Attachment T
Report and Recordkeeping Plan

Attachment T
Report and Recordkeeping Plan
Basic Remediation Company (BRC)
Corrective Action Management Unit (CAMU)
Henderson, Nevada

Table of Contents

1.0 INTRODUCTION

Records of all data, drawings and calculations concerning work proposed or completed at the BRC CAMU will be kept on permanent file in the BRC Data Repository, consistent with document retention requirements specified in the AOC3. In addition, records will be maintained at the BRC offices in conjunction with investigative work at the BRC CAMU. Included in the reports will be appendices with copies of data sheets, log books, and laboratory analysis results. All investigative results will be incorporated into the reports detailed in the following section.

Prior to any document destruction, NDEP will be provided an opportunity to acquire the documents in question.

2.0 OPERATIONAL RECORDKEEPING AND REPORTS

2.1 Training Records

The Construction Manager will maintain records of training received by onsite workers. These records will include copies of certificates received prior to BRC CAMU site activities and site specific training received. The training records will be stored onsite at the facilities maintained by the Construction Manager. Records will be maintained onsite for a period consistent with that specified in the AOC3.

2.2 Operational Records

The contractor will prepare daily progress reports documenting daily BRC CAMU activities. Daily activity records include, but are not limited to, documentation that evidences the quantity of waste materials placed in the BRC CAMU and log in/out forms. In addition to this summary daily report, the contractor will also keep detailed field notes and daily logs documenting:

- 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; and
- a summary of meetings held and attendees.

Daily photographic and video record will be kept by BRC documenting disposal activities. Records and results of inspections will be maintained and kept onsite for a period consistent with the AOC3.

Records and results of waste analysis and waste determinations will be maintained and kept on site. In addition, monitoring, testing or analytical data, and corrective action records resulting from BRC CAMU releases shall be maintained for three years. Groundwater monitoring and clean up records will be maintained until closure of the BRC CAMU.

Summary reports of all incidents that require the use of the Contingency Plan will contain the details outlined in Attachment G, Accident Prevention, Contingency, and Emergency Response Plan. The incident reports will be maintained and kept onsite until completion of the post-closure period for the BRC CAMU.

Additional items to be included with the operating records are: closure and post-closure cost estimates, plans for closure and post-closure, EPA identification number, detailed chemical and physical analysis of a representative waste sample, Quality Control (QC) and Quality Assurance (QA) documentation.

2.2.1 Construction Quality Control and Quality Assurance

The CQA Site Manager will prepare daily reports that document the activities observed during each day of activity as detailed in the CQA Plans for the Base Liner and Final Cover Systems (Sections 3 and 6 in the SRAPI, respectively). The daily reports may include monitoring logs and testing data sheets. At a minimum, these logs and data sheets will include the following information:

- 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 inspected;
- type of construction and inspection 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.

2.3 Interim Status Reports

During remediation activities at the Site, BRC will submit monthly status reports to the NDEP. The purpose of the monthly status reports will be to keep the NDEP informed of the progress of remediation activities at the Site. The reports will present a summary of the remediation progress during the previous month, including as appropriate, significant milestones in BRC CAMU construction, locations of completed pond and ditch excavation (including graphical format), and estimates of soil volumes excavated and placed in the BRC CAMU.

2.4 Annual Reports

BRC will submit an annual report of the solid wastes received at the site to the NDEP. This report will be submitted in a format mutually agreed to between BRC and NDEP. The report will consist of data reported in units of tons and cubic yards for waste materials received at the BRC CAMU.

The annual report will be submitted to NDEP and will include data compiled throughout the year. The total quantity of wastes deposited and the remaining capacity of the BRC CAMU in cubic yards will be incorporated. In addition, the leachate quality data will be compiled and the status of the leachate collection, including quantity of leachate collected on-site and disposed on a monthly basis, reported. Finally, any changes from the approved report, plans, and specifications, with justifications.

2.5 Biennial Reports

BRC will submit a biennial report by March 1 of each even year to the NDEP on EPA form 8700-13B. The report will summarize activities at the BRC CAMU for the previous two years. The report will contain: EPA identification number, BRC CAMU address, dates covered, description of wastes received, method of waste disposal, most recent closure and post-closure cost estimates, methods utilized to reduce waste volume, and any changes in waste volumes. This report will be signed by BRC.

3.0 POST CAMU CLOSURE REPORTING AND RECORDKEEPING

Maintenance inspection records will kept and maintained in a log book in order to clearly document any changes in physical conditions. Copies of the inspection report will be provided to the NDEP annually.

Results of monitoring will be kept and maintained onsite for a period of at least 30 years after closure. Copies of the monitoring will be provided to NDEP with the inspection report annually.

The condition of the facility will be documented with field notes, maps, and photographs, as appropriate. Evidence of potential compromises in the cover will be recorded including eroded patches, patches of dead vegetation, animal burrows, subsidence, and cracks along the cover. Surface water drainage features will be inspected for the presence of debris, physical integrity, and evidence of conditions that exceeded design assumptions.

Attachment U

Compliance with applicable Federal Laws

Attachment U
Compliance with Applicable Federal Laws
Basic Remediation Company (BRC)
Corrective Action Management Unit (CAMU)
Henderson, Nevada

Table of Contents

1.0 INTRODUCTION

This attachment discusses how the BRC CAMU is in compliance with the following Federal Laws.

1.1 The Wild and Scenic Rivers Act

The Wild and Scenic Rivers Acts protects river areas and their immediate environments for the present and future generations. Rivers are eligible for protection if they display one or more of the following characteristics:

1. *Wild river areas* -- Those rivers or sections of rivers that are free of impoundments and generally inaccessible except by trail, with watersheds or shorelines essentially primitive and waters unpolluted. These represent vestiges of primitive America.

2. *Scenic river areas* -- Those rivers or sections of rivers that are free of impoundments, with shorelines or watersheds still largely primitive and shorelines largely undeveloped, but accessible in places by roads.

3. *Recreational river areas* -- Those rivers or sections of rivers that are readily accessible by road or railroad, that may have some development along their shorelines, and that may have undergone some impoundment or diversion in the past.

There are no rivers in the vicinity of the BRC CAMU site with the exception of the Las Vegas Wash which is approximately 2 miles north of the Site, and the Las Vegas Wash is not designated as a Wild and Scenic River.

1.2 The National Historic Preservation Act of 1966

The National Historic Preservation Act protects, rehabilitates, restores and reconstructs the districts, sites, buildings, structures, and objects significant in American history, architecture, archaeology, or culture.

There are no locations of historical significance at the BRC CAMU site. BRC contacted the Nevada Natural Heritage Program and requested a search of their database. No locations of historical significance were identified as a result of this search. A copy of the seach result is provided in Appendix A to this Attachment.

1.3 <u>The Endangered Species Act</u>

The city of Henderson, along with other cities within Clark County, submitted an application to the EPA for a permit to incidentally take desert tortoises (*gopherus agassizii*), pursuant to section 10(a)(1)(B) of the Endangered Species Act of 1973, as amended (Act), in association with various proposed public and private projects in Clark County, Nevada. The permit allows incidental take of desert tortoises for a period of 30 years, resulting from development on up to 113,900 acres of private lands within Clark County, Nevada. The permit application was received September 28, 1994, and was accompanied by the Clark County Desert Conservation Plan (CCDCP), which serves as the Applicant's habitat conservation plan and details their proposed measures to minimize, monitor, and mitigate the impacts of the proposed take on the desert tortoise.

To minimize the impacts of take, Henderson provides a free pick-up and collection service for desert tortoises encountered in harm's way within the city. These desert tortoises will be made available for beneficial uses such as translocation studies and programs, research, education, zoos, museums, or other programs approved by the Service and Nevada Division of Wildlife. Sick or injured desert tortoises will be humanely euthanized.

Henderson approves the issuance of land development permits for otherwise lawful public and private project proponents during the 30-year period in which the proposed Federal permit is in effect. Henderson imposes a fee of $550 per acre of habitat disturbance to fund the measures to minimize and mitigate the impacts of the proposed action on desert tortoises.

The BRC CAMU will be in compliance with the Endangered Species Act through a fee of $550 per acre submitted with the grading permit application. If a Desert Tortoise is encountered during construction, the City of Henderson will be contacted and the tortoise relocated.

The above discussion notwithstanding, BRC does not believe that there are any threatened and endangered species in the CAMU area. BRC requested that Nevada Natural Heritage conduct a search related to such species. The letter from Nevada Natural Heritage provided in Appendix A to the Attachment confirms that there are no at risk taxa in this area.

1.4 <u>The Coastal Zone Management Act</u>

The Coastal Zone Management Act seeks to manage and preserve the nation's coastal resources, ensuring their protection for future generation.

The BRC CAMU site is not located near any coasts.

1.5 The Fish and Wildlife Coordination Act

The Act provides that whenever the waters or channel of a body of water are modified by a department or agency of the U.S., the department or agency first shall consult with the U.S. Fish and Wildlife Service and with the head of the agency exercising administration over the wildlife resources of the state where construction will occur, with a view to the conservation of wildlife resources. The Act provides that land, water and interests may be acquired by federal construction agencies for wildlife conservation and development. In addition, real property under jurisdiction or control of a federal agency and no longer required by that agency can be utilized for wildlife conservation by the state agency exercising administration over wildlife resources upon that property.

The BRC CAMU will not modify the waters or channel of a body of water.

Appendix A

Copy of Letter Received from Nevada Natural Heritage

Nevada Natural Heritage Program

Nevada Natural Heritage Program

Nevada Department of Conservation and Natural Resources
Richard H. Bryan Building
901 South Stewart Street, suite 5002 • Carson City, Nevada 89701-5245, U.S.A.
tel: (775) 684-2900 • internet: http://heritage.nv.gov

28 February 2007

Ranajit Sahu
Basic Remediation Company
875 W Warm Springs Rd.
Henderson, NV 89011

RE: Data request received 23 February 2007

Dear Mr. Sahu:

We are pleased to provide the information you requested on endangered, threatened, candidate, and/or At Risk plant and animal taxa recorded within or near the BRC Common Areas-CAMU Project area. We searched our database and maps for the following a 5 kilometer radius around including:

Township 22S Range 62E Sections 11 and 12

There are no at risk taxa recorded within the given area. However, habitat may be available for: the big free-tailed bat, *Nyctinomops macrotis*, a Nevada Bureau of Land Management (BLM) Sensitive Species; the spotted bat, *Euderma maculatum*, a Nevada BLM Special Status Species; the Arizona toad, *Bufo microscaphus*, a Nevada BLM Sensitive Species; the desert tortoise, *Gopherus agassizii*, a Federally Threatened Taxon; the chuckwalla, *Sauromalus ater*, a Nevada BLM Sensitive Species; and the banded Gila monster, *Heloderma suspectum cinctum*, a Nevada BLM Special Status Species. We do not have complete data on various raptors that may also occur in the area; for more information contact Ralph Phenix, Nevada Division of Wildlife at (775) 688-1565. Note that all cacti, yuccas, and Christmas trees are protected by Nevada state law (NRS 527.060-.120), including taxa not tracked by this office.

Please note that our data are dependent on the research and observations of many individuals and organizations, and in most cases are not the result of comprehensive or site-specific field surveys. Natural Heritage reports should never be regarded as final statements on the taxa or areas being considered, nor should they be substituted for on-site surveys required for environmental assessments.

Thank you for checking with our program. Please contact us for additional information or further assistance.

Sincerely,

Eric S. Miskow
Biologist III/Data Manager

Attachment V
Additional information required by NDEP

SUPPLEMENTAL
REMEDIAL ACTION PLAN (RAP)
PERMIT APPLICATION INFORMATION FOR
CORRECTIVE ACTION MANAGEMENT UNIT (CAMU)
HENDERSON, NEVADA

Submitted to:

Nevada Division of Environmental Protection
901 South Stewart Street – 4th Floor
Carson City, Nevada 89701
(775) 687-4670

Prepared for:

Basic Remediation Company
875 West Warm Springs Road
Henderson, Nevada 89015
(702) 567-0400

Prepared by:

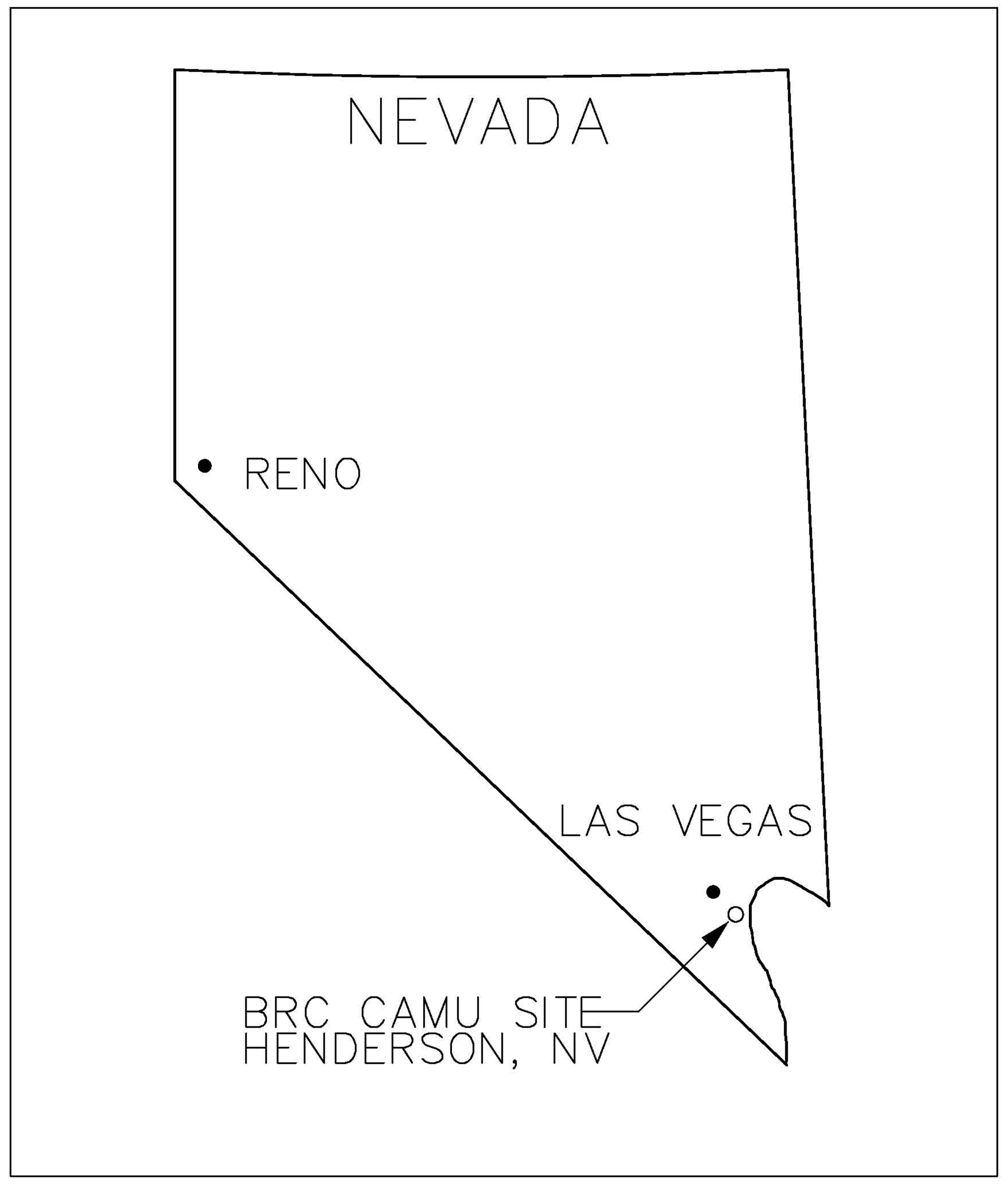
GeoSyntec Consultants
10875 Rancho Bernardo Road, Ste. 200
San Diego, California 92127
(858) 674-6559

November 2006

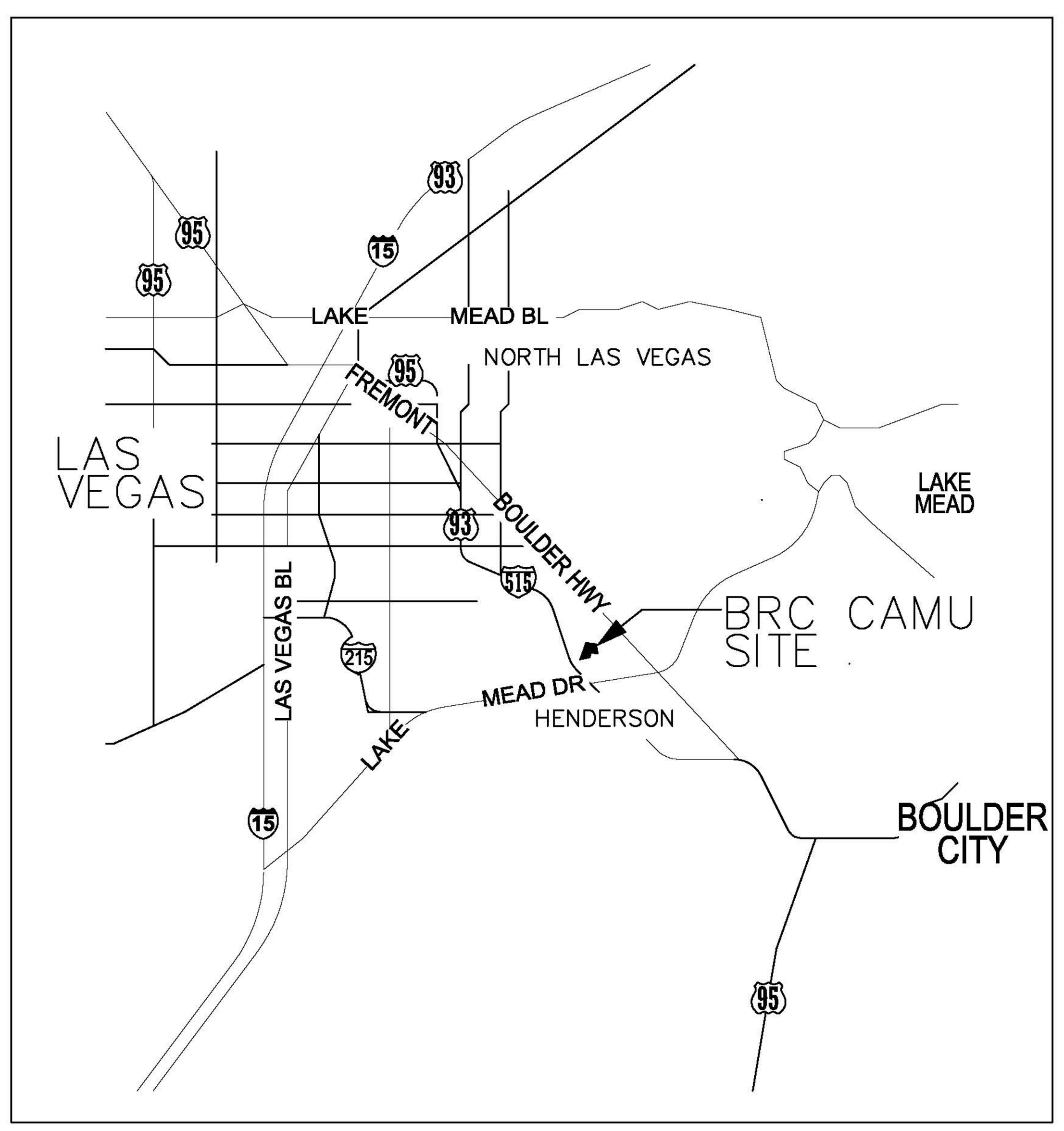
Table of Contents

Section 1
Permit Drawings

PERMIT DRAWINGS
Basic Remediation Company
CORRECTIVE ACTION MANAGEMENT UNIT
CONTROL SYSTEMS DESIGN
HENDERSON, NEVADA
NOVEMBER 2006

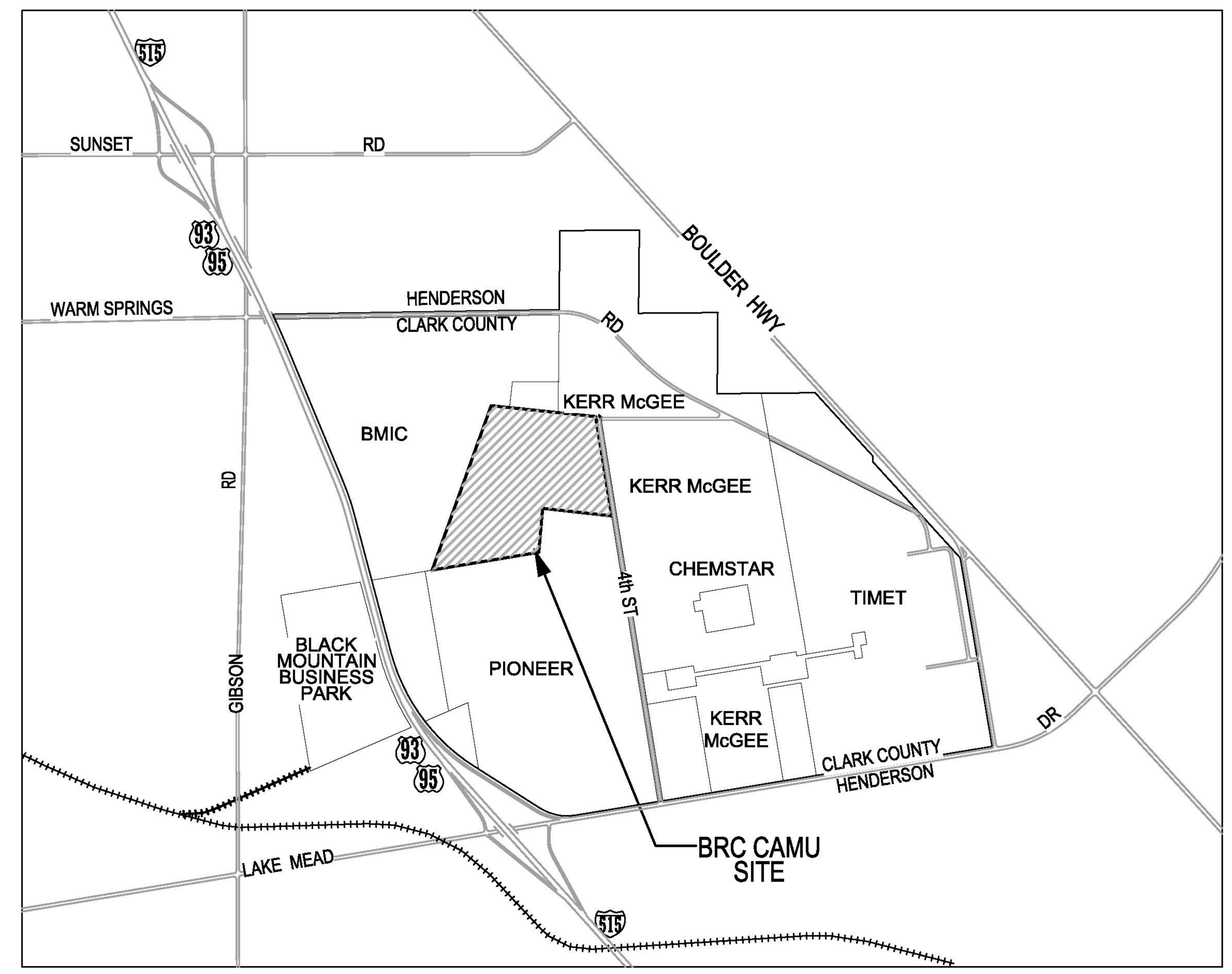


LOCATION MAP
NTS



VICINITY MAP
Approximate Scale: 1"=3 Miles

DRAWING LIST	DRAWING TITLE
J–1	TITLE SHEET AND DRAWING LIST
J–2	LEGEND, ABBREVIATIONS AND GENERAL NOTES
J–3	OVERALL SITE PLAN
J–4	CONSTRUCTION SEQUENCE PLAN
J–5	SURCHARGE PLAN
J–6	SITE PREPARATION PLAN
J–7	CONSTRUCTION GRADING PLAN
J–8	STORM DRAIN PLAN
J–9	FINAL COVER GRADING PLAN
J–10	CROSS SECTIONS
J–11	FINAL COVER AND LINER DETAILS AND SECTIONS
J–12	FENCE PLAN



SITE PLAN
Approximate Scale: 1"=1/2 Mile

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: TITLE SHEET

MARK	DATE	REVISION	BY	APPROVED

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

SIGNATURE

DATE

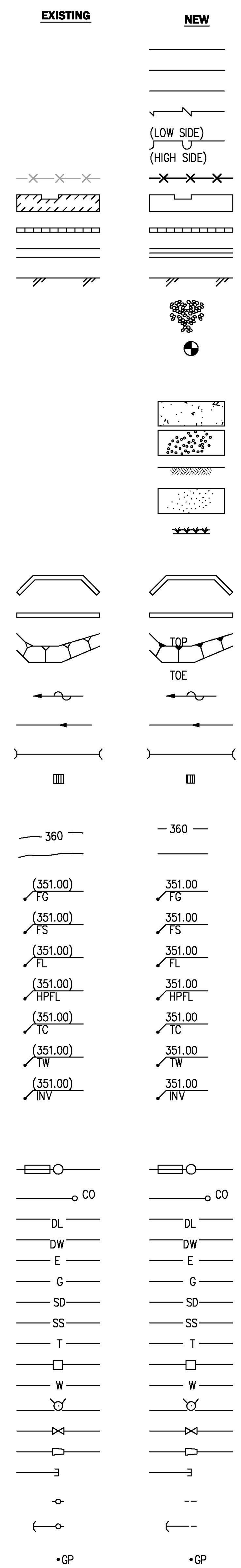
SEAL

DATE:	NOVEMBER 2006	SCALE:	AS SHOWN
DESIGN BY:	G.T.C.	JOB NO.:	SC0313
DRAWN BY:	J.R.H.	FILE NO:	
CHECKED BY:	G.T.C.	DOCUMENT NO:	
REVIEWED BY:	G.T.C.	DRAWING NO:	J–1
APPROVED BY:	G.T.C.		

ABBREVIATIONS

ABAN	ABANDON ABANDON IN PLACE .
CIP	CAST IRON PIPE
C-C	CENTER TO CENTER
CL	CENTER LINE
COH	CITY OF HENDERSON
CC	CLARK COUNTY
CCSD	CLARK COUNTY SANITATION DISTRICT
CO	CLEANOUT
COMM	COMMERCIAL
CONC, PCC	CONCRETE, PORTLAND CEMENT CONCRETE
CONST	CONSTRUCTION OR CONSTRUCT
CMP	CORRUGATED METAL PIPE
CU FT, CF	CUBIC FOOT
CU YD, CY	CUBIC YARD
CULV	CULVERT
CF	CURB FACE
DEPT	DEPARTMENT
Δ	DELTA = ANGLE
DIA, ⌀	DIAMETER
DWG	DRAWING
DI	DROP INLET
DL	DRAIN LINE
EA	EACH
ESNT	EASEMENT
EO	EDGE OF OIL
EP	EDGE OF PAVEMENT
E	ELECTRIC OR ELECTRICAL
ELEV	ELEVATION
EMBK	EMBANKMENT
ECR	END OF CURB RETURN
EC	END OF CURVE
EVC	END OF VERTICAL CURVE
EXIST	EXISTING
FT	FEET, FOOT
FG	FINISH GRADE
FS	FINISH SURFACE
FH	FIRE HYDRANT
FL, F_L	FLOW LINE
GALV	GALVANIZED
G	GAS
GB	GRADE BREAK
HDWL	HEADWALL
HP	HIGH POINT
HPFL	HIGH POINT FLOW LINE
HORIZ	HORIZONTAL
IN	INCH
INT	INTERSECTION
INV	INVERT
LVVWD	LAS VEGAS VALLEY WATER DISTRICT
LF	LINEAR FEET
MH	MANHOLE
MAX	MAXIMUM
MIN	MINIMUM
NTS	NOT TO SCALE
OC	ON CENTER
PVMT	PAVEMENT
PCC	POINT OF COMPOUND CURVE
PC	POINT OF CURVE
PI	POINT OF INTERSECTION
PRC	POINT OF REVERSE CURVE
PT	POINT OF TANGENCY
PVI	POINT OF VERTICAL INFLECTION
PVC	POINT OF VERTICAL CURVE
PCC	PORTLAND CEMENT CONCRETE
PL, P_L	PROPERTY LINE
R	RADIUS OR RIDGE
REINF	REINFORCED
RCP	REINFORCED CONCRETE PIPE
R/W	RIGHT-OF-WAY
RD	ROAD
SS	SANITARY SEWER
SHT	SHEET
SW	SIDEWALK
SQ FT, SF	SQUARE FOOT
SQ YD, SY	SQUARE YARD
STA	STATION
SD	STORM DRAIN
STD	STANDARD
STL	STEEL
T	TELEPHONE
TEMP	TEMPORARY
TBA	TO BE ADJUSTED
TBR	TO BE REMOVED
TC	TOP OF CURB
TG	TOP OF GRATE
TOC	TOP OF CONCRETE
TP	TOP OF PIPE
TW	TOP OF WALL
TYP	TYPICAL
UG	UNDERGROUND
VAR	VARIABLE
VERT	VERTICAL
VC	VERTICAL CURVE
VCP	VITRIFIED CLAY PIPE
W	WATER
WWF	WELDED WIRE FABRIC
YD	YARD

LEGEND



DESCRIPTION

CENTERLINE ~
SECTION LINE
RIGHT-OF-WAY OR PROPERTY LINE
CUT LINE - ASPHALTIC OR PORTLAND CEMENT CONCRETE
RETAINING WALL
CHAINLINK FENCE
BUILDING OR STRUCTURE
CONCRETE BLOCK WALL
ROAD
EDGE OF PAVEMENT
RIPRAP
TEST HOLE
CONCRETE (SHOWN ON CROSS SECTIONS)
GRAVEL (SHOWN ON CROSS SECTIONS)
GROUND (SHOWN ON CROSS SECTIONS)
SAND (SHOWN ON CROSS SECTIONS)
VEGETATIVE COVER (SHOWN ON CROSS SECTIONS)
WING TYPE HEADWALL
PLAIN HEADWALL
CUT OF FILL SLOPES
DIRECTION OF SHEET FLOW
FLOW LINE OF DITCH OR CHANNEL
CULVERT
STORM DRAIN CATCH BASIN
MAJOR CONTOUR LINE AND ELEVATION
MINOR CONTOUR LINE
FINISH GRADE ELEVATION
FINISH SURFACE ELEVATION
FLOW LINE ELEVATION
HIGH POINT FLOW LINE ELEVATION
TOP OF CURB ELEVATION
TOP OF WALL ELEVATION
INVERT ELEVATION
UNDERGROUND WITH MANHOLE AND CASING
CLEANOUT
DRAIN LINE
DOMESTIC WATER
ELECTRICAL
GAS
STORM DRAIN
SANITARY SEWER
TELEPHONE/COMMUNICATIONS
TELEPHONE/COMMUNICATIONS MANHOLE
WATER
FIRE HYDRANT
GATE VALVE
REDUCER
PLUG OR CAP
UTILITY POLE
UTILITY POLE WITH GUY ANCHOR
GUARD POST

GENERAL NOTES

1. UNLESS OTHERWISE INDICATED ON THE DRAWINGS, THE TOPOGRAPHY, INCLUDING HORIZONTAL AND VERTICAL CONTROLS ARE FROM A SURVEY BY PENTACORE GEOGRAPHIC INFORMATION SYSTEMS, 6763 WEST CHARLESTON BLVD, LAS VEGAS, NEVADA, DATED SEP. 9, 1999.

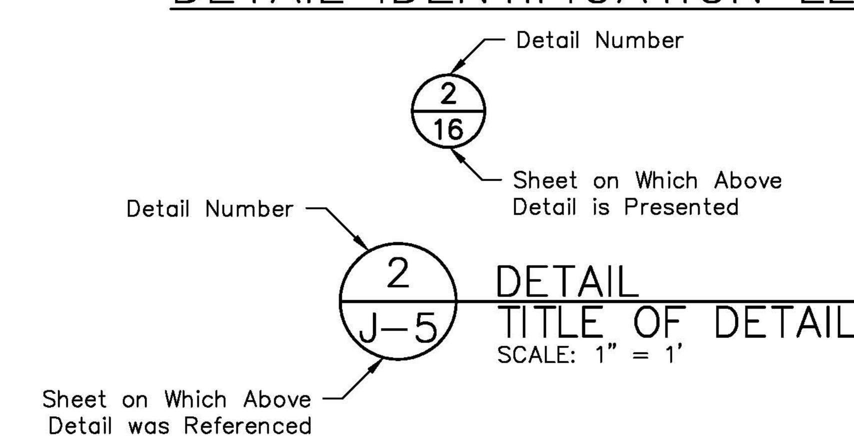
   1.1. BASIS OF VERTICAL CONTROL:
   CITY OF HENDERSON BENCHMARK NO. 5 - BOLT AND WASHER IN THE TOP OF THE CURB ON THE EAST SIDE OF HIGHWAY 93, 100 FEET +/- NORTHWEST OF THE CENTERLINE OF KING STREET.

   NAVD 1988 DATUM ELEVATION = 519.021 METERS 1702.821 FEET

   1.2. BASIS OF HORIZONTAL CONTROL:
   THE BASIS OF BEARINGS FOR THIS PROJECT IS GRID NORTH AS DEFINED BY THE NEVADA COORDINATE SYSTEM OF 1983 (NCS83), EAST ZONE, (2701), DETERMINED BY GIS CONTROL POINTS, "851" , "884" AND "W51" AS SHOWN ON A RECORD OF SURVEY ON FILE IN THE CLARK COUNTY RECORDER'S OFFICE, IN FILE 88 OF SURVEYS, AT PAGE 53.

2. A PRELIMINARY GEOTECHNICAL AND GEOLOGIC INVESTIGATION NO. 99-33437-01, DATED OCTOBER 22, 1999 PREPARED BY CONVERSE CONSULTANTS IS PART OF THE CONSTRUCTION DOCUMENTS.
3. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE FIELD INVESTIGATION OF THE EXISTING CONDITIONS AND FOR THE FIELD VERIFICATION OF EXISTING UNDERGROUND UTILITIES WITHIN THE PROPOSED WORK AREA. IF UNKNOWN UNDERGROUND UTILITIES ARE ENCOUNTERED DURING EXCAVATION, THE SUBCONTRACTOR SHALL IMMEDIATELY NOTIFY CONSTRUCTION MANAGER PRIOR TO CONTINUING THE WORK IN THE IMMEDIATE VICINITY.
4. THE CONTRACTOR SHALL PROTECT EXISTING UNDERGROUND AND ABOVEGROUND LINES AND ANY EXISTING IMPROVEMENTS AND EQUIPMENT WITHIN THE PROJECT LIMITS. ANY EXISTING CONSTRUCTION DAMAGED BY CONTRACTOR OPERATIONS SHALL BE REPAIRED OR REPLACED AT THE CONTRACTOR'S EXPENSE.
5. ALL UNDERGROUND UTILITIES SHALL BE PROPERLY PROTECTED DURING CONSTRUCTION FROM HEAVY EQUIPMENT BY THE USE OF SUITABLE TIMBER MAT OR STEEL MATTING.
6. DIMENSIONS, ELEVATIONS, AND LOCATION OF EXISTING UTILITIES ARE TO BE FIELD VERIFIED BY CONTRACTOR PRIOR TO START OF CONSTRUCTION.
7. STRAIGHT GRADE BETWEEN SPOT ELEVATIONS SHOWN UNLESS OTHERWISE INDICATED ON THE PLANS.
8. FINISHED SURFACES SHALL BE SLOPED UNIFORMLY FROM HIGH POINTS, RIDGE LINES, AND AROUND OBJECTS TO FLOW LINES AND AREA DRAINS UNLESS INDICATED OTHERWISE.
9. THE GEOTECHNICAL ENGINEER SHALL PROVIDE SUFFICIENT INSPECTIONS DURING THE EXCAVATION OF THE NATURAL GROUND AND THE PLACEMENT AND COMPACTION OF THE FILL TO BE SATISFIED THAT THE WORK IS BEING PERFORMED IN ACCORDANCE WITH THE PLANS AND SPECIFICATIONS.
10. EXCAVATION, EMBANKMENT, AND BACKFILL WORK AREAS SHALL BE CONTINUALLY AND EFFECTIVELY DRAINED. WATER SHALL NOT BE PERMITTED TO ACCUMULATE IN EXCAVATION OR FOUNDATION AREAS. THE SUBCONTRACTOR SHALL PROVIDE SUITABLE DIKES, DRAINS OR SHALL PROVIDE PUMPING EQUIPMENT AS REQUIRED TO DIVERT WATER FLOW AWAY FROM THE WORK AREAS.
11. SLOPES SHOWN ON GRADING PLANS ARE FOR INFORMATION ONLY. ACTUAL LINES SHALL BE VERIFIED BY FIELD MEASUREMENTS.
12. THE CONTRACTOR SHALL PERFORM GRADING OPERATIONS IN ACCORDANCE WITH EARTHWORK SPECIFICATION, SECTION 02200.
13. SUBCONTRACTOR SHALL COORDINATE WITH OTHER SUBCONTRACTORS IN THE WORK AREA. ANY DISPUTES BETWEEN SUBCONTRACTORS SHALL BE RESOLVED THROUGH THE CONSTRUCTION MANAGER.
14. THE SUBCONTRACTOR SHALL IMMEDIATELY NOTIFY CONSTRUCTION MANAGER FOR PROVISIONS OF THE APPROPRIATE MEANS OF DECOMMISSIONING UNDERGROUND UTILITIES ENCOUNTERED IN THE FIELD WITHIN HIS WORK PACKAGE LIMITS.

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.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: GENERAL NOTES, LEGEND, AND ABBREVIATIONS

MARK	DATE	REVISION	BY	APPROVED

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.	DATE: NOVEMBER 2006	SCALE: AS SHOWN	
	DESIGN BY: G.T.C.	JOB NO.: SC0313	
	DRAWN BY: J.R.H.	FILE NO:	
	CHECKED BY: G.T.C.	DOCUMENT NO:	
SIGNATURE	REVIEWED BY: G.T.C.	DRAWING NO: J-2	
DATE	SEAL	APPROVED BY: G.T.C.	

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LEGEND

—PL—	BRC LANDFILL SITE BOUNDARY
— — —	EXISTING LIMITS OF FORMER BMI LANDFILL
- - - -	EXISTING SURFACE WATER FLOW PATH
~~	EXISTING MINOR ELEVATION CONTOUR
~~	EXISTING MAJOR ELEVATION CONTOUR
— — —	PROPOSED LIMIT OF WASTE
-------	APPROXIMATE SLIT TRENCH LIMITS

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: OVERALL SITE PLAN

MARK	DATE	REVISION	BY	APPROVED

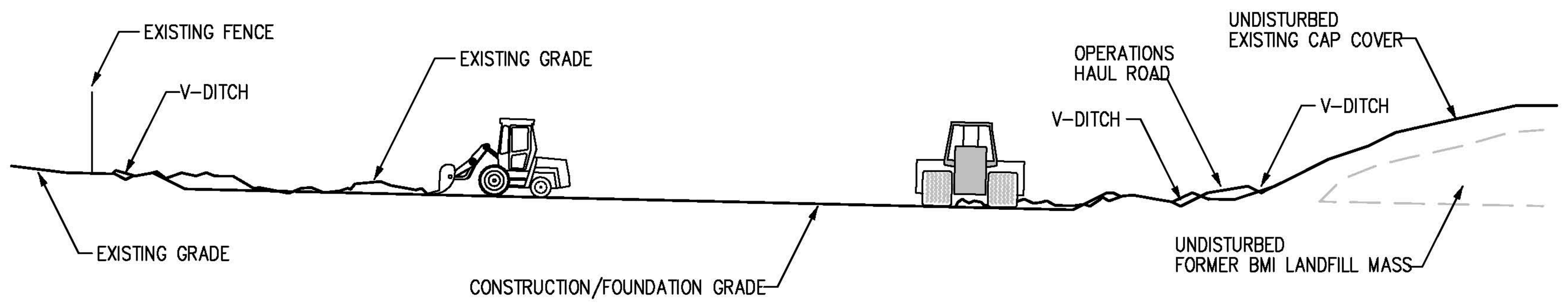
THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.	DATE: NOVEMBER 2006	SCALE: AS SHOWN
	DESIGN BY: G.T.C.	JOB NO.: SC0313
	DRAWN BY: J.R.H.	FILE NO:
	CHECKED BY: G.T.C.	DOCUMENT NO:
SIGNATURE	REVIEWED BY: G.T.C.	DRAWING NO: J-3
DATE / SEAL	APPROVED BY: G.T.C.	

P:\PR\SDCadd\CADD\SC0313\BRC\figures\Construction\Revised 11-6-06\SC0313_J-3_Overall Site Plan.dwg 11/14/06 10:24 jhall

1

- REMOVE ANY INTERFERING PORTION OF EXISTING FENCE
- CLEAR AND REMOVE BRUSH AND RUBBLE
- INSTALL INTERIM PERIMETER FENCE AND ACCESS GATES
- CONSTRUCT OPERATIONS SUPPORT FACILITIES
- GRADE TO FOUNDATION TO RECEIVE LINER LAYER

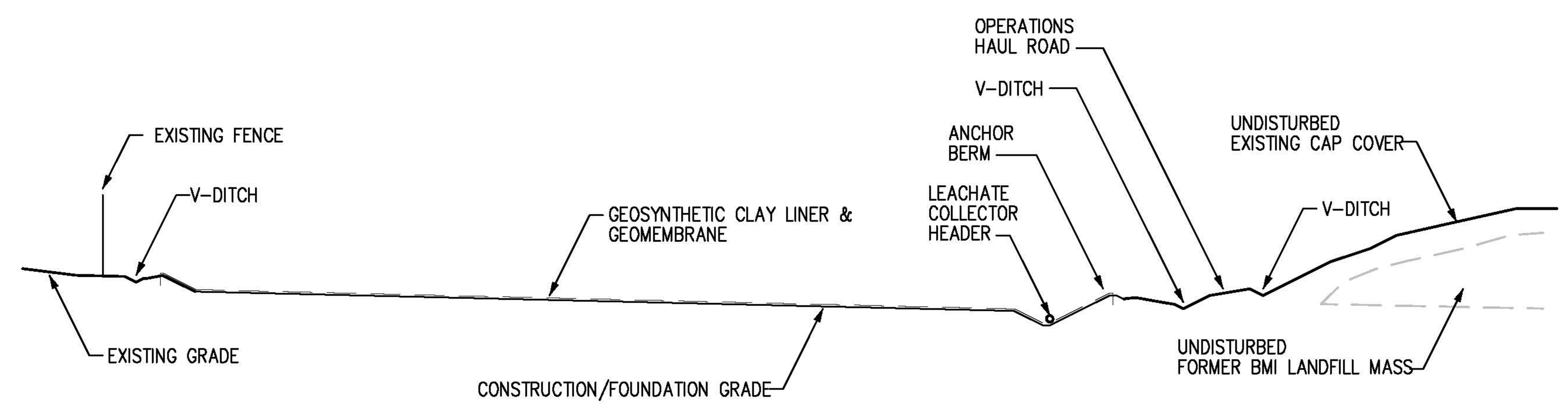
- CONSTRUCT OPERATIONS HAUL ROAD
- CONSTRUCT EROSION CONTROL FEATURES – DITCHES, CULVERTS, SILT FENCES, CHECK DAMS, AND BASINS



NOT TO SCALE

2

- CONSTRUCT FLOWLINES, SUMPS, AND BERMS FOR LEACHATE COLLECTION SYSTEM AND LINER ANCHOR
- INSTALL GEOSYNTHETIC CLAY LINER, GEOMEMBRANE AND GEOCOMPOSITE

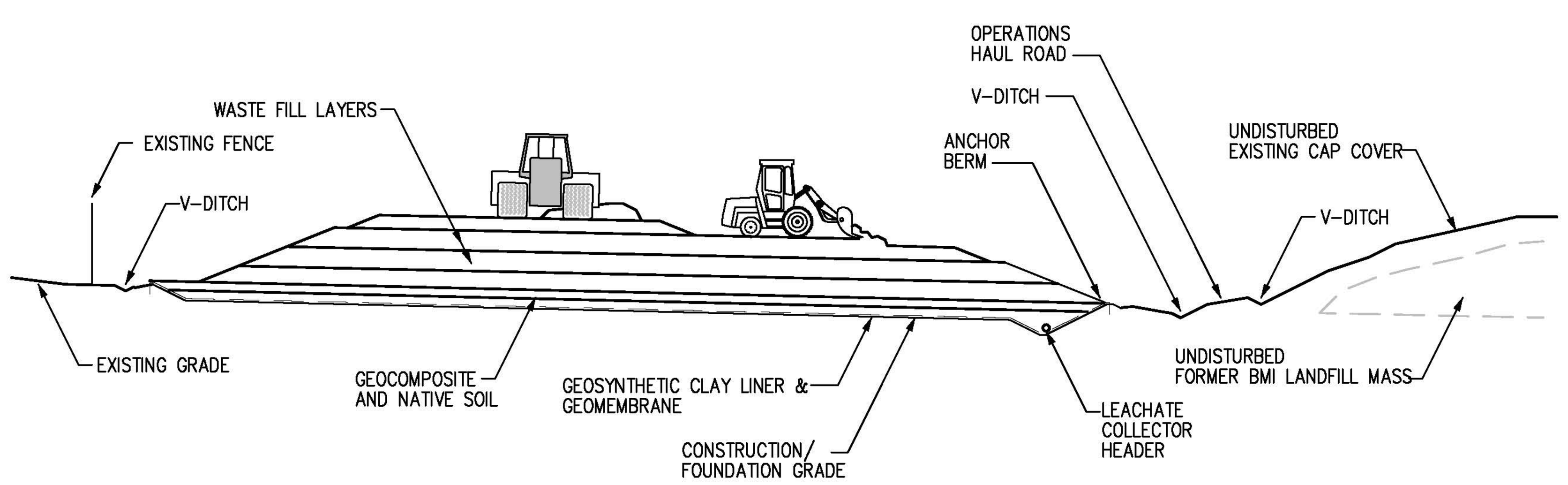


NOT TO SCALE

3

- INSTALL LEACHATE COLLECTION SYSTEM
- INSTALL BALANCE OF COMPOSITE LINER: GEOCOMPOSITE AND NATIVE SOIL

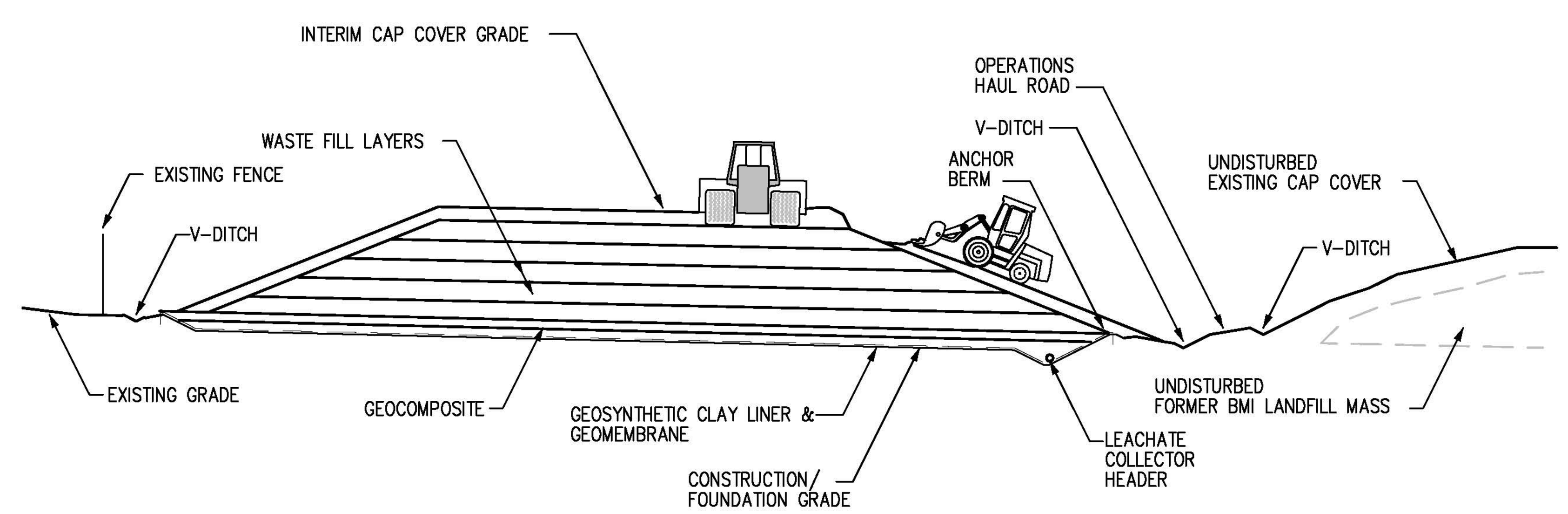
- AT REMOVAL SITE: STOCKPILE AND LOAD WASTE FOR TRANSPORT
- AT LANDFILL SITE: UNLOAD, PLACE, COMPACT AND GRADE WASTE FILL IN LAYERS



NOT TO SCALE

4

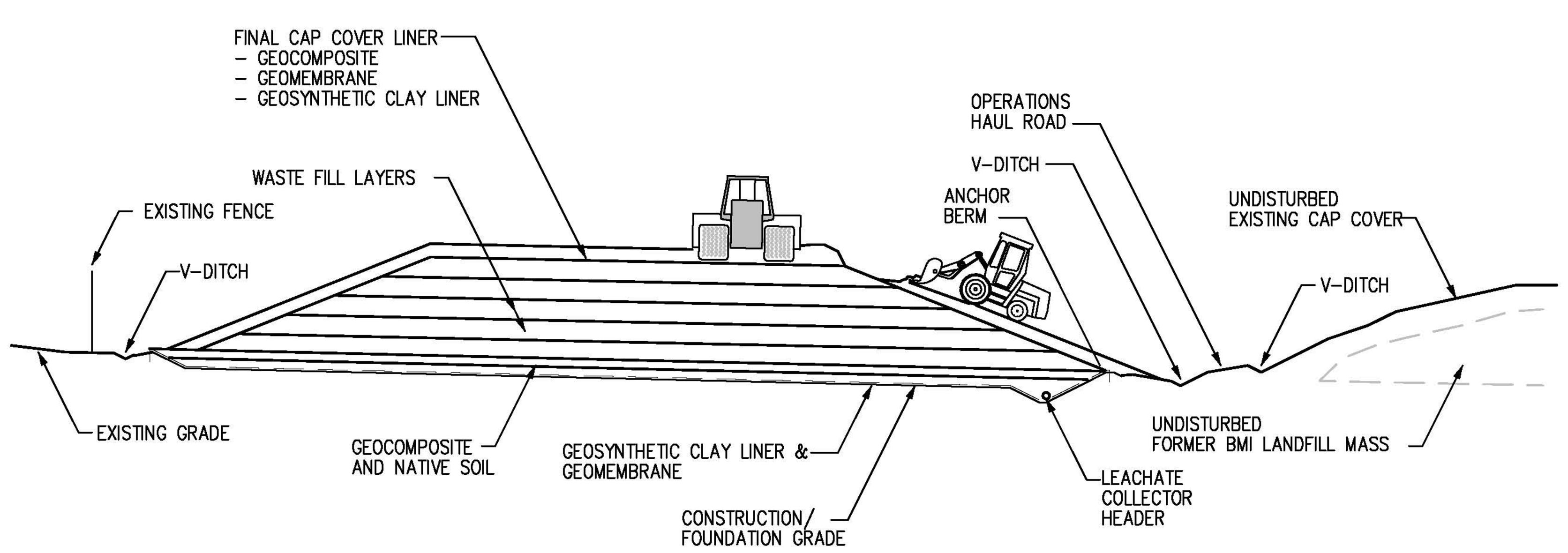
- PLACE, COMPACT AND GRADE SOILS MATERIAL FOR INTERIM CAP COVER



NOT TO SCALE

5

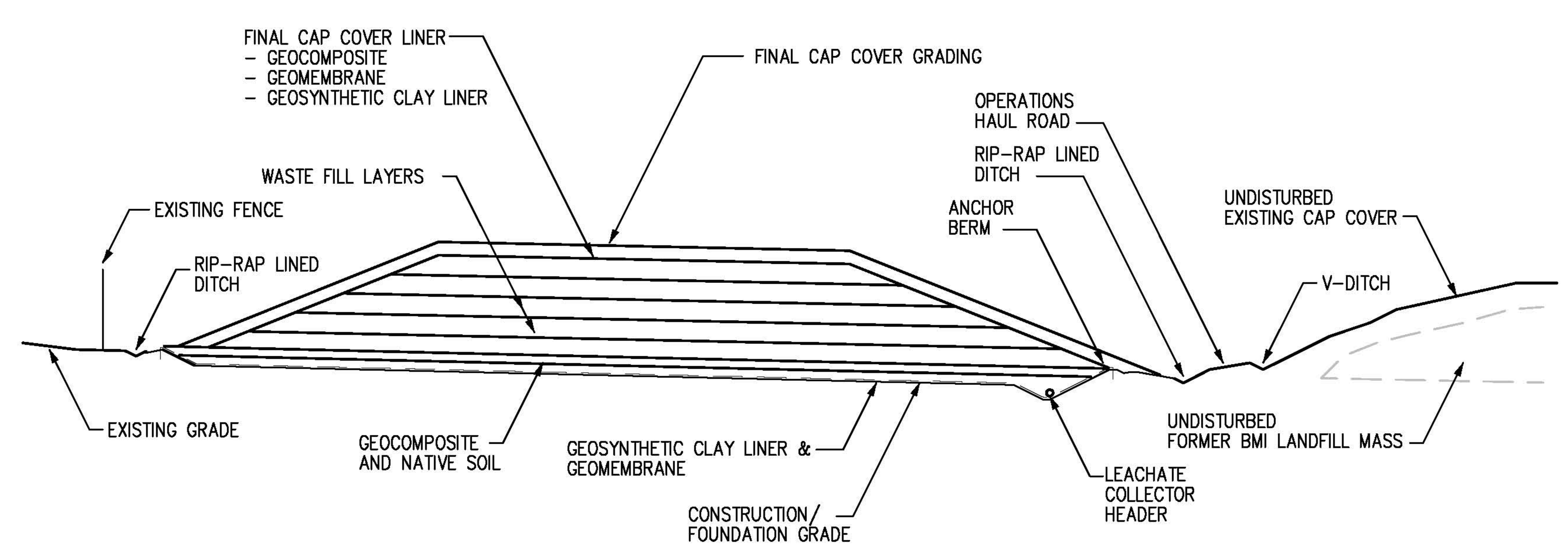
- REMOVE INTERIM CAP COVER
- INSTALL FINAL CAP COVER LINER – GCL, GEOMEMBRANE AND GEOCOMPOSITE



NOT TO SCALE

6

- PLACE, COMPACT AND GRADE SOILS MATERIAL FOR FINAL CAP COVER
- INSTALL PERMANENT STORM DRAINAGE FACILITIES (GUTTERS, DITCHES, CULVERTS, HEADWALLS, ETC)



NOT TO SCALE

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: CONSTRUCTION SEQUENCE PLAN

MARK	DATE	REVISION	BY	APPROVED

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SIGNATURE

DATE

SEAL

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DRAWN BY:	J.R.H.	FILE NO:	
CHECKED BY:	G.T.C.	DOCUMENT NO:	
REVIEWED BY:	G.T.C.	DRAWING NO:	J-4
APPROVED BY:	G.T.C.		

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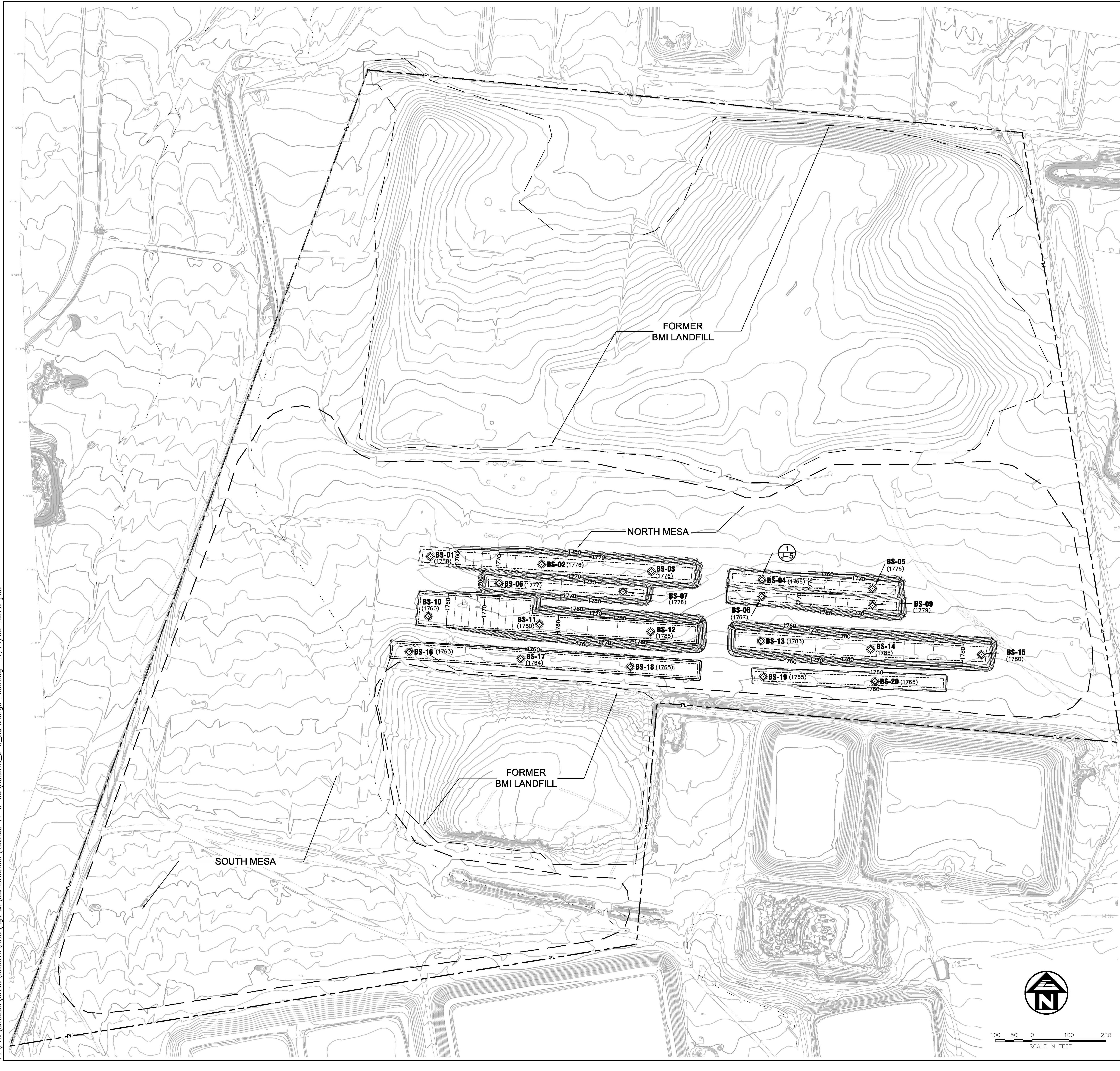

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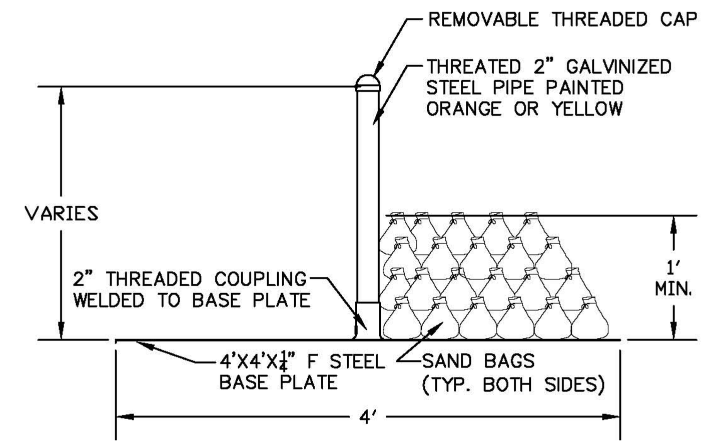
LEGEND

-PL-	BRC LANDFILL SITE BOUNDARY
	EXISTING LIMITS OF FORMER BMI LANDFILL
	EXISTING SURFACE WATER FLOW PATH
	EXISTING MINOR ELEVATION CONTOUR
	EXISTING MAJOR ELEVATION CONTOUR
	PROPOSED LIMIT OF WASTE
	APPROXIMATE SLIT TRENCH LIMITS
◈	PROPOSED SETTLEMENT PLATE (DRAWN AT LARGER SCALE FOR CLARITY)
BS-01	EXISTING BORING
1760	PROPOSED MAJOR SURCHARGE ELEVATION CONTOUR
	PROPOSED MINOR SURCHARGE ELEVATION CONTOUR



1
J-5
DETAIL
SETTLEMENT PLATE
N.T.S.

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT:
BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE:
SURCHARGE PLAN

MARK	DATE	REVISION	BY	APPROVED

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	DRAWN BY: J.R.H.	FILE NO:
	CHECKED BY: G.T.C.	DOCUMENT NO:
SIGNATURE	REVIEWED BY: G.T.C.	DRAWING NO: J-5
DATE	SEAL	APPROVED BY: G.T.C.	

LEGEND

—PL—	BRC LANDFILL SITE BOUNDARY
— — —	EXISTING LIMITS OF FORMER BMI LANDFILL
- - - -	EXISTING SURFACE WATER FLOW PATH
～	EXISTING MINOR ELEVATION CONTOUR
～	EXISTING MAJOR ELEVATION CONTOUR
— — —	PROPOSED LIMIT OF WASTE
- - - -	APPROXIMATE SLIT TRENCH LIMITS

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: SITE PREPARATION PLAN

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SIGNATURE	REVIEWED BY: G.T.C.	DRAWING NO: J-6
DATE SEAL	APPROVED BY: G.T.C.	

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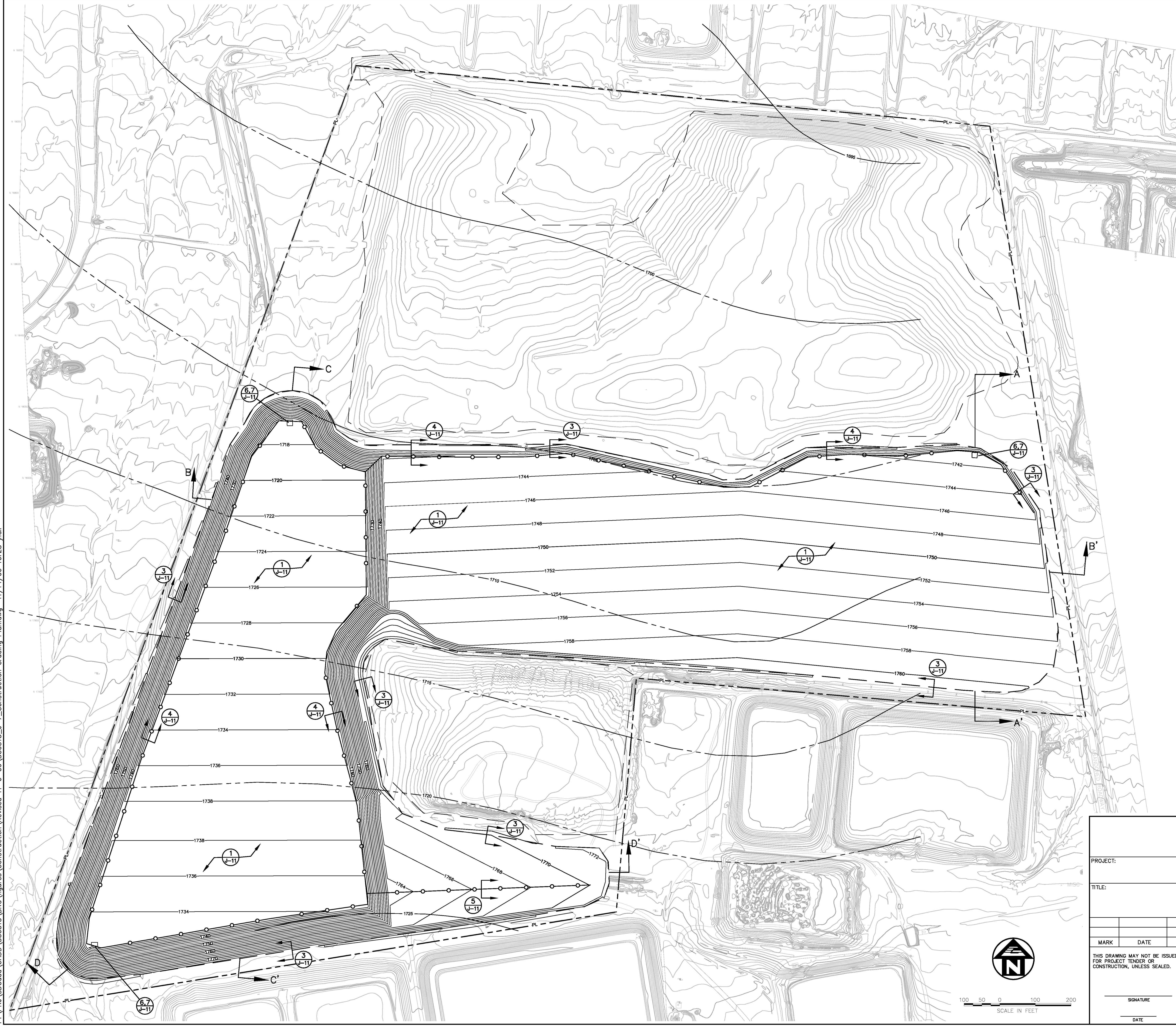

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LEGEND

- -PL- BRC LANDFILL SITE BOUNDARY
- EXISTING LIMITS OF FORMER BMI LANDFILL
- EXISTING SURFACE WATER FLOW PATH
- EXISTING MINOR ELEVATION CONTOUR
- EXISTING MAJOR ELEVATION CONTOUR
- PROPOSED MINOR ELEVATION CONTOUR
- PROPOSED MAJOR ELEVATION CONTOUR
- PROPOSED LIMIT OF WASTE
- GROUNDWATER SURFACE CONTOUR
- LCRS PIPE

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.
8. GROUNDWATER CONTOURS PROVIDED BY MWH, DATED JULY 2005

GEOSYNTEC CONSULTANTS
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SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: CONSTRUCTION GRADING PLAN

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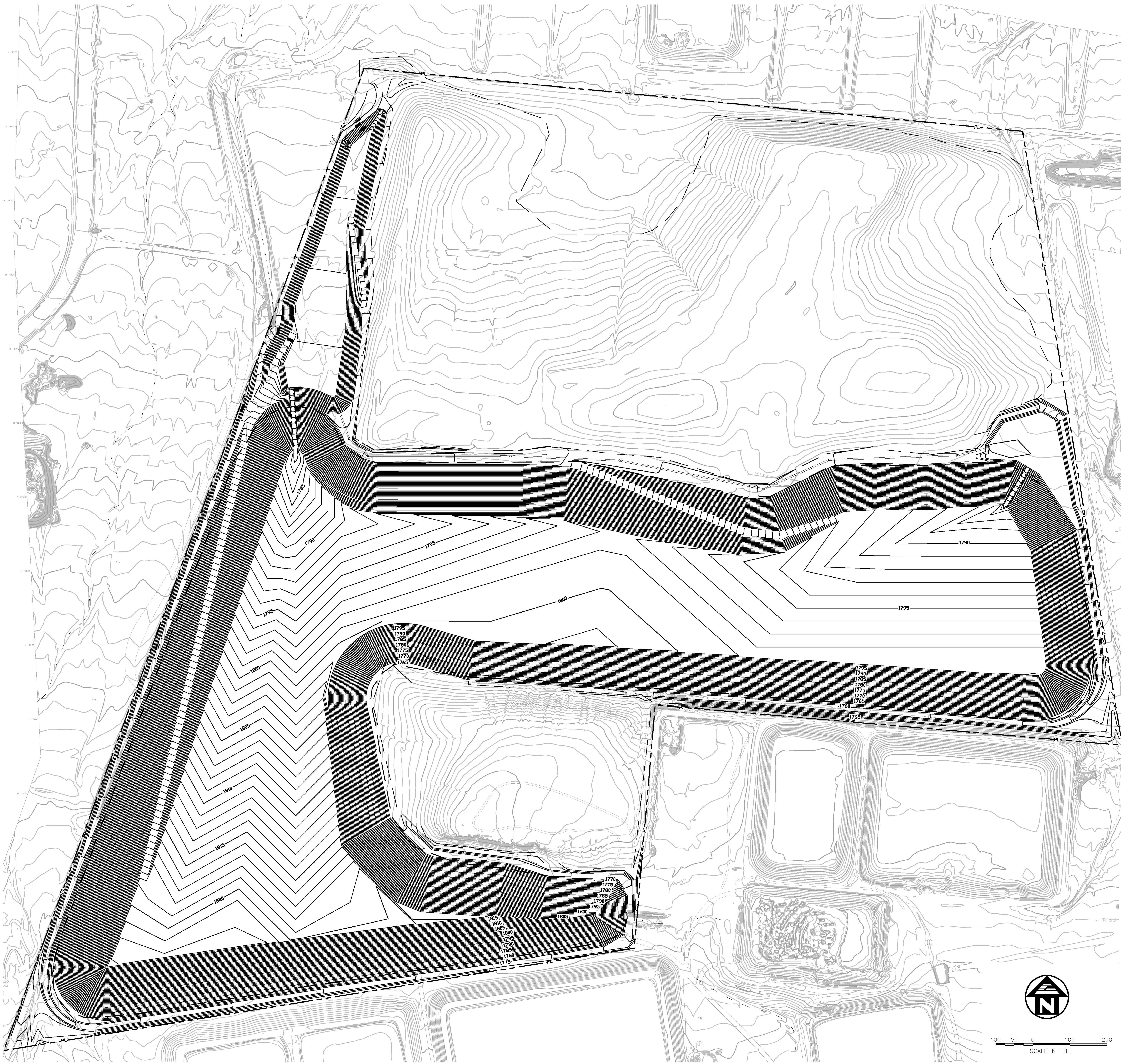
		DESIGN BY: G.T.C.	JOB NO.: SC0313
		DRAWN BY: J.R.H.	FILE NO:
		CHECKED BY: G.T.C.	DOCUMENT NO:
SIGNATURE		REVIEWED BY: G.T.C.	DRAWING NO: J-7
DATE	SEAL	APPROVED BY: G.T.C.	



LEGEND

—PL—	BRC LANDFILL SITE BOUNDARY
— — —	EXISTING LIMITS OF FORMER BMI LANDFILL
- - - -	EXISTING SURFACE WATER FLOW PATH
	EXISTING MINOR ELEVATION CONTOUR
	EXISTING MAJOR ELEVATION CONTOUR
	PROPOSED MINOR ELEVATION CONTOUR
	PROPOSED MAJOR ELEVATION CONTOUR
— — —	PROPOSED LIMIT OF WASTE

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.
8. PLAN PROVIDED BY PBS&J FROM "TECHNICAL DRAINAGE STUDY FOR EASTSIDE LANDFILL", DATED 6 OCT 2006.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: STORM DRAIN PLAN

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APPROVED BY:	G.T.C.		

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LEGEND

—PL—	BRC LANDFILL SITE BOUNDARY
— — —	EXISTING LIMITS OF FORMER BMI LANDFILL
- - -	EXISTING SURFACE WATER FLOW PATH
	EXISTING MINOR ELEVATION CONTOUR
	EXISTING MAJOR ELEVATION CONTOUR
	PROPOSED MAJOR GRADING ELEVATION CONTOUR
	PROPOSED MINOR GRADING ELEVATION CONTOUR
	PROPOSED MINOR COVER ELEVATION CONTOUR
	PROPOSED MAJOR COVER ELEVATION CONTOUR
— — —	PROPOSED LIMIT OF WASTE

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: FINAL COVER GRADING PLAN

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DATE SEAL	APPROVED BY: G.T.C.	

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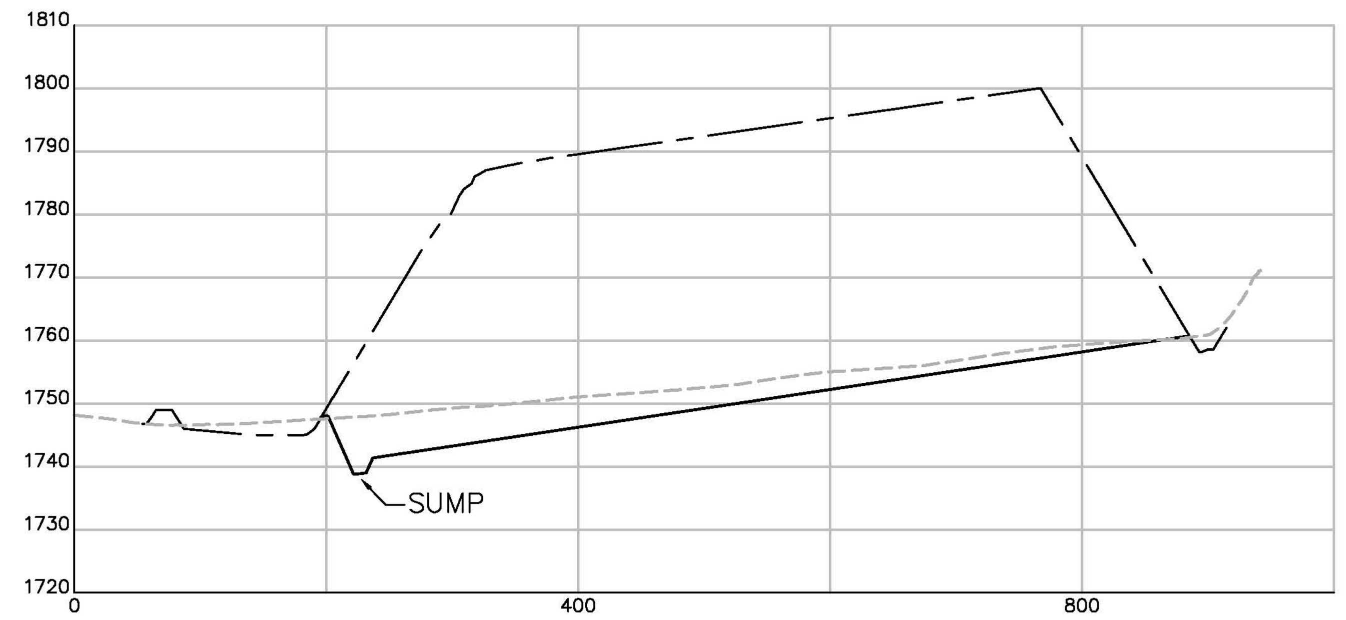
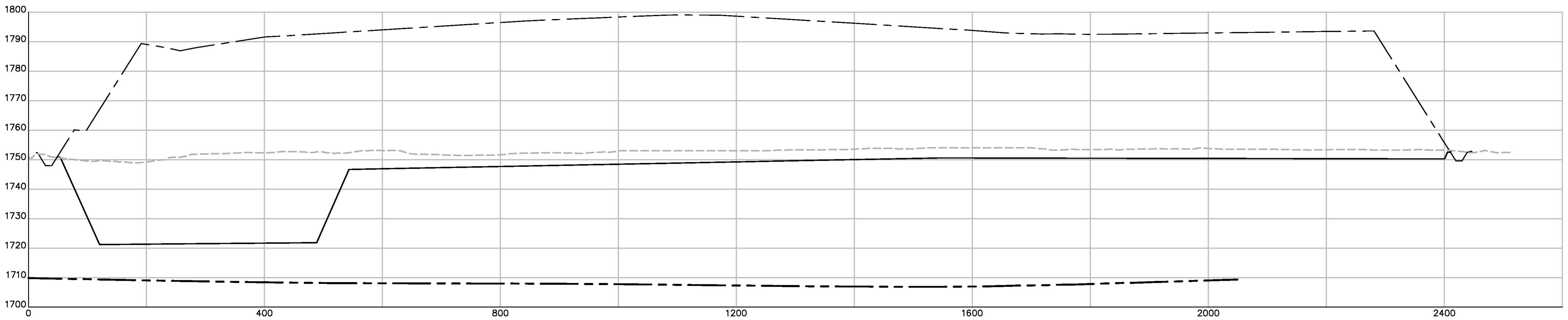
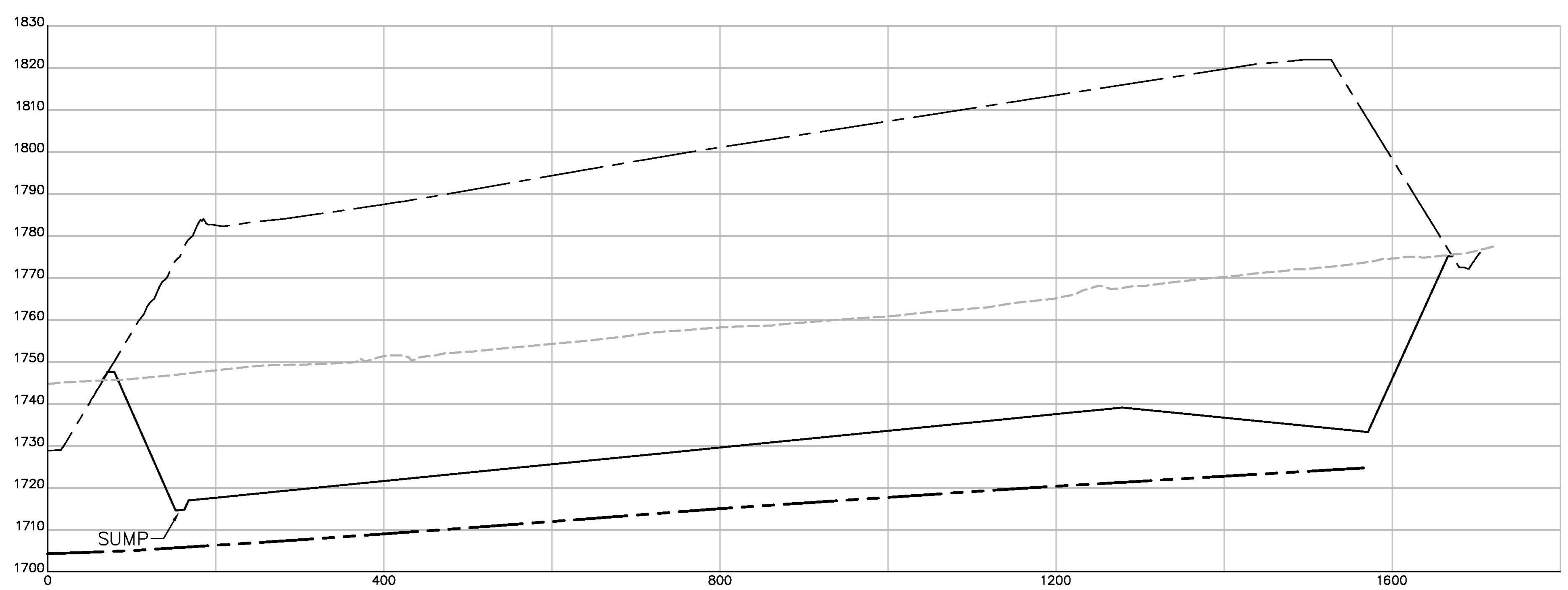


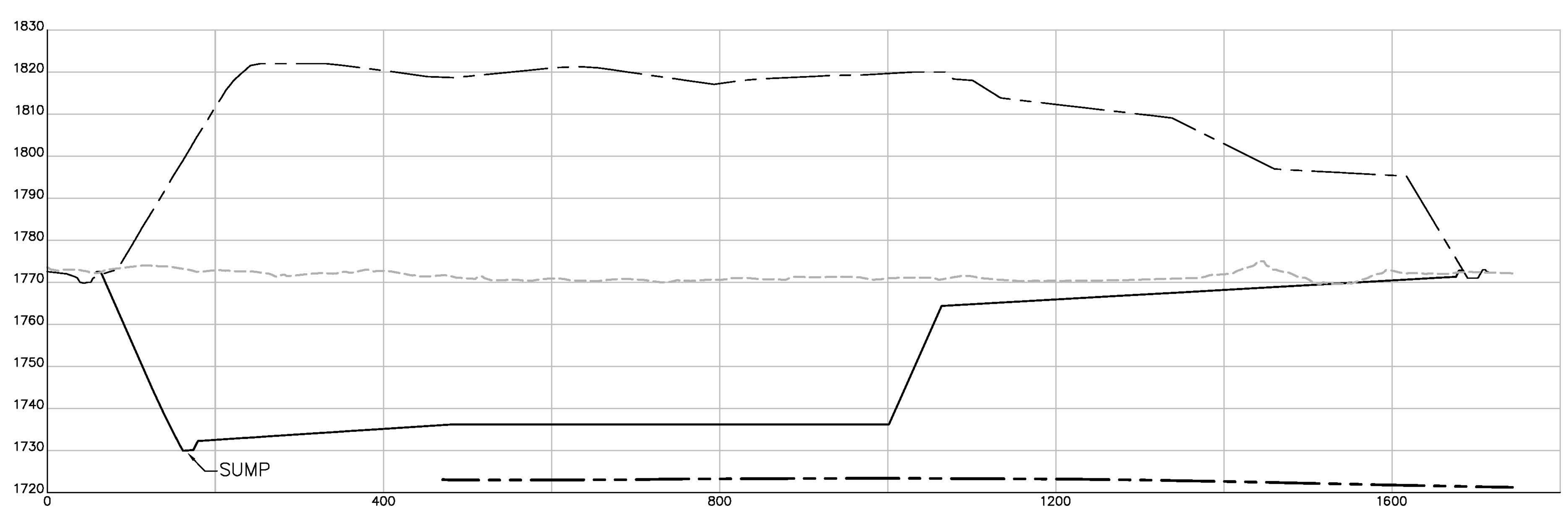
A–A 7,9 SECTION
SCALE: HORIZ. 1" = 100', VERT. 1" = 20'



B–B 7,9 SECTION
SCALE: HORIZ. 1" = 100', VERT. 1" = 20'



C–C 7,9 SECTION
SCALE: HORIZ. 1" = 100', VERT. 1" = 20'



D–D 7,9 SECTION
SCALE: HORIZ. 1" = 100', VERT. 1" = 20'

LEGEND

FINAL COVER SURFACE
EXCAVATION SURFACE
EXISTING GROUND SURFACE
GROUNDWATER SURFACE

NOTES:
1. GROUNDWATER CONTOURS PROVIDED BY MWH, DATED JULY 2005.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: GRADING CROSS SECTIONS

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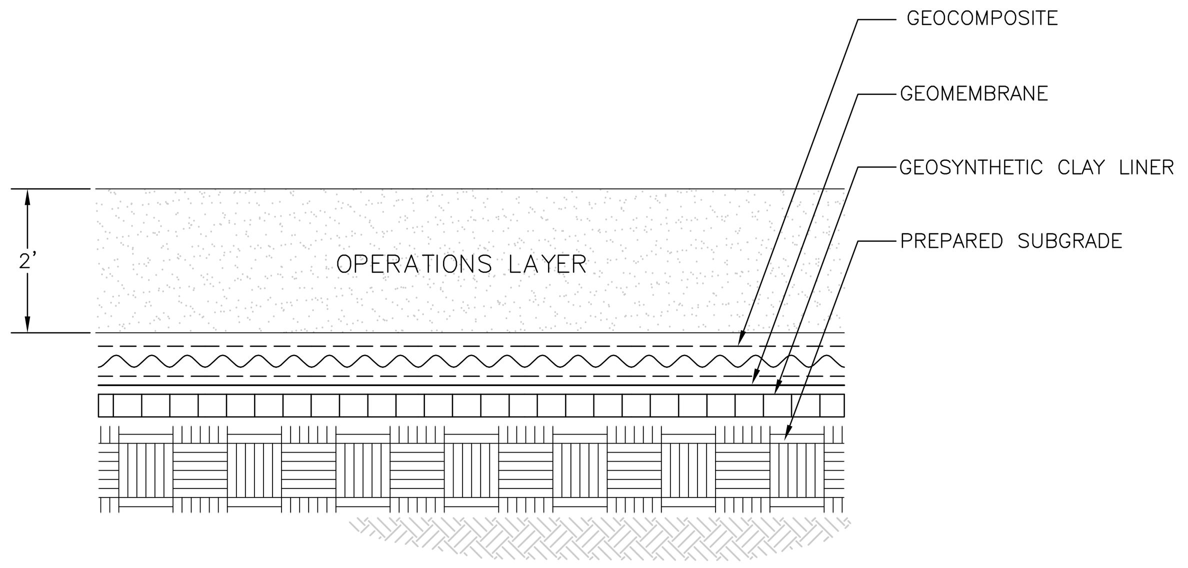
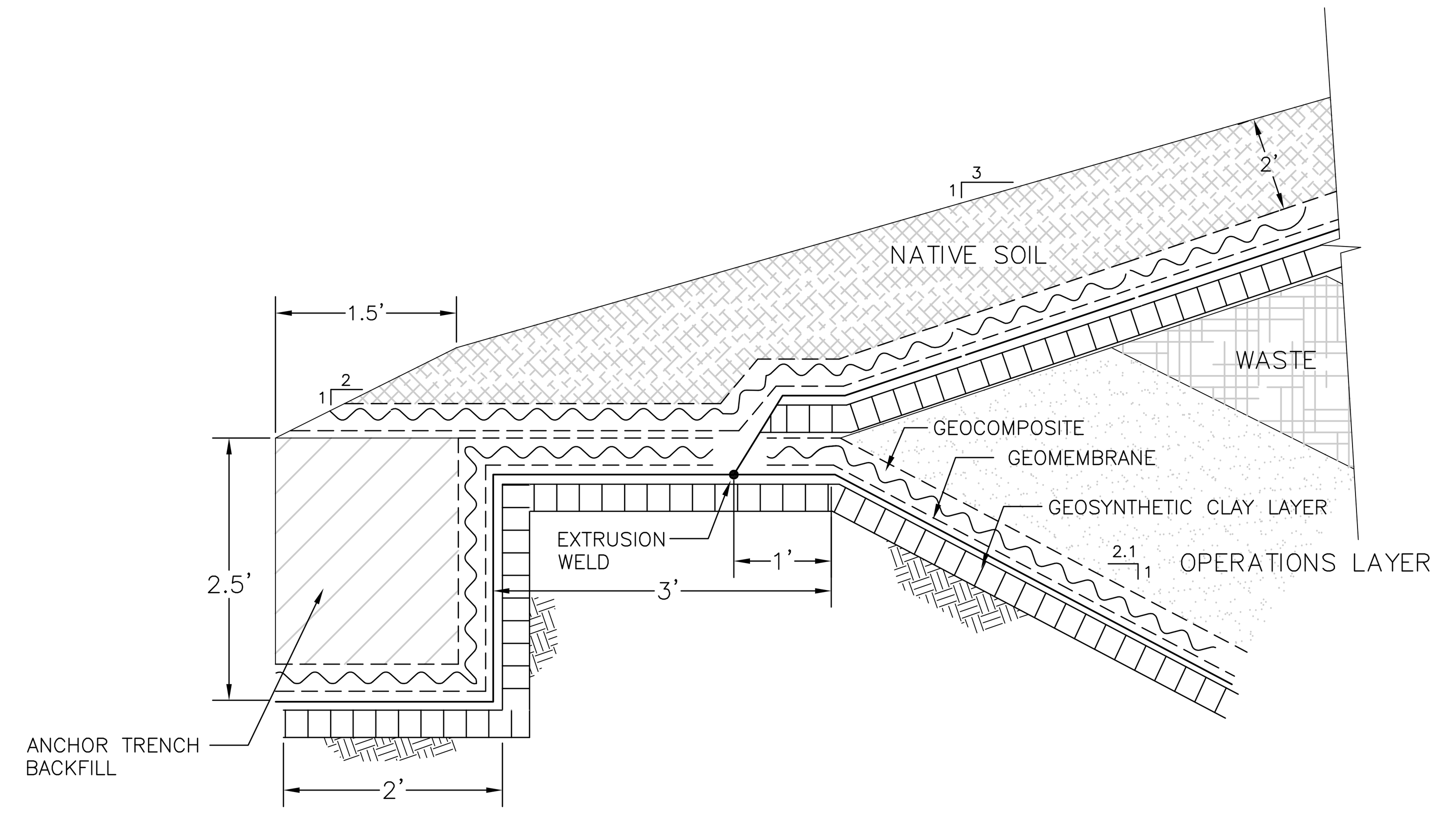
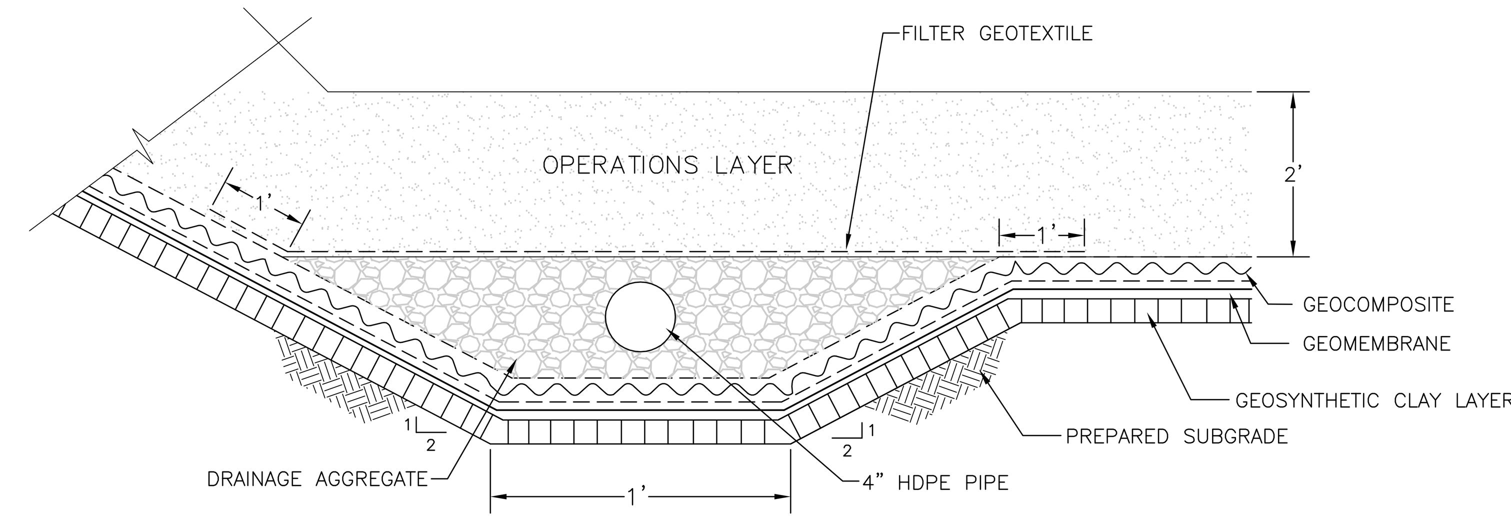
REVIEWED BY:	G.T.C.	DRAWING NO:	J–10
APPROVED BY:	G.T.C.		



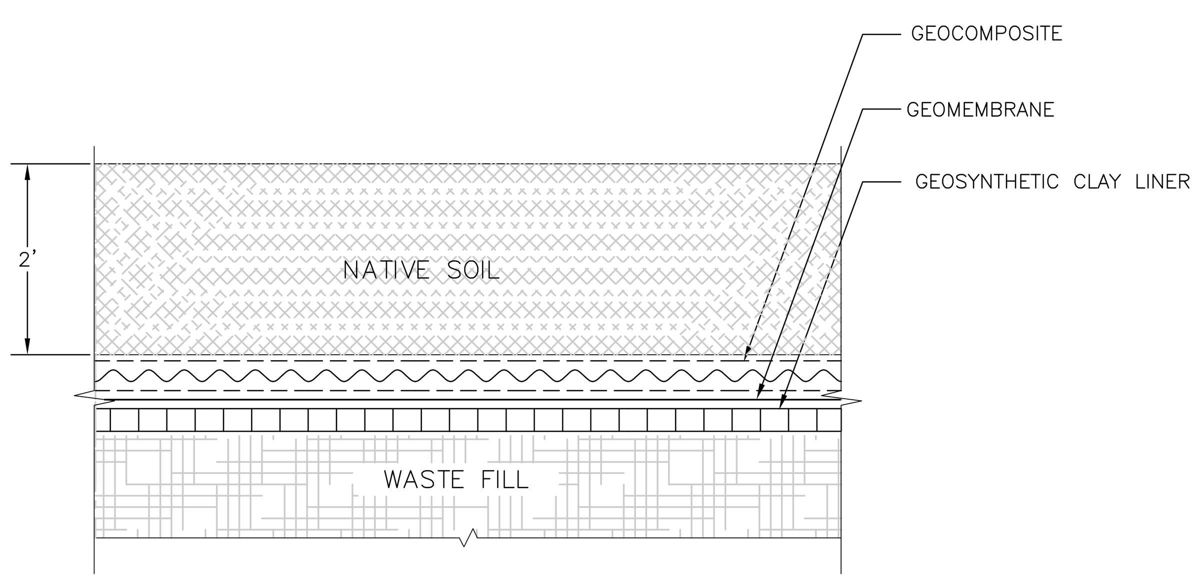
COMPOSITE LINER SECTION
NOT TO SCALE
1
J-7



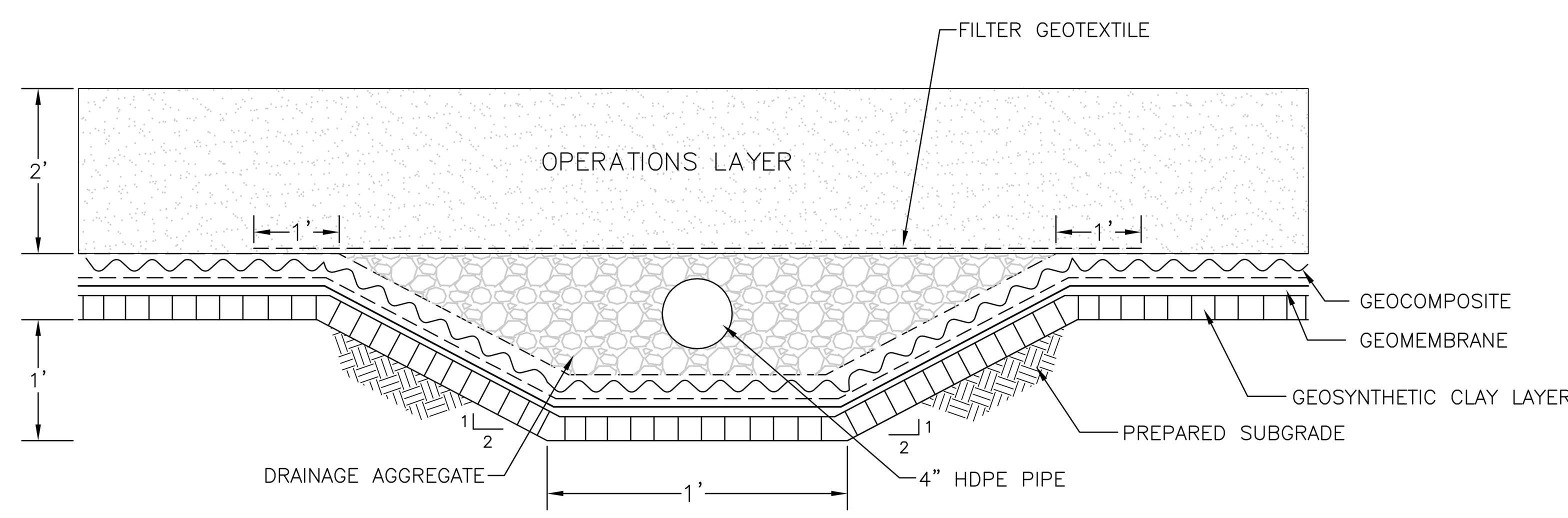
CROSS SECTION
ANCHOR TRENCH
NOT TO SCALE
3
J-7



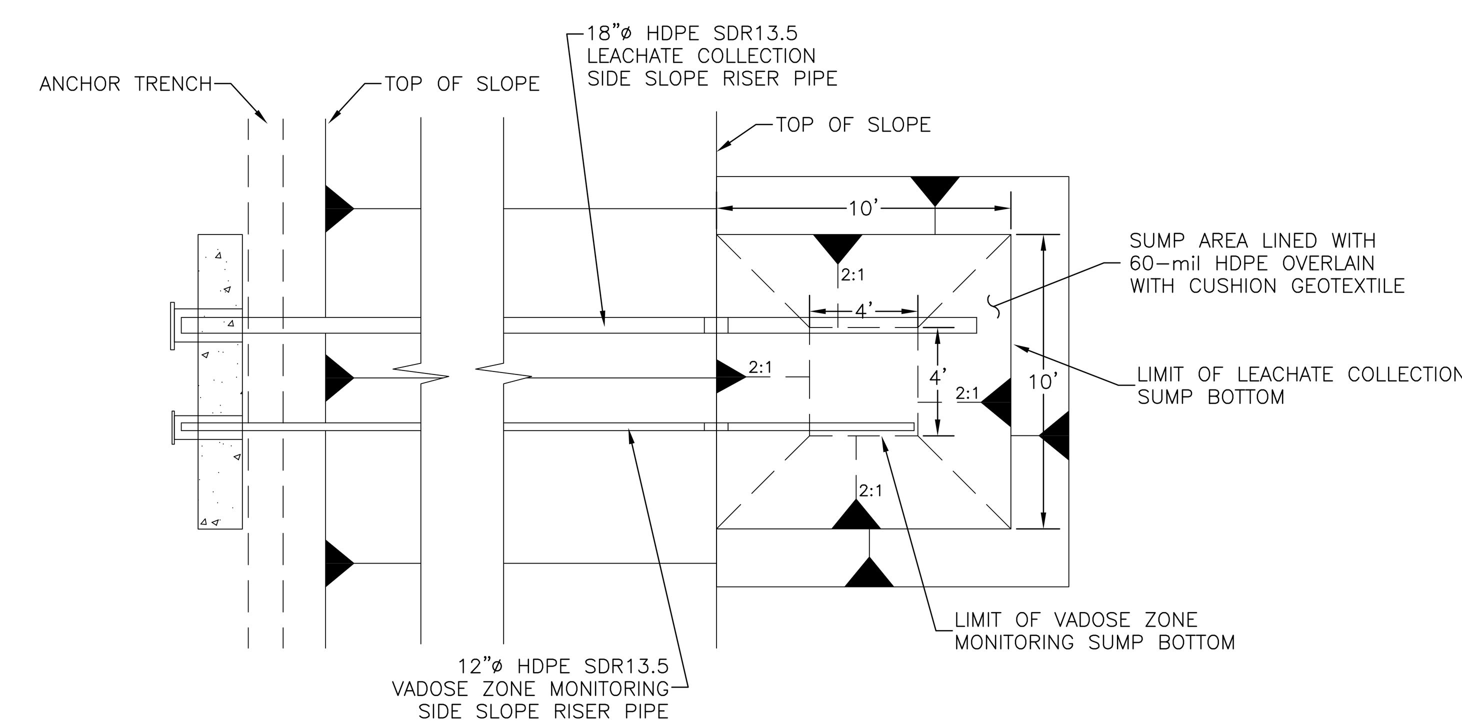
CROSS SECTION
LCRS COLLECTION TRENCH AT TOE OF SLOPE
NOT TO SCALE
4
J-7



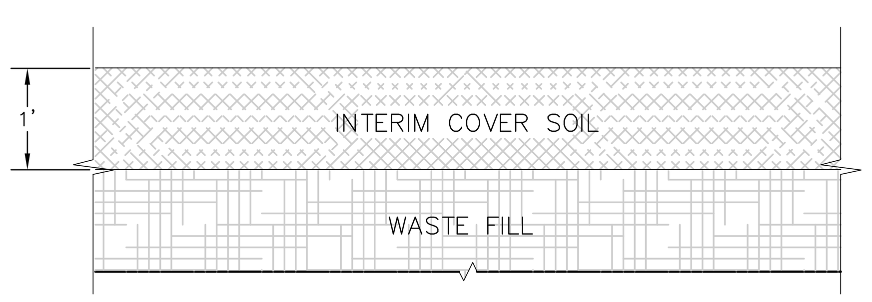
FINAL CAP COVER SECTION
NOT TO SCALE
2A
J-9



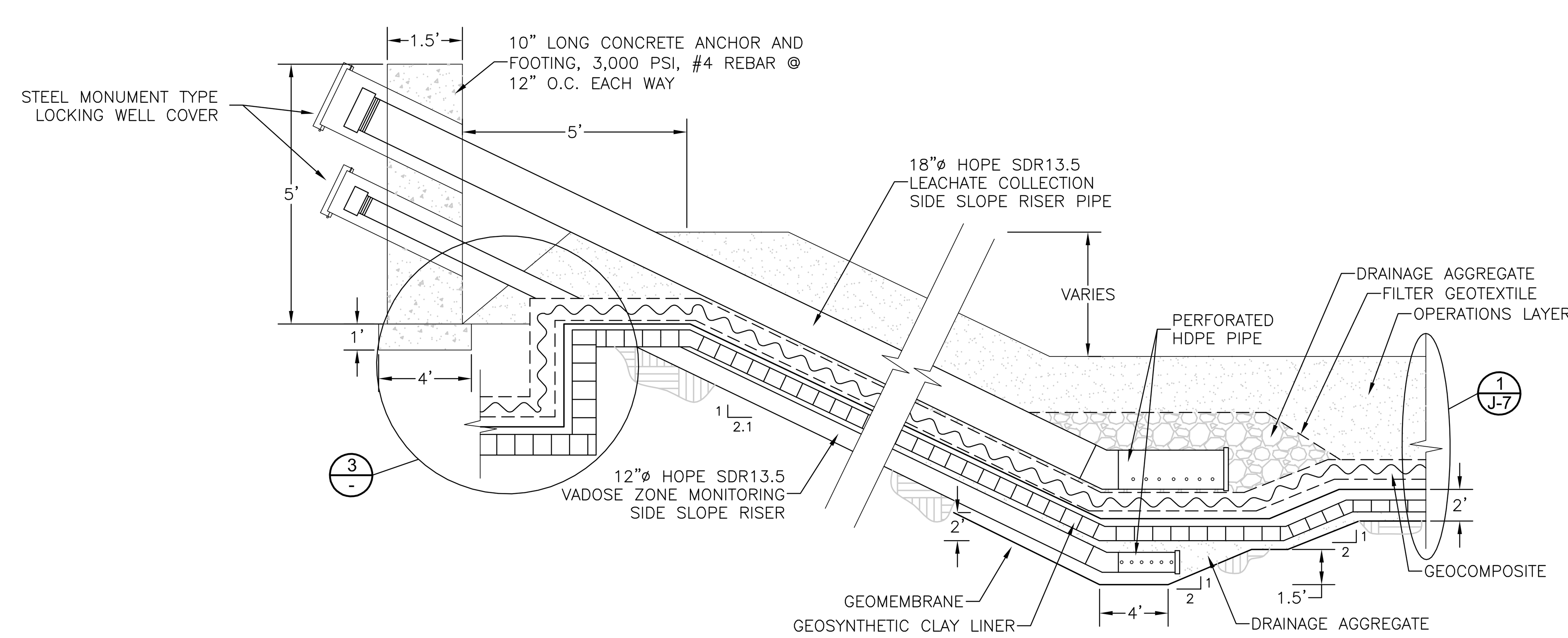
CROSS SECTION
LCRS COLLECTION TRENCH
NOT TO SCALE
5
J-7



DETAIL
LEACHATE COLLECTION AND VADOSE
ZONE MONITORING SUMP
NOT TO SCALE
6
J-7



INTERIM CAP COVER SECTION
NOT TO SCALE
2B
J-9



CROSS SECTION
VADOSE ZONE MONITORING
SIDE SLOPE RISER
NOT TO SCALE
7
J-7

NOTES:
1. GEOSYNTHETIC MATERIALS SHOWN AT EXAGGERATED SCALE FOR CLARITY.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT:
BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE:
FINAL COVER AND LINER DETAILS AND SECTIONS

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	CHECKED BY: G.T.C.	DOCUMENT NO:
SIGNATURE	REVIEWED BY: G.T.C.	DRAWING NO: J-11
DATE / SEAL	APPROVED BY: G.T.C.	

LEGEND

- —PL— BRC LANDFILL SITE BOUNDARY
- EXISTING LIMITS OF FORMER BMI LANDFILL
- EXISTING SURFACE WATER FLOW PATH
- EXISTING MINOR ELEVATION CONTOUR
- EXISTING MAJOR ELEVATION CONTOUR
- PROPOSED MINOR ELEVATION CONTOUR
- PROPOSED MAJOR ELEVATION CONTOUR
- PROPOSED LIMIT OF WASTE
- EXISTING FENCE
- —x—x— PROPOSED FENCE

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG J-2.
2. FOR SITE PREPARATION PLAN SEE DWG J-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG J-7.
4. FOR STORM DRAIN PLAN SEE DWG J-8.
5. FOR FINAL COVER GRADING PLAN SEE DWG J-9.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG J-11.
7. FOR FENCE PLAN SEE DWG J-12.

GEOSYNTEC CONSULTANTS
10875 RANCHO BERNARDO ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE: FENCE PLAN

MARK	DATE	REVISION	BY	APPROVED

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DATE

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DRAWN BY:	J.R.H.	FILE NO:	
CHECKED BY:	G.T.C.	DOCUMENT NO:	
REVIEWED BY:	G.T.C.	DRAWING NO:	J-12
APPROVED BY:	G.T.C.		

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Section 2
Base Liner Calculation Packages

Section 2
Base Liner Calculation Packages

Calculation Package A
Cut Slope Stability

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: BRC **Project:** BRC CAMU **Project #:** SC0313 **Task #:** 01 02

Title of Computations: Slope Stability Evaluation – Cut Slopes

Computations By:

SIGNATURE — 27 March 2005 DATE

Edward M. Zielanski, P.E./Project Civil Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE — 6/2/05 DATE

Gregory T. Corcoran, P.E./Associate
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE — 6/2/05 DATE

Gregory T. Corcoran, P.E./Associate
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE — 27 March 2005 ~~23 March 2000~~ DATE

Edward M. Zielanski, P.E./Project Civil Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE — 6/2/05 DATE

Gregory T. Corcoran, P.E./Associate
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Ed Zielanski Date: 5 / 27 / 05 Reviewed by: GRC Date: 6 / 2 / 05
MM DD YY MM DD YY

Client: BRC Project: BRC CAMU Project No.: SC0313 Task No. 01-02

SLOPE STABILITY EVALUATION
CUT SLOPES
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

The objective of this calculation package is to evaluate the stability of cut slopes (inclined at 2.1H:1V) at the BRC Corrective Action Management Unit (CAMU) in Henderson, Nevada.

DESIGN CRITERION

Because the cut slopes at the BRC CAMU will eventually be buttressed by waste they are considered interim slopes, which are normally required to exceed a static factor of safety of 1.3. However, due to limited information regarding shear strength parameters for on-site soils, GeoSyntec employed a static factor of safety equal to or greater than 1.4 as the interim slope static stability criterion. In consideration of the limited duration which the interim slopes will exist, seismic stability analyses of the cut slopes were not conducted.

METHOD OF ANALYSIS

The slope stability computer program PCSTABL5 (Achilleos, 1988) was employed for this analysis. PCSTABL5 employs limit equilibrium principles to provide general solutions to slope stability problems. Potential sliding surfaces, both circular and polygonal, can be specified or randomly generated. The Modified Janbu Method is used herein, as recommended in the PCSTABL5 manual (Achilleos 1998) for circular and polygonal failure surfaces.

In addition to the above analyses, the stability of the cut slopes is evaluated using the infinite slope methodology. An inifinite slope (or surficial sliding) without seepage in the slope is defined as:

$$FS = \frac{c}{\gamma H \cos^2 \beta \tan \beta} \frac{\tan \phi}{\tan \beta} \quad \text{(Das 1994)} \qquad \text{(Equation 1)}$$

SUBSURFACE INFORMATION

Based on results of subsurface explorations by others (Converse 1999), the subsurface is characterized by alluvial granular soils overlying fine-grained soils encountered at depths from approximately 34-ft to 55-ft below the surface. The granular subsoils generally consist of medium to very dense granular fill and native soils overlying localized zones of moderately hard to hard cemented sand and gravel. The fine-grained soils consist of lean clays and high plasticity silts. Field and laboratory test results indicated that the native granular soils at the site have a low compressibility and moderate to high internal friction angles (Converse 1999). The fine-grained soils encountered at depth at the site generally were found to be moderately compressible, have a high expansion potential, and have relatively low permeability (Converse 1999). Groundwater was encountered at depths between 30-ft and 58-ft and was, in general, located at the approximate elevation of the lean clay layer (Converse 1999).

NATIVE MATERIAL PARAMETERS

Native material within the limits of the BRC CAMU consists of alluvial granular soils overlying fine-grained soils. Shear strength parameters for the native soil material were previously estimated and reported in the *Preliminary Geotechnical and Geologic Investigation - Industrial Non-Hazardous Disposal Facility* (Converse 1999). Twelve exploratorary borings were conducted by others to depths ranging from 33-ft to 60-ft (Converse 1999). In general, the native materials appear to be consistent between borings. Direct Shear tests were performed on selected samples retrieved from the exploratory borings. A summary of the Direct Shear test results as reported by Converse (1999) are presented in Attachment 1.

Based on data obtained by Converse (1999), the *in situ* silty sand with gravel material (fill and native) can be characterized by a moist unit weight of approximately 117 pcf (see Attachment 1 - boring logs). The sandy lean clay can be characterized by a saturated unit weight of approximately 102 pcf (see Attachment 1 - boring logs).

Based on the boring logs provided by Converse (1999), the average groundwater elevation in the vicinity of the cut slopes (borings B-8, B-12, B-5, B-4, and B-10) is approximately 49 feet below the surface (at the top of the CL layer).

Upon review of the direct shear tests performed by Converse at the BRC CAMU site, GeoSyntec has employed its own interpretation of the on-site material properties. The direct shear tests performed by Converse involved shearing the soil at a relatively low normal stress, then returning the specimen to its original position, placing a larger normal stress and re-shearing the sample. This process was repeated for three different normal loads. Therefore, based on the description of the testing methods, it is unclear as to the appropriate design shear strength parameters for the on site soil.

In recognition of this, GeoSyntec has reviewed the borings logs in order to evaluate the design shear strength parameters. The boring logs indicate that the *in situ* silty sand with gravel soil (SM) is primarily dense to very dense (i.e, very high blow counts). Based on a correlation by Peck, Hanson, and Thornburn (1974), dense silty sands typically have internal friction angles on the order of 30 to 35 degrees (Attachment 4). Due to the high density of the SM material, it is assumed that the effective friction angle of the SM material to be 35 degrees. At numerous locations shown in the boring logs, the SM material was described as partially cemented. However, based on the borings logs, it is unclear as to the extent or the degree of cementation in the profile. Therefore, it is conservatively assumed that the effective cohesion of the SM material is zero.

A sandy lean clay (CL) layer exists approximately 34 to 55 feet below the surface. Based on a review of the boring logs in the area of the proposed cut slopes, the average depth to the CL layer (for borings B-8, B-12, B-5, B-4, and B-10) is approximately 49 ft. The moisture content of the lean clay is relatively close to the liquid limit, indicating a normally consolidated deposit. Therefore, the undrained shear strength of the lean clay layer can be approximated using correlations based on the plasticity index. A representative value from Converse (1999) for the plasticity index of the CL material is on the order of 32 percent (Attachment 1). Ladd (1990), presents a relationship between the plasticity index and the c/p ratio. Based on this correlation, the c/p ratio is approximately 0.24 (Attachment 2). An example calcuation to estimate the undrained shear strength at a depth of 54 feet is shown below:

Assume the SM material and water table extends to a depth of 49 ft. Therefore, the effective vertical stress, σ'_v *is:* $(\gamma_{SM})(z_{SM})+(\gamma' t_{CL}) = (117)(49\ ft)+(102-62.4)(54-49) = 5{,}931$ *psf. The c/p ratio is assumed to be 0.24, therefore, the undrained strength is* $(0.24)(5931) \approx 1{,}400$ *psf.*

CROSS SECTIONS

One cross section (cross section A-A') was developed to evaluate the stability of the cut slopes at the BRC CAMU. The location of this cross section is indicated on Figure 1. Cross section A-A' is a 44-ft high cut slope and represents the most critical cross section. The slope of the cross section is inclined at 2.1H:1V (horizontal:vertical).

ANALYSIS AND RESULTS

Proposed Cut Slopes – 2.1H:1V

Global Stability. Based on data from Converse (1999), the groundwater table appears to be located approximately at the elevation of the lean clay layer. In recognition of this, a piezometric surface was included in the analyses to represent the water table. The water table is placed at the top of the lean clay layer.

Two types of potential failure surfaces were analyzed, (i) circular, and (ii) polygonal. The polygonal potential failure surface assumes the failure surface occurs between the weakest interface between the silty sand (SM) and the lean clay (CL). Potential failure surfaces were evaluated using the search option of the PCSTABL5 program. Program input parameters, specifying the range of beginning and ending locations for potential circular failure surfaces, were varied by the user to focus on the location of the most critical potential failure surfaces for the given cross section.

Graphical output of the most critical potential failure surfaces of cross section A-A' is presented in Figures 2, 3, and 4. The graphical output of Figures 2 and 3 illustrates the ten most critical potential circular failure surfaces found by the PCSTABL5 analysis. The graphical output of Figures 2 and 3 illustrates the ten most critical polygonal failure. Analyses indicate that the most critical potential failure surface of cross section A-A', possesses a factor of safety of 1.46. This factor of safety satisfies the design criterion of a factor of safety equal to or greater than 1.4.

Computer output of the stability analyses for the potential surfaces of cross section A-A' is included in Attachment 3.

Surficial Sliding or Sloughing. Based on the assumption of zero cohesion for the SM material, Equation 1 becomes:

$$FS = \frac{\tan \phi}{\tan \beta} \qquad \text{(Equation 2)}$$

For the proposed slopes, the inclination is 2.1H:1V, therefore the factor of safety is:

$$FS = \frac{\tan 35}{1/2.1} = 1.47$$

CONCLUSIONS

- The minimum factor of safety for the proposed cut slopes (2.1H:1V) is 1.46;
- The undrained shear strength of the CL layer can be characterized with a c/p ratio of 0.24 and a unit weight of 102 pcf; and
- The shear strength of the SM material can be conservatively characterized as c'=0 and ϕ'=35 degrees and a unit weight of 117 pcf.

REFERENCES

Achilleos, E. (1988), "PC STABL5M, User Manual" *Information Report,* School of Civil Engineering, Purdue University, West Lafayette, Indiana, 132 p.

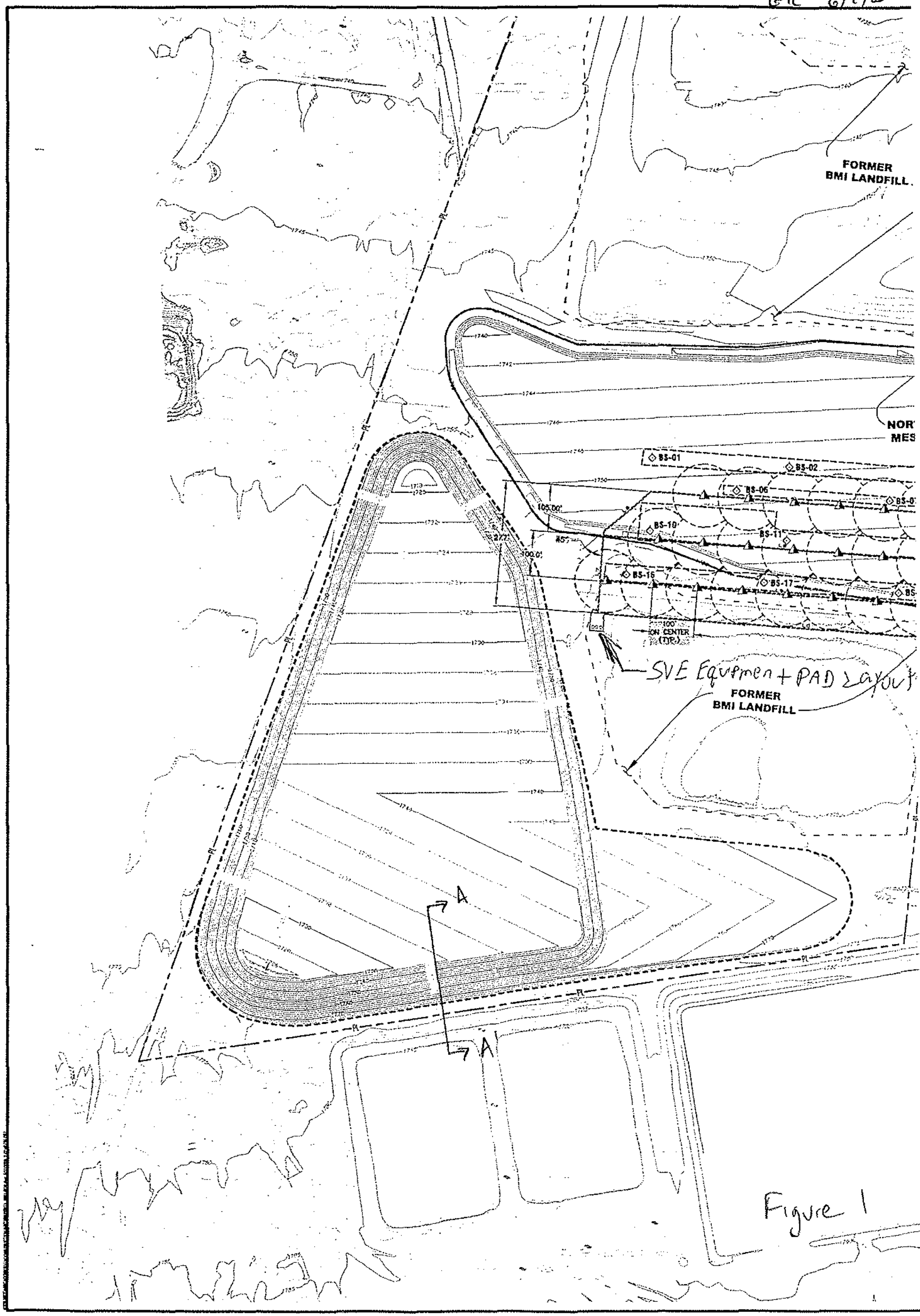
Converse (1999), "Preliminary Geotechnical and Geologic Investigation - Industrial Non-Hazardous Disposal Facility", prepared for Basic Management, Inc., October 1999. *(Attachment 1)*

Das B. M. (1994), "Principles of Geotechnical Engineering," Third Edition, PWS Publishing Company, Boston, MA

Peck, Hanson, and Thornburn (1974), "Foundation Engineering," Second Edition, John Wiley & Sons, Inc. New York. *(Attachment 4)*

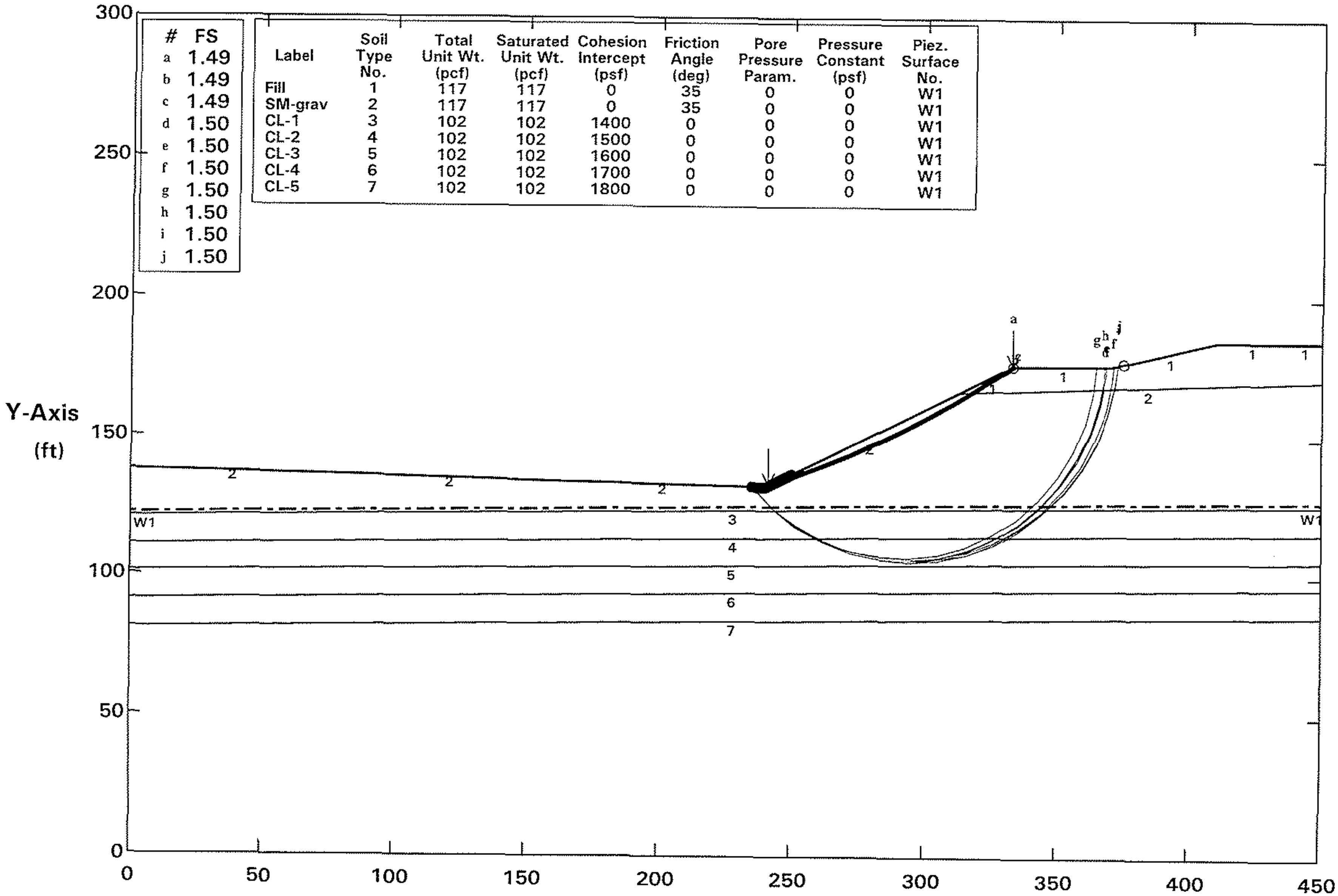
Ladd, C. C., (1990) "Soft Ground Engineering," 13th Annual Spring Seminar, Geotechnical Engineering Group, Los Angeles Section of ASCE. *(Attachment 2)*

GTC 6/2/05


FORMER
BMI LANDFILL
NORT
MES
BS-01
BS-02
BS-06
BS-10
BS-11
BS-16
BS-17
100.00'
100'
ON CENTER
(TYP.)
SVE Equipment + PAD Layout
FORMER
BMI LANDFILL
A
A'
Figure 1

Section A-A' BRC CAMU, Henderson, Nevada

Ten Most Critical. C:ANEW.PLT By: EMZ 5/27/2005 9:56am



#	FS
a	1.49
b	1.49
c	1.49
d	1.50
e	1.50
f	1.50
g	1.50
h	1.50
i	1.50
j	1.50

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	117	117	0	35	0	0	W1
SM-grav	2	117	117	0	35	0	0	W1
CL-1	3	102	102	1400	0	0	0	W1
CL-2	4	102	102	1500	0	0	0	W1
CL-3	5	102	102	1600	0	0	0	W1
CL-4	6	102	102	1700	0	0	0	W1
CL-5	7	102	102	1800	0	0	0	W1

PCSTABL5 FSmin = 1.49 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

Figure 2

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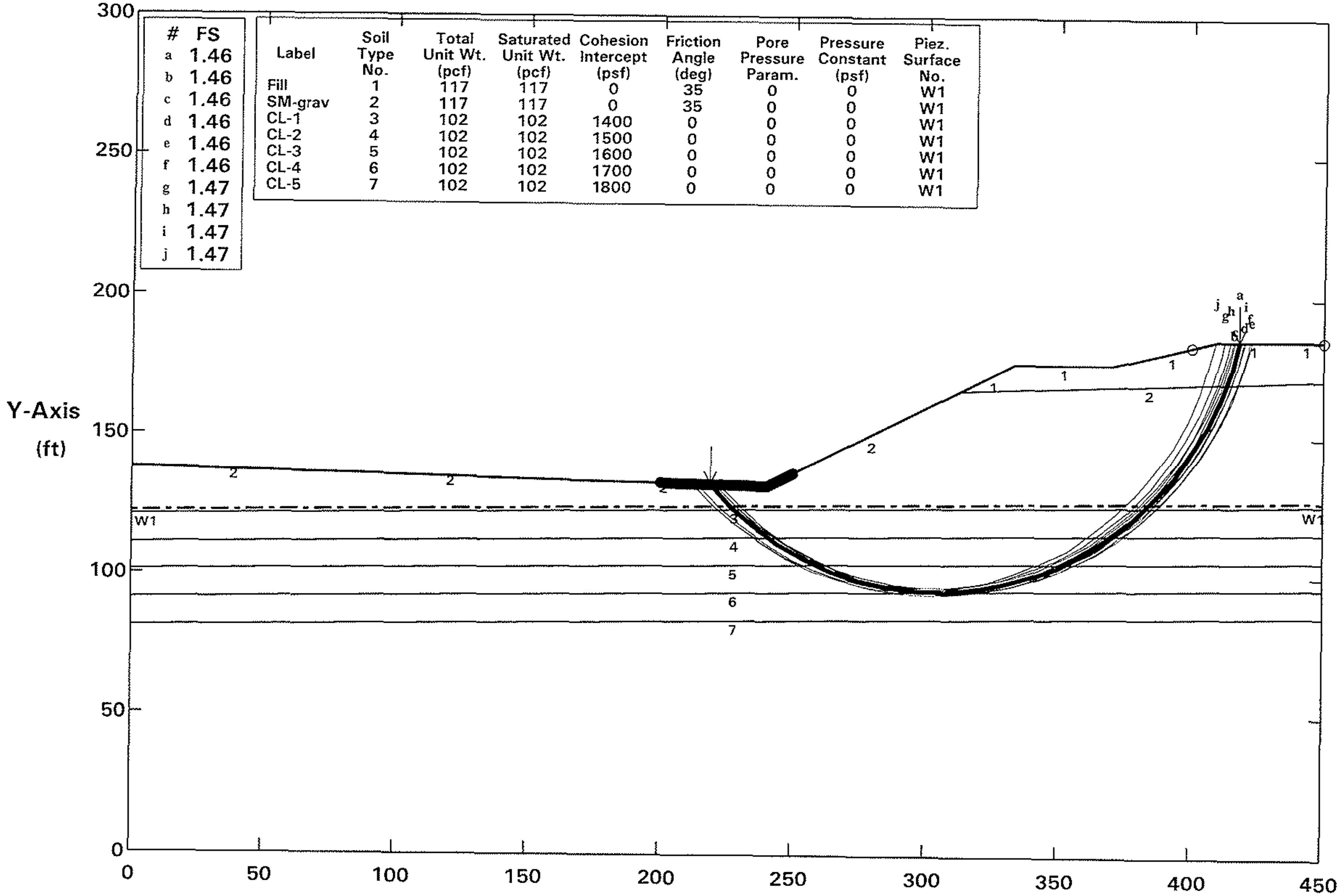
Section A-A' BRC CAMU, Henderson, Nevada

Ten Most Critical. C:A2NEW.PLT By: EMZ 5/27/2005 10:06am



#	FS
a	1.46
b	1.46
c	1.46
d	1.46
e	1.46
f	1.46
g	1.47
h	1.47
i	1.47
j	1.47

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	117	117	0	35	0	0	W1
SM-grav	2	117	117	0	35	0	0	W1
CL-1	3	102	102	1400	0	0	0	W1
CL-2	4	102	102	1500	0	0	0	W1
CL-3	5	102	102	1600	0	0	0	W1
CL-4	6	102	102	1700	0	0	0	W1
CL-5	7	102	102	1800	0	0	0	W1

PCSTABL5 FSmin = 1.46 X-Axis (ft)

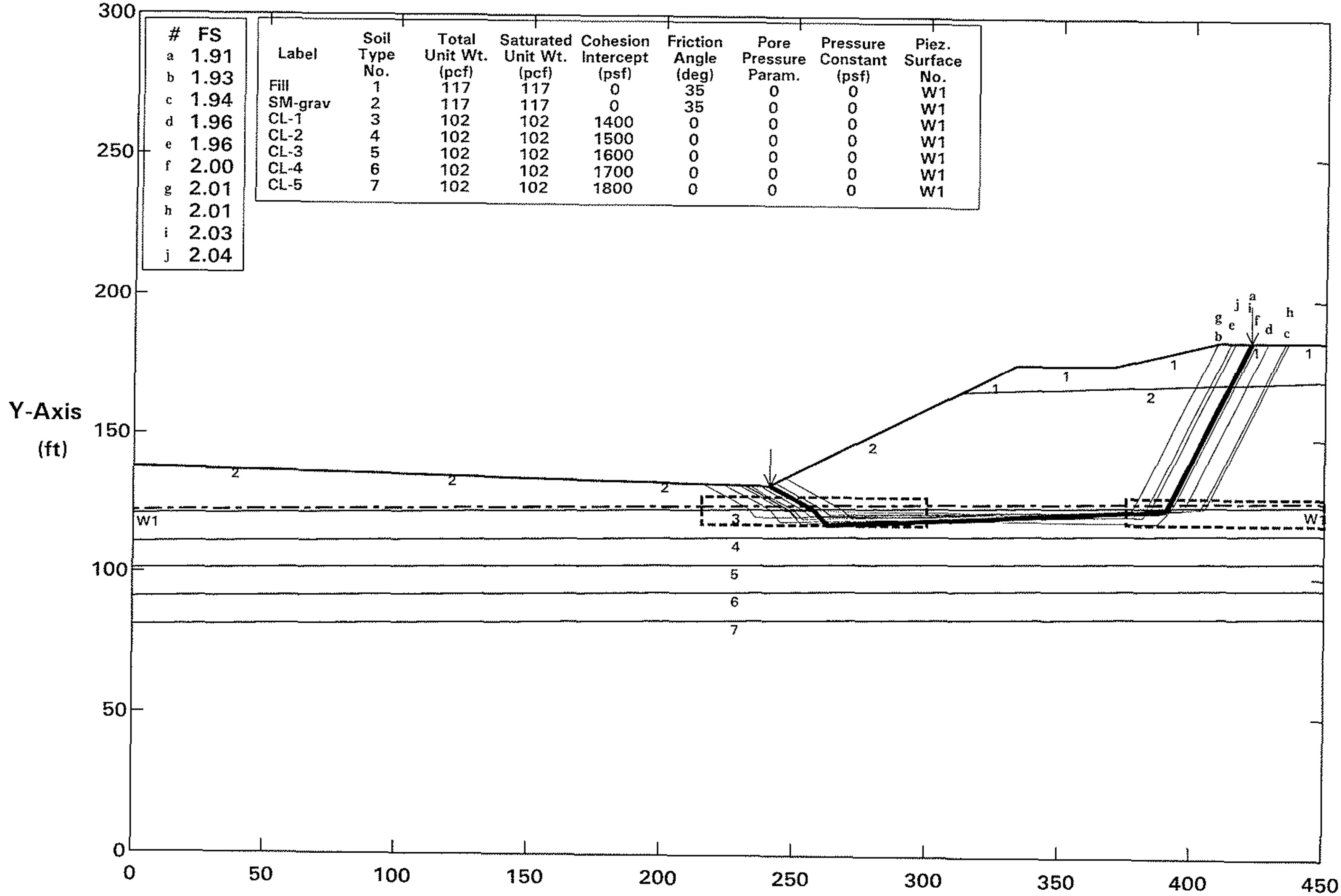
Factors Of Safety Calculated By The Modified Janbu Method

Figure 3

CTC 6/2/05

Section A-A' BRC CAMU, Henderson, Nevada

Ten Most Critical. C:A3NEW.PLT By: EMZ 5/27/2005 10:14am



#	FS
a	1.91
b	1.93
c	1.94
d	1.96
e	1.96
f	2.00
g	2.01
h	2.01
i	2.03
j	2.04

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	117	117	0	35	0	0	W1
SM-grav	2	117	117	0	35	0	0	W1
CL-1	3	102	102	1400	0	0	0	W1
CL-2	4	102	102	1500	0	0	0	W1
CL-3	5	102	102	1600	0	0	0	W1
CL-4	6	102	102	1700	0	0	0	W1
CL-5	7	102	102	1800	0	0	0	W1

PCSTABL5 FSmin = 1.91 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

Figure 4

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Direct Shear Strength

A progressive direct shear test was performed on selected undisturbed samples using a constant strain rate direct shear machine in general accordance with ASTM D3080. The test specimen was trimmed and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until maximum shear strength was developed. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. Another normal load was then applied, and the specimen was sheared a second time. This process was repeated for three different normal loads. Results of the direct shear test are presented on Figures A-62 through A-69 and in the following table:

Exploration Location	Depth (feet)	Soil Description	Angle of Internal Friction (deg)	Coulomb Cohesion (ksf)
B-4	14-14.5	Silty sand with gravel	31	0.7
B-5	14-15	Silty sand with gravel	43	0.3
B-10	54-54.5	Sandy lean clay	26	0.85
B-12	14-15	Silty sand with gravel	40	0.3
B-101	39-40	Sandy lean clay	26	0.9
B-102	20-25	Silty sand with gravel	37	0.2
B-103	49-50	Sandy lean clay	37	1.0
B-104	10-15	Silty sand with gravel	43	0.1

Chemical Analysis

Chemical tests were performed on a representative soil samples to investigate the potential for soil corrosivity and chemical heave. Atlas Chemical Testing Laboratories, Inc. in Las Vegas performed the chemical analysis for water-soluble sulfates and sodium in general accordance with ASTM D516. The results of the chemical tests are presented on Drawing No. A-70.

AH. 1
1/3

GTC MLD

Grain Size Distribution

Grain size distribution for soil samples were determined by sieve analysis in accordance with ASTM C136. A sieve analysis is conducted by passing the soil through a number of different sized sieves and measuring the amount of soils retained on each sieve. The test results and grain size distribution curves are presented on Drawing Nos. A-37 through A-48.

Atterberg Limits

The liquid limit, plastic limit and plasticity index of a representative sample of the fine-grained soils were determined to aid in the classification of the soils and in the evaluation of other engineering parameters. The test was performed in general accordance with ASTM test method D4318. The results of the tests are tabulated in the following table:

Exploration Location	Sample Depth, ft.	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Unified Soils Classification
B-1	30-35	NP	NP	NP	SM
B-5	20-25	NP	NP	NP	SM
B-10	30-35	NP	NP	NP	SM
B-12	10-15	NP	NP	NP	SM
B-101	39-40	105	71	34	MH
B-101	54-55	54	44	10	ML
B-102	20-25	NP	NP	NP	SM
B-102	49-50	88	58	30	MH
B-103	30-35	NP	NP	NP	SM
B-104	10-15	NP	NP	NP	SM
B-105	20-25	NP	NP	NP	SW-SM
B-106	0-5	NP	NP	NP	SM

NP = Nonplastic

Attachment 1

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Log No. B- 8

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 58.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Description	Drive	Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
40	SANDY LEAN CLAY With Gravel (CL); very stiff, dark brown, slightly moist	X		49				
42								
44	---dry	X		35/8"				K
46								
48	---partially cemented, moderately hard, white, dry							
49	SANDY LEAN CLAY (CL); very stiff, dark brown, moist	X		35/10"				pp=3.0
50								
52								
54		X		16		52	71 108	pp=2.0
56								
58	SILTY SAND (SM); dense, brown, wet							
59	SANDY LEAN CLAY (CL); stiff, dark brown, wet	X		13		51	70 106	pp=3.0

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

3701 GPJ

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Log No. B-12

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 37.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Drive	Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
40	SANDY LEAN CLAY (CL); very stiff, dark brown, moist	X		44				
45	---wet, stiff	X		15		80	53	95 pp=1.5
49.5		X		11				
53	---with gravel, very stiff							
54.5		X		16		31	87	114 pp=1.5
59.5		X		18				pp=1.5

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Log No. B-101

Date of Drilling: 9/20/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		X		10				
42	SILTY SAND With Gravel (SM); medium dense, grayish brown, dry							
	SANDY LEAN CLAY (CL); stiff, dark brown, moist							
44		X		11		45	67	97
46								
48								
		X		25				pp=1.5
50	---partially cemented lens, moderately hard, brownish white, wet							
52								
54		X		11		42	75	107 A
	---with gravel							
56								
58								
		X		14		42	74	106

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Clark County, Nevada

Project No.
99-33437-01

43701.GPJ

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Log No. B- 1

Date of Drilling: 9/23/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 53.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
	SILTY SAND With Gravel (SM); dense, light brown, slightly moist	X	X	35				
22								G;P
24		X		35		7	108	116
26								
28	---grayish brown							
		X		35/6"				
30	---very dense	X	X	53				A
								G
32								
34		X		35/3"				
36								
38								
	---dense	X		35/6"				

End of Exploration at 55.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

~~48~~ GTC MLD

Log No. B- 4

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 54.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Description	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); very dense, brown, dry			35/2"				
22									
24		---dense			49				K
26									
28									
		---few gravel			35		7	107	114
30					42				
32									
34		---with gravel, very dense			35/8"		4		
36									
		---with boulders							
38									
		CEMENTED SAND AND GRAVEL; hard, brown, dry			20/0"				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
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Clark County, Nevada

Project No.
99-33437-01

/18 GTC MLD

Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		FILL: SILTY SAND With Gravel; tan, dry			9				
2									
4		---slightly moist			35/5"				
					58				
6									
8		---with cobbles							
		SILTY SAND With Gravel (SM); dense, tan, slightly moist			31		5	115	121
10					29				
12									K;Sol; Ch
14					43		4	111 115	DS
16									
18									
		---very dense			57/8"				

End of Exploration at 60.0'   Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

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Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

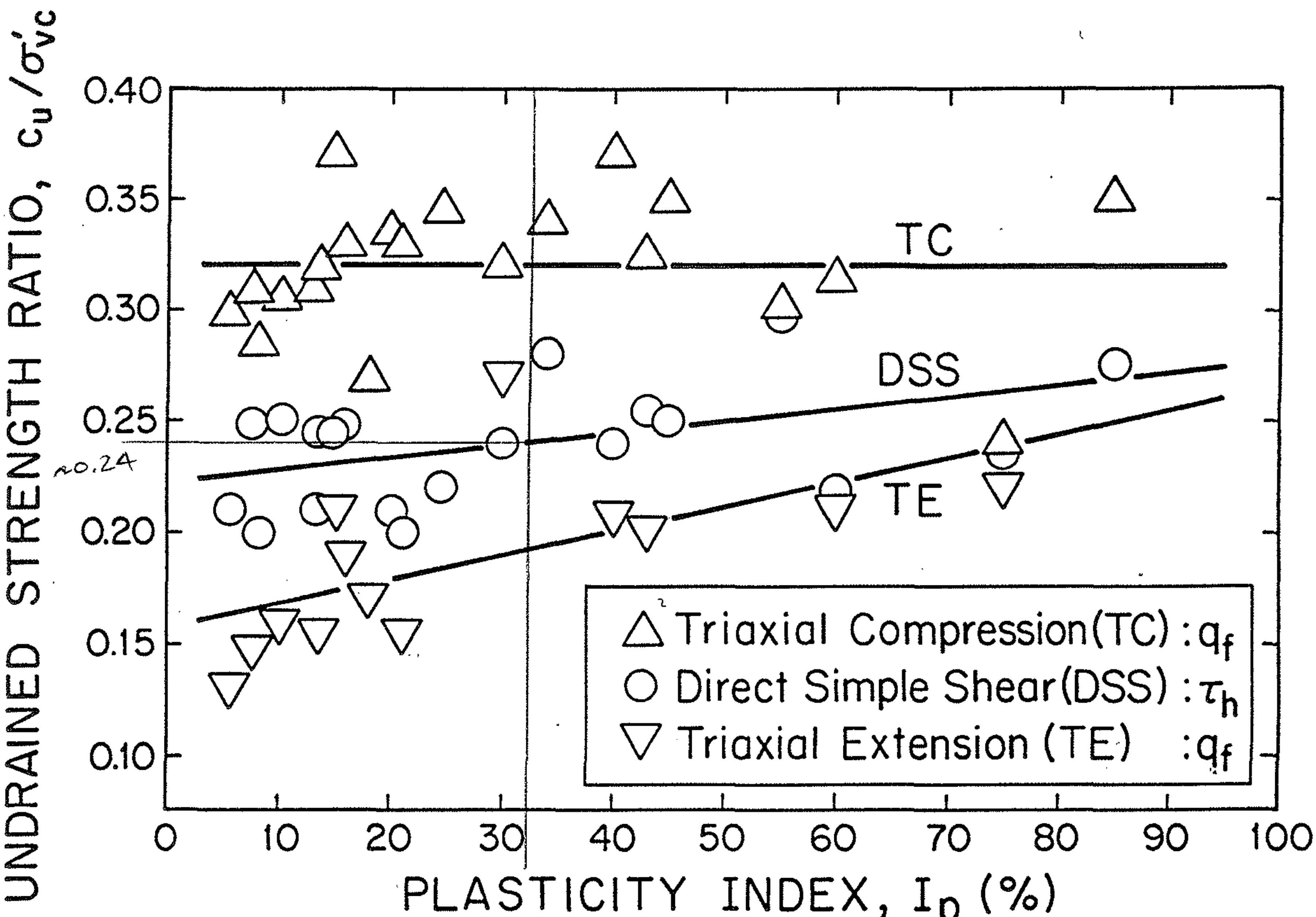
This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); dense, tan, slightly moist			32				A
22									G;P;Ch
24					58/8"				
		---very dense							
26									
28		---brown							
		CLAYEY SAND With Gravel (SC); dense, dark brown, slightly moist			35/5"		6	112	119
30					35/6"				
		SILTY SAND With Gravel (SM); dense, black, slightly moist							
32		---dark brown							
34		---very dense			50/3"				
36									
38									
		---with cobbles			50/6"				

APPROVED BY ON

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)



Attachment 2
1/1

FIG. 15.-Undrained Strength Anisotropy from CK_oU Tests on Normally Consolidated Clays and Silts (data from Lefebvre et al. 1983; Vaid Campanella 1974; and various MIT and NGI reports)

** PCSTABL5 **

by
Purdue University

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

Run Date: 5/27/2005
Time of Run: 9:56am
Run By: EMZ
Input Data Filename: C:ANEW
Output Filename: C:ANEW.OUT
Plotted Output Filename: C:ANEW.PLT

PROBLEM DESCRIPTION Section A-A'
BRC CAMU, Henderson, Nevada

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	138.00	80.00	136.00	2
2	80.00	136.00	164.00	134.00	2
3	164.00	134.00	240.00	132.00	2
4	240.00	132.00	313.00	166.50	2
5	313.00	166.50	333.00	176.00	1
6	333.00	176.00	370.00	176.00	1
7	370.00	176.00	410.00	185.00	1
8	410.00	185.00	436.00	185.00	1
9	436.00	185.00	450.00	185.00	1
10	313.00	166.50	450.00	170.75	2
11	.00	121.00	450.00	126.00	3
12	.00	111.00	450.00	116.00	4
13	.00	101.00	450.00	106.00	5
14	.00	91.00	450.00	96.00	6
15	.00	81.00	450.00	86.00	7

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Attachment 3A

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	117.0	117.0	.0	35.0	.00	.0	1
2	117.0	117.0	.0	35.0	.00	.0	1
3	102.0	102.0	1400.0	.0	.00	.0	1
4	102.0	102.0	1500.0	.0	.00	.0	1
5	102.0	102.0	1600.0	.0	.00	.0	1
6	102.0	102.0	1700.0	.0	.00	.0	1
7	102.0	102.0	1800.0	.0	.00	.0	1

---

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	122.00
2	450.00	127.00

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

2000 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of200 Points Equally Spaced Along The Ground Surface Between X = 235.00 ft.
and X = 250.00 ft.

Each Surface Terminates Between X = 333.00 ft.
and X = 375.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

11.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

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

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	241.18	132.56
2	251.59	136.12
3	261.90	139.96
4	272.10	144.08
5	282.18	148.48
6	292.14	153.14
7	301.97	158.08
8	311.67	163.27
9	321.22	168.73
10	330.62	174.45
11	333.02	176.00

*** 1.487 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	249.77	136.62
2	260.26	139.95
3	270.62	143.64
4	280.84	147.70
5	290.92	152.12
6	300.83	156.89
7	310.56	162.01
8	320.11	167.47
9	329.46	173.27
10	333.55	176.00

*** 1.490 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	242.24	133.06
2	252.62	136.69
3	262.91	140.57
4	273.11	144.69
5	283.21	149.05
6	293.20	153.65
7	303.09	158.48
8	312.85	163.54
9	322.50	168.83
10	332.02	174.34
11	334.73	176.00

*** 1.491 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	235.00	132.13
2	242.78	124.36
3	251.61	117.79
4	261.29	112.57
5	271.62	108.80
6	282.39	106.57
7	293.37	105.91
8	304.33	106.85
9	315.04	109.37
10	325.27	113.41
11	334.82	118.88
12	343.46	125.68
13	351.04	133.65
14	357.38	142.64
15	362.36	152.45
16	365.87	162.88
17	367.83	173.70
18	367.91	176.00

*** 1.497 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	235.30	132.12

2	243.09	124.36
3	251.92	117.80
4	261.60	112.58
5	271.94	108.81
6	282.71	106.58
7	293.69	105.92
8	304.65	106.86
9	315.36	109.37
10	325.60	113.40
11	335.14	118.86
12	343.80	125.65
13	351.39	133.61
14	357.75	142.59
15	362.74	152.39
16	366.27	162.81
17	368.26	173.63
18	368.35	176.00

*** 1.498 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	235.08	132.13
2	242.89	124.39
3	251.72	117.83
4	261.38	112.57
5	271.69	108.72
6	282.43	106.36
7	293.40	105.53
8	304.38	106.25
9	315.14	108.51
10	325.48	112.26
11	335.20	117.42
12	344.09	123.90
13	351.98	131.56
14	358.72	140.26
15	364.16	149.81
16	368.21	160.04
17	370.79	170.73
18	371.32	176.30

*** 1.499 ***

Failure Surface Specified By 17 Coordinate Points

Point X-Surf Y-Surf

No.	(ft)	(ft)
1	235.08	132.13
2	242.89	124.39
3	251.77	117.89
4	261.51	112.79
5	271.90	109.18
6	282.72	107.16
7	293.71	106.76
8	304.64	108.00
9	315.26	110.85
10	325.35	115.25
11	334.67	121.09
12	343.02	128.25
13	350.21	136.57
14	356.09	145.87
15	360.53	155.93
16	363.42	166.54
17	364.54	176.00

*** 1.500 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	235.45	132.12
2	243.26	124.37
3	252.11	117.83
4	261.81	112.64
5	272.15	108.92
6	282.93	106.73
7	293.92	106.12
8	304.87	107.11
9	315.57	109.68
10	325.78	113.77
11	335.29	119.29
12	343.90	126.14
13	351.43	134.15
14	357.72	143.18
15	362.64	153.02
16	366.08	163.47
17	367.97	174.31
18	368.01	176.00

*** 1.500 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	235.38	132.12
2	243.17	124.36
3	251.97	117.76
4	261.61	112.45
5	271.89	108.55
6	282.62	106.11
7	293.58	105.19
8	304.56	105.81
9	315.35	107.96
10	325.73	111.59
11	335.51	116.64
12	344.48	123.00
13	352.48	130.55
14	359.35	139.14
15	364.95	148.61
16	369.18	158.76
17	371.95	169.41
18	372.78	176.63

*** 1.501 ***

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	235.23	132.13
2	243.07	124.41
3	251.91	117.87
4	261.57	112.61
5	271.87	108.75
6	282.61	106.35
7	293.57	105.47
8	304.55	106.11
9	315.34	108.28
10	325.72	111.92
11	335.49	116.97
12	344.47	123.33
13	352.47	130.87
14	359.36	139.45
15	364.98	148.91
16	369.24	159.05
17	372.05	169.68
18	372.88	176.65

*** 1.501 ***

** PCSTABL5 **

by
Purdue University

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

Run Date:	5/27/2005
Time of Run:	10:14am
Run By:	EMZ
Input Data Filename:	C:A3NEW
Output Filename:	C:A3NEW.OUT
Plotted Output Filename:	C:A3NEW.PLT

PROBLEM DESCRIPTION Section A-A'
BRC CAMU, Henderson, Nevada

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	138.00	80.00	136.00	2
2	80.00	136.00	164.00	134.00	2
3	164.00	134.00	240.00	132.00	2
4	240.00	132.00	313.00	166.50	2
5	313.00	166.50	333.00	176.00	1
6	333.00	176.00	370.00	176.00	1
7	370.00	176.00	410.00	185.00	1
8	410.00	185.00	436.00	185.00	1
9	436.00	185.00	450.00	185.00	1
10	313.00	166.50	450.00	170.75	2
11	.00	121.00	450.00	126.00	3
12	.00	111.00	450.00	116.00	4
13	.00	101.00	450.00	106.00	5
14	.00	91.00	450.00	96.00	6
15	.00	81.00	450.00	86.00	7

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Attachment 3B

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	117.0	117.0	.0	35.0	.00	.0	1
2	117.0	117.0	.0	35.0	.00	.0	1
3	102.0	102.0	1400.0	.0	.00	.0	1
4	102.0	102.0	1500.0	.0	.00	.0	1
5	102.0	102.0	1600.0	.0	.00	.0	1
6	102.0	102.0	1700.0	.0	.00	.0	1
7	102.0	102.0	1800.0	.0	.00	.0	1

---

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	122.00
2	450.00	127.00

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

The Active And Passive Portions Of The Sliding Surfaces Are Generated According To The Rankine Theory.

100 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 11.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	215.00	123.00	300.00	123.00	10.00
2	375.00	124.00	450.00	124.00	10.00

**** ERROR - BK12 ****
Points on active or passive wedges are outside defined ground surface

---

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

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

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	240.76	132.36
2	247.35	128.94
3	257.10	123.86
4	262.66	118.30
5	389.77	124.35
6	390.76	125.34
7	395.84	135.10
8	400.92	144.86
9	406.00	154.61
10	411.08	164.37
11	413.82	169.63
12	418.90	179.38
13	421.82	185.00

*** 1.908 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	223.66	132.43
2	230.72	128.75
3	240.48	123.67
4	244.90	119.25
5	375.27	122.34
6	378.13	125.20
7	383.21	134.96
8	388.29	144.72
9	393.37	154.47
10	398.45	164.23
11	401.05	169.23
12	406.13	178.99
13	409.16	184.81

*** 1.926 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	231.25	132.23
2	237.78	128.83
3	247.54	123.75
4	252.78	118.50
5	402.74	124.63
6	403.59	125.48
7	408.67	135.24
8	413.74	145.00
9	418.82	154.76
10	423.90	164.51
11	426.77	170.03
12	431.85	179.79
13	434.57	185.00

*** 1.939 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	233.47	132.17
2	239.85	128.85
3	249.60	123.77
4	253.12	120.26
5	396.00	124.20
6	397.21	125.41
7	402.29	135.17
8	407.36	144.93
9	412.44	154.68
10	417.52	164.44
11	420.33	169.83
12	425.41	179.59
13	428.23	185.00

*** 1.956 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	237.40	132.07
2	243.50	128.89
3	253.26	123.81
4	255.69	121.38
5	381.28	123.37
6	383.16	125.26
7	388.24	135.01
8	393.32	144.77
9	398.40	154.53
10	403.48	164.29
11	406.13	169.39
12	411.21	179.15
13	414.26	185.00

*** 1.959 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	246.87	135.25
2	249.00	134.14
3	258.76	129.06
4	268.51	123.98
5	272.04	120.46
6	386.92	119.71
7	392.57	125.36
8	397.64	135.12
9	402.72	144.88
10	407.80	154.63
11	412.88	164.39
12	415.64	169.68
13	420.72	179.44
14	423.61	185.00

*** 2.002 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	242.98	133.41
2	251.48	128.98
3	261.24	123.90
4	263.33	121.81
5	376.30	123.49
6	378.00	125.20
7	383.08	134.96
8	388.16	144.71
9	393.24	154.47
10	398.32	164.23
11	400.92	169.23
12	406.00	178.98
13	409.02	184.78

*** 2.013 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	231.76	132.22
2	238.26	128.83
3	248.01	123.76
4	250.87	120.89
5	403.98	124.60
6	404.87	125.50
7	409.95	135.26
8	415.03	145.01
9	420.11	154.77
10	425.19	164.53
11	428.08	170.07
12	433.16	179.83
13	435.85	185.00

*** 2.014 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	229.33	132.28
2	236.00	128.81
3	245.76	123.73
4	246.14	123.35
5	387.50	123.23
6	389.60	125.33
7	394.68	135.09

8	399.76	144.84
9	404.84	154.60
10	409.92	164.36
11	412.64	169.59
12	417.72	179.35
13	420.67	185.00

*** 2.028 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	215.59	132.64
2	223.22	128.67
3	232.98	123.59
4	235.35	121.22
5	381.50	122.19
6	384.58	125.27
7	389.66	135.03
8	394.73	144.79
9	399.81	154.54
10	404.89	164.30
11	407.56	169.43
12	412.64	179.19
13	415.67	185.00

*** 2.041 ***

** PCSTABL5 **

by
Purdue University

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

Run Date: 5/27/2005
Time of Run: 10:06am
Run By: EMZ
Input Data Filename: C:A2NEW
Output Filename: C:A2NEW.OUT
Plotted Output Filename: C:A2NEW.PLT

PROBLEM DESCRIPTION Section A-A'
BRC CAMU, Henderson, Nevada

BOUNDARY COORDINATES

9 Top Boundaries
15 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	138.00	80.00	136.00	2
2	80.00	136.00	164.00	134.00	2
3	164.00	134.00	240.00	132.00	2
4	240.00	132.00	313.00	166.50	2
5	313.00	166.50	333.00	176.00	1
6	333.00	176.00	370.00	176.00	1
7	370.00	176.00	410.00	185.00	1
8	410.00	185.00	436.00	185.00	1
9	436.00	185.00	450.00	185.00	1
10	313.00	166.50	450.00	170.75	2
11	.00	121.00	450.00	126.00	3
12	.00	111.00	450.00	116.00	4
13	.00	101.00	450.00	106.00	5
14	.00	91.00	450.00	96.00	6
15	.00	81.00	450.00	86.00	7

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Attachment 3C

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	117.0	117.0	.0	35.0	.00	.0	1
2	117.0	117.0	.0	35.0	.00	.0	1
3	102.0	102.0	1400.0	.0	.00	.0	1
4	102.0	102.0	1500.0	.0	.00	.0	1
5	102.0	102.0	1600.0	.0	.00	.0	1
6	102.0	102.0	1700.0	.0	.00	.0	1
7	102.0	102.0	1800.0	.0	.00	.0	1

---

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	122.00
2	450.00	127.00

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

2000 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of200 Points Equally Spaced Along The Ground Surface Between X = 200.00 ft.
and X = 250.00 ft.

Each Surface Terminates Between X = 400.00 ft.
and X = 450.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

11.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

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

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	218.59	132.56
2	226.38	124.79
3	234.86	117.79
4	243.97	111.63
5	253.62	106.35
6	263.73	102.01
7	274.20	98.64
8	284.95	96.28
9	295.87	94.94
10	306.86	94.64
11	317.84	95.38
12	328.69	97.16
13	339.33	99.95
14	349.66	103.74
15	359.58	108.49
16	369.01	114.15
17	377.86	120.68
18	386.06	128.02
19	393.53	136.10
20	400.19	144.84
21	406.01	154.18
22	410.91	164.03
23	414.86	174.30
24	417.82	184.89
25	417.84	185.00

*** 1.459 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	212.06	132.74
2	219.84	124.96
3	228.31	117.94
4	237.39	111.73
5	247.00	106.38
6	257.07	101.94

7	267.50	98.46
8	278.21	95.95
9	289.10	94.44
10	300.09	93.94
11	311.08	94.46
12	321.97	95.99
13	332.68	98.52
14	343.10	102.03
15	353.16	106.49
16	362.76	111.85
17	371.83	118.08
18	380.28	125.12
19	388.05	132.91
20	395.06	141.39
21	401.26	150.47
22	406.59	160.09
23	411.02	170.16
24	414.49	180.60
25	415.52	185.00

*** 1.463 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	220.60	132.51
2	228.41	124.76
3	236.93	117.80
4	246.08	111.69
5	255.77	106.49
6	265.92	102.25
7	276.43	99.01
8	287.21	96.80
9	298.14	95.63
10	309.14	95.52
11	320.10	96.47
12	330.92	98.48
13	341.49	101.51
14	351.72	105.55
15	361.52	110.56
16	370.78	116.49
17	379.44	123.28
18	387.39	130.87
19	394.58	139.20
20	400.94	148.18
21	406.40	157.73
22	410.91	167.76
23	414.44	178.18
24	416.03	185.00

*** 1.463 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	222.36	132.46
2	230.15	124.69
3	238.64	117.70
4	247.76	111.55
5	257.42	106.30
6	267.54	101.99
7	278.03	98.67
8	288.79	96.37
9	299.72	95.10
10	310.71	94.89
11	321.68	95.72
12	332.52	97.60
13	343.13	100.50
14	353.41	104.41
15	363.28	109.27
16	372.63	115.06
17	381.39	121.71
18	389.48	129.17
19	396.81	137.37
20	403.33	146.23
21	408.98	155.67
22	413.69	165.61
23	417.44	175.95
24	419.77	185.00

*** 1.464 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	222.11	132.47
2	229.92	124.73
3	238.43	117.75
4	247.55	111.60
5	257.21	106.34
6	267.32	102.00
7	277.79	98.63
8	288.53	96.26
9	299.44	94.90
10	310.44	94.58
11	321.42	95.29

12	332.28	97.02
13	342.93	99.76
14	353.28	103.50
15	363.23	108.18
16	372.70	113.78
17	381.60	120.24
18	389.86	127.51
19	397.39	135.52
20	404.15	144.21
21	410.06	153.48
22	415.07	163.28
23	419.14	173.49
24	422.24	184.05
25	422.42	185.00

*** 1.464 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	219.60	132.54
2	227.38	124.76
3	235.85	117.75
4	244.94	111.55
5	254.57	106.23
6	264.65	101.82
7	275.09	98.37
8	285.81	95.90
9	296.71	94.45
10	307.70	94.01
11	318.69	94.59
12	329.57	96.20
13	340.26	98.81
14	350.65	102.40
15	360.67	106.95
16	370.22	112.40
17	379.23	118.72
18	387.60	125.86
19	395.28	133.73
20	402.19	142.29
21	408.27	151.46
22	413.48	161.15
23	417.76	171.28
24	421.09	181.76
25	421.79	185.00

*** 1.464 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	217.84	132.58
2	225.69	124.87
3	234.24	117.96
4	243.42	111.90
5	253.15	106.76
6	263.32	102.59
7	273.86	99.41
8	284.65	97.27
9	295.59	96.18
10	306.59	96.15
11	317.54	97.18
12	328.34	99.27
13	338.89	102.39
14	349.09	106.52
15	358.84	111.61
16	368.05	117.62
17	376.64	124.49
18	384.53	132.16
19	391.64	140.55
20	397.90	149.59
21	403.26	159.20
22	407.67	169.28
23	411.09	179.73
24	412.26	185.00

*** 1.465 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	220.60	132.51
2	228.45	124.80
3	237.00	117.88
4	246.18	111.83
5	255.91	106.69
6	266.09	102.53
7	276.63	99.37
8	287.42	97.25
9	298.37	96.18
10	309.37	96.18
11	320.32	97.25
12	331.11	99.38
13	341.65	102.54
14	351.83	106.71
15	361.55	111.84
16	370.74	117.90

17	379.29	124.82
18	387.13	132.53
19	394.18	140.97
20	400.39	150.06
21	405.68	159.70
22	410.01	169.81
23	413.35	180.29
24	414.35	185.00

*** 1.465 ***

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	213.57	132.70
2	221.36	124.93
3	229.82	117.91
4	238.89	111.68
5	248.49	106.30
6	258.53	101.82
7	268.94	98.26
8	279.63	95.67
9	290.51	94.05
10	301.50	93.44
11	312.49	93.82
12	323.40	95.20
13	334.15	97.56
14	344.63	100.89
15	354.77	105.15
16	364.48	110.32
17	373.68	116.35
18	382.29	123.19
19	390.25	130.79
20	397.49	139.07
21	403.94	147.98
22	409.56	157.44
23	414.29	167.37
24	418.11	177.69
25	420.08	185.00

*** 1.465 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	212.31	132.73
2	220.09	124.95
3	228.58	117.96
4	237.70	111.81
5	247.37	106.56
6	257.50	102.27
7	267.99	98.97
8	278.75	96.69
9	289.68	95.45
10	300.68	95.26
11	311.65	96.14
12	322.48	98.05
13	333.08	101.00
14	343.34	104.95
15	353.18	109.87
16	362.50	115.71
17	371.22	122.41
18	379.26	129.93
19	386.54	138.17
20	393.00	147.08
21	398.57	156.56
22	403.20	166.54
23	406.86	176.91
24	408.80	184.73

*** 1.465 ***

GTC

stresses associated with failure. As the number of tests increases indefinitely, it is apparent that the envelope of the failure circles (Fig. 4.6*a*) represents the locus of points associated with failure of the specimens. The envelope is known as the *rupture line* for the given material under the specific conditions of the series of tests. For materials in general, the rupture line may be curved, and it may have an intercept c on the axis of shearing stress. Since the values of shearing strength t corresponding to the rupture line all represent failure, they are designated as values of shearing strength s, and the vertical axis in Fig. 4.6*a* is called the axis of shearing strength. If the rupture line is considered to be straight, it may be represented by

$$s = c + p \tan \phi \qquad 4.2$$

known as *Coulomb's equation*.

From the geometry of Fig. 4.6*b*, it may be seen that for any failure circle

$$2\alpha = 90^\circ + \phi$$

Therefore, the angle between the planes on which failure occurs and the plane on which the major principal stress acts is

$$\alpha = 45^\circ + \frac{\phi}{2} \qquad 4.3$$

4.6. Shearing Strength of Dry Sands and Gravels

The rupture lines for dry sands and gravels pass through the origin of the rupture diagram; hence, the intercept c is equal to zero. If the material is in a loose state, the rupture line is linear and may be represented accurately by the equation

$$s = p \tan \phi_d \qquad 4.4$$

where ϕ_d is the angle between the rupture line and the p-axis. For the same materials in a dense state, the rupture line has a slight downward curvature, but for practical purposes in foundation engineering it may also be represented by eq. 4.4.

For gravels, sands, silty sands, and inorganic cohesionless silts the value of ϕ_d depends primarily on the relative density, the grain-size distribution, and the shape of the grains. It may be estimated with the aid of Table 4.1.

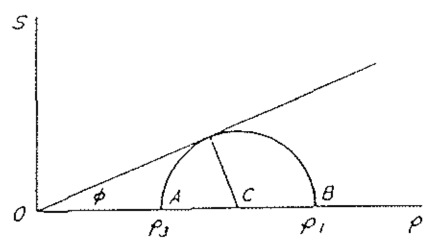
Table 4.1 Representative Values of ϕ_d for Sands and Silts

Material	Degrees Loose	Degrees Dense
Sand, round grains, uniform	27.5	34
Sand, angular grains, well graded	33	45
Sandy gravel	35	50
Silty sand	27–33	30–35
Inorganic silt	27–30	30–34

ILLUSTRATIVE PROBLEM

A drained triaxial test is to be performed on a uniform dense sand with rounded grains. The all-around pressure p_3 is to be 2 tons/sq ft. At about what vertical pressure should the sample fail?

Solution. If $s = p \tan \phi$, it can be seen from the sketch that



$$\frac{p_1}{p_3} = \frac{OC + CB}{OC - AC} = \frac{OC + OC \sin \phi}{OC - OC \sin \phi}$$
$$= \frac{1 + \sin \phi}{1 - \sin \phi}$$

Whence by trigonometric transformation

$$\frac{p_1}{p_3} = \tan^2\left(45^\circ + \frac{\phi}{2}\right) = \frac{1}{\tan^2[45^\circ - (\phi/2)]}$$

According to Table 4.1, the value of ϕ_d is likely to be about 34°. Therefore,

$$\tan^2\left(45^\circ + \frac{\phi_d}{2}\right) = \tan^2(45^\circ + 17^\circ)$$
$$= 1.881^2 = 3.54$$

Attachment-2 4

Calculation Package B
Final Waste Slope Stability

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Slope Stability Evaluation – Final Waste Slopes

Computations By:
SIGNATURE DATE 11/2/06
Meghan Lithgow, Staff Engineer
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Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE DATE 11/3/06
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Computations Checked By:
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Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Meghan Lithgow Date: 08 / 06 / 06 (MM DD YY) Reviewed by: GTC Date: 11 / 3 / 06 (MM DD YY)

Client: BRC Project: BRC CAMU Project No.: SC0313 Task No.: 01 -02

SLOPE STABILITY EVALUATION
FINAL WASTE SLOPES
BRC CAMU

OBJECTIVE

The objective of this calculation package is to evaluate the stability of the final waste soil slopes of the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada.

APPROACH OF ANALYSES

One of the important aspects in performing slope stability analyses is the selection of appropriate material parameters. With respect to landfill design, experience dictates that the weakest materials are typically the geosynthetic materials of the composite liner system. Hence, selection of material parameters for the composite liner system, specifically internal and interface shear strength properties are typically the most critical.

Ideally, lining shear strength properties should be determined from the results of laboratory testing conducted on site specific lining materials. In lieu of conducting a laboratory testing program with site specific materials, material shear strength properties may be adopted from values reported in literature or values gained from experience. However, shear strength properties are dependent on the laboratory testing conditions (e.g. confining pressures, shearing rate, degree of saturation). In addition, published material shear strength properties may be very general or explicitly product specific. Subsequently, employing published or experience-based shear strength parameters in slope stability analyses may require that restrictions be placed on the actual products and specific application conditions to ensure that the constructed slope is representative of the slope evaluated in stability analyses.

To minimize the restrictions placed on the actual products used and the specific application condition of the products in the construction of the BRC CAMU, the slope stability analyses presented herein were conducted using the following approach:

1. Determine the minimum shear strength parameters (internal or interface) of the composite liner components satisfying design criteria as indicated by results of slope stability analyses (i.e. back-analysis);

2. Demonstrate, using shear strength values reported in literature, that commercially available products exist which reportedly satisfy the minimum shear strength parameters; and
3. Based on the minimum shear strength parameters and shear strength values reported in literature, develop specific requirements for the composite liner system components which must be satisfied as demonstrated through results of laboratory testing of site specific materials to be included in the Technical Specifications.

DESIGN CRITERION

In current practice, a static factor of safety of 1.5 is generally required for final refuse slopes. To evaluate static stability at the BRC CAMU, GeoSyntec established a minimum static factor of safety of 1.5 as the stability criteria.

METHOD OF ANALYSIS

The slope stability computer program SLOPE/W (GeoStudio, 2006) was employed for this analysis. SLOPE/W employs limit equilibrium principles to provide general solutions to slope stability problems for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods, and loading conditions. Potential sliding surfaces, both circular and polygonal, can be specified or randomly generated.

CROSS SECTIONS

Two cross sections were developed (cross sections A-A' and B-B') to evaluate stability of the waste soil slopes of the BRC CAMU. The locations of these cross sections are indicated in Figure 1. The cross sections were selected and developed considering that the most likely potential failure surfaces would propagate along the composite liner system. In addition, the slopes with the largest heights were chosen to represent the most critical conditions. Cross sections A-A' and B-B' are representative of approximately a 2% liner slope behind a 3H:1V (horizontal:vertical) 47-ft high slope of cover soil.

The slope stability calculations are based on the conceptual design grading plans developed by GeoSyntec. Additional calculations may have to be performed if changes are made to the aforementioned design grading plans.

MATERIAL PARAMETERS

In order to determine the critical material parameters of the composite liner system through back-analyses, it is necessary to establish the material parameters for non-liner system components of the slope configurations. Non-liner system components involved in the evaluation of waste soil stability include waste soil and native geologic materials. The selection of material parameters for the non-liner system components used for slope stability analyses are presented herein.

Waste Soil Material

For the purposes herein, the waste soil material is assumed to be similar to soils located at the BRC CAMU site. The *in situ* properties determined by Converse (1999) characterize the BRC CAMU site soils as alluvial granular soils overlying fine-grained soils encountered at depths from approximately 34-ft to 55-ft below the surface. Based on Converse (1999) the alluvial granular soils are classified as silty sand with gravel (SM). The fine-grained soils are classified as sandy lean clay (CL). Therefore, it is assumed that the waste soil can be classified as a silty sand with gravel (SM). A sample boring log is presented in Attachment 1 to represent the typical subsurface profile.

Direct shear tests were preformed on retrieved samples by Converse (1999). The *in situ* material properties evaluated by Converse (1999) are presented in Attachment 2. Based on results by Converse (1999) the silty sand with gravel is assumed to have a friction angle of 40 degrees and cohesion of 300 psf. For this analysis, GeoSyntec assumes a friction angle equal to 31 degrees and a cohesion of 300 psf.

Converse reported a maximum dry density of 132 pcf and an optimum water content of 8.7 percent for materials at the site. Therefore, assuming 95% relative compaction, the dry density in the field is approximately 125 pcf. Adding the weight of water, the unit weight is approximately 136 pcf.

Native Material

Native material within the limits of the BRC CAMU consists of alluvial granular soils overlying fine-grained soils. Shear strength parameters for the native soil material were previously estimated and reported in the *Preliminary Geotechnical and Geologic Investigation - Industrial Non-Hazardous Disposal Facility* (Converse 1999). Twelve exploratory borings were conducted by others to depths ranging from 33-ft to 60-ft (Converse 1999). In general, the native materials appear to be consistent between borings. Direct Shear tests were performed on selected undisturbed samples retrieved from the exploratory borings. A summary of the Direct Shear test results as reported by Converse (1999) is presented in Attachment 2.

GeoSyntec performed stability calculations for the proposed fill slopes at the BRC CAMU. Fill slope stability calculations are presented in Appendix X, "*Slope Stability Evaluation - Fill Slopes*".

RESULTS OF ANALYSES

Stability analyses were conducted on cross sections A-A' and B-B' using the computer program SLOPE/W. Because the composite liner system of solid waste landfills introduces the weakest materials within a typical cross section, polygonal or wedge shaped potential failure surfaces propagating along the composite liner system were investigated. Potential wedge failure surfaces were evaluated using the solver function of the SLOPE/W program to search for the lowest factor of safety using both force and moment analyses. The locations of potential failure surface searches were varied throughout the cross section by the user to focus on the location of the most critical potential failure surface (defined as the potential failure surface yielding the lowest factor of safety) for the given cross section.

A composite liner system represented by an apparent friction angle of 12 degrees and no geosynthetic adhesion yields a factor of safety of 1.7 for the most critical potential failure surface of cross section A-A' and 1.6 for cross section B-B' (Attachment 3).

Written by: Meghan Lithgow Date: 08 / 06 / 06 (MM DD YY) Reviewed by: GC Date: 11 / 3 /06 (MM DD YY)

Client: BRC Project: BRC CAMU Project No.: SC0313 Task No.: 01 -02

REVIEW OF MATERIAL SHEAR STRENGTHS REPORTED IN LITERATURE

Results of the stability analyses indicate that the minimum allowable apparent internal or interface friction angle for components of the composite liner system is 12 degrees. The objective herein is to identify the potential internal and interface sliding surfaces of the composite liner system proposed for the BRC CAMU and to estimate potential values of internal and interface friction angles based on values reported in literature. The proposed base and side slope composite liner system is comprised of, from top to bottom:

- 2 ft. thick operations layer;
- Drainage geocomposite with an 8 oz/sy geotextile bonded to both sides of the geonet;
- 60 mil HDPE geomembrane liner (textured top and bottom);
- Geosynthetic clay liner (GCL); and
- Prepared subgrade or waste soil.

The estimated internal and/or interface shear strengths for each of the potential composite liner system materials as reported in literature and the corresponding reference are presented below. Values listed below, unless otherwise noted, represent post-peak or residual friction angles.

The type of GCL considered is a geotextile-backed GCL. The geotextile-backed GCL consists of a layer of bentonite between two geotextiles.

Written by: Meghan Lithgow Date: 08 / 06 / 06 (MM DD YY) Reviewed by: GTC Date: 11 / 3 / 06 (MM DD YY)

Client: BRC Project: BRC CAMU Project No.: SC0313 Task No.: 01 -02

The following presents a summary of published interface friction values:

Material/Interface	c', Friction Angle[1]	Reference
-Operations layer	0, 32 degrees	NAVFAC (1982) (Att. 4)[2,3]
-Operations layer/GT	0, 29 degrees	Koerner (1995) (Att. 5)[4]
-GT/leachate collection aggregate	0, 34 degrees	Koerner (1995) (Att. 5)[4,5]
-Leachate collection aggregate	0, 37 degrees	NAVFAC (1982) (Att. 4)[2,5]
-Geotextile/textured GM	0, 19 degrees	Li and Gilbert (1999) (Att. 7)[6]
-Textured GM/GCL	0, 12 degrees	Bentomat (Att. 6)[8]
-GCL (internal)	0, 16 degrees	Bentomat (Att. 6)[7]
-GCL (GT backed)/subgrade	0, 31 degrees	Bentomat (Att. 6)[8]
-Prepared Subgrade	0, 35 degrees	Converse (1999) (Att. 2)

where:
GT = geotextile
GM = geomembrane
GCL = geosynthetic clay liner

Notes:

1. For the sake of comparison herein, the adhesion of the geosynthetic interfaces (geosynthetic cohesion) is neglected. All values are presented in terms of effective stress strength parameters.

2. NAVFAC (1982) lists typical shear strength values for various soils based on 100 percent standard Proctor compaction. Actual construction materials would likely be placed at 90 percent of the modified Proctor compaction which for the sake of the comparison presented herein roughly corresponds to 100 percent standard Proctor compaction.

3. Value of friction angle for a silty sand designated under the USCS classification system as a SM.

4. Koerner (1995) suggests that an efficiency of greater than 90 percent for the interface of nonwoven, needle-punched geotextiles to various soils can be achieved. Efficiency values are based on the relationship, Efficiency = tan(interface friction angle)/tan(soil friction angle). The interface friction angle presented herein was calculated using a 90 percent efficiency and the estimated soil friction angle. Adhesion is neglected.

5. Value of friction angle for a well graded or poorly graded gravel designated under the USCS classification system as a GW or GP.

6. Li and Gilbert (1999) reports an average secant friction angle of a Gundline HDT 60-mil HDPE geomembrane/Trivira 1145 interface of 19 degrees under a confining pressure of 50 psi at a large-displacement (Attachment 8).

7. According to the Summary of Bentomat Direct Shear Test Data (Attachment 6) a friction angle of 24 degrees is represents typical reported peak displacement internal shear strength values for the GCLs. Note that this represents typical results of hydrated test conditions within the range of normal stresses from 14 to 142 psi. The peak displacement internal shear strength equals approximately 16 degrees when reduced by a factor of safety of 1.5. This reduction in the internal shear strength is conservative considered in the absence of site specific test data.

8. Reported data from the Summary of Bentomat Direct Shear Test Data (Attachment 6). Note that this represents typical results of hydrated test conditions within the range of normal stresses from 7.5 to 30 psi for the respective interface.

CONCLUSIONS

Results of stability analyses presented herein indicate that an apparent internal or interface friction angle (residual) of 12 degrees for any component of the composite liner system is the minimum allowable value providing for a static factor of safety that satisfies the design criteria of 1.5.

A review of potential internal and interface shear strengths reported in literature for materials representative of the components of the proposed composite liner system at the BRC CAMU was conducted. Based on the shear strength values reported in literature, proposed composite liner interfaces exhibit apparent internal and interface friction angles greater than the minimum allowable value of 12 degrees evaluated herein.

Furthermore, it is crucial that interface shear tests be conducted on the actual materials proposed for use in the composite liner system. Based on the analyses presented herein, results of interface shear tests on the actual materials proposed for use in the composite liner system must indicate that the weakest **apparent residual friction angle** of the composite liner system is equal to or greater than 12 degrees. Note that the **apparent friction angle** differs from the friction angle determined from the failure envelope developed directly from results of direct shear tests. The **apparent friction angle** is determined from the failure envelope of the data points from direct shear test results which have been linearly regressed through the origin of the normal stress/shear stress plot.

