CLEAN HARBORS (DEER TRAIL), LLC.
108555 East Highway 36
Deer Trail, Colorado 80105

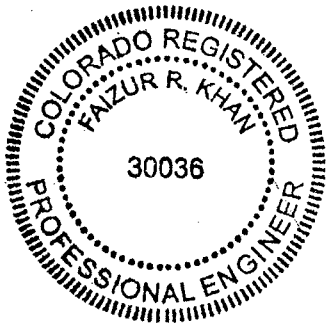
WASTE FILLING PLAN AND DRAWINGS

FOR

SECURE CELL NUMBER 2

OCTOBER 2004
REVISION 1

Prepared by:
Clean Harbors Deer Trail, LLC.
108555 East Highway 36
Deer Trail, Colorado, 80105



Faiz Rah Kh

10 - 22 - 04

TABLE OF CONTENTS

1.0 INTRODUCTION

2.0 REGULATORY CRITERIA

3.0 FILLING PLAN AND COVER CONSTRUCTION
- 3.1 WASTE FILLING PLAN
- 3.2 ACCESS RAMP
- 3.3 INTERIM COVER MAINTENANCE

4.0 ENGINEERING ANALYSIS
- 4.1 STABILITY ANALYSIS
- 4.2 BASE LINER STRESS ANALYSIS
- 4.3 RUNOFF ANALYSIS AND CONTAMINATED WATER STORAGE CAPACITY

LIST OF TABLES

Table No.	Description
1	Summary of Permit Requirements from LF-6
2	Summary of Waste Strength Parameters
3	Summary of Sump Storage Capacity

LIST OF DRAWINGS

Drawing No.	Description
1	Stage 1- Waste Filling Plan
2	Stage 1 – Waste Filling Sections
3	Stage 2 – Sump Filling Plan
4	Stage 2 – Sump Filling Sections
5	Stage 3 – Sump Filling Plan
6	Stage 3 – Sump Filling Sections
7	Stage 4 – Sump Filling Plan

LIST OF APPENDICES

Appendix	Description
1	Stability Analyses
2	Base Liner Stress Analyses
3	Stormwater Storage Volume Calculation

1.0 INTRODUCTION

Clean Harbors Deer Trail, LLC has prepared a plan for the final filling of Secure Cell 2 located in Deer Trail, Colorado.

The objective of the study was to develop a detailed plan for filling of Secure Cell 2 that met both operational and regulatory criteria. The regulatory criteria are presented in the Part B Permit for the site and the associated landfill attachments to the Part B Permit. The specific criteria that are being addressed are detailed in Section 2.0. The operational criteria include maintaining functional haul roads into the cell, providing suitable waste placement areas to allow efficient operations and providing adequate storage areas for contaminated runoff from the active work areas.

The scope of this study included the following tasks:

- Preparation of filling plan that indicated the sequential filling of the cell with waste that maintains sufficient storage capacity within the cell to contain the runoff from the waste from an 8.2-inch rainfall with 1-inch of freeboard.

- Preparation of the sequential cover construction that separates the runoff from the waste areas and from the covered areas. This plan must also maintain sufficient storage capacity within the cell to contain the runoff from an 8.2-inch rainfall with 1-inch of freeboard.

- Preparation of a letter report that describes the filling and cover construction sequence, summarizes the design analyses that were completed for the cover design, summarizes all of the regulatory requirements for landfilling operations and inspections during active operations and during closure.

The major design analyses to support the cover section to be used for Secure Cell 2 were presented in the Class 2 Permit Modification approved in July 1995. The design analyses that were completed as part of this study are related to the following:

- Stability of the waste materials during filling;

- Control of contaminated surface water runoff during filling above the perimeter grade; and

- Control of the uncontaminated (UC) water during and after construction of the cover.

Additional engineering analyses were performed to support the height increase of waste by 6-feet. This height increase was requested by Clean Harbors Deer Trail on August 9, 2004 as a Class 3 Permit Modification request to increase the capacity of the Secure Cell 2 by 30,000 cubic yards. The following analyses were perform to support the height increase:

- Revised slope stability analyses (see Appendix 1);
- Base Liner Stress Analyses (see Appendix 2); and
- Stormwater Storage Volume Calculation (see Appendix 3).

2.0 REGULATORY CRITERIA

The regulatory criteria that address the waste filling operations for each Secure Cell are defined in Permit Attachment LF-6. The criteria are summarized in Table 1.

TABLE 1
SUMMARY OF PERMIT REQUIREMENTS FROM LF-6

Permit Attachment	Section	Requirement
LF-6	II	The leachate collection sump will be inspected for liquids after storm events.
	II	The leachate collection sump will be inspected for the presence of liquids as specified in Part I.D of the permit.
	II	Bulk solids will consist of solidified sludges and direct landfilled cell solids. The bulk solids will consist of solidified sludges and direct landfilled cell solids.
	II	Space between drums will be filled with a bulk solid and compacted.
	II	A geosynthetic cover or six inches of select fill cover will be applied over the waste daily.
	II	The waste will be placed in horizontal lifts and compacted in place.
	III	Intermediate cover shall be placed over waste which has been placed above the crest of the landfill at any time when the waste placement in non-active or waste placed is expected to be non-active for a period of two hours.
	III	During waste placement above the crest, special measures will be taken to prevent the generation of wind blown waste. These measures may include placement of a stockpile of soil or geosynthetic cover near the working face to be used to cover waste immediately after placement, if wind conditions warrant. In addition, soil stabilization products or tackifiers may be used to seal the surface soils and reduce dust generation during high wind periods.
	III	Standing water in the area will be removed using the facility's vacuum truck or portable pumps which discharge to the leachate collection line to the extent necessary to prevent excessive ponding.
	III	During waste placement above the crest, standing surface liquids that collect in the V-ditch trench shall be removed within 12 hours after a storm event.
	III	During waste placement above the crest, special inspections as detailed in Section I.D.9.a.i shall be conducted.

The regulatory criterion for construction of the cover system is detailed in the specifications (LF-1.3) and the Construction Quality Assurance (CQA) plan (LF 10.3). Regulatory criteria for regular inspections of the facility are detailed in the Part B Permit and are not summarized in this document. The design criteria for the perimeter drainage ditches are detailed in the Part B Permit Attachment LF-12 that was included with the Class 2 Permit Modification approved in July 1995.

3.0 FILLING PLAN AND COVER CONSTRUCTION

3.1 WASTE FILLING PLAN

The waste filling plan for Secure Cell 2 has been divided into a series of stages to reach the final waste grading plan. The stages are listed below:

- Final Waste Waste Grading Plan (Stage 1)
- Sump Backfilling Plan (Stages 2, 3 and 4)

A description of the work to be performed during each stage is presented below:

Stage 1: Final Waste Grading Plan

Secure Cell 2 is currently being filled and the waste level has approached the perimeter of the cell. In accordance with the approved Class 2 permit modification (July 1995), the final waste filling plan for the cell will include cover slopes of 5 percent (above the perimeter of the cell). However, at anytime during filling, the cell must have storage capacity for the contaminated (C) water runoff from an 8.2-inch rainfall with 1-inch of freeboard.

Based on the requirement for storage of the runoff from an 8.2-inch rainfall, a sump has been designed to hold the expected runoff in the south side of the cell. The sump area is formed by the south east and west sides of the landfill and 2H:1V slopes in the waste. The sump area is shown on drawings 1 and 2. Runoff from the side slopes is directed into the sump area via a series of V-ditches around the perimeter of the cell. The V-ditches are formed by the side slopes of the landfill and by 2H:1V slopes in the waste. The base grade on the V-ditch is 1 percent. Drawings 1 and 2 indicate the layout for the V-ditches around the perimeter. The V-ditch on secure Cell 2 has been designed with 2H:1V side slopes.

Waste shall be placed above the crest of Secure Cell 2 and will require that the V-ditches and sump area are operational. Waste should be placed starting in the northeast corner and progress to the south. The lift thickness should be approximately 4 feet thick; however the lift thickness will vary as it intersects the final surface contours. The material used for the final 12-inches should not contain any oversized materials that would prevent achieving the final lines and grades shown on Figures 1 and 2.

Stage 2: Sump Filling Plan

The north one-third portion of Secure Cell 2 final waste surface shall receive 6-inches of compacted interim cover soil once waste reaches the final grade as shown in Figure 3. However, the V-ditch will be be backfilled first in the areas receiving interim cover. As the waste is being backfilled into the V-ditch, intermediate soil cover must be placed directly over the waste so that waste will not be exposed in the V-ditch at any time during filling. Clean Harbors shall only proceed with filling the V-ditch when the weather forecast supports this activity. Backfilling of V-ditch with waste shall

not be permitted during rain or inclement weather. Figure 3 and 4 illustrate the Stage 2 sump-filling plan.

Once the north third portion of the final waste surface receives interim cover and the runoff has been re-directed into the perimeter uncontaminated ditch, the contaminated water drainage area into the sump area will be reduced and therefore a portion of the sump can be filled with waste. The extent of the waste placement in the sump, after the north third portion of the final waste surface has been covered with interim soil cover, is shown in Figures 3 and 4.

Stage 3: Continued Sump Filling Plan

The middle one-third portion of Secure Cell 2 final waste surface shall receive 6-inches of compacted interim cover soil once waste reaches the final grade as shown in Figure 5. However, the V-ditch will be backfilled first in the areas receiving interim cover. As the waste is being backfilled into the V-ditch, intermediate soil cover must be placed directly over the waste so that waste will not be exposed in the V-ditch at any time during filling. Clean Harbors shall only proceed with filling the V-ditch when the weather forecast supports this activity. Backfilling of V-ditch with waste shall not be permitted during rain or inclement weather. Figure 5 and 6 illustrate the Stage 2 sump-filling plan.

Once the middle third of the final waste surface receives interim cover and the runoff has been re-directed into the perimeter uncontaminated ditch, the contaminated water drainage area into the sump area will be reduced and therefore a portion of the sump can be filled with waste. The extent of the waste placement in the sump, after the middle third portion of the final waste surface has been covered with interim soil cover, is shown in Figures 5 and 6.

Stage 4: Sump Filling Plan

The south one-third portion of Secure Cell 2 final waste surface shall receive 6-inches of compacted interim cover soil once waste reaches the final grade as shown in Figure 7. In this phase the final capacity of the sump is filled with waste and covered with interim soil cover. As the sump is being filled, additional interim cover will be placed to reduce the volume of the sump while maintaining storm water storage capacity from a 8.2-inch rainfall. The waste filling and cover operation will have to be completed together to prevent potential contaminated water runoff from exiting the landfill. The sump backfill for this stage is shown in Figure 7.

3.2 Access Ramp

A new access ramp is proposed on the south east side of the Secure Cell 2, immediately north of the sump. This new access ramp will eliminate the need to drive over interim cover and will be used to transport waste and daily cover into the cell. The existing ramp located at the northeast corner of Secure Cell 2 will be used to transport interim cover. Interim cover will be spread and compacted with a dozer starting from north to south. Interim cover shall be placed in a sequential manner such that run-off in contact with the waste does not flow over the interim cover.

The movement of waste trucks and all terrain operational equipment should not be a problem given the fact the maximum slope height is only 16 feet for 6 to 1 slope and stability of the side slope is greater than 2.0 even with assuming a very conservative set of strength parameters. Waste trucks are allowed to travel a restricted route in the landfill that is considered safe and stable. Unloading of waste is permitted away from the slopes and in areas designated by the cell operator. Waste truck and heavy equipment will not be allowed to get close to top of sump and the 6 to 1 side slope. A minimum setback distance equal to two times the maximum height will be maintained in order to prevent localized slough-in of waste into the sump. The operational equipment such as bulldozer and compactor will be used for spreading, mixing and compacting of the waste, which will improve the overall stability of the slope.

3.3 Interim Cover Maintenance

At least 6-inches of interim cover soil will be placed upon waste reaching the final grade. The surface of the interim cover will be covered with erosion control matting or straw mulch. The interim cover will be inspected daily and a routine maintenance program will be implemented. The interim cover soil will be reapplied in areas where soil loss is evident due to erosion. The interim cover will be installed in multiple phases in accordance with the revised Waste Filling Plan.

4.0 ENGINEERING ANALYSIS

4.1 Stability Analyses

Stability analyses were conducted to determine if the proposed filling plan and cover construction sequence would result in stable slopes for interim and final conditions. As part of the Class 2 permit modification that was approved in July 1995, calculations for the stability of the cover were prepared for five percent cover slopes. The calculations are presented in the Design Report (LF-13) and previously indicated stable conditions for the following failure planes.

- Infinite slope failure planes through the cover component interfaces
- Circular failure planes through the cover components and the waste
- Overall stability of the waste along a basal sliding plane.

These analyses no longer represent the exiting condition of Secure Cell 2 since waste in Cell 2 has reached an elevation above the crest of the landfill.

Additional slope stability analyses were conducted as part of this study to evaluate the stability of the waste slopes into the sump area and the final waste grade using compound slope. Clean Harbors plans to add additional 30,000 cubic yards of waste (if approved), which will increase the capacity of Cell 2 to 220,600 cubic yards. This will be accomplished by placing waste above the crest of the landfill using a compound slope. The lower and the upper slope are designed to have a slope steepness of 16.6 and 5 percent, respectively. Stability analyses were conducted for circular failure planes extending from the crest of the slope to the toe of the slope. Stability of waste was performed using two sets of assumed strength parameters as follows:

Strength Parameters Set 1: Testing of the waste materials to determine strength parameters is not considered practical, therefore, strength properties were assumed from a previous report entitled " Filling Plan and Cover Design Drawings for Secure Cell Number 1" dated August 1995 and our experience with other hazardous waste materials. The selection of the waste parameter is considered conservative since vast majority of the waste stream is stabilized with cement kiln dust (CKD). The strength properties are summarized in Table 2.

TABLE 2A
SUMMARY OF WASTE STRENGTH PARAMETERS SET 1

Unit Weight	110 pcf
Friction Angle	31 degrees
Cohesion [1]	100 psf

Note 1: The waste material is characterized as debris and non-debris waste. Non-debris waste is either soil like material or waste stabilized with cement kiln dust. Due to the blending and mixing of the waste an unconfined compressive strength requirement of 0.33 tsf, it is appropriate to assume a cohesion of 100 psf (conservative).

Strength Parameters Set 2: Slope stability analyses have been evaluated to determine if the Permit Condition VI.E.8.a is still adequate. The Permit condition VI.E.8.a requires an in-place compressive strength of 0.33 tsf (660 psf) of the waste. However, it is appropriate to assume that the waste mass will possess some frictional property. Therefore, it is conservatively assumed that the in-place waste has a friction angle of 8-degrees. The types of waste placed in the landfill may be classified as debris and non-debris waste. Non-debris waste is either soil like material or waste stabilized with cement kiln dust (CKD). Debris waste is spread in the landfill and mixed with non-debris waste and subsequently compacted with a sheepsfoot compactor. The compaction of the mixed waste fills the voids, resulting in a denser and consolidated waste mass. It should be noted that the unconfined compressive strength of waste is measured prior to landfilling. Therefore, it is expected that the strength properties of the in-place waste should be greater following the compaction and hydration process.

TABLE 2B
SUMMARY OF WASTE STRENGTH PARAMETERS SET 2

Unit Weight	110 pcf
Friction Angle	8 degrees
Cohesion (1)	330 psf

(1) – Cohesion value is equal to one half of the compressive strength. Required compressive strength by Permit condition = 660 psf.

The water condition within the waste slope has been based on very conservative conditions. We have assumed that the sump is completely full from an 8.2-inch rainfall and that the sump is pumped dry in a relatively short period of time. This will result in a rapid drawdown condition within the waste slope below the maximum water elevation. The water levels assumed for the analysis are shown on a cross-section in Appendix 1.

Two cases were analyzed for waste stability. These cases are described as follows:

Case 1: Stability of Side slope and final cover slope:

In this analysis, the worst-case scenario was investigated. The worst case was assumed when the waste conforms to the lower slope (6 to 1) followed by 5 percent upper slope and the waste has reached the maximum permissible height. Static slope stability analyses were conducted. The result indicates a factor of safety of 2.5 and 2.1 for Strength parameters 1 and 2, respectively, which is considered acceptable.

Case 2: Stability of Storm water Storage Sump:

In this analysis, the stability of sump slope was investigated. The sump is located in the south side of Secure Cell 2 and is formed by the south east and west sides of the landfill and 2H:1V slopes in the waste on the north side of sump. The north side of the sump will have a maximum 15-feet high slope at 2 to 1 when waste reaches the maximum capacity. Based on these conditions, static slope stability analyses were conducted. The result indicates a factor of safety of 1.7 and 1.6 for strength parameters 1 and 2, respectively, which is considered acceptable.

Waste will be placed in the landfill within 2 to 3 years and therefore dynamic stability due to seismic event is not considered for this short term filling operation. Seismic stability is normally performed for areas with a 10 percent or greater probability that the maximum horizontal acceleration will exceed 0.10g in 250 years. In addition, based on the EPA seismic design guidance document, the Deer Trail facility is outside the limits of "seismic impact zone" (see attached reference).

It should be recognized that if the sump is filled and pumped out over a relatively short period of time (a few days) that some surficial slumping of the operations layer on the side slopes of the sump might occur. However, it is not expected that this will impact the performance of the cell and could easily be replaced during waste filling in the sump. Slope stability analysis is presented in Appendix 1.

