ENVIRONMENTAL SERVICES, INC.

Clean Harbors Environmental Services, LLC

Lone Mountain Facility

Waynoka, Oklahoma

RCRA/HSWA Permit Renewal Application

Volume 11

October 1, 2020

Lone Mountain RCRA Permit Renewal

Volume 11

Volume 11 Contents:

6.4

Design Engineering Report

Landfill Cell 15

Table of Contents

1.0 Landfill Cell 15

Landfill Cell 15 was originally permitted with Cells 12 through 14. The design of Cell 15 was subsequently modified in 2014. As of this time (June 2020), Cell 15 Subcells 9 through 13 are currently active. Subcells 1-8 are closed. In addition, subcell 14 has been constructed, but is not yet in use. A detailed discussion of the design, geotechnical considerations, construction methods, and operational procedures to be used in Cell 15 are provided in the Design and Engineering Report (DER). The original DER Report for Cell 15 is contained herein as Appendix A, and the DER Report for the most recent Cell 15 expansion is contained herein as Appendix B.

The current permitted capacity of Landfill Cell 15 is 8,065,500 cubic yards, based on the Permit Modification approved by DEQ in August 2015.

The specific construction details for Cell 15 may be found in the DER, Run-On Control System, and Construction Quality Assurance (CQA) Plan. The following information from the Cell 15 application not found in those documents has been provided.

1.1 Landfill Liner System

The base liner system for Cell 15 is designed and will be constructed with a triple liner/leachate collection system configuration. The base liner system was modified and subsequently approved by ODEQ in 2010. The proposed Cell 15 base liner system for subcells (6 through 22) is composed of (from bottom to top):

- 3-ft thick compacted clay liner (k ≤ 1 x 10-7 cm/s) ;
- bottom 60-mil HDPE textured geomembrane;
- bottom double-sided geocomposite leak detection drainage layer;
- middle 60-mil HDPE textured geomembrane;
- GCL;
- upper 60-mil HDPE textured geomembrane;
- upper double-sided geocomposite leachate collection drainage layer; and
- 2-ft thick protective cover layer.

1.1.1 Liner Location Relative to High Water Table

Cell 15 will be constructed above ground. The DER indicates that the lowest elevation at which the waste will be disposed of in Cell 15 is at an elevation of 1,365.5 feet above mean sea level in the northeast corner of the landfill. The groundwater elevation data indicates a potentiometric surface elevation of approximately 1,360 feet above mean sea level beneath the northeast corner of Cell 15 and higher in the western and southern portions of the cell. The cell has been designed so that the bottoms of the lowest sumps are indicated to be below the high water table, the Lone Mountain Facility will submit a plan to the DEQ to raise the elevation of the floor/sumps, where necessary.

1.1.2 Loads of Liner System

A discussion of the stresses considered in defining liner strength requirements is included in the DER (dated June 2014) for Landfill Cell 15, contained in Appendix B.

1.1.3 Liner System Coverage

The liner system extends to the top of the cell embankment and is anchored in a trench. The stormwater containment (run-off control) system, consisting of contours and/or ditches inside the perimeter of the active subcells and phase of the cell, will be operated with an allowance of one foot of depth ("freeboard") above that required to contain the precipitation falling on the active portions of the landfill from a 24-hour, 25-year storm event. Thus, the liner will cover all areas likely to contact the waste.

1.1.4 Liner Exposure Prevention

The uppermost HDPE liner will be exposed to the elements for a short period of time before waste is placed over it. In order to minimize degradation due to ultraviolet rays from sunlight, the liner contains up to three percent carbon black anti-oxidants. The bottom of the cell with a protective layer of select/screened waste or soils to guard against puncture damage. The protective layer will gradually be extended up the sides of the cell, ahead of actual waste placement, to continue this protection. As a consequence, there is minimal risk of damage to the liners from climatic exposure or mechanical sources.

1.1.5 Liner Repairs During Operations

Any liner repairs needed during cell operation will occur in accordance with the Construction Quality Assurance (CQA) plan for landfill cell construction and closure in effect at the time of repair.

1.1.6 Synthetic Liner

The synthetic liners shall consist of 60mil high density polyethylene (HDPE) sheeting. The liner materials are composed of new, first quality products designed and manufactured specifically for this type of application and have been satisfactorily demonstrated by prior use to be suitable and durable for such purposes.

Samples of HDPE liner material and other structural components such as drainage net, geotextile (filter fabric), and HDPE pipe have been tested for compatibility with leachate. These tests verify that the landfill liner system materials of construction are compatible with the wastes and leachate found inside the cell.

Because leachate does not adversely affect the integrity of the liner system, the stress calculations provide adequate assurances of sufficient liner strength for Cell 15 as discussed in the DER.

1.1.7 Clay Liner

The three-foot clay liner will be constructed with on-site borrow materials, if possible, although off-site borrow materials will be used, as necessary. The approved CQA plan for the facility includes pre-placemat specifications for soils and a discussion of the procedures used and the testing performed to ensure that the required hydraulic conductivity is achieved in the constructed clay liner.

Landfill Cell 15 will contain wastes similar as those disposed in Cells 10 through 14. These wastes and associated leachate do not present any compatibility problems with respect to the clay liner system.

1.2 Leachate Collection, Detection, and Removal System

The design of the leachate collection, removal, and detection system for Cell 15 includes sump drainage areas or leachate collection areas for each subcell constructed. Each section will be sloped on flat surfaces at a minimum grade of two percent towards leachate collection "ditches" which are themselves graded (also at a minimum one percent slope) to a sump at the low point of each area. Leachate collected within each sump will be removed via leachate removal pipes nested within large diameter HDPE pipes in the uppermost, and bottom sumps that extend from the sumps up the embankment slope to the top of the embankment. The leachate systems are each designed in accordance with regulatory requirements and guidance. The uppermost system will be used for the collection and removal of expected leachate and rainfall due to direct precipitation into the cell. The bottom system is used for detection, collection, and removal of leakage (if any) past the upper two liners and for removal of residual liquid resulting from initial construction, condensation, and other miscellaneous infiltration.

For additional information, the reader is advised to see the DER for Landfill Cell 15 (dated June 2014) which includes detailed discussions and drawings of the leachate collection, detection, and removal systems.

1.3 Foundation

The foundation preparation will consist of removing excessively wet and/or soft (unsuitable) soils. This material will be removed down to more competent natural soils. Foundation preparation also involves removing vegetation and other organic matter, as well as debris and deleterious materials from the area. The ground surface to receive the clay liner and embankment materials will be prepared in accordance with the facility's approved CQA plan at the time of construction.

Previously approved permit applications and modifications (most recently, the permit modification for Cells 12 through 14 and Cell 15) included details concerning subsurface exploration and laboratory testing of the soils. Those details are not repeated here. The results of the additional geotechnical Investigation performed for the location of Cell 15 are included in the DER.

The test results discussed in the DER indicate unconfined compressive strengths ranging form 560 to 5,460 PSF for the overburden soil, with bedrock values ranging from 8,050 to 36,500 pounds per square foot. These strengths are consistent with those previously encountered and used in earlier investigations for landfill cells at the Lone Mountain Facility.

Previously approved permit applications discuss the settlement analysis performed by Chen and Associates, Inc. and detailed in their August 1, 1986 report. Additional settlement analysis performed for the Cell 15 permit modification request is discussed in the DER. The allowable bearing capacity of the clay liner is 2,000 pounds per square foot for live loads and dead loads and 3,000 pounds per square foot for impact loads. The DER presents the calculations of the clay bearing capacity.

The original, approved application for Cells 12 through 15 contained a stability analysis performed by Chen and Associates, Inc. Supplemental stability analysis, specific to the Landfill Cell 15 design, is provided in the DER. Stability calculations indicate that the embankment has a static safety factor under long term conditions of 1.8 with a dynamic safety factor of 1.6. EPA recommends a static safety factor of 1.5 and a dynamic safety factor of 1.3. The above analyses indicate that the design of Landfill Cell 15 exceeds these recommendations.

1.4 Run-On/Run-Off Controls

A complete discussion of the run-on/run-off controls may be found in the Run-On Control System, Landfill Operations Procedures, and the Cell 15 DER sections of the permit application.

1.5 Construction Quality Assurance Plan

CHESI has developed a Construction Quality Assurance (CQA) Plan document to ensure that the construction and closure of all landfill cells complies with the Oklahoma Department of Environmental Quality (DEQ) and EPA regulations. The CQA Plan discusses project organization, responsibilities of personnel, and qualifications for each position. The inspection, sampling, and testing activities are associated with construction are also defined. The CQA Plan also details the documentation required to provide evidence of adherence to the plan. When the various components of the plan are combined, the resultant effort produces a well-constructed and operational project. The CQA Plan is an evolving document, with modifications made when technologies change, regulations change, and hands-on experience provides better or more efficient means of monitoring and assuring quality construction is maintained.

1.6 Construction Schedule

A construction schedule is dependent upon many factors and cannot be fully developed until the approximate date of permit approval or capacity depletion is known. A construction schedule is developed using input regarding materials availability, contractor capabilities, expected seasonal delays, waste receipt volumes, and other information relevant to construction.

Appendix A

Design Engineering Report – Cell 15
Dated May 1999

CONSULTANTS/ENGINEERS
HA&L
ENGINEERING
INCORPORATED
6771 SOUTH 900 EAST
MIDVALE, UTAH 84047
(801) 566-5599
FAX 801-566-5581

May 18, 1999

Safety-Kleen, Inc.
5665 Flat Iron Parkway
Boulder, Colorado 80301

Attention: Mr. Don Durr

Subject: Lone Mountain Facility
Typical Closure Sections & General Specifications

Gentlemen:

As requested, attached are the typical closure sections proposed for landfill cell closures at the Lone Mountain Facility. The typical sections have been modified to flatten the exterior slopes around the closure caps from 2H:1V to a maximum slope of 2.5H:1V. Attached are two design drawings which illustrate the proposed design for the closures; one drawing reflects using a geosynthetic clay liner (GCL) for the soil portion of the composite liner system of the cap, and the other drawing reflects using a 2-foot thick compacted clay liner for the soil portion of the composite liner system of the cap. Some general specifications that should be included in the construction documents pertaining to cell closures at the Lone Mountain Facility (associated with the typical sections included herein) are summarized below.

HDPE Liner Flap, Drainage Net, and Filter Fabric

In order to provide a more free flowing discharge from the drainage net into the riprap erosion protective rock covering (Type V Riprap), the HDPE Textured Liner Flap should extend to a position approximately midway through the thickness of the Type II Granular Filter material. Therefore, we recommend that the tolerance specification for the HDPE Textured Liner Flap be as follows (see Note 3 on the drawings):

> The 60 mil HDPE Textured Liner Flap shall extend into the Type II Granular Filter material a distance of between 4 to 6 inches (slope distance). This Liner Flap shall be placed on a slope equal to the top of the closure slope (i.e. 10 percent or flatter, depending on the designation for the particular cap) and shall not extend down the 2.5H:1V slope.

The tolerance specification for the point at which the Drainage Net and Filter Fabric should terminate should be as follows (see Note 4 on the drawing):

The Drainage Net and Filter Fabric shall extend into the Type II Granular Filter Material a distance of between 1 inch beyond the termination of the 60 mil HDPE Textured Liner Flap and the interface between the Type II and Type V materials. These Drainage Net and Filter Fabric materials shall be placed on a slope equal to the top of the closure slope (i.e. 10 percent or flatter, depending on the designation for the particular cap) and shall not extend down the 2.5H:1V slope.

Type I Granular Filter

The thickness for the Type I Granular Filter material is specified to be 3 inches. It is important that this thickness not exceed 3 inches. Therefore, the recommended thickness tolerance for the Type I material is that the in-place thickness shall be between 2 to 3 inches.

Based on information provided by Applied Geotechnical Engineering Consultants (AGEC), the internal coefficient of friction for Type I Granular Filter must be at least 38 degrees.

The gradation for the Type I Granular Filter shall be as follows:

TYPE I GRANULAR FILTER

Sieve Size	Percent Passing
3/8	100
#4	95-100
#16	45-85
#50	5-30
#100	0-10
#200	0-3

It will be important during construction to verify that the Type I Granular Filter Material placed meets the grain size criteria. We recommend that, as a minimum, one grain size distribution test for the Type I Granular Filter material be conducted for every 1,000 cubic feet of material placed, with no less than 3 tests per side area of each closure cap. It will also be important to observe the material as it is placed so that a change in the material can be detected and tests conducted to verify compliance.

Type II Granular Filter

The thickness for the Type II Granular Filter material is specified to be 4 inches. The recommended thickness tolerance for the Type II material is that the in-place thickness shall be between 3 to 5 inches.

Based on information provided by AGEC, the internal coefficient of friction for Type II Granular Filter must be at least 38 degrees.

The gradation for the Type II Granular Filter has been determined by AGEC such that the material will have a permeability in excess of 4 cm/sec and will also serve as a filter medium between the Type I Granular Filter and the Type V Riprap. This is a revised specification from that used previously at the Lone Mountain Facility for Type II Granular Filter material. Based on AGEC's testing, the gradation for the Type II Granular Filter shall be as follows:

TYPE II GRANULAR FILTER

Sieve Size	Percent Passing
3 inch	90-100
3/4 inch	35-70
#4	0-20
#16	0-3
#200	0-1

It will be important during construction to verify that the Type II Granular Filter Material placed meets the grain size criteria. We recommend that, as a minimum, one grain size distribution test for the Type II Granular Filter material be conducted for every 1,000 cubic feet of material placed, with no less than 3 tests per side area of each closure cap. It will also be important to observe the material as it is placed so that a change in the material can be detected and tests conducted to verify compliance.

Type V Riprap

The thickness for the Type V Riprap is specified to be 6 inches. The recommended thickness tolerance for the Type V material is that the in-place thickness shall be a minimum of 6 inches, with the additional stipulation that the average slope of the surface of the riprap be maintained at 2.5H:1V or flatter.

Based on information provided by AGEC, the internal coefficient of friction for Type V Riprap must be at least 38 degrees.

The gradation for the Type V Riprap has been revised, from that used previously at the Lone Mountain Facility, to include a limitation on the allowable fines in the material. The gradation for the Type V Riprap shall be as follows:

TYPE V RIPRAP

Riprap Designation	% Smaller Than Given Size By Weight	Intermediate Rock*		D_{50}** inches
		Weight lbs	Dimension inches	
Type V	70-100	43	8	4
	50-70	18	6	
	35-50	5.3	4	
	2-10	0.7	2	
	0-1	0.04	3/4	

* Dimension is based on volume of a cube and Specific Gravity = 2.3 for Type V Riprap.

** D_{50} = Nominal particle size

It will be important during construction to verify that the Type V Riprap placed meets the grain size criteria. We recommend that, as a minimum, 3 grain size distribution tests for the Type V Riprap material be conducted per closure project. It will also be important to observe the material as it is placed so that a change in the material can be detected and tests conducted to verify compliance

If you have any questions regarding the information included herein, please call.

Sincerely,

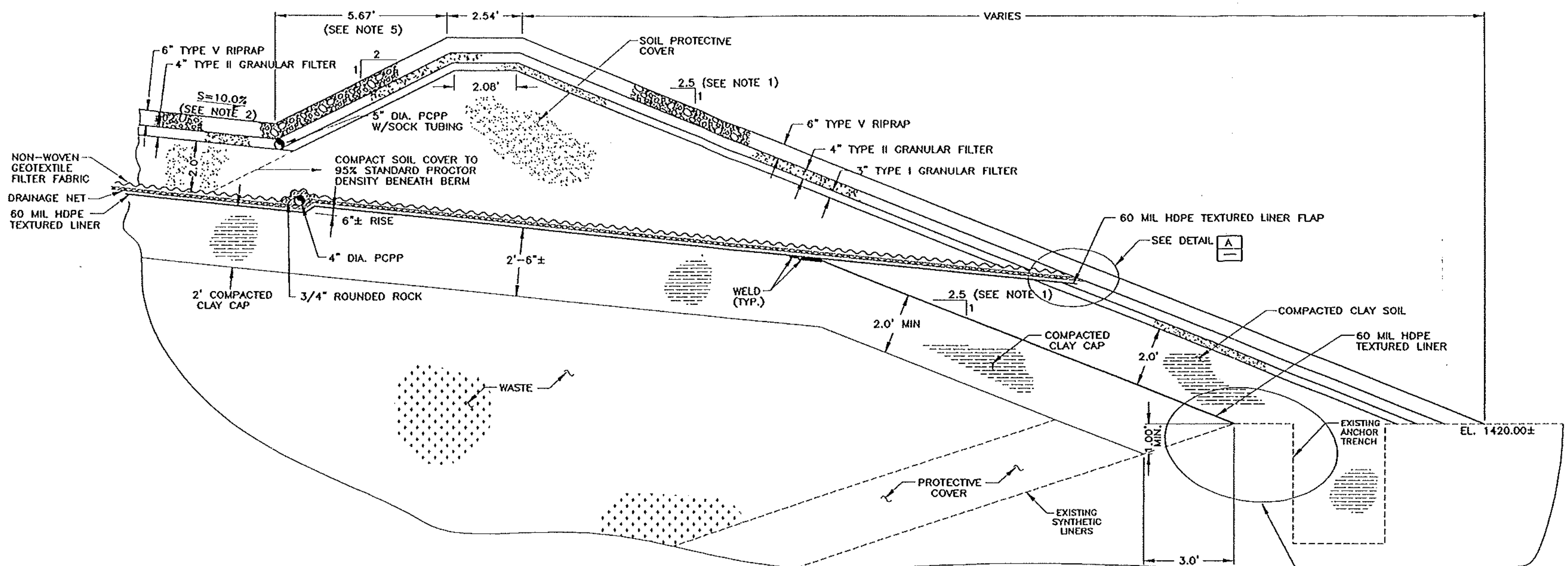
HA&L ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
MARVIN E
ALLEN
14802
OKLAHOMA

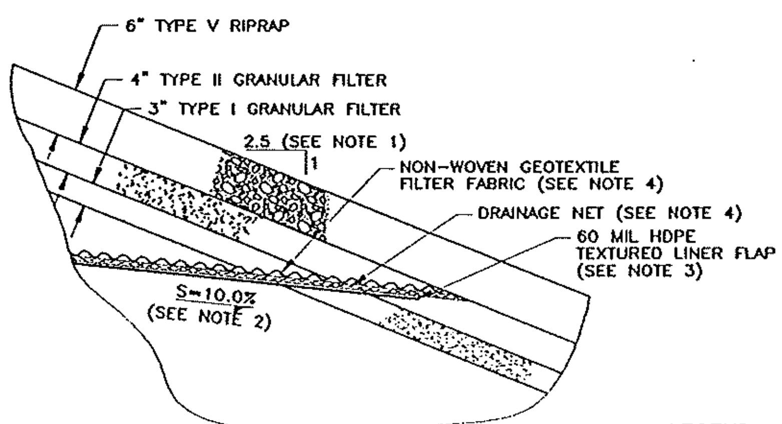
Marvin E. Allen

Marvin E. Allen, P.E.
Principal

attachments



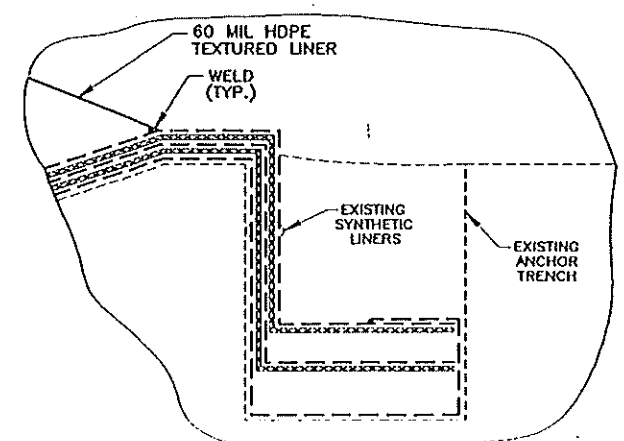
TYPICAL CLOSURE SECTION



DETAIL A

LEGEND

PCPP – PERFORATED CORRUGATED POLYETHYLENE PIPE



CAP HDPE LINER TIE IN
DETAIL AT ANCHOR TRENCH B

NOTES:

1. EXTERIOR SLOPES OF CLOSURES SHALL BE 2.5H:1V OR FLATTER.
2. TOP OF CLOSURE SLOPES SHALL BE 10% OR FLATTER.
3. THE 60 MIL HDPE TEXTURED LINER FLAP SHALL EXTEND INTO THE TYPE II GRANULAR FILTER MATERIAL A DISTANCE OF BETWEEN 4 TO 6 INCHES (SLOPE DISTANCE). THIS LINER FLAP SHALL BE PLACED ON A SLOPE EQUAL TO THE TOP OF CLOSURE SLOPE (AS NOTED) AND SHALL NOT EXTEND DOWN THE 2.5H:1V SLOPE.
4. THE DRAINAGE NET AND FILTER FABRIC SHALL EXTEND INTO THE TYPE II MATERIAL A DISTANCE OF BETWEEN 1 INCH BEYOND THE TERMINATION OF THE 60 MIL HDPE TEXTURED LINER FLAP AND THE INTERFACE BETWEEN THE TYPE II AND TYPE V MATERIALS. THESE DRAINAGE NET AND FILTER FABRIC MATERIALS SHALL BE PLACED ON A SLOPE EQUAL TO THE TOP OF CLOSURE SLOPE (AS NOTED) AND SHALL NOT EXTEND DOWN THE 2.5H:1V SLOPE.
5. DIMENSION MAY VARY DEPENDING ON HEIGHT REQUIRED FOR BERM TO CONTROL RUNOFF FROM THE CLOSURE SURFACE.

HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City, Utah

PROJECT ENGINEER

REGISTERED PROFESSIONAL ENGINEER — MARTIN E. ALLEN — 5/18/99

DESIGNED	MEA	3			
DRAFTED	SS	2			
CHECKED	MEA	1			
DATE	MAY 1999	NO.	DATE	REVISIONS	BY / APVD.

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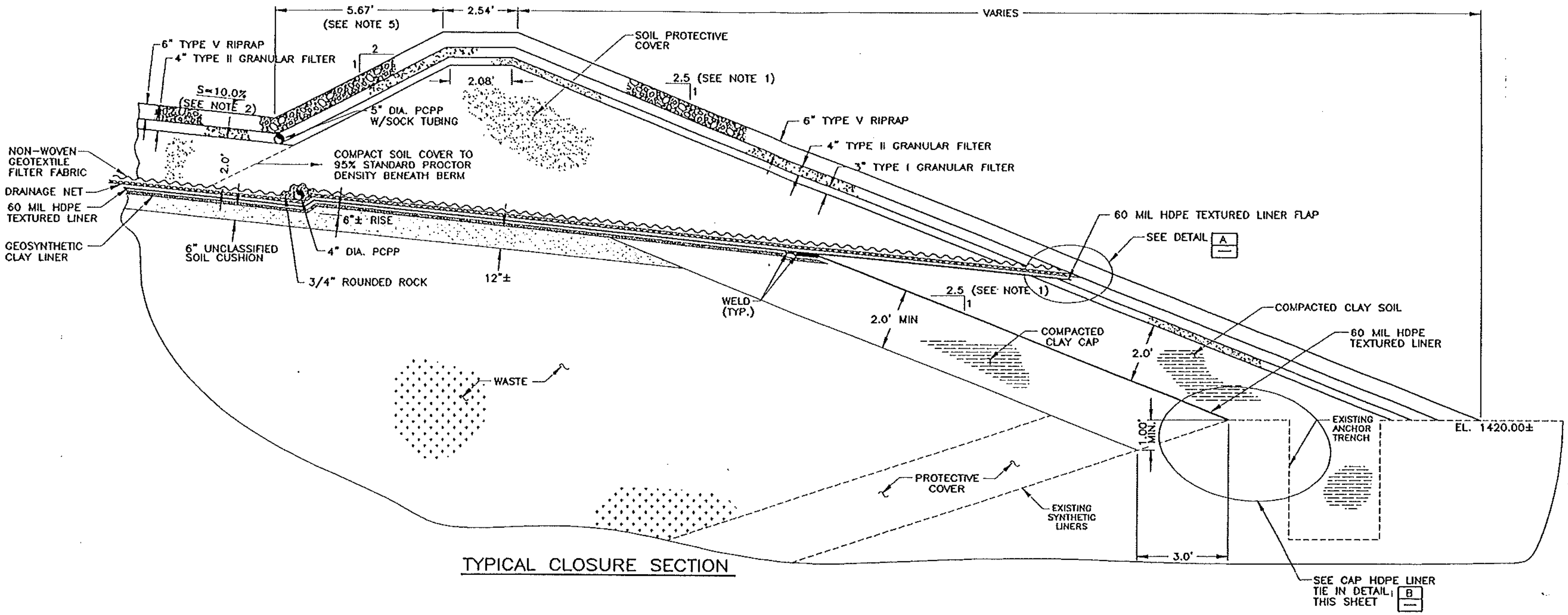
VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

safety-kleen corp.

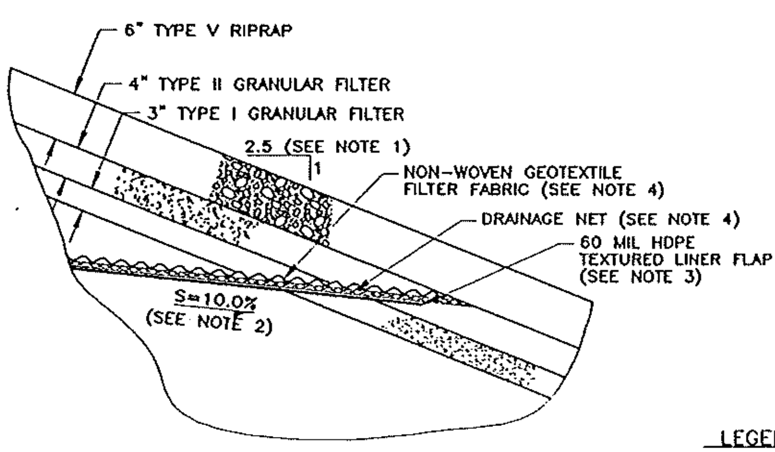
LONE MOUNTAIN FACILITY
TYPICAL CLOSURE SECTION
CLAY LINER OPTION

SHEET NO. 1 OF 1

64-76-100



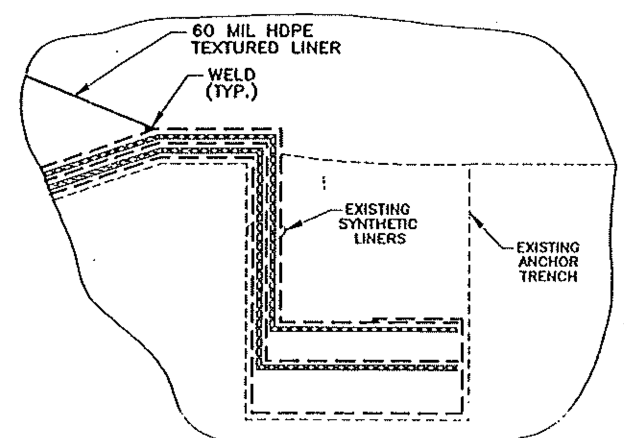
TYPICAL CLOSURE SECTION



DETAIL A

LEGEND

PCPP - PERFORATED CORRUGATED POLYETHYLENE PIPE



CAP HDPE LINER TIE IN DETAIL AT ANCHOR TRENCH B

NOTES:

1. EXTERIOR SLOPES OF CLOSURES SHALL BE 2.5H:1V OR FLATTER.
2. TOP OF CLOSURE SLOPES SHALL BE 10% OR FLATTER.
3. THE 60 MIL HDPE TEXTURED LINER FLAP SHALL EXTEND INTO THE TYPE II GRANULAR FILTER MATERIAL A DISTANCE OF BETWEEN 4 TO 6 INCHES (SLOPE DISTANCE). THIS LINER FLAP SHALL BE PLACED ON A SLOPE EQUAL TO THE TOP OF CLOSURE SLOPE (AS NOTED) AND SHALL NOT EXTEND DOWN THE 2.5H:1V SLOPE.
4. THE DRAINAGE NET AND FILTER FABRIC SHALL EXTEND INTO THE TYPE II MATERIAL A DISTANCE OF BETWEEN 1 INCH BEYOND THE TERMINATION OF THE 60 MIL HDPE TEXTURED LINER FLAP AND THE INTERFACE BETWEEN THE TYPE II AND TYPE V MATERIALS. THESE DRAINAGE NET AND FILTER FABRIC MATERIALS SHALL BE PLACED ON A SLOPE EQUAL TO THE TOP OF CLOSURE SLOPE (AS NOTED) AND SHALL NOT EXTEND DOWN THE 2.5H:1V SLOPE.
5. DIMENSION MAY VARY DEPENDING ON HEIGHT REQUIRED FOR BERM TO CONTROL RUNOFF FROM THE CLOSURE SURFACE.

HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	PROJECT ENGINEER	DESIGNED MEA	3	
			DRAFTED SS	2	
			CHECKED MEA	1	
			DATE MAY 1999	NO.	DATE / REVISIONS / BY / APVD.

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safety-kleen corp.

LONE MOUNTAIN FACILITY
TYPICAL CLOSURE SECTION
GEOSYNTHETIC CLAY LINER OPTION

SHEET NO. 1 OF 1

64-76-100

REF FILE Z:\FILES\064\60-100\CAD\C15TB.DWG

DESIGN ENGINEERING REPORT
LANDFILL CELL 15
LONE MOUNTAIN FACILITY

REGISTERED PROFESSIONAL ENGINEER
10-16-97
KENT
CHARLES
STAHELI
16595
OKLAHOMA

Prepared for

Laidlaw Environmental Services
(Lone and Grassy Mountain), Inc.
220 Outlet Pointe Boulevard
Columbia, South Carolina 29210

by

HA&L ENGINEERING, INC
Salt Lake City, Utah

Revised
October 1997

TABLE OF CONTENTS

Page

TABLE OF CONTENTS (continued)

LIST OF TABLES

1.0 INTRODUCTION

1.1 Background and Physiographic Setting

Laidlaw Environmental Services (Lone and Grassy Mountain), Inc. is proposing to modify the design of RCRA Landfill Cell 15 at their Lone Mountain Facility near Waynoka, Major County, Oklahoma. HA&L Engineering was retained by Laidlaw to provide the modified design for the cell. Design information, design drawings and details related to the original design were submitted previously in a report entitled, "Summary of Design and Engineering Report Landfill Cell Expansion Lone Mountain Facility," dated July 1989. A modification to the original design and the corresponding design information, drawings and details were submitted in a report entitled Landfill Cell 15 Design Engineering Report," dated June, 1993. Included in the original and the modified design reports were the results of geotechnical investigations conducted at the facility; storm drainage facilities design, including a discussion of the runoff management system and the run-on control system to control runoff from surrounding watersheds impacted by Cell 15; and the design criteria associated with the proposed landfill cell.

Landfill Cell 15 will be located in Section 28, T. 23 N., R. 15 W., I. M. and will be adjacent to Landfill Cells 12 and 14. The north embankments of Cells 12 and 14 will be shared as the south embankments of Cell 15 and the east embankment of Cell 14 will be shared as the west embankment of Cell 15. Landfill Cell 15 will extend to the north such that the exterior toe of the north embankment parallels approximately 20 feet to the south of existing channel 4. The east embankment of Landfill Cell 15 will be an extension, to the north, of the east embankment of Cell 12, and the west embankment of Cell 15 will be an extension, to the north, of the west embankment of Cell 14.

1.2 Scope of Services

The scope of this project included revising the design of Landfill Cell 15 in meeting the design requirements of 40 CFR Part 264 of the Code of Federal Regulations and Oklahoma Administrative Code (OAC) 252:200 of the Oklahoma Department of Environmental Quality, Waste Management Service. Other design criteria not included in the referenced documents but otherwise provided as a requirement by the Oklahoma Department of Environmental Quality have also been implemented in the design.

The following sections are presented in this report: 1) Stormwater Management, 2) Landfill Cell Design, and 3) Landfill Cell Closure.

Design drawings for Landfill Cell 15 including the closure of Cell 15 have been prepared and are included in Exhibit A. A geotechnical investigation for Landfill Cell 15 was conducted previously by Applied Geotechnical Engineering Consultants (AGEC). The AGEC report, which presents the results of their geotechnical investigations, is included in Exhibit B. Calculations for the stormwater management system are presented in Exhibit C. The design criteria for the HDPE geomembrane liners are presented in Exhibit D. The design criteria and calculations associated with the leachate collection/detection systems associated with the synthetic/composite triple liner system for the cell are presented in Exhibit E. Calculations associated with closure of Landfill Cell are contained in Exhibit F.

2.0 STORMWATER MANAGEMENT

2.1 Stormwater Facilities

Stormwater management associated with landfills at the Lone Mountain Facility will provide for the control of surface water drainage, resulting from precipitation events on areas that are tributary to the landfill cells. A portion of the precipitation that falls on the site will infiltrate directly into the ground, a portion will evaporate, some will adhere directly to vegetation and some will run off and be transported towards collection points or drainage ways. The stormwater management plan consists of facilities to control runoff inside and outside of the cell. Facilities outside of the cells control runoff from precipitation which falls outside of the cells, whereas the control systems inside of the cells will control runoff from precipitation which falls inside of the cells. Control facilities inside of Landfill Cell 15 will be referred to as the run-off management system. Control facilities outside of the cell will be referred to as the run-on management system. The run-on management system must be capable of preventing flow onto the active portion of the landfill cell during the peak discharge from at least a 25-year, 24-hour storm (as per 40 CFR 264.301). The run-off management system must be capable of collecting and controlling the run-off water volume from the active portion of the landfill, resulting from a 25-year, 24-hour storm.

A more detailed description of the run-off management system and the run-on management system is provided below.

2.2 Run-On Management System

Since Landfill Cell 15 will have embankments that are to be constructed above the existing ground surface around the entire perimeter of the cell, run-on to active portions of the landfill cell from surrounding watershed areas is not possible. Thus, potential areas which could contribute run-on to active portions of the landfill would be restricted to runoff from the top of the embankments themselves, from the closure caps of adjacent cells, or from closed phases of Cell 15.

Runoff from precipitation falling on top of the embankment of the landfill cell, on top of the closure caps of adjacent cells, or on closed phases of Cell 15 will be collected and controlled by providing cross slopes on top of the cell embankments, via ditches and storm drainage pipes to be constructed around the tops of the embankments and via berms and ditches to be constructed in conjunction with the closure caps of adjacent landfills and with the closure cap of Cell 15. These berms and ditches direct runoff from the closure caps and from the tops of the cell embankments towards pipe downspouts located at key locations around the cell. The pipe downspouts will convey the collected runoff down the exterior 2.1 horizontal to 1 vertical (2.1H:1V) side slopes of the landfill cell and will discharge the runoff into existing drainage channels. The detailed design of the ditches, berms and downspouts associated with the closure design of Landfill Cell 15 will be discussed in the landfill cell closure section of this report.

Facilities associated with the run-on management system include conveyance facilities designed to prevent runoff from adjacent watersheds from collecting and concentrating along the toes of the exterior embankment slopes. The run-on management system consists of five run-on conveyance channels which collect runoff from the drainage areas south and west of the landfill cells and from the closure caps of surrounding landfill cells, and which convey this collected flow around the cells to an existing drainage way located on the west side of the county road. Landfill Cell 15 will be located inside of the area controlled by these five run-on conveyance channels. Design information associated with these channels was submitted previously in a report prepared by HA&L Engineering entitled "Comparison of Developed vs. Predeveloped Conditions and Run-On Conveyance Channel Design Lone Mountain Facility," (HA&L Engineering, June 1990).

2.3 Run-Off Management System

The run-off management system will control runoff from precipitation that falls directly onto active areas of Landfill Cell 15. The run-off management system will consist of a conveyance channel or ditch around the top inside perimeter of the cell (and other ditches and/or berms constructed on the surface of the waste material) which will direct precipitation runoff from active areas of the cell toward temporary holding areas located inside the cell.

Active areas of the cell may be minimized by implementing the following options so that management of runoff from active areas of the cell can likewise be minimized.

1. Two types of berms (phase or sub-phase division berms and temporary area berms) will be constructed on the floor inside the landfill cell that are designed to contain runoff generated from active areas of the cell from entering areas that have not received waste materials. The location of the phase or sub-phase division and temporary area berms are illustrated on Sheet 4 of the drawings presented in Exhibit A. The landfill cell has been designed to have eight sump drainage areas (referred to hereafter as sump areas). When sump area no. 1 receives waste and becomes active, the temporary area berm between sump areas 1 and 2 will retain within sump area no. 1 all or part of the runoff from the waste material. As additional sump areas receive waste material, the additional sump areas will become active and the berms between the active sump areas and the adjacent non-active sump areas will retain all or part of the runoff from the active sump areas from entering the non-active areas. Cross-sections taken through a temporary area berm and through a phase or sub-phase division berm are presented on Sheet 4 of the drawings in Exhibit A. As illustrated on these cross-sections, the berms will be covered with HDPE

liner to prevent runoff water collected on the active side of the berms from infiltrating through the berms and into inactive sump areas of the cell. Runoff from precipitation falling on inactive sump areas of the cell will not be considered contaminated since it has not contacted hazardous waste materials located in active sump areas. The temporary area berms may be removed to the top of the two-foot thick protective cover prior to placing waste in the next adjacent sump area. If the temporary area berms are removed, the HDPE liner over the berms will be removed by cutting the liner just above the uppermost protective cover to prevent damage to the uppermost liner and maintain a barrier between the uppermost sumps.

2. As stated above, the area behind the temporary area and phase or sub-phase division berms may provide capacity to contain all or part of the estimated runoff from the waste material. If insufficient capacity has been provided behind the berms to contain all estimated runoff, additional capacity may be provided on the surface of the waste material (by providing berms, ditches and/or depressions in the waste) and it may be provided in a ditch around the perimeter of the waste and against the interior slopes of the cell.

3. Filling of Landfill Cell 15 is planned to begin at the south end of the east leg of the cell (north of Cell 12) and will proceed northward and then westward into the west leg of the cell (north of Cell 14). The waste may be placed in the cell such that the waste will be brought to design grade (if operationally desired and where stability concerns are not restrictive) as the filling progresses northward and then westward. As the waste is brought to design grade, the cell may be closed in phases and runoff from the closed areas of the cell will be directed away from the active working areas of the cell as part of the run-on management system. This process of filling and closing the cell in phases may proceed in such a manner so as to limit the active working area in the cell. Thus, the volume of runoff water that must be controlled inside of the cell can be minimized to that volume that would be generated from the open and active waste areas.

The temporary holding areas for runoff within the cell may be provided behind the temporary area berms or phase or sub-phase division berms; between the waste and the interior sideslopes of the cell; in depressions and ditches, or behind dikes on the waste; or any combination of the above. Capacity that is formed behind the temporary area berms or phase or sub-phase division berms in the active sump areas of the cell will be removed as waste placement progresses and inactive sump areas become active. The previous temporary holding areas behind the berms separating sump areas will be filled with waste and a new temporary holding area will be formed behind the berms of the newly activated sump areas.

Sufficient storage capacity should be maintained within the active areas of the cell to totally contain the runoff volume resulting from a 25-year, 24-hour precipitation event (6.0 inches). Assuming a curve number of 91 for bare soil conditions and a hydrologic soil group C, the runoff volume from the 25-year, 24-hour precipitation event is 0.41 acre-feet per acre of area. Thus, sufficient capacity should be maintained in the temporary holding areas to contain 0.41 acre-feet of runoff water per acre of active area contributing runoff to the temporary holding areas. Table 1 presents the amount of area contained within each sump drainage area and the potential runoff volume that may be generated from those areas assuming the areas are entirely open. If the active waste area consists of sump area 1 only, the temporary holding areas within sump area 1 must have enough total capacity for 1.88 acre-feet. The total capacity can be provided behind the temporary area berm (with one foot of freeboard) if the toe of the waste material is maintained at a minimum distance of 35 feet from the toe of the berm. Table 2 presents set-back distances between the toe of the waste material and the toe of the berms for some combinations of active areas assuming all runoff storage capacity is provided behind the berms. The waste set-backs presented in the table provide for the calculated runoff storage capacities and provide for one-foot of freeboard (calculations are presented in Appendix 1 of Exhibit C). Other combinations of active sump areas and required holding capacities may be evaluated to contain runoff from the active areas as the need arises during operation of the cell. Where holding areas are provided other than behind the berms, runoff will be directed to those areas. When waste placement progresses in active sump areas and the temporary holding areas no longer meet capacity requirements for the 25-year, 24-hour precipitation event, inspections will be made to check for any spill over that may occur from the temporary holding areas into inactive (or clean) sump areas during precipitation events. Once spill-over occurs, the inactive sump areas will be considered active unless they are cleaned-up.

TABLE 1

SUMP DRAINAGE AREAS AND POTENTIAL RUNOFF VOLUMES FROM ENTIRELY OPEN AND ACTIVE SUMP AREAS.

Sump Drainage Area	Area (acres)	Potential Runoff Volumes (acre-feet)
1	4.54	1.88
2	3.06	1.26
3	3.06	1.26
4	2.77	1.14
5	2.72	1.12
6	3.17	1.30
7	3.88	1.59
8	2.36	0.97

TABLE 2

SUMMARY OF ACTIVE SUMP AREAS, BERM ELEVATIONS AND WASTE MATERIAL OFFSETS REQUIRED TO PROVIDE TEMPORARY HOLDING CAPACITIES FOR RUNOFF FROM THE WASTE MATERIAL IN THE ACTIVE SUMP AREAS.

Sump Area Combinations	Required Holding Capacity (acre-feet)	Top of Berm Elevations (feet)	Waste offsets from Berms (feet)	Calculated Holding Capacity (acre-feet)
1	1.88	1381.9	35	1.88
1 & 2	3.14	1381.4	60	3.16
1 - 3	4.40	1382.81	60	4.40
1 - 4	5.54	1380.4	80	5.68
1/2 of 2 & 3 - 5	4.15	1376.7	70	4.16

Once the waste in the cell reaches an elevation within approximately 7 feet below the top of the embankments, conveyance channels will be formed between the waste and the inside slopes of the cell that will direct runoff from the active portions of the cell toward the temporary holding areas inside of the cell. The conveyance channels prevent runoff from overtopping the cell embankments and have been designed with sufficient capacity to convey the peak flow rate generated from the 25-year, 24-hour precipitation event with one-foot of freeboard. The conveyance channels were also analyzed to determine how they would function if they were to receive runoff from a 100-year, 24-hour precipitation event.

Construction of the conveyance channels will be dynamic in conjunction with the segmented closure of the landfills. A minimum depth of 1.6 feet must be maintained at the upstream end of each channel and the channel will be sloped with a minimum downhill gradient of 0.5 percent from the upstream end of the channel toward the temporary holding area. However, as the cell is filled and phased closure takes place over a portion of the cell, the portion of the cell that has been closed will no longer contribute runoff to the conveyance channel. The channel will then be reconstructed such that the depth at the new upstream end of the channel (the point where phased closure has progressed to) is 1.6 feet with the channel sloping at the designated slope from that new point of construction toward the temporary holding area.

These conveyance channels have been designed to have a trapezoidal cross-section with the 3H:1V inside slope of the cell forming one side of the channel, the waste placed on an approximate 2H:1V side slope forming the other side of the channel, and a bottom width of approximately 6.3 feet (which is the horizontal thickness of the 2-foot protective cover on the 3H:1V slopes). The conveyance channels will be constructed on an approximate 0.5 percent grade. The estimated peak flow rates at the upstream end of the channel from a 25-year, 24-hour precipitation event and from a 100-year, 24-hour precipitation event are 11.7 cfs and 16.1 cfs, respectively. The estimated peak flow rates at the downstream end of the channel from a 25-year, 24-hour precipitation event and from a 100-year, 24-hour precipitation event are 37.2 cfs and 50.7 cfs respectively. The downstream peak flow rates were calculated assuming the maximum length of channel to be about 950 feet or less.

Assuming the typical channel cross-section indicated above, a 0.5 percent channel gradient, and a Manning's n of 0.024, the normal flow depth at the upstream end of the channel is 0.6 foot (at the velocity of 2.6 fps for the 25-year, 24-hour precipitation event) and 0.7 foot (at a velocity of 2.9 fps for the 100-year, 24-hour precipitation event). The normal flow depth at the downstream end of a 950-foot long channel would be 1.08 feet (with a velocity of 3.8 fps for the 25-year, 24-hour precipitation event) and 1.28 feet (with a velocity of 4.16 fps for the 100-year, 24-hour precipitation event). A channel constructed to the dimensions and gradient presented would provide a minimum of 1-foot of freeboard for the 25-year, 24-hour precipitation event and 0.9 foot of freeboard for the 100-year, 24-hour precipitation event. Calculations are presented in Appendix 2 of Exhibit C.

After all phases of the cell have received waste and the waste in the final active areas nears the top of the cell embankments, either the perimeter ditches or a ponding area, to which runoff is directed, will function as the holding areas for precipitation runoff from the waste material in areas of the cell not yet closed. If ditches are used for containment of runoff, the ditches will be constructed at a minimum depth of 7.2 feet with a bottom width of about 6.3 feet around the top inside perimeter of the cell embankments. Ditches constructed to a minimum depth of 7.2 feet and with a 6.3-foot bottom width will have sufficient capacity to contain runoff generated by a 100-year, 24-hour precipitation event from active portions of the waste with one-foot of freeboard (see calculations in Appendix 2 of Exhibit C).

3.0 LANDFILL CELL DESIGN

3.1 Landfill Cell Layout and General Design Description

Landfill Cell 15 is designed to allow construction to occur in three phases with the first phase being constructed as two separate sub-phases as shown on Sheets 4, 5, 6 and 7 of the drawings in Exhibit A. Phase I is located east of Landfill Cell 14 and North of Landfill Cell 12. Phase I will be constructed as two separate sub-phases referred to as Phase IA and Phase IB. Phase II is located north of Phase I between Phase I and channel no. 4. Phase III is located west of Phase II between the north side of Landfill Cell 14 and channel no. 4. Each phase of Cell 15 will be separated from the other phases by constructing phase division berms designed to contain precipitation runoff from the waste material (from the 25-year, 24-hour storm event) in completed and active phases of the cell from entering adjacent and inactive phases of the cell. Phase IA will be separated from Phase IB by constructing a sub-phase division berm designed to contain precipitation runoff from the waste material (from the 25-year, 24-hour storm event). The phase or sub-phase division berms will be constructed similar to the cell embankments such that they will consist of all the same liner and leachate collection systems characteristic of those for the cell embankments. Waste storage capacities were calculated for each of the phases of the cell. Waste storage capacities for the different phases of the cell are presented in Table 3 and were calculated assuming the maximum waste storage that can be placed in each phase prior to placement of waste in subsequent phases of the cell. During actual waste placement, Laidlaw may decide to begin waste placement in subsequent phases of the cell prior to maximizing waste placement in the previous phases.

The capacities presented in this table reflect the maximum waste volume that can be placed in each phase or sub-phase prior to requiring waste placement to continue in adjacent phases or sub-phases. For example, the Phase I volume excludes the capacity above the waste set-back from the Phases I and II division berm and above the slope along the leading face of the waste. The Phase II volume includes the capacity that was excluded from Phase I but excludes the capacity above the set-back from the Phases II and III division berm and above the slope along the leading face of the waste. Initial filling of Phase II is also restricted to a height level with the top of the cell embankments due to liner stability concerns, therefore, the Phase II volume also excludes the capacity in the waste mound forming the subgrade to the closure cap above Phase II. Phase III volume includes all the capacity of Phase III and the capacity excluded from the Phase II volume.

TABLE 3

WASTE STORAGE CAPACITIES OF PHASES WITHIN LANDFILL CELL 15

Description of Waste Level	Phase IA cy	Phase IB cy	Total Phase I[1] cy	Phase II [2] cy	Phase III [3] cy
Storage capacity at elevation 1419 (one-foot below the top of the cell embankments)	241,000	209,100	450,100	447,100	407,900
Storage capacity within the waste mound forming the closure cap sub-grade	105,100	100,400	205,500	68,000	398,300
Total storage capacity for each phase	346,100	309,500	655,600	515,100	806,200

1) The maximum volume of waste that may be placed in Phase I prior to expanding waste placement into Phase II, taking into consideration the waste placement versus liner stability criteria discussed in Section 3.2.5 of this report and the waste set-back from the Phases I and II division berm.
2) The maximum volume of waste that may be placed in Phase II and the remainder of Phase I prior to expanding waste placement into Phase III, taking into consideration the waste placement versus liner stability criteria discussed in Section 3.2.5 of this report and the waste set-back from the Phases II and III division berm.
3) The maximum volume of waste that may be placed in Phase III and the remainder of Phases I and II.

The total waste storage capacity of Landfill Cell 15 at elevation 1419, (which is one-foot below the top of the cell embankments) will be approximately 809 acre-feet (1,305,100 cy). The total waste storage capacity to the top of the waste mound forming the subgrade to the closure cap is 1225.4 acre-feet (1,976,900 cy) using geosynthetic clay liner (GCL) for the closure cap. A more detailed discussion of the closure cap design is contained in section 4.0 of this report entitled Landfill Cell Closure.

Federal and State regulations require that hazardous waste landfills be constructed with two or more liner systems and with a leachate collection system above each liner system (40 CFR 264.301). Landfill Cell 15 has been designed and will be constructed with three liner systems and a leachate collection/detection system above each of the liner systems. The liner systems will consist of an uppermost synthetic liner (80 mil high density polyethylene geomembrane, HDPE), a middle synthetic liner (60 mil HDPE geomembrane), and a bottom composite synthetic/clay liner (consisting of 60 mil HDPE geomembrane overlying a minimum three-foot thick compacted clay liner). The leachate collection and removal systems (LCRS) will consist of a synthetic drainage net material (SLT GS-228, Gundle XL-14, or other drainage nets that are approved in meeting design requirements) placed over each liner system. The proposed composite triple liner system is illustrated in Detail A located on Sheet 38 of the design drawings presented in Exhibit A.

Although only a double liner and leachate collection/detection system is required to meet governmental regulations, Laidlaw has chosen to design and construct the cell with a triple rather than a double liner system. Thus, the extra liner and LCRS acts as a supplemental (but not required) feature. Most of the leachate collected in the landfill will be retained above the uppermost liner, collected in the uppermost sump, and will be removed via the uppermost LCRS. The middle LCRS will be preserved from the rigors of active operations and will function to determine if there is a leak in the uppermost liner system; whereas, the bottom LCRS will function to determine if there is a leak in excess of the proposed action leakage rate (ALR) in both the uppermost and middle liner systems.

The floor of Landfill Cell 15 has been divided into eight sump drainage areas or leachate collection removal systems. The floor of each sump area will consist of planar surfaces graded to slope toward a sump and toward leachate collection drains consisting of perforated corrugated polyethylene pipe and gravel backfill. The collection drains will be graded on a slope toward the sump to be constructed at the lower elevations of each sump area. The individual sumps provide a reservoir where leachate is collected and from which leachate in the landfill can be removed (see Sheets 10 through 36 in Exhibit A). Cell 15 has been designed to have a minimum bottom slope of one percent (40 CFR 264.301) which, based on the geotechnical investigation provided by AGEC (Exhibit B), should not be impacted by differential settlement. Leachate collected within each sump will be removed via 16-inch diameter HDPE pipes in the uppermost sumps and 12-inch diameter HDPE pipes in the middle and bottom sumps, which extend from the sumps to the top of the embankments.

The uppermost LCRS will consist of a continuous single layer of drainage net on the floor of the cell. The boundary conditions for the uppermost LCRS will be the underlying 80-mil uppermost HDPE geomembrane liner and a non-woven geotextile filter fabric (Tensar TG-700, or other geotextile filter fabric materials that are approved in meeting design requirements) which will be placed over the uppermost drainage net. Leachate collection drains (consisting of perforated corrugated polyethylene pipe and gravel backfill) will be constructed on the floor of the cell above the uppermost liner system. The drains will extend out onto the floor of the cell along the line formed by the intersecting plane surfaces forming the floor of each sump area and along the interior toe of the north embankment in sump areas 6, 7 and 8. The drains will collect leachate contribution from the intersecting planes on the floor of the cell and from the north embankment and will convey the leachate directly into the uppermost sumps.

The middle LCRS will have the same general design and configuration as the uppermost LCRS except that the drainage net will also extend up the interior slopes of the cell and the sumps will have less capacity. Boundary conditions for the middle drainage net on the floor of the cell will be the 60 mil middle HDPE geomembrane liner below and the Tensar TG-700 non-woven geotextile filter fabric (or other approved geotextile filter fabric meeting design requirements) above. Boundary conditions for the middle drainage net on the inside slopes of the cell will be the 60 mil middle HDPE geomembrane liner below and the 80 mil uppermost HDPE geomembrane liner above. Similar to the uppermost system, leachate collection drains will be constructed on the floor of the cell above the middle liner system and the slope of the floor and leachate collection drains will be toward the middle sumps.

The bottom leachate detection/collection and removal system (LDCRS) will consist of a continuous layer of drainage net on the floor and inside slopes of the cell. The bottom LDCRS will be bounded below by the 60-mil bottom HDPE geomembrane liner and bounded above by the 60 mil middle HDPE geomembrane liner.

The clay liner which forms the lowest most member of the triple liner system will be constructed from clay material at or near the facility that will be processed, placed, and compacted such that the in-place saturated hydraulic conductivity is less than or equal to 1×10^{-7} cm/sec. Construction procedures and construction quality control to ensure that the permeability requirement will be met are included in the construction quality assurance/quality control plan prepared by Laidlaw for the Lone Mountain Facility.

3.2 Geotechnical Investigation

The following data are summarized from the report submitted by Applied Geotechnical Engineering Consultants (AGEC) of Salt Lake City, Utah entitled "Geotechnical Investigation - Landfill Cell 15 - Lone Mountain Facility - Waynoka, Oklahoma, April 13, 1993. This report is presented in Exhibit B.

3.2.1 Bearing Capacity

Based on exploratory borings and test pits, the subsurface conditions beneath the proposed landfill consists of less than one foot to more than twelve feet of natural clay soil overlying claystone/siltstone bedrock. Most of the natural soils and all of the bedrock are suitable to support the proposed construction. It is anticipated that unsuitable foundation soil (consisting of excessively wet and soft soils) will be encountered in limited areas beneath the proposed Landfill Cell 15. All unsuitable material will need to be removed prior to construction. Classical bearing capacity calculations have been conducted to determine the bearing capacity of the bedrock and natural clay materials. A safety factor greater than 3 was calculated for the cell embankments and for the entire landfill.

3.2.2 Slope Stability Analysis

Interior embankment sideslopes are designed on a 3-foot horizontal to 1-foot vertical (3H:1V) slope. Exterior embankment side slopes are designed such that the embankment surface will be on a 2.1H:1V slope. Slope stability calculations indicate that the embankment section for a 66-foot high embankment has a static safety factor under long term condition of approximately 1.8 with a dynamic safety factor of 1.6. A horizontal ground acceleration of 0.04g was used to evaluate the embankment under seismic conditions. This was based on studies conducted by Algermissen and Perdins (U.S. Geological Survey Open File Report, 76-416, 1976) which indicate that the horizontal acceleration (expressed as a percentage of gravity) in rock with a 90 percent probability of not being exceeded in 50 years at the Lone Mountain Facility is estimated to be approximately 0.04g.

3.2.3 Ramp Stability Analysis

Ramps will be constructed down the 4.24H:1V slopes at the interior southwest corner of Phase I and at the interior southwest corner of Phase III to provide access into the landfill cell. The ramp in the southwest corner of Phase I will be constructed with an eight-inch thick protective layer above the uppermost liner, an eight-inch thick concrete or soil cement slab above the protective layer and two feet of protective cover above the concrete slab (see Sheet 39 of the drawings in Exhibit A). Based on stability calculations presented in the AGEC report in Exhibit B, the ramp has a safety factor against sliding of 1.5. A ramp in the southwest corner of Phase III may be constructed of waste material by first placing waste material on the cell floor at the bottom of the southwest corner and continuing placement up the corner on a maximum slope of 8H:1V. This type of ramp would provide a minimum safety factor of 1.3.

Additional access points may be provided into the cell along the west embankment of Phase I and the south embankment of Phase III. These additional access points may be provided as the cell fills with waste material to near the top of the cell embankment and as phased closure begins. The access points into the cell will consist of constructing a roadway between the top of the embankment and the top of the waste material in the cell.

3.2.4 Settlement

According to AGEC, settlement will occur within the overburden soil, foundation bedrock materials and within the embankment soils resulting from Cell 15 construction. Calculations indicate the proposed embankment may experience up to 3-1/2 to 8-1/2 inches of settlement due to the consolidation of foundation material. Embankment constructed directly on bedrock will experience less settlement than embankment constructed directly on overburden soils. The entire landfill is estimated to settle approximately 4-1/2 to 9 inches due to consolidation of the foundation material. Maximum settlement will occur in the central portions of the cell, reducing down to less than an inch at the outside edge of the embankment. A large portion of the settlement will occur during initial placement of the material within the embankment areas and/or within the cell. Since a large portion of the settlement will occur during initial placement of the material within the embankment areas and/or within the cell and since the overall settlement projections are low, differential settlement after cell construction will be negligible.

3.2.5 Waste Stability

Filling of the landfill cell will begin at the south end of Phase I and will move northward to the north end of Phase II and will continue from Phase II toward the west to the west end of Phase III. With the exception of some stability restrictions, the waste may be placed in the cell such that the waste is brought to design grade as waste placement in the cell progresses. As the waste is brought to design grade, the cell may be closed in phases in order to reduce the area of waste material exposed to precipitation and to promote precipitation runoff away from the waste material in the cell. To maintain stability of the synthetic liner/waste system, the waste within the cell should be placed such that the horizontal distance along the top of the waste (extending out away from the embankment) is at least 5 times the height of the waste. The height of the waste is measured from the top of the uppermost liner to the top of the waste. This criteria applies to all leading surfaces of waste placement until the waste level within the entire cell has reached the top of the cell embankments and the grade of the final waste surface is being achieved.

This is an extremely important aspect of the operation of the landfill cell due to the fact that the materials on the floor and interior side slopes of the cell have very low resistance to sliding. Placement of waste outside of this criteria may cause sliding which may result in damage to the synthetic materials. A safety factor of 1.5 is calculated with a phreatic surface located 1-foot above the bottom of the waste extending from the top of embankment down the interior embankment slope and across the cell floor to the end of the waste. This is a conservative assumption as the uppermost leachate collection and removal system on the floor of the cell is designed to reduce the depth of water head on the uppermost liner to approximately the thickness of the drainage net, which is less than one inch. This condition has, however, been evaluated to determine if water would result in unacceptable performance of the waste disposal system.

Slippages in the waste itself are very difficult if not impossible to evaluate due to the unknown characteristics and non-uniformity of the waste material. Stability analyses conducted using strength parameters that would apply to relatively weak soils indicate that slopes constructed on the order of 3H:1V are anticipated to be stable. Safety factors of 1.3 are obtained with a friction angle of 23.7 degrees with no cohesion or with 650 pounds per square foot cohesion with no friction. Using typical strength parameters that would apply for a highly plastic clay (cohesion of 79 pounds per square foot and a friction of 20 degrees) would provide a safety factor of 1.3.

3.2.6 Construction Considerations

Foundation Preparation. Foundation preparation will consist of removing the unsuitable foundation soils. This material will be removed down to more competent material, which will consist of the bedrock materials, very stiff embankment materials or more competent natural soils. Foundation preparation will also consist of stripping vegetation and other organic or deleterious materials from areas to receive fill.

Embankment Construction. The embankment may be constructed using on-site materials consisting of overburden soils and/or the claystone/siltstone bedrock broken down to soil size particles. Fragments of bedrock as large as six inches, if surrounded by soil size particles, are acceptable. All fill material placed in the embankment should be compacted to at least 95 percent of the maximum Standard Proctor Density within four percent of optimum moisture content. Fill compacted using heavy compaction equipment should be placed in uniform lifts not more than 8 inches thick prior to compaction. Fill compacted by hand operated equipment should be placed in lifts no more than 4 inches thick prior to compaction.

New fill material should be benched into existing embankments. The benching should extend at least one foot horizontally into the existing embankments for each lift placed.

Clay Liner. Landfill Cell 15 will be lined with a clay liner material that will be a minimum of 3-feet thick. This clay liner must meet the permeability requirement of being less than or equal to 1×10^{-7} cm/sec. Materials for clay liner are likely available from the surrounding area. A test fill(s) will be constructed to define the construction procedure(s) needed to obtain the required permeability of the clay liner.

Placement and compaction procedures will be defined from the test fill(s) to obtain the desired permeability. Compaction should be at least 95 percent of the maximum Standard Proctor Density as determined by ASTM D-698. Moisture content will likely need to be maintained near or above the optimum moisture content. To prevent surface cracking of the clay, positive measures should be taken to keep the surface of the clay liner moist.

Protective Cover. Approximately 1.5 feet of soil will be placed above the middle synthetic liner and LCRS on the floor of the Landfill Cell 15 as protection for the middle synthetic liner. The soil can consist of on-site or imported materials that have been broken down to soil-sand size. Approximately 2.0 feet of soil, select waste and/or screened waste will be placed above the uppermost synthetic liner and LCRS on the floor and inside slopes of the cell as a protection for the uppermost synthetic liner. The protective covers must be free of materials and objects that may damage the liner.

The placement of the soil protective cover above the middle liner should be conducted to prevent displacement of the underlying clay liner soils. Placement of the protective cover above the uppermost liner should be conducted to prevent displacement of the underlying soil protective cover above the middle liner. This is to be accomplished by only allowing equipment on top of the soil protective cover above the middle liner that will not impose a pressure greater than the allowable bearing capacity of the clay liner beneath the middle and bottom liners. Equipment used to place the protective cover above the uppermost liner and during operation of the cell should be restricted to equipment that will not exceed the allowable bearing capacity of the soil protective cover beneath the uppermost liner on the floor of the cell.

3.3 Synthetic Composite Triple Liner System

3.3.1 Design - HDPE Liners

The HDPE liners must have sufficient strength to resist stresses caused by the following conditions: compression, settlement, climatic conditions, uplift, external and internal pressure gradients, and stresses imposed during operation and installation of the liners. It must also be demonstrated that the synthetic liners are compatible with the waste materials deposited in the landfill cell. Summarized below are design considerations in analyzing the above indicated stresses for conditions to be encountered specifically in Landfill Cell 15. Chemical compatibility testing procedures and data are described elsewhere by Laidlaw.

Methodology and Assumptions Used in the Analysis of the HDPE Liner. Stresses resulting in the HDPE liner from the conditions listed above are interrelated. For example, climatic conditions affect the strength properties of the liner thereby affecting the liner's ability to withstand forces due to compression, settlement, construction and operation. External and internal pressure gradients are a result of normal forces (or compressive forces) which would include uplift and live and dead loads imposed during the construction and operation of the landfill cell.

Stresses resulting from the conditions indicated above could be grouped into distributed normal stresses and tangential stresses on the liner and the effects therefrom. Distributed normal stresses would include the dead load of overburden material placed on the liner, the live load of machinery used during installation and operation of the landfill cell, and any uplift pressures caused by the accumulation of liquids or gases underneath the liner. The effects of these normal stresses on the HDPE liner are twofold; the liner could be pushed into a depression if the allowable bearing capacity of the underlying subgrade is exceeded, or the liner could be pushed into a crack between two ridges of the drainage net which will be placed between the HDPE liners. Should the strength properties of the liner be exceeded by either failure of the underlying subgrade material or the liners inability to bridge the small span between ridges of the drainage net, the yield strength of the liner could be exceeded. Distributed tangential stresses would result primarily from differential settlement of the landfill cell under the dead load of materials placed therein, resulting in elongation of the liner. Since differential settlement of the embankments has been projected to be small, elongation of the liner and therefore distributed tangential stresses are negligible.

The integrity of the HDPE liner has been analyzed for dead and live loads, resulting in normal stresses to the liner during construction and operation of the landfill cell. Assumptions made in the analysis and/or construction requirements developed from the analysis include the following:

(1) Equipment used in spreading the protective cover on top of the HDPE liners in the bottom of the landfill cell shall be restricted to the following (or other equipment with prior approval from the Engineer, after evaluating loading characteristics of that equipment to ensure it does not exceed the allowable bearing capacity of the underlying soils):

   a. Track-Type Tractors of equivalent or improved loading characteristics (i.e. weight, center of gravity, etc.) to the Caterpillar D6 Track-Type Tractor or to the John Deere 750 Dozer.

   b. Wheel-Type Rubber Tire Dozer Tractors of equivalent or improved loading characteristics (i.e. weight, center of gravity, etc.) to the Caterpillar 824B or 824C Wheel-Type Rubber Tire Dozer Tractor.

c. Track-Type Front End Loaders of equivalent or improved loading characteristics (i.e. weight, center of gravity, etc.) to the Caterpillar 977L Track Front End Loader with a three and one quarter yard bucket.

d. Wheel-Type Rubber Tire Front End Loaders of equivalent or improved loading characteristics (i.e. weight, center of gravity, etc.) to the Caterpillar 966C Wheel Front End Loader with a three and one quarter yard bucket, or to the John Deere 644B and 644C Rubber Tire Loaders with a four and one half cubic yard bucket.

d. Motor Graders of equivalent or improved loading characteristics (i.e. weight, center of gravity, etc.) to the Caterpillar 14G Motor Grader.

e. Track-Type Excavator/Backhoes of equivalent or improved loading characteristics (i.e. weight, center of gravity, etc.) to the Caterpillar 235 Track-Type Excavator/Backhoe.

f. Trucks that do not exceed the maximum highway wheel loads specified by AASHTO for a HS-20 truck.

(2) Track type tractors or front-end loaders used in placing the liner protective cover in the bottom of the cell must push the protective cover out in front maintaining a minimum cover of 1.5 feet between the liner and the tracks of the vehicle for the soil protective cover placed over the middle liner, and maintaining a minimum cover of 2 feet between the liner and the tracks of the vehicle for the protective cover placed over the uppermost liner.

(3) Wheel type tractors must maintain a minimum cover of 1.5 feet (unless otherwise approved by the Engineer) between the liner and the wheels of the vehicle during placement of the soil protective cover over the middle liner, and a minimum cover of 2 feet must be maintained (unless otherwise approved by the Engineer) between the liner and the wheels of the vehicle during placement of the protective cover over the uppermost liner.

(4) The minimum cover that must be maintained over areas traversed by trucks, with an HS-20 loading, hauling the protective cover material into the cell for placement is two feet for either the soil protective cover over the middle liner or the protective cover over the uppermost liner. Trucks with loading conditions lower than the HS-20 designation may be analyzed and approved by the Engineer for operation on less than two feet of protective cover.

(5) The two-foot protective cover above the uppermost liner on the inside slopes of the cell will be placed in five-foot high lifts as the cell is filled with waste. These lifts must be placed by equipment reaching from the bottom up and from the top down. No machinery will be allowed on the side slope while placing the protective cover.

Stress to the liner due to deformation of the subgrade material beneath the uppermost, middle and bottom liners from the normal stresses described above can be avoided by assuring that loadings are within the allowable bearing capacity of the subgrade material. According to AGEC (see Exhibit B), the allowable bearing capacity of the clay liner material subgrades to the middle and bottom liners (assuming a safety factor of 3 for live and dead loads and a safety factor of 2 for impact loads) is 2,000 pounds per square foot for live and dead loads and 3,000 pounds per square foot for impact loads. The allowable bearing capacity of the soil protective cover underneath the uppermost liner was determined by the following equations recommended by AGEC (see Exhibit B). Allowable bearing pressure assuming a safety factor of 3 for dead and live loading:

(1) $ABC = 540 + 120\ W + 510\ SC$

Where: W = width of the bearing area of the load on the ground surface of a single track or tire in feet.

SC = height of protective cover above the uppermost liner in feet.

ABC= allowable bearing pressure in Lbs/ft^2.

Allowable bearing pressure for impact loading assuming a safety factor of 2:

(2) $ABCI = 1.5(ABC)$

Where: ABCI = allowable bearing pressure for impact loading in lbs/ft^2.

Strength of the liner to withstand stresses caused by bridging the small gap between ridges of the drainage net which separates the two HDPE liners, was analyzed by a methodology proposed by J.P. Giroud in a publication entitled, "Design of Geotextiles Associated with Geomembranes," (Giroud, J. P., 1982). Normal loads used in the analysis included forces caused by the weight of the soil and waste material placed on top of the liner as well as live and impact loadings caused by machinery used during construction and filling the cell.

Uplift pressure resulting from the accumulation of gases or liquids beneath the liner is also a normal stress that could act on the liner. However, the effect of uplift pressure on the liner of a landfill cell with solid waste deposited and compacted therein is significantly different from the effect of uplift pressures on a liner of a surface impoundment filled with a liquid that can be displaced if the uplift pressures are significant enough. In a landfill cell uplift forces will not be significant enough to displace the overburden material thereby creating a void in the overburden and additional stress in the liner. An analysis of a free-body diagram at the surface of the liner at rest would indicate that the force applied to the liner from the top would be countered by an equivalent reaction from the forces of the subgrade material on the bottom of the liner. Any force created by uplift pressures would not be an accumulative force added to the reaction from the subgrade soil, but would be a replacement force for an equivalent amount of the reaction force from the subgrade soil, since the combined forces from beneath must equal the force from overburden above the liner. In order for uplift pressures to totally replace the reactions of the subgrade soil to the liner and displace the overburden thereby causing damage to the liner, uplift pressures would have to exceed 8,400 lbs/ft^2 which is not anticipated.

Based on hydrogeologic characterization of the Lone Mountain Facility as presented elsewhere in the permit application, hydrostatic pressure from beneath the HDPE liners due to ground water is not anticipated. Although the underlying clay soils have some inclusions of naturally occurring organics, accumulations of gases beneath the embankment and clay liner are anticipated to be negligible, if any at all.

Results. The results from the analysis conducted to demonstrate that the HDPE liners can withstand normal and tangential stresses created during installation, operation, and ultimate completion of Landfill Cell 15 are summarized in Tables 4 and 5, with the results for the middle and bottom liners presented in Table 4 and the results for the uppermost liner presented in Table 5. The results presented in Tables 4 and 5 are categorized under two major headings; Gap Analysis - or the ability of the HDPE liner to bridge the small gap between ridges of the drainage net to be placed between the two 60 mil HDPE liners, and Loadings During Installation of the Protective Cover Over HDPE Liners and During Operation.

The results presented in Tables 4 and 5 indicate that as long as the minimum cover requirements are met for machinery and equipment used during installation and operation of the cell, the minimum safety factor against failure determined in the analysis was 2.4. Actually this minimum value is the safety factor against failure of the protective cover under the uppermost liner. As long as the protective cover does not fail, the liner will not be stressed from normal loadings, with the exception of the gap analysis which demonstrated a safety factor of 3.1 against failure. Should the clay or soil sub-base fail and the liner become stressed as a result, the liner itself would have an additional safety factor against failure up to its yield strength. Therefore, the factor of safety against failure of the liner is much greater than 2.4.

The results presented in Tables 4 and 5 indicate that the HDPE liners can withstand the normal and tangential stresses created during installation, operation, and ultimate completion of the cell. Supporting calculations are contained in Appendix 2 of Exhibit D.

TABLE 4

RESULTS OF THE ANALYSIS TO DETERMINE THE INTEGRITY OF THE MIDDLE AND BOTTOM HDPE LINERS AGAINST NORMAL AND TENSILE STRESSES IMPOSED ON THE LINER DURING CONSTRUCTION AND ULTIMATE CLOSURE OF LANDFILL CELL 15.

Conditions for Analysis	Tensile Force in Liner (lbs/in. of width)	% of Yield Tensile Strength	Live + Dead Load Bearing Pressure on Clay Sub-base (lbs/ft²)	% of 2000 lb/ft² Allowable Bearing Capacity on Subgrade	Live + Dead + Impact Load Bearing Pressure on Clay Sub-base (lbs/ft²)	% of 3000 lb/ft² Allowable Impact Load Bearing Capacity	Safety Factor S.F.
GAP ANALYSIS							
1. HDPE Liner bridging the gap of the drainage net under ultimate dead load conditions after the landfill cell is closed (governing load condition for the gap analysis).	45.0	32.2	-	-	-	-	3.1
LOADING DURING INSTALLATION OF THE PROTECTIVE COVER OVER MIDDLE HDPE LINER							
1. Analysis of HS-20 Truck loading on bearing capacity of clay sub-base assuming 2' minimum cover (governing load condition results from single axle loading).	-	-	1892	95	2057	69	2.9
2. Analysis of Caterpillar 977L Track Type Loader (3 1/4 CY bucket) on bearing capacity of clay sub-base assuming 1.5' minimum cover.	-	-	1703	85	2230	74	2.7
3. Analysis of D-6 Track Type Dozer on bearing capacity of clay sub-base assuming 1.5' minimum cover.	-	-	1478	74	1736	58	3.5
4. Analysis of Caterpillar 824C Wheel-Type Dozer assuming 40 psi maximum tire pressure and 1.5' minimum cover.	-	-	1923	96	2281	76	2.6
5. Analysis of Caterpillar 966C Wheel-Type Loader assuming 40 psi maximum tire pressure and 1.5' minimum cover.	-	-	1934	97	2283	76	2.6
6. Analysis of Caterpillar 14G Motor Grader assuming 45 psi maximum tire pressure and 1.5' minimum cover.	-	-	1944	97	2296	77	2.6
7. Analysis of Caterpillar 235 Track-Type Excavator Backhoe assuming 1.5' minimum cover.	-	-	1646	82	1937	65	3.1

TABLE 5

RESULTS OF THE ANALYSIS TO DETERMINE THE INTEGRITY OF THE UPPERMOST HDPE LINER AGAINST NORMAL AND TENSILE STRESSES IMPOSED ON THE LINER DURING CONSTRUCTION AND DURING OPERATION OF LANDFILL CELL 15

Conditions for Analysis	Tensile Force in Liner (lbs/in. of width)	% of Yield Tensile Strength	Live + Dead Load Bearing Pressure on Clay Sub-base (lbs/ft²)	% of Allowable Bearing Capacity on Subgrade for SF = 3	Live + Dead + Impact Load Bearing Pressure on Liner (lbs/ft²)	% of Allowable Impact Load Bearing Capacity	Safety Factor S.F.
LOADING DURING INSTALLATION OF THE PROTECTIVE COVER ABOVE THE UPPERMOST LINER AND DURING OPERATION							
1. Analysis of HS-20 Truck loading on bearing capacity of soil sub-base assuming 2.0' minimum cover (governing load condition results from single axle loading).	-	-	1892	113	2057	82	2.4
2. Analysis of Caterpillar 977L Track Type Loader (3 1/4 CY bucket) on bearing capacity of soil sub-base assuming 2' minimum cover.	-	-	1614	93	1887	72	2.8
3. Analysis of D-6 Track Type Dozer on bearing capacity of soil sub-base assuming 2' minimum cover.	-	-	1273	73	1478	57	3.5
4. Analysis of Caterpillar 824C Wheel-Type Dozer assuming 40 psi maximum tire pressure and 2' minimum cover.	-	-	1573	91	1840	71	2.8
5. Analysis of Caterpillar 966C Wheel-Type Loader assuming 40 psi maximum tire pressure and 2' minimum cover.	-	-	1570	89	1834	69	2.9
6. Analysis of Caterpillar 14G Motor Grader assuming 45 psi maximum tire pressure and 2' minimum cover.	-	-	1553	89	1814	69	2.9
7. Analysis of Caterpillar 235 Track-Type Excavator Backhoe assuming 2' minimum cover.	-	-	1572	90	1837	70	2.8

3.3.2 Design - Uppermost Leachate Collection and Removal System (LCRS)

Design of the ULCRS consists of hydrologic evaluations for projecting anticipated leachate rates and volumes, hydraulic design of the leachate collection system, and design of the sumps and leachate removal process. The hydrologic evaluation was performed using EPA's Hydrologic Evaluation of Landfill Performance (HELP) Model. Projected leachate rates and volumes generated by the HELP model were used for design of the leachate collection systems and the sumps and leachate removal systems.

HELP Modeling. The HELP Model is a quasi-two-dimensional hydrologic computer model used for conducting water balance analyses of landfills, cover systems and other solid waste containment systems. The model accepts weather, soil and design data and uses solution techniques that account for the effects of surface storage, snowmelt, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through soil, geomembrane or composite liners.

Landfill Cell 15 is designed to contain all direct precipitation during filling of the cell and to divert outside runoff from entering the cell. Therefore, runoff from the cell and introduction of other than direct precipitation were not a consideration. Vegetative growth and leachate recirculation were also not considered in the HELP Model.

Precipitation and daily temperature data were obtained on CD ROM from Hydrosphere Data Products for the time period between January 1980 and September 1994. Additional precipitation and daily temperature data between October 1994 and December 1995 were obtained from the USGS Climatological Data for Oklahoma. All precipitation and temperature data was obtained from the gauging station located in Waynoka, Oklahoma (near the facility). Solar radiation and evapotranspiration data were not available for the Waynoka station, therefore, the data was synthetically generated by the HELP model using data available within the model for Tulsa, Oklahoma. Tulsa was selected because of it's latitude proximity to the facility.

HELP Model Calibration. Model calibration was accomplished using actual waste elevations within Landfill Cell 13 and comparing the predicted leachate quantities generated by the HELP model against actual records of leachate quantities pumped from the cell. Adjustments were made to the input data for physical characteristics of the waste material until a relatively close comparison was obtained between the leachate quantities generated by the model and the actual leachate quantities.

Calibration of the HELP model appears to be most sensitive to soil and waste thicknesses, the saturated hydraulic conductivity of the soil and waste materials and the evaporation zone depth at the surface of the soil and waste materials. Since varying the waste thickness as a function of time is not an option provided in the HELP model, waste elevations were evaluated for Landfill Cell 13 in order to determine a time period when waste elevations remained relatively constant. Waste elevation data were obtained from Jividen Land Surveyors based on periodic waste surface surveys conducted within Cell 13. The conditions of Landfill Cell 13 in 1993 were selected for model calibration because the data obtained from the waste surveys indicate that the waste elevation remained somewhat constant, at a near level full condition, for that year.

Although there were differences between monthly leachate quantities generated by the model and actual monthly leachate quantities pumped from Cell 13, the annual quantity for 1993 is nearly equal. The variation in monthly values may result from waste thickness variations, irregularity in the waste surface which may cause variations in runoff and infiltration patterns and possible variations regarding the physical characteristics within the waste materials. The model provides a higher peak month leachate quantity than the actual peak month quantity pumped from Cell 13 in 1993. This would indicate that values generated by the model should be somewhat conservative for design purposes.

The actual annual and peak month leachate quantities from Landfill Cell 13 for 1993 are 11.11 inches and 1.83 inches, respectively. The annual and peak month leachate quantities for 1993 as generated by the HELP model are 11.42 inches and 3.72 inches, respectively. The quantity in inches represents the average depth of leachate generated over the area of the cell.

HELP Modeling of Landfill Cell 15. Landfill Cell 15 has been designed with eight different sump drainage areas. Each of the sump drainage areas are unique in size, configuration and slopes. Therefore, each sump area was modeled separately for waste thicknesses of about 24 inches (near empty), half full (based on height and not capacity), level full (with the top of the cell embankments) and full (waste height at closure). Weighted averages were used as input to the HELP model for waste thickness and for lateral drainage slope and transmissivity because of the impact the embankment height and sideslopes have on these characteristics.

Tables 6, 7 and 8 present leachate quantities generated by the HELP model for peak day, peak month and for average day based on peak month for each of the waste levels modeled. The peak day and peak month values represent the peaks over the 15 years (1980 through 1995) of weather data that were used.

Leachate Collection System. EPA requires (40 CFR 264.301) that the leachate collection and removal system should be capable of collecting and conveying leachate to the sumps such that the maximum depth of leachate on the liner system outside the sumps is one foot. The Oklahoma Department of Environmental Quality has determined that the maximum depth of leachate which will be allowed is 16 inches, measured at the low point of the liner or sump system. The uppermost leachate collection and removal system has been designed such that the maximum leachate depth within the leachate collection system above the uppermost liner should not exceed one foot. The drainage net of the uppermost system serves as a continuous pipe drainage system between the uppermost liner system and the overlying geotextile and protective cover material. The drainage net, therefore, must be sized such that the capacity of the net is not exceeded for the projected leachate rate when considering the design slope of, and the normal loading on, the net. As long as the capacity of the net is not exceeded, flow in the drainage net will be in essence "open channel" flow and the depth of fluid on the liner system below the net will be limited by the thickness of the net, which is less than one inch.

SLT GS-228 drainage net (or other drainage nets, such as Gundle XL-14, that are approved in meeting design requirements) has been specified as the drainage net to be used in construction of the uppermost LCRS. Test data are available for SLT GS-228 (and from Gundle XL-14) from which the transmissivity of the net can be determined under the boundary conditions and ultimate loading conditions proposed in the design of Landfill Cell 15. A contractor may propose a substitute drainage net for approval as long as it can be demonstrate that the transmissivity of the proposed drainage net meets design transmissivity requirements (under the boundary and loading conditions and under the slopes used in the Cell 15 design) for the uppermost LCRS and as long as it can be demonstrate that the substitute net is chemically compatible with potential leachate generated in Cell 15. Transmissivity test data for SLT GS-228 and Gundle XL-14 drainage nets are presented in Appendix 1 of Exhibit E for varied loading conditions and slopes and for the upper boundary condition of Tensar TG-700 non-woven geotextile fabric and the lower boundary condition of HDPE liner.

The drainage net has been designed using the design-by-function concept recommended by EPA for the design of RCRA hazardous waste facilities. According to EPA (1989, pg. 56), "whatever parameter of a specific material one is evaluating, a required value for the material must be found using a design model and an allowable value for the material must be determined by a test method. The allowable value divided by the required value yields the design ratio, or the resulting factor of safety." Thus, in evaluating the drainage net for the leachate collection system, an allowable transmissivity is divided by the required transmissivity to determine the factor of safety for the design.

TABLE 6

PEAK DAY LEACHATE VOLUMES

Sump No.	Drainage Area (ft^2)	Near Empty (in)	Near Empty (ft^3)	Half Full (in)	Half Full (ft^3)	Level Full (in)	Level Full (ft^3)	Full (in)	Full (ft^3)
1	197690	0.82	13509	0.22	3624	0.16	2636	0.15	2471
2	133300	0.83	9220	0.21	2333	0.15	1666	0.15	1666
3	133160	0.82	9099	0.21	2330	0.15	1665	0.15	1665
4	114440	0.82	7820	0.20	1907	0.15	1431	0.14	1335
5	117930	0.81	7960	0.19	1867	0.15	1474	0.14	1376
6	136960	0.97	11071	0.21	2397	0.15	1712	0.15	1712
7	167910	0.82	11474	0.22	3078	0.16	2239	0.15	2099
8	102850	0.96	8228	0.35	3000	0.17	1457	0.16	1371

TABLE 7

PEAK MONTH LEACHATE VOLUMES

Sump No.	Drainage Area (ft^2)	Near Empty (in)	Near Empty (ft^3)	Half Full (in)	Half Full (ft^3)	Level Full (in)	Level Full (ft^3)	Full (in)	Full (ft^3)
1	197690	5.76	94891	3.64	59966	3.44	56671	2.93	48269
2	133300	5.76	63984	3.72	41323	3.22	35769	2.79	30992
3	133160	5.76	63917	3.72	41280	3.23	35842	2.76	30627
4	114440	5.76	54931	3.76	35858	3.09	29468	2.64	25177
5	117930	5.77	56705	3.77	37050	3.01	29581	2.60	25552
6	136960	5.78	65969	3.72	42458	3.23	36865	2.82	32186
7	167910	5.76	80597	3.64	50933	3.40	47575	2.92	40858
8	102850	5.78	49539	3.20	27427	3.71	31798	3.41	29227

TABLE 8

AVERAGE DAY LEACHATE VOLUMES BASED ON PEAK MONTH

Sump No.	Drainage Area (ft^2)	Near Empty (ft^3)	Half Full (ft^3)	Level Full (ft^3)	Full (ft^3)
1	197690	3163	1999	1889	1609
2	133300	2133	1377	1154	1000
3	133160	2131	1376	1156	988
4	114440	1831	1195	951	812
5	117930	1890	1235	954	824
6	136960	2199	1415	1189	1038
7	167910	2687	1698	1535	1318
8	102850	1651	914	1026	943

Koerner (1990) suggests that additional factors of safety be applied to the allowable value found by test method to account for creep deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space and for biological and chemical clogging in the geonet's core space. In accordance with the procedures recommended by Koerner (1990), an additional factor of safety of 1.4 has been applied to the allowable transmissivity for creep deformation or intrusion of the adjacent geosynthetic into the geonet's core space, and an additional factor of safety of 2 has been applied to the allowable transmissivity for potential biological and chemical clogging of the geonet. This is in addition to a factor of safety of 1.5 to be used in the design-by-function concept discussed above. The combined safety factor for the drainage net is therefore 4.2, which is determined by multiplying the three safety factors indicated above.

Calculations derived in determining the required drainage net to sump configuration for the uppermost LCRS are presented in Appendix 2 of Exhibit E. These calculations indicate that a single layer of SLT GS-228 drainage net is adequate to convey the leachate to the leachate collection drains and to the sumps. Data supplied by Gundle Lining Systems indicated that the transmissivity of Gundle XL-14 drainage net is higher than the transmissivity of SLT GS-228 under the same boundary and loading conditions. Gundle XL-14 drainage net would, therefore, be acceptable for use.

Leachate collection drains will extend out across the floor of the cell along the line formed by the intersection of the plane surfaces of the floor and along the interior toe of the north embankment in sump areas 6, 7 and 8. The leachate collection drains will consist of three-inch diameter perforated corrugated polyethylene pipe (PCPP) and will be backfilled with 3/4-inch rounded washed drain rock. The leachate collection drains will intercept leachate contribution to the floor and from the north interior side slope of the cell and convey the collected leachate directly into the sumps. Based on the maximum tributary area to the longest leachate collection drain and based on the design leachate infiltration rate presented above, the maximum flow rate to be conveyed by the drain would be on the order of 2.14 cubic feet per minute. The three-inch diameter PCPP has a flow carrying capacity of 3.36 cubic feet per minute on a 1 percent slope. Thus, the proposed pipe has more than sufficient capacity to handle design leachate flows.

A geotextile filter fabric is to be placed between the drainage net and the overlying liner protective soil cover to prevent migration of the soil into the drainage net. The geotextile fabric must have sufficient filtering capability to retain the soil, must be permeable enough to convey water percolating through the soil cover into the underlying drainage net, and must not become clogged by the overlying soil material. According to the "Geotextile Engineering Manual" (U.S. Department of Transportation Federal Highway Administration), to meet the soil retention criteria (for soils with a gradation such that less than 50 percent by weight of the soil passes the #200 sieve) the equivalent opening size (E.O.S.) of the fabric must be less than or equal to BxD_{85}, where D_{85} is the soil particle size for which 85 percent is finer by weight and B is equal to 1 for a uniformity coefficient of the soil (C_u) less than or equal to 2 or greater than or equal to 8, B is equal to 0.5 x C_u for a C_u greater than or equal to 2 or less than or equal to 4, or B is equal to $8/C_u$ for C_u greater than 4 or less than 8. To meet the permeability criteria, the permeability of the fabric must be greater than ten times the permeability of the soil. Calculations presented in Appendix 3 of Exhibit E indicate that the Polyfelt TS 700 geotextile filter fabric provides the required soil retention capability to retain the soil above the net, has sufficient permeability when compared with the permeability of the soil protective cover and meets the permeability criteria presented above.

Sumps. The sumps were designed using the contour of the floor above the uppermost liner system to form the bottom of the sumps. The top of the uppermost sump is formed by a level plain surface 1.5 feet above the lowest point on the floor formed by the uppermost liner system.

Projected leachate quantities generated by the HELP model were used to determine the amount of holding capacity the sumps have with respect to leachate generation time. In other words, the frequency at which leachate should be pumped from the sumps depends on the potential the cell has for generating leachate. According to results from the HELP model, the potential of leachate generation depends on the level of waste material in the specific sump area. As the waste level becomes higher, the required pumping frequency can be reduced because the quantity of leachate will be reduced and the sumps have the capacity to hold leachate volumes generated over a longer period of time.

The design details for the uppermost sumps are presented on Sheets 29 through 36 of the drawings in Exhibit A. The sumps will be filled with 3/4-inch rounded washed rock and will have a system of 6-inch and 3-inch diameter perforated corrugated polyethylene pipes that will collect and convey leachate and water stored in the sumps towards the low point of the sumps. HDPE leachate withdrawal pipes (16-inch diameter for the uppermost sumps) will be placed up the slope of the cell from the sump to the top of the embankment. The leachate withdrawal pipes allow pumps to be inserted into the sumps for removal of leachate from the sumps. A porosity of 32 percent was used for the sump rock and 100 percent of the volume of the perforated pipes was used to calculated the storage capacity in the sumps. The capacity of the uppermost sumps was only assumed to a depth of 16 inches which is the ponding depth approved by Oklahoma DEQ within the sumps. This capacity is exclusive of any storage in the leachate collection drains outside of the sumps. Sump storage capacity calculations are presented in Appendix 5 of Exhibit E. The capacities and a relationship between waste height and potential pumping frequency for the uppermost sumps are presented in Table 9. The numbers presented in Table 9 were generated by dividing the peak day leachate volume provided by the HELP model (see Table 6) by the calculated sump storage capacity. The pumping frequencies presented in Table 9 represent the most frequent pumping that may occur based on maximum peak day conditions as generated by the HELP model. Actual pumping frequencies may be substantially less than those presented depending on weather conditions at the facility and physical characteristics within the waste material and on the waste surface.

<u>Leachate Withdrawal Pipes.</u> The required wall thickness for the leachate withdrawal pipes must be designed to prevent failure or significant loss of cross sectional area from the ultimate loading that will be placed on them. The HDPE leachate withdrawal pipes have been designed with sufficient wall thickness to prevent failure by wall crushing, failure by wall buckling, and failure by ring deflection. Manufacturers test data used in the calculations represent a maximum period of analysis of 50 years.

TABLE 9

UPPERMOST SUMP STORAGE CAPACITIES AND POTENTIAL PUMPING FREQUENCIES

Sump No.	Storage Capacity (gallons)	Pumping Frequency for a Near Empty Condition and Peak Day Leachate Volume (days)	Pumping Frequency for a Half Full Condition and Peak Day Leachate Volume (days)	Pumping Frequency for a Full Condition and Peak Day Leachate Volume (days)
1	19,320	0.2	0.7	1
2	20,470	0.3	1.2	1.6
3	9,650	0.1	0.6	0.8
4	9,980	0.2	0.7	1
5	14,020	0.2	1	1.4
6	16,380	0.2	0.9	1.3
7	14,550	0.2	0.6	0.9
8	13,560	0.2	0.6	1.3

Calculations presented in Appendix 4 of Exhibit E, indicate all HDPE leachate withdrawal pipes with a SDR of 15.5 are safe against wall crushing with a safety factor of 2.9, are safe against wall buckling with a safety factor of 2.0, and have a ring deflection of 2.9 percent compared to an allowable ring deflection of 3.9 percent. Backfill for the leachate withdrawal pipes should consist of a mixture of 50 percent sand soils and 50 percent clay soils for the maximum height of fill anticipated above the pipe proposed in the design of the cell. These safety factors were derived assuming the soil around the pipes to have been compacted to a minimum of 85 percent of Standard Proctor. Specifications will require a minimum compaction around the pipes of 90 percent of Standard Proctor. Therefore, these safety factors are conservative.

The eight uppermost sump drainage areas for Landfill Cell 15 are designed to direct all flow toward the sumps through the drainage net and leachate collection drains placed as designated above. The maximum head of water on top of the uppermost liner will be less than one foot except possibly for short periods of time during severe storm events which may occur while the landfill is open and except in the sumps.

3.3.3 Design - Middle Leachate Collection and Removal System (LCRS)

Drainage Collection System. The middle leachate collection and removal system on the side slopes of the cell and on the slopes and the top of the phase division berms will be bounded above by the uppermost liner and bounded below by the middle liner. The leachate collection system on the cell floor will be located between the Tensar TG-700 geotextile filter fabric and protective cover above and the middle liner below. The middle LCRS will consist of a continuous layer of SLT GS-228 drainage net (or other drainage nets, such as

Gundle XL-14 that are approved as meeting design requirements) placed on the floor and inside slopes of the landfill cell. Tensar TG-700 geotextile filter fabric and a 1.5-foot thick soil protective cover will be placed on the floor of the cell, above the middle drainage net, as a protective cover between the middle and uppermost synthetic liners. Thus, similar to the uppermost system, a filter fabric will be placed above the drainage net on the floor of the cell as a permeable barrier between the soil cover material and the drainage net. The boundary conditions for the middle drainage net on the floor of the cell consist of the filter fabric above and the 60 mil middle HDPE synthetic liner below. The boundary conditions for the middle drainage net on the inside slopes of the cell consists of the 80 mil uppermost HDPE synthetic liner above and the 60 mil middle HDPE synthetic liner below.

The middle LCRS has been designed to have the same general configuration as the uppermost LCRS. As such the middle LCRS functions to convey leachate towards the middle sumps under the same design conditions as the uppermost LCRS as if the uppermost HDPE liner were not present. Leachate collection drains will also be constructed for the middle LCRS extending across the floor of the cell and at the interior toe of the north embankment as was done with the uppermost LCRS. These collector drains will intercept leachate contribution to the floor area and the interior side slopes of the cell and will convey the collected leachate into the sumps.

The middle LCRS is also a backup system that can be used to check for leaks in the uppermost system and to allow the removal of leachate should it leak through the uppermost liner system. Regulations (Federal Register, Volume 57, No. 19, January 29, 1992) for RCRA hazardous waste landfills indicate that a backup system should be designed to be capable of detecting, collecting and removing leaks of hazardous constituents at the earliest practicable time through all areas of the top liner likely to be exposed to waste or leachate during the active life and post closure care period. The middle LCRS will have the same slope, boundary conditions, and loading characteristics as used in the uppermost LCRS design. Transmissivity values used to design the middle LCRS are the same transmissivity values as those used for the uppermost leachate collection and removal system. Similar safety factors as discussed in the design for the uppermost leachate collection and removal system were applied to the design of the middle LCRS. The longest possible distance between a point of leakage and the point of leachate detection was used to determine the longest possible theoretical response time. A response time of 5.5 hours was determined using SLT GS-228 drainage net and is slightly less using Gundle XL-14 drainage net.

Design considerations for the geotextile filter fabric and the 12-inch diameter HDPE leachate withdrawal pipe for the middle LCRS are the same as those summarized in the design for the uppermost LCRS.

Sumps. The design details for the middle sumps are similar to those for the uppermost sumps and are presented on Sheets 21 through 28 of the drawings in Exhibit A. The bottom of the middle sumps consists of the floor formed by the middle liner system. The top of the middle sumps consists of a level plain surface one foot above the low point of the middle sumps. The sumps will be filled with 3/4-inch rounded washed rock and will have a system of 6-inch and 3-inch diameter perforated corrugated polyethylene pipes that will collect and convey leachate and water stored in the sumps towards the low point of the sumps. HDPE leachate withdrawal pipes (12-inch diameter for the middle sumps) will be placed up the slope of the cell from the sump to the top of the embankment. The leachate withdrawal pipes allow pumps to be inserted into the sumps for removal of leachate from the sumps.

3.3.4 Design - Bottom Leachate Detection, Collection, and Removal System (LDCRS)

Drainage Collection System. The bottom leachate detection, collection, and removal system will be located between the middle liner above and the bottom liner below and will be the lowest leachate collection and removal system in the cell. According to EPA (EPA, January 1992) no maximum has been set for the level of liquids in the sumps, but the head on the bottom liner and backup of liquids into the drainage layer must be minimized by removing pumpable liquids from the sumps. The Oklahoma Department of Environmental Quality has determined that the maximum depth of leachate which will be allowed is 16 inches, measured at the low point of the liner or sump system. The bottom LDCRS will consist of a continuous layer of SLT GS-228 drainage net (or other drainage nets, such as Gundle XL-14, that are approved in meeting design requirements) placed on the floor and on the inside slopes of the landfill cell between the middle and bottom liners. The boundary conditions for the bottom drainage net on the floor and on the inside slopes of the cell consist of the 60 mil middle HDPE synthetic liner above and the 60 mil bottom HDPE synthetic liner below. The bottom sumps are located vertically beneath the middle and uppermost sumps in each sump area.

The bottom LDCRS is a second or redundant backup system used to check for leaks in excess of the action leakage rate from the middle liner system and to allow the removal of leachate that may leak through the middle liner system. As indicated above, regulations (Federal Register, Volume 57, No. 19, January 29, 1992) for RCRA hazardous waste landfills indicate that a backup system should be designed to be capable of detecting, collecting and removing leaks of hazardous constituents at the earliest practicable time through all areas of the top liner likely to be exposed to waste or leachate during the active life and post closure care period.

The bottom LDCRS will have the same slope and loading characteristics as used in the uppermost and middle LCRS designs. However, the boundary conditions for the bottom LDCRS are somewhat different since it is bounded both above and below by a 60 mil HDPE geomembrane liner, instead of having an upper boundary condition of a geotextile filter fabric. The transmissivity test value used in the design of the uppermost system was also used in the design of the bottom system. The test results are conservative when applied to the bottom system because the boundary conditions of the bottom system provide better flow characteristics than the boundary conditions of the uppermost system. When a load is applied to the geotextile filter fabric, which provides the upper boundary layer in the uppermost system, the filter fabric tends to push into the gaps between the ribs of the drainage net, restricting the flow of leachate through the drainage net. However, when a load is applied to HDPE geomembrane liner, providing the upper boundary layer in the bottom system, the liner tends to bridge the gap between the ribs in the drainage net, thereby allowing leachate to flow more freely through the open spaces in the drainage net.

The transmissivity values used to evaluate the bottom LDCRS are the same transmissivity values as those used for the uppermost and middle LCRS's. Similar safety factors, as discussed in the design for the uppermost LCRS, were applied to the design of the bottom LDCRS. The longest possible distance between a point of leakage and the point of leachate detection was used to determine the longest possible theoretical response time. A response time of 5.5 hours was determined using SLT GS-228 drainage net and is slightly less using Gundle XL-14 drainage net.

Design considerations for the 12-inch diameter HDPE leachate withdrawal pipe for the bottom LDCRS are the same as those summarized in the design for the uppermost LCRS.

Sumps. The design details for the bottom sumps are presented on Sheets 13 through 20 of the drawings in Exhibit A. The sumps will be filled with 3/4-inch rounded washed rock and will have a system of 4-inch diameter perforated corrugated polyethylene pipes that will collect and convey water stored in the sumps towards the 12-inch diameter HDPE leachate withdrawal pipe to be placed up the slope of the cell from the sump to the top of the embankment. This leachate withdrawal pipe is located at the low point of each sump and a pump, for pumping leachate from the cell, will be placed inside of the 12-inch diameter HDPE pipe.

The sump capacities for the bottom sumps are presented in Table 10. A porosity of 32 percent was used for the sump rock and 100 percent of the volume of the perforated pipes was used to calculate the storage capacity in the sumps. Sump capacity calculations, including stage capacity relationships for each sump, are presented in Appendix 7 of Exhibit E.

TABLE 10

BOTTOM SUMP STORAGE CAPACITIES

Sump No.	Storage Capacity (gallons)
1	1575
2	1575
3	1558
4	1558
5	1617
6	1419
7	3712
8	1740

Action Leakage Rate (ALR). Based on the January 29, 1992 EPA rule, owners or operators of a hazardous waste disposal unit must calculate an action leakage rate based on the maximum design leakage rate that the lowermost leak detection system (in this case the bottom LDCRS system) can remove without the fluid head on the bottom liner exceeding one foot. This leakage rate must account for an adequate margin of safety for uncertainties in design which, EPA indicates in the rule, should be a factor of safety of 2. The ALR must take into consideration both the drainage layer in the unit as well as the pumping capacity of the leak detection sump.

The drainage layer associated with the LDCRS, as described above, consists of a continuous layer of drainage net placed on the floor and inside slopes of the landfill cell between the middle and bottom liners. The boundary conditions for the bottom drainage net on the floor and inside slopes of the cell consist of the 60 mil middle HDPE synthetic liner above and the 60 mil bottom HDPE synthetic liner below. The bottom LDCRS slopes toward sumps and toward leachate collection drains that slope toward sumps at the low point in each sump drainage area.

The bottom sumps are filled with 3/4-inch rounded washed rock and contain a series of 4-inch diameter perforated corrugated polyethylene pipes. The sumps are graded toward 12-inch diameter HDPE leachate withdrawal pipes, located at low points in the sumps, into which pumps are placed and which extend up the inside slope of the cell between the middle and bottom HDPE liners. Two or three of the 4-inch diameter corrugated polyethylene pipes (two in sump 7 and three in sumps 1 through 6 and 8) extend directly into and convey water collected in the sump into the 12-inch diameter HDPE leachate withdrawal pipe. These pipes provide a hydraulic conduit for conveyance of water collected within the sump directly to the pump.

Pumps having a capacity of at least 40 gpm are placed in the sumps to pump leachate collected in the sumps to the top of the embankment where it is collected and then treated and disposed of as required in the operating permit for the facility.

Design calculations associated with the ALR are presented in Appendix 8 of Exhibit E. The calculations include an analysis of the capacity of the drainage layer and drainage system tributary to the bottom sump and a comparison of the capacity of the pump and operation of the pumping system, to the capacity of the sump and drainage layer. The analysis of the drainage layer includes an analysis of the flow capacity of the bottom drainage net in the vicinity of the sump. The transmissivity test values for the drainage net used in the design of the middle and uppermost systems were also used in the analysis of the bottom system. As described previously, the test results are conservative when applied to the bottom system because the boundary conditions of the bottom system provide better flow characteristics than the boundary conditions of the middle and uppermost systems.

Transmissivity values were determined from test results for the slopes and loading conditions of each sump area of the cell. Based on the transmissivity values obtained, the areas tributary to the bottom sumps and the capacities of the bottom sumps, the controlling sump area for the ALR is sump area no. 7 with a transmissivity of 7.5×10^{-3} m^2/sec (0.48 ft^2/min) and a resultant floor slope of 1.04 percent. This transmissivity value includes a safety factor of 4.2, as described above.

As presented in the calculations in Exhibit E, the flow capacity of the drainage net into bottom sump no. 7 is 53.8 gallons per day per foot (gpd/ft). Using the flow capacity of the drainage net of 53.8 gpd/ft, a total tributary area to the sump of 3.88 acres, and a safety factor of 2, the ALR based on the limiting factors of the drainage system is 391 gallons per acre per day.

The pumping system was determined not to be a limiting factor in determination of the ALR, with the ALR of the pumping system being on the order of 6,344 gallons per acre per day assuming a minimum pumping rate of 40 gpm and applying a safety factor of 2.

The system ALR is further limited by the operation of the leachate withdrawal system. If the pumping system is not automated and Laidlaw follows an inspection and pumping program of once a week for the bottom sumps, the sump capacity and pumping system control the ALR. According to the calculations presented in Exhibit E, sump no. 1 becomes the controlling sump without the use of an automated pumping system. Using a bottom sump capacity of 1,575 gallons, an inspection and pumping program of once per week, and a sump area of 4.54 acres, the resulting ALR is 25 gallons per acre per day. If the inspection and pumping program for the bottom sumps is conducted once each day, then the resulting ALR would be 173 gallons per acre per day.

This analysis has been conducted in accordance with the suggestions and requirements of the January 29, 1992 Federal Register "Part II Environmental Protection Agency 40 CFR Parts 260, 264, 265, 270, and 271 Liners and Leak Detection Systems for Hazardous Waste Land Disposal Units; Final Rule" (Federal Register, Volume 57, No. 19, Wednesday, January 29, 1992, Rules and Regulations).

3.4 <u>Erosion Control of Exterior Embankment Slopes</u>

Accounting for the expected precipitation and the exterior side slopes (2.1H:1V) of the landfill cell, design for embankment erosion protection has to be approached in a different manner than simply providing protection against erosion from overland flow. Standard solutions for erosion protection (such as vegetation) would not be feasible on the side slopes of the cell. A vegetative cover as dense as Bermuda Grass would be required to limit erosion to 2 tons/acre/year (as recommended by EPA), which would require extensive irrigation to maintain the cover in this semi-arid area. Thus, vegetation would not provide a good long term solution without ongoing maintenance.

Soil stabilizing chemicals have also been investigated as a means for controlling erosion. However, there appears to be little to no information available regarding the effectiveness of soil stabilizing chemicals in reducing erosion on steep slopes. There is also little supporting data regarding the longevity of soil stabilizing chemicals. This type of solution could therefore require significant maintenance over the active life of the cell, as well as during closure and beyond.

It has been determined that a rock covering would provide the best long term low-maintenance solution. However, for slopes steeper than 3H:1V, the overriding factor in designing rock as erosion protection is not the tangential shear stress acting on the surface of the rock from water flowing down the slope as overland flow, but the overriding factor becomes slope stability of the rock cover under saturated conditions (Duncan and Buchiganani, 1975). The movement of water down the side slopes of the cell from the 100-year, 24-hour precipitation event must be approached as ground-water interflow within the rock cover and not as a surface overland flow problem as will be demonstrated by the safety factors against slope failure for 2.1H:1V slopes presented hereafter.

Water will infiltrate into the void spaces between the rocks as precipitation falls on the rock cover. Runoff will then flow down the slopes through the void spaces in the rock simulating ground-water movement. Assuming laminar flow within the rocks, Darcy's Law for ground-water flow then becomes valid and the depth of flow within the rock is determined by the permeability of the rock material, the flow rate, and the hydraulic gradient.

Assuming interflow within the rock erosion protective cover, the ability of the rock to stay on 2H:1V side slopes was analyzed as a slope stability problem. Angular rock and filter materials with a friction angle of thirty-eight degrees and with seepage occurring through the void spaces in the rock and filter materials were assumed in the stability analysis. Applied Geotechnical Engineering Consultants (AGEC) determined that if the rock cover is totally saturated throughout its depth (a condition that must exist before overland flow over the rock surface could begin) the safety factor for the rock cover against slope failure is less than 1.0 for a 2H:1V side slope. Therefore, under saturated conditions the rock cover is not stable against slope failure for slopes as steep as 2H:1V. As the saturated depth of rock decreases with respect to the total depth of the rock, the safety factor increases. AGEC determined that in order to provide a safety factor of 1.5 for the rock cover against slope failure the saturated depth of the rock should not exceed 20 percent.

Therefore, the design of a stable rock cover for erosion protection requires that the thickness versus permeability of the rock be sized such that the depth of flow within the rock cover lies within the lower fifth of the total thickness of the rock cover (assuming angular rock). The more permeable the rock, the smaller will be the depth of flow within the rock. Use of a more permeable rock to limit the depth of flow will require a coarse rock gradation without fines, which would necessitate the use of a filter blanket between the rock cover and the embankment material. Otherwise, the embankment material would erode due to a pumping action created by flow within the rock. When considering the depth of flow and the corresponding required thickness of the rock material, the thickness of the filter blanket must be considered in determining the thickness of the rock cover. The filter blanket must likewise be composed of angular not rounded material.

Actually a two-layered filter blanket is required, the lower layer consisting of a three-inch thick finer-grained material (referred to as Type I) designed as a suitable filter for the embankment material, and the upper layer consisting of a four-inch thick coarser-grained material (referred to as Type II) designed as a suitable intermediate filter between the Type I filter blanket and the overlying protective rock covering. The gradations for these two filter layers are presented in Table 11.

The granular filter blanket acts as a part of the erosion protective rock structure except that the lower fine-grained filter blanket material has a lower permeability than the clean, poorly graded, angular rock of the upper layer of filter blanket material and the rock covering itself, so it can better resist erosion of the embankment soils. The saturation or seepage depth within the rock structure, therefore, includes the filter blanket materials.

Assuming a permeability of about 7.3 feet per minute for the upper filter blanket material and overlying protective rock covering and 0.018 feet per minute for the lower filter blanket material, the required thickness of the slope protective covering (including the filter blanket) to maintain a minimum safety factor against slope failure of 1.5 varies from about 21 inches at the top of the slopes to about 39 inches at the toe of the longest slope (see Appendix 3 in Exhibit C for calculations). Of this slope protective cover thickness, 7 inches would be granular filter material (3 inches for Type I and 4 inches for Type II materials) and the remainder would be a rock (riprap) protective cover. A mean rock diameter (D_{50}) of 9 inches was selected for this riprap protective cover. The gradation for this riprap layer having a mean rock diameter of 9 inches is presented as Type L riprap in Table 12.

The riprap protective cover will be keyed down into the ground along the toe of slope to a minimum depth (at the top of the riprap cover) of 1 foot in order to provide erosion protection for the slope protective cover around the outside toe of the embankment. In addition to the erosion protection on the 2.1H:1V exterior slopes, as described above, temporary erosion protection will be placed on the 3H:1V exterior slope located along the west embankment of Landfill Cell 15. By applying the same design procedures that were used for the 2.1H:1V exterior slopes, the required thickness of the slope protective cover (including the granular filters) to maintain a minimum safety factor against slope failure of 1.5 would be approximately 15 inches. Of this 15 inches, seven inches would be granular filter materials (three inches of the Type I and four inches of the Type II filter materials) and 8 inches would be riprap cover. A mean rock diameter of four inches was selected for the 8-inch thick riprap cover . The gradation for riprap with a mean diameter of four inches is also presented as Type V riprap in Table 12.

TABLE 11

GRANULAR FILTER BLANKET GRADATIONS

U.S. Standard Sieve Size	Percent Passing by Weight
TYPE I GRANULAR FILTER:	
3/8"	100
No. 4	95-100
No. 16	45-80
No. 50	10-30
No. 100	2-10
No. 200	0-2
TYPE II GRANULAR FILTER:	
3"	90-100
3/4"	35-90
No. 4	0-30
No. 16	0-15
No. 200	0-3

TABLE 12
RIPRAP GRADATIONS

Riprap Designation	% Smaller Than Given Size By Weight	Intermediate Rock* Weight (Lbs)	Intermediate Rock* Dimension (Inches)	D_{50}** (Inches)
Type L	100	350	16.2	9
	50	70-125	9.4-11.5	
	20	30	7.1	
Type V	70-100	43	8	4
	50-70	18	6	
	35-50	5.3	4	
	2-10	0.7	2	

* Dimension based on volume of cube and SG=2.3

** D_{50} = Nominal particle size

4.0 LANDFILL CELL CLOSURE

4.1 Closure Cap Layout and General Description

A final cover for a landfill should be designed to: (1) Provide long-term minimization of migration of liquid through the closed landfill; (2) function with minimum maintenance; (3) Promote drainage and minimize erosion or abrasion of the cover; (4) Accommodate settling and subsidence so that the cover's integrity is maintained; and (5) Have a cap liner system that has a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present. The closure cap for Landfill Cell 15 has been designed taking into consideration these requirements.

The closure cap of Landfill Cell 15 will consist of a geosynthetic clay liner (GCL) or two feet of compacted clay meeting a maximum permeability of 1×10^{-7} cm/sec, a HDPE geomembrane liner with a drainage system above the liner, a protective cover over the liner and drainage system, and an erosion protective cover over the protective cover. The closure cap will be constructed in the general shape of a "hipped roof" or elongated pyramid, with the cap surface sloping toward the outer edges of the cap at maximum slope of approximately ten percent. Grading the closure cap as proposed will assist in accommodating subsidence so that the cover's integrity is maintained. At the proposed slopes of ten percent, the cap could subside an additional eight feet over a horizontal distance of 100 feet and still maintain a slope of approximately two percent, thus, promoting drainage off the surface of the cap.

Landfill Cell 15 may be closed or partially closed in phases as the cell is filled with waste material in order to minimize rainfall impingement on active waste areas in the cell. Design drawings for the closure of Landfill Cell 15 are presented on sheets 40 through 44 in Exhibit A. Downspout and storm drainage pipes will be located around the cell to convey precipitation runoff from the surface of the closure cap (as the cell is partially closed and upon final closure) and from the top of the cell embankments to drainage channels located at the exterior toe of the north and east embankments.

4.2 Phased Closure

Three general phases of cell closure are presented in the design drawings which correspond approximately to the three phases of Landfill Cell 15 construction. For example, Phases I, II and III of the closure cap are located (with some minor variation) approximately above Phases I, II and III, respectively, of the landfill cell. Each of the three phases of closure may also be partially closed in sub-phases as waste material in portions of each phase is graded to its final elevations providing the subgrade to the closure cap.

Thus, closure of Landfill Cell 15 may be progressive during cell operation, the cell may be closed or partially closed in the designated phases or several phases may be closed as a single project.

It is anticipated that the landfill cell will be filled from the southern end of Phase I moving northward through Phases I and II, and then moving westward through Phase III. The waste may be placed in the cell such that the waste will regularly be brought to design grade as the filling of the cell progresses. As the waste is brought to design grade, the cell may be closed on a regular basis in phases and runoff from the closed portion of the cell will be directed away from the active working area of the cell as part of the run-on management system. This process of filling and closing the cell in phases may proceed in such a manner so as to limit the active working area in the cell. Thus, the volume of runoff water generated from the active areas of the cell can be minimized and the volume of leachate generation will also be minimized.

4.3 Design

Typical cross-sections and details of the closure cap are illustrated on sheet 44A (for a cap consisting of a GCL and sheet 44B (for a cap consisting of two feet of compacted clay) of the drawings in Exhibit A. The closure cap will consist of the following:

1. A final waste surface that has been graded, compacted and prepared to receive cap materials.

2. A low permeable soil layer consisting of either a GCL or two feet of compacted clay as follows:

   a. If a GCL is used:
      - A 6-inch thick compacted layer of soil will be placed upon the waste surface if a GCL is used in order to provide a better subgrade condition for the GCL.
      - A geosynthetic clay liner (GCL) which has equivalent or improved permeability characteristics to a two-foot thick compacted clay liner.

   b. Two feet of compacted clay cap material meeting a maximum in-place permeability of 1×10^{-7} cm/sec.

3. A 60-mil HDPE geomembrane liner. Since the cap will consist of a geomembrane liner, it will have a permeability that is less than or equal to the permeability of the bottom liner system in the cell.

4. A middle drainage layer consisting of a drainage net with overlying geotextile filter fabric. The middle drainage layer will convey water, which percolates through the overlying cap materials, off the underlying geomembrane liner. The drainage net will be placed on the ten percent slope paralleling the surface of the closure cap. The edge of the drainage net will extend into the erosion protective cover around the edges of the cap to allow water that enters the drainage net to drain freely. However, the majority of the drainage water collected in the net will be intercepted by a perforated drainage pipe, which is to be located around the perimeter of the cap directly underneath the flow line of the drainage collection ditch of the

cap. This pipe will be placed on the same slope as the drainage collection ditch (i.e. 0.5%) and will be connected into the proposed downspouts. The cap drainage layer consists of SLT GS-228 (or other drainage nets, such as Gundle XL-14, that meet or exceed the drainage characteristics of SLT GS-228) underlying a Tensar TG-700 filter fabric (or other equivalent or improved filter fabric materials). Test information supplied by SLT under a normal load of 6500 pounds per square foot (much greater than will be experienced by the cell cap liner) indicates that SLT GS-228 drainage has a transmissivity of 2.5×10^{-3} m^2/sec on a ten percent slope.

Koerner (1990) suggests that safety factors can be applied to the test results value to account for creep deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space and for biological and chemical clogging in the geonet's core space. In accordance with the procedures recommended by Koerner (1990), a safety factor of 1.4 against creep deformation or intrusion of the adjacent geosynthetic into the geonet's core space, and an additional safety factor of 2 against potential biological and chemical clogging of the geonet have been applied to the SLT GS-228 test results. Applying an additional design-by-function safety factor of 1.5 (EPA, 1989) produces a combined safety factor for the drainage net of 4.2 and an allowable design transmissivity for the SLT GS-228 drainage net of 0.6×10^{-3} m^2/sec which is approximately equivalent to a foot of sand with a saturated hydraulic conductivity of 0.20 cm/sec.

5. A 2-foot soil protective cover that will provide frost protection for the liner. The regional depth of frost penetration map for the United States (EPA, 1980) indicates that the frost depth at the Lone Mountain Facility is about 10 inches. The protective cover and erosion protective cover thicknesses should, therefore, provide adequate frost protection.

6. Erosion protective cover consisting of Type II granular filter and Type V riprap inside the ditches around the perimeter of the cap and Type I and Type II granular filters and riprap on the berm and the 2H:1V slopes around the perimeters of the cap.

7. Berms, ditches, downspout pipes, storm drainage pipes and other drainage facilities to control and convey runoff from the cap.

Based on calculations performed by AGEC, safety factors against failure of the main closure cap area are 1.6 under static conditions and 1.1 under seismic conditions assuming the geonet is placed such that the length of the roll is placed parallel to the slope of the cap. The safety factors are 3.3 under static conditions and 1.7 under seismic conditions assuming the geonet is placed such that the length of the rolls is perpendicular to the slope of the cap. The stability calculations were based on use of textured HDPE liner.

Safety factors against failure of the perimeter berms around the closure cap are 1.5 under static conditions and 1.4 under seismic conditions. The letter provided by AGEC regarding the results of the stability analysis is included near the end of Exhibit B.

Closure actions include the following:

1. Preparation of the Waste Mound

The waste surface at the top of the cell must be amenable for closure. Proper selection, compaction, slope and grading of the waste is necessary to ensure the integrity of the cap design. Incoming waste free of sharp objects and debris will make up the final or top one-foot of waste placed in the cell in order to protect the overlying cell cap. The cell will be shaped and contoured to conform to the final grading plan for the waste. The cap will be graded at a maximum slope of approximately ten percent. The contouring of the waste will reduce the subsequent need for additional fill material, facilitate grading of the cap, and reduce the possible formation of depressions that could pond water.

2. Unclassified Soil Material

Following completion of the waste surface preparation, an unclassified soil material will be placed and compacted on top of the waste surface at a thickness of approximately 6 inches where a GCL is to be placed. This unclassified soil will be graded to conform to the designated cap slopes (i.e. maximum slopes of approximately ten percent).

3. Low Permeable Soil Layer (Geosynthetic Clay Liner (GCL) or Compacted Clay Cap)

As indicated above, closure of the cell may proceed in phases soon after waste in a given phase has been prepared to receive the low permeable soil layer. Closure may begin at the southern end of Phase I and progress northward to coincide with the proposed placement plan for waste in the cell. Placement of the low permeable soil layer will be initiated and will progress such that drainage of precipitation runoff from the closure cap and from the adjacent waste material will be away from the low permeable soil layer. The HDPE liner will immediately be placed above any GCL that is placed to prevent moisture resulting from precipitation from coming into contact with the GCL.

4. HDPE Liner

A 60-mil HDPE geomembrane liner will be installed above the low permeable soil layer. The HDPE liner in conjunction with the underlying low permeable soil layer will provide for the long-term minimization of liquid migration through the closed cell. Quality control and quality assurance for HDPE liner installation will be ensured during construction by implementation of the CQA plan.

Slope stability analysis of the 2H:1V side slope around the perimeter of the cap indicates that the plane governing stability occurs along the HDPE liner interface with the soil. The slope stability analysis results (see computations in Exhibit B) indicate that the safety factor against slippage on the 2H:1V side slopes between the textured HDPE liner and the compacted soil is 1.8 under static conditions and 1.6 under dynamic conditions.

5. Drainage Net and Filter Fabric

A drainage net will be placed on top of the HDPE liner to function as a drainage media for water that infiltrates the surface soil. A layer of geotextile filter fabric will be installed directly above the drainage net to prevent clogging of the drainage net by the overlying soil. The drainage net and the filter fabric will be installed at the same time as the soil protective cover.

6. Soil Protective Cover

A 2-foot thick soil protective cover layer will be placed over the drainage layer. It is anticipated that the soil protective cover meeting the Unified Soil Classification designations CL, ML, SM, SC, SP, SW or combinations thereof may be obtained from borrow sources near the Lone Mountain Facility.

7. Erosion Protective Cover

The erosion protective cover across the interior portion of the cap (inside the ditch around the perimeter of the cap) will consist of a 4-inch layer of Type II granular filter material above the soil protective cover and a 6-inch layer of Type V riprap. The erosion protective cover on the berm and on the 2H:1V slopes around the perimeter of the cap will consist of a 3-inch layer of Type I granular filter, a 4-inch layer of Type II granular filter, and a 6-inch layer of Type V riprap on the berm and a 12-inch layer of Type V riprap on the 2H:1V sideslope of the cap..

4.3.1 Surface Water Drainage and Erosion Control

The final closure cap, as illustrated by the closure details presented in Exhibit A, will be constructed such that it is separated into eight drainage areas consisting of plane surfaces sloped toward drainage collection ditches around the outside perimeter of the closure cap. The drainage collection ditches will be graded on an approximate 0.5 percent slope toward pipe downspouts that will be constructed at low points in each drainage area around the perimeter of the cap. Runoff water will be introduced into the pipe downspouts via a concrete inlet box. The pipe downspouts will consist of either a single 18-inch diameter corrugated polyethylene pipe or a set of two 18-inch diameter corrugated polyethylene pipes. The downspouts will convey the runoff from the cap areas described above to either existing channel no. 4 located along the north side of Cell 15, existing channel no. 5 located along the east side of Cell 15, or to storm drainage pipes that will be constructed in the common embankments between Cells 14 and 15 and in the common embankment between Cells 12 and 15. The storm drainage pipes will convey stormwater runoff to the extreme northwest corner and the extreme southeast corner of Landfill Cell 15, down the exterior embankment slopes and into channel no.'s 4 and 5.

The design of the drainage collection ditches, downspouts and storm drainage pipes for the closure cap is based on the 100-year, 24-hour precipitation event with peak flows generated using the curve number methodology and unit hydrograph procedure developed by the USDA Soil Conservation Service (SCS). Hydrologic calculations are presented in Exhibit F. The rainfall depth for the 100-year, 24-hour precipitation event at the Lone Mountain Facility is 8.0 inches. Determination of the value of curve number for use in the SCS procedure was accomplished through the use of information published by the Soil Conservation Service.

A curve number of 75 was selected for runoff design from the surface of the final cover based on a Hydrologic Soil Type A with a gravel or rock cover.

1. Drainage Collection Ditches

Using the SCS curve number methodology, the peak discharge from the 100-year, 24-hour precipitation event of 8.0 inches to the drainage collection ditches with the largest tributary area on the top of the closure cap was estimated to be 20.2 cfs. The analysis assumed that all of the flow generated from the largest tributary area would be contained within a single drainage collection ditch. Actually, the flow will be divided between two drainage collection ditches, therefore the analysis performed is conservative. The drainage collection ditches will have a triangular cross sectional area with the approximate 10 percent slope (10H:1V) of the cap forming one side of the ditch and a berm forming the outside of the ditch, constructed with 2H:1V side slopes to a height of 2.83 feet from the flow line of the ditch to the top of the riprap cover on the berm (see design drawings in Exhibit A). The riprap and Type II granular filter will not contain runoff water without the water seeping through the riprap and granular filter. Thus, the actual design of the drainage collection ditches is based on the flow depth in the ditches and providing one-foot of freeboard to the top of the Type I granular filter. The actual design depth of the collection ditches is considered to be 2 feet. At a flow rate of 20.2 cfs and a channel slope of 0.5 percent with a rock lining, these ditches will have a flow depth of 1.2 feet and a velocity of about 2.3 feet per second (see hydrologic calculations in Exhibit F). At a velocity of 2.3 feet per second, the riprap erosion protective cover of the cap will be adequate to prevent erosion along the drainage collection ditches. With the design depth of the drainage collection ditches of 2 feet and the maximum flow depth of 1.2 feet, the minimum freeboard in the drainage collection ditches during the 100-year, 24-hour precipitation event will be 0.8 foot to the top of the Type I granular filter material.

2. Downspouts

The closure cap will be drained by eight downspouts that are numbered as Downspout No. D1 through Downspout No. D8 (see sheet 40 of the drawings in Exhibit A). Table 13 presents a summary of the design information for each of the downspouts. Supporting calculations for each of the downspouts are presented in Exhibit F.

3. Storm Drainage Pipes

The storm drainage pipes will consist of smooth lined corrugated polyethylene pipe and will receive and convey runoff water from Downspouts D1, D2, D3 and D4; from the north half of the closure cap for Landfill Cell 14; from the roadways formed by the common embankments between Cells 14 and 15 and between Cells 12 and 15; and from half of the roadways formed by the common embankments between Cells 12 and 13 and between Cells 13 and 14. The storm drainage pipes are numbered P1 through P8 and are shown on sheet 40 of the drawings in Exhibit A. The design information for the storm drainage pipes is summarized in Table 14.

Manholes will be located at every downspout junction with the storm drainage pipes, at the southeast and southwest corners of Phase I, and at the northwest and southwest corners of Phase III.

Inlets to the storm drainage pipes from runoff water generated on the roadways on top of the cell embankments will consist of inlet boxes or grated covers in the manholes.

TABLE 13

DOWNSPOUT PIPE DESIGN INFORMATION

Downspout	Contributing Area (acres)	Flow Rate (cfs)	Number and Size of Downspout Pipes	Headwater Depth at Downspout Inlet (feet)
D1	2.0	9.6	1-18" Dia.	<3.0
D2	3.6	16.8	2-18" Dia.	<2.6
D3	2.3	11.7	1-18" Dia.	<3.0
D4	2.3	10.8	1-18" Dia.	<3.0
D5	2.3	10.9	1-18" Dia.	<3.0
D6	3.4	16.2	2-18" Dia.	<2.6
D7	4.4	20.2	2-18" Dia.	<2.6
D8	3.6	16.7	2-18" Dia.	<2.6

Note: Inlets to all of the downspouts are to be USBR Type 4 inlets.

TABLE 14

STORM DRAINAGE PIPE DESIGN INFORMATION

Pipe Number	Contributing Area (acres)	Flow Rate (cfs)	Pipe Size (inches)	Pipe Slope (ft/ft)
P1	8.6	50.8	36	0.005
P2	5.5	26.9	30	0.006
P3	2.8	14.0	24	0.006
P4	4.2	19.5	30	0.005
P5	6.1	28.8	30	0.005
P6	8.5	40.3	36	0.005
P7	8.7	41.3	30	0.15
P8	8.7	51.5	30	0.15

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EXHIBIT A

DESIGN DRAWINGS
LANDFILL CELL 15

HA&L
ENGINEERING

EXHIBIT A

DESIGN DRAWINGS
LANDFILL CELL 15

LAIDLAW ENVIRONMENTAL SERVICES (LONE AND GRASSY MOUNTAIN), INC.

LONE MOUNTAIN FACILITY

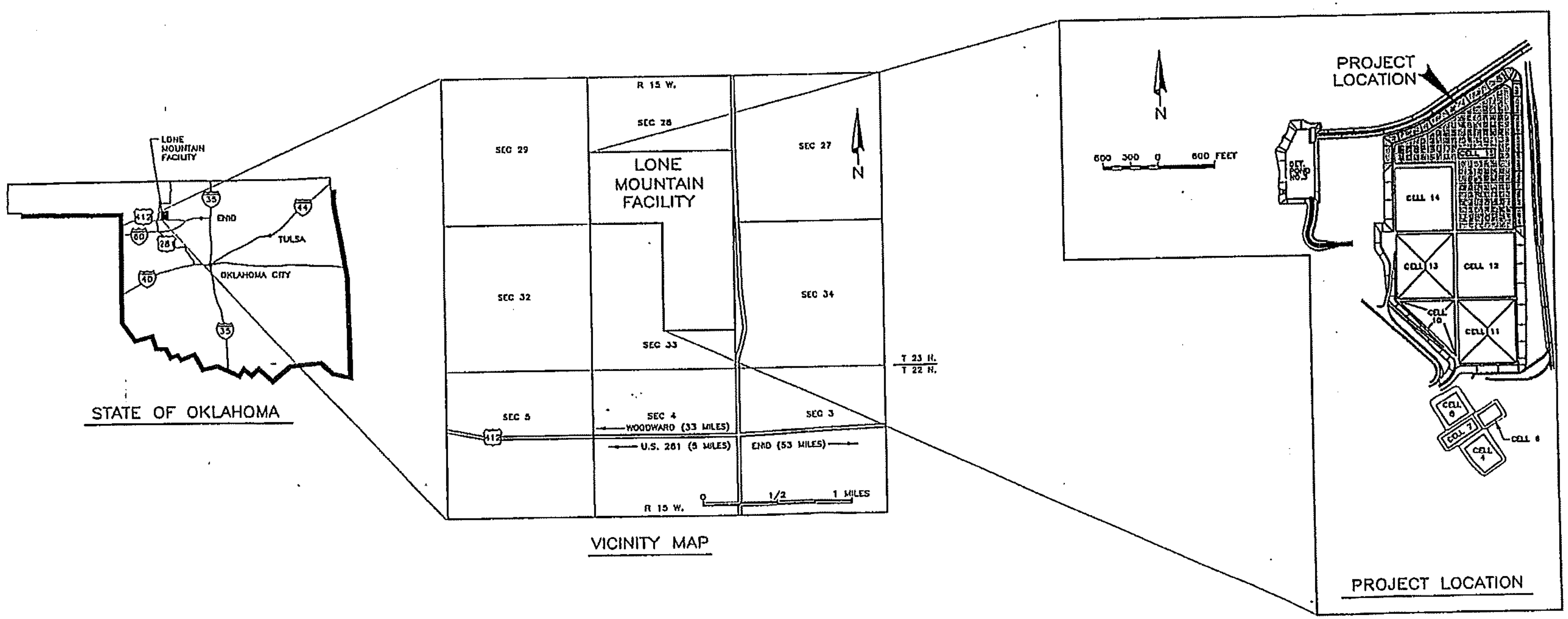
LANDFILL CELL 15

CORPORATE OFFICES

220 OUTLET POINTE BOULEVARD
COLUMBIA, SOUTH CAROLINA 29210
PHONE: (803)798-2993

LONE MOUNTAIN FACILITY

ROUTE 2, BOX 170
WAYNOKA, OKLAHOMA 73860-9622
Phone: (405) 697-3500



DESIGNED	MPW	3			
DRAFTED	JVH	2			
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY APVD.

HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

PROJECT ENGINEER

SCALE AS SHOWN

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
TITLE SHEET

USPCI, INC.
RECOMMENDED FOR APPROVAL

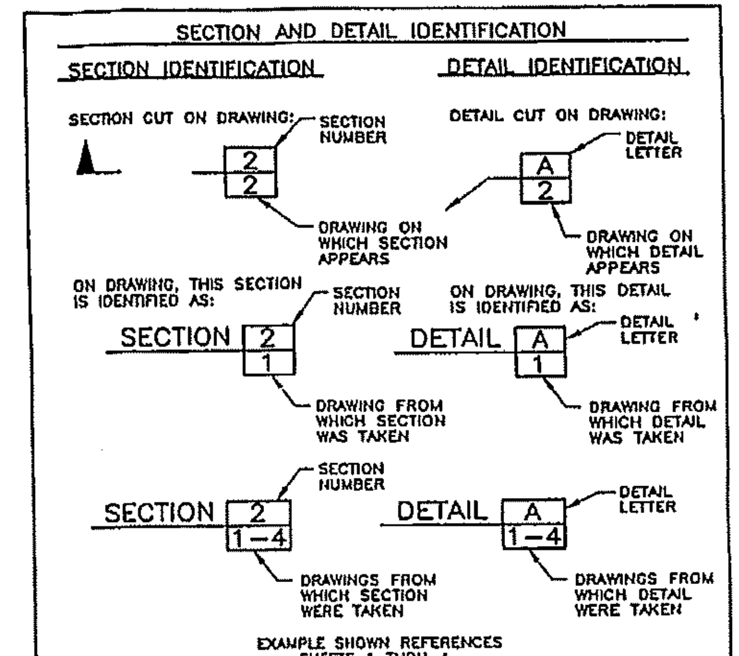
NAME TITLE DATE

INDEX OF DRAWINGS

SHEET NO.	TITLE	REVISION
1.	TITLE SHEET	1
2.	INDEX OF DRAWINGS	1
3.	SITE PLAN	1
4.	PLAN VIEW – BERM PLACEMENT	1
5A.	PLAN VIEW – PHASE IA	1
5B.	PLAN VIEW – PHASE IB	1
6.	PLAN VIEW – PHASE II	1
7.	PLAN VIEW – PHASE III	1
8.	TYPICAL SECTIONS – PHASE I	1
9.	TYPICAL SECTIONS – PHASES II & III	1
10.	UPPERMOST LEACHATE COLLECTION SYSTEM PHASE I	1
11.	UPPERMOST LEACHATE COLLECTION SYSTEM PHASE II	1
12.	UPPERMOST LEACHATE COLLECTION SYSTEM PHASE III	1
13.	BOTTOM SUMP NO. 1 – PHASE IA	1
14.	BOTTOM SUMP NO. 2 – PHASE IA	1
15.	BOTTOM SUMP NO. 3 – PHASE IB	1
16.	BOTTOM SUMP NO. 4 – PHASE II	1
17.	BOTTOM SUMP NO. 5 – PHASE II	1
18.	BOTTOM SUMP NO. 6 – PHASE II	1
19.	BOTTOM SUMP NO. 7 – PHASE III	1
20.	BOTTOM SUMP NO. 8 – PHASE III	1
21.	MIDDLE SUMP NO. 1 – PHASE IA	1
22.	MIDDLE SUMP NO. 2 – PHASE IA	

SHEET NO.	TITLE	REVISION
23.	MIDDLE SUMP NO. 3 – PHASE IB	1
24.	MIDDLE SUMP NO. 4 – PHASE II	1
25.	MIDDLE SUMP NO. 5 – PHASE II	1
26.	MIDDLE SUMP NO. 6 – PHASE II	1
27.	MIDDLE SUMP NO. 7 – PHASE III	1
28.	MIDDLE SUMP NO. 8 – PHASE III	1
29.	UPPERMOST SUMP NO. 1 – PHASE IA	1
30.	UPPERMOST SUMP NO. 2 – PHASE IA	1
31.	UPPERMOST SUMP NO. 3 – PHASE IB	1
32.	UPPERMOST SUMP NO. 4 – PHASE II	1
33.	UPPERMOST SUMP NO. 5 – PHASE II	1
34.	UPPERMOST SUMP NO. 6 – PHASE II	1
35.	UPPERMOST SUMP NO. 7 – PHASE III	1
36.	UPPERMOST SUMP NO. 8 – PHASE III	1
37.	SUMP SECTIONS	1
38.	TYPICAL SECTIONS & DETAILS	1
39.	INTERIOR RAMP DETAILS – PHASE IA	1
	CLOSURE DRAWINGS	
40.	CLOSURE – PLAN VIEW	1
41.	CLOSURE – PLAN VIEW PHASE I	1
42.	CLOSURE – PLAN VIEW PHASE II	1
43.	CLOSURE – PLAN VIEW PHASE III	1
44A.	CLOSURE – SECTIONS & DETAILS GEOSYNTHETIC CLAY LINER OPTION	1
44B.	CLOSURE – SECTIONS & DETAILS COMPACTED CLAY CAP OPTION	1

SECTION & DETAIL IDENTIFICATION



NOTES:
1. IF PLAN AND SECTION (OR DETAIL CALL-OUT AND DETAIL) ARE SHOWN ON SAME DRAWING, DRAWING NUMBER IS REPLACED BY A LINE.
2. DETAIL LETTERS "I" AND "O" NOT USED

ABRIDGED TABLE OF ABBREVIATIONS

EL.	ELEVATION	CC	CENTER TO CENTER
INV. EL.	INVERT ELEVATION	FL	FLOW LINE
STA.	STATION	℄	CENTER LINE
PI	POINT OF INTERSECTION	BPS	BOTTOM OF MIDDLE SUMP
PC	POINT OF CURVE	TPS	TOP OF MIDDLE SUMP
PT	POINT OF TANGENT	TL	TOP OF LINER
NTS	NOT TO SCALE	SDR	STANDARD DIMENSIONAL RATIO
DIA.	DIAMETER	PVC	POLYVINYL CHLORIDE
TYP.	TYPICAL	HDPE	HIGH DENSITY POLYETHYLENE
CLR.	CLEAR	MIN.	MINIMUM
PL	PLATE	MAX.	MAXIMUM
CPP	CORRUGATED POLYETHYLENE PIPE		
SCPP	SMOOTH WALL CORRUGATED POLYETHYLENE PIPE		
PCPP	PERFORATED CORRUGATED POLYETHYLENE PIPE		

HA&L ENGINEERING CONSULTANTS ENGINEERS Salt Lake City Utah

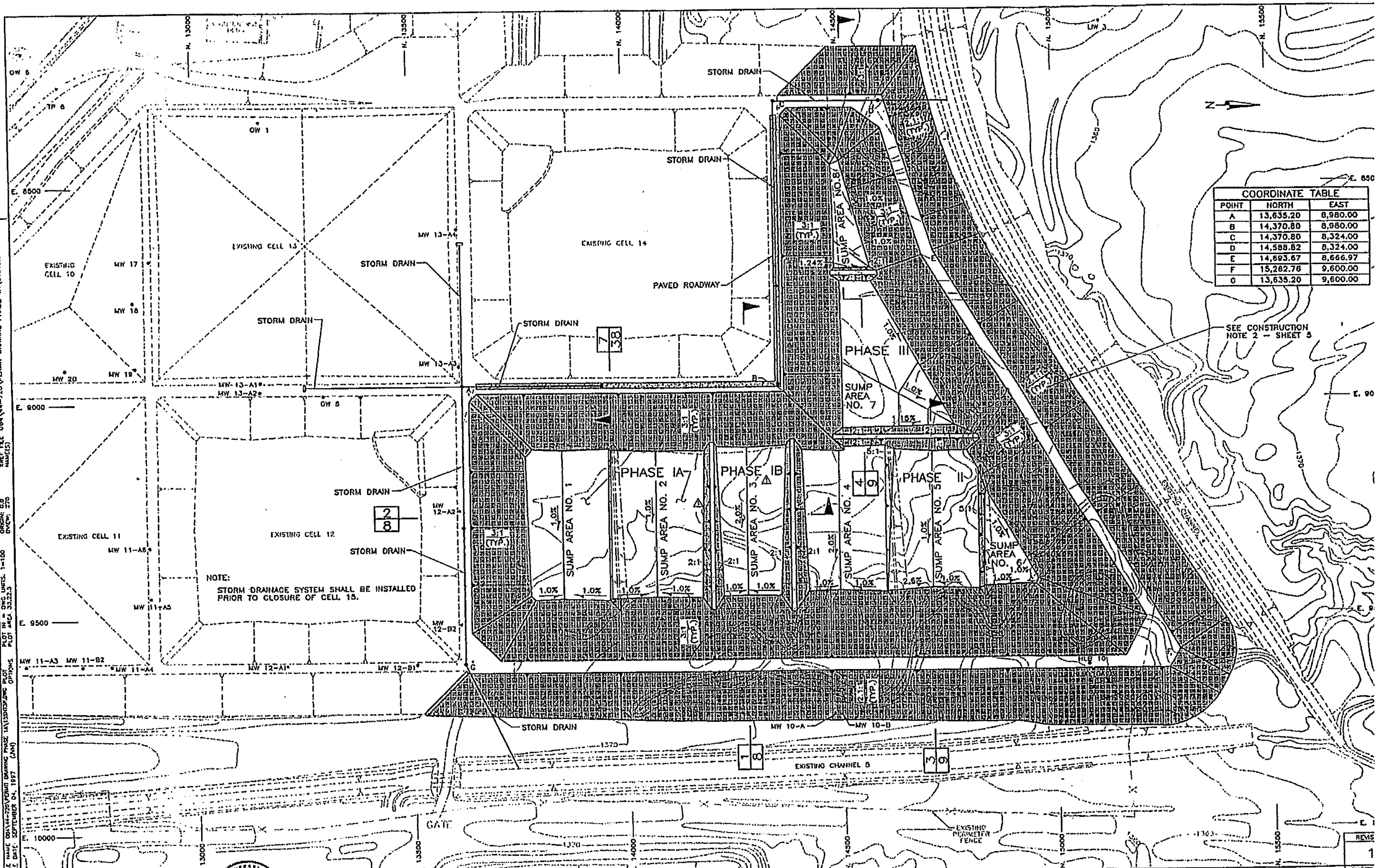
DESIGNED	MPW	3		
DRAFTED	JVH	2		
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME – ADDED SHEET NO. 5B — JVH KCS
DATE	JULY 1996	NO.	DATE	REVISIONS — BY APVD.

SCALE NOT TO SCALE

VERIFY SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
INDEX OF DRAWINGS



COORDINATE TABLE		
POINT	NORTH	EAST
A	13,635.20	8,980.00
B	14,370.80	8,980.00
C	14,370.80	8,324.00
D	14,588.82	8,324.00
E	14,693.67	8,666.97
F	15,262.76	9,600.00
G	13,635.20	9,600.00

HA&L ENGINEERING
CONSULTANTS ENGINEERS
Salt Lake City Utah

PROJECT ENGINEER

DESIGNED MPW	3					
DRAFTED JVH	2					
CHECKED KCS	△	9/97	UPDATED COMPANY NAME – MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASES 1A & 1B	JMI	KCS
DATE JULY 1996	NO.	DATE	REVISIONS	BY	APVD.

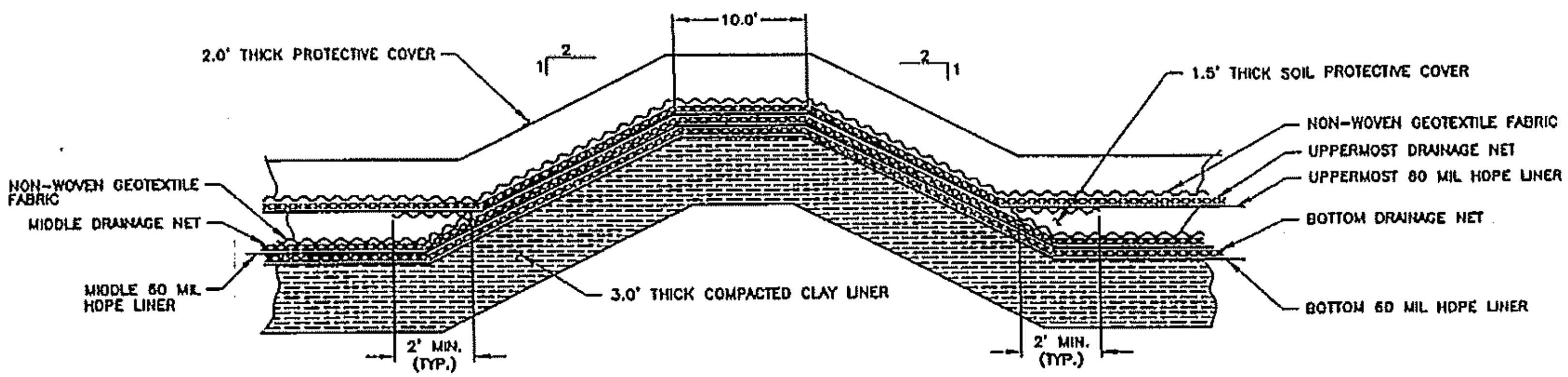
SCALE 1"=100"

VERIFY SCALE
0" 1"
BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

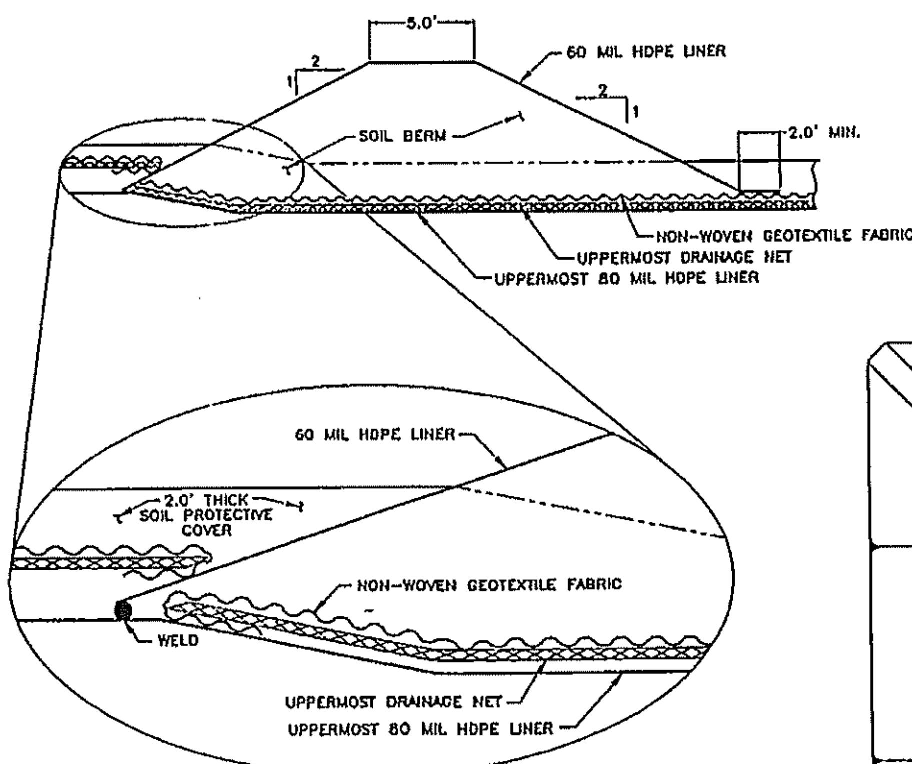
LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
SITE PLAN

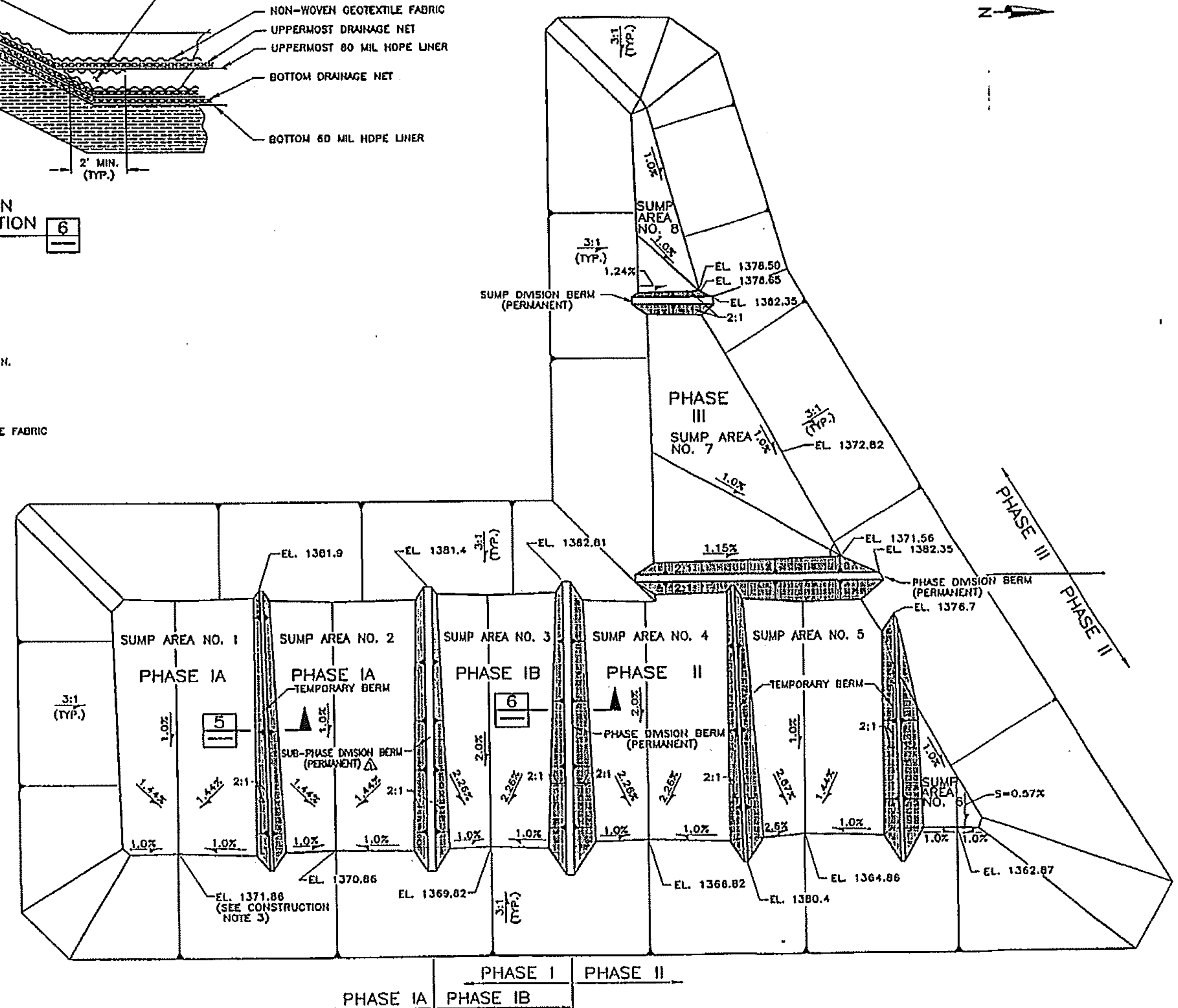
REVIS 1
SHEET 3 OF 4



PHASE & SUB-PHASE DIVISION AND SUMP DIVISION BERM SECTION 6
N.T.S.



TEMPORARY BERM SECTION 5
N.T.S.



PLAN VIEW
1"=80'

CONSTRUCTION NOTES:

1. THE INTERIOR AND EXTERIOR EMBANKMENT SLOPES SHALL BE NO STEEPER THAN THE APPROXIMATE DESIGNATED SLOPE.
2. THE DESIGNATED SLOPE ON THE INTERIOR FLOOR OF THE CELL IS AN APPROXIMATE AVERAGE SLOPE FOR THE CELL FLOOR.
3. ELEVATIONS INSIDE OF CELL ARE TOP OF UPPERMOST LINER SYSTEM.

OPERATIONAL NOTE:

1. EACH TEMPORARY AREA BERM MAY BE REMOVED BY USPCI OPERATIONS AS WASTE PLACEMENT IN CELLS PROGRESSES INTO ADJACENT SUMP AREAS BEYOND EACH TEMPORARY BERM.

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PROJECT ENGINEER

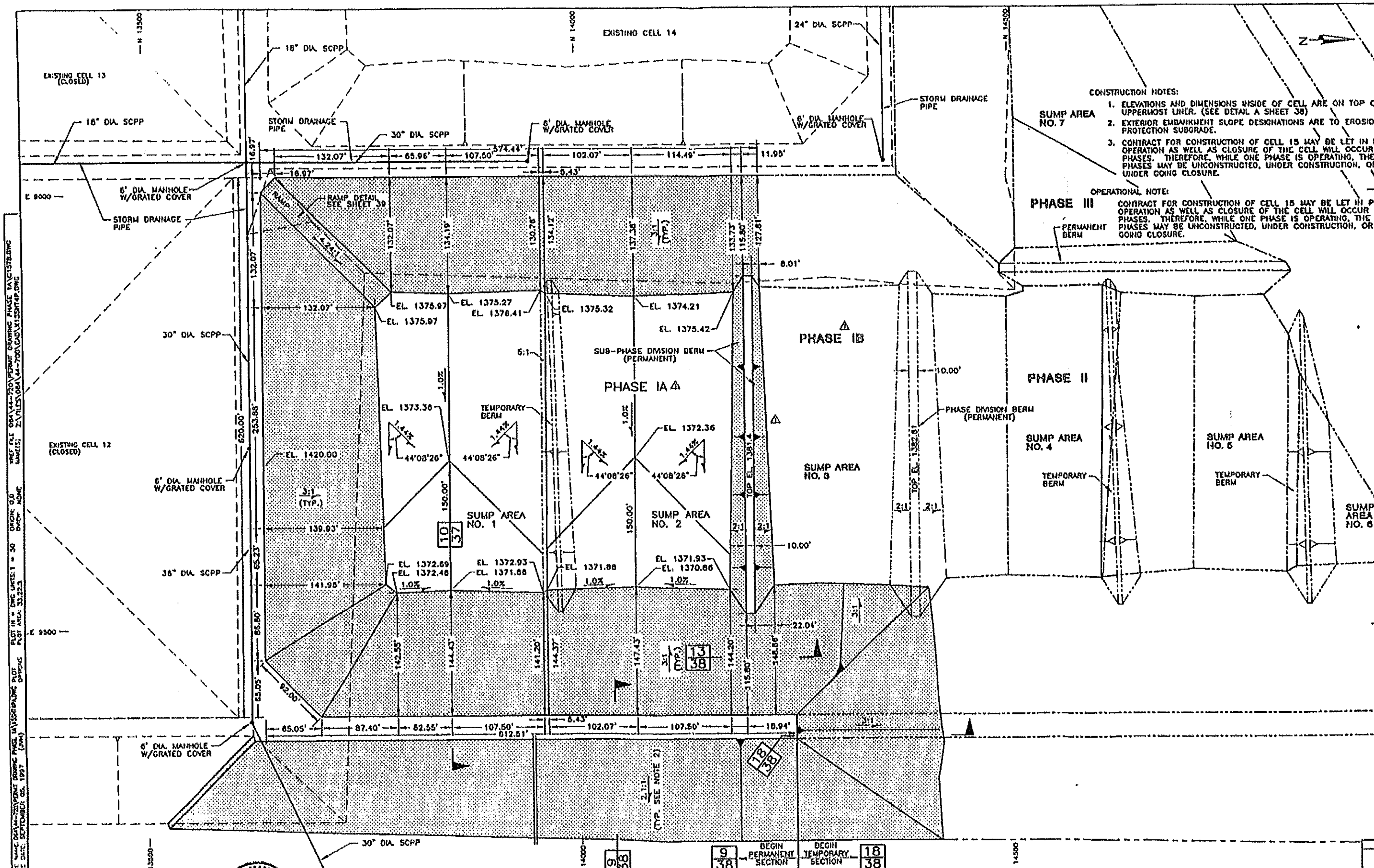
DESIGNED	MPW	3		
DRAFTED	JVH	2		
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME – MODIFICATIONS TO DIVIDE PHASE 1 INTO SUB-PHASES 1A & 1B — JVH — KCS
DATE	JULY 1996	NO.	DATE	REVISIONS — BY — APVD.

