SCS ENGINEERS

RESPONSES TO REQUEST FOR ADDITIONAL INFORMATION (RAI) - MINOR MODIFICATION APPLICATION OPERATIONS PLAN PHASES I-VI FILL SEQUENCE

SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

Prepared for:

Hillsborough County Florida

Public Works Department
Solid Waste Management Division
332 North Falkenburg Road
Tampa, FL 33619
FDEP Permit No. 35435-022-SO-01
WACS ID No. 41193

Prepared by:

SCS Engineers
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Florida Board of Professional Engineers
Certificate Number 00004892

June 13, 2017
File No. 09215600.04

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SCS ENGINEERS

June 13, 2017
File No. 09215600.04

Mr. Henry Freedenberg, P.E., P.G.
Solid Waste Section
Florida Department of Environmental Protection
2600 Blair Stone Road, MB 4565
Tallahassee, FL 32399

Subject: Southeast County Landfill (SCLF), Hillsborough County
Operation Permit Minor Modification Application
Responses to Request for Additional Information (RAI)
WACS No. 41193
DEP Application Nos. 35435-024-SO-MM Revised Fill Sequence Phases I-IV

Dear Mr. Freedenberg:

On behalf of the Hillsborough County Public Works Department Solid Waste Management Division (SWMD), SCS Engineers (SCS) submits the following responses to your Request for Additional Information (RAI) in a letter dated May 8, 2017.

We have provided additional information, where applicable. If a response modifies a section of the application, the respective section(s) is updated accordingly. A complete version of the documents that include all revisions made in responding to this RAI are attached to this letter, using a strikethrough (e.g., ~~deleted~~) and underline (added) format, to facilitate review. We have included the revision date as part of the header/footer for all revised pages and provided an original and two copies of all revised materials.

For ease of review, each Florida Department of Environmental Protection (FDEP) comment is reiterated in bold type, followed by our response. The following are our responses:

1. **Please supply an updated set of sequence drawings. Please adjust the drawings labels (or the text of the document) so that the entire application displays consistent labeling. The submitted drawing package needs to display the title block associated with the engineer of record (SCS).**

   Response: Please refer to Attachment 1 for a complete set of sequence drawings with proposed revisions. The drawings provided in Attachment 1 include the title blocks associated with the drawing author and engineer of record at the time that the drawing was finalized. The cover sheet by SCS indicates the drawings revised by SCS. The set includes previously approved drawings by HDR Engineering, Inc. which have not changed.

2. **Please supply veneer stability calculations for the new sequence geometry. If the slope of the cell and the cover material configuration has not changed, a reference to a previous report that included these calcuations will be sufficent.**

   Response: A copy of the new sequence geometry veneer stability calculations, prepared by SCS and dated June 2, 2017, is provided in Attachment 2 and the Attachment 2 appendices (Attachments 2-1 through 2-6). The calculations and the corresponding attachments depict why we have concluded that veneer cover will be stable. Additionally, no past incidents involving veneer stability have been recorded at SCLF.

3. **A spreadsheet output is supplied that displays the expected compaction at multiple locations within the fill. Please provide a calculation that displays the procedure used in calculating the spreadsheet values. Doing this for a single critical location is sufficient.**

   Response: The procedure for calculating the expected compaction is provided in Attachment 3-A through Attachment 3-C. As an example, we have provided the method to calculating the value at TH-48 with references appended to the method.

4. **Please provide time rate of settlement calculations that reflect both the compaction of the fill material and the continuous adjustment of the surcharge load. We are interested in time to both 50% and 90% settlement and, also, developing cross sections showing the settlement to be expected at 5 year intervals. Please develop two cross sections that shows the bottom expected elevation of two LCS lines at 5 year intervals. These cross sections should be consistent with your time rate of settlement calculations. Please use the most critical LCS paths for your cross drawings. Also, Chapter 3 of Qian and Koerner ("Geotechnical Aspects of Landfill Design and Construction") provides a discussion of compacted clay liner design and performance. Select portions of this chapter might be relevant to the current application.**

   Response: Please refer to Attachment 4 and the Attachment 4 appendices for a response to comment No. 4 of the RAI. This attachment, produced by Ardaman & Associates, Inc., includes a discussion regarding the rate of consolidation of the clay layer and cross sections depicting the time settlement calculations along two critical Leachate Collection and Recovery System (LCRS) paths. The figures indicate that the clays will continue to consolidate and the projected settlement will provide downward slopes leading to the projected low point in Phase VI, PS-B, as predicted.

5. **Please provide a complete updated Operation Plan that includes a drawing displaying all LFG monitoring points as listed in the table on page 18 of the application submittal.**

   Response: A complete updated Operations Plan that includes a drawing displaying all LFG monitoring points as listed in the table on page 18 of the application submittal is provided in the Updated Operations Plan located in Attachment 5 of this submittal. Upon FDEP approval of this Minor Modification to the Operating Permit, a "conformed" copy of the Operations Plan will be provided.

As previously discussed with the FDEP, waste filling operations at the SCLF have moved from Phases I-IV to the Capacity Expansion Area (CEA). The CEA is near capacity, and the SWMD will need to move operations back to Lift 16A very soon. To this end, we would appreciate a quick response to this submittal.

As required, this response has been certified and signed by a professional engineer. We have enclosed one copy of our response. Please call us if you have any questions.

Sincerely,

B Clark

Robert B. Curtis, P.E.
Project Manager
SCS ENGINEERS

Bruce J. Clark, P.E.
Project Director
SCS ENGINEERS

RBC:kls

cc: Kimberly A. Byer, HCSWMD
Larry Ruiz, HCSWMD
Ron Cope, HCEPC
Melissa Madden, FDEP Tampa

ATTACHMENT 1

UPDATED SEQUENCE DRAWINGS

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY

TAMPA, FLORIDA
APRIL 2017

Hillsborough County Florida
EST. 1834

BOARD OF COUNTY COMMISSIONERS

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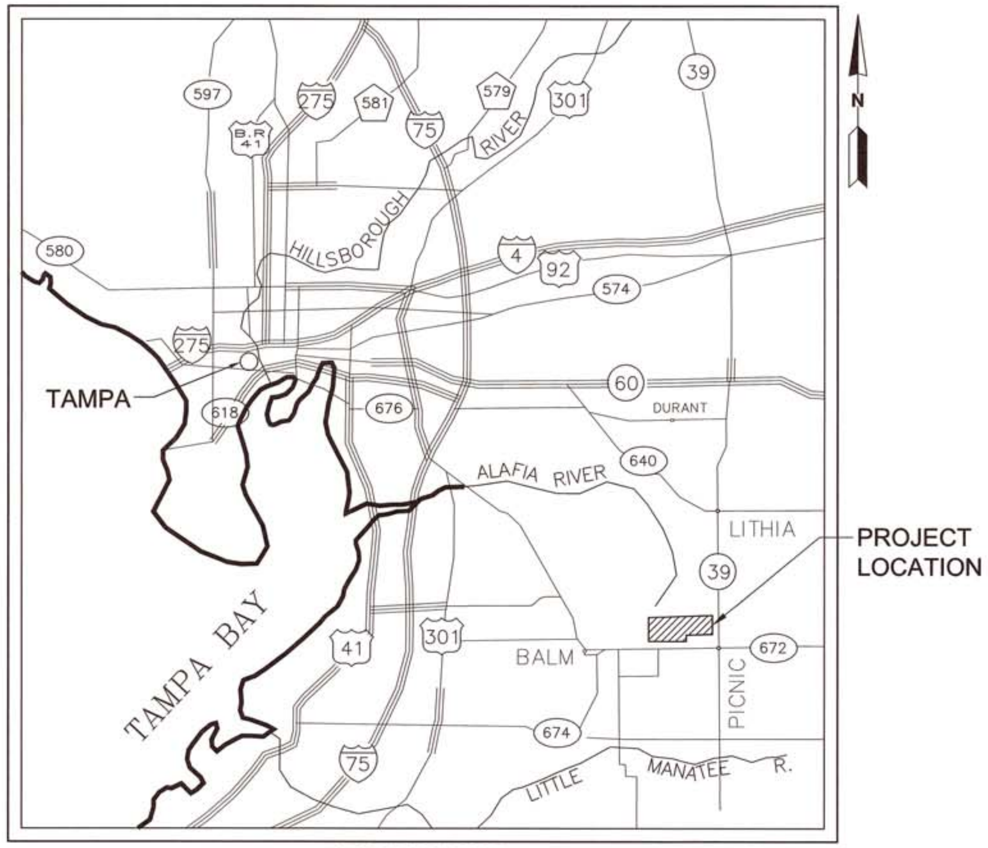

LOCATION MAP
NOT TO SCALE

INDEX OF DRAWINGS

SHEET	SHEET TITLE
1	COVER SHEET
2	INDEX, LEGENDS AND GENERAL NOTES
3	FACILITY SITE PLAN AND EXISTING TOPOGRAPHY
~~4~~	~~PHASES I TO IV - LIFTS 13 TO 16~~
4A	PHASES I - LIFT 13
4B	PHASES VI, IV, I - LIFT 16A
4C	PHASES IV, VI, V - LIFT 17A
~~5~~	~~PHASES V AND VI - LIFT 17~~
5A	PHASES I, III - LIFTS 14, 15
6	PHASES I TO IV - LIFTS 18 TO 21
7	PHASES V AND VI - LIFT 22
8	PHASES V AND VI - LIFT 23 (FINAL LIFT)
9	SINKHOLE REMEDIATION PLAN
10	LANDFILL CROSS SECTIONS
11	SINKHOLE REMEDIATION CROSS SECTION
12	DETAILS 1
13	DETAILS 2
14	DETAILS 3
15	DETAILS 4

NOTE:

THIS UPDATE TO PHASE I-VI OPERATING SEQUENCE DRAWINGS INCLUDE MODIFICATIONS TO LIFT SEQUENCES 13 THROUGH 17; AS SUCH, SCS ENGINEERS IS ONLY SIGNING AND SEALING SHEETS 4A, 4B, 4C, AND 5A. THE REMAINING LIFT SEQUENCES 18 THROUGH 23 (FINAL LIFT) WILL CONTINUE IN ACCORDANCE WITH THE CURRENTLY FDEP APPROVED OPERATING SEQUENCE DRAWINGS, DATED JUNE 2013, PREPARED, SIGNED AND SEALED BY HDR ENGINEERING, INC.

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SCS PROJECT NO. 09215600.03

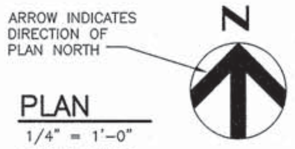
ROBERT B. CURTIS
No. 73758
4-7-2017
ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

\\Tam-fs02\tampa\dwg\PROJECT\Hillsborough\09215600.03 – Solid Waste\Engineering Drawings\09215600.03 – C-00.dwg Apr 06, 2017 – 4:33pm Layout Name: COVER SHEET By: 3630rlc

ENGINEERING SYMBOLOGY

9A	LIFT NUMBER
➡	DAILY PROGRESSION
——→	FILL PROGRESSION
∿←	DRAINAGE FLOW DIRECTION
- - - - - - -	APPROXIMATE PHASE FOOTPRINT
—— · · ——	APPROXIMATE LANDFILL LIMITS
— — — —	APPROXIMATE LIFT LIMITS
— — · —	APPROXIMATE LIMITS OF BORROW AREA
▨	APPROXIMATE TEMPORARY FINAL COVER AREA AFTER EACH LIFT
—·—·—·—	EXISTING SWALE
—— — ——	SIDESLOPE DITCH
——140——	EXISTING CONTOUR
——**140**——	PROPOSED CONTOUR
————	PROPOSED DOWNCHUTE
BVC	BEGIN VERTICAL CURVE
CTRD	CENTERED
ERCP	ELLIPTICAL REINFORCED CONCRETE PIPE
EVC	END VERTICAL CURVE
∼∼150∼∼	EXIST. CONTOURS
EXP. JT.	EXPANSION JOINT
I.E.	INVERT ELEVATION
LF	LINEAR FEET
LT	LEFT
PC	POINT OF CURVATURE
PI	POINT OF INTERSECTION
PT	POINT OF TANGENCY
PVI	POINT OF VERTICAL INTERSECTION
RT	RIGHT
TYP.	TYPICAL
∼∼∼∼	VEGETATION
VC	VERTICAL CURVE
Ø	DIAMETER
'	FOOT
"	INCH
Ⓐ	CELL DESIGNATION
(S-21)	EXISTING DRAINAGE STRUCTURE
TS-4	TEMPORARY DRAINAGE STRUCTURE
—— — — ——	PROPERTY LINE
>——<	STORMWATER STRUCTURE
CDW →	DIRECTION OF CONDENSATE/ DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)

GENERAL SYMBOLOGY



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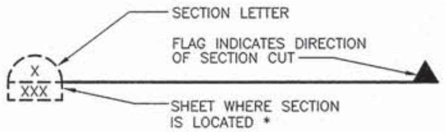
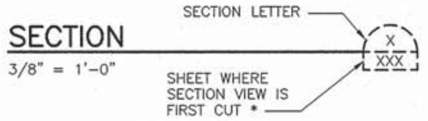


SECTION CUT MARKER



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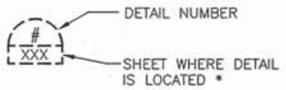
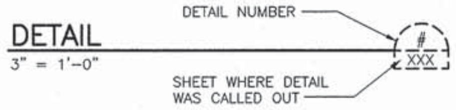


DETAIL MARKER

FOR REFERENCING DETAILS INCLUDED IN DRAWING SET.

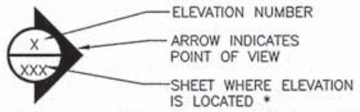
DETAIL MARKER

FOR REFERENCING DETAILS BOUND IN SPECIFICATIONS OR SEPARATE VOLUME.



DETAIL TITLE

* FOR COMMON DETAILS AND SECTIONS OR DETAILS THAT ARE CUT OR CALLED OUT ON MULTIPLE SHEETS, THE SHEET NUMBER IS REPLACED BY A DASH (–).



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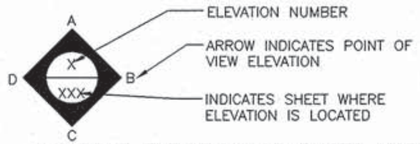
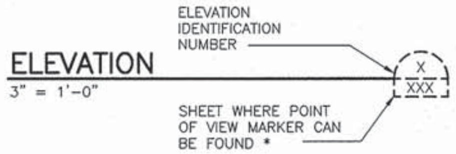


MULTIPLE ELEVATION OR PHOTO MARKER



ELEVATION TITLE

ABBREVIATIONS

APPROX – APPROXIMATE, APPROXIMATELY
BLDG – BUILDING
BTM – BOTTOM
CB – CATCH BASIN
CM – CONCRETE MONUMENT
CMP – CORRUGATED METAL PIPE
CONC – CONCRETE
CONT – CONTINUOUS
CORR – CORRUGATED
DET – DETAIL
DIA – DIAMETER
DIM – DIMENSION
DWG – DRAWING
EA – EACH
EOL – EDGE OF LINER
ETC – ET CETERA
ENCL – ENCLOSE, ENCLOSURE
EL – ELEVATION
EQUIP – EQUIPMENT
EXIST – EXISTING
FDEP – FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
FDOT – FLORIDA DEPARTMENT OF TRANSPORTATION
FES – FLARED END SECTION
FIN – FINISHED
FM – FORCE MAIN
GALV – GALVANIZED
GCL – GEOSYNTHETIC CLAY LINER
GFFR – GROUT FILLED FIBER REVETMENT
GR – GRADE
GDL – GEOSYNTHETIC DRAINAGE LINER
GM – GAS MONITORING LOCATION
GP – GAS PROBE
HCSWMG – HILLSBOROUGH COUNTY SOLID WASTE MANAGEMENT GROUP
HDPE – HIGH DENSITY POLYETHYLENE
HP – HIGH POINT
ID – INSIDE DIAMETER
IE – INVERT ELEVATION
LF – LINEAL FEET
LFG – LANDFILL GAS
LLDPE – LINEAR LOW DENSITY POLYETHYLENE
LP – LOW POINT
MES – MITRED END SECTION
MAX – MAXIMUM
MH – MANHOLE
MIN – MINIMUM
MISC – MISCELLANEOUS
MSL – (ABOVE) MEAN SEAL LEVEL
MT – MOUNT
MW – GROUNDWATER MONITORING WELL
N/A – NOT APPLICABLE
N/AVAIL – NOT AVAILABLE
NGVD – NATIONAL GEODETIC VERTICAL DATUM
NIC – NOT IN CONTRACT
No – NUMBER
NTS – NOT TO SCALE
OC – ON CENTER
OD – OUTSIDE DIAMETER
OSHA – OCCUPATIONAL SAFETY & HEALTH ADMINISTRATION
PLS – PROFESSIONAL LAND SURVEYOR
PS – PUMP STATION
R – RADIUS
RCP – REINFORCED CONCRETE PIPE
REF – REFERENCE
REQD – REQUIRED
SCH – SCHEDULE
SDR – STANDARD DIMENSION RATIO
SHT – SHEET
SIM – SIMILAR
SS – STAINLESS STEEL
STD – STANDARD
STL – STEEL
STW – STORMWATER MONITORING STATION
TPO – THERMOPLASTIC POLYOLEFIN
TYP – TYPICAL
USC&GS – UNITED STATES COASTAL AND GEODETIC SURVEY
USGS – UNITED STATES GEOLOGICAL SURVEY
WGT – WEIGHT
W.E. – WATER ELEVATION
WWTP – WASTE WATER TREATMENT PLANT

GENERAL NOTES

1. THE EXISTING TOPOGRAPHY, DATED 1/5/2013, WAS OBTAINED FROM DRAWINGS PROVIDED BY PICKETT & ASSOCIATES, INC.
2. THE PROPOSED OPERATING SEQUENCES (LIFTS 13 – 23) ARE BASED ON THE EXISTING TOPOGRAPHY SHOWN ON THE 1/5/2013 AERIAL SURVEY. ACTUAL OPERATING SEQUENCES MAY NEED TO BE MODIFIED IN THE FIELD TO ALLOW FOR LANDFILL SETTLEMENT. REVISED GRADES WILL BE DETERMINED BASED ON THE MAXIMUM DESIGNED 20-FOOT LIFT HEIGHT.
3. THE LANDFILL LINER AND EXISTING DRAINAGE STRUCTURES WERE SURVEYED BY WEIDENER SURVEY AND MAPPING PA ON 2/23/94.

Hillsborough County
Florida

HDR

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS RBC
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

ROBERT B. CURTIS
No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

INDEX, LEGENDS AND GENERAL NOTES

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DWG No.
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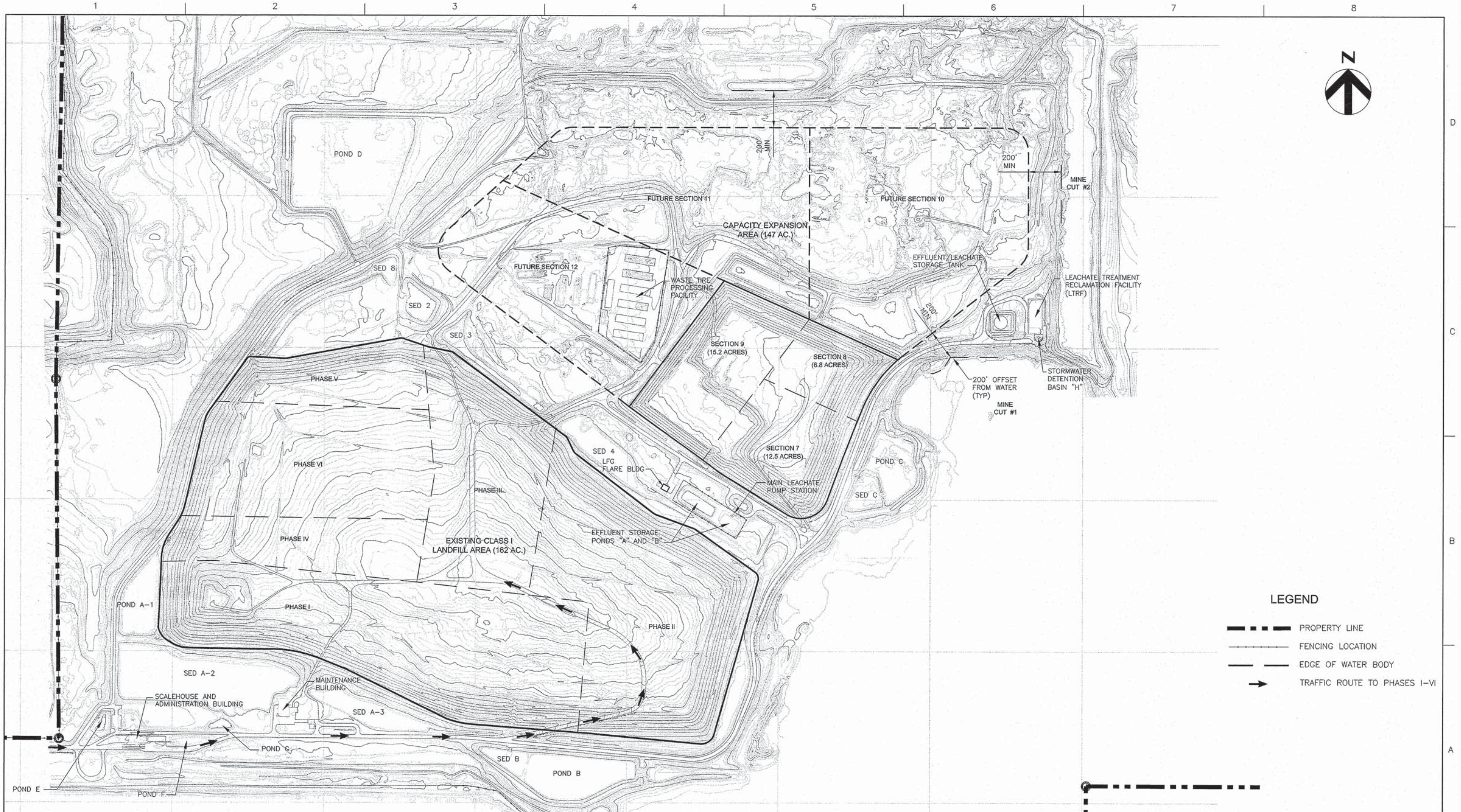

NOTE:
EXISTING TOPOGRAPHY FROM AERIAL PHOTOGRAPHY PROVIDED BY PICKETT AND ASSOCIATES, INC. DATED JANUARY 5, 2013.

Hillsborough County
Florida

HDR

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HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096–193806–001

ROBERT B. CURTIS
No. 73758
6/7/13

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CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

FACILITY SITE PLAN AND EXISTING TOPOGRAPHY

0 1" 2"

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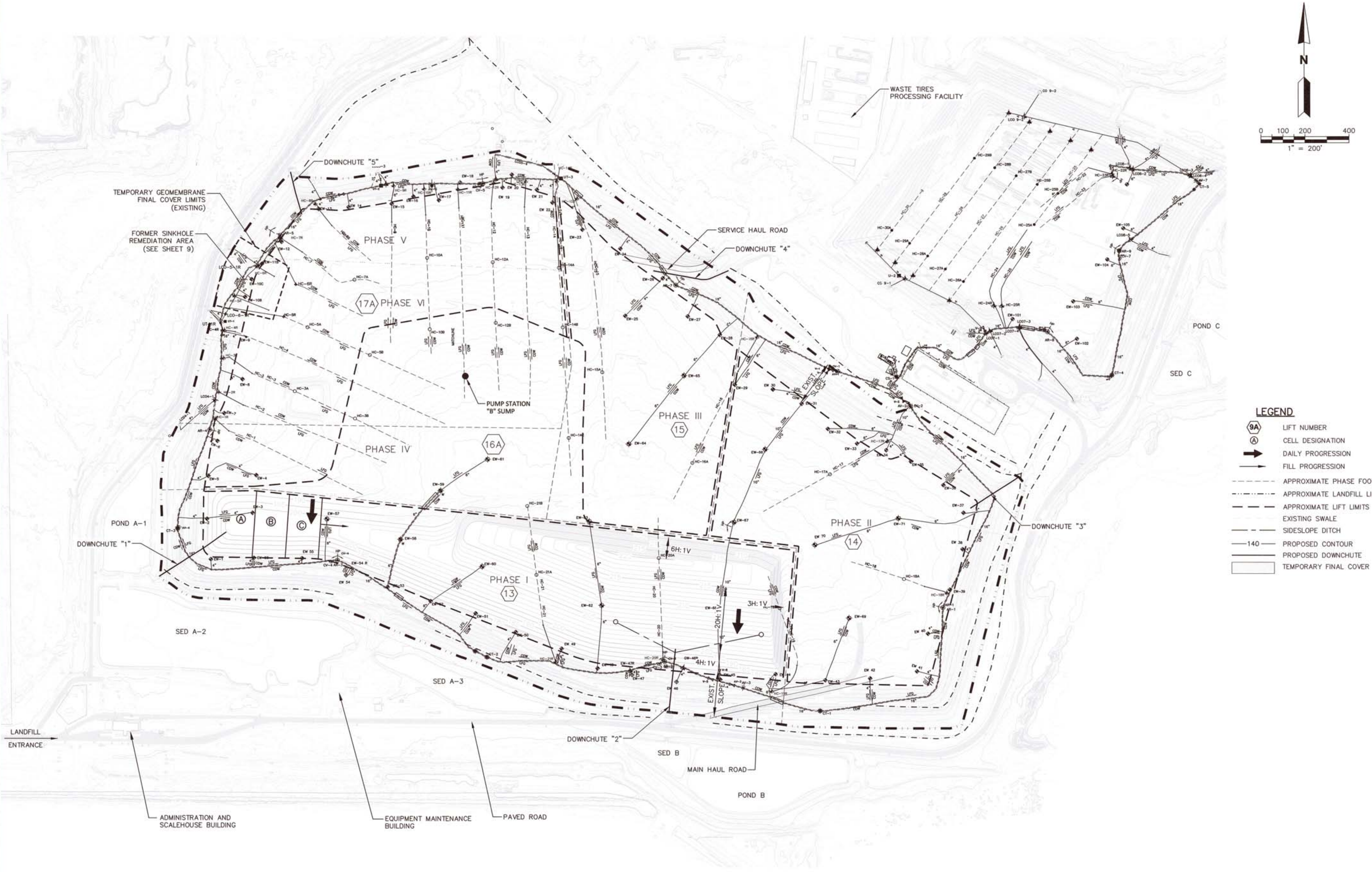

LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
- - - - - -	APPROXIMATE PHASE FOOTPRINT
-··-··-	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
	EXISTING SWALE
—— - ——	SIDESLOPE DITCH
—140—	PROPOSED CONTOUR
———	PROPOSED DOWNCHUTE
▭	TEMPORARY FINAL COVER

PROGRESSION:

1. BEGIN LIFT 13 FILL IN CELL "A", ON WEST SIDE OF PHASE I AND CONTINUE EASTERLY ACROSS PHASE I TO THE PHASE II BOUNDARY.
2. EXTEND DOWNCHUTES 1 AND 2 DURING PLACEMENT OF INTERMEDIATE COVER.

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ROBERT B. CURTIS LICENSE No. 73758 STATE OF FLORIDA PROFESSIONAL ENGINEER
4-7-17

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
△	4/7/17	FDEP SUBMITTAL	RBC
△			
△			
△			
△			
△			

SHEET TITLE
PHASE I - LIFT 13

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL
FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

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FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO. 09215600.03	DWN. BY: RLC	Q/A RVW BY: RBC
DSN. BY: CAR	CHK. BY: RBC	APP. BY: RBC

CADD FILE:
09215600.03 – C-4A

DATE:
APRIL 2017

SCALE:
AS SHOWN

DRAWING NO.
4A

SHEET **4A** of **15**

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LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
⟶	FILL PROGRESSION
- - - - - -	APPROXIMATE PHASE FOOTPRINT
-··—··—	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
—·—·—	EXISTING SWALE
—— - ——	SIDESLOPE DITCH
—140—	PROPOSED CONTOUR
———	PROPOSED DOWNCHUTE
▭	TEMPORARY FINAL COVER

PROGRESSION:

BEGIN LIFT 16A FILL IN CELL "A", WEST SIDE OF PHASE III AND CONTINUE WEST-SOUTHWEST ACROSS PHASES VI, IV, AND I TO THE NORTHERN LIMIT OF LIFT 13.

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
1	4/7/17	FDEP SUBMITTAL	RBC

SHEET TITLE
PHASE VI, IV, I - LIFT 16A

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL
FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

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FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO.	DWN. BY:	Q/A RVW BY:
09215600.03	RLC	RBC
DSN. BY: CAR	CHK. BY: RBC	APP. BY: RBC

CADD FILE:
09215600.03 – C-4B

DATE:
APRIL 2017

SCALE:
AS SHOWN

DRAWING NO.
4B

SHEET 4B of 15

PROGRESSION:

BEGIN LIFT 17A FILL IN CELL "A", NORTH SIDE OF PHASE I AND CONTINUE CLOCKWISE ACROSS PHASES IV, VI, AND V TO THE WESTERN LIMIT OF PHASE III.

REV	DATE	DESCRIPTION	CHK. BY
1	4/7/17	FDEP SUBMITTAL	RBC

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

SHEET TITLE
PHASE IV, VI, V - LIFT 17A

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

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FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO. 09215600.03
DWN. BY: RLC
Q/A RVW BY: RBC
DSN. BY: CAR
CHK. BY: RBC
APP. BY: RBC

CADD FILE:
09215600.03 – C-4C

DATE:
APRIL 2017

SCALE:
AS SHOWN

DRAWING NO.
4C

SHEET 4C of 15

PROGRESSION:

BEGIN LIFT 14 FILL IN CELL "A", WEST SIDE OF PHASE II AND CONTINUE COUNTERCLOCKWISE TO THE PHASE III BOUNDARY. BEGIN LIFT 15 ON THE NORTHERN SIDE SLOPES AND PREPARE THE PHASE III SIDE SLOPES AND THE SERVICE LANE ROAD FOR CLOSURE. THEN CONTINUE LIFT 15 FILL IN CELL "A", EAST SIDE PHASE III AND CONTINUE WEST ACROSS PHASE III TO THE WEST SIDE OF PHASE III.

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ROBERT B. CURTIS
No. 73758
4-7-17

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
1	4/7/17	FDEP SUBMITTAL	RBC

SHEET TITLE
PHASES I, II, III - LIFTS 14, 15

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL
FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

SCS ENGINEERS
STEARNS, CONRAD AND SCHMIDT
CONSULTING ENGINEERS, INC.
4041 PARK OAKS BLVD., SUITE 100, TAMPA, FL 33610
PH. (813) 621-0080 FAX. (813) 623-6757
FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO. 09215600.03
DWN. BY: RLC
Q/A RVW BY: RBC
DSN. BY: CAR
CHK. BY: RBC
APP. BY: RBC

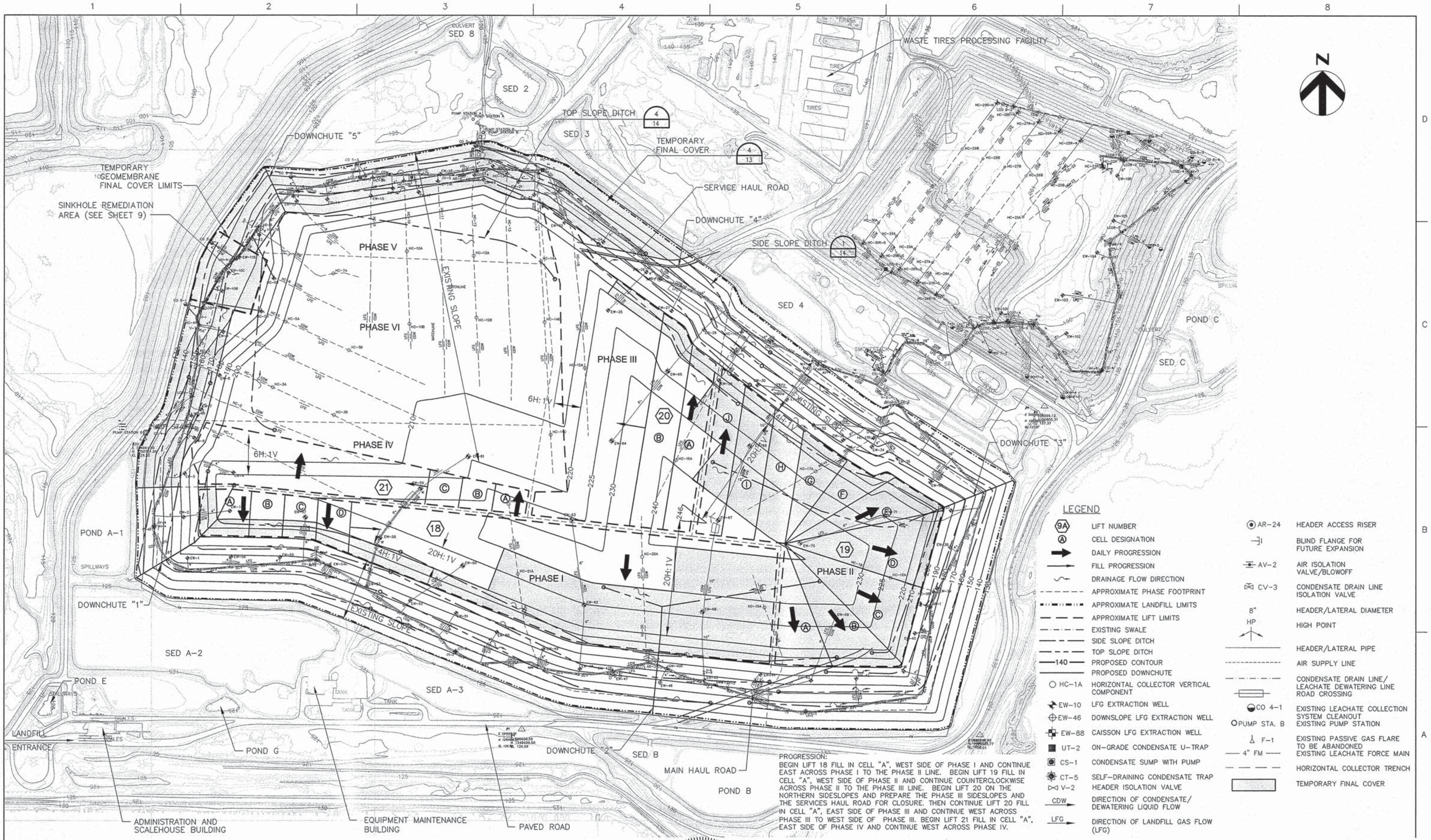
CADD FILE:
09215600.03 – C-5A

DATE:
APRIL 2017

SCALE:
AS SHOWN

DRAWING NO.
5A

SHEET **5A** of **15**



LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
	DRAINAGE FLOW DIRECTION
- - - - - -	APPROXIMATE PHASE FOOTPRINT
	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
	EXISTING SWALE
	SIDE SLOPE DITCH
	TOP SLOPE DITCH
—140—	PROPOSED CONTOUR
	PROPOSED DOWNCHUTE
○ HC-1A	HORIZONTAL COLLECTOR VERTICAL COMPONENT
EW-10	LFG EXTRACTION WELL
EW-46	DOWNSLOPE LFG EXTRACTION WELL
EW-88	CAISSON LFG EXTRACTION WELL
UT-2	ON-GRADE CONDENSATE U-TRAP
CS-1	CONDENSATE SUMP WITH PUMP
CT-5	SELF-DRAINING CONDENSATE TRAP
⋈ V-2	HEADER ISOLATION VALVE
CDW →	DIRECTION OF CONDENSATE/ DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)
◉ AR-24	HEADER ACCESS RISER
	BLIND FLANGE FOR FUTURE EXPANSION
AV-2	AIR ISOLATION VALVE/BLOWOFF
CV-3	CONDENSATE DRAIN LINE ISOLATION VALVE
8"	HEADER/LATERAL DIAMETER
HP	HIGH POINT
	HEADER/LATERAL PIPE
	AIR SUPPLY LINE
	CONDENSATE DRAIN LINE/ LEACHATE DEWATERING LINE
	ROAD CROSSING
CO 4-1	EXISTING LEACHATE COLLECTION SYSTEM CLEANOUT
○ PUMP STA. B	EXISTING PUMP STATION
F-1	EXISTING PASSIVE GAS FLARE TO BE ABANDONED
— 4" FM —	EXISTING LEACHATE FORCE MAIN
	HORIZONTAL COLLECTOR TRENCH
▨	TEMPORARY FINAL COVER

PROGRESSION:
BEGIN LIFT 18 FILL IN CELL "A", WEST SIDE OF PHASE I AND CONTINUE EAST ACROSS PHASE I TO THE PHASE II LINE. BEGIN LIFT 19 FILL IN CELL "A", WEST SIDE OF PHASE II AND CONTINUE COUNTERCLOCKWISE ACROSS PHASE II TO THE PHASE III LINE. BEGIN LIFT 20 ON THE NORTHERN SIDESLOPES AND PREPARE THE PHASE III SIDESLOPES AND THE SERVICES HAUL ROAD FOR CLOSURE. THEN CONTINUE LIFT 20 FILL IN CELL "A", EAST SIDE OF PHASE III AND CONTINUE WEST ACROSS PHASE III TO WEST SIDE OF PHASE III. BEGIN LIFT 21 FILL IN CELL "A", EAST SIDE OF PHASE IV AND CONTINUE WEST ACROSS PHASE IV.