4.2 BASE LINER STRESS ANALYSIS

The effect of 6-additional feet of weight on the base and the slope liner system has been evaluated. Based on the analysis, the base liner will experience only 0.27 % strain, which is below the manufacturer allowable limit of 12 %. Therefore, the base liner should perform satisfactorily without yielding. The liner stress analysis is presented in the Waste Filling Plan (Appendix 2).

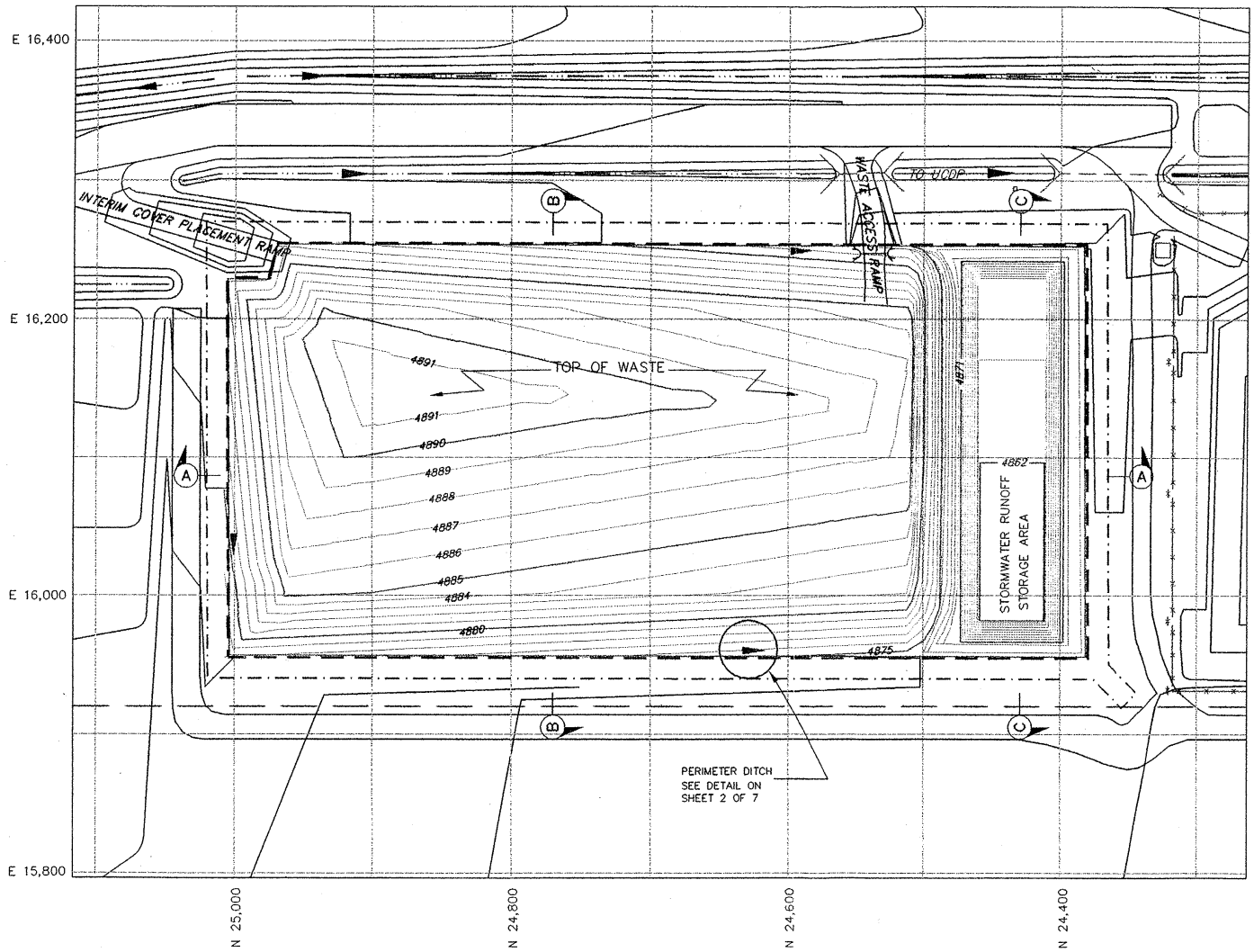
4.3 RUNOFF ANALYSIS AND CONTIMINATED WATER STORAGE CAPACITY

The contaminated water runoff from the exposed waste was determined for each stage of filling. The runoff was calculated assuming 100 percent runoff from the waste. This is considered to be a very conservative assumption (i.e.; evaporation, infiltration and absorption was neglected), but appropriate for sizing the sump capacity. The required runoff storage capacity can be estimated by multiplying the area of Cell 2, not covered with the interim soil cover, by 8.2-inches.

The required sump storage capacity was calculated to be equal to the area of Cell 2 not covered with interim soil cover multiplied by 8.2-inches for any stage of filling. Therefore, the required sump capacity could be reduced as interim soil cover was placed over portion of the cell.

Table 3 presents a summary of the required sump storage capacity and available sump storage capacity for each stage of waste placement. Detailed calculations for the runoff volumes and the available storage capacity are presented in Appendix 3 and Figures 1, 3, and 5.

TABLE 3 SUMMARY OF SUMP STORAGE CAPACITY				
Stage of Filling	**Referenced Figures**	**Description**	**Storage Capacity Required (Cubic Feet)**	**Sump Capacity Available (Cubic Feet)**
1	1	Final waste grading plan	122,500	134,000
2	3	Place Interim Cover on Northern Third of Final Surface	86,000	96,500
3	5	Place Interim Cover on Middle Third of Final Surface	45,000	54,400
4	7	Place Interim Cover on Southern Third of Final Surface	0	0



LEGEND

- CULVERT
- ANCHOR TRENCH
- CREST OF CELL
- DIRECTION OF FLOW
- FENCE

UCDP. – UNCONTAMINATED RAINWATER DETENTION POND

NOTE:
STORMWATER STORAGE REQUIREMENT:
100 YR. 24 HR RAIN STORM = 8.2"

REQUIRED SUMP CAPACITY =
AREA (179,134 SQ. FT.) x 8.2/12 = 122,500 CU. FT.

AVAILABLE SUMP CAPACITY = 134,000 CU. FT.

COLORADO REGISTERED
FAIZUR R. KHAN
30036
PROFESSIONAL ENGINEER

Faizur Rahman
10-22-04

SCALE
0 40 80 120

Rev. No	NOTES	DATE	BY	CHKD
0	ISSUED FOR REVIEW	8/25/04	WDS	FK

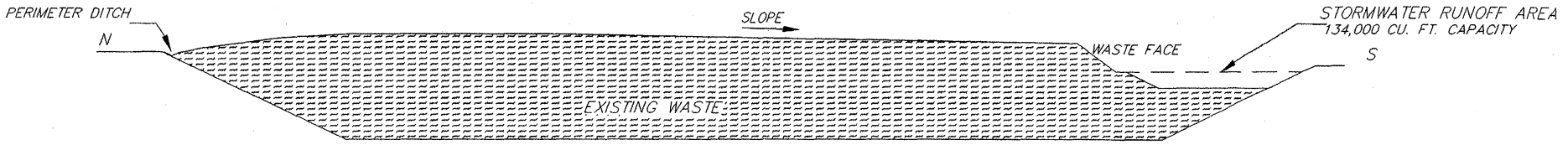
SCALE: SEE BAR SCALE
DESIGN BY: WDS
DRAWN BY: WDS
CHECK BY: FK
PROJECT No.
FILE NAME: DEERTRAIL-00A

CleanHarbors DEER TRAIL, LLC

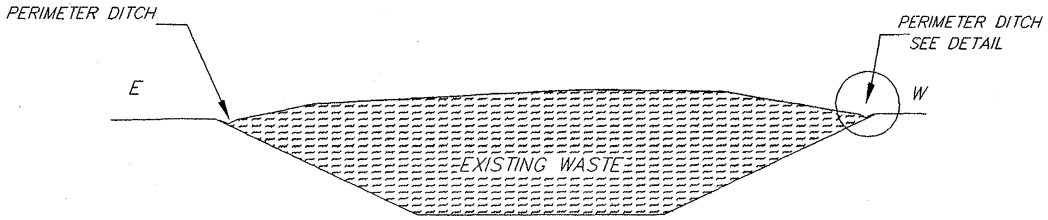
PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2
DRAWING TITLE: WASTE FILLING PLAN STAGE 1

SHEET: 1 OF 7
DRAWING No.: 001
REV.: 0

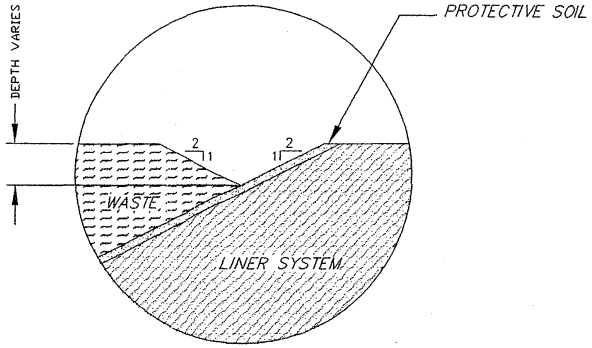
FIGURE 1



SECTION A–A



SECTION B–B



PERIMETER DRAINAGE DITCH DETAIL

SECTION C–C

COLORADO REGISTERED
FAIZUR R. KHAN
30036
PROFESSIONAL ENGINEER

Faiz Rab Khan
10-22-04

Rev. No.	NOTES	DATE	BY	CHKD
0	ISSUED FOR REVIEW	8/30/04	WDS	FK

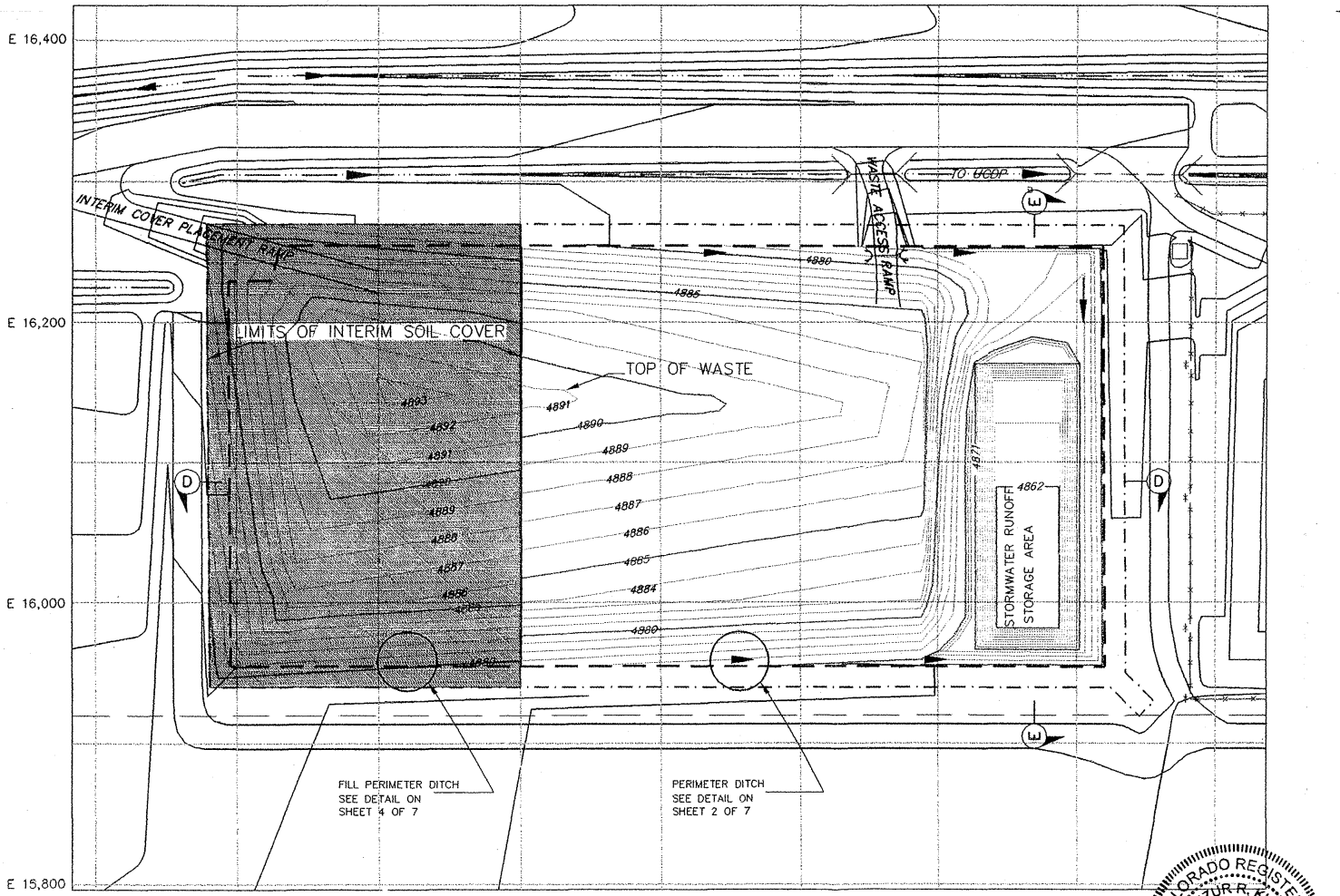
SCALE: SEE BAR SCALE
DESIGN BY: WDS
DRAWN BY: WDS
CHECK BY: FK
PROJECT No.
FILE NAME: DEERTRAIL-008

Clean Harbors DEER TRAIL, LLC.

PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2
DRAWING TITLE: WASTE FILLING SECTIONS STAGE 1

SHEET: 2 OF 7
DRAWING No.: 001
REV.: 0

FIGURE 2



LEGEND

- CULVERT
- ANCHOR TRENCH
- CREST OF CELL
- DIRECTION OF FLOW
- FENCE

UCDP - UNCONTAMINATED RAINWATER DETENTION POND

NOTE:
STORMWATER STORAGE REQUIREMENT:
100 YR. 24 HR RAIN STORM = 8.2"
REQUIRED SUMP CAPACITY =
AREA (125,575 SQ. FT.) x 8.2/12 = 86,000 CU. FT.
AVAILABLE SUMP CAPACITY = 96,500 CU. FT.

SCALE
0 40 80 120

COLORADO REGISTERED
FAZUR R. KHAN
30036
PROFESSIONAL ENGINEER
10-22-04

Rev. No.	NOTES	DATE	BY	CHKD
0	ISSUED FOR REVIEW	8/30/04	WDS	FK

SCALE: SEE BAR SCALE
DESIGN BY: WDS
DRAWN BY: WDS
CHECK BY: FK
PROJECT No.
FILE NAME: DEERTRAIL-00C

CleanHarbors
DEER TRAIL, LLC.

PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2
DRAWING TITLE: WASTE FILLING PLAN STAGE 2

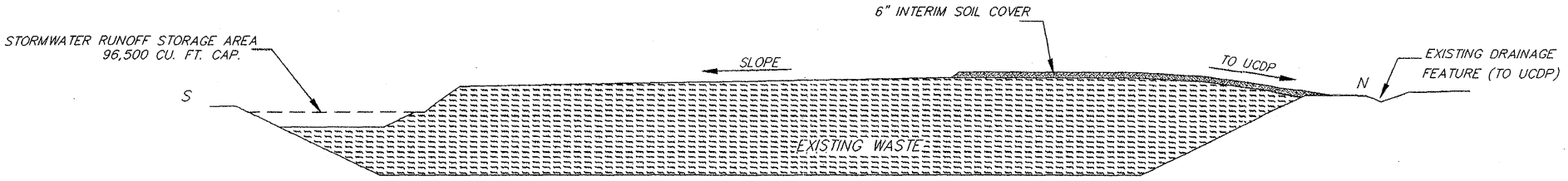
FIGURE 3

SHEET: 3 OF 7
DRAWING No.: 001
REV.: 0

LEGEND

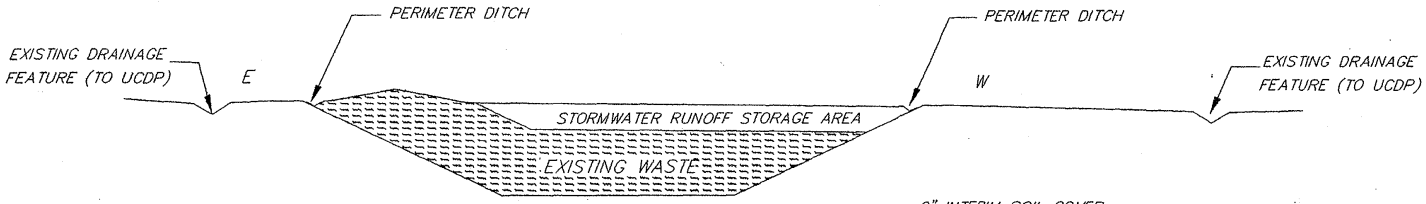
UCDP – UNCONTAMINATED RAINWATER DETENTION POND