The results of interface shear testing are directly dependent on the test conditions including normal stress levels, rate of shear, degree of saturation, and amount of displacement. It is imperative that these test conditions are representative of the anticipated conditions at the BRC CAMU when testing actual materials proposed for use in the composite liner system.

LIST OF FIGURES

LIST OF ATTACHMENTS

REFERENCES

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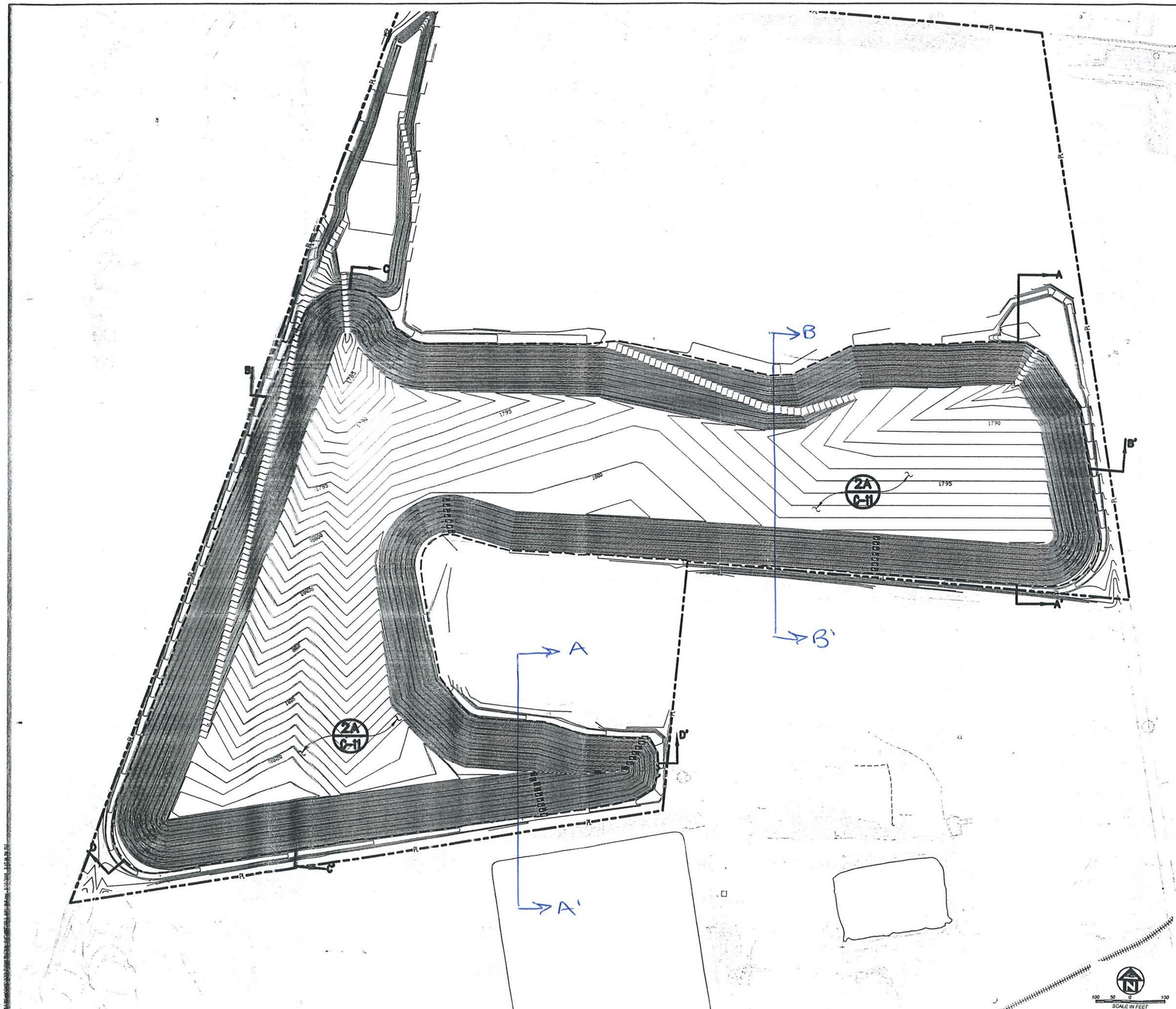
G-TC
11/7/06



LEGEND

- BRC LANDFILL SITE BOUNDARY
- EXISTING LIMITS OF FORMER BMI LANDFILL
- EXISTING MINOR ELEVATION CONTOUR
- EXISTING MAJOR ELEVATION CONTOUR
- PROPOSED MINOR ELEVATION CONTOUR
- PROPOSED MAJOR ELEVATION CONTOUR
- PROPOSED LIMIT OF WASTE

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG C-2.
2. FOR SITE PREPARATION PLAN SEE DWG C-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG C-7.
4. FOR EROSION CONTROL AND STORM DRAIN PLAN SEE DWG C-8.
5. FOR CROSSECTIONS SEE DWG C-10.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG C-11.
7. FOR FENCE PLAN SEE DWG C-12.

FIGURE 1

GEOSYNTEC CONSULTANTS
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PROJECT:
BRC CAMU
HENDERSON/CLARK COUNTY NEVADA

TITLE:
FINAL COVER GRADING PLAN
PERMITTING CONSTRUCTION PLAN

DATE: OCTOBER 2006	CHECKED BY: G.T.C.	SCALE: 1"=100'	FIGURE NO:
DESIGN BY: TAW	REVIEWED BY: (PEER REV.) G.T.C.	JOB NO.: SC0313-01-04	89
DRAWN BY: T.L.Z.	DOCUMENT NO:	FILE NO:	

𝒜𝒱 GTC MLD

Direct Shear Strength

A progressive direct shear test was performed on selected undisturbed samples using a constant strain rate direct shear machine in general accordance with ASTM D3080. The test specimen was trimmed and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until maximum shear strength was developed. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. Another normal load was then applied, and the specimen was sheared a second time. This process was repeated for three different normal loads. Results of the direct shear test are presented on Figures A-62 through A-69 and in the following table:

Exploration Location	Depth (feet)	Soil Description	Angle of Internal Friction (deg)	Coulomb Cohesion (ksf)
B-4	14-14.5	Silty sand with gravel	31	0.7
B-5	14-15	Silty sand with gravel	43	0.3
B-10	54-54.5	Sandy lean clay	26	0.85
B-12	14-15	Silty sand with gravel	40	0.3
B-101	39-40	Sandy lean clay	26	0.9
B-102	20-25	Silty sand with gravel	37	0.2
B-103	49-50	Sandy lean clay	37	1.0
B-104	10-15	Silty sand with gravel	43	0.1

Chemical Analysis

Chemical tests were performed on a representative soil samples to investigate the potential for soil corrosivity and chemical heave. Atlas Chemical Testing Laboratories, Inc. in Las Vegas performed the chemical analysis for water-soluble sulfates and sodium in general accordance with ASTM D516. The results of the chemical tests are presented on Drawing No. A-70.

AH. 1
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GTC MLD

Grain Size Distribution

Grain size distribution for soil samples were determined by sieve analysis in accordance with ASTM C136. A sieve analysis is conducted by passing the soil through a number of different sized sieves and measuring the amount of soils retained on each sieve. The test results and grain size distribution curves are presented on Drawing Nos. A-37 through A-48.

Atterberg Limits

The liquid limit, plastic limit and plasticity index of a representative sample of the fine-grained soils were determined to aid in the classification of the soils and in the evaluation of other engineering parameters. The test was performed in general accordance with ASTM test method D4318. The results of the tests are tabulated in the following table:

Exploration Location	Sample Depth, ft.	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Unified Soils Classification
B-1	30-35	NP	NP	NP	SM
B-5	20-25	NP	NP	NP	SM
B-10	30-35	NP	NP	NP	SM
B-12	10-15	NP	NP	NP	SM
B-101	39-40	105	71	34	MH
B-101	54-55	54	44	10	ML
B-102	20-25	NP	NP	NP	SM
B-102	49-50	88	58	30	MH
B-103	30-35	NP	NP	NP	SM
B-104	10-15	NP	NP	NP	SM
B-105	20-25	NP	NP	NP	SW-SM
B-106	0-5	NP	NP	NP	SM

NP = Nonplastic

Attachment 1

GTC

Direct Shear Strength

A progressive direct shear test was performed on selected undisturbed samples using a constant strain rate direct shear machine in general accordance with ASTM D3080. The test specimen was trimmed and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until maximum shear strength was developed. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. Another normal load was then applied, and the specimen was sheared a second time. This process was repeated for three different normal loads. Results of the direct shear test are presented on Figures A-62 through A-69 and in the following table:

Exploration Location	Depth (Feet)	Soil Description	Angle of Internal Friction (deg)	Coulomb Cohesion (ksf)
B-4	14-14.5	Silty sand with gravel	31	0.7
B-5	14-15	Silty sand with gravel	43	0.3
B-10	54-54.5	Sandy lean clay	26	0.85
B-12	14-15	Silty sand with gravel	40	0.3
B-101	39-40	Sandy lean clay	26	0.9
B-102	20-25	Silty sand with gravel	37	0.2
B-103	49-50	Sandy lean clay	37	1.0
B-104	10-15	Silty sand with gravel	43	0.1

Chemical Analysis

Chemical tests were performed on a representative soil samples to investigate the potential for soil corrosivity and chemical heave. Atlas Chemical Testing Laboratories, Inc. in Las Vegas performed the chemical analysis for water-soluble sulfates and sodium in general accordance with ASTM D516. The results of the chemical tests are presented on Drawing No. A-70.

Attachment 2

1/1

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Log No. B- 8

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 58.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SANDY LEAN CLAY With Gravel (CL); very stiff, dark brown, slightly moist			49				
42									
44		—dry			35/8"				K
46									
48		---partially cemented, moderately hard, white, dry							
		SANDY LEAN CLAY (CL); very stiff, dark brown, moist			35/10"				pp=3.0
50									
52									
54					16		52	71 108	pp=2.0
56									
58		SILTY SAND (SM); dense, brown, wet							
		SANDY LEAN CLAY (CL); stiff, dark brown, wet			13		51	70 106	pp=3.0

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

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MB GTC MLD

Log No. B-12

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 37.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY OLE

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SANDY LEAN CLAY (CL); very stiff, dark brown, moist			44				
42									
44									
		—wet, stiff			15		80	53	95 pp=1.5
46									
48									
					11				
50									
52									
		---with gravel, very stiff							
54					16		31	87	114 pp=1.5
56									
58									
					18				pp=1.5

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

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gg GTC MLD

Log No. B-101

Date of Drilling: 9/20/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
40			X		10				
		SILTY SAND With Gravel (SM); medium dense, grayish brown, dry							
42	▼								
		SANDY LEAN CLAY (CL); stiff, dark brown, moist							
44			X		11		45	67	97
46									
48									
			X		25				pp=1.5
50		---partially cemented lens, moderately hard, brownish white, wet							
52									
54			X		11		42	75	107 A
		---with gravel							
56									
58									
			X		14		42	74	106

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
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Clark County, Nevada

Project No.
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Log No. B- 1

Date of Drilling: 9/23/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 53.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
	SILTY SAND With Gravel (SM); dense, light brown, slightly moist	X	X	35				
22								G;P
24		X		35		7	108	116
26								
28	---grayish brown							
		X		35/6"				
30	---very dense	X	X	53				A
								G
32								
34		X		35/3"				
36								
38								
	---dense	X		35/6"				

End of Exploration at 55.0'

Converse Sampler (white symbol=no recovery)
SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

3343701.GPJ

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Log No. B- 4

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 54.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); very dense, brown, dry	X		35/2"				
22									
24		---dense	X		49				K
26									
28									
		---few gravel	X		35		7	107	114
30			X		42				
32									
34		---with gravel, very dense	X		35/8"		4		
36									
		---with boulders							
38									
		CEMENTED SAND AND GRAVEL; hard, brown, dry			20/0"				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

1701 GPJ

98 GTC MLD

Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		**FILL:** SILTY SAND With Gravel; tan, dry	X	X	9				
4.2		---slightly moist	X		35/5"				
5			X		58				
8.2		---with cobbles							
9		SILTY SAND With Gravel (SM); dense, tan, slightly moist	X		31		5	115	121
10			X	X	29				
12									K;Sol; Ch
14.5			X		43		4	111 115	DS
19.5		---very dense	X		57/8"				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

701 GPJ

MS GTC MLD

Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.							
		SILTY SAND With Gravel (SM); dense, tan, slightly moist			32				A
22									G;P;Ch
24					58/8"				
		---very dense							
26									
28		---brown							
		CLAYEY SAND With Gravel (SC); dense, dark brown, slightly moist			35/5"		6	112	119
30					35/6"				
		SILTY SAND With Gravel (SM); dense, black, slightly moist							
32		---dark brown							
34		---very dense			50/3"				
36									
38									
		---with cobbles			50/6"				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

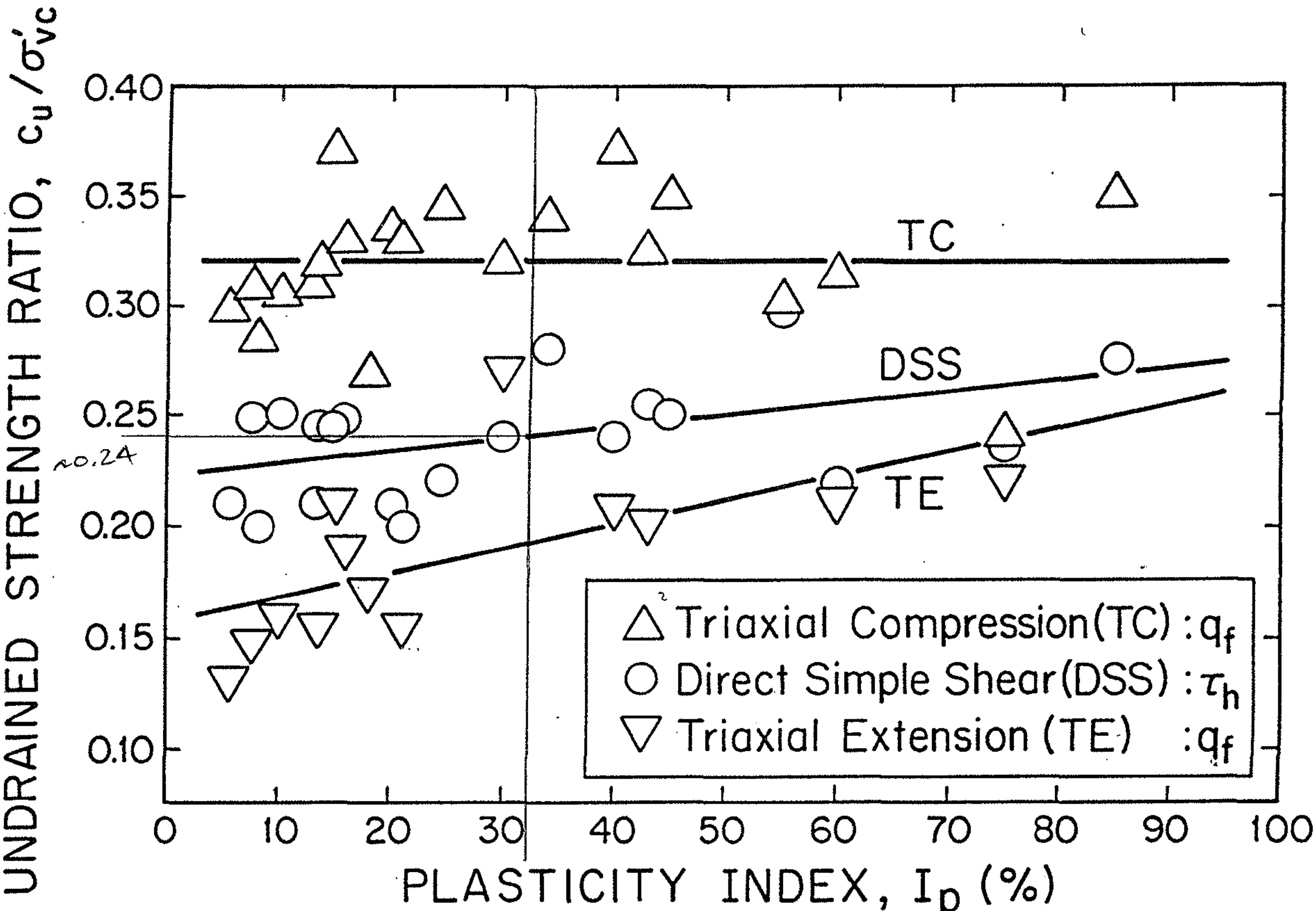
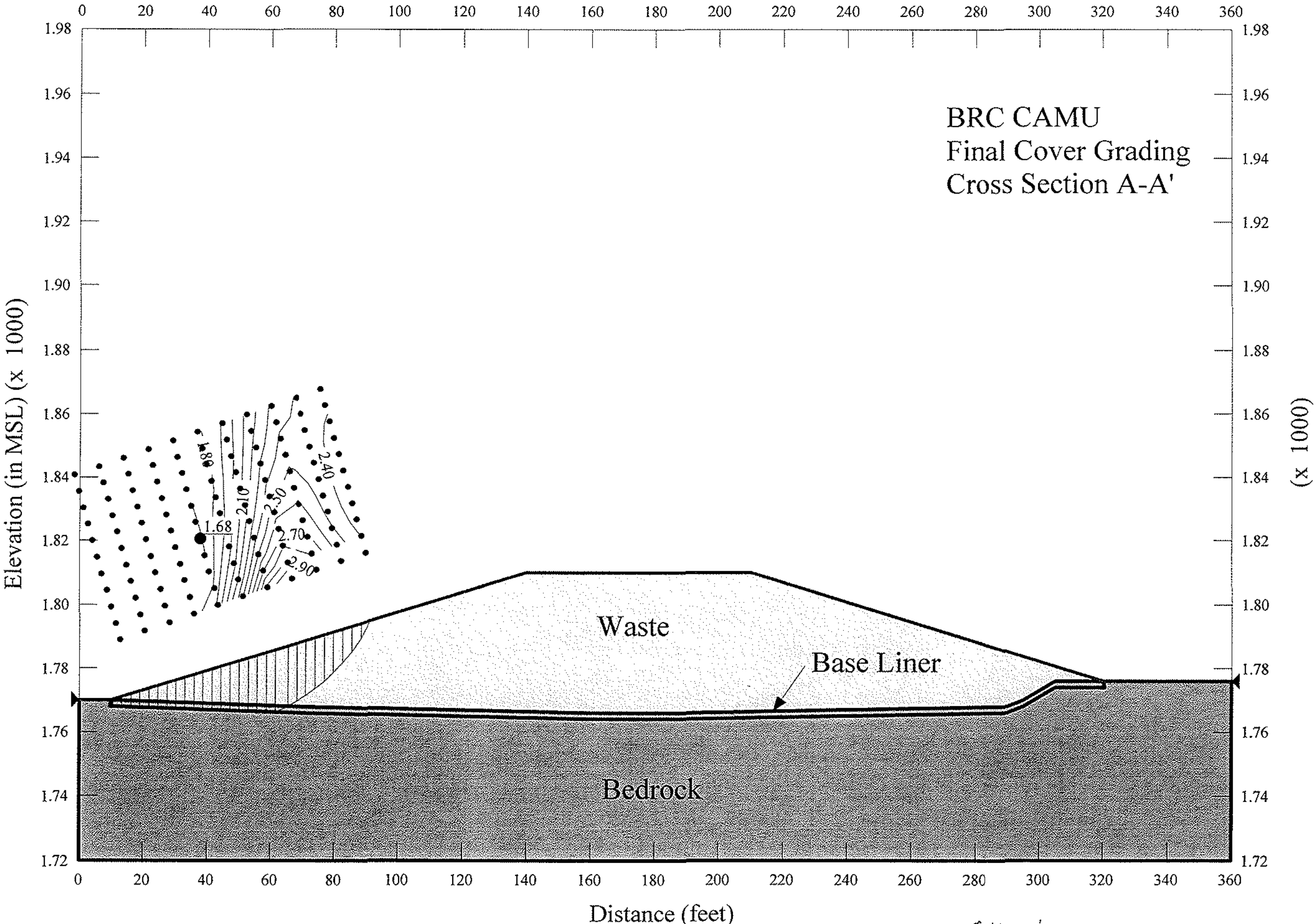


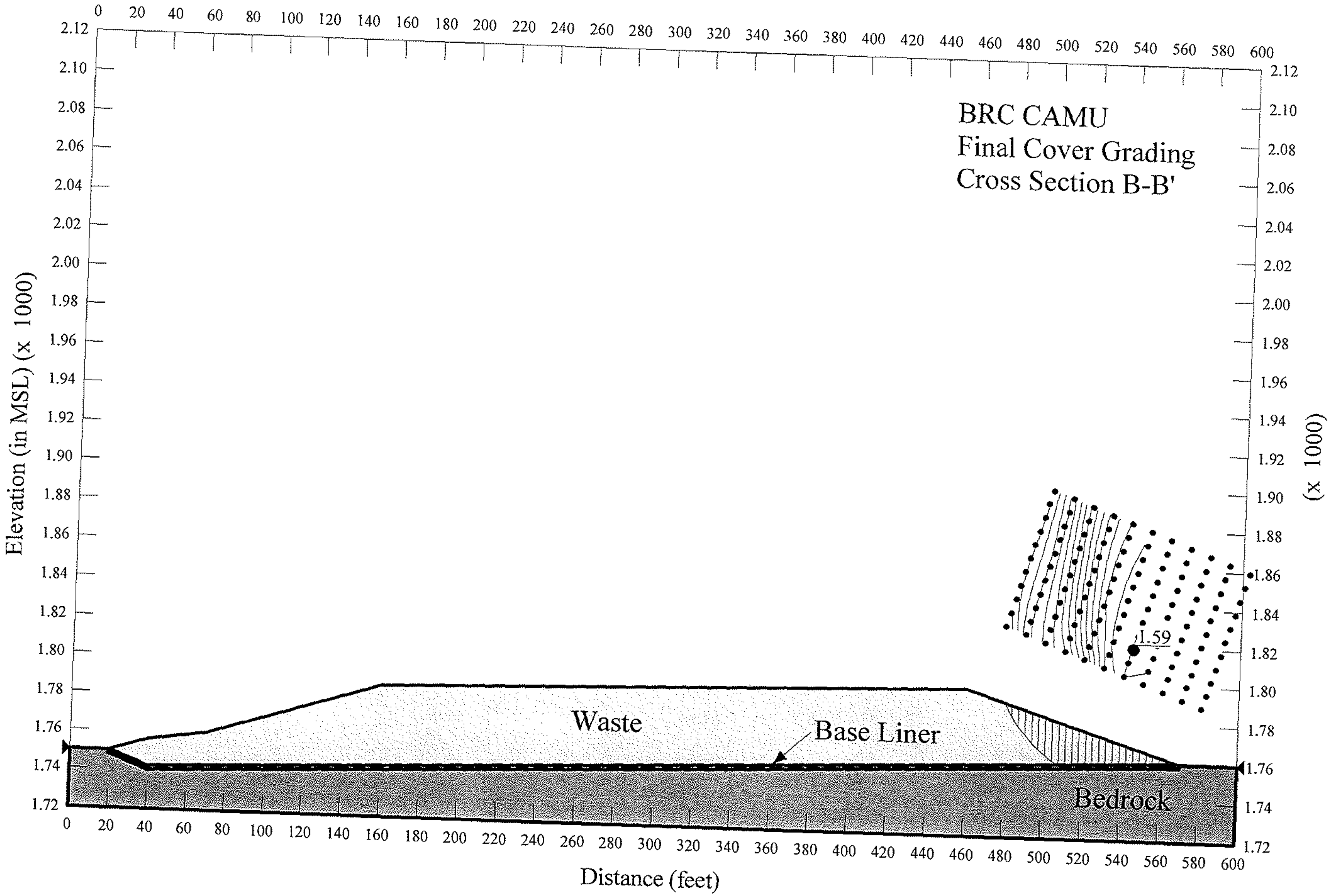
FIG. 15.-Undrained Strength Anisotropy from CK_oU Tests on Normally Consolidated Clays and Silts (data from Lefebvre et al. 1983; Vaid Campanella 1974; and various MIT and NGI reports)

Attachment 2

1/1



Attachment 3 1/2



Attachment 3

2/2

GTC

stresses associated with failure. As the number of tests increases indefinitely, it is apparent that the envelope of the failure circles (Fig. 4.6*a*) represents the locus of points associated with failure of the specimens. The envelope is known as the *rupture line* for the given material under the specific conditions of the series of tests. For materials in general, the rupture line may be curved, and it may have an intercept c on the axis of shearing stress. Since the values of shearing strength t corresponding to the rupture line all represent failure, they are designated as values of shearing strength s, and the vertical axis in Fig. 4.6*a* is called the axis of shearing strength. If the rupture line is considered to be straight, it may be represented by

$$s = c + p \tan \phi \qquad 4.2$$

known as *Coulomb's equation.*

From the geometry of Fig. 4.6*b*, it may be seen that for any failure circle

$$2\alpha = 90° + \phi$$

Therefore, the angle between the planes on which failure occurs and the plane on which the major principal stress acts is

$$\alpha = 45° + \frac{\phi}{2} \qquad 4.3$$

4.6. Shearing Strength of Dry Sands and Gravels

The rupture lines for dry sands and gravels pass through the origin of the rupture diagram; hence, the intercept c is equal to zero. If the material is in a loose state, the rupture line is linear and may be represented accurately by the equation

$$s = p \tan \phi_d \qquad 4.4$$

where ϕ_d is the angle between the rupture line and the p-axis. For the same materials in a dense state, the rupture line has a slight downward curvature, but for practical purposes in foundation engineering it may also be represented by eq. 4.4.

For gravels, sands, silty sands, and inorganic cohesionless silts the value of ϕ_d depends primarily on the relative density, the grain-size distribution, and the shape of the grains. It may be estimated with the aid of Table 4.1.

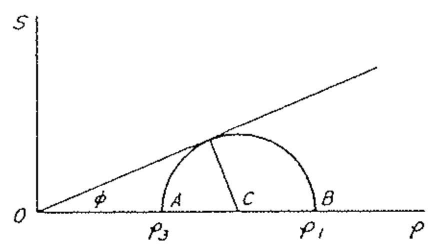
Table 4.1 Representative Values of ϕ_d for Sands and Silts

Material	Degrees Loose	Degrees Dense
Sand, round grains, uniform	27.5	34
Sand, angular grains, well graded	33	45
Sandy gravel	35	50
Silty sand	27–33	30–35
Inorganic silt	27–30	30–34

ILLUSTRATIVE PROBLEM

A drained triaxial test is to be performed on a uniform dense sand with rounded grains. The all-around pressure p_3 is to be 2 tons/sq ft. At about what vertical pressure should the sample fail?

Solution. If $s = p \tan \phi$, it can be seen from the sketch that



$$\frac{p_1}{p_3} = \frac{OC + CB}{OC - AC} = \frac{OC + OC \sin \phi}{OC - OC \sin \phi} = \frac{1 + \sin \phi}{1 - \sin \phi}$$

Whence by trigonometric transformation

$$\frac{p_1}{p_3} = \tan^2\left(45° + \frac{\phi}{2}\right) = \frac{1}{\tan^2 [45° - (\phi/2)]}$$

According to Table 4.1, the value of ϕ_d is likely to be about 34°. Therefore,

$$\tan^2\left(45° + \frac{\phi_d}{2}\right) = \tan^2 (45° + 17°) = 1.881^2 = 3.54$$

Attachment-~~2~~. 4

Calculation Package C
Drainage Pipe Size Requirement

GeoSyntec Consultants

GeoSyntec Consultants
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Drainage Pipe Size Requirements

Computations By:
SIGNATURE DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE DATE 11/3/06
Greg Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE DATE 11/3/06
Greg Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE DATE 11/3/06
Greg Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebecca Flynn Date: 06 / 08 / 29 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

DRAINAGE PIPE SIZE REQUIREMENTS

OBJECTIVE

The objective of this calculation package is to evaluate the drainage pipe size requirements for the base liner proposed for the drainage system for the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada. It is proposed that the drainage system include an geocomposite drainage layer with a single drainage pipe, bedded in a drainage aggregate trench, located along the base of each cell. The pipe diameter and perforations must be able to handle the maximum flow into and through the pipe. This calculation will evaluate the required performance diameter and perforation size and spacing for the drainage pipe. In addition, the drainage aggregate must be sized as to not allow transport of materials through the pipe perforations.

SUMMARY OF ANALYSIS

The calculations suggest that a 4-inch diameter pipe with four ¼-inch perforations spaced at 1 foot on center will accommodate the maximum flow predicted by the HELP Model. The maximum particle size of the drainage aggregate is 1 in or less (as evaluated in the geomembrane puncture protection calculation) and D_{85} must be 0.5 inch or higher to prevent piping and material loss. AASHTO 67 material meets the drainage aggregate requirements.

SITE CONDITIONS

The proposed composite liner system will be comprised of the following components, from top to bottom (Figure 2):

- 2 ft of operations layer material;
- a drainage geocomposite;
- 60-mil (1.5 mm) HDPE geomembrane, textured on both sides;
- a geosynthetic clay liner (GCL); and
- prepared subgrade.

ANALYSIS

HELP Analyses:

The Hydrologic Evaluation of Landfill Performance (HELP) model was used to estimate the peak daily quantity of liquid expected to be generated in the drainage pipe during or after a rainfall event. The HELP model is used to evaluate the worst case scenario which occurs *during the period when the cell contains a small quantity of waste*, such that any collected liquid is considered leachate, yet the majority of the cell is empty so the largest quantity of liquid will infiltrate through the operations layer to the drainage layer. This time period exists between the following activities:

- *Immediately after the composite liner system construction has been completed* (any collected liquid during construction is considered construction water and will not be contaminated);

- *Before the period of significant waste placement.* Leachate generation is reduced by the dry landfill moisture retention capacity. Separate calculation packages address the moisture retention capacity of the dry landfill soil and respective dry landfill requirements so that leachate generation is greatly reduced or completely stored in the waste soil.

No further analyses are needed because the above condition represents the worst case during the life of the landfill.

The rainfall history was synthetically generated by HELP over a 20 year period. The drainage pipes, pipe perforations, and portions of the drainage geocomposite layer (maintaining less than 1 ft of head over the liner) must accommodate the peak daily quantity over a 20 year rainfall history. This is conservative because the 20-year peak daily quantity is assumed to occur during a relatively short exposure period.

The initial water content of the placed soil is assumed to be the default water content initialized by HELP. This is conservative because the water content of the drainage aggregate and operations layer soil is likely to be relatively low due to high temperatures and low humidity at the site, creating a high pan evaporation rate. Therefore, the initial water content evaluated by HELP (based on a water content near steady state) will likely be higher than the actual site conditions.

Written by: Rebecca Flynn Date: 06 / 08 / 29 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

Model Parameters

Vertical Percolation Layer: The vertical percolation layer (type 1) is represented by default properties for a silty sand in HELP (texture number 3). However, the hydraulic conductivity is changed to represent worst case (high hydraulic conductivity) as estimated by Converse Consultants (Attachment G).

Geocomposite: The default material properties for the drainage geocomposite in the HELP model are used to represent the lateral drainage layer (type 2). The defalt properties for the geocomposite are represented by HELP texture number 20. The hydraulic conductivity of the goecomposite is assumed to be 10 cm/sec (required by 40 CFR 264.301 (c) (3) (ii)).

Geosynthetics: The geomembrane and GCL properties are estimated from the HELP model. The GCL is represented by texture number 17 and the geomembrane is represented by texture number 35.

Runoff: The SCS runoff curve number was evaluated assuming HELP texture number 3 at the surface (silty sand).

The default values in the HELP model are shown in the HELP output presented in Attachment A.

Figure 1 presents the locations of drainage pipes and HELP Model analyses listen in the following Table. Figure 2 shows the cross section of the base liner system.

HELP Results

The following results were obtained from the HELP analysis:

Location	Peak Daily Quantity	Area	Slope and Length[1]	File
Unit 1	306 ft^3	3.34 acres	2% and 200 ft	SOIL1SUM
Unit 2	568 ft^3	5.98 acres	2% and 400 ft	SOIL2SUM
Unit 3	1062 ft^3	10.9 acres	2% and 1050 ft	SOIL3SUM
Unit 4	1169 ft^3	11.9 acres	3% and 550 ft	SOIL4SUM
Unit 5	1313 ft^3	13.3 acres	3% and 650 ft	SOIL5SUM
Unit A	9833 ft^3	2.84 acres	48% and 100 ft	SOILASUM
Unit B	10471 ft^3	3.15 acres	48% and 75 ft	SOILBSUM

1) see Figure 1 for slope and length evaluation.

Written by: Rebecca Flynn Date: 06 / 08 / 29 (YY MM DD) Reviewed by: ETC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

Pipe Diameter Analysis

From the HELP Model analyses performed for each of the 5 units and 2 side slope conditions at this facility (Attachment A), the maximum peak daily quantity of liquid expected to be generated is 10457 cubic feet on Slope B (see Figure 1). Two pipes will be used to convey the drainage from Slopes B to the sump. The following flow rate for each pipe is calculated:

(10457 CF/day) / (24 hours/day) / (60 min/hour) / (60 sec/min)/2 = 0.0605 cfs

The following equation (Attachment B) can be used to estimate the flow rate in the pipe when flowing full:

$Q_{½\ full} = (1.486/n)(A)(R)^{2/3}(S)^{1/2}$

where;

n = the Mannings roughness coefficient, 0.009 for plastic pipe (Attachment B)

A = the area of the pipe

R = hydraulic radius = area/perimeter of the pipe (Attachment B)

S = minimum slope of pipe = 0.5 %

Assuming a 4" diameter pipe will be sufficient, and using the standard dimension ratio (SDR) of 13.5 determined in the pipe crushing calculation, the following values will be used in the above equation:

r = 1.92 in (SDR 13.5 HDPE pipe) (Attachment ~~A~~ C)

$A = \pi r^2 = \pi (1.92/12)^2 = 0.080 \text{ ft}^2$

$R = (0.080) / (2 \pi (1.92/12)) = 0.080 \text{ ft}$

Placing the above values into the Mannings equation results in the following:

$Q_{½\ full} = (1.486/0.009)(0.080)(0.080)^{2/3}(.005)^{1/2}$

$Q_{½\ full}$ = 0.173 cfs, which is larger than the required flow rate of 0.061 cfs, therefore 4" diameter pipe will be sufficient for this application.

Perforations Analysis

A minimum of four penetrations, each 45 and 90 degrees from the bottom of the pipe, will be used to limit the amount of particulates entering the drainage pipe directly. Verify that 4 rows of ¼-inch holes spaced at 12 inches on center are adequate.

The holes will be modeled as submerged orifices using the following equation:

$Q = C_d A (2gh)^{1/2}$ (Attachment D)

where:

C_d = coefficient of discharge = 0.62 (Attachment D)

A = area of hole = $\pi (0.125/12)^2 = 0.0003$ ft^2

h = head in feet = 1 ft based on maximum head allowed over the liner system

g = gravity = 32.2 ft/sec^2

$Q = (0.62)(0.0003)(2 \times 32.2 \times 1)^{1/2} = 0.0015$ cfs per hole

Q = 0.0015 cfs x 4 holes/foot of pipe = 0.006 cfs per foot of pipe

The conservative maximum discharge to the pipe is approximately 0.061 cfs. Since there is more than 100 ft of drainage pipe for each unit (6.1 cfs/100 ft of pipe) the perforation size and spacing is more than adequate for the predicted flow rates to the pipe.

Required Drainage Aggregate

The size of the perforations are 1/4 inch in diameter. The gradation of the drainage aggregate must be designed to insure that piping and material loss will not occur. The U.S. Bureau of Reclamation (1973) use the following criteria for gradation of filter materials in relation to holes:

(D_{85} of the filter material) / (hole diameter) > 2.0 (Attachment E)

Therefore, D_{85} of the filter material must be greater than 1/2 inch.

AASHTO 67 material has a maximum particle size of 1 inch, which is the criteria for puncture protection of the geomembrane. D_{85} of AASHTO 67 material ranges from approximately 0.5 to 0.7 inches, which satisfies the above criteria (see Attachment F).

CONCLUSIONS

In accordance with the above analysis, the drainage pipe shall be 4-inch diameter pipe with four ¼-inch perforations per foot of pipe. The drainage aggregate shall have a hydraulic conductivity greater than 1 x 10^{-2} cm/sec, a maximum particle diameter of 1 inch, and D_{85} greater than 1/2 inch. AASHTO 67 material satisfies this criteria.

REFERENCES

Written by: Rebecca Flynn Date: 06 / 08 / 29 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

Cedergren, H. R., (1989), "Seepage, Drainage, and Flow Nets," Third Edition, John Wiley and Sons, New York. p. 161

Chevron (1999), Plexco/Spirolite Engineering Manual

Converse (1999), "Preliminary Geotechnical and Geologic Investigation - Industrial Non-Hazardous Disposal Facility", prepared for Basic Management, Inc., October 1999.

Hwang, (1987), "Fundamentals of Hydraulic Engineering Systems," 2nd Edition, Prentice-Hall

Phillips 66, Driscopipe Sizes and Dimensions (1999)

U.S. Bureau of Reclamation (1973), "Design of Small Dams, U.S. Government Printing Office, Washington D.C., 2nd edition.

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                 DEVELOPED BY ENVIRONMENTAL LABORATORY                    **
**                   USAE WATERWAYS EXPERIMENT STATION                      **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:        C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:    C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:      C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:    C:\HLP3\BRC\BRCSOIL1.D10
OUTPUT DATA FILE:             C:\HLP3\BRC\Summary\SOIL1SUM.OUT


TIME:   8: 3      DATE:   8/31/2006


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     TITLE:  BRCUnit1

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER   3
          THICKNESS                    =      24.00   INCHES
          POROSITY                     =       0.4570 VOL/VOL
          FIELD CAPACITY               =       0.0830 VOL/VOL
          WILTING POINT                =       0.0330 VOL/VOL
          INITIAL SOIL WATER CONTENT   =       0.1055 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.310000009000E-02 CM/SEC


                                   LAYER  2
                                   --------

                        TYPE 2 - LATERAL DRAINAGE LAYER
```

MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0110 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC
SLOPE	=	2.50 PERCENT
DRAINAGE LENGTH	=	200.0 FEET

LAYER 3

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	0.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	0.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 4

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.20 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 200. FEET.

SCS RUNOFF CURVE NUMBER	=	81.00
FRACTION OF AREA ALLOWING RUNOFF	=	100.0 PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000 ACRES
EVAPORATIVE ZONE DEPTH	=	18.0 INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.704 INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.226 INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.594 INCHES
INITIAL SNOW WATER	=	0.000 INCHES

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INITIAL WATER IN LAYER MATERIALS     =       2.684  INCHES
TOTAL INITIAL WATER                  =       2.684  INCHES
TOTAL SUBSURFACE INFLOW              =       0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

```
STATION LATITUDE                          = 36.08 DEGREES
MAXIMUM LEAF AREA INDEX                   =  0.00
START OF GROWING SEASON (JULIAN DATE)     =    62
END OF GROWING SEASON (JULIAN DATE)       =   321
EVAPORATIVE ZONE DEPTH                    = 18.0  INCHES
AVERAGE ANNUAL WIND SPEED                 =  9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY     = 39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY     = 21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY     = 24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY     = 36.00 %
```

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.005	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.024	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.274	0.206	0.271	0.277	0.225	0.132
	0.364	0.338	0.298	0.156	0.379	0.248
STD. DEVIATIONS	0.306	0.171	0.285	0.258	0.200	0.109
	0.560	0.369	0.350	0.135	0.451	0.242
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.0579	0.0687	0.0968	0.0973	0.0851	0.0617
	0.0658	0.0649	0.0946	0.0838	0.0585	0.0585
STD. DEVIATIONS	0.0332	0.0691	0.1033	0.0592	0.0452	0.0303
	0.0441	0.0511	0.0657	0.0580	0.0360	0.0271
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0003	0.0003	0.0004	0.0005	0.0004	0.0003
	0.0003	0.0003	0.0004	0.0004	0.0003	0.0003
STD. DEVIATIONS	0.0002	0.0003	0.0005	0.0003	0.0002	0.0001
	0.0002	0.0002	0.0003	0.0003	0.0002	0.0001

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	4.04	(1.532)	14676.1	100.00
RUNOFF	0.005	(0.0236)	19.14	0.130
EVAPOTRANSPIRATION	3.169	(1.3330)	11504.54	78.390
LATERAL DRAINAGE COLLECTED FROM LAYER 2	0.89371	(0.36544)	3244.172	22.10515
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.009	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.000	(0.000)		
CHANGE IN WATER STORAGE	-0.025	(0.4578)	-91.76	-0.625

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.105	382.7216
DRAINAGE COLLECTED FROM LAYER 2	0.02521	91.50326
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.004	
MAXIMUM HEAD ON TOP OF LAYER 3	0.010	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.74	2687.3430
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1784	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0330	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	2.0260	0.0844
2	0.0029	0.0148
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:        C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:    C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:      C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:    C:\HLP3\BRC\BRCSOIL2.D10
OUTPUT DATA FILE:             C:\HLP3\BRC\Summary\SOIL2SUM.OUT


TIME:    8: 5      DATE:   8/31/2006


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     TITLE:  BRCUnit2

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
                                  --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   3
          THICKNESS                   =     24.00   INCHES
          POROSITY                    =      0.4570 VOL/VOL
          FIELD CAPACITY              =      0.0830 VOL/VOL
          WILTING POINT               =      0.0330 VOL/VOL
          INITIAL SOIL WATER CONTENT  =      0.1083 VOL/VOL
          EFFECTIVE SAT. HYD. COND.   =  0.310000009000E-02 CM/SEC


                                  LAYER  2
                                  --------

                       TYPE 2 - LATERAL DRAINAGE LAYER
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MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0108	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	400.0	FEET

LAYER 3

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	0.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	0.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR	

LAYER 4

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.20	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 400. FEET.

SCS RUNOFF CURVE NUMBER	=	80.20	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	18.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.969	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.226	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.594	INCHES
INITIAL SNOW WATER	=	0.000	INCHES

```
INITIAL WATER IN LAYER MATERIALS     =      2.752  INCHES
TOTAL INITIAL WATER                  =      2.752  INCHES
TOTAL SUBSURFACE INFLOW              =      0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

```
STATION LATITUDE                        = 36.08 DEGREES
MAXIMUM LEAF AREA INDEX                 =  0.00
START OF GROWING SEASON (JULIAN DATE)   =    62
END OF GROWING SEASON (JULIAN DATE)     =   321
EVAPORATIVE ZONE DEPTH                  = 18.0  INCHES
AVERAGE ANNUAL WIND SPEED               =  9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY   = 39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY   = 21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY   = 24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY   = 36.00 %
```

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.005	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.021	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.346	0.320	0.328	0.281	0.286	0.166
	0.431	0.257	0.256	0.178	0.422	0.296
STD. DEVIATIONS	0.363	0.338	0.284	0.211	0.239	0.117
	0.712	0.200	0.317	0.117	0.493	0.264
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.0366	0.0424	0.0505	0.0443	0.0435	0.0330
	0.0351	0.0298	0.0460	0.0576	0.0407	0.0411
STD. DEVIATIONS	0.0305	0.0358	0.0484	0.0353	0.0389	0.0219
	0.0173	0.0175	0.0738	0.0683	0.0377	0.0284
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0004	0.0005	0.0006	0.0005	0.0005	0.0004
	0.0004	0.0003	0.0005	0.0007	0.0005	0.0005
STD. DEVIATIONS	0.0003	0.0005	0.0006	0.0004	0.0004	0.0003
	0.0002	0.0002	0.0009	0.0008	0.0004	0.0003

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	4.04	(1.532)	14676.1	100.00
RUNOFF	0.005	(0.0208)	16.91	0.115
EVAPOTRANSPIRATION	3.567	(1.4731)	12948.16	88.226
LATERAL DRAINAGE COLLECTED FROM LAYER 2	0.50069	(0.25506)	1817.490	12.38402
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.008	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.000	(0.000)		
CHANGE IN WATER STORAGE	-0.029	(0.5316)	-106.48	-0.726

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.093	338.2043
DRAINAGE COLLECTED FROM LAYER 2	0.02617	95.01099
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.009	
MAXIMUM HEAD ON TOP OF LAYER 3	0.021	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.74	2687.3430
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1924	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0330	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	2.0124	0.0838
2	0.0032	0.0162
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

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0
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******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
******************************************************************************
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PRECIPITATION DATA FILE:     C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:       C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:   C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:     C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:   C:\HLP3\BRC\BRCSOIL3.D10
OUTPUT DATA FILE:            C:\HLP3\BRC\Summary\SOIL3SUM.OUT


TIME:    8: 6      DATE:    8/31/2006


******************************************************************************

     TITLE:  BRCUnit3

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   3
          THICKNESS                  =     24.00    INCHES
          POROSITY                   =      0.4570 VOL/VOL
          FIELD CAPACITY             =      0.0830 VOL/VOL
          WILTING POINT              =      0.0330 VOL/VOL
          INITIAL SOIL WATER CONTENT =      0.1083 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.310000009000E-02 CM/SEC


                                   LAYER  2
                                   --------

                       TYPE 2 - LATERAL DRAINAGE LAYER
```

```
              MATERIAL TEXTURE NUMBER  20
THICKNESS                    =       0.20    INCHES
POROSITY                     =       0.8500 VOL/VOL
FIELD CAPACITY               =       0.0100 VOL/VOL
WILTING POINT                =       0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =       0.0122 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =    10.0000000000     CM/SEC
SLOPE                        =       2.00    PERCENT
DRAINAGE LENGTH              =    1050.0     FEET


                     LAYER  3
                     --------

           TYPE 4 - FLEXIBLE MEMBRANE LINER
              MATERIAL TEXTURE NUMBER  35
THICKNESS                    =       0.06    INCHES
POROSITY                     =       0.0000 VOL/VOL
FIELD CAPACITY               =       0.0000 VOL/VOL
WILTING POINT                =       0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =       0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY          =       0.00    HOLES/ACRE
FML INSTALLATION DEFECTS     =       0.00    HOLES/ACRE
FML PLACEMENT QUALITY        =  4 - POOR


                     LAYER  4
                     --------

              TYPE 3 - BARRIER SOIL LINER
              MATERIAL TEXTURE NUMBER  17
THICKNESS                    =       0.20    INCHES
POROSITY                     =       0.7500 VOL/VOL
FIELD CAPACITY               =       0.7470 VOL/VOL
WILTING POINT                =       0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT   =       0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.300000003000E-08 CM/SEC


          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

 NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
          SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE
          GROUND CONDITIONS, A SURFACE SLOPE OF  2.% AND
          A SLOPE LENGTH OF 1050. FEET.

SCS RUNOFF CURVE NUMBER            =     79.10
FRACTION OF AREA ALLOWING RUNOFF   =    100.0     PERCENT
AREA PROJECTED ON HORIZONTAL PLANE =      1.000   ACRES
EVAPORATIVE ZONE DEPTH             =     18.0     INCHES
INITIAL WATER IN EVAPORATIVE ZONE  =      1.969   INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE =      8.226   INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE =      0.594   INCHES
INITIAL SNOW WATER                 =      0.000   INCHES
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INITIAL WATER IN LAYER MATERIALS	=	2.753	INCHES
TOTAL INITIAL WATER	=	2.753	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

STATION LATITUDE	=	36.08	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	62	
END OF GROWING SEASON (JULIAN DATE)	=	321	
EVAPORATIVE ZONE DEPTH	=	18.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	39.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	21.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	24.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	36.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.004	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.016	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.348	0.321	0.330	0.283	0.287	0.167
	0.431	0.258	0.258	0.183	0.429	0.303
STD. DEVIATIONS	0.362	0.339	0.280	0.211	0.239	0.117
	0.712	0.199	0.316	0.125	0.489	0.260
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.0339	0.0402	0.0566	0.0436	0.0383	0.0330
	0.0351	0.0310	0.0438	0.0422	0.0262	0.0320
STD. DEVIATIONS	0.0305	0.0365	0.0626	0.0387	0.0309	0.0244
	0.0189	0.0193	0.0750	0.0597	0.0281	0.0211
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0010	0.0013	0.0017	0.0013	0.0011	0.0010
	0.0010	0.0009	0.0014	0.0013	0.0008	0.0010
STD. DEVIATIONS	0.0009	0.0012	0.0019	0.0012	0.0009	0.0008
	0.0006	0.0006	0.0023	0.0018	0.0009	0.0006

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	4.04	(1.532)	14676.1	100.00
RUNOFF	0.004	(0.0159)	12.92	0.088
EVAPOTRANSPIRATION	3.598	(1.4548)	13062.02	89.002
LATERAL DRAINAGE COLLECTED FROM LAYER 2	0.45591	(0.24477)	1654.963	11.27659
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.009	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.001	(0.001)		
CHANGE IN WATER STORAGE	-0.015	(0.5598)	-53.82	-0.367

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.071	258.3089
DRAINAGE COLLECTED FROM LAYER 2	0.02685	97.45744
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.025	
MAXIMUM HEAD ON TOP OF LAYER 3	0.050	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.74	2687.3430

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1937

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0330

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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              FINAL WATER STORAGE AT END OF YEAR   20
    ----------------------------------------------------------------------
               LAYER        (INCHES)       (VOL/VOL)
               -----        --------       ---------
                 1           2.3023          0.0959

                 2           0.0036          0.0184

                 3           0.0000          0.0000

                 4           0.1500          0.7500

              SNOW WATER     0.000

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**                                                                        **
**                                                                        **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE             **
**               HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)               **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                  USAE WATERWAYS EXPERIMENT STATION                     **
**            FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                        **
**                                                                        **
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****************************************************************************


PRECIPITATION DATA FILE:     C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:       C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:   C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:     C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:   C:\HLP3\BRC\BRCSOIL4.D10
OUTPUT DATA FILE:            C:\HLP3\BRC\Summary\SOIL4SUM.OUT


TIME:   8: 8     DATE:   8/31/2006


****************************************************************************

     TITLE:  BRCUnit4

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                 LAYER  1
                                 --------

                    TYPE 1 - VERTICAL PERCOLATION LAYER
                       MATERIAL TEXTURE NUMBER  3
           THICKNESS                  =     24.00   INCHES
           POROSITY                   =      0.4570 VOL/VOL
           FIELD CAPACITY             =      0.0830 VOL/VOL
           WILTING POINT              =      0.0330 VOL/VOL
           INITIAL SOIL WATER CONTENT =      0.1083 VOL/VOL
           EFFECTIVE SAT. HYD. COND.  =  0.310000009000E-02 CM/SEC


                                 LAYER  2
                                 --------

                      TYPE 2 - LATERAL DRAINAGE LAYER
```

MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0108	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	3.00	PERCENT
DRAINAGE LENGTH	=	550.0	FEET

LAYER 3

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	0.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	0.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR	

LAYER 4

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.20	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 3.% AND A SLOPE LENGTH OF 550. FEET.

SCS RUNOFF CURVE NUMBER	=	80.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	18.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.969	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.226	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.594	INCHES
INITIAL SNOW WATER	=	0.000	INCHES

```
INITIAL WATER IN LAYER MATERIALS    =      2.752  INCHES
TOTAL INITIAL WATER                 =      2.752  INCHES
TOTAL SUBSURFACE INFLOW             =      0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

```
STATION LATITUDE                          = 36.08 DEGREES
MAXIMUM LEAF AREA INDEX                   =  0.00
START OF GROWING SEASON (JULIAN DATE)     =    62
END OF GROWING SEASON (JULIAN DATE)       =   321
EVAPORATIVE ZONE DEPTH                    = 18.0  INCHES
AVERAGE ANNUAL WIND SPEED                 =  9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY     = 39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY     = 21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY     = 24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY     = 36.00 %
```

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.004	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.020	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.346	0.320	0.328	0.281	0.286	0.166
	0.431	0.257	0.256	0.178	0.422	0.296
STD. DEVIATIONS	0.363	0.338	0.284	0.211	0.239	0.117
	0.712	0.200	0.317	0.117	0.493	0.264
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.0366	0.0424	0.0505	0.0443	0.0435	0.0330
	0.0351	0.0297	0.0463	0.0576	0.0407	0.0411
STD. DEVIATIONS	0.0305	0.0358	0.0484	0.0353	0.0389	0.0219
	0.0174	0.0174	0.0748	0.0683	0.0377	0.0284
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0004	0.0005	0.0005	0.0005	0.0005	0.0004
	0.0004	0.0003	0.0005	0.0006	0.0004	0.0004
STD. DEVIATIONS	0.0003	0.0004	0.0005	0.0004	0.0004	0.0002
	0.0002	0.0002	0.0008	0.0007	0.0004	0.0003

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SOIL4SUM.OUT

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	4.04	(1.532)	14676.1	100.00
RUNOFF	0.004	(0.0199)	16.14	0.110
EVAPOTRANSPIRATION	3.567	(1.4731)	12948.16	88.226
LATERAL DRAINAGE COLLECTED FROM LAYER 2	0.50090	(0.25564)	1818.280	12.38940
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.008	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.000	(0.000)		
CHANGE IN WATER STORAGE	-0.029	(0.5316)	-106.49	-0.726

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.089	322.7272
DRAINAGE COLLECTED FROM LAYER 2	0.02705	98.20731
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.009	
MAXIMUM HEAD ON TOP OF LAYER 3	0.004	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	436.3 FEET	
SNOW WATER	0.74	2687.3430
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1927	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0330	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	2.0124	0.0838
2	0.0031	0.0157
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

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****

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*******************************************************************************
*******************************************************************************
**                                                                           **
**                                                                           **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                **
**               HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                  **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY                      **
**                  USAE WATERWAYS EXPERIMENT STATION                        **
**            FOR USEPA RISK REDUCTION ENGINEERING LABORATORY                **
**                                                                           **
**                                                                           **
*******************************************************************************
*******************************************************************************


PRECIPITATION DATA FILE:     C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:       C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:   C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:     C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:   C:\HLP3\BRC\BRCSOIL5.D10
OUTPUT DATA FILE:            C:\HLP3\BRC\Summary\SOIL5SUM.OUT


TIME:   8:10     DATE:   8/31/2006


*******************************************************************************

     TITLE:  BRCUnit5

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
                                  --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   3
           THICKNESS                  =      24.00   INCHES
           POROSITY                   =       0.4570 VOL/VOL
           FIELD CAPACITY             =       0.0830 VOL/VOL
           WILTING POINT              =       0.0330 VOL/VOL
           INITIAL SOIL WATER CONTENT =       0.1083 VOL/VOL
           EFFECTIVE SAT. HYD. COND.  =   0.310000009000E-02 CM/SEC


                                  LAYER  2
                                  --------

                       TYPE 2 - LATERAL DRAINAGE LAYER
```

```
              MATERIAL TEXTURE NUMBER  20
THICKNESS                  =       0.20    INCHES
POROSITY                   =       0.8500 VOL/VOL
FIELD CAPACITY             =       0.0100 VOL/VOL
WILTING POINT              =       0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT =       0.0109 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =    10.0000000000     CM/SEC
SLOPE                      =       3.00    PERCENT
DRAINAGE LENGTH            =     675.0     FEET


                      LAYER  3
                      --------

           TYPE 4 - FLEXIBLE MEMBRANE LINER
              MATERIAL TEXTURE NUMBER  35
THICKNESS                  =       0.06    INCHES
POROSITY                   =       0.0000 VOL/VOL
FIELD CAPACITY             =       0.0000 VOL/VOL
WILTING POINT              =       0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT =       0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY        =       0.00    HOLES/ACRE
FML INSTALLATION DEFECTS   =       0.00    HOLES/ACRE
FML PLACEMENT QUALITY      =  4 - POOR


                      LAYER  4
                      --------

              TYPE 3 - BARRIER SOIL LINER
              MATERIAL TEXTURE NUMBER  17
THICKNESS                  =       0.20    INCHES
POROSITY                   =       0.7500 VOL/VOL
FIELD CAPACITY             =       0.7470 VOL/VOL
WILTING POINT              =       0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT =       0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.300000003000E-08 CM/SEC


         GENERAL DESIGN AND EVAPORATIVE ZONE DATA
         ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE
         GROUND CONDITIONS, A SURFACE SLOPE OF  3.% AND
         A SLOPE LENGTH OF  675. FEET.

SCS RUNOFF CURVE NUMBER            =     79.80
FRACTION OF AREA ALLOWING RUNOFF   =    100.0     PERCENT
AREA PROJECTED ON HORIZONTAL PLANE =      1.000   ACRES
EVAPORATIVE ZONE DEPTH             =     18.0     INCHES
INITIAL WATER IN EVAPORATIVE ZONE  =      1.969   INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE =      8.226   INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE =      0.594   INCHES
INITIAL SNOW WATER                 =      0.000   INCHES
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INITIAL WATER IN LAYER MATERIALS     =      2.752  INCHES
TOTAL INITIAL WATER                  =      2.752  INCHES
TOTAL SUBSURFACE INFLOW              =      0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

```
STATION LATITUDE                          = 36.08 DEGREES
MAXIMUM LEAF AREA INDEX                   =  0.00
START OF GROWING SEASON (JULIAN DATE)     =    62
END OF GROWING SEASON (JULIAN DATE)       =   321
EVAPORATIVE ZONE DEPTH                    = 18.0  INCHES
AVERAGE ANNUAL WIND SPEED                 =  9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY     = 39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY     = 21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY     = 24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY     = 36.00 %
```

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.004	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.019	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.346	0.319	0.330	0.284	0.286	0.166
	0.430	0.257	0.256	0.180	0.424	0.301
STD. DEVIATIONS	0.363	0.338	0.282	0.210	0.239	0.117
	0.712	0.200	0.317	0.127	0.487	0.260
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.0366	0.0449	0.0524	0.0395	0.0416	0.0328
	0.0349	0.0303	0.0463	0.0541	0.0359	0.0383
STD. DEVIATIONS	0.0302	0.0391	0.0487	0.0306	0.0372	0.0217
	0.0181	0.0189	0.0756	0.0669	0.0312	0.0257
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0005	0.0006	0.0007	0.0005	0.0005	0.0004
	0.0004	0.0004	0.0006	0.0007	0.0005	0.0005
STD. DEVIATIONS	0.0004	0.0006	0.0006	0.0004	0.0005	0.0003
	0.0002	0.0002	0.0010	0.0009	0.0004	0.0003

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	4.04	(1.532)	14676.1	100.00
RUNOFF	0.004	(0.0190)	15.38	0.105
EVAPOTRANSPIRATION	3.580	(1.4744)	12994.72	88.543
LATERAL DRAINAGE COLLECTED FROM LAYER 2	0.48768	(0.24897)	1770.263	12.06223
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.008	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.001	(0.000)		
CHANGE IN WATER STORAGE	-0.029	(0.5295)	-104.28	-0.711

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.085	307.6796
DRAINAGE COLLECTED FROM LAYER 2	0.02719	98.70956
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.011	
MAXIMUM HEAD ON TOP OF LAYER 3	0.022	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.74	2687.3430
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1929	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0330	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	2.0231	0.0843
2	0.0046	0.0232
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

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**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:        C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:    C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:      C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:    C:\HLP3\BRC\BRCSOILa.D10
OUTPUT DATA FILE:             C:\HLP3\BRC\Summary\SOILASUM.OUT


TIME:   8:11      DATE:   8/31/2006


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     TITLE:  BRCUnitA

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                         MATERIAL TEXTURE NUMBER   3
          THICKNESS                    =     24.00    INCHES
          POROSITY                     =      0.4570 VOL/VOL
          FIELD CAPACITY               =      0.0830 VOL/VOL
          WILTING POINT                =      0.0330 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.0565 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.310000009000E-02 CM/SEC


                                   LAYER  2
                                   --------

                        TYPE 2 - LATERAL DRAINAGE LAYER
```

MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0101	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	48.00	PERCENT
DRAINAGE LENGTH	=	100.0	FEET

LAYER 3

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	0.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	0.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR	

LAYER 4

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.20	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 48.% AND A SLOPE LENGTH OF 100. FEET.

SCS RUNOFF CURVE NUMBER	=	83.20	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	18.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	0.594	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.226	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.594	INCHES
INITIAL SNOW WATER	=	0.000	INCHES

INITIAL WATER IN LAYER MATERIALS	=	1.507	INCHES
TOTAL INITIAL WATER	=	1.507	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

STATION LATITUDE	=	36.08 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	62
END OF GROWING SEASON (JULIAN DATE)	=	321
EVAPORATIVE ZONE DEPTH	=	18.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	36.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.005	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.022	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.018	0.015	0.009	0.010	0.008	0.002
	0.036	0.006	0.008	0.003	0.016	0.019
STD. DEVIATIONS	0.041	0.037	0.017	0.026	0.013	0.004
	0.094	0.008	0.018	0.003	0.033	0.040
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.3228	0.4337	0.3217	0.2672	0.2977	0.1287
	0.4109	0.4867	0.3369	0.2050	0.4156	0.2610
STD. DEVIATIONS	0.3156	0.3623	0.2745	0.1676	0.2297	0.0926
	0.6111	0.3682	0.4862	0.1405	0.4363	0.1747
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000
	0.0001	0.0001	0.0001	0.0000	0.0001	0.0000
STD. DEVIATIONS	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000
	0.0001	0.0001	0.0001	0.0000	0.0001	0.0000

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SOILASUM.OUT

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES	CU. FEET	PERCENT
PRECIPITATION	4.04 (1.532)	14676.1	100.00
RUNOFF	0.005 (0.0223)	18.10	0.123
EVAPOTRANSPIRATION	0.149 (0.1470)	539.37	3.675
LATERAL DRAINAGE COLLECTED FROM LAYER 2	3.88799 (1.37123)	14113.411	96.16602
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000 (0.00000)	0.009	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.000 (0.000)		
CHANGE IN WATER STORAGE	0.001 (0.1516)	5.20	0.035

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.100	361.7648
DRAINAGE COLLECTED FROM LAYER 2	0.95379	3462.24854
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.005	
MAXIMUM HEAD ON TOP OF LAYER 3	0.007	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.74	2687.3430

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.0406

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0330

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	1.3840	0.0577
2	0.0020	0.0101
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\HLP3\BRC\BRCPREC.D4
TEMPERATURE DATA FILE:        C:\HLP3\BRC\BRCTEMP.D7
SOLAR RADIATION DATA FILE:    C:\HLP3\BRC\BRCSOLAR.D13
EVAPOTRANSPIRATION DATA:      C:\HLP3\BRC\BRCEVAP.D11
SOIL AND DESIGN DATA FILE:    C:\HLP3\BRC\BRCSOILB.D10
OUTPUT DATA FILE:             C:\HLP3\BRC\Summary\SOILBSUM.OUT


TIME:    8:14     DATE:    8/31/2006


******************************************************************************

     TITLE:  BRCUnitB

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
                                  --------

                        TYPE 1 - VERTICAL PERCOLATION LAYER
                            MATERIAL TEXTURE NUMBER   3
          THICKNESS                    =      24.00    INCHES
          POROSITY                     =       0.4570 VOL/VOL
          FIELD CAPACITY               =       0.0830 VOL/VOL
          WILTING POINT                =       0.0330 VOL/VOL
          INITIAL SOIL WATER CONTENT   =       0.0565 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.310000009000E-02 CM/SEC


                                  LAYER  2
                                  --------

                          TYPE 2 - LATERAL DRAINAGE LAYER
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```
              MATERIAL TEXTURE NUMBER  20
THICKNESS                   =       0.20   INCHES
POROSITY                    =       0.8500 VOL/VOL
FIELD CAPACITY              =       0.0100 VOL/VOL
WILTING POINT               =       0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =    10.0000000000     CM/SEC
SLOPE                       =     48.00    PERCENT
DRAINAGE LENGTH             =     75.0     FEET


                         LAYER  3
                         --------

            TYPE 4 - FLEXIBLE MEMBRANE LINER
              MATERIAL TEXTURE NUMBER  35
THICKNESS                   =       0.06   INCHES
POROSITY                    =       0.0000 VOL/VOL
FIELD CAPACITY              =       0.0000 VOL/VOL
WILTING POINT               =       0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =       0.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =       0.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  4 - POOR


                         LAYER  4
                         --------

              TYPE 3 - BARRIER SOIL LINER
              MATERIAL TEXTURE NUMBER  17
THICKNESS                   =       0.20   INCHES
POROSITY                    =       0.7500 VOL/VOL
FIELD CAPACITY              =       0.7470 VOL/VOL
WILTING POINT               =       0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC


          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

 NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
          SOIL DATA BASE USING SOIL TEXTURE # 3 WITH BARE
          GROUND CONDITIONS, A SURFACE SLOPE OF 48.% AND
          A SLOPE LENGTH OF    75. FEET.

 SCS RUNOFF CURVE NUMBER                =      83.40
 FRACTION OF AREA ALLOWING RUNOFF       =     100.0     PERCENT
 AREA PROJECTED ON HORIZONTAL PLANE     =       1.000   ACRES
 EVAPORATIVE ZONE DEPTH                 =      18.0     INCHES
 INITIAL WATER IN EVAPORATIVE ZONE      =       0.594   INCHES
 UPPER LIMIT OF EVAPORATIVE STORAGE     =       8.226   INCHES
 LOWER LIMIT OF EVAPORATIVE STORAGE     =       0.594   INCHES
 INITIAL SNOW WATER                     =       0.000   INCHES
```

```
INITIAL WATER IN LAYER MATERIALS     =        1.507  INCHES
TOTAL INITIAL WATER                  =        1.507  INCHES
TOTAL SUBSURFACE INFLOW              =        0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

```
NOTE:  EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
         LAS VEGAS             NEVADA

        STATION LATITUDE                          = 36.08 DEGREES
        MAXIMUM LEAF AREA INDEX                   =  0.00
        START OF GROWING SEASON (JULIAN DATE)     =    62
        END OF GROWING SEASON (JULIAN DATE)       =   321
        EVAPORATIVE ZONE DEPTH                    = 18.0  INCHES
        AVERAGE ANNUAL WIND SPEED                 =  9.10 MPH
        AVERAGE 1ST QUARTER RELATIVE HUMIDITY     = 39.00 %
        AVERAGE 2ND QUARTER RELATIVE HUMIDITY     = 21.00 %
        AVERAGE 3RD QUARTER RELATIVE HUMIDITY     = 24.00 %
        AVERAGE 4TH QUARTER RELATIVE HUMIDITY     = 36.00 %
```

```
NOTE:  PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
         COEFFICIENTS FOR    LAS VEGAS           NEVADA
```

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

```
NOTE:  TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
         COEFFICIENTS FOR    LAS VEGAS           NEVADA
```

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

```
NOTE:  SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
         COEFFICIENTS FOR    LAS VEGAS           NEVADA
           AND STATION LATITUDE  =  36.08 DEGREES
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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS   1 THROUGH   20

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.33	0.45	0.33	0.26	0.29	0.07
	0.51	0.51	0.34	0.17	0.49	0.28
STD. DEVIATIONS	0.37	0.49	0.34	0.23	0.33	0.13
	0.73	0.40	0.55	0.16	0.50	0.22
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.005	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.023	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.018	0.015	0.009	0.010	0.008	0.002
	0.036	0.006	0.007	0.003	0.016	0.019
STD. DEVIATIONS	0.041	0.037	0.017	0.026	0.013	0.004
	0.094	0.008	0.015	0.003	0.033	0.040
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.3226	0.4341	0.3212	0.2677	0.2978	0.1285
	0.4111	0.4865	0.3374	0.2053	0.4149	0.2614
STD. DEVIATIONS	0.3153	0.3621	0.2753	0.1679	0.2298	0.0927
	0.6112	0.3682	0.4889	0.1407	0.4363	0.1748
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
DAILY AVERAGE HEAD ON TOP OF LAYER 3						
AVERAGES	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0001	0.0000	0.0001	0.0000	0.0001	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 20

	INCHES		CU. FEET	PERCENT
PRECIPITATION	4.04	(1.532)	14676.1	100.00
RUNOFF	0.005	(0.0235)	19.05	0.130
EVAPOTRANSPIRATION	0.148	(0.1460)	537.00	3.659
LATERAL DRAINAGE COLLECTED FROM LAYER 2	3.88839	(1.37242)	14114.859	96.17588
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00000	(0.00000)	0.009	0.00006
AVERAGE HEAD ON TOP OF LAYER 3	0.000	(0.000)		
CHANGE IN WATER STORAGE	0.001	(0.1520)	5.16	0.035

****

****

PEAK DAILY VALUES FOR YEARS 1 THROUGH 20

	(INCHES)	(CU. FT.)
PRECIPITATION	1.83	6642.900
RUNOFF	0.105	380.3883
DRAINAGE COLLECTED FROM LAYER 2	0.91576	3324.21558
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000000	0.00002
AVERAGE HEAD ON TOP OF LAYER 3	0.005	
MAXIMUM HEAD ON TOP OF LAYER 3	0.005	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.74	2687.3430

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.0397

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0330

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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□
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FINAL WATER STORAGE AT END OF YEAR 20

LAYER	(INCHES)	(VOL/VOL)
1	1.3838	0.0577
2	0.0020	0.0101
3	0.0000	0.0000
4	0.1500	0.7500
SNOW WATER	0.000	

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Table VI Values of n for use with Manning Equation

Surface	n. Range	n, design
Polyethylene pipe	0.008-0.011	0.009 *
Uncoated cast or ductile iron pipe	0.012-0.015	0.013
Corrugated steel pipe	0.021-0.030	0.024
Concrete pipe	0.012-0.016	0.015
Vitrified clay pipe	0.011-0.017	0.013
Brick & cement mortar sewers	0.012-0.017	0.015
Wood stave	0.010-0.013	0.011
Rubble masonry	0.017-0.030	0.021

Some gravity flow piping systems may become very complex, especially if the pipeline grade varies, because friction loss will vary along the run. With a varying grade, parts of the line may develop internal pressure, or vacuum, and may have varying liquid levels in the bore.

Manning

For open channel water flow under conditions of constant grade, and uniform channel cross section, the Manning equation may be used. Open channel flow exists in a pipe when it runs partially full. Like the Hazen-Williams formula, the Manning equation is limited to water or liquids with a kinematic viscosity equal to water.

Manning Equation **(20)**

$$V = \frac{1.486}{n} r^{2/3} S^{1/2}$$

where

V = flow velocity, ft/sec
n = roughness coefficient, dimensionless
r = hydraulic radius, ft

$$r = \frac{A}{P} \quad * \quad (21)$$

A = channel cross section area, ft^2
P = perimeter wetted by flow, ft
S = hydraulic slope, ft/ft

$$S = \frac{h_1 - h_2}{L} = \frac{h_f}{L} \quad (22)$$

h_1 = upstream pipe elevation, ft
h_2 = downstream pipe elevation, ft
h_f = friction loss, ft of liquid

It is convenient to combine the Manning equation with

$$Q = A V \quad (23)$$

to obtain

$$Q = \frac{1.486 A}{n} r^{2/3} S^{1/2} \quad * \quad (24)$$

where terms are as defined above, and

Q = flow, ft^3/secWhen a circular pipe is running full or half-full,

$$r = \frac{D}{4} = \frac{d}{48} \quad (25)$$

ATTACHMENT B, 1/1

CHEVRON PLEXCO/SPIROLITE ENGINEERING MANUAL

Driscopipe 1000 Series Industrial Pipe; Sizes and Dimensions

Nom. Size, in.	Dimensions, inches		DR	$P_{73.4°F}$ psig	Weight lb/100ft
	OD	Min.Wall			
3/4	1.050	0.095	11.0	160	12
1	1.315	0.120	11.0	160	19
1 3/4	1.660	0.151	11.0	160	31
1 1/2	1.900	0.173	11.0	160	41

Nom. Size, in.	Dimensions, in.		DR	$P_{73.4°F}$ psig	Weight lb/100ft
	OD	Min.Wall			
2	2.375	0.339	7.0	267	94
2	2.375	0.264	9.0	200	76
2	2.375	0.216	11.0	160	64
2	2.375	0.176	13.5	128	53
2	2.375	0.140	17.0	100	43
3	3.500	0.500	7.0	267	205
3	3.500	0.389	9.0	200	166
3	3.500	0.318	11.0	160	139
3	3.500	0.259	13.5	128	115
3	3.500	0.206	17.0	100	93
3	3.500	0.167	21.0	80	77
3	3.500	0.135	26.0	64	62
4	4.500	0.643	7.0	267	339
4	4.500	0.500	9.0	200	274
4	4.500	0.409	11.0	160	229
4	4.500	0.333	13.5	128	190
4	4.500	0.265	17.0	100	154
4	4.500	0.214	21.0	80	126
4	4.500	0.173	26.0	64	103
4	4.500	0.138	32.5	51	83
5	5.563	0.795	7.0	267	517
5	5.563	0.618	9.0	200	418
5	5.563	0.506	11.0	160	351
5	5.563	0.412	13.5	128	291
5	5.563	0.327	17.0	100	235
5	5.563	0.265	21.0	80	193
5	5.563	0.214	26.0	64	157
5	5.563	0.171	32.5	51	127
5.375	5.375	0.316	17.0	100	220
5.375	5.375	0.256	21.0	80	180
5.375	5.375	0.207	26.0	64	147
5.375	5.375	0.165	32.5	51	118
6	6.625	0.946	7.0	267	733
6	6.625	0.736	9.0	200	593
6	6.625	0.602	11.0	160	487
6	6.625	0.491	13.5	128	413
6	6.625	0.390	17.0	100	334
6	6.625	0.315	21.0	80	273
6	6.625	0.255	26.0	64	223
6	6.625	0.204	32.5	51	180

Nom. Size, in.	Dimensions, in.		DR	$P_{73.4°F}$ psig	We lb/
	OD	Min.Wall			
7	7.125	1.018	7.0	267	84
7	7.125	0.792	9.0	200	6
7	7.125	0.648	11.0	160	5
7	7.125	0.528	13.5	128	4
7	7.125	0.420	17.0	100	3
7	7.125	0.340	21.0	80	3
7	7.125	0.274	26.0	64	2
7	7.125	0.220	32.8	51.0	2
8	8.625	1.232	7.0	267	12
8	8.625	0.958	9.0	200	10
8	8.625	0.784	11.0	160	8
8	8.625	0.639	13.5	128	7
8	8.625	0.507	17.0	100	5
8	8.625	0.411	21.0	80	4
8	8.625	0.332	26.0	64	3
8	8.625	0.265	32.5	51	3
10	10.750	1.536	7.0	267	19
10	10.750	1.194	9.0	200	15
10	10.750	0.977	11.0	160	13
10	10.750	0.796	13.5	128	10
10	10.750	0.632	17.0	100	8
10	10.750	0.512	21.0	80	7
10	10.750	0.413	26.0	64	5
10	10.750	0.331	32.5	51	4
12	12.750	1.821	7.0	267	27
12	12.750	1.417	9.0	200	21
12	12.750	1.159	11.0	160	18
12	12.750	0.944	13.5	128	15
12	12.750	0.750	17.0	100	12
12	12.750	0.607	21.0	80	10
12	12.750	0.490	26.0	64	8
12	12.750	0.392	32.5	51	66
14	14.000	2.000	7.0	267	32
14	14.000	1.556	9.0	200	26
14	14.000	1.273	11.0	160	22
14	14.000	1.037	13.5	128	18
14	14.000	0.824	17.0	100	14
14	14.000	0.667	21.0	80	12
14	14.000	0.538	26.0	64	9
14	14.000	0.431	32.5	51	8
16	16.000	2.286	7.0	267	42
16	16.000	1.778	9.0	200	34
16	16.000	1.455	11.0	160	29
16	16.000	1.185	13.5	128	24
16	16.000	0.941	17.0	100	19
16	16.000	0.762	21.0	80	15
16	16.000	0.615	26.0	64	13
16	16.000	0.492	32.5	51	10

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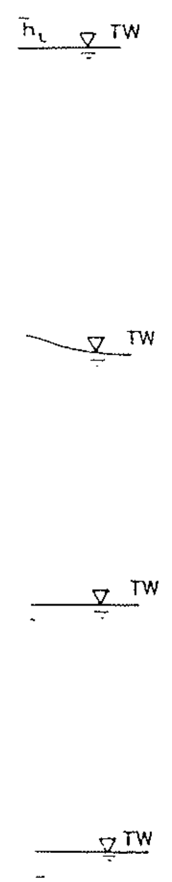
raulic conditions are dis-

the result of inadequate
ert discharge is primarily
and the head loss of the
ulvert flow can be treated
culvert, h_L, is the sum of



'verts.

an entrance loss, h_{ent}, friction loss, h_f, and the barrel.

$$h_L = h_{ent} + h_f + \frac{V^2}{2g}$$

Substituting Equations (3.32) and (3.16) for h_{ent} and h_f, respectively, we have

$$h_L = k_{ent}\left(\frac{V^2}{2g}\right) + \frac{n^2V^2L}{R_h^{4/3}} + \frac{V^2}{2g} \tag{8.17}$$

The entrance coefficient, k_{ent}, is approximately 0.5 for a square-edged entrance and approximately 0.1 for a well-rounded entrance. Common values used for the Manning's roughness coefficient are $n = 0.012$ for concrete pipe and $n = 0.024$ for corrugated steel pipe. Equation (8.17) may be rearranged to express the direct relationship between the discharge and the dimensions of the culvert at any given elevation difference, h_L, between tail water and head water. For a circular culvert,

$$h_L = \left[K_{ent} + \left(\frac{n^2L}{R_h^{4/3}}\right)(2g) + 1\right]\frac{8Q^2}{\pi^2 gD^4} \tag{8.18}$$

where Q is the discharge, D is the diameter, and R_h is the hydraulic radius ($R_h = D/4$) of the culvert barrel. For culverts with noncircular cross sections, the head loss may be calculated by Equation (8.17) with the corresponding hydraulic radius calculated by using the cross-sectional area, A, and the wetted perimeter, P.

2. If the discharge carried in a culvert has a normal depth that is larger than the barrel height, the culvert will flow full even if the tail water level drops below that of the outlet. In this case, the discharge is controlled by the head loss and the level of the head water (HW). The hydraulics are the same as discussed before.
3. If the normal depth is less than the barrel height, with the inlet submerged and free discharge at the outlet, a partially full pipe flow condition will normally result, as illustrated in Figure 8.17(c). The culvert discharge is controlled by the entrance condition, and the flow is said to be under *entrance* control. The discharge can be calculated by

$$* \qquad Q = C_d A\sqrt{2gh} \tag{8.19}$$

where h is the hydrostatic head above the center of the orifice and A is the cross-sectional area. C_d is the coefficient of discharge; common

HWANG "FUNDAMENTALS OF HYDRAULIC ENGINEERING SYSTEMS"
2nd EDITION, PRENTICE-HALL, 1987

ATTACHMENT D 1/2

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*

values used in practice are $\boxed{C_d = 0.62}$ for a square-edged entrance and $C_d = 1.0$ for a well-rounded entrance.

4. When the hydrostatic head at the entrance is less than $1.2D$, air will break into the barrel and the culvert will flow under no pressure. In this case, the culvert slope and the barrel wall friction determine the flow condition in the culvert for open channel flow. Due to a sudden reduction of the water area at the entrance, the flow usually enters the culvert in a supercritical condition. The critical depth takes place at the entrance of the barrel. The friction of the barrel wall gradually dissipates the energy. If the rate of dissipation is higher than the flow could gain from the barrel slope, the depth of the flowing water will increase in the downstream direction. Depending on the tail water level, the supercritical flow may convert to subcritical flow through a hydraulic jump. The flow conditions can be computed by applying the water surface profiles developed for open channels.

Example 8.4

A corrugated steel pipe is used as a culvert that must carry a flow rate of 5.3 m^3/sec and discharge into the air. At the entrance, the maximum available water head is 3.2 m above the bottom as shown in Figure 8.18. The culvert is 35 m long and has a square-edged entrance and slope of 0.003. Determine the diameter of the pipe.

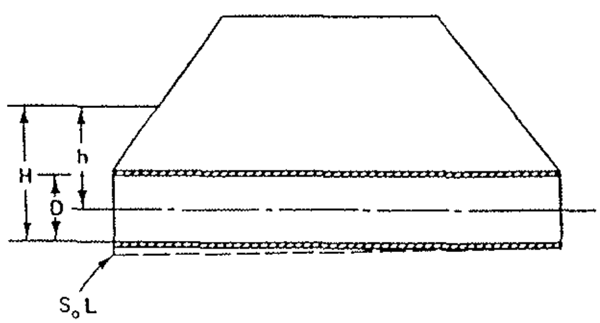


Figure 8.18

Solution

(a) Allowing full pipe flow, the energy balance of the culvert flow may be expressed as (see Figure 8.18),

$$h_L = H - D + S_0 L = 3.2 - D + 0.003 \cdot 35$$
$$= 3.305 - D$$

where D is the diameter of the pipe. Also, from Equation (8.18), we have,

ATTACHMENT D, 2/2

$h_L =$

$=$

Combining bot

D

By trial, it is fo

(b) If the pipe flow entrance cond the centerline

so that

and the orifice discharge

$Q =$

$=$

By trial, it is f

Because the it is evident t

8.9.1 A culvert is install the maximum he corrugated steel p

8.9.2 A rectangular con culvert is 15 m l elevation is 1.8 m evation necessary

/C

The U.S. Bureau of Reclamation (1973) uses the following criterion for grain size of filter materials in relation to openings in pipes:

$$\frac{D_{85} \text{ of the filter nearest the pipe}}{\text{maximum opening of pipe drain}} = 2 \text{ or more} \qquad (5.6)$$

Equations 5.4, 5.5, and 5.6 represent a reasonable range over which satisfactory performance can be expected.

An important development in the manufacture of drainage pipes is the slotted PVC pipe (Cedergren, 1987) which has slots machined to specified widths from a minimum of 0.010 in. (0.25 mm) up to 0.10 in. (2.54 mm) or larger. Figure 5.3 shows a PVC pipe 6 in. (15.2 cm) in diameter with sawed slots of uniform size. Close control over the width of the slots ensures free flow of water into the pipe without danger of clogging with soil when the slotwidths have been correctly established with Eq. 5.4.

5.3 EXAMPLES OF FILTER DESIGNS TO PREVENT PIPING

Historical

Before the development of rational and experimental filter design criteria drain design was considered more of an art than a science. Designers depended on judgment, instinct, or precedent. In many instances coarse stone or gravel was placed in direct contact with fine-grained soils with the result that drains often became clogged or soil piped through them, thus causing structural failures.

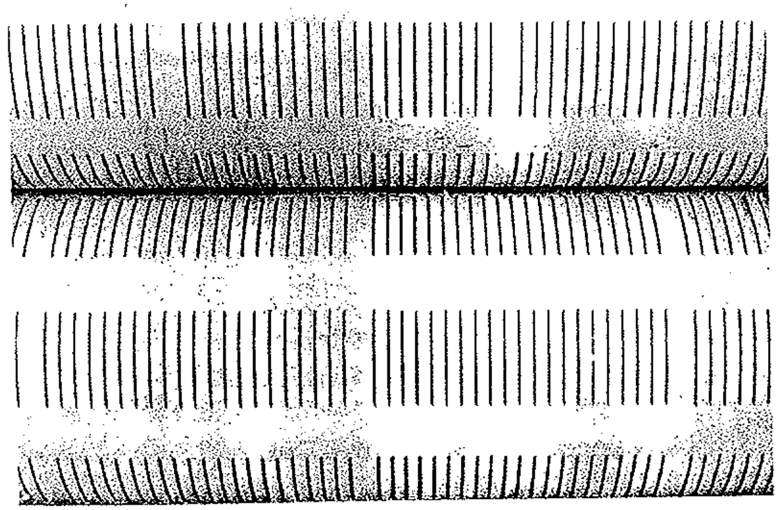
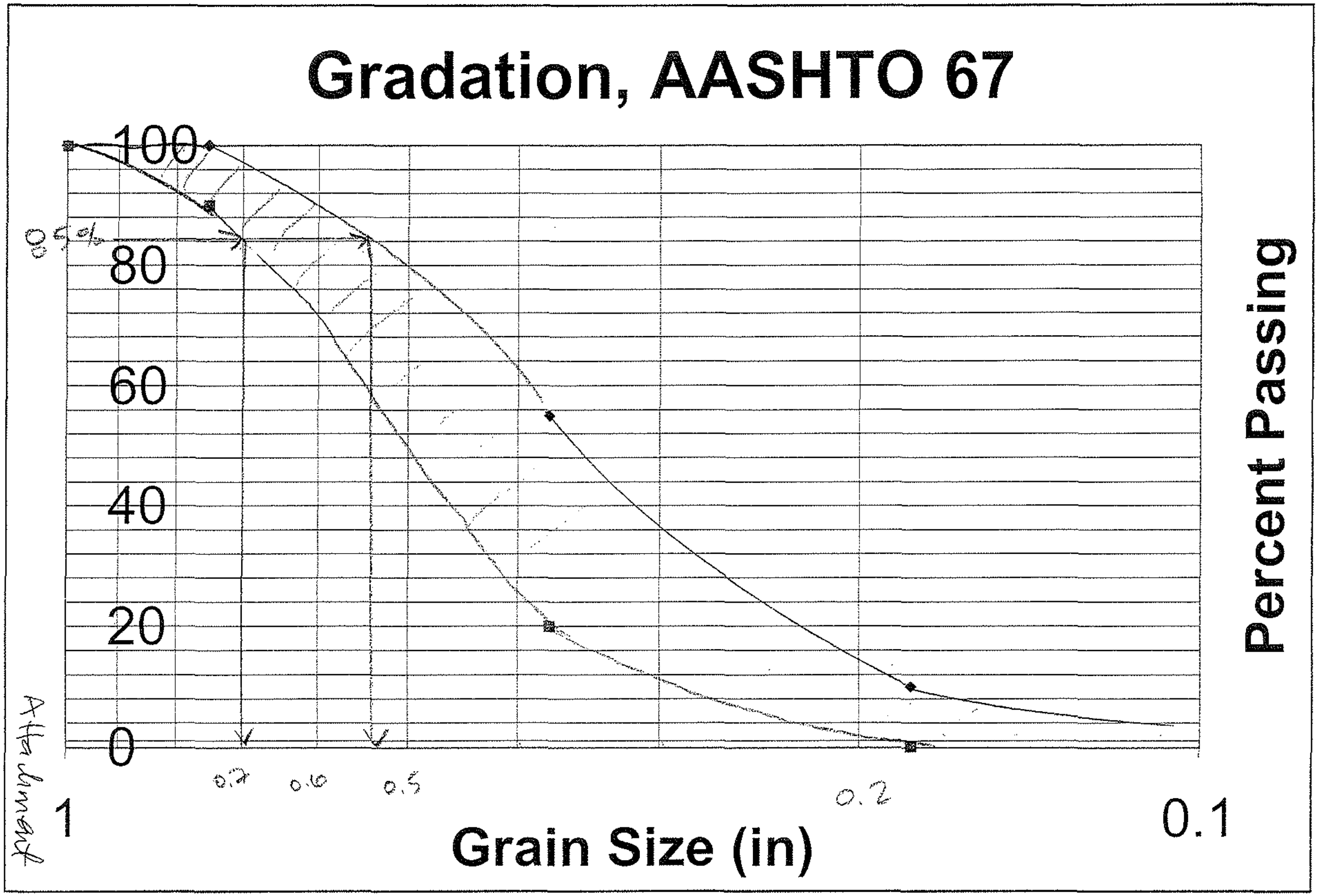


FIG. 5.3 Slotted PVC drain pipe 6-in. in diameter. Machined slots of uniform width provide good filter protection.

Attachment E

1/1



Attachment F

1/4

pu 5A

STANDARD SPECIFICATIONS

for

TRANSPORTATION MATERIALS

and

METHODS OF SAMPLING AND TESTING

Seventeenth Edition

1995

PART I SPECIFICATIONS

Adopted by the

AMERICAN ASSOCIATION OF STATE HIGHWAY
AND TRANSPORTATION OFFICIALS

Standard Specification
for

Sizes of Aggregate for Road and Bridge Construction

AASHTO DESIGNATION: M 43-88
(ASTM DESIGNATION: D 448-86)

1. SCOPE

1.1 This specification defines aggregate size designations and ranges in mechanical analyses for standard sizes of coarse aggregate and screenings for use in the construction and maintenance of various types of highways and bridges.

1.2 The values stated in SI units are to be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards:*

- T 27 Sieve Analysis of Fine and Coarse Aggregates
- T 2 Sampling Aggregates
- M 92 Wire-Cloth Sieves for Testing Purposes

3. SIGNIFICANCE AND USE

3.1 Contract documents may specify certain of these aggregate sizes for specific uses or may suggest one or more of these sizes as appropriate for the preparation of various end-product mixtures. In some cases, closer limits on variability of the aggregate grading may be required.

4. MANUFACTURE

4.1 The standard sizes of aggregate described in this classification may be manufactured by means of any suitable process used to separate raw material into the desired size ranges. Standard sizes may also be produced by blending two or more different components.

5. STANDARD SIZES

5.1 Standard sizes of coarse aggregate shall comply with the sizes given in Table 1. All sizes shall be determined by means of laboratory sieves having square openings and conforming to M 92.

6. BASIS OF CLASSIFICATION

6.1 Classification is based upon the size number and size ranges shown in Table 1 with the aggregate sampled in accordance with T 2 and tested for grading by T 27.

AH. F

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TABLE 1 Standard Sizes of Processed Aggregate

Size Number	Nominal Size, Square Openings	Amounts Finer than Each Laboratory Sieve (Square Openings), Mass Percent														
		100-mm (4-in.)	90-mm (3½-in.)	75-mm (3-in.)	63-mm (2½-in.)	50-mm (2-in.)	37.5-mm (1½-in.)	25.0-mm (1-in.)	19.0-mm (¾-in.)	12.5-mm (½-in.)	9.5-mm (⅜-in.)	4.75-mm (No. 4)	2.36-mm (No. 8)	1.18-mm (No. 16)	300-µm (No. 50)	150-µm (No. 100)
1	90 to 37.5-mm (3½ to 1½-in.)	100	90 to 100	—	25 to 60	—	0 to 15	—	0 to 5	—	—	—	—	—	—	—
2	63 to 37.5-mm (2½ to 1½-in.)	—	—	100	90 to 100	35 to 70	0 to 15	—	0 to 5	—	—	—	—	—	—	—
24	63 to 19.0-mm (2½ to ¾-in.)	—	—	100	90 to 100	—	25 to 60	—	0 to 10	0 to 5	—	—	—	—	—	—
3	50 to 25.0-mm (2 to 1-in.)	—	—	—	100	90 to 100	35 to 70	0 to 15	—	0 to 5	—	—	—	—	—	—
357	50 to 4.75-mm (2-in. to No. 4)	—	—	—	100	95 to 100	—	35 to 70	—	10 to 30	—	0 to 5	—	—	—	—
4	37.5 to 19.0-mm (1½ to ¾-in.)	—	—	—	—	100	90 to 100	20 to 55	0 to 15	—	0 to 5	—	—	—	—	—
467	37.5 to 4.75-mm (1½ to No. 4)	—	—	—	—	100	95 to 100	—	35 to 70	—	10 to 30	0 to 5	—	—	—	—
5	25.0 to 12.5-mm (1 to ½-in.)	—	—	—	—	—	100	90 to 100	20 to 55	0 to 10	0 to 5	—	—	—	—	—
56	25.0 to 9.5-mm (1 to ⅜-in.)	—	—	—	—	—	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	—	—	—	—
57	25.0 to 4.75-mm (1 to No. 4)	—	—	—	—	—	100	95 to 100	—	25 to 60	—	0 to 10	0 to 5	—	—	—
6	19.0 to 9.5-mm (¾ to ⅜-in.)	—	—	—	—	—	—	100	90 to 100	20 to 55	0 to 15	0 to 5	—	—	—	—
67	19.0 to 4.75-mm (¾ to No. 4)	—	—	—	—	—	—	100	90 to 100	—	20 to 55	0 to 10	0 to 5	—	—	—
68	19.0 to 2.36-mm (¾ to No. 8)	—	—	—	—	—	—	100	90 to 100	—	30 to 65	5 to 25	0 to 10	0 to 5	—	—
7	12.5 to 4.75-mm (½ to No. 4)	—	—	—	—	—	—	—	100	90 to 100	40 to 70	0 to 15	0 to 5	—	—	—
78	12.5 to 2.36-mm (½ to No. 8)	—	—	—	—	—	—	—	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	—	—
8	9.5 to 2.36-mm (⅜ to No. 8)	—	—	—	—	—	—	—	—	100	85 to 100	10 to 30	0 to 10	0 to 5	—	—
89	9.5 to 1.18-mm (⅜ to No. 16)	—	—	—	—	—	—	—	—	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	—
9	4.75 to 1.18-mm (No. 4 to No. 16)	—	—	—	—	—	—	—	—	—	100	85 to 100	10 to 40	0 to 10	0 to 5	—
10	4.75-mm (No. 4 to 0)[A]	—	—	—	—	—	—	—	—	—	100	85 to 100	—	—	—	10 to 30

[A] Screenings.

Att. E
4/4

RC
5MM

Exploration Location	Depth (Feet)	Soil Description	Percent Sodium	Percent Sulfate	Total Available Water Soluble sodium Sulfate (%)
B-5	10-15	Silty sand with gravel	0.07	0.13	0.20
B-8	19-20	Silty sand with gravel	0.07	0.06	0.08
B-101	5-10	Silty sand with gravel	0.17	0.06	0.08
B-102	0-5	Fill – Silty sand with gravel	0.17	0.03	0.05
B-106	0-5	Silty sand with gravel	0.15	0.08	0.12
B-106	29-30	Silty sand with gravel	0.15	0.06	0.08

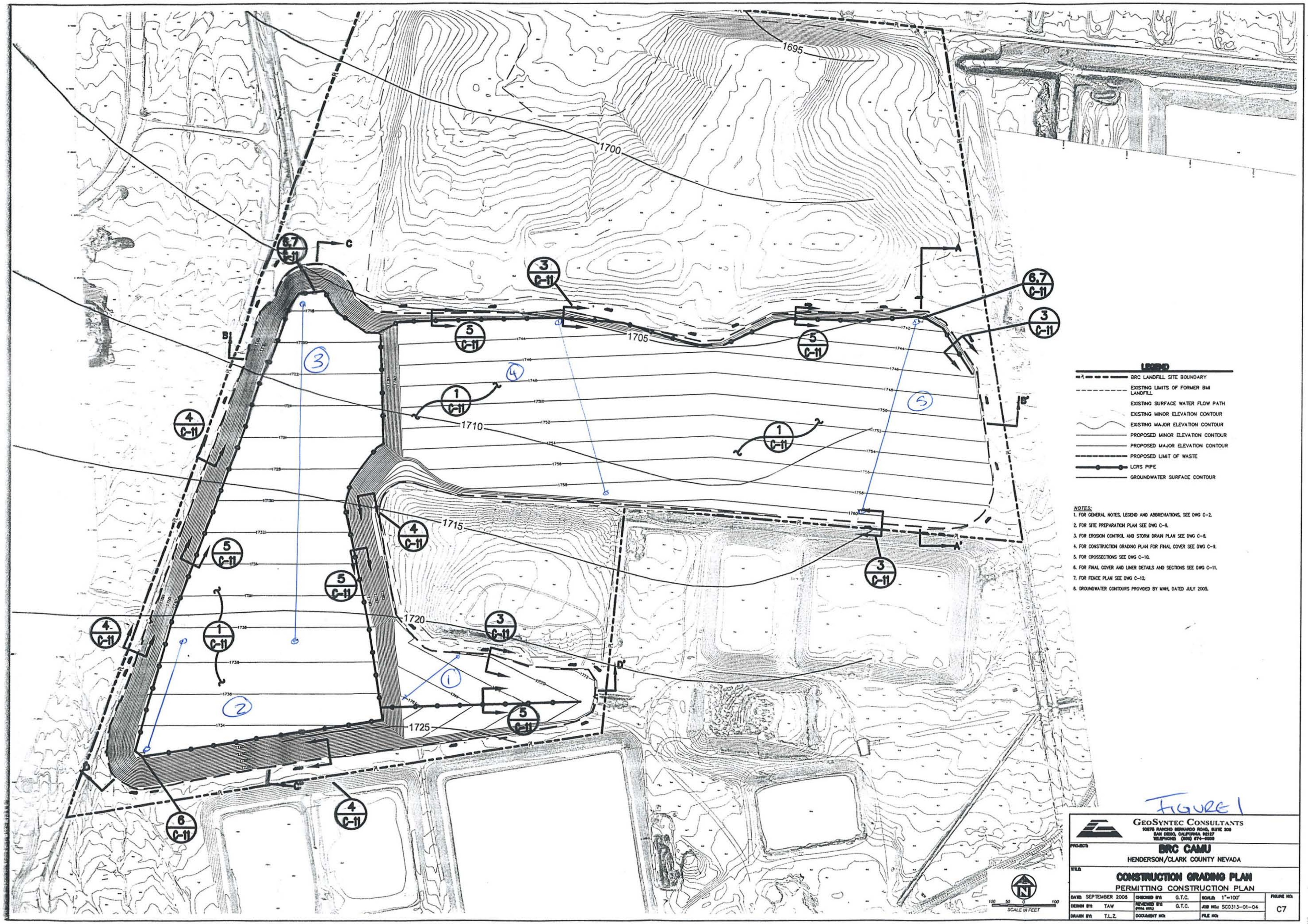
Permeability

Falling head permeability tests were conducted on remolded samples in general accordance with modified ASTM procedure D2434. The soil was compacted in a mold 4.6 inches long and 4.0 inches in diameter to 85 or 90 percent of maximum dry density and at optimum moisture content. A falling head was applied to the sample and the flow of water through the sample was monitored. The permeability was calculated after the flow rate had stabilized. The result of the falling head permeability test is presented in the following table:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-5	20-25	Silty sand with gravel	5.3×10^{-4}
B-12	10-15	Silty sand with gravel	4.0×10^{-4}
B-102	20-25	Silty sand with gravel	~~1.0×10^{-4}~~
B-105	20-25	Well graded sand with silt and gravel	1.2×10^{-3}

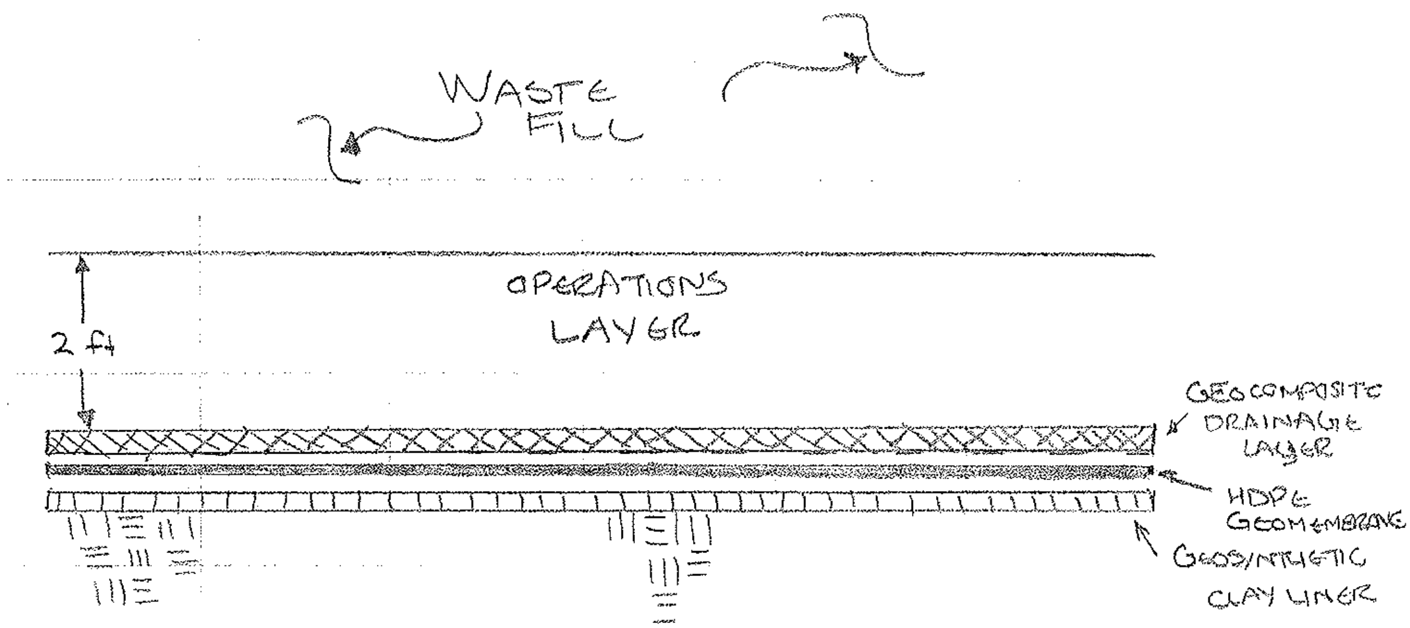
Flexible wall permeameter tests were performed on selected samples by AP Engineering and Testing, Inc according to ASTM D5084. With the exception of one sample (B-105), all tested samples were undisturbed ring samples. The samples were placed in a triaxial machine with a constant confining pressure at the approximate in-place effective stress pressures. Results were generally consistent with the fal-

Attachment G 1/1


1695
1700
1705
1710
1715
1720
1725
LEGEND
BRC LANDFILL SITE BOUNDARY
EXISTING LIMITS OF FORMER BMI LANDFILL
EXISTING SURFACE WATER FLOW PATH
EXISTING MINOR ELEVATION CONTOUR
EXISTING MAJOR ELEVATION CONTOUR
PROPOSED MINOR ELEVATION CONTOUR
PROPOSED MAJOR ELEVATION CONTOUR
PROPOSED LIMIT OF WASTE
LCRS PIPE
GROUNDWATER SURFACE CONTOUR
NOTES:
1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG C-2.
2. FOR SITE PREPARATION PLAN SEE DWG C-6.
3. FOR EROSION CONTROL AND STORM DRAIN PLAN SEE DWG C-8.
4. FOR CONSTRUCTION GRADING PLAN FOR FINAL COVER SEE DWG C-9.
5. FOR CROSSECTIONS SEE DWG C-10.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG C-11.
7. FOR FENCE PLAN SEE DWG C-12.
8. GROUNDWATER CONTOURS PROVIDED BY MWH, DATED JULY 2005.
Figure 1
GEOSYNTEC CONSULTANTS
PROJECT:
BRC CAMU
HENDERSON/CLARK COUNTY NEVADA
TITLE:
CONSTRUCTION GRADING PLAN
PERMITTING CONSTRUCTION PLAN
DATE: SEPTEMBER 2006
CHECKED BY: G.T.C.
SCALE: 1"=100'
FIGURE NO:
DESIGN BY: TAW
REVIEWED BY: G.T.C.
JOB NO: SC0313-01-04
C7
DRAWN BY: T.L.Z.
DOCUMENT NO:
FILE NO:
100 50 0 100
SCALE IN FEET

Written by: R. FLYNN Date: 21 / 08 / 06 (DD MM YY) Reviewed by: ______________ Date: ___ / ___ / ___ (DD MM YY)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No: 01 - 04



COMPOSITE LINER SYSTEM

NOT TO SCALE

FIGURE 2

Calculation Package D
Pipe Strength

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Pipe Strength Calculations

Computations By:
SIGNATURE — DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE 11/6/06
Keaton Botelho, Staff Engineer
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — DATE 11/3/06
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE — DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE — DATE 11/3/06
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 03 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

PIPE STRENGTH CALCULATIONS
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

A 4-in diameter HDPE leachate collection pipe and 18-in diameter side slope riser will be constructed at the BRC Corrective Action Management Unit (CAMU) in Henderson, Nevada. The objective of this calculation package is to evaluate the pipe strength performance and size the pipe wall thickness (i.e., determine the SDR).

SITE CONDITIONS

Crushed gravel will be backfilled around the 4-in and 18-in pipes. The maximum height of waste soil placed above the pipes will be 93 ft. Short term construction and long-term conditions will be evaluated.

LOADING CONDITIONS

The following two loading conditions were evaluated:

1. Short-Term Loading: Haul Truck (H-20)

   The ground pressure applied by the haul truck was estimated to be 100 psi as shown in Attachment A, p. 2. Based on the maximum axle load of 18 kips, the total load on the tire is 9 kips. Therefore, the footprint of the tire is approximately 0.63 ft^2 (see Attachment A). During construction, it was assumed that a minimum of 1 ft (0.3 m) of cover soil will separate haul trucks from the leachate collection pipes. Using Boussinesq's solution for a uniformly loaded square area, the assumed vertical pressure on the top of the pipe is 25 psi (see Attachment A).

2. Long-Term: Waste Soil Overburden Pressure

   Post construction, the maximum overburden soil will exist over the pipe at the southwest sump at a total depth of 93 ft (28.3 m). The unit weight is assumed to be 136 pcf. Additional loads from equipment is assumed to be negligible. The vertical pressure is calculated by:

$P = 93\text{ ft }(136\text{ pcf})/144\text{ in}^2/\text{ft}^2 = 87.8\text{ psi}.$

Therefore, the vertical pressure on the top of pipe in the long-term condition is 87.8 psi.

Since the long-term load (#2) is greater than short-term load (#1), only the long-term load (#2) was considered herein.

METHOD OF ANALYSES

Ring deflection, wall buckling, and wall crushing of the pipe were evaluated for the loading conditions. The Spangler's Modified Iowa Formula was used to calculate ring deflection. Recent literature indicates that the Modified Iowa Formula results in conservative values for pipe deformation (Brachman 1998). The actual deflection is likely lower due to the arching effects of soil via pipe deflection that are neglected in the Modified Iowa Formula. The manufacturer's design manual for Driscopipe (Philips 66, 1991) and Koerner's *Designing with Geosynthetics* (Koerner 1998) was used to evaluate wall buckling and wall crushing. The design criteria were based on the manufacturer's design manual for Driscopipe (Philips 66, 1991).

The method of analysis shown below solves directly for the SDR (Equation 2) of the pipe. Since the SDR is a dimensionless parameter, the diameter of the pipe does not need to be known.

ANALYSIS

Evaluating Variables

E' = 3,000 psi for crushed rock (Philips 66 1998)

P = Long-term loading. Soil Load = 93 ft of soil at 136 pcf = 12,648 psf = 87.8 psi (Assume haul truck loading is negligible)

E = 30,000 psi
Attachment D (Philips 66, 1991) shows that the modulus of elasticity (E) is approximately 30,000 psi for a 100 year design life.

Design by Wall Buckling

Wall buckling is generally the critical failure case for buried pipes. Naturally, this is a starting point for initial values for the SDR.

The SDR is defined as:

SDR $= D/t$ (Equation 1)

where D= outside diameter of pipe
t = minimum pipe wall thickness

Assume a factor of safety of 2.0 for buckling (Philips 66 1998). Therefore:

$FS = 2.0 = P_{cb}/P_T$ (Equation 2) (Attachment D)

where P_{cb} = critical buckling pressure at top of the pipe, and
P_T = total soil pressure at the top of the pipe = 87.8 psi

Solving Equation 2 for the critical buckling pressure, P_{cb}, yields:

$P_{cb} =$ 175.6 psi

The critical buckling pressure, P_{cb} is defined (Philips 66 1998) as:

$$P_{cb} = 0.8\sqrt{(E')(P_c)}$$ (Equation 3) (Attachment D)

where: P_c = critical collapse pressure
E' = soil modulus = 3,000 psi

The critical collapse pressure can be determined by the following equation:

$$P_c = \frac{2E(t/D)^3(D_{min}/D_{max})^3}{1-\mu^2}$$ (Equation 4)(Attachment D)

where: E = pipe modulus = 30,000 psi
D = outside diameter
t = thickness
$(D_{min}/D_{max}) = 0.95$
μ = Poisson's Ratio = 0.45 for HDPE pipe

Equation 4 can be reduced to the following equation:

$$P_c = \frac{2.15(E)}{SDR^3}$$ (Equation 5) (Attachment D)

Inserting Equation 5 and rearranging, Equation 3 becomes:

$$SDR^3 = 0.64(E')\frac{2.15E}{P_{cb}^{\ 2}}$$ (Equation 6)

By inserting the appropriate value determined above, the following result is obtained:

SDR = 15.9, use 13.5 (13.5 < 15.9, OK)

Check Wall Crushing

Wall crushing occurs when the compressive strength of the pipe is exceeded by the overburden soil pressure. For example, the compressive yield strength for HDPE pipe manufactured by Driscopipe is 1,500 psi (Philips 66, 1998). Assuming a factor of safety of 2.0, the required compressive strength of the pipe becomes:

$FS = 2.0 = 1500$ psi / S_A, therefore $S_A = 750$ psi (Equation 7)(Attachment D)

The hoop stress in the pipe is expressed as:

$S_A = (SDR-1)P_T/2$ (Equation 8) (Attachment D)

where: P_T = total external pressure = 87.8 psi

Rearranging, Equation 9 becomes:

$SDR = [2(S_A)/P_T] + 1$ (Equation 9)

Therefore, the design SDR is shown below:

SDR = 18.1 > 13.5 O.K.

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: ETC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

Check Ring Deflection

Ring deflection is the change in the vertical diameter of the pipe as the pipe/aggregate system deforms under the external vertical pressure. Ring deflection can be evaluated using Spangler's Modified Iowa formula and can be expressed as follows (Koerner 1998):

$$\Delta y = \frac{D_L K_b W_c}{(EI / r^3) + 0.061E'}$$ (Equation 10) (Attachment B)

Rearranging Equation 11 to express SDR and the percent pipe deflection directly:

$$\frac{\Delta y}{D} = \frac{D_L K_b P}{(2E) / (3(SDR - 1)^3) + 0.061E'}$$ (Equation 11)

where:

Δy	= pipe deflection or change in diameter, in.
D	= pipe diameter, in.
W_c	= prism soil load, lb/in of pipe
P	= prism soil load, psi = 87.8 psi
K	= bedding constant, typically 0.083 (Attachment E)
SDR	= standard dimension ratio (SDR) = 13.5
E	= modulus of elasticity of pipe, 30,000 psi
E'	= modulus of soil reaction, 3,000 psi
D_L	= deflection lag factor, 1.3 (range 1.0 to 1.5)

Solving for Equation 12 for the critical load yields:

$$\frac{\Delta y}{D} = \frac{(1.3)(0.083)(87.8)}{2(30{,}000)/3(13.5-1)^3 + 0.061(3000)} = 4.9\%$$

The maximum allowable ring deflection for SDR 13.5 pipe is approximately 5.1% at 1.5% strain (Attachment D). Therefore, the estimated ring deflection is acceptable.

SUMMARY AND CONCLUSIONS

Based upon these calculations for pipe ring deflection, wall buckling, and wall crushing, an HDPE pipe with an SDR of 13.5 satisfies the design criteria.

The pipe compressive strength and the design methods and criteria were based on the Driscopipe manufacturer's design manual (Philips 66, 1998). These parameters might be slightly different for the specific pipe used at BRC CAMU. However, the factors of safety against wall buckling and wall crushing calculated herein will account of uncertainty or differences in the pipe compressive strength or in the design methods and criteria.

CONSIDERATIONS FOR SPECIFICATIONS

In accordance with the above analyses, the following items should be included in the specifications for construction at the BRC CAMU:

- A pipe with a maximum SDR of 13.5.
- A minimum of 1 ft (0.3 m) of cover soil shall be placed over the pipes before a haul truck is allowed to drive over them.
- Compacted crushed rock shall be placed in the pipe trench.

REFERENCES

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Attachment A

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Attachment A

Brachman, R.W. I., "Laboratory Investigation of the Effect of Coarse Stone Backfill on the Performance of Leachate Collection Pipe", Geosythetics 99, Atlanta, Georgia, March 1998

Koerner R.B. (1998), "Designing with Geosynthetics", Fourth Edition, Prentice Hall, p.676

Attachment B

Philips 66 (1991), "Driscopipe Systems Design", 800-527-0062
Attachment C

Philips 66 (1998), "Driscopipe Systems Design", 800-527-0062
Attachment D

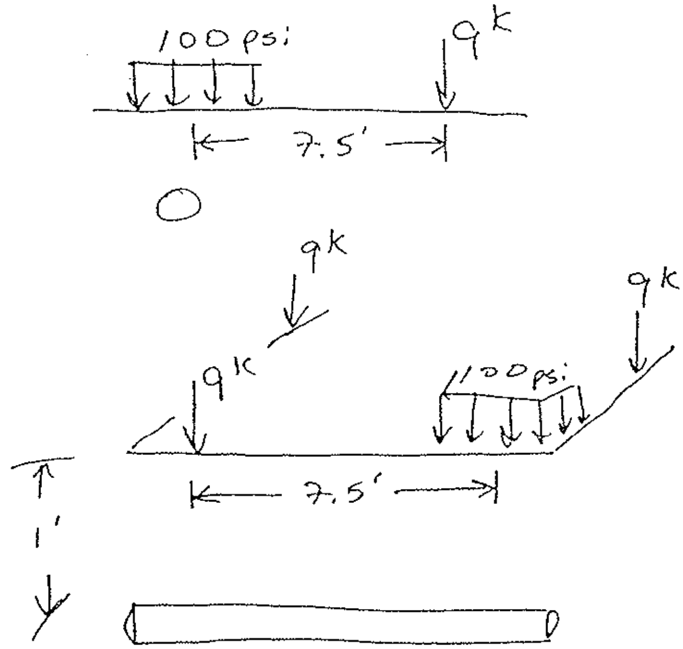
Written by: GLS Date: 00/01/26 Reviewed by: MLD GTC 5/10/05 Date: 00/07/03

Client: Parsons Project: BRC Landfill Project/Proposal No.: HL0389 Task No.: 01

Haul truck loading (H-20)

- Assumptions: (AASHTO 94)
  - → max single axle load of 18 kip
  - → tire pressure of 100 psi (assumed) 1ft wide tires
  - → truck is 8 1/2 wide

- Distribute Load
  - → assume distributed load of one wheel over pipe
  - ⇒ assume other loads are point loads



- Assume 1 ft cover soil

Attachment A
1/4

→ From Das (1994)

Table 7.1, Variation of I_1

Due to shallow depths and large distance to point loads, influence of point load wheels are negligable.

→ From Das (1994)

Evaluate stress increases due to wheel directly above pipe:

- contact pressure of 100 psi
- radius of footprint = $R = \sqrt{\frac{P}{\rho \pi}} = \left[\frac{9k}{(100)(144)\pi}\right]^{1/2}$

$R = 0.45$ ft

- assume flexible circular footprint (Das, 1994)

$$\Delta p = q\left\{1 - \frac{1}{[(R/z)^2+1]^{3/2}}\right\}$$

$z = 1$ ft, $R = 0.45$ ft, $q = 100$ psi

$$\Delta p = 100\left\{1 - \frac{1}{[(0.45/1)^2+1]^{3/2}}\right\}$$

$\Delta p = 24$ psi

add soil 1 ft = 136 psf/144 = 0.94 psi

$\boxed{\Delta p = 25 \text{ psi}}$ = total external pressure

Attachment A

2/4

/YY GTC (3) 5/10/05 MLD

vertical stress, dp, at point A caused by the load on the elemental area (which may be assumed to be a concentrated load) can be obtained from Eq. (7.11):

$$dp = \frac{3(qr\,dr\,d\alpha)}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}} \tag{7.24}$$

The increase in the stress at A caused by the entire loaded area can be found by integrating Eq. (7.24), or

$$\Delta p = \int dp = \int_{\alpha=0}^{\alpha=2\pi}\int_{r=0}^{r=R}\frac{3q}{2\pi}\frac{z^3 r}{(r^2+z^2)^{5/2}}\,dr\,d\alpha$$

So

$$\boxed{\Delta p = q\left\{1-\frac{1}{[(R/z)^2+1]^{3/2}}\right\}} \tag{7.25}$$

The variation of $\Delta p/q$ with z/R as obtained from Eq. (7.25) is given in Table 7.5. A plot of this is also shown in Figure 7.17. The value of Δp decreases rapidly with depth, and, at $z = 5R$, it is about 6% of q, which is the intensity of pressure at the ground surface.

TABLE 7.5 Variation of $\Delta p/q$ with z/R [Eq. (7.25)]

z/R	$\Delta p/q$
0	1
0.02	0.9999
0.05	0.9998
0.10	0.9990
0.2	0.9925
0.4	0.9488
0.5	0.9106
0.8	0.7562
1.0	0.6465
1.5	0.4240
2.0	0.2845
2.5	0.1996
3.0	0.1436
4.0	0.0869
5.0	0.0571

Attachment A
3/4

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TABLE 7.4 Values of $\Delta p/q$ [Eq. (7.23)]

	2z/B								
2x/B	0	0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0
−3	0	0.0003	0.0018	0.00054	0.0107	0.0170	0.0235	0.0347	0.0422
−2	0	0.0008	0.0053	0.0140	0.0249	0.0356	0.0448	0.0567	0.0616
−1	0	0.0041	0.0217	0.0447	0.0643	0.0777	0.0854	0.0894	0.0858
0	0	0.0748	0.1273	0.1528	0.1592	0.1553	0.1469	0.1273	0.1098
1	0.5	0.4797	0.4092	0.3341	0.2749	0.2309	0.1979	0.1735	0.1241
2	0.5	0.4220	0.3524	0.2952	0.2500	0.2148	0.1872	0.1476	0.1211
3	0	0.0152	0.0622	0.1010	0.1206	0.1268	0.1258	0.1154	0.1026
4	0	0.0019	0.0119	0.0285	0.0457	0.0596	0.0691	0.0775	0.0776
5	0	0.0005	0.0035	0.0097	0.0182	0.0274	0.0358	0.0482	0.0546

7.8 VERTICAL STRESS BELOW THE CENTER OF A UNIFORMLY LOADED CIRCULAR AREA

Using Boussinesq's solution for vertical stress Δp caused by a point load [Eq. (7.11)], one can also develop an expression for the vertical stress below the center of a uniformly loaded flexible circular area.

From Figure 7.16, let the intensity of pressure on the circular area of radius R be equal to q. The total load on the elemental area (shaded in the figure) $= qr\,dr\,d\alpha$. The

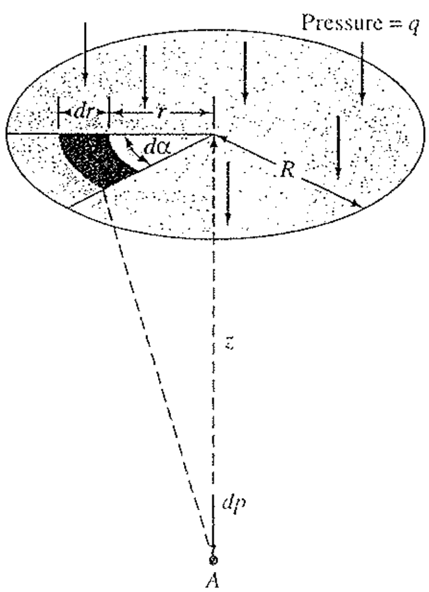

FIGURE 7.16 Vertical stress below the center of a uniformly loaded flexible circular area

Attachment A

FACTORS

Tire Pressure (psi)
180
170
148
166
204
150
175
175
177
127
184
174

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GTC (3) 5/10/05 MLD

TABLE 1.2. Typical Runway Lengths for Several Aircraft and Conditions[a]

Plane Type	Normal Max Temp. of Hottest Month (°F)	Elevation (ft)	Length[b] (ft)
Boeing 707-100	100	Sea level	10,500
Boeing 707-100	75	3000	11,500
Boeing 707-100	75	1000	10,500
Boeing 727	75	1000	7,800
Boeing 747	75	1000	10,500
Douglas DC 9	75	1000	8,000
Convair Cv 880	75	1000	10,500
BAC 1-11	75	1000	7,500

[a] Data from charts in FAA publication (ref. 4).
[b] The lengths shown in the table are relative and are for illustrative purposes only since the required lengths are dependent upon many factors, including effective grade of the runway, setting of the wing flaps, and takeoff weight. Each runway must be analyzed for its own particular conditions and the critical plane using the runway.

In the majority of the problems, circular tire imprints are assumed. Hence the radius of contact is as follows:

$$a = \sqrt{\frac{P}{p\pi}} \qquad A = \frac{P}{p} \tag{1.1}$$

where a = radius of contact
P = total load on the tire
p = tire pressure (assumed to be equal to contact pressure)

For some cases tire imprints as illustrated on Figure 1.8 are used. The relationship between pressure and the geometry of the imprint is as shown on the figure.

DESIGN FACTORS

Pavement design consists of two broad categories: (1) design of the paving mixtures, and (2) structural design of the pavement components. These two design steps must go hand in hand.

The structural design of pavements is basically different from the structural design of bridges and buildings in that the pavement structure lies exposed upon the ground surface and, hence, is greatly influenced by environmental factors. Likewise, a highway, for example, will cross many different soil deposits and it becomes necessary for the design engineer to select in a rational manner a design value representative of the area under question. The strength of soil is affected by many factors, including density, moisture content, soil texture, soil structure, rate of load application, and degree of confinement. In addition, soils

Attachment A
4/4

GTC (3) 5/10/05 MYD

7.2.2 Deflection Issues

An engineering approach to the quantification of deflection of buried pipelines has been developed by a sequential group of research faculty and students at Iowa State University. Beginning with Marston in the 1920s evaluating rigid conduits (the term used for shallow buried pipes), followed by Spangler in 1950–1970 evaluating flexible conduits, and into the present by Watkins, the group and their colleagues have "written the book" for this type of research [12]. Key issues in the development are the use of arching theory for gravitational force dissipation, the importance of subgrade stability, backfill type, and compaction conditions, and finally the flexibility of the pipe structure itself. Moser [13] presents the following equation, summarizing the Iowa State group's effort for the deflection behavior of flexible (in our case plastic) pipe.

$$\Delta X = \frac{D_L K_b W_c}{(\mathrm{EI}/r^3) + (0.061E')} \cong y \tag{7.17}$$

where

ΔX = horizontal increase in diameter (m), 1.5 (conservative)
y = vertical deflection (m),
D_L = deflection lag factor, which varies from 1.0 to 1.5 (dimensionless),
K_b = bedding constant, which varies from 0.83 to 0.110 (dimensionless), 0.1
W_c = Marston's prism load per unit length of pipe (kN/m) (note that arching is not taken into account in this formula),
E = modulus of elasticity of the pipe material (kPa),
I = moment of inertia of the pipe wall per unit length (m^3),
EI = bedding stiffness of the pipe ring per unit length (kN-m),
r = mean radius of the pipe (m), and
E' = modulus of soil reaction (kPa).

The last term (E') has been the subject of intense discussion and research. Howard [14] of the U.S. Bureau of Reclamation has recommended the values given in Table 7.9, which have relatively wide acceptance.

Eq. (7.17) can also be cast in terms of the laboratory plate loading test with the following result. The equation assumes a bedding constant $K_b = 0.2$ and uses the ring stiffness constant (RSC).

$$\frac{y}{D} = \frac{P(0.1L)}{[14.9(\mathrm{RSC})/D + 0.061E']} \tag{7.18}$$

where

y = vertical deflection (m),
D = inside pipe diameter (m),
P = load on pipe (kPa),
L = deflection lag factor (usually 1.0 to 1.5),
RSC = ring stiffness constant (kN/m), and
E' = modulus of soil reaction (kPa).

Attachment B

1/1

GTC (3) 5/10/05 MWD

/33

Where: S_A = Actual compressive stress, psi
SDR = Standard Dimension Ratio
P_T = External Pressure, psi

Safety Factor = 1500 psi ÷ S_A where 1500 psi is the Compressive Yield Strength of Driscopipe.

Design by Wall Buckling: Local wall buckling is a longitudinal wrinkling of the pipe wall. Tests of *non-pressurized* Driscopipe show that buckling and collapse do not occur when the soil envelope is in full contact with the pipe and is compacted to a dense state. However, it can be forced to occur over the long term in *non-pressurized* pipe if the total external soil pressure, P_t, is allowed to exceed the pipe-soil system's critical buckling pressure, P_{cb}. If $P_t > P_{cb}$, gradual collapse may occur over the long term. A calculated, conservative value for the critical buckling pressure may be obtained by the following approximate formula. All pipe diameters with the same SDR in the same burial situation have the same critical collapse and critical buckling endurance

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

Where:

P_t = Total vertical soil pressure at the top of the pipe, psi

P_{cb} = Critical buckling soil pressure at the top of the pipe, psi

E' = Soil modulus in psi calculated as the ratio of the vertical soil pressure to vertical soil strain at a specified density

P_C = Hydrostatic, critical-collapse differential pressure, psi

$$P_C = \frac{2E\,(t/D)^3\,(D_{MIN}/D_{MAX})^3}{(1-\mu^2)}$$

$$P_c = \frac{2.32\,E}{(SDR)^3}$$

Where: $(D_{MIN}/D_{MAX}) = .95$
μ = Poission's Ratio
μ = .45 for Driscopipe
E = stress and time dependent tensile modulus of elasticity, psi

In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Design by Wall Buckling Guidelines
Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_t < P_{cb}$

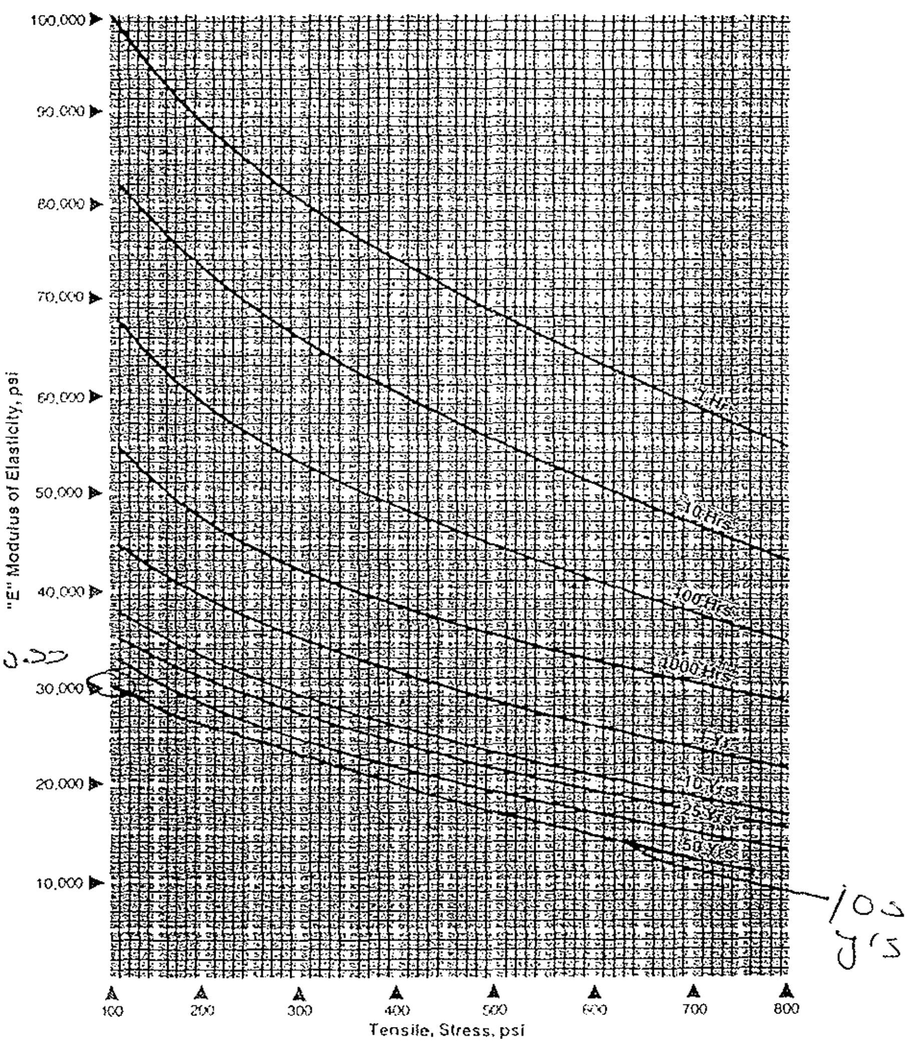
1. Calculate or estimate the total soil pressure, P_t, at the top of the pipe
2. Calculate the stress "S_A" in the pipe wall according to the formula:

$$S_A = \frac{(SDR - 1)\,P_t}{2}$$

3. Based upon the stress "S_A" and the estimated time duration of non-pressurization, use Chart 25 to find the value of the pipe's modulus of elasticity, E, in psi

Chart 25

Time Dependent Modulus of Elasticity for Polyethylene Pipe vs. Stress Intensity (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

Attachment C
1/1

GTC (3) 5/10/05 MSD

TABLE 13: VALUE OF E' BASED ON SOIL TYPE (ASTM D2321) AND DEGREE OF COMPACTION

E' (psi) for Degree of Compaction (Standard Proctor Density, %)

	Soil Type of Initial Backfill Material	Loose 70%	Slight (70 - 85%)	Moderate (85-95%)	High > 95%
I	**Manufactured angular, granular materials** (Crushed Stone, or rock, broken coral, cinders, etc.)	1,000	3,000	3,000	3,000
II	Coarse grained soils with little or no fines	Not Recommended	1,000	2,000	3,000
III	Coarse grained soils with fines	Not Recommended	Not Recommended	1,000	2,000
IV	Fine Grained Soils	Not Recommended	Not Recommended	Not Recommended	Not Recommended
V	Organic Soils (Peat, Muck, Clay, etc.)	Not Recommended	Not Recommended	Not Recommended	Not Recommended

Note: This summary of ASTM D 2321 is provided for the design engineer's convenience. This specification should be reviewed in detail before specifying burial conditions.

MINIMUM COVER There are no firm rules regarding minimum burial depth. The variables change for each installation, and the designer should check each design for wall crushing, wall buckling, and ring deflection. However, the following guidelines may be helpful.

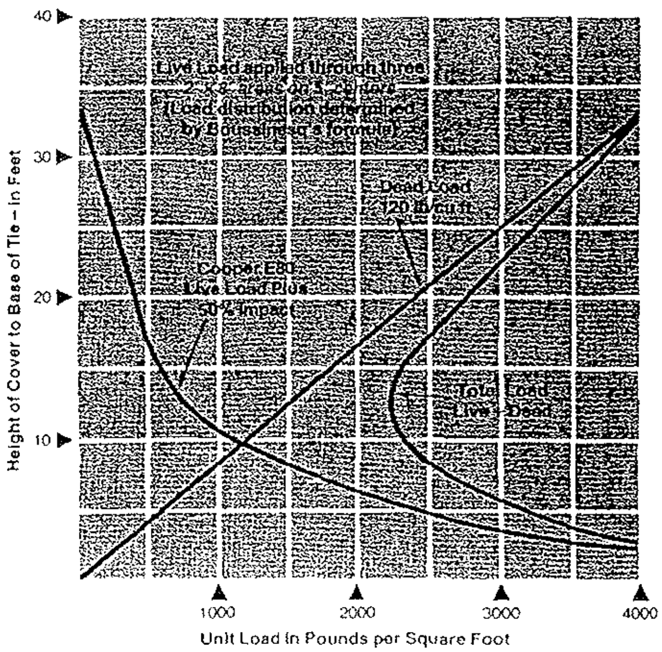
- Consider a burial depth below the local frost line.
- Where there will be no overland traffic, the designer may wish to consider a cover of 18" or one diameter, whichever is greater.
- Where truck traffic may be expected, the designer may wish to consider a burial depth of 36" or one diameter, whichever is greater.
- Where heavy off-the-road truck or locomotive traffic is expected, the designer may wish to consider a minimum cover of 5 feet or more.

CALCULATION OF TOTAL SOIL PRESSURE BY COMPONENTS Proper design of the polyethylene "pipe-soil" system balances the response of the pipe and surrounding soil against the total external soil pressure. Burial design by wall crushing, wall buckling, and ring deflection require the calculation of the total soil pressure, P_T, at the top of the pipe. There are many sources of soil pressure above the pipe. It is helpful to examine the total soil pressure as the sum of its components.

Attachment D
1/6

GTC (3) 5/10/05 MLD

FIGURE 6: COOPER E-80 LIVE LOADING



Note: Cooper E-80 live load assumes 80,000 pounds applied to three 2' x 8' areas on 5' centers such as might be encountered through live loading from a locomotive with three 80,000 pound axle loads.

Source: American Iron and Steel Institute, Washington, DC

<u>APPARENT EXTERNAL PRESSURE DUE TO INTERNAL VACUUM, P_I</u> Vacuum generates a compressive hoop stress in the wall of a pipe and acts to collapse the pipeline. Under vacuum conditions, the value of P_I is positive. P_I is added to the other two external pressure components, P_S and P_L, to obtain the total external pressure, P_T, acting on the pipe. An internal vacuum generates pressure equal to the absolute value of the vacuum. The maximum apparent external pressure due to a vacuum inside the pipe is 14.7 psi (2,117 psf).

BURIAL DESIGN GUIDELINES

The design engineer must select the proper pipe DR and specify the backfill conditions to obtain the desired performance of the "pipe-soil" system.

<u>DESIGN BY WALL CRUSHING</u> Wall crushing occurs when external vertical pressure causes the compressive stress in the pipe wall to exceed the long-term compressive strength of the pipe material. To design for wall crushing, the following check should be made:

$$S_A = \frac{(SDR - 1)}{2} P_T$$

Where:
- S_A = Actual compressive stress, psi
- SDR = Standard Dimension Ratio
- P_T = Total external pressure on the top of the pipe, psi

Safety Factor = 1500 psi /S_A (where 1500 psi is the compressive yield strength of Driscopipe HDPE pipe)

<u>DESIGN BY WALL BUCKLING</u> Local wall buckling is a longitudinal wrinkling of the pipe wall. Buckling can occur over the long term in non-pressurized pipe if the total external soil pressure, P_T, exceeds the pipe-soil system's critical buckling pressure, Pcb . Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_T < P_{cb}$. All pipe diameters with the same DR in the same burial situation have the same critical collapse and critical buckling endurance.

Attachment D
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CTC ③ 5/10/05 MLD

1. Calculate or estimate the total soil pressure, P_T, at the top of the pipe.

2. Calculate the stress, S_a, in the pipe wall:

$$S_A = \frac{(SDR-1)}{2} P_T$$

3. Based upon the stress, S_a, and the estimated time duration of non-pressurization, find the value of the pipe's modulus of elasticity, E, in psi (approximate value for E is 35,000 psi).

4. Calculate the pipes hydrostatic, critical-collapse differential pressure, P_c

$$P_C = \frac{2E(t/D)^3 (D_{MIN}/D_{MAX})^3}{(1-\mu^2)} \quad \text{or} \quad P_c = \frac{2.32(E)}{SDR^3}$$

$$P_c = \frac{2.15(E)}{SDR^3}$$

Where: $(D_{MIN}/D_{MAX}) = 0.95$
μ = Poission's Ratio = 0.45 for polyethylene pipe
E = stress and time dependent tensile modulus of elasticity, psi
E = 35,000 psi (approximate)
D = Outside Diameter, in.
t = thickness, in.

5. Calculate the soil modulus, E', by plotting the total external soil pressure, P_T, against a specified soil density to derive the soil strain as shown in the example problem below Figure 7.

6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{cb} = 0.8\sqrt{(E')(P_c)}$$

Where: P_{cb} = Critical buckling soil pressure at the top of the pipe, psi
E' = Soil Modulus, psi
P_c = Hydrostatic critical-collapse differential pressure, psi

7. Calculate the Safety Factor: SF = P_{cb} / P_T.

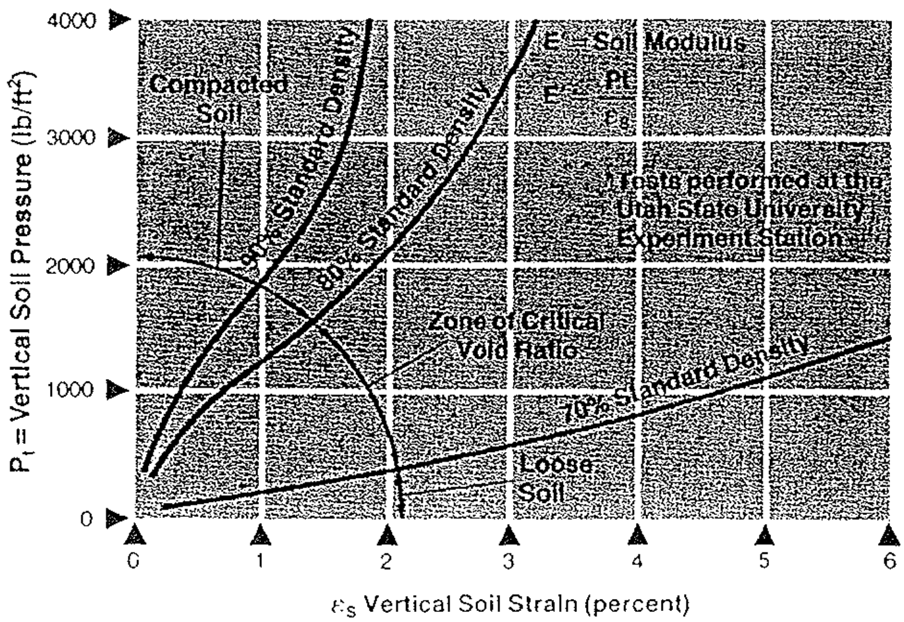
8. The above procedures can be reversed to calculate the minimum pipe DR required for a given soil pressure and an estimated soil density.

In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Attachment D
3/4

LTC ⑤ 5/10/05 MLD

FIGURE 7: PLOT OF VERTICAL STRESS-STRAIN DATA FOR TYPCIAL TRENCH BACKFILL (EXCEPT CLAY) FROM ACTUAL TESTS



Example:
Find: E' @ 2000 psf and 80% density
Formula: $E' = PT/\varepsilon_s$
Calculations: E' = 2000 psf / (0 018 * 144) = 771 psi

Note: The curves shown on this chart are sample curves for a granular soil. If other types of soil are used for backfill such as clay or clay loam, curves should be developed from laboratory test data for the material used. Soil pressures greater than 4000 psf may be extrapolated with the slope of the curve or curves can be generated by testing at higher soil pressures. Probable error of curves is about half the distance between adjacent lines.

Design by Ring Deflection Ring deflection, by definition, is the ratio of the vertical change in diameter to the pipe's original diameter. It is often expressed as a percentage.

Driscopipe HDPE pipe is designed to be "flexible". This assumes the pipe will deflect the same as the vertical compression of the soil around it. Design by ring deflection matches the ability of the pipe to accommodate, without structural distress, the vertical compression of the surrounding soil. Design by ring deflection calculates the vertical soil strain and compares it to the allowable ring deflection of the pipe.

Attachment D
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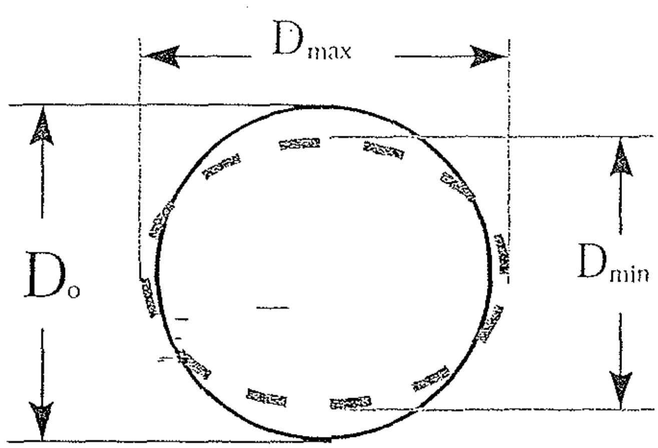
GTC ⑧ 5/10/05 MLD

TABLE 15: ALLOWABLE RING DEFLECTION OF DRISCOPIPE® POLYETHYLENE PIPE BASED UPON DR

DR	Allowable Ring Deflection
32.5	8.1%
26	6.5%
21	5.2%
19	4.7%
17	4.2%
15.5	3.9%
* 13.5	3.4% *
11	2.7%

The allowable ring deflection of polyethylene pipe is limited to create no more than 1 to 1.5% tangential strain in the outer surface of the pipe wall. As the wall of a pipe becomes thicker (a "lower" DR value), the distance from the neutral axis to the outer surface increases. As a result, less deflection is required to create the allowable tangential strain. Deflection of the pipe-soil system is controlled by proper specification of the backfill compaction. *

FIGURE 8: CALCULATING RING DEFLECTION



$$\%\,Ring Deflection = \left(1 - \frac{D_{min}}{D_o}\right) \times 100\%$$

The percentage ring deflection based upon strain for a given DR pipe can be calculated as follows:

$$\frac{\Delta Y}{D} = (0.25)(\varepsilon)(SDR)$$ *

Attachment D
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61C ⑧ 5/10/05 LD

Where: ΔY = Vertical deflection, in.
D = Pipe OD, in.
ε = Tangential strain in the surface of the pipe ring, in./in.
SDR = Standard Dimension Ratio

Driscopipe recommends limiting tangential surface strain to 0.01. This value is based upon the following criteria:

- Most of the deflection of a flexible pipe occurs within a few days after final backfill is completed. Development of a soil arch over the pipe relieves the pipe of much of the vertical soil load by the arching action of the soil envelope and by the development of soil restraint at the sides of the pipe.
- An allowable strain value of 0.01 will allow for reasonable additional deflection due to disturbance of the backfill by earthquake, fluctuations of the water table, etc.
- An allowable design strain value of 0.01 allows for the normal deviation of temperature encountered during installation.

In summary, a soil density can be specified for the bedding and initial backfill so that total soil pressure at the top of the pipe, P_T, will not cause a given DR pipe to exceed its maximum allowable ring deflection.

Attachment D
6/6

Table 5-6
Support Spacing for Schedule 80 CPVC Pipe

Nominal Pipe Size, mm (in)	Maximum Support Spacing, m (ft) at Various Temperatures					
	23• C (73• F)	38• C (100• F)	49• C (120• F)	60• C (140• F)	71• C (160• F)	82• C (180• F)
25 (1)	1.83 (6.0)	1.83 (6.0)	1.68 (5.5)	1.52 (5.0)	1.07 (3.5)	0.91 (3.0)
40 (1.5)	2.13 (7.0)	1.98 (6.5)	1.83 (6.0)	1.68 (5.5)	1.07 (3.5)	0.91 (3.0)
50 (2)	2.13 (7.0)	2.13 (7.0)	1.98 (6.5)	1.83 (6.0)	1.22 (4.0)	1.07 (3.5)
80 (3)	2.44 (8.0)	2.44 (8.0)	2.29 (7.5)	2.13 (7.0)	1.37 (4.5)	1.22 (4.0)
100 (4)	2 59 (8.5)	2 59 (8.5)	2 59 (8.5)	2.29 (7.5)	1.52 (5.0)	1.37 (4.5)
150 (6)	3.05 (10.0)	2.90 (9.5)	2.74 (9.0)	2.44 (8.0)	1.68 (5.5)	1.52 (5.0)
200 (8)	3.35 (11.0)	3.20 (10.5)	3.05 (10.0)	2.74 (9.0)	1.83 (6.0)	1.68 (5.5)
250 (10)	3.51 (11.5)	3.35 (11.0)	3.20 (10.5)	2.90 (9.5)	1.98 (6.5)	1.83 (6.0)
300 (12)	3.81 (12.5)	3.66 (12.0)	3.51 (11.5)	3.20 (10.5)	2.29 (7.5)	1.98 (6.5)

Note: The above spacing values are based on test data developed by the manufacturer for the specific product and continuous spans. The piping is insulated and is full of liquid that has a specific gravity of 1.0.
Source: Harvel Plastics, Product Bulletin 112/401 (rev. 10/1/95), p. 63.

Table 5-7
Bedding Factor, K_x

Type of Installation	K_x
Shaped bottom with tamped backfill material placed at the sides of the pipe, 95% Proctor density or greater	0.083
Compacted coarse-grained bedding and backfill material placed at the side of the pipe, 70-100% relative density	0.083
Shaped bottom, moderately compacted backfill material placed at the sides of the pipe, 85-95% Proctor density	0.103
Coarse-grained bedding, lightly compacted backfill material placed at the sides of the pipe, 40-70% relative density	0.103
Flat bottom, loose material placed at the sides of the pipe (not recommended); <35% Proctor density, <40% relative density	0.110

Source: Reprinted from Schweitzer, Corrosion-Resistant Piping Systems, p. 49, by courtesy of Marcel Dekker, Inc.

Attachment E

Calculation Package E
Veneer Stability

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 02

Title of Computations: Veneer Stability of Geosynthetic-Lined Slopes

Computations By:
(GS) SIGNATURE
3 May 2000 DATE
Geoffrey L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE
7/3/00 DATE
GREGORY T CORCORAN / PROJ. ENG.
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE
2 May 2000 DATE
Matt Darr Staff Engr.
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE
3 May 2000 DATE
Geoffrey L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE
10 July 2000 DATE
Bradford H. Cooley / Project Manager
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 00 / 01 / 26 (YY MM DD) Reviewed by: MLD GTC Date: 00 / 07 / 03 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

SLOPE STABILITY EVALUATION
VENEER STABILITY OF GEOSYNTHETIC-SOIL LINED SIDESLOPES

OBJECTIVE

To evaluate the tension developed within the geosynthetic-soil layered sideslope of the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada.

METHOD OF ANALYSIS

The stability analysis of the geosynthetic-soil layered systems was carried out using the approach outlined by McKelvey (1994). McKelvey (1994) calculates the tension of a geosynthetic component along a geosynthetic-soil layered sideslope and compares it to the allowable geosynthetic strength to evaluate the overall factor of safety against tension. McKelvey (1994) allows for the consideration of tapered slopes and equipment loads. The calculations herein were performed by a spreadsheet developed by McKelvey that has been verified numerous times, so verification will not be presented herein.

SIDESLOPE LINER SYSTEM

The sideslope liner system (Attachment A) consists of, from top to bottom:

- 2 ft (min) 2.5H:1V tapered operations layer material (10 ft max height),
- a drainage geocomposite;
- ~~a non-woven cushion geotextile;~~ GLS
- a 60-mil (1.5-mm) thick textured high density polyethylene (HDPE) geomembrane;
- a geosynthetic clay liner (GCL); and
- subgrade.

The sideslope inclination is 2.1H:1.0V. The maximum height of the side slope is 30 vertical feet.

Written by: Geoff L. Smith GLS Date: 00 / 01 / 26 (YY MM DD) Reviewed by: MLD GTC Date: 00 / 07 / 03 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

MATERIAL SHEAR STRENGTHS

Operations Layer Material and Drainage Aggregate:

The soil materials to be used overlying the side slope liner system will be native materials such as silty sands (SM) for the operations layer. The operations layer material is characterized by an internal angle of friction of 31 degrees (Attachment B). Converse reported a maximum dry density of 132 pcf and a optimum water content of 8.7 percent for materials at the site. Therefore, assuming 95% relative compaction, the dry density in the field is approximately 125 pcf. Adding the weight of water, the unit weight is approximately 136 pcf (Attachment B).

For this analysis, a shear strength of ~~31~~ 30 GLS degrees and a unit weight of 13~~2~~6 GLS pcf will be used for the analyses performed herein.

Geosynthetic interface:

The following values for the interface friction between geosynthetic and soil components of the liner system will be used in this calculation:

Operations Layer to Geocomposite (NWGT side)	25 (Attachment D)
~~Geocomposite (GN side) to Cushion Geotextile~~	~~30 (Attachment D)~~ GLS
Cushion Geotextile to Textured HDPE	28 (Attachment E)
Textured HDPE to Dry GCL	18 (Attachment F)
Dry GCL to Subgrade	28 (Attachment F)

The critical loading for the side slope liner system occurs during construction when dozers are placing the ~~drainage aggregate and~~ operations layer materials up the slope. Global stability analyses have been performed to determine the long-term stability of the liner system. Therefore, the GCL component of the liner system was selected to be dry, since hydration will likely occur only after the construction phase is complete, and the normal stress used in obtaining the interface friction will be low (i.e. less than 5 psi).

The minimum shear strength along the geosynthetic interfaces of the sideslope liner system was assumed to occur at the interface of the textured HDPE geomembrane/GCL. A shear strength value of 18 degrees will be used in the tension analysis for the textured HDPE geomembrane/GCL interface as suggested in Attachment F.

Written by: Geoff L. Smith Date: 00 / 01 / 26 Reviewed by: MLD GTC Date: 00/07/03
YY MM DD YY MM DD

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

DESIGN CRITERION

The geosynthetic-soil lined sideslopes are considered interim slopes because they will eventually be buttressed by placement of waste. Due to the limited duration during which the geosynthetic-soil lined slope will exist before it is buttressed by waste, seismic stability analyses were not conducted.

RESULTS AND CONCLUSIONS

The computer output for the analyses are presented in pages 4-~~7~~5. The results suggest the following:

Live Load Case (assuming placement of operations layer with a bulldozer no larger than Caterpillar D6H-LGP dozer in terms of operating weight and ground pressure), the side slope liner system will not be placed into tension.

REFERENCES

Converse (1999), "Preliminary Geotechnical and Geologic Report - Industrial Non-Hazardous Disposal Facility (Converse 1999)", prepared for Basic Management, Inc., October 1999.

Hunt R.E. (1986), *Geotechnical Engineering Techniques and Practices*, McGraw-Hill Book Company.

Koerner, R.M. (1990), *Designing with Geosynthetics*, Third Edition, Prentice Hall, New Jersey.

Calculated Tension for : BRC CAMU - Side Slope, Final Thickness -Construction loads

Project variables

Cover soil unit weight :	136.00 pcf
Cover soil thickness :	2.00 feet
Cover soil internal angle of friction :	30.00 degrees
Angle of slope (ie nH:1V) :	2.10
Angle of cover (ie nH:1V) :	2.50
Slope height :	10.00 feet
Equipment load :	D6H

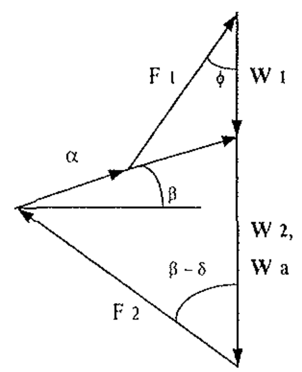
Geosynthetic variables

Minimum friction angle :	18.00 degrees
Load bearing product name :	60 MIL HDPE
Yield tensile strength :	126.00 ppi
Creep factor of safety :	5.00
installation damage factor of safety:	1.20
chemical degradation factor of safety:	1.50
biological degradation factor of safety:	1.00
Break strain factor of safety:	1.00

ANALYSIS OUTPUT

Slope angle:	25.46 degrees
Ramp angle:	21.80 degrees
Resisting force (W1) :	2,390 ppf
Driving force (W2) :	14,776 ppf
Equipment and special loads (Wa) :	6,477 ppf
estimated tension :	**0 ppf**
Geomembrane allowable strength :	168 ppf
Overall Factor of Safety for tension :	**INFINITE OK**

Free body diagram



Notes

1. 60 MIL HDPE
   assumed to carry load.

2. Minimum interface at
GM/GCL

Taper Distances

x' (ft) :	4.00
y' (ft) :	1.60

GeoSyn
Dat
Written By:
Client:

GeoSyntec Consultants

Written By: Geoff Smith GS Date: 26-Apr-00 Reviewed by: MLD LTC Date: 7/3/00

Client: Parsons Project: BRC CAMU Project No.:

Equipment load

PLACEMENT EQUIPMENT:	D6H
operating weight (lbs):	38,605
track width (in):	22.00
ground contact (in^2)	4,564
ground pressure (psi):	8.46
depth (ft):	2.74
ANALYSIS OUTPUT	
Surface stress (q) in psf :	1218.04
Track length in inches :	103.73
Length to width ratio (L/B) :	4.72
Length to depth ratio (N) :	1.58
Width to depth ratio (M) :	0.33
Influence coefficient (Io) :	0.0946
Stress at geosynthetic surface (s) in psf :	460.91
Equivalent track width due to attenuation (B') in feet :	2.98
Driving force of equipment (Wa) in ppf :	6,477
Special load (ppf) :	0

References

1. McKelvey, J.A. and Deutsch, W.L., (1991) "The Effect of Equipment Loading and Tapered Cover Soil Layers on Geosynthetic Lined Landfill Slopes", Proc. 14th Ann. Madison Waste Conference, Madison, WI:UWM, pp 395 - ‹

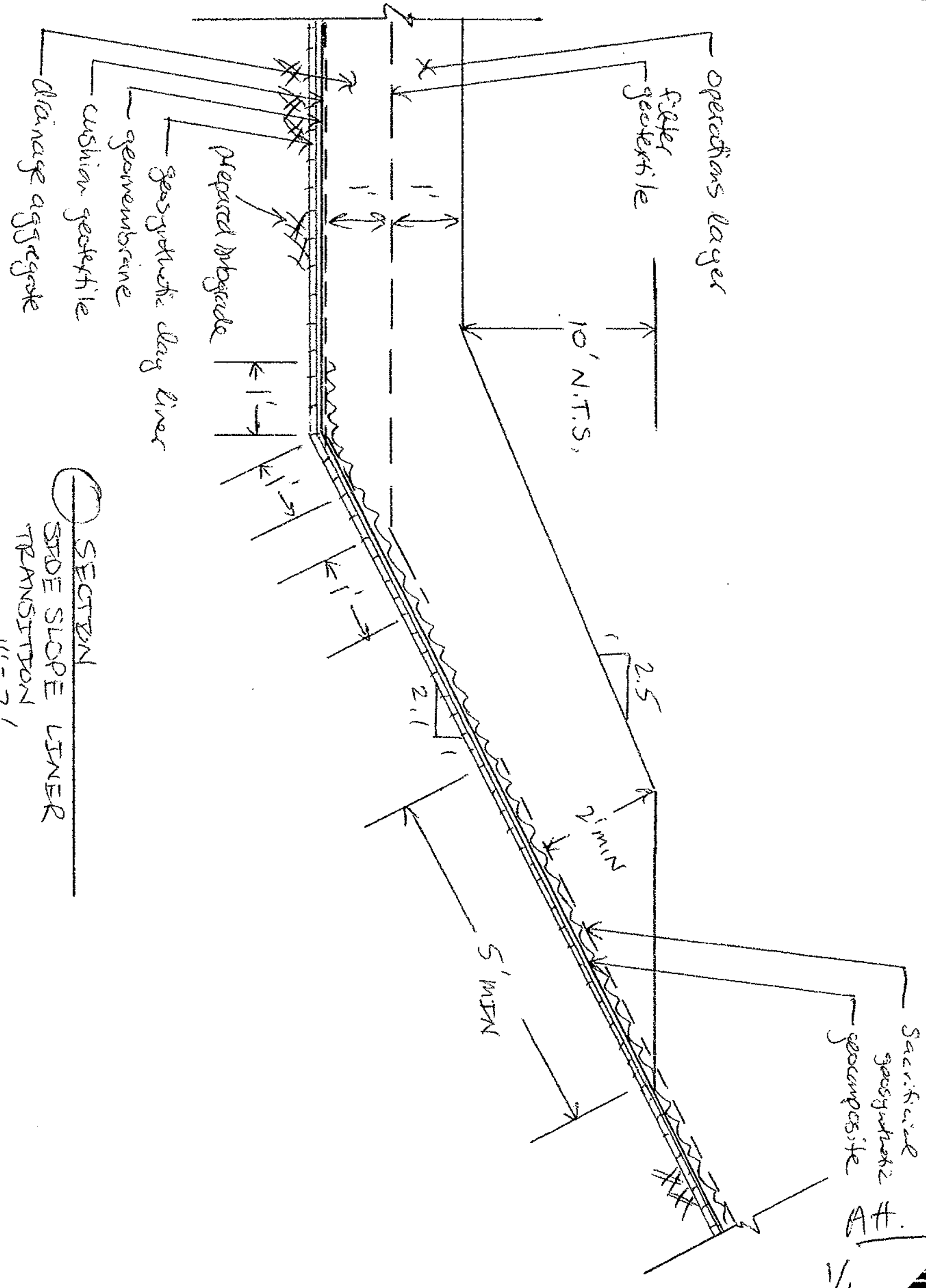
2. McKelvey, J.A. (1994) "Consideration of equipment loadings in geosynthetic lined slope designs", Proc. 8th Inte Conf. of the International Association for Computer Methods and Advances in Geomechanics, Morgantown, WV:Balkema, pp. 1371-1377.

GEOSYNTEC CONSULTANTS

Page ______ of ______

Written by: Geoff Smith Date: 00/04/18 (YY/MM/DD) Reviewed by: MW GTC Date: 00/07/03 (YY/MM/DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0398 Task No.: 02



Att. A

1/1

GTC

Direct Shear Strength

A progressive direct shear test was performed on selected undisturbed samples using a constant strain rate direct shear machine in general accordance with ASTM D3080. The test specimen was trimmed and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until maximum shear strength was developed. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. Another normal load was then applied, and the specimen was sheared a second time. This process was repeated for three different normal loads. Results of the direct shear test are presented on Figures A-62 through A-69 and in the following table:

Exploration Location	Depth (Feet)	Soil Description	Angle of Internal Friction (deg)	Coulomb Cohesion (ksf)
B-4	14-14.5	Silty sand with gravel	31	0.7
B-5	14-15	Silty sand with gravel	43	0.3
B-10	54-54.5	Sandy lean clay	26	0.85
B-12	14-15	Silty sand with gravel	40	0.3
B-101	39-40	Sandy lean clay	26	0.9
B-102	20-25	Silty sand with gravel	37	0.2
B-103	49-50	Sandy lean clay	37	1.0
B-104	10-15	Silty sand with gravel	43	0.1

Chemical Analysis

Chemical tests were performed on a representative soil samples to investigate the potential for soil corrosivity and chemical heave. Atlas Chemical Testing Laboratories, Inc. in Las Vegas performed the chemical analysis for water-soluble sulfates and sodium in general accordance with ASTM D516. The results of the chemical tests are presented on Drawing No. A-70.

Converse Consultants, 1999 MLD

Attachment B, 1/2

shown on Drawing Nos. A-49 through A-56, entitled *Consolidation Test* and are summarized on the following table:

Exploration Location	Depth (feet)	Soil Description	Dry Unit Weight, pcf	Moisture Content, %	Hydrocollapse (percent)*
B-1	29-30	Silty sand with gravel	105	6	3.2
B-8	39-40	Sandy lean clay	57.4	64	0.4
B-8	49-50	Sandy lean clay	69.5	51.1	-0.6
B-10	54-54.5	Sandy lean clay	60.7	67.7	-0.6
B-101	39-40	Sandy lean clay	65.8	45	-0.2
B-101	59-60	Sandy lean clay	73.2	38.3	-0.6
B-102	49-50	Sandy lean clay	67.3	48.7	-0.5
B-105	34-35	Well graded sand with silt and gravel	101	5	0.1

NA: Not available

* A negative sign indicates swell occurred upon inundation with water instead of collapse.

Laboratory Maximum Density

Laboratory maximum density tests were performed on selected samples of the granular soils. The purpose of the test was to define the compaction characteristics of these soils, and to aid in estimating soil shrinkage. The laboratory maximum density test was performed in general accordance with the ASTM D1557 test method. This test procedure uses 25 blow of a 10-pound hammer falling a height of 18 inches on each of five layers of soil in a 1/30 or 1/13 cubic foot cylinder. The test results are presented on Drawing Nos. A-57 through A-61 and in the following table:

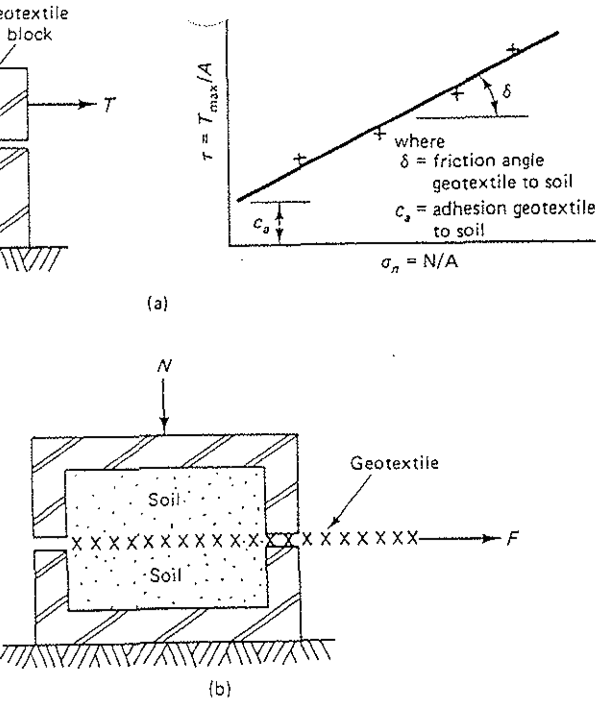
Exploration Location	Depth (Feet)	Soil Description	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (percent) of dry weight)
B-1	20-25	Silty sand with gravel	129.4	8.2
B-5	20-25	Silty sand with gravel	132.1	8.2
B-12	10-15	Silty sand with gravel	129.7	7.9
B-101	5-10	Silty sand with gravel	130.6	8.7
B-105	20-25	Well graded sand with silt and gravel	131.8	7.5

MLD Attachment 3 2/2

TABLE 3.31
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

Hunt, 1986
Attachment C 1/1
MLD



nematic diagrams of test setups for friction and pullout evaluation of
ls. (a) Soil-to-fabric friction test and results. (b) Fabric pullout (anchor-

between the fabric and the soil with no further increase in
n the test is repeated at different normal stresses, a trend is
shear strength parameters can be obtained. Note that these
parameters are related to, but not necessarily the same as, the
rs. They (the soil parameters) are, however, the conservative
fabric parameters. Often an efficiency, as defined below, is
of soil shear strength parameters that is mobilized, e.g.,

$$E_c = (c_a/c) \times 100 \tag{2.7}$$

$$E_\phi = (\tan \delta / \tan \phi) \times 100 \tag{2.8}$$

n cohesion,
n friction angle,
f soil to fabric,

c = the cohesion of soil,
δ = the friction angle of soil to fabric, and
ϕ = the friction angle of soil.

Results from such a test setup by Martin, et al. [13], are presented in Table 2.6 for four geotextile types against three different cohesionless soils. Soil-to-fabric friction angles are given, as well as the fabric efficiency versus the soil friction angle by itself as per Equation 2.8. Here it is seen that most geotextiles can mobilize a high percentage of the soil's friction and can be used to advantage in situations requiring this feature. A review and compilation of a number of direct shear tests on various fabrics against different granular soils is given by Richards and Scott [14]. Another review by Williams and Houlihan [15] covers a wider range of soils, including some sands, silts, and mixed soils.

2.2.3.11 Pullout (Anchorage) Tests

Geotextiles are often called upon to provide anchorage for many applications within the reinforcement function. Such anchorage usually has the fabric sandwiched between soil on each side of it. The resistance can be modeled in the laboratory using a pullout test, shown schematically in Figure 2.8b. The pullout resistance is obviously dependent on the normal force applied to the soil surrounding it, which mobilizes shear forces on both sides of the fabric.

Test results by Collios et al. [16] show a relationship of pullout test results to shear test results with some notable exceptions. If the soil particles are smaller than the fabric openings, efficiencies are higher; if not, they can be lower. In all cases, however, pullout test resistances are less than shear test resistances. This is due to the fact that the fabric is taut and exhibits large deformations. This in turn causes the soil particles to reorient themselves into a reduced-shear-strength situation at the soil-fabric interfaces, resulting in lower pullout resistance. The stress state mobilized in this test is a very complex one requiring additional research.

TABLE 2.6 SOIL-TO-FABRIC FRICTION ANGLES AND EFFICIENCIES (IN PARENTHESES) IN COHESIONLESS SOIL

Geotextile type	Manufacturer's designation	Concrete sand ϕ = 30 deg.	Rounded sand ϕ = 28 deg.	Silty sand ϕ = 26 deg.
Woven, monofilament	Polyfilter X	26 deg. (84%)	-	—
Woven, silt film	500X	24 deg. (77%)	24 deg. (84%)	23 deg. (87%)
Nonwoven, melt-bonded	3401	26 deg. (84%)	-	-
Nonwoven, needle-punched	CZ600	30 deg. (100%)	26 deg. (92%)	25 deg. (96%)

Source: After Martin et al. [13]

SLT North America, Inc.

FrictionFlex™ Application Data

SLT's FrictionFlex process provided the industry's first textured liner. It is the only geomembrane texturing process ever to be granted a U.S. Patent. It, in fact, has been awarded two*. In direct contrast to blown-film geomembranes which are textured or made rough by a process which actually erodes the sides of the sheet, the FrictionFlex process is additive. SLT begins with 24-foot wide SLT HyperFlex™ or UltraFlex™ sheet manufactured to the industry's most exacting standards. Only after the sheet passes all QC, is texturing added to one or both sides as required by the application. When the engineer utilizes SLT geomembranes textured by the FrictionFlex process, increased facility design capacity, service life and total revenue potential can be obtained. Containment slopes, vertical expansions and perimeter slopes in closures share the benefits of greater air-space and superior cover stability.

Most importantly, the advantages of FrictionFlex are available without compromise of any performance property or other issue of secure containment. The patented manufacturing process enables SLT to produce a textured liner exhibiting similar mechanical and chemical properties demanded of SLT's premium grades of smooth geomembrane liners, whether HDPE, VLDPE or LDPE.

An added feature of SLT's process is that an edge, 6-to-8 inches wide, is left smooth to aid in welding and field quality control. This allows standard installation equipment and procedures to ensure expedient construction.

The following reflects independent data confirming superior FrictionFlexed liner performance in contact with soils and synthetics:

- Highest coefficient of friction with soils
- Highest coefficient of friction with synthetics
- Premium grade mechanical and chemical properties

SLT Textured Liner Materials				Typical Smooth HDPE
Material	Coefficient of Friction	Adhesion (per square foot)	Average Friction Angle (degrees)	Comparable Friction Angle
Sandy Glacial Till	0.74	27	36	20
Sandy Clay	0.70	65	35	18
Smooth Clay	0.62	39	32	16
Ottawa Sand	0.59	21	30	19
Non-woven Polyester Geotextile	0.54	116	28 *	11
Non-woven Poly-propylene Geotextile	0.65	133	33	12

NOTE: The above data is approximate. SLT recommends that specific data be developed for all application designs. Shear box testing of the specific geosynthetic and natural components of the composite is necessary to establish an appropriate design basis. SLT will be pleased to provide any necessary material samples for such purposes and invites comparative procedures.

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

*U.S. Patent No. 4,885,201
5,075,135

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MLD

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)	Apparent Cohesion (psf)	Comments
J & L	05-30-90	NW/Sand	1 - 2 - 3	Hydrated	0.02	35	Not determined	10	
		NW/Sand *	1 - 2 - 3	Dry *	0.02	28 *	"	85	
		NW/Clay	1 - 2 - 3	Hydrated	0.02	41	"	77	
		NW/Clay	1 - 2 - 3	Dry	0.02	32	"	105	
STS	09-11-90	NW/40-mil Text. HDPE *	35 - 52 - 70	Dry *	0.2	18 *	Not determined	0	
		NW/80-mil Text. HDPE	35 - 52 - 70	Dry	0.2	37	"	0	
		W/80-mil Text. HDPE	35 - 52 - 70	Dry	0.2	24	"	0	
J & L	11-06-90	NW/Sandy soil	2 - 3.5 - 5	Dry	0.02	23	Not determined	119	
GRI	04-18-91	Internal	.5 - 1 - 5 - 10 - 20	Dry	0.035	42	Not determined	288	
		Internal	.1 - .5 - 1 - 5 - 10	Hydrated	0.035	37	"	115	
		Internal	.1 - .5 - 1 - 5 - 10	Hydrated	0.035	39	"	173	Hydrated in leachate
STS	05-28-91	NW/40-mil Text. HDPE	35 - 52 - 70	Hydrated	0.2	20	Not determined	0	
		W/80-mil Text. HDPE	35 - 52 - 70	Hydrated	0.2	19	"	0	
UTA	08-12-91	Internal	6 - 9 - 14 - 19	Hydrated	0.000131	26	Not determined	619	
J & L	09-09-91	W/Soil cover	0.6 - 1.25 - 1.88	Hydrated	0.035	22.5	20.5	55	
		W/Geonet	0.6 - 1.25 - 1.88	Hydrated	0.035	17	16	64	
		NW/2B Stone	0.6 - 1.25 - 1.88	Hydrated	0.035	53	52	10	
TRI	05-06-92	W/60-mil text. VLDPE	2 - 8 - 14	Hydrated	0.04	22	Not determined	113	Limited hydration
		W/60-mil sm. VLDPE	2 - 8 - 14	Hydrated	0.04	15	"	77	"
TRI	11-12-92	W/40-mil text. LLDPE	3.5 - 7 - 14	Hydrated	0.2	25	16.5	230	
TRI	03-16-93	W/Saturated soil	1 - 2 - 3	Hydrated	0.04	24	Not determined	100	Bentomat HS
		W/Dry soil	1 - 2 - 3	Hydrated	0.04	20	"	153	"
		NW/Drainage geocomp.	1 - 2 - 3	Dry	0.04	17	"	20	"

Calculation Package F
Geotextile/Separation/Filtration Requirements

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Geotextile Performance Requirements

Computations By:
SIGNATURE DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE DATE 11/3/06
Gregory T. Corcoran, PE / Principal
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Computations Checked By:
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Gregory T. Corcoran, PE / Principal
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Computations Backchecked By (Originator):
SIGNATURE DATE 11/3/06
Rebecca Flynn, Staff Engineer
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Approved By (PM or Designate):
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Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebecca Flynn Date: 06/09/06 (YY MM DD) Reviewed by: GTC Date: 06/11/03 (YY MM DD)

Client: BRC Project: CAMU Project/Proposal No.: SC0313 Task No.: 01-04

GEOTEXTILE PERFORMANCE REQUIREMENTS
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

It is proposed that the drainage system for the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada include a geonet overlaid by an 8 oz/sy geotextile on the top side of the geonet acting as a filter geotextile. The geotextile must retain the overlying protective soil to minimize impairment to the underlying geocomposite drainage layer material flow properties and have sufficient mechanical properties for durability. This calculation will evaluate the required performance properties of the filter geotextile.

SUMMARY OF ANALYSIS

The calculations suggest:

- that the separation/filtration geotextile overlying the aggregate drainage layer have an AOS less than sieve No. 70 (0.21 mm), a permittivity greater than 0.8 sec^{-1}, a minimum mass per unit area of 6 oz./yd^2, and sufficient mechanical strength properties as outlined in federal guidelines; and

- that the separation/filtration geotextile adhered to the geonet layer have an AOS less than sieve No. 70 (0.21 mm), a permittivity greater than 0.6 sec^{-1}, a minimum mass per unit area of 8 oz./yd^2, and sufficient mechanical strength properties as outlined in federal guidelines.

SITE CONDITIONS

The liner system consists of, from top to bottom:

- 2 ft. of operations layer material;
- Drainage geocomposite with an 8 oz/sy geotextile bonded to both sides of the geonet;
- a 60-mil (1.5-mm) textured high density polyethylene (HDPE) geomembrane;
- a geosynthetic clay liner (GCL); and
- subgrade.

A cross-section of the lining system is presented as Attachment A.

The operations layer material will consist of on-site material, which has been classified as silty sand to well-graded sand (SM, SM-SW according to the Unified Soils Classification System) (Converse Consultants, 1999) (Attachment F).

ANALYSIS

Filtration Requirements: The geotextile will minimize fine particles of the operations layer material from migrating into the geocomposite drainage layer material. Migration of fine particles would have the adverse effect of decreasing the transmissivity of the geocomposite drainage layer.

The filtration requirements for geotextiles can be evaluated using the "Geotextile Filter Design Manual" developed by Luettich et al., (1991) (Attachment B). Page 2 of Attachment B shows a chart in which soil properties are used to evaluate the retention criteria of the geotextile by determining the maximum allowable apparent opening size (AOS or O_{95}).

The soil cover has been classified as silty or clayey sand and well-graded sand. Both of these classifications suggest that less than fifty percent of the material is fine-grained soils (i.e., smaller than the No. 200, or 0.075 mm, sieve size). To be conservative in the calculations herein, the operations layer is assumed to consist of more than 20 percent clay and to be non-dispersive. Therefore, using page 2 of Attachment B,

> $\mathbf{O_{95} < 0.21}$ **mm**, which corresponds to sieve No. 70, meaning that the geotextile apparent opening size (AOS) must be less than a No. 70 sieve size.

Permeability: The following equation can be used to evaluate the minimum allowable geotextile permeability:

$$k_g > i_s\, k_s \qquad \text{(Luettich et al. (1991), Att. B, p. 1)}$$

where: k_g = permeability of geotextile (cm/s)
i_s = hydraulic gradient (dimensionless)
k_s = permeability of the protective soil cover (cm/s)

Hydraulic Gradient, i_s: Attachment B, page 3 from Luettich et al. (1991) lists typical hydraulic gradients for various geotextile drainage applications. In this attachment, a hydraulic gradient of 1.5 for landfill LCRS (landfill leachate collection and removal system) applications is recommended.

Soil Permeability, k_s: A permeability of 1.2×10^{-3} cm/s was used based on permeability testing of site specific soils. (Attachment F)

Therefore,

$$k_g > \quad i_s k_s = (1.5)(1.2\text{x}10^{-3})$$
$$k_g > \quad 1.8 \times 10^{-3} \text{ cm/s}$$

Koerner (1994) suggests applying partial factors of safety to the ultimate flow capacity of the geotextile to account for clogging of the geotextile. Using recommendations given in Table 2.13 on p. 160 of Koerner (1994) (Attachment D), the following partial safety values were applied:

soil clogging and blinding:	10 (5 – 10)	creep reduction of voids:	2.0 (1.5 – 2.0)
intrusion into voids:	1.2 (1.0 – 1.2)	chemical clogging:	1.5 (1.2 – 1.5)
biological clogging:	2.0 (2 – 50)		

Therefore,

$$k_g > \quad (1.8 \times 10^{-3})(10)(2)(1.2)(1.5)(2)$$
$$k_g > \quad 0.13 \text{ cm/s}$$

The thickness of 6 oz/yd^2 (205 g/m^2) and 8 oz/yd^2 (273 g/m^2) nonwoven geotextiles are approximately 65 mils (0.165 cm) and 90 mils (0.229 cm), respectively (Amoco technical literature, Attachment E, p. 1). Dividing the permeability by the thickness of the geotextile results in the following permittivity values:

6 oz./SY = 0.78 sec^{-1}

8 oz./SY = 0.57 sec^{-1}

Mechanical Property Requirements: To ensure proper manufacturing and durability of the geotextile, the geotextile should have appropriate strength requirements. Based on guidelines developed by Task Force 25 (see note below) (Attachment C) for mechanical properties of geotextiles used in applications requiring moderate survivability, the geotextile should have the following properties:

Property	**Criteria**
Grab strength	≥ 130 lb.
Puncture strength	≥ 40 lb.
Mullen burst	≥ 210 lb
Trapezoidal tear	≥ 40 lb
Ultraviolet strength retention	≥ 70 %

Note: Task Force 25 consisted of the American Associated of State and Transportation Officials (AASHTO), the American Building Contractors (ABC), and the American Road Builders and Transportation Association (ARBTA).

CONCLUSIONS

In accordance with the above analysis, the geotextile component of the drainage composite or separation/filtration geotextile shall have the following properties:

6 oz/yd^2

Property	**Separation/Filtration Criteria**
matrix	nonwoven
mass per unit area	6 oz/yd^2 (205 g/m^2)
apparent opening	≤ 0.21 mm (sieve No. 70)
permittivity	≥ 0.8 s^{-1}
grab strength	≥ 130 lb.
puncture strength	≥ 40 lb.
mullen burst	≥ 210 lb.
trapezoidal tear	≥ 40 lb.
ultraviolet strength retention	≥ 70 %

8 oz/yd^2

Property	**Separation/Filtration Criteria**
matrix	nonwoven
mass per unit area	8 oz/yd^2 (273 g/m^2)

Written by: Rebecca Flynn	Date: 06/09/06 (YY MM DD)	Reviewed by: GTC	Date: 06/11/3 (YY MM DD)
Client: BRC	Project: CAMU	Project/Proposal No.: SC0313	Task No.: 01-04

apparent opening	≤ 0.21 mm (sieve No. 70)
permittivity	≥ 0.6 s^{-1}
grab strength	≥ 130 lb.
puncture strength	≥ 40 lb.
mullen burst	≥ 210 lb.
trapezoidal tear	≥ 40 lb.
ultraviolet strength retention	≥ 70 %

The following is a partial list of geotextile products that should meet the material requirements.