Hillsborough County
Florida

HDR

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

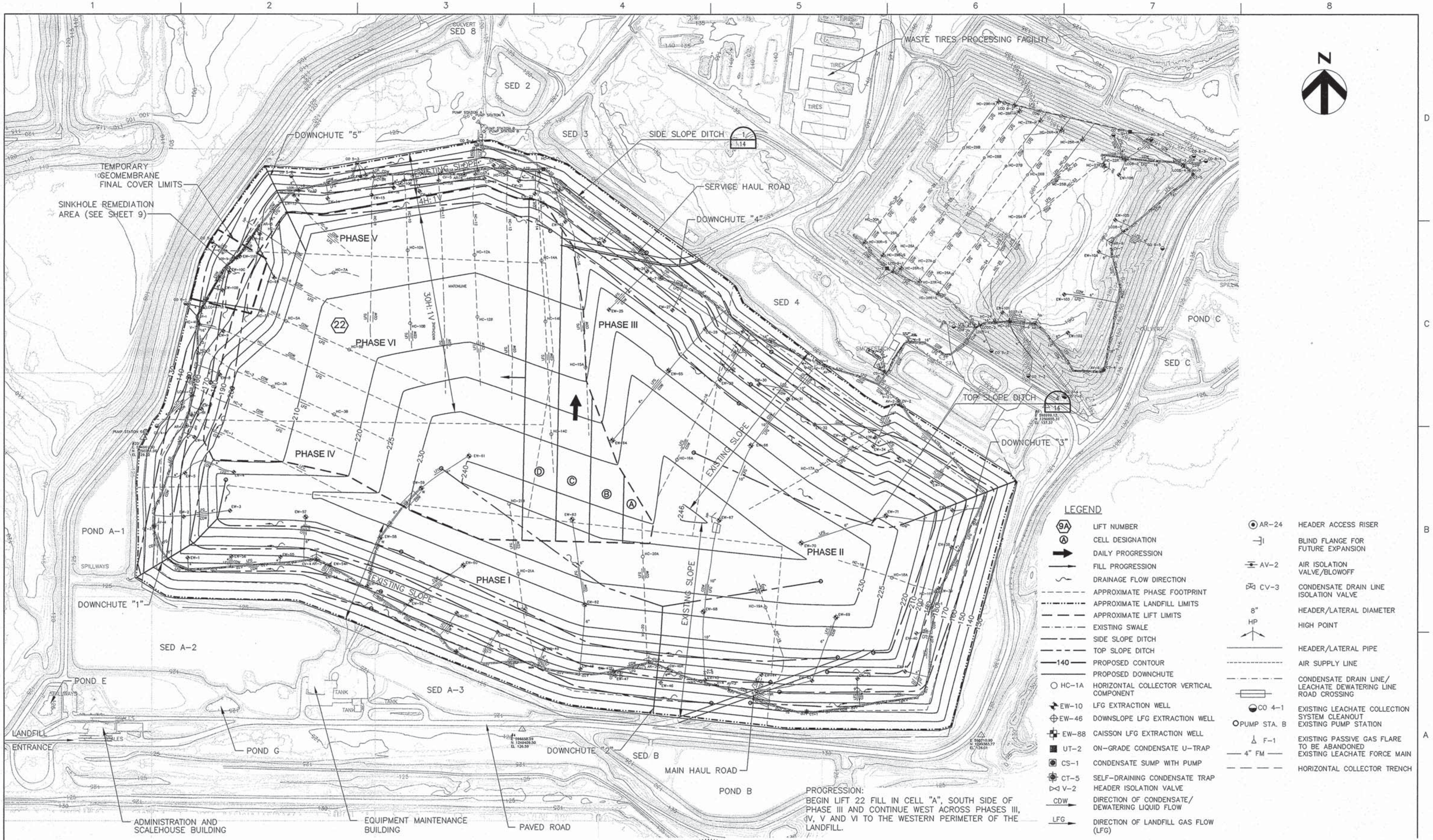
No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES I TO IV - LIFTS 18 TO 21

0 1" 2"

FILENAME	00C-06.DWG
SCALE	1"=200'
DWG No.	6



PROGRESSION:
BEGIN LIFT 22 FILL IN CELL "A", SOUTH SIDE OF PHASE III AND CONTINUE WEST ACROSS PHASES III, IV, V AND VI TO THE WESTERN PERIMETER OF THE LANDFILL.

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES V AND VI - LIFT 22

0 1" 2"

FILENAME	00C-07.DWG
SCALE	1"=200'

DWG No.
7

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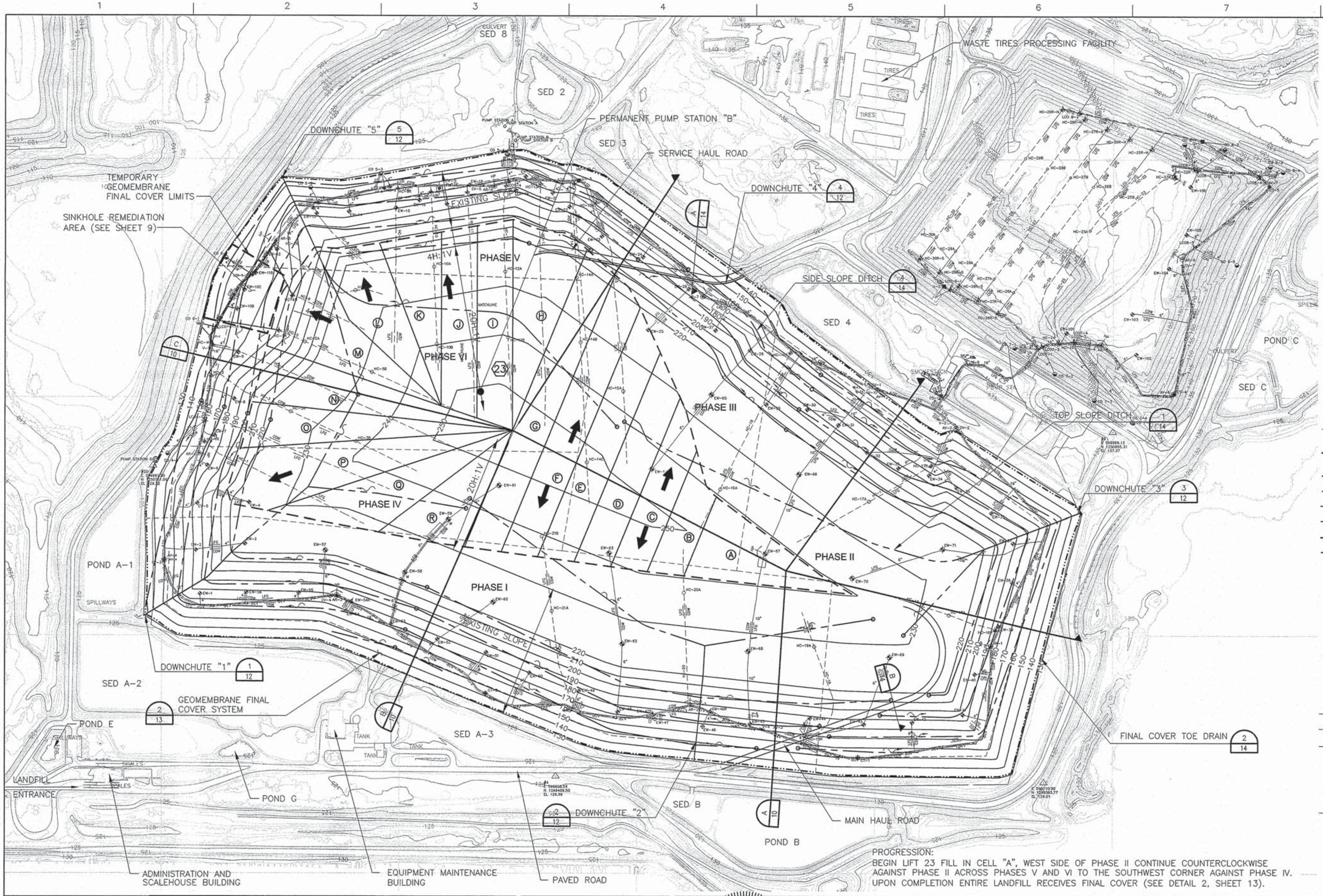

PROGRESSION:
BEGIN LIFT 23 FILL IN CELL "A", WEST SIDE OF PHASE II CONTINUE COUNTERCLOCKWISE AGAINST PHASE II ACROSS PHASES V AND VI TO THE SOUTHWEST CORNER AGAINST PHASE IV. UPON COMPLETION ENTIRE LANDFILL RECEIVES FINAL COVER (SEE DETAIL 2, SHEET 13).

LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
∼→	DRAINAGE FLOW DIRECTION
- - - - - - - -	APPROXIMATE PHASE FOOTPRINT
—··—··—	APPROXIMATE LANDFILL LIMITS
— — — —	APPROXIMATE LIFT LIMITS
—·—·—·—	EXISTING SWALE
—— - ——	SIDE SLOPE DITCH
—— - - - ——	TOP SLOPE DITCH
—140—	PROPOSED CONTOUR
————	PROPOSED DOWNCHUTE
EW-10	LFG EXTRACTION WELL
⊕EW-46	DOWNSLOPE LFG EXTRACTION WELL
EW-88	CAISSON LFG EXTRACTION WELL
■ UT-2	ON-GRADE CONDENSATE U-TRAP
CS-1	CONDENSATE SUMP WITH PUMP
CT-5	SELF-DRAINING CONDENSATE TRAP
⊙ AR-24	HEADER ACCESS RISER
————	HEADER/LATERAL PIPE
------------	AIR SUPPLY LINE
—·—·—·—	CONDENSATE DRAIN LINE/LEACHATE DEWATERING LINE
CO 4-1	EXISTING LEACHATE COLLECTION SYSTEM CLEANOUT
— 4" FM —	EXISTING LEACHATE FORCE MAIN
CDW →	DIRECTION OF CONDENSATE /DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)

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PROJECT NUMBER	0096-193806-001

No. 73758
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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES V AND VI - LIFT 23 (FINAL LIFT)

0 1" 2"

FILENAME	00C-08.DWG
SCALE	1"=200'

DWG No.
8

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LIMITS OF FINAL COVER			
DESCRIPTION	NORTHING	EASTING	ELEVATION
W1	1,251,380.1	595,181.34	129.32
W2	1,251,332.6	595,169.56	129.21
W3	1,251,312.3	595,247.07	142.69
W4	1,251,291.4	595,336.55	158.01
W5	1,251,265.5	595,442.88	162.72
W6	1,251,385.8	595,481.43	162.31
W7	1,251,505.8	595,517.36	162.22
W8	1,251,545.4	595,453.35	159.23
W9	1,251,597.7	595,367.51	145.05
W10	1,251,642.4	595,291.38	126.82
W11	1,251,548.6	595,233.61	128.73
W12	1,251,484.4	595,207.36	129.31

LEGEND:

(W1) (W2)	WOODEN MARKER POST (LOCATIONS ON TABLE THIS SHEET)
——— TD ———	TOE DRAIN
— — 155 — —	EXISTING CONTOUR
———155———	PROPOSED CONTOUR

NOTES:
1. EXISTING TOPOGRAPHY PROVIDED BY PICKETT AND ASSOCIATES, INC. FROM AERIAL PHOTOGRAPHY DATED JANUARY 5, 2013.
2. LFG SYSTEM NOT SHOWN FOR CLARITY OF DRAWING.
3. EXISTING STORMWATER PIPES TO BE RELOCATED PER STORMWATER PLANS.

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SINKHOLE REMEDIATION PLAN

0 1" 2"

FILENAME	00C-09.dwg
SCALE	1"=50'
DWG No.	9

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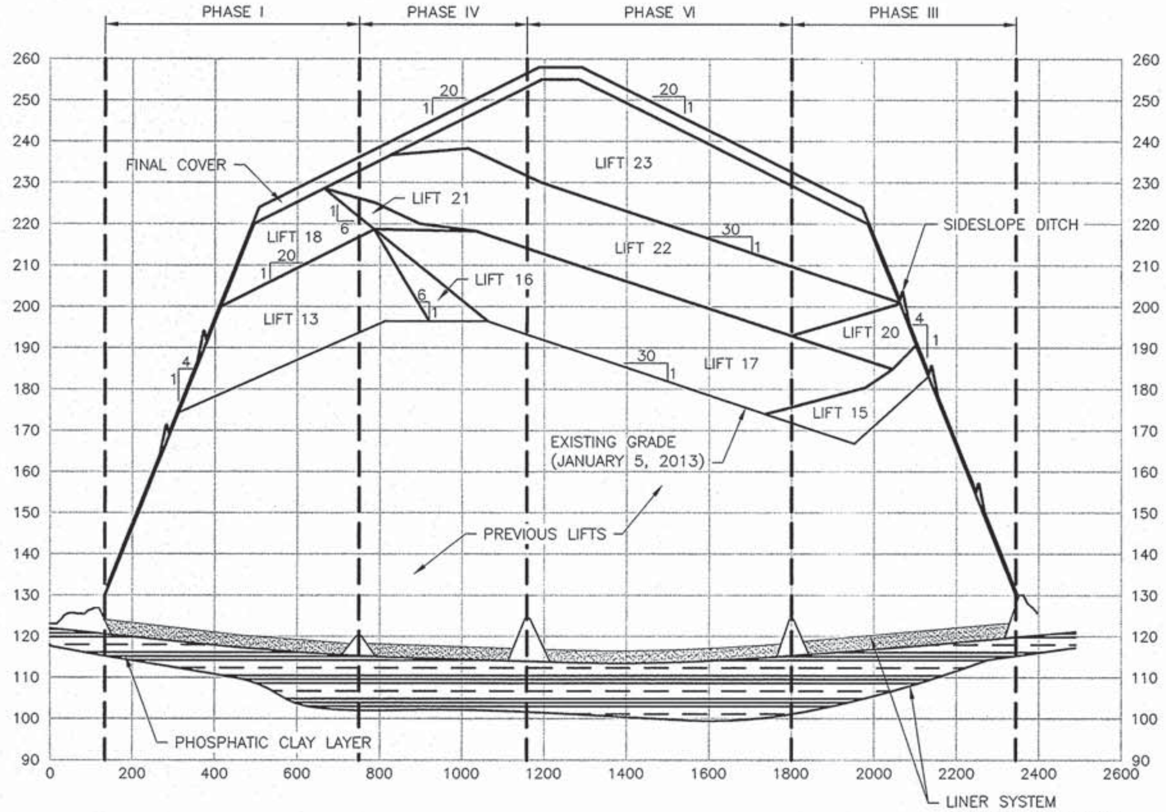
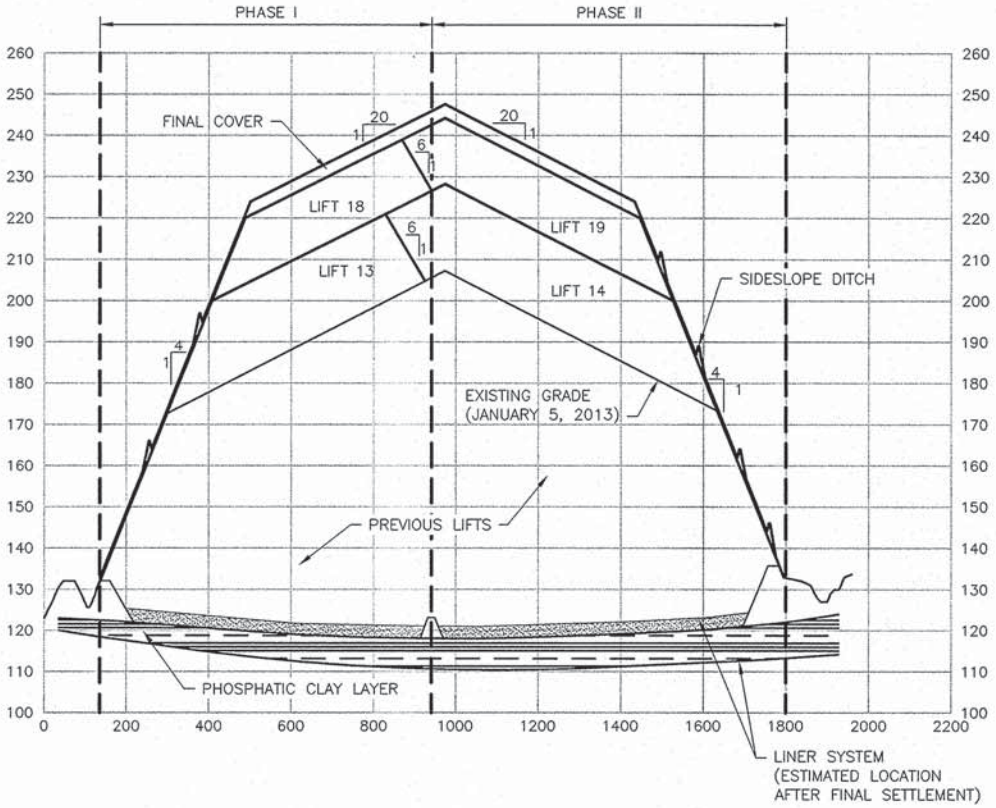
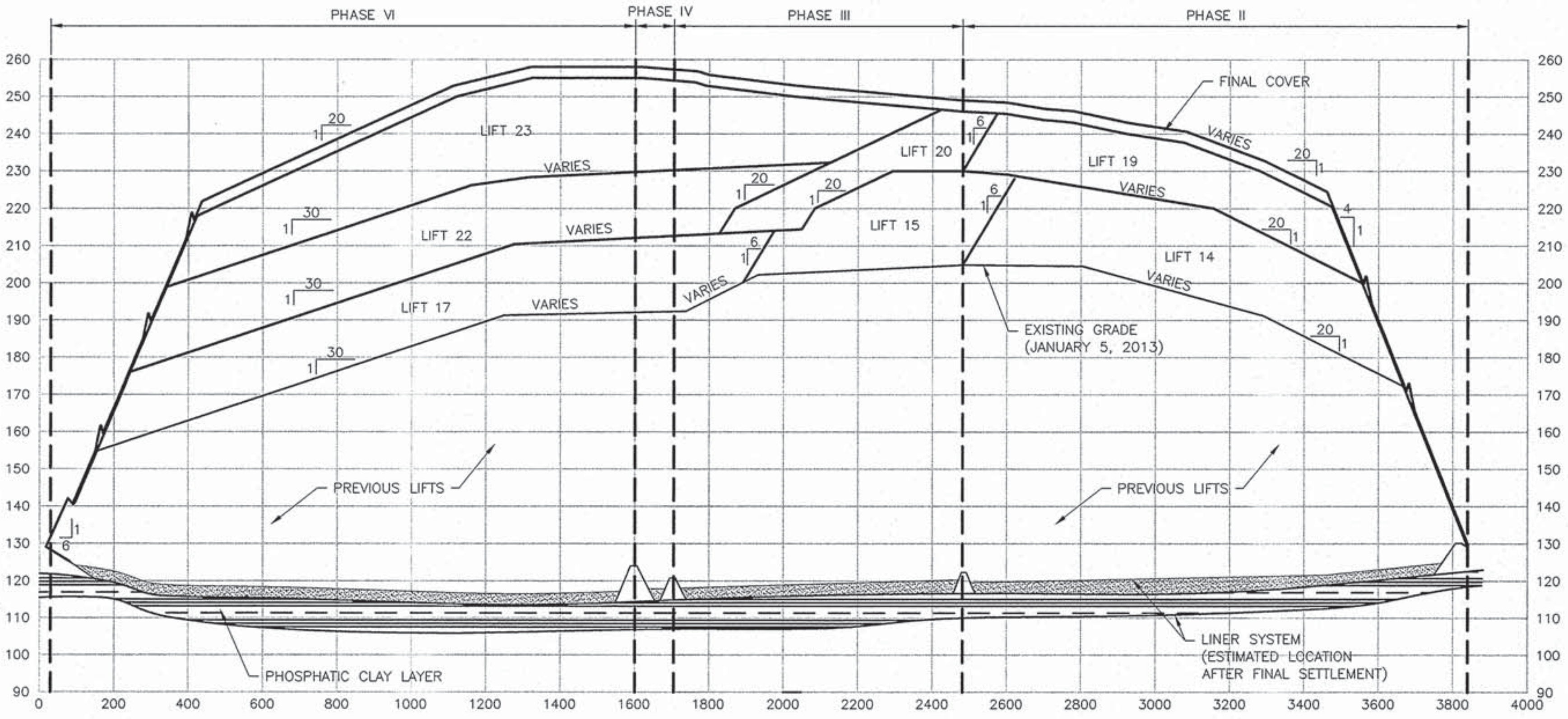


LANDFILL CROSS SECTION B/8



LANDFILL CROSS SECTION A/8



LANDFILL CROSS SECTION C/8

LEGEND:

▨ LINER SYSTEM

☰ PHOSPHATIC CLAY LAYER

NOTE:
SOME SLOPES AND LIFT SIZE VARY DUE TO SECTION CUT ORIENTATION. SEE SHEET C-08 FOR SECTION LOCATIONS.

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

LANDFILL CROSS SECTIONS

0 1" 2"

FILENAME	00C-10.DWG
SCALE	NOT TO SCALE

DWG No.
10

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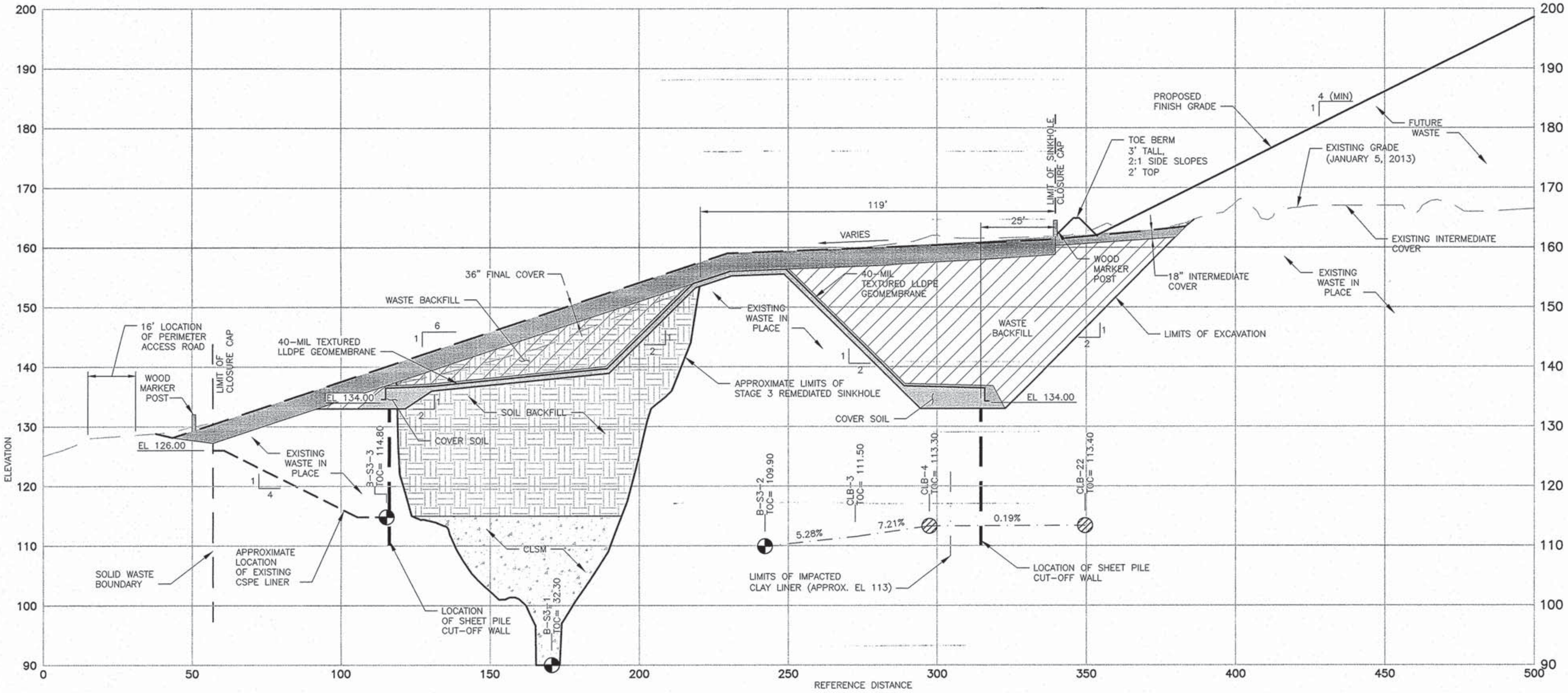

SECTION

D
9

SCALE: 1"=20' (HORIZONTAL)
1"=10' (VERTICAL)

LEGEND:

●	GEOTECHNICAL BORING LOCATION (SDII)
▨	GEOTECHNICAL BORING LOCATION (TIERRA)

TOC = TOP OF CLAY ELEVATION

CLSM = CONTROLLED LOW STRENGTH MATERIAL

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SINKHOLE REMEDIATION CROSS SECTION

0 1" 2"

FILENAME	00C-11.DWG
SCALE	AS SHOWN

DWG No.
11

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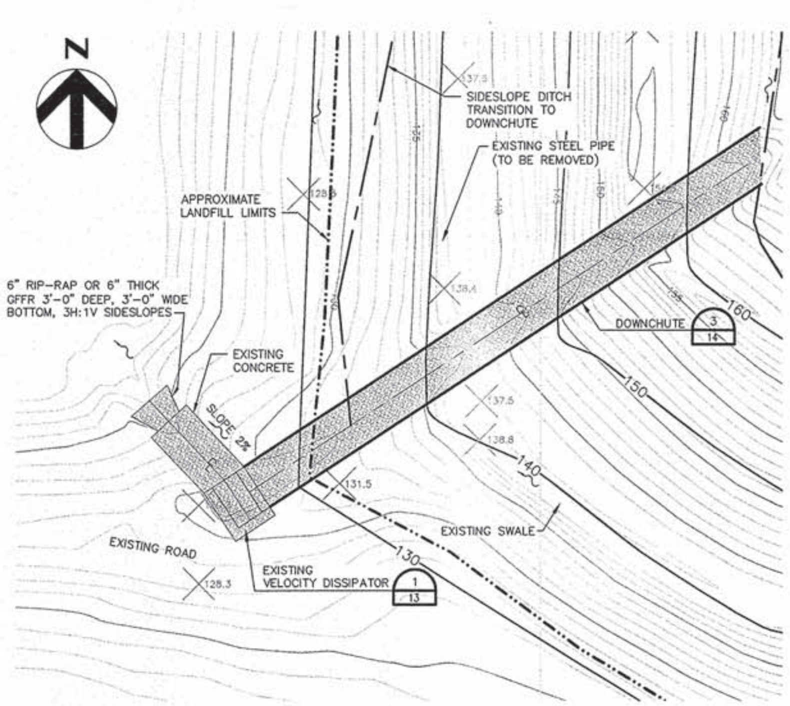
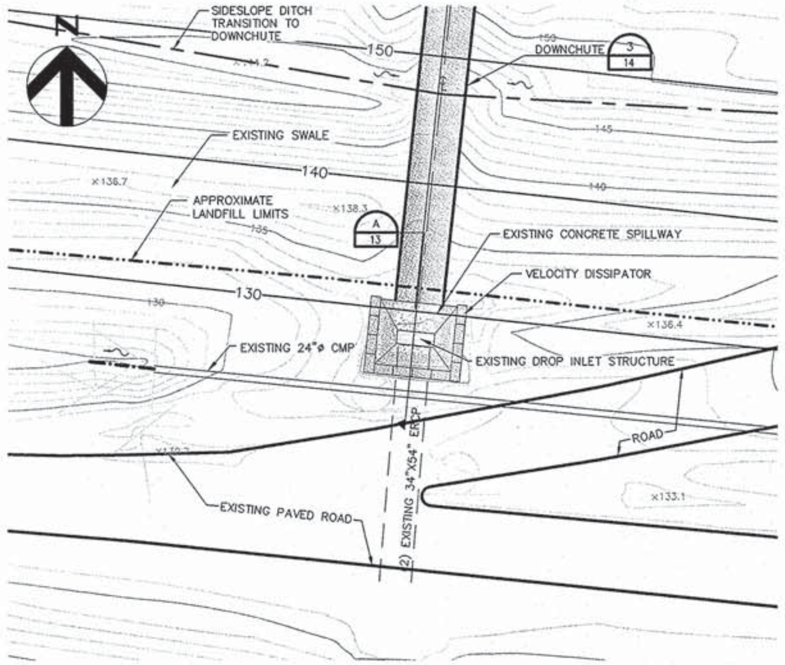
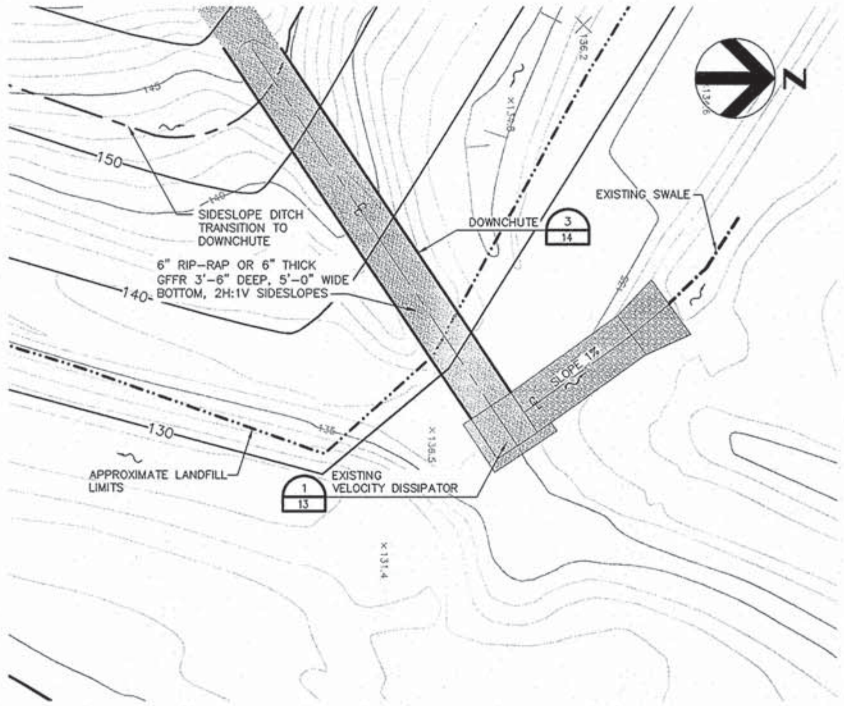


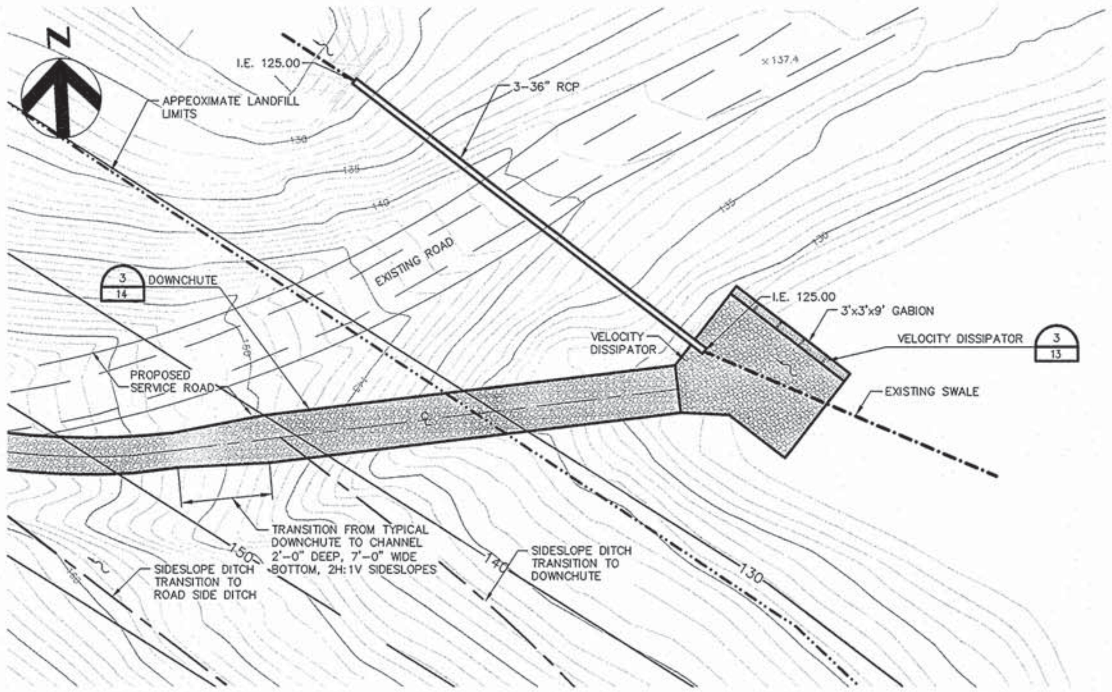
DOWNCHUTE "1" – PLAN DETAIL 1/8



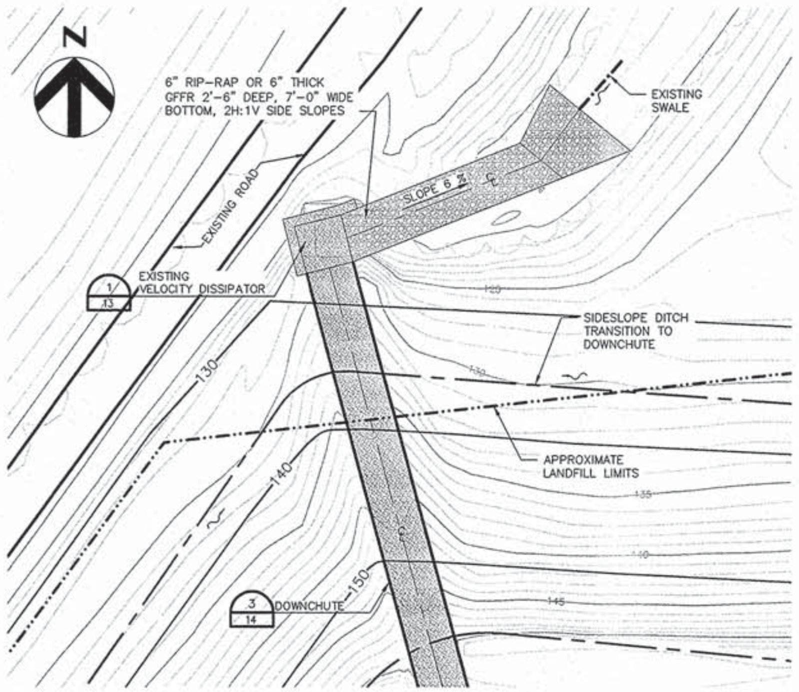
DOWNCHUTE "2" – PLAN DETAIL 2/8



DOWNCHUTE "3" – PLAN DETAIL 3/8



DOWNCHUTE "4" – PLAN DETAIL 4/8



DOWNCHUTE "5" – PLAN DETAIL 5/8

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REVIEWED BY	T. YANOSCHAK
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PROJECT NUMBER	0096–193806–001

No. 73758
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CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 1

0 1" 2"

FILENAME	00C–12.DWG
SCALE	1"=30'

DWG No.
12

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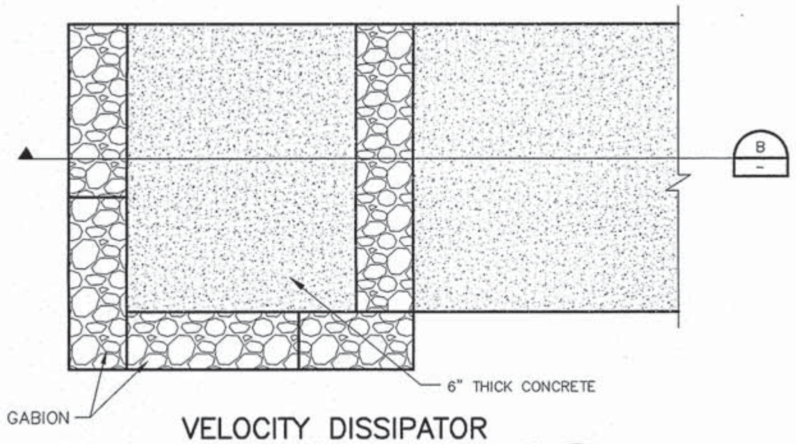
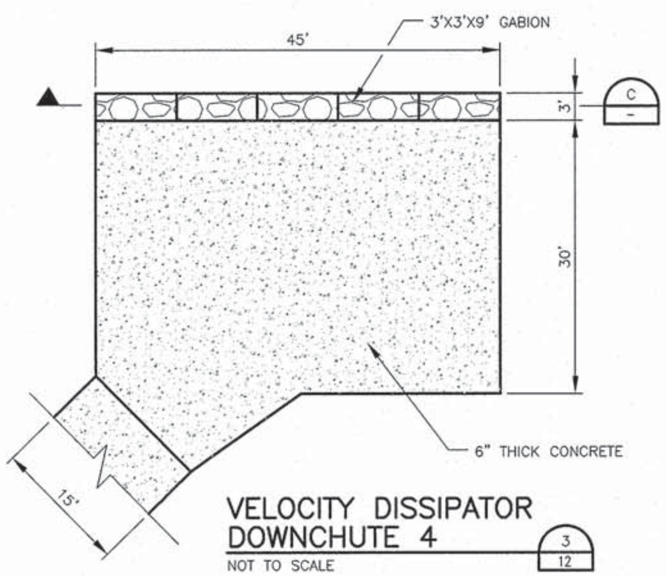