FLOW DIRECTION



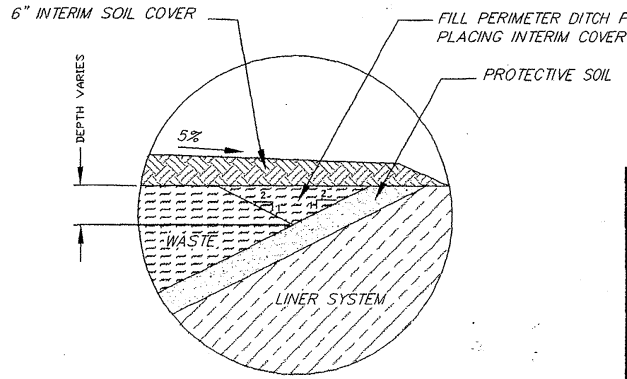
SECTION D–D

SEE SHEET 3 OF 7



SECTION E–E

SEE SHEET 3 OF 7



PERIMETER DRAINAGE DITCH FILLING DETAIL

COLORADO REGISTERED
FAIZUR R. KHAN
30036
PROFESSIONAL ENGINEER

10-22-04

Rev. No	NOTES	DATE	BY	CHKD
0	ISSUED FOR REVIEW	8/30/04	WDS	FK

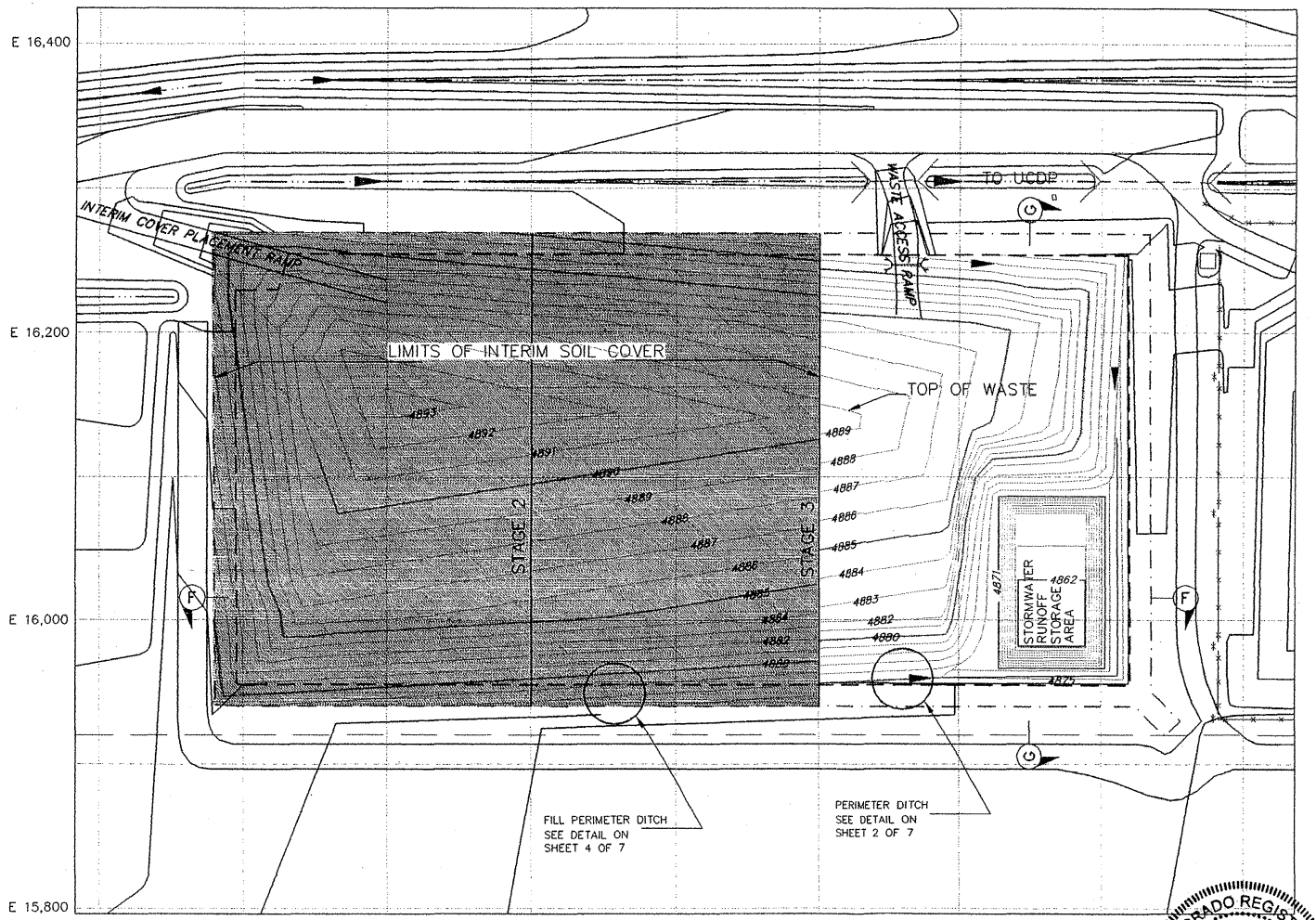
SCALE: SEE BAR SCALE
DESIGN BY: WDS
DRAWN BY: WDS
CHECK BY: FK
PROJECT No.
FILE NAME: DEERTRAIL-000

CleanHarbors DEER TRAIL, LLC.

PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2
DRAWING TITLE: WASTE FILLING SECTIONS STAGE 2

FIGURE 4

SHEET: 4 OF 7
DRAWING No.: 001
REV.: 0



LEGEND

- CULVERT
- ANCHOR TRENCH
- CREST OF CELL
- DIRECTION OF FLOW
- FENCE

UCDP – UNCONTAMINATED RAINWATER DETENTION POND

NOTE:
STORMWATER STORAGE REQUIREMENT:
100 YR. 24 HR RAIN STORM = 8.2"
REQUIRED SUMP CAPACITY =
AREA (65,680 SQ. FT.) x 8.2/12 = 44,881 CU. FT.
AVAILABLE SUMP CAPACITY = 54,400 CU. FT.

SCALE
0 40 80 120

COLORADO REGISTERED
FAIZUR R. KHAN
30036
PROFESSIONAL ENGINEER
10-22-04

Rev. No	NOTES	DATE	BY	CHKD
4				
3				
2				
1	REVISED TO SHOW NEW OWNERSHIP	8/04	WDS	FK
0	ISSUED FOR REVIEW	8/31/04	WDS	FK

SCALE: SEE BAR SCALE	Clean Harbors DEER TRAIL, LLC
DESIGN BY: WDS	
DRAWN BY: WDS	
CHECK BY: FK	PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2
PROJECT No. 603	DRAWING TITLE: WASTE FILLING PLAN STAGE 3
FILE NAME: DEERTRAIL-DOE	

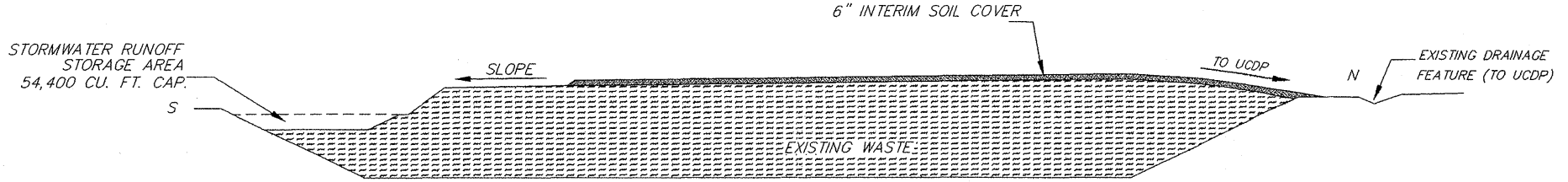
FIGURE 5

SHEET: 5 OF 7
DRAWING No.: 001
REV.: 0

LEGEND

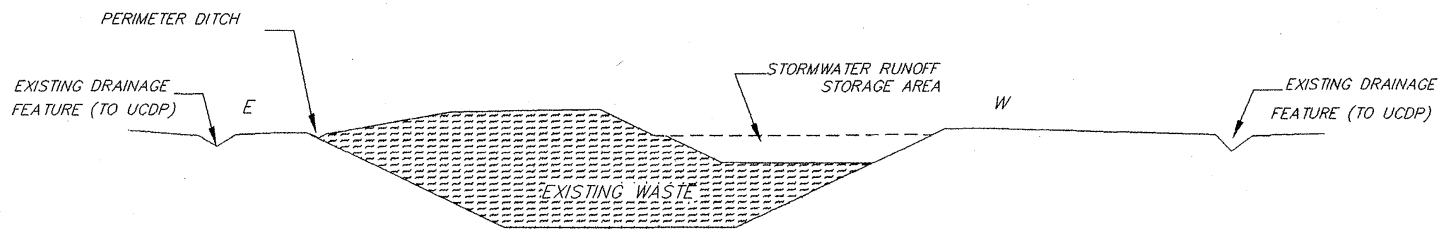
UCDP – UNCONTAMINATED RAINWATER DETENTION POND

FLOW DIRECTION



SECTION F–F

SEE SHEET 5 OF 7



SECTION G–G

SEE SHEET 5 OF 7

COLORADO REGISTERED
FAZUR R. KHAN
30036
PROFESSIONAL ENGINEER

10-22-04

Rev. No.	NOTES	DATE	BY	CHKD
4				
3				
2				
1				
0	ISSUED FOR REVIEW	8/31/04	WDS	FK

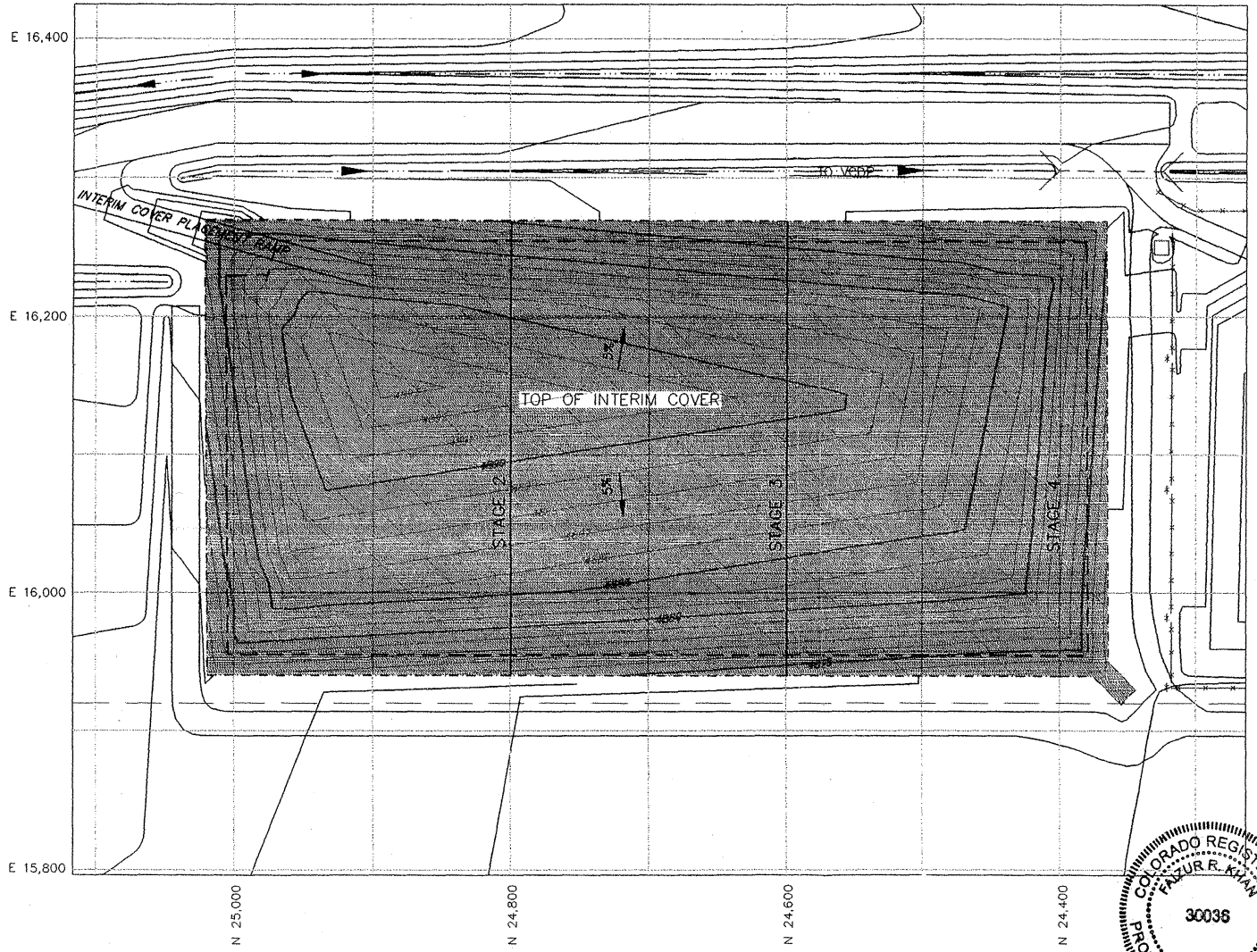
SCALE: SEE BAR SCALE
DESIGN BY: WDS
DRAWN BY: WDS
CHECK BY: FK
PROJECT No.
FILE NAME: DEERTRAIL-00F

CleanHarbors DEER TRAIL, LLC

PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2
DRAWING TITLE: WASTE FILLING PLAN STAGE 3

FIGURE 6

SHEET: 6 OF 7
DRAWING No.: 001
REV.: 0



LEGEND

- CULVERT
- ANCHOR TRENCH
- CREST OF CELL
- DIRECTION OF FLOW
- FENCE

SCALE

0 40 80 120

COLORADO REGISTERED
FAIZUR R. KHAN
30036
PROFESSIONAL ENGINEER

10-22-04

Rev. No	NOTES	DATE	BY	CHKD
0	ISSUED FOR REVIEW	8/31/04	WDS	FK

SCALE: SEE BAR SCALE
DESIGN BY: WDS
DRAWN BY: WDS
CHECK BY: FK
PROJECT No.
FILE NAME: DEERTRAIL-006

CleanHarbors DEER TRAIL, LLC.

PROJECT: CLEAN HARBORS (DEER TRAIL) CELL 2

DRAWING TITLE: WASTE FILLING PLAN STAGE 4

FIGURE 7

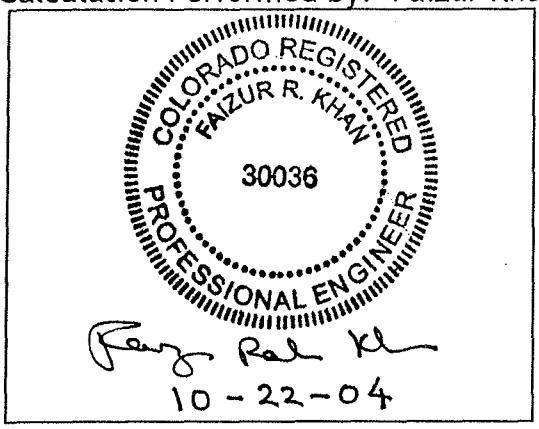
SHEET: 7 OF 7
DRAWING No.: 001
REV.: 0

Appendix 1

Slope Stability Analysis

Calculation Summary Sheet

Project Name: Clean Harbors Deer Trail, LLC
Date: October 22, 2004
Calculation Name: Slope Stability Analyses
Original? Y N Revised? Y N
Calculation Performed by: Faizur Khan



Seal of Certifying P.E.

See Attached Figures

Graphical Representation

PROBLEM: Determine stability of Cell 2 waste fill slopes for the proposed 220,000 cubic yard capacity.

ANALYTICAL APPROACH: The stability analysis will be performed for two cases having the greatest height (worst case).

a) Case 1: Stability of side slope for 6 to 1 steepness (9.46 degrees).

b) Case 2: Stability of sump side slope for 2 to 1 steepness (26.6 degrees).

For each case, stability of waste will be investigated using two separate sets of shear strength of waste as follows:

Strength parameter #1: Friction Angle (ϕ) = 31 degrees
Cohesion (C) = 100 PSF
Unit Weight (γ) = 110 PCF

The above strength parameters were used to perform stability analysis for secure Cell No. 1 by TerraMatrix in 1994. The materials to be disposed in Cell 2 will vary based market demands. The types of waste expected to be disposed in Cell 2 may be categorized as debris and non-debris type waste. Non-debris type wastes are either solid type or waste stabilized with cement kiln dust (which meet the 0.33 TSF unconfined compressive strength requirement for direct landfill). The

standard operational practice is to spread out the debris waste and mix with non-debris waste, which fills the void spaces resulting in a denser, monolithic waste mass. In addition, the mixed waste is subsequently compacted with sheepsfoot compactor. Therefore, it is anticipated that the strength parameters acquired for this analysis is very conservative given the fact a good percentage of the waste is stabilized with cement kiln dust.

Strength parameter #2: Friction Angle (ϕ) = 8 degrees
Cohesion (C) = 330 PSF
Unit Weight (γ) = 110 PCF

Permit condition VI.E.8.a requires that the waste be compacted in place to an unconfined compressive strength of 0.33 tons per square foot, which converts to a cohesion of 330 PSF. Due to the debris disposed in Cell 2, it is appropriate to include a small contribution of frictional property to the strength. Therefore, a friction value of 8 degrees is conservatively selected for this purpose.

ASSUMPTIONS: The Cell 2 disposal capacity is planned to increase by 30,000 cubic yards. This will be accomplished by using a 6 to 1 side slope followed by final cover slope. The final waste height will be raised by only 6-feet from the original top of waste contour.

FACTOR OF SAFETY AND BASIS

Short term F.S. = minimum 1.5 against circular and sliding stability

F.S. = Σ Resisting Forces/Σ Driving Forces

REFERENCES USED: a) *Geotechnical Properties Of Municipal Solid Wastes* by Jeffney Fassatt.

b) Filling and Cover Design for Secure Cell 1 Drawing prepared by Terra Matrix dated August 1995.

c) Clean Harbors (Deer Trail) final permit dated March 1998.