SCALE: AS SHOWN

VERIFY SCALE — BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
PLAN VIEW – BERM PLACEMENT



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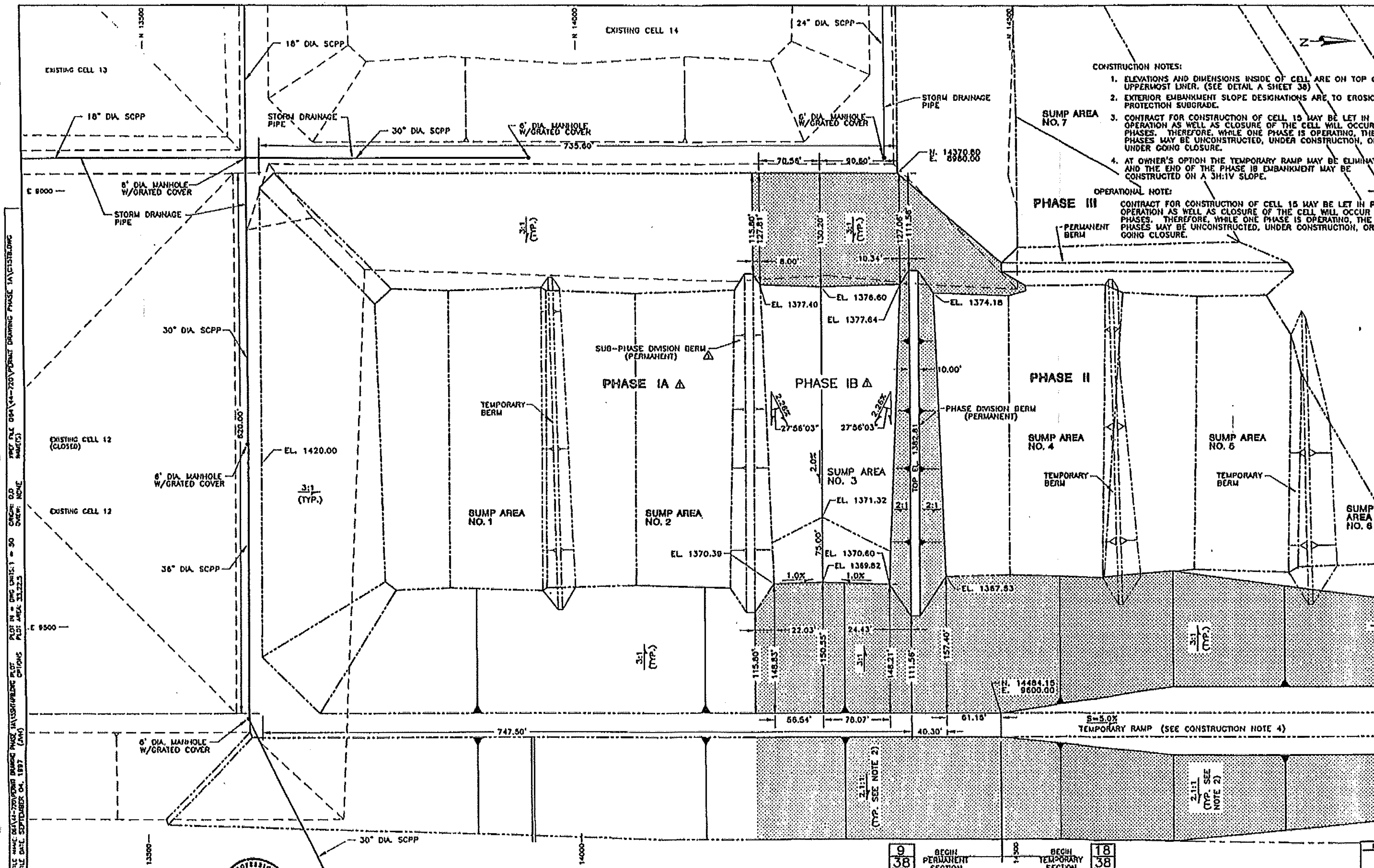
DATE	JULY 1996	NO.	DATE	REVISIONS	BY	APVD.

SCALE 1" = 50'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
PLAN VIEW – PHASE IA



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DATE	JULY 1996	NO.	DATE — REVISIONS — BY — APVD.

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PROJECT ENGINEER

SCALE 1" = 50'

VERIFY SCALE — BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

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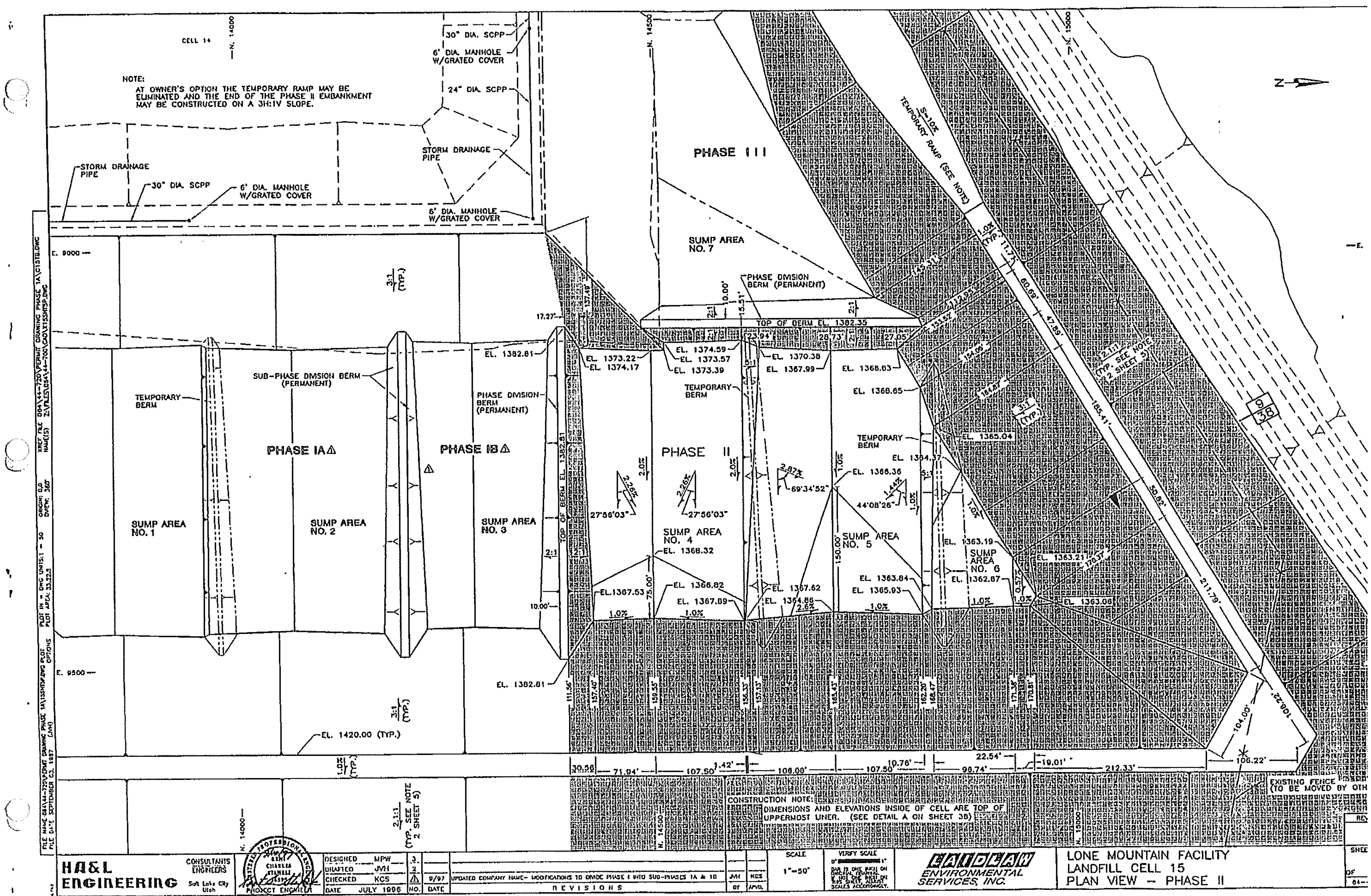
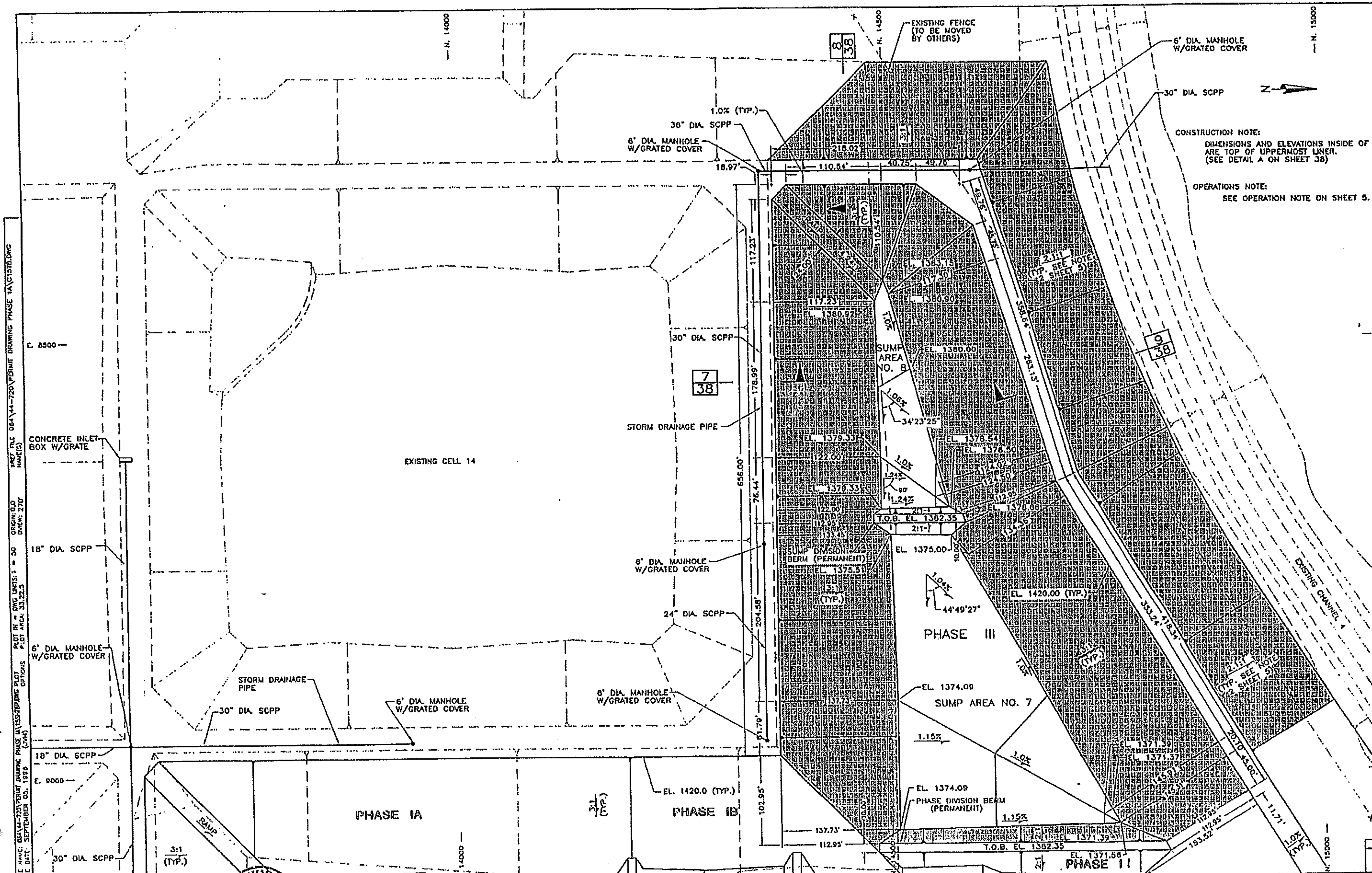
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
PLAN VIEW – PHASE IB

SHEET 5



CELL 14
NOTE:
AT OWNER'S OPTION THE TEMPORARY RAMP MAY BE ELIMINATED AND THE END OF THE PHASE II EMBANKMENT MAY BE CONSTRUCTED ON A 3H:1V SLOPE.
30" DIA. SCPP
6' DIA. MANHOLE W/GRATED COVER
24" DIA. SCPP
STORM DRAINAGE PIPE
30" DIA. SCPP
6' DIA. MANHOLE W/GRATED COVER
PHASE III
SUMP AREA NO. 7
PHASE DIVISION BERM (PERMANENT)
TOP OF BERM EL. 1382.35
5-10% TEMPORARY RAMP (SEE NOTE)
TEMPORARY BERM
SUB-PHASE DIVISION BERM (PERMANENT)
PHASE DIVISION BERM (PERMANENT)
PHASE IA
PHASE IB
PHASE II
SUMP AREA NO. 1
SUMP AREA NO. 2
SUMP AREA NO. 3
SUMP AREA NO. 4
SUMP AREA NO. 5
SUMP AREA NO. 6
TOP OF BERM EL. 1382.81
EL. 1382.81
EL. 1420.00 (TYP.)
3:1 (TYP.)
2.1:1 (TYP. SEE NOTE 2 SHEET 5)
EXISTING FENCE (TO BE MOVED BY OTH
CONSTRUCTION NOTE:
DIMENSIONS AND ELEVATIONS INSIDE OF CELL ARE TOP OF UPPERMOST LINER. (SEE DETAIL A ON SHEET 38)
HA&L ENGINEERING
CONSULTANTS ENGINEERS Salt Lake City Utah
DESIGNED MPW
DRAFTED JVH
CHECKED KCS
DATE JULY 1996
9/97 UPDATED COMPANY NAME- MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASES 1A & 1B
REVISIONS
SCALE 1"=50'
LAIDLAW ENVIRONMENTAL SERVICES, INC.
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
PLAN VIEW - PHASE II



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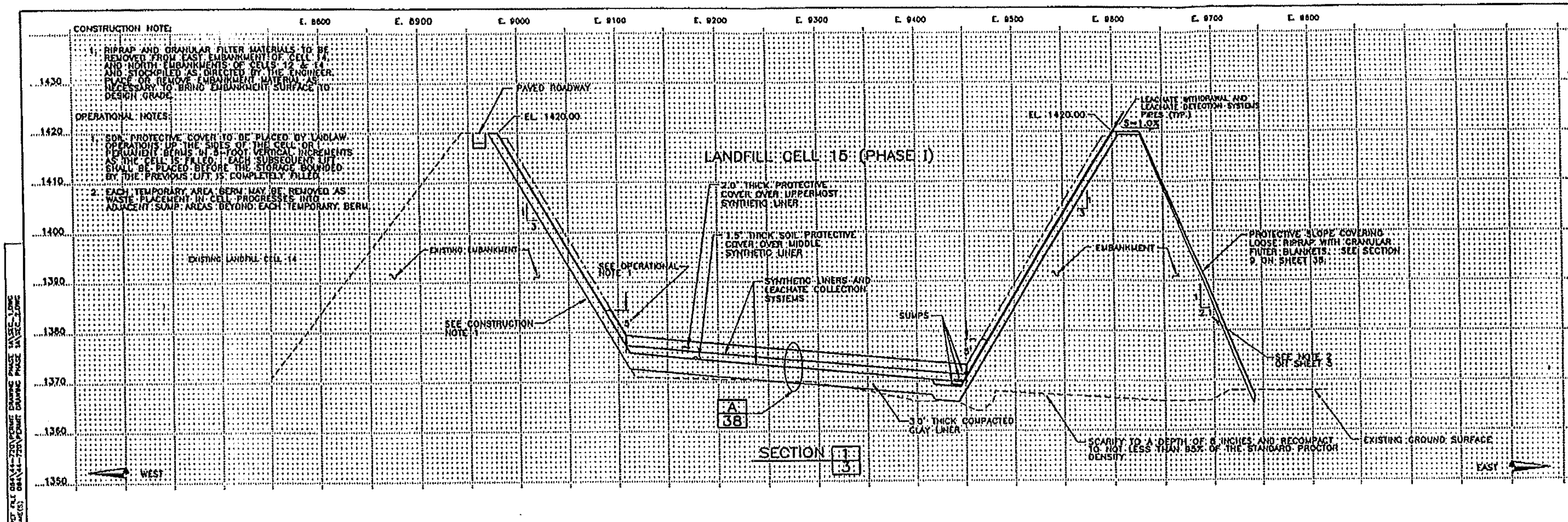
DESIGNED	MPW	3	
DRAFTED	JVH	2	
CHECKED	KCS	1	9/97 UPDATED COMPANY NAME ~ MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASE IA & IB — JVH — KCS
DATE	JULY 1996	NO.	DATE — REVISIONS — BY — APVD.

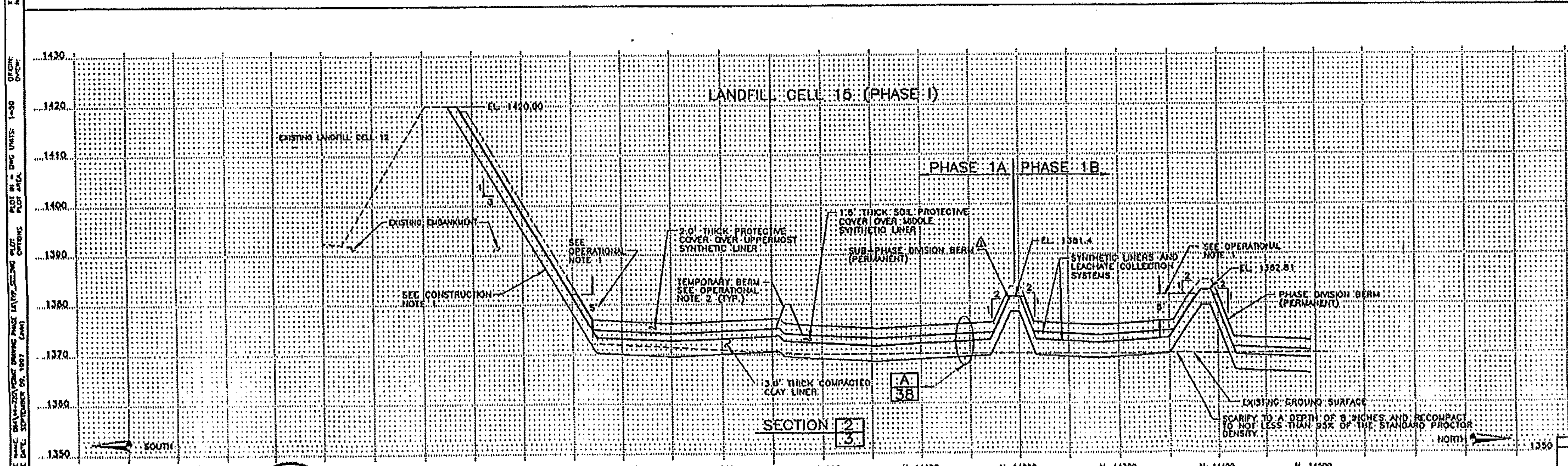
SCALE 1" = 50'

VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
PLAN VIEW – PHASE III





DESIGNED MPW	3			
DRAFTED JVH	2			
CHECKED KCS	1	9/97	UPDATED COMPANY NAME – MODIFICATIONS TO DMOE PHASE I INTO SUB-PHASES IA & IB	JVH KCS
DATE JULY 1996	NO.	DATE	REVISIONS	BY APVD.

HA&L ENGINEERING CONSULTANTS ENGINEERS Salt Lake City Utah

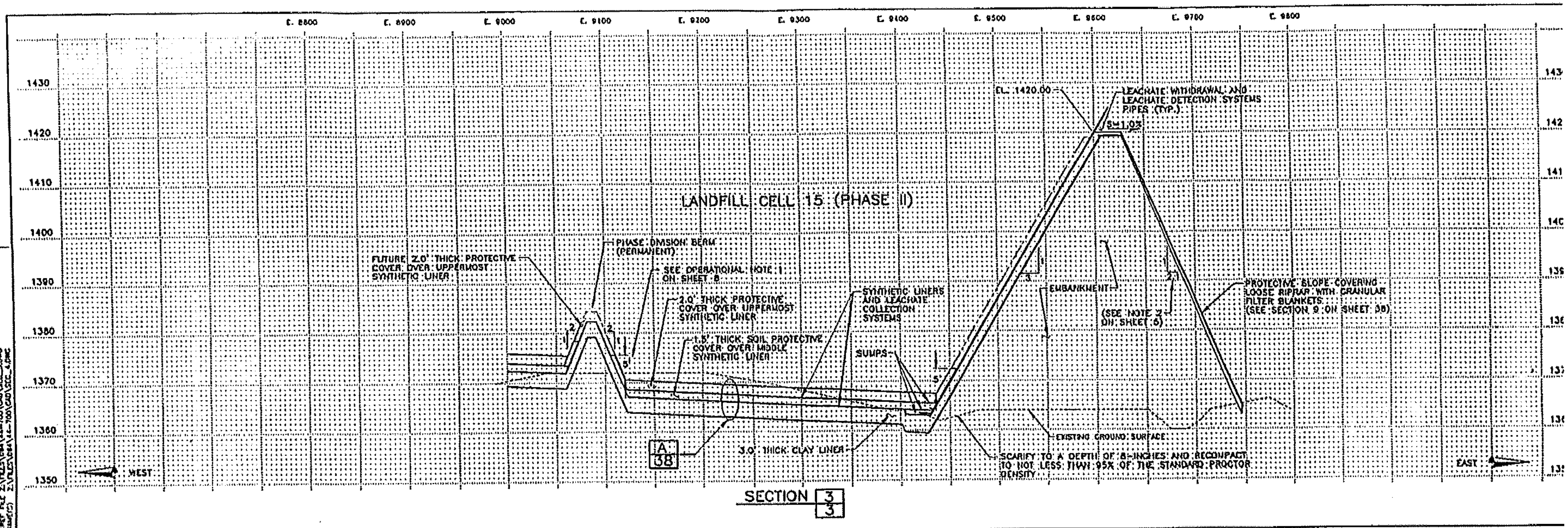
PROJECT ENGINEER

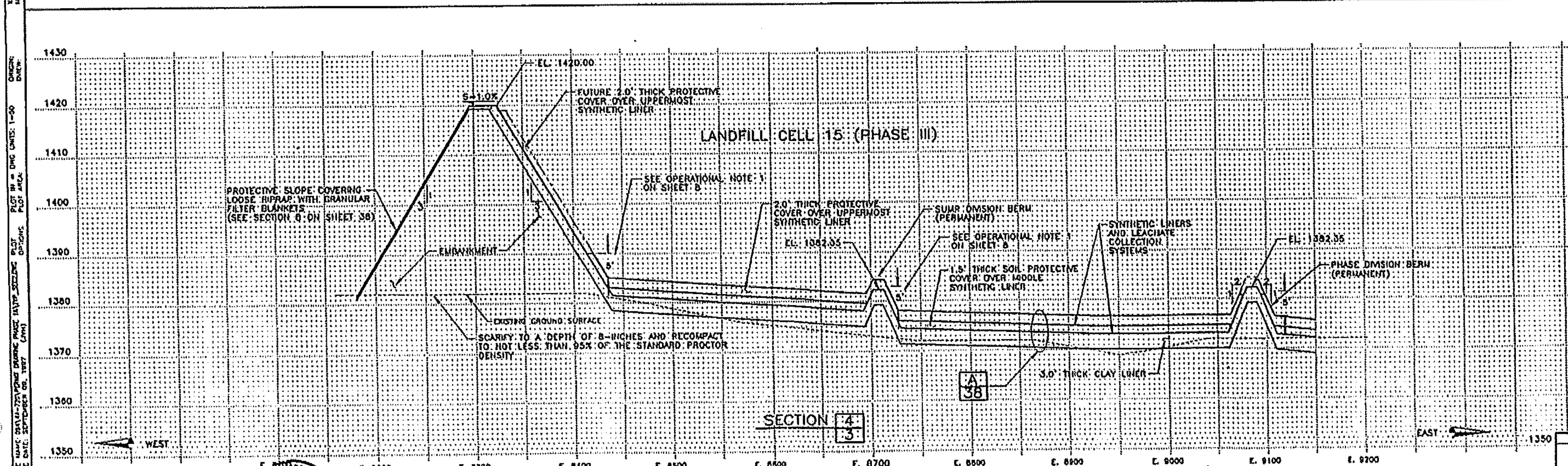
SCALE HORIZONTAL 1" = 50' VERTICAL 1" = 10'

VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
TYPICAL SECTIONS – PHASE I





HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

PROJECT ENGINEER

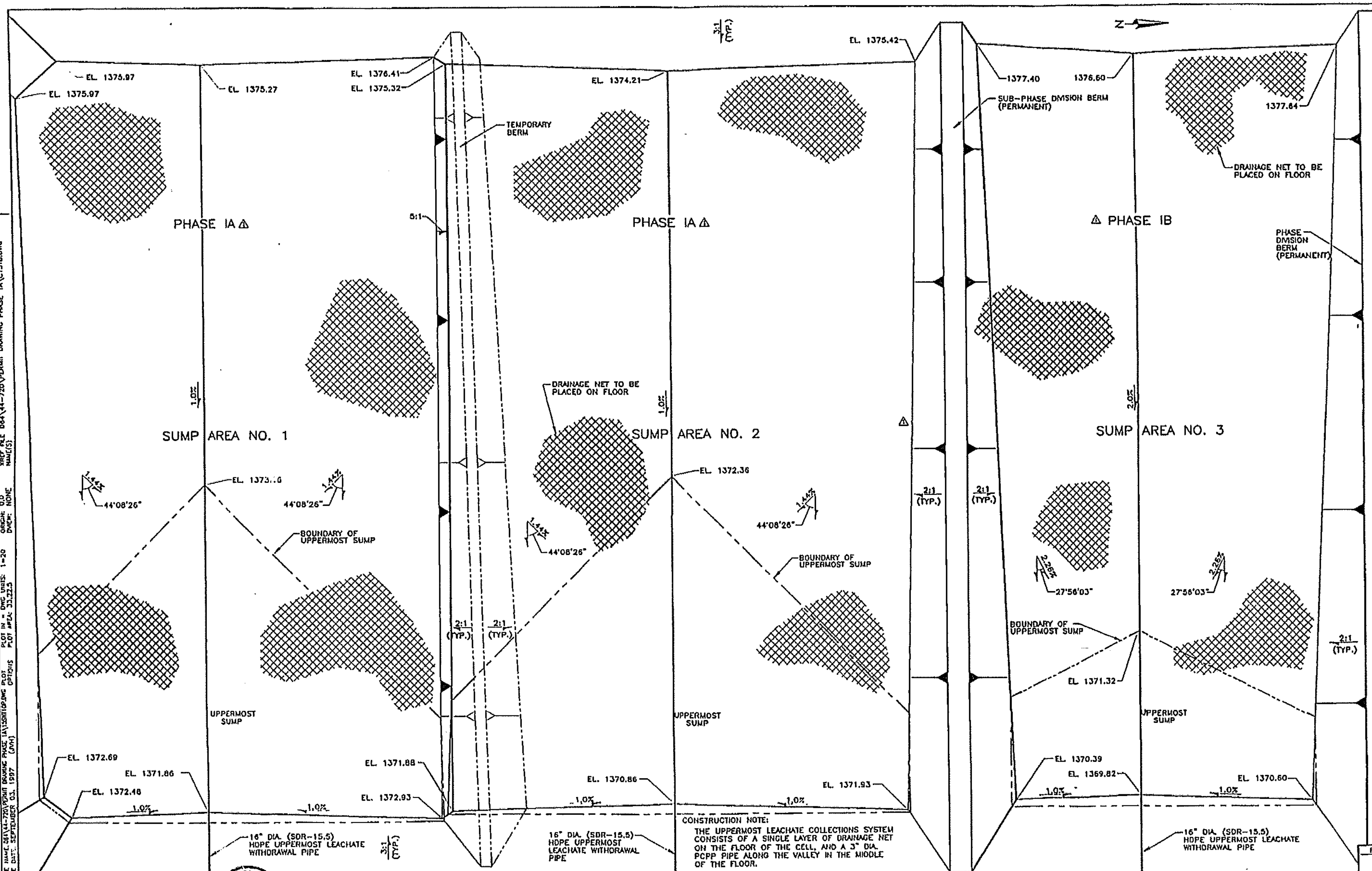
DESIGNED	MPW	3				
DRAFTED	JVH	2				
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH	KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY	APVD.

SCALE HORIZONTAL 1"=50' VERTICAL 1"=10'

VERIFY SCALE 0" 1" BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
TYPICAL SECTIONS – PHASES II & III

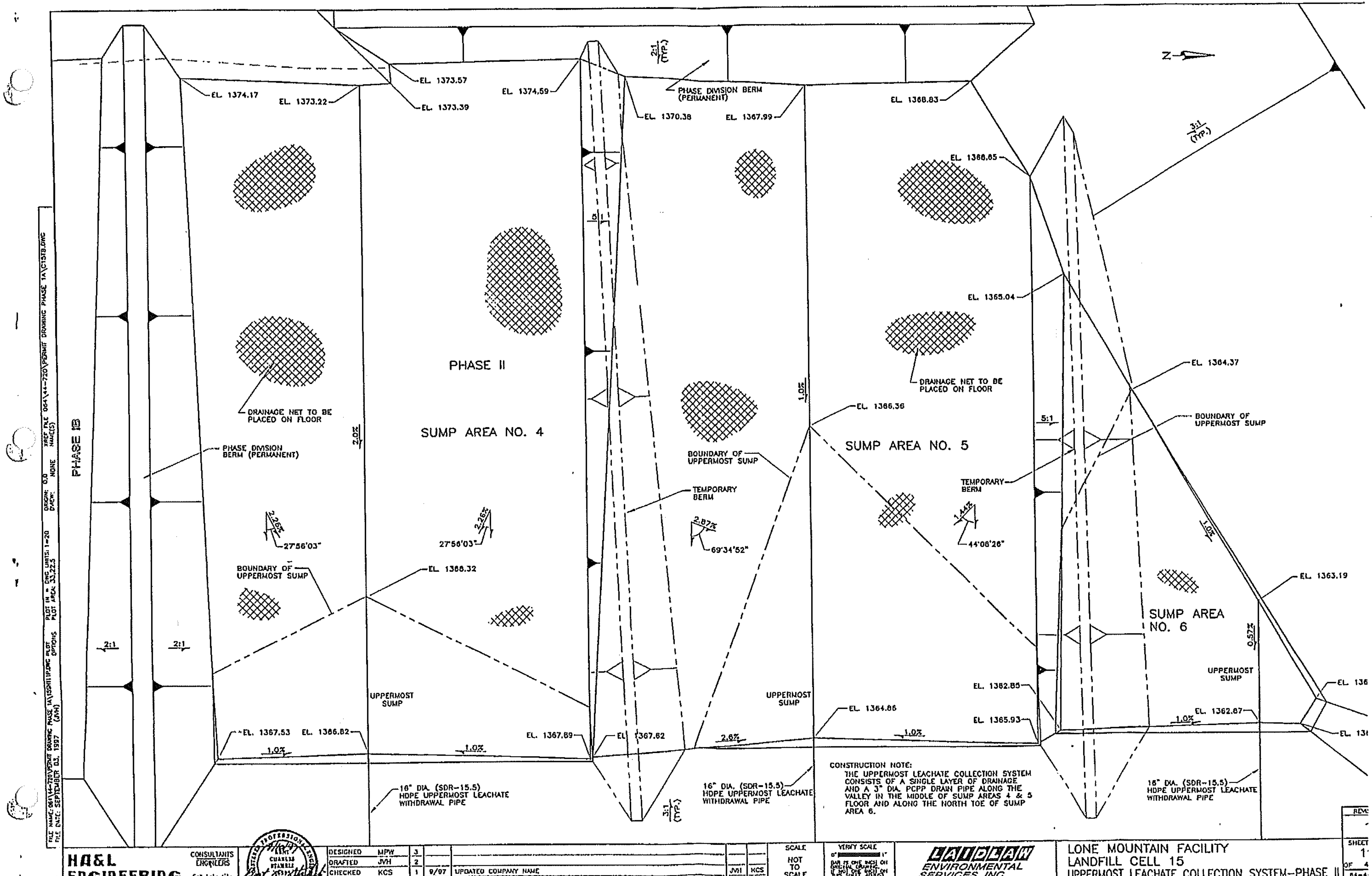


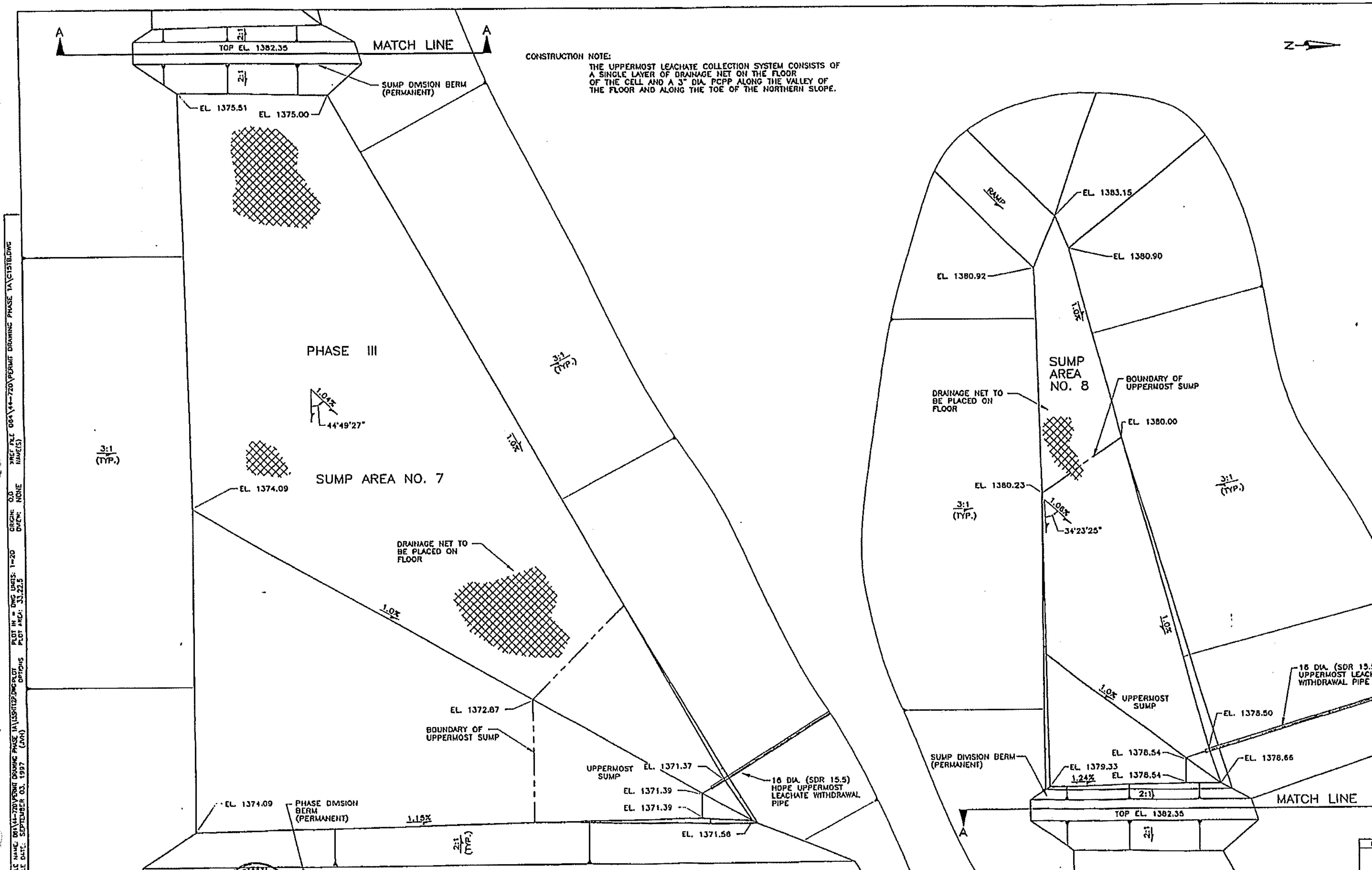
HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah	DESIGNED MPW	3		
	DRAFTED JVH	2		
	CHECKED KCS	1	9/97	UPDATED COMPANY NAME – MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASES 1A & 1B — JVH — KCS
PROJECT ENGINEER	DATE JULY 1990	NO.	DATE	REVISIONS — BY — APVD.

SCALE: NOT TO SCALE

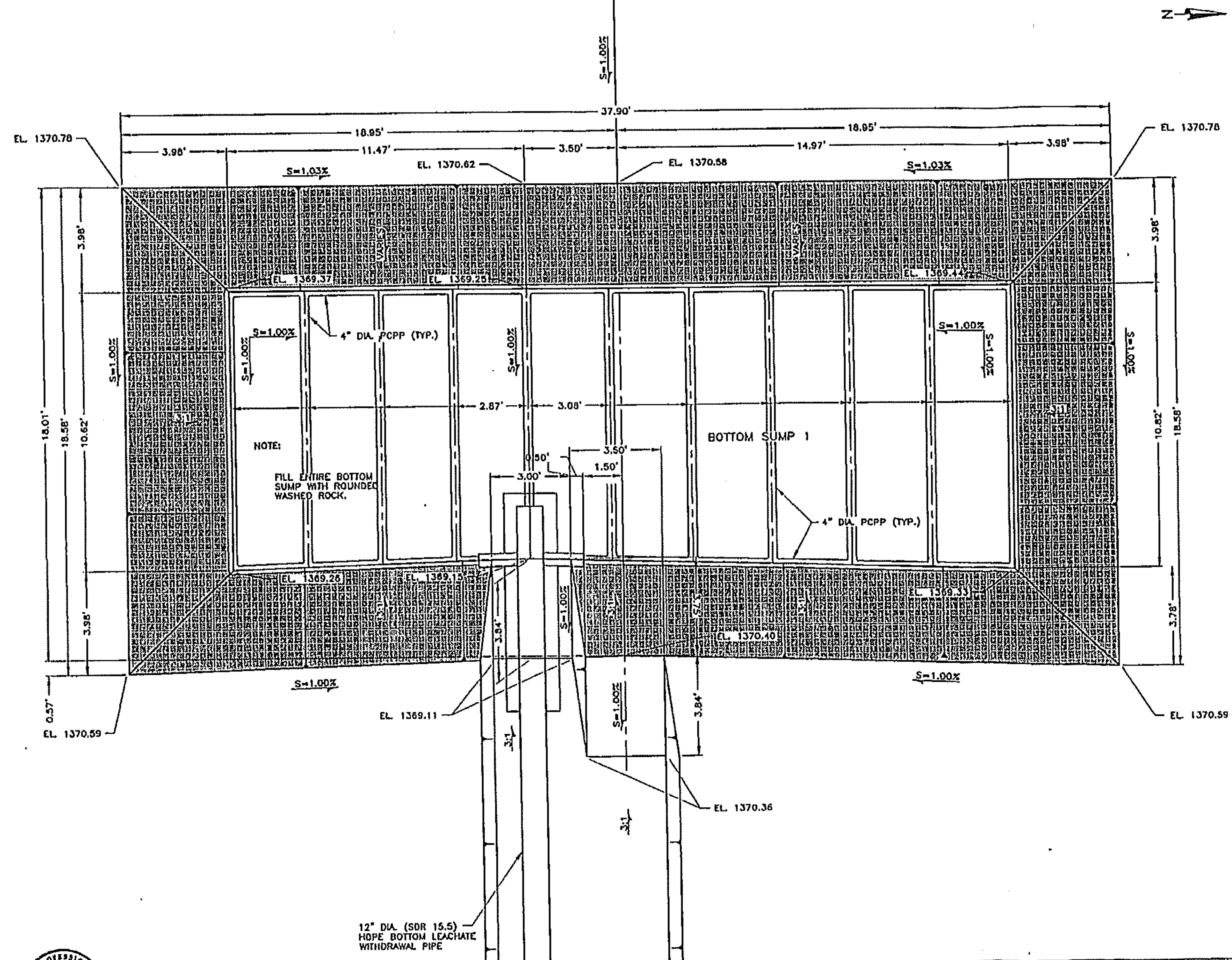
LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST LEACHATE COLLECTION SYSTEM–PHASE I

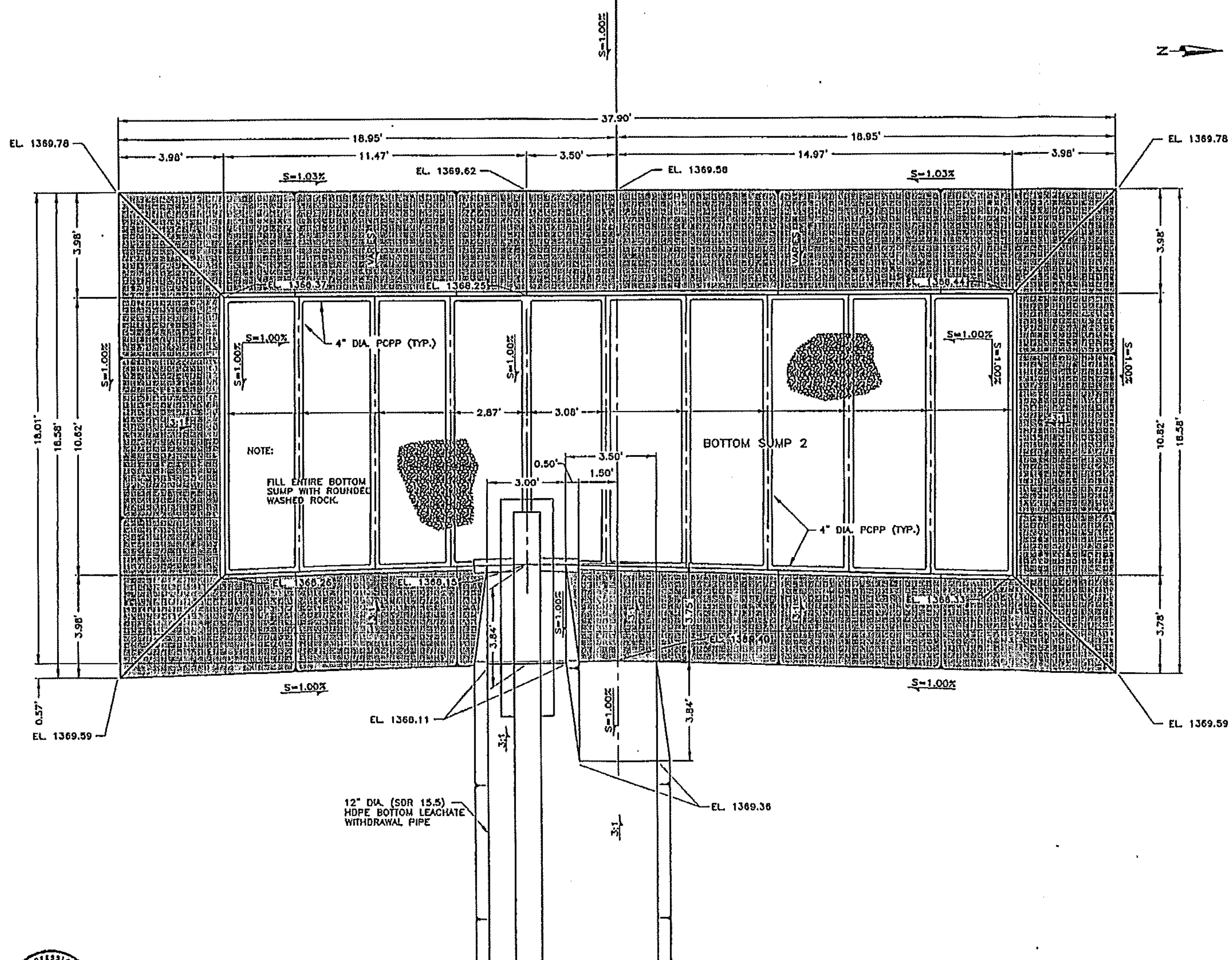

PHASE IB
PHASE II
SUMP AREA NO. 4
SUMP AREA NO. 5
SUMP AREA NO. 6
EL. 1374.17
EL. 1373.22
EL. 1373.57
EL. 1373.39
EL. 1374.59
EL. 1370.38
EL. 1367.99
EL. 1368.83
EL. 1366.65
EL. 1365.04
EL. 1364.37
EL. 1366.36
EL. 1366.32
EL. 1363.19
EL. 1362.85
EL. 1365.93
EL. 1364.86
EL. 1362.87
EL. 1367.53
EL. 1366.82
EL. 1367.69
EL. 1367.62
PHASE DIVISION BERM (PERMANENT)
DRAINAGE NET TO BE PLACED ON FLOOR
BOUNDARY OF UPPERMOST SUMP
TEMPORARY BERM
UPPERMOST SUMP
2:1 (TYP.)
3:1 (TYP.)
5:1
2:1
1.0%
2.0%
2.6%
0.57%
2.26%
2.87%
1.44%
27°56'03"
69°34'52"
44°08'26"
16" DIA. (SDR-15.5) HDPE UPPERMOST LEACHATE WITHDRAWAL PIPE
CONSTRUCTION NOTE:
THE UPPERMOST LEACHATE COLLECTION SYSTEM CONSISTS OF A SINGLE LAYER OF DRAINAGE AND A 3" DIA. PCPP DRAIN PIPE ALONG THE VALLEY IN THE MIDDLE OF SUMP AREAS 4 & 5 FLOOR AND ALONG THE NORTH TOE OF SUMP AREA 6.
HA&L ENGINEERING
CONSULTANTS ENGINEERS
DESIGNED MPW
DRAFTED JVH
CHECKED KCS
9/97 UPDATED COMPANY NAME JVH KCS
REVISIONS
SCALE NOT TO SCALE
VERIFY SCALE
LAIDLAW ENVIRONMENTAL SERVICES, INC.
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST LEACHATE COLLECTION SYSTEM-PHASE II



HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah	DESIGNED MPW; DRAFTED JVH; CHECKED KCS; DATE JULY 1996	1 — 9/97 — UPDATED COMPANY NAME — JVH — KCS; REVISIONS	SCALE: NOT TO SCALE
LAIDLAW ENVIRONMENTAL SERVICES, INC.	LONE MOUNTAIN FACILITY	LANDFILL CELL 15	UPPERMOST LEACHATE COLLECTION SYSTEM – PHASE III



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	DESIGNED MPW / DRAFTED RGA / CHECKED KCS / DATE JULY 1990	1 9/07 UPDATED COMPANY NAME — REVISIONS
SCALE 1"=2'	VERIFY SCALE	LAIDLAW ENVIRONMENTAL SERVICES, INC.	LONE MOUNTAIN FACILITY LANDFILL CELL 15 BOTTOM SUMP NO. 1 – PHASE IA



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City	DESIGNED MPW	3			
		DRAFTED RGA	2			
		CHECKED KCS	1	9/97	UPDATED COMPANY NAME	JVH KCS

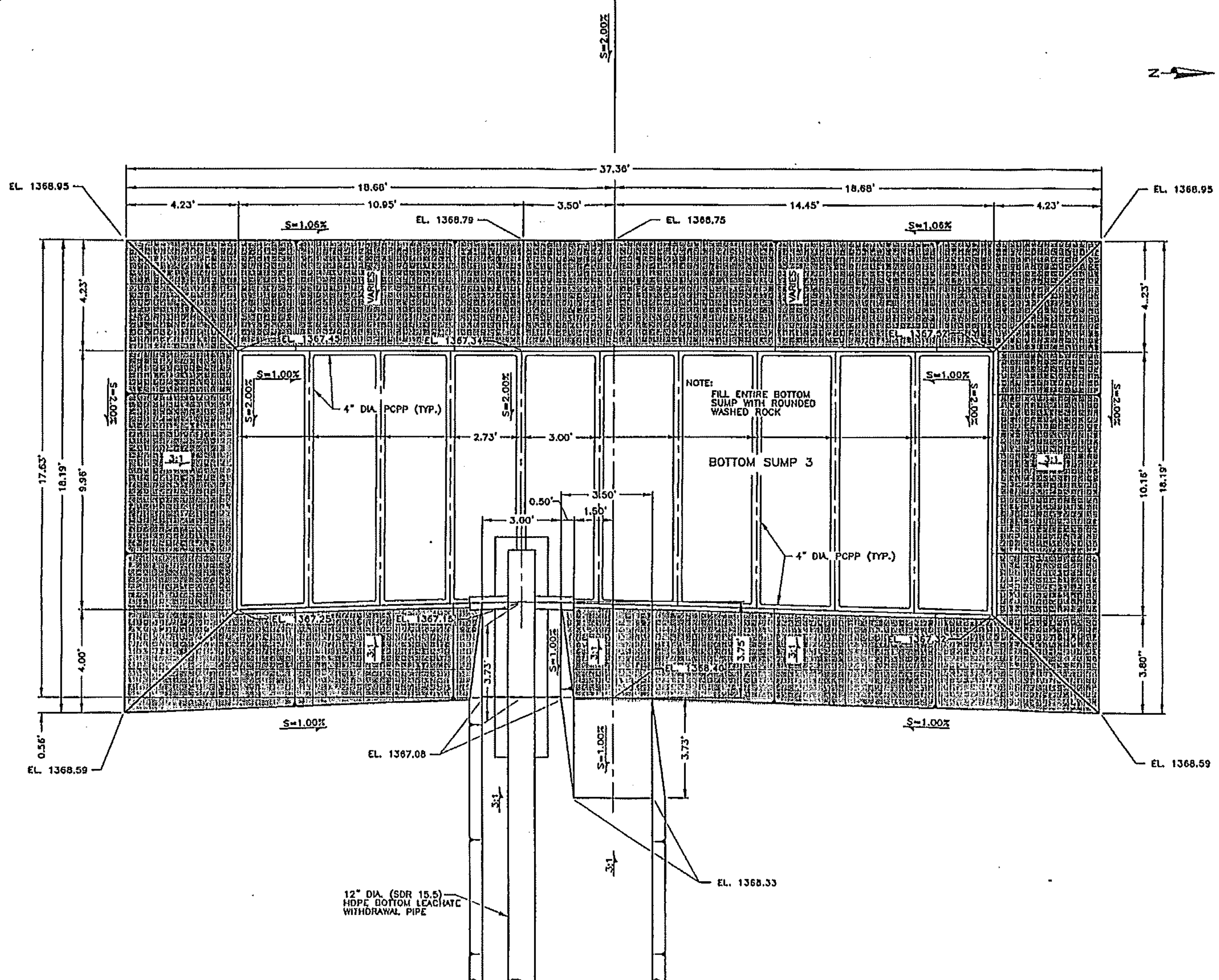
SCALE 1"=2'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
BOTTOM SUMP NO. 2 — PHASE IA

SHEET 14 OF 44

REVISION 1



HA&L ENGINEERING	CONSULTANTS ENGINEERS, Salt Lake City Utah	PROJECT ENGINEER	DESIGNED MPW	3		
			DRAFTED RGA	2		
			CHECKED KCS	1	9/97	UPDATED COMPANY NAME — JVH — KCS
			DATE JULY 1998	NO.	DATE	REVISIONS — BY — APVD.

SCALE 1"=2'

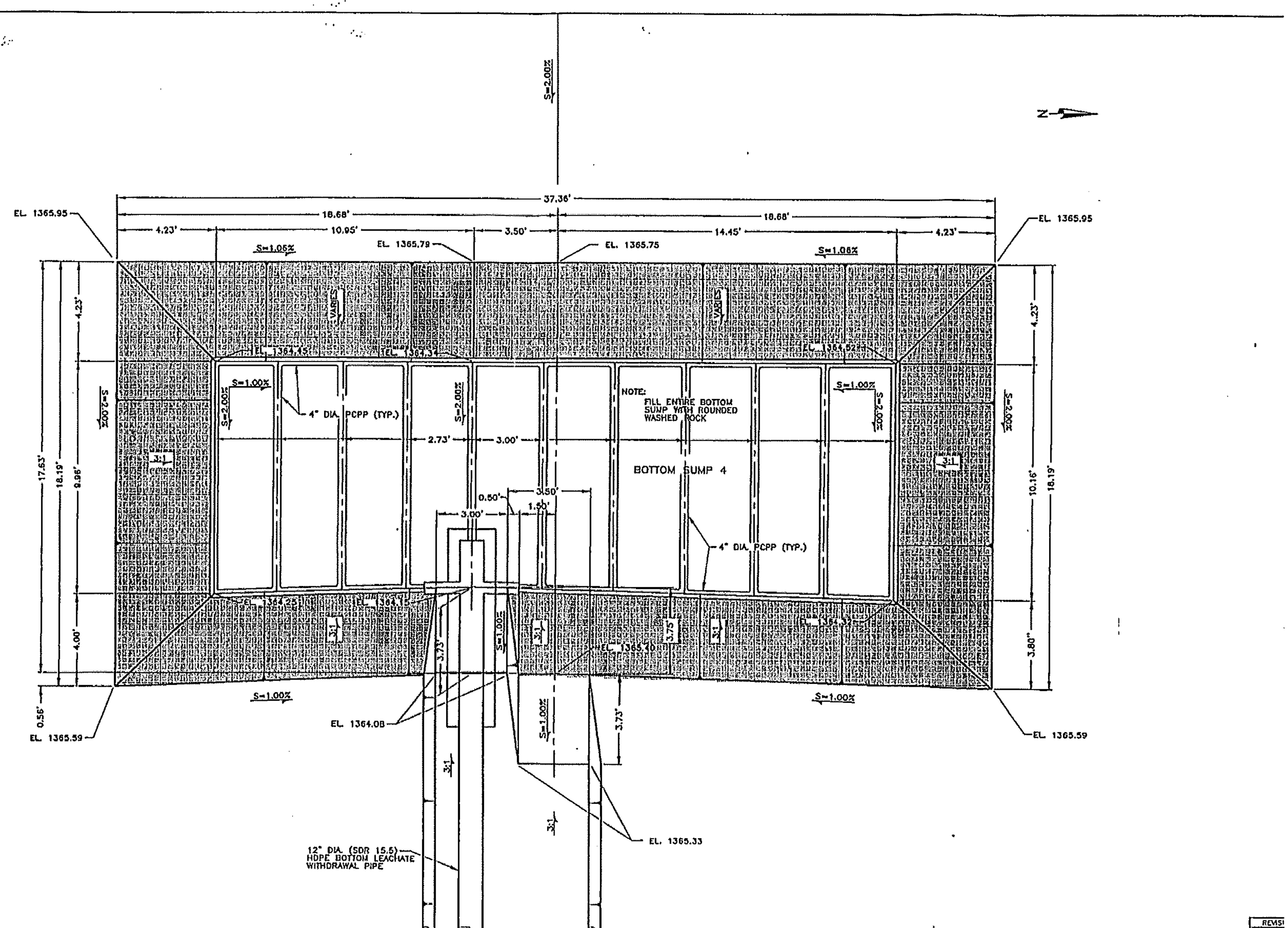
VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
BOTTOM SUMP NO. 3 — PHASE IB

REVISION 1

SHEET 15 OF 44



HA&L ENGINEERING CONSULTANTS ENGINEERS

DESIGNED MPW | DRAFTED RGA | CHECKED KCS

SCALE 1"=2'

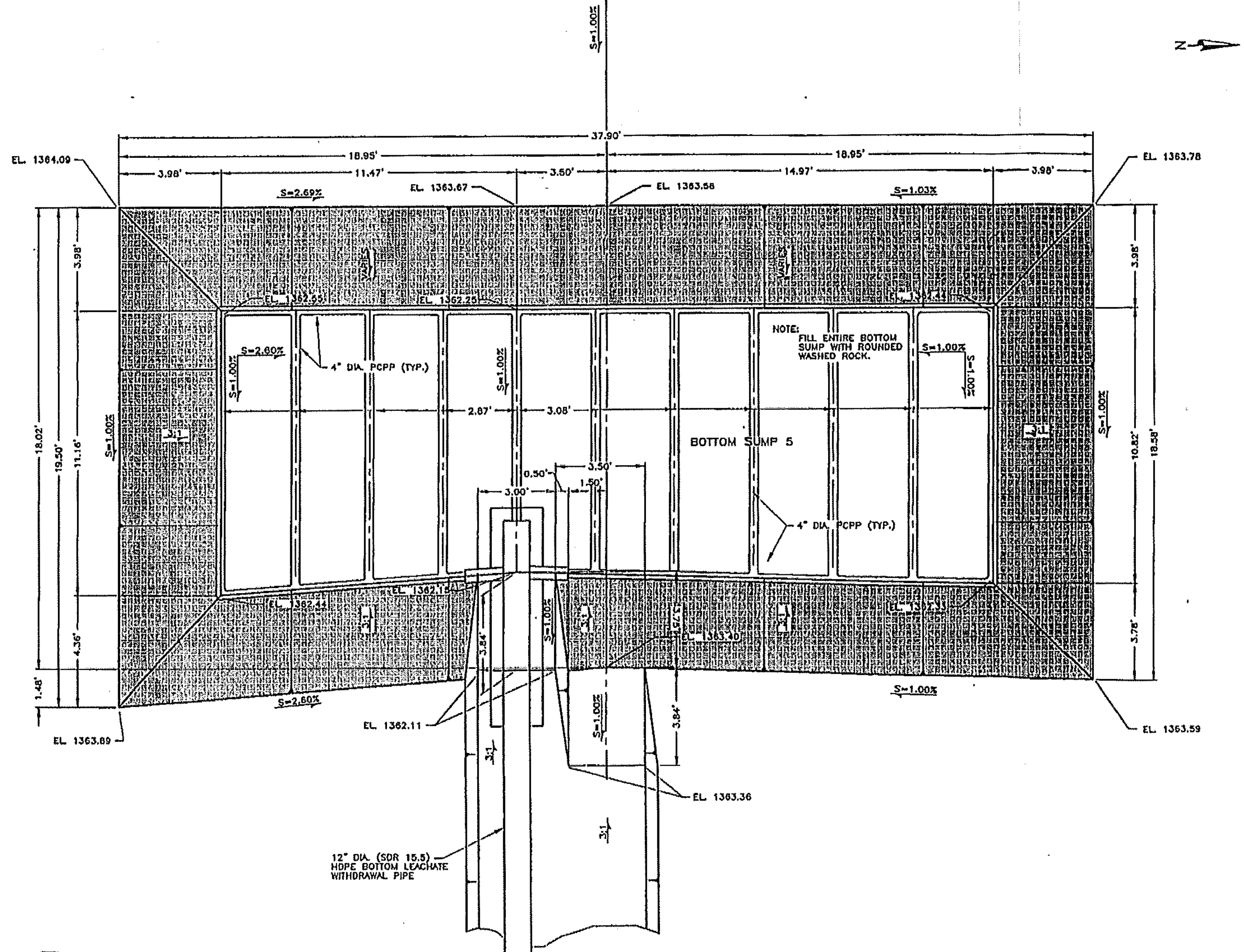
VERIFY SCALE

LAIDLAW ENVIRONMENTAL

LONE MOUNTAIN FACILITY
LANDFILL CELL 15

REVISION 1

SHEET 16

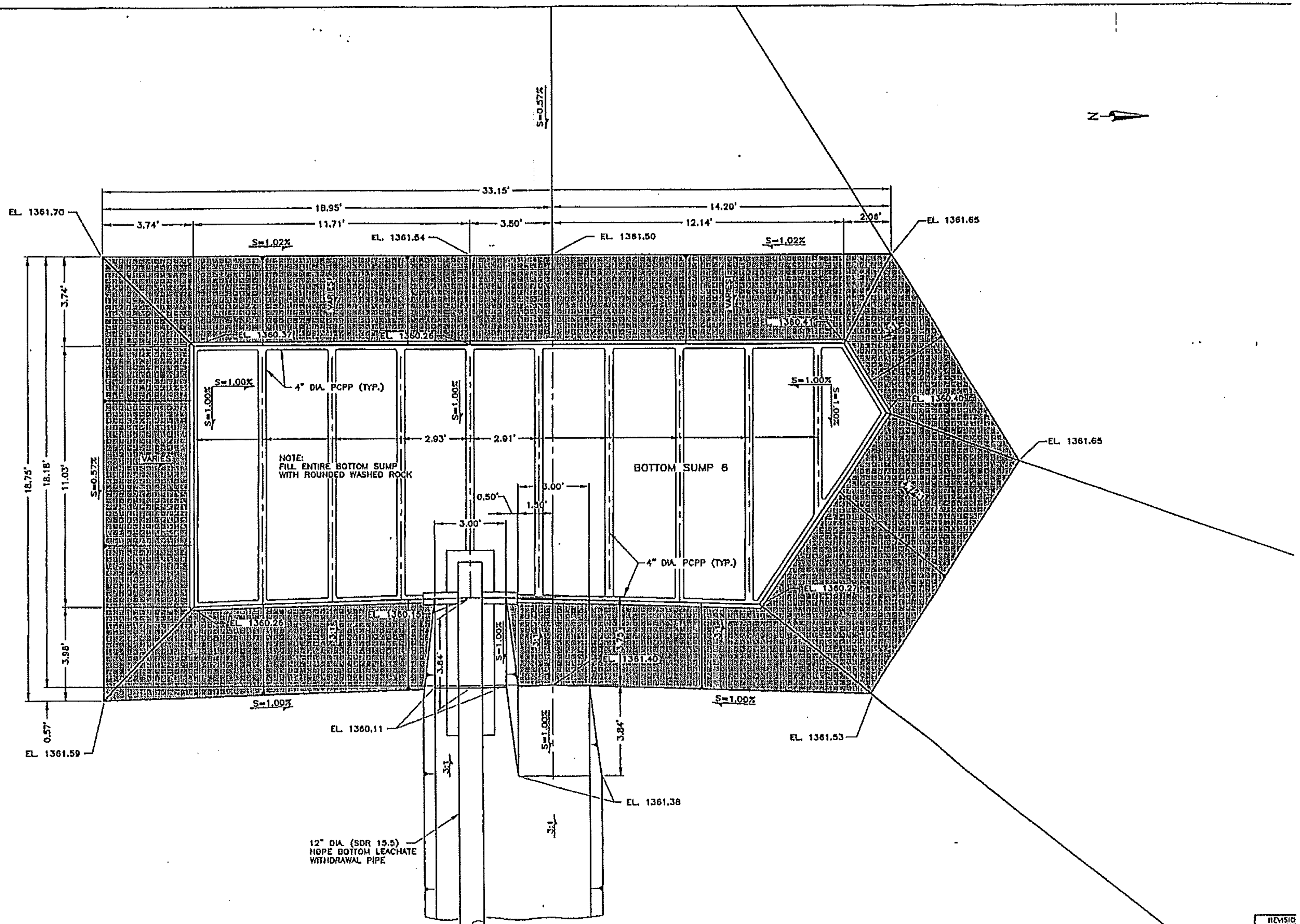


REVISION 1

CONSULTANTS | DESIGNED MPW | SCALE | VERIFY SCALE

LONE MOUNTAIN FACILITY

SHEET NO. 17



DESIGNED	MPW	3			
DRAFTED	RGA	2			
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH KCS

HA&L ENGINEERING — CONSULTANTS ENGINEERS — Salt Lake City

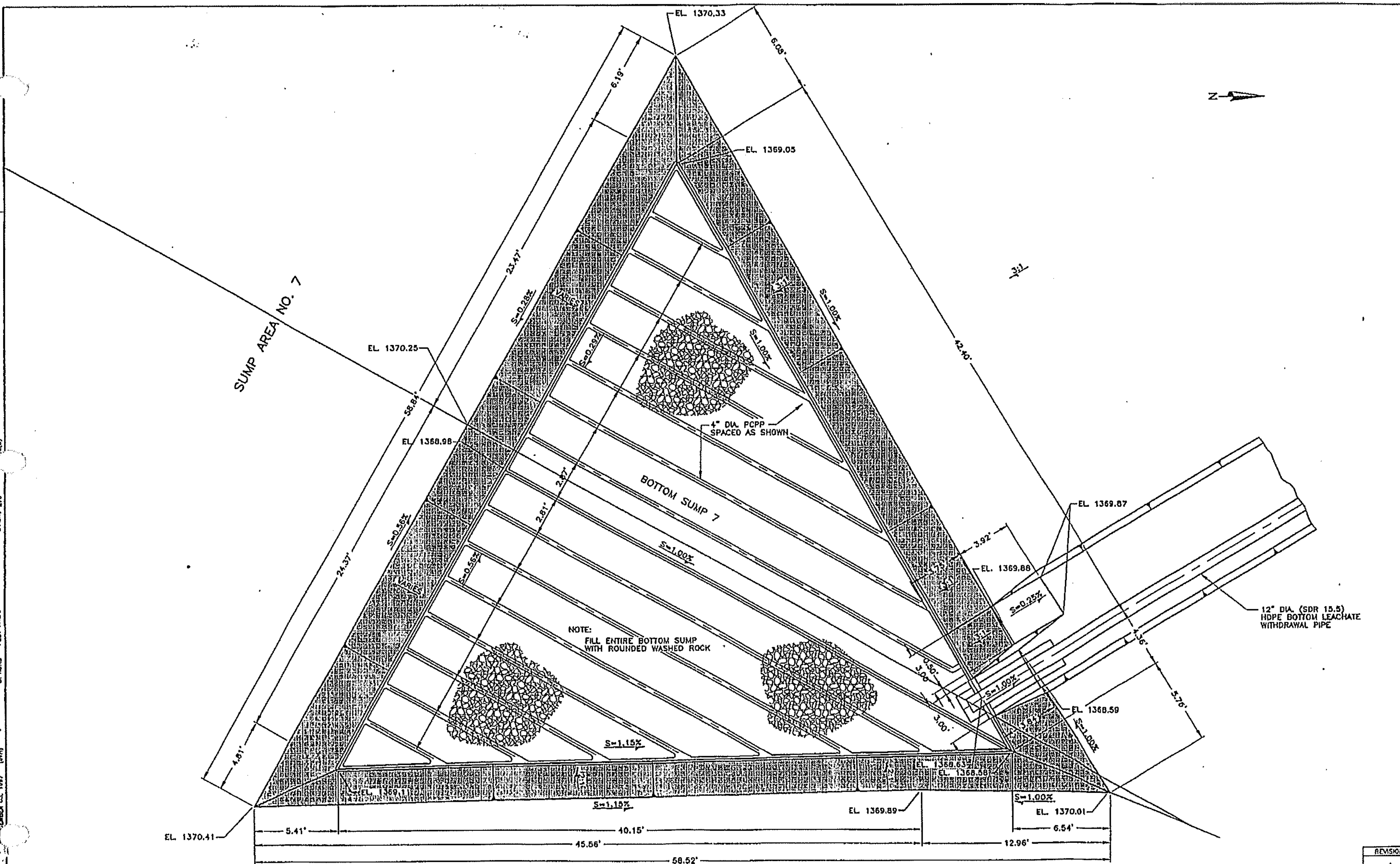
SCALE 1"=2'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
BOTTOM SUMP NO. 6 — PHASE II

REVISION 1

SHEET 18 OF 44



HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

DESIGNED	MPW	3				
DRAFTED	RGA	2				
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH	KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY	APVD.

PROJECT ENGINEER

SCALE 1"=3'

VERIFY SCALE — BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

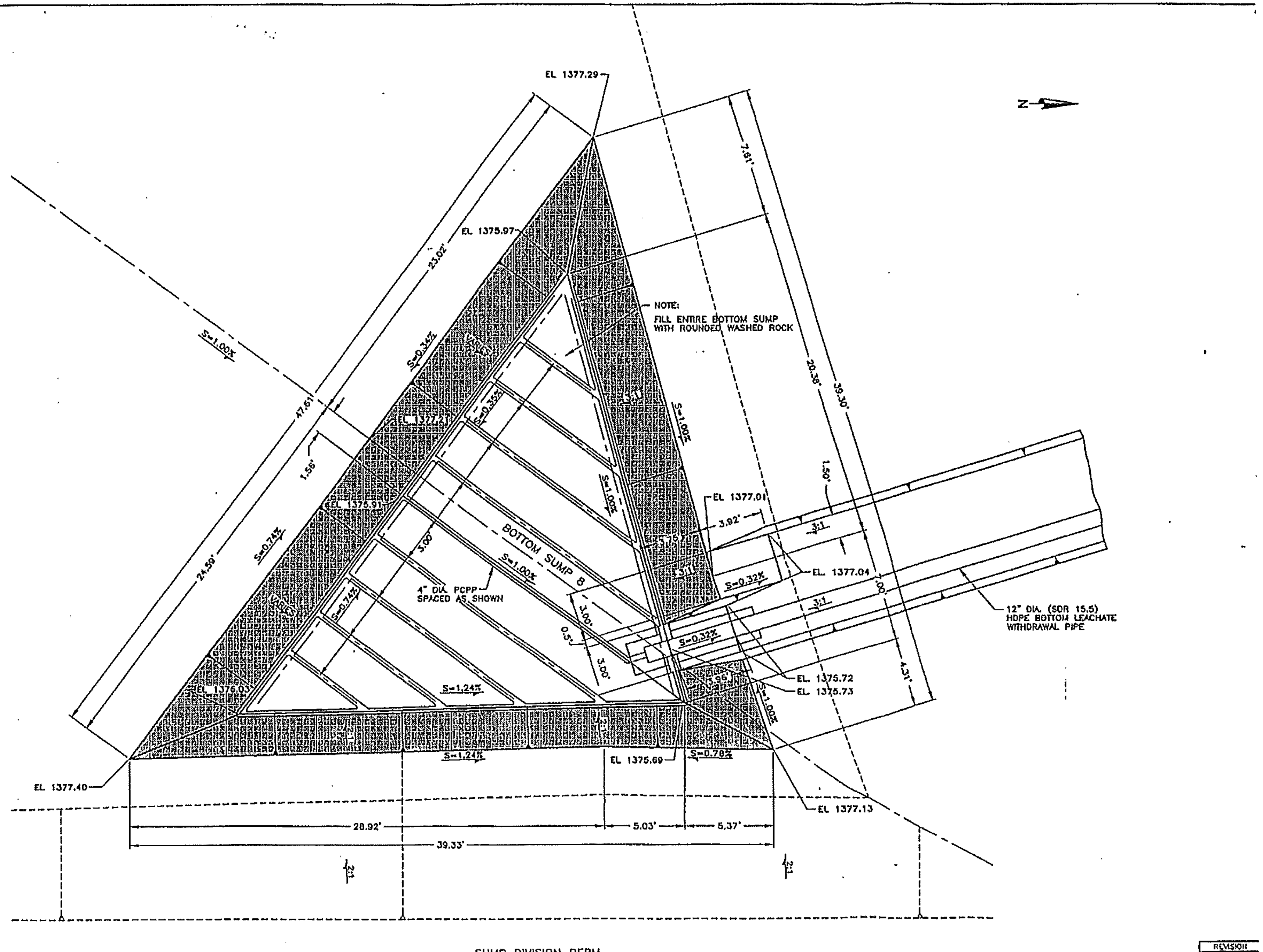
LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
BOTTOM SUMP NO. 7 — PHASE III

REVISION 1

SHEET NO. 19 OF 44

64-44-700



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah
DESIGNED	MPW
DRAFTED	RGA
CHECKED	KCS
DATE	JULY 1996

NO.	DATE	REVISIONS	BY	APPD.
3				
2				
1	9/97	UPDATED COMPANY NAME	JMI	KCS

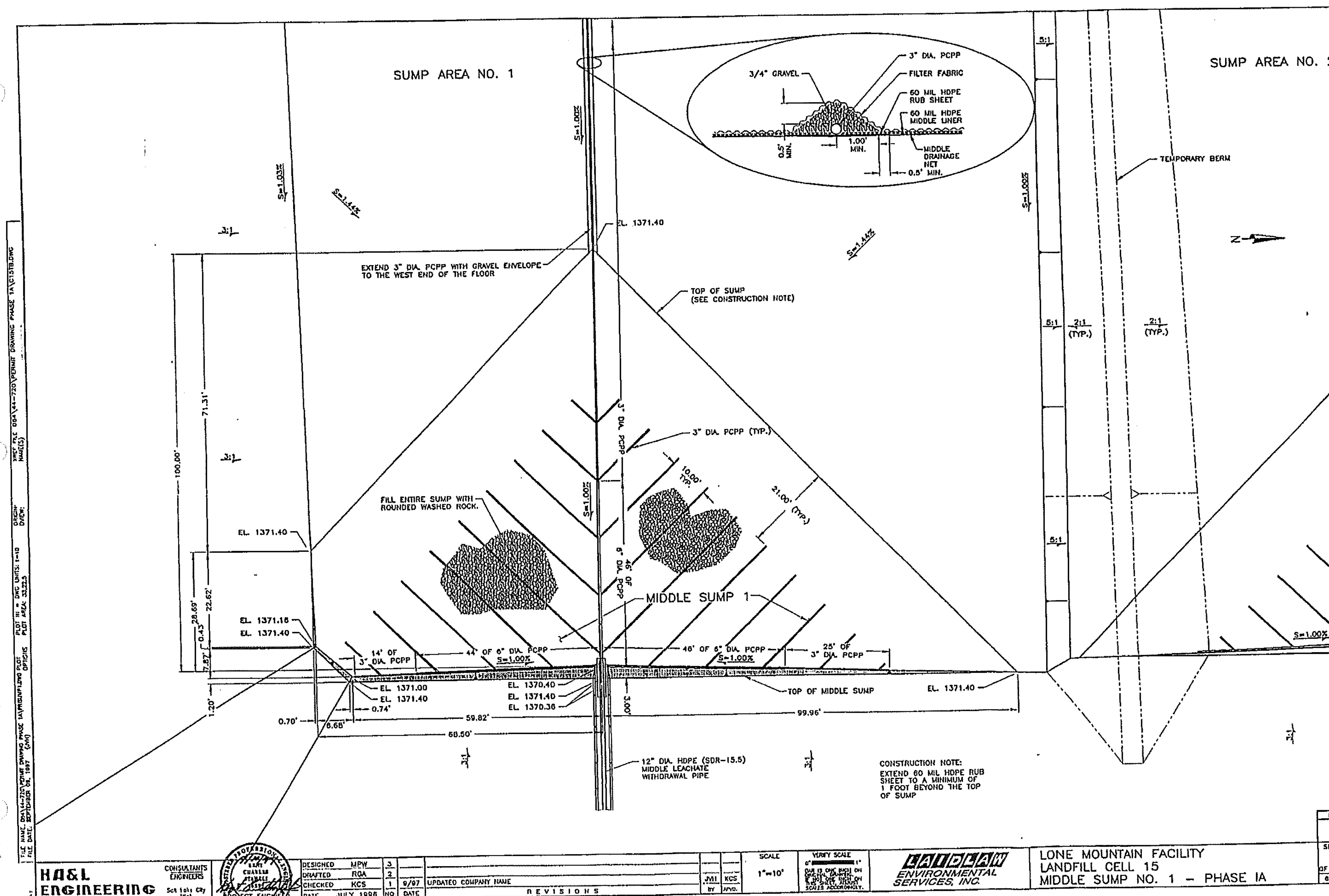
SCALE 1"=3'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

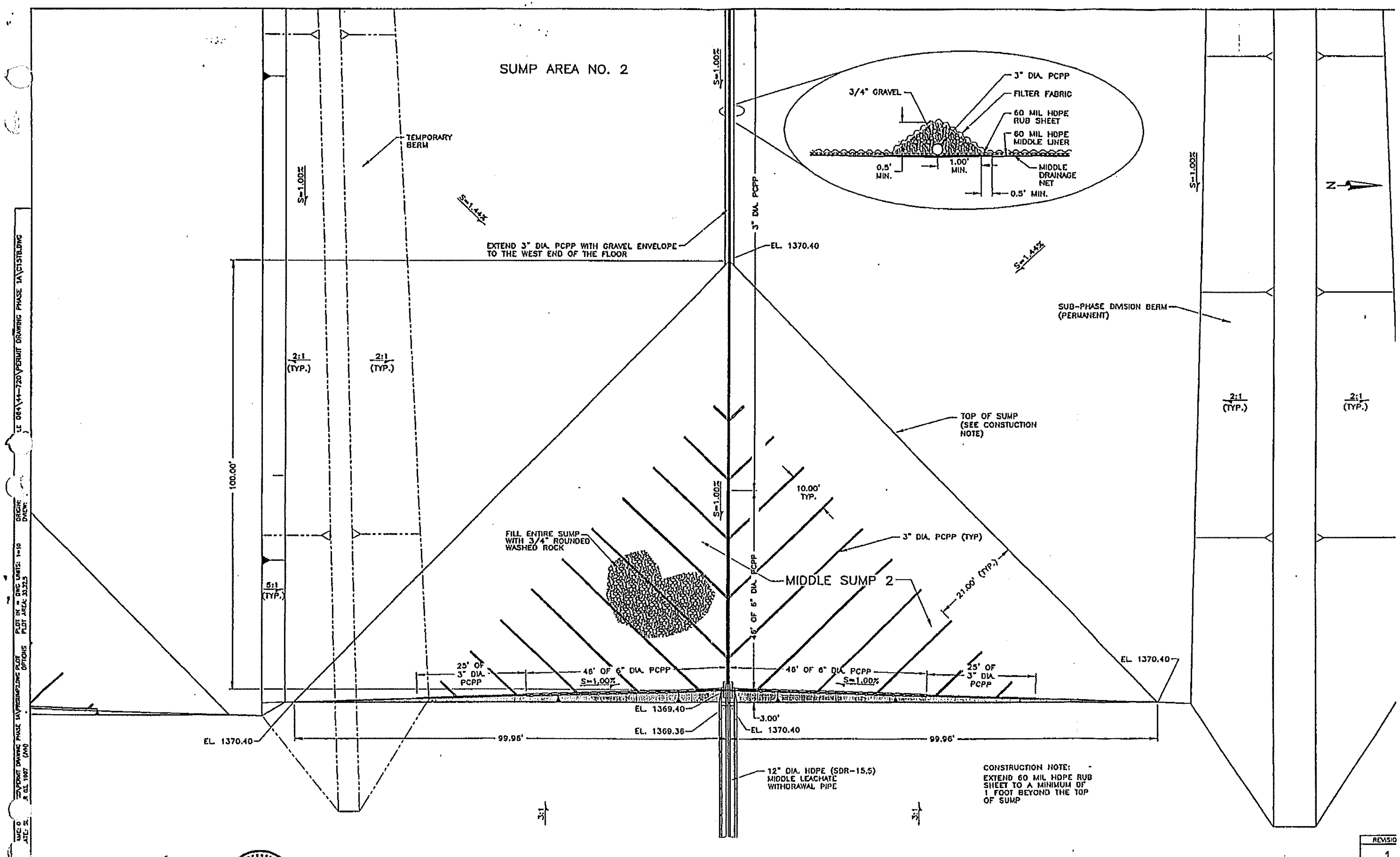
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
BOTTOM SUMP NO. 8 – PHASE III

REVISION 1

SHEET NO. 20 OF 44



SUMP AREA NO. 1
SUMP AREA NO. 2
3/4" GRAVEL
3" DIA. PCPP
FILTER FABRIC
60 MIL HDPE RUB SHEET
60 MIL HDPE MIDDLE LINER
1.00' MIN.
0.5' MIN.
MIDDLE DRAINAGE NET
0.5' MIN.
TEMPORARY BERM
EL. 1371.40
EXTEND 3" DIA. PCPP WITH GRAVEL ENVELOPE TO THE WEST END OF THE FLOOR
TOP OF SUMP (SEE CONSTRUCTION NOTE)
3" DIA. PCPP (TYP.)
10.00' TYP.
21.00' (TYP.)
FILL ENTIRE SUMP WITH ROUNDED WASHED ROCK.
MIDDLE SUMP 1
46' OF 6" DIA. PCPP
14' OF 3" DIA. PCPP
44' OF 6" DIA. PCPP
25' OF 3" DIA. PCPP
S=1.00%
S=1.03%
S=1.44%
EL. 1371.16
EL. 1371.40
EL. 1371.00
EL. 1370.40
EL. 1370.36
TOP OF MIDDLE SUMP
100.00'
71.31'
28.69'
22.62'
7.87'
0.43'
1.20'
0.70'
0.74'
6.68'
59.82'
68.50'
99.96'
3.00'
12" DIA. HDPE (SDR-15.5) MIDDLE LEACHATE WITHDRAWAL PIPE
CONSTRUCTION NOTE:
EXTEND 60 MIL HDPE RUB SHEET TO A MINIMUM OF 1 FOOT BEYOND THE TOP OF SUMP
3:1
5:1
2:1 (TYP.)
N
HA&L ENGINEERING
CONSULTANTS ENGINEERS
DESIGNED MPW
DRAFTED RGA
CHECKED KCS
DATE JULY 1996
PROJECT ENGINEER
1 9/97 UPDATED COMPANY NAME
NO DATE REVISIONS BY APVD.
SCALE 1"=10'
VERIFY SCALE
LAIDLAW ENVIRONMENTAL SERVICES, INC.
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 1 – PHASE IA



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City	DESIGNED MPW	3			
		DRAFTED RGA	2			
		CHECKED KCS	1	9/97	UPDATED COMPANY NAME – MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASES IA & IB	JVI KCS

SCALE 1"=10'

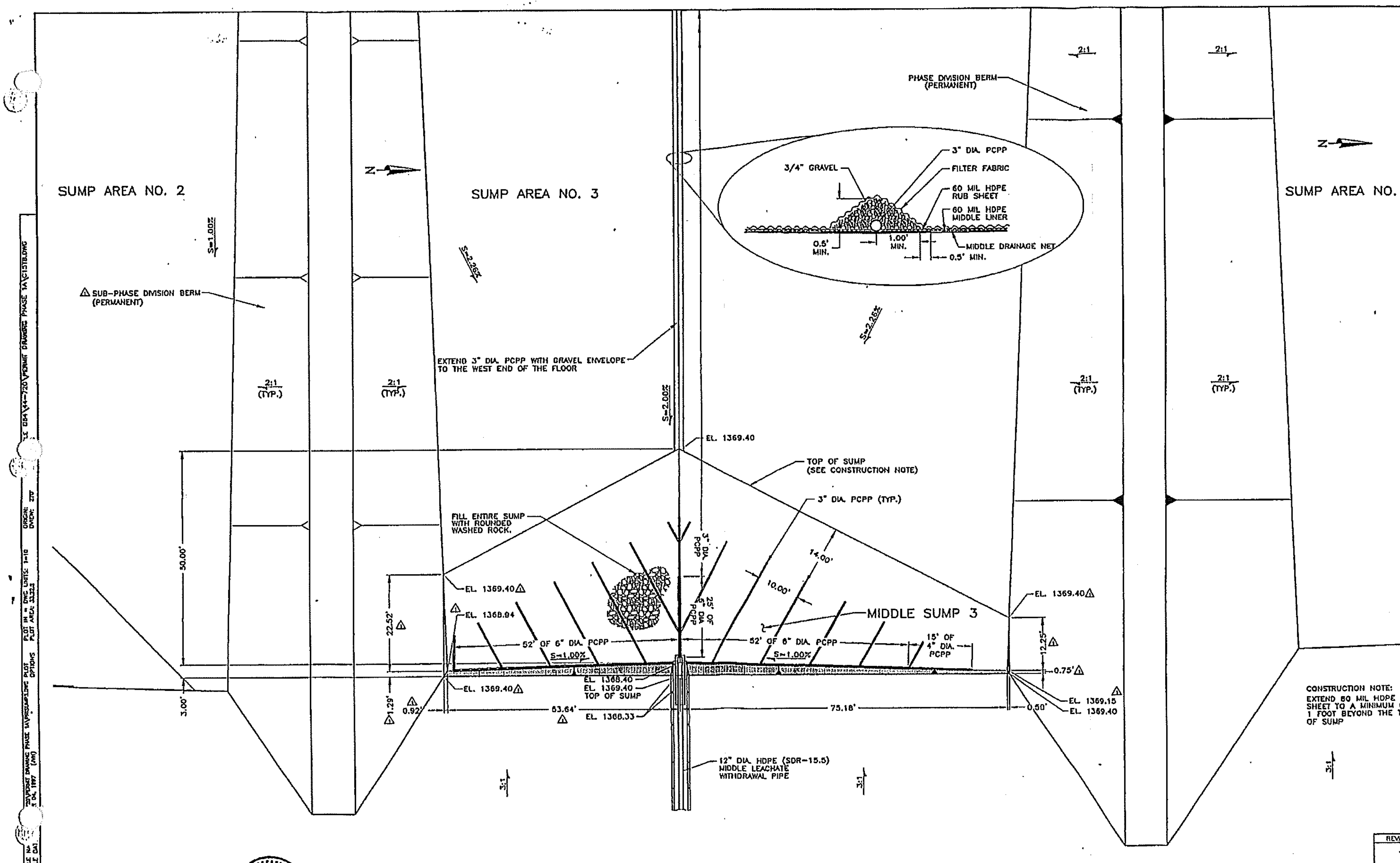
VERIFY SCALE — BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON ...

LAIDLAW ENVIRONMENTAL SERVICES INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 2 — PHASE IA

REVISION 1

SHEET 22 OF 44



HA&L ENGINEERING — CONSULTANTS ENGINEERS — Salt Lake City, Utah

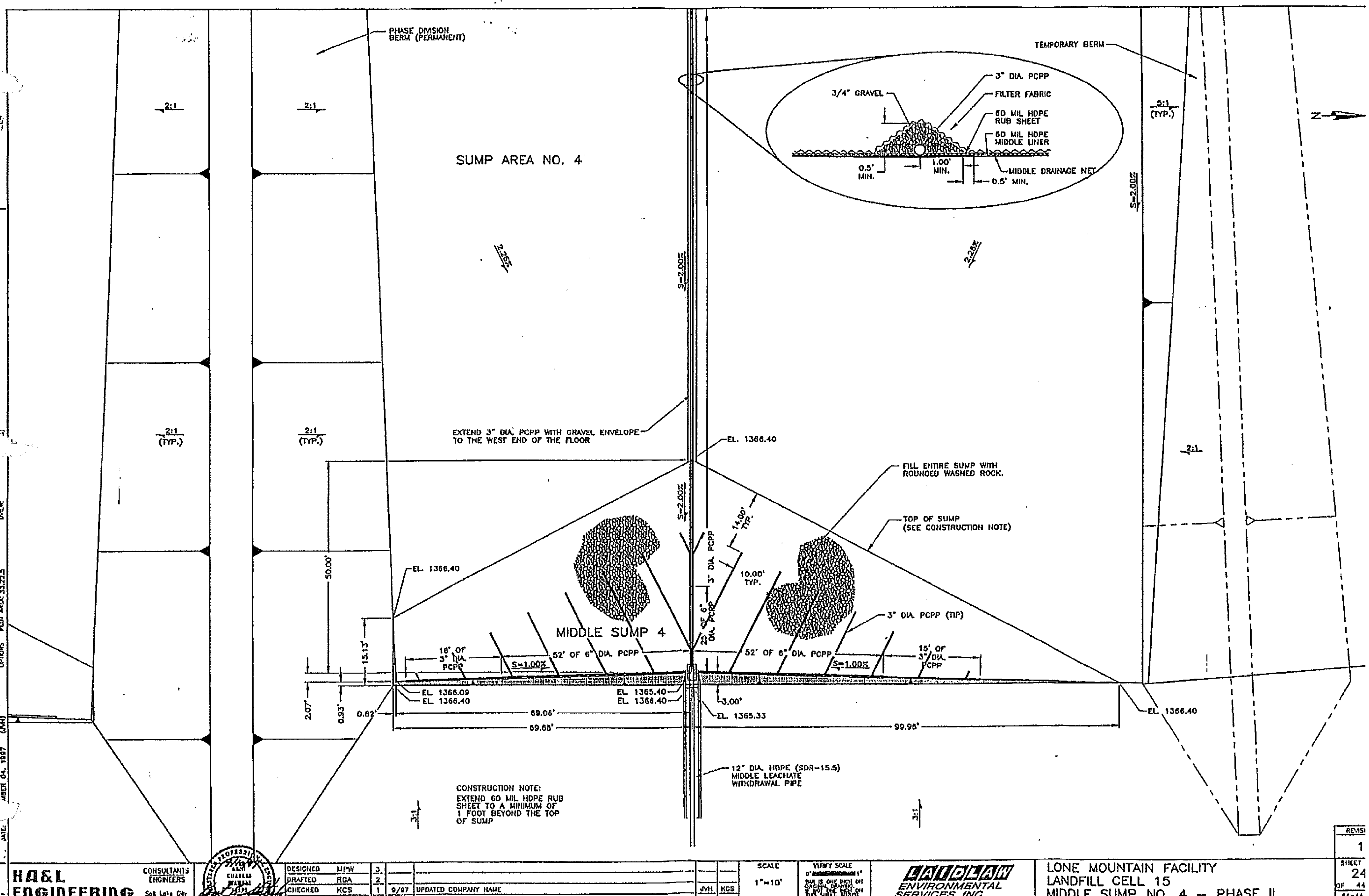
DESIGNED	MPW	3	
DRAFTED	SDM	2	
CHECKED	KCS	1	9/97 UPDATED COMPANY NAME - MODIFICATIONS TO DIVIDE PHASE 1 INTO SUB-PHASES IA & IB — JWH — KCS
DATE	JULY 1996	NO.	DATE — REVISIONS — BY — APPD.

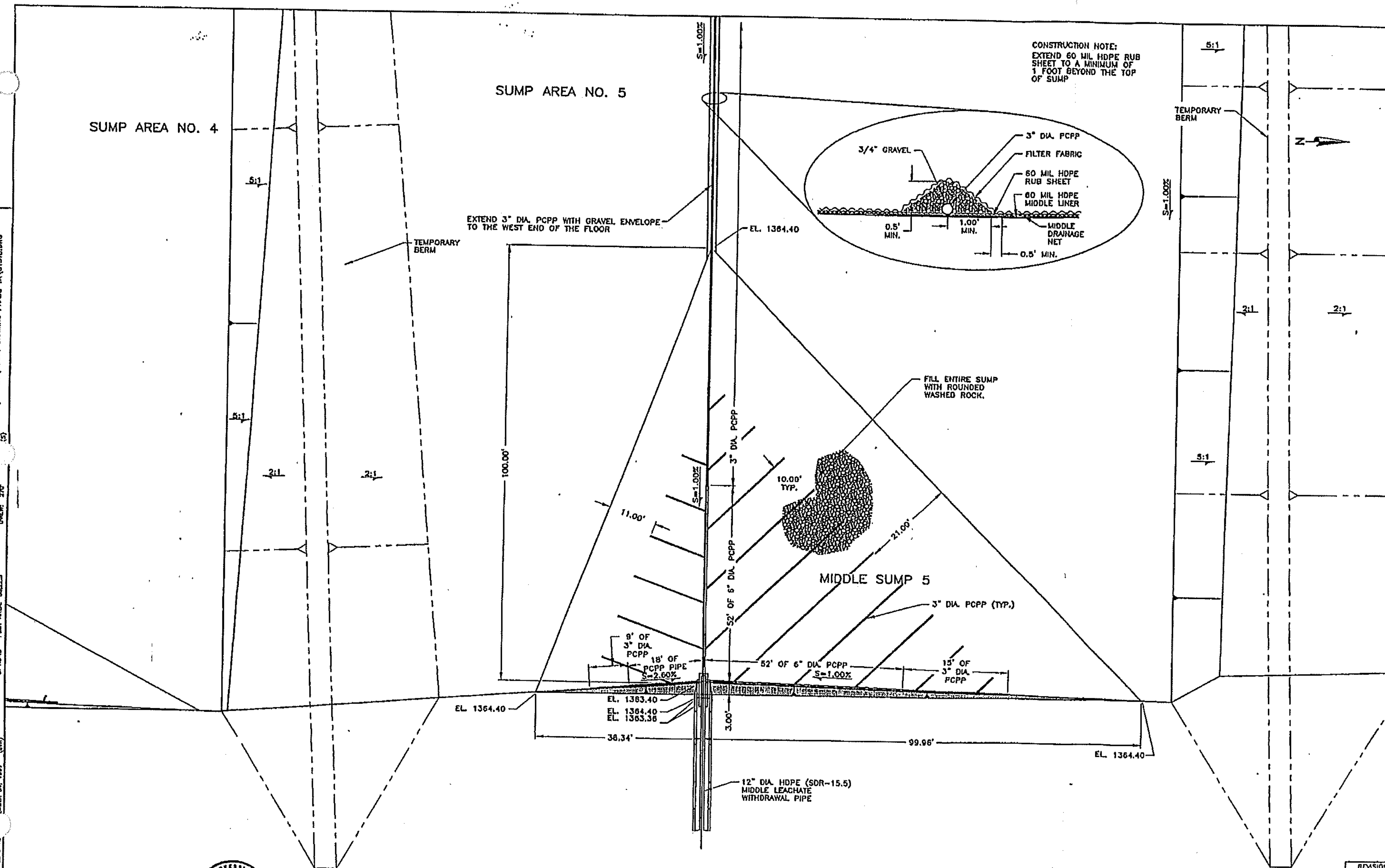
SCALE 1"=10'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 3 - PHASE IB

REVISION 1
SHEET 23 OF 44


PHASE DIVISION BERM (PERMANENT)
SUMP AREA NO. 4
TEMPORARY BERM
3" DIA. PCPP
3/4" GRAVEL
FILTER FABRIC
60 MIL HDPE RUB SHEET
60 MIL HDPE MIDDLE LINER
MIDDLE DRAINAGE NET
0.5' MIN.
1.00' MIN.
0.5' MIN.
2:1
2:1 (TYP.)
5:1 (TYP.)
S=2.00%
2.26%
2.28%
EXTEND 3" DIA. PCPP WITH GRAVEL ENVELOPE TO THE WEST END OF THE FLOOR
EL. 1366.40
FILL ENTIRE SUMP WITH ROUNDED WASHED ROCK.
TOP OF SUMP (SEE CONSTRUCTION NOTE)
14.00' TYP.
10.00' TYP.
3" DIA. PCPP
3" DIA. PCPP (TIP)
MIDDLE SUMP 4
18' OF 3" DIA. PCPP
52' OF 6" DIA. PCPP
S=1.00%
25' OF 6" DIA. PCPP
15' OF 3" DIA. PCPP
50.00'
15.13'
2.07'
0.93'
0.62'
EL. 1366.09
EL. 1366.40
EL. 1365.40
EL. 1366.40
3.00'
EL. 1365.33
69.06'
69.68'
99.98'
12" DIA. HDPE (SDR-15.5) MIDDLE LEACHATE WITHDRAWAL PIPE
CONSTRUCTION NOTE:
EXTEND 60 MIL HDPE RUB SHEET TO A MINIMUM OF 1 FOOT BEYOND THE TOP OF SUMP
3:1
N
HA&L ENGINEERING
CONSULTANTS ENGINEERS Salt Lake City Utah
DESIGNED MPW
DRAFTED RGA
CHECKED KCS
DATE JULY 1996
1 9/97 UPDATED COMPANY NAME JVH KCS
NO. DATE REVISIONS BY APVD.
SCALE 1"=10'
LAIDLAW ENVIRONMENTAL SERVICES, INC.
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 4 – PHASE II
REVISION 1
SHEET 24 OF 44



HA&L ENGINEERING
CONSULTANTS ENGINEERS
Salt Lake City Utah

DESIGNED	MPW	3		
DRAFTED	RGA	2		
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME
DATE	JULY 1990	NO.	DATE	REVISIONS

PROJECT ENGINEER

SCALE 1"=10'

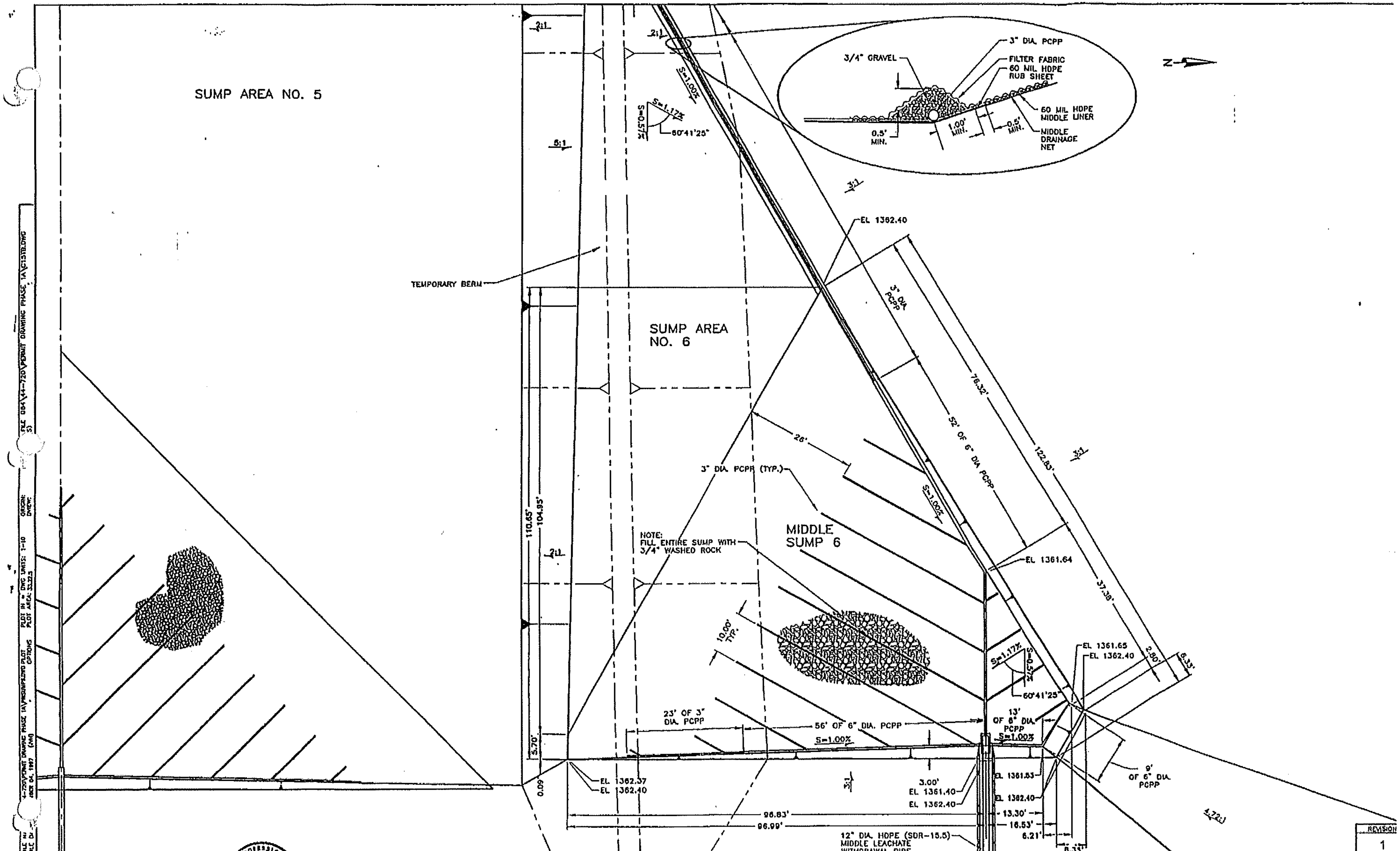
VERIFY SCALE

LAIDLAW ENVIRONMENTAL

LONE MOUNTAIN FACILITY
LANDFILL CELL 15

REVISION 1

SHEET NO. 25



DESIGNED	MPW	3
DRAFTED	RGA	2
CHECKED	KCS	1 — 9/97 — UPDATED COMPANY NAME — JVH — KCS
DATE	JULY 1996	NO. — DATE — REVISIONS

HA&L ENGINEERING — CONSULTANTS ENGINEERS — Salt Lake City Utah

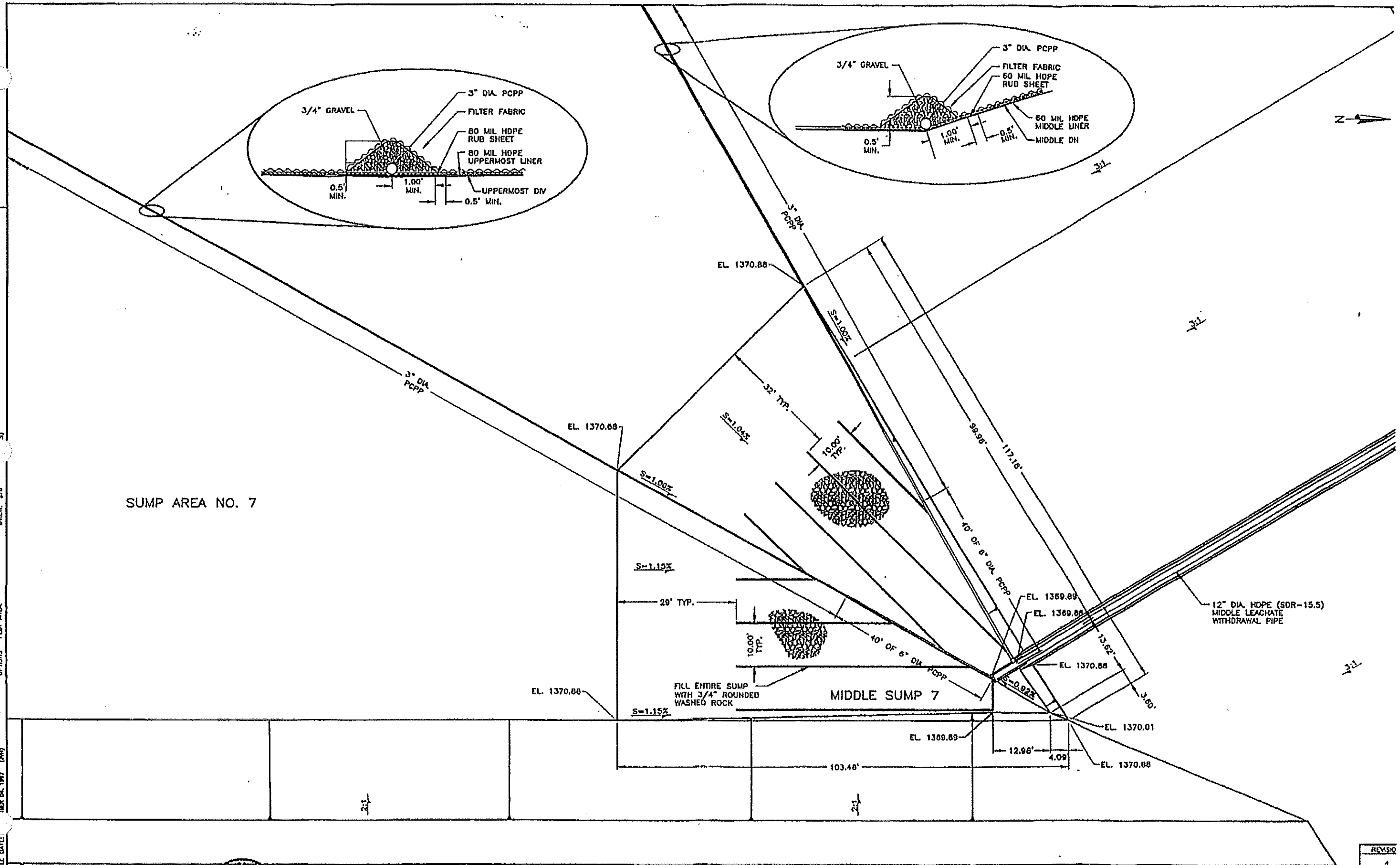
SCALE 1"=10'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 6 – PHASE II

REVISION 1

SHEET NO. 26 OF 44



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	DESIGNED MPW	3		
		DRAFTED RGA	2		
		CHECKED KCS	1	9/97 UPDATED COMPANY NAME	JVH KCS

SCALE 1"=10'

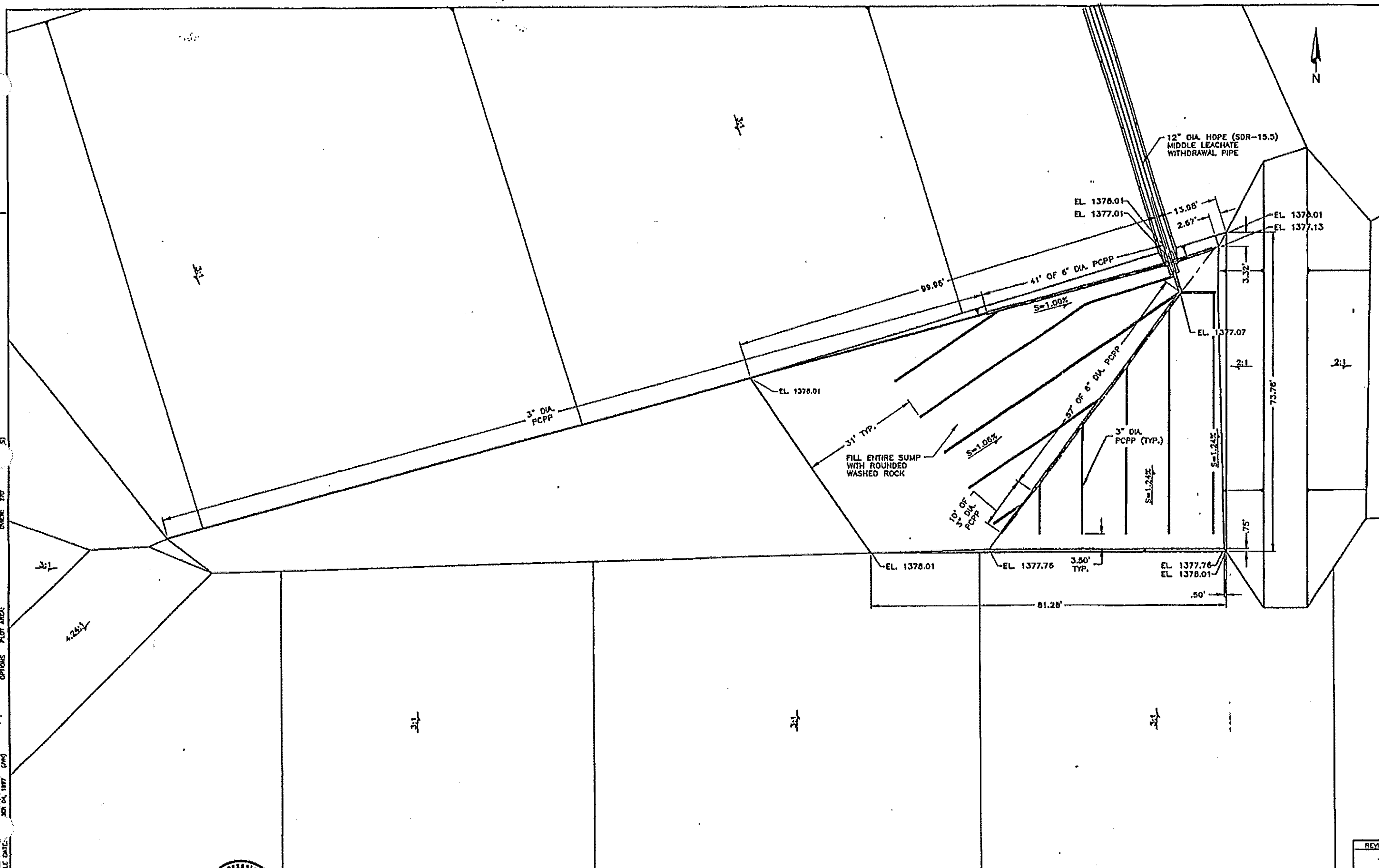
VERIFY SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 7 PHASE III

REVISION 1

SHEET NO. 27 OF 44



HA&L ENGINEERING CONSULTANTS ENGINEERS Salt Lake City Utah

PROJECT ENGINEER

DESIGNED	MPW	3			
DRAFTED	RGA	2			
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY APVD.

SCALE 1"=10'

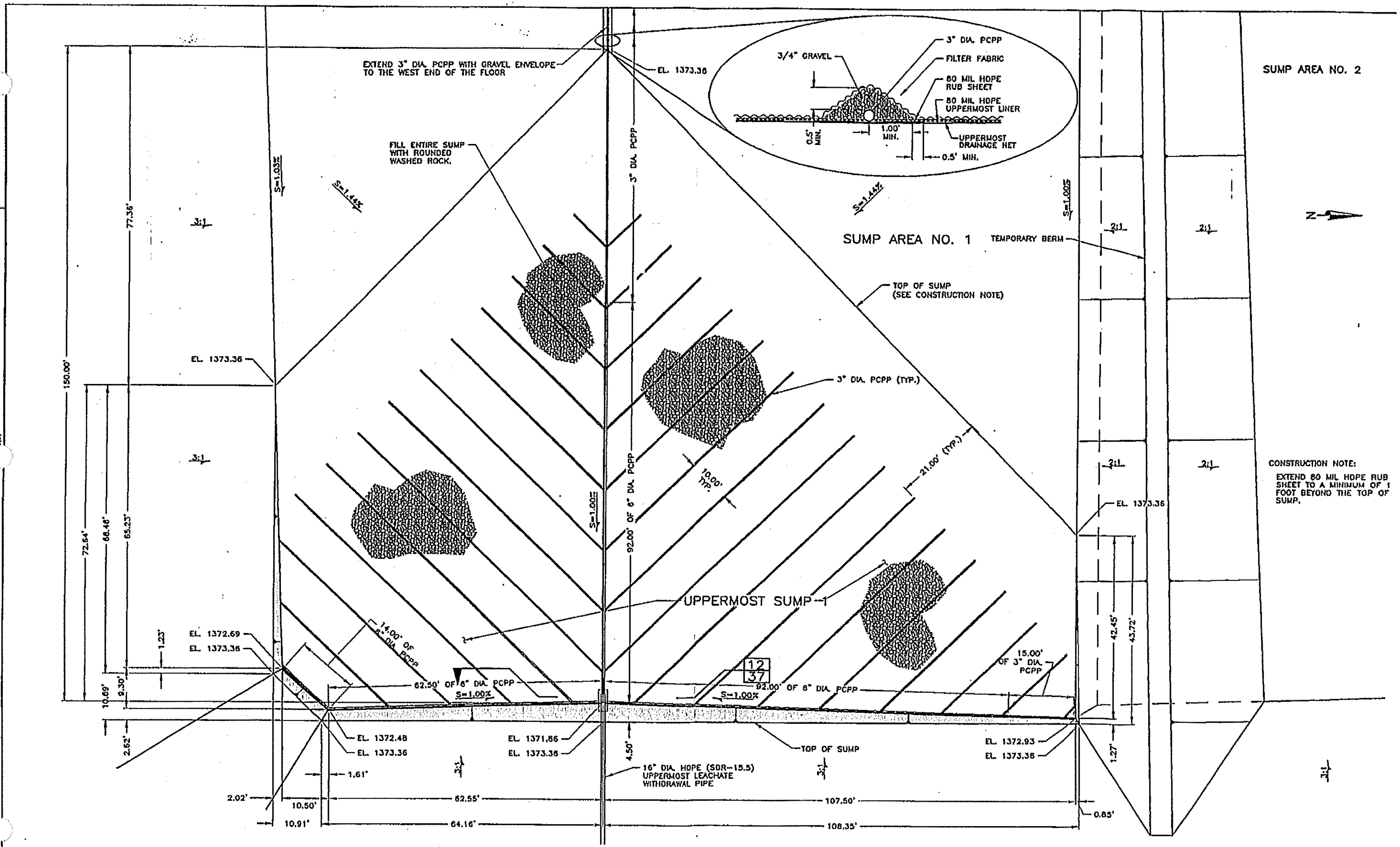
VERIFY SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

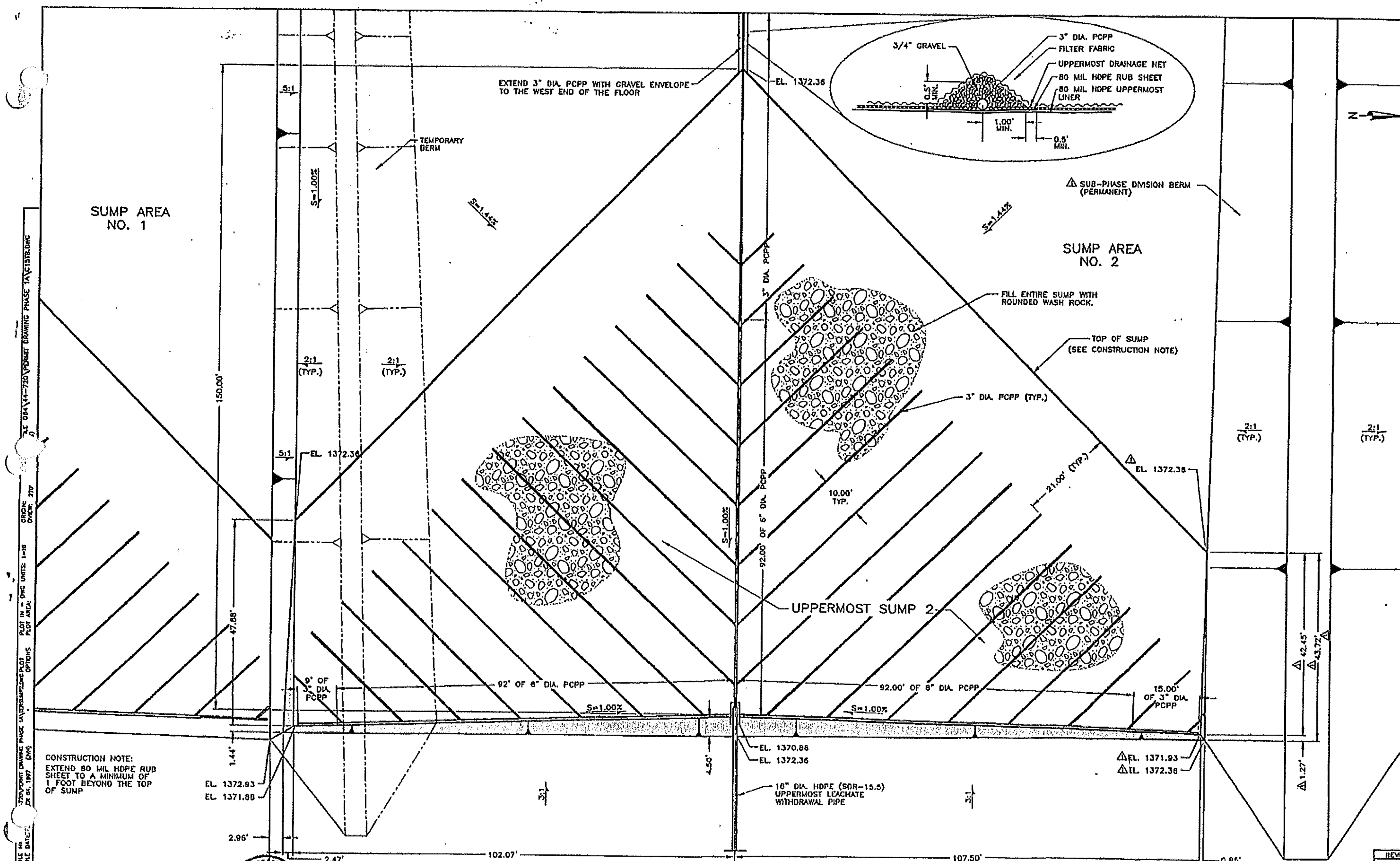
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
MIDDLE SUMP NO. 8 – PHASE III

REVISION 1

SHEET NO. 28 OF 44



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	PROJECT ENGINEER	DESIGNED MPW DRAFTED RGA CHECKED KCS DATE JULY 1996	3 2 1 9/97 UPDATED COMPANY NAME JVH KCS NO. DATE REVISIONS BY APP.	SCALE 1"=10'	VERIFY SCALE	LAIDLAW ENVIRONMENTAL SERVICES, INC.	LONE MOUNTAIN FACILITY LANDFILL CELL 15 UPPERMOST SUMP NO. 1 – PHASE IA	REVISION 1 SHEET NO. 29 OF 44



HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City, Utah

DESIGNED	MPW	3	
DRAFTED	RGA	2	
CHECKED	KCS	1	9/97 UPDATED COMPANY NAME – MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASES IA & IB — JVH KCS

SCALE 1"=10'

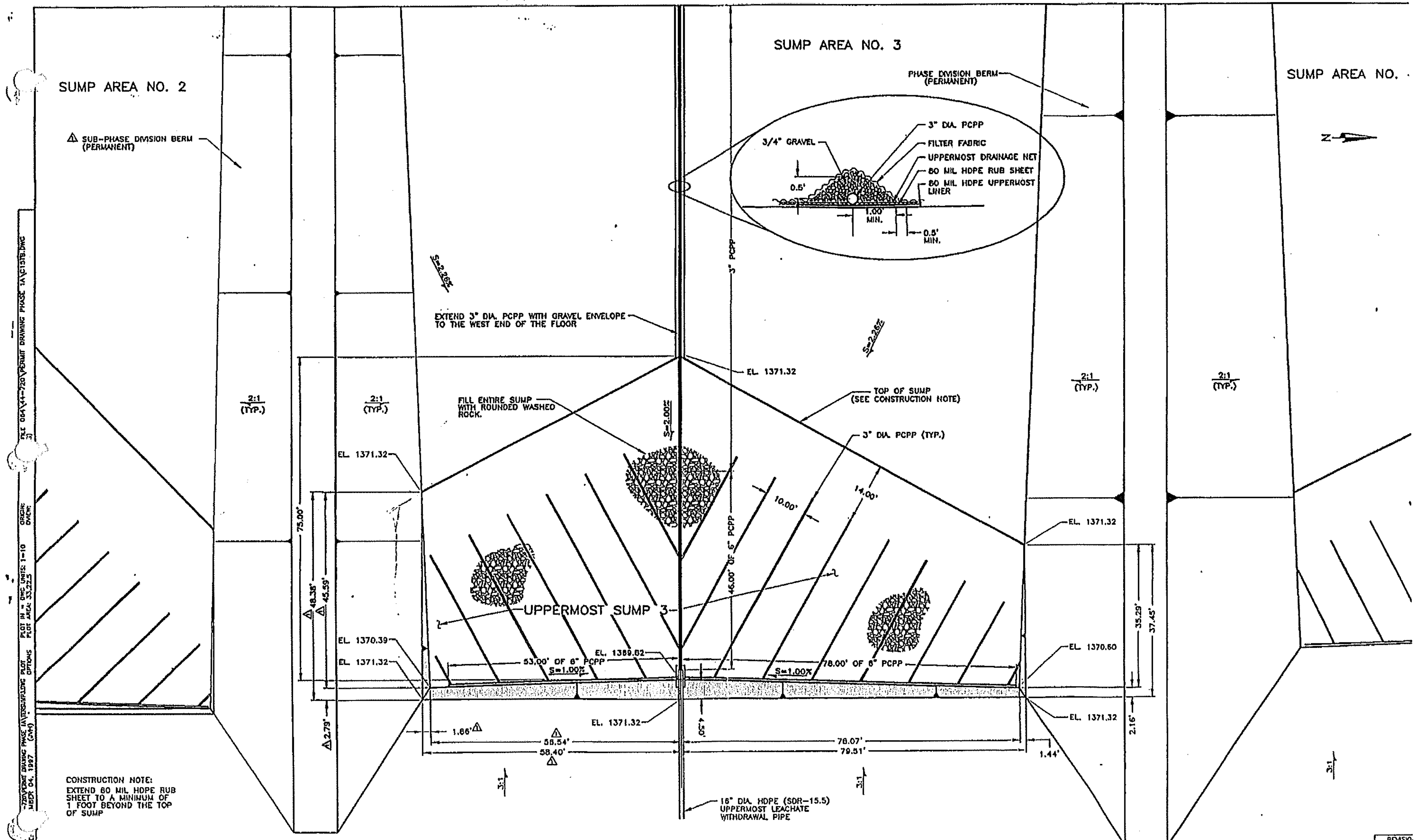
VERIFY SCALE

LAIDLAW ENVIRONMENTAL

LONE MOUNTAIN FACILITY
LANDFILL CELL 15

REVISION 1

SHEET 30



HA&L ENGINEERING CONSULTANTS ENGINEERS Salt Lake City Utah

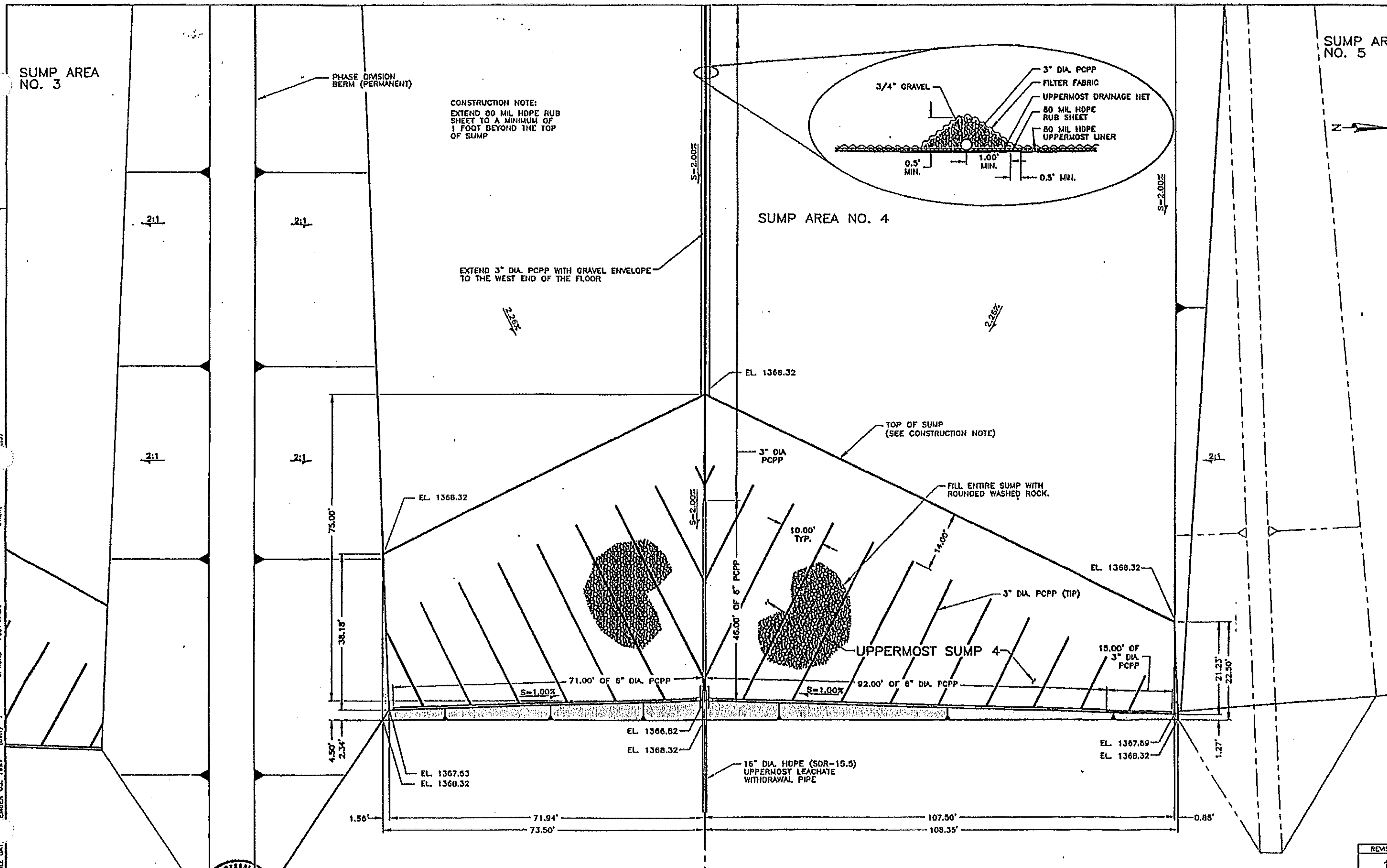
DESIGNED	MPW	3		
DRAFTED	SDM	2		
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME – MODIFICATIONS TO DIVIDE PHASE I INTO SUB-PHASES IA & IB — JVH KCS
DATE	JULY 1995	NO.	DATE	REVISIONS

SCALE 1"=10'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST SUMP NO.3 – PHASE IB

REVISION 1
SHEET NO. 31 OF 44



DESIGNED	MPW	3		
DRAFTED	RGA	2		
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME
DATE	JULY 1996	NO.	DATE	REVISIONS

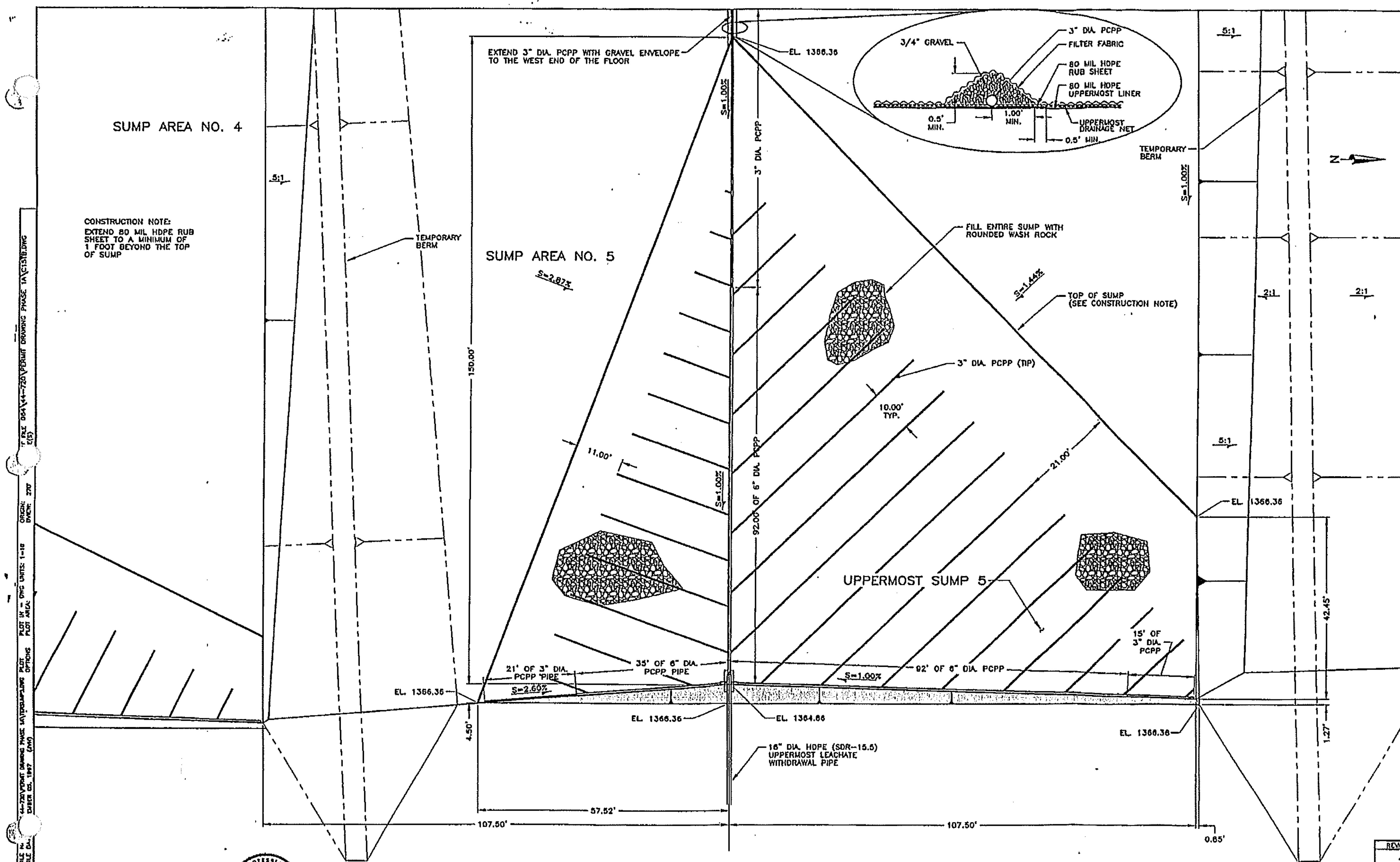
HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

SCALE 1"=10'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST SUMP NO. 4 – PHASE II

REVISION 1
SHEET NO. 32 OF 44



HA&L ENGINEERING	CONSULTANTS ENGINEERS	Salt Lake City Utah
DESIGNED	MPW	3
DRAFTED	RGA	2
CHECKED	KCS	1 · 9/97 · UPDATED COMPANY NAME · JVH · KCS
DATE	JULY 1996	NO. · DATE · REVISIONS · BY · APVD.

SCALE 1"=10'

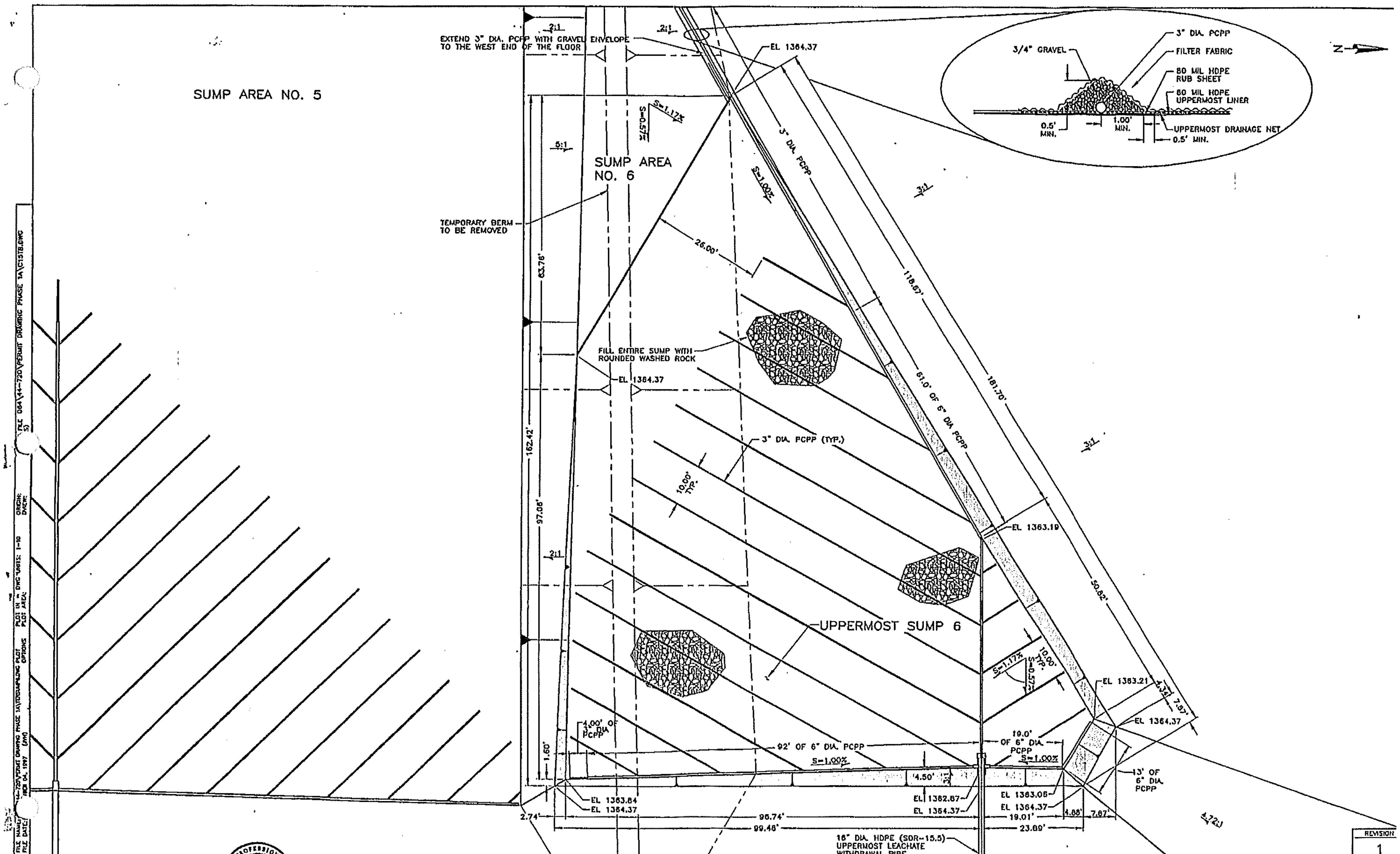
VERIFY SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

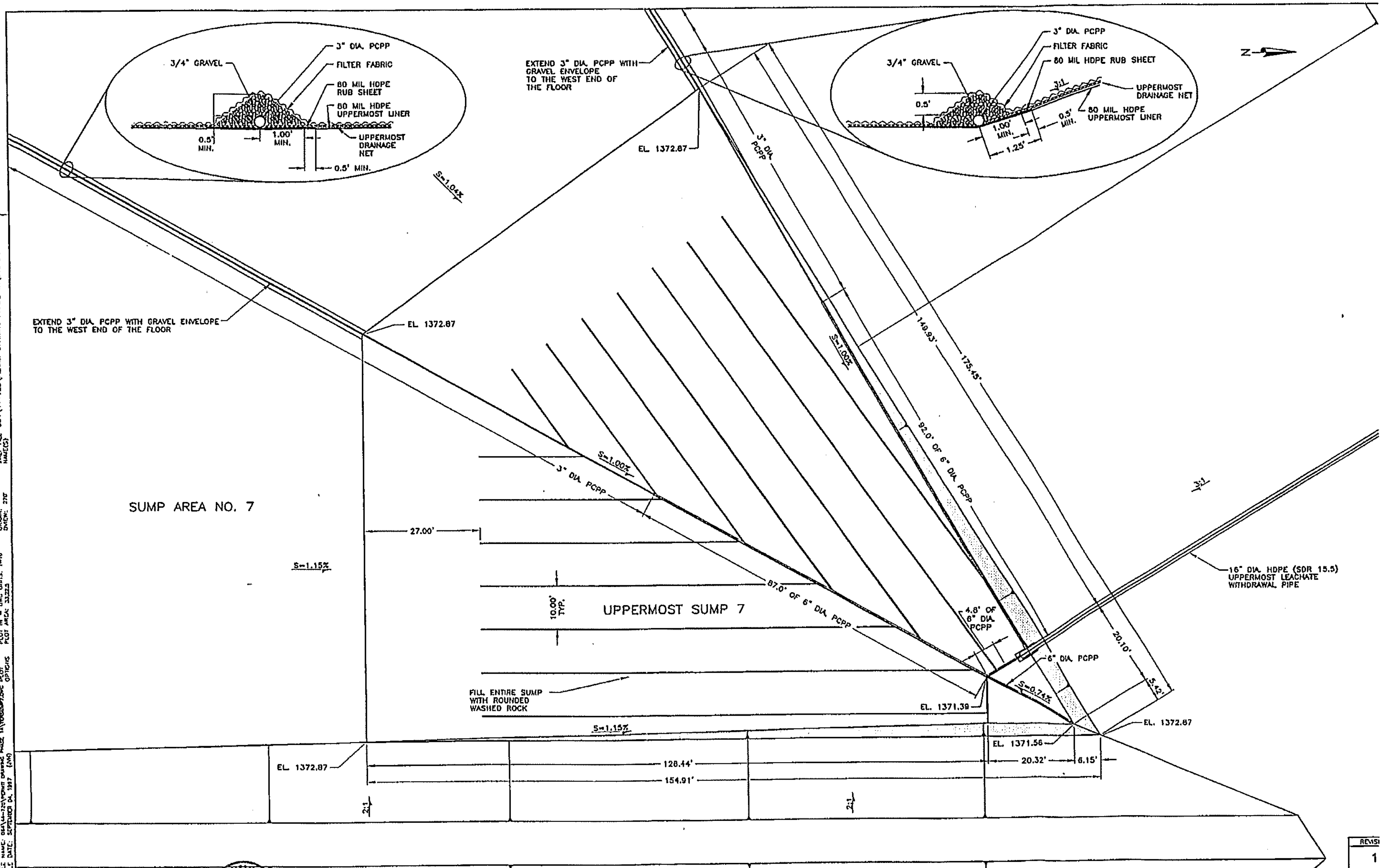
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST SUMP NO. 5 – PHASE II

REVISION 1

SHEET NO. 33 OF 44



HA&L ENGINEERING, Consultants Engineers, Salt Lake City Utah	DESIGNED MPW / DRAFTED RGA / CHECKED KCS / DATE JULY 1996	REVISIONS: 1 — 9/97 — UPDATED COMPANY NAME — JVH — KCS	SCALE 1"=10'
LAIDLAW ENVIRONMENTAL SERVICES, INC.	LONE MOUNTAIN FACILITY LANDFILL CELL 15 UPPERMOST SUMP NO. 6 – PHASE II	SHEET NO. 34 OF 44	REVISION 1



HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah	DESIGNED MPW	3			
PROJECT ENGINEER	DRAFTED RGA	2			
	CHECKED KCS	1	9/97	UPDATED COMPANY NAME	JVH KCS
	DATE JULY 1996	NO.	DATE	REVISIONS	BY APVD.

SCALE 1"=10'

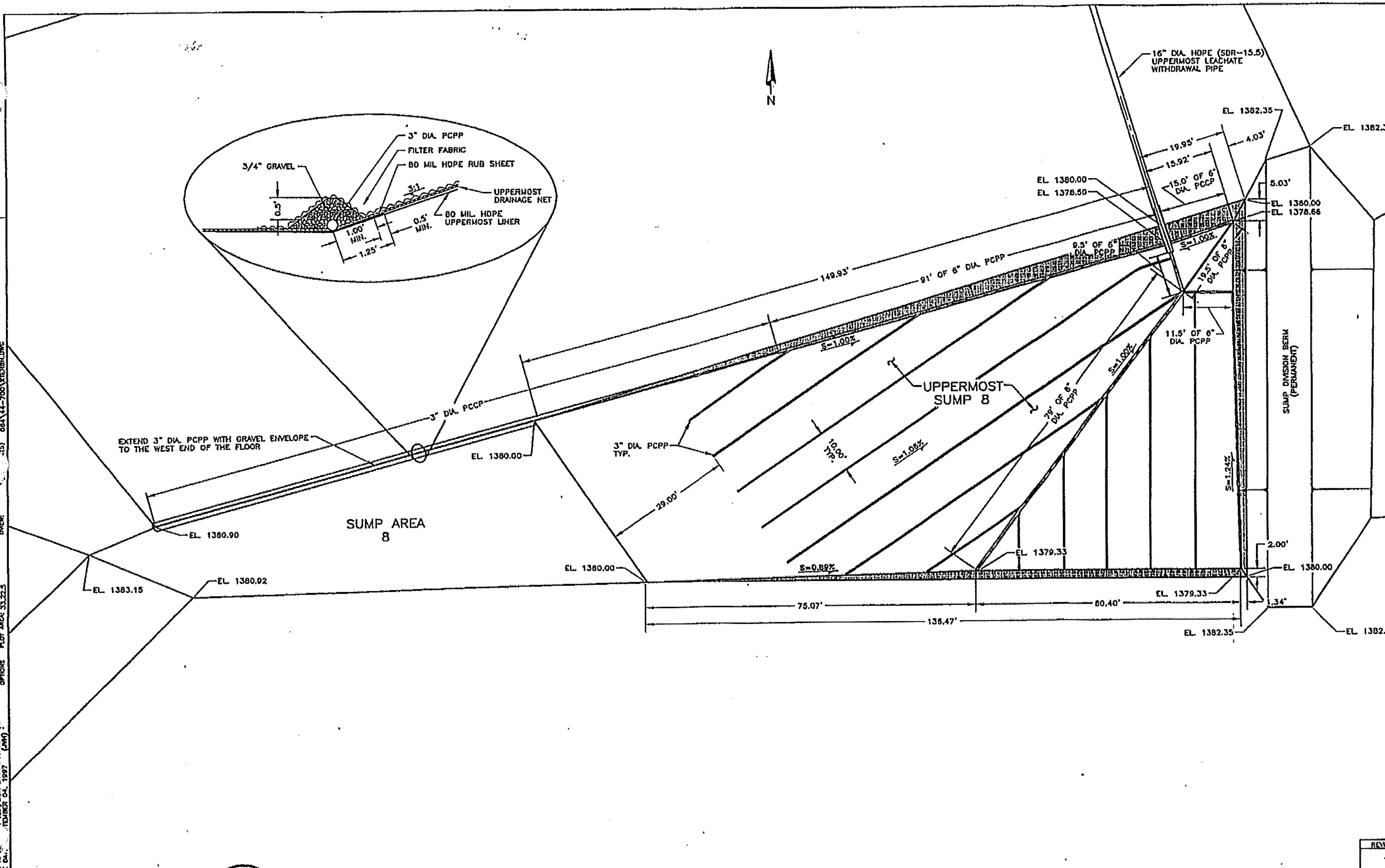
VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST SUMP NO. 7 – PHASE III

REVISION 1

SHEET 35 OF 44



HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

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DRAFTED	DRB	2				
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH	KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY	APVD.

PROJECT ENGINEER

SCALE 1"=10'

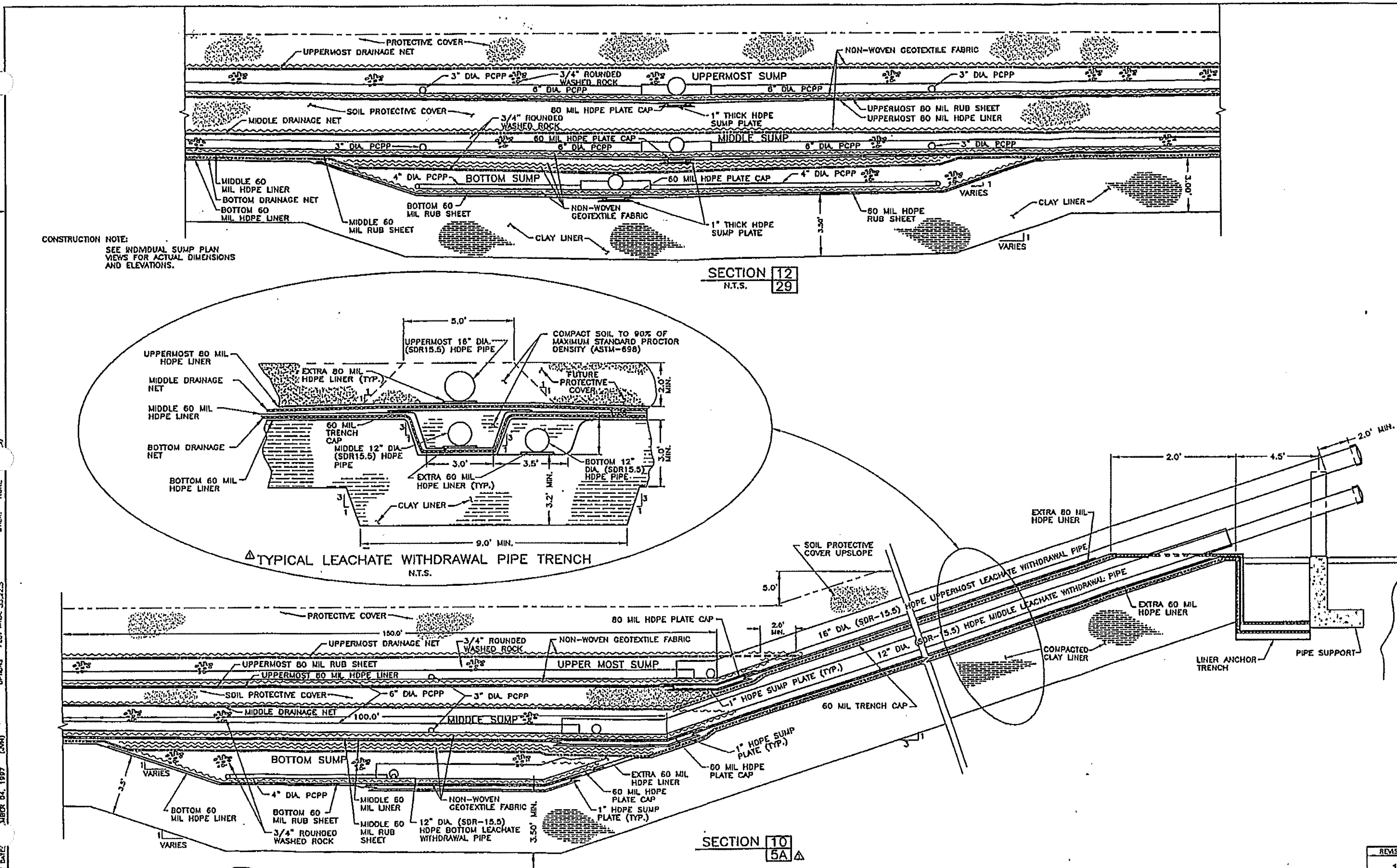
VERIFY SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
UPPERMOST SUMP NO. 8 – PHASE III

REVISION 1

SHEET NO. 36 OF 44



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DATE JULY 1996	NO.	DATE	REVISIONS	BY	APVD.	

HA&L ENGINEERING CONSULTANTS ENGINEERS Salt Lake City Utah

PROJECT ENGINEER

SCALE NOT TO SCALE

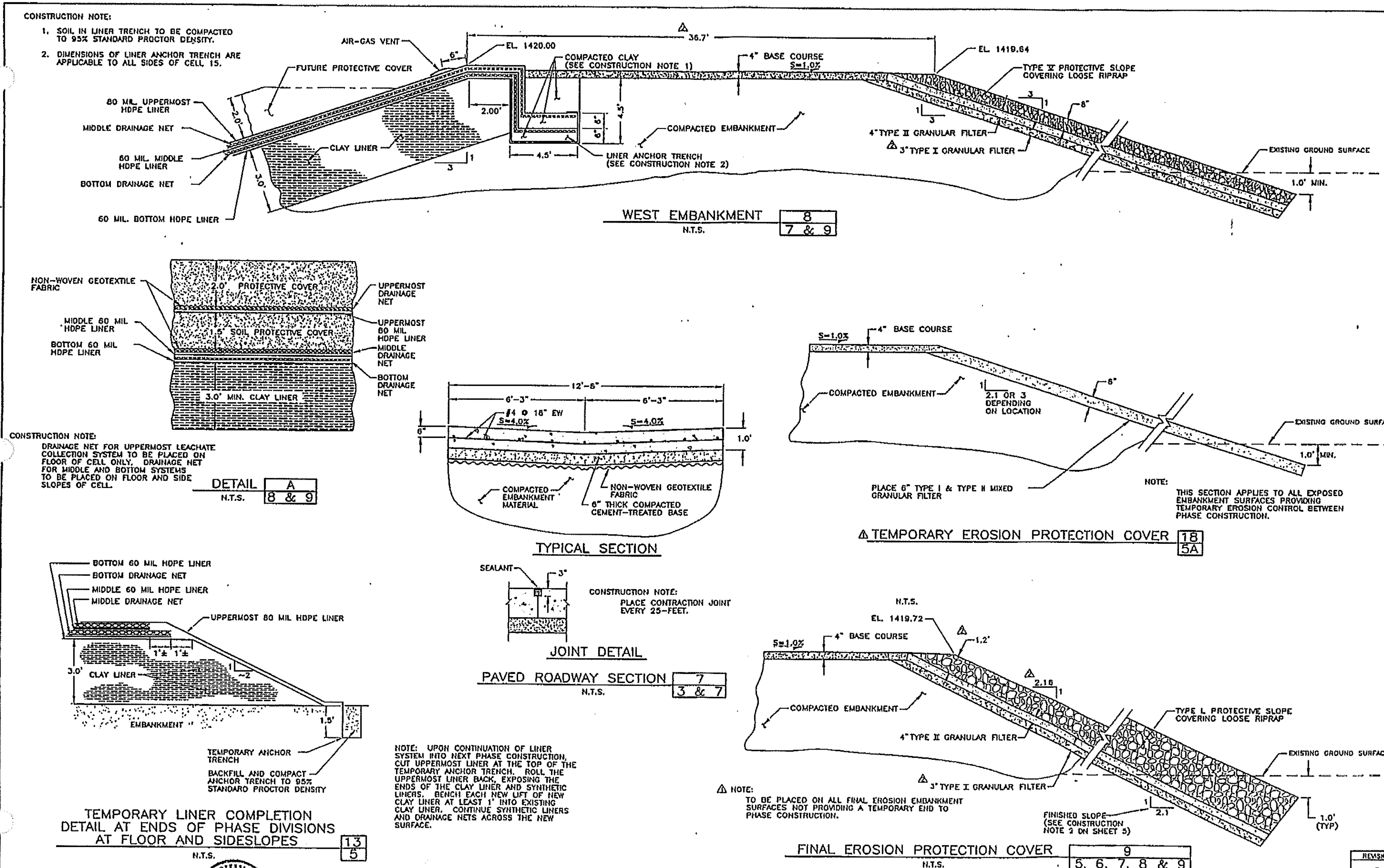
VERIFY SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

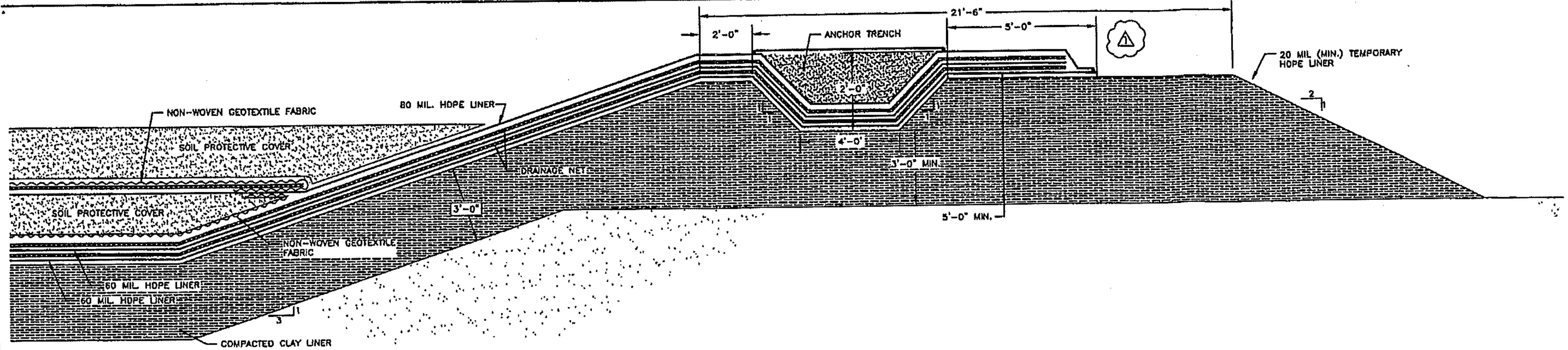
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
SUMP SECTIONS

REVISION 1

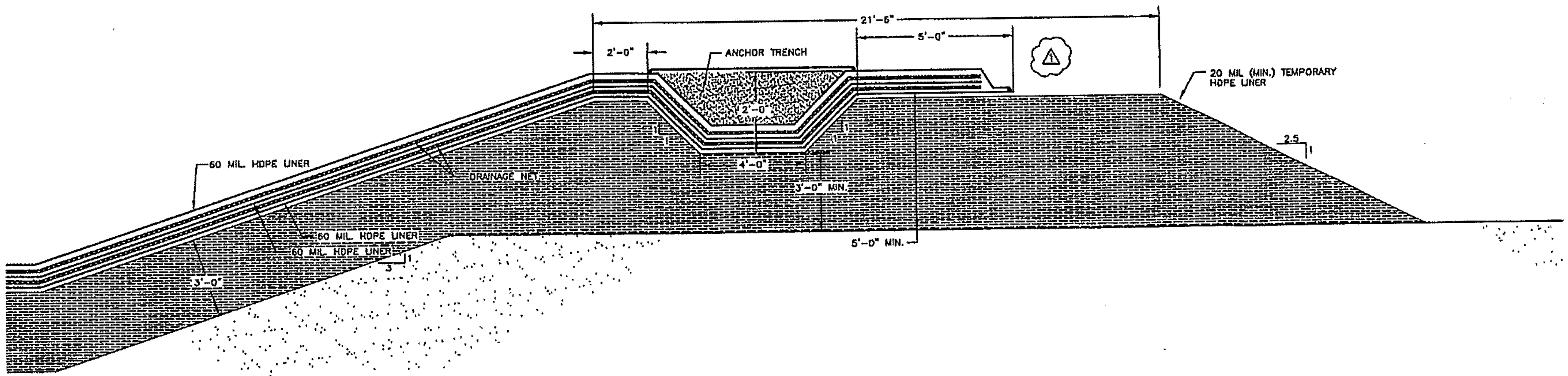
SHEET NO. 37 OF 44



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	DESIGNED MPW / DRAFTED JSJ / CHECKED KCS / DATE JULY 1996	
REVISIONS	3 — 10/97 — MODIFIED PROTECTIVE SLOPE THICKNESS – SECTIONS 8 & 9 — JVH — KCS	2 — 9/97 — UPDATED COMPANY NAME – ADDED TEMPORARY EROSION PROTECTION COVER DETAIL — JVH — KCS	
SCALE: NOT TO SCALE	LAIDLAW ENVIRONMENTAL SERVICES, INC.	LONE MOUNTAIN FACILITY LANDFILL CELL 15 TYPICAL SECTIONS & DETAILS	REVISION 2 / SHEET NO. 38 OF 44



WEST – SECTION 1/8
N.T.S.



WEST – SECTION 2/8
N.T.S.

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DRAFTED CAH	2		
CHECKED KCS	⚠	08/2005	MODIFIED SECTIONS, DIMENSIONS & ANCHOR TRENCH — CAH — KCS
DATE JANUARY 2005	NO.	DATE	REVISIONS — BY — APVD.

HANSEN ALLEN & LUCE INC ENGINEERS

PROJECT ENGINEER

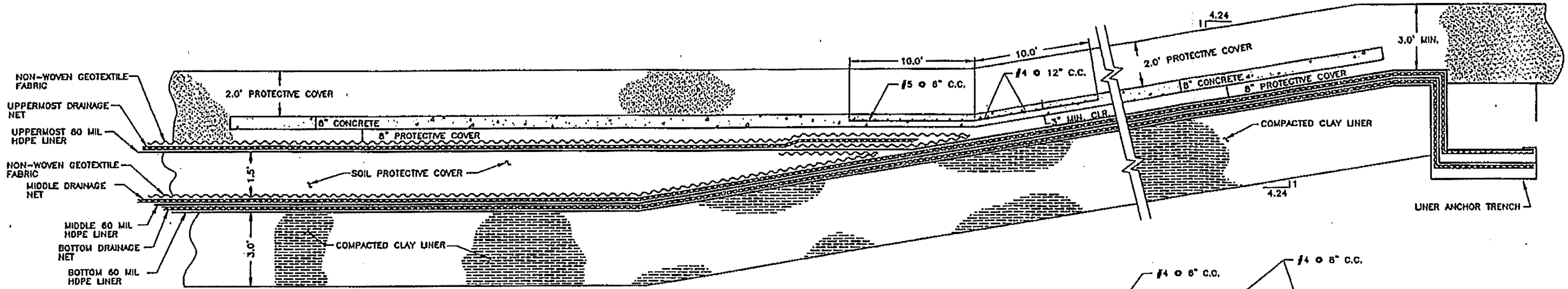
SCALE: NOT TO SCALE

VERIFY SCALE: 0" — 1". BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

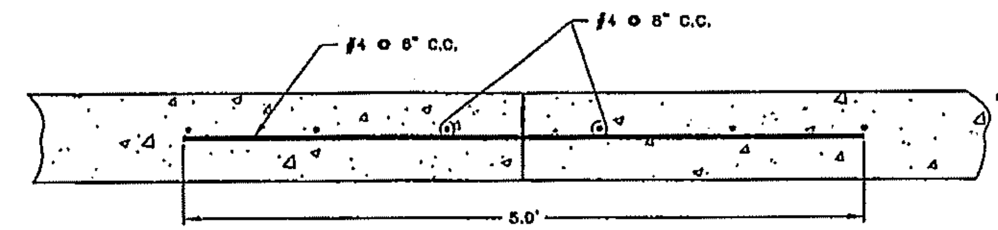
CleanHarbors ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15 – PHASE 2A
WEST – SECTIONS

SHEET 3



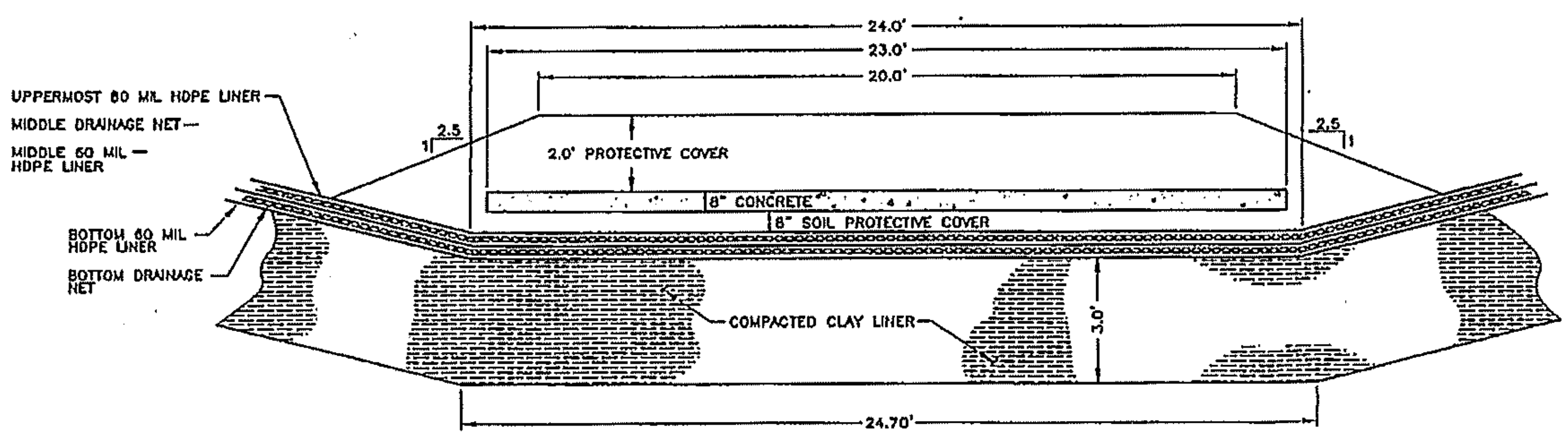
SECTION 14
N.T.S.