Amoco Fabrics & Fibers Co., Amoco 4506
Trevira 011/120
Synthetic Industries, Geotex 701

REFERENCES

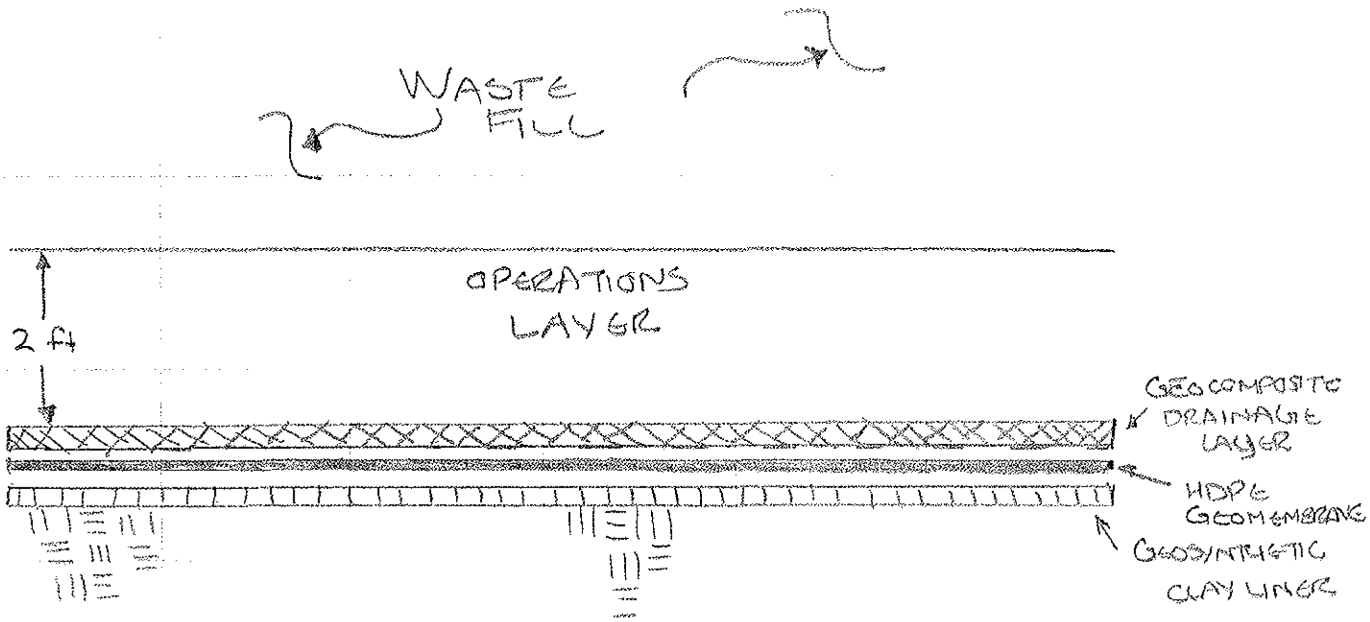
Amoco Fabrics and Fibers Company, Atlanta, Georgia, 404-984-4444

Luettich, S.M., Giroud, J.P., and Bachus, R.C. (1991), "Geotextile Filter Design Manual", report prepared for Nicolon Corporation, Norcross, GA *(Attachment B).*

Converse Consultants (1999), "Preliminary Geotechnical and Geological Investigation", Prepared for Parsons Engineering Science, Inc., October 1999.

Written by: R. FLYNN Date: 21 / 08 / 06 (DD MM YY) Reviewed by: ETC Date: 06 / 11 / 13 (DD MM YY)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No: 01 - 04



COMPOSITE LINER SYSTEM

ATTACHMENT A 1/1

MW

MLD
5/11/12
GTC

4.2 Define the Hydraulic Gradient for the Application (i_s)

The hydraulic gradient will vary depending on the application of the filter. Anticipated hydraulic gradients for various applications may be estimated using Figure 3.

4.3 Determine the Minimum Allowable Geotextile Permeability (k_g)

After determining the soil hydraulic conductivity and the hydraulic gradient, the following equation can be used to determine the minimum allowable geotextile permeability [Giroud, 1988]:

$$k_g > i_s k_s$$

The hydraulic conductivity (permeability) of the geotextile can be calculated from the permittivity test method ASTM D 4491; this value can often be obtained from the manufacturer's literature as well. The geotextile permeability is defined as the product of the permittivity, ψ, and the geotextile thickness, t_g:

$$k_g > \varphi t_g$$

STEP 5. DETERMINE ANTI-CLOGGING REQUIREMENTS

To minimize the risk of clogging, the following criteria should be met:

- Use the largest opening size (O_{95}) that satisfies the retention criteria.
- For nonwoven geotextiles, use the largest porosity available, but not less than 30 percent.
- For woven geotextiles, use the largest percent open area available, but not less than 4 percent.

Source: Luettich, S.M. Giroud, J.P. and Bachus R.C (1991) "Geotextile Filter Design Manual" Report prepared For Nicolon Corporation Norcross, GA

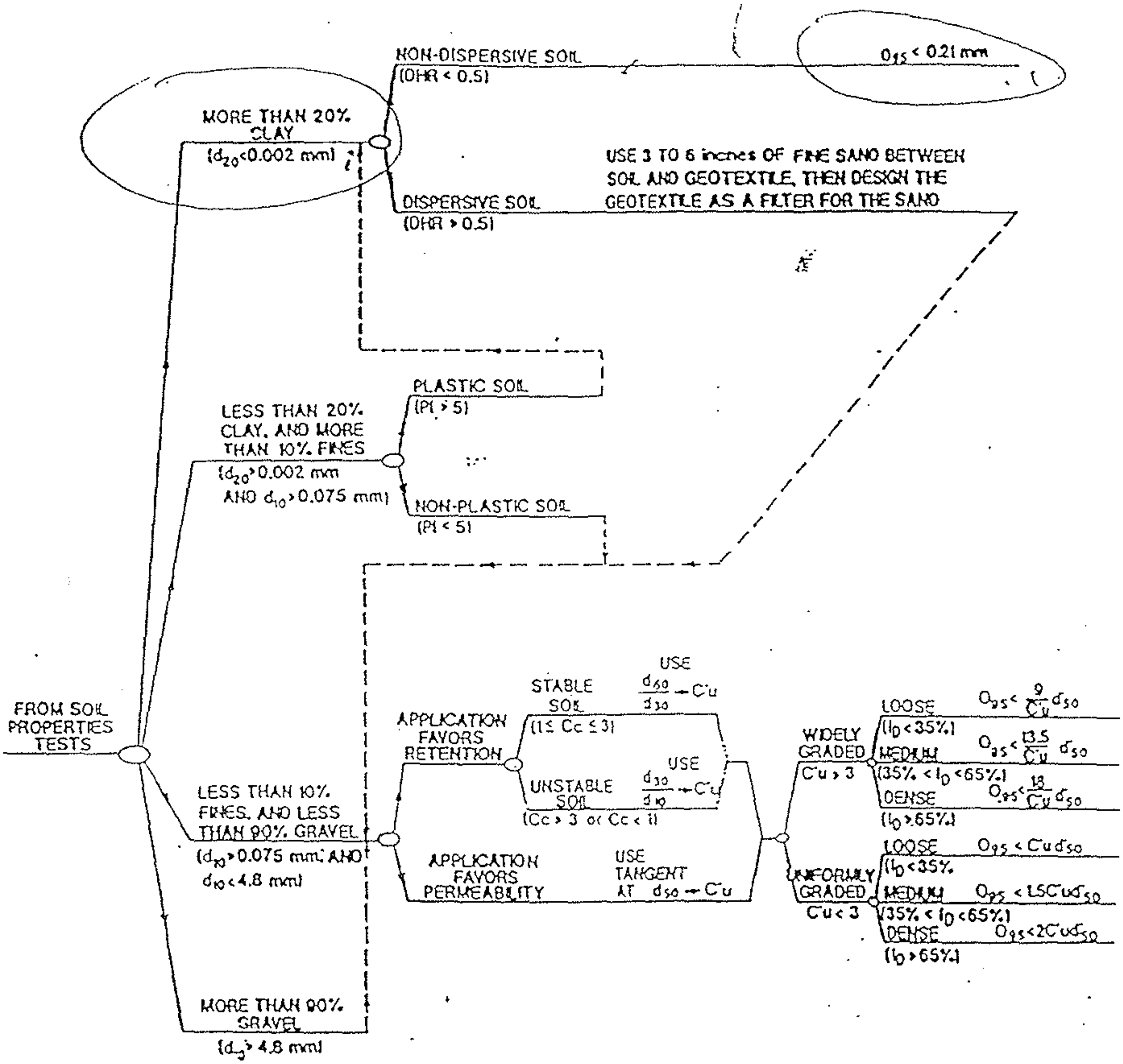
Attachment B page 1/3

GTC MSD

ATTACHMENT B

P 2/3

CHART 1
SOIL RETENTION CRITERIA
FOR STEADY-STATE FLOW CONDITIONS



NOTES:

d_x is the particle size of which x percent is smaller

$C'u = \sqrt{\frac{d'_{100}}{d'_0}}$ where: d'_{100} and d'_0 are the extremities of a straight line drawn through the particle-size distribution, as directed above; and d'_{50} is the midpoint of this line.

$Cc = \frac{(d_{30})^2}{d_{60} \times d_{10}}$

I_D is the relative density of the soil

PI is the plasticity index of the soil

DHR is the double-hydrometer ratio of the soil

Portions of this flow chart modified from Giroud (1988)

Source: Luettich, S.M., Giroud, J.P., and Bachus, R.C. (1991) "Geotextile Filter Design Manual" Report prepared for

ATTACHMENT B, 2/3

HN

MLD GTC

FIGURE 3

TYPICAL HYDRAULIC GRADIENTS[a]

DRAINAGE APPLICATION	TYPICAL HYDRAULIC GRADIENT
STANDARD DEWATERING TRENCH	1.0
VERTICAL WALL DRAIN	1.5
PAVEMENT EDGE DRAIN	1[b]
LANDFILL LCDRS	1.5
LANDFILL LCRS	1.5
LANDFILL SWCRS	1.5
DAMS	10[b]
INLAND CHANNEL PROTECTION	1[b]
SHORELINE PROTECTION	10[b]
LIQUID IMPOUNDMENTS	10[b]

NOTES: (a) Table developed after Giroud, 1988.

(b) Critical applications may require designing with higher gradients than those given.

Attachment B

p. 3/3

Specification for *Survivability*

TABLE C-4 MINIMUM FABRIC PROPERTIES RECOMMENDED FOR FABRIC SURVIVABILITY[a]

Required degree of fabric survivability	Grab strength (lbs.)	Puncture strength[b] (lbs.)	Burst strength[c] (lb./in.^{2})	Trap tear[d] (lbs.)
Low	90	30	145	30
* Moderate	130	40	210	40
High	180	75	290	50
Very high	270	110	430	75

(a) All values represent minimum values (i.e., any roll in a lot should meet or exceed the minimum values in this table).

(b) ASTM D751-68, tension testing machine with ring clamp, steel ball replaced with a 5/16-in.-diameter solid steel cylinder with hemispherical tip centered within the ring clamp.

(c) ASTM D751-68, diaphragm test method.

(d) ASTM D1117, either principal direction.

Koerner 2nd Edition

ATTACHMENT C 1/1

TABLE C-5 REQUIRED DEGREE OF SI ABILITY AS A FUN CONDITIONS AND CONSTRUCTION EQ ENT[a]

Subgrade conditions	Construction ec: Low ground-pressure equipment ($\leq$4 lb./in.^{2})
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level such that any shallow depressions and humps do not exceed 6 in. in depth and height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Low
Subgrade has been cleared of obstacles larger than small to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 18 in. in depth and height. Larger depressions should be filled.	Moderate
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than 6 in. ± above subgrade. Fabric may be draped directly over tree trunks, stumps, large depressions and humps, holes, steam channels, and large boulders. Items should be removed only if placing the fabric and cover material over them will distort the finished road surface.	High

(a) Recommendations are for 6-12 in. initial lift thickness. For other init
12–18 in: reduce survivability requirement one level
18–24 in.: reduce survivability requirement two levels
>24 in.: reduce survivability requirement three levels
Survivability levels are, in increasing order: low, moderate, high, an
For special construction techniques such as prerutting, increase fabric
Placement of excessive initial cover material thickness may cause be

Source: After Christopher, B., and Holtz, R. D., Federal Highway Adı Training Manual, Washington, DC.

MS

GTC MLD

Table 2.13 Recommended partial factors of safety values for use in Equation 2.25

Application	Soil Clogging and Blinding	Creep Reduction of Voids	Intrusion into Voids	Chemical Clogging	Biological Clogging
	Various Partial Factors of Safety				
Retaining wall filters	2.0 to 4.0	1.5 to 2.0	1.0 to 1.2	1.0 to 1.2	1.0 to 1.3
Underdrain filters	5.0 to 10	1.0 to 1.5	1.0 to 1.2	1.2 to 1.5	2.0 to 4.0
Erosion control filters	2.0 to 10	1.0 to 1.5	1.0 to 1.2	1.0 to 1.2	2.0 to 4.0
Landfill filters	5.0 to 10	1.5 to 2.0	1.0 to 1.2	1.2 to 1.5	2.0 to 5.0
Gravity drainage	2.0 to 4.0	2.0 to 3.0	1.0 to 1.2	1.2 to 1.5	1.2 to 1.5
Pressure drainage	2.0 to 3.0	2.0 to 3.0	1.0 to 1.2	1.1 to 1.3	1.1 to 1.3

do not serve this function, the other, sometimes primary, function will not be served properly. This should not give the impression that geotextiles as separators always play a secondary role. Many situations call for separation only, and in such cases the geotextiles do serve a significant and worthwhile function.

2.5.1 Overview of Applications

Perhaps the target application that can best illustrate the use of geotextiles as separators is their placement between an underlying reasonably firm soil subgrade and a stone base course, aggregate, or ballast placed above the geotextile. We say "reasonably firm" because it is assumed that the subgrade deformation is not sufficiently large to mobilize uniformly high tensile stress in the geotextile. (The application of geotextiles in unpaved roads on soft soils wherein membrane-type reinforcement is developed is treated later in Section 2.6.) Thus for such a separation function to occur, the geotextile must be placed on the soil subgrade and then have stone placed, spread, and compacted on top of it. A number of scenar[ios] can be developed showing what geotextile properties are required for a given situation.

2.5.2 Burst Resistance

Consider a geotextile on a soil subgrade with stone of average particle diameter (d_a) placed above it. If the stone is uniformly sized, there will be voids within it that will be available for the geotextile to enter into. This entry is caused by the simultaneous action of the traffic loads being transmitted to the stone, through the geotextile, and into the underlying soil. The stressed soil then tries to push the geotextile up into the voids within the stone. The situation is shown schematically in Figure 2.26. Giroud [59] provides a formulation for the required geotextile strength which can be adopted for this application.

$$T_{reqd} = \frac{1}{2} p' d_v [f(\epsilon)] \tag{2.26}$$

where T_{reqd} = the required geotextile strength,
p' = the stress at the geotextile's surface, which is less than, or equal to,

Koerner (1994)

ATTACHMENT D 1/1

AMOCO WASTE RELATED GEOTEXTILES

MINIMUM PHYSICAL PROPERTIES
(Minimum Average Roll Values)

Property	Test Method	Units	4504	4506	4508	4510	4512	4516
Unit Weight	ASTM D-3776	Oz./yd.²	4.0	6.0	8.0	10.0	12.0	16.0
Grab Tensile	ASTM D-4632	lbs.	95	150	200	235	275	350
Grab Elongation	ASTM D-4632	%	50	50	50	50	50	50
Mullen Burst	ASTM D-3786	psi	225	350	450	550	600	750
Puncture	ASTM D-4833	lbs.	55	90	130	165	185	220
Trapezoid Tear	ASTM D-4533	lbs.	35	65	80	95	115	130
Apparent Opening Size	ASTM D-4751	US Sieve Number	70	70	100	100	100	100
Permittivity	ASTM D-4491	gal/min/ft² sec⁻¹	100 2.0	90 1.7	80 1.5	70 1.1	60 0.9	50 0.7
Permeability	ASTM D-4491	cm/sec	.2	.2	.2	.2	.2	.2
Thickness	ASTM D-1777	mils	40	65 *	90 *	110	130	175
U.V. Resistance	ASTM D-4355[1]	%[2]	70	70	70	70	70	70

1. Fabric conditioned per ASTM D-4355 2. Percent of minimum grab tensile after conditioning.

TYPICAL PHYSICAL PROPERTIES

Property	Test Method	Units	4504	4506	4508	4510	4512	4516
Grab Tensile	ASTM D-4632	lbs.	130/115	225/200	275/270	315/310	410/370	510/470
Grab Elongation	ASTM D-4632	%	75	65	65	65	65	65
Mullen Burst	ASTM D-3786	psi	285	410	575	650	825	920
Puncture	ASTM D-4833	lbs	75	120	170	190	210	270
Trapezoid Tear	ASTM D-4533	lbs	60/50	100/80	140/120	160/140	185/155	220/180
Apparent Opening Size	ASTM D-4751	US Sieve Number	70/120	70/140	100/200	100 -	100 -	100 -
Permittivity	ASTM D-4491	gal/min/ft² sec⁻¹	150 3.1	110 2.0	100 1.8	80 1.5	70 1.3	60 1.0
Permeability	ASTM D-4491	cm/sec	.35	.31	.27	.26	.25	.23
Thickness	ASTM D-1777	mils	50	75	115	130	150	195

PACKAGING

Dimensions		4504	4506	4508	4510	4512	4516
Roll Width	ft	15	15	15	15	15	15
Roll Length	ft.	1200	900	600	600	450	300

Amoco Techical Literature Appendix E
404 984-4441

MLD

accuracy or reliability of said information or the performance of any product. Any specifications/properties ... provided as information only and in no way modify, amend, enlarge or create any warranty. Nothing contained herein is to be construed as permission or as a recommendation to infringe any patent.

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Exploration Location	Depth (feet)	Soil Description	Percent Sodium	Percent Sulfate	Total Available Water Soluble sodium Sulfate (%)
B-5	10-15	Silty sand with gravel	0.07	0.13	0.20
B-8	19-20	Silty sand with gravel	0.07	0.06	0.08
B-101	5-10	Silty sand with gravel	0.17	0.06	0.08
B-102	0-5	Fill – Silty sand with gravel	0.17	0.03	0.05
B-106	0-5	Silty sand with gravel	0.15	0.08	0.12
B-106	29-30	Silty sand with gravel	0.15	0.06	0.08

Permeability

Falling head permeability tests were conducted on remolded samples in general accordance with modified ASTM procedure D2434. The soil was compacted in a mold 4.6 inches long and 4.0 inches in diameter to 85 or 90 percent of maximum dry density and at optimum moisture content. A falling head was applied to the sample and the flow of water through the sample was monitored. The permeability was calculated after the flow rate had stabilized. The result of the falling head permeability test is presented in the following table:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-5	20-25	Silty sand with gravel	5.3×10^{-4}
B-12	10-15	Silty sand with gravel	4.0×10^{-4}
B-102	20-25	Silty sand with gravel	1.0×10^{-4}
* B-105	20-25	Well graded sand with silt and gravel	1.2×10^{-3}

Flexible wall permeameter tests were performed on selected samples by AP Engineering and Testing, Inc according to ASTM D5084. With the exception of one sample (B-105), all tested samples were undisturbed ring samples. The samples were placed in a triaxial machine with a constant confining pressure at the approximate in-place effective stress pressures. Results were generally consistent with the fal-

Calculation Package G
Geotextile Cushion Protection

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Geotextile Puncture Protection of Geomembrane

Computations By:

SIGNATURE DATE 11/3/06

Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE DATE 11/6/06

Keaton Botelho, Staff Engineer
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE DATE 11/3/06

Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

GEOTEXTILE PUNCTURE PROTECTION OF GEOMEMBRANE

OBJECTIVE

A composite liner system is proposed for the Corrective Action Maintenance Unit (CAMU) located in Henderson, Nevada. The objective of this calculation is to evaluate the maximum particle size of soil materials adjacent to the geomembrane that will not puncture the geomembrane. Specifically, the evaluation will consider the drainage aggregate overlying the geocomposite and geomembrane components of the liner system and the subgrade underlying the geomembrane and GCL components of the liner system.

SUMMARY OF ANALYSIS

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

Soil Component of Liner	Maximum Particle Size	Minimum Mass Per Unit Area
Subgrade	0.75 in	9 oz./SY (GCL)
Angular Drainage Aggregate	1.00 in	16 oz./SY

SITE CONDITIONS

The composite liner system will be comprised of the following components, from top to bottom (Attachment A):

- 2 ft of operations layer material;
- a geocomposite drainage layer;
- 60-mil (1.5 mm) HDPE geomembrane, textured on both sides;
- a geosynthetic clay liner (GCL); and
- prepared subgrade.

The maximum height of waste to be placed within the lined area is 93 ft overlying the drainage sump in the South Mesa, based on the proposed waste fill plan. (Attachment B)

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: RTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

- **OVERLYING PRESSURE**:

The overlying pressure can be estimated based on the maximum future fill height of 93 feet. The unit weight of the waste material was selected to be 136 pcf based on modified proctor tests conducted on soil samples from the site that are similar to the waste material to be placed with in the CAMU. The maximum dry density was determined to be 132 pcf at an optimum moisture content of 8.2%. Assuming that the material will be placed at a density less than 95% degree of compaction, the resulting dry density is 125.4 pcf. Adding the weight of the moisture in the soil results in a wet density of approximately 136 pcf. (Attachment C). Therefore, the overlying pressure is estimated as follows:

$$P = (93\text{ ft})(136\text{ pcf}) = 12{,}648\text{ psf or }606\text{ kPa}$$

ANALYSIS

- **APPROACH – Protected Geomembrane**

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

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

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

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

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

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

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

- **MASS PER UNIT AREA OF GEOTEXTILE**

Two different mass per unit areas will be evaluated. The cushion geotextile overlying the liner system will be 16 oz./SY and the GCL underlying the liner system will have geotextile components with a minimum 9 oz./SY mass per unit area. (Attachment D)

- **SIZING MAXIMUM PARTICLE OF SOIL**

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

$H^2 = 450\ M_A / P_{allow}$ (Attachment E)

where:

M_A = mass per unit area geotextile (g/m^2)
= 542 (16 oz./SY) and 305 (9 oz./SY) g/m^2

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

P_{allow} = maximum long term allowable pressure

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

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 E)

where: FS = global factor of safety, 3.0 (Attachment E)
$P_{actual\ field\ pressure}$ = 606 kPa
P'_{allow} = (606)(3) = 1,818 kPa

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

MF_{PD} packing density: (Attachment E)
0.5 (assume packed stones)

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

MF_A	soil arching: 0.75 (assume moderate)	(Attachment E)
FS_{CR}	partial factor of safety for creep 1.5 (see Table 12)	(Attachment E)
FS_{CBD}	partial factor of safety for chemical and biological degradation 1.5 (based on average value)	(Attachment E)

Solving for P_{allow} provides:

$$P_{allow} = (1{,}818)\ (1.0 \times 0.5 \times 0.75)(1.5 \times 1.5)$$
$$P_{allow} = 1{,}534 \text{ kPa}$$

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

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

$$H_{cushion} = [(450)(M_A)/(1{,}534)]^{1/2}$$
$M_A = 542$ g/m^2 = 12.6 mm = 0.50 inches

$$H_{GCL} = [(450)(M_A)/(1{,}534)]^{1/2}$$
$M_A = 305$ g/m^2 = 9.45 mm = 0.37 inches

The predicted effective protrusion height equals one half the maximum stone size. Therefore, the maximum stone size is twice the values listed above.

$M_A = 542$ g/m^2 = 0.50 inches x 2 = 1.0 inches
$M_A = 305$ g/m^2 = 0.37 inches x 2 = 0.75 inches

CONCLUSIONS

Assuming the following:

- the particle shape is angular for the drainage aggregate, and
- the approach presented by Wilson-Fahmy, Narejo, and Koerner for evaluating puncture protection of geomembranes is appropriate for the analysis herein,

then, the calculations suggest that THE MAXIMUM PARTICLE SIZE IS 1.0 IN. for a 16 oz/yd^2 geocomposite drainage layer and 0.75 IN. for a GCL with two geotextiles equating to 9 oz/yd^2.

- **COMPLETE SET OF GEOTEXTILE PROPERTIES**

Typical nonwoven, needlepunched geotextile, when subjected to laboratory testing, should provide the following mechanical property values:

Property	**16 oz./SY Value**
puncture strength	> 240 lb
grab strength	> 390 lb
trapezoidal tear	> 150 lb
ultraviolet strength retention	> 70 %

REFERENCES

Geotechnical Fabrics Report, "2005 Specifiers Guide", Vol 22, No. 9, Dec. 2004.

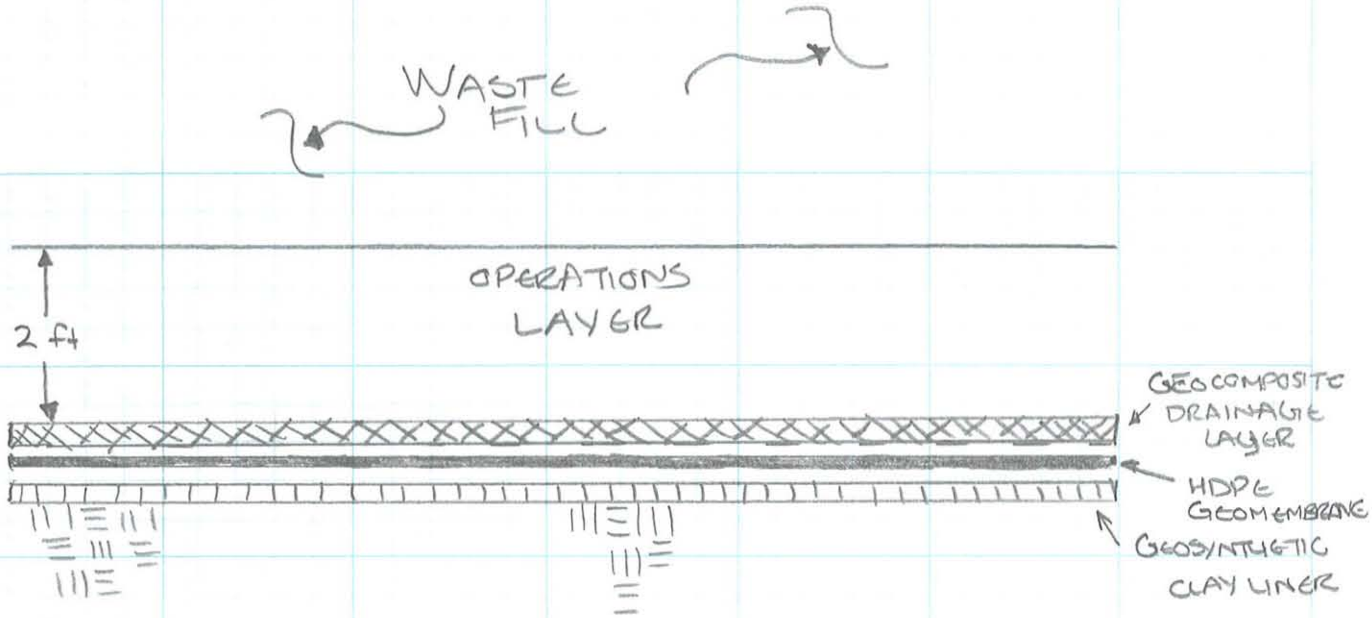
Koerner, R.M., Wilson-Fahmy, R.F. and Narejo, D. (1996) "Puncture Protection of Geomembranes Part III: Examples", Geosynthetics International, Vol. 3, No. 5, pp. 655-675.

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.

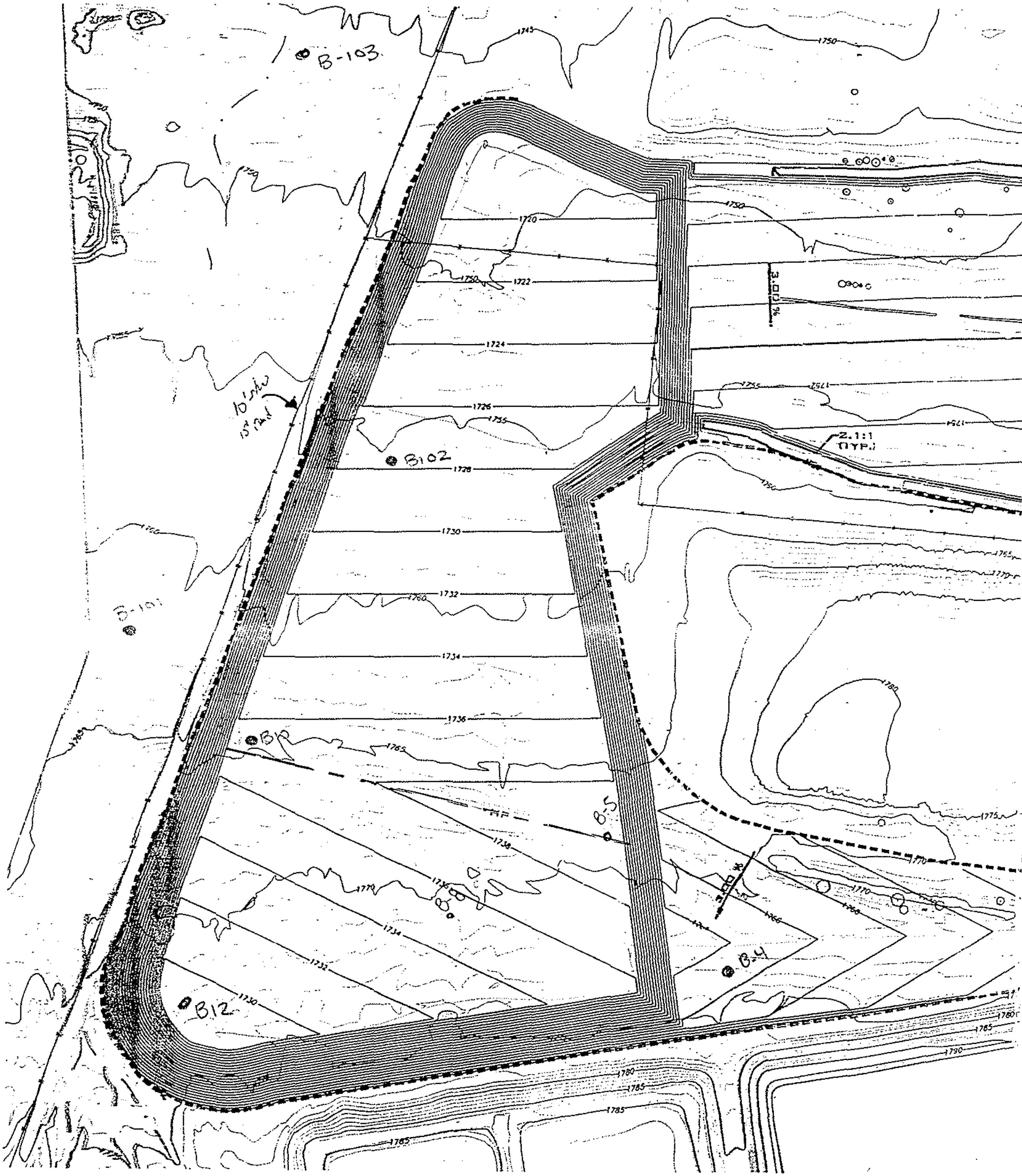
Written by: R. FLYNN Date: 21 / 08 / 06 (DD MM YY) Reviewed by: GTC Date: 03 / 11 / 06 (DD MM YY)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No: 01 - 04



COMPOSITE LINER SYSTEM

ATTACHMENT A 1/1

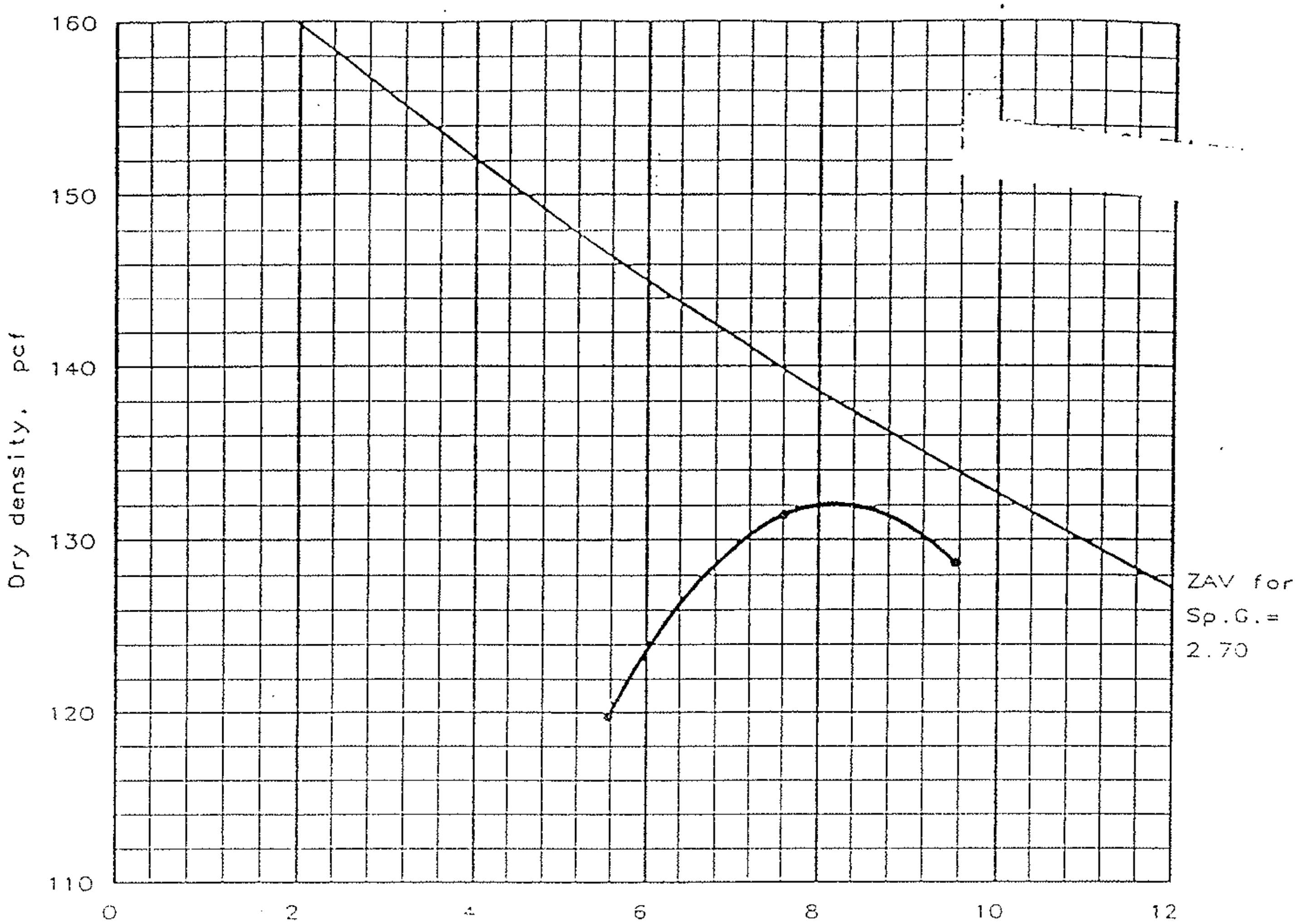


Attachment B

1/1

(3)5/10/05

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 1557-91 Procedure C, Modified

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in	% < No.200
	SM				NV	NP	0 %	19.2 %

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 132.1 pcf Optimum moisture = 8.2 %	Silty Sand with Gravel

Project No.: 99-33437-01

Project: BMI Landfill

Location: B5 @ 20-25'

Date: 10-15-1999

MOISTURE-DENSITY RELATIONSHIP TEST

CONVERSE CONSULTANTS

Fig. No. A-58

ATTACHMENT C, 1/1 MWD

GEOSYNTHETIC CLAY LINERS

Product Name	Bonding Method	GCL Dimensional Properties: Panel Size Roll Width/Length m/m (ft/ft)	GCL Dimensional Properties: Average Roll Weight kg (lb)	GCL Dimensional Properties: Bentonite Mass/Unit Area ASTM D 5993 gm/m² (lb/ft²)	GCL Hydraulic Properties: Flux [1] ASTM D 5887 [2] m³/m²-s	Base Bentonite Properties: Swell Index ASTM D 5890 (min.) ml/2g	Base Bentonite Properties: Fluid Loss ASTM D 5891 ml	GCL Structural Components, Upper Geosynthetic: Type or Structure	GCL Structural Components, Upper Geosynthetic: Weight ASTM D 3776 g/m² (oz/yd²)	GCL Structural Components, Lower Geosynthetic: Type or Structure	GCL Structural Components, Lower Geosynthetic: Weight ASTM D 3776 or Thickness ASTM D 5199 g/m² or mm (oz/yd² or mil)	Manufacturer's Suggested Applications [3]
CETCO									✱		✱ www.cetco.com	
Bentomat ST	needle-punched	4.6/45.7 (15/150)	1180 (2600)	3670 (0.75)	1 x 10[illegible]	24	18	woven	105 (3.2)	nonwoven	200 (6.0)	LL, LC, SIC
Bentomat SDN	needle-punched	4.4/45.7 (14.5/150)	1200 (2650)	3670 (0.75)	1 x 10[illegible]	24	18	nonwoven	90 (2.7)	nonwoven	200 (6.0)	LL, LC, SIC
Bentomat DN	needle-punched	4.4/45.7 (14.5/150)	1200 (2650)	3670 (0.75)	1 x 10[illegible]	24	18	nonwoven	200 (6.0)	nonwoven	200 (6.0)	LL, LC, SIC
Bentomat YSDN	needle-punched	4.4/60.9 (14.5/200)	1180 (2600)	2440 (0.50)	3 x 10[illegible]	24	18	nonwoven	90 (2.7)	nonwoven	200 (6.0)	LC
Bentomat CL	needlepunched laminated	4.6/45.7 (15/150)	1250 (2750)	3670 (0.75)	1 x 10[illegible]	24	18	FML/geotextile composite	NA	woven	105 (3.2)	LL, LC, SIC, CL, SIL
Bentomat CLT	needle-punched	4.6/45.7 (15/150)	1340 (2950)	3670 (0.75)	1 x 10[illegible]	24	18	textured FML/geotextile composite	NA	woven	105 (3.2)	LL, LC, SIC, CL, SIL
Claymax 200R	adhesive	4.6/45.7 (15/150)	1225 (2700)	3670 (0.75)	1 x 10[illegible]	24	18	nonwoven	75 (2.2)	nonwoven	90 (2.7)	LL, LC, SIC
Claymax [illegible]	adhesive	4.4/[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	18	FML/nonwoven [illegible]	NA	nonwoven	75 (2.2)	LL, LC, SIC, CL, SIL

Attachment D 1/2

(3/5/05)

GSE Lining Technology Inc. www.gseworld.com

GSE Gundseal Smooth HDPE	adhesive	5.3/61 (17.5/200)	1900 (4200)	3660 (0.75)	≤4 x 10^{11}	24	18	smooth HDPE geomembrane	0.4–2.0 mm (15–80 mil)	spunbonded geotextile	25 (0.75)	all
GSE Gundseal Textured HDPE	adhesive	5.3/55 (17.5/180)	1900 (4200)	3660 (0.75)	≤4 x 10^{11}	24	18	textured HDPE geomembrane	0.75–2.0 mm (30–80 mil)	spunbonded geotextile	25 (0.75)	all
Bentofix EC	needle-punched	4.7/46 (15.5/150)	980 (2160)	3660 (0.75)	1 x 10[illegible]	24	18	PP nonwoven	100 (3.0)	PP woven	105 (3.1)	all
Bentofix NWL	needle-punched	4.7/46 (15.5/150)	980 (2160)	3660 (0.75)	1 x 10[illegible]	24	18	PP nonwoven	200 (6.0)	PP nonwoven	200 (6.0)	all
Bentofix NSL	needle-punched	4.7/46 (15.5/150)	980 (2160)	3660 (0.75)	1 x 10[illegible]	24	18	PP nonwoven	200 (6.0)	PP woven	105 (3.1)	all
Bentofix CNSL	needlepunched, polymer-coated	4.7/46 (15.5/150)	1180 (2600)	3660 (0.75)	≤1 x 10[illegible]	24	18	PP FML/nonwoven composite	200 (6.0)	PP woven	105 (3.1)	all

[1] Flux is defined as "Flow rate/unit area" which can be converted to permeability using the equation:
Permeability = flux/hydraulic gradient
[2] Report result at a confining stress of 69 kN/m^2 (10psi) and 34 Kpa (5psi) head pressure

[3] CL = Canal liner
LL = Landfill liner
SIC = Surface impoundment cover
LC = Landfill cover
SIL = Surface impoundment liner
NP = Not provided by manufacturer
NA = Not applicable, per manufacturer

Companies were requested to provide minimum average roll values (MARV). All claims are the responsibility of the manufacturer.

GFR recommends you contact the manufacturers before making any specifying/purchasing decisions.

Attachment D31 2/2

(3) 5/10/05

...tion ...tile	Failure pressure (kPa)	Applied pressure (% of failure pressure)	Applied pressure (kPa)	Failure time (hours)
...e	140	75	100	130
		50	70	170
		25	35	260
.../m²	1750*	75	1300	10,000**
.../m²	3400*	40	1300	10,000**
...e	69	75	52	24
		50	34	42
		25	17	68
...m²	320	75	240	140
		50	160	110
		25	80	310
...m²	450	85	380	240
		70	310	390
		60	270	1000**
...m²	610	75	460	10,000**
...e	55	75	41	0.5
		50	28	2.5
		25	14	40
...m²	83	75	62	3
		50	41	12
		25	21	200
...m²	103	75	77	192
		50	52	1000**
...m²	365	75	270	10,000**

...ues using Equation 3. **Geomembrane showed signs of yield. † From short term ...cated cone puncture tests.

...RMULATION

Attachment E

...on is presented in this section based on the experimental puncture ...previous sections. The resulting equations predict the allowable ...DPE geomembranes both with and without geotextile protection.

5/5

...ure pressure based on the short term hydrostatic truncated cone test us... A series ...odification factors are then applied to correlate the truncated cone ... actual ...d conditions. The modification factors consider the stone shape, arr...ent 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/m²); 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 cont...s filaments, and polypropylene geotextiles made of staple fibers. Hence, Equatio... plies to essentially all of the polymer and fiber types used in the nonwoven ...e-punched geotextiles.

MW GTC

5/10/05 MW

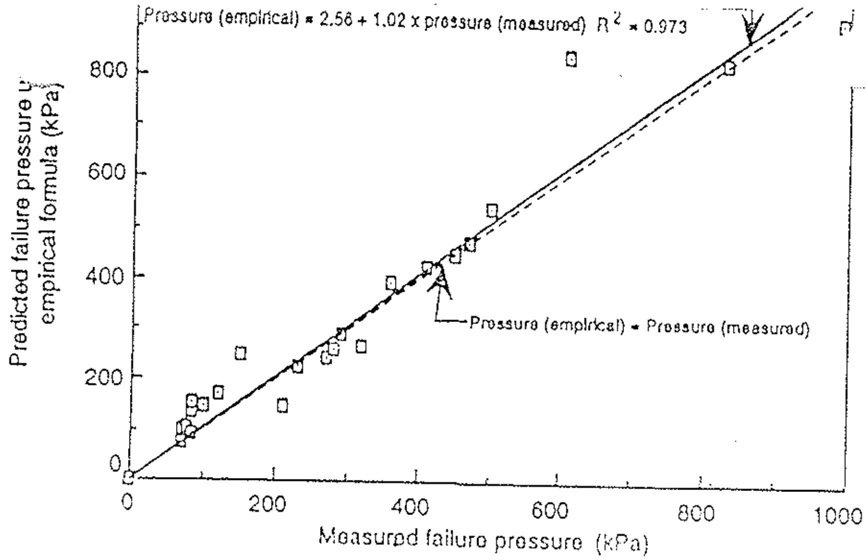


Figure 6. Measured versus empirically predicted failure pressures using Equation 3 for all nonwoven needle-punched geotextiles evaluated with a 1.5 mm thick HDPE geomembrane. Note: R = correlation coefficient.

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.

Attachment E 2/5

Stone shape	Modificati
Angular	
Subrounded	
Rounded	

6.3.2 *Modification Factor for Packing Density*

It is shown in Section 5.2.2 that the allowable pressure for higher than for isolated stones. Unfortunately, within the capac device, no failure could be achieved with the packed stones, ar relation with isolated stones could be made. However, using presented in Part I of this series of papers (Wilson-Fahmy et a yield for packed stones ($R_o/H = 2$) could be compared with the lated stones ($R_o/H = 4$) where R_o is the horizontal distance from brane point of tangency with the protrusion tip to the undeform of tangency with the soil subgrade. The analysis was performed and without protection. Based on the results, a modification fa which provides a conservative estimate of the effect of packing 4 can be rewritten after introducing a modification factor for pac

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

where MF_{PD} is the modification factor for packing density. The r sented in Table 10 can be used for isolated protrusions and pack

6.3.3 *Modification Factor for Soil Arching*

Equation 5 can be further modified as follows to include ing:

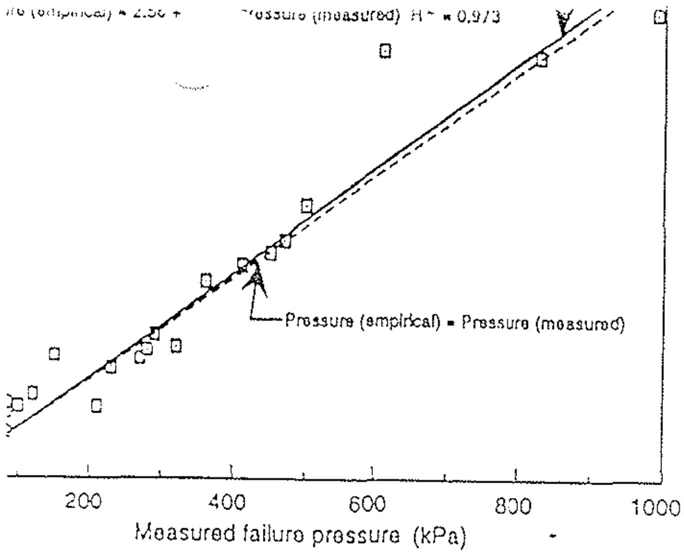
$$p'_{allow} = p_{allow}\left(\frac{1}{MF_S \times MF_{PD} \times MF_A}\right)$$

where MF_A is the modification factor for soil arching.

Table 10. Modification factors for packing density.

Protrusion arrangement	Modificati
Isolated protrusions	
Packed stones	

5/10/05



...ed versus empirically predicted failure pressures using Equation 3 for all ...unched geotextiles evaluated with a 1.5 mm thick HDPE geomembrane. ...ion coefficient.

...n Factors

...ification factors is now sequentially applied to Equation 3 in order ...ure representing field conditions. The modified pressure will be re-

...n *Factor for the Protrusion Shape*

... shown that the failure pressure depends on the protrusion shape. ...ve the highest failure pressure followed by subrounded stones. The ...ure is associated with angular stones and is approximately equal to ...of truncated cones. In order to account for the effect of stone shape, ...or is introduced into Equation 3 as follows:

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

...odification factor for the protrusion shape. Hereafter, p'_{allow} refers ...odified value of p_{allow} as is illustrated in Figure 6.
...ysis of the data presented in Section 5.2.1, the modification factors ...hapes are presented in Table 9.

Attachment E 3/5

Stone shape	Modification factor, MF
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

3 5/10/05

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 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 Greep*

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

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

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/m²)	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.

Attachment E 4/5

low in comparison with the factors of safety found in the literatu[re for geotex]tiles in tension. This may be explained by the fact that, in the pu[ncture mode, the geo]membrane and its protection material will conform more to the s[hape of the protrusion] 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 app[lied pressure, the maxi]mum stress mobilized at the protrusion tip will decrease as the u[nsupported length de]creases. Thus, a decrease in stress in the geomembrane and its [protection material is] expected with time. Accordingly, a lower factor of safety for cr[eep is warranted in the] puncture mode in comparison to the stress mode in which the m[aterial is subjected to] a constant tensile stress.

6.4.2 *Partial Factor of Safety for Chemical/Biological Degrad[ation]*

The partial factor of safety against chemical/biological deg[radation is in]cluded in Equation 7 as follows:

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

Although not assessed in this study, the value of FS_{CBD} is felt to [range from 1.0 to] 2.0 with an average value of 1.5; see Koerner (1994) for discuss[ion].

6.5 Global Factor of Safety

After determining an allowable pressure that is suitably adj[usted by modification] factors and partial factors of safety (Equation 8), a global factor [of safety is obtained] by dividing the allowable pressure by the required pressure as f[ollows]:

$$FS = \frac{p'_{allow}}{p_{reqd}}$$

where: p_{reqd} = maximum stress required on the geomembrane; a[nd FS = global] factor of safety for uncertainties related to site specific conditio[ns].

It is felt that the global factor of safety should never be less t[han ... values] may be used depending on site specific conditions. For example[, ...] should be used in situations where large isolated stones are fre[quently present in] the subgrade. Also, a tightly installed geomembrane may also [require a higher] factor of safety compared to a geomembrane installed with slac[k. ... No mo]dification has been included for in situ temperatures different f[rom the laboratory test] temperature, i.e. ≈ 20°C. More definitive recommendations f[or factors of] safety are made in Part III of this series of papers (Koerner et a[l. ...]

5/10/05 (3) MW

…r of six in comparison with hydrostatic loading. This corresponds to factor of 0.17 … be noted, however, that the effect of soil arching … at yield may … as great as the effect on the failure pressure. The …f the geomembrane up to yield may not be large enough to mobilize the …fect; therefore, caution must be exercised when using the data in Table … is recommended that the values in Table 11 be used when soil arching …

Factors of Safety

…icing the various modification factors (all of which are 1.0 or less), sever…s of safety should be applied in order to determine the allowable pressure …nbrane. The partial factors of safety are equal to 1.0 or greater. Two fac…ered below, a partial factor of safety for long term creep and a partial fac… account for long term chemical/biological degradation of the materials

Factor of Safety for Creep

…or of safety for creep is incorporated into Equation 6, and the allowable … calculated as follows:

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

…he partial factor of safety for creep. Based on the creep data presented …recommended partial factors of safety for creep are given in Table 12.

…lification factors for soil arching.

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

…ial factors of safety for creep.

	Partial factors of safety for creep			
	Protrusion height (mm)			
	38	25	12	6
	N/R	N/R	N/R	>>1.5
	N/R	N/R	>1.5	1.5
	N/R	1.5	1.3	1.2
	1.3	1.2	1.1	1.0
	~1.2	~1.1	~1.0	1.0

…ecommended.

Attachment E-5

…low in comparison with the factors of safety found in the literature for creep of geotex…s in tension. This may be explained by the fact that, in the puncture … the geo…membrane and its protection material will conform more to the subgrad… 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:

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

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:

$$FS = \frac{p'_{allow}}{p_{reqd}} \tag{9}$$

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).

5/10/05 MG

Calculation Package H
Sump Capacity

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Sump Capacity

Computations By:
SIGNATURE DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE DATE 11/3/06
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE DATE 11/3/06
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE DATE 11/3/06
Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE DATE 11/3/06
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebecca B. Flynn Date: 08 /31 /06 Reviewed by: GTC Date: 11 / 3 /06
MM DD YY MM DD YY

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 04

SUMP CAPACITY
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

The objective of this calculation is to evaluate if the sump and leachate aggregate have the capacity to hold the maximum average daily volume (evaluated by the HELP model in the drainage pipe sizing calculation) without exceeding the 12-inch head requirement.

SUMMARY OF ANALYSIS

The HELP model suggest that the maximum, average daily volume collected by the leachate collection system is 231 cubic feet. The capacity of the sump and a "pool" of leachate 1 ft deep within the leachate collection system is 453 ft^3. Therefore, the sump and adjacent drainage geocomposite capacity exceeds the estimated maximum average daily leachate volume from the drainage pipe.

SITE CONDITIONS

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

- 2 ft of operations layer material;
- a drainage geocomposite;
- 60-mil (1.5 mm) HDPE geomembrane, textured on both sides;
- a geosynthetic clay liner (GCL); and
- prepared subgrade.

The proposed base grading plans are shown on in Figure 1. A berm will be placed around the cells and within the cells to contain and control runoff during construction and during the worst case scenario when the cell contains a small quantity of waste. A sump will be located at the low point of each cell. Each sump will be approximately 10 ft x 10ft x 2 ft deep. The grade of the lining system immediately adjacent to the sumps ranges from 2 to 3%.

The Hydrologic Evaluation of Landfill Performance (HELP) model was used to estimate the peak daily quantity of liquid expected to be generated in the drainage layer during or after a rainfall event. The HELP model is used to evaluate the worst case scenario which occurs *during the period where the cell contains a small quantity of waste*, such that any collected liquid is considered leachate, yet the majority of the cell is empty so the largest quantity of liquid will infiltrate through the operations layer to the drainage layer. The output and assumptions of the HELP analyses are presented in the pipe sizing calculation package. It is assumed that surface runoff will be controlled, therefore this sump capacity calculation package compares the capacity of the sump to the quantity of leachate collected from the drainage layer.

ANALYSIS

Each sump will be approximately 10 ft x 10ft x 2 ft deep (see attachment). The side slopes of the sumps are proposed to be inclined at 2:1 (horizontal:vertical), although the calculation conservatively assumes side slopes of 1:1. The porosity (n) of the drainage aggregate is assumed to be 0.4 (HELP value for gravel). Therefore, the capacity of each sump is:

$$V_{sump} = \text{[volume of square + volume of triangle around circumference]*porosity}$$
$$V_{sump} = [10*10*2+(2*2)/2*40]*0.4 = 112 \text{ ft}^3$$

Additional storage capacity within the drainage aggregate within the upgradient drainage collection layer is available provided that the leachate level does not exceed 12 inches over the liner (in order to remain compliant with 40 CFR 264.301(c)(2)).

The HELP model runs performed for the pipe sizing calculation indicates the following maximum average daily volumes:

Sump 1	169 ft^3
Sump 2	231 ft^3
Sump 3	65 ft^3

As Sump 2 has the largest peak daily volume of the four sumps, the capacity of the drainage aggregate will be analyzed for this sump. The plan view of Sump 2 is presented in Attachment A. For Sump 2, the slope of the drainage aggregate is assumed to be 2%. Therefore, for a 12-inch height of leachate, the edge of the "pool" of water is located at:

$$\text{distance from sump} = 1 \text{ ft} / 0.02 = 50 \text{ ft.}$$

Written by: Rebecca B. Flynn Date: 08 /31 /06 (MM DD YY) Reviewed by: GTC Date: 11/03/06 (MM DD YY)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 04

Based on the topography of Sump 2, the grading from Sump 2 varies from 1 to 2 percent from the sump (Figure 1). Therefore, the 2% slope assumption is conservative.

The limits of the "pool" of water is assumed to have a radius of 50 ft from the sump (i.e., constant 2% slope). Again, this assumption is conservative (see attachment). The area of the "pool" around the sump is approximately 1,963 ft^2 as shown in the attachment. The capacity of this volume of drainage geocomposite and operation layer is approximately 342 ft^3, based on a maximum leachate depth of 12 inches and considering the porosity of the geocomposite and operations layer. Adding the volume within the sump yields a total capacity of 453 ft^3 (112 + 342 ft^3). The geometry for the other sumps is similar to Sump 2, and can be considered adequate to handle the estimated maximum average daily leachate volume.

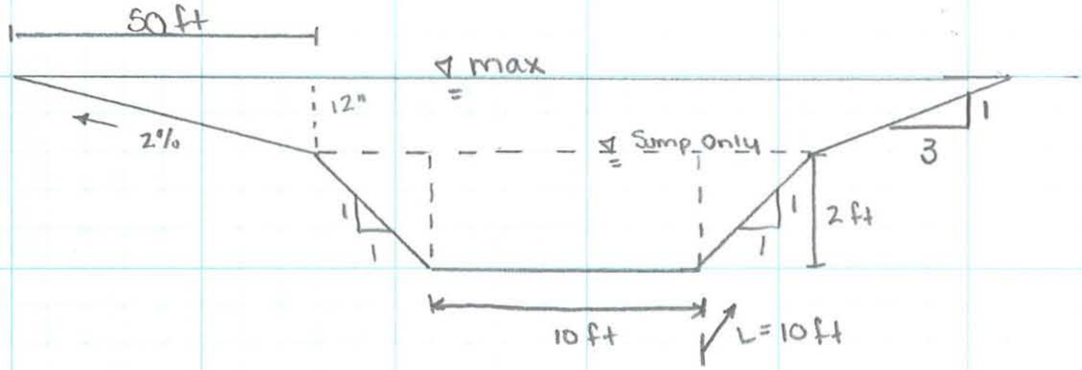
SUMMARY AND CONCLUSIONS

- The sump capacity by itself is 112 ft^3; and
- The capacity of the drainage aggregate sloping at 2% away from the sump without exceeding the 1 ft head requirement is approximately 342 ft^3.
- The capacity of the sump and drainage aggregate far exceeds the maximum average daily volume evaluated from the HELP model (pipe size calculations) without exceeding 12 inches of head on the liner.

Written by: ______ Date: __/__/__ Reviewed by: GTC Date: 3/11/06

Client: ______ Project: ______ Project/Proposal No.: ______ Task No: ______

Elevation - Sump 2



Capacity of sump only = [10' x 10' x 2' + $\frac{2' \times 2'}{2}$ x 40'] x 0.4
↑ circum. ↑ porosity

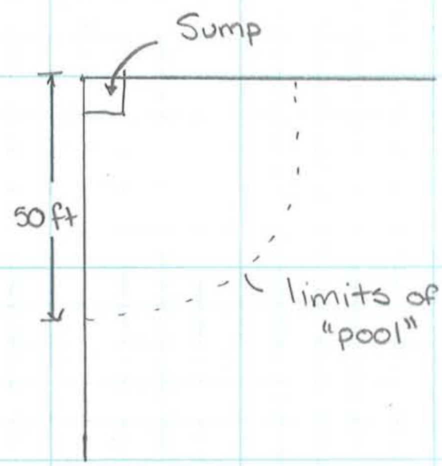
Sump Capacity: 112 ft^3

→ Since average daily flow is 231 ft^3 (HELP model) exceeds capacity, look at influence of overflow (into geocomposite and operations layer) capacity. max head on liner = 12". See next page.

Written by: ______ Date: __/__/__ Reviewed by: GTC Date: 03/11/06

Client: ______ Project: ______ Project/Proposal No.: ______ Task No: ______

PLAN SUMP 2



To be conservative, and for ease of calculation, assume radius of +1 ft contour to be 50 ft from sump. Also, add volume directly above sump.

The area of the above "pool" is:

$$A = (50\,ft)^2 \pi \; 90/360 = 1963\,ft^2$$

The average depth of the leachate is 6" (mid-point of triangle shown on page 1). Therefore, the volume is:

$$V = (1963\,ft^2)\Big[\underbrace{\big(\underset{\text{Thickness}}{0.2/12}\big)\big(\underset{\text{Porosity}}{0.8}\big)}_{\text{Geocomposite}} + \underbrace{\big(\underset{\text{leachate level}}{\tfrac{6-0.2}{12}}\big)\big(\underset{\text{Porosity}}{0.3}\big)}_{\text{Operations layer}}\Big]$$

$$V = 311\ ft^3$$

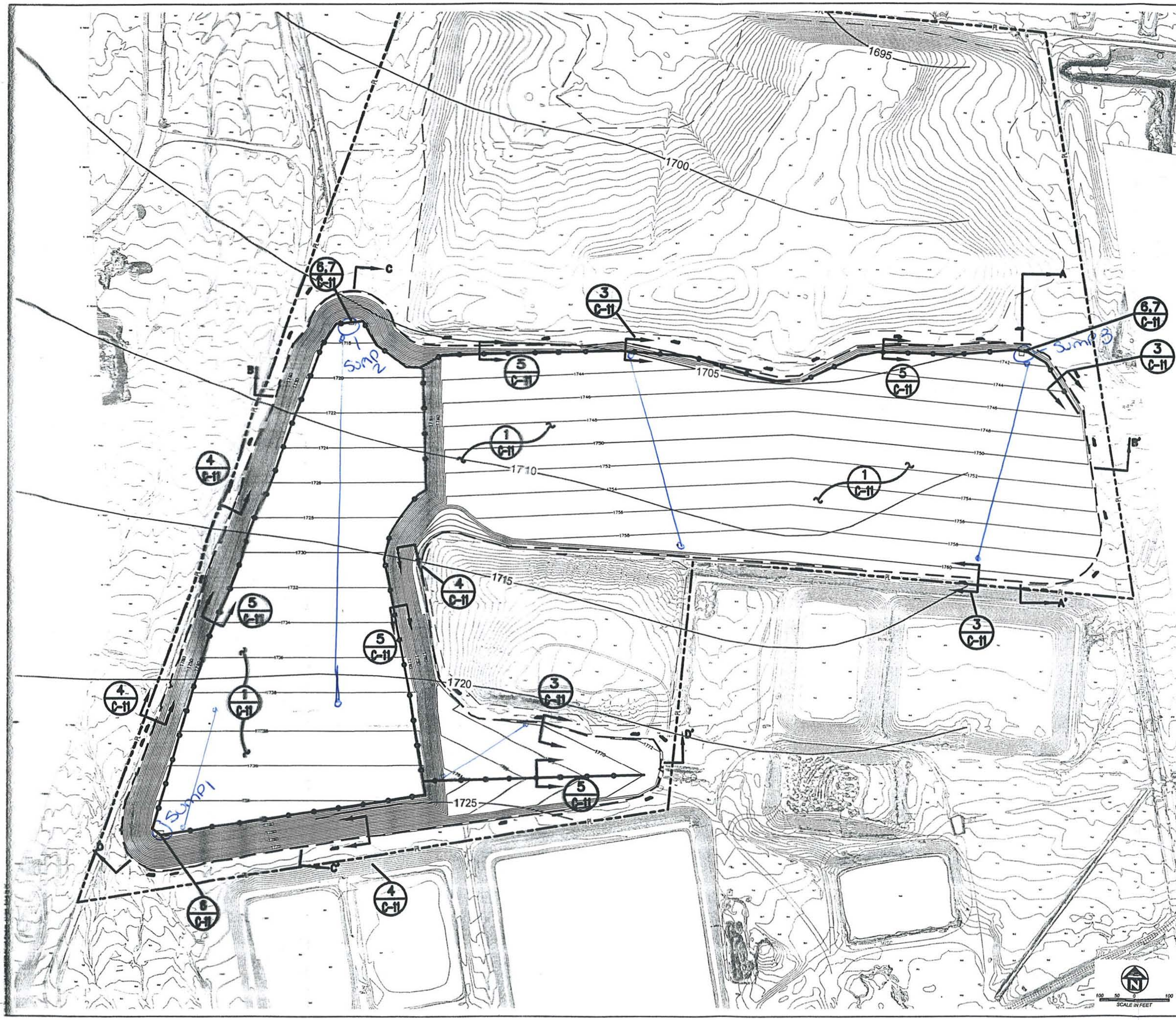
The capacity of the geocomposite and operations layer directly above the sump is:

$$V_s = (\text{length})(\text{width})\Big[\big(\tfrac{0.2}{12}\big)(0.8) + \big(\tfrac{12-0.2}{12}\big)(0.3)\Big]$$

$$V_s = 30.8\,ft^3$$

Total Volume of voids for 1 ft "pool" of leachate is:

$342\,ft^3$



LEGEND

- BRC LANDFILL SITE BOUNDARY
- EXISTING LIMITS OF FORMER BMI LANDFILL
- EXISTING SURFACE WATER FLOW PATH
- EXISTING MINOR ELEVATION CONTOUR
- EXISTING MAJOR ELEVATION CONTOUR
- PROPOSED MINOR ELEVATION CONTOUR
- PROPOSED MAJOR ELEVATION CONTOUR
- PROPOSED LIMIT OF WASTE
- LCRS PIPE
- GROUNDWATER SURFACE CONTOUR

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG C-2.
2. FOR SITE PREPARATION PLAN SEE DWG C-6.
3. FOR EROSION CONTROL AND STORM DRAIN PLAN SEE DWG C-8.
4. FOR CONSTRUCTION GRADING PLAN FOR FINAL COVER SEE DWG C-9.
5. FOR CROSSECTIONS SEE DWG C-10.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG C-11.
7. FOR FENCE PLAN SEE DWG C-12.
8. GROUNDWATER CONTOURS PROVIDED BY MWH, DATED JULY 2005.

FIGURE 1

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SAN DIEGO, CALIFORNIA 92127
TELEPHONE: (858) 674-6559

PROJECT: **BRC CAMU**
HENDERSON/CLARK COUNTY NEVADA

TITLE: **CONSTRUCTION GRADING PLAN**
PERMITTING CONSTRUCTION PLAN

DATE:	SEPTEMBER 2006	CHECKED BY:	G.T.C.	SCALE:	1"=100'	FIGURE NO:
DESIGN BY:	TAW	REVIEWED BY: (FUNA UNK)	G.T.C.	JOB NO.:	SC0313-01-04	C7
DRAWN BY:	T.L.Z.	DOCUMENT NO:		FILE NO:		

Calculation Package I
Comparison of Flow Between CCL and GCL

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 02

Title of Computations: Comparison of Flow Through Prescriptive CCL and Alternative GCL

Computations By:
SIGNATURE
13 March 2000
DATE
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
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Computations Checked By:
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26 April 2000
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Computations Backchecked By (Originator):
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3 May 2000
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Geoff L. Smith / Staff Engineer
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Approved By (PM or Designate):
SIGNATURE
10 July 2000
DATE
Bradford H. Cooley / Project Manager
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Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 01 /25 /00 (MM DD YY) Reviewed by: GTC Date: 07/03/00 (MM DD YY)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 02

COMPARISON OF FLOW THROUGH THE PRESCRIPTIVE COMPOSITE LINER AND A PROPOSED ALTERNATIVE COMPOSITE LINER

OBJECTIVE

Evaluate an alternative composite lining 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 the prescriptive 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.

SUMMARY OF ANALYSIS

The calculations suggest that the flow rate through the prescriptive composite liner system is approximately 1.64 times greater than the flow rate through the alternative composite liner system. The fluid head on the geomembrane is assumed to be 12 in (305 mm), the maximum allowed as required in regulations 40 CFR §258.40(2).

In terms of limiting flow through the composite liner system, the alternative liner system performs better than the prescriptive liner system.

ANALYSIS

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

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

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 Eq. (1) the ratio between the rate of leachate flow through the prescriptive composite liner system and the alternative composite liner system can be compared as follows (Attachment 1):

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

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

and other terms have previously been defined for Eq. (1).

EVALUATE FLOW THROUGH COMPOSITE LINER SYSTEMS

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

Properties of Compacted Clay Liner (CCL):

- k_{CCL} = 1 x 10^{-7} cm/s (the maximum hydraulic conductivity for the low-permeability soil component of the composite liner prescribed by 40 CFR §258.40(2))
- t_{CCL} = 2 ft (0.6 m) (minimum thickness prescribed by 40 CFR §258.40(2))

Properties of Geosynthetic Clay Liner (GCL):

- k_{GCL} = 5 x 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):

40 CFR §258.40(2) states that the maximum allowable liquid head on a composite liner is less than 12 in. (305 mm).

Three cases comparing the flow rates through the prescriptive and alternative composite liner systems are evaluated using Eq. (2). The calculation is presented below:

Calculation:

Plugging the above values for Case 1, Eq. (2) becomes as follows:

$$q_{comp\ CCL} / q_{comp\ GCL} = (1 \times 10^{-7} / 5 \times 10^{-9})^{0.74} [(1+0.1(12.0/24.0)^{0.95}) / (1+0.1(12.0/0.20)^{0.95}]$$
$$= 1.64$$

Thus, for a liquid head of 12.0 in. (305 mm) on the geomembrane, the flow through the prescriptive composite liner system that includes a CCL is 1.64 times greater than the flow through the alternative 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 prescriptive liner system (with CCL) was evaluated to be greater than the flow rate through the proposed alternative liner system (with GCL instead of CCL).

- The amount of flow through the prescriptive liner system was evaluated to be 1.64 times greater than flow through the alternative liner system for a leachate head of 12 in. (305 mm).

- In terms of limiting fluid flow through the composite liner system, the alternative liner system performs better than the prescriptive liner system.

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 1)

ATTACHMENT A
P 1/2 MLD

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:

$$\boxed{Q = 0.21\,[1 + 0.1(h/t)^{0.95}]\,a^{0.1}\,h^{0.9}\,k^{0.74}} \qquad (51)$$

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:

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

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

ATTACHMENT A
P 2/2 MLD

in the composite liner; and N = number of geomembrane defects per unit area. After simplification, Equation 52 becomes:

$$\boxed{\frac{q_{comp\,CCL}}{q_{comp\,GCL}} = \left(\frac{k_{CCL}}{k_{GCL}}\right)^{0.74} \frac{1 + 0.1(h/t_{CCL})^{0.95}}{1 + 0.1(h/t_{GCL})^{0.95}}} \tag{53}$$

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.

Calculation Package J
Tension Due to Wind Uplift

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 02

Title of Computations: Geotextile Tension due to Wind Uplift

Computations By:
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28 June 2000
DATE
Geoff L. Smith / Staff Engineer
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Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 00 / 01 / 26 (YY MM DD) Reviewed by: GTC Date: 00 / 07 / 03 (YY MM DD)

Client: Parsons Project: BRC Landfill Project/Proposal No.: HL0389 Task No.: 01

GEOTEXTILE TENSION DUE TO WIND UPLIFT
BRC LANDFILL
HENDERSON, NEVADA

OBJECTIVE

Evaluate tension in a woven geotextile due to wind uplift. Use method outlined by Giroud, et al (1995). Tension generated by wind uplift will be compared to the anchor trench capacity and the ultimate strength of the woven geotextile.

SITE CONDITIONS

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

- woven geotextile U.V. protection layer
- geocomposite;
- 60-mil HDPE geomembrane
- GCL

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 A). Giroud et al offer the following equation for estimating the effective suction on a geomembrane (Attachment A):

$$S_E = 0.050\lambda V^2 e^{-[1.252E-4]z} - 9.81M_{GM} \qquad \text{(Equation 1)}$$

where: S_E = effective suction (P_a)
λ = suction factor (dimensionless)
V = wind velocity (km/h)
z = altitude above sea level (m)
M_{GM} = mass per unit area of geomembrane (kg/m^2)

Evaluate Variables

λ Suction factor = 0.85 for the upper third of the slope being considered

V wind velocity = 90 mph = 145 km/h (the peak gust ever recorded after 1961, see Attachment B)

z altitude above sea level (m). A representative elevation for the base side slopes is approximately 1760 ft = 536 m

M_{GM} mass per unit area of geotextile (kg/m^2)
$M_{GM} = 0.15\ kg/m^2$ (Attachment C)

Evaluate Suction

$S_E = (0.05)(0.85)(145)^2\ e^{-(1.252E-4)536} - 9.81(0.15) = 834$ Pa

The height of the exposed slope (2.1H:1V) after the first phase of operations layer is placed is 20 vertical feet, so the total length of exposed slope, L, is $L/\sin\beta = 46.5$ ft = 14.2 m

$S_E L = (821)(14.2) = 11.84$ kN/m

Evaluate Tension

The objective of this case is to evaluate tension in the geomembrane. Assuming the geomembrane stress-strain curve is linear, the tension can be expresses as:

$$T = J\varepsilon$$

where: T = tension
J = stiffness
ε = strain

To evaluate the tension in the geomembrane, the stiffness and stain must be evaluated.

Stiffness, J

For strains typically less than 10 percent, the stress-strain curve for woven geotextiles is approximately linear. Therefore, stiffness can be approximated as:

$$J = ET_{GT}$$

where: E = elastic modulus = 1535 kN/m (Koerner 1994)
T_{GM} = geotextile thickness, approximately 5 mils (Koerner 1994)

$$J = (1535\ \text{MPa})(0.000127\ \text{m}) = 195\ \text{kN/m}$$

Strain, ε

The strain on the woven geotextile induced by wind uplift loading can be estimated using the table by Giroud et al. (Attachment A).

$$\text{For } \frac{J}{S_E L} = \frac{195}{11.84} = 16.7$$

from Table 4, ε = 5.57%

The estimated strain in the woven geotextile is 5.57% percent, so the assumption of a linear stress-strain relationship is valid.

Evaluate tension

$$\mathbf{T} = 195\frac{5.57\%}{100} = \mathbf{10.9\ \ kN/m}$$

GEOTEXTILE ALLOWABLE TENSION

Since the woven geotextile (for U.V. protection) will be subjected to wind uplift forces, the woven geotextile will be designed for tension.

The estimated tensile force imposed on the geomembrane due to wind uplift suction is 14.5 kN/m for a design wind speed of 90 mph (145 kph). To account for uncertainties in the woven geotextile ultimate strength and wind speed, a factor of safety of 1.5 will be employed.

Therefore, the required ultimate strength of the geotextile shall be greater than or equal to 16 kN/m (91 ppi).

CONCLUSIONS

Based on the results herein, the woven geotextile U.V. protection layer shall have a wide width ultimate tensile strength greater than or equal to 16 kN/m (91 ppi).

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., pp. 897-952
Attachment A

Western Regional Climate Center, Summary of Las Vegas Climate
www.wrcc.dri.edu
Attachment B

AB GTC

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

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~~ ~~A~~ ~~Page~~ ~~2/8~~

Attachment A 1/3

Client

• 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/m²)

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

with S_e(Pa), V(km/h), z(m), μ_{GM}(kg/m²)

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:

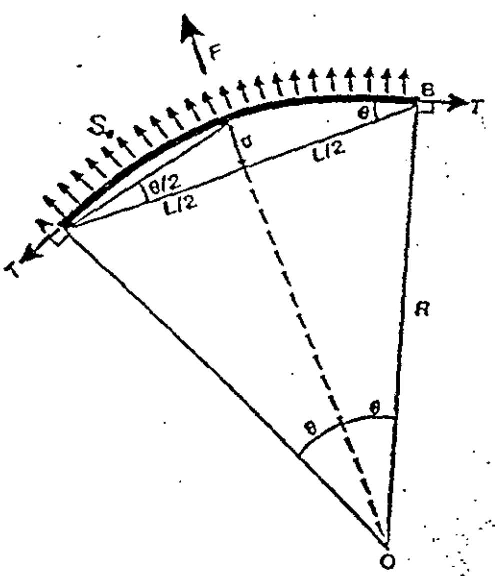

Figure 9. Schematic representation of uplifted geomembrane used for developing equations.

Attachment 2A2 2 pages

Attachment A 2/3

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

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

ATTACHMENT A
3/3

LAS VEGAS, NEVADA

to GTC

NORMALS, MEANS, AND EXTREMES

LATITUDE: 36 Deg. 02 Min. N LONGITUDE: 115 Deg. 11 Min. W ELEVATION: FT. GRND 2286 BARO 2179 TIME ZONE: PACEFIC WBAN: 23169

	(a)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
TEMPERATURE (Deg. F)										
Normals										
-Daily Maximum		57.3	63.3	68.8	77.5	87.8	100.3	105.9	103.2	94.7
-Daily Minimum		33.6	38.8	43.8	50.7	60.2	69.4	76.2	74.2	66.2
-Monthly		45.5	51.1	56.3	64.1	74.0	84.9	91.1	88.7	80.5
Extremes										
-Record Highest	47	77	87	91	99	109	115	116	116	113
-Year		1975	1986	1966	1981	1951	1994	1985	1979	1950
-Record Lowest	47	8	16	23	31	40	48	60	56	46
-Year		1963	1989	1971	1975	1964	1993	1987	1968	1965
NORMAL DEGREE DAYS										
Heating (base 65 Deg. F)		605	389	292	143	14	0	0	0	0
Cooling (base 65 Deg. F)		0	0	22	116	293	597	809	735	465
% OF POSSIBLE SUNSHINE	46	77	81	83	87	88	93	88	88	91
MEAN SKY COVER(tenths)										
Sunrise - Sunset	47	4.9	4.8	4.6	3.9	3.5	2.1	2.8	2.5	2.1
MEAN NUMBER OF DAYS:										
Sunrise to Sunset										
-Clear	47	13.6	12.3	13.6	15.6	17.7	22.3	20.1	21.6	22.5
-Partly Cloudy	47	6.3	6.8	8.4	7.7	8.0	5.2	7.5	6.7	5.0
-Cloudy	47	11.1	9.1	9.0	6.8	5.3	2.5	3.4	2.8	2.5
Precipitation										
.01 inches or more	47	3.3	2.8	3.2	1.7	1.4	0.7	2.6	2.8	1.6
Snow, Ice Pellets, Hail										
1.0 inches or more	47	0.3	0.*	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Thunderstorms	47	0.*	0.2	0.4	0.5	1.0	1.0	3.9	3.9	1.5
Heavy Fog Visibility										
1/4 mile or less	47	0.3	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.*
Temperature Deg. F										
-Maximum										
90 Deg. F and above	35	0.0	0.0	0.*	3.3	15.1	25.7	30.5	29.9	22.4
32 Deg. F and below	35	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-Minimum										
32 Deg. F and below	35	11.9	4.2	1.1	0.1	0.0	0.0	0.0	0.0	0.0
0 Deg. F and below	35	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AV. STATION PRES. (mb)	23	942.4	940.7	937.3	935.9	933.9	933.6	934.9	935.5	936.0
RELATIVE HUMIDITY (%)										
Hour 04	35	56	51	46	35	32	25	28	34	33
Hour 10 (Local Time)	35	42	37	31	22	19	15	19	23	22
Hour 16	35	32	27	23	16	14	11	14	17	17
Hour 22	35	50	43	37	26	23	17	22	26	26
PRECIPITATION (in.)										
Water Equivalent										
-Normal		0.48	0.49	0.42	0.21	0.28	0.12	0.35	0.49	0.28
-Maximum Monthly	47	3.00	2.52	4.80	2.44	0.96	0.97	2.48	2.59	1.58
-Year		1995	1993	1992	1965	1969	1990	1984	1957	1963

Attachment B
1/2

SW BTC

-Minimum Monthly	47	T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-Year		1984	1977	1972	1962	1970	1982	1981	1980	1971
-Maximum in 24 hrs	47	1.09	1.30	1.27	0.97	0.83	0.97	1.36	2.59	1.07
-Year		1990	1993	1992	1965	1987	1990	1984	1957	1963
Snow, Ice Pellets, Hail										
-Maximum Monthly	47	16.7	1.4	0.1	T	0.0	0.0	0.0	T	0.0
-Year		1949	1990	1976	1970				1989	
-Maximum in 24 hrs	7 9	.0 6	.9 0	.1 T	0	.0 0	.0 0	.0 T	0	.0 T
-Year		1974	1979	1976	1970				1989	
WIND										
Mean Speed (mph)	47	7.4	8.5	10.2	11.0	11.1	11.1	10.3	9.6	9.0
Prevailing Direction through 1964		W	SW	SW	SW	SW	SW	SW	SW	SW
Fastest Mile										
-Direction(!!)	10	23	23	23	22	32	34	02	14	22
-Speed(mph) Avg. 44 mph		30	50	51	49	53	48	44	40	35
-Year		1987	1989	1989	1988	1991	1989	1994	1989	1989
Peak Gust										
-Direction(!!)	12	SW	NW	NW	W	NW	NE	NE	SE	SW
-Speed(mph) Avg 68	12	54	67	82	69	72	59	71	90	49
-Date		1987	1984	1994	1988	1991	1984	1994	1989	1989

(a) - Length of Record in Years, although individual months may be missing.
0.* or * - The value is between 0.0 and 0.05.
Normals - Based on the 1961 - 1990 record period.
Extremes - Dates are the most recent occurrence.
Wind Dir.- Numerals show tens of degrees clockwise from true north. "00" indicates calm.
Resultant Directions are given to whole degrees.

90 mph

Attachment B
2/2

Attachment C

MR GTC

GEOTEXTILES

Product Name (Structure [1]/ Polymer Type [2])	Mass Per Unit Area ASTM D 5261 g/m² (oz/yd²)	M288 Transportation-Related Applications: Filtration/Hydraulic Properties: Percent Open Area CWO-22125 %	Apparent Opening Size ASTM D 4751 mm (U.S. sieve)	Permittivity ASTM D 4491 sec⁻¹ Flow Rate (FH or CH) [3] l/min/m² (gal/min/ft²)	Physical Properties: Puncture ASTM D 4833 kN (lb)	Trapezoid Tearing Strength ASTM D 4533 kN (lb)	Grab Tensile/ Elongation ASTM D 4632 kN (lb)/%	M288 Survivability Class	M288 Applications [4]	Reinforcement Applications: Wide Width Tensile Properties ASTM D 4595 kN/m (lb/in.): Strength @ 5% Strain [5] MD	Strength @ 5% Strain [5] XD	Ultimate Strength % (T_{ult}) [5],[6] MD	Ultimate Strength % (T_{ult}) [5],[6] XD	Creep Limited Strength ASTM D 5262 [6], [7] kN/m (lb/ft)	T_{allow} GRI GT7 (in sand) [6]	Other Manufacturer's Suggested Applications [8]
Maccaferri Gabions Inc. (cont.)																
MacTex MXF2 (W-SF-PP)	95 (2.9)	<1 [A]	0.850 (20)	0.2/610 (15), FH	0.255 (58)	0.220x0.220 (50x50)	0.445x0.445 (100x100)/15x15	--	S/F	NA	NA	NA	NA	NA	NP	S/F
MacTex MXM10 (W-PP)	200 (5.9)	4 [A]	0.212 (70)	0.28/730 (18), CH	0.535 (120)	0.445x0.310 (100x70)	1.645x1.110 (370x250)/24x24	--	E, F, D	NA	NA	NA	NA	NA	NP	F
MacTex MXM20 (W-PP)	190 (5.7)	11 [A]	0.600 (30)	1.10/4480 (110), CH	0.510 (115)	0.510x0.335 (115x75)	1.845x0.980 (370x220)/25x15	--	E, F, D	NA	NA	NA	NA	NA	NP	E, F, D
MacTex MXW9 (W-SF-PP)	150 (4.5)	<1 [A]	0.425 (40)	0.07/240 (6), FH	0.445 (100)	0.330x0.330 (75x75)	0.890x0.890 (200x200)/15x15	--	SP, ST	8.7 (50)	9.6 (55)	21/9 (120)	21/7 (120)	NA	NP	ST, SP, R
MacTex MXW13 (W-SF-PP)	220 (6.5)	<1 [A]	0.212 (70)	0.06/200 (5), FH	0.555 (125)	0.530x0.530 (120x120)	1.400x1.400 (315x315)/15x15	--	SP, ST	8.7 (50)	10.5 (60)	30.6/10 (175)	30.6/8 (175)	NA	NP	ST, SP, R
Serrot International Inc./Fluid Systems																
Trevira 011/140	136 (4.0)	NA	0.3 (50)	2.01/102 (150)	0.222 (50)	0.178 (40)	0.489 (110)/60	NA	NA	NA	NA	NA	NA	NA	NA	SP, ST, F, A/O, D, E
Trevira 011/120	193 (5.7)	NA	0.21 (70)	1.74/88 (130)	0.356 (80)	0.267 (60)	0.711 (160)/60	NA	NA	NA	NA	NA	NA	NA	NA	SP, ST, F, A/O, D, E

Calculation Package K
Anchorage Capacity

GeoSyntec Consultants

GeoSyntec Consultants
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 02

Title of Computations: Evaluation of Geosynthetic Anchorage Capacity

Computations By:
SIGNATURE ______ DATE 28 June 2000
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE ______ DATE 7/3/00
GREGORY T CORCORAN/PROF. ENG.
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Computations Checked By:
SIGNATURE ______ DATE 7/3/00
GREGORY T. CORCORAN/PROF. ENG.
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Computations Backchecked By (Originator):
SIGNATURE ______ DATE 28 June 2000
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE ______ DATE 10 July 2000
Bradford H. Cooley / Project Manager
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GY Date: 01 / 27/ 00 (MM DD YY) Reviewed by: GTC Date: 07 / 03 / 00 (MM DD YY)

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 01

EVALUATION OF LINER SYSTEM ANCHOR CAPACITY
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

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 U.V. protective woven geotextile. One liner termination detail will be evaluated for the BRC CAMU.

SITE CONDITIONS

The proposed design for the BRC CAMU consists of a trench located a minimum of 3 ft from the crest of the slope. The width and depth of the trench is 2 ft and 2.5 ft, respectively. During the period of wind uplift exposure, there will be a 1 ft (0.3 m) high overburden placed above the anchor trench. The geosynthetic anchorage design is presented in Figure 1.

METHOD OF ANALYSIS

Koerner (1994) presents design equations developed from static equilibrium to evaluate the allowable geosynthetic tension from an anchor trench. 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:

$$T_{allow} = F_U + F_L + F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2} \qquad (1)$$

Where: T_{allow} = Allowable tensile force in the geotextile;

F_U = Friction force above the geotextile;

F_L = Friction force below the geotextile;

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

q = surcharge pressure due to soil overburden

δ = minimum friction angle between liner system interfaces and the soil

L_{RO} = Runout length

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

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

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

d_{AT} = depth of the anchor trench

K_o = coefficient of earth pressure at rest, = $(1\text{-}\sin\phi)$

σ_v = vertical overburden stress (at average depth of the anchor trench)

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

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

L_{AT} = width of the anchor trench

ANALYSIS

Evaluating Variables

Since tension will develop in the U.V. protective layer (see the calculation package *Tension due to Wind Uplift*), frictional forces of the U.V. protective layer woven geotextile will be mobilized along the geosynthetic and soil interfaces on the side and base of the anchor trench. For the analysis presented herein:

- A friction angle of 8 degrees will be used to represent the friction angle between the geocomposite (non-woven geotextile side) and the woven geotextile. (Attachment 1 presents a typical friction angle between a non-woven geotextile and a *smooth* HDPE sheet as 8 degrees. The interface between a woven geotextile and the non-woven geotextile will be higher due to the slit film and raised surfaces of the woven geotextile. Therefore, it is conservative to assume a friction angle of 8 degrees);

- A friction angle of 26 degrees to represent the soil/ woven geotextile interface. (Attachment 1 presents typical efficiency values between woven and soil interfaces. The typical efficiency is on the order of 85 percent. The anchor trench soil compacted to 90 % modified proctor will have a friction angle on the order of 30 degrees - see final slope stability calculation package. Therefore, the interface friction angle is $\tan^{-1}(0.85*\tan(30))$=26 deg.).

For determination of the surcharge due to soil overburden, q, and the vertical and horizontal overburden stresses, σ_h and σ_v, a unit weight of overburden soil of 136 pcf was assumed (γ = 136 pcf). For evaluation of the effective horizontal overburden stress based on the coefficient of earth pressure at rest, a friction angle of 30° was assumed for the cover soil.

Analysis

From Equation (1):

$$T_{allow} = F_U + F_L + F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2}$$

$$\begin{aligned} F_U &= q\tan\delta(L_{RO}) \\ &= 1\text{ ft }(136\text{ pcf})\tan 8°(2\text{ ft}) \\ &= 38.2\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_L &= q\tan\delta(L_{RO}) \\ &= 1\text{ ft }(136\text{ pcf})\tan 26°(2\text{ ft}) \\ &= 132.7\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_{AT\text{-}SIDE1} &= (\sigma_h)_{ave}\tan\delta(d_{AT}) = Ko(\sigma_v)_{ave}\tan\delta(d_{AT}) \\ &= (1-\sin 30^o)(2.25\text{ ft})(136\text{ pcf})\tan 8^o(2.5\text{ ft}) \\ &= 53.8\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_{AT\text{-}SIDE2} &= (\sigma_h)_{ave}\tan\delta(d_{AT}) = Ko(\sigma_v)_{ave}\tan\delta(d_{AT}) \\ &= (1-\sin 30^o)(2.25\text{ ft})(136\text{ pcf})\tan 26^o(2.5\text{ ft}) \\ &= 186\text{ lb/ft} \end{aligned}$$

$$\begin{aligned} F_{AT\text{-}BASE1} &= q\tan\delta(L_{AT}) \\ &= (3.5\text{ft})(136\text{ pcf})(\tan 8^o)(2\text{ ft}) \\ &= 133.8\text{ lb/ft} \end{aligned}$$

$$F_{AT\text{-}BASE2} = q\tan\delta(L_{AT})$$
$$= (3.5\text{ft})(136\text{ pcf})(\tan 26^\circ)(2\text{ ft})$$
$$= 464\text{ lb/ft}$$

$$T_{allow} = F_U + F_L + F_{AT\text{-}SIDE1} + F_{AT\text{-}SIDE2} + F_{AT\text{-}BASE1} + F_{AT\text{-}BASE2}$$
$$= 38.2 + 132.7 + 53.8 + 186 + 134 + 464$$
$$\boldsymbol{T_{allow} = 1{,}008\text{ lb/ft} = 14.7\text{ kN/m}}$$

CONCLUSIONS

The allowable tensile capacity of the anchorage system as calculated herein (as $\boldsymbol{T_{allow}}$) 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 10.9 kN/m. Since the design wind speed is the maximum peak gust recorded (90 mph), the required factor of safety against anchorage pullout is greater than 1.0. Therefore, the anchor capacity is adequate.

Based on the methods employed herein, results of analysis indicates that the design anchorage evaluated provides adequate tensile capacity to resist the U.V. protective geotextile tension induced by wind uplift forces.

REFERENCES

Koerner, R.M. (1994), "*Designing with Geosynthetics*," 3rd Edition., Englewood Cliffs, NJ, Prentice Hall
(Attachment 1)

Written by: JW Date: __/__/__ (YY MM DD) Reviewed by: GTC Date: 00/07/03 (YY MM DD)

Client: ________ Project: ________ Project/Proposal No.: ________ Task No.: ________

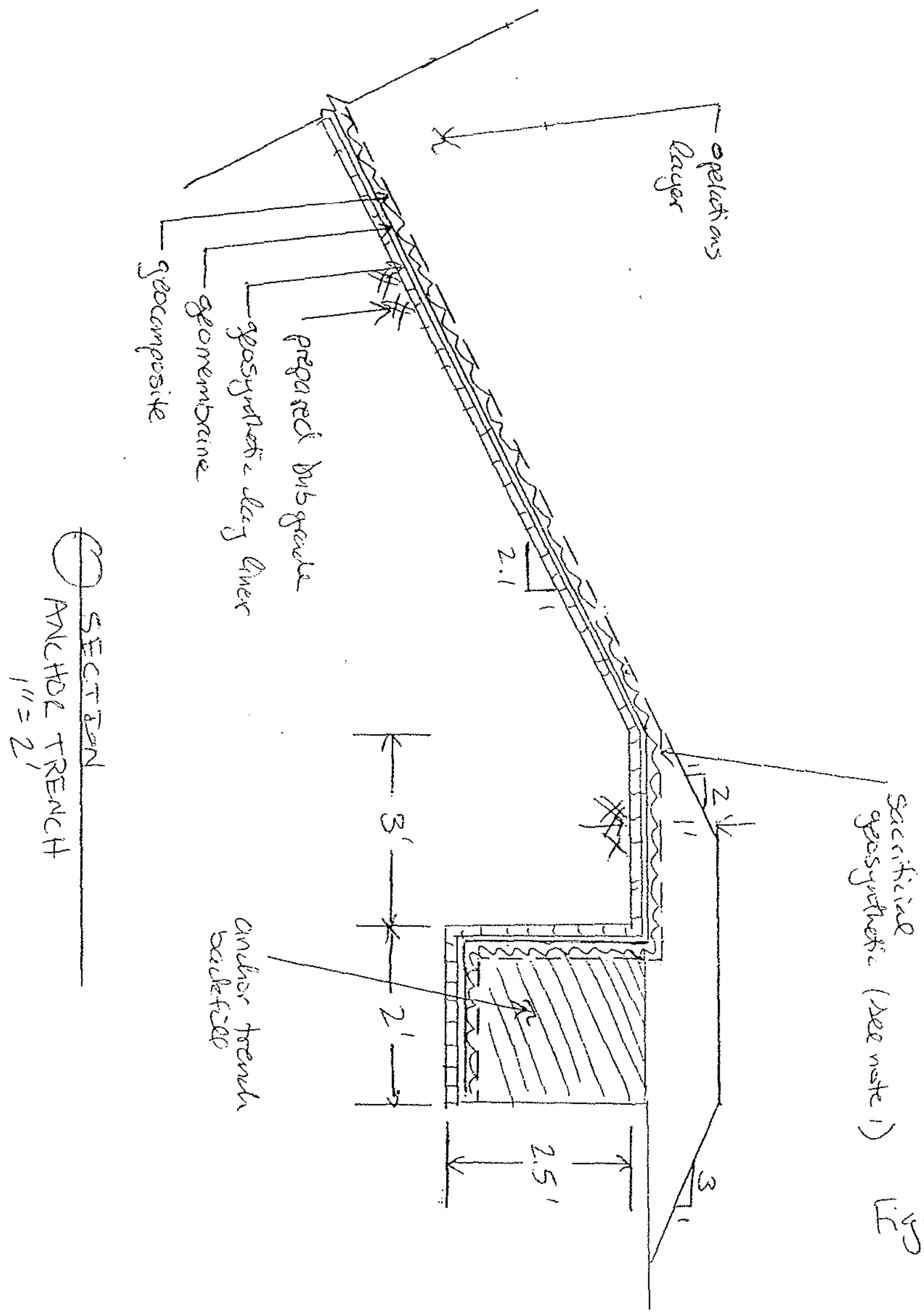

Figure 1

rg GTC

Solution: From the design equations just presented,

$$T \cos \beta = 350\ (144)(0.030/12) \cos 18.4$$
$$= 120 \text{ lb./ft.}$$
$$T \sin \beta = 39.8 \text{ lb./ft.}$$
$$q_L = d_{CS}\, \gamma_{CS}$$
$$= (1.0)\ (100)$$
$$= 100 \text{ lb./ft.}^2$$

which, when substituted into Equation 5.27, gives

$$T \cos \beta = q_L \tan \delta\ (L_{RO}) + T \sin \beta \tan \delta$$
$$120 = 100 \tan 20\ (L_{RO}) + 39.8 \tan 20$$
$$120 = 36.4 L_{RO} + 14.5$$
$$L_{RO} = 2.9 \text{ ft.; use } 3.0 \text{ ft.}$$

Note that this value is strongly dependent on the value of mobilized allowable stress used in the analysis. To mobilize the full strength of the geomembrane would require a longer runout length or 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. To establish static equilibrium, Daniel [41] has suggested using an imaginary and frictionless pulley as shown in Figure 5.31, which allows for the geomembrane to be considered in its continuous form.

$$T_{\text{allow}} = F_U + F_L + 2\ F_{AT} \qquad (5.28)$$

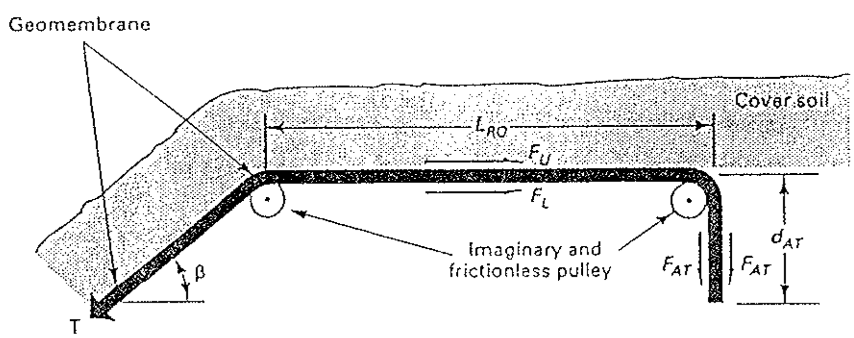

Figure 5.31 Cross section of geomembrane runout section with anchor trench and related stresses and forces involved.

Attachment A 1/3

AN GTC

where $T_{\text{allow}} = \sigma_{\text{allow}}\, t$, in which
σ_{allow} = the mobilized allowable geomembrane stress = σ_{ult}/FS,
σ_{ult} = the ultimate geomembrane stress (e.g., yield or break),
FS = the factor of safety, and
t = the geomembrane thickness,
F_U = the friction force above geomembrane (assumed to be negligible, since the cover soil probably moves along with the liner as it deforms),
$F_L = q \tan \delta\, (L_{RO})$, in which
q = the surcharge pressure = $d_{CS}\, \gamma_{CS}$,
d_{CS} = the depth of cover soil,
γ_{CS} = the unit weight of cover soil,
δ = the friction angle between geomembrane and soil, and
L_{RO} = (unknown) length of runout, and
$F_{AT} = (\sigma_h)_{\text{ave}} \tan \delta\, (d_{AT})$, in which
σ_h = the average horizontal stress in anchor trench = $K_O\, \sigma_V$,
$\sigma_V = \gamma\, H_{\text{ave}}$,
γ = the unit weight of backfill soil,
H_{ave} = the average depth of anchor trench (requires an estimate),
$K_O = 1 - \sin \phi$,
ϕ = the angle of shearing resistance of backfill soil, and
d_{AT} = the (unknown) depth of anchor trench.

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 (i.e., T_{allow}). An example illustrates the procedure.

Example:

Repeat the previous problem of a 30-mil VLDPE geomembrane of allowable stress 350 lb./in.^{2} on a 3(H) to 1(V) side slope. There is a 12-in. cover soil placed over the geomembrane weighing 100 lb./ft.^{3} (also the unit weight of the backfill soil). The friction angle between the liner and soil is 20 deg. and the soil itself is 30 deg. Determine the required length of runout for a 12-in.-deep anchor trench and for a zero-depth anchor trench to check the preceding example.

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

$$T_{\text{allow}} = F_U + F_L + 2F_{AT}$$

$$\sigma_{\text{allow}}(t) = 0 + q \tan\delta\,(L_{RO}) + 2(K_O \sigma_{V_{\text{ave}}}) \tan\delta(d_{AT})$$

$$(350 \times 144)\left(\frac{0.030}{12}\right) = 0 + (1.0)(100)\tan 20(L_{RO}) + 1.5(0.5)(2.0 \times 100)\tan 20(d_{AT})$$

$$126 = 36.4L_{RO} + 56.6d_{AT}$$

Attachment A
2/3

 YY GTC

Table 5.7 Friction values and efficiencies (in parentheses) for (a) soil-to-geomembrane, (b) geomembrane-to-geotextile, and (c) soil-to-geotextile combinations*

(a) Soil-to-Geomembrane Friction Angles

	Soil Types		
Geomembrane	*Concrete Sand* ($\phi = 30°$)	*Ottawa Sand* ($\phi = 28°$)	*Micha Schist Sand* ($\phi = 26°$)
EPDM-R	24° (0.77)	20° (0.68)	24° (0.91)
PVC			
Rough	27° (0.88)	—	25° (0.96)
Smooth	25° (0.81)	—	21° (0.79)
CSPE-R	25° (0.81)	21° (0.72)	23° (0.87)
HDPE – Smooth	18° (0.56)	18° (0.61)	17° (0.63)

(b) Geomembrane-to-Geotextile Friction Angles

	Geomembrane				
		PVC			
Geotextile	*EPDM-R*	*Rough*	*Smooth*	*CSPE-R*	*HDPE*
Nonwoven, needle punched	23°	23°	21°	15°	8°
Nonwoven, heat bonded	18°	20°	18°	21°	11°
Woven, monofilament	17°	11°	10°	9°	6°
Woven, slit film	21°	28°	24°	13°	10°

(c) Soil-to-Geotextile Friction Angles

	Soil Types		
Geotextile	*Concrete Sand* ($\phi = 30°$)	*Ottawa Sand* ($\phi = 28°$)	*Mica Schist Sand* ($\phi = 26°$)
Nonwoven, needle punched	30° (1.00)	26° (0.92)	25° (0.96)
Nonwoven, heat bonded	26° (0.84)	—	—
Woven, monofilament	26° (0.84)	—	—
Woven, slit film	24° (0.77)	24° (0.84)	23° (0.87)

*Efficiency values in parentheses are based on the relationship $E = (\tan \delta)/(\tan \phi)$.

Source: After Martin et al. [14].

The frictional behavior of geomembranes placed on clay soils is of considerable importance in the composite liners of waste landfills. Current requirements are for the clay to have a hydraulic conductivity equal to or less than 2×10^{-7} ft./min. (1×10^{-7} cm/sec.) and for the geomembrane to be placed directly on the clay. While an indication of the shear strength parameters has been investigated (e.g., reference 15), the data are so sensitive to the variables listed previously that site-specific and material-specific tests should always be performed. In such cases, literature values should never be used for final design purposes.

5.1.3.9 Geomembrane Anchorage In certain problem situations a geomembrane might be sandwiched between two materials and then tensioned by an external force. The termination of a geomembrane liner within an anchor trench is such a situation. To simulate this behavior in a laboratory environment, one can use an 8.0-in. (200-mm)-wide geomembrane sandwiched between back-to-back channels.

Attachment A 3/3

Koerner, R.M. (1995), Designing with Geosynthetics, Third Edition, Prentice Hall, N.J.

Calculation Package L
Geocomposite Equivalency Demonstration

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 02

Title of Computations: Drainage Geocomposite Equivalency Demonstration

Computations By:

SIGNATURE (GLS)

28 June 2000
DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE

7/3/00
DATE

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PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE

7/3/00
DATE

GREGORY T. CORCORAN / PROJ. ENG.
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Computations Backchecked By (Originator):

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28 June 2000
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Geoff L. Smith / Staff Engineer
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Approved By (PM or Designate):

SIGNATURE

10 July 2000
DATE

Bradford H. Cooley / Project Manager
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 00 / 06 / 28 (YY MM DD) Reviewed by: GTC Date: 00 / 07 / 03 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 02

DRAINAGE COMPOSITE EQUIVALENCY DEMONSTRATION
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

The objective of this analysis is to evaluate the hydraulic performance of a drainage composite, and compare it to the prescriptive leachate collection layer, consisting of drainage aggregate, within the BRC CAMU. A drainage composite, consisting of two 8 oz/sy nonwoven geotextiles bonded to either side of a geonet, is proposed. This analysis will demonstrate equivalence or performance exceedance of the drainage composite to the prescriptive 1-foot thick (0.3m) aggregate drainage layer. The method of analysis will compare the current transmissivity of the aggregate drainage layer and the equivalent transmissivity of the drainage composite.

SUMMARY OF ANALYSIS

The calculations suggest that a drainage composite having a transmissivity of $6.1*10^{-5}$ m^2/sec, at a maximum stress of 12,000 psf (574 kN/m^2) and a hydraulic gradient of ~~0.02~~, will provide equivalence to the aggregate drainage layer. 0.10 GLS

METHOD OF ANALYSIS

The analysis was performed using procedures recommended by Koerner (1994). The procedure first evaluates the flow rate (transmissivity) through the aggregate drainage layer, using the basic flow equation described by Darcy, and then calculates the equivalent flow rate (transmissivity) of the drainage composite, and includes appropriate partial factors of safety for geosynthetic materials.

ANALYSIS

This calculation evaluates the flow rate within the drainage composite.

- **FLOW RATE (TRANSMISSIVITY) OF THE AGGREGATE DRAINAGE LAYER**

The maximum flow rate within the leachate collection zone is determined from Darcy's Law by the equation:

$$q = K\,i\,A$$

where:

q = flow rate within the leachate collection layer (m^3/sec)
K = hydraulic conductivity (m/sec)
i = hydraulic gradient (dimensionless)
A = area of flux (m^2)

The following properties will be used for the aggregate drainage layer material.

$K = 1*10^{-2}$ cm/sec $= 1*10^{-4}$ m/sec.
$i = 0.02$ (minimum slope of base liner system)
$A = (1 \text{ ft} * 1 \text{ ft}) = 1 \text{ ft}^2 = 0.093 \text{ m}^2$

The hydraulic conductivity is the prescriptive minimum value for the aggregate drainage layer, and the hydraulic gradient is a function of the minimum base slope of the cell. The area of flux is based on the unit thickness of the aggregate drainage layer (1 foot minimum).

Therefore,

$$q_{req} = (1*10^{-4} \text{ m/sec.})(0.02)(0.093 \text{ m}^2)$$
$$= 1.86*10^{-7} \text{ m}^3/\text{sec}$$

This maximum flow rate through the aggregate drainage layer, is the required flow through the drainage composite (q_{req}).

The allowable flow rate is obtained from laboratory testing for design purposes. This value is determined using appropriate safety factors against the required flow rate. The factor of safety is expressed as the ratio of the allowable flow rate (q_{all}) to the required flow rate (q_{req}).

$$FS = q_{all} / q_{req}$$

Similarly, the factor of safety equation can be expressed as the ratio of allowable to required in plane flow, transmissivity (θ), where the factor of safety equals:

$$FS = \theta_{all} / \theta_{req}$$

Where the transmissivity is calculated by:

$$\theta_{req} = q_{req} / (i*W) \qquad \text{(Attachment A)}$$

W = unit width of the drainage layer = 1 ft = 0.3408 m

Therefore,

$$\theta_{req} = \frac{(1.86*10^{-7}\ m^3/sec)}{(0.3048m)(0.02)}$$

$$= 3.05*10^{-5}\ m^2/sec.$$

The transmissivity of the aggregate drainage layer then becomes the minimum required transmissivity (θ_{req}) of the drainage composite.

- **TRANSMISSIVITY OF THE DRAINAGE COMPOSITE**

To ensure that the transmissivity of the proposed drainage composite meets or exceeds the required values over the life of the landfill, the required transmissivity must be increased through the use of appropriate partial factors of safety. These partial factors of safety make the adequate adjustment between the laboratory transmissivity values for drainage composite and actual field conditions.

As seen in Attachment A, Koerner suggests four factors of safety which should be accounted for: the intrusion of the adjacent geotextile into the core of the geonet (FS_{IN}), creep deformation of the geonet (FS_{CR}), factor of safety against chemical clogging of the geonet (FS_{CC}), and factor of safety against biological clogging of the geonet (FS_{BC}). Partial factor of safety values were applied to the geotextile in the filtration geotextile calculation to account for flow through the geotextile component of the drainage composite.

Attachment A shows the ranges for the partial factors of safety. For the purposes of the calculations made, the factors of safety were assumed to be:

FS_{IN} = 1.0 (Accounted for during the testing of the drainage composite)
FS_{CR} = 2.0
FS_{CC} = 1.0 (Accounted for in the Filter Calculation)
FS_{BC} = 1.0 (Accounted for in the Filter Calculation)

The ultimate transmissivity of the drainage composite then becomes:

$$\theta_{ultimate} = \theta_{req} * (\Sigma\ FS)$$

Written by: Geoff L. Smith GLS Date: 00 / 06 / 28 (YY MM DD) Reviewed by: GTC Date: 00 / 07 / 03 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 02

Σ FS = product of all the partial factors of safety for the site specific conditions

The ultimate transmissivity of the drainage composite is then calculated as:

$$\theta_{geonet} = \theta_{req} * (\Sigma \text{ FS}) = 3.05*10^{-5} \text{ m}^2/\text{sec} \; [\, 1.0*2.0*1.0*1.0]$$

$$= 6.1*10^{-5} \text{ m}^2/\text{sec}$$

- **MAXIMUM STRESS**

The maximum height of the waste fill is 60 ft (18.3 m). Assuming a unit weight of 136 pcf and a factor of safety of 1.5, this translates to a overburden stress of approximately 12,000 psf (574 kPa).

- CONCLUSIONS

The required transmissivity of the geocomposite shall be $6.1*10^{-5}$ m^2/sec at a maximum stress of 12,000 psf and a gradient of ~~0.02~~. 0.10 GLS

REFERENCES

Koerner, R.M. (1994) "Designing with Geosynthetics", 3rd Edition, Prentice Hall, New Jersey *(Attachment A)*

GTC

transportation-related systems, such as roads and walls, the problem does not appear to be too serious. In waste leachate related systems (e.g., landfill leachate collection systems), it might be another story. At the bottom of a landfill, temperatures are high, ample carbon (as a biological food source) is available, and bacteria and fungi could indeed thrive. Whether oxygen is available or not only dictates whether aerobic or anaerobic conditions prevail. No data regarding microorganisms in geonets are presently available, although research is currently ongoing. Procedurally, one must use a high flow rate factor of safety, or have systems designed so that flushing is possible. This area begs for future inquiry.

The last environmental consideration, resistance to light and weather, is not felt to be a serious concern for most situations in which geonets are used. Polyethylene is resistant to weather-related degradation, and carbon black is included in all of the known products. Nevertheless geonets should be covered as soon as possible after placement.

4.1.6 Allowable Flow Rate

As described previously, the essence of the design-by-function concept is the establishment of an adequate global factor of safety. For geonets, where flow rate is the primary function, this takes the following form:

$$FS = \frac{q_{allow}}{q_{req}} \tag{4.3}$$

where FS = the global factor of safety (to handle unknown loading conditions or uncertainties in design methods, etc.),
q_{allow} = the allowable flow rate as obtained from laboratory testing, and
q_{req} = the required flow rate as obtained from design of the actual system.

Alternatively, one could also work from a transmissivity basis to obtain the equivalent relationship.

$$FS = \frac{\theta_{allow}}{\theta_{req}} \tag{4.4}$$

where θ is the transmissivity under similar definitions as above. As described previously, however, it is preferable to design with flow rate rather than transmissivity because of nonlaminar flow conditions in geonets.

Concerning the allowable value, which comes from hydraulic testing of the type just described, one must assess the realism of the test setup in contrast to the actual field system. If it does not model real life adequately, then some adjustments to the laboratory value must be made. This is often the case. Thus the laboratory-

-/18

ATTACHMENT A, [illegible] 1/2

3/16/99

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Table 4.2 Recommended preliminary factor of safety values for determining allowable flow rate or transmissivity of geonets

	Partial Factor of Safety Value in Equation 4.5			
Application Area	FS_{IN}	FS_{CR}*	FS_{CC}	FS_{BC}
Sport fields	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.1 to 1.3	1.0 to 1.2	1.1 to 1.5	1.1 to 1.3
Roof and plaza decks	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock and soil slopes	1.3 to 1.5	1.2 to 1.4	1.1 to 1.5	1.0 to 1.5
Drainage blankets	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2
Surface water drains for landfill caps	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.2 to 1.5
Secondary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Primary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0 *

*These values assume that the q_{ult} value was obtained using an applied normal pressure of 1.5 to 2 times the field-anticipated maximum value. If not, values must be increased.

done at the proper design load and hydraulic gradient and that this testing yielded a short-term between-rigid-plates value of 1.2 gal./min.-ft.

Solution: Since better information is not known, average values from Table 4.2 are used.

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

$$= 1.2\left[\frac{1}{1.1 \times 1.1 \times 1.1 \times 1.2}\right]$$

$$= 1.2\left[\frac{1}{1.60}\right]$$

$$= 0.75 \text{ gal./min.-ft.}$$

Example:

What is the allowable geonet flow rate to be used in the design of a secondary leachate collection system? Assume that laboratory testing at proper design load and proper hydraulic gradient gave a short-term between-rigid-plates value of 1.2 gal./min.-ft.

Solution: Average values from Table 4.2 are used; however, note the large reduction.

2/2

ATTACHMENT A, ~~#4~~

4.2 DESIGNING FC

This section wi
transportation-r
field.

4.2.1 Theoreti

Design by func

For geonets ser
aforementioned

where q_{allow} = t
q_{reqd} = t

As stated prev
saturated condi
flow rate, namc
equations 4.1 a

SJM 3/16/99

Calculation Package M
Consolidation Analysis of the South Mesa

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Consolidation Analysis of South Mesa

Computations By:

SIGNATURE DATE 11/3/06

Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE DATE 11/6/06

Keaton Botelho, Staff Engineer
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE DATE 11/3/06

Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: [signature] Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

CONSOLIDATION ANALYSIS OF SOUTH MESA
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

The objective of this calculation package is to evaluate the consolidation settlement potential of the south mesa at the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada. In particular, the amount of differential settlement and strain will be evaluated.

SUMMARY OF ANALYSIS

The calculations suggest that the strain on the liner system due to differential settlement is within tolerable amounts. In addition, the proposed grading plan will accommodate the calculated differential settlement without significant effects on drainage. Since limited information is available regarding the consolidation characteristics of the soil strata, interpretation was required and the analyses were approached on a conservative basis.

SITE BACKGROUND

Native material within the limits of the BRC CAMU consists of alluvial granular soils overlying fine-grained soils. Shear strength parameters for the native soil material were previously estimated and reported in the *Preliminary Geotechnical and Geologic Investigation - Industrial Non-Hazardous Disposal Facility* (Converse 1999). Twelve exploratory borings were conducted by others to depths ranging from 33-ft to 60-ft (Converse 1999). In general, the native materials appear to be consistent between borings. Six consolidation tests were performed on retrieved samples of sandy clay (CL). All samples were inundated with water at 2000 psf. The author has approximated the saturated consolidation curve for the purposes herein. A summary of the consolidation test results as reported by Converse (1999) are presented in Attachment 1.

ANALYSIS

Terzaghi's one-dimensional consolidation theory will be used to evaluate the consolidation potential of the south mesa at the BRC CAMU. The method of analysis to evaluate the maximum differential settlement is to compare the calculated settlement at each boring location.

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

After the expected settlement is known for each location, the maximum differential settlement and strain can be calculated. The location of each boring is presented in Figure 1. An idealized profile for the borings is presented in Figure 2. The sandy clay (CL) layers are highlighted.

The Casagrande construction method was used to evaluate if the lean clay is normally consolidated and to construct the field consolidation curve. The construction is shown in Attachment 1. The following results are obtained:

Sample	Depth (ft)	Initial e_0	C_c	Liquid Limit (%)	Moisture Content (%)	Present Consolidation Pressure (ksf)	Previous Consolidation Pressure (ksf)
B-8	39.0	1.9	0.55		64	3.71	about 3.7
B-8	49.0	1.41	0.98		51	5.18	about 5.0
B-102	49.0	1.52	0.86	88	49	4.89	about 4.1
B-10	54.0	1.76	0.81		68	5.06	about 4.1
B-101	59.0	1.31	0.52		38	4.88	about 4.4
B-101	39.0	1.55	0.47	105	45	3.7	about 4.0
Avg.		1.58	0.73				

Based on the above results, the lean clay deposit can be categorized as normally consolidated (i.e., the previous consolidation pressure is relatively close to the present consolidation pressure).

The settlement calculations are presented in Attachment 2. The results are summarized below:

Boring	Thickness of CL (ft)	Calculated Settlement (ft)
B-10	22 ft (interbedded layers of SM)	2.54
B-12	24 ft	2.58
B-8	22 ft	2.35
B-102	8 ft	0.92
B-5	10 ft	1.18
B-4	5 ft	0.37

As shown in Figure 1, borings B-4, B-5, B-8, B-10, B-12, and B-102 were considered for this analysis. Borings B-101 and B-103 were not considered because they lie well outside the liner

Written by: Rebecca Flynn Date: 06 / 08 / 21 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

limits. Settlement was calculated at the aforementioned boring locations (known CL thickness). However, consolidation tests were not necessarily performed at each of these locations. For locations were there was no consolidation data, the average values for e_o and C_c were used. See Attachment 2 for settlement calculations.

The shortest distance between borings and the largest differential settlement will result in the largest strain. This relationship is defined as:

$$\varepsilon = \Delta h/L$$

The maximum differential settlement at the shortest distance was between borings B-4, and boring B-8 (see Figure 1). The distance between the borings is 470 ft. The strain is:

$$\varepsilon = \Delta h/L = (2.35\text{-}0.37)/470 = 0.42 \text{ percent.}$$

Assuming a linear differential settlement profile, the calculated strain is well below the typical yield strength of HDPE geomembrane. Typical strain at the yield stress for HDPE geomembrane is approximately 12 percent (Koerner 1994).

CONCLUSIONS

Based on available information and the author's interpretation of the appropriate consolidation curves, the following can be concluded:

- The CL deposit can be characterized as normally consolidated;
- The maximum settlement expected in the south mesa is approximately 2 ft;
- The maximum strain expected in the south mesa is approximately 0.4%
- The liner system and proposed grading can accommodate the expected differential settlement and strain.

REFERENCES

Converse (1999), "Preliminary Geotechnical and Geologic Report - Industrial Non-Hazardous Disposal Facility (Converse 1999)", prepared for Basic Management, Inc., October 1999.

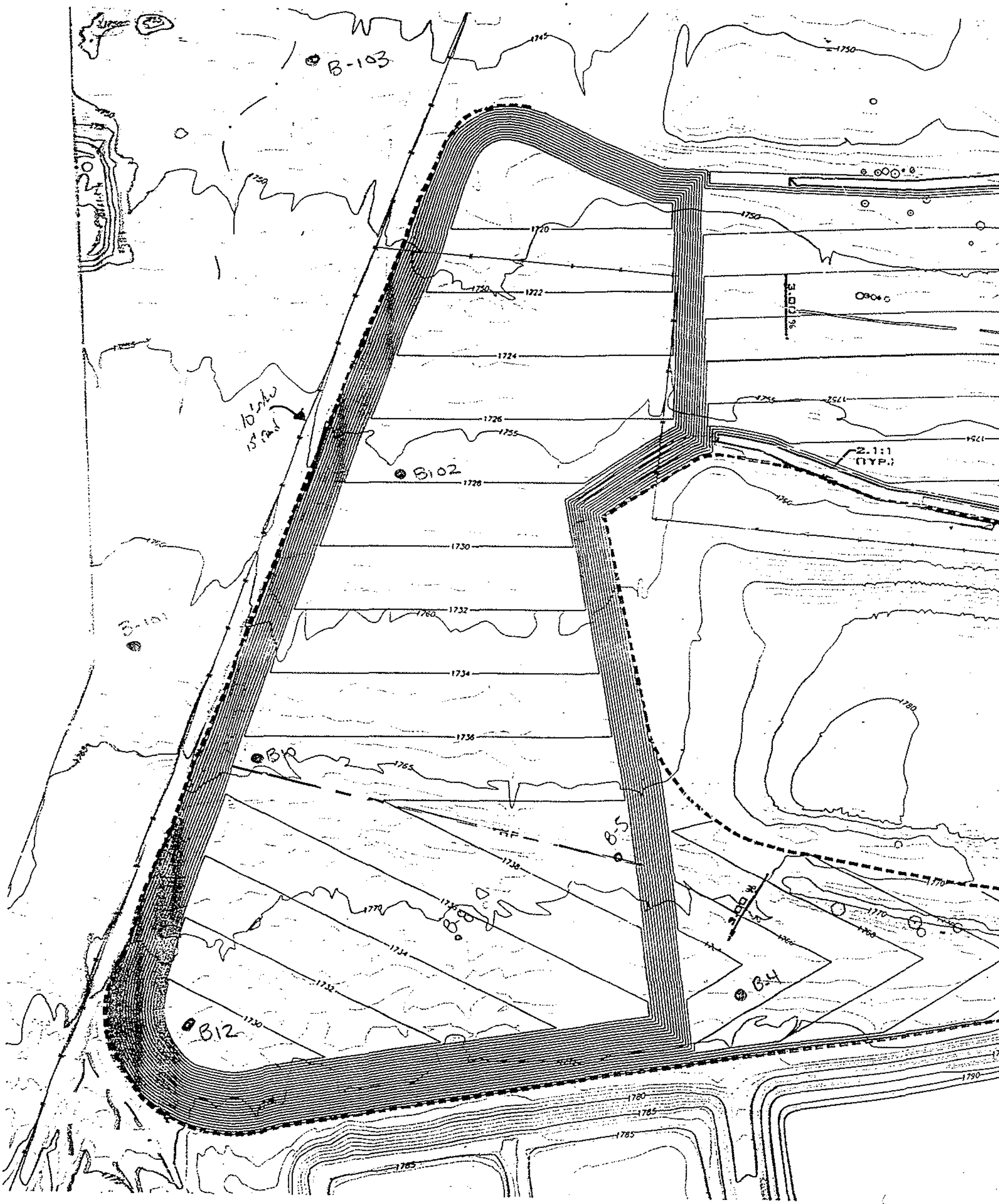

B-103
1745
1750
1720
1722
1724
1726
1728
1730
1732
1734
1736
1738
1755
1760
1765
1770
1780
1785
1790
B102
B-101
B10
B-5
B-4
B12
2.1:1
(TYP.)

FIGURE 1

GEOSYNTEC CONSULTANTS Page ______ of ______

Written by: R. FINN Date: 21 / 08 / 06 (DD MM YY) Reviewed by: ______________ Date: ___ / ___ / ___ (DD MM YY)

Client: ______________ Project: ______________ Project/Proposal No.: ______________ Task No: ______________

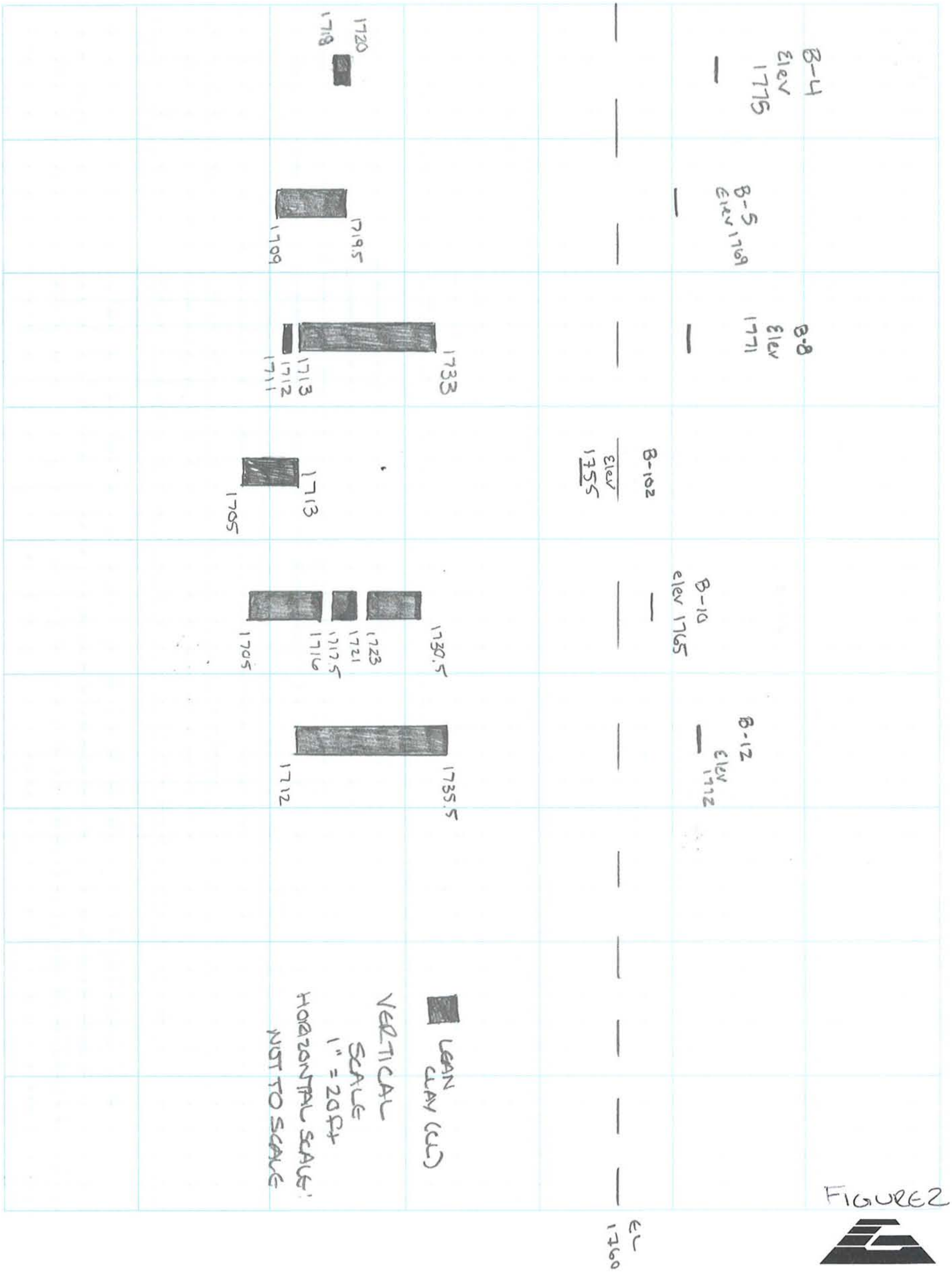
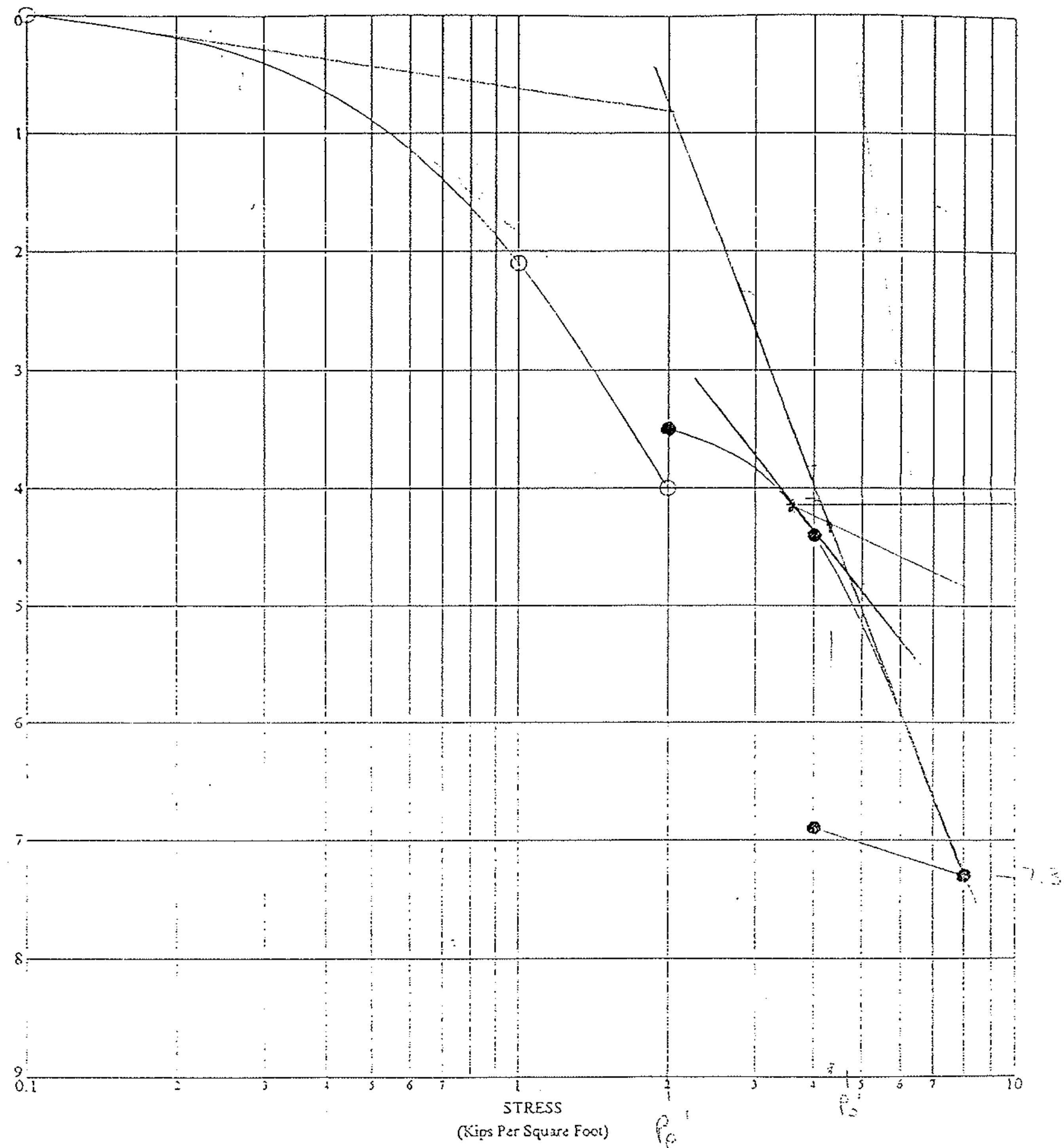


FIGURE 2

411 GTC



BORING NO: B-102 $e_o = 1.52$

DEPTH: 49.0

DESCRIPTION:

DRY DENSITY: 67 PCF

MOISTURE CONTENT: 49%

Attachment 1

1/6

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

Project No. 99-33437-01

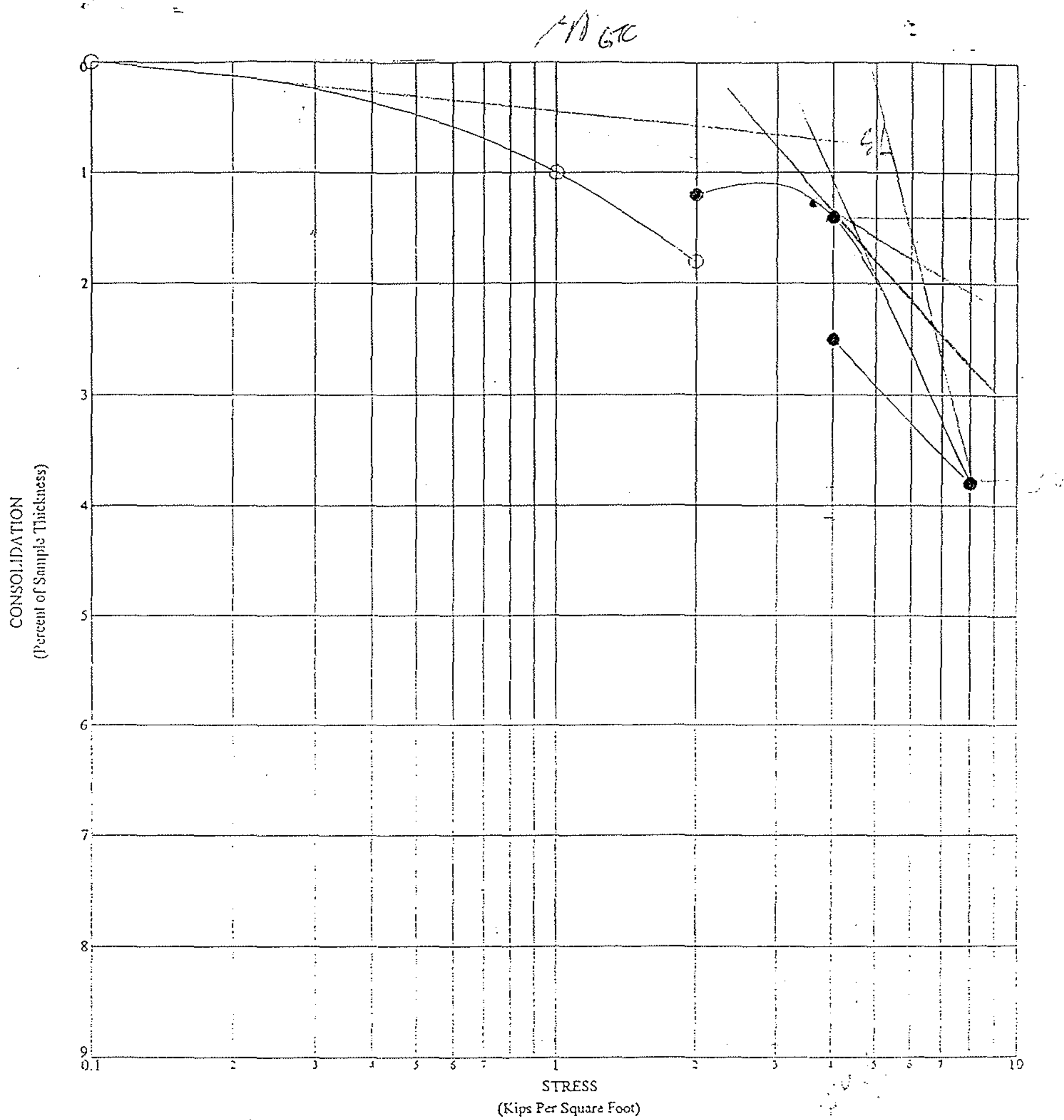
APPROVED BY ON

Converse Consultants

Geotechnical Engineering

Drawing No.

DRAFTED



APPROVED BY ON

BORING NO: B-101
DEPTH: 59.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 73 PCF
MOISTURE CONTENT: 38 %

Attachment 1
2/6

● READINGS AFTER SATURATION WITH WATER

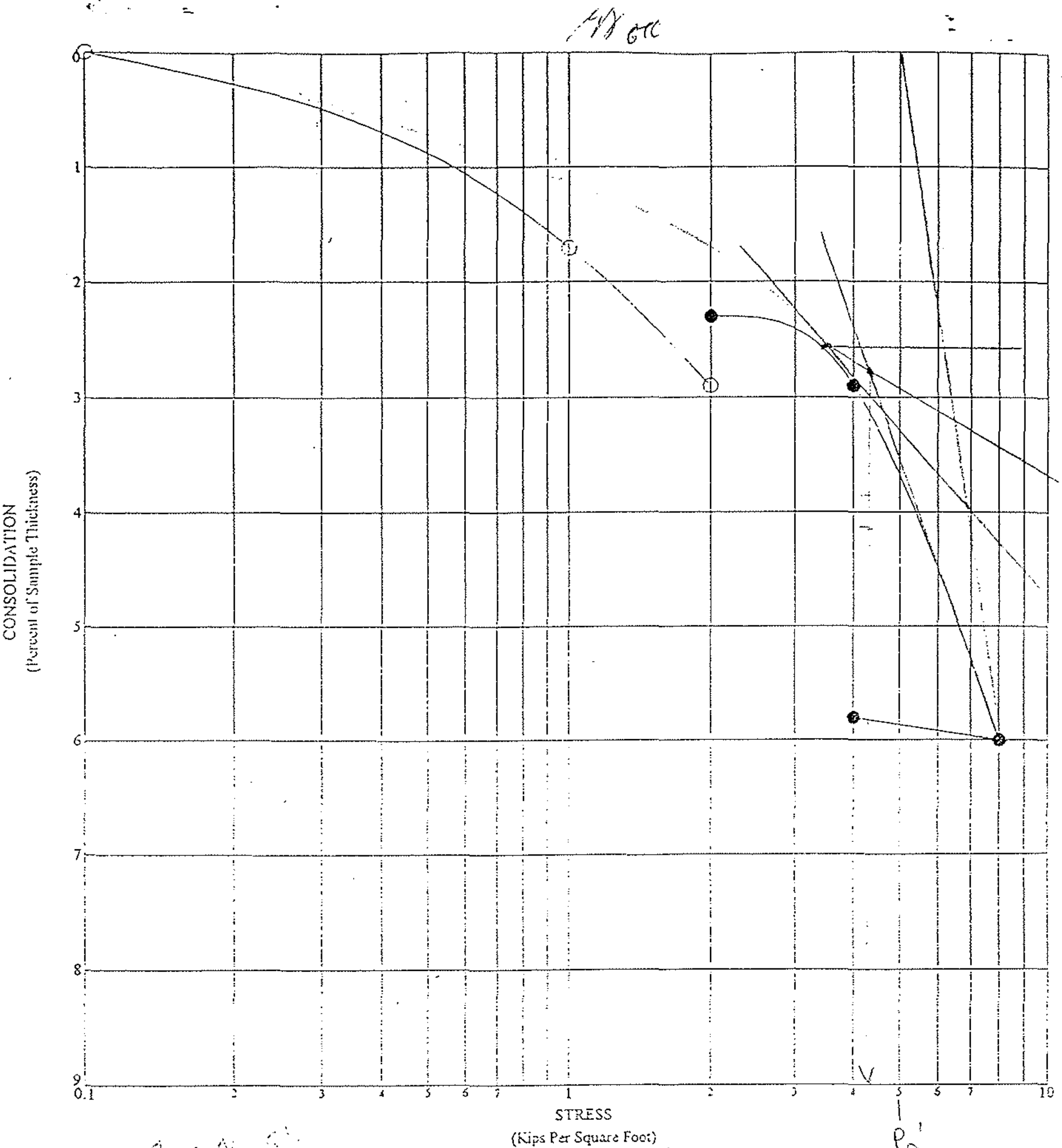
CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

Project No. 99-33437-01

Geotechnical Engineering

Drawing No.



DRAFT

BORING NO: B- 10
DEPTH: 54.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 61 PCF
MOISTURE CONTENT: 68%

$e_o = 1.76$

Attachment 1
3/6

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

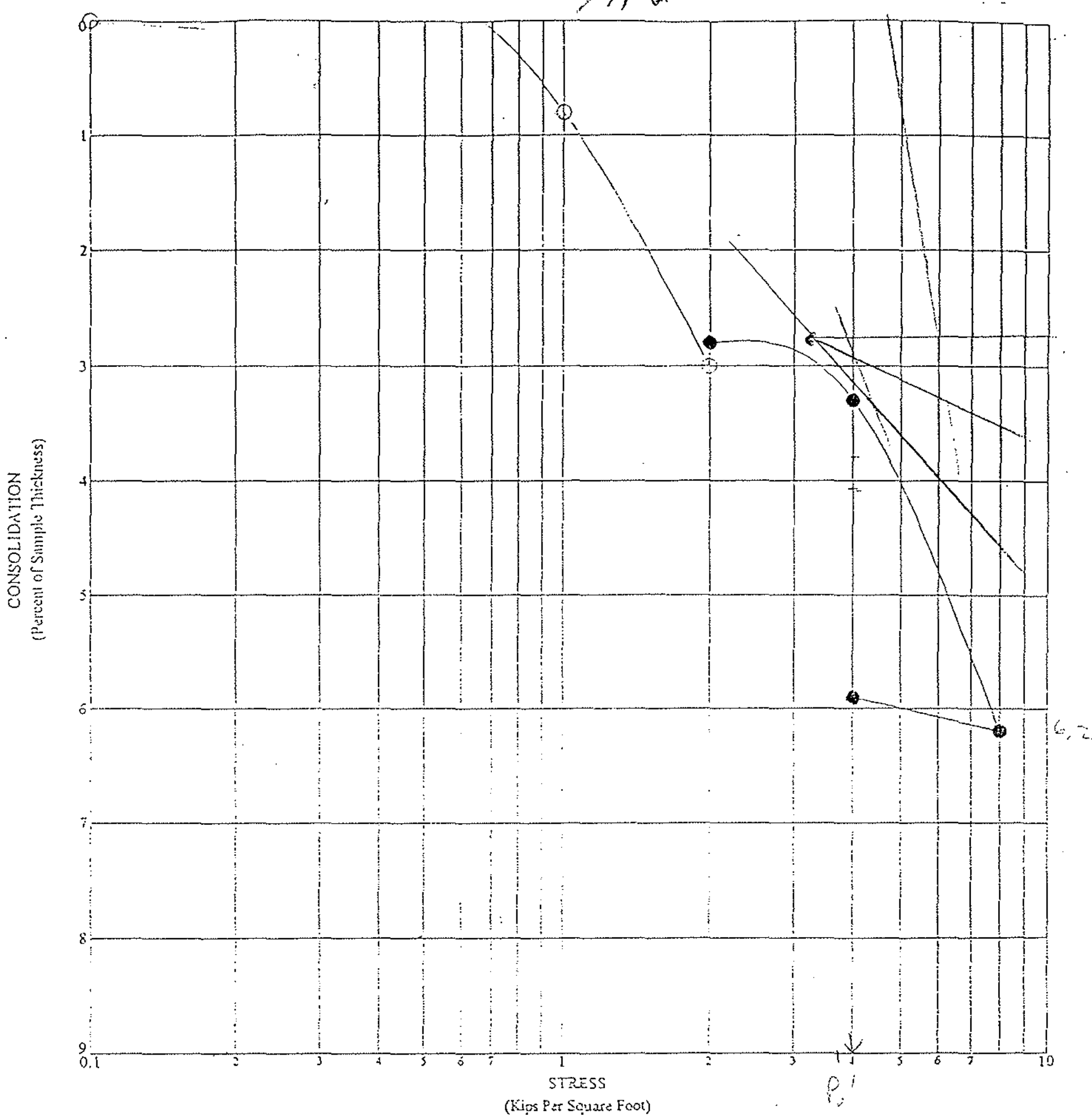
Project No.
99-33437-01

Drawing No.

Converse Consultants

Geotechnical Engineering and Applied Sciences

DRAFTED BY



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BORING NO: B-101
DEPTH: 39.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 66 PCF
MOISTURE CONTENT: 45%

Attachment 1
4/6

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

Project No.
99-33437-01

Geotechnical Engineering

Drawing No.

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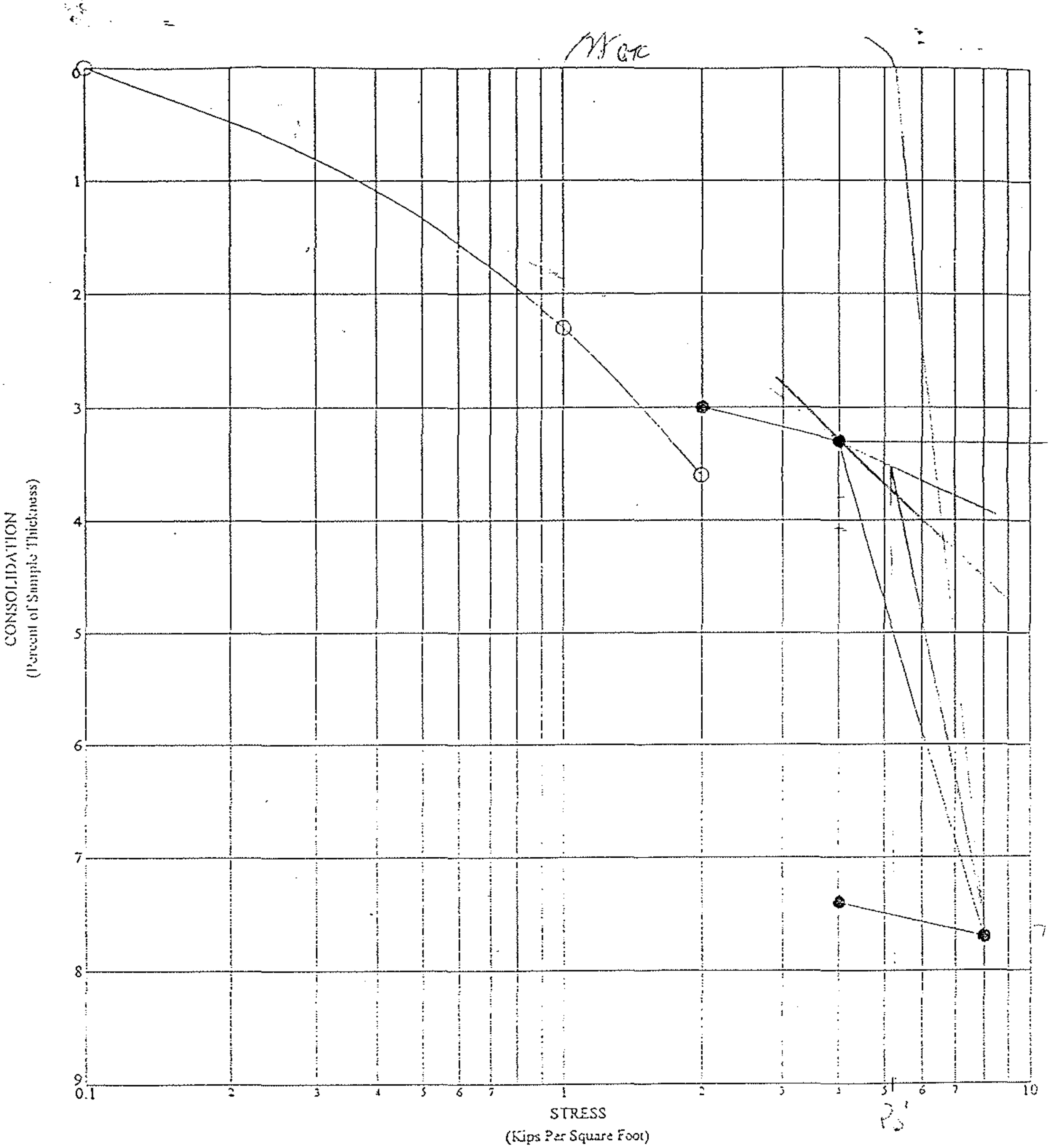

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BORING NO: B- 8
DEPTH: 49.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 70 PCF
MOISTURE CONTENT: 51 %

● READINGS AFTER SATURATION WITH WATER

Attachment 1
5/6

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

Project No.
99-33437-01

Geotechnical Engineering

Drawing No.

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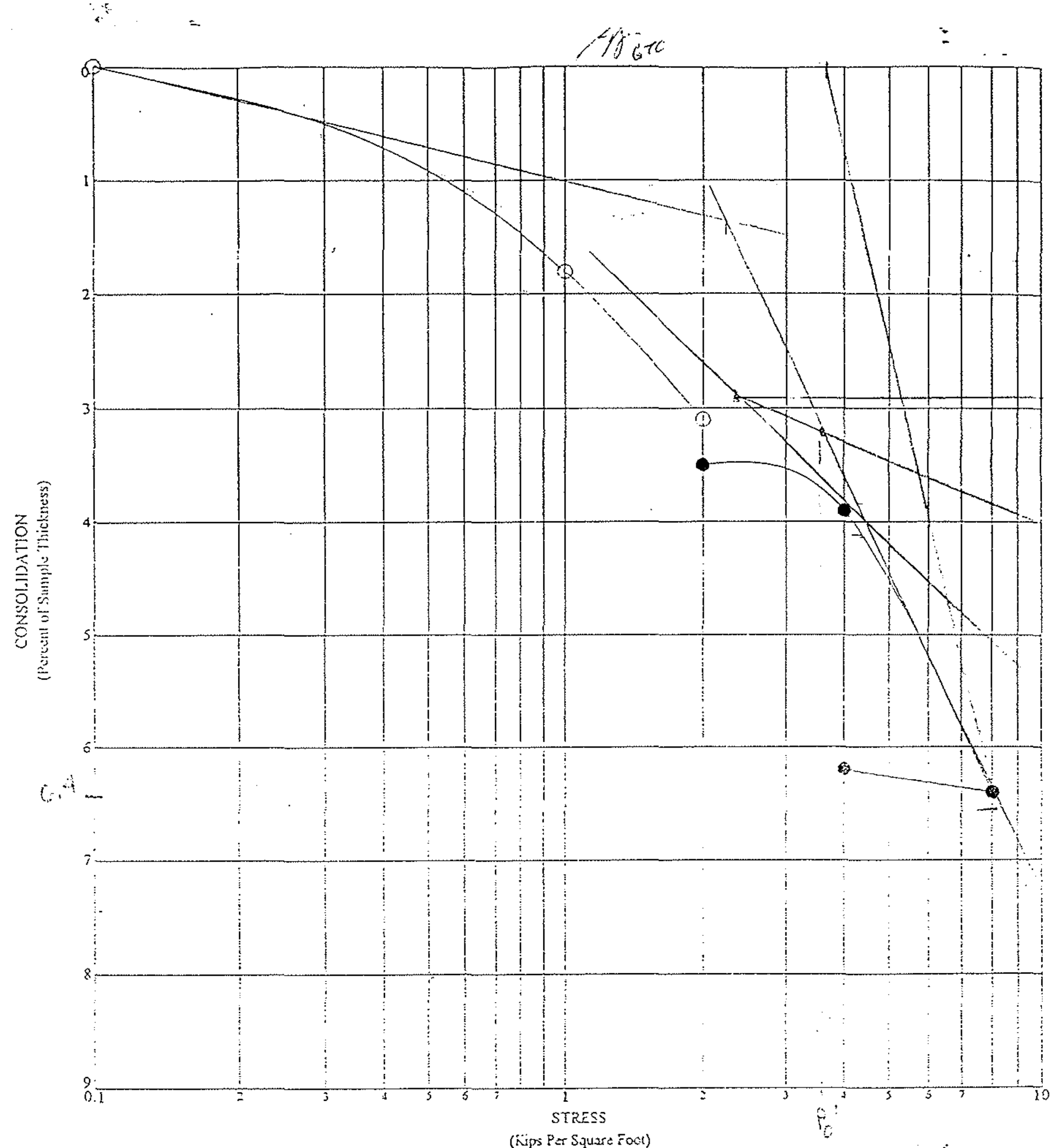

APPROVED BY ON

BORING NO: B- 8 $e_0 = 1.9$
DEPTH: 39.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 58 PCF
MOISTURE CONTENT: 64 %

$p_0' = 3110$ psf

$C_c = \frac{\Delta e}{\log(p_1/p_0)} = \frac{0.156}{\log(8/3.2)}$

$e_0 = \frac{G_s \gamma_w}{\gamma_d} - 1$

$\Delta e = \frac{S(1+e)}{H} = 0.156$

$H = 1''$

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

~~Attachment 1~~

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

Project No.
99-33437-01

Drawing No.

Geotechnical Engineering

Written by: R. FLYNN Date: 21 / 08 / 06 Reviewed by: GTC Date: 03 / 11 / 06

Client: ______ Project: ______ Project/Proposal No.: ______ Task No: ______

SETTLEMENT ANALYSIS FOR BORING B-102

→ ASSUME LAYER THICKNESS OF 8 ft
O.K THICKNESS FOR ANALYSIS

→ ASSUME $C_c = 0.86$ $e_0 = 1.52$

→ Assume 29 ft excavation of 117 pcf and replace at 136 pcf plus 42 ft of fill at 136 pcf

Therefore, $\Delta p = (29')(136-117) + 42'(136)$
$\Delta p = 6263$

→ Depth of mid-layer at depth = 46'
water table at 43 ft
Therefore,
$P_o' = 43\text{ft}\,(117\,\text{pcf}) + (117-62.4)(3\text{ft})$

$P_o' = 5195$

$$S = \frac{C_c}{1+e_0} H \log\left(\frac{P_o' + \Delta p}{P_o'}\right)$$

$$S = \frac{0.86}{1+1.52}(8')\log\left(\frac{5159+6263}{5159}\right)$$

$S = 0.\cancel{924}$ ft $= 11.3$ inches

.942 GTC

Attachment 2

1/4

Written by: R FLYNN Date: 21 / 08 / 06 Reviewed by: ______ Date: ___ / ___ / ___

Client: ______ Project: ______ Project/Proposal No.: ______ Task No: ______

SETTLEMENT ANALYSIS FOR BORING B-5

→ ASSUME LAYER THICKNESS OF 10 ft
Too thick: do analysis for two 5ft layers

→ Assume $C_c = 0.73$ and $e_0 = 1.58$

→ ASSUME 30 ft of excavation and replacement at 136 pcf, plus 70 ft of fill at 136 pcf
therefore

$$\Delta p = (30)(136-117) + 70(136)$$
$$\Delta p = 10{,}090$$

→ Depth of mid-layer #1 = 52.5 ft
$P_o' = 52.5(117) = 6142$ psf

→ Depth of mid-layer #2 = 57.5 ft
$P_o' = 52\text{ ft}(117\text{ pcf}) + (117-62.4)(57.5-52)$
$P_o' = 6385$ psf

$$S_1 = \frac{0.73}{1+1.58}(5)\log\left(\frac{6142+10090}{6142}\right)$$

$S_1 = 0.597$ Ft. $= 7.17$ inches

$$S_2 = \frac{0.73}{1+1.58}(5)\log\left(\frac{6385+10090}{6385}\right)$$

$S_2 = 0.582$ ft $= 6.99$ inches

$S_{TOTAL} = 1.18$ ft $= 14.15$ inches

Attachment 2

Written by: R. FLYNN Date: 21 / 08 / 06 Reviewed by: __________ Date: ___/___/___

Client: __________ Project: __________ Project/Proposal No.: __________ Task No: __________

SETTLEMENT ANALYSIS FOR BORING B-4

→ ASSUME LAYER THICKNESS OF 5 ft

→ Assume $C_c = 0.73$ and $e_o = 1.58$

→ $S = \frac{C_c}{1+e_o} H \log\left(\frac{P_o + \Delta p}{P_o}\right)$

→ Assume 10 ft of excavation and replace at 136 pcf, plus 38 ft fill at 136 pcf

Therefore $\Delta p = (10)(136-117) + 38(136)$

$\Delta p = 5358$

→ DEPTH AT MID LAYER = 56 ft
WATER TABLE DEPTH AT 54 ft

THEREFORE $P_o' = 54\,(117\,\text{pcf}) + 2'\,(117-62.4)$

$P_o' = 6427$ psf

$S = \frac{0.73}{1+1.58}(5\,\text{ft}) \log\left[\frac{6427 + 5358}{6427}\right)$

$S = 0.373$ ft → 4.5 inches

Attachment 2
3/4

Settlement Analysis for Boring B-8

H	22	ft						
w%	51	%						
Gs	2.7		Layer	Thickness	Mid-Depth	Po'	Si
$\gamma_{d,\,max}$	70	pcf	1	3	39.5	4621.5	0.367447
% comp.	100	%	2	3	42.5	4972.5	0.349927
γ	105.7	pcf	3	3	45.5	5323.5	0.334088
e_o	1.406857		4	3	48.5	5674.5	0.319688
C_c	0.76		5	3	51.5	6025.5	0.306533
Δp	6668		6	3	54.5	6376.5	0.294462
			7	4	58	6786	0.375428
							2.347574

Settlement Analysis for Boring B-10

H	22	ft					
w%	49	%					
Gs	2.7		Layer	Thickness	Mid-Depth	Po'	Si
$\gamma_{d,\,max}$	67	pcf	1	3	39.5	4621.5	0.38723
% comp.	100	%	2	3	42.5	4972.5	0.368582
γ	99.83	pcf	3	3	45.5	5323.5	0.351735
e_o	1.514627		4	3	48.5	5518.5	0.343057
C_c	0.86		5	3	51.5	5682.3	0.336106
Δp	6399		6	3	54.5	5846.1	0.329444
water table	46		7	4	58.5	6064.5	0.427971
				22			2.544125

Settlement Analysis for Boring B-12

H	24	ft					
e_o	1.58						
C_c	0.73		Layer	Thickness	Mid-Depth	Po'	Si
Δp	7753		1	4	38	4446	0.496122
			2	4	42	4914	0.465433
			3	4	46	5382	0.43855
			4	4	50	5850	0.414774
			5	4	54	6318	0.393572
			6	4	58	6786	0.37453
							2.582983

Attachment 2
4/4

Calculation Package N
Consolidation Analysis of the North Mesa

GeoSyntec Consultants

GeoSyntec Consultants
Computation Cover Sheet

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Consolidation Analysis of North Mesa

Computations By:

SIGNATURE DATE 11/3/06

Rebecca Flynn, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE DATE 11/3/06

Rebecca Flynn, Staff Engineer
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Approved By (PM or Designate):

SIGNATURE DATE 11/3/06

Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Rebeccca Flynn Date: 06 / 08 / 22 (YY MM DD) Reviewed by: GTC Date: 06 / 11 / 3 (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-04

CONSOLIDATION ANALYSES OF SLIT TRENCHES
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

The objective of this calculation package is to evaluate the consolidation settlement potential of landfilled waste in the "slit trenches" of the North Mesa at the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada. In particular, the amount of differential settlement and potential strain in an overlying geosynthetic liner will be evaluated. Finally, the height of surcharge material to be used in pre-loading the slit-trenches will be calculated.

SUMMARY OF ANALYSES

Since limited information is available regarding the consolidation characteristics of the soil strata, interpretation was required and the analyses were approached on a conservative basis. Should the liner system be constructed overlying the existing, unconsolidated "slit trenches", the calculations suggest that the strain on the liner system due to differential settlement is not within tolerable amounts. The strain on the liner system may be reduced to within tolerable amounts if the trenches are first 'pre-loaded' with approximately 40 feet of surcharge material.

SITE BACKGROUND

Native material within the limits of the BRC CAMU consists of alluvial granular soils overlying fine-grained soils. Shear strength parameters for the native soil material were previously estimated and reported in the *Preliminary Geotechnical and Geologic Investigation - Industrial Non-Hazardous Disposal Facility* (Converse 1999). Twelve exloratorary borings were conducted by others to depths ranging from 33-ft to 60-ft (Converse 1999). In general, the native materials appear to be consistent between borings.

In January 2005, BRC performed a subsurface investigation of the slit trenches, consisting of continuous, split-spoon sampling to determine the depth of landfilled waste. Twenty exploratory borings were advanced to depths ranging from 22-ft to 32-ft (Attachment A). The thickness of landfilled waste ranges between 5-ft to 26-ft, and consists of typical industrial waste and construction/demolition debris: wood, paper, fabric, plastic, glass, and metal. The width of slit trenches ranges between 20 feet and 76 feet

The design of the proposed North Mesa landfill includes approximately 38-ft of waste and 2-ft of cover soil, resulting in approximately 40-ft of overburden material on the present base grades.

ANALYSIS

Consolidation Settlement

Terzaghi's one-dimensional consolidation theory will be used to evaluate the consolidation potential of the slit trenches of the North Mesa at the BRC CAMU. The method of analysis to evaluate the maximum differential settlement is to compare the calculated settlement at each boring location. After the expected settlement is known for each location, the maximum differential settlement and strain can be calculated.

There are two components to the total settlement calculated by one-dimensional consolidation theory, primary settlement and secondary settlement. The calculation of total consolidation settlement is governed by the following equation:

$$S_T = S_P + S_S$$

Where:

S_p = magnitude of primary consolidation settlement, ft
S_S = magnitude of secondary consolidation settlement, ft

The calculation of primary settlement is governed by the following equation:

$$S_p = C'_c \cdot H \cdot \log\left(\frac{p_o + \Delta p}{p_o}\right) \qquad \text{(Attachment B, 1 of 2)}$$

Where:

S_p = magnitude of primary consolidation settlement, ft
C'_c = compression index for the compressible layer (waste) = 0.3
H = initial thickness of compressible layer (waste), ft
p_o = initial overburden pressure on the compressible layer, lb/ft^2
Δp = change in overburden pressure due to loading, lb/ft^2

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Furthermore:

$$p_o = \frac{H}{2} \cdot \gamma_{waste} + H_{soil,overburden} \cdot \gamma_{soil,overburden}$$

H = initial thickness of compressible layer, ft
γ_{waste} = unit weight of waste = 85 lb/ft^3
$H_{soil,overburden}$ = thickness of soil overburden over waste, ft
$\gamma_{soil,\ overburden}$ = unit weight of soil overburden = 115 lb/ft^3

and

$$\Delta p = H_{soil,waste} \cdot \gamma_{soil,waste} + H_{soil,cov\,ersystem} \cdot \gamma_{soil,cov\,ersystem}$$

$H_{soil,\ waste}$= thickness of waste to be placed in landfill, ft
γ_{waste} = unit weight of waste to be placed in landfill = 136 lb/ft^3
$H_{soil,\ coversystem}$ = thickness of soil in landfill cover system, ft
$\gamma_{soil,\ coversystem}$= unit weight of soil in landfill cover system = 125 lb/ft^3

The calculation of secondary settlement is governed by the following equation:

$$S_S = C'_\alpha \cdot H \cdot \log\left(\frac{t_2}{t_1}\right) \qquad \text{(Attachment B, 2 of 2)}$$

Where:

S_S = magnitude of secondary consolidation settlement, ft
C'_α = secondary compression index for the compressible layer (waste) = 0.019
H = initial thickness of compressible layer (waste), ft
t_1 = time at beginning of secondary consolidation = 26 year (assuming waste placement ended in 1980)
t_2 = time at end of consolidation, years = 59 years (t_1 + 30 years post-closure maintenance + 3 years of landfill operation)

The compression index (C_c) and secondary compression index (C'_α) are normally determined through laboratory analyses of samples collected during subsurface investigation. In the absence of such testing, the following calculations use a conservatively high C_c value, as obtained from Deutsch, et al. (Attachment C), and a conservatively high C'_α value, as obtained from Bjarngard, et al. (Attachment D).

The consolidation settlement calculations are presented in Attachment E (1 of 3). The results are summarized below:

Location	**Initial Waste Thickness (ft)**	**Total Settlement, S_T (ft)**
BS-01	1	0.35
BS-02	6	1.10
BS-03	19	3.53
BS-04	10	1.86
BS-05	18	3.78
BS-06	20	4.46
BS-07	18	3.54
BS-08	12	1.74
BS-09	22	3.82
BS-10	0	0.00
BS-11	24	4.18
BS-12	26	5.42
BS-13	20	4.55
BS-14	26	5.00
BS-15	10	2.76
BS-16	5	0.04
BS-17	22	0.15
BS-18	11	1.24
BS-19	21	1.35
BS-20	8	0.83

Strain Due to Differential Settlement

The differential settlement between the compressible material in a slit trench and the relatively incompressible native material adjacent to the trench will result in strain in the overlying liner system. The maximum strain is assumed to occur at the middle of the trench, at the point of maximum settlement. This relationship is defined as:

$$\varepsilon = \frac{\sqrt{S_T^{\,2} + L^2}}{L} - 1$$

Where:

ε = strain in liner system due to differential settlement
L = half the width of the slit trench, ft
S_T = total settlement, ft

The maximum allowable strain in the liner system is selected to be 1% to maintain drainage of the leachate collection system (3%, as constructed) and acceptable for the geomembrane material properties. Those locations that exceed the allowable strain will require "pre-loading" prior to liner placement. The purpose of this "pre-loading" is to achieve consolidation settlement of the waste and minimize settlement induced strain. The required settlement to be achieved by pre-loading is calculated as follows:

$$S_{req} = S_T - \left(\varepsilon_{allow} \cdot L\right)$$

Where:

S_{req} = required settlement, ft
S_T = total settlement, ft
ε_{allow} = maximum allowable strain in liner system = 1%
L = half the width of the slit trench, ft

The strain calculations are presented in Attachment E (2 of 3). The results are summarized below:

Location	L (ft)	Strain, ε	S_{req} (ft)
BS-01	20	0.02%	-
BS-02	20	0.15%	-
BS-03	20	1.55%	2.52
BS-04	10	1.72%	0.85
BS-05	10	6.91%	2.77
BS-06	10	9.50%	3.45
BS-07	10	6.08%	2.53
BS-08	10	1.50%	0.73
BS-09	10	7.05%	2.81
BS-10	38	0.00%	-
BS-11	38	0.60%	-
BS-12	20	3.61%	4.41
BS-13	20	2.56%	3.54
BS-14	20	3.08%	3.99
BS-15	20	0.95%	-
BS-16	23	0.00%	-
BS-17	23	0.00%	-
BS-18	23	0.15%	-
BS-19	13	0.54%	-
BS-20	13	0.20%	-

Required Surcharge Height

The surcharge height necessary to preload the slit trenches and obtain the required settlement is calculated by rearranging the equation for primary consolidation settlement (S_P), as below:

$$H_{surcharge} = \frac{\left(10^{\left(\frac{S_{req}}{C_c \cdot H}\right)} \cdot p_o\right) - p_o}{\gamma_{surcharge}}$$

Where:

$H_{surcharge}$ = height of surcharge, ft
S_{req} = required settlement, ft
C_c = compression index for the compressible layer (waste) = 0.3
H = initial thickness of compressible layer (waste), ft
p_o = initial overburden pressure on the compressible layer, lb/ft^2
$\gamma_{surcharge}$ = unit weight of surcharge soil = 135 lb/ft^3

The surcharge height calculations are presented in Attachment E (3 of 3). The results are summarized below:

Location	**S_{req} (ft)**	**$H_{surcharge}$ (ft)**
BS-01	-	-
BS-02	-	-
BS-03	2.52	23
BS-04	0.85	12
BS-05	2.77	22
BS-06	3.45	24
BS-07	2.53	23
BS-08	0.73	12
BS-09	2.81	24
BS-10	-	-
BS-11	-	-
BS-12	4.41	29
BS-13	3.54	26
BS-14	3.99	28
BS-15	-	-
BS-16	-	-
BS-17	-	-
BS-18	-	-
BS-19	-	-
BS-20	-	-

Thus, it appears that a maximum of 29 feet of surcharge should be placed overlying the slit trenches to minimize the effects of settlement induced strain in the base liner system.

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CONCLUSIONS

Based on available information and the results of these consolidation analyses, the following can be concluded:

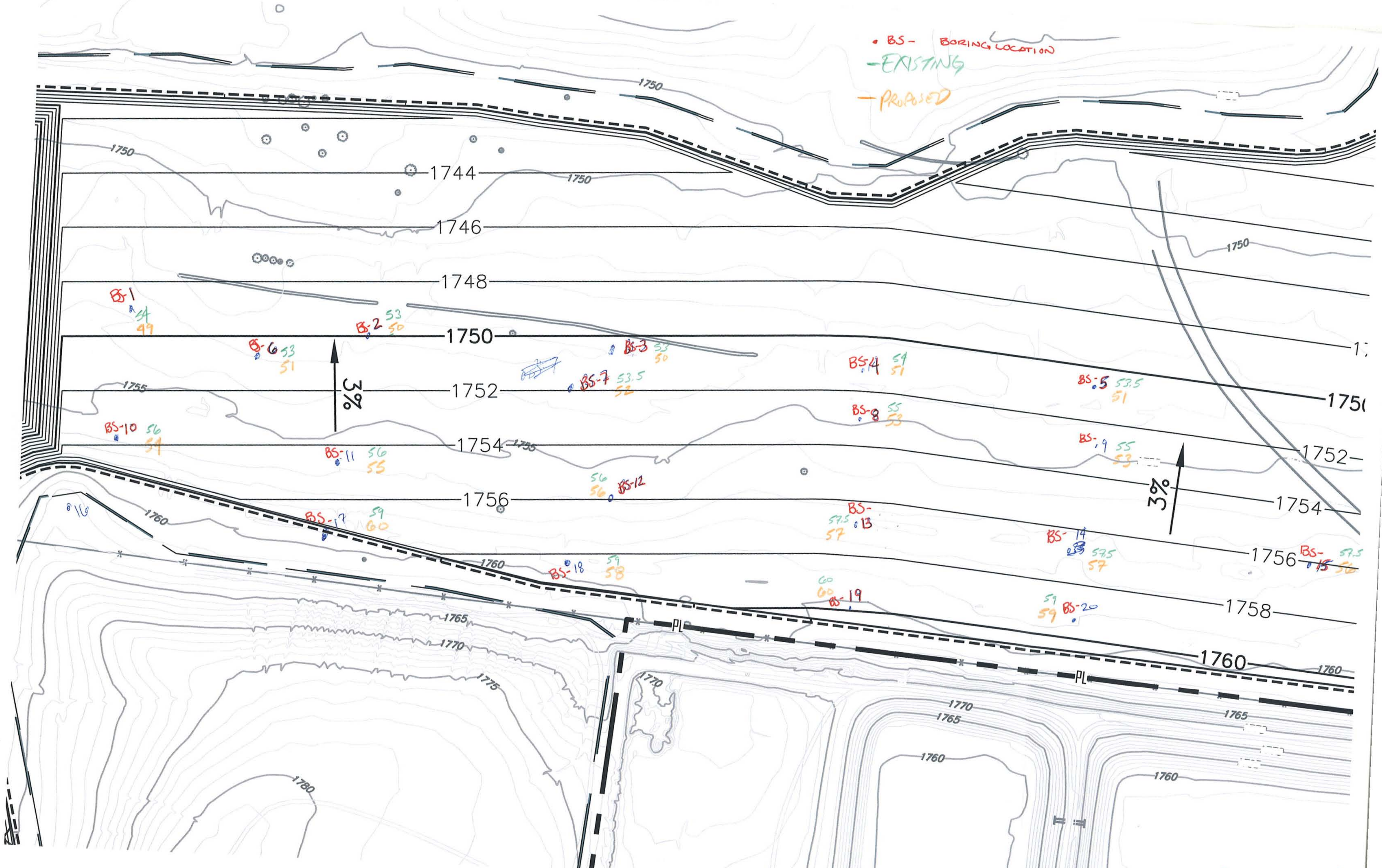
- The maximum differential settlement expected in the slit trenches of the North Mesa is approximately 4.96 ft;
- The height of surcharge required to pre-load the North Mesa is a maximum of 29 ft;
- The maximum strain expected in the slit trenches of the North Mesa, AFTER PRELOADING, is approximately 1% ; and
- The liner system and proposed grading can accommodate the expected differential settlement and strain.

REFERENCES

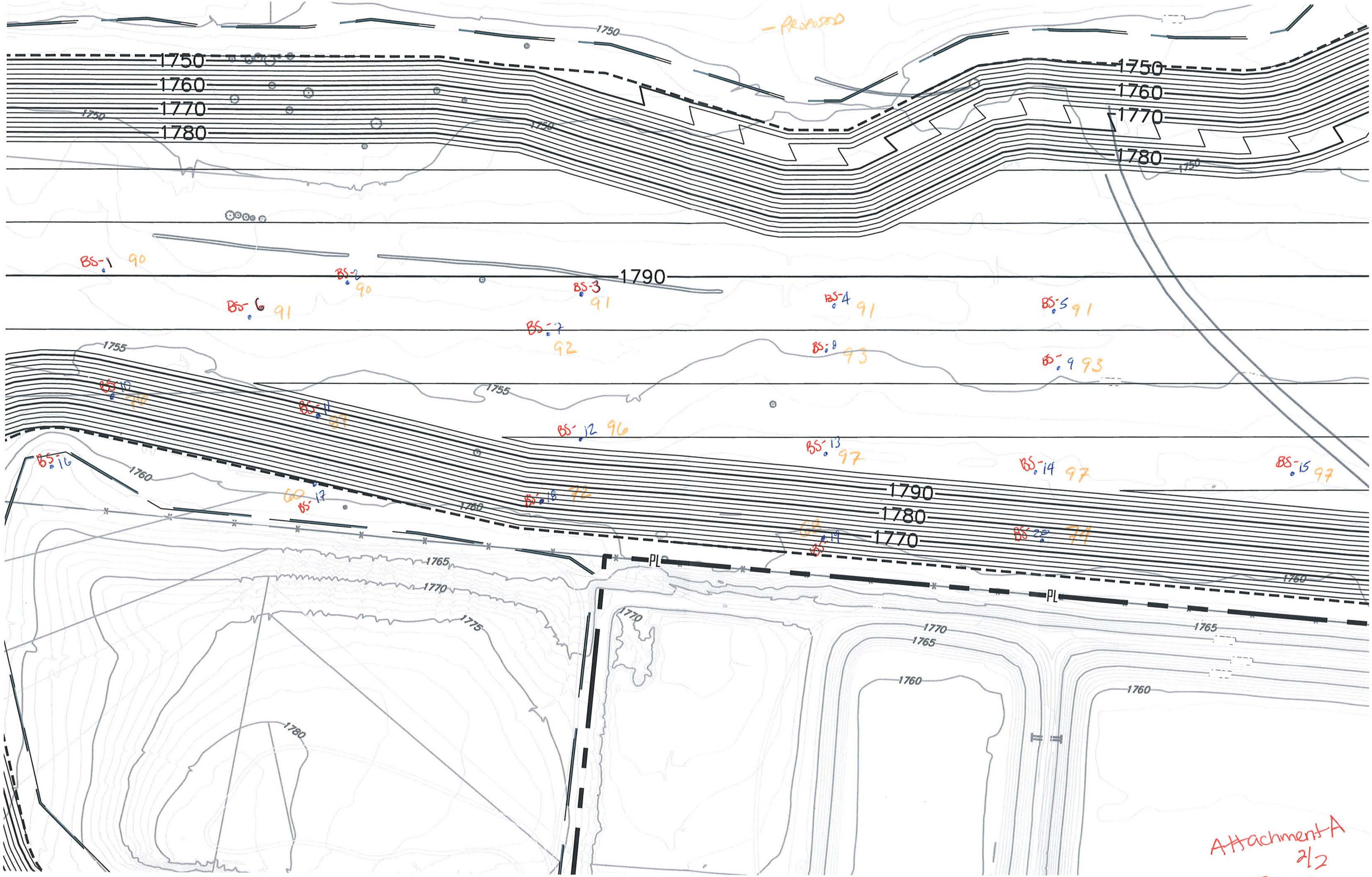
GeoSyntec (2005), "Permit Application for Class III Landfill, Henderson, Nevada", submitted to Clark County Health District, Las Vegas, Nevada. GeoSyntec Consultants, 2005.

Converse (1999), "Preliminary Geotechnical and Geologic Report - Industrial Non-Hazardous Disposal Facility (Converse 1999)", prepared for Basic Management, Inc., October 1999.

Das, Braja M, (1998), "Principles of Geotechnical Engineering, 4th Edition", PWS Publishing Company, Boston, Massachusetts.



Attachment A



Attachment A
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where e_O = initial void ratio at volume V_0. Thus, from Eqs. (8.11) through (8.14),

$$\Delta V = SA = \Delta e V_s = \frac{AH}{1 + e_O} \Delta e$$

or

$$S = H \frac{\Delta e}{1 + e_O} \tag{8.15}$$

For normally consolidated clays that exhibit a linear e–log p relationship (see Figure 8.9),

$$\Delta e = C_c[\log (p_O + \Delta p) - \log p_O] \tag{8.16}$$

where C_c = slope of the e–log p plot and is defined as the compression index. Substitution of Eq. (8.16) into Eq. (8.15) gives

$$\boxed{S = \frac{C_c H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_O} \right)} \tag{8.17}$$

For a thicker clay layer, it is more accurate if the layer is divided into a number of sublayers and calculations for settlement are made separately for each sublayer. Thus, the total settlement for the entire layer can be given as

$$S = \sum \left[\frac{C_c H_i}{1 + e_O} \log \left(\frac{p_{O(i)} + \Delta p_{(i)}}{p_{O(i)}} \right) \right]$$

where H_i = thickness of sublayer i
$p_{O(i)}$ = initial average effective overburden pressure for sublayer i
$\Delta p_{(i)}$ = increase of vertical pressure for sublayer i

In overconsolidated clays (see Figure 8.10), for $p_O + \Delta p \le p_c$, field e–log p variation will be along the line hj, the slope of which will be approximately equal to that for the laboratory rebound curve. The slope of the rebound curve, C_s, is referred to as the *swell index*, so

$$\Delta e = C_s[\log (p_O + \Delta p) - \log p_O] \tag{8.18}$$

From Eqs. (8.15) and (8.18),

$$\boxed{S = \frac{C_s H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_O} \right)} \tag{8.19}$$

If $p_O + \Delta p > p_c$, then

$$\boxed{S = \frac{C_s H}{1 + e_O} \log \frac{p_c}{p_O} + \frac{C_c H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_c} \right)} \tag{8.20}$$

Attachment B, 1/2

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The magnitude of the secondary consolidation can be calculated as

$$\boxed{S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right)} \qquad (8.27)$$

where

$$\boxed{C'_\alpha = \frac{C_\alpha}{1 + e_p}} \qquad (8.28)$$

e_p = void ratio at the end of primary consolidation (see Figure 8.18)
H = thickness of clay layer

The general magnitudes of C'_α as observed in various natural deposits are given in Figure 8.19.

Secondary consolidation settlement is more important than primary consolidation in organic and highly compressible inorganic soils. In overconsolidated inorganic clays, the secondary compression index is very small and of less practical significance.

Several factors might affect the magnitude of secondary consolidation, some of which are not clearly understood (Mesri, 1973). The ratio of secondary to primary compression for a given thickness of soil layer depends on the ratio of the stress increment (Δp) to the initial effective stress (p). For small $\Delta p/p$ ratios, the secondary-to-primary compression ratio is larger.

EXAMPLE 8.4

For a normally consolidated clay layer in the field, the following are given:

- thickness of clay layer = 8.5 ft
- Void ratio (e_O) = 0.8
- Compression index (C_c) = 0.28
- Average effective pressure on the clay layer (p_O) = 2650 lb/ft^2
- Δp = 970 lb/ft^2
- Secondary compression index (C_α) = 0.02

What is the total settlement of the clay layer five years after the completion of primary consolidation settlement? (*Note:* Time for completion of primary settlement = 1.5 years.)

Solution From Eq. (8.28),

$$C'_\alpha = \frac{C_\alpha}{1 + e_p}$$

Attachment B 2/2

COFFMAN

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6/3/05

MODELING SETTLEMENTS OF AN EXISTING MUNICIPAL SOLID WASTE LANDFILL SIDESLOPE USING AN EARTHEN SURCHARGE PILE

by

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INTRODUCTION

The large-scale field study discussed in this paper was completed for the purpose of determining design parameters for potential overfilling of existing landfill sideslopes at the Lanchester Landfill facility in Honey Brook, Pennsylvania. In particular, to maximize future waste disposal quantities at this facility, the Chester County Solid Waste Authority (CCSWA) is proposing to overfill additional municipal solid waste (MSW) within the available air space between the eastern sideslope of their closed Municipal Site Landfill and the adjacent western sideslope of Cell No. 1 of their currently active Area B landfill. In accordance with current Pennsylvania Department of Environmental Resources (DER) MSW landfill lining system regulations, CCSWA must initially install a geosynthetic lining/leachate collection system over the existing landfill sideslope prior to placement of additional waste at this location. The intent of this project was to simulate the total settlements and lateral movements of the existing waste mass when subjected to the additional applied stress of overfilled waste materials. These vertical and lateral movements would allow the maximum strains that these movements would generate within the geosynthetics of an overfill lining system to be calculated. Based on these calculated strains, appropriate geosynthetic reinforcement criteria for the overfill lining system could be developed. The reinforcement would protect the geosynthetic components of this lining system, in particular, the geomembrane, from excessive straining and possible tear resulting from underlying waste settlements.