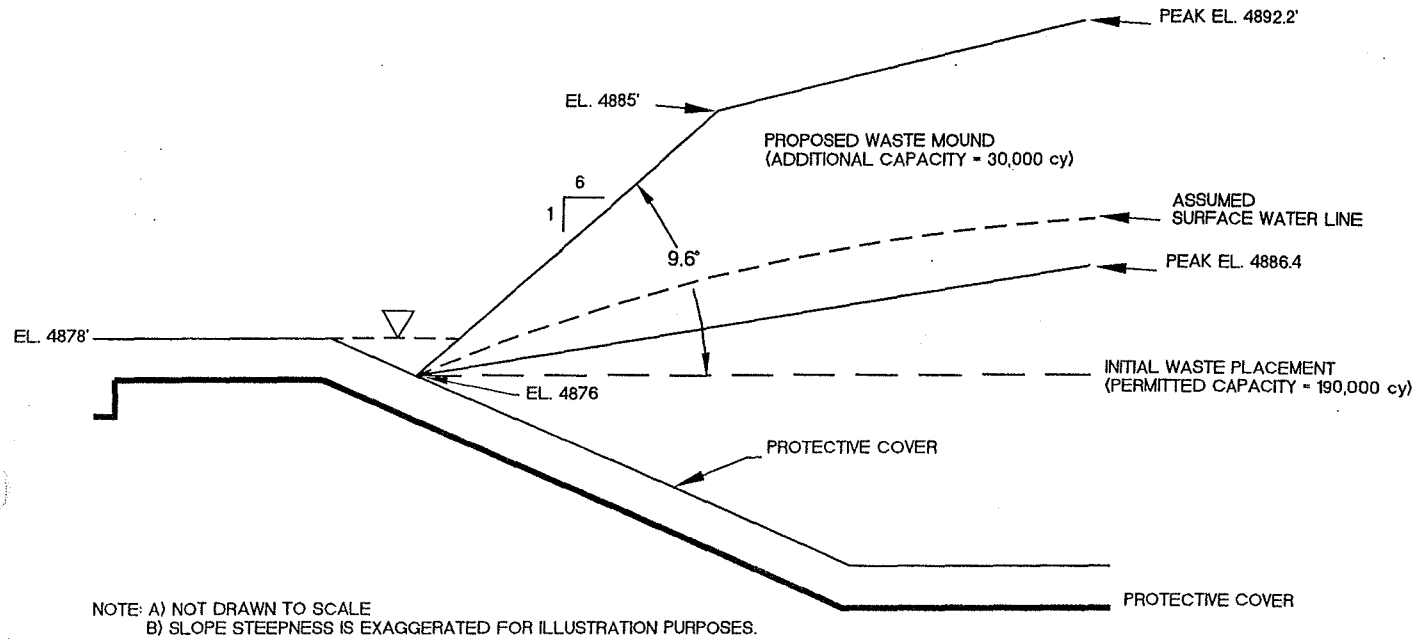
d) Design of Waste Containment Liner and Final Cover System - American Society of Civil Engineer Seminar - 1998

e) *Foundation Analysis And Design* by Joseph Bowels.

Calculation Summary Sheet

SOLUTION:

Case 1. Stability of side slope for 6 to 1.



CASE 1: ANALYTICAL PROFILE FOR SIDE SLOPE STABILITY OF WASTE

Assume maximum waste height = 16 feet

Side Slope = 6:1 = 9.46 degrees

Maximum height is conservatively assumed to be 16 feet at 6 to 1 slope (although first 9 feet at 6 to 1 and the upper 7 feet at 5% slope).

ANALYSIS 1: Based on hook chart (see attached) and using strength parameter #1.

$$C / (\gamma \ .H.\text{Tan}\ \phi) = 100 / ((110\ \text{PCF})\ (16\ \text{Ft})\ (\text{Tan}\ 31^\circ)) = 0.10$$

Calculation Summary Sheet

$C / (\gamma \ .H.F) = 0.025$ (from attached circular failure chart #2)

$F = C / ((.025) (\gamma\ ) (H)) = 100 / ((.025) (110\ PCF) (16)) = 2.5 > 1.5$ OK

ANALYSIS 2: Using strength parameter #2

$C / (\gamma \ .H.\text{Tan}\ \phi) = 330 / ((110\ PCF) (16\ Ft) (\text{Tan}\ 8^\circ)) = 1.33$

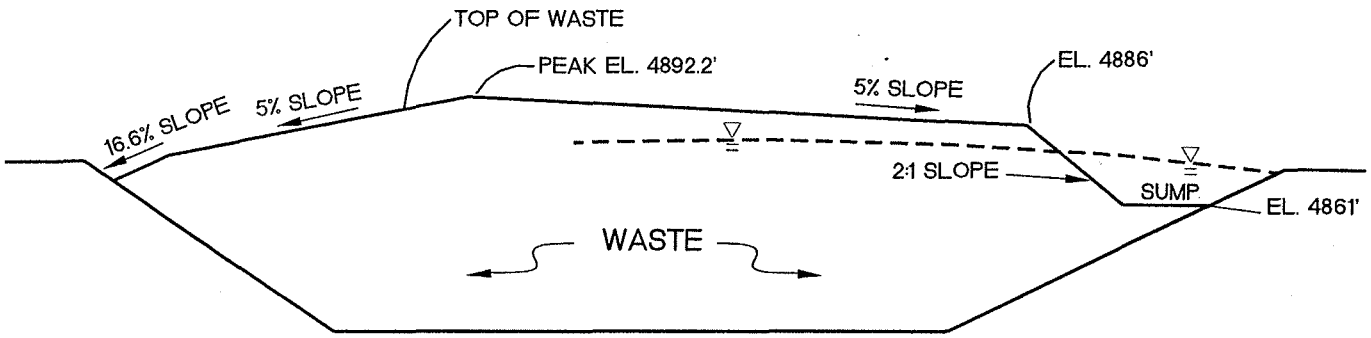
$C / (\gamma \ .H.F) = 0.09$ (from attached circular failure chart #2)

$F = 330 / ((.09) (110) (16))$

$F = 2.08 > 1.5$ OK

Calculation Summary Sheet

Case 2. Stability of Sump Side-Slope



CASE 2: STABILITY OF SUMP SIDE SLOPE

Given: Maximum slope height = 15-feet at 2 to 1 slope

Maximum water elevation = 10 feet assuming the slope is full with water.

Slope steepness = 2 to 1 = 26.6 degrees.

ANALYSIS 3. Based on hook chart and using strength parameter 1.

$C / (\gamma \cdot H \cdot \tan \phi) = 100 / ((110 \text{ PCF}) (15 \text{ Ft}) (\tan 31^\circ)) = 0.100$

$C / (\gamma \cdot H \cdot F) = 0.035$ (from attached circular chart #2)

$F = C / ((.035) (\gamma \cdot H)) = 100 / ((.035) (110) (15))$

F = 1.73 > 1.5 OK

ANALYSIS 4. Based on hook chart and using strength parameter 2.

$C / (\gamma .H.\tan \phi) = 330 / ((110)(15\ Ft)(\tan 8^\circ)) = 1.42$

$C / (\gamma .H.F) = 0.125$

$F = C / ((0.125)(\gamma)(H)) = 330 / ((.125)(110)(15))$

F = 1.6 > 1.5 OK

CONCLUSIONS: Slope stability analyses were performed based on worst-case scenario. The conclusions are as follows:

1. Stability of waste with 6 to 1 and 5 percent compound slope is satisfactory.

2. Stability of waste at sump location having 2 to 1 side slope is satisfactory.

3. Waste will be placed in the landfill in short duration (between 1 to 2 years) and therefore dynamic stability due to seismic event is not considered for filling operation. Seismic stability normally is performed for a return period of 250 years. In addition, based on the EPA seismic design guidance, the Deer Trail facility is outside the limits of " seismic impact zone" (see attached reference).

4. The movement waste trucks and all terrain operational equipment should not be a problem given the fact the maximum slope height is only 16 feet for 6 to 1 slope and Stability of the side slope is greater than 2.0 even with conservative strength parameter. Waste truck and heavy equipment should not be allowed to get close to

top of sump and 6 to 1 side slope. A set-back equal to two times the maximum height shall be maintained in order to prevent localized slough-in of waste into the sump. The operational equipment such as bulldozer and compactor will be used for spreading and compacting the waste, which improve the stability of the slope.

Cited References

RCRA SUBTITLE D (258)
SEISMIC DESIGN GUIDANCE
FOR
MUNICIPAL SOLID WASTE LANDFILL FACILITIES

Prepared By:

Gregory N. Richardson
G.N. Richardson & Associates
Raleigh, North Carolina 27603

and

Edward Kavazanjian, Jr.
and
Neven Matasović
GeoSyntec Consultants
Huntington Beach, California 92648

Tel #(714) 969-0800
Fax #(714) 969-0820
2100 Main Street
Ste 150
Huntington Beach
CA 92648

PROJECT MANAGER

Robert Landreth
Waste Minimization, Destruction and
Disposal Research Division
Risk Reduction Engineering Laboratory
Cincinnati, Ohio 45268

Risk Reduction Engineering Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

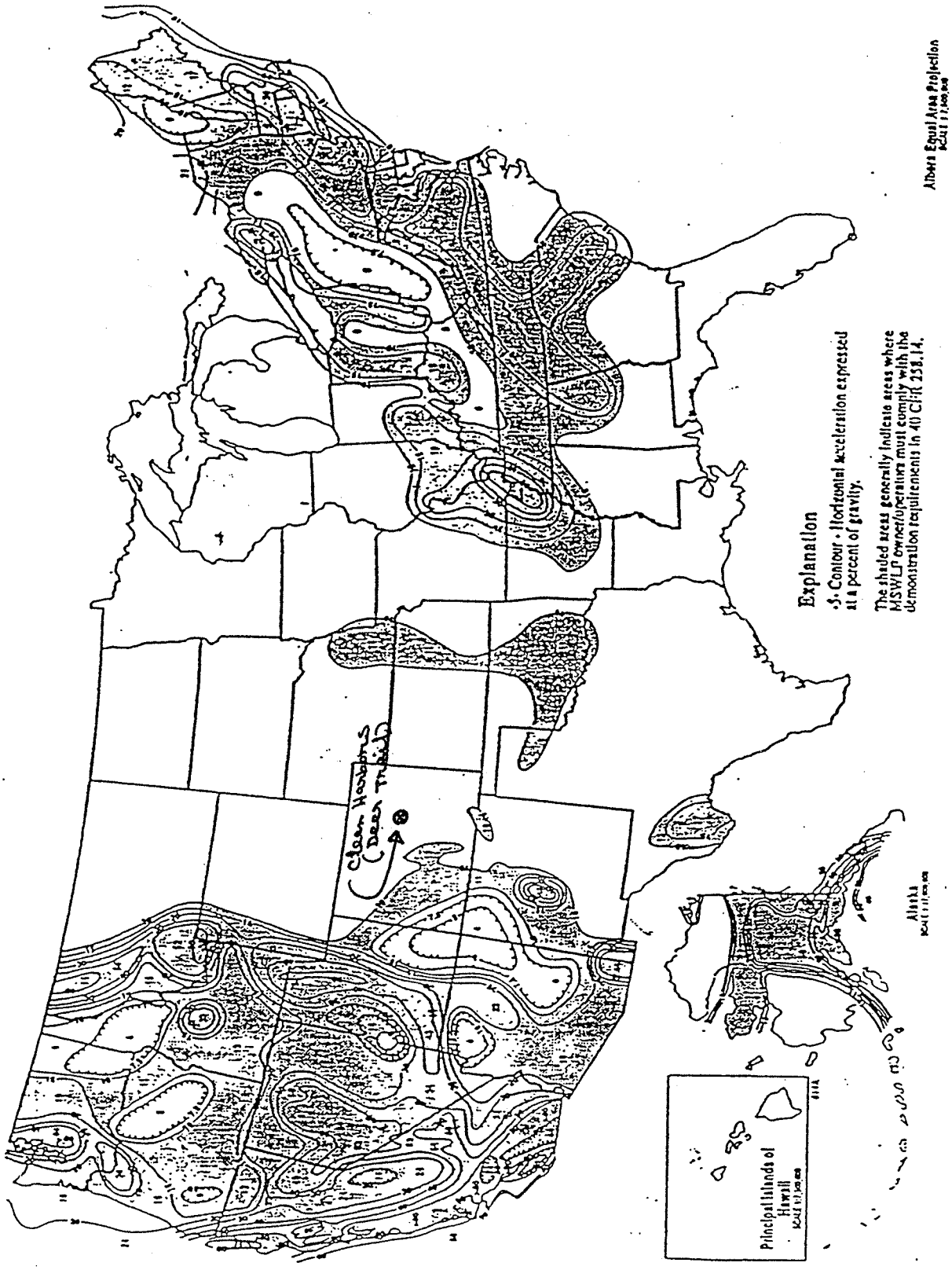


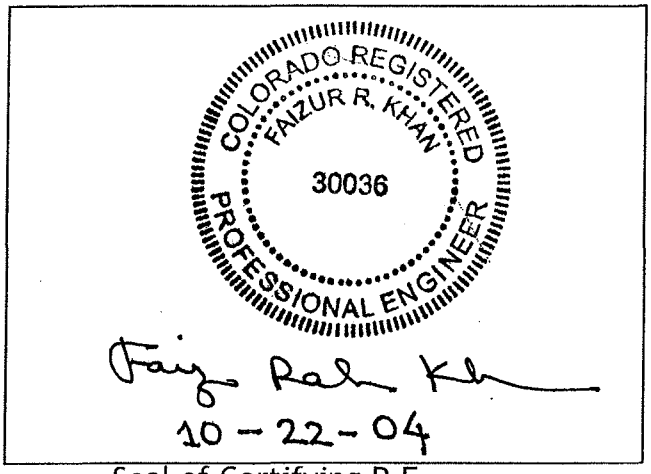
Figure 1.1 Seismic Impact Zones (Areas With a 10% or Greater Probability that the Maximum Horizontal Acceleration Will Exceed 0.10 g in 250 Years) (EPA, 1993).

Appendix 2

Base Liner Stress Analysis

Calculation Summary Sheet

Project Name: Clean Harbors (Deer Trail), LLC
Date: October 22, 2004
Calculation Name: Base Liner Strain Analysis
Original? Y N Revised? Y N



Seal of Certifying P.E.

See Attached Figure

Graphical Representation

PROBLEM: Estimate the amount of strain experienced by the base liner as a result of Settlement.

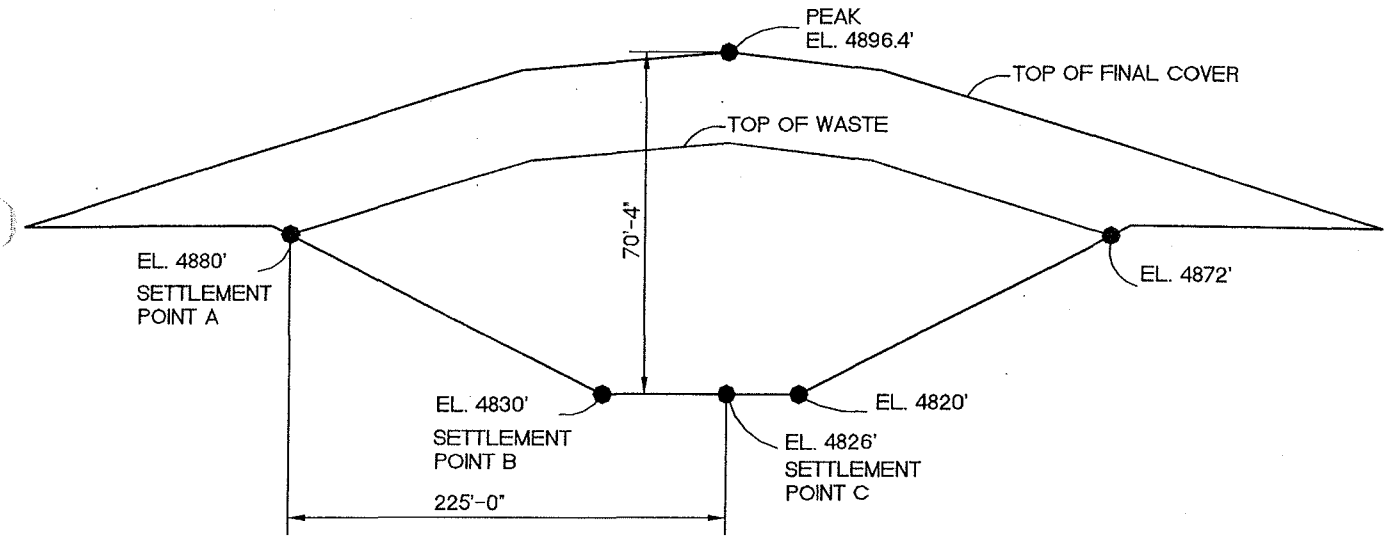
GIVEN: The settlement of the base liner was performed using both uni-axial stress and bi-axial stress calculation. Settlement difference (differential settlement) between two adjacent analysis points was investigated to determine the worst-case scenario. Settlement of the subgrade for Deer-Trail Landfill was performed by TerraMatrix (see attached) as a part of permit modification in 1994. The analyses were performed on the deepest Cell, which is Cell 5 to simulate the worst-case settlement scenario. The settlement for Cell 5 was performed based on maximum height of waste (including liner material) of 96.8 feet (i.e.; 72' below grade +24.8' above grade). The proposed Cell 2 mounding will increase the waste height by only 6-feet. The maximum height of waste (including liner material) for Cell 2 will be 70-feet, which is substantially below the worst-case scenario of 96.8-feet previously investigated by TerraMatrix. A cross-section of Cell 2 showing the maximum height is presented in Figure 1. Therefore, the settlement analyses performed for Cell 5 by Terramatrix still represents the worst-case scenario and is a very conservative estimate for Cell 2. A total of three settlement points (Points A, B and C) were investigated by TerraMatrix to simulate the worst case. These points are located at the top of cell, toe of cell and the center,

Calculation Summary Sheet

where it represents the maximum height of waste. Based on the attached calculation performed by Terramatrix, for the base liner system, the total settlements are as follows:

Point A (Top of Cell):	0.039	
Point B (Toe of Cell):	0.065	0.026 (Point A to B)
Point C (Center of Cell):	0.072 (maximum)	0.033 (Point A to C)

A graphical presentation of Cell 2 having maximum settlement is shown below.