VELOCITY DISSIPATOR DOWNCHUTES 1, 3 & 5

NOT TO SCALE

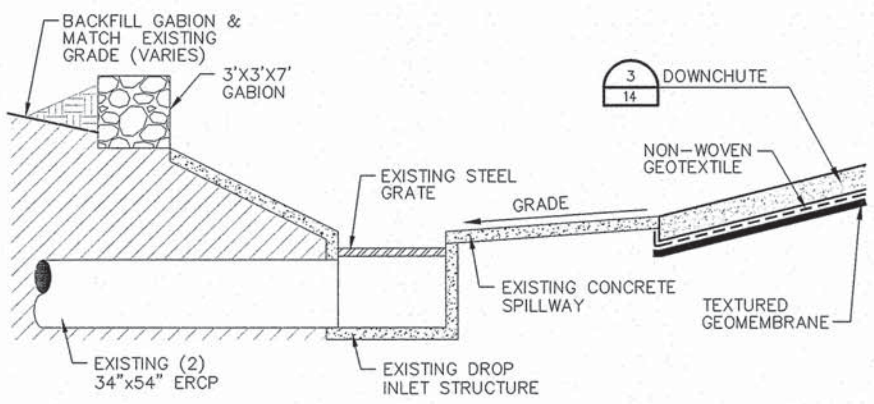
1 / 12



VELOCITY DISSIPATOR DOWNCHUTE 4

NOT TO SCALE

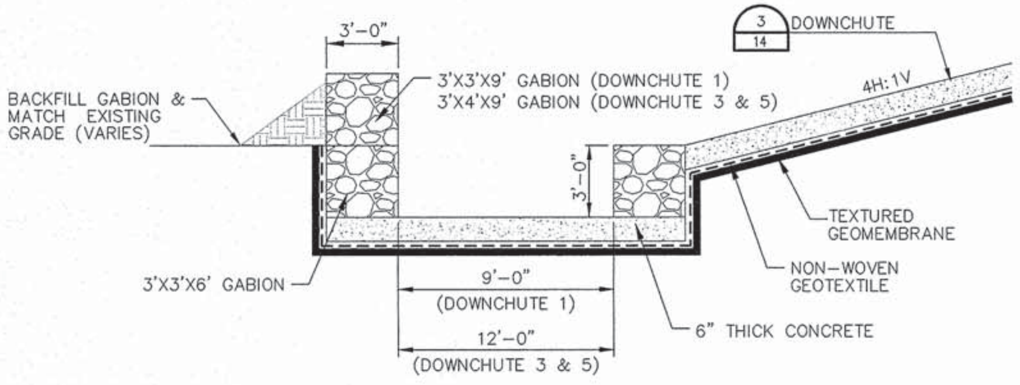
3 / 12



VELOCITY DISSIPATOR SECTION

NOT TO SCALE

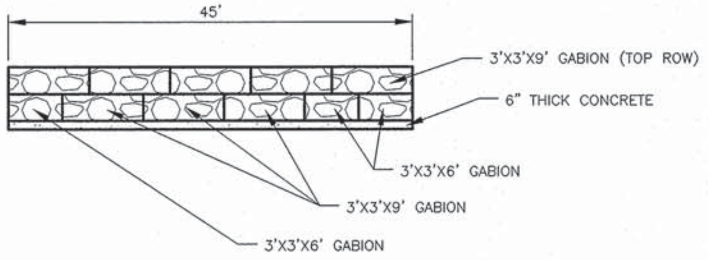
A / 12



SECTION

NOT TO SCALE

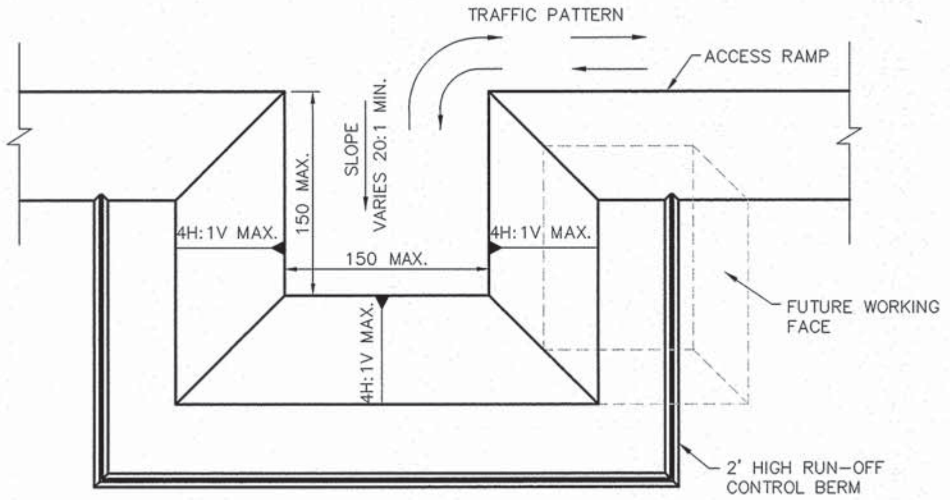
B / –



SECTION

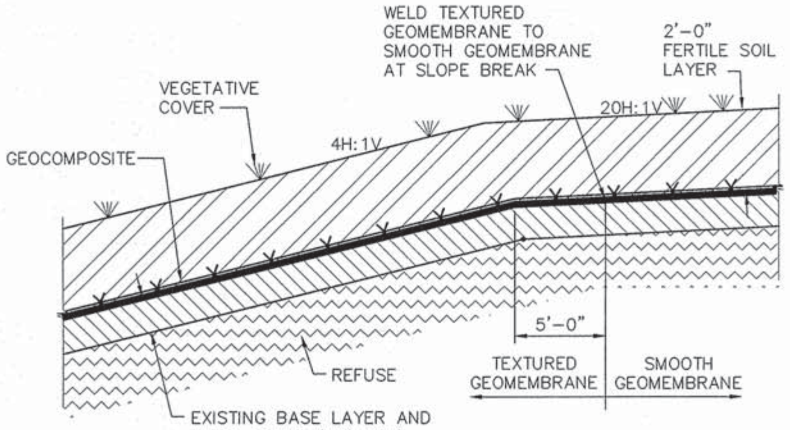
NOT TO SCALE

C / –



TYPICAL WORKING FACE CELL PLAN

NOT TO SCALE

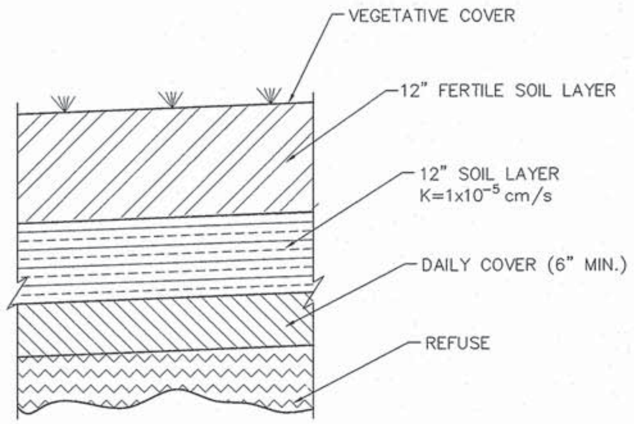


NOTE:
TEXTURED GEOMEMBRANE TO BE INSTALLED ON SLOPES EXCEEDING 20H:1V AND SMOOTH GEOMEMBRANE TO BE INSTALLED ON SLOPES LESS THAN OR EQUAL TO 20H:1V

TYPICAL GEOMEMBRANE FINAL COVER SYSTEM DETAIL

NOT TO SCALE

2 / 8 / 14



TYPICAL TEMPORARY FINAL COVER DETAIL

NOT TO SCALE

4 / 4 / 5

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ROBERT B. CURTIS
No. 73758

6/19/13

ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 2

0 1" 2"

FILENAME	00C-13.DWG	DWG No.
SCALE	AS SHOWN	13

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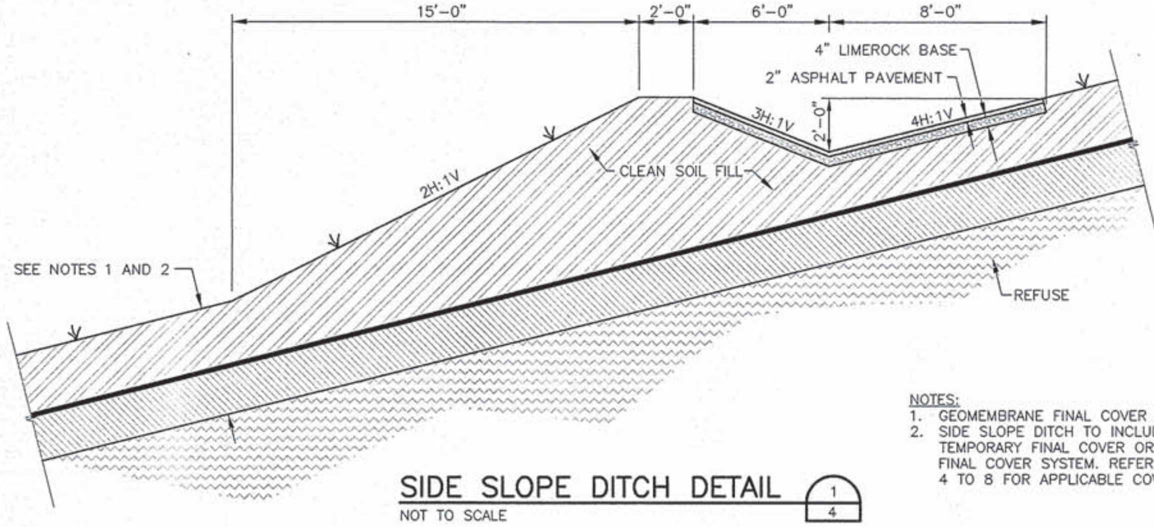
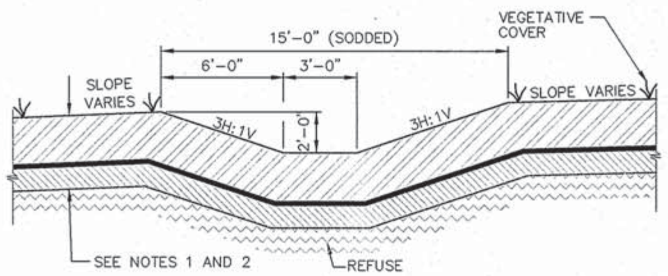


SIDE SLOPE DITCH DETAIL 1/4

NOT TO SCALE

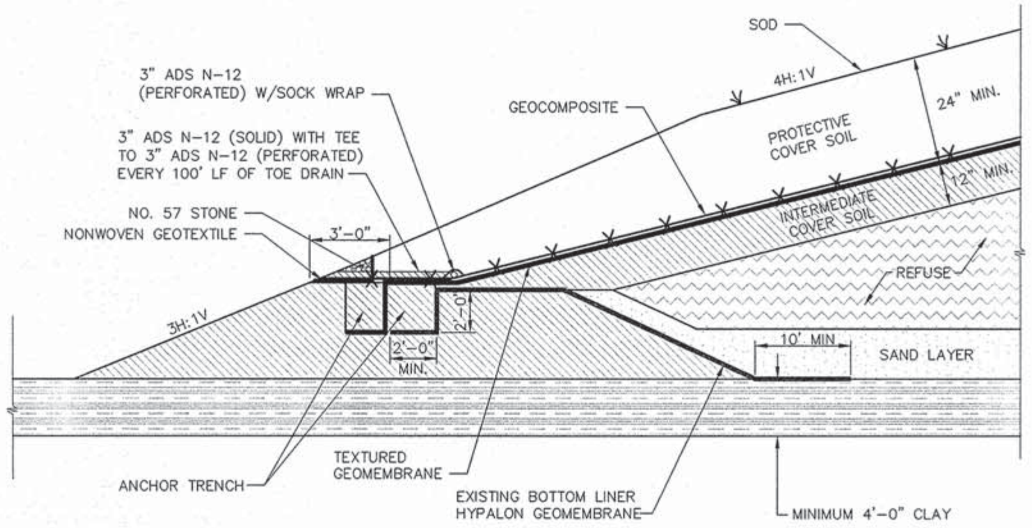
NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. SIDE SLOPE DITCH TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.



TOP SLOPE DITCH DETAIL 4/4

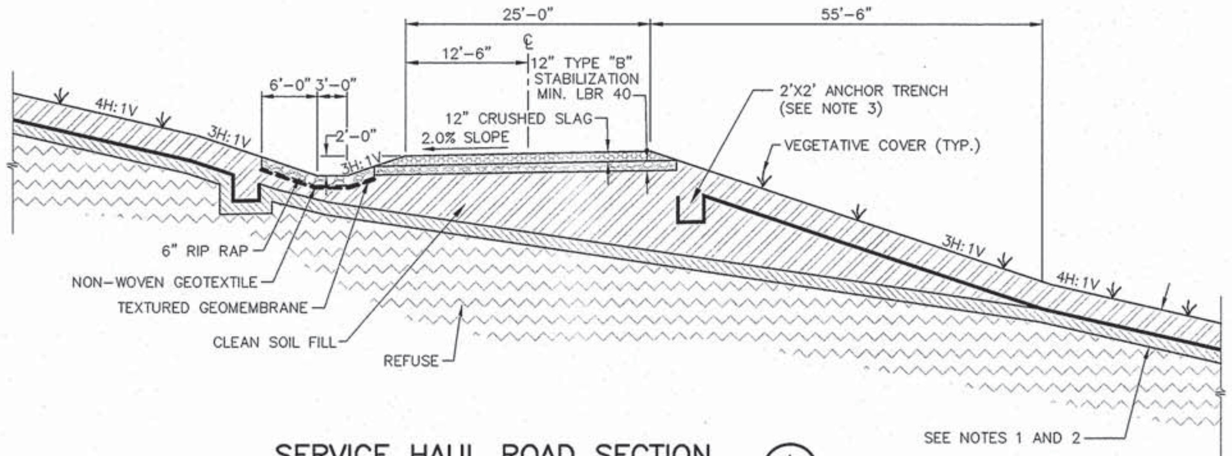
NOT TO SCALE

NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. TOP SLOPE DITCH TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.



FINAL COVER TOE DRAIN DETAIL 2/8

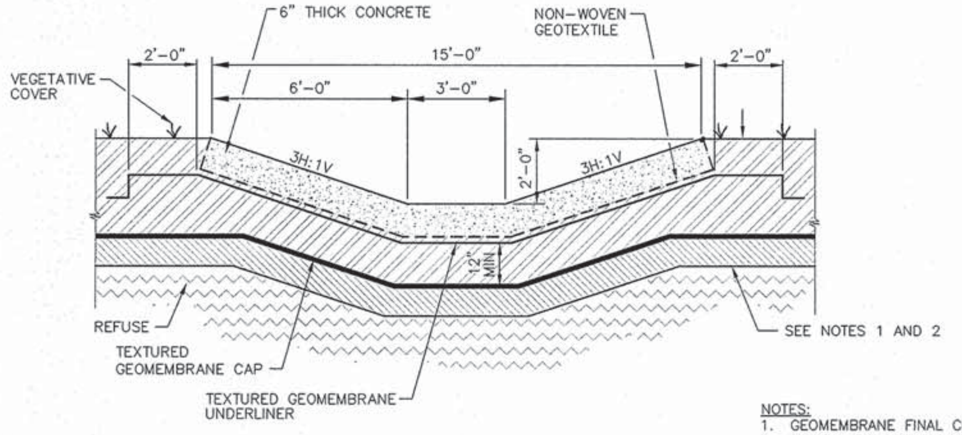
NOT TO SCALE



SERVICE HAUL ROAD SECTION A/8

NOT TO SCALE

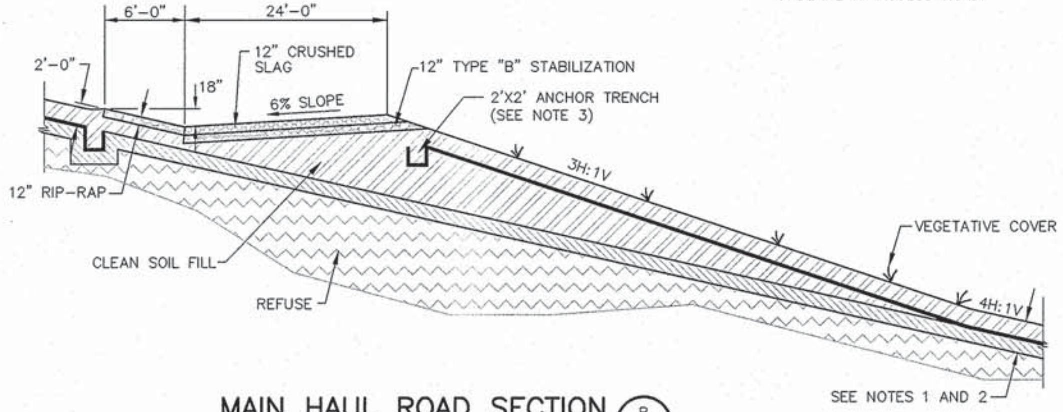
NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. SERVICE HAUL ROAD TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.
3. GEOMEMBRANE LINER NOT INSTALLED UNDERNEATH ACCESS ROAD.



TYPICAL DOWNCHUTE DETAIL 3/12

NOT TO SCALE

NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. TYPICAL DOWNCHUTE TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.



MAIN HAUL ROAD SECTION B/8

NOT TO SCALE

NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. MAIN HAUL ROAD TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.
3. GEOMEMBRANE LINER NOT INSTALLED UNDERNEATH ACCESS ROAD.

Hillsborough County
Florida

HDR

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 3

0 1" 2"

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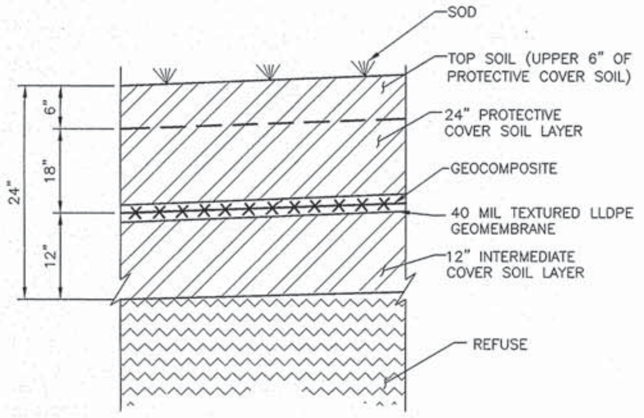
SCALE	AS SHOWN

DWG No. 14



TEMPORARY GEOMEMBRANE FINAL COVER DETAIL

1/9

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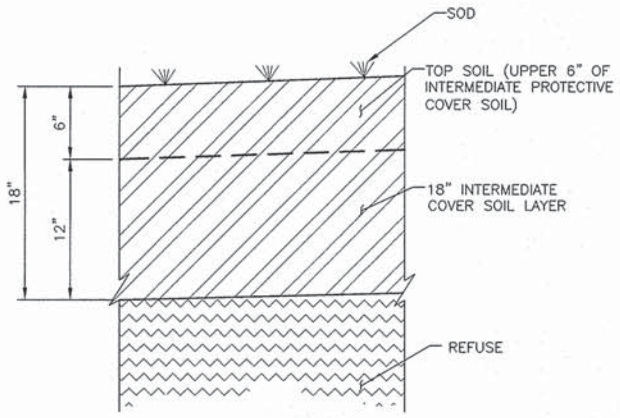

INTERMEDIATE COVER DETAIL

2/9

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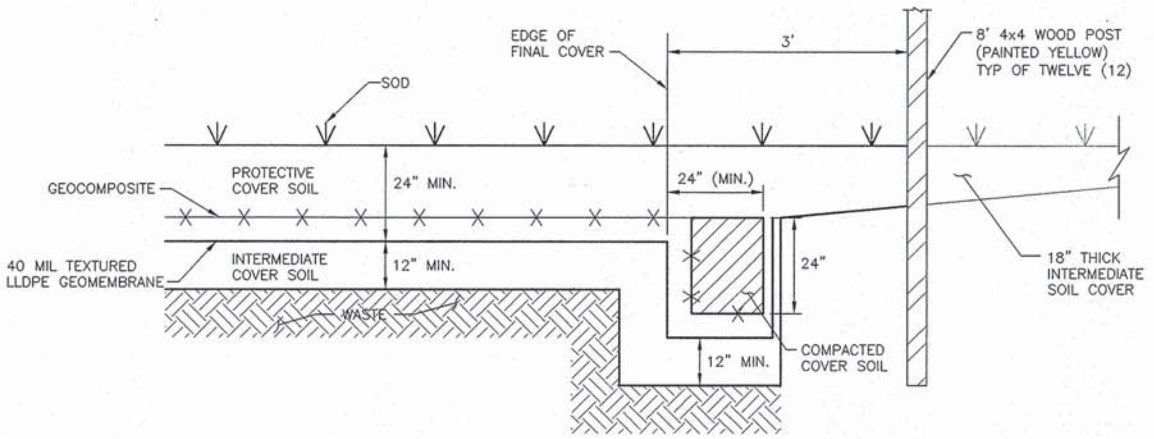
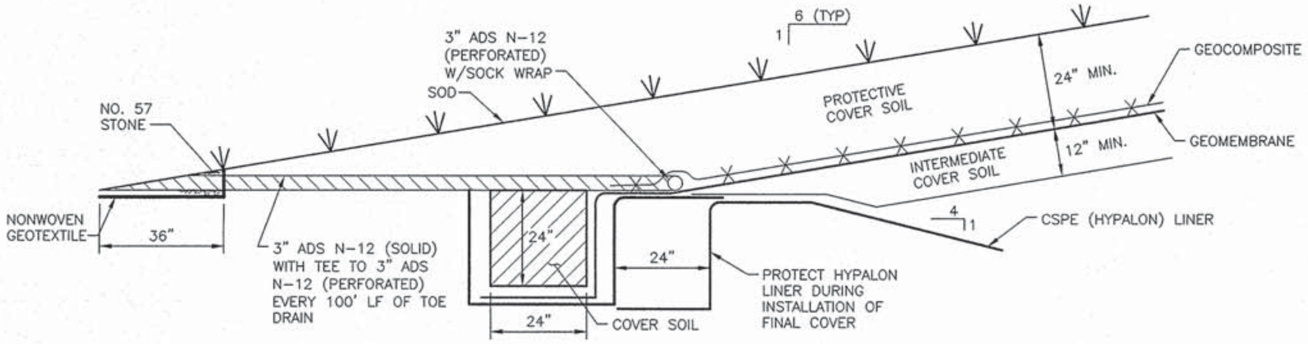


ANCHOR TRENCH DETAIL

3/9

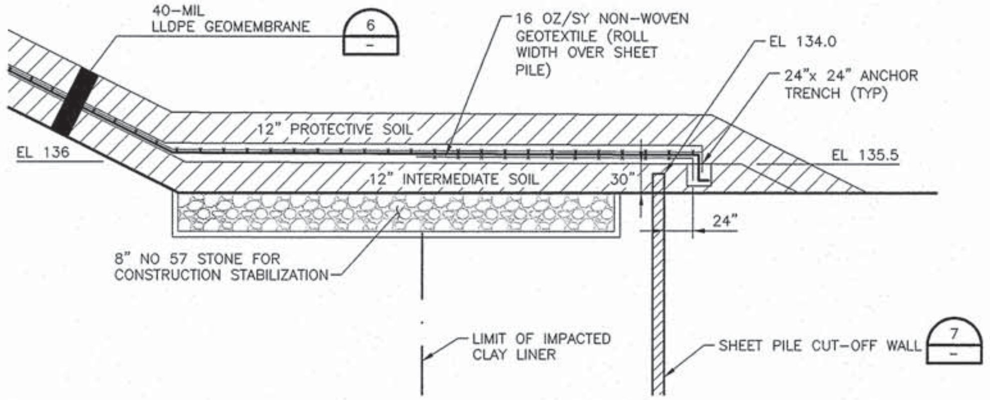
NOT TO SCALE



TEMPORARY TOE DRAIN DETAIL

4/9

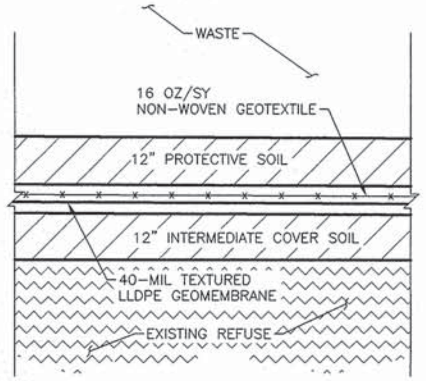
NOT TO SCALE

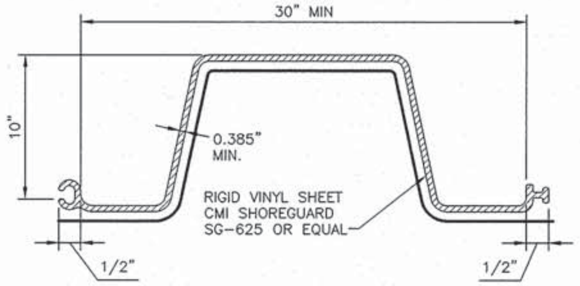


40-MIL LLDPE GEOMEMBRANE ANCHOR DETAIL

5/9

NOT TO SCALE





* STEEL GUIDE METAL THICKNESS SUFFICIENT TO PENETRATE THROUGH MATERIALS SHOWN ON BORINGS.

TYPICAL SHEET PILE CUT-OFF WALL DETAIL

7/–

NOT TO SCALE

40-MIL LLDPE GEOMEMBRANE DETAIL

6/9

NOT TO SCALE

Hillsborough County
Florida

HDR

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5426 Bay Center Drive
Suite 400
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ISSUE	DATE	DESCRIPTION
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No. 73758

6/7/13

ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 4

0 1" 2"

FILENAME 00C-15.dwg
SCALE AS SHOWN

DWG No.
15

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ATTACHMENT 2

NEW SEQUENCE GEOMETRY VENEER STABILITY REPORT

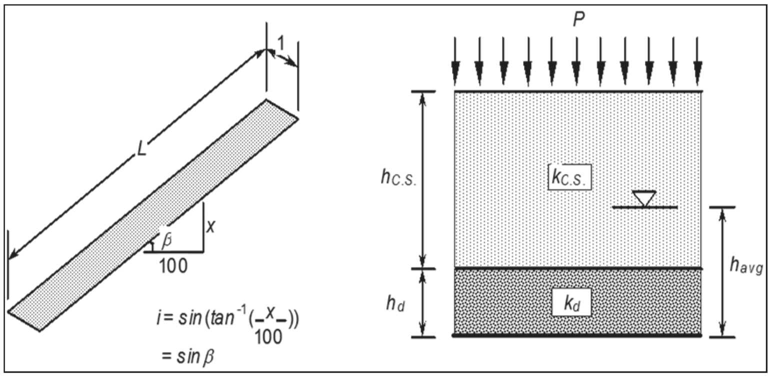
SCS ENGINEERS

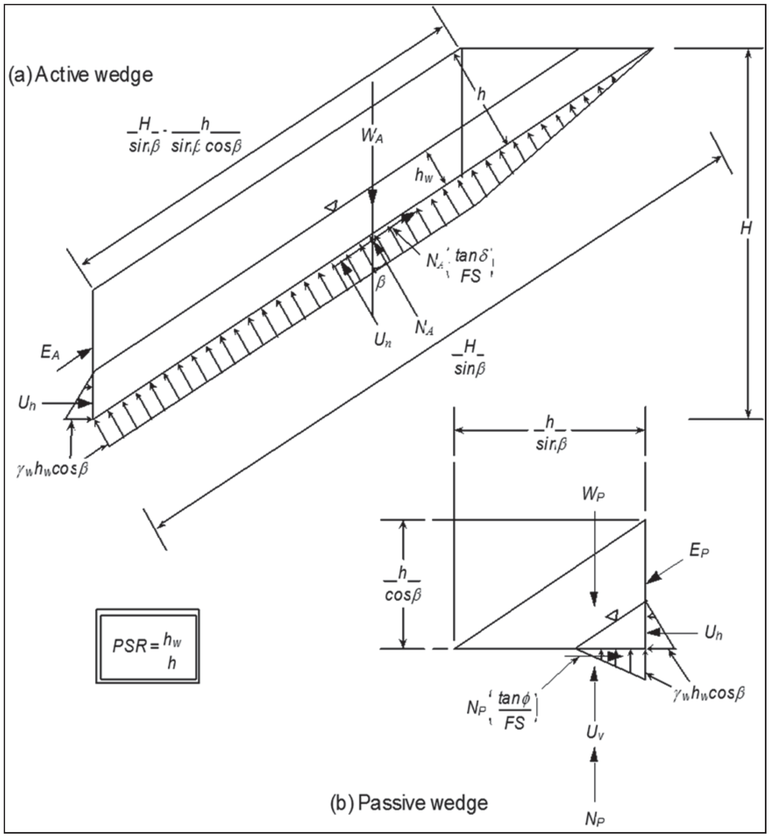
SHEET 1 OF 5

CLIENT	PROJECT	JOB NO.
Hillsborough County	Southeast County Landfill	09215600.03

SUBJECT	BY	DATE
Veneer Stability Calculations	SRF	6/2/2017
	CHECKED	DATE
	KLS	6/6/2017

Consideration: To evaluate the stability of the protective cover soil with seepage forces applied using the method described by Koerner and Soong (1998) referenced below.





Ref.: R.M. Koerner, and T-Y.Soong, 1998. "Analysis and Design of Veneer Cover Soils". Proceeding of 6th International Conference on Geosynthetics, Vol. 1, pp. 1-23, Atlanta, Georgia, USA.

Refer to Attachment 2

SCS ENGINEERS

SHEET 2 OF 5

CLIENT Hillsborough County	PROJECT Southeast County Landfill	JOB NO. 09215600.03
SUBJECT Veneer Stability Calculations	BY SRF	DATE 6/2/2017
	CHECKED KLS	DATE 6/6/2017

Parameters:

DLC = drainage layer capacity
$FLUX_{allow}$ = allowable flow rate of the drainage layer per unit width of slope
k_d = permeability of drainage soil or geosynthetic
h_d = thickness of the drainage soil or geosynthetic
i = $\sin \beta$ = slope gradient
$FLUX_{req'd}$ = actual flow rate per unit width of slope
PERC = the rate of percolation
P = probable maximum (hourly) precipitation (25-year storm event)
RC = runoff coefficient
L = length of drainage slope
k_{cs} = permeability of cover soil
β = slope angle
w = 1.0 m = unit width of drainage slope
PSR = parallel submergence ratio
h_{avg} = average head buildup above the geomembrane
h_{cs} = thickness of cover soil
FS = factor of safety against instability
W_A = total weight of the active wedge
W_P = total weight of the passive wedge
U_h = resultant of the pore pressures acting on the interwedge surfaces
U_n = resultant of the pore pressures acting perpendicular to the slope
U_v = resultant of the vertical pore pressures acting on the passive wedge
N_A = effective force normal to the failure plane of the active wedge
h = thickness of the cover soil
H = vertical height of the slope measured from the toe
h_w = (PSR)(h) = height of the free water surface measured from the geomembrane
γ_{dry} = dry unit weight of the cover soil
$\gamma_{sat'd}$ = saturated unit weight of the cover soil
γ_w = unit weight of water
ϕ = cover soil friction angle
δ = interface friction angle between weakest interface of the final cover system

SCS ENGINEERS

SHEET 3 OF 5

CLIENT	PROJECT	JOB NO.
Hillsborough County	Southeast County Landfill	09215600.03

SUBJECT	BY	DATE
Veneer Stability Calculations	SRF	6/2/2017
	CHECKED KLS	DATE 6/6/2017

Calculate Drainage Layer Capacity (DLC):

$PERC = P(1-RC)$, for $P(1-RC) \leq k_{cs}$
$PERC = k_{cs}$, for $P(1-RC) > k_{cs}$

See Equations 21a, 21b on p. 34 in Attachment 1

k_{CS} = 1.00E-04 cm/s = 3.60 mm/hr — Anticipated value to be specified
P = 0.44 in/hr = 11.18 mm/hr — Refer to Attachment 4 for Rainfall Data
RC = 0.40 — Refer to page 26 of Attachment 1 for RC values, this was taken as the default value for this design

P(1-RC) = 6.71 mm/hr

PERC = 3.60 mm/hr

$$FLUX_{req'd} = \frac{PERC}{1000} \times L(\cos\beta) \times w$$

See Equation 22 on p. 40 in Attachment 1

L = 360.0 feet 109.7 m
β = 14.04 ° = 0.25 rad

L(cosβ) = 106.45 m

"L" represents the slope length between terraces of landfill at 4:1 slope
w = 1.0 = unit width (constant) of drainage slope

$FLUX_{req'd}$ = 0.383 m^3/hr

$FLUX_{allow} = k_d \times i \times h_d$

k_d = 9.30 cm/s = 0.09 m/s
h_d = 300 mil
h_d = 7.62 mm = 0.01 m
i = 0.24

Refer to Attachment 5 for Geocomposite Transmissivity Calculations

$FLUX_{allow}$ = 0.619 m^3/hr

$$DLC = \frac{FLUX_{allow}}{FLUX_{req'd}}$$

See Equation 23 on p. 40 in Attachment 1

DLC = 1.62

Notes:
1) If only one soil layer above geomembrane, treat it as a drainage layer.
2) DLC needs to be greater than one to avoid saturation of the drainage layer.
Therefore, the proposed geocomposite meets drainage capacity requirements.

SCS ENGINEERS

SHEET 4 OF 5

CLIENT	PROJECT	JOB NO.
Hillsborough County	Southeast County Landfill	09215600.03

SUBJECT	BY	DATE
Veneer Stability Calculations	SRF	6/2/2017
	CHECKED	DATE
	KLS	6/6/2017

Calculate Parallel Submergence Ratio (PSR):

$h_{avg} = \frac{FLUX_{req'd}/3600}{k_d \times i}$, for DLC $\geq$ 1.0

See Equation 24 on p. 42 in Attachment 1

$h_{avg} = \frac{[FLUX_{req'd}/(3600 \times i)] - [h_d \times (k_d - k_{cs})]}{k_{cs}}$, for DLC < 1.0

See Equation 26 on p. 42 in Attachment 1

h_{avg} for DLC $\geq$ 1.0 = 0.005 m

h_{avg} for DLC < 1.0 = -269.86 m

h_{avg} = 0.005 m

$PSR = \frac{h_{avg}}{h_{cs} + h_d}$

if PSR $\geq$ 1, set PSR = 1

See Equation 27 on p. 42 in Attachment 1

h_{CS} = 609.60 mm = 0.61 m — Thickness of cover soil (2 ft)

PSR = 0.00764

PSR = 0.00764

Calculate Factor of Safety (FS):

$W_A = \frac{\gamma_{dry} (h - h_w)[2H\cos\beta - (h + h_w)] + \gamma_{sat'd} (h_w)(2H\cos\beta - h_w)}{\sin 2\beta}$

See Equation 32 on p. 12 in Attachment 2

γ_{dry} = 100 lb/ft^3 = 15.71 kN/m^3

$\gamma_{sat'd}$ = 110 lb/ft^3 = 17.28 kN/m^3

$h = h_d + h_{cs}$ = 617.22 mm = 0.62 m

h_w = 4.72 mm = 0.0047 m

$H = L \times \sin\beta$ = 26.62 m

W_A = 1051.99 kN

$U_h = \frac{\gamma_w (h_w)^2}{2}$

See Equation 34 on p. 12 in Attachment 2

γ_w = 9.81 kN/m^3

U_h = 0.000109 kN

$U_n = \frac{\gamma_w (h_w)(\cos\beta)(2H\cos\beta - h_w)}{\sin 2\beta}$

See Equation 33 on p. 12 in Attachment 2

U_n = 4.93 kN

SCS ENGINEERS

SHEET 5 OF 5

CLIENT Hillsborough County	PROJECT Southeast County Landfill	JOB NO. 09215600.03

SUBJECT Veneer Stability Calculations	BY SRF	DATE 6/2/2017
	CHECKED KLS	DATE 6/6/2017

$N_A = W_A(\cos\beta) + U_h(\sin\beta) - U_n$

See Equation 26 on p. 10 in Attachment 2

$N_A =$ 1,015.64 kN

$$W_P = \frac{\gamma_{dry}(h^2 - h_w^2) + \gamma_{sat'd}(h_w)^2}{\sin 2\beta}$$

See Equation 35 on p. 12 in Attachment 2

$W_P =$ 12.71 kN

$U_V = U_h(\cot\beta)$

See Equation 29 on p. 11 in Attachment 2

$U_V =$ 0.000437 kN

$$FS = \frac{-b + (b^2 - 4ac)^{1/2}}{2a}$$

See Equation 15 on p. 5 in Attachment 2

$a = W_A(\sin\beta)(\cos\beta) - U_h(\cos^2\beta) + U_h$

See Equation 31 on p. 11 in Attachment 2 for variables "a", "b", and "c"

a = 247.59

$b = -W_A(\sin^2\beta)(\tan\phi) + U_h(\sin\beta)(\cos\beta)(\tan\phi) - N_A(\cos\beta)(\tan\delta) - (W_P - U_V)(\tan\phi)$

Friction angle $\phi =$ 30.0 ° = 0.52 rad

Shear resistance $\delta =$ 20.0 ° = 0.35 rad

Refer to Attachment 3 for typical protective cover soil material properties (friction angle ϕ) and representative test results (shear resistance δ)

b = -401.71

$c = N_A(\sin\beta)(\tan\delta)(\tan\phi)$

c = 51.78

FS = 1.5

Summary:

DLC	**1.6**
PSR	**0.0076**
δ =	**20.0**
FS	**1.5**

At the minimum interface friction angle indicated in the summary table for all soil-geosynthetic and geosynthetic-geosynthetic interfaces, the calculated factor of safety is (static), indicating that there is adequate shear strength available to prevent the cover soil from sliding. Therefore, the cover soil will be stable under the slope conditions analyzed.

The resulting drainage layer capacity of greater than 1.0, indicating the saturation of the cover soil above the liner would not occur. Therefore the anticipated flow capacity within the drainage layer is sufficient to handle a 25-year 24-hour storm event.

Attachment 2-1

"The Design of Drainage Systems Over Geosynthetically Lined Slopes", T-Y. Soong, 1997

DREXEL
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THE DESIGN OF DRAINAGE SYSTEMS OVER GEOSYNTHETICALLY LINED SLOPES

by

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Research Engineer

and

Robert M. Koerner, Ph.D.
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Drexel University
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Philadelphia, PA 19104

GRI Report #19

JUNE 17, 1997

The Design of Drainage Systems Over Geosynthetically Lined Slopes

Table of Contents

Abstract

Upon investigating eight recent seepage induced slides of leachate collection and final cover systems, it was felt that many designs underestimate the site-specific required flux (lateral flow rate) value. Rather than rely on the HELP model, an hourly-interval procedure for calculating the required flux is presented. It is based on a severe storm event and subsequent water balance analysis over a 6 hour period. The various types of natural and geosynthetic drainage materials are presented and assessed in light of the 25 to 40 times higher required flux-values from such storm events.