To complete this study, an earthen surcharge pile, whose maximum height modeled the stress of the proposed overfilled waste, was constructed on the eastern sideslope of the closed Municipal Site Landfill following an initial survey of the topography of this slope.

Attachment C 1/2

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Tabulated values of the independent variables are presented in Table 2 for each of the 64 nodal point locations. These values were subsequently input into Equation 2 in order to calculate the "modified compressibility index" (C') of the waste. These values are also presented in Table 2 for each of the 64 nodal points.

It is evident from an inspection of Table 2 that the calculated values of C' define a fairly limited range of values, with the exception of nodal points H1 through H8. In particular, the C' values for the 56-nodal point database consisting of lines A through G varied from 0.15 to 0.39, averaging 0.22, while the C' database consisting of H line values varied from 0.66 to 0.98, averaging 0.80. It is clear that the H line values are anomalous. This is believed to be due to the minimal and highly variable surcharge pile thicknesses that existed in the vicinity of the nodal points along the H line resulting from the sloping geometry of the back face of the pile at these locations. As a result, the surcharge loading varies considerably in the vicinity of each H line nodal point location. Therefore, there is significant error in assuming that the load at these nodal point locations can be accurately calculated using Equation 4. Based on the above discussion, it was believed appropriate to disregard the H line C' values in generating pertinent statistics for this database as discussed in the following paragraph.

The 56-point database was analyzed statistically. A quantitative procedure was used to confirm that the database is normally distributed. The mean ($\bar{x}$), standard deviation (σ), variance (σ^2), and coefficient of variation (C_v) of the database were also calculated. These values are as follows:

$\bar{x} = 0.22$ $\qquad\qquad$ $\sigma^2 = 0.0016$

$\sigma = 0.0402$ $\qquad\qquad$ $C_v = 10.27\%$

.0402

Based on the properties of a normal distribution, it is known that 97.5% of the database lies below the random variable value of $\bar{x} + 2\sigma$. For this database, this value is equal to 0.22 + 2(~~0.402~~) = 0.30. Therefore, it can be concluded that calculation of surcharge load induced MSW settlement using Equation 1 with known values of H_w, $\bar{\sigma}_{o_w}$, and $\Delta\sigma_s$ will yield a conservative estimate of this settlement 97.5% of the time if a value of

$$\boxed{C' = \frac{C_c}{1 + e_o} = 0.30}$$ is used in the equation.

no italics *

II. Analysis Using a Multiple Linear Regression Model

The 56-point database was also analyzed using a multiple linear regression model, in which the dependent variable waste settlement (ΔH_w) was assumed to be a linear function of the independent variables waste thickness (H_w) and applied surcharge loading ($\Delta\sigma_s$). The form of this equation is as follows:

Attachment C, 2/2

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Attachment D, 1/3

GTC 6/3/03

SETTLEMENT OF MUNICIPAL SOLID WASTE LANDFILLS

by Anders Bjarngard[1] and Lewis Edgers[2]

Abstract

Landfill settlement data from 24 case histories were collected and analyzed to establish engineering parameters for the prediction of landfill settlement. Several one-dimensional laboratory compression tests were performed to evaluate the load and time dependent compression characteristics of paper. Standard geotechnical engineering parameters were used to describe settlements.

Both the field and laboratory data show large initial settlements, followed by high logarithmic rates of delayed compression. Primary consolidation was generally not observed in the field or laboratory data. The field data suggest two phases of delayed compression, one dominated by mechanical compression and a later phase possibly showing the added effects of decomposition. The delayed (secondary) compression coefficient (C_α) ranged from 0.01 to 0.056 for the laboratory data and from 0.003 to 0.038 for the first phase of the field data. The second phase, at long time periods, showed field values of C_α as large as 0.51.

Introduction

The settlement of landfills is important in a number of ways. Settlement will affect the:

- design of protection systems such as caps, leachate collection and drainage systems.

---

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2 Professor of Civil Engineering, Tufts University, Medford, Massachusetts.

Presented at the Thirteenth Annual Madison Waste Conference, September 19-20, 1990, Department of Engineering Professional Development, University of Wisconsin-Madison.

Attachment D, 2/3

TABLE 6
COMPARISON OF FIELD AND LABORATORY SETTLEMENT PARAMETERS

	INITIAL SETTLEMENTS %		C_α	
	Avg.	Range	Avg.	Range
Field Case Histories	6.3	3-11.8	$\boxed{0.019}$ 0.125	0.003-0.038 (min) 0.017-0.51 (max)
Laboratory Data	6.4[1] 2.5[2]	3.6-11[1] 0.7-5.4[2]	0.026	0.010-0.056

1. Initial settlements from the first load increment.
2. Initial settlements excluding first load increment.

Conclusions

Settlement of MSW landfills can be large and continue for very long time periods. This settlement is believed to occur due to mechanical compression, ravelling of particles and decomposition of the waste. This paper has reviewed available MSW landfill settlement data, compared these data with the results of one-dimensional laboratory tests on paper specimens, and quantified the measured compressions.

Review of settlement data from MSW landfills has shown little settlement due to primary consolidation. On the other hand, the long-term settlements due to mechanical effects (secondary compression), possible ravelling, piping or collapse, and possible chemical and biological decomposition develop over a long period of time and may become large. They are very difficult to predict because they are highly dependent upon site-specific factors such as moisture, density and composition which are usually not well defined.

The complex time-settlement behavior of landfills has been idealized by separating it into three phases: initial compression; early delayed compression; and late delayed compression. In the first phase, settlements are believed to occur because of rapid compression of the refuse and reduction of the gas voids. In the second phase, settlements are believed to be caused mostly by long-term reorientation, slippage at the particle contacts, and delayed compression of some of the MSW constituents. It is believed that in the third phase, the rate of delayed compression is greatly increased by decomposition. Ravelling may also be a contributing factor to the delayed compression phases.

The following measures of landfill settlement were observed from the case histories:

Basic Remediation Company
Corrective Action Management Unit
Henderson, Nevada

Consolidation Calculation

Material Properties			Secondary Consolidation Time		
C_{cr}	0.3	*Note 1*	time t_1	26	years
$C_{\alpha r}$	0.019	*Note 1*	time t_2	59	years
e_o	0				
γ_{waste}	85	pcf			
$\gamma_{soil\ overburden}$	115	pcf			
$\gamma_{soil\ waste}$	136	pcf			
$\gamma_{cover\ system}$	125	pcf			

Location	Depth to Waste (ft)	Depth to Native (ft)	Initial Waste Thickness (ft) H_{waste}	Soil Overburden Thickness (ft) $H_{soil\ overburden}$	Initial Overburden Presure (psf) p_0	Waste Soil Thickness (ft) $H_{soil\ waste}$	Cover System Thickness (ft) $H_{cover\ system}$	Change Overburden Pressure (psf) Δp	Primary Settlement (ft) S_P	Secondary Settlement (ft) S_S	Total Settlement (ft) S_T	Approximate Elevations: Existing Grade	Approximate Elevations: Base Liner Grade	Approximate Elevations: Final Cover Grade
MWH - BRC - BS-01	3	4	1	3	388	34	2	4,874	0.34	0.01	0.35	1754.0	1749.0	1790.0
MWH - BRC - BS-02	13	19	6	13	1,750	35	2	5,010	1.06	0.04	1.10	1753.0	1750.0	1790.0
MWH - BRC - BS-03	8	27	19	8	1,728	36	2	5,146	3.42	0.11	3.53	1753.0	1750.0	1791.0
MWH - BRC - BS-04	11	21	10	11	1,690	35	2	5,010	1.80	0.06	1.86	1754.0	1751.0	1791.0
MWH - BRC - BS-05	5	23	18	5	1,340	36	2	5,078	3.68	0.10	3.78	1753.5	1751.0	1791.0
MWH - BRC - BS-06	3	23	20	3	1,195	36	2	5,146	4.35	0.11	4.46	1753.0	1751.0	1791.0
MWH - BRC - BS-07	7	25	18	7	1,570	37	2	5,214	3.44	0.10	3.54	1753.5	1752.0	1792.0
MWH - BRC - BS-08	19	31	12	19	2,695	36	2	5,146	1.67	0.07	1.74	1755.0	1753.0	1793.0
MWH - BRC - BS-09	9	31	22	9	1,970	36	2	5,146	3.69	0.13	3.82	1755.0	1753.0	1793.0
MWH - BRC - BS-10	0	0	0	0	0	18	2	2,698	0.00	0.00	0.00	1756.0	1754.0	1776.0
MWH - BRC - BS-11	5	29	24	5	1,595	29	2	4,194	4.04	0.14	4.18	1756.0	1755.0	1787.0
MWH - BRC - BS-12	3	29	26	3	1,450	38	2	5,418	5.27	0.15	5.42	1756.0	1756.0	1796.0
MWH - BRC - BS-13	3	23	20	3	1,195	38	2	5,350	4.44	0.11	4.55	1757.5	1757.0	1797.0
MWH - BRC - BS-14	5	31	26	5	1,680	38	2	5,350	4.85	0.15	5.00	1757.5	1757.0	1797.0
MWH - BRC - BS-15	3	13	10	3	770	38	2	5,350	2.71	0.05	2.76	1757.5	1756.0	1797.0
MWH - BRC - BS-16	3	8	5	3	558	0	0	0	0.00	0.04	0.04	1762.5	1762.5	1762.5
MWH - BRC - BS-17	3	25	22	3	1,280	0	0	0	0.00	0.15	0.15	1759.0	1760.0	1760.0
MWH - BRC - BS-18	8	19	11	8	1,388	11	2	1,746	1.17	0.07	1.24	1759.0	1758.0	1772.0
MWH - BRC - BS-19	9	30	21	9	1,928	6	2	1,066	1.21	0.14	1.35	1760.0	1760.0	1768.0
MWH - BRC - BS-20	13	21	8	13	1,835	13	2	2,018	0.78	0.05	0.83	1759.0	1759.0	1774.0

Note 1.

$C_c' = 0.01$ to 0.04, $C_\alpha C = 0.001$ to 0.006 - low organic waste, C&D - "Settlement Evaluation for Cap Closure Performance", Zamiskie et. Al.

$C_c' = 0.30$ - MSW - "Modeling Settlements of an Existing MSW Landfill Side Slope Using an Earthen Surcharge Pile", Deutsch et. Al.

$C_c' = 0.063$, $C_\alpha C = 0.019$ - MSW LFs - "Settlement of Municipal Waste Landfills", Bjarngard et. Al.

High values selected to reflect the uncompacted nature of the waste materials placed in the "slit trenches".

- input value

Basic Remediation Company
Corrective Action Management Unit
Henderson, Nevada

Strain Calculation				
Maximum Allowable Strain	1.00%			
Location	**Total Settlement** **(ft)** **S_T**	**Distance to Native** *Note 1* **(ft)** **L**	**Strain** **(%)** **ε**	**Surcharge Settlement (Required)** **(ft)**
MWH - BRC - BS-01	0.35	20	0.02%	0.00
MWH - BRC - BS-02	1.10	20	0.15%	0.00
MWH - BRC - BS-03	3.53	20	1.55%	2.52
MWH - BRC - BS-04	1.86	10	1.72%	0.85
MWH - BRC - BS-05	3.78	10	6.91%	2.77
MWH - BRC - BS-06	4.46	10	9.50%	3.45
MWH - BRC - BS-07	3.54	10	6.08%	2.53
MWH - BRC - BS-08	1.74	10	1.50%	0.73
MWH - BRC - BS-09	3.82	10	7.05%	2.81
MWH - BRC - BS-10	0.00	38	0.00%	0.00
MWH - BRC - BS-11	4.18	38	0.60%	0.00
MWH - BRC - BS-12	5.42	20	3.61%	4.41
MWH - BRC - BS-13	4.55	20	2.56%	3.54
MWH - BRC - BS-14	5.00	20	3.08%	3.99
MWH - BRC - BS-15	2.76	20	0.95%	0.00
MWH - BRC - BS-16	0.04	23	0.00%	0.00
MWH - BRC - BS-17	0.15	23	0.00%	0.00
MWH - BRC - BS-18	1.24	23	0.15%	0.00
MWH - BRC - BS-19	1.35	13	0.54%	0.00
MWH - BRC - BS-20	0.83	13	0.20%	0.00

Note 1
Half the width of the "Slit Trench".

| | - input value |

Basic Remediation Company
Corrective Action Management Unit
Henderson, Nevada

Surcharge Height Calculation

Material Properties		
C_{cr}	0.3	*Note 1*
γ_{waste}	85	pcf
$\gamma_{soil\ overburden}$	115	pcf
$\gamma_{surcharge}$	135	pcf

Location	Depth to Waste (ft)	Depth to Native (ft)	Initial Waste Thickness (ft) H_{waste}	Soil Overburden Thickness (ft) $H_{soil\ overburden}$	Initial Overburden Presure (psf) p_0	Settlement Required (ft)	Surcharge Height (ft) $H_{surcharge}$	Minimum Surcharge Elevation
MWH - BRC - BS-01	3	4	1	3	388	0.00	0	1,754
MWH - BRC - BS-02	13	19	6	13	1,750	0.00	0	1,753
MWH - BRC - BS-03	8	27	19	8	1,728	2.52	23	1,776
MWH - BRC - BS-04	11	21	10	11	1,690	0.85	12	1,766
MWH - BRC - BS-05	5	23	18	5	1,340	2.77	22	1,776
MWH - BRC - BS-06	3	23	20	3	1,195	3.45	24	1,777
MWH - BRC - BS-07	7	25	18	7	1,570	2.53	23	1,776
MWH - BRC - BS-08	19	31	12	19	2,695	0.73	12	1,767
MWH - BRC - BS-09	9	31	22	9	1,970	2.81	24	1,779
MWH - BRC - BS-10	0	0	0	0	0	0.00	0	1,756
MWH - BRC - BS-11	5	29	24	5	1,595	0.00	0	1,756
MWH - BRC - BS-12	3	29	26	3	1,450	4.41	29	1,785
MWH - BRC - BS-13	3	23	20	3	1,195	3.54	26	1,783
MWH - BRC - BS-14	5	31	26	5	1,680	3.99	28	1,785
MWH - BRC - BS-15	3	13	10	3	770	0.00	0	1,758
MWH - BRC - BS-16	3	8	5	3	558	0.00	0	1,763
MWH - BRC - BS-17	3	25	22	3	1,280	0.00	0	1,759
MWH - BRC - BS-18	8	19	11	8	1,388	0.00	0	1,759
MWH - BRC - BS-19	9	30	21	9	1,928	0.00	0	1,760
MWH - BRC - BS-20	13	21	8	13	1,835	0.00	0	1,759

- input value

Section 3
Base Liner Construction Quality Assurance Plan

CONSTRUCTION QUALITY ASSURANCE PLAN
FOR THE
CONSTRUCTION OF

BASE LINER SYSTEM
AT
CORRECTIVE ACTION MANAGEMENT UNIT
BASIC REMEDIATION COMPANY
HENDERSON, NEVADA

Prepared for:

BasicEnvironmental
COMPANY

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3 November 2006

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1. INTRODUCTION

1.1 Terms of Reference

GeoSyntec Consultants (GeoSyntec) has prepared this Construction Quality Assurance (CQA) Plan for the construction of the Corrective Action Management Unit (CAMU) Base Liner System for Basic Remediation Company (BRC) located in Henderson, Nevada. Hereinafter, the CAMU construction is referred to as the Project.

This CQA Plan was prepared by Mr. Gregory T. Corcoran, P.E. of GeoSyntec Consultants (GeoSyntec) under the direction of Mr. James A. McKelvey III, P.E. In general accordance with the peer review policies of the firm, Mr. James A. McKelvey III, P.E. of GeoSyntec was responsible for senior peer review of the work presented in this plan.

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 engineered fill, prepared subgrade, operations layer material, drainage aggregate, geosynthetic clay liner, geomembrane, geotextile, geocomposite, and polyethylene pipe. It should be emphasized that care and documentation are required in the placement and compaction of the soils and aggregate and in the production and installation of the geosynthetic materials placed during construction. The CQA Plan, therefore, delineates the procedures to be followed for monitoring construction of these materials.

The scope of this CQA Plan includes the CQA of the soil and geosynthetic components of the Project. The CQA monitoring activities during the selection, evaluation, treatment, placement, and compaction of soils for earthworks, and 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 geosynthetic clay liner;
- Section 13 presents CQA of the geocomposite; and
- Section 16 presents CQA surveying.

2. DEFINITIONS RELATING TO CONSTRUCTION QUALITY ASSURANCE

This CQA Plan is devoted to Construction Quality Assurance. In the context of this document, Construction Quality Assurance and Construction Quality Control are defined as follows:

Construction Quality Assurance (CQA) - A planned and systematic pattern of means and actions designed to assure adequate confidence that materials and/or services meet contractual and regulatory requirements and will perform satisfactorily in service.

Construction Quality Control (CQC) - Those actions which provide a means to measure and regulate the characteristics of an item or service in relation to contractual and regulatory requirements.

In the context of this document:

- 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*.

- 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 soil components, CQC is combined with CQA and is provided by the CQA Consultant. In the case of the geosynthetic components and piping of the Work, CQC is provided by the Manufacturer and Geosynthetic Installer and the Contractor. CQA testing of soil, pipe, and geosynthetic components is provided by the CQA Consultant.

3. PARTIES INVOLVED WITH CONSTRUCTION QUALITY ASSURANCE

3.1 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 Parsons Engineering Science, Inc. (Parsons) and GeoSyntec.

Qualifications

The Engineer of Record shall be a qualified engineer, registered as required by Nevada state regulations. The Engineer should have expertise, which demonstrates significant familiarity with piping, geosynthetics and soils, as appropriate, including design and construction experience related to landfill liner systems.

3.2 Project Manager

Responsibilities

The Project Manager is responsible for implementing the design, and overseeing subcontractors. In this CQA Plan, the term “Project Manager” refers to a qualified BRC employee.

Qualifications

The Project Manager shall be a qualified engineer having familiarity with earthwork construction and installation of geosynthetic materials.

3.3 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 and geosynthetic components of the liner system. This work will include excavation, placement and compaction of engineered fill and prepared subgrade, placement of drainage aggregate and native soil (operations layer material), installation and of piping and concrete manhole, installation of temporary erosion control features, 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 construction and maintain current state and federal licenses as appropriate.

3.4 <u>Resin Supplier</u>

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.

3.5 <u>Geosynthetics Manufacturer</u>

Responsibilities

The Manufacturer is responsible for the production of finished material (geomembrane, geotextile, geosynthetic clay liner, geocomposite, pipe, and other specified material) 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*.

3.6 Geosynthetic Installer

Responsibilities

The Geosynthetic Installer is responsible for field handling, storage, placement, seaming, loading 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*.

3.7 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 the geosynthetic components used in the construction of the Project. The CQA Consultant will also be responsible for issuing a CQA report at the completion of the Project construction, which details the earthworks, piping, and geosynthetic installation activities 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 Nevada.

The CQA Consultant will be responsible for obtaining and testing representative samples of all components used in construction of the Project as required by this CQA Plan and *Technical Specifications*. All tests will be conducted in general accordance with ASTM or other applicable state or federal standards. Test results must be submitted to the Project Manager within a reasonable timeframe, which will not impede or delay construction of the Project. The CQA Consultant will be responsible for inspecting all earthwork, piping, and geosynthetic operations to verify that the components are installed in general accordance with this CQA Plan and *Technical Specifications*.

Qualifications

The CQA Consultant is a well established firm specializing in geotechnical and geosynthetic engineering and 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 Nevada State regulations.

The CQA Consultant will be experienced with earthwork construction and the 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 the installation of geosynthetic materials and will be trained by the CQA Consultant in the duties of a CQA Site Manager.

3.8 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 Owner. 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 experience in the profession of surveying services in the State of Nevada. The Surveyor will be a licensed professional as required by the State of Nevada regulations. The Surveyor shall be fully equipped and experienced in the use of total stations and AutoCAD Version 14. All surveying will be performed under the direct supervision of the Owner.

3.9 CQA Laboratory

Responsibilities

The CQA Laboratory is a party, independent from the Contractor, Manufacturer and 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.

4. CQA CONSULTANTS PERSONNEL ORGANIZATION AND DUTIES

4.1 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:

- excavation and screening of materials;
- placement and compaction of engineered fill, surcharge prepared subgrade, and operations layer material;
- installation of geotextile;
- installation of geosynthetic clay liner;
- installation of geomembrane;
- installation of drainage aggregate;
- installation of geocomposite;
- installation of cast-in-place concrete; and
- installation of piping.

The duties of the CQA personnel are discussed in the remainder of this section.

4.2 CQA Personnel

For construction of the Project, the CQA Consultant's personnel will include:

- the CQA Officer, who operates 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.

The duties of the CQA Personnel are discussed in the following subsections.

4.2.1 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 QC Plan;

- attends resolution and/or 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.

4.2.2 CQA Site Manager

The CQA Site Manager:

- acts as the on-site representative of the CQA Consultant;

- attends CQA-related meetings (e.g., resolution, pre-construction, daily, weekly (or designates a representative to attend the meeting));

- prepares or 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 Manufacturer's QC documentation;

- reviews the Geosynthetic Installer’s personnel Qualifications for conformance with those pre-approved for work on site;

- notes in the daily summary report and reports to the CQA Officer and Project Manager on-site activities that could result in damage to the geosynthetic materials or other completed work;

- reports unresolved deviations from the CQA Plan, *Drawings*, and *Technical Specifications* to the Project Manager; and

- assists with the preparation of the CQA report.

5. SITE AND PROJECT CONTROL

5.1 <u>Project Coordination Meetings</u>

Meetings of key project personnel are necessary to assure a high degree of quality during installation, and 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) resolution meetings; (ii) pre-construction meetings; (iii) progress meetings; and (iv) problem or work deficiency meetings.

5.1.1 Resolution Meeting

Following the completion of the design, *Drawings*, and *Technical Specifications* for the project and prior to the start of construction, a Resolution Meeting will be held. This meeting may include the CQA Officer, the CQA Site Manager, the Engineer, and the Project Manager.

The purpose of this meeting is to begin planning for coordination of construction tasks, anticipate installation problems which might cause difficulties and delays in construction, and, above all, present the CQA Plan to the parties involved. It is very important that the criteria regarding testing, repair, and other CQA activities be known and accepted by the parties involved in the work prior to the installation of geosynthetic materials and construction of the soil components for the Project.

The first part of the Resolution Meeting may be devoted to a review of the *Drawings* and *Technical Specifications* for familiarity. This is different from the peer review of the design, including design calculations, which will have been carried out previously.

The Resolution Meeting may include the following activities:

- distribute relevant documents to all parties;
- review critical design details of the project;

- review this CQA Plan;
- review the Drawings and Technical Specifications;
- make appropriate modifications to the design criteria, *Drawings*, and *Technical Specifications* so that the fulfillment of the design specifications or performance standards can be determined through the implementation of the CQA Plan;
- reach a consensus on the quality control procedures, especially on methods of evaluating acceptability of the soils and geosynthetic materials;
- assign the responsibilities of each party;
- establish work area security and health and safety protocol;
- confirm the methods for documenting observations, reporting, and distributing documents and reports; and
- confirm the lines of authority and communication.

The Project Manager will appoint one of the meeting attendees to record the discussions and decisions of the Resolution Meeting. The record of the meeting will be documented by the appointee in the form of meeting minutes, which will be subsequently distributed to all attendees.

5.1.2 Pre-Construction Meeting

A Pre-Construction Meeting will be held at the site prior to construction of the Project. As a minimum, the Pre-Construction Meeting will be attended by the Contractor, the Geosynthetic Installer’s Superintendent, the CQA Consultant, the Engineer, and the Project 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 Project 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 Project 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.

5.1.3 Progress Meetings

Progress meetings will be held between the CQA Site Manager, the Contractor, Project 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.

5.1.4 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 Project 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 Project 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.

6. DOCUMENTATION

6.1 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 all quality assurance requirements have been addressed and satisfied.

The CQA Site Manager will provide the Project 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.

6.2 Daily Recordkeeping

Preparation of daily CQA documentation will consist of daily 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 Project Manager.

The CQA Site Manager will prepare daily reports that document the activities observed during each day of activity. The daily 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 inspected;
- type of construction and inspection 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.

6.3 Construction Problems and Resolution Data Sheets

Construction Problems and Resolution Data Sheets, to be submitted with the daily reports prepared by the CQA Site Manager, describing special construction situations will be cross-referenced with daily 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 Project Manager.

The Project 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 Engineer 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.

6.4 Photographic Documentation

Photographs will be taken and documented in order to serve as a pictorial record of work progress, problems, and mitigation activities. The basic file will contain color prints. Negatives will also be stored in a separate file in chronological order. These records will be presented to the Project 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). Photographs used for documentation will be identified with the date, time, and location of the photograph.

6.5 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 Project Manager. Design and/or specification changes will be made with the written agreement of the Project Manager and the Engineer and will take the form of an addendum to the *Drawings* and *Technical Specifications*.

6.6 CQA Report

At the completion of the Project, the CQA Consultant will submit to the Project Manager the CQA report signed and sealed by the Professional Engineer licensed in the State of Nevada. 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:

- Manufacturers' quality control 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 personnel resumes and qualifications;
- 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 inspected 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;
- documentation showing that piping was tested in general accordance with the *Technical Specifications*; and
- daily inspection reports.

The record drawings will include scale drawings depicting the location of the construction and details pertaining to the extent of construction (e.g., depths, plan dimensions, elevations, soil component thicknesses). Base maps required for development of the record drawings and the record drawings will be prepared by a qualified Professional Land Surveyor registered in the State of Nevada. These documents will be reviewed by the CQA Consultant and included as part of the CQA Report.

7. EARTHWORKS

7.1 <u>Introduction</u>

This section prescribes the CQA activities to be performed to monitor that earthwork components are constructed in general accordance with *Drawings* and *Technical Specifications*. The earthworks construction procedures to be monitored by the CQA Consultant include:

- excavation;
- surcharge placement;
- engineered fill placement;
- anchor trench excavation and backfill;
- subgrade preparation; and
- operations layer material placement.

7.2 <u>Testing Activities</u>

Soil testing will be performed for material qualification, material conformance, and construction quality control (CQC). These three stages of testing are defined as follows:

- Material qualification tests are used to evaluate the conformance of a proposed soil source to the material specifications for qualification of the source prior to construction.

- Soils conformance testing is used to evaluate the conformance of a particular batch of soil from a qualified source to the material specifications prior to installation of the soil.

- CQC tests are performed on completed portions of the earthwork during construction to demonstrate that the placement procedures are resulting in a product that meets or exceeds both material and performance specifications.

The Contractor will be responsible for submitting material qualification test results to the Project Manager and to the CQA Site Manager for review. The CQA Laboratory will perform the conformance testing and CQC testing. Soil 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 soils testing for material qualification will conform to the minimum frequencies presented in Table 2. The frequency of soils testing for material conformance will conform to the minimum frequencies presented in Table 3. 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 or Test Location 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 soil available for sampling in a timely and organized manner so that the test results can be obtained before the material is installed. The CQA Site Manager must document sample locations so that failing areas can be immediately isolated. The CQA Site Manager will follow standard sampling procedures to obtain representative samples of the proposed soil materials.

CQC sample and test locations will be selected by the CQA Site Manager at the minimum test frequency specified in Table 4. Samples and test locations will generally be selected at random, however a special testing frequency will be used at the discretion of the CQA Site Manager when visual observations of construction performance indicate a potential problem. Additional testing for suspected areas will be considered when:

- rollers slip during rolling operation;

- lift thickness is greater than specified;
- fill is at improper and/or variable moisture content;
- less than specified number of roller passes are made;
- dirt-clogged rollers are used to compact the material;
- rollers may not have used optimum ballast;
- fill materials differ substantially from those specified;
- the degree of compaction is doubtful; and
- as directed by the Project Manager or the CQA Site Manager.

The frequency of testing may also be increased in the following situations:

- adverse weather conditions;
- breakdown of equipment;
- at the start and finish of grading;
- material fails to meet specifications; and
- the work area is reduced.

7.3 CQA Monitoring Activities

7.3.1 Earthwork

The CQA Site Manager will monitor and document the earthworks required for the Project. In general, monitoring the construction for earthwork includes the following activities:

- reviewing documentation of the material qualification test results provided by the Contractor;

- monitoring the prepared subgrade and subgrade surfaces for compliance with the *Technical Specifications* before geosynthetic materials are placed;

- sampling and testing for conformance of the materials to the *Technical Specifications*;

- documenting that the earthwork is constructed using the specified equipment and procedures;

- documenting that the earthwork is constructed to the lines and grades shown on the *Drawings*;

- monitoring that the construction activities do not cause damage to underlying geosynthetic materials;

- quality control testing to determine the acceptability of the work during construction; and

- monitoring the action of the compaction and heavy hauling equipment on the construction surface (i.e., penetration, pumping, cracking, etc.).

The specific activities required for CQA of each of the major soil components of the Liner System are presented in the following sections.

7.3.2 Engineered Fill Material

Monitoring the earthwork for the engineered fill material specifically includes the following:

- reviewing documentation of the qualification and conformance test results;

- monitoring soil for maximum particle size and deleterious materials;

- monitoring the thickness of lifts during placement of the materials;

- monitoring compaction operations; and

- measuring and recording the field density and the field moisture content of the in-place material.

7.3.3 Prepared Subgrade

During construction, the CQA Site Manager will monitor the prepared subgrade to document that the prepared subgrade soil characteristics are consistent with those specified in the *Technical Specifications*. The CQA Site Manager will monitor the construction activities to document that sharp rocks and other undesirable materials are removed and that the subgrade is prepared using the procedures and equipment specified in the *Technical Specifications*.

The upper portion of the subgrade can be damaged by excess moisture (causing softening) or insufficient moisture (causing desiccation and shrinkage). At a minimum, the CQA Site Manager will determine the suitability of the subgrade for geomembrane placement by:

- documenting that the surface is free of sharp rocks, debris and other undesirable materials;

- documenting that the surface is smooth, uniform, and free from desiccation cracks by visually monitoring proof rolling activities; and

- documenting that the subgrade surface meets the lines and grades shown on the *Drawings* by reviewing certified survey results.

7.3.4 Operations Layer Material

The CQA Site Manager will monitor the earthwork of the operations layer material for the following:

- the Contractor’s submittals and qualification test results for consistency between the proposed methods and the approved methods;

- the conformance testing of the material and notifying the Contractor of results for compliance with material specifications;

- the thickness of lifts during placement;

- the placement equipment operation on the sideslopes is in general accordance with the *Technical Specifications*;

- the construction procedures to monitor that completed sections of liner and geomembrane are protected from damage; and

- the survey data to monitor that operations layer material is constructed to the proposed lines and grades and to the specified thickness.

7.4 Deficiencies

If a defect is discovered in the earthwork product, the CQA Site Manager will immediately determine the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Site Manager will determine the extent of the deficient 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 surface desiccation, the CQA Site Manager will define the limits and nature of the defect.

7.4.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Project Manager and Contractor and schedule appropriate re-tests when the work deficiency is to be corrected.

7.4.2 Repairs and Re-Testing

At locations where the field testing indicates densities below the requirements of the specification, the failing area will be reworked. 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 Engineer and/or Project Manager suggested solutions for his approval.

All re-tests recommended by the CQA Site Manager must verify 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. DRAINAGE AGGREGATE

8.1 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.

8.2 Testing Activities

Aggregate testing will be performed for material qualification and material conformance. These two stages of testing are defined as follows:

- Material qualification tests are used to evaluate the conformance of a proposed aggregate source to the material 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 material specifications prior to installation of the aggregate.

The Contractor will be responsible for submitting material qualification test results to the Project 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 5 are those that will be used for this testing unless the test methods are updated or revised prior to construction. Revisions to the test methods will be reviewed and approved by the Engineer and the CQA Site Manager prior to their usage.

8.2.1 Sample Frequency

The frequency of aggregate testing for material qualification will conform to the minimum frequencies presented in Table 6. The frequency of aggregate testing for material conformance will conform to the minimum frequencies presented in Table 7.

The actual frequency of testing required will be increased by the CQA Site Manager as necessary if variability of materials is noted at the site, during adverse conditions, or to isolate failing areas of the construction.

8.2.2 Sample Selection

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

8.3 CQA Monitoring Activities

8.3.1 Drainage Aggregate

The CQA Site Manager will monitor and document the installation of the drainage aggregates. In general, monitoring the installation of the drainage aggregates 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.

8.4 Deficiencies

If a defect is discovered in the drainage aggregates, the CQA Site Manager will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Site Manager will determine the extent of the deficient area by additional tests, observations, a review of records, or other means that the CQA Site Manager deems appropriate.

8.4.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Project Manager and Contractor and schedule appropriate re-tests when the work deficiency is to be corrected.

8.4.2 Repairs and Re-testing

The Contractor will correct the deficiency to the satisfaction of the CQA Site Manager. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Site Manager will develop and present to the Engineer and/or Project Manager suggested solutions for approval.

All re-tests recommended by the CQA Site Manager must verify that the defect has been corrected before any additional work is performed by the Contractor in the area of the deficiency. The CQA Site Manager will also verify that installation requirements are met and that submittals are provided.

9. HIGH DENSITY POLYETHYLENE (HDPE) PIPE AND FITTINGS

9.1 Material Requirements

HDPE pipe and fittings must conform to the requirements of the *Technical Specifications*. The CQA Consultant will document that the HDPE pipe and fittings meet those requirements through manufacturer's quality control certificates, conformance testing, and visual examination of materials arriving on site.

9.2 Manufacturer

9.2.1 Submittals

Prior to the installation of HDPE 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

- a certification that property values given in the properties sheet are minimum values and are guaranteed by the Manufacturer.

The CQA Consultant will document that:

- the property values certified by the Manufacturer meet the *Technical Specifications*; and

- the measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

9.2.2 Identification

Prior to shipment, the Manufacturer will provide the Project Manager and the CQA Site Manager with a quality control certificate for each lot/batch of HDPE pipe provided. The quality control certificate will be signed by a responsible party employed by the Manufacturer, such as the Production Manager. The quality control certificate will include:

- lot/batch numbers and identification; and
- sampling procedures and results of quality control tests.

The CQA Site Manager will:

- document that the quality control certificates have been provided at the specified frequency for all lots/batches of pipe, and that each certificate identifies the pipe lot/batch related to it; and

- review the quality control certificates and document that the certified properties meet the *Technical Specifications*.

9.3 Handling and Laying

Care will be taken during transportation of the pipe such that it will not be cut, kinked, or otherwise damaged.

Ropes, fabric, or rubber-protected slings and straps will be used when handling pipes. Chains, cables, or hooks inserted into the pipe ends will not be used. Two slings spread apart will be used for lifting each length of pipe. Pipe or fittings will not be dropped onto rocky or unprepared ground.

Pipes will be handled and stored in 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 butt-fused joints. Sections of the pipes with deep cuts and gauges will be removed and the ends of the pipe rejoined.

9.4 Joints

Lengths of pipe will be assembled into suitable installation lengths by the butt-fusion process. Butt-fusion means the butt-joining of the pipe by softening by heat the aligned faces of the pipe ends in a suitable apparatus and pressing them together under controlled pressure. This process will be applied by personnel experienced with the process. Certification will be provided that the person performing this work is qualified by experience and instruction in the procedure. All pipe so joined will be made from the same class and type of raw material made by the same raw material supplier.

10. GEOMEMBRANE

10.1 General

This section discusses and outlines the CQA activities to be performed for high density polyethylene (HDPE) geomembrane installation. The CQA Site Manager will review the *Drawings*, and the *Technical Specifications*, and any approved Addenda regarding this material.

10.2 Geomembrane Material Conformance

10.2.1 Introduction

The CQA Site Manager will document that the geomembrane delivered to the site meets the requirements of the *Technical Specifications* prior to installation. The CQA Site Manager will:

- review the manufacturer's submittals for compliance with the *Technical Specifications*;

- document the delivery and proper storage of geomembrane rolls; and

- conduct conformance testing of the rolls before the geomembrane is installed.

The following sections describe the CQA activities required to verify the conformance of geomembrane.

10.2.2 Review of Quality Control

10.2.2.1 Material Properties Certification

The Manufacturer will provide the Project Manager and the CQA Site Manager with the following:

- a properties sheet including, at a minimum, all specified properties, measured using test methods indicated in the *Technical Specifications*, or equivalent;
- the sampling procedure and results of testing; and
- a certification that property values given in the properties sheet are guaranteed by the Manufacturer.

The CQA Site Manager will document that:

- the property values certified by the Manufacturer meet all of the *Technical Specifications*; and
- the measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

10.2.2.2 Resin Certification

The Manufacturer will also provide the Project Manager with the following information concerning the resin used to manufacture the geomembrane:

- the origin (Resin Supplier's name and resin production plant), identification (brand name, lot number), and production date of the resin; and
- the raw material quality control certificates.

The CQA Site Manager will:

- evaluate that the quality control certificates have been provided at the specified frequency, and that the certificate identifies the rolls related to it; and

- review the quality control certificates and evaluate that the certified properties meet the specifications.

10.2.2.3 Geomembrane Roll QC Certification

Prior to shipment, the Manufacturer will provide the Project Manager and the CQA Site Manager with a quality control certificate for every roll of geomembrane provided. The quality control certificate will be signed by a responsible party employed by the Geomembrane Manufacturer, such as the production manager. The quality control certificate will include:

- roll numbers and identification; and

- results of quality control tests - as a minimum, results will be given for thickness, specific gravity, carbon black content, carbon black dispersion, tensile properties, tear resistance, puncture resistance, and single point stress rupture evaluated in general accordance with the methods indicated in the specifications or equivalent methods approved by the Engineer.

The CQA Site Manager will:

- evaluate that the quality control certificates have been provided at the specified frequency, and that the certificate identifies the rolls related to the roll represented by the test results; and

- review the quality control certificates and evaluate that the certified roll properties meet the specifications.

10.2.3 Conformance Testing

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 will not be installed. The CQA Site Manager shall obtain conformance samples 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 8. 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 (1 m) of material. Unless otherwise specified, samples will be 3 ft (1 m) 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 8.

The CQA Site Manager will examine results from laboratory conformance testing and will report any non-conformance to the Project Manager and the Geosynthetic Installer. The procedure prescribed in the *Technical Specifications* will be followed in the event of a failing conformance test.

10.3 <u>Delivery</u>

10.3.1 Transportation and Handling

The CQA Site Manager will document that the transportation and handling does not pose a risk of damage to the geomembrane.

Upon delivery at the site, the Geosynthetic Installer and the CQA Site Manager will conduct a surface observation of the rolls for defects and damage. This inspection will be conducted without unrolling unless defects or damages are found or suspected. The CQA Site Manager will indicate to the Project Manager:

- rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and

- rolls that include minor repairable flaws.

10.3.2 Storage

The Geosynthetic Installer will be responsible for the storage of the geomembrane on site. The Contractor will provide storage space in a location (or several locations) such that on-site transportation and handling are optimized if possible.

The CQA Site Manager will document that storage of the geomembrane provides adequate protection against sources of damage.

10.4 Geomembrane Installation

10.4.1 Introduction

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

10.4.2 Earthwork

10.4.2.1 Surface Preparation

The CQA Site Manager will document that:

- a qualified land surveyor has verified lines and grades;

- that the supporting prepared subgrade or subgrade meets 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 the 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 will be given by the Geosynthetic Installer to the Project Manager prior to commencement of geomembrane installation in the area under consideration. The CQA Site Manager will be given a copy of this certificate by the Project Manager.

After the supporting subgrade has been accepted by the Geosynthetic Installer, it will be the Geosynthetic Installer's responsibility to indicate to the Project Manager any change in the supporting soil condition that may require repair work. If the CQA Site Manager concurs with the Geosynthetic Installer, then the Project Manager will document that the supporting soil is repaired.

At any time before and during the geomembrane installation, the CQA Site Manager will indicate to the Project Manager locations that may not provide adequate support to the geomembrane.

10.4.2.2 Geosynthetic Termination

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

10.4.3 Geomembrane Placement

10.4.3.1 Panel Identification

A field panel is the unit area of geomembrane which is to be seamed in the field, i.e., a field panel is a roll or a portion of roll cut in the field. It will be the responsibility of the CQA Site Manager to document that each field panel is given an "identification code" (number or letter- number) consistent with the layout plan. This identification code will be agreed upon by the Project 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 quality assurance records.

10.4.3.2 Field Panel Placement

Location

The CQA Site Manager will document that field panels are installed at the location indicated in the Geosynthetic Installer's layout plan, as approved or modified by the Engineer.

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 Project Manager on the acceptability of that change. The CQA Site Manager will document that the condition of the supporting 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 104°F;
- when the geomembrane sheet temperature is below 40°F or above 104°F; or
- when wind gusts are in excess of 20 mph.

Geomembrane placement will not be performed 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 (i.e., wind gusts in excess of 20 mph).

The CQA Site Manager will document that the above conditions are fulfilled. Additionally, the CQA Site Manager will document that the supporting soil has not been damaged by weather conditions. The Geosynthetics Installer will inform the Project 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 Project 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 10.4.5.

10.4.4 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.

10.4.4.1 Seam Layout

The Geosynthetic Installer will provide the Project Manager and the CQA Site Manager with a seam layout drawing, i.e., a drawing of the facility to be lined showing all expected seams. The CQA Site Manager will review the seam layout drawing and evaluate that it is consistent with the preliminary geomembrane panel layout. No panels may be seamed in the field without the Project Manager's approval. In addition, panels not specifically shown on the seam layout drawing may be used without the Project Manager's prior approval.

In general, seams should be oriented parallel to the line of maximum slope, i.e., oriented along, not across, the slope. In corners and odd-shaped geometric locations, the number of seams should be minimized. No horizontal seam should be less than 5 ft (1.5 m) from the toe of the slope, or areas of potential stress concentrations, unless otherwise authorized.

A seam numbering system compatible with the panel numbering system will be agreed upon at the Resolution and/or Pre-Construction Meeting.

10.4.4.2 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 Project Manager and the CQA Site Manager with a list of proposed seaming personnel and their experience records. This document will be reviewed by the Project Manager and the Geosynthetics CQA Manager.

10.4.4.3 Seaming Equipment and Products

Approved processes for field seaming are fillet extrusion welding and 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 extrudate to the Project Manager and the CQA Site Manager, and will certify that the extrudate is compatible with the specifications, and in any event is comprised of the same resin as the geomembrane sheeting.

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 Resolution 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 pressures.

The CQA Site Manager will also document that:

- the Geosynthetic Installer maintains on-site the number of spare operable seaming apparatus decided at the Resolution 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 base such that no damage occurs to the geomembrane;

- a smooth insulating plate or fabric is placed beneath the hot welding apparatus after usage;

- the geomembrane is protected from damage in heavily trafficked areas; and

- a movable protective layer may be used directly below each overlap of geomembrane that is to be seamed to prevent build- up of moisture between the sheets.

10.4.4.4 Seam Preparation

The CQA Site Manager will document that:

- prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris, and foreign material; and

- seams are aligned with the fewest possible number of wrinkles and "fishmouths."

10.4.4.5 Weather Conditions for Seaming

The normally required weather conditions for seaming are as follows unless authorized in writing by the Project Manager:

- seaming will only be approved between ambient temperatures of 40°F (4°C) and 104°F (40°C); and

- seaming will not be approved if sustained wind speed is in excess of 20 mph (32 km/hr).

If the Geosynthetic Installer wishes to use methods that may allow seaming at ambient temperatures below 40°F (4°C) or above 104°F (40°C), the Geosynthetic Installer will demonstrate and certify that such methods produce seams which are entirely equivalent to seams produced within acceptable temperature and wind requirements, 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 Project Manager if they are not. The Project Manager will then decide if the installation will be stopped or postponed.

10.4.4.6 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. (75 mm) for both extrusion and fusion welding;

- no solvent or adhesive bonding material are to be used; and

- the procedure used 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 to the Project Manager non-compliances.

10.4.4.7 Trial Seams

Trial seams will be made on fragment pieces of geomembrane liner to verify that seaming conditions are adequate. Such trial seams will be made at the beginning of each seaming period, beginning of the day and after lunch, for each seaming apparatus used that day. Also, each seamer will make at least one trial seam each day. Trial seams will be made under the same conditions as actual seams.

Extrusion welded trial seam samples will be at least 3 ft (0.9 m) long by 1 ft (0.3 m) wide (after seaming) with the seam centered lengthwise. Fusion welded trial seam samples will be at least 5 ft (1.5 m) long by 1 ft (0.3 m) wide (after seaming) with the seam centered lengthwise. Seam overlap will be as indicated in Section 10.5.3.6.

Four specimens, each 1 in. (25 mm) wide, will be cut from the trial seam sample by the Geosynthetic Installer. One specimen will be tested for shear strength and three specimens will be tested for peel adhesion using a gauged tensiometer. All specimens tested will exhibit a Film Tear Bond (FTB) and will not fail in the seam. In addition, all specimens will meet or exceed the minimum strength requirements described in the *Technical Specifications*. If any of the four specimens fails, the entire trial seaming operation will be repeated. If any of the four additional specimens fails, the seaming apparatus and seamer will not be approved for production seaming until the deficiencies are corrected and two consecutive trial seam tests achieve the FTB requirements outlined above.

The CQA Site Manager will observe trial seam procedures. Trial seam samples will be assigned a number. The CQA Site Manager, will log the date, time, machine temperature(s), number of the seaming unit, name of the seamer, and pass or fail description for each trial seam sample tested.

10.4.4.8 General Seaming Procedure

Unless otherwise specified, the general seaming procedure used by the Geosynthetic Installer will be as follows:

- Fishmouths or wrinkles at the seam overlaps will be cut along the ridge of the wrinkle in order to achieve a flat overlap. The cut fishmouths or wrinkles will be seamed and any portion where the overlap is inadequate will then be patched with an oval or round patch of the same geomembrane extending a minimum of 6 in. (150 mm) beyond the cut in all directions.

- 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 will document that the above seaming procedures are followed, and will inform the Project Manager if they are not.

10.4.4.9 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 approved method. 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, test unit number, name of person conducting the test, and the results of tests; and

- inform the Geosynthetic Installer and Project Manager of required repairs.

The Geosynthetic Installer will complete any required repairs in general accordance with Section 10.4.5.

The CQA Site Manager will:

- observe the repair and re-testing of the repair;
- mark on the geomembrane that the repair has been made; and
- document the results.

The following procedures will apply to locations where seams cannot be non-destructively tested:

All such seams will be cap-stripped with the same geomembrane.

- If the seam is accessible to testing equipment prior to final installation, the seam will be non-destructively tested prior to final installation.

- If the seam cannot be tested prior to final installation, the seaming and cap-stripping operations will be observed by the CQA Site Manager and Geosynthetic Installer for uniformity and completeness.

The seam number, date of observation, name of tester, and outcome of the test or observation will be recorded by the CQA Site Manager.

Vacuum Testing

The equipment will be comprised of the following:

- a vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge;

- a steel vacuum tank and pump assembly equipped with a pressure controller and pipe connections;

- a rubber pressure/vacuum hose with fittings and connections;

- an approved applicator; and

- a soapy solution.

The following procedures will be followed:

- energize the vacuum pump and reduce the tank pressure to approximately 5 psi (35 kPa) (10 in. of Hg.) gauge;

- wet a strip of geomembrane approximately 12 in. by 48 in. (0.3 m by 1.2 m) with the soapy solution;

- place the box over the wetted area;

- close the bleed valve and open the vacuum valve;

- document that a leak tight seal is created;

- for a period of not less than ten seconds, examine the geomembrane through the viewing window for the presence of leaks indicated by soap bubbles;

- if no leaks appear after ten seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3 in. (75 mm) overlap, and repeat the process;

- areas where soap bubbles appear will be marked and repaired in general accordance with Section 10.4.5 and retested using the vacuum testing method.

Air Pressure Testing (For Double-Track Fusion Seam Only)

The following procedures are applicable to those processes that produce a double seam with an enclosed space.

The equipment will be comprised of the following:

- an air pump (manual or motor driven) equipped with pressure gauge capable of generating and sustaining a pressure of 30 psi (200 kPa) and mounted on a cushion to protect the geomembrane;

- a rubber hose with fittings and connections;

- a sharp hollow needle, or other approved pressure feed device.

The following procedures will be followed:

- seal both ends of the seam to be tested;

- insert needle or other approved pressure feed device into the tunnel created by the fusion weld;

- insert a protective cushion between the air pump and the geomembrane;

- energize the air pump to a pressure of 25 to 30 psi (170 to 204 kPa), close valve, and sustain pressure for not less than 5 minutes;

- if loss of pressure exceeds 3 psi (20 kPa) or does not stabilize, locate faulty area and repair in general accordance with Section 10.4.5;

- cut end of tested seam area, opposite the location of the pressure gauge, after completion of the five minute pressure hold period to verify complete testing of the seam. If the pressure gauge does not indicate a release of pressure, locate blockage of the air channel and retest until entire seam is tested; and

- remove needle or other approved pressure feed device and repair any holes in the geomembrane resulting from the air pressure testing procedure in general accordance with Section 10.4.5.

10.4.4.10 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 phase.

- Test locations will be evaluated during seaming at 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 cut by the Geosynthetic Installer 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;
- assign a number to each sample, and mark it accordingly;
- record sample location on 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 10.4.5. The continuity of the new seams in the repaired area will be tested in general accordance with Section 10.4.4.9.

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 Contractor 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. Five 1 in. (25 mm) wide strips will be taken for peel. 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 log 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 Project Manager will document that packaging and shipping conditions are acceptable. The Project Manager will be responsible for storing the archive samples. This procedure will be outlined at the Resolution 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 Project Manager.

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 Project Manager.

Geosynthetic Installer's Laboratory Testing

The Geosynthetic Installer's laboratory test results will be presented to the Project 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. In cases where the failed seam segment exceeds 150 ft (50 m), a destructive sample will be taken from the zone in which the seam has been reconstructed. Repairs will be made in general accordance with Section 10.4.5.

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

10.4.5 Defects and Repairs

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

10.4.5.1 Identification

Seams and non-seam areas of the geomembrane will 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 will be clean at the time of examination.

10.4.5.2 Evaluation

Each suspect location both in seam and non-seam areas will be non-destructively tested using the methods described in Section 10.4.4.9 as appropriate. Each location that fails the nondestructive testing will be marked by the CQA Site Manager and repaired by the Geosynthetic Installer. Work will not proceed with any materials that will cover locations which have been repaired until laboratory test results with passing values are available.

10.4.5.3 Repair Procedures

Portions of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, will be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure will be at the discretion of the CQA Consultant with input from the Project 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 Project Manager 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); and

- the geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

10.4.5.4 Verification of Repairs

Each repair will be numbered and logged. Each repair will be non-destructively tested using the methods described in Section 10.4.4.9 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, at the discretion of the CQA Site Manager. Failed tests indicate that the repair will be redone and re-tested until a passing test results. The CQA Site Manager will observe all non-destructive testing of repairs and will record the number of each repair, date, and test outcome.

10.4.5.5 Large Wrinkles

When seaming of the geomembrane is completed (or when seaming of a large area of the geomembrane liner is completed) and prior to placing overlying materials, the CQA Site Manager will observe the geomembrane wrinkles. The CQA Site Manager will indicate to the Project Manager which wrinkles should be cut and re-seamed by the Geosynthetic Installer. The seam thus produced will be tested like any other seam.

10.4.6 Lining System Acceptance

The Geosynthetic Installer and the Manufacturer(s) will retain all responsibility for the geosynthetic materials in the liner system until acceptance by the Owner.

The geosynthetic liner system will be accepted by the Owner 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
- CQA report, including “as built” drawing(s), sealed by a registered professional engineer has been received by the Project Manager.

The CQA Site Manager will document that installation has proceeded in general accordance with the *Technical Specifications* for the project except as noted to the Project Manager.

11. GEOTEXTILE

11.1 Introduction

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

11.2 Manufacturing

The Manufacturer will provide the Project 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 Project 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 quality control testing.

The Manufacturer will provide, as a minimum, test results for the following:

- mass per unit area (cushion geotextile only);
- grab strength (cushion and filtration geotextiles only);
- tear strength (cushion and filtration geotextiles only);
- burst strength (cushion and filtration geotextiles only);
- puncture strength (cushion and filtration geotextiles only);
- wide width tensile strength (UV protection geotextile only);
- permittivity (filtration geotextile only); and
- apparent opening size (filtration and UV protection geotextiles only).

Quality control tests must be performed, in general accordance with the test methods specified in Table 9, on geotextile produced for the project. The Manufacturer will also provide a written certification that the nonwoven, needle-punched geotextiles are continuously inspected and found to be needle-free.

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 Project Manager.

11.3 Labeling

The Manufacturer will identify all rolls of geotextile with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The CQA Site Manager will examine rolls upon delivery and deviation from the above requirements will be reported to the Project Manager.

11.4 Shipment and Storage

During shipment and storage, the geotextile will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. To that effect, geotextile rolls will be shipped and stored in relatively opaque and watertight wrappings.

Protective wrappings will be removed less than one hour prior to unrolling the geotextile. After the wrapping has been removed, a geotextile will not be exposed to sunlight for more than 15 days, except for UV protection geotextile, unless otherwise specified and guaranteed by the Manufacturer.

The CQA Site Manager will observe rolls upon delivery at the site and deviation from the above requirements will be reported to the Project Manager.

11.5 <u>Conformance Testing</u>

11.5.1 Tests

Upon delivery of the rolls of geotextiles, the CQA Site Manager will document that samples are removed and forwarded 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 9.

These conformance tests will be performed in general accordance with the test methods specified in the *Technical Specifications*.

11.5.2 Sampling Procedures

Samples will be taken across the width of the roll and will not include the first three feet (linear meter). Unless otherwise specified, samples will be 3 ft (1 m) 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 9 for geotextiles.

11.5.3 Test Results

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

11.5.4 Conformance Sample Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of geotextile that is in nonconformance with the *Technical Specifications* with a roll(s) that meets *Technical Specifications*.

- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these samples fail, every roll of geotextile on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*. This additional conformance testing will be at the expense of the Manufacturer.

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

11.6 <u>Handling and Placement</u>

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:

- On slopes, the geotextiles will be securely anchored in the anchor trench and then rolled down the slope in such a manner as to continually keep the geotextile sheet in tension.

- 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 Project Manager.

11.7 Seams and Overlaps

All geotextiles will be continuously sewn in accordance with *Technical Specifications*. Geotextiles will be overlapped 6 in. (0.15 m) prior to seaming. No horizontal seams will be allowed on side slopes (i.e. seams will be along, not across, the slope), except as part of a patch.

Sewing will be done using polymeric thread with chemical and ultraviolet resistance properties equal to or exceeding those of the geotextile.

11.8 Repair

Holes or tears in the geotextile will be repaired as follows:

- On slopes: A patch made from the same geotextile will be double seamed into place. Should a tear exceed 10 percent of the width of the roll, that roll will be removed from the slope and replaced.

- Non-slopes: A patch made from the same geotextile will be spot-seamed in place with a minimum of 6 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 Project Manager.

11.9 Placement of Soil or Aggregate Materials

The Contractor will place all soil or aggregate materials located on top of a geotextile, in such a manner as to document:

- no damage of the geotextile;
- minimal slippage of the geotextile on underlying layers; and
- no excess tensile stresses in the geotextile.

Unless otherwise specified by the Engineer, all lifts of soil material will be in conformance with the following guidelines:

Equipment Ground Pressure		Minimum Loose Lift Thickness	
Psi	**kPa**	**in.**	**m**
<10	< 68	12	0.30
<20	< 138	24	0.60
>20	> 138	36	0.90

If portions of the geotextile are exposed, the CQA Site Manager will periodically place two (or more, at his discretion) marks on the geotextile 10 ft (3 m) apart along the slope and measure the elongation of the geotextile during the placement of soil. This elongation will be related, by the Engineer, to the tensile stress in the geotextile.

Non-compliance will be noted by the CQA Site Manager and reported to the Project Manager.

12. GEOSYNTHETIC CLAY LINER (GCL)

12.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geosynthetic clay liner (GCL) installation. The CQA Consultant will review the *Drawings*, and the *Technical Specifications*, and approved addenda or changes.

12.2 Manufacturing

The Manufacturer will provide the Project 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 Project 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 10, 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 Project Manager.

12.3 Labeling

The Manufacturer will identify all rolls of GCL with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The CQA Site Manager will examine rolls upon delivery and deviation from the above requirements will be reported to the Project Manager.

12.4 Shipment and Storage

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

12.5 Conformance Testing

12.5.1 Tests

CQA personnel will sample the GCL either during production at the manufacturing facility or after delivery to the construction site. The samples will be forwarded to the Geosynthetics CQA Laboratory for testing to assess conformance with the *Technical Specifications*. The test methods and minimum testing frequencies are indicated in Table 10.

Samples will be taken across the width of the roll and will not include the first 3 ft (0.9 m) if the sample is cut on site. Unless otherwise specified, samples will be 3 ft (0.9 m) 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 Project Manager.

12.5.2 Conformance Sample Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of GCL that is in nonconformance with the *Technical Specifications* with a roll(s) that meets *Technical Specifications*.

- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these samples fail, every roll of GCL on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*. This additional conformance testing will be at the expense of the Manufacturer.

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

12.6 <u>GCL Delivery and Storage</u>

Upon delivery to the site, the CQA Consultant will check the GCL rolls for defects (e.g., tears, holes) and for damage. The CQA Consultant will report to the Project Manager and the Geosynthetics Installer:

- any rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and

- any rolls which include minor repairable flaws.

The GCL rolls delivered to the site will be checked by the CQA Consultant to document that the roll numbers correspond to those on the approved Manufacturer's quality control certificate of compliance.

12.7 GCL Installation

The CQA Consultant will monitor and document that the GCL is installed in 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 Project Manager.

13. GEOCOMPOSITE

13.1 <u>Introduction</u>

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

13.2 <u>Manufacturing</u>

The Manufacturer will provide the CQA Consultant with a list of certified "minimum average roll value" properties for the type of geocomposite 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 geocomposite actually delivered have "minimum average roll values" properties which meet or exceed all certified property values for that type of geocomposite.

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 geocomposite (geotextile and geonet). Deviations will be reported to the Project Manager.

13.3 <u>Labeling</u>

The Manufacturer will identify all rolls of geocomposite 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 Project Manager.

13.4 <u>Shipment and Storage</u>

During shipment and storage, the geocomposite will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. Therefore, geocomposite rolls will be shipped and stored in relatively opaque and watertight wrappings. The CQA Site Manager will observe rolls upon delivery to the site and deviation from the above requirements will be reported to the Project Manager. Damaged rolls will be rejected and replaced.

Wrapping protecting geocomposite rolls will be removed less than one hour prior to unrolling geocomposite before placement. After the wrapping has been removed, geocomposite should not be exposed to sunlight for more than 15 days, unless otherwise approved by the Manufacturer. Approval by the Manufacturer will be a guarantee that the properties of the exposed geotextile will not degrade upon prolonged exposure to such values that would cause the material to not meet the *Technical Specifications*. Any material that is exposed for more than 15 days, which has been approved for prolonged exposure by the Manufacturer, will be tested by the CQA Laboratory to document that the material properties are still in conformance with the *Technical Specifications*. Any material that fails to meet the *Technical Specifications* will be replaced by the Manufacturer.

The CQA Site Manager will observe that geocomposite is free of dirt and dust just before installation. The CQA Site Manager will report the outcome of this observation to the Project Manager, and if the geocomposite is judged dirty or dusty, they will be cleaned by the Geosynthetic Installer prior to installation.

13.5 <u>Conformance Testing</u>

13.5.1 Tests

The geocomposite material will be tested for transmissivity (ASTM D 4716) and for peel strength (ASTM D 413) at the frequencies presented in Table 11.

13.5.2 Sampling Procedures

Upon delivery of the geocomposite rolls, the CQA Site Manager will document that samples are obtained from individual rolls at the frequency specified in this CQA Plan. The geocomposite 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 (1 linear m). Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The CQA Consultant will mark the machine direction on the samples with an arrow.

13.5.3 Test Results

The CQA Site Manager will examine results from laboratory conformance testing and compare results to the *Technical Specifications*. The criteria used to evaluate acceptability are presented in the *Technical Specifications*. The CQA Site Manager will report any nonconformance to the Project Manager.

13.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of geocomposite that is in nonconformance with the *Technical Specifications* with a roll that meets specifications.

- 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 geocomposite on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*.

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

13.6 Handling and Placement

The Geosynthetic Installer will handle all geocomposite in such a manner as to document they are not damaged in any way. The Geosynthetic Installer will comply with the following:

- In the presence of wind, the geocomposite will be weighted with sandbags or the equivalent. Sandbags will be used during installation only and will remain until replaced with the appropriate cover material.

- If in place, special care must be taken to protect other materials from damage, which could be caused by the cutting of the geocomposite.

- The Geosynthetic Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geocomposite.

- During placement of geocomposite, 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 geocomposite, 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 geocomposite 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 Project Manager.

13.7 Drainage composite Seams and Overlaps

Adjacent geocomposite panels will be joined in general accordance with *Construction Drawings* and *Technical Specifications*. As a minimum, the following requirements will be met:

- Adjacent rolls will be overlapped by at least 4 in. (100 mm).
- Each component of the geocomposite will be secured or seamed to the like component at overlaps.
- The geocomposite overlaps will be secured by tying, in general accordance with the *Technical Specifications*.
- The bottom layers of geotextile will be overlapped.
- The top layers of geotextile will be continuously sewn.

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

13.8 Repair

Holes or tears in the geocomposite will be repaired by placing a patch extending 2 ft (0.6 m) beyond edges of the hole or tear. The patch will be secured by tying with approved tying devices every 6 in. (150 mm) through the bottom geotextile and the geonet of the patch, and through the top geotextile and geonet components of the geocomposite needing repair. The top geotextile component of the patch will be heat sealed to the top geotextile of the geocomposite needing repair. 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 geocomposite will be joined in general accordance with Section 13.7.

The CQA Site Manager will observe repairs, note noncompliances with the above requirements and report them to the Project Manager.

13.9 Placement of Soil Materials

The Contractor will place all soil materials located on top of a geocomposite in such a manner as to document:

- the geocomposite and underlying liner materials are not damaged;
- minimal slippage of the geocomposite on underlying layers occurs; and
- no excess tensile stresses occur in the geocomposite.

Unless otherwise specified by the CQA Consultant, lifts of soil material will be in conformance with the *Technical Specifications*. If portions of the geocomposite are exposed, the CQA Consultant will periodically place marks on the geocomposite and the underlying geomembrane and measure the elongation of the geonet during the placement of soil.

Noncompliance will be noted by the CQA Consultant and reported to the Project Manager.

14. SURVEYING

14.1 Survey Control

Survey control will be performed by the Owner 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.

14.2 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. Surveys shall be performed at 2nd order accuracy.

14.3 Lines and Grades

The following surfaces will be surveyed to verify the lines and grades achieved during soil placement and compaction.

- Excavation:
  - original grade surface;
  - completed excavation surface prior to fill placement.
- Engineered Fill:
  - subgrade surface; and
  - finished compacted engineered fill surface.
- Prepared Subgrade:
  - prepared subgrade surface.

The following structures will be surveyed to verify and document the lines and grades achieved during construction of the Project:

- all culverts, inlet, and drop structures;
- ditch bottoms and sideslopes;
- permanent erosion control features;
- geomembrane terminations and selected geomembrane seams, as indicated by the CQA Manager; and
- centerlines of pipes.

14.4 Frequency and Spacing

Surveying should be carried out immediately upon completion of a given installation to facilitate progress and avoid delaying commencement of the next installation. In addition, spot checks during placement and compaction will be necessary to assist the Contractor in compliance with required grades.

At the least the following minimum spacings and locations should be provided for survey points:

- all "flat" surfaces, such as the base of the landfill, with gradients less than 10 percent, should be surveyed on a square grid not wider spaced than 100 ft (30 m);
- on all slopes greater than 10 percent, a square grid not wider than 100 ft (30 m) should be used, but in any case, a line at the crest, midpoint, and toe of the slope should be taken;
- a line of survey points no further than 100 ft (30 m) apart must be taken along any slope break (this will include the inside edge and outside edge of any bench on a slope); and
- a line of survey points no further than 50 ft (15 m) apart must be taken at the invert of pipes or other appurtenances to the liner.

14.5 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 Engineer in AutoCADD 2000 format or other suitable format as directed by the Owner.

TABLE 1
TEST PROCEDURES FOR THE EVALUATION OF SOILS

TEST METHOD	DESCRIPTION	TEST STANDARD
Laboratory Test Procedures:		
Classification	Classification of Soils	ASTM D 2487
Modified Proctor	Moisture/Density Relationship of Soil (10 lb (4.54 kg) rammer and 18 in. (457 mm) drop)	ASTM D 1557
Hydrometer Analysis	Particle Size Distribution of Fine Fraction of Soils	ASTM D 422
Sieve Analysis	Particle Size Distribution of Coarse Fraction of Soils	ASTM D 422
Field Test Procedures:		
Nuclear Densometer	In Situ Soil Unit Weight In Situ Moisture Content	ASTM D 2922 ASTM D 3017
Sand Cone	In Situ Soil Unit Weight Moisture Content	ASTM D 1556 ASTM D 2216
Drive Cylinder	In Situ Soil Unit Weight Moisture Content	ASTM D 2937 ASTM D 2216

TABLE 2

MINIMUM SOILS TESTING FREQUENCIES FOR MATERIAL QUALIFICATION TESTING

TEST	ENGINEERED FILL
Sieve Analysis	1 per source
Hydrometer Analysis	1 per source
Soil Classification	1 per source
Modified Proctor	1 per source

TABLE 3

MINIMUM SOILS TESTING FREQUENCIES FOR CONFORMANCE TESTING

TEST	ENGINEERED FILL
Sieve Analysis	1 per 10,000 yd^3 (7,646 m^3)
Hydrometer Analysis	1 per 10,000 yd^3 (7,646 m^3)
Soil Classification	1 per 10,000 yd^3 (7,646 m^3)
Modified Proctor	1 per 10,000 yd^3 (7,646 m^3)

TABLE 4

MINIMUM SOIL TESTING FREQUENCIES FOR CONSTRUCTION QUALITY CONTROL

TEST	ENGINEERED FILL
Nuclear densometer	1 per 500 yd^3 (76 m^3)
Sand cone or drive cylinder	1 per 20 nuclear densometer tests

TABLE 5

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 6

MINIMUM AGGREGATE TESTING FREQUENCIES FOR MATERIAL QUALIFICATION TESTING

TEST	DRAINAGE AGGREGATE
Sieve Analysis	1 per source
Hydraulic Conductivity	1 per source

TABLE 7

MINIMUM AGGREGATE TESTING FREQUENCIES FOR CONFORMANCE TESTING

TEST	TEST METHOD	DRAINAGE AGGREGATE
Sieve Analysis	ASTM C 136	1 per 5,000 yd^3 (3,823 m^3)
Hydraulic Conductivity	ASTM D 2434	1 per 10,000 yd^3 (7,646 m^3)

TABLE 8

GEOMEMBRANE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	FREQUENCY
Specific Gravity	ASTM D 792 Method A or ASTM D 1505	100,000 ft^2 (9,290 m^2)
Thickness	ASTM D 5994	100,000 ft^2 (9,290 m^2)
Tensile Strength at Yield	ASTM D 638	100,000 ft^2 (9,290 m^2)
Tensile Strength at Break	ASTM D 638	100,000 ft^2 (9,290 m^2)
Elongation at Yield	ASTM D 638	100,000 ft^2 (9,290 m^2)
Elongation at Break	ASTM D 638	100,000 ft^2 (9,290 m^2)
Carbon Black Content	ASTM D 1603	100,000 ft^2 (9,290 m^2)
Carbon Black Dispersion	ASTM D 5596	100,000 ft^2 (9,290 m^2)

TABLE 9

GEOTEXTILE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY CUSHION	MINIMUM FREQUENCY FILTRATION	MINIMUM FREQUENCY UV PROTECTION
Mass per Unit Area	ASTM D 5261	1 test per 200,000 ft^2 (18,580 m^2)	---	---
Grab Strength	ASTM D 4632	1 test per 200,000 ft^2 (18,580 m^2)	1 test per 200,000 ft^2 (18,580 m^2)	---
Puncture Resistance	ASTM D 4833	1 test per 200,000 ft^2 (18,580 m^2)	1 test per 200,000 ft^2 (18,580 m^2)	---
Permittivity	ASTM D 4491	---	1 test per 200,000 ft^2 (18,580 m^2)	---
Apparent Opening Size	ASTM D 4751	---	1 test per 200,000 ft^2 (18,580 m^2)	1 test per 200,000 ft^2 (18,580 m^2)
Wide Width Tensile	ASTM D 4595	---	---	1 test per 200,000 ft^2 (18,580 m^2)

TABLE 10

GCL CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Mass per Unit Area	ASTM D 3776	100,000 ft^2 (9,290 m2)
Index Flux	ASTM D 5887	400,000 ft^2 (37,160 m^2)
Residual Shear Strength	ASTM D 5321	See Technical Specifications

TABLE 11

GEOCOMPOSITE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Peel Strength	ASTM D 413	1 test per 200,000 ft^2 (18,580 m^2)
Hydraulic Transmissivity	ASTM D 4716	1 test per 200,000 ft^2 (18,580 m^2)

Note: Testing will be carried out at a frequency of one per lot or at listed frequency, whichever yields the greater number of samples.

Section 4
Base Liner Technical Specifications

TECHNICAL SPECIFICATIONS
FOR THE
CONSTRUCTION OF

BASE LINER SYSTEM
AT
CORRECTIVE ACTION MANAGEMENT UNIT
BASIC REMEDIATION COMPANY
HENDERSON, NEVADA

Prepared for:

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3 November 2006

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SECTION 02110
SITE CLEARING

PART 1 – GENERAL

1.01 Description of Work

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, and equipment necessary to perform all work specified herein and as shown on the Drawings.

B. The Contractor shall remove and dispose of all debris, vegetation, other organic and deleterious material, and other materials not suitable for Engineered fill materials that exist within the designated construction limits.

1.02 Related Sections

Section 02200 — Earthwork

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

PART 2 – PRODUCTS

A. Not Used.

PART 3 – EXECUTION

3.01 General

A. The Contractor shall be responsible for all clearing and grubbing operations within the limits of work.

B. All vegetation, debris, deleterious and other organic material not suitable for Engineered fill materials shall be removed completely from within the construction limits.

C. No open burning of combustible materials will be allowed.

D. All materials removed during the clearing and grubbing operations shall be disposed of properly.

E. Prior to site clearing, Contractor shall have implemented erosion control plan.

PART 4 – MEASUREMENT AND PAYMENT

4.01 General

A. Providing for and complying with the requirements set forth in this Section for Clearing and Grubbing will be incidental to earthworks (Section 02200).

B. The following are considered incidental to the Work:

- Mobilization
- Layout survey

END OF SECTION

SECTION 02200
EARTHWORK

PART 1 – GENERAL

1.01 Description of Work

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary to perform all Earthwork. The work shall be carried out as specified herein and in accordance with the Drawings.

B. The Work shall include, but not be limited to excavating, hauling, placing, moisture conditioning, backfilling, compacting, grading, stockpiling, surcharging, and subgrade preparation. 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 02110 — Site Clearing

Section 02771 — Geotextile

Section 02772 — Geosynthetic Clay Liner

Section 02773 — Geocomposite

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 422	Standard Method for Particle-Size Analysis of Soils
ASTM D 1557	Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))
ASTM D 2216	Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
ASTM D 2487	Standard Test Method for Classification of Soils for Engineering Purposes
ASTM D 2922	Standard Test Methods for Density of Soil and Soil-Aggregate In-Place by Nuclear Density Methods (Shallow Depth)
ASTM D 3017	Standard Test Method for Water Content of Soil and Rock In-Place by Nuclear Methods (Shallow Depth)

1.04 Submittals

A. The Contractor shall submit to the Engineer a description of equipment and methods proposed for Engineered Fill, Operations Layer, Anchor Trench Backfill, and Prepared Subgrade placement and compaction at least 7 days prior to the start of activities covered by this Section.

B. If the work of this Section is interrupted for reasons other than inclement weather, the Contractor shall notify the Engineer a minimum of 24 hours prior to the resumption of work.

C. The Contractor shall provide the Engineer with sufficient time to perform as-built surveys of the completed excavation, engineered fill, prepared subgrade, and operations layer.

D. If foreign borrow materials are proposed for any earthwork material on this project, the Contractor shall provide the Engineer information regarding the source of the material. In addition, the Contractor shall provide the Engineer an opportunity to obtain the necessary samples for conformance testing, prior to delivery of foreign borrow materials to the site.

1.05 Quality Assurance

A. The Contractor shall ensure that the materials and methods used for Earthwork meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Engineer will be rejected and shall be repaired or replaced by the Contractor.

B. 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 Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Contractor will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 Materials

A. Engineered fill shall consist of relatively homogeneous, natural soils that are free of debris, foreign objects, large rock fragments (greater than 6 inches in maximum dimension), roots, and organics. No materials larger than 6 inches shall be allowed within the Engineered fill. The Engineered fill shall be classified according to the Unified Soil Classification System (per ASTM D 2487) as SC, ML, CL, SM, SW, SP, GW, GP, GM, GC, or combinations of these materials. The Contractor may propose the use of other soil types as Engineered fill, but then such use shall be at the sole discretion of the Engineer.

B. Prepared subgrade is defined as the material directly underlying the geosynthetic liner system which shall meet the requirements listed above for Engineered fill. No materials larger than 3/4 inch shall project or protrude from the surface of the prepared subgrade.

C. Anchor Trench Backfill materials shall meet the requirements listed above for the Engineered fill.

D. Operations Layer materials shall meet the requirements listed above for the Engineered fill, except that the 12 inches of layer material to be placed overlying the geocomposite shall have a maximum particle size of 1 inch.

E. Surcharge materials shall have a minimum wet density, as compacted in place of 135 pounds per cubic foot.

2.02 Equipment

A. The Contractor shall furnish, operate, and maintain compaction equipment as is necessary to produce the required in-place soil density and moisture content.

B. The Contractor shall furnish, operate and maintain tank trucks, pressure distributors, or other equipment designed to apply water uniformly and in controlled quantities to variable surface widths.

C. The Contractor shall furnish, operate, and maintain miscellaneous equipment such as scarifiers or disks, earth excavating equipment, earth hauling equipment, and other equipment, as necessary for Earthwork construction.

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:

- 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.
- 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 continuance of the work of this Section will be construed as Contractor's acceptance of the related work of all other Sections.

3.02 Site Preparation

A. The Contractor shall perform clearing and grubbing in accordance with the Drawings and Section 02110 of these Specifications prior to any Earthwork activity.

B. Prior to performing any earthworks on the site, the Contractor shall perform a baseline topographic survey. This survey shall be conducted by a Professional Land Surveyor licensed in the state of Nevada. This survey will serve as the starting point for earthwork quantities, both excavation and fill placement.

3.03 General Excavation

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

B. All excavated materials not used for Engineered fill or operations layer shall be stockpiled in an area designated by the Owner in accordance with Part 3.06 of this Section.

C. Excavations in native soil shall not have slopes steeper than 2.1H:1V, unless otherwise approved by the Engineer.

3.04 Anchor 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 or Engineered fill shall be stockpiled in an area designated by the Owner in accordance with Part 3.06 of this Section.

3.05 Surcharge

A. The surcharge soil shall be placed and compacted as shown on the drawings.

B. Surcharge shall have slopes no steeper than 1H:1V.

C. Settlement plates shall be installed as shown on the drawings and shall be surveyed on a weekly basis, at a minimum.

D. Surcharge shall be removed once settlement has reached levels differing no more than 5% from previous readings or no more than 0.05 feet.

3.06 Subgrade Surface Preparation

A. The subgrade shall be prepared and made suitable as a foundation for placement and compaction of soil material, where applicable. The subgrade shall be firm and able to support the Contractor's construction equipment without the development of depressions or ruts. In addition, the subgrade shall provide adequate support such that the overlying fill material may be placed and compacted to the specified density.

3.07 Prepared Subgrade

A. The prepared subgrade shall be made suitable as a foundation for placement of the geosynthetic components of the liner system (prepared subgrade). The prepared subgrade shall be firm, meet the requirements outlined in Part 2.01, and be able to support the geosynthetic components of the liner system.

3.08 Stockpiling

A. Soil shall be stockpiled in areas designated by the Owner and shall be free of incompatible soil, clearing, clearing debris, or other objectionable materials.

B. Stockpiles shall be no steeper than 2.1H:1V (Horizontal:Vertical) or other slope approved by the Engineer, graded to drain, sealed by tracking parallel to the slope with a dozer or other means approved by the 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 stockpile areas.

3.09 Engineered Fill And Anchor Trench Backfill

A. The Engineered fill and Anchor Trench Backfill shall be placed to the lines and grades shown on the Drawings.

B. Soil used for the Engineered fill and Anchor Trench Backfill shall meet the requirements of Part 2.01 of this Section.

C. Soil used for the Engineered fill and Anchor Trench Backfill shall be placed in a loose lift that results in a compacted lift thickness of no greater than 12 inches. The maximum permissible pre-compaction soil clod size is 6 inches.

D. Each 12-inch horizontal lift of Engineered fill placed against a slope shall be keyed into the slope a minimum of 3 feet, as measured horizontally from the top of the 12-inch lift.

E. The Contractor shall compact each lift to at least 90 percent of its modified Proctor maximum dry density (ASTM D 1557). The Contractor shall utilize compaction equipment suitable for achieving the soil compaction requirements.

F. During wetting or drying, the material shall be regularly disced or otherwise mixed so that uniform moisture conditions in the appropriate range are obtained.

3.10 Operations Layer

A. Place only when underlying drainage aggregate and filter geotextile or geocomposite installation is complete including all Construction Quality Control (CQC) and CQA work.

B. The subgrade to the operations layer consists of a geotextile or geocomposite. Therefore, the Contractor shall avoid tearing, puncturing, folding, or damaging in any way the filter geotextile or geocomposite geotextile during placement of the operations layer material.

C. Any damage to the geosynthetic liner system which is caused by the Contractor or representatives of the Contractor shall be repaired by the Geosynthetics Installer at the expense of the Contractor.

D. No density requirements are specified for placement of the operations layer material. Operations layer material shall be placed at a moisture content less than the optimum moisture content for the soil.

E. The operations layer material shall be placed out in front of the equipment used to place the operations layer such that the minimum thickness requirements are maintained at all times between the geosynthetic materials and the wheels or tracks of the equipment used to place the operations layer material.

F. Care must be exercised by the operators of tracked equipment to avoid sharp pivoting turns that could displace the operations layer material and result in damage to the liner system.

G. The Contractor shall not push operations layer material down the side slope. All soil materials shall be placed from the toe of slope upward.

H. Equipment used in spreading the operations layer material on top of the geosynthetic liner system shall be restricted to the following maximum allowable equipment ground pressures:

MAXIMUM ALLOWABLE EQUIPMENT GROUND PRESSURE (psi)	INITIAL LIFT THICKNESS OF OVERLYING AGGREGATE (ft)
<20	2.0
>20	3.0

I. The operations layer shall be placed to a maximum vertical height of 10 ft at a slope inclination no steeper than 2.5H:1V as shown on the Drawings.

3.11 Field Testing

A. The minimum frequency and details of quality control testing are provided below. This testing will be performed by the Engineer. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. Engineered fill material quality control testing:

a. particle-size analyses conducted in accordance with ASTM D 422 at a frequency of one test per 10,000 yd^3;

b. soil classification tests conducted in accordance with ASTM D 2487 at a frequency of one test per 10,000 yd^3; and

c. modified Proctor compaction tests conducted in accordance with ASTM D 1557 at a frequency of one test per 10,000 yd^3.

2. The Engineer will perform conformance tests on placed and compacted Engineered fill to evaluate compliance with these Specifications. These tests will include in-situ moisture content and dry density. The frequency and procedures for moisture-density testing are given in the CQA Plan. At a minimum, the dry density and moisture content of the soil will be measured in-situ in accordance with ASTM D 2922 and ASTM D 3017, respectively.

3. A special testing frequency will be used by the Engineer when visual observations of construction performance indicate a potential problem. Additional testing will be considered when:

a. the rollers slip during rolling operation;
b. the lift thickness is greater than specified;
c. the fill is at improper and/or variable moisture content;
d. fewer than the specified number of roller passes are made;
e. dirt-clogged rollers are used to compact the material;
f. the rollers do not have optimum ballast; or
g. the degree of compaction is doubtful.

4. During construction, the frequency of testing will be increased by the Engineer in the following situations:

a. adverse weather conditions;
b. breakdown of equipment;
c. at the start and finish of grading;
d. if the material fails to meet specifications; or
e. the work area is reduced.

B. Defective Areas:

1. If a defective area is discovered in the Earthwork, the Engineer will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the Engineer will determine the extent of the defective area by additional tests, observations, a review of records, or other means that the Engineer deems appropriate. If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the Engineer shall define the limits and nature of the defect.

2. Once the extent and nature of a defect is determined, the Contractor shall correct the deficiency to the satisfaction of the Engineer. The Contractor shall not perform additional work in the area until the Engineer approves the correction of the defect.

3. Additional testing may be performed by the Engineer to verify that the defect has been corrected. This additional testing will be performed before any additional work is allowed in the area of deficiency. The cost of the additional testing shall be borne by the Contractor.

3.12 Survey Control

A. The Contractor shall perform all surveys necessary for construction layout and control.

3.13 Construction Tolerance

A. The Contractor shall perform the Earthwork construction to within ±0.1 ft on areas with a slope less than 10 percent and ±0.2 ft on areas with a slope greater than 10 percent of the grades indicated on the Drawings.

3.14 Protection of Work

A. The Contractor shall use all means necessary to protect completed work of this Section.

B. At the end of each day, the Contractor shall verify that the entire work area is left in a state that promotes drainage of surface water away from the area and from finished work. If threatening weather conditions are forecast, at a minimum, compacted surfaces shall be seal-rolled to protect finished work.

C. In the event of damage to prior work, the Contractor shall make repairs and replacements to the satisfaction of the Engineer.

PART 4 – MEASUREMENT AND PAYMENT

4.01 General

A. Providing for and complying with the requirements set forth in this Section for Excavation will be measured as in-place cubic yards (CY), prior to excavation, and payment will be based on the unit price provided on the Bid Schedule.

B. Providing for and complying with the requirements set forth in this Section for Engineered Fill will be measured as compacted and moisture conditioned in-place cubic yards (CY), and payment will be based on the unit price provided on the Bid Schedule.

C. Providing for and complying with the requirements set forth in this Section for Prepared Subgrade will be measured as square feet (SF), and payment will be based on the unit price provided on the Bid Schedule.

D. Providing for and complying with the requirements set forth in this Section for Anchor Trench Excavation and Backfill will be measured as linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

E. Providing for and complying with the requirements set forth in this Section for Operations Layer will be measured as in-place cubic yards (CY), and payment will be based on the unit price provided on the Bid Schedule.

F. Providing for and complying with the requirements set forth in this section for Surcharge will be measured as in-place cubic yards (CY), and payment will be based on the unit price provided on the Bid Schedule.

G. The following are considered incidental to the Work:

- material samples, sampling, and testing.
- layout survey.
- rejected material removal, re-testing, handling, and repair.

- rejected material.
- mobilization.
- Stockpiling.
- settlement plates.
- surcharge removal.

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.

B. The work shall include, but not be limited to, delivery, storage, and placement of drainage aggregate (aggregate).

1.02 Related Sections

Section 02200 — Earthwork

Section 02771 — Geotextile

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

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

ASTM C 33	Standard Specification for Concrete Aggregates
ASTM C 136	Test Method for Sieve Analysis of Fine and Coarse Aggregates
ASTM D 2434	Standard Test Method for Permeability of Granular Soils (Constant Head)

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 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 Quality Assurance

A. The Contractor shall ensure that the materials and methods used for Earthwork meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Engineer will be rejected and shall be repaired or replaced by the Contractor.

B. The Contractor shall be aware of 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 Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Contractor will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 Materials

A. Aggregate shall meet the requirements specified in ASTM C-33 and shall have a maximum particle size of 1-inch. Aggregate shall have a minimum permeability of 1 x 10^{-2} cm/sec when tested in accordance with ASTM D 2434.

2.02 Equipment

A. The Contractor shall furnish, operate, and maintain hauling, placing, and grading equipment as necessary for aggregate placement.

PART 3 – EXECUTION

3.01 Familiarization

A. Prior to implementing any of the work in this Section, the Contractor shall become thoroughly familiar with the site, the site conditions, and all portions of the work falling within this and other related Sections.

B. Inspection:

1. The Contractor shall carefully inspect the installed work of all other Sections and verify that all work is complete to the point where the installation of the work specified in this Section may properly commence without adverse impact.

2. If the Contractor has any concerns regarding the installed work of other Sections, the Engineer shall be notified in writing prior to commencing work. Failure to notify the Engineer or continuance 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 only when underlying geosynthetic installation is complete including all CQC and CQA work.

B. Place to the lines, grades, and dimensions shown on the Drawings.

C. The subgrade to 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. Any damage to the geosynthetic liner system which is caused by the Contractor or his representatives shall be repaired by the Geosynthetic Installer.

E. No density or moisture requirements are specified for placement of the aggregate material.

F. Equipment used in spreading the aggregate material on top of the geosynthetic liner system shall be restricted to the following maximum allowable equipment ground pressures:

MAXIMUM ALLOWABLE EQUIPMENT GROUND PRESSURE (psi)	**INITIAL LIFT THICKNESS OF OVERLYING AGGREGATE (ft)**
<10	1.0
<20	2.0
>20	3.0

G. The aggregate material shall be placed out in front of the equipment used to place the aggregate such that the minimum thickness requirements are maintained at all times between the geosynthetic materials and the wheels or tracks of the equipment used to place the aggregate material.

H. All equipment to be used in placing the aggregate material must be approved in writing by the Engineer prior to use. The Contractor shall provide a list of the equipment to be used for placing the aggregate material and the necessary technical information (equipment specifications) on each piece of equipment to be approved at least two working days prior to use.

I. Care must be exercised by the operators of tracked equipment to avoid sharp pivoting turns that could displace the aggregate material and result in damage to the geosynthetic liner system. Care must also be exercised by the operators to avoid the formation of waves greater than 6-inches high in the underlying geosynthetic materials which, when formed and pushed out in front of the leading face of the aggregate material, may grow to such magnitude as to result in a fold in the underlying geosynthetic materials. The Contractor shall place, by backhoe or some other acceptable method, aggregate material on the geosynthetic liner system out in front of the leading face of the aggregate material to trap small waves in the underlying geosynthetic materials and prevent the small waves from combining and growing as the aggregate material is spread. Folds in the underlying geosynthetic materials shall be considered as damage to the liner and must be repaired by the Geosynthetic Installer at the expense of the Contractor.

J. Place filter geotextile overlying aggregate as shown on the Drawings and as specified in Section 02771.

3.03 Field Testing

A. The minimum frequency and details of quality control testing are provided below. This testing will be performed by the Engineer. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. Aggregates quality control testing:

   a. particle-size analyses conducted in accordance with ASTM C-136 at a frequency of one test per 5,000 yd^3;

   b. permeability tests conducted in accordance with ASTM D 2434 at a frequency of one test per 10,000 yd^3.

3.04 Survey Control

A. The Contractor shall perform all surveys necessary for construction layout and control.

3.05 Construction Tolerance

A. The Contractor shall perform the aggregate construction to within +0.1 ft of the thickness indicated on the Drawings.

3.06 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.

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 measured as in-place cubic yards (CY) and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- material samples, sampling, and testing.
- layout survey.
- rejected material.
- rejected material removal, re-testing, handling, and repair.
- mobilization.

END OF SECTION

SECTION 02711
POLYETHYLENE 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 high density polyethylene (HDPE) pipe and fittings as shown on the Drawings and specified herein.

1.02 Related Sections

Section 02225 — Drainage Aggregate

Section 02774 — Geotextile

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

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

ASTM F 714	Specification for Polyethylene Plastic Pipe (SDR-PR) Based on Outside Diameter
ASTM D 1248	Specification for Polyethylene Plastics Molding and Extrusion
ASTM D 2657	Standard Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings
ASTM D 3035	Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Controlled Outside Diameter
ASTM D 3350	Specification for Polyethylene Plastic Pipe and Fitting Materials

1.04 Definitions

A. Standard Dimensional Ratio (SDR) is defined as the actual outside pipe diameter divided by the wall thickness.

1.05 Submittals

A. The Contractor shall submit, at least 7 days prior to installation of this material, to the Engineer, certificates of compliance for the pipe materials and fittings to be furnished.

B. The Contractor shall submit, at least 7 days prior to installation of this material, to the Engineer, copies of certifications for each operator responsible for welding pipe.

C. The Engineer will supply a surveyor to document the as-built conditions of the piping. The Contractor shall notify and allow the Engineer sufficient time to survey piping prior to backfilling the pipe.

1.06 Quality Assurance

A. The Contractor shall ensure that the materials and methods used for polyethylene pipe 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.

PART 2 – PRODUCTS

2.01 Pipe

A. HDPE pipe sizes shown on the Drawings and specified in this Section reference nominal inside diameter. Pipe size shall be in accordance with ASTM F 714 and ASTM D 3035.

B. Pipe shall be, 4-inch, 6-inch, and 18-inch diameter, and shall be HDPE with a minimum standard dimension ratio (SDR) of 13.5, and have a cell classification of 345434C in accordance with ASTM D 3350.

C. Pipe shall conform to the following requirements:

1. Pipe and fittings shall contain no recycled compound except that generated in the Manufacturer's own plant and from resin of the same specification as the raw material supplier.

2. Pipe and fittings shall be homogeneous throughout and free of visible cracks, holes, foreign inclusions, or other deleterious defects, being uniform in color, capacity, density, and other physical properties.

D. The following information shall be continuously marked on the pipe or spaced at intervals not exceeding 5 feet.

1. Name and/or trademark of the pipe Manufacturer.

2. Nominal pipe size.

3. Standard Dimensional Ratio (SDR).

4. PE 3408.

5. A production code from which the date and place of manufacture can be determined.

PART 3 – EXECUTION

3.01 General

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 job site. Handle and store pipe and fittings in accordance with the Manufacturer's recommendation.

3.02 Placing and Laying Pipe

A. Follow the Manufacturer's recommendations when hauling, unloading, and stringing the pipe.

B. HDPE solid and perforated pipe shall be installed as shown on the Drawings.

C. HDPE pipe shall be inspected for cuts, scratches, or other damages prior to installation. Any pipe showing damage, which in the opinion of the CQA Engineer will affect performance of the pipe, must be removed from the Site. The Contractor shall replace any material found to be defective at no additional cost to the Owner.

D. The Contractor shall place the Trench Backfill material around the polyethylene pipe so as to not deform or otherwise damage the pipe and fittings. Special care shall be taken when placing pipe bedding material beneath the spring-line of the pipe and fittings.

E. The Contractor shall clean out pipe interior, as necessary, to remove debris that may affect performance of pipe.

3.03 Fusion Welding Pipe

A. All pipe fusion shall be performed by the Supplier, or a by fusion operator certified by the Manufacturer.

B. Join the polyethylene pipe by the method of thermal butt fusion, as outlined in ASTM D 2657. Electro-fusion couplings shall not be used. Perform butt-fusion joining of pipe and fittings in accordance with the procedures established by the pipe Manufacturer. Of particular importance is the use of proper interface pressures and heater plate temperatures.

C. Do not perform pipe fusion on wet or excessively dirty pipe or when conditions are unsuitable for the work. Secure open ends of pipe when work is not in progress, so that no water, earth, or other substance will enter the pipe or fittings. Plug, cap, or valve off ends of pipe left for future connections, if any.

D. In order to allow the joining operation to continue in adverse weather conditions, a shelter may be required for the joining machine. Particular caution should be exercised to prevent water from entering the pipe and from coming in contact with the heater plate.

E. Only fully trained personnel will be allowed to perform the fusion, installation, supervision, or inspection of polyethylene fusion joints.

3.04 Construction Tolerance

A. The Contractor shall perform the work to within ±0.1 ft of the grades indicated on the Drawings.

PART 4 – MEASUREMENT AND PAYMENT

4.01 General

A. Providing for and complying with the requirements set forth in this Section for 4-inch HDPE solid wall pipe and fittings will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

B. Providing for and complying with the requirements set forth in this Section for 4-inch HDPE perforated pipe and fittings will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

C. Providing for and complying with the requirements set forth in this Section for 6-inch HDPE solid wall pipe and fittings will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

D. Providing for and complying with the requirements set forth in this Section for 18-inch HDPE perforated and solid wall pipe and fittings will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

E. Providing for and complying with the requirements set forth in this Section for 4-inch HDPE solid and perforated pipe and fittings for vertical installation in wells will be incidental to Section 02074 for well installation.

F. The following are considered incidental to the Work:

- shipping, handling and storage.
- layout survey.
- mobilization.
- Fittings, and other pipe appurtenances.
- Fusing and Joining.
- rejected material.
- rejected material removal, handling, re-testing, repair, and replacement.
- filtration Geotextile in accordance with Section 02771.
- trench Excavation and Backfill in accordance with Section 02200.

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 textured high-density polyethylene (HDPE) geomembrane, as shown on the Drawings. The work shall be carried out as specified herein and in accordance with Drawings.

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

1.02 Related Sections

Section 02771 — Geotextile

Section 02772 — Geosynthetic Clay Liner

Section 02773 — Geocomposite

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

C. Latest version of the American Society for Testing and Materials (ASTM) standards:

ASTM 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 1004	Standard Test Method of Initial Tear Resistance of Plastic Film and Sheeting
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 5321	Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method
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 5994 Standard Test Method for Measuring Core Thickness of Textured Geomembranes

ASTM D 6392 Test Method for Determining the Integrity of Non-reinforced Geomembrane Seams Produced using Thermo-Fusion Methods.

1.04 Qualifications

A. 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.

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 10,000,000 ft^2 of polyethylene geomembrane on previous projects.

3. The installation crew shall have the following experience.

   a. The Superintendent shall have supervised the installation of a minimum of 2,000,000 ft^2 of polyethylene geomembrane on at least five (5) different projects.

   b. At least one seamer shall have experience seaming a minimum of 1,000,000 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 Installer shall furnish the Engineer 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 Engineer and Owner.

B. The Geosynthetic Installer shall furnish the Engineer 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 Engineer and 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 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 Part 2.02 of this Section.

2. The manufacturer warranty specified in Part 1.05.A of this Section.

C. The Geosynthetic Installer shall submit the following information to the Engineer for approval 14 days prior to mobilization.

1. A drawing showing the installation layout identifying geomembrane panel configurations, dimensions, details, locations of seams, as well as any variance or additional details that deviate from the Drawings. The layout shall be adequate for use as a construction plan and shall include dimensions, details, etc. The layout drawings, as modified and/or approved by the Engineer, shall become part of these 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 installer has installed a minimum of 10,000,000 ft^2 of polyethylene geomembrane, in accordance with Part 1.04 of this Specification. The following information shall be provided for each facility:

   a. the name and purpose of the facility, its location, and dates of installation;
   b. the names of the owner, project manager, 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 Part 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 Part 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 Part 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 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.

1.07 Quality Assurance

A. The Geosynthetic Installer shall ensure that the materials and methods used for installation of the geomembrane 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 Geosynthetic Installer.

B. 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 Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer’s materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 Geomembrane Properties

A. The Geomembrane Manufacturer shall furnish double-sided, textured geomembrane having properties that comply with the required property values shown in Table 02770-1.

B. In addition to the property values listed in Table 02770-1, the geomembrane shall:

1. Contain a maximum of 1 percent by weight of additives, fillers, or extenders (not including carbon black).

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

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 5994
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.

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. Handling and care of the geomembrane prior to and following installation at the site 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 Engineer.

B. 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.

C. The geomembrane shall be delivered at least 14 days prior to the planned deployment date to allow the Engineer adequate time to perform conformance testing on the geomembrane samples as described in Part 3.05 or this Section. If the Engineer performed a visit to the manufacturing plant and performed the required conformance sampling, geomembrane can be delivered to the site within the 14 days prior to the planned deployment date as long as there is sufficient time for the Engineer to complete the conformance testing and confirm that the rolls shipped to the site are in compliance with this Section.

PART 3 – 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 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 composite liner system.

3.02 Geomembrane Deployment

A. Layout Drawings:

1. The Geosynthetic Installer shall deploy the geomembrane panel in general accordance with the layout drawing specified. The layout drawings 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 an 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 layout drawings.

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 104°F, 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 with adjacent components.

f. Temporary ballast and/or anchors (e.g., sand bags), not likely to damage the geomembrane, are placed on the geomembrane to prevent wind uplift.

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.

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, which 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 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.

B. Personnel:

1. All personnel performing seaming operations shall be qualified as indicated in Part 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 104°F. If the Geosynthetic Installer wishes to use methods that may allow seaming at ambient temperatures below 40°F or above 104°F, he shall use a procedure approved by the Engineer.

2. A meeting will be held with the Geosynthetic Installer and Engineer to establish acceptable installation procedures. In all cases, the geomembrane shall be dry and protected from wind damage.

3. Ambient temperatures shall be measured between 0 to 6 in. above the geomembrane surface.

D. Overlapping:

1. 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. 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. The electric generator shall be properly grounded.

G. Seaming Process:

1. Approved processes for field seaming are extrusion welding and fusion welding. Only equipment identified as part of the approved submittal specified in Part 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 apparatus shall be equipped with gauges giving the temperature in the apparatus.

   c. Prior to beginning a seam, the extruder shall be purged until all heat-degraded extrudate has been removed from the barrel.

   d. The Geosynthetics Installer shall provide documentation regarding the welding rod to the Engineer and shall certify that the welding rod is compatible with the specifications.

   e. 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, beginning of the day and after lunch, for each seaming apparatus used each day. 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 Part 3.03.D of this Section.

2. Four adjoining coupon specimens, each 1-in. wide, shall be cut from the trial seam sample by the installer using a die cutter to ensure precise 1-in. wide coupons. The coupons shall be tested in peel (outside (fusion only) and inside track) and shear using an electronic readout field tensiometer in accordance with ASTM D 4437, at a strain rate of 2 in./min., and they shall not fail in the seam (i.e., Film Tear Bond (FTB), which is failure in the parent material, is required). The required peel and shear seam strength is listed in Table 02770-2. Ideally, samples shall be conditioned at $23 \pm 2^{o}C$ at a relative humidity of $50 \pm 5\%$ for two hours prior to testing. If test conditions vary from these conditions, a 1-in. wide coupon of the parent geomembrane material (no weld) shall be tested in the same manner as the seam specimens to determine the break strength at this condition.

3. If a coupon specimen fails, the entire operation shall be repeated. If the additional coupon specimen fails, 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 Part 3.03.K of this Section. The following procedures shall apply:

a. Vacuum testing in accordance with ASTM D 5641.

b. Air pressure testing (for double-track fusion seams only) in accordance with ASTM D 5820 and the following:

i. Energize the air pump to a pressure between 25 and 30 pounds per square inches, close valve, and sustain the pressure for not less than 5 minutes.

ii. If loss of pressure exceeds 3 pounds per square inches, or does not stabilize, locate faulty area and repair in accordance with Part 3.03.K of this Section.

iii. Cut opposite end of air channel from pressure gauge and observe release of pressure to ensure air channel is not blocked.

iv. Remove needle, or other approved pressure feed device, and seal repair in accordance with Part 3.03.K of this Section.

c. Spark testing shall be performed 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 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 Engineer will be responsible for choosing the locations. The Geosynthetic Installer shall not be informed in advance of the locations where the seam samples will be taken. The Engineer reserves the right to increase the sampling frequency.

b. Samples shall be cut by the Geosynthetic Installer at the locations designated by the Engineer as the seaming progresses in order to obtain laboratory test results before the geomembrane is covered by another material. Each sample shall be numbered and the sample number and location identified on the panel layout drawing. All holes in the geomembrane resulting from the destructive seam sampling shall be immediately repaired in accordance with the repair procedures described in Part 3.03.K of this Section. The continuity of the new seams in the repaired areas shall be tested according to Part 3.03.I of this Section.

c. Two strips of dimensions 1-in. wide and 12 in. long with the seam centered parallel to the width shall be taken from either side of the sample location. These samples shall be tested in the field in accordance with Part 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 lengthwise. The sample shall be cut into three parts and distributed as follows:

i. One portion 12-in. long to the Geosynthetic Installer.
ii. One portion 18-in. long to the Geosynthetic CQA Laboratory for testing.
iii. One portion 12-in. long to the Engineer for archival storage.

3. Field Testing:

a. The two 1-in. wide strips shall be tested in the field tensiometer in the peel mode. The Engineer has the option to request an additional test in the shear mode. If any field test sample fails to meet the requirements in Table 02770-2, then the procedures outlined in Part 3.03.J.5 of this Section shall be followed.

4. Laboratory Testing:

   a. Testing by the Geosynthetics CQA Laboratory will include "Seam Strength" and "Peel Adhesion" (ASTM D 4437) with the 1-in. wide strip 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 4437. The minimum acceptable values to be obtained in these tests are indicated in Table 02770-2. Both 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 passed destructive test locations.

      ii. The Geosynthetic Installer can trace the welding path to an intermediate location, a minimum of 10 feet from the location 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 field tests, then full laboratory samples shall be taken. These full laboratory samples shall be tested in accordance with Part 3.03.J.4 of this Section. If these laboratory samples pass the tests, then the seam shall be reconstructed between these locations. If either sample fails, then the process shall be repeated to establish the zone in which the seam should be reconstructed. All acceptable seams must be bounded by two locations from which samples passing laboratory destructive tests have been taken. In cases exceeding 150 ft of reconstructed seam, a sample taken from within the reconstructed zone must pass destructive testing.

   b. Whenever a sample fails, the Engineer may require additional tests for seams that were formed by the same seamer and/or seaming apparatus or seamed 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. Each suspected location, both in seam and non-seam areas, shall be nondestructively tested using the methods described Part 3.03.I of this Section, as appropriate. Each location that fails nondestructive testing shall be marked by the Engineer and repaired by the Geosynthetic Installer.

3. When seaming of a geomembrane is completed (or when seaming of a large area of a geomembrane is completed) and prior to placing overlying materials, the Engineer shall identify all excessive geomembrane wrinkles. The Geosynthetic Installer shall cut and reseam all wrinkles so identified. The seams thus produced shall be tested like any 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 exist. 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, used to repair holes larger than 1/16 inch, tears, undispersed raw materials, and contamination by foreign matter;

      ii. abrading and reseaming, used to repair small sections of extruded seams;

      iii. spot seaming, used to repair minor, localized flaws;

      iv. capping, used to repair long lengths of failed seams; and

      v. removing bad seam and replacing with a strip of new material seamed into place (used with long lengths of fusion seams).

   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;

      iv. the repair procedures, materials, and techniques 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 shall be rounded with a radius of at least 3 in.; and

      vi. the geomembrane below large caps shall be appropriately cut to avoid water or gas collection between the two sheets.

5. Repair Verification:

   a. Each repair shall be nondestructively tested using the methods described in Part 3.03.I of this Section, as appropriate. Repairs that pass the nondestructive test shall be taken as an indication of an adequate repair. Failed tests will require the repair to be redone and retested until a passing test results. At the discretion of the Engineer, destructive testing may be required on large 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 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. Equipment shall not be driven directly on the geomembrane.

3.05 Conformance Testing

A. Samples of the geomembrane will be removed by the Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The Geosynthetic Installer shall assist the Engineer in obtaining conformance samples. The Geosynthetic Installer and Contractor shall account for this testing in the installation schedule. Only material that meets the requirements of Part 2.02 this Section shall be installed.

B. Samples will be selected by the Engineer in accordance with this Section and with the procedures outlined in the CQA Plan.

C. Samples will be taken at a minimum frequency of one sample per 100,000 ft^2.

D. The Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Part 2.02 of this Section.

E. The following tests will be performed by the Engineer:

Test	Test Method
Specific Gravity	ASTM D 792 or D 1505
Thickness	ASTM D 5994
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 Part 1.07.C of this Section, or that conformance testing indicates do not comply with Part 2.02 of this Section, will 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 Engineer.

B. The geomembrane shall be accepted by the Engineer when:

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 textured 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:

- shipping, handling and storage.
- layout survey.
- mobilization.
- rejected material.
- overlaps and seaming.
- rejected material removal, handling, re-testing, and repair.
- temporary anchorage.

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 5994
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 %	90 100 126 12	ASTM D 638
Puncture	Minimum	lb	90	ASTM D 4833
Tear Resistance	Minimum	lb	42	ASTM D 1004
Interface Shear Strength	-	-	Note 1	ASTM D 5321
Environmental Properties				
Carbon Black Content	Range	%	2-3	ASTM D 1603
Carbon Black Dispersion	N/A	none	Note 2	ASTM D 5596
Environmental Stress Crack	Minimum	hr	400	ASTM D 5397

Notes: (1) Interface shear strength test(s) shall be performed, by the Engineer, on the composite liner system in accordance with Section 02772 — Geosynthetic Clay Liner.
(2) Minimum 8 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	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)]				
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.

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.

B. The work shall include, but not be limited to, delivery, storage, placement, and seaming of the various geotextile components of the project.

C. Filter geotextile shall be used overlying the drainage aggregate. Cushion geotextile shall be used overlying the geomembrane and underlying the drainage aggregate. A UV protective geotextile shall be used overlying the exposed portions of geosynthetic components of the side slope liner system.

1.02 Related Sections

Section 02200 — Earthwork

Section 02225 — Drainage Aggregate

Section 02770 — Geomembrane

Section 02773 — Geocomposite

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 3786	Standard Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabric-Diaphragm Bursting Strength Test Method
ASTM D 4355	Standard Test Method for Deterioration of Geotextile from Exposure to Ultraviolet Light and Water
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 4595	Standard Test Method for Wide Width Tensile Properties of Geosynthetics
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 Geosynthetic Installer shall submit to the Engineer, at least 7 days prior to geotextile delivery, the following information regarding the proposed geotextile:

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 Geosynthetic Installer shall submit to the Engineer the manufacturing quality control 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 quality control certificates shall include:

1. lot, batch, and/or roll numbers and identification; and
2. results of quality control tests, including a description of the test methods used.

1.05 Quality Assurance

A. The Geosynthetic Installer shall ensure that the geotextile and installation methods used 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 Geosynthetic Installer.

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 Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer's materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 Geotextile Properties

A. Geotextile suppliers shall furnish materials in which the "Minimum Average Roll Values", as defined by the Federal Highway Administration (FHWA), meet or exceed the criteria specified in Table 02771-1.

B. The geotextile shall be nonwoven materials, suitable for use in filter/separation and cushion applications and woven geotextile for use as a UV protective layer.

2.02 Manufacturing Quality Control

A. The geotextile shall be manufactured with quality control 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 Geosynthetic Installer shall replace any rejected rolls.

D. If a geotextile sample fails to meet the quality control requirements of this Section the Geotextile Manufacturer shall 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 bound the failed roll(s).

E. Additional sample testing may be performed, at the Geotextile Manufacturer's discretion and expense, to identify more closely any 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, at a minimum once every 100,000 ft^2, to demonstrate that the geotextile properties conform to the values specified in Table 02771-1. At a minimum, the following manufacturing quality control tests shall be performed on each type of geotextile:

Test	Procedure	Cushion	Filtration	UV Protective
Mass per unit area	ASTM D 5261	Yes	No	No
Grab strength	ASTM D 4632	Yes	Yes	No
Puncture strength	ASTM D 4833	Yes	Yes	No
Permittivity	ASTM D 4491	No	Yes	No
A.O.S.	ASTM D 4751	No	Yes	Yes
Wide Width Tensile	ASTM D 4595	No	No	Yes

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 impermeable 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 Installer shall be liable for any damage to the materials 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 Engineer adequate time to perform conformance testing on the geotextile samples as described in Part 3.06 of this Section.

C. Handling, unloading, storage, and care of the geotextile prior to and following installation at the site, is the responsibility of the Geosynthetic Installer. The Geosynthetic Installer shall be liable for any damage to the materials incurred prior to final acceptance by the Engineer.

D. The Geosynthetic Installer shall be responsible for storage of the geotextile at the site.

E. The geotextile shall be protected from sunlight, excessive heat or cold, 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 Geosynthetic Installer.

PART 3 – EXECUTION

3.01 Familiarization

A. Prior to implementing any of the work described in this Section, the Geosynthetic Installer shall become thoroughly familiar with the site, the site conditions, and 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 such work is complete to the point where the installation of this Section may properly commence without adverse effect.

2. If the Geosynthetic Installer 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 installation of the geotextile will be construed as Geosynthetic Installer's acceptance of the related work of all other Sections.

3.02 Placement

A. Geotextile installation shall not commence until CQA conformance evaluations, by the Engineer, of previous work 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, he does so at his own risk. The Contractor shall account for the CQA conformance evaluations in the construction schedule.

B. The Geosynthetic Installer shall handle all geotextile in such a manner as to ensure they are not damaged in any way.

C. The Geosynthetic Installer 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 filtration and cushion 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 Geosynthetic Installer shall take care not to entrap stones, excessive dust, or moisture in the geotextile during placement.

F. The Geosynthetic Installer shall anchor or weight all geotextile with sandbags, or the equivalent, to prevent wind uplift.

G. The Geosynthetic Installer 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.

B. Geotextile shall be continuously sewn (i.e., spot sewing is not allowed) using a "single prayer" seam, with the stitching a minimum of 1.5 inches from the edge of the geotextile. Cushion to filtration geotextile shall be overlapped a minimum of 12 inches.

C. Geotextile shall be sewn with polymeric thread, having similar strength characteristics as the geotextile.

3.04 Repair

A. Any holes or tears in the geotextile shall be repaired using a patch made from the same geotextile. Geotextile patches will be sewn into place no closer than 1 inch from any panel edge. Should any tear exceed 50% of the width of the roll, that roll shall be removed and replaced.

B. Where geosynthetic materials underlie the geotextile being placed, care shall be taken to remove any soil or other material that may have penetrated the torn geotextile.

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.

C. Unless otherwise approved in writing by the Engineer, all equipment operating on materials overlying the geotextile shall comply with Section 02200 and Section 02225.

3.06 Conformance Testing

A. Samples of the geotextile materials will be removed by the Engineer after the material has been received at the site and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. 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 Engineer, at a minimum frequency of one sample per 200,000 ft^2.

C. The Engineer may increase the frequency of sampling in the event that test results do not comply with requirements of Part 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:

Test	Procedure	Cushion	Filtration	UV Protective
Mass per unit area	ASTM D 5261	Yes	No	No
Grab strength	ASTM D 4632	Yes	Yes	No
Puncture strength	ASTM D 4833	Yes	Yes	No
Permittivity	ASTM D 4491	No	Yes	No
A.O.S.	ASTM D 4751	No	Yes	Yes
Wide Width Tensile	ASTM D 4595	No	No	Yes

E. Any geotextile that is not certified in accordance with Part 1.04 of this Section, or that conformance testing results do not comply with Part 2.01 of this Section, will be rejected. The Geosynthetic Installer shall replace the rejected material with new material.

3.07 Protection of Work

A. The Geosynthetic Installer shall use all means necessary to protect all work of this Section.

B. In the event of damage, the Geosynthetic Installer 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 Cushion Geotextile will be measured as in-place square feet (SF), including Cushion Geotextile 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. Providing for and complying with the requirements set forth in this Section for Filtration Geotextile will be measured as in-place square feet (SF), including Filtration Geotextile 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.

C. Providing for and complying with the requirements set forth in this Section for UV Protection Geotextile will be measured as in-place square feet (SF), including UV Protection Geotextile 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.

D. The following are considered incidental to the Work:

- 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	CUSHION SPECIFIED VALUES	FILTER SPECIFIED VALUES	UV PROTECTIVE SPECIFIED VALUES	TEST METHOD
Type			Nonwoven	nonwoven	Woven	(-)
Mass per unit area	minimum	oz/yd^2	16	6[1]	-	ASTM D 5261
Filter Requirements						
Apparent opening size (O_{95})	maximum	mm	-	0.21	0.43	ASTM D 4751
Permittivity	minimum	s^{-1}	-	0.8	-	ASTM D 4491
Mechanical Requirements						
Grab strength	minimum	lb	350	130	-	ASTM D 4632
Puncture strength	minimum	lb	155	40	-	ASTM D 4833
Wide Width Tensile Strength	minimum	ppi	-	-	110	ASTM D 4595
Durability						
Ultraviolet Resistance @ 500 hours	minimum	%	70	70	70	ASTM D 4355

Notes: (1) For information purposes only, not a required property.

END OF SECTION

SECTION 02772
GEOSYNTHETIC CLAY LINER

PART 1 – GENERAL

A. The Contractor 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.

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

1.02 Related Sections

Section 02200 — Earthworks

Section 02770 — Geomembrane

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

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

ASTM D 2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
ASTM D 5321	Determination of the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method
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. The Manufacturer shall be a well-established firm with more than ten years of experience in the manufacturing of GCL.

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

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 method, to which are attached GCL samples.

2. projected delivery dates for this project.

3. Manufacturing quality control certificates for each shift's production, signed by responsible parties employed by the Manufacturer (such as the production manager).

4. The quality control certificates shall include:

    a. roll numbers and identification; and

    b. results of quality control tests, including description of test methods used, outlined in Part 2.01 of this Section.

5. The Manufacturer shall certify that the GCL meets all the properties outlined in 2.01 of this Section.

1.06 Construction Quality Assurance Monitoring

A. The Geosynthetic Installer shall ensure that the materials and methods used for the GCL 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.

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 Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 Material Properties

A. The flux 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 liner length shall be long enough to conform with the requirements specified in this Section.

C. The bentonite used to fabricate the GCL shall have at least 90 percent sodium mortmorillonite.

D. The bentonite component of the GCL shall be applied at a minimum concentration of 0.75 pound per square foot, when measured at a water content of less than or equal to 0 percent.

E. The geotextile components of the GCL shall have a minimum combined mass per unit area of 9 oz/yd^2 in accordance with ASTM D 5261.

F. The GCL shall meet the required property values listed in Table 02772-1.

G. 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.

H. An alternative GCL having a textured vapor barrier (i.e., geomembrane) as an integral component of the GCL may be provided. If an alternative GCL is used, the textured HDPE vapor barrier component shall be placed against the prepared subgrade and shall meet the following requirements:

1. have an average thickness of 30-mils in accordance with ASTM D 5994;

2. not have striations, pinholes, or bubbles on the surface or in the interior; and

3. be produced so as to be free of holes, blisters, undispersed raw materials, or any sign of contamination by foreign matter.

2.02 Interface Shear Testing

A. Interface Shear test(s) shall be performed on the proposed geosynthetic and soil components in accordance with ASTM D 5321. Tests shall be performed on several geosythetic interfaces as outlined below.

1. Dry GCL interface - the GCL shall be underlain by prepared subgrade compacted to 90% of the maximum dry density (ASTM D 1557) at the optimum moisture content and overlain by a textured 60-mil HDPE geomembrane, geocomposite, and operations layer material. The geosynthetic components of the liner system shall be allowed to “float” (i.e., not fixed) such that the failure surface can occur between any of the interfaces.

    a. The test shall be performed, under dry conditions, at normal stresses of 1, 3, and 5 psi at a shear rate of no more than 0.04 in./min. (1 mm/min.).

    b. The results of this test shall have a post peak apparent friction angle in excess of 18 degrees.

2. Hydrated GCL interface - the GCL shall be underlain by prepared subgrade compacted to 90% of the maximum dry density (ASTM D 1557) at the optimum moisture content and overlain by a textured 60-mil HDPE geomembrane, geocomposite, and operations layer material. The GCL component of the liner system shall be allowed to “float” (i.e., not fixed) such that the failure surface can occur at the top, bottom, or internal GCL interfaces.

    a. Before shearing, the GCL shall be hydrated under a loading of 240 psf (11 Kpa) for 48 hours. The test shall be performed under saturated conditions, at normal stresses of 20, 40, and 80 psi at a shear rate of no more than 0.04 in./min. (1 mm/min.).

    b. The results of this test shall have a post-peak apparent friction angle in excess of 12 degrees.

3. Hydrated GCL interface - If a GCL containing a geomembrane vapor barrier (i.e. Gundseal) is to be used, one additional shear strength test shall be performed. The GCL shall be underlain by prepared subgrade compacted to 90% of the maximum dry density (ASTM D 1557) at the optimum moisture content and covered by a textured 60 mil HDPE geomembrane, geocomposite, and operations layer material.

   a. The test shall evaluate the interface between the bentonite side of an unhydrated (i.e., dry) GCL containing a geomembrane vapor barrier and a textured 60 mil HDPE geomembrane. The test shall be set up such that the failure occurs between the bentonite component of the GCL and the textured geomembrane. The test shall be performed under dry conditions, at normal stresses of 20, 40, and 80 psi at a shear rate of no more than 0.04 in./min. (1 mm/min.).

   b. The acceptance criterion for the interface between the GCL consisting of a geomembrane vapor barrier component and the overlying geomembrane shall be as follows:

$$\tan^{-1}(0.25 \tan \delta_{hydrated} + 0.75 \tan\delta_{unhydrated}) \geq 12°$$

where $\delta_{hydrated}$ is the post-peak apparent friction angle in degrees determined in test No. 2 and $\delta_{unhydrated}$ is the post-peak apparent friction angle in degrees determined in test No. 3.

2.03 Manufacturing Quality Control

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 bound the failed roll(s).

E. Additional sample testing may be performed, at the Manufacturer's discretion and expense, to more closely identify 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 tests shall be performed by the Manufacturer: dry mass per unit area and index flux at frequencies of at least 1 per 50,000 ft^2 and 1 per 200,000 ft^2, respectively.

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

2.04 Packing and Labeling

A. GCLs shall be supplied in rolls wrapped in impermeable and opaque protective covers.

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

1. Manufacturer’s name;
2. product identification;
3. lot number;
4. roll number; and
5. roll dimensions.

2.05 Transportation, Handling and Storage

A. Handling, storage, and care of the GCL, prior to and following installation, is the responsibility of the Geosynthetic Installer, until final acceptance by the Engineer.

B. The GCL shall be stored and handled in accordance with ASTM D 5888.

C. The Geosynthetic Installer shall be liable for all damage to the materials incurred prior to and during transportation to the site.

D. 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 Part 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. 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 installation of this Section may properly commence without adverse impact.

2. 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 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 prior to commencement of geomembrane 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 Crest Anchorage System

A. The anchor trench shall be excavated, prior to GCL placement, to the lines and grades shown on the Drawings.

B. No loose soil shall be allowed in the anchor trench beneath the GCL.

C. The GCL shall be temporarily anchored in the anchor trench until all geosynthetic layers are installed in the anchor trench as shown on the Drawings.

3.04 Handling and Placement

A. The Geosynthetic Installer shall handle all GCL in such a manner that they are not damaged in any way and so that they do not become hydrated prior to, or during, installation.

B. In the presence of wind, all GCLs 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 Part 3.07 by the Geosynthetic Installer.

D. If an alternative GCL is used, the vapor barrier portion of the GCL shall be installed against the underlying prepared subgrade.

E. 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.

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

G. All hydrated GCL shall be removed and replaced by the Geosynthetic Installer.

3.05 Overlaps

A. On slopes steeper than 10 horizontal to 1 vertical, all GCL shall be continuous down the slope; that is, 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. As a minimum, along the length (i.e., the sides) of the GCL the overlap shall be 6 inches, and along the width (i.e., the ends) the overlap shall be 12 inches.

3.06 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. Material shall not be placed on a GCL that is hydrated.

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

D. Equipment shall not 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.

3.07 Repair

A. Any holes or tears in the GCL shall be repaired by placing a GCL patch over the hole. 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 water-based adhesive approved by the Manufacturer.

B. Care shall be taken to remove any soil or other material, which may have penetrated the torn GCL.

C. The patch shall not be nailed or stapled.

3.08 Conformance Testing

A. Samples of the GCL will be removed by the Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The Geosynthetic Installer shall assist the Engineer in obtaining conformance samples. The Geosynthetic Installer shall account for this testing in the installation schedule.

B. Samples shall be taken at a minimum frequency rate of one sample per 100,000 square feet.

C. The Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Part 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. As a minimum, the following conformance tests will be performed: mass per unit area and index flux. All tests shall be carried out at a frequency of one sample per 100,000 ft^2 and 400,000 ft^2, respectively. In addition, the Engineer will perform a minimum of two interface shear strength tests in accordance with Part 2.02.

E. Any GCL that is not certified by the Manufacturer in accordance with Part 1.05 of this section or that does not meet the requirements specified in Part 2.01 shall be rejected and replaced by the Geosynthetic Installer.

3.09 Protection of Work

A. The Geosynthetic Installer shall use all means necessary to 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.

PART 4 – MEASUREMENT AND PAYMENT

A. Providing for a complying with the requirements set forth in this Section for GCL will be measured as in-place square feet (SF), including GCL 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:

- shipping, handling and storage.
- overlaps and seaming.
- layout survey.
- mobilization.
- rejected material.
- rejected material removal, handling, re-testing, and repair.
- temporary anchorage.

TABLE 02772-1
REQUIRED GCL PROPERTY VALUES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED[1] VALUES	TEST METHOD
<u>Liner System Properties</u>				
Interface Shear Strength	minimum	degrees	18° (unhydrated) 12° (hydrated)	ASTM D 5321[2]
<u>GCL Properties</u>				
Bentonite Content[4]	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[3]
Moisture Content (Bentonite)	maximum	percent	25	ASTM D 2216

Notes: (1) All values represent minimum average roll values (i.e., any roll in a lot should meet or exceed the values in this table).
(2) Interface shear strength testing shall be performed, by the Engineer, in accordance with Part 2.02 of this Section.
(3) Hydraulic flux testing shall be performed under an effective confining stress of 5 pounds per square inch.
(4) Measured at a moisture content of 0 percent.

END OF SECTION

SECTION 02773
GEOCOMPOSITE

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 geocomposite. The work shall be carried out as specified herein and in accordance with the Drawings.

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

C. Geocomposite shall be used overlying the geomembrane and underlying the operations layer on the side slopes for the base liner system. Geocomposite shall be used overlying the geomembrane for the final cover liner system.

1.02 Related Sections

Section 02200 — Earthwork

Section 02770 — Geomembrane

Section 02771 — Geotextiles

1.03 References

A. Drawings

B. Construction Quality Assurance (CQA) Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 413. Standard Test Method for Rubber Property-Adhesion to Flexible Substrate.

ASTM D 792. Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement.

ASTM D 1603. Standard Test Method for Carbon Black in Olefin Plastics.

ASTM D 4491. Standard Test Methods for Water Permeability of Geotextiles by Permittivity.

ASTM D 4533. Standard Test Method for Trapezoid Tearing Strength of Geotextiles.

ASTM D 4632. Standard Test Method for Grab Breaking Load and Elongation of Geotextiles.

ASTM D 4716. Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products.

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 Geotextiles, Geomembranes, and Related Products.

ASTM D 5199. Standard Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes.

ASTM D 5261. Standard Test Method for Measuring Mass per Unit Area of Geotextiles.

1.04 Qualifications

A. The manufacturer shall be a well-established firm with more than one year experience in the manufacturing of geocomposite.

B. The Geosynthetic Installer shall install the geocomposite and shall meet the requirements of Section 02770 and this Section.

1.05 Submittals

A. The Geosynthetic Installer shall submit to the Engineer, at least 7 days prior to geocomposite delivery, the following information regarding the proposed geocomposite:

1. manufacturer and product name;
2. minimum property values of the proposed geocomposite and the corresponding test procedures;
3. projected geocomposite delivery dates; and
4. list of geocomposite roll numbers for rolls to be delivered to the site.

B. At least 7 days prior to geocomposite placement, the Geosynthetic Installer shall submit to the Engineer the manufacturing quality control certificates for each roll of geocomposite. The certificates shall be signed by responsible parties employed by the geocomposite manufacturer (such as the production manager). The quality control certificates shall include:

1. lot, batch, and/or roll numbers and identification; and
2. results of quality control tests, including a description of the test methods used.

1.06 Construction Quality Assurance Monitoring

A. The Geosynthetic Installer shall ensure that the geocomposite and installation methods used 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 Geosynthetic Installer.

B. 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 Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer's materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 – PRODUCTS

2.01 Geocomposite Properties

A. The Geocomposite Manufacturer shall furnish geocomposites having properties that comply with the required property values shown in Table 02773-1. The Geocomposite Manufacturer shall provide results of tests performed using the procedures listed in Table 02773-1, as well as certification that the materials meet or exceed the specified values.

B. Geotextiles will be thermally bonded to both sides of the geonet component of geocomposite material rather than chemically bonded.

C. Geocomposite suppliers shall furnish materials in which the “Minimum Average Roll Values”, as defined by the Federal Highway Administration (FHWA), meet or exceed the criteria specified in Table 02773-1.

D. The geocomposite’s geotextile components shall be nonwoven materials, suitable for use in filter/separation and cushion applications.

2.02 Manufacturing Quality Control

A. The geocomposite shall be manufactured with quality control procedures that meet or exceed generally accepted industry standards.

B. The geocomposite Manufacturer shall sample and test the geocomposite to demonstrate that the material conforms to the requirements of these Specifications.

C. Any geocomposite sample that does not comply with this Section shall result in rejection of the roll from which the sample was obtained. The Geosynthetic Installer shall replace any rejected rolls.

D. If a geocomposite sample fails to meet the quality control requirements of this Section the geocomposite Manufacturer shall 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 bound the failed roll(s).

E. Additional sample testing may be performed, at the geocomposite Manufacturer's discretion and expense, to identify more closely any non-complying rolls and/or to qualify individual rolls.

F. Sampling shall, in general, be performed on sacrificial portions of the geocomposite material such that repair is not required. The Geocomposite Manufacturer shall sample and test the geocomposite, at a minimum once every 100,000 ft^2, to demonstrate that the geocomposite properties conform to the values specified in Table 02773-1. At a minimum, the following manufacturing quality control tests shall be performed on the geotextile component of the geocomposite:

Test	Procedure
Mass per unit area	ASTM D 5261
Grab strength	ASTM D 4632
Puncture strength	ASTM D 4833
Permittivity	ASTM D 4491
A.O.S.	ASTM D 4751

G. At a minimum, the following manufacturing quality control tests shall be performed on the geonet component of the geocomposite:

Test	Procedure
Specific gravity	ASTM D 792
Nominal thickness	ASTM D 5199

H. At a minimum, the following manufacturing quality control tests shall be performed on the geocomposite:

Test	Procedure
Transmissivity	ASTM D 4716
Peel strength	ASTM D 413

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

2.03 Packing and Labeling

A. Geocomposite shall be supplied in rolls wrapped in relatively impermeable and opaque protective covers.

B. Geocomposite 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 Installer shall be liable for any damage to the materials incurred prior to and during transportation to the site.

B. The geocomposite shall be delivered to the site at least 14 days prior to the planned deployment date to allow the Engineer adequate time to perform conformance testing on the geocomposite samples as described in Part 3.06 of this Section.

C. Handling, unloading, storage, and care of the geocomposite prior to and following installation at the site, is the responsibility of the Geosynthetic Installer. The Geosynthetic Installer shall be liable for any damage to the materials incurred prior to final acceptance by the Engineer.

D. The Geosynthetic Installer shall be responsible for storage of the geocomposite at the site.

E. The geocomposite shall be protected from sunlight, excessive heat or cold, puncture, or other damaging or deleterious conditions. The geocomposite shall be protected from mud, dirt, and dust. Any additional storage procedures required by the geocomposite Manufacturer shall be the responsibility of the Geosynthetic Installer.

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. 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 installation of this Section may properly commence without adverse impact.

2. 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 installation of the geocomposite 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 geocomposite with the installation of other components of the lining system.

3.02 Handling and Placement

A. The Geosynthetic Installer shall handle all geocomposite in such a manner that it is not damaged in any way.

B. Install the geocomposite down the slope not across the slope. Place ends into the anchor trenches in such a manner as to continually keep the geocomposite in tension.

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

D. In the presence of wind, all geocomposites shall be sufficiently weighted with sandbags or the equivalent to prevent movement.

E. The geocomposite shall be positioned by hand after being unrolled to minimize wrinkles.

F. Care shall be taken during placement of geocomposites not to entrap dirt or excessive dust in the geocomposite 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 geocomposite, it should be cleaned prior to placement of the next material on top of it. Care shall be exercised when handling sandbags, to prevent rupture or damage of the sandbags.

G. Geocomposites shall only be cut using a hooked utility blade.

H. After unwrapping the geocomposite from its opaque cover, the geocomposite shall not be left exposed for a period in excess of 15 days.

3.03 Overlaps and Seams

A. Geonet Components:

1. The geonet components shall be overlapped a minimum 4 in. along the length. The geonet shall be overlapped by a minimum 1 ft. across the width.

2. Geonet overlaps shall be secured by tying with nylon cable ties. Tying devices shall be white or yellow for easy inspection. Metallic devices shall not be used.

3. Seaming of the geonet shall be performed by wrap-ties at 12-in. centers for end of panels and at 5-ft centers for edge of panel seams.

4. No end-of-panel seams shall be placed on slopes exceeding 10 %.

B. Geotextile Components:

1. The bottom layers of geotextile shall be overlapped. The top layers of geotextiles shall be continuously sewn.

2. Polymeric thread, with chemical resistance properties equal to or exceeding those of the geotextile component, shall be used for all sewing.

3.04 Placement of Overlying Materials

A. All overlying materials shall be placed in such a manner as to ensure that:

1. The geocomposite and underlying materials are not damaged;

2. Minimal slippage occurs between the geocomposite and underlying layers; and

3. Excess tensile stresses are not produced in the geocomposite.

4. Equipment shall not be driven directly on the geocomposite.

5. Unless otherwise approved in writing by the Engineer, all equipment operating on the materials overlying the geotextile shall comply with Section 02200 and Section 02225.

3.05 Conformance Testing

A. Samples of geocomposite will be removed by the Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The Geosynthetic Installer shall assist the Engineer in obtaining conformance samples. The Geosynthetic Installer shall account for this testing in the installation schedule.

B. Samples shall be taken at a minimum frequency rate of one sample per 200,000 square feet.

C. The Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Part 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. As a minimum, transmissivity and peel strength will be performed on each sample.

E. Any geocomposite that is not certified by the Manufacturer in accordance with Part 1.05 of this section or that does not meet the requirements specified in Part 2.01 shall be rejected and replaced by the Geosynthetic Installer.

3.06 Protection of Work

A. The Geosynthetic Installer shall use all means necessary to 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.

PART 4 – MEASUREMENT AND PAYMENT

A. Providing for a complying with the requirements set forth in this Section for geocomposite will be measured as in-place square feet (SF), including geocomposite 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:

- shipping, handling, and storage.
- overlaps and seaming.
- layout survey.
- mobilization.
- rejected material.
- rejected material removal, handling, re-testing, and repair.
- temporary anchorage.

TABLE 02773 - 1

GEOCOMPOSITE PROPERTY VALUES – BASE LINER SYSTEM

PROPERTIES	QUALIFIER	UNITS	SPECIFIED VALUES (1)	TEST METHOD
Geonet Component:				
Specific gravity	Minimum		0.935	ASTM D 792
Carbon black content	Range	%	2 – 3	ASTM D 1603
Nominal thickness	Minimum	mils	200	ASTM D 5199
Geotextile Components:				
Mass per unit area	Minimum	oz/yd^2	8	ASTM D 5261
Filter Requirements				
Apparent opening size	Maximum	mm	0.21 mm	ASTM D 4751
Permittivity	Minimum	1/s	0.6	ASTM D 4491
Mechanical Requirements				
Grab strength	Minimum	lb	190	ASTM D 4632
Puncture strength	Minimum	lb	110	ASTM D 4833
Geocomposite:				
Transmissivity (2)	Minimum	m^2/s	6.1 x 10^{-5}	ASTM D 4716
Peel Strength	Minimum	gm/in	500	ASTM D 413

Notes:

1. All values except transmissivity represent minimum average roll values (i.e., any roll in a lot should meet or exceed the values in this table).

2. The design transmissivity is the hydraulic transmissivity of the geocomposite measured using water at 68°F ±3°F (20°C ±1.5°C) with a hydraulic gradient of 0.1 under a compressive stress of not less than 12,000 psf (574 kPa). For the test, the geocomposite shall be sandwiched between a layer of operations material and a textured 60-mil HDPE geomembrane. The minimum test duration shall be 24 hours and the report for the test results shall include measurements at intervals over the entire test duration.

END OF SECTION

SECTION 02820
CHAIN LINK FENCE

PART 1 – GENERAL

1.01 Description of Work

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary to construct the Chain Link Fence. The Work shall be performed as specified herein and in accordance with the Drawings.

B. The Work shall include construction of footers, construction of posts, rails, and bracing, placement and securing the fence fabric, construction and installation of swing gates, and all other Work incidental to construction of a completed fence as shown on the Drawings and as described in this Section.

1.02 Related Sections

Section 01025 – Measurement and Payment

Section 01300 – Submittals

Section 01400 – Quality Control

Section 01500 – Construction Facilities

Section 01560 – Temporary Controls

Section 02200 – Earthwork

Section 03400 – Cast-in-Place Concrete

1.03 References

A. Drawings

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

ASTM A 153	Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware
ASTM A 1043	Standard Specification for Strength and Protective Coatings on Metal Industrial Chain Link Fence Framework

1.04 Delivery, Storage, and Handling

A. Deliver materials to Site in good condition, in unopened packaging, and with labels intact. Inspect materials upon delivery and replace damaged or contaminated materials.

B. Store materials above ground, under cover, in a dry place, and in a manner to prevent damage or staining.

C. Handle materials to prevent damage to surfaces, edges, and ends. Replace damaged materials at no additional cost to the Owner.

1.05 Submittals

A. The Contractor shall submit to the Construction Manager, at least 7 days prior to installation of fence material, certificates of compliance with the fence Manufacturer's specifications and that the material meets or exceeds all internal quality control requirements and the requirements of this Section.

1.06 Quality Assurance

A. The Contractor shall ensure that the materials and methods used for security chain link fence construction meet the requirements of the Manufacturer, 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 replaced by the Contractor to the satisfaction of the CQA Engineer at the Contractor's expense.

PART 2 – PRODUCTS

2.01 Security Fence Fabric

A. Fence fabric: 2" by 2" mesh size, 11 gauge core size, hot-dip zinc galvanized steel wire.

B. Fence fabric shall be a commercial-grade fence system, as supplied by Master-Halco, or shall be equivalent in core material, metallic-coating material, and all coating processes and strengths.

C. Manufacturer shall warranty the fence materials against defects and deterioration, other than normal wear and tear, for a minimum of 12 years.

D. Fabric selvages shall be twisted (barbed) on the top and knuckled on the bottom.

E. The heights of fabric shall be sufficiently long so that no horizontal splices are required.

2.02 Framework

A. Framework includes all posts and rails.

B. Fencing framework shall be made of tubular galvanized steel pipe that conforms to ASTM F 1043, Group 1a. The framework shall be standard weight, schedule 40 steel pipe, galvanized by the hot-dip method, with a minimum average of 1.8 ounces per square foot of zinc-coated surface.

C. Dimensions of the framework components shall be as shown on the Drawings.

2.03 Footers

A. Post footers shall be made with concrete rated at a minimum of 2,500 pounds per square inch (psi) and shall be constructed to the dimensions as shown on the Drawings.

2.04 Swing Gates

A. Gates frames shall be of the same materials and coatings as for the fence fabric and framework.

B. Dimensions of the gates and gate components shall be as shown on the Drawings.

C. Gate frame members shall be welded at joints for a rigid connection.

D. Contractor shall provide and install the following hardware for each gate:

E. Hinges: provide hinges of type to, size, and material to suit gate size. Hinges shall be non-liftoff type, offset to permit 180 degree gate opening.

F. Latch: exit control lock and cylinder, as shown on the Drawings.

G. Latch rail: steel plate, weld to stiles at interior side of gate to receive and protect latch.

2.05 Hardware

A. Hardware shall include bolts, tension rods, and truss rods.

B. Hardware shall be made of galvanized steel as per ASTM A 153.

C. Bolts shall be 3/16” diameter self drilling hex head TEK screws with flat washers, as manufactured by Hilti, Red Head, or approved.

D. Tension rods shall be stainless steel, in standard lengths to equal full height of fabric, with maximum cross section to suit fabric openings. Provide one tension rod for each gate post and corner post.

E. Truss rods shall be minimum 3/8” diameter threaded, galvanized steel rod and turnbuckle.

2.06 Fittings

A. Fittings include: tension and brace bands, caps, eye tops, rail ends, sleeves, and tie wires.

B. All fittings, except tie wires, shall be hot-dip galvanized steel. Tie wires shall be zinc-coated steel wire.

PART 3 – EXECUTION

3.01 General

A. When shipping, delivering, and installing all fence materials, do so to ensure a sound, undamaged installation. Provide storage for all materials and equipment delivered to the Site that is protective of stored materials. Handle and store materials in accordance with the Manufacturer's recommendations.

B. Prior to installation, examine surfaces designated to receive Work described in the Section for conditions adversely affecting the finished Work. Repair or replace surfaces not meeting tolerances or quality requirements governing substrate construction prior to initiating this Work.

C. Do not begin installation and erection before construction of the concrete secondary containment portion of the Work is complete.

3.02 Installation

A. Install materials in accordance with accepted shop drawings and Manufacturer’s printed instructions.

B. Provide top and intermediate rails as shown on Drawings. Install each as one piece between posts. Offset as necessary to allow for depth of fabric.

C. Place chain link fabric on the outside of the area to enclosed. Secure one end and apply sufficient tension to remove all slack before making attachments elsewhere. Tighten the fabric to provide a smooth uniform appearance, free from sag.

D. Cut fabric by untwisting on picket and attach each span independently at all terminal posts. Install tension rods with bolts and washers at 15” on center.

E. Fasten fabric to all posts, rails, and gate frames with bolts and washers at 15” on center.

F. Gates: install gates plumb, level and secure, for full operation without interference. Adjust hardware for smooth operation and lubricate where necessary. Gates shall open outward form the area to be secured.

G. Clearances: install fencing and gates with a maximum ½” clearance between the perimeter of the fabric and the framing, between the framing and adjacent construction, and between the perimeter of each gate leaf and surrounding construction. Close off gaps exceeding 1/2”at the direction of the Construction Manager.

3.03 Cleaning Up

A. During the progress of the Work, the premises shall be kept free of debris and waste. Upon completion, remove from the Site and dispose of all debris and surplus materials in a lawful manner.

B. At completion of Work, touch up minor damage to all surfaces to the satisfaction of the Construction Manager. Protect completed Work until final acceptance by the Owner.

3.04 Survey

A. The locations of the fence posts shall be surveyed by the Surveyor, and shall be included in the Record Drawings.

PART 4 – MEASUREMENT AND PAYMENT

4.01 General

A. Providing for and complying with the requirements set forth in this Section for chain link fence will be measured as linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- Submittals.
- Mobilization.
- Quality Control.
- Shipping, handling and storage.
- Footers
- Framework
- Welding.
- Fence fabric
- Gates
- Hardware and fittings
- Clean up
- As-built survey.

END OF SECTION

Section 5
Final Cover Calculation Packages

Section 5
Final Cover Calculation Packages

Calculation Package A
Geocomposite and Pipe Size Calculation

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 04

Title of Computations: Drainage Geocomposite and Pipe Size Requirements

Computations By:
SIGNATURE (GS) — DATE: 10 July 2000
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE: 13 JULY 2000
MICHAEL E. KALINSKI / SR. STAFF ENG.
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — DATE: 7/14/00
GREGORY T CORCORAN / PROJ. ENG.
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):
SIGNATURE — DATE: 7/14/00
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE — DATE: 7/17/2000
Bradford H. Cooley / Project Manager
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 00 / 07 / 10 (YY MM DD) Reviewed by: GBK 7.13.00 GTC Date: 00 / 07 / 14 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

DRAINAGE COMPOSITE AND PIPE SIZE REQUIREMENTS
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

The objective of this analysis is to evaluate the hydraulic performance of a drainage geocomposite and drainage pipe within the final cover system at the BRC Corrective Action Management Unit (CAMU). A drainage pipe is proposed to convey the collected water (from the geocomposite) to perimeter ditches. This calculation will evaluate the required criteria (e.g., transmissivity, pipe diameter) for the geocomposite and pipes.

SUMMARY OF ANALYSIS

The analyses indicate that the ultimate (or laboratory) transmissivity of the drainage composite shall be greater than 1 x 10^{-3} m^2/sec-m at σ_n = 2000 psf and i = 0.10. The drainage path shall be no longer than 1100 ft. The drainage pipe shall be 6-inch diameter perforated, corrugated polyethylene (CPE) pipe. The interior wall of the pipe shall be smooth.

METHOD OF ANALYSIS

The approach of the analysis is to first select a representative geocomposite from manufacturer data. Then, the ultimate transmissivity of the geocomposite will be evaluated from the manufacturer data. The allowable transmissivity of the geocomposite will be evaluated by reducing the ultimate transmissivity by partial factors of safety to account for field conditions. The allowable transmissivity will be used in an analysis performed using the Hydrological Evaluation of Landfill Performance (HELP) model developed by the EPA. The HELP model was developed to conduct water balance analysis of landfills, cover systems, and solid waste disposal and containment facilities.

Precipitation values used in this analysis are based on the assumed irrigation rates of a potential golf course that may be constructed overlying the final cover system. Based on verbal conversations with golf course landscape managers in Palm Springs, CA, approximately 3.5 inches of water per month is used from September to April and approximately 7 inches during the summer months. This equates to a total of about 56 inches per year. In addition, at the end of the model year, a 3-inch storm was simulated in HELP.

Written by: Geoff L. Smith GLS Date: 00 / 07 / 10 (YY MM DD) Reviewed by: MSK 7/13/00 GLC Date: 00 / 07 / 14 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

The maximum drainage length of the geocomposite will be evaluated by varying its value to limit the head on the liner to a value less than the design criterion.

FINAL COVER LINER SYSTEM

The final cover system consists of, from top to bottom:

GLS 7-10-00

- 2 ft native material; top deck: ~~double~~ single-sided nonwoven GT
- a geocomposite; side slope: double-sided nonwoven GT
- a 60-mil (1.5-mm) thick high density polyethylene (HDPE) geomembrane;
- a geosynthetic clay liner (GCL); and
- prepared subgrade.

The sideslope inclination is 3.0H:1.0V and the top deck area has a minimum grade of 2%. The maximum height of the side slope is 30 vertical feet (Figure 1).

DESIGN CRITERION

The design criteria consists of an allowable liquid head over the geomembrane component of the final cover system for the top deck area and side slopes. The allowable liquid head over the geomembrane for the side slopes is a negligible amount to limit potential sloughing failure of the vegetative cover soil component of the final cover system. The allowable head over the geomembrane for the top deck is less than twelve inches (0.3 m), to minimize unstable areas in the vegetative cover soil.

ANALYSIS

CONSERVATIVE

Geocomposite Design

A review of manufacturer literature suggests that an ultimate transmissivity of 1×10^{-3} m^2/sec can be achieved (see Attachment A) under gradients and normal stresses representative of the BRC CAMU final cover.

To ensure that the transmissivity of the proposed drainage composite meets or exceeds the required values over the life of the final cover, the ultimate transmissivity must be reduced through the use of appropriate partial factors of safety. These partial factors of safety make the adequate adjustment between the laboratory transmissivity values for drainage composite and actual field conditions.

As seen in Attachment B, Koerner suggests four partial factor of safety values that should be accounted for: the intrusion of the adjacent geotextile into the core of the geonet (FS_{IN}), creep deformation of the geonet (FS_{CR}), factor of safety against chemical clogging of the geonet (FS_{CC}), and factor of safety against biological clogging of the geonet (FS_{BC}). Partial factor of safety values were applied to the geotextile in the filtration geotextile calculation to account for flow through the geotextile component of the drainage composite.

Attachment B shows the ranges for the partial factors of safety. For the purposes of this analysis, the following factors of safety values were selected:

$FS_{IN} = 1.0$ (Accounted for during the testing of the drainage composite)
$FS_{CR} = 1.3$
$FS_{CC} = 1.0$ (Surface water)
$FS_{BC} = 1.2$ (Surface water)

The allowable transmissivity of the drainage composite then becomes:

$$\theta_{allowable} = \theta_{ultimate} / (\prod FS)$$

$\prod FS$ = product of all the partial factors of safety for the site specific conditions

The allowable transmissivity of the drainage composite is then calculated as:

$$\theta_{allowable} = \theta_{ultimate} / (\prod FS) = 1 \times 10^{-3}\ m^2/sec\ /\ [\ 1.0*1.3*1.0*1.2]$$

$$\mathbf{= 6.4 \times 10^{-4}\ m^2/sec\ at\ \sigma_n = 2000\ psf,\ i = 0.10}$$

The hydraulic conductivity and the thickness of the geocomposite are input for the HELP model to evaluate the maximum drainage distance. The hydraulic conductivity can be translated to transmissivity by using Darcy's Law as shown below:

$q = kiA = ki(t * W) = kti = \theta i$, therefore, for a unit width,
$k = \theta / t$

The thickness of the geocomposite can be ascertained from manufacturer information. (ATTACHMENT A) The thickness used herein is 225 mils (0.0057 m) for the Gundnet XL-14. Hence, the allowable hydraulic conductivity is 11 cm/sec (6.4×10^{-4} m^2/sec / 0.0057 m * 100) for the geocomposite.

HELP model output is presented in Attachment D. Typical near surficial soils exhibit a permeability of 5.3×10^{-4} cm/sec (Converse 1999) (Attachment C). Input parameters and results from the HELP model are presented below for the top deck area:

K_{veg} (cm/sec)	drainage length (ft)	Gradient (%)	$t_{geocomposite}$ (in)	$k_{geocomposite}$ (cm/sec)	Head on Liner (in)
5.3×10^{-4}	1100	2.0	0.225	11	8.2

Input parameters and results from the HELP model are presented below for the side slope area (Figure 1):

K_{veg} (cm/sec)	drainage length (ft)	Gradient (%)	$t_{geocomposite}$ (in)	$k_{geocomposite}$ (cm/sec)	Head on Liner (in)
5.3×10^{-4}	150	33.3	0.225	11	0.0

Since the head on the side slope liner is negligable, slope stability analyses for the side slope that includes seepage forces are not needed. BHC 7/17/00

Based on the above analyses, the drainage geocomposite and drainage distances meet the design criteria.

Pipe Size Design

A pipe diameter must be chosen for the final cover collection pipes and the downchutes for the final cover drainage system. The pipe is assumed to be a CPE pipe with a smooth wall interior. Figure 1 shows the maximum contributing area that a lateral must collect. This maximum contributing area is 13.2 acres (located in the South Mesa). From the HELP model analyses (Attachment D), the peak daily quantity of water expected to be generated is 926 cubic feet per acre. Assuming that all of this liquid will have to flow through one pipe, the following flow rate can be calculated:

(926 CF/day/acre) * 13.2 acres / (24 hours/day) / (60 min/hour) / (60 sec/min) = 0.14 cfs

Manning's Equation (Attachment E) can be used to estimate the flow rate in the pipe when flowing full:

$$Q_{full} = (1.486/n)(A)(R)^{2/3}(S)^{1/2}$$

where:

n = Manning's roughness coefficient, 0.012 from product literature (Attachment F)
A = area of pipe
R = hydraulic radius = area/wetting perimeter of the pipe (Attachment E)
S = slope of pipe, assume 0.5%

Assuming a 6-inch diameter corregated pipe flowing half-full, and using literature from ADS (Attachment F), the following values will be used in the above equation:

r = 3.0 in (Attachment F)
$A = \pi r^2 = \pi(3/12)^2 = 0.196\ ft^2$
$R = (0.196) / (2\pi(3/12)) = 0.125\ ft$

Placing the above values into the Manning's equation results in the following:

$$Q_{full} = (1.486/0.012)(0.196)(0.125)^{2/3}(0.005)^{1/2}$$

$$Q_{full} = 0.43\ cfs > 0.14\quad O.K.$$

Therefore, a 6-inch nominal diameter pipe is acceptable.

NOTE TO SPECIFICATIONS

The ultimate transmissivity of the drainage composite shall be greater than or equal to $1 \times 10^{-3}\ m^2/sec$ at $\sigma_n = 2000$ psf and i = 0.10. The drainage path shall be no longer than 1100 ft. The pipe shall be 6-inch diameter perforated corrugated polyethylene pipe with smooth interior wall.

Written by: Geoff L. Smith GLS Date: 00 / 07 / 10 Reviewed by: GTC Date: 00 / 07 / 14
YY MM DD YY MM DD

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

REFERENCES

Advanced Drainage Systems, Inc., Product Literature, (614) 457-3051. *(Attachment F)*

Koerner, R.M. (1994) "Designing with Geosynthetics", 3rd Edition, Prentice Hall, New Jersey *(Attachment B)*

Environmental Protection Agency (1994), "The Hydrologic Evaluation of Landfill Performance (HELP) Model," September 1994. *(model ouput presented in Attachment D)*

Converse Consultants (1999), "Preliminary Geotechnical and Geologic Investigation, Industrial Non-Hazardous Disposal Facility, Basic Management Incorporated, Clark County, Nevada," October 27, 1999, Las Vegas, Nevada. *(Attachment C)*

Chow, V.T. (1959) "Open Channel Hydraulics," McGraw-Hill, *(Attachment E)*

Product Description

Gundnet® XL-14 Drainage Media is a net-like product of two overlapping polyethylene strands which transmits fluids in the plane of the net.

Fabri-Net® XL-14 Drainage Composite consists of a geotextile fabric heat bonded to Gundnet® XL-14 Drainage Net on one or both sides.

- Gundnet® XL-14 and Fabri-Net® XL-14 drainage systems are rolled on 6 in. (15 cm) I.D. hollow cores.
- Each Gundnet® XL-14 roll is provided with a sling to aid handling on site.
- Fabri-Net® XL-14 Drainage Composite is wrapped in a protective plastic bag.
- Dimensions and weights are approximate.
- Custom lengths available.

Gundnet® XL-14 Drainage Net and Fabri-Net® XL-14 Drainage Composite

Gundnet® XL-14 Drainage Net and Fabri-Net® XL-14 Drainage Composite replace traditional one foot (0.3 m) thicknesses of drainage sand with typically 100 times the equivalent hydraulic conductivity of the sand drainage layer. They simultaneously add nearly one foot of air space to the project for each drainage layer they replace. Fabri-Net® XL-14 Drainage Composite allows single step deployment where geotextile separation of soils from drainage net is required. Double-sided Fabri-Net® XL-14 Drainage Composite (geotextile heat bonded on both sides) provides tremendous slope stability in combination with textured liner by gripping the geomembrane surface with velcro-like friction.

Standard Roll Dimensions

Gundnet® XL-14

Roll Width	6.5 ft. (2 m)	14 ft. (4.3 m)
Roll Length	100 ft. (30 m)	300 ft. (91 m)
Roll Weight	130 lbs (60 kg)	840 lbs (382 kg)

Fabri-Net® XL-14

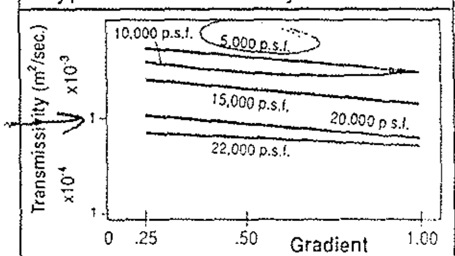
Roll Width [1]	6.5 ft. (2 m)	14 ft. (4.3 m)
Roll Length [2]	100 ft. (30 m)	200 ft. (61 m)
Roll Weight [2]	194 lbs (88 kg)	835 lbs (380 kg)

[1] Width with geotextile overlap is 7 ft. for 6.5 ft. width and 15 ft. for 14 ft. width.

[2] Value for Fabri-Net® XL-14 double-sided with 6 ounce geotextile only. Bonding geotextile with different weights will change the final roll weight and length.

Material: Gundnet® XL-14
Fluid: Water Temperature: 60°F

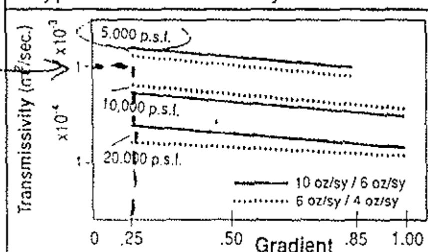
Typical Transmissivity Test Result



Section:
- Upper Load Plate
- Gundline® HD
- Gundnet® XL-14
- Gundline® HD
- Lower Load Plate

Material: Fabri-Net® XL-14
Fluid: Water Temperature: 70°F

Typical Transmissivity Test Result



Section:
- Upper Load Plate
- Sand
- Geotextile
- Gundnet® XL-14
- Geotextile
- Gundline® HD
- Lower Load Plate

Gundle Lining Systems Inc.

Gundle®

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Attachment A 1/1

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Table 4.2 Recommended preliminary factor of safety values for determining allowable flow rate or transmissivity of geonets

	Partial Factor of Safety Value in Equation 4.5			
Application Area	FS_{IN}	FS_{CR}*	FS_{CC}	FS_{BC}
Sport fields	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.1 to 1.3	1.0 to 1.2	1.1 to 1.5	1.1 to 1.3
Roof and plaza decks	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock and soil slopes	1.3 to 1.5	1.2 to 1.4	1.1 to 1.5	1.0 to 1.5
Drainage blankets	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2
Surface water drains for landfill caps	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.2 to 1.5
Secondary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Primary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0

*These values assume that the q_{ult} value was obtained using an applied normal pressure of 1.5 to 2 times the field-anticipated maximum value. If not, values must be increased.

done at the proper design load and hydraulic gradient and that this testing yielded a short-term between-rigid-plates value of 1.2 gal./min.-ft.

Solution: Since better information is not known, average values from Table 4.2 are used.

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

$$= 1.2\left[\frac{1}{1.1 \times 1.1 \times 1.1 \times 1.2}\right]$$

$$= 1.2\left[\frac{1}{1.60}\right]$$

$$= 0.75 \text{ gal./min.-ft.}$$

Example:

What is the allowable geonet flow rate to be used in the design of a secondary leachate collection system? Assume that laboratory testing at proper design load and proper hydraulic gradient gave a short-term between-rigid-plates value of 1.2 gal./min.-ft.

Solution: Average values from Table 4.2 are used; however, note the large reduction.

Attachment B

1/1

SOURCE Koerner (1994)

FROM CONVERSE (1999)

7-10-05
7-13-05 GTC

Exploration Location	Depth (feet)	Soil Description	Percent Sodium	Percent Sulfate	Total Available Water Soluble sodium Sulfate (%)
B-5	10-15	Silty sand with gravel	0.07	0.13	0.20
B-8	19-20	Silty sand with gravel	0.07	0.06	0.08
B-101	5-10	Silty sand with gravel	0.17	0.06	0.08
B-102	0-5	Fill – Silty sand with gravel	0.17	0.03	0.05
B-106	0-5	Silty sand with gravel	0.15	0.08	0.12
B-106	29-30	Silty sand with gravel	0.15	0.06	0.08

Permeability

Falling head permeability tests were conducted on remolded samples in general accordance with modified ASTM procedure D2434. The soil was compacted in a mold 4.6 inches long and 4.0 inches in diameter to 85 or 90 percent of maximum dry density and at optimum moisture content. A falling head was applied to the sample and the flow of water through the sample was monitored. The permeability was calculated after the flow rate had stabilized. The result of the falling head permeability test is presented in the following table:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-5	20-25	Silty sand with gravel	5.3×10^{-4} *
B-12	10-15	Silty sand with gravel	4.0×10^{-4}
B-102	20-25	Silty sand with gravel	1.0×10^{-4}
B-105	20-25	Well graded sand with silt and gravel	1.2×10^{-3}

Flexible wall permeameter tests were performed on selected samples by AP Engineering and Testing, Inc according to ASTM D5084. With the exception of one sample (B-105), all tested samples were undisturbed ring samples. The samples were placed in a triaxial machine with a constant confining pressure at the approximate in-place effective stress pressures. Results were generally consistent with the fal-

Attachment 7, 1/2

MN 7-13-00 GTC

ling head permeability test results for the granular materials. Laboratory results are presented on Drawing Nos. A-71 through A-76 and summarized below:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-1	14-15	Silty sand with gravel	1.57×10^{-4}
B-4	24-25	Silty sand with gravel	1.47×10^{-4}
B-8	44-45	Sandy silt	2.90×10^{-5}
B-12	39-39.5	Silty clay	1.76×10^{-7}
B-103	44-45	Silty clay	3.83×10^{-7}
B-105*	30-35	Silty sand	3.05×10^{-4}

* Sample remolded to 85% relative compaction at optimum moisture.

FROM CONVERSE (1999)

Attachment C
2/2

GB 7-13-00
MSK 7.13.00 GTC

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                 DEVELOPED BY ENVIRONMENTAL LABORATORY                    **
**                   USAE WATERWAYS EXPERIMENT STATION                      **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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******************************************************************************


PRECIPITATION DATA FILE:      r:\cawp\gls\help307\DATA44.D4
TEMPERATURE DATA FILE:        r:\cawp\gls\help307\DATA74.D7
SOLAR RADIATION DATA FILE:    r:\cawp\gls\help307\DATA134.D13
EVAPOTRANSPIRATION DATA:      r:\cawp\gls\help307\DATA114.D11
SOIL AND DESIGN DATA FILE:    r:\cawp\gls\help307\DATA104.D10
OUTPUT DATA FILE:             r:\cawp\gls\help307\TEST4.OUT


TIME:  11:11      DATE:   7/10/2000


******************************************************************************

     TITLE:  BRC CAMU, Condition 4        Top deck w/ geocomposite

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
               COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                              LAYER  1
                              --------

                    TYPE 1 - VERTICAL PERCOLATION LAYER
                        MATERIAL TEXTURE NUMBER   7
          THICKNESS                    =       1.00   INCHES
          POROSITY                     =       0.4730 VOL/VOL
          FIELD CAPACITY               =       0.2220 VOL/VOL
          WILTING POINT                =       0.1040 VOL/VOL
          INITIAL SOIL WATER CONTENT   =       0.2494 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.520000001000E-03 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
                FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
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Att. D

67C

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	23.00	INCHES
POROSITY	=	0.4730	VOL/VOL
FIELD CAPACITY	=	0.2220	VOL/VOL
WILTING POINT	=	0.1040	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3271	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.530000019000E-03	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	0.22	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.8500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	11.0000000000	CM/SEC
SLOPE	=	2.00	PERCENT
DRAINAGE LENGTH	=	1100.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	2.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	1.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 5

GTC

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.20 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 7 WITH A POOR STAND OF GRASS, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 1100. FEET.

SCS RUNOFF CURVE NUMBER	=	81.70	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	18.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	5.337	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	8.514	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.872	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	8.113	INCHES
TOTAL INITIAL WATER	=	8.113	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

STATION LATITUDE	=	36.08 DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00
START OF GROWING SEASON (JULIAN DATE)	=	62
END OF GROWING SEASON (JULIAN DATE)	=	321
EVAPORATIVE ZONE DEPTH	=	18.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	36.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR LAS VEGAS NEVADA

67C

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.72	224043.187	100.00
RUNOFF	0.845	3066.880	1.37
EVAPOTRANSPIRATION	57.247	207807.844	92.75
DRAINAGE COLLECTED FROM LAYER 3	3.4317	12457.194	5.56
PERC./LEAKAGE THROUGH LAYER 5	0.000008	0.028	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0548		
CHANGE IN WATER STORAGE	0.196	711.629	0.32
SOIL WATER AT START OF YEAR	8.113	29451.367	
SOIL WATER AT END OF YEAR	8.309	30162.996	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

GTC

ANNUAL WATER BUDGET BALANCE	-0.0001	-0.391	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 1

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.72	3.36	3.72	3.30	3.41	7.20
	7.75	8.05	7.52	3.55	3.60	6.54
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.845
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	2.805	3.292	4.494	3.871	3.646	7.066
	7.823	7.942	6.748	4.260	2.823	2.479
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	2.4674	0.5061	0.2495	0.0039	0.0014	0.0008
	0.0001	0.0002	0.0000	0.0004	0.0004	0.2015
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MC

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.5174	0.0159	0.0071	0.0001	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.1174
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.72 (0.000)	224043.2	100.00
RUNOFF	0.845 (0.0000)	3066.88	1.369
EVAPOTRANSPIRATION	57.247 (0.0000)	207807.84	92.753
LATERAL DRAINAGE COLLECTED FROM LAYER 3	3.43173 (0.00000)	12457.194	5.56018
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00001 (0.00000)	0.028	0.00001
AVERAGE HEAD ON TOP OF LAYER 4	0.055 (0.000)		
CHANGE IN WATER STORAGE	0.196 (0.0000)	711.63	0.318

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MA 7-10-03

6TC

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 1

	(INCHES)	(CU. FT.)
PRECIPITATION	3.00	10890.000
RUNOFF	0.845	3066.8796
DRAINAGE COLLECTED FROM LAYER 3	0.25524	926.53894
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000002	0.00546
AVERAGE HEAD ON TOP OF LAYER 4	4.490	
MAXIMUM HEAD ON TOP OF LAYER 4	8.210	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	93.9 FEET	
SNOW WATER	0.16	579.4747

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3497
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1515

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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Att. D

6TC

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FINAL WATER STORAGE AT END OF YEAR 1

LAYER	(INCHES)	(VOL/VOL)
1	0.2492	0.2492
2	7.7189	0.3356
3	0.1913	0.8500
4	0.0000	0.0000
5	0.1500	0.7500
SNOW WATER	0.000	

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MY 7-10-00
MSK 7.13.00

MC

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**                                                                      **
**                                                                      **
**             HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE            **
**               HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)             **
**                 DEVELOPED BY ENVIRONMENTAL LABORATORY                **
**                   USAE WATERWAYS EXPERIMENT STATION                  **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY          **
**                                                                      **
**                                                                      **
**************************************************************************
**************************************************************************


PRECIPITATION DATA FILE:     r:\cawp\gls\help307\DATA44.D4
TEMPERATURE DATA FILE:       r:\cawp\gls\help307\DATA74.D7
SOLAR RADIATION DATA FILE:   r:\cawp\gls\help307\DATA134.D13
EVAPOTRANSPIRATION DATA:     r:\cawp\gls\help307\DATA114.D11
SOIL AND DESIGN DATA FILE:   r:\cawp\gls\help307\DATA105.D10
OUTPUT DATA FILE:            r:\cawp\gls\help307\TEST5.OUT


TIME:  13:22     DATE:   7/10/2000


**************************************************************************

     TITLE:  BRC CAMU, Condition 5                  Side Slopes

**************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                       TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER   7
           THICKNESS                    =      1.00   INCHES
           POROSITY                     =      0.4730 VOL/VOL
           FIELD CAPACITY               =      0.2220 VOL/VOL
           WILTING POINT                =      0.1040 VOL/VOL
           INITIAL SOIL WATER CONTENT   =      0.2892 VOL/VOL
           EFFECTIVE SAT. HYD. COND.    =  0.520000001000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
                 FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                   LAYER  2
                                   --------

                       TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER   0
           THICKNESS                    =     23.00   INCHES
           POROSITY                     =      0.4730 VOL/VOL
           FIELD CAPACITY               =      0.2220 VOL/VOL
           WILTING POINT                =      0.1040 VOL/VOL
           INITIAL SOIL WATER CONTENT   =      0.2546 VOL/VOL
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Att. D

6TC

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EFFECTIVE SAT. HYD. COND.    =  0.530000019000E-03 CM/SEC


                              LAYER  3
                              --------

                  TYPE 2 - LATERAL DRAINAGE LAYER
                    MATERIAL TEXTURE NUMBER   0
THICKNESS                    =        0.22   INCHES
POROSITY                     =        0.8500 VOL/VOL
FIELD CAPACITY               =        0.0100 VOL/VOL
WILTING POINT                =        0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =        0.0158 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =     11.0000000000     CM/SEC
SLOPE                        =       33.30   PERCENT
DRAINAGE LENGTH              =      150.0    FEET


                              LAYER  4
                              --------

                  TYPE 4 - FLEXIBLE MEMBRANE LINER
                    MATERIAL TEXTURE NUMBER  35
THICKNESS                    =        0.06   INCHES
POROSITY                     =        0.0000 VOL/VOL
FIELD CAPACITY               =        0.0000 VOL/VOL
WILTING POINT                =        0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =        0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY          =        2.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =        1.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD


                              LAYER  5
                              --------

                    TYPE 3 - BARRIER SOIL LINER
                    MATERIAL TEXTURE NUMBER  17
THICKNESS                    =        0.20   INCHES
POROSITY                     =        0.7500 VOL/VOL
FIELD CAPACITY               =        0.7470 VOL/VOL
WILTING POINT                =        0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT   =        0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.300000003000E-08 CM/SEC


              GENERAL DESIGN AND EVAPORATIVE ZONE DATA
              ----------------------------------------

NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE # 7 WITH A
         POOR STAND OF GRASS, A SURFACE SLOPE OF 33.%
         AND A SLOPE LENGTH OF  150. FEET.

SCS RUNOFF CURVE NUMBER              =      84.80
FRACTION OF AREA ALLOWING RUNOFF     =     100.0     PERCENT
AREA PROJECTED ON HORIZONTAL PLANE   =       0.520   ACRES
EVAPORATIVE ZONE DEPTH               =      18.0     INCHES
INITIAL WATER IN EVAPORATIVE ZONE    =       4.344   INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE   =       8.514   INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE   =       1.872   INCHES
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61C

INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	6.298	INCHES
TOTAL INITIAL WATER	=	6.298	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
LAS VEGAS NEVADA

STATION LATITUDE	=	36.08 DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00
START OF GROWING SEASON (JULIAN DATE)	=	62
END OF GROWING SEASON (JULIAN DATE)	=	321
EVAPORATIVE ZONE DEPTH	=	18.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	9.10 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	39.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	21.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	24.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	36.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.50	0.46	0.41	0.22	0.20	0.09
0.45	0.54	0.32	0.25	0.43	0.32

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
44.60	50.10	55.30	63.50	73.30	83.60
90.30	88.00	80.10	67.60	53.60	45.40

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR LAS VEGAS NEVADA
AND STATION LATITUDE = 36.08 DEGREES

****

ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.72	116502.453	100.00
RUNOFF	0.941	1776.331	1.52
EVAPOTRANSPIRATION	45.182	85284.914	73.20
DRAINAGE COLLECTED FROM LAYER 3	15.2152	28720.299	24.65

G-TC

PERC./LEAKAGE THROUGH LAYER 5	0.000002	0.005	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0003		
CHANGE IN WATER STORAGE	0.382	721.107	0.62
SOIL WATER AT START OF YEAR	6.298	11888.534	
SOIL WATER AT END OF YEAR	6.680	12609.642	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.200	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 1

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.72	3.36	3.72	3.30	3.41	7.20
	7.75	8.05	7.52	3.55	3.60	6.54
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.941
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	2.535	2.748	3.640	3.724	2.661	5.384
	6.433	5.925	4.995	3.087	2.107	1.943
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	1.7294	0.7077	0.5946	0.3187	0.5837	1.5292
	1.3191	2.0582	2.4341	0.9180	1.1996	1.8228
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

6C

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0004	0.0002	0.0002	0.0001	0.0002	0.0004
	0.0003	0.0005	0.0007	0.0002	0.0003	0.0005
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	61.72 (0.000)	116502.5	100.00
RUNOFF	0.941 (0.0000)	1776.33	1.525
EVAPOTRANSPIRATION	45.162 (0.0000)	85284.91	73.204
LATERAL DRAINAGE COLLECTED FROM LAYER 3	15.21525 (0.00000)	28720.299	24.65210
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00000 (0.00000)	0.005	0.00000
AVERAGE HEAD ON TOP OF LAYER 4	0.000 (0.000)		
CHANGE IN WATER STORAGE	0.382 (0.0000)	721.11	0.619

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GTC

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 1

	(INCHES)	(CU. FT.)
PRECIPITATION	3.00	5662.800
RUNOFF	0.941	1776.3307
DRAINAGE COLLECTED FROM LAYER 3	0.16183	305.47559
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00001
AVERAGE HEAD ON TOP OF LAYER 4	0.001	
MAXIMUM HEAD ON TOP OF LAYER 4	0.002	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.16	301.3268

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.2709

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.1434

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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67C

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FINAL WATER STORAGE AT END OF YEAR 1

LAYER	(INCHES)	(VOL/VOL)
1	0.2881	0.2881
2	6.2395	0.2713
3	0.0027	0.0121
4	0.0000	0.0000
5	0.1500	0.7500
SNOW WATER	0.000	

****
****

CHANNEL DESIGN

MB 7-10-00

7-13-00

well-known form

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{5-6}$$

where V is the mean velocity in fps, R is the hydraulic radius in ft, S is the slope of energy line, and n is the coefficient of roughness, specifically known as *Manning's n*. This formula was developed from seven different formulas, based on Bazin's experimental data, and further verified by 170 observations.[1] Owing to its simplicity of form and to the satis-

---

a linear measure of roughness and $\phi(R/k)$ is a function of R/k. If $\phi(R/k)$ is considered dimensionless, n will have the same dimensions as those of $k^{1/6}$, that is, $L^{1/6}$.

On the other hand, of course, it is equally possible to assume that the numerator of $1.486/n$ can absorb the dimensions of $L^{1/3}T^{-1}$, or that $\phi(R/k)$ involves a dimensional factor, thus leaving no dimensions for n. Some authors, therefore, preferring the simpler choice, consider n a dimensionless coefficient.

It is interesting to note that the conversion of the units for the Manning formula is independent of the dimensions of n, as long as the same value of n is used in both systems of units. If n is assumed dimensionless, then the formula in English units gives the numerical constant $3.2808^{1/3} = 1.486$ since 1 meter = 3.2808 ft. Now, if n is assumed to have the dimensions of $L^{1/6}$, its numerical value in English units must be different from its value in metric units, unless a numerical correction factor is introduced for compensation. Let n be the value in metric units and n' the value in English units. Then, $n' = (3.2808^{1/6})n = 1.2190n$. When the formula is converted from metric to English units, the resulting form takes the numerical constant $3.2808^{1/3+1/6} = 3.2808^{1/2} = 1.811$, since n has the dimensions of $L^{1/6}$. Thus, the resulting equation should be written $V = 1.811R^{2/3}S^{1/2}/n'$. Since the same value of n is used in both systems, the practical form of the formula in the English system is $V = 1.811R^{2/3}S^{1/2}/1.2190n = 1.486R^{2/3}S^{1/2}/n$, which is identical with the formula derived on the assumption that n has no dimensions.

In a search of early literature on hydraulics, the author has failed to find any significant discussion regarding the dimensions of n. It seems that this was not a problem of concern to the forefathers of hydraulics. It is most likely, however, that n was unconsciously taken as dimensionless in the conversion of the Manning formula, because such a conversion, as shown above, is more direct and simpler.

Now, considering the approximations involved in the derivation of the formula and the uncertainty in the value of n, it seems unjustifiable to carry the numerical constant to more than three significant figures. For practical purposes, a value of 1.49 is believed to be sufficiently accurate [16].

Manning mentioned that the simplified form of the formula had been suggested independently by G. H. L. Hagen prior to Manning's own work, according to a statement by Major Cunningham [17]. Hagen's formula was believed to have appeared first in 1876 [7]. It is also known that Philippe-Gaspard Gauckler [18] had an early proposal of the simplified form of Manning's formula in 1868 and that Strickler [19] presented independently the same form of the formula in 1923.

[1] For the derivation of the exponent of R, use was made of Bazin's experimental data on artificial channels [12]. For different shapes and roughnesses, the average value of the exponent was found to vary from 0.6499 to 0.8395. Considering these variations, Manning adopted an approximate value of $\frac{2}{3}$ for the exponent. On the

Attachment E

1/1

Chow, V.T. "Open Channel Hydraulics", (1959) McGraw-Hill

WGK 7-13-00

Technical Notes

ADS
ADVANCED DRAINAGE SYSTEMS, INC.

Technical Note 2.109
Re: Flow Capacity
Date: March 1, 1995

It is the intent of this Technical Note to provide current hydraulic performance data for use by the engineering community. A bibliography is included for the engineer's use if further information or guidance is needed.

Manning's "n" values are offered for design purposes based on the best available data assembled from a variety of sources as indicated. Table 1 presents the Manning's "n" values recommended by the A.D.S. engineering staff for use in design.

WB 7-10-00 GK

Table 1
Manning's "n" Value For Design
(Storm & Sanitary Sewer and Culverts)

Pipe Type	"n"
A.D.S. Corrugated Polyethylene Pipe	
3" - 6" Diameter	0.015
8" Diameter	0.016
10" Diameter	0.017
12" - 15" Diameter	0.018
18" - 36" Diameter	0.020
A.D.S. N-12	0.012
Concrete Pipe	0.013
Corrugated Metal Pipe (2 2/3" x 1/2" corrugation)	
Annular	
Plain	0.024
Paved Invert	0.020
Fully Paved (smooth lined)	0.013
Helical	
Plain 15" Diameter	0.013
Plain 18" Diameter	0.015
Plain 24" Diameter	0.018
Plain 36" Diameter	0.021
Spiral-Rib	0.012
Plastic Pipe (SDR, S&D, Etc.)	0.011
Vitrified Clay	0.013

Attachment F 1/2

ADS N-12® PRODUCT ORMATION SHEET

Nominal Diameter	Inside Diameter, Average	Outside Diameter, Average	Wall Thickness, Minimum	Pipe Stiffness @ 5% Deflection	Weight lbs./20 ft. (kg./6 m.)	Area $in.^2/in.$	"T" $in.^4/in.$	"C" in.
4" (100 mm)	4.10" (104 mm)	4.78" (120 mm)	0.020" (0.50 mm)	50 psi (340 kN/m^2)	8.10 lbs. (3.60 kg.)	0.070	0.0014	0.29
6" (150 mm)	6.00" (152 mm)	6.92" (176 mm)	0.020" (0.50 mm)	50 psi (340 kN/m^2)	17.00 lbs. (7.71 kg.)	0.107	0.0036	0.25
8" (200 mm)	7.90" (200 mm)	9.11" (233 mm)	0.025" (0.64 mm)	50 psi (340 kN/m^2)	30.80 lbs. (13.97 kg.)	0.135	0.0070	0.27
10" (250 mm)	9.90" (251 mm)	11.36" (287 mm)	0.025" (0.64 mm)	50 psi (340 kN/m^2)	45.20 lbs. (20.50 kg.)	0.145	0.0110	0.34
12" (300 mm)	12.15" (308 mm)	14.45" (367 mm)	0.035" (0.89 mm)	50 psi (340 kN/m^2)	63.80 lbs. (28.96 kg.)	0.188	0.0410	0.53
15" (375 mm)	14.98" (380 mm)	17.57" (448 mm)	0.035" (0.89 mm)	42 psi (290 kN/m^2)	92.50 lbs. (42.00 kg.)	0.217	0.0660	0.66
18" (450 mm)	18.07" (459 mm)	21.20" (536 mm)	0.050" (1.27 mm)	40 psi (280 kN/m^2)	128.60 lbs. (58.38 kg.)	0.250	0.0890	0.75
24" (600 mm)	24.08" (612 mm)	27.80" (719 mm)	0.050" (1.27 mm)	34 psi (240 kN/m^2)	224.60 lbs. (101.97 kg.)	0.338	0.2310	1.07
30" (750 mm)	30.20" (767 mm)	36.07" (917 mm)	0.050" (1.27 mm)	28 psi (190 kN/m^2)	308.30 lbs. (139.97 kg.)	0.353	0.4870	0.69
36" (900 mm)	36.20" (919 mm)	42.46" (1073 mm)	0.050" (1.27 mm)	22 psi (150 kN/m^2)	361.20 lbs. (163.98 kg.)	0.361	0.5500	0.46
42" (1050 mm)	41.50" (1054 mm)	46.75" (1187 mm)	0.050" (1.27 mm)	19 psi (140 kN/m^2)	530.00 lbs. (240.62 kg.)	0.420	0.7400	1.25
48" (1200 mm)	47.55" (1208 mm)	52.70" (1339 mm)	0.050" (1.27 mm)	17 psi (120 kN/m^2)	640.00 lbs. (290.56 kg.)	0.420	0.7400	1.25

Date: March 1, 1996

Attachment E 2/2

SOURCE ADS 1996

Calculation Package B
Veneer Stability

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Basic Remediation Company

Project: BRC CAMU **Project/Proposal #:** SC0313 **Task #:** 01

Title of Computations: Veneer Stability of Geosynthetic – Soil Lined Sideslope

Computations By: Meghan Lithgow — 11/6/06
SIGNATURE DATE
Meghan Lithgow, Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer): — 11/4/06
SIGNATURE DATE
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Checked By: — 11/4/06
SIGNATURE DATE
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Computations Backchecked By (Originator): Meghan Lithgow — 11/6/06
SIGNATURE DATE
Meghan Lithgow, Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate): — 11/4/06
SIGNATURE DATE
Gregory T. Corcoran, PE / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Meghan Lithgow Date: 09 / 25 / 06 Reviewed by: GTC Date: 11 / 4 / 06
MM DD YY MM DD YY

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 1.2

SLOPE STABILITY EVALUATION
VENEER STABILITY OF GEOSYNTHETIC-SOIL LINED SIDESLOPES
COVER LINER SYSTEM
BRC CAMU

OBJECTIVE

To evaluate the tension developed within the geosynthetic-soil layered sideslopes of the cover liner system of the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada.