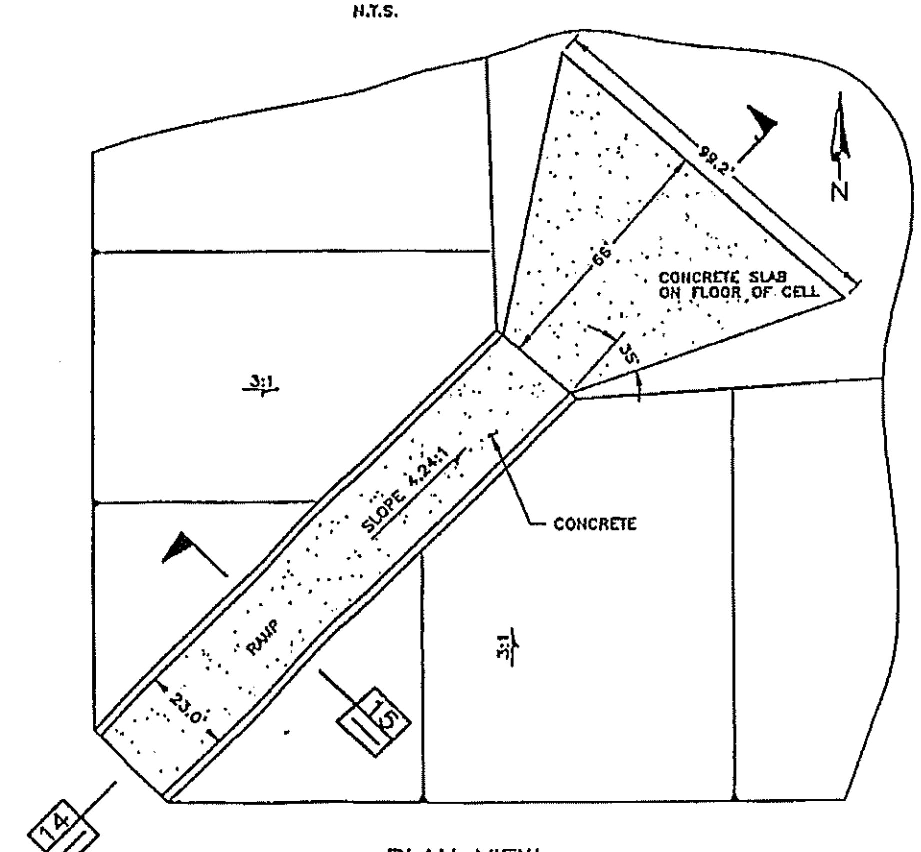
TYPICAL CONSTRUCTION JOINT
N.T.S.

SPECIFICATIONS FOR PLACEMENT OF RAMP

1. THE RAMP, INCLUDING THE PROTECTIVE COVER AND THE CONCRETE SHALL BE CONSTRUCTED FROM THE BOTTOM OF THE SLOPE TO THE TOP. THE 8-INCH SOIL PROTECTIVE COVER (UNDERLYING THE CONCRETE), THE CONCRETE SLAB, AND THE 2-FOOT COVER OVER THE CONCRETE SHALL BE CONSTRUCTED ON THE FLOOR OF THE CELL (AT THE BOTTOM OF THE SLOPE) PRIOR TO THE CONSTRUCTION OF THE RAMP.
2. THE 8-INCH PROTECTIVE COVER, THE 8-INCH CONCRETE SLAB, AND THE 2-FOOT PROTECTIVE COVER SHALL BE CONSTRUCTED IN SECTIONS NOT EXCEEDING 45 FEET IN LENGTH AS MEASURED ALONG THE SLOPE OF THE RAMP. THE CONCRETE MATERIAL SHALL ACHIEVE A MINIMUM STRENGTH OF 200 PSI, BEFORE COMMENCING CONSTRUCTION OF THE OVERLAYING 2-FOOT SOIL PROTECTIVE COVER, AND THE 8-INCH SOIL PROTECTIVE COVER AND CONCRETE LAYER FOR THE NEXT SECTION OF THE RAMP.
3. THE CONCRETE SHALL HAVE A MINIMUM 28 DAY STRENGTH OF 4000 PSI.
4. THE SURFACE OF THE CONCRETE SHALL BE LEFT IN A ROUGHENED CONDITION. THE SURFACE SHALL BE SCREEDED ONLY.
5. THE SECTIONS OF THE CONCRETE SHALL BE FORMED SO AS TO PROVIDE A CONSTRUCTION JOINT THAT IS PERPENDICULAR TO THE SLOPE OF THE RAMP BETWEEN SECTIONS AS THE RAMP IS CONSTRUCTED UP THE SLOPE.



SECTION 15
N.T.S.



PLAN VIEW
N.T.S.

HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	DESIGNED MPW	DRAFTED JSJ	CHECKED KCS	DATE JULY 1995
1	9/97	UPDATED COMPANY NAME	JVH	KCS	

REVISIONS

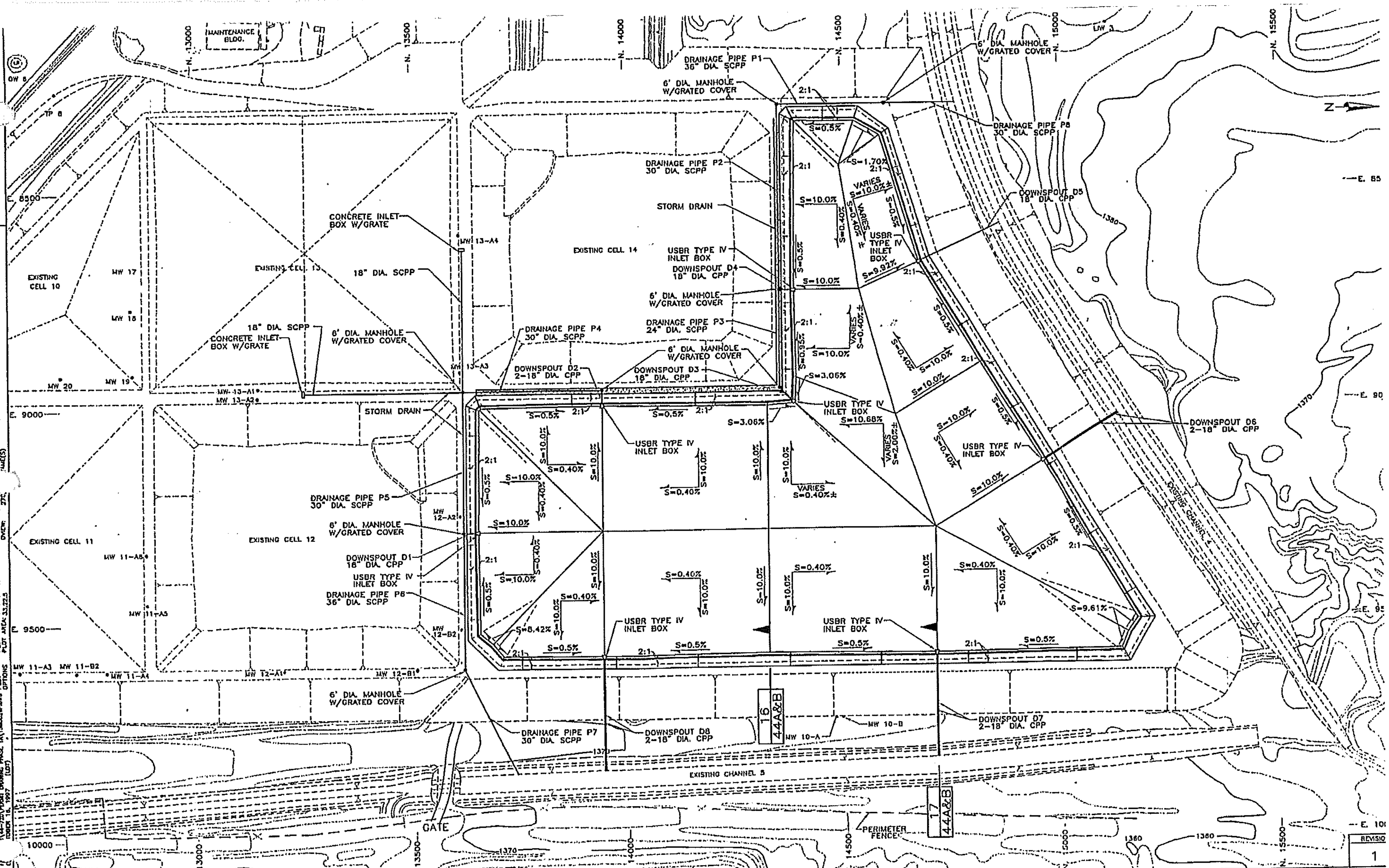
SCALE AS SHOWN

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
INTERIOR RAMP DETAILS – PHASE IA

REVISION 1

SHEET NO 39 OF 44



DESIGNED	MPW	3		
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CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	LDT	MEA
DATE	JULY 1996	NO.	DATE	REVISIONS	BY	APVD.

HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

PROJECT ENGINEER

SCALE 1"=100"

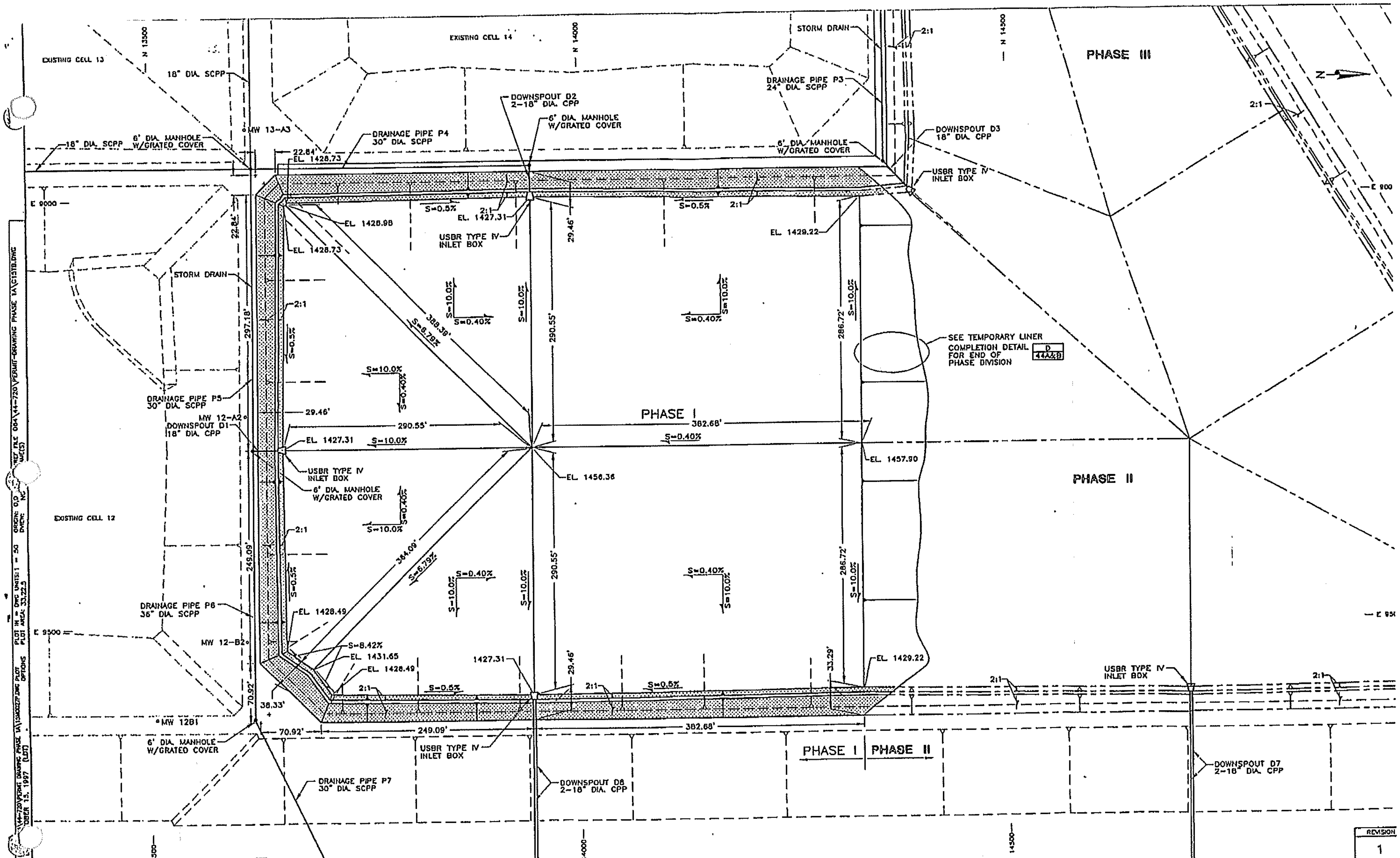
VERIFY SCALE: 0" — 1". BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
CLOSURE — PLAN VIEW

REVISION 1

SHEET 40 OF 44



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CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH / KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY / APVD.

HA&L ENGINEERING CONSULTANTS ENGINEERS Salt Lake City Utah

PROJECT ENGINEER

SCALE 1" = 50'

VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

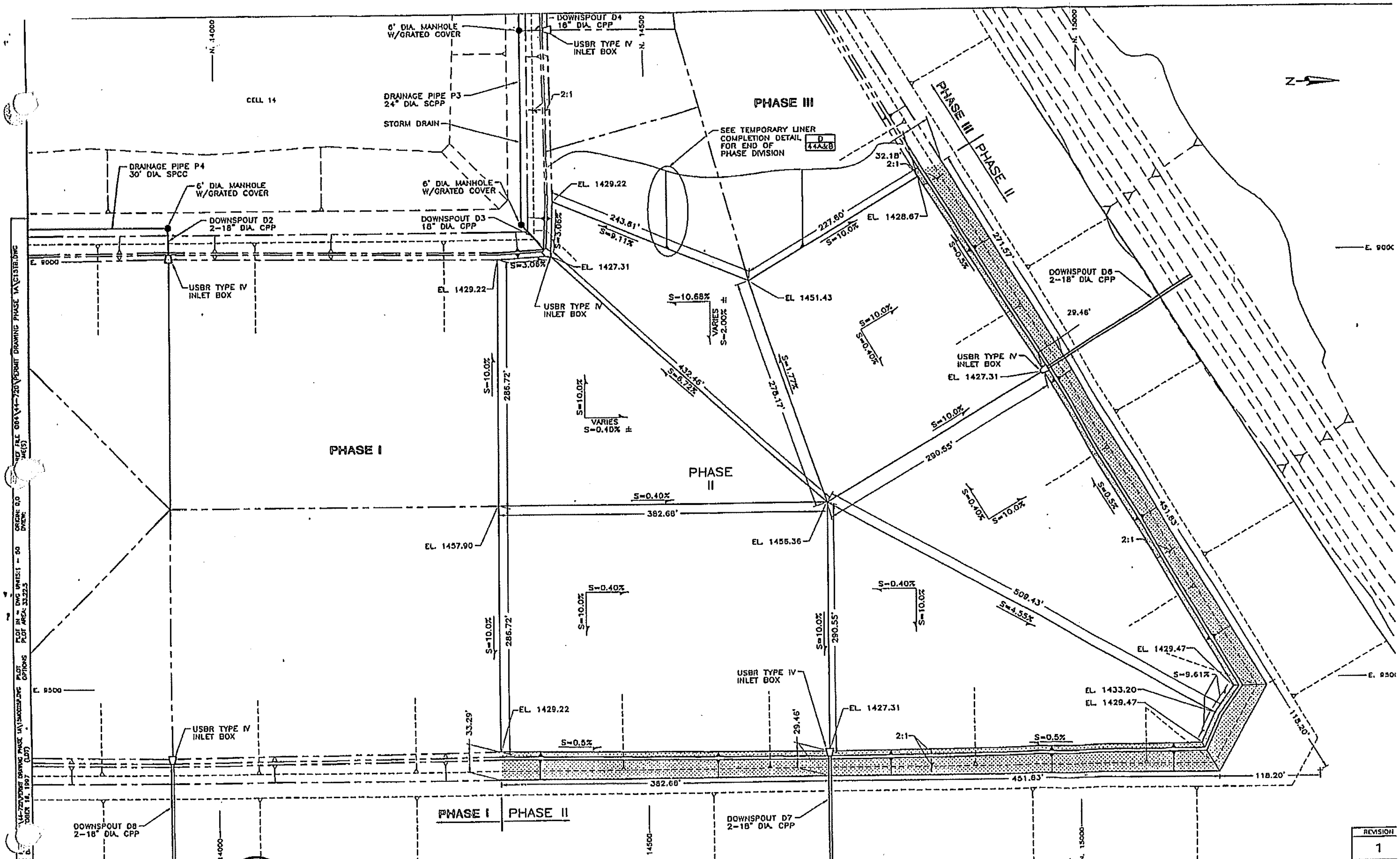
LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
CLOSURE – PLAN VIEW PHASE I

REVISION 1

SHEET NO 41 OF 44

64-44-70



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CHECKED	KCS	1	9/97	UPDATED COMPANY NAME
DATE	JULY 1996	NO.	DATE	REVISIONS

H&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

PROJECT ENGINEER

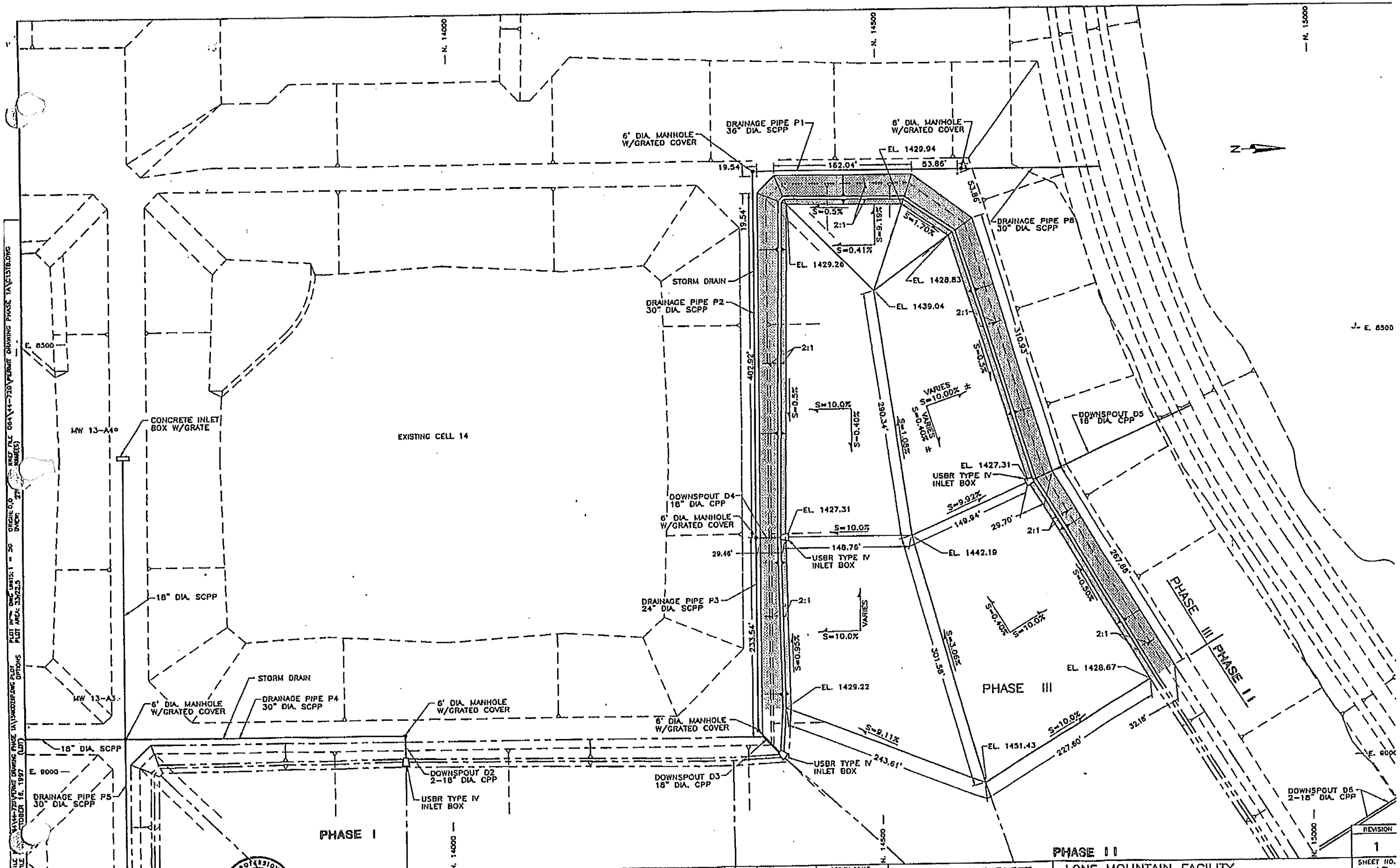
SCALE 1"=50'

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
CLOSURE – PLAN VIEW PHASE II

REVISION 1

SHEET NO. 42 OF 44



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	DESIGNED MPW	3			
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		CHECKED KCS	1	9/97	UPDATED COMPANY NAME	JWI KCS
	PROJECT ENGINEER	DATE JULY 1996	NO.	DATE	REVISIONS	BY APVD.

SCALE 1" = 50'

VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

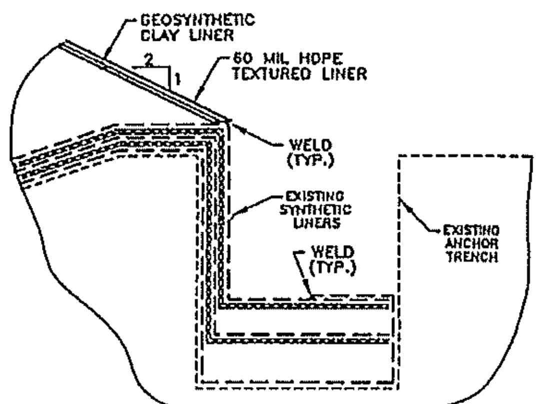
LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
CLOSURE – PLAN VIEW PHASE III

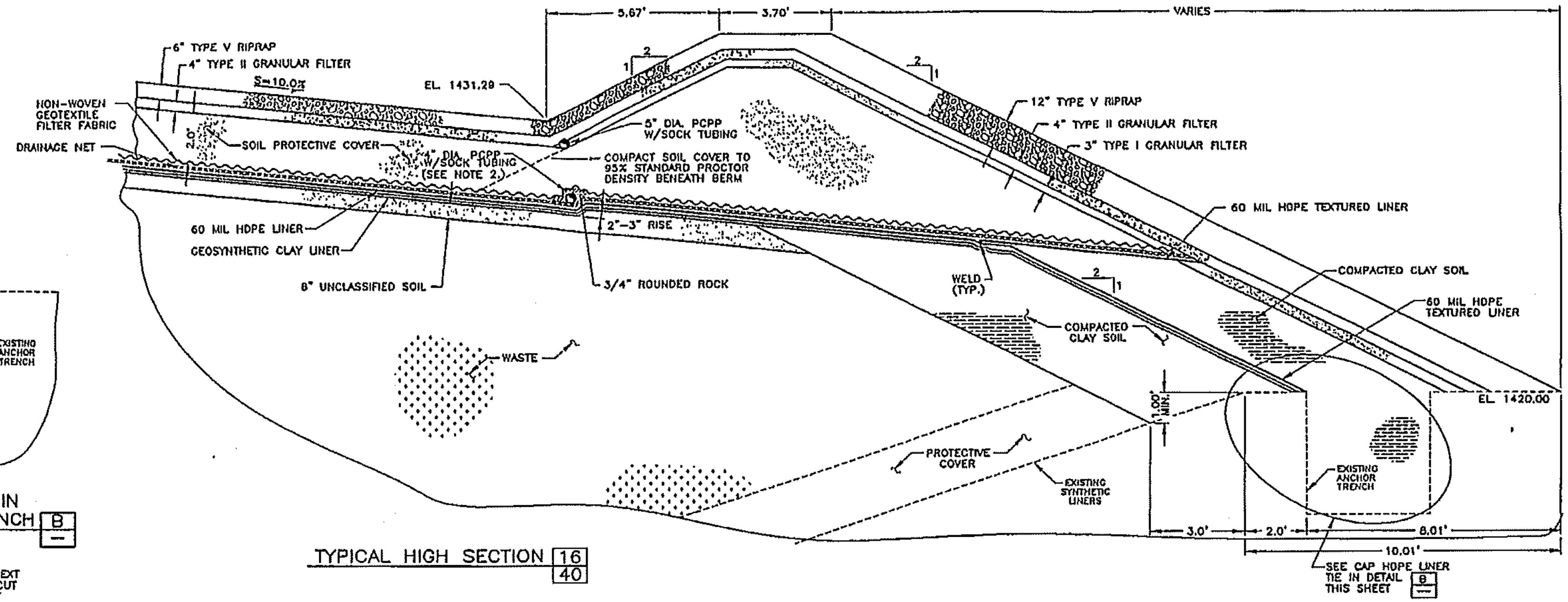
REVISION 1

SHEET NO. 43 OF 44

64-44-70



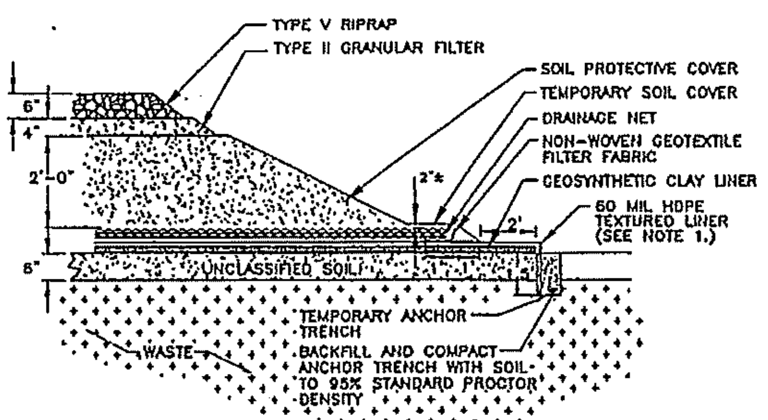
CAP HDPE LINER TIE IN DETAIL AT ANCHOR TRENCH B/—



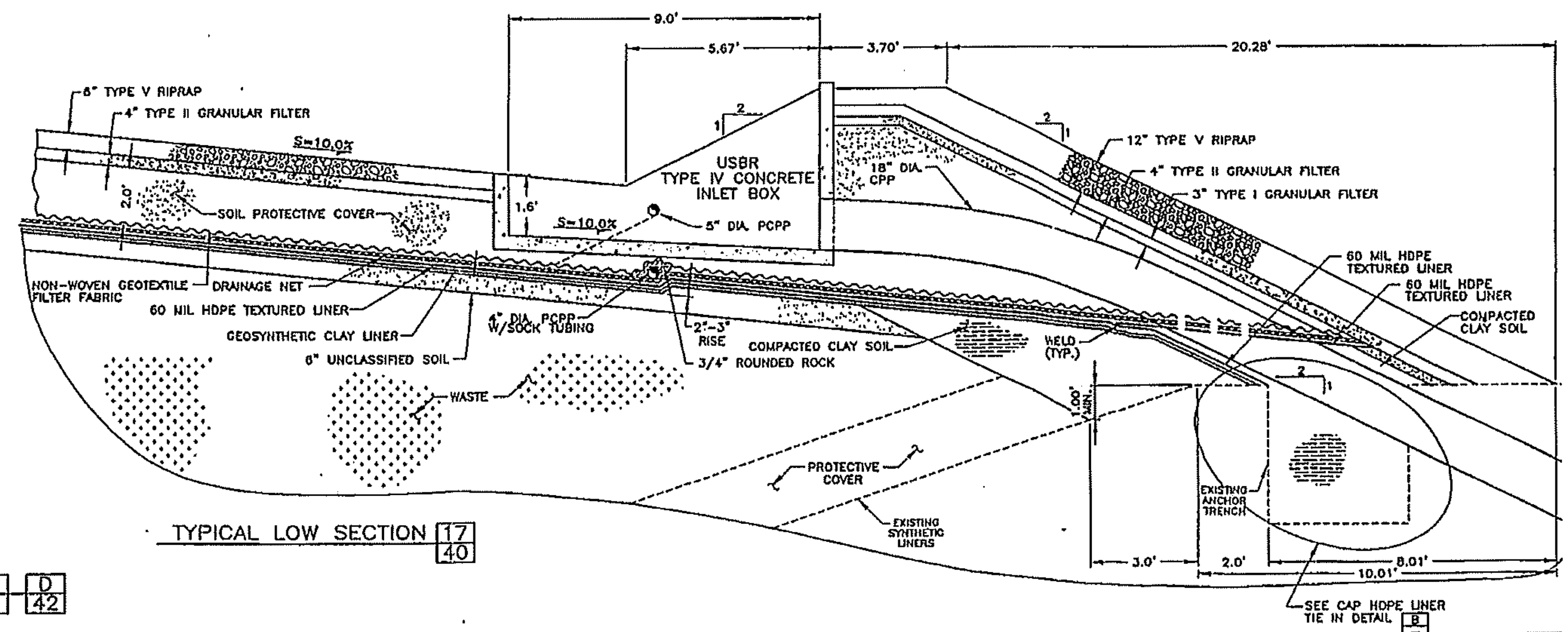
TYPICAL HIGH SECTION 16/40

NOTE:

1. UPON CONTINUATION OF LINER SYSTEM INTO THE NEXT SUBCELL OR PHASE OF CLOSURE CONSTRUCTION, CUT THE HDPE LINER AT THE EDGE OF THE TEMPORARY ANCHOR TRENCH. CONTINUE PLACEMENT OF THE COMPACTED CLAY MATERIAL AND SYNTHETIC LINER ACROSS NEW SURFACE. UNCOVER THE EDGE OF FILTER FABRIC & DRAINAGE NET AND TIE THE NEW NET TO THE EXISTING NET AND FABRIC TO FABRIC IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS. THEN CONTINUE PLACEMENT OF SOIL COVER, GRANULAR FILTER AND RIPRAP.
2. THE 4" DIA. PCPP IS TO BEND 90° ON EACH SIDE OF THE INLET BOX SUCH THAT THE 4" DIA. PCPP CAN EXTEND ALONG THE CAP SLOPE AND TIE INTO THE TOP OF THE 18" DIA. CPP FOR DRAINAGE.



TEMPORARY LINER COMPLETION DETAIL AT ENDS OF SUBCELL AND PHASE CLOSURES D/41 D/42



TYPICAL LOW SECTION 17/40

GEOSYNTHETIC CLAY LINER OPTION

HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	DESIGNED MPW	
		DRAFTED RGA	
	PROJECT ENGINEER	CHECKED KCS	
		DATE JULY 1996	

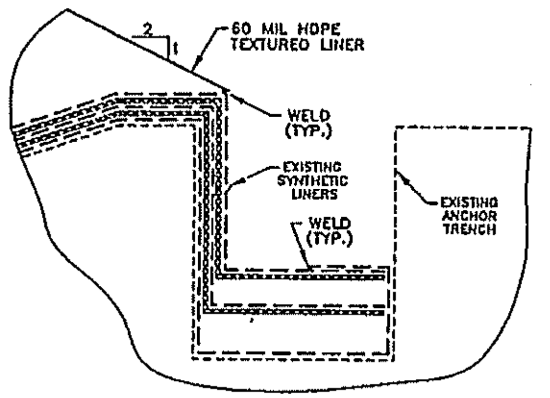
NO	DATE	REVISIONS	BY	APVD.
3				
2	9/97	MODIFIED EROSION THICKNESS	LDT	MEA
1	9/97	UPDATED COMPANY NAME	JAI	KCS

SCALE NOT TO SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
CLOSURE – SECTIONS & DETAILS

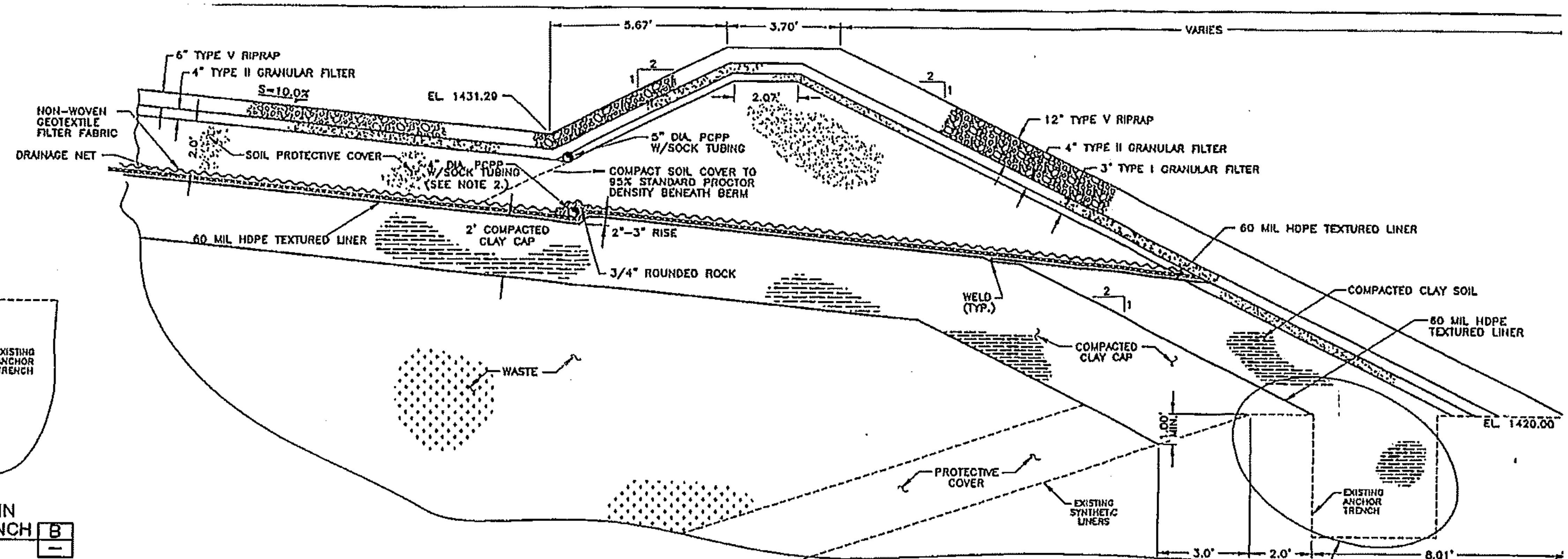
SHEET 4



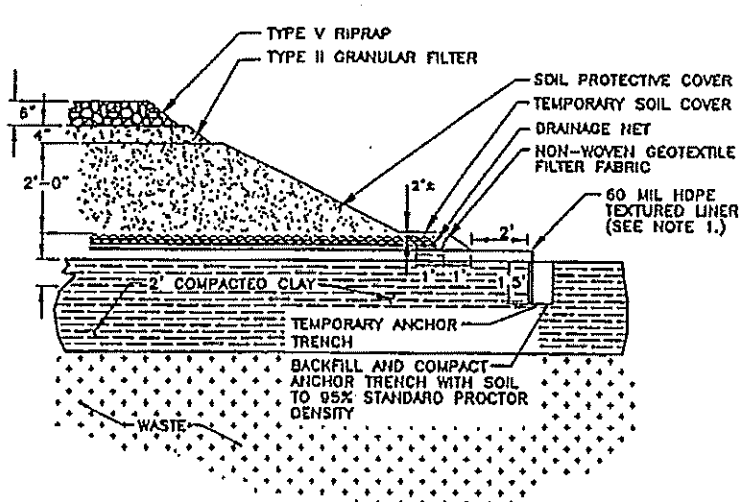
CAP HDPE LINER TIE IN DETAIL AT ANCHOR TRENCH B/—

NOTE:

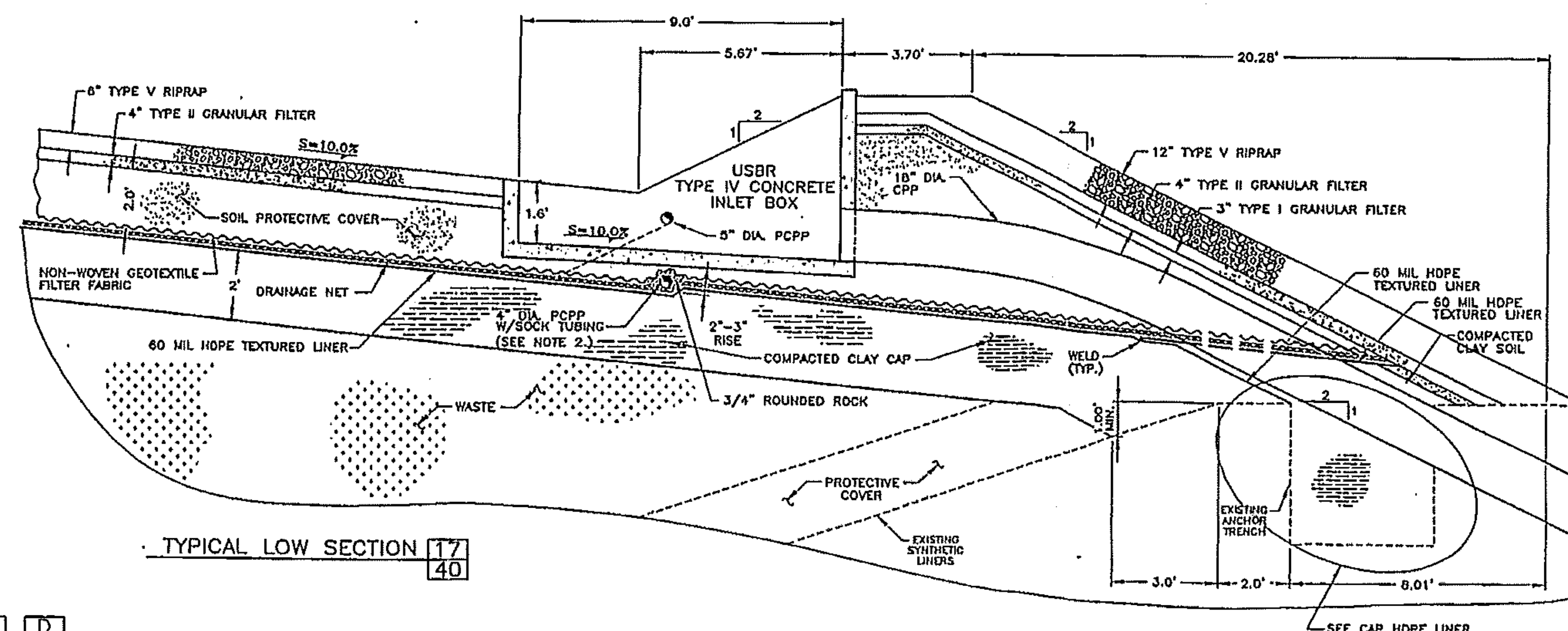
1. UPON CONTINUATION OF LINER SYSTEM INTO THE NEXT SUBCELL OR PHASE OF CLOSURE CONSTRUCTION, CUT THE HDPE LINER AT THE EDGE OF THE TEMPORARY ANCHOR TRENCH. CONTINUE PLACEMENT OF THE COMPACTED CLAY MATERIAL AND SYNTHETIC LINER ACROSS NEW SURFACE. UNCOVER THE EDGE OF FILTER FABRIC & DRAINAGE NET AND TIE THE NEW NET TO THE EXISTING NET AND FABRIC TO FABRIC IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS. THEN CONTINUE PLACEMENT OF SOIL COVER, GRANULAR FILTER AND RIPRAP.
2. THE 4" DIA. PCPP IS TO BEND 90° ON EACH SIDE OF THE INLET BOX SUCH THAT THE 4" DIA. PCPP CAN EXTEND ALONG THE CAP SLOPE AND TIE INTO THE TOP OF THE 18" DIA. CPP FOR DRAINAGE.



TYPICAL HIGH SECTION 16/40



TEMPORARY LINER COMPLETION DETAIL AT ENDS OF SUBCELL AND PHASE CLOSURES D/41 D/42



TYPICAL LOW SECTION 17/40

COMPACTED CLAY CAP OPTION

HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City Utah

DESIGNED	MPW	3			
DRAFTED	RGA	2	9/97	MODIFIED EROSION THICKNESS	LDT / MEA
CHECKED	KCS	1	9/97	UPDATED COMPANY NAME	JVH / KCS
DATE	JULY 1996	NO.	DATE	REVISIONS	BY / APVD.

SCALE: NOT TO SCALE

LAIDLAW ENVIRONMENTAL SERVICES, INC.

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
CLOSURE – SECTIONS & DETAILS

EXHIBIT B

GEOTECHNICAL INVESTIGATION
LANDFILL CELL 15
LONE MOUNTAIN FACILITY
USPCI
WAYNOKA, OKLAHOMA

Prepared by

Applied Geotechnical Engineering Consultants
Salt Lake City, Utah

Applied Geotechnical Engineering Consultants, Inc.

June 10, 1993

Mr. Kent Staheli
Hansen, Allen and Luce, Inc.
6771 South 900 East
Salt Lake City, Utah 84047-1436

Subject: Changes to Text for Landfill Cell 15
AGEC Project No. 24292

Dear Kent:

We have made the changes requested by Walter Sonne to the text of the report for Landfill Cell 15.

Enclosed are pages which required changes.

Best regards,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

Jim

James E. Nordquist, P.E.

JEN/cs
enclosure (3)

7109 South 185 West, Suite A • Midvale, Utah 84047 • (801) 566-6399 • FAX (801) 566-6493

Applied Geotechnical Engineering Consultants, Inc.

GEOTECHNICAL INVESTIGATION

LANDFILL CELL 15

LONE MOUNTAIN FACILITY

USPCI

WAYNOKA, OKLAHOMA

PREPARED FOR:

USPCI, INC.
515 WEST GREENS ROAD, SUITE 500
HOUSTON, TEXAS 77067

ATTN: WALTER SONNE

PROJECT NO. 24292

APRIL 13, 1993

7109 South 185 West, Suite A • Midvale, Utah 84047 • (801) 566-6399 • FAX (801) 566-6493

TABLE OF CONTENTS

CONCLUSIONS

1. The natural soil and bedrock are suitable for construction and support of the proposed landfill disposal cell.

2. The existing embankments of Landfill Cells 12 and 14 are suitable to incorporate in Landfill Cell 15.

3. Unsuitable material will need to be removed from the subgrade before placement of embankment materials. Unsuitable material is located along the north portion of the area where overburden material removed during previous cell construction has been stockpiled. We anticipate that unsuitable material will be encountered in the northeast corner, where a natural drainage is located. We also anticipate that other areas will have unsuitable material.

4. Exterior slopes of 2.1:1 (horizontal to vertical) and interior slopes of 3:1 (horizontal to vertical) are geotechnically stable.

5. Further design details and construction precautions are contained within the text of the report.

JAMES E.
15327
OKLAHOMA

SCOPE

This report presents the results of a Geotechnical Investigation for a proposed hazardous waste landfill to be constructed at the Lone Mountain Facility of USPCI, located near Waynoka, Oklahoma. The landfill is located within the southeast quarter of Section 28, Township 23 N, Range 50 W, IM in Major County, Oklahoma. The proposed landfill will be located north of existing Landfill Cell 12, north and east of Landfill Cell 14.

This report summarizes the data obtained and presents our conclusions and recommendations based on the subsurface conditions encountered and the proposed construction. Design and construction considerations related to the geotechnical aspects of the facility are included. A report for the proposed Industrial Waste Cell (IWC) 1 was prepared and submitted on May 18, 1990 under Project No. 14590. The information used for the design on Landfill Cells 12 and 13 were obtained from previous studies at the Lone Mountain Facility, primarily conducted for Landfill Cell 10 and also for the areas covered by Landfill Cells 12 and 13. A report for Landfill Cell 14 was prepared and submitted on December 3, 1991 under Project No. 19091. The information obtained from these previous studies are included in the analysis for Landfill Cell 15.

PROPOSED CONSTRUCTION

Landfill Cell 15 will be configured as shown on Figure 1. The east half of the south embankment of the landfill cell will be in common with the north embankment of Landfill Cell 12. The south portion of the west embankment will be in common with the east embankment of proposed Landfill Cell 14. The west portion of the south embankment will be in common with the north embankment of proposed Landfill Cell 14.

Three phases of construction are planned for the landfill cell. The proposed end of the embankment for Phase 1 and Phase 2 is shown in Figure 1.

The interior slopes of the landfill are planned to be constructed at 3:1 (horizontal to vertical). The floor elevation at the sump locations above the uppermost liner range from approximately 1363 to 1382 feet. The embankment crest elevation is planned at elevation 1420 feet. The exterior slopes are planned at 2.1:1 (horizontal to vertical) for the embankment and 2.15:1 (horizontal to vertical) for the rock protective cover. The ground surface around the exterior of the landfill ranges from approximately 1354 to 1370 feet. With these elevations, the maximum interior embankment height is approximately 57 feet, with the maximum exterior embankment height being approximately 66 feet.

Two ramps are proposed to enter the landfill cell, one from the west corner of the southwest embankment and the other from the west corner of the southeast embankment (as shown on Figure 1).

Closure will result in placing the waste and the cover materials up to elevations ranging from approximately 1420 to 1441 feet.

The interior portion of the cell will be constructed with flexible membrane liners, drainage nets, sump rock and soil protective cover. Materials along the bottom of the landfill cell consist of the following, extending from the top down:

2 feet of protective cover
Non-woven geotextile fabric
Drainage net
Uppermost liner (80 mil. HDPE)
1-1/2 feet of protective cover
Non-woven geotextile fabric
Drainage net
Middle liner (60 mil. HDPE)
Drainage net
Bottom liner (60 mil. HDPE)
3 feet of clay

Drainage pipes will collect any leachate within the sump areas. The pipes will be supported on the interior embankment slopes and will extend up to the top of the embankments. Within the sump areas, a double HDPE liner will be placed underneath the pipe section.

Material at or near the site will be used to construct embankments and the clay liner. Soil and bedrock worked into a soil-like material will be used for the embankment.

The area inside the landfill cell will be used to dispose of waste.

Prior to operation of each subcell area of the landfill cell, a protective cover will be placed on the bottom of the cell and on the interior slopes. The initial protective layer on the interior slopes will extend only 5 vertical feet. Protective material will extend higher up the slopes as waste is placed.

SITE CONDITIONS

At the time of the field investigation, November 13, 1992, the area for the proposed landfill cell was used for the processing of clay cap material and the stockpiling of overburden soils removed during the construction of previous landfill cells.

It appears that an old drainage traverses through the site. The drainage exists near the northeast corner of the proposed landfill. Zones of material unsuitable for support of the proposed landfill will likely be encountered in old drainages. Other areas which have since been filled with soil will likely also contain materials unsuitable for support of the proposed landfill.

FIELD INVESTIGATION

The field exploration for the landfill cell was conducted on November 13, 1992. Exploratory Borings B-18 through B-21 were drilled at the locations indicated on Figure 1. Information obtained from the borings drilled from previous studies in the area are included within the report.

Exploratory borings were advanced using 4-inch diameter solid flight power auger. Samples of the subsurface materials were obtained with a 2-inch inside diameter California spoon sampler and a 1-3/8-inch inside diameter standard penetration sampler. The samplers were driven into the subsoil and bedrock with blows from a 140 pounds hammer falling 30 inches. This test is similar to the standard penetration test as described by ASTM Method D-1587. When using the California sampler, the actual measured penetration resistance values are adjusted to determine an equivalent penetration resistance, if the standard penetration sampler were to have been used (Goodman and Carroll, Theory and Practice of Foundation Engineering, McMillan Company, New York, 1968, pp 54).

Measurements were made in the exploratory borings to determine the presence of free water. Water measurements were obtained at the time of drilling and several hours after drilling. Free water was encountered in Borings B-20 and B-21 at a depth of 6 feet in both of the borings.

After conducting water level measurements, the earlier exploratory borings were backfilled using the soil and bedrock cuttings with two bags of Bentonite. Borings drilled in November, 1991 and November, 1992 were backfilled with a cement/Bentonite grout.

LABORATORY TESTING

Laboratory testing was conducted to determine the engineering characteristics of the material obtained from the exploratory borings. Laboratory testing conducted during the study include natural moisture content, dry density, percent finer than the No. 200 sieve, consolidation, unconfined compressive strength and consolidated undrained triaxial shear. Test results are shown on Figures 2 through 8. A Summary of Laboratory Test Results is shown on Table I.

SUBSURFACE CONDITIONS

Subsurface conditions encountered within the borings consist of fill and natural clay soils, overlying bedrock. Fill material encountered from 0 to 5 feet in Boring B-20 consists of unsuitable clay soil which was stockpiled in the area during the construction of previous landfill cells. The subsurface profile as determined from the other borings at the site consists of one-half to 12 feet of natural clay soil, overlying claystone/siltstone bedrock.

Material Description

1. Fill Material
   The fill material consisted of clay and silt material stockpiled in the area during the construction of previous landfill cells. The fill was sandy, slightly moist and red in color.

2. Clay
   The natural clay soil was found to be silty. Consistency was generally medium stiff to very stiff with slightly moist to wet moisture condition and red color.

3. Bedrock
   The claystone/siltstone bedrock was found to be firm to very hard. Moisture condition was slightly moist. Gravel size gypsum was observed within the bedrock materials. Color of the bedrock was primarily red with some turquoise areas.

Subsoil Characteristics

The laboratory testing conducted on samples obtained from the field investigation indicate the following conditions:

1. The unconfined compressive strength of samples of the natural soil range from 560 to 5,460 pounds per square foot. The lower strengths were obtained from Boring B-15, which is located in the east central area of the proposed landfill.

2. The unconfined compressive strength of the bedrock materials ranged from 8,050 to 36,500 pounds per square foot. These strength are consistent with those previous encountered and used in earlier investigations for landfill cells at the Lone Mountain Facility.

LANDFILL

A. Foundations

Most of the natural soils and all of the bedrock are suitable to support the proposed construction. We anticipate that unsuitable soils will be encountered in the area of old drainages and in areas where overburden soil materials have been stockpiled. All unsuitable material will need to be removed prior to construction.

B. Section

The typical embankment section for the proposed landfill consists of an interior slope of 3H:1V and exterior slope of 2.1H:1V. The embankments will be constructed by placing material above the prepared subgrade.

The soil profile used in the analysis was defined from the information obtained from the exploratory borings. The soil profile is shown in the Stability Analysis Section of the Appendix.

C. Moisture Condition

The potential for water entering the embankment will be limited to surface infiltration from the exterior portion of the embankment. The interior portion of the embankment will be covered with impervious flexible membrane liners. With these considerations, the embankments were evaluated both in the laboratory and during the stability evaluation, assuming drained conditions. The natural soil and bedrock was evaluated in their natural moisture condition.

D. Seismic Conditions

Studies conducted by Algermissen and Perkins (U.S. Geological Survey Open File Report 76-416, 1976) indicate that the horizontal acceleration (expressed as a percentage of gravity) in rock with a 90 percent probability of not being exceeded in 50 years at the Lone Mountain Facility is estimated to be approximately 0.04g.

Based on this information, a horizontal ground acceleration of 0.04g has been used to evaluate the embankment under seismic conditions.

E. Tension Cracking

With the claystone/siltstone bedrock as foundation material, the potential for tension cracking within the embankment is low. Calculations indicate that with the very stiff foundation soils, the critical height of embankment above which tension cracking would occur is greater than the proposed embankment height. Based on this information, we believe tension cracking will not influence the stability of the proposed embankment. There is, however, the potential of desiccation cracking which has been observed by others at the Lone Mountain Facility to extend 2 to 3 feet below grade. Should cracking occur, the cracking would not significantly influence the stability of the embankment.

F. Strength Parameters

Strength parameters for use in the stability analysis were determined from field and laboratory test results. Included in the Appendix is a summary of the field and laboratory test results on potential borrow and materials at the site. The testing consisted of penetration resistances, unconfined compressive strength, triaxial shear and direct shear tests. Based on these conditions, the soil strength profiles for long term conditions as previous indicated were determined.

Material	Density (pcf)	Friction Angle	Cohesion (psf)
Landfill Material	120	10°	50
Cover Material	110	28°	0
Embankment Material	120	23°	550
Soil (Clay-Silt)	125	10°	1800
Claystone-Siltstone	128	10°	5000

G. Bearing Capacity

The capacity of the foundation soils to support the proposed landfill cell was evaluated. Stability calculations which will be summarized in the next section, also model bearing capacity type failure. A bearing capacity type failure occurs if the foundation soils are not able to support the proposed construction. Typically, the bearing capacity of an embankment is evaluated by conducting stability analysis.

Classical bearing capacity calculations have been conducted to determine bearing capacity of bedrock and natural clay materials. A safety factor greater than 3 was calculated for the embankment and entire landfill placed at the site.

Attached in the Appendix is the classical bearing capacity calculations performed in regards to the proposed facility.

H. Bearing Capacity of Embankment Materials

The support above the embankment materials for construction equipment and for design of the liner system may be evaluated using a bearing capacity of 2,000 pounds per square foot. Under impact loading, a bearing capacity of 3,000 pounds per square foot may be used.

I. Bearing Capacity of Clay Liner

The support above the clay liner for construction equipment and for design on the liner system may be evaluated using a bearing capacity of 2,000 pounds per square foot. Under impact loading, a bearing capacity of 3,000 pounds per square foot may be used.

J. Bearing Capacity of Protective Cover

The support of the protective cover for construction equipment and for design of the liner system may be evaluated using an allowable bearing capacity as calculated for the following equation:

q all (psf) = 540 + 510(d) + 120(B)
Where: d = depth of embedment (ft)
B = width of loaded area (ft)

Under impact loading, the allowable bearing capacity may be increased by 50 percent. This assumes that the cover material will behave like a sandy soil.

K. Stability Calculations

The stability of the proposed embankment was analyzed under several loading conditions. Factors of safety for the embankment was determined against mass rotational and sliding wedge failures. Static and dynamic (pseudo static) analysis of the embankments were conducted using the proposed configuration as described. Strength parameters as described earlier were used in the stability analysis.

The stability of the embankment was evaluated using a computer using the Simplified Janbu Method of analysis. The computer program is entitled "STABL" which was developed by Ronald A. Seagull, Graduate Instructor in

Research, Purdue University, conducted as a joint highway research project in cooperation with Indiana State Highway Commission.

Stability calculations indicate that the embankment section for a 66 foot high embankment has a static safety factor under long term conditions of 1.8 with a dynamic safety factor of 1.6. A summary of the stability calculations is shown on Figure 5.

The stability calculations indicate that the closure cap has a 1.8 safety factor under static conditions and 1.6 under seismic.

Recommended minimum factors of safety are dependent upon the uncertainty of soil strength parameters and the cost and consequences of slope failure. The Environmental Protection Agency recommends use of a minimum static safety factor of 1.5 with a slope where the cost of repairs is comparable to the cost of construction and where there is no danger to human life or other valuable property if the slope fails with large uncertainty of soil parameters. The recommended minimum factor of safety under seismic conditions is 1.3.

The EPA also recommends the same safety factors where there is little uncertainty in the strength parameters and a high cost of repair of damage if the slope fails.

Based on the recommended minimum safety factors and the safety factors calculated, we believe the landfill will perform satisfactorily in regards to overall slope stability.

SETTLEMENT

With the proposed embankment and disposal cell, settlement will occur within the overburden soil, foundation bedrock materials and within the embankment soils. Calculations indicate the proposed embankment may experience up to 3-1/2 to 8-1/2 inches of settlement due to the consolidation of foundation material. Embankment founded on bedrock will experience less settlement than embankment founded in areas where overburden soils exist. The entire landfill is estimated to settle approximately 4-1/2 to 9 inches due to the consolidation of the foundation material. Maximum settlement will occur in the central portion of the cell, reducing down to less than one inch at the outside edge of the embankment. A large portion of the settlement will occur during initial placement of material within the embankment areas and/or within the cell.

CONSTRUCTION CONSIDERATIONS

Based on the subsurface conditions, the proposed materials for construction, and our experience with similar construction projects the following precautions should be observed during design and construction of the proposed landfill disposal cell.

1. <u>Foundation Preparation</u>

   Foundation preparation should consist of removing the excessively wet and soft soils. This material should be removed down to more competent material which would most likely consist of the bedrock materials, very stiff embankment materials or natural soils.

   Foundation preparation should also consist of stripping any vegetation and other organic or deleterious material from areas to receive fill.

2. <u>Embankment</u>

   The embankment may be constructed using the on-site materials consisting of overburden soils and/or claystone/siltstone bedrock broken down to soil size particles. The bedrock materials should be handled so as to break them down into soil size particles. Pieces of bedrock to 6 inches, surrounded by soil size particles is acceptable. All fill materials placed in the embankment should be compacted to at least 95 percent of the maximum standard Proctor density within 4 percent of the optimum moisture content. Fill should be placed in uniform lifts not more than 8 inches thick, before compaction by heavy compaction equipment. Fill compact by hand operated equipment should be placed no more than 4 inches in loose thickness.

New fill material should be benched into the existing embankments. The benching should extend at least one foot horizontal for each lift into the existing embankments.

3. Clay Liner

The clay liner should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698. The moisture content during compaction should be near or above the optimum moisture content.

A test fill should be constructed to define the construction procedure needed to obtain the required permeability of the low permeable clay liner.

To prevent cracking, positive measures should be taken to keep the surface of the clay liner material moist.

4. Material Sources

Materials for construction of the embankment and clay liner are likely available from the surrounding area. There is a potential that selective borrowing would be required to prevent placement of gypsum near the embankment surface.

5. <u>Erosion</u>

The exterior portions of the embankment should be protected to reduce erosion. Erosion on existing embankments at the site has been reduced by placing granular filters and riprap on the exterior slopes. Special care should be taken to maintain uniform compaction of exterior embankment slopes to prevent isolated areas of shallow slippage.

6. <u>Quality Control</u>

The embankment material should be continuously observed and frequently tested by a representative of the soils engineer to verify that material type, densities, moisture contents and permeability meet the project specifications.

LINER COVER MATERIALS

To protect the synthetic liner system along the interior side slopes, protective material should be placed on the floor and side slopes. The critical slippage upon which sliding could occur on the side slopes would be between the drainage net and the HDPE liner material. Assuming no stress within the drainage net, liner materials or fabric in holding up the protective cover, a safety factor of slightly less than 1 is calculated for the cover extending up a vertical distance of approximately 3 feet.

Utilizing some tension in the synthetic materials would allow placement of protective materials further up the slope. Approximately 250 pounds per linear foot is required to keep a safety factor of 1.0 for the protective cover to extend 5 vertical feet. Using the yield strength of the 80 mil. HDPE, a safety factor of 2.7 is calculated for the cover extending 5 vertical feet.

It should be expected during rainfall and snow melt that erosion will occur and may require repair of the protective material on the side slopes.

ATTACHMENT TO NOD COMMENT NO. 43-5

RAMP STABILITY

To provide access into the landfill cell, two ramps are being considered down the corners of the landfill cell. The ramp entering the cell from the south has a slope of 4.24:1 (horizontal to vertical) at the top of the tertiary liner and a slope distance of approximately 184 feet. The ramp entering the cell from the west has a slope of 4.24:1 (horizontal to vertical) at the top of the tertiary liner and a slope distance of approximately 167 feet.

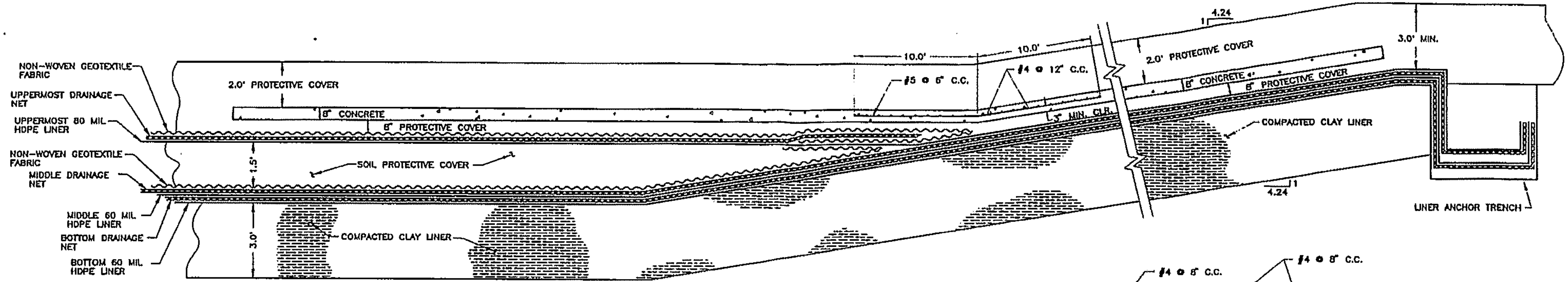
A. <u>South Ramp</u>

We anticipate that the ramp will have a 20 foot wide traffic surface and will be approximately 3 feet thick. We anticipate that an 8-inch thick reinforced lean mix concrete support material may be used within the ramp.

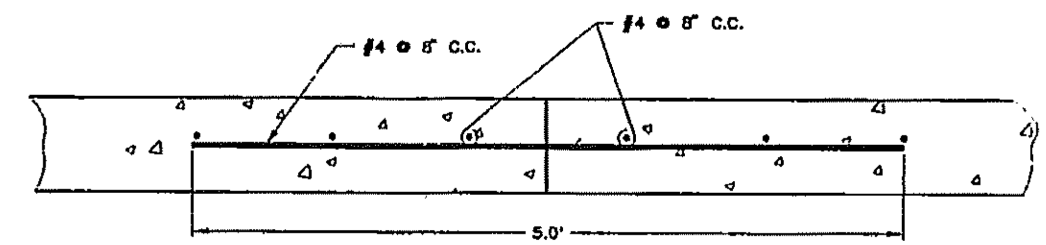
With the 20 foot wide surface and a similar design configuration as used in Cells 12 and 13, the ramp may be designed with the 8-inch thick, 23 foot wide, reinforced lean mix concrete. The lean mix concrete slab on the cell floor should be constructed with a length of approximately 65 feet and an end width of approximately 98 feet. It is important that the base lean mix concrete slab, overlying protective cover material, and protective cover in front of the lean mix concrete base be placed to provide the lateral support prior to construction of the ramp. To maintain suitable safety factors on the tension of the liner system, we recommend that the ramp continue to be constructed with sections no longer than 45 feet long.

B. <u>West Ramp</u>

The west ramp may be constructed using similar procedures as the south ramp or if placement of waste can be used to construct the west ramp, the waste should be placed at a slope of approximately 8:1 (horizontal to vertical). This would provide a safety factor of at least 1.3. Waste materials must be conveyed onto the floor and the ramp built from bottom up.



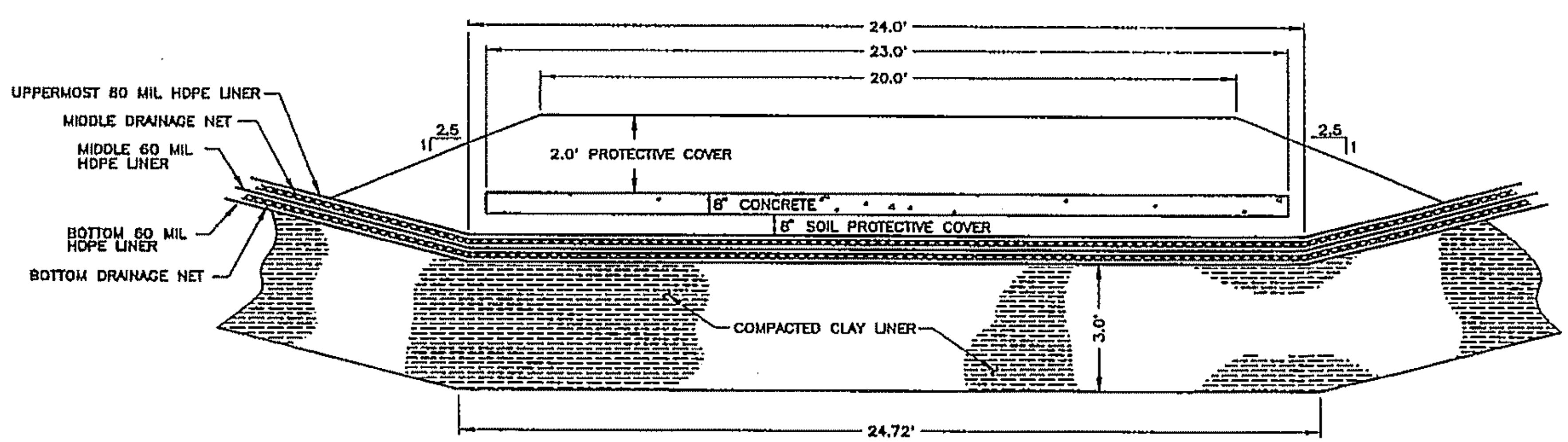
SECTION 14
N.T.S.



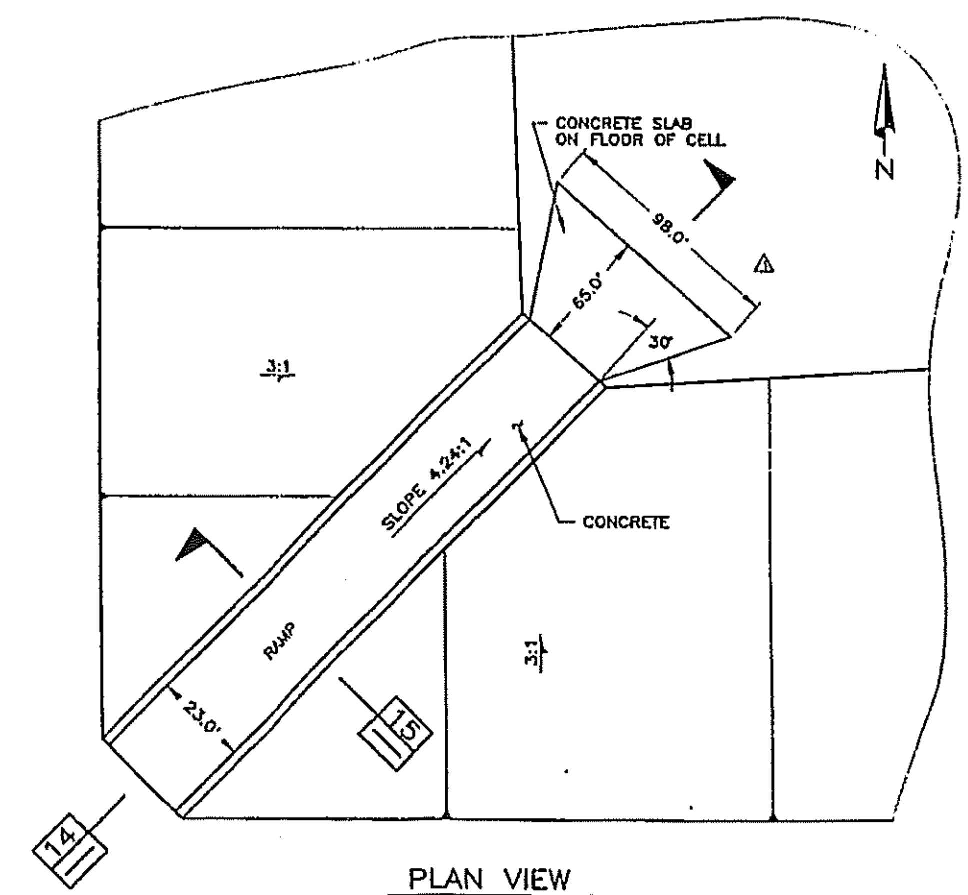
TYPICAL CONSTRUCTION JOINT
N.T.S.

SPECIFICATIONS FOR PLACEMENT OF RAMP

1. THE RAMP, INCLUDING THE PROTECTIVE COVER AND THE CONCRETE SHALL BE CONSTRUCTED FROM THE BOTTOM OF THE SLOPE TO THE TOP. THE 8-INCH SOIL PROTECTIVE COVER (UNDERLYING THE CONCRETE), THE CONCRETE SLAB, AND THE 2-FOOT COVER OVER THE CONCRETE SHALL BE CONSTRUCTED ON THE FLOOR OF THE CELL (AT THE BOTTOM OF THE SLOPE) PRIOR TO THE CONSTRUCTION OF THE RAMP.
2. THE 8-INCH PROTECTIVE COVER, THE 8-INCH CONCRETE SLAB, AND THE 2-FOOT PROTECTIVE COVER SHALL BE CONSTRUCTED IN SECTIONS NOT EXCEEDING 45 FEET IN LENGTH AS MEASURED ALONG THE SLOPE OF THE RAMP. THE CONCRETE MATERIAL SHALL ACHIEVE A MINIMUM STRENGTH OF 200 PSI, BEFORE COMMENCING CONSTRUCTION OF THE OVERLAYING 2-FOOT SOIL PROTECTIVE COVER, AND THE 8-INCH SOIL PROTECTIVE COVER AND CONCRETE LAYER FOR THE NEXT SECTION OF THE RAMP.
3. THE CONCRETE SHALL HAVE A MINIMUM 28 DAY STRENGTH OF 4000 PSI.
4. THE SURFACE OF THE CONCRETE SHALL BE LEFT IN A ROUGHENED CONDITION. THE SURFACE SHALL BE SCREEDED ONLY.
5. THE SECTIONS OF THE CONCRETE SHALL BE FORMED SO AS TO PROVIDE A CONSTRUCTION JOINT THAT IS PERPENDICULAR TO THE SLOPE OF THE RAMP BETWEEN SECTIONS AS THE RAMP IS CONSTRUCTED UP THE SLOPE.



SECTION 15
N.T.S.



PLAN VIEW
N.T.S.

HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City, Utah	DESIGNED MPW	3		
		DRAFTED JSJ	2		
		CHECKED KCS	1	6/8/94	MODIFIED RAMP DIMENSIONS — RGA — MEA
	PROJECT ENGINEER	DATE JUNE, 1993	NO.	DATE	REVISIONS — BY — APVD.

SCALE AS SHOWN

VERIFY SCALE — BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

USPCI — A Subsidiary of Union Pacific Corporation

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
INTERIOR RAMP DETAILS PHASE I

REVISION 0

SHEET NO. 28 OF 34

64-44-200

RAMP STABILITY

To provide access into the landfill cell, two ramps are being considered down the corners of the landfill cell. The ramp entering the cell from the south has a slope of 4.24:1 (horizontal to vertical) at the top of the tertiary liner and a slope distance of approximately 184 feet. The ramp entering the cell from the west has a slope of 4.24:1 (horizontal to vertical) at the top of the tertiary liner and a slope distance of approximately 167 feet.

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We anticipate that the ramp will have a 20 foot wide traffic surface and will be approximately 3 feet thick. We anticipate that an 8-inch thick reinforced lean mix concrete support material may be used within the ramp.

With the 20 foot wide surface and a similar design configuration as used in Cells 12 and 13, the ramp may be designed with the 8-inch thick, 23 foot wide, reinforced lean mix concrete. The base lean mix concrete slab should be constructed approximately 65 feet long with an end width of approximately 98 feet. It is important that the base lean mix concrete slab, overlying protective cover material, and protective cover in front of the lean mix concrete base be placed to provide the lateral support prior to construction of the ramp. To maintain suitable safety factors on the tension of the liner system, we recommend that the ramp continue to be constructed with sections no longer than 45 feet long.

B. West Ramp

The west ramp may be constructed using similar procedures as the south ramp or if placement of waste can be used to construct the west ramp, the waste should be placed at a slope of approximately 8:1 (horizontal to vertical). This would provide a safety factor of at least 1.3. Waste materials must be conveyed onto the floor and the ramp built from bottom up.

WASTE STABILITY

We understand that the landfill cells will be operated by disposing of waste on one side of the cell and then continuing to place waste up to the design finished grade. Once the finished grade is achieved over a certain area, the filled area will be closed with clay, synthetic materials and protective cover material.

A. Waste/Synthetic Liners/Clay Interface Stability

To maintain stability of the synthetic liner/waste system, the waste should be placed a horizontal distance of at least 5 times the height of the waste. The height of the waste is measured from the top of the tertiary liner to the top of the waste. The horizontal distance is measured on top of the waste from the waste-embankment slope contact to the edge (crest) of the waste. This criteria applies to all open faces of the waste. Once this criteria has been met along the long axis of the landfill cell, the criteria would only apply to waste extending from the side slopes.

This is an extremely important aspect of the landfill operation due to the fact that the materials on the floor and sidewalls of the cells have very low resistance to sliding. Placement of waste outside of this criteria may result in sliding of the synthetic materials and may possibly damage the protective layers.

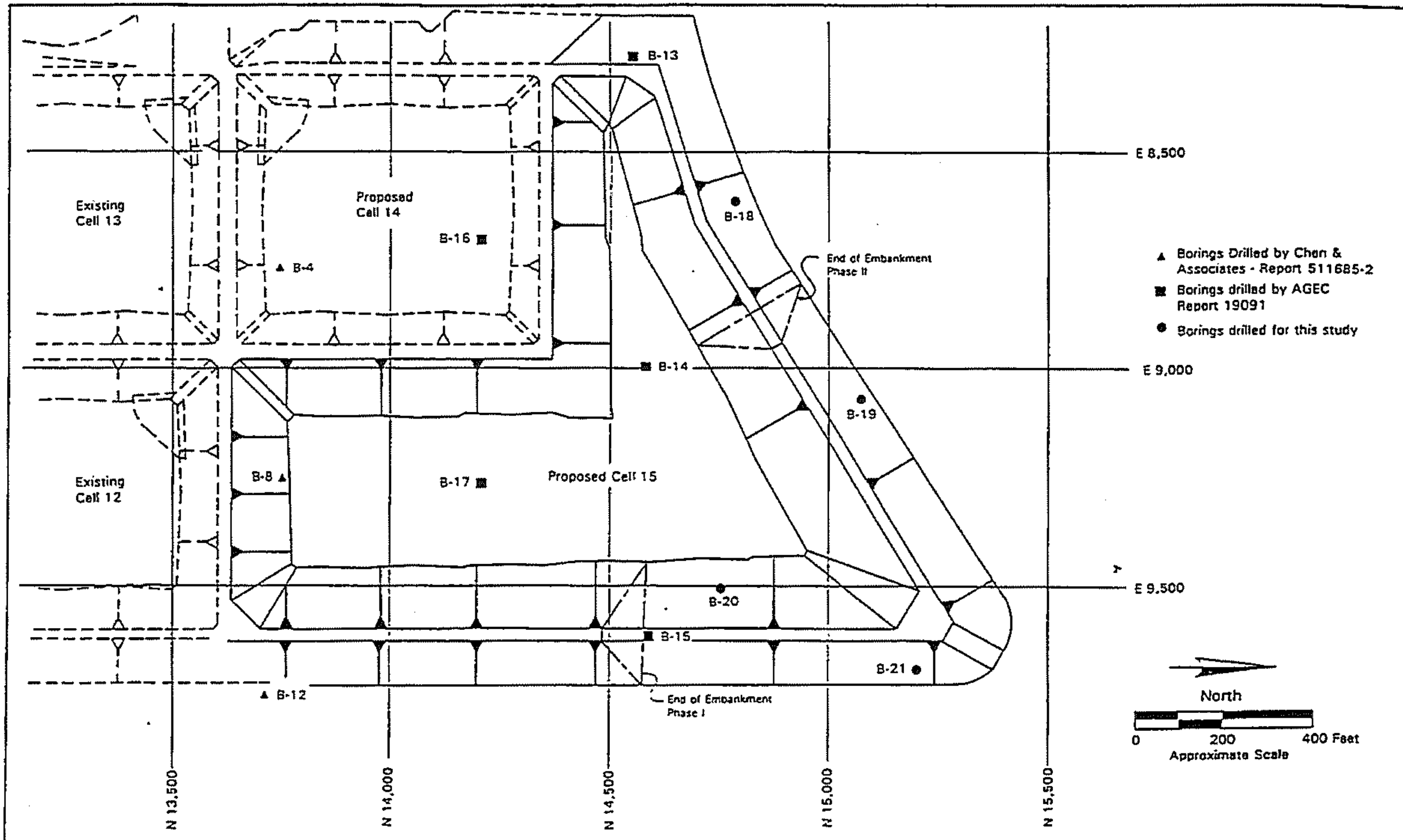
A safety factor of 1.5 has been calculated for this waste placement configuration with a phreatic surface located 1 foot above the drainage media at the bottom of the waste extending from the embankment top down the interior embankment slope and across the cell floor to the end of the waste. This water condition is not anticipated to occur during operation of the landfill, however this condition has been evaluated to determine if water would result in unacceptable performance of the waste disposal system.

B. Waste Stability

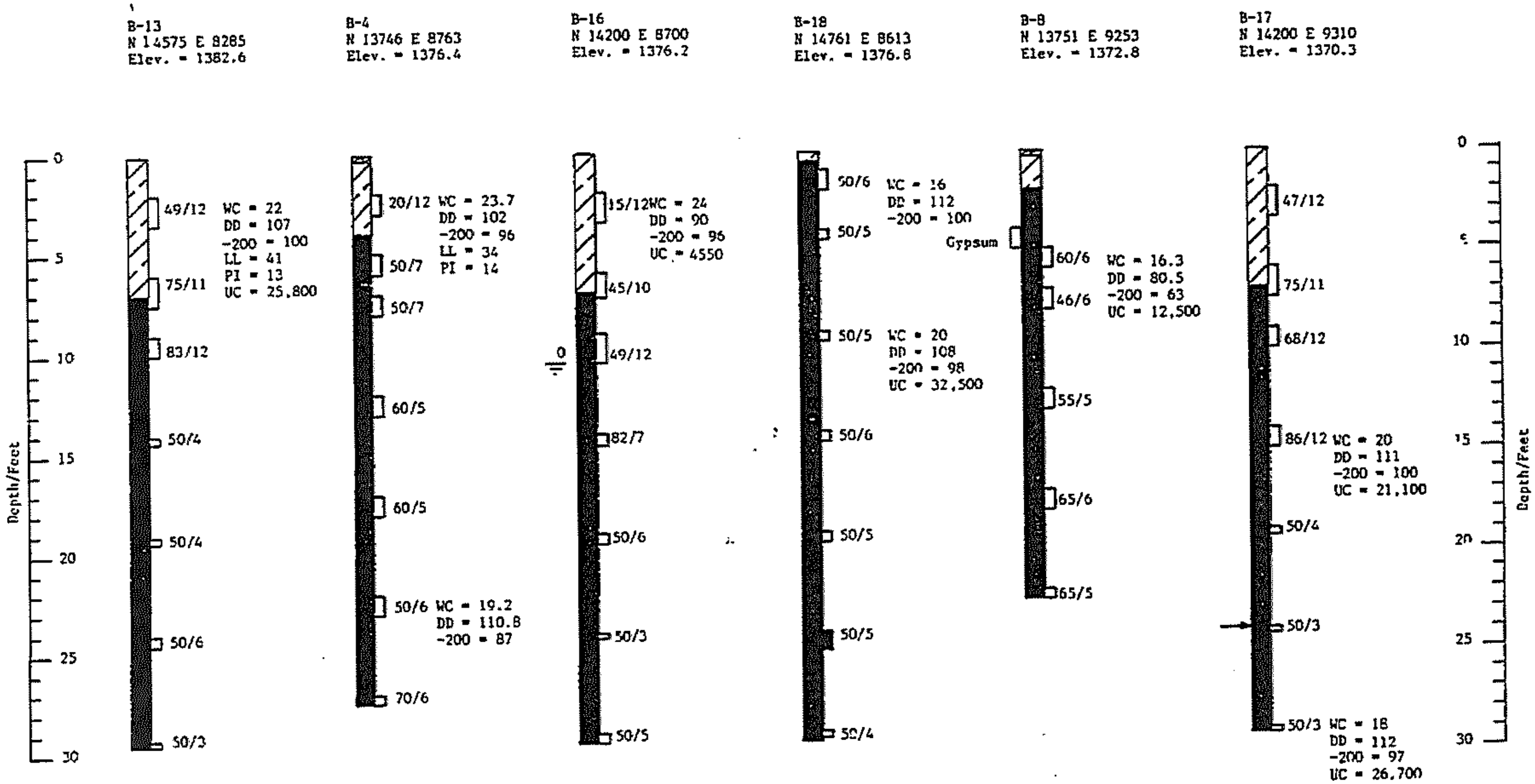
Slippages in the waste itself are very difficult if not impossible to evaluate due to the unknown characteristics and non-uniformity of the waste material. Stability analyses conducted using strength parameters that would apply to relatively weak soils indicate that slopes constructed on the order of 3 (horizontal) to 1 (vertical) are anticipated to be stable.

Safety factors of 1.3 are obtained with a friction angle of 23.7 degrees with no cohesion or with approximately 650 pounds per square foot cohesion with no friction. Using typical strength parameters that would apply for a highly plastic clay (cohesion of 79 psf and a friction of 20 degrees) would provide a safety factor of approximately 1.3.

Stability calculations are presented in Appendix F.

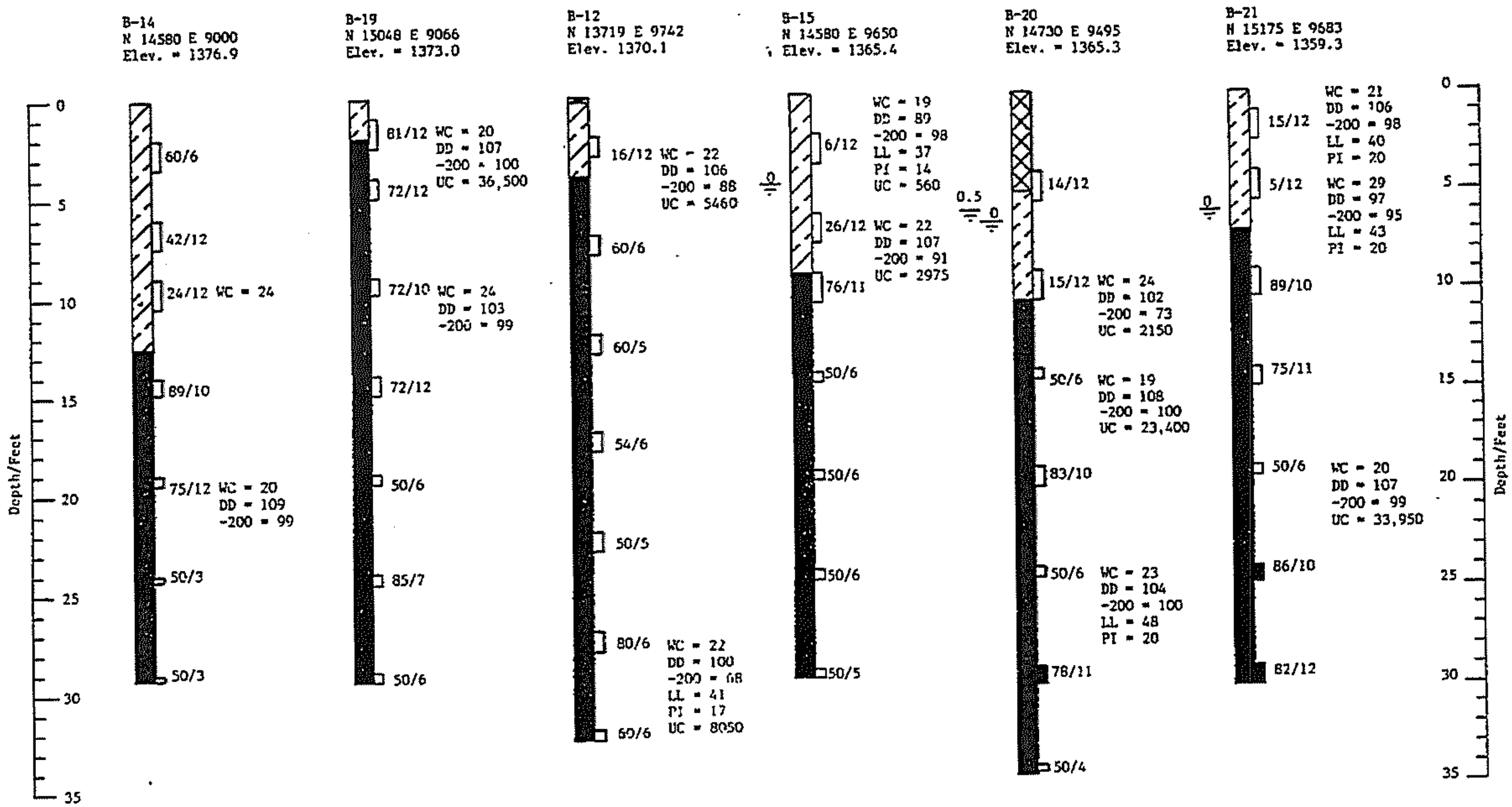


24292	AGEC	Location of Exploratory Borings	Figure 1



See Figure 4 for Legend and Notes

24292	AGEC	Logs of Exploratory Borings	Figure 2



See Figure 4 for Legend and Notes

24292	AGEC	Logs of Exploratory Borings	Figure 3

NOTES:

1. Listed below are the dates that the borings were drilled and the report in which they first were reported.

Borings	Date Drilled	Report
B-4, B-8, B-12	1/29/85 - 2/2/85	Chen & Associates 511685-2
B-13 through B-17	11/4/91 - 11/5/91	AGEC 19091
B-18 through B-21	11/13/92	This Report

2. Locations of exploratory borings were survey located by others.
3. Elevations of exploratory borings were surveyed by others.
4. The exploratory boring locations and elevations should be considered accurate only to the degree implied by the method used.
5. The lines between the materials shown on the boring logs represent the approximate boundaries between material types and the transitions may be gradual.
6. Water level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.
7. WC = Water Content (%);
DD = Dry Density (pcf);
-200 = Percent Passing No. 200 Sieve;
LL = Liquid Limit (%);
PI = Plasticity Index (%);
UC = Unconfined Compressive Strength (psf).

LEGEND:

Fill; clay and silt, sandy, slightly moist, red.

Topsoil; clay, silty, dry to moist, red.

Clay (CL); silty, stiff, moist to wet, red.

Clay and Silt (CL-ML); medium stiff to very stiff, moist to wet, red.

Claystone/Siltstone; firm to very hard, slightly moist, gravel sized gypsum, red and turquoise.

10/12 California Drive Sample. The symbol 10/12 indicates that 10 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 12 inches.

Standard Drive Sample.

Indicates depth to free water surface and number of days after drilling that measurement was taken.

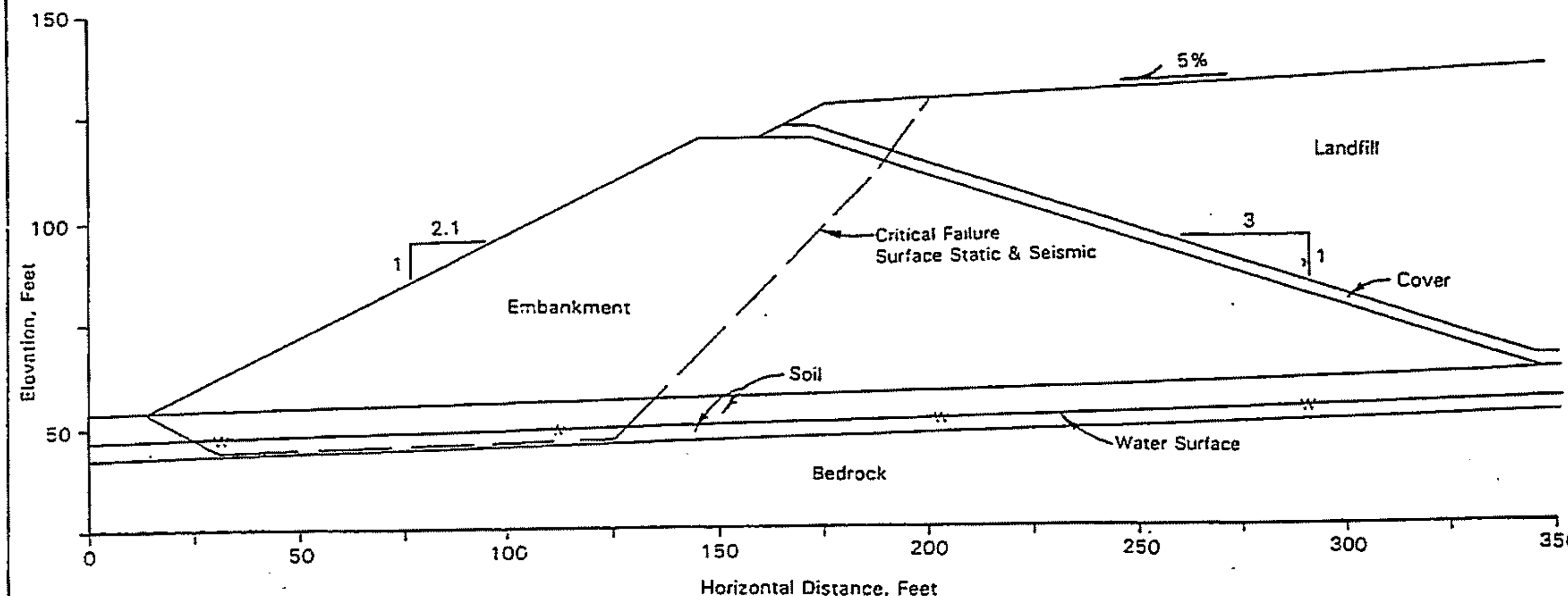
Indicates depth at which boring caved.

24292	AGEC	Legend and Notes of Exploratory Borings	Figure 4

Long Term Safety Factors

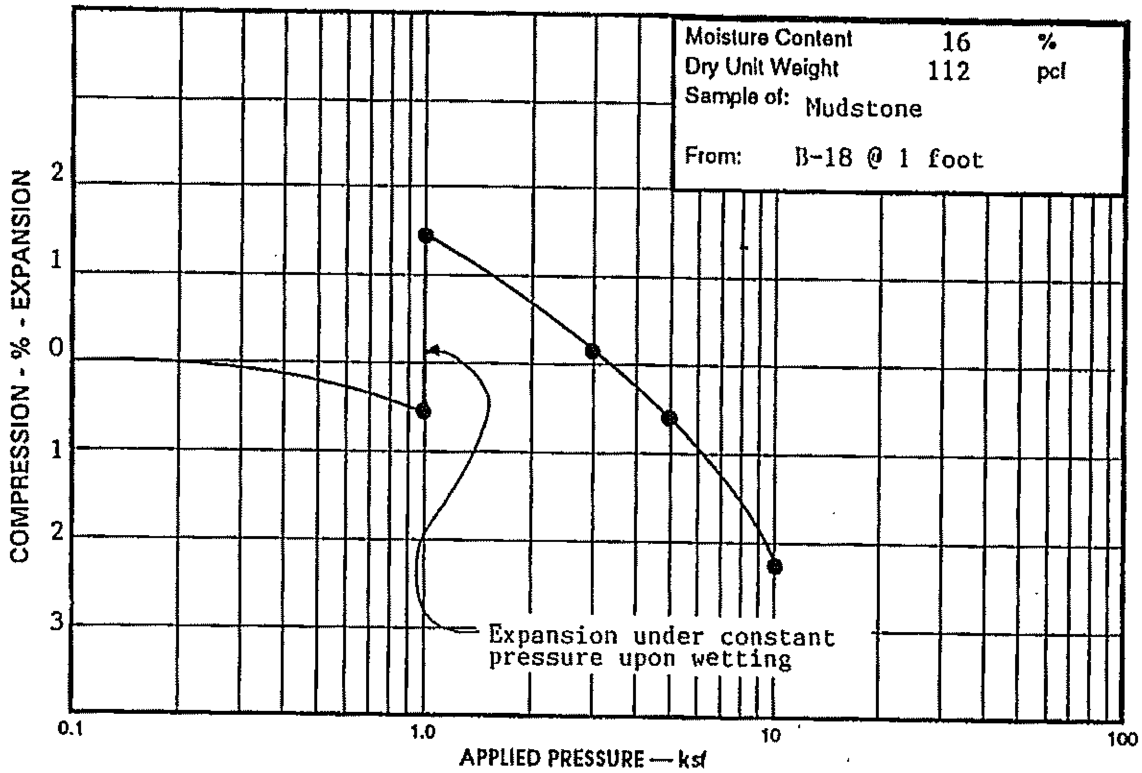
1.8 Static
1.6 Seismic - 0.04g acceleration

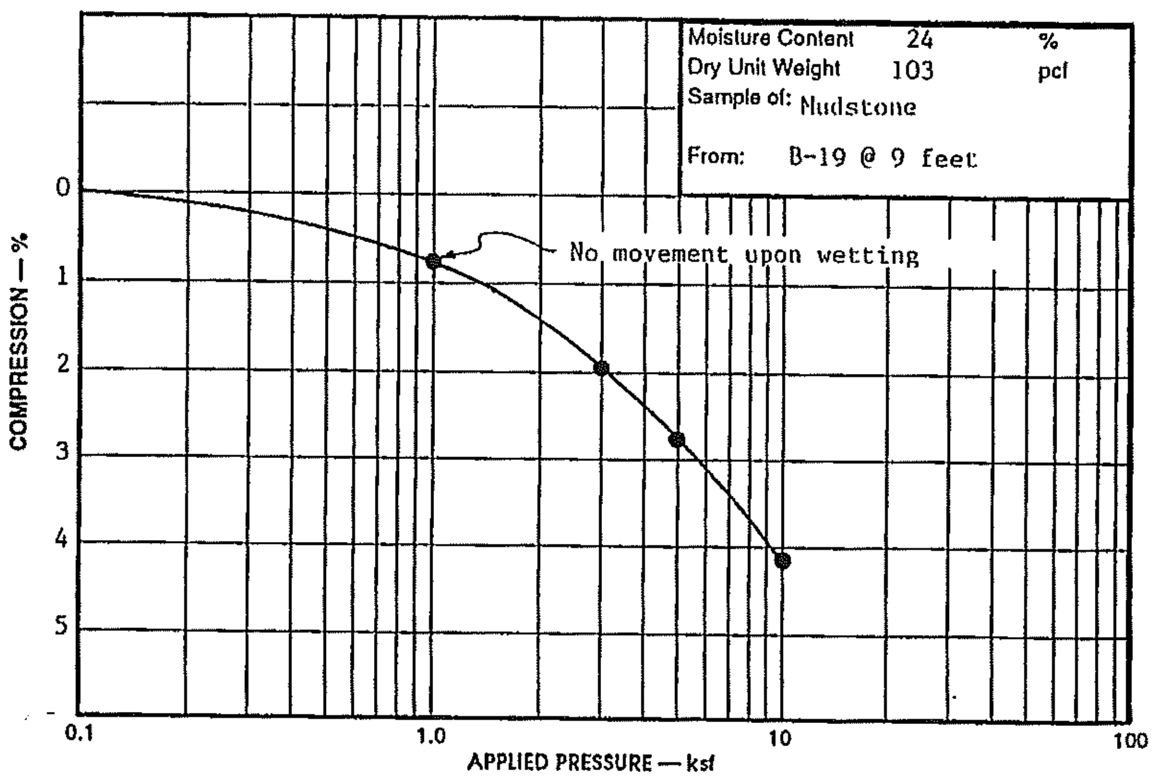
SOIL PARAMETERS			
Material	Density, pcf	ϕ, degrees	c, psf
Landfill	120	10	50
Cover	110	28	0
Embankment & Liner	120	23	550
Soil (CL-ML)	125	10	1800
Bedrock	128	10	5000



24293	AGEC	STABILITY ANALYSIS SUMMARY	FIGURE 5

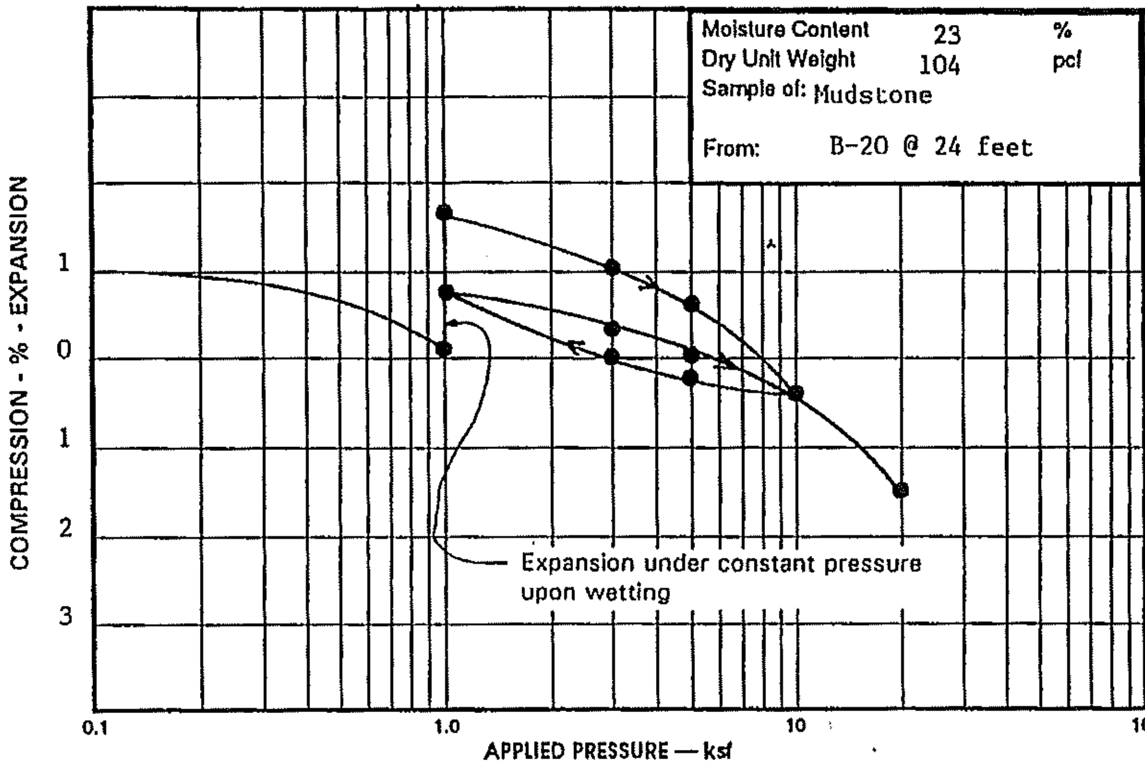
Applied Geotechnical Engineering Consultants, Inc.

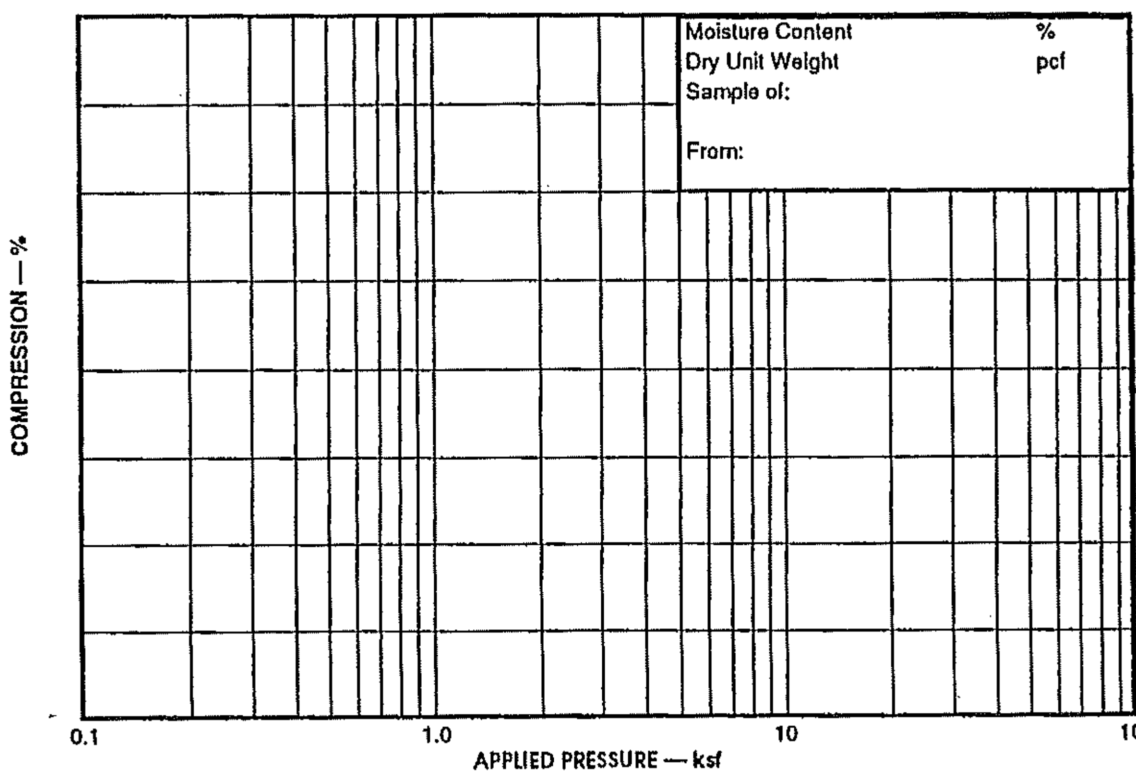




Project No. 24292 **CONSOLIDATION TEST RESULTS** Figure 6

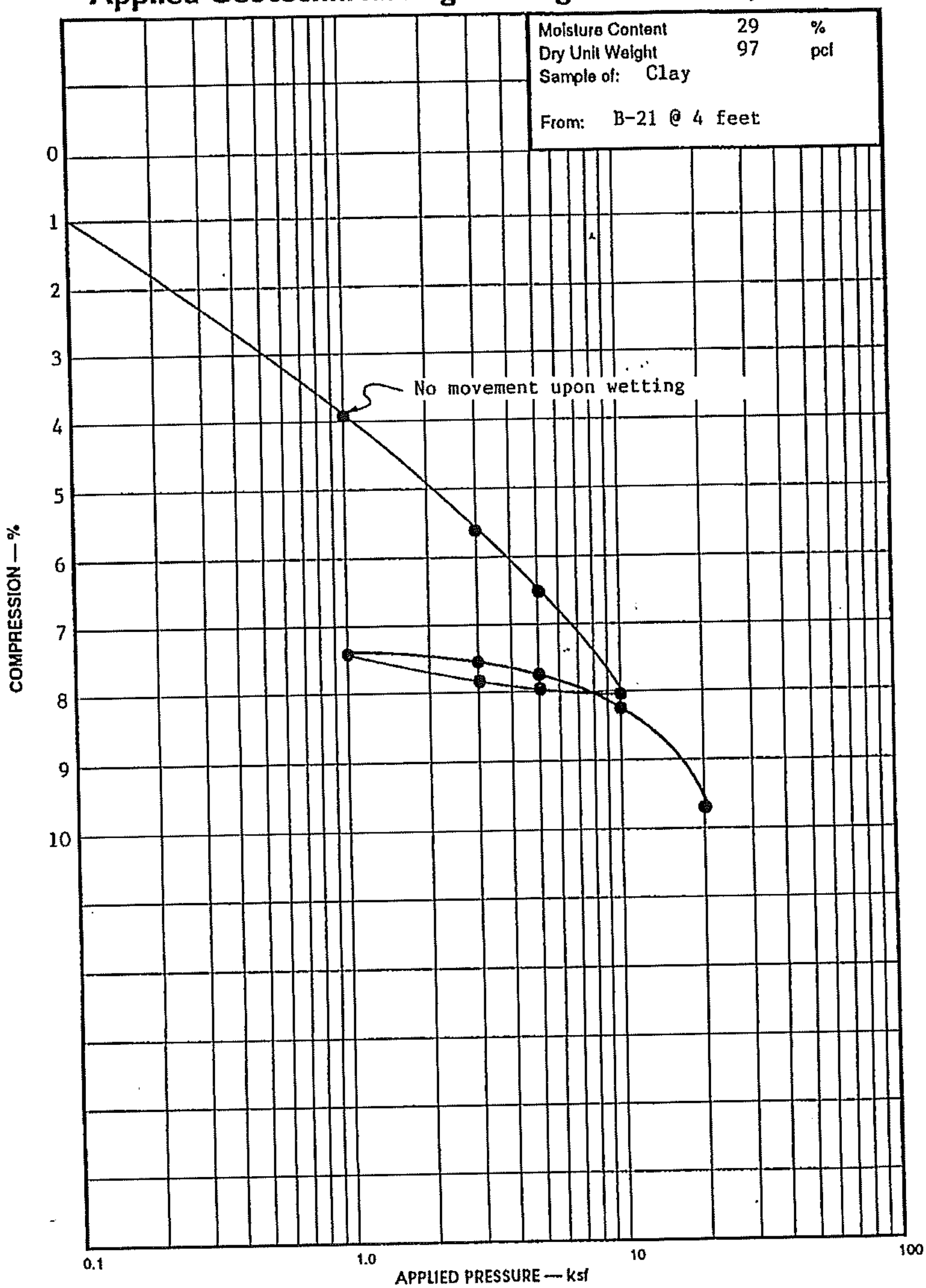
Applied Geotechnical Engineering Consultants, Inc.



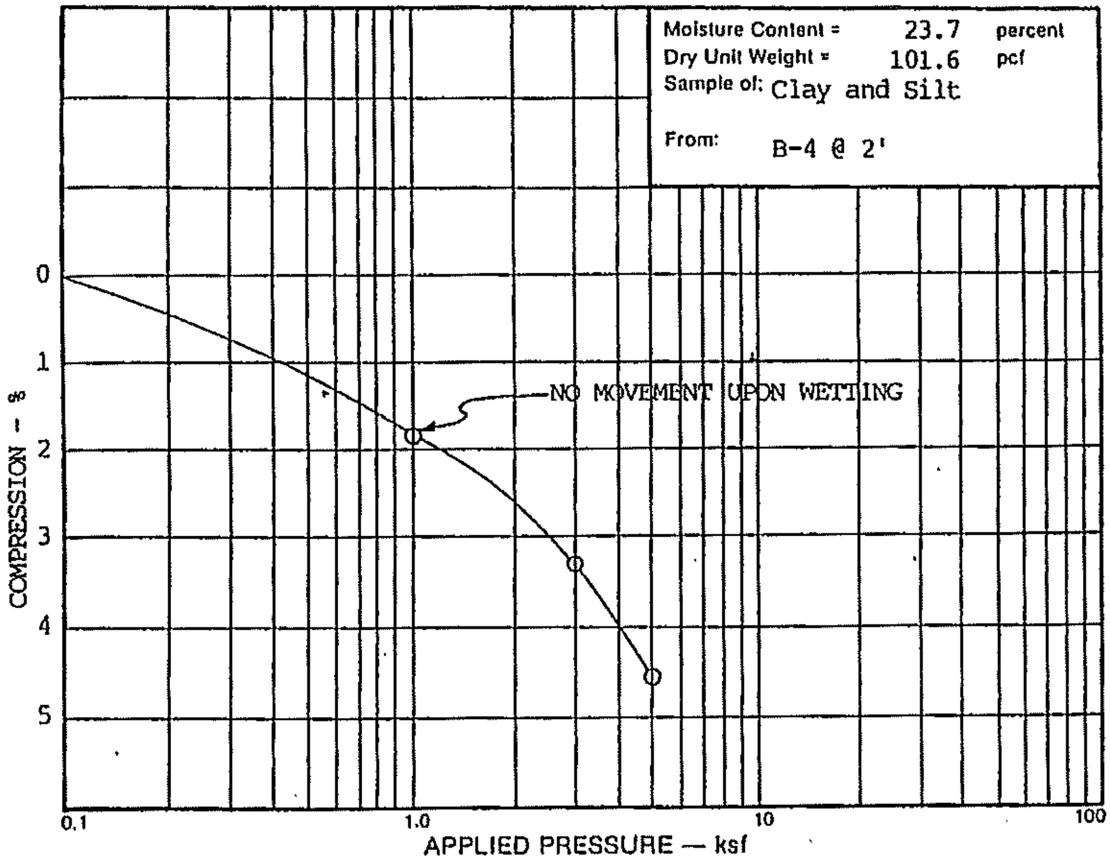


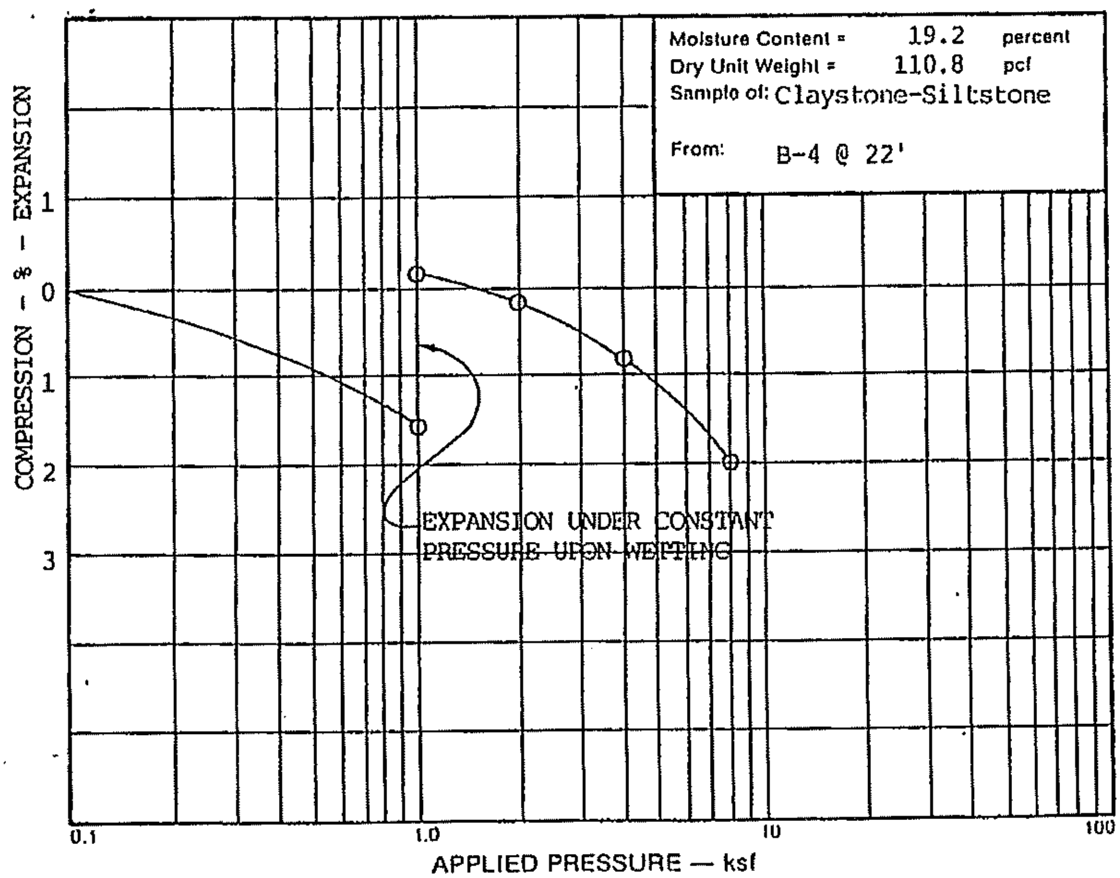
Project No. 24292 **CONSOLIDATION TEST RESULTS** Figure 7

Applied Geotechnical Engineering Consultants, Inc.



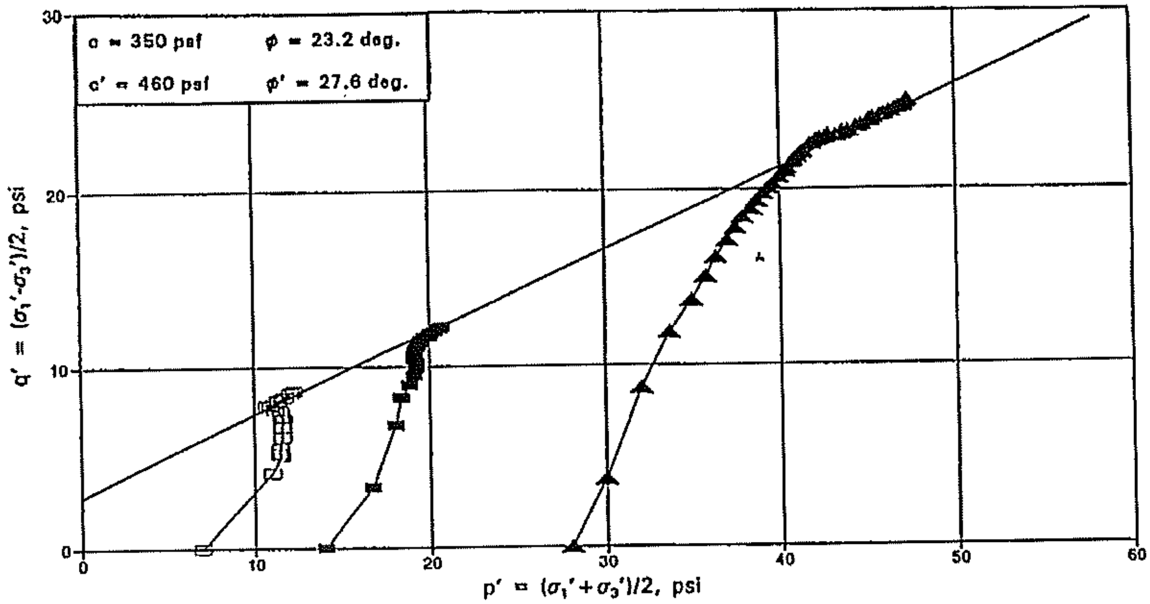
Project No. 24292 **CONSOLIDATION TEST RESULTS** Figure 8

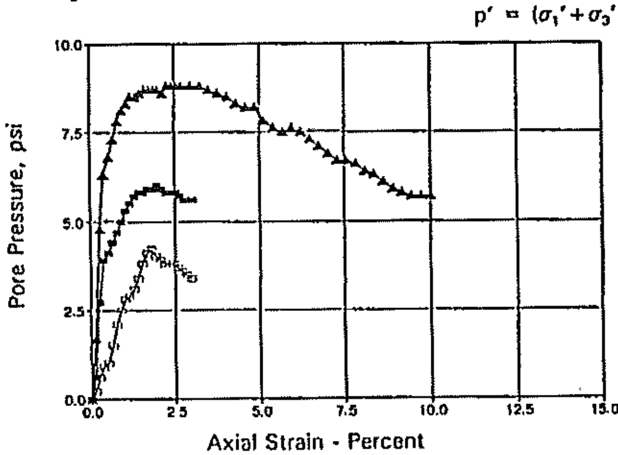


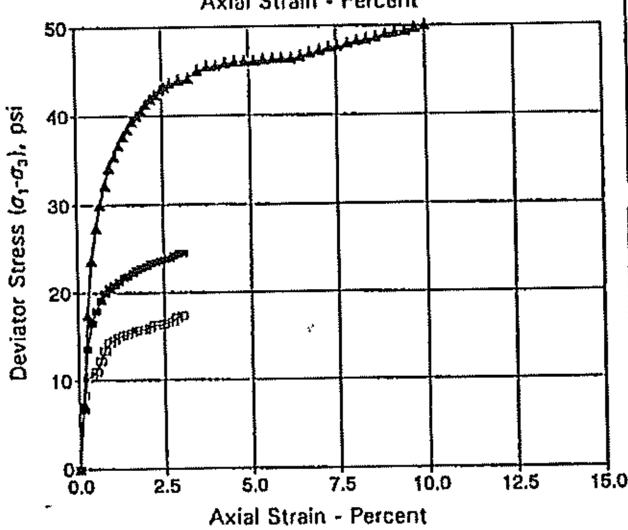


#511685-2 SWELL-CONSOLIDATION TEST RESULTS Fig. 9

Applied Geotechnical Engineering Consultants, Inc.







Test No.(Symbol)	1(□)	2(■)	3(▲)
Sample Type	undisturbed		
Length, in.	3.85	3.77	3.70
Diameter, in.	1.93	1.95	1.97
Dry Density, pcf	106.4		
Moisture Content, %	21.0		
Consol. Pressure, psi	7.0	14.0	28.0
"B" Parameter	1.00	1.00	1.00
Total Conf. Stress(σ_3), psi	7.0	14.0	28.0
Total Axial Stress(σ_1), psi	24.3	38.5	72.0
Deviator Stress(σ_1-σ_3), psi	17.3	24.5	44.0
Eff. Lateral Stress(σ_3'),	3.6	8.4	19.2
Eff. Axial Stress(σ_1'), psi	20.9	32.9	63.2
Pore Pressure(u), psi	3.4	5.6	8.8
Strain(ϵ), %	3.0	6.0	9.0
Remarks	Test Type: (CIU) Consolidated Undrained with pore pressure measurements. Sample was back pressure saturated.		

Sample Index Properties	
Natural Dry Density, pcf	106.4
Natural Moisture Content, %	21.0
Liquid Limit, %	40
Plasticity Index, %	20
Percent Gravel	-
Percent Sand	-
Percent Passing No. 200 Sieve	98

Sample Description: Red, Lean Clay (CL)

From: Boring B-21 @ 1 foot

Project No.24292

TRIAXIAL COMPRESSION TEST RESULTS

Figure 10

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

PROJECT NUMBER 24292

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION			ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	SAMPLE CLASSIFICATION
BORING	DEPTH (FEET)			GRAVEL (%)	SAND (%)	SILT/ CLAY (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
B-4	2	23.7	102			96	34	14		Lean Clay
	22	19.2	110.8			87				Mudstone
B-8	5	16.3	80.5			63			12,500	Mudstone
B-12	2	22	106			88			5,460	Clay-Silt
	27	22	100			68	41	17	8,050	Mudstone
B-13	2	22	107			100	41	13	25,800	Clay
B-14	9	24								Clay-Silt
	19	20	109			99				Mudstone
B-15	2	19	80			98	37	14	560	Clay-Silt
	6	22	107			91			2,975	Clay-Silt
B-16	2	24	90			96			4,550	Clay-Silt
B-17	14	20	111			100			21,100	Mudstone

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

PROJECT NUMBER 242[illegible]

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION			ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	SAMPLE CLASSIFICATION
BORING	DEPTH (FEET)			GRAVEL (%)	SAND (%)	SILT/ CLAY (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
B-18	1	16	112			100				Mudstone
	9	20	108			98			32,500	Mudstone
B-19	1	20	107			100			36,500	Mudstone
	9	24	103			99				Mudstone
B-20	9	24	102			73			2,150	Clay-Silt
	14	19	108			100			23,400	Mudstone
	24	23	104			100	48	20		Mudstone
B-21	1	21	106			98	40	20		Clay
	4	29	97			95	43	20		Clay
	19	20	107			99			33,950	Mudstone

ATTACHMENT TO NOD COMMENT NO. 49-11

APPENDIX A

SOIL PARAMETERS

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/8/93 BY JRM
SHEET 1 OF 1
SUBJECT Design Configuration

PROFILE:

Embankments

- 3:1 interior slopes
- 2.15:1 exterior slopes (gravel) 2.1:1 exterior slopes (embankment)
- 28 foot wide crest
- 1354 to 1380 natural ground surface elevation
- 1420 embankment crest elevation
- 1381 to 1384 floor elevation at top of uppermost liner
- 24 foot wide interior ramps with 4.24:1 (H:V) slopes
- maximum embankment height = 66 feet

Closure 1420 to 1441 elevation, feet

Floor from top down

- 2' Cover
- Fabric
- Drainage Net
- 80 mil HDPE - uppermost liner
- 2' Cover
- Fabric
- Drainage Net
- 60 mil HDPE - middle liner
- Drainage Net
- 60 mil HDPE - bottom liner
- 3 feet clay

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/11/93 BY JRM
SUBJECT Soil Strength Parameters SHEET 1 OF 4

REMOLDED CLAYSTONE

Compaction Test Results

Hole	Depth (ft)	Max. Dry Density (pcf)	Optimum Moisture (%)
15	6-10	94.2	28.2
22	0-5	102.8	18.6
23	7-11	111.8	16.7
24	12-16	109.2	19.3

Average Maximum Dry Density = 104.5 pcf
Average Optimum Moisture = 20.7%
95% of Average Total Density = 119.8 pcf use : 120 pcf

Strength Test Results

Triaxial Compression Test
Cu (consolidated-undrained)

	ϕ	c
Effective Stress	23.5°	100 psf
Total Stress	13.7°	140 psf

Direct Shear Test (cu) $\phi = 6°$ c = 1140 psf

Soil Classification: LL = 29-48% CL - ML
PI = 10-21%
-200 = 86-100%

From NAVFAC DM - 7.02 '86 pg. 39
Cohesion (as compacted) = 1350 psf
Cohesion (saturated) = 460 psf
ϕ' (effective) = 32°

Patton & Hendron
PI = 10-21%
ϕ residual = 13.5° - 24° min PI
10.5° - 17.5° max PI

NAVFAC DM - 7 Fig. 3.7 Based on PI

PI = 10% $\phi_r = 26°$ $\phi' = 31° - 42°$
PI = 21% $\phi_r = 22°$ $\phi' = 28° - 34°$

End of Construction
use $\phi = 0$ c = 1100 psf or $\phi = 13°$ c = 140 psf

Long Term
use $\phi = 23.5°$ c = 100 psf

Upper Clay

Average total unit weight = 124.2 pcf

Laboratory Test Results

Uncontinued = 5460, 560, 2975, 2150, 4550, 25,800 psf

c = 2730, 280, 1487, 1075, 2225, 12,900 psf
excluding c = 280 → unsuitable material to be removed
also excluding c = 12,900 → not typical value
$c_{ave} = 1886$ psf

Field Test Results - Penetration Resistavce

N = 49, 20, 15, 47, 60, 42, 24, 16, 6, 26, 15, 15, 5

N_{corr} for sample size (California Sample)

N = 40, 16, 12, 38, 49, 34, 20, 13, 5, 21, 12, 12, 4

Correlation Terz. & Peck or Sowers

q_{ult} = 0.075 to 0.133 N (TSF)

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/11/93 BY JRM
SUBJECT Soil Strength Parameters SHEET 3 OF 4

q_{ult} (TSF)

N	0.075 N	0.133 N
40	3.0	5.3
16	1.2	2.1
12	0.9	1.6
38	2.9	5.0
49	3.7	6.5
34	2.6	4.5
20	1.5	2.7
13	1.0	1.7
5	0.4	0.7
21	1.6	2.8
12	0.9	1.6
12	0.9	1.6
4	0.3	0.5

q_{ult} = 600 - 13,000 psf
c = 300 to 6,500 psf
conservative values

Triaxial Compression Test
Cu

	ϕ	c
Effective Stress	27.6°	460 psf
Total Stress	23.2°	350 psf

End of Construction
Use $\phi = 0°$ c = 1800 psf

Long Term
Use $\phi = 10°$ c = 1800 psf

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/11/93 BY JRM
SUBJECT Soil Strength Parameters SHEET 4 OF 4

Claystone/Siltstone

Average Density

$\gamma = 126.0$ pcf
$\gamma_{cell\ 14} = 126.5$ pcf
$\gamma_{cell\ 10} = 128.3$ pcf
$\gamma_{average} = 126.9$ pcf

Laboratory Strength Testing

Boring	Depth (ft)	C (unconfined) psf
B-18	9	16,250
B-19	1	18,250
B-20	14	11,700
B-21	19	16,957
B-8	5	6,250
B-17	14	10,550
B-17	29	13,350
B-12	27	4,025

Penetration Resistance 49/12 and higher

N_{corr} for sampler = 40

using sowers $q_u = 0.075\,N$ (TSF)

$= (.075)(40) = 3$ TSF

$$c = \frac{3(2000)}{2} = 3{,}000\ psf$$

using Terzagi and Peck $q_u = (0.133)(40) = 5.3$ TSF

$$c = \frac{5.3(2000)}{2} \approx 5300\ psf$$

use C = 5,000 psf with $\phi = 0$

APPENDIX B

TENSION CRACK POTENTIAL

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/11/93 BY JRM
SUBJECT Tension Crack Potential SHEET 1 OF 2

H_T = Height of Embankment when cracking will begin

$$H_T = N_T \frac{S_u}{\gamma_E}$$

S_u = undrained strength of foundation
γ_E = unit weight of embankment
γ_E = 120 pcf

Soil Foundation
γ = 124.2 pcf
S_u = 3,600 psf

Bedrock Foundation
γ = 126.9 pcf
S_u = 4,000 psf

$$N_T = f\left(\frac{K_E}{K_F}, \frac{W}{D}\right)$$ *see DM - 7*

$$\frac{W}{D} = \frac{280 \ feet}{0 - 4 \ feet} = > 70$$

$$\frac{K_E}{K_F} = \frac{Emb.modulus}{Fnd.modulus} = \frac{30 - 1000}{71000} = < 1$$

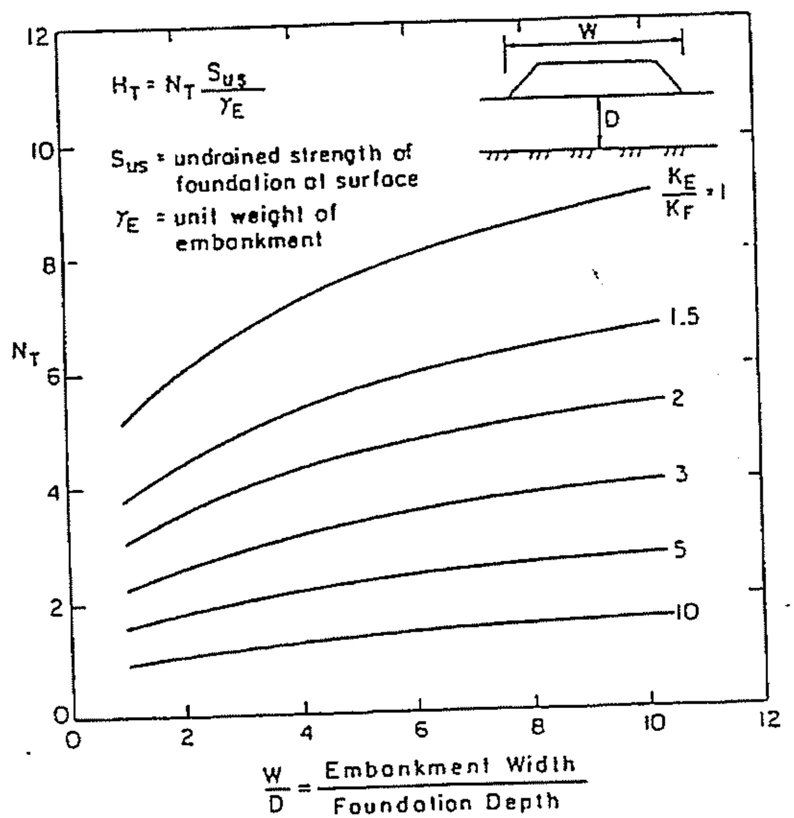
chart only goes $\frac{W}{D} = 10 \qquad \frac{K_E}{K_F} = 1$

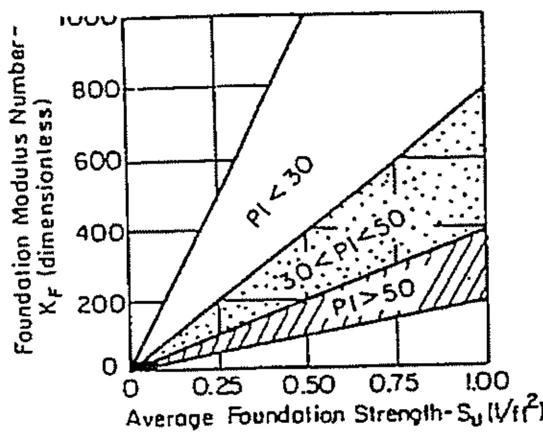
from chart $N_T > 9$

$$H_T = \frac{9(3600)}{120} = 270 \, feet$$

potential for tension cracking is low
at 66' high embankment.

2/2





Typical values of K_E for compacted fills

Unified Class.	Compaction Water Content		
	Optimum - 3%	Optimum	Optimum + 3%
GC	300 - 1200	200-500	75 - 300
SP	400 - 1000	400-1000	400-1000
SM	300 - 750	300-750	300-750
SC	250 - 1000	150-600	50-250
ML	250 - 1000	150-600	50-250
CL	250 - 1000	100-400	30-200
CH	100 - 400	50-200	20-100

Values shown apply to fill materials compacted to dry densities from 90% to 95% of the Std. AASHO maximum. In general, the value of K_E increases with increasing dry density at a given water content.

Fig 4 CHART FOR ESTIMATING H_T = HEIGHT OF EMBANKMENT WHEN CRACKING WILL BEGIN. (after Chirapuntu and Duncan, 1975) An Engineering manual for Slope Stability Studies by Duncan and Buchignani, 1975, Univ. of CA Berkeley

APPENDIX C

BEARING CAPACITY

Foundation Material Parameters

Soil γ = 124.2 pcf
c = 1,800 psf

Bedrock γ = 126.9 pcf
c > 5,000 psf

Embankment and Cell Parameters

Height = 66'

Anticipated Cap height = 81'

σ = 81(120) = 9,720 psf (cell)
σ = 66(120) = 7,920 psf (embankment)

Embankment width = 320 feet

Inside crest to inside crest = 1540' x 620'

Bearing Capacity

$q_{ult} = CN_cS_cd_c + qN_qS_qd_q$

D = 0, B = 320', when L = 1540', 620' length connects to Cell 14

B/L = 0.2078 emb. $\frac{B}{L} = \frac{1540}{2000} = 0.77$

$\phi = 0$, $N_c = 5.14$, $S_c = 1 + 0.2$ $d_c = 1 + 0.2$

$q' = \gamma P$ $N_q = 1$, $S_q = 1$, $d_q = 1$

Embankment where D = 0

q_{ult} = (c) 5.14 [1 + (0.2)(0.21)] = c (5.36)

= 1,800(5.36) = 9648 psf on soil SF = 9648/7920 = 1.2

= > 5,000(5.36) = 26,800 psf on bedrock SF = 26800/7920 = 3.4 ok

ok on bedrock - on clay the SF is not good enough - look at layering effect

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/2/93 BY JRM
SUBJECT Bearing Capacity SHEET 2 OF 3

Cell

$$q_{ult} = c(5.14)[\ 1 + 0.2(0.77)\ ] = c(5.93)$$

$$\doteq 1800(5.93) = 10674$$

$$SF = \frac{10674}{9720} = 1.1 \text{ on soil - NG - Investigate layering effect}$$

$$= > 5000(5.93) = > 29{,}650$$

$$SF = \frac{29650}{9720} = 3.1 \text{ on bedrock} \qquad OK$$

Bearing Capacity on two-layered systems (Bowles p. 211)

C_1 = 1800 psf C_2 = 5000 psf

d_1 = depth upper layer = 8 ft

$$H = 0.5B \tan(45 + \quad)$$

$$= 0.5(320) \tan(45 + \quad) = 191$$

$$c' = \frac{c_1 d_1 + (H - d_1)c_2}{H}$$

$$= \frac{(1800)(8) + (191 - 8)(5000)}{191} = 4866$$

$$q_{ult} = c(5.14)[\ 1 + 0.2(0.21)\ ] = c(5.36)$$

$$= (4866)(5.36) = 26{,}082 \text{ psf}$$

$$SF = \frac{26{,}082}{7920} = 3.3$$

Bearing Capacity is OK - using the two layered system analysis.

Two layered System for cell

$$q_{ult} = c\ (5.93)$$

$$= (4866)\ (5.93) = 28{,}885 \text{ psf}$$

$$SF = \frac{28855}{9720} = 3.0 \qquad OK$$

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/14/93 BY JRM
SUBJECT Clay liner & Intermediate Soil layer bearing cap SHEET 1 OF 1

Clay liner and Embankment

For conservative approach, use the bearing capacity of the clay for the embankment materials too. The liner materials will be lower strength material.

Laboratory Test Results

Unconfined Compression tests

-remolded to 105.3 pcf @ 23.5% moisture

UC = 2820 & 2870 psf

c = 1400 psf

Undrained Bearing Dapacity

$q_{ult} = cN_c$

$= (5.14)\ (1400) = 7196$ psf

w/SF = 3 $q_{all} = 2399$ psf use 2,000 psf

for temporary loading SF = 2 $q_{all} = 3{,}598$ psf use 3,000 psf

Intermediate Soil

No laboratory results
material will not be compacted
will likely be dry and granular

Assume it will behave as a granular soil

$\phi = 25°$ $c = 50$ psf

$q_{ult} = 1.3cN_c + qNq + 0.3\gamma BN_\gamma$

$= 1.3(50)(25.1) + q(12.7) + 0.3\gamma B9.7$

$= 1631.5 + 12.7q + 349.2B$

q = thickness (d) x 120 pcf

for SF = 3 $q_{all} = 540 + 510d + 120B$

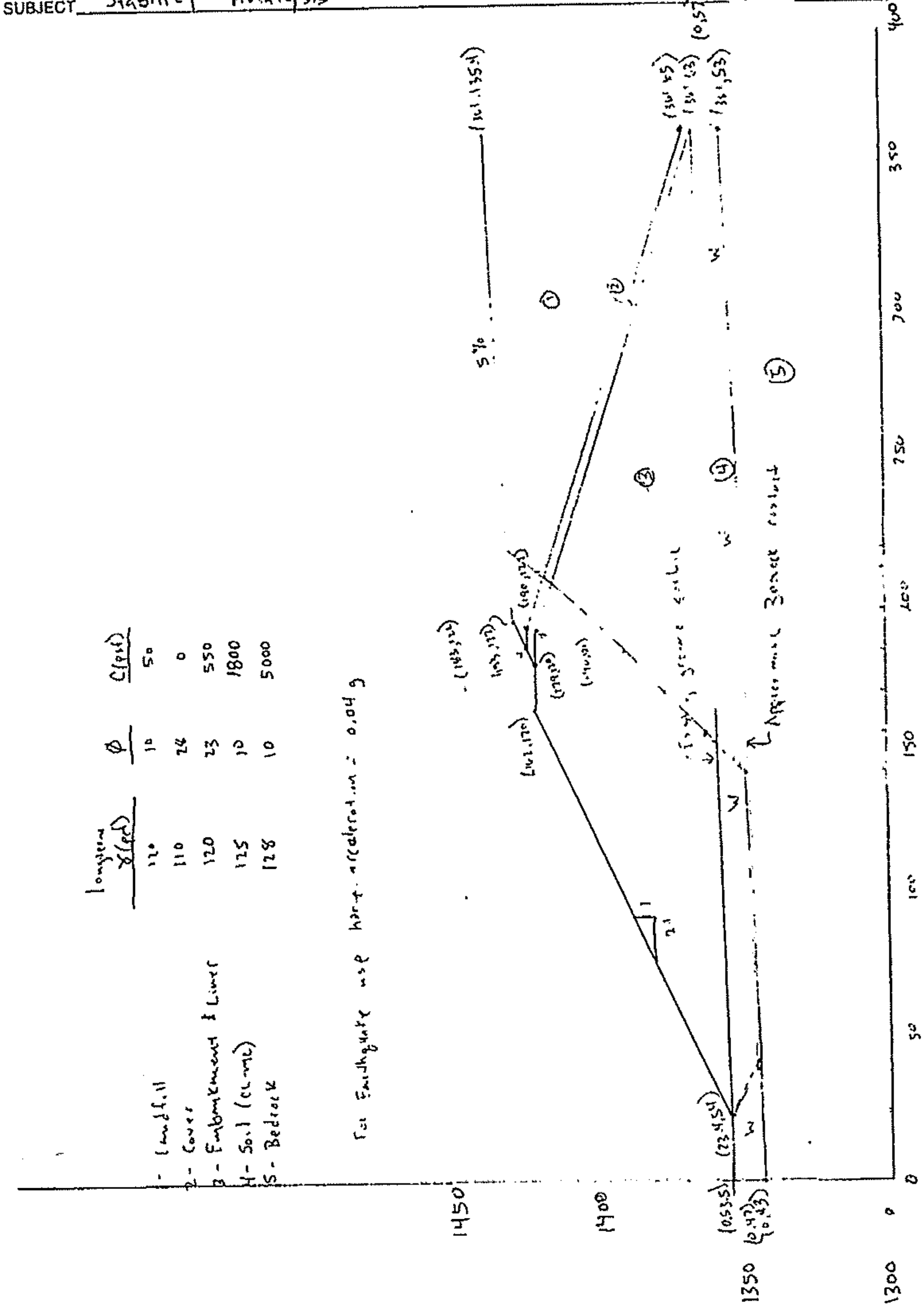
APPENDIX D

STABILITY ANALYSIS

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 24293 TITLE Landfill Cell 15 DATE 3/30/93 BY JRM

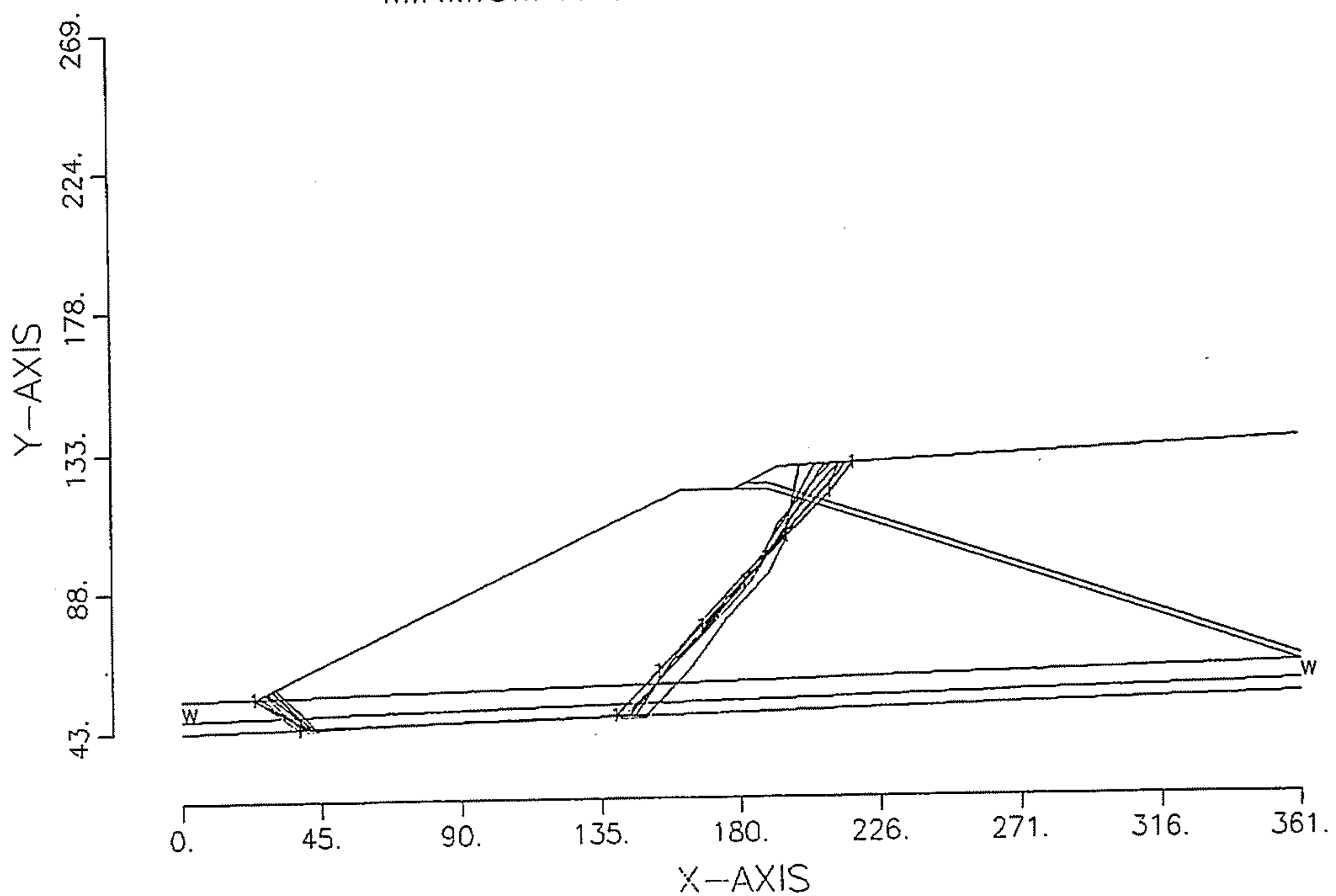
SUBJECT Stability Analysis SHEET 1 OF 7



AG
Midvale UT s/n5206

Cell 15 Long Term – Static

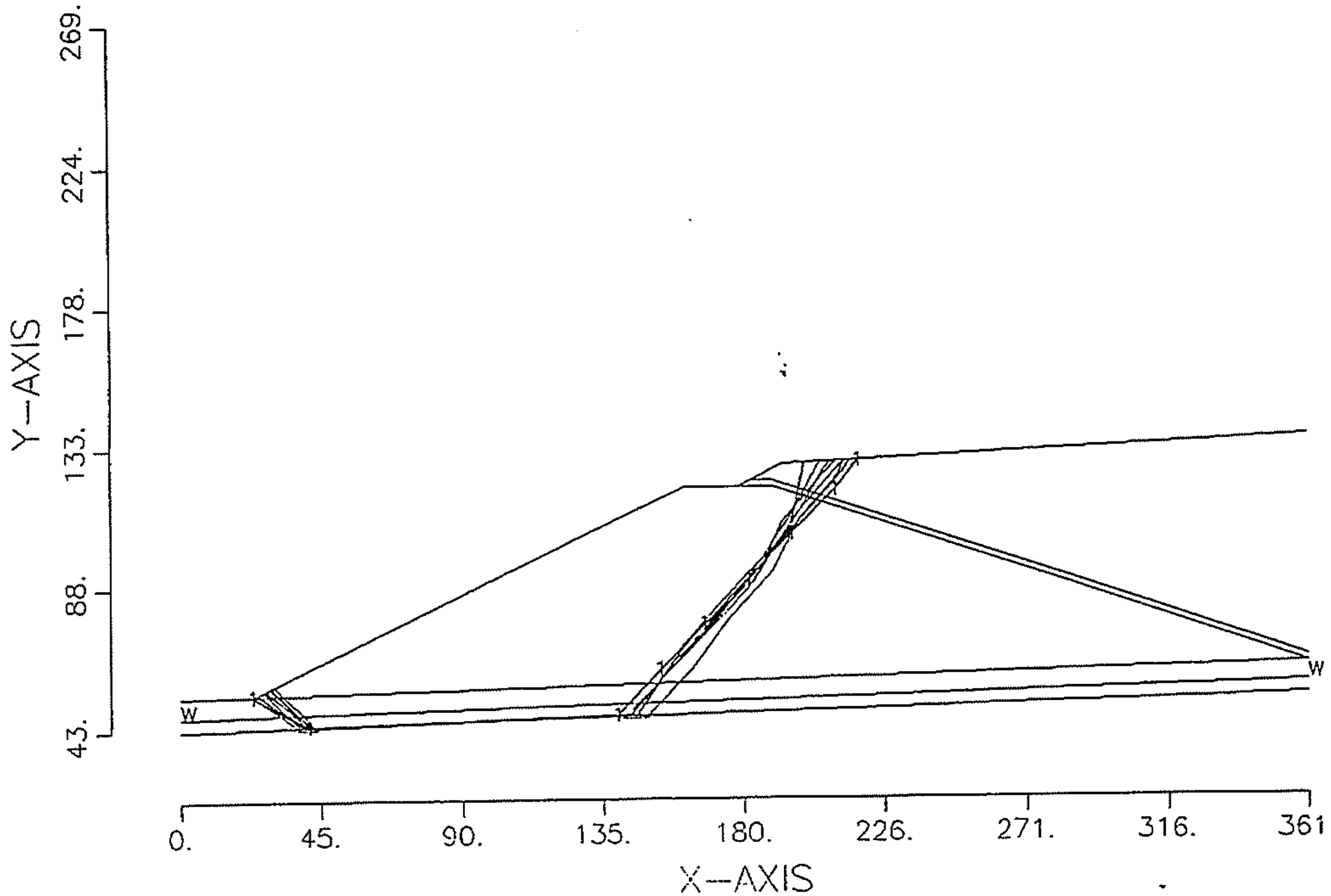
1000SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.766



AGE
Midvale UT s/n5206

Cell 15 Long Term – 0.04g Earthquake

1000SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.593



3 of 7

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION Cell 15 Long Term - Static

BOUNDARY COORDINATES

6 TOP BOUNDARIES
12 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SOIL TYPE BELOW BND
1	.00	53.50	23.40	54.00	4
2	23.40	54.00	162.00	120.00	3
3	162.00	120.00	179.00	120.00	3
4	179.00	120.00	183.00	122.00	2
5	183.00	122.00	193.00	127.00	1
6	193.00	127.00	361.00	135.40	1
7	183.00	122.00	190.00	122.00	2
8	190.00	122.00	361.00	65.00	2
9	179.00	120.00	190.00	120.00	3
10	190.00	120.00	361.00	63.00	3
11	23.40	54.00	361.00	63.00	4
12	.00	43.00	361.00	53.00	5

ISOTROPIC SOIL PARAMETERS

5 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT.	SATURATED UNIT WT.	COHESION INTERCEPT	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT	PIEZOMETRIC SURFACE NO.
1	120.0	120.0	50.0	10.0	.00	.0	1
2	110.0	110.0	.0	28.0	.00	.0	1
3	120.0	120.0	550.0	23.0	.00	.0	1
4	125.0	125.0	1800.0	10.0	.00	.0	1
5	128.0	128.0	5000.0	10.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER	Y-WATER
1	.00	47.00
2	361.00	57.00

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING SLIDING BLOCK SURFACES, HAS BEEN SPECIFIED.

1000 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 20.0

BOX NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	WIDTH
1	38.00	44.00	44.00	44.00	1.00
2	140.00	47.00	150.00	47.00	1.00

* * SAFETY FACTORS ARE CALCULATED BY THE MODIFIED JANBU METHOD * *

FAILURE SURFACE # 1 SPECIFIED BY 9 COORDINATE POINTS

SAFETY FACTOR = 1.766

POINT NO.	X-SURF	Y-SURF	ALPHA (DEG)
1	23.48	54.04	-34.31
2	38.03	44.11	1.83
3	140.00	47.37	45.44
4	154.04	61.62	45.37
5	168.09	75.86	45.59
6	182.08	90.14	46.32
7	195.90	104.61	45.79
8	209.84	118.94	51.11
9	217.32	128.22	

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION   Cell 15 Long Term - 0.04g Earthquake

BOUNDARY COORDINATES

6 TOP BOUNDARIES
12 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SOIL TYPE BELOW BND
1	.00	53.50	23.40	54.00	4
2	23.40	54.00	162.00	120.00	3
3	162.00	120.00	179.00	120.00	3
4	179.00	120.00	183.00	122.00	2
5	183.00	122.00	193.00	127.00	1
6	193.00	127.00	361.00	135.40	1
7	183.00	122.00	190.00	122.00	2
8	190.00	122.00	361.00	65.00	2
9	179.00	120.00	190.00	120.00	3
10	190.00	120.00	361.00	63.00	3
11	23.40	54.00	361.00	63.00	4
12	.00	43.00	361.00	53.00	5

ISOTROPIC SOIL PARAMETERS

5 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT.	SATURATED UNIT WT.	COHESION INTERCEPT	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT	PIEZOMETRIC SURFACE NO.
1	120.0	120.0	50.0	10.0	.00	.0	1
2	110.0	110.0	.0	28.0	.00	.0	1
3	120.0	120.0	550.0	23.0	.00	.0	1
4	125.0	125.0	1800.0	10.0	.00	.0	1
5	128.0	128.0	5000.0	10.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER	Y-WATER
1	.00	47.00
2	361.00	57.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .040 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE = .0

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING SLIDING BLOCK SURFACES, HAS BEEN SPECIFIED.

1000 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 20.0

BOX NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	WIDTH
1	38.00	44.00	44.00	44.00	1.00
2	140.00	47.00	150.00	47.00	1.00

FAILURE SURFACE # 1 SPECIFIED BY 10 COORDINATE POINTS

SAFETY FACTOR = 1.593

POINT NO.	X-SURF	Y-SURF	ALPHA (DEG)
1	23.24	54.00	-29.61
2	23.77	53.70	-27.55
3	41.50	44.45	1.72
4	140.30	47.41	46.69
5	154.02	61.97	45.20
6	168.11	76.16	45.14
7	182.22	90.34	45.35
8	196.27	104.57	45.66
9	210.25	118.87	52.27
10	217.49	128.22	

APPENDIX E

ENTRY RAMP STABILITY

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/2/93 BY JRM
SUBJECT Ramp Stability SHEET 1 OF 4

South Ramp

Configuration

top of uppermost liner:
24' wide ramp
slope 4.24:1 = 13.27°
Length = 188.82 ft. (along slope)
Elevation difference 1420 to 1376.68 ft.
= 43.32 ft.
Horizontal length = 183.78 ft.