The design methodology used to incorporate the site-specific required flux and the material specific allowable flux-values into a slope stability analysis is developed and illustrated. Example problems and a parametric study are presented. Based on the results, the recommendations of the report are as follows:

- The site-specific precipitation rate should be based on a severe storm event basis, particularly for the final covers of landfills.
- Permeability of natural soils and geosynthetic drains must be significantly increased over those currently used in practice.
- Well graded and poorly graded gravels, and possibly sandy gravels, are the obvious choice for natural soils.
- Higher flow rate geosynthetic drains than are currently used, e.g., triaxial geonets and composite sheet drains, are necessary to meet the higher flux requirements.
- The length of slope should probably be limited to 30 m, unless the site is in an arid region. The cumulative effect of long slopes was seen to be a major cause of seepage induced slope instability.
- The drainage outlet at the toe of the slope must have the greatest capacity of any part of the drainage system. Some design scenarios are offered.

Using the method proposed herein, the eight seepage induced slides were back calculated to estimate the site specific precipitation values. They were quite high for leachate collection layers, 14 to 44 mm/hour, except for one with very low permeability soil. For the final cover system slides, the precipitation values were remarkably low, i.e., 0.38 to 1.34 mm/hour. Clearly, the permeability of the drainage layer soil was far too low, i.e., 0.01 cm/sec. Interestingly, this is the regulatory minimum value in federal and many state regulations.

It is hoped that the report stimulates an increased awareness in the possibility of seepage induced slope instability. While instability of the leachate collection layer before waste is placed is often not a critical issue (the slope can often be repaired by on-site personnel), instability of final covers is a serious issue. Such instability could occur many years after closure of a facility, when the expense of repair is a very contentious issue. Such seepage induced instability situations can be avoided by the type of conservative drainage design presented herein.

Acknowledgments

This study was funded through general membership fees of the organizations in the Geosynthetic Institute consortium. We are grateful for their generosity and support. The organizations are as follows; along with the primary contact person within that organization or company. Members of the GSI Board of Directors (BOD) are identified accordingly.

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THE DESIGN OF DRAINAGE SYSTEMS OVER GEOSYNTHETICALLY LINED SLOPES

The previous report in this series, GRI Report #18 dated December 9, 1996, presented numerous *analyses* involving the stability of cover soils overlying geomembrane lined slopes. In so doing, the report highlighted the precarious nature of several situations. For example, equipment loads and seismic forces can be critical, as can be multi-geosynthetic lined slopes. Nowhere, however, was stability more adversely effected than when seepage forces were involved. Paradoxically, this is one situation that can be completely avoided by use of proper drainage materials, either natural drainage soils or geosynthetic drains. Yet, slopes continue to fail due to seepage induced slope instability. This report focuses completely on the issue of proper drainage layer ***design*** and the subsequent analysis of the slope's factor of safety for soils located above geosynthetically lined slopes with the hope that seepage-related slides can be avoided in the future.

1.0 INTRODUCTION

For most geosynthetically lined slope applications like landfill liners and the final covers of closed landfills and waste piles, a geomembrane (*GM*), geosynthetic clay liner (*GCL*), or compacted clay liner (*CCL*) is used as a hydraulic barrier. Furthermore, the liner is directly oriented in the direction of the critical potential sliding plane. While this is unfortunate from a stability perspective, it does allow for a tractable solution of the problem in a relatively straightforward manner. The solution used by numerous researchers is a linear failure plane oriented along the direction of the slope angle, of finite length and of constant thickness e.g., Giroud and Beech (1989), Koerner and Hwu (1991), McKelvey and Deutsch (1991), Thiel and Stewart (1993), Bordeau, et al (1993), Soong and Koerner (1996), and others. In each case, the analysis uses limit equilibrium concepts where the destabilizing actions involved (gravity, live loads, etc.) create driving forces, and the shearing resistance of the materials at the critical interface provides the resisting force. This assumes that the shearing resistance of the critical interface is less than the shearing resistance of the soil itself, which is usually the case with geosynthetically lined slopes. In terms of a factor of safety (*FS*), this concept is expressed as follows:

$$FS = \frac{Resisting\ Force}{Driving\ Forces} \tag{1}$$

When the *FS* is less than 1.0, the slope fails by sliding along the critical interface. When the *FS* is greater than 1.0, stability is suggested with the higher the value, the greater the stability. For temporary slopes, *FS*-values are typically 1.2 to 1.4. For permanent slopes, the *FS*-value should be at least equal to 1.5. Liu, et al (1997) give greater insight in this regard.

A critical issue, and one which has not seen much attention [the exceptions being Thiel and Stewart (1993), Soong and Koerner (1996) and Richardson (1997)] is the negative influence of seepage forces within the drainage layer and/or cover soil above the geosynthetically lined interface. The tacit assumption of most designers appears to be that the cover soil can readily handle the required drainage, or that a drainage layer (often regulatory suggested insofar as thickness and permeability) will be adequate. Unfortunately, neither assumption is accurate and seepage-mobilized slope instability has all too frequently occurred.

This report focuses completely on the issue of the *design of adequate drainage systems* so as to prevent seepage-mobilized slope instability. The report will present background information, water balance analyses, drainage layer considerations (using both natural soils and geosynthetic drainage materials), slope stability analysis, behavior of selected cross-sections, parametric evaluations, related discussion, summary and recommendations.

2.0 BACKGROUND

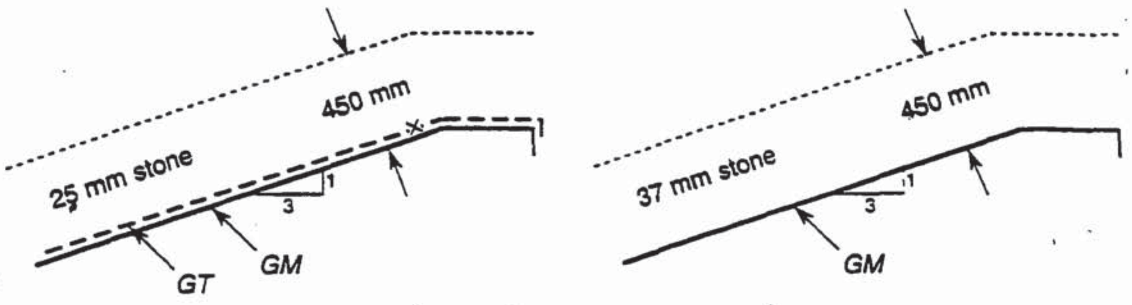
This section of the report describes eight recent seepage induced slides known to the writers. It also presents the possible magnitude of heavy rainstorm events and the idiosyncrasies of various drainage systems.

2.1 Seepage Induced Slides

The occurrence of seepage induced instability was originally daylighted by Boschuk (1991) and actually challenged in a field trial reported by Giroud, et al. (1990). Yet, such incidents still occur and appear to have occurred more frequently in the intervening years. Figure 1 illustrates four case histories of slides occurring in the leachate collection soils above a geomembrane liner before waste was placed in the respective landfills. Figure 2 illustrates an additional four case histories of slides occurring in the drainage and cover soils above barrier layers after waste was placed in the respective landfills, i.e., final cover situations. While all four cases in the latter category involved compacted clay liners, the situations would probably have been similar with geosynthetic liners. A brief description of each slide follows, and then all eight are compared and contrasted in Table 1.

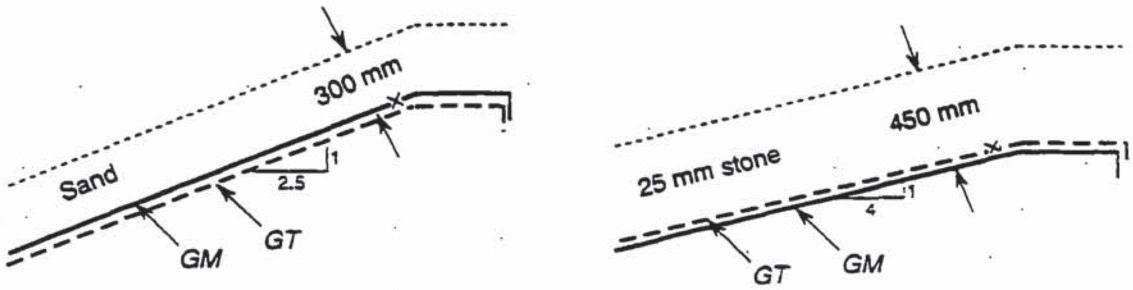
Case #1 occurred in 1992 with a 25 mm average diameter leachate collection stone underlain by a needle punched nonwoven protection geotextile sliding on a stationary smooth HDPE geomembrane. The geotextile failed at the top of the slope carrying it and the stone above into the base of the landfill. The slope was 3(H)-to-1(V) and a number of successive slides occurred during several heavy rainfalls. The stone was AASHTO #57 quarried limestone.

Case #2 occurred in 1993 with a 37 mm average diameter leachate collection stone placed directly on a smooth HDPE geomembrane. The stone slid on the surface of the stationary geomembrane down to the toe of the landfill. The slope was approximately 3(H)-to-1(V) and the slide occurred immediately after a heavy rainfall. The stone was a very coarse AASHTO #3 quarried material.



Case #1 - GT failure

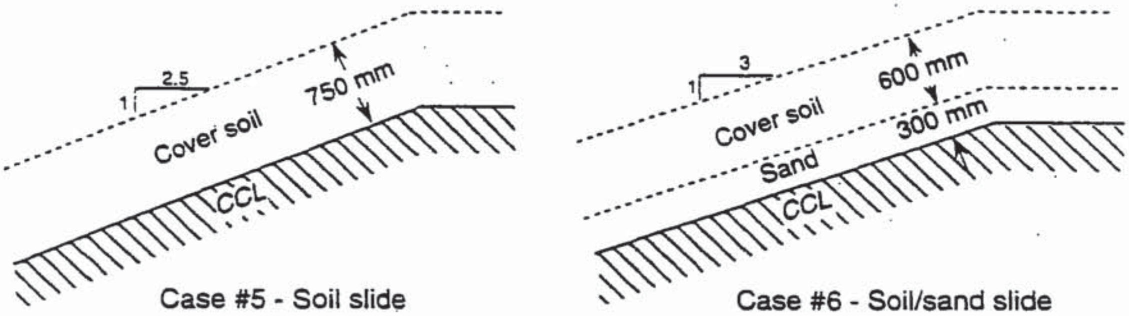
Case #2 - Stone slide



Case #3 - GM failure

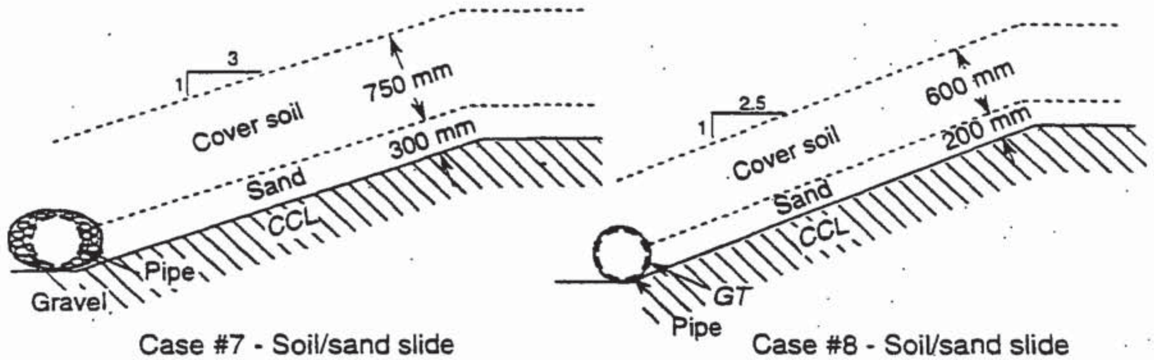
Case #4 - GT failure

Figure 1 - Various seepage involved slides of leachate collection systems in landfill liner systems



Case #5 - Soil slide

Case #6 - Soil/sand slide



Case #7 - Soil/sand slide

Case #8 - Soil/sand slide

Figure 2 - Various seepage involved slides of final cover systems above solid waste landfills

Table 1 - Recent Slope Instability Case Histories Involving Seepage Forces

No.	Upper Interface	Lower Interface	Slope Inclination (Hor. : Vert.)	Cover Soil Thickness, (mm)	Approx. Slope Length, (m)	Approx. Time after Construction, (yr)	Cause of Seepage Force
(a) Slides of leachate collection layers before waste placement							
1	NW-NP-GT	HDPE-GM	3 : 1	450	45	1 - 2	fines in stone
2	Stone	HDPE-GM	3 : 1	450	30	3 - 4	fines in stone
3	VFPE-GM	NW-NP-GT	2.5 : 1	300	20	0.2 - 0.5	low initial permeability
4	NW-NP-GT	PVC-GM	4 : 1	450	90 (3 benches of 30 m each)	1 - 2	ice wedge at toe of slope
(b) Slide of final cover/drainage layers after waste placement							
5	Silty sand	CCL	2.5 : 1	750	40	2 - 3	no drainage layer
6	Sand	CCL	3 : 1	600 + 300	50	5 - 6	low initial sand permeability
7	Sand	CCL	3 : 1	750 + 300	45	5 - 6	fines clogging gravel around pipe
8	Sand	CCL	2.5 : 1	600 + 200	90 (2 benches of 45 m each)	4 - 5	fines clogging GT around pipe

Notes:
GT = Geotextile
GM = Geomembrane
CCL = compacted clay liner
NW-NP = Nonwoven needle punched
HDPE = High density polyethylene
VFPE = Very flexible polyethylene
PVC = Polyvinyl chloride

Case #3 occurred in 1994 with a sand leachate collection material and VFPE geomembrane sliding on a stationary needle punched nonwoven geotextile. The slope was approximately 2.5(H)-to-1(V) and the slide occurred during a relatively light rainfall. The geomembrane failed along the crest of the slope for a distance of approximately 30 m with its upper end remaining in the anchor trench.

Case #4 occurred in 1995 with a 25 mm average diameter quarried leachate collection stone underlain by a needle punched nonwoven protection geotextile sliding on a geomembrane. The difference between it and Case #1 was that the geomembrane was PVC, the slope was 4(H)-to-1(V) and the toe blockage was via a frozen ice wedge with sun-melted seepage forces being mobilized upslope. Approximately 3 ha of geomembrane was exposed after the geotextile and stone slid down to the toe of the landfill.

Case #5 occurred in 1995 with 750 mm of silty sand ($k \approx 0.001$ cm/s) cover soil sliding on a compacted clay liner (*CCL*) during a storm event. The slide was relatively small and localized. The slope was 2.5(H)-to-1(V).

Case #6 occurred in 1996 with 900 mm of sand drainage layer ($k \approx 0.01$ cm/s) and cover soil sliding on a CCL immediately after a storm event. At least four localized slides occurred. The slope was 3(H)-to-1(V).

Case #7 also occurred in 1996 under very similar circumstances to Case #6, except exhuming the gravel around the toe drain showed the gravel to be highly contaminated with fines which migrated through the cover soil and/or sand. A number of localized slides occurred at this site. The slope was 3(H)-to-1(V).

Case #8 also occurred in 1996 under very similar circumstances to Case #7 except the geotextile filter surrounding the prefabricated toe drain pipe was excessively clogged with fines from the cover soil and/or sand. There were a number of small localized slides at this site. This is the so-called "socked pipe" design which is known to be problematic in other situations, e.g., in leachate collection filters beneath the waste mass, Koerner G. R. et al (1993). The slope was 2.5(H)-to-1(V).

2.2 Storm Event Characteristics

In seven of the eight cases of seepage induced slides just described, the occurrence was during, or immediately after, rain storm events. Unfortunately, the exact storm magnitudes were not recorded. It is assumed, however, that localized short-term seepage forces created enough of an additional driving force to decrease the FS-value to less than 1.0 and thereby result in the slope's instability. The other case, Case #4, of an ice wedge at the toe of the slope and seepage forces due to thawing at the top of the slope is certainly a plausible situation depending on site specific climatic conditions. However, this case is somewhat unique and is somewhat outside of the main thrust of this report. Clearly its teaching, however, is that toe blockage of any type must be avoided in order to have a free up-gradient drainage system without mobilizing seepage forces.

It should be obvious that rain storms are not well-behaved, uniform events. Figure 3 illustrates just how random a short-term storm event can be. The peaks occur over extremely short time periods, i.e., minutes, and can reach dramatic rates. In light of this behavior, a slope will undoubtedly be most susceptible during periods of high rainfall and particularly during or immediately after the highest rainfall rate. In this regard, a seepage-related slope stability analyses should be analyzed as a severe storm event and the drainage system designed accordingly. This is not unlike all types of engineering design when considering live load circumstances, e.g., snow loads, seismic loads, equipment loads, etc.

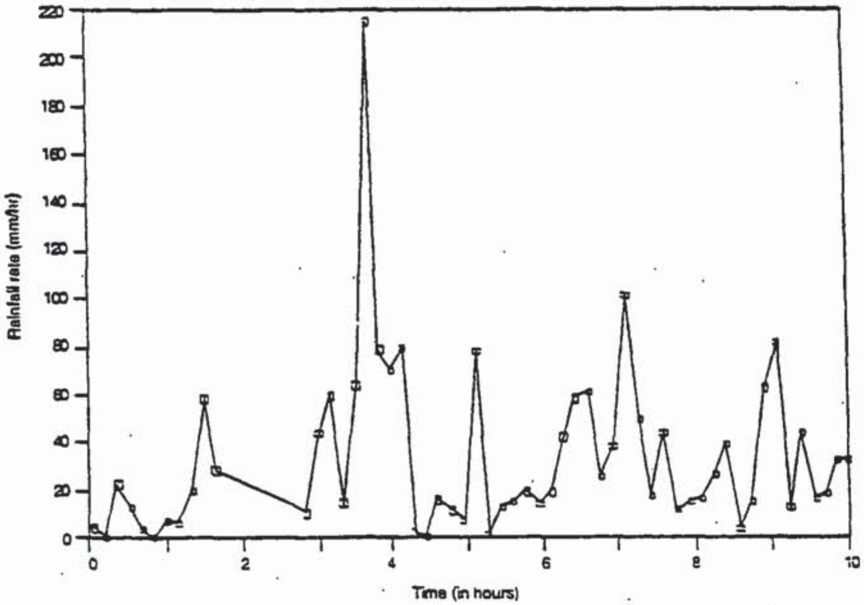


Figure 3 - Precipitation time-rate data for an extreme storm in Oklahoma on May 27, 1987, as measured by the National Storm Service Laboratory. Values are for a 2- by 2-km area, after Maidment (1993).

Ideally, one would like to select a design storm for which there is no risk of exceedance. This concept, however, is most troublesome and hydrologists even argue about the existence of an upper limit. More practical, and accepted in the design of spillways for dams, is the concept of the probable maximum precipitation (*PMP*). This term is defined by the World Meteorological Organization as:

> *"theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year."*

Four critical issues are related to the above definition: storm duration, storm intensity, orientation (slope) effects and infiltration into the cover soil. For the first two issues, Table 2 is available for the selected cases in the United States. It is seen that extremely high rates can occur over small, localized areas. For the second two issues, one must proceed on the basis of site specific material properties and an appropriate water balance analysis.

Table 2 - Maximum observed rainfall amount,.area and duration data for selected locations in the United States

[Table values are for average rainfall in millimeters, after the World Meteorological Organization (1986).]

	Duration, hour						
Area	6	12	18	24	36	48	72
26 km^2	627^{a}	757^{b}	922^{e}	983^{e}	1062^{e}	1095^{e}	1148^{e}
260 km^2	498^{b}	668^{e}	826^{e}	894^{e}	963^{e}	988^{e}	1031^{e}
520 km^2	455^{b}	650^{e}	798^{e}	869^{e}	932^{e}	958^{e}	996^{e}
1300 km^2	391^{b}	625^{e}	754^{e}	831^{e}	889^{e}	914^{e}	947^{e}
2600 km^2	340^{b}	574^{e}	696^{e}	767^{e}	836^{e}	856^{e}	886^{e}
5200 km^2	284^{b}	450^{e}	572^{e}	630^{e}	693^{e}	721^{e}	754^{e}
13000 km^2	206^{h}	282^{b}	358^{b}	394^{e}	475^{i}	526^{i}	620^{i}
26000 km^2	145^{h}	201^{j}	257^{k}	307^{k}	384^{i}	442^{i}	541^{i}
52000 km^2	102^{h}	152^{j}	201^{k}	244^{k}	295^{i}	351^{i}	447^{i}
130000 km^2	64^{m}	107^{n}	135^{k}	160^{k}	201^{k}	251^{r}	335^{r}
260000 km^2	43^{m}	64^{m}	89^{k}	109^{k}	152^{p}	170^{p}	226^{q}

Storm	Date		Location of Center		Remark
a	July 17-18	1942	Smethport	PA	
b	Sept. 8-10	1921	Thrall	TX	
e	Sept. 3-7	1950	Yankeetown	FL	Hurricane
i	June 27-July 1	1899	Hearne	TX	
k	Mar. 13-15	1929	Elba	AL	
q	July 5-10	1916	Bonifay	FL	Hurricane
n	Apr. 15-18	1900	Eutaw	AL	
m	May 22-26	1908	Chattanooga	OK	
o	Nov. 19-22	1934	Millry	AL	
h	June 27-July 4	1936	Bebe	TX	
j	Apr. 12-16	1927	Jefferson Parish	LA	
r	Sept. 19-24	1967	Cibolo Ck.	TX	Hurricane
p	Sept. 29-Oct. 3	1929	Vernon	FL	Hurricane

For the cases of sliding of cover soils as described previously, it appears to the authors that a 6-hour duration storm event falls acceptably close to the concept of a *PMP* event, i.e., a 6-hour duration storm can be considered as a severe storm event and, arguably, a worst-case event. Local weather conditions would prevail and the nearest meteorological station would be the logical source of the hour-by-hour precipitation data. As far as the infiltration into the cover soil calculated via a water balance analysis, one is immediately drawn to the use of the U.S. EPA computer model entitled Hydrologic Evaluation of Landfill Performance (*HELP*). Clearly, the methodology of this model is beyond reproach. At issue, however, is the periodicity of monitoring the infiltration (hence drainage) quantity and some of the assumptions generally used by designers. The *HELP*-model proceeds on the basis of a daily monitoring of precipitation. As will be seen, this significantly underestimates the drainage quantities which must be efficiently removed in the site specific cross-section on the basis of hourly monitoring. Monthly, daily and hourly monitoring examples will be illustrated later in this report so as to illustrate the significance of this issue.

2.3 Types of Drainage Systems

The traditional material used for the drainage of liquids has been naturally occurring granular soils, e.g., sands and gravels. Beginning in the mid-1980's, geosynthetic drainage materials emerged. First geonets and later different types of drainage geocomposites. Each type, under the collective name "geosynthetic drains", will be described in this section.

2.3.1 Natural Soils

The drainage capacity of natural soils is usually analyzed using Darcy's formula:

$$q = kiA \qquad (2)$$

where q = flow rate (through or within the soil),

k = coefficient of permeability (the term used herein but more properly, the hydraulic conductivity),

i = hydraulic gradient, and

A = cross sectional area perpendicular to flow.

Critical in the above formulation is the value of "k" for which many relationships exist. Formulas range from the empirical Hazen relationship;

$$k(\text{cm}/\text{sec}) = Cd_{10}^2 \tag{3}$$

where C = constant ranging from 0.4 to 1.2,

d_{10} = 10% finer particle size (mm).

to the more complex Kozeny-Carman equation:

$$k = \frac{1}{k_0 T^2 S_0^2}\left(\frac{e^3}{1+e}\right)\left(\frac{\gamma_p}{\mu}\right) \tag{4}$$

where k_0 = slope factor (≈2.5),

T = tortuosity (factor (≈1.4),

S_0 = wetted surface per unit volume of particles,

e = void ratio,

γ_p = unit weight of the permeating liquid,

μ = viscosity of the permeating liquid.

All formulas of this type indicate that particle size and gradation play the major role insofar as drainage of granular soils is concerned. Typical values of permeability for granular soils are provided in Table 3.

Table 3 - Typical values of permeability for granular soils.

Type of Soil	USCS* Classification	Range of "k"-values (cm/sec)
clean, poorly graded gravel	GP	5 - 20
clean, well graded gravel	GW	1 - 10
clean, poorly graded sand	SP	0.5 - 5
clean, well graded sand	SW	0.2 - 2
mixed, poorly graded sandy gravel	SP - GP	0.1 - 2
mixed, well graded sandy gravel	SW - GW	0.01 - 0.5
mixed, poorly graded gravely sand	GP - SP	0.005 - 0.05
mixed, well graded gravely sand	GW - SW	0.001 - 0.01
silty gravels	ML-GP, ML-GW,	0.0005 - 0.01
silty sands	ML-SP or ML-SW	0.0001 - 0.005

* Unified Soil Classification System

Of course, the use of estimated or typical values as presented in Table 3 is for illustrative purposes only and should never be used for final design. Testing by ASTM D2434 is necessary in this regard. Upon obtaining the value of "*k*" for the candidate drainage soil, it must be compared to the site-specific required value to arrive at a factor of safety. Alternatively, "k" can be used to calculate a flow rate, *q*, and used in a similar manner, for example:

$$FS = \frac{k_{allow}}{k_{req'd}} \tag{5}$$

or,

$$FS = \frac{q_{allow}}{q_{req'd}} \tag{6}$$

where FS = factor of safety,

k_{allow} = allowable permeability,

q_{allow} = allowable flow rate (using Darcy's formula),

$k_{req'd}$ = required permeability, and

$q_{req'd}$ = required flow rate (using Darcy's formula).

Depending on the drainage soil that is being used, a filter may also be necessary, e.g., when using *GP* or *GW* gravel in the final cover above the barrier layer, and perhaps with other coarse granular soils as well. Insofar as soil filters are concerned, the material will typically be a well-graded sand with particle sizes intermediate between the overlying protection or cover soil, and the underlying drainage soil. The following filtration criteria for sand filters are from the U.S. Army Corps of Engineers (1948).

To prevent piping:

$$\frac{d_{15}(\text{filter})}{d_{85}(\text{cover soil})} < 4 \text{ to } 5, \text{ and}$$

$$\frac{d_{15}(\text{drainage soil})}{d_{85}(\text{filter})} < 4 \text{ to } 5 \qquad (7)$$

To maintain permeability:

$$\frac{d_{15}(\text{filter})}{d_{15}(\text{cover soil})} > 4 \text{ to } 5, \text{ and}$$

$$\frac{d_{15}(\text{drainage soil})}{d_{15}(\text{filter})} > 4 \text{ to } 5 \qquad (8)$$

The d_{85}-values refer to the size of particle at which 85% by dry weight of the particles are smaller. Similarly, d_{15} refers to the size of particle below which 15% by dry weight is smaller.

2.3.2 Geosynthetics

Geosynthetic drains are always composites in that the drainage core transmitting the flow must be protected by a geotextile which acts as both a filter and a separator with respect to the overlying soil. There are many types of drainage cores that are available:

- Biaxial extruded geonets
- Triaxial extruded geonets
- Stiff 3-D entangled webs
- Vacuum formed cuspated sheets
- Extruded columns or nubbed sheets

The design of a geonet, or other type of drainage core is straightforward. It results in the quantification of a flow rate factor of safety as follows:

$$FS = \frac{q_{allow}}{q_{req'd}} \qquad (9)$$

where FS = factor of safety,

q_{allow} = allowable flow rate as obtained from laboratory testing, and

$q_{req'd}$ = required flow rate as obtained from design requirements of the actual system.

The allowable flow rate comes from in-plane (transmissivity) laboratory testing of the geosynthetic drainage product under consideration. Options in this regard are ASTM D4716 and ISO/DIS 12958. The test setup must simulate the actual field system as closely as possible. If it does not model the field system accurately, then adjustments to the laboratory value must be made. This is generally the case. Thus, the laboratory generated flow rate is often an ultimate (or index) value which must be reduced before use in design; that is,

$$q_{allow} < q_{ult} \tag{10}$$

One way of doing this is to ascribe reduction factors[*] on each of the items not simulated in the laboratory test. This can be accommodated as follows:

$$q_{allow} = q_{ult}\left[\frac{1}{RF_{IN} \times RF_{CR} \times RF_{CC} \times RF_{BC}}\right] \tag{11}$$

Alternatively, if all of the reduction factors are grouped together:

$$q_{allow} = q_{ult}\left[\frac{1}{\Pi RF}\right] \tag{12}$$

where q_{allow} = allowable flow rate to be used for final design purposes,

q_{ult} = flow rate determined from a short-term transmissivity test between solid plates, e.g., see the index data of Figure 4 which was generated according to ASTM D4716,

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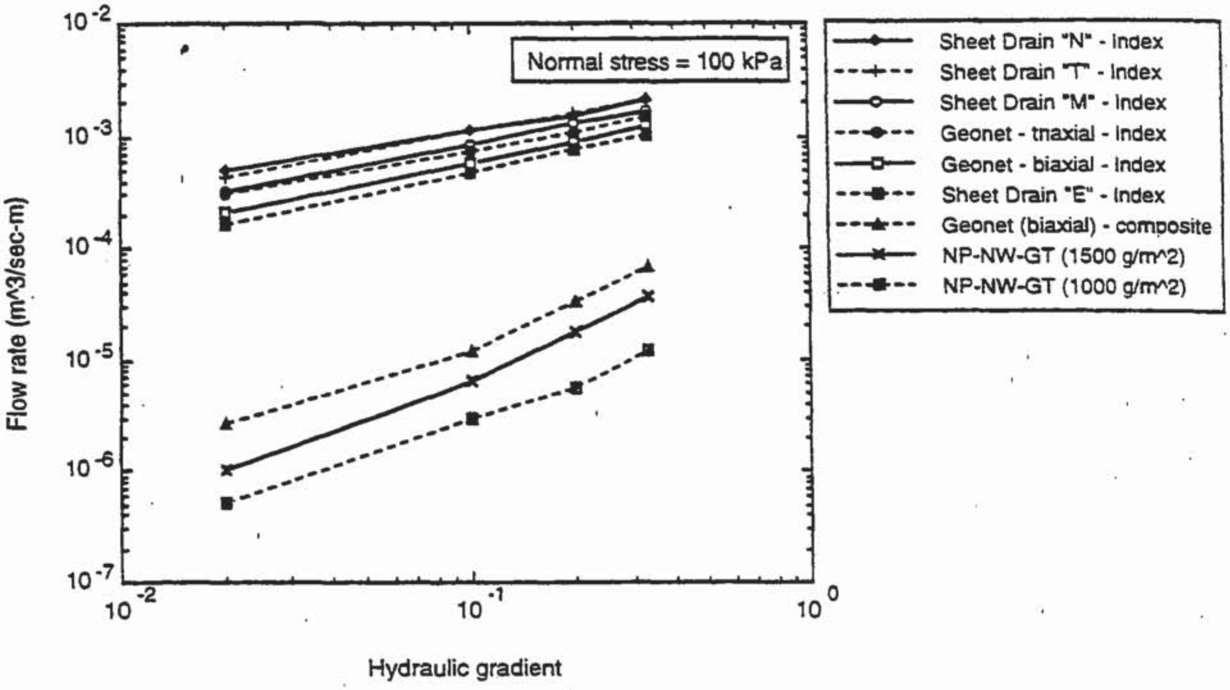
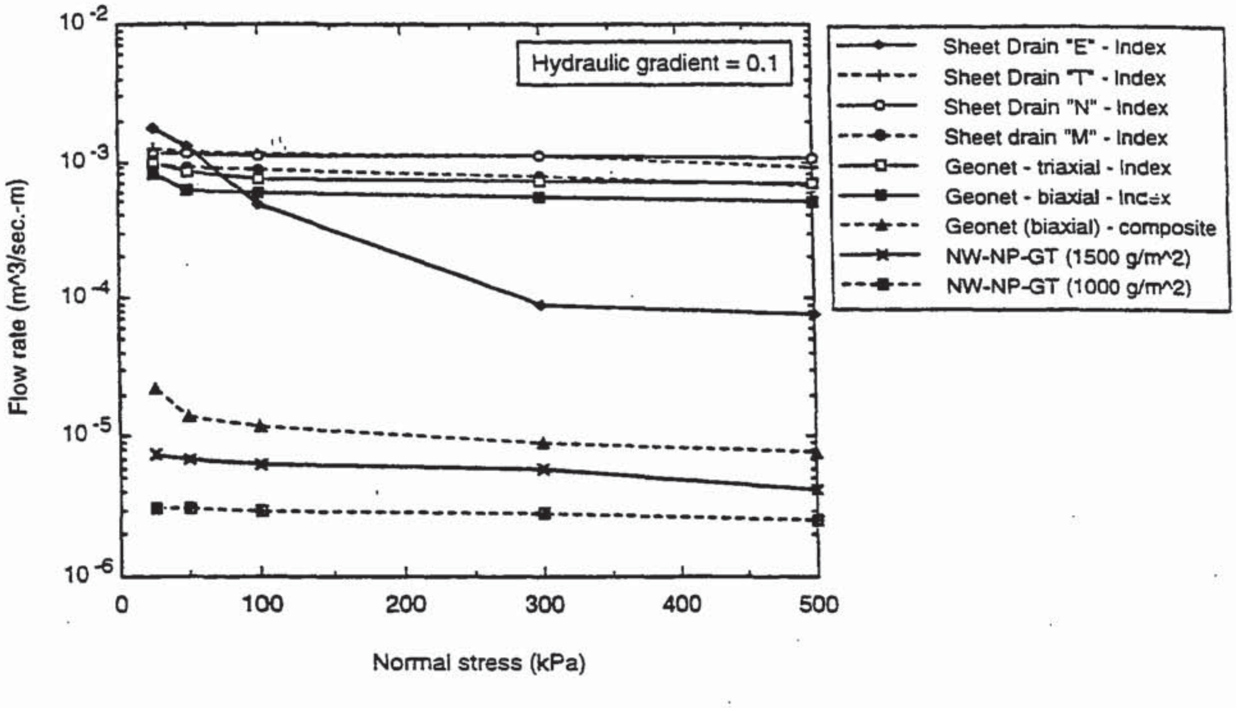
[*] The term "reduction factor" is synonymous with the term "partial factor of safety" which has been used in past literature. This newer definition leaves the traditional term "factor-of-safety" to be uniquely associated with uncertainties in the design process.



(a) Variation of hydraulic gradient with normal stress constant



(b) Variation of normal stress with hydraulic gradient constant

Figure 4 - Flow rate behavior of various geosynthetic drainage materials and composites compared to the drainage capability of geotextiles and geonets.

RF_{IN} = reduction factor for elastic deformation, or intrusion, of the adjacent geotextile into the drainage core space,

RF_{CR} = reduction factor for creep deformation of the drainage core and/or adjacent geotextile into the drainage core space,

RF_{CC} = reduction factor for chemical clogging and/or precipitation of chemicals in the drainage core space,

RF_{BC} = reduction factor for biological clogging in the drainage core space, and

ΠRF = product of all relevant reduction factors for the site specific conditions.

Additional reduction factors, such as core overlap flow restriction, temperature effects and liquid turbidity, might also be considered. If needed, they can be included on a site-specific basis. On the other hand, if the test has included the particular item, the reduction factor would appear in the foregoing formulation as a value of unity. Details of the design and guidelines for the various reduction factors are given in Koerner (1997).

As noted previously, a geotextile must cover the geonet or drainage core and its primary function will be to serve as a filter. In so doing, the geotextile must allow the liquid to pass without mobilizing upstream pore water pressure and, simultaneously, must retain the upstream soil so that up-gradient piping and down-gradient clogging of the geonet or drainage core do not occur. Thus the design is a two-step process; first, openness for permeability (or permittivity) and second, tightness for soil retention (via the geotextile's apparent opening size).

Geotextile permeability is the first part of a geotextile filter design. A factor of safety is formulated using permittivity, which is the permeability divided by the geotextile's thickness, as follows:

$$FS = \frac{\psi_{allow}}{\psi_{req'd}} \tag{13}$$

$$\psi = \frac{k_n}{t} \tag{14}$$

where

ψ = permittivity

k_n = cross-plane permeability coefficient, and

t = thickness at a specified normal pressure.

The testing for geotextile permittivity follows similar lines as used for testing soil permeability. The method is standardized as ASTM D4491 and ISO/DIS 11058. Alternatively, some designers prefer to work directly with permeability and require the geotextile's permeability to be some multiple of the adjacent soil's permeability (e.g., 1.0 to 10.0, or higher).

The second part of a geotextile's filter design is focused on adequate upstream soil retention. There are many approaches toward a soil retention design, most of which use some characteristic of the upstream soil particle size and then compares it to the 95% opening size of the geotextile (i.e., defined as O_{95} of the geotextile). The test method used in the United States to determine this value is called the apparent opening size (*AOS*) test, designated as ASTM D4751. "*AOS*" is defined as the approximate largest soil particle that would effectively pass through the geotextile. In Canada and Europe, the test method is called filtration opening size (FOS) and is accomplished by hydrodynamic sieving. One variation is designated as ISO/DIS 12956. Wet sieving is felt by the writers to be the preferred method.

The simplest of the design methods examines the percentage of soil passing the No. 200 sieve, which has openings of 0.074 mm.

1. For soil with ≤ 50% passing the No. 200 sieve: $O_{95} < 0.59$ mm (i.e., *AOS* of the fabric ≥ No. 30 sieve)
2. For soil with > 50% passing the No. 200 sieve: $O_{95} < 0.30$ mm (i.e., *AOS* of the fabric ≥ No. 50 sieve)

Alternatively, a series of direct comparisons of geotextile opening size (O_{95}, O_{50}, or O_{15}) can be made to a specific soil particle size to be retained (d_{90}, d_{85}, d_{50}, or d_{15}). The numeric value depends on the geotextile type, soil type, flow regime, etc. For example, Carroll (1983) recommends the following widely used relationship.

$$O_{95} < (2\ or\ 3)d_{85} \tag{15}$$

where O_{95} = the 95% opening size of the geotextile (in mm), and

d_{85} = soil particle size (in mm) for which 85% of the soil particle is finer.

More detailed procedures, for both static and dynamic flow are available, see Luettich, et al. (1992). Details of the design and example problems are given in Koerner (1997).

2.3.3 Long-Term Effects

All too often when designing natural soil or geosynthetic drainage systems the focus is on the as-received materials. While this may be appropriate for temporary slopes, it is not appropriate for permanent situations like the drainage layer of final covers above closed landfills.