SUMMARY OF RESULTS

The calculations suggest that a minimum geosynthetic interface friction angle of 20 degrees is required to prevent the development of tension in the geosynthetic components of the side slope liner final cover system. A review of the literature indicates that achieving a friction angle of 20 degrees is obtainable. The critical interface of the geosynthetics is likely the internal friction angle of the GCL.

METHOD OF ANALYSIS

The stability analysis of the geosynthetic-soil layered systems was carried out using the approach outlined by Koerner and Soong [1998] (Attachment F). This approach calculates the driving force of an active soil wedge along a geosynthetic-soil layered sideslope and compares it to the resisting force of the complementary passive soil wedge to evaluate the overall factor of safety against failure. The method presented by Koerner and Soong [1998] allows for the consideration of a uniform depth soil layer and the influence of dynamic equipment loading.

SIDESLOPE LINER SYSTEM

The sideslope liner system consists of, from top to bottom:

- 2 ft (min) cover soil;
- Double-sided geocomposite consisting of geonet sandwiched between two non-woven geotextiles;

Written by: Meghan Lithgow Date: 09 / 25 / 06 (MM DD YY) Reviewed by: GTC Date: 11 / 14 / 06 (MM DD YY)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 1.2

- 60-mil (1.5-mm) textured high density polyethylene (HDPE) geomembrane;
- Geosynthetic clay liner (GCL) consisting of bentonite sandwiched between a top nonwoven geotextile and a bottom woven geotextile needle-punched together
- Prepared subgrade.

The sideslope inclination is 3.0H:1.0V. The maximum height of a side slope is 47 vertical feet, located in the southwest corner of the South Mesa.

MATERIAL SHEAR STRENGTHS

Cover Soil Material:

The soil materials to be used overlying the side slope liner system will be native materials such as silty sands (SM) for the operations layer. Converse reported a maximum dry density of 132 pcf and a optimum water content of 8.~~7~~2 percent for materials at the site (Attachment A). Therefore, assuming 95% relative compaction, the dry density in the field is approximately 125 pcf. Adding the weight of water, the unit weight is approximately 136 pcf. The cover soil material is characterized by a minimum internal angle of friction of 32 degrees in accordance with NAVFAC and the expected compaction criteria (see *review of reported interface strengths*).

For this analysis, a shear strength of 32 degrees, with 500 psf cohesion, and a unit weight of 136 pcf will be used for the analyses performed herein.

Geosynthetic Interface:

The geosynthetic interface friction angle was varied to evaluate the minimum allowable value to obtain no tension. A literature review was performed to evaluate if the calculated minimum allowable interface friction value is achievable. Adhesion is neglected for the analyses herein.

REVIEW OF REPORTED INTERFACE STRENGTHS IN THE LITERATURE

The following values for the interface friction between the geosynthetic and soil components of the liner system represent values reported in the literature:

Native Material (SM)	32	NAVFAC[1,2], 500 psf cohesion (Attachment B)
Cover Soil to Geocomposite (NWGT side)	29	Koerner[3] (Attachment C)
Geocomposite to Textured HDPE	28	(Professional Experience)
Textured HDPE to Hydrated GCL	20	Bentomat[4] (Attachment D)
Hydrated GCL to Subgrade	22	Bentomat[5] (Attachment D)

1. NAVFAC (1982) lists typical shear strength values for various soils based on 100 percent standard Proctor compaction. Actual construction materials would likely be placed at 90 percent of the modified Proctor compaction. To be conservative, a value of phi = 32 degree was used.

2. Value of friction angle for a silty sand designated under the USCS classification system as a SM. This value is a conservative friction angle estimate as compared to an SC soil, which would yield a greater friction angle.

3. Koerner (1995) suggests that an efficiency of greater than 90 percent for the interface of nonwoven, needle-punched geotextiles to various soils can be achieved. Efficiency values are based on the relationship, Efficiency = tan(interface friction angle)/tan(soil friction angle). The interface friction angle presented herein was calculated using a 90 percent efficiency and the estimated soil friction angle. Adhesion is neglected.

4. Values of residual friction angle reported for interface shear strength between 60 mil textured HDPE geomembrane and woven geotextile side of CETCO Bentomat GCL under hydrated conditions at a shearing rate of 0.04 in/min. Maximum displacement was not reported. Normal stresses of 1.4, 3.5, and 7.0 psi yielded a friction angle of 20.4 degrees.

5. Values of residual friction angle reported for interface shear strength between 60 mil textured HDPE geomembrane and woven geotextile side of CETCO Bentomat GCL under hydrated conditions at a shearing rate of 0.04 in/min. Maximum displacement was not reported. Normal stresses of 1.4, 3.5, and 7.0 psi yielded a friction angle of 22.1 degrees.

DESIGN CRITERION

For the geosynthetic-soil lined side slopes of BRC CAMU final cover system it was desired to evaluate the combination of operations layer soil height, inclination, and equipment loading which would introduce no geosynthetic tension. Subsequently, zero geosynthetic tension was established as the design criterion for veneer stability of the geosynthetic-soil lined side slopes.

The geosynthetic-soil lined side slopes are considered permanent slopes because they will not eventually be buttressed by placement of waste. In consideration of the significance of no tension in the geosynthetic liner system and consistent with current practice, GeoSyntec will adopt a factor of safety (FS) equal to or greater than 1.5 for slope stability (CASE I, completed cover system). However, the construction of the cover system involves a very short period where the cover soil will be placed by construction equipment moving parallel to the slope, which imparts additional forces parallel to the liner system and may induce geosynthetic tension. Therefore, a factor of safety of 1.1 is established as the stability criterion for this case (CASE II, during construction).

ANALYSIS

CASE I: Veneer Stability on Side Slope

According to the Koerner and Soong approach, a soil veneer on a side slope is stable when the resultant driving force on the passive wedge (E_P) is equal to the resultant resistant force on the active wedge (E_A) (Figure 1). The following equations represent the resultant resistance and active forces, respectively:

$$E_A = \frac{FS(W_A - N_A \cos\beta) - (N_A \tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$ (Attachment E, 2 of 6)

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi}$$ (Attachment E, 2 of 6)

The variables will be defined in a subsequent section.

By setting E_A=E_P, the resulting equation may be arranged in the form of the quadratic equation $ax^2+bx+c=0$. Considering FS as the variable of interest, the resulting equation is as follows:

$$a(FS)^2 + b(FS) + c = 0$$ (Attachment E, 2 of 6)

The factor of safety may be obtained from the solution of the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ (Attachment E, 2 of 6)

Where the constants are defined, as in Attachment E, 6 of 6:

$$a = \left(W_A - N_A \cos\beta\right)\cos\beta \tag{1}$$

$$b = -\left[\left(W_A - N_A \cos\beta\right)\sin\beta\tan\phi + \left(N_A \tan\delta + C_a\right)\sin\beta\cos\beta + \sin\beta\left(C + W_P \tan\phi\right)\right] \tag{2}$$

$$c = \left(N_A \tan\delta + C_a\right)\sin^2\beta\tan\phi \tag{3}$$

In which the variables are indicated on Figure 1 and defined as follows:

$$C = \frac{c \cdot h}{\sin\beta} \tag{4}$$ (Attachment E, 2 of 6)

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) \tag{5}$$ (Attachment E, 2 of 6)

$$W_A = \gamma \cdot h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) \tag{6}$$ (Attachment E, 2 of 6)

$$N_A = W_A \cos\beta \tag{7}$$ (Attachment E, 2 of 6)

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta} \tag{8}$$ (Attachment E, 2 of 6)

β = soil slope angle beneath the geomembrane, **18.4°** for 3H:1V slope
δ = minimum interface friction angle of side slope liner system, **20°**
ϕ = friction angle of cover soil, **32°**
γ = unit weight of the cover soil, **136 pcf**
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the cover soil, **500 psf**
C_a = adhesive force between the cover soil of the active wedge and the geomembrane
c_a = adhesion between the cover soil of the active wedge and the geomembrane, **0 psf**
h = thickness of the operations layer, **2 feet**
L = length of slope measured along the geomembrane beneath cover soil, **148.6 feet**
N_A = effective force normal to the failure plane of the active wedge
W_A = total weight of the active wedge
W_P = total weight of the passive wedge

Substituting the variables and solving equations (4)-(8), above:

$$C = \frac{c \cdot h}{\sin\beta} = \frac{500 \cdot 2}{\sin(18.4)} = \mathbf{3163\ lbs/ft} \qquad (4)$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = 0\left(148.7 - \frac{2}{\sin 21.8}\right) = \mathbf{0\ lbs/ft} \qquad (5)$$

$$W_A = \gamma \cdot h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) = 136 \cdot 2^2\left(\frac{148.7}{2} - \frac{1}{\sin 18.4} - \frac{\tan 18.4}{2}\right) = \mathbf{38{,}627\ lbs/ft} \qquad (6)$$

$$N_A = W_A \cos\beta = 38{,}627 \cdot \cos 18.4. = \mathbf{36{,}646\ lbs/ft} \qquad (7)$$

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta} = \frac{136 \cdot 2^2}{\sin 2 \cdot 18.4} = \mathbf{907\ lbs/ft} \qquad (8)$$

Next, substituting the solutions to equations (4)-(8) into equations (1)-(3):

$$a = (38{,}627 - 36{,}646 \cdot \cos 18.4)\cos 18.4 = \mathbf{3{,}663\ lbs/ft} \qquad (1)$$

$$\begin{aligned} b = &-[(38{,}627 - 36{,}646 \cos 18.4) \sin 18.4 \tan 32 \\ &+ (36{,}646 \tan 20 + 0) \sin 18.4 \cos 18.4 \\ &+ \sin 18.4\,(3163 + 907 \tan 32) = \mathbf{-5942\ lbs/ft} \end{aligned} \qquad (2)$$

$$c = (36{,}646 \cdot \tan 20 + 0)\sin^2 18.4 \cdot \tan 32 = \mathbf{833\ lbs/ft} \qquad (3)$$

Finally, inserting the solutions to (1), (2) and (3) and solving the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} = \frac{-(-5942) + \sqrt{(-5942)^2 - 4(3663)(833)}}{2(3663)} = \mathbf{1.47}$$

Therefore, the factor of safety for the veneer stability of the operations layer on a sideslope composite liner system is 1.47. This factor of safety satisfies the stability criterion for permanent slopes of 1.5, as previously described.

CASE II: Veneer Stability during Dynamic Equipment Loading

In consideration of the loading due to soil placement equipment moving on the slope, the following equations represent the resultant resistant and active forces, respectively:

$$E_A = \frac{FS[(W_A + W_e)\sin\beta + F_e]}{FS} - \frac{[(N_e + N_A)\tan\delta + C_a]}{FS}$$ (Attachment E, 5 of 6)

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi}$$ (Attachment E, 6 of 6)

Where the constants are defined, as in Attachment A, 6 of 6:

$$a = [(W_A + W_e)\sin\beta + F_e]\cos\beta \tag{9}$$

$$b = -\{[(N_e + N_A)\tan\delta + C_a]\cos\beta + [(W_A + W_e)\sin\beta + F_e]\sin\beta\tan\phi + (C + W_P\tan\phi)\} \tag{10}$$

$$c = [(N_e + N_A)\tan\delta + C_a]\sin\beta\tan\phi \tag{11}$$

In which the variables are indicated on Figure 1, defined previously and as follows:

$$q = \frac{W_b}{(2 \cdot w \cdot b)}$$ (12) (Attachment E, 3 of 6)

$$W_e = qwI$$ (13) (Attachment E, 3 of 6)

$$F_e = W_e\left(\frac{a}{g}\right)$$ (14) (Attachment E, 5 of 6)

$$N_e = W_e\cos\beta$$ (15) (Attachment E, 5 of 6)

- a = acceleration of placement equipment (12km/hr, 2secs), **0.19 g** (Attachment E, 5of 6)
- b = width of placement equipment track, **22 inches (D6R XL)** (Attachment F)
- F_e = dynamic equipment loading force per unit parallel to slope at geomembrane interface
- g = acceleration due to gravity, **32.2 ft/s²**
- I = influence factor at the geomembrane interface, **0.9** (Attachment E, 4 of 6)
- N_e = effective equipment force normal to the failure plane of the active wedge
- w = length of placement equipment track, **111 inches (D6R XL)** (Attachment F)
- W_b = actual weight of the placement equipment, **42,300 lbs (D6R XL)** (Attachment F)
- W_e = equivalent equipment force per unit width at the geomembrane interface

Substituting the variables and solving equations (4)-(8) and (12)-(15), above:

$$C = \frac{c \cdot h}{\sin\beta} = \frac{500 \cdot 2}{\sin(18.4)} = \mathbf{3163\ lbs/ft} \tag{4}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = 0\left(148.7 - \frac{2}{\sin 18.4}\right) = \mathbf{0\ lbs/ft} \qquad (5)$$

$$W_A = \gamma \cdot h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) = 136 \cdot 2^2\left(\frac{148.7}{2} - \frac{1}{\sin 18.4} - \frac{\tan 18.4}{2}\right) = \mathbf{38{,}627\ lbs/ft} \qquad (6)$$

$$N_A = W_A \cos\beta = 38{,}627 \cdot \cos 18.4 = \mathbf{36{,}646\ lbs/ft} \qquad (7)$$

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta} = \frac{136 \cdot 2^2}{\sin 2 \cdot 18.4} = \mathbf{907\ lbs/ft} \qquad (8)$$

$$q = \frac{W_b}{(2 \cdot w \cdot b)} = \frac{42{,}300}{(2 \cdot 111 \cdot 22)} = 8.66\ \text{psi} = \mathbf{1247\ psf} \qquad (12)$$

$$W_e = qwl = 8.66 \cdot 111 \cdot 0.9 = 865\ \text{lbs/in} = \mathbf{10{,}381\ lbs/ft} \qquad (13)$$

$$F_e = W_e\left(\frac{a}{g}\right) = 10{,}381\left(\frac{0.19 \cdot 32.2}{32.2}\right) = \mathbf{1{,}972\ lbs/ft} \qquad (14)$$

$$N_e = W_e \cos\beta = 10{,}381 \cdot \cos 18.4 = \mathbf{9{,}482\ lbs/ft} \qquad (15)$$

Next, substituting the solutions to equations (4)-(12) and (12)-(15) into equations (9)-(11):

$$a = [(38627 + 10381)\sin 18.4 + 1972]\cos 18.4 = \mathbf{16{,}571\ lbs/ft} \qquad (9)$$

$$\begin{aligned} b = -\{&[9482 + 36646)\tan 20 + 0]\cos 18.4 \\ &+ [(38627 + 10381)\sin 18.4 + 1972]\sin 18.4 \tan 32 \\ &+ (3163 + 907 \tan 32)\} = \mathbf{-23{,}108\ lbs/ft} \end{aligned} \qquad (10)$$

$$c = [(9482 + 36646)\tan 20 + 0]\sin 18.4 \tan 32 = \mathbf{3{,}317\ lbs/ft} \qquad (11)$$

Finally, inserting the solutions to (9), (10) and (11) and solving the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} = \frac{-(-23108) + \sqrt{(-23108)^2 - 4(16571)(3317)}}{2(16571)} = \mathbf{1.23}$$

Therefore, the factor of safety for the stability of the operations layer on a sideslope composite liner system considering additional dynamic loading due to placement equipment is 1.23. This factor of safety satisfies the stability criterion for interim slopes of 1.1, as previously described.

RESULTS AND CONCLUSIONS

The results suggest that the proposed final cover system satisfies the design criteria of no geosynthetic tension development:

Stability Condition	Soil Cover Layer Inclination (H:V)	Equipment Loading	Tension FS
Case I	3.0H:1V	NONE	1.47
Case II	3.0H:1V	CAT-D6H	1.23

Live Load Case (assuming placement of operations layer with a bulldozer no larger than Caterpillar D6R XL dozer in terms of operating weight and ground pressure), the side slope liner system will not be placed into tension.

Results of veneer stability analyses presented herein indicate that an apparent internal friction or interface friction angle (residual) of 20 degrees for any component of the composite liner system is the minimum allowable value providing for a static factor of safety that satisfies that design criteria of no tension in the geosynthetic liner system.

Based on the analyses herein, results of interface shear tests on the actual materials proposed for use in the composite liner system must indicate that the weakest apparent residual friction angle of the composite liner is equal to or greater than 20 degrees.

The results suggest that the following conditions satisfy the design criteria of no geosynthetic tension development:

- 3H:1 V side slopes;
- 2 feet thickness of operations layer soil/leachate collection layer on side slopes, placed to a vertical height of 47 feet;
- 136 pcf unit weight of operations layer soil;
- Minimum shear properties of operations layer soil - 500 psf cohesion and 32° friction angle;
- Minimum shear strength of composite liner system – 0 psf adhesion and 20° friction angle;
- Caterpillar D6R XL used as operations layer placement equipment;
- Acceptable factor of safety for interim slope stability is 1.3 (CASE I); and

- Acceptable factor of safety for dynamically loaded interim slopes is 1.1 (CASE II).

REFERENCES

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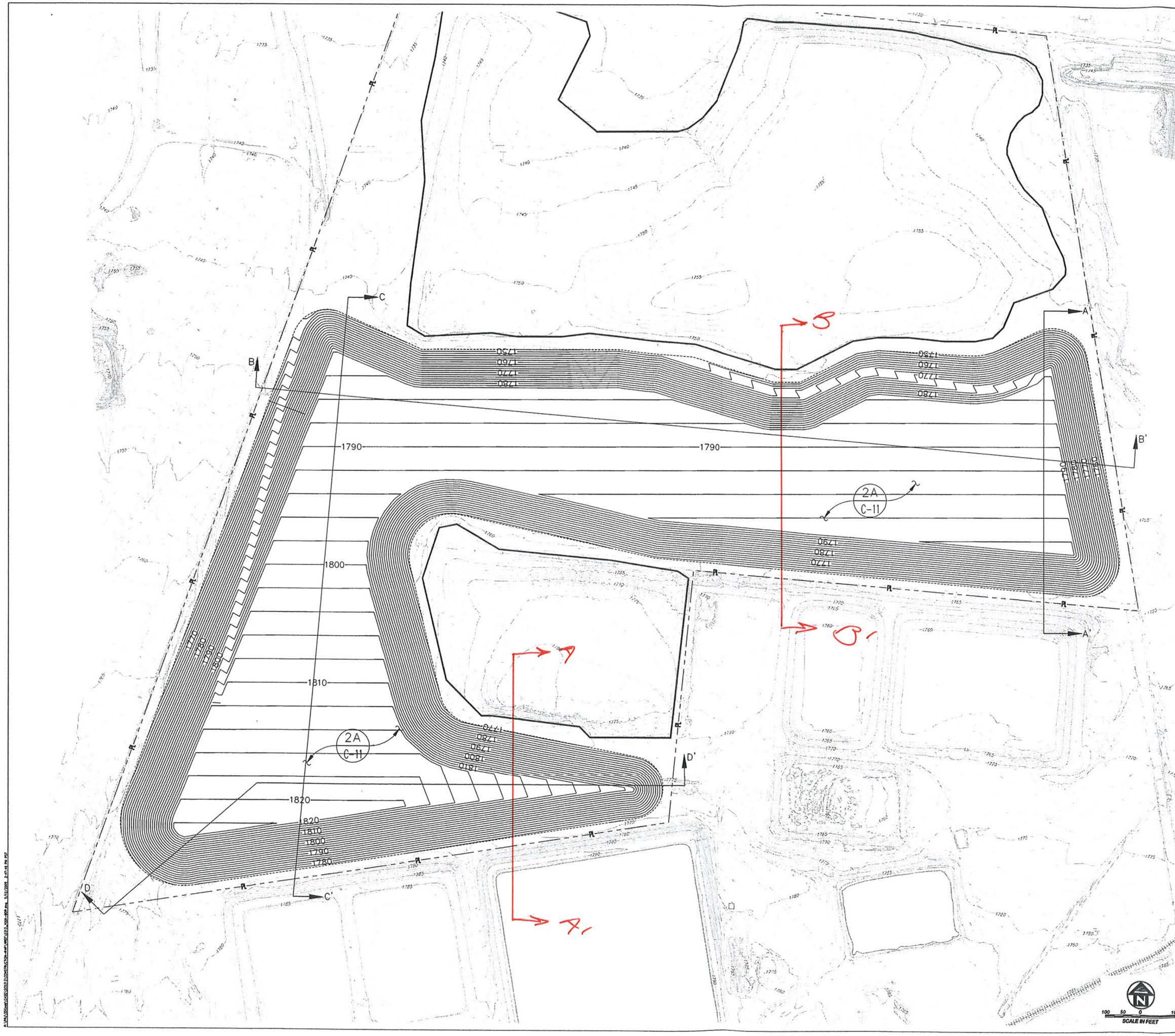
Koerner, R. M. (1997), "Designing with Geosynthetics," Simon and Schuster / A Viacom Company, Upper Saddle New Jersey, 07458, 1998 *(Attachment C)*

NAVFAC (1982), "Foundations and Earth Structures, Design Manual - 7.2," Department of Navy, Naval Facilities Engineering Command, May 1982 *(Attachment B)*

Bentomat Attachment! (A*ttachment D)*



LEGEND

—PL— — — ——	BRC LANDFILL SITE BOUNDARY
	EXISTING LIMITS OF FORMER BMI LANDFILL
	EXISTING SURFACE WATER FLOW PATH
	EXISTING MINOR ELEVATION CONTOUR
	EXISTING MAJOR ELEVATION CONTOUR
	PROPOSED MINOR EXCAVATION ELEVATION CONTOUR
——1770——	PROPOSED MAJOR EXCAVATION ELEVATION CONTOUR
- - - - - - -	PROPOSED LIMITS OF REFUSE

NOTES:

1. FOR GENERAL NOTES, LEGEND AND ABBREVIATIONS, SEE DWG C-2
2. FOR SITE PREPARATION PLAN SEE DWG C-6.
3. FOR CONSTRUCTION GRADING PLAN SEE DWG C-7.
4. FOR EROSION CONTROL AND STORM DRAIN PLAN SEE DWG C-8.
5. FOR CROSSECTIONS SEE DWG C-10.
6. FOR FINAL COVER AND LINER DETAILS AND SECTIONS SEE DWG C-11
7. FOR FENCE PLAN SEE DWG C-12.

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PROJECT: BRC CAMU

HENDERSON/CLARK COUNTY NEVADA

TITLE: FINAL COVER GRADING PLAN

PERMITTING CONSTRUCTION PLAN

DATE: SEPTEMBER 2006	CHECKED BY: G.T.C	SCALE: 1"=100'	FIGURE NO
DESIGN BY: TAW	REVIEWED BY: (PROJ MGR) G.T.C	JOB NO.: SC0313-01-04	C9
DRAWN BY: T.L.Z.	DOCUMENT NO:	FILE NO:	

Figure 1

Mail B All cc
GTC

shown on Drawing Nos. A-49 through A-56, entitled *Consolidation Test* and are summarized on the following table:

Exploration Location	Depth (feet)	Soil Description	Dry Unit Weight, pcf	Moisture Content, %	Hydrocollapse (percent)*
B-1	29-30	Silty sand with gravel	105	6	3.2
B-8	39-40	Sandy lean clay	57.4	64	0.4
B-8	49-50	Sandy lean clay	69.5	51.1	-0.6
B-10	54-54.5	Sandy lean clay	60.7	67.7	-0.6
B-101	39-40	Sandy lean clay	65.8	45	-0.2
B-101	59-60	Sandy lean clay	73.2	38.3	-0.6
B-102	49-50	Sandy lean clay	67.3	48.7	-0.5
B-105	34-35	Well graded sand with silt and gravel	101	5	0.1

NA: Not available

* A negative sign indicates swell occurred upon inundation with water instead of collapse.

Laboratory Maximum Density

Laboratory maximum density tests were performed on selected samples of the granular soils. The purpose of the test was to define the compaction characteristics of these soils, and to aid in estimating soil shrinkage. The laboratory maximum density test was performed in general accordance with the ASTM D1557 test method. This test procedure uses 25 blow of a 10-pound hammer falling a height of 18 inches on each of five layers of soil in a 1/30 or 1/13 cubic foot cylinder. The test results are presented on Drawing Nos. A-57 through A-61 and in the following table:

Exploration Location	Depth (Feet)	Soil Description	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (percent) of dry weight)
B-1	20-25	Silty sand with gravel	129.4	8.2
B-5	20-25	Silty sand with gravel	132.1	8.2
B-12	10-15	Silty sand with gravel	129.7	7.9
B-101	5-10	Silty sand with gravel	130.6	8.7
B-105	20-25	Well graded sand with silt and gravel	131.8	7.5

assume
$\gamma_d = 130$ pcf
$w\% = 8\%$

Source: Converse (1999)

Attachment A 1/1

TABLE 1

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi) — Percent of Original Height	Typical Value of Compression: At 3.6 tsf (50 psi) — Percent of Original Height	Typical Strength Characteristics: Cohesion (as compacted) psf	Typical Strength Characteristics: Cohesion (saturated) psf	Typical Strength Characteristics: ϕ (Effective Stress Envelope Degrees)	Typical Strength Characteristics: Tan ϕ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
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	$5 \times >10^{-5}$	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	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	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	$5 \times >10^{-7}$	.....	
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	$5 \times >10^{-7}$	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

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical stength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) Indicates that typical property is greater than the value shown.
   (..) Indicates insufficient data available for an estimate.

Source- NAVFAC (1982)

Attachment B

1/1

Table 5.7 Friction values and efficiencies (in parentheses) for (a) soil-to-geomembrane, (b) geomembrane-to-geotextile, and (c) soil-to-geotextile combinations*

(a) Soil-to-Geomembrane Friction Angles

	Soil Types		
Geomembrane	*Concrete Sand* ($\phi = 30°$)	*Ottawa Sand* ($\phi = 28°$)	*Micha Schist Sand* ($\phi = 26°$)
EPDM-R	24° (0.77)	20° (0.68)	24° (0.91)
PVC			
Rough	27° (0.88)	—	25° (0.96)
Smooth	25° (0.81)	—	21° (0.79)
CSPE-R	25° (0.81)	21° (0.72)	23° (0.87)
HDPE -Smooth	18° (0.56)	18° (0.61)	17° (0.63)

(b) Geomembrane-to-Geotextile Friction Angles

	Geomembrane				
		PVC			
Geotextile	*EPDM-R*	*Rough*	*Smooth*	*CSPE-R*	Smooth *HDPE*
Nonwoven, needle punched	23°	23°	21°	15°	8°
Nonwoven, heat bonded	18°	20°	18°	21°	11°
Woven, monofilament	17°	11°	10°	9°	6°
Woven, slit film	21°	28°	24°	13°	10°

(c) Soil-to-Geotextile Friction Angles

	Soil Types		
Geotextile	*Concrete Sand* ($\phi = 30°$)	*Ottawa Sand* ($\phi = 28°$)	*Mica Schist Sand* ($\phi = 26°$)
Nonwoven, needle punched	30° (1.00)	26° (0.92)	25° (0.96)
Nonwoven, heat bonded	26° (0.84)	—	—
Woven, monofilament	26° (0.84)	—	—
Woven, slit film	24° (0.77)	24° (0.84)	23° (0.87)

*Efficiency values in parentheses are based on the relationship $E = (\tan\delta)/(\tan\phi)$.

Source: After Martin et al. [14].

The frictional behavior of geomembranes placed erable importance in the composite liners of waste land are for the clay to have a hydraulic conductivity equa ft./min. (1×10^{-7} cm/sec.) and for the geomembrane t clay. While an indication of the shear strength param (e.g., reference 15), the data are so sensitive to the var site-specific and material-specific tests should always b literature values should never be used for final design

5.1.3.9 Geomembrane Anchorage In certain proble might be sandwiched between two materials and the force. The termination of a geomembrane liner within an anchor trench is such a situation. To simulate this behavior in a laboratory environment, one can use an 8.0-in. (200-mm)-wide geomembrane sandwiched between back-to-back channels.

Judge - Koerner (1997)

Attachment C 1/1

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ SOIL

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GSC	03-09-95	NW/ Soaked GC Soil	7.5 - 15 - 30	Hydrated	0.04	33	31	385	
CETCO	08-29-95	W/Silty Sand	1 - 2 - 3	Hydrated	0.04	32.3	ND	127	
GSC	03-12-96	NW/ Soaked Soil	2 - 4 - 6	Hydrated	0.04	29	29	70	
STS	05-30-96	NW/Silty Sandy Gravel	0.7 – 2 – 3.5	Hydrated	0.04	28	ND	222	
CETCO	07-08-96	NW/ Sandy Silty Gravel	0.7 - 2.0 - 3.5	Hydrated	0.04	40.4	ND	238	
	07-31-96	W/ Sandy Silty Gravel	0.7 - 2.0 - 3.5	Hydrated	0.04	37.8	ND	194	
AGP	11-08-96	NW/ SC Soil	1.4 - 3.5 - 7.0	Hydrated	0.04	27.8	ND	186	
		W/ SC Soil	1.4 - 3.5 - 7.0	Hydrated	0.04	23.5	22.1	187	
GSC	01-08-97	NW/ Soaked Soils	14 - 40 - 70	Hydrated	0.04	18	18	155	
TRI	09-23-97	NW/ Subbaase Soil	3.5 – 7 – 14	Hydrated	0.001	24.0	20.7	4	
		NW/ Subbase Soil	.35 - .7 – 1.4	Hydrated	0.001	35.5	37.2	88	
GSC	01-22-98	CLS/ SM soil	5 - 25 - 45	Hydrated	0.04	17	13	70	
CETCO	02-10-98	CLS/ SC Soil	0.5 – 1 – 1.5	Hydrated	0.04	9.5	ND	46	
VE	03-03-98	W/ Cover soil	1 - 2.1 – 4.9	Hydrated	0.04	37	33	50	
CETCO	08-11-98	W/ SC soil	2.1 – 4.2 – 6.3	Hydrated	0.04	33.6	35.2	183	
		NW/ Non-Pag Quartz Monzonite Waste	2.1 – 4.2 – 6.3	Hydrated	0.04	39.8	37.5	190	
TRI	09-22-98	NW/ soil	16 – 32 – 73	Hydrated	0.04	17.5	16.8	505	Residual @ 3"
CETCO	11-3-98	CLS/SM soil	0.5 – 1 – 1.5	Hydrated	0.04	27	27	80	
		CLS/SP soil	0.5 – 1 – 1.5	Hydrated	0.04	15.2	15.2	55	
GSC	6-24-00	CLS/drainage soil	139	Hydrated	0.04	24	0	13	Angle w/ origin

ATTACHMENT D 1/2

SUMMARY OF BENTOMAT DIRECT SHEAR TEST DATA
INTERFACE W/ GEOMEMBRANE

Lab[1]	Report Date	Interface Tested[2]	Normal Stresses (psi)	Bentomat Moisture[3]	Shear Rate (in/min)	Peak Friction Angle (deg)	Residual Friction Angle (deg)[4]	Apparent Peak Cohesion (psf)	Comments
GA	09-04-92	W/60-mil sm. HDPE	0.5 - 1 - 2 - 4 - 10	Hydrated	0.02	8	7	0	
		W/60-mil text. HDPE	0.5 - 1 - 2 - 4 - 10	Hydrated	0.02	28	28 / 6	29	bi-modal residual
GSC	12-08-94	W/60-mil text. HDPE	7.5 - 15 - 30	Hydrated	0.04	18	17	175	Diff. membrane
		W/60-mil text. HDPE	7.5 - 15 - 30	Hydrated	0.04	16	12	345	manufacturers
GSC	12-16-94	W/60-mil text. HDPE	1 -3 - 6 - (15)	Hydrated	0.04	25 (19)	10	100	(lower d at 15 psi)
AGP	07-12-95	W/80-mil Text. HDPE	14 - 28 - 69 - 104	Hydrated	0.04	18	8	192	
		W/80-mil Text. HDPE	14 - 28 - 69 - 104	Dry	0.04	30	14	0	
AGP	11-30-95	W/ Text. HDPE	10 - 26 - 38	Dry	0.08	30.2	13.3	0	
GSC	03-12-96	W/ 30 mil PVC	2 - 4 - 6	Hydrated	0.04	17	17	24	
GSC	05-29-96	NW/80mil Text. HDPE	140	Hydrated	0.04	19	5	475	Consol 24 hrs @140 psi
AGP	11-08-96	NW/60mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	34.8	22.7	149	
		W/60 mil Text. HDPE	1.4 - 3.5 - 7.0	Hydrated	0.04	28.8	20.4 *	83	
GSC	01-08-97	NW/60mil Text. HDPE	14 - 40 - 70	Hydrated	0.04	17	9	255	
TRI	4-15-97	NW/60mil Text. HDPE	14 – 28 – 56	Hydrated	0.04	21.9	10.8	722	
Emcon	6-16-97	NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	32.0	18.5	111	
		NW/40mil Text. LLPE	1.0 – 2.0 – 3.5	Hydrated	0.04	37.5	27.1	118	
TRI	10-15-97	NW/ 60mil Text.HDPE	3.5 – 7 – 14	Hydrated	0.04	20.3	18.6	278	
		NW/ 60mil Text.HDPE	.35 - .7 – 1.4	Hydrated	0.04	36.6	25.8	2	
TRI	12-01-97	NW/60mil Text. HDPE	0.35 – 0.7 – 1.4	Hydrated	0.001	25.6	23.3	54	Residual @ 4"
		NW/60mil Text. HDPE	3.5 – 7 – 14	Hydrated	0.001	23.2	17.8	85	"
Emcon	04-06-98	NW/60mil Text. HDPE	14 – 28 – 70	Hydrated	0.04	24.1	12.2	290	

Gzshidos (3)
GTC 5/11/01

- The issue of appropriate normal stress is greatly complicated if gas pressures are generated in the underlying waste. These gas pressures will counteract some (or all) of the gravitational stress of the cover soil. The resulting shear strength, and subsequent stability, can be significantly decreased. See Liu et al (1997) for insight into this possibility.
- Shear rates necessary to attain drained conditions (if this is the desired situation) are extremely slow, requiring long testing times.
- Deformations necessary to attain residual strengths require large relative movement of the two respective halves of the shear box. So as not to travel over the edges of the opposing shear box sections, devices should have the lower shear box significantly longer than 300 mm. However, with a lower shear box longer than the upper traveling section, new surface is constantly being added to the shearing plane. This influence is not clear in the material's response or in the subsequent behavior.
- The attainment of a true residual strength is difficult to achieve. ASTM D5321 states that one should "run the test until the applied shear force remains constant with increasing displacement". Many commercially available shear boxes have insufficient travel to reach this condition.
- The ring torsion shearing apparatus is an alternative device to determine true residual strength values, but is not without its own problems. Some outstanding issues are the small specimen size, nonuniform shear rates along the width of the specimen, anisotropic shearing with some geosynthetics and no standardized testing protocol. See Stark and Poeppel (1994) for information and data using this alternative test method.

2.3 Various Types of Loadings

There are a large variety of slope stability problems that may be encountered in analyzing and/or designing final covers of engineered landfills, abandoned dumps and remediation sites as well as leachate collection soils covering geomembranes beneath the waste. Perhaps the most common situation is a uniformly thick cover soil on a geomembrane placed over the soil subgrade at a given and constant slope angle. This "standard" problem will be analyzed in the next section. A variation of this problem will include equipment loads used during placement of cover soil on the geomembrane. This problem will be solved with equipment moving up the slope and then moving down the slope.

Unfortunately, cover soil slides have occurred and it is felt that the majority of the slides have been associated with seepage forces. Indeed, drainage above a geomembrane (or other barrier material) in the cover soil cross section must be accommodated to avoid the possibility of seepage forces. A section will be devoted to this class of slope stability problems.

Lastly, the possibility of seismic forces exists in earthquake prone locations. If an earthquake occurs in the vicinity of an engineered landfill, abandoned dump or remediation site, the seismic wave travels through the solid waste mass reaching the upper surface of the cover. It then decouples from the cover soil materials, producing a horizontal force which must be appropriately analyzed. A section will be devoted to the seismic aspects of cover soil slope analysis as well.

All of the above actions are destabilizing forces tending to cause slope instability. Fortunately, there are a number of actions that can be taken to increase the stability of slopes.

Other than geometrically redesigning the slope with a flatter slope angle or shorter slope length, a designer can add soil mass at the toe of the slope thereby enhancing stability. Both toe berms and tapered soil covers are available options and will be analyzed accordingly. Alternatively, the designer can always use geogrids or high strength geotextiles within the cover soil acting as reinforcement materials. This technique is usually referred to as veneer reinforcement. Cases of both intentional and nonintentional veneer reinforcement will be presented.

Thus it is seen that a number of strategies influence slope stability. Each will be described in the sections to follow. First, the basic gravitational problem will be presented followed by those additional loading situations which tend to decrease slope stability. Second, various actions that can be taken by the designer to increase slope stability will be presented. The summary will contrast the FS-values obtained in the similarly crafted numeric examples.

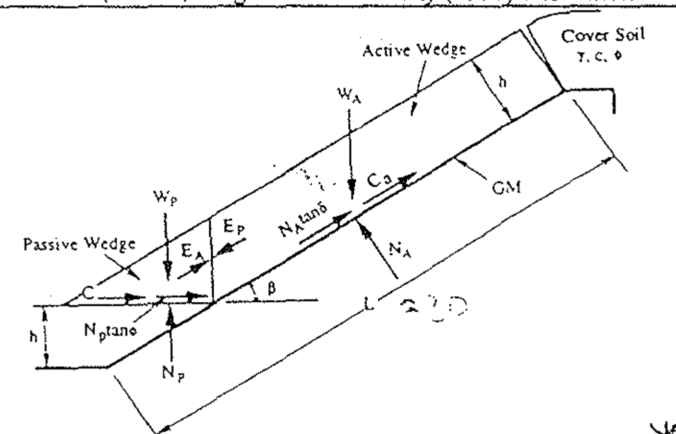
3 SITUATIONS CAUSING DESTABILIZATION OF SLOPES

This section treats the standard veneer slope stability problem and then superimposes upon it a number of situations, all of which tend to destabilize slopes. Included are gravitational, construction equipment, seepage and seismic forces. Each will be illustrated by a design graph and a numeric example.

3.1 Cover Soil (Gravitational) Forces

Figure 3 illustrates the common situation of a *finite* length, uniformly thick cover soil placed over a liner material at a slope angle "β". It includes a passive wedge at the toe and has a tension crack of the crest. The analysis that follows is after Koerner and Hwu (1991), but comparable analyses are available from Giroud and Beech (1989), McKelvey and Deutsch (1991), Ling and Leshchinsky (1997) and others.

*



*

Figure 3. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil.

Attachment E 1/6

5/11/01 2
CTC 5/11/01

..e symbols used in Figure 3 are defined below.

W_A = total weight of the active wedge
W_P = total weight of the passive wedge
N_A = effective force normal to the failure plane of the active wedge
N_P = effective force normal to the failure plane of the passive wedge
γ = unit weight of the cover soil
h = thickness of the cover soil
L = length of slope measured along the geomembrane
β = soil slope angle beneath the geomembrane
ϕ = friction angle of the cover soil
δ = interface friction angle between cover soil and geomembrane
C_a = adhesive force between cover soil of the active wedge and the geomembrane
c_a = adhesion between cover soil of the active wedge and the geomembrane
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the cover soil
E_A = interwedge force acting on the active wedge from the passive wedge
E_P = interwedge force acting on the passive wedge from the active wedge
FS = factor of safety against cover soil sliding on the geomembrane

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge,

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) \quad (3)$$

$$N_A = W_A \cos\beta \quad (4)$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) \quad (5)$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin\beta = W_A - N_A \cos\beta - \frac{N_A \tan\delta + C_a}{FS}\sin\beta \quad (6)$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} \quad (7)$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} \quad (8)$$

$$N_P = W_P + E_P \sin\beta \quad (9)$$

$$C = \frac{(c)(h)}{\sin\beta} \quad (10)$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cos\beta = \frac{C + N_P \tan\phi}{FS} \quad (11)$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \quad (12)$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$ which in our case, using FS-values, is:

$$a(FS)^2 + b(FS) + c = 0 \quad (13)$$

where

$$a = (W_A - N_A\cos\beta)\cos\beta$$

$$b = -\left[(W_A - N_A\cos\beta)\sin\beta\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\beta + \sin\beta(C + W_P\tan\phi)\right]$$

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi \quad (14)$$

The resulting FS-value is then obtained from the solution of the quadratic equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \quad (15)$$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated. Thus a value of greater than 1.0 must be targeted as being the minimum factor of safety. How much greater than 1.0 the FS-value should be, is a design and/or regulatory issue. The issue of minimum allowable FS-values under different conditions will be assessed at the end of the paper. In order to better illustrate the implications of Eqs. 13, 14 and 15, typical design curves for various FS-values as a function of slope angle and interface friction angle are given in Figure 4. Note that the curves are developed specifically for the variables stated in the legend of the figure. Example 1 illustrates the use of the curves in what will be the standard example to which other examples will be compared.

Example 1:
Given a 30 m long slope with a uniformly thick 300 mm cover soil at a unit weight of 18 kN/m³. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is placed directly on a geomembrane as shown in Figure 3. Direct shear testing has resulted in a interface friction angle between the cover soil and geomembrane of 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg?

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Solution:

Substituting Eq. 14 into Eq. 15 and solving for the FS-value results in the following which is seen to be in agreement with the curves of Figure 4.

$$\left.\begin{aligned} a &= 14.7\,\text{kN/m} \\ b &= -21.3\,\text{kN/m} \\ c &= 3.5\,\text{kN/m} \end{aligned}\right\} \text{FS} = 1.25$$

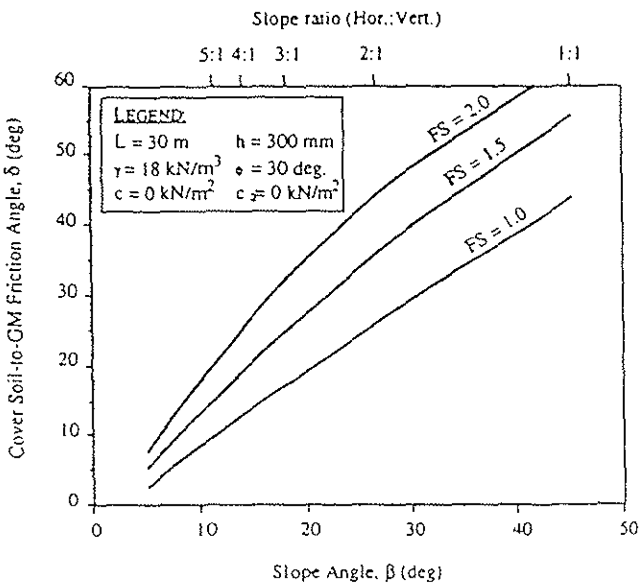


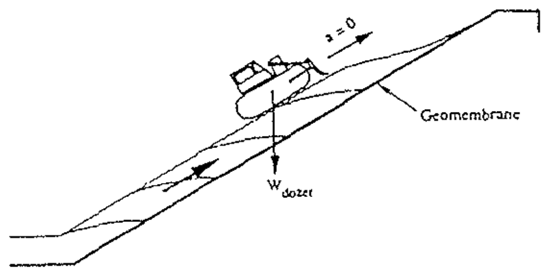
Figure 4. Design curves for stability of uniform thickness cohesionless cover soils on linear failure planes for various global factors-of-safety.

Comment:
In general, this is too low of a value for a final cover soil factor-of-safety and a redesign is necessary. While there are many possible options of changing the geometry of the situation, the example will be revisited later in this section using toe berms, tapered cover soil thickness and veneer reinforcement. Furthermore, this general problem will be used throughout the main body of this paper for comparison purposes to other cover soil slope stability situations.

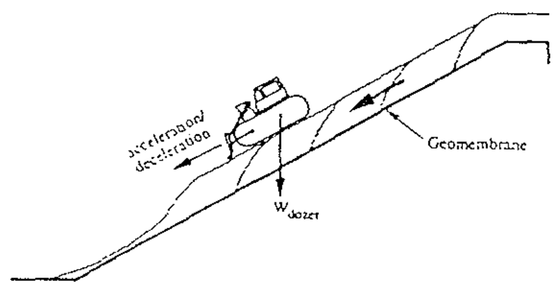
3.2 Tracked Construction Equipment Forces

The placement of cover soil on a slope with a relatively low shear strength inclusion (like a geomembrane) should always be from the toe upward to the crest. Figure 5a shows the recommended method. In so doing, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever present passive wedge and stable lower-portion beneath the active wedge. While it is necessary to specify low ground pressure equipment to place the soil, the reduction of the FS-value for this situation of equipment working up the slope will be seen to be relatively small.

For soil placement down the slope, however, a stability analysis cannot rely on toe buttressing and also a dynamic stress should be included in the calculation. These conditions decrease the FS-value and in some cases to a great extent. Figure 5b shows this procedure. Unless absolutely necessary, it is not recommended to place cover soil on a slope in this manner. If it is necessary, the design must consider the unsupported soil mass and the dynamic force of the specific type of construction equipment and its manner of operation.



(a) Equipment backfilling up slope
(the recommended method)



(b) Equipment backfilling down slope
(method is not recommended)

Figure 5. Construction equipment placing cover soil on slopes containing geosynthetics.

For the first case of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 6a. The analysis uses a specific piece of tracked construction equipment (like a bulldozer characterized by its ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the geomembrane. A Boussinesq analysis is used, see Poulos and Davis (1974). This results in an equipment force per unit width as follows:

$$W_e = q w I \tag{16}$$ *

where

W_e = equivalent equipment force per unit width at the geomembrane interface

q = $W_b / (2 \times w \times b)$ *

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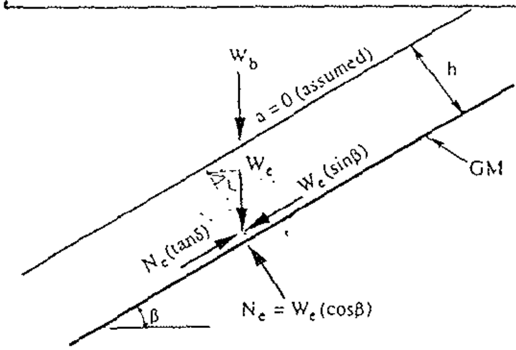
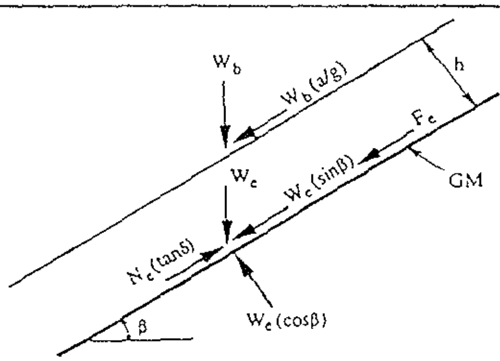
W_b = actual weight of equipment (e.g., a bulldozer)
w = length of equipment track
b = width of equipment track
I = influence factor at the geomembrane interface see Figure 7



(a) Equipment moving up slope
(load with no assumed acceleration)



(b) Equipment moving down slope
(load plus acceleration or deceleration)

Figure 6. Additional (to gravitational forces) limit equilibrium forces due to construction equipment moving on cover soil (see Figure 3 for the gravitational soil force to which the above forces are added).

Upon determining the additional equipment force at the cover soil-to-geomembrane interface, the analysis proceeds as described in Section 3.1 for gravitational forces only. In essence, the equipment moving up the slope adds an additional term, W_e, to the W_A-force in Eq. 3. Note, however, that this involves the generation of a resisting force as well. Thus, the net effect of increasing the driving force as well as the resisting force is somewhat neutralized insofar as the resulting FS-value is concerned. It should also be noted that no acceleration/deceleration forces are included in this analysis which is somewhat optimistic. Using these concepts (the same equations used in Section 3.1 are used here), typical design curves for various FS-values as a function of equivalent ground contact equipment pressures and cover soil thicknesses are given in Figure 8. Note that the curves are developed specifically for the variables stated in the legend. Example 2a illustrates the use of the formulation.

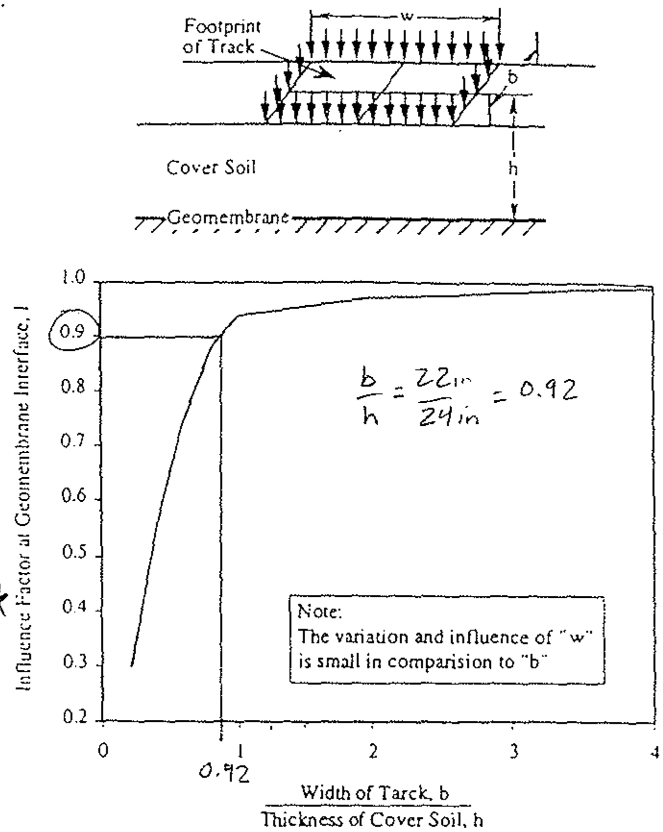


Figure 7. Values of influence factor, "I", for use in Eq. 16 to dissipate surface force of tracked equipment through the cover soil to the geomembrane interface, after Poulos and Davis (1974).

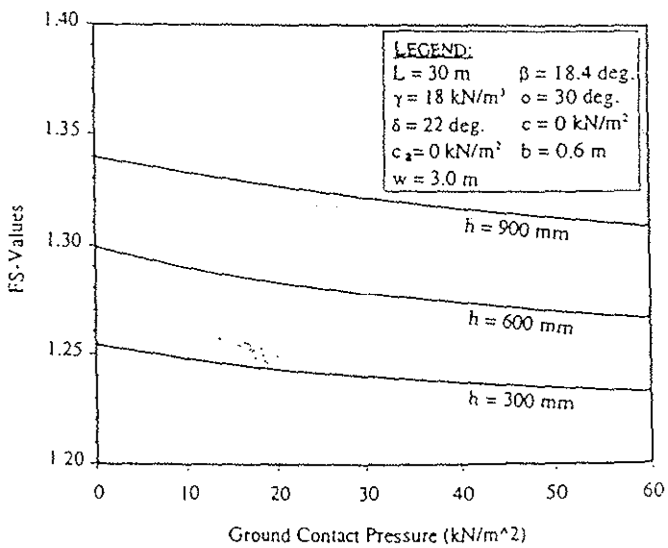


Figure 8. Design curves for stability of different thickness of cover soil for various values of tracked ground contact pressure construction equipment.

Attachment E 4/6

Example 2a:

Given 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m³. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the toe of the slope up to the crest. The bulldozer has a ground pressure of 30 kN/m² and tracks that are 3.0 m long and 0.6 m wide. The cover soil to geomembrane friction angle is 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:
This problem follows Example 1 exactly except for the addition of the bulldozer moving up the slope. Using the additional equipment load Eq. 16, substituted into Eqs. 14 and 15 results in the following.

$$\left.\begin{aligned} a &= 73.1\,\text{kN/m} \\ b &= -104.3\,\text{kN/m} \\ c &= 17.0\,\text{kN/m} \end{aligned}\right\} \text{FS} = 1.24$$

Comment:
While the resulting FS-value is low, the result is best assessed by comparing it to Example 1, i.e., the same problem except without the bulldozer. It is seen that the FS-value has only decreased from 1.25 to 1.24. Thus, in general, a low ground contact pressure bulldozer placing cover soil up the slope with negligible acceleration/deceleration forces does not significantly decrease the factor-of-safety.

For the second case of a bulldozer pushing cover soil down from the crest of the slope to the toe as shown in Figure 5b, the analysis uses the force diagram of Figure 6b. While the weight of the equipment is treated as just described, the lack of a passive wedge along with an additional force due to acceleration (or deceleration) of the equipment significantly changes the resulting FS-values. This analysis again uses a specific piece of construction equipment operated in a specific manner. It produces a force parallel to the slope equivalent to W_b (a/g), where W_b = the weight of the bulldozer, a = acceleration of the bulldozer and g = acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed, see Figure 9. A similar behavior will be seen for deceleration.

The acceleration of the bulldozer, coupled with an influence factor "I" from Figure 7, results in the dynamic force per unit width at the cover soil to geomembrane interface, "F_e". The relationship is as follows:

$$F_e = W_e\left(\frac{a}{g}\right) \qquad (17)$$

*

where

F_e = dynamic force per unit width parallel to the slope at the geomembrane interface,

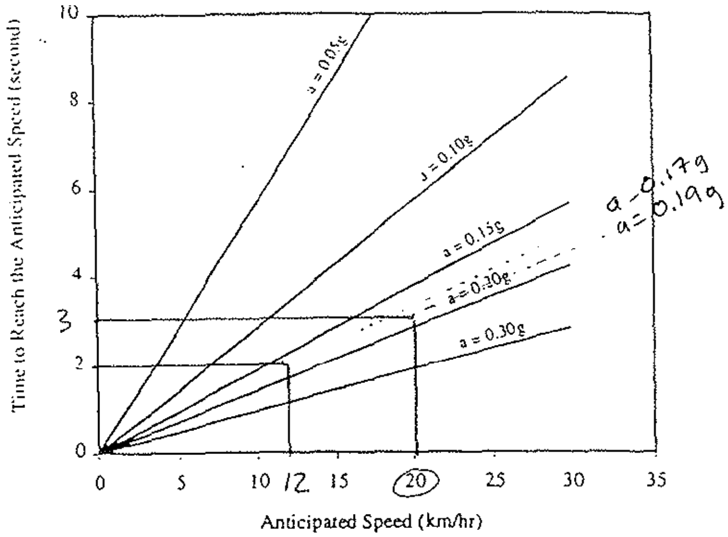


Figure 9. Graphic relationship of construction equipment speed and rise time to obtain equipment acceleration.

W_e = equivalent equipment (bulldozer) force per unit width at geomembrane interface, recall Eq. 16.
β = soil slope angle beneath geomembrane
a = acceleration of the bulldozer
g = acceleration due to gravity

Using these concepts, the new force parallel to the cover soil surface is dissipated through the thickness of the cover soil to the interface of the geomembrane. Again, a Boussinesq analysis is used, see Poulos and Davis (1974) The expression for determining the FS-value can now be derived as follows:

Considering the active wedge, and balancing the forces in the direction parallel to the slope, the following formulation results:

$$E_A + \frac{(N_e + N_A)\tan\delta + C_a}{FS} = (W_A + W_e)\sin\beta + F_e \qquad (18)$$

where

N_e = effective equipment force normal to the failure plane of the active wedge
$= W_e \cos\beta$ (19) *

Note that all the other symbols have been previously defined.

The interwedge force acting on the active wedge can down be expressed as:

$$E_A = \frac{(FS)\left[(W_A + W_e)\sin\beta + F_e\right]}{FS} - \frac{\left[(N_e + N_A)\tan\delta + C_a\right]}{FS} \qquad (20)$$

*

The passive wedge can be treated in a similar manner. The following formulation of the interwedge force acting on the passive wedge results:

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$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \quad (21)$$

By setting $E_A = E_P$, the following equation can be arranged in the form of Eq. 13 in which the "a", "b" and "c" terms are as follows:

$$a = \left[(W_A + W_e)\sin\beta + F_e\right]\cos\beta$$

$$b = -\left\{\left[(N_e + N_A)\tan\delta + C_a\right]\cos\beta + \left[(W_A + W_e)\sin\beta + F_e\right]\sin\beta\tan\phi + (C + W_P\tan\phi)\right\}$$

$$c = \left[(N_e + N_A)\tan\delta + C_a\right]\sin\beta\tan\phi \quad (22)$$

Finally, the resulting FS-value can be obtained using Eq. 15. Using these concepts, typical design curves for various FS-values as a function of equipment ground contact pressure and equipment acceleration can be developed, see Figure 10. Note that the curves are developed specifically for the variables stated in the legend. Example 2b illustrates the use of the formulation.

Example 2b:

Given a 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m³. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the crest of the slope down to the toe. The bulldozer has a ground contact pressure of 30 kN/m² and tracks that are 3.0 m long and 0.6 m wide. The estimated equipment speed is 20 km/hr and the time to reach this speed is 3.0 sec. The cover soil to geomembrane friction angle is 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:

Using the design curves of Figure 10 along with Eqs. 22 substituted into Eq. 15 the solution can be obtained:

- From Figure 9 at 20 km/hr and 3.0 sec. the bulldozer's acceleration is 0.19g.
- From Eq. 22 substituted into Eq. 15 we obtain

$$\left.\begin{array}{l} a = 88.8\,\mathrm{kN/m} \\ b = -107.3\,\mathrm{kN/m} \\ c = 17.0\,\mathrm{kN/m} \end{array}\right\} \quad FS = 1.03$$

Comment:

This problem solution can now be compared to the previous two examples:

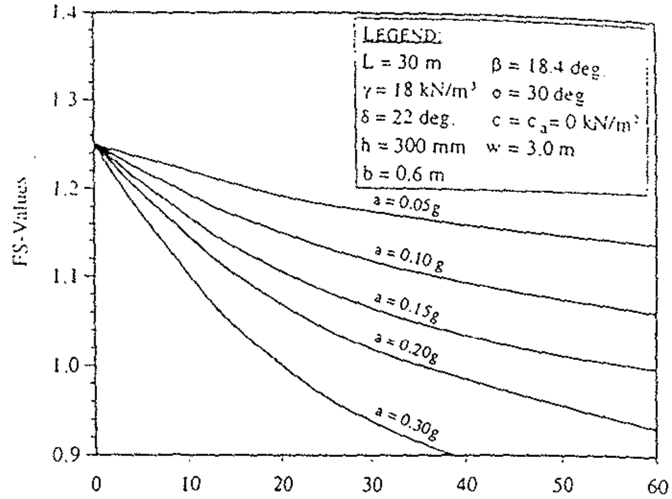

Figure 10. Design curves for stability of different construction equipment ground contact pressure for various equipment accelerations.

Ex. 1:	cover soil alone with no bulldozer loading	FS = 1.25
Ex. 2a:	cover soil plus bulldozer moving up slope	FS = 1.24
Ex. 2b:	cover soil plus bulldozer moving down slope	FS = 1.03

The inherent danger of a bulldozer moving down the slope is readily apparent. Note, that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguable the more severe condition due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should the cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the exact conditions made in the design. The maximum allowable weight and ground contact pressure of the equipment should be stated along with suggested operator movement of the cover soil placement operations. Truck traffic on the slopes can also give as high, or even higher, stresses and should be avoided unless adequately designed. Additional detail is given in McKelvey (1994). The issue of access ramps is a unique subset of this example and one which deserves focused attention due to the high loads and decelerations that often occur.

3.3 Consideration of Seepage Forces

The previous sections presented the general problem of slope stability analysis of cover soils placed on slopes under different conditions. The tacit assumption throughout was that either permeable soil or a drainage layer was placed above the barrier layer with adequate flow capacity to efficiently remove permeating water safely way from the cross section. The amount of water to be removed is obviously a site specific situation. Note that in extremely

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MODEL	D6R		D6R XL		D6R XL (IG)▶		D6R XR	
Flywheel Power	123 kW	165 hp	130 kW	175 hp	138 kW	185 hp	130 kW	175 hp
Operating Weight:*								
Power Shift	18 000 kg	39,700 lb	19 000 kg	41,900 lb	19 780 kg	43,600 lb	18 780 kg	41,400 lb
Direct Drive	18 053 kg**	39,800 lb	—		—		—	
Power Shift Differential Steer	18 200 kg	40,000 lb	19 200 kg	42,300 lb	19 960 kg	44,000 lb	18 910 kg	41,700 lb
Engine Model	3306T		3306T		3306T		3306T	
Rated Engine RPM	1900		1900		1900		1900	
No. of Cylinders	6		6		6		6	
Bore	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"
Stroke	152 mm	6"	152 mm	6"	152 mm	6"	152 mm	6"
Displacement	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	6		7		7		7	
Width of Standard Track Shoe	560 mm	1'10"	560 mm	1'10"	762 mm	2'6"	560 mm	1'10"
Length of Track on Ground	2.61 m	8'7"	2.82 m	9'3"	2.82 m	9'3"	2.75 m	9'0"
Ground Contact Area (W/Std. Shoe)	2.92 m²	4523 in²	3.16 m²	4888 in²	4.3 m²	6661 in²	3.08 m²	4771 in²
Track Gauge	1.88 m	6'2"	1.88 m	6'2"	2.03 m	6'8"	1.88 m	6'2"
GENERAL DIMENSIONS:								
Height (Stripped Top)***	2.38 m	7'5"	2.38 m	7'5"	2.38 m	7'5"	2.38 m	7'5"
Height (To Top of ROPS)	3.19 m	10'5"	3.19 m	10'5"	3.19 m	10'5"	3.19 m	10'5"
Height (To Top of Cab ROPS)	3.19 m	10'5"	3.19 m	10'5"	3.19 m	10'5"	3.19 m	10'5"
Height (To Top of ROPS Canopy)	—		—		—		—	
Overall Length (With S Blade)	5.11 m	16'9"	—		—		5.26 m	17'3"
(Without Blade)	4.08 m	13'4"	4.08 m	13'4"	—		4.22 m	13'10"
Width (Over Trunnion)	2.64 m	8'8"	2.64 m	8'8"	2.95 m	9'8"	2.64 m	8'8"
Width (W/O Trunnion — Std. Shoe)	2.44 m	8'0"	2.44 m	8'0"	2.74 m	9'0"	2.44 m	8'0"
Ground Clearance	383 mm	14.8"	383 mm	14.8"	383 mm	14.8"	383 mm	14.8"
Blade Types and Widths:								
Straight	3.35 m	11'0"	—		—		3.36 m	11'0"
Angle	—		—		—		—	
Angle Straight	4.16 m	13'7.8"	4.16 m	13'8"	—		4.16 m	13'8"
Full Angle	3.78 m	12'4.7"	3.78 m	12'5"	—		3.78 m	12'5"
Universal	—		—		—		—	
Semi-U	3.26 m	10'8"	3.26 m	10'8"	3.56 m	11'8"	3.26 m	10'8"
Fuel Tank Refill Capacity	383 L	101 U.S. gal	383 L	101 U.S. gal	383 L	101 U.S. gal	383 L	101 U.S. gal

*Operating Weight includes ROPS canopy, operator, lubricants, coolant, full fuel tank, hydraulic controls and fluid, straight dozer with tilt, horn, back-up alarm, retrieval hitch and front pull hook.

**Japan only.

***Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.

▶Intermediate Gauge offered as custom product.

ATTACHMENT F, 1/1

~~Attachment G~~

Calculation Package C
Cover Liner Sloughing Stability

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: BRC Project: BRC CAMU Project/Proposal #: SC0313 Task #: 01-02

Title of Computations: Slope Stability Evaluation – Sloughing Stability

Computations By: (3) 24 May 2005
SIGNATURE DATE

Edward M. Zielanski / Project Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer): 6/3/05
SIGNATURE DATE

Gregory T. Corcoran / Associate Engineer
PRINTED NAME AND TITLE

Computations Checked By: 6/3/05
SIGNATURE DATE

Gregory T. Corcoran / Associate Engineer
PRINTED NAME AND TITLE

Computations Backchecked By (Originator): 6/3/05
SIGNATURE DATE

Edward M. Zielanski / Project Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate): 6/3/05
SIGNATURE DATE

Gregory T. Corcoran / Associate Engineer

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Ed Zielanski (G) Date: 05 / 05 / 24 (YY MM DD) Reviewed by: ______ Date: __/__/__ (YY MM DD)

Client: BRC Project: BRC CAMU Project/Proposal No.: SC0313 Task No.: 01-02

SLOPE STABILITY EVALUATION
SLOUGHING STABILITY OF GEOSYNTHETIC-SOIL LINED SIDESLOPES
COVER LINER SYSTEM
BRC CAMU

OBJECTIVE

To evaluate the infinite slope stability (i.e., sloughing failure) within the geosynthetic-soil layered sideslopes of the cover liner system of the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada.

SUMMARY OF RESULTS

Assuming that no head buildup is expected above the liner, the proposed slope meets the required factor of safety of 1.5. The minimum interface friction angle between the cover soil and geocomposite shall be greater than or equal to 27 degrees. The yield acceleration is 0.15 g.

METHOD OF ANALYSIS

The stability analyses presented herein are based on the principles of an infinite slope failure. The critical failure plane may occur within the cover soil mass or along the geotextile/cover soil interface. The procedure outlined below is based on published data from Thiel and Steward (1993) (Attachment A). The factor of safety, defined as the ratio of resisting shear strength divided by the driving shear stress, is:

$$FS = \frac{S}{\tau} = \frac{c' + \left[h_1\gamma_1 + (h_2 - h_w)\gamma_2 - h_w\gamma_w\right]\tan\phi'}{\left[h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{sat}\right]\tan\beta} \qquad \text{(Equation 1)}$$

where:
- h_1 = thickness of topsoil;
- h_2 = thickness of drainage layer;
- h_w = average height of water in drainage layer normal to slope;
- γ_1 = saturated unit weight of topsoil;
- γ_2 = moist unit weight of drainage layer;
- γ_{2sat} = saturated unit weight of drainage layer;
- β = slope angle; and
- ϕ, c' = interface strength parameters.

Since the results of the veneer stability calculation package indicate that there will be no tension developed in the geosynthetics, only the stability of the cover soil and interface between the cover soil and drainage composite will be analyzed herein.

The yield acceleration is the horizontal acceleration (in terms of gravity) required for the slope to have a factor of safety of 1.0. Matasovic (1991) (Attachment D) presents a formula to calcualate the yield acceleration for an infinite slope, defined as:

$$k_y = \frac{c/(\gamma z \cos^2 \beta) + \tan\phi[1 - \gamma_w (z - d_w)/(\gamma z)] - \tan\beta}{1 + \tan\beta \tan\phi} \qquad \text{(Equation 2)}$$

where:
k_y = yield acceleration (g)
ϕ = friction angle (deg.)

Since the head on the liner is negligable and the material is cohesionless, Equation 2 becomes:

$$k_y = \frac{\tan\phi - \tan\beta}{1 + \tan\beta \tan\phi} \qquad \text{(Equation 3)}$$

DESIGN CRITERION

For long term conditions, a design criterion is a factor of safety of 1.5.

SIDESLOPE LINER SYSTEM

The sideslope liner system of the cover liner system consists of, from top to bottom:

- 2 ft native material;
- a drainge geocomposite;
- a 60-mil (1.5-mm) thick textured high density polyethylene (HDPE) geomembrane;
- a geosynthetic clay liner (GCL); and
- waste soil fill.

The sideslope inclination is 3.0H:1.0V. The maximum height of a side slope is 42 vertical feet, located in the southwest corner of the South Mesa.

MATERIAL SHEAR STRENGTHS

Cover Soil Material:

The soil materials to be used overlying the side slope liner system will be native materials such as silty sands (SM) for the operations layer. Converse reported a maximum dry density of 132 pcf and an optimum water content of 8.7 percent for materials at the site. Therefore, assuming 95% relative compaction, the dry density in the field is approximately 125 pcf. Adding the weight of water, the unit weight is approximately 136 pcf.

NAVFAC (1982) lists typical shear strength values for various soils based on 100 percent standard Proctor compaction (Attachment C). Actual construction materials would likely be placed at 90 percent of the modified Proctor compaction which for the sake of the comparison presented herein roughly corresponds to 95 percent standard Proctor compaction. NAVFAC (1982) lists typical shear strength values for a SM material to be 34 degrees. For the analysis herein, cohesion was neglected. Therefore, the shear strength can be approximately estimated to be 0.95(34 deg) = 32 degrees.

For this analysis, a shear strength of 32 degrees and a unit weight of 136 pcf will be used for the analyses performed herein.

ANALYSIS

Based on HELP model analyses (presented in a separate calculation package titled *Geocomposite and Pipe Size Requirements*), the side slopes will not develop a head on the liner system. In addition, a geocomposite will be used as a drainage layer in the cover liner system. Based on the assumption of a cohesionless material, Equation 1 becomes:

$$FS = \frac{S}{\tau} = \frac{\tan\phi'}{\tan\beta}$$

The design criteria dictates a minimum factor of safety of 1.5 for long term conditions. Therefore, the minimum allowable friction angle (interface or internal) is evaluated as:

$$\varphi = \tan^{-1}(FS\tan\beta)$$
$$\varphi = \tan^{-1}(1.5\tan 18.43) = 27$$

To calculate the yield accleration, Equation 3 becomes:

$$k_y = \frac{\tan 27 - 0.333}{1 + (0.333)\tan 27} = 0.15 \text{ g}$$

RESULTS AND CONCLUSIONS

The calculations suggest that the minimum friction angle (interface or internal) of 26.5 degrees will meet the design criteria of a factor of safety equal to 1.5. The yield accleration is evaluated to be 0.15 g. The minimum friction angle should be obtained in the cover soil and the interface between the cover soil and geocomposite interface. A review of current literature suggests that the minimum friction angle can be obtained.

REFERENCES

Koerner, R. M. (1997), "Designing with Geosynthetics," Simon and Schuster / A Viacom Company, Upper Saddle New Jersey, 07458, 1998 *(Attachment B)*

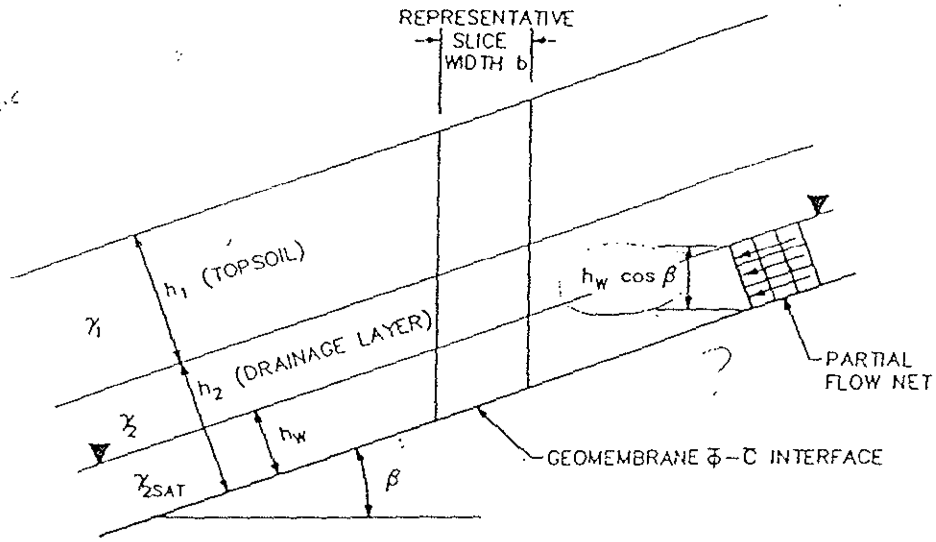
NAVFAC (1982), "Foundations and Earth Structures, Design Manual - 7.2," Department of Navy, Naval Facilities Engineering Command, May 1982 *(Attachment C)*

Thiel, R. S., Stewart, M.G., "Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific Northwest," Proceedings from Geosynthetics 1993, Vancouver, Canada. *(Attachment A)*

[redacted]

MCK 7-13-00
RC 7/27/00 GTC (3)

a) INFINITE SLOPE GEOMETRY AND MATERIAL PARAMETERS



b) FORCES

$W = [h_1\ \gamma_1 + (h_2 - h_W)\gamma_2 + h_W\ \gamma_{2SAT}]\ b/\cos\beta$

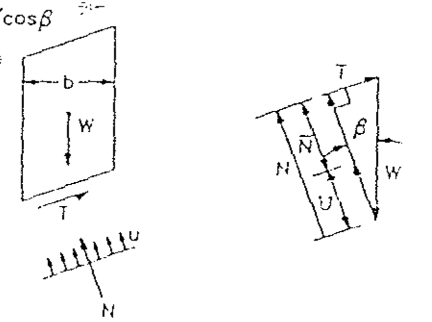
$u = \gamma_W h_W \cos\beta$

$\bar{U} = ub/\cos\beta = \gamma_w h_w b$

$N = W\cos\beta$

$T = W\sin\beta$

$\bar{N} = N - U$



FREE BODY DIAGRAM FORCE POLYGON

Geometric Parameters: β = slope angle; h_1 = thickness of topsoil; h_2 = thickness of drainage layer; h_w = average height of water in drainage layer normal to slope; b = width of representative slice.

Material Parameters: γ_1 = saturated unit weight of topsoil; γ_2 = moist unit weight of drainage layer; γ_{2SAT} = saturated unit weight of drainage layer; γ_w = unit weight of water; $\bar{\phi}$ = effective friction parameter for shear strength at base of drainage layer; $\bar{c}$ = effective cohesion parameter for shear strength at base of drainage layer.

Forces: u = pore pressure on base of drainage layer; U = uplifting water force; W = total weight of slice; N = total force normal to slope exerted by weight; T = tangential force to slope exerted by weight; $\bar{N}$ = effective normal force.

FIGURE 4. INFINITE SLOPE STABILITY WITH SEEPAGE PARALLEL TO SLOPE
(MODIFIED AFTER DUNN ET AL, 1980, p.241)

$$FS = \frac{c' + \left[\left[\gamma_1 h_1 + h_2\gamma_2 - h_w\gamma_w\right]\frac{\cos\beta}{\cos\beta}\right]\tan\phi}{\left[h_1\gamma_1 + h_2\gamma_2 - h_w\gamma_w\right]\frac{\sin\beta}{\cos\beta} + \left(\frac{\gamma_w h_w}{\cos\beta}\right)}$$

Attachment A
1/2

SOURCE: THIEL & STEWART (1993)

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The effective stress normal to base of the slice, $\bar{\sigma}$, is

$$\bar{\sigma} = \frac{\bar{N}}{b/\cos\beta} = [h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2\,SAT} - h_w\gamma_w]\cos\beta \quad (4)$$

The shear stress exerted tangential to the slice base, τ, is

$$\tau = \frac{T}{b/\cos\beta} = [h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2SAT}]\sin\beta \quad (5)$$

The shear strength at base of the slice, S, is

$$S = \bar{c} + \bar{\sigma}\tan\bar{\phi} \quad (6)$$

The factor of safety, defined as the ratio of the resisting shear strength divided by the driving shear stress, is

$$FS = \frac{S}{\tau} = \frac{\bar{c} + [h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2SAT} - h_w\gamma_w]\tan\bar{\phi}}{[h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2SAT}]\tan\beta} \quad (7)$$

OK

Because the depth of saturation in the drainage layer varies, the FS would vary also. A common procedure is to compute the average FS by using the average water depth in the drainage layer, assumed to be half the maximum water depth (D) used in Equation (2). The method therefore computes an average factor of safety for the slope length between drainage discharge points. Locations upgradient of the average flow depth will have a slightly higher FS, and downgradient locations, a slightly lower FS.

The design methodology would be to compute the FS for a given cover geometry and materials properties using Equation (7), and using one half the maximum water depth (D) used in Equations (2) and (3). If the FS is acceptable, use the maximum drainage discharge spacing (L) computed in Equation (3). If the FS is unacceptably low, reduce the distance (L), recompute the average flow depth (D/2) in the drainage layer, and recompute the FS. Iterate until the FS is acceptable.

Design Example. Given: thickness (h_1) of 1.5 feet (45 cm) of topsoil with a saturated unit weight (γ_1) of 115 pcf (18 KN/m^3) and hydraulic conductivity (k_1) of 2×10^{-4} ft/min (1×10^{-4} cm/sec); thickness (h_2) of 1 foot (30 cm) of drainage layer with moist unit weight (γ_2) of 100 pcf (15.7KN/m^3), saturated unit weight (γ_{2SAT}) of 105 pcf (16.5 KN/m^3) and hydraulic conductivity (k_2) of 0.2 ft/min (0.1 cm/sec); slope angle (β) of 3:1 (18.4 degrees); and interface friction parameter (ϕ) of 30 degrees. Unit weight of water (γ_w) = 62.4 pcf (9.8 KN/m^3).

Find: Maximum allowable spacing (L_{max}) between drainage outlets designed subparallel to slope contours such that the maximum depth (D) of accumulated water in the drainage layer is one foot (30 cm), and a minimum average FS of 1.5 is maintained.

Solution:

$$L_{(max)} = (k_2)\sin(\beta)(D)/(k_1) = 0.2\sin(18.4)(1)/0.0002 = 316\,ft\,(96\,m)$$

$$FS = \frac{[(h_1)(\gamma_1) + (h_2 - D/2)(\gamma_2) + (D/2)(\gamma_{2SAT}) - (D/2)(\gamma_w)]\tan(\phi)}{[(h_1)(\gamma_1) + (h_2 - D/2)(\gamma_2) + (D/2)(\gamma_{2SAT})]\tan(\beta)} \quad (8)$$

$$= \frac{[(1.5)(115) + (1-.5)(100) + (.5)(105) - (.5)(62.4)]\tan(30)}{[(1.5)(115) + (1-.5)(100) + (.5)(105)]\tan(18.43)} = 1.5\ (ok)$$

Factor of Safety. Geotechnical engineers often feel comfortable with a minimum FS of 1.5 for long-term static slope stability conditions. This value originated from dam

Attachment A
2/2

MSK 1-13-00
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Table 5.7 Friction values and efficiencies (in parentheses) for (a) soil-to-geomembrane, (b) geomembrane-to-geotextile, and (c) soil-to-geotextile combinations*

(a) Soil-to-Geomembrane Friction Angles

	Soil Types		
Geomembrane	*Concrete Sand* ($\phi = 30°$)	*Ottawa Sand* ($\phi = 28°$)	*Micha Schist Sand* ($\phi = 26°$)
EPDM-R	24° (0.77)	20° (0.68)	24° (0.91)
PVC			
Rough	27° (0.88)	—	25° (0.96)
Smooth	25° (0.81)	—	21° (0.79)
CSPE-R	25° (0.81)	21° (0.72)	23° (0.87)
HDPE	18° (0.56)	18° (0.61)	17° (0.63)

(b) Geomembrane-to-Geotextile Friction Angles

	Geomembrane				
		PVC			
Geotextile	*EPDM-R*	*Rough*	*Smooth*	*CSPE-R*	*HDPE*
Nonwoven, needle punched	23°	23°	21°	15°	8°
Nonwoven, heat bonded	18°	20°	18°	21°	11°
Woven, monofilament	17°	11°	10°	9°	6°
Woven, slit film	21°	25°	24°	13°	10°

(c) Soil-to-Geotextile Friction Angles

	Soil Types		
Geotextile	*Concrete Sand* ($\phi = 30°$)	*Ottawa Sand* ($\phi = 28°$)	*Mica Schist Sand* ($\phi = 26°$)
Nonwoven, needle punched	30° (1.00)	26° (0.92)	25° (0.96)
Nonwoven, heat bonded	26° (0.84)	—	—
Woven, monofilament	26° (0.84)	—	—
Woven, slit film	24° (0.77)	24° (0.84)	23° (0.87)

*Efficiency values in parentheses are based on the relationship $E = (\tan \delta)/(\tan \phi)$.

Source: After Martin et al. [14].

The frictional behavior of geomembranes placed on clay soils is of considerable importance in the composite liners of waste landfills. Current requirements are for the clay to have a hydraulic conductivity equal to or less than 2×10^{-7} ft./min. (1×10^{-7} cm/sec.) and for the geomembrane to be placed directly on the clay. While an indication of the shear strength parameters has been investigated (e.g., reference 15), the data are so sensitive to the variables listed previously that site-specific and material-specific tests should always be performed. In such cases, literature values should never be used for final design purposes.

5.1.3.9 Geomembrane Anchorage In certain problem situations a geomembrane might be sandwiched between two materials and then tensioned by an external force. The termination of a geomembrane liner within an anchor trench is such a situation. To simulate this behavior in a laboratory environment, one can use an 8.0-in. (200-mm)-wide geomembrane sandwiched between back-to-back channels.

From Koerner (1997)

Attachment B
1/1

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KC 7/27/00 GTC

BLE 1

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi) — Percent of Original Height	Typical Value of Compression: At 3.6 tsf (50 psi) — Percent of Original Height	Typical Strength Characteristics: Cohesion (as compacted) psf	Cohesion (saturated) psf	φ (Effective Stress Envelope Degrees)	Tan φ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5 x 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 x >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 x >10^{-6}	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5 x >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 x >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 x >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

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) Indicates that typical property is greater than the value shown.
   (..) Indicates insufficient data available for an estimate.

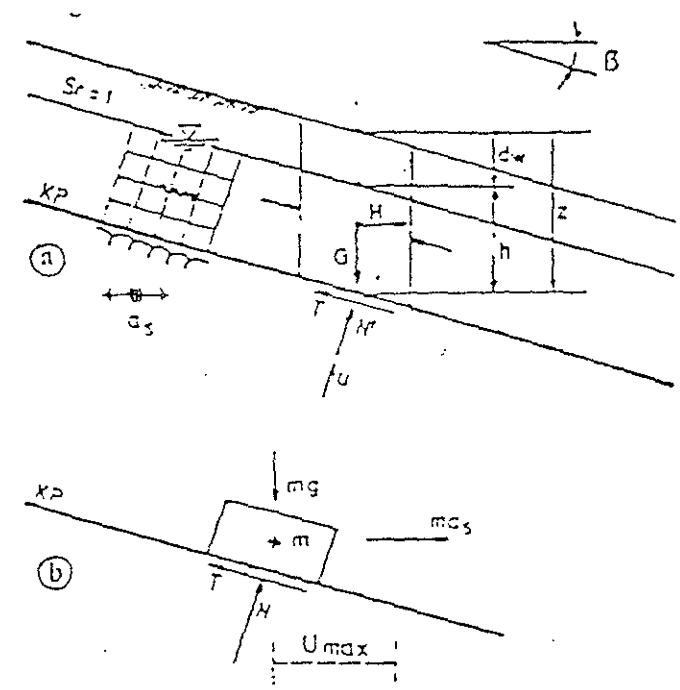
NAVFAC

Att. C
1/1

to get estimate of permanent displacement.

KC 4/27/00

(3)



- Ground Water Level
- Steady Seepage
KP - Sliding Surface
- Seismic Exitation
Umax - Permanent Displacement of sliding mass

Fig. 2 Model of an infinite slope

Based on the above assumptions, the principles of limit equilibrium and the notation introduced in Figure 2, the following expression for the factor of safety, F_s, has been derived (Matasovic, 1989):

$$F_s = \frac{c/(\gamma z \cos^2\beta) + \tan\Phi[1 - \gamma_w(z-d_w)/(\gamma z)] - k_s \tan\beta \tan\Phi}{k_s + \tan\beta} \quad (1)$$

where γ, γ_w, c and Φ are the unit weight of slope material, the unit weight of water, cohesion and the angle of internal friction respectively.

Equation (1) defines the factor of safety for a general case of infinite slope stability. A similar expression, but for stability of cohesionless materials with pore pressure increase due to seismic loading, has been used by Hadj-Hamou and Kavazanjian (1985).

It should be noted that the value of factor of safety calculated by Equation (1) diminishes with depth in cohesive ($c \neq 0$, $\varphi = 0$) materials. Also, since the equation has been set for a case of limit equilibrium when $F_s = 1$, it is assumed that slope will generally resist seismic loading and will be stable if $F_s > 1.0$.

To estimate u_{max}, it is necessary, as a first step, to determine a_c expressed by the product $k_c\, g$. The coefficient k_c can be determined iteratively by varying the amount of horizontal force until it reaches the value that gives the $F_s = 1$. However, for the model of infinite slope the coefficient of critical acceleration can be expressed explicitly by inserting $F_s = 1$ in Equation (1) and rearranging the variables:

$$k_c = \frac{c/(\gamma z \cos^2\beta) + \tan\Phi[1 - \gamma_w(z - d_w)/(\gamma z)] - \tan\beta}{1 + \tan\beta \tan\Phi} \quad (2)$$

For a case without steady state seepage in the slope (if we insert $d_w = z$), Equation (2) becomes the expression for k_c used by Chang et Al. (1984).

REFERENCE: [Matasovic, 1991]

Attachment D

1/1

Calculation Package D
Geotextile Filtration Requirements

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 04

Title of Computations: Geotextile Performance Requirements

Computations By: (GLS) SIGNATURE — 10 July 2000 DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer): SIGNATURE — 7-13-00 DATE

Michael Kalinski / Sr. Staff Eng.
PRINTED NAME AND TITLE

Computations Checked By: SIGNATURE — 7/14/00 DATE

Gregory T Corcoran
PRINTED NAME AND TITLE

Computations Backchecked By (Originator): SIGNATURE — 7/14/00 DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate): SIGNATURE — 7/17/2000 DATE

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PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GS Date: 00 / 07 / 13 (YY MM DD) Reviewed by: MSK 7.13.00 GTC Date: 00 / 07 / 14 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 04

GEOTEXTILE PERFORMANCE REQUIREMENTS BRC CAMU

OBJECTIVE

It is proposed that the final cover for the BRC Corrective Action Management Unit (CAMU) located in Henderson, Nevada include a drainage geocomposite and a geotextile filter overlying the drainage aggregate. One geotextile bonded to a geonet will be used for the top deck and two geotextiles bonded to a geonet (geocomposite) will be used for the side slopes. The filtration geotextile must retain the overlying protective soil to minimize impairment of the drainage capacity of the underlying geonet or drainage aggregate. This calculation package focuses on the separation/filtration performance of the top geotextile and the required geotextile properties.

SUMMARY OF RESULTS

The calculations suggest that the separation/filtration geotextile must have an AOS less than sieve No. 70 (0.21 mm), a permittivity greater than 0.5 sec^{-1}, a minimum mass per unit area of 6 oz./sy, and sufficient mechanical strength properties as outlined in federal regulations.

SITE CONDITIONS

The final cover system is presented in Attachment F. The final cover system consists of, from top to bottom:

- 2 ft. cover soil;
- geocomposite;
- geomembrane;
- geosynthetic clay liner; and
- subgrade.

The final cover soils will consist of on-site material, which has been classified as silty sand to well-graded sand (SM, SM-SW according to the Unified Soils Classification System) (Converse Consultants, 1999) (Attachment A).

ANALYSIS

Filtration Requirements: The geotextile will minimize fine particles of the cover soil layer from migrating into the underlying geonet or the underlying drainage aggregate. Migration of fine particles would have the adverse effect of decreasing the transmissivity of the geocomposite or clogging the drainage aggregate.

The filtration requirements for geotextiles can be evaluated using the "Geotextile Filter Design Manual," developed by Luettich et. al. (1991) (Attachment B). Page 2 of Attachment B shows a chart in which soil properties are used to evaluate the retention criteria of the geotextile by determining the maximum allowable apparent opening size (AOS or O_{95}).

The soil cover has been classified as silty or clayey sand and well-graded sand. Both of these classifications suggest that less than fifty percent of the material is fine-grained soils (i.e., smaller than the No. 200 sieve or 0.075 mm, sieve size). To be conservative in the calculations herein, the cover soil is assumed to consist of more than 20 percent clay and to be non-dispersive. Therefore, using Page 2 of Attachment B:

O_{95} < 0.21 mm, which corresponds to sieve No. 70, meaning that the geotextile apparent opening size (AOS) must be less than a No. 70 sieve size.

Permeability: The following equation can be used to evaluate the minimum allowable geotextile permeability:

$$k_g > i_s \, k_s$$

where: k_g = permeability of geotextile (cm/s)
i_s = hydraulic gradient (dimensionless)
k_s = permeability of the protective soil cover (cm/s)

Hydraulic gradient, i_s: Attachment B, page 3 from Luettich et al. (1991) lists typical hydraulic gradients for various geotextile drainage applications. In this attachment, a hydraulic gradient of 1.5 for landfill cover systems is recommended.

Soil Permeability, k_s: A permeability of 1.2×10^{-3} cm/s was used based on permeability testing of site specific soils (Attachment A).

Therefore,

$$k_g > \quad i_s k_s = (1.5)(1.2 \times 10^{-3})$$
$$k_g > \quad 1.8 \times 10^{-3} \text{ cm/s}$$

Koerner (1994) suggests applying partial factors of safety to the ultimate flow capacity of the geotextile to account for clogging of the geotextile. Using recommendations given in Table 2.13 on p. 160 of Koerner (1994) (Attachment D), the following partial factor of safety values were applied:

soil clogging and blinding	10 (5-10)
intrusion into voids	1.2 (1.0-1.2) (Surface water – minimum)
biological clogging:	2.0 (2.0-4.0) GLS
creep reduction of voids:	1.5 (1.0-1.5)
chemical clogging:	1.2 (1.2-1.5) (Surface water – minimum)

Therefore,

$$k_g > (1.8 \times 10^{-3})(10)(1.2)(2.0)(1.5)(1.2)$$
$$k_g > 0.08 \text{ cm/s}$$

The thickness of a 6 oz/yd^2 (205 g/m^2) geotextile is approximately 65 mils (0.165 cm) (Amoco technical literature, Attachment E, p.1). Dividing the permeability by the thickness of the geotextile results in the following permittivity values:

$$6 \text{ oz/sy} = 0.5 \text{ sec}^{-1}$$

Mechanical Property Requirements: To ensure proper manufacturing and durability of the geotextile, the geotextile should have appropriate strength requirements. Based on guidelines developed by Task Force 25 (see note below) (Attachment C) for mechanical properties of geotextiles used in applications requiring moderate survivability, the geotextile should have the following properties:

Property	**Criteria**
Grab Strength	≥130 lb
Puncture Protection	≥40 lb
Mullen Burst	≥210 psi
Trapezoidal Tear	≥40 lb
Ultraviolet strength retention	≥70% (typical value for geotextiles)

Note: Task Force 25 consisted of the American Association of State and Transporation Officials (AASHTO), the American Building Contractors (ABC), and the American Road Builders and Transportation Association (ARBTA).

CONCLUSIONS

In accordance with the above analyses, the top geotextile component of the drainage composite and the filtration geotextile overlying the drainage aggregate shall have the following properties:

Property	**Separation/Filtration Criteria**
matrix	nonwoven
mass per unit area	6 oz/sy (205 g/m^2)
apparent opening	≤0.21 mm (sieve No. 70)
permittivity	≥0.5 sec^{-1}
grab strength	≥130 lb
puncture strength	≥40 lb
Mullen burst	≥210 psi
trapezoidal tear	≥40 lb
ultraviolet strength reduction	≥70%

The following is a partial list of geotextile products that should meet the material requirements:

Amoco Fabrics & Fibers Co., Amoco 4506
Trevira 011/120
Synthetic Industries, Geotex 701

REFERENCES

Amoco Fabrics and Fibers Company, Atlanta, Georgia 404-984-4444 *(Attachment E)*

Converse Consultants (1999), "Preliminary Geotechnical and Geologic Investigation, Industrial Non-Hazardous Disposal Facility, Basic Management Incorporated, Clark County, Nevada," October 27, 1999, Las Vegas, Nevada. *(Attachment A)*

Koerner, R. M. (1994), "Designing with Geosynthetics," Prentice Hall, New Jersey, 1994 *(Attachment C, D)*

Luettich, S.M., Giroud, J.P., and Bachus, R.C. (1991), "Geotextile Filter Design Manual," report prepared for Nicolon Corporation, Norcross, GA *(Attachment B)*

MY MKK 7-13-00 GTC MLD

Exploration Location	Depth (feet)	Soil Description	Percent Sodium	Percent Sulfate	Total Available Water Soluble sodium Sulfate (%)
B-5	10-15	Silty sand with gravel	0.07	0.13	0.20
B-8	19-20	Silty sand with gravel	0.07	0.06	0.08
B-101	5-10	Silty sand with gravel	0.17	0.06	0.08
B-102	0-5	Fill – Silty sand with gravel	0.17	0.03	0.05
B-106	0-5	Silty sand with gravel	0.15	0.08	0.12
B-106	29-30	Silty sand with gravel	0.15	0.06	0.08

Permeability

Falling head permeability tests were conducted on remolded samples in general accordance with modified ASTM procedure D2434. The soil was compacted in a mold 4.6 inches long and 4.0 inches in diameter to 85 or 90 percent of maximum dry density and at optimum moisture content. A falling head was applied to the sample and the flow of water through the sample was monitored. The permeability was calculated after the flow rate had stabilized. The result of the falling head permeability test is presented in the following table:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-5	20-25	Silty sand with gravel	5.3×10^{-4}
B-12	10-15	Silty sand with gravel	4.0×10^{-4}
B-102	20-25	Silty sand with gravel	1.0×10^{-4}
* B-105	20-25	Well graded sand with silt and gravel	1.2×10^{-3}

Flexible wall permeameter tests were performed on selected samples by AP Engineering and Testing, Inc according to ASTM D5084. With the exception of one sample (B-105), all tested samples were undisturbed ring samples. The samples were placed in a triaxial machine with a constant confining pressure at the approximate in-place effective stress pressures. Results were generally consistent with the fal-

Converse (1999)

Attachment A

MG 7/3.C.

GAC

4.2 Define the Hydraulic Gradient for the Application (i_s)

The hydraulic gradient will vary depending on the application of the filter. Anticipated hydraulic gradients for various applications may be estimated using Figure 3.

4.3 Determine the Minimum Allowable Geotextile Permeability (k_g)

After determining the soil hydraulic conductivity and the hydraulic gradient, the following equation can be used to determine the minimum allowable geotextile permeability [Giroud, 1988]:

$$k_g > i_s\, k_s$$

The hydraulic conductivity (permeability) of the geotextile can be calculated from the permittivity test method ASTM D 4491; this value can often be obtained from the manufacturer's literature as well. The geotextile permeability is defined as the product of the permittivity, ψ, and the geotextile thickness, t_g:

$$k_g > \varphi\, t_g$$

STEP 5. DETERMINE ANTI-CLOGGING REQUIREMENTS

To minimize the risk of clogging, the following criteria should be met:

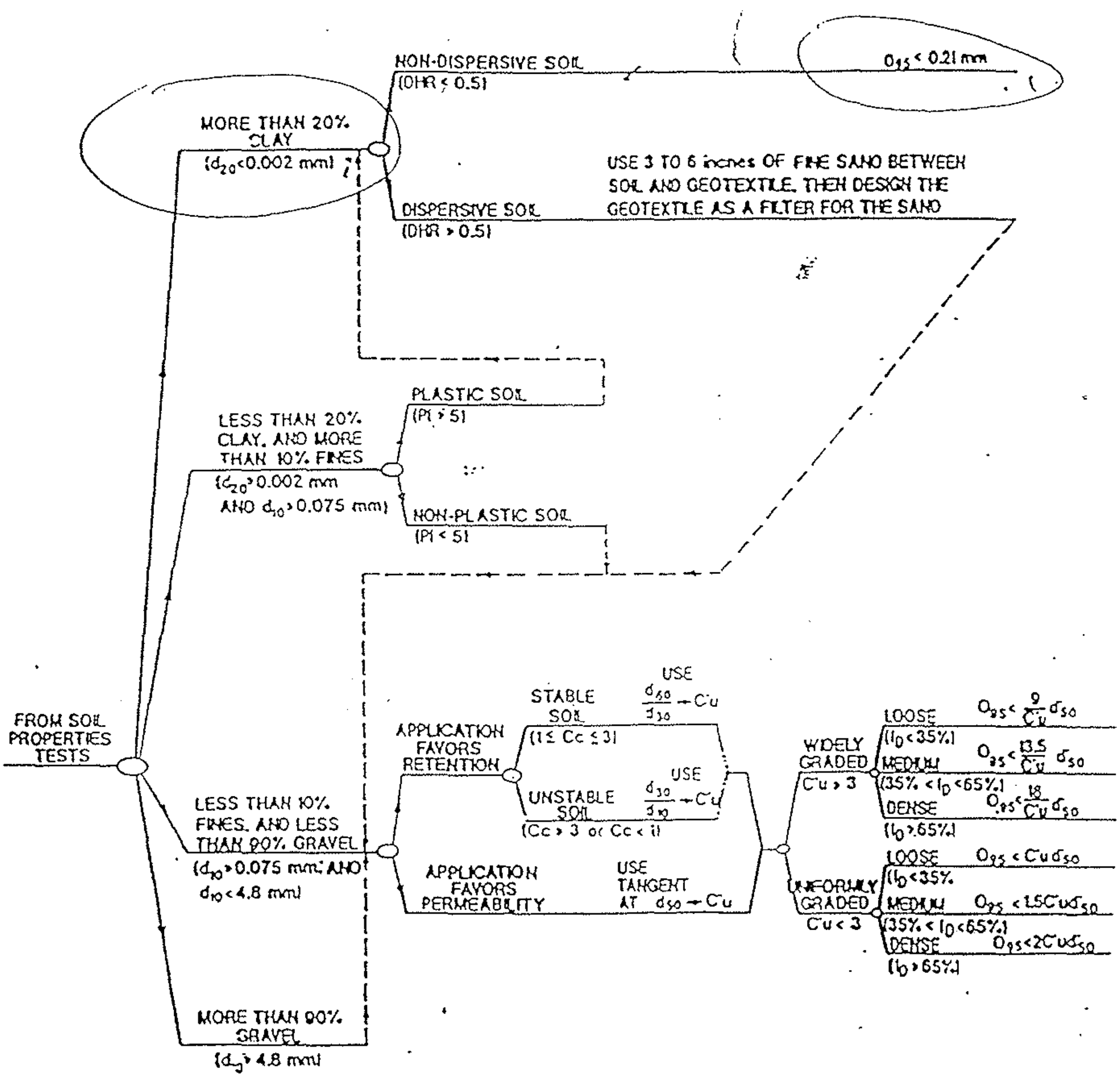
- Use the largest opening size (O_{95}) that satisfies the retention criteria.
- For nonwoven geotextiles, use the largest porosity available, but not less than 30 percent.
- For woven geotextiles, use the largest percent open area available, but not less than 4 percent.

Source: Luettich, S.M., Giroud, J.P. and Bachus R.C (1991) "Geotextile Filter Design Manual" Report prepared For Nicolon Corporation Norcross, GA

Attachment B page 1/3

7-13-00 GTC
ATTACHMENT
P 2/3

CHART 1
SOIL RETENTION CRITERIA
FOR STEADY-STATE FLOW CONDITIONS



NOTES:

d_x is the particle size of which x percent is smaller

$C'u = \sqrt{\frac{d'_{100}}{d'_0}}$ where: d'_{100} and d'_0 are the extremities of a straight line drawn through the particle-size distribution as directed above; and d'_{50} is the midpoint of this line.

$Cc = \frac{(d_{30})^2}{d_{60} \times d_{10}}$

I_D is the relative denisty of the soil
PI is the plasticity index of the soil
DHR is the double-hydrometer ratio of the soil
Portions of this flow chart modified from Giroud (1988)

Source: Luettich, S.M., Giroud, J.P., and Bachus, R.C. (1991) "Geotextile Filter Design ..."

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FIGURE 3
TYPICAL HYDRAULIC GRADIENTS[a]

DRAINAGE APPLICATION	TYPICAL HYDRAULIC GRADIENT
STANDARD DEWATERING TRENCH	1.0
VERTICAL WALL DRAIN	1.5
PAVEMENT EDGE DRAIN	1[b]
LANDFILL LCDRS	1.5
~~LANDFILL LCRS~~	1.5
LANDFILL SWCRS	1.5
DAMS	10[b]
INLAND CHANNEL PROTECTION	1[b]
SHORELINE PROTECTION	10[b]
LIQUID IMPOUNDMENTS	10[b]

NOTES: (a) Table developed after Giroud, 1988.

(b) Critical applications may require designing with higher gradients than those given.

Luettich et al. (1991) Attachment B

p. 3/3

Specification for *Survivability*

TABLE C-4 MINIMUM FABRIC PROPERTIES RECOMMENDED FOR FABRIC SURVIVABILITY[a]

Required degree of fabric survivability	Grab strength (lbs.)	Puncture strength[b] (lbs.)	Burst strength[c] (lb./in.^{2})	Trap tear[d] (lbs.)
Low	90	30	145	30
* Moderate	130	40	210	40
High	180	75	290	50
Very high	270	110	430	75

(a) All values represent minimum values (i.e., any roll in a lot should meet or exceed the minimum values in this table).

(b) ASTM D751-68, tension testing machine with ring clamp, steel ball replaced with a 5/16-in.-diameter solid steel cylinder with hemispherical tip centered within the ring clamp.

(c) ASTM D751-68, diaphragm test method.

(d) ASTM D1117, either principal direction.

(1994)

KOERNER 2nd EDITION

ATTACHMENT C 1/

TABLE C-5 REQUIRED DEGREE OF SI ...ITY AS A FUNC CONDITIONS AND CONSTRUCTION EQ... IT[a]

Subgrade conditions	Construction equi... Low ground-pressure equipment ($\leq$4 lb./in.^{2})
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level such that any shallow depressions and humps do not exceed 6 in. in depth and height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Low
Subgrade has been cleared of obstacles larger than small to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 18 in. in depth and height. Larger depressions should be filled.	Moderate
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than 6 in. ± above subgrade. Fabric may be draped directly over tree trunks, stumps, large depressions and humps, holes, steam channels, and large boulders. Items should be removed only if placing the fabric and cover material over them will distort the finished road surface.	High

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(a) Recommendations are for 6-12 in. initial lift thickness. For other initial
12–18 in: reduce survivability requirement one level
18–24 in.: reduce survivability requirement two levels
>24 in.: reduce survivability requirement three levels
Survivability levels are, in increasing order: low, moderate, high, and v
For special construction techniques such as prerutting, increase fabric s
Placement of excessive initial cover material thickness may cause beari

Source: After Christopher, B., and Holtz, R. D., Federal Highway Admi
Training Manual, Washington, DC.

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Table 2.13 Recommended partial factors of safety values for use in Equation 2.25

	Various Partial Factors of Safety				
Application	*Soil Clogging and Blinding*	*Creep Reduction of Voids*	*Intrusion into Voids*	*Chemical Clogging*	*Biological Clogging*
Retaining wall filters	2.0 to 4.0	1.5 to 2.0	1.0 to 1.2	1.0 to 1.2	1.0 to 1.3
Underdrain filters	5.0 to 10	1.0 to 1.5	1.0 to 1.2	1.2 to 1.5	2.0 to 4.0
Erosion control filters	2.0 to 10	1.0 to 1.5	1.0 to 1.2	1.0 to 1.2	2.0 to 4.0
Landfill filters	5.0 to 10	1.5 to 2.0	1.0 to 1.2	1.2 to 1.5	2.0 to 50
Gravity drainage	2.0 to 4.0	2.0 to 3.0	1.0 to 1.2	1.2 to 1.5	1.2 to 1.5
Pressure drainage	2.0 to 3.0	2.0 to 3.0	1.0 to 1.2	1.1 to 1.3	1.1 to 1.3

do not serve this function, the other, sometimes primary, function will not be served properly. This should not give the impression that geotextiles as separators always play a secondary role. Many situations call for separation only, and in such cases the geotextiles do serve a significant and worthwhile function.

2.5.1 Overview of Applications

Perhaps the target application that can best illustrate the use of geotextiles as separators is their placement between an underlying reasonably firm soil subgrade and a stone base course, aggregate, or ballast placed above the geotextile. We say "reasonably firm" because it is assumed that the subgrade deformation is not sufficiently large to mobilize uniformly high tensile stress in the geotextile. (The application of geotextiles in unpaved roads on soft soils wherein membrane-type reinforcement is developed is treated later in Section 2.6.) Thus for such a separation function to occur, the geotextile must be placed on the soil subgrade and then have stone placed, spread, and compacted on top of it. A number of scenarios can be developed showing what geotextile properties are required for a given situation.

2.5.2 Burst Resistance

Consider a geotextile on a soil subgrade with stone of average particle diameter (d_a) placed above it. If the stone is uniformly sized, there will be voids within it that will be available for the geotextile to enter into. This entry is caused by the simultaneous action of the traffic loads being transmitted to the stone, through the geotextile, and into the underlying soil. The stressed soil then tries to push the geotextile up into the voids within the stone. The situation is shown schematically in Figure 2.26. Giroud [59] provides a formulation for the required geotextile strength which can be adopted for this application.

$$T_{reqd} = \frac{1}{2} p' d_a [f(\epsilon)] \qquad (2.26)$$

where T_{reqd} = the required geotextile strength,
p' = the stress at the geotextile's surface, which is less than, or equal to,

Koerner (1994) Attachment D 1/1

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AMOCO WASTE RELATED GEOTEXTILES

MINIMUM PHYSICAL PROPERTIES
(Minimum Average Roll Values)

Property	Test Method	Units	4504	4506	4508	4510	4512	4516
Unit Weight	ASTM D-3776	Oz./yd.²	4.0	6.0	8.0	10.0	12.0	16.0
Grab Tensile	ASTM D-4632	lbs.	95	150	200	235	275	350
Grab Elongation	ASTM D-4632	%	50	50	50	50	50	50
Mullen Burst	ASTM D-3786	psi	225	350	450	550	650	750
Puncture	ASTM D-4833	lbs.	55	90	130	165	185	220
Trapezoid Tear	ASTM D-4533	lbs.	35	65	80	95	115	130
Apparent Opening Size	ASTM D-4751	US Sieve Number	70	70	100	100	100	100
Permittivity	ASTM D-4491	gal/min/ft² sec⁻¹	100 2.0	90 1.7	80 1.5	70 1.1	60 0.9	50 0.7
Permeability	ASTM D-4491	cm/sec	.2	.2	.2	.2	.2	.2
Thickness	ASTM D-1777	mils	40	65 *	90 *	110	130	175
U.V. Resistance	ASTM D-4355[1]	%[2]	70	70	70	70	70	70

1. Fabric conditioned per ASTM D-4355 2. Percent of minimum grab tensile after conditioning.

TYPICAL PHYSICAL PROPERTIES

Property	Test Method	Units	4504	4506	4508	4510	4512	4516
Grab Tensile	ASTM D-4632	lbs.	130/115	225/200	275/270	315/310	410/370	510/470
Grab Elongation	ASTM D-4632	%	75	65	65	65	65	65
Mullen Burst	ASTM D-3786	psi	285	410	575	650	825	920
Puncture	ASTM D-4833	lbs.	75	120	170	190	210	270
Trapezoid Tear	ASTM D-4533	lbs.	60/50	100/80	140/120	160/140	185/155	220/180
Apparent Opening Size	ASTM D-4751	US Sieve Number	70/120	70/140	100/200	100+	100+	100+
Permittivity	ASTM D-4491	gal/min/ft² sec⁻¹	150 3.1	110 2.0	100 1.8	80 1.5	70 1.3	60 1.0
Permeability	ASTM D-4491	cm/sec	.35	.31	.27	.26	.25	.23
Thickness	ASTM D-1777	mils	50	75	115	130	150	195

PACKAGING

Dimensions		4504	4506	4508	4510	4512	4516
Roll Width	ft.	15	15	15	15	15	15
Roll Length	ft.	1200	900	600	600	450	300

Amoco Techical Literature Appendix E

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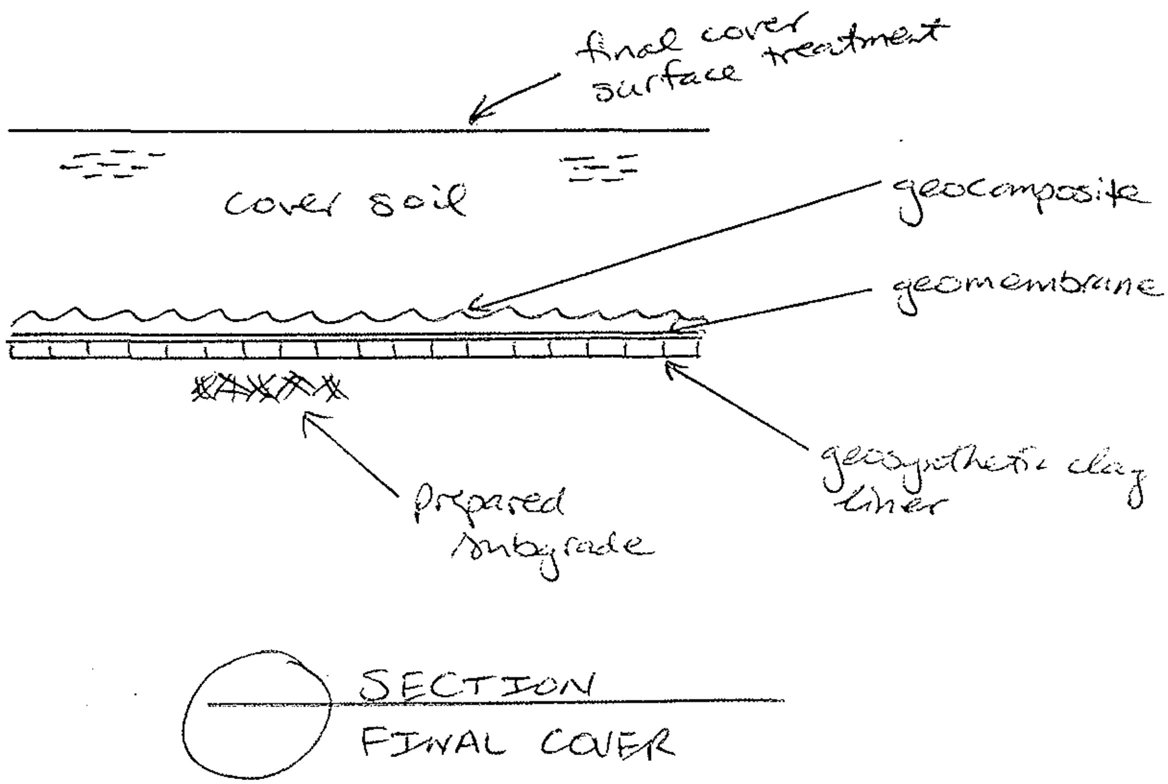
GTC

accuracy or reliability of said information or the performance of any product. Any specifications, properties ... provided as information only and in no way modify, amend, enlarge or create any warranty. Nothing contained herein is to be construed as permission or as a recommendation to infringe any patent.

Written by: GLS Date: 00/07/07 Reviewed by: 7.13.00 GTC Date: 00/07/14

Client: Project: Project/Proposal No.: Task No.:

Final Cover Configuration



Attachment F

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Calculation Package E
Geotextile Puncture Protection

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 04

Title of Computations: Geotextile Cushion Calculations

Computations By:
SIGNATURE (GSS) — DATE: 6 July 2000
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE — DATE: 7/11/00
GREGORY T CORCORAN / PROJECT ENG
PRINTED NAME AND TITLE

Computations Checked By:
SIGNATURE — DATE: 7/11/00
GREGORY T CORCORAN / PROJECT ENG
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Computations Backchecked By (Originator):
SIGNATURE — DATE: 14 July 2000
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):
SIGNATURE — DATE: 7/17/2000
Bradford H. Cooley / Project Manager
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 00 / 07 / 10 (YY MM DD) Reviewed by: GTK Date: 00 / 07 / 11 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 4

GEOTEXTILE PUNCTURE PROTECTION OF GEOMEMBRANE COVER LINER SYSTEM FOR BRC CAMU HENDERSON, NEVADA

OBJECTIVE

A composite final cover liner system is proposed for the Corrective Action Maintenance Unit (CAMU) located in Henderson, Nevada. The objective of this calculation is to evaluate the maximum particle size of soil materials adjacent to the geomembrane that will not puncture the geomembrane. Specifically, the evaluation will consider the gravel component of the local soils (silty sand with gravel, Converse 1999) overlying the cushion geotextile and geomembrane components of the cover liner system and the waste soil underlying the geomembrane and GCL components of the cover liner system.

SUMMARY OF ANALYSIS

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

Soil Component of Liner	Maximum Particle Size	Minimum Mass Per Unit Area
Subgrade	1.5 in	9 oz/yd^2 (GCL)
Silty Sand with Gravel	1.5 in	12 oz./yd^2 (double-sided geocomposite)
Silty Sand with Gravel	1 in	6 oz/yd^2 (single-sided geocomposite)

SITE CONDITIONS

The composite liner system will be comprised of the following components, from top to bottom (Attachment A):

- 2 ft of native soil;
- double-sided geocomposite on side slopes (geonet sandwiched between two 6 oz/yd^2 geotextiles (combined thickness of 12 oz/sy)) OR single-sided geocomposite on top deck (one 6 oz/yd^2 geotextile attached to top of geonet);

- 60-mil (1.5 mm) high-density polyethylene (HDPE) geomembrane;
- a geosynthetic clay liner (GCL) with a combined geotextile mass per unit area of 9 oz/sy; and
- prepared subgrade.

The maximum height of soil to be placed overlying the final cover system is assumed to be 10 ft overlying the geomembrane. This assumes future end-use fill of the area (e.g., golf course fill).

OVERLYING PRESSURE

The unit weight of the cover soil was selected to be 136 pcf based on modified proctor tests conducted on soil samples from the site that are similar to the waste material to be placed as cover soil (Attachment B). The maximum dry density was determined to be 132 pcf at an optimum moisture content of 8.2%. Assuming that the material will be placed at a density less than 95% degree of compaction, the resulting dry density is 125.4 pcf. Adding the weight of the moisture in the soil results in a wet density of approximately 136 pcf.

The following loading conditions was evaluated:

Haul Truck (H-20) Loading

The live load applied by the haul truck was estimated to be 8.7 psi as shown in Attachment D. The dead load consists of the 1.5 ft of cover soil overburden. Therefore the dead load is 1.4 psi (1.5 ft * 136 pcf / 144 psi/psf). The combined load from haul truck (H-20) loading is:

$P = P_{live} + P_{dead} = 8.7 + 1.4 = 10.1$ psi.

Therefore, the vertical pressure on the top of pipe due to H-20 loading is 10.1 psi.

ANALYSIS

APPROACH – Protected Geomembrane

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

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

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

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

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

According to these papers, the important parameters that affect the puncture protection of geomembranes are: overlying pressure, mass per unit 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.

MASS PER UNIT AREA OF GEOTEXTILE

The combined geotextile density overlying the geomembrane will be a minimum of 12 oz/yd^2 (405 g/m^2) on the sides slopes (double-sided geocomposite) and a minimum of 6 oz/yd^2 on the top deck (single-sided geocomposite). The combined geotextile density for the GCL is 9 oz/sy (305 g/m^2).

SIZING MAXIMUM PARTICLE OF SOIL

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

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

where:

M_A = mass per unit area geotextile (g/m^2)
= 405 (12 oz./yd^2 for double sided composite), 305 (9 oz/yd^2 for GCL), and 205 (6 oz./yd^2 single-sided geocomposite);

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

P_{allow} = maximum allowable pressure = 10.1 psi (

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

where: FS = global factor of safety, 3.0
$P_{actual\ field\ pressure}$ = 70 kPa
P'_{allow} = (70)(3) = 210 kPa

MF_S = shape factor:
1.0 (assume angular particles)

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

MF_A soil arching:
0.75 (assume moderate)

FS_{CR} partial factor of safety for creep
1.5 (see Table 12)

FS_{CBD} partial factor of safety for chemical and biological degradation
1.5 (based on average value) (Attachment C)

Solving for P_{allow} provides:

P_{allow} = (210) (1.0 x 1.0 x 0.75)(1.5 x 1.5)
P_{allow} = 354 kPa

RESULTS

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

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

For the three geotextile weights, the following results are:

$H_{cushion} = ((450)(M_A)/(354))^{1/2}$
$M_A = 405\ g/m^2\ (12\ oz/yd^2)$ = 22.7 mm = 0.89 inches

$H_{cushion} = ((450)(M_A)/(354))^{1/2}$
$M_A = 305\ g/m^2\ (9\ oz/yd^2)$ = 19.7 mm = 0.78 inches

$H_{cushion} = ((450)(M_A)/(354))^{1/2}$
$M_A = 205\ g/m^2\ (6\ oz/yd^2)$ = 16.1 mm = 0.64 inches

The predicted effective protrusion height equals one half the maximum stone size. Therefore, the maximum stone size is twice the values listed above.

$M_A = 405\ g/m^2$ = 0.89 inches x 2 = 1.78 inches
$M_A = 305\ g/m^2$ = 0.78 inches x 2 = 1.56 inches
$M_A = 205\ g/m^2$ = 0.64 inches x 2 = 1.28 inches

CONCLUSIONS

Assuming the following:

- the particle shape is angular for the gravel component of the cover soils, and
- the approach presented by Wilson-Fahmy, Narejo, and Koerner for evaluating puncture protection of geomembranes is appropriate for the analysis herein.

then, the calculations suggest that THE MAXIMUM PARTICLE SIZE IS 1.5 IN. for a 12 oz/sy geotextile (combined weight of the double-sided geocomposite) and the 9 oz/sy geotextile (combined weight for the GCL) and 1 inch for the 6 oz/sy geotextile (single-sided geocomposite).

REFERENCES

Advanced Drainage Systems (ADS) (1994), "ADS Specification Manual", PH: 615-457-3051
Attachment D

Converse Consultants, Inc (1999), *Preliminary Geotechnical and Geologic Investigation, Industrial Non-Hazardous Disposal Facility*, October 1999.
Attachment B

Attachment C:

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.

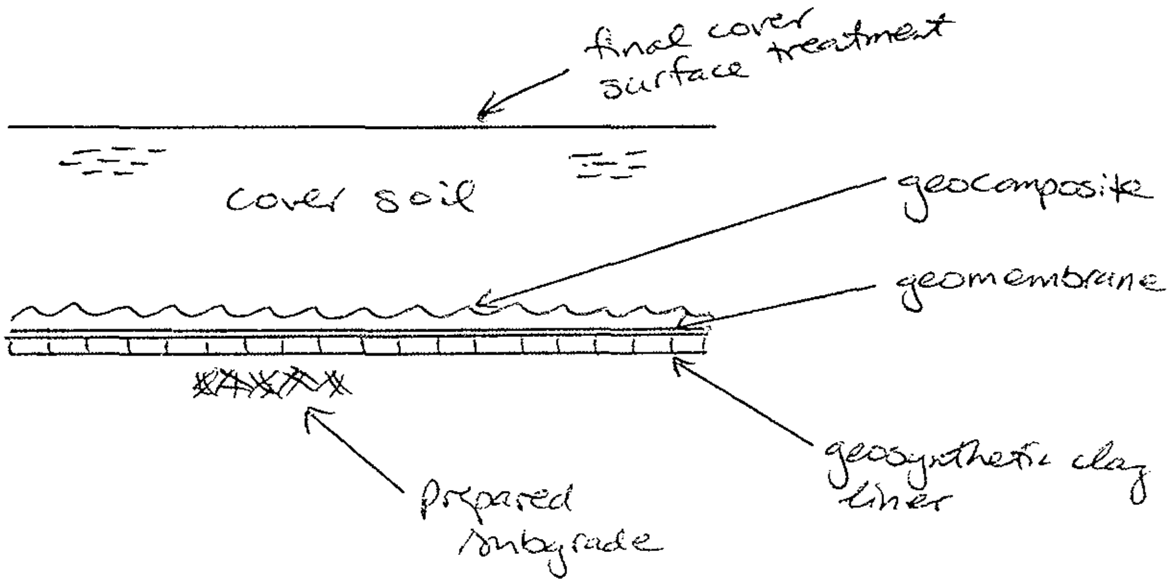
Narejo, D., Koerner, R.M. and Wilson-Fahmy, R.F. (1996) "Puncture Protection of Geomembranes Part II: Experimental", Geosynthetics International, Vol. 3, No. 5, pp. 629-653.

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: GLS MY Date: 00/07/07 (YY MM DD) Reviewed by: GTC Date: 00/07/11 (YY MM DD)

Client:________ Project:________ Project/Proposal No.:________ Task No.:________

Final Cover Configuration



SECTION

FINAL COVER

Attachment A

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shown on Drawing Nos. A-49 through A-56, entitled *Consolidation Test* and are summarized on the following table:

Exploration Location	Depth (feet)	Soil Description	Dry Unit Weight, pcf	Moisture Content, %	Hydrocollapse (percent)*
B-1	29-30	Silty sand with gravel	105	6	3.2
B-8	39-40	Sandy lean clay	57.4	64	0.4
B-8	49-50	Sandy lean clay	69.5	51.1	-0.6
B-10	54-54.5	Sandy lean clay	60.7	67.7	-0.6
B-101	39-40	Sandy lean clay	65.8	45	-0.2
B-101	59-60	Sandy lean clay	73.2	38.3	-0.6
B-102	49-50	Sandy lean clay	67.3	48.7	-0.5
B-105	34-35	Well graded sand with silt and gravel	101	5	0.1

NA: Not available

* A negative sign indicates swell occurred upon inundation with water instead of collapse.

Laboratory Maximum Density

Laboratory maximum density tests were performed on selected samples of the granular soils. The purpose of the test was to define the compaction characteristics of these soils, and to aid in estimating soil shrinkage. The laboratory maximum density test was performed in general accordance with the ASTM D1557 test method. This test procedure uses 25 blow of a 10-pound hammer falling a height of 18 inches on each of five layers of soil in a 1/30 or 1/13 cubic foot cylinder. The test results are presented on Drawing Nos. A-57 through A-61 and in the following table:

Exploration Location	Depth (Feet)	Soil Description	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (percent of dry weight)
B-1	20-25	Silty sand with gravel	129.4	8.2
B-5	20-25	Silty sand with gravel	132.1	8.2
B-12	10-15	Silty sand with gravel	129.7	7.9
B-101	5-10	Silty sand with gravel	130.6	8.7
B-105	20-25	Well graded sand with silt and gravel	131.8	7.5

Attachment B

›n le	Failure pressure† (kPa)	Applied pressure (% of failure pressure)	Applied pressure (kPa)	Failure time (hours)
	140	75	100	130
		50	70	170
		25	35	260
2	1750*	75	1300	10,000**
₁²	3400*	40	1300	10,000**
	69	75	52	24
		50	34	42
		25	17	68
2	320	75	240	140
		50	160	110
		25	80	310
2	450	85	380	240
		70	310	390
		60	270	1000**
₁²	610	75	460	10,000**
	55	75	41	0.5
		50	28	2.5
		25	14	40
2	83	75	62	3
		50	41	12
		25	21	200
2	103	75	77	192
		50	52	1000**
₁²	365	75	270	10,000**

s using Equation 3. **Geomembrane showed signs of yield. † From short term
ted cone puncture tests.

RMULATION

›n is presented in this section based on the experimental puncture ›revious sections. The resulting equations predict the allowable DPE geomembranes both with and without geotextile protection.

›dification factors are then applied to correlate the truncated cone da actual conditions. The modification factors consider the stone shape, arran‹ .t and soil arching. All of these modification factors have a magnitude of 1.0 or le.. .nce 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/m²); 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:

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

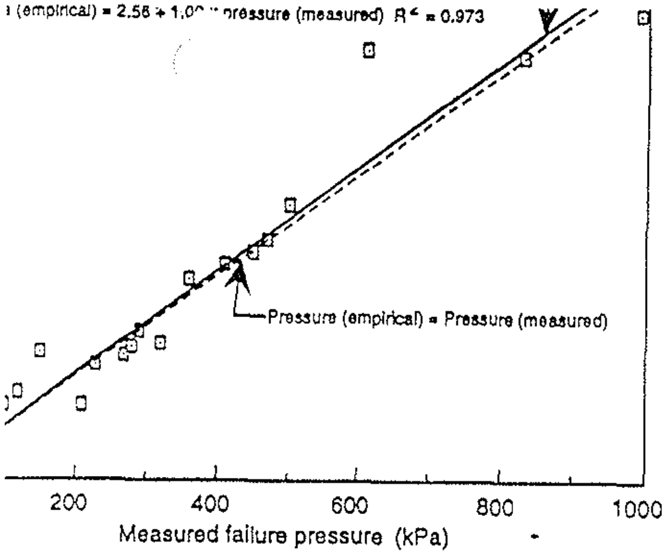
where H is in millimeters.

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



d versus empirically predicted failure pressures using Equation 3 for all nched geotextiles evaluated with a 1.5 mm thick HDPE geomembrane. on coefficient.

n Factors

ication factors is now sequentially applied to Equation 3 in order re representing field conditions. The modified pressure will be re-

Factor for the Protrusion Shape

shown that the failure pressure depends on the protrusion shape. e the highest failure pressure followed by subrounded stones. The ire is associated with angular stones and is approximately equal to of truncated cones. In order to account for the effect of stone shape, r is introduced into Equation 3 as follows:

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

odification factor for the protrusion shape. Hereafter, p'_{allow} refers odified value of p_{allow} as is illustrated in Figure 6.
sis of the data presented in Section 5.2.1, the modification factors iapes are presented in Table 9.

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

Attachment C
2/3

'actor of 0.17. It be noted, however, that the effect of soil arching at yield may n s great as the effect on the failure pressure. The the geomembra up to yield may not be large enough to mobilize the ect; therefore, caution must be exercised when using the data in Table is recommended that the values in Table 11 be used when soil arching

'actors of Safety

ing the various modification factors (all of which are 1.0 or less), sever- of safety should be applied in order to determine the allowable pressure rane. The partial factors of safety are equal to 1.0 or greater. Two fac- ed below, a partial factor of safety for long term creep and a partial fac- ccount for long term chemical/biological degradation of the materials

'actor of Safety for Creep

r of safety for creep is incorporated into Equation 6, and the allowable calculated as follows:

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

e partial factor of safety for creep. Based on the creep data presented ecommended partial factors of safety for creep are given in Table 12.

fication factors for soil arching.

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

al factors of safety for creep.

Partial factors of safety for creep			
Protrusion height (mm)			
38	25	12	6
N/R	N/R	N/R	>>1.5
N/R	N/R	>1.5	1.5
N/R	1.5	1.3	1.2
1.3	1.2	1.1	1.0
~1.2	~1.1	~1.0	1.0

commended.

s in tension. This may be explained by the fact that, in the puncture m the geo- ibrane and its protection material will conform more to the subgrade y creep nd hence the unsupported length will decrease with time. It was shown i. 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:

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

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:

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

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).

Attachment C
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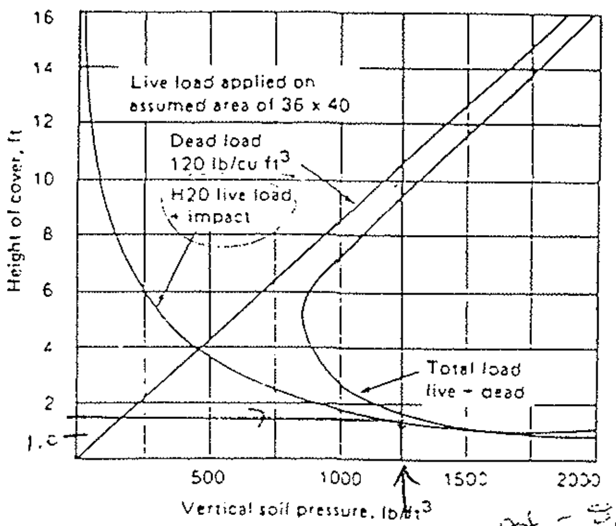
ATTACHMENT E

6TC

Figure 4



1250 psf = 8.7 ft

Figure 5

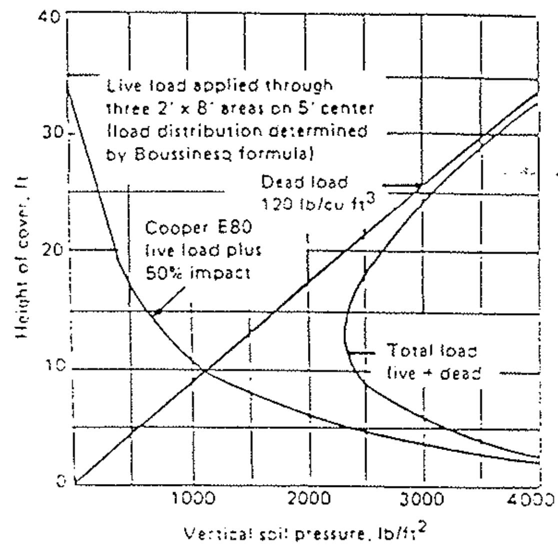
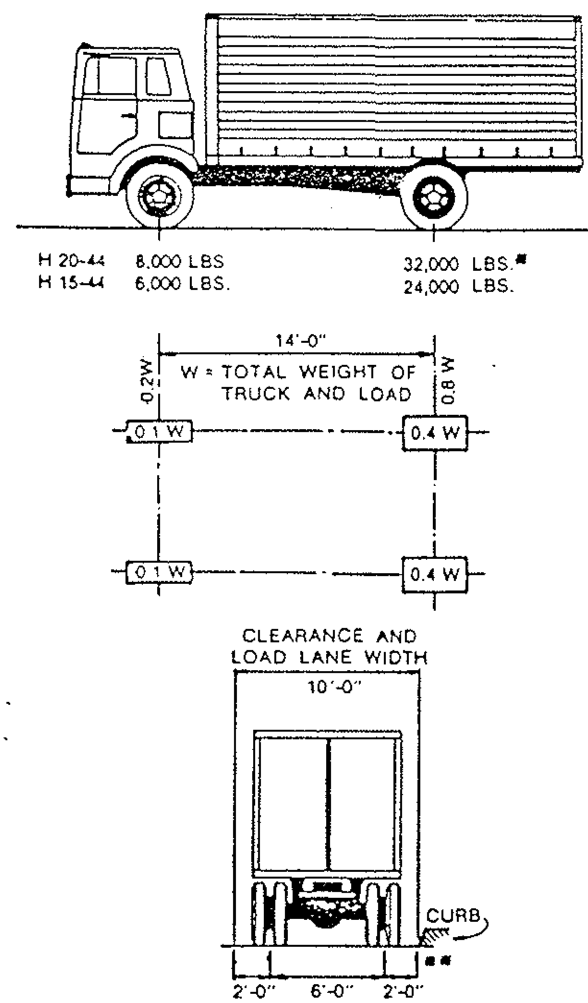
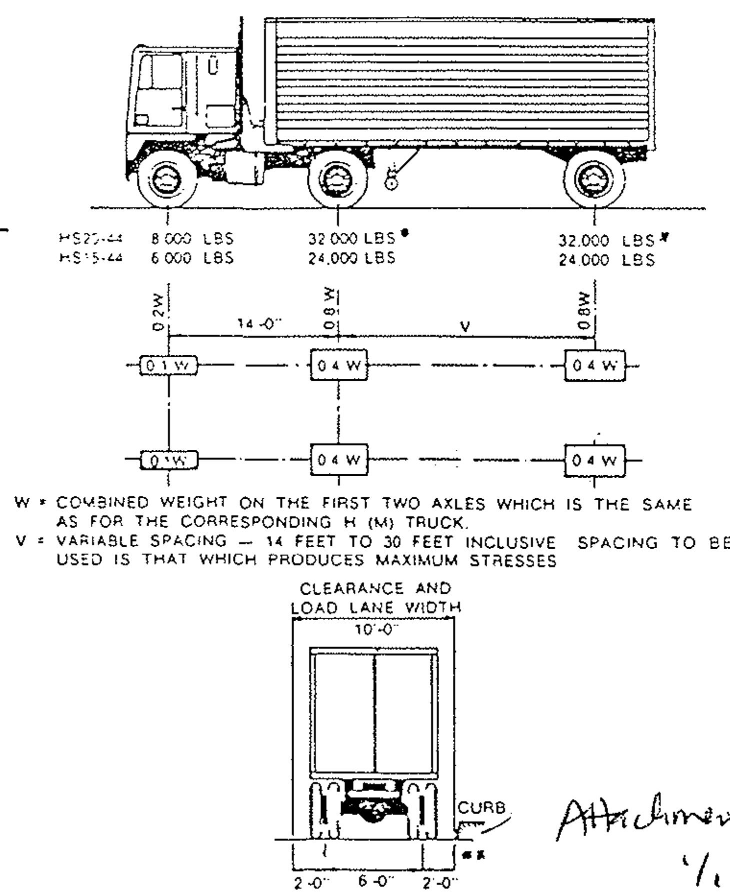


Figure 6





Attachment D
1/1

MB

Calculation Package F
Pipe Strength

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 04

Title of Computations: Pipe Strength Evaluation

Computations By:
SIGNATURE
10 July 2000
DATE
Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):
SIGNATURE
13 JULY 2000
DATE
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Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GS Date: 00 / 07 / 10 (YY MM DD) Reviewed by: [illegible] 7-13-00 GTC Date: 00 / 07 / 14 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 02

PIPE STRENGTH CALCULATIONS
BRC CAMU
HENDERSON, NEVADA

OBJECTIVE

A 6-in diameter corrugated perforated polyethylene (CPE) pipe will be constructed at the BRC Corrective Action Management Unit (CAMU) in Henderson, Nevada. This pipe will collect and transport the water collected in the geocomposite located above the geomembrane. The objective of this calculation package is to evaluate the pipe strength performance.

SITE CONDITIONS

Crushed gravel will be backfilled around the 6-in pipes. The maximum height of cover soil placed above the pipes will be 1.5-ft.

LOADING CONDITIONS

The following loading condition was evaluated:

Haul Truck (H-20) Loading

The live load applied by the haul truck was estimated to be 8.7 psi as shown in Attachment C, p. 13. The dead load consists of the 1.5 ft of cover soil overburden. Therefore the dead load is 1.4 psi (1.5 ft * 136 pcf / 144 psi/psf). The combined load from haul truck (H-20) loading is:

$P = P_{live} + P_{dead} = 8.7 + 1.4 = 10.1$ psi.

Therefore, the vertical pressure on the top of pipe in the long-term condition is 10.1 psi.

METHOD OF ANALYSES

Ring deflection, wall buckling, and wall crushing of the pipe were evaluated for the loading conditions. The Spangler's Modified Iowa Formula was used to calculate ring deflection. The actual deflection is likely lower due to the arching effects of soil via pipe deflection that are neglected in the Modified Iowa Formula. The manufacturer's design manual for ADS corrugated HDPE pipe (ADS, 1994) was used to evaluate wall buckling and wall crushing. The design criteria were based on the manufacturer's design manual for ADS corrugated polyethylene pipe.

ANALYSIS

Evaluating Variables

E' = 1,000 psi for crushed rock, dumped (Attachment C, p. 8/19)

P = Loading of 10.1 psi

EI = pipe stiffness. ADS recommends replacing the pipe EI values with a minimum pipe stiffness value defined as $D^3PS/53.77$ (Attachment C, p. 9/19) where PS = 50 psi for 6-inch diameter pipe at 5% deflection (Attachment C, p. 1/19).

Design by Wall Buckling

Wall buckling is generally the critical failure case for buried pipes. Naturally, this is a starting point. GeoSyntec set the minimum factor of safety for buckling as 2.0.

The critical buckling pressure, P_{cr} is defined (ADS 1994) as:

$$P_{cr} = 2\sqrt{\frac{E'}{1-\nu^2}\left(\frac{D^3PS}{R^3 53.77}\right)}$$ (Equation 1) (Attachment C, p. 10/19)

where: P_{cr} = critical buckling pressure (psi)
E' = soil modulus = 1,000 psi
ν = Poisson's Ratio = 0.45 for polyethylene pipe (Attachment D)
R = Pipe Radius (in) = 3 in
PS = 50 psi
D = 6 in

Solving Eqn. 4 for P_{cr}, the critical buckling is:

$P_{cr} = 193$ psi

The factor of safety for pipe buckling is defined as:

$FS = P_{cr}/P_T > 2.0$

where P_{cr} = critical buckling pressure at top of the pipe, and
P_T = total soil pressure at the top of the pipe = 10.1 psi

Therefore, the factor of safety for buckling is:

FS = 193 psi / 10.1 psi = 19 > 2.0 O.K.

Check Wall Crushing

The potential for wall crushing under load is checked in the AASHTO design procedure. Wall crushing occurs when the compressive strength of the pipe is exceeded by the overburden soil pressure. The thrust in the wall is calculated as follows:

$T = P * D/2$ (Equation 2) (Attachment C, p. 11/19)

where: T = thrust (lb/ft);
P = design load (psi); and
D = diameter (in).

The design load is 10.1 psi. Therefore, the thrust is evaluated to be:

T = 10.1 psi * 4 in / 2
T = 20.2 lb/in

Using the service load design, the following equation is used to determine the required wall thickness:

$A = T/f_a$

where: A = required wall area (in^2/ft);
T = thrust (lb/ft); and

f_a = allowable minimum compressive strength (psi) divided by a factor of safety of 2.

The compressive yield strength for pipe manufactured by ADS should be limited to 3000 psi (Attachment A, p. 2/3). Assuming a factor of safety of 2.0, the allowable compressive strength of the pipe becomes:

FS = 2.0 = 3000 psi / f_a, therefore f_a = 1500 psi

The required area can then be calculated to be:

A = 50 lb/in / 1500 psi
A = 0.033 in^2 / in

The wall area for 6-inch ADS pipe is 0.107 in^2/in. Therefore, the pipe is sufficient to withstand compressive crushing.

$F.S. = \frac{.107}{.033} = 3.2$ ok.

Written by: Geoff L. Smith GLS Date: 00 / 07 / 10 (YY MM DD) Reviewed by: GTC Date: 00 / 07 / 14 (YY MM DD)

Client: Parsons Project: BRC CAMU Project/Proposal No.: HL0389 Task No.: 02

Check Ring Deflection

Ring deflection is the change in the vertical diameter of the pipe as the pipe/sand system deforms under the external vertical pressure. Ring deflection can be evaluated using Spangler's Modified Iowa Formula formula and can be expressed as follows (ADS 1994):

$$\Delta y = \frac{D_L K_b W_c r^3}{(D^3 PS / 53.77) + 0.061 E' r^3}$$ (Equation 3) (Attachment C, p. 9/19)

where:

Δy	= pipe deflection or change in diameter, in.
r	= pipe radius, 3-inch
W_c	= prism soil load, lb/in of pipe, 136 pcf * 1.5 ft = 204 psf = 1.42 psi * 6-in pipe width = 8.5 lb/in of pipe
K_b	= bedding constant, typically 0.1 (Attachment B) (Attachment C, 6/19)
D	= pipe diameter, 6 inches
PS	= pipe stiffness, 50 psi at 5% deflection (Attachment C, 1/19)
E'	= modulus of soil reaction, 1,000 psi (Attachment C, 8/19)
D_L	= deflection lag factor, 1.5 long-term conditions (Attachment B)(Attachment C, 6/19)

Solving for Equation 12 for the critical load yields:

$$\Delta y = \frac{(1.5)(0.1)(8.5)(3^3)}{(6^3)(50)/53.77 + 0.061(1{,}000)(3^3)} = 0.019 \text{ in} \checkmark$$

Koerner (1994) recommends ring deflections of less than 5 % (Attachment B). The ring deflection for this application is: 0.019-in / 6 in = 0.3 %. Therefore, the ring deflection is acceptable.

SUMMARY AND CONCLUSIONS

Based upon these calculations for pipe ring deflection, wall buckling, and wall crushing, ADS CPE pipe satisfies the design criterion.

CONSIDERATIONS FOR SPECIFICATIONS

In accordance with the above analyses, the following items should be included in the specifications for construction at the BRC CAMU:

- Pipe shall be corrugated HDPE pipe with a smooth interior wall; and
- A minimum of 1 ft (0.3 m) of cover soil shall be placed over the pipes before a haul truck is allowed to drive over them.

REFERENCES

Advanced Drainage Systems (ADS) (1994), "ADS Specifications Manual", Ph: 614-457-3051
Attachments A and C

Philips 66 (1998), "Driscopipe Systems Design" 800-527-0062
Attachment D

Koerner R.B. (1998), "Designing with Geosynthetics", Fourth Edition, Prentice Hall, p.676
Attachment B

MNSK 1-17-00 GC

Technical Notes

ADS ADVANCED DRAINAGE SYSTEMS, INC.

Technical Note 2.130

Re: Structural Performance of Corrugated PE Pipe Using the Burns and Richard Solution

By: James B. Goddard, Norman E. Kampbell and David P. Kozman

INTRODUCTION

In 1964, Jerome Burns and Ralph Richard presented a break-through paper on the "Attenuation of Stresses for Buried Cylinders" which provided an improved understanding of the stresses around a buried pipe. The analysis is applicable to deeply buried structures where the structure is made from an elastic material and the soil is assumed to be an elastic medium. The circumferential stiffness of the pipe, the bending stiffness of the pipe, and the load transfer between the soil and pipe all influence the loads, in both magnitude and direction (tensile or compression loads). The solution is applicable to any pipe buried in an linearly elastic medium.

The elastic medium parameters are the modulus of elasticity (E′), Poisson's ratio of the soil (μ), the constrained modulus (M′), and the lateral stress ratio (K). These parameters are related by the following equations:

$$M' = \frac{E'(1-\mu)}{(1+\mu)(1-2\mu)} \tag{1}$$

$$K = \frac{\mu}{(1-\mu)} \tag{2}$$

Two additional constants relate to the lateral stress ratio:

$$B = \frac{1}{2}(1+K) = \frac{1}{2}\left(\frac{1}{1-\mu}\right) = \text{symmetrical lateral stress ratio} \tag{3}$$

and

$$C = \frac{1}{2}(1-K) = \frac{1}{2}\left(\frac{1-2\mu}{1-\mu}\right) = \text{antisymmetrical lateral stress ratio} \tag{4}$$

The pipe parameters are the mean radius of the pipe, the circumferential stiffness, and the pipe stiffness (bending stiffness). The circumferential stiffness (or ring compression stiffness) is given by the equation:

M

Attachment A

1/3

W-19K 7-13-00 OAC

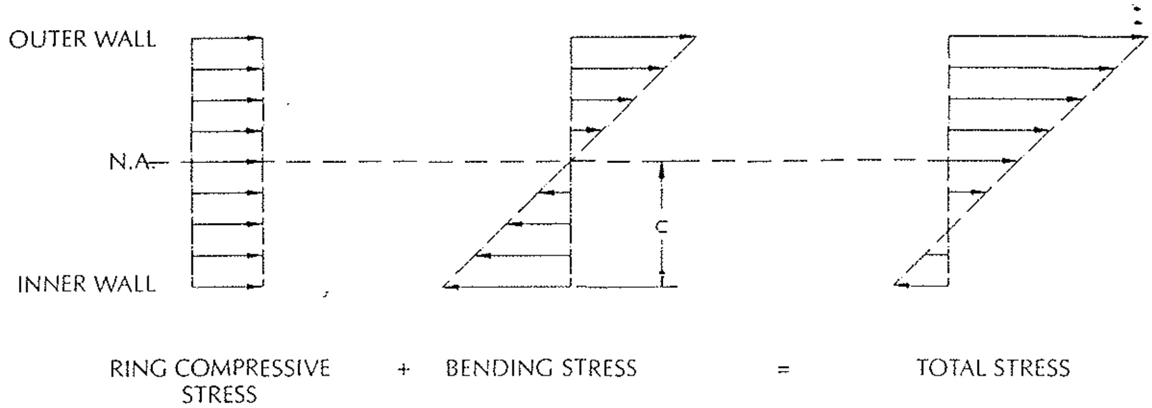

Figure 2: Stress distribution of pipe profile

A great deal can be learned from plots of pipe deflection versus pipe stiffness; moment versus pipe stiffness; thrust versus pipe stiffness; tension versus pipe stiffness ; and compression versus pipe stiffness. As pipe stiffness increases so does the moment, thrust, tension and compression in the pipe wall. All other things remaining constant, as pipe stiffness increases deflection changes very little; it is the soil stiffness that defines deflection performance. Thus it can be said a pipe that is more more compliant is a more structurally capable pipe.

For the design engineer, vertical deflection limits typically will determine the design limits; However, other parameters should also be checked. Circumferential shortening should be limited to less than 2%. Under total stress, inner and outer wall stress should be limited to less than 1,000 psi tensile stress and/or 3,000 psi compressive stress.

SUMMARY

This spreadsheet provides a powerful tool for the design engineer. Installation limits based on deflection, buckling, and circumferential shortening can be selected by the designer, based on his or her experience with pipe installations. It will provide more accurate predictions of pipe performance than the traditional approaches, particularly the "Iowa Formula" for thermoplastic pipes.

The problems with the Iowa Formula:

$$\Delta Y = \frac{W}{\left(EI/R^3 + E'\right)}$$

are:
1. It is assumed that the total stiffness (resistance to deformation) of the soil-pipe interaction system can be estimated by adding the separate stiffnesses of the pipe and the soil. It is far more complicated.
2. The pipe stiffness is a composite of a material stiffness (E) and a geometric stiffness (I/R^3). The soil stiffness (E') is only a material stiffness.

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3. The soil stiffness (E′) is empirically arrived at by back calculation of existing installations. This data is from installations with limited cover; typical 25' or less. This makes extrapolations above and beyond the pipes studied vulnerable to error. A single E′ is typically used for a given backfill material and compaction level regardless of depth, which is clearly in error.
4. The load (W) is not truly known. For flexible pipes it is often taken as the Marston load; $W = c_d \gamma B_c B_d$ where c_d is a coefficient dependent upon the depth of burial in the trench, the type of backfill soil, and the nature and extent of a soil arch; typically read from prepared charts. B_c is the OD of the pipe and B_d is the trench width.
5. For viscoelastic materials, like HDPE, the modulus value (E) typically used is based on a test specimen in bending only. There is no consideration of the effect of hoop compression and circumferential shortening; which do effect soil arching and, therefore, soil pressure on the pipe.

The Burns and Richard solution deals with these issues and provides a much more thorough analysis of the pipe response.

Table 1. Predefined MATNAM values and associated soil classes for the overburden dependent model

Young's Modulus (psi) for Overburden Pressures

Soil	MATNAM	5 psi	10 psi	15 psi	20 psi	25 psi	30 psi	40 psi	50 psi	Poisson's Ratio	Density (lb/ft³)
Granular											
Good	G.GOOD	1,100	1,300	1,500	1,650	1,800	1,900	2,100	2,250	0.30-0.35	110-150
Fair	G.FAIR	550	750	850	1,000	1,100	1,150	1,300	1,400		
Mixed											
Good	M.GOOD	600	850	1,000	1,100	1,200	1,250	1,350	1,450	0.30-0.40	100-140
Fair	M.FAIR	400	550	600	700	750	800	900	900		
Cohesive											
Good	C.GOOD	250	325	375	375	400	400	400	400	0.35-0.40	100-130
Fair	C.FAIR	150	200	225	250	250	250	250	250		

AH

Att. A
3/3

where ΔX = the horizontal deflection or change in diameter, in.,
D_L = the deflection lag factor (varies from 1.0 to 1.5), *
K = the bedding constant (varies from 0.83 to 0.110), *
W_c = the Marston's prism load per unit length of pipe, lb./in. (note that arching is not taken into account in this formula),
E = the modulus of elasticity of the pipe material, lb./in.^{2},
I = the moment of inertia of the pipe wall per unit length, in.^{4}/in. = in.^{3},
r = the mean radius of the pipe, in., and
E' = the modulus of soil reaction, lb./in.^{2}.

The last mentioned term (E') has been the subject of intense discussion and research. Howard [13] of the U.S. Bureau of Reclamation has recommended the values given in Table 7.6, which have seen relatively wide acceptance.

The preceding equation can also be cast in terms of the laboratory plate loading test with the following result. The equation assumes a bedding constant $K = 0.2$ and uses the ring stiffness constant (RSC).

$$\frac{\Delta y}{d} = \frac{P}{144} \frac{0.1L}{(1.24(RSC)/d + 0.061\ E')} \tag{7.20}$$

where Δy = the vertical pipe deformation (in.),
d = the inside pipe diameter (in.),
P = the load on pipe (lb./ft.^{2}),
L = the deflection lag factor (usually 1.0 to 1.5),
RSC = the ring stiffness constant (lb./ft.), and
E' = the modulus of soil reaction (lb./ft.^{2}).

The ring stiffness constant reflects the sensitivity of the pipe to installation stresses. It is defined in terms of the pipe's deflection resulting from the load applied between parallel plates as per ASTM D2412 (recall Section 7.1.2.1). As described in ASTM F894, *RSC* is the valued obtained by dividing the parallel plate load in pounds per foot of length by the resulting deflection, in percent, at 3% deflection. Most plastic pipe manufacturers have an empirical formula, along with the necessary tables of their pipe products, for evaluation of *RSC* values (e.g., see reference 14). The equation also reflects strongly on the type, condition, and placement of backfill both on the sides of the pipe and above it (recall Table 7.6) for values of the modulus of soil reaction (E').

Recognizing the importance of the preceding formulation, several full-scale field and large-scale laboratory trials have been published which give valuable information. Watkins and Reeve [3] have evaluated 15-, 18-, and 24-in. corrugated plastic pipe under standard H-20 truck loadings to determine the minimum cover necessary to prevent pipe damage. They also performed high-pressure, large-scale laboratory tests. Regarding the minimum cover tests, their results showed the response given in Figure 7.9. It can be seen that for a limiting ring deflection of 5% (for this particular pipe), 12 to 15 in. of soil cover is necessary. For the large-scale laboratory tests the setup and typical data were shown in Figure 7.4e.

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ADS N-12® PRODUCT FORMATION SHEET

Nominal Diameter	Inside Diameter, Average	Outside Diameter, Average	Wall Thickness, Minimum	Pipe Stiffness @ 5% Deflection	Weight lbs./20 ft. (kg./6 m.)	Area in.^{2}/in.	"I" in.^{4}/in.	"C" in.
4" (100 mm)	4.10" (104 mm)	4.78" (120 mm)	0.020" (0.50 mm)	50 psi (340 kN/m^2)	8.10 lbs. (3.60 kg.)	0.070	0.0014	0.29
6" (150 mm)	6.00" (152 mm)	6.92" (176 mm)	0.020" (0.50 mm)	50 psi (340 kN/m^2)	17.00 lbs. (7.71 kg.)	0.107	0.0036	0.25
8" (200 mm)	7.90" (200 mm)	9.11" (233 mm)	0.025" (0.64 mm)	50 psi (340 kN/m^2)	30.80 lbs. (13.97 kg.)	0.135	0.0070	0.27
10" (250 mm)	9.90" (251 mm)	11.36" (287 mm)	0.025" (0.64 mm)	50 psi (340 kN/m^2)	45.20 lbs. (20.50 kg.)	0.145	0.0110	0.34
12" (300 mm)	12.15" (308 mm)	14.45" (367 mm)	0.035" (0.89 mm)	50 psi (340 kN/m^2)	63.80 lbs. (28.96 kg.)	0.188	0.0410	0.53
15" (375 mm)	14.98" (380 mm)	17.57" (448 mm)	0.035" (0.89 mm)	42 psi (290 kN/m^2)	92.50 lbs. (42.00 kg.)	0.217	0.0660	0.66
18" (450 mm)	18.07" (459 mm)	21.20" (536 mm)	0.050" (1.27 mm)	40 psi (280 kN/m^2)	128.60 lbs. (58.38 kg.)	0.250	0.0890	0.75
24" (600 mm)	24.08" (612 mm)	27.80" (719 mm)	0.050" (1.27 mm)	34 psi (240 kN/m^2)	224.60 lbs. (101.97 kg.)	0.338	0.2310	1.07
30" (750 mm)	30.20" (767 mm)	36.07" (917 mm)	0.050" (1.27 mm)	28 psi (190 kN/m^2)	308.30 lbs. (139.97 kg.)	0.353	0.4870	0.69
36" (900 mm)	36.20" (919 mm)	42.46" (1073 mm)	0.050" (1.27 mm)	22 psi (150 kN/m^2)	361.20 lbs. (163.98 kg.)	0.361	0.5500	0.46
42" (1050 mm)	41.50" (1054 mm)	46.75" (1187 mm)	0.050" (1.27 mm)	19 psi (140 kN/m^2)	530.00 lbs. (240.62 kg.)	0.420	0.7400	1.25
48" (1200 mm)	47.55" (1208 mm)	52.70" (1339 mm)	0.050" (1.27 mm)	17 psi (120 kN/m^2)	640.00 lbs. (290.56 kg.)	0.420	0.7400	1.25

Date: March 1, 1996

Attach. C 1/14

MYK 1/13-00 GTC

Technical Report

Technical Report 4.103
Re: Plastic Pipe Design
Date: November 1, 1994
By: J. B. Goddard

ADVANCED DRAINAGE SYSTEMS, INC.

Introduction

To most of the engineering profession there was little time spent on underground structure design, particularly small pipe design, in our undergraduate studies. Bridges, large buildings, and pavement design occupied most of our time and our interest. This occured despite the fact that nearly 10% of our transportation construction dollars go for drainage structures. This is particularly interesting since a number of engineering professors have stated that the three most important considerations in pavement design are drainage, drainage, and drainage.

This lack of emphasis in our training is further exacerbated when considering plastic pipe design in that our structural design courses focused on rigid and elastic materials but spent little or no time on viscoelastic materials. A review of my own college texts reveal a total of three pages on viscoelastic properties.

In considering pipe design, generally, pipes are divided into two categories, rigid and flexible. Rigid pipes are defined as those that will not accept deflection without structural distress. Flexible pipes are defined as those that will deflect at least 2% without structural distress. Concrete, clay, and cast iron pipe are examples of rigid pipes. Steel, aluminum and plastic pipes are usually considered flexible. Within those pipes defined as flexible, the metal pipes would be considered elastic and the thermoplastic pipes would be viscoelastic or viscoplastic. Individual pipe types may have different performance limits based on type, material and wall design. The strength to resist wall stresses due to external load is critical for rigid pipe; while for flexible pipe, stiffness is important in resisting deflection and possibly buckling. Wall area may also be a factor to consider in design. For all buried pipe, rigid or flexible, the "structural performance is dependent on soil structure interaction. The type and anticipated behavior of the material beneath the structure, adjacent to the structure, and over the structure must be considered". (From paragraph 17.1.6, AASHTO Standard Specifications for Highway Bridges). Also, "It must be recognized that a buried plastic pipe is a composite structure made up of the plastic ring and the soil envelope, and that both materials play a vital part in the structural design of plastic pipe". (From paragraph 18.1.1, AASHTO Standard Specifications for Highway Bridges). Both these statements apply to rigid or flexible pipe.

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To put the differences between rigid, elastic, and viscoelastic materials in the simplest possible terms consider the following; the hard candy stick, licorice, and a hershey bar. The hard candy (the rigid structure) shatters if you attempt to bend it, regardless of loading rate. The hershey bar (the elastic structure) flexes under load but returns to shape unless that load exceeds the yield point. Beyond the yield point, the material takes a permanent set or deformation. At some amount of strain, the elastic material fails. The licorice (the viscoelastic material) responds differently depending on the rate at which the load is applied. If the load is applied very rapidly, the strength of the material is quite high. If a much lower load is placed on the licorice, it will slowly elongate. If the elongation is fixed at some constant strain, the licorice will relieve itself of stress. Although helpful in visualizing the differences in these materials,this perspective is inaccurate in that the pipe wall in non-pressure pipe is normally in compression, not tension. Because of that, the tendency is for the pipe wall to compress and thicken under load rather than stretching and thinning (or necking down). The impact of that is to increase cross-sectional or wall area while, at the same time, stress relaxation is taking place. The impact of this is discussed later in the design section.

Design Theory

The proof of any design theory should be how accurately it predicts the point and mode of failure in the product under the anticipated loading conditions. Unfortunately, current non-pressure pipe design procedures do not pass this test, regardless of major pipe types. Rigid pipe practice tends to predict quartering of the pipe as a failure mode when in fact wall shear is more common. Metal pipe design predicts circumferential wall crushing as the failure mode, a phenomenum I have never seen in the field, where localized buckling is a more typical failure mode. In defense of both theories; however, they appear to be generally conservative in that there are few structural failures of standard production pipe supplied by either industry, unless blatantly abused in installation and handling.

The same can generally be said for standard production thermoplastic pipe supplied by the major producers in that there have been few structural failures of these products. Design theory for these products is considerably more confusing, in part because the products represented are only about 30 years old (versus 100 years for steel and antiquity for concrete) and in part because of the variation in wall design and materials (primarily PVC and HDPE).

Prior to developing a design procedure, performance limits must be established. Deflection, wall buckling, stress, and strain are normally considered performance limits for flexible pipes.

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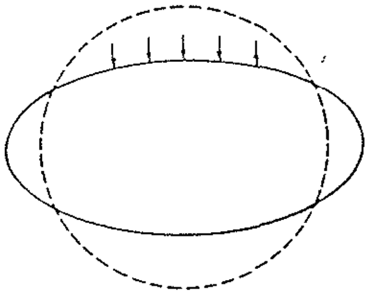
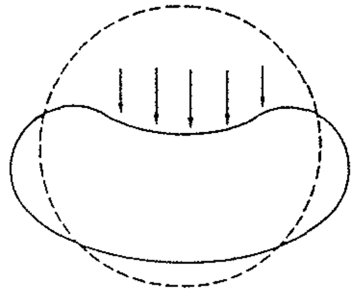
Deflection limits are set to avoid reversal of curvature, limit bending stress and strain, and avoid pipe flattening. Excessive deflection may reduce the flow capacity of the pipe and may cause joint leakage. Deflection of flexible pipe is primarily controlled by the method of installation and the backfill and insitu soil properties.

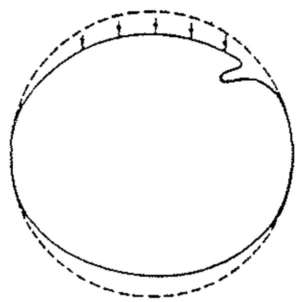


Ring deflection in a flexible pipe



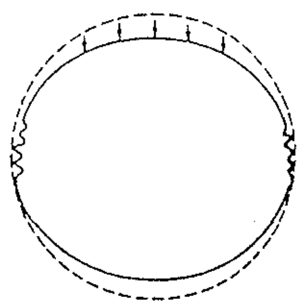
Reversal of curvature due to over-deflection

Wall buckling should be considered. Large diameter flexible pipe design may be governed by buckling, particularly when subjected to high soil pressures in low stiffness soils.



Localized wall buckling

Wall stress in compression can theoretically lead to wall crushing if excessive. If the ring compressive stress is greater than the compressive strength of the wall of the pipe, wall crushing may occur. The viscoelastic properties of thermoplastic material make this mode of failure very unlikely and field and lab tests tend to confirm that view.



Wall crushing at the 3 and 9 o'clock positions

Pipe wall strain, generally in bending should be checked. Typically, these are outer wall fiber strains brought about by excessive deflection or localized deformations. Strain limits for thermoplastic pipe materials are generally assumed to be from 3.5 to 8% depending on wall design and resin used. Note that this is fiber strain, not deflection.

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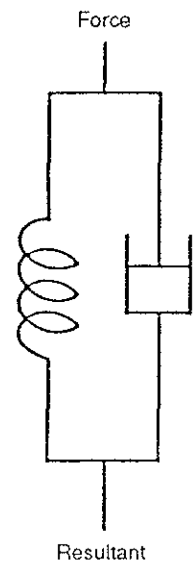
The equations to determine deflections, wall buckling, wall stress, and wall strain were developed primarily for evaluating flexible pipes manufactured from elastic materials and do not adequately reflect the effect of viscoelastic properties; in some cases treating a positive attribute as a negative. Again, using an analogy, the viscoelastic material is treated as a spring and dash pot (or shock absorber) connected in parallel with the spring handling sudden or short term loads and the dash pot responding to long term loads. The effect of this combined response is significant on the soil structure interaction system. (Figure 1)

Figure 1



Design Practice

1. Deflection

Probably the most commonly used formula in plastic pipe design is Spangler's Iowa Deflection Formula. It, at least in some form, is referenced or utilized in the ASCE Plastic Design Manual; by Moser in his textbook, Buried Pipe Design; by Koerner in his textbook, Designing With Geosynthetics; by the Bureau of Reclamation; and by the Environmental Protection Agency. The most common form of the equation is:

$$\Delta x = D_L (kWr^3)/(EI + 0.061\ E'r^3) \qquad (1)$$

Where:	Δx	=	Horizontal deflection of the pipe in inches
	D_L	=	Deflection lag factor (usually 1.5)
	k	=	Bedding constant
	W	=	Load per unit length of pipe in lbs/linear inch
	r	=	Pipe radius in inches
	E	=	Modulus of elasticity of pipe material in lbs/in2
	I	=	Moment of inertia of the pipe wall in in4/in
	E'	=	Modulus of soil reactions in lbs/in2

Developed by Dr. Merlin G. Spangler based on work begun in 1927 with rigid and flexible pipes, this built on previous work by Dr. Marston which predicted loads on culverts. The form above is the modified formula developed by Dr. Reynold Watkins based on his work in 1958.

It should be noted that this equation was developed largely from test installations with from 15 to 25 feet of cover.

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A number of factors in the equation are contentious and deserve explanation:

A. The deflection lag factor (D_L) was included in the equation because Dr. Spangler believed that deflections could increase as much as 30% over a period of 40 years. He recommended a D_L of 1.5 to be conservative. We now know virtually all of the deflection occurs during the first year, therefore a D_L of 1.0 may be used.

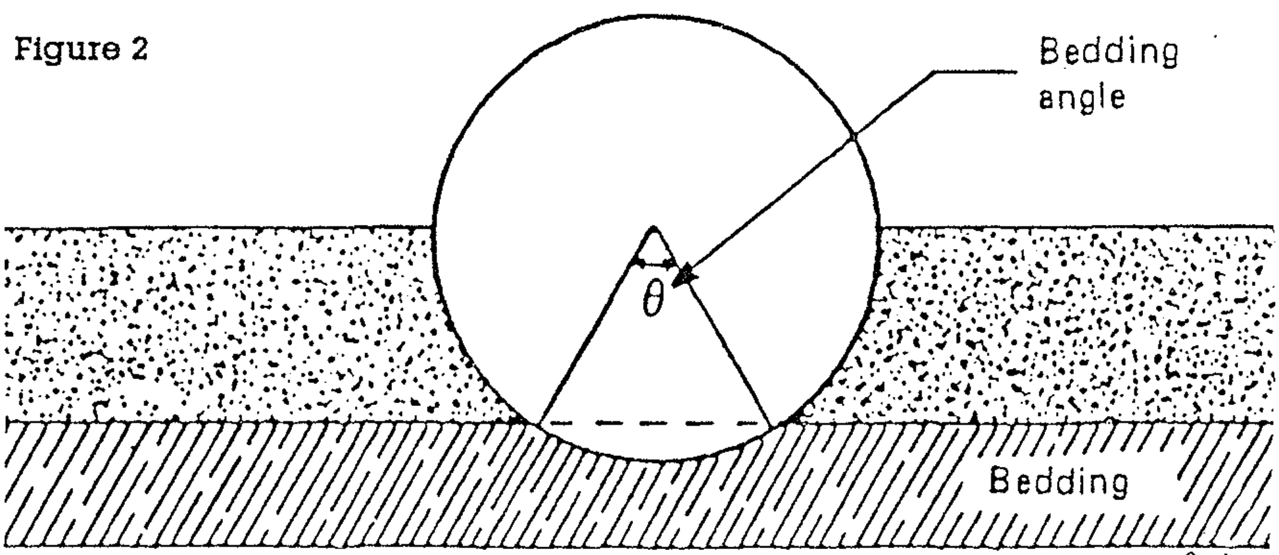
B. The bedding constant (k) is usually assumed to equal 0.1, although, as shown in Table 1, other values may be appropriate for specific installation conditions. A bedding angle (see Figure 2) of 0° would indicate a very firm foundation which would not be recommended for any pipe type.

Table 1

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

Figure 2



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C. The load per unit length of pipe (W) is Marston's prism load, which assumes that the entire weight of the vertical prism of soil over the pipe is pressing down on the pipe. For very deep fills, this is probably very conservative in that it assumes no soil arching. This may be unconservative for very shallow cover.

D. The modulus of soil reaction E' has been studied extensively and continues to be a point of contention between rigid and flexible pipe manufacturers. Probably the most used values are those developed by Amster Howard of the U.S. Bureau of Reclamation and shown in Table 2. These values are based on field measurements of flexible pipe installations whose installation conditions were known and then back calculating to find the E' values.

Recent work by Dr. Mike Duncan at V.P.I. indicates that E' varies with depth. When looked upon as a confining pressure, this seems logical. Amster Howard's work limits his E' values to 50 foot or less. Richard Chamber's work published in 1980 showed that E' can be replaced by M_s (constrained soil modulus) in the Iowa Formula. M_s does vary with depth. Dr. Duncan's values are shown in Table 3. These values may be more appropriate than those shown in Table 2.

Values of E' have been given as high as 8,000 psi in very high fills.

Selection of the appropriate E' value is up to the design engineer who must make that decision based on experience and knowledge of the project conditions. Clearly, values less than 400 psi would indicate backfill conditions inappropriate for pipe installation.

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Table 2

Average Values of Modulus of Soil Reaction, E'
(For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System*)	E' for degree of compaction of bedding, lb/in²			
	Dumped	Slight, < 85% proctor, < 40% relative density	Moderate, 85%-95% proctor, 40%-70% relative density	High > 95% proctor > 70% relative density
Fine-grained soils (LL > 50) † 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	1000
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 contains more than 12% fines	100	400	1000	2000
Coarse-grained soils with little or no fines GW, GP, SW, SP‡ contains less than 12% fines	200	1000	2000	3000
Crushed rock	1000	3000	3000	3000
Accuracy in terms of percentage deflection§	± 2	± 2	± 1	± 0.5

* ASTM Designation D2487, USBR Designation E-3
†LL - liquid limit
‡ Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC)
§ For ± 1% acuracy 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/ft³ (598,000 J/m¹) (ASTM D698, AASHTO T-99, USBR Designation E-11). 1 lb/in² = 6.9 kN/m².

SOURCE: Amster K. Howard, "Soil Reaction for Buried Flexible Pipe", U.S. Bureau of Reclamation, Denver, Colorado. Reprinted with Permission from American Society of Civil Engineers J. Geotech Eng. Div., January 1977, pp. 33-43.

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One variation to the Modified Iowa Formula that can simplify its use is replacing the pipe EI values with a minimum pipe stiffness value as shown below:

$$\Delta x = D_L\,[kWr^3/[(D^3PS/53.77) + (0.061\,E'r^3)]] \qquad (2)$$

Minimum pipe stiffness values are provided in the pipe specification in ASTM and AASHTO.

PS = Pipe stiffness in #/in/in
D = Pipe diameter in inches

One additional design approach intended in part to limit installation deflections and ensure construction survivability is the use of flexibility factor in the AASHTO design procedure. Based on earlier experience with corrugated steel and corrugated aluminum pipe, AASHTO has set a minimum flexibility factor for thermoplastic pipes at 0.095, based on the following formula:

$$FF = D^2/EI \qquad (3)$$

Where: D = Pipe diameter in inches
E = Modulus of elasticity in PSI
I = Moment of inertia of the pipe wall in in^4/in.

To utilize minimum specified pipe stiffness (PS), this equation becomes:

$$FF = 53.77/(PS \times D) \qquad (4)$$

From this, Table 4 can be generated:

Table 4

Pipe Stiffness For FF = 0.095

Pipe Diameter (in)	Pipe Stiffness (#/in/in)
12	47.17
15	37.73
18	31.44
24	23.58
30	18.87
36	15.72
42	13.48
48	11.79
54	10.48
60	9.43

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Considerable experience with pipe sizes 48" and smaller and with stiffness values equal to or greater than those given has shown that these products perform well with good installation procedures. More flexible structures can be successfully installed if special care is exercised.

2. Wall Buckling

Wall buckling can govern design of flexible pipes subjected to high soil pressures, external hydrostatic pressure, or internal vacuum. The more flexible the pipe, the lower the resistance to buckling. Caution should be exercised when considering large diameter pipes or pipes in shallow burial. Buckling equations assume the external pressure is reasonably uniform around the pipe. From Dr. Moser's textbook, the following equation offers a relatively simple equation that has been shown to be conservative for thermoplastic pipe.

$$P_{cr} = 2\sqrt{\frac{E'}{1-v^2}\left(\frac{EI}{R^3}\right)} \qquad (5)$$

Where:			
	Pcr	=	Critical Buckling Pressure (PSI)
	E'	=	Soil Modulus (PSI)
	v	=	Poisson's Ratio
	E	=	Modulus of Elasticity (Pipe Material) (PSI)
	I	=	Moment of Inertia (in_4/in)
	R	=	Pipe Radius (in)

AASHTO and ASCE use a somewhat different approach, relying on variations of the AWWA equation. The current AASHTO version is as follows:

$$Pcr = 9.24\ (R/Ap)\sqrt{C_W\ M_S\ EI/0.149R^3} \qquad (6)$$

Where:			
	R	=	Pipe Radius (in)
	M_S	=	Soil Modulus (PSI)
	C_W	=	Water Buoyancy Factor = (1 - 0.33 hw/h) Where hw = Height of water above top of pipe h = Height of ground surface above top of pipe
	Ap	=	Pipe Wall area (in^2/in)

For viscoelastic materials, the E value in this equation is normally the long-term E value, either 10 yr or 50 yr.

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3. Wall Crushing

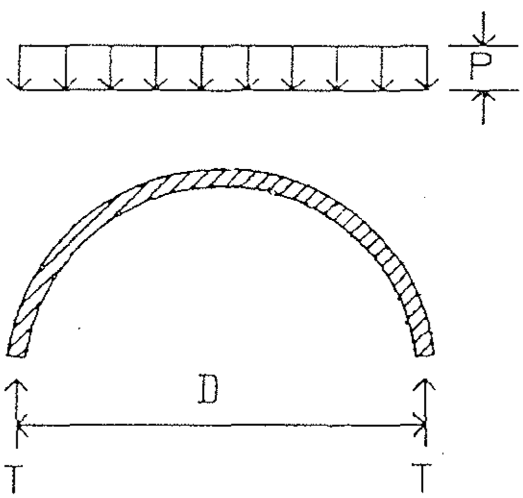
Based on ring compression theory developed for metal pipe, the potential for wall crushing under load is checked in the AASHTO design procedure. According to the AASHTO procedure, this can be addressed in two ways, using service load design or load factor design. Both start by calculating the thrust in the wall as follows:

$$T = P \times D/2 \qquad (7)$$

Where T = Thrust in pounds/foot
P = Design Load in ~~PSF~~.
D = Diameter in ~~Feet~~ inches

This is represented by the free body diagram in Figure 3.

Figure 3



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The design load is generally assumed to be the weight of the soil load above the pipe calculated by multiplying the soil density times the height of cover. Any anticipate live load must be added to this dead load. Live loads are given in Table 5 and shown on Figure 4, 5, and 6.

Table 5

Live Loads on Flexible Pipe
(Live Load Transferred to Pipe, lb/in^2)

Height of Cover	Highway	Railway	Airports
(FT)	H20	E80	
1	12.50	N.R.	N.R.
2	5.56	26.39	13.14
3	4.17	23.61	12.28
4	2.78	18.40	11.27
5	1.74	16.67	10.09
6	1.39	15.63	8.79
7	1.22	12.15	7.85
8	0.69	11.11	6.93
10	N.S.	7.64	6.08
12	-	5.56	4.76
14	-	4.17	3.06
16	-	3.47	2.29
18	-	2.78	1.91
20	-	2.08	1.53
22	-	1.91	1.14
24	-	1.74	1.05
26	-	1.39	N.S.
28	-	1.04	-
30	-	0.69	-
35	-	N.S.	-
40	-	-	-

Notes: H20 load simulates 20 ton truck traffic and impact.
E80 load simulates 80,000 lb./ft. railway load and impact.
Airport load simulates 180,000 lb. dual tandem gear, 26 inch spacing between tires and 66 inch center-to-center spacing between fore and aft tires under rigid pavement 12" thick plus impact.

N.S.= Not Significant
N.R.= Not Recommended

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Figure 4

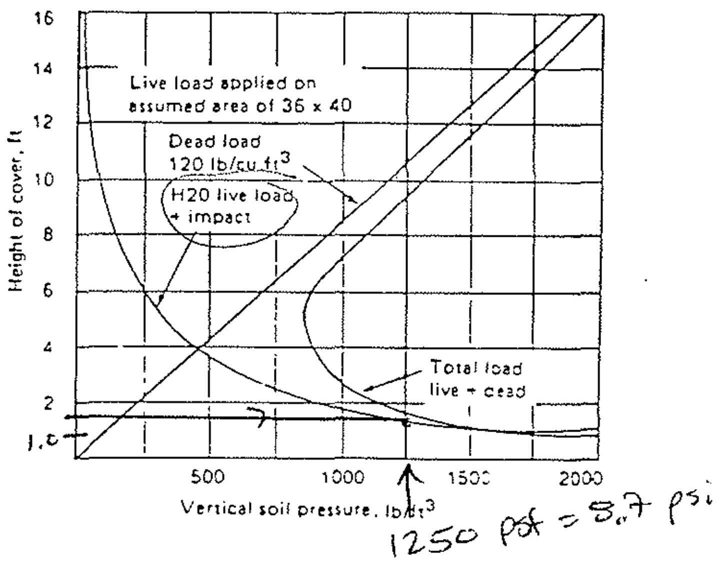


Figure 5

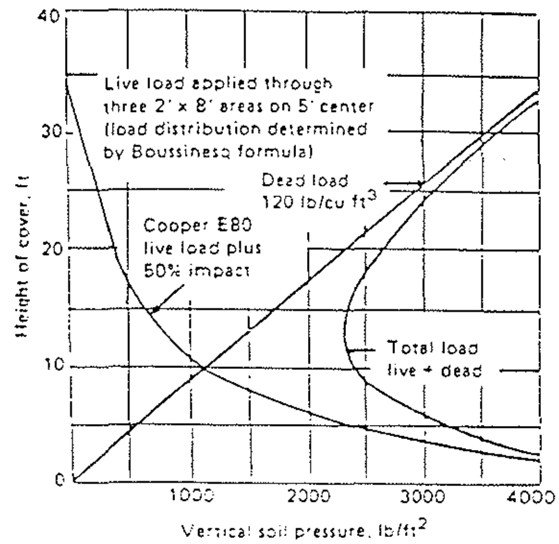
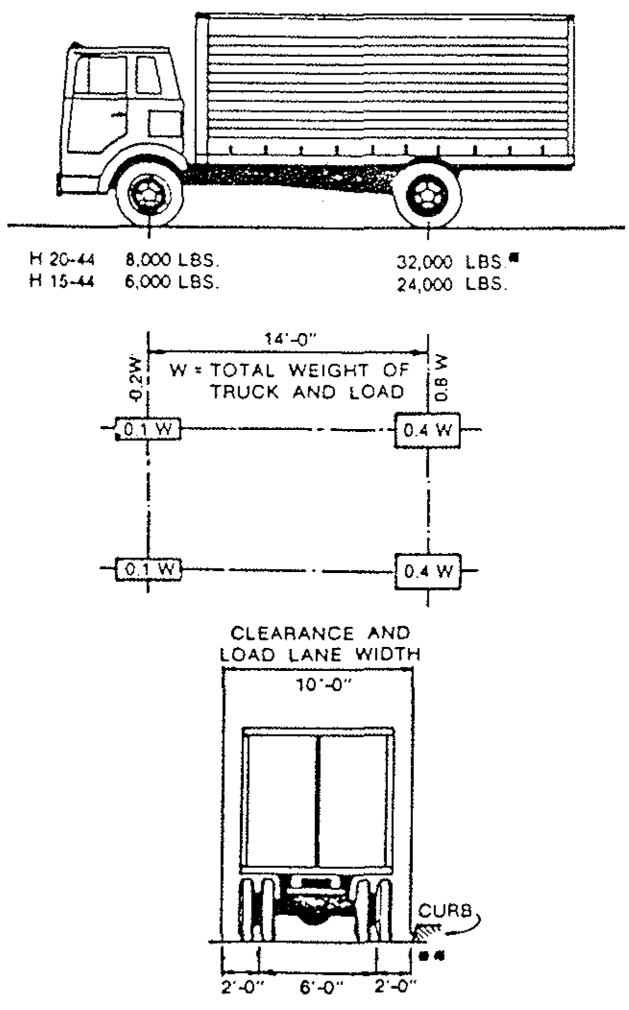
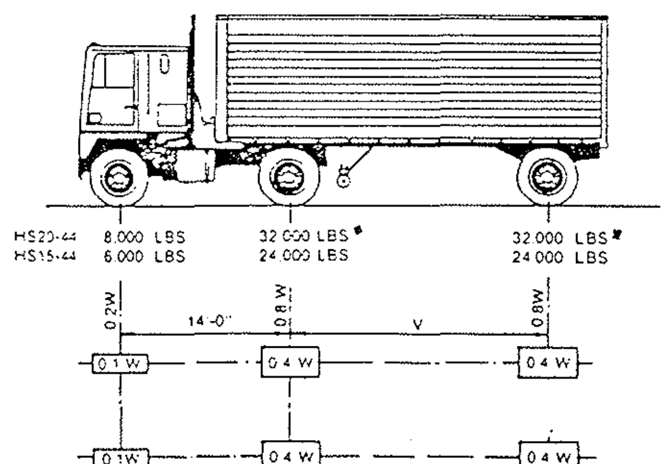
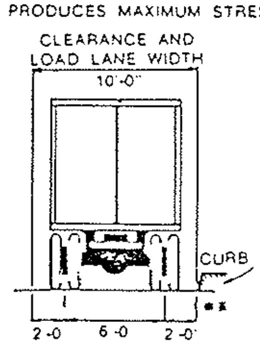


Figure 6





W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H (M) TRUCK
V = VARIABLE SPACING — 14 FEET TO 30 FEET INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES



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With the wall thrust determined, the required pipe wall area necessary can then be calculated. AASHTO provides two approaches to this, Service Load Design and Load Factor Design. Service Load Design is a working stress method while Load Factor Design is based on ultimate strength principles. Using Service Load Design, the following equation is used to determine required wall area:

$$A = T/fa \qquad (8)$$

Where:

A	=	Required wall area in in^2/ft
T	=	Thrust in #/ft
fa	=	Allowable minimum tensile strength in PSI divided by a safety factor of 2

Using Load Factor Design, the following approach is used:

$$A = T_L/\varnothing\ fu \qquad (9)$$

Where:

T_L	=	Load factor modified thrust
fu	=	Specified minimum tensile strength in PSI
Ø	=	Capacity modification factor

For flexible conduits, $T_L = T/(1.5 \times 1.3)$, where 1.3 = load factor and 1.5 = beta factor (from section 3, AASHTO Bridge Guide Manual). Phi (Ø) equals 1.0 (from section 18).