Ramp Section:
2' Cover
8" Lean Mix Concrete
8" Protective Cover
80 mil HDPE
XL-14 - drainage net

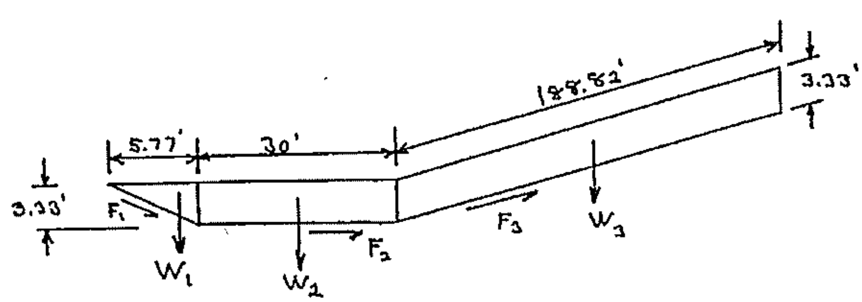
Friction/Cohersion of materials
Cover $\phi = 30°$
Lean Mix c = 500 psi
Between HDPE and XL-14 $\phi = 9°$
Between soil cover and soil cement $\phi = 30°$

Analysis

1) Stability of Protective material over Lean Mix Concrete.

$$SF = \frac{\tan 30°}{\tan 13.27°} = 2.45 \qquad OK$$

2) Stability between Net and HDPE (2 dimensional)



PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/2/93 BY JRM
SUBJECT Ramp Stability SHEET 2 OF 4

Weights

$W_1 = (3.33)(5.77)(1/2)(100) = 961$ plf

$W_2 = (30)(3.33)(100) = 9{,}990$ plf

$W_3 = (188.82)(3.33)(100) = 62{,}877$ plf

Driving Forces

$= (62{,}877)(\sin 13.27°) - (961)(\sin 30°) = 13{,}952$ lb/ft

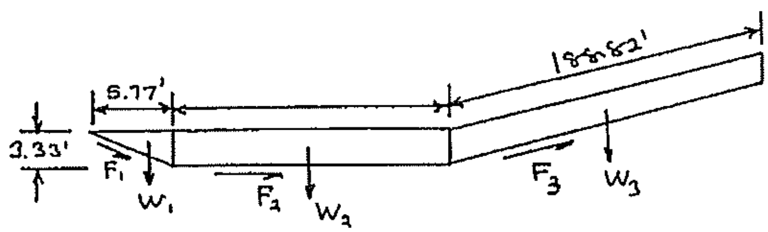
Resisting Forces

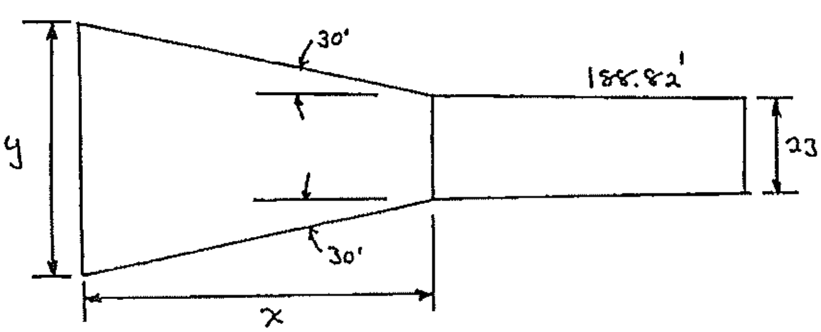
$= (62{,}877)(\cos 13.27°)(\tan 9°) + (9990)\tan 9°$

$+ (961)(\cos 30°)(\tan 30°) = 11{,}756$ lb/ft

$SF = \dfrac{11756}{13952} = 0.84$ *NG*

Must try 3-D analysis





PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/2/93 BY JRM

SUBJECT Ramp Stability SHEET 3 OF 4

$$y = 23 + 2(x \tan 30°) = 23 + 1.155x$$

Weights

$$W_1 = (3.33)(5.77)(1/2)(100)[\ 23 + 1.155x\ ] = (22096.2 + 1109.6x)\ lb$$

$$W_2 = (3.33)(100)(x) \left(\frac{23 + 1.155x + 23}{2}\right) = (7659x = 192.3x^2)\ lb$$

$$W_3 = (188.82)(3.33)(100)[\ 20 + (2)(3.5)\ ] = 1{,}697{,}680\ lb$$

Driving Forces

$$= (1{,}697{,}680)\sin 13.27° - (22096.2 + 1109.6x)\sin 30°$$

$$= 378{,}637.6 - 554.8x$$

Resisting Forces

$$= (22096.2 + 1109.6x)(\cos 30°)(\tan 30°) + (7659x + 192.3x^2)(\tan 9°)$$

$$+ 1{,}697{,}680(\cos 13.27°)(\tan 9°)$$

$$= 272{,}754.7 + 1{,}767.9x + 30.5x^2$$

$$FS = \frac{272754.7 + 1767.9x + 30.5x^2}{378637.6 - 554.8x^2} = 1.5$$

Solving for x

$x = 64.6$ ft

use $x = 65$ ft

$y = 98$ ft

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/2/93 BY JRM

SUBJECT Ramp Stability SHEET 4 OF 4

Check Stability

$$W_1 = (3.33)(5.77)(1/2)(100)[23 + (1.155)(65)] = 94{,}221 \text{ lb}$$

$$W_2 = (3.33)(100)(65)\ \frac{23 + 1.155(65) + 23}{2} = 1{,}310{,}334\ lb$$

$$W_3 = (188.82)(3.33)(100)[20 + (2)(3.5)] = 1{,}697{,}680 \text{ lb}$$

Driving Forces

$$= (1{,}697{,}680)(\sin 13.27°) - (94{,}221)(\sin 30°) = 342{,}575 \text{ lb}$$

Resisting Forces

$$= (94{,}221)(\cos 30°)(\tan 30°) + (1{,}310{,}334)(\tan 9°) + (1{,}697{,}680)(\cos 13.27°)(\tan 9°)$$

$$= 516{,}354 \text{ lb}$$

$$FS = \frac{516354}{342575} = 1.5 \qquad OK$$

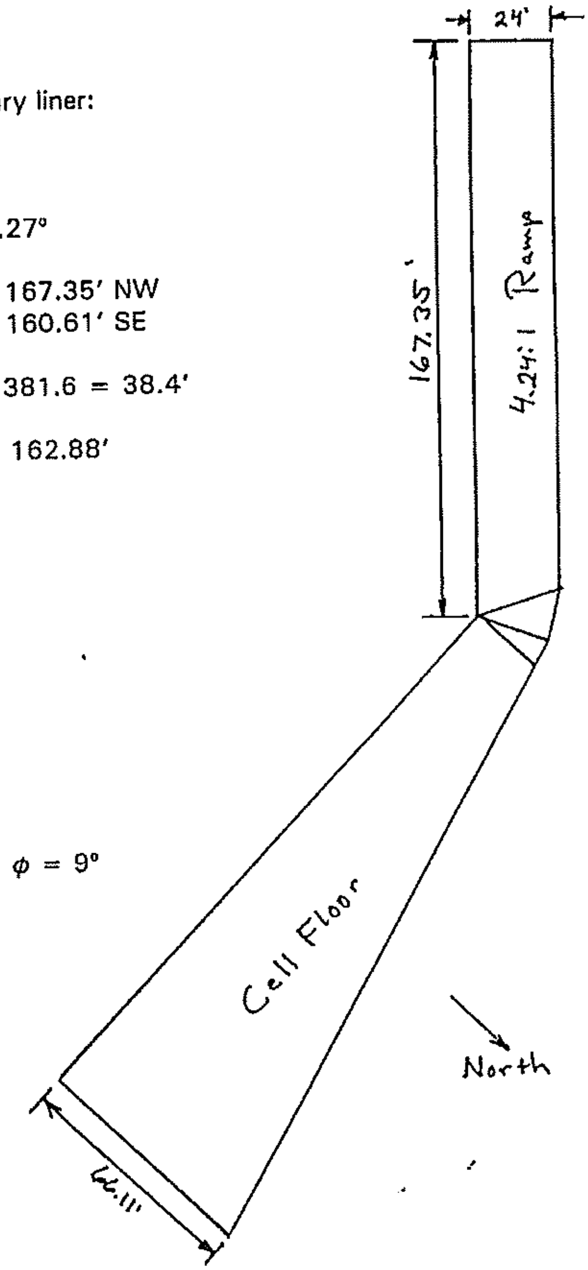
Volume of Concrete Required

$$Vol = (23 \times 188.82 \times \frac{8}{12}) + (6.24 \times \frac{98.06 + 23}{2} \times \frac{8}{12}) = 200.5\ yd^3$$

Cell Volume Reduced

$$Vol = (62.4 \times \frac{98.06 + 23}{2} \times 1) = 140\ yd^3$$

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/6/93 BY JRM
SUBJECT Ramp Stability SHEET 1 OF 2



West Ramp

Configuation at top of tertiary liner:

24' wide

slope 4.24:1 = 13.27°

Length (slope) 167.35' NW
160.61' SE

Elevation 1420 to 1381.6 = 38.4'

Horizontal Length = 162.88'

Ramp Section - proposed

min 3' soil cover

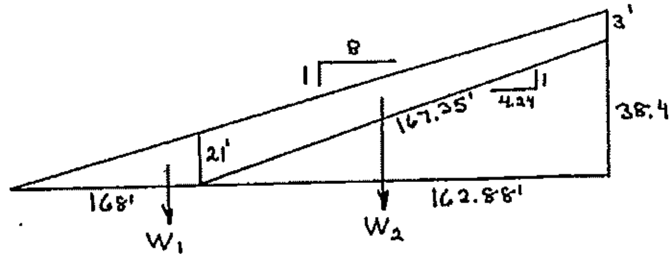
Friction/Cohesion of Materials

Cover 30°

Between HDPE and Net $\phi = 9°$

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 4/8/93 BY JRM
SUBJECT Ramp Stability SHEET 2 OF 2

West Ramp



Weights

$$W_1 = \frac{1}{2}(168)(21)(100) = 176{,}400\,plt$$

$$W_2 = \frac{(21+3)}{2}(167.35)(100) = 200{,}820\,plt$$

Driving Forces

$$= (200{,}820)(\sin 13.27^\circ) = 46{,}096\,plt$$

Resisting Forces

$$= (176{,}400)(\tan 9^\circ) + (200{,}820)(\cos 13.27^\circ)(\tan 9^\circ) = 58{,}897\,plt$$

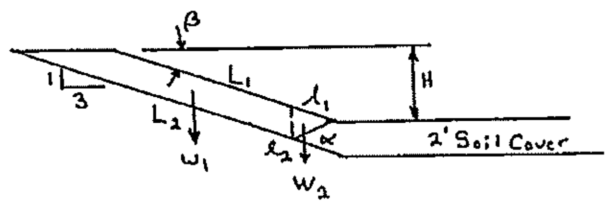
$$SF = \frac{58897}{46096} = 1.28$$

APPENDIX F

PROTECTIVE COVER STABILITY

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/19/93 BY JRM
SUBJECT Protective Cover Stability SHEET 1 OF 2

Configuration



Determine:

- maximum height (H) of cover material that can be placed up the 3:1 slope to protect liner
- Tensile strength required in liner if H is increased

Strength Parameters:	ϕ
Cover	20°
HDPE/Drainage net interface	9.1°
Embankment/HDPE interface	17°

Stability Calculations

$\alpha = 45 - \frac{26}{2} = 32°$, $d = 2'$ $\gamma = 110$ pcf, $\beta = 18.43°$

Failure Slope $= \alpha - \beta = 13.57° = \theta$

$$W_1 = \left(\frac{L_1 + L_2}{2}\right) d\,\gamma$$

$$l_2 = \frac{2}{\tan\beta} - \frac{2}{\tan\alpha} = 2.8 \qquad L_2 = \frac{H}{\sin\beta} + l_2$$

$$l_1 = \frac{2}{\tan\alpha} + 2\tan\beta = 3.867 \qquad L_1 = \frac{H}{\sin\beta} - l_1$$

$$W_1 = \left(\frac{\frac{H}{\sin\beta} - 3.867 + \frac{H}{\sin\beta} + 2.800}{2}\right)(2)(10)$$

$$= 695.9\ H - 117.4$$

$$W_2 = l_1 d\gamma = (3.867)(2)(110) = 425.4 \text{ lb}$$

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/19/93 BY JRM
SUBJECT Protective Cover Stability SHEET 2 OF 2

$$SF = \frac{\textit{Resisting Forces}}{\textit{Driving Forces}}$$

$$= \frac{W_1 \cos\beta \tan\phi_2 + W_2 \cos\theta \tan\phi_s + W_2 \sin\theta}{W_1 \sin\beta}$$

$$= \frac{(695.9H - 117.4)\ 0.15195 + 425.4\ (0.47413) + 425.4\ (0.23455)}{(695.9\ H - 117.4)\ 0.31623}$$

$$= \frac{105.74H + 283.62}{220H + 37.13}$$

Using SF = 1 = 105.74 H + 283.62 = 220 H - 37.13

H = 2.8'

Tension is required to increase solid cover height

$$SF = 1 = \frac{105.74\ H + 283.62 + T}{220\ H - 37.13}$$

T = 114.26 H - 320.75

H (ft)	Tension (plf)
4	136
6	365
8	593

Tensile Strength of 80 mil HDPE liner at yield is 2016 plf for 5' height

$$SF = \frac{105.74(5) + 783.62 + 2016}{220(5) - 37.13} = 2.7 \qquad OK$$

APPENDIX G

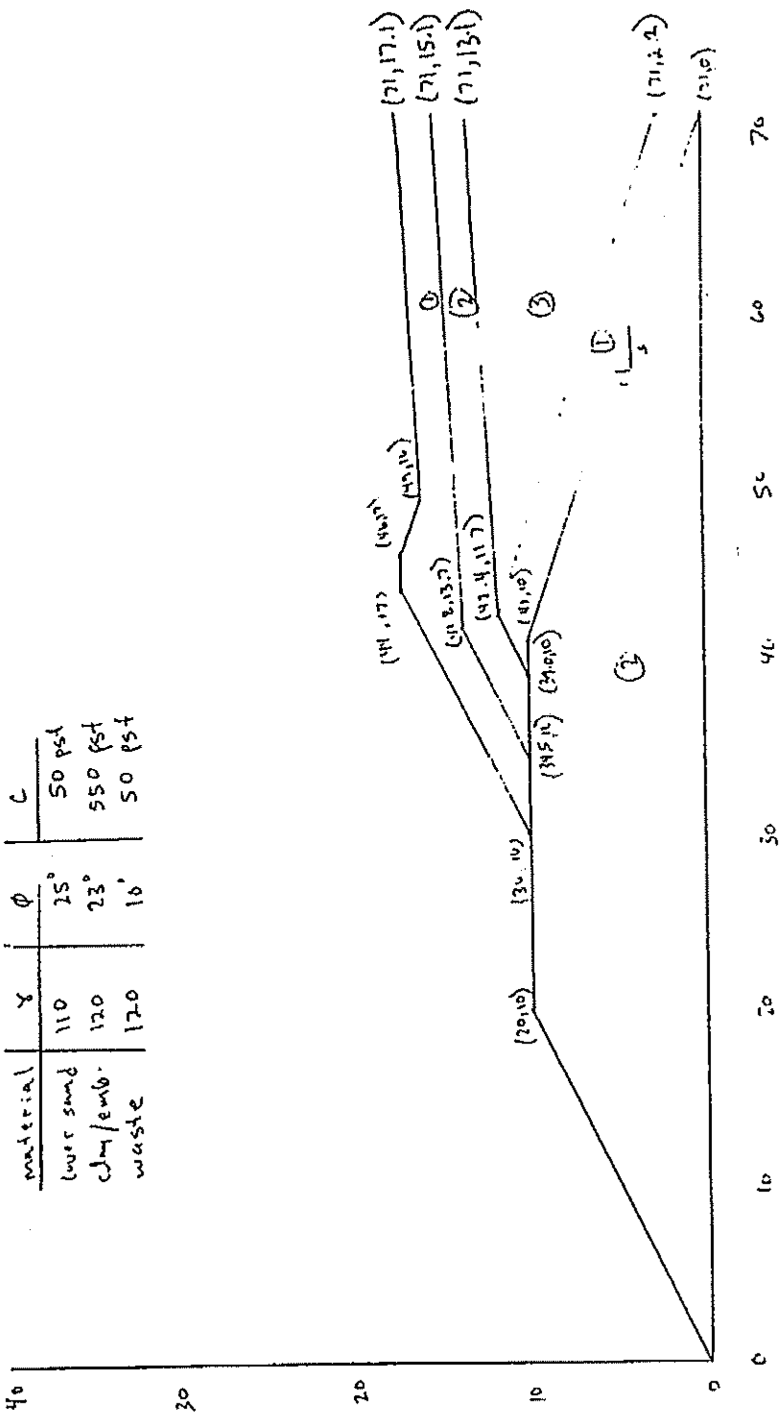
CELL CAP STABILITY

AGEC

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/19/93 BY JRM

SUBJECT Cell cap Stability SHEET 1 OF 5

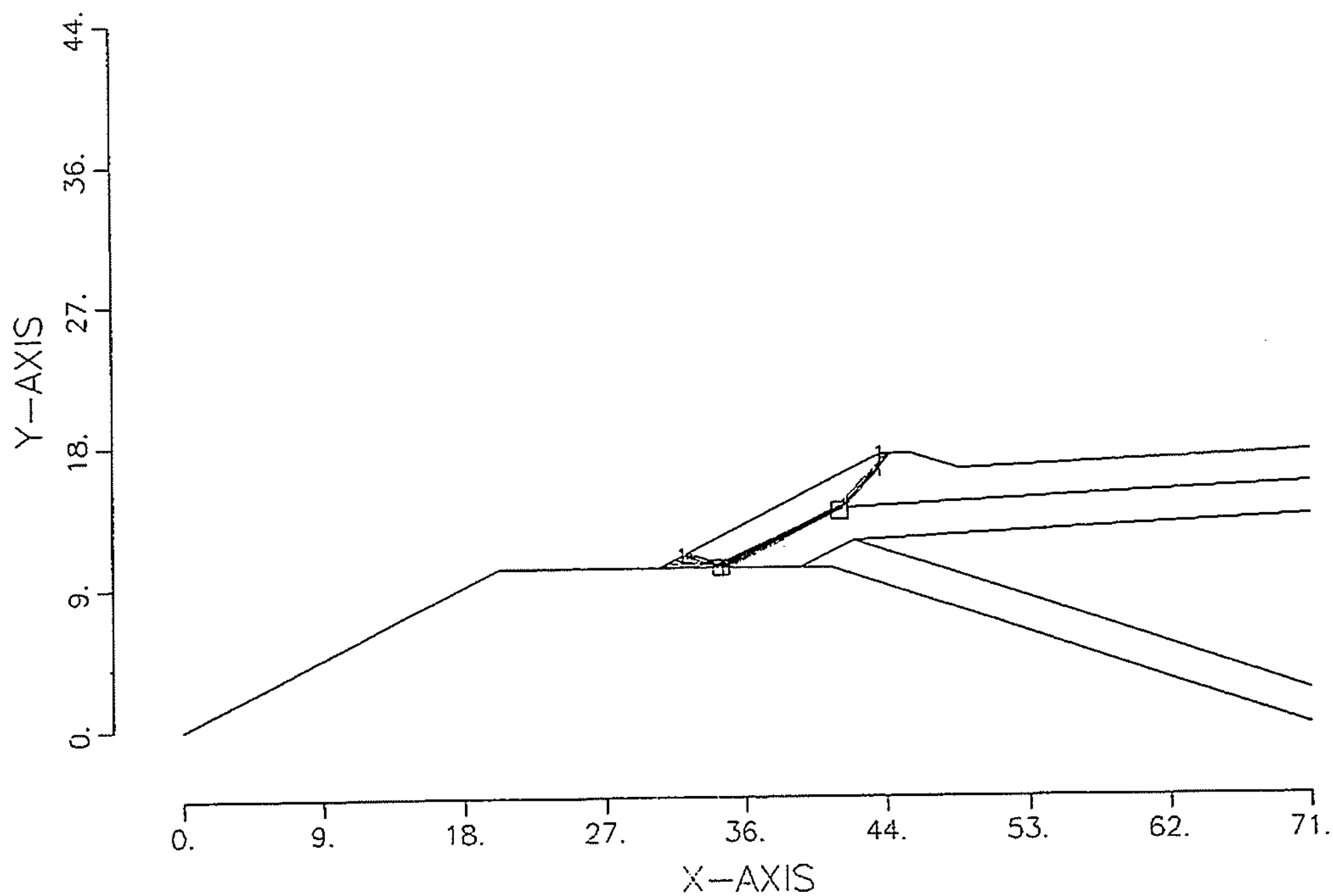


material	γ	φ	c
cover sand	110	25°	50 psf
clay/emb.	120	23°	550 psf
waste	120	10°	50 psf

AGI
Midvale UT s/n5206

Cell 15 Cap / Long Term / Static

1000SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.810

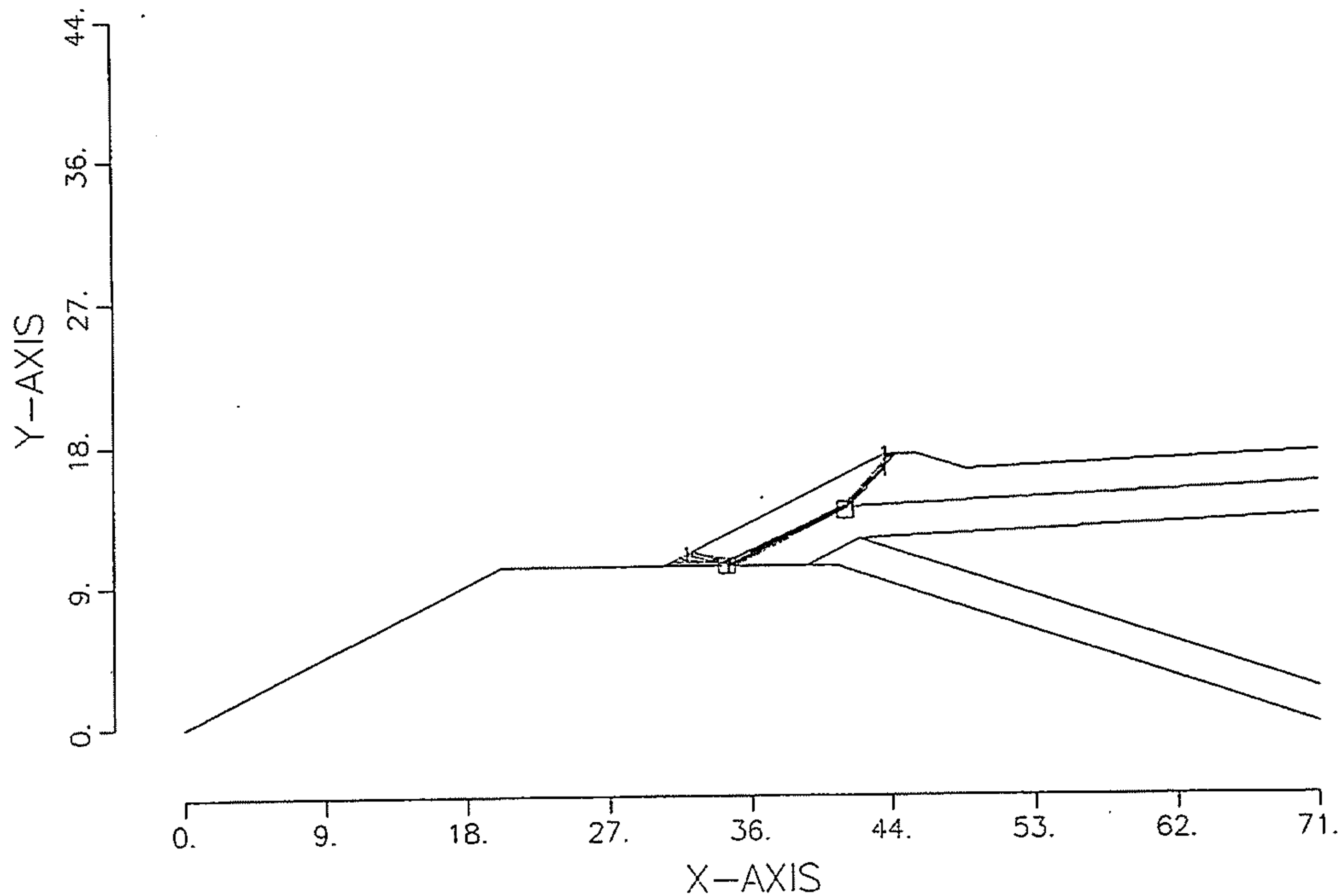


2 of 5

AG
Midvale UT s/n5206

Cell 15 Cap/Long Term/0.04g Earthquake

1000SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.653



--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION Cell 15 Cap / Long Term / Static

BOUNDARY COORDINATES
6 TOP BOUNDARIES
15 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SOIL TYPE BELOW BND
1	.00	.00	20.00	10.00	2
2	20.00	10.00	30.00	10.00	2
3	30.00	10.00	44.00	17.00	1
4	44.00	17.00	46.00	17.00	1
5	46.00	17.00	49.00	16.00	1
6	49.00	16.00	71.00	17.10	1
7	30.00	10.00	34.50	10.00	2
8	34.50	10.00	41.80	13.70	2
9	41.80	13.70	71.00	15.10	2
10	34.50	10.00	39.00	10.00	2
11	39.00	10.00	42.40	11.70	1
12	42.40	11.70	71.00	13.10	3
13	42.40	11.70	71.00	2.20	1
14	39.00	10.00	41.00	10.00	2
15	41.00	10.00	71.00	.00	2

ISOTROPIC SOIL PARAMETERS
3 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT.	SATURATED UNIT WT.	COHESION INTERCEPT	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT	PIEZOMETRIC SURFACE NO.
1	110.0	110.0	50.0	25.0	.00	.0	1
2	120.0	120.0	550.0	23.0	.00	.0	1
3	120.0	120.0	50.0	10.0	.00	.0	1

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING SLIDING BLOCK SURFACES, HAS BEEN SPECIFIED.

1000 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 3.0

BOX NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	WIDTH
1	33.50	10.00	34.50	10.00	1.00
2	41.00	13.50	42.00	13.50	1.00

* * SAFETY FACTORS ARE CALCULATED BY THE MODIFIED JANBU METHOD * *
FAILURE SURFACE # 1 SPECIFIED BY 5 COORDINATE POINTS

SAFETY FACTOR = 1.810

POINT NO.	X-SURF	Y-SURF	ALPHA (DEG)
1	31.47	10.73	-15.42
2	34.06	10.02	26.08
3	41.88	13.85	45.09
4	44.00	15.97	89.89
5	44.00	17.00	

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION   Cell 15 Cap/Long Term/0.04g Earthquake

BOUNDARY COORDINATES
6 TOP BOUNDARIES
15 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SOIL TYPE BELOW BND
1	.00	.00	20.00	10.00	2
2	20.00	10.00	30.00	10.00	2
3	30.00	10.00	44.00	17.00	1
4	44.00	17.00	46.00	17.00	1
5	46.00	17.00	49.00	16.00	1
6	49.00	16.00	71.00	17.10	1
7	30.00	10.00	34.50	10.00	2
8	34.50	10.00	41.80	13.70	2
9	41.80	13.70	71.00	15.10	2
10	34.50	10.00	39.00	10.00	2
11	39.00	10.00	42.40	11.70	1
12	42.40	11.70	71.00	13.10	3
13	42.40	11.70	71.00	2.20	1
14	39.00	10.00	41.00	10.00	2
15	41.00	10.00	71.00	.00	2

ISOTROPIC SOIL PARAMETERS
3 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT.	SATURATED UNIT WT.	COHESION INTERCEPT	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT	PIEZOMETRIC SURFACE NO.
1	110.0	110.0	50.0	25.0	.00	.0	1
2	120.0	120.0	550.0	23.0	.00	.0	1
3	120.0	120.0	50.0	10.0	.00	.0	1

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .040 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE =   .0

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING SLIDING BLOCK SURFACES, HAS BEEN SPECIFIED.

1000 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   3.0

BOX NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	WIDTH
1	33.50	10.00	34.50	10.00	1.00
2	41.00	13.50	42.00	13.50	1.00

* * SAFETY FACTORS ARE CALCULATED BY THE MODIFIED JANBU METHOD * *
FAILURE SURFACE # 1 SPECIFIED BY 5 COORDINATE POINTS

SAFETY FACTOR =   1.653

POINT NO.	X-SURF	Y-SURF	ALPHA (DEG)
1	31.47	10.73	-15.42
2	34.06	10.02	26.08
3	41.88	13.85	45.09
4	44.00	15.97	89.89
5	44.00	17.00	

APPENDIX H

SETTLEMENT ANALYSIS

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/20/93 BY JRM
SUBJECT Settlement Analysis SHEET 1 OF 7

Loading Parameters

Embankment

2.1:1 & 3:1 Slopes   320′ wide base   ~1700′ long at base

Height 66′ maximum

$\sigma = 0$ to 7920 psf

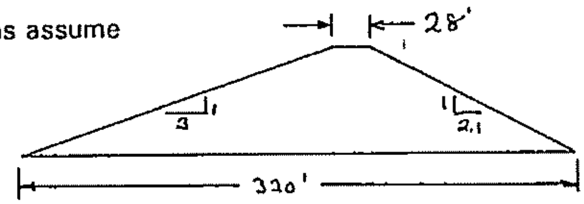
Cell

~ 3100 x 1500   including cells 10, 11, 12, 13, & 14

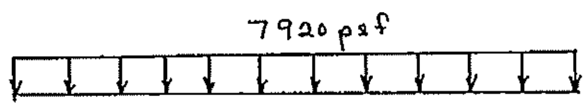
$\sigma = 9600$ to 0   use 60′ high ave $\Rightarrow \sigma = 7200$ psf

For Settlement Calculations assume

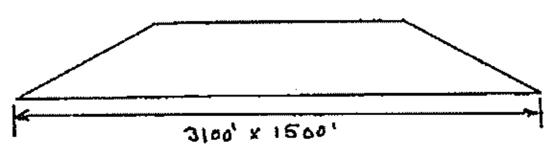
Embankment
Configuration



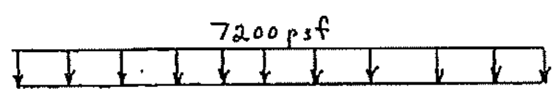
Conservative
Loading



Cell
Configuration



Conservative
Loading



PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/20/93 BY JRM
SUBJECT Settlement Anlaysis SHEET 2 OF 7

Consolidation Test Results

Boring	Depth	Cc′	Cr′	Swell	Material
4	2′	0.055	0.015	---	clay
4	22′	0.038	0.013	1.7	claystone
13	12′	0.015	---	1.0	claystone
16	7′	0.013	---	0	claystone
18	1′	0.029	---	2.0	mudstone
19	9′	0.030	---	0	mudstone
20	24′	0.019	0.010	1.6	mudstone
21	4′	0.079	0.006	---	clay
1	7′	0.014	0.002	0.3	claystone
2	22′	0.0125	---	0.3	claystone
5	7′	0.02	0.005	0.7	claystone
9	15′	0.027	0.005	1.4	claystone
11	5′	0.015	0.005	1.3	claystone
19	17′	0.016	---	1.3	claystone
Average		0.024	0.006	1.16	

Extraplate expansion into results and assume all samples are under recompression.

	Cr′
B-1 @ 7	0.005
B-2 @ 22	0.006
B-4 @ 22	0.005
B-5 @ 7	0.004
B-9 @ 15	0.006
B-11 @ 5	0.005
B-13 @ 15	0.003
B-19 @ 17	0.005
Average	0.00049

PROJECT NO 24292 TITLE Landfill Cell 15 DATE 1/20/93 BY JRM
SUBJECT Settlement Analysis SHEET 3 OF 7

Material is overconsolidated - check pre-consolidation pressure

DM 7.01 p. 142 LI vs. preconsolidation

$$LI = \frac{WC - PL}{LL - PL}$$

Hole	Depth	WC	LL	PI	LI
1	7	22.3	38	11	-0.4
7	7	17.4	28	11	0.036
9	5	19.2	33	7	-0.97
11	5	17.9	33	13	-0.16
12	27	22.3	41	17	-0.1
20	24	22.8	48	20	-0.26

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/20/93 BY JRM
SUBJECT Settlement Analysis SHEET 4 OF 7

Based on LI, the preconsolidation pressure is at least 12,000 psf.
Elastic Settlement - Immediate

$$\delta_V = qB\ \frac{1-\nu}{E_u}\ I$$ *DM* 7.01 *p.* 209

66' dike & cell

cell q = 9600 psf B = 1500' I = 1.12
ν = 0.5 saturated soil - no volume change upon loading
E_u = undrained modulus
Empirical
E_u = 600 c = 600(9287 psf) = 5.6×10^6 psf
based on shear wave velocity (empirical)

$$C_s = \sqrt{\frac{E}{P}\ \frac{1}{2(1+\nu)}} \qquad 3000\ ft/sec = \sqrt{\frac{E(32.2\ ft/s^2)}{128\,pcf(2)(1+.5)}}$$

E = 1.07×10^8 psf or 7.45×10^5 psi

Hole	Depth	σ	ϵ	E
20	12	20,632	0.024	8.6×10^5
21	17	18,313	0.019	9.6×10^5
13	7	16,822	0.029	5.8×10^5
14	17	22,559	0.02	1.13×10^5
15	6	16,708	0.027	6.2×10^5
17	27	23,100	0.054	4.3×10^5
3	7	22,856	0.058	3.9×10^5
7	7	22,115	0.033	6.9×10^5
9	5	20,674	0.174	2.8×10^5
10	20	25,211	0.045	5.6×10^5
			average	5.5×10^5 psf

$$\delta_V = 9600\ psf(1500ft)\left(\frac{1-0.5^2}{5.5\times10^5 psf}\right) 1.12 = 22\ feet \quad (too\ high)$$

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 1/20/93 BY JRM
SUBJECT Settlement Analysis SHEET 5 OF 7

If $E_u = 5.6 \times 10^6$ psf $\delta_v = 2.2$ ft or 26" (too high)

If Foundation Mtr was concrete $E_u = 2$ to 6×10^6 psi
$\delta_v = 0.5" - 0.17"$

The calculated elastic settlement appears to be higher than would logically occur

Consolidation Settlement - Calculated from Consolidation Tests

Profile	C_r'	C_c'
0 - 10'	--	0.03
9' - 300'	0.0033	--
300' - 900'	0.0022	--
900' - 2100'	0.0011	--

Settlements

66' embankment $\rho = 8\frac{1}{2}"$

1500' x 3100' x 60' cell $\rho = 9"$

OB NUMBER: 24292
andfill Cell 15
ength(X): 1700.0 ft Width(Y): 320.0 ft Load: 7920 psf X-Coord = .0 ft
a Depth: 5 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

S LAYER	SOIL TYPE	LAYER THICKNESS (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	CL-ML	10	124.0	.0300	.0060	4.646	.000
2	mudst	290	128.0	.0033	.0000	3.233	.000
3	mudst	600	128.0	.0022	.0000	.523	.000
4	mudst	1200	128.0	.0011	.0000	.064	.000

TOTAL SETTLEMENT= 8.466 inches

)B NUMBER: 24292
ındfill Cell 15
ɔngth(X): 3100.0 ft Width(Y):1500.0 ft Load: 7200 psf X-Coord = .0 ft
ıter Depth: 5 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICKNESS (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	CL-ML	10	124.0	.0300	.0060	4.507	.000
2	mudst	290	128.0	.0033	.0000	3.350	.000
3	mudst	600	128.0	.0022	.0000	1.108	.000
4	mudst	1200	128.0	.0011	.0000	.274	.000

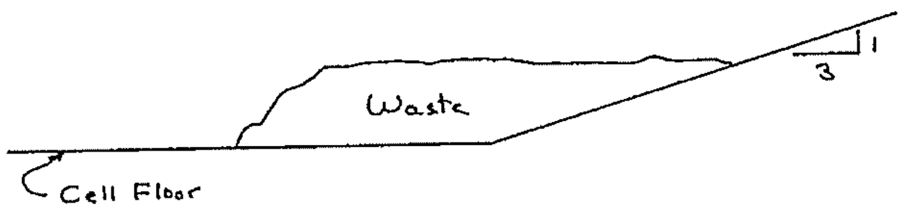
TOTAL SETTLEMENT= 9.239 inches

APPENDIX I

INTERIOR WASTE STABILITY

PROJECT NO. 24292 TITLE Landfill Cell 15 DATE 3/31/93 BY JRM
SUBJECT Interior Waste Stability SHEET 1 OF 3

Configuration:



Concerns:

-The friction between synthetic materials on the floor and side slopes will likely be around 9 degrees. This should be verified once the materials are delivered.

Calculations:

-In order to resist the potential to slide - the amount of waste on the floor should be sufficient to provide the required resistance.

-Maintain a safety Factor of at least 1.5.

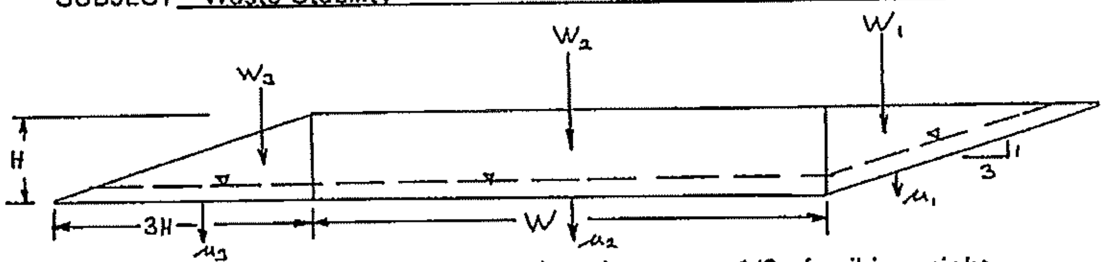
-Assume a waste face of 3:1 (horizontal:vertical)

-Highest depth is 72.5' from top of uppermost liner to the top of cap.

-Stability calculations assuming that the friction of 9° applies along the entire synthetic material profile.

PROJECT NO. 24292 TITLE Cell 15 DATE 4/2/93 BY JRM

SUBJECT Waste Stability SHEET 2 OF 3



assume 1′ of water along bottom ~ 1/2 of soil in weight

$W_1 \propto \frac{1}{2}(3H)(H) = \frac{3}{2}H^2$ $\quad u_1 \propto 1.58H$

$W_2 \propto HW$ $\quad u_2 \propto \frac{1}{2}W$

$W_3 = W_1 = \frac{3}{2}H^2$ $\quad u_3 \propto \frac{3}{2}H$

Driving Forces

$$W_1 \sin 18.43^\circ = \frac{3}{2}H^2 \sin 18.43^\circ + 1.58H = 0.474H^2 + 1.58H$$

Resisting Forces

$$= W_1 \cos 18.43^\circ \tan 9^\circ + W_2 \tan 9^\circ + W_3 \tan 9^\circ$$

$$= \frac{3}{2}H^2 \sin 18.43^\circ \tan 9^\circ + HW \tan 9^\circ + W_3 H^2 \tan 9^\circ$$

$$= 0.463H^2 + 0.158HW$$

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

$$= \frac{0.463H^2 + 0.158HW}{0.474H^2 + 1.58H}$$

PROJECT NO. 24292 TITLE CELL 15 DATE 4/2/93 BY JRM

SUBJECT Waste Stability SHEET 3 OF 3

For FS = 1.5

H	W	W/H	W + 3H/H Top Width/Height
5	22.8	4.6	7.6
10	30.7	3.1	6.1
20	46.4	2.3	5.3
40	77.8	1.9	4.9
60	109.2	1.8	4.8
80	140.6	1.8	4.8

To provide a simple relationship of waste height and length, and to provide a safety factor > 1.5

recommend Top width = 5 x height

PROJECT NO. 24292 TITLE CELL 15 DATE 6/15/94 BY JEN

SUBJECT Waste Stability SHEET 1 OF 7

Stability of Waste Only - would like to determine if a 3:1 slope would be appropriate. Due to the unknown characteristics and non-uniformity of the waste material, it is very difficult to assign strength parameters for the waste for stability evaluation.

If we were to assume that the waste has strength characteristics similar to those used in the overall cell stability ($c = 50$ psf, $\phi = 10°$) a fairly flat slope would be needed to maintain a safety factor of at least 1.3. If a 3 to 1 slope is used, and these strength parameters were included in the analysis for the strength of the waste, a safety factor of less than one would be achieved.

In order to predict if a 3 to 1 slope would be suitable for the waste face, three stability analyses are conducted to determine the strengths needed to maintain a safety factor of at least 1.3. The analyses will back calculate the required strengths assuming that the waste behaves like a cohesionless material, a cohesive material and material with both cohesion and friction.

Cohesionless:

$$S.F. = \frac{\tan\phi}{\tan\alpha}$$

Where: ϕ = material friction angle
α = Slope angle
If: S.F. = 1.3 and $\alpha = 18.3°$
Then: $\phi = 23.3°$

Cohesive:

Computor analysis indicates that c would need to be at least 1020 psf.

Mixture of friction and cohesion:

Computor analysis indicates the following possible parameters:
$c = 100$ psf with $\phi = 20°$

Conclusions:

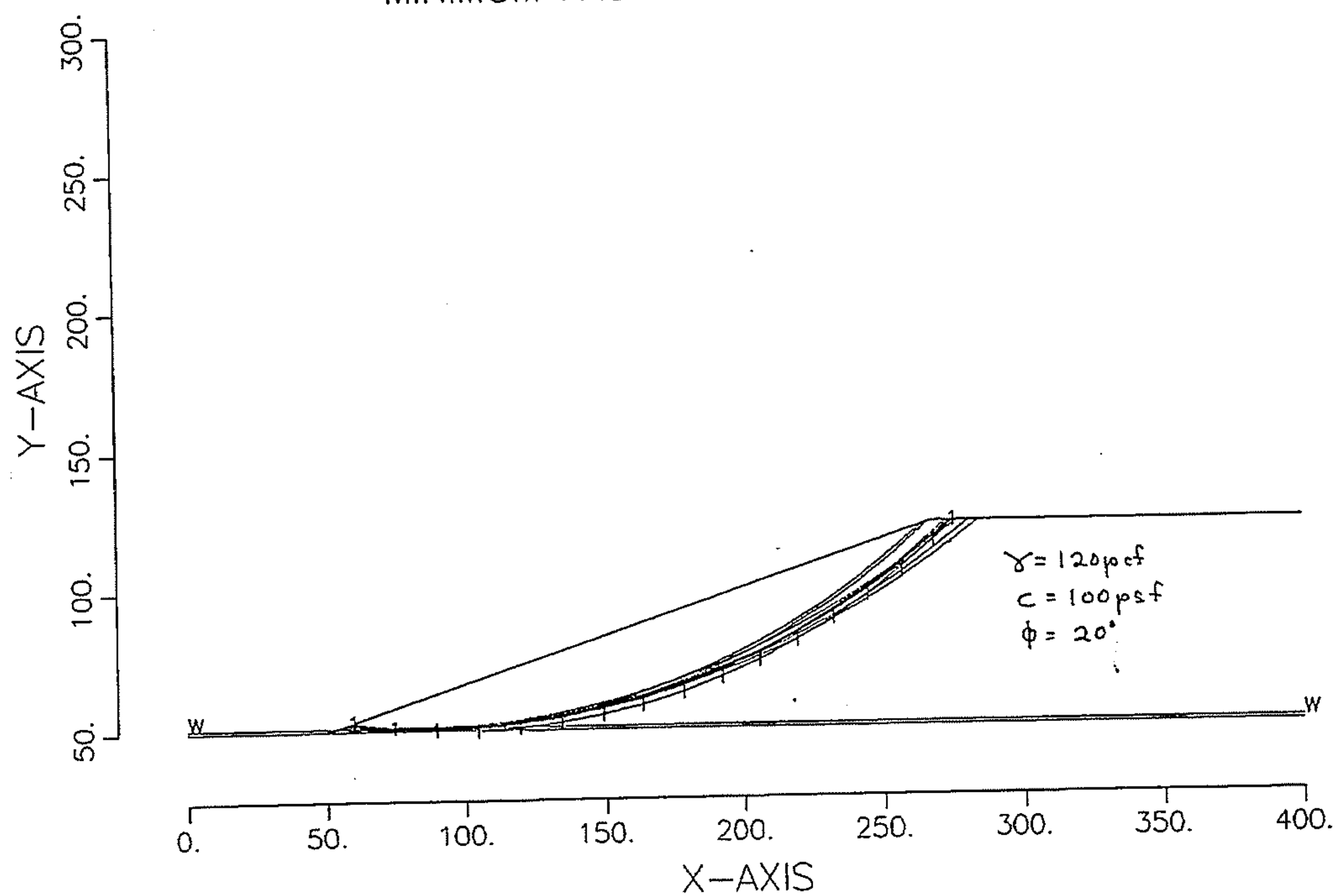
1. The waste will stay at the 3:1 slope due to the relatively low strength parameters required to maintain a S.F. = 1.3.

2. The waste strengths will be at least those calculated based on the following conditions:
   a. A saturated weak clay would typically have strengths of $\phi = 19°$ with $c = 230$ psf.
   b. Other in-organic soils have higher strength parameters.
   c. Loose sand has a friction angle of 26° - which is significantly higher (stronger) than the minimum required for the waste to be stable.
   d. Waste will be placed in relatively horizontal layers so the potential of a weak zone falling along a failure surface is very low.

AGEC
Midvale UT s/n5206

CELL 15 WASTE STABILITY

900 SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.300



--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION   CELL 15 WASTE STABILITY

BOUNDARY COORDINATES
3 TOP BOUNDARIES
4 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SOIL TYPE BELOW BND
1	.00	50.00	50.00	50.00	1
2	50.00	50.00	267.50	122.50	2
3	267.50	122.50	400.00	122.50	2
4	50.00	50.00	400.00	50.00	1

ISOTROPIC SOIL PARAMETERS

2 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT.	SATURATED UNIT WT.	COHESION INTERCEPT	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT	PIEZOMETRIC SURFACE NO.
1	120.0	120.0	550.0	23.0	.00	.0	1
2	120.0	120.0	100.0	20.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER	Y-WATER
1	.00	51.00
2	400.00	51.00

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

900 TRIAL SURFACES HAVE BEEN GENERATED.

30 SURFACES INITIATE FROM EACH OF 30 POINTS EQUALLY SPACED ALONG THE GROUND SURFACE BETWEEN X = 45.00
AND X = 60.00

EACH SURFACE TERMINATES BETWEEN X = 260.00
AND X = 400.00

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = 45.00

15.00 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 # 1 SPECIFIED BY 17 COORDINATE POINTS

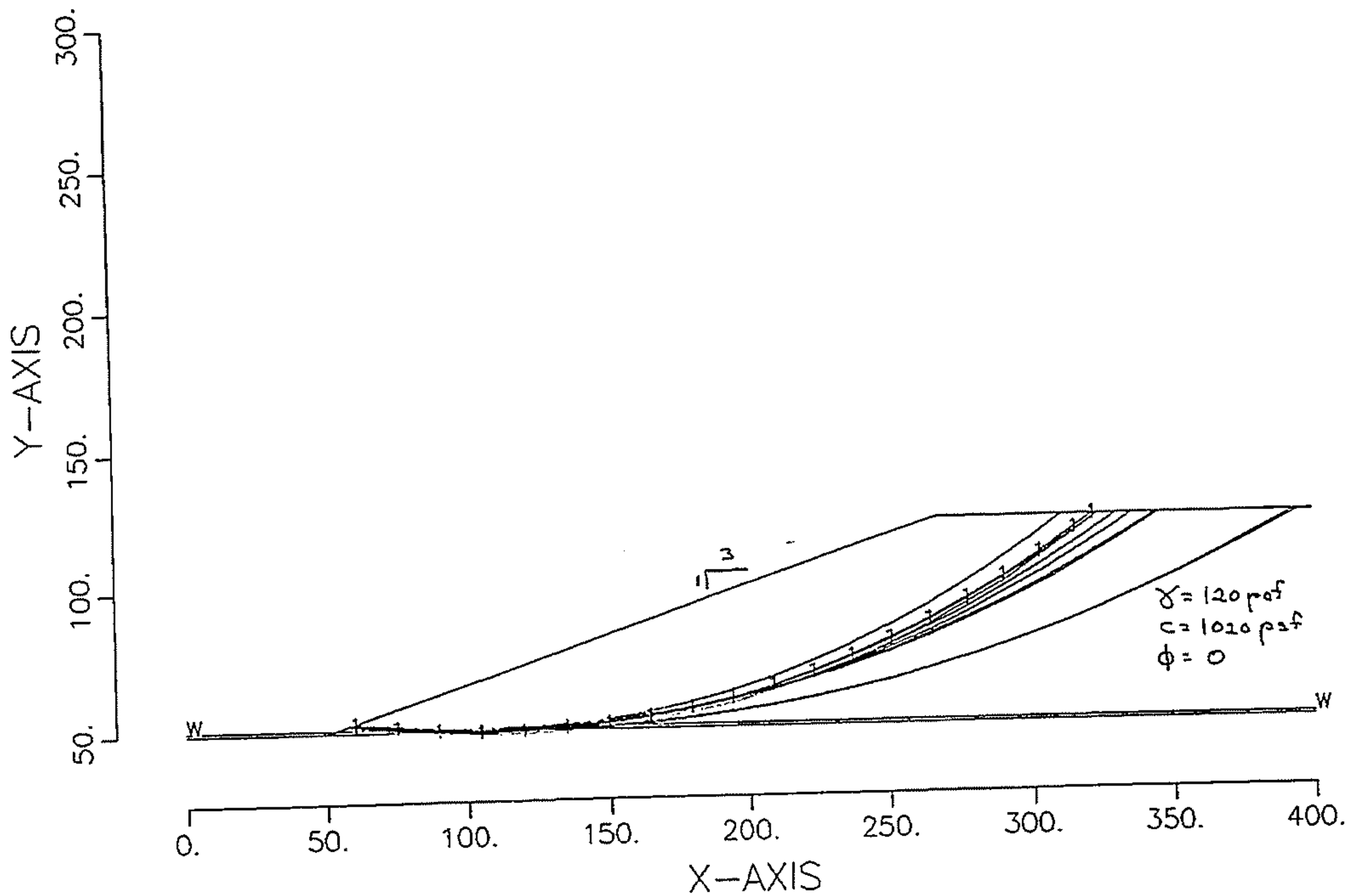
SAFETY FACTOR = 1.300

POINT NO.	X-SURF	Y-SURF	ALPHA (DEG)
1	59.48	53.16	-7.43
2	74.36	51.22	-3.98
3	89.32	50.18	-.54
4	104.32	50.04	2.90
5	119.30	50.80	6.35
6	134.21	52.45	9.79
7	148.99	55.00	13.23
8	163.59	58.44	16.67
9	177.96	62.74	20.12
10	192.05	67.90	23.56
11	205.80	73.90	27.00
12	219.16	80.71	30.45
13	232.09	88.31	33.89
14	244.54	96.67	37.33
15	256.47	105.77	40.78
16	267.83	115.57	44.22
17	274.95	122.50	

AGEC
Midvale UT s/n5206

CELL 15 WASTE STABILITY

900 SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.307



--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION   CELL 15 WASTE STABILITY

BOUNDARY COORDINATES
3 TOP BOUNDARIES
4 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT	Y-LEFT	X-RIGHT	Y-RIGHT	SOIL TYPE BELOW BND
1	.00	50.00	50.00	50.00	1
2	50.00	50.00	267.50	122.50	2
3	267.50	122.50	400.00	122.50	2
4	50.00	50.00	400.00	50.00	1

ISOTROPIC SOIL PARAMETERS

2 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT.	SATURATED UNIT WT.	COHESION INTERCEPT	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT	PIEZOMETRIC SURFACE NO.
1	120.0	120.0	550.0	23.0	.00	.0	1
2	120.0	120.0	1020.0	.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER	Y-WATER
1	.00	51.00
2	400.00	51.00

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

900 TRIAL SURFACES HAVE BEEN GENERATED.

30 SURFACES INITIATE FROM EACH OF 30 POINTS EQUALLY SPACED ALONG THE GROUND SURFACE BETWEEN X = 45.00
AND X = 60.00

EACH SURFACE TERMINATES BETWEEN X = 260.00
AND X = 400.00

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = 45.00

15.00 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 # 1 SPECIFIED BY 20 COORDINATE POINTS

SAFETY FACTOR = 1.307

POINT NO.	X-SURF	Y-SURF	ALPHA (DEG)
1	60.00	53.33	-6.76
2	74.90	51.57	-4.31
3	89.85	50.44	-1.85
4	104.85	49.95	.60
5	119.85	50.11	3.06
6	134.82	50.91	5.51
7	149.75	52.35	7.97
8	164.61	54.43	10.42
9	179.36	57.15	12.88
10	193.98	60.49	15.33
11	208.45	64.46	17.79
12	222.73	69.04	20.24
13	236.81	74.23	22.70
14	250.65	80.02	25.15
15	264.22	86.39	27.61
16	277.51	93.34	30.06
17	290.50	100.86	32.52
18	303.14	108.92	34.98
19	315.44	117.52	37.43
20	321.94	122.50	

Applied Geotechnical Engineering Consultants, Inc.

August 21, 1996

Hansen, Allen & Luce, Inc.
6771 South 900 East
Midvale, Utah 84047-1436

Attention: Marv Allen

Subject: Stability Analysis
Closure Cap
Landfill Cell 15
Lone Mountain Facility
Waynoka, Oklahoma
Project No. 29793

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. was requested to evaluate the stability of the closure cap for Landfill Cell 15 at the Lone Mountain facility utilizing a geosynthetic clay liner. Previous analyses have been conducted with 2 feet of clay as opposed to a geosynthetic clay liner. This letter was submitted on August 18, 1993.

<u>Closure Profile</u>

We understand that the profile for the closure will consist of the following from the top down.

6 inches of riprap
4 or 8 inches of granular filter
2 feet of soil cover
Non-woven geotextile
Drainage net (J Drain 200N)
Textured HDPE Liner
Geosynthetic clay liner (GCL)
Soil/Waste

The following unit weights and strength parameters were used in our analysis.

Material or Interface	Unit Weight	Friction	Cohesion
Filter and riprap	120	37°	0
Soil cover (compacted)	120	30°	100 psf
Soil cover/geotextile	—	25°	80 psf
Geotextile/drainage net	—	See Note	—
Drainage net/HDPE	—	See Note	—
HDPE/GCL	—	25.5°	—
GCL	—	26° (dry)	—
Soil	120	23°	550 psf

Note: The friction along the interfaces of the drainage net are dependant on the orientation of the net with the direction of movement. The lowest friction value with movement along the roll of the net is 15 degrees (between the geotextile and the drainage net). The lowest friction value with movement across the roll of the net is 18 degrees (between the net and geotextile or textured HDPE). A friction value of 8.3° was used for the interface friction angle between the geotextile/drainage net and the drainage net/textured HDPE along the ribs of the drainage net.

Perimeter Berm

Around the perimeter of the closure cap, a berm has been designed to control run off water. The berm is constructed above the synthetic materials and has exterior slopes of 2:1 (horizontal to vertical). The top width of the berm is approximately 2.6 feet and the berm extends approximately 2.8 feet above the main slope of the closure cap. The soil cover material placed above the synthetics in the berm area is to be compacted to at least 95 percent of the maximum density as determined by the Standard Proctor method. An additional 4-inch layer of filter material is placed above the soil cover layer in the berm area.

Stability Analysis

A. Main Cover

An infinite slope analysis was conducted assuming that the drainage net is rolled down the slope to evaluate the stability of the cover away from the perimeter berm. Calculations indicate a static safety factor of approximately 1.6, assuming slippage along the weakest layer, which would be the drainage net and the geotextile or textured liner along the rib of the net. Using a 0.04g horizontal acceleration, a safety factor for a seismic event was calculated to be approximately 1.1. If the drainage net is placed perpendicular (across) the slope, the safety factors would be 3.3 and 1.7 for the static and dynamic conditions, respectively.

B. Exterior Perimeter Berm

Calculations were conducted on the exterior perimeter berm. Safety factors were found to be approximately 1.5 under static conditions and 1.4 under dynamic conditions. The weakest slip plane was found to be within the exterior riprap/filter materials. A higher factor of safety was obtained for slippage surfaces going through the synthetic materials. If the GCL becomes wet, which is not likely, the factors of safety would be lower.

If you have any questions, or if we can be of further service, please call.

Sincerely,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
JAMES E. NORDQUIST
15327
8/21/96
OKLA

James E. Nordquist, P.E.

JEN/cs

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 29793 TITLE USPCI Cell 15 DATE 8/21/96 BY [illegible]

SUBJECT Perimeter Closure Stability SHEET 1 OF 4

Profile:

Closure is 10% slope w/perimeter berm (2:1 slope)

Materials (Top Down)	Internal Strengths φ	Internal Strengths c	Interface Strengths φ	Interface Strengths c	
6" Riprap	37°	0			
			33°	0	
4" or 8" Granular Filter	33°	0			
			33°	0	
2' Soil Cover	30°	100psf			
			25°	80psf	
Geotextile	–				
			8–11°	0	along rib
Drainage Net (J Drain 200N)	–				
			9.7°	0	along roll
HDPE textured (Polyflex)	–				
			25.5°	0	
GCL	10° or 400psf (26° if dry)				
Soil					

The critical interfaces will be:

	Along Roll φ	Along Roll c	Along Rib φ	Along Rib c	Across Rib φ	Across Rib c
Geotextile / Drainage Net	15°	0	8.3°	0	18°	0
Drainage Net / Textured Liner	18°	0	8.3°	0	18°	0

The dry strength of the GCL will be assumed due to its location below the HDPE.

For stability analysis use 15° down slope
8.3° along drainage net rib.

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 29793 TITLE USPCI Cell 15 DATE 8/21/96 BY [illegible]

SUBJECT Perimeter Closure Stability SHEET 2 OF 4

Top Slope.

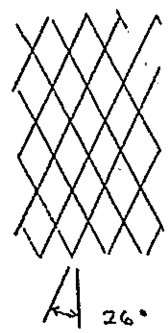
10% down slope - net rolled down slope

Static $F.S. = \frac{\tan 15°}{\tan 5.71°} = 2.7$ ok.

Dynamic. w/ a = 0.04 g 10% exceedence in 50yrs

$$F.S. = \frac{\cos 5.71 \tan 15}{\sin 5.71 + (0.04) \cos 5.71} = 1.91 \text{ ok}$$

Along net rib



dip along grid

$\tan \alpha = \tan 5.71 \cos 26$

$\alpha = 5.14°$

Static $S.F. = \frac{\tan 8°}{\tan 5.14°} = 1.56$

Dynamic $S.F. = \frac{\cos 5.14 \tan 8.3}{\sin 5.14 + (0.04) \cos 5.14} = 1.12$

10% downslope - net rolled perpendicular to slope along rib

Angle of Slope $\tan \alpha = \tan 5.71 \cos 64°$

$\alpha = 2.51°$

Static $S.F. = \frac{\tan 8.3}{\tan 2.51} = 3.3$ ok

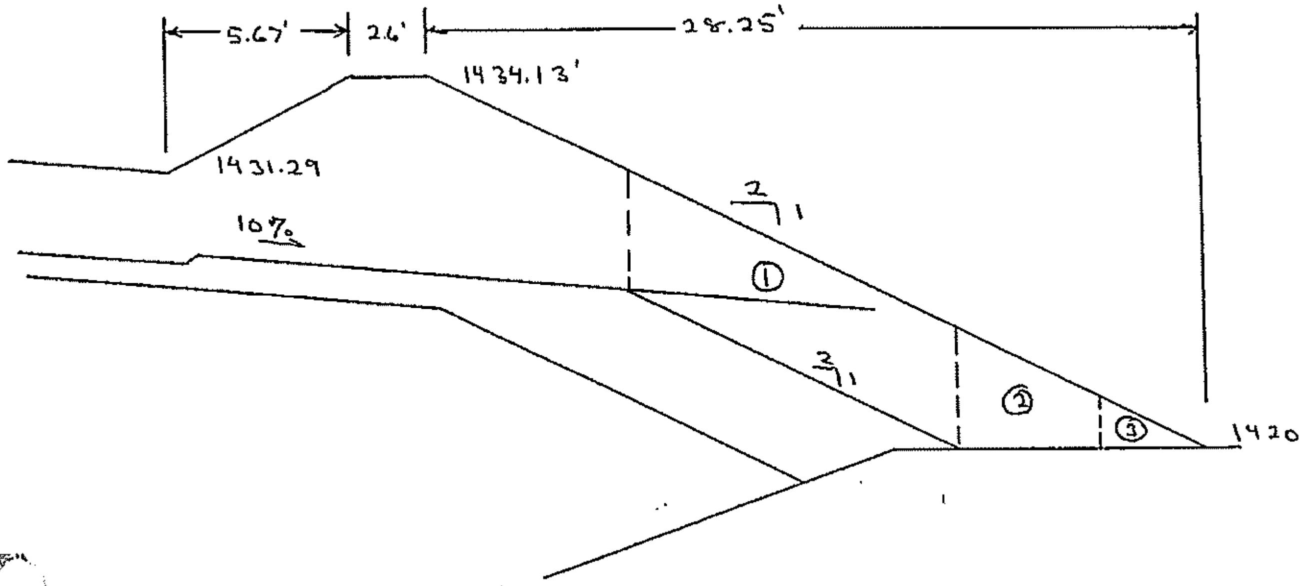
Dynamic $S.F. = \frac{\cos 2.51 \tan 8.3}{\sin 2.51 + (0.04) \cos 2.51} = 1.74$ ok

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 29793 TITLE USPCI Cell 15 DATE 8/21/96 BY [illegible]

SUBJECT Closure Stability SHEET 3 OF 4

Berm.



- Ignoring the 10% "Flap" extending beyond the 2:1 slope
- All soil above the 10% / 2:1 break will support itself

Slice	w	l	α	ϕ	c
1	(9.2)(3.5)(120) 3864	10.25	26.6	26	0
2	(2.4)(4)(120) 1152	4	0	30	100
3	(1.25)(3)(1/2)(120) 225	3	0	37	0

$$S.F. = \frac{3864 \cos 26.6 \tan 26 + 1152 \cos 0 \tan 30 + (4)(100) + 225 \tan 37}{3864 \sin 26.6}$$

$$= \frac{2919}{1730} = 1.69 \text{ ok}$$

AGEC

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 29793 TITLE USPCI Cell 15 DATE 8/21/96 BY [illegible]

SUBJECT Closure Stability SHEET 4 OF 4

Seismic

$$S.F. = \frac{2919}{1730 + 5241(0.04)} = 1.5 \text{ ok}$$

Filter Materials

Static $$SF = \frac{\tan 37}{\tan 26.5} = 1.5 \text{ ok}$$

Dynamic $$SF = \frac{\cos 26.5 \tan 37}{\sin 26.5 + 0.04 \cos 26.5} = 1.4 \text{ ok}$$

Applied Geotechnical Engineering Consultants, Inc.

July 21, 1994

HA&L Engineering
6771 South 900 East
Midvale, Utah 84047-1436

Attention: Marv Allen

Subject: Clay/Driscopipe Compression
Lone Mountain Facility
USPCI Waynoka, Oklahoma
Project No. 24292A

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. conducted laboratory tests on samples of lean clay and mixtures of lean clay with sand to measure the vertical strain when loaded from 200 to 9,250 pounds per square foot. The tests were conducted to assist in the design of the leachate withdrawal pipes.

The laboratory tests were conducted in one-dimensional consolidometers on remolded samples that were submerged during testing. A letter summarizing our test results was submitted on July 12, 1994.

Subsequent to our original testing, we visited with Dr. Reynold Watkins of Utah State University with respect to the procedures developed by Dr. Watkins on buried flexible pipe design. The standard design charts indicate the vertical stress-strain data for typical trench backfill from actual tests. The chart indicates that the values do not apply for clay soils.

Due to the fact that the backfill for the USPCI facility is clay soil, Dr. Watkins was asked to recommend a procedure to determine the strain which should be used in design. Dr. Watkins indicated that a conservative approach would be to conduct one-dimensional consolidation tests and incorporate the amount of strain measured up to the design load. He also indicated that the lateral restraint is conservative with the one-dimensional consolidation, due to the fact that as the flexible pipe is compressed, the pipe will push into the adjacent soil. With this in mind, Dr. Watkins recommended that a realistic strain for our analysis would be to use one-half of the one-dimensional strain.

Additional Testing

In review of the actual field conditions, the clay backfill around the pipe will not be submerged. With this condition, additional testing was conducted to determine the stress-strain relationship in a one-dimensional consolidometer with the sample out of water. The tests indicate the following amounts of strain when loaded from 200 to 9,250 pounds per square foot.

90% Compaction	95% Compaction
14 percent	4½ percent

Test results are attached.

Recommendations

Based on our understanding of the procedure used for designing buried flexible pipe, we recommend that a strain ranging from 2-1/4 to 3 percent be utilized. This value ranges from 1/2 of the unwetted compression to 1/2 of the average between the wetted and the unwetted conditions.

For these strain values to apply, the material would need to be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698.

If you have any questions, or if we can be of further service, please call.

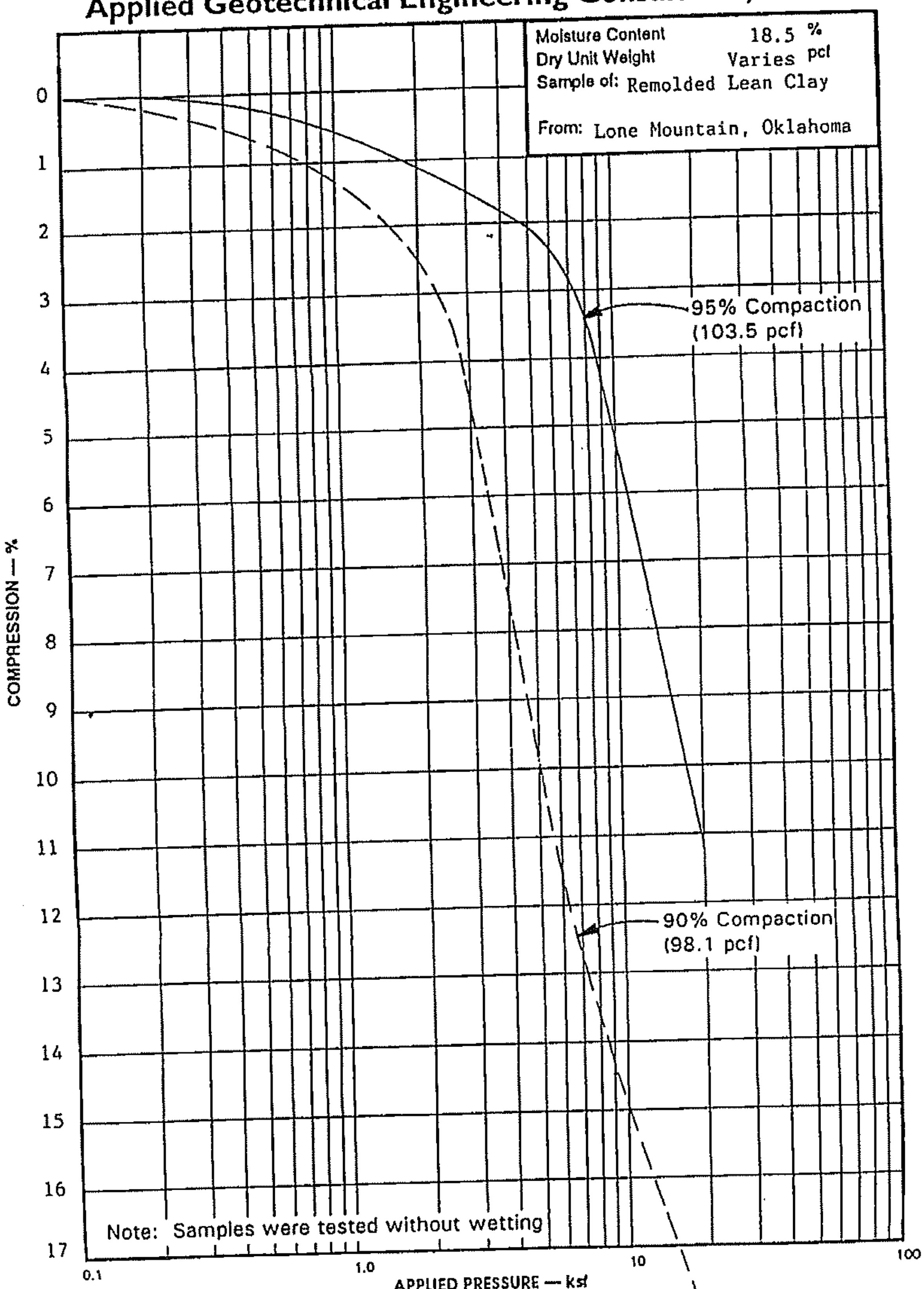
Sincerely,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

James E. Nordquist, P.E.

JEN/cs
enclosure

Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A CONSOLIDATION TEST RESULTS Figure 1

Applied Geotechnical Engineering Consultants, Inc.

July 12, 1994

HA&L Engineering
6771 South 900 East
Midvale, Utah 84047-1436

Attention: Marv Allen

Subject: Clay/Clay-Sand Mixture Compression
Lone Mountain Facility
USPCI, Waynoka, Oklahoma
Project No. 24292A

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. was requested to conduct laboratory tests on samples of lean clay and mixtures of lean clay with sand to determine the strain between 200 to 9,250 pounds per square foot. We understand that a strain of less than 3.9 percent is needed for backfill around the leachate withdrawl pipes.

Testing

A sample of Lone Mountain clay was submitted to our laboratory and tested to determine Atterberg Limits, percent finer than the number 200 sieve, moisture/density relationship and consolidation. The consolidation tests were conducted on the clay sample remolded to 90, 95 and 101 percent of the maximum dry density as determined by ASTM D-698. The amount of strain measured from these tests was found to exceed the strain needed for the facility. Results of the testing is shown on Figure 4.

In order to reduce the amount of strain using material that will hold itself together, the on-site clay soil was mixed with sand similar to the sand that was previously obtained and tested from the Lone Mountain area. A mixture of 50 percent sand and 50 percent lean clay was tested for moisture/density relationship and consolidation. The consolidation samples were remolded to 92 and 97 percent of the maximum dry density as determined by ASTM D-698. The amount of strain measured with this mixture exceeded the amount of strain desired in the design. Results of the testing is shown on Figure 3.

A mixture of 75 percent sand and 25 percent clay was then tested for compressibility when remolded. Samples were remolded to 90 and 95 percent of the maximum dry density with results as shown on Figure 2.

The tests indicate the following amount of strain.

Strain from 200 to 9250 pounds per square foot

Mixture Ratio Clay/Sand	Percent Fines	Strain, 90% Compaction	Strain, 95% Compaction
100:0	93%	13	7½
50:50	55%	9	5
25:75	35%	6	2

Summary

Based on the tests conducted, in order to maintain strain below or equal to 3½ percent when loaded from 200 to 9,250 pounds per square foot, we recommend that the material contain from 25 to 42 percent fines. The fines need to be clay and the mixture should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698.

If you have any questions, or if we can be of further service, please call.

Sincerely,

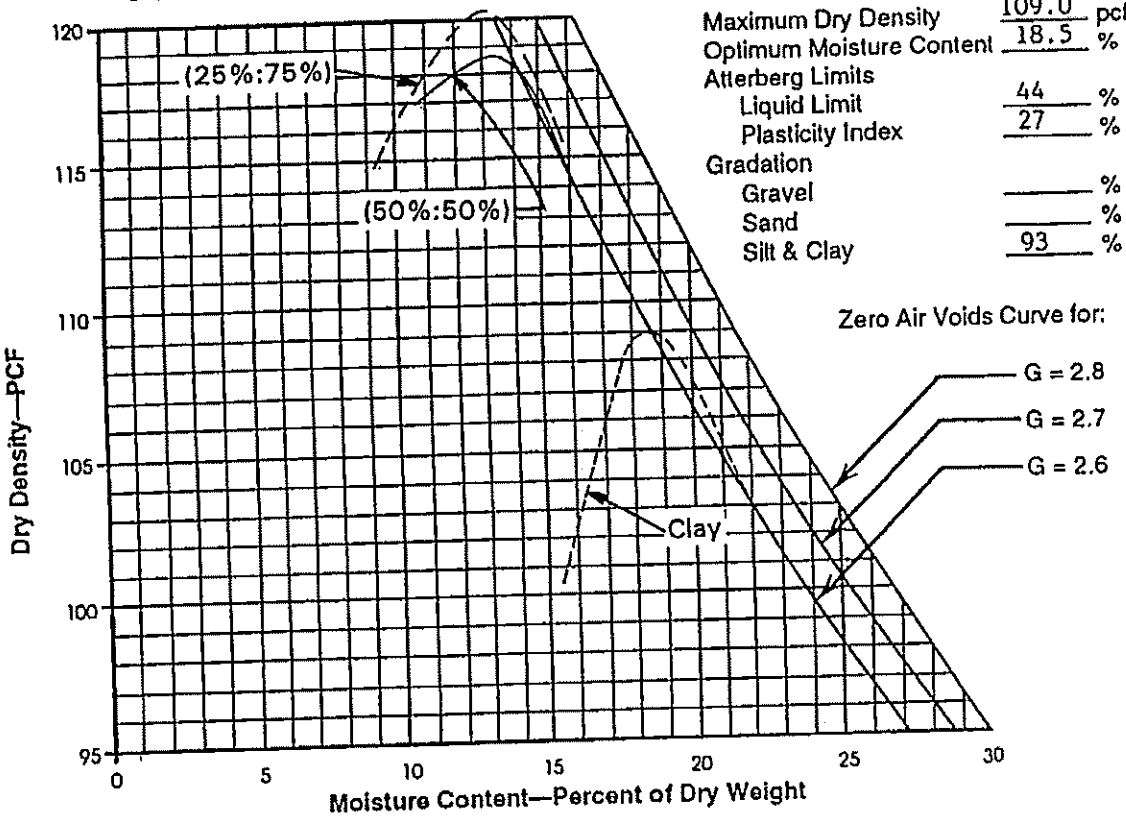
APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

James E. Nordquist, P.E.

JEN/cs

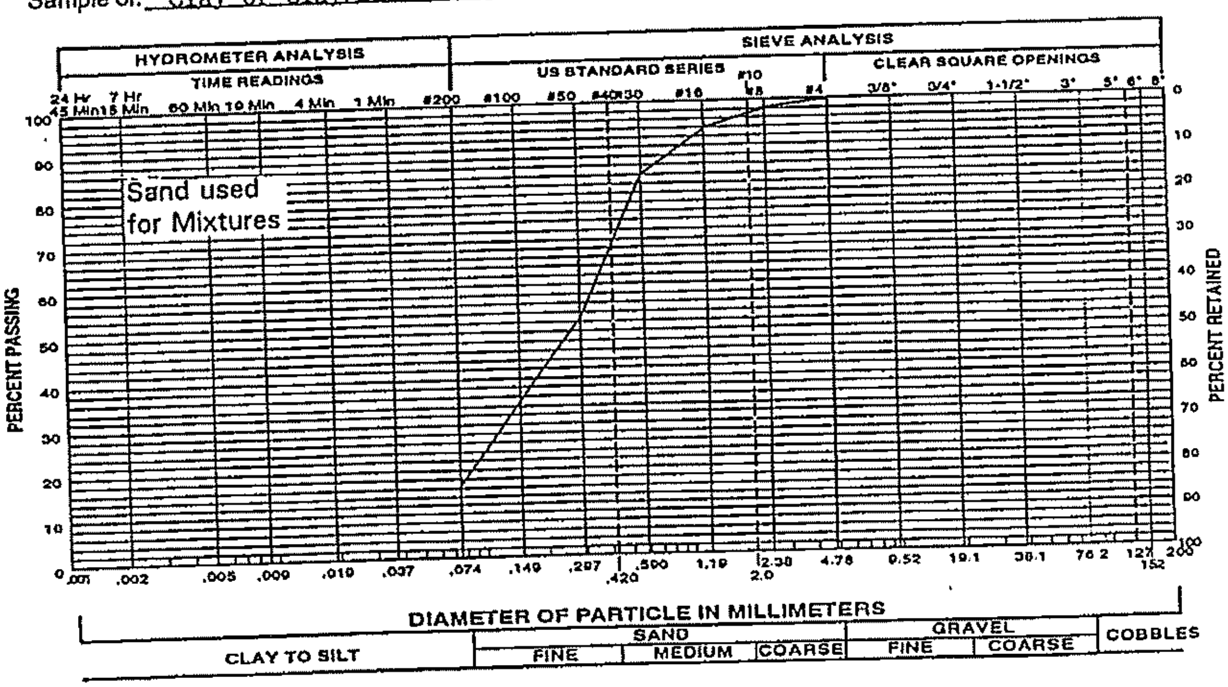
Applied Geotechnical Engineering Consultants, Inc.

Maximum Dry Density	109.0	pcf
Optimum Moisture Content	18.5	%
Atterberg Limits		
Liquid Limit	44	%
Plasticity Index	27	%
Gradation		
Gravel		%
Sand		%
Silt & Clay	93	%



Compaction Test Procedure ASTM D-698

Sample of: Clay or Clay/Sand Mixture From: Lone Mountain, Oklahoma

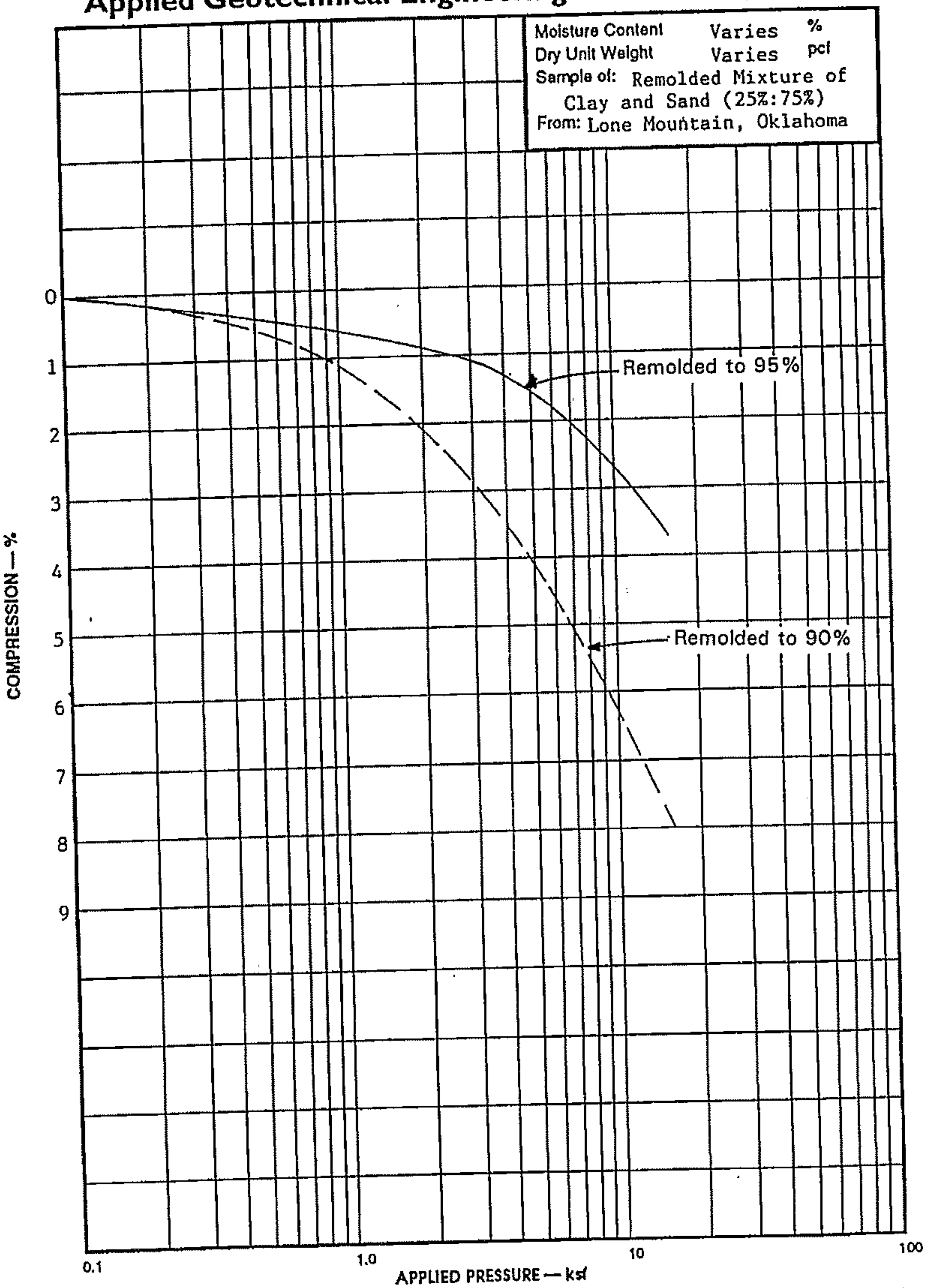


GRADATION & COMPACTION TEST RESULTS

Project No. 24292A

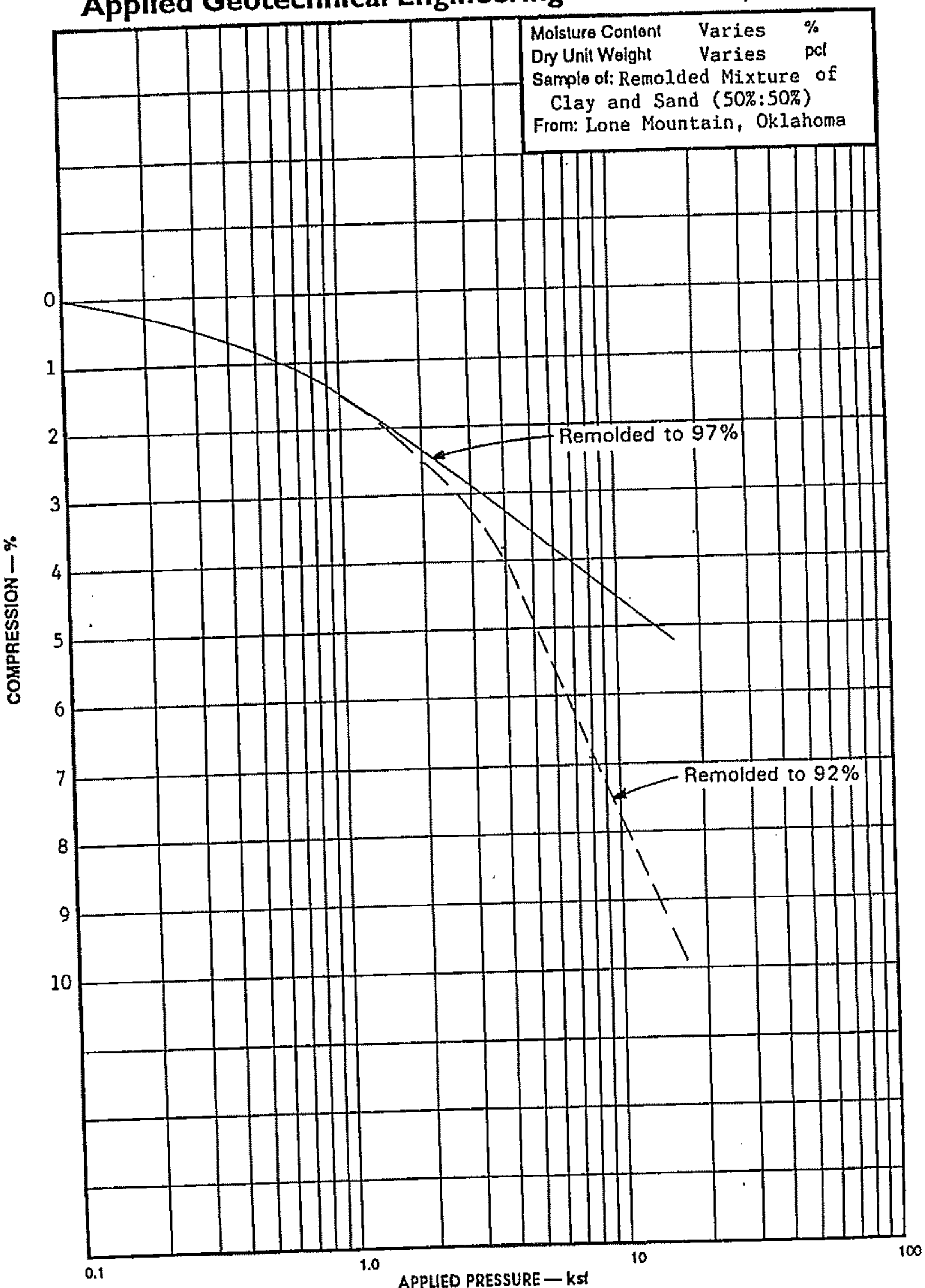
Figure 1

Applied Geotechnical Engineering Consultants, Inc.



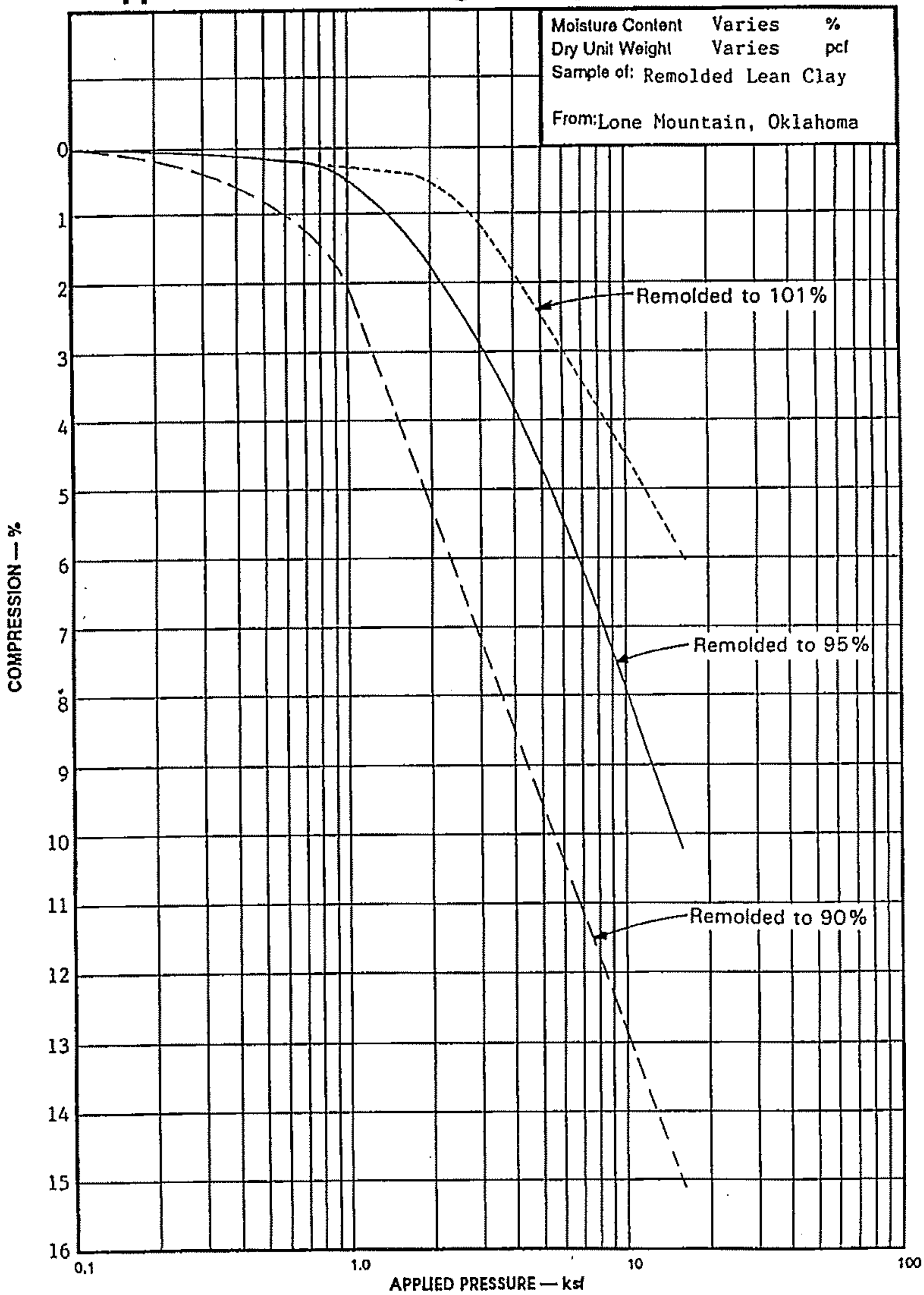
Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 2

Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 3

Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 4

EXHIBIT C

STORMWATER MANAGEMENT CALCULATIONS

Appendix 1 - Phase Division and Temporary Area Berms

Appendix 2 - Run-off Control

Appendix 3 - Embankment Erosion Protection

HA&L ENGINEERING

APPENDIX I

Phase Division and Temporary Area Berms

HA&L ENGINEERING

CLIENT USPCI
PROJECT Lone Mtn - Landfill Cell 15
FEATURE Setback waste from temporary Berms
PROJECT NO. 64-44-200

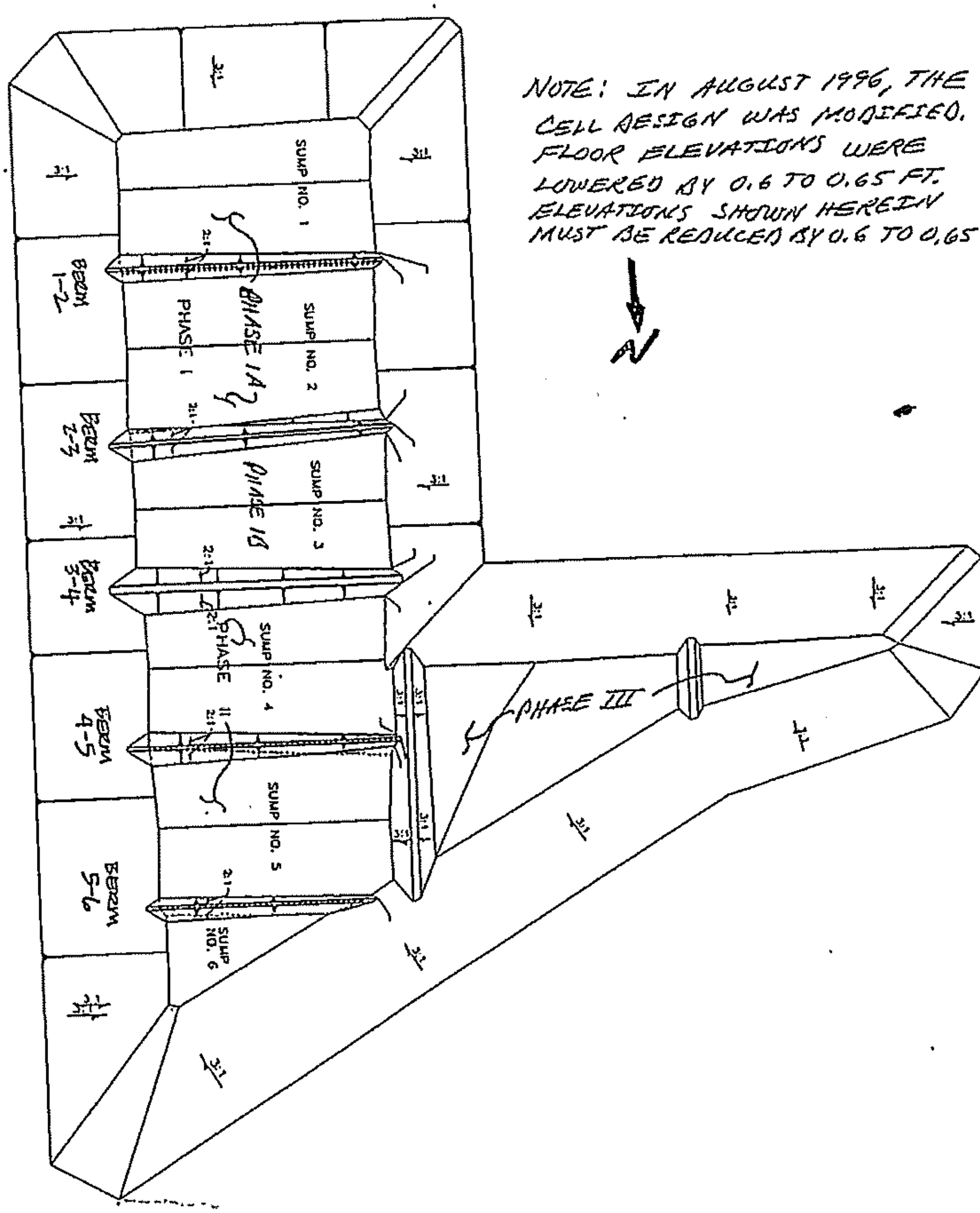
SHEET 1 OF 9
COMPUTED PLH
CHECKED KLS
DATE 3-10-93

Rev. 9/4/97

Purpose: Determine Run-off volume and required set back for operation of subcell areas within phases 1-3 of landfill cell #15.

The proposed layout is as follows:

NOTE: IN AUGUST 1996, THE CELL DESIGN WAS MODIFIED. FLOOR ELEVATIONS WERE LOWERED BY 0.6 TO 0.65 FT. ELEVATIONS SHOWN HEREIN MUST BE REDUCED BY 0.6 TO 0.65 FT.



The following shall be determined

1. Runoff volume for each sump area
2. Storage volume for Sump 1 active
3. " " " Sumps 1-2 active
4. " " " " 1-3 active
5. " " " " 1-4 active
6. " " " ½ sump 2 and 3-5 active (Sump 1 + ½ 2 closed)

The following assumptions are made:

1. Use 25 year, 24 hour precipitation event
2. 2' tertiary soil cover placed adjacent to temporary berms.
3. Use CN = 91 for waste areas.

1.) Runoff volume for each Sump area

Utilize the SCS Curve number methodology

P = 6.0 inches (25 year, 24 hour) { Technical Paper #40 "Rainfall frequency Atlas of the United States" Department of Commerce

CN = 91

Sump No	Area (ft²)	Area (acres)	P_N	Volume (acre-feet)
1	197,700	4.54	0.41	1.88
2	133,300	3.06	0.41	1.26
3	133,300	3.06	0.41	1.26
4	120,700	2.77	0.41	1.14
5	118,400	2.72	0.41	1.12

Note: Above runoff based on Curve Number Method

$$S = \frac{1000}{CN} - 10 \Rightarrow \frac{1000}{91} - 10 \Rightarrow 0.989$$

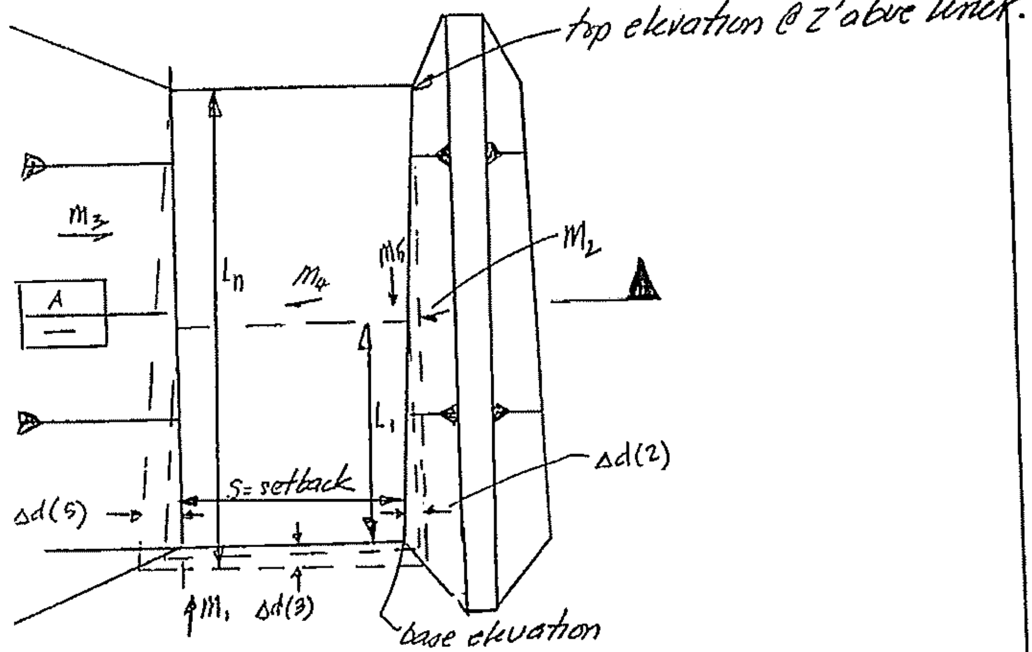
$$P_N = \frac{(P-0.2S)^2}{(P+0.8S)} \Rightarrow \frac{[6.0-0.2(.989)]^2}{[6.0+0.8(.989)]} \Rightarrow 4.96''$$

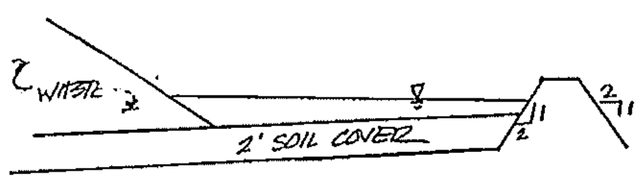
Effective runoff = 4.96 inches (25 year, 24 hour)
= 0.41 feet

2.) Storage Volume for Sump No 1 active

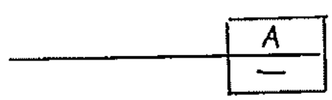
As shown above, Runoff volume for sump No 1 equals 1.88 acre-feet.

Develop spreadsheet program based upon sump parameters which will calculate stage-storage vs. width.





assume base elevation 2' above tertiary liner



area$_n$

$$L_n = \left[\frac{\text{Stage}_n\ \text{Elev} - \text{Base Elev}}{m_5} + \left(\frac{\text{Stage}_n - \text{base}}{\text{floor slope}} + S(m_4)/m_5\right)\right]/2 + (\text{Stage} - \text{base})$$

$$W_n = \frac{\text{Setback} + \left[\text{Setback} + (\text{Stage}_n - \text{base Elev})(m_2 + m_3)\right]}{2}$$

$$\text{area}_n = L_1 * W_1$$

Volume between area$_n$ and area$_{n+1}$ = $\frac{\text{area}_n + \text{area}_{n+1}}{2}(\text{Stage}_{n+1} - \text{Stg}$

See Attached computer run sheets

ULATION OF STORAGE VOLUME -- LANDFILL CELL 15

CONTRIBUTING SUMPS   TO BERM 1-2

SUMP AREA NO. 1=   1.88
SUMP AREA NO. 2=   0
SUMP AREA NO. 3=   0
SUMP AREA NO. 4=   0
SUMP AREA NO. 5=   0

TOTAL=   1.88 AF

INPUT

M1=   3
M2=   3
M3=   5
M4=   0.01
M5=   0.01
BASE ELEV=   1375.6 (TOP OF SAND)
TOP ELEV=   1379.1 (TOP OF SAND)
TOP OF BERM=   1382.5 (W/1' FREEBOARD)
SETBACK=   35 (FROM TOE OF BERM)

HEIGHT OF BERM AT BASE   6.9 FT
HEIGHT OF BERM AT TOP   3.4 FT
ABOVE TOP OF SOIL COVER

STAGE ELEV	AVG LENGTH (FT)	AVG WIDTH (FT)	AREA (FT^2)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
1375.6	0	0	0	0.00	0.00
1376.0	58.7	36.4	2,136.7	0.02	0.02
1376.5	109.0	38.2	4,158.4	0.05	0.07
1377.0	159.0	39.9	6,344.1	0.07	0.14
1377.5	209.0	41.7	8,704.9	0.10	0.24
1378.0	259.0	43.4	11,240.6	0.13	0.37
1378.5	309.0	45.2	13,951.4	0.16	0.53
1379.0	370.4	46.9	17,371.8	0.20	0.73
1379.5	373.4	48.7	18,165.9	0.21	0.94
1380.0	376.4	50.4	18,970.6	0.22	1.16
1380.5	379.4	52.2	19,785.7	0.23	1.39
1381.0	382.4	53.9	20,611.4	0.24	1.62
1381.5	385.4	55.7	21,447.5	0.25	1.87
1382.0	388.4	57.4	22,294.2	0.26	2.12

3.) Storage Volume for Sumps 1-2 Active

CALCULATION OF STORAGE VOLUME - LANDFILL CELL 15

CONTRIBUTING SUMPS 1-2 TO BERM 2-3

SUMP AREA NO. 1= 1.88
SUMP AREA NO. 2= 1.26
SUMP AREA NO. 3= 0
SUMP AREA NO. 4= 0
SUMP AREA NO. 5= 0

TOTAL= 3.14 AF

INPUT

M1= 3
M2= 4
M3= 5
M4= 0.01
M5= 0.01
BASE ELEV= 1374.6 (TOP OF SAND)
TOP ELEV= 1377.9 (TOP OF SAND)
TOP OF BERM= 1382.0 (W/1' FREEBOARD)
SETBACK= 40 (FROM TOE OF BERM)

HEIGHT OF BERM AT BASE 7.4 FT
HEIGHT OF BERM AT TOP 4.1 FT
ABOVE TOP OF SOIL COVER

STAGE ELEV	AVG LENGTH (FT)	AVG WIDTH (FT)	AREA (FT^2)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
1374.6	0	0	0	0.02	0.02
1375.0	71.2	61.4	4,371.7	0.04	0.06
1375.5	121.5	63.2	7,672.7	0.09	0.14
1376.0	171.5	64.9	11,130.4	0.13	0.27
1376.5	221.5	66.7	14,763.0	0.17	0.44
1377.0	271.5	68.4	18,570.6	0.21	0.66
1377.5	347.4	70.2	24,370.1	0.28	0.93
1378.0	350.4	71.9	25,193.8	0.29	1.22
1378.5	353.4	73.7	26,027.9	0.30	1.52
1379.0	356.4	75.4	26,872.6	0.31	1.83
1379.5	359.4	77.2	27,727.7	0.32	2.15
1380.0	362.4	78.9	28,593.4	0.33	2.48
1380.5	365.4	80.7	29,469.5	0.34	2.82
1381.0	368.4	82.4	30,356.2	0.35	3.16

4.) Storage Volume for Sumps 1-3 active

CALCULATION OF STORAGE VOLUME - LANDFILL CELL 15

CONTRIBUTING SUMPS 1-3 TO BERM 3-4

SUMP AREA NO. 1= 1.88
SUMP AREA NO. 2= 1.26
SUMP AREA NO. 3= 1.26
SUMP AREA NO. 4= 0
SUMP AREA NO. 5= 0

TOTAL= 4.40 AF

INPUT

M1= 3
M2= 2
M3= 5
M4= 0.01
M5= 0.02
BASE ELEV= 1373.3 (TOP OF SAND)
TOP ELEV= 1380.4 (TOP OF SAND)
TOP OF BERM= 1383.4 (W/1' FREEBOARD)
SETBACK= 60 (FROM TOE OF BERM)

HEIGHT OF BERM AT BASE 10.1 FT
HEIGHT OF BERM AT TOP 3 FT
ABOVE TOP OF SOIL COVER

STAGE ELEV	AVG LENGTH (FT)	AVG WIDTH (FT)	AREA (FT^2)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
1373.3	0	0	0	0.01	0.01
1375.0	105.1	66.0	6,931.3	0.27	0.28
1375.5	126.5	67.7	8,564.1	0.10	0.38
1376.0	151.5	69.5	10,521.7	0.12	0.50
1376.5	176.5	71.2	12,566.8	0.14	0.64
1377.0	201.5	73.0	14,699.4	0.17	0.81
1377.5	226.5	74.7	16,919.6	0.19	1.01
1378.0	251.5	76.5	19,227.2	0.22	1.23
1378.5	276.5	78.2	21,622.3	0.25	1.47
1379.0	301.5	80.0	24,104.9	0.28	1.75
1379.5	326.5	81.7	26,675.1	0.31	2.06
1380.0	395.2	83.5	32,979.4	0.38	2.44
1380.5	398.2	85.2	33,926.6	0.39	2.82
1381.0	401.2	87.0	34,884.3	0.40	3.23
1381.5	404.2	88.7	35,852.5	0.41	3.64
1382.0	407.2	90.5	36,831.2	0.42	4.06
1382.4	409.6	91.9	37,621.8	0.35	4.40

7/

5) Storage Volume for Sumps 1-4 active

CALCULATION OF STORAGE VOLUME - LANDFILL CELL 15

CONTRIBUTING SUMPS 1-4 TO BERM 4-5

SUMP AREA NO. 1= 1.88
SUMP AREA NO. 2= 1.26
SUMP AREA NO. 3= 1.26
SUMP AREA NO. 4= 1.14
SUMP AREA NO. 5= 0

TOTAL= 5.54 AF

INPUT

M1= 3
M2= 2
M3= 5
M4= 0.01
M5= 0.02
BASE ELEV= 1370.6 (TOP OF SAND)
TOP ELEV= 1377.4 (TOP OF SAND)
TOP OF BERM= 1381.0 (W/1' FREEBOARD)
SETBACK= 50 (FROM TOE OF BERM)

HEIGHT OF BERM AT BASE 10.4 FT
HEIGHT OF BERM AT TOP 3.6 FT
ABOVE TOP OF SOIL COVER

STAGE ELEV	AVG LENGTH (FT)	AVG WIDTH (FT)	AREA (FT^2)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
1370.6	0	0	0	0.02	0.02
1371.0	41.2	81.4	3,353.7	0.03	0.05
1371.5	66.5	83.2	5,529.5	0.06	0.11
1372.0	91.5	84.9	7,768.4	0.09	0.20
1372.5	116.5	86.7	10,094.7	0.12	0.32
1373.0	141.5	88.4	12,508.6	0.14	0.46
1373.5	166.5	90.2	15,010.0	0.17	0.63
1374.0	191.5	91.9	17,598.9	0.20	0.84
1374.5	216.5	93.7	20,275.2	0.23	1.07
1375.0	241.5	95.4	23,039.1	0.26	1.33
1375.5	266.5	97.2	25,890.5	0.30	1.63
1376.0	291.5	98.9	28,829.4	0.33	1.96
1376.5	316.5	100.7	31,855.7	0.37	2.33
1377.0	378.4	102.4	38,748.2	0.44	2.77
1377.5	381.4	104.2	39,722.8	0.46	3.23
1378.0	384.4	105.9	40,708.0	0.47	3.70
1378.5	387.4	107.7	41,703.6	0.48	4.17
1379.0	390.4	109.4	42,709.8	0.49	4.66
1379.5	393.4	111.2	43,726.4	0.50	5.17
1380.0	396.4	112.9	44,753.6	0.51	5.68

:) Storage Volume for ½ Sump 2, 3-5 open (1 + ½ area 2 closed) 8/9

LATION OF STORAGE VOLUME - LANDFILL CELL 15

ONTRIBUTING SUMPS1/2 SUMP 2, 3,4,5 Berm 5-6

SUMP AREA NO. 1= 0
SUMP AREA NO. 2= 0.63
SUMP AREA NO. 3= 1.26
SUMP AREA NO. 4= 1.14
SUMP AREA NO. 5= 1.12

TOTAL= 4.15 AF

NPUT

M1= 3
M2= 2
M3= 5
M4= 0.01
M5= 0.01
BASE ELEV= 1368.6 (TOP OF SAND)
TOP ELEV= 1371.4 (TOP OF SAND)
TOP OF BERM= 1377.3 (W/1' FREEBOARD)
SETBACK= 70 (FROM TOE OF BERM)

HEIGHT OF BERM AT BASE 8.7 FT } above soil cover
HEIGHT OF BERM AT TOP 5.9 FT

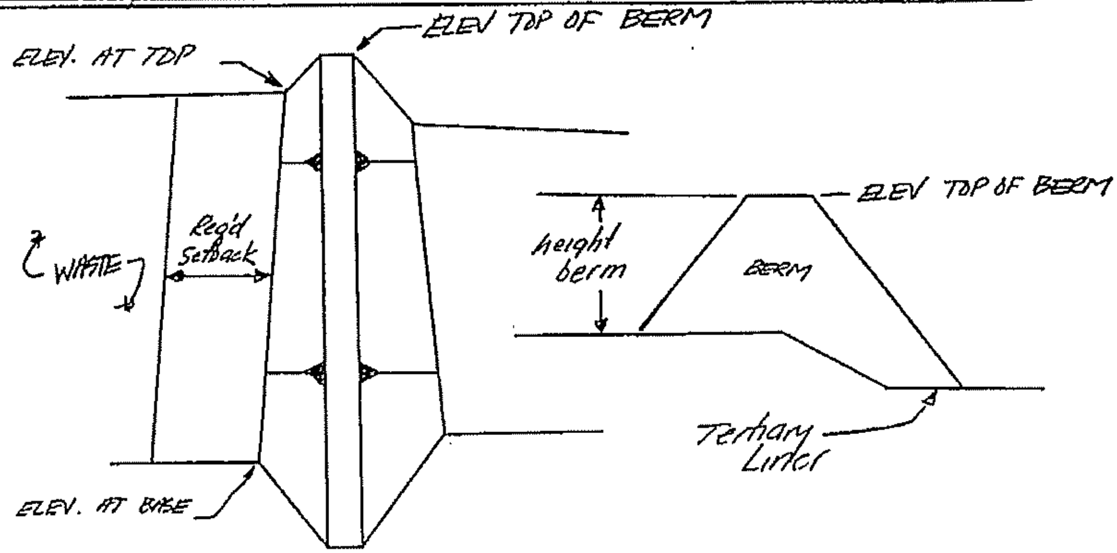
STAGE ELEV	AVG LENGTH (FT)	AVG WIDTH (FT)	AREA (FT^2)	INCREMENTAL VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)
1368.6	0	0	0	0.03	0.03
1369.0	76.2	71.4	5,440.7	0.05	0.08
1369.5	126.5	73.2	9,253.5	0.11	0.18
1370.0	176.5	74.9	13,219.9	0.15	0.33
1370.5	226.5	76.7	17,361.2	0.20	0.53
1371.0	294.4	78.4	23,081.0	0.26	0.80
1371.5	297.4	80.2	23,836.6	0.27	1.07
1372.0	300.4	81.9	24,602.8	0.28	1.35
1372.5	303.4	83.7	25,379.4	0.29	1.65
1373.0	306.4	85.4	26,166.6	0.30	1.95
1373.5	309.4	87.2	26,964.2	0.31	2.26
1374.0	312.4	88.9	27,772.4	0.32	2.57
1374.5	315.4	90.7	28,591.0	0.33	2.90
1375.0	318.4	92.4	29,420.2	0.34	3.24
1375.5	321.4	94.2	30,259.8	0.35	3.59
1376.0	324.4	95.9	31,110.0	0.36	3.94
1376.1	325.0	96.3	31,281.3	0.07	4.02
1376.2	325.6	96.6	31,453.0	0.07	4.09
1376.3	326.2	96.9	31,625.1	0.07	4.16

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Lone Mtn - Landfill Cell 15
FEATURE: Setback Waste from temporary berms
PROJECT NO.: 64-44-200

SHEET 9 OF 9
COMPUTED: PGH
CHECKED: KCS
DATE: 3-11-93

Summary of Temporary Berm, Runoff Volume and Required setback.



BERM NO.	DESIGN BERM ELEV	HEIGHT BERM AT BASE (above tert)	HEIGHT BERM AT TOP (above tert)	TRIBUTARY AREAS	TRIBUTARY VOLUME (AF)	REQUIRED SETBACK (ft)
1-2	1382.5	8.9	5.4	1	1.88	35
2-3	1382.0	9.4	6.1	1, 2	3.14	60'
3-4	1383.4	12.1	5.0	1,2,3	4.40	60'
4-5	1381.0	12.4	5.6	1,2,3,4	5.54	80'
5-6	1377.3	10.7	7.9	½ area 2 3,4,5	4.15	70'

Notes:
- Above values allow for 1.0' min freeboard, 25 year 24 hour precipitation event.
- height of berm indicates height above tertiary liner. preceding analysis were based on volume capacity of berm w/ 2' soil cover in place.
- Berm 5-6, above, assumes that USPCI will have sump area 1 and ½ of area 2 closed prior to this point. Cap drainage water to be directed away from active portions of cell.

APPENDIX 2

Run-off Control

HA&L ENGINEERING

CLIENT: USPLF
PROJECT: CELL 15 CLOSURE
FEATURE: DEPTH OF PERIMETER DITCHES
PROJECT NO. 64.14.200

SHEET 1 OF 10
COMPUTED: JAH
CHECKED: KCS
DATE: 4/1/93

DETERMINE REQUIRED DEPTH OF PERIMETER DITCHES

RUN-OFF AREA (SUMP 1) = 360' x 325' = 117,000 ft^2

= 2.69 ACRES

RAINFALL = 6" 25yr · 24hr.

DETERMINE (T_c) TIME OF CONCENTRATION

$T_1 + T_2 = T_c$

$$T_1 = \frac{0.007(nL)^{0.8}}{P^{1/2}\ S^{0.4}}$$ (FROM SCS, TR-55)

Cap
L = 300'
n = 0.024 (Manning's)
P = 6" (25yr-24hr)
S = 5%

= 0.05 Hrs

$T_2 = D/V$. Assume Velocity = 2.6 fps in perimeter ditch
D = 290'

$T_2 = 290/2.6 = 111.5$ sec = 0.03 Hrs

$T_c = 0.08$ Hrs

AVERAGE SLOPE = 300' @ 5% = ΔELEV = 15'
290' @ 0.5% = 1.45'
590 16.45'

SLOPE = 2.7%

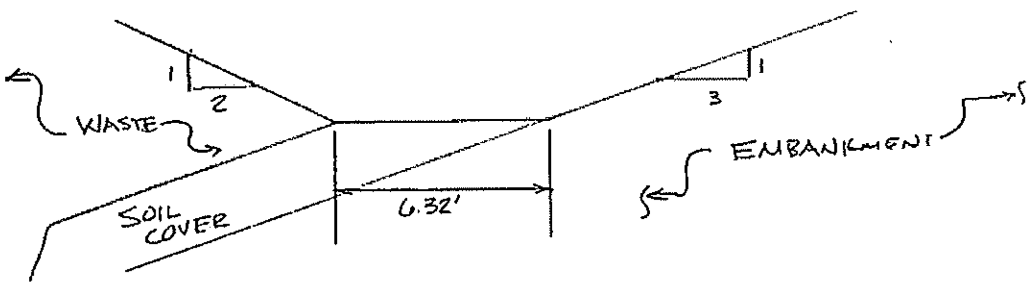
FROM HYDRO (SEE ATTACHED SHEET) Q = 11.69 cfs

HA&L ENGINEERING

CLIENT USPCI LONE MT.
PROJECT CELL 15 CLOSURE
FEATURE DEPTH OF PERIMETER DITCHES
PROJECT NO. 64.44.200

SHEET 2 OF 10
COMPUTED JAH
CHECKED KCS
DATE 4/1/93

CROSS-SECTION OF PERIMETER DITCHES



Flow (Q, cfs)	n	s	Depth	Side 1 (H:1V)	Side 2 (H:1V)	Area	WP	R	Slope
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Velocity (fps) [illegible]

DEPTH OF FLOW = 0.6' 1'-0" REQUIRED FREE BOARD

DITCH DEPTH REQUIRED = 1.6'

HA&L ENGINEERING	CLIENT: USPCI, Lone Mt.
	PROJECT: Cell K Closure
	FEATURE: Depth of Perimeter Ditches
	PROJECT NO. 64.44.200
	SHEET 3 OF 10
	COMPUTED JAH
	CHECKED KCS
	DATE 4/1/93

Run-off Area, Sumps 1, 2, 3, 4, & Portal 5

$1145 \times 325 = 372{,}125\ ft^2 = \underline{8.54\ Acres}$

<u>AVERAGE SLOPE</u>

300'	@ 5%	ΔELEV =	15'
950'	@ 0.5%	ΔELEV	4.75
Σ 1250'			19.75

$19.75/1250 = \underline{1.58\%}$

<u>TIME OF CONCENTRATION</u>

$T_1 = 0.05$ Hrs (See Sheet 1)

$T_2 =$ (Assume V = 3.8 fps) $950/3.8 = 0.07$ Hrs

$T_c = \underline{0.12\ Hrs}$

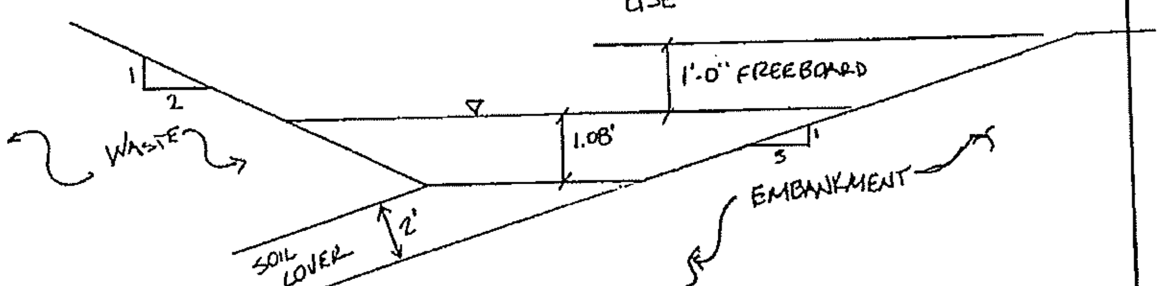
FROM Hydro (SEE ATTACHED SHEET) Q = 37.21 cfs

Flow (Q, cfs)	[illegible]	n	Depth	Side 1 2H:1V	Side 2 3H:1V	Area	WP	R	Slope
37.21	1.49	0.0240	1.08	2.00	3.00	[illegible]	[illegible]	[illegible]	[illegible]
				2.24	3.16				

Velocity (fps) 3.80

DEPTH OF DITCH REQUIRED = 2.08'

USE



PROJECT : USPCI. LONE MT. CELL 15 CLOSURE 25yr-24hr
DEPTH OF PERIMETER DITCHES, SUMP 1

AREA= 2.7 ACRES
AVERAGE BASIN SLOPE= 2.7 PERCENT
CURVE NUMBER= 91.0
DESIGN STORM= 6.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 590. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .08 HOURS

TP= .0533 HOURS QPCFS= 37.86 CFS QPIN=14.0620 INCHES
C3= 69.3116 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
11.89	3.4784	2.5207	.0767	.0	11.52
11.91	3.5596	2.5976	.0769	.0	11.56
11.93	3.6408	2.6747	.0771	.0	[illegible]
11.94	3.7219	2.7519	.0772	.0	[illegible]
11.96	3.8031	2.8292	.0773	.0	[illegible]
11.98	3.8843	2.9067	.0775	.0	[illegible]
12.00	3.9655	2.9843	.0776	.0	[illegible]
12.02	3.9910	3.0087	.0244	.0	[illegible]
12.03	4.0064	3.0235	.0147	.0	[illegible]
12.05	4.0218	3.0382	.0147	.0	[illegible]
12.07	4.0371	3.0529	.0147	.0	5.35
12.09	4.0525	3.0676	0147	.0	3.92
12.10	4.0679	3.0824	.0147	.0	3.07

HYDROGRAPH PEAK= 11.69 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= 1.10 Acre-Feet

PROJECT : USPCI, LONE MT., CELL 15 CLOSURE, PERIMETER DITCHES
Sumps 1, 2, 3, 4, & Partial 5 — 25yr 24 hr.

AREA= 8.5 ACRES
AVERAGE BASIN SLOPE= 1.6 PERCENT
CURVE NUMBER= 91.0
DESIGN STORM= 6.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 1250. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .12 HOURS

[illegible] .0800 HOURS QP[illegible]= 80.73 CFS QPIN= 9.3747 [illegible]
[illegible] .2077 [illegible]ATIONS [illegible] [illegible] 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
11.90	3.5398	2.5788	.0691	.0	36.72
11.92	3.6128	2.6481	.0692	.0	36.81
11.94	3.6857	2.7174	.0693	.0	36.90
11.95	3.7587	2.7868	.0695	.0	36.99
11.97	3.8315	2.8564	.0696	.0	37.06
11.98	3.9046	2.9261	.0697	.0	37.14
12.00	3.9776	2.9959	.0698	.0	37.21
12.02	3.9917	3.0094	.0136	.0	37.05
12.03	4.0056	3.0227	.0132	.0	35.70
12.05	4.0194	3.0359	.0132	.0	32.74
12.06	4.0332	3.0491	.0132	.0	28.60
12.08	4.0470	3.0624	.0132	.0	24.07
12.10	4.0609	3.0756	.0132	.0	19.82

HYDROGRAPH PEAK= 37.21 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= 3.53 Acre-Feet

HA&L ENGINEERING

CLIENT	USPCI Lone Mt.	SHEET	6 OF 10
PROJECT	Cell 15 Closure	COMPUTED	JAH
FEATURE	Depth of Perimeter Ditches	CHECKED	KCS
PROJECT NO.	64.44.200	DATE	4/5/93

EVALUATE THE 100yr-24hr EVENT = 8"

SUMP 1

$T_c = T_1 + T_2$

$T_1 = 0.04$ Hrs

$T_2 = (\text{Assume } V = 3\text{ fps}) = 0.03$

$T_c = 0.07$

From Hydro (See Attached Sheet) $Q_p = 16.08$ cfs

Depth of Flow = 0.68 Use 0.7'

Flow (Q cfs)	b	n	Depth	Side 1 (H:1V)	Side 2 (H:1V)	Area	WP	R	Slope
16.08	[illegible]	0.0240	0.68	2.00	3.00	[illegible]	[illegible]	[illegible]	[illegible]
				[illegible]	[illegible]				

Velocity (fps) [illegible]

FREEBOARD, CHANNEL DEPTH = 1.6'
- 0.7

FREEBOARD 0.9'

HA&L ENGINEERING	CLIENT: USPCI, LONE MT.
	PROJECT: CELL 15 CLOSURE
	FEATURE: DEPTH OF PERIMETER DITCHES
	PROJECT NO. 64.44.200
	SHEET 7 OF 10
	COMPUTED: JAH
	CHECKED: KCC
	DATE: 4/1/93

SUMPS 1, 2, 3, 4, & Partial 5

$T_1 = 0.04$ Hrs

$T_2 =$ (ASSUME V = 4.2 fps) $= 950/4.2 = $ 0.06 Hrs

$T_c =$ 0.10 Hrs

FROM Hydro (SEE ATTACHED SHEET) $Q_p = 50.70$ cfs

DEPTH OF FLOW = 1.28'
DEPTH OF CHANNEL = 2.08

FREEBOARD 0.80'

Flow (cfs)	b	n	Depth	Side 1 (H:V)	Side 2 (H:V)	Area	WP	R	Slope
50.70	[illegible]	[illegible]	1.28	2.00	3.00	[illegible]	[illegible]	[illegible]	[illegible]
				[illegible]	[illegible]				

Velocity (fps) [illegible]

8/10

PROJECT : USPCI, LONE MT. CELL 15 CLOSURE, 100 YR, PERIMETER DITCHES

64.44.200
SUMP 1-

AREA= 2.7 ACRES
AVERAGE BASIN SLOPE= 2.7 PERCENT
CURVE NUMBER= 91.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 590. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .07 HOURS

TP= .0467 HOURS QPCFS= 43.75 CFS QPIN=16.0709 INCHES
C3= 79.2132 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
11.90	4.7136	3.7045	.0917	.0	[illegible]
11.92	4.8085	3.7963	.0918	.0	[illegible]
11.93	4.9033	[illegible]	.0919	.0	[illegible]
11.95	4.9982	[illegible]	.0920	.0	[illegible]
11.97	5.0930	4.0724	.0921	.0	[illegible]
11.98	5.1879	4.1646	.0922	.0	[illegible]
[illegible]	5.2827	4.2569	.0923	.0	[illegible]
[illegible]	5.3179	4.2912	.0343	.0	[illegible]
[illegible]	5.3359	4.3087	.0175	.0	[illegible]
12.04	5.3539	4.3262	.0175	.0	10.[illegible]
12.06	5.3719	4.3437	.0175	.0	7.52
12.07	5.3898	4.3613	.0175	.0	5.49
12.09	5.4078	4.3788	.0175	.0	4.2[illegible]

HYDROGRAPH PEAK= 16.08 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= 1.56 Acre-Feet

PROJECT : USPCI. LONE MT. CELL 15 CLOSURE, 100 YR, PERIMETER DITCHES

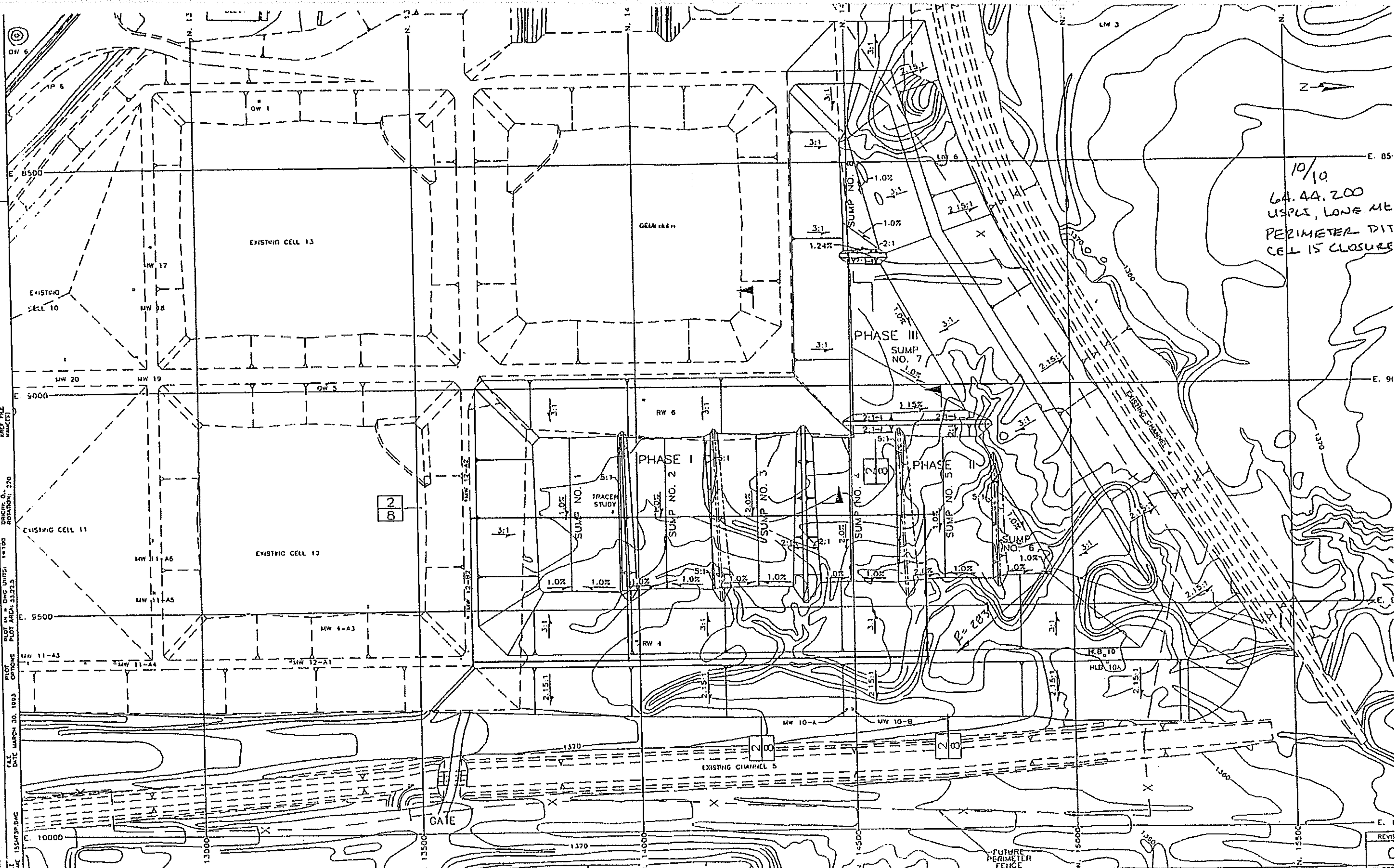
64.44.200
SUMPS 1, 2, 3, 4,
& Partial-5

AREA= 3.5 ACRES
AVERAGE BASIN SLOPE= 1.6 PERCENT
CURVE NUMBER= 91.0
DESIGN STORM= [illegible].00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 1250. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .10 HOURS

TP= .0667 HOURS QPCFS= 76.88 CFS QPIN=11.2476 INCHES
C3= 55.4492 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
11.89	4.6371	3.6305	.0981	.0	50.27
11.91	4.7386	3.7287	.0982	.0	50.35
11.92	4.8401	3.8270	.0983	.0	50.4[illegible]
11.94	4.9417	[illegible]	.0985	.0	50.5[illegible]
[illegible]	[illegible]	[illegible]	.0986	.0	50.[illegible]
11.97	[illegible]	[illegible]	.0987	.0	[illegible]
11.99	[illegible]	[illegible]	.0988	.0	50.70
[illegible]	[illegible]	[illegible]	.0641	.0	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	.0	[illegible]
[illegible]	[illegible]	[illegible]	.0187	.0	[illegible]
12.04	5.3700	[illegible]	.0187	.0	[illegible]
12.07	5.3893	4.3607	.0187	.0	23.82
12.09	5.4085	4.3795	.0187	.0	22.50

HYDROGRAPH PEAK= 50.70 cfs
TIME TO PEAK= 11.99 Hours
RUNOFF VOLUME= 4.93 Acre-Feet



HA&L ENGINEERING — CONSULTANTS ENGINEERS — Salt Lake City Utah

PROJECT ENGINEER

DESIGNED	MPW	3		
DRAFTED	RGA	2		
CHECKED	KCS	1		
DATE APRIL 1993		NO	DATE	REVISIONS

SCALE 1" = 100'

VERIFY SCALE — BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

USPCI — A Subsidiary Union Pacific Corporation

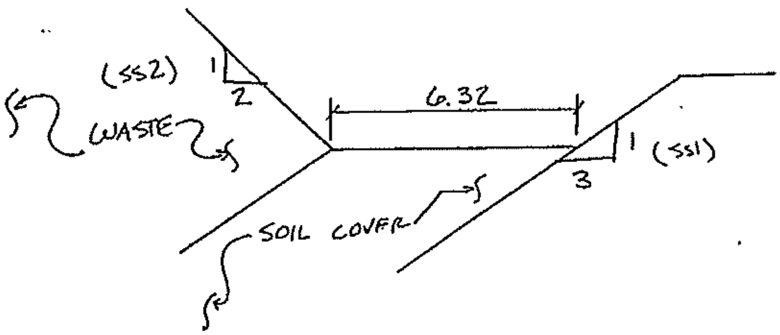
LONE MOUNTAIN FACILITY
LANDFILL CELL 15
SITE PLAN

SHEET 3 OF 3

64.44.

HA&L ENGINEERING	CLIENT: USPCI	SHEET 1 OF 6	
	PROJECT: CELL 15 CLOSURE, LONE MT.	COMPUTED: JAH	
	FEATURE: INTERIOR CONTAINMENT DITCH	CHECKED: KLS	
	PROJECT NO. 64.44.200	DATE: 8/10/93	

Perimeter Storage Requirements – Phase I closed, 2+3 open
– Phase I+II closed, 3 open



RUN-OFF AREA PHASE III (SCALE OF DRAWING)		Area (ft²)
	250 x 350	87,500
	350 x 100 x ½	17,500
	350 x 315	110,250
	315 x 190 x ½	29,925
	70 x 105 x ½	3,675
	105 x 415	43,575
	120 x 105 x ½	6,300
		298,725 ft²
		= 6.86 Acres

TOTAL DITCH LENGTH =
656
20
150
80
310
500
1716 ft

RUN OFF AREA PHASE II		
	605 x 540	326,700
	110 x 105 x ½	5,775
	320 x 540 x ½	86,400
		418,875 ft²
		= 9.62 Acres

TOTAL DITCH LENGTH
800
100
500
1400 ft

USPCI
CELL 15 CLOSURE. LONE MOUNTAIN
PROJECT NO. 64.44.200
11-Mar-93
JAH

CN	91	
100 yr-24 Hr (Pg)	8 inches	

S=(1000/CN)-10
Pn=(Pg-0.2S)^2/(Pg+0.8S)

S =	0.99	
Pn =	6.92 inches	
Side Slope 1 (ss1)	3 H	1 V
Side Slope 2 (ss2)	2 H	1 V

Phase I and Phase II have been capped
Run-off Area for Phase III

Run-off Area(III) =	6.87 Acres	299,257.20 Sq. Ft.
Run-off =	4.00 Acre-Ft	174,240.00 Cubic Ft.
Length of Ditches(III) =	1716 Ft.	

Area = (wd)+(.5(ss1d^2))+(.5(ss2d^2))

Width	Area	Volume	Water Depth	Depth + 1' Freebo
6.32	101.54	174,240	5.23	6.23
10	101.54	174,240	4.68	5.68
15	101.54	174,240	4.04	5.04
20	101.54	174,240	3.52	4.52

USPCI
CELL 15 CLOSURE, LONE MOUNTAIN
PROJECT NO. 64.44.200
11-Mar-93
JAH

CN	91	
100 yr-24 Hr (Pg)	8 inches	

S=(1000/CN)-10
Pn=(Pg-0.2S)^2/(Pg+0.8S)

S =	0.99	
Pn =	6.92 inches	
Side Slope 1 (ss1)	3 H	1 V
Side Slope 2 (ss2)	2 H	1 V

Phase I has been capped
Run-off Area for Phase II and Phase III

Run-off Area(II) =	9.62 Acres	419,047.20 Sq. Ft.
Run-off Area(III) =	6.87 Acres	299,257.20 Sq. Ft.
Total Run-off Area =	16.50 Acres	718,740.00 Sq. Ft.
Run-off =	9.52 Acre-Ft	414,739.97 Cubic Ft.
Length of Ditches(II) =	1400 Ft.	
Length of Ditches(III) =	1716 Ft.	
Total Length =	3116 Ft.	

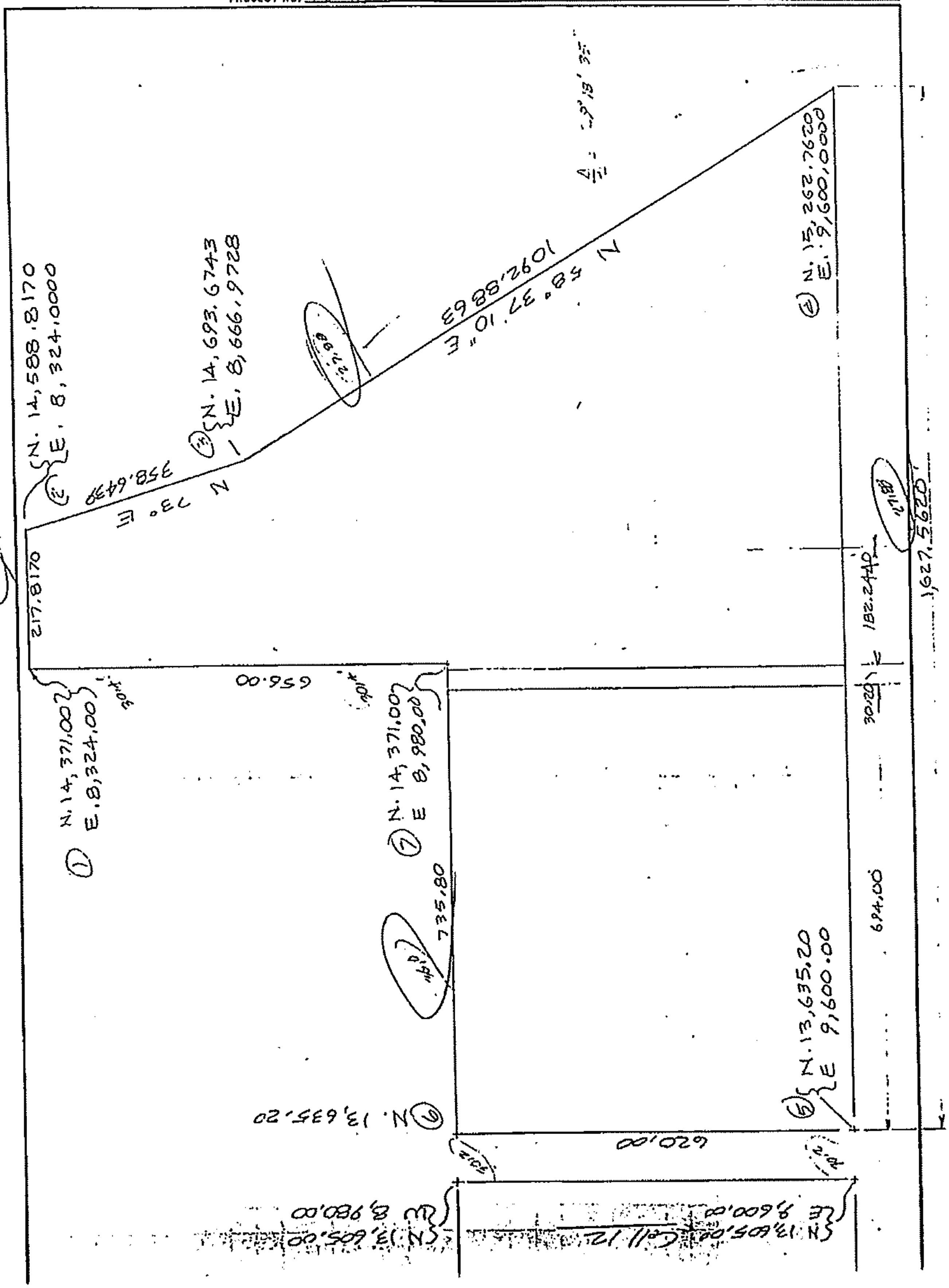
Area = (wd)+(.5(ss1d^2))+(.5(ss2d^2))

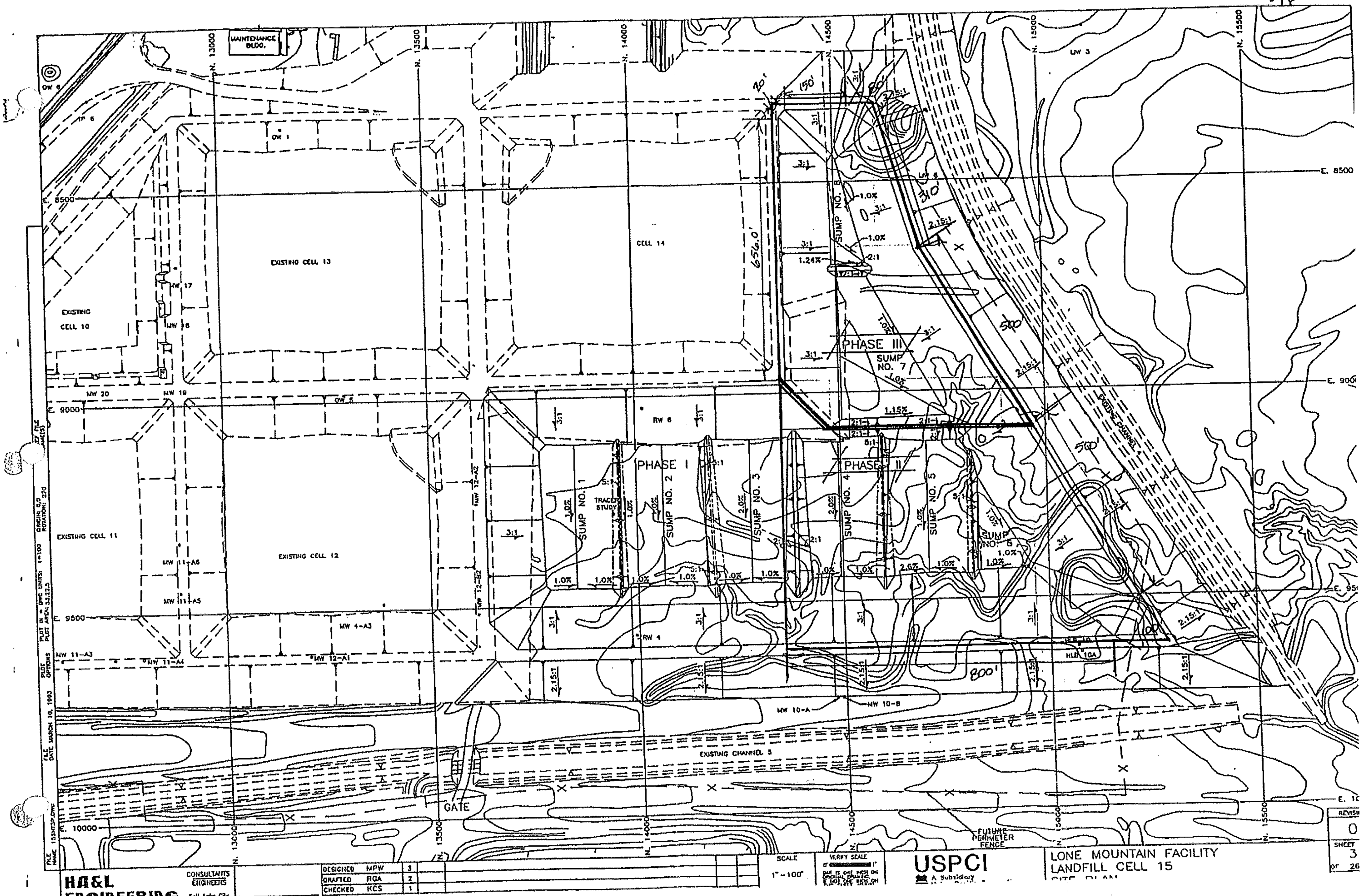
Width	Area	Volume	Depth	Depth + 1' Freebo
6.32	133.10	414,740	6.14	7.14
10	133.10	414,740	5.57	6.57
15	133.10	414,740	4.89	5.89
20	133.10	414,740	4.32	5.32

HA&L ENGINEERING

CLIENT ______
PROJECT ______
FEATURE Cell ______
PROJECT NO. 62-______

SHEET 4 OF 6
COMPUTED ______
CHECKED KCS
DATE 3-13-93




LONE MOUNTAIN FACILITY
LANDFILL CELL 15
USPCI
HA&L ENGINEERING
CONSULTANTS ENGINEERS
Salt Lake City
DESIGNED MPW
DRAFTED RGA
CHECKED KCS
SCALE 1" = 100'
VERIFY SCALE
SHEET 3
0
MAINTENANCE BLDG.
EXISTING CELL 10
EXISTING CELL 11
EXISTING CELL 12
EXISTING CELL 13
CELL 14
PHASE I
PHASE II
PHASE III
SUMP NO. 1
SUMP NO. 2
SUMP NO. 3
SUMP NO. 4
SUMP NO. 5
SUMP NO. 6
SUMP NO. 7
SUMP NO. 8
TRACER STUDY
EXISTING CHANNEL 5
EXISTING CHANNEL
GATE
FUTURE PERIMETER FENCE
N. 13000
N. 13500
N. 14000
N. 14500
N. 15000
N. 15500
E. 8500
E. 9000
E. 9500
E. 10000
656.0'
310'
500'
560'
800'
150'
20'
100'

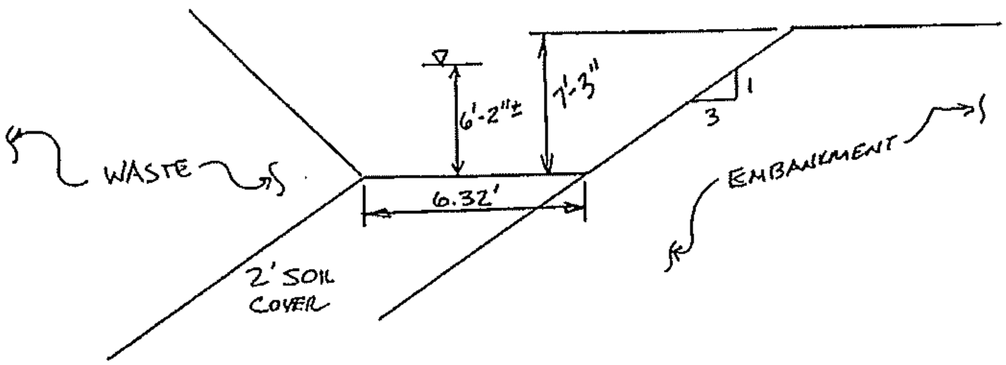
HA&L ENGINEERING

CLIENT: USPCI
PROJECT: CELL 15 CLOSURE, LONE MT.
FEATURE: INTERIOR CONTAINMENT DITCHES
PROJECT NO. 64.44.200

SHEET 6 OF 6
COMPUTED: JAH
CHECKED: KCS
DATE: 3/10/93

Summary

USE DITCH BOTTOM WIDTH 6.32' - Use larger of ditch depths shown on sheets 2 & 3.



As shown on attached computation sheets, for a 100 year, 24 hour precipitation event, a ditch having the above configuration would be adequate (allowing 1' freeboard). Use above configuration for both phase II and III closure.

APPENDIX 3

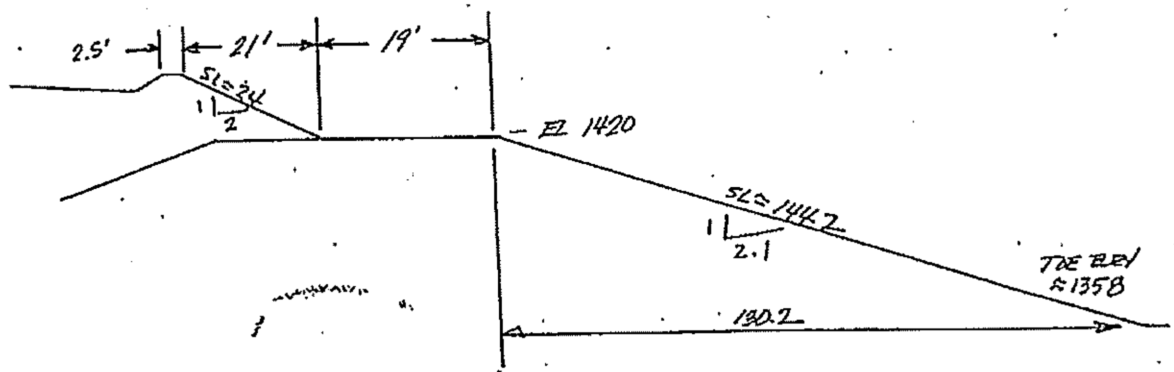
Embankment Erosion Protection

I. Design of sideslope erosion protection for the 2.1:1 (horizontal to vertical exterior slopes along the north and east sides of Landfill Cell 15.

The longest 2.1H:1V slope is at the northeast corner of Cell 15 goes from an elevation of 1420 feet at the top of the cell embankments to an elevation of about 1358 feet at the exterior toe of the embankment slopes. Thus the embankment is about 62 feet high at the highest point. The erosion protection is to consist of a 3-inch thick Type I granular filter blanket, a 4-inch thick Type II granular filter blanket and a required depth of riprap for the rock to be stable on the sideslopes with a reasonable safety factor. According to the information provided by Applied Geotechnical Engineering Consultants (AGEC), the two types of granular filter are required below the riprap for protection of the embankment sideslopes. The type I filter will be used to filter and hold the embankment soils in place and the Type II filter will be used to filter and hold the Type I filter material in place.

1. Hydrology

The surface area which contributes runoff to the sideslopes of the cell includes the sideslope of the cell itself, the top of the cell embankment, the 2:1 (horizontal to vertical) exterior slope of the cap, and the top of the berm around the perimeter of the cap. The dimensions of these areas are as follows:



The total horizontal distance over which precipitation would fall and that would be tributary to the side slope of the cell would be = 2.5' + ~21' + ~19' + ~131' = ~173.5 ft. The length along the embankment surfaces (according for slope lengths) is 2.5' + ~24' + ~19' + ~145' = ~190.5'.

Since the flow will be interflow in the rock itself, then the time of concentration is equal to the time for water to flow through the rock from the top to the bottom of the slope.

The velocity (V) of flow through the rock = ki/n, where k= permeability of the rock, i= the hydraulic gradient assumed to be equal to the slope of the sideslopes of the cell, and n= the porosity of the rock or filters. Chen-Northern tested the permeability of the Type I filter material to be 9 x 10^{-3} cm/sec. Thus, there will be very little flow in the Type I filter. Chen-Northern also tested the Type II filter material to have a permeability of 3.7 cm/sec = 0.121 ft/sec. Assume the porosity of the Type II filter to be 0.25.

Thus V in the Type II = 0.121 * 0.476 / 0.25 = 0.23 ft/sec.

The time of concentration T_c = slope length/V
= 190.5/0.23
= 828 sec. = 13.8 min. = 0.23 hrs.

Using the SCS Unit Hydrograph Procedure, the peak discharge Qp from 1 acre of area using the following data is 5.80 cfs (see attached computer printout).

Average basin slope	={2.5(0)+21(0.5)+19(0)+131(1/2.1)}/{2.5+21+19+131}
	= 42.0 percent
Curve Number	= 90
100-yr, 24-hr precipitation	= 8.0 inches
Storm Duration	= 24 hours
Hydraulic Length	= 190.5 ft.
Time of Concentration	= 0.23 hours

Checking the flow rate at 14%, one fourth, one half, three fourths, and full slope length gives:

14% of Slope Length:

The horizonal length along the slope for 1 acre of slope with a horizontal slope length of 14% of the total slope length (173.5 *0.14) = 24.29 ft.

= (43,560)/24.29 = 1793 ft.
Thus, q_p = 5.80 cfs / 1,793 ft = 0.0032 cfs/ft

One Fourth Slope Length:

The horizontal length along the slope for 1 acre of slope with a horizontal slope length of one fourth of the total slope length (173.5*0.25 = 43.38 ft) should be:

= (43,560)/43.38 = 1,004 ft.
Thus, q_p = 5.80 cfs / 1,004 ft = 0.0058 cfs/ft

One Half Slope Length:

The horizontal length along the slope for 1 acre of slope with a horizontal slope length of one half of the total slope length (173.5*0.5 = 86.75 ft) would be:

= (43,560)/86.75 = 502 ft.
Thus, q_p = 5.80 cfs / 502 ft = 0.0116 cfs/ft

HA&L ENGINEERING	CLIENT: LESI-Lone Mountain Facility PROJECT: RCRA Landfill Cell 15 FEATURE: Sideslope Erosion Protection PROJ. NO.: 64.44.200	SHEET 3 OF 10 COMPUTED: PGH CHECKED: KCS DATE: April 29, 1993

REVISED 10/14/97

PROJECT : 'USPCI Ln. Mtn.' Landfill Cell 15. Sideslope Erosion Protection

AREA= 1.0 ACRES
AVERAGE BASIN SLOPE= 42.0 PERCENT
CURVE NUMBER= 90.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 191. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .23 HOURS

TP= .1533 HOURS   QPCFS= 4.93 CFS   QPIN= 4.8911 INCHES
C3= 24.1084   ITERATIONS= 8   SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.39	.2199	.0000	.0000	.0	.00
2.42	.2233	.0000	.0000	.2	.00
2.45	.2268	.0000	.0000	1.5	.00
2.48	.2302	.0000	.0000	3.3	.00
2.51	.2332	.0001	.0000	4.5	.00
2.55	.2356	.0002	.0000	4.9	.00
2.58	.2381	.0002	.0000	4.6	.00
2.61	.2405	.0003	.0000	3.9	.00
2.64	.2430	.0004	.0000	3.1	.00
2.67	.2454	.0005	.0000	2.3	.00
2.70	.2479	.0006	.0001	1.6	.00
2.73	.2503	.0007	.0001	1.1	.00
2.76	.2528	.0008	.0001	.7	.00
2.79	.2553	.0010	.0001	.5	.00
2.82	.2577	.0011	.0001	.3	.00
2.85	.2602	.0013	.0002	.2	.00
2.88	.2626	.0014	.0002	.1	.00
2.91	.2651	.0016	.0002	.0	.00
11.84	4.3148	3.2188	.1776	.0	5.40
11.87	4.5013	3.3970	.1782	.0	5.53
11.90	4.6878	3.5758	.1788	.0	5.63
11.93	4.8742	3.7551	.1793	.0	5.70
11.96	5.0607	3.9348	.1797	.0	5.76
11.99	5.2471	4.1150	.1801	.0	5.80
12.02	5.3285	4.1938	.0788	.0	5.80
12.05	5.3639	4.2280	.0342	.0	5.66
12.08	5.3992	4.2622	.0342	.0	5.27
12.11	5.4345	4.2964	.0342	.0	4.68
12.14	5.4699	4.3307	.0342	.0	3.99
12.17	5.5052	4.3649	.0343	.0	3.31
12.21	5.5405	4.3992	.0343	.0	2.71

HYDROGRAPH PEAK= 5.80 cfs
TIME TO PEAK= 12.02 Hours
RUNOFF VOLUME= .57 Acre-Feet

Full Slope Length:

The horizontal length along the slope for 1 acre of slope with a horizontal slope of 173.5 would be:

$$= (43{,}560)/173.5 = 251 \text{ ft.}$$

Thus, $q_p = 5.80 \text{ cfs} / 251 \text{ ft} = 0.0231 \text{ cfs/ft}$

2. Required Riprap Thickness

According to Chen and Associates, for the riprap to be stable on the 2H:1V sideslopes of the cells with a S.F. of 1.5, the flow must occur in the lower 25% of the rock thickness, which must include the flow in the two filter materials. Since the lower filter (Type I) consists of a sandy material, with a minimum of 2% passing the #200 sieve, the permeability as determined by Chen-Northern is only 9×10^{-3} cm/sec = 3×10^{-4} ft/sec. The Type II filter is made up of larger gravel relatively free of fines. From tests conducted on the Type II filter materials by Chen-Northern, the permeability of the Type II material is 3.7 cm/sec = 0.121 ft/sec.

To determine the required rock thickness a seepage depth was calculated by applying Darcy's Law which states:

$$Q = kiA$$

Where:
Q = Flow rate,
k = permeability,
i = hydraulic gradient, and
A = flow area.

For a one foot flow width, the darcy's equation becomes:

$$q = kiy$$

Where: y = the flow depth.

If y_{fl} = the flow depth in the lower filter
y_{fu} = the flow depth in the upper filter, and
y_r = the flow depth in the rock riprap,

Then:

total flow depth $y_t = y_{fl} + y_{fu} + y_r$

Using the permeability of the Type I filter and Darcy's law, the flow that the Type I filter would carry would be:

$$q = (3 \times 10^{-4})*0.50*(4/12)$$
$$= 4.95 \times 10^{-5} \text{cfs/ft}$$

Then:

Total flow depth $y_t = y_n + y_{fu} + y_r$

Using the permeability of the Type I filter and Darcy's law, the flow that the Type I filter would carry would be:

$$q = (3 \times 10^{-4})*0.50*(3/12)$$

$$= 3.75 \times 10^{-5} \text{ cfs/ft}$$

Thus the Type I filter will carry very little flow.

Using the permeability for the Type II material and assuming this permeability to conservatively apply to the overlying rock riprap, then the flow depth above the Type I filter required to convey the peak flow rate at the 14%, one-fourth, one-half, three-fourths, and full slope length from the 100-year, 24-hour storm event would be:

14% of slope length:
$y = q/(ki) = 0.0032/(0.121*0.5) = 0.05$ ft $= 0.6$ inches

One-fourth slope length:
$y = q/(ki) = 0.0058/(0.121*0.476) = 0.1007$ ft $= 1.21$ inches

One-half slope length:
$y = q/(ki) = 0.0116/(0.121*0.476) = 0.2013$ ft $= 2.42$ inches

Three-fourth slope length:
$y = q/(ki) = 0.0173/(0.121*0.476) = 0.3002$ ft $= 3.62$ inches

Full slope length:
$y = q/(ki) = 0.0231/(0.121*0.476) = 0.4009$ ft $= 4.81$ inches

Assuming the Type I filter to be saturated, then the total required thickness of rock and filter (y_t), which must be five times the seepage depth to maintain a safety factor of 1.5 is:

14% of slope length:
$y_t = 5\ (y_n + y) = 5\ (3.0 + 0.64) = 18.2$ inches

One-fourth slope length:
$y_t = 5\ (y_n + y) = 5\ (3.0 + 1.21) = 21.05$ inches

One-half slope length:
$y_t = 5\ (y_n + y) = 5\ (3.0 + 2.42) = 27.1$ inches

Three-fourths slope length:

$y_t = 5\ (y_n + y) = 5\ (3.0 + 3.60) = 33.0$ inches

Full slope length:

$y_t = 5\ (y_n + y) = 5\ (3.0 + 4.81) = 39.0$ inches

Of the thicknesses indicated above, 3 inches is Type I filter, 4 inches is Type II filter, and the remaining is riprap. The riprap thicknesses y_r are therefore:

14% of slope length:

$y_r = 18.2 - 7 = 11.2$ inches

One-fourth slope length:

$y_r = 21.1 - 7 = 14.1$ inches

One-half slope length:

$y_r = 27.1 - 7 = 20.1$ inches

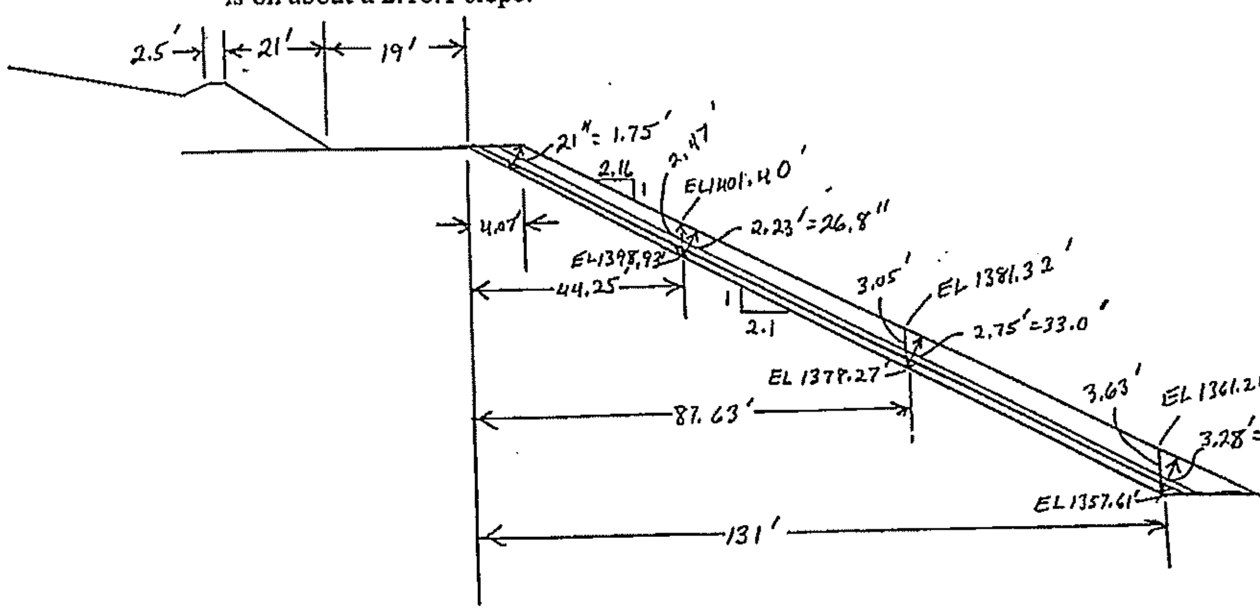
Three-fourth slope length:

$y_r = 33.0 - 7 = 26.0$ inches

Full slope length:

$y_r = 39.0 - 7 = 32.0$ inches

As illustrated on the figure below, the required riprap thickness increases in depth in the flow gradient direction on the 2.1:1 (horizontal to vertical) slopes. The upper surfaces of the Type I and Type II granular filters parallel the 2.1:1 exterior embankment slope. The riprap starts out about 14 inches thick at the top of the exterior embankment slopes and increases in thickness such that the upper surface of the riprap is on about a 2.16:1 slope.

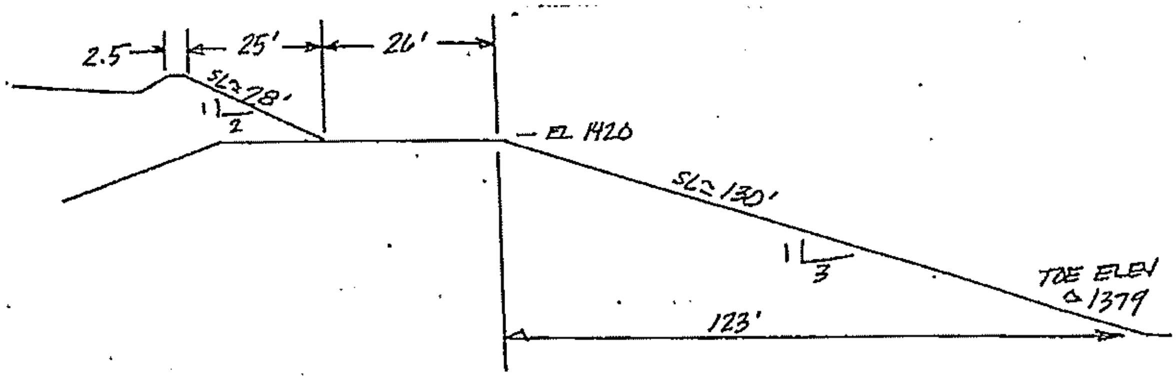


II. Design of temporary sideslope erosion protection for the 3:1 (horizontal to vertical) exterior slope along the west side of Landfill Cell 15.