The overriding long-term effect on drainage systems is the potential for fine particle migration and contamination of the drainage and/or filter materials. As seen in the case histories presented in Table 1, seepage induced slides have occurred in gravel soils having 25 to 38 mm average particle sizes. While these coarse drainage gravels may have appeared initially acceptable, it must be remembered that quarried stone always contains fines and furthermore with the weaker mineral types, e.g., limestone, many fracture surfaces exist to generate even more fines. Furthermore, the filter (if one is present) may allow fines from overlying soils to pass into the underlying drain. Over time and successive rain events, fines from various sources migrate down through the thickness of the drainage layer and can then further migrate downgradient. Obviously, the permeability of the stone (which always appears clean and porous on its surface) decreases over time. The potential clogging mechanisms can be modeled in the laboratory, but to the writers' knowledge long-term drainage tests of soils are rarely conducted and have never (?) been reported in the open literature.

In a similar manner, long-term clogging can also negatively influence geosynthetic drainage systems; both the drainage core and the geotextile filter. Focus in geosynthetic drainage systems has been on the geotextile due to its relatively small openings in comparison to the drainage core of geocomposites and geonets. Three candidate tests aimed at an assessment of long-term geotextile clogging are available. They are the following:

- Long-Term Flow (*LTF*) test via GRI GT-1.
- Gradient Ratio (*GR*) test via ASTM D 5101.
- Hydraulic Conductivity Ratio (*HCR*) test via ASTM D 5084.

Of these tests, the hydraulic conductivity ratio test is preferred by the authors since it can model the field situation under closely simulated conditions. The test is performed using a flexible wall soil permeameter of the type that is readily available in most soil testing laboratories, e.g., ASTM D5084.

As described in section 2.2, the precipitation (*P*) that we will focus upon is the hourly storm event over a 6-hour period. This will be seen to be very intense in comparison to daily or monthly monitoring of precipitation on the basis of the flux that is generated.

The infiltration (*I*) into the cover soil is minimized by increasing the surface runoff (*R*). For the cross sections we are considering, the runoff is relatively high since slope angles where instability occurs are usually greater than 14 deg. which is 4(H)-to-1(V). Of course, high surface runoff can easily lead to surface soil erosion but this consideration is not addressed in this report, see Koerner and Daniel (1997) for details in this regard. The infiltration is also influenced by the type of surface soil. For example, a coarse drainage gravel as shown in Figure 5a will accept significantly more infiltration and less runoff than will a fine grained soil as shown in Figure 5b.

Water that enters the cover soil as infiltration flows downward by gravitational forces. However, capillary action tends to retain water in the soil. Storage of water in soil, coupled with removal of water by evapotranspiration, are important mechanisms in limiting the percolation of water through the cover soils. Much of the water that falls on the soil surface infiltrates into the soil and is returned to the atmosphere over time by plants through evapotranspiration. Unfortunately, for very intense storms, the actual evapotranspiration (*AET*) is very limited due to the short time periods considered.

An important major retarding mechanism toward high percolation values is the water storage capacity of soils (*WS*). For dry, or partially saturated soils, infiltrating water will simply fill the available space in the soil voids. For sporadic and relatively mild rain events, the retardation of percolation by water storage is a major factor in limiting percolation through the system. When the voids in the cover soils are at field capacity or are fully saturated, however, there is no additional storage capacity and the infiltrating water all passes through the system as percolation in accordance with Darcy's formula. When the soils involved have high k-values the quantities can be quite large. Cover soils at field capacity, or fully saturated, are the likely case for the extreme storm events which are focused upon in this report.

The vertical percolation (*PERC*) value itself (in units of mm/hour) is based on a horizontal unit area, thus its units are mm/hour-m^2. It would continue downward except for the underlying hydraulic barrier. In this report we make the assumption that there is "zero leakage" through the hydraulic barrier layer (*GM, GCL* and/or *CCL*) beneath the drainage layer.. This is done for the following reasons:

1. For slopes of 4(H)-to-1(V), and greater, the value will be quite small, e.g., roofs of homes at these angles (generally) do not leak.
2. The velocity of flow will be quite high for the short duration and intense storm events considered herein further minimizing leakage rates.
3. The no leakage assumption gives rise to conservative estimates of percolation.
4. We have no idea what value to assume for leakage and would much prefer to assume good CQC and CQA of the barrier system with no leakage.

Finally, whatever value of percolation arrives at the drainage layer, it translates completely into lateral drainage, or flux (*FLUX*). The flux accumulates as it flows on top of the hydraulic barrier to a maximum value at the toe of the slope. Thus, the flux is at a maximum at the toe of the slope and the drainage system is designed on the basis of this value. It is a worst case scenario assumption and is recommended for design so as to avoid seepage related slope instability problems.

3.2 Calculation Options

There are many possible calculation options for percolation and we have selected three of them; manually for peak *monthly* averages, computer modeling for peak *daily* averages, and manually for peak *hourly* averages. Each will be explained.

3.2.1 Manual Method for Monthly Averages

A water balance analysis can be performed on a monthly average basis. The procedure can be performed manually as proposed by Dr. D. E. Daniel of the University of Illinois-Urbana, however, it is highly amenable to use of a computer spread sheet to facilitate the actual computations. Three publications provide the basis of Daniel's procedure; Thornthwaite and Mather (1957), Fenn, et al. (1975), and Kmet (1982).

A table or spread sheet should be set up with twelve columns established for the twelve months of the year. In a progressive sequence of steps, an additional twelve rows (from A through P) are developed for each of the twelve months of the year. Table 4 gives an overview of the information needed and the respective calculations to eventually arrive at a percolation value (*PERC*) passing through the cross-section arriving at the drainage layer. The flow units are in "mm/month" over a square meter of horizontal surface. Table 5 gives an illustration of this procedure for a final cover system as shown in Figure 5b. Details of the procedure are found in Koerner and Daniel (1997). The target value in Table 5 is the maximum monthly value of "*PERC*", i.e., the required percolation value which is used to design the drainage system. Note that the value in this example is 8.54 mm/month in the month of January and thereafter the evapotranspiration has eliminated all of the infiltration resulting in zero percolation for the rest of the year.

3.2.2 Computer Method for Daily Averages

Nearly all water balance analyses performed in the United States are conducted using the computer program "*HELP*" (Hydraulic Evaluation of Landfill Performance). The HELP program was written by Dr. P. R. Schroeder of the U.S. Army Corps of Engineers, Waterways Experiment Station under sponsorship of the U.S. EPA. The program, which has been periodically updated, is available in the public domain. At the time of this writing, the latest version is Version 3.0 and is available by purchasing "The Hydraulic Evaluation of Landfill Performance Model, Engineering Documentation for Version 3", EPA/600/R-94/168b, from the National Technical Information Service in Springfield, Virginia. A user's manual is supplied with a diskette that contains the program, which is written in FORTRAN for use on a personal computer.

Table 4 - Manual Procedure for "PERC" Calculation, Based on Monthly Average Rainfall Values, see Table 5 for Example

Row	Value	Units	Comment or Calculation
A	average monthly temperature	°C	local weather station data
B	monthly heat index	—	calculated value needed to determine evapotranspiration
C	unadjusted daily potential evapotranspiration	mm/mo.	calculated value using data from Row A & Row B
D	monthly duration of sunlight	—	values taken from published tables
E	potential evapotranspiration	mm/mo.	multiply Row C by Row D
F	mean monthly precipitation	mm/mo.	local weather station data
G	runoff coefficient	—	estimated value, but guidance is available
H	runoff	mm/mo.	multiply Row F by Row G
I	infiltration	mm/mo.	subtract Row H from Row F
J	infiltration minus potential evapotranspiration	mm/mo.	subtract Row E from Row I
K	accumulated water loss	mm/mo.	sum of negative values in Row J
L	water stored	mm/mo.	calculated value having many details
M	change in water storage	mm/mo.	difference in monthly water storage from Row L data
N	actual evapotranspiration	mm/mo.	comparison to potential evapotranspiration
O	percolation (PERC)	mm/mo.	comparison to determine if percolation occurs (or not) and to what amount
P	check of calculations	mm/mo.	validation of water balance calculations

Table 5 - Illustration of the water balance analysis in a typical final cover system using the manual method for monthly averages.

Row	Parameter	January	Feburary	March	April	May	June	July	August	September	October	November	December	Total	%
A	Avg. monthly Temp., °C	9.5	11.8	15.8	20.4	23.9	27.6	29.3	29.2	28.2	21.0	14 8	11.2		
B	Monthly heat Index (H_a)	2.64	3.87	5.71	8 41	10.68	13.28	14.54	14.47	12.28	8.78	5.17	3.39	103.02	
C	Unadjusted daily potential evapotranspiration (UPET), mm/mo.	0 44	0.72	1.39	2.48	3 55	4.53	4.94	4 92	4.36	2.65	1.20	0.64		
D	Possible monthly duration of sunlight (N)	27	26.1	30.9	32.4	35.4	35.1	36	34.2	30.9	29.4	28.7	28.4		
E	Potential evapotranspiration (PET), mm/mo.	11.92	18.80	43.02	80.33	125.49	159.09	177.82	168 20	134.60	77.82	32.07	16.90		
F	Precipitation (P), mm/mo.	34.09	31.34	26.67	114.4	108.81	68.73	42.72	100.13	100.58	95.71	95.5	27.99	846.67	100%
G	Runoff Coefficient (RC)	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40		
H	Runoff (R), mm/mo.	13 636	12.536	10.668	45.76	43.524	27.492	17.008	40.052	40.232	38.284	38.2	11.196	338.67	40%
I	Infiltration (IN), mm/mo.	20 45	18 80	16.00	68 64	65 29	41.24	25 63	60 08	60.35	57.43	57.30	16.79	508 00	
J	IN - PET, mm/mo.	8.64	0.01	-27.02	-11.69	-60.21	-117.85	-152.19	-108.12	-74.45	-20.40	25.23	-0.11		
K	Accumulated water loss (WL), mm/mo.	0.00	0 00	-27.02	-38 71	-98.92	-216 77	-368.96	-477.07	-551.53	-571.92	-571.92	-572.03		
L	Water stored (WS), mm/mo.	118.50	118.50	93.32	81.63	69 59	41.80	30.86	45.56	61.38	40 98	118.60	118 50		
M	Change in water storage (CWS), mm/mo.	0.00	0.00	-25.18	-11.69	-12.04	-27.79	-10 95	14.70	15.80	-20.40	77.54	0.00		
N	Actual evapotranspiration (AET), mm/mo.	11.92	18.80	41.18	80.33	77.33	69.02	36.58	45.38	44.55	77.82	32.07	16.78	551.77	65%
O	Percolation (PERC), mm/mo.	8.54	0.01	0.00	0.00	0.00	0.00	0.00	0 00	0.00	0 00	-52.31	0.00	-43.77	-5%
P	Check (CK), mm/mo.	34.09	31.34	26.67	114.40	108.81	68.73	42.72	100.13	100.58	95.71	95.50	27.99	846.67	100%

The computer program employs the same principles as the method of manual analysis described in section 3.2.1, but *HELP* uses a daily (rather than monthly) time internal and employs sophisticated algorithms for many of the computations. The model accepts weather, soil, and geometric data. It then uses solution techniques that account for the effects of surface storage, snowmelt, runoff, infiltration, evapotranspiration, vegetative growth, storage of soil moisture, lateral drainage of water in drainage layers, leachate recirculation, vertical percolation of soil water, and leakage through hydraulic barriers (*GM, GCL, CCL* or composite liners).

Engineering documentation of *HELP* is provided by Schroeder et al. (1994). We will not attempt to repeat the documentation here. Instead, we will provide an overview of *HELP*'s capability and discuss the key technical components of the model. The *HELP* program contains a number of default values for soil and other parameters, which can prove to be helpful even for manual analyses.

3.2.2.1 Design Profile

A schematic view of the profile that *HELP* was designed to simulate is shown in Figure 6. The profile is divided into three subprofiles (cover, waste and bottom liner system) to simulate a landfill. For purposes of this report, attention is focused on the cover.

The layers that are analyzed with *HELP* are categorized by the hydraulic function that they perform. Four types of layers are available, as summarized in Table 6.

(a) Vertical Percolation Layer

A vertical percolation layer is any layer permitting vertical movement of water (downward due to gravity or upward due to evapotranspiration) within it, and not serving as a lateral drainage layer. Examples of layers that are treated as a vertical percolation layers are top soil, protection soil, gas collection layer, foundation soil, and waste.

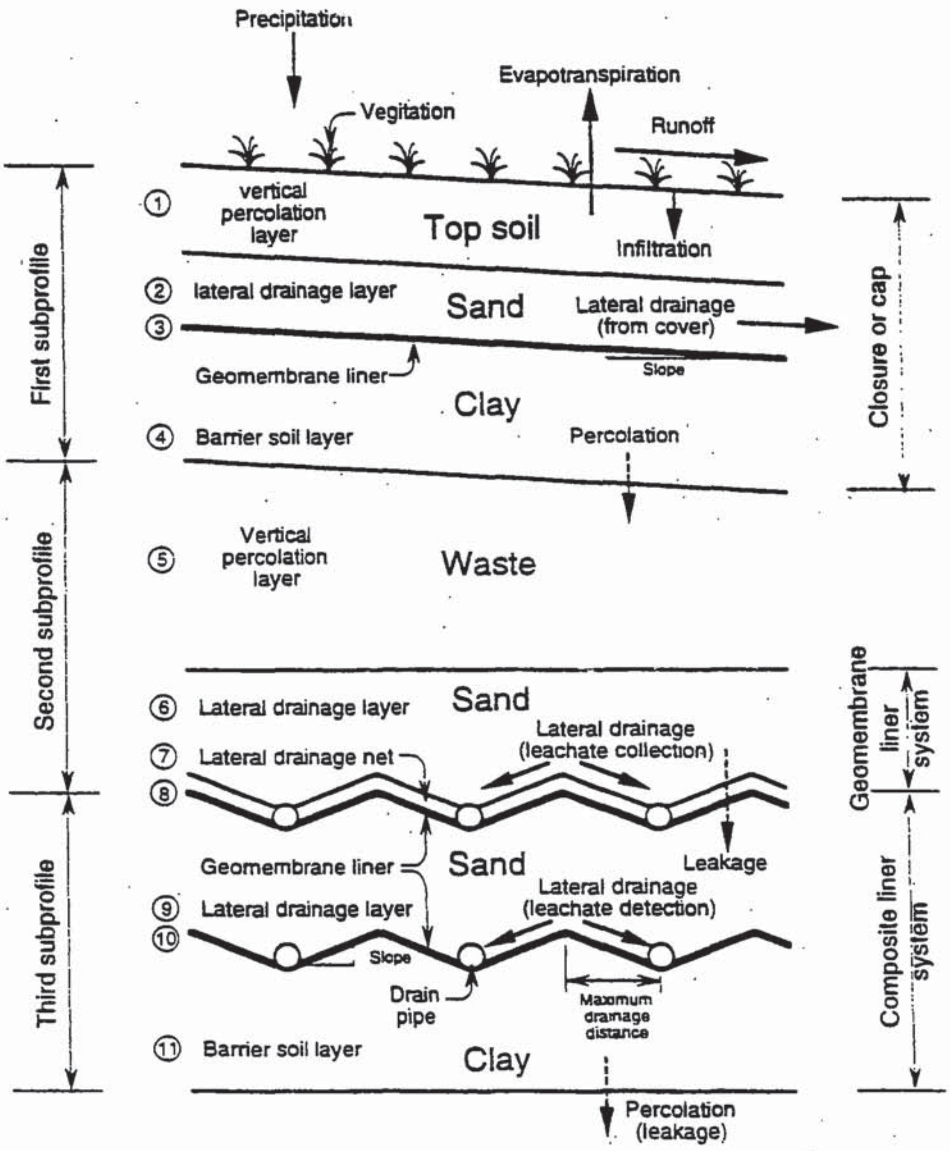


Figure 6 - Elevation view of a typical solid waste landfill

Table 6 - Four Types of Layers Allowed in the HELP Program

Type of Layer	Hydraulic Characteristics
Vertical Percolation Layer	Flow in this layer is strictly vertical (downward due to gravity or upward due to evapotranspiration). Hydraulic conductivity (permeability) at saturation is typically in the range of 10^{-3} to 10^{-6} cm/sec.
Lateral Drainage Layer	This layer promotes lateral drainage to collection systems, e.g., drains at the perimeter of the cover. Hydraulic conductivity (permeability) can vary greatly. (This layer is the focus of the present report). The underlying layer is normally a barrier consisting of some type of liner.
Barrier Soil Liner	Barrier soil liners are low-permeability soils; a compacted clay liner (*CCL*) with a permeability of 10^{-6} to 10^{-7} cm/sec or a geosynthetic clay liner (*GCL*) with a permeability of 10^{-8} to 10^{-9} cm/sec.
Geomembrane	Geomembranes can be of many types. In the HELP program, they are assumed to permit leakage via vapor diffusion, manufacturing flaws (pinholes), and installation defects (e.g., flaws).

The method of calculating the downward movement of water in the unsaturated vertical percolation layer is approximate. More rigorous analytic techniques are available that more carefully compute hydraulic gradients and consider vapor and thermal transport mechanisms. However, computer codes that account for unsaturated flow more rigorously tend to be difficult to use because of their complexity and, therefore, are rarely employed for water balance analyses. Nevertheless, *HELP* is not considered a particularly accurate simulation program for covers that are located in arid areas, where the subtleties of unsaturated moisture movement can dominate the water balance.

(b) Lateral Drainage Layer

Lateral drainage layers may consist of granular soils or geosynthetic materials. Vertical drainage in a lateral drainage layer is modeled in the same manner as a vertical percolation layer. However, lateral flow in the saturated zone at the base of the lateral drainage layer is allowed.

Unconfined lateral flow in the drainage layer is modeled using Darcy's formula, assuming continuity and employing the Depuit-Forcheimer assumptions (seepage parallel to the slope of the layer and hydraulic gradient proportional to the slope of the underlying barrier layer). The algorithm used by *HELP* is reasonably rigorous and accurate. The accuracy with which the permeability value of the lateral drainage is determined, not the method of analysis, limits the overall accuracy of the calculations.

(c) Low-Permeability Soil Barrier Layer

Compacted clay liners (*CCLs*) and geosynthetic clay liners (*GCLs*) are frequently used as hydraulic barrier layers. The soil is assumed to be saturated, i.e., to have no capacity to store water without drainage occurring. Leakage through the *CCL* or *GCL* is assumed to occur whenever there is a head of water on top of the barrier.

When the soil liner is located near to the surface of the cover and there is no geomembrane overlying the clay, the low-permeability soil layer will probably desiccate at times, invalidating the assumption of continuous saturation. To model this process, the low-permeability soil layer can be treated as a vertical percolation layer. Also, clay liners are not completely saturated with water at the time of construction, so the liners must first absorb some nominal amount of water before drainage is initiated.

(d) Geomembrane Layer

Geomembranes are widely and routinely used in well engineered covers and liners beneath the waste. Geomembranes can be extremely effective hydraulic barriers and can withstand many of the forces (e.g., differential settlement and freeze/thaw or wet/dry cycles) that are destructive to clay liners.

The *HELP* program assumes that liquids can leak through geomembranes by three mechanisms: (1) vapor diffusion through the intact geomembrane; (2) leakage through manufacturing defects (pinholes); and (3) leakage through construction defects (mainly flaws in seams). The equations are complex and involve a number of possible cases. The reader is referred to Schroeder, et al. (1994) for details.

3.2.2.2 Default Properties

One of the useful aspects of the *HELP* model is that it contains default parameters for various soil and waste properties based upon data available for more than a thousand soils. Default properties are available for low-density, moderate-density and high-density soils. Information is also available on default waste characteristics, on saturated hydraulic conductivity (permeability) of wastes, and on default material characteristics for various geosynthetic materials. In addition to the manual which documents the HELP program, these default tables are reproduced in Koerner and Daniel (1997).

3.2.2.3 Method of Solution

The *HELP* program models both surface processes and subsurface processes. The surface processes include snowmelt, interception of rainfall by vegetation, surface runoff, and evaporation of water. The subsurface processes modeled are evaporation of water from the soil, transpiration of water by plants, vertical percolation of water through unsaturated soil, lateral drainage in drainage layers, and leakage of water through clay barrier soils, geomembranes, or composite liners. Daily infiltration of water into the surface of the cover is determined indirectly from a surface water balance. Each day, infiltration is assumed to equal the sum of rainfall and snowmelt, minus the sum of runoff, surface storage (e.g., on the surfaces of plants), and surface evaporation (e.g., evaporation of water stored on the surfaces of plants).

The daily surface water accounting procedure used in *HELP* is as follows. Snowfall and rainfall are added to the surface snow storage, if present, and then snowmelt plus excess storage of rainfall is computed. The total outflow from the snow cover is then treated as rainfall in the absence of a snow cover for the purpose of computing runoff. A rainfall-runoff relationship is used to calculate runoff. Surface evaporation is then computed, but surface evaporation is not allowed to exceed the sum of surface snow storage and intercepted rainfall. The snowmelt and rainfall that does not run off or evaporate is assumed to infiltrate into the landfill. Computed infiltration in excess of the storage and drainage capacity of the soil is routed back to the surface and is added to the runoff or held as surface storage.

The subsurface processes modeled by *HELP* are as follows. The first subsurface process considered is evaporation of water from the soil. Next, transpiration of water from the evaporative zone by plants is computed. Other processes are modeled using a time step varying from 30 minutes to 6 hours. For vertical percolation layers, a water balance is performed on each layer to determine the water content of the material. Hydraulic conductivity is computed from the water content, and then the amount of gravity drainage (if any) is determined. For lateral drainage layers, a water balance is used to determine whether the drainage layer is saturated at any point, and if so, lateral drainage is computed for that portion of the layer that is saturated. Vertical percolation is assumed to occur in the lateral drainage layer above the zone of saturation. The same equations employed for analyzing gravity drainage in vertical percolation layers are used to analyze vertical flow above the saturated zone in lateral drainage layers. Soil barrier layers are assumed to be continuously saturated and, therefore, no water balance is performed for them. Leakage is computed from the hydraulic properties of the drainage layer and the amount of head acting on the barrier layer. Leakage through geomembranes is computed from vapor diffusion, leakage through pinholes, and leakage through installation defects.

The *HELP* program allows the user to select the number of years to simulate as well as the output frequency. The user may use a maximum of 100 years of simulation provided the weather are available for that many years. The user may also select any, all or none of the available output options - namely, daily, monthly or annual output. Note that daily output is the shortest time-interval available using the *HELP* program. Of the resulting output information, the peak daily percolation ($PERC_{peak\ daily}$, in units of mm/day) into the drainage layer within the cover soil system is the target value for this report. This value will be used to calculate the value of flux which is then used to design the drainage system.

3.2.3 Manual Method for Hourly Averages

Under the hypothesis that seepage induced slope instability occurs in periods consisting of hourly intervals, and recognition that the minimum time-internal from *HELP* is days, a manual method to calculate hourly averages is presented. Obviously, it requires hourly precipitation data. Based on the basic concepts of water balance analysis shown in Figure 5, the following relationships hold:

$$P = I + SR \tag{16}$$

and

$$I = PERC + AET + \Delta WS \tag{17}$$

where P = probable maximum (hourly) precipitation
I = infiltration
SR = surface runoff
$PERC$ = percolation
AET = actual evapotranspiration
ΔWS = change in water stored in cover soil
= (field capacity) - (actual water content)

Under the assumptions that the immediate time before the *PMP* event has been a period of regular rainfall, the actual evapotranspiration is negligible for a intense rainfall over a short period of time (e.g., a few hours), and the cover soil is at *field capacity* before the storm reaches its highest intensity (i.e., there is only nominal excess water storage capacity available at the time), the infiltration results directly in percolation, i.e., $I = PERC$. Therefore, the following relationships result:

$$P = PERC + SR \tag{18}$$

$$\text{or } PERC = P - SR$$

$$\text{but } \quad SR = P(RC) \tag{19}$$

where "RC " equals the runoff coefficient

$$\text{thus } \quad PERC = P\,(1 - RC) \tag{20}$$

Note that Equation (20) is valid only when the cover soil is sufficiently permeable so that the amount of water which does not runoff [i.e., $P(1 - RC)$] can percolate through the cover soil into the drainage layer. When the cover soil is not permeable enough to handle such amount of water, the difference will occur as sheet flow over the ground surface. The amount is governed by the permeability of the cover soil ($k_{cover\ soil}$). Thiel and Stewart (1993) showed that the percolation into the drainage layer, under such a situation, should be determined as:

$$PERC = k_{cover\ soil}; \qquad \text{when } P(1 - RC) > k_{cover\ soil} \qquad \text{(21a)}$$

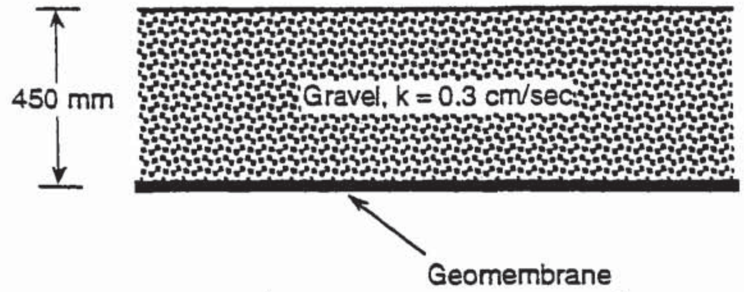
otherwise:

$$PERC = \text{as calculated}; \qquad \text{when } P(1 - RC) \leq k_{cover\ soil} \qquad \text{(21b)}$$

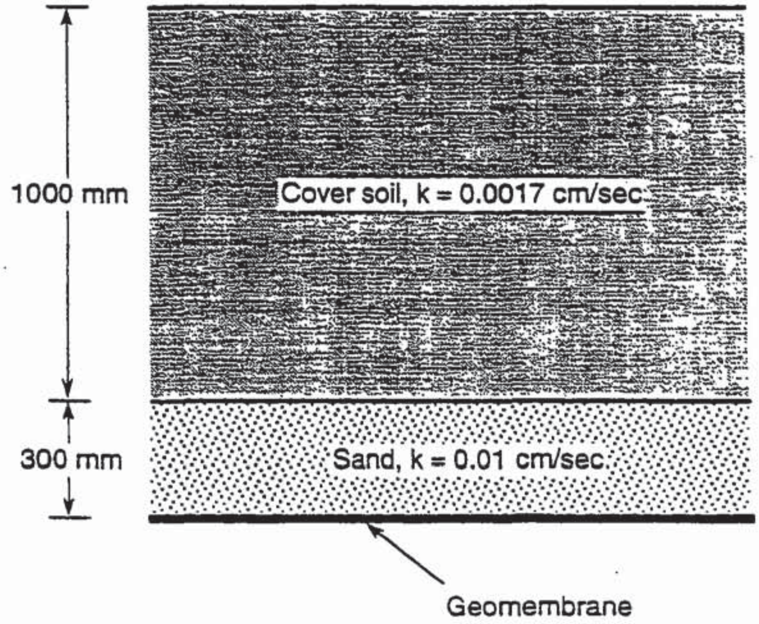
3.3 Comparison of Results

The following example is used to demonstrate the dramatic differences between the three calculation options just presented; namely, *monthly, daily and hourly* averages.

Example: A landfill is to be built in Thrall, Texas (60 kilometers northeast of Austin). The site is a 200 m by 200 m square, i.e., it is 4 hectares. The side slopes of the leachate collection layer in the liner system, as well as the final cover, have slope inclinations of 3(H)-to-1(V). The runoff coefficients for the leachate collection layer is 0.18 and for the cover soil is 0.4. Calculate the percolation (*PERC*) and flux (*FLUX*) values of the leachate collection layer in the side slope liner system (figure "a" following) and the final cover system (figure b" following) for slope lengths of 10, 30, 60 and 100 m on the basis of monthly precipitation (per Section 3.2.1), daily precipitation (per section 3.2.2), and hourly precipitation (per section 3.2.3). The soil permeability values are default values suggested in the HELP manual.



(a) Leachate collection system



(b) Final cover system

<u>Solution</u>: Each of the three calculation options presented in the previous section were used to obtain the percolation (i.e., "*PERC*") and the results were multiplied by the respective slope lengths using a unit width to obtain the respective values of flow rates (i.e., "FLUX"). The results are summarized in Table 7.

Table 7 - Results of the example problem using various time interval options of water balance analyses to obtain PERC and varying slope lengths to obtain FLUX.

Type	Time Internal for Calculations	PERC (mm/hr)	FLUX (m^3/hr)			
			L = 10 m	L = 30 m	L = 60 m	L = 100 m
(a) leachate collection system	monthly	0.046[1]	4.4×10^{-4}	1.3×10^{-3}	2.6×10^{-3}	4.4×10^{-3}
	daily	varies[2]	0.025	0.079	0.16	0.28
	hourly	68.1[3]	0.65	1.9	3.9	6.5
(b) final cover system	monthly	0.011[1]	1.1×10^{-4}	3.3×10^{-4}	6.6×10^{-4}	1.1×10^{-3}
	daily	varies[2]	0.013	0.041	0.088	0.14
	hourly	49.9[3]	0.50	1.5	3.0	5.0

Note:
1. Via spread sheets as shown in Table 5, using the average monthly temperature, duration of sunlight and precipitation data from Austin, Texas.
2. Via the *HELP* model using evapotranspiration , synthetic temperature and solar radiation data from Austin, Texas and historical precipitation data (1974-1978) from San Antonio, Texas. The PERC and FLUX-values vary since the *HELP* model takes the slope length into consideration when calculating the amount of runoff.
3. Using the 6-hour rainfall data recorded at Thrall, Texas over an area of 260 km^2 (see Table-2) and Equations 20 and 21.

For the above example, the values of *FLUX* for the various slope lengths can be put into a comparison format by assuming that the *HELP* model gives the conventionally used values for design purposes. Thus the *HELP* generated *FLUX*-values will be assigned a value of 100% (or 1.0), and the monthly and hourly values compared accordingly. As seen in Table 8, it is readily apparent that the precipitation time interval plays a dominate role in the calculations. Using monthly intervals, the *FLUX*-values vastly <u>underestimate</u> the *HELP* generated values (≈ 60 to 120 times), whereas the hourly interval *FLUX*-values vastly <u>overpredict</u> the *HELP* generated values (≈ 25 to 40 times). *In the writers' opinion, it is the hourly interval calculations that result in flux-values which create seepage induced slope instability and calculations using this time interval should be used in the design of drainage layers for applications as described in this report.* This will be the approach taken in the remainder of the report. At the outset, however, it should be stated that drainage systems designed as just noted (i.e., on an hourly interval basis with the worst case assumptions stated in section 3.2.3) will require significantly greater hydraulic capacity than the comparable drainage systems designed using the *HELP* model.

Table 8 - Comparison of *FLUX*-values for different calculation options normalized to the conventionally used HELP generated values.

Type	Calculation option	Slope length (m)			
		10	30	60	100
(a) leachate collection system	monthly	0.018	0.016	0.016	0.016
	daily (HELP)	1.0	1.0	1.0	1.0
	hourly	26.0	24.0	24.4	23.2
(b) final cover system	monthly	0.008	0.008	0.008	0.008
	daily (HELP)	1.0	1.0	1.0	1.0
	hourly	38.5	36.6	34.1	35.7

4.0 DRAINAGE LAYER CONSIDERATIONS

As long as there is percolation into the drainage layer beyond its field capacity, there will be water flowing within the slope's drainage system. When the drainage layer is capable of handling this flow rate, which is generally the assumption made in the design stage, seepage will occur in the drainage layer only. Giroud and Houlihan (1995) describe the situation for both steady state and transient flow conditions. They caution that the drainage layer must be able to accommodate the required flow rate. However, when the flow rate is too large to be handled by the drainage layer and/or its toe drain, seepage will buildup above the drainage layer into the overlying cover soil or even flow above grade as an addition to runoff. Such seepage in the drainage layer or overlying cover soil could build up in a horizontal or a parallel manner, or as a combination of both. Since water tends to uplift soil particles due to a buoyancy effects and seepage tends to drag particles in the direction of flow, such seepage forces lead to a decrease in the slope's factor of safety and can easily result in seepage induced sliding.

From the above discussion, two issues are significant in conducting the design of the drainage layer above a lined slope: the flow (phreatic surface) orientation and the depth of submergence. Both issues are discussed in this section.

4.1 Patterns of Seepage Buildup in Cover Soils

Consider a cover soil of uniform thickness placed directly above a geomembrane or other barrier material at a slope angle of "β" as shown in Figure 7. Two discrete zones are illustrated; a small passive wedge at the toe of the slope resisting a long, thin active wedge extending the length of the slope. Only one type of soil is placed directly against the geomembrane and it is cohesionless, i.e., typical of a leachate collection layer or a drainage layer in a final cover. For the case of a drainage layer in a final cover, the profile can also consist of different soil materials placed in parallel layers. In this case, the drainage soil would be granular and placed directly above the geomembrane and then a locally available finer grained soil (including topsoil) would be placed above the drainage layer. Other soil properties, soil-to-geomembrane friction angle and the dimensions of the considered profile are shown in Figure 7.

Note should be made in Figure 7 of two possible phreatic surface orientations. This is necessary because seepage can be built-up in two different ways: horizontal or parallel to the slope. Thus, orientation is quantified as a horizontal submergence ratio (*HSR*), or a parallel submergence ratio (*PSR*). As to the depth of submergence, it is a function of the amount of infiltration, the permeability of the drainage layer and the drainage layer capacity. The dimensional definitions of both ratios are given in Figure 7.

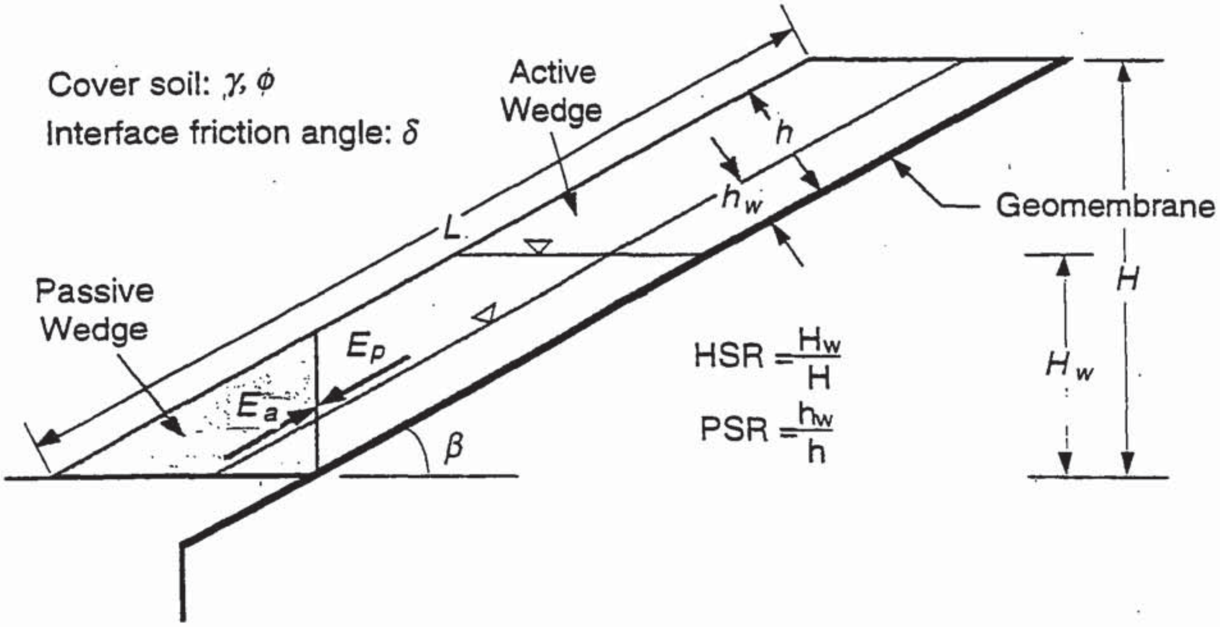


Figure 7 - Cross-section of cover soil on a geomembrane with different seepage buildup patterns.

Of the two seepage orientation possibilities shown in Figure 7, it is felt that extremely low permeabilities at the toe of slope will result in a horizontal seepage buildup, Soong and Koerner (1996). This would typify cases where toe blockage occurs due to fines migrating downgradient over time, or due to ice buildup at the toe of the slope as the up-gradient drainage layer thaws producing seepage pressure. However, in most steady-state situations, it is generally assumed that water flows parallel to the slope, e.g., Giroud et al. (1995), Thiel and Stewart

(1993). This would likely occur when the drainage system is underdesigned from the outset. In a separate study, however, it has been shown that different seepage orientations, under the same submergence ratio, make little difference in the resulting slope stability factor of safety values, Soong and Koerner (1996). Furthermore, a specific amount of percolation results in a unique submergence ratio regardless of the seepage orientation assumption, i.e., $HSR = PSR$, since the total submerged volume of soil remains the same. Based on the above reasons, only the parallel seepage orientation will be considered in this report.

4.2 Drainage Layer Capacity (*DLC*)

The rate of percolation per unit area (in units of m^3/hour) coming through a given cross section, assuming no leakage through the underlying hydraulic barrier layer (which is a conservative assumption), is determined as follows:

$$\boxed{FLUX_{reqd} = \frac{PERC}{1000} \times L(\cos\beta) \times w} \tag{22}$$

where $PERC$ = the rate of percolation in units of mm/hr [see Equations 20 and 21],

L = length of drainage slope, m

β = slope angle,

w = 1.0 = unit width of drainage slope, m

When designing the drainage layer in a soil covered slope, the following concept of drainage layer capacity should be evaluated:

$$\boxed{DLC = \frac{FLUX_{allow}}{FLUX_{reqd}}} \tag{23}$$

where DLC = drainage layer capacity

$FLUX_{allow}$ = allowable flow rate of the drainage layer per unit width of slope,

$FLUX_{req'd}$ = actual flow rate per unit width of slope.

It is good design practice and is generally required by regulatory agencies that the drainage layer capacity cannot be exceeded, i.e., $DLC \geq 1.0$. That is, complete saturation of the drainage layer should not be allowed at any time.

4.3 Parallel Submergence Ratio (*PSR*)

In a cover soil slope stability analysis, it is necessary to determine the depth of submergence in the cross section so as to quantify the value of parallel submergence ratio (*PSR*). The value of *PSR* can then be used in the slope stability analysis and ultimately results in a factor of safety (*FS*) regarding slope stability. The following procedure can be used to calculate the parallel submergence ratio (*PSR*). The typical cover system configuration of Figure 5b and dimensions are illustrated in Figure 8. Note that the analysis also applies for full thickness drainage layers typical of leachate collection layers beneath the waste material as shown in Figure 5a.