This approach has been used successfully with metal pipe for many years. For plastic pipe (viscoelastic), there are a couple of fundamental errors that lead the designer to very conservative designs. The most obvious error is the use of tensile strength values in calculating wall area in compression. At least with the principle resins used in the manufacture of thermoplastic pipe, PVC, and HDPE, the allowable values should be higher in compression.

The second error is that the calculated soil load is still based on the weight of the soil prism over the pipe without any consideration of soil arching, which has been proven in a number of research studies to reduce the load considerably in very deep fills.

The third error is the use of long term material properties rather than initial strength in these calculations. When backfilled with Type I, Type II, or compacted Type III soils, it is appropriate to assume the pipe is subjected to repeated dynamic loads from the successive settling of the soil. Because stresses in the pipe wall relax, design should be based on the instantaneous modulus of elasticity and compressive strength.

4. Pipe wall strain is mostly a post-construction concern. Within the normally specified deflection limitations, allowable outer fiber tensile strains are not a concern. If, however, due to poor installation localized deformations occur, wall strain should be checked. Allowable strains for the resins used for thermoplastic

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pipe are 3.5 to 5% for PVC and 4 to 8% for HDPE. To check bending strain, the following equation should be used:

$$E_b = 6\left(\frac{t}{D}\right)\left(\frac{\Delta Y}{D}\right) \tag{10}$$

Where: E_b = Bending strain
t = Wall thickness
D = Diameter
ΔY = Vertical deflection

Total circumferential strain may also include (in addition to bending strain) ring compression strain and strain due to Poisson's effect. Ring compression strain is:

$$E_c = P_v D/2tE \tag{11}$$

Where: E_c = Ring compression strain
P_v = Vertical soil pressure
E = Modulus of elasticity

The contribution to circumferential strain due to the Poisson effect caused by longitudinal strain is:

$$E = -v \times L_S \tag{12}$$

Where: E = Circumferential Poisson's strain
v = Poisson's ratio
L_S = Longitudinal strain

As noted, these strains are additive. Compressive strains reduce tensile strains.

In order to properly design any plastic pipe, it is necessary to know the section properties of the pipe; including inside and outside diameter, pipe wall area; wall centroid, and moment of inertia. Also, the minimum resin properties including short and long term tensile strength, modulus of elasticity, and compression strength as well as allowable long term strain. These values will be available in the referenced specifications or from the manufacturers.

Installation

As noted in the introduction, design of any buried structure, be it rigid or flexible, depends on the interaction of the pipe structure and the surrounding soil (or backfill). Sound installation practice assures satisfactory structural performance. Fortunately, for thermoplastic pipe, there is an excellent installation specification in ASTM, ASTM D2321, Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications. This specification is particularly helpful in its classification of embedment and backfill materials and its recommendations for

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their use. It also includes guidance as to the compatibility of backfill materials with various soil types, particularly in terms of migration of fines. Minimum compaction levels for different backfill classes are provided and should be followed. These are based on providing an E' (modulus of soil reaction) value of 1,000 psi, as used in the deflection formula (1).

In typical conditions, the minimum trench width is determined by the size of the pipe and the ability to get compaction equipment between the pipe and the trench walls. The minimum trench width should not be less than the outside diameter plus 16 inches or the pipe outside diameter times 1.25 plus 12 inches; whichever is greater. High speed trenchers may enable satisfactory installation of pipe in narrower trenches. Poor insitu soil conditions such as peat, muck, running sands, or expansive clays will require substantially wider backfill as well as deeper foundation and bedding. Trench width and foundation depth should be based on a thorough site investigation.

Other means of trench control through poor insitu soils include wrapping the backfill and bedding material with a geotextile. Particularly severe conditions may require a geonet or geogrid, often in combination with a geotextile.

Bedding, to provide a stable and uniform base for the pipe should be 3 to 4 inches thick. Over rock or unyielding foundations, a minimum of 6" of bedding should be provided.

Backfill in the area up to the springline should be carefully placed and compacted to achieve a minimum E' value of 1,000 psi as detailed in ASTM D2321. A minimum of 12" of backfill should be placed and compacted above the crown of the pipe. It is typical for trenches to be backfilled entirely with Type I or Type II materials when under pavement. (Figure 7)

For pipe up to 48" diameter, and with pipe stiffness equal to or in excess of those required in AASHTO Section 18 (Table 4), a minimum of 12" of compacted cover is required prior to vehicle loadings. For larger or less stiff pipe, additional cover is recommended.

Recent development of flowable, low strength cement or fly ash backfill provides the ability to reduce trench widths and still get adequate backfill support. This can be particularly helpful in municipal street installations. Manufacturer's recommendations should be closely followed.

Flexible pipe should never be installed in a concrete cradle, as done for rigid pipe in a Class A installation. This type of installation could create concentrated forces at the ends of the cradle when the pipe has deformed.

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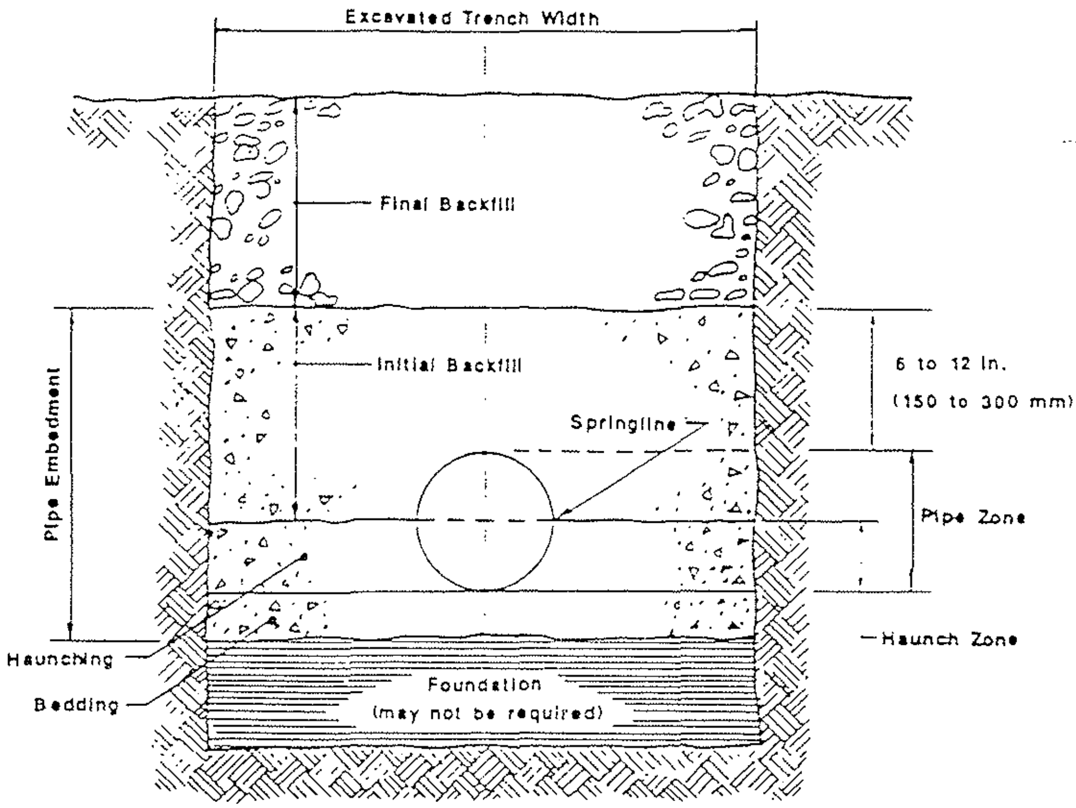
Figure 7



Trench Cross Section

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Research

One way to verify design and installation procedures for any product is to compare research findings with predicted performance. Over the past ten years, there has been a substantial amount of research done by the plastic pipe industry or by users of plastic pipe to verify the existing design procedures or to improve upon them.

A considerable amount of research has been conducted at Utah State University by either Dr. Reynold Watkins or Dr. Al Moser. Much of Dr. Watkins work has involved the large soil cell at U.S.U. to attempt to simulate very large soil pressures on buried pipe. Depending on the specific test, different backfill material and installation practice have been used as well. Based on work done in 1982 (TRR 903) (Figures 8 and 9) and 1990 on corrugated polyethylene pipe, the measured deflections were 1/2 to 2/3 those predicted by the Modified Iowa Formula (1) (Figure 10). At the soil pressures in the test cells in both tests, the resultant wall thrust exceeded that predicted by the AASHTO equations (8 & 9) by a factor of 2 using short-term material values and by a factor of 10 using long term (50 year) material values. In these tests, however, no wall thrust failure occurred, so ultimate wall thrust loads must be greater than those in these tests. These tests also exceeded the predicted wall buckling pressures by approximately 50%. With deflections less than 5% in these tests, wall strain was about 1%, well under the strain limit for HDPE.

In 1987, under the direction of Dr. Ernest Selig, a 24" diameter corrugated polyethylene pipe was installed in a 100' highway fill under I-279 North of Pittsburgh, PA as a test of the pipe's performance under high soil pressures in a realistic installation. Pipe shape and circumference have been monitored along with soil pressure at crown and springline, free field soil strain, and trench strain (pipe & backfill). Under 100 feet of fill, this pipe has shortened vertically 4.3%, with 1.6% of that reflected in circumferential shortening. The actual deflection is therefore only 2.7%. The free field soil strain is 4.2%. Because of the combination of circumferential shortening and deflection, a soil arch has developed over the pipe in the fill reducing the vertical soil pressure at the crown to only 22% of the predicted (by Marsten) soil pressure. Total vertical shortening is 55% of that predicted by the Iowa Formula (1) as deflection. Actual deflection (out of roundness) is only 35% of that predicted by the Iowa Formula. This study demonstrates that the soil arching and the circumferential shortening, which are not taken into consideration in the traditional calculations add a considerable degree of conservatism to the predicted performance values. Using the AASHTO design calculations, this pipe should have failed in wall crushing at about one half of the actual fill height. Dr. Selig has shown that finite element analysis programs, specifically CANDE and SOILCON, can predict the kind of results found in this study.

Att. C

18/19

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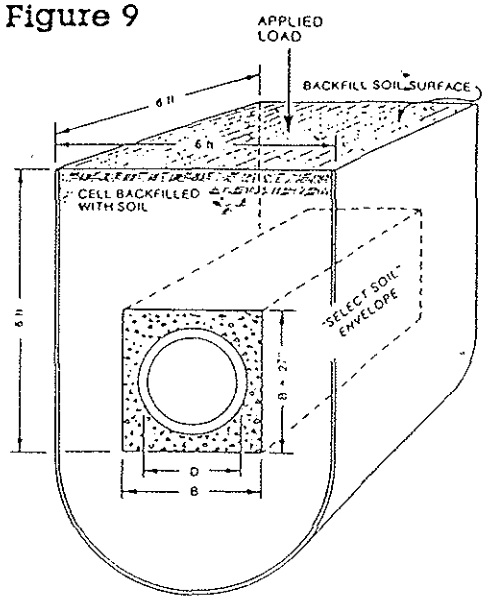
GTC

Figure 8

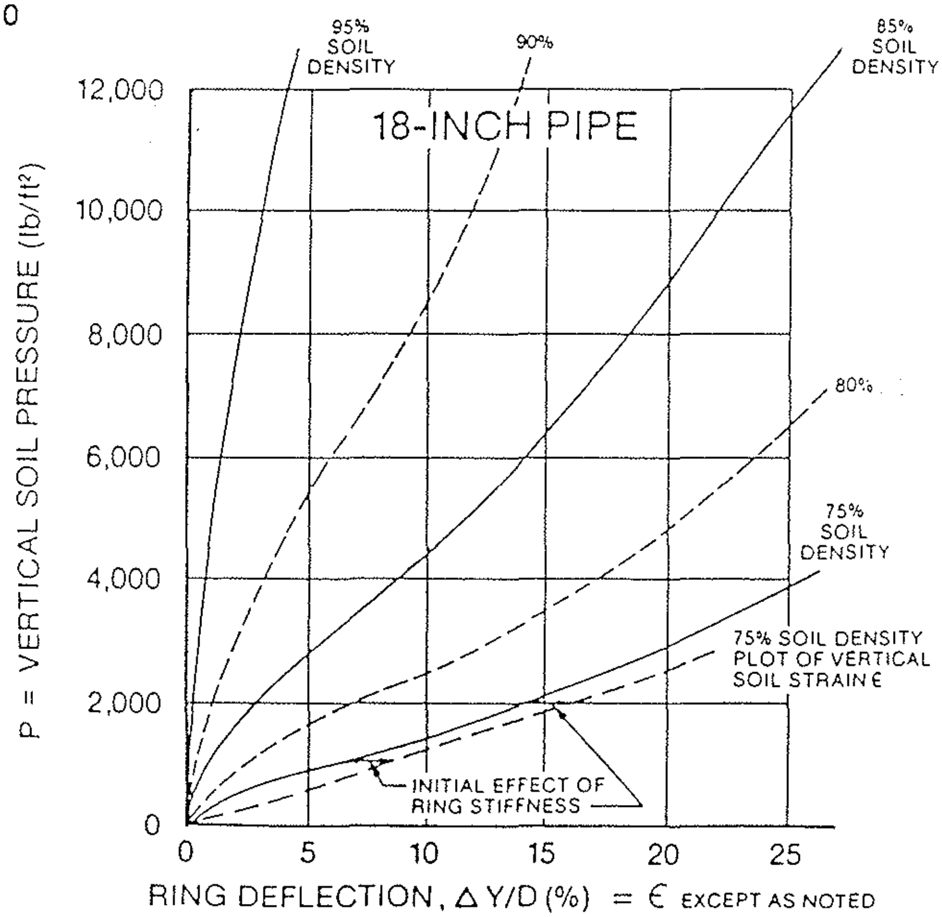
H-20 (32 kip axle load) being applied on 24-inch pipe with cover removed

Figure 9



Sketch of the small Utah State University high vertical soil pressure test cell, showing the setup for testing corrugated plastic pipe. The test can be conducted with or without the "select soil" envelope (gravel).

Figure 10



AH-C

19/17

MSK 7-13-00 GTC MLD GTC

DRISCOPIPE

1. Calculate or estimate the total soil pressure, P_T, at the top of the pipe.

2. Calculate the stress, S_a, in the pipe wall:

$$S_A = \frac{(SDR-1)}{2} P_T$$

3. Based upon the stress, S_a, and the estimated time duration of non-pressurization, find the value of the pipe's modulus of elasticity, E, in psi (approximate value for E is 35,000 psi).

4. Calculate the pipes hydrostatic, critical-collapse differential pressure, P_c

$$P_c = \frac{2E(t/D)^3 (D_{MIN}/D_{MAX})^3}{(1-\mu^2)} \quad \text{or} \quad P_c = \frac{2.32(E)}{SDR^3}$$

Where:
- $(D_{MIN}/D_{MAX}) = 0.95$
- μ = Poission's Ratio = 0.45 for polyethylene pipe
- E = stress and time dependent tensile modulus of elasticity, psi
- E = 35,000 psi (approximate)
- D = Outside Diameter, in.
- t = thickness, in.

5 Calculate the soil modulus, E', by plotting the total external soil pressure, P_T, against a specified soil density to derive the soil strain as shown in the example problem below Figure 7.

6. Calculate the critical buckling pressure at the top of the pipe by the formula:

$$P_{cb} = 0.8\sqrt{(E')(P_c)}$$

Where:
- P_{cb} = Critical buckling soil pressure at the top of the pipe, psi
- E' = Soil Modulus, psi
- P_c = Hydrostatic critical-collapse differential pressure, psi

7. Calculate the Safety Factor: SF = P_{cb} / P_T.

8. The above procedures can be reversed to calculate the minimum pipe DR required for a given soil pressure and an estimated soil density.

In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Attachment D

1/1

Calculation Package G
Seismic Performance Evaluation

Calculation Package G-1
Seismic Hazard Evaluation and Evaluation of Design Ground Motions

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL 0389 **Task #:** 04

Title of Computations: Seismic Hazard Evaluation and Evaluation of Design Ground Motions

Computations By:
Neven Matasovic
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05-04-2000
DATE
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07-26-00
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Neven Matasovic / Project Engineer
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Approved By (PM or Designate):
Edward Kavazanjian
SIGNATURE
07/24/00
DATE
Edward Kavazanjian, Jr / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Neven Matasovic Date: 05 / 03 / 00 Reviewed by: ________ Date: / / (MM DD YY)

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

SEISMIC HAZARD EVALUATION AND EVALUATION OF DESIGN GROUND MOTIONS BRC CAMU HENDERSON, NEVADA

BACKGROUND AND PURPOSE

The proposed BRC CAMU (BRC site) is located in Henderson, Nevada, approximately 10 miles (16 km) southeast of Las Vegas. The approximate site location is shown on Figure 1. Approximate coordinates of the geometric center of the site are 36.0412 North Latitude and -114.9964 West Longitude.

The purpose of the evaluations documented herein is:

- to estimate Peak Horizontal Ground Acceleration (PHGA) at hypothetical bedrock outcrop at the geometric center of the BRC site; and
- to develop representative ground motions for seismic site response analyses at the site.

EVALUATION OF SEISMIC HAZARD

GeoSyntec evaluated the seismic hazard at the site using the most recent United States Geological Survey (USGS) probabilistic seismic hazard maps [Frankel et al., 1996]. For compliance with the State of Nevada regulations, GeoSyntec considered motions with 2 percent probability of being exceeded in 50 years [2% PE in 50 yrs., which is equivalent to the 10% PE in 250 yrs. used by Algermissen et al., 1990]. The Frankel et al. [1996] maps are accessible via the Internet. In addition to bedrock free-field PHGA values, these maps also provide 5 percent damping elastic Spectral Acceleration (S_a) values for spectral periods of 0.2, 0.3 and 1.0 seconds. This data was used to establish the design free-field PHGA and set limits on the target acceleration response spectrum for design ground motions.

Information obtained from the USGS web site is enclosed in Appendix A. The Frankel et al. [1996] 2% PE in 50 yrs. map values indicate that the free-field bedrock PHGA at the site equals to 0.34 g. The same maps indicate that the 0.2, 0.3 and 1.0 seconds spectral acceleration values for 5 percent spectral damping equal 0.82 g, 0.64 g and 0.21 g, respectively. The map-derived spectral acceleration values are plotted on Figure 2. The acceleration response spectrum defined by the above cited (spectral) acceleration points is referred to herein as the "target" spectrum and serves as the basis for evaluation of design ground motions for the site, as explained below.

Frankel et al. [1996] also provide limited disaggregated seismic hazard that can be used to establish the design earthquake magnitude. Tabulated disaggregated seismic hazard (contributing earthquake magnitudes) can be obtained from the USGS web site for the major cities in the area. To evaluate the design earthquake magnitude for the BRC CAMU site, GeoSyntec downloaded the disaggregated seismic hazard data for Las Vegas. Information obtained from the USGS web site is enclosed in Appendix A. The disaggregation of the seismic hazard for Las Vegas indicates that a Moment Magnitude (M_w) 6.5 earthquake governs the seismic hazard in the area.

EVALUATION OF DESIGN GROUND MOTIONS

A suite of representative acceleration time histories, time histories which conform to the target acceleration response spectrum and magnitude of the design earthquake, was selected for use in the seismic response analyses. GeoSyntec selected a suite of three time histories that enveloped the target response spectrum using the following methodology: (i) screen the database of acceleration time histories on the basis of earthquake magnitude to select a reduced set of accelerograms; and (ii) plot the acceleration response spectra of the candidate accelerograms against target acceleration response spectrum and select the representative accelerograms for use in design analyses.

Using the above methodology, GeoSyntec selected the following three candidate accelerograms to represent design ground motions at the BRC site:

- The Cholame Shandon Array No. 5 (355 deg) accelerogram, recorded during the M_w 6.3 Parkfield earthquake. The Parkfield earthquake occurred on 27 June 1966 on a strike-slip fault.

- The Superstition Mountain (135 deg) accelerogram, recorded during the M_w 6.5 Imperial Valley earthquake. The Imperial Valley earthquake occurred on 15 October 1979 on a strike-slip fault.

- The Big Bear Lake - Civic Center Grounds (360 deg) accelerogram, recorded during the M_w 6.7 Big Bear earthquake. The Big Bear earthquake occurred on 28 June 1992 on a strike-slip fault.

The acceleration response spectra of the scaled candidate design accelerograms are plotted against the target acceleration response spectrum on Figure 2. Figure 2 indicates that the acceleration response spectra of Cholame Shandon Array No. 5, Superstition Mountain and Big Bear Lake - Civic Center Grounds accelerograms match and exceed the target acceleration response spectrum in the period range of 0.1 to 1.0 seconds, the period range of interest for seismic design considerations at the site. Therefore, the Cholame Shandon Array No. 5, Superstition Mountain and Big Bear Lake - Civic Center Grounds accelerograms scaled to 0.34 g were selected as the representative acceleration time histories for use in seismic site response analyses at the BRC site.

REFERENCES

Algermissen, S.T., Perkins, D.M., Thenhaus, P.C., Hanson, S.L. and Bender, B.L. [1990], "Probabilistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico," Miscellaneous Field Studies Map MF-2120, U.S. Geological Survey, Reston, Virginia.

Frankel, F., Mueller, C., Bernhard, T., Perkins, D., Leyendecker, Dickman, N., Hanson, S. and Hopper, M. [1996], "Interim National Hazard Maps: Documentation," *Draft Report*, U.S. Geological Survey, Denver, Colorado.

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[illegible]

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Fig. 1

Approximate Site Location

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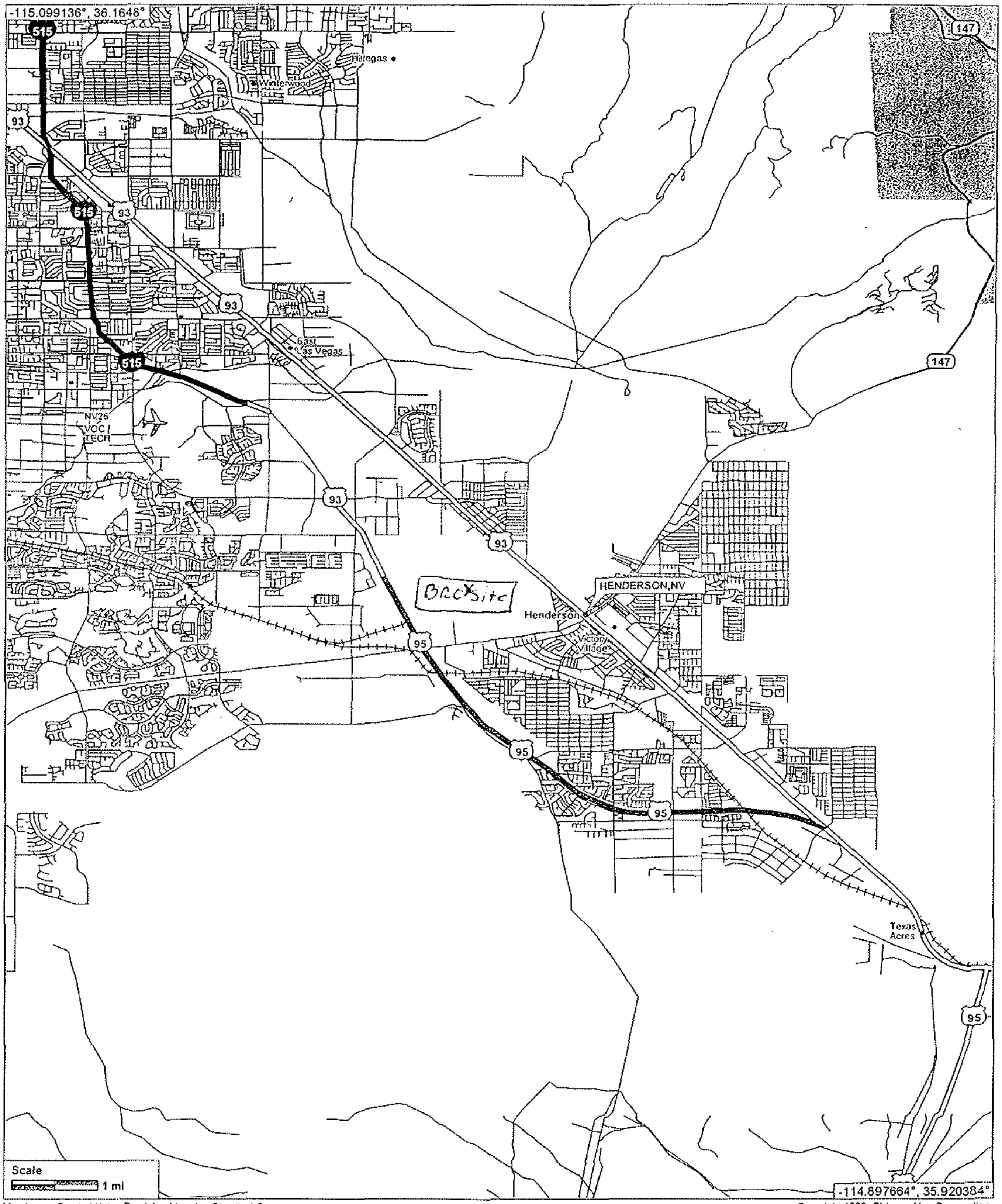

Map Image Created Using Precision Mapping Streets 4.0

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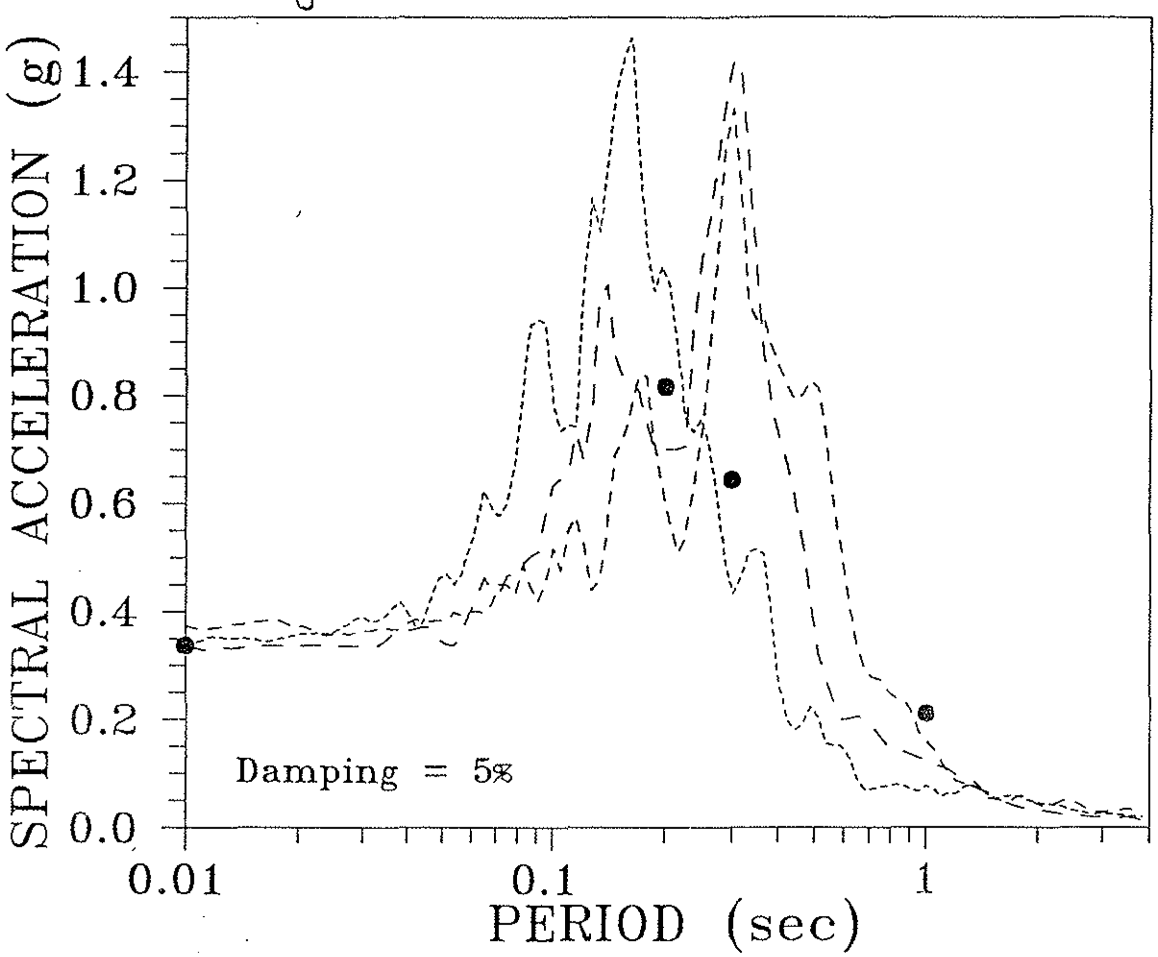
Fig. 2



●●●●● Spectral values from USGS web site (2% PE in 50 yrs.)
------ Superstition Mountain Accelerogram (135 deg)
- - - - Cholame Shandon Array No. 5 Accel. (355 deg)
— — Big Bear Lake – Civic Center Grounds (360 deg)

APPENDIX A

SEISMIC HAZARD FOR THE BRC SITE
AND
DISAGGREGATED SEISMIC HAZARD FOR LAS VEGAS
(2% PE IN 50 YEARS)

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USGS

Home Page

Lat/Lon Lookup page

NATIONAL SEISMIC HAZARD MAPPING PROJECT

USGS, Central Region, Geologic Hazards Team
Golden, Colorado

```
The ground motion values for the requested point:
LOCATION                    36.0412 Lat.  -114.9964 Long.
DISTANCE TO
  NEAREST GRID POINT        4.58287299074001 kms
NEAREST GRID POINT          36.00000  Lat.   -115.0000  Long.
Probabilistic ground motion values, in %g, at this point are:
               10%PE in 50 yr   5%PE in 50 yr   2%PE in 50 yr
  PGA              13.85668        20.33881        33.70004 ✓
0.2 sec SA         31.82535        47.43614        81.64591
0.3 sec SA         27.39132        41.04289        64.31845
1.0 sec SA         8.801240        13.04513        21.02698
- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
```

The program has detected a zero latitude and has assumed the end of valid input data.

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USGS

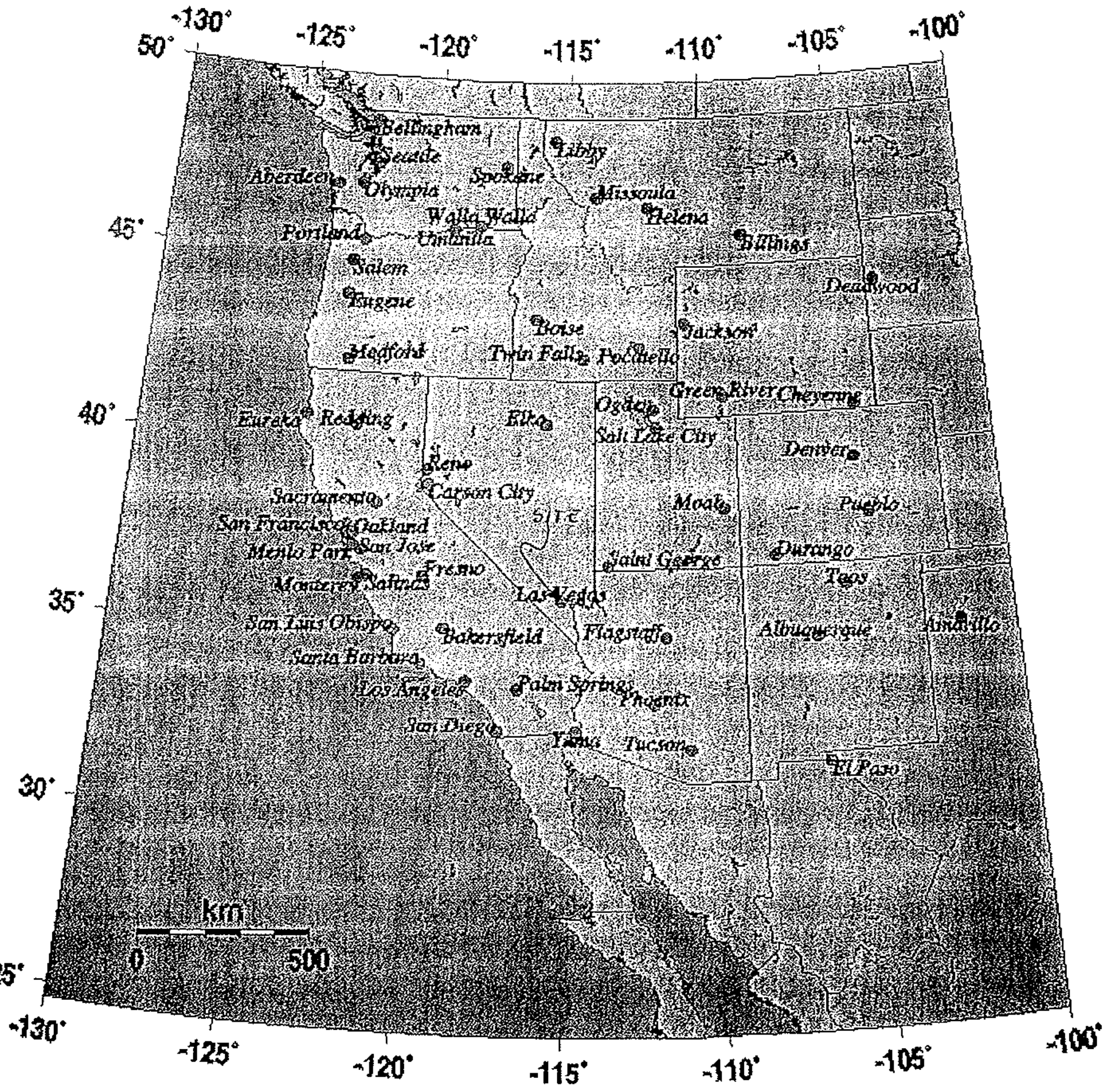
NATIONAL SEISMIC HAZARD MAPPING PROJECT 8/12
Golden, Colorado

24 07/24/00

Please note that the image map on this page is a client-side image map.
YOU WILL NEED A BROWSER WHICH SUPPORTS CLIENT-SIDE IMAGE MAPS TO USE IT!!!
(such as Netscape Navigator 2.0 or Microsoft Internet Explorer 3.0 or the equivalent)

To obtain the four hazard matrices click on the city (red dot). The entries are per cent contribution to hazard. They will sum to 100 per cent for each matrix.

WUS Cities



Calc

http://geohazards.cr.usgs.gov/eq/deagg/lasvegas.html

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Deaggregated Seismic Hazard PE = 2% in 50 years pga
Las_Vegas NV 36.175 deg N 115.136 deg W PGA=0.27640 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	0.000	16.873	15.484	47.082	16.578	0.000
50.	0.000	0.101	0.310	0.353	3.037	0.000
75.	0.000	0.003	0.007	0.051	0.064	0.000
100.	0.000	0.000	0.000	0.005	0.010	0.001
125.	0.000	0.000	0.000	0.002	0.005	0.000
150.	0.000	0.000	0.000	0.001	0.019	0.006
175.	0.000	0.000	0.000	0.000	0.002	0.005
200.	0.000	0.000	0.000	0.000	0.001	0.000

Deaggregated Seismic Hazard PE = 2% in 50 years 1hz
Las_Vegas NV 36.175 deg N 115.136 deg W SA= 0.18000 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	0.000	3.479	7.223	47.502	19.347	0.000
50.	0.000	0.076	0.318	3.177	15.119	0.000
75.	0.000	0.004	0.022	0.510	1.095	0.000
100.	0.000	0.000	0.001	0.050	0.184	0.037
125.	0.000	0.000	0.000	0.023	0.125	0.000
150.	0.000	0.000	0.000	0.008	0.641	0.389
175.	0.000	0.000	0.000	0.006	0.073	0.413
200.	0.000	0.000	0.000	0.002	0.042	0.076
225.	0.000	0.000	0.000	0.005	0.012	0.000
300.	0.000	0.000	0.000	0.000	0.001	0.000
325.	0.000	0.000	0.000	0.000	0.003	0.000
350.	0.000	0.000	0.000	0.001	0.006	0.000
375.	0.000	0.000	0.000	0.001	0.009	0.000
400.	0.000	0.000	0.000	0.002	0.004	0.000
425.	0.000	0.000	0.000	0.002	0.002	0.000
450.	0.000	0.000	0.000	0.001	0.003	0.000
475.	0.000	0.000	0.000	0.001	0.002	0.000
500.	0.000	0.000	0.000	0.001	0.001	0.000

Deaggregated Seismic Hazard PE = 2% in 50 years 3.33 hz (0.3 s)
Las_Vegas NV 36.175 deg N 115.136 deg W SA= 0.55200 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	0.000	10.013	12.605	50.737	17.888	0.000
50.	0.000	0.125	0.256	1.394	6.637	0.000
75.	0.000	0.003	0.008	0.111	0.158	0.000
100.	0.000	0.000	0.000	0.007	0.016	0.002
125.	0.000	0.000	0.000	0.002	0.008	0.000
150.	0.000	0.000	0.000	0.000	0.019	0.004
175.	0.000	0.000	0.000	0.000	0.001	0.003
200.	0.000	0.000	0.000	0.000	0.001	0.000

Deaggregated Seismic Hazard PE = 2% in 50 years 5hz
Las_Vegas NV 36.175 deg N 115.136 deg W SA= 0.62990 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	0.000	13.541	14.177	49.428	16.715	0.000
50.	0.000	0.171	0.247	1.004	4.559	0.000
75.	0.000	0.003	0.005	0.061	0.070	0.000
100.	0.000	0.000	0.000	0.003	0.005	0.001
125.	0.000	0.000	0.000	0.001	0.002	0.000
150.	0.000	0.000	0.000	0.000	0.004	0.001

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USGS

NATIONAL SEISMIC HAZARD MAPPING PROJECT
Golden, Colorado

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LAS VEGAS NV

Hazard matrices

The distance scale on the following two maps is accurate in the east-west direction,
but there is foreshortening in the north-south direction.

Disaggregated Seismic Hazard for 1 second Spectral Acceleration
GIF, PDF, PS
Disaggregated Seismic Hazard for 0.2 second Spectral Acceleration
GIF, PDF, PS

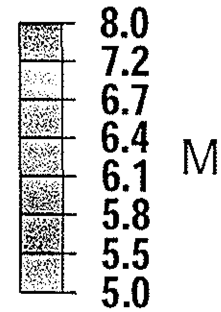
Las Vegas NV Disaggregated Seismic Hazard

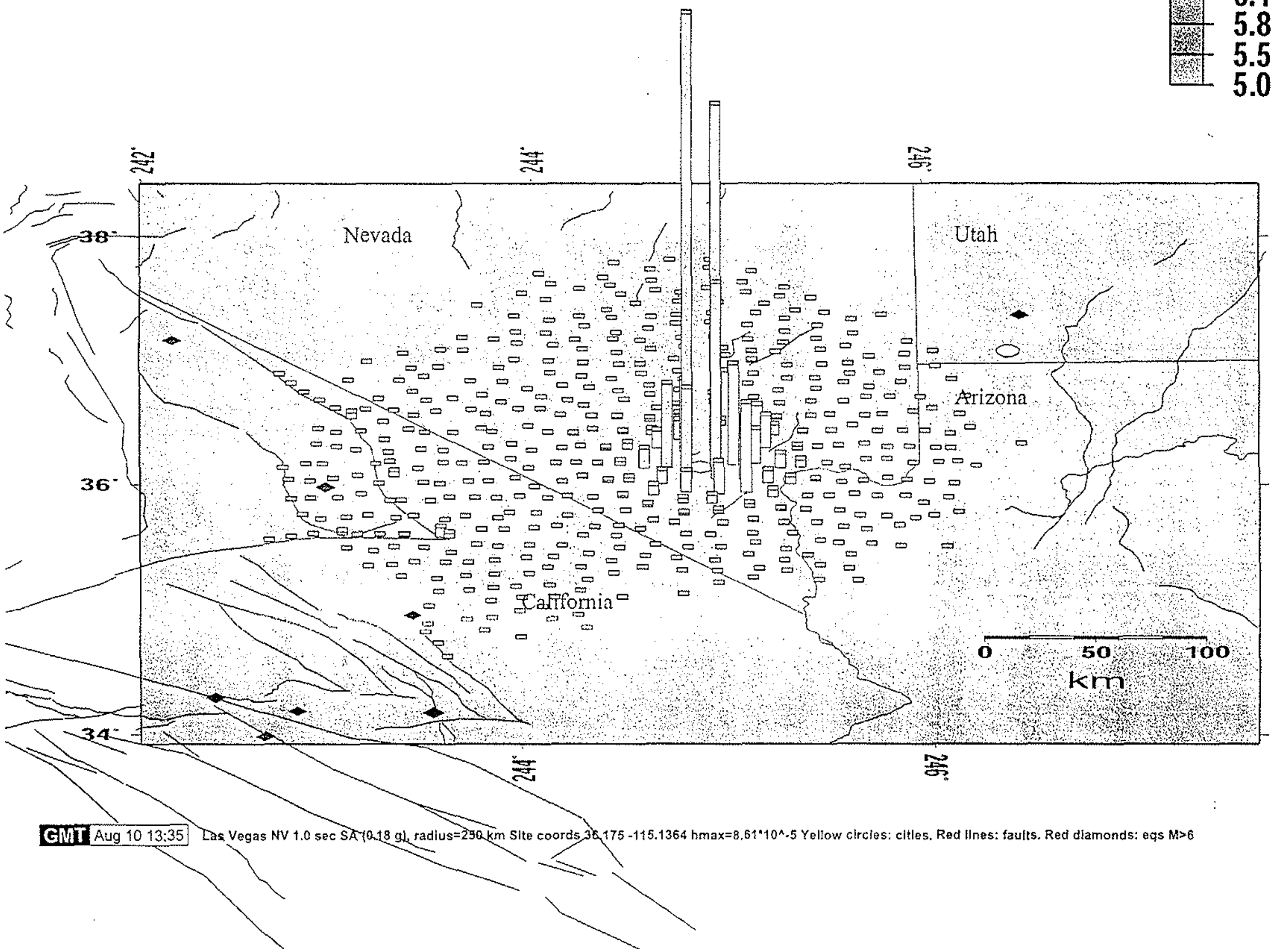
for 1 second Spectral Acceleration, 0.18 g

PE = 2% per 50 yr. Hazard radius 250 km, DeltaR=10 km

Mw: Binned average; equal-area bins.

Most hazard from gridded seismicity; Eglington Fault ca. 5%





GMT Aug 10 13:35 Las Vegas NV 1.0 sec SA (0.18 g), radius=250 km Site coords 36.175 -115.1364 hmax=8.61*10^-5 Yellow circles: cities. Red lines: faults. Red diamonds: eqs M>6

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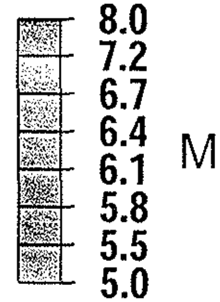
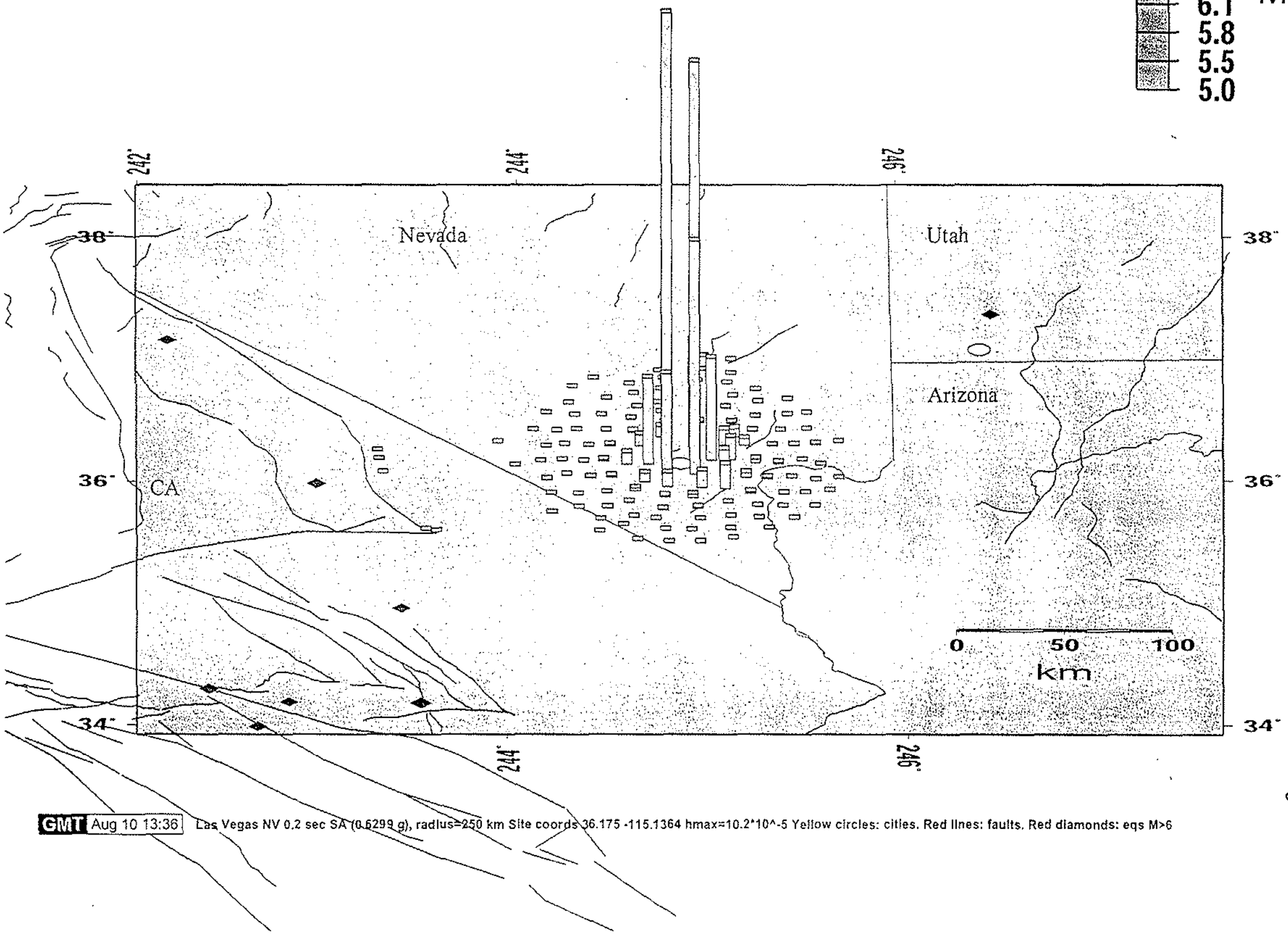
Las Vegas NV Disaggregated Seismic Hazard

for 0.2 second Spectral Acceleration, 0.63 g

PE = 2% per 50 yr. Hazard radius 250 km, DeltaR=10 km
Mw: Binned average; equal-area bins.
Most hazard from gridded seismicity; Eglington Fault ca. 5%





GMT Aug 10 13:36 Las Vegas NV 0.2 sec SA (0.6299 g), radius=250 km Site coords 36.175 -115.1364 hmax=10.2*10^-5 Yellow circles: cities. Red lines: faults. Red diamonds: eqs M>6

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Calculation Package G-2
Site Response Analysis

GeoSyntec Consultants
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project #:** HL0389 **Task #:** 04

Title of Computations: Seismic Site Response Analysis

Computations By:

SIGNATURE 26 July 2000
DATE

Michael E. Kalinski / Senior Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE 07-26-00
DATE

Neven Matasovic / Project Engineer
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE 7-26-00
DATE

Geoff Smith/Staff Engineer
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE 7-26-2000
DATE

Michael E. Kalinski / Senior Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE 07/26/00
DATE

Edward Kavazanjian, Jr. / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions:

No.	Date	Description	Approval

PURPOSE

One-dimensional site response analyses were performed for the BRC Landfill in Henderson, Nevada using the SHAKE91 computer program [SHAKE91, 1992]. Analyses were performed to assess the dynamic site response of the compacted landfill to three design earthquake seismograms. This ~~memo~~ Calculation Package provides the details of the analyses. Synthetic accelerograms for the ground surface derived from these analyses were used in the companion computation package titled "Site-Specific Seismic Deformation Charts."

SUMMARY OF SHAKE91

SHAKE91 is a computer program for conducting an equivalent linear seismic response analysis of a horizontally layered soil deposit [SHAKE91, 1992]. SHAKE91 was originally written in 1972, but has undergone several modifications and revisions to its present form. SHAKE91 performs a frequency-domain analysis of a one-dimensional soil column subjected to earthquake shaking in bedrock at its base. SHAKE91 is based on the assumption that the earthquake is a vertically propagating shear wave.

The three main sets of input parameters for SHAKE91 are 1) information about the soil profile, 2) information about how the dynamic properties of the soil (shear modulus and damping) vary with strain and 3) input ground motion. Information about the soil profile includes layer thickness, small-strain shear modulus (G_{max}) and unit weight. Information about how the dynamic properties of the soil vary with strain are provided using curves of modulus reduction (G/G_{max}) and damping versus shearing strain. Different curves are used for the different types of material (i.e. sand, clay, rock and waste) in the profile. Seismograms recorded at earthquake stations near the site being evaluated are typically used as input ground motion.

To perform the site response analysis, SHAKE91 implements an iterative procedure using an 'equivalent linear' method. In this method, the linear response of the soil column to earthquake shaking is calculated using initial estimates for G_{max} and material damping. The resulting shear strains are plotted against the modulus reduction and damping curves and the dynamic soil properties are adjusted accordingly. The linear response of the soil column is re-calculated using the adjusted parameters. The re-calculated shear strains are used to re-adjust the dynamic soil properties and the linear response of the soil column is calculated again. This procedure is repeated until the difference in dynamic soil properties between iterations is sufficiently small. The final result is typically expressed in terms of peak accelerations calculated at the surface of the model.

PROBLEM APPROACH

Site response analyses were performed using three seismogram records. Each input record was adjusted to a maximum amplitude of 0.337 g. This value is the predicted peak ground acceleration (PGA) for Las Vegas, Nevada with a 2% probability of occurrence in a 50-year period and is based on probabilistic analysis of available earthquake data by the United States Geological Survey [USGS, 2000]. The three seismograms (Parkfield, Superstition and Big Bear) were selected based on similarities between their response spectra and the target acceleration response spectrum as described in the companion computation package titled "Seismic Hazard Evaluation and Evaluation of Design Ground Motions."

Shear wave velocity and layer thickness data were derived from SASW testing performed by Dr. Barbara Luke at the University of Nevada at Las Vegas on February 25, 2000. SASW testing was performed on native material as shown in Fig. 1. The resulting shear wave velocity data are summarized in Table 1. These data were integrated with soil boring data [Converse Consultants, 1999] to identify units of silty sand (SM), lean clay (CL), waste and bedrock within the profiles. The interpreted shear wave velocity profile is included in Fig. 2 (GeoSyntec, 2000).

Estimated unit weights for the native soil spanned a reasonable range of 120 to 130 pcf. These values were varied according to shear wave velocity. Higher values were assigned to layers with higher shear wave velocities.

For characterizing bedrock, it was assumed that the bedrock material was a hard, competent rock with a relatively high shear wave velocity (7000 ft/s) and high unit weight (140 pcf). These values are comparable to typical values observed in other hard, competent rocks.

Separate modulus reduction and damping curves were used for the materials encountered in the two profiles. Reduction curves for the clay and sand were taken from research performed by Vucetic and Dobry [1991] where the effects of plasticity index on cyclic response were investigated for a wide variety of soil types and studies. To apply the Vucetic and Dobry data to this analysis, the plasticity indices of the silty sand and lean clay were assumed to be 0% and 15%, respectively. Modulus reduction curves corresponding to silty sand were used for the

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compacted waste based on the assumption that the compacted waste will be classifiable as a silty sand.

Two SHAKE91 analyses were performed on hypothetical profiles representative of the final design of the new landfill. These profiles, shown in Fig. 3, consist of 1) the existing native material overlain by 30 ft of compacted waste and 2) 60 ft of waste embedded 30 ft into the native soil. For these analyses, shear wave velocities estimated for the native material were modified to account for the effect of increased confining stress on increased shear wave velocity. As shown in Table 2, shear wave velocities of the native material were used, but shear wave velocities less than 800 ft/s were set equal to 800 ft/s to account for the increased confining stress. ← North mesa ← South mesa

To characterize the compacted waste material, shear wave velocities and unit weights of 1000 ft/s and 120 pcf, respectively, were assumed. These are reasonable values for silty sands that are compacted at 90 to 95% of optimum water content, which is the anticipated nature of the compacted waste.

Based on results from SASW testing, the depth to bedrock could not be conclusively estimated. Therefore, three sets of SHAKE91 analyses were performed by varying the thickness of the silty sand layer present at the bottom of the native material profile (Fig. 2). Silty sand layer thicknesses of 50, 100 and 150 ft were used for the analyses.
Corresponding to bedrock either 201, 251, & 301 ft bgs EK or 231, 281, & 331 ft below top of landfill. EK 07/26/00

RESULTS

Site response analyses were performed for the two scenarios (unembedded and embedded waste) using the procedures described in the previous section. A sample SHAKE91 input file is attached to this memorandum. Peak accelerations at the surface of the landfill and native material are summarized in Table 3. The calculated accelerations are all less than 0.50 g. Embedding the compacted waste causes a minor (less than 10%) reduction in predicted surface accelerations.

Calculated profile periods are summarized in Table 4. Profile periods for modeling with the 150-ft thick bottom soil layer are all greater than 1.09 s. Since this period is longer than the maximum-amplitude period on typical response spectra, it was believed that thicker bottom layers would produce longer periods, less amplification and smaller accelerations. Therefore, modeling with layer thicknesses greater than 150 ft was not considered necessary.

Written by: Michael E. Kalinski Date: 07 / 26 / 00 Reviewed by: Date: 07 / 26 / 00

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

EK 07/26/00

REFERENCES

Converse Consultants, (1999), "Preliminary Geotechnical and Geologic Investigation, Industrial Non-Hazardous Disposal Facility, Basic Management Incorporated, Clark County, Nevada," Converse Project No. 99-33437-01, October 27, 1999, Converse Consultants, Las Vegas, Nevada.

GeoSyntec, (2000), "Preliminary Results of SASW Testing, BRC Landfill, Henderson, Nevada," interoffice memorandum from Michael E. Kalinski to Jay McKelvey, 7 April 2000.

SHAKE91, (1992), "User's Manual for SHAKE91," Center for Geotechnical Modeling, Department of Civil & Environmental Engineering, University of California at Davis.

USGS, (2000), "National Seismic Hazard Mapping Project," USGS, Central Region, Geologic Hazards Team, http://geohazards.cr.usgs.gov/eq/.

Written by: Michael E. Kalinski Date: 07 / 26 / 00 Reviewed by: ____ Date: 7 / 26 / 00

MM DD YY

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

EK 07/26/00

TABLE 1

SHEAR WAVE VELOCITY PROFILES FROM SASW MEASUREMENTS PERFORMED AT THE BRC LANDFILL, HENDERSON NEVADA

Thickness (ft)	Depth (ft)	Shear Wave Velocity (ft/s)	Material Type
0.98	0.98	426	SM
1.31	2.29	754	SM
10.8	13.09	984	SM
3.28	16.37	1082	SM
16.4	32.77	1279	SM
7.28	40.05	1312	SM
81.9	121.95	1115	CL
29.5	151.45	1246	CL
Half-space	-	1968	SM

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TABLE 2
SHEAR WAVE VELOCITY PROFILES USED FOR SHAKE91 ANALYSIS
NATIVE MATERIAL WITH 30 FT OF OVERBURDEN WASTE

a) 30 ft waste placed on top of native material

Thickness (ft)	Depth (ft)	Shear Wave Velocity (ft/s)	Material Type
30.00	30.00	1000	compacted waste
0.98	30.98	800	SM
1.31	32.29	800	SM
10.8	43.09	984	SM
3.28	46.37	1082	SM
16.4	62.77	1279	SM
7.28	70.05	1312	SM
81.9	151.95	1115	CL
29.5	181.45	1246	CL
50, 100 or 150	231.45, 281.45 or 331.45	1968	SM
Half-space	-	7000	bedrock

b) 30-ft excavation filled with waste to elevation of 30 ft above native ground surface (60 ft total waste thickness)

Thickness (ft)	Depth (ft)	Shear Wave Velocity (ft/s)	Material Type
30.00	30.00	1000	compacted waste
30.00	60.00	1000	compacted waste
2.77	62.77	1279	SM
7.28	70.05	1312	SM
81.9	151.95	1115	CL
29.5	181.45	1246	CL
50, 100 or 150	231.45, 281.45 or 331.45	1968	SM
Half-space	-	7000	bedrock

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TABLE 3
PEAK GROUND SURFACE ACCELERATIONS CALCULATED USING SHAKE91

a) 30 ft Compacted Waste over Native Material

Earthquake	Bottom soil layer = 50 ft thick	Bottom soil layer = 100 ft thick	Bottom soil layer = 150 ft thick
Big Bear	0.47 g	0.50	0.46
Parkfield	0.43	0.47	0.48
Superstition	0.40	0.35	0.31
AVERAGE	**0.43**	**0.44**	**0.42**

b) 60 ft Compacted Waste Embedded into Native Material

Earthquake	Bottom soil layer = 50 ft thick	Bottom soil layer = 100 ft thick	Bottom soil layer = 150 ft thick
Big Bear	0.41 g	0.44	0.41
Parkfield	0.40	0.44	0.42
Superstition	0.38	0.34	0.31
AVERAGE	**0.40**	**0.41**	**0.38**

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Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

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TABLE 4
SOIL PROFILE PERIODS CALCULATED USING SHAKE91

a) 30 ft Compacted Waste Over Native Material

Earthquake	Bottom soil layer = 50 ft thick	Bottom soil layer = 100 ft thick	Bottom soil layer = 150 ft thick
Big Bear	0.95 s	1.10	1.22
Parkfield	0.94	1.05	1.19
Superstition	0.91	1.01	1.09

b) 60 ft Compacted Waste Embedded into Native Material

Earthquake	Bottom soil layer = 50 ft thick	Bottom soil layer = 100 ft thick	Bottom soil layer = 150 ft thick
Big Bear	0.98 s	1.11	1.22
Parkfield	0.95	1.06	1.21
Superstition	0.92	1.01	1.09

Written by: Michael E. Kalinski Date: 07 / 26 / 00 Reviewed by: Date: 7 / 26 / 00

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

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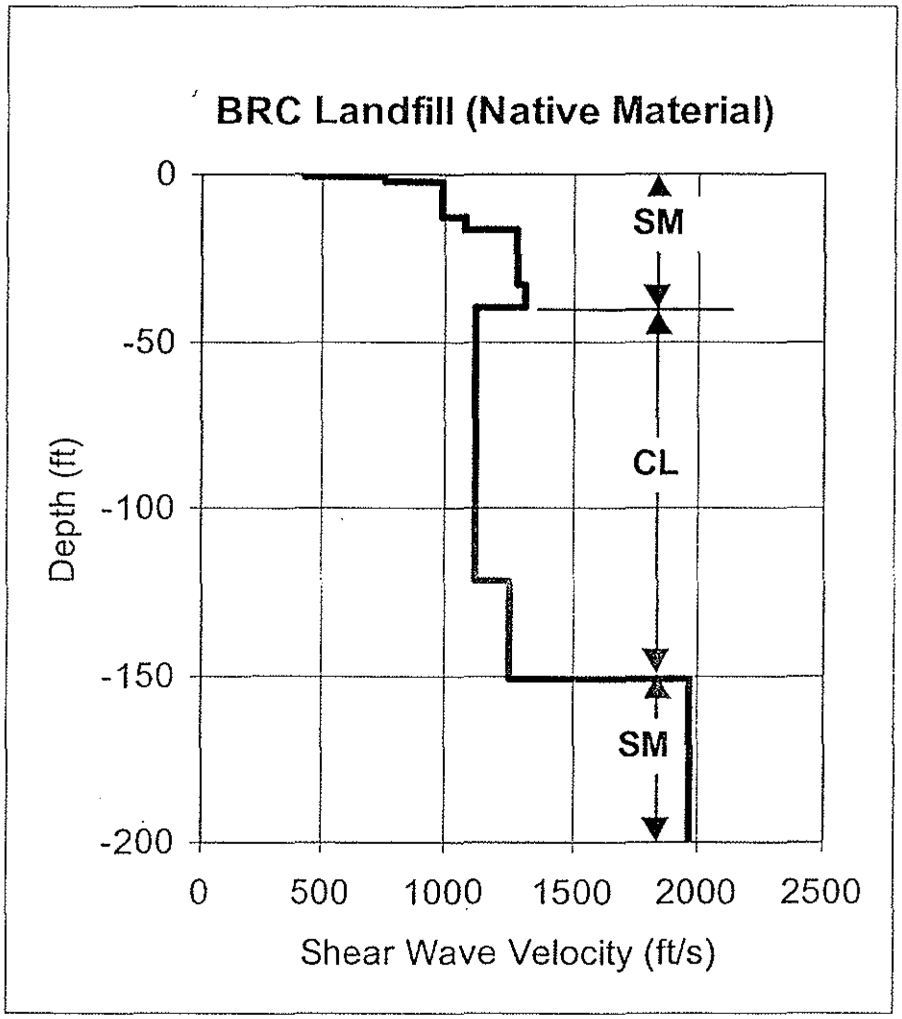

FIGURE 2. **SHEAR WAVE VELOCITY PROFILE OF NATIVE MATERIAL FROM SASW DATA**

Written by: [signature] Date: __/__/__ (YY/MM/DD) Reviewed by: [signature] Date: 7/26/00 (YY/MM/DD)

Client: ____ Project: ____ Project/Proposal No.: ____ Task No.: ____

FIG 3 SOIL PROFILES USED FOR NEW COMPACTED LANDFILL.

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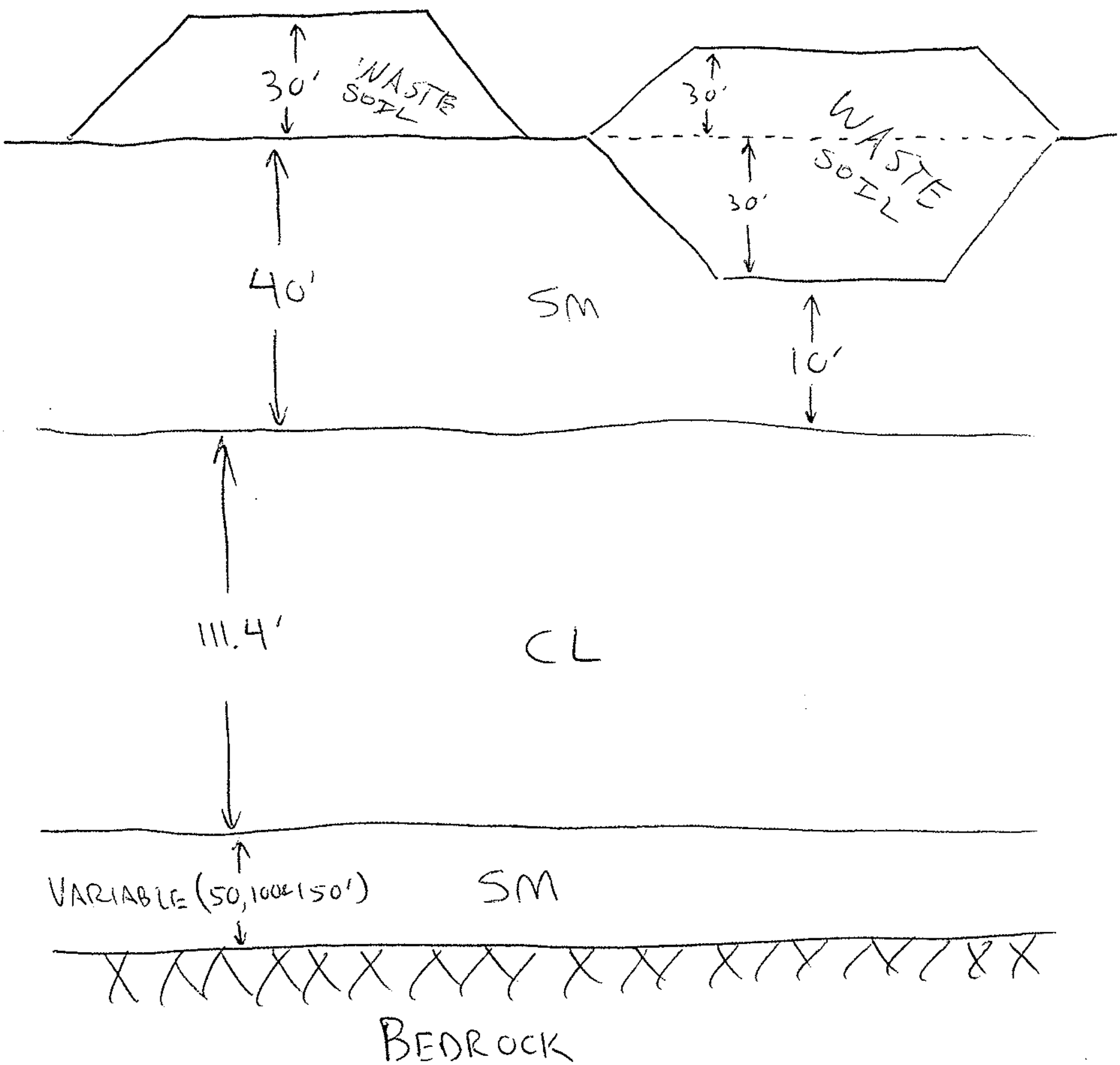
CASE 1 - NOT EMBEDDED

CASE 2 - EMBEDDED



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ATTACHMENT

SAMPLE SHAKE91 INPUT FILE

(Embedded Waste, Big Bear Earthquake, Bottom Soil Layer Thickness = 100 ft)

embedN2

option 1 - dynamic soil properties - (max is thirteen):
```
    1
    3
   10  #1 Modulus for Clay (PI=15) (Vucetic and Dobry, 1991)
    0.0001  0.000316     0.001   0.00316      0.01    0.0316
 0.1      0.316
        1.        3.16
      1.000      1.000     0.995     0.936     0.818     0.640     0
.405      0.210
     0.095      0.034
   10  #1 Damping for Clay (PI=15) (Vucetic and Dobry, 1991)
    0.0001  0.000316     0.001   0.00316      0.01    0.0316
 0.1      0.316
        1.        3.16
       0.85      1.04      1.55      2.58     4.635      7.77     1
1.67    16.085
     20.12     23.17
   10  #2 Modulus for Sand (PI=0) (Vucetic and Dobry, 1991)
   0.0001   0.000316     0.001   0.00316      0.01    0.0316
 0.1      0.316
      1.0        3.16
    1.000      1.000     0.964     0.870     0.712     0.474     0
.253      0.103
    0.028      0.004
    9  #2 Damping for Sand (PI=0) (Vucetic and Dobry, 1991)
   0.0001   0.000316    0.0010    0.0032      0.01    0.0316
 0.1      0.316
      1.0        3.16
     0.85       1.04      1.66     2.999      5.48     10.01     1
5.40     20.23
    23.94     26.17
    8  #3 Modulus for Rock
    0.0001    0.0003     0.001     0.003      0.01      0.03
 0.1       1.0
     1.000     1.000    0.9875    0.9525     0.900     0.810     0
.725      0.550
    5  #3 Damping for Rock
    0.0001     0.001      0.01       0.1       1.0
       0.4       0.8       1.5       3.0       4.6
    3    1    2    3
Option 2 -- Soil Profile
    2
    1    8      Profile embedN2 -- 60 ft waste embedded 30 ft
    1    2         30.00                    .010      .120      1000
    2    2         30.00                    .010      .120      1000
    3    2          2.77                    .010      .125      1279
    4    2          7.28                    .010      .125      1312
    5    1          81.9                    .020      .125      1115
```

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```
    6    1          29.5                     .020      .125      1246
    7    2         100.0                     .010      .130      1968
    8    3                                   .005      .140      7000
Option 3 -- input motion:
    3
 2000 4096        .01 bigbear.acc                      (8f9.6)
              0.337       50.0    2    8
Option 4 -- sublayer for input motion (within (1) or outcropping (
0):
    4
    8    0
Option 5 -- number of iterations & ratio of avg strain to max stra
in
    5
    0   12        0.6
Option 6 -- sublayers for which accn time histories are computed &
 saved:
    6
    1    2    3    4    5    6    7    8
    1    1    1    1    1    1    1    1
    1    0    0    0    0    0    0    0
Option 7 -- sublayer for which shear stress or strain are computed
 & saved:
    7
    8    1    1      2000            -- stress at top of bedrock
    8    0    1      2000            -- strain at top of bedrock
Option 10 -- compute & save amplification spectrum:
   10
    8    0    1    0      0.25 -- ground surface/rock outcrop
execution will stop when program encounters 0
    0
```

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FIG. 1

Calc 7-26
10/15

PARSONS
ENGINEE
SCIENCE,
PASADENA, CAL
JOB NO. 7

Approximate Boring Locations (… 1/21/90 … No. 2, File # 43701002)

Calculation Package G-3
Seismic Deformation Charts

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 04

Title of Computations: Site-Specific Seismic Deformation Charts

Computations By:

SIGNATURE

26 ~~25~~ July 2000
DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE

07-26-00
DATE

Neven Matasovic / Project Engineer
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE

26 JULY 2000
DATE

Michael E. Kalinski / Senior Staff Engineer
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE

7-26-00
DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE

07/26/00
DATE

Edward Kavazanjian, Jr. / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 07 / 25 / 00 Reviewed by: ERIC 7.26 Date: __/__/__
MM DD YY MM DD YY

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

SITE-SPECIFIC SEISMIC DEFORMATION CHARTS
BRC CAMU
HENDERSON, NEVADA

EK 07/26/00

BACKGROUND AND PURPOSE

The purpose of the analyses documented herein is to develop site-specific seismic deformation charts for the proposed BRC CAMU in Henderson, Nevada. Due to different geometry, the deformation response for the borth North Mesa and the South Mesa were evaluated. The North Mesa is characterized by a 30 ft (10 m) contaminated soil fill placed on native ground. The South Mesa is characterized by a 60 ft (20 m) thick comtaiminated soil fill that is embedded 30 ft (10 m) below native ground surface. The analyses presented herein are based on results documented in a companion calculation package (seismic site response analyses).

METHOD OF ANALYSIS

The site-specific seismic deformation charts were developed using the computer program YSLIP_PM [Yan et al. 1996]. YSLIP_PM is based upon the Newmark [1965] seismic deformation analysis.

The Newmark method assumes a rigid block on a plane. The block base is subjected to earthquake-induced accelerations. YSLIP_PM calculates the permanent displacement of the rigid block from pseudostatically-evaluated yield accelerations and block base accelerations evaluated in a site response analysis.

ANALYSIS

The analysis consisted of inputting site response acceleration time histories generated by SHAKE91 [Idriss and Sun 1992] into YSLIP_PM. Since bedrock is deep below the ground surface, the depth to the bedrock was varied. Depth to bedrock was assumed to be 201, 251, and 301 ft below the ground surface and 231, 281, and 331 feet below the top of the the landfill.

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A suite of three accelerograms were used to represent the conditions at the BRC CAMU. The three accelerograms were:

- The Cholame Shandon Array No. 5 (355 deg.) accelerogram, recorded during the M_w 6.3 Parkfield earthquake. The Parkfield earthquake occurred on 27 June 1996 on a strike-slip fault.

- The Superstition Mountain (135 deg.) accelerogram, recorded during the M_w 6.5 Imperial Valley earthquake. The Imperial Valley earthquake occurred on 15 October 1979 on a strike-slip fault.

- The Big Bear Lake - Civic Center Grounds (360 deg.) accelerogram, recorded during the M_w 6.7 Big Bear earthquake. The Big Bear earthquake occurred on 28 June 1992 on a strike-slip fault.

The three depths to bedrock for each accelerogram for native and landfill site conditions were analyzed to evaluate a suite of site-specific seismic deformation responses.

RESULTS

The results of evaluations documented herein are plotted on the charts shown in Figure 1 (North Mesa characterized by 30 ft high landfill) and Figure 2 (South Mesa characterized by a 60 ft waste soil thickness embedded 30 ft below native ground suface). Inspection of Figures 1 and 2 indicate that the North Mesa displacement response is more critical than the South Mesa, although both ~~locations~~ responses are similar. The results further indicate that the largest seismic displacement response is calculated by applying the Parkfield record to the base of the 331 ft (101 m) high soil (soil/landfill) column.

Written by: Geoff L. Smith GLS Date: 07 / 25 / 00 (MM DD YY) Reviewed by: MCL 7-26 Date: __/__/__ (MM DD YY)

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

EK 07/26/00

REFERENCES

Idriss, I.M., Sun, J. I. (1992), "User's Manual for SHAKE91, A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits," Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, California, November 1992.

Newmark, N.M. (1965), "Effects of Earthquakes on Dams and Embankments," Geotechnique, Vol. 15, No. 2, pp. 139-160.

Yan, L., Matasovic, N., and Kavazanjian, E. (1996), "YSLIP_PM - User's Guide," Research Report, GeoSyntec Consultants, Huntington Beach, California

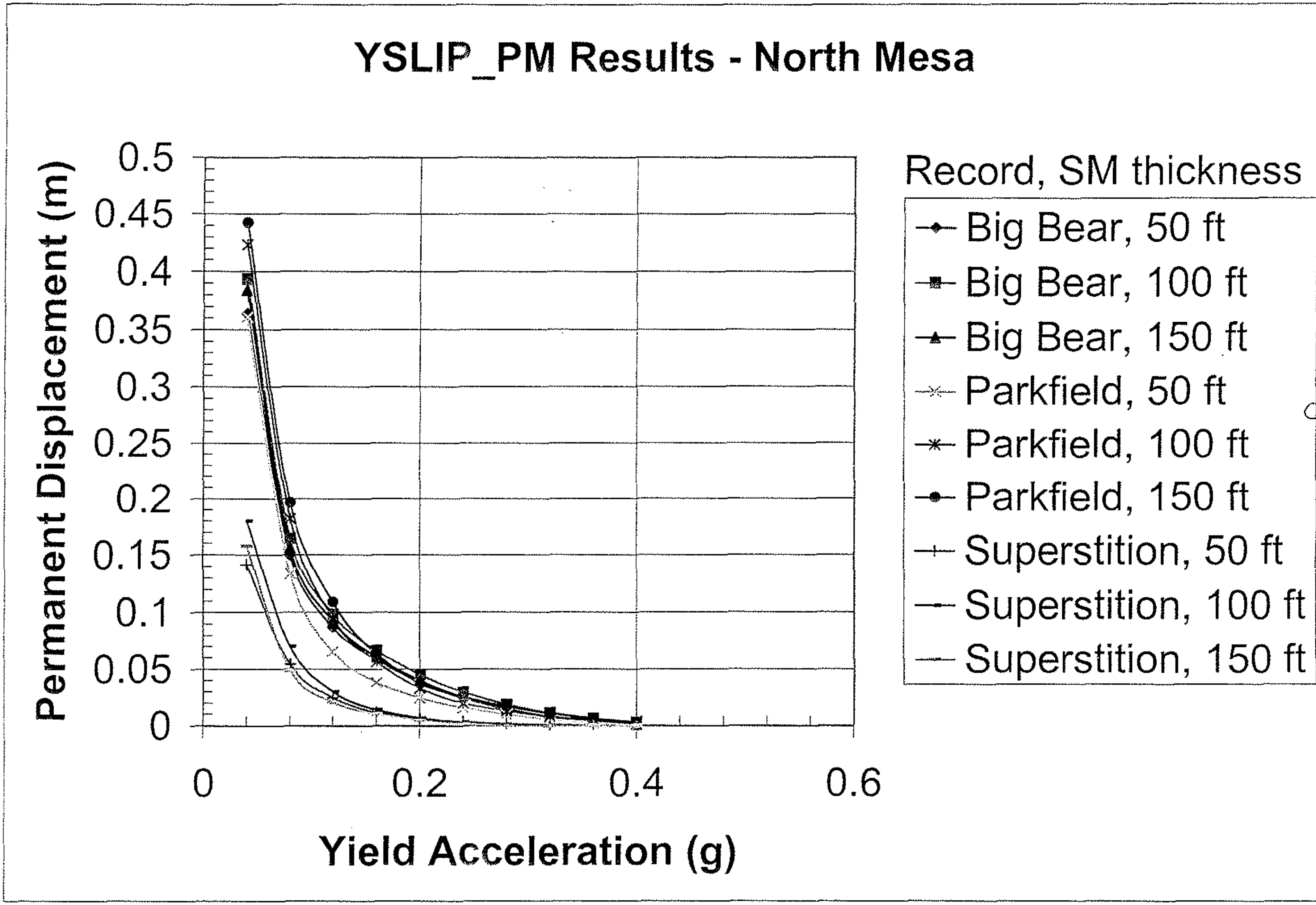
YSLIP_PM Results - North Mesa
Permanent Displacement (m)
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
0.2
0.4
0.6
Yield Acceleration (g)
Record, SM thickness
Big Bear, 50 ft
Big Bear, 100 ft
Big Bear, 150 ft
Parkfield, 50 ft
Parkfield, 100 ft
Parkfield, 150 ft
Superstition, 50 ft
Superstition, 100 ft
Superstition, 150 ft

Figure 1

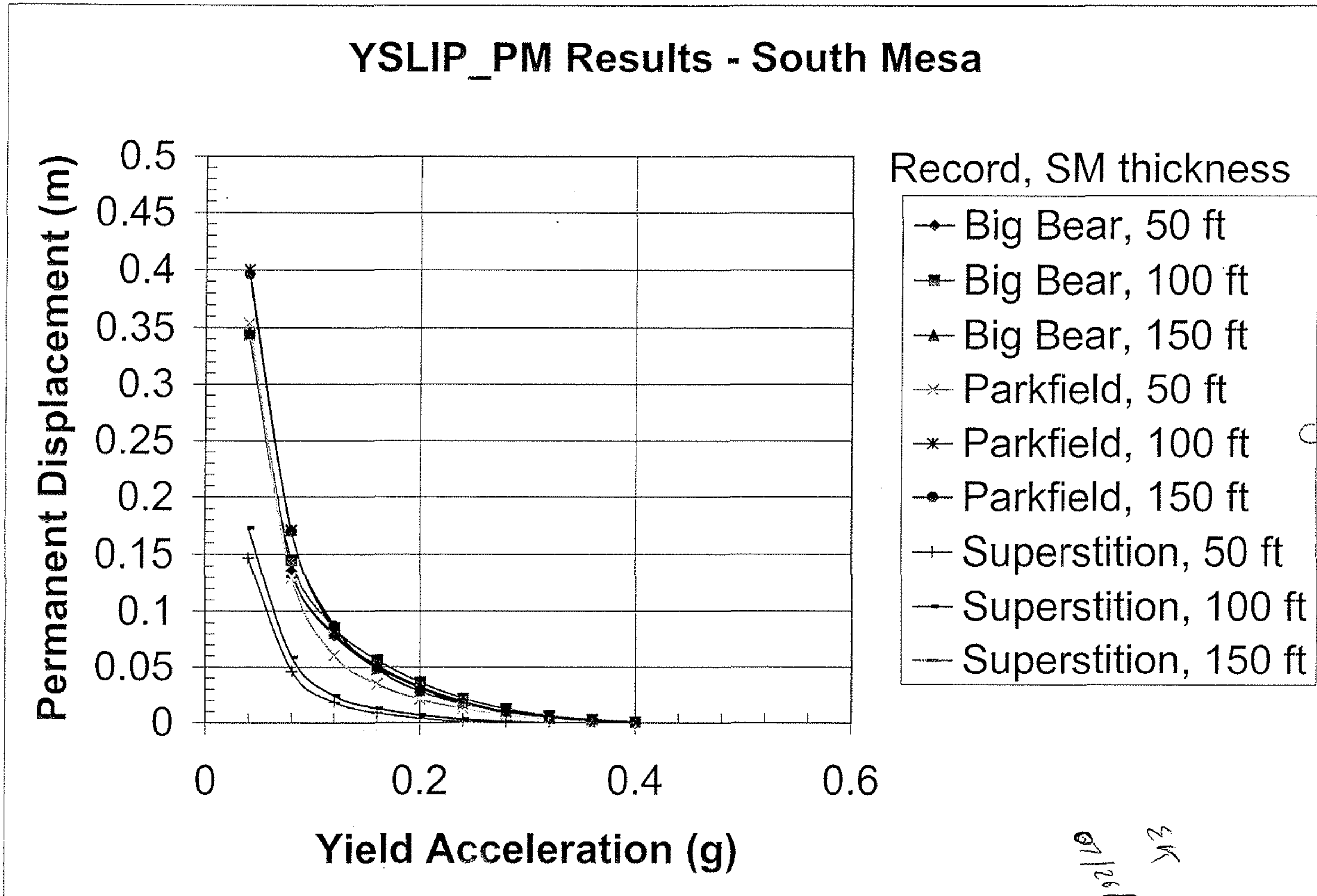

Figure 2

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MEK 7-26

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*************************************************
**                                             **
**                 YSLIP_PM                    **
**        (VERSION 2.2, JANUARY 1996)          **
**                                             **
**   A COMPUTER PROGRAM FOR SIMULATION OF      **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON      **
**   AN INCLINED PLANE AND CALCULATION OF      **
**   PERMANENT DISPLACEMENTS OF THE BLOCK      **
**                                             **
**                    BY                       **
**                                             **
**                Liping Yan                   **
**             Neven Matasovic                 **
**          Edward Kavazanjian, Jr.            **
**                                             **
*************************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Parkfield Record 60 ft, SM thickness = 50 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10     1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         p6050.out
CARD  7: NIDH  KREAD     NP       NLINE     DT       APH
         2     2         4096     512       0.02     0.4001
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP       NLINE     DT       APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .400062
INPUT HORI. ACC. TIMES A FACTOR = 1.000095 TO GET APH =  .400100


           *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .3310 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3520 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .4161 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3537 m     *

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .2446 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0984 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .3471 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1303 m     *

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .1719 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0473 m
```

MSK 7-26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .2833 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0604 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .1207 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0249 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .2267 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0349 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0758 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0111 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .1724 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0209 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0421 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0034 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .1231 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0128 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0156 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0008 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0817 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0072 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0004 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0430 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0032 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0155 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0008 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m
```

NSK 7.26

```
*********************************************
**                                         **
**                YSLIP_PM                 **
**       (VERSION 2.2, JANUARY 1996)       **
**                                         **
**   A COMPUTER PROGRAM FOR SIMULATION OF  **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON  **
**   AN INCLINED PLANE AND CALCULATION OF  **
**   PERMANENT DISPLACEMENTS OF THE BLOCK  **
**                                         **
**                   BY                    **
**                                         **
**               Liping Yan                **
**             Neven Matasovic             **
**         Edward Kavazanjian, Jr.         **
**                                         **
*********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Parkfield Record 60 ft, SM thickness = 100 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         p60100.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.02      0.4395
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .439500
INPUT HORI. ACC. TIMES A FACTOR = 1.000000 TO GET APH =  .439500


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3957 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3981 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .4384 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3999 m   *

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3226 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1384 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .3716 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1708 m   *

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2582 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0690 m
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YMSK 7-7C

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .3115 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0834 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .1966 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0386 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .2535 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0480 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .1459 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0215 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .1977 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0280 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0982 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0108 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .1518 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0163 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0610 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0050 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .1085 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0100 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0285 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0018 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0676 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0055 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0076 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0003 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0373 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0024 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0115 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0006 m
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MSK 7-26

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**********************************************
**                                          **
**               YSLIP_PM                   **
**       (VERSION 2.2, JANUARY 1996)        **
**                                          **
**   A COMPUTER PROGRAM FOR SIMULATION OF   **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON   **
**   AN INCLINED PLANE AND CALCULATION OF   **
**   PERMANENT DISPLACEMENTS OF THE BLOCK   **
**                                          **
**                  BY                      **
**                                          **
**              Liping Yan                  **
**           Neven Matasovic                **
**        Edward Kavazanjian, Jr.           **
**                                          **
**********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Parkfield Record 60 ft, SM thickness = 150 ft
CARD  3: KPAR  N    KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10   1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         p60150.out
CARD  7: NIDH  KREAD    NP       NLINE     DT       APH
         2     2        4096     512       0.02     0.4249
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD    NP       NLINE     DT       APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .424949
INPUT HORI. ACC. TIMES A FACTOR =  .999885 TO GET APH =  .424900


           *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .4676 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .3960 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .4437 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .3740 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3909 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1531 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .3734 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1696 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3212 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0811 m
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MSK 7-76

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .3081 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0873 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .2540 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0457 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .2504 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0476 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1972 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0281 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1949 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0266 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1431 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0172 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1433 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0157 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0978 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0100 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1010 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0093 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0573 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0049 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0613 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0048 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0272 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0017 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0295 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0018 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0053 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0002 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0056 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0002 m
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YMSK 7-26

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**********************************************
**                                          **
**                YSLIP_PM                  **
**        (VERSION 2.2, JANUARY 1996)       **
**                                          **
**   A COMPUTER PROGRAM FOR SIMULATION OF   **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON   **
**   AN INCLINED PLANE AND CALCULATION OF   **
**   PERMANENT DISPLACEMENTS OF THE BLOCK   **
**                                          **
**                   BY                     **
**                                          **
**               Liping Yan                 **
**            Neven Matasovic               **
**         Edward Kavazanjian, Jr.          **
**                                          **
**********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Parkfield Record 30 ft, SM thickness = 50 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         p3050.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.02      0.4298
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =   .429801
INPUT HORI. ACC. TIMES A FACTOR =   .999998 TO GET APH =   .429800


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .3252 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .3609 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .4408 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .3555 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .2552 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0970 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .3711 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .1340 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1973 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0509 m
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MSK 7-26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .3112 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0656 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .1451 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0291 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .2536 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0387 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .1006 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0151 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .1981 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0246 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0625 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0062 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .1495 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0158 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0323 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0020 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .1067 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0098 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0092 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0004 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0663 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0053 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0342 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0022 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0085 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0004 m
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UGK 7.76

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**********************************************
**                                          **
**               YSLIP_PM                   **
**       (VERSION 2.2, JANUARY 1996)        **
**                                          **
**   A COMPUTER PROGRAM FOR SIMULATION OF   **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON   **
**   AN INCLINED PLANE AND CALCULATION OF   **
**   PERMANENT DISPLACEMENTS OF THE BLOCK   **
**                                          **
**                  BY                      **
**                                          **
**              Liping Yan                  **
**           Neven Matasovic                **
**        Edward Kavazanjian, Jr.           **
**                                          **
**********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Parkfield Record 30 ft, SM thickness = 100 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         p30100.out
CARD  7: NIDH  KREAD     NP        NLINE     DT       APH
         2     2         4096      512       0.02     0.469
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT       APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .469017
INPUT HORI. ACC. TIMES A FACTOR =  .999964 TO GET APH =  .469000


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .4464 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .4231 m     ✓

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .4670 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .4123 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3777 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1421 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .4022 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1820 m     ✓

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3113 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0753 m
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MSK 7.26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .3420 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0950 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2518 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0458 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .2839 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0566 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1979 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0290 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .2280 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0330 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1466 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0173 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1799 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0198 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1053 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0101 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1363 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0128 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0671 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0054 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0951 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0079 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0368 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0023 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0583 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0042 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0127 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0006 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0307 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0017 m
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MSK 7-26

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***********************************************
**                                           **
**                 YSLIP_PM                  **
**        (VERSION 2.2, JANUARY 1996)        **
**                                           **
**   A COMPUTER PROGRAM FOR SIMULATION OF    **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
**   AN INCLINED PLANE AND CALCULATION OF    **
**   PERMANENT DISPLACEMENTS OF THE BLOCK    **
**                                           **
**                    BY                     **
**                                           **
**                Liping Yan                 **
**             Neven Matasovic               **
**          Edward Kavazanjian, Jr.          **
**                                           **
***********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Parkfield Record 30 ft, SM thickness = 150 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP   KDIR
         0     10    1     1      9.807     0      0       0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         p30150.out
CARD  7: NIDH  KREAD      NP         NLINE      DT        APH
         2     2          4096       512        0.02      0.4797
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD      NP         NLINE      DT        APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .479704
INPUT HORI. ACC. TIMES A FACTOR =  .999992 TO GET APH =  .479700


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .5281 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .4422 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .4656 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .4124 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .4522 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1726 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .3967 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1964 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .3817 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0971 m
```

MSK 7-26

```
*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .3344 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .1090 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .3180 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0582 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .2744 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0634 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .2566 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0372 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .2168 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0356 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .2011 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0255 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .1679 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0195 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .1521 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0170 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .1232 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0116 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .1055 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0105 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .0812 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0067 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .0689 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0057 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .0469 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0032 m

 RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
            MAX. SLIDING VELOCITY =    .0356 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0024 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
            MAX. SLIDING VELOCITY =    .0196 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0010 m
```

MSK 7-26

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     ***********************************************
     **                                           **
     **               YSLIP_PM                    **
     **       (VERSION 2.2, JANUARY 1996)         **
     **                                           **
     **   A COMPUTER PROGRAM FOR SIMULATION OF    **
     **   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
     **   AN INCLINED PLANE AND CALCULATION OF    **
     **   PERMANENT DISPLACEMENTS OF THE BLOCK    **
     **                                           **
     **                  BY                       **
     **                                           **
     **              Liping Yan                   **
     **           Neven Matasovic                 **
     **        Edward Kavazanjian, Jr.            **
     **                                           **
     ***********************************************

     **************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Superstition 30'- SM layer thickness = 50'
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         sn3050.out
CARD  7: NIDH  KREAD    NP      NLINE     DT      APH
         2     2        4096    512       0.005   0.4046
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD    NP      NLINE     DT      APV      TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .404609
INPUT HORI. ACC. TIMES A FACTOR =  .999978 TO GET APH =  .404600


     *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2177 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .1389 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .2152 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .1415 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1495 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0532 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1580 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0542 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000
```

VNSK 7-26

```
*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .1128 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0207 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .1253 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0248 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0801 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0089 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0980 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0117 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0518 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0035 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0735 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0060 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0282 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0013 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0523 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0030 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0100 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0003 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0338 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0014 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0184 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0006 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =   .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =   .0068 m/sec
      PERMANENT SLIDING DISPLACEMENT =   .0001 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .400000
```

MYK 7-26

```
*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
             MAX. SLIDING VELOCITY =    .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
             MAX. SLIDING VELOCITY =    .0001 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m
```

MSK 7-76

```
**********************************************
**                                          **
**              YSLIP_PM                    **
**       (VERSION 2.2, JANUARY 1996)        **
**                                          **
**  A COMPUTER PROGRAM FOR SIMULATION OF    **
**  DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
**  AN INCLINED PLANE AND CALCULATION OF    **
**  PERMANENT DISPLACEMENTS OF THE BLOCK    **
**                                          **
**                 BY                       **
**                                          **
**             Liping Yan                   **
**           Neven Matasovic                **
**        Edward Kavazanjian, Jr.           **
**                                          **
**********************************************

    ***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Superstition 30'- SM layer thickness = 100'
CARD  3: KPAR   N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0      10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         sn30100.out
CARD  7: NIDH  KREAD     NP        NLINE      DT        APH
         2     2         4096      512        0.005     0.3467
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE      DT        APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .346668
INPUT HORI. ACC. TIMES A FACTOR = 1.000092 TO GET APH =  .346700


     ******************RESULTS*****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .2369 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .1598 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .2412 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .1800 m    ✓

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .1683 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0592 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .1843 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0702 m    ✓

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000
```

MGK 7-26

```
*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .1305 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0235 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .1344 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0302 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .0966 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0117 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .0946 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0139 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .0667 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0064 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .0627 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0066 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .0411 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0029 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .0369 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0029 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .0203 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0009 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .0172 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0009 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .0050 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0001 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .0037 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0001 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000
```

MSK 7-26

```
*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m
```

MKK 7-26

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         ************************************************
         **                                            **
         **                 YSLIP_PM                   **
         **        (VERSION 2.2, JANUARY 1996)         **
         **                                            **
         **   A COMPUTER PROGRAM FOR SIMULATION OF     **
         **   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON     **
         **   AN INCLINED PLANE AND CALCULATION OF     **
         **   PERMANENT DISPLACEMENTS OF THE BLOCK     **
         **                                            **
         **                    BY                      **
         **                                            **
         **                Liping Yan                  **
         **             Neven Matasovic                **
         **          Edward Kavazanjian, Jr.           **
         **                                            **
         ************************************************

         ***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Superstition 30'- SM layer thickness = 150'
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         sn30150.out
CARD  7: NIDH  KREAD     NP        NLINE      DT       APH
         2     2         4096      512        0.005    0.3088
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE      DT       APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .308773
INPUT HORI. ACC. TIMES A FACTOR = 1.000087 TO GET APH =  .308800

          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .1835 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1271 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                  MAX. SLIDING VELOCITY =   .2060 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1579 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .1431 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0485 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                  MAX. SLIDING VELOCITY =   .1518 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0491 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000
```

MSK 7-26

```
*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .1064 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0209 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .1086 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0180 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .0736 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0102 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .0729 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0084 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .0450 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0050 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .0437 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0035 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .0212 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0018 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .0210 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0011 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .0049 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0002 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .0054 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0001 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
```

MMSK 7.26

```
          MAX. SLIDING VELOCITY =    .0000 m/sec
 PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
          MAX. SLIDING VELOCITY =    .0000 m/sec
 PERMANENT SLIDING DISPLACEMENT =    .0000 m
```

Mail 7-26

```
***********************************************
**                                           **
**                YSLIP_PM                   **
**        (VERSION 2.2, JANUARY 1996)        **
**                                           **
**   A COMPUTER PROGRAM FOR SIMULATION OF    **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
**   AN INCLINED PLANE AND CALCULATION OF    **
**   PERMANENT DISPLACEMENTS OF THE BLOCK    **
**                                           **
**                   BY                      **
**                                           **
**               Liping Yan                  **
**            Neven Matasovic                **
**         Edward Kavazanjian, Jr.           **
**                                           **
***********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Superstition 60'- SM layer thickness = 150'
CARD  3: KPAR  N    KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10   1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         sn60150.out
CARD  7: NIDH  KREAD     NP        NLINE      DT        APH
         2     2         4096      512        0.005     0.3090
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE      DT        APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI.'ACC. =  .308963
INPUT HORI. ACC. TIMES A FACTOR = 1.000120 TO GET APH =  .309000


           *****************RESULTS*****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1797 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .1272 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1887 m/sec  ✓
        PERMANENT SLIDING DISPLACEMENT =    .1468 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1401 m/sec  ✓
        PERMANENT SLIDING DISPLACEMENT =    .0461 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1434 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0446 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1044 m/sec  ✓
        PERMANENT SLIDING DISPLACEMENT =    .0183 m
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Usd 7-26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1044 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0157 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0726 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0086 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0711 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0069 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0450 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0037 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0432 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0029 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0222 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0011 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0210 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0010 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0054 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0001 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0055 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0001 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m
```

MSC 7.76

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*********************************************
**                                         **
**               YSLIP_PM                  **
**        (VERSION 2.2, JANUARY 1996)      **
**                                         **
**   A COMPUTER PROGRAM FOR SIMULATION OF  **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON  **
**   AN INCLINED PLANE AND CALCULATION OF  **
**   PERMANENT DISPLACEMENTS OF THE BLOCK  **
**                                         **
**                   BY                    **
**                                         **
**               Liping Yan                **
**            Neven Matasovic              **
**        Edward Kavazanjian, Jr.          **
**                                         **
*********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Superstition 60'- SM layer thickness = 100'
CARD  3: KPAR  N    KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10   1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         sn60100.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.005     0.3379
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .337891
INPUT HORI. ACC. TIMES A FACTOR = 1.000027 TO GET APH =  .337900


            *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .2146 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .1509 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .2143 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .1729 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1588 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0550 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1591 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0587 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1225 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0243 m
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Mill 9-26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1170 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0217 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0901 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0126 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0823 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0102 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0614 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0066 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0537 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0046 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0366 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0030 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0302 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0018 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0168 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0009 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0123 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0004 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0029 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0001 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0012 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m
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MSK 7-26

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*********************************************
**                                         **
**              YSLIP_PM                   **
**      (VERSION 2.2, JANUARY 1996)        **
**                                         **
**  A COMPUTER PROGRAM FOR SIMULATION OF   **
**  DYNAMIC BEHAVIOR OF A RIGID BLOCK ON   **
**  AN INCLINED PLANE AND CALCULATION OF   **
**  PERMANENT DISPLACEMENTS OF THE BLOCK   **
**                                         **
**                 BY                      **
**                                         **
**             Liping Yan                  **
**           Neven Matasovic               **
**        Edward Kavazanjian, Jr.          **
**                                         **
*********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Superstition 60'- SM layer thickness = 50'
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT    KSLIP  KDIR
         0     10    1     1      9.807     0       0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         sn6050.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.005     0.3763
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .376289
INPUT HORI. ACC. TIMES A FACTOR = 1.000029 TO GET APH =  .376300


          ******************RESULTS*****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2217 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1409 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1932 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1427 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1351 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0519 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1526 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0510 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0990 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0202 m
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9/15/C 7-26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .1203 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0201 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0674 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0095 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0917 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0101 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0403 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0045 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0663 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0051 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0188 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0014 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0442 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0024 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0048 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0002 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0257 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0010 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0112 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0003 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0016 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
              MAX. SLIDING VELOCITY =    .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
              MAX. SLIDING VELOCITY =    .0000 m/sec
      PERMANENT SLIDING DISPLACEMENT =    .0000 m
```

TASK 7.26

```
***********************************************
**                                           **
**                 YSLIP_PM                  **
**         (VERSION 2.2, JANUARY 1996)       **
**                                           **
**   A COMPUTER PROGRAM FOR SIMULATION OF    **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
**   AN INCLINED PLANE AND CALCULATION OF    **
**   PERMANENT DISPLACEMENTS OF THE BLOCK    **
**                                           **
**                    BY                     **
**                                           **
**                Liping Yan                 **
**             Neven Matasovic               **
**          Edward Kavazanjian, Jr.          **
**                                           **
***********************************************

 ***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Big Bear Record 30' - SM thickness = 100 ft
CARD  3: KPAR  N      KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10     1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         b30100.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.01      0.4969
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .496930
INPUT HORI. ACC. TIMES A FACTOR =  .999940 TO GET APH =  .496900


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .5224 m/sec
         PERMANENT SLIDING DISPLACEMENT =   .3651 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                  MAX. SLIDING VELOCITY =   .3417 m/sec
         PERMANENT SLIDING DISPLACEMENT =   .3938 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .4477 m/sec
         PERMANENT SLIDING DISPLACEMENT =   .1647 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                  MAX. SLIDING VELOCITY =   .2547 m/sec
         PERMANENT SLIDING DISPLACEMENT =   .1513 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .3783 m/sec
         PERMANENT SLIDING DISPLACEMENT =   .0984 m
```

MSK 7.76

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1779 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0649 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3145 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0665 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1184 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0288 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2562 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0452 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0807 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0131 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2032 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0297 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0499 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0047 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1557 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0188 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0249 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0011 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1136 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0111 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0069 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0002 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0763 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0061 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0445 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0029 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m
```

0#SK 7.2i

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***********************************************
**                                           **
**                 YSLIP_PM                  **
**        (VERSION 2.2, JANUARY 1996)        **
**                                           **
**   A COMPUTER PROGRAM FOR SIMULATION OF    **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
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**   PERMANENT DISPLACEMENTS OF THE BLOCK    **
**                                           **
**                    BY                     **
**                                           **
**               Liping Yan                  **
**             Neven Matasovic               **
**          Edward Kavazanjian, Jr.          **
**                                           **
***********************************************

    ***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Big Bear Record 30' - SM thickness = 50 ft
CARD  3: KPAR  N      KACC  KCOM   g(m/s/s)  KOUT    KSLIP   KDIR
         0     10     1     1      9.807     0       0       0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         b3050.out
CARD  7: NIDH  KREAD     NP        NLINE      DT        APH
         2     2         4096      512        0.01      0.4652
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE      DT        APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .465208
INPUT HORI. ACC. TIMES A FACTOR =  .999983 TO GET APH =  .465200


    *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .4741 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3345 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                  MAX. SLIDING VELOCITY =   .3380 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3638 m   ✓

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .3983 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1498 m   ✓

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                  MAX. SLIDING VELOCITY =   .2529 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1481 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                  MAX. SLIDING VELOCITY =   .3309 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0872 m   ✓
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MSK 7.76

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .1761 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0623 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .2716 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0584 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .1094 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0265 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .2179 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0385 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0562 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0102 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .1688 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0240 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0272 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0027 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .1257 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0141 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0080 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0002 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0869 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0075 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0529 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0036 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0256 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0013 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m
```

MSC 7.26

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**********************************************
**                                          **
**                YSLIP_PM                  **
**        (VERSION 2.2, JANUARY 1996)       **
**                                          **
**   A COMPUTER PROGRAM FOR SIMULATION OF   **
**   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON   **
**   AN INCLINED PLANE AND CALCULATION OF   **
**   PERMANENT DISPLACEMENTS OF THE BLOCK   **
**                                          **
**                   BY                     **
**                                          **
**               Liping Yan                 **
**            Neven Matasovic               **
**         Edward Kavazanjian, Jr.          **
**                                          **
**********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Big Bear Record 30' - SM thickness = 150 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10    1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         b30150.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.01      0.4570
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .457006
INPUT HORI. ACC. TIMES A FACTOR =  .999987 TO GET APH =  .457000


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .5083 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3525 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .3590 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .3839 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .4312 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1558 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .2764 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .1452 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .3584 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0923 m
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MSK 7:26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .2026 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0643 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .2907 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0607 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .1370 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0297 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .2288 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0399 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0818 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0142 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .1734 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0251 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0386 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0053 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .1247 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0149 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0112 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0009 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0830 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0079 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0483 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0036 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .0211 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0011 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =   .0000 m
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WSK 7.76

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 ***********************************************
 **                                           **
 **                 YSLIP_PM                  **
 **         (VERSION 2.2, JANUARY 1996)       **
 **                                           **
 **   A COMPUTER PROGRAM FOR SIMULATION OF    **
 **   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
 **   AN INCLINED PLANE AND CALCULATION OF    **
 **   PERMANENT DISPLACEMENTS OF THE BLOCK    **
 **                                           **
 **                     BY                    **
 **                                           **
 **                 Liping Yan                **
 **              Neven Matasovic              **
 **         Edward Kavazanjian, Jr.           **
 **                                           **
 ***********************************************

 ***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Big Bear Record 60' - SM thickness = 150 ft
CARD  3: KPAR  N    KACC  KCOM   g(m/s/s)  KOUT   KSLIP  KDIR
         0     10   1     1      9.807     0      0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         b60150.out
CARD  7: NIDH  KREAD     NP        NLINE      DT       APH
         2     2         4096      512        0.01     0.4084
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE      DT       APV       TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =   .408391
INPUT HORI. ACC. TIMES A FACTOR = 1.000022 TO GET APH =   .408400


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .4684 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .3155 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .3114 m/sec ✓
       PERMANENT SLIDING DISPLACEMENT =    .3454 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .3884 m/sec ✓
       PERMANENT SLIDING DISPLACEMENT =    .1351 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =    .2265 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .1202 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =    .3136 m/sec
       PERMANENT SLIDING DISPLACEMENT =    .0802 m ✓
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MGK 7.26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .1512 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0485 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .2451 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0512 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0871 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0185 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .1836 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0321 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0402 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0065 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .1302 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0189 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0142 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0011 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0853 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0097 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0005 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0481 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0039 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0191 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0011 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .0014 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m
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MGK 7-26

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*********************************************
**                                         **
**                YSLIP_PM                 **
**        (VERSION 2.2, JANUARY 1996)      **
**                                         **
**  A COMPUTER PROGRAM FOR SIMULATION OF   **
**  DYNAMIC BEHAVIOR OF A RIGID BLOCK ON   **
**  AN INCLINED PLANE AND CALCULATION OF   **
**  PERMANENT DISPLACEMENTS OF THE BLOCK   **
**                                         **
**                   BY                    **
**                                         **
**               Liping Yan                **
**            Neven Matasovic              **
**        Edward Kavazanjian, Jr.          **
**                                         **
*********************************************

***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Big Bear Record 60' - SM thickness = 100 ft
CARD  3: KPAR  N    KACC  KCOM   g(m/s/s)  KOUT  KSLIP  KDIR
         0     10   1     1      9.807     0     0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         b60100.out
CARD  7: NIDH  KREAD     NP        NLINE     DT        APH
         2     2         4096      512       0.01      0.4448
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD     NP        NLINE     DT        APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bbl.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bbl.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .444781
INPUT HORI. ACC. TIMES A FACTOR = 1.000043 TO GET APH =  .444800


          *****************RESULTS****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .4677 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .3317 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .3117 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .3436 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .3908 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1448 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =   .2199 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1205 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =   .3210 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0858 m
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MGK 7.26

```
*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .1377 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0488 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .2595 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0563 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0865 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0180 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .2044 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0363 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0506 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0049 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1549 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0220 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0226 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0011 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .1107 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0124 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0043 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0001 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0727 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0062 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0405 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0026 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                 MAX. SLIDING VELOCITY =    .0153 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0007 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                 MAX. SLIDING VELOCITY =    .0000 m/sec
        PERMANENT SLIDING DISPLACEMENT =    .0000 m
```

MCL 7.26

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 ***********************************************
 **                                           **
 **                YSLIP_PM                   **
 **        (VERSION 2.2, JANUARY 1996)        **
 **                                           **
 **   A COMPUTER PROGRAM FOR SIMULATION OF    **
 **   DYNAMIC BEHAVIOR OF A RIGID BLOCK ON    **
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 **                                           **
 **                   BY                      **
 **                                           **
 **               Liping Yan                  **
 **            Neven Matasovic                **
 **        Edward Kavazanjian, Jr.            **
 **                                           **
 ***********************************************

 ***************REPRINT INPUT DATA**************

CARD  1: TITLE
         VARIATION OF Ky/Kh
CARD  2: SUBTITLE
         Big Bear Record 60' - SM thickness = 50 ft
CARD  3: KPAR  N     KACC  KCOM   g(m/s/s)  KOUT    KSLIP  KDIR
         0     10    1     1      9.807     0       0      0
CARD  4: Ky(1), ..., Ky(N)
         0.040 0.080 0.120 0.160 0.200 0.240 0.280 0.320 0.360 0.400
CARD  5: YIELD ACCELERATION DEGRADATION DATA
         Ky is constant.
CARD  6: INPUT HORIZONTAL ACC. TIME HISTORY FILE NAME
         b6050.out
CARD  7: NIDH  KREAD    NP       NLINE     DT       APH
         2     2        4096     512       0.01     0.4108
CARD  8: INPUT VERTICAL ACC. TIME HISTORY FILE NAME
         NO Av
CARD  9: NIDV  KREAD    NP       NLINE     DT       APV        TSHIFT
         NO INPUT
CARD 10: FILE NAME FOR OUTPUT OF SLIDING MOTION
         bb1.OUS
CARD 11: FILE NAME FOR OUTPUT OF ABSOLUTE MOTION
         bb1.OUA

MAX. VALUE OF INPUT HORI. ACC. =  .410755
INPUT HORI. ACC. TIMES A FACTOR = 1.000110 TO GET APH =  .410800


 *****************RESULTS*****************

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .040000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .4468 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .3073 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .3389 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .3441 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .080000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .3635 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1357 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
               MAX. SLIDING VELOCITY =   .2567 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .1319 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =   .120000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
               MAX. SLIDING VELOCITY =   .2871 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0784 m
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MGK 7.26

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*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1816 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0547 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .160000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .2249 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0500 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .1151 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0228 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .200000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1723 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0311 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0598 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0087 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .240000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .1255 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0175 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0232 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0020 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .280000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0841 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0087 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0020 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0001 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .320000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0488 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0036 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .360000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0205 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0011 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

  RESULTS FOR DOWNSLOPE YIELD ACCELERATION (g) =    .400000

*** RUN 1: THE DIRECTION OF Ah IS AS INPUT ***
                MAX. SLIDING VELOCITY =   .0018 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m