The longest 3H:1V slope is at the west edge of Cell 15 and goes from an elevation of 1420 feet at the top of the cell embankments to an elevation of about 1379 feet at the exterior toe of the embankment slopes. Thus the embankment is about 41 feet high at the highest point. The erosion protection is to consist of a 3-inch thick Type I granular filter blanket, a 4-inch thick Type II granular filter blanket and a required depth of riprap for the rock to be stable eon the sideslopes with a reasonable safety factor. According to the information provided by AGEC, the two types of granular filter are required below the riprap for protection of the embankment sideslopes. The Type I filter will be used to filter and hold the embankment soils in place and the Type II filter will be used to filter and hold the Type I filter material in place.

1. Hydrology

The surface area which contributes runoff to the sideslopes of the cell includes the sideslope of the cell itself, the top of the cell embankment, the 2:1 (horizontal to vertical) exterior slope of the cap, and the top of the berm around the perimeter of the cap. The dimensions of these areas are as follows:



The total horizontal distance over which precipitation would fall and that would be tributary to the side slope of the cell would be = 2.5' + ~25' + ~26' + ~123' = ~176.5 ft. The length along the embankment surfaces (accounting for slope lengths) is 2.5' + ~28' + ~26' + ~130' = 186.5'.

Since the flow will be interflow in the rock itself, then the time of concentration is equal to the time for water to flow through the rock from the top to the bottom of the slope.

The velocity (V) of flow through the rock = ki/n, where k= permeability of the rock, i= the hydraulic gradient assumed to be equal to the slope of the sideslopes of the cell, and n= the porosity of the rock or filters. Chen-Northern tested the permeability of the Type I filter material to be 9 x 10^{3} cm/sec. Thus, there will be very little flow in the Type I filter. Chen-Northern also tested the Type II filter material to have a permeability of 3.7 cm/sec = 0.121 ft/sec. Assume the porosity of the Type II filter to be 0.25.

Thus V in Type II = 0.121*0.333/0.25 = 0.16 ft/sec.

The time of concentration T_c = slope length/V
= 186.5/0.16
= 1,166 sec. = 19.4 min. = 0.32 hrs.

Using the SCS Unit Hydrograph Procedure, the peak discharge Qp from 1 acre of area using the following data is 5.62 cfs (see attached computer printout).

Average basin slope	= {2.5(0)+25(0.5)+26(0)+123(1/3)}/{2.5+25+26+123}
	= 30.3 percent
Curve Number	= 90
100-yr, 24-hr precipitation	= 8.0 inches
Storm Duration	= 24 hours
Hydraulic Length	= 186.5 ft.
Time of Concentration	= 0.32 hours

Checking the flow rate at the full slope length gives:

The horizontal length along the slope for 1 acre of slope with a horizontal slope of 176.5 would be:
= (43,560)/176.5 = 247 ft.
Thus, q_p = 5.62 cfs / 247 ft = 0.0228 cfs/ft

2. Required Riprap Thickness

Based on information provided by AGEC (see attached letter), it was determined that for the riprap to be stable on the 3H:1V sideslopes of the cells with a S.F. of 1.5, the flow must occur in the lower 72% of the rock thickness, which must include the flow in the two filter materials. Since the lower filter (Type I) consists of a sandy material, with a minimum of 2% passing the #200 sieve, the permeability as determined by Chen-Northern is only 9 x 10^{3} cm/sec = 3 x 10^{-4} ft/sec. The Type II filter is made up of larger gravel relatively free of fines. From tests conducted on the Type II filter materials by Chen-Northern, the permeability of the Type II material is 3.7 cm/sec = 0.121 ft/sec.

To determine the required rock thickness a seepage depth was calculated by applying Darcy's Law with states:

$$Q = kiA$$

HA&L ENGINEERING	CLIENT: LESI-Lone Mountain Facility PROJECT: RCRA Landfill Cell 15 FEATURE: Sideslope Erosion Protection PROJ. NO.: 64.44.200	SHEET 9 OF 10 COMPUTED: PGH CHECKED: KCS DATE: April 29, 1993

REVISED 10/14/97

PROJECT : USPCI Ln. Mtn. Cell 15, West Sideslope Erosion Protection

AREA= 1.0 ACRES
AVERAGE BASIN SLOPE= 30.3 PERCENT
CURVE NUMBER= 90.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 187. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .32 HOURS

TP= .2133 HOURS QPCFS= 3.54 CFS QPIN= 3.5155 INCHES
C3= 17.3279 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.39	.2196	.0000	.0000	.0	.00
2.43	.2244	.0000	.0000	.2	.00
2.47	.2292	.0000	.0000	1.1	.00
2.52	.2334	.0001	.0000	2.4	.00
2.56	.2368	.0002	.0000	3.3	.00
2.60	.2402	.0003	.0000	3.5	.00
2.65	.2436	.0004	.0001	3.3	.00
2.69	.2470	.0005	.0001	2.8	.00
2.73	.2505	.0007	.0002	2.2	.00
2.77	.2539	.0009	.0002	1.6	.00
2.82	.2573	.0011	.0002	1.1	.00
2.86	.2607	.0013	.0002	.8	.00
2.90	.2641	.0015	.0002	.5	.00
2.94	.2675	.0018	.0003	.3	.00
2.99	.2709	.0020	.0003	.2	.00
3.03	.2758	.0025	.0004	.1	.00
3.07	.2812	.0030	.0005	.0	.00
11.78	3.9418	2.8641	.2449	.0	3.79
11.82	4.2012	3.1104	.2464	.0	4.35
11.86	4.4606	3.3581	.2477	.0	4.78
11.90	4.7201	3.6068	.2487	.0	5.11
11.95	4.9795	3.8565	.2497	.0	5.35
11.99	5.2389	4.1070	.2505	.0	5.52
12.03	5.3408	4.2057	.0986	.0	5.62
12.07	5.3900	4.2533	.0476	.0	5.53
12.12	5.4391	4.3009	.0476	.0	5.18
12.16	5.4883	4.3485	.0476	.0	4.61
12.20	5.5374	4.3962	.0477	.0	3.93
12.25	5.5866	4.4439	.0477	.0	3.25
12.29	5.6357	4.4916	.0477	.0	2.66

HYDROGRAPH PEAK= 5.62 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= .57 Acre-Feet

Where: Q = Flow rate,
k = permeability,
i = hydraulic gradient, and
A = flow area.

For a one foot flow width, the darcy's equation becomes:

$$q = kiy$$

Where: y = the flow depth.

If y_{fl} = the flow depth in the lower filter
y_{fu} = the flow depth in the upper filter, and
y_r = the flow depth in the rock riprap,

Then:

$$\text{total flow depth } y_t = y_{fl} + y_{fu} + y_r$$

Using the permeability of the Type I filter and Darcy's law, the flow that the Type I filter would carry would be:

$$q = (3 \times 10^{-4})*0.33*(3/12)$$
$$= 2.475 \times 10^{-5} \text{cfs/ft}$$

Thus the Type I filter will carry very little flow.

Using the permeability for the Type II material and assuming this permeability to conservatively apply to the overlying rock riprap, then the flow depth above the Type I filter required to convey the peak flow rate at the full slope length from the 100-year, 24-hour storm event would be:

$$y = q/(ki) = 0.0228/(0.121*0.333) = 0.57 \text{ ft} = 6.8 \text{ inches}$$

Assuming the Type I filter to be saturated, then the total required thickness of rock and filter (y_t), which must be 1.39 times the seepage depth to maintain a safety factor of 1.5, is:

$$y_t = 1.39\,(y_{fl} + y) = 1.39\,(3.0 + 6.8) = 13.6 \text{ inches}$$

Of the thickness indicated above, 3 inches is Type I filter, 4 inches is Type II filter, and the remaining is riprap. The riprap thicknesses y_r are therefore:

$$y_r = 13.6 - 7 = 6.6 \text{ inches}$$

The 3:1 exterior (west) slope will have 8 inches of riprap material, therefore the design meets the above design criteria.

Applied Geotechnical Engineering Consultants, Inc.

October 15, 1997

Hansen, Allen & Luce
6771 South 900 East
Midvale, UT 84047

Attention: Marv Allen

Subject: Exterior Side Slope Protection
Landfill Cell 15
Lone Mountain Facility
Waynoke, Oklahoma
Project No. 973021

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. was requested to provide recommendations for the exterior slope protection for Landfill Cell 15 at the Lone Mountain Facility. Our analysis includes protection of the embankment slope from particle migration and stability.

PARTICLE MIGRATION

The project documents indicate placement of a Type I filter immediately adjacent to the embankment, a Type II filter and Type L riprap. Listed below is a summary of the specification for each of these materials.

Grain Size Distribution

Sieve Size	Percent Passing		
	Type I	Type II	Type L
15"			100
11"			50
7"			20
3"		90-100	
¾"		35-90	
⅜"	100		

Sieve Size	Percent Passing		
	Type I	Type II	Type L
No. 4	85-100	0-30	
No. 16	45-80	0-15	
No. 50	10-30		
No. 100	2-10		
No. 200	0-2	0-3	

The grain size distribution of the embankment materials indicate that the Type I filter will prevent particle migration from the embankment slope. The Type II filter will protect the Type I and Type L will protect the Type II.

Based on the specifications, the embankment materials will be protected with the layers being placed from smallest to largest.

STABILITY

The exterior slope stability was evaluated with the embankment material Type I and Type II filters along with the riprap placed on 2:1 and 3:1 (horizontal to vertical) slopes. The critical parameter for design is the height of water in the slope protection material and the soil strength.

Earlier recommendations indicated that the filter material be relatively strong. In order to allow greater flexibility in the type of material used and to provide a factor of safety of at least 1.5 under seepage conditions, we recommend that the filter material have the following internal soil strengths.

A. Filters Placed on 2:1 Slopes

2:1 slopes will require that the internal coefficient of friction be at least 38 degrees and that the water level be maintained no higher than 20 percent of the total thickness of the slope protection materials.

B. Filters Placed on 3:1 Slopes

On 3:1 slopes, filter materials should have an internal friction angle of 38 degrees to maintain a safety factor of at least 1.5 if the water level is no higher than 72 percent of the total thickness of the protection materials.

Deeper water in the slope protection material would require the filter materials to have a higher friction angle or the factor of safety will drop below 1.5.

We would recommend that the materials proposed for use be tested to verify what compactive effort and material characteristics would provide the suitable soil strengths.

If you have any questions or if we can be of further service, please call.

Sincerely,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
JAMES E. NORDQUIST
15327
10/15/97
OKLAHOMA

James E. Nordquist, P.E.

JEN/js

Applied Geotechnical Engineering Consultants, Inc.

SUBJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY [illegible]

SHEET 1 OF 8

SUBJECT Embankment Protection

Cell 15 Soil Cover - Protection of embankment

Criteria 1. $\frac{D_{15F}}{D_{85S}} < 4-5 < \frac{D_{15F}}{D_{15S}}$

permeability

$\frac{D_{50F}}{D_{50S}} \le 25$

	D_{15}	D_{50}	D_{85}
Soil	0.0001 - 0.001	.0007 - .004	0.007 - 0.074
Type 1	0.16 - 0.35	.5 - 1.5	1.7 - 4.8
Type 1	1.19 - 9	7.5 - 25	17 - 65
Type L	152	280	380

Possible Value

Soil / Type 1

OK $\frac{D_{15F}}{D_{85S}} = \frac{0.16 - 0.35}{0.007 - 0.074}$ 2.16 4.73 6.3 23 50

No $\frac{D_{50F}}{D_{50S}} = \frac{0.5 - 1.5}{0.0007 - 0.004}$ 125 375 425 714 2143

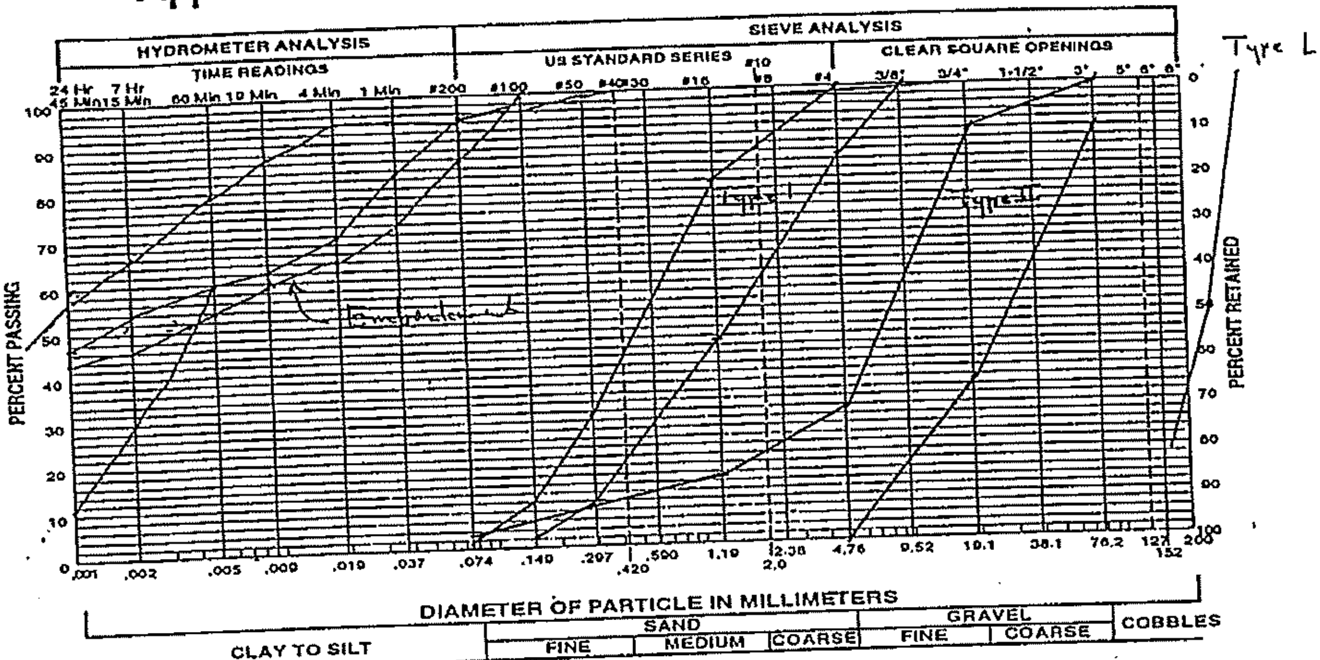
Soil / Type II

$\frac{D_{15F}}{D_{85S}} = \frac{1.19 - 9}{0.007 - 0.074}$ 16.08 121 125 170 1286

$\frac{D_{50F}}{D_{50S}} = \frac{7.5 - 25}{0.0007 - 0.004}$ 1875 6250 6900 10,000 35,714

Sheet 2 of 8

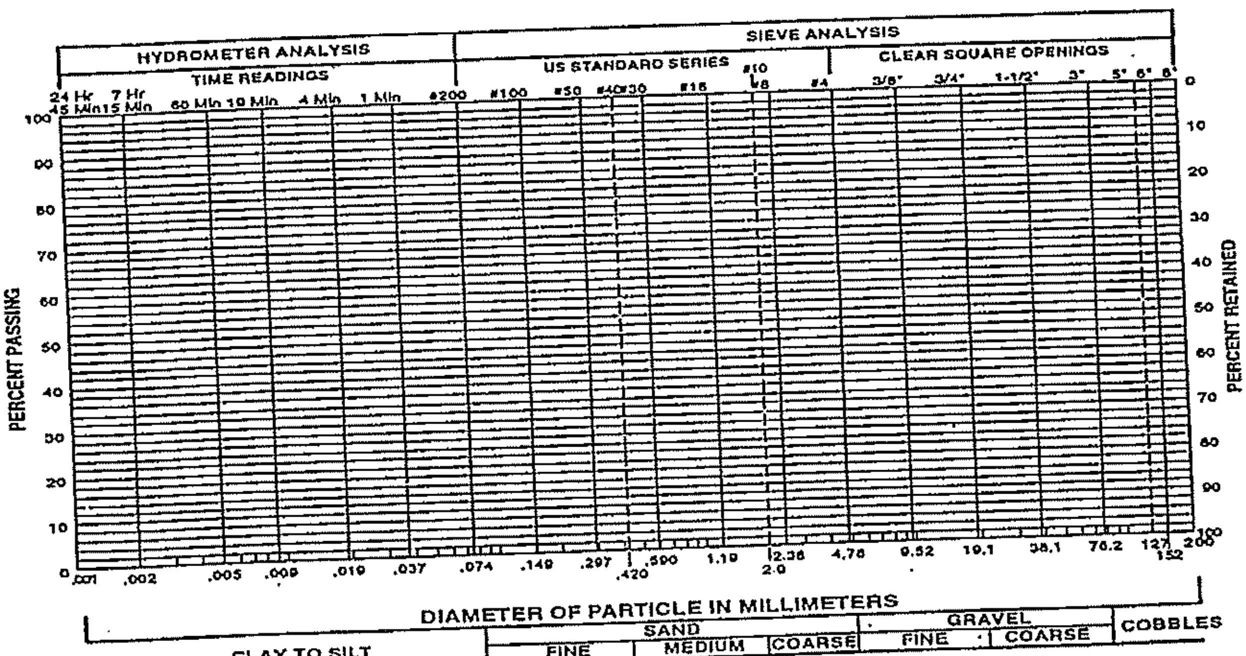
Applied Geotechnical Engineering Consultants, Inc.



Gravel__________ % Sand__________ % Silt and Clay__________ %

Liquid Limit__________ % Plasticity Index__________ %

Sample of__________ From __________



Gravel__________ % Sand__________ % Silt and Clay__________ %

Liquid Limit__________ % Plasticity Index__________ %

Sample of__________ From __________

Project No. 973021 **GRADATION TEST RESULTS** Figure __________

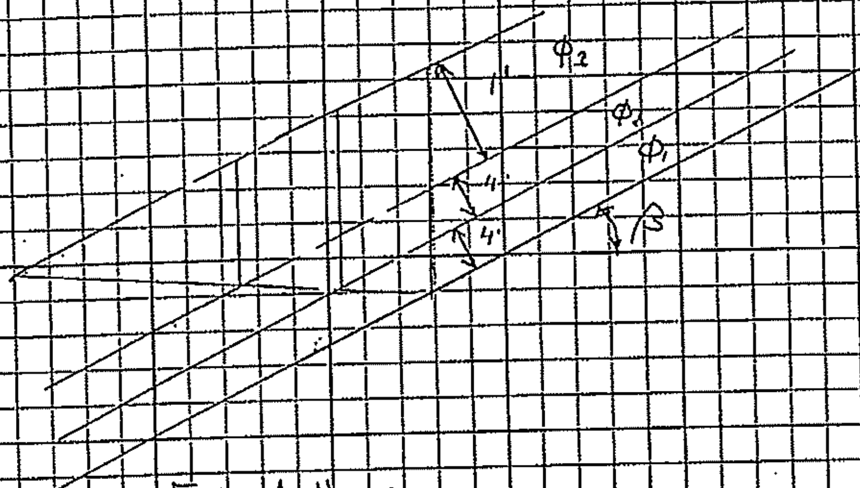
Applied Geotechnical Engineering Consultants, Inc.

SUBJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY [illegible]

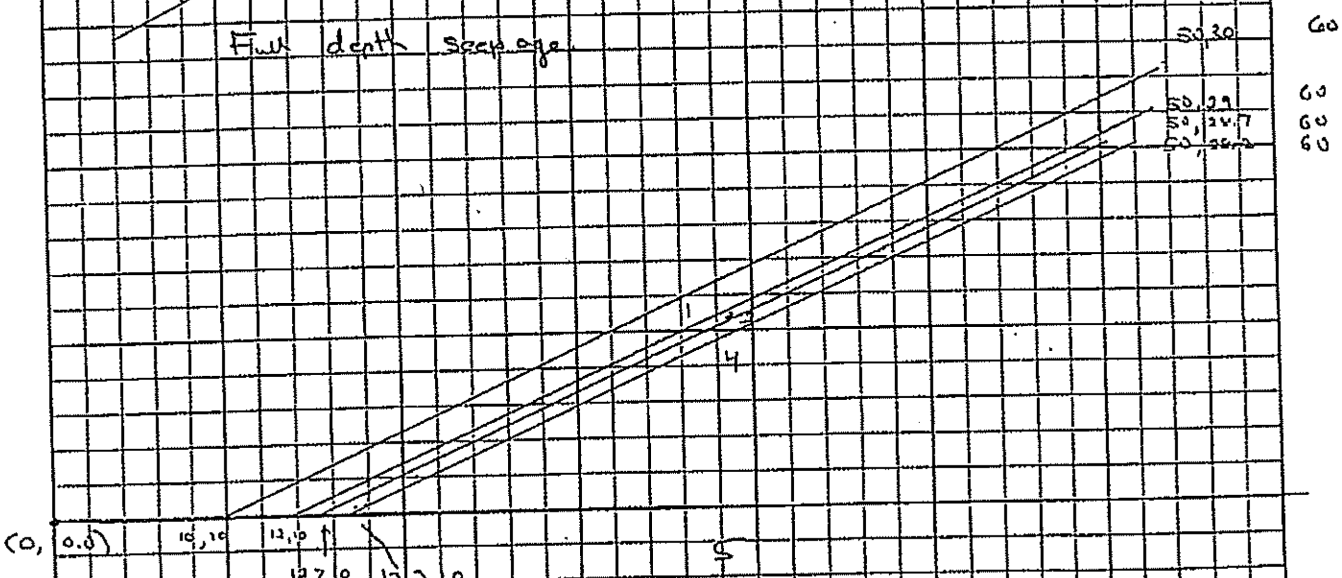
SUBJECT Embankment Protection / stability SHEET 3 OF 8

$D_{15} < 0.4$ mm to protect clay — Filter Conclusion

Side slope protection from water flow.



Full depth seepage



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PROJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY [illegible]

SUBJECT Protection Stability SHEET 4 OF 8

Oklahoma Side Slope Stability

2:1 slope

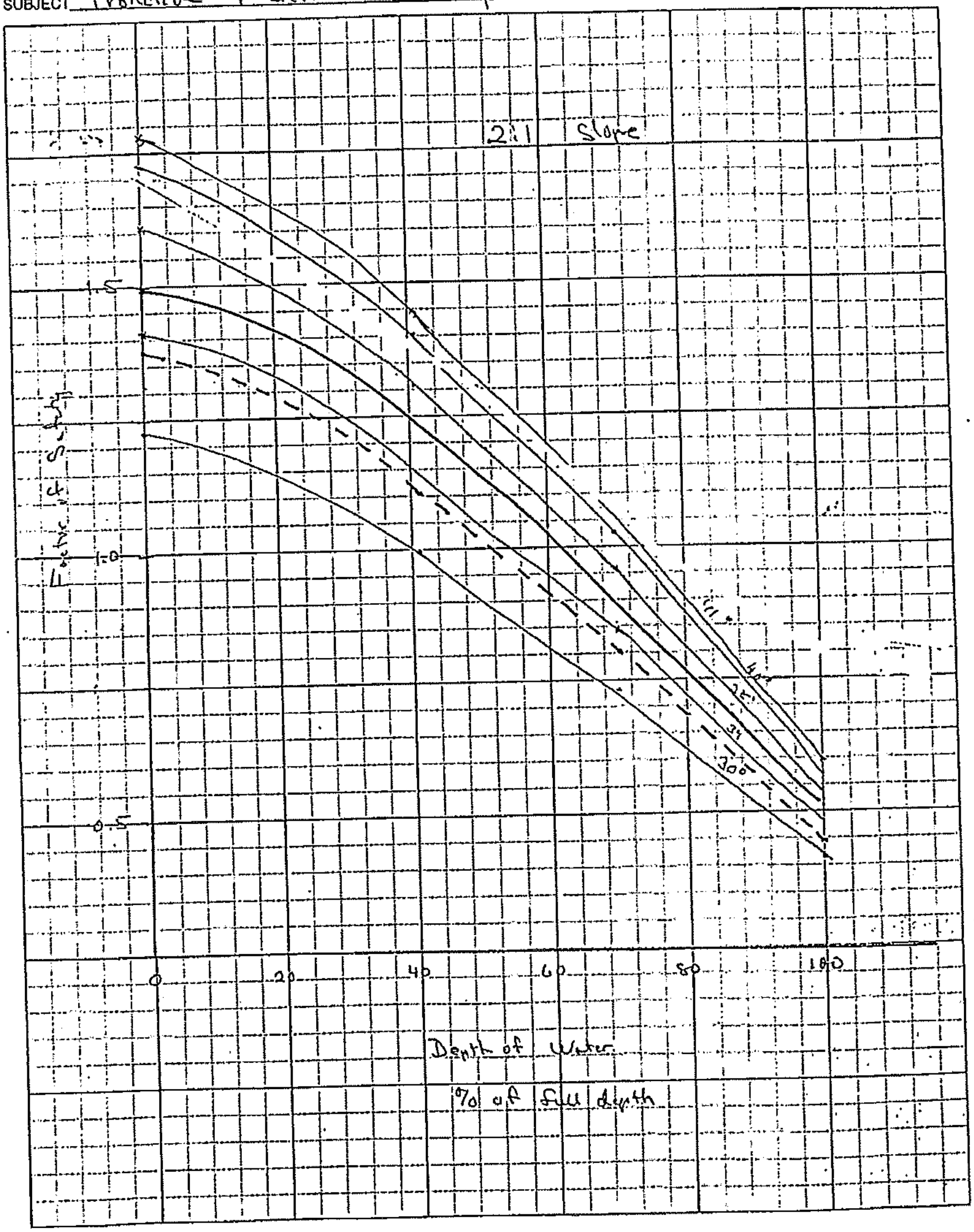
Armor Plate	Filter	Water Depth		F.S.	File
41°	30°	Full		0.413	OK01
	32°	Full		0.443	OK02
	34	Full	100%		
	36	Full			
	38	Full			
	40	Full		0.575	OK03
	41	"		0.543	OK018
41°	30°	1' down	(Filter only)	0.997	OK05
	34	"		1.146	OK06
	38	"	41	1.310	OK07
	40	1' down		1.400	OK04
	41			1.446	OK17
41°	30	0.5 down		0.737	OK08
	34			0.844	OK09
	38		70%	0.960	OK010
	40			1.024	OK011
	41				
41°	30	no water in granular		1.224	OK012
	34			1.411	13
	38			1.614	14
	40			1.725	15
	41			1.782	16
41	36	Full	EQ	0.444	OK019
	36	0.5 dn		0.811	OK020
	36	1 dn		1.112	OK021
	36	no.		1.372	OK022

AGEC

Applied Geotechnical Engineering Consultants, Inc.

PROJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY [illegible]

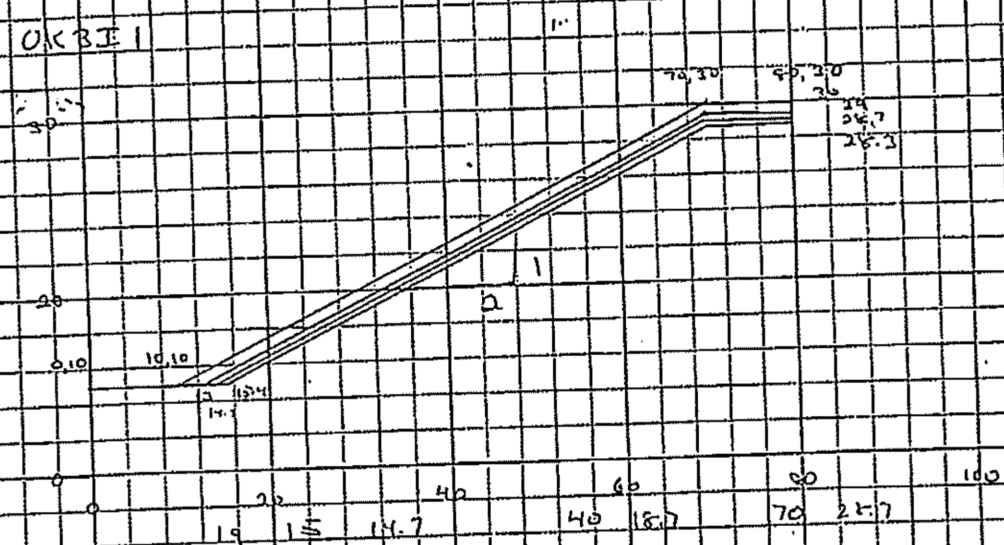
SUBJECT Protection Material Stability SHEET 5 OF 8



Applied Geotechnical Engineering Consultants, Inc.

SUBJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY

SUBJECT Stability of Protection Material SHEET 6 OF 8

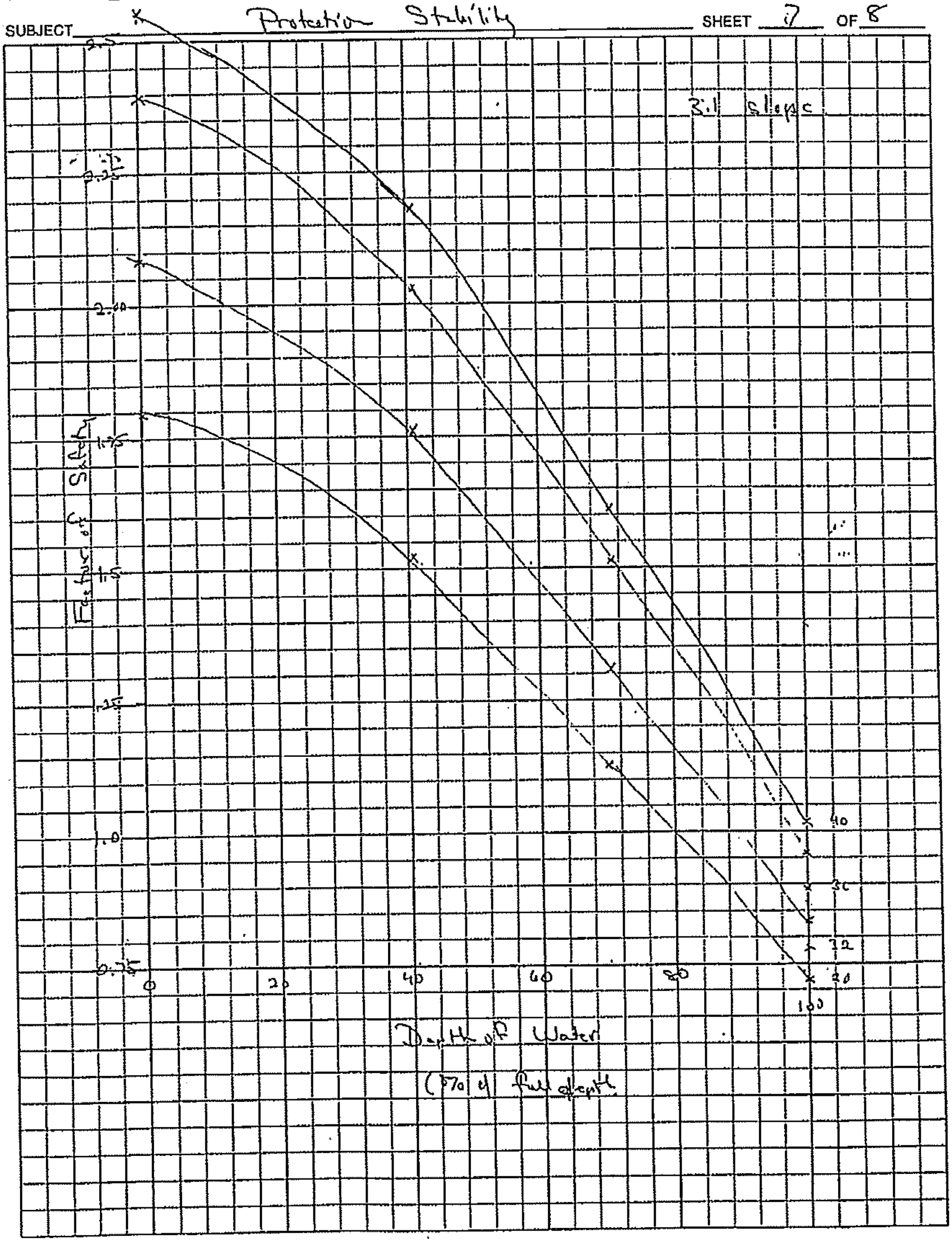


Water	Armor	Filter	SF	File
Full	41°	30	0.72	OK301
		32	0.777	OK302
		34	0.833	OK303
		36	0.893	OK304
		38	0.956	OK305
		40	1.022	OK306
none	41	30	1.799	OK308
		34	2.065	OK309
		38	2.394	OK3010
		40	2.560	OK307
1' down	41	30	1.520	OK3012
		34	1.745	OK3013
		38	2.034	OK3011
11%		40	2.179	OK3014
0.5 down	41	30	1.132	OK3016
		34	1.311	17
		38	1.508	18
71%		40	1.615	OK3015

Applied Geotechnical Engineering Consultants, Inc.

SUBJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY [illegible]

SUBJECT Protection Stability SHEET 7 OF 8



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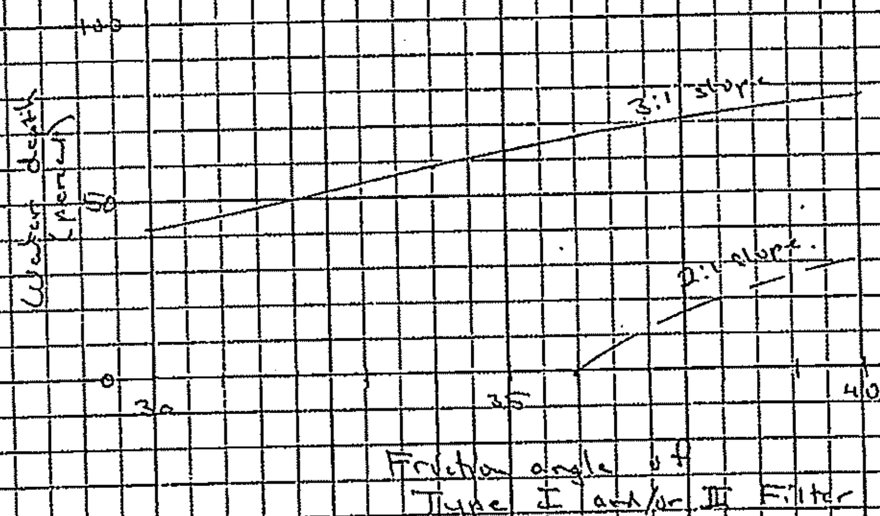
SUBJECT NO. 973021 TITLE Cell 15 DATE 10/15/97 BY [illegible]

SUBJECT Slope Stability SHEET 8 OF 8

Summary -

For F.S = 1.5



Design criteria associated with the Type II granular filter to be placed on the 3H:1V and the 2H:1V slopes is different from the design criteria required for the Type II granular filter material to be placed on the 10 percent closure cap slopes. This calculation is provided to describe the differences in design criteria and to evaluate the design for the Type II granular filter material to be placed below the Type V riprap on the 10 percent Closure Cap Slope.

I. Design Criteria on the Steeper 3H:1V and the 2H:1V Slopes:

A. Design of the Type II granular filter to be placed on the 3H:1V and on the 2H:1V slopes depends on two criteria:

1. One design criterion is to provide the erosion protection required as a result of the high potential flow velocities within the filter materials. Two filters were designed with material gradations to provide protection of the underlying embankment and soil protective cover materials from eroding from under the riprap and off the slopes. Under the steep slope design with the higher potential of flow velocities, the Type I filter is designed to provide erosion protection to the embankment and soil protective cover materials, the Type II filter is designed to provide erosion protection to the Type I filter and the riprap is designed to provide erosion protection to the Type II filter.

2. The other criterion depends on flow depth within the filter and riprap materials which is critical to the slope stability design issues on the steeper slopes. Flow depth is dependent on permeability within the filter and riprap materials. The permeability of the Type I and Type II filter materials was determined based on laboratory testing of the materials performed by Applied Geotechnical Engineering Consultants (AGEC). Flow depth within the filter materials and the riprap was calculated using Darcy's Law, applying the permeability established by AGEC, and using the slopes upon which the materials are placed as the hydraulic gradient. AGEC provided design criteria with recommended safety factors to establish the riprap thickness required to provide adequate slope stability of the erosion protection materials.

3. All calculations for design of the filter materials and riprap are provided with the Design Engineering Reports for the Landfill Cell and Closure designs.

II. Design Criteria on the 10 Percent Closure Cap Slopes:

A. Design of the filter and riprap materials on the 10 percent closure cap slope is based upon potential flow velocities within the Type II material. Potential flow velocities on the 10 percent closure cap slopes are significantly lower than the potential velocities on the steeper 3H:1V and the 2H:1V slopes. This allows for a variation in gradation criteria for the Type II granular filter from that used on the steeper slopes.

B. The friction angle between the riprap, granular filter and the soil protective cover materials is higher than the friction angle between the HDPE geomembrane liner and the geonet (drainage net) materials. The slippage plane controlling stability of the closure cap on the 10 percent slope is, therefore, between the HDPE geomembrane liner and the geonet. Since the planes between the riprap, granular filter and soil protective cover do not control stability of the closure cap, design of the Type II granular filter is not based on stability criteria. Flow will occur within the erosion protective materials for the full depth without causing stability concerns.

III. Design Calculations for the Type II Granular Filter on the 10 Percent Closure Cap Slopes.

A. Determine potential flow velocities using Darcy's Law:

$$V = Ki/n$$

Where: V = Velocity, fps
K = Hydraulic Conductivity, fps
i = Hydraulic Gradient, ft/ft
n = Porosity, decimal

B. Hydraulic Conductivity, based on testing conducted by AGEC is K = 3.7 cm/sec. (0.121 ft/sec)

C. Hydraulic gradient is the cap slope, i = 0.10.

D. Typical porosity for the Type II filter material (estimate), n = 25% (0.25)

E. Flow Velocity:

$$V = (0.121)(0.10) / (0.25) = 0.05 \text{ fps}$$

1. The velocity of 0.05 fps is representative of the velocity that may be expected in the Type II granular filter material specified. Providing a specification for the Type II material that allows more fine material effectively reduces the hydraulic conductivity by much greater proportions than the reduction in porosity, thus decreasing the velocity. Therefore, the velocity calculated above should represent a maximum velocity that may be expected in the Type II granular filter assuming that the specification for the Type II material to be placed on the 10 percent slope is at least equivalent or finer than the Type II material specified for the steeper 3H:1V and 2H:1V slopes.

IV. According to the U.S. Bureau of Reclamation (USBR), maximum flow velocities prior to erosion and corresponding Safety Factors against erosion for the velocity calculated are as follows:

Soil Type	Maximum Permissible Velocity, fps	Safety Factor Against Erosion
Silt	0.49	9.8
Fine Sand	0.66	13.2
Fine Sand with Colloidal Properties	1.50	30.0
Medium Sand	0.98	19.6
Sandy Loam	1.75	35
Silt Loam	2.00	40

A. The soil protective cover to be used on the closure cap generally consists of fine sands containing some silts. Since silts have the lowest erodible velocities, the values in the table above represent worst case conditions.

V. Based on the above calculations, we recommend that the specification for the Type II granular filter material be modified to allow for a wider range of finer particles than allowed for the material specified for the steeper slopes. Gradation curves for the Type V riprap were evaluated to determine the range to which the Type II material may be modified. The following table provides the gradation as previously specified for the steeper (3H:1V to 2H:1V) slopes and the modified gradation for the material to be placed on the 10 percent slopes.

U. S. Standard Sieve Size	Percent Passing by Weight as Specified for Steeper 3H:1V to 2H:1V Slopes	Percent Passing by Weight as Recommended for the 10 Percent Cap Slopes
3 inches	90 - 100	90 - 100
3/4 inches	35 - 70	35 - 80
No. 4	0 - 20	0 - 35
No. 16	0 - 3	0 - 15
No. 200	0 - 1	0 - 5

EXHIBIT D

DESIGN CALCULATIONS HDPE LINERS

Appendix 1 - Laboratory Report #443 Feb. 14, 1984 and
Laboratory Report # 207 Mar. 7, 1983
by Gundle Lining Systems, Inc.

Appendix 2 - Calculations - Integrity of the HDPE Liner
Against Failure from Normal and
and Tensile Stresses

Appendix 3 - Liner Anchor Trench Design Calculations

HA&L ENGINEERING

APPENDIX 1

Laboratory Report #443
February 14, 1984
and
Laboratory Report # 207
March 7, 1983

by

Gundle Lining Systems, Inc.

LABORATORY REPORT #443

FEBRUARY 14, 1984

SUBJECT

Comparative tensile and tear resistance testing of GUNDLINE® HD and HDA, as well as PVC and Hypalon at various cold temperatures.

TEST METHOD

Tensile and elongation properties were determined according to ASTM D638, a 2 inch-per-minute strain rate, and Type IV tensile specimens. For the notched results, a razor blade was used to make a notch approximately .01 inches deep perpendicular to the length of the test specimen.

Tongue tear resistance was determined according to ASTM D751. A 12 inch-per-minute strain rate was used.

Cold temperatures were maintained in an Instron environmental test chamber according to ASTM D3847 to an accuracy of +/- 1°C. The test specimens were acclimated to the test temperature for fifteen minutes before testing.

The materials tested were: 80 mil HDPE, 40 mil HDPE, 40 mil HDA, 30 mil HDA, 36 mil 10x10x1,000 denier scrim-reinforced Hypalon, and 30 mil PVC.

OBSERVATIONS (see attachments for data)

The tongue tear resistance of all the materials tested showed a sharp initial decrease in strength between 20°C and 0°C, except for the HDA material. The following table describes the percentage decrease in strength at 0°C and -50°C. From 0°C to-50°C, the strength remained fairly constant for the HDPE and HDA materials (see graph of tear resistance vs temperature). The PVC material remained fairly constant in strength, but became brittle at -30°C. The Hypalon also demonstrated brittle failure at -30°C. The HDPE and HDA materials did not demonstrate brittle failure at any temperature tested, but tore in a manner similar to that observed at +20°C.

	20°C	0°C	% Change @ 0°C	-50°C	% Change @ -50°C
80 HD	260 lb	180 lb	-31%	182 lb	-30%
40 HD	145.5 lb	95 lb	-35%	80 lb	-45%
30 HDA	70 lb	63 lb	-10%	48 lb	-31%
36 Hypalon	118 lb	50 lb	-58%	28 lb	-76%
30 PVC	10 lb	7 lb	-30%	6.9 lb	-34%

The tensile tests demonstrated an increase in strength accompanied by a decrease in elongation as the test temperature decreased. The yield strength appeared to increase along a straight line of definite slope. The point of scrim failure for the Hypalon which was compared to the yield strength of the HDPE and HDA materials seemed to follow the same trend. PVC which does not demonstrate a yielding phenomenon cannot be compared. The following table lists the percentage increase in tensile yield strength as well as loss of elongation from +20°C to -50°C.

	Yield Strength (lb/in) +20°C	Yield Strength (lb/in) -50°C	Percent Increase	Elongation (%) +20°C	Elongation (%) -50°C	Percent Decrease
80 HD	214	524	145	15	6.7	55
40 HD	84	212	152	15	6.7	55
40 HDA	88	240	173	15	6.7	55
30 PVC	N/A	N/A	N/A	N/A	N/A	N/A
36 Hypalon	76	220	189	44	6.7	85

Due to limitations of the environmental test chamber's size, the materials that elongated greater than 350% could not be accurately evaluated for ultimate tensile strength or ultimate elongation (except for the 20°C results which were taken without the chamber). Those materials that did break in less than 350% can be considered accurate.

Examining the graph of ultimate elongation versus temperature, one can see the trend of decreasing elongation. The dotted lines demonstrate the elongation one would expect as extrapolated between the 20°C result and the first accurate cold temperature result. The PVC and Hypalon results were not extrapolated.

The tensile strength at break increased for all of the materials except for the 80 mil HDPE. The following table summarizes the difference from 20°C to -50°C.

	Break Strength (lb/in) +20°C	-50°C	Percent Change	Ultimate Elongation (%) +20°C	-50°C	Percent Change
80 HD	378	347	-8	880	86	-90
40 HD	134	160	+19	685	249	-64
40 HDA	166	232	+40	890	>359	≈-60
30 PVC	80	264	+230	435	10	-98
36 Hypalon	76	220	+189	227	6.7	-97

The notched tensile results demonstrated a steady increase in strength as temperature decreased. The Hypalon results, as demonstrated by the graph of notched tensile strength versus temperature, were somewhat erratic. All of the samples snapped with very little elongation, regardless of the test temperature.

CONCLUSION

All of the materials tested were stiffened by the decrease in temperature. All were affected by notching such that elongation was severely decreased. The effect of notching was seen even at the +20°C test temperature. All of the materials showed a decrease in tongue tear resistance.

The PVC material became brittle at -30°C. This was demonstrated by shattering in tongue tear testing and as a severe decrease in elongation at break, approximately 98%. The break strength increased 230%.

The Hypalon material became brittle at -30°C which was demonstrated by cracking during tongue tear testing and splitting of the polymer before the first seam fiber broke in tensile testing.

The HDA material did not demonstrate brittle failure or cracking at any time. The 40 mil HDA retained approximately 40% of its ultimate elongation at -50°C and increased in break strength by 40%.

The 80 mil HDPE lost approximately 90% of its ultimate elongation properties at -50°C. It did not demonstrate brittleness during tear testing. The yield strength increased 145% from +20°C to -50°C. The break strength decreased by 8%.

The 40 mil HDPE performed better than 80 mil HDPE. The break strength increased by 19% and the ultimate elongation decreased only 64%. The tongue tear resistance was similar.

Upon evaluating all of the test results, the HDA material demonstrated the best overall cold temperature properties. The 40 mil HDPE was second, with the 80 mil HDPE third. The Hypalon and PVC materials would be unsuitable for use at temperatures of -30°C and below.

Chuck Crisman

Chuck Crisman, QC Technician

CC/bj
1207.443

TENSILE TESTING*
ASTM D638

	YIELD STRENGTH (LB/IN)	ELONGATION @ YIELD (%)	BREAK STRENGTH (LB/IN)	ULTIMATE ELONGATION (%)
80 HD				
20°C	214	15	378	880
0°C	308	11.7	364	>350**
-10°C	344	10	414	>350
-30°C	440	10	304	253
-50°C	524	6.7	347	86
40 HD				
20°C	84	15	134	685
0°C	128	10	188	>350
-10°C	136	10	152	>350
-30°C	176	10	198	>350
-50°C	212	6.7	160	249
40 HDA				
20°C	88	15	166	890
0°C	124	13.3	182	>350
-10°C	140	11	240	>350
-30°C	176	8.4	264	>350
-50°C	240	6.7	232	>350
30 PVC				
20°C			80	435
0°C			116	347
-10°C			142	335
-30°C			176	230
-50°C			264	10

*Results are average value of two determinations.
**350 = Capacity of Environmental test chamber

	SCRIM FAILURE STRENGTH (LB/IN)	SCRIM FAILURE ELONGATION (%)	POLYMER FAILURE STRENGTH (LB/IN)	POLYMER FAILURE ELONGATION (%)
36 Hypalon				
20°C	76	44	28	227
0°C	142	37	80	124
-10°C	146	30	90	82
-30°C	192	16.7	Fails with Scrim	
-50°C	220	6.7	Fails with Scrim	

TONGUE TEAR TESTING
ASTM D751

	30 mil PVC	36 mil Hypalon	30 mil HDA	40 mil HD	80 mil HD
-50°C	6.9 lb.	28.5 lb.	48.0 lb.	80.0 lb.	182.0 lb.
-30°C	7.8 lb.	20.5 lb.	70.0 lb.	89.0 lb.	180.0 lb.
-10°C	7.0 lb.	65.0 lb.	68.5 lb.	95.0 lb.	180.0 lb.
0°C	7.0 lb.	50.0 lb.	63.0 lb.	95.0 lb.	180.0 lb.
20°C	10.0 lb.	118.0 lb.	70.0 lb.	145.5 lb.	260.0 lb.

VISUAL OBSERVATIONS

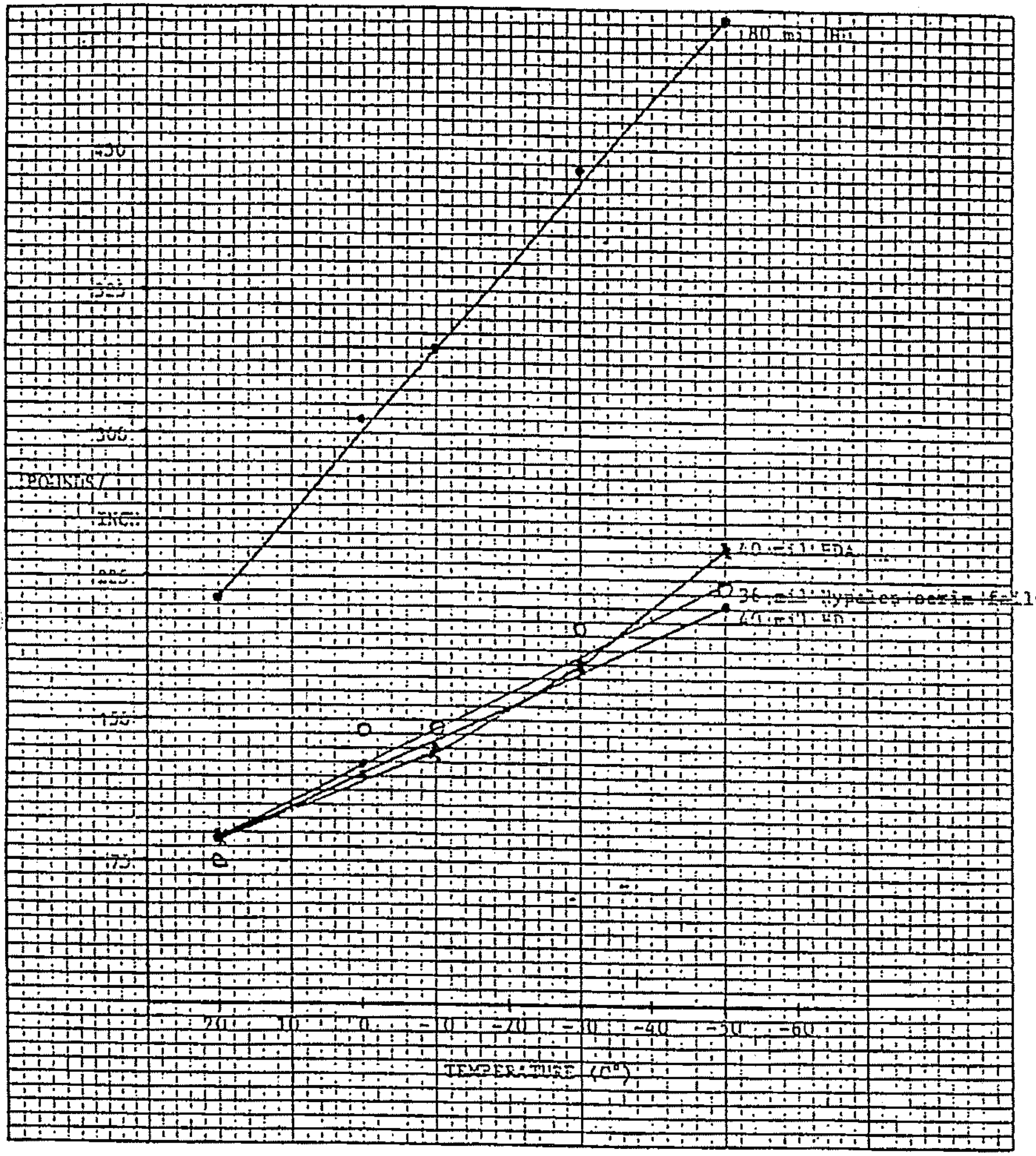
Temperature					
-50°C	brittle	brittle cracking	normal	normal	normal
-30°C	brittle	brittle cracking	normal	normal	normal
-10°C	normal	normal	normal	normal	normal
0°C	normal	normal	normal	normal	normal
20°C	normal	normal	normal	normal	normal

NOTCHED TENSILE

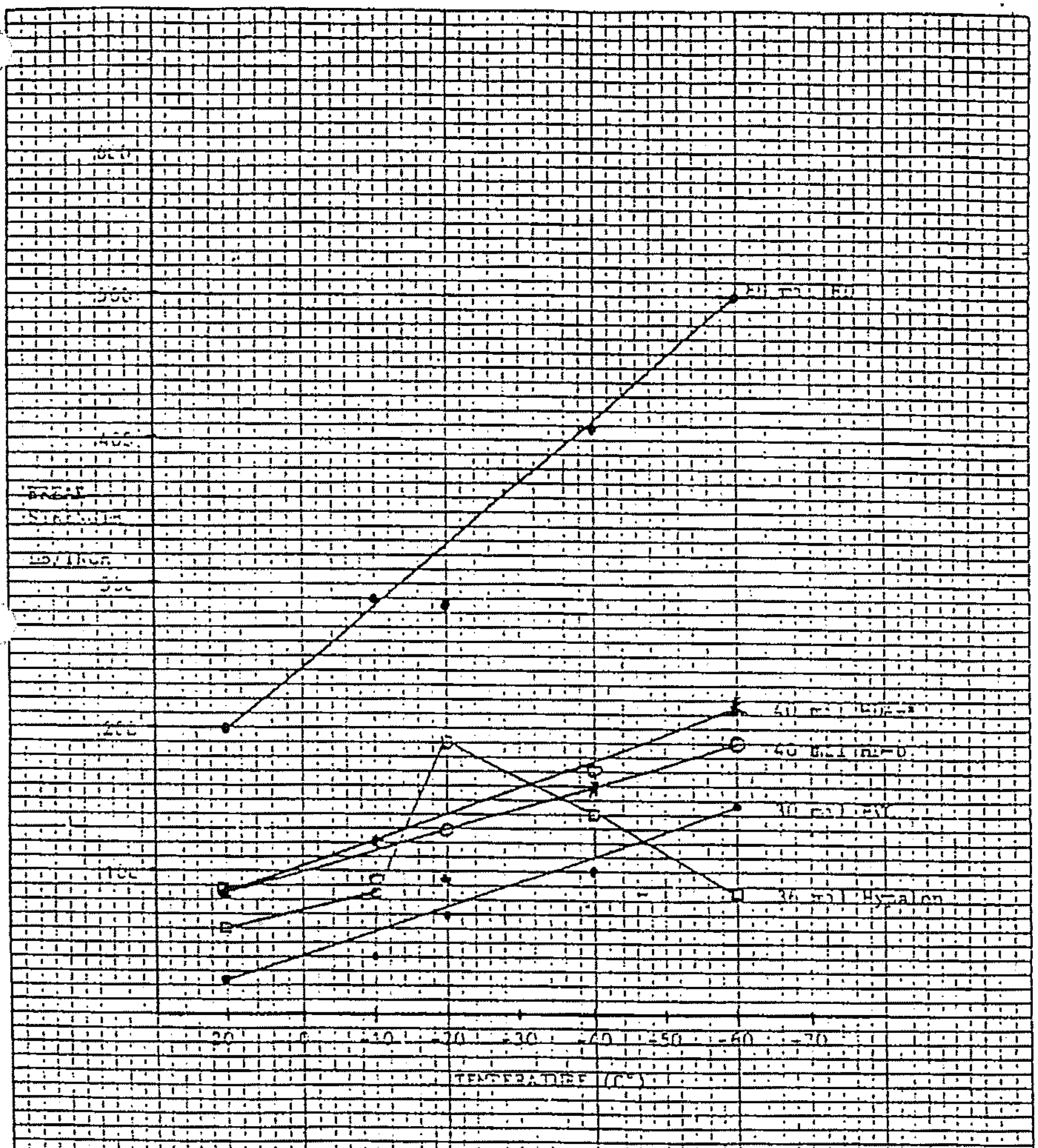
		Break Strength (lb/in)	Elongation (%)
40 HD	-50°C	192	3.3
	-30°C	168	6.7
	-10°C	130	6.7
	0°C	96	6.7
	20°C	86	10.0
40 HDA	-50°C	224	3.3
	-30°C	160	6.7
	-10°C	98	3.3
	0°C	120	6.7
	20°C	86	10.0
80 HD	-50°C	500	6.7
	-30°C	408	6.7
	-10°C	284	6.7
	0°C	288	6.7
	20°C	200	10.0
30 PVC	-50°C	148	3.3
	-30°C	100	10.0
	-10°C	70	64.0
	0°C	40	40.0
	20°C	24	35.0
36 Hypalon	-50°C	82	18.0
	-30°C	140	37.0
	-10°C	188	37.0
	0°C	86	45.0
	20°C	62	84.0

YIELD STRENGTH VS TEMPERATURE

ASTM D638

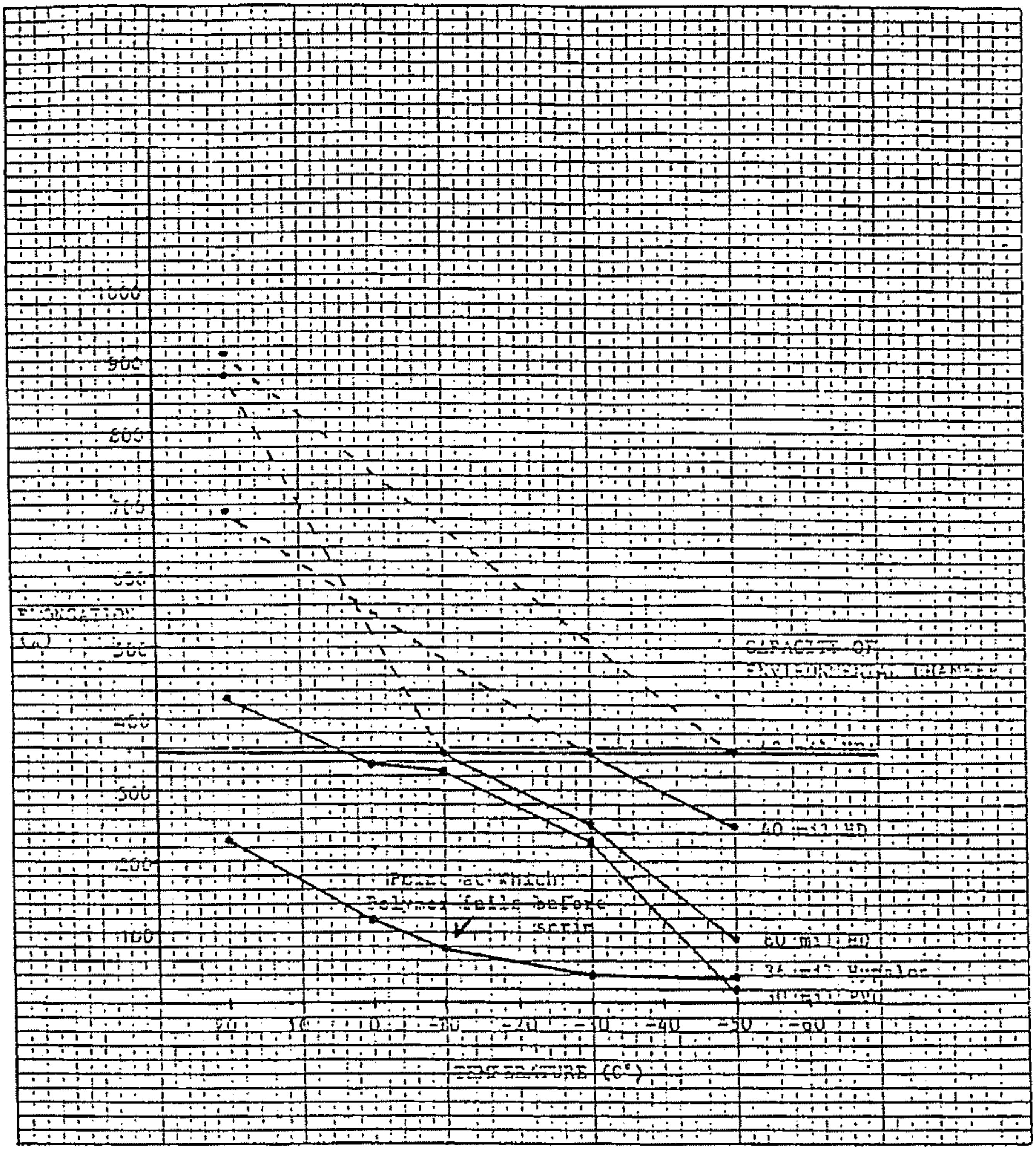


NOTCHED TENSILE STRENGTH VS TEMPERATURE



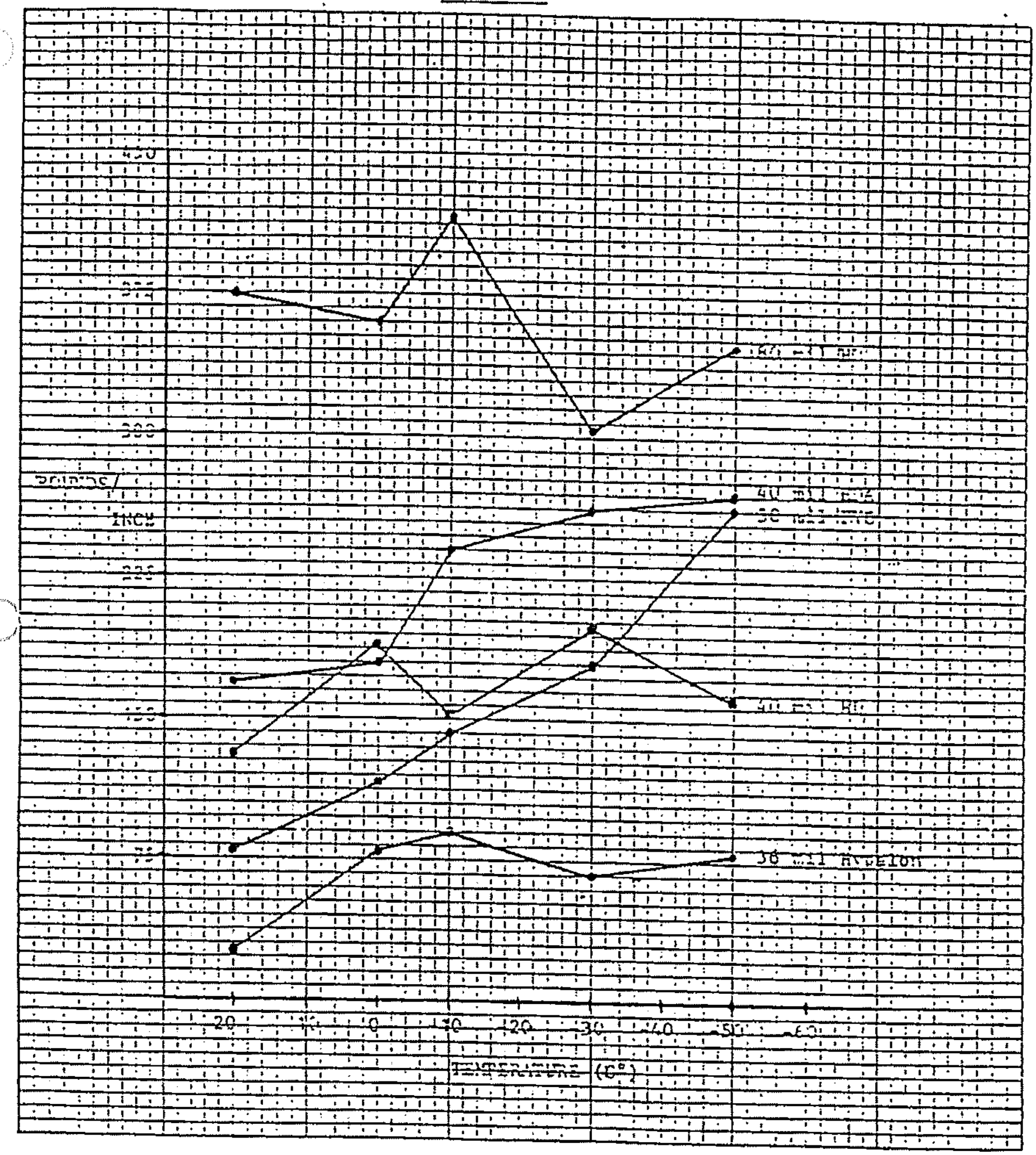
ULTIMATE ELONGATION VS TEMPERATURE

ASTM D638



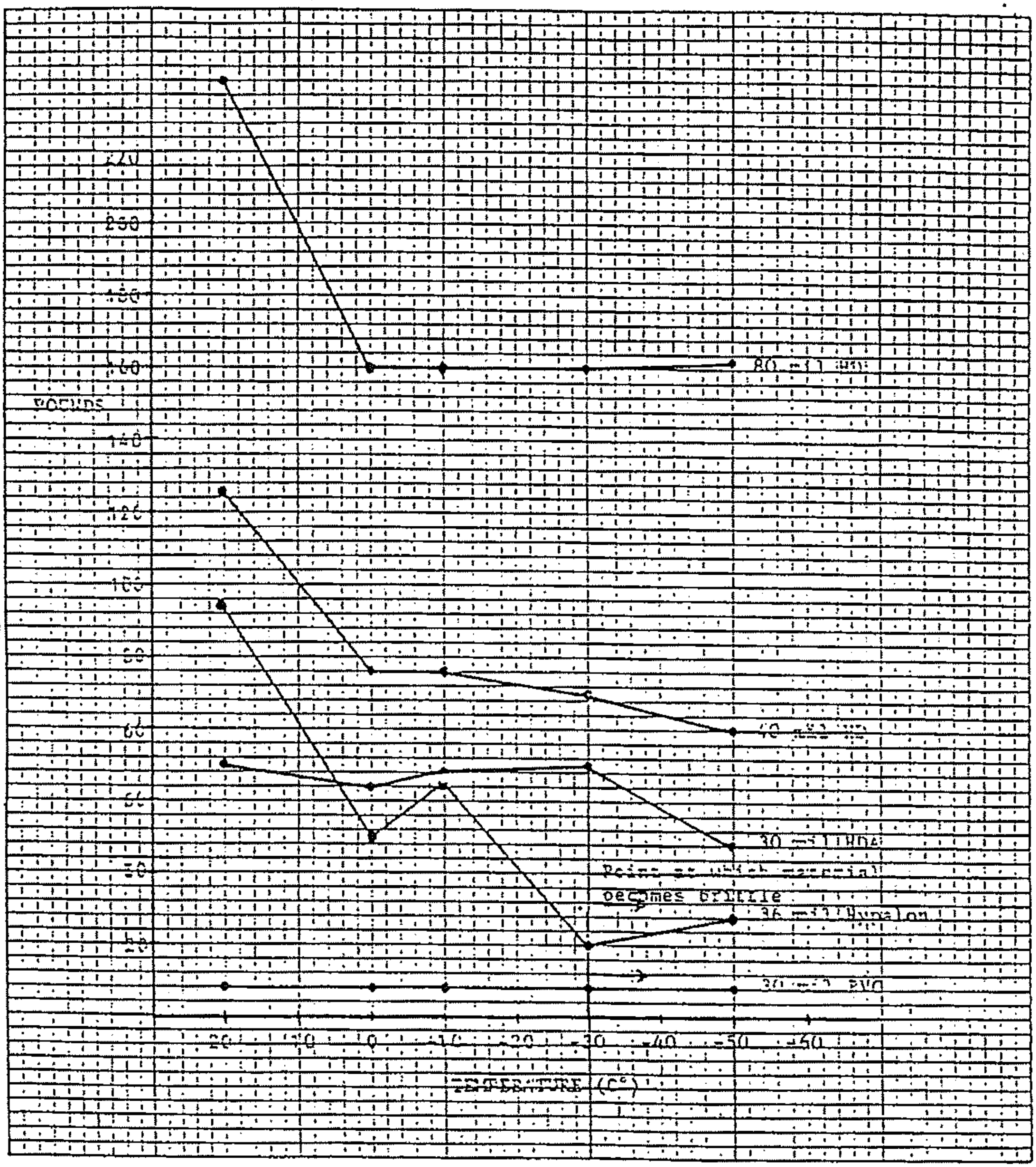
BREAKING STRENGTH VS TEMPERATURE

ASTM D638



TONGUE TEAR RESISTANCE VS TEMPERATURE

ASTM D751



LABORATORY REPORT #207

MARCH 7, 1983

SUBJECT

Tensile & Elongation Properties of GUNDLINE HD® and 36 mil Hypalon at Elevated and Subnormal Temperatures

INTRODUCTION

GUNDLINE HD® and 36 mil 10 x 10 x 1,000 denier scrim-reinforced Goodrich Hypalon were tested at various temperatures in order to determine the effect of temperature on the tensile and elongation properties of the two materials.

TEST METHOD

Tensile and elongation properties were evaluated according to ASTM D638-80 utilizing a crosshead separation rate of 2 ipm. A Type IV dumb-bell specimen was used.

Temperatures were maintained in an Instron Environmental Test Chamber according to ASTM D3547-79 at an accuracy of ±1°C. The tensile specimens were acclimated to the test temperatures of -15, 0, +10, +20, +35, +50, and +70°C for 30 minutes before testing.

In the event that the material did not break in the first 400% elongation allowed due to space limitations of the test chamber, the sample was reclamped and stressed until failure. This method has limitations, as the material tends to fail in the grips when reclamped, giving low values. For this reason, the ultimate elongation and break values of the GUNDLINE® material at temperatures other than 20°C should be viewed as indicative but not accurate. The yield values and data up to 400% elongation is accurate. The Hypalon material failed within 250% elongation. The Hypalon data obtained is, therefore, accurate.

TEST RESULTS

Temperature	Yield Strength (Lb/In) 40 HD	Yield Strength (Lb/In) 100 HD	Scrim Failure 36 Hypalon
70°C	48	128	122
50°C	76	192	144
35°C	94	248	126
20°C	104	320	132
10°C	132	368	188
0°C	150	430	162
-15°C	176	460	218

T-058

Temperature	Elongation at Yield (%) 40 HD	100 HD	Scrim Failure 36 Hypalon
70°C	23	23	25
50°C	20	20	47
35°C	15	15	33
20°C	13	13	30
10°C	10	10	34
0°C	7	7	34
-15°C	5	5	25

Temperature	Breaking Strength (Lb/In) 40 HD	100 HD	36 Hypalon
70°C	96	216	6
50°C	122	282	8
35°C	150	302	24
20°C	174	468	24
10°C	172	380	40
0°C	202	290	56
-15°C	200	448	84

Temperature	Ultimate Elongation (%) 40 HD	100 HD	36 Hypalon
70°C	960	1080	25
50°C	920	1067	180
35°C	1067	773	140
20°C	930	895	97
10°C	820	867	214
0°C	900	727	107
-15°C	720	740	40

The GUNDLINE HDP material (40 and 100 mil) demonstrated superior ultimate elonga-tion properties compared to Hypalon (as seen in photos A & B) at all temperatures. The breaking strength of the HD-material was superior to Hypalon in all cases. The Hypalon was severely weakened at +70°C and failed with the scrim. The elonga-tion of Hypalon was also severely affected at -15°C.

At the yield point, the Hypalon scrim was not as temperature-dependent as the GUNDLINE HDP. The elongation was not severely affected, although it did decrease to 25% at +70 and -15°C. The HD material steadily stiffened and decreased in elongation from 23% to 5% in the range of +70 to -15°C. The yield strength also steadily increased from +70 to -15°C. The Hypalon scrim also increased in strength as well.

CONCLUSION

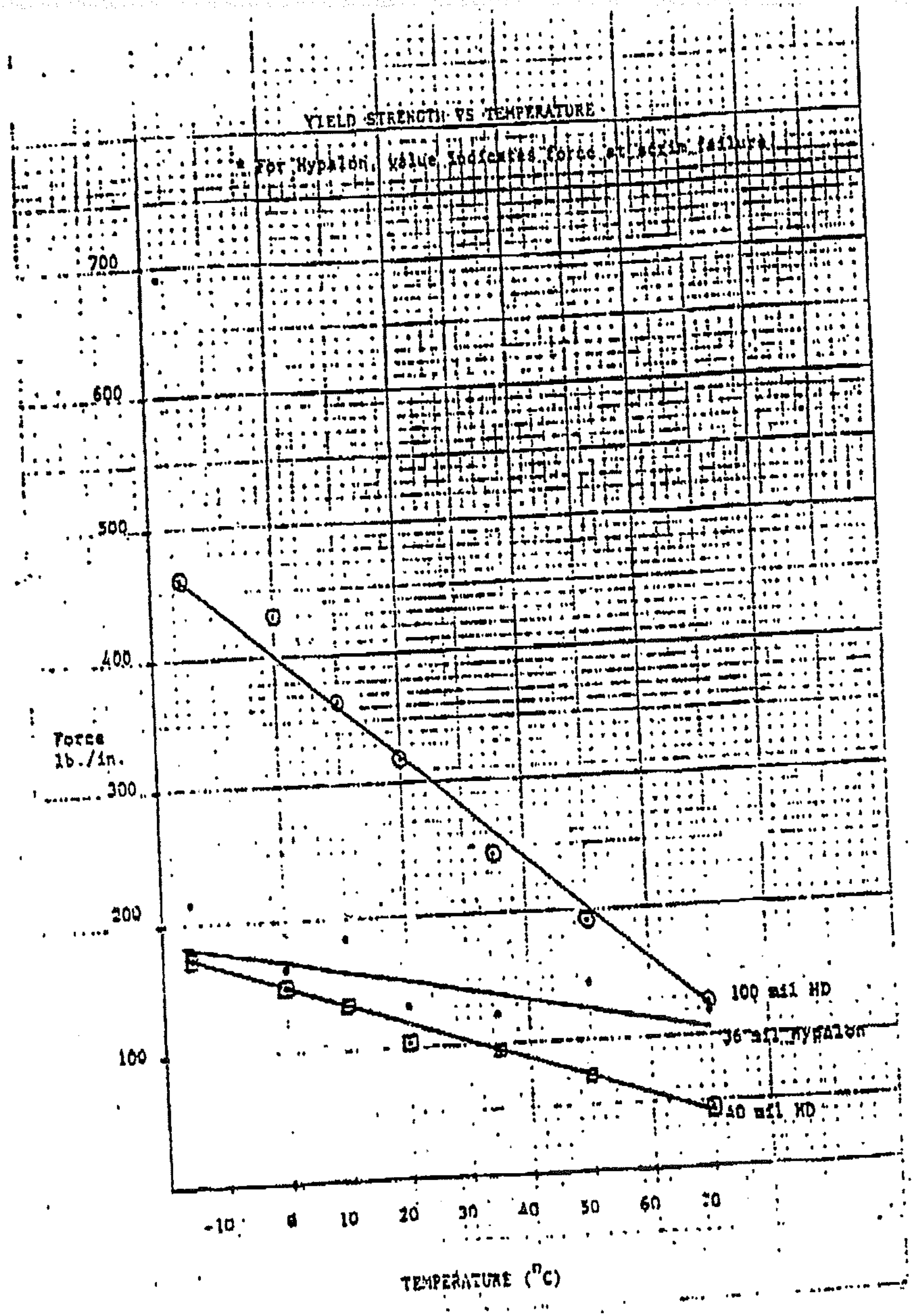
At -15°C and +70°C, Hypalon experiences a severe loss in break properties, as well as ultimate elongation. The HD material retains its elongation properties well over 500% in the full range of -15 to +70°C.

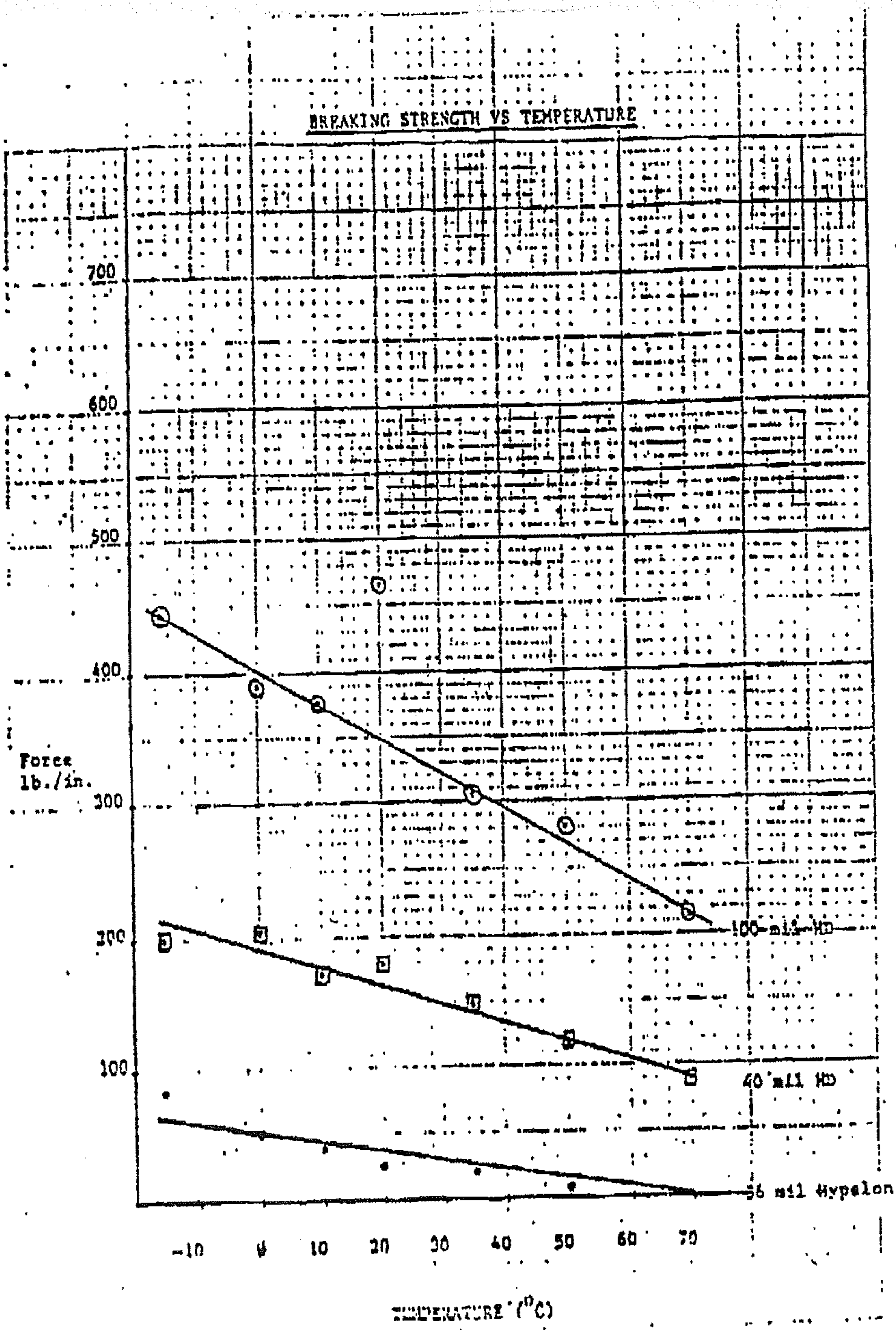
The GUNDLINE HD® material decreased in elongation at yield as temperature decreases. This is accompanied by a proportional increase in strength. The material actually becomes tougher as temperature decreases.

Considering physical strength, GUNDLINE HD® is a superior material compared to Hypalon in the full temperature range of -15°C to +70°C.

Chuck Crisman
C. Crisman, QC Technician

CC/bj


YIELD STRENGTH VS TEMPERATURE
* For Hypalon, value indicates force at scrim failure
700
600
500
400
300
200
100
Force
lb./in.
-10
0
10
20
30
40
50
60
70
TEMPERATURE (°C)
100 mil HD
36 mil Hypalon
40 mil HD


BREAKING STRENGTH VS TEMPERATURE
700
600
500
400
Force
lb./in.
300
200
100
100 mil HD
40 mil HD
36 mil Hypalon
-10
0
10
20
30
40
50
60
70
TEMPERATURE (°C)

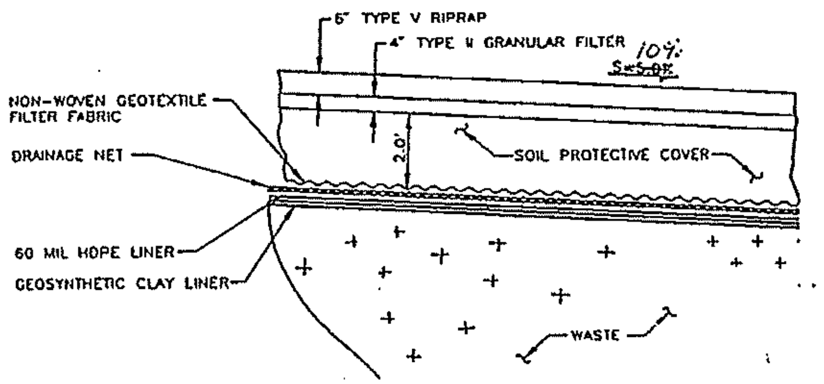
APPENDIX 2

Calculations - Integrity of the HDPE Liner Against Failure from Normal and Tensile Stresses

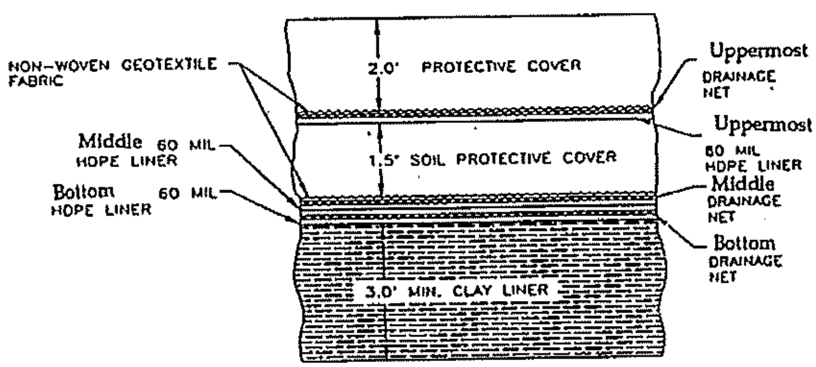
H&L ENGINEERING	CLIENT:	USPCI - LONE MOUNTAIN FACILITY	SHEET 1	OF 30
	PROJECT:	LANDFILL CELL 15	COMPUTED:	JDB
	FEATURE:	HDPE LINER - INTEGRITY ANALYSIS	CHECKED:	KCS
	PROJECT NO.:	64.44.700	DATE:	May 31, 1996

I. Gap Analysis

Analyze the 60 mil HDPE liner for bridging the small gap of the drainage net located between the liners of the triple liner system. The following diagrams illustrate the soil, liner, drainage net, and filter fabric configuration for the interior of the cell and cell closure.



Closure profile



Cell Floor profile

A. Properties of the 60 mil and 80 mil SLT HDPE liner are tabulated below.

Property	60 mil	80 mil
20°C:		
Tensile Strength at Break (lbs/inch of width)	240	320
Ultimate Elongation at Break (percent)	700	700
Yield Strength (lbs./in. of width)	140	190
Elongation at Yield (percent)	13	13

B. Load on Liner

Since the bearing capacity of the material under the liner is 2000 lbs./ft.^{2}, operational loading that could create an uneven loading distribution on the clay, and therefore on the liner, must be maintained less than 2000 lbs./ft.^{2}.

For the gap analysis, the ultimate loading at closure is the critical load. The maximum height of fill, and therefore the maximum loading, will occur at the center ridge line of the closure cap above the sump 5 flow line.

Maximum height of fill over the liner on the net = 1,456.4 - 1,363.2 = 93.2 ft.

Unit Weights are:

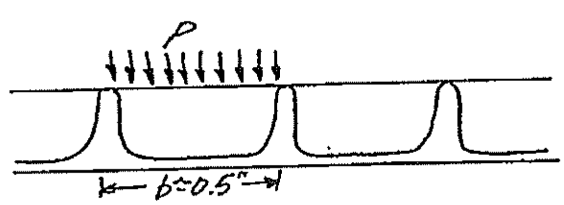
Soil Cover	=	125	lbs./ft.^{3}
Waste	=	120	lbs./ft.^{3}
Gravel	=	110	lbs./ft.^{3}

Ultimate Dead Load at the center of the closure cap is:

$$L_D = 5.5(125) + 0.8(110) + 86.9(120)$$
$$= 11,204 \text{ lbs./ft.}^2$$
$$= 530.3 \text{ KN/m}^2$$

C. Check bridging capability over gap in drainage net.

The drainage net will consist of SLT GS-228. The gap between ridges of the drainage net is 0.461 inches (1.171 cm). Use a gap of 0.5 inches (1.27 cm) to be conservative.



P_b = 530.3 KN/m^2 x 1.27 cm x 1 m/100 cm = 6.73 KN/m

Analyze the liner when covered with soil, at which time the temperature of the liner would be fairly constant. The minimum tensile yield strength of the 60 mil liner @ 20°C is 140 lbs/in.

Yield Strength = 140 lbs./in. x 4.4 N/lb. x 39.37 in./m x 1 KN/1000 N
= 24.3 KN/m

Presented below is a figure entitled "Fig. 7 Chart for the design of a geotextile bridging a crack." This figure was obtained from a paper entitled "Design of Geotextiles Associated with Geomembranes" by J. P. Giroud, which is presented in a publication entitled, "Geotextiles and Geomembranes Definitions, Properties and Design Selected Papers, Revisions and Comments, Third Edition, Industrial Fabrics Association International, 1985, St. Paul, Minnesota. Curves A, B, and C on the graph in the figure represent three different geotextiles-geomembranes having different properties, i.e. tensile yield strength and elongation at yield. Thus, the points A, B, and C were plotted in the above referenced paper based on α being the tensile strength at yield and ϵ being the elongation at yield for each specific geotextile-geomembrane. The dashed curves were then drawn by the author of the paper between zero and the points as plotted. These curves represent the relationship between elongation of the geotextile-geomembrane and the tensile force on the geotextile-geomembrane, with the tensile force varying between zero and the tensile strength at yield for the material.

Plotting the yield point (23.9 KN/m-tensile yield strength, 13% elongation at yield) on the figure below, a curve for the 60 mil liner can be developed. Thus, in the same way that the author had generated curves A, B, and C, a curve for the 60 mil HDPE liner was generated by plotting the yield point for the 60 mil liner consisting of the tensile yield strength α = 23.9 kN/m and elongation ϵ = 13%.

The pb curves presented on the graph were generated by the author based on tension membrane theory identified by the same author, J. P. Giroud in a paper entitled "Designing with Geotextiles" contained in the same publication referenced above. The parameter p represents the overburden pressure on the liner, whereas the parameter b represents the width of a crack that the liner must span, in this case identified as the width between ribs of the drainage net. According to the tension membrane theory, α the tensile force per unit width on the liner can be determined from the following equation:

$$\alpha = p\, b\, f(\epsilon)$$

Where:

α = tensile force per unit width
p = pressure exerted on the geotextile/geomembrane
b = width of the crack that the geotextile/geomembrane is spanning
$f(\epsilon)$ = function of elongation ϵ defined in Table II of the publication as:

ϵ (%)	$f(\epsilon)$
0	$\propto$
2	1.47
3	1.23
4	1.08
5	0.97
6	0.9
8	0.8
10	0.73
12	0.69
15	0.64
20	0.58
30	0.53
40	0.51
45-70	0.50

The family of pb curves presented on the graph were generated from the above equation for a given value of pb and varying values of elongation ϵ. Using the above equation, a curve has been plotted on Figure 7 for a value of pb = 6.73 kN/m for Landfill Cell 15. This curve intersects the dashed curve (which was generated and plotted on the graph for the 60 mil HDPE liner) at a value of elongation ϵ equal to approximately 3.5%. From the above table with a value for elongation ϵ of 3.5%, $f(\epsilon)$ would be 1.16. With $f(\epsilon)$ equal to 1.16 and pb equal to 6.73 kN/m, α would be equal to 7.8 kN/m tensile force on the liner (7.8 = 1.16 x 6.73), based on the equation presented above. Thus, the actual factor of safety (which is the yield strength divided by the tensile force on the liner) would be 3.1 (3.1 = 24.3 /7.8).

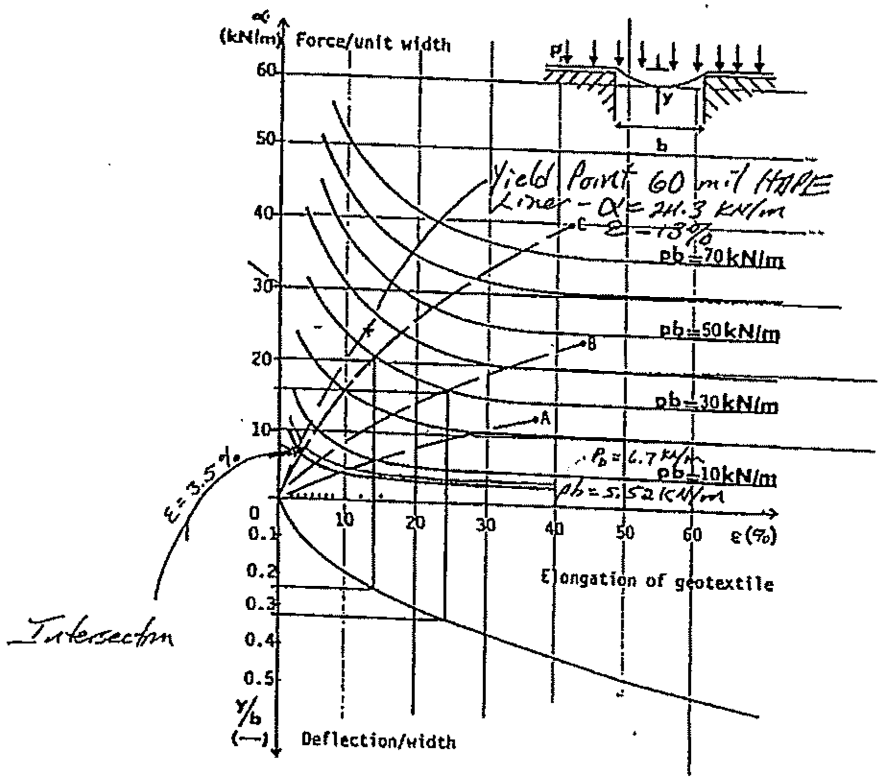
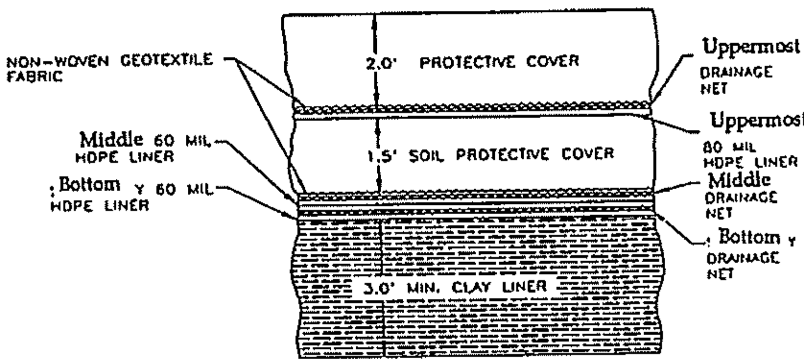


Fig. 7 Chart for the design of a geotextile bridging a crack.

III. Loading During Installation of 2-foot Soil Protective Cover and during cell operation.

The triple liner system for Cell 15 consists of:



In order to protect the synthetic liner and leachate collection systems from stress due to uneven loadings from installation and operational machinery, the bearing capacity of the underlying clay or soil must not be exceeded. As long as the foundation for the synthetic liner remains firm and does not fail, then differential stresses on the liner, other than settlement already discussed, should not occur that could damage the liner.

Assumed possible loading to be checked are:

A. HS-20 Truck Loading
B. Standard Caterpillar Track-Type Loader with 3.25 cy bucket
C. Standard Caterpillar D6D Track-Type Dozer
D. Caterpillar 824C Wheel-Type Dozer Tractor (40 psi)
E. Caterpillar 966C Wheel Loader with 3.25 cy bucket (40 psi)
F. Caterpillar 14G Motor Grader
G. Caterpillar 235 Excavator/Backhoe

The bearing capacity of the clay liner material under the primary and secondary HDPE liners as provided by Applied Geotechnical Engineering Consultants are:

Condition	Criteria
Ultimate Clay Bearing Capacity	6,000 lbs/ft^2
Allowable Clay Bearing Capacity	2,000 lbs/ft^2
Allowable Clay Bearing Capacity with Impact Loading	3,000 lbs/ft^2
Load Distribution through Soil Protective Cover	0.5 H : 1.0 V
Soil Protective Cover Density	125 lbs/ft^3

The bearing capacity of the soil protective cover can be determined from the following equation which assumes a Safety Factor of 3.

Allowable Bearing Capacity = 540 + (120 x width of load) + (510 x depth of soil cover)

The above equation is valid for a single track, or dual tire.

The Allowable Bearing Capacity due Impact Loading, is obtained by multiplying the above value by 1.5. The Factor of Safety against failure is reduced to 2.0.

A. HS-20 TRUCK LOADING

1. - Check impact and static HS-20 Truck Loadings for several combination of tire pressures and soil protective cover thicknesses using the following assumptions and equations:

   i) The contact area for the truck tires approximates a rectangular area with the length approximately 40 percent greater than the width. Therefore the width equals:

   width of load = ((16,000 lbs/tire pressure)/1.4)$^{1/2}$

   The resulting length of the load equals:

   length of load = 1.4(width of load)

   ii) The area over which the load is distributed on the clay assuming a load distribution 0.5H to 1.0 V is:

   Length = (soil cover thickness)(0.5)(2 directions) + length of load
   Width = (soil cover thickness)(0.5)(2 directions) + width of load
   Area of load applied = Length x Width

   iii) Bearing Pressure on the Clay

   $$= \frac{\text{applied truck load + fill material load}}{\text{Area}}$$

   iv) The impact loading factor to be applied is 1.1, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

   $$= \frac{\text{1.1 x applied truck load + fill material load}}{\text{Area}}$$

The results of the calculations are given on the following page. Results indicate that the static and impact loadings on the clay liner are acceptable for all of the conditions analyzed with 2.0' minimum soil cover depth.

H&L ENGINEERING	CLIENT: USPCI - LONE MOUNTAIN FACILITY PROJECT: LANDFILL CELL 15 FEATURE: HDPE LINER - INTEGRITY ANALYSIS PROJECT NO.: 64.44.700	SHEET 9 OF 30 COMPUTED: JDB CHECKED: KCS DATE: May 31, 1996

USPCI LONE MOUNTAIN - LANDFILL CELL 15
EQUIPMENT FOR SOIL PROTECTIVE COVER - HS-20 TRUCK LOADINGS

GIVEN:					
	equipment description =	DUMP TRUCKS WITH HS-20 LOADINGS			
	loading on tire =	16000	pounds		
	axel loadings =	8000.00	front	32000.00	rear
	type and size of tire =	11Lx16	front	23.1-26	rear
	width of tire (Wc) =	11.0	in. front	23.10	in. rear
	soil cover density =	125	lbs / C.F.		

FRONT:

Tire Operating Pressure (P) psi	Tire Contact Area (Ac) sq. in.	Tire Contact Width (Wc) in.	Tire Contact Length (Lc) in.	Soil Cover Height (H) ft.	Width of Load on Liner (L) in.	Length of Load on Liner (L) in.	Liner Loading Area (A) sq. ft.	Soil Weight (Ws) lbs.	Applied Bearing Pressure (Tp) lbs./s.f.	Allowable Bearing Pressure (Ab) lbs./s.f.	Safety Factor for Ultimate Bearing Pressure
STATIC LOADING											
100	40.00	5.35	7.48	2.00	29.35	31.48	6.42	1603.97	873.45	2000.00	6.87
95	42.11	5.48	7.68	2.00	29.48	31.68	6.49	1621.51	866.71	2000.00	6.92
90	44.44	5.63	7.89	2.00	29.63	31.89	6.56	1640.60	859.53	2000.00	6.98
100	40.00	5.35	7.48	2.50	35.35	37.48	9.20	2875.12	747.26	2000.00	8.03
95	42.11	5.48	7.68	2.50	35.48	37.68	9.28	2901.39	743.33	2000.00	8.07
90	44.44	5.63	7.89	2.50	35.63	37.89	9.38	2929.94	739.13	2000.00	8.12
IMPACT LOADING											
100	40.00	5.35	7.48	2.00	29.35	31.48	6.42	1603.97	935.80	2600.00	8.34
95	42.11	5.48	7.68	2.00	29.48	31.68	6.49	1621.51	928.38	2600.00	8.40
90	44.44	5.63	7.89	2.00	29.63	31.89	6.56	1640.60	920.49	2600.00	8.47
100	40.00	5.35	7.48	2.50	35.35	37.48	9.20	2875.12	790.74	2600.00	9.86
95	42.11	5.48	7.68	2.50	35.48	37.68	9.28	2901.39	786.41	2600.00	9.92
90	44.44	5.63	7.89	2.50	35.63	37.89	9.38	2929.94	781.79	2600.00	9.98

REAR:

Tire Operating Pressure (P) psi	Tire Contact Area (Ac) sq. in.	Tire Contact Width (Wc) in.	Tire Contact Length (Lc) in.	Soil Cover Height (H) ft.	Width of Load on Liner (L) in.	Length of Load on Liner (L) in.	Liner Loading Area (A) sq. ft.	Soil Weight (Ws) lbs.	Applied Bearing Pressure (Tp) lbs./s.f.	Allowable Bearing Pressure (Ab) lbs./s.f.	Safety Factor for Ultimate Bearing Pressure
STATIC LOADING											
100	160.00	[illegible]	14.97	2.00	34.69	38.97	9.39	2346.82	1954.43	2000.00	3.07
95	168.42	10.97	15.36	2.00	34.97	39.36	9.56	2389.21	1924.19	2000.00	3.12
90	177.78	11.27	15.78	2.00	35.27	39.78	9.74	2435.51	1892.36	2000.00	3.17
100	160.00	10.69	14.97	2.50	40.69	44.97	12.71	3970.73	1571.71	2000.00	3.82
95	168.42	10.97	15.36	2.50	40.97	45.36	12.90	4032.40	1552.46	2000.00	3.86
90	177.78	11.27	15.78	2.50	41.27	45.78	13.12	4099.67	1532.11	2000.00	3.92
IMPACT LOADING											
100	160.00	10.69	14.97	2.00	34.69	38.97	9.39	2346.82	2124.88	2600.00	3.67
95	168.42	10.97	15.36	2.00	34.97	39.36	9.56	2389.21	2091.61	2600.00	3.73
90	177.78	11.27	15.78	2.00	35.27	39.78	9.74	2435.51	2056.60	2600.00	3.79
100	160.00	10.69	14.97	2.50	40.69	44.97	12.71	3970.73	1697.64	2600.00	4.59
95	168.42	10.97	15.36	2.50	40.97	45.36	12.90	4032.40	1676.45	2600.00	4.65
90	177.78	11.27	15.78	2.50	41.27	45.78	13.12	4099.67	1654.07	2600.00	4.72

2. Check height of cover requirements for cover over the tertiary liner system with the soil over the primary liner providing the base for the tertiary liner.

Assume 24 inches of soil cover above the tertiary liner and a tire pressure of 90 psi.

Bearing Pressure applied by HS-20 truck loading on the soil sub-base would be the same as that applied to the clay in the previous calculation assuming a height of cover of 24 inches and a tire pressure of 90 psi.

Bearing Pressure on the soil base = 1,892 lbs/ft^2

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(11.27"/(12"/ft)) + 510(24"/(12"/ft))$

$= 1,673$ lbs/ft^2

Since 1,892 lbs/ft^2 ≈ 1,673 lbs/ft^2 OK

Actual Safety Factor = 3(1,673)/1,892 = 2.7 OK

Bearing Pressure for impact loading = 2,057 lbs/ft^2

Allowable $= (3/2)(1,673$ lbs/ft$^2)$

$= 2,510$ lbs/ft^2

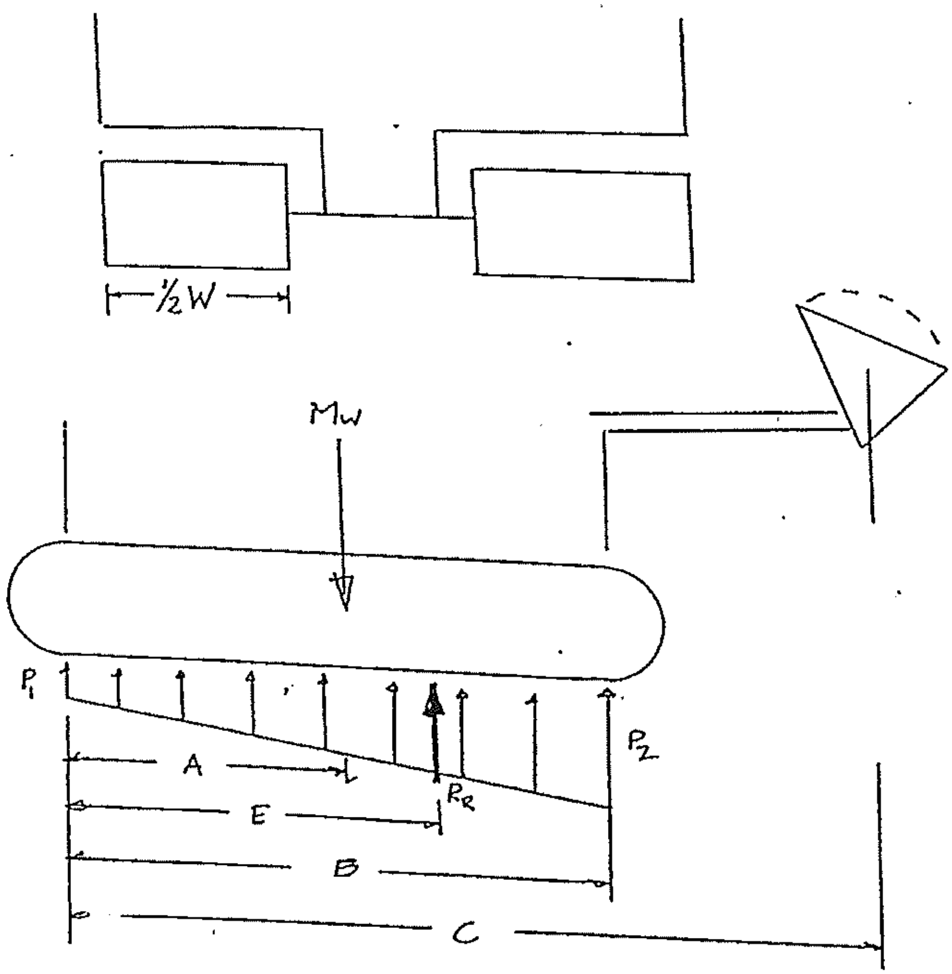
Since 2,057 lbs/ft^2 < 2,510 lbs/ft^2 OK

Actual Safety Factor = 3(1,673)/2,057 = 2.4 OK

The single axle HS-20 loading was analyzed instead of the double axle HS-20 loading because it gives the most conservative value. The results are more conservative because the load per dual on the double axle is 12,000 lbs and the load distributions will not overlap between axles in the 30-inch layer of soil protective cover. This loading applies also to end dump trucks of H-20 loading and 10-wheel and dump trucks with double axles.

B. Caterpillar 977L with 3.25 cy bucket

All of the following calculations are based on information obtained from Caterpillar Machinery. The older machinery is assumed to be worse case due to the motor being located at the front section rather than the rear, as in the case of the newer equipment.



A =	Distance from back drive to empty machine center of gravity with the bucket extended to its furthest horizontal distance
B =	Distance between sprockets - Wheel base
C =	Distance from back drive to load center of gravity
D =	Track Width
R_r=	Resultant reaction from the pressure distribution
P_1=	Pressure on minimum side of pressure distribution
P_2=	Pressure on maximum side of pressure distribution
M_w=	Machine operating weight with an empty bucket
L_w=	Load weight in bucket
E =	Distance of R_r from rear drive

The standard dimension to be used for the Caterpillar 977L with the 3.25 cy bucket are:

A = 57.48"	M_w = 49,380 lbs
B = 111.1"	(1/2)W = 18"
C = 185.02"	Υ = 125 lbs/ft³ = 3,375 lb/cy

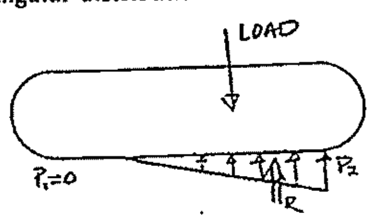
L_w= 3.25(3,375) = 10,969 lbs
R_r=49,380 + 10,969 = 60,349 lbs

$\Sigma M_n = 0 = 60{,}349(E) - 10{,}969(185.02) - 49{,}380(57.48)$

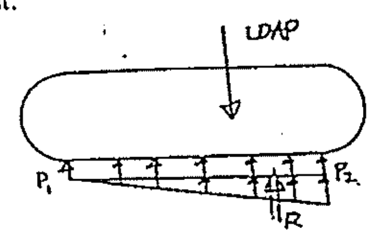
Solving for E = E=80.66 in. = 6.72 feet

If (B-E)(3) $\leq$ B, then the loading placed on the soil under the track is triangular as shown below (left) with $P_1 = 0$.

If (B-E)(3) > B, then the loading is a triangular distribution superimposed on a rectangular distribution as shown below at the right.



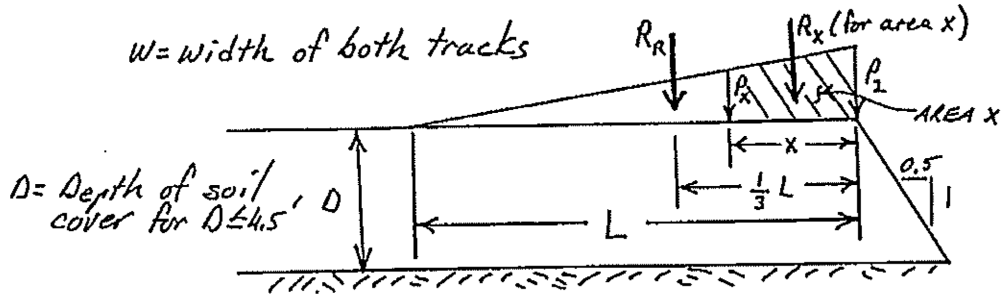
(B-E)(3) $\leq$ B



(B-E)(3) > B

(B-E)(3) = (111.1-80.66)(3) = 91.32 < 111.11 therefore the loading distribution is triangular as shown above (left).

The worst case load distributed through the soil layer to the clay is not obtained by assuming the entire triangular distribution acting over the applicable area of the track is transferred to the clay surface. Obviously, from the triangular distribution, the larger loading occurs as P_2 is approached. For example, if only loading on the clay created by the pressure distribution right of R_r is compared with the loading on the clay from the pressure distribution left of R_r it can be shown that the loading created right of Rr is much greater than that created left of R_r. This is obvious due to the fact that the total load right of R_r is greater, but the area over which the maximum loading will occur can be derived mathematically as follows:



Note: R_x is assumed to distribute in 3 directions (the front and two sides) but not to the back since the back part of the pressure triangle would tend to counter R_x in the backward direction. This is a more conservative approach than assuming Rx is being distributed in four directions, because it will distribute the same load over a smaller area of the underlying clay liner.

$R_r = 0.5P_2LW \qquad P_x/(L\text{-}X) = P_2/L$

$P_2 = 2R_r/(LW) \qquad P_x = P_2(L\text{-}X)/L$

$R_x = (P_2 + P_x)(W)(X)/2$

$$\begin{aligned} R_x &= 0.5(P_2 + (P_2(L\text{-}X)/L))(W)(X) \\ &= 0.5P_2WX(1 + ((L\text{-}X)/L)) \\ &= 0.5P_2WX(2\text{-}(X/L)) \end{aligned}$$

Given that the bearing area from one track does not overlap the other track, the Bearing Area is as follows:

$$\begin{aligned} \text{Area} &= 2 \text{ tracks}[(0.5D + X)(2D(0.5) + (W/2))] \\ &= ((D/2) + X)(2D + W) \\ &= D^2 + D(W/2) + X(2D + W) \end{aligned}$$

Bearing on the Clay:

$$= (R_x + \text{Weight of Soil})/\text{Bearing area}$$

$$= (R_x + \Upsilon_s(\text{Bearing Area})(\text{Soil depth}))/\text{Bearing area}$$

$$= \frac{0.5P_2WX(2\text{-}(X/L)) + \Upsilon_s D(D^2 + D(W/2) + X(2D + W))}{(D^2 + D(W/2) + X(2D + W))}$$

$$= \frac{0.5P_2WX(2\text{-}(X/L))}{(2D + W)((D/2) + X)} + \Upsilon_s D$$

To derive the maximum, take the derivative of the bearing with respect to x, and set the equation equal to zero and solve.

$$\frac{d}{dx}\left(\frac{u}{v}\right)=\frac{\left(V\left(\frac{du}{dx}\right)-U\left(\frac{dv}{dx}\right)\right)}{V^2}$$

$$\frac{du}{dx}=\frac{P_2WX}{2}\left(-\frac{1}{L}\right)+\left(2-\left(\frac{X}{L}\right)\right)\left(\frac{P_2W}{2}\right)$$

$$=-\frac{(P_2WX)}{2L}+(P_2W)-\frac{(P_2WX)}{2L}$$

$$=P_2W-\frac{P_2WX}{L}=P_2W\left(1-\frac{X}{L}\right)$$

$$\frac{dv}{dx}=(2D)+W$$

$$\frac{\Upsilon bearing}{\Upsilon x}=\frac{(2D+W)\left(\frac{D}{2}+X\right)\left[P_2W\left(1-\frac{X}{L}\right)\right]-\frac{P_2WX}{2}\left(2-\frac{X}{L}\right)(2D+W)}{\left[(2D+W)\left(\frac{D}{2}+X\right)\right]^2}$$

Reducing the equation leads to:

$$0=x^2+DX-DL$$

From the quadratic equation ax^2+bx+c, where

$$x=\frac{-b\pm\sqrt{b^2-4ac}}{2a}$$

..and by substituting the correct values into the equation gives the formula for maximum loading:

$$x=\frac{-D\pm\sqrt{D^2-4(1)(-DL)}}{2(1)}=\frac{-D\pm\sqrt{D^2+4DL}}{2}$$

1. Check maximum loading for Caterpillar 977L with standard bucket on clay base under primary liner.

Assume the depth of soil cover (D) equals 1.5 feet.

$$L = (B\text{-}E)(3) = (111.1\text{-}80.66)(3) = 91.32" = 7.61 \text{ feet}$$

$$W = 2(18") = 36 \text{ inches} = 3 \text{ feet}$$

$$x = \frac{-1.5 \pm \sqrt{1.5^2+4(1.5)(7.61)}}{2} = 2.71 \ feet$$

$$P_2 = \frac{2R_r}{LW} = \frac{2(60,349 \ lbs)}{7.61(3)} = 5,287 \ lbs/ft^2$$

$$R_x = \frac{P_2WX}{2}\left(2-\frac{X}{L}\right) = \frac{5,287(3)(2.71)}{2}\left(2-\frac{2.71}{7.61}\right)$$

$$= 35,330 \text{ lbs}$$

$$Bearing \quad Area = D^2+D\frac{W}{2}+X(2D+W)$$

$$= 1.5^2+1.5\left(\frac{3}{2}\right)+2.71[2(1.5)+3] = 20.76 \ ft^2$$

$$\text{Bearing Pressure on the Clay} = \frac{R_x + \Upsilon_2(\text{Bearing area})(\text{soil depth})}{\text{Area}}$$

$$= \frac{35,330 \text{ lbs} + (18"/12)(125)(20.76)}{20.76}$$

$$= 1,703 \text{ lbs/ft}^2 < 2,000 \text{ lbs/ft}^2 \text{ OK}$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition. 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(35,330) + (18"/12)(125)(20.76)}{20.76} = 2,230 \text{ lbs/ft}^2$$

Since 2,230 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate for the clay under the primary liner for the 977L with the 3.25 cy bucket.

2. Check maximum loading for Caterpillar 977L with standard bucket on soil base under tertiary liner.

Assuming the same depth of soil cover of 1.5 feet used in the pervious calculation, the bearing on the soil sub-base of the tertiary liner would be the same as that calculated on the clay sub-base.

Bearing Pressure on the soil base = 1,703 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(1.5) + 510(1.5)$

$= 1{,}485$ lbs/ft²

Since 1,703 lbs/ft² > 1,485 lbs/ft² - NOT ADEQUATE

Increase the depth of soil cover to 2.0 feet.

Bearing Pressure on the soil base (with 2.0' cover) = 1,614 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(1.5) + 510(2)$

$= 1{,}740$ lbs/ft²

Since 1,614 lbs/ft² < 1,740 lbs/ft² OK

Actual Safety Factor = 3(1,740)/1,614 = 3.2 OK

Bearing Pressure for impact loading (with 2.0' cover) = 1,887 lbs/ft²

Allowable $= (3/2)(1{,}740)$

$= 2{,}610$ lbs/ft²

Since 1,887 lbs/ft² < 2,610 lbs/ft² OK

Actual Safety Factor = 3(1,740)/1,887 = 2.8 OK

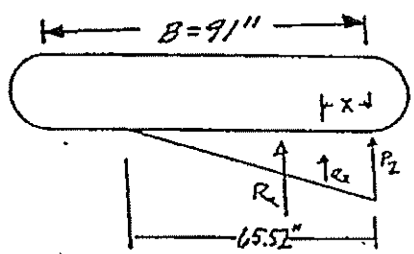
C. Track Type Dozer - Caterpillar D6D

The standard track type loader analyzed (977L) had an effective track length carry weight of the equipment with a full bucket of approximately 72 percent. During a discussion with Don Miller (an engineer for the Caterpillar Tractor Company) Mr. Miller said that for flat dozing,as would be the case while spreading the soil protective cover, the assumption of 72% effective track area would be conservative. The 72% effective track length will therefore be used in the following calculations.

Weight = 35,500 lbs (highest weight assuming ripper attachment)
Track Width (W/2) = 18 inches
Track length on ground (B) = 91 inches
Effective Track Length (L) = 0.72(91) = 65.52 inches = 5.46 ft

Assume that triangular loading applies.

The worst case condition utilized the same equations that were developed for the worst case conditions in the front end loader section (977L).



R_r = 35,500 lbs

1. Check Clay sub-base for primary liner.

Assume a height of cover = 1.5 feet

$$x = \frac{-1.5 \pm \sqrt{1.5^2 + 4(1.5)(5.46)}}{2} = 2.21 \ feet$$

$$P_2 = \frac{2R_r}{LW} = \frac{2(35,500 \ lbs)}{5.46(3)} = 4,335 \ lbs \ / ft^2$$

$$R_x = \frac{P_2 WX}{2}\left(2 - \frac{X}{L}\right) = \frac{4,335(3)(2.21)}{2}\left(2 - \frac{2.21}{5.46}\right)$$

$$= 22,924 \text{ lbs}$$

$$\text{Bearing Area} = D^2 + D\frac{W}{2} + X(2D+W)$$

$$= 1.5^2 + 1.5\left(\frac{3}{2}\right) + 2.21[2(1.5)+3] = 17.76\ ft^2$$

$$\text{Bearing Pressure on the Clay} = \frac{R_x + \Upsilon_2(\text{Bearing area})(\text{soil depth})}{\text{Area}}$$

$$= \frac{22{,}924 \text{ lbs} + (1.5)(125)(17.76)}{17.76}$$

$$= 1{,}478 \text{ lbs/ft}^2 < 2{,}000 \text{ lbs/ft}^2$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(22{,}924) + (1.5)(125)(17.76)}{17.76} = 1{,}736 \text{ lbs/ft}^2$$

Since 1,776 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate.

2. Check maximum loading on soil base under tertiary liner.

Utilize soil bearing for 2 foot cover.

Bearing Pressure on the soil base = 1,273 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$$= 540 + 120(1.5) + 510(2.0) = 1{,}740 \text{ lbs/ft}^2$$

Since 1,273 lbs/ft² < 1,740 lbs/ft² OK

Actual Safety Factor = 3(1,740)/1,273 = 4.1 OK

Bearing Pressure for impact loading = 1,478 lbs/ft²

Allowable = (3/2)(1,740) = 2,610 lbs/ft²

Since 1,478 lbs/ft² < 2,610 lbs/ft² OK

Actual Safety Factor = 3(1,740)/1,478 = 3.5 OK

D. Caterpillar 824C and 824B Wheel Type Dozer

1. Machine Specifications - reference "Caterpillar Performance Handbook" edition 16.

Model	Weight	Wheel Base
824C	66,975 lbs	11' 7"
824B	73,480 lbs	11' 8"

The 824B is an older model. Because the 824B is heavier, loading for the 824B will be analyzed. If the 824B proves to be acceptable, extrapolate to the lighter 824C.

Caterpillar representatives in Peoria, Illinois indicated that the weight distribution is 55% to the rear and 45% to the front. Based upon this load distribution, the maximum load for a single tire would be:

$$= 0.55*(73,480)/2 = 20,207 \text{ lbs.}$$

Assuming a maximum tire pressure of 40 psi, the area over which the load is spread at the surface of the soil cover is:

$$= 20,207 \text{ lbs} / 40 \text{ psi} = 505 \text{ in}^2$$

Given that the standard tire width is 29.5 inches, the dimensions over which the load is spread is calculated as follows:

$$\text{length} = 505 \text{ in}^2 / 29.5 \text{ in} = 17.1 \text{ inches}$$

The area over which the load is distributed on the clay assuming a load distribution 0.5H to 1.0 V, and a soil protective cover thickness of 18 inches is:

$$\text{Length} = (18")(0.5)(2 \text{ directions}) + 29.5" = 47.5 \text{ inches}$$
$$\text{Width} = (18")(0.5)(2 \text{ directions}) + 17.1" = 35.1 \text{ inches}$$

Area of load applied $= (47.5)(35.1) = 1,667 \text{ in}^2 = 11.58 \text{ ft}^2$

$$\text{Bearing Pressure on the Clay} = \frac{\text{applied truck load + fill material load}}{\text{Area}}$$

$$= \frac{20,207 \text{ lbs} + (18"/12)(125)(11.58)}{11.58}$$

$$= 1,923 \text{ lbs/ft}^2 < 2,000 \text{ lbs/ft}^2 \text{ OK}$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(20,207) + (1.5)(125)(11.58)}{11.58} = 2,281 \text{ lbs/ft}^2$$

Since 2,281 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate.

2. Check maximum loading on soil base under tertiary liner.

Assuming the same depth of soil cover of 1.5 feet used in the previous calculation, the bearing on the soil sub-base of the tertiary liner would be the same as that calculated on the clay sub-base.

Bearing Pressure on the soil base = 1,932 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(17.1/12) + 510(1.5)$

$= 1,476$ lbs/ft²

Since 1,932 lbs/ft² > 1,476 lbs/ft² NOT ADEQUATE

Increase soil cover depth to 2.0 feet. Bearing pressure on the soil base under 2.0 foot soil cover depth equals 1,573 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(17.1/12) + 510(2)$

$= 1,731$ lbs/ft²

Since 1,573 lbs/ft² < 1,731 lbs/ft² OK

Actual Safety Factor = 3(1,731)/1,573 = 3.3 OK

Bearing Pressure for impact loading (with 2.0 foot cover) = 1,840 lbs/ft²

Allowable $= (3/2)(1,731)$

$= 2,597$ lbs/ft²

Since 1,840 lbs/ft² < 2,597 lbs/ft² OK

Actual Safety Factor = 3(1,731)/1,840 = 2.8 OK

E. Caterpillar 966C Wheel Loader with 3.25 cy bucket

According to the Caterpillar Tractor Company in Peoria, Illinois, with the bucket empty and under static conditions, it can be assumed that 50 to 55% of the loader weight is on the front axle. With the bucket fully loaded and under static conditions, it can be assumed that 70 to 80% of the total weight of the machine and the load is on the front axle of the rubber tired loader. To be conservative, this analysis assumes that 80% of the load is on the front end of the loader.

1. Machine Specifications

Shipping weight = 37,100 lbs
rated capacity = 3.43 cy
Load weight = 3.43(125 lbs/ft^3)(27 ft^3/cy) = 11,576 lbs
Total weight = 48,676 lbs

Load on one front tire = 0.5(48,676)(80%) = 19,470

Assuming a maximum tire pressure of 40 psi, the area over which the load is spread at the surface of the soil cover is:

= 19,470 lbs / 40 psi = 486.8 in^2

Given that the standard tire width is 20.5 inches, the dimensions over which the load is spread is calculated as follows:

length = 486.8 in^2 / 20.5 in = 23.74 inches

The area over which the load is distributed on the clay assuming a load distribution 0.5H to 1.0 V, and a soil protective cover thickness of 18 inches is:

Length = (18")(0.5)(2 directions) + 23.74" = 41.7 inches
Width = (18")(0.5)(2 directions) + 20.50" = 38.5 inches

Area of load applied = (41.7)(38.5) = 1,605 in^2 = 11.15 ft^2

$$\text{Bearing Pressure on the Clay} = \frac{\text{applied truck load + fill material load}}{\text{Area}}$$

$$= \frac{19{,}470 \text{ lbs} + (18''/12)(125)(11.15)}{11.15}$$

$$= 1{,}934 \text{ lbs/ft}^2 < 2{,}000 \text{ lbs/ft}^2 \text{ OK}$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(19,470) + (1.5)(125)(11.15)}{11.15} = 2,283 \text{ lbs/ft}^2$$

Since 2,283 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate.

2. Check maximum loading on soil base under tertiary liner.

Assuming the same depth of soil cover of 1.5 feet used in the pervious calculation, the bearing on the soil sub-base of the tertiary liner would be the same as that calculated on the clay sub-base.

Bearing Pressure on the soil base = 1,934 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(20.5/12) + 510(1.5)$

$= 1,510$ lbs/ft²

Since 1,934 lbs/ft² < 1,510 lbs/ft² NOT ADEQUATE, therefore increase soil cover depth to 2.0 foot.

Increase soil cover thickness to 2.0 foot.

Bearing Pressure on the soil base (with 2.0' depth)= 1,570 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(20.5/12) + 510(2)$

$= 1,765$ lbs/ft²

Since 1,570 lbs/ft² < 1,765 lbs/ft² OK

Actual Safety Factor = 3(1,765)/1,570 = 3.4 OK

Bearing Pressure for impact loading (with 2.0' depth) = 1,834 lbs/ft²

Allowable $= (3/2)(1,765)$

$= 2,648$ lbs/ft²

Since 1,834 lbs/ft² < 2,648 lbs/ft² OK

Actual Safety Factor = 3(1,765)/1,834 = 2.9 OK

F. Caterpillar 14G Motor Grader

Dan Cordray of the Caterpillar Tractor Company (phone # 309-675-4655) in Peoria, Illinois, provided the following information regarding the 14G Motor Grader:

Wheel Loading Distribution	w/out ripper	with ripper
Front Axles	10,700 lbs	11,010 lbs
Rear Axles	29,950 lbs	34,310 lbs
Total	40,650 lbs	45,320 lbs

Wheel base - from front axle to center of tandem axles in rear = 21' 2"

Distance from the center of the tandem axle to either rear wheel = 32.5"

1. Assuming the load to be distributed equally on the rear tandem axle and assuming the weight distribution to be equal on all four tires of the rear axle, then the load per tire on the rear axle is:

Load on one rear tire = 34,310/4 = 8,576 lbs (use 9,000 lbs)

Assuming a maximum tire pressure of 45 psi, the area over which the load is spread at the surface of the soil cover is:

$$= 9{,}000 \text{ lbs} / 45 \text{ psi} = 200 \text{ in}^2$$

Given that the standard tire width is 20.5 inches, the dimensions over which the load is spread is calculated as follows:

length = $200 \text{ in}^2 / 20.5 \text{ in} = 9.8$ inches

The area over which the load is distributed on the clay assuming a load distribution 0.5H to 1.0 V, and a soil protective cover thickness of 18 inches is:

Length = (18")(0.5)(2 directions) + 9.8" = 27.8 inches
Width = (18")(0.5)(2 directions) + 20.5" = 38.5 inches

Area of load applied = $(27.8)(38.5) = 1{,}070 \text{ in}^2 = 7.43 \text{ ft}^2$

$$\text{Bearing Pressure on the Clay} = \frac{\text{applied truck load + fill material load}}{\text{Area}}$$

$$= \frac{9{,}000 \text{ lbs} + (18''/12)(125)(7.43)}{7.43}$$

$$= 1{,}399 \text{ lbs/ft}^2 < 2{,}000 \text{ lbs/ft}^2 \text{ OK}$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(9,000) + (1.5)(125)(7.43)}{7.43} = 1,641 \text{ lbs/ft}^2$$

Since 1,641 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate.

Check the bearing pressure if for some reason two of the back tires were to carry all of the load distributed to the rear of the 14G.

Load per tire = 34,310/2 = 17,155 lbs (use 17,200 lbs)

Assuming a maximum tire pressure of 45 psi, the area over which the load is spread at the surface of the soil cover is:

= 17,200 lbs / 45 psi = 382 in²

Given that the standard tire width is 20.5 inches, the dimensions over which the load is spread is calculated as follows:

length = 382 in² / 20.5 in = 18.6 inches

The area over which the load is distributed on the clay assuming a load distribution 0.5H to 1.0 V, and a soil protective cover thickness of 18 inches is:

Length = (18")(0.5)(2 directions) + 18.6" = 36.6 inches
Width = (18")(0.5)(2 directions) + 20.5" = 38.5 inches

Area of load applied = (36.6)(38.5) = 1,409 in² = 9.79 ft²

$$\text{Bearing Pressure on the Clay} = \frac{\text{applied truck load} + \text{fill material load}}{\text{Area}}$$

$$= \frac{17,200 \text{ lbs} + (18''/12)(125)(9.79)}{9.79}$$

$$= 1,944 \text{ lbs/ft}^2 < 2,000 \text{ lbs/ft}^2 \text{ OK}$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(17,200) + (1.5)(125)(9.79)}{9.79} = 2,296 \text{ lbs/ft}^2$$

Since 2,296 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate.

2. Check maximum loading on soil base under tertiary liner.

Assuming the same depth of soil cover of 1.5 feet used in the pervious calculation, the bearing on the soil sub-base of the tertiary liner would be the same as that calculated on the clay sub-base. Also assume that the loading is distributed between all four of the rear tires.

Bearing Pressure on the soil base = 1,399 lbs/ft^2

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(9.8/12) + 510(1.5)$

$= 1,403\ lbs/ft^2$

Since 1,399 lbs/ft^2 < 1,403 lbs/ft^2 OK

Actual Factor of Safety = 3(1,403)/1,399 = 3.0 OK

Bearing Pressure for impact loading = 1,641 lbs/ft^2

Allowable = (3/2)(1,403) = 2,104 lbs/ft^2

Since 1,641 lbs/ft^2 < 2,104 lbs/ft^2 OK

Actual Factor of Safety = 3(1,403)/1,641 = 2.6 OK

Now, assume that the loading is distributed carried by only two of the rear tires.

Bearing Pressure on the soil base = 1,944 lbs/ft^2

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(18.6/12) + 510(1.5)$

$= 1,491\ lbs/ft^2$

Since 1,944 lbs/ft^2 > 1,746 lbs/ft^2 NOT ACCEPTABLE

Therefore, increase soil cover thickness to 2.0 feet.

Bearing Pressure on the soil base (with 2.0 foot cover) = 1,553 lbs/ft^2

Allowable Bearing Pressure for the soil (S.F.=3)

$= 540 + 120(18.6/12) + 510(2.0)$

$= 1,746\ lbs/ft^2$

Since 1,553 lbs/ft² > 1,746 lbs/ft² OK

Actual Factor of Safety = 3(1,746)/1,553 =3.4 OK

Bearing Pressure for impact loading = 1,814 lbs/ft²

Allowable = (3/2)(1,746) = 2,619 lbs/ft²

Since 1,814 lbs/ft² < 2,619 lbs/ft² OK

Actual Factor of Safety = 3(1,746)/1,814 =2.9 OK

G. Caterpillar 235 Excavator - Backhoe

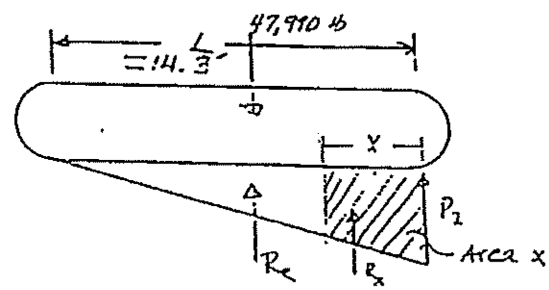
Based upon information provided by Caterpillar Machinery, the following characteristics belong to the 235 Excavator - Backhoe:

Operating weight = 86,700 lbs
Weight of Material in 2.75 cy bucket, assuming soil density of 125 lbs/ft³ = 9,280 lbs
Total weight loaded = 95,980 lbs
Weight on one track = 0.5(95,980) = 47,990 lbs

Loading Distribution:

Assume that triangular loading applies. The worst case condition utilized the same equations that were developed for the worst case conditions in the front end loader section (977L).

W= track width
=3'



1. Check Clay sub-base for primary liner.

Assume a height of cover = 1.5 feet

$$x=\frac{-1.5\pm\sqrt{1.5^2+4(1.5)(14.3)}}{2}=3.9\ feet$$

$$P_2=\frac{2R_r}{LW}=\frac{2(47,990\ lbs\ )}{14.3(3)}=2,237\ lbs\ /ft^2$$

$$P_x=\frac{P_2(L-X)}{L}=\frac{2,237\ (14.3-3.9)}{14.3}=1,626\ lbs\ /ft^2$$

$$R_x=\frac{P_2+P_x}{2}(W)(X)=\frac{2,237+1,626}{2}(3)(3.9)$$

$$=22,599\ lbs$$

The area over which the load is distributed on the clay assuming a load distribution 0.5H to 1.0 V, and a soil protective cover thickness of 18 inches is:

Length = (1.5')(0.5)(2 directions) + 1.5' = 3.0 feet
Width = (1.5')(0.5)(1 direction) + 4.4' = 5.15 feet
Area = (3.0)(5.15) = 15.5 ft²

$$\text{Bearing Pressure on the Clay} = \frac{R_x + T_2(\text{Bearing area})(\text{soil depth})}{\text{Area}}$$

$$= \frac{22{,}599 \text{ lbs} + (1.5)(125)(15.5)}{15.5}$$

$$= 1{,}646 \text{ lbs/ft}^2 < 2{,}000 \text{ lbs/ft}^2$$

The impact loading factor to be applied is 1.2, supplied by the American Association of State Highway and Transportation Officials in "Standard Specifications for Highway Bridges," Edition 12. Therefore Bearing Pressure on the clay due to impact loading:

$$= \frac{1.2(22{,}599) + (1.5)(125)(15.5)}{15.5} = 1{,}937 \text{ lbs/ft}^2$$

Since 1,937 lbs/ft² < 3,000 lbs/ft², the 18 inch soil protective layer is adequate.

2. Check maximum loading on soil base under tertiary liner.

Assuming the same depth of soil cover of 1.5 feet used in the pervious calculation, the bearing on the soil sub-base of the tertiary liner would be the same as that calculated on the clay sub-base.

Bearing Pressure on the soil base = 1,646 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$$= 540 + 120(1.5) + 510(1.5)$$
$$= 1{,}485 \text{ lbs/ft}^2$$

Since 1,646 lbs/ft² > 1,485 lbs/ft² NOT ACCEPTABLE

Therefore, increase soil cover thickness to 2.0 foot.

Bearing Pressure on the soil base (with 2.0 foot cover) = 1,572 lbs/ft²

Allowable Bearing Pressure for the soil (S.F.=3)

$$= 540 + 120(1.5) + 510(2)$$
$$= 1{,}740 \text{ lbs/ft}^2$$

Since 1,572 lbs/ft² < 1,740 lbs/ft² OK

Actual Safety Factor = 3(1,740)/1,572 = 3.3 OK

Bearing Pressure for impact loading = 1,837 lbs/ft^2

Allowable = (3/2)(1,740) = 2,610 lbs/ft^2

Since 1,837 lbs/ft^2 < 2,610 lbs/ft^2 OK

Conclusions

The following types of equipment may be used on top of the **PRIMARY** soil protective cover with the following conditions:

	Equipment	Conditions
A.	HS-20 Loadings	2.0' min. cover
B.	Caterpillar Track Type Loader (977L with 3.25 cy bucket)	1.5' min. cover
C.	Caterpillar D6D Track Type Dozer	1.5 min. cover
D.	Caterpillar 824C/824B Wheel Type Dozer	1.5' min. cover
E.	Caterpillar 966C Wheel Type Dozer	1.5' min. cover
F.	Caterpillar 14G Motor Grader	1.5' min. cover
G.	Caterpillar 235 Track Type Excavator/Backhoe	1.5' min. cover

The following types of equipment may be used on top of the **TERTIARY** protective cover with the following conditions:

	Equipment	Conditions
A.	HS-20 Loadings	2.0' min. cover
B.	Caterpillar Track Type Loader (977L with 3.25 cy bucket)	2.0' min. cover
C.	Caterpillar D6D Track Type Dozer	2.0' min. cover
D.	Caterpillar 824C/824B Wheel Type Dozer	2.0' min. cover
E.	Caterpillar 966C Wheel Type Dozer	2.0' min. cover
F.	Caterpillar 14G Motor Grader	2.0' min. cover
G.	Caterpillar 235 Track Type Excavator/Backhoe	2.0' min. cover

APPENDIX 3

Liner Anchor Trench Design Calculations

I. Determine the tensile force potentially acting upon the liner.

A. Determine the tensile force created by a wind load upon the liner.

Wind coming across the top of the cell creates an uplift pressure on the liner. According to "Fundamental Theory of Structures" by D. Allan Firmage, Robert E. Krieger Publishing Company, Huntington New York, the uplift pressure on the leeward side of a roof can be determined from:

$P' = -0.70p$ for all values of the slope of the roof.

for $p = 0.002558(V^2)$

where v equals the wind velocity in miles/hour.

From Figure 3.6 of the above referenced publication, the fastest wind velocity, having a 50 year recurrence interval in the vicinity of the USPCI Lone Mountain site is between 80 and 85 miles per hour.

At 80 miles per hour:

$p = 0.002558(80^2) = 16.37$ lbs/ft^2

and

$P' = -0.70(16.37) = -11.46$ lbs/ft^2 normal to the slope

At 85 miles per hour:

$p = 0.002558(85^2) = 18.48$ lbs/ft^2

and

$P' = -0.70(18.48) = -12.94$ lbs/ft^2 normal to the slope

Note: Because the liner is flexible and not rigid, it is unlikely that the full force due to wind loading (based on the above equations) will be developed. Thus, the above uplift pressures are most likely conservative.

The following assumptions will apply in this analysis:

1. Assume that the wind load acts on the upper 30' of slope length. This is based on observation of what has occurred during major wind events at the USPCI's Grassy Mountain, Grayback Mountain and Lone Mountain Facilities.
2. Assume that the wind load will act perpendicular to the line between point A-B as shown on the attached sheet.
3. Assume, based upon observation, that the liner lifts a maximum of 0.5' to 1.0' above the slope due to the lift forces created by the wind. Thus, it would be 1.0' to 1.5' above the line A-B'.
4. Assume a uniform loading condition along the span length of the liner between point A and B'. The liner will therefore result in a parabolic configuration over the lifted portion of the liner between points A and B'.

HA&L ENGINEERING

CLIENT: USPCI/Laidlaw - LONE MOUNTAIN FACILITY
PROJECT: LANDFILL CELL 15
FEATURE: LINER TRENCH DESIGN
PROJECT NO.: 64.44.700

COMPUTED: JDB
CHECKED: KCS
DATE: May 96

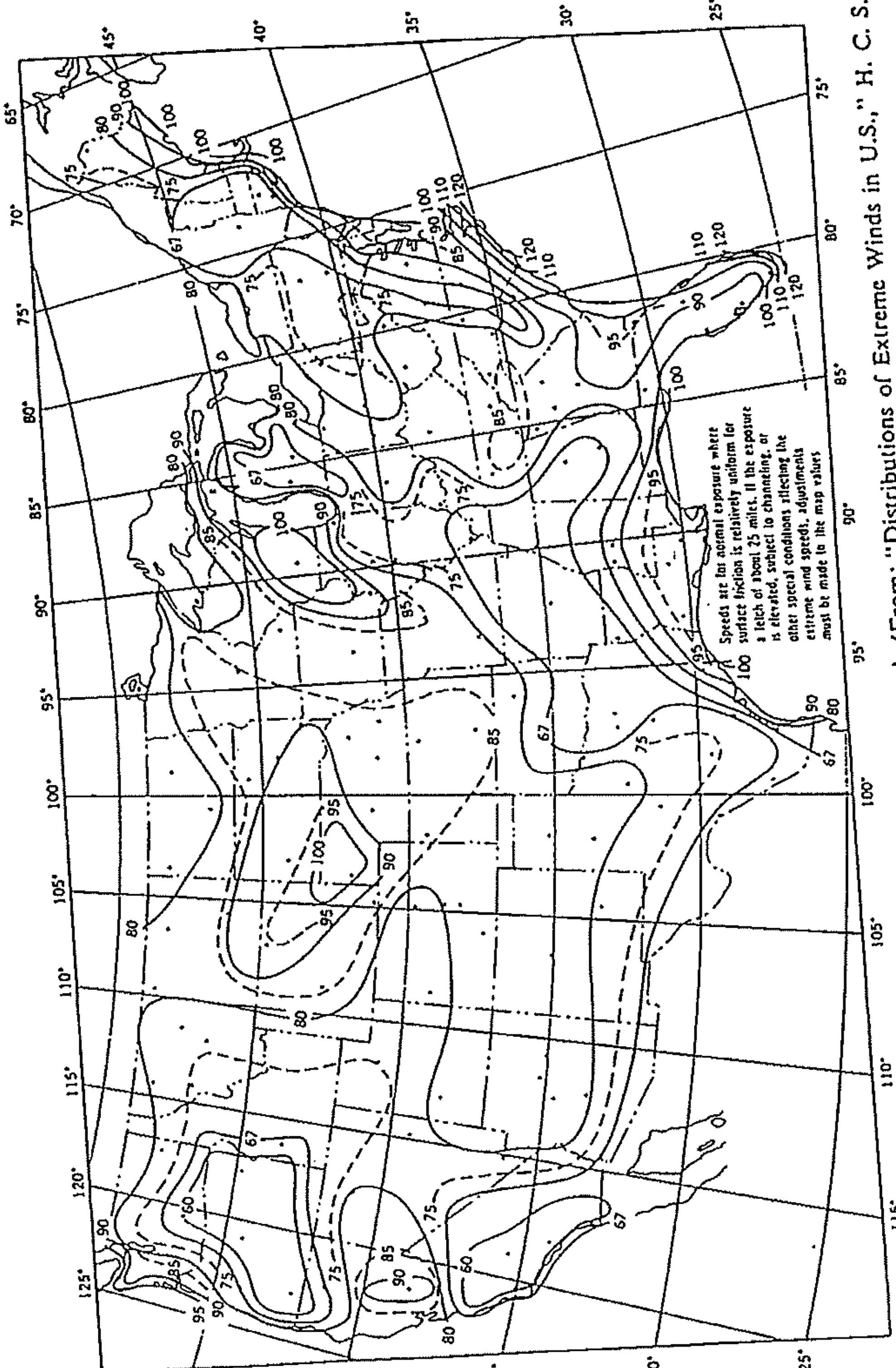


Fig. 3.6. Fastest mile of wind, fifty year mean recurrence interval. (From: "Distributions of Extreme Winds in U.S.," H. C. S. Thom, *Proceedings of the Structural Division of ASCE*, Paper 2433, April 1960.)

HA&L ENGINEERING

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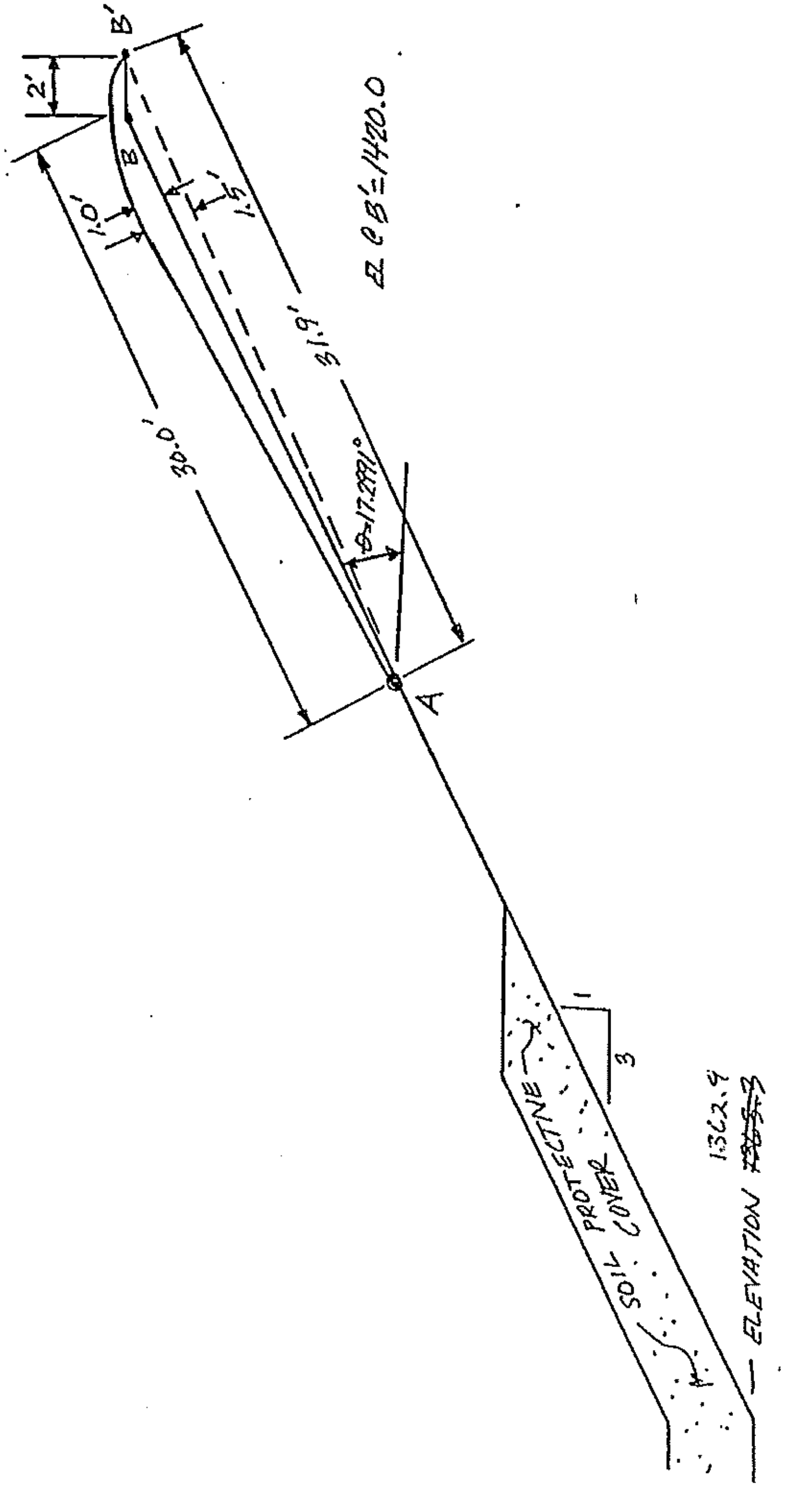
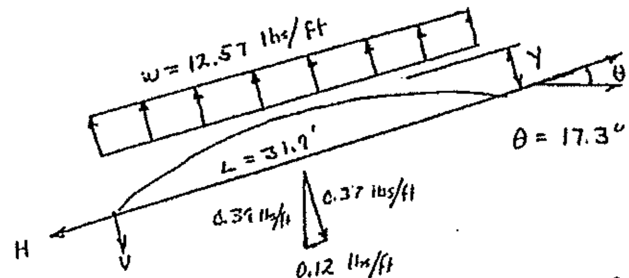


The loading due to wind would be as follows:



@ 85 mph
w = (12.94 - 0.37) lb/ft
w = 12.57 lb/ft

Counter to the wind load would be the component of the weight of the liner perpendicular to the slope. The unit weight of the 80 mil HDPE liner is 0.39 lbs/ft², or considering a strip one unit foot wide would be 0.39 lbs/ft of length.

Using equations for a uniformly loaded cable with respect to span length:

$$H=\frac{WL^2}{8y'}$$

$$V=\frac{WL}{2}$$

y' = maximum deflection y assumed to be 1.5 feet above the liner A-B'. Check maximum deflection.
L = span length = 31.9 ft.

Wind Velocity (mph)	Horizontal Force (H) (lbs/ft)	Vertical Force (V) (lbs/ft)	Tension (T) (lbs/ft)
80	940	177	957
85	1066	200	1085

Use the average of the values presented in the table above (i.e. 1021 lbs/ft)

B. Tensile force caused by load due to soil cover placed on the slope.

A tensile load can occur in the HDPE liner due to the protective soil cover placed up the slope of the cell on top of the liner. The tensile load placed on the HDPE liner due to the soil cover depends on the vertical height that the soil cover is placed up the slope.

To this must be added the component of the weight of the liner which is parallel to the slope:

Slope Length:
Vertical differential = 1420 - 1364.9 = 55.1'
Horizontal distance = 3(55.1) = 165.3'
Slope Length = $(165.3^2 + 55.1^2)^{1/2}$ = 174.2'
Weight/ft = 0.39 (174.2) = 67.9 lbs/ft
Weight parallel to slope = 67.9 sin(18.4349°) = 21.5 lbs/ft

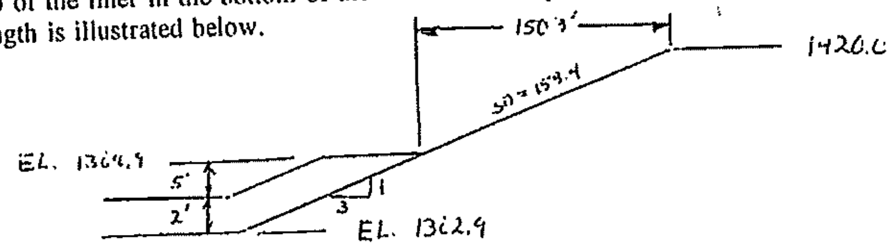
Vertical Height ft.	Tension in Liner lbs/ft width*	Tension + Weight Parallel to Slope
4	134	156
5	248	270
6	363	385
8	592	614

*Values provided by Applied Geotechnical Engineering Consultants (AGEC) of SLC, Utah.

Use the values for a vertical height of 5 feet.

According to specification, no equipment should be allowed on the inside slopes of the facility once the liner is in place. Thus no other loading should be present than those discussed herein.

C. Tensile force created by temperature variation in the liner and thermal contraction. Check the longest exposed slope length, which would be at the sump. A soil cover will be placed on top of the liner in the bottom of the cell and initially 5' vertical feet up the slopes. The slope length is illustrated below.



Coefficient of Thermal Expansion $\alpha = 1.2 \times 10^{-4}$ in/in/ $\circ$ F

Thermal Strain $\epsilon = (\alpha)(\Delta T) = (\Delta L)/L$

Where:

$\Delta L =$ Change in Length $= L\epsilon$

$L =$ Length of liner exposed to temperature extremes

$\Delta T =$ Assumed to be 115° F

Therefore:

$$\Delta L = 158.4(1.2 \times 10^{-4} \text{ in/in/} \circ \text{F})(115)(12 \text{ in/ft})$$

$$= 26.2 \text{ inches} = 2.2 \text{ foot}$$

All of this potential 2.2 foot of change in length will not result in stress being created in the liner. When the liner is placed, there is slack left in the liner. The liner is generally deployed and welded on the sideslopes during the cooler periods of the day, so that when the liner expands as it heats up the welders don't have to deal with wrinkles in the liner. Thus, it would be reasonable to assume at a minimum there would be at least 1.0 foot of slack in the liner down the 158.4-foot slope of the cell. Thus, this slack would be removed before a tensile stress is created in the liner. Assume that ΔL which creates stress = 2.2-1.0=1.2'.

Thus the stress in the liner (ϵ) equals $\Delta L/L = (1.2)/(158.4) = 0.008$

The theoretical tensile stress $\sigma = E\epsilon$ Where E = Modulus of Elasticity

Poly-Flex Lining has indicated a Modulus of Elasticity for their 60 and 80 mil liner to be 80,000 psi.

$$\sigma_{theoretical} = 80,000\ (.008) = 640 \text{ psi}$$

This is the theoretical stress. The actual stress is approximately 50% of the theoretical stress, due to a property associated with polyethylene material known as thermal stress relaxation. As addressed in the polyethylene pipe design manual, according to ASTM 2513, when a thermal gradient develops in polyethylene material due to a temperature change, the viscoelastic polyethylene molecules react in a manner which significantly dissipates the thermally imposed stress. Thus, a major portion of the stress induced by a temperature change is dissipated as the polyethylene material tries to contract. The measured thermal stress has been found to be half of the theoretical or calculated value where an instantaneous temperature change has occurred.

$$\sigma_{act} = 0.5\sigma_{theoretical}$$

$$\sigma_{act} = 0.5(640) = 320 \text{ psi}$$

$$\text{Tensile Force (T)} = 320(A)$$

where A = area = (12 inches)(.08) = 0.96 in^2/ft for 80 mil liner

Therefore: $T = 320(.96) = 307$ lbs/ft

D. The total tensile force would be the summation of the various forces analyzed above, including the force created by the wind load, the force created by the soil cover material placed 5 feet up the side slope, and the force created by temperature differential. Thus, the total tensile force is:

$$T_{total} = 1021 + 270 + 307 = 1598 \text{ lbs/ft}$$

The anchor trench should be designed such that it will resist pull-out loads up to loads that approach some design strength value of the liner with a safety factor applied. It is desirable to allow the liner to pull out of the trench prior to liner failure. It is much easier to repair a liner trench than to repair a failed liner. Therefore, the trench should be designed to resist liner pull out up to a certain percentage of the actual liner strength. The tensile force computed above must be compared with this percentage of actual liner strength to ensure that the tensile forces on the liner do not exceed this value.

II. Determine the required configuration of the anchor trench to resist liner pullout up to a percentage of the actual liner yield strength or liner seam shear strength. Reference for design "Designing with Geosynthetics" by Robert M. Koerner, Prentice-Hall, Englewood Cliffs, New Jersey. As indicated above, the anchor trench should be designed such that it will resist pull-out loads up to loads that approach some design strength value of the liner with a safety factor applied. It is desirable to allow the liner to pull out of the trench prior to liner failure. It is much easier to repair a liner trench than to repair a failed liner. Therefore, the trench should be designed to resist liner pull out up to a certain percentage of the actual liner strength.

The liner tensile yield strength and liner seam strength in shear for various liners are presented below. These values were obtained from the manufacturer's data sheets for Gundle and Poly-Flex and from the "Geotextile Fabrics Report - 1995 Specifiers Guide", December 1994 for NSC and SLT liners.

A. Liner Tensile Strength at Yield

Liner Thickness Mils	SUPPLIER							
	Gundle		NSC		Poly-Flex		SLT	
	HD	HDT	HD	HD-T	HD	HD-T	HD	HD-T
60 mil (lbs/in) 60 mil (lbs/ft)	140 1680	140 1680	132 1584	132 1584	138 1656	126 1512	132 1584	132 1584
80 mil (lbs/in) 80 mil (lbs/ft)	185 2220	185 2220	176 2112	176 2112	184 2208	160 1920	176 2112	176 2112

Use 1584 lbs/ft for 60 mil smooth HDPE
Use 2112 lbs/ft for 80 mil smooth HDPE

Use 1512 lbs/ft for 60 mil textured HDPE
Use 1920 lbs/ft for 80 mil textured HDPE

B. Liner Seam Shear Strength

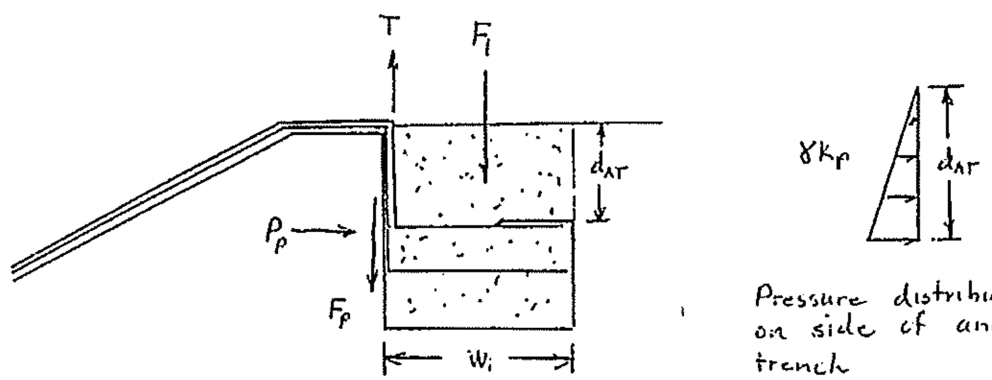
Liner Thickness Mils	SUPPLIER							
	Gundle		NSC		Poly-Flex		SLT	
	HD	HDT	HD	HD-T	HD	HD-T	HD	HD-T
60 mil (lbs/in) 60 mil (lbs/ft)	126 1512	113 1356	120 1440	120 1440	131 1572	120 1440	121 1452	121 1452
80 mil (lbs/in) 80 mil (lbs/ft)	166 1992	151 1812	160 1920	160 1920	175 2100	152 1824	161 1932	161 1932

Note: Seam Strengths are based on extrusion welds since they have lower strength than fusion welds.

Use 1440 lbs/ft for 60 mil smooth HDPE
Use 1920 lbs/ft for 80 mil smooth HDPE

Use 1356 lbs/ft for 60 mil textured HDPE
Use 1812 lbs/ft for 80 mil textured HDPE

C. The anchor trench and potential forces acting upon the liners in the trench are illustrated on the following diagram:



Pressure distribution on side of anchor trench

Because the lower 60 mil liner is to be welded to the upper 80 mil liner in the anchor trench, with a 6-inch layer of soil between the two liners, the soil material above the 80 mil liner would have to be displaced for the liner system to pull out of the trench. Therefore the resisting forces to liner pull out are the weight of the soil material above the 80 mil liner and the friction resistance force along the side of the trench between the weakest plane (i.e. the drainage net liner interface). To evaluate pull-out, the forces in the y-direction are summed and compared with the tensile force acting on the liner which is assumed to be equal to a percentage of the liner strengths indicated above. Terms used in the evaluation are defined below:

β = slope angle
Υ = unit weight of backfill soil
d_{AT} = depth of anchor trench
δ = Soil friction angle
$\delta_{l\text{-}s}$ = angle of shearing resistance of backfill soil to liner material
P_p = $0.5\Upsilon d^2_{At}K_p$
K_p = coefficient of passive earth pressure
= $\tan^2(45 + \delta/2)$
F_p = $P_p(\tan \delta_{l\text{-}n})$ = friction force on bottom of liner along the anchor trench vertical wall
$\delta_{l\text{-}n}$ = friction angle between the liner and drainage net
F_l = $d_{AT}(w_l)\Upsilon$ = force due to the weight of the soil in the anchor trench above the liner
w_l = width of the anchor trench
FS = factor of safety

substituting the correct values gives:

β = 18.4349
Υ = 120 lbs/ft^3
δ = 36° (Recommended by AGEC)
$\delta_{l\text{-}n}$ = 5.5° for liner against the drainage net (Based on testing by AGEC)
K_p = $\tan^2(45 + 36/2) = 3.85$
P_p = $0.5(120)d^2_{AT}(3.85) = 231\ d^2_{At}$
F_p = $P_p(\tan\delta_{l\text{-}n}) = 231\ d^2_{At}(\tan 5.5) = 22.24d^2_{At}$
F_t = $d_{AT}(w_t)\Upsilon$

Σ Forces in the y direction

Pulling Forces = T_{liner}

Resisting Forces = $F_p + F_t$

Safety Factor (SF) = $(T_{liner})/(F_p + F_t)$ or

$$F_p + F_t = T_{liner}/SF$$

The following sheets provide computer printouts which present liner and liner seam strengths with applied safety factors with anchor trench sizes and pull-out potential. Although liner and liner seam strengths and pullout potential for both the 60 mil and 80 mil liner are compared on the following computer printouts, the key liner is the upper 80 mil liner. Design of the anchor trench will be based on the 80 mil liner for the following reason:

1) The 80 mil liner will carry nearly all of the tensile loads due to wind and the soil cover on the slopes.
2) If the 80 mil liner pulls out the anchor trench the resisting forces on the middle and lower 60 mil liners will be much lower than is indicated on the following computer printouts. This effectively increases the safety factors against failure of the 60 mil liners.
3) Failure of the weld in the anchor trench (joining the 60 mil and 80 mil liners) is less critical than pullout of 80 mil liner because failure of this seam is not a failure of the liner systems. If this seam fails the 80 mil liner would likely pullout from the trench, which is preferable to failure of the liner inside the cell.