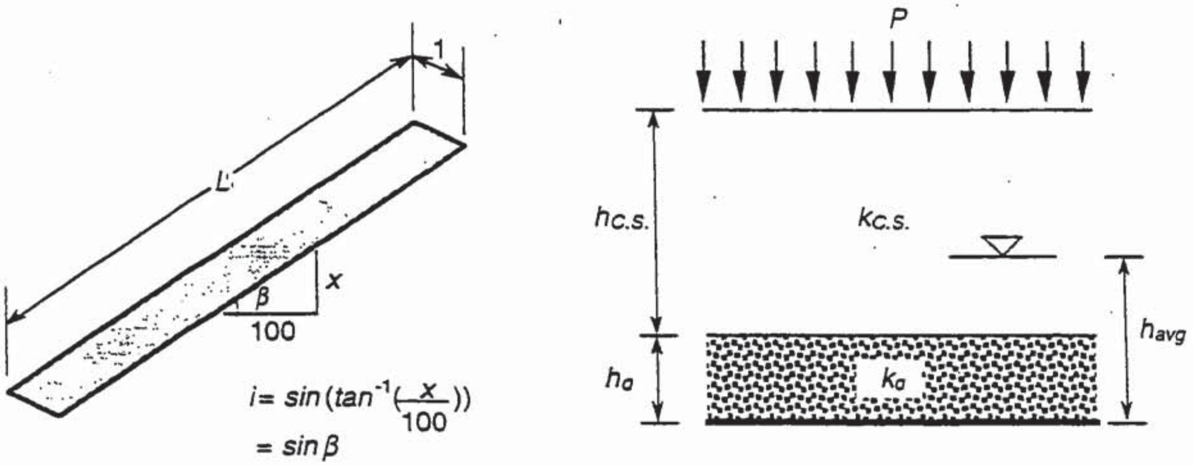


Figure 8 - Typical cover system configuration and dimensions used to calculate parallel submergence ratio

The average head buildup (h_{avg}) above the barrier layer can then be determined as follows:

When $h_{avg} \leq h_d$, i.e., $DLC \geq 1.0$ (and the average phreatic surface level is within the drainage layer).

$$h_{avg} = \frac{(FLUX_{reqd} / 3600)}{k_d \times i} \tag{24}$$

When $h_{avg} > h_d$, i.e., $DLC < 1.0$ (the average phreatic surface level is within the cover soil layer),.

$$FLUX_{reqd} / 3600 = i \times [\, k_{C.S.} \; (h_{avg} - h_d) + k_d h_d \,] \tag{25}$$

where $FLUX_{reqd}$ = required flux, m^3/hr

$k_{C.S.}$ = permeability of cover soil, m/sec

k_d = permeability of drainage soil, m/sec

$h_{avg.}$ = average head buildup above the geomembrane, m, and

h_d = thickness of the drainage layer, m.

$$h_{avg} = \frac{\left(\dfrac{FLUX_{reqd}}{3600 \times i} \right) - [\, h_d (k_d - k_{C.S.} \;)]}{k_{C.S.}} \tag{26}$$

Finally, the parallel submergence ratio, "*PSR*", can be calculated as follows:

$$PSR = \frac{h_{avg}}{h_{c.s} + h_d} \tag{27}$$

The parallel submergence ratio is then used in the slope stability analysis as the mechanism to incorporate seepage forces into the calculation. Note that the above discussion has been focused on natural drainage materials. However, the procedure is also applicable to geosynthetic drainage composites, providing the thickness and the equivalent permeability of the drainage geocomposite under the site specific normal pressure and hydraulic gradient is known.

5.0 SLOPE STABILITY ANALYSIS INCORPORATING SEEPAGE FORCES

Figure 9 shows the free body diagrams of both the active and passive wedges assuming parallel seepage buildup resulting in a parallel submergence ratio (PSR). As noted previously, it follows the same concept as does horizontal seepage buildup. The symbols used are defined below.

W_A = total weight of the active wedge

W_P = total weight of the passive wedge

$(Area)'_A$ = area of the active wedge below the free water surface

$(Area)''_A$ = area of the active wedge above the free water surface

$(Area)_P$ = area of the passive wedge

$\gamma_{sat'd}$ = saturated unit weight of the cover soil

γ_{dry} = dry unit weight of the cover soil

γ_W = unit weight of water

h = thickness of the cover soil

H = vertical height of the slope measured from the toe

h_W = (PSR) (h) = height of the free water surface measured from the geomembrane

PSR = parallel submergence ratio

β = slope angle

U_h = resultant of the pore pressures acting on the interwedge surfaces

U_n = resultant of the pore pressures acting perpendicular to the slope

U_v = resultant of the vertical pore pressures acting on the passive wedge

N_A = effective force normal to the failure plane of the active wedge

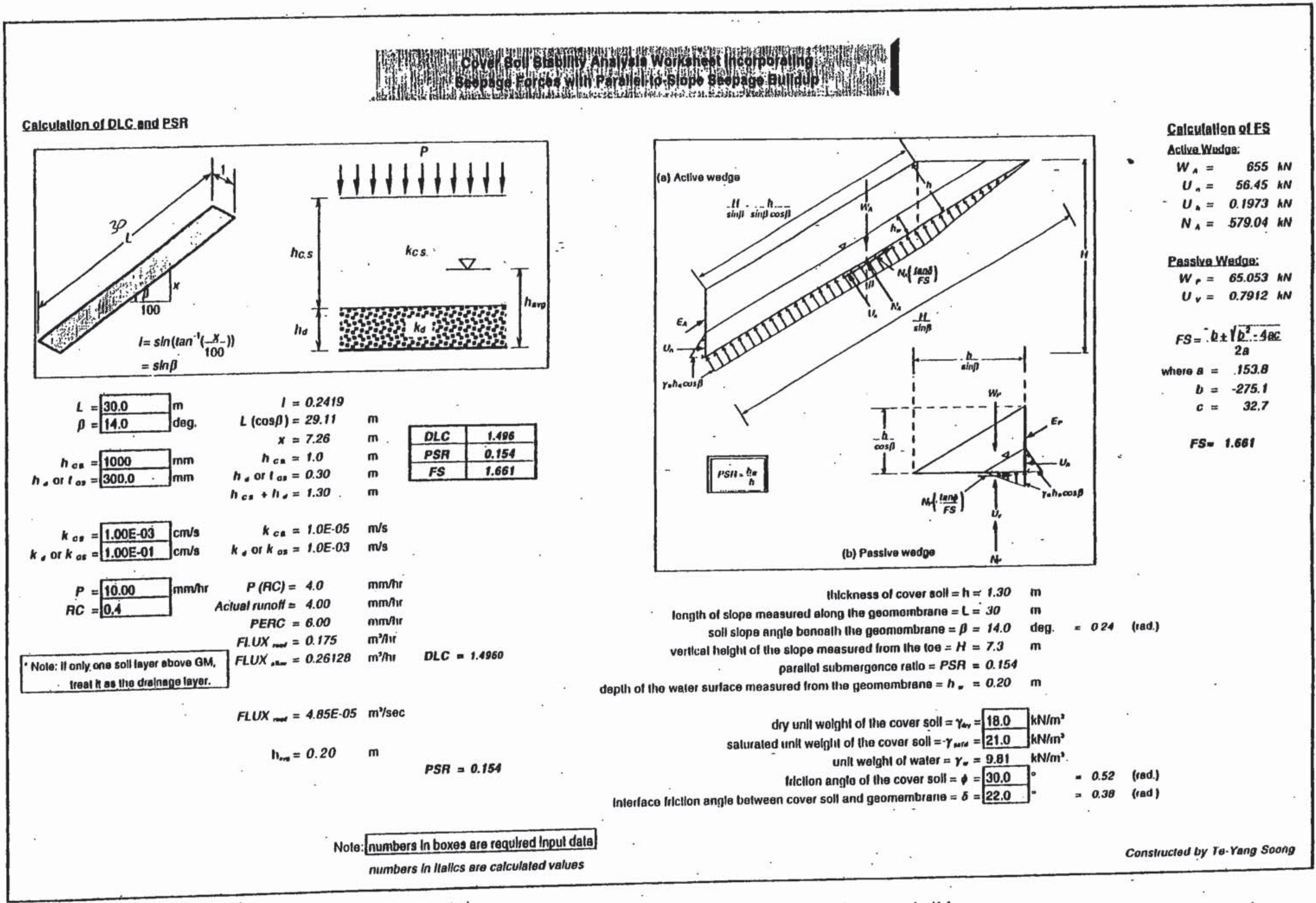
N_P = effective force normal to the failure plane of the passive wedge

ϕ = cover soil friction angle

δ = interface friction angle between cover soil and geomembrane

E_A = interwedge force acting on the active wedge from the passive wedge

E_P = interwedge force acting on the passive wedge from the active wedge



Cover Soil Stability Analysis Worksheet Incorporating Seepage Forces with Parallel-to-Slope Seepage Buildup

Calculation of DLC and PSR

$I = \sin(\tan^{-1}(\frac{x}{100}))$
$= \sin\beta$

L = 30.0 m
β = 14.0 deg.

h_{cs} = 1000 mm
h_d or t_{ds} = 300.0 mm

k_{cs} = 1.00E-03 cm/s
k_d or k_{ds} = 1.00E-01 cm/s

P = 10.00 mm/hr
RC = 0.4

* Note: If only one soil layer above GM, treat it as the drainage layer.

I = 0.2419
L (cosβ) = 29.11 m
x = 7.26 m
h_{cs} = 1.0 m
h_d or t_{ds} = 0.30 m
h_{cs} + h_d = 1.30 m

k_{cs} = 1.0E-05 m/s
k_d or k_{ds} = 1.0E-03 m/s

P (RC) = 4.0 mm/hr
Actual runoff = 4.00 mm/hr
PERC = 6.00 mm/hr
$FLUX_{reqd}$ = 0.175 m³/hr
$FLUX_{allow}$ = 0.26128 m³/hr

DLC = 1.4960

$FLUX_{reqd}$ = 4.85E-05 m³/sec

h_{avg} = 0.20 m

PSR = 0.154

DLC	*1.496*
PSR	*0.154*
FS	*1.661*

thickness of cover soil = h = *1.30* m
length of slope measured along the geomembrane = L = *30* m
soil slope angle beneath the geomembrane = β = *14.0* deg. = *0 24* (rad.)
vertical height of the slope measured from the toe = H = *7.3* m
parallel submergence ratio = PSR = *0.154*
depth of the water surface measured from the geomembrane = h_w = *0.20* m

dry unit weight of the cover soil = γ_{dry} = 18.0 kN/m³
saturated unit weight of the cover soil = γ_{sat} = 21.0 kN/m³
unit weight of water = γ_w = 9.81 kN/m³
friction angle of the cover soil = φ = 30.0 ° = *0.52* (rad.)
Interface friction angle between cover soil and geomembrane = δ = 22.0 ° = *0.38* (rad)

Calculation of FS

Active Wedge:

W_A = 655 kN
U_n = 56.45 kN
U_h = 0.1973 kN
N_A = 579.04 kN

Passive Wedge:

W_P = 65.053 kN
U_v = 0.7912 kN

$$FS = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

where *a = .153.8*
b = -275.1
c = 32.7

FS = 1.661

Note: numbers in boxes are required input data
numbers in italics are calculated values

Constructed by Te-Yang Soong

Figure 10 - Cover soil stability analysis worksheet incorporating seepage forces with parallel seepage buildup

6.0 BEHAVIOR OF SELECTED CROSS SECTIONS

In this section, several cross sections typical of leachate collection systems and final cover systems will be analyzed. These were the two general categories of the different failures described in Table 1 and illustrated in Figure 2.

6.1 General Slope Configurations and Dimensions

So as to minimize the large number of variables that are possible, the general configuration shown in Figure 11a will be used. It consists of a geomembrane lined slope which is either 30 m long at a 3(H)-to-1(V) slope, or 100 m long at a 4(H)-to-1(V) slope. These are commonly seen geometric choices by designers of both leachate collection systems and final cover soil systems. To keep the number of variables at a minimum, a single type of cover soil is used having the following properties:

$\gamma_{dry} = 18$ kN/m^3

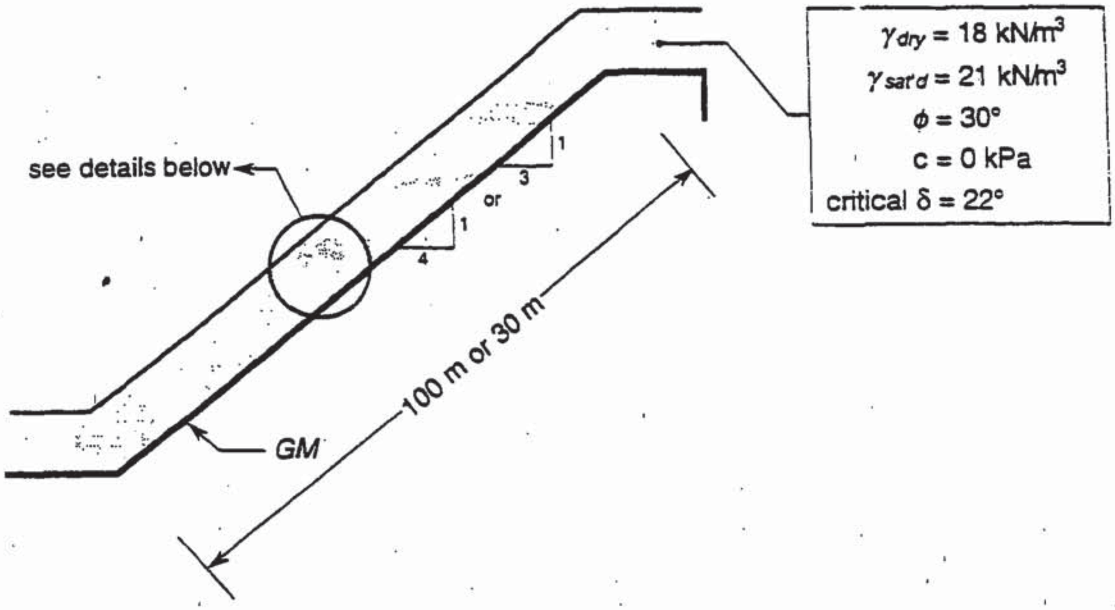
$\gamma_{sat'd} = 21$ kN/m^3

$\phi = 30$ deg. (soil-to-soil)

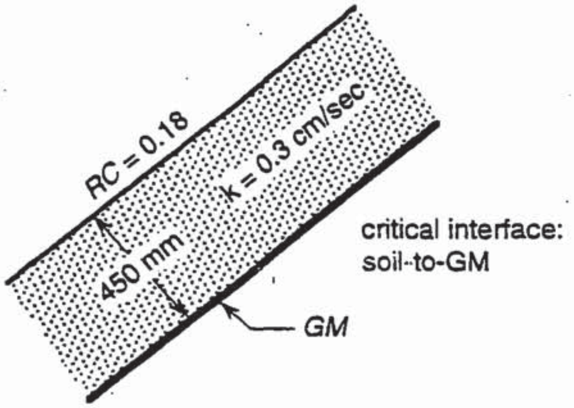
$c = 0$

$\delta = 22$ deg. (soil-to-geosynthetics)

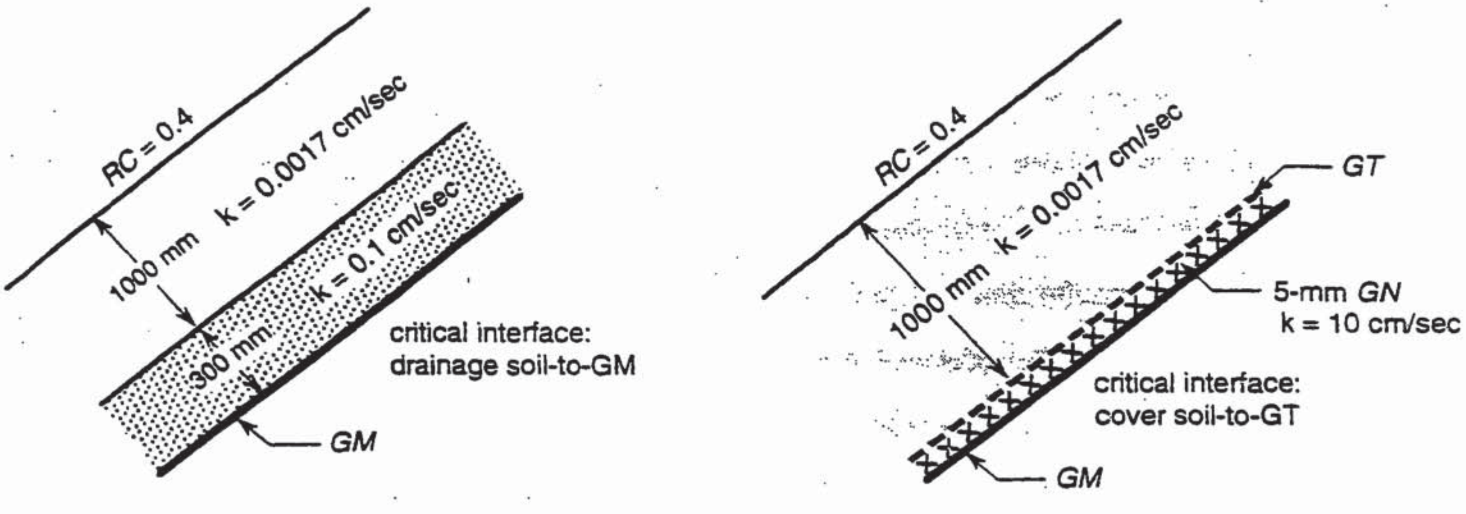
In order to typify a leachate collection system which will eventually be covered by waste, the drainage soil will be constant in its thickness and uncovered; see Figure 11b. For final cover systems, a drainage layer will be incorporated between the underlying geomembrane and the overlying cover soil. The drainage layer will be considered as being either natural soil (Figure 11c) or a geocomposite drain (Figure 11d). Thus, three separate cases will be analyzed; each having two geometric lengths and slope angles. Note that in all cases the precipitation is calculated on an hourly basis as described in Chapter 3 and uses the assumptions stated therein.



(a) General configuration and dimensions



(b) Leachate collection system



(c) Cover system over drainage soil

(d) Cover system over geosynthetic drain

Figure 11 - General configuration and specific dimensions of slopes to be analyzed.

6.2 Leachate Collection Systems

Using the general slope configuration shown in Figure 11a, along with the details shown in Figure 11b, an analysis for leachate collection soil stability was undertaken per the concepts developed in Chapters 3, 4 and 5. The homogeneous drainage layer is 450 mm thick and has a permeability of 0.3 cm/sec. This permeability was selected because it is the default value suggested in the HELP manual. A relatively low runoff coefficient of 0.18 is used since the soil is granular (sand or gravel) and will accept a large portion of the precipitation. The stability analysis has been performed for two separate geometric slopes:

- 100 m long slope at 4(H)-to-1(V)
- 30 m long slope at 3(H)-to-1(V)

The precipitation has been systematically varied between 5 mm/hr and 100 mm/hr. The results are presented in Figure 12 for drainage layer capacity (*DLC*), the resulting parallel submergence ratio (*PSR*), and the resulting slope's factor of safety (*FS*) against instability. The following trends can be observed.

- Only for relatively low values of precipitation, e.g., less than 5 mm/hr, is the *DLC* high, giving a low *PSR* and a *FS*-value greater than 1.2 for both slopes evaluated. Note that this relatively low value of factor of safety may be acceptable since the situation is temporary and stability will be established when waste is placed in the landfill.
- For precipitation values between approximately 15 and 65 mm/hr for the two slopes analyzed, the *DLC* drops below 1.0, the PSR is rapidly increasing and the FS-value is less than 1.0.
- The above trends, in *PSR* and *FS* values are very abrupt and they result in a discontinuity in the *PSR* and *FS* response curves when the *DLC* values drop lower than 1.0.

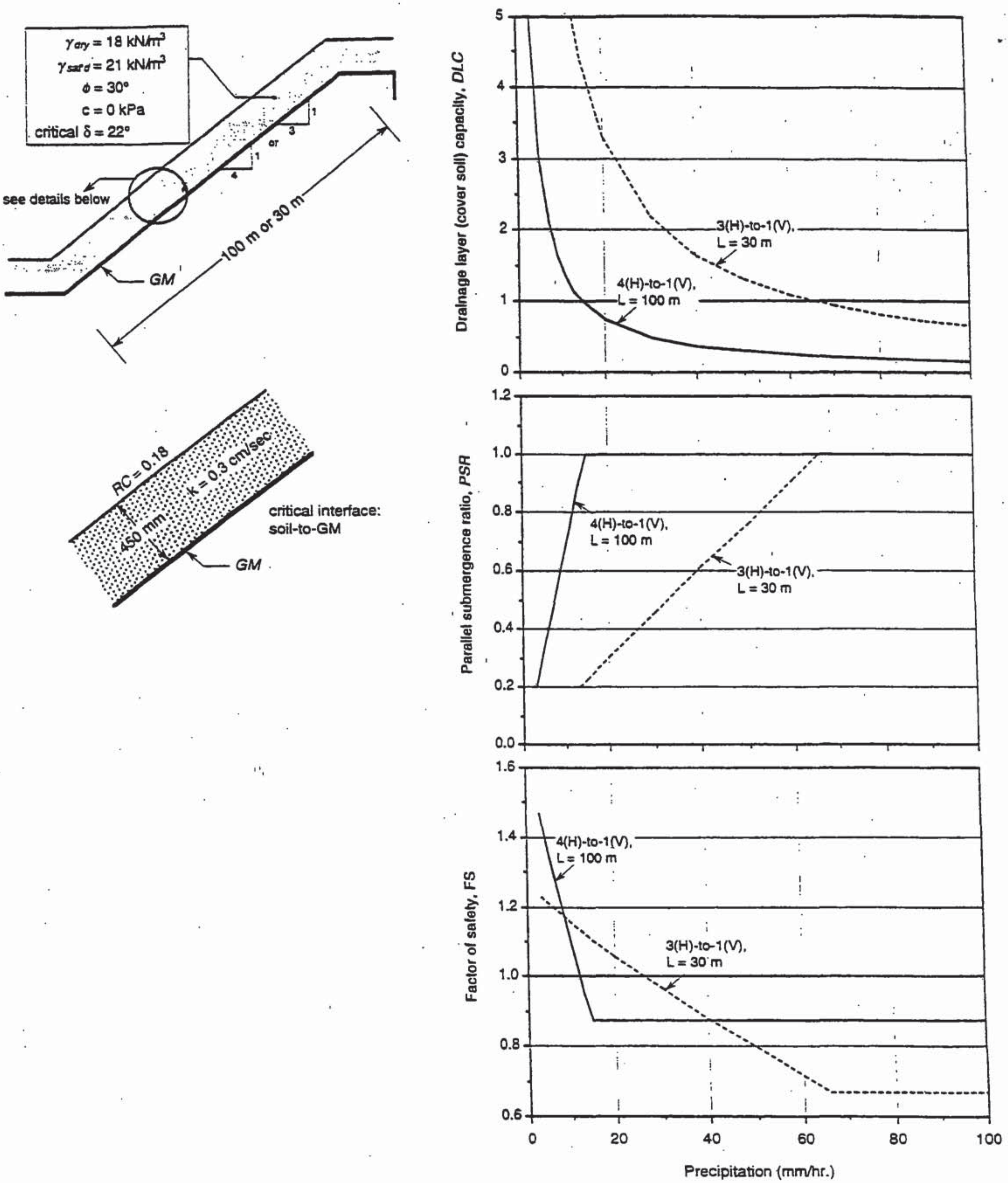


Figure 12 - Results of leachate collection system example problem

- The physical significance of the *DLC* decreasing to a value of 1.0, and continuing to values less than 1.0, is that water has filled the layer and will begin to flow on the surface of the leachate collection layer and add to the naturally occurring runoff.
- For the two geometric cross sections analyzed, the 100 m long 4(H)-to-1(V) slope reaches full drainage capacity sooner than the 30 m long 3(H)-to-1(V) slope, thus the FS-value is less than 1.0 at lower intensity precipitation storms.
- The reason for the above is more related to the length of slope than to its slope angle, since the require flux is cumulative over the length of slope. Long slope lengths will be seen to be very challenging in this regard.

6.3 Final Cover Systems Over Drainage Soils

Using the general slope configuration shown in Figure 11a, along with the details shown in Figure 11c, an analysis for stability was undertaken per the concepts developed in Chapters 3, 4 and 5. The cover soil is 1000 mm thick and has a permeability of 0.0017 cm/sec. This permeability is the default value of "SM" soils (commonly used for cover soils) suggested in the HELP manual. A relatively high runoff coefficient of 0.40 is used since the soil is fine grained and is probably somewhat cohesive. The underlying soil drainage layer is 300 mm thick and has a permeability of 0.1 cm/sec. This value of permeability is 10-times greater than the HELP manual's default value of "SP" soils and is used because the default value of 0.01 cm/sec *always results in FS-values less than 1.0.* The stability analysis has been performed for two separate geometric cases:

- 100 m long slope at 4(H)-to-1(V)
- 30 m long slope at 3(H)-to-1(V)

The precipitation has been systematically varied between 5 mm/hr and 100 mm/hr. The results are presented in Figure 13 for drainage layer capacity (*DLC*), the resulting parallel submergence ratio (*PSR*), and the resulting slope's factor of safety (*FS*) against instability. The following trends can be observed:

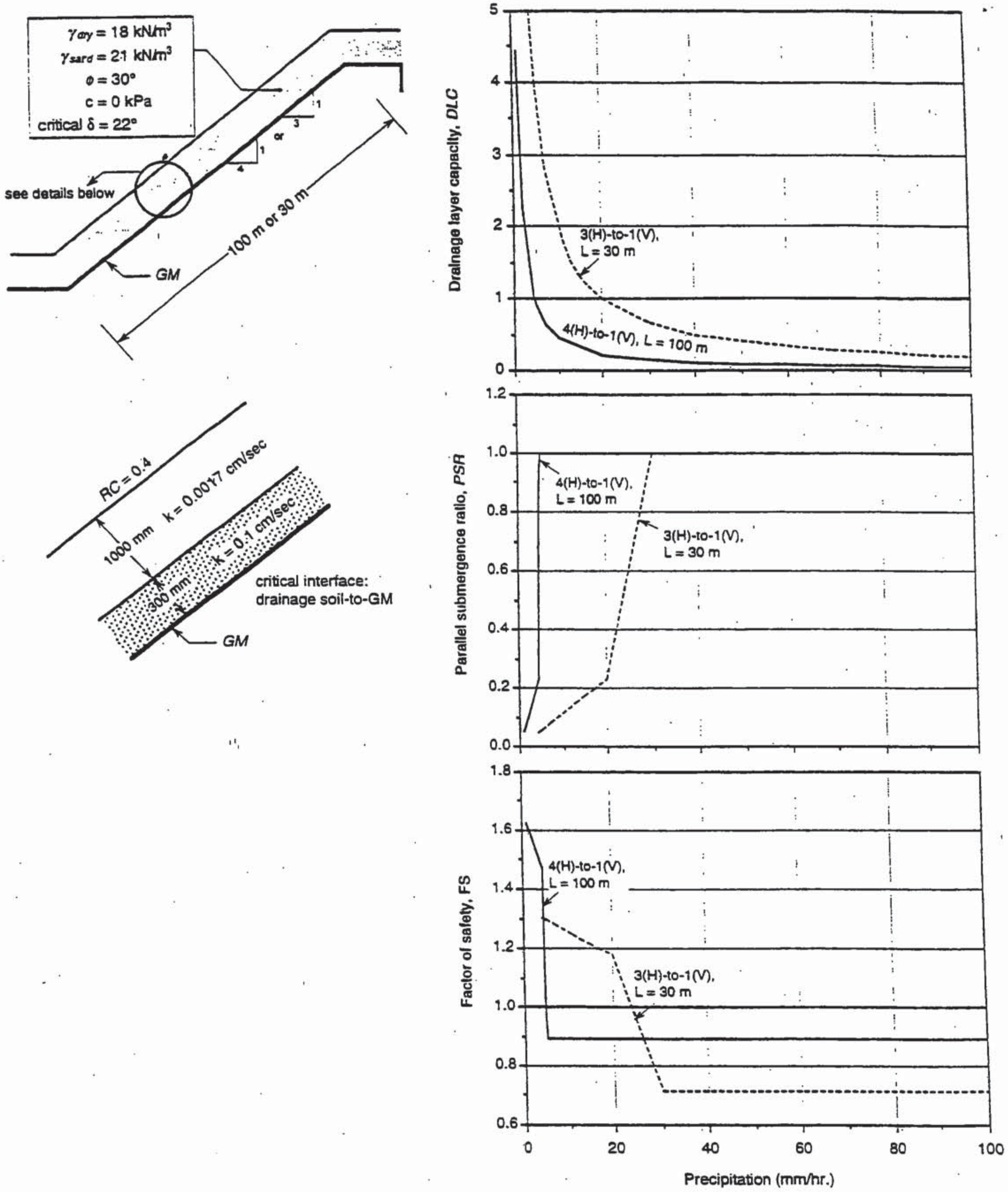


Figure 13 - Results of cover system over drainage soil example problem

- Only for relatively low values of precipitation, i.e., less than 5 mm/hr for the 100 m long 4(H)-to-1(V) slope and less than 20 mm/hr for the 30 m long 3(H)-to-1(V) slope. is the *DLC* high, giving low *PSR* values and *FS* values greater than 1.0.
- Furthermore, a *FS* greater than 1.5, which is recommended for permanent slopes, only occurs for the 100 m long 4(H)-to-1(V) slope at a precipitation value of less than 5 mm/hr.
- Water abruptly fills the drainage layer beyond this precipitation value rapidly decreasing the FS-value to less than 1.0.
- For the 30 m long 3(H)-to-1(V) slope between precipitation values of 5 and 20 mm/hr. the *DLC* falls to a value of 1.0. This increases the *PSR* and decreases the *FS* -value to 1.2. Water has completely filled the drainage layer at this point.
- As precipitation increases beyond 20 mm/hr for the 30 m long 3(H)-to-1(V) slope, the *DLC* becomes less than 1.0, the *PSR* increases rapidly to a value of 1.0 and the *FS*-values becomes less than 1.0.
- The above trends in *PSR* and *FS* values are very abrupt and result in discontinuities in the *PSR* and *FS* response curves when the *DLC* values drop lower than 1.0.
- When the *DLC* is less than 1.0, which occurs for both geometric slopes above 20 mm/hr, the phreatic surface rises above the drainage layer into the cover soil. This is clearly unacceptable insofar as slope stability is concerned. [Had the drainage layer permeability been used as 0.01 cm/sec, which is the U.S. EPA minimum technology guidance value and also the HELP default value, the FS-value would never have been acceptable.]
- For these two geometric considerations, the 100 m long 4(H)-to-1(V) slope is more sensitive to intense rain storms than is the 30 m long 3(H)-to-1(V) slope due to the cumulative nature of required flux value over the longer length of slope.

6.4 Final Cover Systems Over Geosynthetic Drains

Using the general slope configuration shown in Figure 11a, along with the details shown in Figure 11d, an analysis for stability was undertaken per the concepts developed in Chapters 3, 4 and 5. The cover soil is 1000 mm thick and has a permeability of 0.0017 cm/sec. This permeability is the default value suggested in the HELP manual for "SM" soils, which are commonly used for cover soils. A relatively high runoff coefficient of 0.40 is used since the soil is fine grained and probably somewhat cohesive. The underlying geosynthetic drainage layer is 5.0 mm thick and has a permeability of 10 cm/sec. This value is not available as a default value in the HELP manual and must be evaluated for the candidate geosynthetic drainage material as illustrated in Figure 4. The stability analysis has been performed for two separate cases:

- 100 m long slope at 4(H)-to-1(V)
- 30 m long slope at 3(H)-to-1(V)

The precipitation has been systematically varied between 5 mm/hr and 100 mm/hr. The results are presented in Figure 14 for drainage layer capacity (*DLC*), the resulting parallel submergence ratio (*PSR*), and the resulting slope's factor of safety (*FS*) against instability. The following trends can be observed:

- Only for relatively low values of precipitation, i.e., less than 10 mm/hr for the 100 m long 4(H)-to-1(V) slope and 30 mm/hr for the 30 m long 3(H)-to-1(V) slope, is the *DLC* high, giving a near zero *PSR* value and *FS* -values of 1.6 and 1.3, respectively.
- At the above precipitation limits the *PSR* response curves go from near zero to 1.0 very quickly because the geosynthetic drains are quite thin with respect to soil drainage layers and they fill very rapidly.
- At the above precipitation limits, the *FS*-values drop rapidly to values less than 1.0.
- When the *DLC* is less than 1.0, the phreatic surface rises above the geocomposite drainage layer into the cover soil. This is clearly unacceptable insofar as slope stability is concerned.

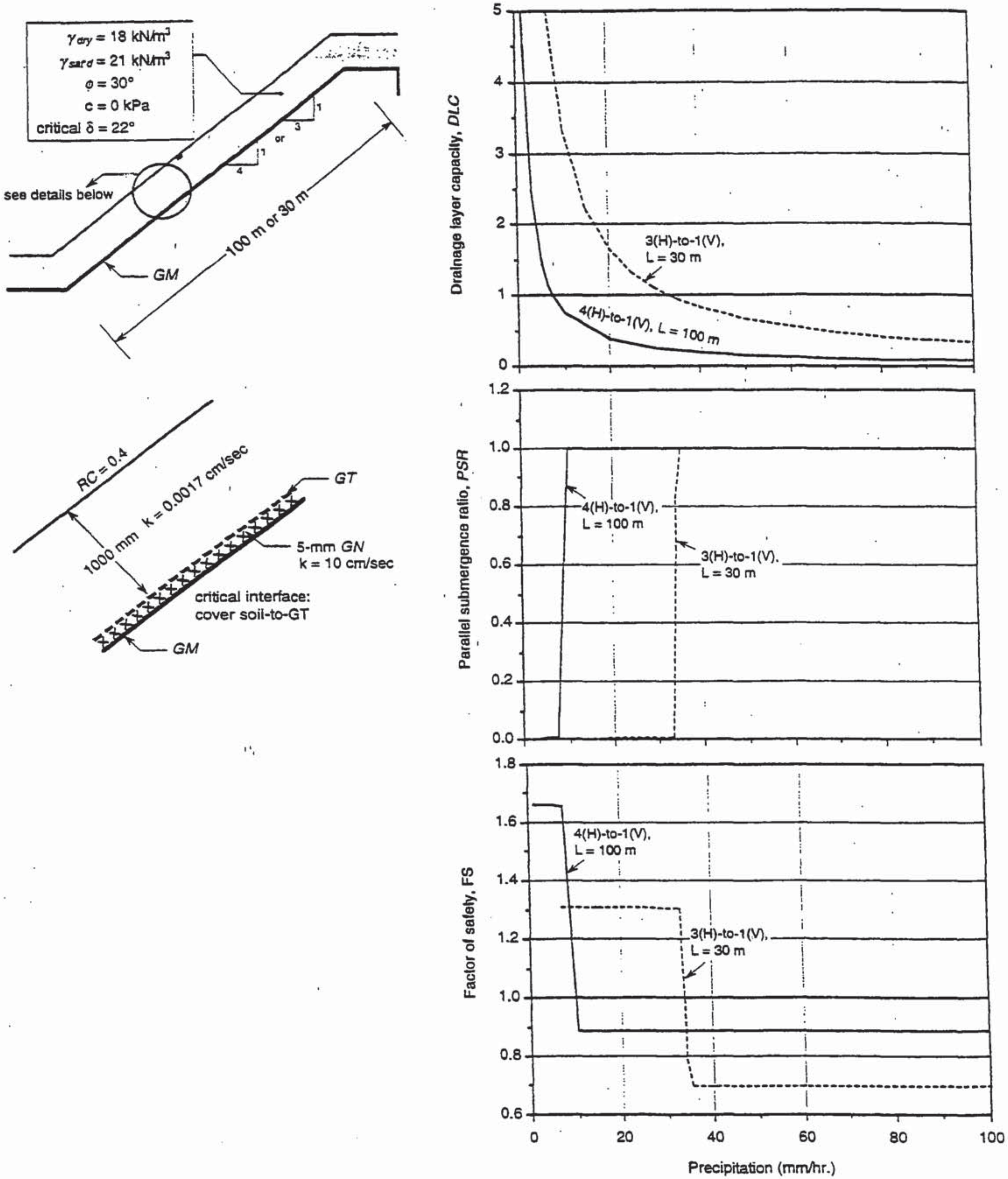


Figure 14 - Results of cover system over geosynthetic drain example problem

- For these two slopes, the 100 m long 4(H)-to-1(V) slope is somewhat more sensitive to intense rain storms than is the 30 m long 3(H)-to-1(V) slope since the required flux is cumulative over the relatively long slope length.

7.0 PARAMETRIC EVALUATIONS

Based on discontinuous trends in drainage layer capacity (*DLC*), parallel submergence ratio (*PSR*) and factor of safety (*FS*) in the previous section for only two slope conditions, it should be obvious that the selection of variables for illustrative purposes is very sensitive and quite subjective. Rather than select specific conditions, it is perhaps instructive to conduct a parametric evaluation on a range of variables. This section presents this type of parametric variation for the three profiles shown in Figures 11b, c and d. It includes variation of precipitation between 5 and 100 mm/hr, as well as variation in other selected variables.

7.1 Leachate Collection Systems

Using the general slope configuration shown in Figure 11a, along with details shown in Figure 11b, a parametric evaluation of leachate collection systems was undertaken per Table 16.