ANALYTICAL PROFILE OF CELL 2

ASSUMPTIONS: The amount of settlement will be from 100 percent (%) of the ultimate settlement caused by the weight of waste and the liner system including final cover. For stress analysis, a differential settlement between analysis points will be used (between Points A and C). The value is .033 feet. Uni-axial and Bi-axial subsidence will be used to calculate the liner stress. Settlement estimated by TerraMatrix Engineering and Engineering Service, which is still valid for Cell 2. The strain at yield (conservative) for the flexible geomembrane liner is 12 %.

Calculation Summary Sheet

FACTOR OF SAFETY AND BASIS: For the uni-axial stress analysis a factor of safety of 5 will be used to determine the basis for passing or failing. For the bi-axial stress analysis, the yield stress of the membrane (as reported by the manufacturer) will be compared to the estimated bi-axial stress.

REFERENCES USED:

a) Settlement Analysis in Appendix F (LF-13), Class 2 Permit Modification dated November 1994, Highway 36 Land Development Company prepared by TerraMatrix Engineering & Environmental Services (attached).

b) Design of waste containment liner and final cover systems, ASCE Seminar.

c) Designing with geosynthetis, Second Edition by Robert Koerner.

SOLUTION: Uniaxial Stress Analysis of Base Liner: (see attached reference)

Given:

L = Length for differential settlement = 225 feet (from Point A to C); however a 25-feet length will be use for localized settelement (conservative)

Depth of Waste (including liner thickness) = 70 feet

S = Defferential Settlement depth = .033 feet

Yield strength of geomembrane = 12 %

1. Calculate Settlement Ratio, SR
   SR = S/2L= .033 ft /50 ft = .00066

2. Obtain Uniform Strain, ϵ_{unif}
   From Figure => ϵ_{unif} = 0.1 %

3. Strain at Yield, ϵyield
   ϵyield = 12.0 % (manufacturer data attached)

4. Calculate Design Ratio, DR
   DR = ϵ_{rupt}/ ϵ_{unif} = 12/0.1 =120 >5 OK

Bi-axial Stress Analysis of Base Liner (see attached reference)

Given:

Maximum differential settlement =.033 feet = d

D = 25 feet

Analysis:

Calculation Summary Sheet

$$R = \frac{D^2 + 4d^2}{8d} = 2367.4$$ see attached for definition of terms

$$z = \frac{4dD}{D^2 - 4d^2} = 0.0053$$

$\theta = 2 \operatorname{atan}(z) = .607$ degree $= 0.0105888$ rad

$L = R \cdot \theta = 2367.4 * .0105888 = 25.068$ feet

$$\epsilon = \frac{(L - D) \cdot 100}{D} = 0.27\ \% < 12$$ yield stress of geomembrane OK

CONCLUSION: The factor of safety for uni-axial strain is 120, which is above the desired factor of safety of 5. The elongation at yield of the geomembrane is 0.27 %, which is below the manufactures limit of 12 %. The base liner should perform without yielding.

PASS/FAIL? PASS

ADDITIONAL COMMENTS: The geotextile and the geonets used in the base liner system have a minimum elongation at yield that is greater than that of the value of a geomembrane. Therefore, geomembrane strain is the most critical. Separate evaluations for geotextile and geonet strains are not necessary.

TERRAMATRIX INC.

CALCULATION COVER SHEET

PROJECT TITLE	Highway 36 Class 2 Permit Modification
PROJECT NO.	603
CALCULATION TITLE	Calculate Subgrade Settlement

	NAME	DATE
PREPARED BY	J. Souther	7/27/94
CHECKED BY	J. Pellicer	9/15/94
REVIEWED BY	P. Corser	10/11/94

REVISIONS	DATE	BY	CHECKED BY	REVIEWED BY
UPDATE REF. FIG	10/7/94	JTS	P. Corser	

TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name: HWY 36
Project Number: 603-2000 Sheet: 1 Of 16
Prepared By: JTS Date: 7-27-94
Checked By: J. Pellicer Date: 9/15/94

OBJECTIVE: CALCULATE SUBGRADE SETTLEMENT.

METHODOLOGY — ELASTIC SETTLEMENT THEORY. (pp. 169-177 - Fang 1991.) (Sheets 2-4)

Assumptions — Groundwater Table well below the base of the excavation
— stratigraphy similar to GeoSyntec 1990. (Sheet 5)
0 - 24' silty clay
24 - 48 weathered shale
> 48' unweathered shale
(see attached fig)
(based on a summary of all site borings)

cell plan: TERRAMATRIX DRAFT - LARGEST (DEEPEST) CELL CCA # 5 (Sheet 6)

assume 1) 4:1 waste fill
2) horiz waste fill. (8% crown)

Estimated Physical Parameters

SOIL TYPE	ν	E	GeoSyntec 1990 TABLE 1
SITY CLAY n: 11 to 62	0.35	480,000 ✓	C = 800 psf PI = 24 LL = 44
weathered shale n: 32 - 62	0.35	600,000 ✓	C = >2000 psf PI = 36 LL = 58
unweathered shale n = > 50	0.30	72,000,000 ✓ (conservative)	C = >8,000 psf NA

from OM 7.1 - 215 TABLE 2. (Sheet 7-8)

SILTY CLAY OCR < 3 PI < 30 $E_u/c = 600$ $E_u = 600 \times 8000 = 480{,}000$ psf

WEATHERED SHALE " " 30 < PI < 50 $E_u/c = 300$ $E_u = 300 \times 2{,}000 = 600{,}000$

UNWEATHERED SHALE → (Sheets 9-10)

$= 1.20 \times 10^7$ kN/m² $\times$ 20.886 = 250,632,000 psf] Ref Jumiki's 1983, p. 181

624.2-2000
JPP
9/15/94
2/16

FOUNDATION ENGINEERING HANDBOOK

Second Edition

Edited by

HSAI-YANG FANG Ph.D.

Professor of Civil Engineering and Director, Geotechnical Engineering Division, Fritz Engineering Laboratory, Lehigh University

1991

VNR VAN NOSTRAND REINHOLD
New York

and less than at the edges (Fig. 5.2b). Contact stress distributions are important for the design of foundations and footings Bowles, 1975a). For the structural design of footings, a linear contact stress distribution is often assumed although this is obviously incorrect from a soil mechanics point of view.

5.4.2 Immediate Settlement of Cohesive Foundations

For soils that are predominately cohesive, linear theory of elasticity is used to estimate the magnitude of initial settlements. Soil profiles are typically simplified, although some solutions involving multiple layer theory are available. Homogeneity and isotropy are implicitly assumed so that only two elastic parameters, the modulus of elasticity E and Poisson's ratio ν are needed. This approach works reasonably well on clay soils if the applied stress level is low; that is, if the factor of safety is large and we do not have plastic yielding in the foundation. If foundation yielding is likely to occur, another approach is recommended, which will be described below.

In many foundations on cohesive soils, the immediate or distortion settlement is a relatively small part of the total vertical movement and, thus, rough estimates are acceptable. A discussion of relative importance of immediate and consolidation settlement will be given later in this section.

A Distributed Load at or Near the Surface of a Deep Layer When the foundation problem can be approximated as one or more uniformly distributed loads acting on circular or rectangular areas near the surface of a relatively deep stratum, the vertical settlement can be estimated by

$$s_i = C_s q B \left(\frac{1 - \nu^2}{E_u} \right) \tag{5.2}$$

where

s_i = settlement of a point on the surface
C_s = shape and rigidity factor
q = magnitude of the uniformly distributed load
B = characteristic dimension of the loaded area as shown in Figure 5.3
E_u = Young's modulus (undrained)
ν = Poisson's ratio

The coefficient C_s accounts for the shape and rigidity of the loaded area and for the position of the point for which the settlement is being calculated. Values of C_s are given in Table 5.3.

EXAMPLE 5.1

A structure is to be supported on a stiff reinforced concrete mat foundation whose dimensions are 20 m by 50 m. The load on the mat is to be uniformly distributed; its magnitude is 65 kPa. The mat rests on a deep saturated deposit of saturated clay for which the average undrained Young's modulus is approximately 40 MPa. Estimate the immediate settlement at the center and corner of the mat.

Solution

Since the mat is stiff, use the rigid factors from Table 5.3a. With $L/B = 50/20 = 2.5$, the shape factors for both the center and corner are determined by interpolation to be $C_s = 1.20$. Thus from Equation 5.2 the immediate surface settlement at both the center and corner of the mat is $s_i = 1.20(65)(20)\ [(1 - 0.5^2)]$ divided by $[(40 \times 10^3)] = 0.029$ m = 29 mm.

For comparison, the shape factors for a flexible mat would be determined by interpolation to be

At the center $C_s = 1.63$
At the corner $C_s = 0.81$

Thus, the immediate surface settlements are

At the center $s_i = 40$ mm
At the corner $s_i = 20$ mm

A mat foundation is usually neither completely flexible nor completely rigid, depending on its size and thickness and how heavily reinforced it is. If it is large, the distribution of contact pressure may be nearly uniform over its center portion. At the corners and edges, however, the rigidity of the mat may be significant (owing to its thickness and the amount of reinforcing), and settlements are likely to be less than predicted. In a saturated clay, because of settlement in the middle portion of the foundation, some heave may occur in the outer portions because of undrained (constant volume) loading and shear.

Effect of Layered Systems In actuality, most soil profiles are not homogeneous and deep. If the thickness of the top layer is large relative to the dimensions of the loaded area, immediate surface settlement may be calculated as if the soil were a homogeneous layer of infinite depth. However, if the upper stratum is relatively thin, the effect of layering must be taken into consideration. This is likely to be especially important when a soft compressible stratum is underlain by rock or very hard or dense soils. This special case may be approximated by a layer of elastic material of finite thickness underlain by a rigid base. Settlements for this case may be determined by Equation 5.2, but using a shape factor C_s that accounts for the presence of the rigid base. Values for these shape factors C_s are tabulated in Table 5.3b for the settlement under the center of a rigid circular area and under the corner of flexible rectangular areas. These shape factors depend upon both the shape of the loaded area and the thickness of the compressible stratum relative to the width of the loaded area, as illustrated in Figure 5.3.

Examination of Table 5.3 indicates the importance of the presence of a rigid boundary. When $H/B = 0.5$, the reduction in surface displacements of the center of the loaded area relative to that for the halfspace is greater than 50 percent.

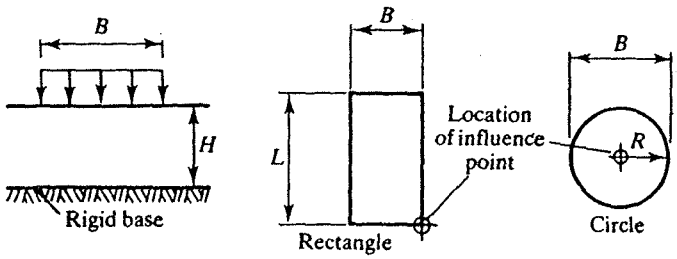


Fig. 5.3 Notation for loaded areas, shown in plan view. (*U.S. Navy, 1982.*)

EXAMPLE 5.2

Compute the immediate settlement at the center of the uniformly loaded (flexible) area measuring 6 m × 6 m. The applied surface stress is 200 kPa and the depth to firm bottom is 3 m. Assume the undrained elastic modulus is 10 000 kPa and $\nu = 0.5$.

Solution

Use the C_s values for the corners of four equally sided rectangles 3 m × 3 m. In this case, $H/B = 1$, $L/B = 1$, and,

TABLE 5.3 SHAPE AND RIGIDITY FACTORS, C_s, FOR CALCULATING SETTLEMENTS OF POINTS ON LOADED AREAS AT THE SURFACE OF AN ELASTIC HALFSPACE[a].

a. Infinite Depth

Shape and Rigidity	*Center*	*Corner*	*Edge/Middle of Long Side*	*Average*
Circle (flexible)	1.00		0.64	0.85
Circle (rigid)	0.79		0.79	0.79
Square (flexible)	1.12	0.56	0.76	0.95
Square (rigid)	0.82	0.82	0.82	0.82
Rectangle:				
(flexible)				
length/width				
2	1.53	0.76	1.12	1.30
5	2.10	1.05	1.68	1.82
10	2.56	1.28	2.10	2.24
Rectangle:				
(rigid)				
length/width				
2	1.12	1.12	1.12	1.12
5	1.6	1.6	1.6	1.6
10	2.0	2.0	2.0	2.0

b. Limited Depth Over a Rigid Base

		Corner of Flexible Rectangular Area				
H/B	*Center of Rigid Circular Area Diameter = B*	*L/B = 1*	*L/B = 2*	*L/B = 5*	*L/B = 10*	*L/B = ∞ (strip)*
			$\nu = 0.50$			
0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.14	0.05	0.04	0.04	0.04	0.04
1.0	0.35	0.15	0.12	0.10	0.10	0.10
1.5	0.48	0.23	0.22	0.18	0.18	0.18
2.0	0.54	0.29	0.29	0.27	0.26	0.26
3.0	0.62	0.36	0.40	0.39	0.38	0.37
5.0	0.69	0.44	0.52	0.55	0.54	0.52
10.0	0.74	0.48	0.64	0.76	0.77	0.73
			$\nu = 0.33$			
0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.20	0.09	0.08	0.08	0.08	0.08
1.0	0.40	0.19	0.18	0.16	0.16	0.16
1.5	0.51	0.27	0.28	0.25	0.25	0.25
2.0	0.57	0.32	0.34	0.34	0.34	0.34
3.0	0.64	0.38	0.44	0.46	0.45	0.45
5.0	0.70	0.46	0.56	0.60	0.61	0.61
10.0	0.74	0.49	0.66	0.80	0.82	0.81

[a] After U.S. Navy (1982).

from Table 5.3, $C_s = 0.15$:

$$S_i = 0.15(200)(3)\left(\frac{1 - 0.5^2}{10\,000}\right) \times 4 = 27\,\text{mm}$$

If the soil profile consists of a relatively thin stiffer layer underlain by a less stiff layer of greater depth, then the stresses from the surface load must be distributed to the top of the less compressible layer. Use the stress distribution techniques discussed in Section 5.6.

Analytical and/or numerical methods for the determination of displacements in multilayered systems are available for cases other than those in Table 5.3 (see Poulos and Davis, 1974). A number of multilayer solutions are now available in computer codes. Except for pavements and special foundations, however, the use of multilayered computer analyses is generally not justified, because the material parameters are not accurately determined, the boundary conditions and interface conditions between the strata are not that well known, and, finally, approximations may be required to fit the geometry of the real problem to that for which the solution is available. In many situations an approximate analysis of the intermediate settlement is sufficient, as illustrated in the following two examples.

EXAMPLE 5.3

The mat foundation of Example 5.1, 20 m × 50 m supporting a uniform normal load of 65 kPa, is founded on a soil profile shown in Figure 5.4. The profile indicates a layer of stiff clay over a more compressible layer that is in turn underlain by shale. Assume these conditions are representative of the entire site. Estimate the immediate surface settlement at the center of the mat.

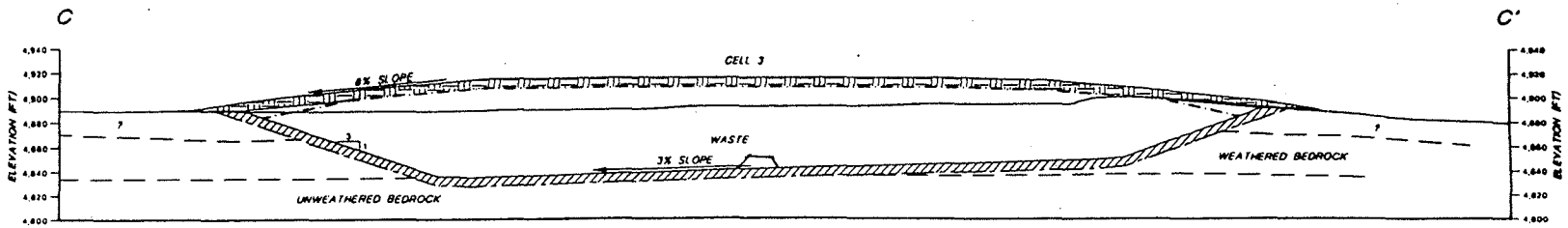
Solution

Assume the shale acts as a rigid base and above it a *single* stratum of thickness $H = 15$ m. Thus,

$$\frac{H}{B} = \frac{15}{10} = 1.5 \qquad \frac{L}{B} = \frac{25}{10} = 2.5$$

The shape factor C_s obtained from Table 5.3b by linear

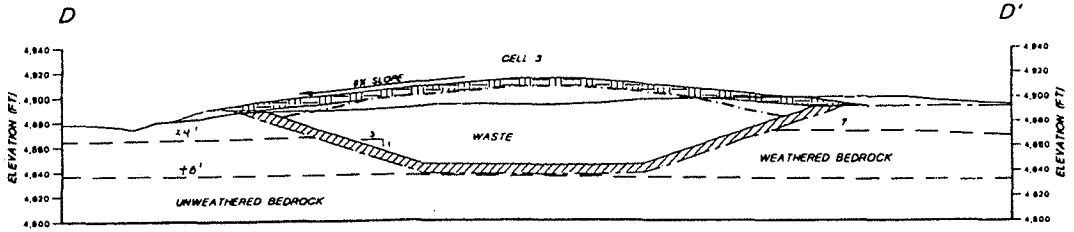
5/16
603-2000
JTS
10/7/94.