Using the anchor trench with a depth of 2.75 feet above the top liner and a bottom width of 4.5 feet, the safety factors representing liner pull-out of the 80 mil liners prior to liner failure are as follows:

SF = 1.3 that the 80 mil smooth liner will pull out prior to liner failure. The resisting forces are 1650 lbs/ft, which is greater than the total tensile forces acting on the liner computed previously of 1600 lbs/ft.

SF = 1.2 that the 80 mil textured liner will pull out prior to liner failure. The resisting forces are 1650 lbs/ft, which is greater than the total tensile forces acting on the liner computed previously of 1600 lbs/ft.

Client:	USPCI/LAIDLAW
Project:	LONE MT. LANDFILL CELL 15
Feature:	Liner Anchor Trench
Date:	05/31/96

Sheet 10 of 13
By: JDB
Check:
Date: 05/31/96

80 Mil SMOOTH LINER

Ultimate Liner Tensile Strength at Yield (T, liner):	2112	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	2112	lbs./ft
Computed Safety Factor (FS):	1.28		Computed Safety Factor (FS):	1	
Allowable Liner Tensile Strength at Yield (T, allow):	1653	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	2112	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	4.43	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	3.43	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1851	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	2714	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	261	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	2112	lbs/ft
Solve Equation (set resisting = to T, allow):	-0		Solve Equation (set resisting = to T, allow):	-0	

80 Mil TEXTURED LINER

Ultimate Liner Tensile Strength at Yield (T, liner):	1920	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1920	lbs./ft
Computed Safety Factor (FS):	1.16		Computed Safety Factor (FS):	1	
Allowable Liner Tensile Strength at Yield (T, allow):	1653	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1920	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	4.15	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	3.15	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1700	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	2289	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	220	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	1920	lbs/ft
Solve Equation (set resisting = to T, allow):	-0		Solve Equation (set resisting = to T, allow):	-0	

Client:	USPCI/LAIDLAW
Project:	LONE MT. LANDFILL CELL 15
Feature:	Liner Anchor Trench
Date:	05/31/96

Sheet 11 of 13
By: JDB
Check:
Date: 05/31/96

60 Mil SMOOTH LINER SEAM IN SHEAR

Ultimate Liner Tensile Strength at Yield (T, liner):	1440	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1440	lbs./ft
Computed Safety Factor (FS):	0.87		Computed Safety Factor (FS):	1	
Allowable Liner Tensile Strength at Yield (T, allow):	1653	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1440	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	3.42	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	2.42	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1309	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	1358	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	131	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	1440	lbs/ft
Solve Equation (set resisting = to T, allow):	-0		Solve Equation (set resisting = to T, allow):	-0	

60 Mil TEXTURED LINER SEAM IN SHEAR

Ultimate Liner Tensile Strength at Yield (T, liner):	1356	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1356	lbs./ft
Computed Safety Factor (FS):	0.82		Computed Safety Factor (FS):	1	
Allowable Liner Tensile Strength at Yield (T, allow):	1653	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1356	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	3.29	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	2.29	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1239	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	1216	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	117	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	1356	lbs/ft
Solve Equation (set resisting = to T, allow):	-0		Solve Equation (set resisting = to T, allow):	-0	

Client:	USPCI/LAIDLAW
Project:	LONE MT. LANDFILL CELL 15
Feature:	Liner Anchor Trench
Date:	05/31/96

Sheet 12 of 13
By: JDB
Check:
Date: 05/31/96

80 Mil SMOOTH LINER SEAM IN SHEAR

Ultimate Liner Tensile Strength at Yield (T, liner):	1920	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1920	lbs./ft
Computed Safety Factor (FS):	2.16		Computed Safety Factor (FS):	1.5	
Allowable Liner Tensile Strength at Yield (T, allow):	889	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1280	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	4.4	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	3.4	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1839	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	2681	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	258	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	2097	lbs/ft
Solve Equation (set resisting = to T, allow):	-764		Solve Equation (set resisting = to T, allow):	-817	

80 Mil TEXTURED LINER SEAM IN SHEAR

Ultimate Liner Tensile Strength at Yield (T, liner):	1812	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1812	lbs./ft
Computed Safety Factor (FS):	0.84		Computed Safety Factor (FS):	1.5	
Allowable Liner Tensile Strength at Yield (T, allow):	2163	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1208	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	3.1	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	2.1	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1113	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	982	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	94.6	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	1208	lbs/ft
Solve Equation (set resisting = to T, allow):	509		Solve Equation (set resisting = to T, allow):	0	

Client:	USPCI/LAIDLAW
Project:	LONE MT. LANDFILL CELL 15
Feature:	Liner Anchor Trench
Date:	05/31/96

Sheet 13 of 13
By: JDB
Check:
Date: 05/31/96

60 Mil SMOOTH LINER

Ultimate Liner Tensile Strength at Yield (T, liner):	1584	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1584	lbs./ft
Computed Safety Factor (FS):	0.96		Computed Safety Factor (FS):	1	
Allowable Liner Tensile Strength at Yield (T, allow):	1653	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1584	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	3.65	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	2.65	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1428	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	1617	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	156	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	1584	lbs/ft
Solve Equation (set resisting = to T, allow):	-0		Solve Equation (set resisting = to T, allow):	-0	

60 Mil TEXTURED LINER

Ultimate Liner Tensile Strength at Yield (T, liner):	1512	lbs./ft	Ultimate Liner Tensile Strength at Yield (T, liner):	1512	lbs./ft
Computed Safety Factor (FS):	0.91		Computed Safety Factor (FS):	1	
Allowable Liner Tensile Strength at Yield (T, allow):	1653	lbs./ft	Allowable Liner Tensile Strength at Yield (T, allow):	1512	lbs./ft
Assumed Soil Friction Angle (phi)	36	degrees	Assumed Soil Friction Angle (phi)	36	degrees
Assumed Soil Unit Weight (gamma):	120	pcf	Assumed Soil Unit Weight (gamma):	120	pcf
Assumed Anchor Trench Bottom Width (w1):	4.5	ft	Assumed Anchor Trench Bottom Width (w1):	4.5	ft
Calculated Total Anchor Trench Depth (dT):	3.75	ft	Calculated Total Anchor Trench Depth (dT):	3.54	ft
Assumed Backfill Depth Above Liners (d1):	2.75	ft	Computed Backfill Depth Above Liners (d1):	2.54	ft
Backfill Thickness Between Liners (d2):	0.5	ft	Backfill Thickness Between Liners (d2):	0.5	ft
Anchor Trench Backfill Wt. above Liners (F1):	1485	lbs/ft	Anchor Trench Backfill Wt. above Liners (F1):	1369	lbs/ft
Friction Angle Between Liner and Net	5.5	degrees	Friction Angle Between Liner and Net	5.5	degrees
Coefficient of Passive Earth Pressure (Kp)	3.85		Coefficient of Passive Earth Pressure (Kp)	3.85	
Passive Earth Pressure (Pp)	1748	lbs/sf/ft	Passive Earth Pressure (Pp)	1485	lbs/sf/ft
Friction Force along Trench Vertical Wall (Fp)	168	lbs/ft	Friction Force along Trench Vertical Wall (Fp)	143	lbs/ft
Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft	Anchor Trench Backfill Weight between Liners (P2):	540	lbs/ft
Total Resisting Forces:	1653	lbs/ft	Total Resisting Forces:	1512	lbs/ft
Solve Equation (set resisting = to T, allow):	-0		Solve Equation (set resisting = to T, allow):	-0	

EXHIBIT E

LEACHATE COLLECTION AND REMOVAL SYSTEM DESIGN CRITERIA AND CALCULATIONS

Appendix 1 - Test Data-SLT GS-228 and Gundle XL-14 Drainage Net

Appendix 2 - Uppermost and Middle Leachate Collection System

Appendix 3 - Geotextile Filter Fabric

Appendix 4 - Leachate Withdrawal Pipes

Appendix 5 - Uppermost Sump Capacities

Appendix 6 - Bottom Leachate Detection and Removal System and Action Leakage Rate

Appendix 7 - Bottom Sump Capacities

HA&L ENGINEERING

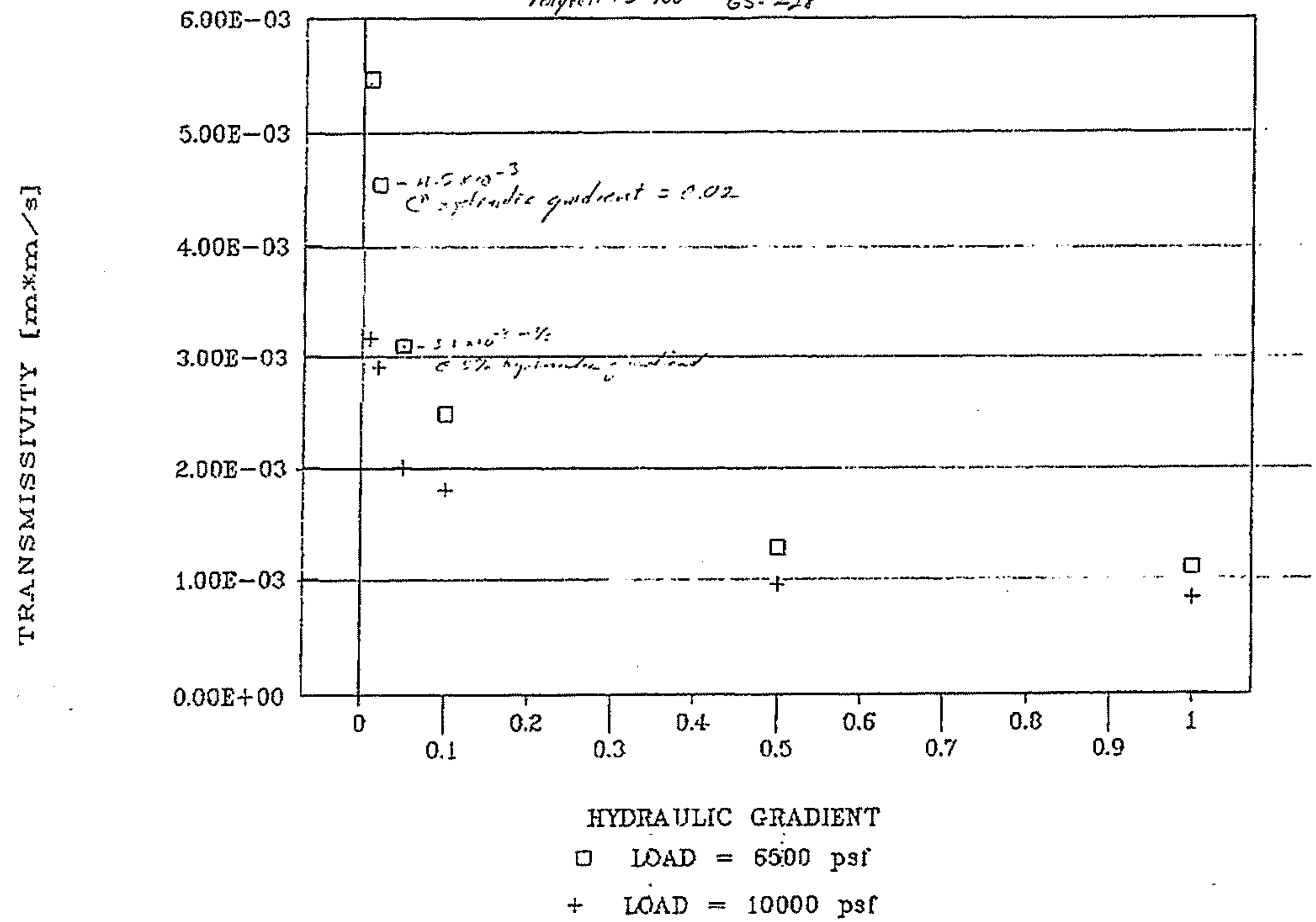
APPENDIX 1

Test Data - SLT GS-228 and Gundle XL-14 Drainage Net

TRANSMISSIVITY TEST RESULTS

PLT/SAND/TEXTILE/GEONET/0.060 HDPE/PLT

Polyfelt TS-700 GS-228



J & L TESTING COMPANY, INC.

GEOTECHNICAL, GEOMEMBRANE, GEOTEXTILE AND CONSTRUCTION MATERIALS TESTING AND RESEARCH

June 25, 1990

Job No. 90G741-05

Gundle Lining Systems, Inc.
19103 Gundle Road
Houston, Texas 77073

ATTENTION: Mr. Mark Cadwallader

RE: TRANSMISSIVITY TEST RESULTS

Dear Mr. Cadwallader:

Attached are the results of the transmissivity tests performed on the following section:

POLYFFLT TS-700 SOIL GEOTEXTILE
GUNDLE XL-14 GEONET
60 MIL HDPE GEOMEMBRANE

The tests were performed in accordance with ASTM D-4716 using normal loads of 6,500 and 10,000 psf and gradients of 0.02, 0.25, and 0.50.

Should you have any questions, please do not hesitate to call.

Sincerely,

J&L TESTING COMPANY, INC.

Richard S. Lacey, P.E.

Richard S. Lacey, P.E.
Manager-Geosynthetic Testing

RSL/dlz
L-D#318

J & L TESTING COMPANY, INC.

TRANSMISSIVITY TEST RESULTS
FOR
GUNDLE LINING SYSTEMS,INC.

ASTM D-4716

TEST CONFIGURATION
TOP LOAD PLATE
SOIL
POLYFELT TS-700
GUNDLE XL-14 GEONET
60 MIL HDPE
BOTTOM LOAD PLATE

DATE: 6-19-90
JOB NO.: 90G741-05
UNIT NO.: 2
TESTED BY: J.B.

SAMPLE: 12"x12"
FLUID: WATER

NORMAL LOAD: 6,500psf

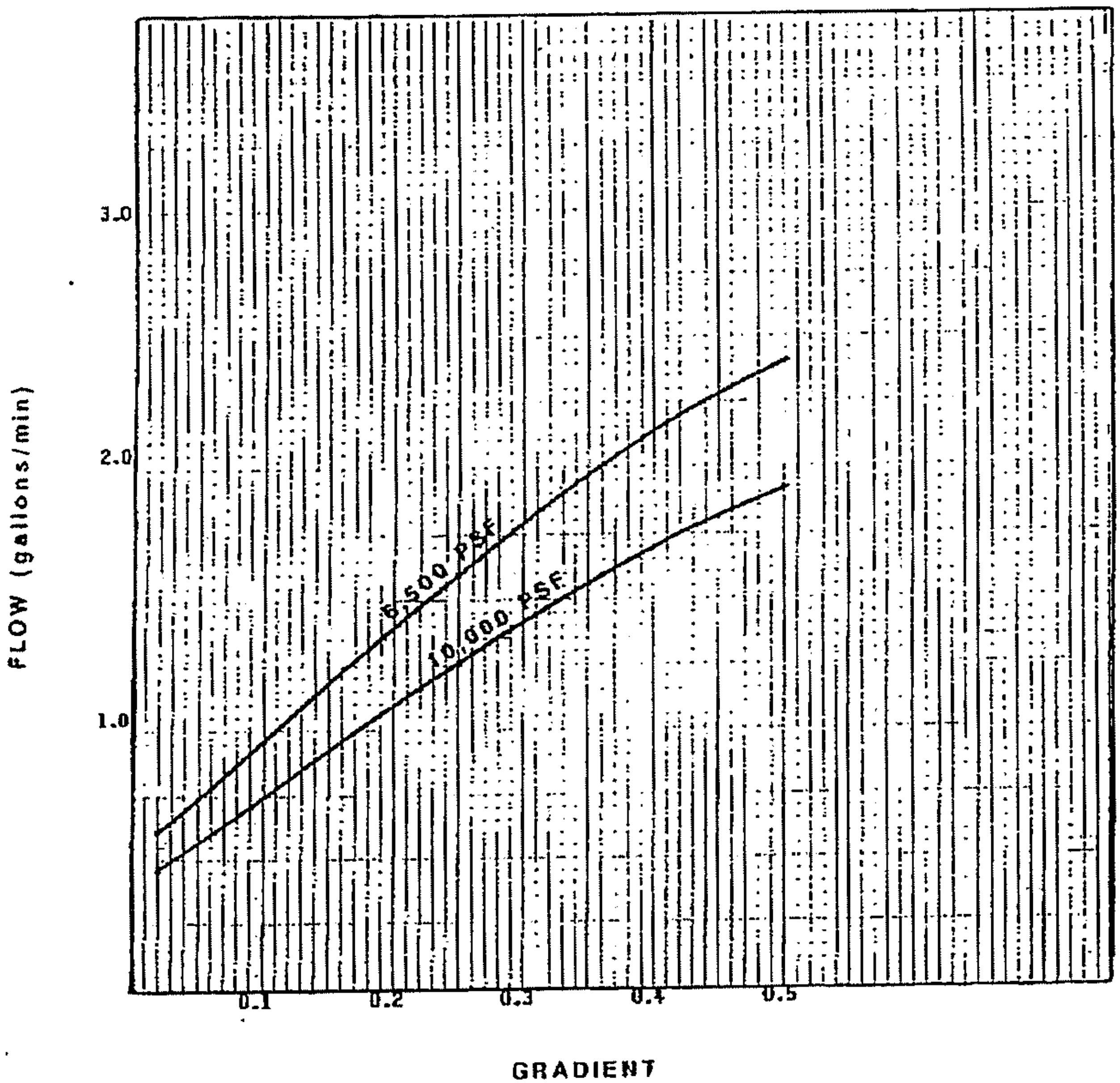
GRADIENT	INITIAL READING (cm)	FINAL READING (cm)	ELAPSED TIME (sec)	FLOW RATE Q (gal/min)	TRANSMISSIVITY M2/SEC
0.02	28.0	30.5	164.3	0.609	6.2993E-03
0.25	32.0	37.0	124.9	1.601	1.3258E-03
0.50	35.0	40.0	82.7	2.418	1.0012E-03

NORMAL LOAD: 10,000psf

GRADIENT	INITIAL READING (cm)	FINAL READING (cm)	ELAPSED TIME (sec)	FLOW RATE Q (gal/min)	TRANSMISSIVITY M2/SEC
0.02	26.0	28.5	218.7	0.457	4.7324E-03
0.25	23.0	28.0	158.7	1.260	1.0435E-03
0.50	33.0	38.0	103.7	1.929	7.9844E-04

Richard S. Lacey, P.E.

Richard S. Lacey, P.E.
Manager Geosynthetic Testing



J & L TESTING COMPANY, I[illegible]

Geotechnical Geomembrane and Geotextile [illegible]

MATERIALS:

Gundle XL-14 Geonet
Polyfelt TS-700 Geotextile
Gundle 60 Mil HDPE

FLUID: Water

UNIT NO.: 2 TEMPERATURE: 70°F

SECTION:

UPPER LOAD PLAT[illegible]
Soil
Geotextile
Geonet
HDPE
LOWER LOAD PLAT[illegible]

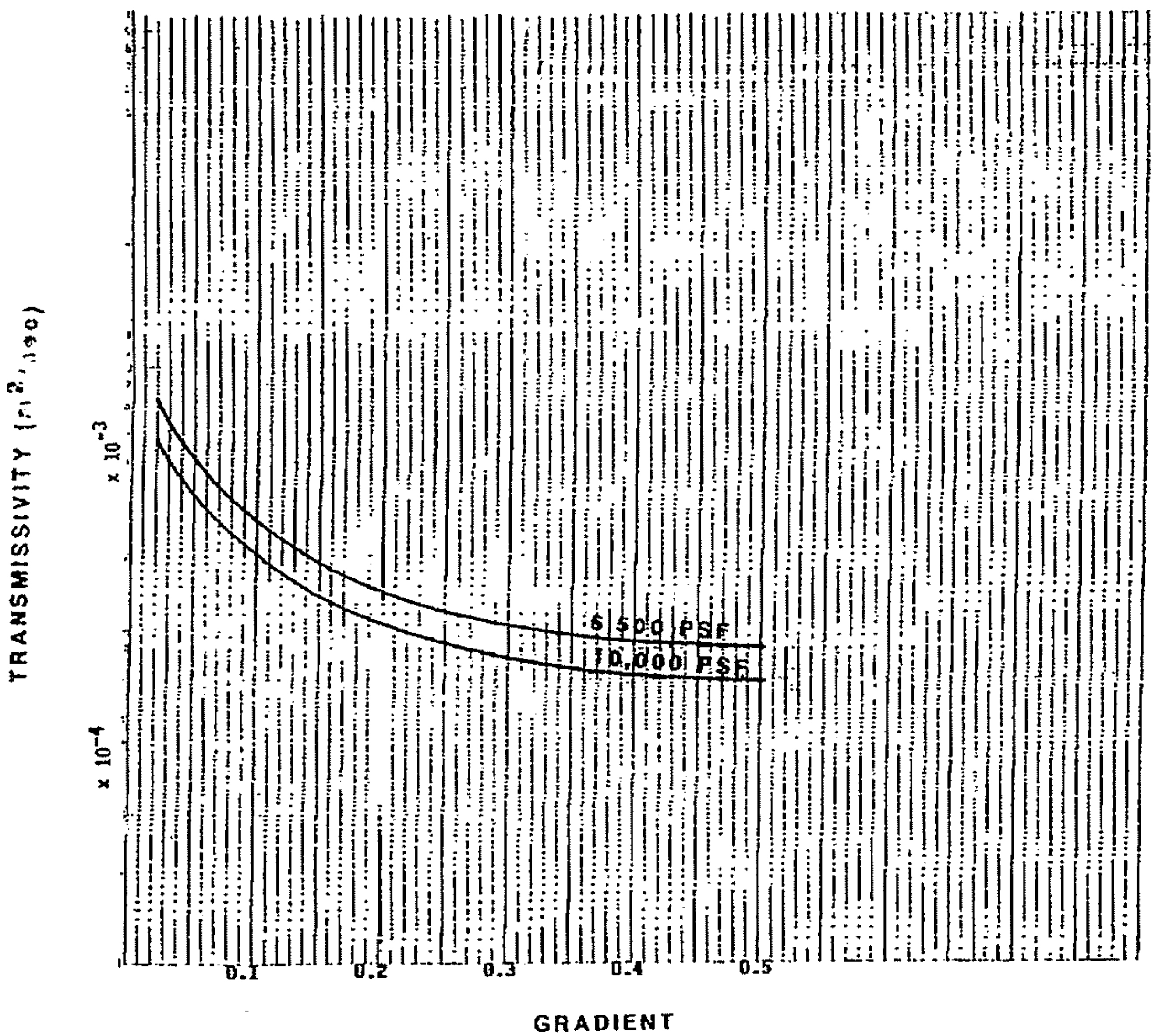
TRANSMISSIVITY TEST RESULT[illegible]

Gundle Lining Systems, Inc.

PROJECT NO : 906741-05

DATE : 6/22/90 FIGURE 1A

G 1.0 16



J & L TESTING COMPANY, I

Geotechnical, Geomembrane and Geotextile I

MATERIALS:

Gundle XL-14 Geonet
Polyfelt TS-700 Geonet
Gundle 60 Mil HDPE

FLUID: Water

UNIT NO.: 2 TEMPERATURE: 70°F

SECTION:

UPPER LOAD PLAT

Soil

Geotextile

Geonet

HDPE

LOWER LOAD PLA

TRANSMISSIVITY TEST RESULT

Gundle Lining Systems, Inc.

PROJECT NO: 906741-05

DATE: 6/22/90

FIGURE 18

NOTE: POLYFELT WAS SOLD TO TENSAR CORPORATION WHO THEN BEGAN MANUFACTURING POLYFELT TS-700 UNDER A NEW PRODUCT NAME OF TENSAR TG-700. ATTACHED IS A COPY OF THE PRODUCT SPECIFICATIONS FOR TENSAR TG-700 VERIFYING THAT THE MATERIALS ARE THE SAME.

Evergreen Technologies

June 21, 1996

Tensar Corporation
1210 Citizens Parkway
Morrow, GA 30260

Subj: TG700 Geotextile Certificate of Compliance

Re : Laidlaw Environmental, Lone Mountain Facility, Order # 001061, PO # 6-8097

Dear Sir/Madam:

This letter certifies that TG700, shipped FOB Evergreen, Alabama, on 6/17/96, manufactured by Evergreen Technologies, meets or exceeds the minimum requirements listed below.

PROPERTY	TEST PROCEDURE	VALUE(1)	
Weight	ASTM D 5261	8.0	oz/yd2
Thickness	ASTM D 5199	90 *	Mil
Grab Strength	ASTM D 4632	210	lbs
Grab Elongation	ASTM D 4632	50	%
Tear Strength	ASTM D 4533	80	lbs
Mullen Burst	ASTM D 3786	400	psi
Puncture Resistance	ASTM D 4833	100	lbs
A.O.S.	ASTM D 4751	.212 *	US Std Sieve
		(70) *	mm
Permittivity	ASTM D 4491	1.3 *	1/sec
Water Permeability	ASTM D 4491	0.3 *	cm/sec
Water Flow Rate	ASTM D 4491	100 *	gpm/sq ft
U.V. Resistance (500 hours)	ASTM D 4355	70	%

(1) Values in weaker principle direction. Unless noted otherwise, these values represent minimum average roll values (i.e. test results from any sampled roll in a lot, tested in accordance with ASTM D 4759-88 shall meet or exceed the minimum values listed).

* Determined at the time of manufacturing, storage and handling conditions which differ from those found in ASTM D 4873-88 may influence these properties.

Unless noted otherwise, this certification is based on testing conducted by Evergreen Technologies Quality Assurance & Quality Control testing laboratories at the time of manufacturing. Evergreen Technologies issues this letter of certification to indicate our commitment to providing our customers with a quality product which will meet or exceed the minimum average roll values in accordance with the applicable American Society for Testing and Materials (ASTM) test method.

Sincerely,

Manoj Tyagi
QA Manager

200 Miller Sellers Drive • Evergreen, Alabama 36401 • Tel. 334.578.9003 • Fax 334.578.6141
A subsidiary of The Tensar Corporation

APPENDIX 2

Uppermost and Middle Leachate Collection System

Design of the Uppermost Leachate Collection and Removal System (ULCRS) consists of the following:

A. Use of EPA's HELP model in order to estimate the amount of leachate that may enter the ULCRS.
B. Based on the results obtained from the HELP model, design the Leachate Collection System with sufficient capacity to convey the leachate to the sumps for removal.
C. Determine the capacity of the Uppermost Sumps.
D. Determine the pumping frequency for leachate removal from the sumps (such that a maximum depth of 16 inches above the uppermost liner system in the area of the sumps is not exceeded. It is our understanding that the Oklahoma DEQ has approved a maximum depth in the sumps of 16 inches.

Each sump drainage area varies in size, depth, configuration and floor slope. The design criteria, therefore, varies and each sump area will be evaluated individually.

I. HELP model

Modeling of leachate generation within Landfill Cell 15 will consist of two steps: 1) Model calibration, and 2) Modeling of each sump area within the cell.

Climatic data was obtained on CD ROM from Hydrosphere Data Products for the time period between January 1980 and September 1994. Climate data between October 1994 and December 1995 was obtained from Oklahoma USGS Climatological Data. All precipitation and temperature data was based on the weather station located in Waynoka, Oklahoma. Solar Radiation and Evapotranspiration data were generated by the HELP model using the Tulsa, Oklahoma area since the latitude at Tulsa is similar to that at the Lone Mountain Facility.

A. Model Calibration

Leachate quantities pumped from the leachate collection and removal systems for Landfill Cells 12 and 13 were obtained from facility records. Surveyed waste elevations during operation of Landfill Cells 12 and 13 were obtained from Jividen Land Surveyors.

Since the waste elevation within the cells is very dynamic and the waste configuration at the surface of the waste is frequently changing, we reviewed the elevation data we received in order to determine a condition when the waste elevations remained somewhat constant for as long a period of time as possible. The more dynamic the filling of the cell and moving of the waste within the cell is, the more difficult it is to calibrate the model. This is because leachate generation is impacted greatly by the waste thickness.

After reviewing waste elevation data, we observed that the waste level within Landfill Cell 13 remained somewhat constant during the year of 1993. Although there was some variation in the waste surface during 1993, the HELP model calibration will be based on the conditions of and generation of leachate within Landfill Cell 13 for 1993. The average waste thickness at that time is about 22 feet or 264 inches.

The model is very sensitive to waste material thickness and saturated hydraulic conductivity as well as the thickness of the soil evaporation zone at the surface of the waste. These parameters dictate how much precipitation will not be lost to evaporation and will infiltrate into deeper zones and how rapidly the precipitin will enter the leachate collection system. The slower the leachate moves through the waste, the more the peaks will be reduced for leachate generation. From our understanding, another difficulty in modeling the existing cell conditions is that much of the precipitation runs to the interior edges of the cell and seeps much more rapidly through the more permeable sand layer (acting as a soil cover to the HDPE liner systems) than it would seep through the waste material.

Other parameters which may affect the results of model calibration are:

- Differences in precipitation events between the Waynoka station and the facility. There may be large differences in daily and monthly precipitation, however the over total annual precipitation at the station should approximate the annual precipitation at the facility.
- Types of waste material placed in various areas of the cell will impact how rapidly leachate will move through the waste in various areas of the cell.
- There may be numerous other factors affecting the accuracy of the model.

Table I below provides a comparison of the results obtained from the calibrated model and the actual quantities pumped from the sumps in the cell. Although there are differences with monthly data, the annual total between the model and measured quantities is very close. Since the model generates higher values than measured quantities, using the data generated from the calibration effort should provide some additional safety factor to the system.

TABLE I

Month	Measured Leachate Pumped During 1993 (inches)	HELP Model Generated Leachate Quantities (inches)
January	0.87	2.47
February	0.73	0.86
March	0.66	0.37
April	0.59	0.18
May	1.26	1.21
June	1.36	3.72
July	1.48	1.59
August	1.83	0.35
September	1.18	0.21
October	0.56	0.13
November	0.35	0.19
December	0.24	0.14
TOTALS	11.11	11.42

Actual leachate generation within the cell has a more reduced peak. Using the data generated by the model may provide a safety factor of about 2 for of peak values and should be a relatively close approximation for annual totals. The data presented in Table 1 represents depth in inches over the areas of the cell.

B. HELP Modeling of Landfill Cell 15

As stated above, each sump area is unique in its physical characteristics and should, therefore, be modeled individually. Each sump area will be modeled assuming the waste level is near empty, approximately half full (based on elevation and not capacity), level full (even with the top of the cell embankments, elev. 1420) and completely full (just prior to closure).

Weighted averages were used for slope and saturated hydraulic conductivity in order to provide the model with more realistic physical data. The waste thickness was also based on a weighted average to account for the length of the slopes and the decreasing thickness going up the interior slopes of the cell and for the decreasing thickness going toward the edges of the closure cap.

Areas consisting of the various slopes within each drainage area are included on sheet 4 of these calculations. Calculated weighted averages used within the HELP model are calculated and presented on sheets 5 through 12 of these calculations.

Data generated from the HELP modeling of each sump area, at varying waste thicknesses, is very extensive and has not been included with these calculations but is compiled separately. It consists of over 1200 pages of computer printout. The main results from the help model that were used for these calculations are summarized in numerical and graphical form on sheets 13 through 28 of these calculations.

The graphical presentation provides peak day leachate volumes for varying waste elevations. Numerical data presents peak day as well as peak month and daily averages based on peak month data.

Calculations to determine the capacity of the uppermost sumps were performed separately and are only summarized and presented in these calculations in numerical and graphical form showing stage vs. capacity relationships. The sump capacities at depths of 16 inches is presented on the graphs. The sump capacities area presented on sheets 35 through 38 of these calculations.

Sheet 40 of these calculations presents pumping frequencies that would be required for peak day and average day (based on peak month leachate quantities) for the various uppermost sumps within the cell. Pumping frequencies are calculated by dividing the estimated quantity of leachate generated by the sump capacity.

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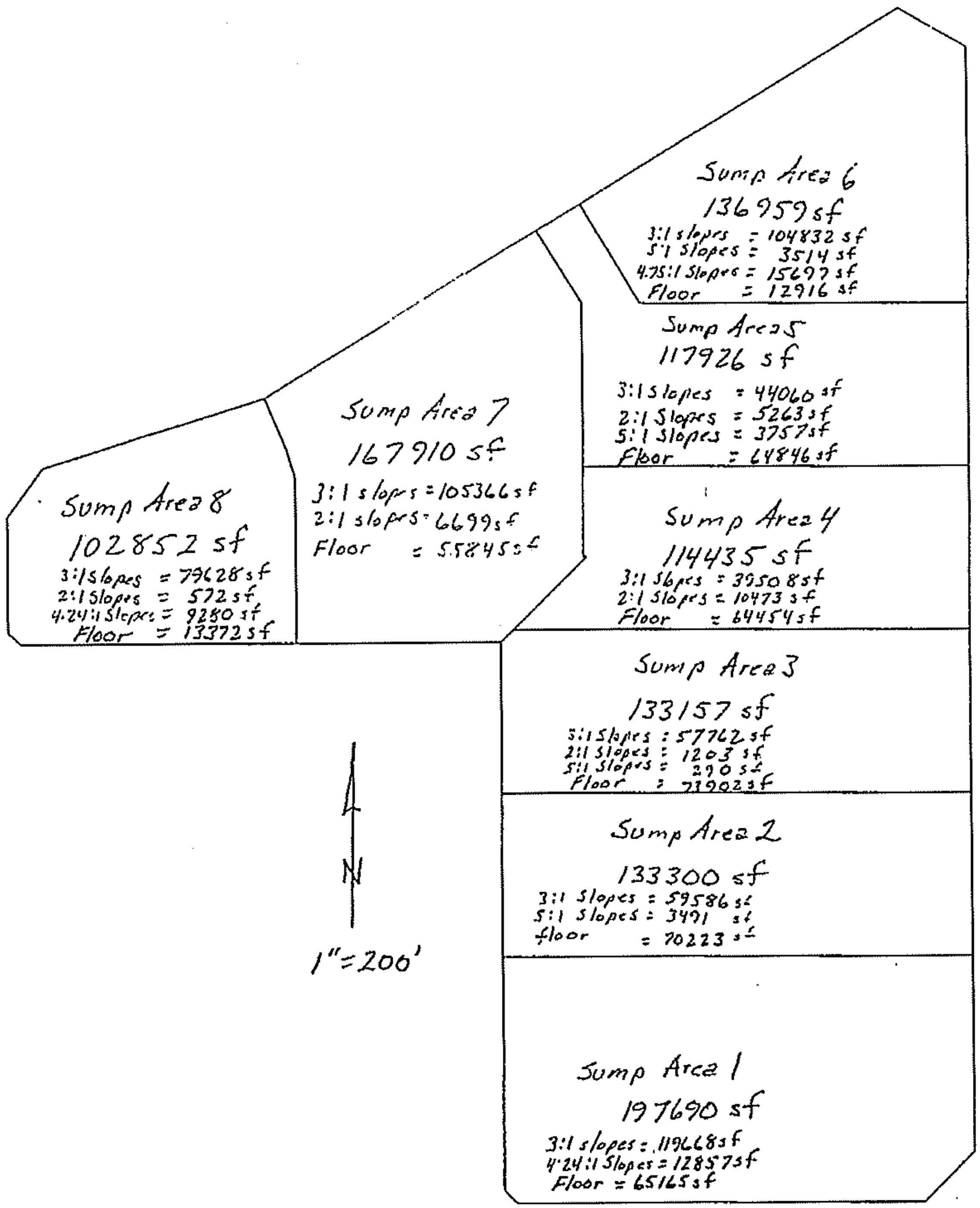
CLIENT:	USPCI - Lone Mountain Facility	SHEET 4	OF 40
PROJECT:	RCRA Cell 15	COMPUTED:	KCS
FEATURE:	Uppermost Leachate System	CHECKED:	
PROJECT NO.:	64.44.700	DATE:	August, 1996



HA&L ENGINEERING

CLIENT:	USPCI - Lone Mountain Facility	SHEET 5	OF 40
PROJECT:	RCRA Cell 15	COMPUTED:	KCS
FEATURE:	Uppermost Leachate System	CHECKED:	
PROJECT NO.:	64.44.700	DATE:	August, 1996

Bare portion of sideslopes =	95	percent (near empty)
Bare Sideslope Hyd. Cond. =	95	cm/sec
Soil Cover Hyd. Cond. =	0.001	cm/sec

Sump No. 1 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	119668	60.53	33	3949044	soil cover	90.25005/[illegible] ×	0.2 × 60/144 =	2.9610	354332.12
~4.24:1 Slopes	12857	6.50	24	308568	soil cover	90.25005	0.2	2.9610	38069.06
Floor (1.44%)	65165	32.96	1.44	93837.6	Geonet @ 6500 psf			3.2000	208528.00
Totals	197690			4351449.6					600929.18
Weighted Averages			22.01			92.65		3.0398	

Sump No. 2 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	59586	44.70	33	1966338	soil cover	90.25005	0.2	2.9610	176431.74
5:1 Slopes	3491	2.62	50	174550	soil cover	90.25005	0.2	2.9610	10336.71
Floor (1.44%)	70223	52.68	1.44	101121.12	Geonet @ 6500 psf			3.2000	224713.60
Totals	133300			2242009.1					411482.05
Weighted Averages			11.34			[illegible]		2.0815	

Sump No. 3 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	57762	43.38	33	1906146	soil cover	90.25005	0.2	2.9610	171030.95
2:1 Slopes	1203	0.90	50	60150	soil cover	90.25005	0.2	2.9610	3562.03
5:1 Slopes	290	0.22	20	5800	soil cover	90.25005	0.2	2.9610	858.68
Floor (2.26%)	73902	55.50	2.26	167018.52	Geonet @ 6500 psf			2.7900	206186.58
Totals	133157			2139114.5					381638.24
Weighted Averages			10.82			54.83		1.9305	

Sump No. 4 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	39508	34.52	33	1303764	soil cover	90.25005	0.2	2.9610	116981.59
2:1 Slopes	10473	9.15	50	523650	soil cover	90.25005	0.2	2.9610	31010.13
Floor (2.26%)	64454	56.32	2.26	145666.04	Geonet @ 6500 psf			2.7900	179826.66
Totals	114435			1973080					327818.38
Weighted Averages			9.98			50.54		1.6582	

HA&L ENGINEERING	*CLIENT:* USPCI - Lone Mountain Facility		*SHEET* 6 *OF* 40
	PROJECT: RCRA Cell 15		*COMPUTED:* KCS
	FEATURE: Uppermost Leachate System		*CHECKED:*
	PROJECT NO.: 64.44.700		*DATE:* August, 1996

Sump No. 5 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	44060	37.36	33	1453980	soil cover	90.25005	0.2	2.9610	130459.88
2:1 Slopes	5263	4.46	50	263150	soil cover	90.25005	0.2	2.9610	15583.53
5:1 Slopes	3757	3.19	20	75140	soil cover	90.25005	0.2	2.9610	11124.33
Floor (2.87%)	32423	27.49	2.87	93054.01	Geonet @ 6500 psf			2.5400	82354.42
Floor (1.44%)	32423	27.49	1.44	46689.12	Geonet @ 6500 psf			3.2000	103753.60
Totals	117926			1885324					239522.16
Weighted Averages			9.54			36.93		1.2116	

Sump No. 6 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	104832	76.54	33	3459456	soil cover	90.25005	0.2	2.9610	310403.32
5:1 Slopes	3514	2.57	20	70280	soil cover	90.25005	0.2	2.9610	10404.81
4.75:1 Slopes	15697	11.46	21	329637	soil cover	90.25005	0.2	2.9610	46478.18
Floor (1.06%)	12916	9.43	1.06	13690.96	Geonet @ 6500 psf			3.4700	44818.52
Totals	136959			3873064					412104.84
Weighted Averages			19.59			63.54		2.0846	

Sump No. 7 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	105366	62.75	33	3477078	soil cover	90.25005	0.2	2.9610	311984.47
2:1 Slopes	6699	3.99	50	334950	soil cover	90.25005	0.2	2.9610	19835.47
Floor (−1.06%)	55845	33.26	1.06	59195.7	Geonet @ 6500 psf			3.4700	193782.15
Totals	167910			3871223.7					525602.09
Weighted Averages			19.58			81.04		2.6587	

Sump No. 8 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	79628	77.42	33	2627724	soil cover	90.25005	0.2	2.9610	235775.29
2:1 Slopes	572	0.56	50	28600	soil cover	90.25005	0.2	2.9610	1693.67
4.24:1 Slopes	9280	9.02	24	222720	soil cover	90.25005	0.2	2.9610	27477.71
Floor (1.06%)	13372	13.00	1.06	14174.32	Geonet @ 6500 psf			3.4700	46400.84
Totals	102852			2893218.3					311347.51
Weighted Averages			14.64			48.00		1.5749	

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 7 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

Bare portion of sideslopes = 50 percent (1/3 full)
Bare Sideslope Hyd. Cond. = 95 cm/sec
Soil Cover Hyd. Cond. = 0.001 cm/sec

Sump No. 1 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	119668	60.53	33	3949044	soil cover	47.5005	0.2	1.5584	186492.45
~4.24:1 Slopes	12857	6.50	24	308568	soil cover	47.5005	0.2	1.5584	20036.55
Floor (1.44%)	65165	32.96	1.44	93837.6	Geonet @ 6500 psf			3.2000	208528.00
									415056.99
Totals	197690			4351449.6					
Weighted Averages			22.01			63.99		2.0995	

Sump No. 2 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	59586	44.70	33	1966338	soil cover	47.5005	0.2	1.5584	92859.74
5:1 Slopes	3491	2.62	50	174550	soil cover	47.5005	0.2	1.5584	5440.43
Floor (1.44%)	70223	52.68	1.44	101121.12	Geonet @ 6500 psf			3.2000	224713.60
									323013.77
Totals	133300			2242009.1					
Weighted Averages			11.34			49.80		1.6339	

Sump No. 3 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	57762	43.38	33	1906146	soil cover	47.5005	0.2	1.5584	90017.19
2:1 Slopes	1203	0.90	50	60150	soil cover	47.5005	0.2	1.5584	1874.77
5:1 Slopes	290	0.22	20	5800	soil cover	47.5005	0.2	1.5584	451.94
Floor (2.26%)	73902	55.50	2.26	167018.52	Geonet @ 6500 psf			2.7900	206186.58
									298530.48
Totals	133157			2139114.5					
Weighted Averages			10.82			46.03		1.5101	

Sump No. 4 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	39508	34.52	33	1303764	soil cover	47.5005	0.2	1.5584	61569.87
2:1 Slopes	10473	9.15	50	523650	soil cover	47.5005	0.2	1.5584	16321.28
Floor (2.26%)	64454	56.32	2.26	145666.04	Geonet @ 6500 psf			2.7900	179826.66
									257717.82
Totals	114435			1973080					
Weighted Averages			9.98			39.73		1.3036	

Sump No. 5 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	44060	37.36	33	1453980	soil cover	47.5005	0.2	1.5584	68663.78
2:1 Slopes	5263	4.46	50	263150	soil cover	47.5005	0.2	1.5584	8201.94
5:1 Slopes	3757	3.19	20	75140	soil cover	47.5005	0.2	1.5584	5854.97
Floor (2.87%)	32423	27.49	2.87	93054.01	Geonet @ 6500 psf			2.5400	82354.42
Floor (1.44%)	32423	27.49	1.44	46689.12	Geonet @ 6500 psf			3.2000	103753.60
									165075.11
Totals	117926			1885324		25.45		0.8350	
Weighted Averages			9.54						

Sump No. 6 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	104832	76.54	33	3459456	soil cover	47.5005	0.2	1.5584	163371.80
5:1 Slopes	3514	2.57	20	70280	soil cover	47.5005	0.2	1.5584	5476.27
4.75:1 Slopes	15697	11.46	21	329637	soil cover	47.5005	0.2	1.5584	24462.45
Floor (1.06%)	12916	9.43	1.06	13690.96	Geonet @ 6500 psf			3.4700	44818.52
									238129.04
Totals	136959			3873064		36.72		1.2046	
Weighted Averages			19.59						

Sump No. 7 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	105366	62.75	33	3477078	soil cover	47.5005	0.2	1.5584	164203.99
2:1 Slopes	6699	3.99	50	334950	soil cover	47.5005	0.2	1.5584	10439.82
Floor (~1.06%)	55845	33.26	1.06	59195.7	Geonet @ 6500 psf			3.4700	193782.15
									368425.97
Totals	167910			3871223.7		56.81		1.8637	
Weighted Averages			19.58						

Sump No. 8 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	79628	77.42	33	2627724	soil cover	47.5005	0.2	1.5584	124093.50
2:1 Slopes	572	0.56	50	28600	soil cover	47.5005	0.2	1.5584	891.41
4.24:1 Slopes	9280	9.02	24	222720	soil cover	47.5005	0.2	1.5584	14462.09
Floor (1.06%)	13372	13.00	1.06	14174.32	Geonet @ 6500 psf			3.4700	46400.84
									185847.85
Totals	102852			2893218.3		28.65		0.9401	
Weighted Averages			14.64						

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 9 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

Bare portion of sideslopes = 0 percent (full)
Bare Sideslope Hyd. Cond. = 91.44 cm/sec
Soil Cover Hyd. Cond. = 0.001 cm/sec

Sump No. 1 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	119668	60.53	33	3949044	soil cover	0.001	12	3.2808E-05	3.9261
~4.24:1 Slopes	12857	6.50	24	308568	soil cover	0.001	12	3.2808E-05	0.4218
Floor (1.44%)	65165	32.96	1.44	93837.6	Geonet @ 6500 psf			3.2000E+00	208528.0000
									208532.3479
Totals	197690			4351449.6					
Weighted Averages			22.01			32.15		1.0548E+00	

Sump No. 2 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	59586	44.70	33	1966338	soil cover	0.001	12	3.2808E-05	1.9549
5:1 Slopes	3491	2.62	50	174550	soil cover	0.001	12	3.2808E-05	0.1145
Floor (1.44%)	70223	52.68	1.44	101121.12	Geonet @ 6500 psf			3.2000E+00	224713.6000
									224715.6695
Totals	133300			2242009.1					
Weighted Averages			11.34			34.25		1.1367E+00	

Sump No. 3 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	57762	43.38	33	1906146	soil cover	0.001	12	3.2808E-05	1.8951
2:1 Slopes	1203	0.90	50	60150	soil cover	0.001	12	3.2808E-05	0.0395
5:1 Slopes	290	0.22	20	5800	soil cover	0.001	12	3.2808E-05	0.0095
Floor (2.26%)	73902	55.50	2.26	167018.52	Geonet @ 6500 psf			2.7900E+00	206186.5800
									206188.5241
Totals	133157			2139114.5					
Weighted Averages			10.82			31.79		1.0430E+00	

Sump No. 4 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	39508	34.52	33	1303764	soil cover	0.001	12	3.2808E-05	1.2962
2:1 Slopes	10473	9.15	50	523650	soil cover	0.001	12	3.2808E-05	0.3436
Floor (2.26%)	64454	56.32	2.26	145666.04	Geonet @ 6500 psf			2.7900E+00	179826.6600
									179828.2998
Totals	114435			1973080					
Weighted Averages			9.98			27.73		9.0965E-01	

HA&L ENGINEERING		
CLIENT: USPCI - Lone Mountain Facility		*SHEET* 10 *OF* 40
PROJECT: RCRA Cell 15		*COMPUTED:* KCS
FEATURE: Uppermost Leachate System		*CHECKED:*
PROJECT NO.: 64.44.700		*DATE:* August, 1996

Sump No. 5 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	44060	37.36	33	1453980	soil cover	0.001	12	3.2808E-05	1.4455
2:1 Slopes	5263	4.46	50	263150	soil cover	0.001	12	3.2808E-05	0.1727
5:1 Slopes	3757	3.19	20	75140	soil cover	0.001	12	3.2808E-05	0.1233
Floor (2.87%)	32423	27.49	2.87	93054.01	Geonet @ 6500 psf			2.5400E+00	82354.4200
Floor (1.44%)	32423	27.49	1.44	46689.12	Geonet @ 6500 psf			3.2000E+00	103753.6000
									82356.1615
Totals	117926			1885324					
Weighted Averages			9.54			12.70		4.1659E-01	

Sump No. 6 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	104832	76.54	33	3459456	soil cover	0.001	12	3.2808E-05	3.4394
5:1 Slopes	3514	2.57	20	70280	soil cover	0.001	12	3.2808E-05	0.1153
4.75:1 Slopes	15697	11.46	21	329637	soil cover	0.001	12	3.2808E-05	0.5150
Floor (1.06%)	12916	9.43	1.06	13690.96	Geonet @ 6500 psf			3.4700E+00	44818.5200
									44822.5897
Totals	136959			3873064					
Weighted Averages			19.59			6.91		2.2673E-01	

Sump No. 7 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	105366	62.75	33	3477078	soil cover	0.001	12	3.2808E-05	3.4569
2:1 Slopes	6699	3.99	50	334950	soil cover	0.001	12	3.2808E-05	0.2198
Floor (~1.06%)	55845	33.26	1.06	59195.7	Geonet @ 6500 psf			3.4700E+00	193782.1500
									193785.8267
Totals	167910			3871223.7					
Weighted Averages			19.58			29.88		9.8025E-01	

Sump No. 8 Characteristics

Description	Area (A, sf)	Percent of Area	Slope (S, %)	A*S	Drainage Medium	Drainage Hydraulic Conductivity (HC, cm/sec)	Drainage Layer Thickness (t, in)	Drainage Transmissivity (T, ft^2/min)	A*HC
3:1 Slopes	79628	77.42	33	2627724	soil cover	0.001	12	3.2808E-05	2.6125
2:1 Slopes	572	0.56	50	28600	soil cover	0.001	12	3.2808E-05	0.0188
4.24:1 Slopes	9280	9.02	24	222720	soil cover	0.001	12	3.2808E-05	0.3045
Floor (1.06%)	13372	13.00	1.06	14174.32	Geonet @ 6500 psf			3.4700E+00	46400.8400
									46403.7757
Totals	102852			2893218.3					
Weighted Averages			14.64			7.15		2.3473E-01	

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 11 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

SUMP NO. 1

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1374.84 feet
Average Embankment Height = 45.16 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	65165	2.00	132525	1.33	197690	1.55	19
Near 1/3 Full	65165	22.58	132525	15.05	197690	17.53	210
Level Full Full	65165	45.16	132525	30.10	197690	35.07	421 +120 = 541

SUMP NO. 2

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1373.90 feet
Average Embankment Height = 46.10 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	70223	2.00	63077	1.33	133300	1.68	20
Near 1/3 Full	70223	23.05	63077	15.37	133300	19.42	233
Level Full Full	70223	46.10	63077	30.74	133300	38.83	466 +120 = 586

SUMP NO. 3

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1374.61 feet
Average Embankment Height = 45.39 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	73902	2.00	59255	1.33	133157	1.70	20
Near 1/3 Full	73902	22.69	59255	15.13	133157	19.33	232
Level Full Full	73902	45.39	59255	30.26	133157	38.66	464 +132 = 596

SUMP NO. 4

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1371.43 feet
Average Embankment Height = 48.57 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	64454	2.00	49981	1.33	114435	1.71	21
Near 1/3 Full	64454	24.28	49981	16.19	114435	20.75	249
Level Full Full	64454	48.57	49981	32.38	114435	41.50	498 +147 = 645

HA&L ENGINEERING

CLIENT: USPCI - Lone Mountain Facility
PROJECT: RCRA Cell 15
FEATURE: Uppermost Leachate System
PROJECT NO.: 64.44.700

SHEET 12 OF 40
COMPUTED: KCS
CHECKED:
DATE: August, 1996

SUMP NO. 5

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1368.24 feet
Average Embankment Height = 51.76 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	64846	2.00	53080	1.33	117926	1.70	20
Near 1/3 Full	64846	25.88	53080	17.25	117926	22.00	264
Level Full	64846	51.76	53080	34.51	117926	43.99	528
Full							+ 150 = 678

SUMP NO. 6

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1364.60 feet
Average Embankment Height = 55.40 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	12916	2.00	124043	1.33	136959	1.40	17
Near 1/3 Full	12916	27.70	124043	18.47	136959	19.34	232
Level Full	12916	55.40	124043	36.93	136959	38.67	464
Full							+ 170 = 574

SUMP NO. 7

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1374.20 feet
Average Embankment Height = 45.80 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	55845	2.00	112065	1.33	167910	1.56	19
Near 1/3 Full	55845	22.90	112065	15.27	167910	17.81	214
Level Full	55845	45.80	112065	30.53	167910	35.61	427
Full							+ 115 = 542

SUMP NO. 8

Top of Embankment Elevation = 1420 feet
Average Floor Elevation = 1380.36 feet
Average Embankment Height = 39.64 feet

Description	Floor Area (sf)	Average Height on Floor (ft)	Slope Area (sf)	Average Height on Slope (ft)	Total Area (sf)	Overall Average Height (ft)	Overall Average Height (in)
Near Empty	13372	2.00	89480	1.33	102852	1.42	17
Near 1/3 Full	13372	19.82	89480	13.21	102852	14.07	169
Level Full	13372	39.64	89480	26.43	102852	28.14	338
Full							+ 94 = 432

HA&L ENGINEERING

CLIENT: USPCI - Lone Mountain Facility
PROJECT: RCRA Cell 15
FEATURE: Uppermost Leachate System
PROJECT NO.: 64.44.700

SHEET 13 OF 40
COMPUTED: KCS
CHECKED:
DATE: August, 1996

Sump 1 Empty
Peak Day = 0.82 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.44	0.048	5.47
1981	3.89	0.130	13.31
1982	1.67	0.056	4.35
1983	1.34	0.045	6.97
1984	0.87	0.029	2.77
1985	2.74	0.091	9.70
1986	2.27	0.076	7.70
1987	2.31	0.077	7.30
1988	1.17	0.039	2.89
1989	1.70	0.057	5.92
1990	1.88	0.063	3.97
1991	1.32	0.044	5.49
1992	2.68	0.089	6.36
1993	5.76	0.192	8.75
1994	0.82	0.027	3.08
1995	2.57	0.086	6.01

Sump 1 Half Full
Peak Day = 0.22 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.47
1981	3.03	0.101	13.20
1982	1.78	0.059	4.65
1983	1.47	0.049	6.89
1984	0.53	0.018	2.11
1985	2.43	0.081	10.33
1986	2.39	0.080	7.69
1987	1.60	0.053	7.43
1988	1.07	0.036	2.90
1989	1.72	0.057	5.96
1990	1.03	0.034	4.02
1991	1.94	0.065	5.33
1992	0.96	0.032	4.57
1993	3.64	0.121	10.71
1994	0.84	0.028	2.86
1995	2.27	0.076	6.24

Sump 1 Level Full
Peak Day = 0.16 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.57	0.052	5.48
1981	2.66	0.089	12.93
1982	1.77	0.059	4.90
1983	1.39	0.046	6.90
1984	0.54	0.018	2.14
1985	2.66	0.089	10.24
1986	2.19	0.073	7.69
1987	2.04	0.068	7.45
1988	1.20	0.040	2.90
1989	1.78	0.059	5.97
1990	1.42	0.047	4.00
1991	1.75	0.058	5.29
1992	0.98	0.033	3.98
1993	3.44	0.115	11.31
1994	0.81	0.027	2.85
1995	2.42	0.081	6.28

Sump 1 Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.57	0.052	5.48
1981	2.56	0.085	12.83
1982	1.88	0.063	5.00
1983	1.38	0.046	6.90
1984	0.54	0.018	2.14
1985	2.58	0.086	10.20
1986	2.12	0.071	7.70
1987	2.10	0.070	7.48
1988	1.19	0.040	2.91
1989	1.81	0.060	5.97
1990	1.48	0.049	3.40
1991	1.69	0.056	5.29
1992	0.98	0.033	3.95
1993	2.93	0.098	11.34
1994	0.81	0.027	2.84
1995	2.37	0.079	6.28

Sump 2 Empty
Peak Day = 0.83 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.44	0.048	5.47
1981	3.89	0.130	13.31
1982	1.67	0.056	4.35
1983	1.35	0.045	6.98
1984	0.86	0.029	2.77
1985	2.76	0.092	9.72
1986	2.27	0.076	7.70
1987	2.35	0.078	7.29
1988	1.17	0.039	2.90
1989	1.72	0.057	5.97
1990	1.94	0.065	4.03
1991	1.31	0.044	5.50
1992	2.69	0.090	6.36
1993	5.76	0.192	8.77
1994	0.83	0.028	3.08
1995	2.57	0.086	6.02

Sump 2 Half Full
Peak Day = 0.21 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.48
1981	2.98	0.099	13.13
1982	1.65	0.055	4.69
1983	1.45	0.048	6.91
1984	0.54	0.018	2.12
1985	2.37	0.079	10.32
1986	2.39	0.080	7.69
1987	1.70	0.057	7.45
1988	1.11	0.037	2.90
1989	1.71	0.057	5.93
1990	1.12	0.037	3.96
1991	1.91	0.064	5.34
1992	0.96	0.032	4.46
1993	3.72	0.124	10.85
1994	0.84	0.028	2.87
1995	2.34	0.078	6.25

Sump 2 Level Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.48
1981	2.55	0.085	12.85
1982	1.81	0.060	4.96
1983	1.39	0.046	6.92
1984	0.54	0.018	2.15
1985	2.64	0.088	10.22
1986	2.16	0.072	7.69
1987	2.05	0.068	7.57
1988	1.20	0.040	2.93
1989	1.78	0.059	5.96
1990	1.47	0.049	3.93
1991	1.73	0.058	5.30
1992	0.98	0.033	3.96
1993	3.22	0.107	11.34
1994	0.81	0.027	2.85
1995	2.42	0.081	6.31

Sump 2 Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.48
1981	2.59	0.086	12.75
1982	1.91	0.064	5.06
1983	1.38	0.046	6.92
1984	0.54	0.018	2.15
1985	2.53	0.084	10.19
1986	2.09	0.070	7.70
1987	2.09	0.070	7.59
1988	1.19	0.040	2.93
1989	1.80	0.060	5.96
1990	1.52	0.051	3.93
1991	1.68	0.056	5.30
1992	0.99	0.033	3.95
1993	2.79	0.093	11.35
1994	0.81	0.027	2.84
1995	2.36	0.079	6.31

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility	SHEET 15 OF 40
	PROJECT: RCRA Cell 15	COMPUTED: KCS
	FEATURE: Uppermost Leachate System	CHECKED:
	PROJECT NO.: 64.44.700	DATE: August, 1996

Sump 3 Empty
Peak Day = 0.82 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.44	0.048	5.47
1981	3.87	0.129	13.27
1982	1.67	0.056	4.36
1983	1.34	0.045	6.98
1984	0.86	0.029	2.78
1985	2.76	0.092	9.70
1986	2.27	0.076	7.70
1987	2.35	0.078	7.28
1988	1.17	0.039	2.90
1989	1.72	0.057	5.96
1990	1.88	0.063	3.95
1991	1.30	0.043	5.46
1992	2.69	0.090	6.36
1993	5.76	0.192	8.76
1994	0.83	0.028	3.09
1995	2.57	0.086	6.03

Sump 3 Half Full
Peak Day = 0.21 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.48
1981	2.99	0.100	13.18
1982	1.65	0.055	4.69
1983	1.45	0.048	6.91
1984	0.54	0.018	2.12
1985	2.37	0.079	10.32
1986	2.38	0.079	7.69
1987	1.67	0.056	7.44
1988	1.10	0.037	2.90
1989	1.72	0.057	5.97
1990	1.12	0.037	4.03
1991	1.91	0.064	5.36
1992	0.96	0.032	4.45
1993	3.72	0.124	10.84
1994	0.84	0.028	2.86
1995	2.33	0.078	6.26

Sump 3 Level Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.48
1981	2.56	0.085	12.90
1982	1.81	0.060	4.96
1983	1.39	0.046	6.92
1984	0.54	0.018	2.15
1985	2.64	0.088	10.21
1986	2.16	0.072	7.70
1987	2.05	0.068	7.56
1988	1.20	0.040	2.92
1989	1.80	0.060	5.97
1990	1.48	0.049	3.96
1991	1.74	0.058	5.35
1992	0.99	0.033	3.97
1993	3.23	0.108	11.34
1994	0.81	0.027	2.86
1995	2.42	0.081	6.29

Sump 3 Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.48
1981	2.59	0.086	12.79
1982	1.91	0.064	5.07
1983	1.38	0.046	6.92
1984	0.54	0.018	2.15
1985	2.52	0.084	10.18
1986	2.09	0.070	7.70
1987	2.10	0.070	7.59
1988	1.19	0.040	2.92
1989	1.83	0.061	5.97
1990	1.54	0.051	3.96
1991	1.68	0.056	5.35
1992	0.99	0.033	3.96
1993	2.76	0.092	11.35
1994	0.81	0.027	2.86
1995	2.35	0.078	6.29

Sump 4 Empty
Peak Day = 0.82 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.44	0.048	5.48
1981	3.88	0.129	13.27
1982	1.67	0.056	4.36
1983	1.34	0.045	6.98
1984	0.86	0.029	2.78
1985	2.76	0.092	9.70
1986	2.27	0.076	7.70
1987	2.35	0.078	7.29
1988	1.18	0.039	2.90
1989	1.72	0.057	5.96
1990	1.90	0.063	3.95
1991	1.30	0.043	5.46
1992	2.68	0.089	6.36
1993	5.76	0.192	8.77
1994	0.83	0.028	3.07
1995	2.57	0.086	6.03

Sump 4 Half Full
Peak Day = 0.20 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.48
1981	2.92	0.097	13.16
1982	1.61	0.054	4.71
1983	1.44	0.048	6.91
1984	0.54	0.018	2.13
1985	2.44	0.081	10.31
1986	2.35	0.078	7.69
1987	1.75	0.058	7.45
1988	1.13	0.038	2.91
1989	1.72	0.057	5.97
1990	1.19	0.040	3.99
1991	1.87	0.062	5.31
1992	0.97	0.032	4.37
1993	3.76	0.125	10.93
1994	0.84	0.028	2.86
1995	2.37	0.079	6.26

Sump 4 Level Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.49
1981	2.57	0.086	12.86
1982	1.84	0.061	5.00
1983	1.39	0.046	6.92
1984	0.54	0.018	2.15
1985	2.61	0.087	10.21
1986	2.14	0.071	7.69
1987	2.07	0.069	7.51
1988	1.20	0.040	2.92
1989	1.80	0.060	5.97
1990	1.48	0.049	3.95
1991	1.73	0.058	5.35
1992	0.99	0.033	3.95
1993	3.09	0.103	11.35
1994	0.81	0.027	2.85
1995	2.40	0.080	6.29

Sump 4 Full
Peak Day = 0.14 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.49
1981	2.60	0.087	12.75
1982	1.94	0.065	5.11
1983	1.37	0.046	6.92
1984	0.54	0.018	2.16
1985	2.47	0.082	10.17
1986	2.08	0.069	7.69
1987	2.11	0.070	7.53
1988	1.19	0.040	2.92
1989	1.82	0.061	5.97
1990	1.54	0.051	3.95
1991	1.67	0.056	5.35
1992	0.99	0.033	3.95
1993	2.64	0.088	11.35
1994	0.80	0.027	2.85
1995	2.34	0.078	6.30

Sump 5 Empty
Peak Day = 0.81 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.44	0.048	5.47
1981	3.89	0.130	13.28
1982	1.68	0.056	4.36
1983	1.35	0.045	7.00
1984	0.86	0.029	2.78
1985	2.75	0.092	9.70
1986	2.27	0.076	7.69
1987	2.33	0.078	7.24
1988	1.18	0.039	2.91
1989	1.72	0.057	5.97
1990	1.90	0.063	3.97
1991	1.31	0.044	5.47
1992	2.68	0.089	6.38
1993	5.77	0.192	8.77
1994	0.83	0.028	3.08
1995	2.58	0.086	6.03

Sump 5 Half Full
Peak Day = 0.19 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.48
1981	2.94	0.098	13.15
1982	1.55	0.052	4.75
1983	1.45	0.048	6.92
1984	0.54	0.018	2.13
1985	2.50	0.083	10.32
1986	2.32	0.077	7.68
1987	1.81	0.060	7.50
1988	1.15	0.038	2.92
1989	1.74	0.058	5.99
1990	1.25	0.042	3.96
1991	1.89	0.063	5.37
1992	0.97	0.032	4.31
1993	3.77	0.126	11.00
1994	0.83	0.028	2.87
1995	2.42	0.081	6.31

Sump 5 Level Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.61	0.054	5.53
1981	2.58	0.086	12.86
1982	1.89	0.063	5.06
1983	1.40	0.047	6.96
1984	0.50	0.017	2.08
1985	2.58	0.086	10.23
1986	2.11	0.070	7.74
1987	2.08	0.069	7.58
1988	1.21	0.040	2.99
1989	1.79	0.060	6.02
1990	1.48	0.049	3.96
1991	1.73	0.058	5.32
1992	1.03	0.034	4.02
1993	3.01	0.100	11.47
1994	0.83	0.028	2.91
1995	2.40	0.080	6.38

Sump 5 Full
Peak Day = 0.14 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.62	0.054	5.53
1981	2.60	0.087	12.75
1982	1.97	0.066	5.17
1983	1.39	0.046	6.96
1984	0.50	0.017	2.09
1985	2.45	0.082	10.19
1986	2.05	0.068	7.74
1987	2.12	0.071	7.60
1988	1.21	0.040	2.99
1989	1.81	0.060	6.02
1990	1.54	0.051	3.96
1991	1.68	0.056	5.32
1992	1.04	0.035	4.02
1993	2.60	0.087	11.47
1994	0.82	0.027	2.91
1995	2.35	0.078	6.38

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 18 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

Sump 6 Empty
Peak Day = 0.97 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.46	0.049	5.47
1981	3.89	0.130	13.30
1982	1.67	0.056	4.35
1983	1.35	0.045	6.98
1984	0.88	0.029	2.79
1985	2.71	0.090	9.69
1986	2.28	0.076	7.70
1987	2.21	0.074	7.30
1988	1.18	0.039	2.89
1989	1.70	0.057	5.92
1990	1.88	0.063	3.97
1991	1.38	0.046	5.49
1992	2.65	0.088	6.37
1993	5.78	0.193	8.74
1994	0.81	0.027	3.08
1995	2.58	0.086	6.01

Sump 6 Half Full
Peak Day = 0.21 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.47
1981	2.99	0.100	13.16
1982	1.65	0.055	4.69
1983	1.45	0.048	6.89
1984	0.53	0.018	2.12
1985	2.38	0.079	10.32
1986	2.38	0.079	7.68
1987	1.71	0.057	7.45
1988	1.11	0.037	2.90
1989	1.72	0.057	5.93
1990	1.12	0.037	3.94
1991	1.92	0.064	5.33
1992	0.96	0.032	4.44
1993	3.72	0.124	10.84
1994	0.84	0.028	2.86
1995	2.33	0.078	6.24

Sump 6 Level Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.49
1981	2.56	0.085	12.92
1982	1.81	0.060	4.95
1983	1.40	0.047	6.92
1984	0.54	0.018	2.15
1985	2.64	0.088	10.22
1986	2.16	0.072	7.69
1987	2.05	0.068	7.58
1988	1.20	0.040	2.93
1989	1.80	0.060	5.97
1990	1.46	0.049	3.96
1991	1.74	0.058	5.35
1992	0.99	0.033	3.97
1993	3.23	0.108	11.34
1994	0.81	0.027	2.85
1995	2.42	0.081	6.31

Sump 6 Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.49
1981	2.58	0.086	12.83
1982	1.90	0.063	5.04
1983	1.39	0.046	6.92
1984	0.54	0.018	2.15
1985	2.54	0.085	10.18
1986	2.10	0.070	7.70
1987	2.09	0.070	7.60
1988	1.20	0.040	2.93
1989	1.82	0.061	5.97
1990	1.51	0.050	3.96
1991	1.70	0.057	5.35
1992	0.99	0.033	3.96
1993	2.82	0.094	11.35
1994	0.81	0.027	2.85
1995	2.37	0.079	6.31

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 19 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

Sump 7 Empty
Peak Day = 0.82 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.44	0.048	5.47
1981	3.89	0.130	13.31
1982	1.67	0.056	4.35
1983	1.34	0.045	6.97
1984	0.87	0.029	2.77
1985	2.75	0.092	9.70
1986	2.27	0.076	7.70
1987	2.31	0.077	7.30
1988	1.17	0.039	2.89
1989	1.70	0.057	5.92
1990	1.88	0.063	3.97
1991	1.33	0.044	5.49
1992	2.68	0.089	6.36
1993	5.76	0.192	8.75
1994	0.82	0.027	3.08
1995	2.57	0.086	6.01

Sump 7 Half Full
Peak Day = 0.22 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.59	0.053	5.47
1981	3.03	0.101	13.19
1982	1.75	0.058	4.66
1983	1.46	0.049	6.89
1984	0.53	0.018	2.11
1985	2.41	0.080	10.33
1986	2.39	0.080	7.69
1987	1.61	0.054	7.44
1988	1.08	0.036	2.90
1989	1.72	0.057	5.93
1990	1.04	0.035	3.94
1991	1.94	0.065	5.33
1992	0.96	0.032	4.55
1993	3.64	0.121	10.74
1994	0.84	0.028	2.86
1995	2.28	0.076	6.24

Sump 7 Level Full
Peak Day = 0.16 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.47
1981	2.64	0.088	12.90
1982	1.78	0.059	4.93
1983	1.39	0.046	6.89
1984	0.54	0.018	2.13
1985	2.66	0.089	10.23
1986	2.18	0.073	7.69
1987	2.05	0.068	7.52
1988	1.20	0.040	2.90
1989	1.79	0.060	5.97
1990	1.44	0.048	3.99
1991	1.75	0.058	5.28
1992	0.98	0.033	3.97
1993	3.40	0.113	11.31
1994	0.81	0.027	2.85
1995	2.42	0.081	6.27

Sump 7 Full
Peak Day = 0.15 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.47
1981	2.60	0.087	12.79
1982	1.88	0.063	5.04
1983	1.38	0.046	6.90
1984	0.54	0.018	2.14
1985	2.57	0.086	10.19
1986	2.12	0.071	7.70
1987	2.10	0.070	7.55
1988	1.19	0.040	2.90
1989	1.81	0.060	5.97
1990	1.51	0.050	3.99
1991	1.69	0.056	5.27
1992	0.98	0.033	3.95
1993	2.92	0.097	11.33
1994	0.81	0.027	2.85
1995	2.36	0.079	6.27

Sump 8 Empty
Peak Day = 0.96 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.46	0.049	5.47
1981	3.89	0.130	13.31
1982	1.67	0.056	4.35
1983	1.34	0.045	6.98
1984	0.88	0.029	2.79
1985	2.71	0.090	9.70
1986	2.27	0.076	7.70
1987	2.21	0.074	7.30
1988	1.18	0.039	2.90
1989	1.72	0.057	5.96
1990	1.97	0.066	4.02
1991	1.37	0.046	5.45
1992	2.65	0.088	6.37
1993	5.78	0.193	8.74
1994	0.81	0.027	3.09
1995	2.58	0.086	6.01

Sump 8 Half Full
Peak Day = 0.35 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.58	0.053	5.48
1981	2.86	0.095	13.28
1982	2.14	0.071	4.59
1983	1.49	0.050	6.90
1984	0.53	0.018	2.10
1985	2.77	0.092	10.36
1986	2.26	0.075	7.68
1987	1.74	0.058	7.41
1988	0.98	0.033	2.90
1989	1.71	0.057	5.97
1990	1.27	0.042	4.03
1991	2.00	0.067	5.36
1992	1.12	0.037	4.91
1993	3.20	0.107	10.38
1994	0.86	0.029	2.88
1995	2.06	0.069	6.23

Sump 8 Level Full
Peak Day = 0.17 inch

	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.60	0.053	5.51
1981	2.83	0.094	13.04
1982	1.65	0.055	4.87
1983	1.43	0.048	6.94
1984	0.51	0.017	2.07
1985	2.65	0.088	10.34
1986	2.23	0.074	7.73
1987	1.95	0.065	7.53
1988	1.21	0.040	2.98
1989	1.78	0.059	6.01
1990	1.34	0.045	3.96
1991	1.83	0.061	5.32
1992	1.01	0.034	4.15
1993	3.71	0.124	11.29
1994	0.84	0.028	2.89
1995	2.45	0.082	6.30

Sump 8 Full
Peak Day = 0.16 inch

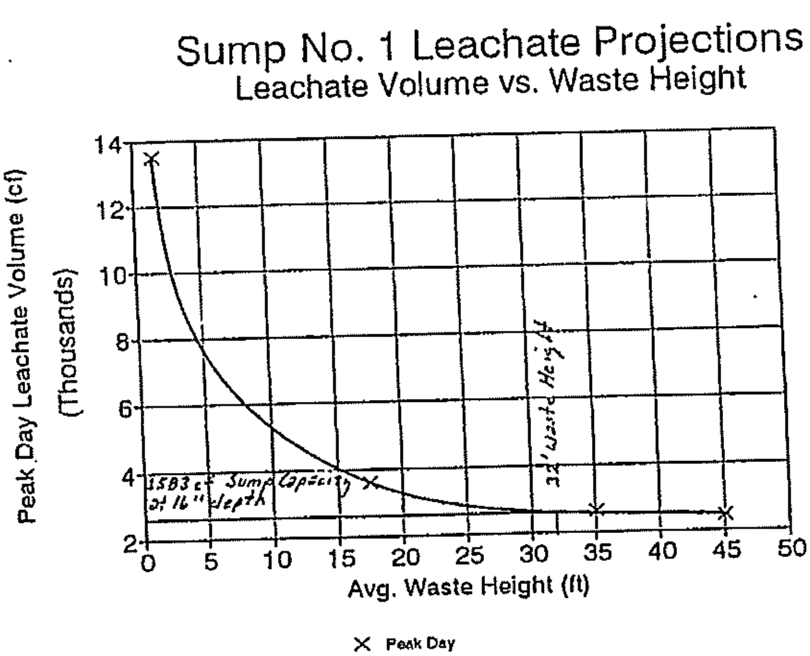
	Peak Month (inches)	Peak Day from Peak Month (inches)	Annual Total (inches)
1980	1.60	0.053	5.51
1981	2.65	0.088	12.93
1982	1.80	0.060	4.98
1983	1.42	0.047	6.94
1984	0.51	0.017	2.07
1985	2.65	0.088	10.30
1986	2.18	0.073	7.74
1987	2.04	0.068	7.56
1988	1.22	0.041	2.98
1989	1.81	0.060	6.01
1990	1.43	0.048	3.96
1991	1.76	0.059	5.32
1992	1.02	0.034	4.03
1993	3.41	0.114	11.40
1994	0.83	0.028	2.88
1995	2.44	0.081	6.30

HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 21 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

Sump No. 1 - Peak Daily & Peak Monthly Leachate Values

Area = 197690 sf

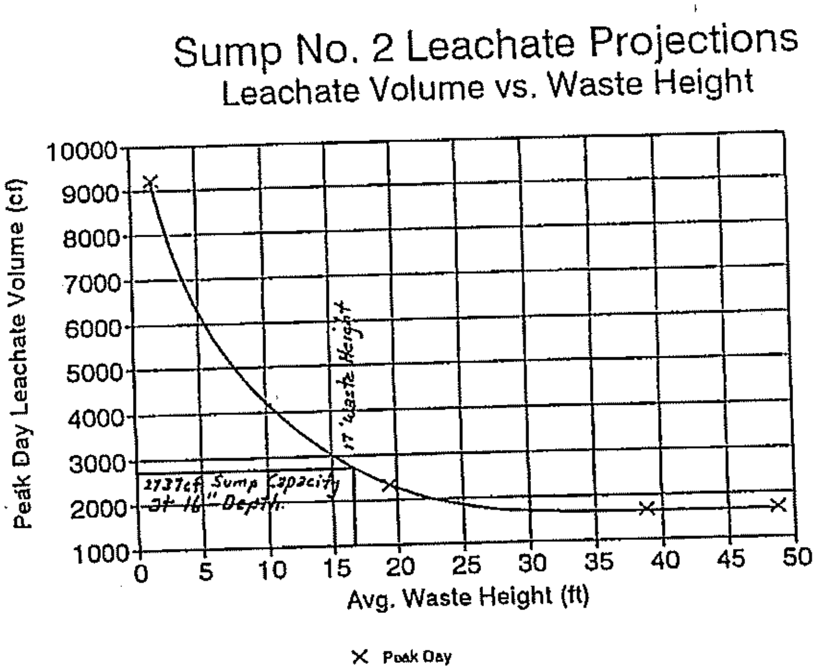
Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume		Month		
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)		(cf)	(gal)
19	1.58	0.82000	13509	101046	5.76000	94891	709786	30	3163	23660
210	17.50	0.22000	3624	27110	3.64000	59966	448545	30	1999	14952
421	35.08	0.16000	2636	19716	3.44000	56671	423900	30	1889	14130
541	45.08	0.15000	2471	18484	2.93000	48269	361054	30	1609	12035



Sump No. 2 - Peak Daily & Peak Monthly Leachate Values

Area = 133300 sf

Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume				
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)	Month	(cf)	(gal)
20	1.67	0.83000	9220	68965	5.76000	63984	478600	30	2133	15953
233	19.42	0.21000	2333	17449	3.72000	41323	309096	30	1377	10303
466	38.83	0.15000	1666	12464	3.22000	35769	267551	31	1154	8631
586	48.83	0.15000	1666	12464	2.79000	30992	231822	31	1000	7478

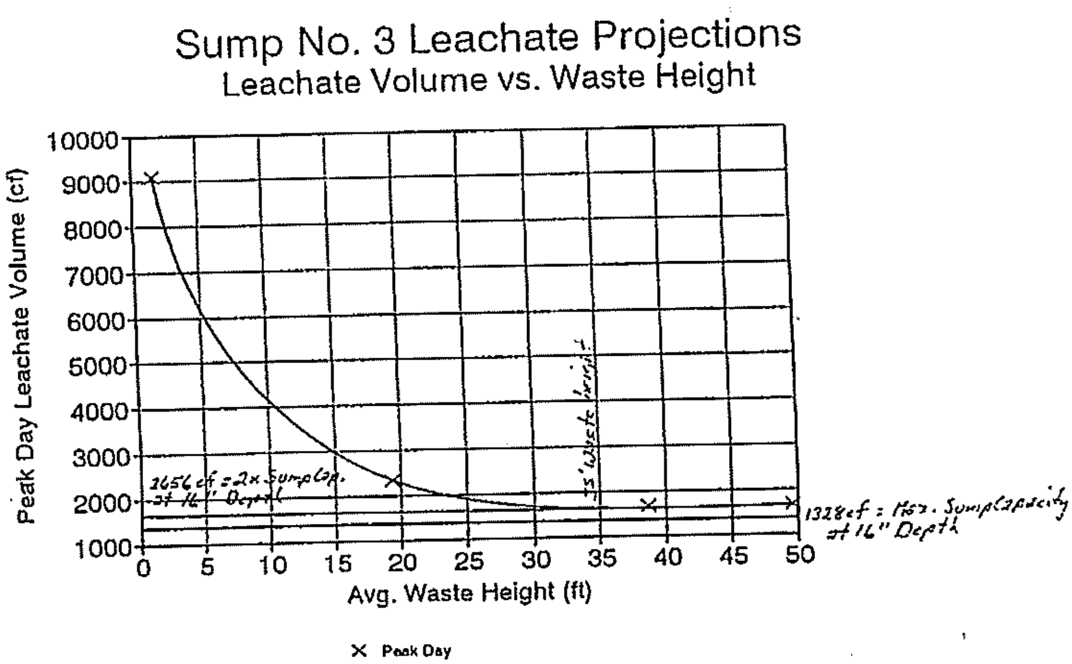


HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 23 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996

Sump No. 3 - Peak Daily & Peak Monthly Leachate Values

Area = 133157 sf

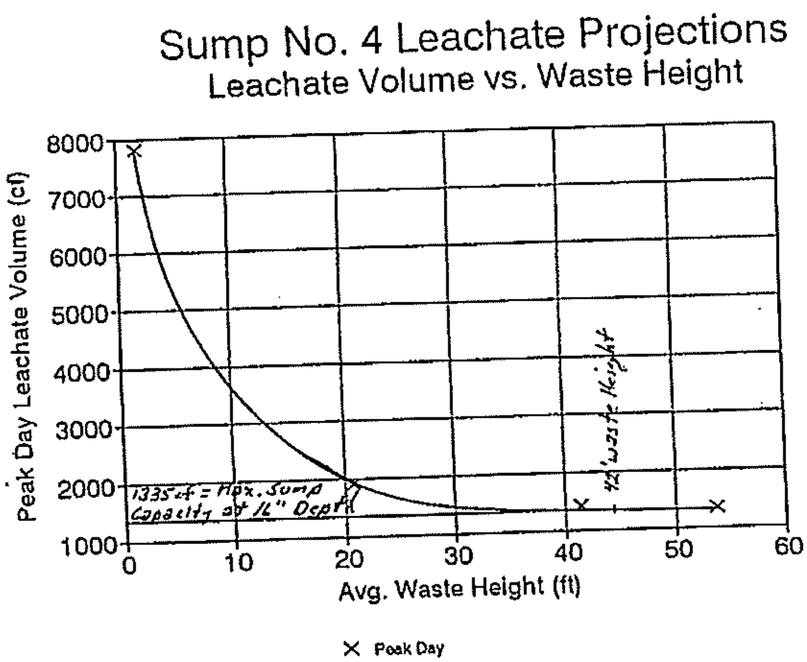
Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume				
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)	Month	(cf)	(gal)
20	1.67	0.82000	9099	68061	5.76000	63915	478087	30	2131	15936
232	19.33	0.21000	2330	17430	3.72000	41279	308764	30	1376	10292
464	38.67	0.15000	1664	12450	3.23000	35841	268094	31	1156	8648
596	49.67	0.15000	1664	12450	2.76000	30626	229083	31	988	7390



Sump No. 4 - Peak Daily & Peak Monthly Leachate Values

Area = 114435 sf

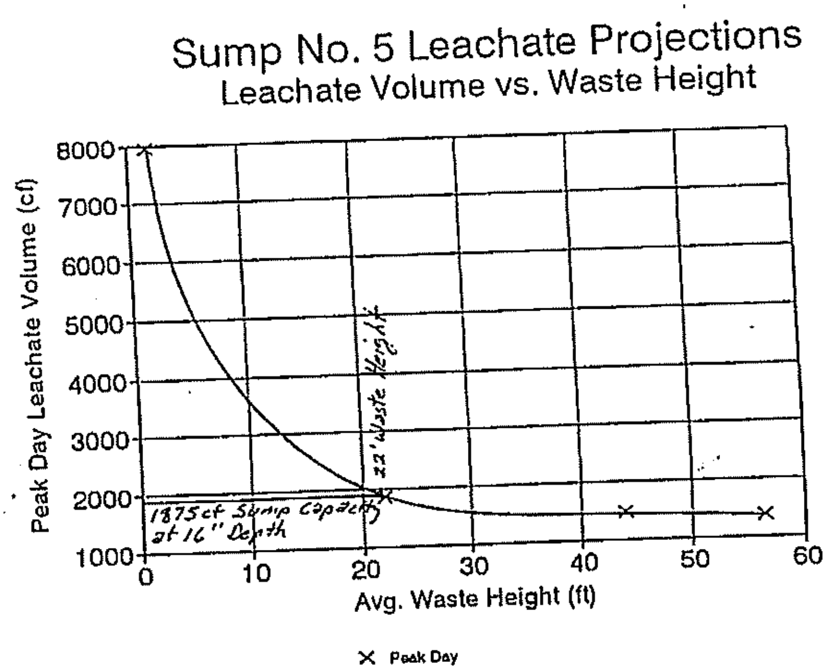
Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per Month	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume				
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)		(cf)	(gal)
21	1.75	0.82000	7820	58492	5.76000	54929	410867	30	1831	13696
249	20.75	0.20000	1907	14266	3.76000	35856	268205	30	1195	8940
498	41.50	0.15000	1430	10700	3.09000	29467	220413	31	951	7110
645	53.75	0.14000	1335	9986	2.64000	25176	188314	31	812	6075



Sump No. 5 - Peak Daily & Peak Monthly Leachate Values

Area = 117926 sf

Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume				
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)	Month	(cf)	(gal)
20	1.67	0.81000	7960	59541	5.77000	56703	424137	30	1890	14138
264	22.00	0.19000	1867	13966	3.77000	37048	277122	30	1235	9237
528	44.00	0.15000	1474	11026	3.01000	29580	221257	31	954	7137
678	56.50	0.14000	1376	10291	2.60000	25551	191119	31	824	6165

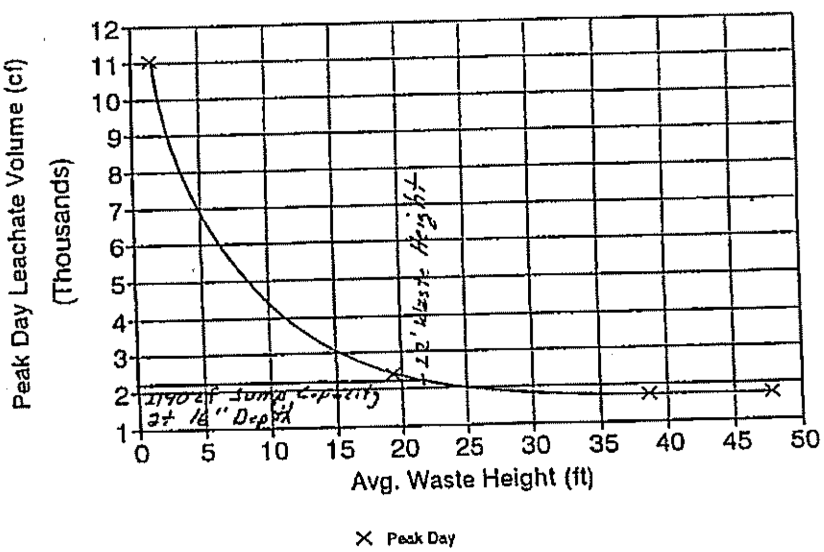


Sump No. 6 - Peak Daily & Peak Monthly Leachate Values

Area = 136959 sf

Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per Month	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume				
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)		(cf)	(gal)
17	1.42	0.97000	11071	82810	5.78000	65969	493445	30	2199	16448
232	19.33	0.21000	2397	17928	3.72000	42457	317581	30	1415	10586
464	38.67	0.15000	1712	12806	3.23000	36865	275749	31	1189	8895
574	47.83	0.15000	1712	12806	2.82000	32185	240747	31	1038	7766

Sump No. 6 Leachate Projections
Leachate Volume vs. Waste Height

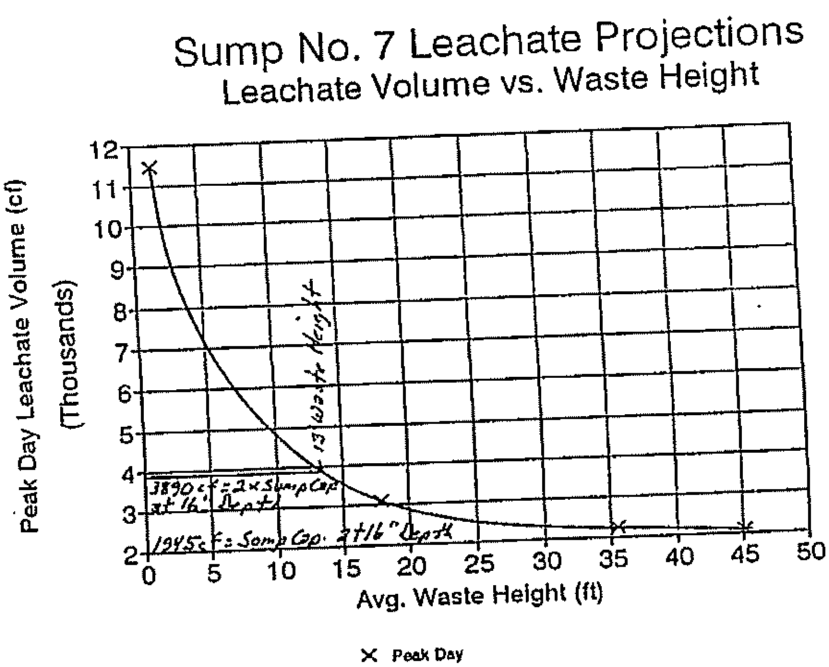


HA&L ENGINEERING		
CLIENT:	USPCI - Lone Mountain Facility	SHEET 27 OF 40
PROJECT:	RCRA Cell 15	COMPUTED: KCS
FEATURE:	Uppermost Leachate System	CHECKED:
PROJECT NO.:	64.44.700	DATE: August, 1996

Sump No. 7 - Peak Daily & Peak Monthly Leachate Values

Area = 167910 sf

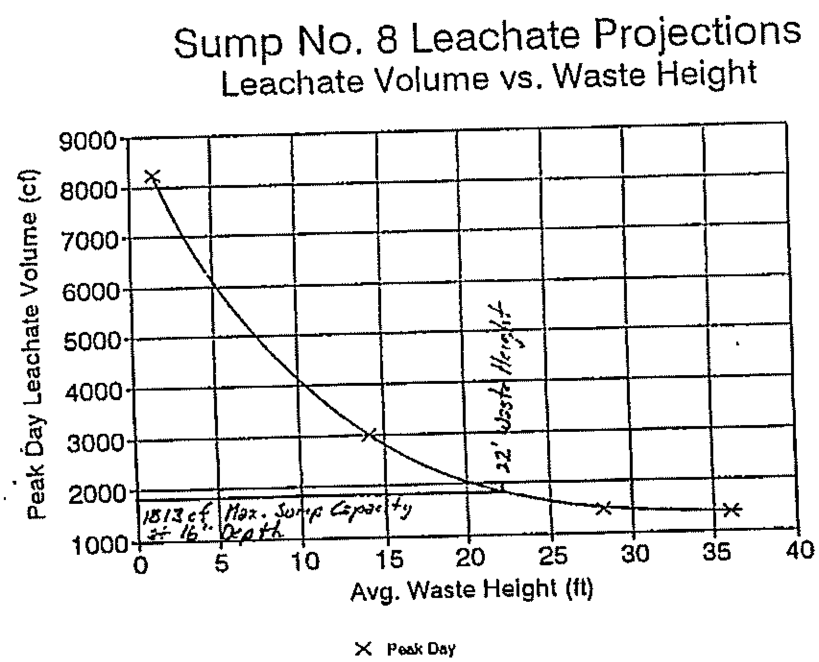
Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume		Month		
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)		(cf)	(gal)
19	1.58	0.82000	11474	85824	5.76000	80597	602864	30	2687	20095
214	17.83	0.22000	3078	23026	3.64000	50933	380977	30	1698	12699
427	35.58	0.16000	2239	16746	3.40000	47575	355857	31	1535	11479
542	45.17	0.15000	2099	15700	2.92000	40858	305619	31	1318	9859



Sump No. 8 - Peak Daily & Peak Monthly Leachate Values

Area = 102852 sf

Avg. Waste Height		Peak Daily Leachate Quantity			Peak Monthly Leachate Quantity			Days Per Month	Avg. Day from Pk. Month	
		Depth	Volume		Depth	Volume				
(in)	(ft)	(in)	(cf)	(gal)	(in)	(cf)	(gal)		(cf)	(gal)
17	1.42	0.96000	8228	61547	5.78000	49540	370562	30	1651	12352
169	14.08	0.35000	3000	22439	3.20000	27427	205155	30	914	6839
338	28.17	0.17000	1457	10899	3.71000	31798	237852	31	1026	7673
432	36.00	0.16000	1371	10258	3.41000	29227	218619	31	943	7052



II. Leachate Collection System

Design components of the uppermost leachate collection system consist of a geonet drainage medium across the cell floor and a collection drain along the valley of the cell floor where the plain surfaces forming the cell floor meet. The geonet collects leachate generated from the sideslopes and floor area of the cell and conveys the leachate to the valley area of the floor. The collection drain collects leachate entering the valley of the floor from the geonet and conveys the leachate to the sumps located at the low point of the each sump drainage area.

A. Geonet Drainage Medium

Two conditions that need to be considered in evaluating the geonet are checking the maximum length of geonet that can be placed to convey leachate to the leachate collection drain and to check the capacity of the net in areas where leachate will accumulate (such as the bottom of the southwest corner in sump no. 1).

The synthetic drainage net or geonet will be designed using the design-by-function concept recommended by EPA for the design of RCRA hazardous waste facilities. According to EPA (1989, pg. 56), "whatever parameter of a specific material one is evaluating, a required value for the material must be found using a design model and an allowable value for the material must be determined by a test method. The allowable value divided by the required value yields the design ratio, or the resulting factor of safety." Thus, in evaluating the drainage net for the leachate collection system, an allowable transmissivity is divided by the required transmissivity to determine the factor of safety for the design, or:

$$\text{Factor of Safety (FS)} = \theta_{allow}/\theta_{req}$$

where

θ_{allow} = the allowable transmissivity as obtained from laboratory testing, and

θ_{req} = the required transmissivity as obtained from design of the actual system.

Koerner (1990) in "Designing with Geosynthetics" suggests that additional factors of safety be applied to the transmissivity value found by test method to account for creep deformation, or intrusion, of the adjacent geosynthetics into the geonet's core space, and for biological and chemical clogging in the geonet's core space. In accordance with the procedures recommended by Koerner (1990), an additional factor of safety of 1.4 will be applied to the transmissivity found by test method for creep deformation of the geonet or intrusion of adjacent geosynthetics into the geonet's core space, and an additional factor of safety of 2 will be applied to the test transmissivity for potential biological and chemical clogging of the geonet. This value thus becomes the allowable value to be used in the equation above. This is in addition to a factor of safety of 1.5 to be used in the design-by-function concept discussed above.

Thus;

$$\theta_{req} = \theta_{allow}/(1.4 \times 2 \times 1.5)$$

$$\theta_{req} = \theta_{allow}/4.2$$

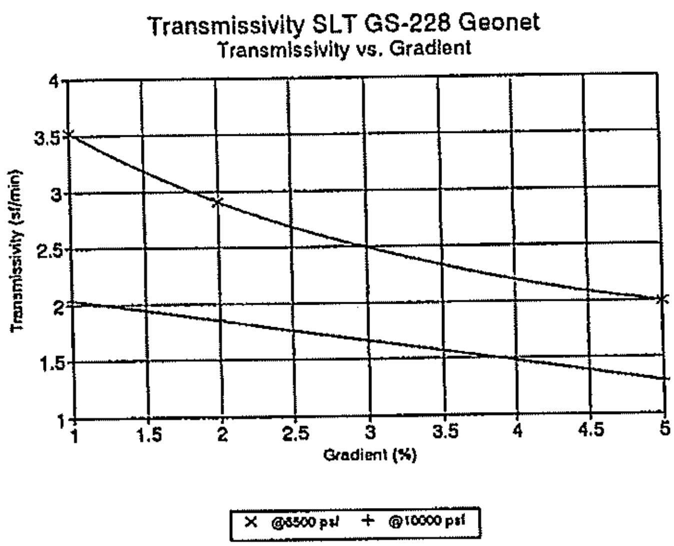
The equation governing flow within the geonet is:

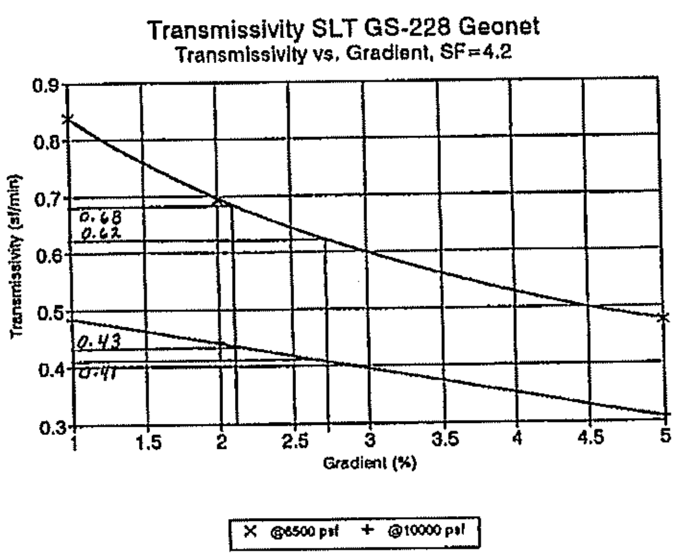
$\theta i = Q/\beta$ Where: θ = Geonet Transmissivity,
i = Gradient,
Q = Flowrate in the Geonet,
β = Width Perpendicular to the Flow,

The following tables and graphs present actual test values and design values for the geonet. The design values assuming a safety factor of 4.2 is applied to the test data provided.

SLT GS-228 Geonet laboratory test data based on the boundary conditions as follows:
Load Plate/Soil (sand)/Polyfelt TS-700 Geotextile/SLT GS-228 Geonet/HDPE Geomembrane/Load Plate

Gradient	Transmissivity at 6500 psf		Transmissivity at 10000 psf		Transmissivity at 6500 psf		Transmissivity at 10000 psf	
(percent)	sm/sec	sf/min	sm/sec	sf/min	sm/sec	sf/min	sm/sec	sf/min
1	5.45E-03	3.52	3.15E-03	2.03	1.30E-03	0.84	7.50E-04	0.48
2	4.50E-03	2.91	2.88E-03	1.86	1.07E-03	0.69	6.86E-04	0.44
5	3.10E-03	2.00	2.00E-03	1.29	7.38E-04	0.48	4.76E-04	0.31





1. The maximum length for which a single layer of GS-228 geonet can be placed, assuming non-converging flow, is calculated as follows:

Continuity Equation:

Flow into the geonet is equal to the downward percolation (design leachate rate) times the area over which percolation occurs.

$Q = AV = VL\beta$ Where:
- V = Design leachate rate,
- L = Length of flow path,
- β = Width perpendicular to flow
- Q = Total flow into the net from vertical percolation in the area of $L \times \beta$.

The areas where the longest flow path will occur through the net is where the flow will be conveyed down a sideslope and along the bottom of the cell to intersect with the collection drain. The longest flow path and the least potential gradient for the geonet which would provide a condition that design can be based on is in either sump no. 1 or sump no. 3. Sump no. 1 has the least gradient and sump no. 3 has the longest flow path. The flow capacity of the geonet on the floor of the cell will govern the length of the flow path. The total flow path in sump no. 1 is about 300 feet of which about 150 feet is on the floor of the cell with a floor slope of 1.44 percent. Assuming a maximum head of 1 foot is allowed on the floor the resulting gradient is:

$$(150 \times 0.0144+1)/150 = .0211 \text{ ft/ft} = 2.11 \text{ percent}$$

The flow length in sump no. 3 is about 350 feet of which about 230 feet is on the floor which has a slope of 2.26 percent. Assuming a maximum head of 1 foot is allowed on the floor, the resulting gradient is:

$$(230 \times 0.0226+1)/230 = .0269 \text{ ft/ft} = 2.69 \text{ percent}$$

The following table provides the calculations for the maximum length that can be allowed for a single layer of GS-228 geonet. Assume β = 1 foot

Condition	Design Leachate Rate, V		Geonet Allowable Transmissivity, θ (sf/min)	Gradient, i (percent)	Maximum Drainage Length to a Single Layer of Geonet, L (feet)
	(in/day)	(ft/min)			
Sump No. 1, Empty	0.82	4.7454e-05	0.68	2.11	302.36 (ok)
Sump No. 1, Half Full	0.22	1.2731e-05	0.68	2.11	1126.97
Sump No. 1, Level Full	0.16	9.2593e-06	0.43	2.11	979.88
Sump No. 1, Full	0.15	8.6806e-06	0.43	2.11	1045.21
Sump No. 3, Empty	0.82	4.7454e-05	0.62	2.69	351.46 (ok)
Sump No. 3, Half Full	0.21	1.2153e-05	0.62	2.69	1372.36
Sump No. 3, Level Full	0.15	8.6806e-06	0.41	2.69	1270.54
Sump No. 3, Full	0.15	8.6806e-06	0.41	2.69	1270.54

Based on the data presented in the table, the geonet functions appropriately for all conditions at a design safety factor of 4.2 and a maximum head on the uppermost liner system of 1 foot.

2. Converging flow to the drainage net from the southwest corner of sump no.1 is also a controlling design condition. The contributing area to the net is 24,417 sf. which needs to be conveyed within a flow path width of 24 feet (for a presentation of drainage areas see sheet 34). The flow based on an empty condition is:

   (4.7454e-05 ft/min) x (24,417 sf) = 1.16 cf/min.

   The flow capacity of the drainage net is:

   (0.68 sf/min) x (.0211 ft/ft) x (24 feet) = 0.98 cf/min NG at a safety factor of 4.2.

   Determine the actual safety factor: 0.98 x 4.2/1.16 = 3.55 which should be ok since the design leachate rate will drop by a factor of (0.82/0.22) = 3.72 shortly after the cell is in operation. The safety factor will then be much greater than 4.2.

B. Leachate Collection Drain (for a presentation of drainage areas see sheet 34)

The largest area contributing to a leachate collection drain in sump areas 1 through 8 is within sump area no. 1. The area contributing drainage to the valley of sump area no. 1 is about 85,677 sf. The design flow that the leachate collection drain should be able to carry is:

(4.7454e-05 ft/min) x (85,677 sf) = 4.07 cf/min at a near empty condition
(1.2731e-05 ft/min) x (85,677 sf) = 1.09 cf/min at a half full condition
(9.2593e-06 ft/min) x (85,677 sf) = 0.79 cf/min at a level full condition
(8.6806e-06 ft/min) x (85,677 sf) = 0.74 cf/min at a full condition

A 3-inch diameter perforated corrugated polyethylene pipe will provide sufficient capacity, assuming open channel flow occurs, to meet all flow conditions above. The capacity at a near empty condition provides a safety factor just greater than 1, however, as the cell fills the safety factor increases to 3.7 at half full, 5.15 at level full and 5.5 at a full condition. The leachate collection drain should, therefore, function adequately. All other sump drainage areas have substantially less area contributing to the 3-inch diameter drain pipe, therefore, the leachate collection drain should function properly for all sump areas.

III. Sump Capacities and Pumping Frequencies.

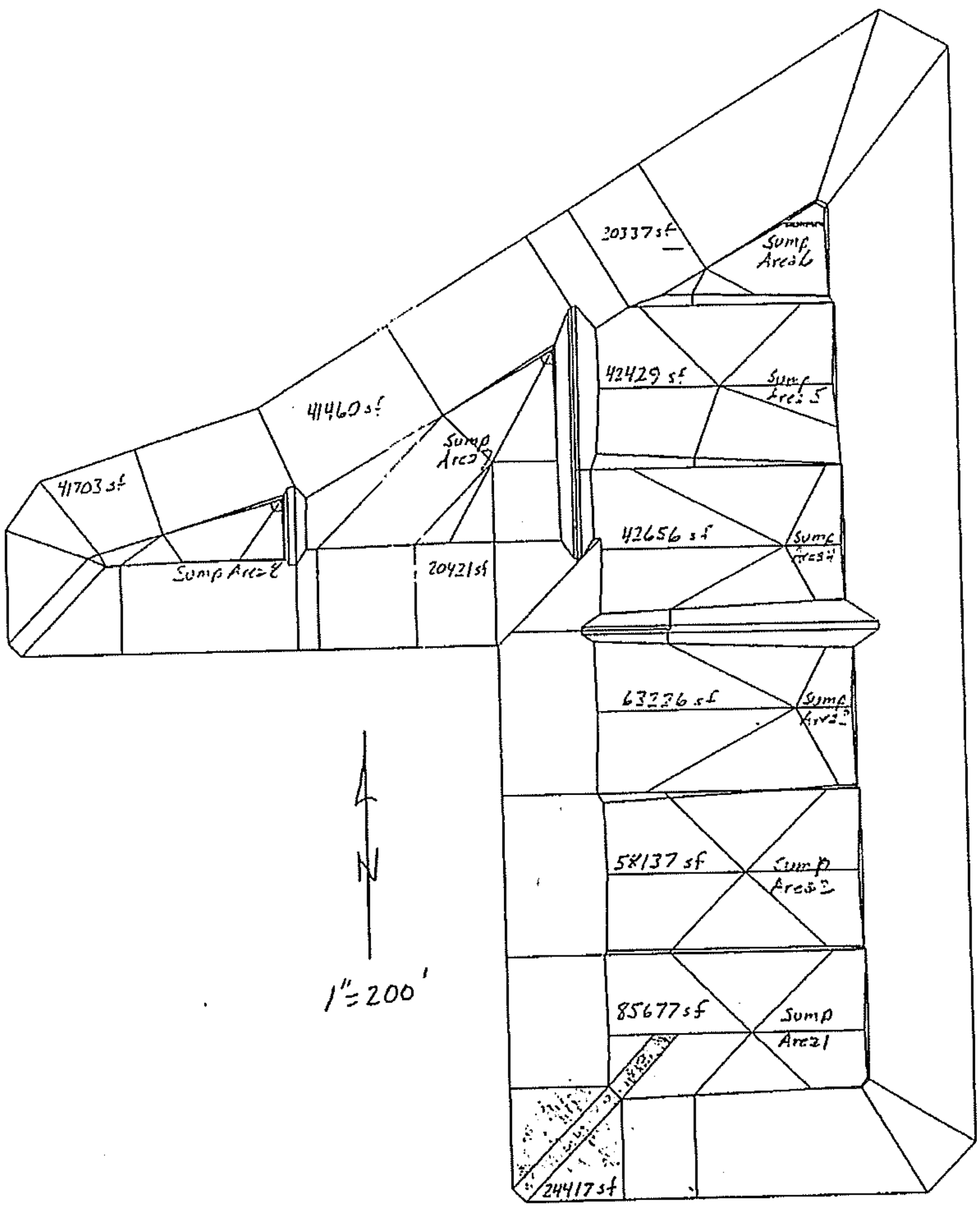
The sump capacities were determined by calculations separate from the calculations presented herein. Results from the calculations are presented on the following pages of stage and elevation vs. capacity tables and graphs. The data presented on the following graphs also present the maximum sump capacity at the 16-inch depth above the uppermost liner system that we understand the Oklahoma DEQ approved in the sumps of the cell.

As each sump area fills with waste, the daily volume of leachate generated from precipitation events decreases which also decreases the frequency for which pumping of leachate from the sumps is necessary. There will also be periods of time when very little leachate will be generated due to dryer weather conditions. The information presented herein presents the pumping frequency that may be required during peak precipitation events, similar to past events, in order the maintain a maximum leachate depth in the sumps of 16 inches. If USPCI opts to install level sensors in the sumps, then the pumping frequency may be determined by monitoring of the level sensors rather than active daily pumping.

The data presented indicates that pumping activities may be required several times per day following precipitation events that generate leachate rates similar to the peak day events resulting from the HELP model. In general, however, the leachate rates generated will be much less than the peak day rates provided by the model and the average daily rates based on the peak month event may be more reasonable for a standard pumping frequency. The tables presented on sheet 40 of these calculations provide the pumping frequencies based on peak day leachate rates and average day rates based on peak month for the various waste levels in the cell.

HA&L ENGINEERING

CLIENT:	USPCI - Lone Mountain Facility	SHEET 34 OF 40	
PROJECT:	RCRA Cell 15	COMPUTED:	KCS
FEATURE:	Uppermost Leachate System	CHECKED:	
PROJECT NO.:	64.44.700	DATE:	August, 1996



HA&L ENGINEERING	*CLIENT:* USPCI - Lone Mountain Facility *PROJECT:* RCRA Cell 15 *FEATURE:* Uppermost Leachate System *PROJECT NO.:* 64.44.700	*SHEET* 35 *OF* 40 *COMPUTED:* KCS *CHECKED:* *DATE:* August, 1996

Sump No. 1 - Stage vs. Capacity

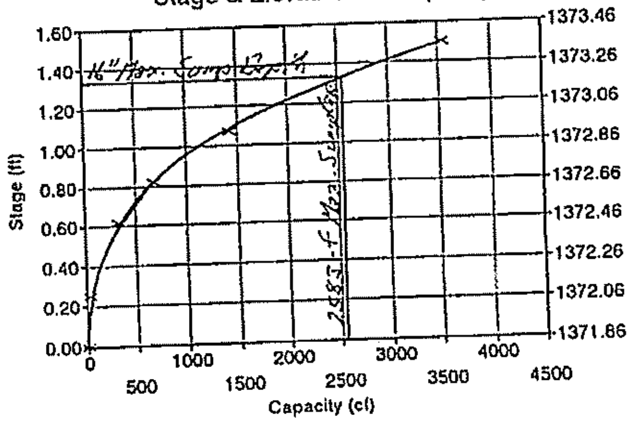
Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1371.86	0.0
0.25	1372.11	28.3
0.62	1372.48	321.3
0.82	1372.68	683.7
1.07	1372.93	1424.4
1.50	1373.36	3538.8

Sump No. 2 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1370.86	0.0
0.25	1372.11	28.3
0.62	1372.48	321.3
1.11	1372.97	1635.2
1.50	1373.36	3713.8

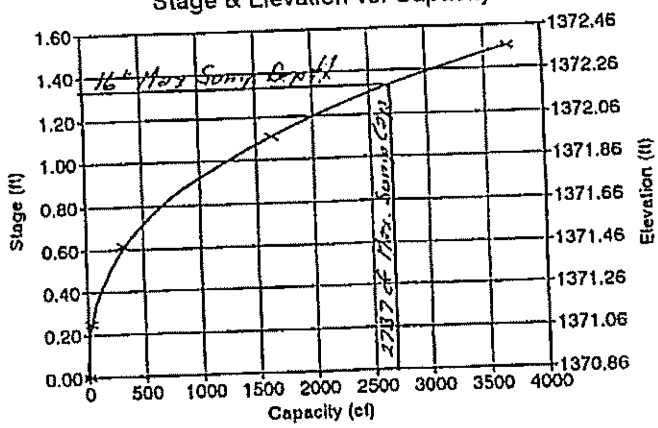
Uppermost Sump No. 1 Capacity

Stage & Elevation vs. Capacity



Uppermost Sump No. 2 Capacity

Stage & Elevation vs. Capacity



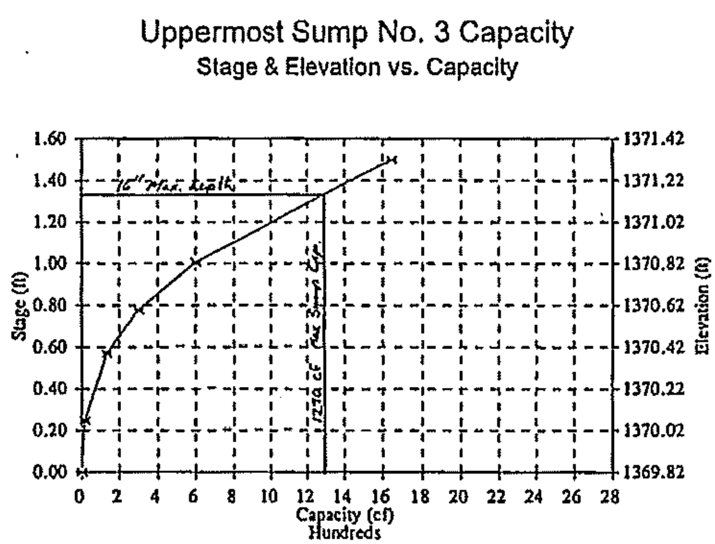
HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility PROJECT: RCRA Cell 15 FEATURE: Uppermost Leachate System PROJECT NO.: 64.44.700	SHEET 36 OF 40 COMPUTED: KCS CHECKED: DATE: August, 1996 Revised 10/14/97

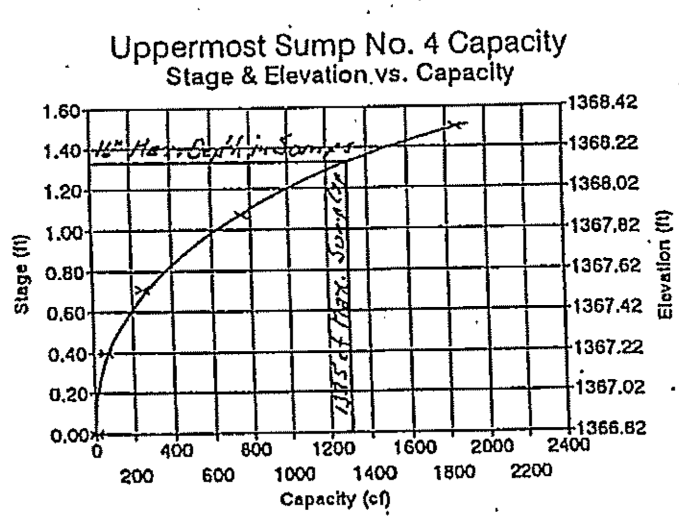
Sump No. 3 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1369.82	0.0
0.25	1370.07	16.7
0.57	1370.39	133.0
0.78	1370.60	303.0
1.01	1370.83	603.0
1.50	1371.32	1644.0

Sump No. 4 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1366.82	0.0
0.40	1370.22	61.0
0.71	1370.53	253.0
1.07	1370.89	757.3
1.50	1371.32	1851.2



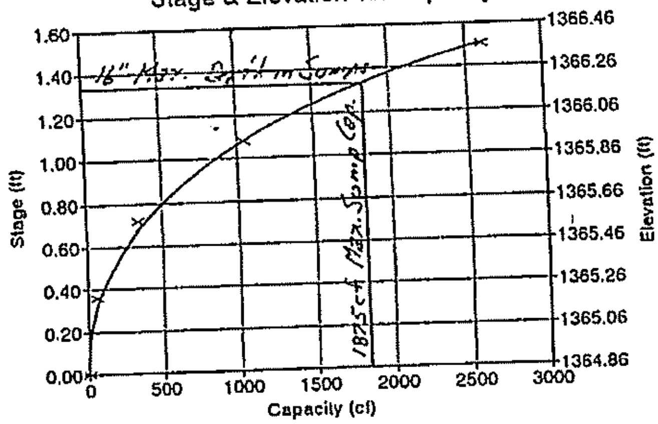


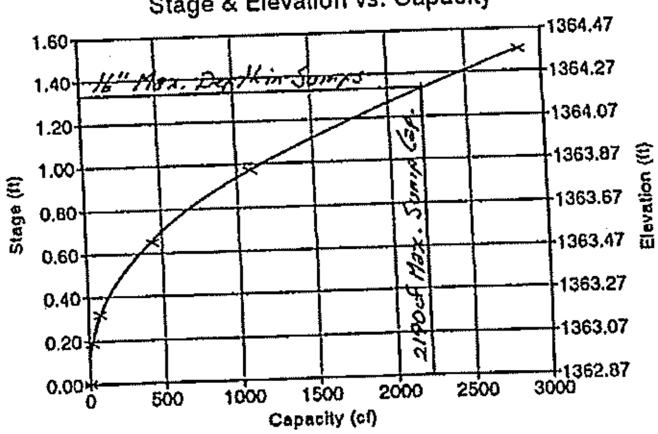
Sump No. 5 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1364.86	0.0
0.36	1365.22	58.2
0.72	1365.58	342.4
1.08	1365.94	1052.8
1.50	1366.36	2605.8

Sump No. 6 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1362.87	0.0
0.19	1365.05	21.4
0.32	1365.18	74.8
0.65	1365.51	439.7
0.97	1365.83	1091.0
1.50	1364.37	2857.9





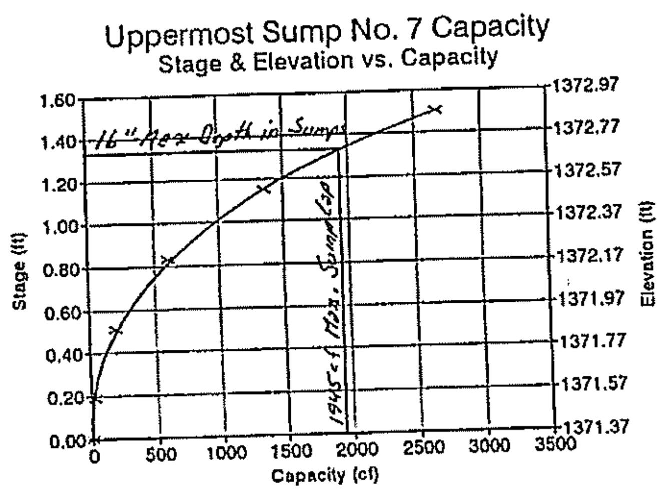
HA&L ENGINEERING	CLIENT: USPCI - Lone Mountain Facility	SHEET 38 OF 40
	PROJECT: RCRA Cell 15	COMPUTED: KCS
	FEATURE: Uppermost Leachate System	CHECKED:
	PROJECT NO.: 64.44.700	DATE: August, 1996

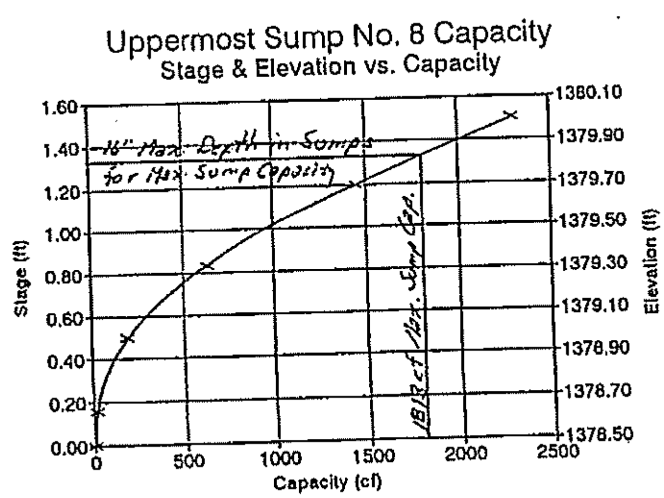
Sump No. 7 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1371.37	0.0
0.19	1371.56	24.4
0.51	1371.88	200.1
0.83	1372.20	612.5
1.15	1372.52	1358.0
1.50	1372.87	2665.9

Sump No. 8 - Stage vs. Capacity

Stage (ft)	Elevation (ft)	Capacity (cf)
0.00	1378.50	0.0
0.16	1371.53	15.6
0.50	1371.87	192.4
0.83	1372.20	638.6
1.20	1372.57	1453.0
1.50	1380.00	2297.1





FIND: Determine the capacity of a 3-inch diameter corrugated polyethylene pipe in conveying leachate along the floor valley created by the intersecting floor plains.

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q) =	0.068 cfs	4.07 cfm
Manning n (n) =	0.020	
Pipe Diameter (d) =	0.258 feet	3.1 inches
Slope (So) =	0.01	
Normal Depth (y) =	0.237 feet	
Flow x-section area (A) =	0.050 sq. ft.	
Flow Top Width (T) =	0.143 feet	
Perimeter (P) =	0.659 feet	
Hyd. Radius (R) =	0.076 feet	
Flow Velocity (V) =	1.349 ft/sec.	
Froude Number =	0.401	
Theta =	5.105 radians	
Solve Equation =	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc) =	0.258 feet
Critical area (Ac) =	0.052 sq. ft.
Top Width (Tc) =	0.000 feet
Perimeter (Pc) =	0.812 feet
Hyd. Radius (Rc) =	0.065 feet
Flow Velocity (Vc) =	1.294 ft/sec.
Froude Number =	0.000
Theta =	6.283 radians

Sump 1 = 2583 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	13509	0.2	3163	0.8
1/2 full	3624	0.7	1999	1.3
Level full	2636	1.0	1889	1.4
full	2471	1.0	1609	1.6

Sump 2 = 2737 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	9220	0.3	2133	1.3
1/2 full	2333	1.2	1377	2.0
Level full	1666	1.6	1154	2.4
full	1666	1.6	1000	2.7

Sump 3 = 1290 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	9099	0.1	2131	0.6
1/2 full	2330	0.6	1376	0.9
Level full	1664	0.8	1156	1.1
full	1664	0.8	988	1.3

Sump 4 = 1335 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	7820	0.2	1831	0.7
1/2 full	1907	0.7	1195	1.1
Level full	1430	0.9	951	1.4
full	1335	1.0	812	1.6

Sump 5 = 1875 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	7960	0.2	1890	1.0
1/2 full	1867	1.0	1235	1.5
Level full	1474	1.3	954	2.0
full	1376	1.4	824	2.3

Sump 6 = 2190 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	11071	0.2	2199	1.0
1/2 full	2397	0.9	1415	1.5
Level full	1712	1.3	1189	1.8
full	1712	1.3	1038	2.1

Sump 7 = 1945 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	11474	0.2	2687	0.7
1/2 full	3078	0.6	1698	1.1
Level full	2239	0.9	1535	1.3
full	2099	0.9	1318	1.5

Sump 8 = 1813 cf

	Peak Day		Avg Day from Peak Mont	
Condition	Volume (cf)	Frequency (days)	Volume (cf)	Frequency (days)
Empty	8228	0.2	1651	1.1
1/2 full	3000	0.6	914	2.0
Level full	1457	1.2	1026	1.8
full	1371	1.3	943	1.9

NOTE: POLYFELT WAS SOLD TO TENSAR CORPORATION WHO THEN BEGAN MANUFACTURING POLYFELT TS-700 UNDER A NEW PRODUCT NAME OF TENSAR TG-700. ATTACHED IS A COPY OF THE PRODUCT SPECIFICATIONS FOR TENSAR TG-700 VERIFYING THAT THE MATERIALS ARE THE SAME.

Evergreen Technologies

June 21, 1996

Tensar Corporation
1210 Citizens Parkway
Morrow, GA 30260

Subj: TG700 Geotextile Certificate of Compliance

Re : Laidlaw Environmental, Lone Mountain Facility, Order # 001061, PO # 6-8097

Dear Sir/Madam:

This letter certifies that TG700, shipped FOB Evergreen, Alabama, on 6/17/96, manufactured by Evergreen Technologies, meets or exceeds the minimum requirements listed below.

PROPERTY	TEST PROCEDURE	VALUE(1)		
Weight	ASTM D 5261	8.0		oz/yd2
Thickness	ASTM D 5199	90	*	Mil
Grab Strength	ASTM D 4632	210		lbs
Grab Elongation	ASTM D 4632	50		%
Tear Strength	ASTM D 4533	80		lbs
Mullen Burst	ASTM D 3786	400		psi
Puncture Resistance	ASTM D 4833	100		lbs
A.O.S.	ASTM D 4751	.212	*	US Std Sieve
		(70)	*	mm
Permittivity	ASTM D 4491	1.3	*	1/sec
Water Permeability	ASTM D 4491	0.3	*	cm/sec
Water Flow Rate	ASTM D 4491	100	*	gpm/sq ft
U.V. Resistance (500 hours)	ASTM D 4355	70		%

(1) Values in weaker principle direction. Unless noted otherwise, these values represent minimum average roll values (i.e. test results from any sampled roll in a lot, tested in accordance with ASTM D 4759-88 shall meet or exceed the minimum values listed).

* Determined at the time of manufacturing, storage and handling conditions which differ from those found in ASTM D 4873-88 may influence these properties.

Unless noted otherwise, this certification is based on testing conducted by Evergreen Technologies Quality Assurance & Quality Control testing laboratories at the time of manufacturing. Evergreen Technologies issues this letter of certification to indicate our commitment to providing our customers with a quality product which will meet or exceed the minimum average roll values in accordance with the applicable American Society for Testing and Materials (ASTM) test method.

Sincerely,

Manoj Tyagi
QA Manager

APPENDIX 3

Geotextile Filter Fabric

I. Geotextile filter fabric is to be placed on top of the drainage net to serve as a filter for the overlying materials. Check design criteria of Table 3-3 p3-30 "Geotextile Engineering Manual" by U.S. Department of Transportation" to determine the soil retention and permeability criteria that must be met with the soil protective cover material. The geotextile filter fabric that is proposed for use is Polyfelt TS-700.

A. Polyfelt TS-700 Geotextile Filter Fabric.

Product design specs. for Polyfelt TS-700 are:

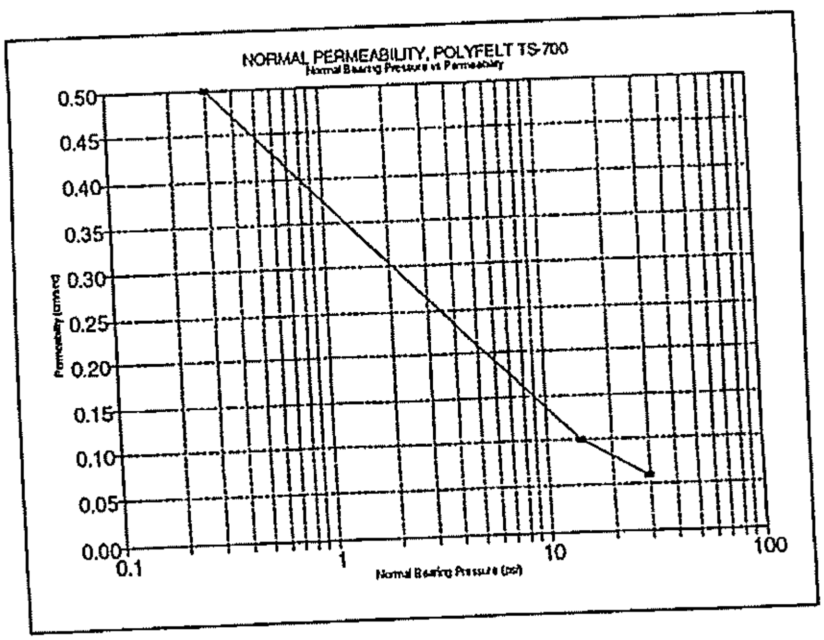
U. S. Standard sieve size - Equivalent Opening Size (EOS) of the fabric:

EOS = 70 to 120 sieve

Water permeability (k_v) normal to the plane of the fabric taken from the manufacturers specification sheet for the fabric is:

$k_v = 50 \times 10^{-2}$ for a normal bearing pressure of 0.3 psi.
$k_v = 10 \times 10^{-2}$ for a normal bearing pressure of 14.5 psi.
$k_v = 6 \times 10^{-2}$ for a normal bearing pressure of 29.0 psi.

Below is a curve of permeability normal to the plane of the fabric versus the normal bearing pressure on the fabric generated from the data presented above.



B. Soil Retention
Criteria from Table 3-3 for:

$\leq$ 50% passing the #200 sieve.

$$AOS\ (O_{95}) = EOS \leq B^*D_{85\ (soil)}$$

where: $B = 1$ for $C_u \leq 2$ or $C_u \geq 8$
$B = 0.5C_u$ for $2 \leq C_u \leq 4$
$B = 8/C_u$ for $4 < C_u < 8$

and:

$$C_u = D_{60\ (soil)}/D_{10\ (soil)}$$

$D_{85\ (soil)} > EOS/B$ therefore, for Polyfelt TS-700:

$D_{85\ (soil)} > 0.212$ mm for $C_u \leq 2$ or $C_u \geq 8$
$D_{85\ (soil)} > 0.212/0.5C_u$ for $2 \leq C_u \leq 4$
$D_{85\ (soil)} > 0.212C_u/8$ for $4 < C_u < 8$

$\geq$ 50% passing the #200 sieve.

$O_{95} = EOS$
$O_{95} \leq 1.8D_{85(soil)}$
and AOS $No._{(fabric)} \geq$ No. 50 sieve

$D_{85\ (soil)} > EOS/1.8$ therefore, for Polyfelt TS-700:

$$D_{85\ (soil)} > 0.10 \text{ mm}$$

C. Permeability Criteria

$$k_{v\ (fabric)} \geq 10^*k_{v\ (soil)} \text{ "or" } k_{v\ (soil)} \leq k_{v\ (fabric)}/10$$

The fabric permeability is dependent on the normal bearing pressure as presented with the fabric specifications for Polyfelt TS-700. The soil permeability criteria is, therefore, also dependent on the normal bearing pressure on the fabric.

II. Check the soil protective cover material with the above criteria for soil retention and permeability using the Polyfelt TS-700 Geotextile Filter Fabric.

A. Normal Bearing Pressure (N)

The maximum elevation difference between the top of the future closure cap and the fabric occurs at the center ridge line of the closure cap above the sump 5 center flow line. Evaluate elevation to surface of middle liner.

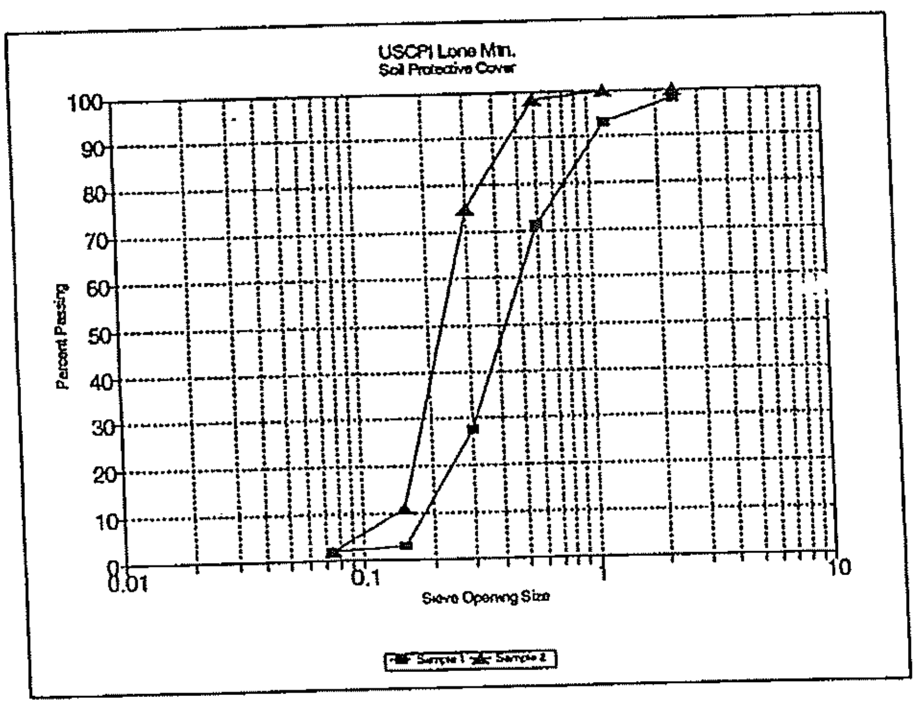
$$\Delta Elev. = 1441.9 - 1365.5 = 76.4 \text{ feet}$$

The Normal Bearing Pressure is:

0.83′ Erosion Protective Rock at 110 pcf	= 88.0 psf
70.1′ Waste at 120 pcf	= 8412.0 psf
5.50′ Soil Protective Cover at 125 pcf	= 687.5 psf
	9,187.5 psf
N TOTAL	= 63.8 psi

B. Check the Soil Protective Cover material used of previous construction projects with the soil retention criteria above. Gradation analyses were conducted on two samples of the Soil Cover material and the results are shown below:

Sieve No.	Opening Size (mm)	Sample 1 % Passing	Sample 2 % Passing
8	2.380	98	100
16	1.190	93	100
30	0.590	71	98
50	0.297	28	75
100	0.149	3	11
200	0.074	0	0



In the case of both samples taken, less than 50 percent passes the No. 200 sieve. The Uniformity Coefficient of each sample is:

$C_{u1} = 0.22/0.075 = 2.93 \qquad C_{u2} = 0.23/0.037 = 6.22$

$D_{85(1)} > 0.212/0.5(2.93) = 0.14$ mm "and" $D_{85(1)} = 0.39$ mm OK

$D_{85(2)} > 0.212(6.22)/8 = 0.16$ mm "and" $D_{85(2)} = 0.46$ mm OK

C. Check the permeability of Soil Protective Cover material used on previous construction projects with the above criteria. Information obtained during the design of Containment Facilities 1 thru 5 in Minnesota indicated that for a normal bearing pressure of 63.5 psi the $k_{(fabric)} = 4 \times 10^{-2}$ cm/sec.

$k_{(fabric)} = \sim 0.04$ cm/sec $\therefore k_{(soil)} < .03/10 = .003$ cm/sec "or" 3×10^{-3} cm/sec.

According to permeability tests conducted by Chen-Northern, Inc. during a previous construction project the Soil Protective Cover material had the following permeabilities:

at 90% compaction 1×10^{-3} cm/sec $< 3 \times 10^{-3}$ cm/sec OK
at 95% compaction 8×10^{-4} cm/sec $< 3 \times 10^{-3}$ cm/sec OK

III. Check the strength of the Filter Fabric against Burst Resistance. Since the geotextile fabric is being placed on the geonet, the fabric must have sufficient strength to bridge the ridges of the geonet without failure. According to Robert M. Koerner (1990) in "Designing with Geotextiles" (published by Prentice-Hall, Inc.) the required fabric burst strength to bridge the gap is:

$$T_{req'd} = p' d_v$$

where

$T_{req'd}$ = the required fabric strength
p' = the stress at the fabric's surface, which in the worst case would equal the overburden stress at closure = 63.8 psi.
d_v = the maximum void diameter, or in this case the gap distance between ridges of the geonet = 0.5 inches.

Thus, $T_{req'd} = (63.8)(0.5) = 31.9$ psi

The geotextile will be designed using the design-by-function concept recommended by EPA for the design of hazardous waste facilities. According to EPA (1989, pg. 56), "whatever parameter of a specific material one is evaluating, a required value for the material must be found using a design model and an allowable value for the material must be determined by a test method. The allowable value divided by the required value yields the design ratio, or the resulting factor of safety." Thus in evaluating the tensile resulting factor of safety." Thus in evaluating the tensile strength requirement for the filter fabric, an allowable tensile strength is divided by the required tensile strength to determine the factor of safety for the design, or:

Factor of Safety (FS) = $T_{allow}/T_{req'd}$

where

T_{allow} = the allowable tensile strength as obtained from laboratory testing, and

$T_{req'd}$ = the required tensile strength as obtained from design of the actual system

Koerner (1990) in "Designing with Geosynthetics" suggests that additional factors of safety be applies to the tensile strength value found by test method to account for installation damage, creep and for biological and chemical degradation. IN accordance with the procedures recommended by Koerner (1190), an additional factor of safety of 1.5 will be applies to the tensile strength found by test method for installation damage, an additional factor of safety of 1.2 will be applied to the tensile strength value for creep, and an additional factor of safety of 1.8 will be applies to the test tensile strength for potential biological and chemical degradation. This value becomes the allowable value to be used in the equation above. This is in addition to the factor of safety to be used in the design-by-function concept discussed above. The test value is the Mullen burst Strength which is equal to 320 psi for Polyfelt TS-700. Thus,

$$T_{allow} = \frac{320}{(1.5x1.2x1.8)} = 98.8 \frac{lbs}{ft^2}$$

$$FS = \frac{98.8}{31.9} = 3.1$$

IV. Koerner (1990) also defines another process acting on the fabric at the same time as the tendency to burst. This is one of tensile stress being mobilized by in-place deformation. This would occur when the geotextile fabric is locked into position by the soil above it and the ridges of the geonet below it. A lateral or in-place stress could be mobilized if two ridges of the geonet were to give or spread outward form the load of the soil placed on top. The maximum strain would occur if the ridges folded over completely, thus stressing the filter fabric. This maximum strain would be equal to the height of the tow ridges divided by the original gap separation. The height of each ridge is approximately 0.1 inches. The ga; separation between the ridges in 0.5 inches. Thus, the maximum strain would be 0.2/0.5 = .4 or 40%. Koerner defines the tensile force being mobilized as being related to the pressure exerted on the fabric as follows:

$$T_{req'd} = p'(e)^2$$

$T_{req'd}$	=	the mobilized tensile force
p'	=	the applied pressure which would equal the overburden stress at closure = 63.8 psi.
e	=	the strain of the geotextile between contact points, = 0.4

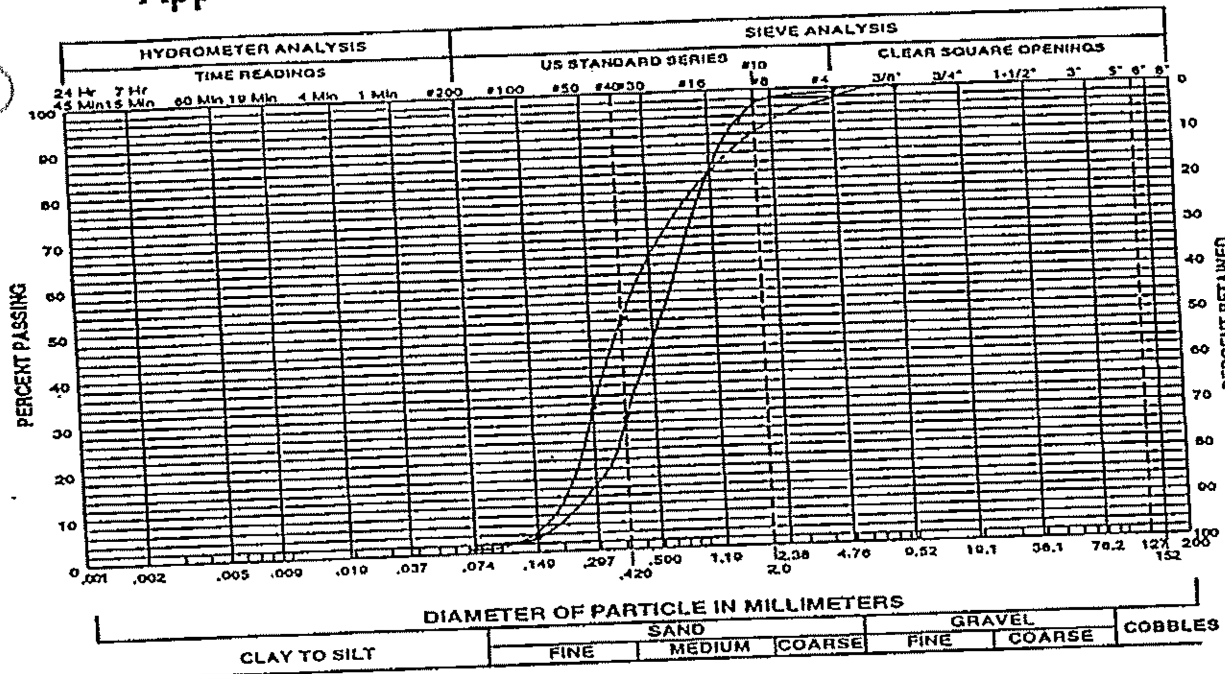
Thus, $T_{req'd} = 63.8(0.4)^2 = 10.2$ lbs.

To determine the factor of safety (FS), $T_{req'd}$ is compared with an allowable T which is the grab strength divided by the additional factors of safety referred to above. The Grab Tensile Strength for Polyfelt TS-700 is 210 lbs.

$$T_{allow} = \frac{210}{(1.5x1.2x1.8)} = 64.8\ \frac{lbs}{ft^2}$$

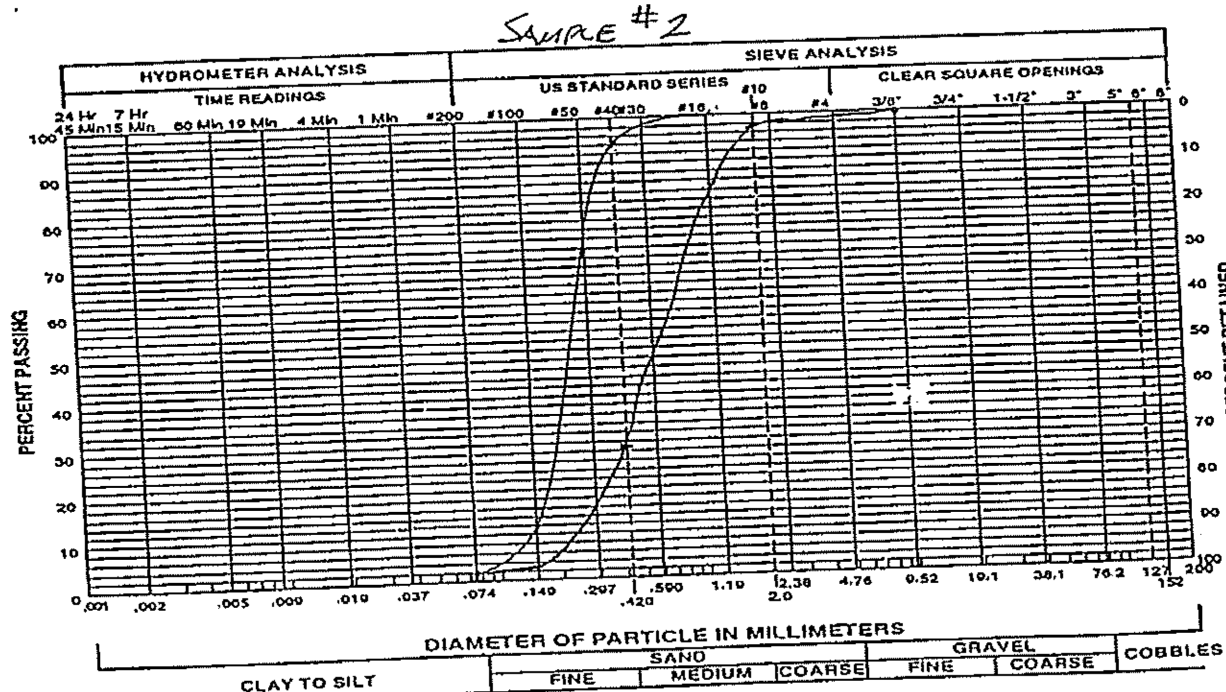
$$FS = \frac{64.8}{10.2} = 6.4$$

Applied Geotechnical Engineering Consultants, Inc.



Gravel 2 %
Sand 97 %
Silt and Clay 1 %
Liquid Limit ___ %
Plasticity Index N.P. %
Sample of WASH SAND
From ___

SAMPLE #2



Gravel 0 %
Sand 98 %
Silt and Clay 2 %
Liquid Limit ___ %
Plasticity Index N.P %
Sample of BLOW SAND
From ___

Project No. ___

GRADATION TEST RESULTS

Figure ___

EAGLE [illegible]AND +
GRAVEL

APPLIED GEOTECHNICAL ENGI[illegible]ERING CONSULTANTS, INC.

Sheet Prep. By DH. Date 6/24/92
Sheet Calc. By_____ Sheet___of___

GRADATION ANALYSIS
WORKSHEET

ob No.________
ob Name________

Hole @ Depth	6-24-92	6-24-92				
Sample No.	WASH SAND	BLOW SAND				
	-200 / Gradation	-200 / Gradation	-200 / Gradation	-200 / Gradation	-200 / Gradation	-200 / Gradation
Dish Name						
Dry Soil & Dish						
Dish Wt.						
Dry Soil Wt.	360.7 G.	255.7 G				

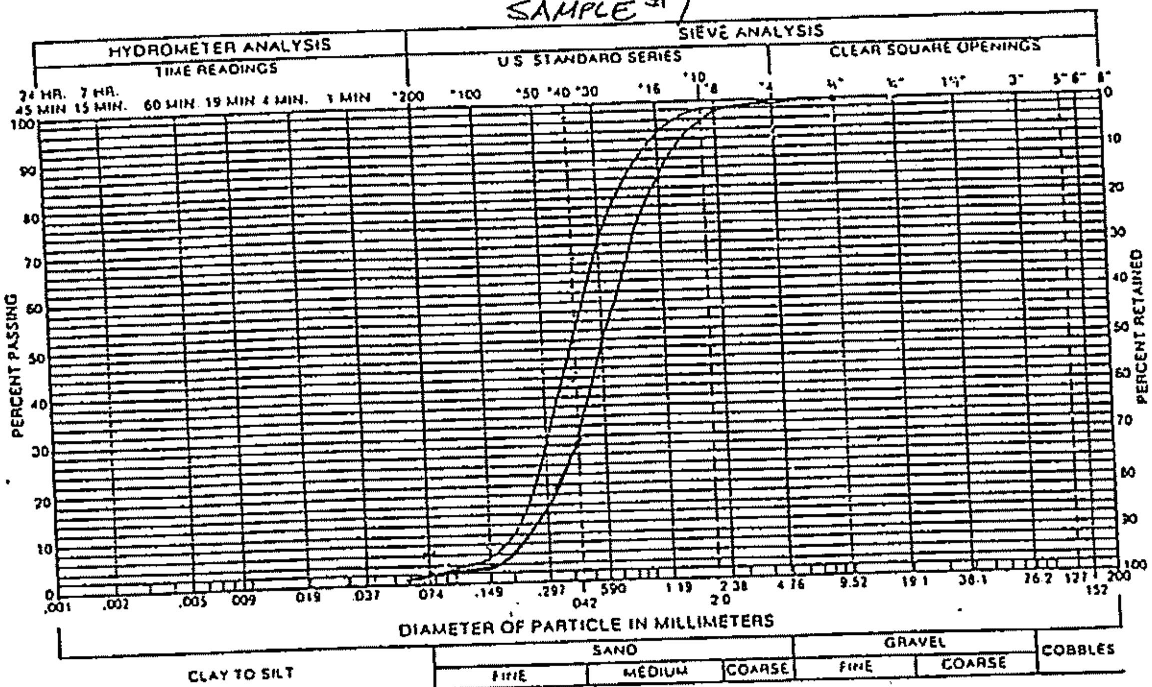
SIEVE SIZE	WT.		% PASS	WT.		% PASS	WT.		% PASS	WT.		% PASS	WT.		% PASS	WT.		% PASS
3.0"																		
1.5"																		
.75"																		
.375"	0	0	100															
# 4	7.2	2.0	98															
# 8																		
# 16	59.6	16.5	82	0	0	100												
# 30																		
# 40	114.6	31.8	50	14.6	5.7	94												
# 50	66.7	18.5	31	60.6	23.7	71												
# 100	98.1	27.2	4	151.9	59.4	11												
# 200	12.5	3.5	1	24.0	9.4	2												

Percent Gravel	2					
Percent Sand	97	98				
Percent Fines	1	2				

AGEC

Applied Geotechnical Engineering Consultants

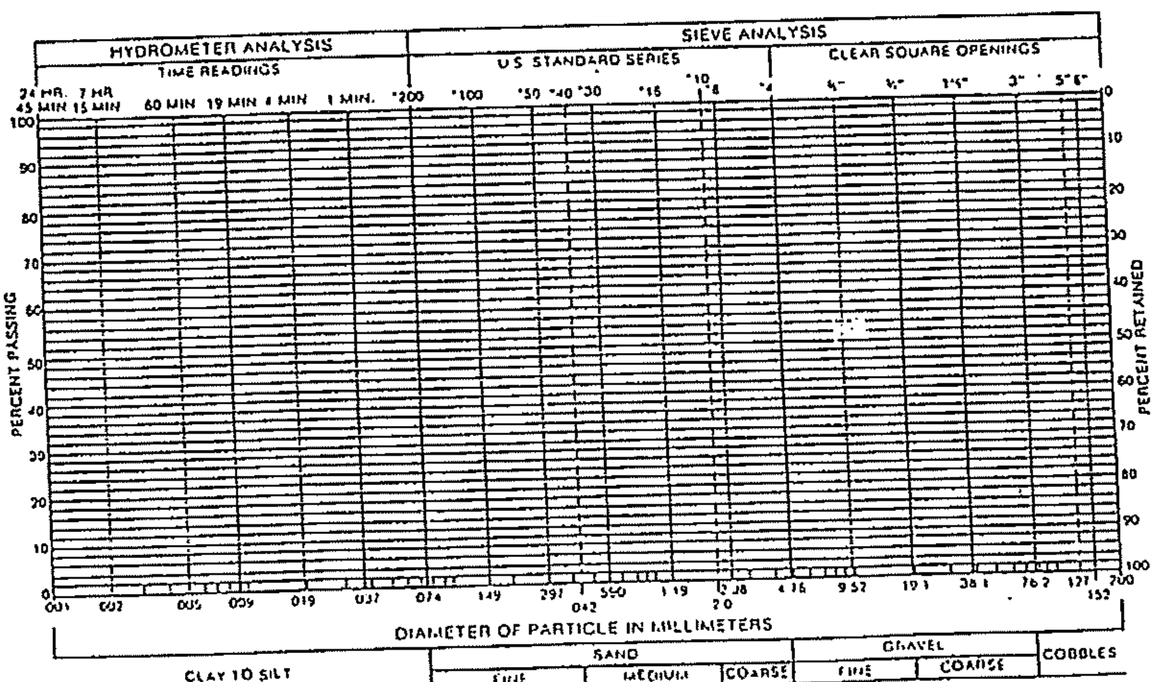
SAMPLE #1



GRAVEL 1 % SAND 97 % SILT AND CLAY 2 %

LIQUID LIMIT N/A % PLASTICITY INDEX N.P. %

SAMPLE OF Poorly Graded Sand FROM Tertiary Soil Protective Cover Stockpile

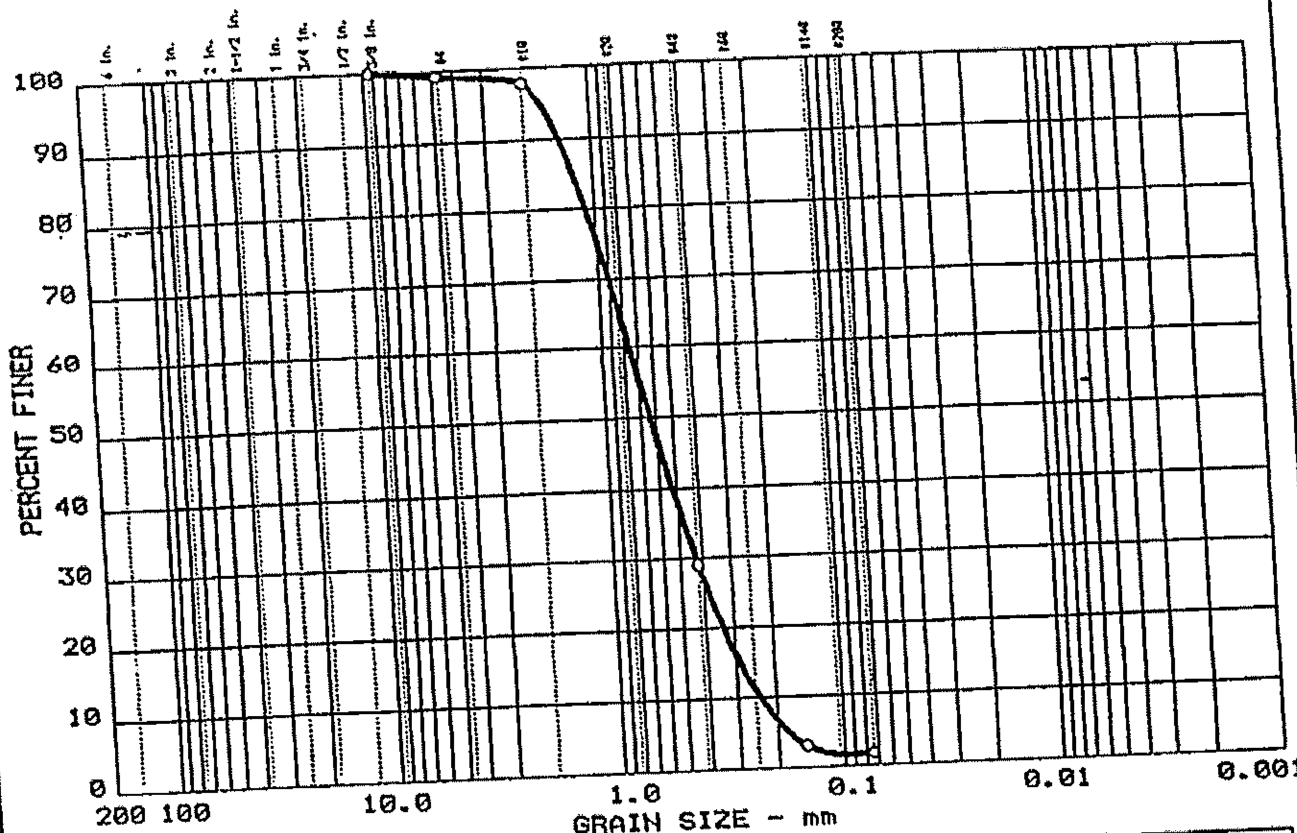


GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

GRAIN SIZE DISTRIBUTION TEST REPORT



	Test	%+75	% GRAVEL	% SAND	% SILT	% CLAY
o	4	0.0	0.8	97.4	1.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
o			1.24	0.76	0.63	0.429	0.2867	0.2358	1.03	3.2

MATERIAL DESCRIPTION	USCS	AASHTO
o SAND, MEDIUM GRAINED	SP	

Project No.: 4220 90-590.05
Project: MN. INDUSTRIAL CONTAINMENT FACILITY
o Location: ROSEMOUNT, MINNESOTA

Date: 7-2-90

GRAIN SIZE DISTRIBUTION TEST REPORT
TWIN CITY TESTING CORPORATION

Remarks:
BORING NO. F-5
SAMPLE NO. 11
DEPTH (ft.): 24.5-26.5

Figure No.