*** RUN 2: THE DIRECTION OF Ah IS REVERSED ***
                MAX. SLIDING VELOCITY =   .0000 m/sec
       PERMANENT SLIDING DISPLACEMENT =   .0000 m
```

Calculation Package G-4
Seismically-Induced Permanent Deformations

GeoSyntec Consultants

GEOSYNTEC CONSULTANTS
COMPUTATION COVER SHEET

Client: Parsons **Project:** BRC CAMU **Project/Proposal #:** HL0389 **Task #:** 04

Title of Computations: Evaluation of Seismic-Induced Permanent Deformations

Computations By:

SIGNATURE — 26 ~~28~~ July 2000 DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Assumptions and Procedures Checked By (Peer Reviewer):

SIGNATURE — 07-26-00 DATE

Neven Matasovic / Project Engineer
PRINTED NAME AND TITLE

Computations Checked By:

SIGNATURE — 7-26-00 DATE

Michael E. Kalinski / Senior Staff Engineer
PRINTED NAME AND TITLE

Computations Backchecked By (Originator):

SIGNATURE — 7-26-00 DATE

Geoff L. Smith / Staff Engineer
PRINTED NAME AND TITLE

Approved By (PM or Designate):

SIGNATURE — 07/26/00 DATE

Edward Kavazanjian, Jr. / Principal
PRINTED NAME AND TITLE

Approval Notes:

Revisions: (Number and Initial All Revisions)

No.	Sheet	Date	By	Checked By	Approval

Written by: Geoff L. Smith GLS Date: 07 / 25 / 00 Reviewed by: MLL 7.26 Date: __/__/__
MM DD YY MM DD YY

Client: Parsons Project: BRC CAMU Project No.: HL0389 Task No.: 04

EVALUATION OF SEISMICALLY-INDUCED PERMANENT DEFORMATIONS
BRC CAMU
HENDERSON, NEVADA

ECK 07/26/00

PURPOSE

The purpose of the analyses documented herein is to evaluate seismically-induced permanent deformations of the proposed BRC CAMU in Henderson, Nevada. The analyses are based on site-specific seismic deformation charts documented in a companion calculation package (*site-specific seismic deformation charts*).

METHOD OF ANALYSIS

Using the site-specific seismic deformation charts, seismically-induced permanent deformations may be evaluated. The site-specific seismic deformation charts are presented in two figures which represent the expected deformations of the North Mesa area and at the South Mesa area of the proposed landfill (see *site-specific deformation charts* calculation package). The North Mesa site-specific deformation chart shows a larger deformation response than the South Mesa chart. Therefore, for simplicity and conservatism, the North Mesa site-specific deformation chart will be used herein to represent the expected deformations at all locations. Figure 1 presents the North Mesa site-specific deformation chart. The site-specific seismic deformation chart has numerous curves that represent a suite of potential site responses. To be conservative, the curve that indicates the largest seismic deformation is used.

Seismically-induced permanent deformations are evaluated for the following locations:

- side slopes of the final cover;
- top deck of the final cover; and
- base liner.

RKC 07/26/00

DESIGN CRITERION

To evaluate seismically-induced permanent deformations at the BRC CAMU site, GeoSyntec established a maximum deformation of 12 in (0.3 m) for final cover slopes and 6 in (150 mm) for the base liner as the design criterion.

YIELD ACCELERATIONS

The yield accelerations presented here in were calculated in the final waste slopes and the final cover stability companion calculation packages. The yield accelerations evaluated from the stability calculations are as follows:

Location	Yield Acceleration
Final Cover Side Slopes	0.15 g
Final Cover Top Deck	>0.15 g
Base Liner	0.10 g

The yield acceleration of the final cover top deck is larger than the final cover side slopes due to the smaller slope inclination (e.g., 2% for the top deck compared to 3H:1V for the side slopes). The smaller slope inclination of the top deck produces a higher factor of safety than the final cover side slopes resulting in larger yield accelerations.

RESULTS

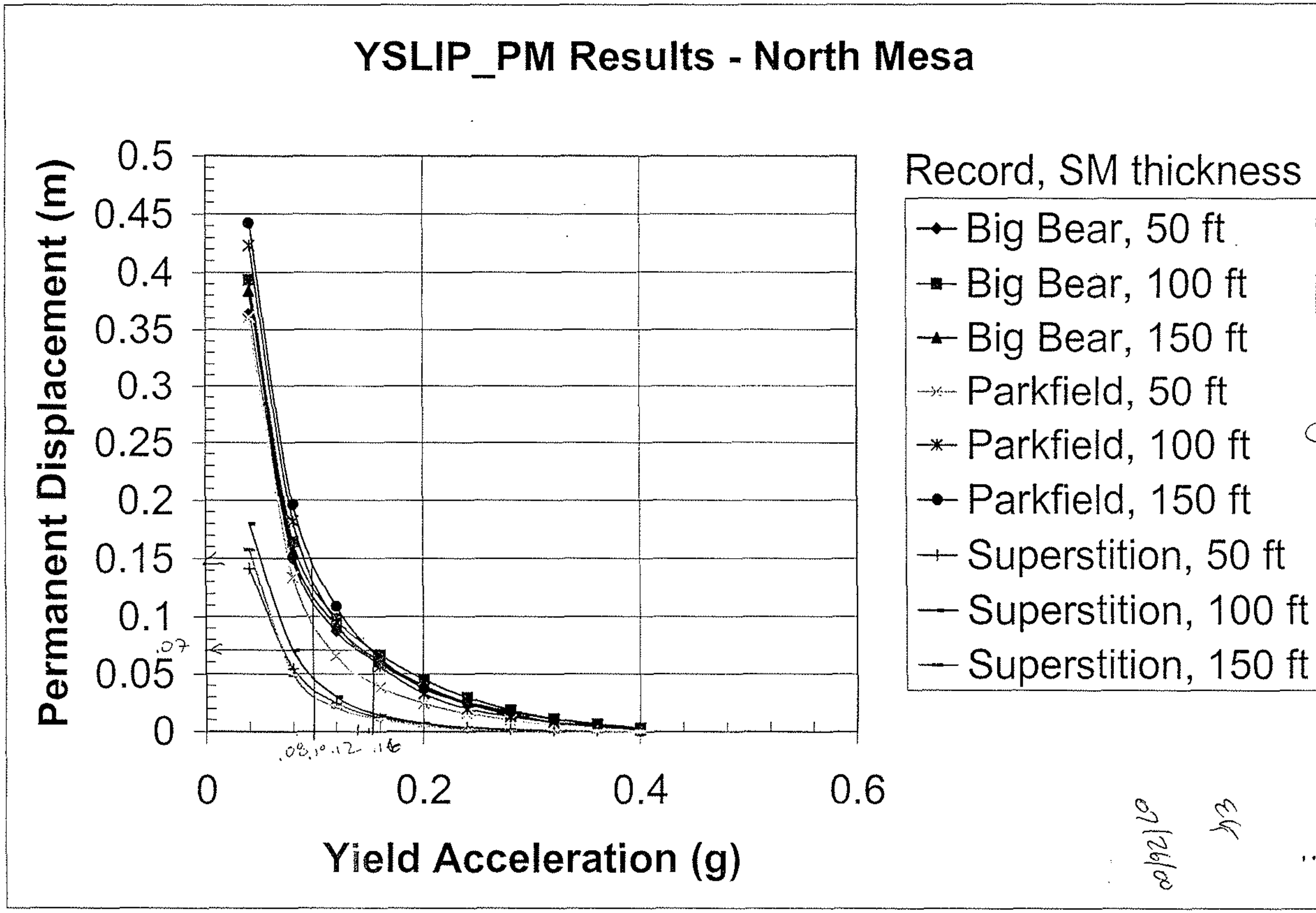
By inputting the yield acceleration into the site-specific deformation charts, the seismically-induced permanent deformations may be evaluated. Based on the site-specific seismic deformation charts, the permanent seismic deformations are estimated to be:

Location	Estimated Permanent Deformation
Final Cover Side Slopes	0.07 m (2.8 in)
Final Cover Top Deck	< 0.07 m (2.8 in)
Base Liner	0.15 m (5.9 in)

Based on the above results, the estimated seismically-induced permanent deformations meet the design criteria.

REFERENCES - COMPANION CALCULATION PACKAGES

- Site-specific Deformation Charts
- Seismic Hazard Evaluation and Design Ground Motions
- Seismic Site Response Analyses
- Slope Stability Evaluation - Final Waste Slopes
- Final Cover Sloughing Stability Calculations



YSLIP_PM Results - North Mesa
Permanent Displacement (m)
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
.07
.08 .10 .12 .16
0
0.2
0.4
0.6
Yield Acceleration (g)
Record, SM thickness
Big Bear, 50 ft
Big Bear, 100 ft
Big Bear, 150 ft
Parkfield, 50 ft
Parkfield, 100 ft
Parkfield, 150 ft
Superstition, 50 ft
Superstition, 100 ft
Superstition, 150 ft

Figure 1

Section 6
Final Cover Construction Quality Assurance Plan

CONSTRUCTION QUALITY ASSURANCE PLAN
FOR THE
CONSTRUCTION OF

FINAL COVER SYSTEM
AT
CORRECTIVE ACTION MANAGEMENT UNIT
BASIC REMEDIATION COMPANY
HENDERSON, NEVADA

Prepared for:

Basic Environmental COMPANY

Basic Remediation Company
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3 November 2006

TABLE OF CONTENTS

1. INTRODUCTION

1.1 Terms of Reference

GeoSyntec Consultants (GeoSyntec) has prepared this Construction Quality Assurance (CQA) Plan for the construction of the Corrective Action Management Unit (CAMU) Final Cover System for Basic Remediation Company (BRC) located in Henderson, Nevada. Hereinafter, the CAMU construction is referred to as the Project.

This CQA Plan was prepared by Mr. Gregory T. Corcoran, P.E. of GeoSyntec Consultants (GeoSyntec) under the direction of Mr. Brad Cooley, P.E. In general accordance with the peer review policies of the firm, Mr. Brad Cooley, P.E. of GeoSyntec was responsible for senior peer review of the work presented in this plan.

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 final cover system for the project. The soils, geosynthetic, and appurtenant components include cover soil, prepared subgrade, drainage aggregate, geosynthetic clay liner, geomembrane, geotextile, geocomposite, and corrugated polyethylene (CPE) pipe. It should be emphasized that care and documentation are required in the placement and compaction of the soils and aggregate and in the production and installation of the geosynthetic materials placed during construction. The CQA Plan, therefore, delineates the procedures to be followed for monitoring construction of these materials.

The scope of this CQA Plan includes the CQA of the soil and geosynthetic components of the Project. The CQA monitoring activities during the selection, evaluation, treatment, placement, and compaction of soils for earthworks, and 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 geosynthetic clay liner;
- Section 13 presents CQA of the geocomposite; and
- Section 14 presents CQA surveying.

2. DEFINITIONS RELATING TO CONSTRUCTION QUALITY ASSURANCE

This CQA Plan is devoted to Construction Quality Assurance. In the context of this document, Construction Quality Assurance and Construction Quality Control are defined as follows:

Construction Quality Assurance (CQA) - A planned and systematic pattern of means and actions designed to assure adequate confidence that materials and/or services meet contractual and regulatory requirements and will perform satisfactorily in service.

Construction Quality Control (CQC) - Those actions which provide a means to measure and regulate the characteristics of an item or service in relation to contractual and regulatory requirements.

In the context of this document:

- 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*.

- 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 soil components, CQC is combined with CQA and is provided by the CQA Consultant. In the case of the geosynthetic components and piping of the Work, CQC is provided by the Manufacturer and Geosynthetic Installer and the Contractor. CQA testing of soil, pipe, and geosynthetic components is provided by the CQA Consultant.

3. PARTIES INVOLVED WITH CONSTRUCTION QUALITY ASSURANCE

3.1 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 Parsons Engineering Science, Inc. (Parsons) and GeoSyntec.

Qualifications

The Engineer of Record shall be a qualified engineer, registered as required by Nevada state regulations. The Engineer should have expertise, which demonstrates significant familiarity with piping, geosynthetics and soils, as appropriate, including design and construction experience related to landfill liner systems.

3.2 Project Manager

Responsibilities

The Project Manager is responsible for implementing the design, and overseeing subcontractors. In this CQA Plan, the term “Project Manager” refers to a qualified BRC employee.

Qualifications

The Project Manager shall be a qualified engineer having familiarity with earthwork construction and installation of geosynthetic materials.

3.3 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 and geosynthetic components of the liner system. This work will include excavation, placement and compaction of engineered fill and prepared subgrade, placement of drainage aggregate and native soil (operations layer material), installation and of piping and concrete manhole, installation of temporary erosion control features, 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 construction and maintain current state and federal licenses as appropriate.

3.4 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.

3.5 Geosynthetics Manufacturer

Responsibilities

The Manufacturer is responsible for the production of finished material (geomembrane, geotextile, geosynthetic clay liner, geocomposite, pipe, and other specified material) 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*.

3.6 Geosynthetic Installer

Responsibilities

The Geosynthetic Installer is responsible for field handling, storage, placement, seaming, loading 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*.

3.7 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 the geosynthetic components used in the construction of the Project. The CQA Consultant will also be responsible for issuing a CQA report at the completion of the Project construction, which details the earthworks, piping, and geosynthetic installation activities 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 Nevada.

The CQA Consultant will be responsible for obtaining and testing representative samples of all components used in construction of the Project as required by this CQA Plan and *Technical Specifications*. All tests will be conducted in general accordance with ASTM or other applicable state or federal standards. Test results must be submitted to the Project Manager within a reasonable timeframe, which will not impede or delay construction of the Project. The CQA Consultant will be responsible for inspecting all earthwork, piping, and geosynthetic operations to verify that the components are installed in general accordance with this CQA Plan and *Technical Specifications*.

Qualifications

The CQA Consultant is a well established firm specializing in geotechnical and geosynthetic engineering and 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 Nevada State regulations.

The CQA Consultant will be experienced with earthwork construction and the 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 the installation of geosynthetic materials and will be trained by the CQA Consultant in the duties of a CQA Site Manager.

3.8 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 Owner. 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 experience in the profession of surveying services in the State of Nevada. The Surveyor will be a licensed professional as required by the State of Nevada regulations. The Surveyor shall be fully equipped and experienced in the use of total stations and AutoCAD Version 14. All surveying will be performed under the direct supervision of the Owner.

3.9 CQA Laboratory

Responsibilities

The CQA Laboratory is a party, independent from the Contractor, Manufacturer and Geosynthetic Installer, that is responsible for conducting tests in general accordance with ASTM and other applicable test standards on samples of geosynthetic materials and soil 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.

4. CQA CONSULTANTS PERSONNEL ORGANIZATION AND DUTIES

4.1 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:

- screening of materials;
- placement and compaction of cover soil and prepared subgrade;
- installation of geotextile;
- installation of geosynthetic clay liner;
- installation of geomembrane;
- installation of drainage aggregate;
- installation of geocomposite; and
- installation of piping.

The duties of the CQA personnel are discussed in the remainder of this section.

4.2 CQA Personnel

For construction of the Project, the CQA Consultant's personnel will include:

- the CQA Officer, who operates 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.

The duties of the CQA Personnel are discussed in the following subsections.

4.2.1 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 QC Plan;

- attends resolution and/or 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.

4.2.2 CQA Site Manager

The CQA Site Manager:

- acts as the on-site representative of the CQA Consultant;

- attends CQA-related meetings (e.g., resolution, pre-construction, daily, weekly (or designates a representative to attend the meeting));

- prepares or 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 Manufacturer's QC documentation;

- reviews the Geosynthetic Installer's personnel Qualifications for conformance with those pre-approved for work on site;

- notes in the daily summary report and reports to the CQA Officer and Project Manager on-site activities that could result in damage to the geosynthetic materials or other completed work;

- reports unresolved deviations from the CQA Plan, *Drawings*, and *Technical Specifications* to the Project Manager; and

- assists with the preparation of the CQA report.

5. SITE AND PROJECT CONTROL

5.1 Project Coordination Meetings

Meetings of key project personnel are necessary to assure a high degree of quality during installation, and 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) resolution meetings; (ii) pre-construction meetings; (iii) progress meetings; and (iv) problem or work deficiency meetings.

5.1.1 Resolution Meeting

Following the completion of the design, *Drawings*, and *Technical Specifications* for the project and prior to the start of construction, a Resolution Meeting will be held. This meeting may include the CQA Officer, the CQA Site Manager, the Engineer, and the Project Manager.

The purpose of this meeting is to begin planning for coordination of construction tasks, anticipate installation problems which might cause difficulties and delays in construction, and, above all, present the CQA Plan to the parties involved. It is very important that the criteria regarding testing, repair, and other CQA activities be known and accepted by the parties involved in the work prior to the installation of geosynthetic materials and construction of the soil components for the Project.

The first part of the Resolution Meeting may be devoted to a review of the *Drawings* and *Technical Specifications* for familiarity. This is different from the peer review of the design, including design calculations, which will have been carried out previously.

The Resolution Meeting may include the following activities:

- distribute relevant documents to all parties;
- review critical design details of the project;

- review this CQA Plan;
- review the Drawings and Technical Specifications;
- make appropriate modifications to the design criteria, *Drawings*, and *Technical Specifications* so that the fulfillment of the design specifications or performance standards can be determined through the implementation of the CQA Plan;
- reach a consensus on the quality control procedures, especially on methods of evaluating acceptability of the soils and geosynthetic materials;
- assign the responsibilities of each party;
- establish work area security and health and safety protocol;
- confirm the methods for documenting observations, reporting, and distributing documents and reports; and
- confirm the lines of authority and communication.

The Project Manager will appoint one of the meeting attendees to record the discussions and decisions of the Resolution Meeting. The record of the meeting will be documented by the appointee in the form of meeting minutes, which will be subsequently distributed to all attendees.

5.1.2 Pre-Construction Meeting

A Pre-Construction Meeting will be held at the site prior to construction of the Project. As a minimum, the Pre-Construction Meeting will be attended by the Contractor, the Geosynthetic Installer's Superintendent, the CQA Consultant, the Engineer, and the Project 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 Project 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 Project 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.

5.1.3 Progress Meetings

Progress meetings will be held between the CQA Site Manager, the Contractor, Project 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.

5.1.4 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 Project 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 Project 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.

6. DOCUMENTATION

6.1 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 all quality assurance requirements have been addressed and satisfied.

The CQA Site Manager will provide the Project 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.

6.2 Daily Recordkeeping

Preparation of daily CQA documentation will consist of daily 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 Project Manager.

The CQA Site Manager will prepare daily reports that document the activities observed during each day of activity. The daily 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 inspected;
- type of construction and inspection 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.

6.3 Construction Problems and Resolution Data Sheets

Construction Problems and Resolution Data Sheets, to be submitted with the daily reports prepared by the CQA Site Manager, describing special construction situations will be cross-referenced with daily 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 Project Manager.

The Project 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 Engineer 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.

6.4 Photographic Documentation

Photographs will be taken and documented in order to serve as a pictorial record of work progress, problems, and mitigation activities. The basic file will contain color prints. Negatives will also be stored in a separate file in chronological order. These records will be presented to the Project 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). Photographs used for documentation will be identified with the date, time, and location of the photograph.

6.5 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 Project Manager. Design and/or specification changes will be made with the written agreement of the Project Manager and the Engineer and will take the form of an addendum to the *Drawings* and *Technical Specifications*.

6.6 CQA Report

At the completion of the Project, the CQA Consultant will submit to the Project Manager the CQA report signed and sealed by the Professional Engineer licensed in the State of Nevada. 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:

- Manufacturers' quality control 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 personnel resumes and qualifications;

- 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 inspected 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;

- documentation showing that piping was tested in general accordance with the *Technical Specifications*; and

- daily inspection reports.

The record drawings will include scale drawings depicting the location of the construction and details pertaining to the extent of construction (e.g., depths, plan dimensions, elevations, soil component thicknesses). Base maps required for development of the record drawings and the record drawings will be prepared by a qualified Professional Land Surveyor registered in the State of Nevada. These documents will be reviewed by the CQA Consultant and included as part of the CQA Report.

7. EARTHWORKS

7.1 Introduction

This section prescribes the CQA activities to be performed to monitor that earthwork components are constructed in general accordance with *Drawings* and *Technical Specifications*. The earthworks construction procedures to be monitored by the CQA Consultant include:

- cover soil placement;
- anchor trench excavation and backfill; and
- subgrade preparation.

7.2 Testing Activities

Soil testing will be performed for material qualification, material conformance, and construction quality control (CQC). These three stages of testing are defined as follows:

- Material qualification tests are used to evaluate the conformance of a proposed soil source to the material specifications for qualification of the source prior to construction.

- Soils conformance testing is used to evaluate the conformance of a particular batch of soil from a qualified source to the material specifications prior to installation of the soil.

- CQC tests are performed on completed portions of the earthwork during construction to demonstrate that the placement procedures are resulting in a product that meets or exceeds both material and performance specifications.

The Contractor will be responsible for submitting material qualification test results to the Project Manager and to the CQA Site Manager for review. The CQA Laboratory will perform the conformance testing and CQC testing. Soil 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 soils testing for material qualification will conform to the minimum frequencies presented in Table 2. The frequency of soils testing for material conformance will conform to the minimum frequencies presented in Table 3. 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 or Test Location 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 soil available for sampling in a timely and organized manner so that the test results can be obtained before the material is installed. The CQA Site Manager must document sample locations so that failing areas can be immediately isolated. The CQA Site Manager will follow standard sampling procedures to obtain representative samples of the proposed soil materials.

CQC sample and test locations will be selected by the CQA Site Manager at the minimum test frequency specified in Table 4. Samples and test locations will generally be selected at random, however a special testing frequency will be used at the discretion of the CQA Site Manager when visual observations of construction performance indicate a potential problem. Additional testing for suspected areas will be considered when:

- rollers slip during rolling operation;
- lift thickness is greater than specified;

- fill is at improper and/or variable moisture content;
- less than specified number of roller passes are made;
- dirt-clogged rollers are used to compact the material;
- rollers may not have used optimum ballast;
- fill materials differ substantially from those specified;
- the degree of compaction is doubtful; and
- as directed by the Project Manager or the CQA Site Manager.

The frequency of testing may also be increased in the following situations:

- adverse weather conditions;
- breakdown of equipment;
- at the start and finish of grading;
- material fails to meet specifications; and
- the work area is reduced.

7.3 CQA Monitoring Activities

7.3.1 Earthwork

The CQA Site Manager will monitor and document the earthworks required for the Project. In general, monitoring the construction for earthwork includes the following activities:

- reviewing documentation of the material qualification test results provided by the Contractor;

- monitoring the prepared subgrade and subgrade surfaces for compliance with the *Technical Specifications* before geosynthetic materials are placed;

- sampling and testing for conformance of the materials to the *Technical Specifications*;

- documenting that the earthwork is constructed using the specified equipment and procedures;

- documenting that the earthwork is constructed to the lines and grades shown on the *Drawings*;

- monitoring that the construction activities do not cause damage to underlying geosynthetic materials;

- quality control testing to determine the acceptability of the work during construction; and

- monitoring the action of the compaction and heavy hauling equipment on the construction surface (i.e., penetration, pumping, cracking, etc.).

The specific activities required for CQA of each of the major soil components of the Final Cover System are presented in the following sections.

7.3.2 Cover Soil Material

Monitoring the earthwork for the cover soil material specifically includes the following:

- reviewing documentation of the qualification and conformance test results;

- monitoring soil for maximum particle size and deleterious materials;

- monitoring the thickness of lifts during placement of the materials;

- monitoring compaction operations; and

- measuring and recording the field density and the field moisture content of the in-place material.

7.3.3 Prepared Subgrade

During construction, the CQA Site Manager will monitor the prepared subgrade to document that the prepared subgrade soil characteristics are consistent with those specified in the *Technical Specifications*. The CQA Site Manager will monitor the construction activities to document that sharp rocks and other undesirable materials are removed and that the subgrade is prepared using the procedures and equipment specified in the *Technical Specifications*.

The upper portion of the subgrade can be damaged by excess moisture (causing softening) or insufficient moisture (causing desiccation and shrinkage). At a minimum, the CQA Site Manager will determine the suitability of the subgrade for geomembrane placement by:

- documenting that the surface is free of sharp rocks, debris and other undesirable materials;

- documenting that the surface is smooth, uniform, and free from desiccation cracks by visually monitoring proof rolling activities; and

- documenting that the subgrade surface meets the lines and grades shown on the *Drawings* by reviewing certified survey results.

7.4 <u>Deficiencies</u>

If a defect is discovered in the earthwork product, the CQA Site Manager will immediately determine the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Site Manager will determine the extent of the deficient 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 surface desiccation, the CQA Site Manager will define the limits and nature of the defect.

7.4.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Project Manager and Contractor and schedule appropriate re-tests when the work deficiency is to be corrected.

7.4.2 Repairs and Re-Testing

At locations where the field testing indicates densities below the requirements of the specification, the failing area will be reworked. 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 Engineer and/or Project Manager suggested solutions for his approval.

All re-tests recommended by the CQA Site Manager must verify 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. DRAINAGE AGGREGATE

8.1 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.

8.2 Testing Activities

Aggregate testing will be performed for material qualification and material conformance. These two stages of testing are defined as follows:

- Material qualification tests are used to evaluate the conformance of a proposed aggregate source to the material 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 material specifications prior to installation of the aggregate.

The Contractor will be responsible for submitting material qualification test results to the Project 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 5 are those that will be used for this testing unless the test methods are updated or revised prior to construction. Revisions to the test methods will be reviewed and approved by the Engineer and the CQA Site Manager prior to their usage.

8.2.1 Sample Frequency

The frequency of aggregate testing for material qualification will conform to the minimum frequencies presented in Table 6. The frequency of aggregate testing for material conformance will conform to the minimum frequencies presented in Table 7. The actual frequency of testing required will be increased by the CQA Site Manager as necessary if variability of materials is noted at the site, during adverse conditions, or to isolate failing areas of the construction.

8.2.2 Sample Selection

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

8.3 CQA Monitoring Activities

8.3.1 Drainage Aggregate

The CQA Site Manager will monitor and document the installation of the drainage aggregates. In general, monitoring the installation of the drainage aggregates 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.

8.4 Deficiencies

If a defect is discovered in the drainage aggregates, the CQA Site Manager will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the CQA Site Manager will determine the extent of the deficient area by additional tests, observations, a review of records, or other means that the CQA Site Manager deems appropriate.

8.4.1 Notification

After evaluating the extent and nature of a defect, the CQA Site Manager will notify the Project Manager and Contractor and schedule appropriate re-tests when the work deficiency is to be corrected.

8.4.2 Repairs and Re-testing

The Contractor will correct the deficiency to the satisfaction of the CQA Site Manager. If a project specification criterion cannot be met, or unusual weather conditions hinder work, then the CQA Site Manager will develop and present to the Engineer and/or Project Manager suggested solutions for approval.

All re-tests recommended by the CQA Site Manager must verify that the defect has been corrected before any additional work is performed by the Contractor in the area of the deficiency. The CQA Site Manager will also verify that installation requirements are met and that submittals are provided.

9. CORRUGATED POLYETHYLENE (CPE) PIPE AND FITTINGS

9.1 Material Requirements

CPE pipe and fittings must conform to the requirements of the *Technical Specifications*. The CQA Consultant will document that the CPE pipe and fittings meet those requirements through manufacturer's quality control certificates, conformance testing, and visual examination of materials arriving on site.

9.2 Manufacturer

9.2.1 Submittals

Prior to the installation of CPE 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

- a certification that property values given in the properties sheet are minimum values and are guaranteed by the Manufacturer.

The CQA Consultant will document that:

- the property values certified by the Manufacturer meet the *Technical Specifications*; and

- the measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

9.2.2 Identification

Prior to shipment, the Manufacturer will provide the Project Manager and the CQA Site Manager with a quality control certificate for each lot/batch of CPE pipe provided. The quality control certificate will be signed by a responsible party employed by the Manufacturer, such as the Production Manager. The quality control certificate will include:

- lot/batch numbers and identification; and
- sampling procedures and results of quality control tests.

The CQA Site Manager will:

- document that the quality control certificates have been provided at the specified frequency for all lots/batches of pipe, and that each certificate identifies the pipe lot/batch related to it; and

- review the quality control certificates and document that the certified properties meet the *Technical Specifications*.

9.3 Handling and Laying

Care will be taken during transportation of the pipe such that it will not be cut, kinked, or otherwise damaged.

Ropes, fabric, or rubber-protected slings and straps will be used when handling pipes. Chains, cables, or hooks inserted into the pipe ends will not be used. Two slings spread apart will be used for lifting each length of pipe. Pipe or fittings will not be dropped onto rocky or unprepared ground.

Pipes will be handled and stored in 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 gauges will be removed and the ends of the pipe rejoined.

9.4 Joints

Lengths of pipe will be assembled into suitable installation lengths by a manufacturer-recommended method.

10. GEOMEMBRANE

10.1 General

This section discusses and outlines the CQA activities to be performed for high density polyethylene (HDPE) geomembrane installation. The CQA Site Manager will review the *Drawings*, and the *Technical Specifications*, and any approved Addenda regarding this material.

10.2 Geomembrane Material Conformance

10.2.1 Introduction

The CQA Site Manager will document that the geomembrane delivered to the site meets the requirements of the *Technical Specifications* prior to installation. The CQA Site Manager will:

- review the manufacturer's submittals for compliance with the *Technical Specifications*;
- document the delivery and proper storage of geomembrane rolls; and
- conduct conformance testing of the rolls before the geomembrane is installed.

The following sections describe the CQA activities required to verify the conformance of geomembrane.

10.2.2 Review of Quality Control

10.2.2.1 Material Properties Certification

The Manufacturer will provide the Project Manager and the CQA Site Manager with the following:

- a properties sheet including, at a minimum, all specified properties, measured using test methods indicated in the *Technical Specifications*, or equivalent;

- the sampling procedure and results of testing; and

- a certification that property values given in the properties sheet are guaranteed by the Manufacturer.

The CQA Site Manager will document that:

- the property values certified by the Manufacturer meet all of the *Technical Specifications*; and

- the measurements of properties by the Manufacturer are properly documented and that the test methods used are acceptable.

10.2.2.2 Resin Certification

The Manufacturer will also provide the Project Manager with the following information concerning the resin used to manufacture the geomembrane:

- the origin (Resin Supplier's name and resin production plant), identification (brand name, lot number), and production date of the resin; and

- the raw material quality control certificates.

The CQA Site Manager will:

- evaluate that the quality control certificates have been provided at the specified frequency, and that the certificate identifies the rolls related to it; and

- review the quality control certificates and evaluate that the certified properties meet the specifications.

10.2.2.3 Geomembrane Roll QC Certification

Prior to shipment, the Manufacturer will provide the Project Manager and the CQA Site Manager with a quality control certificate for every roll of geomembrane provided. The quality control certificate will be signed by a responsible party employed by the Geomembrane Manufacturer, such as the production manager. The quality control certificate will include:

- roll numbers and identification; and

- results of quality control tests - as a minimum, results will be given for thickness, tensile properties, specific gravity, carbon black content, carbon black dispersion, tear resistance, puncture resistance, and single point stress rupture evaluated in general accordance with the methods indicated in the specifications or equivalent methods approved by the Engineer.

The CQA Site Manager will:

- evaluate that the quality control certificates have been provided at the specified frequency, and that the certificate identifies the rolls related to the roll represented by the test results; and

- review the quality control certificates and evaluate that the certified roll properties meet the specifications.

10.2.3 Conformance Testing

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 will not be installed. The CQA Site Manager shall obtain conformance samples 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 8. 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 (1 m) of material. Unless otherwise specified, samples will be 3 ft (1 m) 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 8.

The CQA Site Manager will examine results from laboratory conformance testing and will report any non-conformance to the Project Manager and the Geosynthetic Installer. The procedure prescribed in the *Technical Specifications* will be followed in the event of a failing conformance test.

10.3 <u>Delivery</u>

10.3.1 Transportation and Handling

The CQA Site Manager will document that the transportation and handling does not pose a risk of damage to the geomembrane.

Upon delivery at the site, the Geosynthetic Installer and the CQA Site Manager will conduct a surface observation of the rolls for defects and damage. This inspection will be conducted without unrolling unless defects or damages are found or suspected. The CQA Site Manager will indicate to the Project Manager:

- rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and

- rolls that include minor repairable flaws.

10.3.2 Storage

The Geosynthetic Installer will be responsible for the storage of the geomembrane on site. The Contractor will provide storage space in a location (or several locations) such that on-site transportation and handling are optimized if possible.

The CQA Site Manager will document that storage of the geomembrane provides adequate protection against sources of damage.

10.4 Geomembrane Installation

10.4.1 Introduction

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

10.4.2 Earthwork

10.4.2.1 Surface Preparation

The CQA Site Manager will document that:

- a qualified land surveyor has verified lines and grades;

- that the supporting prepared subgrade or subgrade meets 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 the 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 will be given by the Geosynthetic Installer to the Project Manager prior to commencement of geomembrane installation in the area under consideration. The CQA Site Manager will be given a copy of this certificate by the Project Manager.

After the supporting subgrade has been accepted by the Geosynthetic Installer, it will be the Geosynthetic Installer's responsibility to indicate to the Project Manager any change in the supporting soil condition that may require repair work. If the CQA Site Manager concurs with the Geosynthetic Installer, then the Project Manager will document that the supporting soil is repaired.

At any time before and during the geomembrane installation, the CQA Site Manager will indicate to the Project Manager locations that may not provide adequate support to the geomembrane.

10.4.2.2 Geosynthetic Termination

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

10.4.3 Geomembrane Placement

10.4.3.1 Panel Identification

A field panel is the unit area of geomembrane which is to be seamed in the field, i.e., a field panel is a roll or a portion of roll cut in the field. It will be the responsibility of the CQA Site Manager to document that each field panel is given an "identification code" (number or letter- number) consistent with the layout plan. This identification code will be agreed upon by the Project 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 quality assurance records.

10.4.3.2 Field Panel Placement

Location

The CQA Site Manager will document that field panels are installed at the location indicated in the Geosynthetic Installer's layout plan, as approved or modified by the Engineer.

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 Project Manager on the acceptability of that change. The CQA Site Manager will document that the condition of the supporting 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 104°F;
- when the geomembrane sheet temperature is below 40°F or above 104°F; or
- when wind gusts are in excess of 20 mph.

Geomembrane placement will not be performed 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 (i.e., wind gusts in excess of 20 mph).

The CQA Site Manager will document that the above conditions are fulfilled. Additionally, the CQA Site Manager will document that the supporting soil has not been damaged by weather conditions. The Geosynthetics Installer will inform the Project 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 Project 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 10.4.5.

10.4.4 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.

10.4.4.1 Seam Layout

The Geosynthetic Installer will provide the Project Manager and the CQA Site Manager with a seam layout drawing, i.e., a drawing of the facility to be lined showing all expected seams. The CQA Site Manager will review the seam layout drawing and evaluate that it is consistent with the preliminary geomembrane panel layout. No panels may be seamed in the field without the Project Manager's approval. In addition, panels not specifically shown on the seam layout drawing may be used without the Project Manager's prior approval.

In general, seams should be oriented parallel to the line of maximum slope, i.e., oriented along, not across, the slope. In corners and odd-shaped geometric locations, the number of seams should be minimized. No horizontal seam should be less than 5 ft (1.5 m) from the toe of the slope, or areas of potential stress concentrations, unless otherwise authorized.

A seam numbering system compatible with the panel numbering system will be agreed upon at the Resolution and/or Pre-Construction Meeting.

10.4.4.2 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 Project Manager and the CQA Site Manager with a list of proposed seaming personnel and their experience records. This document will be reviewed by the Project Manager and the Geosynthetics CQA Manager.

10.4.4.3 Seaming Equipment and Products

Approved processes for field seaming are fillet extrusion welding and 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 extrudate to the Project Manager and the CQA Site Manager, and will certify that the extrudate is compatible with the specifications, and in any event is comprised of the same resin as the geomembrane sheeting.

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 Resolution 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 pressures.

The CQA Site Manager will also document that:

- the Geosynthetic Installer maintains on-site the number of spare operable seaming apparatus decided at the Resolution 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 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.

10.4.4.4 Seam Preparation

The CQA Site Manager will document that:

- prior to seaming, the seam area is clean and free of moisture, dust, dirt, debris, and foreign material; and

- seams are aligned with the fewest possible number of wrinkles and "fishmouths."

10.4.4.5 Weather Conditions for Seaming

The normally required weather conditions for seaming are as follows unless authorized in writing by the Project Manager:

- seaming will only be approved between ambient temperatures of 40°F (4°C) and 104°F (40°C); and

- seaming will not be approved if sustained wind speed is in excess of 20 mph (32 km/hr).

If the Geosynthetic Installer wishes to use methods that may allow seaming at ambient temperatures below 40°F (4°C) or above 104°F (40°C), the Geosynthetic Installer will demonstrate and certify that such methods produce seams which are entirely equivalent to seams produced within acceptable temperature and wind requirements, 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 Project Manager if they are not. The Project Manager will then decide if the installation will be stopped or postponed.

10.4.4.6 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. (75 mm) for both extrusion and fusion welding;

- no solvent or adhesive bonding material are to be used; and

- the procedure used 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 to the Project Manager non-compliances.

10.4.4.7 Trial Seams

Trial seams will be made on fragment pieces of geomembrane liner to verify that seaming conditions are adequate. Such trial seams will be made at the beginning of each seaming period, beginning of the day and after lunch, for each seaming apparatus used that day. Also, each seamer will make at least one trial seam each day. Trial seams will be made under the same conditions as actual seams.

Extrusion welded trial seam samples will be at least 3 ft (0.9 m) long by 1 ft (0.3 m) wide (after seaming) with the seam centered lengthwise. Fusion welded trial seam samples will be at least 5 ft (1.5 m) long by 1 ft (0.3 m) wide (after seaming) with the seam centered lengthwise. Seam overlap will be as indicated in Section 10.5.3.6.

Four specimens, each 1 in. (25 mm) wide, will be cut from the trial seam sample by the Geosynthetic Installer. One specimen will be tested for shear strength and three specimens will be tested for peel adhesion using a gauged tensiometer. All specimens tested will exhibit a Film Tear Bond (FTB) and will not fail in the seam. In addition, all specimens will meet or exceed the minimum strength requirements described in the *Technical Specifications*. If any of the four specimens fails, the entire trial seaming operation will be repeated. If any of the four additional specimens fails, the seaming apparatus and seamer will not be approved for production seaming until the deficiencies are corrected and two consecutive trial seam tests achieve the FTB requirements outlined above.

The CQA Site Manager will observe trial seam procedures. Trial seam samples will be assigned a number. The CQA Site Manager, will log the date, time, machine temperature(s), number of the seaming unit, name of the seamer, and pass or fail description for each trial seam sample tested.

10.4.4.8 General Seaming Procedure

Unless otherwise specified, the general seaming procedure used by the Geosynthetic Installer will be as follows:

- Fishmouths or wrinkles at the seam overlaps will be cut along the ridge of the wrinkle in order to achieve a flat overlap. The cut fishmouths or wrinkles will be seamed and any portion where the overlap is inadequate will then be patched with an oval or round patch of the same geomembrane extending a minimum of 6 in. (150 mm) beyond the cut in all directions.

- 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 will document that the above seaming procedures are followed, and will inform the Project Manager if they are not.

10.4.4.9 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 approved method. 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, test unit number, name of person conducting the test, and the results of tests; and

- inform the Geosynthetic Installer and Project Manager of required repairs.

The Geosynthetic Installer will complete any required repairs in general accordance with Section 10.4.5.

The CQA Site Manager will:

- observe the repair and re-testing of the repair;
- mark on the geomembrane that the repair has been made; and
- document the results.

The following procedures will apply to locations where seams cannot be non-destructively tested:

All such seams will be cap-stripped with the same geomembrane.

- If the seam is accessible to testing equipment prior to final installation, the seam will be non-destructively tested prior to final installation.

- If the seam cannot be tested prior to final installation, the seaming and cap-stripping operations will be observed by the CQA Site Manager and Geosynthetic Installer for uniformity and completeness.

The seam number, date of observation, name of tester, and outcome of the test or observation will be recorded by the CQA Site Manager.

Vacuum Testing

The equipment will be comprised of the following:

- a vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge;
- a steel vacuum tank and pump assembly equipped with a pressure controller and pipe connections;
- a rubber pressure/vacuum hose with fittings and connections;
- an approved applicator; and
- a soapy solution.

The following procedures will be followed:

- energize the vacuum pump and reduce the tank pressure to approximately 5 psi (35 kPa) (10 in. of Hg.) gauge;
- wet a strip of geomembrane approximately 12 in. by 48 in. (0.3 m by 1.2 m) with the soapy solution;
- place the box over the wetted area;
- close the bleed valve and open the vacuum valve;
- document that a leak tight seal is created;

- for a period of not less than ten seconds, examine the geomembrane through the viewing window for the presence of leaks indicated by soap bubbles;

- if no leaks appear after ten seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3 in. (75 mm) overlap, and repeat the process;

- areas where soap bubbles appear will be marked and repaired in general accordance with Section 10.4.5 and retested using the vacuum testing method.

Air Pressure Testing (For Double-Track Fusion Seam Only)

The following procedures are applicable to those processes that produce a double seam with an enclosed space.

The equipment will be comprised of the following:

- an air pump (manual or motor driven) equipped with pressure gauge capable of generating and sustaining a pressure of 30 psi (200 kPa) and mounted on a cushion to protect the geomembrane;

- a rubber hose with fittings and connections;

- a sharp hollow needle, or other approved pressure feed device.

The following procedures will be followed:

- seal both ends of the seam to be tested;

- insert needle or other approved pressure feed device into the tunnel created by the fusion weld;

- insert a protective cushion between the air pump and the geomembrane;

- energize the air pump to a pressure of 25 to 30 psi (170 to 204 kPa), close valve, and sustain pressure for not less than 5 minutes;

- if loss of pressure exceeds 3 psi (20 kPa) or does not stabilize, locate faulty area and repair in general accordance with Section 10.4.5;

- cut end of tested seam area, opposite the location of the pressure gauge, after completion of the five minute pressure hold period to verify complete testing of the seam. If the pressure gauge does not indicate a release of pressure, locate blockage of the air channel and retest until entire seam is tested; and

- remove needle or other approved pressure feed device and repair any holes in the geomembrane resulting from the air pressure testing procedure in general accordance with Section 10.4.5.

10.4.4.10 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 phase.

- Test locations will be evaluated during seaming at 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 cut by the Geosynthetic Installer 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;
- assign a number to each sample, and mark it accordingly;
- record sample location on 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 10.4.5. The continuity of the new seams in the repaired area will be tested in general accordance with Section 10.4.4.9.

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 Contractor 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. Five 1 in. (25 mm) wide strips will be taken for peel. 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 log 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 Project Manager will document that packaging and shipping conditions are acceptable. The Project Manager will be responsible for storing the archive samples. This procedure will be outlined at the Resolution 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 Project Manager.

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 Project Manager.

Geosynthetic Installer's Laboratory Testing

The Geosynthetic Installer's laboratory test results will be presented to the Project 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. In cases where the failed seam segment exceeds 150 ft (50 m), a destructive sample will be taken from the zone in which the seam has been reconstructed. Repairs will be made in general accordance with Section 10.4.5.

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

10.4.5 Defects and Repairs

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

10.4.5.1 Identification

Seams and non-seam areas of the geomembrane will 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 will be clean at the time of examination.

10.4.5.2 Evaluation

Each suspect location both in seam and non-seam areas will be non-destructively tested using the methods described in Section 10.4.4.9 as appropriate.

Each location that fails the nondestructive testing will be marked by the CQA Site Manager and repaired by the Geosynthetic Installer. Work will not proceed with any materials that will cover locations which have been repaired until laboratory test results with passing values are available.

10.4.5.3 Repair Procedures

Portions of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, will be repaired. Several procedures exist for the repair of these areas. The final decision as to the appropriate repair procedure will be at the discretion of the CQA Consultant with input from the Project 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 Project Manager 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); and

- the geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

10.4.5.4 Verification of Repairs

Each repair will be numbered and logged. Each repair will be non-destructively tested using the methods described in Section 10.4.4.9 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, at the discretion of the CQA Site Manager. Failed tests indicate that the repair will be redone and re-tested until a passing test results. The CQA Site Manager will observe all non-destructive testing of repairs and will record the number of each repair, date, and test outcome.

10.4.5.5 Large Wrinkles

When seaming of the geomembrane is completed (or when seaming of a large area of the geomembrane liner is completed) and prior to placing overlying materials, the CQA Site Manager will observe the geomembrane wrinkles. The CQA Site Manager will indicate to the Project Manager which wrinkles should be cut and re-seamed by the Geosynthetic Installer. The seam thus produced will be tested like any other seam.

10.4.6 Lining System Acceptance

The Geosynthetic Installer and the Manufacturer(s) will retain all responsibility for the geosynthetic materials in the liner system until acceptance by the Owner.

The geosynthetic liner system will be accepted by the Owner 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
- CQA report, including "as built" drawing(s), sealed by a registered professional engineer has been received by the Project Manager.

The CQA Site Manager will document that installation has proceeded in general accordance with the *Technical Specifications* for the project except as noted to the Project Manager.

11. GEOTEXTILE

11.1 Introduction

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

11.2 Manufacturing

The Manufacturer will provide the Project 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 Project 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 quality control testing.

The Manufacturer will provide, as a minimum, test results for the following:

- grab strength;
- tear strength;
- burst strength;
- puncture strength;
- permittivity; and
- apparent opening size.

Quality control tests must be performed, in general accordance with the test methods specified in Table 9, on geotextile produced for the project. The Manufacturer will also provide a written certification that the nonwoven, needle-punched geotextiles are continuously inspected and found to be needle-free.

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 Project Manager.

11.3 Labeling

The Manufacturer will identify all rolls of geotextile with the following:

- manufacturer's name;
- product identification;
- lot number;
- roll number; and
- roll dimensions.

The CQA Site Manager will examine rolls upon delivery and deviation from the above requirements will be reported to the Project Manager.

11.4 Shipment and Storage

During shipment and storage, the geotextile will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. To that effect, geotextile rolls will be shipped and stored in relatively opaque and watertight wrappings.

Protective wrappings will be removed less than one hour prior to unrolling the geotextile. After the wrapping has been removed, a geotextile will not be exposed to sunlight for more than 15 days, except for UV protection geotextile, unless otherwise specified and guaranteed by the Manufacturer.

The CQA Site Manager will observe rolls upon delivery at the site and deviation from the above requirements will be reported to the Project Manager.

11.5 Conformance Testing

11.5.1 Tests

Upon delivery of the rolls of geotextiles, the CQA Site Manager will document that samples are removed and forwarded 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 9.

These conformance tests will be performed in general accordance with the test methods specified in the *Technical Specifications*.

11.5.2 Sampling Procedures

Samples will be taken across the width of the roll and will not include the first three feet (linear meter). Unless otherwise specified, samples will be 3 ft (1 m) 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 9 for geotextiles.

11.5.3 Test Results

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

11.5.4 Conformance Sample Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of geotextile that is in nonconformance with the *Technical Specifications* with a roll(s) that meets *Technical Specifications*.

- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these samples fail, every roll of geotextile on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*. This additional conformance testing will be at the expense of the Manufacturer.

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

11.6 <u>Handling and Placement</u>

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 Project Manager.

11.7 Seams and Overlaps

Geotextiles will be overlapped a minimum of 12 in. (0.3 m) prior to seaming.

11.8 Repair

Holes or tears in the geotextile will be repaired as follows:

- 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 Project Manager.

11.9 Placement of Soil Materials

The Contractor will place all soil 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 Project Manager.

12. GEOSYNTHETIC CLAY LINER (GCL)

12.1 Introduction

This section of the CQA Plan outlines the CQA activities to be performed for the geosynthetic clay liner (GCL) installation. The CQA Consultant will review the *Drawings*, and the *Technical Specifications*, and approved addenda or changes.

12.2 Manufacturing

The Manufacturer will provide the Project 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 Project 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 10, 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 Project Manager.

12.3 <u>Labeling</u>

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 Project Manager.

12.4 <u>Shipment and Storage</u>

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 Project Manager.

12.5 <u>Conformance Testing</u>

12.5.1 Tests

CQA personnel will sample the GCL either during production at the manufacturing facility or after delivery to the construction site. The samples will be forwarded to the Geosynthetics CQA Laboratory for testing to assess conformance with the *Technical Specifications*. The test methods and minimum testing frequencies are indicated in Table 10.

Samples will be taken across the width of the roll and will not include the first 3 ft (0.9 m) if the sample is cut on site. Unless otherwise specified, samples will be 3 ft (0.9 m) 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 Project Manager.

12.5.2 Conformance Sample Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of GCL that is in nonconformance with the *Technical Specifications* with a roll(s) that meets *Technical Specifications*.

- The Geosynthetic Installer will remove conformance samples for testing by the CQA Laboratory from the closest numerical rolls on both sides of the failed roll. These two samples must conform to the *Technical Specifications*. If either of these samples fail, the numerically closest rolls on the side of the failed sample will be tested by the CQA Laboratory. These samples must conform to the *Technical Specifications*. If any of these samples fail, every roll of GCL on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*. This additional conformance testing will be at the expense of the Manufacturer.

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

12.6 GCL Delivery and Storage

Upon delivery to the site, the CQA Consultant will check the GCL rolls for defects (e.g., tears, holes) and for damage. The CQA Consultant will report to the Project Manager and the Geosynthetics Installer:

- any rolls, or portions thereof, which should be rejected and removed from the site because they have severe flaws; and
- any rolls which include minor repairable flaws.

The GCL rolls delivered to the site will be checked by the CQA Consultant to document that the roll numbers correspond to those on the approved Manufacturer's quality control certificate of compliance.

12.7 GCL Installation

The CQA Consultant will monitor and document that the GCL is installed in 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 Project Manager.

13. GEOCOMPOSITE

13.1 Introduction

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

13.2 Manufacturing

The Manufacturer will provide the CQA Consultant with a list of certified "minimum average roll value" properties for the type of geocomposite 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 geocomposite actually delivered have "minimum average roll values" properties which meet or exceed all certified property values for that type of geocomposite.

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 geocomposite (geotextile and geonet). Deviations will be reported to the Project Manager.

13.3 Labeling

The Manufacturer will identify all rolls of geocomposite 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 Project Manager.

13.4 Shipment and Storage

During shipment and storage, the geocomposite will be protected from ultraviolet light exposure, precipitation or other inundation, mud, dirt, dust, puncture, cutting or any other damaging or deleterious conditions. Therefore, geocomposite rolls will be shipped and stored in relatively opaque and watertight wrappings. The CQA Site Manager will observe rolls upon delivery to the site and deviation from the above requirements will be reported to the Project Manager. Damaged rolls will be rejected and replaced.

Wrapping protecting geocomposite rolls will be removed less than one hour prior to unrolling geocomposite before placement. After the wrapping has been removed, geocomposite should not be exposed to sunlight for more than 15 days, unless otherwise approved by the Manufacturer. Approval by the Manufacturer will be a guarantee that the properties of the exposed geotextile will not degrade upon prolonged exposure to such values that would cause the material to not meet the *Technical Specifications*. Any material that is exposed for more than 15 days, which has been approved for prolonged exposure by the Manufacturer, will be tested by the CQA Laboratory to document that the material properties are still in conformance with the *Technical Specifications*. Any material that fails to meet the *Technical Specifications* will be replaced by the Manufacturer.

The CQA Site Manager will observe that geocomposite is free of dirt and dust just before installation. The CQA Site Manager will report the outcome of this observation to the Project Manager, and if the geocomposite is judged dirty or dusty, they will be cleaned by the Geosynthetic Installer prior to installation.

13.5 Conformance Testing

13.5.1 Tests

The geocomposite material will be tested for transmissivity (ASTM D 4716) and for peel strength (ASTM D 413) at the frequencies presented in Table 11.

13.5.2 Sampling Procedures

Upon delivery of the geocomposite rolls, the CQA Site Manager will document that samples are obtained from individual rolls at the frequency specified in this CQA Plan. The geocomposite 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 (1 linear m). Unless otherwise specified, samples will be 3 ft (1 m) long by the roll width. The CQA Consultant will mark the machine direction on the samples with an arrow.

13.5.3 Test Results

The CQA Site Manager will examine results from laboratory conformance testing and compare results to the *Technical Specifications*. The criteria used to evaluate acceptability are presented in the *Technical Specifications*. The CQA Site Manager will report any nonconformance to the Project Manager.

13.5.4 Conformance Test Failure

The following procedure will apply whenever a sample fails a conformance test that is conducted by the CQA Laboratory:

- The Manufacturer will replace every roll of geocomposite that is in nonconformance with the *Technical Specifications* with a roll that meets specifications.

- 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 geocomposite on site from this lot and every subsequently delivered roll that is from the same lot must be tested by the CQA Laboratory for conformance to the *Technical Specifications*.

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

13.6 Handling and Placement

The Geosynthetic Installer will handle all geocomposite in such a manner as to document they are not damaged in any way. The Geosynthetic Installer will comply with the following:

- In the presence of wind, the geocomposite will be weighted with sandbags or the equivalent. Sandbags will be used during installation only and will remain until replaced with the appropriate cover material.

- If in place, special care must be taken to protect other materials from damage, which could be caused by the cutting of the geocomposite.

- The Geosynthetic Installer will take any necessary precautions to prevent damage to underlying layers during placement of the geocomposite.

- During placement of geocomposite, 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 geocomposite, 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 geocomposite 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 Project Manager.

13.7 Geocomposite Seams and Overlaps

Adjacent geocomposite panels will be joined in general accordance with *Construction Drawings* and *Technical Specifications*. As a minimum, the following requirements will be met:

- Adjacent rolls will be overlapped by at least 4 in. (100 mm).

- Each component of the geocomposite will be secured or seamed to the like component at overlaps.

- The geocomposite overlaps will be secured by tying, in general accordance with the *Technical Specifications*.

- The bottom layers of geotextile, if applicable, will be overlapped.

- The top layers of geotextile will be continuously sewn.

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

13.8 Repair

Holes or tears in the geocomposite will be repaired by placing a patch extending 2 ft (0.6 m) beyond edges of the hole or tear. The patch will be secured by tying with approved tying devices every 6 in. (150 mm) through the bottom geotextile and the geonet of the patch, and through the top geotextile and geonet components of the geocomposite needing repair. The top geotextile component of the patch will be heat sealed to the top geotextile of the geocomposite needing repair. 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 geocomposite will be joined in general accordance with Section 13.7.

The CQA Site Manager will observe repairs, note noncompliances with the above requirements and report them to the Project Manager.

13.9 Placement of Soil Materials

The Contractor will place all soil materials located on top of a geocomposite in such a manner as to document:

- the geocomposite and underlying liner materials are not damaged;
- minimal slippage of the geocomposite on underlying layers occurs; and
- no excess tensile stresses occur in the geocomposite.

Unless otherwise specified by the CQA Consultant, lifts of soil material will be in conformance with the *Technical Specifications*. If portions of the geocomposite are exposed, the CQA Consultant will periodically place marks on the geocomposite and the underlying geomembrane and measure the elongation of the geonet during the placement of soil.

Noncompliance will be noted by the CQA Consultant and reported to the Project Manager.

14. SURVEYING

14.1 Survey Control

Survey control will be performed by the Owner 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.

14.2 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. Surveys shall be performed at 2nd order accuracy.

14.3 Lines and Grades

The following surfaces will be surveyed to verify the lines and grades achieved during soil placement and compaction.

- Prepared Subgrade:
  - prepared subgrade surface.
- Cover Soil:
  - finished compacted cover soil surface.

The following structures will be surveyed to verify and document the lines and grades achieved during construction of the Project:

- all culverts, inlet, and drop structures;

- ditch bottoms and sideslopes;

- permanent erosion control features;

- geomembrane terminations and selected geomembrane seams, as indicated by the CQA Manager; and

- centerlines of pipes.

14.4 Frequency and Spacing

Surveying should be carried out immediately upon completion of a given installation to facilitate progress and avoid delaying commencement of the next installation. In addition, spot checks during placement and compaction will be necessary to assist the Contractor in compliance with required grades.

At the least the following minimum spacings and locations should be provided for survey points:

- all "flat" surfaces with gradients less than 10 percent, should be surveyed on a square grid not wider spaced than 100 ft (30 m);

- on all slopes greater than 10 percent, a square grid not wider than 100 ft (30 m) should be used, but in any case, a line at the crest, midpoint, and toe of the slope should be taken;

- a line of survey points no further than 100 ft (30 m) apart must be taken along any slope break (this will include the inside edge and outside edge of any bench on a slope); and

- a line of survey points no further than 50 ft (15 m) apart must be taken at the invert of pipes or other appurtenances to the liner.

14.5 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 Engineer in AutoCADD V.14 format or other suitable format as directed by the Owner.

TABLE 1
TEST PROCEDURES FOR THE EVALUATION OF SOILS

TEST METHOD	DESCRIPTION	TEST STANDARD
Laboratory Test Procedures:		
Classification	Classification of Soils	ASTM D 2487
Moisture Content	Moisture Content	ASTM D 2216
Modified Proctor	Moisture/Density Relationship of Soil (10 lb (4.54 kg) rammer and 18 in. (457 mm) drop)	ASTM D 1557
Hydrometer Analysis	Particle Size Distribution of Fine Fraction of Soils	ASTM D 422
Sieve Analysis	Particle Size Distribution of Coarse Fraction of Soils	ASTM D 422
Field Test Procedures:		
Nuclear Densometer	In Situ Soil Unit Weight In Situ Moisture Content	ASTM D 2922 ASTM D 3017
Sand Cone	In Situ Soil Unit Weight Moisture Content	ASTM D 1556 ASTM D 2216
Drive Cylinder	In Situ Soil Unit Weight Moisture Content	ASTM D 2937 ASTM D 2216

TABLE 2

MINIMUM SOILS TESTING FREQUENCIES FOR MATERIAL QUALIFICATION TESTING

TEST	ENGINEERED FILL
Moisture Content	1 per source
Sieve Analysis	1 per source
Hydrometer Analysis	1 per source
Soil Classification	1 per source
Modified Proctor	1 per source

TABLE 3

MINIMUM SOILS TESTING FREQUENCIES FOR CONFORMANCE TESTING

TEST	COVER SOIL
Moisture Content	1 per 10,000 yd^3 (7,646 m^3)
Sieve Analysis	1 per 10,000 yd^3 (7,646 m^3)
Hydrometer Analysis	1 per 10,000 yd^3 (7,646 m^3)
Soil Classification	1 per 10,000 yd^3 (7,646 m^3)
Modified Proctor	1 per 10,000 yd^3 (7,646 m^3)

TABLE 4

MINIMUM SOIL TESTING FREQUENCIES FOR CONSTRUCTION QUALITY CONTROL

TEST	ENGINEERED FILL
Nuclear densometer	1 per 750 yd^3
Sand cone or drive cylinder	1 per 20 nuclear densometer tests

Notes: (1) Nuclear densometer testing of the first lift of cover soil placed above the final cover system geosynthetics shall be performed at a depth no greater than 6 in. (i.e., 8-in. deep hole in 12-in. thick cover).

TABLE 5

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 6

MINIMUM AGGREGATE TESTING FREQUENCIES FOR MATERIAL QUALIFICATION TESTING

TEST	DRAINAGE AGGREGATE
Sieve Analysis	1 per source
Hydraulic Conductivity	1 per source

TABLE 7

MINIMUM AGGREGATE TESTING FREQUENCIES FOR CONFORMANCE TESTING

TEST	TEST METHOD	DRAINAGE AGGREGATE
Sieve Analysis	ASTM C 136	1 per 5,000 yd^3 (3,823 m^3)
Hydraulic Conductivity	ASTM D 2434	1 per 10,000 yd^3 (7,646 m^3)

TABLE 8

GEOMEMBRANE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	FREQUENCY
Specific Gravity	ASTM D 792 Method A or ASTM D 1505	100,000 ft^2 (9,290 m^2)
Thickness	ASTM D 5994 or ASTM D 5199	100,000 ft^2 (9,290 m^2)
Tensile Strength at Yield	ASTM D 638	100,000 ft^2 (9,290 m^2)
Tensile Strength at Break	ASTM D 638	100,000 ft^2 (9,290 m^2)
Elongation at Yield	ASTM D 638	100,000 ft^2 (9,290 m^2)
Elongation at Break	ASTM D 638	100,000 ft^2 (9,290 m^2)
Carbon Black Content	ASTM D 1603	100,000 ft^2 (9,290 m^2)
Carbon Black Dispersion	ASTM D 5596	100,000 ft^2 (9,290 m^2)

TABLE 9

GEOTEXTILE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
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 10

GCL CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Mass per Unit Area	ASTM D 3776	100,000 ft^2 (9,290 m2)
Index Flux	ASTM D 5084	400,000 ft^2 (37,160 m^2)
Residual Shear Strength	ASTM D 5321	See Technical Specifications

TABLE 11

GEOCOMPOSITE CONFORMANCE TESTING REQUIREMENTS

TEST NAME	TEST METHOD	MINIMUM FREQUENCY
Peel Strength	ASTM D 413	1 test per 200,000 ft^2 (18,580 m^2)
Hydraulic Transmissivity	ASTM D 4716	1 test per 200,000 ft^2 (18,580 m^2)

Note: Testing will be carried out at a frequency of one per lot or at listed frequency, whichever yields the greater number of samples.

Section 7
Final Cover Technical Specifications

TECHNICAL SPECIFICATIONS
FOR THE
CONSTRUCTION OF

FINAL COVER SYSTEM
AT
CORRECTIVE ACTION MANAGEMENT UNIT
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HENDERSON, NEVADA

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3 November 2006

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SECTION 02200
EARTHWORK

PART 1 — GENERAL

1.01 DESCRIPTION OF WORK

A. The Contractor shall furnish all labor, materials, tools, supervision, transportation, equipment, and incidentals necessary to perform all Earthwork. The work shall be carried out as specified herein and in accordance with the Drawings.

B. The Work shall include, but not be limited to excavating, hauling, placing, moisture conditioning, backfilling, compacting, grading, and subgrade preparation. 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 02772 — Geosynthetic Clay Liner

Section 02773 — Geocomposite

1.03 REFERENCES

A. Drawings.

B. Site Construction Quality Assurance (CQA) Plan.

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 422	Standard Method for Particle-Size Analysis of Soils
ASTM D 1557	Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))
ASTM D 2216	Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
ASTM D 2487	Standard Test Method for Classification of Soils for Engineering Purposes
ASTM D 2922	Standard Test Methods for Density of Soil and Soil-Aggregate In-Place by Nuclear Density Methods (Shallow Depth)
ASTM D 3017	Standard Test Method for Water Content of Soil and Rock In-Place by Nuclear Methods (Shallow Depth)

1.04 SUBMITTALS

A. The Contractor shall submit to the Engineer a description of equipment and methods proposed for Cover Soil, Anchor Trench Backfill, and Prepared Subgrade placement and compaction at least 7 days prior to the start of activities covered by this Section.

B. If the work of this Section is interrupted for reasons other than inclement weather, the Contractor shall notify the Engineer a minimum of 24 hours prior to the resumption of work.

C. The Contractor shall provide the Engineer with sufficient time to perform as-built surveys of the completed cover soil and prepared subgrade.

D. If foreign borrow materials are proposed for any earthwork material on this project, the Contractor shall provide the Engineer information regarding the source of the material. In addition, the Contractor shall provide the Engineer an opportunity to obtain the necessary samples for conformance testing, prior to delivery of foreign borrow materials to the site.

1.05 QUALITY ASSURANCE

A. The Contractor shall ensure that the materials and methods used for Earthwork meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Engineer will be rejected and shall be repaired or replaced by the Contractor.

B. 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 Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Contractor will be required to repair the deficiency or replace the deficient materials.

PART 2 — PRODUCTS

2.01 MATERIALS

A. Cover soil shall consist of relatively homogeneous, natural soils that are free of debris, foreign objects, large rock fragments (greater than 6 inches in maximum dimension), roots, and organics. The first lift of cover soil placed directly overlying the geosynthetic components of the cover system shall have a maximum particle size of 1 inch. The cover soil shall be classified according to the Unified Soil Classification System (per ASTM D 2487) as SC, ML, CL, SM, SW, GW, GM, GC, or combinations of these materials. The Contractor may propose the use of other soil types as cover soil, but then such use shall be at the sole discretion of the Engineer.

B. Prepared subgrade is defined as the material directly underlying the geosynthetic liner system which shall meet the requirements listed above for cover soil. No materials larger than 1.5 inch shall project or protrude from the surface of the prepared subgrade.

C. Anchor Trench Backfill materials shall meet the requirements listed above for the cover soil.

2.02 EQUIPMENT

A. The Contractor shall furnish, operate, and maintain compaction equipment as is necessary to produce the required in-place soil density and moisture content.

B. The Contractor shall furnish, operate and maintain tank trucks, pressure distributors, or other equipment designed to apply water uniformly and in controlled quantities to variable surface widths.

C. The Contractor shall furnish, operate, and maintain miscellaneous equipment such as scarifiers or disks, earth excavating equipment, earth hauling equipment, and other equipment, as necessary for Earthwork construction.

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:

- 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.

- 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 continuance of the work of this Section will be construed as Contractor's acceptance of the related work of all other Sections.

3.02 SITE PREPARATION

A. Prior to performing any earthworks on the site, the Contractor shall perform a baseline topographic survey. This survey shall be conducted by a Professional Land Surveyor licensed in the state of Nevada. This survey will serve as the starting point for earthwork quantities.

3.03 ANCHOR 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 or cover soil shall be stockpiled in an area designated by the Owner in accordance with Part 3.06 of this Section.

3.04 SUBGRADE SURFACE PREPARATION

A. The subgrade shall be prepared and made suitable as a foundation for placement and compaction of soil material, where applicable. The subgrade shall be firm and able to support the Contractor's construction equipment without the development of depressions or ruts. In addition, the subgrade shall provide adequate support such that the overlying fill material may be placed and compacted to the specified density.

3.05 PREPARED SUBGRADE

A. The prepared subgrade shall be made suitable as a foundation for placement of the geosynthetic components of the liner system (prepared subgrade). The prepared subgrade shall be firm, meet the requirements outlined in Part 2.01, and be able to support the geosynthetic components of the liner system.

3.06 STOCKPILING

A. Soil shall be stockpiled in areas designated by the Owner and shall be free of incompatible soil, clearing, clearing debris, or other objectionable materials.

B. Stockpiles shall be no steeper than 2.1H:1V (Horizontal:Vertical) or other slope approved by the Engineer, graded to drain, sealed by tracking parallel to the slope with a dozer or other means approved by the 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 stockpile areas.

3.07 ANCHOR TRENCH BACKFILL

A. The Anchor Trench Backfill shall be placed to the lines and grades shown on the Drawings.

B. Soil used for the Anchor Trench Backfill shall meet the requirements of Part 2.01 of this Section.

C. Soil used for the Anchor Trench Backfill shall be placed in a loose lift that results in a compacted lift thickness of no greater than 12 inches. The maximum permissible pre-compaction soil clod size is 6 inches.

D. The Contractor shall compact each lift to at least 90 percent of its modified Proctor maximum dry density (ASTM D 1557). The Contractor shall utilize compaction equipment suitable for achieving the soil compaction requirements.

3.08 COVER SOIL

A. The Cover Soil shall be placed to the lines and grades shown on the Drawings.

B. Soil used for the Cover Soil shall meet the requirements of Part 2.01 of this Section.

C. Place only when underlying geocomposite installation is complete including all Construction Quality Control (CQC) and CQA work.

D. Soil used for the Cover Soil shall be placed in a loose lift that results in a compacted lift thickness of no greater than 12 inches. The maximum permissible pre-compaction soil clod size is 6 inches.

E. During wetting or drying, the material shall be regularly disced or otherwise mixed so that uniform moisture conditions in the appropriate range are obtained.

F. The subgrade to the cover soil layer consists of a geocomposite. Therefore, the Contractor shall avoid tearing, puncturing, folding, or damaging in any way the geocomposite during placement of the cover soil material.

G. Any damage to the geosynthetic liner system which is caused by the Contractor or representatives of the Contractor shall be repaired by the Geosynthetics Installer at the expense of the Contractor.

H. The Contractor shall compact each lift to at least 90 percent of its modified Proctor maximum dry density (ASTM D 1557). The Contractor shall utilize compaction equipment suitable for achieving the soil compaction requirements.

I. The cover soil material shall be placed out in front of the equipment used to place the cover soil such that the minimum thickness requirements are maintained at all times between the geosynthetic materials and the wheels or tracks of the equipment used to place the cover soil material.

J. Care must be exercised by the operators of tracked equipment to avoid sharp pivoting turns that could displace the cover soil material and result in damage to the liner system.

K. The Contractor shall not push cover soil material down the side slope. All soil materials shall be placed from the toe of slope upward.

L. Equipment used in spreading the cover soil material on top of the geosynthetic liner system shall be restricted to the following maximum allowable equipment ground pressures:

MAXIMUM ALLOWABLE EQUIPMENT GROUND PRESSURE (psi)	INITIAL LIFT THICKNESS OF OVERLYING COVER SOIL (ft)
<10	1.0
>20	2.0

3.09 FIELD TESTING

A. The minimum frequency and details of quality control testing are provided below. This testing will be performed by the Engineer. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. Cover Soil material quality control testing:

a. particle-size analyses conducted in accordance with ASTM D 422 at a frequency of one test per 10,000 yd^3;

b. moisture content tests conducted in accordance with ASTM D 2216 at a frequency of one test per 10,000 yd^3;

c. soil classification tests conducted in accordance with ASTM D 2487 at a frequency of one test per 10,000 yd^3; and

d. modified Proctor compaction tests conducted in accordance with ASTM D 1557 at a frequency of one test per 10,000 yd^3.

2. The Engineer will perform conformance tests on placed and compacted cover soil to evaluate compliance with these Specifications. These tests will include in-situ moisture content and dry density. The frequency and procedures for moisture-density testing are given in the CQA Plan. At a minimum, the dry density and moisture content of the soil will be measured in-situ in accordance with ASTM D 2922 and ASTM D 3017, respectively.

3. A special testing frequency will be used by the Engineer when visual observations of construction performance indicate a potential problem. Additional testing will be considered when:

a. the rollers slip during rolling operation;

b. the lift thickness is greater than specified;
c. the fill is at improper and/or variable moisture content;
d. fewer than the specified number of roller passes are made;
e. dirt-clogged rollers are used to compact the material;
f. the rollers do not have optimum ballast; or
g. the degree of compaction is doubtful.

4. During construction, the frequency of testing will be increased by the Engineer in the following situations:

   a. adverse weather conditions;
   b. breakdown of equipment;
   c. at the start and finish of grading;
   d. if the material fails to meet specifications; or
   e. the work area is reduced.

B. Defective Areas:

1. If a defective area is discovered in the Earthwork, the Engineer will evaluate the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the Engineer will determine the extent of the defective area by additional tests, observations, a review of records, or other means that the Engineer deems appropriate. If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the Engineer shall define the limits and nature of the defect.

2. Once the extent and nature of a defect is determined, the Contractor shall correct the deficiency to the satisfaction of the Engineer. The Contractor shall not perform additional work in the area until the Engineer approves the correction of the defect.

3. Additional testing may be performed by the Engineer to verify that the defect has been corrected. This additional testing will be performed before any additional work is allowed in the area of deficiency. The cost of the additional testing shall be borne by the Contractor.

3.10 SURVEY CONTROL

A. The Contractor shall perform all surveys necessary for construction layout and control.

3.11 CONSTRUCTION TOLERANCE

A. The Contractor shall perform the Earthwork construction to within ±0.1 ft on areas with a slope less than 10 percent and ±0.2 ft on areas with a slope greater than 10 percent of the grades indicated on the Drawings.

3.12 PROTECTION OF WORK

A. The Contractor shall use all means necessary to protect completed work of this Section.

B. At the end of each day, the Contractor shall verify that the entire work area is left in a state that promotes drainage of surface water away from the area and from finished work. If threatening weather conditions are forecast, at a minimum, compacted surfaces shall be seal-rolled to protect finished work.

C. In the event of damage to prior work, the Contractor shall make repairs and replacements to the satisfaction of the Engineer.

PART 4 — MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for Prepared Subgrade will be measured as square feet (SF), and payment will be based on the unit price provided on the Bid Schedule.

B. Providing for and complying with the requirements set forth in this Section for Anchor Trench Excavation and Backfill will be measured as linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

C. Providing for and complying with the requirements set forth in this Section for Cover Soil will be measured as in-place cubic yards (CY), and payment will be based on the unit price provided on the Bid Schedule.

D. The following are considered incidental to the Work:

- material samples, sampling, and testing.
- layout survey.
- rejected material removal, re-testing, handling, and repair.
- rejected material.
- mobilization.

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.

B. The work shall include, but not be limited to, delivery, storage, and placement of drainage aggregate (aggregate).

1.02 RELATED SECTIONS

Section 02200 — Earthwork

Section 02771 — Geotextile

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (CQA) Plan

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

ASTM C 33	Standard Specification for Concrete Aggregates
ASTM C 136	Test Method for Sieve Analysis of Fine and Coarse Aggregates
ASTM D 2434	Standard Test Method for Permeability of Granular Soils (Constant Head)

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 and source of aggregate materials.

1.05 QUALITY ASSURANCE

A. The Contractor shall ensure that the materials and methods used for Earthwork meet the requirements of the Drawings and this Section. Any material or method that does not conform to these documents, or to alternatives approved in writing by the Engineer will be rejected and shall be repaired or replaced by the Contractor.

B. 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 Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Contractor will be required to repair the deficiency or replace the deficient materials.

PART 2 — PRODUCTS

2.01 MATERIALS

A. Aggregate shall meet the requirements specified in ASTM C-33 and shall have a maximum particle size of 1-inch. Aggregate shall have a minimum permeability of 1 x 10^{-2} cm/sec when tested in accordance with ASTM D 2434.

2.02 EQUIPMENT

A. The Contractor shall furnish, operate, and maintain hauling, placing, and grading equipment as necessary for aggregate placement.

PART 3 — EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work in this Section, the Contractor shall become thoroughly familiar with the site, the site conditions, and all portions of the work falling within this and other related Sections.

B. Inspection:

- 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.

- 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 continuance 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 only when underlying geosynthetic and CPE pipe installation is complete including all CQC and CQA work.

B. Place to the lines, grades, and dimensions shown on the Drawings.

C. The subgrade to the aggregate consists of a geocomposite 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. Any damage to the geosynthetic liner system which is caused by the Contractor or his representatives shall be repaired by the Geosynthetic Installer.

E. No density or moisture requirements are specified for placement of the aggregate material.

F. Place filter geotextile overlying aggregate as shown on the Drawings and as specified in Section 02771.

3.03 FIELD TESTING

A. The minimum frequency and details of quality control testing are provided below. This testing will be performed by the Engineer. The Contractor shall take this testing frequency into account in planning the construction schedule.

1. Aggregates quality control testing:

   a. particle-size analyses conducted in accordance with ASTM C-136 at a frequency of one test per 5,000 yd^3;

   b. permeability tests conducted in accordance with ASTM D 2434 at a frequency of one test per 10,000 yd^3.

3.04 SURVEY CONTROL

A. The Contractor shall perform all surveys necessary for construction layout and control.

3.05 CONSTRUCTION TOLERANCE

A. The Contractor shall perform the aggregate construction to within +0.1 ft of the thickness indicated on the Drawings.

3.06 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.

PART 4 — MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for Drainage Aggregate around CPE pipe will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

B. Providing for and complying with the requirements set forth in this Section for Drainage Aggregate at geocomposite perimeter termination will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

C. The following are considered incidental to the Work:

- material samples, sampling, and testing.
- layout survey.
- rejected material.
- rejected material removal, re-testing, handling, and repair.
- mobilization.

END OF SECTION

SECTION 02711
CORRUGATED POLYETHYLENE 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 corrugated polyethylene (CPE) pipe and fittings as shown on the Drawings and specified herein.

1.02 RELATED SECTIONS

Section 02200 — Earthwork

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (CQA) Plan

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

ASTM F 405	Standard Specification for Corrugated Polyethylene (CPE) Pipe and Fittings

Latest American Association of State Highway and Transportation Officials (AASHTO) Standards:

AASHTO M252M-96	Corrugated Polyethylene Drainage Pipe

1.04 SUBMITTALS

A. The Contractor shall submit, at least 7 days prior to installation of this material, to the Engineer, certificates of compliance for the pipe materials and fittings to be furnished.

B. The Engineer will supply a surveyor to document the as-built conditions of the piping. The Contractor shall notify and allow the Engineer sufficient time to survey piping prior to backfilling the pipe.

1.05 QUALITY ASSURANCE

A. The Contractor shall ensure that the materials and methods used for corrugated polyethylene pipe 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.

PART 2 — PRODUCTS

2.01 PIPE

A. HDPE pipe sizes shown on the Drawings and specified in this Section reference nominal inside diameter. Pipe material, markings, properties, and size shall be in accordance with AASHTO M252.

B. Pipes shall be corrugated polyethylene pipe with a smooth interior wall, Type SP with Class 2 perforations in accordance with AASHTO M252.

PART 3 — EXECUTION

3.01 GENERAL

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 job site. Handle and store pipe and fittings in accordance with the Manufacturer's recommendation.

3.02 PLACING AND LAYING PIPE

A. Follow the Manufacturer's recommendations when hauling, unloading, and stringing the pipe.

B. Corrugated polyethylene solid and perforated pipe shall be installed as shown on the Drawings.

C. Corrugated polyethylene pipe shall be inspected for cuts, scratches, or other damages prior to installation. Any pipe showing damage, which in the opinion of the Engineer will affect performance of the pipe, must be removed from the site. Replace any material found to be defective.

3.03 CONSTRUCTION TOLERANCE

A. The Contractor shall perform the work to within ±0.1 ft of the grades indicated on the Drawings.

PART 4 — MEASUREMENT AND PAYMENT

4.01 GENERAL

A. Providing for and complying with the requirements set forth in this Section for 6-inch HDPE perforated and solid wall CPE pipe and fittings will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

B. Providing for and complying with the requirements set forth in this Section for 6-inch HDPE perforated and solid wall CPE pipe and fittings will be measured in linear feet (LF), and payment will be based on the unit price provided on the Bid Schedule.

C. The following are considered incidental to the Work:

- shipping, handling and storage.
- layout survey.
- mobilization.
- 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 double-sided textured high-density polyethylene (HDPE) and smooth HDPE geomembrane, as shown on the Drawings. The work shall be carried out as specified herein and in accordance with Drawings.

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

C. Double-sided textured geomembrane shall be used on the side slopes and smooth geomembrane shall be used on the top deck as shown on the Drawings.

1.02 RELATED SECTIONS

Section 02772 — Geosynthetic Clay Liner

Section 02773 — Geocomposite

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (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 1004	Standard Test Method of Initial Tear Resistance of Plastic Film and Sheeting
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 4437	Standard Test Methods for Determining the Integrity of Field Seams Used in Joining Flexible Polymeric Geomembranes
ASTM D 5321	Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method
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 5994	Standard Test Method for Measuring Core Thickness of Textured Geomembranes

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. Prequalified Geomembrane Manufacturers include the following:

GSE Lining Technology, Inc.
19103 Gundle Road
Houston, TX 77073
(800) 435-2008

Serrot International, Inc.
125 Cassia Way
Henderson, NV 89014
(800) 237-1777

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 2,000,000 ft^2 of polyethylene geomembrane on previous projects.

3. The installation crew shall have the following experience.

a. The Superintendent shall have supervised the installation of a minimum of 2,000,000 ft^2 of polyethylene geomembrane on at least five (5) different projects.

b. At least one seamer shall have experience seaming a minimum of 1,000,000 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 Installer shall furnish the Engineer 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 Engineer and Owner.

B. The Geosynthetic Installer shall furnish the Engineer with a 5-year written warranty against defects in workmanship. Warranty conditions concerning limits of liability will be evaluated by, and must be acceptable to, the Engineer and 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 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 Part 2.02 of this Section.

2. The manufacturer warranty specified in Part 1.05.A of this Section.

C. The Geosynthetic Installer shall submit the following information to the Engineer for approval 14 days prior to mobilization.

1. A drawing showing the installation layout identifying geomembrane panel configurations, dimensions, details, locations of seams, as well as any variance

or additional details that deviate from the Drawings. The layout shall be adequate for use as a construction plan and shall include dimensions, details, etc. The layout drawings, as modified and/or approved by the Engineer, shall become part of these 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 installer has installed a minimum of 2,000,000 ft^2 of polyethylene geomembrane, in accordance with Part 1.04 of this Specification. The following information shall be provided for each facility:

   a. the name and purpose of the facility, its location, and dates of installation;

   b. the names of the owner, project manager, 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 Part 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 Part 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 Part 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 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.

1.07 QUALITY ASSURANCE

A. The Geosynthetic Installer shall ensure that the materials and methods used for installation of the geomembrane 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 Geosynthetic Installer.

B. 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 Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer's materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 — PRODUCTS

2.01 GEOMEMBRANE PROPERTIES

A. The Geomembrane Manufacturer shall furnish double-sided and smooth-textured geomembrane having properties that comply with the required property values shown in Table 02770-1.

B. In addition to the property values listed in Table 02770-1, the geomembrane shall:

1. Contain a maximum of 1 percent by weight of additives, fillers, or extenders (not including carbon black).

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

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 5994
Specific Gravity	ASTM D 792 Method A or ASTM D 1505
Tensile Properties	ASTM D 638
Puncture Resistance	ASTM D 4833
Tear Resistance	ASTM D 1004
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.

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. Handling and care of the geomembrane prior to and following installation at the site 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 Engineer.

B. 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.

C. The geomembrane shall be delivered at least 14 days prior to the planned deployment date to allow the Engineer adequate time to perform conformance testing on the geomembrane samples as described in Part 3.05 or this Section. If the Engineer performed a visit to the manufacturing plant and performed the required conformance sampling, geomembrane can be delivered to the site within the 14 days prior to the planned deployment date as long as there is sufficient time for the Engineer to complete the conformance testing and confirm that the rolls shipped to the site are in compliance with this Section.

PART 3 — GEOMEMBRANE INSTALLATION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work described in this Section, the Geosynthetic Installer shall become thoroughly familiar with all portions of the work falling within this Section.

B. Inspection:

1. The Geosynthetic Installer shall carefully inspect the installed work of all other Sections and verify that all work is complete to the point where the work of this Section may properly commence without adverse effect.

2. If the Geosynthetic Installer has any concerns regarding the installed work of other Sections, he shall notify the Engineer in writing prior to the start of the work of this Section. Failure to inform the Engineer in writing or 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 composite liner system.

3.02 GEOMEMBRANE DEPLOYMENT

A. Layout Drawings:

1. The Geosynthetic Installer shall deploy the geomembrane panel in general accordance with the layout drawing specified. The layout drawings 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 an 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 layout drawings.

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 104°F, 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 with adjacent components.

f. Temporary ballast and/or anchors (e.g., sand bags), not likely to damage the geomembrane, are placed on the geomembrane to prevent wind uplift.

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.

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, which 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 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.

B. Personnel:

1. All personnel performing seaming operations shall be qualified as indicated in Part 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 104°F. If the Geosynthetic Installer wishes to use methods that may allow seaming at ambient temperatures below 40°F or above 104°F, he shall use a procedure approved by the Engineer.

2. A meeting will be held with the Geosynthetic Installer and Engineer to establish acceptable installation procedures. In all cases, the geomembrane shall be dry and protected from wind damage.

3. Ambient temperatures shall be measured between 0 to 6 in. above the geomembrane surface.

D. Overlapping:

1. 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. 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. The electric generator shall be properly grounded.

G. Seaming Process:

1. Approved processes for field seaming are extrusion welding and fusion welding. Only equipment identified as part of the approved submittal specified in Part 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 apparatus shall be equipped with gauges giving the temperature in the apparatus.

c. Prior to beginning a seam, the extruder shall be purged until all heat-degraded extrudate has been removed from the barrel.

d. The Geosynthetics Installer shall provide documentation regarding the welding rod to the Engineer and shall certify that the welding rod is compatible with the specifications.

e. 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, beginning of the day and after lunch, for each seaming apparatus used each day. 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 Part 3.03.D of this Section.

2. Four adjoining coupon specimens, each 1-in. wide, shall be cut from the trial seam sample by the installer using a die cutter to ensure precise 1-in. wide coupons. The coupons shall be tested in peel (outside (fusion only) and inside track) and shear using an electronic readout field tensiometer in accordance with ASTM D 4437, at a strain rate of 2 in./min., and they shall not fail in the seam (i.e., Film Tear Bond (FTB), which is failure in the parent material, is required). The required peel and shear seam strength is listed in Table 02770-2. Ideally, samples shall be conditioned at $23 \pm 2^{o}C$ at a relative humidity of $50 \pm 5\%$ for two hours prior to testing. If test conditions vary from these conditions, a 1-in. wide coupon of the parent geomembrane material (no weld) shall be tested in the same manner as the seam specimens to determine the break strength at this condition.

3. If a coupon specimen fails, the entire operation shall be repeated. If the additional coupon specimen fails, 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 Part 3.03.K of this Section. The following procedures shall apply:

    a. Vacuum testing in accordance with ASTM D 5641.

    b. Air pressure testing (for double-track fusion seams only) in accordance with ASTM D 5820 and the following:

        i. Energize the air pump to a pressure between 25 and 30 pounds per square inches, close valve, and sustain the pressure for not less than 5 minutes.

        ii. If loss of pressure exceeds 3 pounds per square inches, or does not stabilize, locate faulty area and repair in accordance with Part 3.03.K of this Section.

        iii. Cut opposite end of air channel from pressure gauge and observe release of pressure to ensure air channel is not blocked.

iv. Remove needle, or other approved pressure feed device, and seal repair in accordance with Part 3.03.K of this Section.

c. Spark testing shall be performed 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 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 Engineer will be responsible for choosing the locations. The Geosynthetic Installer shall not be informed in advance of the locations where the seam samples will be taken. The Engineer reserves the right to increase the sampling frequency.

b. Samples shall be cut by the Geosynthetic Installer at the locations designated by the Engineer as the seaming progresses in order to obtain laboratory test results before the geomembrane is covered by another material. Each sample shall be numbered and the sample number and location identified on the panel layout drawing. All holes in the geomembrane resulting from the destructive seam sampling shall be immediately repaired in accordance with the repair procedures described in Part 3.03.K of this Section. The continuity of the new seams in the repaired areas shall be tested according to Part 3.03.I of this Section.

c. Two strips of dimensions 1-in. wide and 12 in. long with the seam centered parallel to the width shall be taken from either side of the sample location. These samples shall be tested in the field in accordance with Part 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 lengthwise. The sample shall be cut into three parts and distributed as follows:

i. One portion 12-in. long to the Geosynthetic Installer.

ii. One portion 18-in. long to the Geosynthetic CQA Laboratory for testing.

iii. One portion 12-in. long to the Engineer for archival storage.

3. Field Testing:

The two 1-in. wide strips shall be tested in the field tensiometer in the peel mode. The Engineer has the option to request an additional test in the shear mode. If any field test sample fails to meet the requirements in Table 02770-2, then the procedures outlined in Part 3.03.J.5 of this Section shall be followed.

4. Laboratory Testing:

Testing by the Geosynthetics CQA Laboratory will include "Seam Strength" and "Peel Adhesion" (ASTM D 4437) with the 1-in. wide strip 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 4437. The minimum acceptable values to be obtained in these tests are indicated in Table 02770-2. Both 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 passed destructive test locations.

ii. The Geosynthetic Installer can trace the welding path to an intermediate location, a minimum of 10 feet from the location 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 field tests, then full laboratory samples shall be taken. These full laboratory samples shall be tested in accordance with Part 3.03.J.4 of this Section. If these laboratory samples pass the tests, then the seam shall be reconstructed between these locations. If either sample fails, then the process shall be repeated to establish the zone in which the seam should be reconstructed. All acceptable seams must be bounded by two locations from which samples passing laboratory destructive tests have been taken. In cases exceeding 150 ft of reconstructed seam, a sample taken from within the reconstructed zone must pass destructive testing.

b. Whenever a sample fails, the Engineer may require additional tests for seams that were formed by the same seamer and/or seaming apparatus or seamed 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. Each suspected location, both in seam and non-seam areas, shall be nondestructively tested using the methods described Part 3.03.I of this Section, as appropriate. Each location that fails nondestructive testing shall be marked by the Engineer and repaired by the Geosynthetic Installer.

3. When seaming of a geomembrane is completed (or when seaming of a large area of a geomembrane is completed) and prior to placing overlying materials, the Engineer shall identify all excessive geomembrane wrinkles. The Geosynthetic Installer shall cut and reseam all wrinkles so identified. The seams thus produced shall be tested like any 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 exist. 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, used to repair holes larger than 1/16 inch, tears, undispersed raw materials, and contamination by foreign matter;

      ii. abrading and reseaming, used to repair small sections of extruded seams;

      iii. spot seaming, used to repair minor, localized flaws;

      iv. capping, used to repair long lengths of failed seams; and

      v. removing bad seam and replacing with a strip of new material seamed into place (used with long lengths of fusion seams).

   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;

      iv. the repair procedures, materials, and techniques 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 shall be rounded with a radius of at least 3 in.; and

      vi. the geomembrane below large caps shall be appropriately cut to avoid water or gas collection between the two sheets.

5. Repair Verification:

a. Each repair shall be nondestructively tested using the methods described in Part 3.03.I of this Section, as appropriate. Repairs that pass the nondestructive test shall be taken as an indication of an adequate repair. Failed tests will require the repair to be redone and retested until a passing test results. At the discretion of the Engineer, destructive testing may be required on large 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 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. Equipment shall not be driven directly on the geomembrane.

3.05 CONFORMANCE TESTING

A. Samples of the geomembrane will be removed by the Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The Geosynthetic Installer shall assist the Engineer in obtaining conformance samples. The Geosynthetic Installer and Contractor shall account for this testing in the installation schedule. Only material that meets the requirements of Part 2.02 this Section shall be installed.

B. Samples will be selected by the Engineer in accordance with this Section and with the procedures outlined in the CQA Plan.

C. Samples will be taken at a minimum frequency of one sample per 100,000 ft^2.

D. The Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Part 2.02 of this Section.

E. The following tests will be performed by the Engineer:

Test	Test Method
Specific Gravity	ASTM D 792 or D 1505
Thickness	ASTM D 5994
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 Part 1.07.C of this Section, or that conformance testing indicates do not comply with Part 2.02 of this Section, will 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 Engineer.

B. The geomembrane shall be accepted by the Engineer when:

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 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:

- shipping, handling and storage.
- layout survey.
- mobilization.
- rejected material.
- overlaps and seaming.
- rejected material removal, handling, re-testing, and repair.
- temporary anchorage.

TABLE 02770-1
REQUIRED HDPE GEOMEMBRANE PROPERTIES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED VALUES	TEST METHOD
Physical Properties				
Thickness	Average	mils	60	ASTM D 5994 or
	Minimum	mils	54	ASTM D 5199
Specific Gravity	Minimum	N/A	.94	ASTM D 792 Method A or ASTM D 1505
Asperity Height	Minimum	mils	10	GRI GM12
Mechanical Properties				
Tensile Properties (each direction)				
1. Tensile (Break) Strength	Minimum	lb/in	90	ASTM D 638
2. Elongation at Break		%	100	
3. Tensile (Yield) Strength		lb/in	126	
4. Elongation at Yield		%	12	
Puncture	Minimum	lb	90	ASTM D 4833
Tear Resistance	Minimum	lb	42	ASTM D 1004
Interface Shear Strength	-	-	Note 1	ASTM D 5321
Environmental Properties				
Carbon Black Content	Range	%	2-3	ASTM D 1603
Carbon Black Dispersion	N/A	none	Note 2	ASTM D 5596
Environmental Stress Crack	Minimum	hr	400	ASTM D 5397

Notes: (1) Interface shear strength test(s) shall be performed, by the Engineer, on the composite liner system in accordance with Section 02772 — Geosynthetic Clay Liner.

(2) 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	TEST METHOD
Shear Strength[1]				
Fusion	minimum	lb/in	120	ASTM D 4437
Extrusion	minimum	lb/in	120	ASTM D 4437
Peel Adhesion				
FTB[2]				
Fusion	minimum	lb/in	91	ASTM D 4437
Extrusion	minimum	lb/in	78	ASTM D 4437

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.

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.

B. The work shall include, but not be limited to, delivery, storage, placement, and seaming of the various geotextile components of the project.

C. Filter geotextile shall be used overlying the drainage aggregate.

1.02 RELATED SECTIONS

Section 02200 — Earthwork

Section 02225 — Drainage Aggregate

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (CQA) Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

ASTM D 4355	Standard Test Method for Deterioration of Geotextile from Exposure to Ultraviolet Light and Water
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 4595	Standard Test Method for Wide Width Tensile Properties of Geosynthetics
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 Geosynthetic Installer shall submit to the Engineer, at least 7 days prior to geotextile delivery, the following information regarding the proposed geotextile:

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 Geosynthetic Installer shall submit to the Engineer the manufacturing quality control 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 quality control certificates shall include:

1. lot, batch, and/or roll numbers and identification; and

2. results of quality control tests, including a description of the test methods used.

1.05 QUALITY ASSURANCE

A. The Geosynthetic Installer shall ensure that the geotextile and installation methods used 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 Geosynthetic Installer.

B. 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 Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer's materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 — PRODUCTS

2.01 GEOTEXTILE PROPERTIES

A. Geotextile suppliers shall furnish materials in which the "Minimum Average Roll Values", as defined by the Federal Highway Administration (FHWA), meet or exceed the criteria specified in Table 02771-1.

B. The geotextile shall be nonwoven materials, suitable for use in filter/separation applications.

2.02 MANUFACTURING QUALITY CONTROL

A. The geotextile shall be manufactured with quality control 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 Geosynthetic Installer shall replace any rejected rolls.

D. If a geotextile sample fails to meet the quality control requirements of this Section the Geotextile Manufacturer shall 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 bound the failed roll(s).

E. Additional sample testing may be performed, at the Geotextile Manufacturer's discretion and expense, to identify more closely any 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, at a minimum once every 130,000 ft^2, to demonstrate that the geotextile properties conform to the values specified in Table 02771-1. At a minimum, the following manufacturing quality control tests shall be performed on each type of geotextile:

Test	Procedure	Filtration
Grab strength	ASTM D 4632	Yes
Tear strength	ASTM D 4533	Yes
Puncture strength	ASTM D 4833	Yes
Permittivity	ASTM D 4491	Yes
A.O.S.	ASTM D 4751	Yes

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 impermeable 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 Installer shall be liable for any damage to the materials 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 Engineer adequate time to perform conformance testing on the geotextile samples as described in Part 3.06 of this Section.

C. Handling, unloading, storage, and care of the geotextile prior to and following installation at the site, is the responsibility of the Geosynthetic Installer. The Geosynthetic Installer shall be liable for any damage to the materials incurred prior to final acceptance by the Engineer.

D. The Geosynthetic Installer shall be responsible for storage of the geotextile at the site.

E. The geotextile shall be protected from sunlight, excessive heat or cold, 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 Geosynthetic Installer.

PART 3 — EXECUTION

3.01 FAMILIARIZATION

A. Prior to implementing any of the work described in this Section, the Geosynthetic Installer shall become thoroughly familiar with the site, the site conditions, and 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 such work is complete to the point where the installation of this Section may properly commence without adverse effect.

2. If the Geosynthetic Installer 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 installation of the geotextile will be construed as Geosynthetic Installer's acceptance of the related work of all other Sections.

3.02 PLACEMENT

A. Geotextile installation shall not commence until CQA conformance evaluations, by the Engineer, of previous work 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, he does so at his own risk. The Contractor shall account for the CQA conformance evaluations in the construction schedule.

B. The Geosynthetic Installer shall handle all geotextile in such a manner as to ensure they are not damaged in any way.

C. The Geosynthetic Installer 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 filtration and cushion 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 Geosynthetic Installer shall take care not to entrap stones, excessive dust, or moisture in the geotextile during placement.

F. The Geosynthetic Installer shall anchor or weight all geotextile with sandbags, or the equivalent, to prevent wind uplift.

G. The Geosynthetic Installer 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.

B. Filtration 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. Geotextile patches will be sewn into place no closer than 1 inch from any panel edge. Should any tear exceed 50% of the width of the roll, that roll shall be removed and replaced.

B. Where geosynthetic materials underlie the geotextile being placed, care shall be taken to remove any soil or other material that may have penetrated the torn geotextile.

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.

C. Unless otherwise approved in writing by the Engineer, all equipment operating on materials overlying the geotextile shall comply with Section 02200.

3.06 CONFORMANCE TESTING

A. Samples of the geotextile materials will be removed by the Engineer after the material has been received at the site and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. 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 Engineer, at a minimum frequency of one sample per 260,000 ft^2.

C. The Engineer may increase the frequency of sampling in the event that test results do not comply with requirements of Part 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:

Test	Procedure	Filtration
Grab strength	ASTM D 4632	Yes
Puncture strength	ASTM D 4833	Yes
Permittivity	ASTM D 4491	Yes
A.O.S.	ASTM D 4751	Yes

E. Any geotextile that is not certified in accordance with Part 1.04 of this Section, or that conformance testing results do not comply with Part 2.01 of this Section, will be rejected. The Geosynthetic Installer shall replace the rejected material with new material.

3.07 PROTECTION OF WORK

A. The Geosynthetic Installer shall use all means necessary to protect all work of this Section.

B. In the event of damage, the Geosynthetic Installer 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 Filtration Geotextile will be measured as in-place square feet (SF), and payment will be based on the unit price provided on the Bid Schedule.

B. The following are considered incidental to the Work:

- 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	FILTER SPECIFIED VALUES	TEST METHOD
Type			nonwoven	(-)
Mass per unit area	minimum	oz/yd^2	6[(1)]	ASTM D 5261
Filter Requirements				
Apparent opening size (O_{95})	maximum	mm	0.21	ASTM D 4751
Permittivity	minimum	s^{-1}	0.5	ASTM D 4491
Mechanical Requirements				
Grab strength	minimum	lb	130	ASTM D 4632
Tear strength	minimum	lb	40	ASTM D 4533
Puncture strength	minimum	lb	40	ASTM D 4833
Durability				
Ultraviolet Resistance @ 500 hours	minimum	%	70	ASTM D 4355

Notes: (1) For information purposes only, not a required property.

END OF SECTION

SECTION 02772
GEOSYNTHETIC CLAY LINER

PART 1 — GENERAL

1.01 SCOPE

A. The Contractor 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.

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

1.02 RELATED SECTIONS

Section 02200 — Earthworks

Section 02770 — Geomembrane

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (CQA) Plan

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

ASTM D 2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
ASTM D 5321	Determination of the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method
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. The Manufacturer shall be a well-established firm with more than one year of experience in the manufacturing of GCL.

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

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 method, to which are attached GCL samples.

2. projected delivery dates for this project.

3. Manufacturing quality control certificates for each shift's production, signed by responsible parties employed by the Manufacturer (such as the production manager).

4. The quality control certificates shall include:

   a. roll numbers and identification; and

   b. results of quality control tests, including description of test methods used, outlined in Part 2.01 of this Section.

5. The Manufacturer shall certify that the GCL meets all the properties outlined in 2.01 of this Section.

1.06 CONSTRUCTION QUALITY ASSURANCE MONITORING

A. The Geosynthetic Installer shall ensure that the materials and methods used for the GCL 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.

B. 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 Engineer. If nonconformances or other deficiencies are found in the materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 — PRODUCTS

2.01 MATERIAL PROPERTIES

A. The flux 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 liner length shall be long enough to conform with the requirements specified in this Section.

C. The bentonite used to fabricate the GCL shall have at least 90 percent sodium mortmorillonite.

D. The bentonite component of the GCL shall be applied at a minimum concentration of 0.75 pound per square foot, when measured at a water content of less than or equal to 0 percent.

E. The geotextile components of the GCL shall have a minimum combined mass per unit area of 9 oz/yd^2 in accordance with ASTM D 5261.

F. The GCL shall meet the required property values listed in Table 02772-1.

G. 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.

2.02 INTERFACE SHEAR TESTING

A. Interface Shear test(s) shall be performed on the proposed geosynthetic and soil components in accordance with ASTM D 5321. Tests shall be performed as outlined below.

1. Dry GCL interface - the GCL shall be underlain by prepared subgrade compacted to 90% of the maximum dry density (ASTM D 1557) at the optimum moisture content and overlain by a GCL textured 60-mil HDPE geomembrane, geocomposite, and cover soil. The GCL, geomembrane, and geocomposite components of the liner system shall be allowed to “float” (i.e., not fixed) such that the failure surface can occur at the any of these interfaces.

a. Before shearing, the GCL shall be hydrated under a loading of 120 psf (6 Kpa) for 48 hours. The test shall be performed under saturated conditions, at normal stresses of 1, 2, and 4 psi at a shear rate of no more than 0.04 in./min. (1 mm/min.).

b. The results of this test shall have a post-peak apparent friction angle in excess of 18 degrees.

2.03 MANUFACTURING QUALITY CONTROL

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 bound the failed roll(s).

E. Additional sample testing may be performed, at the Manufacturer's discretion and expense, to more closely identify 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 tests shall be performed by the Manufacturer: dry mass per unit area and index flux at frequencies of at least 1 per 50,000 ft^2 and 1 per 200,000 ft^2, respectively.

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

2.04 PACKING AND LABELING

A. GCLs shall be supplied in rolls wrapped in impermeable and opaque protective covers.

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

1. Manufacturer's name;
2. product identification;
3. lot number;
4. roll number; and
5. roll dimensions.

2.05 TRANSPORTATION, HANDLING AND STORAGE

A. Handling, storage, and care of the GCL, prior to and following installation, is the responsibility of the Geosynthetic Installer, until final acceptance by the Engineer.

B. The GCL shall be stored and handled in accordance with ASTM D 5888.

C. The Geosynthetic Installer shall be liable for all damage to the materials incurred prior to and during transportation to the site.

D. 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 Part 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. 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 installation of this Section may properly commence without adverse impact.

2. 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 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 prior to commencement of geomembrane 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 CREST ANCHORAGE SYSTEM

A. The anchor trench shall be excavated, prior to GCL placement, to the lines and grades shown on the Drawings.

B. No loose soil shall be allowed in the anchor trench beneath the GCL.

C. The GCL shall be temporarily anchored in the anchor trench until all geosynthetic layers are installed in the anchor trench as shown on the Drawings.

3.04 HANDLING AND PLACEMENT

A. The Geosynthetic Installer shall handle all GCL in such a manner that they are not damaged in any way and so that they do not become hydrated prior to, or during, installation.

B. In the presence of wind, all GCLs 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 Part 3.07 by the Geosynthetic Installer.

D. If an alternative GCL is used, the vapor barrier portion of the GCL shall be installed against the underlying prepared subgrade.

E. 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.

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

G. All hydrated GCL shall be removed and replaced by the Geosynthetic Installer.

3.05 OVERLAPS

A. On slopes steeper than 10 horizontal to 1 vertical, all GCL shall be continuous down the slope; that is, 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. As a minimum, along the length (i.e., the sides) of the GCL the overlap shall be 6 inches, and along the width (i.e., the ends) the overlap shall be 12 inches.

3.06 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. Material shall not be placed on a GCL that is hydrated.

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

D. Equipment shall not 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.

3.07 REPAIR

A. Any holes or tears in the GCL shall be repaired by placing a GCL patch over the hole. 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 water-based adhesive approved by the Manufacturer.

B. Care shall be taken to remove any soil or other material, which may have penetrated the torn GCL.

C. The patch shall not be nailed or stapled.

3.08 CONFORMANCE TESTING

A. Samples of the GCL will be removed by the Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The Geosynthetic Installer shall assist the Engineer in obtaining conformance samples. The Geosynthetic Installer shall account for this testing in the installation schedule.

B. The Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Part 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.

C. As a minimum, the following conformance tests will be performed: mass per unit area and index flux. All tests shall be carried out at a frequency of one sample per 100,000 ft^2 and 400,000 ft^2, respectively. In addition, the Engineer will perform a minimum of one interface shear strength test in accordance with Part 2.02.

D. Any GCL that is not certified by the Manufacturer in accordance with Part 1.05 of this section or that does not meet the requirements specified in Part 2.01 shall be rejected and replaced by the Geosynthetic Installer.

3.09 PROTECTION OF WORK

A. The Geosynthetic Installer shall use all means necessary to 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.

PART 4 — MEASUREMENT AND PAYMENT

A. Providing for a complying with the requirements set forth in this Section for GCL will be measured as in-place square feet (SF), including GCL 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:

- shipping, handling and storage.
- overlaps and seaming.
- layout survey.
- mobilization.
- rejected material.
- rejected material removal, handling, re-testing, and repair.
- temporary anchorage.

TABLE 02772-1
REQUIRED GCL PROPERTY VALUES

PROPERTIES	QUALIFIERS	UNITS	SPECIFIED[1] VALUES	TEST METHOD
<u>Liner System Properties</u>				
Interface Shear Strength	minimum	degrees	20°	ASTM D 5321[2]
<u>GCL Properties</u>				
Bentonite Content	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
Index Flux	minimum	m^3/m^2-s	1×10^{-8}	ASTM D 5887[3]
Moisture Content (Bentonite)	maximum	percent	25	ASTM D 2216

Notes: (1) All values represent minimum average roll values (i.e., any roll in a lot should meet or exceed the values in this table).
(2) Interface shear strength testing shall be performed, by the Engineer, in accordance with Part 2.02 of this Section.
(3) Hydraulic flux testing shall be performed under an effective confining stress of 5 pounds per square inch.
(4) Measured at a moisture content of 0 percent.

END OF SECTION

SECTION 02773
GEOCOMPOSITE

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 geocomposite. The work shall be carried out as specified herein and in accordance with the Drawings.

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

C. Single-sided geocomposite shall be used overlying the geomembrane and underlying the cover soil on the top deck area of the cover system. Double-sided geocomposite shall be used overlying the geomembrane and underlying the cover soil on the side slopes of the cover system.

1.02 RELATED SECTIONS

Section 02200 — Earthwork

Section 02770 — Geomembrane

1.03 REFERENCES

A. Drawings

B. Site Construction Quality Assurance (CQA) Plan

C. Latest version of American Society for Testing and Materials (ASTM) standards:

1. ASTM D 413. Standard Test Method for Rubber Property-Adhesion to Flexible Substrate.

2. ASTM D 792. Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement.

3. ASTM D 1603. Standard Test Method for Carbon Black in Olefin Plastics.

4. ASTM D 4491. Standard Test Methods for Water Permeability of Geotextiles by Permittivity.

5. ASTM D 4533. Standard Test Method for Trapezoid Tearing Strength of Geotextiles.

6. ASTM D 4632. Standard Test Method for Grab Breaking Load and Elongation of Geotextiles.

7. ASTM D 4716. Standard Test Method for Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products.

8. ASTM D 4751. Standard Test Method for Determining Apparent Opening Size of a Geotextile.

9. ASTM D 4833. Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.

10. ASTM D 5199. Standard Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes.

11. ASTM D 5261. Standard Test Method for Measuring Mass per Unit Area of Geotextiles.

1.04 QUALIFICATIONS

A. The manufacturer shall be a well-established firm with more than one year experience in the manufacturing of geocomposite.

B. The Geosynthetic Installer shall install the geocomposite and shall meet the requirements of Section 02770 and this Section.

1.05 SUBMITTALS

A. The Geosynthetic Installer shall submit to the Engineer, at least 7 days prior to geocomposite delivery, the following information regarding the proposed geocomposite:

1. manufacturer and product name;

2. minimum property values of the proposed geocomposite and the corresponding test procedures;

3. projected geocomposite delivery dates; and

4. list of geocomposite roll numbers for rolls to be delivered to the site.

B. At least 7 days prior to geocomposite placement, the Geosynthetic Installer shall submit to the Engineer the manufacturing quality control certificates for each roll of geocomposite. The certificates shall be signed by responsible parties employed by the geocomposite manufacturer (such as the production manager). The quality control certificates shall include:

1. lot, batch, and/or roll numbers and identification; and

2. results of quality control tests, including a description of the test methods used.

1.06 CONSTRUCTION QUALITY ASSURANCE MONITORING

A. The Geosynthetic Installer shall ensure that the geocomposite and installation methods used 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 Geosynthetic Installer.

B. 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 Engineer. If nonconformances or other deficiencies are found in the Geosynthetic Installer's materials or completed work, the Geosynthetic Installer will be required to repair the deficiency or replace the deficient materials.

PART 2 — PRODUCTS

2.01 GEOCOMPOSITE PROPERTIES

A. The Geocomposite Manufacturer shall furnish geocomposites having properties that comply with the required property values shown in Table 02773-1. The Geocomposite Manufacturer shall provide results of tests performed using the procedures listed in Table 02773-1, as well as certification that the materials meet or exceed the specified values.

B. Geotextiles will be thermally bonded to one and two sides of the geonet component of geocomposite material rather than chemically bonded.

C. Geocomposite suppliers shall furnish materials in which the "Minimum Average Roll Values", as defined by the Federal Highway Administration (FHWA), meet or exceed the criteria specified in Table 02773-1.

D. The geocomposite's geotextile components shall be nonwoven materials, suitable for use in filter/separation and cushion applications.

2.02 MANUFACTURING QUALITY CONTROL

A. The geocomposite shall be manufactured with quality control procedures that meet or exceed generally accepted industry standards.

B. The geocomposite Manufacturer shall sample and test the geocomposite to demonstrate that the material conforms to the requirements of these Specifications.

C. Any geocomposite sample that does not comply with this Section shall result in rejection of the roll from which the sample was obtained. The Geosynthetic Installer shall replace any rejected rolls.

D. If a geocomposite sample fails to meet the quality control requirements of this Section the geocomposite Manufacturer shall 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 bound the failed roll(s).

E. Additional sample testing may be performed, at the geocomposite Manufacturer's discretion and expense, to identify more closely any non-complying rolls and/or to qualify individual rolls.

F. Sampling shall, in general, be performed on sacrificial portions of the geocomposite material such that repair is not required. The Geocomposite Manufacturer shall sample and test the geocomposite, at a minimum once every 100,000 ft^2, to demonstrate that the geocomposite properties conform to the values specified in Table 02773-1. At a minimum, the following manufacturing quality control tests shall be performed on the geotextile component of the geocomposite:

Test	Procedure
Mass per unit area	ASTM D 5261
Grab strength	ASTM D 4632
Tear strength	ASTM D 4533
Puncture strength	ASTM D 4833
Burst Strength	ASTM D 3786
Permittivity	ASTM D 4491
A.O.S.	ASTM D 4751

At a minimum, the following manufacturing quality control tests shall be performed on the geonet component of the geocomposite:

Test	Procedure
Specific gravity	ASTM D 792
Nominal thickness	ASTM D 5199

At a minimum, the following manufacturing quality control tests shall be performed on the geocomposite:

Test	Procedure
Transmissivity	ASTM D 4716
Peel strength	ASTM D 413

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

2.03 PACKING AND LABELING

A. Geocomposite shall be supplied in rolls wrapped in relatively impermeable and opaque protective covers.

B. Geocomposite 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 Installer shall be liable for any damage to the materials incurred prior to and during transportation to the site.

B. The geocomposite shall be delivered to the site at least 14 days prior to the planned deployment date to allow the Engineer adequate time to perform conformance testing on the geocomposite samples as described in Part 3.06 of this Section.

C. Handling, unloading, storage, and care of the geocomposite prior to and following installation at the site, is the responsibility of the Geosynthetic Installer. The Geosynthetic Installer shall be liable for any damage to the materials incurred prior to final acceptance by the Engineer.

D. The Geosynthetic Installer shall be responsible for storage of the geocomposite at the site.

E. The geocomposite shall be protected from sunlight, excessive heat or cold, puncture, or other damaging or deleterious conditions. The geocomposite shall be protected from mud, dirt, and dust. Any additional storage procedures required by the geocomposite Manufacturer shall be the responsibility of the Geosynthetic Installer.

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. 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 installation of this Section may properly commence without adverse impact.

2. 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 installation of the geocomposite 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 geocomposite with the installation of other components of the lining system.

3.02 HANDLING AND PLACEMENT

A. The Geosynthetic Installer shall handle all geocomposite in such a manner that it is not damaged in any way.

B. Install the double-sided geocomposite down the slope not across the slope. Place ends into the anchor trenches in such a manner as to continually keep the geocomposite in tension.

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

D. In the presence of wind, all geocomposites shall be sufficiently weighted with sandbags or the equivalent to prevent movement.

E. The geocomposite shall be positioned by hand after being unrolled to minimize wrinkles.

F. Care shall be taken during placement of geocomposites not to entrap dirt or excessive dust in the geocomposite 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 geocomposite, it should be cleaned prior to placement of the next material on top of it. Care shall be exercised when handling sandbags, to prevent rupture or damage of the sandbags.

G. Geocomposites shall only be cut using a hooked utility blade.

H. After unwrapping the geocomposite from its opaque cover, the geocomposite shall not be left exposed for a period in excess of 15 days.

3.03 OVERLAPS AND SEAMS

A. Geonet Components:

1. The geonet components shall be overlapped a minimum 4 in. along the length. The geonet shall be overlapped by a minimum 1 ft. across the width.

2. Geonet overlaps shall be secured by tying with nylon cable ties. Tying devices shall be white or yellow for easy inspection. Metallic devices shall not be used.

3. Seaming of the geonet shall be performed by wrap-ties at 12-in. centers for end of panels and at 5-ft centers for edge of panel seams.

4. No end-of-panel seams shall be placed on slopes exceeding 10 %.

B. Geotextile Components:

1. The bottom layers of geotextile shall be overlapped, if applicable. The top layers of geotextiles shall be continuously sewn.

2. Polymeric thread, with chemical resistance properties equal to or exceeding those of the geotextile component, shall be used for all sewing.

3.04 PLACEMENT OF OVERLYING MATERIALS

A. All overlying materials shall be placed in such a manner as to ensure that:

1. The geocomposite and underlying materials are not damaged;
2. Minimal slippage occurs between the geocomposite and underlying layers; and
3. Excess tensile stresses are not produced in the geocomposite.

B. Equipment shall not be driven directly on the geocomposite.

C. Unless otherwise approved in writing by the Engineer, all equipment operating on the materials overlying the geotextile shall comply with Section 02200.

3.05 CONFORMANCE TESTING

A. Samples of geocomposite will be removed by the Engineer and sent to a Geosynthetic CQA Laboratory for testing to ensure conformance with the requirements of this Section. The Geosynthetic Installer shall assist the Engineer in obtaining conformance samples. The Geosynthetic Installer shall account for this testing in the installation schedule.

B. Samples shall be taken at a minimum frequency rate of one sample per 200,000 square feet.

C. The Engineer may increase the frequency of sampling in the event that test results do not comply with the requirements of Part 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. As a minimum, transmissivity and peel strength will be performed on each sample.

E. Any geocomposite that is not certified by the Manufacturer in accordance with Part 1.05 of this section or that does not meet the requirements specified in Part 2.01 shall be rejected and replaced by the Geosynthetic Installer.

3.06 PROTECTION OF WORK

A. The Geosynthetic Installer shall use all means necessary to 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.

PART 4 — MEASUREMENT AND PAYMENT

A. Providing for a complying with the requirements set forth in this Section for geocomposite will be measured as in-place square feet (SF), including geocomposite 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:

- shipping, handling, and storage.
- overlaps and seaming.
- layout survey.
- mobilization.
- rejected material.
- rejected material removal, handling, re-testing, and repair.
- temporary anchorage.

TABLE 02773 - 1
GEOCOMPOSITE PROPERTY VALUES – FINAL COVER LINER SYSTEM

PROPERTIES	QUALIFIER	UNITS	SPECIFIED VALUES [1]	TEST METHOD
Geonet Component:				
Specific gravity	Minimum		0.935	ASTM D 792
Carbon black content	Range	%	2 – 3	ASTM D 1603
Nominal thickness	Minimum	mils	200	ASTM D 5199
Geotextile Components:				
Mass per unit area	Minimum	oz/yd^2 (g/m^2)	6 (203)	ASTM D 5291
Filter Requirements				
Apparent opening size	Maximum	mm	$0_{95} \leq$ 0.21 mm	ASTM D 4751
Permittivity	Minimum	1/s	0.5	ASTM D 4491
Mechanical Requirements				
Grab strength	Minimum	lb (N)	130 (578)	ASTM D 4632
Tear strength	Minimum	lb (N)	40 (178)	ASTM D 4533
Puncture strength	Minimum	lb (N)	40 (178)	ASTM D 4833
Geocomposite:				
Transmissivity [2]	Minimum	m^2/s	5×10^{-4}	ASTM D 4716
Peel Strength	Minimum	lb	0.5	ASTM D 413

Notes: (1) All values except transmissivity represent minimum average roll values (i.e., any roll in a lot should meet or exceed the values in this table).

(2) The design transmissivity is the hydraulic transmissivity of the geocomposite measured using water at 68°F ±3°F (20°C ±1.5°C) with a hydraulic gradient of 0.1 under a compressive stress of not less than 2,000 psf (96 kPa). For the test, the geocomposite shall be sandwiched between a layer of cover soil material and a textured 60-mil HDPE geomembrane. The minimum test duration shall be 24 hours and the report for the test results shall include measurements at intervals over the entire test duration.

END OF SECTION

Section 8
Previous Explorations

Converse Consultants
Over 50 Years of Dedication in Geotechnical Engineering and Environmental Sciences

PRELIMINARY GEOTECHNICAL AND GEOLOGIC INVESTIGATION

INDUSTRIAL NON-HAZARDOUS DISPOSAL FACILITY
BASIC MANAGEMENT INCOPORATED
CLARK COUNTY, NEVADA

Prepared for:

Parsons Engineering Science, Inc.
100 West Walnut Street
Pasadena, CA 91124

Converse Project No. 99-33437-01

October 27, 1999

18/69BG

Printed on Recycled Paper

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Converse Consultants
Over 50 Years of Dedication in Geotechnical Engineering and Environmental Sciences

October 27, 1999 99-33437-01

Parsons Engineering Science, Inc.
100 West Walnut Street
Pasadena, CA 91124

Attention: Mr. Jim Goepel

Subject: **Preliminary Geotechnical and Geologic Investigation**
Industrial Non-Hazardous Disposal Facility
Basic Management Incorporated
Clark County, Nevada

Gentlemen:

We are pleased to submit the results of our preliminary geotechnical and geologic investigation conducted for an industrial non-hazardous disposal facility at property owned by Basic Management Incorporated (BMI). The site is located immediately south and west of the operating BMI facility off Lake Mead Drive in Clark County, Nevada and is approximately 20 acres in size. The study was performed in general accordance with our proposal dated August 30, 1999, and your Notice-to-Proceed dated September 1, 1999.

The on-site soils are suitable for use as materials for structural and embankment fills for support of a disposal facility. There do not appear to be adverse geologic or engineering considerations that would severely restrict the development of the proposed facility.

Soils generally consisted of medium dense to very dense granular soils with occasional zones of moderately hard to hard cemented sand and gravel overlying very stiff clay and silts. Groundwater encountered in the borings explored for this project and ranged from 30 to 58 feet be-

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low the ground surface. Cemented soils were encountered at 9 out of 12 boring locations beginning at depths ranging from 7 to 49½ feet below ground surface. Rock excavation techniques may be required for deep cuts.

If you have questions concerning information contained in this report, please contact us at your convenience.

Respectfully submitted,

CONVERSE CONSULTANTS

Al Leskys

Algirdas G. Leskys, P.E.
Principal

AGL:MKK:gm
18/69BG

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Drawing Nos. 1 through 3

Appendix A – Field and Laboratory Investigations

1.0 Introduction

This report presents the results of our preliminary geotechnical and geologic investigation performed for development of an industrial non-hazardous disposal facility at property owned by Basic Management Incorporated (BMI). The site is located immediately south and west of the operating BMI facility off Lake Mead Drive in Clark County, Nevada and is approximately 20 acres in size. We understand that BMI desires to excavate the existing near-surface soils at the site to approximate depths of the proposed disposal facility, which will be constructed some time in the future. Ideally, the excavated soils will be suitable as commercial aggregate materials that may be sold.

A vicinity map showing the location of the project within the Las Vegas Valley area is provided on Drawing No. 1. This scope of work included performing geologic mapping, field exploration, and laboratory testing and engineering analyses to provide preliminary geotechnical design criteria for the site. In conjunction with this scope of work, an environmental evaluation was performed to assess the existing residual chemical concentrations of the soils at the site. Results from this work have been submitted under separate cover.

The purposes of this investigation were to: (1) determine the geologic conditions and presence of any hazards or controlling features at the proposed site; (2) define general subsurface conditions at the sites, and delineate or determine, if possible, the presence of any features that might impact location of the project features; and (3) provide preliminary geotechnical design recommendations.

Designs are in the conceptual stage at this time. The specific type of disposal facility, related structures and other details have not been determined. Future studies will be necessary to address liner designs, slope stability, earthwork recommendations and foundation design.

2.0 Scope of Services

The following tasks were included in our scope of services:

1. A geologic and geotechnical reference search was conducted to confirm the geology and soil conditions in the area.

2. A field reconnaissance of the site was performed by our field geologist to determine the presence of geologic features that could have an impact on the project.

3. A field exploration program was conducted which consisted of drilling, logging, and sampling of twelve (12) exploratory borings to depths ranging from 33 to 60 feet. The approximate location of the borings is shown on Drawing No. 2. The location of the borings was determined in the field by PBS&J surveyors at locations requested by Parsons Engineering Science, Inc. Coordinates and elevations of the individual soil boring locations were not available at the time this report was prepared. Summaries of the subsurface conditions encountered are presented on the boring summary sheets, Drawing Nos. A-1 through A-34 presented in Appendix A. Field drilling and investigation procedures are further described in Appendix A. Samples of the subsurface soils were obtained from the borings and were taken to our laboratory for further evaluation and testing.

4. Laboratory tests were conducted on selected soil samples. Tests included solubility, Atterberg limits, grain size analysis, direct shear, moisture/density relationship, chemical analyses, solubility and permeability. Descriptions and results of the laboratory tests are presented in Appendix A.

5. Results of the field exploration and laboratory testing were evaluated and engineering analyses were performed develop appropriate preliminary recommendations for the design and construction of the proposed project.

6. This geotechnical report was prepared to present the findings, conclusions, and preliminary recommendations. Based on the soil boring logs, geologic cross sections were developed which show interpolated subsurface conditions and are presented on Drawing No. 3

3.0 Existing Site Conditions

3.1 Site Description

The site is located on the northwest portion of the Black Mountain Industrial Complex near Lake Mead Drive and Interstate 515. Two potential areas were investigated at the project site and evaluated for the proposed disposal facility. Six borings were drilled in each area.

Area 1 was located north and west of several evaporation ponds and north of the Pioneer Chlor Alkali Plant. The area was rough-graded and relatively level. A drainage ditch was located adjacent and north of Area 1. Several dirt roads crossed this area. Numerous groundwater wells were observed in this area during our site visit; however, they did not appear to be pumping. Access to this area was either through the BMI plant or by a gate accessed from Warm Springs Road. Soil borings B-1, B-4, B-5, B-8, B-10, and B12 were drilled to investigate this area and cross section A-A' developed from this information.

Area 2 was located north and west of Area 1 and nearly paralleled the west and north fence of the BMI property. Area 2 was less disturbed except near dirt roadways. Undeveloped areas generally were covered with desert vegetation and scattered debris at the time of our field investigation. Soil borings B-101 through B-106 were drilled to investigate this area and cross section B-B' shown on Drawing No. 3 was developed from this information. Only one of the borings (B-102) in Area 2 was located inside the fence of the BMI property.

Based on our investigation, no major subsurface variations were observed at either of the above areas at the site. The soil materials encountered in both areas were found to be of the same type and have the same engineering properties. The following sections present information applicable for both areas.

3.2 Subsurface

Based on results of our subsurface explorations and subsurface explorations performed by others, the subsurface is characterized by alluvial granular soils overlying fine-grained soils, the top of which generally coincides with the groundwater table. The depth of the contact between the granular soil top and the fine-grained soils was encountered from approximately 34 feet to 55 feet below ground surface. The location of this material contact is shown on geologic cross sections A-A' and B-B' on Drawing No. 3.

The granular subsoils generally consisted of medium dense to very dense granular fill and native soils overlying localized zones of moderately hard to hard cemented sand and gravel. The fine-grained soils consisted high plasticity silts and lean clays. Granular fill soils 2 to 14 feet deep were encountered in 10 of the borings. Small boulders and large cobbles were encountered in several of the borings. Cemented soils were not encountered in Boring Nos. 1, 5 and 8 for this investigation. The depths to the cemented soils for the other borings are provided below in Table No. 1.

Table 1 - Depth to and Thickness of Cemented Soils

Boring Location	Depth To Cemented Soils (ft)	Thickness of Cemented Soils (ft)	Cemented Soil Description
B-4	39	1	Hard
B-4	48	6	Moderately Hard to Hard

Boring Location	Depth To Cemented Soils (ft)	Thickness of Cemented Soils (ft)	Cemented Soil Description
B-10	14	16	Moderately Hard
B-12	21	2	Hard
B-101	49.5	0.5	Moderately Hard
B-102	7	10.5	Moderately Hard
B-102	38	2	Moderately Hard
B-103	37	5	Moderately Hard
B-104	25	4	Moderately Hard
B-104	40	3.5	Hard
B-105	17	3	Moderately Hard
B-105	38	>2	Hard
B-106	31	>2	Hard

Field and laboratory test results indicate that the native granular soils at the site have a low compressibility, moderate to high internal angles of friction, low potential for gypsum solubility, a low chemical (salt) heave potential, and contain sulfate salts in concentrations considered harmful to normal strength concrete. The fine-grained soils encountered at depth at the site generally were found to be moderately compressible, have a high expansion potential, and have relatively low permeabilities. Ranges of laboratory test results for the soils are summarized in Table No. 2 and the individual test results and procedures are presented in detail in Appendix A.

Table No. 2 - Summary of Laboratory Test Results

*Properties	Range of Results
Solubility	0.0 to 0.6 percent
Laboratory Max Density (ASTM D1557)	129.7 to 132.1 pcf
Optimum Moisture Content D 1557	7.5 to 8.7 percent
Angle of Internal Friction	26 to 43 degrees
Percent Passing the No. 200 Sieve	8 to 20 percent
Permeability (cm/s)	1.2×10^{-3} to 1.7×10^{-7}
Plasticity Index	Nonplastic to 34

*The laboratory tests are described in detail in Appendix A.

Free groundwater was encountered at depths between 30 and 58 feet below ground surface. Groundwater was measured in the borings immediately following drilling and levels may not have stabilized in the boreholes. A summary of the groundwater elevations encountered in the borings, which extended to groundwater is summarized in the following table:

Table 3 - Depth to Groundwater at Time of Drilling

Boring Location	Depth to Groundwater (ft)
B-1	53
B-4	54.5
B-5	52.5
B-8	58
B-10	46.5
B-12	37.5
B-101	42
B-102	43
B-103	42.5
B-104	43.5
B-105	30
B-106	30

4.0 Site Conditions

4.1 Topography and Vegetation

The native topography of the site is characterized by moderately sloping alluvial fans that lie at the base of the McCullough Range. These piedmont surfaces (areas geologically formed at the base of mountains) slope to the north and northeast, and coalesce in several areas near the proposed disposal facility. The relatively flat surfaces of the alluvial fans in the area are sparsely vegetated with creosote bushes, weeds, and other native plants typical of the Mojave Desert. The eastern two-thirds of Area 1 had been rough graded and was relatively level. Vegetation was very sparse in this area.

Several small north to northeast-trending washes cross the fans and the western portion of the site providing drainage for natural storm-water runoff from the uplands to the south. Most of these washes vary in size from about 10 to 20 feet across. The terrain within the washes consist of gentle to moderate bar-and-swale topography with numerous bar deposits of cobbles and gravel.

4.2 Geologic Soil Units

Our mapping has been based upon review of aerial photographs, published geologic and soils maps, site reconnaissance, and field exploration program. Within the general project area, two different geologic soil units were identified for the purposes of this work. The approximate boundary between the two units is the BMI property fence. Geologic descriptions of the units are based on the *Soil Conservation Service* (SCS) of soil classification and are given below:

Caliza Soil

This soil corresponds to *SCS Soil Map Unit 187* and is described as a very deep, well drained, cobbly fine sandy loam with 2 to 8 percent slopes. This soil is formed in alluvium derived from various types of rock and found on inset fan remnants. The subsurface is predominantly very gravelly coarse sand to a depth of 5 feet or more. Permeability is moderately rapid, available water capacity is low, and runoff is medium. The hazard of water erosion is slight, and the hazard of soil blowing is moderate if the surface is disturbed. Intermittent streams form the drainages in this unit. These drainages are subject to rare or occasional periods of high-velocity flooding.

Urban Land

This unit corresponds to *SCS Soil Map Unit 615* and consists of areas covered by asphalt, concrete, and buildings or other urban structures. The entire BMI complex including the undeveloped areas within the fenced property is mapped in this unit.

5.0 Geologic Conditions

5.1 Geologic Setting

The project area is located in the southeastern portion of the Las Vegas Valley, a structural basin of late Mesozoic and Tertiary block faulting origin. The valley is physiographically characteristic of the Basin and Range province. Valley deposits are Tertiary and Quaternary Age unconsolidated sediments derived from the surrounding mountains; the local sources of deposition are the McCullough Range to the south and the River Mountains to the east. Alluvial deposits consisting of gravel, sand, silt, and clay are overlain by lacustrine deposits of sand, silt, and clay in some portions of the valley. The alluvial and lacustrine sediments can be up to 4,000 feet thick in some parts of the valley. Coarse-grained alluvial fan deposits are located near the base of the mountains and grade into and interfinger with fine-grained sand, silt, and clay deposits in the central portion of the valley.

5.2 Drainage

Primary drainage for the valley is toward the southeast along four major wash systems: Las Vegas Wash, Flamingo Wash, Tropicana Wash, and Duck Creek Wash. These four systems consist of a series of channels of varying lengths and depths that originate along alluvial fans at the base of the Spring Mountains. The channels drain the valley to the southeast where they all coalesce into Las Vegas Wash. Several drainages near the proposed disposal facility divert stormwater runoff from the area to Las Vegas Wash and its tributaries.

5.3 Geologic and Environmental Hazards Review

Existing geologic data, including past Converse project files and references from the U.S. Soil Conservation Service, U.S. Geological Survey, Nevada Bureau of Mines and Geology, aerial photographs, and information through the State Universities were reviewed as part of this investigation. According to our review, two north-trending fault scarps are located approximately ½ mile northwest and southwest of the proposed site. The first is located along the fringe of the McCullough Mountains. The northern terminus of this fault scarp is located approximately 690 feet south of Lake Mead Drive and 1,050 feet west of Interstate 515. The second scarp is located near the vicinity of Gibson Road and American Pacific Drive. No evidence for faulting was observed within the proposed disposal facility site.

The *Subsidence-Related Faults and Fissures of the Las Vegas Valley Map*, published by the *Nevada Bureau of Mines and Geology, Subsidence in Las Vegas Valley 1980-91 Final Project Report*, John W. Bell and Jonathan G. Price, 1991, did not indicate topographic lineations regarded as a subsidence-related (compaction) fault were located near the proposed site. It is generally agreed that subsidence-related faults are not bedrock faults, although their displacement may have been at least partly induced by a seismic event. Others have dated the age of one of these escarpments in the southern part of the valley at about 14,000 to 35,000 years old. Fissures, surface expressions of differential stress resulting from regional and local subsidence due to withdrawal of groundwater, have been localized near subsidence-related faults. The nearest subsidence related faults and fissures are located approximately 2.3 miles northwest of the proposed disposal facility near Whitney Mesa.