TYPICAL CROSS-SECTION (C / 11 / 17)

HORIZONTAL SCALE
0 50' 100'

VERTICAL SCALE
0 50' 100'



TYPICAL CROSS-SECTION (D / 11 / 17)

HORIZONTAL SCALE
0 50' 100'

VERTICAL SCALE
0 50' 100'

LEGEND

- EXISTING GRADE
- TOP OF LINER
- SUBGRADE
- LINER LAYER
- COVER LAYER

DETAIL/SECTION NUMBER (A / 6 / 6)

DRAWING No. WHERE DETAIL/SECTION IS REFERENCED

DRAWING No. WHERE DETAIL/SECTION IS SHOWN

No.	REVISION	DATE	BY	CHKD
0				

SCALE: SEE BAR SCALE
DESIGN BY: TEK
DRAWN BY: JGH
CHECK BY: PCC
PROJECT No: 803
FILE NAME: HWY-11.DWG

HIGHWAY 36 LAND DEVELOPMENT COMPANY

HIGHWAY-36

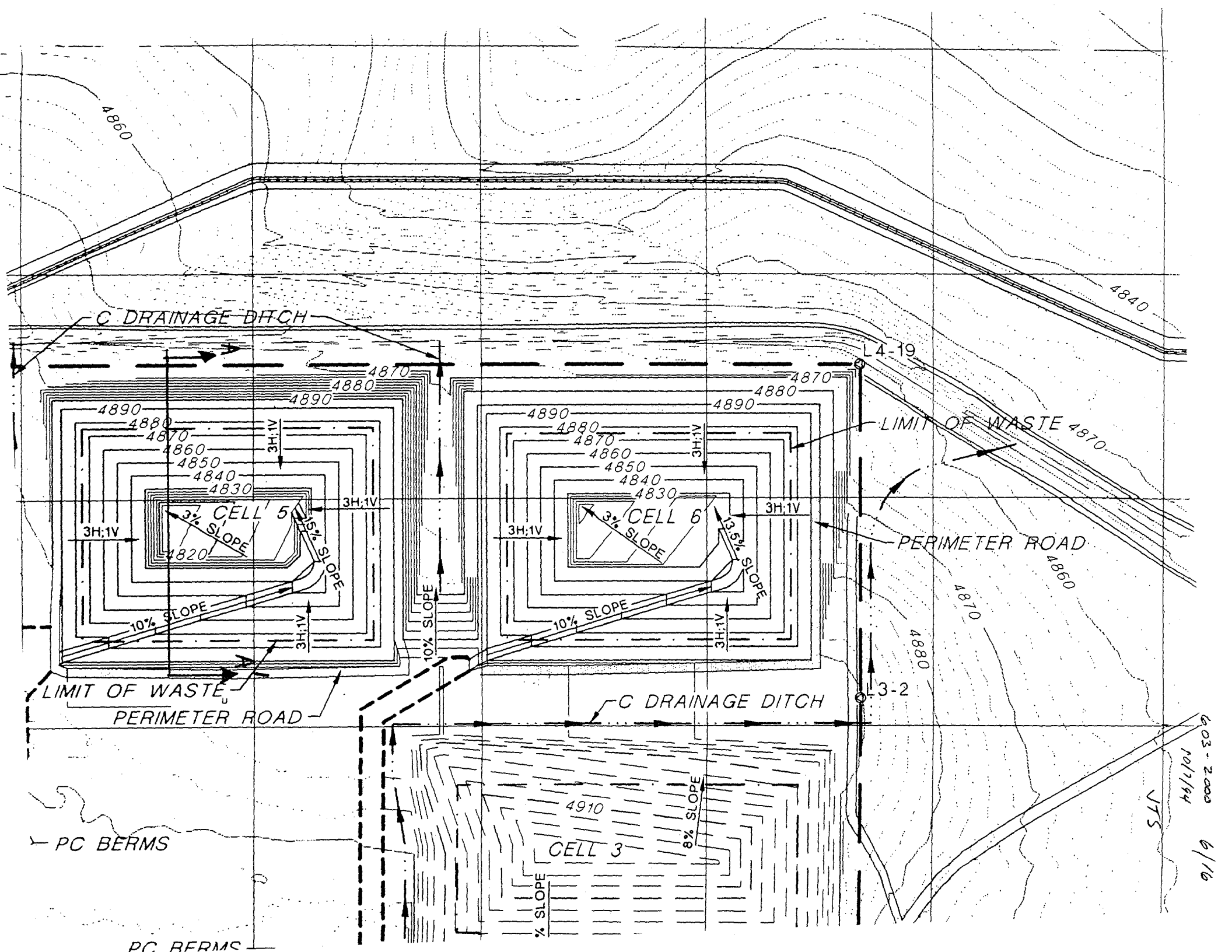
PROJECT: SITE DEVELOPMENT PLAN

DRAWING TITLE: TYPICAL CELL CROSS-SECTION

TerraMatrix

SHEET 1 OF 1
DRAWING No. | REV

5/16


C DRAINAGE DITCH
L 4-19
LIMIT OF WASTE
PERIMETER ROAD
CELL 5
CELL 6
CELL 3
3% SLOPE
13.5% SLOPE
10% SLOPE
3H:1V
8% SLOPE
PC BERMS
3-2
4860
4870
4880
4890
4840
4830
4820
4910

7/16

Naval Facilities Engineering Command
[illegible] Stovall Street
[illegible]andria, Virginia 22332-2300

APPROVED FOR PUBLIC RELEASE

603-2000

9/15/94

Soil Mechanics

DESIGN MANUAL 7.01
[illegible]ALIDATED BY CHANGE 1 SEPTEMBER 1986

8/16

603-2000
9/15/94

pace

TABLE 2
Relationship Between Undrained Modulus and Overconsolidation Ratio

OCR*	Eu/c		
	PI<30	30<PI<50	PI>50
<3	600	300	125
3 - 5	400	200	75
>5	150	75	50

* OCR = Overconsolidation ratio

c = Undrained shear strength

PI = Plastic index

Other books published within the Series on Rock and Soil Mechanics

& v. Eisenhart Rothe, M.:
nkers
; Bulk Materials

of Rock Mechanics
gth Properties of Rocks

.:
Instrumentation

:.:

nerican Engineers

. & A.:
lls Volume I
nd Sheet Piling

S. & Lama, R.D.:
1 Mechanical Properties

. & A.:
lls Volume II
ive Resistance
1 Structures

Jr. & Leighton, F.W.:
nce on Acoustic
roseismic Activity
tructures and Materials

Karafiath, L.L. & Nowatzki, E.A.:
Soil Mechanics
for Off-Road Vehicle Engineering
1977

Baguelin, F., Jézéquel, J.F. & Shields, D.H.:
The Pressuremeter
and Foundation Engineering
1978

Lama, R.D. & Vutukuri, V.S.:
Handbook on Mechanical Properties
of Rocks
Volumes II, III & IV
1978

Gregory, C.E.:
Explosives
for North American Engineers
Second Edition
1979

Assonyi, Cs. & Richter, R.:
The Continuum Theory
of Rock Mechanics
1979

Hardy, H.R., Jr. & Leighton, F.W.:
Second Conference on Acoustic
Emission/Microseismic Activity
in Geologic Structures and Materials
1979

Jumikis, A.R.:
Rock Mechanics
1979

Hanna, T. H.:
Foundations in Tension
Ground Anchors
1982

Series on Rock and Soil Mechanics
Vol. 7 (1983)

ROCK MECHANICS

Second Edition

by

Alfreds R. Jumikis
Professor Emeritus of Rutgers University
The State University of New Jersey
USA

Second Edition
1983

603-2000
SPO
9/15/94

TRANS TECH PUBLICATIONS

9/16

24. Modulus of Elasticity of Rocks

Knowledge about the modulus of elasticity of rocks is an important factor evaluating rock deformation under various loading conditions. It is also cessary in studies of seismics in rocks. The modulus of elasticity of rock ries from one geologic region to another because of the existence of rious types of rock formation. For this reason alone, different values of e modules of elasticity of different rocks can be anticipated. Hence, there ists a need for the determination of the numerical values of the modulus of asticity of rocks.

Generally, the modulus of elasticity of rock is affected by rock type, po-sity, particle size, and water content. The variation in the values of the odulus of elasticity is more pronounced in clayey shales than in sandstone, r example.

The modulus of elasticity is greater perpendicular to stratification and sure than that determined parallel to the bedding planes and fissures.

The various modes of genesis or formation of rocks also affect the mag-tude of the modulus of elasticity.

The reason for high modulus of elasticity values of some rocks probably s in their structures themselves, as well as in their mineral chemical com-unds. The known values of the modulus of elasticity of various rocks may ggest that eruptive igneous rocks of basic nature—like basalts, for example have a larger modulus value than acidic rocks—such as granite, for ample.

Technically, the modulus of elasticity of a rock can be increased con-lerably by means of rock grouting.

asticity constants E and μ of rocks are determined by:

- static tests
- longitudinal resonance tests
- electrical resistance (strain gauge) tests
- ultrasonic velocity measurements on laboratory rock test specimens, and
- dynamic tests, in which velocities of propagated energy waves in situ are measured. Measurements of the primary wave (P-wave) and secondary wave (S-wave) velocities thus afford the indirect determination of E and μ of geologic materials in situ.

TABLE 4-13
Elasticity Constants E and μ of Some Rocks

Rock	Young's modulus of elasticity E		Poisson's ratio μ	References
	kgf/cm²	N/m²		
	Multiply by 10^5	Multiply by 10^{10}	Average values	
1	2	3	4	5
Igneous rocks				
Basalt	2.0–10.0	1.96– 9.81	0.14 –0.25	37, 63, 68, 107
		4.85–11.15	0.22 –0.25	18
Diabase	3.0– 9.0	2.94– 8.83	0.125–0.25	42, 63
		–	0.333	68
		2.20–11.40	0.103–0.184	68
		8.00–10.75	–	18
Gabbro	6.0–11.0	5.88–10.78	0.125–0.25	37, 63, 68
		5.84– 8.71	0.154–0.48	18
Granite	2.6– 7.0	2.55– 6.86	0.125–0.25	37, 63
		–	0.155–0.338	18
		–	0.150–0.240	112, 68
		2.13– 7.05	–	18
Syenite	6.0– 8.0	5.88– 7.85	0.25	37
		6.29– 8.63	0.17 –0.319	18
			0.15 –0.34	68
Sedimentary rocks				
Dolomite	2.0– 8.4	1.96– 8.24	0.08 –0.20	37, 63
		7.10– 9.30	0.08 –0.20	68, 107
		–	0.32 –0.37	18
Limestone	1.0– 8.0	0.98– 7.85	0.10 –0.20	63, 107
		–	0.16 –0.23	68
		–	0.33 –	63
		0.80– 2.10	0.14 –0.30	18
Sandstone	0.5– 8.6	0.49– 8.43	0.066–0.125	112
		–	0.230–0.300	56
		–	0.17	63, 68
	163,848,000 psf.	163,970,500 psf	0.07	112
		–	0.62	56
Shale (clay)	0.8– 3.0	0.78– 2.94	0.11 –0.54	37, 68
		0.98– 2.35	0.10	56
		1.20– 4.40	0.23 –0.30	107
		–	0.04 –0.12	18
Sandstone	4.5– 5.2	4.41– 5.10	0.21 –0.24	63

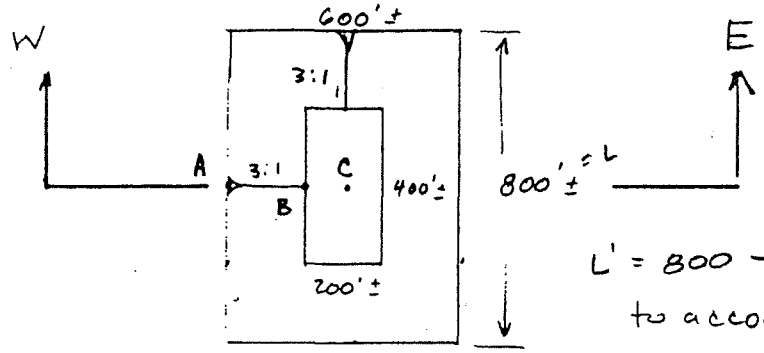
TABLE 12-3
Properties of Various Rocks as Foundation Materials

Type	Rock	Typical dry unit weight, p.c.f.	Range of modulus of elasticity, k.s.i.	Range of compressive strength, k.s.i.	Structural characteristics
Igneous:					
Intrusive (coarse grained).	Feldspar predominates, light colored:				
	Granite (abundant quartz)	168	4,000 to 7,000	10 to 25	Generally present as intrusion of great mass. May contain fracture system, closed except where weathering has proceeded downward from the surface. Deep localized weathering may occur at intersection of major fracture systems.
	Diorite (little quartz).....	176	5,000 to 8,000		
	Iron, magnesium predominate, dark colored Gabbro.	180	7,000 to 12,000	15 to 30	
Extrusive (fine grained).	Feldspar predominates, light colored:				
	Rhyolite (abundant quartz)	162	5,000 to 8,000	10 to 25	Formed in sheet-like masses characterized by extensive joint system, more open in basalt than rhyolite or andesite. May contain holes, voids, layers of volcanic ash or pumice resulting from volcanism concurrent with extrusion.
	Andesite (little quartz)	166	6,000 to 9,000		
	Iron, magnesium predominate, dark colored Basalt.	178	7,000 to 13,000	25 to 40	
	Glassy Obsidian	140	1,000 to 4,000	2 to 8	Exhibits flow structures, may be highly vesicular.
	Fragmental Tuff	100	200 to 1,000	0.2 to 1	Light and relatively porous structure formed in volcanic discharge.
Metamorphic:					
Foliated (platy) ...	Medium grained, abundant mica Schist.	167	2,000 to 5,000	5 to 15	Often intricately folded and distorted. Fracturing, softening, weathering or deep erosion occurs in zones of intense movement. Weathering produces clayey, micaceous residue.
	Fine grained, dark colored Slate	168	5,000 to 8,000	10 to 20	
Banded (imperfectly foliated).	Coarse grained, abundant quartz Gneiss.	169	4,000 to 8,000	10 to 20	Less distortion than in the highly foliated rocks. Weathered residue is gritty with resistant silica particles.
Massive	Hard and brittle:				
	Quartzite (mainly quartz)	166	6,000 to 8,000	15 to 35	Quartzite and marble may be extremely hard with only fine fracture system. Some serpentines are soft to great depths.
	Marble (mainly calcite)	168	7,000 to 10,000	12 to 30	
	Relatively soft Serpentine	158	1,000 to 5,000	1 to 10	
Sedimentary:			72,000 ksf		
Argillaceous (clay minerals predominate).	Fine grained, laminated:				
	Shale (clay size)	100 to 140	500 to 2,000	0.1 to 5	Wide variation in engineering properties between materials formed by compaction alone or with cementation. Compaction shales may soften, slake and swell on exposure. Cementation types (argillite) are not sensitive to exposure.
	Siltstone (silt size)	110 to 150	500 to 2,000	0.1 to 5	
Siliceous (silica predominates).	Medium grained Sandstone	147	1,000 to 3,000	4 to 12	Strength and permeability depends on type and degree of cementation. Fracturing, folding and jointing leads to deterioration of cementing materials.
	Coarse grained:				
	Conglomerate (rounded particles) ...	155	1,000 to 5,000	5 to 15	
	Breccia (angular fragments)	158	1,000 to 5,000		

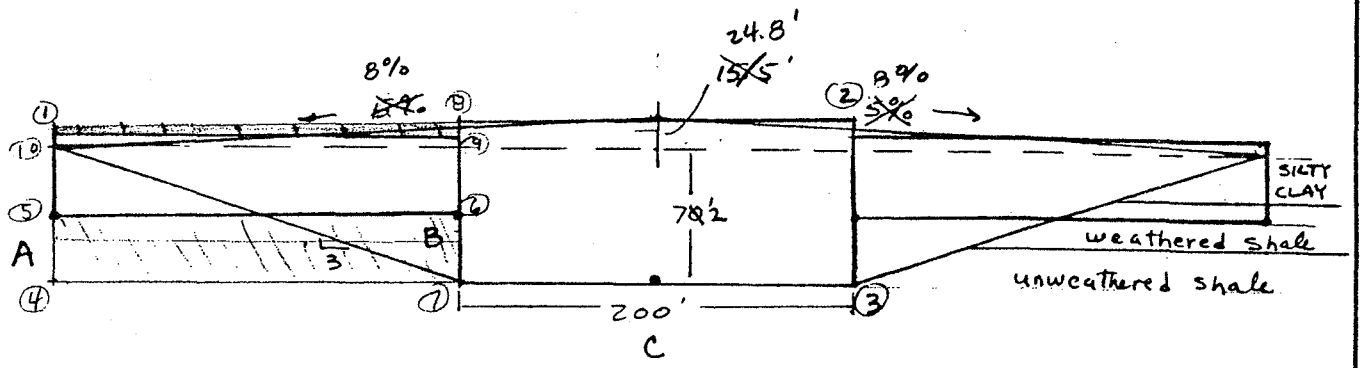
TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name: HWY 36
Project Number: 603-2000 Sheet: 12 Of 16
Prepared By: VTS Date: 7/28/94
Checked By: J. Pellicer Date: 9/15/94

PROPOSED CELL #~~X~~5



$L' = 800 - (2 \times 70') = 660$
to account for 3:1 end slopes.