Table 16 - Conditions Evaluated for Leachate Collection Systems

Parameter Evaluated (in addition to precipitation)	Conditions				
	P (mm/hr.)	$k_{d.s.}$ (cm/sec)	$h_{d.s.}$ (mm)	L (m)	β (deg.)
Permeability of drainage soil, $k_{d.s.}$	5-100	**10^{-3}-10^{1}**	1000	100	14.0
Thickness of drainage soil, $h_{d.s}$	5-100	10^{-1}	**300-2000**	100	14.0
Length of slope, *L*	5-100	10^{-1}	1000	**10-300**	14.0
Slope angle, β	5-100	10^{-1}	1000	100	**2.9-40.0**

Values held constant for all iterations are as follows:

γ_{dry} = 18 kN/m^3

$\gamma_{sat'd}$ = 21 kN/m^3

ϕ = 30 deg. (soil-to-soil)

δ = 22 deg. (soil-to-geomembrane)

RC = 0.18

The response for the first variation in permeability of leachate collection soil between 0.001 and 10 cm/sec is given in Figure 15. The results are striking.

- With a permeability of leachate collection drainage soil equal, or less, than 0.05 cm/sec, the FS-values for all precipitation values, even as low as 5 mm/hr, are always less than one, signifying instability.
- Paradoxically, a permeability of 0.01 cm/sec drainage soil is the value noted in U.S. EPA regulations as being minimum technology guidance. As expected, this value is used widely. Here it is seen that such low permeability drainage soil will always lead to seepage induced slope instability under the conditions assumed herein.
- Depending on the precipitation intensity, *FS*-values of 1.5 require drainage soil *k*-values of 0.3 to 6.0 cm/sec.
- Referring back to Table 3, this value of permeability can only be achieved using "GP" or "GW" gravels, and possibly "SP" sand. However, the poorly graded gravels and sands are often unstable, leaving only well graded gravel as being the candidate material for leachate collection layers of the type being analyzed.
- The above gravel is typical of AASHTO #1, #3 or #5. In general, AASHTO #57 must be screened of its fines to meet such a permeability requirement.
- Of course, with such coarse sized gravel the underlying geomembrane must be protected using a thick needle punched nonwoven geotextile, or equivalent, see Koerner, et al. (1996).
- Furthermore, the issue of placing waste directly on the surface of the gravel versus using a geotextile filter, must be carefully considered, see Koerner, G. R. et al. (1993).

The second variation in the leachate collection system profile varied the thickness of the drainage layer between 300 and 2000 mm. The response curves are given in Figure 16. At a constant drainage layer permeability value of 0.1 cm/sec, essentially all of the resulting *FS*-values are less than 1.5. It should be noted that the minimum technology guidance of the U.S.

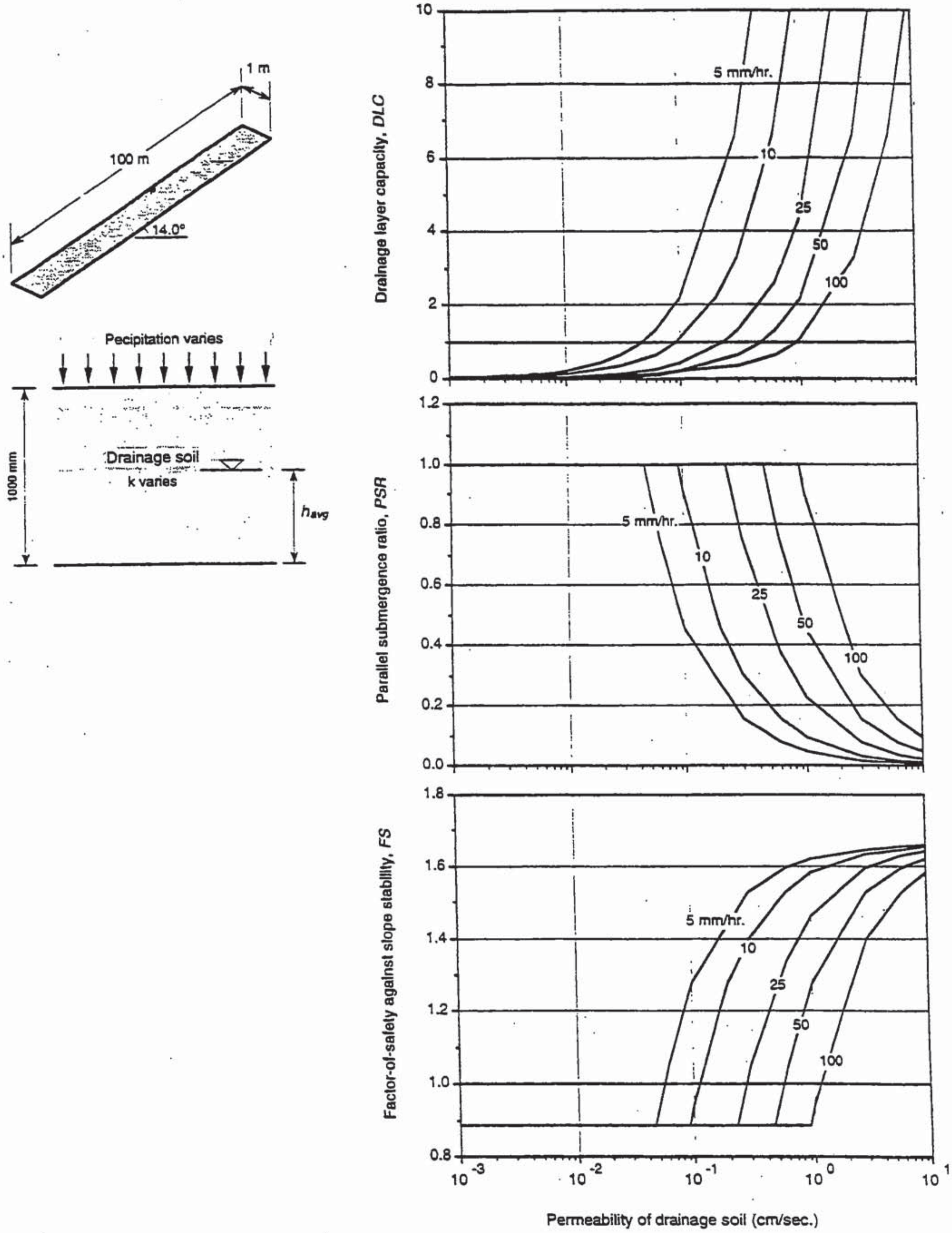


Figure 15 - Parametric study of leachate collection system: permeability variation

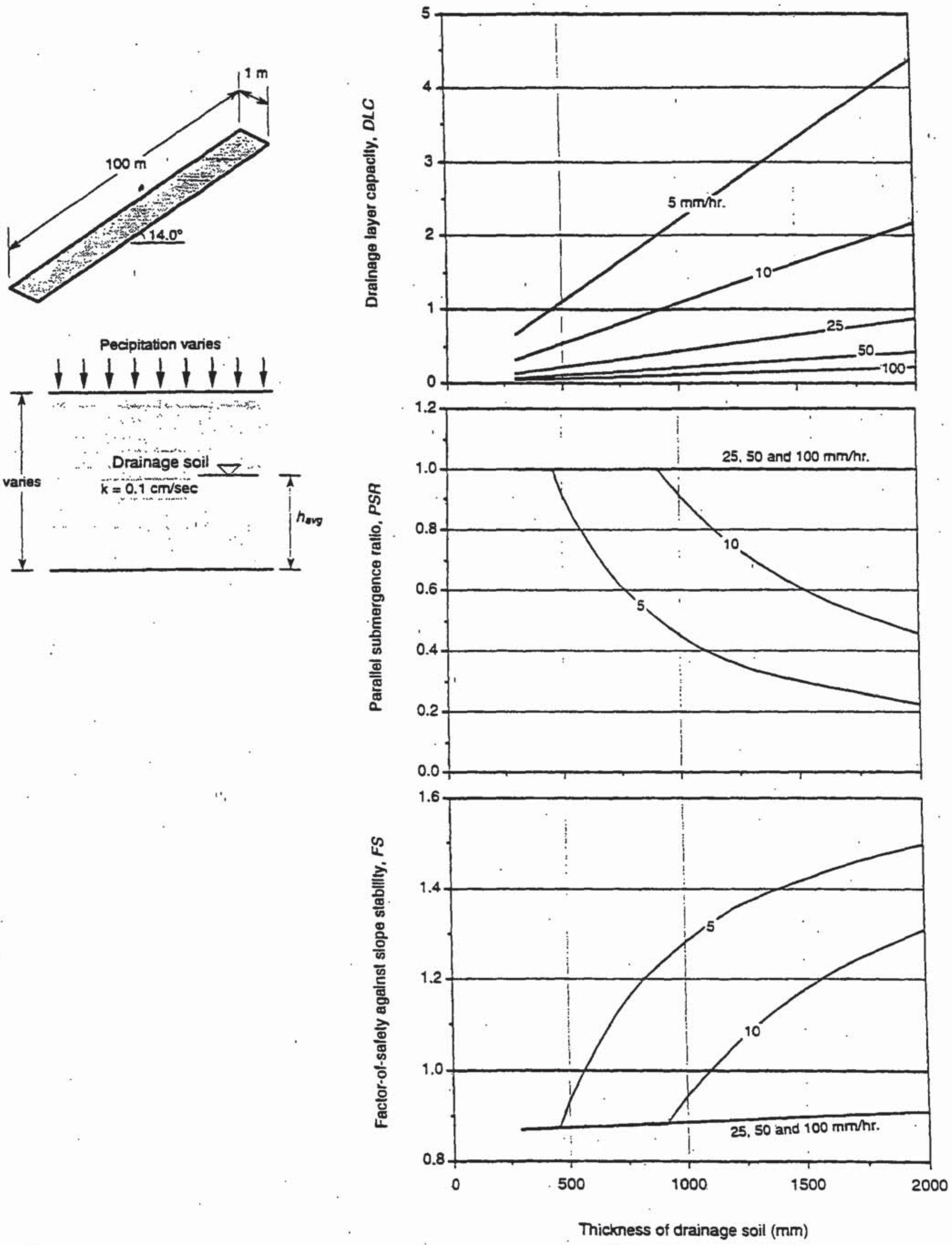


Figure 16 - Parametric study of leachate collection system: thickness variation

EPA regulations is an order of magnitude lower, i.e., 0.01 cm/sec, which (if analyzed) would produce proportionally even lower *FS*-values.

The third variation in the leachate collection system profile varied the slope length between 10 and 300 m. The response curves are given in Figure 17. With a constant drainage layer permeability value of 0.1 cm/sec, the *FS*-values are only acceptable for slope lengths between 10 and 50 m, for precipitation values between 100 and 5 mm/hr, respectively. In such cases, the storm intensity is a significant factor and therefore, careful selection of the design storm is necessary.

As discussed a number of times in Section 6.0 for the two example slopes of 30 m and 100m lengths, the longer slopes with cumulatively increasing required flux values are generally troublesome. If long slope lengths are necessary, it is suggested that they be segmented by berms and that the drainage be removed at each berm level. An illustration will be given later.

The fourth variation in the leachate collection system profile varied the slope angle between 2.9 and 40 deg. The response curves are given in Figure 18. With a constant permeability 0.1 cm/sec, it is seen that only relatively flat slopes are stable, e.g., less than approximately 10 deg. which is approximately 5(H)-to-1(V). The storm intensity is only nominally a factor, the major constituent being the permeability of the drainage layer as noted earlier in this section.

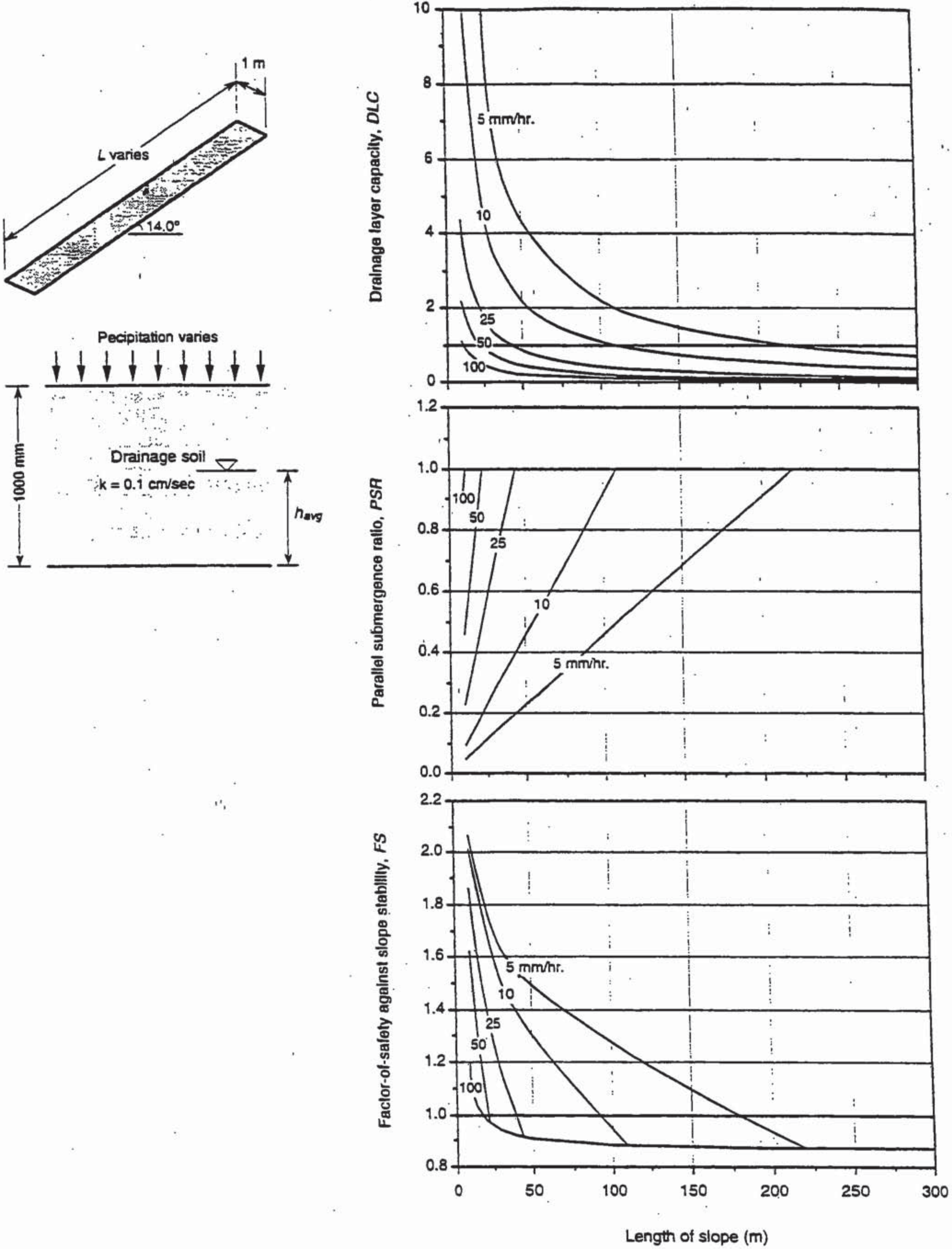


Figure 17 - Parametric study of leachate collection system: slope length variation

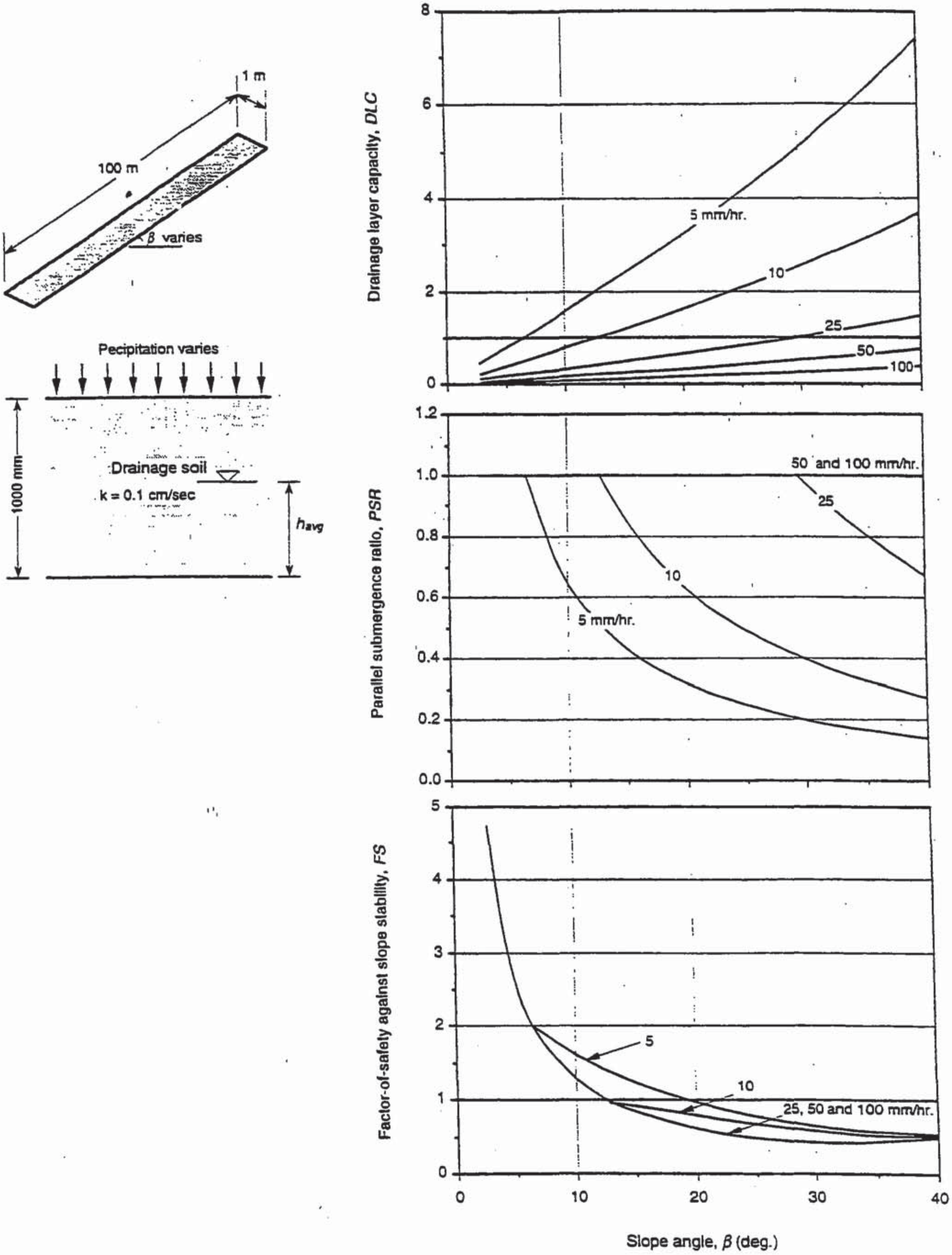


Figure 18 - Parametric study of leachate collection system: slope angle variation

7.2 Final Cover Systems Over Drainage Soils

Using the general slope configuration shown in Figure 11a, along with details shown in Figure 11c, a parametric evaluation of cover systems over drainage soils was undertaken per Table 17.

Table 17 - Conditions Evaluated for Cover Systems Over Drainage Soils

Parameter Evaluated (in addition to precipitation)	Conditions				
	P (mm/hr.)	$k_{d.s.}$ (cm/sec)	$k_{c.s}$ (cm/sec)	L (m)	β (deg.)
Permeability of drainage soil, $k_{d.s.}$	5-100	**10^{-2}-10^{1}**	10^{-3}	100	14.0
Permeability of cover soil, $k_{c.s.}$	5-100	10^{-1}	**10^{-5}-10^{-1}**	100	14.0
Length of slope, *L*	5-100	10^{-1}	10^{-3}	**10-300**	14.0
Slope angle, β	5-100	10^{-1}	10^{-3}	100	**2.9-40.0**

Values held constant for all iterations are as follows:

γ_{dry} = 18 kN/m^3

$\gamma_{sat'd}$ = 21 kN/m^3

ϕ = 30 deg. (soil-to-soil)

δ = 22 deg. (soil-to-geomembrane)

RC = 0.4

$t_{cover\ soil}$ = 1000 mm

$t_{drainage\ soil}$ = 300 mm

The response for the first variation of drainage soil permeability between 0.01 and 10 cm/sec is given in Figure 19. As with the leachate collection system described in section 7.1, the results are striking.

- Drainage soil permeabilities less than 0.1 cm/sec result in *DLC*-values less than 1.0 (i.e., the drainage layer is at full capacity), producing *PSR*-values equal to 1.0 and the *FS*-values are always less than 1.0.

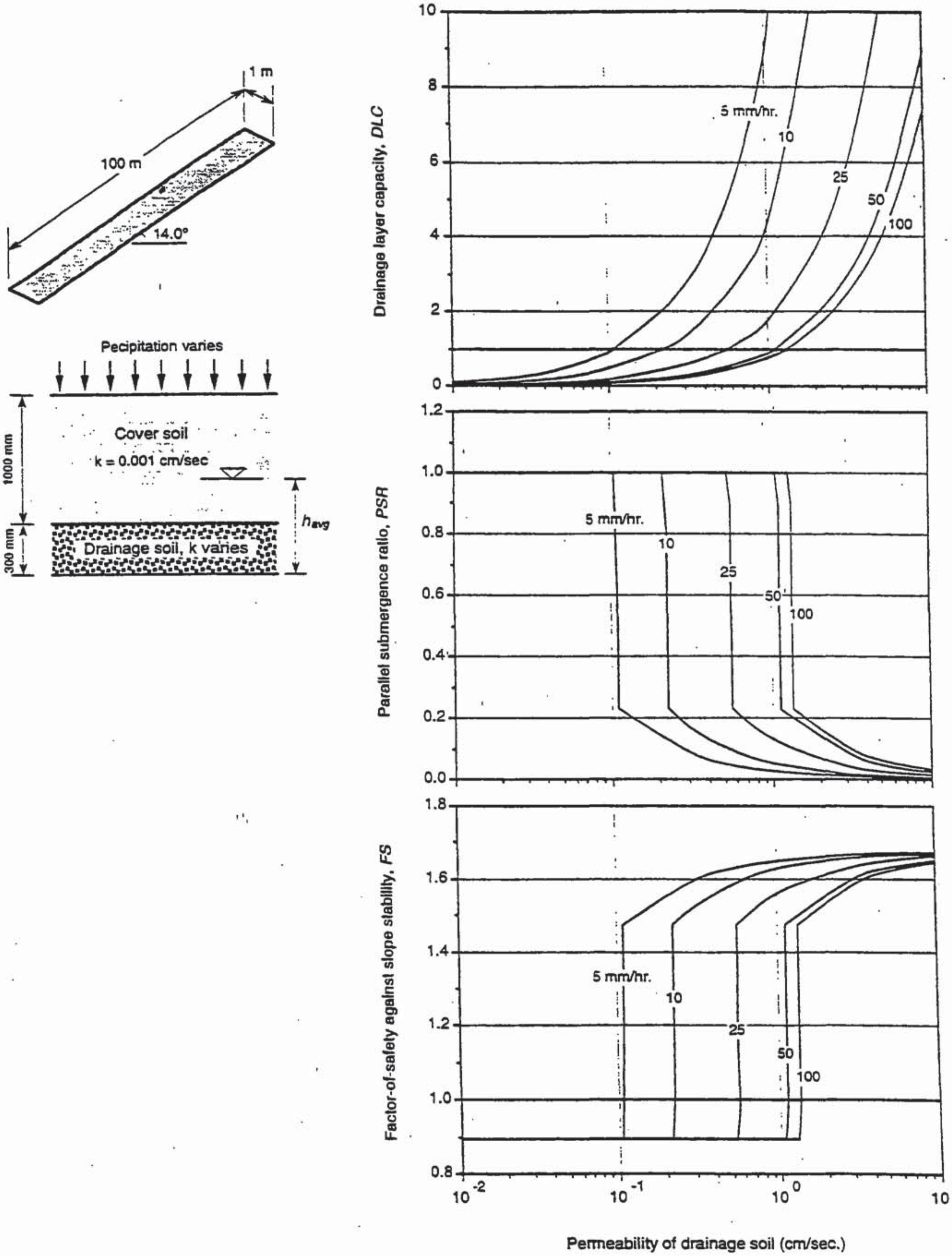


Figure 19 - Parametric study of cover system over drainage soil: permeability (drainage soil) variation

- The *FS*-values are less than 1.0 even for the 5 mm/hr precipitation, which is the lowest value analyzed.
- As precipitation increases, the permeability of the drainage layer must also increase for suitable *FS*-values. For example, for a factor of safety of 1.5:
    - A 5 mm/hr precipitation storm requires $k \geq 0.12$ cm/sec
    - A 10 mm/hr precipitation storm requires $k \geq 0.22$ cm/sec
    - A 25 mm/hr precipitation storm requires $k \geq 0.55$ cm/sec
    - A 50 mm/hr precipitation storm requires $k \geq 1.3$ cm/sec
    - A 100 mm/hr precipitation storm requires $k \geq 1.5$ cm/sec
- The implication of the above is that coarse sand or gravel must be used as discussed in section 7.1.
- Alternatively, the permeability of the cover soil could be reduced thereby allowing less percolation through this layer. (This alternative is treated in the next section.) Of course, this strategy will add to the runoff value and potentially create erosion of the cover soil, but this issue not treated in this report.

The second variation in the cover soil over drainage soil profile varied the permeability of the cover soil between 10^{-5} and 10^{-1} cm/sec. The response curves are given in Figure 20. The curves are somewhat challenging to interpret.

At cover soil permeability values less than 7×10^{-5} cm/sec, the *FS*-values can be quite reasonable. This permeability is sufficiently low that the underlying drainage layer ($k = 0.1$ cm/sec) can handle the relatively low percolation and its subsequent flux requirement. Similarly, at very high cover soil permeability values of greater than 0.05 cm/sec, the *FS*-values can also be acceptable but only for light precipitation, i.e., less than 5 mm/hr. In this case there is drainage within the cover soil which adds to the capability of the drainage layer. When the permeability of cover soil increases to 0.1 cm/sec, the entire profile becomes a homogeneous drainage layer. For cover soil permeability ranges between 7×10^{-5} and 5×10^{-2} cm/sec, however, unacceptable *FS*-values result under all precipitation conditions. Unfortunately, this is a very common

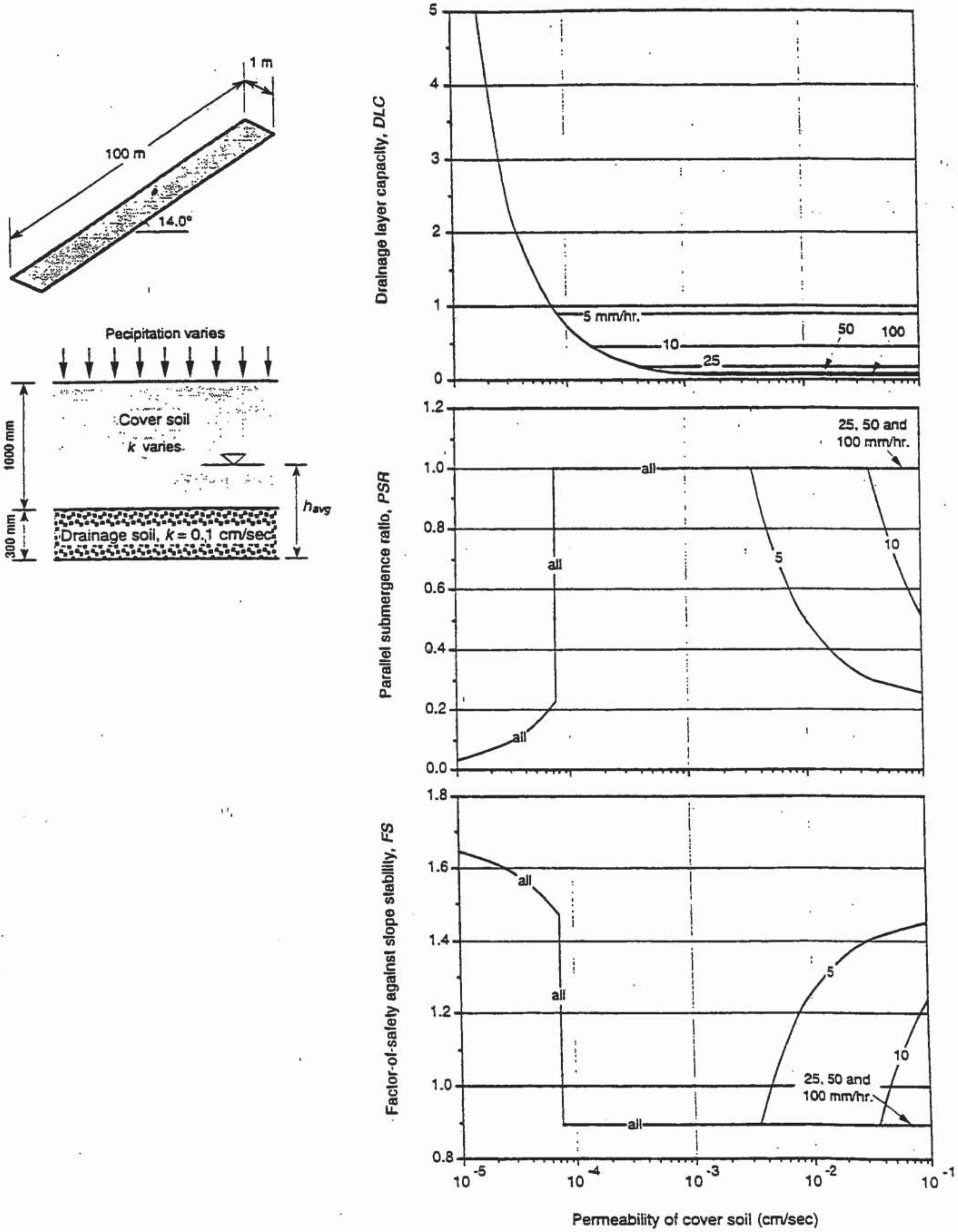


Figure 20 - Parametric study of cover system over drainage soil: permeability (cover soil) variation

permeability range for cover soil materials which are usually on-site borrow soils. If only such cover soils were available, the design strategy would be to increase the drainage layer capacity or shorten the slope length with benches.

The third variation in the cover soil drainage soil profile varied the length of slope from 10 to 300 m. The response curves are given in Figure 21. Here it is seen that slope lengths of less than 80 m can be acceptable depending on the magnitude of precipitation. The higher the precipitation, the shorter the slope must be in order to result in an acceptable *FS*-value, for example:

- For 5 mm/hr precipitation, the slope can be up to 80 m in length.
- For 10 mm/hr precipitation, the slope can be up to 45 m in length.
- For 25 mm/hr precipitation, the slope can be up to 20 m in length.
- For greater than 25 mm/hr precipitation, the slope must be less than 20 m in length.

The fourth variation in the cover soil over drainage soil profile varied the slope angle from 2.9 to 40 degrees. The response curves are given in Figure 22. Note that the *FS*-values are unacceptable for all cases except very shallow slope angles, e.g., less than 10 degrees (i.e., less than 5(H)-to-1(V)). The reason for this response is (a) the poorly selected permeability value of cover soil (held constant at 0.001 cm/sec) which is in the unacceptable mid-range in Figure 20, and (b) the unacceptably low value of drainage soil permeability (held constant at 0.1 cm/sec), recall Figure 19.

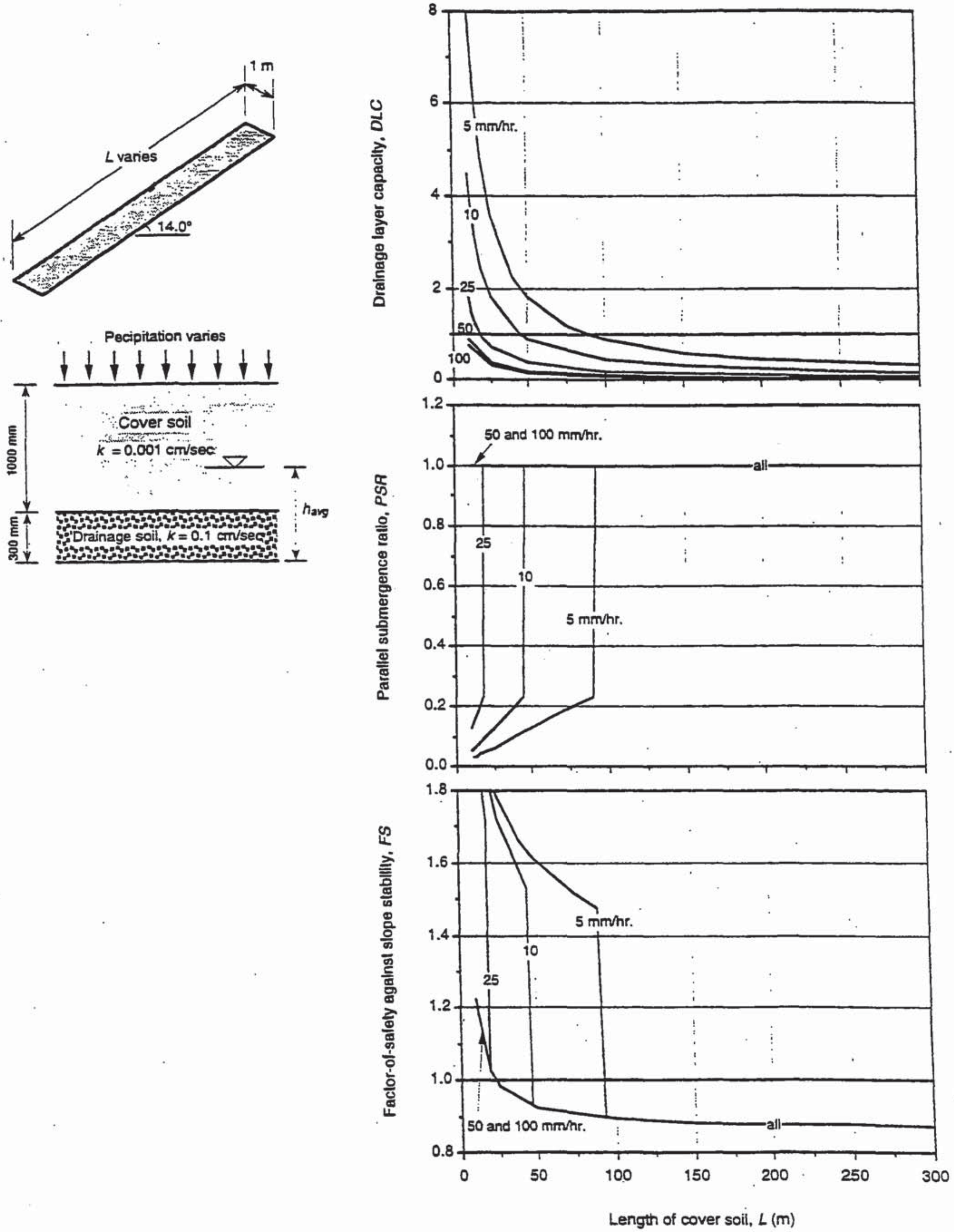


Figure 21 - Parametric study of cover system over drainage soil: slope length variation

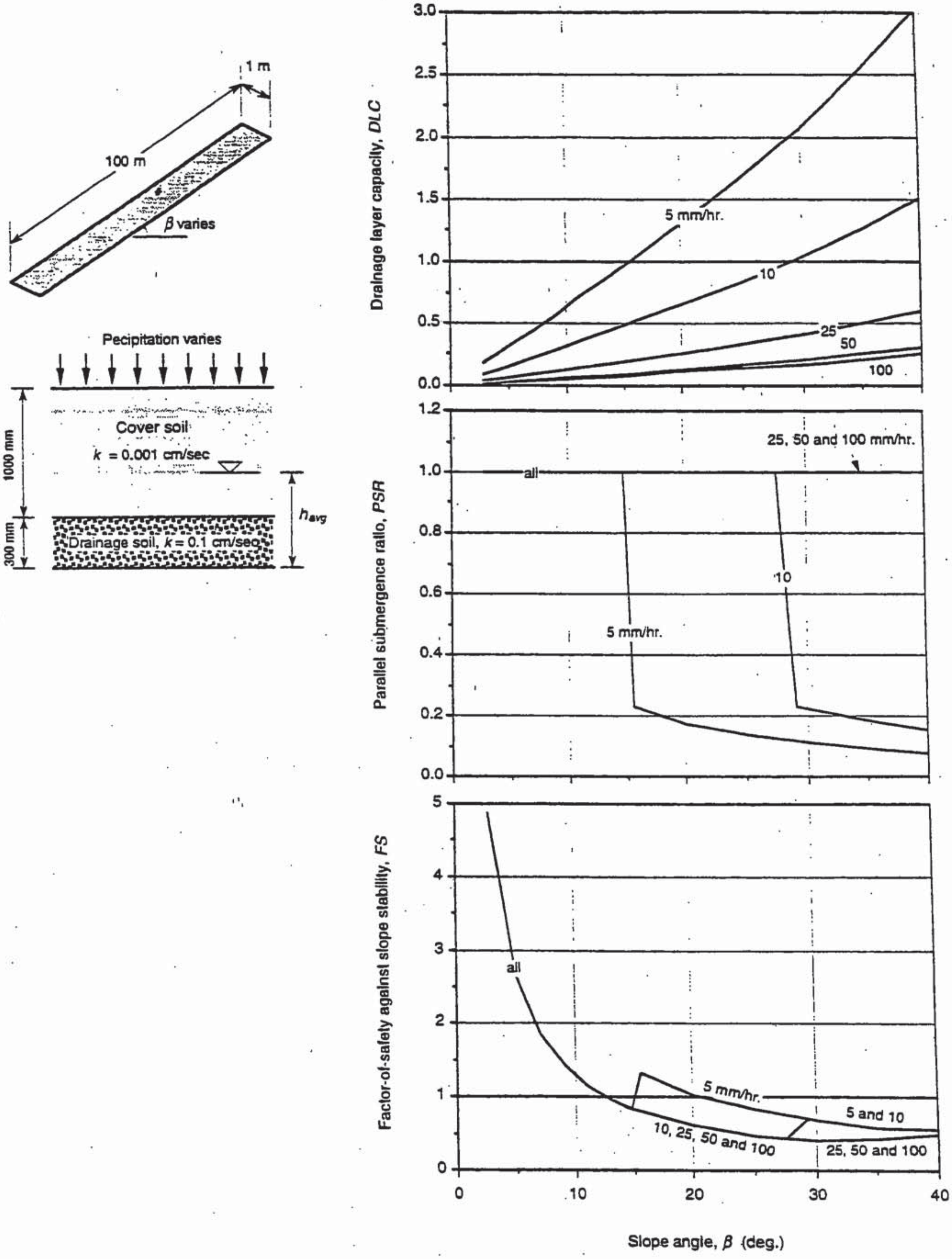


Figure 22 - Parametric study of cover system over drainage soil: slope angle variation

7.3 Final Cover Systems Over Geosynthetic Drains

Using the slope configuration shown in Figure 11a, along with details shown in Figure 11d, a parametric evaluation of cover systems over geosynthetic drains was undertaken per Table 18.