The potential for landslides is believed to be low due to the nature and proximity of the topographic highlands to the project area and low precipitation in the region.

5.4 Estimated Ground Accelerations

Las Vegas Valley is located in Seismic Zone 2B as categorized in the Uniform Building Code. Zone 2B represents a low to moderately active seismic area. A regional map published by Algermissen and Perkins (1976) presents the expected peak horizontal ground acceleration for the Las Vegas Valley as approximately 0.1g. This value has a 10 percent chance of being exceeded in a 50-year period. Site specific seismicity model analyses have been performed by Converse on other projects in the Las Vegas Valley. This analysis has typically found the peak horizontal ground acceleration of an event having a 10 percent chance of exceedance during a 100-year design life to range between 0.2g and 0.3g. For an event having a 10 percent chance of exceedance during a 50-year design life, the peak horizontal ground acceleration has ranged between 0.1g and 0.2g. The peak horizontal ground acceleration recommended for the design of the project is 0.15g. Based on our subsurface explorations and well drillers' logs in the area, a soil profile type of S_D per Table 16-J of the 1997 Uniform Building Code should be used for the site.

6.0 Site Evaluation and Preliminary Recommendations

6.1 General

Based on (1) our review of published geologic maps; (2) our geologic site reconnaissance, (3) the results of the preliminary field and laboratory investigations ; and (4) and assuming that the proposed facilities will not be developed to within 8 feet or closer of the fine-grained soils encountered at depth, it is our opinion that from a geotechnical engineering perspective, the site is suitable for support of the proposed disposal facility. We did not identify geologic hazards that would severely restrict development of the proposed disposal facility. Granular native site soils will provide support of the facility with some reworking. Cemented soils were found at shallow to moderate depths in 9 of the 12 borings drilled for this investigation. Based on observed excavation activities performed on other nearby projects, the majority of the soils should be rippable with conventional earthwork equipment, however, deep cuts into fully cemented soil deposits could require rock excavation techniques.

One design consideration will be requirements pertaining to liner permeability as related to the availability of near-surface, low-permeability native soils. As previously mentioned, the subsurface soils are characterized by alluvial granular soils overlying fine-grained soils, the top of which coincides with the groundwater table. The granular soils above the fine-grained soils and groundwater table were tested to have relatively high permeabilities (greater that 1×10^{-5} cm/sec) and are not suitable as potential liner material. Due to regulatory requirements and the subsurface site conditions, development of the site as a disposal facility will probably require placement of either synthetic or non-synthetic (clay) liners. Synthetic liners could include membranes manufactured of high density polyethylene or a similar material. If a clay liner is used, a borrow source containing materials which meet regulatory permeability criteria would need to be identified and investigated. Another possible alternative, which is a combination of the above, is a geosynthetic clay liner which is constructed both of geotextile and/or geomembrane and bentonite. The appropriate regulatory agencies should be contacted during the design process to establish a dialogue as to which liner systems are allowed for the given waste stream and site conditions.

In order to provide geotechnical design recommendations for the proposed project, additional field explorations, laboratory testing, and engineering analyses should be performed. Additional field investigations related to excavatability of the on-site cemented soils may be desirable to perform and could include additional borings at cut areas and seismic refraction tests. After details of the project are finalized and design information is available, specific design recommendations should be developed in the design-level investigation.

The following sections present an engineering evaluation and preliminary considerations for the proposed site and facilities. The preliminary recommendations are for planning and should be confirmed with a final design-level investigation.

6.2 Foundations

Disturbed soils and undocumented fill soils are not considered suitable for the support of structures or retaining walls in the their present condition. It is our opinion that structures may be supported on spread footings founded on a zone of properly placed and compacted structural fill, undisturbed medium dense to very dense granular native soils or on moderately hard to hard cemented soils. Individual footings should not bear on both cemented soils and uncemented soils. Actual bearing materials can be determined after foundation loads and elevations have been determined.

Depending on the type of construction, the proposed structural loads, and the depths of footings, we estimate that the maximum allowable bearing pressures for conventional spread footings will range from 4,000 to 6,000 psf on cemented or non-cemented native granular soils or granular structural fill. Concrete floor slabs may be supported by a 4 to 6 inch layer of processed and compacted granular fill material underlain by structural fill or undisturbed, dense native soils.

6.3 Cut Slopes and Fill Embankments

Cut slopes into medium dense to very dense native granular soils or cemented soils should provide stable slopes on which waste disposal liners may be placed. For any proposed embankments, medium dense to very dense native granular soils or a zone of properly placed and compacted structural fill or cemented soils should provide adequate support for the embankments and disposal facilities after the surface vegetation and organics have been removed and the foundation preparation has been conducted. For embankments, we recommend a homogeneous embankment section consisting of either on-site or import soils. The on-site soils are suitable for use as materials for embankment fill or other structural fills. Fill soils and disturbed native granular soil beneath the embankment sections or any areas which will support liner systems will require scarification, moisture conditioning, and recompaction.

Surfaces of the cut or embankment slopes will need to be adequately protected from erosion due to rainfall and runoff. The stability of the cut or embankment slopes should be analyzed during the final design-level investigation after locations, geometry and heights have been determined. The ranges of cut and embankment slopes given in Table No. 4 may be used for preliminary planning.

Table No. 4 - Slopes for Preliminary Planning

Cut Slope or Embankment Soil Type	Approximate Slope (Horizontal:Vertical)
Sands and Gravels	1½:1 to 2:1

It should be noted that typical slopes on which liners will be constructed are usually not recommended to be steeper than 3:1 (horizontal to vertical). Slopes would need to analyzed on an individual basis with proposed liner types and expected loads for the appropriate maximum steepness in the design-level investigation.

6.4 Retaining Walls

We anticipate that conventional concrete retaining walls may be used for the project. The appropriateness of concrete walls will depend on the height, length, and configuration of the walls. After wall details have been determined, information on different options and approximate costs can be provided in the design-level investigation. Ranges of lateral earth pressures for restrained and unrestrained walls are given in Table 5 below:

Table No. 5 - Lateral Earth Pressures

Wall Type	Lateral Earth Pressures psf/ft
Restrained	38-45
Unrestrained	30-40

The site soils generally contain too high a percentage of fines to be free draining. On-site material may be processed to reduce the silt and clay content and to remove oversize material, to obtain a free draining backfill material. If free draining material is not used, a drainage system will need to be provided or the wall designed to resist hydrostatic forces.

7.0 Construction Considerations

7.1 Site Grading

It is anticipated that site grading will be a large component of development of the site. Site grading should consist of: (1) the removal of existing vegetation, pavements, debris, surface trash and possibly some underground utilities that may be relocated from the site; (2) the undocumented fill, loose or disturbed native soils should be processed and stockpiled for later use as engineered fill; (3) excavating down to medium dense to very dense granular soils or down to expose the underlying cemented soils for support of any foundations; (4) overexcavation and recompaction of the natural soils will be required for the support of any structures; and (5) the exposed native soils will require scarification, moisture conditioning and recompaction prior to placing structural fill in fill areas. Scarification and recompaction can be terminated where cemented soils are exposed. The existing on-site soils should be suitable for use as compacted structural fill. All fill at the site should be considered undocumented unless records of proper placement were prepared and are obtained. All undocumented fill at the site will be unsuitable for support of structures and settlement-sensitive facilities at the site will need to be reworked as structural fill. Rubble and debris resulting from excavating cemented soil deposits should be considered undocumented fill. Contingency plans should be considered for removing small boulders, cobbles, and broken cemented material resulting from excavations of cemented soil from the project site. Boulders, cobbles, and cemented material between 4 and 24 inches in diameter may possibly be used in deep fill where potential settlement may be tolerated areas if special compaction procedures are used and full-time observation during placement is provided.

There will be shrinkage when excavating and compacting or scarifying and recompacting the non-cemented on-site soils, and swell when excavating and compacting cemented soils. The shrinkage and swell factors provided in Table No. 6 may be used for preliminary planning.

Table No. 6 - Shrinkage and Swell Factors for Preliminary Planning

Soil Type	Estimated Shrinkage (-) or Swell (+) Factor (percent)
Sands and Gravels	5 to 15 (-)
Cemented Soils	0 to 10 (+)

A shrinkage factor of 5 to 15 percent may be used for preliminary planning in areas where the exposed native soils will be compacted to a depth of 6 inches. For final design, the anticipated shrinkage and swell factors for the on-site soils should be determined in the design-level investigation.

7.2 Excavations

Based on observations made during our field explorations, the majority of non-cemented soils should be readily excavatable with conventional earthwork equipment. Partially to fully cemented (moderately hard to hard) soils were encountered during this investigation at depths of 7 to 49½ feet below the existing ground surface. Heavy-duty ripping, heavy-duty backhoe, headache ball, rocksaw, blasting, or Ho-ram should be anticipated for any deep excavations. The Contractor should be aware of the potential for vibrational damage to adjacent or nearby structures when using blasting or heavy impact equipment during removal of the hard cemented materials.

7.3 Soil Corrosivity

Laboratory test results indicate that according to Clark County Building Department standards, soils should be considered moderately to severely corrosive to buried metal and have sulfate levels above that considered harmful to normal strength concrete and soil cement. Consideration should be given to corrosion protection systems for buried metal. The concrete mixture recommendations to accommodate severely corrosive soils should be based on the design-level investigation.

7.4 Final Design Recommendations

Additional design-level investigations will be required, when final facility and structure layouts and dimensions have been determined. Borings should be located within the foot-print of the facilities or structures and laboratory testing should be performed to evaluate the nature and engineering properties of the native subsoils at those locations and, if possible, any potential import materials. It may also be desirable to evaluate the extent and rippability of cemented soils.

8.0 Closure

Our assumptions, conclusions, recommendations, and opinions presented herein are: (1) based upon the geologic site reconnaissance, (2) based upon our evaluation and interpretation of the findings of the preliminary field exploration and laboratory programs, (3) based upon an interpolation of soil conditions between and extrapolation beyond the boring locations, (4) based on our geotechnical experience in the locale, (5) not based on environmental regulatory requirements for disposal facility design and construction and are only based on standard geotechnical engineering considerations (6) subject to confirmation of the conditions encountered during a design-level investigation, and (7) prepared in accordance with generally accepted professional geotechnical engineering principles and practice. We make no other warranty, either express or implied.

It has been our pleasure to serve you on this project. If you have any questions, please contact this office.

Respectfully submitted,

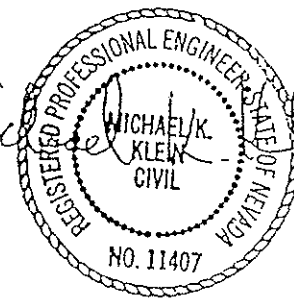
CONVERSE CONSULTANTS

Lorraine Linnert Dunford
Project Geologist

Mike Klein, P.E.
Senior Engineer

Reviewed by:

James L. Werle, P.G.
Principal Geologist



10-27-99

MKK:CMK:gm
18/69BG

Encl: Drawing Nos. 1 through 3
Appendix A

Dist: 3/Addressee

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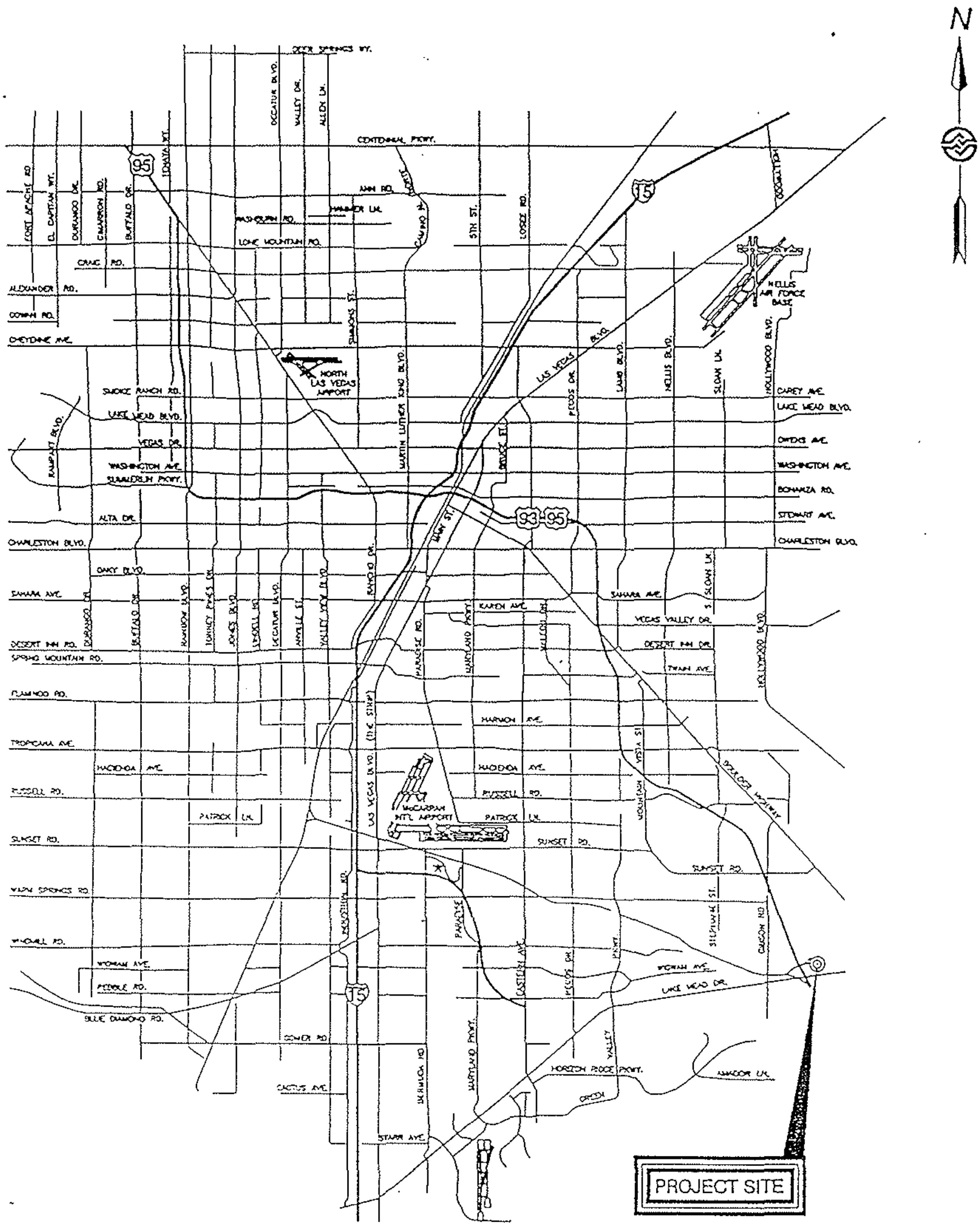
Bell, J.W., 1978, *Geologic Constraints Map, Las Vegas SE Folio*, Nevada Bureau of Mines and Geology, Environmental Series, Las Vegas Area.

Bell, J.W. and Price, J.G., 1991, *Subsidence-Related Faults and Fissures of the Las Vegas Valley Map*, Nevada Bureau of Mines and Geology, Subsidence in the Las Vegas Valley 1980-91 Final Project Report.

Soil Conservation Service, 1985, *Soil Survey of Las Vegas Valley Area, Nevada, Part of Clark County*: U.S. Department of Agriculture.



LAS VEGAS VICINITY

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Scale N.T.S.	File No.	
Date 10/21/99	Project No. 99-33437-01	
Drafted By CLE	Drawing No.	
Checked By MKK	1	
Approved By		

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REF: AutoCAD drawing file "site.dwg", supplied by client

GRAPHIC SCALE

100 0 200 400

(In Feet)

EXPLANATION

APPROXIMATE SOIL BORING LOCATION

APPROXIMATE LOCATION OF GEOLOGIC SECTION (See Drawing No. 3)

LOCATION OF SUBSURFACE EXPLORATIONS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Scale	1" = 200'	File No.	43701002
Date	10/21/99	Project No.	99-33437-01
Drafted By	GLE	Drawing No.	2
Checked By	MKK		
Approved By			

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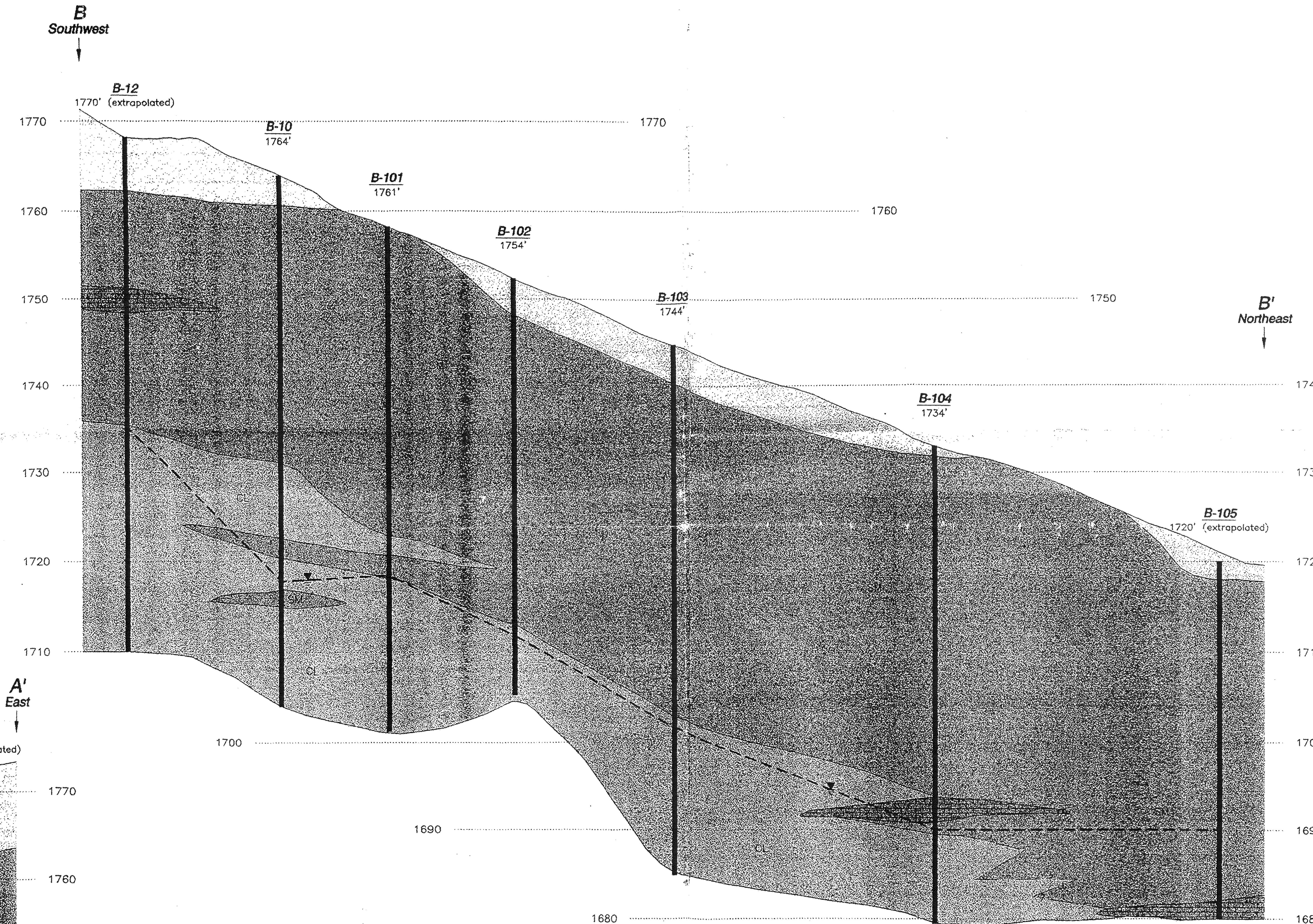
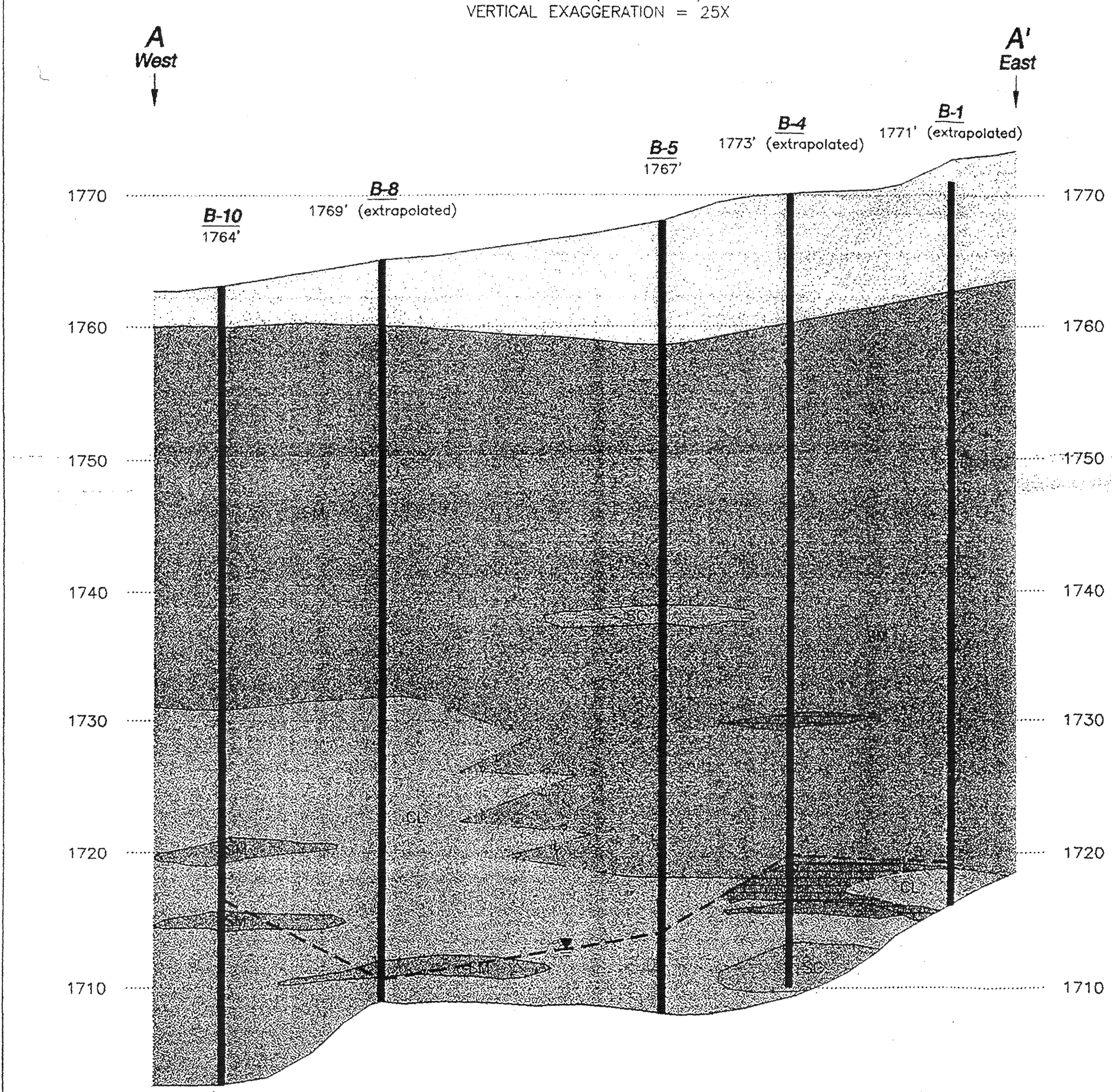


SECTION B-B'

SCALE: 1" = 10' (VERTICAL)
1" = 250' (HORIZONTAL)
VERTICAL EXAGGERATION = 25X

SECTION A-A'

SCALE: 1" = 10' (VERTICAL)
1" = 250' (HORIZONTAL)
VERTICAL EXAGGERATION = 25X



LEGEND

- FILL
- CLAY
- CLAYEY SAND
- SILTY SAND
- SAND
- CEMENTED SAND AND GRAVEL
- APPROXIMATE GROUNDWATER LEVEL MEASURED IMMEDIATELY AFTER DRILLING

NOTE: Ground surface elevations were extrapolated from a topographic map provided by the client. Locations and elevations of the soil borings are approximate. Data concerning subsurface conditions has been obtained at exploration locations only. Actual conditions between explorations may differ from the generalized profiles shown here.

GEOLOGIC SECTIONS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

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Scale	1" = 300'
Date	10/28/99
Drafted By	GLE
Checked By	LLD
Approved By	[signature]
File No.	43701003
Project No.	99-33437-01
Drawing No.	3

Field and Laboratory Investigations

Field Investigation

The subsurface soil conditions were explored by drilling 6 borings in each of the two areas (total of 12 for the site) to depths ranging from 33 to 60 feet below ground surface. The approximate locations of the explorations are shown on Drawing No. 2, Locations of Subsurface Explorations. Boring locations were located, surveyed, and staked by PBS&J surveyors, however, at the time this report was prepared this information was not available. Continuous logs of the subsurface conditions as encountered in the explorations were recorded at the time of drilling by a field geologist. The subsurface conditions encountered were visually classified in accordance with the Unified Soil Classification System. Summaries of the subsurface conditions encountered are presented on the boring summary sheets, Drawing Nos. A-1 through A-34. A key to soil symbols and terms is found on Drawing No. A-35. The soil classification system for engineering purposes is further explained on Drawing No. A-36.

Drilling was accomplished with a BK-81 hollow-stem auger drill rig equipped for soil sampling. Relatively undisturbed soil samples were obtained using a 2.42-inch inside diameter Converse sampler driven with a 140-pound hammer free-falling through a distance of 30 inches. Sampler driving resistance, expressed as blows per 12 inches of penetration, is presented on the boring logs at the respective sampling depths. The sampled soil is retained in brass rings 1-inch in height which line the sampler. A representative portion of each sample was retained and carefully sealed in waterproof plastic containers for transport to the geotechnical laboratory. Additional samples were collected in 6-inch high stainless steel sleeves for environmental laboratory analysis. Soils with stains and orders due to potential chemical contamination are noted on the soil boring logs. It should be noted that given the coarse grained materials encountered at the site, undisturbed sample recovery was low. Also, the encountered materials in the borings classified as boulders could have also been cobble size as well as boulder size particles.

Pocket Penetrometer

At the time of drilling, pocket penetrometer tests (pp) were conducted in the ends of selected brass ring samples of fine-grained soils as they were received from the borings. The purpose of the tests was to give an indication of the unconfined compressive strength in tons per square foot (tsf) or unconfined shear strength in kips per square foot (ksf) of the soil. A Brainard-Kilman S-170 pocket penetrometer was used. The results of the tests are presented in the Field or Laboratory Tests column of the boring logs, Drawing Nos. A-1 through A-34.

Laboratory Investigation

Laboratory tests were conducted on representative soil samples for the purpose of classification, and determination of their physical properties and engineering characteristics. The amount and selection of the types of testing for a given study are based on the geotechnical conditions of the project. Test results are presented in the summary boring logs and in this appendix. A summary of the various laboratory tests conducted by our office for engineering purposes is presented as follows.

The soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.

Moisture Content and Dry Density

Data obtained from these tests, performed on relatively undisturbed samples obtained from the field and in accordance with ASTM D2435, were used in the classification and correlation of the soils and to provide qualitative information regarding soils strength and compressibility. Test results are presented on the boring logs on Drawing Nos. A-1 through A-34.

Grain Size Distribution

Grain size distribution for soil samples were determined by sieve analysis in accordance with ASTM C136. A sieve analysis is conducted by passing the soil through a number of different sized sieves and measuring the amount of soils retained on each sieve. The test results and grain size distribution curves are presented on Drawing Nos. A-37 through A-48.

Atterberg Limits

The liquid limit, plastic limit and plasticity index of a representative sample of the fine-grained soils were determined to aid in the classification of the soils and in the evaluation of other engineering parameters. The test was performed in general accordance with ASTM test method D4318. The results of the tests are tabulated in the following table:

Exploration Location	Sample Depth, ft.	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Unified Soils Classification
B-1	30-35	NP	NP	NP	SM
B-5	20-25	NP	NP	NP	SM
B-10	30-35	NP	NP	NP	SM
B-12	10-15	NP	NP	NP	SM
B-101	39-40	105	71	34	MH
B-101	54-55	54	44	10	ML
B-102	20-25	NP	NP	NP	SM
B-102	49-50	88	58	30	MH
B-103	30-35	NP	NP	NP	SM
B-104	10-15	NP	NP	NP	SM
B-105	20-25	NP	NP	NP	SW-SM
B-106	0-5	NP	NP	NP	SM

NP = Nonplastic

Solubility

Solubility tests were performed to determine the amount of water-soluble materials (principally gypsum) present in the soil. After drying a soil specimen weighing approximately 150 grams in a 60-degree centigrade oven, about 2 liters of tap water are passed through the specimen. The soil is then oven-dried and the amount of soluble materials lost is calculated based on the original dry weight of the soil. The results of the solubility test are presented in the following table:

Exploration Location	Sample Depth (Feet)	Soil Description	Solubility (% by Unit Weight)
B-4	2-2.5	Fill – Poorly graded sand	0.2
B-5	10-15	Silty sand with gravel	0.6
B-8	19-20	Silty sand with gravel	0.2
B-101	5-10	Silty sand with gravel	0.0
B-102	0-5	Fill - Silty sand with gravel	0.6
B-104	0-5	Silty sand with gravel and cobbles	0.4
B-106	0-5	Silty sand with gravel	0.0

Consolidation

The apparatus used for the consolidation tests is designed to receive a one-inch high brass ring containing an undisturbed soil sample as it comes from the field. Tests were performed in general accordance with ASTM D2435 test method. Loads are applied to the test specimen in several increments, while resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with both ends of the specimen to permit the ready addition or release of water. Samples are initially tested at their field moisture content. After consolidating at the field moisture content with a 2 ksf surcharge load, the specimens are inundated with water. Additional consolidation that occurs with a 2 ksf load after the specimens are inundated with water (hydrocollapse) is measured. Subsequent consolidation with additional loads is measured at the increased moisture content to determine soil behavior under saturated conditions. Results of the tests are shown on Drawing Nos. A-49 through A-56, entitled *Consolidation Test* and are summarized on the following table:

Exploration Location	Depth (feet)	Soil Description	Dry Unit Weight, pcf	Moisture Content, %	Hydrocollapse (percent)*
B-1	29-30	Silty sand with gravel	105	6	3.2
B-8	39-40	Sandy lean clay	57.4	64	0.4
B-8	49-50	Sandy lean clay	69.5	51.1	-0.6
B-10	54-54.5	Sandy lean clay	60.7	67.7	-0.6
B-101	39-40	Sandy lean clay	65.8	45	-0.2
B-101	59-60	Sandy lean clay	73.2	38.3	-0.6
B-102	49-50	Sandy lean clay	67.3	48.7	-0.5
B-105	34-35	Well graded sand with silt and gravel	101	5	0.1

NA: Not available

* A negative sign indicates swell occurred upon inundation with water instead of collapse.

Laboratory Maximum Density

Laboratory maximum density tests were performed on selected samples of the granular soils. The purpose of the test was to define the compaction characteristics of these soils, and to aid in estimating soil shrinkage. The laboratory maximum density test was performed in general accordance with the ASTM D1557 test method. This test procedure uses 25 blow of a 10-pound hammer falling a height of 18 inches on each of five layers of soil in a 1/30 or 1/13 cubic foot cylinder. The test results are presented on Drawing Nos. A-57 through A-61 and in the following table:

Exploration Location	Depth (Feet)	Soil Description	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (percent) - of dry weight)
B-1	20-25	Silty sand with gravel	129.4	8.2
B-5	20-25	Silty sand with gravel	132.1	8.2
B-12	10-15	Silty sand with gravel	129.7	7.9
B-101	5-10	Silty sand with gravel	130.6	8.7
B-105	20-25	Well graded sand with silt and gravel	131.8	7.5

Direct Shear Strength

A progressive direct shear test was performed on selected undisturbed samples using a constant strain rate direct shear machine in general accordance with ASTM D3080. The test specimen was trimmed and placed in the shear machine, a specified normal load was applied, and the specimen was sheared until maximum shear strength was developed. After the soil specimen had developed maximum shear resistance under the first normal load, the normal load was removed and the specimen was pushed back to its original undeformed configuration. Another normal load was then applied, and the specimen was sheared a second time. This process was repeated for three different normal loads. Results of the direct shear test are presented on Figures A-62 through A-69 and in the following table:

Exploration Location	Depth (feet)	Soil Description	Angle of Internal Friction (deg)	Coulomb Cohesion (ksf)
B-4	14-14.5	Silty sand with gravel	31	0.7
B-5	14-15	Silty sand with gravel	43	0.3
B-10	54-54.5	Sandy lean clay	26	0.85
B-12	14-15	Silty sand with gravel	40	0.3
B-101	39-40	Sandy lean clay	26	0.9
B-102	20-25	Silty sand with gravel	37	0.2
B-103	49-50	Sandy lean clay	37	1.0
B-104	10-15	Silty sand with gravel	43	0.1

Chemical Analysis

Chemical tests were performed on a representative soil samples to investigate the potential for soil corrosivity and chemical heave. Atlas Chemical Testing Laboratories, Inc. in Las Vegas performed the chemical analysis for water-soluble sulfates and sodium in general accordance with ASTM D516. The results of the chemical tests are presented on Drawing No. A-70.

Exploration Location	Depth (feet)	Soil Description	Percent Sodium	Percent Sulfate	Total Available Water Soluble sodium Sulfate (%)
B-5	10-15	Silty sand with gravel	0.07	0.13	0.20
B-8	19-20	Silty sand with gravel	0.07	0.06	0.08
B-101	5-10	Silty sand with gravel	0.17	0.06	0.08
B-102	0-5	Fill – Silty sand with gravel	0.17	0.03	0.05
B-106	0-5	Silty sand with gravel	0.15	0.08	0.12
B-106	29-30	Silty sand with gravel	0.15	0.06	0.08

Permeability

Falling head permeability tests were conducted on remolded samples in general accordance with modified ASTM procedure D2434. The soil was compacted in a mold 4.6 inches long and 4.0 inches in diameter to 85 or 90 percent of maximum dry density and at optimum moisture content. A falling head was applied to the sample and the flow of water through the sample was monitored. The permeability was calculated after the flow rate had stabilized. The result of the falling head permeability test is presented in the following table:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-5	20-25	Silty sand with gravel	5.3×10^{-4}
B-12	10-15	Silty sand with gravel	4.0×10^{-4}
B-102	20-25	Silty sand with gravel	1.0×10^{-4}
B-105	20-25	Well graded sand with silt and gravel	1.2×10^{-3}

Flexible wall permeameter tests were performed on selected samples by AP Engineering and Testing, Inc according to ASTM D5084. With the exception of one sample (B-105), all tested samples were undisturbed ring samples. The samples were placed in a triaxial machine with a constant confining pressure at the approximate in-place effective stress pressures. Results were generally consistent with the falling head permeability test results for the granular materials. Laboratory results are presented on Drawing Nos. A-71 through A-76 and summarized below:

Exploration Location	Sample Depth (Feet)	Soil Description	k (cm/s)
B-1	14-15	Silty sand with gravel	1.57×10^{-4}
B-4	24-25	Silty sand with gravel	1.47×10^{-4}
B-8	44-45	Sandy silt	2.90×10^{-5}
B-12	39-39.5	Silty clay	1.76×10^{-7}
B-103	44-45	Silty clay	3.83×10^{-7}
B-105*	30-35	Silty sand	3.05×10^{-4}

* Sample remolded to 85% relative compaction at optimum moisture.

Log No. B- 1

Date of Drilling: 9/23/99 | Location: See Drawing No. 2 | Ground Surface Elevation (ft): Not Available
Driller: T. High | Borehole Diameter: 8" | Equipment: BK-81 Hollow Stem Auger
Logged By: M. Stacy | Groundwater Depth (ft): 53.0 | Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Description	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		FILL: SILTY SAND With Gravel; light brown, dry			21				
4		---moist			26				
5					42				
9					26		4		
10		SILTY SAND With Gravel (SM); dense, light brown, slightly moist			33				
14					44				K
19		---very dense			35/5"				

End of Exploration at 55.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

Converse Consultants

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Drawing No. A-1

LV-B1 93343701 GPJ

Log No. B- 1

Date of Drilling: 9/23/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 53.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Graphic Log	Description	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
20		SILTY SAND With Gravel (SM); dense, light brown, slightly moist			35				
22									G;P
24					35		7	108	
26									
28		---grayish brown							
29.5					35/6"				
30		---very dense			53				A
31									G
32									
34					35/3"				
36									
38									
39.5		---dense			35/6"				

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End of Exploration at 53.0'

Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

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Drawing No. A-2

Log No. B- 1

Date of Drilling: 9/23/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 53.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); dense, grayish brown, slightly moist	X		42				
42									
44		—very dense	X		58/7"				
46									
48									
			X		35/4"				
50									
52									
		---wet							
54		SANDY LEAN CLAY (CL); very stiff, dark brown, moist	X		35/6"				
56									
58									

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End of Exploration at 55.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Drawing No. A-3

Log No. B- 4

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 54.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		FILL: SILTY SAND With Gravel; brown, dry	X		17				
2		---POORLY GRADED GRAVELLY SAND	X		45				Sol
4			X		66/7"		3		
5			X		35/4"				
9		---SILTY SAND With Gravel	X		35/5"				
10			X		36				
14		SILTY SAND With Gravel (SM); very dense, brown, dry	X		35/8"		5		DS
18		---dense							
19			X		43				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Project No. 99-33437-01

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Drawing No. A-4

Log No. B- 4

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 54.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); very dense, brown, dry			35/2"				
22									
24		---dense			49				K
26									
28									
		---few gravel			35		7	107	
30					42				
32									
34		---with gravel, very dense			35/8"		4		
36									
		---with boulders							
38									
		CEMENTED SAND AND GRAVEL; hard, brown, dry			20/0"				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Project No. 99-33437-01

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Drawing No.

Log No. B- 4

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 54.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
	SILTY SAND With Gravel (SM); very dense, brown, dry			35/1"				
42								
44	---with boulders			35/0"				
46								
48	---partially cemented, moderately hard							
				35/0"				
50								
	CEMENTED SAND AND GRAVEL; hard, gray, dry							
52								
54	POORLY GRADED SAND (SP); dense, black, moist, very strong odor, heavy staining			44				
	LEAN CLAY (CL); stiff, grayish brown, dry							pp=2.0
56								
	CLAYEY SAND (SC); medium dense, brown, wet							
58								
	---strong odor			23				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Drawing No. A-6

99 33437-01 GPJ

Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0	FILL: SILTY SAND With Gravel; tan, dry	X	X	9				
4	---slightly moist	X	X	35/5"				
5		X		58				
8	---with cobbles							
9	SILTY SAND With Gravel (SM): dense, tan, slightly moist	X		31		5	115	
10		X	X	29				
12			X					K;Sol; Ch
14		X	X	43		4	111	DS
19	---very dense	X		57/8"				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Drawing No.

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Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
20	SILTY SAND With Gravel (SM); dense, tan, slightly moist	32				A
22						G;P;C
24		58/8"				
25	---very dense					
28	---brown					
29	CLAYEY SAND With Gravel (SC); dense, dark brown, slightly moist	35/5"		6	112	
30		35/6"				
31	SILTY SAND With Gravel (SM); dense, black, slightly moist					
32	---dark brown					
34	---very dense	50/3"				
39	---with cobbles	50/6"				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Drawing No.

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Log No. B- 5

Date of Drilling: 9/15/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 52.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.							
40		SILTY SAND With Gravel (SM); dense, black, slightly moist	X		35/6"				
42									
44					35/0"				
46									
48									
49.5			X		36				
50		SANDY LEAN CLAY With Gravel (CL); very stiff, dark brown, slightly moist							
52									
54		---moist	X		32				pp=3.0
56									
58									
59.5			X		18				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Project No. 99-33437-01

Drawing No.

Log No. B- 8

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 58.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Graphic Log	Description	Drive	Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		<u>FILL:</u> **SILTY SAND With Gravel**; tan, dry			8				
4					28				
5					16				
9		SILTY SAND With Gravel (SM); very dense, tan, dry			35/8"		2		
10					32				
14.5					33/6"				
19		---dense			38		4		Ch Sol

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

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Drawing No.
A-10

B1 9334370 1 GPJ

Log No. B- 8

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 58.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.							
		SILTY SAND With Gravel (SM); very dense, tan, dry			60/9"				
22									
24					54/9"		4		
26									
28									
					35/8"				
30							4		
					45				
32									
34					55				
									G
36									
38		SANDY LEAN CLAY (CL); very stiff, dark brown, slightly moist							
					30		46	69	
		---with gravel							

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End of Exploration at 60.0'
Converse Sampler (white symbol=no recovery)
SPT Sampler (white symbol=no recovery)

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Project No.
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Drawing No.

1 93343701 GPJ

Log No. B- 8

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 58.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SANDY LEAN CLAY With Gravel (CL); very stiff, dark brown, slightly moist			49				
42									
44		—dry			35/8"				K
46									
48		---partially cemented, moderately hard, white, dry							
		SANDY LEAN CLAY (CL); very stiff, dark brown, moist			35/10"				pp=3.
50									
52									
54					16		52	71	pp=2.(
56									
58		SILTY SAND (SM); dense, brown, wet							
		SANDY LEAN CLAY (CL); stiff, dark brown, wet			13		51	70	pp=3.(

End of Exploration at 60.0' Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery

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Drawing No.
A-12

B1 9334370I GPJ /99

Log No. B-10

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 46.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		FILL: SILTY SAND With Gravel; tan, dry			26				
4		SILTY SAND With Gravel (SM); medium dense, tan, slightly moist			25		5		
					20				
9		---very dense			39				
10					60/8"				
11		---cobbles							
14		---partially cemented, moderately hard			35/1"				
19					35/1"				

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End of Exploration at 60.0'   Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

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Drawing No. A-13

-B1 93343701 GPJ

Log No. B-10

Date of Drilling: 9/13/99	Location: See Drawing No. 2	Ground Surface Elevation (ft): Not Available
Driller: T. High	Borehole Diameter: 8"	Equipment: BK-81 Hollow Stem Auger
Logged By: M. Stacy	Groundwater Depth (ft): 46.5	Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
20	SILTY SAND With Gravel (SM); partially cemented, moderately hard, tan, slightly moist			35/0"				
24	—with cobbles			35/1"				
29				35/0"				
30	SILTY SAND With Gravel and Cobbles (SM); very dense, brown, slightly moist			35/3"				
32								A.G
34	---cobbles and boulders			35/3"				
34.5	LEAN CLAY With Gravel (CL); very stiff, light brown, dry							
39				35/6"				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Drawing No. A-14

B1 99343701.GPJ

Log No. B-10

Date of Drilling: 9/13/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 46.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		LEAN CLAY (CL); very stiff, light brown, dry			35				
42		SILTY SAND With Boulders (SM); very dense, brown, dry							
44		LEAN CLAY With Gravel (CL); stiff, dark brown, moist			13				pp=1.2
46									
48		SILTY SAND With Gravel and Caliche Nodules (SM); medium dense, dark brown, moist							
		SANDY LEAN CLAY (CL); very stiff, dark grayish brown, wet			30		17	106	
50									
52									
54					26		68	60	DS pp=1.5
56									
58					16				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

Converse Consultants

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Drawing No. A-15

V-01 99343701.GPJ

Log No. B-12

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy
Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 37.5
Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		FILL: SILTY SAND With Gravel; tan, dry							
					16				
2									
4					25				
					35/4"				
6									
8									
					35/2"				
10		SILTY SAND With Gravel (SM); very dense, tan, dry			35/7"				
		---cobbles							
12									A;G;P K;DS
14					35/7"		2		
16									
		---cobbles							
18									
		SILTY SAND With Gravel (SM); very dense, light brown, dry			35/6"				

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End of Exploration at 60.0'
Converse Sampler (white symbol=no recovery)
SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

Converse Consultants

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Drawing No. A-16

Log No. B-12

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 37.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
	SILTY SAND With Gravel (SM); very dense, light brown, dry			35/6"				
22	CEMENTED SAND AND GRAVEL; hard, grayish brown, dry							
	SILTY SAND With Gravel (SM); very dense, tan, dry							
24				35/5"		2		
26								
28								
	---with gypsum, dense			35		3		
30	---with clay			35				
32								
34				28		5		
36								
	SANDY LEAN CLAY (CL); very stiff, dark brown, moist							
38	---slight odor							
				21				K pp=3.0

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End of Exploration at 60.0' Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-17

LV-B1 99343701.GPJ

Log No. B-12

Date of Drilling: 9/14/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 37.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SANDY LEAN CLAY (CL); very stiff, dark brown, moist			44				
42									
44									
		—wet, stiff			15		80	53	pp=1..
46									
48									
					11				
50									
52									
		---with gravel, very stiff							
54					16		31	87	pp=1.:
56									
58									
					18				pp=1.:

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recover

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-18

.21 9334370 1.GPJ

Log No. B-101

Date of Drilling: 9/20/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		**SILTY SAND With Gravel (SM);** occasional cobbles, medium dense, tan, slightly moist			11				
4		—dense			35				
5					21				
7.5									G;P; Sol;Ch
9					42		4		
10					39				
14.5					42				
19.5		---very dense			35/1"				

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-19

Log No. B-101

Date of Drilling: 9/20/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
	SILTY SAND With Gravel (SM); occasional cobbles, very dense, tan, slightly moist	55				
22						
24		35/7"		4		
26						
28						
	---dense	35/8"		4		
30		35				
32						
34		35/9"				
36						
38	SANDY LEAN CLAY (CL): very stiff, dark brown, slightly moist					
		21		52	64	A:DS

End of Exploration at 60.0' Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-20

Log No. B-101

Date of Drilling: 9/20/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
40		Drive		10				
	SILTY SAND With Gravel (SM); medium dense, grayish brown, dry							
42								
	SANDY LEAN CLAY (CL); stiff, dark brown, moist							
44		Drive		11		45	67	
46								
48								
49		Drive		25				pp=1.5
50	---partially cemented lens, moderately hard, brownish white, wet							
52								
54		Drive		11		42	75	A
	---with gravel							
56								
58								
59		Drive		14		42	74	

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-21

91 9334370I.GPJ

Log No. B-102

Date of Drilling: 9/23/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 43.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); very dense, light brown, slightly moist			35/5"				
22									A;G;K; DS
24					35/8"				
		---with cobbles							
26									
28									
					35/4"				
30					35/10"				
32									
34		---dense			35/6"		5		
36									
38		---partially cemented, moderately hard							
					35/0"				

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End of Exploration at 50.0'

Converse Sampler (white symbol=no recovery)
SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

Converse Consultants

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Drawing No.
A-22

B1 93343701.GPJ

Log No. B-102

Date of Drilling: 9/23/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 43.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); trace clay, very dense, light brown, slightly moist			35/9"				
		---with cobbles							
42		SANDY LEAN CLAY (CL); very stiff, dark brown, moist							
44					28				
46									
48									
		---stiff			13		48	68	A
50									
52									
54									
56									
58									

End of Exploration at 50.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-24

B1 993343701.GPJ

Log No. B-103

Date of Drilling: 9/21/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Drive	Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		FILL: SILTY SAND With Clay; tan, dry			14				
4					14				
5		SILTY SAND With Gravel (SM); dense, tan, dry			26				
6		---with cobbles							
9					35		4		
10					25				
14					42				
19					26		3		

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End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
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Clark County, Nevada

Project No. 99-33437-01

Converse Consultants

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Drawing No. A-25

V-81 9933437-01 GPJ

Log No. B-103

Date of Drilling: 9/21/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Summary of Subsurface Conditions	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
	SILTY SAND With Gravel (SM); medium dense, tan, dry	24				
22						
	---with cobbles, very dense					
24		35/5"		3		
26						
28						
		35/0"				
30		59/8"				
32						A.G
34		62/8"				
36						
	---partially cemented, moderately hard					
38						
		35/0"				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

Drawing No.

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B1 99343701.GPJ

Log No. B-103

Date of Drilling: 9/21/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 42.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
40–42	SILTY SAND With Gravel (SM); partially cemented, moderately hard, tan, dry							
42	SANDY LEAN CLAY (CL); very stiff, dark brown, slightly moist							
44		X		21				K pp=2.5
45	---stiff	X		13				
49		X		12		88	51	DS
54		X		10				
59		X		12				

End of Exploration at 60.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-37

81 9334370 1.GPJ

Log No. B-104

Date of Drilling: 9/21/99
Driller: D. Wilson
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 43.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Graphic Log	Description	Drive	Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		FILL: SILTY SAND With Gravel; tan, dry			9				
2		SILTY SAND With Gravel and Cobbles (SM); dense, tan, slightly moist							Sol
4					35/7"		3		
		—very dense			35/3"				
6									
8									
					45				
10					35/6"				
12									A,G,DS
14					53/7"		1		
16									
18									
					35/2"				

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End of Exploration at 54.0' | Converse Sampler (white symbol=no recovery) | SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

Converse Consultants

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Drawing No. A-28

91 03343701.GPJ

Log No. B-104

Date of Drilling: 9/21/99
Driller: D. Wilson
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 43.5

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		CEMENTED SAND AND GRAVEL; hard, tan, dry							
42									
		SILTY SAND (SM); medium dense, gray, wet							
44		SANDY LEAN CLAY (CL); very stiff, dark brown, slightly moist	X		18				pp=1..
46									
48									
			X		16				
50									
52									
54									
56									
58									

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End of Exploration at 54.0'

Converse Sampler (white symbol=no recovery)   SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

Drawing No.
A-20

Converse Consultants

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01 9334370 1.GPJ

Log No. B-105

Date of Drilling: 9/22/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 30.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
0		FILL: SILTY SAND With Clay and Gravel; tan, dry			6				
2		WELL GRADED SAND With Silt and Gravel (SW-SM); trace clay, dense, tan, slightly moist							
		—with cobbles, very dense							
4					35/4"				
5					35/7"				
9		---dense			35/8"		5		
10					35/4"				
14		---very dense			60/9"		4		
17		---partially cemented, moderately hard							
19					52				

End of Exploration at 40.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

Drawing No.

Log No. B-105

Date of Drilling: 9/22/99 | Location: See Drawing No. 2 | Ground Surface Elevation (ft): Not Available
Driller: T. High | Borehole Diameter: 8" | Equipment: BK-81 Hollow Stem Auger
Logged By: M. Stacy | Groundwater Depth (ft): 30.0 | Driving Wt. and Drop: 140#/30"

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SUMMARY OF SUBSURFACE CONDITIONS

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Depth (ft)	Description	Drive	Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
20	WELL GRADED SAND With Silt and Gravel (SW-SM); dense, grayish brown, slightly moist			44				
22								A;P;K
24				35/1"				
29	---with cobbles			35/3"				
30				35/2"		3		
32								K
34				35		13	122	
38	CEMENTED SAND AND GRAVEL; hard, grayish brown, slightly moist							
39				35/0"				

End of Exploration at 40.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

Converse Consultants

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Drawing No. A-20

93343701.GPJ

Log No. B-106

Date of Drilling: 9/22/99
Driller: T. High
Logged By: M. Stacy

Location: See Drawing No. 2
Borehole Diameter: 8"
Groundwater Depth (ft): 30.0

Ground Surface Elevation (ft): Not Available
Equipment: BK-81 Hollow Stem Auger
Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.	Samples: Drive	Samples: Bulk	Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
		SILTY SAND With Gravel (SM); medium dense, grayish brown, dry			23				
2									A;G;Sol;Ch
4		—very dense			35/6"		7	111	
		---tan			52/9"				
6									
8									
					44				
10		---dense			37				
12									
		---very dense							
14					60/11"		5		
16									
18									
					58/7"				

APPROVED BY ___ ON ___

End of Exploration at 33.0'
Converse Sampler (white symbol=no recovery)
SPT Sampler (white symbol=no recovery)

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Project No.
99-33437-01

Converse Consultants

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Drawing No.
A-23

9334370 1.GPJ

Log No. B-106

Date of Drilling: 9/22/99	Location: See Drawing No. 2	Ground Surface Elevation (ft): Not Available
Driller: T. High	Borehole Diameter: 8"	Equipment: BK-81 Hollow Stem Auger
Logged By: M. Stacy	Groundwater Depth (ft): 30.0	Driving Wt. and Drop: 140#/30"

DRAFTED BY GLE

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read with the report. This summary applies only at the location and time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplified model of the actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Samples: Blow Count	Drill Rate (sec/ft)	Moisture (%)	Dry Density (lb/cf)	Field or Lab Tests
20		SILTY SAND With Gravel (SM); dense, grayish brown, dry	X	X	35/2"				
22									
23		—with cobbles							
24			X		35/6"				
26									
28									
29			X		35/8"		15	114	Ca
30		---gray, wet							
31		CEMENTED SAND AND GRAVEL; hard, gray, wet		X					
32									
34									
36									
38									

End of Exploration at 33.0'

Converse Sampler (white symbol=no recovery) SPT Sampler (white symbol=no recovery)

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Basic Management Incorporated
Clark County, Nevada

Project No. 99-33437-01

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Drawing No. A-34

93343701.GPJ

KEY TO SOILS SYMBOLS AND TERMS

Terms used in this report for describing soils according to thier texture and grain size distributions are generally in accordance with the UNIFIED SOILS CLASSIFICATION SYSTEM.

TERMS DESCRIBING CONDITION, CONSISTENCY, AND HARDNESS

COARSE GRAINED SOILS (major portion retained on No. 200 sieve) includes clean gravels, silty or clayey gravels, and silty, clayey, or gravelly sands. Consistency is rated according to relative density, as determined by laboratory tests.

DESCRIPTIVE TERM	RELATIVE DENSITY
very loose	0 to 15%
loose	15 to 40%
medium dense	40 to 70%
dense	70 to 85%
very dense	85 to 100%

FINE GRAINED SOILS (major portion passing No. 200 sieve) includes inorganic and organic silts and clays, gravelly, silty, or sandy clays, and clayey silts. Consistency is rated according to shearing strength as indicated by penetrometer readings or by direct shear tests.

DESCRIPTIVE TERM	SHEAR STRENGTH (ksf)
very soft	less than 0.25
soft	0.25 to 0.50
firm	0.50 to 1.00
stiff	1.00 to 2.00
very stiff	2.00 to 4.00
hard	4.00 and up

ROCK includes gravels, cobbles, rock, caliche, and bedrock materials. Hardness is related to field identification procedures described below.

DESCRIPTIVE TERM	SHEAR STRENGTH (ksf)
soft	can be dug by hand and crushed by fingers
moderately hard	friable, can be gouged deeply with knife and will crumble readily under light hammer blows
hard	knife scratch leaves dust trace and will withstand a few hammer blows before breaking
very hard	scratched with knife with difficulty and is difficult to break with hammer blows

SIZE PROPORTIONS

DESIGNATION	PERCENT BY WEIGHT
trace	0 to 5
few	5 to 10
little	15 to 25
some	30 to 45

SOIL TYPE GRAPHIC KEY

Silt

Lean Clay

Elastic Silt

Fat Clay

Gravel

Sand

Caliche or Cemented Soil

Gypsum

Partially Cemented

Fill

SOIL TYPE GRAPHIC KEY

MOISTURE CONTENT IS INDICATED BY

dry
slightly moist
moist
very moist
wet

LEGEND OF LABORATORY TESTS

A	Liquid & Plastic Limits	G	Grain	R	Resistivity
C	Consolidation	H	Horticultural Tests	RV	R–Value
Ch	Chemical	K	Permeability	S	Swell
Disp	Dispersion	N	Chemical Heave	Sol	Solubility
DR	Drill Rate	P	Compaction	T	Triaxial
DS	Direct Shear	pp	Unconfined Compressive Strength (tsf)	UU	Unconsolidated, Undrained

GROUNDWATER LEVEL KEY

Water level during drilling

Stabilized water level

SAMPLER TYPES

Converse Sampler

Converse Sampler (no recovery)

Standard Penetration Test (SPT)

SPT Sampler (no recovery)

Shelby Sampler

Bulk Sample

WELL DESIGN GRAPHIC KEY

Grout

Bentonite

PVC Screen

Silica Sand

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Drawing No.

A-35

CLASSIFICATION OF SOILS

ASTM Designation: D2487-93
(ASTM version of Unified Soil Classification System)

Criteria for Assigning Group Symbols and Group Names using Laboratory Tests[a]				Soil Classification: Group Symbol	Soil Classification: Group Name[b]
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on #4 sieve	Clean Gravels Less than 5% fines[c]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[e]	GW	Well-graded gravel[f]
			$Cu < 4$ and/or $Cc < 1$ or $Cc > 3$[e]	GP	Poorly graded gravel[f]
		Gravels with Fines More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	Sands 50% or more of coarse fraction passes #4 sieve	Clean Sands Less than 5% fines[d]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[e]	SW	Well-graded sand[i]
			$Cu < 6$ and/or $Cc < 1$ or $Cc > 3$[e]	SP	Poorly graded sand[i]
		Sands with Fines More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI>7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
			PI<4 or plots below "A" line[j]	ML	Silt[k,l,m]
		Organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay[k,l,m,n] / Organic silt[k,l,m,o]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
			PI plots below "A" line	MH	Elastic silt[k,l,m]
		Organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay[k,l,m,p] / Organic silt[k,l,m,q]
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor			PT	Peat

a Based on the material passing the 3-in. (75-mm) sieve.

b If field sample contained cobbles or boulders, or both, add "with cobbles" or "with boulders", or both to group name.

c GW-GM well graded gravel with silt
GW-GC well graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

d Sands with 5-12% fines require dual symbols:
SW-SM well graded sand with silt
SW-SC well graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

e $Cu = \frac{D_{60}}{D_{10}}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

f If soil contains $\geq$ 15% sand, add "with sand" to group name.

g If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.

h If fines are organic, add "with organic fines" to group name.

i If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

j If Atterberg limits plot in hatched area, soil is a CL-ML silty clay.

k If soil contains 15-29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

l If soil contains $\geq$ 30% plus No. 200, predominantly sand, add "sandy" to group name.

m If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

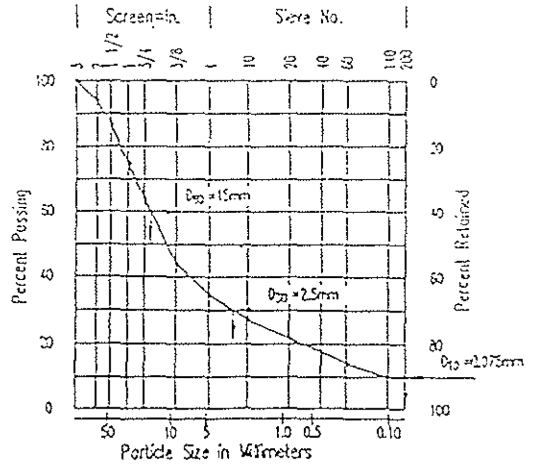
n PI $\geq$ 4 and plots on or above "A" line

o PI < 4 or plots below "A" line

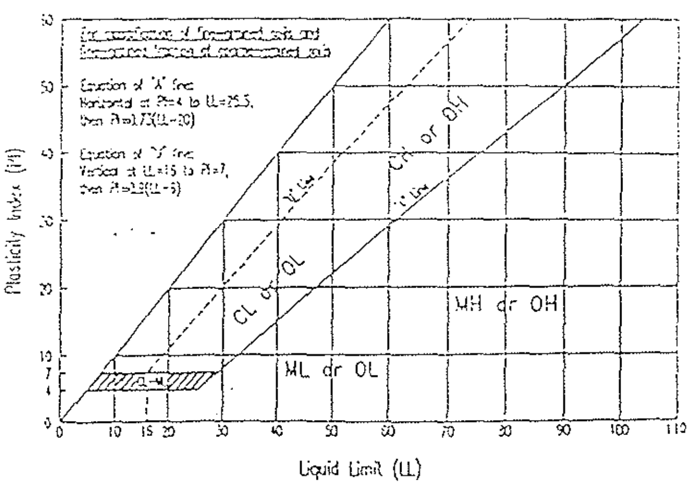
p PI plots on or above "A" line

q PI plots below "A" line

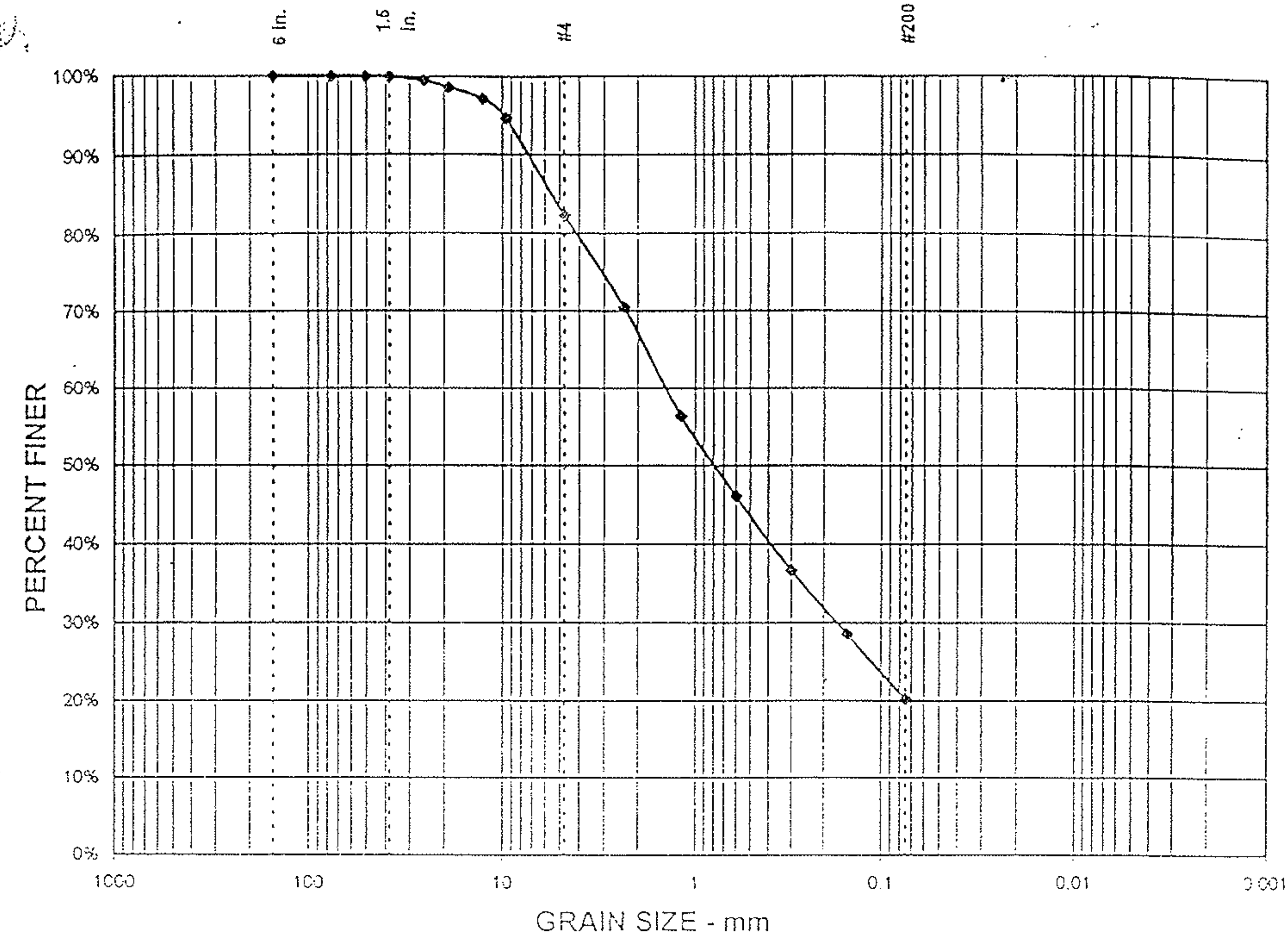
SIEVE ANALYSIS



$Cu = \frac{D_{60}}{D_{10}} = \frac{15}{0.075} = 200$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(2.5)^2}{0.075 \times 15} = 5.6$



GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	18%	D_{85} =	5.5	D_{15} =	
% SAND =	62%	D_{60} =	1.4	D_{10} =	
% SILT & CLAY =	20%	D_{50} =	0.8	C_u =	
		D_{30} =	0.2	C_c =	

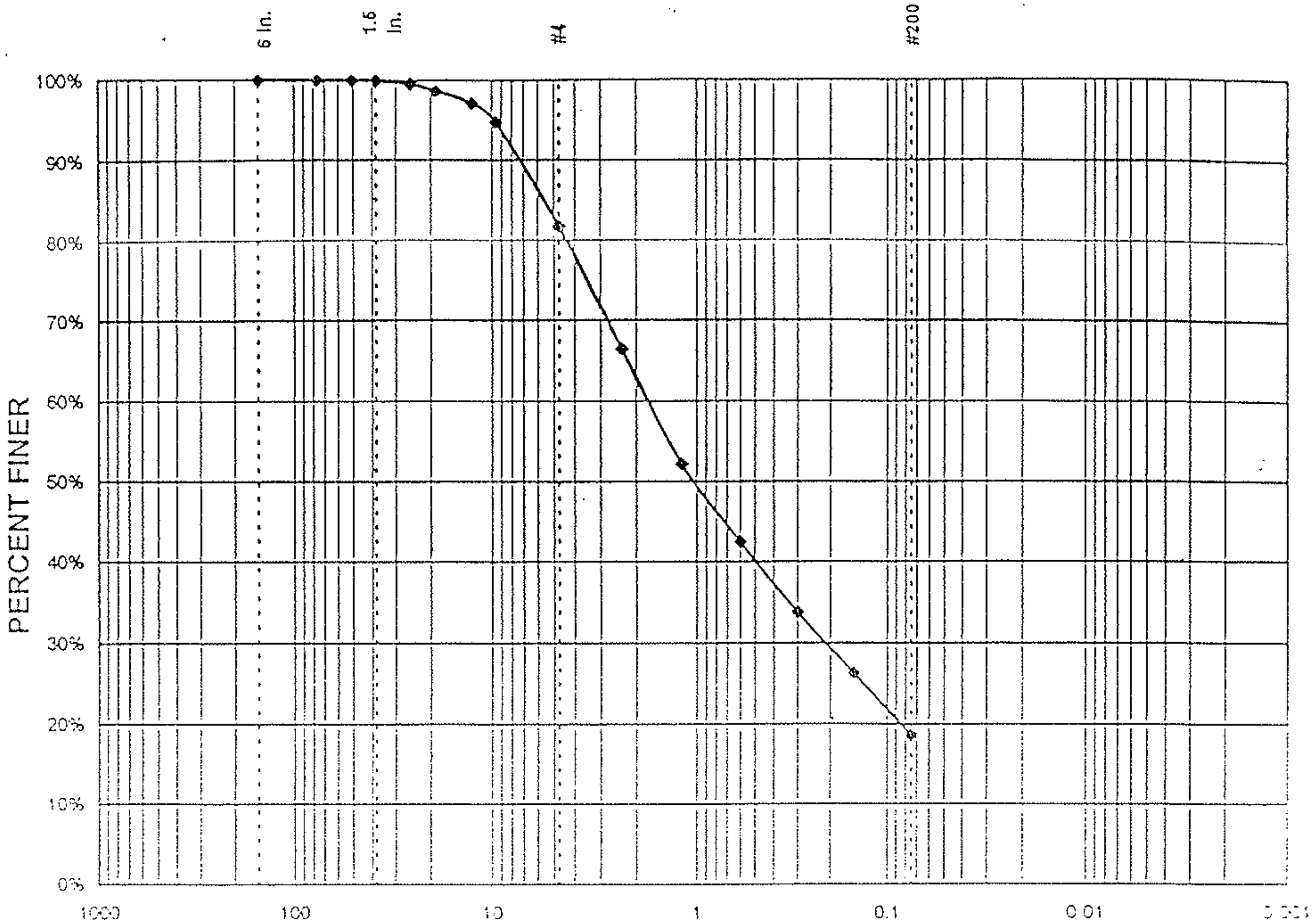
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 1 @ 20-25'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-37

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL = 18%	D_{85} = 5.6	D_{15} =
% SAND = 63%	D_{60} = 1.7	D_{10} =
% SILT & CLAY = 19%	D_{50} = 1.0	C_u =
	D_{30} = 0.2	C_c =

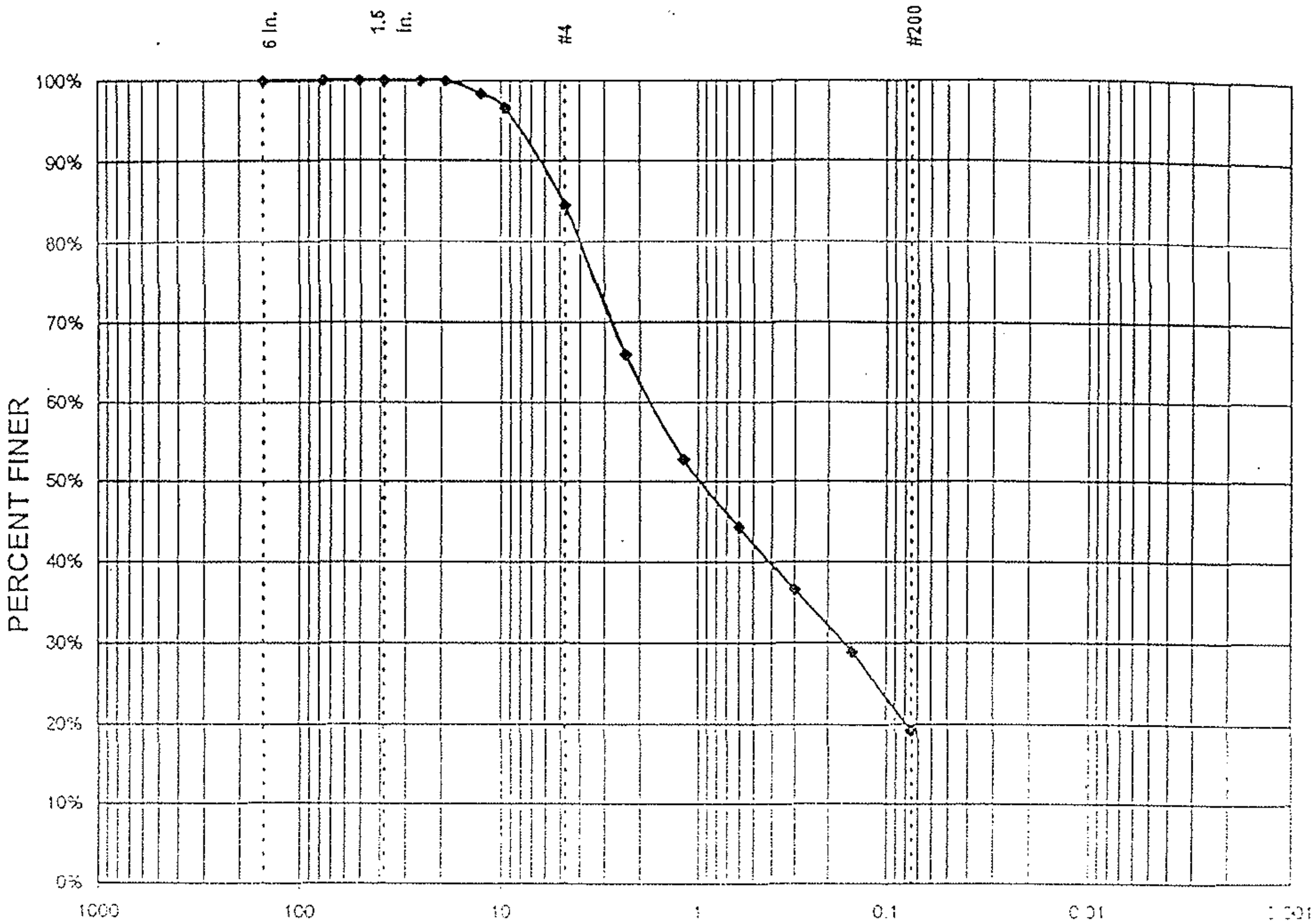
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 1 @ 30-35'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-38

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	16%	D_{85} = 4.9	D_{15} =
% SAND =	65%	D_{60} = 1.7	D_{10} =
% SILT & CLAY =	19%	D_{50} = 0.9	C_u =
		D_{30} = 0.2	C_c =

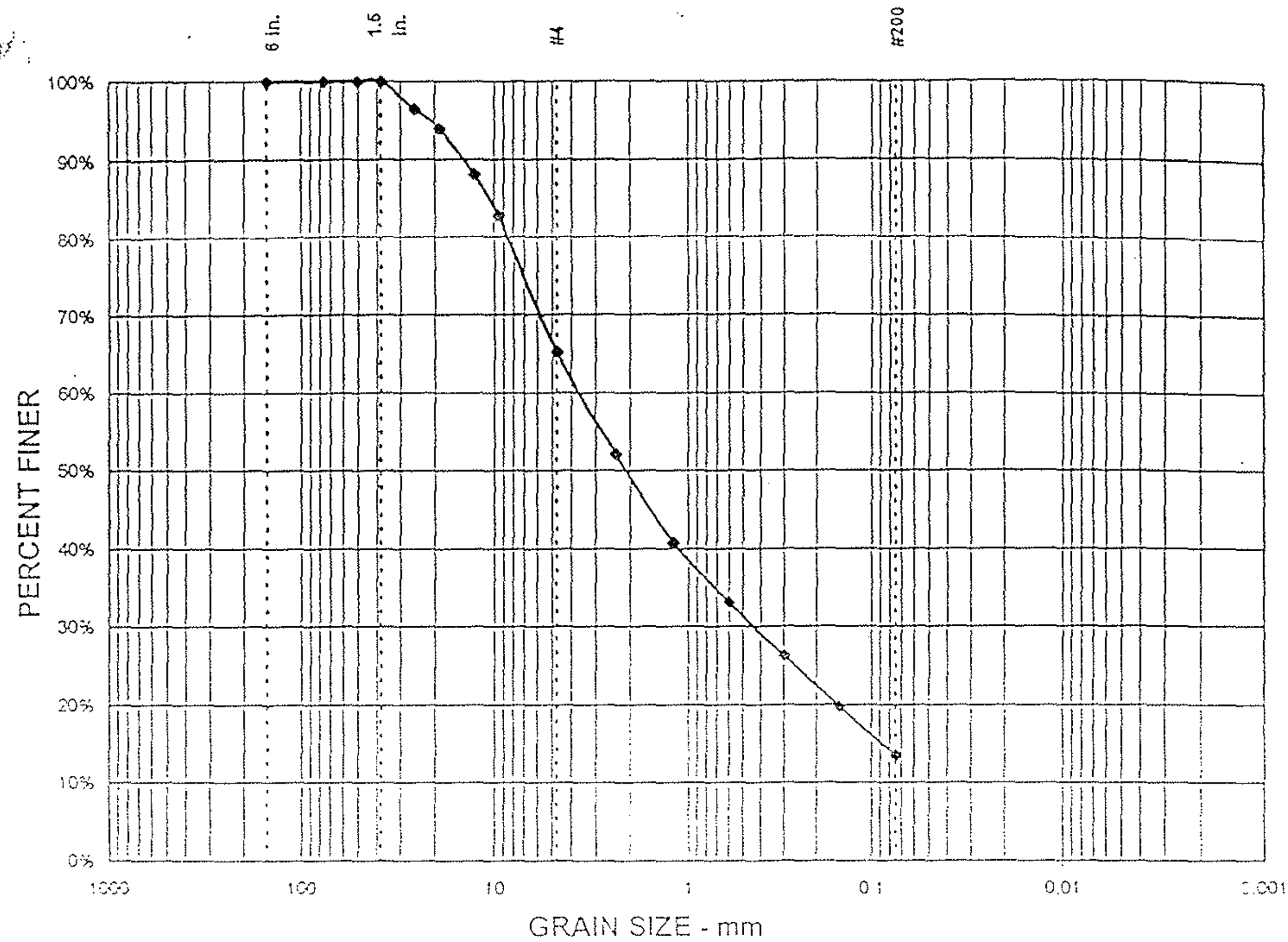
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B5 @ 20-25'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-39

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	35%	D_{85} = 10.8	D_{15} = 0.1
% SAND =	52%	D_{60} = 3.6	D_{10} =
% SILT & CLAY =	14%	D_{50} = 2.1	C_u =
		D_{30} = 0.4	C_c =

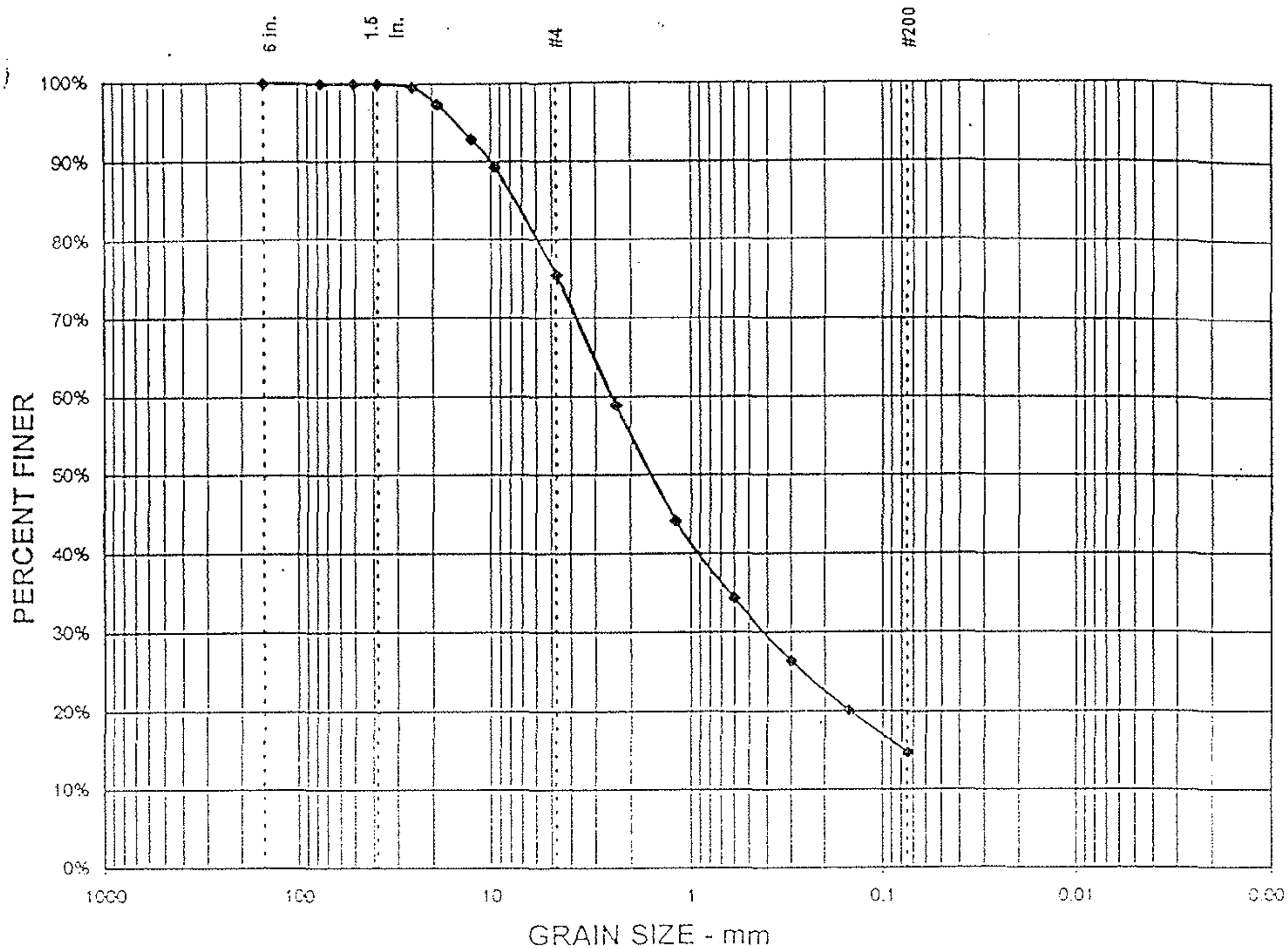
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 8 @ 35-40'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-4

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	24%	D_{35} = 7.7	D_{15} = 0.1
% SAND =	61%	D_{50} = 2.5	D_{10} =
% SILT & CLAY =	15%	D_{50} = 1.5	C_u =
		D_{30} = 0.4	C_c =

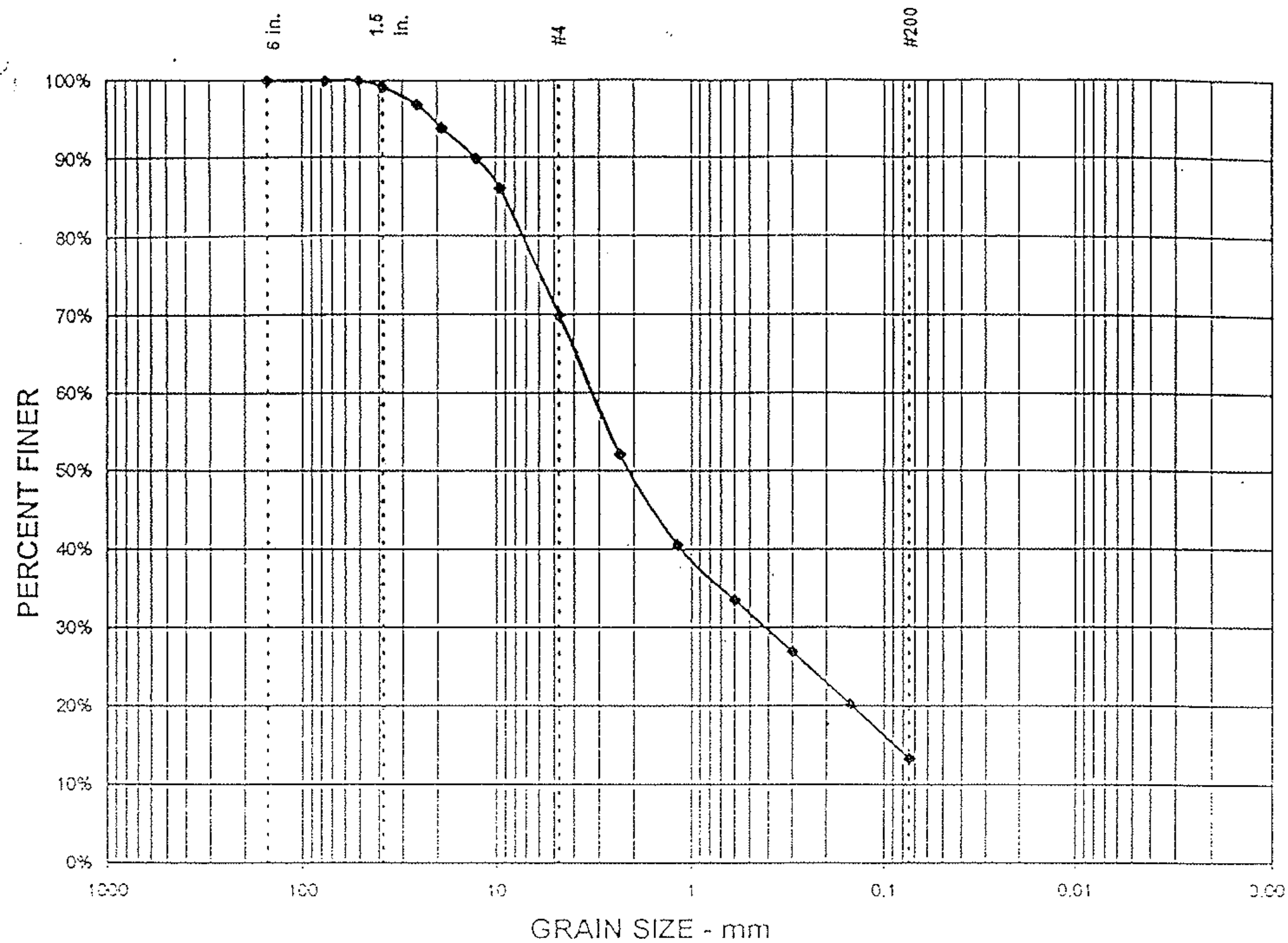
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 10 @ 30-35'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-41

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	30%	D_{85} = 9.1	D_{15} = 0.1
% SAND =	57%	D_{60} = 3.2	D_{10} =
% SILT & CLAY =	13%	D_{50} = 2.1	C_u =
		D_{30} = 0.4	C_c =

Project No.: 99-33437-01

Project Name: BMI Landfill

Date sampled: 10/01/99

Location: B12 @ 10-15'

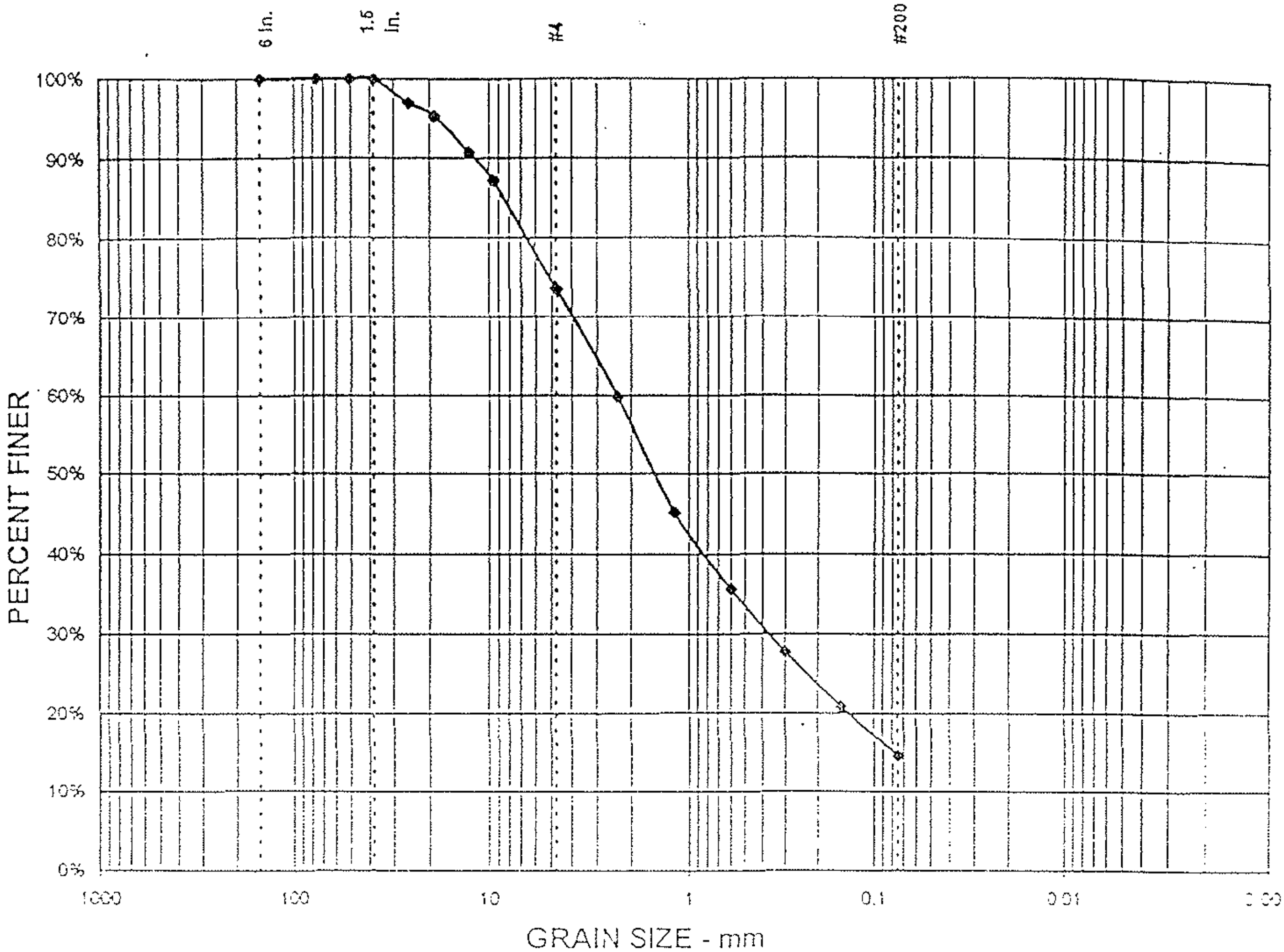
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-42

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	26%	D_{85} = 8.5	D_{15} = 0.1
% SAND =	59%	D_{60} = 2.4	D_{10} =
% SILT & CLAY =	15%	D_{50} = 1.5	C_u =
		D_{30} = 0.4	C_c =

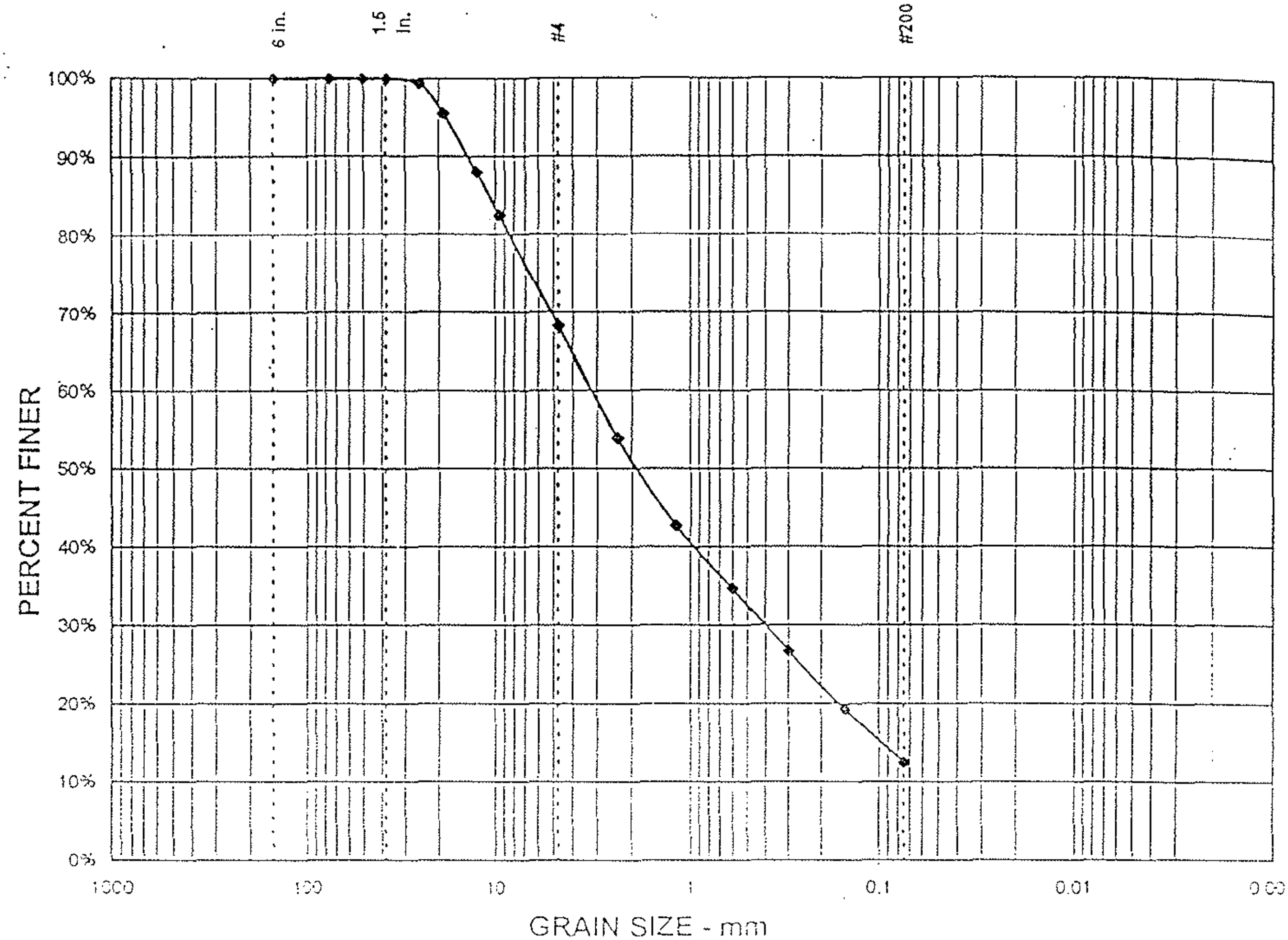
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 101 @ 5-10'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-4

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL = 32%	D_{85} = 10.9	D_{15} = 0.1
% SAND = 56%	D_{60} = 3.2	D_{10} =
% SILT & CLAY = 12%	D_{50} = 1.9	C_u =
	D_{30} = 0.4	C_c =

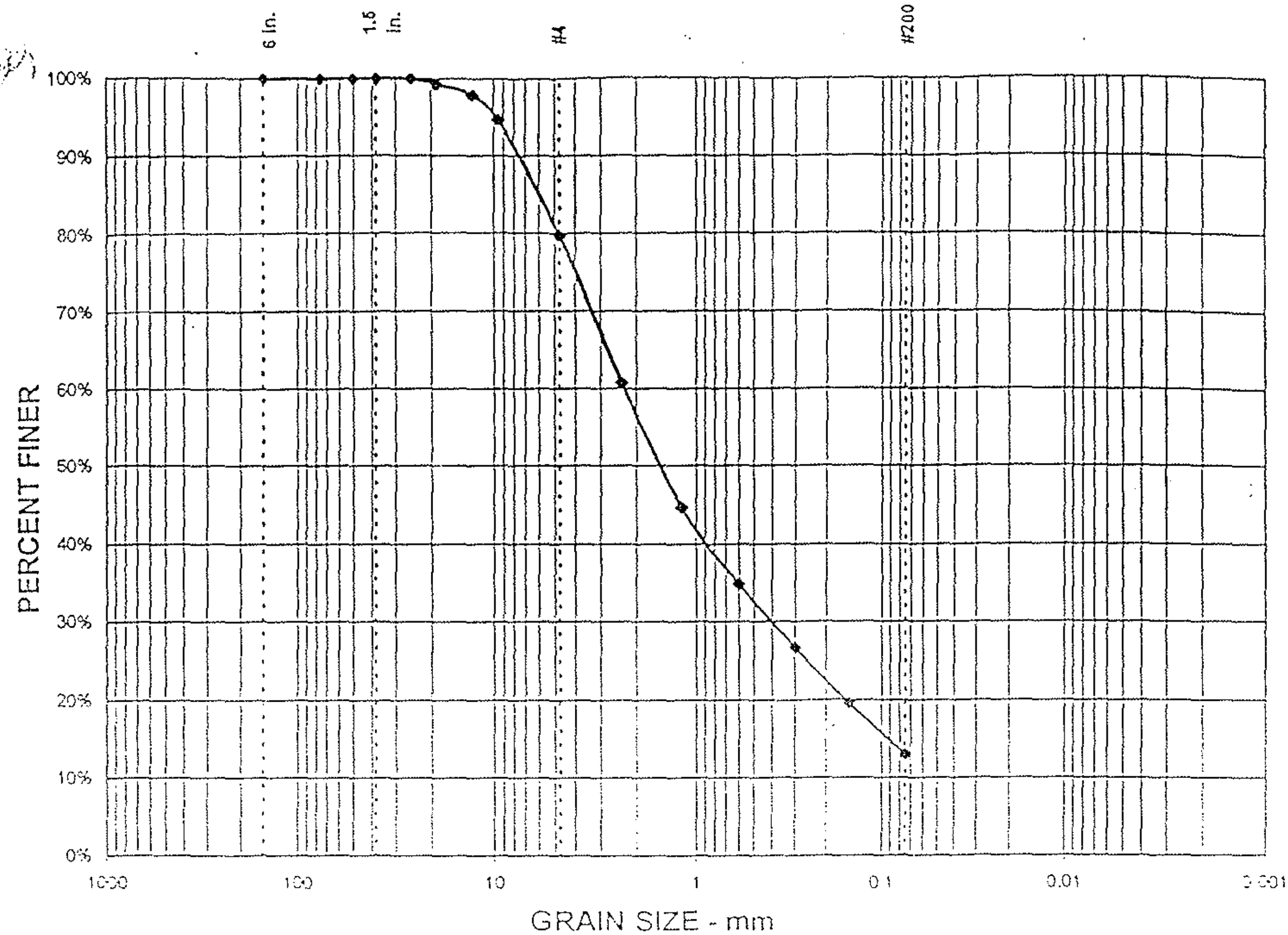
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 102 @ 20-25'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-44

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	20%	D_{85} = 6.1	D_{15} = 0.1
% SAND =	67%	D_{60} = 2.3	D_{10} =
% SILT & CLAY =	13%	D_{50} = 1.5	C_u =
		D_{30} = 0.4	C_c =

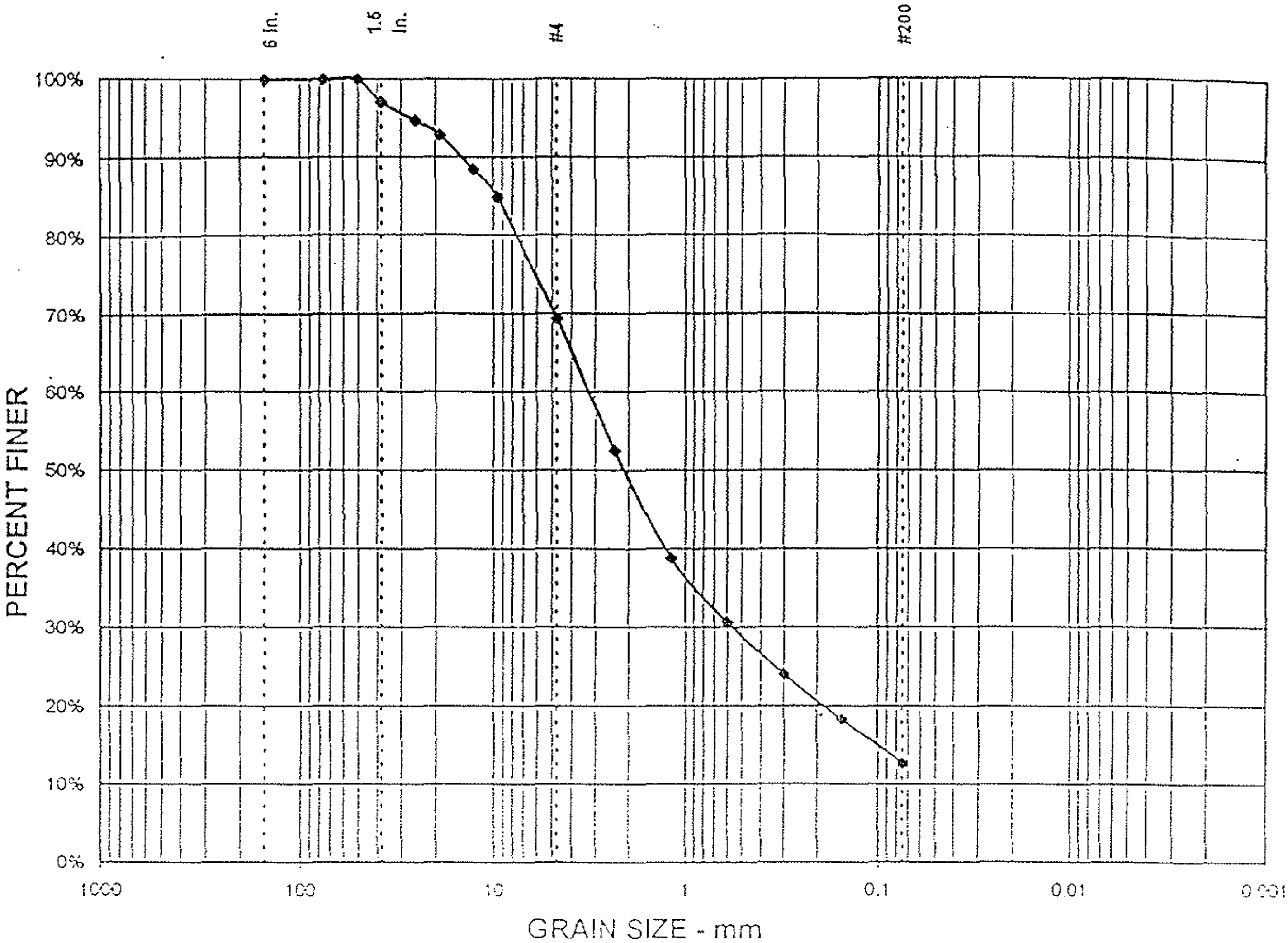
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 103 @ 30-35'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-45

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	31%	D_{85} = 9.6	D_{15} = 0.1
% SAND =	57%	D_{60} = 3.2	D_{10} =
% SILT & CLAY =	13%	D_{50} = 2.1	C_u =
		D_{30} = 0.6	C_c =

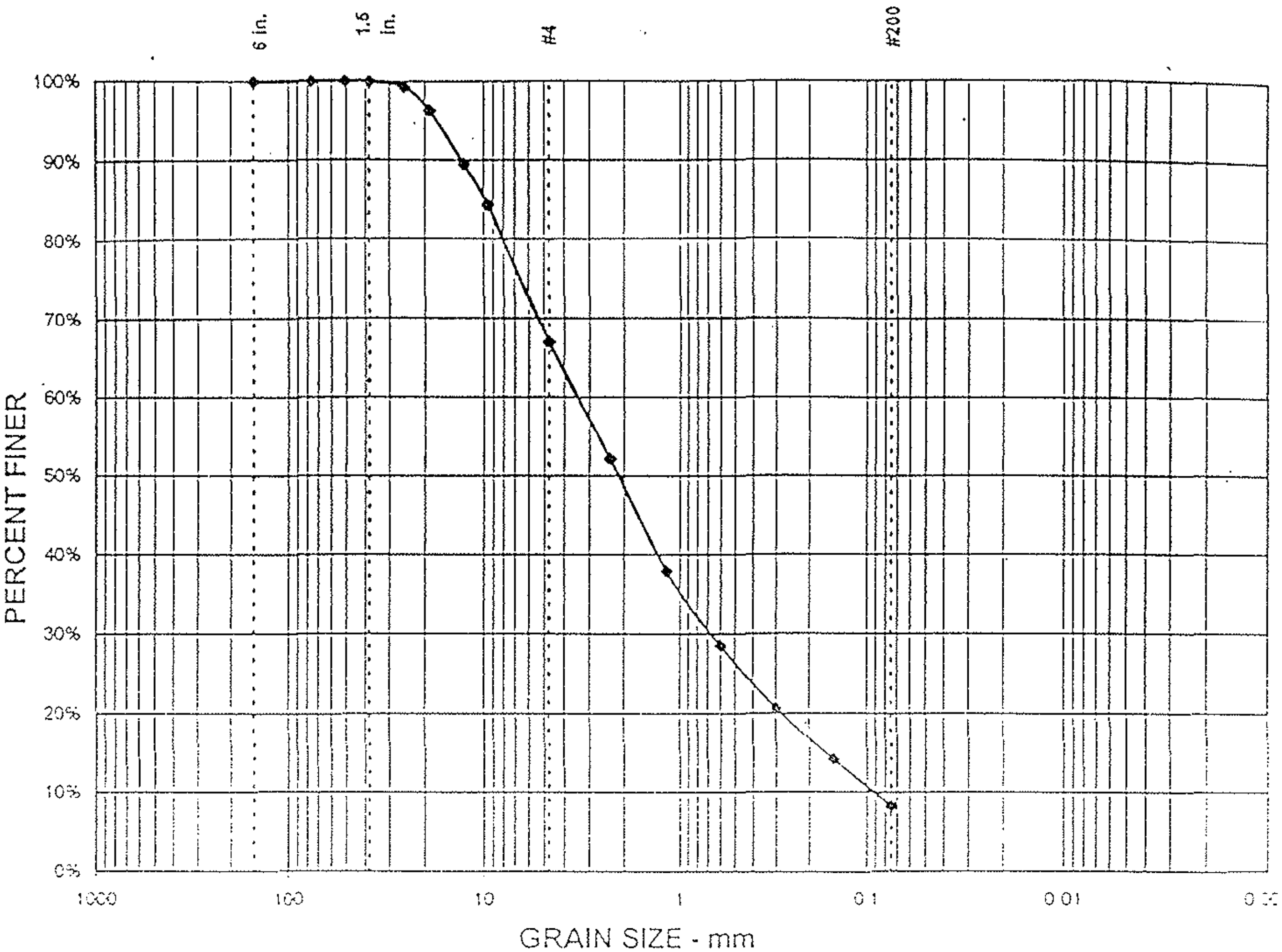
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 104 @ 10-15'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-46

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL = 33%		D_{85} = 9.9	D_{15} = 0.2
% SAND = 59%		D_{60} = 3.4	D_{10} = 0.1
% SILT & CLAY = 8%		D_{50} = 2.1	C_u = 37.4
		D_{30} = 0.7	C_c = 1.4

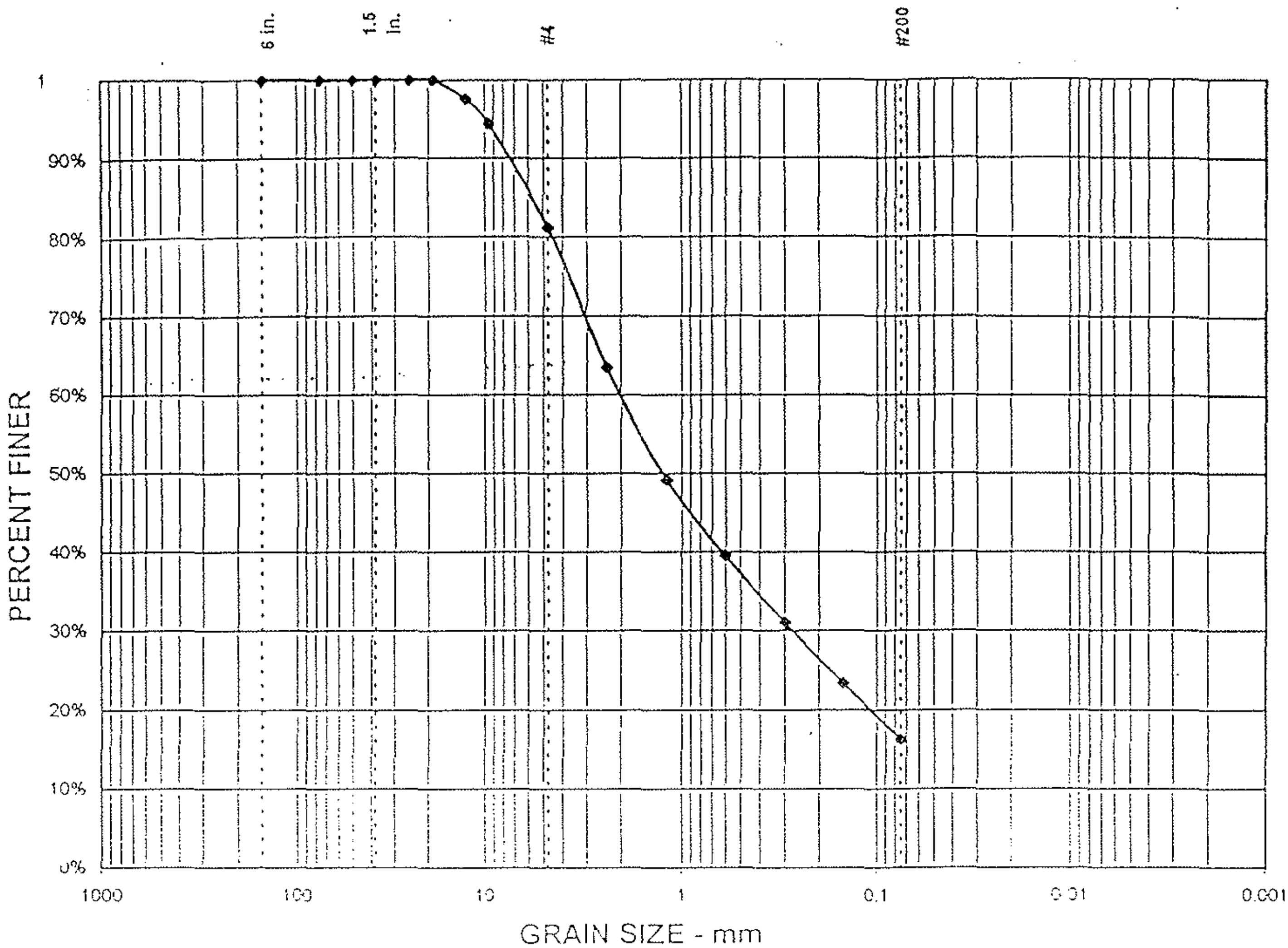
Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B105 @ 20-25'
Well Graded Sand with Silt & Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-47

GRAIN SIZE DISTRIBUTION GRAPH



TEST SUMMARY

% GRAVEL =	19%	D_{65} = 5.8	D_{15} =
% SAND =	65%	D_{50} = 2.0	D_{10} =
% SILT & CLAY =	16%	D_{50} = 1.2	C_u =
		D_{30} = 0.3	C_c =

Project No.: 99-33437-01
Project Name: BMI Landfill
Date sampled: Oct 1999
Location: B 106 @ 0-5'
Silty Sand with Gravel

GRAIN SIZE DISTRIBUTION TEST REPORT

CONVERSE CONSULTANTS

Drawing No. A-4

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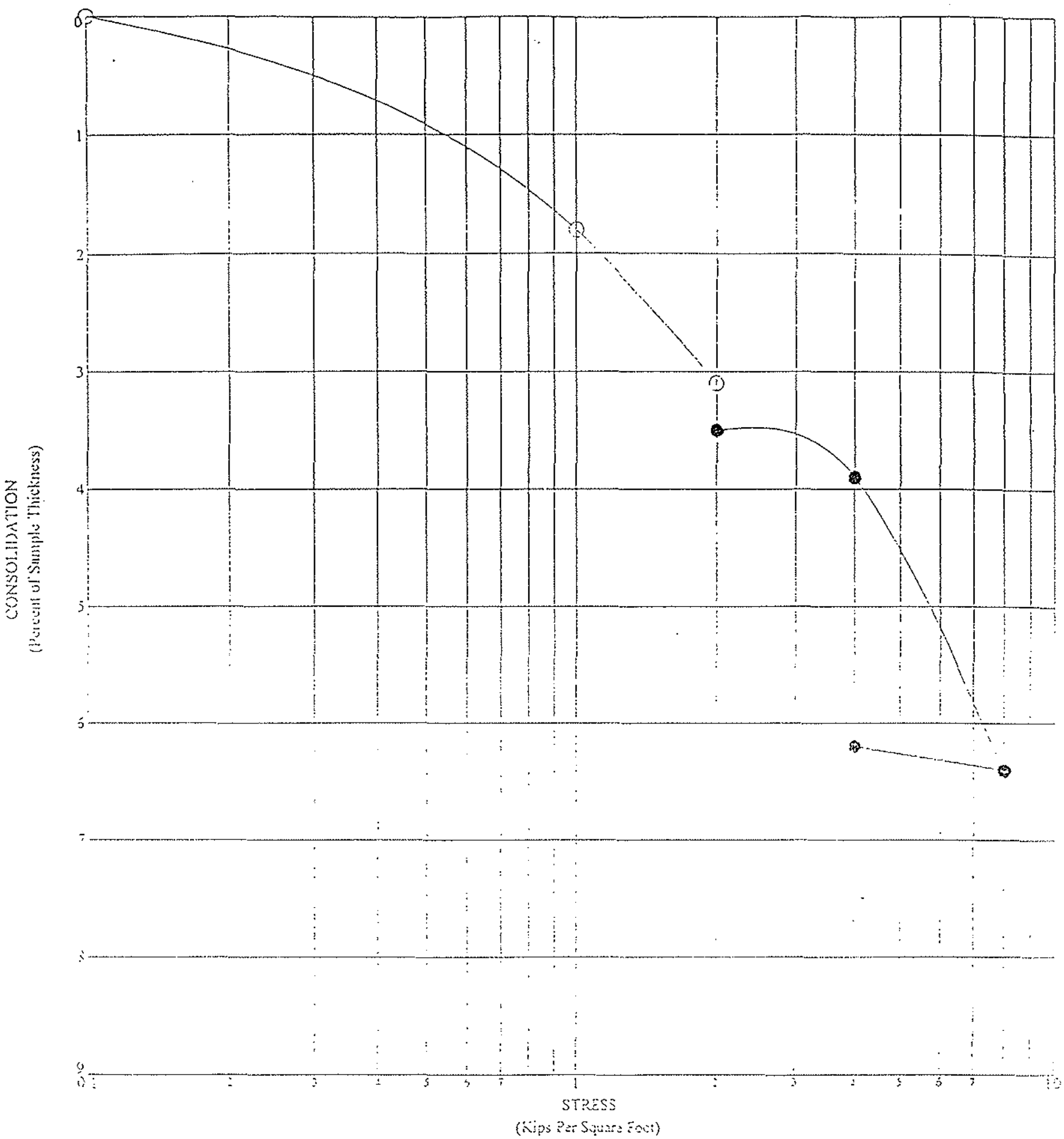


APPROVED BY ... ON

BORING NO: B- 8
DEPTH: 39.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 58 PCF
MOISTURE CONTENT: 64%

READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

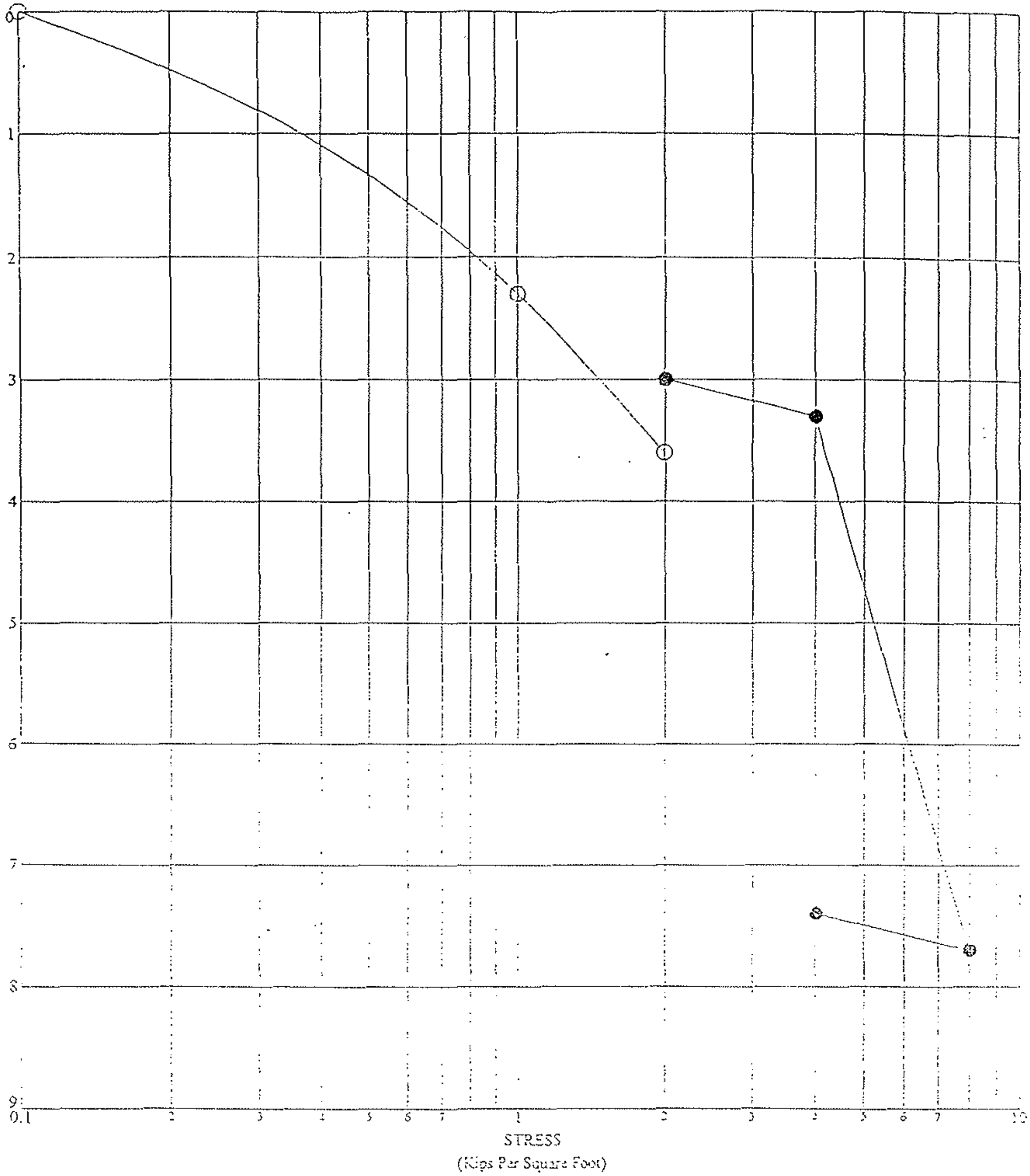
Project No.
99-33437-01

Converse Consultants

Geotechnical Engineering
and Applied Sciences

Drawing No.
A-50

DRAFTED BY GLE



APPROVED BY ON

BORING NO: B- 8
DEPTH: 49.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 70 PCF
MOISTURE CONTENT: 51 %

READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

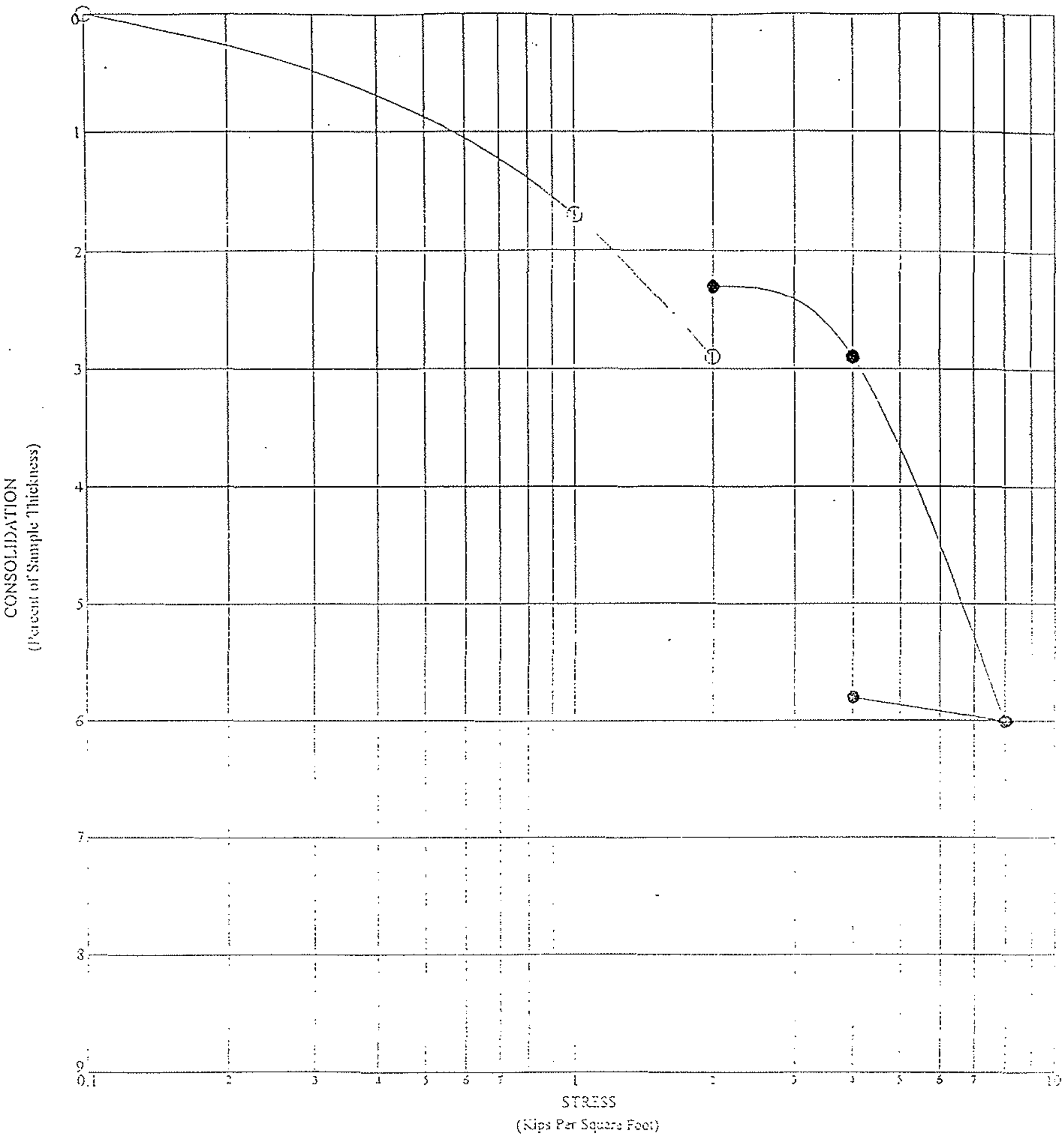
Project No.
99-33437-01

Converse Consultants

Geotechnical Engineering and Applied Sciences

Drawing No.
A-51

DRAFTED BY GLE



APPROVED BY ON

BORING NO: B-10
DEPTH: 54.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 61 PCF
MOISTURE CONTENT: 68%

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

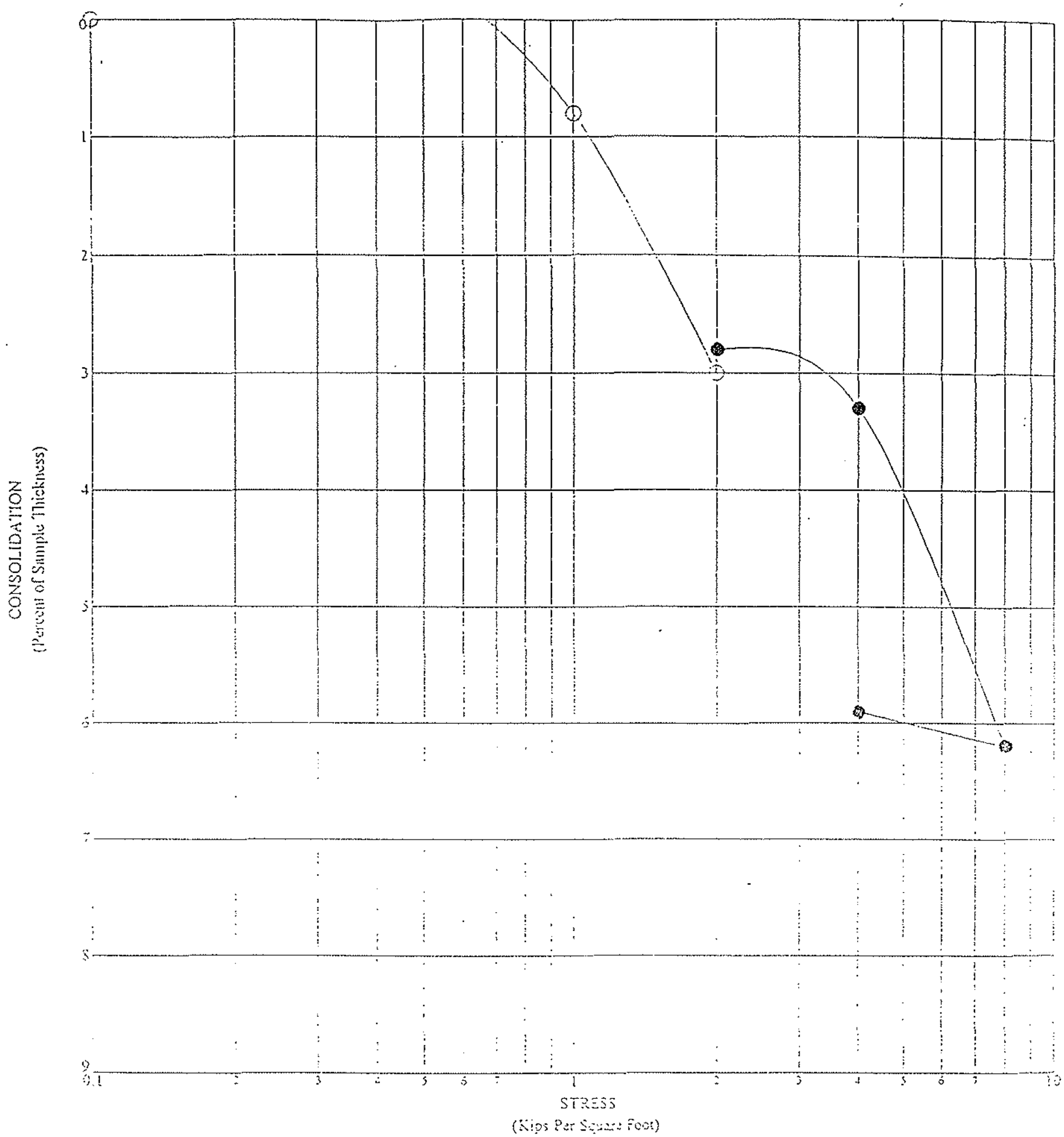
Project No. 99-33437-01

Converse Consultants

Geotechnical Engineering and Applied Sciences

Drawing No. A-52

DRAFTED BY GLE



APPROVED BY ON

BORING NO: B-101
DEPTH: 39.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 66 PCF
MOISTURE CONTENT: 45%

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

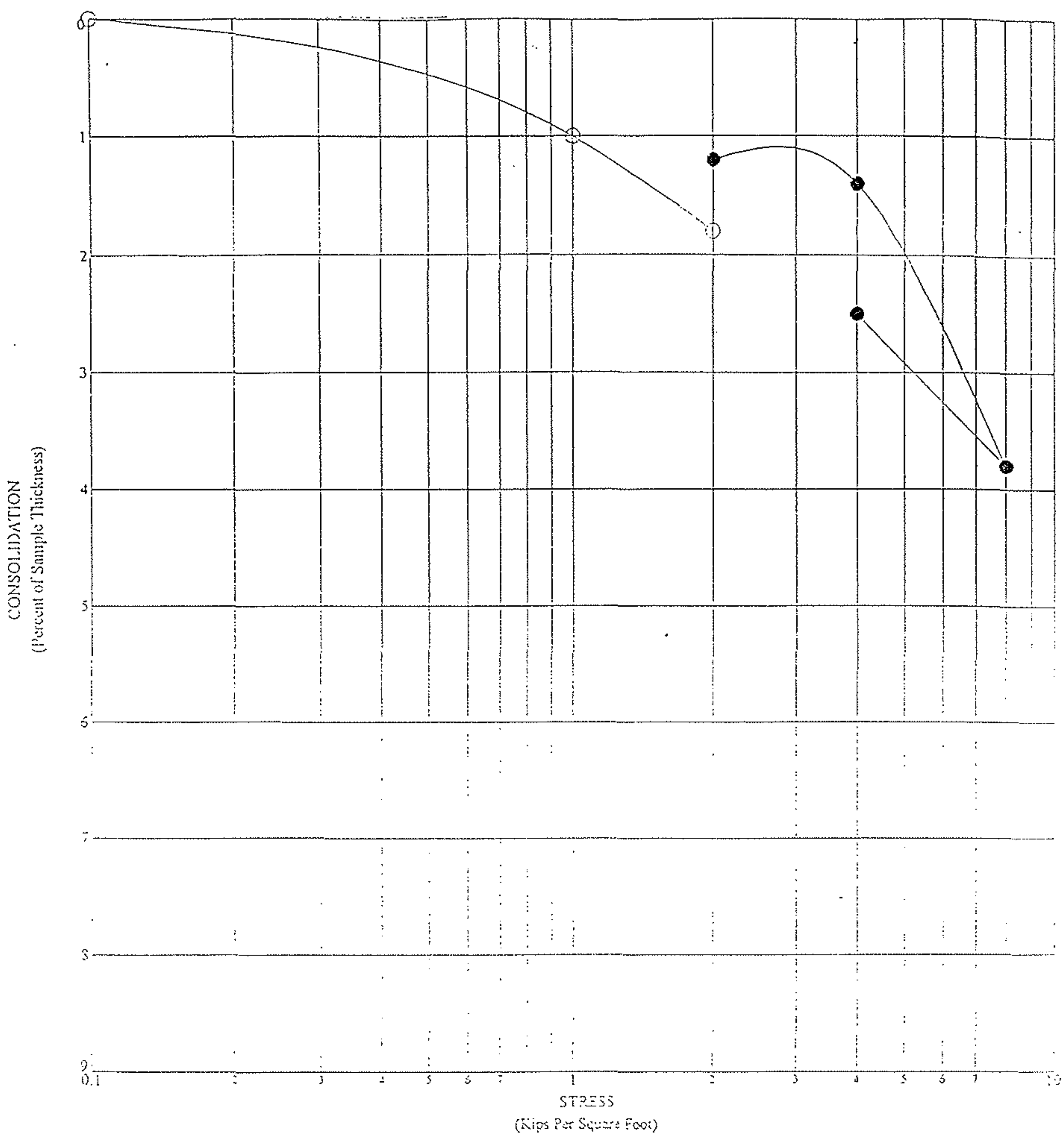
Project No. 99-33437-01

Converse Consultants

Geotechnical Engineering and Applied Sciences

Drawing No. A-53

DRAFTED ... GLE



APPROVED BY ON

BORING NO: B-101
DEPTH: 59.0
DESCRIPTION: LEAN CLAY
DRY DENSITY: 73 PCF
MOISTURE CONTENT: 38%

● READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

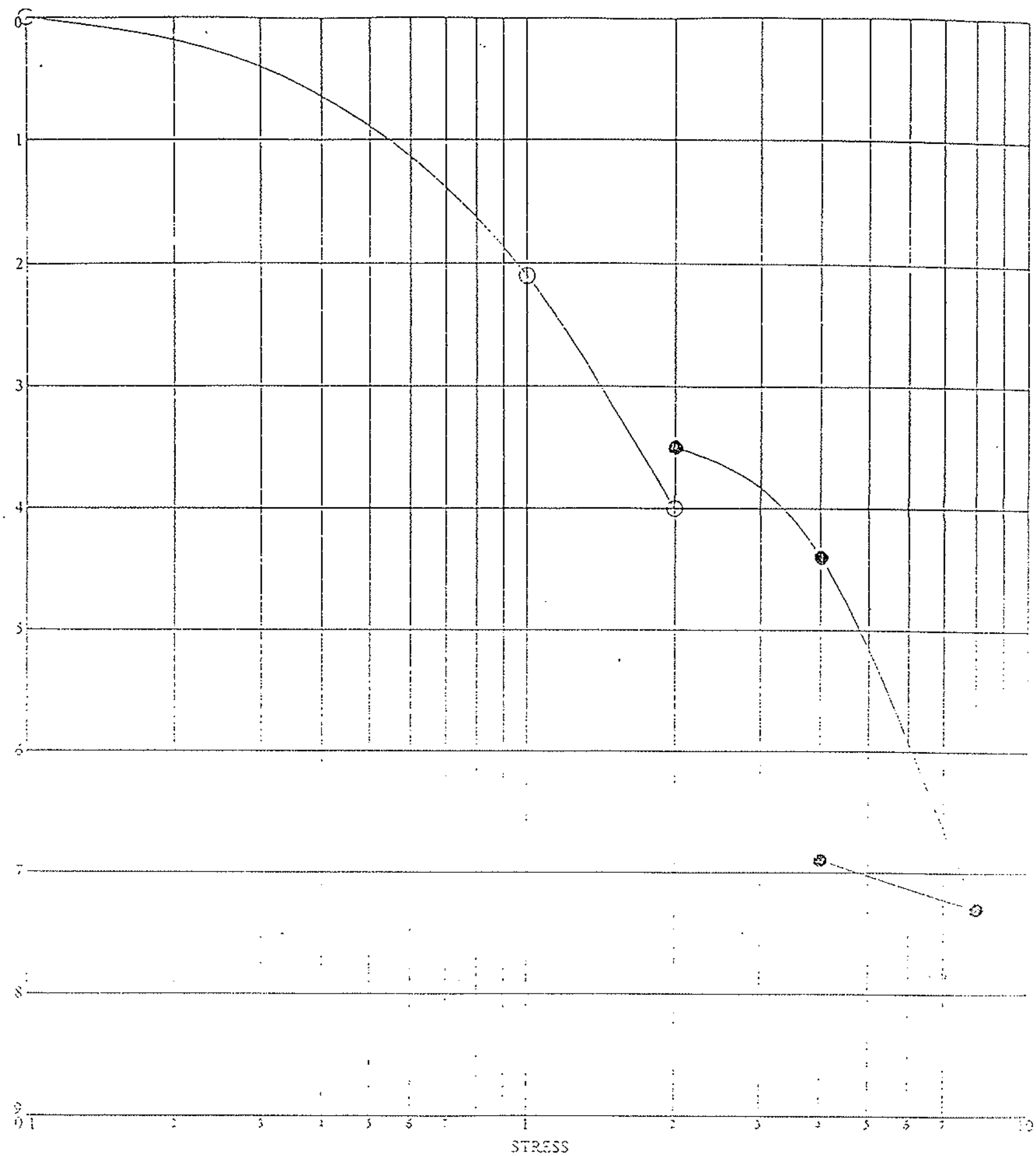
Project No. 99-33437-01

Converse Consultants

Geotechnical Engineering and Applied Sciences

Drawing No. A-54

DRAFTED BY GLE



APPROVED BY ON

BORING NO: B-102
DEPTH: 49.0
DESCRIPTION:
DRY DENSITY: 67 PCF
MOISTURE CONTENT: 49 %

READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

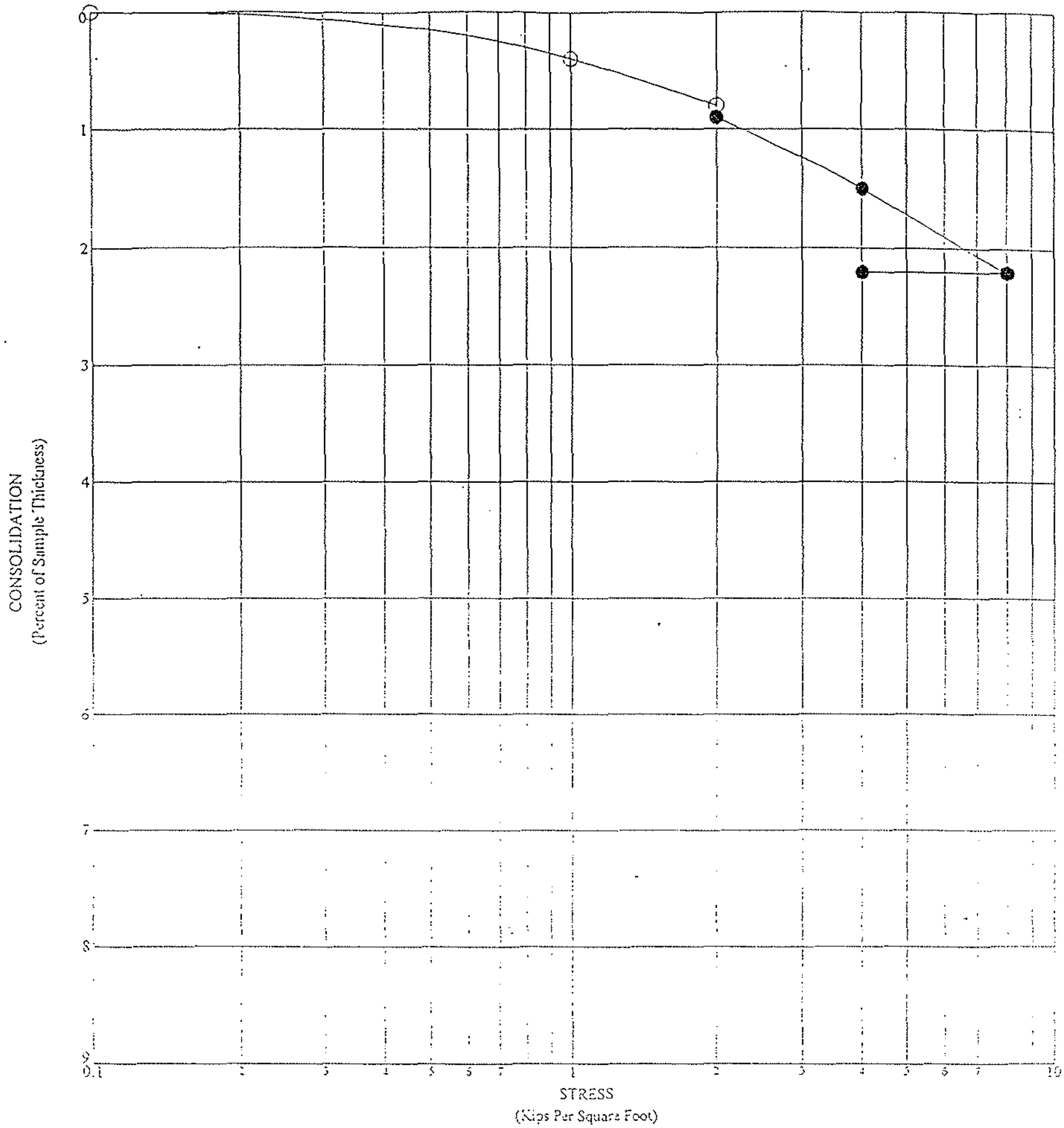
Project No. 99-33437-01

Converse Consultants

Geotechnical Engineering and Applied Sciences

Drawing No. A-55

DRAFTED BY GLE



APPROVED BY ON

BORING NO: B-105

DEPTH: 34.0

DESCRIPTION: SILTY SAND With Gravel

DRY DENSITY: 101 PCF

MOISTURE CONTENT: 5 %

⊗ READINGS AFTER SATURATION WITH WATER

CONSOLIDATION TEST

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION

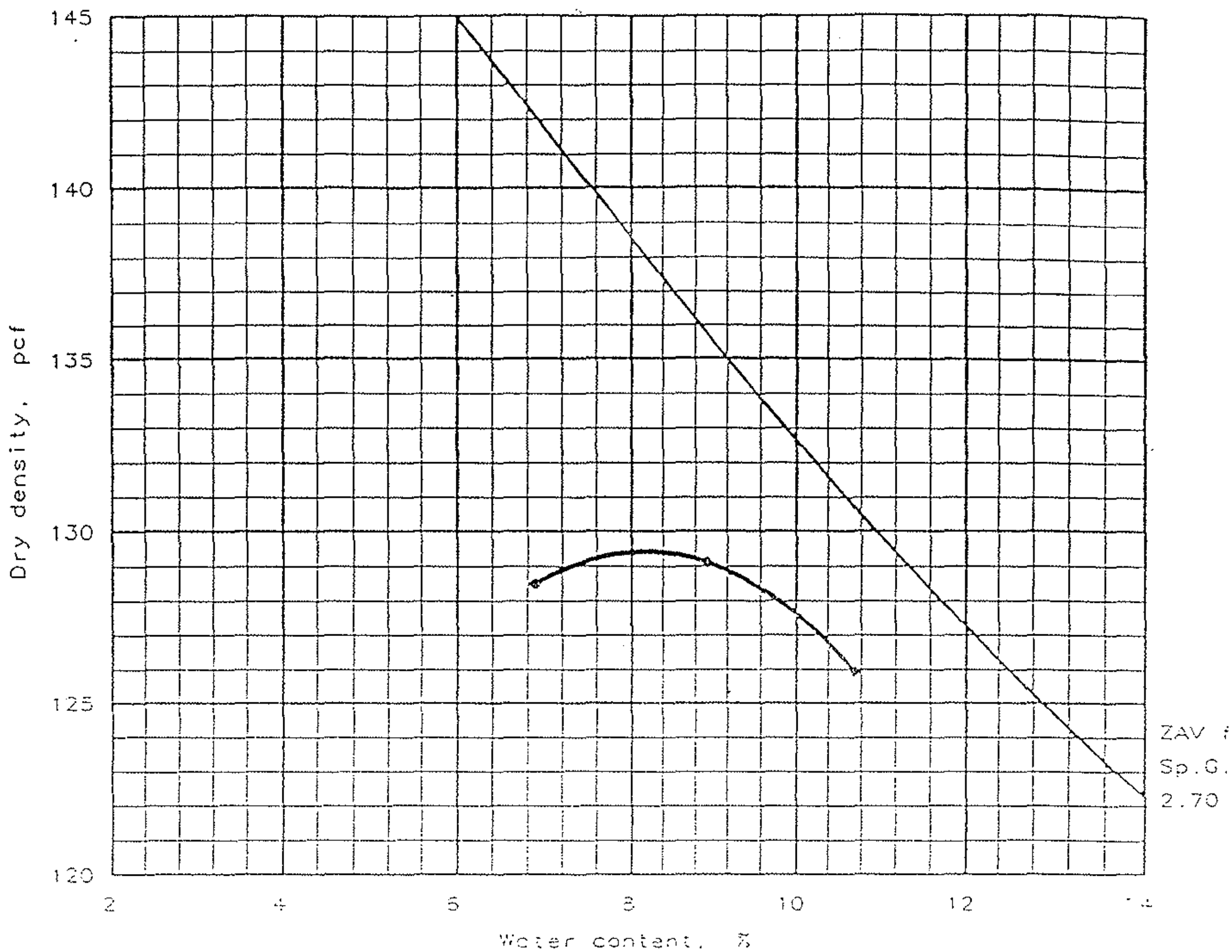
Project No. 99-33437-01

Converse Consultants

Geotechnical Engineering and Applied Sciences

Drawing No. A-55

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 1557-91 Procedure C, Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in	% < No.200
	USCS	AASHTO						
	SM				NV	NP	1.4 %	20.2

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 129.4 pcf Optimum moisture = 8.2 %	Silty Sand with Gravel

Project No.: 99-33437-01

Project: BMI Landfill

Location: B1 @ 20-25'

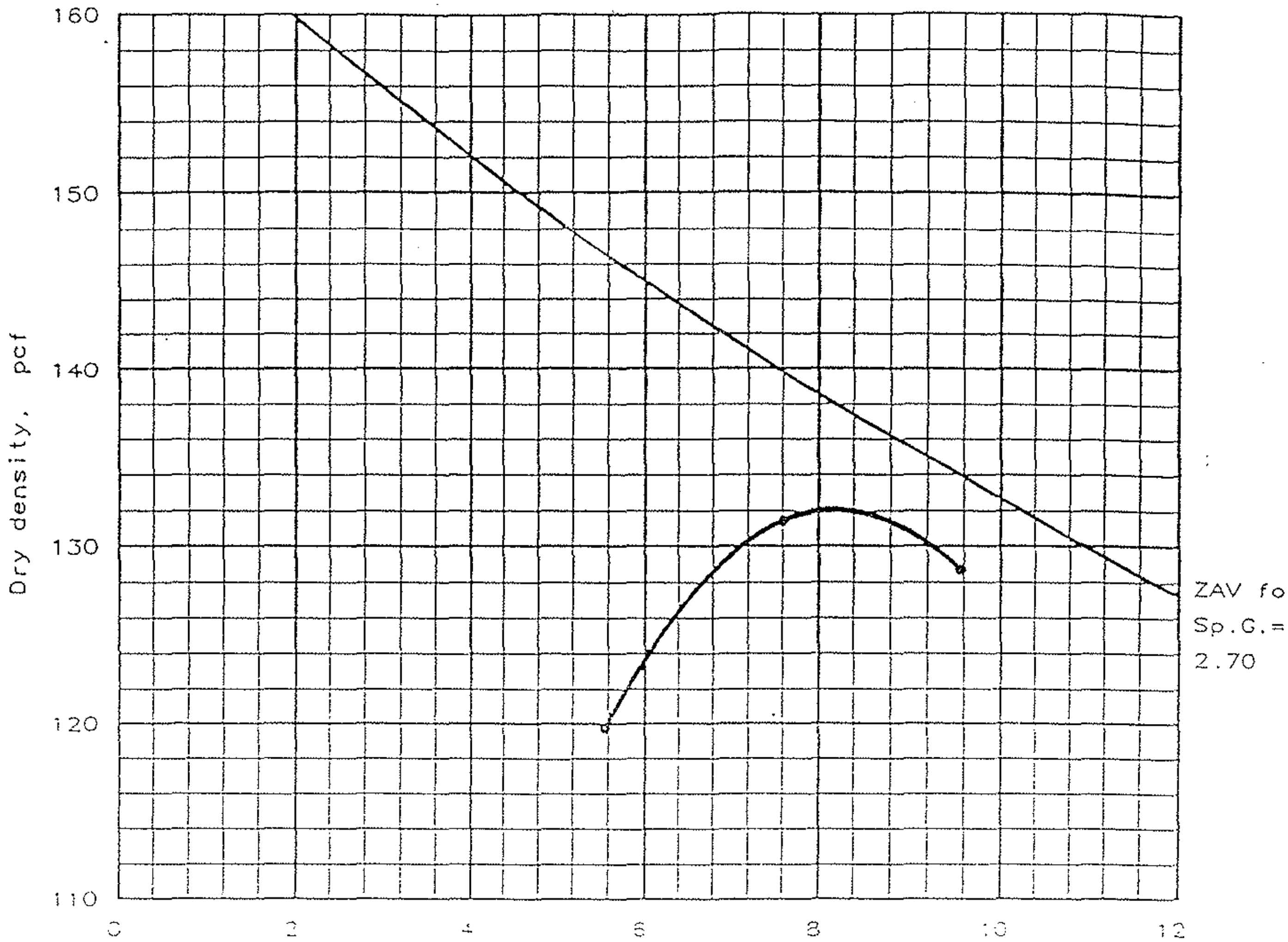
Date: 10-15-1999

MOISTURE-DENSITY RELATIONSHIP TEST

CONVERSE CONSULTANTS

Fig. No. A-57

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 1557-91 Procedure C, Modified

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in	% < No.200
	SM				NV	NP	0 %	19.2 %

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 132.1 pcf Optimum moisture = 8.2 %	Silty Sand with Gravel

Project No.: 99-33437-01

Project: BMI Landfill

Location: B5 @ 20-25'

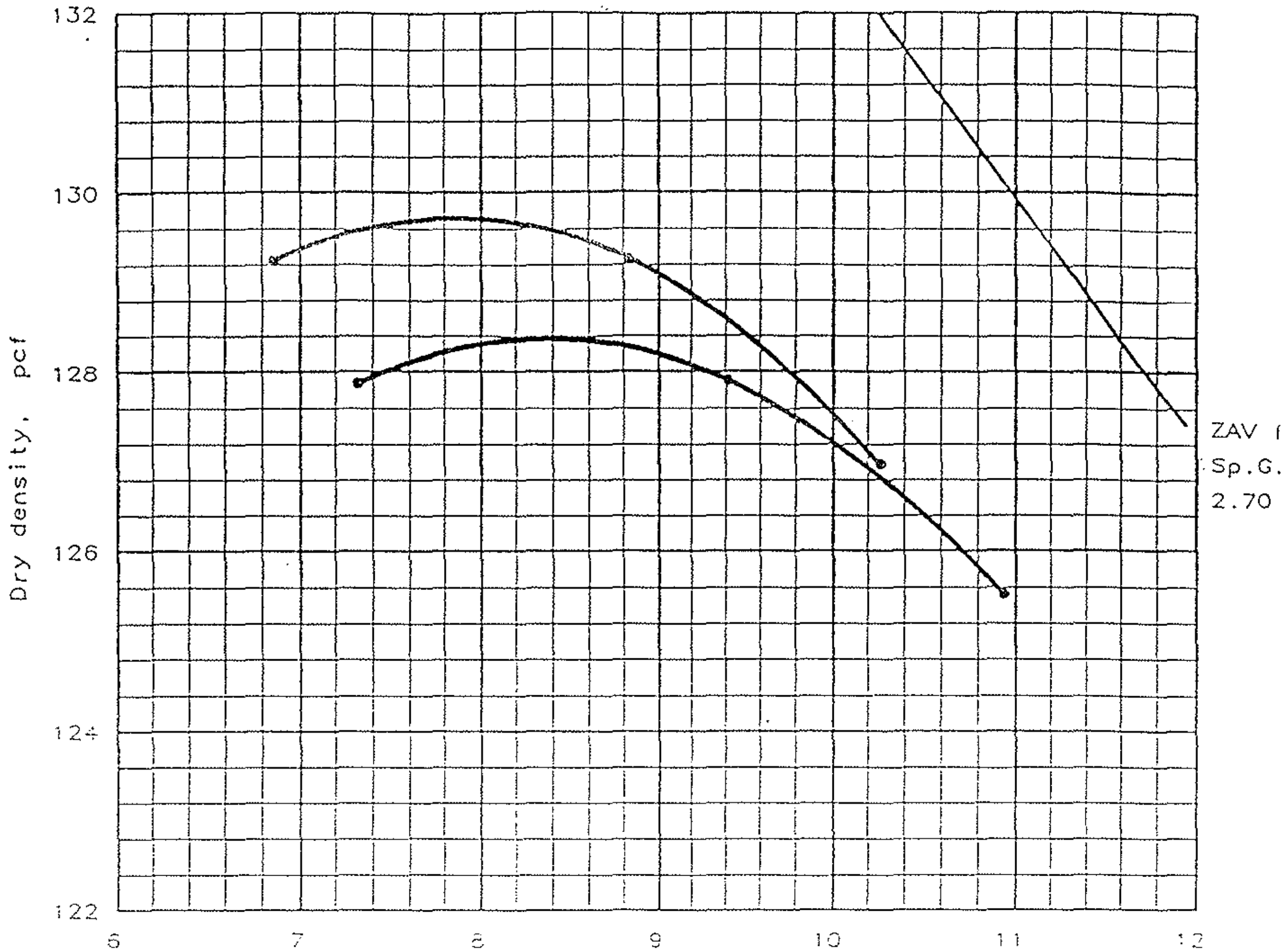
Date: 10-15-1999

MOISTURE-DENSITY RELATIONSHIP TEST

CONVERSE CONSULTANTS

Fig. No. A-58

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 1557-91 Procedure C, Modified

Oversize correction applied to each point

Elev/ Depth	Classification USCS	AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in	% < No.2
	SM				NV	NP	6.2 %	13.3

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 129.7 pcf Optimum moisture = 7.9 %	128.4 pcf 8.4 %	Silty Sand with Gravel

Project No.: 99-33437-01

Project: BMI Landfill

Location: B12 @ 10-15'

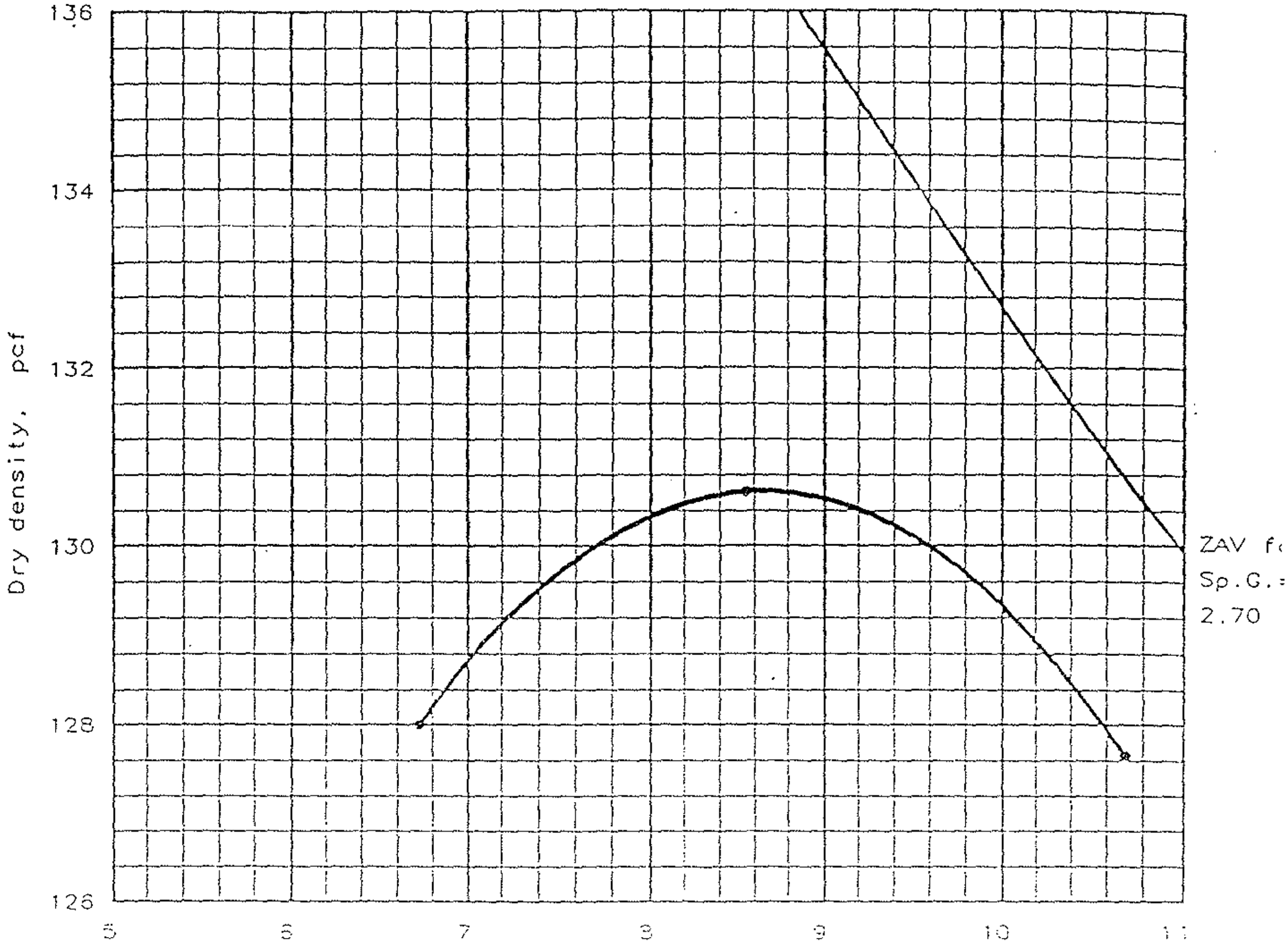
Date: 10-15-1999

MOISTURE-DENSITY RELATIONSHIP TEST

CONVERSE CONSULTANTS

Fig. No. A-59

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 1557-91 Procedure C, Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in	% < No.200
	USCS	AASHTO						
	SM				NV	NP	4.7 %	14.7 %

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 130.6 pcf Optimum moisture = 8.7 %	Silty Sand with Gravel

Project No.: 99-33437-01

Project: BMI Landfill

Location: B 101 @ 5-10'

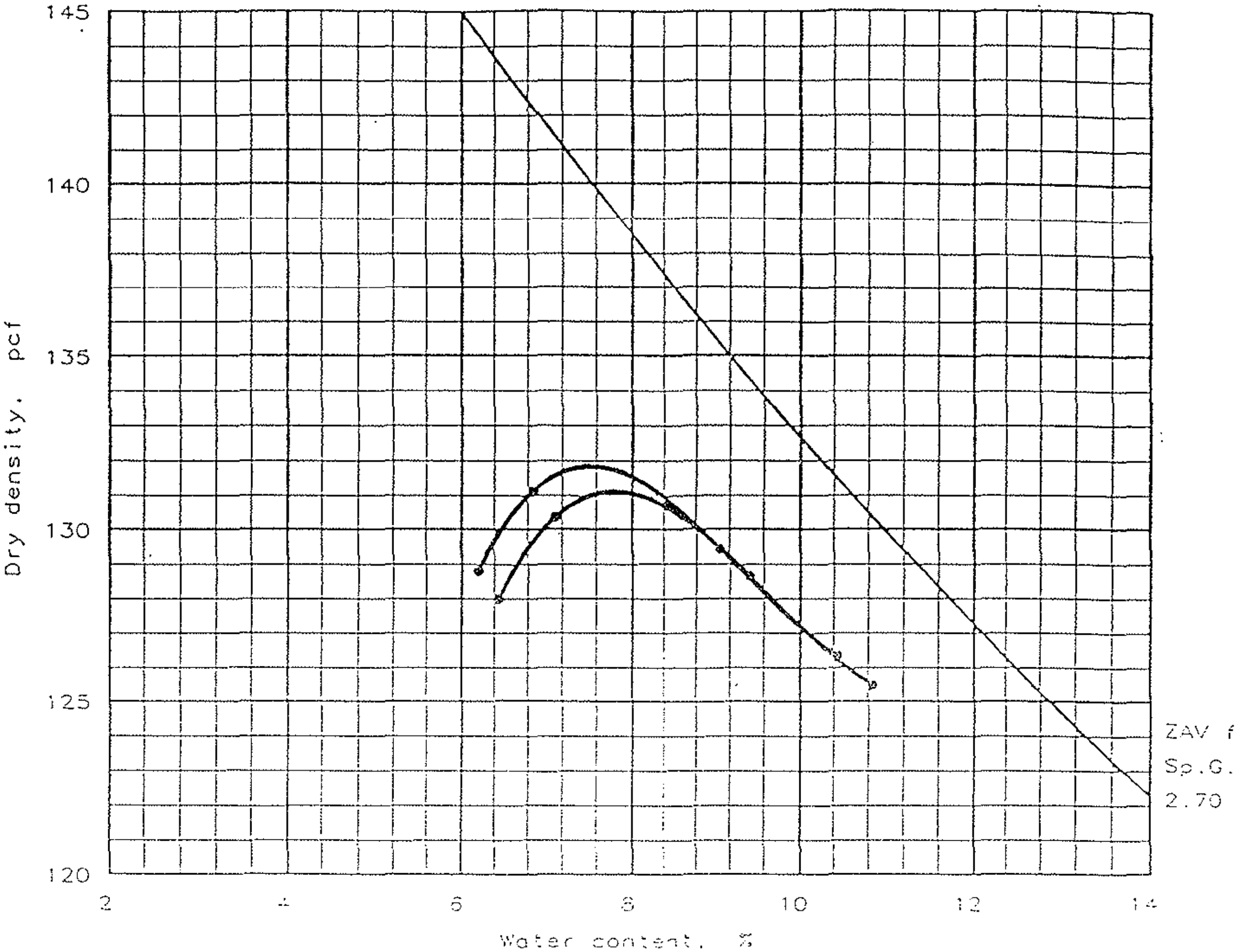
Date: 10-15-1999

MOISTURE-DENSITY RELATIONSHIP TEST

CONVERSE CONSULTANTS

Fig. No. A-60

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 1557-91 Procedure C, Modified

Oversize correction applied to each point

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in	% < No.2
	SW-SM				NV	NP	3.7 %	8.3

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 131.8 pcf Optimum moisture = 7.5 %	131.1 pcf 7.8 %	Well Graded Sand with Silt & Gravel

Project No.: 99-33437-01

Project: BMI Landfill

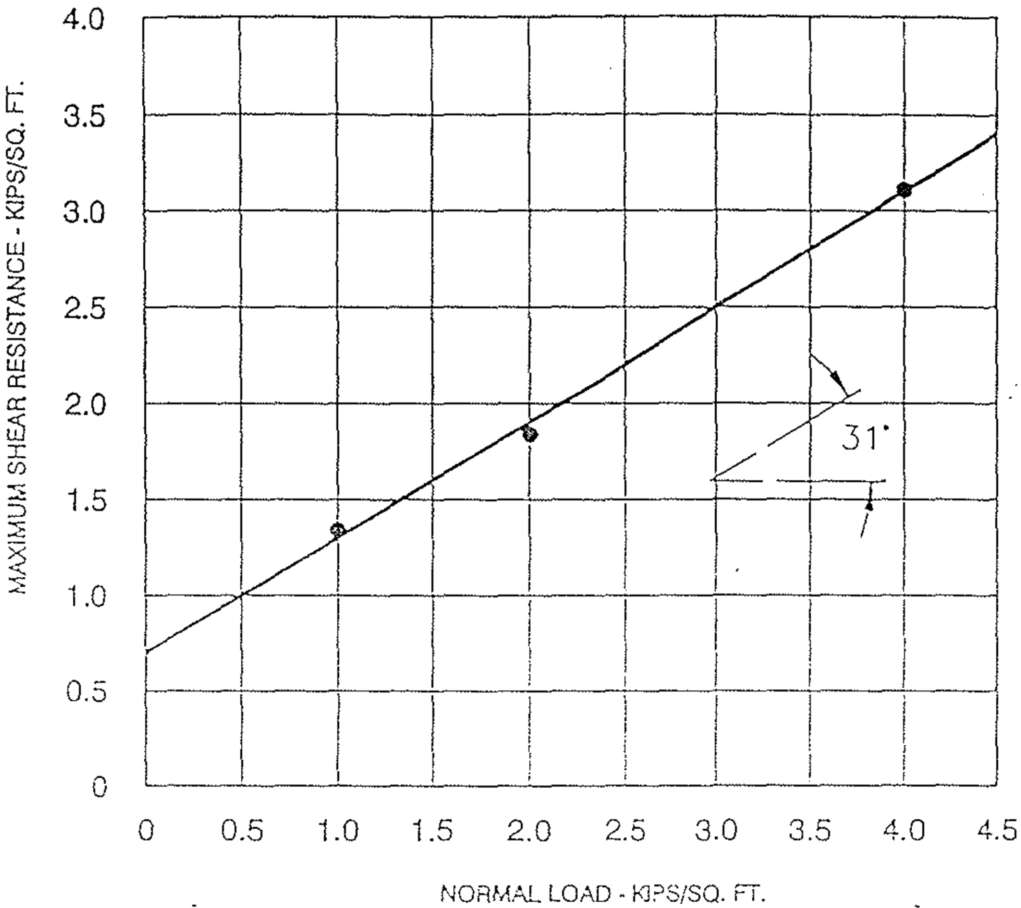
Location: B 105 @ 20-25'

Date: 10-15-1999

MOISTURE-DENSITY RELATIONSHIP TEST

CONVERSE CONSULTANTS

Fig. No. A-61



Exploration Location = B-4

Sample Depth = 14 - 14.5 FT

Angle of Shearing Resistance, Ø = 31°

Cohesion, c = 0.70 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

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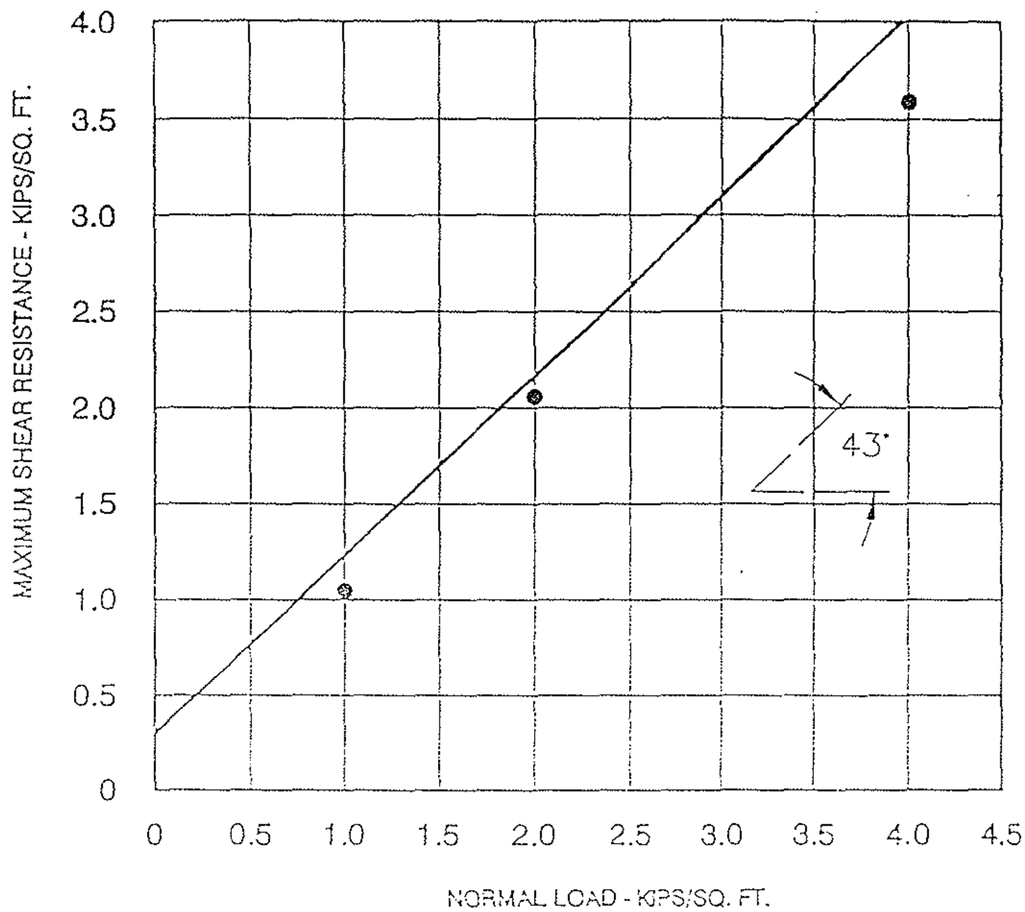
Over 50 Years of Dedication in Engineering and Environmental Sciences

Scale: N/A	File No. 4370136
Date: 10/20/99	Project No. 99-33437-01
Drafted By: CLE	Figure No.
Checked By: MKK	A-62
Approved By	



Exploration Location = B-5

Sample Depth = 14 - 15 FT

Angle of Shearing Resistance, Ø = 43°

Cohesion, c = 0.30 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

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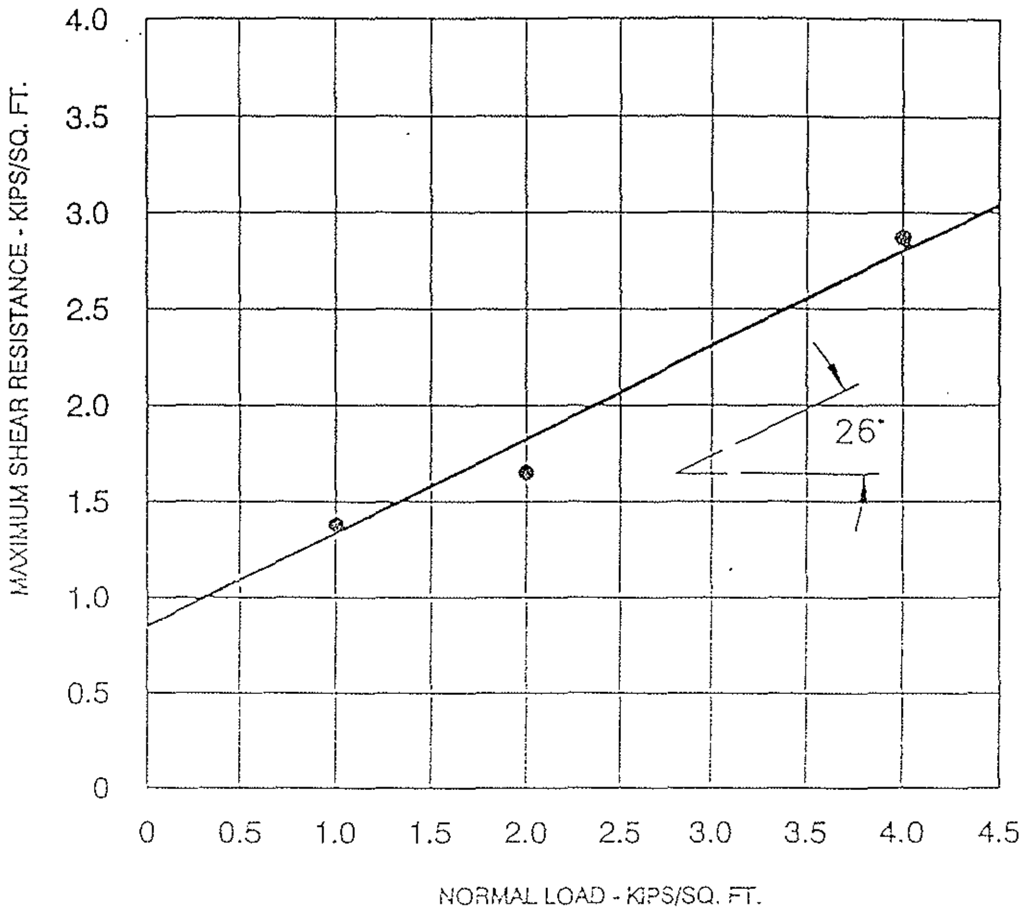
Scale N/A	File No. 43701363
Date 10/20/99	Project No. 99-33437-01
Drafted By GLE	Figure No.
Checked By MKK	A-63
Approved By	



Exploration Location = B-10

Sample Depth = 54 - 54.5 FT

Angle of Shearing Resistance, Ø = 26°

Cohesion, c = 0.85 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Scale N/A	File No. 4370135	
Date 10/20/99	Project No. 99-33437-0	
Drafted By GLE	Figure No.	
Checked By MKK	A-64	
Approved By		

CONVERSE CONSULTANTS

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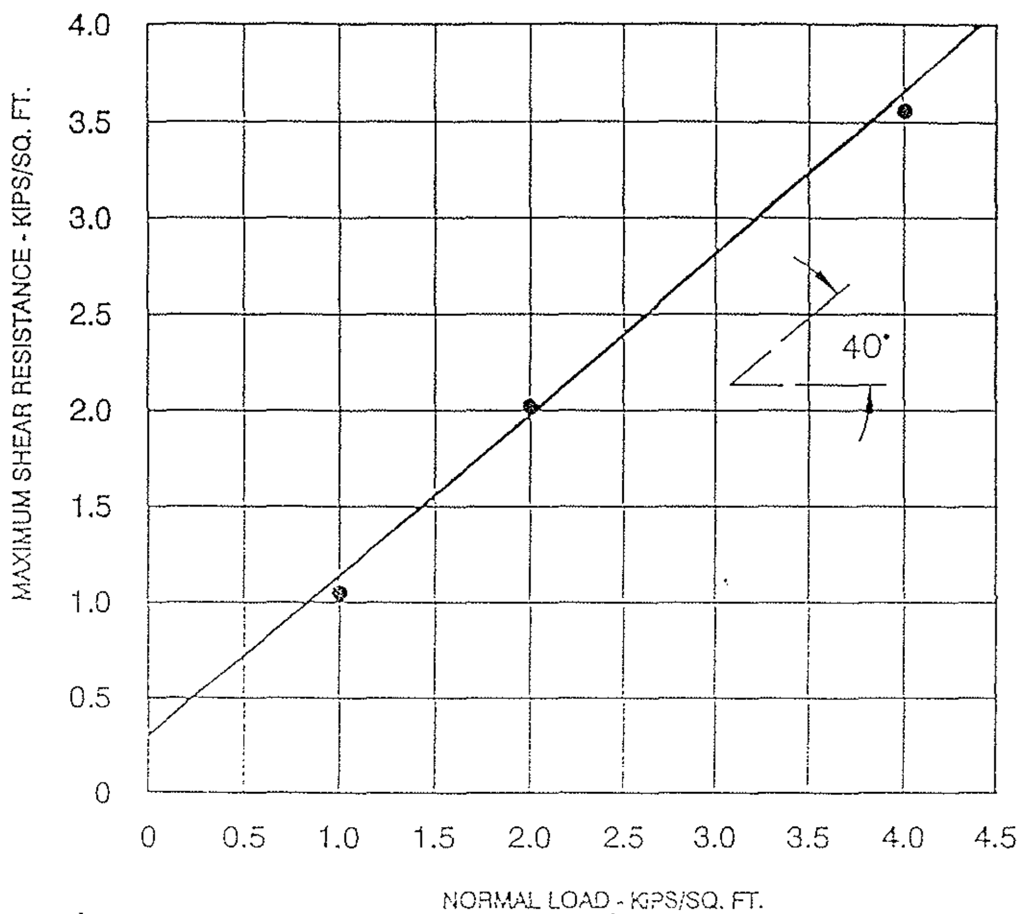

Exploration Location = B-12

Sample Depth = 14 - 15 FT

Angle of Shearing Resistance, Ø = 40°

Cohesion, c = 0.30 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Scale N/A	File No. 4370185	
Date 10/20/99	Project No. 99-33437-0	
Drafted By CLE	Figure No.	
Checked By MKK	A-65	
Approved By		

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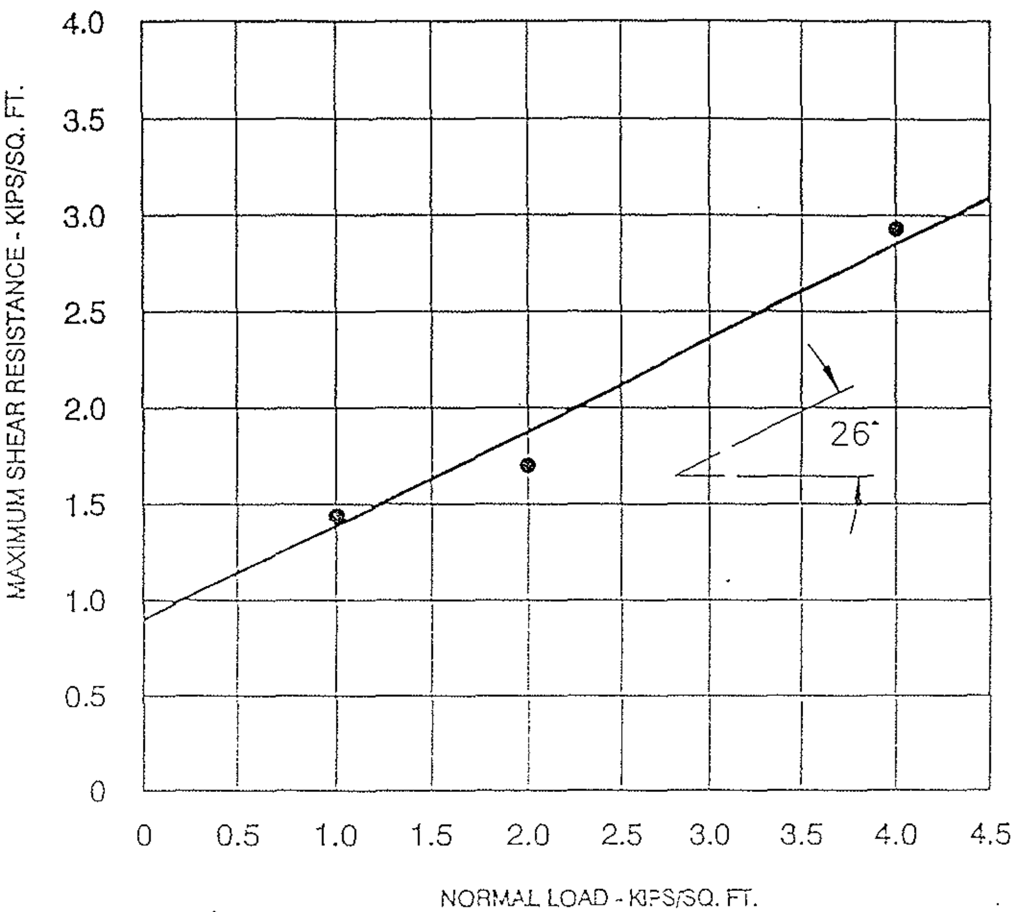

Exploration Location = B-101

Sample Depth = 39 - 40 FT

Angle of Shearing Resistance, Ø = 26°

Cohesion, c = 0.90 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Scale N/A	File No. 4370185:
Date 10/20/99	Project No. 99-33437-0
Drafted By GLE	Figure No.
Checked By MKK	A-66
Approved By	

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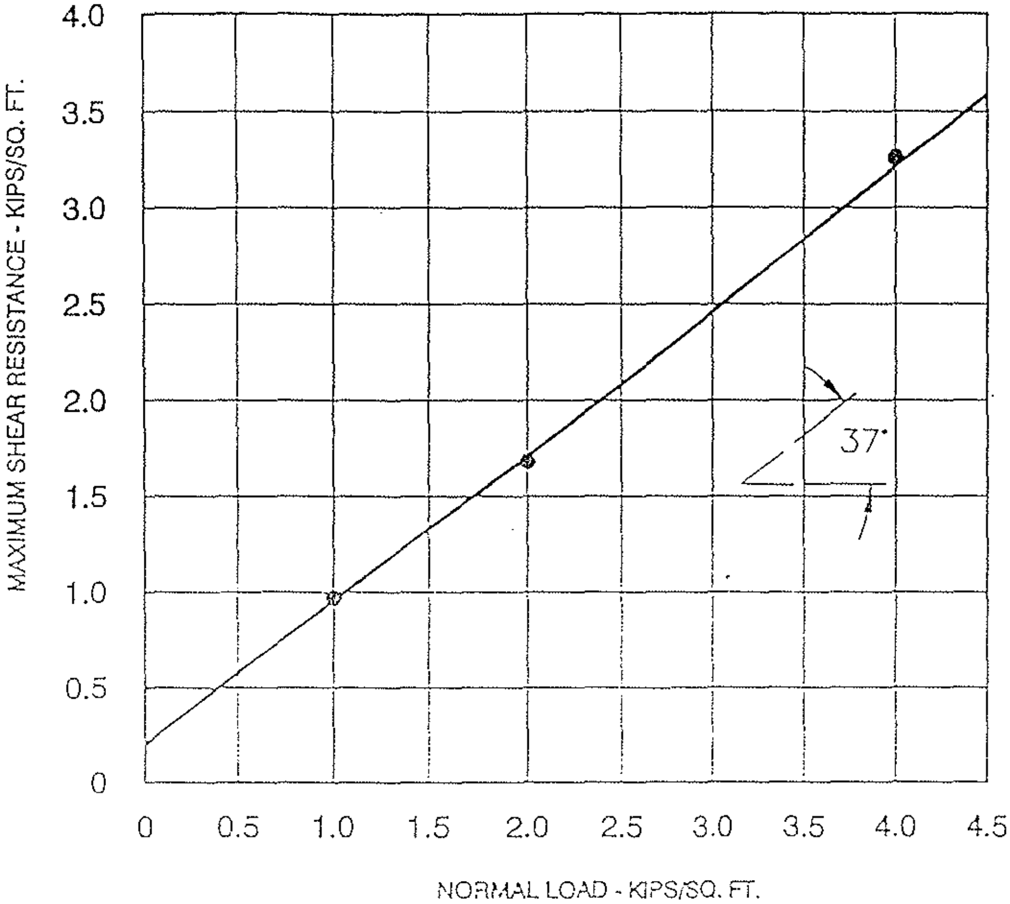

Exploration Location = B-102

Sample Depth = 20 - 25 FT

Angle of Shearing Resistance, Ø = 37°

Cohesion, c = 0.20 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

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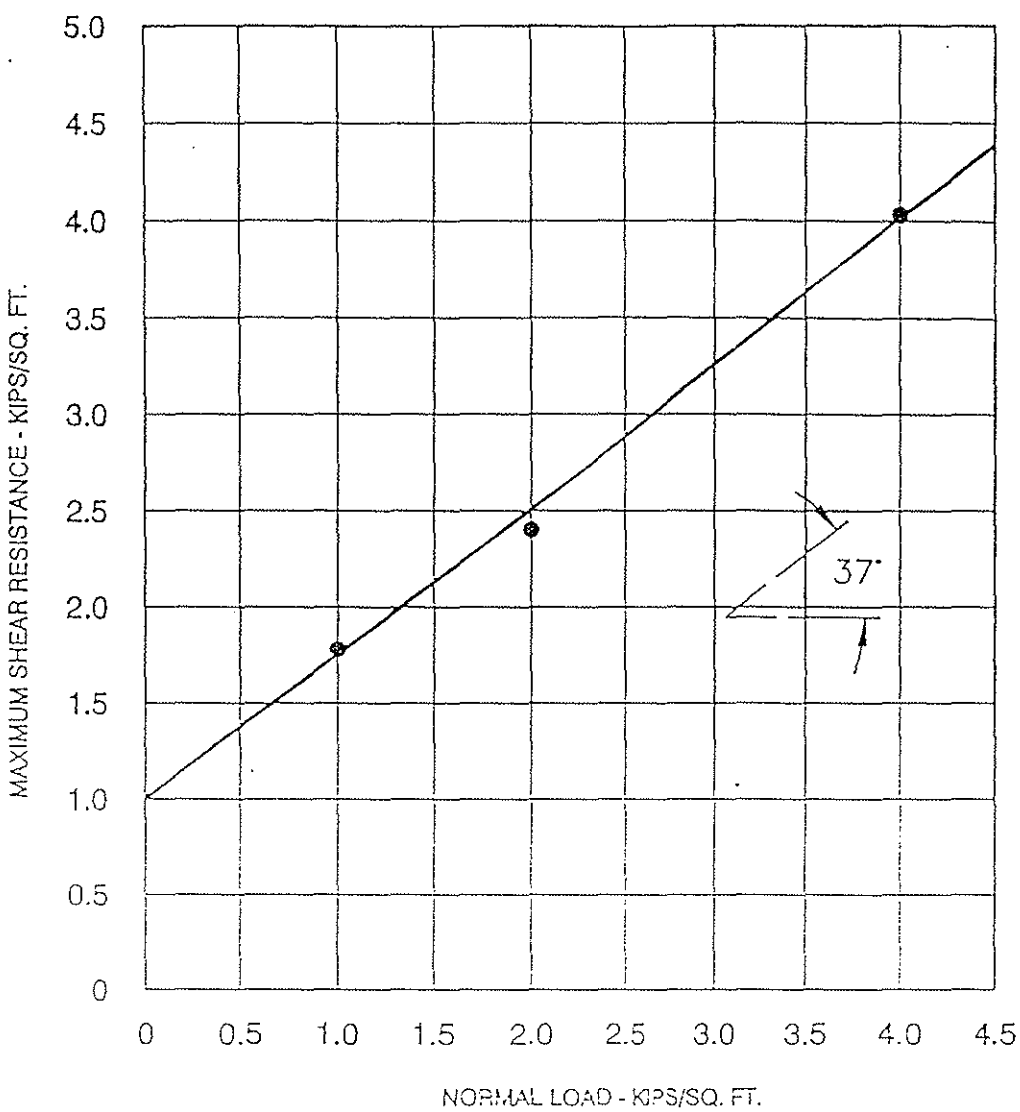
Scale N/A	File No. 4370186
Date 10/20/99	Project No. 99-33437-0
Drafted By GLE	Figure No.
Checked By MKK	A-67
Approved By	



Exploration Location = B-103

Sample Depth = 49 - 50 FT

Angle of Shearing Resistance, Ø = 37°

Cohesion, c = 1.00 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

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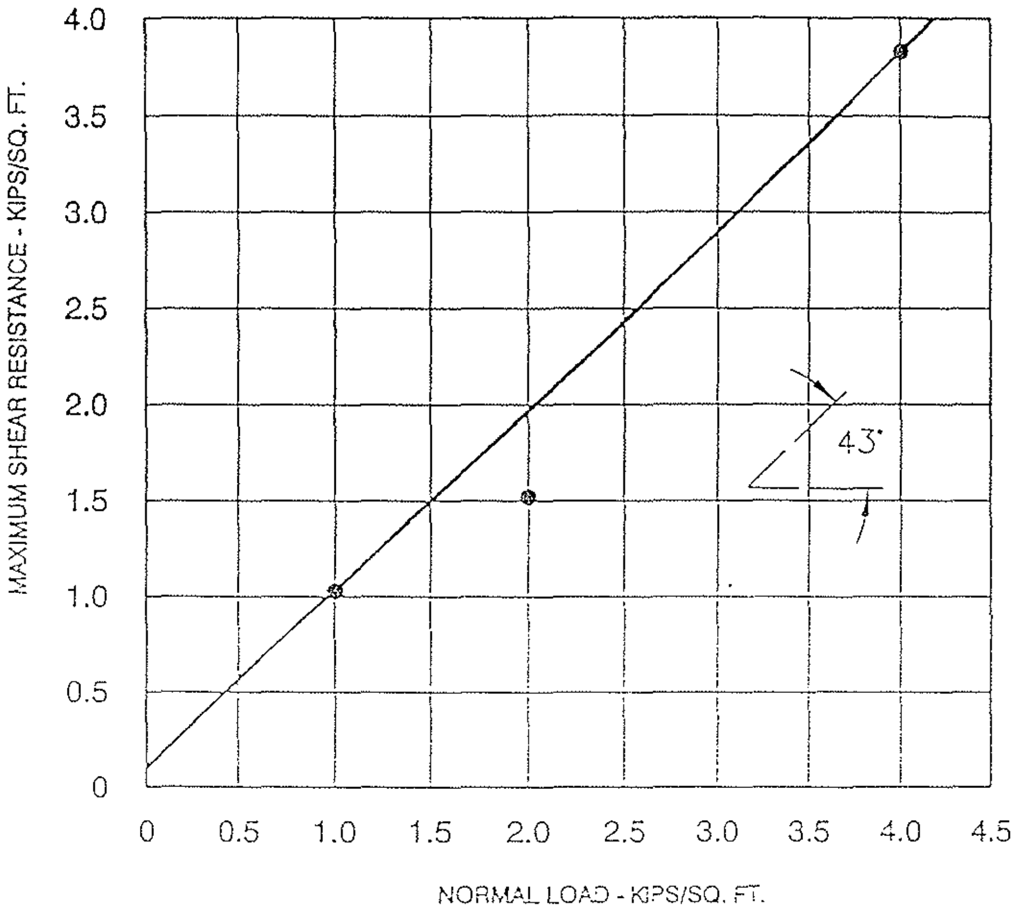
Scale	N/A	File No.	43701359
Date	10/20/99	Project No.	99-33437-01
Drafted By	GLE	Figure No.	A-68
Checked By	MKK		
Approved By			



Exploration Location = B-104

Sample Depth = 10 - 15 FT

Angle of Shearing Resistance, Ø = 43°

Cohesion, c = 0.10 KIPS/SQ. FT.

DIRECT SHEAR TEST RESULTS

PRELIMINARY GEOLOGIC AND GEOTECHNICAL INVESTIGATION
Basic Management Incorporated
Clark County, Nevada

Scale N/A	File No. 4370186:
Date 10/20/99	Project No. 99-33437-0
Drafted By GLE	Figure No.
Checked By MKK	A-69
Approved By	

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Atlas Chemical Testing Laboratories

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(702) 383-1199 • Fax (702) 383-4983

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TESTING MATERIALS

ACT LAB NO: 9218(b) DATE: October 13, 1999

PROJECT NO: 99-33437-01 P.O.: 18154

ANALYZED BY: Robert L. Summers LAB ID:

WATER SOLUBLE SALT ANALYSIS IN SOIL

1:5 (soil:water) Aqueous Extraction

AWWA 3500-Na D, ASTM D 516

BMI LANDFILL

Sample No.	Location	Depth (Feet)	Sodium (Percent)	Water Soluble Sulfate (SO_4) (Percent)	Total Available Water Soluble Sodium Sulfate(Na_2SC (Percent)
	B-5	10-15	0.07	0.13	0.20
	B-8	19-20	0.07	0.06	0.08
	B-101	5-10	0.17	0.06	0.08
	B-102	0-5	0.17	0.03	0.05
	B-106	0-5	0.15	0.08	0.12
	B-106.	29-30	0.15	0.06	0.08

A-7C

Notes: The results for each constituent denote the percentage of that analyte, at a 1:5 (soil:water) extraction ratio, which is present in the soil. Sodium was determined by flame photometry, sulfate turbidimetrically, and sodium sulfate by calculation.

AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

FLEXIBLE WALL HYDRAULIC CONDUCTIVITY TEST
ASTM D5084

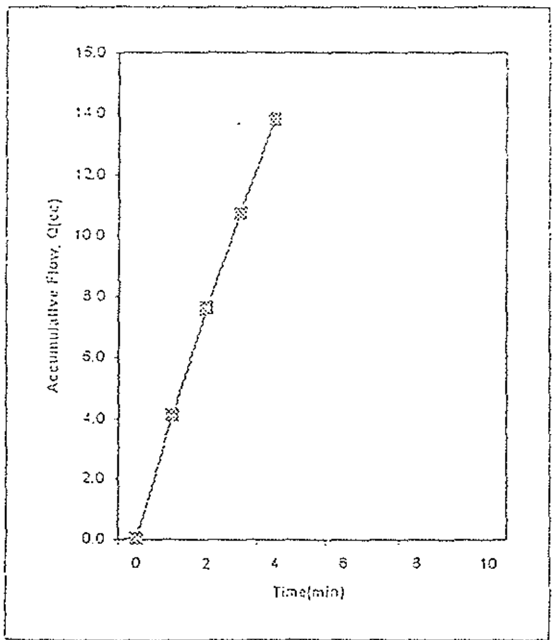
Project Name:	Preliminary Geologic Investigation	Tested by PS	Date	10/12/99
Project No.:	99-33437-01	Calculated by SY	Date	10/20/99
Boring No.:	B-1	Checked by AP	Date	10/20/99
Sample No.:	--- Depth: 14-15' feet			
Soil Description:	Olive Brown Silty Sand w/ gravel			
Test Condition:				
	Confining Pressure = 11 PSI			

INITIAL CONDITION OF SPECIMEN

Diameter (d)	2.42	in
Sample Area (A)	4.58	in^2
Length (L)	2.48	in
Weight Before	286.60	g
Wet Density	95.97	pcf
Dry Density	91.48	pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(gms)	340.95	361.62
Wt. Dry Soil+Container(gms)	327.35	316.16
Wt. Container (gms)	50.51	50.02
Moisture, (%)	4.91	17.07

TEST RESULTS



Time (min)	Flow Rdg (cm)	Burette Factor	Q (cc)	Head, h (psi)	h/L	Q/t (cc/s)
0	29.2	1	0.0	1.0	11.2	0
1	25.1	1	4.1	1.0	11.2	6.83E-02
2	21.6	1	7.6	1.0	11.2	5.83E-02
3	18.5	1	10.7	1.0	11.2	5.17E-02
4	15.4	1	13.8	1.0	11.2	5.17E-02

Hydraulic Conductivity (cm/sec): 1.57E-04

AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

FLEXIBLE WALL HYDRAULIC CONDUCTIVITY TEST
ASTM D5084

Project Name: Preliminary Geologic Investigation
Project No.: 99-33437-01
Boring No.: B-8
Sample No.: — Depth: 44-45' feet
Soil Description: Lt Olive Brown Sandy Silt
Test Condition:
Confining Pressure = 33 PSI

Tested by PS Date 10/12/99
Calculated by SY Date 10/20/99
Checked by AP Date 10/20/99

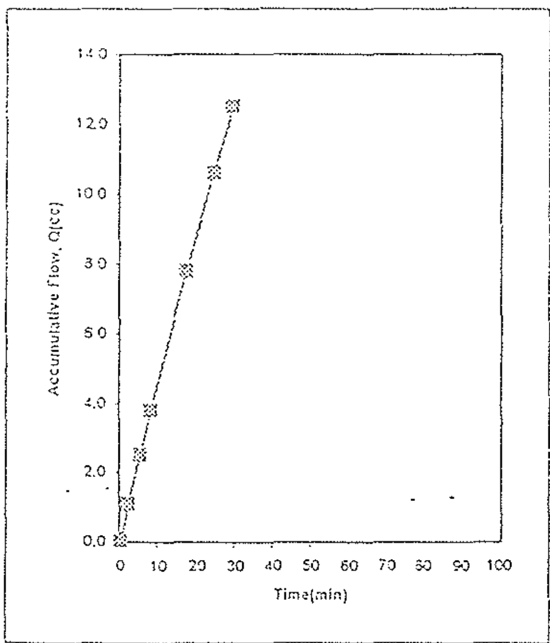
INITIAL CONDITION OF SPECIMEN

Diameter (d) 2.42 in
Sample Area (A) 4.58 in^2
Length (L) 3.48 in
Weight Before 457.60 g

Wet Density 109.24 pcf
Dry Density 85.69 pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(gms)	248.6	412.7
Wt. Dry Soil+Container(gms)	205.71	323.46
Wt. Container (gms)	49.67	53.72
Moisture, (%)	27.49	33.03

TEST RESULTS



Time (min)	Flow Rdg (cm)	Burette Factor	Q (cc)	Head, h (psi)	h/L	Q/t (cc/s)
0	27.2	1	0.0	1.0	7.95	0
2	26.1	1	1.1	1.0	7.95	9.17E-03
5	24.7	1	2.5	1.0	7.95	7.78E-03
8	23.4	1	3.8	1.0	7.95	7.22E-03
17	19.4	1	7.8	1.0	7.95	7.41E-03
24	16.6	1	10.6	1.0	7.95	6.67E-03
29	14.7	1	12.5	1.0	7.95	6.33E-03
34	12.9	1	14.3	1.0	7.95	6.00E-03

Hydraulic Conductivity (cm/sec): 2.90E-05

AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

FLEXIBLE WALL HYDRAULIC CONDUCTIVITY TEST
ASTM D5084

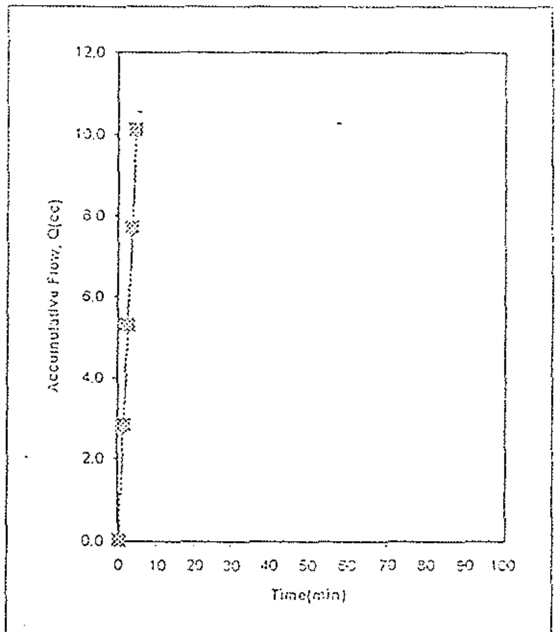
Project Name:	Preliminary Geologic Investigation	Tested by PS	Date 10/12/99
Project No.:	99-33437-01	Calculated by SY	Date 10/20/99
Boring No.:	B-4	Checked by AP	Date 10/20/99
Sample No.:	— Depth: 24-25 feet		
Soil Description:	Lt Olive Brown Silty Sand w/ gravel		
Test Condition:	Confining Pressure = 18 PSI		

INITIAL CONDITION OF SPECIMEN

Diameter (d)	2.42	in
Sample Area (A)	4.58	in²
Length (L)	3.00	in
Weight Before	437.24	g
Wet Density	121.20	pcf
Dry Density	114.78	pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(gms)	308.84	421.32
Wt. Dry Soil+Container(gms)	295.15	372.45
Wt. Container (gms)	50.05	49.46
Moisture, (%)	5.59	15.13

TEST RESULTS



Time (min)	Flow Rdg (cm)	Burette Factor	Q (cc)	Head, h (psi)	h/L	Q/t (cc/s)
0	32.6	1	0.0	1.0	9.23	0
1	29.8	1	2.8	1.0	9.23	4.67E-02
2	27.3	1	5.3	1.0	9.23	4.17E-02
3	24.9	1	7.7	1.0	9.23	4.00E-02
4	22.5	1	10.1	1.0	9.23	4.00E-02

Hydraulic Conductivity (cm/sec): 1.47E-04

AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

FLEXIBLE WALL HYDRAULIC CONDUCTIVITY TEST
ASTM D5084

Project Name: Preliminary Geologic Investigation
Project No.: 99-33437-01
Boring No.: B-12
Sample No.: — Depth: 39-39.5 feet
Soil Description: Yell Brown Silty Clay w/ siltstone
Test Condition:
Confining Pressure = 29 PSI

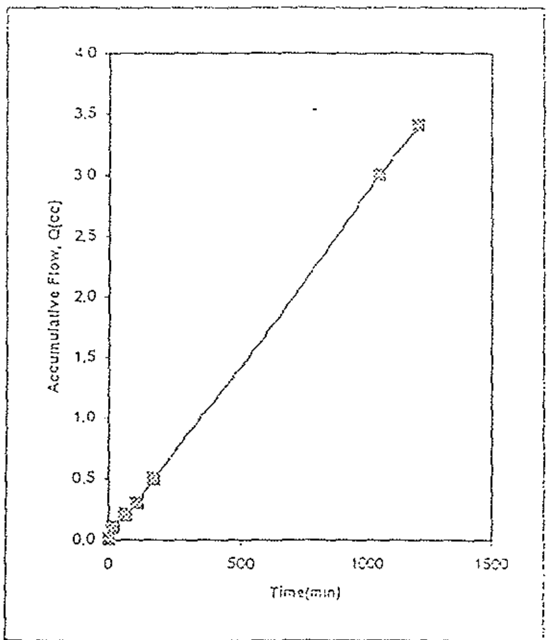
Tested by PS Date 10/14/99
Calculated by SY Date 10/20/99
Checked by AP Date 10/20/99

INITIAL CONDITION OF SPECIMEN

Diameter (d)	2.42	in
Sample Area (A)	4.58	in²
Length (L)	3.00	in
Weight Before	410.32	g
Wet Density	113.73	pcf
Dry Density	86.05	pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(gms)	173.15	470.17
Wt. Dry Soil+Container(gms)	137.46	363.54
Wt. Container (gms)	26.53	50.02
Moisture, (%)	32.17	34.01

TEST RESULTS



Time (min)	Flow Rdg (cm)	Burette Factor	Q (cc)	Head, h (psi)	h/L	Q/t (cc/s)
0	22.3	1	0.0	1.0	9.23	0
19	22.2	1	0.1	1.0	9.23	8.77E-05
63	22.1	1	0.2	1.0	9.23	3.79E-05
109	22.0	1	0.3	1.0	9.23	3.62E-05
171	21.8	1	0.5	1.0	9.23	5.38E-05
1052	19.3	1	3.0	1.0	9.23	4.73E-05
1207	18.9	1	3.4	1.0	9.23	4.30E-05

Hydraulic Conductivity (cm/sec): 1.76E-07

AP Engineering and Testing, Inc.

GEOTECHNICAL TESTING LABORATORY

FLEXIBLE WALL HYDRAULIC CONDUCTIVITY TEST
ASTM D5084

Project Name: Preliminary Geologic Investigation
Project No.: 99-33437-01
Boring No.: B-105
Sample No.: --- Depth: 30-35 feet
Soil Description: Grayish Brown Silty Sand
Test Condition: Remolded to 85 % Relative Compaction @ Opt
Confining Pressure = 23 PSI

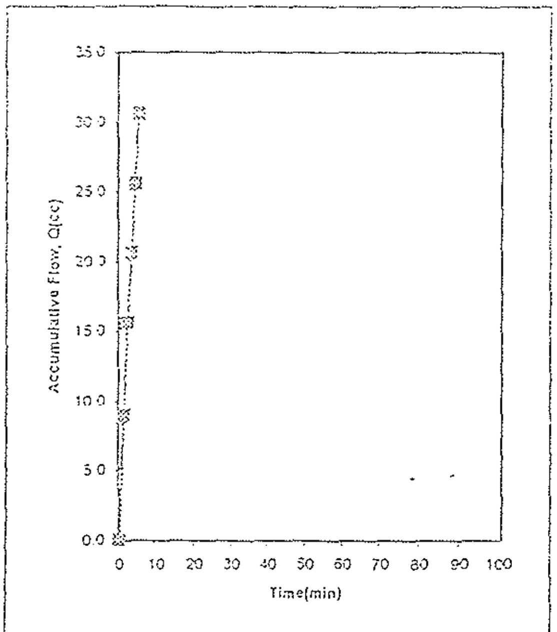
Tested by PS Date 10/14/99
Calculated by SY Date 10/20/99
Checked by AP Date 10/20/99

INITIAL CONDITION OF SPECIMEN

Diameter (d)	2.42	in
Sample Area (A)	4.58	in^2
Length (L)	3.00	in
Weight Before	432.56	g
Wet Density	119.90	pcf
Dry Density	112.01	pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(gms)	309.87	317.64
Wt. Dry Soil+Container(gms)	300.27	282.06
Wt. Container (gms)	164.05	49.93
Moisture, (%)	7.05	15.32

TEST RESULTS



Time (min)	Flow Rdg (cm)	Burette Factor	Q (cc)	Head, h (psi)	h/L	Q/t (cc/s)
0	45.3	1	0.0	1.0	9.23	0
1	36.4	1	8.9	1.0	9.23	1.48E-01
2	29.7	1	15.6	1.0	9.23	1.12E-01
3	24.7	1	20.6	1.0	9.23	8.33E-02
4	19.7	1	25.6	1.0	9.23	8.33E-02
5	14.7	1	30.6	1.0	9.23	8.33E-02

Hydraulic Conductivity (cm/sec): 3.05E-04

Log of Boring No. BRC-BS-01
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/15/05
Date Completed: 01/15/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface. (Odors not being observed due to potential chemical hazards.)	
							SP: @ 3', debris present, grey discolored soil and rubber cord with belt pieces.	
	Grab				0.1		SP: Poorly graded sand, dark greyish brown (10YR4/2), loose to medium dense, moist, fine to coarse subrounded sand and fine gravel, noncemented, no debris. Approximately trace gravel, 5% coarse sand, 85% medium sand and 10% fine sand.	
-10					0.6		Poorly graded sand, light brownish grey (10YR6/2), very dense, moist, fine to coarse, subangular to subrounded sand and fine gravel with increasing volcanics, noncemented silt. Approximately 5% gravel, 5% coarse sand, 65% medium sand, 25% fine sand and trace silt.	
	Grab				1.5		Poorly graded sand, brown (10YR5/3), loose to medium dense, moist, fine to coarse subangular to subrounded snad and fine gravel, no volcanics, noncemented, trace silt. Approximately 5% gravel, 15% coarse sand, 60% medium sand, 20% fine sand, trace silt.	
-20					1.4		Poorly graded sand, brown (10YR5/3), very dense, moist, moist, fine to coarse subangular volcanic sand and fine gravel, noncemented, trace silt. Approximately 5% gravel, 15% coarse sand, 60% medium sand, 20% fine sand, and trace silt.	
	Grab				0.5		Poorly graded sand, brown (10YR5/3) loose to medium dense, moist, fine to coarse subangular to subrounded sand and fine gravel, few volcanics, noncemented, trace silt. Approximately 5% gravel, 15% coarse sand, 60% medium sand, 20% fine sand and trace silt.	
-30					1.9		Poorly graded sand, brown (10YR5/3), very dense, moist, fine to coarse subangular to subrounded volcanic sand, and fine gravel, noncemented, slight solvent odor. Approximately 5% gravel, 15% coarse sand, 60% medium sand, 20% fine sand, and trace silt. Terminated borehole at 32' bgs.	

Log of Boring No. BRC-BS-02
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/14/05
Date Completed: 01/14/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	PID	Soil Description	Well Construction
0			SP: Soil Surface, soil boring located in area of subsidence. (Odors not being observed due to potential chemical hazards.)	
	Grab	0.2	Poorly graded sand, yellowish brown (10YR5/4), loose, moist, fine to coarse, subrounded to rounded sand and fine gravel, noncemented, no debris. Approximately trace gravel, 10% coarse sand, 70% medium sand, 20% fine sand and trace silt.	
-10		0.0	Poorly graded sand, dark yellowish brown (10YR4/4), medium dense, moist, fine to coarse subrounded to rounded sand, and fine gravel, noncemented. Approximately trace gravel, 10% coarse sand, 70% medium sand, 20% fine sand and trace silt.	
	Grab	2.2	SP: Poorly graded sand, brown (10YR5/4), loose to medium dense, moist, fine to coarse subrounded sand and fine gravel, noncemented, minor debris (small paper shreds). Approximately trace gravel, 5% coarse sand, 75% medium sand and 20% fine sand and trace silt.	
			SP: Debris - wood, fabric, plastic, paper bags, white powder, and wet white chalky substance, strong solvent odor.	
-20		3.2	SP: Poorly graded sand, yellowish brown (10YR5/4), dense, moist, fine to coarse subrounded to subangular sand and fine gravel, noncemented, no debris.	
			@ 23' drillers note change in drilling - hard soil.	
	Grab	0.2	Poorly graded sand, brown (10YR5/3), moist, fine to coarse, angular to subrounded sand and fine gravel (includes volcanics), noncemented, no debris, trace silt. Approximately trace gravel, 5% coarse, 65% medium sand, 65% medium sand, 25% fine and 5% silt.	
-30		1.8	Poorly graded sand, brown (10YR5/3), very dense, moist, fine to coarse subanguar sand and fine gravel (including volcanics), noncemented, no debris, trace silt. Approximately trace gravel, trace coarse sand, 50% medium sand, 45% fine sand and 5% silt. Terminated borehole at 32' bgs.	

Log of Boring No. BRC-BS-03
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/12/05
Date Completed: 01/12/05

Monitoring Well Construction

Type of Surface Seal:	N/A	Screen Slot Size:	N/A
Blank Casing Type/Size:	N/A	Top of Screen (ft. bgs):	N/A
Screen Type/Size:	N/A	Bottom of Screen (ft. bgs):	N/A
Transition Sand Type:	N/A	Type of Sand Pack:	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0						GP	GP: Soil surface; boring located within area of slight subsidence. (Odors not being observed due to potential chemical hazards.)	
						SP	SP: Poorly graded gravel with sand, yellowish brown (10YR5/4), loose, moist, medium to coarse subrounded sand and fine subrounded (pea) gravel, trace fine snad, noncemented. Approximately 60% gravel, 20% coarse sand, 20% medium sand and trace fine sand.	
	Grab				0.0			
-10					2.6	SP	SP: Poorly graded sand, yellowish brown (10YR5/4), medium dense, moist, fine to medium subrounded to angular sand and fine subangular gravel, noncemented, debris includes plastic, fabric, concrete chunks, and visqueen. Apporximately 10% gravel, 5% coarse sand, 65% medium sand and 20% fine sand.	
	No Soil				1.0	Debris/Cobbles	DEBRIS/COBBLES: @15' cuttings consist mostly of debris - clothing mostly and plastic parts (one reads "janitorial toilet bowl cleaner"), some paper/cardboard, with cobbles up to 2.5" diameter.	
-20					1.5		@19' cannot pound sample - blocked - no recovery. @20' cannot pound sample - no recovery. Drill to 21' and try again. Cuttings consist of debris (paper/cardboard) with some cobbles/coarse gravel.	
-30					1.7	GP	GP: @27' drillers noted that soil way more compacted - likely out of fill. Poorly graded gravel with sand, light yellowish brown (10YR6/4), very dense, moist, fine to coarse subangular sand and fine subangular gravel, trace silt, no debris. Approximately 60% gravel, 15% coarse sand, 15% medium sand, 10% fine sand and trace silt. Terminated borehole at 32' bgs.	

Log of Boring No. BRC-BS-04

CAMU

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/17/05
Date Completed: 01/17/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface; soil boring located within subsidence cracks. (Odors not being observed due to potential chemical hazards.)	
	Grab				0.4		Poorly raded sand, dark yellowish brown (10YR4/4), loose to medium dense, moist, fine to coarse subrounded to rounded sand and fine gravel - not volcanic, noncemented, no debris. Approximately trace gravel, 20% coarse sand, 65% medium sand and 15% fine sand.	
-10					0.5		Poorly graded sand, dark yellowish brown (10YR4/4), dense, moist, fine to coarse subrounded to rounded sand and fine gravel - not volcanic, noncemented. Debris begins at 11.5', consists of paper bags and white powder, and wood splinters. Approximately 5% gravel, 20% coarse sand, 60% medium sand and 15% fine sand.	
							SP: Poorly graded sand, brown (10YR4/3), medium dense to dense, moist, fine to coarse subrounded to rounded sand and fine gravel - no volcanics, noncemented, paper and plastic debris, wood splinters. Approximately 5% gravel, 20% coarse sand, 60% medium sand and 15% fine sand.	
	Grab				11.3			
-20					3.6			
							SP: Poorly graded sand, brown (10YR4/3), very dense, moist, fine to coarse angular to subangular volcanic sand and fine gravel, noncemented, no debris. Drillers note change in hardness @ 21'. Cuttings appear to be from 15' - 20' interval - have plastic lid (~4' diameter) debris and trace metal straps. Approximately trace gravel, 10% coarse, 65% medium sand and 25% fine sand.	
	Grab				12.0			
-30							Poorly graded sand, brown (10YR4/3), very dense, moist, fine to coarse angular to subangular volcanic sand and fine gravel, noncemented, no debris. Approximately trace gravel, 10% coarse sand, 65% medium sand and 25% fine sand. Terminated borehole at 31' bgs.	

Log of Boring No. BRC-BS-05
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/19/05
Date Completed: 01/19/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface, boring located within area of subsidence. (Odors not being observed due to potential chemical hazards.) Debris begins @ 5'	
	Grab				0.7		SP: Poorly graded sand, yellowish brown (10YR5/4), loose, moist, fine to coarse sand and fine grained - subrounded to rounded - no volcanics, noncemented. Debris includes wood, metal, cobbles, and plastic. Approximately trace gravel, trace coarse sand, 70% medium sand and 30% fine sand.	
-10					4.3		Poorly graded sand, yellowish brown (10YR5/4), dense, moist, fine to coarse subrounded to rounded sand and fine gravel - no volcanics, noncemented, debris includes mostly wood, some plastic, metal, and white chalky material. Approximately 5% gravel, 10% coarse sand, 75% medium sand and 10% fine sand.	
	Grab				21.0		Poorly graded sand, brown (10YR4/3), loose to dense, moist, fine to coarse subangular to subrounded sand and fine gravel, no volcanics, noncemented, little debris, paper (small shreds). Approximately trace gravel, 5% coarse sand, 65% medium sand and 30% fine sand.	
-20					19.0		Poorly graded sand, dark grayish brown (10YR4/2), very dense, moist, fine to coarse subangular to subrounded sand and fine gravel, no volcanics, noncemented, debris includes wood, plastic lined paper bags with white chalky material, metal and newspaper. Approximately trace gravel, 5% coarse sand, 65% medium sand and 30% fine sand.	
	Grab				5.2		SP: Poorly graded sand, yellowish brown (10YR5/4), loose to dense, moist, fine to coarse angular to rounded sand and fine gravel - some volcanics, noncemented, no debris, trace silt. Approximately trace gravel, 5% coarse sand, 60% medium sand, 35% fine sand and trace silt.	
-30					0.4		Poorly graded sand, yellowish brown (10YR5/4), very dense, moist, fine to coarse angular to subangular sand and fine gravel - volcanic, noncemented, no debris. Approximately trace gravel, 5% coarse sand, 60% medium sand, 35% fine sand and trace silt. Terminated borehole at 32' bgs.	

Log of Boring No. BRC-BS-06
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous
Logged By: J. Wiley
Date Started: 01/14/05
Date Completed: 01/14/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface. (Odors not being observed due to potential chemical hazards.) Poorly graded sand, brown (10YR4/3), loose, moist, fine to coarse subrounded to rounded sand and fine grained, noncemented. Approximately 5% gravel, 10% coarse sand, 65% medium sand and 20% fine sand.	
	Grab				30.6		SP: Begin debris at 3' bgs. wood, solvent odor. Poorly graded sand, brown (10YR4/3) loose to medium dense, moist, fine to coarse subrounded to rounded sand and fine gravel, noncemented, solvent odor, debris includes wood. Approximately trace gravel, 10% coarse sand 70% medium sand and 20% fine sand.	
-10					4.0		Poorly graded sand, brown (10YR4/3), dense, moist, fine to coarse subangular to subrounded sand, and trace fine gravel, noncemented, strong solvent odor, very little wood debris. Approximately trace gravel, 5% coarse sand, 75% medium sand and 20% fine sand.	
							@12' increased debris: wood, fabric, plastic, solvent odor.	
	Grab				15.0		Poorly graded sand, dark yellowish brown (10YR4/4), loose to medium dense, fine to coarse subrounded sand and fine gravel, noncemented, debris includes fabric with plastic and wood, solvent odor.	
-20					1.0		Poorly graded sand, dark yellowish brown (10YR4/4), dense, fine to coarse, subangular sand and fine gravel, noncemented, solvent odor. Approximately 5% gravel, 5% coarse sand, 50% medium sand, 40% fine sand and trace silt.	
	Grab				35.1		SP: Poorly graded sand, dark yellowish brown (10YR4/4), loose to medium dense, fine to coarse subangular sand and fine gravel, noncemented, solvent odor, no debris. Approximately 10% gravel, 5% coarse sand, 55% medium sand, 30% fine sand and trace silt.	
-30					0.8		Poorly graded sand, brown (10YR5/3), very dense, moist, fine to coarse volcanic sand (angular to subangular) and fine gravel, noncemented, no debris. Approximately trace gravel, 5% coarse snad 55% medium sand and 40% fine sand.	
							Terminated borehole at 32' bgs.	

Log of Boring No. BRC-BS-07

CAMU

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

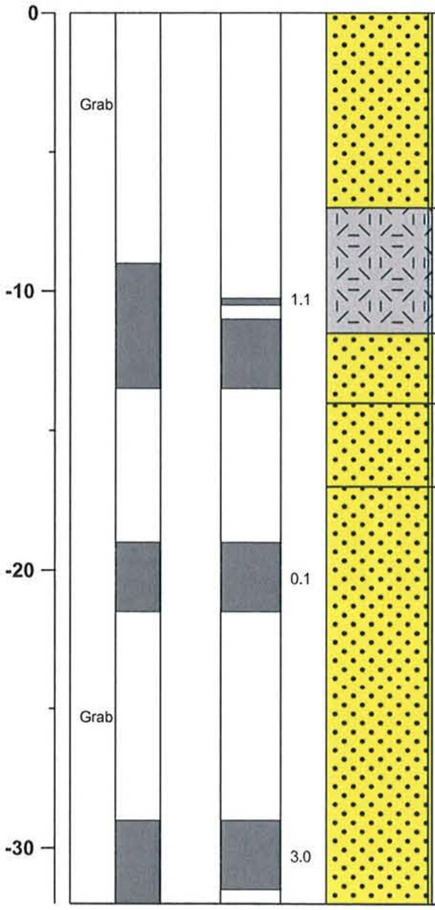
Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/11/05
Date Completed: 01/11/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction



SP: Soil surface; boring located adjacent to deep (3') area of subsidence, and is in line of subsidence areas. (Odors not being observed due to potential chemical hazards.) Poorly graded sand with gravel, yellowish brown (10YR5/4), loose, moist, fine to coarse subrounded to angular sand and fine angular gravel (fill), noncemented. Approximately 15% gravel, 10% coarse sand, 55% medium sand, 20% fine sand and trace silt.

DEBRIS: Debris-fabric (black) and plastic chunks (white to grey). Poor recovery.

SP: Poorly graded sand with gravel, pale brown (10YR6/3), moist, medium dense, fine to coarse subrounded to angular sand and fine subangular gravel, noncemented. Approximately 15% gravel, 20% coarse sand, 50% medium sand, 15% fine sand and trace silt.

SP: No recovery from cuttings due to fabric material caught in auger - likely debris in this interval.

SP: Poorly graded sand with gravel, pale brown (10YR6/3), moist, very dense, fine to coarse subrounded to angular sand and fine subangular gravel, noncemented, trace cobbles, has black fabric fragments. Approximately 15% gravel, 10% coarse sand, 45% medium sand, 30% fine sand and trace silt.

Poorly graded sand with gravel, brown (10YR5/3), moist, fine to coarse subrounded to subangular sand and subangular gravel, noncemented. Approximately 15% gravel, 5% coarse sand, 40% medium sand, 40% fine sand and trace silt.

Poorly graded sand with gravel, brown (10YR5/3), very dense, moist, fine to coarse subrounded to subangular sand and fine subangular gravel, noncemented, no fabric observed, drillers observed harder drilling after 25'. Approximately 15% gravel, 5% coarse sand, 40% medium sand, 40% fine sand and trace silt.

Terminated borehole at 32' bgs.

Log of Boring No. BRC-BS-08
CAMU
Henderson, Nevada

Basic Remediation Company

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/18/05
Date Completed: 01/18/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	PID	Soil Description	Well Construction
0			SP: Soil surface, boring located in slight depression area. (Odors not being observed due to potential chemical hazards.)	
	Grab	-6.5	Poorly graded sand, dark yellowish brown (10YR4/4), loose to dense, fine to coarse subrounded to rounded sand and fine gravel - no volcanics, trace silt, noncemented. Approximately 5% gravel, 5% coarse sand, 75% medium sand, 15% fine sand and trace silt.	
-10		16.2	Poorly graded sand, brown (10YR4/3), dense, moist, fine to coarse angular to subangular volcanic sand and fine gravel plus nonvolcanics, noncemented, solvent odor, grains have slight coating (dark brown), oily.	
	Grab	57.2	Poorly graded sand, very dark grayish brown, dense, fine to coarse angular to subangular volcanic sand and fine gravel, noncemented, oily coating (dark brown), solvent odor. Approximately trace gravel, 10% coarse sand, 70% medium sand and 20% fine sand.	
-20		1361	SP: Poorly graded sand, brown (10YR4/3), dense, moist, fine to coarse, subangular to subrounded sand and fine gravel - some volcanic. Thick oily residue covers grains. Has sheen when washed, strong solvent odor, weakly cemented from residues. Fill begins at ~19', consisting of wood fragments and white powder and paper bags - strong solvent odor. From 20'-25' soil is oily coated, difficult to wash grains - heavy sheen, some loose oil/sludge between grains.	
	Grab	1140	GP: Poorly graded gravel, very dark grey (10YR3/1), loose to dense, moist, fine to medium sand, fine gravel, heavy oily residue on grains, sheen when washed, strong solvent odor, traces of wood splinters, areas of pure sludge between grains. Approximately 80% gravel, 10% coarse sand, 5% medium sand and 5% fine sand.	
-30		2900	Poorly graded sand, dark brown (7.5YR3/4), very dense, fine to coarse angular to subangular volcanic sand and fine gravel, trace silt, noncemented sheen when washing grains. Approximately trace gravel, 10% coarse sand, 70% medium sand, 20% fine sand and trace silt. Terminated borehole at 31.5' bgs.	

Log of Boring No. BRC-BS-09
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/19/05
Date Completed: 01/19/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	PID	Lithology	Soil Description	Well Construction
0				SP: Soil surface, boring located within slight depression area. (Odors not being observed due to potential chemical hazards.)	
	Grab				
		2.1		Poorly graded sand, brown (10YR5/3), loose, moist, fine to coarse subrounded to rounded sand and fine gravel - no volcanic, noncemented, no debris. Approximately trace gravel 5% coarse sand, 70% medium sand and 25% fine sand.	
				Trash begins at 9' bgs.	
-10		9.8		SP: Poorly graded sand as at 5', layered between debris - gray and white speckled cement/concrete - like material. Approximately trace gravel, 5% coarse sand, 70% medium sand and 25% fine sand.	
				Drillers noted a change in drilling at ~16'.	
	Grab	5.2		Poorly graded sand, brown (10YR4/3), dense, moist, fine to coarse sand and fine gravel - subrounded to rounded, no volcanics, noncemented. Debris includes wood, fabric and plastic. Approximately trace gravel, trace coarse sand, 80% medium sand and 20% fine sand.	
-20		4.2		Poorly graded sand, dark gray (10YR4/1), dense, moist fine to coarse subrounded to rounded sand and fine gravel, noncemented, debris includes glass, paper and white chalky material, plastic bottles. Approximately trace gravel, trace coarse sand, 80% medium sand and 20% fine sand.	
	Grab	9.7		Poorly graded sand, brown (10R4/3), dense, moist, fine to coarse subangular to rounded sand and fine gravel, noncemented, debris including glass and paper, some metal. Approximately 10% gravel, 20% coarse sand, 50% medium sand and 20% fine sand.	
				Poorly graded sand, brown (10YR5/3), very dense, fine to coarse subangular to angular sand and fine gravel - volcanic, noncemented, no debris. Approximately 10% gravel, 20% coarse sand, 50% medium sand and 20% fine sand.	
-30		7.4		Terminated borehole at 32' bgs.	

Log of Boring No. BRC-BS-10
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 22 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/15/05
Date Completed: 01/15/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface - soil boring apears to be in possible slightly subsided area. (Odors not being observed due to potential chemical hazards.)	
	Grab							
					13.6		Poorly graded sand, brown (10YR4/3), loose to medium dense, fine to coarse, subrounded to rounded sand and fine gravel, no volcanics, trace silt, noncemented, no debris. Approximately 5% gravel, 5% coarse sand, 80% medium sand, 10% fine sand and trace silt.	
-10					3.5		Poorly graded sand, yellowish brown (10YR5/4), very dense, moist, fine to coarse subangular volcanic sand and fine gravel, noncemented, no odor, no debris. Approximately 5% gravel, 10% coarse sand, 60% medium sand and 25% fine sand.	
	Grab				4.1		Poorly graded sand, yellowish brown (10YR5/4), medium dense, moist, fine to coarse angular to subrounded sand and fine gravel including some volcanics, noncemented, trace silt. Approximately 5% gravel, 10% coarse sand, 65% medium sand, 10% fine sand and trace silt.	
							Poorly graded sand, yellowish brown (10YR5/4), very dense, moist, fine to coarse angular to subangular volcanic sand and fine gravel, noncemented. Aproximately 5% gravel, 15% coarse sand, 60% medium sand and 20% fine sand.	
-20							Terminate borehole at 22' due to no debris encountered. Fill material ended by 9'.	

Log of Boring No. BRC-BS-11
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/13/05
Date Completed: 01/13/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface. Poorly graded sand, yellowish brown (10YR5/4), loose, moist, fine to medium subangular to subrounded sand, trace subangular to subrounded gravel, trace silt, noncemented. Approximately trace gravel, 10% coarse sand, 60% medium sand, 30% fine sand and trace silt.	
	Grab				0.5		@ 5' bgs, debris encountered consisting of fabric and cobbles.	
-10							Poorly graded sand, yellowish brown (10YR5/4), loose, moist, fine to medium subangular to subrounded sand trace fine grained, trace silt, noncemented from 10-10.5' bgs, black fine material and gravel debris. Approximately trace gravel, 10% coarse sand, 60% medium sand, 30% fine sand and trace silt.	
	Grab				9.0		Poorly graded sand, very dark grayish brown (2.5Y3/2), loose, moist, fine to coarse subangular to subrounded sand, fine subrounded gravel, noncemented. Approximately 5% gravel, 5% coarse sand, 30% medium sand and 60% fine sand.	
-20					42.5		Poorly graded sand, brown (10YR5/3), medium dense, moist, fine to medium sand and fine gravel (as at 15'), noncemented, with paper debris and solvent odor. Approximately 10% gravel, 5% coarse, 25% medium sand and 60% fine sand.	
							@ 23', mostly debris-cobbles, wood, paper (receipts, paper pads), fabric, minor amounts of metal and plastic, paper bags, strong odor (solvent).	
-30					70 0.2		Drillers hit hard soil at 29.5' bgs. Poorly graded sand, yellowish brown 10YR5/4), very dense, moist, fine to coarse sand, subangular to subrounded volcanic, trace gravel, noncemented. Approximately trace gravel, 5% coarse sand, 50% medium sand, 45% fine sand and trace silt. Borehole terminated @ 32 ft bgs.	

Log of Boring No. BRC-BS-12
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/12/05
Date Completed: 01/12/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface; boring located within area of slight subsidence. (Odors not being observed due to potential chemical hazards.)	
	Grab						Debris encountered - plastic lined paper bag pieces.	
					1.0		Poorly graded sand, brown (10YR4/3), loose, moist, fine to coarse subangular to subrounded sand and subrounded fine gravel, trace silt, noncemented, few paper and plastic debris fragments. Approximately 5% gravel, 20% coarse sand, 55% medium sand, 20% fine sand and trace silt.	
-10					7.6		Poorly graded sand, brown (10YR4/3), very dense, moist, fine to coarse, subrounded sand and subrounded fine gravel, trace silt, noncemented debris including paper, plastic and white cement fragments. Approximately 5% gravel, 10% coarse sand, 65% medium sand, 20% fine sand and trace silt.	
	Grab				7.0		Poorly graded sand, brown (10YR4/3), moist, fine to coarse subrounded sand and fine subrounded gravel, noncemented, paper and plastic debris. Approximately 5% gravel, 10% coarse sand, 65% medium sand, 20% fine sand and trace silt.	
							@18' bgs, increase in plastic lined paper bag remnants - writing on them reads "Montrose ODT".	
-20					0.6		Poorly graded sand, yellowish brown (10YR5/4), moist, dense, fine to coarse, subrounded sand and fine subrounded gravel, noncemented, paper and plastic debris. Approcimately 5% gravel, 15% coarse sand, 65% medium sand, 15% fine sand and trace silt.	
	Grab				11.7		Poorly graded sand, yellowish brown (10YR5/4), moist, fine to coarse subrounded sand and fine subrounded gravel, noncemented, small ammounts of paper and plastic. Approximately 5% gravel, 5% coarse sand, 60% medium sand, 30% fine sand and trace silt.	
							Poorly graded sand with gravel, yellowish brown (10YR5/4), moist, fine to coarse subrounded sand and fine subangular to angular gravel, noncemented, no debris. Approximately 15% gravel, 5% coarse sand, 40% medium sand and 40% fine sand.	
-30					1.5		Terminate borehole at 32' bgs.	

Log of Boring No. BRC-BS-13
CAMU
Henderson, Nevada

Basic Remediation Company

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 31 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/18/05
Date Completed: 01/18/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface, boring located in area of subsidence. Poorly graded sand, yellowish brown (10YR5/4), loose to dense, fine to coarse subrounded to rounded sand and fine gravel, no volcanics, noncemented, trace debris, consisting of glass shards (very little). Approximately trace gravel, 10% coarse sand, 70% medium sand, 20% fine sand and trace silt.	
	Grab				22			
-10					29.1		At 9' bgs, debris begins, consisting of visqueen and rope. Poorly graded sand, brown (10YR4/3), yellowish brown (10YR5/4), medium dense, moist, fine to coarse subrounded sand and fine gravel, very few volcanics, noncemented, small layer of black (burned) paper bags. Approximately trace gravel, 10% coarse sand, 70% medium sand 20% fine sand and trace silt.	
	Grab				85.6		Poorly graded sand, dark grayish brown (10YR4/2), loose, fine to coarse subrounded to rounded sand and fine gravel, few volcanics, noncemented, abundant debris, entire paper (plastic lined) bags of lime (~80%) and DDT (~20%), solvent odor (flowery). Approximately trace gravel, 5% coarse sand, 65% medium sand, 30% fine sand and trace silt.	
-20					53.1		Entire bags of lime (~80%) and DDT (~20%) with very little sand mixed in, loose trash, solvent odor (flowery). Approximately trace gravel, 5% coarse sand, 65% medium sand, 30% fine sand, and trace silt.	
							Drillers indicated change of drilling at 23', out of trash zone.	
	Grab				0.4		Poorly graded sand, brown (7.5YR5/3), loose to dense, moist, fine to coarse subangular sand and fine gravel, increased volcanics, noncemented. Approximately trace gravel, 10% coarse sand, 60% medium sand, 30% fine sand and trace silt.	
							Poorly graded sand, light yellowish brown (10YR 6/4), very dense, moist, fine to coarse angular to subrounded volcanic sand and fine gravel, noncemented, trace silt. Approximately trace gravel, 5% coarse sand, 55% medium sand, 40% fine sand and trace silt.	
-30					1.1		Terminate borehole at 31' bgs.	

Log of Boring No. BRC-BS-14
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 31.5 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/19/05
Date Completed: 01/19/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soi surface, boring located with in slight depression area. Poorly graded sand, strong brown (7.5YR4/6), loose, moist, fine to coarse subrounded to rounded sand, no volcanics, noncemented.	
	Grab				0.2		Debris begins at ~5' bgs: fabric, paper bags and plastic.	
-10					11.7		Poorly graded sand, strong brown (7.5YR4/6), dense, moist, fine to coarse subrounded to rounded sand, no volcanics, with trace medium sand size angular red brick fragments, noncemented, debris including paper and burned wood fragments, with some plastic. Approximately 5% gravel, 5% coarse sand 80% medium sand and 10% fine sand.	
	Grab				9.3		Poorly graded sand, very dark gray (7.5YR3/1), loose to dense, moist, fine to coarse subangular to rounded sand (some consisting of red brick fragments), fine gravel, noncemented, debris consists of plastic, paper, wood splinters and fabric. Approximately 5% gravel, 5% coarse sand, 70% medium sand and 20% fine sand.	
-20					13.2		Poorly graded sand, very dark gray (7.5YR3/1), very dense, moist, fine to course subangular to rounded sand (some consisting of red brick fragments), fine gravel and few cobbles, noncemented, debris is paper, plastic, fabric, and wood. Approximately 5% gravel, 5% coarse sand, 70% medium sand and 20% fine sand.	
	Grab				6.1		Poorly graded sand, very dark gray (7.5YR3/1), loose to dense, moist, fine to coarse subangular to rounded sand and fine gravel (with red brick fragments), noncemented, debris is paper and plastic. Approximately 5% gravel, 10% coarse sand, 60% medium sand and 25% fine sand.	
-30					0.4		Poorly graded sand, brown (7.5YR5/4), very dense, moist, fine to coarse angular to subangular volcanic sand and fine gravel, noncemented, no debris, drillers also noted a change in drilling. Approximately trace gravel, 5% coarse sand, 55% medium sand and 40% fine sand. Terminatel boring at 31.5' bgs.	

Log of Boring No. BRC-BS-15
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/19/05
Date Completed: 01/19/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface located approximately 10' east of sunken pit from trench (moved from origional location, which was ~30' east of current).	
	Grab				1.5		Poorly graded sand, dark grayish brown (10YR4/2), moist, loose, fine to coarse subangular to rounded sand and fine gravel, few cobbles, debris includes wood, glass and cobbles coated in black residue. Approximately 10% gravel, 10% coarse sand, 60% medium sand and 20% fine sand.	
-10					0.8		Poorly graded sand, brown (10YR5/3), dense, moist, fine to coarse, angular to subangular sand and fine gravel, few volcanics, debris includes few wood splinters and paper (from bags). Approximately trace gravel, 5% coarse sand, 65% medium sand, 30% fine sand and trace silt.	
	Grab				0.3		Poorly graded sand, yellowish brown (10YR5/4), dense, moist, fine to coarse angular to subrounded sand and fine gravel, no volcanics, noncemented, no debris, trace silt, slight solvent odor. Approximately trace gravel, 5% coarse sand, 65% medium sand, 30% fine sand and trace silt.	
-20					0.4		Poorly graded sand, yellowish brown (10YR5/4), medium dense, moist, fine to coarse angular to subrounded sand and fine gravel, abundant volcanics, noncemented, no debris, trace silt, slight solvent odor. Approximately trace gravel, 5% coarse sand, 55% medium sand, 40% fine sand and trace silt.	
	Grab				1.3		Poorly graded sand, as at 15', no debris, medium dense, slight solvent odor. Approximatley trace gravel, 5% coarse sand, 65% medium sand, 30% fine sand and trace silt.	
-30					0.1		Poorly graded sand, yellowish brown (10YR5/4), very dense, moist, fine to coarse angular to subangular sand and fine gravel, volcanic areas of weak to moderate cementation, trace silt, no debris, slight solvent odor. Approximately trace gravel, 5% coarse sand, 55% medium sand, 40% fine sand and trace silt. Terminate borehole at 32' bgs.	

Log of Boring No. BRC-BS-16

CAMU

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/15/05
Date Completed: 01/15/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	PID	Soil Description	Well Construction
0			SP: Soil surface soil boring in a hill.	
			Debris begins at surface, consists of white powdery/chalky substance mixed with soil fill.	
	Grab	94.3	Poorly graded sand, grayish brown (10YR5/2), loose, fine to coarse subrounded to rounded sand and fine gravel, not volcanic, noncemented, solvent odor, with traces of debris - white chalky pieces and wood splinters. Approximately trace gravel, 20% coarse sand, 70% medium sand and 10% fine sand.	
-10		92.5	Poorly graded sand, brown (10YR5/3), medium dense, fine to coarse angular to subrounded sand and fine gravel - volcanic, strong solvent odor. Approximately 5% gravel, 15% coarse sand, 60% medium sand, 20% fine sand and trace silt.	
	Grab	2773	SP (as at 10'), very strong solvent odor (flowery smell). Approximately 5% gravel, 15% coarse sand, 60% medium sand, 20% fine sand and trace silt.	
-20		30.1	Poorly graded sand, yellowish brown (10YR5/4), fine to coarse subangular volcanic sand and fine gravel, noncemented, strong solvent odor. Approximately 5% gravel, 5% coarse sand, 70% medium sand and 20% fine sand.	
	Grab	NM	Poorly graded sand, yellowish brown (10YR5/4), fine to coarse subangular to rounded sand and fine gravel - some volcanic, noncemented, strong solvent odor. Approximately trace gravel, 20% coarse sand, 70% medium sand and 10% fine sand.	
-30		213	Poorly graded sand, yellowish brown (10YR5/4), fine to coarse angular to subangular sand and fine gravel- volcanic, noncemented, no odor, trace silt. Approximately 5% gravel, 5% coarse sand, 70% medium sand, 20% fine sand and trace silt.	
		6.3	Terminate borehole at 32' bgs.	

Log of Boring No. BRC-BS-17

CAMU

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/13/05
Date Completed: 01/13/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	PID	Soil Description
0			SP: Soil surface, boring located with in area of subsidence.
	Grab	NM	Poorly graded sand with gravel, pale brown (10YR6/3), loose, moist, fine to coarse subrounded to rounded sand and fine rounded gravel, noncemented, contains debris consisting of plastic-lined paper bags and wood. Approximately 20% gravel, 10% coarse sand, 50% medium sand and 20% fine sand.
			@9' bgs, wood chunks.
			DEBRIS: @ 9.75'-10' bgs, black powder (fine to fine end of medium sand size) carbon(?).
-10			SP: Poorly graded sand with gravel, brown (10YR5/3), very dense, moist, fine to coarse subangular to subrounded sand and fine subrounded gravel, noncemented. Approximately 15% gravel, 10% coarse sand, 55% medium sand and 20% fine sand.
			@13' bgs, debris includes plastic lined paper bags and black fine gravel sized debris.
-20		20.4	Poorly graded sand, brown (10YR4/3), very dense, moist, fine to coarse subangular to subrounded sand and fine angular gravel sized black fragments. trace cobbles up to 2.5" long. Approximately 5% gravel, 10% coarse sand, 50% medium sand and 35% fine sand.
			@ 21' bgs, drillers noted change in drilling to harder soil.
		200	@ 22' bgs, color change to dark reddish brown (5YR2.5/2), strong odor.
	Grab	238	Poorly graded sand, dark brown (7.5YR3/2), moist, fine to coarse subangular to subrounded sand and subangular fine gravel, noncemented, strong odor, no debris.
-30		12.4	Poorly graded sand, brown (7.5YR 5/3), very dense, moist, fine to coarse subangular to subrounded sand and fine subangular to subrounded gravel, noncemented, strong odor. Approximately 5% gravel, 5% coarse sand, 40% medium sand and 40% fine sand.
		345	Terminate borehole at 32' bgs.

Well Construction

Log of Boring No. BRC-BS-18
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: James Duke

Borehole Total Depth: 32 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/13/05
Date Completed: 01/13/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface, boring located within area of subsidence.	
	Grab				2.6		Poorly graded sand with gravel, dark grayish brown (10YR4/2), medium dense to loose, fine to coarse subrounded to rounded sand and subrounded fine gravel, noncemented. Approximately 15% gravel, 40% coarse snad 40% medium sand, 5% fine sand and trace silt.	
-10					1.5		Poorly graded sand, light gray (10YR7/2), dense, moist, fine to medium subangular sand and fine subangular gravel, noncemented, containing debris - mostly rusted metal pipe fragments and small amounts of insullation material (fiberglass or asbestos). Approximately 5% gravel, 35% medium sand and 60% fine sand.	
					1.2		Poorly graded sand, light gray (10YR7/2), medium dense, moist, fine to medium subangular sand and fine subangular gravel, noncemented, coantains rusted metal pipe fragments and small amounts of insulation attached. Approximately 5% gravel, 35% medium sand and 60% fine sand.	
-20					0.2		Poorly graded sand, dark yellowish brown (10YR4/4), medium dense, moist, fine to coarse subangular to subrounded sand and fine subangular gravel, no debris observed. Approximately 5% gravel, 5% coarse sand, 50% medium sand, 40% fine sand and trace silt.	
	Grab				38.5		GP: Poorly graded gravel with sand, brown (10YR4/3), medium dense to loose, moist, fine to coarse subrounded sand and fine subrounded to rounded gravel, noncemented, debris includes fabric with cobbles. Approximately 55% gravel, 10% coarse sand, 30% medium sand and 5% fine sand.	
					5.7		SP: Poorly graded sand, brown (10YR4/3), very dense, moist, fine to medium subangular to subrounded sand and subangular gravel, noncemented, no debris, trace silt. Approximately 10% gravel, 10% coarse sand, 50% medium sand, 30% fine sand and trace silt. At 26' bgs, drilling in hard soil - drillers think we are out of fill.	
-30							Terminate boring at 32' bgs.	

Log of Boring No. BRC-BS-19
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 31.5 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/18/05
Date Completed: 01/18/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface, boring located in area of subsidence.	
	Grab				0.5		Poorly graded sand, brown (7.5YR4/3), loose to dense, moist, fine to coarse subangular to subrounded, sand and fine gravel - no volcanics, noncemented, trace silt. Approximately trace gravel, 5% coarse sand, 55% medium sand, 40% fine sand and trace silt.	
-10					1.3		Poorly graded sand, brown (10YR4/3), moist, fine to coarse subangular to subrounded sand and fine gravel - no volcanics, noncemented, debris consists of white chalky powder. Approximately trace gravel, 5% coarse sand, 55% medium sand and 40% fine sand. At 9.5' bgs debris begins - consists of white powdery/chalky chunks of material from plastic lined paper bags.	
	Grab				0.7		Poorly graded sand, grayish brown (10YR5/2), loose to dense, moist, fine to coarse subangular to rounded sand and fine gravel - no volcanics, some cobbles, noncemented. Approximately 10% gravel, 15% coarse sand, 70% medium sand and 5% fine sand. From 15'-20', few bags - lime.	
-20					0.6		Poorly graded sand, yellowish brown (10YR5/4), dense, moist, fine to coarse subangular to rounded sand and fine gravel - few volcanics, noncemented, debris includes wood splinters and lime bags. Approximately 5% gravel, 15% coarse sand, 70% medium sand and 10% fine sand.	
	Grab				43.5		Poorly graded sand, yellowish brown (10YR5/4), loose to dense, moist, fine to coarse subangular to subrounded sand and fine gravel, few cobbles, no volcanics, noncemented, debris includes gravel fill, wood splinters and traces of white chalky substance. Approximately 5% gravel, 15% coarse sand, 70% medium sand and 10% fine sand.	
-30					12.5		Poorly graded sand , brown (10YR5/3), very dense, moist, fine to coarse angular to subangular sand and fine gravel - volcanic, noncemented, no debris. Approximately trace gravel, 5% coarse sand, 70% medium sand, 25% fine sand and trace silt. Terminate borehole at 31.5' bgs.	

Log of Boring No. BRC-BS-20
CAMU
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: HSA
Drilling Equipment: CME-75/HSA LAR
Drilling Contractor: Water Development Corporation
Driller: Ignacio Gutierrez

Borehole Total Depth: 31 ft bgs
Borehole Diameter: 8 in
Well ID: N/A
Depth to Water (ft. bgs): NA

Sample Type: 2.5" Split Spoon
Sample Interval Continuous

Logged By: J. Wiley
Date Started: 01/18/05
Date Completed: 01/18/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0							SP: Soil surface, boring located in area of subsidence.	
	Grab				0.2		Poorly graded sand, dark yellowish brown (10YR4/4), loose to dense, moist, fine to coarse subangular to subrounded sand and fine gravel - no volcanics, noncemented, no debris, trace silt. Approximately trace gravel, 5% coarse sand, 65% medium sand, 30% fine sand and trace silt.	
-10					1.7		Poorly graded sand, dark yellowish brown (10YR4/4), dense, moist, fine to coarse subangular to subrounded sand and fine gravel - few volcanics, noncemented, trace silt, no debris.	
	Grab				21.0		Poorly graded sand, dark yellowish brwon (10YR4/4), loose, fine to coarse subangular to rounded sand and fine gravel - no volcanics, noncemented, debris consists of DDT and lime bags, solvent odor (flowery). Approximately 5% gravel, 10% coarse sand, 75% medium sand and 10% fine sand.	
							@ 19', plastic debris and paper bags (DDT and lime) with SP.	
-20					22.0		CL: Sandy lean clay, olive yellow (2.5Y6/6), soft, wet, fine sand and trace medium sand, noncemented, medium plasticity, petroleum hydrocarbon odor, clay was found in sampler shoe and stuck to outside of sampler. Approximately trace medium sand, 30% fine sand and 70% clay.	
							SP	
	Grab				21.0		@ 25' Pooly graded sand as at 15', no debris. Drillers noted change of drilling at 20.5'.	
-30					0.4		Poorly graded sand, brown (10YR5/3), very dense, moist, fine to coarse angular to subangular volcanic and fine gravel, weak cementation, no debris. Approximately trace gravel, 5% coarse sand, 80% medium sand and 15% fine sand.	
							Terminate borehole at 31' bgs.	

Log of Boring No. BW-8B

BMI Landfill CAMU Investigation

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 75 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-AA-BW-08A
Depth to Water (ft. bgs): 50' bgs

Sample Type: N/A
Sample Interval Continuous

Logged By: A. Norris
Date Started: 03/15/05
Date Completed: 03/15/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	37.5 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	57.5 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0					0.0		SM: Silty sand with gravel, moist, (fill).	Locking Monument Well Box Surrounded by Concrete slab and Traffic Ballards
					0.0			
							Silty sand with gravel, moist to dry, trace local debris (wood fragments, plastic) (fill).	
					0.0			
					0.0		Silty sand with gravel, dry, trace local debris (plastic, wood, metal fragments) (fill). Note: soils @ 0' to 12' bgs essentially same as logged a BW-8A. See boring log BW-8A for details.	
-10								Cement Backfill
					0.0			
					0.0		Silty sand with gravel, dry.	
					0.0			4" Blank PVC Riser Pipe
					0.0			
							Silty sand with gravel, dry. Note: soils @12' to 20" bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	
					0.0			
-20								
					0.0		Silty sand with gravel, dry.	
					0.0		Silty sand with gravel, dry.	
								Cement Backfill
					0.0			

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
							Silty sand with gravel, dry. Note: soils @ 20' to 30' bgs essentailly same as logged @ BW-8A. See boring log BW-8A for details.	
					0.0			
-30					0.0		Silty sand with gravel, dry.	Bentonite Seal
					0.0		Silty sand with gravel, dry.	
								#2/12 Sand Pack
					0.0		Silty sand with gravel, dry. Note: soils @ 30' to 40' bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	
					0.0			
-40					0.0		Silty sand with gravel, dry.	4" 0.010 Slot PVC screen
					0.0		Silty sand with gravel, dry.	
					0.0		Silty sand with gravel, dry to moist. Note: soils @ 40' to 50' bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	
					0.0			
-50							Silty sand with gravel, wet @ 50' to 57' bgs.	
					0.0		Silty sand with gravel, wet.	
					0.0			
					0.0		GW: Well graded gravel, multicolored, wet, subangular to subrounded fine to coarse volcanic gravel (up to 3" long). Approximately 100% gravel.	Threaded PVC Well Base Cap
					0.0		ML: Silt with sand, light brownish gray (10YR6/2), moist to wet, nonplastic to low plasticity fines, subangular to subrounded sand. Approximately 15% fine sand, 85% silt and trace clay.	
-60					0.0		CALICHE: Caliche, strongly cemented. Silty sand with gravel, white (10YR8/1), dry.	
					0.0			
					0.0			

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					5.5		ML: Clayey silt, yellowish brown (10YR5/4), moist, very low to low plasticity fines. Approximately 5% fine sand, 80% silt and 15% clay.	Basal Bentonite Seal
					14.2			
					3.0		Clayey silt, yellowish brown (10YR5/4), moist, very low to low plasticity fines. Approximately 85% silt and 15% clay.	
					8.2		Silt with sand, yellowish brown (10YR5/4), moist, nonplastic to very low plasticity fines, subangular to subrounded sand. Approximately 15% fine sand, 85% silt and trace clay.	
-70					28.7		Clayey silt, yellowish brown (10YR5/4). moist, very low to low plasticity fines. Approximately 85% silt and 15% clay.	
					2.0			
					1.0		Clayey silt, light yellowish brown (10YR6/4), moist, very low to low plasticity fines. Approximately 85% silt and 15% clay.	
					3.0			
					0.0		Bottom of hole @ 75' bgs.	

Log of Boring No. BW-8C
BMI Landfill CAMU Investigation
Henderson, Nevada

Basic Remediation Company

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 75 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-AA-BW-08B
Depth to Water (ft. bgs): 47' bgs

Sample Type: N/A
Sample Interval Continuous

Logged By: A. Norris
Date Started: 03/16/05
Date Completed: 03/16/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	43 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	63 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0					0.0		SM: Silty sand with gravel, dry.	Locking monument well box with concrete apron and ballards
					0.0		Silty sand with gravel, dry.	Cement Bentonite Grout
					0.0		Silty sand with gravel, dry. Note: Soils @ 0' to 10' bgs essentially same a logged @ BW-8A. See boring log BW-8A for details.	
-10					0.0			4" Blank PVC Riser Pipe
					0.0		Silty sand with gravel, dry.	
					0.0		Silty sand with gravel, dry, trace fill material, (wood fragments).	
					0.0		Note: Soil encountered @ 10' to 20' bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	
-20					0.0		Silty sand with gravel, dry.	
					0.0		Silty sand with gravel, dry.	

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
							Silty sand with gravel, dry.	Cement Bentonite Grout
							Note: Soil encountered @ 20' to 30' bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	
-30					0.0		Silty sand with gravel, dry.	
					0.0			
							Silty sand with gravel, dry.	
					0.0			Bentonite seal
							Silty sand with gravel, dry.	
					0.0		Note: soils encountered @ 30' to 40' bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	
-40							Silty sand with gravel, dry.	
					0.0			#2/12 Sand Pack
					0.0		Silty sand with gravel, dry. Note: Soils encountered @ 40' to 47' bgs essentially same as logged @ BW-8A. See boring log BW-8A for details.	4" 0.010 Slot PVC screen
					3.6		Silty sand with gravel, grayish brown (10YR5/2), wet, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 1.5" long). Approximately 15% gravel, 15% coarse sand, 30%medium sand, 25% fine sand and 15% silt.	
-50					4.0			
					6.2		ML: Silt with sand, light brownish gray (10YR6/2), dry to moist, nonplastic to low plasticity silt, subangular to subrounded sand. Approximately 15% fine sand and 85% silt.	
					113		SM: Silty sand with gravel, light grey (10YR6/1), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 1" long), weakly to strongly cemented. Approximately 15% gravel, 15% coarse sand, 30% medium sand, 25% fine sand, 15% silt.	
					22.6			
					3.5		CALICHE: Caliche: strongly cemented. Silty sand with gravel, white (10YR8/1), dry.	
					4.5		SM: Silty sand with gravel, gray (10YR5/1), wet, subangular to subrounded sand, nonplastic silt, subangular to subrouneded fine to coarse volcanic gravel (up to 1.5" long). Approximately 15% gravel, 15% coarse sand, 35% medium sand, 20% fine sand, 15% silt.	
					3.2			
					0.0		ML: Silt with sand, light brownish gray (10YR6/2), dry, nonplastic to very low plasticity silt, subangular to subrounded sand. Approximately 15% fine sand and 85% silt.	
					2.0			#2/12 Sand Pack
-60							CALICHE: Caliche: strongly cemented. Silty sand with gravel, white (10YR8/1), dry.	
					3.3		ML: Clayey silt, yellowish brown (10YR5/4), moist, very low to low plasticity fines, trace subangular to subrounded sand. Approximately 5% fine sand, 80% silt and 15% clay.	

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					3.8		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximately 85% silt and 15% clay.	
					2.3			Threaded PVC Well Base Cap
					16.1			
					14.4			
					65.3		Clayey silt, brown (7.5YR5/4) moist, very low to low plasticity fines, stiff. Approximately 85% fine sand and 15% silt.	
					2.9			
-70	Grab				77.2			
					104		Silt, pale brown (10YR6/3), moist, nonplastic to very low plasticity silt. Approximately 5% fine sand and 95% silt. (Sample BRC-BW-08C-70 collected).	Basal Bentonite Seal
					10.9			
					6.4			
					3.4			
					15.1		Bottom of hole @ 75' bgs.	

Log of Boring No. BW-10A

BMI Landfill CAMU Investigation

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 80 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-MCF-BW-10
Depth to Water (ft. bgs): 60.0 ft bgs

Sample Type: 2" Split Spoon
Sample Interval Continuous

Logged By: A. Norris
Date Started: 03/23/05
Date Completed: 03/23/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	57 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	72 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12 sand

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0					0.0		SM: Silty sand with gravel, brown (7.5YR5/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 30% fine sand and 20% silt. (Sample BRC-BW-10A-0 collected).	Locking monument well box with concrete apron and ballards
					0.0		Silty sand with gravel, brown (7.5YR5/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	Cement Bentonite Grout
					0.0			
					0.0		Silty sand with gravel, light brown (7.5YR6/3), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2.5" long). Approximately 20% gravel, 15% coarse sand 10% medium sand, 40% fine sand and 15% silt.	
-10					0.0			4" Blank PVC Riser Pipe
					0.0		Silty sand with gravel, brown (7.5YR 5/3) dry subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long). Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
					0.0		Silty sand, light brown (7.5YR6/4), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long). Approximately 20% gravel, 15% coarse sand, 15% medium sand, 30% fine sand and 20% silt. (Sample BRC-BW-10A-18 collected).	
					0.0			
-20					0.0		Silty sand with gravel, pale brown (10YR6/3), dry, angular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 1.5" long), locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 30% fine sand and 20% silt.	
					0.0		Silty sand, light brownish gray (10YR6/2), dry, subangular to subrounded sand,nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented.	

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0		Silty sand with gravel, light brown (7.5YR6/3), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 1" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 20% silt.	
					0.0			
-30					0.0		Silty sand with gravel, yellowish brown (10YR5/4), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximtaely 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt. (Sample BRC-BW-10A-30 collected).	Cement Bentonite Grout
					0.0			
					0.0		Silty sand with gravel, yellowish brown (10YR5/4), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
					0.0			
-40							ML: Silt with sand, strong brown (7.5YR5/6), moist, nonplastic silt, subangular to subrounded sand. Approximately 15% fine sand and 85% silt.	
					0.0		Silt with sand, brown (7.5YR5/4), moist, nonplastic silt, subangular to subrounded sand. Approximately trace medium sand, 20% fine sand and 80% silt.	
					0.0		Silt with sand, strong brown (7.5YR5/6), moist, nonplastic silt, subangular to subrounded sand. Approximtaely 15% fine sand and 85% silt.	
					0.0		Silt with sand, strong brown (7.5YR5/6), moist, nonplastic to low plasticity fines, subangular to subrounded sand. Approximately 15% fine sand, 80% silt and 5% clay.	
-50					0.0		Clayey silt, brown (10YR5/4), moist, very low to low plasticity fines, stiff. Approximately 5% fine sand, 80% silt and 15% clay. (Sample BRC-BW-10A-50 collected).	Bentonite seal
					0.0		Silt with clay, light olive brown (5Y6/2), moist, very low to low plasticity fines, stiff. Approximately 90% silt and 10% clay.	
					0.0		Silt with clay, brwon (10YR5/3), moist, very low to low plasticity fines, stiff. Approximately 90% silt and 10% clay.	#2/12 Sand Pack
								4" 0.010 Slot PVC screen
					0.0		Becomes wet @ 60' to 64' bgs.	
-60					0.0		Silt with sand, brown (7.5YR5/4), wet, nonplastic to low plasticity fines, subangular to subrounded sand. Approximatley 5% medium sand, 20% fine sand, 70% silt and 5% clay.	4" 0.010 Slot PVC screen

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0			
							Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff, trace subangular to subrounded sand. Approximately 5% fine sand, 80% silt and 15% clay.	#2/12 Sand Pack
					0.0			
							Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximately 80% silt and 20% clay.	
					0.0			
-70								
					0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximately 85% silt and 15% clay.	Threaded PVC Well Base Cap
								Basal Bentonite Seal
					0.0			
							Clayey silt, strong brown (7.5YR5/6), moist, very low to low plasticity fines, stiff. Approximately 80% silt and 20% clay.	
					0.0			
-80								

Log of Boring No. BW-11A
BMI Landfill CAMU Investigation
Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 80 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-MCF-BW-11
Depth to Water (ft. bgs): 60' bgs ▽

Sample Type: 2" Split Spoon
Sample Interval Continuous

Logged By: A. Norris
Date Started: 03/22/05
Date Completed: 03/22/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	57 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	72 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12

Depth Elevation (MSLD)	PID	Soil Description	Well Construction
0	0.0	SM: Silty sand with gravel, yellowish brown (10YR5/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long). Approximately 15% gravel, 10% coarse sand, 15% medium sand 45% fine sand and 15% silt. (Sample BRC-BW-11A-0 collected).	Locking monument well box with concrete apron and ballards
	0.0		
	0.0		
	2.0	Silty sand with gravel, yellowish brown (10YR5/4), dry to moist, subangular to subrounded sand, nonplastic to very low plasticity silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long). Approximately 15% gravel, 10% coarse sand, 15% medium sand, 40% fine sand and 20% silt.	
	1.8		
	2.0		Cement Bentonite Grout
	4.0	Silty sand with gravel, brown (7.5YR5/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long). Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
	3.7		
	4.1		
-10	4.7		
	14.2	Silty sand with gravel, brown (7.5YR5/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2.5" long). Approximately 15% gravel, 10% coarse sand 15% medium sand, 45% fine sand and 15% silt.	4" Blank PVC Riser Pipe
	9.1		
	25.3		
	17.7	Silty sand with gravel, brown (7.5YR5/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 1" long). Approximately 15% gravel, 10% coarse sand, 15% medium sand, 45% fine sand and 15% silt.	
	22.5		
	20.9		
	19.3	Silty sand with gravel, pale brown (10YR6/3), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2.5" long), locally partly indurated. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 30% fine sand and 20% silt. (Sample BRC-BW-11A-18 collected).	
	53.5		
	54.3		
-20	30.9		
	14.7	Silty sand with gravel, brown (7.5YR5/3), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long). locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
	57.0		
	49.3		
	59.1		
	118	Silty sand with gravel, light brown (7.5YR6/3), dry subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 20% silt.	
	50.0		

Depth Elevation (MSLD)	Sample Type	PID	Soil Description	Well Construction
		46.7, 44.3, 46.8	Silty sand with gravel, light brown (7.5YR6/4), dry subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 1" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 10% coarse sand, 10% medium sand, 50% fine sand and 15% silt.	Cement Bentonite Grout
-30		100, 29.2	(Sample BRC-BW-11A-30 collected).	
		60.8, 41.1, 45.2	Silty sand with gravel, brown (7.5YR5/4), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long). Approximately 20% gravel, 15% coarse sand 15% medium sand, 35% fine sand and 15% silt.	
		27.2, 25.7, 14.6	Silty sand with gravel, brown (7.5YR5/3), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long). Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
		10.0, 5.7	Silty sand with gravel, gray (10YR6/2), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented, trace local volcanic cobbles (up to 4" long). Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
-40			CALICHE: Caliche: strongly cemented. Silty sand, white (10YR8/1), dry.	
		22.6, 22.5, 38.4, 0.0	SM: Silty sand, light gray (10YR7/2), dry, subangular to subrounded sand, nonplastic silt, locally weakly to strongly cemented. Approximately 5% coarse sand, 15% medium sand, 60% fine sand and 20% silt.	
		0.0, 0.0, 0.0	Silty sand, very pale brown (10YR8/2), dry, subangular to subrounded sand, nonplastic silt, locally partly indurated to weakly cemented. Approximately 5% gravel, 5% coarse sand, 10% medium sand, 60% fine sand and 20% silt.	
	Grab	0.0, 0.0	Silty sand, brown (10YR5/3), dry, subangular to subrounded sand, nonplastic silt, locally weakly to strongly cemented. Approximately 5% gravel, 5% coarse sand, 10% medium sand, 60% fine sand and 20% silt. (Sample BRC-BW-11A-49 collected).	
-50		12.2, 0.0		Bentonite seal
		0.0, 0.0, 0.0	ML: Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximately 80% silt and 20% clay.	
		0.0, 0.0, 0.0, 0.0, 0.0	Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff to very stiff. Approximately 80% silt and 20% clay.	#2/12 Sand Pack
-60	Grab	0.0, 0.0	Silt with sand, brown (7.5YR5/4), wet, nonplastic to low plasticity fines, subangular to subrounded sand. Approximately trace medium sand, 20% fine sand, 80% silt and trace clay. (Sample BRC-BW-11A-60 collected). Becomes wet @ 60' to 61.5' bgs.	4" 0.010 Slot PVC screen

BMI Landfill CAMU Investigation

Henderson, Nevada

Basic Remediation COMPANY

Log of Boring No. BW-11A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0			
					0.0			
					0.0			
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, very low to low plasticity fines, stiff. Approximately trace fine sand, 85% silt and 15% clay.	#2/12 Sand Pack
					0.0			
					0.0			
					0.0			4" 0.010 Slot PVC screen
					0.0			
-70					0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximately trace fine sand, 85% silt and 15% clay.	
					0.0			
					0.0			Threaded PVC Well Base Cap
					0.0		(Sample BRC-BW-11A-73 collected).	
					0.0		Clayey silt, brown (7.5YR5/4), very low to low plasticity fines, stiff to very stiff. Approximately trace fine sand, 85% silt and 15% clay.	Basal Bentonite Seal
					0.0			
					0.0			
					0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximately 85% silt and 15% clay.	
					0.0			
					0.0		Bottom of hole @ 80' bgs.	Native Backfill
-80								

Log of Boring No. BW-12A

BMI Landfill CAMU Investigation

Henderson, Nevada

Basic Remediation COMPANY

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 131.5 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: N/A
Depth to Water (ft. bgs): 68' bgs

Sample Type: 2" Split Spoon
Sample Interval Continuous

Logged By: M. Manwaring
Date Started: 03/25/05
Date Completed: 03/26/05

Monitoring Well Construction

Type of Surface Seal:	N/A	**Screen Slot Size:**	N/A
Blank Casing Type/Size:	N/A	**Top of Screen (ft. bgs):**	N/A
Screen Type/Size:	N/A	**Bottom of Screen (ft. bgs):**	N/A
Transition Sand Type:	N/A	**Type of Sand Pack:**	N/A

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0					0.0		SM: Silty sand with gravel, dark yellowish brown (10YR4/4), dry to slightly moist, sand and gravel is subrounded to subangular (volcanic coarse material), soil may be fill. Approximately 15% gravel, 15% coarse sand, 20% medium sand, 30% fine sand, 20% silt and trace clay. (Sample BRC-BW-12A-0 collected).	No Well
					0.0		Silty sand, yellowish brown (10YR5/6), slightly moist, as above. Approximately 5% gravel, 15% coarse sand, 25% medium sand, 35% fine sand, 20% silt and trace clay.	
					0.0			
					0.0		Silty sand with gravel, light yellowish brown (10YR6/4), dry, gravel and coarse sand material is likely volcanic, material is slightly cemented (the fine sand and silt sizes). Approximately 20% gravel, 20% coarse sand, 15% medium sand, 30% fine sand, 15% silt and trace clay.	
					1.6			
-10					0.4		Silty sand, brown (7.5YR5/4), dry, subrounded to subangular. Approximately 10% gravel, 15% coarse sand, 25% medium sand, 20% fine sand, 30% silt and trace clay.	
					0.0			
					0.4			
					0.8		Silty sand with gravel, light brown (7.5YR6/3), dry gravel is likely volcanic, subrounded to subangular (gravel is up to 2"-3" diameter). Approximately 15% gravel, 15% coarse sand, 20% medium sand, 25% fine sand, 25% silt and trace clay. (Sample BRC-BW-12A-18 collected).	
					0.0			
					0.4		Silty sand, brown (7.5YR5/4), dry, some coarse gravels (2"-3" diameter), subrounded to subangular, slight odor, loose. Approximately 10% gravel, 5% coarse, 15% medium, 30% fine sand, 30% silt and trace clay.	
-20					0.7			
					1.7		Silty sand with clay and gravel, dark yellowish brown (10YR4/4), abundant coarse angular to subangular gravel (3/4"-1" diameter), dry, slight odor, when wetted soil has low to medium plasticity. Approximately 20% gravel, 5% coarse sand, 10% medium sand, 25% fine sand, 30% silt and 10% clay.	

Depth Elevation (MSLD)	PID	Soil Description
	0.3	
	0.3	Silty sand with clay, dry, brown (7.5YR5/4), low to medium plasticity (when wetted), no odor. Approximately 10% gravel, 5% coarse sand, 10% medium sand, 25% fine sand, 35% silt and 15% trace clay. (Sample BRC-BW-12A-30 collected).
-30		
	0.4	
		SM: Silty sand with clay, brown (7.5YR5/4), dry to slightly moist, loose, low plasticity. Approximately 10% gravel, 5% coarse sand, 10% medium sand, 25% fine sand, 35% silt and 15% clay.
	1.1	
		SM: Silty sand, dark yellowish brown (10YR4/4), slightly moist, loose, some subrounded to subangular sand and gravel, low plasticity. Approximately 10% gravel, 5% coarse sand, 15% medium sand, 35% fine sand, 30% silt and 5% clay.
	0.3	
-40		
	0.0	SM: Silty sand, grayish brown (10YR5/2), dry, loose, abundant broken rock, gravel and coarse sand crushed rock. Approximately 10% gravel, 10% coarse sand, 15% medium sand, 25% fine sand, 30% silt and 10% clay.
	0.0	Silty sand with gravel, silt with abundant coarse sand and gravels, angular to subangular, dry matrix, light grayish brown (10YR6/2), rock color varies. Approximately 20% gravel, 15% coarse sand, 20% medium sand, 20% fine sand, 15% silt and 5% clay.
	0.3	
	0.0	Same as above to 50' bgs, brechia-volcanics (rock)
-50		(Sample BRC-BW-12A-50 collected)
	0.0	
	0.6	Silty sand with gravel, grayish brown (10YR5/2), dry to slighty moist, (below 52' bgs), slight odor. Approximately 20% gravel, 5% coarse sand, 10% medium sand, 25% fine sand, 30% silt and 10% clay.
	4.6	
		SM: Silty sand with clay, gray (5Y5/1), with gravels (subangular to angular) moist, slight odor, trace clay. Approximately 10% gravel, 10% coarse sand, 5% medium sand, 30% fine sand, 30% silt and 15% clay.
	4.2	
		ML: Clayey silt with sand and gravel, as above, grayish brown (10YR5/2). Approximately 25% gravel, 10% coarse sand, 5% medium sand, 20% fine sand, 25% silt and 15% clay.
	8.5	
-60		Clayey silt with sand, strong brown (7.5YR5/6), moist, to very moist, slight odor, trace light gray silt size lenses and gravels, firm to stiff, low plasticity. Approximately 5% gravel, trace coarse sand, 20% fine sand, 55% silt and 20% clay.
	2.0	

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					1.9			
					1.6			
							Clayey silt with sand. Approximately 5% gravel, 20% fine sand, 55% silt and 20% clay.	
					0.6		Silt with clay, trace gravel, brown (7.5YR4/4), wet to very moist, medium plasticity no odor, firm (stiff and moist, not wet below 70').	
-70					1.1			
					0.5			
					0.4		Clayey silt with sand, brown (7.5YR5/4), moist, as above, low to medium plasticity. Approximately trace gravel, 20% fine sand, 65% silt and 15% clay. Silt, wet zone (74'-74.5'), brown, with clay, medium plasticity.	
					1.6			
					2.6		Silt with clay, light gray (10YR7/2), very moist to wet, low plasticity fines. Approximately 10% gravel, 5% fine sand, 75% silt and 10% clay.	
					2.8		Clayey silt with sand, very moist to moist. Approximately trace gravel, 15% fine sand, 65% silt and 15% clay.	
-80					0.4			
							No recovery. Likely soft clayey silt.	
-90								

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
							No recovery. Likely soft clayey silt.	
-100								
-110								
					0.0		Clayey silt with trace angular gravel, brown (7.5YR5/4), trace angular gravel, very moist to saturated (not wet), stiff, low to medium plasticity, trace light brown to pale brown silt size/sand size gypsum crystals, (no odor). Approximately trace gravel, trace fine sand, 60% silt and 40% clay.	
-120					0.0			
					0.0		Clayey silt, yellowish brown (10YR5/4), trace angular gravel and sand size fractured rock fragments (gypsum), firm to stiff (some very stiff), medium to low plasticity, saturated to very moist (not wet), no odor. Approximately trace gravel, 70% silt and 30% clay.	
					0.0			
					0.0		Clayey silt, strong brown (7.5YR5/6), medium plasticity, stiff, saturated trace gypsum nodules. Approximately 70% silt and 30% clay.	
-130							End borehole at 130'bgs at 3-26-05.	

Log of Boring No. BW-12B

BMI Landfill CAMU Investigation

Henderson, Nevada

Basic Remediation Company

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 200 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-AA-BW-12
Depth to Water (ft. bgs): 59.5 ft bgs

Sample Type: N/A
Sample Interval Continuous

Logged By: A. Norris
Date Started: 02/13/05
Date Completed: 02/15/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	49 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	69 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12 sand

Depth Elevation (MSLD)	PID	Soil Description	Well Construction
0	0.0	SM: Silty sand with gravel, dark yellowish brown (10YR4/4), moist, subangular to surounded sand, nonplastic to very low plasticity silt, fine to coarse subangular to subrounded volcanic gravel (up to 3" long), locally partly indurated to weakly cemented, trace local volcanic cobbles. Approximately 15% gravel, 15% coarse sand, 20% medium sand, 30% fine sand and 20% silt.	Locking monument well box with concrete apron and ballards
	0.0		
	0.0	Silty sand with gravel, yellowish brown (10YR5/4), moist, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 15% coarse sand, 25% medium sand, 30% fine sand and 15% silt.	Cement Bentonite Grout
	0.0	Silty sand with gravel, pale brown (10YR6/3), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 15% coarse sand, 20% medium sand, 35% fine sand and 15% silt.	
-10	0.0	Silty sand with gravel, pale brown (10YR6/3), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 3" long). Approximately 15% gravel, 10% coarse sand, 20% medium sand, 40% fine sand and 15% silt.	4" Blank PVC Riser Pipe
	0.0	Silty sand with gravel, light yellowish brown (10YR6/4), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 2" long). Approximately 15% gravel, 15% coarse sand, 25% medium sand, 30% fine sand and 15% silt.	
	0.0	Silty sand with gravel, light yellowish brown (10YR6/4), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 2" long). Approximately 15% gravel, 15% coarse sand, 15% medium sand, 40% fine sand and 15% silt.	
	0.0		
-20	0.0	Silty sand with gravel, pale brown (10YR6/3), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 2" long). Approximately 15% gravel, 15% coarse sand, 20% medium sand, 35% fine sand and 15% silt.	
	0.0		
	0.0	Silty sand with gravel, pale brown (10YR6/3), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 3" long). Some local volcanic cobbles (up to 4" long). Approximately 20% gravel, 10% coarse sand, 20% medium sand, 35% fine sand and 15% silt.	

Column headings: Depth Elevation (MSLD) | Sample Type | Sample Interval | Sample Recovery (feet) | Sample Retained for Analysis | PID | Lithology | Soil Description | Well Construction

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0		Silty sand with gravel, pale brown (10YR6/3), dry, angular to subrounded sand, nonplastic silt, fine to coarse angular to subrounded volcanic gravel (up to 1.5" long), locally partly indurated to weakly cemented. Approximately 40% gravel, 15% coarse sand, 15% medium sand, 15% fine sand and 15% silt.	
-30								Cement Bentonite Grout
					0.0		Silty sand with gravel, very pale brown (10YR7/3), dry, subangular to subrounded sand, nonplastic silt, fine to coarse angular to subrounded volcanic gravel (up to 1" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 15% coarse sand, 20% medium sand, 35% fine sand and 15% silt.	
					0.0		Silty sand with gravel, very pale brown (10YR7/3), dry, angular to subrounded sand, nonplastic silt, fine to coarse sngular to subrounded volcanic gravel (up to 2" long), locally partly indurated to weakly cemented. Approximately 15% gravel, 15% coarse sand, 20% medium sand, 35% fine sand and 15% silt.	
					0.0			
-40					0.0		Silty sand with gravel, brown (10YR5/3), dry, angular to subrounded sand, nonplastic silt, fine to coarse angular to subrounded volcanic gravel (up to 1" long), locally partly indurated to weakly cemented. Approximately 15% gravel. 10% coarse sand, 20% medium sand, 35% fine sand and 20% silt.	Bentonite seal
					0.0			
					0.0		Silty sand with gravel, light brownish gray (10YR6/2), dry, subangular to subrounded sand, nonplastic silt, fine to coarse subangular to subrounded volcanic gravel (up to 1.5" long). Approximately 15% gravel, 15% coarse sand, 15% medium sand, 40% fine sand and 15% silt.	
								#2/12 Sand Pack
-50					0.0		Silty sand with gravel, light brownish gray (10YR6/2), dry, subangular to subrounded sand, fine to coarse subangular to subrounded volcanic gravel (up to 1.5" long) locally partly indurated to weakly cemented. Approximately 20% gravel, 10% coarse sand, 15% medium sand, 40% fine sand and 15% silt.	
					0.0			4" 0.010 Slot PVC screen
					0.0		Silty sand with gravel, light brownish gray (10YR6/2), dry, angular to subrounded sand, nonplastic silt, fine to coarse angular to subrounded volanic gravel (up to 3" long), locally partly indurated to weakly cemented. Some local volcanic cobbles. Approximtely 15% gravel, 10% coarse sand, 30% medium sand, 30% fine sand, and 15% silt.	
					1.0			
					2.0		Becomes wet and black (2.5Y2.5/1) @ 59.5' bgs to 60' bgs.	
-60					11.6		ML: Clayey silt, yellowish brown (10YR5/4), moist, stiff, nonplastic to low plasticity fines. Approximately trace fine sand, 80% silt and 20% clay.	#2/12 Sand Pack

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					1.3		Clayey silt, yellowish brown (10YR5/4), moist, stiff, nonplastic to low plasticity fines. Approximtely trace fine sand, 85% silt and 15% clay.	
					6.6			4" 0.010 Slot PVC screen
					0.0		Clayey silt, light gray (10YR7/2), moist, stiff to very stiff, low plasticity fines. Approximately trace fine sand, 65% silt and 35% clay.	
-70								Threaded PVC Well Base Cap
					0.0		Becomes yellowish brown (10YR5/4) @ 71' bgs.	
							Clayey silt, yellowish brown (10YR5/4), moist, stiff, nonplastic to low plasticity fines. Approximately trace fine sand, 85% silt and 15% clay.	
					0.0			
							Clayey silt, yellowish bown (10YR5/4), moist, stiff, nonplastic to low plasticity fines, locally partly indurated. Approximately 5% fine sand, 80% silt and 15% clay.	Basal Bentonite Seal
					0.0			
-80					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 75% silt and 25% clay.	
					0.0			
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines, subangular to subrounded sand. Approximately 5% fine sand, 70% silt and 25% clay.	
-90					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines, subangular to subrounded sand. Approximately trace medium sand, 5% fine sand, 75% silt and 20% clay.	
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately trace fine sand, 85% silt and 15% clay.	
					0.0			

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
							Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 80% silt and 20% clay.	
					0.0			
-100					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 80% silt and 20% clay.	
					0.0			
					0.0			
							Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 80% silt and 20% clay.	
					0.0			
					0.0		Clayey silt, light yellowish brown (10YR6/4), moist, stiff to very stiff, nonplastic to low plasticity fines. Approximately trace fine sand, 85% silt and 15% clay.	
-110								Basal Bentonite Seal
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 80% silt and 20% clay.	
					0.0		Clayey silt, strong brown (7.5YR5/6), moist, very low to low plasticity fines. Approximately trace fine sand, 80% silt and 20% clay.	
					0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, trace subangular to subrounded sand. Approximately 5% fine sand 75% silt and 20% clay.	
-120					1.0		Clayey silt with sand, brown (7.5YR5/4), wet, nonplastic to low plasticity fines, subangular to subrounded sand. Note: wet zone @ 120' to 130.5' bgs. Approximately trace medium sand, 15% fine sand, 70% silt and 15% clay.	
	Chemical				1.0		(Sample BRC-BW-12B-122 collected). Saturated Soil.	
					1.0		Clayey silt with sand, brown (7.5YR5/4), wet, nonplastic to low plasticity fines, subangular to subrounded sand. Approximtely 15% fine sand, 70% silt and 15% clay.	
					1.0			
-130							Clayey silt, brown (7.5YR5/4), wet, nonplastic to low plasticity fines, trace subangular to subrounded sand. Becomes moist @ 130.5' bgs. Approximately 5% fine sand, 75% silt and 20% clay.	
					0.0			
							Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, medium stiff to stiff.	

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
							Approximately trace fine sand, 80% silt and 20% clay.	
					0.0			
							Clayey silt, brown (7.5YR5/4), moist, stiff, very low to low plasticity fines. Approximately trace fine sand, 80% silt and 20% clay.	
					0.0			
							Clayey silt, brown (7.5YR5/4), moist, stiff to very stiff, very low to low plasticity fines. Approximately trace fine sand, 80% silt and 20% clay.	
					0.0			
-140					0.0		Clayey silt, brown (7.5YR5/4), moist, very stiff, low plasticity fines, some local silty clay seams. Approximately 60% silt and 40% clay.	
					0.0			
							Clayey silt, yellowish brown (10YR5/4), moist, very stiff, nonplastic to low plasticity fines. Approximately trace fine sand, 70% silt and 30% clay.	Basal Bentonite Seal
					0.0			
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, very stiff, nonplastic to low plasticity fines. Approximately trace fine sand, 85% silt and 15% clay.	
-150								
					0.0			
							Clayey silt, yellowish brown (10YR5/4), moist, very stiff, nonplastic to low plasticity fines. Approximately 85% silt and 15% clay.	
					0.0			
							Clayey silt, yellowish brown (10YR5/4), moist, very stiff, nonplastic to low plasticity fines. Approximately 85% silt and 15% clay.	
					0.0			
-160					0.0		Clayey silt, brown (7.5YR5/4), moist, stiff, very low to low plasticity fines. Approximately 65% silt and 35% clay.	
					0.0			
							Clayey silt, brown (7.5YR5/4), moist, stiff, very low to low plasticity fines. Approximately 70% silt and 30% clay.	
					0.0			

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0		Clayey silt, brown (7.5YR5/4), moist, stiff, very low to low plasticity fines. Approximately 75% silt and 25% clay.	
-170								
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 80% silt and 20% clay.	
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff to very stiff, very low to low plasticity fines. Approximately trace fine sand, 80% silt and 20% clay.	
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, very stiff, nonplastic to low plasticity fines. Approximately 80% silt and 20% clay.	
-180					0.0			Basal Bentonite Seal
							Clayey silt, brown (7.5YR5/4), moist, stiff, very low to low plasticity fines. Approximately 70% silt and 30% clay.	
					0.0		Clayey silt, brown (7.5YR5/4), moist, stiff, very low to low plasticity fines. Approximately 65% silt and 35% clay.	
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff to very stiff, nonplastic to low plasticity fines, trace local cementation. Approximately trace fine sand 85% silt and 15% clay.	
					0.0			
-190							Clayey silt, yellowish brown (10YR5/4), moist, stiff, very low to low plasticity fines. Approximately 70% silt and 30%.	
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, stiff to very stiff, nonplastic to low plasticity fines. Approximately 75% silt and 25% clay.	
					0.0		Clayey silt, brown (7.5YR5/4), moist, very stiff, nonplastic to low plasticity fines, trace subangular to subrounded sand, trace local cementation. Approximately trace medium sand, 5% fine sand, 80% silt and 15% clay.	
					0.0		Clayey silt, yellowish brown (10YR5/4), moist, very stiff, nonplastic to low plasticity fines, trace local cementation. Approximately trace fine sand, 80% silt and 20% clay. Bottom of hole @ 200' bgs. End Borehole @ 200 ft bgs.	
-200								

Log of Boring No. BW-13A
BMI Landfill CAMU Investigation
Henderson, Nevada

Basic Remediation Company

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 86.5 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-MCF-BW-13
Depth to Water (ft. bgs): 29' bgs

Sample Type: 2" Split Spoon
Sample Interval Continuous

Logged By: M. Manwaring
Date Started: 03/27/05
Date Completed: 03/28/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	59 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	69 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0					0.4 0.2		SM: Silty sand with gravel, dark grayish brown (10YR4/2), silghtly moist to dry, loose, (0"-2" asphalt), hand auger to 3' (refusal in cobbles/gravels begin rotosonic - slowly to 10' bgs), fill to about 4' bgs/road base). Approximately 15% gravel, 10% coarse sand, 20% medium sand, 40% fine sand and 15% silt. (Sample BRC-BW-13A-0 collected).	Flush mount well box with concrete apron
							Silty sand with gravel, dark yellowish brown (10YR3/4), dry to slightly moist, angular, becoming angular volcanic rock below 7' bgs. (gray to strong brown). Approximately 15% gravel, 10% coarse sand, 15% medium sand, 30% fine sand, 30% silt and trace clay. Approximately 10% gravel, 15% coarse sand, 20% medium sand, 25% fine sand, 30% silt and trace clay.	Cement Bentonite Grout
-10							COBBLES: Cobbles/Boulders, gray (7.5YR5/1), vessicular volcanic rock, drill bit crushed rock into fine silt size, dry. Approximately 100% cobbles and boulders.	4" Blank PVC Riser Pipe
					0.0 0.0		SM: Silty sand with gravel, yellowish brown (10YR5/4), gravels are angular to subangular volcanics (up to 2"-3" diameter). Approximately 25% gravel, 5% coarse sand, 15% medium sand, 25% fine sand, 30% silt and trace clay.	Bentonite seal
					0.0		Silty sand with gravel, brown (10YR5/3), subangular to angular (broken) gravels (volcanics). Approximately 20% gravel, 5% coarse sand, 15% medium sand, 30% fine sand, 30% silt and trace clay.	#2/12 Sand Pack
-20					0.0 0.0		GP-GM: Gravel with silt and sand, large gravels and silty sand, large subangular to subrounded, dry, gray (10YR5/1-6/1), when cobbles cored, broken material silty and angular gravel. Approximately 75% gravel and cobbles, 5% coarse sand, 10% fine sand and 10% silt.	
					0.0		Silty gravel with sand, gray (10YR6/1), subangular to angular (broken) moist at 27' bgs, increased silt and fine sand, wet, at 29' bgs (casing to 30') [water lever meter indicates water to 27'8" bgs]. Approximately 60% gravel, 20% fine sand, 20% silt and trace clay. Becomming	4" 0.010 Slot PVC screen

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
							moist at 25' bgs.	
					0.4		SM: Silty sand with gravel, very moist, lenses of medium sand, very moist to wet, brown (10YR4/3), gravel is subangular to angular, firm. Approximately 20% gravel, 15% coarse sand, 10% medium sand, 20% fine sand, 30% silt and 5% clay.	
					1.8		Approximately 20% gravel, 5% coarse sand, 15% medium sand 30% fine sand, 30% silt and trace clay. (Sample BRC-BW-13A-30 collected).	
-30					2.1			4" 0.010 Slot PVC screen
					0.0			
					NR			
					NR			
							ML/SM: Sandy silt with gravel, brown (7.5YR4/3), very moist, increased fines (silt and clay), driller had trouble holding samples from ~30' - 36' in corebarrel. Approximately 10% gravel, 5% coarse sand, 15% medium sand, 20% fine sand, 40% silt and 10% clay.	#2/12 Sand Pack
					1.6			Threaded PVC Well Base Cap
-40								
					2.8		Sandy silt with clay, several thin (<0.3") fine to medium sand lenses, dark yellowish brown (10YR4/4), trace silty gypsum, small gravel are angular volcanics. Approximately 10% gravel, 5% coarse sand, 10% medium sand, 25% fine sand, 40% silt and 10% clay.	
					7.5			
					29			
					44			Basal Bentonite Seal
							ML: Sandy silt with clay, subangular to subrounded, trace thin (0.3" - 0.5"), fine to medium sand lenses (subrounded volcanic sands), very moist (not wet) but saturated, no odors. Approximately 5% gravel, 5% coarse sand, 10% medium sand, 20% fine sand, 50% silt and 10% clay.	
					6.9			
-50							(Casing advanced to 50' bgs).	
							Sandy silt with clay, brown silt (7.5YR4/4), most pronounced at 51.5' - 52.5', very moist to saturated (wet), silt is firm, sand is loose, trace clay, low to medium plasticity, some local sand lenses (<1" thick). Approximately trace gravel, 5% coarse sand, 10% medium sand, 20% fine sand, 55% silt and 10% clay. (Sample BRC-BW-13A-53 collected).	
							Clayey silt with sand, brown (7.5YR4/3), medium plasticity, saturated (wet 56' - 57' bgs). Approximately trace gravel, 15% fine sand, 65% silt and 20% clay.	
					0.5			
							Silt with clay, gravel and sand, (chert/gypsum?), brocken by corebarrel, moist. Approximately 15% gravel, 10% medium sand, 15% fine sand, 55% silt and 10% clay.	
-60					0.3		Clayey silt, strong brown (7.5YR5/6), medium plasticity clay, becomes wet @ 60' - 63' bgs, slightly firm. Approximately 10% gravel, 10% fine sand, 60% silt and 20% clay.	

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0		Clayey silt with gravel, brown (7.5YR5/4), moist to very moist, (saturated not wet), low plasticity, firm to stiff, some gypsum content. Approximately 15% gravel, 10% fine sand, 60% silt and 15% clay.	
					0.0		Silt with clay and gravel, yellowish brown (10YR5/4), moist, firm. Approxiamtely 15% gravel, 75% silt and 10% clay.	
							Silt, brown (7.5YR5/4), low plasticity, firm to stiff, some gypsum content. Approximately 10% gravel, 10% fine sand, 75% silt and 5% clay.	
-70					0.0			
					0.0		Clayey silt with gravel, brown (7.5YR5/4). Approximately 15% gravel, 10% fine sand, 60% silt and 15% clay.	
					0.0		Clayey silt, sand and trace gravels, (gypsum fragments), angular to subangular, firm to stiff, moist (not wet), low plasticity. Approximately 5% gravel, 10% fine sand, 70% silt and 15% clay.	
					0.0			
							Clayey silt, trace gravel size gypsum fragments, (angular) visible crystals on some fragments firm to stiff, low plasticity, moist. Approximately trace gravel, 10% fine sand, 75% silt and 15% clay.	
					0.0			
-80					0.0		Clayey silt, brown (7.5YR5/3), moist, low plasticity, firm to stiff. Approximately 5% fine gravel, 5% medium sand, 10% fine sand, 65% silt and 15% clay. (Casing to 80' bgs)	
					0.0			
							Clayey silt, trace angular gravel size rocks (gypsum), moist, firm/stiff. Approximately 5% gravel, 10% fine sand, 70% silt and 15% clay. (Sample BRC-BW-13A-85 collected). Bottom of hole @ 86.5 ft bgs.	

Log of Boring No. BW-13B
BMI Landfill CAMU Investigation
Henderson, Nevada

Basic Remediation Company

Drilling Method: Rotary Sonic
Drilling Equipment: GEFCO
Drilling Contractor: Water Development Corporation
Driller: Mike Wilkerson

Borehole Total Depth: 60 ft bgs
Borehole Diameter: 7 5/8" O.D. Casing / 7 7/8" O.D. Bit
Well ID: GW-AA-BW-13
Depth to Water (ft. bgs): 29' bgs

Sample Type: N/A
Sample Interval Continuous

Logged By: A. Norris
Date Started: 03/29/05
Date Completed: 03/30/05

Monitoring Well Construction

Type of Surface Seal:	Bentonite-Grout	**Screen Slot Size:**	0.010 in
Blank Casing Type/Size:	4" Sch 80 PVC	**Top of Screen (ft. bgs):**	18 ft bgs
Screen Type/Size:	4" Sch 80 PVC	**Bottom of Screen (ft. bgs):**	38 ft bgs
Transition Sand Type:	N/A	**Type of Sand Pack:**	#2/12

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
0					0.0		SM: Silty sand with gravel, dark yellowish brown (10YR4/4), moist to dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long), locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 30% fine sand and 20% silt. Note: Asphalt pavement (5" thick) @ ground surface.	Locking monument well box with concrete apron and ballards
					0.0		Silty sand with gravel, dark yellowish brown (10YR4/4), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented, trace local volcanic cobbles (up to 4" long). Approximately 35% gravel, 10% coarse sand, 10% medium sand, 30% fine sand and 15% silt.	Cement Bentonite Grout
					0.0			
					0.0		Silty sand with gravel, brown (10YR5/3), dry, subangular to surounded sand, nonplastic silt, subangular to subronded fine to coarse volcanic gravel (up to 3" long), locally partly indurated to weakly cemented. Approximately 30% gravel, 10% coarse sand, 10% medium sand, 30% fine sand and 20% silt.	
-10					0.0			4" Blank PVC Riser Pipe
					0.0 0.0		Silty sand with gravel, light brownish gray (10YR6/2), dry, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), some local volcanic cobbles (up to 5" long). Approximately 30% gravel, 15% coarse sand, 10% medium sand, 30% fine sand and 15% silt.	Bentonite seal
					0.0 0.0		Silty sand with gravel, dark grayish (10YR4/2), dry to moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 3" long), trace local volcanic cobbles (up to 5" long). Approximatley 30% gravel, 15% coarse sand, 10% medium sand, 25% fine sand and 20% silt. Becomes yellowish brown (10YR5/4) @ 17' bgs.	
					0.0 0.0			#2/12 Sand Pack
							ML: Silt with sand, brown (10YR4/3), moist, nonplastic to very low plasticity silt, subangular to subrounded sand. Approximately 5% medium sand, 15% fine sand and 80% silt.	
-20					0.0		SM: Silty sand with gravel, light brownish gray (10YR6/2), moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long), locally partly indurated to weakly cemented. Approximately 20% gravel, 15% coarse sand, 15% medium sand, 35% fine sand and 15% silt.	
					0.0			
							Silty sand with gravel, light brownish gray (10YR6/2), moist, subangular to subrounded sand, nonplastic silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long), locally partly indurated to weakly cemented, trace local strong cementation. Approximately 20% gravel,	4" 0.010 Slot PVC screen

Depth Elevation (MSLD)	Sample Type	Sample Interval	Sample Recovery (feet)	Sample Retained for Analysis	PID	Lithology	Soil Description	Well Construction
					0.0		15% coarse sand, 10% medium sand, 25% fine sand and 30% silt.	
-30					0.0		Silty sand with gravel, light brownish gray (10YR6/2), wet, subangular to subrounded sand, nonplastic to very low plasticity silt, fine to coarse subangular to subrounded volcanic gravel (up to 3" long), locally partly indurated to weakly cemented, trace local volcanic cobbles (up to 5" long). Approximately 20% gravel, 10% coarse sand, 10% medium sand, 35% fine sand and 25% silt. First water @ 29' bgs.	4" 0.010 Slot PVC screen
					0.0		Silty sand with gravel, brown (7.5YR5/2), wet, subangular to subrounded sand, nonplastic to very low plasticity silt, subangular to subrounded fine to coarse volcanic gravel (up to 2" long), locally cemented. Approximately 15% gravel, 10% coarse sand, 10% medium sand, 40% fine sand and 25% silt.	
					0.0		ML: Silt with sand, pink (7.5YR7/3), moist, nonplastic to very low plasticity silt, subangular to subrounded sand, very stiff to hard. Approximately 20% fine sand and 80% silt.	#2/12 Sand Pack
					0.0 0.0		Silt wit clay, brown (7.5YR5/4), moist, very low to low plasticity fines, subangular to subrounded sand, stiff. Approximately 10% fine sand, 80% silt and 10% clay.	Threaded PVC Well Base Cap
-40					0.0			
							Silt with clay, brown (7.5YR5/4), moist, very low to low plasticity fines, subangular to subrounded sand, stiff. Approximately 10% fine sand, 80% silt and 10% clay.	
					0.0		Silt with clay and sand, brown (7.5YR5/4), moist, very low to low plasticity fines, subangular to subrounded sand, very stiff. Approximately 5% gravel, 5% coarse sand 5% medium sand, 10% fine sand, 65% silt and 10% clay.	Basal Bentonite Seal
					0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, very stiff. Approximately 5% fine sand, 80% silt and 15% clay.	
-50					0.0		Clayey silt, strong brown (7.5YR5/6), moist, very low to low plasticity fines, stiff. Approximately 80% silt and 20% clay.	
					0.0 0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff to very stiff. Approximately 85% silt and 15% clay.	
					0.0		Clayey silt, brown (7.5YR5/4), moist, very low to low plasticity fines, stiff, subangular to subrounded sand. Approximately 5% coarse sand, 5% medium sand, 5% fine sand, 70% silt and 15% clay. 1" thick Sandy silt seam @59' bgs. Wet seam bound above and below by moist clayey silt. Seam: Approximately 40% fine to coarse sand and 60% silt. 59.1' to 60' bgs: Clayey silt. brown (7.5YR5/4), moist, very low to low plasticity fines, stiff. Approximtaely 80% silt and 20% clay.	
-60					0.0		Bottom of hole @ 60' bgs.	