Calculate settlement @ Point A.
- use superposition

assume E_u, ν for unweathered shale. - reason - model for a layered profile - soft soil over a rigid base would give lower values due to thiness of layer.

calculate S_i due to 1, 2, 3, 4.

$\frac{L}{B} = \frac{660}{410} = 1.610$ By interpolation Table 5.3a Fang $C_s = 0.98$ ✓ → flexible rectangular footing - Edge.

$P = 95' \times 110 = 10{,}450$ psf ✓ $B = 410$ ft

$S_i = (C_s)(q)(B)\left(\frac{1-\nu^2}{E_u}\right) = (0.98)(9350)(410)\left(\frac{1-.3^2}{7.2\times10^6 \text{ psf}}\right)$

$= 0.053$ feet ✓

CALCULATE (–) settlement due to: ⑤ ⑥ ⑦ ④

$B = 210'$ ✓ $\frac{L}{B} = \frac{660}{210} = 3.143$ ✓ C_s → by interpolation Table 5.3a (FANG)

$C_s = 1.33$ ✓

$P = 35' \times 110 = 3{,}850$ psf

$S_i = (1.33)(3850)(210)\left(\frac{1-.3^2}{27{,}000{,}000}\right) = 0.014''$ ✓

0.2"

TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name: Hwy 36
Project Number: 603-2000 Sheet: 13 Of 16
Prepared By: VTS Date: 7/28/94
Checked By: J. Pellicer Date: 9/15/94

calculate negative settlement due to ① ⑧ ⑨ ⑩

$\beta = 210$ $\frac{L}{B} = \frac{660}{210} = 3.143$ Cs (as before) = 1.33

$\rho = 0.5' \times 110 = 55$

$S_i = (1.33)(55)(210)\left(\frac{(1-.3^2)}{72,000,000}\right) = 0.0002'$ ✓

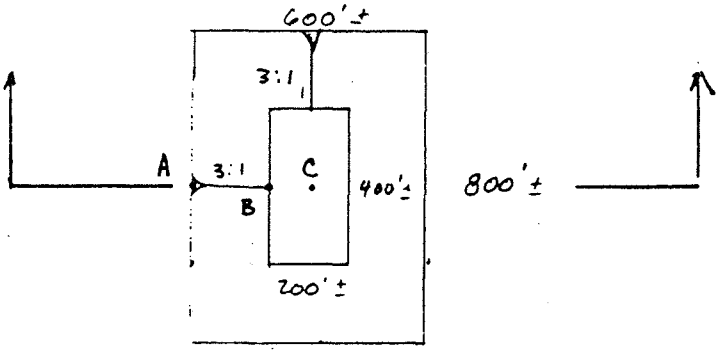
$\Sigma \quad S_{i_4} = 0.053 - .014 - .0002 = .039 \quad = 0.47''$

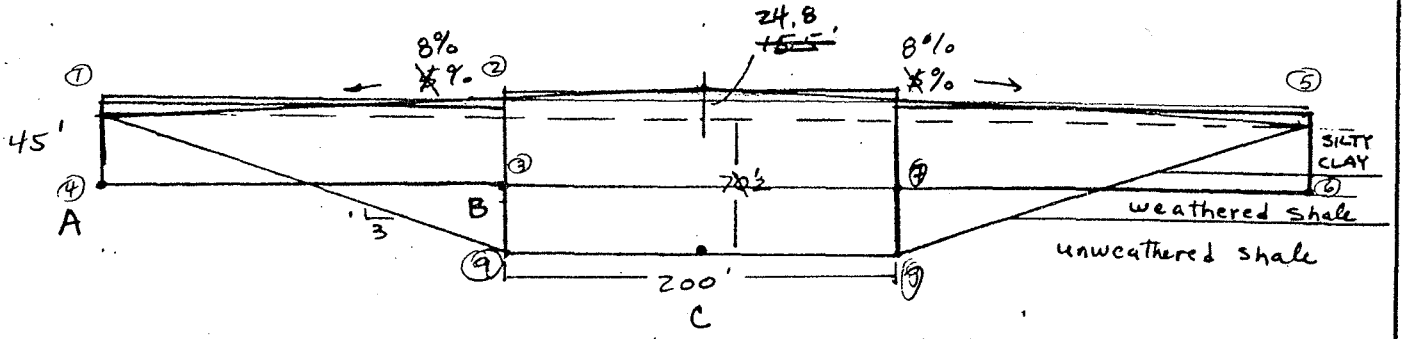
Note: The block of waste fill along the west side of the section was neglected because settlement caused by it would be negligible. This is because the small blocks of waste along the east side caused displacements at point A of 0.17 - 0.002 inches.

TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name: HWY 36
Project Number: 603-2000
Prepared By: VTS
Checked By: J. Pellicer
Sheet: 14 of 16
Date: 7/28/94
Date: 9/15/94

PROPOSED CELL #5





Calculate settlement @ B. - superposition

a) S_i ①②③④ $\frac{L}{W} = \frac{660}{210} = 3.143$ $C_s = 1.333$ (by interpolation FANG 5.3a) Edge/middle rectangle.

$P = 55 \times 110 = 6050$ $B = 210$

$S_i = (1.33)(6050)(210)\left(\frac{1-0.3^2}{72,000,000}\right) = 0.021'$

b) S_i → ②⑤⑥③ $\frac{L}{W} = \frac{660}{410} = 1.61$ $C_s = 0.98$ (as before)

$P = 55 \times 110 = 6050$ $B = 410$

$S_i = (0.98)(4950)(410)\frac{(1-0.3^2)}{72,000,000} = 0.031'$

c) S_i → ③⑦⑧⑨ $\frac{L}{W} = \frac{660}{200} = 3.30$ $C_s = 1.344$ (by interpolation Fang 5.3A) Edge/middle rectangle

$P = 35\text{ft} \times 110\text{pcf} = 3850$ psf

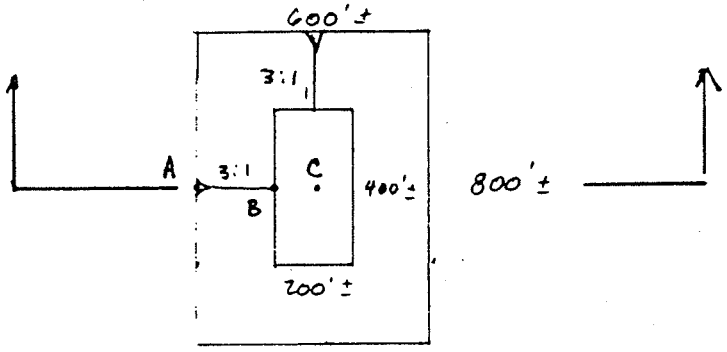
$S_i = (1.34)(3850)(200)\left(\frac{1-0.3^2}{72,000,000}\right) = 0.013$ ft

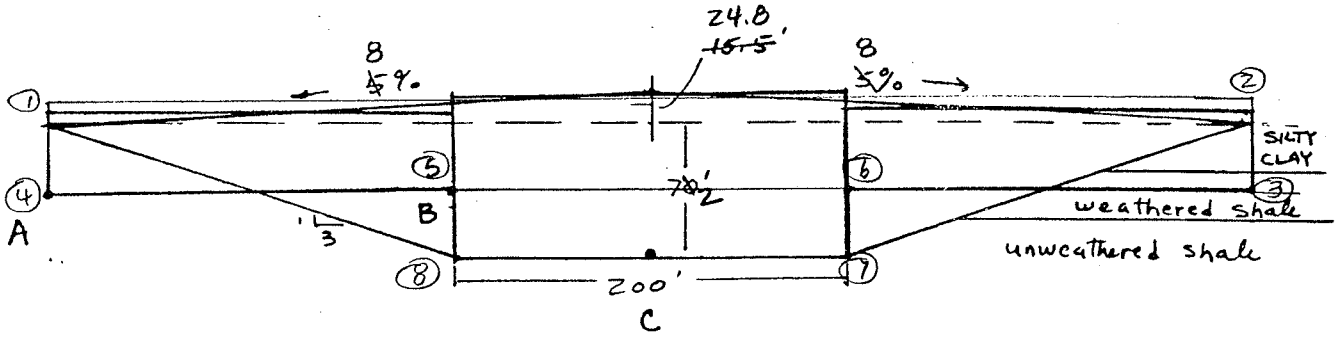
$\sum S_i = 10.021 + .031 + .013 = 0.065$ feet = 0.780 inches

TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name: HWY 36
Project Number: 603-2000 Sheet: 15 of 16
Prepared By: VTS Date: 7/28/94
Checked By: J. Pilcor Date: 9/15/94

PROPOSED CELL #5





Calculate settlement at Point C.

Calculate settlement from ① ② ③ ④

$\frac{L}{W} = \frac{660}{620} = 1.065$ C_s, interpolated from Table 5.3a, p172 - FANG (center - Flexible Rectangle)

$C_s = 1.15$

$P = (55)(110) = 6050$ psf $B = 620'$

$S_i = (1.15)(6050)(620)\left(\frac{1-.3^2}{72,000,000}\right) = 0.055'$

Calculate settlement @ Point C from ⑤ ⑥ ⑦ ⑧

$\frac{L}{W} = \frac{660}{200} = 3.30$ $C_s = 1.78$ by interpolation Table 5.3a center - flexved.

$P = 35' \times 110 = 3850$ $B = 200'$

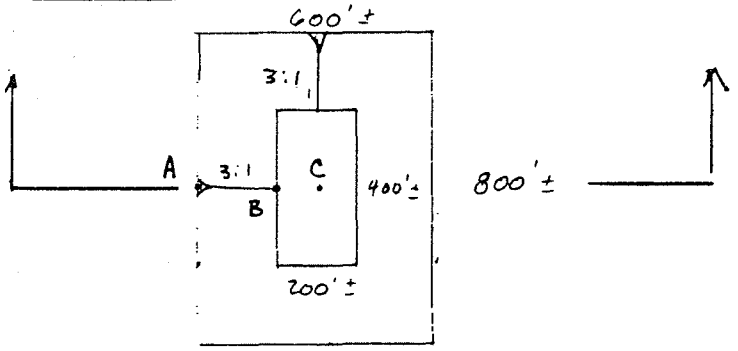
$S_i = (1.78)(3850)(200)\left(\frac{1-.3^2}{72,000,000}\right) = 0.017'$

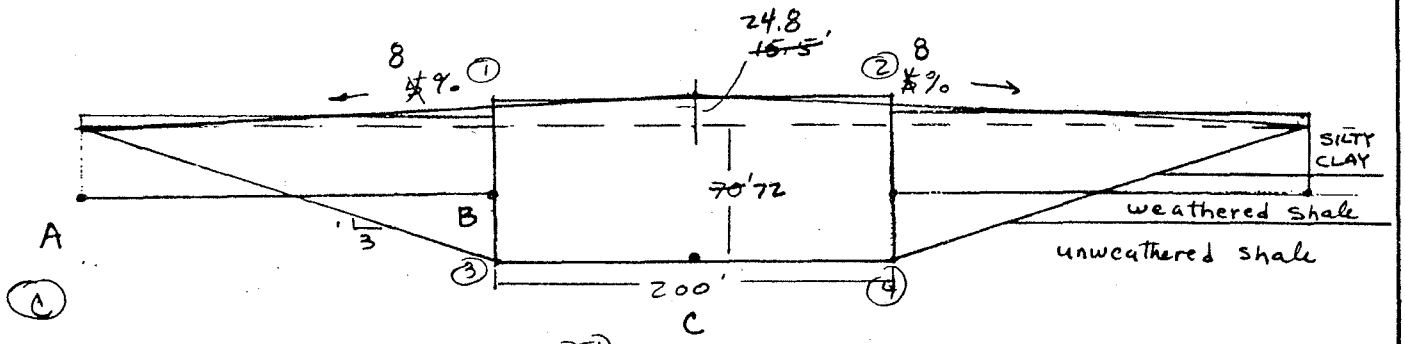
$\Sigma_t s_i = 0.072' = 0.86$ inches

TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name HWY 36 15a
Project Number 603-2000 Sheet 8 of 16
Prepared By VTS Date 7/28/94
Checked By J. Pellicer Date 9/15/94

PROPOSED CELL #5





CALCULATE SETTLEMENT FROM ①②③④

$\frac{L}{W} = \frac{660}{200} = 3.30$ ✓ $C_s = 1.78$ – by interpolation FANG TABLE 5.3a – center-flexible rectangle.

$P = 95 \times 110 = 10{,}450$ psf ✓ $B = 200$

$S_i = (1.78)(10{,}450)(200)\left(\frac{1-.3^2}{72{,}000{,}000}\right) = 0.047'$ ✓ } to check pier. calc.

TerraMatrix
Engineering & Environmental Services
1475 Pine Grove Road, P.O. Box 774018
Steamboat Springs, Colorado 80477

Project Name: Hwy 36
Project Number: 603-2000 Sheet: 16 Of 16
Prepared By: VTS Date: 7/28
Checked By: J. Rillion Date: 9/15/94

DIFFERENTIAL SETTLEMENT

A - B = 0.47" - 0.78" = 0.31"

A - C = .0.47 - .86" = 0.39" Say 1/2" max diff.

Maximum calculated settlement is @ Point A = 0.86 inches ⇒ Say 1"

Conclusion: Total & differential settlements estimated are considered to be negligible and should not impact performance of liner system.

Cited References

Faizur Khan
ph.# 713/478-2116

AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS

DESIGN OF WASTE CONTAINMENT LINER AND FINAL CLOSURE SYSTEMS

...Serving ASCE Members...and Other Technical and Management Professionals

Instructor: Gregory Richardson

5.3.2 Geomembrane (GM) Component

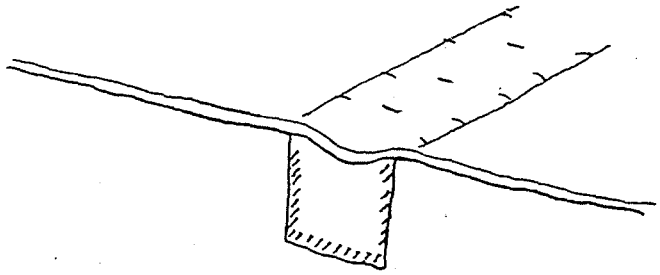
Cover GM vs. Liner GM

- No Significant Chemical Exposure
- Long-term Stability Considerations
- Biaxial Stress-strain Considerations
- Construction Sequencing

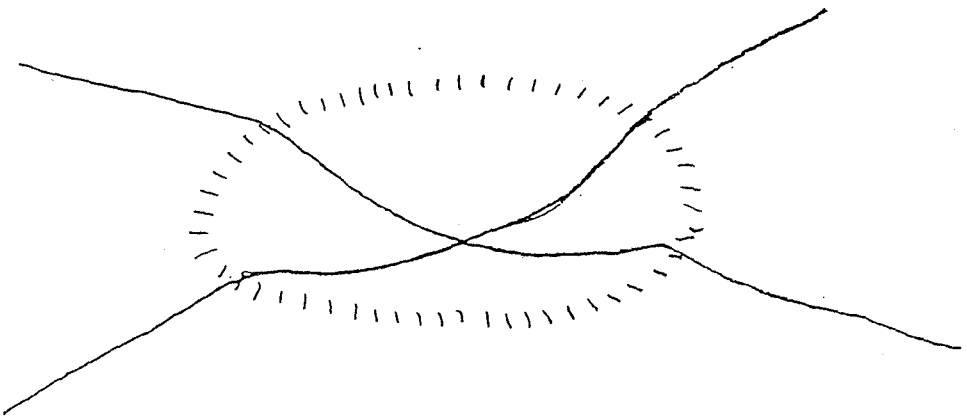
Geomembrane Stresses Due to Localized Subsidence

- Classic "NY Refrigerator"
- Uniaxial vs Biaxial

Uniaxial Subsidence (Trench)



Biaxial Subsidence

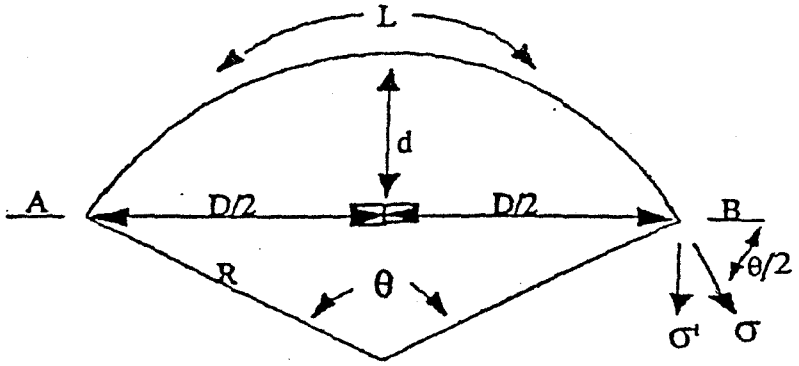


"Ponding"