GRAIN SIZE DISTRIBUTION TEST DATA

Test No.: 4

7-2-90
...t No.: 4220 90-590.05
...ject: MN. INDUSTRIAL CONTAINMENT FACILITY

Sample Data

...cation of Sample: ROSEMOUNT, MINNESOTA
...mple Description: SAND, MEDIUM GRAINED
...CS Class: SP
...SHTO Class:
Liquid limit:
Plasticity index:

Notes

...marks: BORING NO. F-5 SAMPLE NO. 11
DEPTH (ft.): 24.5-26.5
...g. No.:

Mechanical Analysis Data

...eve	Size, mm	Percent finer
.375 inches	9.53	100.0
4	4.760	99.2
10	2.000	97.9
...	0.420	29.0
...	0.149	3.1
200	0.074	1.8

Fractional Components

% + 3 in. = 0.0 % GRAVEL = 0.8 % SAND = 97.4
% FINES = 1.8

D85= 1.24 D60= 0.756 D50= 0.632
D30= 0.4290 D15= 0.26675 D10= 0.23578
Cc = 1.0328 Cu = 3.2063

LABORATORY TEST DATA

PROJECT: MN Industrial Containment Facility - Rosemount, MN   DATE: 7-1-90

REPORTED TO: Environmental Engineering & Management, Ltd   JOB NO.: 4220 90-590.05

Boring No.	F-5	F-5	P-1	P-2
Sample No. / Sample Designation	27	28	Composite 10 & 11	Composite 8 & 9
Depth (ft)	64½-65½	65½-66½	20-26	15-21
Type of Sample	SB	SB	SB	SB
Soil Classification (ASTM:D2487)	Lean Clay (CL)	Sand w/a little gravel, fine to medium grained (SP)	Clayey Sand w/a little gravel (SC)	Sand w/a littl gravel, mediun grained (SP)
In-Place Moisture Content (%)				
Moisture-Density Relation of Soil (ASTM:D698)				
Max. Dry Density (PCF)				
Optimum Moisture Content (%)				
Permeability Test				
Trial No.	1	1	1	1
Type of Test	Constant Head	Constant Head	Falling Head	Constant He
Type of Specimen	In-Situ Natural	Remolded	In-Situ Natural	Remolded
Specimen Height (inches)	1.35	1.31	1.91	3.00
Specimen Diameter (inches)	1.37	1.86	1.31	1.86
Dry Density (PCF)	118.3	114.4	132.7	119.8
Percent of Max. Density				
Moisture Content (%)	4.4	7.4	6.8	0.8
Max. Head Differential (ft)	0.9	0.3	5.0	0.3
Confining Pressure (Effective - PSI)	None	None	2.0	None
Water Temperature (°C)	18	19	22	21
Coefficient of Permeability K @ 20°C (cm/sec)	5.8×10^{-7}	1.5×10^{-4}	1.7×10^{-8}	5.4 x 10
K @ 20°C (ft/min)	1.1×10^{-6}	2.8×10^{-4}	3.3×10^{-8}	1.1 x 10
Atterberg Limits				
Liquid Limit (%)				
Plastic Limit (%)				
Plasticity Index				

twin city testing corporation

SL-26(77-8)

NOTE: POLYFELT WAS SOLD TO TENSAR CORPORATION WHO THEN BEGAN MANUFACTURING POLYFELT TS-700 UNDER A NEW PRODUCT NAME OF TENSAR TG-700. ATTACHED IS A COPY OF THE PRODUCT SPECIFICATIONS FOR TENSAR TG-700 VERIFYING THAT THE MATERIALS ARE THE SAME.

JUL-29-96 MON 15:28 USPCI LONE MOUNTAIN FAX NO. 4056973596 P.02
6-25-1996 10:09AM FROM POLY-FLEX, INC. 214 988 8331 P.2
06/21/96 FRI 16:55 FAX 334 578 6141 EVERGREEN TECH. INC. 002

Evergreen Technologies

June 21, 1996

Tensar Corporation
1210 Citizens Parkway
Morrow, GA 30260

Subj: TG700 Geotextile Certificate of Compliance

Re : Laidlaw Environmental, Lone Mountain Facility, Order # 001061, PO # 6-8097

Dear Sir/Madam:

This letter certifies that TG700, shipped FOB Evergreen, Alabama, on 6/17/96, manufactured by Evergreen Technologies, meets or exceeds the minimum requirements listed below.

PROPERTY	TEST PROCEDURE	VALUE(1)		
Weight	ASTM D 5261	8.0		oz/yd2
Thickness	ASTM D 5199	90	*	Mil
Grab Strength	ASTM D 4632	210		lbs
Grab Elongation	ASTM D 4632	50		%
Tear Strength	ASTM D 4533	80		lbs
Mullen Burst	ASTM D 3786	400		psi
Puncture Resistance	ASTM D 4833	100		lbs
A.O.S.	ASTM D 4751	.212	*	US Std Sieve
		(70)	*	mm
Permitivity	ASTM D 4491	1.3	*	1/sec
Water Permeability	ASTM D 4491	0.3	*	cm/sec
Water Flow Rate	ASTM D 4491	100	*	gpm/sq ft
U.V. Resistance (500 hours)	ASTM D 4355	70		%

(1) Values in weaker principle direction. Unless noted otherwise, these values represent minimum average roll values (i.e. test results from any sampled roll in a lot, tested in accordance with ASTM D 4759-88 shall meet or exceed the minimum values listed).

* Determined at the time of manufacturing, storage and handling conditions which differ from those found in ASTM D 4873-88 may influence these properties.

Unless noted otherwise, this certification is based on testing conducted by Evergreen Technologies Quality Assurance & Quality Control testing laboratories at the time of manufacturing. Evergreen Technologies issues this letter of certification to indicate our commitment to providing our customers with a quality product which will meet or exceed the minimum average roll values in accordance with the applicable American Society for Testing and Materials (ASTM) test method.

Sincerely,

Manoj Tyagi
QA Manager

APPENDIX 4

Leachate Withdrawal Pipes

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: RCRA Landfill Cell 15
FEATURE: Leachate Withdrawal Pipe Design
PROJECT NO.: 64.44.700

SHEET 1 OF 6
COMPUTED: MEA
CHECKED:
DATE: Mod 4/3/96

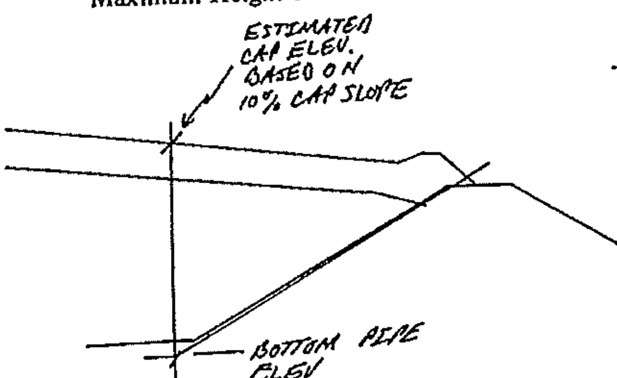
I. Evaluate the long-term strength of the HDPE pipe against failure or significant loss of cross-sectional area.

Reference Manual: "Driscopipe Systems Design", by Phillips Driscopipe, Inc., 1991 .

Design Criteria:

Pipe Diameters = 12 inches - bottom and middle pipes, 16 inches uppermost pipe

Maximum Height of Overburden (See drawing below)



SUMP NO.	BOTTOM PIPE EL.	EST. CAP EL
1	1369.15	1441.0
2	1368.15	1441.4
3	1367.15	1441.1
4	1364.15	1443.0
5	1362.15	1442.7
6	1360.15	1444.2
7	1368.63	1441.5
8	1376.64	1438.4

Maximum height of overburden occurs at Sump 6 over bottom pipe =
1444.2 - 1360.2 = 84.0 ft.

Unit weight of overburden:

Soil cover	= 125 pcf
Waste	= 120 pcf
Unit Weight Rock Cover	= Assume 110 pcf

A. Soil Pressure by components

$$P_T = P_S + P_L$$

where: P_T = Total load pressure
P_S = Static or dead load pressure
P_L = Live load pressure

Chart 30 of the above referenced design manual shows that for a height of cover over 16', the live load becomes insignificant. At 2 ft. of minimum cover, the total dead load plus live load for HS 20 highway loading is only around 1200 psf. Thus, the ultimate dead load is the governing design criteria and P_L+0.

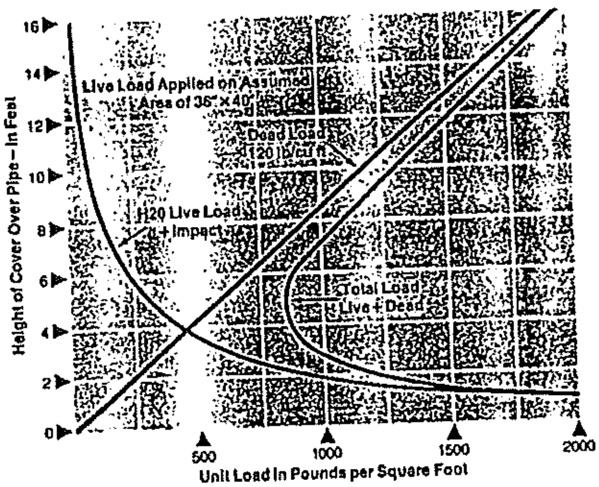
$P_T = P_S$ = height of overburden x unit weight of overburden

P_T = 10/12(110) + 5.5(125) + 77.7(120) = 10,100 psf
= 70.2 psi

HA&L ENGINEERING	CLIENT: USPCI	SHEET 2	OF 6
	PROJECT: RCRA Landfill Cell 15	COMPUTED:	MEA
	FEATURE: Leachate Withdrawal Pipe Design	CHECKED:	
	PROJECT NO.: 64.44.700	DATE:	Mod 4/3/96

Chart 30

H20 Highway Loading



Note: The H20 live load assumes two 16,000 lb. concentrated loads applied to two 18" x 20" areas, one located over the point in question, and the other located at a distance of 72" away. In this manner, a truckload of 20 tons is simulated.

Source: American Iron and Steel Institute, Washington, D.C.

B. <u>Evaluate Wall Crushing</u> (Assume SDR - 15.5)

$$(1)\quad S_A = \frac{(SDR-1)}{2} P_T$$

where; S_A = Actual ring hoop compressive stress

$$(2)\quad Safety \quad Factor \quad \frac{CYS}{S_A}$$

where; CYS = Compressive yield stress
CYS = 1500 psi

Solving the two equations (1) and (2) simultaneously, determine the SDR which would allow for a safety factor of 2.

$$2 = \frac{CYS}{SA} => S_A = \frac{1500}{2} = 750 \ psi$$

$$SDR = \frac{2S_A}{P_t} + 1 = \frac{2(750)}{70.2} + 1 = 22 \leftarrow Not \ a \ limiting \quad factor$$

Actual Safety Factor is: CYS/SA

Where:

S_A = (15.5 - 1)70.2/2 = 509 psi

SF = 1500/509 = 2.9 OK

C. Evaluate Wall Buckling

Wall buckling takes into consideration the soil strain around the pipe. Since, the lower part of the pipe is in a washed gravel which has a lower soil strain than the soil to be placed around the pipe up the slope, determine P_T with 1.25 less feet of overburden. For example, the design manual suggests an E' value of 3000 psi for manufactured rock, which as provided below is sufficiently higher than the clay or clay/sand soil mixture will have.

$$P_T = 10/12(110) + 5.5(125) + 76.5(120) = 9{,}959 \text{ psf} = 69.2 \text{ psi}$$

Safety Factor for wall buckling:

$$SF = \frac{P_{CB}}{P_T}$$

where; P_{CB} = critical buckling soil pressure at the top of the pipe, psi.

$$P_{CB} = 0.8\sqrt{E' x\ P_C}$$

where; E' = Soil modulus in psi calculated as the ratio of the vertical soil pressure to the vertical soil strain (e_s) at a specified density.

P_C = Hydrostatic critical collapse pressure.

According to testing performed by Applied Geotechnical Engineering Consultants (AGEC) on on-site clay soils, AGEC recommends that a soil strain (e_s) of between 2.65% and 3.3% be used with a load of 9,959 psf. Use 3.3% to be conservative. The clay soil should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698 (Standard Proctor Density). The results of the testing performed by AGEC are included in their July 21, 1994 and July 12, 1994 letters attached.

Therefore, use a soil strain (e_s) = 3.3%.

e_s = 3.3 percent = 0.033

$E' = P_T/e_s = 9{,}959/0.033 = 301{,}788 \text{ psf} = 2{,}096 \text{ psi}$

$$P_C = \frac{2.32\,E}{(SDR)^3}$$

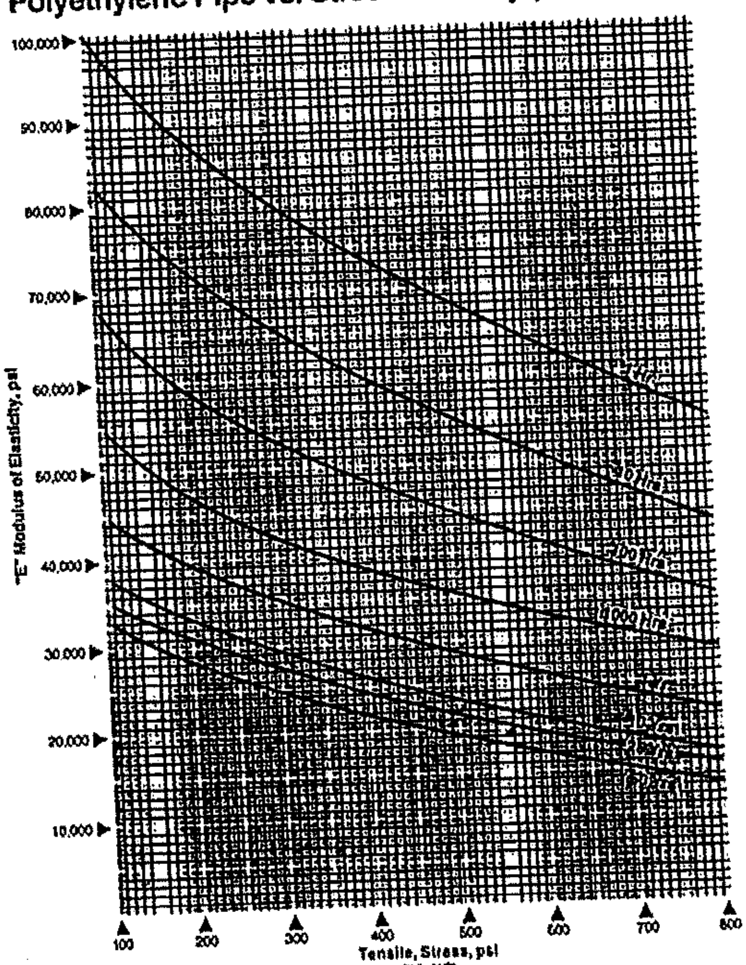
where; E = polyethylene modulus of elasticity solving for SDR;

$$SDR = \sqrt[3]{\frac{2.32E}{P_C}}$$

E is determined from Chart 25 assuming a 50-year period and stress in the pipe wall S_A

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.

Determine actual safety factor for SDR-15.5 against wall buckling.

$P_t = 69.2$ psi

$S_A = (SDR\text{-}1)P_T/2 = (15.5\text{-}1)69.2/2 = 502$ psi

$E = 19{,}100$ psi from Chart 25

$P_C = 2.32E/(SDR)^3 = 2.32(19{,}100)/(15.5)^3 = 11.90$ psi

$E' = 2{,}096$ psi (above)

$P_{CB} = 0.8\ ((E'xP_c))^{1/2} = 0.8\ ((2{,}096)(11.9))^{1/2} = 126.3$ psi

$SF_{(SDR\text{-}15.5)} = P_{CB}/P_T = 126.3/69.2 = 1.8$

Check using a mixture of clay and sand soil at a 50:50 ratio:

Again, according to testing performed by Applied Geotechnical Engineering Consultants (AGEC) on the on-site clay and sand soils, AGEC would recommend a soil strain (e_s) of 2.9% be used with a load of 9,959 psf. Note: that this value assumes a wetted condition. Therefore, the actual value would probably be less than this value and the analysis should be conservative. The clay/sand soil mixture should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698 (Standard Proctor Density). The results of the testing performed by AGEC are included in their July 21, 1994 and July 12, 1994 letters attached.

Therefore, use a soil strain (e_s) = 2.9%.

$e_s = 2.9$ percent $= 0.029$

$E' = P_T/e_s = 9{,}959/0.029 = 343{,}414$ psf $= 2{,}385$ psi

Determine actual safety factor for SDR-15.5 against wall buckling.

$P_t = 69.2$ psi

$S_A = (SDR\text{-}1)P_T/2 = (15.5\text{-}1)69.2/2 = 502$ psi

$E = 19{,}100$ psi from Chart 25

$P_C = 2.32E/(SDR)^3 = 2.32(19{,}100)/(15.5)^3 = 11.90$ psi

$P_{CB} = 0.8\ ((E'xP_c))^{1/2} = 0.8\ ((2{,}385)(11.9))^{1/2} = 134.8$ psi

$SF_{(SDR\text{-}15.5)} = P_{CB}/P_T = 134.8/69.2 = 1.95$ OK

D. Evaluate Ring Deflection Using SDR = 15.5

According to the design manual, "design by ring deflection comprises of a calculation of vertical soil strain to ensure it will be less than the allowable ring deflection of the pipe."

The design manual gives an allowable ring deflection, for SDR-15.5 pipe, of 3.9 percent.

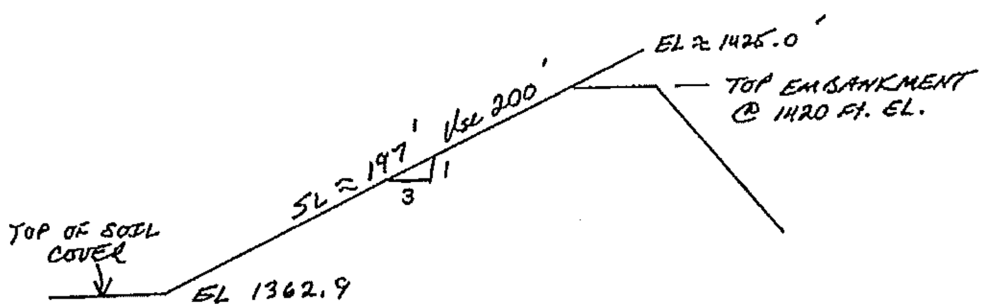
The soil strain (e_s), as defined earlier is 2.9 percent.

Since the soil strain is less than the allowable ring deflection ($2.9 < 3.9$), the pipe is adequately protected against ring deflection.

II. Check the required length of HDPE pipe to allow for contraction/expansion due to thermal changes.

A. Differential Pipe Length Due to Temperature Changes

Check uppermost pipe since that will be the pipe exposed to major temperature differentials. The middle and bottom pipes will be backfilled and therefore not exposed to extreme temperature fluctuations.



Assume maximum $\Delta T = 100° - 10° = 90°$

$$\Delta L = (\alpha)(\Delta T)(L)$$

where; α = coefficient of thermal expansion
= 1.2×10^{-4} in/in/°F

L = pipe length in feet

$$\Delta L = (1.2\times10^{-4})(90°F)(200.1')(12\ ^{in}/ft) = 25.9 \text{ in.} = 2.2 \text{ ft.}$$

AGEC
Applied Geotechnical Engineering Consultants, Inc.

July 21, 1994

RECEIVED
JUL 26 1994

HA&L Engineering
6771 South 900 East
Midvale, Utah 84047-1436

Attention: Marv Allen

Subject: Clay/Driscopipe Compression
Lone Mountain Facility
USPCI Waynoka, Oklahoma
Project No. 24292A

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. conducted laboratory tests on samples of lean clay and mixtures of lean clay with sand to measure the vertical strain when loaded from 200 to 9,250 pounds per square foot. The tests were conducted to assist in the design of the leachate withdrawal pipes.

The laboratory tests were conducted in one-dimensional consolidometers on remolded samples that were submerged during testing. A letter summarizing our test results was submitted on July 12, 1994.

Subsequent to our original testing, we visited with Dr. Reynold Watkins of Utah State University with respect to the procedures developed by Dr. Watkins on buried flexible pipe design. The standard design charts indicate the vertical stress-strain data for typical trench backfill from actual tests. The chart indicates that the values do not apply for clay soils.

Due to the fact that the backfill for the USPCI facility is clay soil, Dr. Watkins was asked to recommend a procedure to determine the strain which should be used in design. Dr. Watkins indicated that a conservative approach would be to conduct one-dimensional consolidation tests and incorporate the amount of strain measured up to the design load. He also indicated that the lateral restraint is conservative with the one-dimensional consolidation, due to the fact that as the flexible pipe is compressed, the pipe will push into the adjacent soil. With this in mind, Dr. Watkins recommended that a realistic strain for our analysis would be to use one-half of the one-dimensional strain.

Additional Testing

In review of the actual field conditions, the clay backfill around the pipe will not be submerged. With this condition, additional testing was conducted to determine the stress-strain relationship in a one-dimensional consolidometer with the sample out of water. The tests indicate the following amounts of strain when loaded from 200 to 9,250 pounds per square foot.

90% Compaction	95% Compaction
~~14~~ percent 14.8	~~4½~~ percent 5.3 ← From attached graph @ load of 9459 lbs/ft².

Test results are attached.

Recommendations

2.65 to 3.3 as per Jim Nordquist for clay backfill @ 95% compaction and 9959 lb/ft². 4/4/96

Based on our understanding of the procedure used for designing buried flexible pipe, we recommend that a strain ranging from ~~2-1/4~~ to 3 percent be utilized. This value ranges from 1/2 of the unwetted compression to 1/2 of the average between the wetted and the unwetted conditions.

For these strain values to apply, the material would need to be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698.

If you have any questions, or if we can be of further service, please call.

Sincerely,

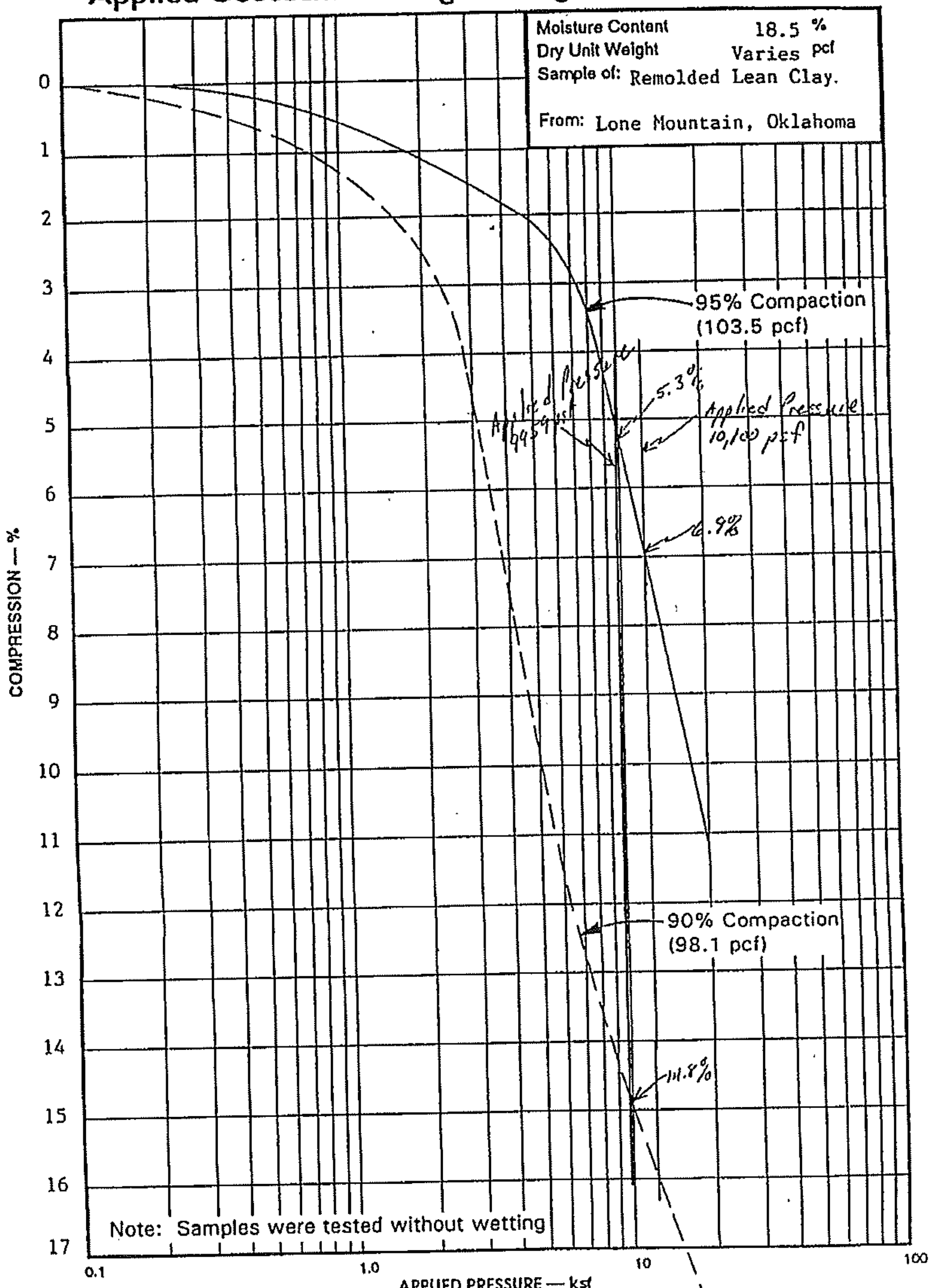
APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

James E Nordquist

James E. Nordquist, P.E.

JEN/cs
enclosure

Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 1

Applied Geotechnical Engineering Consultants, Inc.

RECEIVED JUL 14 1994

July 12, 1994

HA&L Engineering
6771 South 900 East
Midvale, Utah 84047-1436

According to Jim Nordquist, soil strain values presented herein are based on wetted soil conditions. Telecommunication - 4/4/96

Attention: Marv Allen

Subject: Clay/Clay-Sand Mixture Compression
Lone Mountain Facility
USPCI, Waynoka, Oklahoma
Project No. 24292A

Gentlemen:

Applied Geotechnical Engineering Consultants, Inc. was requested to conduct laboratory tests on samples of lean clay and mixtures of lean clay with sand to determine the strain between 200 to 9,250 pounds per square foot. We understand that a strain of less than 3.9 percent is needed for backfill around the leachate withdrawl pipes.

Testing

A sample of Lone Mountain clay was submitted to our laboratory and tested to determine Atterberg Limits, percent finer than the number 200 sieve, moisture/density relationship and consolidation. The consolidation tests were conducted on the clay sample remolded to 90, 95 and 101 percent of the maximum dry density as determined by ASTM D-698. The amount of strain measured from these tests was found to exceed the strain needed for the facility. Results of the testing is shown on Figure 4.

In order to reduce the amount of strain using material that will hold itself together, the on-site clay soil was mixed with sand similar to the sand that was previously obtained and tested from the Lone Mountain area. A mixture of 50 percent sand and 50 percent lean clay was tested for moisture/density relationship and consolidation. The consolidation samples were remolded to 92 and 97 percent of the maximum dry density as determined by ASTM D-698. The amount of strain measured with this mixture exceeded the amount of strain desired in the design. Results of the testing is shown on Figure 3.

A mixture of 75 percent sand and 25 percent clay was then tested for compressibility when remolded. Samples were remolded to 90 and 95 percent of the maximum dry density with results as shown on Figure 2.

The tests indicate the following amount of strain.

From attached graphs
@ 9959 lbs/ft^2
@ 95% Compaction

Strain from 200 to ~~9250~~ 10,013 pounds per square foot

Mixture Ratio Clay/Sand	Percent Fines	Strain, 90% Compaction	Strain, 95% Compaction	
100:0	93%	13	7½	7.8%
50:50	55%	9	5	5.8%
25:75	35%	6	2	2.7%

Summary

Based on the tests conducted, in order to maintain strain below or equal to 3½ percent when loaded from 200 to 9,250 pounds per square foot, we recommend that the material contain from 25 to 42 percent fines. The fines need to be clay and the mixture should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698.

If you have any questions, or if we can be of further service, please call.

Sincerely,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

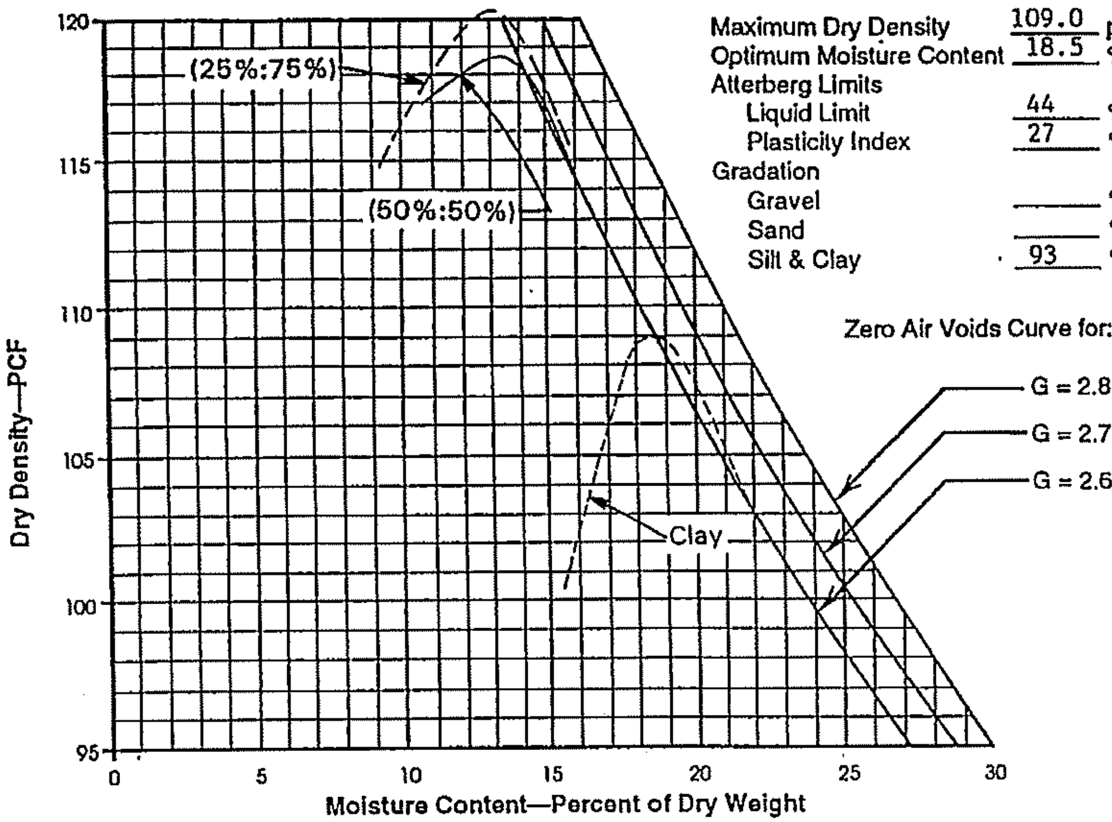
James E. Nordquist

James E. Nordquist, P.E.

JEN/cs

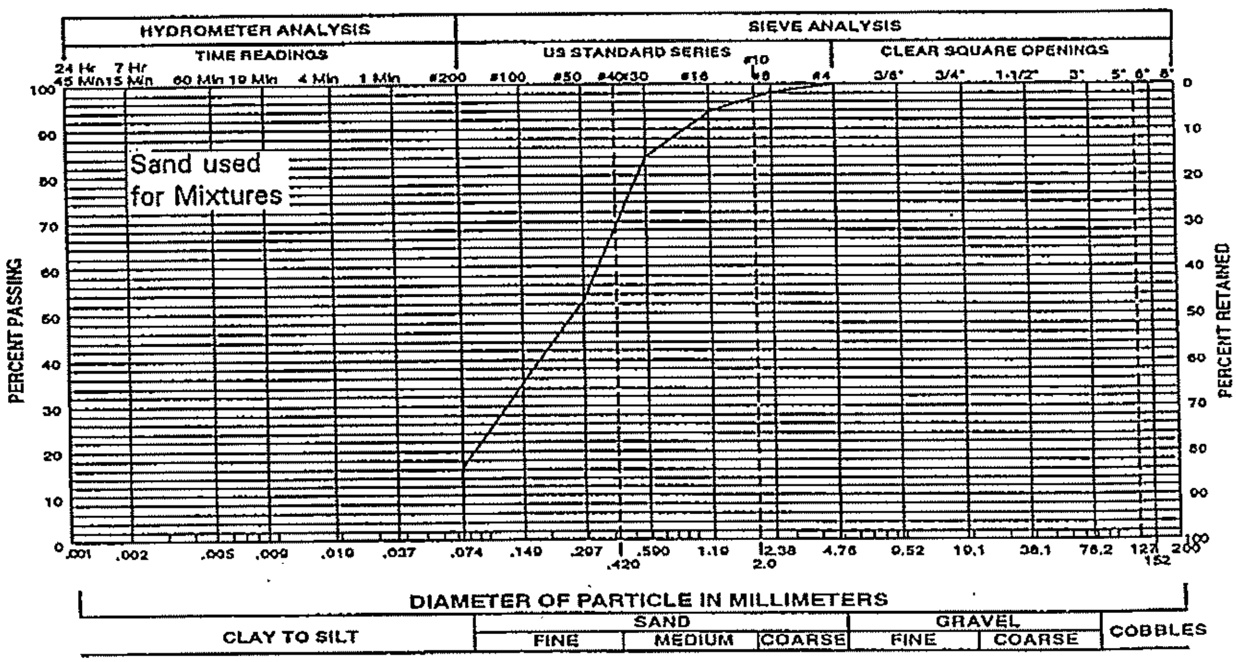
Applied Geotechnical Engineering Consultants, Inc.

Maximum Dry Density	109.0	pcf
Optimum Moisture Content	18.5	%
Atterberg Limits		
Liquid Limit	44	%
Plasticity Index	27	%
Gradation		
Gravel		%
Sand		%
Silt & Clay	93	%



Compaction Test Procedure ASTM D-698

Sample of: Clay or Clay/Sand Mixture From: Lone Mountain, Oklahoma



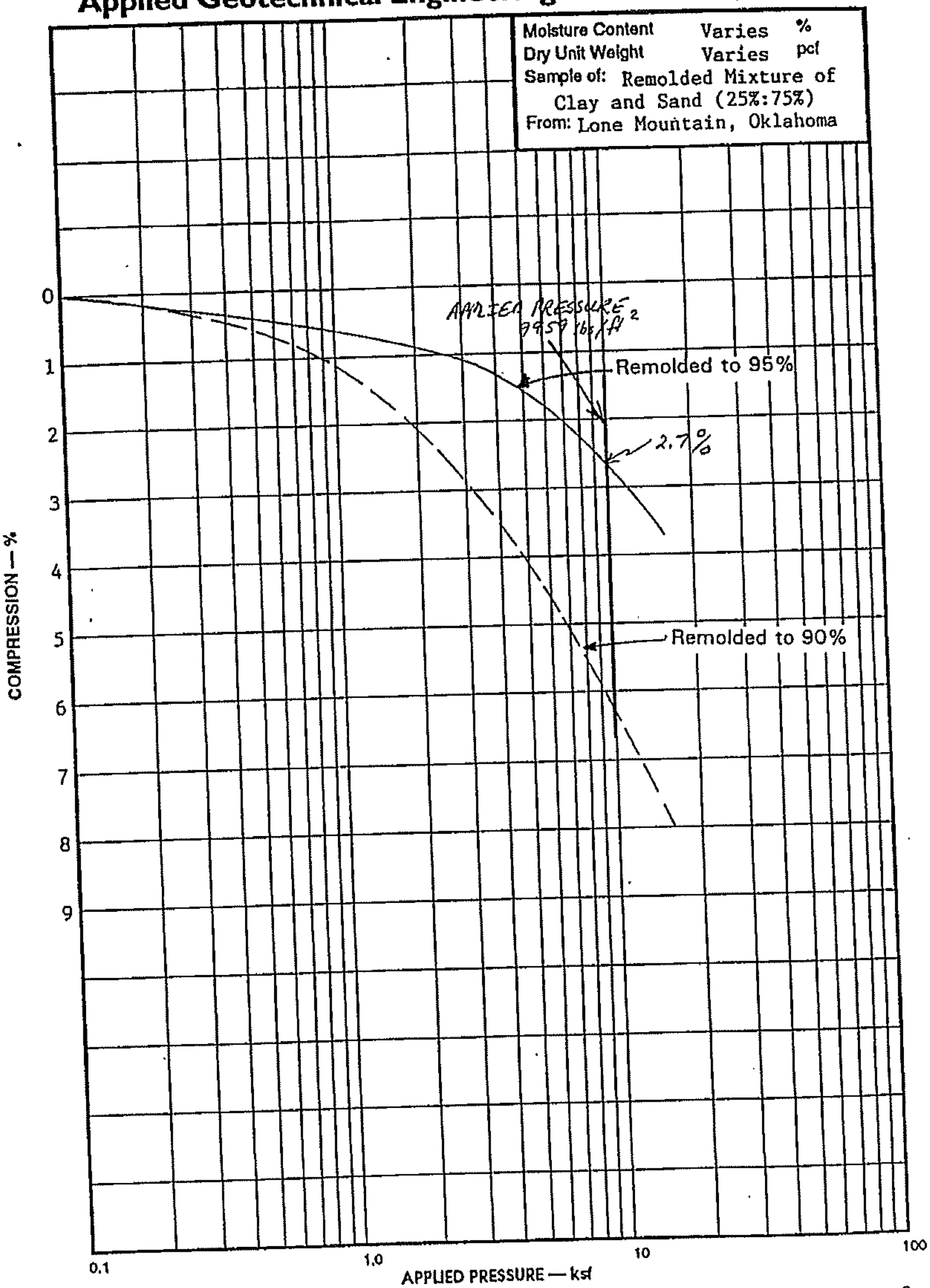
GRADATION & COMPACTION TEST RESULTS

Project No. 24292A

Figure 1

Wet Condition according to Jim Nordquist

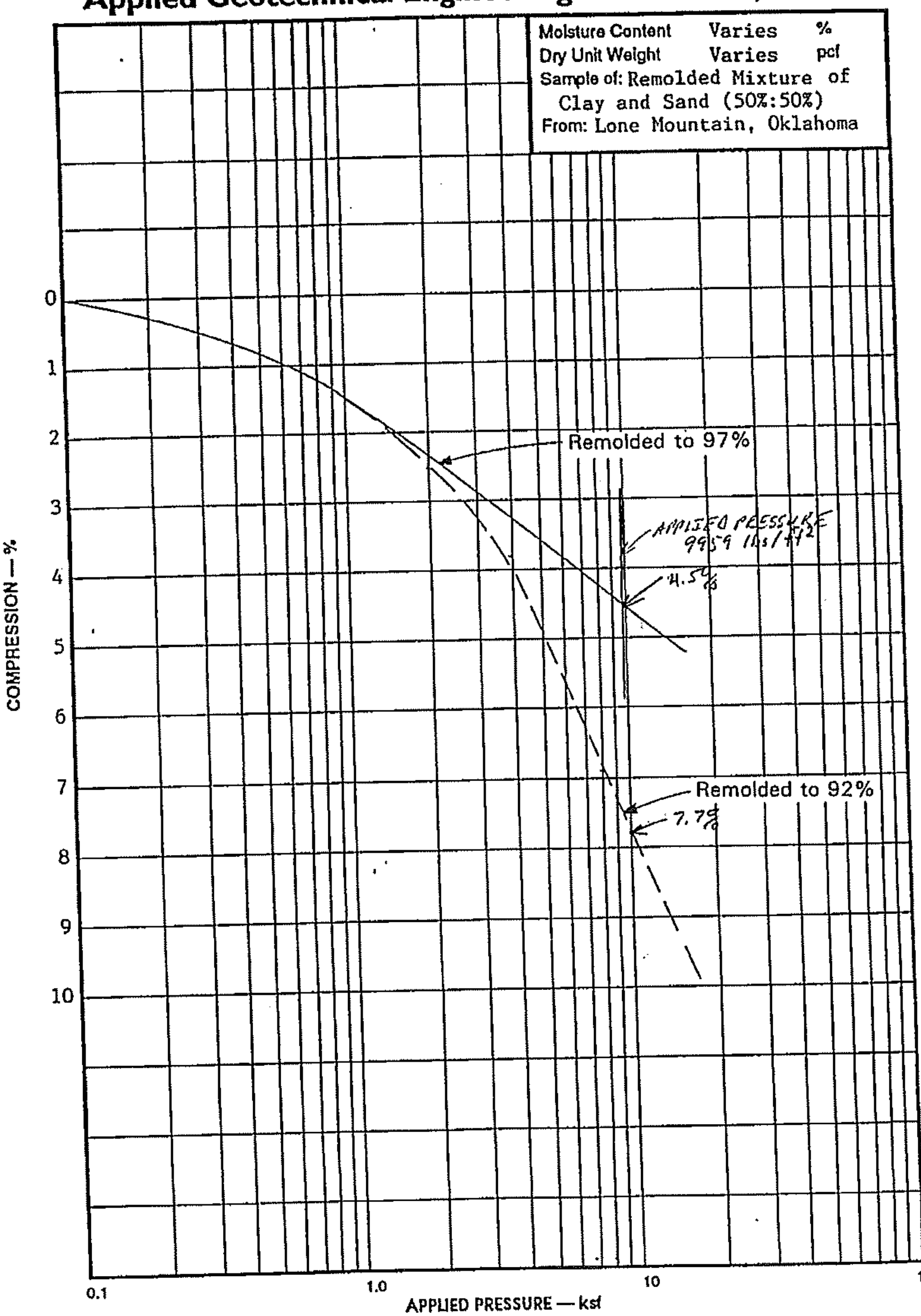
Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 2

Wet Condition
According to Tim Hardy

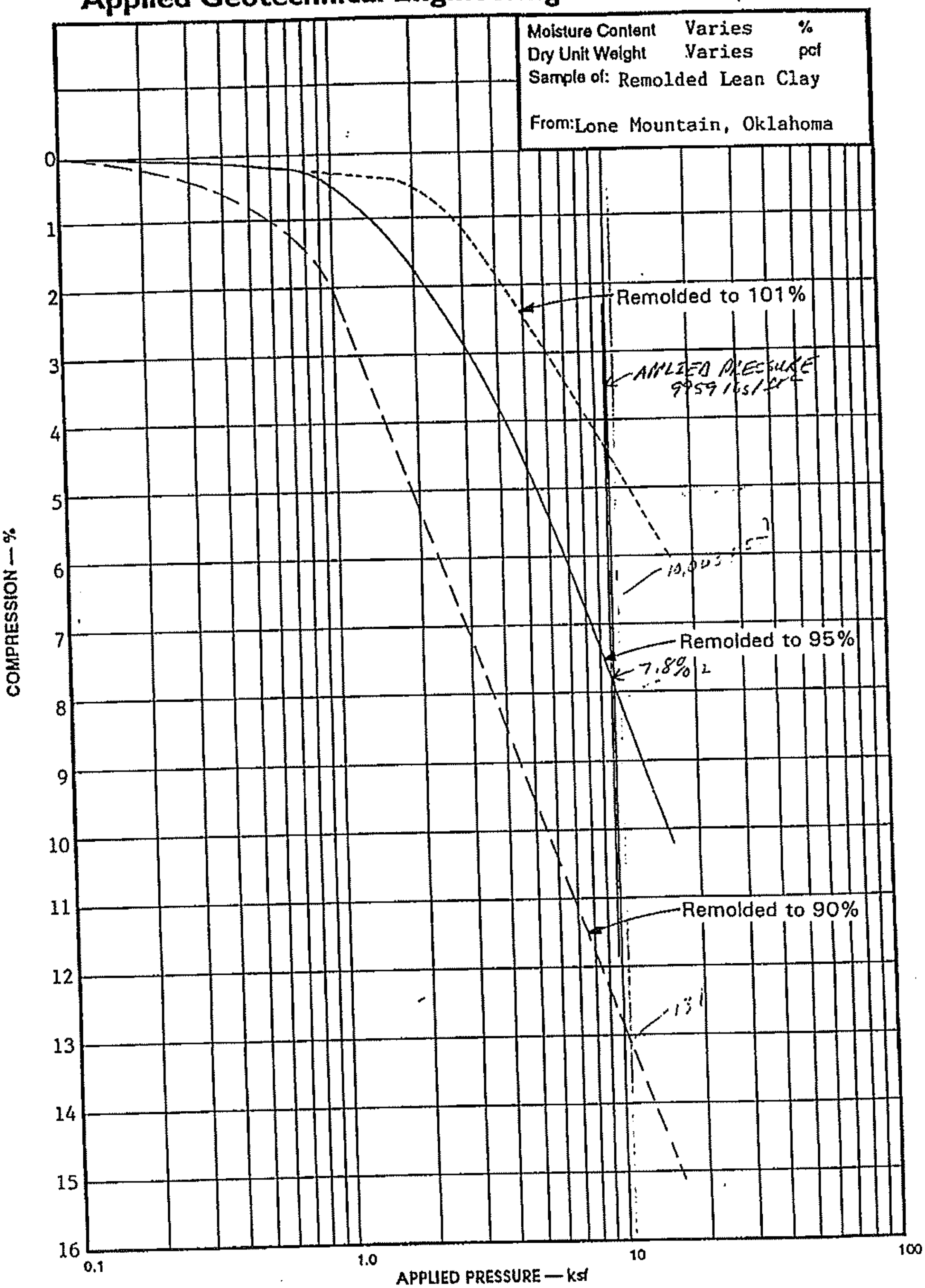
Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 3

Wet Condition
According to Jim Hedquist

Applied Geotechnical Engineering Consultants, Inc.



Project No. 24292A **CONSOLIDATION TEST RESULTS** Figure 4

APPENDIX 5

Uppermost Sump Capacities

HA&L ENGINEERING

CLIENT: USPCI \ Laidlaw
PROJECT: Cell 15 Design
FEATURE: Calculations of Sump Volume
PROJECT NO. 64 44.700

SHEET 1 OF
COMPUTED JDB
CHECKED
DATE 5/22/96

- Calculate the stage-capacity relationships for each of the uppermost sumps in Cell 15. Drawings of each sump are presented in Exhibit A

A) Uppermost Sump No. 1

1) @ Elev 1371.86 (low point)

Surface Area = 0.0 ft^2

Volume = 0.0 ft^3

2) @ Elev 1372.11 ⇒ d = 0.25'

Surface Area: $(0.25/0.01)\left[(0.25/0.01) + 0.25(3)\right] = 643.8\ ft^2$

Total Volume = $\frac{1}{2}(0.0 + 643.75\ ft^2)(0.25\ ft) = 80.5\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area ft^2	Volume ft^3
0.25	64	0.083	0.0143	0.9
0.5'	75'	0.125	0.038	2.9
				3.8 ft^3

Rock Volume = 80.5 − 3.8 = 76.7

Rock porosity = 0.32

Net Volume = 3.8 + 76.7 (0.32) = 28.3 ft^3

3) @ Elev. 1372.48 ⇒ d = 0.62'

Surface Area = $(0.62/0.01)\left[(0.62/0.01) + 0.62(3)\right] \doteq 3959.3\ ft^2$

Total Volume = $\frac{1}{2}(643.8 + 3959.3)(0.62 - 0.25) + 80.5$

= 932.1 ft^3

HA&L ENGINEERING

CLIENT: USPCI \ Laidlaw
PROJECT: Cell 15 Design
FEATURE: Calculations of sump Volume
PROJECT NO.: 64.44.700

SHEET: 2 OF
COMPUTED: JDK
CHECKED:
DATE: 5/22/96

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	120	0.25	0.049	5.9
0.25	248	0.125	0.025	6.1
0.5	36	0.5	0.196	7.1
0.5	150	0.25	0.098	14.7
				33.8 ft^3

Rock Volume = 932.1 − 33.8 = 898.3 ft^3

Rock porosity = 0.32

Net Volume = (898.3)(0.32) + 33.8 = 321.3 ft^3

4) @ Elev 1372.68 ⇒ d = 0.82′

$$SA = (0.82/0.01)\left[(0.82/0.01) + 3(0.82)\right] - \tfrac{1}{2}(10.5)(7.5) = 6875.8\ ft^2$$

$$\text{Total Volume} = \tfrac{1}{2}(6875.8 + 3959.3)(0.82 - 0.62) + 932.1$$

$$= 2{,}015.6\ ft^3$$

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	320′	0.25	0.049	15.7
0.25	336	0.125	0.025	8.2
0.5	96	0.50	0.196	18.8
0.5	145′	0.25	0.098	14.2
				56.9

Rock Volume = 2,015.6 − 56.9 = 1958.7 ft^3

Net Volume = 1958.7 (0.32) + 56.9 = 683.7 ft^3

HA&L ENGINEERING

CLIENT: USPCI\Laidlaw
PROJECT: Lone Mt. Cell 15 Design
FEATURE: Calculations of Sump Volume
PROJECT NO. 64.44 700

SHEET 3 OF
COMPUTED JDB
CHECKED
DATE 5/22/96

5) @ Elev. 1372.93 $\Rightarrow d = 1.07'$

$SA = (107)[107 + 1.07(3)] - \frac{1}{2}(35)(35) - \frac{1}{2}(10.5)(7.5)$

$= 11{,}130.1 \ ft^2$

Total Volume $= \frac{1}{2}(11{,}130.1 + 6875.8)(1.07 - 0.82) + 2015.6$

$= 4{,}266.3 \ ft^3$

Pipe Volume:

Dia. (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	656	0.25	0.049	32.2
0.25	430	0.125	0.025	10.6
0.50	171	0.50	0.196	33.6
0.50	106	0.25	0.098	10.4
0.33	15	≈ 0.10	0.022	0.3
				87.1 ft^3

Rock Volume = 4266.3 − 87.1 = 4,179.2

Net Volume = (4,179.2)(0.32) + 87.1 = 1,424.4

6) @ Elev. 1373.36 $\Rightarrow d = 1.5$

$SA = 150(150 + 1.5(3)) - \frac{1}{2}(77.2)^2 - \frac{1}{2}(10.5)(9.5) - \frac{1}{2}(43)^2$

$= 19{,}220.7 \ ft^2$

Total Vol $= \frac{1}{2}(11{,}130.1 + 19{,}220.7)(1.5 - 1.07) + 4{,}266.3$

$= 10{,}791.7 \ ft^3$

Pipe Volume:

Dia ft	Length (ft)	Ave depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	1420	0.25	0.049	69.7
0.33	18	0.33	0.086	1.5
0.50	277	0.50	0.196	54.4
[illegible]	[illegible]			125.6 ft^3

HA&L ENGINEERING

CLIENT: USPCI \ Laidlaw
PROJECT: Lone Mt Cell 15 Design
FEATURE: Calculations of Sump Volumes
PROJECT NO.: 64.44.700

SHEET: 4 OF
COMPUTED: JDB
CHECKED:
DATE: 5/22/96

Rock Volume = 10,791.7 − 125.6 = 10,666.1 ft^3

Net Volume = $(10,666.1)(0.32) + 125.6 = 3538.8\ ft^3$

7) Summary

depth (ft)	Net Vol (ft^3)
0.0	0.0
0.25	28.3
0.62	321.3
0.82	683.7
1.07	1,424.4
1.50	3,538.8

B) Uppermost Sump No. 2

1) @ Elev. 1370.86 ⇒ d = 0.0 (low point)

Surface Area (SA) = 0.0 ft^2

Volume = 0.0 ft^3

2) @ Elev. 1371.11 ⇒ d = 0.25

Surface Area = 643.8 ft^2

Total Volume = 80.5 ft^3

Net Volume = 28.3 ft^3

Same as d = 0.25' on Sump No 1. (pg. 1)

3) @ Elev. 1371.48 ⇒ d = 0.62

Surface Area = 3959.3 ft^2

Total Volume = 932.1 ft^3

Net Volume = 321.3 ft^3

Same as d = 0.62' on Sump No 1. (pg. 1 & 2)

HA&L ENGINEERING

CLIENT: USPCI \ Laidlaw. Lone Mt
PROJECT: Cell 15 Design
FEATURE: Sump Volume
PROJECT NO. 14.44.700

SHEET 5 OF
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DATE 5/22/96

4) Sump 2 @ Elev 1371.97 $d = 1.11$

$$SA = 107.5\left(1.11/0.01 + 1.11(3)\right) - \tfrac{1}{2}(5.5)^2 = 12{,}275.4 \text{ ft}^2$$

$$\text{Total Vol} = \tfrac{1}{2}(12{,}275.4 + 3959.3)(1.11 - 0.62) + 932.1$$

$$= 4909.6 \text{ ft}^3$$

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	696	0.25	0.049	34.2
0.25	486	0.125	0.025	11.9
0.33	39	≈ 0.1	0.022	0.9
0.5	183	0.5	0.196	35.9
0.5	93	≈ 0.3	0.123	11.4
				94.3

Rock Volume = $4909.6 - 94.3 = 4815.3 \text{ ft}^3$

Net Volume = $4815.3\,(0.32) + 94.3 = 1{,}635.2$

5) Sump 2 @ Elev 1372.36 ⇒ $d = 1.5$

$$SA = 145.8\left(150 + 3(1.5)\right) - \tfrac{1}{2}(41.5)(46.3) - \tfrac{1}{2}(36.4)(40.2)$$

$$= 20{,}833.7 \text{ ft}^2$$

$$\text{Total Volume} = \tfrac{1}{2}(20{,}833.7 + 12{,}275.4)(1.5 - 1.11) + 4909.6$$

$$= 11{,}365.9 \text{ ft}^3$$

HA&L ENGINEERING

CLIENT: USPCI \ Laidlaw Lone Mt
PROJECT: Cell 15 Design
FEATURE: Sump Volume
PROJECT NO. 64 44.700

SHEET 6 OF
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DATE 5/22/96

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	1520	0.25	0.049	74.6
0.33	24	0.33	0.087	2.1
0.5	184	0.50	0.196	36.1
				112.8 ft^3

Rock Volume = 11,365.9 − 112.8 = 11,253.1 ft^3

Net Volume = (0.32)(11,253.1) + 112.8 = 3713.8 ft^3

6) Summary of Stage-Capacity for Sump 2

depth (ft)	Net Vol (ft^3)
0	0.0
0.25	26.3
0.62	321.3
1.11	1,635.2
1.50	3,713.8

HA&L ENGINEERING

CLIENT LESI
PROJECT Lone Mt Cell 15 Design
FEATURE Revised Sump Volume - Uppermost Sump No 3
PROJECT NO. 64 44.700

SHEET 7 OF
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Rev. DATE 10/1/97

Stage - Volume for Uppermost Sump No. 3

1) Sump 3 @ Elev. 1369.82 ⇒ d = 0.0, low point

SA = 0.0
Volume = 0.0

2) Sump 3 @ Elev. 1370.07 ⇒ d = 0.25

$SA = (25.0)[12.5 + 0.25(3)] = 331.3 \ ft^2$
$Vol = \frac{1}{2}(331.3 + 0)(0.25) = 41.4 \ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	26	0.125	0.025	0.3
0.5	62.5	0.125	0.0768	4.8
				5.1 ft^3

Rock Volume = 41.4 - 5.1 = 36.3 ft^3

Net Volume = 0.32(36.3) + 5.1 = 16.7 ft^3

3) Sump 3 @ Elev. 1370.39 ⇒ d = 0.57

$SA = (57)[28.5 + 0.57(3)] = 1,722.0 \ ft^2$

$Total\ Vol = \frac{1}{2}(1,722.0 + 331.3)(.57 - .125) + 41.4 = 369.9 \ ft^3$

Pipe Volume:

L=156 total

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	44	0.25	.0491	2.2
0.25	112	0.125	.0245	2.7
0.50	142.5	0.28	.1131	16.1
				21.0

Rock Volume = 369.9 - 21.0 = 348.9 ft^3

Net Volume = 0.32(348.9) + 21.0 = 132.6 ft^3

HA&L ENGINEERING

CLIENT: IESI
PROJECT: Lone Mt Cell 15 Design
FEATURE: Revised Sump Volume - (Uppermost) Sump 3
PROJECT NO.: 64.44.700

SHEET: 8 OF
COMPUTED: JDB
CHECKED:
Rev. DATE: 10/1/97

4) Sump 3 @ Elev. 1370.60 ⇒ d = 0.78

$SA = (.78)[39 + 3(0.78)] - \frac{1}{2}(19.5)(10) = 3{,}127.0 \ ft^2$

$Volume = \frac{1}{2}(3{,}127.0 + 1{,}722.0)(.78 - .57) + 369.9 = 879.0 \ ft^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
248	0.25	124	0.25	.0491	6.1
	0.25	124	0.125	.0245	3.0
170	0.50	70	0.50	.1693	11.9
	0.50	75	0.25	.0982	7.4
	0.50	25	0.38	.1601	4.0
					32.4 ft^3

Rock Volume = 879.0 - 32.4 = 846.6

Net Volume = 0.32 (846.6) + 32.4 = 303.3 ft^3

5) Sump 3 @ Elev. 1370.83 ⇒ d = 1.01 ft^3

$SA = (1.01)[50.5 + 3(1.01)] - [\frac{1}{2}(43.1)(22.9)] - [\frac{1}{2}(22.5)(12.0)]$

$= 4{,}778 \ ft^2$

$Volume = \frac{1}{2}(4778 + 3127)(1.01 - 0.78) + 879.0 = 1{,}788 \ ft^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
431	0.25	233	0.25	.0491	11.4
	0.25	198	0.125	.0245	4.9
177	0.50	128	0.50	0.1693	21.7
	0.50	20	0.31	0.125	2.5
	0.50	29	0.37	0.156	4.5
					45.0 ft^3

Rock Volume = 1788 - 45 = 1743 ft^3

Net Volume = 0.32(1743) + 45 = 603 ft^3

HA&L ENGINEERING

CLIENT: LEST
PROJECT: Lower MH Cell 15 Design
FEATURE: Revised Sump Volume - Uppermost Sump
PROJECT NO. 64.44.700

SHEET 9 OF
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REV. DATE 10/2/97

6) Sump 3 @ Elev 1371.32 ⇒ d = 1.50

$$SA = (150)[75 + 3(1.5)] - \tfrac{1}{2}(37.5)(70.6) - \tfrac{1}{2}(48.4)(91.1)$$

$$= 8{,}397 \text{ ft}^2$$

$$\text{Volume} = \tfrac{1}{2}(8{,}397 + 4778)(1.5 - 1.01) + 1{,}788 = 5{,}016 \text{ ft}^3$$

Pipe Volume

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	545	0.25	.0491	26.8
0.25	13	0.125	.0245	0.3
0.50	177	0.50	.1693	30.0
				57.1

$$\text{Rock Volume} = 5{,}016 - 57 = 4959 \text{ ft}^3$$

$$\text{Net Volume} = 0.32(4959) + 57 = 1644 \text{ ft}^3$$

7) Summary of Sump 3 Stage-Capacity

Depth (ft)	Net Volume (ft^3)
0.00	0.0
0.25	16.7
0.57	133
0.78	303
1.01	603
1.50	1644

HANSEN ALLEN & LUCE INC	CLIENT USPCT \ Laidlaw	SHEET 10 OF	
	PROJECT Cell 15 Design	COMPUTED JDB	
	FEATURE Calculations of Sump Volume	CHECKED	
	PROJECT NO. 64 44.700	DATE 5/23/96	

D) Uppermost Sump No. 4

1) Sump 4 @ Elev. 1366.82 ⇒ d = 0.0 low point

SA = 0.0

Volume = 0.0

2) Sump 4 @ Elev. 1367.22 ⇒ d = 0.4

$SA = 854.4 \ ft^2$

$Net\ Vol = 61.0 \ ft^3$

same as Sump No. 3, pg. 7

Total Vol = 170.9

3) Sump 4 @ Elev. 1367.53 ⇒ d = 0.71

$SA = (71.9)[35.5 + (3)(0.71)] = 2705.6 \ ft^2$

$Total\ Vol = \tfrac{1}{2}(2705.6 + 854.4)(0.71 - 0.4) + 170.9$

$= 722.7 \ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft²)	Volume (ft³)
0.25	108'	0.25	0.049	5.3
0.25	142'	0.125	0.025	3.5
0.50	55'	0.50	0.196	10.8
0.50	125'	0.25	0.098	12.3
				31.9

250' 1" = 180' ℓ h ℓ

$Rock\ Volume = 722.7 - 31.9 = 690.8 \ ft^3$

$Net\ Volume = (690.8)(0.32) + 31.9 = 253.0$

4) Sump 4 @ Elev. 1367.89 ⇒ d = 1.07

$SA = 107.5[53.5 + 3(1.07)] - \tfrac{1}{2}(19.1)(36.6)$

$= 5{,}746.8 \ ft^2$

$Total\ Vol = \tfrac{1}{2}(5{,}746.8 + 2705.6)(1.07 - 0.71) + 722.7$

$= 2{,}244.1 \ ft^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
532 total 320 full	0.25	320	0.25	0.0491	15.7
	0.25	212	0.125	0.0245	5.4
	0.33	22'	≈ 0.08	0.016	0.4
full	0.50	142	0.5	0.196	27.9
	0.50	67	≈ 0.3	0.123	8.2
					57.6 ft^3

Rock Volume = 2,244.1 - 57.6 = 2186.5 ft^3

Net Volume = (0.32)(2186.5) + 57.6 = 757.3 ft^3

5) Sump 4 @ Elev. 1368.32 ⇒ d = 1.5'

$SA = (149.9)[75 + 3(1.5)] - \frac{1}{2}(38.3)(76.4) - \frac{1}{2}(22.5)(41.6)$

$= 9986.0\ ft^2$

Total Vol $= \frac{1}{2}(9986.0 + 5746.8)(1.5 - 1.07) + 2,244.1$

$= 5626.7\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	640'	0.25	0.0491	31.4
0.33	24	0.33	0.087	2.1
0.5	209	0.5	0.196	41.0
				74.5

Rock Volume = 5626.7 - 74.5 = 5552.2 ft^3

Net Volume = (0.32)(5552.2) + 74.5 = 1,851.2 ft^3

HANSEN ALLEN & LUCE INC	CLIENT USPCI\ Laidlaw Lone Mt	SHEET 12 OF
	PROJECT Cell 15 Design	COMPUTED JDB
	FEATURE Calculations for Sumps Volumes	CHECKED
	PROJECT NO. 64.44.700	DATE 5/23/96

c) Summary for Sump 4

depth (ft)	Net Volume (ft^3)
0.00	0.0
0.40	61.0
0.71	253.0
1.07	757.3
1.50	1851.2

E) Uppermost Sump No. 5

1) Sump 5 @ Elev. 1364.86' $\Rightarrow d = 0.0$. (lowest point)

$SA = 0.0\ ft^2$

Volume $= 0.0\ ft^3$

2) Sump 5 @ Elev. 1365.22 $\Rightarrow d = 0.36$

$SA = \frac{1}{2}(36 + 3(0.36))(13.8 + 36.0) = 923.3\ ft^2$

Total Vol $= \frac{1}{2}(923.3 + 0)(0.36) = 166.2\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	6	0.25	0.049	0.3
0.25	66	0.125	0.025	1.6
0.5	86	0.18	0.064	5.5
				7.4

Rock Volume $= 166.2 - 7.4 = 158.8\ ft^3$

Net Volume $= (0.32)(158.8) + 7.4 = 58.2\ ft^3$

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CLIENT USPCI \ Laidlaw Lone Mt.
PROJECT Cell 15 Design
FEATURE Calculation of Sump Volume
PROJECT NO. 64.44.700

SHEET 13 OF
COMPUTED ADR
CHECKED
DATE 5/28/96

3) Sump 5 @ Elev. 1365.58 ⇒ d = 0.72′

$SA = \frac{1}{2}(72 + 3(0.72))(27.6 + 72.0) = 3693.2 \text{ ft}^2$

$\text{Total Vol.} = \frac{1}{2}(3693.2 + 923.3)(0.72 - 0.36) + 166.2$

$= 997.2 \text{ ft}^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft²)	Volume (ft³)
350 total	0.25	147	0.25	0.049	7.2
	0.25	203	0.125	0.025	5.0
172′ total	0.50	53	0.50	0.196	10.4
	0.50	119	0.25	0.098	11.7
					34.3 ft³

Rock Volume = 997.2 − 34.3 = 962.9 ft³

Net Volume = (0.32)(962.9) + 34.3 = 342.4 ft³

4) Sump 5 @ Elev 1365.94 ⇒ d = 1.08′

$SA = \frac{1}{2}(107.5 + 3(1.08))(41.4 + 107.5) = 8244.6 \text{ ft}^2$

$\text{Total Volume} = \frac{1}{2}(8244.6 + 3693.2)(1.08 - 0.72) + 997.2$

$= 3146.0 \text{ ft}^3$

Pipe Volume.

	Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
774′	0.25′	454′	0.25	0.049	22.3
	0.25′	335′	0.125	0.025	8.2
219′	0.50′	136′	0.5	0.196	26.7
	0.50′	83′	0.3	0.123	10.2
	0.33′	20′	0.1	0.022	0.4
					67.8

Rock Volume = 3146.0 − 67.8 = 3078.2 ft³

Net Volume = (0.32)(3078.2) + 67.8 = 1,052.8 ft³

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PROJECT Cell 15, Design
FEATURE Calculations of Sump Volumes
PROJECT NO. 14.44 700

SHEET 14 OF
COMPUTED JDR
CHECKED
DATE 5/27/96

5) Sump 5 @ Elev. 1366.36 ⇒ d = 1.5

SA = ½(150.0)(57.5 + 150) − ½(45)(150 − 107.5) = 14,606.3

Total Vol = ½(14,606.3 + 8244.6)(1.5 − 1.08) + 3146.0

= 7,944.7 ft^3

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft^3)
0.25	986'	0.25	0.049	48.4
0.33	23'	0.33	0.087	2.0
0.50	219'	0.50	0.196	43.0
				93.4

Rock Volume = 7,944.7 − 93.4 = 7851.3 ft^3

Net Volume = (0.32)(7851.3) + 93.4 = 2605.8 ft^3

6) Sump 5 Summary of Stage-Capacity

depth (ft)	Net Volume (ft^3)
0.0	0.0
0.36	58.2
0.72	342.4
1.08	1,052.8
1.50	2,605.8

F) Uppermost Sump No. 6

1) Sump 6 @ Elev. 1362.87 (low point)

$SA = 0.0\ ft^2$

$Volume = 0.0\ ft^3$

2) Sump 6 @ Elev 1363.06′ ⇒ $d = 0.19'$

$SA = 19.0\left[0.19/0.0057 + 3(0.19)\right] = 644.2\ ft^2$

$\text{Total Volume} = \tfrac{1}{2}(644.2 + 0)(0.19) = 62.2\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
0.25	60	0.1	0.018	1.1
0.50	38	0.1	0.028	1.1
				2.2 ft³

$\text{Rock Volume} = 62.2 - 2.2 = 60.0$

$\text{Net Volume} = 60(0.32) + 2.2 = 21.4\ ft^3$

3) Sump 6 @ Elev 1363.19 ⇒ $d = 0.32'$

$SA = 32\left[0.32/0.0057 + 3(0.32)\right] - \tfrac{1}{2}(13)(11) = 1751.6\ ft^2$

$\text{Total Volume} = \tfrac{1}{2}(1751.6 + 644.2)(0.32 - 0.19) + 62.2$

$= 217.9\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
0.25	10	0.25	0.0491	0.5
0.25	152	0.12	0.0232	3.5
0.5	64	0.16	0.0542	3.5
				7.5

HANSEN ALLEN & LUCE INC

CLIENT: USPCI \ Laidlaw Lone Mt. SHEET 16 OF
PROJECT: Cell 15 Design COMPUTED JDB
FEATURE: Sump Volumes CHECKED
PROJECT NO. 64.44.700 DATE 5/24/96

Rock Volume = 217.9 − 7.5 = 210.4

Net Volume = (0.32)(210.4) + 7.5 = 74.8 ft^3

4) Sump 6 @ Elev. 1363.52 ⇒ d = 0.65

$SA = \frac{1}{2}(86.6)(112) - \frac{1}{2}(14)(12) = 4765.6\ ft^2$

Total Vol. = $\frac{1}{2}(4765.6 + 1751.6)(0.65 - 0.32) + 217.9$

= 1293.2 ft^3

Pipe Volume:

Dia (ft)	Length (ft)	Ave. Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	204	0.25	0.049	10.0
0.25	185	0.125	0.025	4.5
0.5	56	0.5	0.196	11.0
0.5	128	0.25	0.098	12.6
				38.1

184' [illegible]

Rock Volume = 1293.2 − 38.1 = 1255.1 ft^3

Net Volume = (0.32)(1255.1) + 38.1 = 439.7 ft^3

5) Sump 6 @ Elev. 1363.85 ⇒ d = 0.97

$SA = \frac{1}{2}(114.5)(134) - \frac{1}{2}(14)(13) = 7580.5\ ft^2$

Total Volume = $\frac{1}{2}(7580.5 + 4765.6)(0.97 - 0.65) + 1293.2$

= 3268.6 ft^3

Pipe Volume:

	Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
696	0.25	450	0.25	0.049	22.1
	0.25	247	0.125	0.025	6.1
244'	0.50	144	0.50	0.196	28.3
	0.50	100	0.25	0.098	9.8
					66.3

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CLIENT: USPCI \ Laidlaw Lone Mt
PROJECT: Cell 15 Design
FEATURE: Sump Volumes
PROJECT NO.: 64.44.700

COMPUTED: JDB
CHECKED:
DATE: 5/24/96

Rock Volume = 3268.6 − 66.3 = 3202.3

Net Volume = (0.32)(3202.3) + 66.3 = 1091.0 ft^3

6) Sump 6 @ Elev 1364.37 ⇒ d = 1.5'

$SA = \frac{1}{2}(161.7)(194) - \frac{1}{2}(96)(52) - \frac{1}{2}(17)(15) = 13{,}009.4\ ft^2$

Total Volume = $\frac{1}{2}(13{,}009.4 + 7580.5)(1.5 - 0.97) + 3268.6$

= 8724.7 ft^3

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	965	0.25	0.049	48.4
0.33	18	0.33	0.087	1.6
0.50	240	0.50	0.196	47.1
				97.1 ft^3

Rock Volume = 8724.7 − 97.1 = 8627.6 ft^3

Net Volume = 8627.6 (0.32) + 97.1 = 2857.9 ft^3

7) Sump 6 Summary

depth (ft)	Net Volume (ft^3)
0.00	0.0
0.17	21.4
0.32	74.8
0.65	439.7
0.97	1091.0
1.50	2857.9

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CLIENT: USPCI \ Laidlaw Lone Mt
PROJECT: Cell 15 Design
FEATURE: Sump Volumes
PROJECT NO.: 64.44.700

SHEET 18 OF
COMPUTED: JDB
CHECKED:
DATE: 6/6/96

6) Uppermost Sump No. 7

1) Sump 7 @ Elev. 1371.37 ⇒ d = 0.0 (low point)

SA = 0.0
Volume = 0.0

2) Sump 7 @ 1371.56 ⇒ d = 0.19

$SA = \frac{1}{2}(35.1)(19.5) + \frac{1}{2}(39.1)(20.0) = 733.2 \text{ ft}^2$

$\text{Total Volume} = \frac{1}{2}(733.2 + 0.0)(0.19) = 69.7 \text{ ft}^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave. Depth (ft)	Ave Area (ft²)	Volume (ft³)
0.25	69.0	0.10	0.0183	1.3
0.50	69.0	0.10	0.0280	1.8
				3.1

Rock Volume = 69.7 − 3.1 = 66.6 ft³

Net Volume = (0.32)(66.6) + 3.1 = 24.4

3) Sump 7 @ Elev 1371.88 ⇒ d = 0.51

$SA = \frac{1}{2}(143)(35.9) + \frac{1}{2}(72.8)(36.2) = 2471.9 \text{ ft}^2$

$\text{Volume} = \frac{1}{2}(2471.9 + 733.2)(0.51 - 0.19) + 69.7 = 582.5 \text{ ft}^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
226' total	0.25	147	0.125	0.0245	3.6
	0.25	79	0.25	0.049	3.9
128' total	0.50	128	0.25	0.098	12.6
					20.1

Rock Volume = 582.5 − 20.1 = 562.4 ft³

Net Volume = (0.32)(562.4) + 20.1 = 200.1 ft³

HANSEN ALLEN & LUCE INC	CLIENT USPCI \ Laidlaw	SHEET 19 OF
	PROJECT Cell 15 Design	COMPUTED JDB
	FEATURE Sump Volumes	CHECKED
	PROJECT NO. 64.44.700	DATE 6/6/96

4) Sump 7 @ Elev. 1372.2 $\Rightarrow d = 0.83$

$SA = \frac{1}{2}(13.6)(52.1) + \frac{1}{2}(105.8)(52.9) = 5236.7\ ft^2$

Total Volume $= \frac{1}{2}(5236.7 + 2471.9)(0.83 - 0.51) + 582.5$

$= 1815.9\ ft^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft²)	Volume (ft³)
480'	0.25	215	0.125	0.0245	5.3
	0.25	265	0.25	0.049	13.0
192'	0.50	100	0.25	0.098	9.8
	0.50	92'	0.50	0.196	18.1
					46.2

Rock Volume $= 1815.9 - 46.2 = 1769.7\ ft^3$

Net Volume $= (0.32)(1769.7) + 46.2 = 612.5\ ft^3$

5) Sump 7 @ Elev. 1372.52 $\Rightarrow d = 1.15'$

$SA = \frac{1}{2}(122.8)(68.0) + \frac{1}{2}(138.4)(69.0) = 8950.0\ ft^2$

Total Volume $= \frac{1}{2}(8950.0 + 5236.7)(1.15 - 0.83) + 1815.9\ ft^3$

$= 4085.8\ ft^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
840	0.25	288	0.125	0.0245	7.1
	0.25	552	0.25	0.049	27.1
255'	0.50	100	0.25	0.098	9.8
	0.50	155'	0.50	0.196	30.4
					74.4 ft³

Rock Volume $= 4085.8 - 74.4 = 4011.4\ ft^3$

Net Volume $= (0.32)(4011.4) + 74.4 = 1358.0$

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CLIENT: USPCI \ Laidlaw
PROJECT: Cell 15 Design
FEATURE: Sump Volumes
PROJECT NO. 1444.700

SHEET 20 OF
COMPUTED JDB
CHECKED
DATE 6/6/96

6) Sump 7 @ Elev. 1372.87 $d = 1.5$ (maximum)

$SA = \frac{1}{2}(154.9)(85.9) + \frac{1}{2}(175.5)(87.0) = 14{,}287.2\ ft^2$

$\text{Total Volume} = \frac{1}{2}(14{,}287.2 + 8950.0)(1.5 - 1.15) + 4085.8$

$= 8{,}152.3\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	868	0.25	0.049	42.6
0.33	29'	0.33	0.087	2.5
0.50	118	0.50	0.196	38.9
				84.0

Rock Volume = 8152.3 - 84.0 = 8068.3 ft^3

Net Volume = (0.32)(8068.3) + 84.0 = 2665.9 ft^3

7) Sump 7 Summary of Stage-Capacity

depth (ft)	Net Volume (ft^3)
0.0	0.0
0.19	24.4
0.51	200.1
0.83	612.5
1.15	1358.0
1.50	2665.9

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CLIENT: USPCI \ Laidlaw Lone Mt | SHEET: 21 OF
PROJECT: Cell 15 Design | COMPUTED: JMB
FEATURE: Sump Volumes | CHECKED:
PROJECT NO.: 14.44.700 | DATE: 5/29/96

H) Uppermost Sump No. 8

1) Sump 8 @ Elev. 1378.50 ⇒ d = 0.0 (low point)

SA = 0.0
Volume = 0.0

2) Sump 8 @ Elev 1378.66 ⇒ d = 0.16

$SA = \frac{1}{2}(26)(17) + \frac{1}{2}(20)(30) = 547\ ft^2$

Total Volume $= \frac{1}{2}(547)(0.16) = 43.8\ ft^3$

Pipe Volume:

Dia (ft)	Length (ft)	Ave Depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	45	0.08	0.0135	0.6
0.5	83	0.08	0.0203	1.7
				2.3 ft^3

Rock Volume $= 43.8 - 2.3 = 41.5\ ft^3$

Net Volume $= (0.32)(41.5) + 2.3 = 15.6\ ft^3$

3) Sump 8 @ Elev. 1379.0 ⇒ d = 0.50

$SA = \frac{1}{2}(54.2)(39.8) + \frac{1}{2}(42.7)(63.9) = 2442.8\ ft^2$

Total Volume $= \frac{1}{2}(2442.8 + 547.0)(0.5 - 0.16) + 43.8$
$= 552.1\ ft^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft^2)	Volume (ft^3)
238 total {	0.25	141	0.125	0.025	3.5
	0.25	97	0.25	0.049	4.8
	0.50	151	0.25	0.098	14.8
					23.1

Rock Volume $= 552.1 - 23.1 = 529\ ft^3$

Net Volume $= (0.32)(529.0) + 23.1 = 192.4\ ft^3$

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CLIENT: USPCI\Laidlaw Lone Mt
PROJECT: Cell 15 Design
FEATURE: Sump Volumes
PROJECT NO.: 14.44 700

SHEET 22 OF
COMPUTED: JDB
CHECKED:
DATE: 5/22/96

4) Sump 8 @ Elev. 1379.33 $\Rightarrow d = 0.83$ ft

$SA = \frac{1}{2}(82.9)(60.4) + \frac{1}{2}(64.5)(96.4) = 5612.5 \text{ ft}^2$

$\text{Total Volume} = \frac{1}{2}(5612.5 + 2442.8)(0.83 - 0.5) + 552.1$

$= 1881.2 \text{ ft}^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
550'	0.25	248'	0.125	0.0245	6.1
	0.25	302'	0.25	0.049	14.8
218	0.50	100'	0.25	0.098	9.8
	0.50	118'	0.50	0.196	23.2
					53.9 ft³

$\text{Rock Volume} = 1881.2 - 53.9 = 1827.3 \text{ ft}^3$

$\text{Net Volume} = (0.32)(1827.3) + 53.9 = 638.6 \text{ ft}^3$

5) Sump 8 @ Elev. 1379.70 $\Rightarrow d = 1.20'$

$SA = \frac{1}{2}(84.5)(102.3) + \frac{1}{2}(54.0)(132.1) = 7{,}888.9 \text{ ft}^2$

$\text{Total Volume} = \frac{1}{2}(7{,}888.9 + 5612.5)(1.20 - 0.83) + 1881.2$

$= 4379.0 \text{ ft}^3$

Pipe Volume:

	Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft²)	Volume (ft³)
778'	0.25	155'	0.125	0.0245	3.8
	0.25	623	0.25	0.049	30.6
226'	0.50	36	≈ 0.3	0.123	4.4
	0.50	190	0.50	0.196	37.3
					76.1

$\text{Rock Volume} = 4379.0 - 76.1 = 4302.9 \text{ ft}^3$

$\text{Net Volume} = (0.32)(4302.9) + 76.1 = 1453.0 \text{ ft}^3$

6) Sump 8 @ Elev. 1380.0 ⇒ d = 1.50'

$$SA = \tfrac{1}{2}(87.0)(136.8) + \tfrac{1}{2}(45.1)(162.0) = 9603.9 \text{ ft}^2$$

$$\text{Total Volume} = \tfrac{1}{2}(9603.9 + 7888.9)(1.50 - 1.20) + 4379.0$$

$$= 7002.9 \text{ ft}^3$$

Pipe Volume:

Dia (ft)	Length (ft)	Ave depth (ft)	Ave Area (ft^2)	Volume (ft^3)
0.25	776'	0.25	0.0491	38.2
0.50	226'	0.50	0.196	44.4
				82.6 ft^3

$$\text{Rock Volume} = 7002.9 - 82.6 = 6920.3 \text{ ft}^3$$

$$\text{Net Volume} = (0.32)(6920.3) + 82.6 = 2297.1 \text{ ft}^3$$

7) Sump 8 Stage-Capacity Summary

depth (ft)	Net Volume (ft^3)
0.0	0.0
0.16	15.6
0.50	192.4
0.83	638.6
1.20	1453.0
1.50	2297.1

APPENDIX 6

Bottom Leachate Detection and Removal System and Action Leakage Rate

I. Area Tributary to Each Bottom Sump

The area tributary to the bottom sumps is summarized in attached Table No. 3.

II. Transmissivity of the Drainage Net

Maximum height of cover at closure near the perimeter of the bottom sump:

$$= \text{elev of cap} - \text{sump elev} = \Delta h$$

The normal pressure on the drainage net assuming 125 lbs/ft³ unit weight for the soil covers, 110 lbs/ft³ for erosion protection, and 120 lbs/ft³ for the waste material deposited in the landfill cells:

$$= 5.5(125) + 0.8(110) + (\Delta h - 6.3)(120) = \text{Loading in lbs/ft}^2$$

The normal pressure on the drainage net is as follows:

Table 1

Sump No.	Top of Cap Elev. Above Sump ft	Sump Elev @ Perimeter ft	Δh ft.	Normal Pressure lbs/ft²
1	1442.7	1370.58	72.12	8674
2	1443.1	1369.58	73.52	8842
3	1443.3	1368.75	74.55	8966
4	1444.7	1365.75	78.95	9494
5	1444.4	1363.58	80.82	9718
6	1445.9	1361.50	84.40	10,148
7	1446.3	1370.30	76.00	9140
8	1442.2	1377.22	64.98	7817

SLT GS-228 drainage net is evaluated herein. However, any drainage net meeting similar flow characteristics or which has been reevaluated to demonstrate its acceptability may be used. SLT GS-228 drainage net was evaluated under a 6,500 lbs/ft² and a 10,000 lbs/ft² normal stress using the boundary conditions of HDPE liner on the bottom, SLT GS-228 drainage net, an 8 ounce non-woven geotextile fabric on top, and a soil layer above the geotextile fabric, and was tested on 1%, 2%, and 5% gradients. The results of the testing are summarized in Table 2 below.

Table 2

Floor Gradient (percent)	6,500 lbs/ft²		10,000 lbs/ft²	
	Transmissivity m²/sec	Transmissivity ft²/min	Transmissivity m²/sec	Transmissivity ft²/min
1	5.45×10^{-3}	3.52	3.15×10^{-3}	2.03
2	4.50×10^{-3}	2.91	2.88×10^{-3}	1.86
5	3.10×10^{-3}	2.00	2.00×10^{-3}	1.29

Values for slopes other than those shown above were obtained by interpolating data provided by the drainage net manufacturer.

Table 3, summarizes the transmissivity values (θ) of the SLT GS-228 drainage net for the various slopes within Cell 15. Although as indicated above, the overburden pressure on the drainage net is generally less than 10,000 lbs/ft^2, a loading of 10,000 lbs/ft^2 was assumed in all cases. The values shown in Table 3 were interpolated (based on the floor gradient) from the data provided by the manufacturer and a safety factor of 4.2 was applied to the values listed in Table 2. Use of a safety factor of 4.2 is described in the calculations associated with the uppermost and middle leachate collection system.

III. Capacity of Bottom Drainage System into Bottom Sumps

Find: The capacity of the SLT GS-228 drainage net around the perimeter of the bottom sumps.

1. Equation governing the flow in the net is:

$$Q = \beta \cdot \theta \cdot i$$

Where:
θ = Transmissivity of the net,
i = Gradient of the net
Q = Flow rate through the net, and
β = Width perpendicular to the flow.

2. From the equation, and give that β = 1 ft., the flow rate "q" per unit flow width is calculated as follows:

$$q = \beta \cdot \theta \cdot i$$

Based on the transmissivity values obtained from the SLT test data, and using the above equation, the flow rate per unit width and action leakage rate (ALR) for the various floor slopes are as shown in Table 3. The ALR is determined from the following equation:

$$ALR = q \cdot \beta \ / \ area \ / \ safety\ factor$$

As shown in Table 3, the limiting ALR for the drainage net is 391 gallons per acre per day.

Table 3

Sump Area	Major Floor Slope (%)	Area Tributary to Bottom Sumps (Acres)	Loading (lbs/ft^2)	Transmissivity @ 10,000 lbs/ft^2 and S.F = 4.2 θ (ft^2/min)	Flow Rate per Unit Width q (gpd/ft)	Perimeter Length around β Sump (ft.)	ALR$_{allow}$ S.F. = 2 (gpad)
1	1.44	4.54	8,674	0.47	72.9	75.0	602
2	1.44	3.06	8,842	0.47	72.9	75.0	893
3	2.26	3.15	8,966	0.43	104.7	73.7	1,225
4	2.26	2.62	9,494	0.43	104.7	73.7	1,472
5	2.87	1.27	9,718	0.40	123.7	38.4	1,869
	1.46	1.27		0.46	72.3	37.5	1,068
6	1.17	3.20	10,184	0.48	60.5	52.6	497
7	1.04	3.88	9,140	0.48	53.8	56.5	391
8	1.06	2.38	7,817	0.48	54.8	47.2	543
-	33	-	10,000	0.19	675.4		

IV. Action Leakage Rate (ALR) Based on Drainage System

The floors of each of the subcell areas are to be constructed in planes. Drainage on the floor will flow down the plane to the junction of two planes, thence along the junction line or drainage way towards the sumps. Check the ALR of the most critical drainage way in each of the sump or subcell areas. The drainage way will consist of a single layer of drainage net. The most critical drainage way in sumps 1 through 5 is down the center of each subcell area in an east-west orientation. The most critical drainage way in sumps 6 through 8 is along the junction between the northern embankment of the cell and the floor of the cell.

A. Equation governing the flow in the net is:

$$Q = \beta \cdot \theta \cdot i$$

Where:
θ = Transmissivity of the net,
i = Gradient of the net
Q = Flow rate through the net, and
β = Width perpendicular to the flow.

B. Assume a flow width perpendicular to flow (β) of 20 feet. Since the slope into the drainage way in most cases is approximately 1%, the maximum flow depth or head on the liner for sumps 1 through 5 (where the drainage way is down the center of the subcell area and the flow width would consist of 10 feet on each side of the drainage way), would be 0.1 foot (10 *0.01 = 0.1 foot). The maximum flow depth or head on the liner in the drainage way for sumps 6 through 8 (where the drainage way is at the junction of the floor with the toe of the northern cell embankment) would be 0.2 foot (20*0.01=0.2 foot). For β = 20 feet, the governing flow in the net is:

$$Q = 20 \cdot \theta \cdot i$$

C. The drainage way slope, tributary area to the drainage way, assumed transmissivity of the drainage net based on slope, the governing flow and corresponding ALR for the most critical drainage way in each subcell area is presented in Table 4. The ALR is calculated as follows:

$$ALR = Q \,/\, area \,/\, safety\ factor$$

As indicated in Table 4, the governing ALR (assuming a safety factor of 2) based on the drainage system is 178 gallons per acre per day in Subcell Area No. 1.

Table 4

Subcell Area	Drainage Way Slope (%)	Tributary Area (acres)	Loading (lbs/ft²)	Transmissivity @ 10,000 lbs/ft² θ (ft²/min)	Flow $Q = 20 \cdot \theta \cdot i$ (ft³/min)	(gpd)	ALR S.F.=2 (gpad)
1	1	2.91	8,674	0.48	0.096	1,034	178
2	1	1.96	8,842	0.48	0.096	1,034	264
3	2	1.70	8,966	0.44	0.176	1,896	558
4	2	1.22	9,494	0.44	0.176	1,896	777
5	1	1.63	9,718	0.48	0.096	1,034	317
6	1	1.25	10,184	0.48	0.096	1,034	414
7	1	1.58	9,140	0.48	0.096	1,034	327
8	1	1.67	7,817	0.48	0.096	1,034	310

V. Action Leakage Rate (ALR) Based on Pumping System

The pumps should have a capacity of at least 40 gpm. This would be equivalent to a daily pumping rate if the pumps were on all day of 57,600 gpd. The largest tributary area is from Subcell No. 1 with an area of 4.54 acres. This will provide the most critical ALR for the cell. Therefore, the ALR of the system, based upon pumping would be:

$$ALR = 57{,}600 \text{ gpd} / 4.54 \text{ acres}$$

$$ALR = 12{,}687 \text{ gpad}$$

Applying a factor of safety of 2 to this figure, the ALR for Landfill Cell 15, based on the system capacity would be:

$$ALR_{allow} = ALR/2$$

$$ALR_{allow} = 12{,}687 / 2 = 6{,}344 \text{ gpad}$$

Therefore, the pump is not limiting.

VI. Action Leakage Rate Based on Operation of the Pumping System

The action leakage rate can also be limited by the operational criteria established for the pumping system, taking into consideration the storage capacity or void volume of the bottom sumps. Calculations for determination of the storage capacity or void volume of the bottom sumps are contained elsewhere in the design engineering report. The ALR (taking into consideration the sump capacities) is a function of the pumping frequency of the bottom sumps. If for example, the bottom sumps are pumped or checked daily, then the ALR for the bottom sumps would be equal to the sump capacity divided by the area tributary to the sumps. If the bottom sumps are pumped or checked weekly, then the ALR for the bottom sumps would be equal to the sump capacity divided by 7 days/week and divided by the area tributary to the sumps. The sump capacities, tributary area, and ALR based on daily pumping or checking and weekly pumping or checking are presented in Table 5 below.

Based on the information presented in Table 5, the sump capacity becomes the limiting ALR if anything other than daily pumping and checking of the sumps occurs. For daily pumping of the bottom sumps, the most limiting ALR occurs in Sump No. 1, at 173 gpad. With pumping and checking the sumps only once per week, the most limiting ALR again occurs in Sump No. 1 at 25 gpad. Thus, depending on the operational schedule for checking the bottom sumps, the ALR can vary from 25 to 173 gpad.

VII. Summary

As indicated above, depending on the operational schedule for checking the bottom sumps, the ALR based on the capacity of the bottom sumps can vary from 25 gpad for checking once a week to 173 gpad for checking daily. These are the most critical based on the various systems analyzed. Thus, the ALR for Cell 15 can vary between 25 gpad (based on checking and pumping the sumps once per week) to 173 gpad if the sumps are checked daily.

Table 5

Sump No.	Sump Capacity (gallons)	Tributary Area (acres)	ALR	
			Daily Pumping gpad	Weekly Pumping gpad
1	1575	4.54	173	25
2	1575	3.06	257	37
3	1558	3.15	247	35
4	1558	2.62	297	42
5	1617	2.54	318	45
6	1419	3.20	222	32
7	3712	3.88	478	68
8	1740	2.38	366	52

NOTE: POLYFELT WAS SOLD TO TENSAR CORPORATION WHO THEN BEGAN MANUFACTURING POLYFELT TS-700 UNDER A NEW PRODUCT NAME OF TENSAR TG-700. ATTACHED IS A COPY OF THE PRODUCT SPECIFICATIONS FOR TENSAR TG-700 VERIFYING THAT THE MATERIALS ARE THE SAME.

Evergreen Technologies

June 21, 1996

Tensar Corporation
1210 Citizens Parkway
Morrow, GA 30260

Subj: TG700 Geotextile Certificate of Compliance

Re : Laidlaw Environmental, Lone Mountain Facility, Order # 001061, PO # 6-8097

Dear Sir/Madam:

This letter certifies that TG700, shipped FOB Evergreen, Alabama, on 6/17/96, manufactured by Evergreen Technologies, meets or exceeds the minimum requirements listed below.

PROPERTY	TEST PROCEDURE	VALUE(1)	
Weight	ASTM D 5261	8.0	oz/yd2
Thickness	ASTM D 5199	90	* Mil
Grab Strength	ASTM D 4632	210	lbs
Grab Elongation	ASTM D 4632	50	%
Tear Strength	ASTM D 4533	80	lbs
Mullen Burst	ASTM D 3786	400	psi
Puncture Resistance	ASTM D 4833	100	lbs
A.O.S.	ASTM D 4751	.212	* US Std Sieve
		(70)	* mm
Permittivity	ASTM D 4491	1.3	* 1/sec
Water Permeability	ASTM D 4491	0.3	* cm/sec
Water Flow Rate	ASTM D 4491	100	* gpm/sq ft
U.V. Resistance (500 hours)	ASTM D 4355	70	%

(1) Values in weaker principle direction. Unless noted otherwise, these values represent minimum average roll values (i.e. test results from any sampled roll in a lot, tested in accordance with ASTM D 4759-88 shall meet or exceed the minimum values listed).

* Determined at the time of manufacturing, storage and handling conditions which differ from those found in ASTM D 4873-88 may influence these properties.

Unless noted otherwise, this certification is based on testing conducted by Evergreen Technologies Quality Assurance & Quality Control testing laboratories at the time of manufacturing. Evergreen Technologies issues this letter of certification to indicate our commitment to providing our customers with a quality product which will meet or exceed the minimum average roll values in accordance with the applicable American Society for Testing and Materials (ASTM) test method.

Sincerely,

Manoj Tyagi
QA Manager

APPENDIX 7

Bottom Sump Capacities

HA&L ENGINEERING	
CLIENT USPCI / Laidlaw	SHEET 1 OF 4
PROJECT Lone Mtn. Cell 15	COMPUTED JMA
FEATURE Stage Capacity - Bottom Sump No 1 & 2	CHECKED
PROJECT NO. 64.44.600	DATE 6/25/96

I - Stage Capacity - Not adjusted for Porosity or Pipes

Top of Sump = 1370.7 Sump No. 1
Note: Sumps 1 & 2 have identical bottom sump configurations. Only elevations are different by 1'
Elevations shown are for sump No. 1.

Units × 25 in² × 3² ft²/in² / 1634 units

LANDFILL CELL 15 - LONE MOUNTAIN FACILITY
STAGE CAPACITY - BOTTOM SUMPS 1 & 2

DEPTH FT	ELEV. FT.	PLANIMETER UNITS @ 15	AREA FT2	AVE. AREA FT2	UNADJUSTED VOLUME FT3
0	69.1	0	0.0		
				19.5	1.95
0.1	69.2	283	39.0		
				131.2	13.12
0.2	69.3	1623	223.5		
				301.6	60.33
0.4	69.5	2758	379.8		
				448.3	224.14
0.9	70	3753	516.8		
				534.9	267.45
1.4	70.5	4016	553.0		
				303.5	60.70
1.6	70.7	392	54.0		

II - Adjust Volumes for pipes vs. gravel

A - 4" Dia Pipes

Assume all 4" pipe in sump above 1369.2 EL
Assume 15% of 4" pipe volume between 69.2 and 69.3
" 35% " " " " " 69.3 " 69.5
" 50% " " " " " 69.5 " 70.0

Total length of pipe in sumps ≈ 157'
ID = 3.92" ⇒ area = 0.084 ft²
OD = 4.5" ⇒ area = 0.110 ft²

Volume removed from sump = 0.110(157) = 17.27 ft³
Volume added back in to sump = 0.084(157) = 13.19 ft³

HA&L ENGINEERING	CLIENT: USPCI/Laidlaw	SHEET 2 OF 4	
	PROJECT: Lone Mtn. Cell 15	COMPUTED JAA	
	FEATURE: Stage Capacity-Bottom Sumps 1 & 2	CHECKED	
	PROJECT NO. 64.44.600	DATE 6/25/96	

B - 6" Dia Pipes

Assume it lies between 69.2 and 69.7
Volume between 1369.2 ; 1369.3 - Use 25%
" " 1369.3 ; 1369.5 - Use 40%
" " 1369.5 ; 1369.7 - Use 35%

Pipe length = 2' Each way = 4'
ID = 5.771" => area = 0.182 ft^2
OD = 6.625" => area = 0.239 ft^2

Volume removed from sump = 0.239(4) = 0.96 ft^3
" added back in = 0.182(4) = 0.73 ft^3

C - 10" Dia - HDPE Pipe
Assume it lies between El - 1369.1 and 1369.98
Area @ 0.1' depth = 0.036 ft^2
" " 0.2 " = 0.097 ft^2
" " 0.4 " = 0.247 ft^2
" " full " = 0.478 ft^2

Assume $\frac{0.036}{0.478}$ = 7.5% of vol between 69.1 ; 69.2

$\frac{0.097}{0.478}$ = 20% (20-7.5) = 12.5% between 69.2 ; 69.3

$\frac{0.247}{0.478}$ = 52% (52-20) = 32% between 69.3 ; 69.5

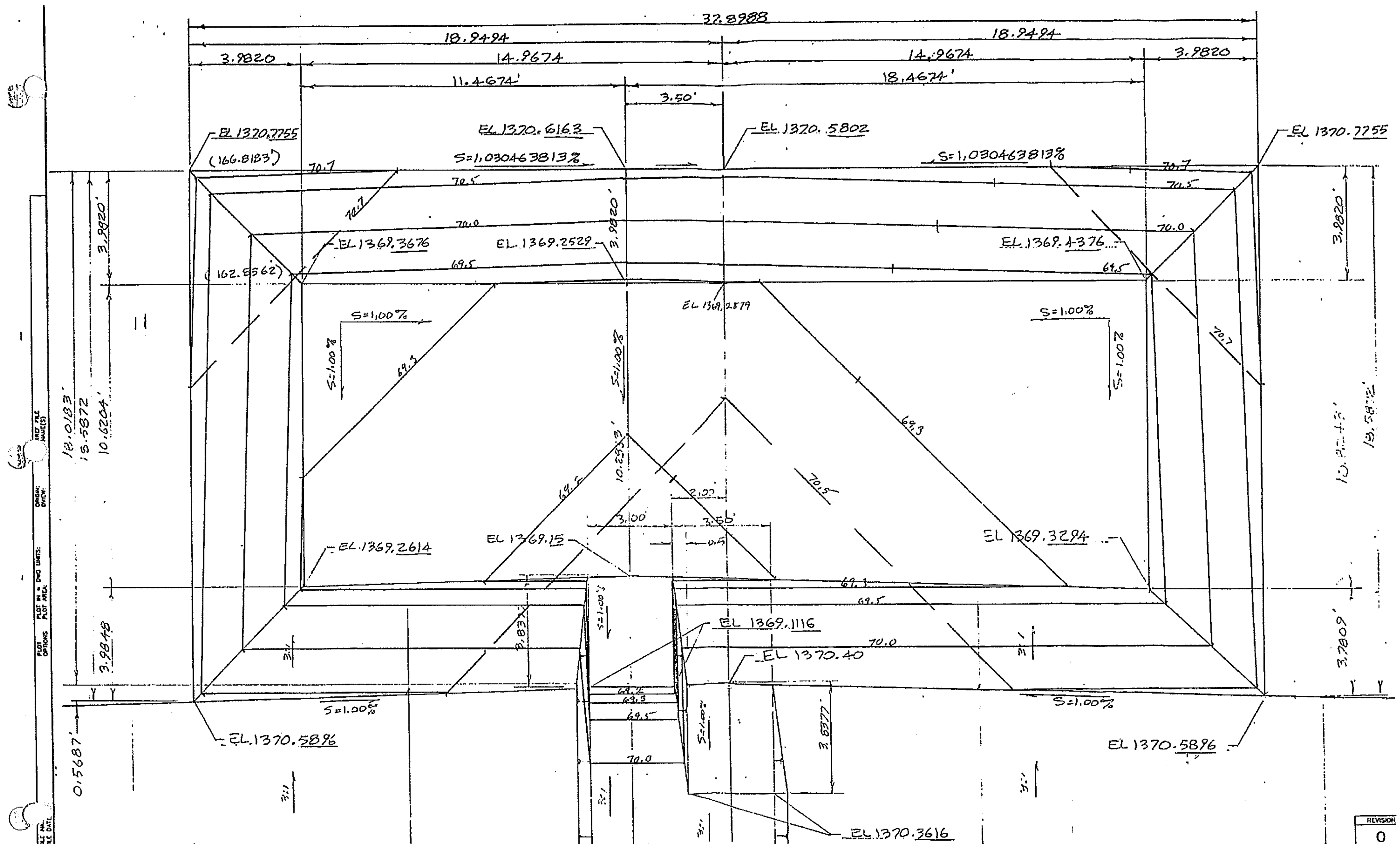
48% between 69.5 and 70.0

ID = 9.362" => area = 0.478 ft^2
OD = 10.75" => area = 0.63 ft^2
Total Length = 5.8'
Volume Out = 5.8(0.63) = 3.65 ft^3
Volume In = 5.8(0.478) = 2.77 ft^3

III - Adjust Volumes for Pipes ; Porosity
Assume porosity = 0.32 (As per testing by AGEC)

3 of 4
6/25/96

DEPTH FT	ELEV. FT	UNADJUSTED VOLUME FT3	VOLUME TAKEN OUT BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE OUT FT3	ADJUSTE VOLUME POROSITY 0.32X FT3	VOLUME ADDED BACK IN BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE IN FT3	ACCUMULATED VOLUME FT3	GALLONS
0	69.1											0.00	0
		1.95	0	0	0.27	1.68	0.54	0	0	0.21	0.75		
0.1	69.2											0.75	6
		13.12	2.59	0.24	0.46	9.83	3.15	1.98	0.18	0.35	5.66		
0.2	69.3											6.40	48
		60.33	6.04	0.38	1.17	52.74	16.88	4.62	0.29	0.89	22.68		
0.4	69.5											29.08	218
		224.14	8.64	0.34	1.75	213.41	68.29	6.6	0.26	1.33	76.48		
0.9	70											105.56	790
		267.45	0	0	0	267.45	85.58	0	0	0	85.58		
1.4	70.5											191.14	1430
		60.70	0	0	0	60.70	19.42	0	0	0	19.42		
1.6	70.7											210.57	1575



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	PROJECT ENGINEER	DESIGNED MPW / DRAFTED / CHECKED / DATE 04-19-96	NO. 3 2 1 / DATE / REVISIONS / BY / APVD.	SCALE 1"=3'	VERIFY SCALE	USPCI A LAIDLAW COMPANY	Lone Mountain Facility Landfill Cell 15 Bottom Sump - Sump Area No. 1	REVISION 0 SHEET NO OF

HA&L ENGINEERING

CLIENT: USPCI / Laidlaw
PROJECT: Lone Mtn. - Cell 15
FEATURE: Stage Capacity - Bottom Sumps 3 & 4
PROJECT NO.: 64.44.600

SHEET 1 OF 4
COMPUTED: SWA
CHECKED:
DATE: 6/25/96

I - Stage Capacity - Not adjusted for porosity or pipes

Top of Sump = 1368.9 - Sump No. 3.
Note: Sumps 3 & 4 are identical configurations, except Sump 3 is 3' higher in elevation than Sump 4

$units^2 \times \frac{25 in^2}{1634 units^2} \times \frac{3^2 ft^2}{in^2}$

LANDFILL CELL 15 - LONE MOUNTAIN FACILITY
STAGE CAPACITY - BOTTOM SUMPS 3 & 4

DEPTH FT	ELEV. FT.	PLANIMETER UNITS @ 15	AREA FT2	AVE. AREA FT2	UNADJUSTED VOLUME FT3
0	67.1	0	0.0		
				13.1	1.31
0.1	67.2	190	26.2		
				79.6	7.96
0.2	67.3	966	133.0		
				234.8	46.97
0.4	67.5	2445	336.7		
				403.3	201.63
0.9	68	3412	469.8		
				511.6	255.81
1.4	68.5	4019	553.4		
				490.3	49.03
1.5	68.6	3103	427.3		
				262.6	52.52
1.7	68.8	711	97.9		
				54.4	5.44
1.8	68.9	79	10.9		

II - Adjust Volumes for pipes vs. gravel

A - 4" Dia. Pipes.
Assume all 4" pipe above EL 1367.2
Assume 15% of 4" pipe volume between 67.2 and 67.3
" 35% " " " " " 67.3 and 67.5
" 50% " " " " " 67.5 and 68.0

Total length of pipe in sump = 2(28.5) + 11(9.1) = 157 ft.
ID = 3.92" ⇒ area = 0.084 ft^2
OD = 4.5" ⇒ area = 0.110 ft^2

Volume removed from sump = 0.110(157) = 17.27 ft^3
Volume added back in " = 0.084(157) = 13.19 ft^3

HA&L ENGINEERING

CLIENT: USPCI / Laidlaw
PROJECT: Lone Mtn - Cell 15
FEATURE: Storage Capacity Bottom Sumps 3 ; 4
PROJECT NO.: 64.44.600

SHEET 2 OF 4
COMPUTED: SMA
CHECKED:
DATE: 6/25/96

B - 6" Air Pipe

$\theta = 1.8235$ rad
$0.3 = \frac{0.48}{2}(1-\cos(\theta))$
Area $= \frac{0.48^2}{4}(1.8235 - \sin 1.8235 \cos 1.8235) = 0.182\ ft^2$
$= 0.119\ ft^2$
Area full pipe $= \frac{\pi(0.48)^2}{4} = 0.65$
Ratio $= \frac{0.119}{0.182}$

Assume it is added between 67.2 and 67.7

Volume between 1367.2 ; 1367.3 - Use 25%
" " 1367.3 ; 1367.5 - Use 40%
" " 1367.5 ; 1368.0 - Use 35%

Pipe length = 2' each way = 4'
ID = 5.771" ⟹ area = $0.182\ ft^2$
OD = 6.625" ⟹ area = $0.239\ ft^2$

Volume removed from sump = 0.239(4) = $0.96\ ft^3$
Volume added back in = 0.182(4) = $0.73\ ft^3$

C - 10" Air - HDPE Pipe

Assume it is added between 67.1 and 67.93
Area @ 0.1' depth = $0.036\ ft^2$
" " 0.2' " = $0.097\ ft^2$
" " 0.4' " = $0.247\ ft^2$
" " full " = $0.478\ ft^2$

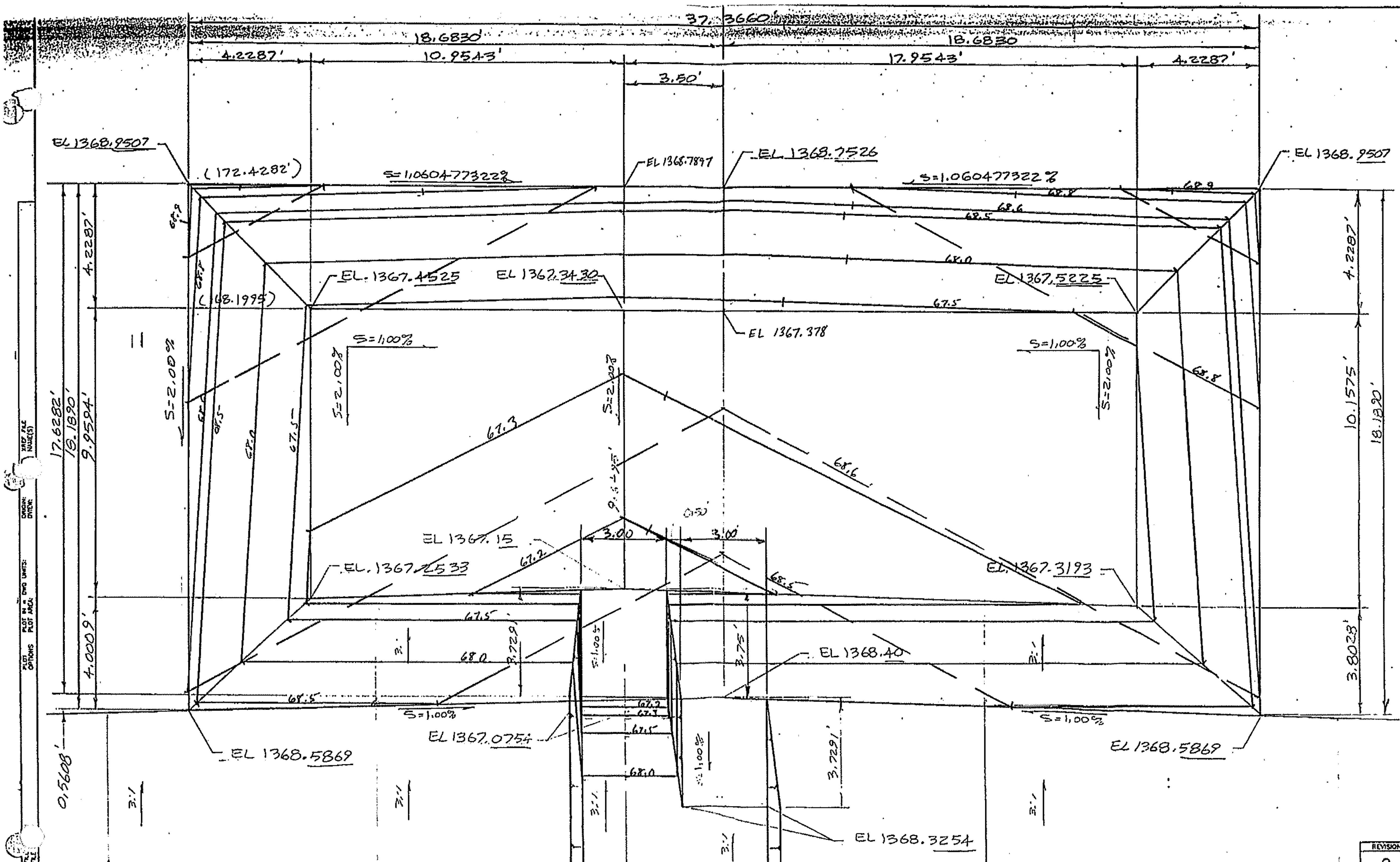
Assume $\frac{0.036}{0.478}$ = 7.5% of volume between 67.1 and 67.2
$\frac{0.097}{0.478}$ = 20% (20-7.5) = 12.5% between 67.2 ; 67.3
$\frac{0.247}{0.478}$ = 52% (52-20) = 32% between 67.3 ; 67.5
48% between 67.5 and 68.0

I.D = 9.362" ⟹ Area = $0.478\ ft^2$
OD = 10.75" ⟹ Area = $0.63\ ft^2$
Total length = 5.7'

Volume taken out = 0.63(5.7) = $3.59\ ft^3$
Volume added in = 0.478(5.7) = $2.72\ ft^3$

III - Adjust Volumes for Pipes ; Porosity
Assume porosity = 0.32 (As per testing by AGEC)

DEPTH FT	ELEV. FT	UNADJUSTED VOLUME FT3	VOLUME TAKEN OUT BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE OUT FT3	ADJUSTED VOLUME POROSITY 0.32X FT3	VOLUME ADDED BACK IN BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE IN FT3	ACCUMULATED VOLUME FT3	GALLONS
												0.00	0
0	67.1	1.31	0	0	0.27	1.04	0.33	0	0	0.2	0.53	0.53	4
0.1	67.2	7.96	2.59	0.24	0.45	4.68	1.50	1.98	0.18	0.34	4.00	4.53	34
0.2	67.3	46.97	6.04	0.38	1.15	39.40	12.61	4.62	0.29	0.87	18.39	22.92	171
0.4	67.5	201.63	8.64	0.34	1.72	190.93	61.10	6.6	0.26	1.31	69.27	92.18	690
0.9	68	255.81	0	0	0	255.81	81.86	0	0	0	81.86	174.04	1302
1.4	68.5	49.03	0	0	0	49.03	15.69	0	0	0	15.69	189.73	1419
1.5	68.6	52.52	0	0	0	52.52	16.81	0	0	0	16.81	206.54	1545
1.7	68.8	5.44	0	0	0	5.44	1.74	0	0	0	1.74	208.28	1558
1.8	68.9												



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	PROJECT ENGINEER	DESIGNED MPW / DRAFTED / CHECKED / DATE 04-18-96	REVISIONS	SCALE 1" = 3'	VERIFY SCALE	USPCI A LAIDLAW COMPANY	Lone Mountain Facility Landfill Cell 15 Bottom Sump Sump Area No. 3	REVISION 0 SHEET NO. - of

HA&L ENGINEERING

CLIENT: USPCI/Laidlaw
PROJECT: Lone Mtn. Cell 15
FEATURE: Stage Capacity Bottom Sump No. 5
PROJECT NO. 64.144.600
SHEET 1 OF 4
COMPUTED SMA
CHECKED
DATE 6/26/96

I - Stage Capacity - Not adjusted for porosity or pipes

LANDFILL CELL 15 - LONE MOUNTAIN FACILITY
STAGE CAPACITY - BOTTOM SUMP 5

DEPTH FT	ELEV. FT.	PLANIMETER UNITS @ 15	AREA FT2	AVE. AREA FT2	UNADJUSTED VOLUME FT3
0	62.1	0	0.0		
				14.8	1.48
0.1	62.2	215	29.6		
				95.8	9.58
0.2	62.3	1176	161.9		
				231.4	23.14
0.3	62.4	2185	300.9		
				332.7	33.27
0.4	62.5	2648	364.6		
				433.4	216.70
0.9	63	3647	502.2		
				533.6	266.79
1.4	63.5	4103	565.0		
				475.5	47.55
1.5	63.6	2803	386.0		
				298.7	29.87
1.6	63.7	1536	211.5		
				168.5	16.85
1.7	63.8	912	125.6		

$\text{Units} \times \frac{25\,in^2}{1654\,\text{Units}} \times \frac{3^2 ft^2}{in^2}$

II - Adjust Volumes for pipes vs. gravel.

A - 4" dia Pipes

Assume all 4" dia pipes in sumps above EL 1362.2
Assume 15% of 4" pipe volume between 62.2 ; 62.3
Assume 35% of 4" pipe volume between 62.3 ; 62.5
Assume 50% " " " " between 62.5 ; 63.0

Total length of pipe in sumps = 157'

ID = 3.92" ⟹ area = 0.084 ft^2

OD = 4.5" ⟹ area = 0.110 ft^2

63.7
63.36

Volume removed from sump = 0.110(157) = 17.27 ft^3

Volume added back in sump = 0.084(157) = 13.19 ft^3

HA&L ENGINEERING

CLIENT USPCI - Laidlaw
PROJECT Lone Mtn - Cell 15
FEATURE Stage Capacity Bottom Sump No. 5
PROJECT NO. 064.44.6001

COMPUTED MA
CHECKED
DATE 6/26/96

B - 6" Dia Pipes

Assume it lies between 62.2 and 62.7
Volume between 62.2 ; 62.3 - Use 25%
" " 62.3 ; 62.5 - Use 40%
" " 62.5 ; 62.7 - Use 35%

Pipe length = 2' each way = 4'
ID = 5.771" => area = 0.182 ft^2
OD = 6.625" => area = 0.239 ft^2

Volume removed from sump = 0.239(4) = 0.96 ft^3
" added back in = 0.182(4) = 0.73 ft^3

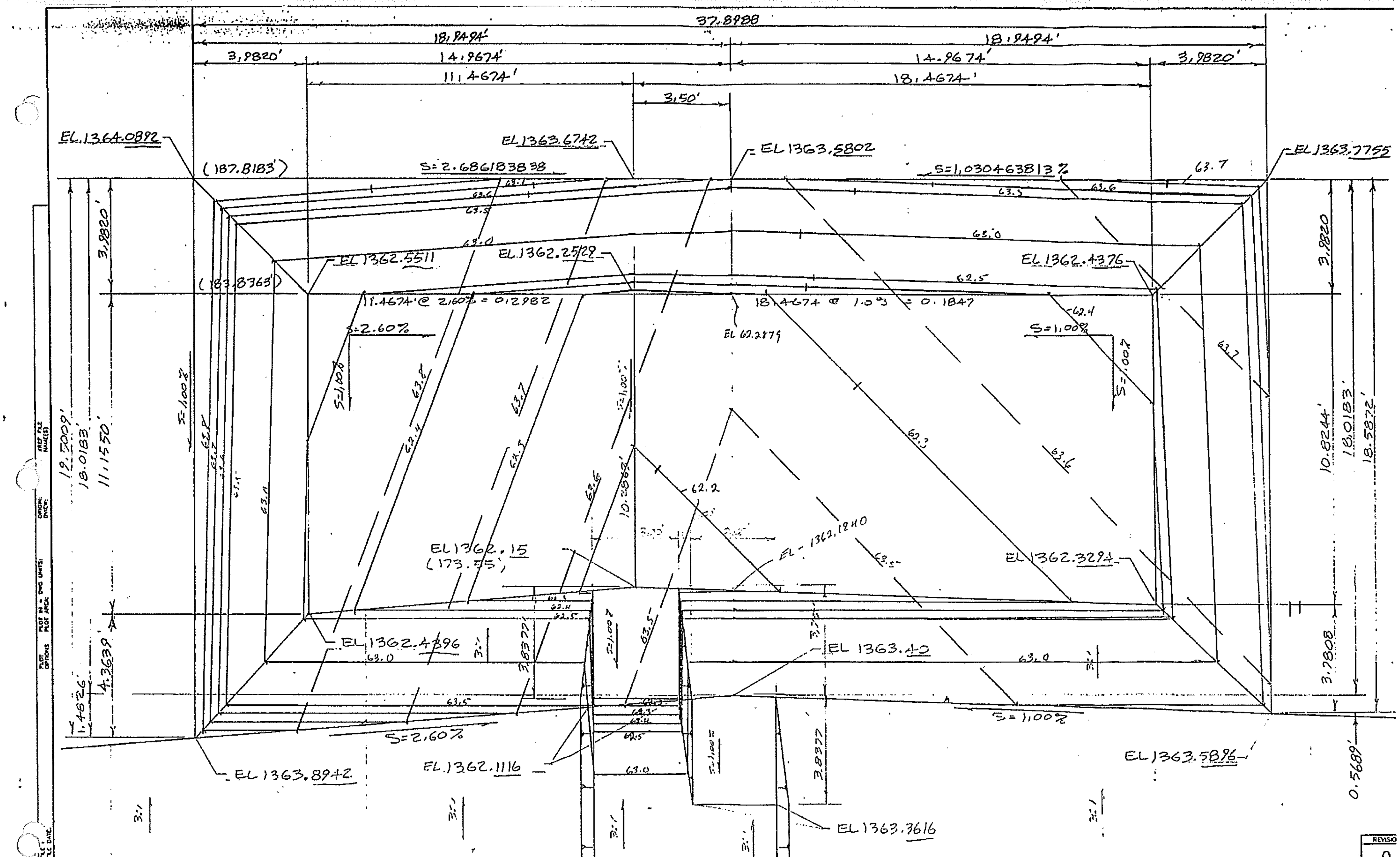
C - 10" Dia - HDPE Pipe
Assume it lies between EL 1362.1 and 1362.93
Area @ 0.1' depth = 0.036 ft^2
" " 0.2' " = 0.097 ft^2
" " 0.4' " = 0.217 ft^2
" " full " = 0.478 ft^2

Assume: $\frac{0.036}{0.478}$ = 7.5% of vol between 62.1 and 62.2
$\frac{0.097}{0.478}$ = 20% (20-7.5) = 12.5% between 62.2 ; 62.3'
$\frac{0.247}{0.478}$ = 52% (52-20) = 32% between 62.3 and 62.5'
48% between 62.5 and 63.0

I.D = 9.362" => area = 0.478 ft^2
O.D = 10.75" => area = 0.63 ft^2
Total Length = 5.8'
Volume Removed = 5.8(0.63) = 3.65 ft^3
Volume In = 5.8(0.478) = 2.77 ft^3

III - Adjust Volumes for Pipes and Porosity
Assume porosity = 0.32 (As per testing by AGEC)

DEPTH FT	ELEV. FT	UNADJUSTED VOLUME FT3	VOLUME TAKEN OUT BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE OUT FT3	ADJUSTED VOLUME POROSITY 0.32X FT3	VOLUME ADDED BACK IN BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE IN FT3	ACCUMULATED VOLUME FT3	GALLONS
												0.00	0
0	62.1	1.48	0	0	0.27	1.21	0.39	0	0	0.21	0.60	0.60	4
0.1	62.2	9.58	2.59	0.24	0.46	6.29	2.01	1.98	0.18	0.35	4.52	5.12	38
0.2	62.3	23.14	3.02	0.19	0.58	19.35	6.19	2.31	0.14	0.44	9.08	14.20	106
0.3	62.4	33.27	3.02	0.19	0.58	29.48	9.44	2.31	0.15	0.44	12.34	26.54	198
0.4	62.5	216.70	8.64	0.34	1.75	205.97	65.91	6.6	0.26	1.33	74.10	100.64	753
0.9	63	266.79	0	0	0	266.79	85.37	0	0	0	85.37	186.01	1391
1.4	63.5	47.55	0	0	0	47.55	15.22	0	0	0	15.22	201.23	1505
1.5	63.6	29.87	0	0	0	29.87	9.56	0	0	0	9.56	210.79	1577
1.6	63.7	16.85	0	0	0	16.85	5.39	0	0	0	5.39	216.18	1617
1.7	63.8												



HA&L ENGINEERING — Consultants Engineers, Salt Lake City, Utah

DESIGNED	MPW	
DRAFTED		
CHECKED		
DATE	04-22-96	

PROJECT ENGINEER

NO. | DATE | REVISIONS | BY | APVD.

SCALE 1" = 3'

VERIFY SCALE

USPCI — A LAIDLAW COMPANY

Lone Mountain Facility
Landfill Cell 15
Bottom Sump – Sump Area No. 5

REVISION 0

SHEET NO. OF

I - Stage Capacity - Not adjusted for porosity or pipes

LANDFILL CELL 15 - LONE MOUNTAIN FACILITY
STAGE CAPACITY - BOTTOM SUMP 6

DEPTH FT	ELEV. FT.	PLANIMETER UNITS @ 15	AREA FT2	AVE. ARE FT2	UNADJUSTED VOLUME FT3
0	60.1	0	0.0		
				19.4	1.94
0.1	60.2	282	38.8		
				131.7	13.17
0.2	60.3	1631	224.6		
				278.2	27.82
0.3	60.4	2409	331.7		
				345.1	34.51
0.4	60.5	2603	358.4		
				424.8	212.40
0.9	61	3567	491.2		
				474.1	237.05
1.4	61.5	3319	457.0		
				293.3	29.33
1.5	61.6	941	129.6		
				64.8	6.48
1.6	61.7	0	0.0		

Units x $\frac{25\ in^2}{1631\ units}$ x $\frac{3^2\ ft^2}{in^2}$

II - Adjust Volumes above for pipes vs. gravel.

A - 4" Dia Pipes

Assume all 4" pipe in sump above 1360.2 EL.
Assume 15% of 4" pipe volume between 60.2 and 60.3 EL.
Assume 35% of 4" pipe volume between 60.3 and 60.5 EL
Assume 50% of 4" pipe volume between 60.5 and 61.0

Total Length of pipe in sumps ≈ 157'

I.D = 3.92" ⇒ area = 0.084 ft^2
O.D = 4.5" ⇒ area = 0.110 ft^2

Volume removed from sump = 0.110(157) = 17.27 ft^3
Volume added back to sump = 0.084(157) = 13.19 ft^3

HA&L ENGINEERING

CLIENT: USPCI/Laidlaw
PROJECT: Cell 15
FEATURE: Stage Capacity Lower Sump No. 6
PROJECT NO.: 64.144.6000

SHEET 2 OF 4
COMPUTED: JMA
CHECKED:
DATE: 6/26/96

B - 6" Dia HDPE Pipes
Assume it is added between EL 1360.2 and 1360.7
Volume between 1360.2 and 1360.3 Use 25%
" " 1360.3 " 1360.5 Use 40%
" above 1360.5 Use 35%

Pipe Length = 2' Each Way = 4'
$ID = 5.771" \Rightarrow area = 0.182 ft^2$
$OD = 6.625" \Rightarrow area = 0.239 ft^2$

Volume removed from sump = $0.239(4) = 0.96 ft^3$
Volume added back in = $0.182(4) = 0.73 ft^3$

C - 10" Dia HDPE Pipe
Assume it is added between EL 1360.1 and 1360.93
Area @ 0.1' depth = $0.036 ft^2$
" " 0.2' " = $0.097 ft^2$
" " 0.4' " = $0.217 ft^2$
" " full " = $0.478 ft^2$

Assume - $\frac{0.036}{0.478} = 7.5\%$ of volume between 60.1 ; 60.2
$\frac{0.097}{0.478} = 20\%$ $(20-7.5) = 12.5\%$ between 60.2 ; 60.3
$\frac{0.217}{0.478} = 52\%$ $(52-20) = 32\%$ between 60.3 ; 60.5
∴ 48% between 60.5 ; 61.0

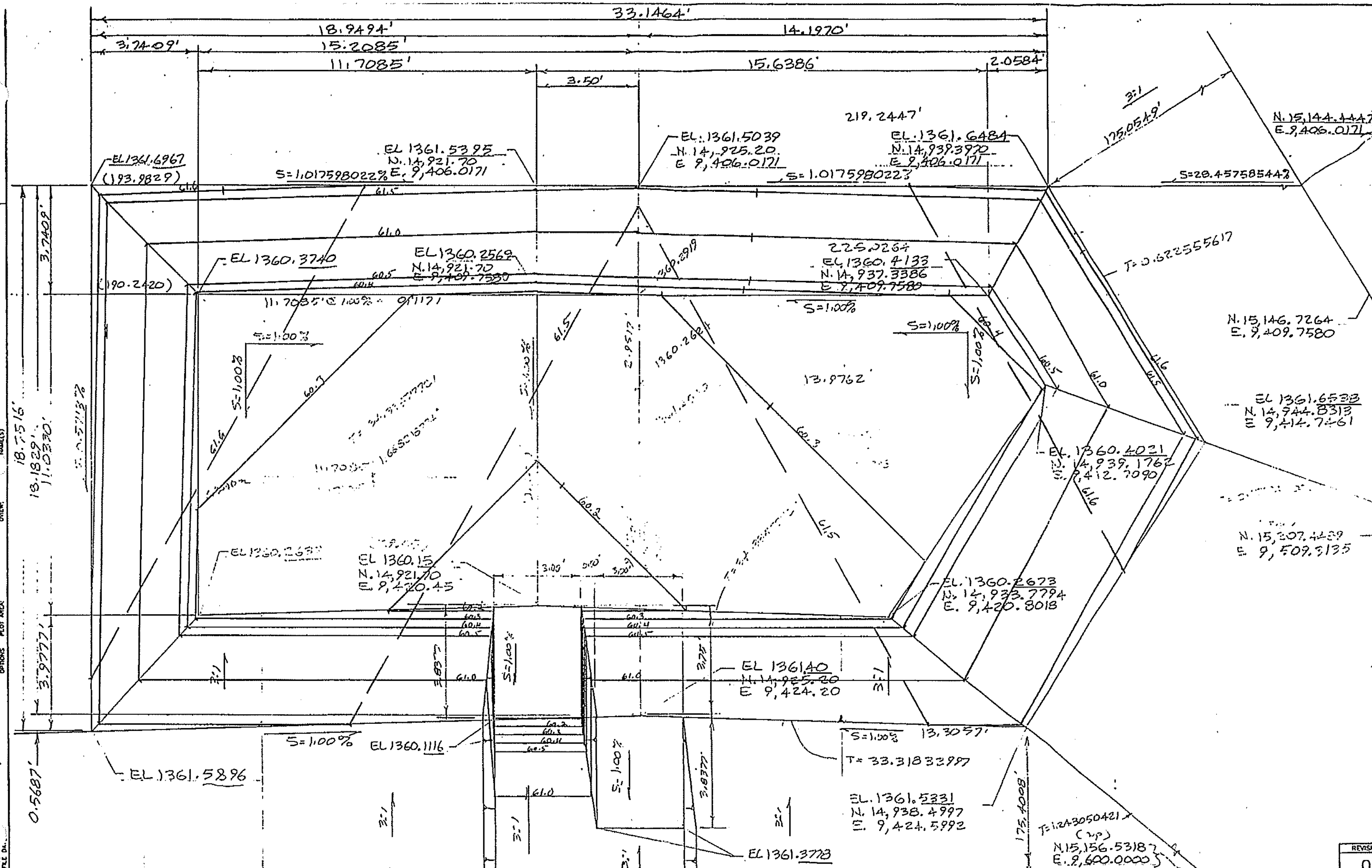
$ID = 9.362" \Rightarrow area = 0.478 ft^2$
$OD = 10.75" \Rightarrow area = 0.63 ft^2$

Total Length = 5.8'
Volume taken out = $0.63(5.8) = 3.65 ft^3$
" added back in = $0.478(5.8) = 2.77 ft^3$

III - Adjust Volumes for Pipes ; Porosity
Assume porosity = 0.32 (As per AGEC tests on sump rock)

6/26/96

DEPTH FT	ELEV. FT	UNADJUSTED VOLUME FT3	VOLUME TAKEN OUT BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE OUT FT3	ADJUSTED VOLUME POROSITY 0.32X FT3	VOLUME ADDED BACK IN BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE IN FT3	ACCUMULATED VOLUME FT3	GALLONS *
0	60.1											0.00	0
		1.94	0	0	0.27	1.67	0.53	0	0	0.21	0.74		
0.1	60.2											0.74	6
		13.17	2.59	0.24	0.46	9.88	3.16	1.98	0.18	0.35	5.67		
0.2	60.3											6.42	48
		27.82	3.02	0.19	0.58	24.03	7.69	2.31	0.14	0.44	10.58		
0.3	60.4											16.99	127
		34.51	3.02	0.19	0.58	30.72	9.83	2.31	0.15	0.44	12.73		
0.4	60.5											29.72	222
		212.40	8.64	0.34	1.75	201.67	64.53	6.6	0.26	1.33	72.72		
0.9	61											102.45	766
		237.05	0	0	0	237.05	75.86	0	0	0	75.86		
1.4	61.5											178.30	1334
		29.33	0	0	0	29.33	9.39	0	0	0	9.39		
1.5	61.6											187.69	1404
		6.48	0	0	0	6.48	2.07	0	0	0	2.07		
1.6	61.7											189.76	1419



HA&L ENGINEERING	CONSULTANTS ENGINEERS Salt Lake City Utah	PROJECT ENGINEER	
DESIGNED MPW	DRAFTED	CHECKED	DATE 04-23-96
SCALE 1"=3'		USPCI a LAIDLAW COMPANY	Lone Mountain Facility Landfill Cell 15 Bottom Sump - Sump Area No 6
REVISION 0	SHEET NO.	OF	

REVISIONS: NO. | DATE | BY | APVD.

HA&L ENGINEERING

CLIENT: USPCI/Laidlaw
PROJECT: Cell 15
FEATURE: Stage Capacity Lower Sump No. 7
PROJECT NO.: 94.44.600

COMPUTED: CMA
CHECKED:
DATE: 6/27/96

Planimeter @ 15

Units $\times \frac{25 in^2}{1634 units} \times \frac{6^2 ft^2}{1 in^2}$

I - Stage Capacity - Not adjusted for porosity or pipes

LANDFILL CELL 15 - LONE MOUNTAIN FACILITY
STAGE CAPACITY - BOTTOM SUMP 7

DEPTH FT	ELEV. FT.	PLANIMETER UNITS @ 15	AREA FT2	AVE. AREA FT2	UNADJUSTED VOLUME FT3
0	68.6	0	0.0		
				41.3	4.13
0.1	68.7	150	82.6		
				169.1	16.91
0.2	68.8	464	255.6		
				571.7	114.35
0.4	69	1612	887.9		
				1039.4	519.68
0.9	69.5	2162	1190.8		
				1248.7	374.60
1.2	69.8	2372	1306.5		
				1197.2	239.43
1.4	70	1975	1087.8		
				782.7	156.54
1.6	70.2	867	477.5		
				238.8	47.75
1.8	70.4	0	0.0		

II - Adjust Volumes above for pipes vs. gravel

A - 4" Dia Pipes

Assume all 4" pipe in sump above 1368.7

Assume 10% of 4" pipe volume between 1368.7 and 1368.8
Assume 50% " " " " " 1368.8 " 1369.0
Assume 40% " " " " " 1369.0 and 1369.5

Total Length of pipe = 467

I.D = 3.92" => Area = 0.084 ft^2
O.D = 4.5" => Area = 0.110 ft^2

Volume removed from sump = 0.110 (467) = 51.37 ft^3
" added back in sump = 0.084 (467) = 39.23 ft^3

B - 6" Dia HDPE Pipe

Assume volume is added between 68.7 and 69.5

Volume between 68.7 and 68.8 - Use 20%
Volume " 68.8 " 69.0 - Use 40%
Volume " 69.0 " 69.5 Use 40%

Pipe Length = 4'
$ID = 5.771" \Rightarrow Area = 0.182 ft^2$
$OD = 6.625" \Rightarrow Area = 0.239 ft^2$

Volume Removed from sump $= 0.239(4) = 0.96 ft^3$
" added back in $= 0.182(4) = 0.73 ft^3$

C - 10" Dia HDPE Pipe

Assume pipe is between EL 1368.6 and 1369.5

Area @ 0.1' depth $= 0.036 ft^2$
" " 0.2' " $= 0.097 ft^2$
" " 0.4' " $= 0.247 ft^2$
" " full " $= 0.478 ft^2$

Assume $\frac{0.036}{0.478} = 7.5\%$ of volume between 68.6 ; 68.7

$\frac{0.097}{0.478} = 20\% (20-7.5) = 12.5\%$ between 68.7 ; 68.8

$\frac{0.247}{0.478} = 52\% (52-20) = 32\%$ " 68.8 ; 69.0

Use = 48% between 69.0 ; 69.5

$I.D = 9.362" \Rightarrow Area = 0.478 ft^2$
$O.D. = 10.75" \Rightarrow Area = 0.63 ft^2$

Total Length = 5.8'

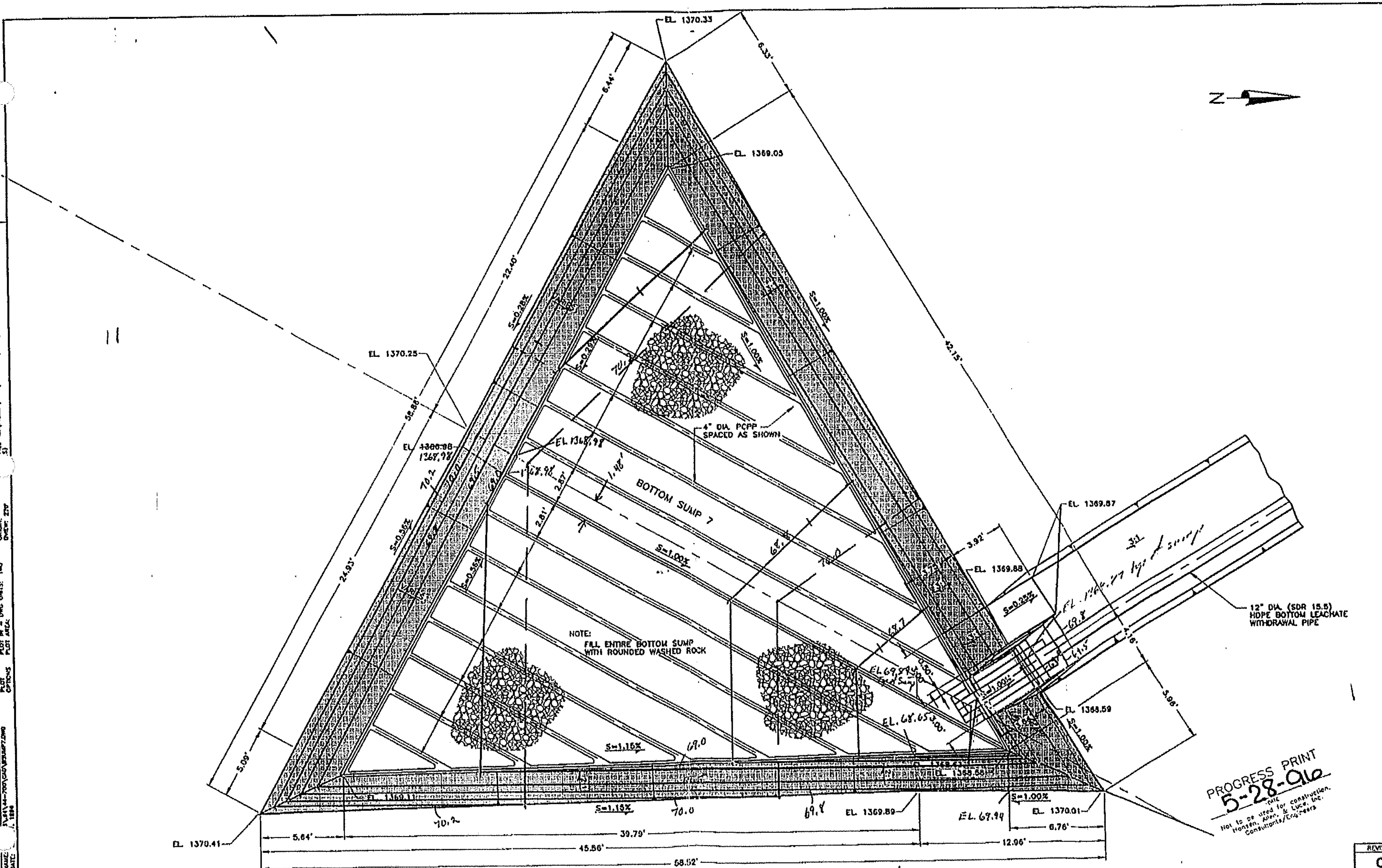
Volume taken out $= 5.8(0.63) = 3.65 ft^3$
" added back in $= 5.8(0.478) = 2.77 ft^3$

III - Adjust Volumes for Pipes ; Porosity

Assume porosity = 0.32 (as per AGEC tests on sump rock)

DEPTH FT	ELEV. FT	UNADJUSTE VOLUME FT3	VOLUME TAKEN OUT BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTE VOLUME PIPE OUT FT3	ADJUSTE VOLUME POROSIT 0.32X FT3	VOLUME ADDED BACK IN BY PI 4" FT3	6" FT3	10" FT3	ADJUSTE VOLUME PIPE IN FT3	ACCUMULATED VOL FT3	GALLONS
0	68.6											0.00	0
		4.13	0	0	0.27	3.86	1.24	0	0	0.21	1.45		
0.1	68.7											1.45	11
		16.91	5.14	0.19	0.46	11.12	3.56	3.92	0.15	0.35	7.98		
0.2	68.8											9.42	70
		114.35	25.68	0.38	1.17	87.12	27.88	19.62	0.29	0.89	48.68		
0.4	69											58.10	435
		519.68	20.55	0.38	1.75	497.00	159.04	15.69	0.29	1.33	176.35		
0.9	69.5											234.45	1754
		374.60	0	0	0	374.60	119.87	0	0	0	119.87		
1.2	69.8											354.32	2650
		239.43	0	0	0	239.43	76.62	0	0	0	76.62		
1.4	70											430.94	3223
		156.54	0	0	0	156.54	50.09	0	0	0	50.09		
1.6	70.2											481.03	3598
		47.75	0	0	0	47.75	15.28	0	0	0	15.28		
1.8	70.4											496.31	3712

3/4
6/27/96



HA&L ENGINEERING — CONSULTANTS ENGINEERS — Salt Lake City, Utah

DESIGNED	MPW
DRAFTED	RGA
CHECKED	KCS
DATE	MAY 1996

REVISIONS

SCALE 1"=3'

VERIFY SCALE: BAR IS ONE INCH ON ORIGINAL DRAWING. IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY.

USPCI — A LAIDLAW COMPANY

LONE MOUNTAIN FACILITY
LANDFILL CELL 15
BOTTOM SUMP NO. 7

REVISION 0

SHEET NO. 19 OF 45

HA&L ENGINEERING

CLIENT USPCI / Laidlaw
PROJECT Cell 15
FEATURE Stage Capacity Lower Sump No. 8
PROJECT NO. 84144.600

COMPUTED SMR
CHECKED
DATE 7/1/96

I - Stage Capacity - Not adjusted for porosity or pipes.

Top of Sump EL = 1377.3'

LANDFILL CELL 15 - LONE MOUNTAIN FACILITY
STAGE CAPACITY - BOTTOM SUMP 8

$\text{Units} \times \frac{25\,in^2}{1634\,units} \times \frac{5^2\,ft^2}{in^2}$

DEPTH FT	ELEV. FT.	PLANIMETER UNITS @ 15	AREA FT2	AVE. AREA FT2	UNADJUSTED VOLUME FT3
0	75.7	0	0.0		
				38.8	3.88
0.1	75.8	203	77.6		
				166.4	16.64
0.2	75.9	667	255.1		
				324.2	32.42
0.3	76	1028	393.2		
				460.0	229.98
0.8	76.5	1377	526.7		
				601.9	300.93
1.3	77	1770	677.0		
				463.0	92.60
1.5	77.2	651	249.0		
				143.6	14.36
1.6	77.3	100	38.2		

II - Adjust Volumes above for pipes vs. gravel

A - 4" Dia Pipes

Assume all 4" pipes in sumps are above EL 1375.8
Assume 15% of 4" pipe volume between 75.8 and 75.9
" 20% " " " " " 75.9 and 76.0
" 65% " " " " " 76.0 and 76.5

Total length of pipe in sump = 195

I.D = 3.92" ⇒ Area = 0.084 ft^2
O.D. = 4.5" ⇒ Area = 0.110 ft^2

Volume removed from sump = 0.110(195) = 21.5 ft^3
" added back in = 0.084(195) = 16.4 ft^3

HA&L ENGINEERING

CLIENT USPCI / Laidlaw
PROJECT Cell 15
FEATURE Storage Capacity Lower Sump No. 8
PROJECT NO. 611.44.600

SHEET 2 OF 4
COMPUTED DMA
CHECKED
DATE 7/1/96

B - 6" Dia HDPE Pipes

Assume it is added between 75.7 and 76.2
Volume between 1375.7 ; 1375.8 Use 25%
" " 1375.8 ; 1376.0 Use 40%
" above 1376.0 Use 35%

Pipe Length = 4'

ID = 5.771" => Area = 0.182 ft^2
OD = 6.625" => Area = 0.239 ft^2

Volume Removed from Sump = 0.239(4) = 0.96 ft^3
" added back in = 0.182(4) = 0.73 ft^3

C - 10" Dia HDPE Pipe

Assume it is added between EL 1375.7 ; 1376.53
Area @ 0.1' depth = 0.036 ft^2
" " 0.2' depth = 0.097 ft^2
" " 0.3' depth = 0.169 ft^2
" full = 0.478 ft^2

Assume $\frac{0.036}{0.478}$ = 7.5% of volume between 75.7 ; 75.8

$\frac{0.097}{0.478}$ = 20% (20 - 7.5) = 12.5% between 75.8 ; 75.9

$\frac{0.169}{0.478}$ = 35% (35 - 20) = 15% between 75.9 ; 76.0

65% between 76.0 and 76.5

I.D = 9.362" => Area = 0.478 ft^2
OD = 10.75" => Area = 0.63 ft^2

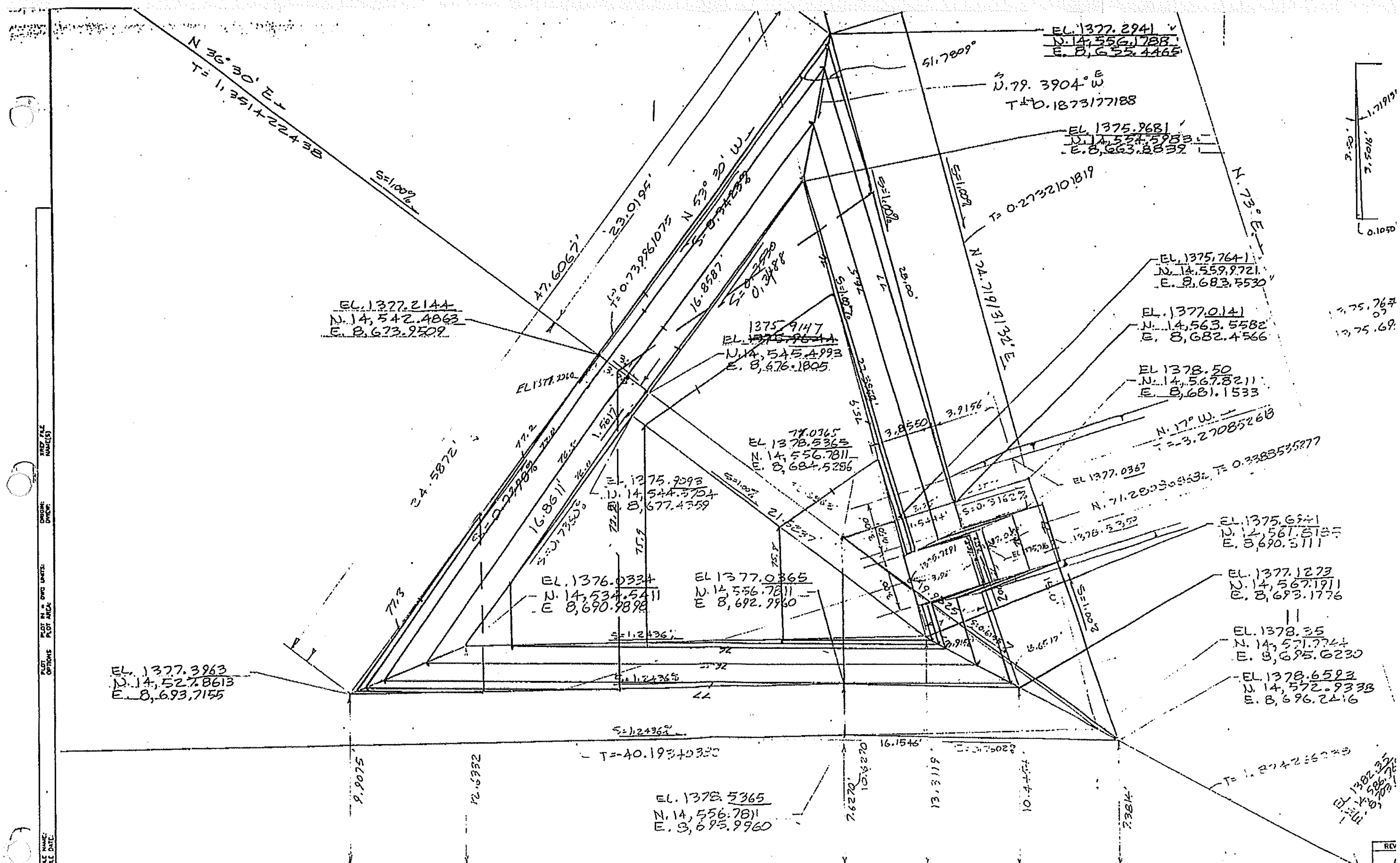
Total Length = 5.9'

Volume taken out = 0.63(5.9) = 3.72 ft^3
" added back = 0.478(5.9) = 2.82 ft^3

III - Adjust Volumes for Pipes ; Porosity

Assume porosity = 0.32 (As per AGEC based on tests on sump rock)

DEPTH FT	ELEV. FT	UNADJUSTED VOLUME FT3	VOLUME TAKEN OUT BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTED VOLUME PIPE OUT FT3	ADJUSTE VOLUME POROSIT 0.32X FT3	VOLUME ADDED BACK IN BY PIPE 4" FT3	6" FT3	10" FT3	ADJUSTE VOLUME PIPE IN FT3	ACCUMULATED VOLUM FT3	GALLONS
0	75.7											0.00	0
		3.88	0	0.24	0.28	3.36	1.08	0	0.18	0.21	1.47		
0.1	75.8											1.47	11
		16.64	3.23	0.19	0.47	12.75	4.08	2.46	0.15	0.35	7.04		
0.2	75.9											8.51	64
		32.42	4.3	0.19	0.56	27.37	8.76	3.28	0.15	0.42	12.61		
0.3	76											21.11	158
		229.98	13.98	0.34	2.42	213.24	68.24	10.66	0.26	1.83	80.99		
0.8	76.5											102.10	764
		300.93	0	0	0	300.93	96.30	0	0	0	96.30		
1.3	77											198.40	1484
		92.60	0	0	0	92.60	29.63	0	0	0	29.63		
1.5	77.2											228.03	1706
		14.36	0	0	0	14.36	4.60	0	0	0	4.60		
1.6	77.3											232.62	1740



HA&L ENGINEERING — CONSULTANTS ENGINEERS, Salt Lake City, Utah	PROJECT ENGINEER	DESIGNED MPW; DRAFTED; CHECKED; DATE 04-30-96	NO. / DATE / REVISIONS / BY / APVD.	SCALE 1" = 5'

USPCI — A LAIDLAW COMPANY

Lone Mountain Facility
Landfill Cell 15
Sump Area No 8 - Bottom Sump

EXHIBIT F

LANDFILL CELL 15 CLOSURE DESIGN CALCULATIONS

HA&L ENGINEERING

HA&L ENGINEERING

CLIENT VSPCI
PROJECT Landfill Cell 15 - modified Closure
FEATURE Cap Hydrology
PROJECT NO. 14-44-300

SHEET 1 OF 24
COMPUTED PGH
CHECKED KCS
DATE 5-20-93

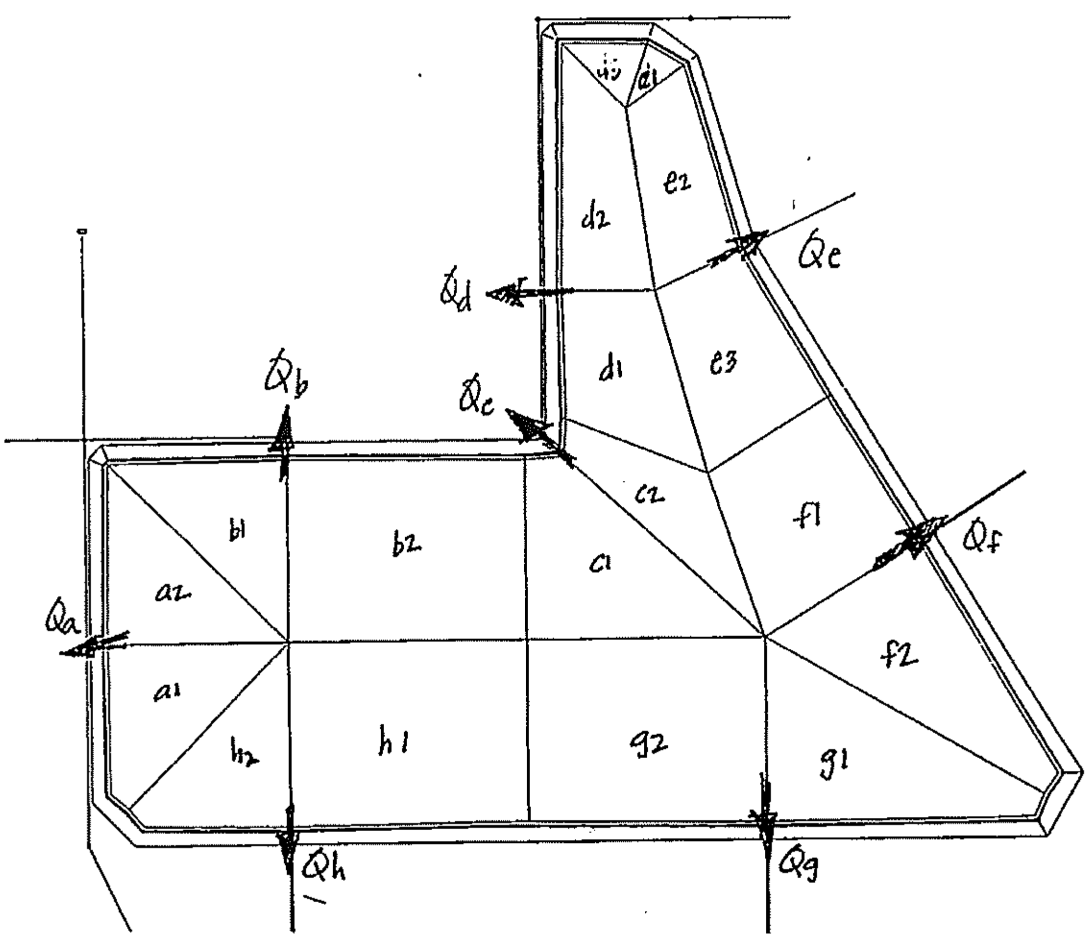
Revised 5/23/96

Purpose: Determine peak flow generated by cell closure cap to the collector ditches and downspouts.

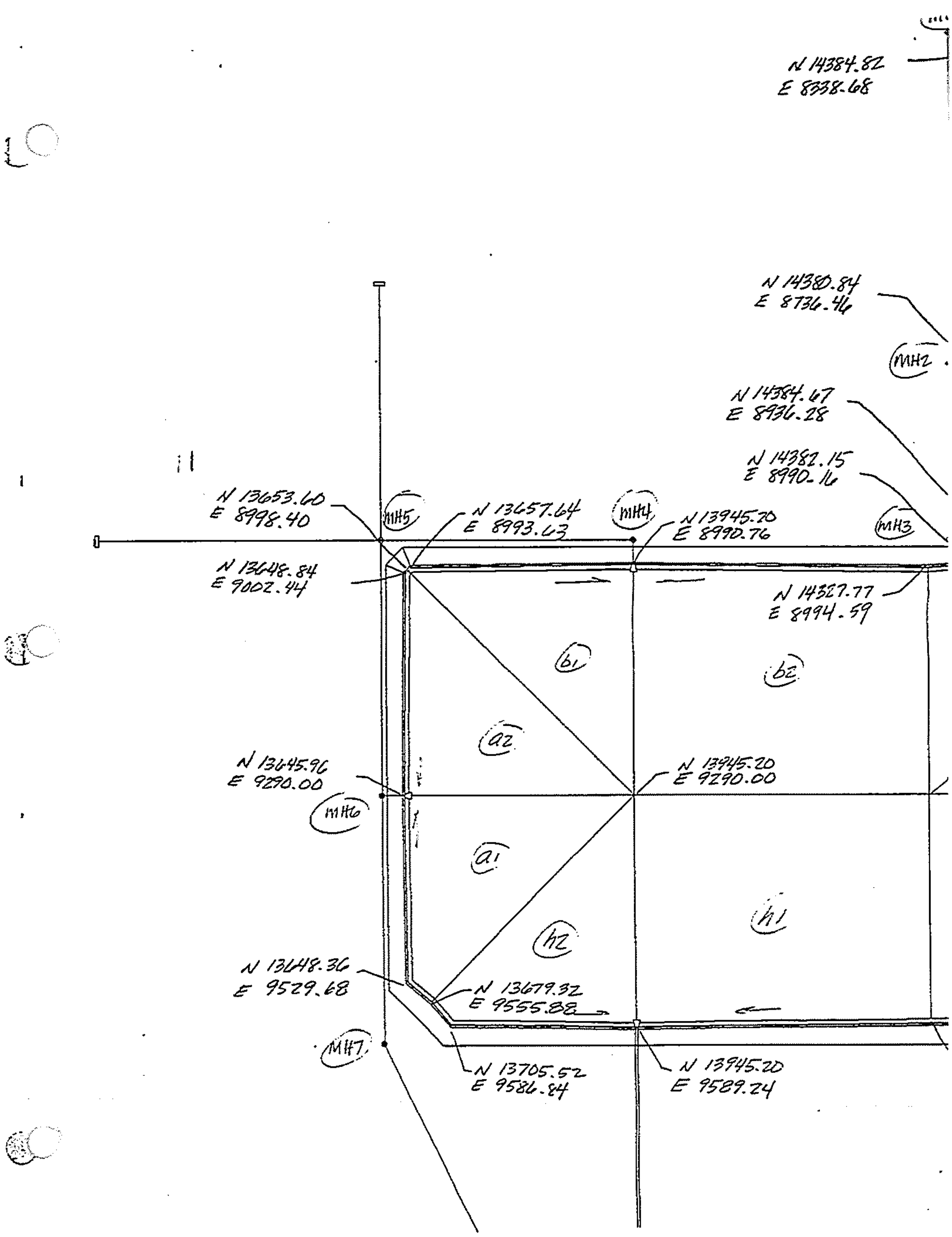
Assume the following:

$P = 8"$ (100 year, 24 hour)
$S = 10\%$
ditch slope $= 0.5\%$

The tributary area to the cap downspouts is as shown below. Each area will be analyzed independently due to a lack of symmetry.



Summarized on the following 2 sheets are the tributary areas, as identified above. The areas were calculated using by the coordinate method.