GM-5

CALCULATIONS (ASTM Draft Standard, GRI Draft Standard)

STRAIN CALCULATIONS



For $d < D / 2$:

$$R = \frac{D^2 + 4d^2}{8d}$$

$$\theta = 2\tan^{-1}\frac{4(D)d}{D^2 - 4d^2} \text{ (radians)}$$

$$L = R * \theta$$

$$\varepsilon = \frac{L - D}{D}(100) \text{ in \%}$$

For $d > D / 2$:

$$L = \pi\sqrt{\frac{D^2 + 4d^2}{8}}$$

$$\varepsilon = \frac{L - D}{D}(100)$$

R	=	sphere radius
L	=	length of segment A - B
D	=	specimen diameter
d	=	vertical deflection
P	=	internal Pressure
t	=	specimen thickness
ε	=	strain (%)
σ	=	stress kPa

STRESS CALCULATIONS

For $d < D / 2$:

$$\sigma = \frac{(D^2 + 4d^2)P}{4dt\sin(\theta / 2)}$$

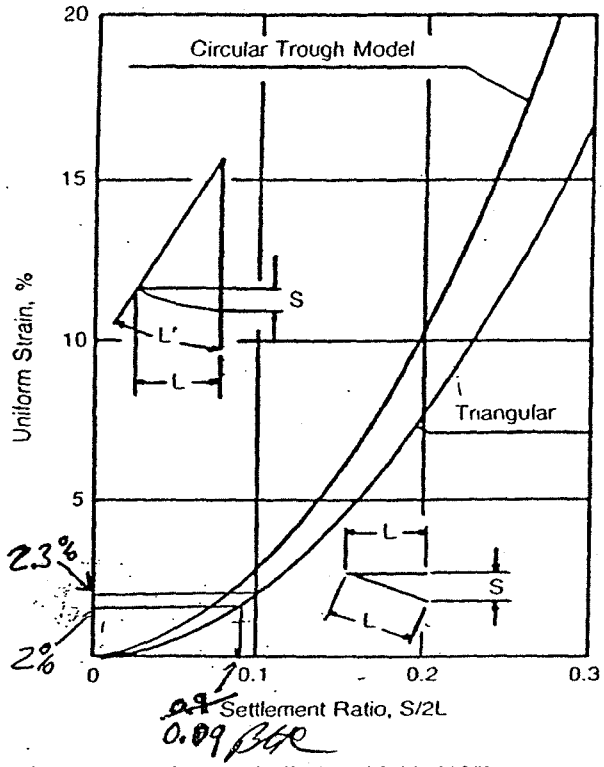
For $d > D / 2$: ($\sigma = \sigma'$)

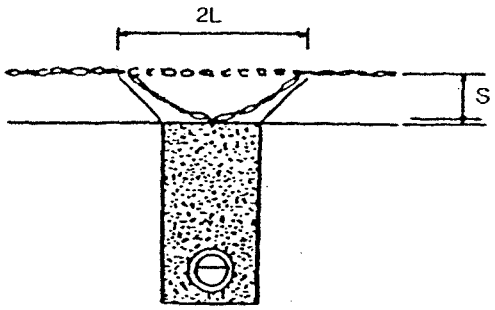
$$\sigma = \frac{dP}{t}$$

BIAXIAL SUBSIDENCE

Exhibit H

10/8/01





re 3-7. Settlement trough models (Knipschield, 1985).

Anchorage

Anchor trenches can cause FMLs to fail in one of two ways: by ripping or by pulling out. The pullout mode is easier to correct. It is possible to calculate pullout capacity for FMLs placed in various anchorage configurations (Figures 3-8 and 3-9). In the "V" anchor configuration, resistance can be increased by increasing the "V" angle. A drawback to using the "V" design for getting an accurate estimate of pullout capacity is that it uses more space. The concrete trench is not presently used.

Ramps

Most facilities have access ramps (Figure 3-10), which are used by trucks during construction and by trucks bringing waste into the facility. Figure 3-11 depicts a cross section of a typical access ramp. The double FML integrity must be maintained over the entire surface of the ramp. Because ramps can fail due to traffic-induced sliding, roadway considerations, and drainage, these three factors must be considered during the design and construction of access ramps.

; weight of the roadway, the weight of a vehicle on je roadway, and the vehicle braking force all must ɔe considered when evaluating the potential for slippage due to traffic (Figure 3-12). The vehicle braking force should be much larger than the dead weight of the vehicles that will use it. Wheelloads also have an impact on the double FML system and the two leachate collection systems below the roadway. Trucks with maximum axle loads (some much higher than the legal highway loads) and 90 psi tires should be able to use the ramps. Figure 3-13 illustrates how to verify that wheel contact loading will not damage the FML. Swells or small drains may be constructed along the inboard side of a roadway to ensure that the ramp will adequately drain water from the roadway. Figure 3-14 illustrates how to verify that a ramp will drain water adequately. The liner system, which must be protected from tires, should be armored in the area of the drainage swells. A sand subgrade contained by a geotextile beneath the roadway can prevent local sloughing and local slope failures along the side of the roadway where the drains are located. The sand subgrade tied together with geotextile layers forms, basically, 800-foot long sandbags stacked on top of one another.

Vertical Standpipes

Landfills have two leachate collection and removal systems (LCRSs): a primary LCRS and a secondary LCRS. Any leachate that penetrates the primary

Cell Component: FLEXIBLE MEMBRANE CAP

Consideration: SETTLEMENT : VERIFY ABILITY OF FMC TO SURVIVE SETTLEMENT RESULTING FROM LONG-TERM WASTE COMPRESSION. SETTLEMENT FEATURES MAY BE BIAXIAL OR UNIAXIAL.

Required Material Properties	Range	Test	Standard
FLEXIBLE MEMBRANE CAP - YIELD STRAIN FOR FML, ϵ_{YIELD}	10 - 25%	WIDE WIDTH	ASTM D4595
Draft MTC Minimum			

Analysis Procedure:

(1) ESTIMATE GEOMETRY OF SETTLEMENT DEPRESSION

- DEPTH BASED ON % SETTLEMENT x WASTE DEPTH (SUGGEST MINIMUM 5%)
- WIDTH BASED ON MINIMUM CELL DIMENSION OR PAST EXPERIENCE

(2) CALCULATE SETTLEMENT RATIO, SR

SR = DEPTH / WIDTH

(3) OBTAIN UNIFORM STRAIN FROM FIG 3A

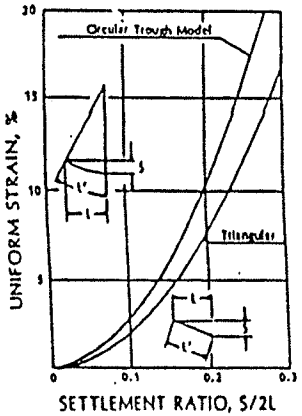
SUGGEST CIRCULAR TROUGH MODEL

(4) OBTAIN STRESS/STRAIN FOR FMC

⇒ ϵ_{RUPT} (STRAIN AT RUPTURE)

(5) CALCULATE DESIGN RATIO

DR = $\epsilon_{RUPT} / \epsilon_{UNIF}$



Design Ratio:

DR_{MIN} > 5.0 RUPTURE

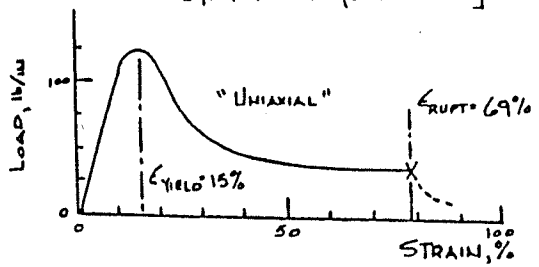
References:

KNIPSCHIELD (1985)

Example:

GIVEN:

- MINIMUM WIDTH OF CELL = 50 FT
- DEPTH OF WASTE = 50 FT
- COMPOSITE FMC [GEOTEXTILE + GEOMEMBRANE]



(1) ESTIMATE GEOMETRY OF SETTLEMENT FEATURE

- SETTLEMENT DEPTH = 5% x 50' = 2.5 FT

(2) CALCULATE SETTLEMENT RATIO, SR

- SR = 2.5/50 = .05

(3) OBTAIN UNIFORM STRAIN, ϵ_{UNIF}

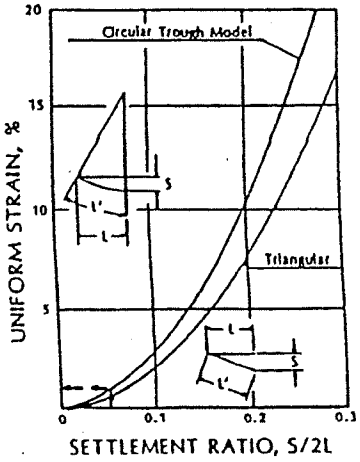
SEE FIGURE ⇒ ϵ_{UNIF} = 1%

(4) STRESS/STRAIN FOR FMC

⇒ ϵ_{RUPT} = 69%

(5) CALCULATE DESIGN RATIO

DR = 69/1 = 69 OK



Example No. 5.4

(UNIAXIAL SIDENCE)

POLY-FLEX
POLYETHYLENE GEOMEMBRANES

TEXTURED HDPE GEOMEMBRANE
METRIC UNITS

Minimum Average Values

Property	Test Method	1.00 mm	1.50 mm	2.00 mm	2.50 mm
Thickness, microns	ASTM D 5994				
minimum average		950	1,425	1,900	2,375
lowest individual of 8 of 10 readings		900	1,350	1,800	2,250
lowest individual of 10 readings		850	1,275	1,700	2,125
Asperity Height, microns	GRI GM12	250	250	250	250
Sheet Density, g/cc	ASTM D 1505/D 792	0.940	0.940	0.940	0.940
Tensile Properties[1]	ASTM D 6693 (Type IV Specimen @ 5 cm/min)				
1. Yield Strength, kN/m		15	22	29	37
2. Break Strength, kN/m		11	16	21	26
3. Yield Elongation, %		12	12	12	12
4. Break Elongation, %		100	100	100	100
Tear Resistance, N	ASTM D 1004	125	187	249	311
Puncture Resistance, N	ASTM D 4833	267	400	534	667
Stress Crack Resistance[2], hrs	ASTM D 5397 (App.)	300	300	300	300
Carbon Black Content[3], %	ASTM D 1603	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596	--Note 4--			
Oxidative Induction Time (OIT) Standard OIT, minutes	ASTM D 3895	100	100	100	100
Oven Aging at 85°C Standard OIT - % retained after 90 days	ASTM D 5721 ASTM D 3895	55	55	55	55
UV Resistance[5] High Pressure OIT[6] - % retained after 1600 hrs	GRI GM11 ASTM D 5885	50	50	50	50
Seam Properties	ASTM D 6392 (@ 5 cm/min)				
1. Shear Strength, kN/m		14	24	28	35
2. Peel Strength, kN/m - Hot Wedge		10.5	15.9	21.2	26.4
- Extrusion Fillet		9.1	13.6	18.2	22.8
Roll Dimensions					
1. Width (meters):		7	7	7	7
2. Length (meters):		228.7	152.4	114.3	91.5
3. Area (square meters):		1,603	1,068	801	641
4. Gross weight (kilograms, approx):		1,588	1,588	1,574	1,574

1 Machine direction (MD) and cross machine direction (XMD) average values should be on the basis of 5 test specimens each direction. Yield elongation is calculated using a gauge length of 33 mm; Break elongation is calculated using a gauge length of 50 mm.
2 The yield stress used to calculate the applied load for the SP-NCTL test should be the mean value via MQC testing.
3 Other methods such as ASTM D 4218 or microwave methods are acceptable if an appropriate correlation can be established.
4 Carbon black dispersion for 10 different views: Nine in Categories 1 and 2 with one allowed in Category 3.
5 The condition of the test should be 20 hr. UV cycle at 75°C followed by 4 hr. condensation at 60°C.
6 UV resistance is based on percent retained value regardless of the original HP-OIT value.

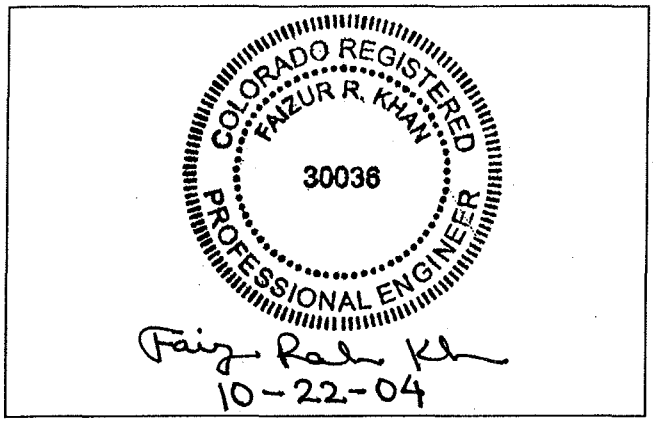
This data is provided for informational purposes only and is not intended as a warranty or guarantee. Poly-Flex, Inc. assumes no responsibility in connection with the use of this data. These values are subject to change without notice. REV. 12/03

Appendix 3

Stormwater Storage Calculation

Calculation Summary Sheet

Project Name: Clean Harbors Deer Trail Cell 2 Temporary Authorization
Date: October 20, 2004
Calculation Name: Deer Trail Cell 2, Stormwater Storage Calculation
Original? [Y] N Revised? Y N



Seal of Certifying P.E.

See Attached Figures
1, 3, and 5

Graphical Representation

PROBLEM: Determine the required stormwater storage sump volume that can accommodate stormwater resulting from a 100 Year, 24-hour storm event

GIVEN: 1) 100-year, 24-hour storm event = 8.2-inches = 0/.683 foot.

a) Stage 1 Filling (see Figure 1 - assumes 0% if area covered with interim cover).
Area covered with interim cover = 0.
Active Area (tributary) = 179,134 sf.

b) Stage 2 Filling (see Figure 3 - assumes approx. 33% of area covered with interim cover).
Area covered with interim cover = 53,559 SF.
Active area (tributary) = 125,575 sf (conservative).

c) Stage 3 Filling (see Figure 5 - assumes approx. 67% of area covered with interim cover).
Area covered with interim cover =113,454 sf.
Active area (tributary) = 65,680 sf (conservative).

d) Stage 4 Filling (see figure 7 - 100% of area covered with interim cover).
Area covered with interim cover = 179,134 sf.
Active area (tributary) = 0 sf.

Calculation Summary Sheet

ASSUMPTIONS: a.) evapotranspiration is neglected

b.) Absorption through the waste is neglected

FACTOR OF SAFETY AND BASIS: Since stormwater will be stored in the sump, a minimum of 1-inch of freeboard is maintained at all times

REFERENCES USED: a.) Technical Paper No. 40, Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years, By David Hershfield.

SOLUTION: General Formula: Required Sump Volume (gallons) = Active Surface Area (ft^2) X 100-year, 24-hour Rainfall Event (inches) X 7.48 gallons per ft^3

Volume of sump is calculated using the following formula: Frustrum of right pyramid.

$$\text{Vol.} = \frac{H}{3}\left(AT + AB + \sqrt{AT + AB}\right)$$

Where:

V = Volume

H = Height

AT= Area Top

AB = Area Bottom

a) Stage 1 Filling

Required Storage Volume = 179,134 X 0.683 ≅ 122,500 cubic feet

Sump Volume Available = 134,000 cubic feet (based on AutoCad measurement & above formula)

b) Stage 2 Filling

Required Storage Volume: = 125,575 X 0.683 ≅ 86,000 cubic feet

Sump Volume Available = 96,500 cubic feet (based on AutoCad measurement & above formula)

c) Stage 3 Filling

Required Storage Volume: = 65,680 X 0.683 ≅ 45,000 cubic feet

Sump Volume Available = 54,400 cubic feet (based on AutoCad measurement and above formula)

d) Stage 4 Filling

No Sump Required. 100 percent area covered with interim cover.

SUMMARY OF SUMP STORAGE CAPACITY

Stage of Filling	Referenced Figures	Description	Storage Capacity Required (Cubic Feet)	Sump Capacity Available (Cubic Feet)
1	1	Final waste grading plan	122,500	134,000
2	3	Place Interim Cover on Northern Third of Final Waste Surface	86,000	96,500
3	5	Place Interim Cover on Middle Third of Final Waste Surface	45,000	54,400
4	7	Place Interim Cover on Southern Third of Final Waste Surface	0	0

Calculation Summary Sheet

CONCLUSION: The stormwater sump capacity as designed is sufficient to contain a 100-year, 24-hour rain event. The sumps are designed to accommodate 8.2 inches of rain, which meets the 100-year, 24-hour storm event.

The facility must size the length, width and depth of the stormwater storage sump so that it can accommodate required volume of stormwater for each dike extension as shown above. Based on the above calculation, the stormwater sump size may be gradually reduced with the addition of each extension dike.

Calculation Made By:	B. Stutes	Date: 10-18-04
Calculation Checked By:	F. Khan	Date: 10-20-04
Approved by	F. Khan	Date: 10-22-04

CITED REFERENCE

U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary

WEATHER BUREAU
F. W. REICHELDERFER, Chief

TECHNICAL PAPER NO. 40

RAINFALL FREQUENCY ATLAS OF THE UNITED STATES

for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years

Prepared by
DAVID M. HERSHFIELD
Cooperative Studies Section, Hydrologic Services Division
for
Engineering Division, Soil Conservation Service
U.S. Department of Agriculture

WASHINGTON, D.C.
May 1961

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C. Price $1.75