N 14384.82
E 8338.68
N 14380.84
E 8736.46
MH2
N 14384.67
E 8936.28
N 14382.15
E 8990.16
MH3
N 13653.60
E 8998.40
MH5
N 13657.64
E 8993.63
MH4
N 13945.20
E 8990.76
N 13648.84
E 9002.44
N 14327.77
E 8994.59
b1
b2
a2
N 13645.96
E 9290.00
N 13945.20
E 9290.00
MH6
a1
h1
h2
N 13648.36
E 9529.68
N 13679.32
E 9555.88
MH7
N 13705.52
E 9586.84
N 13945.20
E 9589.24

3/24

nination of Areas by Coordinate

Project Name : Landfill Cell 15 - Closure Modification
Project Number: 64-44-300
Date : 20-May-93
By: PGH

									(Acres)
Basin I.D.	North :	13,945.20	13,679.32	13,648.36	13,645.96	13,945.20			
A1	East :	9,290.00	9,555.88	9,529.68	9,290.00	9,290.00			
	Area (ft^2):		3,088,887.49	3,057,613.42	1,433,429.57	43,459.77	43,459.77	43,459.77	1.00
Basin I.D.	North :	13,945.20	13,645.96	13,648.84	13,653.60	13,945.20			
A2	East :	9,290.00	9,290.00	9,002.44	8,998.40	9,290.00			
	Area (ft^2):		1,389,969.80	(585,423.93)	(634,420.39)	44,307.77	44,307.77	44,307.77	1.02
									2.02
Basin I.D.	North :	13,945.20	13,653.60	13,657.64	13,945.20	13,945.20			
B1	East :	9,290.00	8,998.40	8,993.63	8,990.76	9,290.00			
	Area (ft^2):		(678,728.16)	(729,468.76)	(2,042,171.60)	44,309.23	44,309.23	44,309.23	1.02
Basin I.D.	North :	14,327.88	13,945.20	13,945.20	14,327.59	14,327.88			
B2	East :	9,290.00	9,290.00	8,990.76	8,994.59	9,290.00			
	Area (ft^2):		1,777,548.60	(308,932.22)	(2,001,215.52)	113,736.94	113,736.94	113,736.94	2.61
									3.63
Basin I.D.	North :	14,710.57	14,327.88	14,327.77	14,382.15	14,710.57			
C1	East :	9,290.00	9,290.00	8,994.59	8,990.16	9,290.00			
	Area (ft^2):		1,777,595.05	(338,193.52)	(614,492.43)	65,405.33	65,405.33	65,405.33	1.50
Basin I.D.	North :	14,710.57	14,382.15	14,384.67	14,619.95	14,710.57			
C2	East :	9,290.00	8,990.16	8,936.28	9,027.13	9,290.00			
	Area (ft^2):		(679,897.75)	(1,078,680.48)	(1,476,520.82)	36,033.05	36,033.05	36,033.05	0.83
									2.33
Basin I.D.	North :	14,619.95	14,384.67	14,380.84	14,538.29	14,619.95			
D1	East :	9,027.13	8,936.28	8,736.46	8,736.45	9,027.13			
	Area (ft^2):		397,840.34	(1,022,219.06)	(1,710,068.78)	46,217.04	46,217.04	46,217.04	1.06
Basin I.D.	North :	14,538.29	14,380.84	14,384.82	14,495.38	14,538.29			
D2	East :	8,736.45	8,736.46	8,338.68	8,449.30	8,736.45			
	Area (ft^2):		687,849.72	(2,189,741.11)	(1,855,078.94)	44,815.51	44,815.51	44,815.51	1.03
Basin I.D.	North :	14,495.38	14,384.82	14,532.47	14,495.38				
D3	East :	8,449.30	8,338.68	8,340.15	8,449.30				
	Area (ft^2):		(334,662.16)	(939,692.37)	8,085.26	8,085.26	8,085.26	8,085.26	0.19
									2.28

I.D.	North :	14,495.38	14,532.47	14,596.05	14,495.38				
	East :	8,449.30	8,340.15	8,376.29	8,449.30				
	Area (ft^2):		(947,777.63)	(950,309.27)	4,140.09	4,140.09	4,140.09	4,140.09	0.10
Basin I.D.	North :	14,538.29	14,495.38	14,596.05	14,683.07	14,538.29			
E2	East :	8,736.45	8,449.30	8,376.29	8,671.45	8,736.45			
	Area (ft^2):		(1,899,894.45)	(2,854,343.81)	(1,064,711.13)	40,214.91	40,214.91	40,214.91	0.92
Basin I.D.	North :	14,619.95	14,538.29	14,683.07	14,821.76	14,619.95			
E3	East :	9,027.13	8,736.45	8,671.45	8,904.03	9,027.13			
	Area (ft^2):		(1,756,285.82)	(2,861,211.86)	(1,755,039.35)	55,701.13	55,701.13	55,701.13	1.28
									2.30
Basin I.D.	North :	14,710.57	14,619.95	14,821.76	14,966.03	14,710.57			
F1	East :	9,290.00	9,027.13	8,904.03	9,134.18	9,290.00			
	Area (ft^2):		(1,512,553.87)	(3,323,294.34)	(2,259,972.52)	72,739.69	72,739.69	72,739.69	1.67
Basin I.D.	North :	14,710.57	14,966.03	15,190.04	15,165.30	14,710.57			
F2	East :	9,290.00	9,134.18	9,509.84	9,545.28	9,290.00			
	Area (ft^2):		(2,332,712.21)	(544,716.62)	(157,912.40)	76,651.30	76,651.30	76,651.30	1.76
									3.43
Basin I.D.	North :	14,710.57	15,165.30	15,147.92	14,710.57	14,710.57			
G1	East :	9,290.00	9,545.28	9,584.86	9,589.24	9,290.00			
	Area (ft^2):		(234,563.70)	148,506.08	2,277,649.28	76,653.80	76,653.80	76,653.80	1.76
Basin I.D.	North :	14,327.88	14,710.57	14,710.57	14,327.88	14,327.88			
G2	East :	9,290.00	9,290.00	9,589.24	9,585.41	9,290.00			
	Area (ft^2):		(1,777,595.05)	423,400.43	2,230,082.82	113,783.30	113,783.30	113,783.30	2.61
									4.37
Basin I.D.	North :	13,945.20	14,327.88	14,327.88	13,945.20	13,945.20			
H1	East :	9,290.00	9,290.00	9,585.41	9,589.24	9,290.00			
	Area (ft^2):		(1,777,548.60)	338,750.92	2,200,261.15	113,780.33	113,780.33	113,780.33	2.61
Basin I.D.	North :	13,945.20	13,945.20	13,705.52	13,679.32	13,945.20			
H2	East :	9,290.00	9,589.24	9,586.84	9,555.88	9,290.00			
	Area (ft^2):		2,086,480.82	3,218,921.11	3,132,347.26	43,459.77	43,459.77	43,459.77	1.00
									3.61

• Calculate the hydraulic length for each area:

basin	hydraulic length (ft)
A	296
B	675
C	440
D	≈500
E	533
F	504
G	675
H	675

• Calculate the time of concentration
Using SCS curve number methodology

$t_c = t_i + t_t$ where t_c = time concentration
t_i = overland flow time
t_t = travel time in ditch

$$t_i = \frac{1.8(1.1 - C_5)\sqrt{L}}{\sqrt[3]{S}}$$ where C_5 = runoff coeff for 5 year frequency (.25)
L = length overland flow
S = avg basin slope

$$t_t = \frac{\text{ditch flow length}}{\text{velocity}}$$ (assume velocity of 2 ft/sec)

(The above was obtained from "Urban Storm Drainage Criteria Manual - Denver Regional Council of Government, Wright McLaughlin Engineers, 1984)

HA&L ENGINEERING

CLIENT USPCI
PROJECT Landfill Cell 15 - Modified Closure
FEATURE Cap Hydrology
PROJECT NO. 124-44-300

SHEET 6 OF 24
COMPUTED PGH
CHECKED KCS
DATE 5/20/93

REVISED 5/23/96

BASIN	OVERLAND FLOW LENGTH (FT)	DITCH FLOW LENGTH (FT)	TOTAL LENGTH (FT)	OVERLAND FLOW TIME (MIN)	DITCH TRAVEL TIME (MIN)	TIME OF CONC. (MIN)	TIME OF CONC. (HRS)
A	296	0	296	12.2	0.0	12.2	0.20
B	292	383	675	12.1	3.2	15.3	0.26
C	0	440	440	0.0	3.7	3.7	0.06
D	154	346	500	8.8	2.9	11.7	0.19
E	233	300	533	10.8	2.5	13.3	0.22
F	233	271	504	10.8	2.3	13.1	0.22
G	292	383	675	12.1	3.2	15.3	0.26
H	292	383	675	12.1	3.2	15.3	0.26

Curve Number: The cap consists of a 6" layer of riprap overlying 4" of granular filter which is over 2' of soil cover. Assume a hydrologic soil group A. Based on the attached tables, and using CN for roads = 72 + 76. Use CN=75.

The peak flow is then calculated, based on the above parameters, using an in-house developed computer program called "Hydro", which is based on the SCS methodology. See attached printouts.

Area	Peak Flow (cfs)
A	9.61
B	16.78
C	11.68
D	10.84
E	10.85
F	16.18
G	20.20
H	16.69

(i) *Terracing.*—Terraces may be graded, open-end level, or closed-end level. The effects of graded and open-end level terraces are considered in table A–2, and the effects of both contouring and the grass waterway outlets are included.

Closed-end level terraces should be handled like contour furrows.

A-4. *Hydrologic Soil-Cover Complexes.*—(a) *Purpose.*—Table A–2 combines soil groups and land use and treatment classes into *hydrologic soil-cover complexes.* The numbers show the relative value of the complexes as direct runoff-producers (see sec. A–5). The higher the number, the greater the amount of direct runoff to be expected from a storm.

(b) *Table A–2.*—Table A–2 was prepared using data from gaged watershed with known soils and cover. Storm rainfall appropriate for antecedent moisture condition[4] AMC-II was plotted versus direct runoff for annual floods for respective watersheds of one soil group and one cover type. The curve from figure A–4 best fitting the plotted points was determined and the curve numbers for table A–2 obtained. Related curve numbers for above average (AMC-III) and below average (AMC-I) points were similarly developed and are shown in table A–7 next to the CN values for AMC-II.

For several of the soil-cover complexes shown in table A–2, curve numbers (CN) were estimated or computed from relations developed by the Soil Conservation Service since hydrologic data were not available for all given soil-cover complexes.

(c) *Forest Service Procedure.*—Chapter 4 of "Forest and Range Hydrology Handbook," U.S. Forest Service, Washington, D.C., 1959, describes how CN are determined for national and commercial forests in the eastern United States. Section 1 of "Handbook on Methods of Hydrologic Analysis," U.S. Forest Service, Washington, D.C., 1959, describes how CN are determined for forest-range regions in the western United States. Selections from these handbooks, which are included in the Soil Conservation Service National Engineering Handbook, issued in 1964, are given here.

[4] Antecedent moisture conditions are defined in section A-5(a).

TABLE A–2.—*Runoff curve numbers (CN) for hydrologic soil-cover complexes*

(FOR WATERSHED CONDITION AMC-II AND I_a=0.2S)

Land use or cover	Treatment or practice	Hydrologic condition for infiltrating	Hydrologic soil group A	B	C	D
Fallow	SR		77	86	91	94
Row crops	SR	Poor	72	81	88	91
	SR	Good	67	78	85	[illegible]
	C	Poor	70	79	84	[illegible]
	C	Good	65	75	82	[illegible]
	C&T	Poor	66	74	80	[illegible]
	C&T	Good	62	71	78	81
Small grain	SR	Poor	65	76	84	88
	SR	Good	63	75	83	87
	C	Poor	63	74	82	85
	C	Good	61	73	81	84
	C&T	Poor	61	72	79	82
	C&T	Good	59	70	78	81
Close-seeded legumes[1] or rotation meadow.	SR	Poor	66	77	85	[illegible]
	SR	Good	58	72	81	[illegible]
	C	Poor	64	75	83	[illegible]
	C	Good	55	69	78	[illegible]
	C&T	Poor	63	73	80	83
	C&T	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	C	Poor	47	67	81	88
	C	Fair	25	59	75	83
	C	Good	6	35	70	79
Meadow (permanent).		do	30	58	71	[illegible]
Woods (farm woodlots).		Poor	45	66	77	[illegible]
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads (dirt)[2]			72	82	87	89
(hard surface).[2]			74	84	90	92

[1] Close-drilled or broadcast.
[2] Including right-of-way.
[3] See sec. A–5.
SR = Straight row.
C = Contoured.
T = Terraced.
C&T = Contoured and terraced.

(U.S. Soil Conservation Service)

(1) *Forest in Eastern United States.*—In the humid forest regions of the eastern United States, soil group, humus type, and humus depth are the principal factors used in the Forest Service method of determining CN. The undecomposed leaves or needles, twigs, bark, and other vegetative debris on the forest floor form the litter from which humus is derived. Natural litter protects humus from oxidation and therefore indirectly enters into

o/k

Table 2-2a.—Runoff curve numbers for urban areas[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1] Average runoff condition, and $I_a = 0.2S$.
[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

PROJECT : USPCI - LANDFILL CELL 15 CLOSURE, AREA A, 100-YR, 24-HR

AREA= 2.0 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 296. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .20 HOURS

TP= .1333 HOURS QPCFS= 11.46 CFS QPIN= 5.6248 INCHES
C3= 27.7246 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.16	.6656	.0000	.0000	.0	.00
6.19	.6699	.0000	.0000	.6	.00
6.21	.6741	.0000	.0000	3.6	.00
6.24	.6784	.0000	.0000	7.6	.00
6.27	.6827	.0000	.0000	10.5	.00
6.29	.6869	.0001	.0000	11.5	.00
6.32	.6912	.0002	.0000	10.7	.00
6.35	.6955	.0002	.0000	9.1	.00
6.37	.6997	.0003	.0000	7.1	.00
6.40	.7040	.0004	.0000	5.2	.00
6.43	.7083	.0005	.0000	3.7	.00
6.45	.7125	.0006	.0001	2.5	.00
6.48	.7168	.0007	.0001	1.6	.00
6.51	.7211	.0009	.0001	1.1	.00
6.53	.7253	.0010	.0001	.7	.00
6.56	.7296	.0012	.0002	.4	.00
6.59	.7339	.0013	.0002	.2	.00
6.61	.7381	.0015	.0002	.1	.00
6.64	.7424	.0017	.0002	.0	.00
11.87	4.4933	2.0451	.1262	.0	8.67
11.89	4.6554	2.1729	.1278	.0	8.88
11.92	4.8175	2.3021	.1293	.0	9.08
11.95	4.9797	2.4328	.1307	.0	9.25
11.97	5.1418	2.5648	.1320	.0	9.40
12.00	5.3039	2.6979	.1332	.0	9.54
12.03	5.3347	2.7234	.0254	.0	9.61
12.05	5.3654	2.7488	.0254	.0	9.34
12.08	5.3961	2.7742	.0254	.0	8.62
12.11	5.4269	2.7997	.0255	.0	7.56
12.13	5.4576	2.8252	.0255	.0	6.39
12.16	5.4883	2.8508	.0256	.0	5.28
12.19	5.5190	2.8764	.0256	.0	4.34

HYDROGRAPH PEAK= 9.61 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= .85 Acre-Feet

PROJECT : USPCI - LANDFILL CELL 15 CLOSURE, AREA B, 100-YR 24-HR

AREA= 3.6 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 675. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .26 HOURS

TP= .1733 HOURS QPCFS= 15.84 CFS QPIN= 4.3268 INCHES
C3= 21.3266 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.14	.6618	.0000	.0000	.0	.00
6.17	.6673	.0000	.0000	.8	.00
6.21	.6729	.0000	.0000	4.9	.00
6.24	.6784	.0000	.0000	10.5	.00
6.27	.6839	.0000	.0000	14.5	.00
6.31	.6895	.0002	.0000	15.8	.00
6.34	.6950	.0002	.0000	14.8	.00
6.38	.7006	.0003	.0001	12.5	.00
6.41	.7061	.0005	.0001	9.8	.00
6.45	.7117	.0006	.0001	7.2	.00
6.48	.7172	.0008	.0002	5.1	.00
6.52	.7228	.0009	.0002	3.5	.00
6.55	.7283	.0011	.0002	2.3	.01
6.59	.7339	.0013	.0002	1.5	.01
6.62	.7394	.0016	.0002	.9	.01
6.66	.7450	.0018	.0002	.6	.02
6.69	.7505	.0021	.0003	.3	.02
6.73	.7561	.0023	.0003	.2	.02
6.76	.7616	.0026	.0003	.1	.02
6.79	.7671	.0029	.0003	.0	.02
11.82	4.2179	1.8318	.1598	.0	13.15
11.86	4.4287	1.9946	.1628	.0	14.09
11.89	4.6394	2.1602	.1656	.0	14.86
11.93	4.8502	2.3284	.1681	.0	15.50
11.96	5.0610	2.4988	.1705	.0	16.03
11.99	5.2718	2.6714	.1726	.0	16.48
12.03	5.3378	2.7259	.0545	.0	16.78
12.06	5.3778	2.7590	.0330	.0	16.52
12.10	5.4177	2.7921	.0331	.0	15.44
12.13	5.4576	2.8253	.0332	.0	13.72
12.17	5.4976	2.8585	.0332	.0	11.70
12.20	5.5375	2.8918	.0333	.0	9.73
12.24	5.5774	2.9252	.0334	.0	8.01

HYDROGRAPH PEAK= 16.78 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= 1.52 Acre-Feet

11/24

PROJECT : USPCI - CELL 15 CLOSURE, AREA C, 100-YR 24-HR

AREA= 2.3 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 440. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .06 HOURS

TP= .0400 HOURS QPCFS= 44.05 CFS QPIN=18.7493 INCHES
C3= 92.4154 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
				.0	.00
6.16	.6656	.0000	.0000	21.6	.00
6.18	.6688	.0000	.0000	44.1	.00
6.20	.6720	.0000	.0000	31.1	.00
6.22	.6752	.0000	.0000	14.2	.00
6.24	.6784	.0000	.0000	5.1	.00
6.26	.6816	.0000	.0000	1.6	.00
6.28	.6848	.0000	.0000	.4	.00
6.30	.6880	.0001	.0000	.1	.00
6.32	.6912	.0002	.0000	.0	.00
6.34	.6944	.0002	.0000		
				.0	11.10
11.88	4.5749	2.1093	.0954	.0	11.21
11.90	4.6965	2.2055	.0963	.0	11.31
11.92	4.8181	2.3026	.0971	.0	11.41
11.94	4.9397	2.4005	.0979	.0	11.51
11.96	5.0613	2.4991	.0986	.0	11.60
11.98	5.1829	2.5984	.0993	.0	11.68
12.00	5.3041	2.6981	.0997	.0	9.99
12.02	5.3271	2.7171	.0190	.0	6.46
12.04	5.3502	2.7361	.0190	.0	3.97
12.06	5.3732	2.7552	.0191	.0	2.83
12.08	5.3963	2.7743	.0191	.0	2.43
12.10	5.4193	2.7934	.0191	.0	2.30
12.12	5.4423	2.8126	.0191		

HYDROGRAPH PEAK= 11.68 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= .98 Acre-Feet

12/24

PROJECT : USPCI - CELL 15 CLOSURE, AREA D, 100-YR 24-HR

AREA= 2.3 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 500. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .20 HOURS

TP= .1333 HOURS QPCFS= 12.93 CFS QPIN= 5.6248 INCHES
C3= 27.7246 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.16	.6656	.0000	.0000	.0	.00
6.19	.6699	.0000	.0000	.6	.00
6.21	.6741	.0000	.0000	4.0	.00
6.24	.6784	.0000	.0000	8.6	.00
6.27	.6827	.0000	.0000	11.9	.00
6.29	.6869	.0001	.0000	12.9	.00
6.32	.6912	.0002	.0000	12.1	.00
6.35	.6955	.0002	.0000	10.2	.00
6.37	.6997	.0003	.0000	8.0	.00
6.40	.7040	.0004	.0000	5.9	.00
6.43	.7083	.0005	.0000	4.2	.00
6.45	.7125	.0006	.0001	2.8	.00
6.48	.7168	.0007	.0001	1.9	.00
6.51	.7211	.0009	.0001	1.2	.00
6.53	.7253	.0010	.0001	.7	.00
6.56	.7296	.0012	.0002	.5	.00
6.59	.7339	.0013	.0002	.3	.00
6.61	.7381	.0015	.0002	.2	.01
6.64	.7424	.0017	.0002	.0	.01
11.87	4.4933	2.0451	.1262	.0	9.78
11.89	4.6554	2.1729	.1278	.0	10.03
11.92	4.8175	2.3021	.1293	.0	10.24
11.95	4.9797	2.4328	.1307	.0	10.44
11.97	5.1418	2.5648	.1320	.0	10.61
12.00	5.3039	2.6979	.1332	.0	10.77
12.03	5.3347	2.7234	.0254	.0	10.84
12.05	5.3654	2.7488	.0254	.0	10.54
12.08	5.3961	2.7742	.0254	.0	9.72
12.11	5.4269	2.7997	.0255	.0	8.54
12.13	5.4576	2.8252	.0255	.0	7.21
12.16	5.4883	2.8508	.0256	.0	5.96
12.19	5.5190	2.8764	.0256	.0	4.89

HYDROGRAPH PEAK= 10.84 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= .96 Acre-Feet

PROJECT : USPCI - CELL 15 CLOSURE, AREA E, 100-YR 24-HR

AREA= 2.3 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 533. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .22 HOURS

TP= .1467 HOURS QPCFS= 11.86 CFS QPIN= 5.1135 INCHES
C3= 25.2042 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.16	.6656	.0000	.0000	.0	.00
6.19	.6703	.0000	.0000	.6	.00
6.22	.6750	.0000	.0000	3.7	.00
6.25	.6797	.0000	.0000	7.9	.00
6.28	.6844	.0000	.0000	10.9	.00
6.31	.6891	.0001	.0000	11.9	.00
6.34	.6938	.0002	.0000	11.1	.00
6.37	.6985	.0003	.0000	9.4	.00
6.39	.7031	.0004	.0000	7.3	.00
6.42	.7078	.0005	.0001	5.4	.00
6.45	.7125	.0006	.0001	3.8	.00
6.48	.7172	.0008	.0001	2.6	.00
6.51	.7219	.0009	.0001	1.7	.00
6.54	.7266	.0011	.0002	1.1	.00
6.57	.7313	.0012	.0002	.7	.00
6.60	.7360	.0014	.0002	.4	.00
6.63	.7407	.0016	.0002	.3	.01
6.66	.7454	.0018	.0002	.2	.01
6.69	.7501	.0020	.0002	.0	.01
11.85	4.3957	1.9690	.1376	.0	9.45
11.88	4.5741	2.1086	.1396	.0	9.79
11.91	4.7524	2.2500	.1415	.0	10.08
11.94	4.9307	2.3932	.1432	.0	10.33
11.97	5.1091	2.5380	.1448	.0	10.54
12.00	5.2874	2.6843	.1463	.0	10.74
12.03	5.3347	2.7233	.0390	.0	10.85
12.06	5.3684	2.7513	.0280	.0	10.60
12.09	5.4022	2.7793	.0280	.0	9.84
12.11	5.4360	2.8073	.0280	.0	8.69
12.14	5.4698	2.8354	.0281	.0	7.38
12.17	5.5036	2.8636	.0281	.0	6.11
12.20	5.5374	2.8917	.0282	.0	5.03

HYDROGRAPH PEAK= 10.85 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= .97 Acre-Feet

PROJECT : USPCI - CELL 15 CLOSURE, AREA F, 100-YR 24-HR

AREA= 3.4 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 504. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .22 HOURS

TP= .1467 HOURS QPCFS= 17.69 CFS QPIN= 5.1135 INCHES
C3= 25.2042 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.16	.6656	.0000	.0000	.0	.00
6.19	.6703	.0000	.0000	.9	.00
6.22	.6750	.0000	.0000	5.5	.00
6.25	.6797	.0000	.0000	11.7	.00
6.28	.6844	.0000	.0000	16.2	.00
6.31	.6891	.0001	.0000	17.7	.00
6.34	.6938	.0002	.0000	16.6	.00
6.37	.6985	.0003	.0000	14.0	.00
6.39	.7031	.0004	.0000	10.9	.00
6.42	.7078	.0005	.0001	8.1	.00
6.45	.7125	.0006	.0001	5.7	.00
6.48	.7172	.0008	.0001	3.9	.00
6.51	.7219	.0009	.0001	2.5	.00
6.54	.7266	.0011	.0002	1.6	.01
6.57	.7313	.0012	.0002	1.0	.01
6.60	.7360	.0014	.0002	.6	.01
6.63	.7407	.0016	.0002	.4	.02
6.66	.7454	.0018	.0002	.2	.02
6.69	.7501	.0020	.0002	.1	.02
6.72	.7548	.0023	.0002	.0	.02
11.85	4.3957	1.9690	.1376	.0	14.10
11.88	4.5741	2.1086	.1396	.0	14.60
11.91	4.7524	2.2500	.1415	.0	15.03
11.94	4.9307	2.3932	.1432	.0	15.40
11.97	5.1091	2.5380	.1448	.0	15.72
12.00	5.2874	2.6843	.1463	.0	16.01
12.03	5.3347	2.7233	.0390	.0	16.18
12.06	5.3684	2.7513	.0280	.0	15.81
12.09	5.4022	2.7793	.0280	.0	14.68
12.11	5.4360	2.8073	.0280	.0	12.97
12.14	5.4698	2.8354	.0281	.0	11.01
12.17	5.5036	2.8636	.0281	.0	9.13
12.20	5.5374	2.8917	.0282	.0	7.51

HYDROGRAPH PEAK= 16.18 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= 1.44 Acre-Feet

PROJECT : USPCI - CELL 15 CLOSURE, AREA G, 100-YR 24-HR

AREA= 4.4 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 675. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .26 HOURS

TP= .1733 HOURS QPCFS= 19.07 CFS QPIN= 4.3268 INCHES
CS= 21.3266 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.14	.6618	.0000	.0000	.0	.00
6.17	.6673	.0000	.0000	1.0	.00
6.21	.6729	.0000	.0000	5.9	.00
6.24	.6784	.0000	.0000	12.7	.00
6.27	.6839	.0000	.0000	17.5	.00
6.31	.6895	.0002	.0000	19.1	.00
6.34	.6950	.0002	.0000	17.9	.00
6.38	.7006	.0003	.0001	15.1	.00
6.41	.7061	.0005	.0001	11.8	.00
6.45	.7117	.0006	.0001	8.7	.00
6.48	.7172	.0008	.0002	6.1	.00
6.52	.7228	.0009	.0002	4.2	.01
6.55	.7283	.0011	.0002	2.7	.01
6.59	.7339	.0013	.0002	1.8	.01
6.62	.7394	.0016	.0002	1.1	.02
6.66	.7450	.0018	.0002	.7	.02
6.69	.7505	.0021	.0003	.4	.02
6.73	.7561	.0023	.0003	.2	.02
6.76	.7616	.0026	.0003	.1	.03
6.79	.7671	.0029	.0003	.0	.03
11.82	4.2179	1.8318	.1598	.0	15.84
11.86	4.4287	1.9946	.1628	.0	16.97
11.89	4.6394	2.1602	.1656	.0	17.90
11.93	4.8502	2.3284	.1681	.0	18.66
11.96	5.0610	2.4988	.1705	.0	19.30
11.99	5.2718	2.6714	.1726	.0	19.85
12.03	5.3378	2.7259	.0545	.0	20.20
12.06	5.3778	2.7590	.0330	.0	19.88
12.10	5.4177	2.7921	.0331	.0	18.59
12.13	5.4576	2.8253	.0332	.0	16.51
12.17	5.4976	2.8585	.0332	.0	14.09
12.20	5.5375	2.8918	.0333	.0	11.71
12.24	5.5774	2.9252	.0334	.0	9.65

HYDROGRAPH PEAK= 20.20 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= 1.84 Acre-Feet

16/24

PROJECT : USPCI - CELL 15 CLOSURE, AREA H, 100-YR 24-HR

AREA= 3.6 ACRES
AVERAGE BASIN SLOPE= 10.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 675. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .26 HOURS

TP= .1733 HOURS QPCFS= 15.75 CFS QPIN= 4.3268 INCHES
C3= 21.3266 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.14	.6618	.0000	.0000	.0	.00
6.17	.6673	.0000	.0000	.8	.00
6.21	.6729	.0000	.0000	4.9	.00
6.24	.6784	.0000	.0000	10.5	.00
6.27	.6839	.0000	.0000	14.5	.00
6.31	.6895	.0002	.0000	15.8	.00
6.34	.6950	.0002	.0000	14.8	.00
6.38	.7006	.0003	.0001	12.5	.00
6.41	.7061	.0005	.0001	9.7	.00
6.45	.7117	.0006	.0001	7.2	.00
6.48	.7172	.0008	.0002	5.1	.00
6.52	.7228	.0009	.0002	3.4	.00
6.55	.7283	.0011	.0002	2.3	.01
6.59	.7339	.0013	.0002	1.5	.01
6.62	.7394	.0016	.0002	.9	.01
6.66	.7450	.0018	.0002	.6	.02
6.69	.7505	.0021	.0003	.3	.02
6.73	.7561	.0023	.0003	.2	.02
6.76	.7616	.0026	.0003	.1	.02
6.79	.7671	.0029	.0003	.0	.02
11.82	4.2179	1.8318	.1598	.0	13.08
11.86	4.4287	1.9946	.1628	.0	14.02
11.89	4.6394	2.1602	.1656	.0	14.78
11.93	4.8502	2.3284	.1681	.0	15.41
11.96	5.0610	2.4988	.1705	.0	15.94
11.99	5.2718	2.6714	.1726	.0	16.39
12.03	5.3378	2.7259	.0545	.0	16.69
12.06	5.3778	2.7590	.0330	.0	16.42
12.10	5.4177	2.7921	.0331	.0	15.36
12.13	5.4576	2.8253	.0332	.0	13.64
12.17	5.4976	2.8585	.0332	.0	11.64
12.20	5.5375	2.8918	.0333	.0	9.67
12.24	5.5774	2.9252	.0334	.0	7.97

HYDROGRAPH PEAK= 16.69 cfs
TIME TO PEAK= 12.03 Hours
RUNOFF VOLUME= 1.52 Acre-Feet

HA&L ENGINEERING

CLIENT USPCI
PROJECT Landfill Cell 15 - Modified Closure
FEATURE
PROJECT NO.

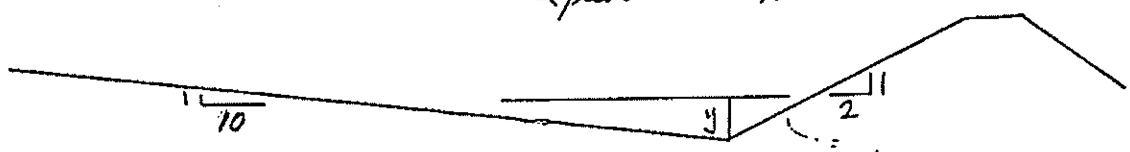
COMPUTED PGH
CHECKED KCS
DATE

Revised 5/23/96

* Determine flow depth in cap drainage ditches.

assume the following cross-section ~ also assume maximum flow Q (conservative.)

$Q_{peak} = 20.20$ cfs



Solve the depth of flow for the above cross section using mannings equation

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Using nickuradse equation to determine n value.

$$n = 0.0395 (D_{50})^{1/6} \quad \text{where } D_{50} = \text{mean dia. riprap} = 4"$$

$$\therefore n = 0.0395 \left(\frac{4}{12}\right)^{1/6} = 0.033$$

Solving for "y" above yields the following

Trapezoidal Channel Flow Calculations using Mannings Equation

Client :	USPCI - CELL 15 CLOSURE	Date :	23-May-96
Project No. :	64.44.710	Time :	10:55 AM
Channel Section:	MAJOR CAP STORM DRAINAGE DITCH	Compute	MEA

			UNITS
GENERAL CRITERIA:	Design Flow:	20.20	cfs
	Bottom Width:	0.0	feet
	Side Slope1:	10.0	1/m1
	Side Slope2:	2.0	1/m2
	Friction Factor:		
	Assumed D50:	0.33	feet
	Calc n Value:	0.033	
	Used:	0.033	
	Min. Bottom Slope:	0.0050	ft/ft
	Max. Bottom Slope:	0.0050	ft/ft
	Freeboard:	0.50	feet
CALCULATION: (Channel Depth)	Depth (Min. S):	1.22	feet ← Y_{max}
	Q-1.49AR(2/3)S(1/2)/n=	0.000	Accuracy
	Required Depth:	1.72	feet
	Area:	8.93	ft2
	Perimeter:	14.99	feet
	Hydraulic Radius:	0.60	feet
	Velocity:	2.26	ft/sec ← $V_{max} = 2.3$ fps OK
	Riprap Ck (V<5?):	Not Needed	

Actual depth, of ditch = 2.83'
Effective depth = 2.0'
Freeboard ≈ 0.8'

Revised 5/23/96

* Determine Inlet box and Downspout design

Assume all inlets to be designed based on maximum peak flow.

$Q_{design} = 20.20$ cfs

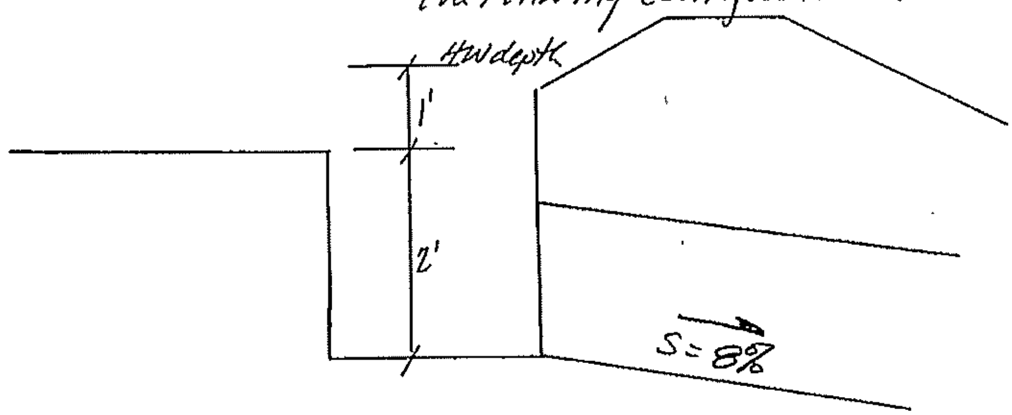
A.) Use two - 18" dia pipes.

$\therefore Q_{pipe} = \frac{20.20}{2} = 10.10$ cfs

Assume the pipe inverts will be set @ 2' below the flow line of the ditch, also assume that 1' minimum freeboard is required.

$\therefore$ AHW = 3'

with this analysis assume the following configuration:



Using Mannings Equation, determine the capacity of a single 18" dia pipe to carry 10.10 cfs

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q) =	10.10	cfs
Manning n (n) =	0.024	
Pipe Diameter (d) =	1.5	feet
Slope (So) =	0.08	
Normal Depth (y) =	0.860	feet ← 18" Pipe Open Channel Flow
Flow x-section area (A) =	1.048	sq. ft.
Flow Top Width (T) =	1.484	feet
Perimeter (P) =	2.577	feet
Hyd. Radius (R) =	0.407	feet
Flow Velocity (V) =	9.638	ft/sec.
Froude Number =	2.021	
Theta =	3.436	radians

Evaluate the head requirements for the inlet box. Use the above referenced "Urban Storm Drainage Criteria Manual".

The procedure is as follows:

1. Assume the flow to be open channel once it enters the 18" pipes. Also assume that critical depth occurs near the pipe inlet. The critical flow conditions for D=18", Q=10.10 cfs are as follows.

CRITICAL FLOW CONDITIONS

Critical Depth (yc) =	1.224	feet
Critical area (Ac) =	1.544	sq. ft.
Top Width (Tc) =	1.162	feet
Perimeter (Pc) =	3.383	feet
Hyd. Radius (Rc) =	0.456	feet
Flow Velocity (Vc) =	6.541	ft/sec.
Froude Number =	1.000	
Theta =	4.511	radians

2. The pressure line at the pipe inlet equals:

$$= y + \frac{V^2}{2g}$$

Given above data, v @ critical depth = 6.5 ft/sec

$$\therefore \quad = 1.224 + \frac{(6.5)^2}{64.4} = 1.88 \text{ feet}$$

3. Estimated water depth d in the box, assuming side flow conditions

$$d = y + K_g \frac{V^2}{2g} \qquad \text{assume } K_g \text{ initial} = 3.3$$

$$d = 1.224 + 3.3\left(\frac{6.5^2}{64.4}\right) = 3.39$$

4.) Calculate ratio headwater depth to pipe diameter

$$\frac{3.39}{1.5} = 2.26$$

5.) from figure 8-6 (following page), with $\frac{d}{D} = 2.26 \Rightarrow$

$$K_g = 4.2$$

HA&L ENGINEERING

CLIENT	USPCI	SHEET	20 OF 24
PROJECT	Landfill Cell 15 - Modified Closure	COMPUTED	PGH
FEATURE	Cap Hydrology	CHECKED	KCS
PROJECT NO.	64-44-308	DATE	5/20/93

Revised 5/23/96

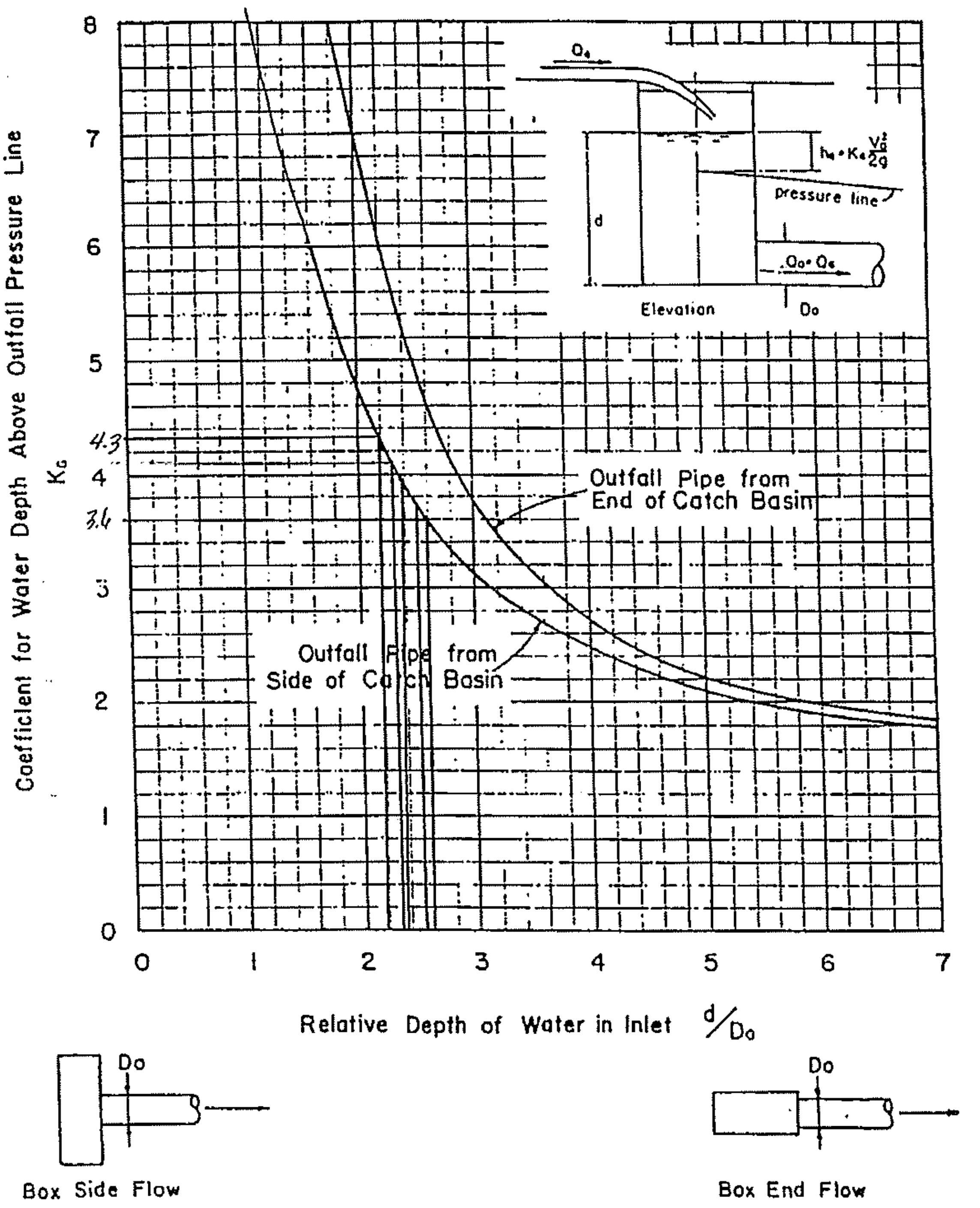


FIGURE 8-6. CATCH BASIN WITH INLET FLOW ONLY (15)

HA&L ENGINEERING

CLIENT USPCI
PROJECT Landfill Cell 15 - modified closure
FEATURE Cap hydrology
PROJECT NO. 21-44-3801

SHEET 21 OF 24
COMPUTED PGH
CHECKED KCS
DATE 5/20/93

Revised 5/23/96

6.) Calculate pressure
$$1.224 + 4.2\left(\frac{6.5^2}{64.4}\right) \Rightarrow 3.98$$

7.) $\frac{d}{D} = \frac{3.98}{1.5} = 2.65$

8.) from figure 8-6 $\Rightarrow k_g = 3.5$

9.) Calculate pressure
$$1.224 + 3.5\left(\frac{6.5^2}{64.4}\right) \Rightarrow 3.52'$$

10) $\frac{d}{D} = \frac{3.52}{1.5} = 2.35$

11.) from figure 8-6 $\Rightarrow k_g = 4.0$

12.) Calculate pressure
$$1.224 + 4.0\left(\frac{6.5^2}{64.4}\right) \Rightarrow 3.85$$

13.) $\frac{d}{D} = \frac{3.85}{1.5} = 2.57$

14.) from figure 8-6 $\Rightarrow k_g = 3.7$

15) Calculate pressure
$$1.224 + 3.7\left(\frac{6.5^2}{64.4}\right) = 3.65$$

16.) $\frac{3.65}{1.5} = 2.43$

17.) from figure 8-6 $\Rightarrow k_g = 3.82$

18.) Calculate pressure
$$1.224 + 3.82\left(\frac{6.5^2}{64.4}\right) \Rightarrow 3.73$$

19.) $\frac{3.73}{1.5} = 2.49$

20.) from figure 8-6 $\Rightarrow k_g = 3.75$

21) Calculate pressure
$$1.224 + 3.75\left(\frac{6.5^2}{64.4}\right) \Rightarrow 3.68$$

22.) $\frac{3.68}{1.5} = 2.45 \Rightarrow k_g = 3.75$ $\therefore y = 3.68'$

HA&L ENGINEERING

CLIENT USPCI
PROJECT Landfill Cell II - Modified Closure
FEATURE Cap Hydrology
PROJECT NO. 64-44-380

SHEET 22 OF 24
COMPUTED RGH
CHECKED KLS
DATE 5/20/93

Revised 5/23/96

Note: the above calculations indicate that the required HW is higher than allowed (given the desire to maintain freeboard.) ∴ Use different inlet box.

- Analyze head requirements for a USBR Type IV box. (as per "Design of Small canal structures" USBR 1978.

See attached sheet for illustration of Type IV inlet

Based on the information presented in the USBR design handbook, the head required to allow passage of the design flow (assuming free flow conditions) is as follows:

$h = 0.0433V^2$ where h = head measured from ℄ of opening
V = design velocity

Assuming $V = 6.5$ ft/sec (critical flow) conservative

$h = 0.0433(6.5)^2 = 1.83$ feet above ℄

The total head equals h + radius of pipe

$\therefore h_{total} = 1.83 + \frac{1.5}{2} \Rightarrow 2.58$ feet.

Since the invert will be set 2' below the flowline of the cap drainage ditches, the head required is only 0.58'. Note that this is lower than the normal flow depth required to pass Q_{total}. Therefore, there will not be any anticipated backing of water in the ditches due inlet conditions. ✓

Also, the freeboard in the ditches will remain in excess of 1 foot. OK

∴ at downspouts b, f, g + h use 2-18" dia. downspouts with a Type IV USBR inlet.

HA&L ENGINEERING	CLIENT USPCI PROJECT Landfill Cell 15 - Modified Closure FEATURE Rev Hydrology PROJECT NO. 64-44-380		SHEET 23 OF 24 COMPUTED PGH CHECKED KLS DATE 5/20/93

Revised 5/23/96

Check flow conditions @ outlets a, c, d and e where a single 18" dia. downspout may be used.

Q_{max} = 11.68 cfs (@ downspout c)

Determine flow characteristics for 18" pipe where Q = 11.68 cfs, S = 8%, n = 0.024

as shown on attached printout, y ≃ 0.95 ft.

- Analyze head requirements using USBR Type IV inlet

$h = 0.0433\,V^2$ where $V_{max} = 7.18$ ft/sec

$h = 0.0433(7.18)^2$

$h = 2.23$

$h_{total} = 2.23 + \frac{1.5}{2} = 2.98'$ OK

There will still be adequate depth and freeboard for these conditions.

Summary:

Downspout Number	Qpeak	# Downspouts
A	9.61 cfs	1
B	16.78 "	2
C	11.68 "	1
D	10.84 "	1
E	10.85 "	1
F	16.18 "	2
G	20.20 "	2
H	16.69 "	2

24/24

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	11.68 cfs
Manning n (n)	=	0.024
Pipe Diameter (d)	=	1.5 feet
Slope (So)	=	0.08
Normal Depth (y)	=	0.946 feet OK
Flow x-section area (A)	=	1.174 sq. ft.
Flow Top Width (T)	=	1.448 feet
Perimeter (P)	=	2.752 feet
Hyd. Radius (R)	=	0.427 feet
Flow Velocity (V)	=	9.950 ft/sec.
Froude Number	=	1.947
Theta	=	3.670 radians
Solve Equation	=	-0.000

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	1.301 feet
Critical area (Ac)	=	1.628 sq. ft.
Top Width (Tc)	=	1.018 feet
Perimeter (Pc)	=	3.593 feet
Hyd. Radius (Rc)	=	0.453 feet
Flow Velocity (Vc)	=	7.175 ft/sec.
Froude Number	=	1.000
Theta	=	4.791 radians

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Landfill Cell 15 - Modified Closure
FEATURE: Storm Drains
PROJECT NO.: 64-44-380

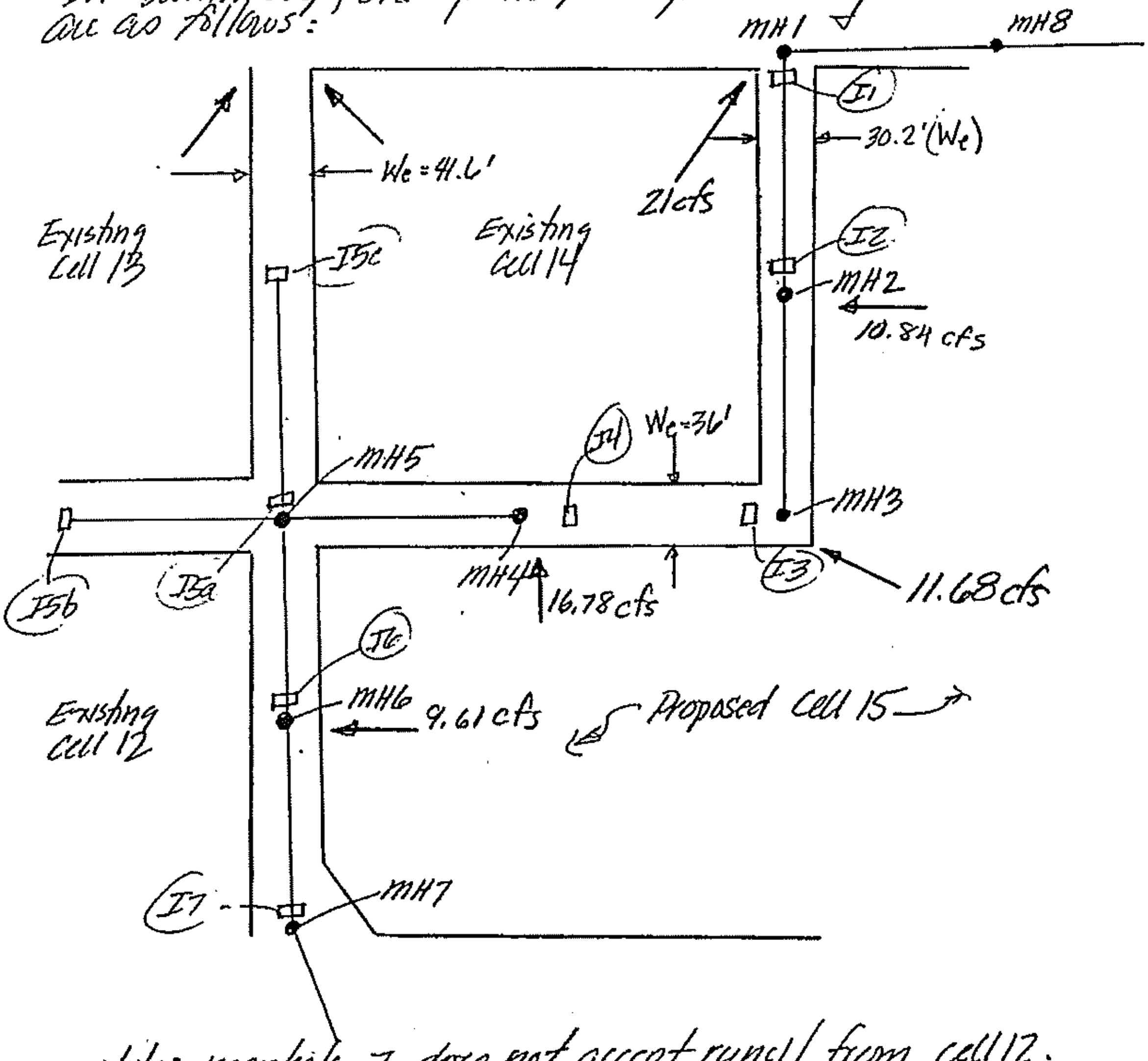
SHEET 1 OF 12
COMPUTED: PGH
CHECKED: KCS
DATE: 5/20/93
Rev. 5/24/96

Purpose: Design storm Drains associated w/ landfill Cell 15 of the USPCI Lone Mtn. Facility.

The storm drains will be designed to convey runoff generated from the embankment tops and closure caps.

Runoff generated from the closure caps is contained in calculations by PGH dated 5/20/93 and entitled Cap Hydrology, and previous work on cells 12, 13 + 14 Closure

In summary, the peak flows previously determined are as follows:



Note: manhole 7 does not accept runoff from cell 12. Cell 12 has independent drainage pipes.

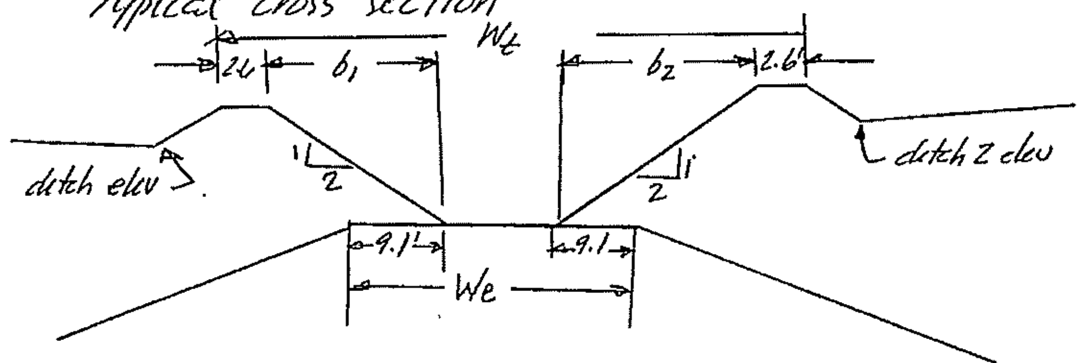
HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Landfill Cell 15 - Modified Closure
FEATURE: Storm Drains
PROJECT NO.: 64-44-300

SHEET 2 OF 12
COMPUTED: RGH
CHECKED: JCS
DATE: 5/20/93
Rev. 5/24/96

A.) Determine the run-off area between cells

Typical cross section



$$b = [(\text{ditch elev} + 2.83) - 1420](2)$$

$$W_t = W_e - 2(9.1) + b_1 + b_2 + 2(2.6)$$

The drainage area to each inlet equals W_t times the interval spacing between inlets. (Summarized below)

Inlet Number	Average Ditch Elev. #1	Average Ditch Elev. #2	B1	B2	We	Wt	Length	Area (acres)
I1	24.5	29.3	14.86	24.26	30.2	56.32	460	0.59
I2	26.1	27.3	17.86	20.26	30.2	55.32	330	0.42
I3	27.5	27.3	20.66	20.26	30.2	58.12	150	0.20
	27.5	27.3	20.66	20.26	36.0	63.92	175	0.26
I4	29.2	27.3	24.06	20.26	36.0	67.32	350	0.54
I5a	27.5	28.4	20.66	22.46	36.0	66.12	175	0.27
	27.5	27.5	20.66	20.66	41.6	69.92	155	0.25
	27.5	27.5	20.66	20.66	36.0	64.32	180	0.27
	27.5	28.4	20.66	22.46	30.2	60.32	150	0.21
I5b	29.2	29.2	24.06	24.06	36.0	71.12	360	0.59
I5c	26.1	26.1	17.86	17.86	41.6	64.32	210	0.31
I6	26.1	27.3	17.86	20.26	30.2	55.32	300	0.38
I7	24.6	30.1	14.86	25.86	30.2	57.92	150	0.20
I8	?	?			30.2	52.00	120	0.14

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Landfill Cell 15 - Modified Closure
FEATURE: Storm Drains
PROJECT NO.: 64-44-300

SHEET 3 OF 12
COMPUTED: PGH
CHECKED: KCS
DATE: 5/20/93
Rev. 5/24/96

- assume that the curve number of the tributary areas between cells ≈ CN caps.

$\therefore CN = 75$

- Calculate an average time of concentration for the between cell areas.

use flow velocity in ditch ~ 1.0 ft/sec
ditch length ~ 150'
length 2:4 slope ≈ 20.5' horizontal
≈ 22.9 (slope length)

$t_c = t_i + t_t \qquad t_i = \dfrac{1.8(1.1 - 0.75)\sqrt{22.9}}{\sqrt[3]{50}} = 2.0 \text{ min}$

$t_t = 150 \times \frac{1}{1} \times \frac{1}{60} = 2.5 \text{ min}$

$\therefore t_c = 2.0 + 2.5 = 4.5 \text{ min} = 0.075 \text{ hr.}$

- assume average basin slope ~ 0.30

Using the above information, and assuming an average area of 0.3 acres, determine peak runoff using "Hydrof" an in-house developed program based on the SCS curve number methodology.

$Q_{peak} \simeq 1.48 \text{ cfs}$

$Q/\text{acre} = \dfrac{1.48}{0.3} \Rightarrow 4.93 \text{ cfs/acre.}$

Since the time of concentration and other basin + hydrologic conditions will be similar for all between cell areas, use the above calculated Q/acre and the previously calculated areas + determine Q at each inlet.

Inlet #	Tributary Area (acres)	Peak flow (cfs)
I1	0.59	2.9
I2	0.42	2.1
I3	0.46	2.3
I4	0.54	2.7
I5a	1.00	4.9
I5b	0.59	2.9
I5c	0.31	1.5
I6	0.38	1.9
I7	0.20	1.0
I8	0.14	0.7

44/12

PROJECT : USPCI - LANDFILL CELL 15 MODIFIED, BETWEEN CELL AREAS

AREA= .3 ACRES
AVERAGE BASIN SLOPE= 30.0 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 8.00 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 173. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR
USER INPUT TIME OF CONCENTRATION= .08 HOURS

TP= .0500 HOURS QPCFS= 4.54 CFS QPIN=14.9995 INCHES
C3= 73.9323 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
6.15	.6642	.0000	.0000	.0	.00
6.17	.6669	.0000	.0000	.9	.00
6.18	.6695	.0000	.0000	3.5	.00
6.20	.6722	.0000	.0000	4.5	.00
6.22	.6749	.0000	.0000	3.8	.00
6.23	.6775	.0000	.0000	2.5	.00
[illegible].25	.6802	.0000	.0000	1.5	.00
[illegible].27	.6829	.0000	.0000	.8	.00
[illegible].28	.6855	.0001	.0000	.4	.00
[illegible].30	.6882	.0001	.0000	.2	.00
[illegible]	[illegible]	.0002	.0000	.0	.00
11.90	4.7111	2.2171	.0804	.0	1.42
11.92	4.8125	2.2981	.0809	.0	1.44
11.94	4.9138	2.3796	.0815	.0	1.45
11.96	5.0152	2.4616	.0820	.0	1.46
11.97	5.1165	2.5441	.0825	.0	1.47
11.99	5.2179	2.6271	.0830	.0	1.48
12.00	5.3069	2.7004	.0733	.0	1.48
12.02	5.3261	2.7162	.0159	.0	1.40
12.04	5.3453	2.7321	.0159	.0	1.16
12.05	5.3645	2.7480	.0159	.0	.86
12.07	5.3837	2.7639	.0159	.0	.62
12.09	5.4029	2.7798	.0159	.0	.46
12.10	5.4221	2.7958	.0159	.0	.37

HYDROGRAPH PEAK= 1.48 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= .13 Acre-Feet

Rev. 5/24/96

II Size the storm Drains.

a.) The flow through each pipe is as shown below

MH3 $Q_{out} = Q_c + I_3 \Rightarrow 11.68 + 2.3 \Rightarrow 14.0$ cfs

MH2 $Q_{out} = MH3 + I_2 + Q_d \Rightarrow 14.0 + 2.1 + 10.84 \Rightarrow 26.9$ cfs

MH1 $Q_{out} = MH2 + Cell14 + I_1 \Rightarrow 26.9 + 21 + 2.9 \Rightarrow 50.8$ cfs

MH8 $Q_{out} = MH1 + I8 \Rightarrow 50.8 + 0.7 \Rightarrow 51.5$ cfs

MH4 $Q_{out} = Q_b + I4 \Rightarrow 16.78 + 2.7 \Rightarrow 19.5$ cfs

MH5 $Q_{out} = MH4 + I5a + I5b + I5c \Rightarrow 19.5 + 4.9 + 2.9 + 1.5 \Rightarrow 28.8$ cfs

MH6 $Q_{out} = MH5 + I_6 + Q_a \Rightarrow 28.8 + 1.9 + 9.61 \Rightarrow 40.3$ cfs

MH7 $Q_{out} = MH6 + I_7 \Rightarrow 40.3 + 1.0 \Rightarrow 41.3$ cfs

b.) a computer spreadsheet was developed using mannings equation to calculate the normal depth of flow in the pipes, based upon the above calculations. (see attached calculations

The results are summarized below:

Upstream MH	Downstream MH	Pipe Dia (ft)	Minimum Slope (%)	Depth (ft)	Velocity (ft/sec)
MH3	MH2	2.0	0.6	1.35	6.21
MH2	MH1	2.5	0.6	1.76	7.28
MH1	MH8	3.0	0.5	2.77	7.45
MH8	OUTLET	2.5	15.0	0.98	28.95
MH4	MH5	2.5	0.5	1.50	6.35
MH5	MH6	2.5	0.5	2.03	6.75
MH6	MH7	3.0	0.5	2.13	7.51
MH7	OUTLET		15.0	0.87	27.25

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH3 AND MH2

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	14.00	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	2.0	feet
Slope (So)	=	0.006	
Normal Depth (y)	=	1.349	feet
Flow x-section area (A)	=	2.255	sq. ft.
Flow Top Width (T)	=	1.874	feet
Perimeter (P)	=	3.855	feet
Hyd. Radius (R)	=	0.585	feet
Flow Velocity (V)	=	6.209	ft/sec.
Froude Number	=	0.998	
Theta	=	3.855	radians
Solve Equation	=	0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	1.348	feet
Critical area (Ac)	=	2.252	sq. ft.
Top Width (Tc)	=	1.875	feet
Perimeter (Pc)	=	3.851	feet
Hyd. Radius (Rc)	=	0.585	feet
Flow Velocity (Vc)	=	6.218	ft/sec.
Froude Number	=	1.000	
Theta	=	3.851	radians

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH2 AND MH1

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	26.90	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	2.5	feet
Slope (So)	=	0.006	
Normal Depth (y)	=	1.761	feet
Flow x-section area (A)	=	3.696	sq. ft.
Flow Top Width (T)	=	2.281	feet
Perimeter (P)	=	4.981	feet
Hyd. Radius (R)	=	0.742	feet
Flow Velocity (V)	=	7.277	ft/sec.
Froude Number	=	1.007	
Theta	=	3.985	radians
Solve Equation	=	0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	1.768	feet
Critical area (Ac)	=	3.712	sq. ft.
Top Width (Tc)	=	2.275	feet
Perimeter (Pc)	=	4.995	feet
Hyd. Radius (Rc)	=	0.743	feet
Flow Velocity (Vc)	=	7.248	ft/sec.
Froude Number	=	1.000	
Theta	=	3.996	radians

7/12

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH1 AND MH8

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	50.80	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	3.0	feet
Slope (So)	=	0.005	
Normal Depth (y)	=	2.772	feet
Flow x-section area (A)	=	6.824	sq. ft.
Flow Top Width (T)	=	1.589	feet
Perimeter (P)	=	7.750	feet
Hyd. Radius (R)	=	0.880	feet
Flow Velocity (V)	=	7.445	ft/sec.
Froude Number	=	0.633	
Theta	=	5.167	radians
Solve Equation	=	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	2.319	feet
Critical area (Ac)	=	5.862	sq. ft.
Top Width (Tc)	=	2.514	feet
Perimeter (Pc)	=	6.444	feet
Hyd. Radius (Rc)	=	0.910	feet
Flow Velocity (Vc)	=	8.666	ft/sec.
Froude Number	=	1.000	
Theta	=	4.296	radians

8/12

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH8 AND OUTLET

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	51.50	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	2.5	feet
Slope (So)	=	0.15	
Normal Depth (y)	=	0.978	feet
Flow x-section area (A)	=	1.779	sq. ft.
Flow Top Width (T)	=	2.440	feet
Perimeter (P)	=	3.378	feet
Hyd. Radius (R)	=	0.527	feet
Flow Velocity (V)	=	28.949	ft/sec.
Froude Number	=	5.975	
Theta	=	2.702	radians
Solve Equation	=	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	2.318	feet
Critical area (Ac)	=	4.748	sq. ft.
Top Width (Tc)	=	1.300	feet
Perimeter (Pc)	=	6.487	feet
Hyd. Radius (Rc)	=	0.732	feet
Flow Velocity (Vc)	=	10.846	ft/sec.
Froude Number	=	1.000	
Theta	=	5.190	radians

9/12

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH4 AND MH5

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	19.50	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	2.5	feet
Slope (So)	=	0.005	
Normal Depth (y)	=	1.498	feet
Flow x-section area (A)	=	3.070	sq. ft.
Flow Top Width (T)	=	2.450	feet
Perimeter (P)	=	4.426	feet
Hyd. Radius (R)	=	0.694	feet
Flow Velocity (V)	=	6.351	ft/sec.
Froude Number	=	1.000	
Theta	=	3.541	radians
Solve Equation	=	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	1.498	feet
Critical area (Ac)	=	3.070	sq. ft.
Top Width (Tc)	=	2.450	feet
Perimeter (Pc)	=	4.426	feet
Hyd. Radius (Rc)	=	0.694	feet
Flow Velocity (Vc)	=	6.352	ft/sec.
Froude Number	=	1.000	
Theta	=	3.541	radians

10/12

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH5 AND MH6

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	28.80	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	2.5	feet
Slope (So)	=	0.005	
Normal Depth (y)	=	2.027	feet
Flow x-section area (A)	=	4.264	sq. ft.
Flow Top Width (T)	=	1.958	feet
Perimeter (P)	=	5.605	feet
Hyd. Radius (R)	=	0.761	feet
Flow Velocity (V)	=	6.754	ft/sec.
Froude Number	=	0.806	
Theta	=	4.484	radians
Solve Equation	=	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	1.830	feet
Critical area (Ac)	=	3.850	sq. ft.
Top Width (Tc)	=	2.215	feet
Perimeter (Pc)	=	5.132	feet
Hyd. Radius (Rc)	=	0.750	feet
Flow Velocity (Vc)	=	7.481	ft/sec.
Froude Number	=	1.000	
Theta	=	4.106	radians

11/12

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH6 AND MH7

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	40.30	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	3.0	feet
Slope (So)	=	0.005	
Normal Depth (y)	=	2.129	feet
Flow x-section area (A)	=	5.364	sq. ft.
Flow Top Width (T)	=	2.724	feet
Perimeter (P)	=	6.010	feet
Hyd. Radius (R)	=	0.892	feet
Flow Velocity (V)	=	7.513	ft/sec.
Froude Number	=	0.943	
Theta	=	4.007	radians
Solve Equation	=	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	2.067	feet
Critical area (Ac)	=	5.194	sq. ft.
Top Width (Tc)	=	2.777	feet
Perimeter (Pc)	=	5.875	feet
Hyd. Radius (Rc)	=	0.884	feet
Flow Velocity (Vc)	=	7.760	ft/sec.
Froude Number	=	1.000	
Theta	=	3.917	radians

12/12

CLIENT: USPCI
PROJECT: LANDFILL CELL 15 CLOSURE WITH 10% CAP SLOPES
FEATURE: STORM DRAINS
PROJECT 64.44.710
DATE: 24-May-96

FIND: FLOW DEPTH IN PIPE BETWEEN MH7 AND OUTLET

Manning Equation Solution for Normal Flow Depth
(Circular Channel)

Flow (Q)	=	41.30	cfs
Manning n (n)	=	0.013	
Pipe Diameter (d)	=	2.5	feet
Slope (So)	=	0.15	
Normal Depth (y)	=	0.869	feet
Flow x-section area (A)	=	1.516	sq. ft.
Flow Top Width (T)	=	2.381	feet
Perimeter (P)	=	3.152	feet
Hyd. Radius (R)	=	0.481	feet
Flow Velocity (V)	=	27.248	ft/sec.
Froude Number	=	6.018	
Theta	=	2.521	radians
Solve Equation	=	-0.000	

CRITICAL FLOW CONDITIONS

Critical Depth (yc)	=	2.156	feet
Critical area (Ac)	=	4.502	sq. ft.
Top Width (Tc)	=	1.722	feet
Perimeter (Pc)	=	5.954	feet
Hyd. Radius (Rc)	=	0.756	feet
Flow Velocity (Vc)	=	9.174	ft/sec.
Froude Number	=	1.000	
Theta	=	4.763	radians

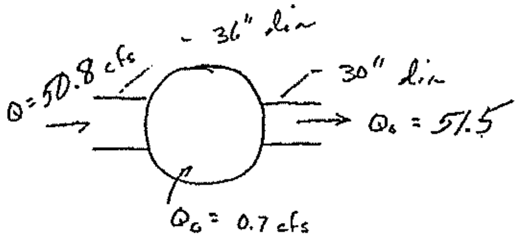
Revised 5/24/96

IV Analyze Northwest Storm Drain

- Round concrete manholes will be used to combine flows from the cap downspouts and runoff between the cells.

Manholes will be analyzed according to the procedures outlined in the "Urban Drainage Criteria Manual", Denver Regional Council of Governments. (DRCOG)

A) Manhole 8



Analyze as in-line through flow with grate flow.

1) Determine the outfall pipe pressure line. The outfall pipe will be steep with super-critical open channel flow conditions. Therefore flow at the inlet will pass through critical depth.

@ critical flow conditions $y = 2.32$ ft

Area $= 4.75$ ft^2

2) Calculate velocity head at the outfall

$$V_o = \frac{Q}{A} = \frac{51.5 \text{ cfs}}{4.75 \text{ ft}^2} = 10.84 \text{ ft/s}$$

$$V_o^2/2g = \frac{10.84^2}{64.4} = 1.82 \text{ ft}$$

3) Calculate the ratios $\frac{D_u}{D_o}$, Q_u/Q_o, $\frac{Q_G}{Q_o}$

$$D_u/D_o = \frac{36"}{30"} = 1.2$$

$$Q_G/Q_o = \frac{0.7}{51.5} = 0.01$$

$$Q_u/Q_o = \frac{50.8}{51.5} = 0.99$$

4) Estimate water depth $d = 5'$

5) Calculate the ratio $\frac{d}{D_0} = \frac{5'}{2.5'} = 2.0$

6) From the lower graph on Figure 8-8.

K_u (base) = 0.8

7) From the upper graph in Figure 8-8

K_u (increment) = 0.0

8) Calculate the total value for K_u

$K_u = 0.8 + 0.0 = 0.8$

9) Reduce K_u for rounded inlet conditions

$K_u = 0.8 - 0.1 = 0.7$

10) Calculate the pressure change h_u

$$h_u = K_u \frac{V_o^2}{2g} = 0.7\,(1.82) = 1.3'$$

11) Calculate upstream in line pipe pressure

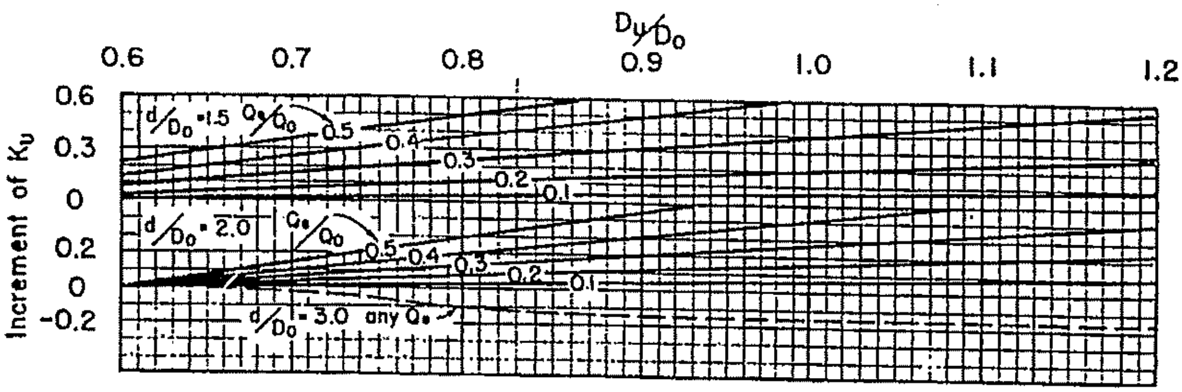
Inv. elev @ MH8 outlet = Z

HGL @ MH8 outlet = Z + 2.3

HGL in upstream pipe at MH8 = Z + 2.3 + 1.3

= Z + 3.6

12) Actual water depth in the manhole is 3.6 feet which is above the top of the inlet pipes. No adjustments to the estimated depth of water in the manhole are necessary because inlet flow is insignificant.



Supplementary Chart for Modification of K_u for Depth in Inlet other than $2.5 D_o$

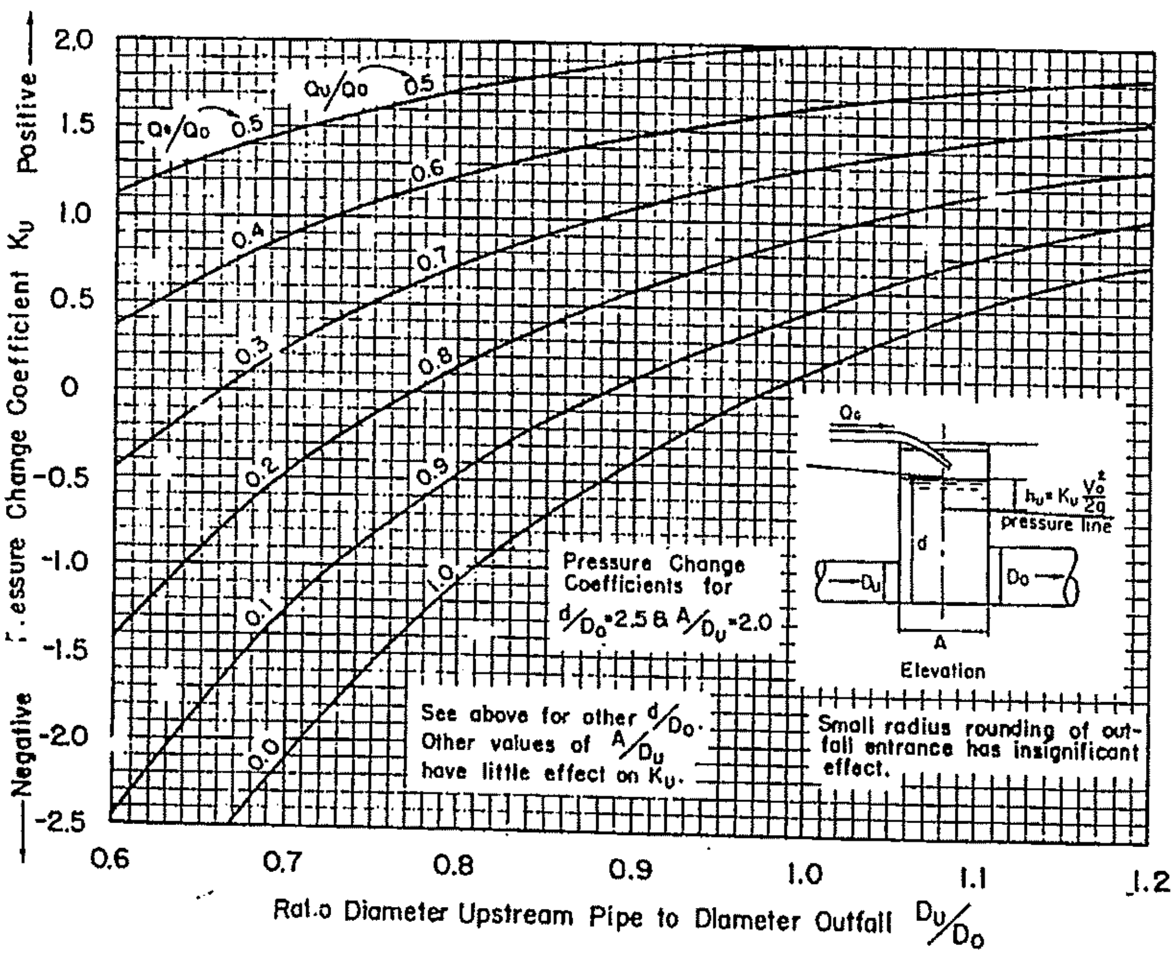


FIGURE 8-8. RECTANGULAR MANHOLE WITH THROUGH PIPELINE AND INLET FLOW (15)

Rev. 5/24/96

B) Calculate friction losses in the pipe between MH 8 and MH 1. Determine if downstream conditions affect flow from MH1.

Solve for the friction slope of the pipe using Mannings equation.

$$S_f = \left[\frac{Qn}{1.49\ A\ R^{2/3}}\right]^2 = \left[\frac{50.8\text{ cfs}\ (0.013)}{1.49\ (7.07)\left(\frac{7.07}{9.42}\right)^{2/3}}\right]^2$$

$$S_f = 0.0058\ \text{ft/ft}$$

With an estimated pipe length of 250', calculate friction losses, h_f.

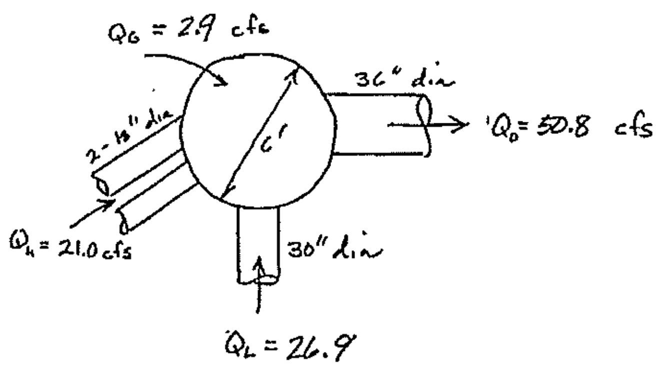
$$h_f = (0.0058\ \text{ft/ft})(250') = 1.45$$

Determine HGL @ MH1 outfall

$$HGL = Z + 3.6 + 1.45 = Z + 5.1$$

$$\text{Top of pipe elevation} = Z + 0.2 + 0.005(250) + 3.0$$
$$= Z + 4.5$$

C) Manhole 1



Analyze as an in-line upstream pipe with a lateral at 90° to the outfall for a round manhole

HA&L ENGINEERING

CLIENT USPCI
PROJECT Lone Mtn. Cell 15 Closure
FEATURE Storm Drains
PROJECT NO. 64.44.300

SHEET 5 OF 14
COMPUTED JB/PLH
CHECKED JCS
DATE 5/21/93
Rev. 5/24/96

1) Calculate the outfall pipe pressure elevation

$$HGL_o = Z + 5.1 \quad (\text{see } B)$$

2) Calculate the velocity head

$$V_o = \frac{Q}{A} = \frac{50.8 \text{ cfs}}{7.07 \text{ ft}^2} = 7.19 \text{ ft/s}$$

$$V_o^2/2g = \frac{7.19^2}{64.4} = 0.80$$

3) Calculate the ratios Q_u/Q_o, D_u/D_o & D_L/D_o

Effective diameter of the two 18" pipes

$$D_u = \sqrt{\frac{4(3.53)}{\pi}} = 2.12 \text{ ft}$$

$$\frac{Q_u}{Q_o} = \frac{21.0}{50.8} = 0.41 \qquad \frac{D_L}{D_o} = \frac{2.5}{3.0} = 0.83$$

$$\frac{D_u}{D_o} = \frac{2.12}{3.0} = 0.71 \qquad D_L/D_o > 0.6 \therefore \text{ use fig. 8-12, 13}$$

4) Calculate the ratio $B/D_o = \frac{6'}{3'} = 2.0$

5) Calculate the factor $\left(Q_u/Q_o\right)\left(D_o/D_u\right) = (0.41)\left(\frac{1}{.71}\right) = 0.58$

$0.58 < 1.0$ so use figure 8-12, 8-13

Lateral pipe

6) Read $\bar{K}_L$ from the lower graph on Figure 8-12

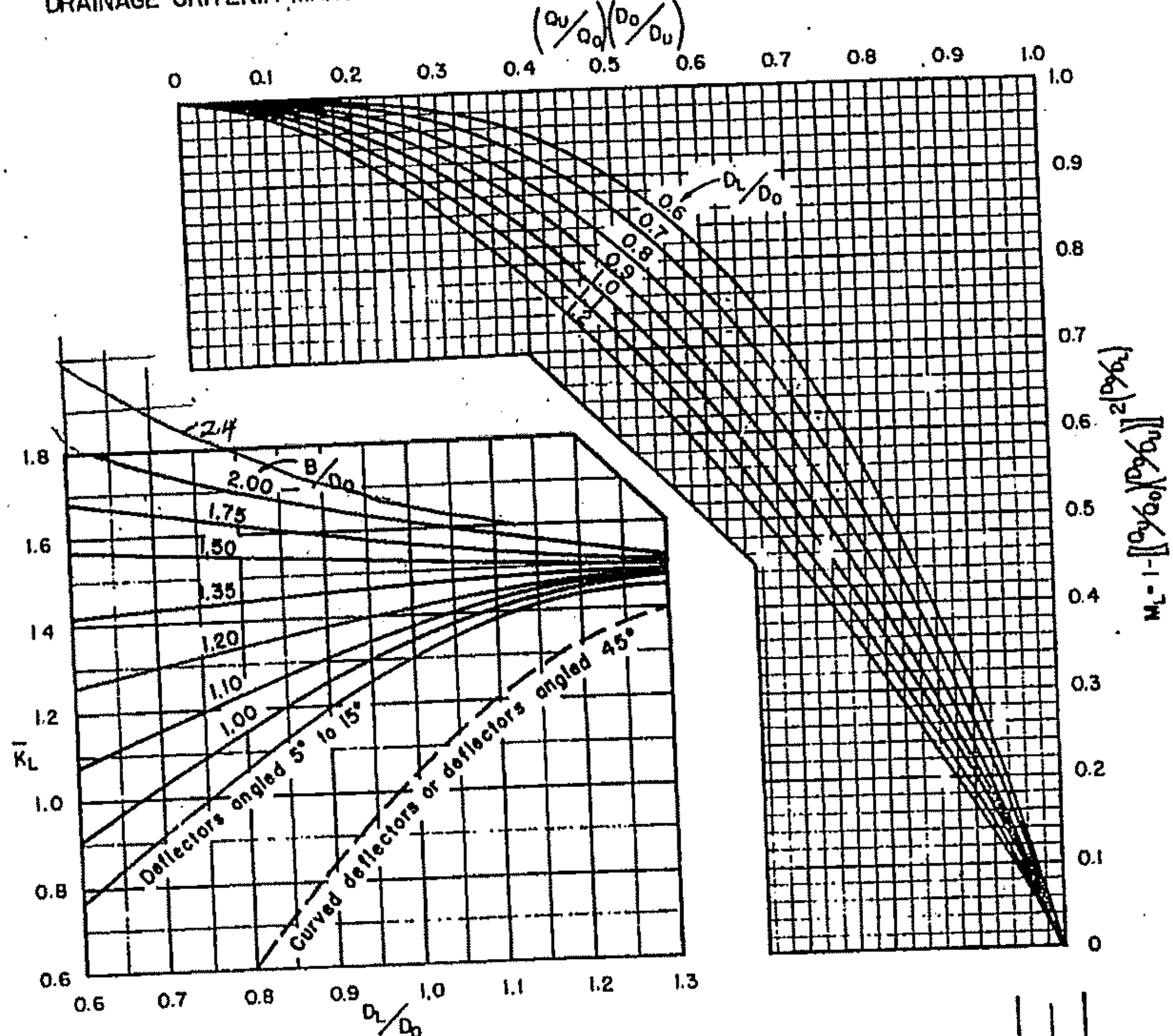
$$\bar{K}_L = 1.68$$

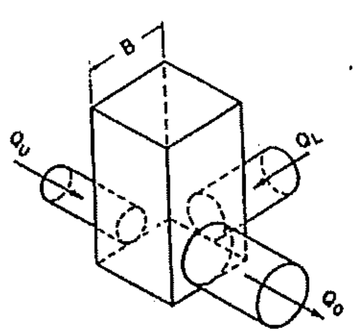
7) Reduce $\bar{K}_L$ by 0.5 for effects of round cross section and a well rounded entrance to the outfall pipe.

$$\bar{K}_L = 1.7 - 0.5 = 1.2$$

8) Read M_L from the upper graph on figure 8-12

$$M_L = 0.74$$





Elevation Sketch

To find K_L for the lateral pipe, first read $\bar{K}_L$ from the lower graph. Next determine M_L. Then

$$K_L = \bar{K}_L \times M_L$$

Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

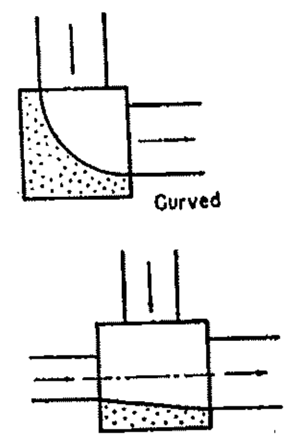
Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L$ by 0.2 for combining flow.

For $Q_U/Q_O \times D_O/D_U > 1$ use Figure 8-14

For $D_L/D_O < 0.6$ use Figure 8-14

$$h_L = K_L \frac{V_O^2}{2g}$$



Plan of Deflectors

FIGURE 8-12. MANHOLE AT 90° DEFLECTION OR ON THROUGH PIPELINE AT JUNCTION OF 90° LATERAL PIPE (LATERAL COEFFICIENT). (15)

9) Calculate $k_L = M_L(\bar{K}_L) = (0.74)(1.2) = 0.89$

10) Calculate the lateral pipe pressure change, h_L

$$h_L = k_L\left(\frac{V_0^2}{2g}\right) = 0.89(0.80) = 0.71'$$

11) Add h_L to the outfall pressure line to obtain the elevation of the lateral pipe pressure line at the branch point

$$HGL_L = (z + 5.1) + 0.7 = z + 5.8$$

Upstream pipe

12) Read $\bar{K}_u$ from the lower graph on figure 8-13

$$\bar{K}_u = 2.08$$

13) Make no reduction to k_u for rounding. This will tend to compensate for the upstream pipe not lining up exactly with the downstream pipe

14) Read M_u from the upper graph on figure 8-13

$$M_u = 0.65$$

15) Calculate $k_u = M_u \bar{K}_u = 0.65(2.08) = 1.35$

16) Calculate the pressure change for the up-stream pipe

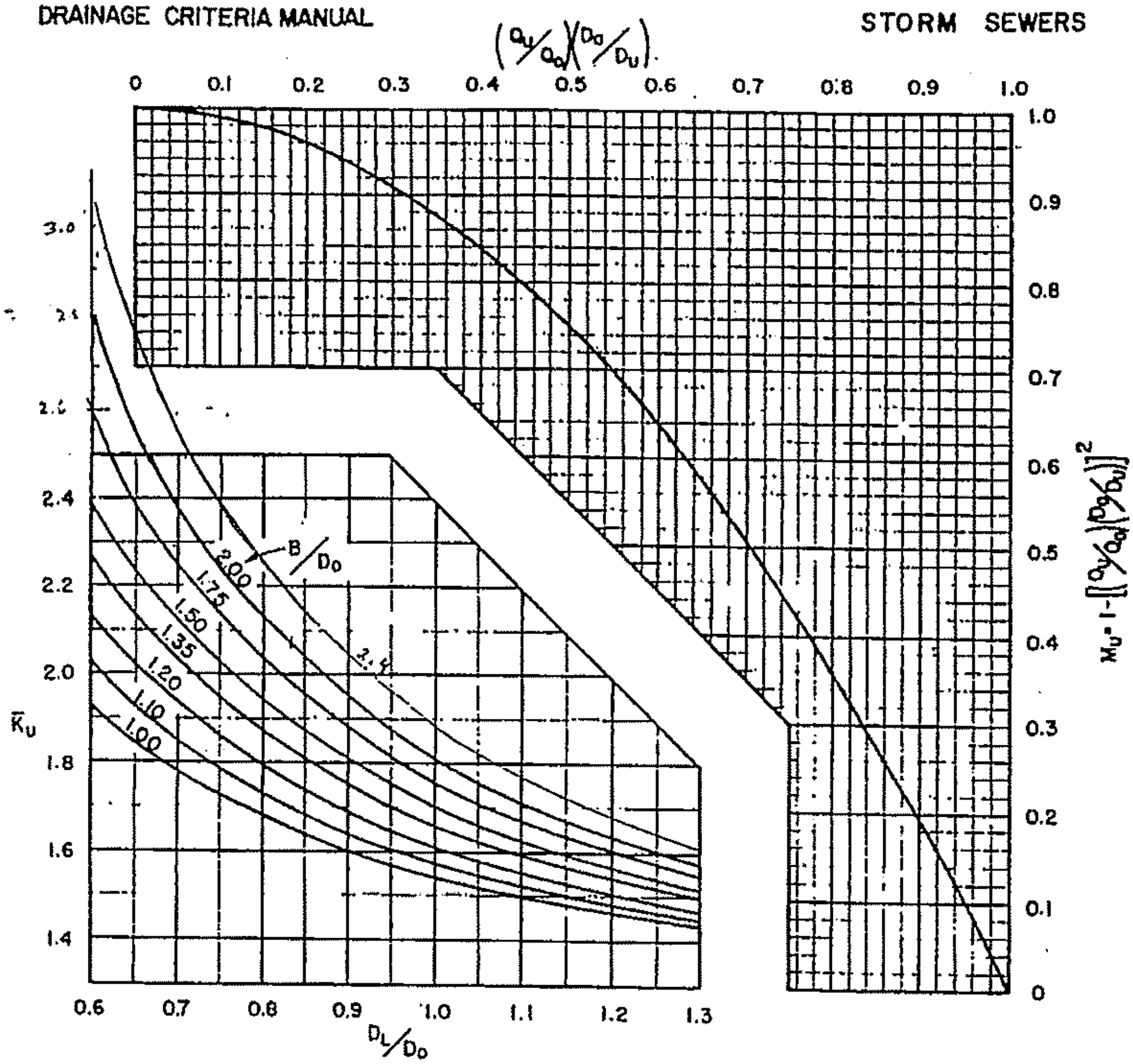
$$h_u = k_u\left(\frac{V_0^2}{2g}\right) = 1.35\,(0.80) = 1.08$$

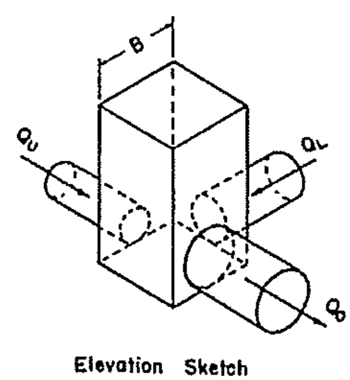
17) Add h_u to the outfall pressure line to obtain the elevation of the upstream pipe pressure line at the branch point.

$$HGL_u = h_u + HGL_o = 1.1' + z + 5.1 = z + 6.2$$

This will correspond to the water surface elevation in the manhole.

8/16





To find K_u for the upstream main, first read $\bar{K}_u$ from the lower graph. Next determine M_u. Then

$$K_u = \bar{K}_u \times M_u$$

For manholes with deflectors at 0° to 15°, read $\bar{K}_u$ on curve for $B/D_o = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_u$ by 0.2 for combining flow.

For deflectors refer to sketches on Figure 8-12

For $Q_u/Q_o \times D_o/D_u > 1$ use Figure 8-14

For $D_L/D_o < 0.6$ use Figure 8-14

$$h_u = K_u \frac{V_o^2}{2g}$$

FIGURE 8-13 MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE (IN-LINE PIPE COEFFICIENT) (15)

D) Calculate friction losses in the pipe between MH1 and MH2, and determine if downstream conditions influence outflow from MH2.

Solve for the friction slope using Mannings Eqn.

$$S_f = \left[\frac{Qn}{1.49\,A\,R^{2/3}}\right]^2 = \left[\frac{26.9 \text{ cfs}\,(.013)}{1.49\,(4.90)\left(\frac{4.90}{7.85}\right)^{2/3}}\right]^2 = 0.0043$$

With an estimated pipe length of 435' calculate pipe friction headloss, h_f.

$$h_f = (0.0043)(435) = 1.87 \text{ ft}$$

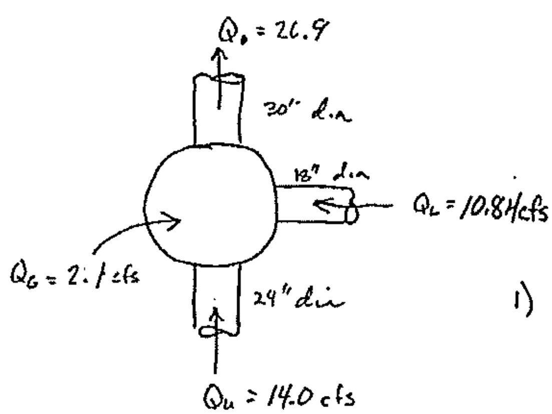
Determine HGL @ MH2 outfall

$$HGL = HGL_L \text{ @ } MH1 + h_f = Z + 6.2 + 1.9 = 8.1$$

With the pipe at a 0.6% slope the top of pipe would be at an elevation of:

$$\text{Top of pipe} = Z + 0.2 + 0.005(250) + 0.3 + .006(435) + 2.5$$
$$= Z + 6.9$$

E) Manhole 2



Analyze as a round manhole on a through pipeline at the junction of a 90° lateral.

1) Determine the outfall pipe pressure line

$$HGL_O = Z + 8.1$$

2) Calculate the velocity head

$$V_o = \frac{Q}{A} = \frac{26.9 \text{ cfs}}{4.90 \text{ ft}^2} = 5.49 \text{ ft/s} \qquad \frac{V_o^2}{2g} = \frac{5.49^2}{64.4} = 0.47$$

3) Calculate the ratios $\frac{Q_u}{Q_o}$, $\frac{D_u}{D_o}$, and $\frac{D_L}{D_o}$

$$\frac{Q_u}{Q_o} = \frac{14.0}{26.9} = 0.52$$

$$\frac{D_u}{D_o} = \frac{2.0}{2.5} = 0.8$$

$$\frac{D_L}{D_o} = \frac{1.5}{2.5} = 0.6$$ use figure 8-12 & 8-13

4) Calculate the ratio $B/D_o = \frac{6.0}{2.5} = 2.4$

5) Calculate the factor $\left(\frac{Q_u}{Q_o}\right)\left(\frac{B}{D_u}\right) = (0.52)\left(\frac{1}{18}\right) = 0.65$

$0.65 < 1$ use figures 8-12 & 8-13

For lateral pipe

6a) Read $\bar{K}_L$ from the lower graph of figure 8-12

extrapolating for $B/D_o = 2.4$ $\Rightarrow$ Base $\bar{K}_L = 2.0$

6b) Reduce $\bar{K}_L$ for effects of round MH

extrapolating for $B/D_o = 2.4$ $\Rightarrow$ reduction $= 0.5$

$\bar{K}_L = 2.0 - 0.5 = 1.5$

7) Reduce $\bar{K}_L$ for well-rounded entrance (round MH)

$\bar{K}_L = 1.5 - 0.1 = 1.4$

8) Determine M_L from upper graph of figure 8-12

$M_L = 0.75$

9) Calculate $K_L = M_L \bar{K}_L = 0.75(1.4) = 1.05$

10) Calculate $h_L = K_L \left(\frac{V_o^2}{2g}\right) = 1.05(0.47) = .49$

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11) Determine HGL of lateral pipe by adding h_L to the outfall pressure line

$$HGL_L = Z + 8.1 + 0.5 = Z + 8.6$$

For upstream pipe

12) Read $\bar{K}_u$ from the lower graph of figure 8-13 extrapolating for $\frac{B}{D_o} = 2.4 \quad \bar{K}_u = 3.1$

13) Reduce $\bar{K}_u$ by 0.2 for rounded entrance

$$\bar{K}_u = 3.1 - 0.2 = 2.9$$

14) Read M_u from upper graph of figure 8-13

$$M_u = 0.56$$

15) Calculate $k_u = \bar{K}_u (M_u) = 2.9\ (0.56) = 1.62$

16) Calculate upstream in-line pressure change

$$h_u = k_u \frac{V_o^2}{2g} = 1.62\ (0.47) = 0.8'$$

17) Upstream HGL = HGL @ outfall + h_u = $Z + 8.1 + 0.8 = Z + 8.9$
The water surface will correspond to the upstream HGL.

F) Evaluate friction losses in the pipe between MH2 and MH3 and determine if outflow from MH3 is influenced by downstream conditions

- Solve for the friction loss using Manning's Eq.

$$S_f = \left[\frac{14.0\ (0.013)}{1.49\ (3.14)\left(\frac{3.14}{6.28}\right)^{2/3}}\right]^2 = 0.0038$$

$$h_f = .0038\,(225') = 0.86 \text{ ft}$$

↑ estimated pipe length

Determine the pressure line at the MH 3 outlet

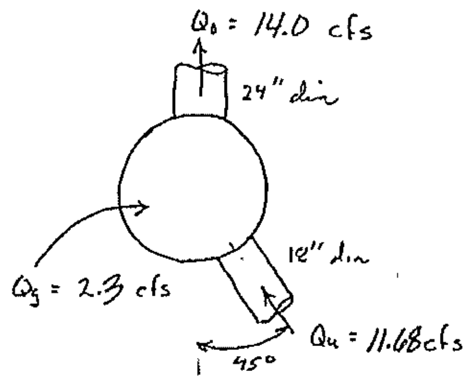
$$HGL @ MH3 = HGL @ MH2 \text{ upstream pipe} + h_f$$
$$= (z + 8.9) + 0.9 = z + 9.8 \text{ ft}$$

Determine top of pipe elevation @ $S_0 = 06\%$

$$Elev = z + 0.7 + 0.005(250) + 0.006(435 + 225) + .2$$
$$= z + 7.9$$

Because HGL is above the top of the pipe, the pipe is flowing full

G. Manhole 3



Analyze as a straight flow through manhole with grate flow and compare with flow through a 45° bend

1) Determine HGL @ outfall

$$HGL = z + 9.8 \quad (\text{see F})$$

2) Calculate energy head @ outfall

$$V_o = \frac{Q}{A} = \frac{14.0 \text{ cfs}}{3.14 \text{ ft}^2} = 4.46 \text{ ft/s}$$

$$V_o^2/2g = \frac{4.46^2}{64.4} = 0.31'$$

3) Calculate the ratios $\frac{D_u}{D_o}$, $\frac{Q_u}{Q_o}$,

$$D_u/D_o = \frac{1.5}{2.0} = 0.75 \qquad \frac{Q_u}{Q_o} = 0.83$$

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Lone Mtn Cell 15 Closure
FEATURE: Storm Design
PROJECT NO. 64.414.300

SHEET 13 OF 16
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CHECKED KGS
DATE 5/21/93
Rev 5/24/96

4) Estimate d = 3' (depth of water above outfall invert)

5) Calculate the ratio $\frac{d}{D_o} = \frac{3.0}{2.0} = 1.5$

6) Read Ku (base) from the lower graph on figure 8-8

$$\text{Base-}k_u = -0.5$$

7) Read incremental adjustment to ku from the upper graph on figure 8-8. (adjustment for $\frac{d}{D_o}$)

increment = 0.1

8) make further adjustments to ku for deflection angle and rounded entrance

well rounded inlet ⇒ reduce Ku by 0.1

From figure 8-15. determine additional losses due to deflection angle. Assume a manhole with no special shaping to be conservative.

from the graph ⇒ K = 0.4

9) Calculate the total combined loss coefficient

$$K_u = -0.5 + 0.1 - 0.1 + 0.4 = -0.1$$

10) Calculate the pressure change

$$h_u = k_u\left(\frac{V_o^2}{2g}\right) = -0.1\,(0.31) = -0.03 \text{ negligible}$$

11) The pressure line for the upstream pipe at the branch point is equal to the pressure line in the outfall

$$HGL_u = z + 9.8$$

This corresponds to the water depth in the manhole

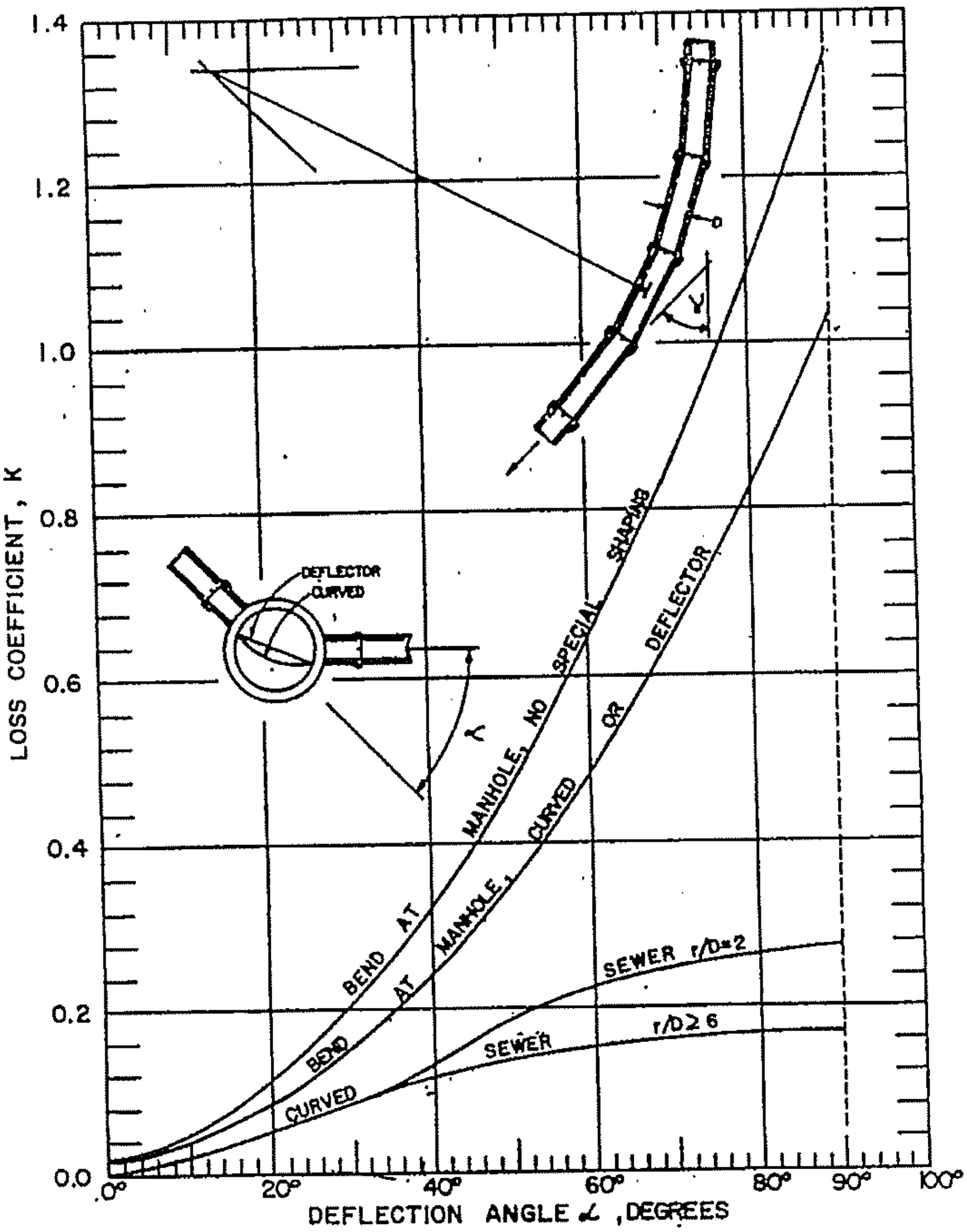


FIGURE 8-15. SEWER BEND LOSS COEFFICIENT (15,16,17)

12) Obtain a more precise estimate of water depth and compare to the original estimate.

$d = HGL_u - \text{top of pipe elev.} + 2.0$

$d = (z+9.8) - (z+7.7) + 2.0 = 3.9$ OK

H) Determine the maximum elevation (z) of the MH 8 outlet and the corresponding outlets for the other manholes.

The tops of the embankments are at Elev. 1420.

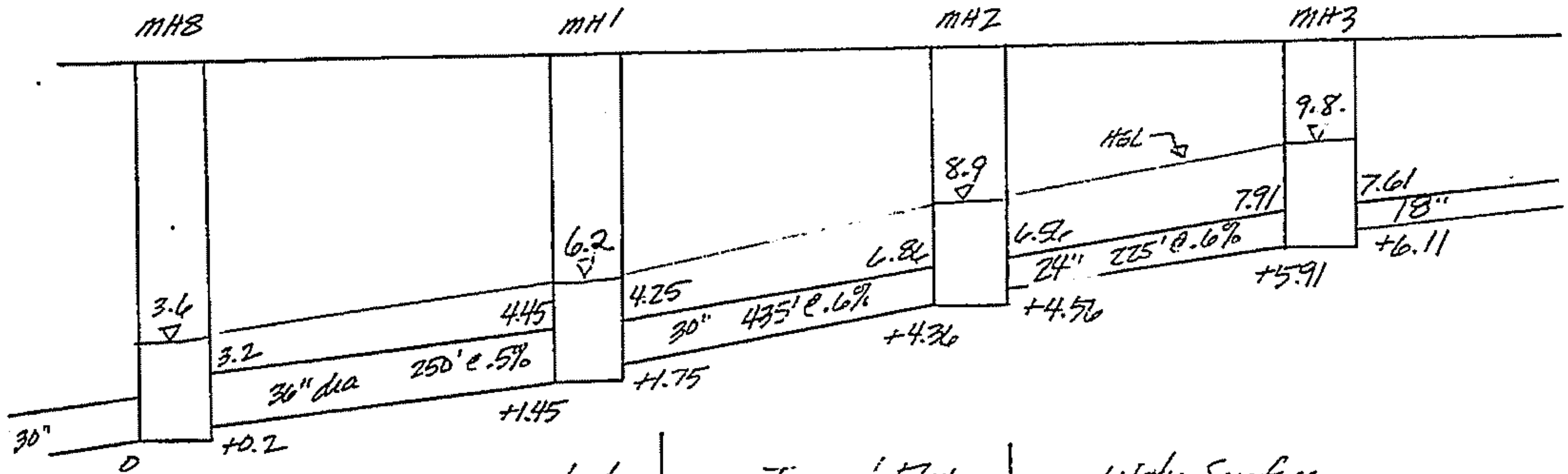
Minor drainage ditches will have a flow line elevation of about 1419 at the inlets or manhole grates. The HGL at the manholes should be at least 0.5 below the ditch flow line to allow flow into an inlet or manhole grate.

Thus, HGL_u @ MH3 ≤ 1418.5

$z + 9.8 \leq 1418.5$

$z \leq 1408.7$

Storm Drain Elevations Relative to MH8 outlet Invert



Manhole #	Invert Elev @ outfall	Water Surface Elevation
8	1408.7	1412.3
1	1410.15	1414.9
2	1413.06	1417.6
3	1414.61	1418.5

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CLIENT: USPCT
PROJECT: Lone Mt. Cell 15 Closure
FEATURE: Design of Storm Drain
PROJECT NO.: 64-44-200

SHEET 1 OF 21
COMPUTED: JD/PGH
CHECKED: KLS
DATE: 9/2/93

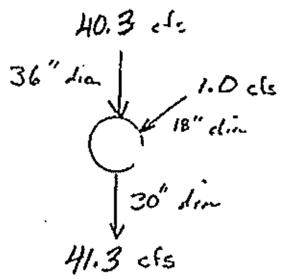
Revised 5/24/96

III Analyze Manholes for Southeast Storm Drain

- Round concrete manholes will be used to combine flows from the cap downspouts and runoff between the cells

Manholes will be analyzed according to the procedures outlined in the "Urban Drainage Criteria Manual", for Denver Regional Council of Governments. (DRCOG)

A) Manhole 7



Analyze the manhole as if the lateral were 90° to the inflow and outflow. This will be conservative

1) Determine the outfall pressure line. Since the flow in the outfall line will be super critical open channel once flow enters the 30" pipe, the flow will pass through critical depth at the inlet

$$y = 2.16 \text{ ft at critical depth}$$

2) Calculate the velocity head at the outfall

$$V_o = \frac{Q}{A} = \frac{41.3 \text{ cfs}}{4.50 \text{ ft}^2} = 9.18 \text{ ft/s}$$

$$\text{velocity head} = \frac{(9.18 \text{ ft/s})^2}{64.4 \text{ ft/s}^2} = 1.31$$

3) Calculate the ratios Q_u/Q_o, D_u/D_o, D_L/D_o

$$Q_u/Q_o = \frac{40.3}{41.3} = 0.98 \qquad D_u/D_o = \frac{30''}{30''} = 1.0$$

$$D_L/D_o = \frac{18''}{30''} = 0.6$$

since $\frac{D_L}{D_o} = 0.6$ use figure 8-12 and 8-13

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CLIENT: USPCT
PROJECT: Lime Mt Cell 15 Closure
FEATURE: Storm Drain
PROJECT NO.: 64-44-200

SHEET 2 OF 21
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CHECKED: KCS
DATE: 5/21/93
Rev 5/24/96

4) Calculate the ratio $\frac{B}{D_o} = \frac{6}{2.5} = 2.4$ (round MH)

5) Calculate $\left(\frac{Q_u}{Q_o}\right)\left(\frac{D_o}{D_u}\right) \Rightarrow (0.98)\left(\frac{30}{36}\right) = 0.82$

since $0.82 < 1.0$ use figures 8-12 & 8-13

For the lateral pipe

6) Enter the lower graph of figure 8-12 at the ratio of $\frac{D_L}{D_o} = 0.6$ and read $\bar{K}_L$ at the curve extrapolating for $\frac{B}{D_o} = 2.4$

from the graph $\bar{K}_L = 2.23$

7) Reduce $\bar{K}_L$ by 0.6 because the pipe entrance will be rounded and effects of round MH

$\bar{K}_L = 2.23 - 0.6 = 1.63$

8) Determine the factor M_L by entering the upper graph of figure 8-12 at the value of

$\left(\frac{Q_u}{Q_o}\right)\left(\frac{D_o}{D_u}\right) = 0.82$ and $\frac{D_L}{D_o} = 0.6$

from the graph $M_L = 0.45$

9) Calculate $K_L = M_L \times \bar{K}_L$

$K_L = (0.45)(1.63) = 0.73$

10) Calculate the lateral pipe pressure change

$h_L = K_L\left(\frac{V_o^2}{2g}\right) = 0.73\,(1.31) = 0.96$

11) Add h_L to the to the outfall pipe pressure line at the branch point to obtain the pressure line at the lateral branch point

$HGL_L = Z + \text{flow depth} + h_L = Z + 2.2 + 0.96 = Z + 3.2$

For the upstream in-line pipe.

12) Enter the lower graph of Figure 8-13 at the values of:

$$\frac{D_L}{D_0} = 0.6 \quad \text{and} \quad \frac{B}{D_0} = 2.4$$

from the graph $\overline{K_u} = 3.2$

13) Reduce $\overline{K_u}$ by 0.2 for effects of a rounded entrance

$$\overline{K_u} = 3.2 - 0.2 = 3.0$$

14) Determine M_u from the upper graph of Fig. 8-13 using the following ratio.

$$\left(\frac{Q_u}{Q_0}\right)\left(\frac{D_0}{D_u}\right) = 0.82 \quad \text{from graph} \Rightarrow M_u = 0.32$$

15) Calculate $K_u = \overline{K_u}\,(M_u) = 3.0\,(0.32) = 0.96$

16) Calculate the upstream in-line pipe pressure change:

$$h_u = K_u\left(\frac{V_0^2}{2g}\right) = 0.96\,(1.31) = 1.26\text{ ft}$$

17) Determine the HGL in the upstream pipe

$HGL_u = z$ + flow depth + losses

$HGL_u = z + 2.2' + 1.26' = z + 3.5$

18) The water surface elevation in the manhole will correspond to the upstream in line pipe pressure at the branch point:

Elev. $= z$ + flow depth + h_u $= z + 2.2 + 1.26$

$= z + 3.5$ OK (water surface elev. is above the pipe crowns)

HA&L ENGINEERING	CLIENT USPCI	SHEET 4 OF 21
	PROJECT Lone Mt Cell 15 Design	COMPUTED JB/PBH
	FEATURE Storm drains	CHECKED KCS
	PROJECT NO. 64-44-200	DATE 5/21/93

Rev 5/24/96

B) Evaluate friction losses in the pipe between MH7 & MH6

Estimated distance between MH6 & MH7 is 315'

Solve for friction slope s_f using Manning's Eq.

$$S_f = \left[\frac{Qn}{1.49\, A\, R^{2/3}}\right]^2 = \left[\frac{40.3\,(.013)}{1.49(7.07)\left(\frac{7.07}{9.42}\right)^{2/3}}\right]^2 = .0036 \text{ ft/ft}$$

$$h_f = (315)(.0036) = 1.14 \text{ ft}$$

Determine if MH6 outlet is influenced by MH7

HGL @ MH6 outlet = Z + 3.5 + 1.14 = Z + 4.64

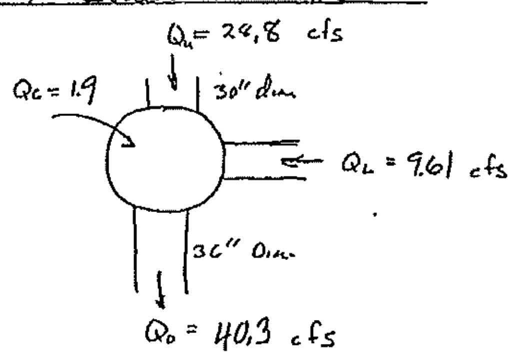
- Assume a minimum pipe slope of .005 ft/ft

Top of pipe @ MH6 outlet = Z + 0.2' + .005(315) + 3'

= Z + 4.8

Because the HGL is below the top of the pipe any interference is negligable

C) Evaluate MH6



- Evaluate the manhole with in-line upstream, 90° lateral, and inlet flow

Critical depth @ 40.3 cfs = 2.07'
HGL @ outlet would be Z + 4.64 - 1.8 = 2.84'
Since HGL > Critical depth, use HGL depth @ outlet for flow depth.

1) HGL @ outlet

- assume depth of flow is depth of HGL @ outlet.

y = 2.84'

HGL = invert + 2.84 = Z + 1.8 + 2.84 = Z + 4.64

2) Calculate velocity head at outfall

@ y = 2.84' ⇒ flow area = 6.92 ft²

$$V_0 = \frac{40.3 \text{ cfs}}{6.92} = 5.82 \text{ ft/s}$$

$$V_0^2/2g = \frac{5.82^2}{64.4} = 0.53 \text{ ft}$$

3) Calculate the ratios $\frac{D_u}{D_0}$, $\frac{Q_u}{Q_0}$, $\frac{Q_G}{Q_0}$

$$\frac{D_u}{D_0} = \frac{30''}{36''} = 0.83$$

$$\frac{Q_u}{Q_0} = \frac{28.8}{40.3} = 0.71$$

$$\frac{Q_G}{Q_0} = \frac{1.9}{40.3} = 0.05$$

4 & 5) Because there is inlet flow into the manhole proceed to step 5. Estimate depth of water in manhole

$d = 4.5'$

6) Calculate the corresponding relative water depth

$$\frac{d}{D_0} = \frac{4.5}{3.0} = 1.5$$

7) Determine k_u from the graph on figure 8-9

@ $\frac{D_u}{D_0} = 0.83$ and $\frac{Q_u}{Q_0} = 0.7$ ⇒ $k_u = 0.6$

8) Determine the increment adjustment to k_u for inlet flow. From the graph on figure 8-9

@ $\frac{d}{D_0} = 1.5$ and $\frac{Q_G}{Q_0} = 0.05$ ⇒ increment = 0.02

9) Because the depth is estimated add an increment of 0.1 to be conservative $k_u = 0.6 + 0.1 = 0.7$

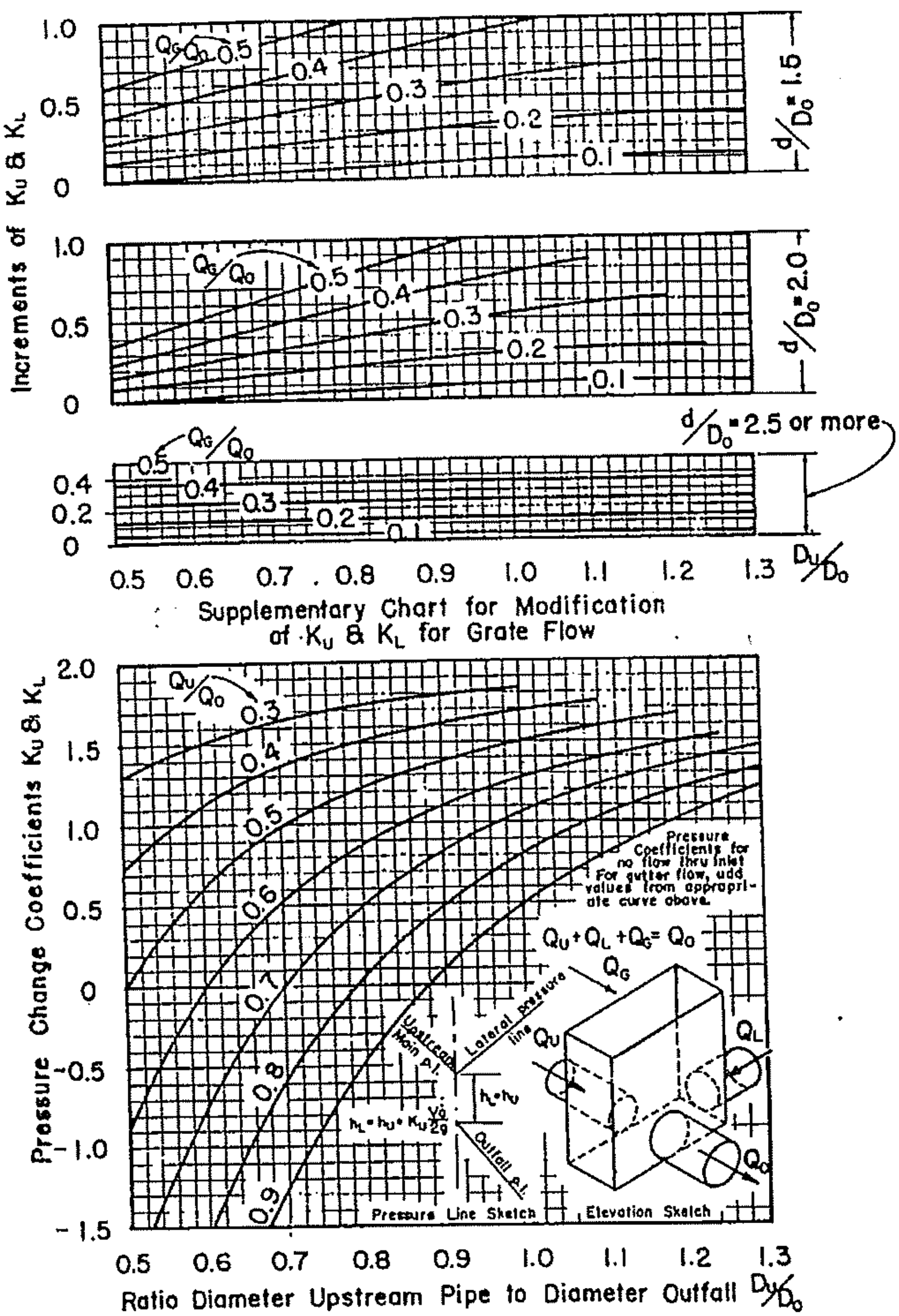


FIGURE 8-9. RECTANGULAR MANHOLE WITH IN-LINE UPSTREAM MAIN & 90° LATERAL PIPE (WITH OR WITHOUT INLET FLOW) (15)

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Lone Mt Cell 15 Design
FEATURE: Storm Drains
PROJECT NO. 64-44-200

SHEET 7 OF 21
COMPUTED: JB/PGH
CHECKED: KCS
DATE: 5/21/93
Rev 5/24/96

10) The pipe entrance to the outfall will be rounded so reduce k_u by 0.1

$$k_u = 0.7 - 0.1 = 0.6$$

11) Calculate h_u:

$$h_u = k_u \frac{V_o^2}{2g} = 0.6(0.53) = 0.32$$

12) Add h_u to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point. The elevations of the lateral pipe pressure line and the water surface at the inlet well correspond.

$$HGL_u = (z + 4.64) + 0.32 = z + 4.96'$$

13) Obtain a more precise value for d

$$d = HGL_u - \text{outfall pipe invert} = 4.96 - (315)(.005) - 0.2$$

$$d = 3.19'$$

14) Because a conservative increment for k_u was assumed in step 9, no adjustments are necessary

D) Evaluate pipe friction loss between MH6 & MH5 and determine if MH5 outlet is influenced by downstream conditions at MH6.

Estimated distance between MH6 & MH5 = 315'

Solve for friction slope S_f using Manning's Eq.

$$S_f = \left[\frac{Qn}{1.49\ A\ R^{2/3}}\right]^2 = \left[\frac{28.8\,(.013)}{1.49(4.90)\left(\frac{4.90}{7.85}\right)^{2/3}}\right]^2 = 0.0049\ \text{ft/ft}$$

$$h_f = 315\,(.0049\ \text{ft/ft}) = 1.54\ \text{ft}$$

Determine if downstream conditions at MH6 influence outflow from MH5.

HGL @ MH5 outlet = MH6 HGL_u + pipe friction losses

$$= (Z + 4.96) + 1.54 = Z + 6.5$$

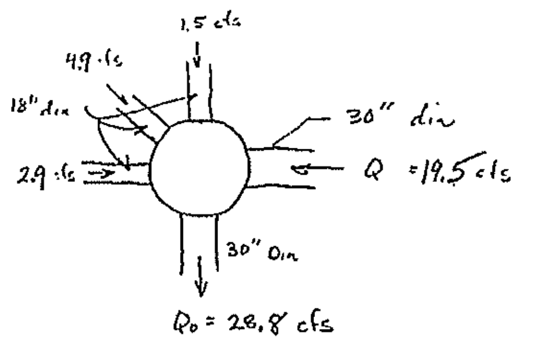
Assuming a minimum pipe slope of 0.005 ft/ft

Top of pipe @ MH5 outlet $= (Z + 1.8) + 0.2 + 315(.005) + 2$

$$= Z + 6.1$$

Because the HGL at the MH5 outlet exceeds the top of the pipe, downstream conditions at MH6 will influence outflow from MH5 and the pipe will be flowing full.

E) Evaluate Conditions at MH 5



- Combine the minor inflows and analyze as if the manhole had opposed in-line laterals carrying 19.5 cfs and 9.3 cfs.

1) HGL @ outfall

HGL = MH6 HGL_u + friction losses

$$= (Z + 4.96) + 1.54 = Z + 6.5$$

2) Calculate velocity head at outfall

$$V_o = \frac{Q}{A} = \frac{28.8 \text{ cfs}}{\pi \frac{(2.5)^2}{4}} = 5.87 \text{ ft/s}$$

$$\frac{V_o^2}{2g} = \frac{5.87^2}{64.4} = 0.54 \text{ ft}$$

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Lone Mt Cell 15 Closure
FEATURE: Storm Drains
PROJECT NO. 64-44-200

COMPUTED JRB/PBH
CHECKED KCS
DATE 5/21/93
Rev 5/24/96

3) Calculate velocities in each lateral. Use the average velocity of the 3 combined 18" dia pipes.

$$V_{HV} = \frac{19.5 \text{ cfs}}{\frac{\pi (2.5)^2}{4}} = 4.0 \text{ ft/s}$$

$$V_{LV} = \frac{9.3 \text{ cfs}}{\frac{3\pi (1.5)^2}{4}} = 1.8 \text{ ft/s}$$

4) Calculate the ratios Q_G/Q_o, Q_{HV}/Q_o, Q_{LV}/Q_o, D_{HV}/D_o, D_{LV}/D_o and D_{HV}/D_{LV}

$$D_{LV} = \text{effective dia} = \sqrt{\frac{4(5.30)}{\pi}} = 2.60'$$

$$Q_G/Q_o = 0$$

$$Q_{HV}/Q_o = \frac{19.5}{28.8} = 0.68$$

$$Q_{LV}/Q_o = \frac{9.3}{28.8} = 0.32$$

$$D_{HV}/D_o = \frac{2.5}{2.5} = 1.0$$

$$D_{LV}/D_o = \frac{2.6}{2.5} = 1.04$$

$$D_{HV}/D_{LV} = \frac{2.5}{2.6} = 0.96$$

5) Determine H from the left-hand graph in Fig. 8-10

from graph $\Rightarrow$ $H = 2.4$

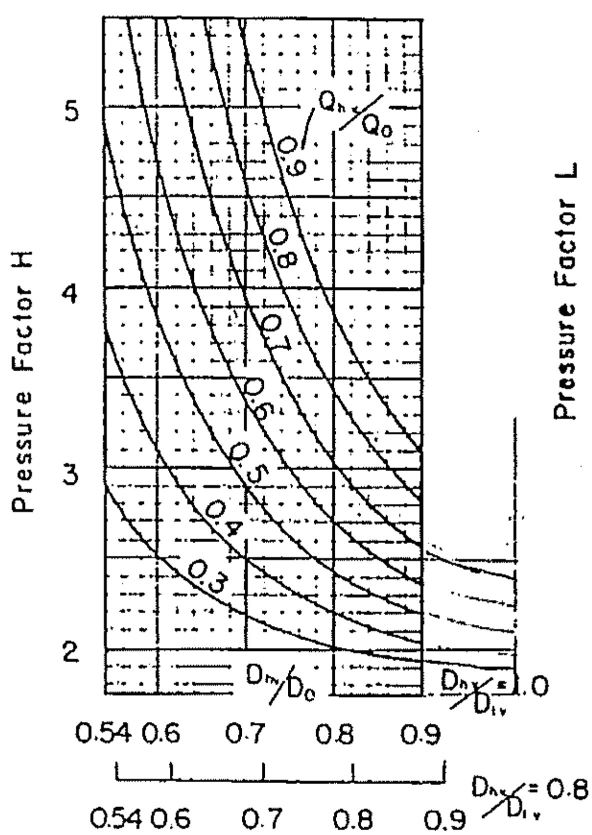
6) Determine L from the right-hand side on Figure 8-10.

from graph $\Rightarrow$ $L = 0.1$

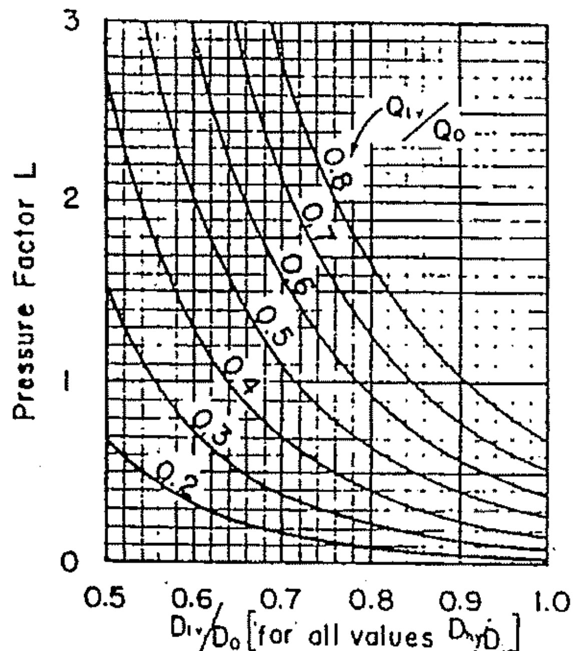
7) Calculate $K_{LV} = (H-L) - 0.2$ (no inlet flow)

$$K_{LV} = (2.4 - 0.1) - 0.2 = 2.1$$

8) $K_{HV} = 1.6$ (no inlet flow)



Ratio of diameter of higher velocity lateral to diameter of outfall



Ratio of diameter of lesser velocity lateral to diameter of outfall

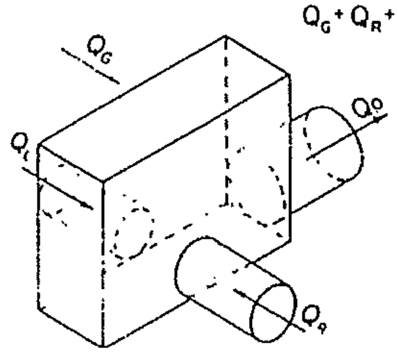
D_{hv} = diameter of lateral with higher-velocity flow.

Q_{hv} = rate of flow in lateral with higher-velocity flow.

D_{lv} = diameter of lateral with lower-velocity flow.

Q_{lv} = rate of flow in lateral with lower-velocity flow.

$Q_G + Q_R + Q_L = Q_o$



Elevation Sketch

To find K_R or K_L for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q; then: K_R (or K_L) = H − L

K_R or K_L for the lateral pipe with higher velocity flow is always 1.8

$$h_L = K_L \frac{V_o^2}{2g} \qquad h_R = K_R \frac{V_o^2}{2g}$$

FIGURE 8-10. RECTANGULAR MANHOLE WITH IN-LINE OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT INLET FLOW) (15)

9) Calculate losses in both laterals

$$h_{LV} = k_{LV}\left(\frac{V_0^2}{2g}\right) = 2.1(0.54') = 1.1 \text{ ft}$$

$$h_{HV} = k_{HV}\left(\frac{V_0^2}{2g}\right) = 1.6(0.54) = 0.9 \text{ ft}$$

10) Determine the pressure line for each lateral at the branch point

$$HGL_{LV} = (z + 6.5) + 1.1 \text{ ft} = z + 7.6$$

$$HGL_{HV} = (z + 6.5) + 0.9 \text{ ft} = z + 7.4 \text{ ft}$$

- The water surface elevation corresponds to the HGL of the higher velocity lateral

F) Evaluate pipe friction loss between MH5 & MH4 and determine if MH4 outlet is influenced by downstream conditions at MH5.

- Estimated distance between MH5 and MH4 = 315'

Using Manning's Eq:

$$S_f = \left[\frac{Q\,n}{1.49\,A\,R^{2/3}}\right]^2 = \left[\frac{19.5(.013)}{1.49(4.90)\left(\frac{4.90}{7.85}\right)^{2/3}}\right]^2 = 0.0023$$

$$h_f = 315\,(.0023) = 0.7'$$

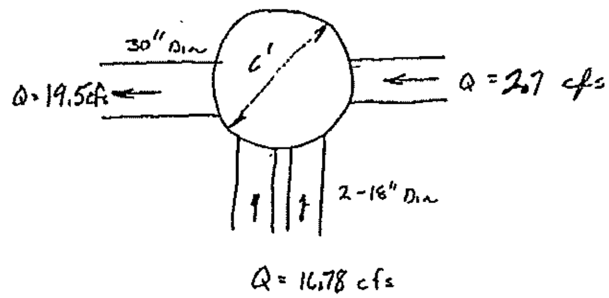
Determine if downstream conditions influences outflow from MH 4.

@ MH4 outlet $HGL = z + 7.4 + 0.7' = z + 8.1'$

Top of pipe @ MH4 outlet $= (z + 6.1) + 0.3 + 315\,(.005)$
$= z + 8.0$

Because the top of the pipe is below the HGL at the MH4 outlet, downstream conditions control.

G. Evaluate Manhole 4



Analyze the manhole as a through pipeline with a 70° lateral with no inlet flow

1) HGL at outfall

$$HGL = MH5\ HGL + \text{Friction loss}$$
$$= z + 7.4 + 0.7 = z + 8.1$$

2) Calculate velocity head at outfall

$$V_o = \frac{Q}{A} = \frac{19.5 \text{ cfs}}{4.91 \text{ ft}^2} = 3.97 \text{ ft/s}$$

$$V_o^2/2g = \frac{(3.97 \text{ ft/s})^2}{64.4 \text{ ft/s}^2} = 0.24 \text{ ft}$$

3) Calculate the ratios $\frac{D_L}{D_o}$ and $\frac{B}{D_o}$

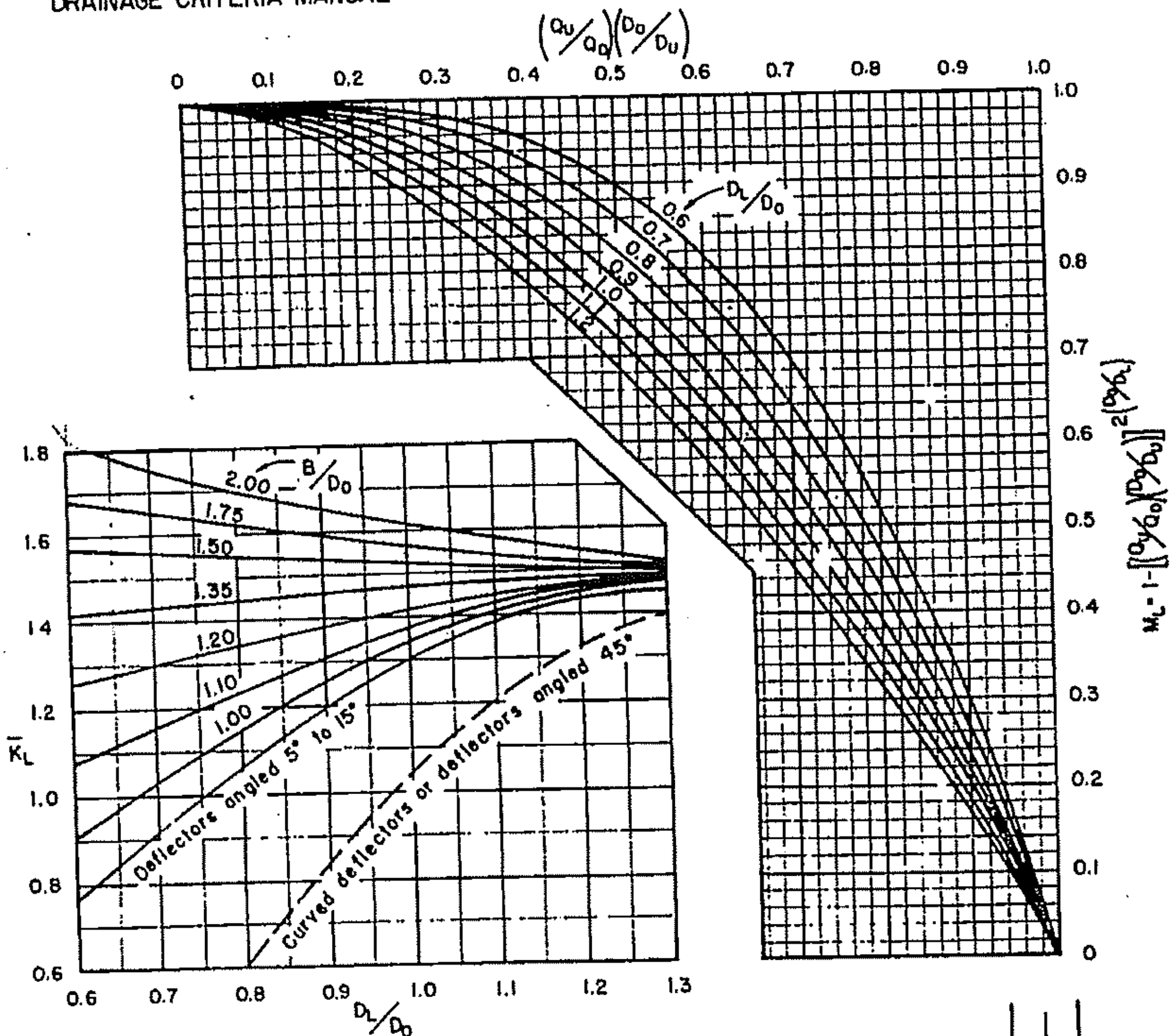
$$\frac{D_L}{D_o} = \frac{\sqrt{\frac{4(3.53)}{\pi}}}{2.5} = 0.85$$

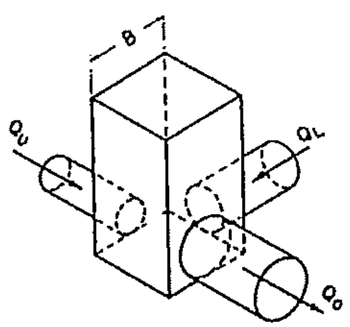
Combined area of two 18" dia pipes = 3.53 ft^2
Equivalent dia = $\sqrt{\frac{4(3.53)}{\pi}} = 2.12'$

$$\frac{B}{D_o} = \frac{6.0}{2.5} = 2.4$$

4) Read $\bar{K}_L$ from the lower graph of Fig. 8-12
extrapolating from graph $\Rightarrow$ $\bar{K}_L = 1.8$

5) Reduce $\bar{K}_L$ by 0.3 for well rounded entrance
$K_L = 1.8 - 0.3 = 1.5$





Elevation Sketch

To find K_L for the lateral pipe, first read $\bar{K}_L$ from the lower graph. Next determine M_L. Then

$$K_L = \bar{K}_L \times M_L$$

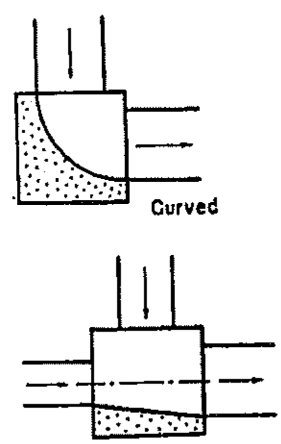
Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L$ by 0.2 for combining flow.

For $Q_U/Q_O \times D_O/D_U > 1$ use Figure 8-14

For $D_L/D_O < 0.6$ use Figure 8-14



Plan of Deflectors

$$h_L = K_L \frac{V_O^2}{2g}$$

FIGURE 8—12. MANHOLE AT 90° DEFLECTION OR ON THROUGH PIPELINE AT JUNCTION OF 90° LATERAL PIPE (LATERAL COEFFICIENT). (15)

6) Calculate change in pressure $h_L = K_L \left(\frac{V_o^2}{2g}\right)$

$$h_L = 1.5(0.24) = 0.4$$

7) Add h_L to the elevation of the outfall pressure line at the branch point to determine the elevation of the lateral pipe pressure line at this point.

$$HGL_L = (z + 8.1) + 0.4 = z + 8.5$$

8) The water surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water-surface elevation use figure 8-13, and steps 12-18

12). Read $\overline{K_u}$ from the lower graph of Fig. 8-13

$$\overline{K_u} = 2.2$$

13) Reduce $\overline{K_u}$ by 0.2 for well rounded entrance

$$\overline{K_u} = 2.2 - 0.2 = 2.0$$

14) Read M_u from the upper graph of Figure 8-13

$$\left(\frac{Q_u}{Q_o}\right)\left(\frac{D_o}{D_u}\right) = \left(\frac{2.7}{19.5}\right)\left(\frac{2.5}{1.5}\right) = 0.23$$

from the graph ⇒ $M_u = 0.95$

15) Calculate $K_u = M_u(\overline{K_u}) = 0.95(2.0) = 1.9$

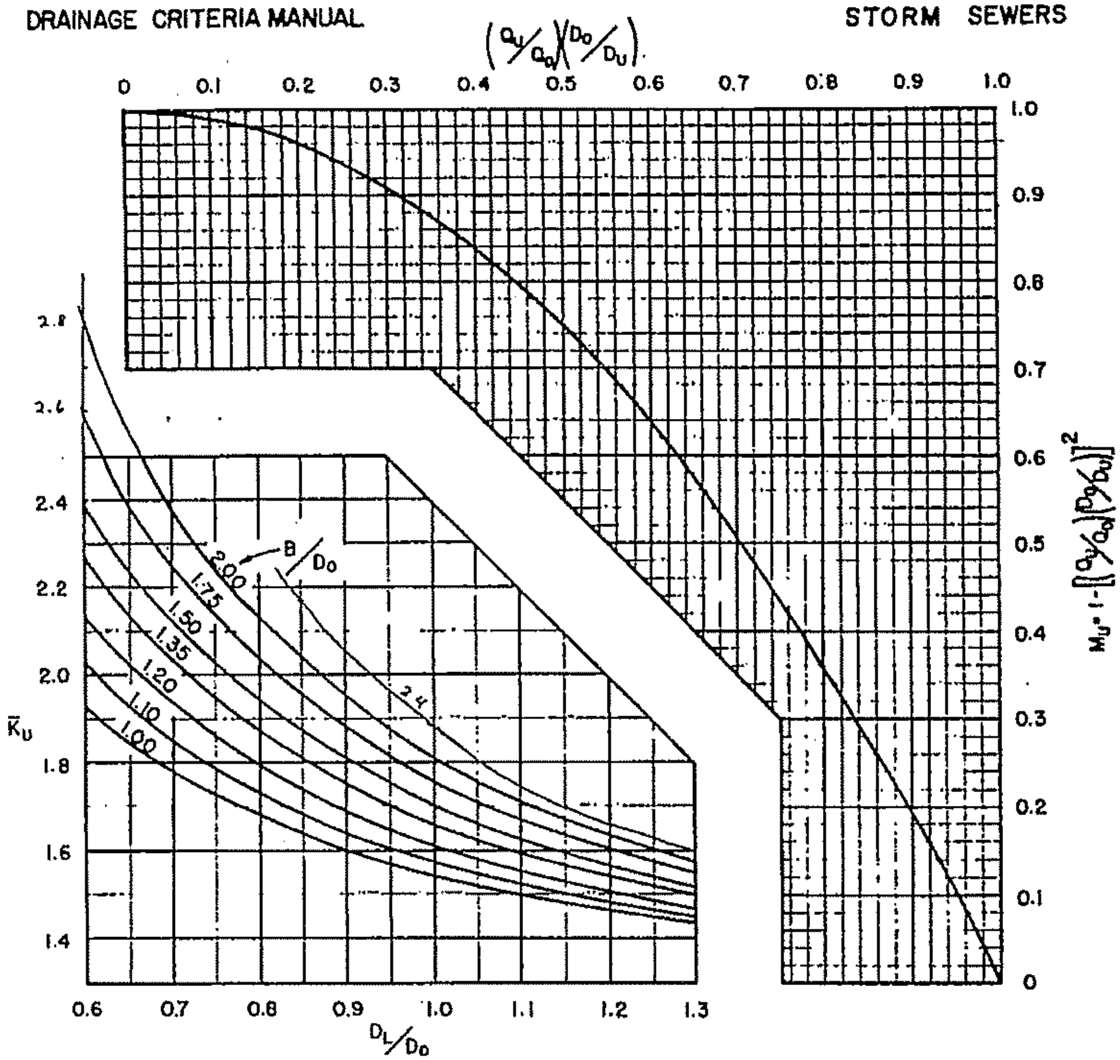
16) Calculate upstream in-line pressure change

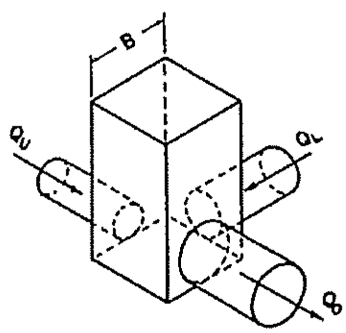
$$h_u = K_u \frac{V_o^2}{2g} = 1.9(0.24) = 0.5$$

17) Upstream in-line pressure line is equal to the water surface elev. in the manhole

$$HGL_u = \text{water surface} = (z + 8.1) + 0.5 = 8.6 + z$$

15/21





Elevation Sketch

To find K_U for the upstream main, first read $\overline{K}_U$ from the lower graph. Next determine M_U. Then

$$K_U = \overline{K}_U \times M_U$$

For manholes with deflectors at 0° to 15°, read $\overline{K}_U$ on curve for $B/D_O = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\overline{K}_U$ by 0.2 for combining flow.

For deflectors refer to sketches on Figure 8-12

For $Q_U/Q_O \times D_O/D_U > 1$ use Figure 8-14

For $D_L/D_O < 0.6$ use Figure 8-14

$$h_U = K_U \frac{V_O^2}{2g}$$

FIGURE 8-13 MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE (IN-LINE PIPE COEFFICIENT) (15)

H. Estimated pipeline elevations

- The elevation, z, of the storm drain outfall at MH 7 will be controlled by either HGL at inlets for drains conveying water to MH 4 or MH5

Calculate HGL @ inlet I5b

HGL = HGL @ MH5 + friction losses + inlet losses

HGL @ MH5 = z + 7.6 (from velocity I[illegible])

friction loss, h_f, according to Mannings Eqn:

$n = 0.015$ pipe length ≈ 365'

$$h_f = \left(\frac{Q}{K}\right)^2 (\text{Length}) = \left(\frac{2.9 \text{ cfs}}{91.3}\right)^2 (365) = 0.4'$$

1) HGL @ I5b = z + 7.6 + 0.4 = z + 8.0

2) Calculate velocity head at outfall. Assume the pipe is flowing full where D = 1.5'

$$V_o = \frac{2.9 \text{ cfs}}{1.77 \text{ ft}^2} = 1.64 \text{ ft/s}$$

$$V_o^2/2g = \frac{1.64^2}{64.4} = 0.042$$

3) Use a drop inlet, 3' deep. Allow the inlet to fill within 0.5' of the gutter line d = 2.5

4) From fig. 8-6 assuming box end flow

with $d/D_o = \frac{2.5}{1.5} = 1.67 \Longrightarrow K_g = 8.6$

5) $h_g = 8.6(.042) = 0.4$ ft

6) HGL = z + 8.0 + 0.4 = z + 8.4

HGL @ I5b must be less than 0.5 below inlet invert

$\therefore z + 8.4 < 1418.5 \Rightarrow z < 1410.1$

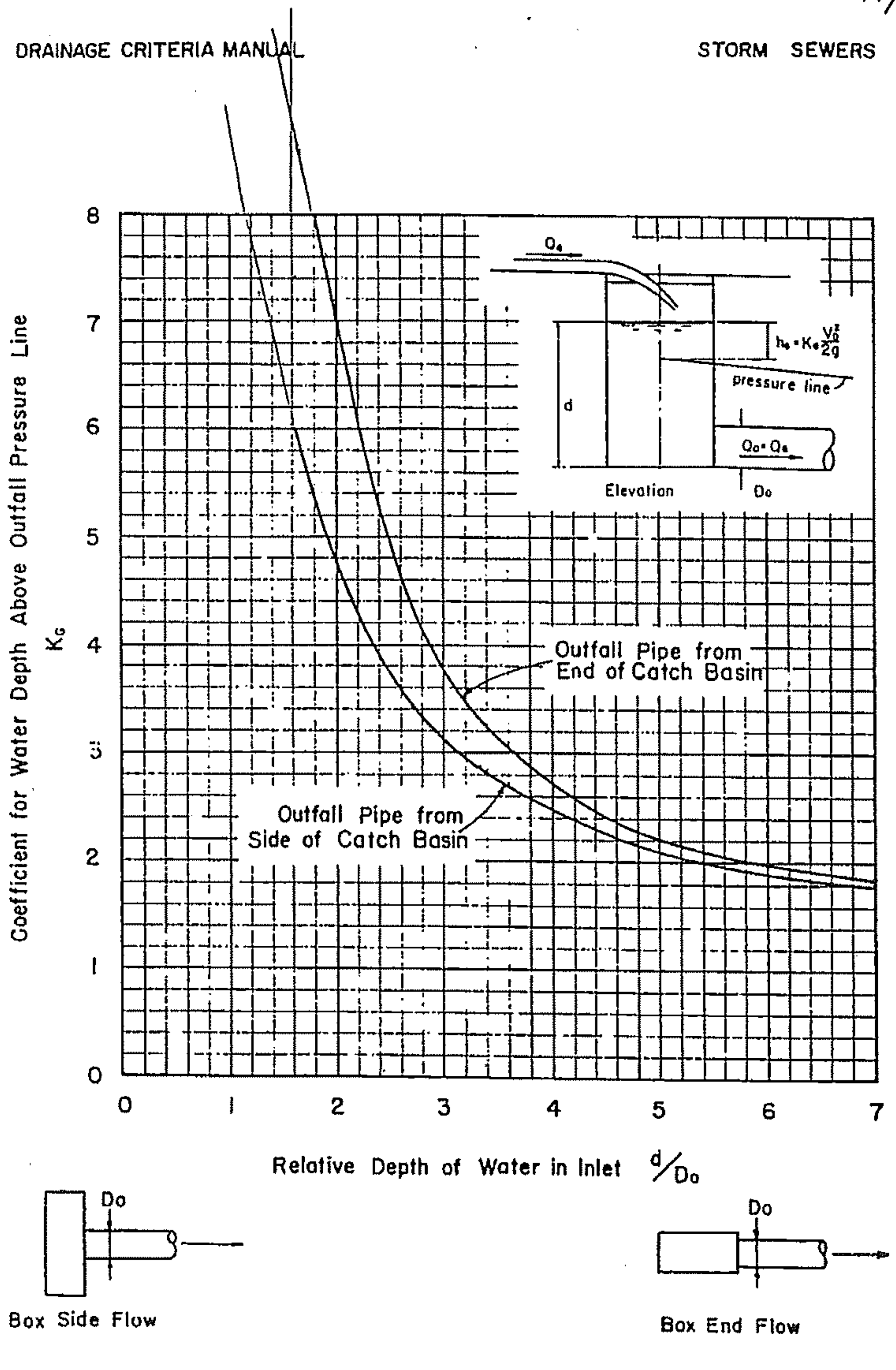


FIGURE 8—6. CATCH BASIN WITH INLET FLOW ONLY (15)

Rev 5/24/96

Calculate HGL @ inlet I4

HGL = HGL @ MH4 + inlet losses (friction losses are negligible because the pipe is short)

HGL_u @ MH4 = z + 8.6 (upstream lateral)

Calculate inlet losses

2) Calculate velocity head. Assume the pipe is flowing full

$$V_o = \frac{2.7 \text{ cfs}}{1.77 \text{ ft}^2} = 1.525 \text{ ft/s}$$

$$V_o^2/2g = \frac{1.53^2}{64.4} = 0.036$$

3) The inlet box will have a total depth of 3', allow the box to fill within 0.5' of the gutter line
d = 2.5'

4) From Figure 8-6 (assuming box end flow)

with $d/D_o = \frac{2.5}{1.5} = 1.67 \Rightarrow k_j = 8.6$

5) Calculate $h_j = k_j \frac{V_o^2}{2g} = 8.6(.036) = 0.3'$

6) HGL = z + 8.6 + 0.3 = z + 8.9

HGL @ I4 must be less than 0.5' below gutter line

z + 8.9 < 1418.5

z < 1409.6

- The pipe from inlet I4 must have a slope of at least 0.005 to maintain velocities adequate to keep the pipe free of sediment deposits.

estimated pipe length is 40'

- pipe invert at I4 = z + 5.7' + 40(.005) = z + 5.9
- with a 3' box and a maximum gutter line elevation of 1419 then

z + 5.9 ≤ 1419 - 3 ⇒ z ≤ 1410.1

- The pipe from inlet I5 must have a slope of at least 0.005 ft/ft to maintain adequate velocity to keep the pipe free of sediment

Calculate the minimum elevation of the inlet relative to Z

$$\text{Elev. of pipe outlet @ MH 5} = Z + 3.6 + 0.3$$
$$= Z + 3.9$$

$$\text{pipe invert at inlet} = Z + 3.9 + 365(.005)$$
$$= Z + 5.7$$

With a 3' box with a gutter line elevation of 1419.0, then

$$Z + 5.7 \leq 1416.0 \Rightarrow Z \leq 1410.3$$

Recheck inlet I-6

- assume $d = 7.5 - 5.7 + 10(.039) = 2.2$

$$d/D_o = 2.2/1.5 = 1.5$$

Extrapolating from graph 8-6 a value of $K_g = 10.0$ appears reasonable. The water depth is 0.8' below the gutter line. Therefore the drop inlet is acceptable.

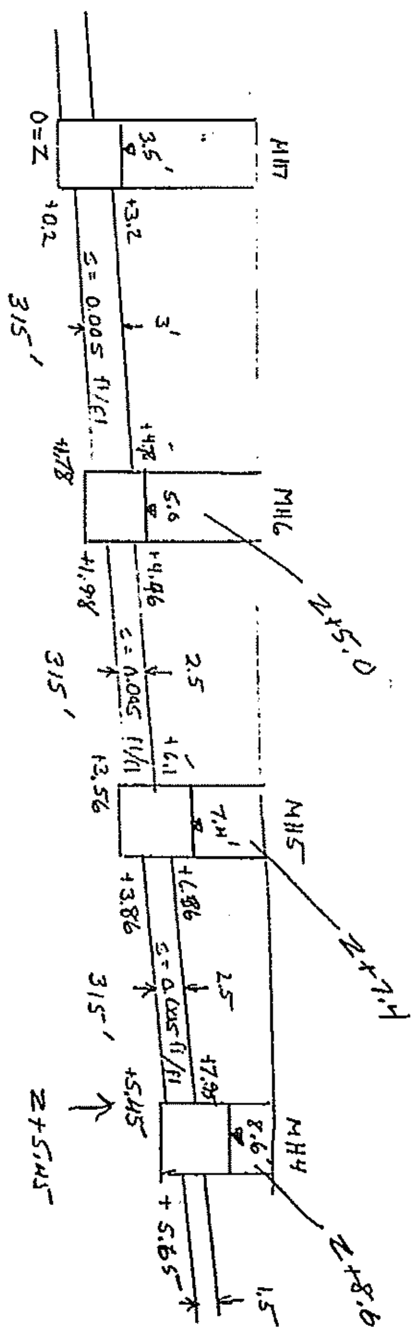
- Flows into other inlets between the cell caps will have lower flowrates. Thus 3' deep inlets will be adequate for all inlets in minor drainage ditches

HA&L ENGINEERING

CLIENT: USPCI
PROJECT: Lone Mtn. Cell 15 Closure
FEATURE: Storm Drains
PROJECT NO.: 64.44.200

SHEET 20 OF 21
COMPUTED: JB/PSH
CHECKED: ICCS
DATE: 5/21/93
Rev 5/24/96

Storm drain elevations relative to the MH 7 outlet invert



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CLIENT USPCI
PROJECT Lone Mtn Cell 15 Closure
FEATURE Storm Drains
PROJECT NO. 64.44.200

SHEET 21 OF 21
COMPUTED JB / PGH
CHECKED KCS
DATE 5/21/93
Rev 5/24/96

Conclusion

- The controlling factor is the minimum slope of the pipe from I4

$z \leq 1409.6$

The maximum elevation of the outlet of MH7 is Elev. 1409.6

Estimated elevation

MH #	Invert Elev. @ outfall	water surface elevations	
7	1409.6	1413.1	ok
6	1411.4	1414.6	ok
5	1413.2	1417.0	ok
4	1415.1	1418.2	ok

HA&L ENGINEERING

CLIENT: USPCI/Laidlaw
PROJECT: Landfill Cell 15 - Closure
FEATURE: Erosion Protection - Cap in Valley Area
PROJECT NO.:
SHEET 1 OF 2
COMPUTED: [illegible]
CHECKED:
DATE: 7/5/96

Problem - Check the Erosion Stability of the valley area on the closure cap that is tributary to Downspout D3.

1- Hydrologic Characteristics.

A - Tributary Area:

$$= \frac{290.6\,(320.3)}{2} + \frac{391.8(134.5)}{2} + \left(\frac{200.2+160}{2}\right)27.5$$

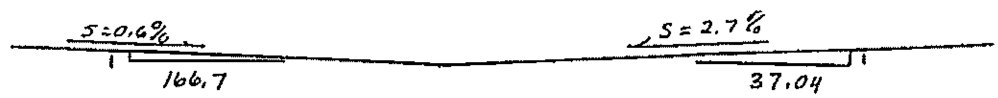
$$= 77{,}804.87\ ft^2 = 1.79\ acres$$

B. Peak flow from 100 yr - 24-hr event to Downspout D3 (identified as Qc in the revised 5/23/96 Cap Hydrology calculations) is 11.68 cfs from a tributary area of 2.33 acres. This provides a flow rate/acre of 11.68/2.33 = 5.01 cfs/acre.

Flowrate to valley area = 5.01 cfs/ac × 1.79 ac = 9.0 cfs

2- Ditch Design - Valley Area

A- Ditch Cross Section



B - Channel Slope = 6.72%

C - Ditch Hydraulics - See Attached sheet.

As indicated on the attached sheet, the velocity is only 2.4 fps. Therefore, the riprap cap should provide adequate protection.

Trapezoidal Channel Flow Calculations using Mannings Equation

Client :	USPCI - CELL 15 CLOSURE	Date :	05-Jul-96
Project No. :	64.44.700	Time :	09:46 AM
Channel Section:	VALLEY TO DOWNSPOUT D3	Compute	MEA

			UNITS
GENERAL CRITERIA:	Design Flow:	9.00	cfs
	Bottom Width:	0.0	feet
	Side Slope1:	166.7	1/m1
	Side Slope2:	37.0	1/m2
	Friction Factor:		
	Assumed D50:	0.33	feet
	Calc n Value:	0.033	
	Used:	0.033	
	Min. Bottom Slope:	0.0672	ft/ft
	Max. Bottom Slope:	0.0672	ft/ft
	Freeboard:	0.50	feet
CALCULATION:	Depth (Min. S):	0.19	feet
(Channel Depth)			
	Q-1.49AR(2/3)S(1/2)/n=	-0.000	Accuracy
	Required Depth:	0.69	feet
	Area:	3.69	ft2
	Perimeter:	38.78	feet
	Hydraulic Radius:	0.10	feet
	Velocity:	2.44	ft/sec
	Riprap Ck (V<5?):	Not Needed	
CALCULATION:	Depth (Max. S):	0.19	feet
(Velocity Check)			
	Q-1.49AR(2/3)S(1/2)/n=	-0.000	Accuracy
	Required Depth:	0.69	feet
	Area:	3.69	ft2
	Perimeter:	38.78	feet
	Hydraulic Radius:	0.10	feet
	Velocity:	2.44	ft/sec
	Riprap Ck (V<5?):	Not Needed	

DESIGN CRITERIA:	Bottom Width:	0.0	feet
	Side Slope 1:	166.7	1/m1
	Side Slope 2:	37.0	1/m2
	Min. Bottom Slope:	6.7	%
	Max. Bottom Slope:	6.7	%
	Min Channel Depth:	0.69	feet