Table 18 - Conditions Evaluated for Cover Soil Systems Over Geosynthetic Drains

Parameter Evaluated (in addition to precipitation)	Conditions						
	P (mm/hr.)	k_{GS} (cm/sec)	$k_{c.s}$ (cm/sec)	L (m)	β (deg)	$h_{c.s}$ (mm)	t_{GS} (mm)
Rainfall intensity, P	1-100	0.6^{GS1}	10^{-3}	100	14.0	1000	5.5^{GS1}
Permeability of cover soil, $k_{c.s.}$	60	10^{GS2}	**10^{-5}-10^{-1}**	100	14.0	1000	5.5^{GS2}
Length of slope, *L*	60	12^{GS3}	10^{-3}	**10-300**	14.0	1000	14.0^{GS3}
Slope angle, β	60		10^{-3}	100	**2.9-40.0**	1000	

Values held constant for all iterations are as follows:

γ_{dry} = 18 kN/m^3

$\gamma_{sat'd}$ = 21 kN/m^3

ϕ = 30 deg. (soil-to-soil)

δ = 22 deg. (soil-to-geocomposite)

RC = 0.4

$t_{cover\ soil}$ = 1000 mm

$k_{cover\ soil}$ = 0.001 cm/sec

GS1 = GT/GN/GT composite[*]

GS2 = plate/GN/plate[*]

GS3 = sheet drain geocomposite[*]

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[*] All geosynthetic drains were evaluated at 25 kPa normal stress and reduced by a cumulative reduction factor of 5.0.

The response for the first variation of precipitation intensity between 1 and 100 mm/hr is given in Figure 23. The response shows that only storm events of less than approximately 30 mm/hr can be handled by the *GS3* drain and approximately 8 mm/hr for the *GS2* drain. The *GS1* drain is unacceptable under all conditions.

The second variation in the cover soil over geosynthetic drain profile varied the permeability of the cover soil from 10^{-5} to 10^{-1} cm/sec. The rainfall intensity was held constant at 60 mm/hr. The response curves are given in Figure 24. Here it is seen that both *GS2* and *GS3* geocomposite drains result in acceptable *FS*-values when the permeability of the cover soil is less than 1.5×10^{-4} cm/sec and 4.5×10^{-4} cm/sec, respectively. At these relatively low values of cover soil permeability the percolation values are sufficiently low that the required flux can be handled. The *GS1* geocomposite is not acceptable at any cover soil permeability value.

The third variation in the cover soil over geosynthetic drain profile varied the length of slope from 10 to 300 m. The rainfall intensity was held constant at 60 mm/hr. The response curves are given in Figure 25. The cover soil permeability was held constant at 0.001 cm/sec. The curves indicate that the *FS*-values are only acceptable for the *GS2* and *GS3* geocomposites at slope lengths of 15 m and 40 m, respectively. Again, the *GS1* drain is never acceptable under these conditions.

The fourth variation in the cover soil over geosynthetic drain profile varied the slope angle between 2.9 and 40 degrees. The rainfall intensity was held constant at 60 mm/hr. The response curves are given in Figure 26. Again, the cover soil permeability was held at 0.001 cm/sec. The resulting *FS*-values are only acceptable at relative shallow slope angles, e.g., less than 9 deg., i.e., approximately 5(H)-to-1(V). All three geosynthetic drains give similar response up to this slope angle. The behavior is dominated by the slope angle, but steeper slopes could be accommodated by cover soil permeability values lower than 0.001 cm/sec (allowing for less percolation) or higher capacity geosynthetic drains (allowing for greater flux capacity).

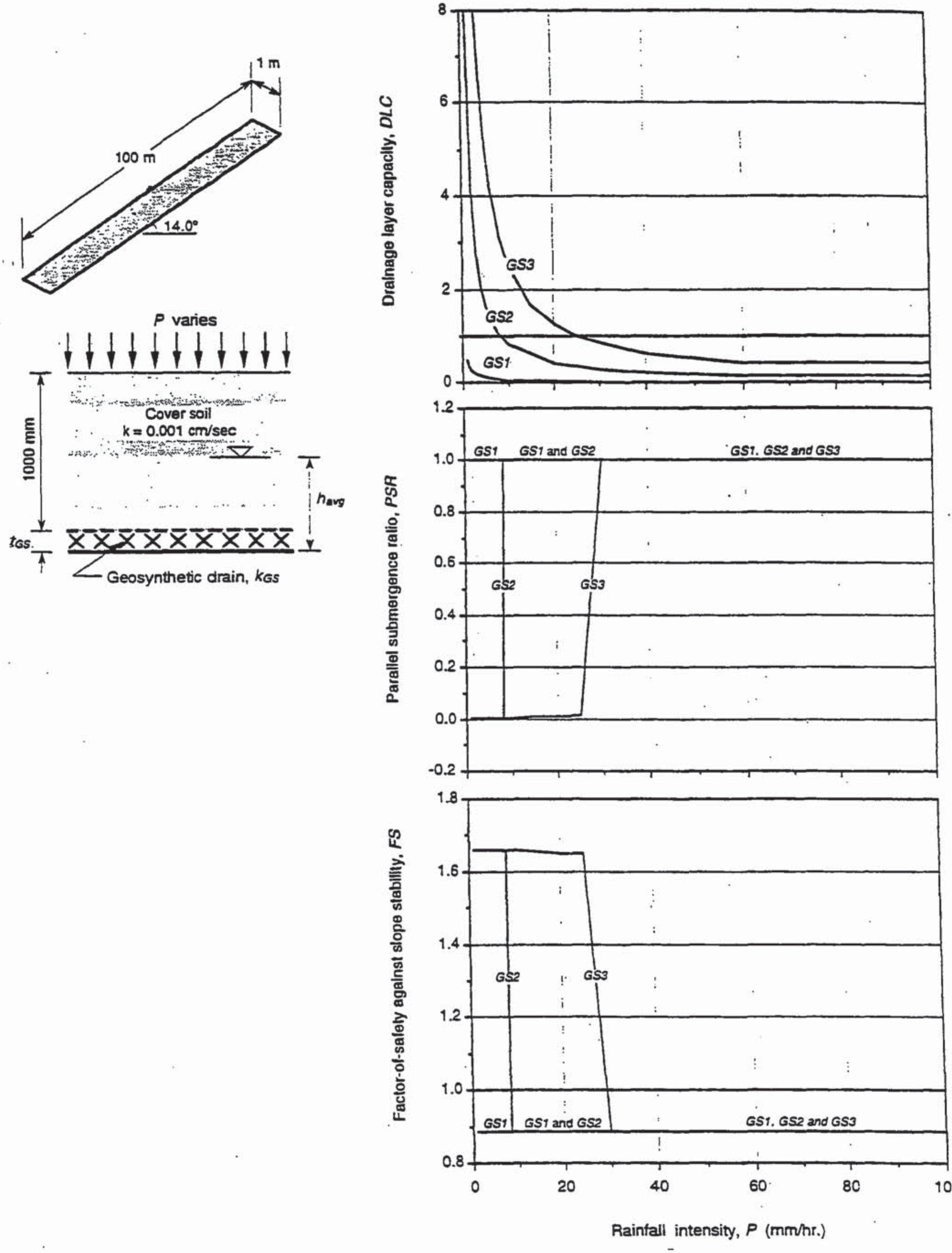


Figure 23 - Parametric study of cover system over geosynthetic drain: rainfall intensity variation

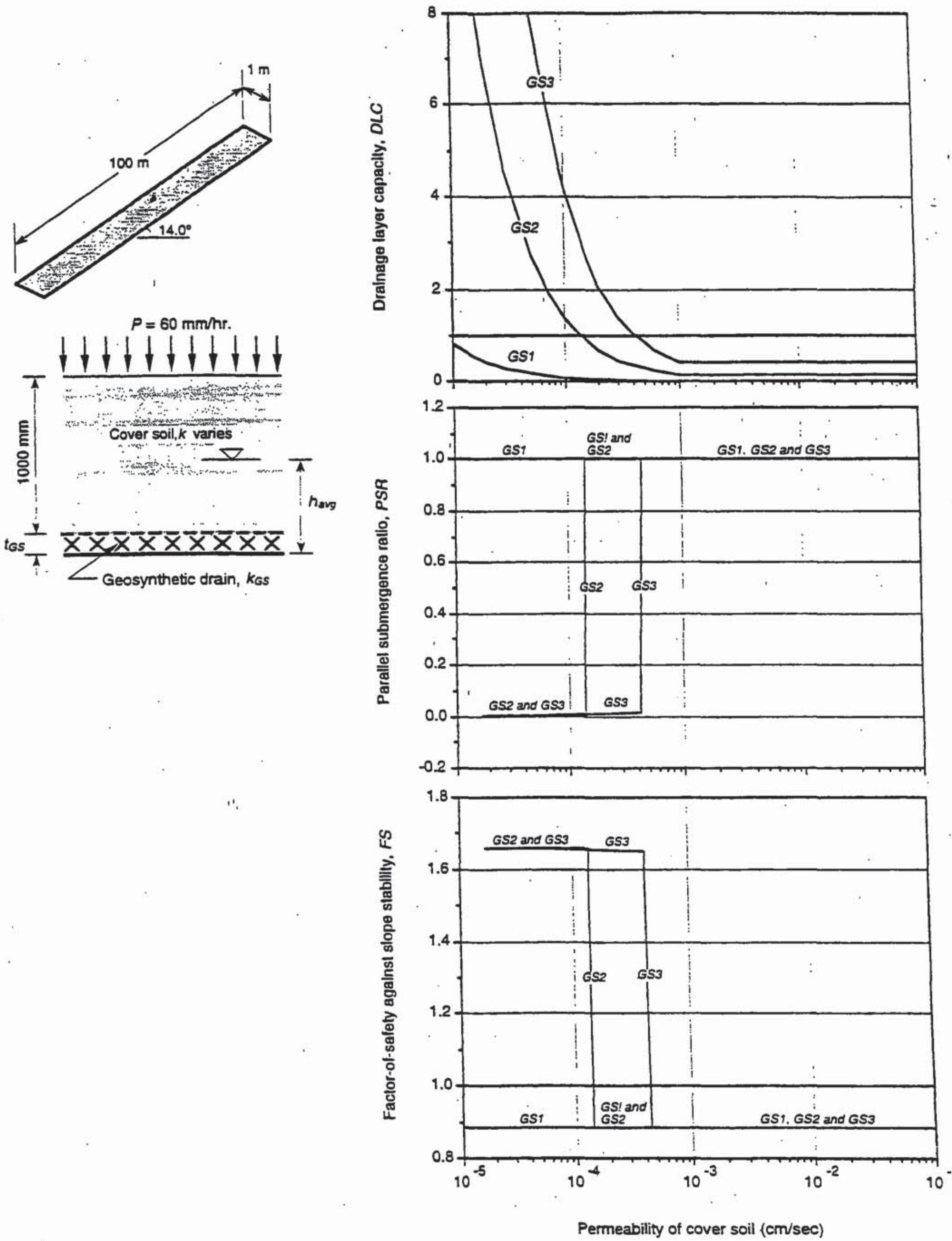


Figure 24 - Parametric study of cover system over geosynthetic drain: permeability (cover soil) variation

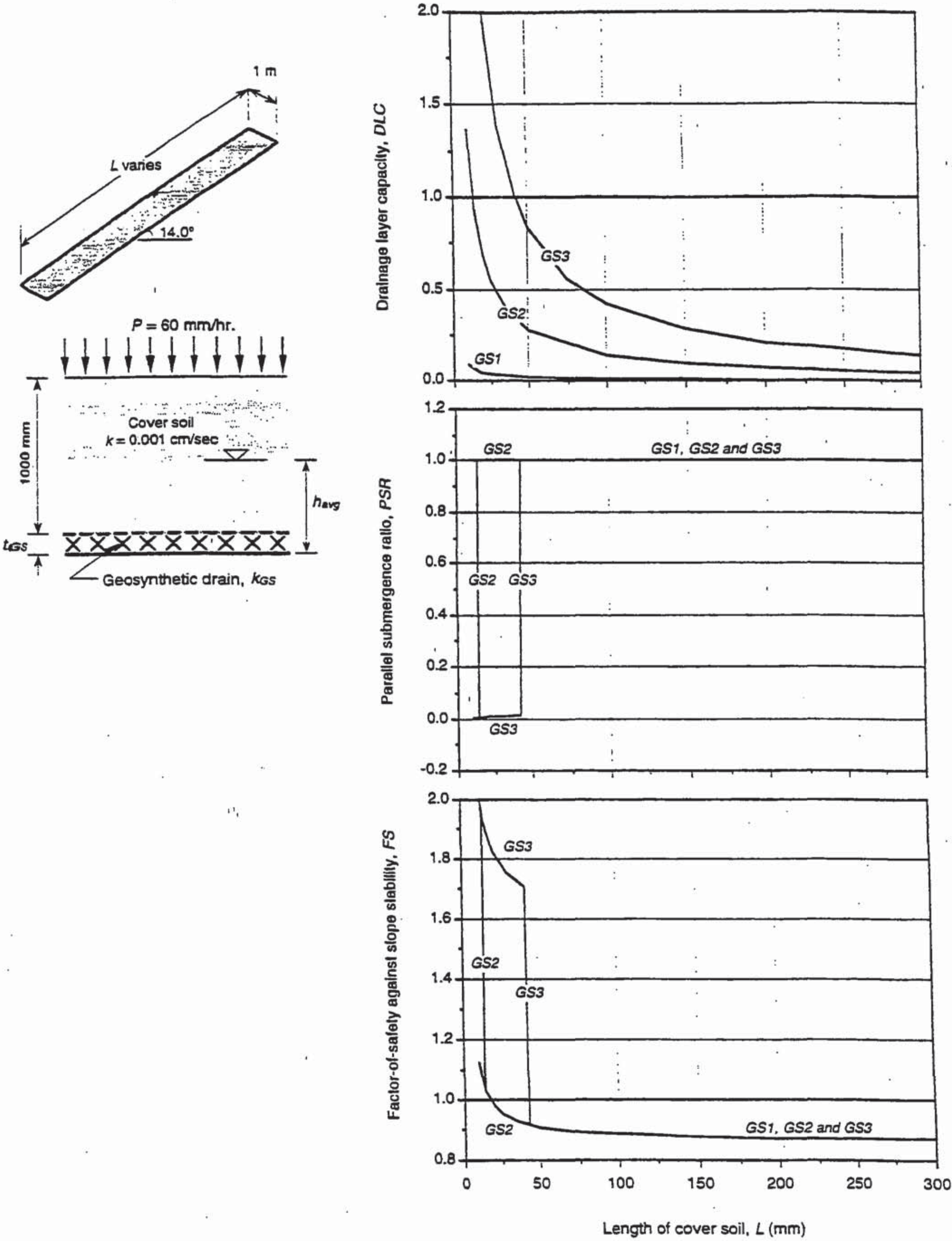


Figure 25 - Parametric study of cover system over geosynthetic drain: slope length variation

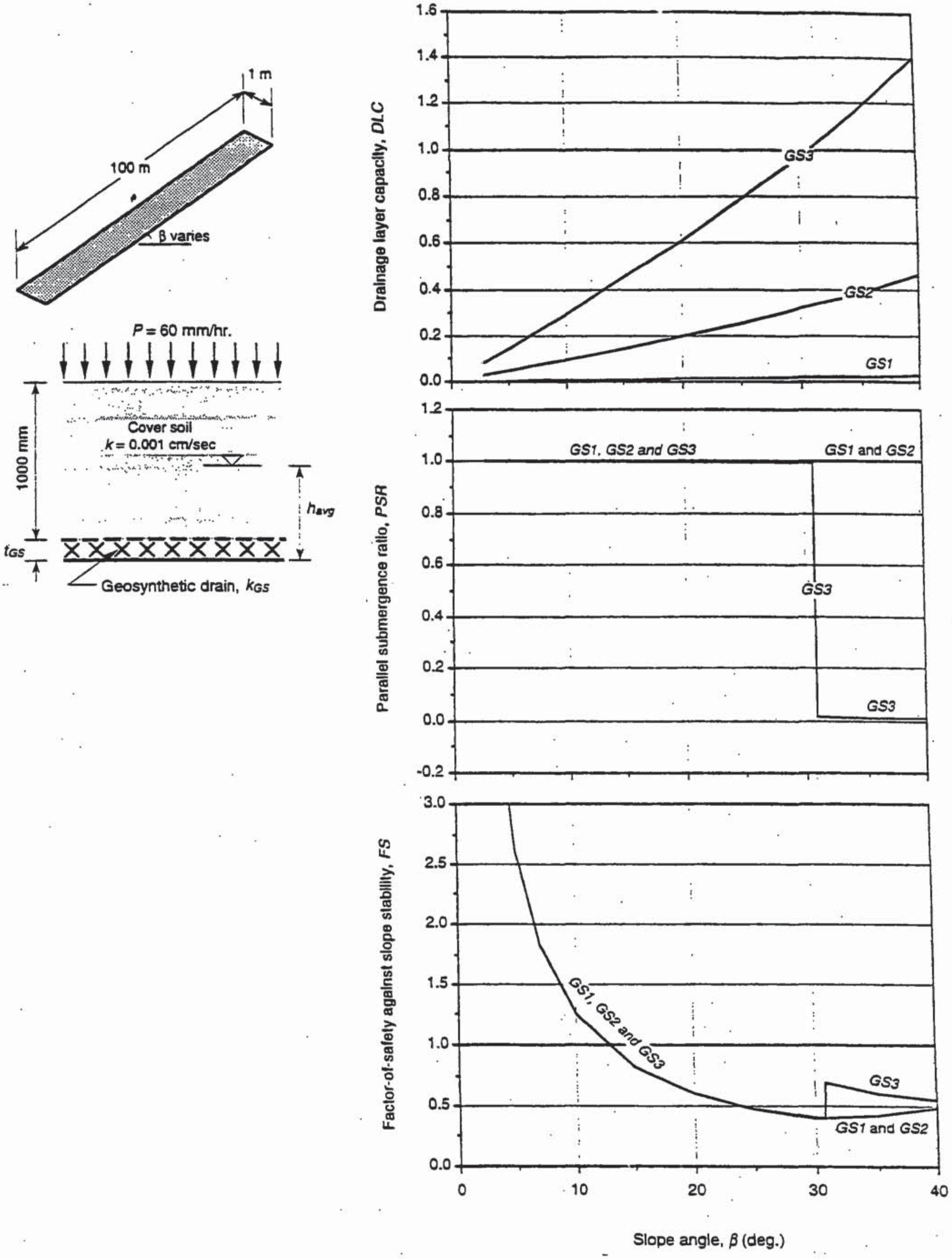


Figure 26 - Parametric study of cover system over geosynthetic drain: slope angle variation

8.0 SUMMARY

Presented in section 2.1 was information on the recent occurrence of four seepage induced slides of leachate collection systems and an additional four seepage induced slides of final cover systems. All occurred during, or immediately after, relatively large storm events (the one exception was by rapid thawing of frozen drainage soil above a still-frozen outlet drain at the toe of the slope). While the exact nature of these storm events are unknown, an idea of their magnitude can be gained by back calculating the various situations. Knowing the dimensions of the slopes and an approximation of the permeability of the soil(s) involved, the design methodology used herein (using an incipient failure FS-value of 1.0) has been followed resulting in the data of Table 19. Here it is seen that the precipitation values for the leachate collection systems was probably quite high, i.e., up to 44 mm/hour. Conversely, precipitation values for the final cover systems were apparently quite low, i.e., between 0.38 and 1.34 mm/hour. The latter are far from extraordinary events and the very low values of drainage soil permeability played strongly into the cause of the instability.

Table 19 - Back Calculated Precipitation Rates to Achieve Slope Instability for the Case Histories Presented in Table 1.

No.	Assumed permeability of cover soil, $k_{c.s.}$ (cm/sec)	Assumed permeability of drainage soil, k_d (cm/sec)	Precipitation at incipient sliding (i.e., FS = 1.0), $P_{critical}$ (mm/hr)
(a) Slides of leachate collection layers before waste placement			
1	none	0.25	14
2	none	0.50	44
3	none	0.05	1.0
4	none	0.25	35
(b) Slides of final cover/drainage layers after waste placement			
5	0.01	0.01	0.42
6	0.0001	0.01	1.20
7	0.0001	0.01	1.34
8	0.0001	0.01	0.38

Note: Values are calculated based on the following assumed constants:

Dry unit weight of soils,	γ_{dry}	$= 18.0$ kN/m^3
Saturated unit weight of soils,	$\gamma_{sat'd}$	$= 21.0$ kN/m^3
Friction angle of soils,	ϕ	= 30 deg.
Critical interface friction angle,	δ	= 22 deg.
Runoff coefficient,	RC	= 0.18 for Type (a) slides and 0.40 for Type (b) slides

To the writers, the occurrence of such a large number of recent slides is an unacceptable situation. It appears that seepage forces are being considerably underestimated by the design community in view of the very low permeability drainage soils used in "conventional" design. Both required flux quantities (lateral flow rates) and drainage system capacities are involved.

8.1 Water Balance Analysis Critique

The occurrence of eight seepage induced cover soil slides (there are probably others not known to the writers) lead directly toward mounting a challenge to the manner in which required drainage quantities are calculated. Agreed upon is the necessity of using a water balance analysis to obtain a required value of percolation through the cover soil and into the drainage layer. This value of percolation over an unit area, is then used to calculate a flux-value (lateral flow rate) which accumulates within the drainage layer reaching a maximum value at the toe of the slope. The maximum flux-value is the required value to use in designing the drainage layer capacity. Not agreed upon is the customary manner of obtaining the percolation-value, hence the required flux is effected accordingly. Typically used in this regard is the computer program entitled Hydrologic Evaluation of Landfill Performance (*HELP*).

It is felt that HELP model is an excellent program for its originally intended use; namely, to estimate the leachate quantities at the base of a landfill. The gravitational flow process through the landfilled waste material is long and slow. The daily monitoring used in the program is an excellent model. HELP should continue to be used to estimate leachate quantities, as well as the hydraulic head on the liner system. However, for short time period intense storms, through relatively thin and often high permeability soils, HELP monitoring on a daily interval is not recommended. The resulting percolation values are too low, resulting in very low required flux values and an underdesigned drainage system capacity.

Recommended and illustrated in this report is to obtain the required percolation and flux values from an hourly monitoring of a short time intensive storm, e.g., a six-hour storm event. Using this type of design scenario for leachate collection layers (before waste is placed) or final cover soil systems (after waste is placed), the following assumptions regarding the mechanisms of the water balance process are felt to be appropriate:

- Evapotranspiration is negligible during such a short time interval.
- The soils are at field capacity before the most intense part of the storm arrives, thus water storage is negligible.
- The barrier system (GM, CCL, GCL) beneath the drainage layer has no appreciable leakage, at least at the slope angles focused upon in dealing with slope stability issues.

Using the above assumptions, the local site-specific precipitation falling on the leachate collection layer or final cover soil system will be initially bifurcated into runoff and infiltration. The runoff is controlled by the surface soil (or vegetation) and the slope angle. The remainder of the precipitation results in water infiltration into the soil. The value of infiltration results directly in the percolation coming to the drainage layer. It is controlled by Darcian flow according to the soil's permeability. This value of vertical flow, in turn, produces the flux-value in the drain which accumulates over the slope length and is the required design value for selecting the drainage material's type, permeability and thickness.

Design in the manner just described results in flux-values that are 25 to 40 times greater than do designs based on HELP modeling. Furthermore, it appears that minimum technology guidance in many federal and state regulations are based on, or substantiated by, HELP modeling. Such a process results in values of required permeability of 0.01 cm/sec, and even as low as 0.001 cm/sec by some state regulatory agencies, which are orders of magnitude lower than values suggested in this report. *It is felt by the authors that this situation is the fundamental reason that seepage induced slides are frequently occurring.*

8.2 Slope Stability Analysis Comments

Once the phreatic surface is established within the specific cross section (i.e., its flow orientation and its depth of submergence), the mechanisms of the calculation procedure are quite straightforward. [The details were not presented completely in this report since the full

9.0 RECOMMENDATIONS

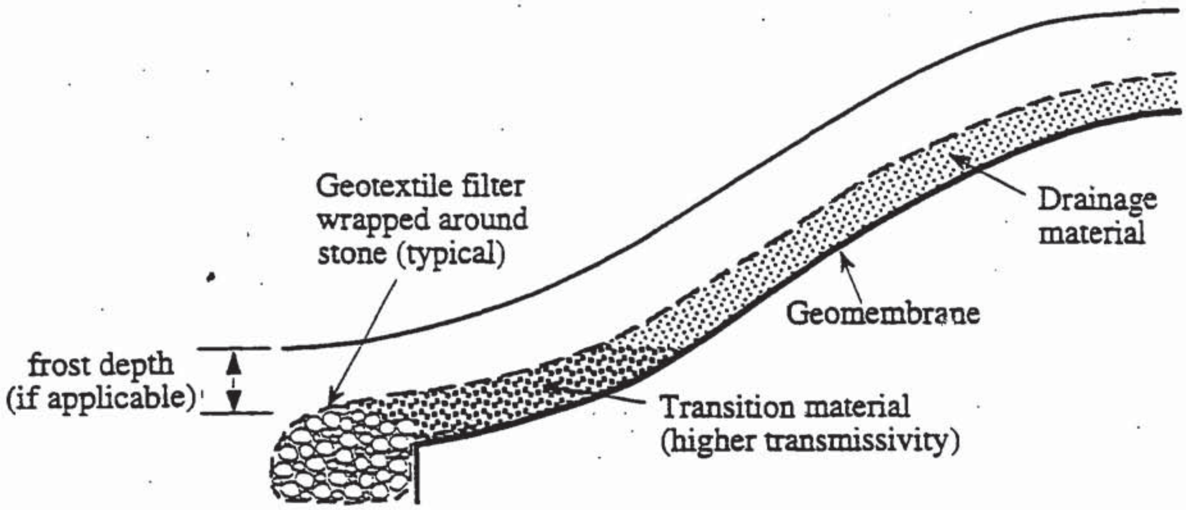
It is hoped the results of this study change some long-standing assumptions and perspectives regarding seepage design in assessing slope instability.

First, and foremost, is the recognition that seepage induced slope instability has occurred often and that its timing is during, or immediately after, intense storm events. This suggests that hourly-interval tracking of precipitation is necessary for use in the water balance analysis used to obtain the required flux (or drainage rate) value. The HELP program, based on daily-intervals is not appropriate as it is currently configured. Furthermore, and related to any type of water balance analysis whatever is its time interval, is that worst case assumptions should be made. For example, evapotranspiration, soil water storage and leakage through barrier layers are all negligible (if not zero) for short interval, high intensity storms, on relatively steep slopes with soils having high drainage rates. There are precisely the conditions where seepage induced slope instability occurs.

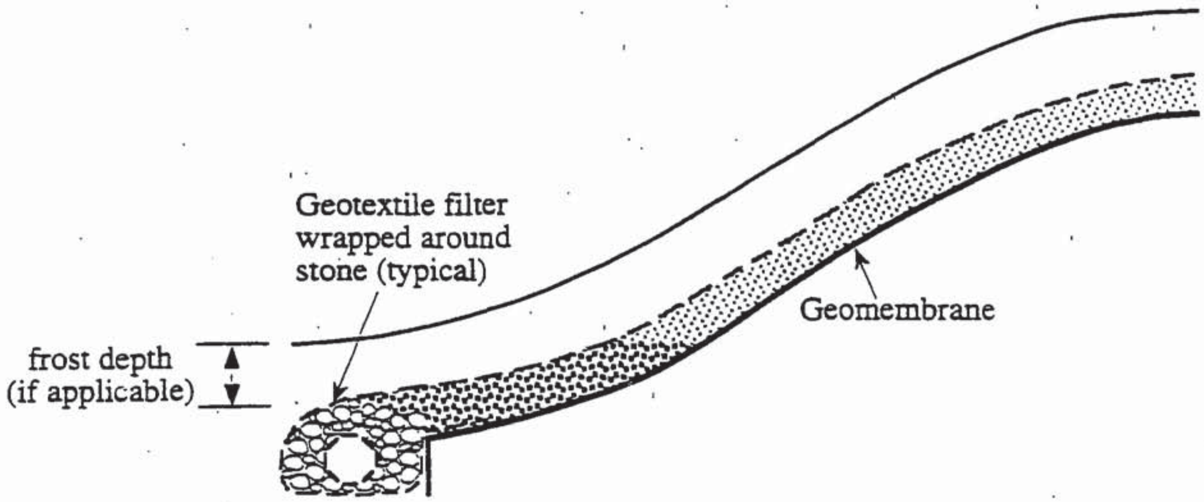
Second, (and certainly related to the high values of required flux), is that allowable flux values of the drainage system must be increased over current practice. The federal and state minimum permeability values for drainage soils (often taken and used directly in design) of 0.01 cm/sec and 0.001 cm/sec are too low by a factor of 10, and in some cases 100. However, the use of higher permeability requirements has profound implications. Natural soil drainage materials can only be gravel and even then the fines can be troublesome. The use of coarse clean gravel requires the underlying geomembrane to be suitably protected against puncture. Further, serious consideration must be given to filter design with respect to overlying fine-grained soils or solid waste. Both are serious design considerations. Geosynthetic drainage materials (geonets and geocomposites) may not be capable of conducting such high required flux-values. Depending on site-specific conditions higher flow rate geosynthetics, or traditional geocomposites augmented by natural drainage soil may be needed.

Third, is that most of the focus of this report has been on the drainage layer but, in reality, the drainage layer is part of the larger drainage system. In this regard, too little attention has been paid to the drainage layer outlet at the toe of the slope. It must be free of excess blockage by fines, as well as physical blockage by ice formations, equipment ramps, access roads, etc. Each toe situation is unique, but the sketches of Figure 28 give some schemes which might be considered. Each shows a gradually increasing drainage layer permeability as the required flux becomes greater in moving from the crest to the toe of slope. Alternatively, a natural soil drainage layer can be augmented by a geosynthetic drainage layer as greater capacity is needed towards the toe of the slope. At the toe, the drainage capability must be at its maximum. Geotextile filters should be placed as far away from the drainage pipes as possible. The pipe itself may have to be increased in diameter as it conveys water to the ultimate off-site outlet.

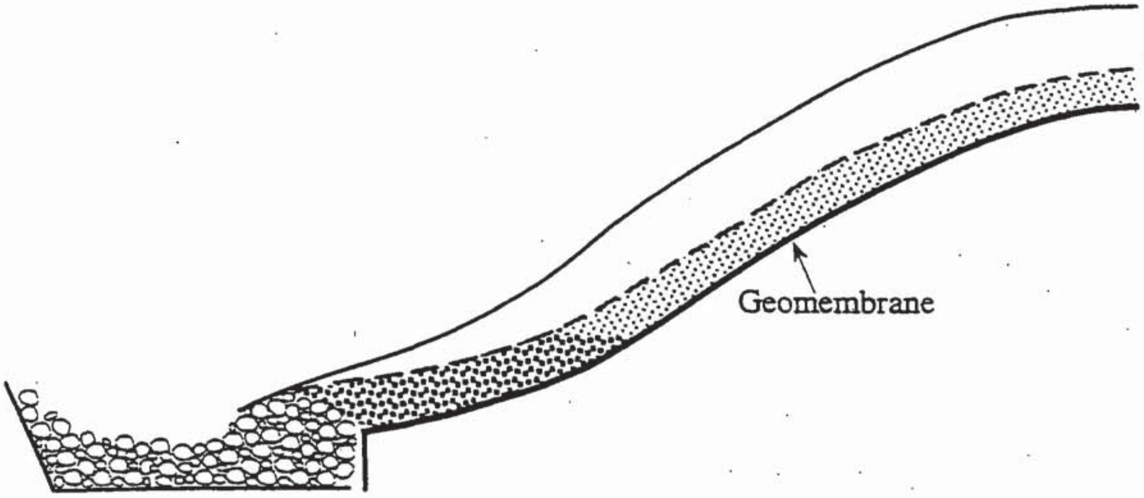
Increasing the drainage capacity of the toe, as with the upgradient drainage layer is clearly within the design community's capability. It remains to see if we are up to the challenge (and the expenses involved to the owner community) to accomplish the task.



(a) Extremely large stone toe drain



(b) Large perforated pipes surrounded by large stones



(c) Daylighting of drainage stone into a drainage channel in non-freezing climates

(Note: only feasible if sediments that run off the surface of the cover do not excessively clog the toe drainage material.)

Figure 28 - Various designs allowing for free drainage at the toe of slopes, after Soong and Koerner (1996).

Attachment 2-2

R.M. Koerner, and T-Y. Soong, 1998. "Analysis and Design of Veneer Cover Soils" Proceeding of 6th International Conference on Geosynthetics, Vol. 1, pp. 1-23, Atlanta, Georgia, USA

Analysis and Design of Veneer Cover Soils

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ABSTRACT: The sliding of cover soils on slopes underlain by geosynthetics is obviously an unacceptable situation and, if the number of occurrences becomes excessive, will eventually reflect poorly on the entire technology. Steeply sloped leachate collection layers and final covers of landfills are situations where incidents of such sliding have occurred. Paradoxically, the analytic formulation of the situation is quite straightforward. This paper presents an analysis of the common problem of a veneer of cover soil (0.3 to 1.0 m thick) on a geosynthetic material at a given slope angle and length so as to arrive at a FS-value. The paper then presents different scenarios that create lower FS-values than the gravitational stresses of the above situation, e.g., equipment loads, seepage forces and seismic loads. As a counterpoint, different scenarios that create higher FS-values also are presented, e.g., toe berms, tapered thickness cover soils and veneer reinforcement. In this latter category, a subdivision is made between intentional reinforcement (using geogrids or high strength geotextiles) and nonintentional reinforcement (cases where geosynthetics overlay a weak interface within a multilayered slope). Hypothetical numeric examples are used in each of the above situations to illustrate the various influences on the resulting FS-value. In many cases, design curves are also generated. Suggested minimum FS-values are presented for final closures of landfills, waste piles, leach pads, etc., which are the situations where veneer slides of this type are the most troublesome. Hopefully, the paper will serve as a vehicle to bring a greater awareness to such situations so as to avert slides from occurring in the future.

KEYWORDS: Analysis, Design, Limit Equilibrium Methods, Steep Slopes, Veneer Stability.

1 INTRODUCTION

There have been numerous cover soil stability problems in the past resulting in slides that range from being relatively small (which can be easily repaired), to very large (involving litigation and financial judgments against the parties involved). Furthermore, the number of occurrences appears to have increased over the past few years. Soong and Koerner (1996) report on eight cover soil failures resulting from seepage induced stresses alone. While such slides can occur in transportation and geotechnical applications, it is in the environmental applications area where they are most frequent. Specifically, the sliding of relatively thin cover soil layers (called "veneer") above both geosynthetic and natural soil liners, i.e., geomembranes (GM), geosynthetic clay liners (GCL) and compacted clay liners (CCL) are the particular materials of concern. These situations represent a major challenge due (in part) to the following reasons:

(a) The underlying barrier materials generally represent a low interface shear strength boundary with respect to the soil placed above them.
(b) The liner system is oriented precisely in the direction of potential sliding.
(c) The potential shear planes are usually linear and are essentially uninterrupted along the slope.
(d) Liquid (water or leachate) cannot continue to percolate downward through the cross section due to the presence of the barrier material.

When such slopes are relatively steep, long and uninterrupted in their length (which is the design goal for landfills, waste piles and surface impoundments so as to maximize containment space and minimize land area), the situation is exacerbated.

There are two specific applications in which cover soil stability has been difficult to achieve in light of this discussion.

- Leachate collection soil placed above a GM, GCL and/or CCL along the sides of a landfill before waste is placed and stability achieved accordingly.
- Final cover soil placed above a GM, GCL and/or CCL in the cap or closure of a landfill or waste pile after the waste has been placed to its permitted height.

For the leachate collection soil situation, the time frame is generally short (from months to a few years) and the implications of a slide may be minor in that repairs can oftentimes be done by on-site personnel. For the final cover soil situation, the time frame is invariably long (from decades to centuries) and the implications of a slide can be serious in that repairs often call for a forensic analysis, engineering redesign, separately engaged contractors and quite high remediation costs. These latter cases sometime involve litigation, insurance carriers, and invariably technical experts, thus becoming quite contentious.

Since both situations (leachate collection and final covers) present the same technical issues, the paper will address them simultaneously. It should be realized, however, that the final cover situation is of significantly greater concern.

In the sections to follow, geotechnical engineering considerations will be presented leading to the goal of establishing a suitable factor of safety (FS) against slope instability. A number of common situations will then be analyzed, all of which have the tendency to decrease stability. As a counterpoint, a number of design options will follow, all of which have the objective of increasing stability. A summary and conclusions section will compare the various situations which tend to either create slope instability or aid in slope stability. It is hoped that an increased awareness in the analysis and design details offered herein, and elsewhere in the published literature which is referenced herein, leads to a significant decrease in the number of veneer cover soil slides that have occurred.

2 GEOTECHNICAL ENGINEERING CONSIDERATIONS

As just mentioned, the potential failure surface for veneer cover soils is usually linear with the cover soil sliding with respect to the lowest interface friction layer in the underlying cross section. The potential failure plane being linear allows for a straightforward stability calculation without the need for trial center locations and different radii as with soil stability problems analyzed by rotational failure surfaces. Furthermore, full static equilibrium can be achieved without solving simultaneous equations or making simplified design assumptions.

2.1 Limit Equilibrium Concepts

The free body diagram of an *infinitely* long slope with uniformly thick cohesionless cover soil on an incipient planar shear surface, like the upper surface of a geomembrane, is shown in Figure 1. The situation can be treated quite simply.

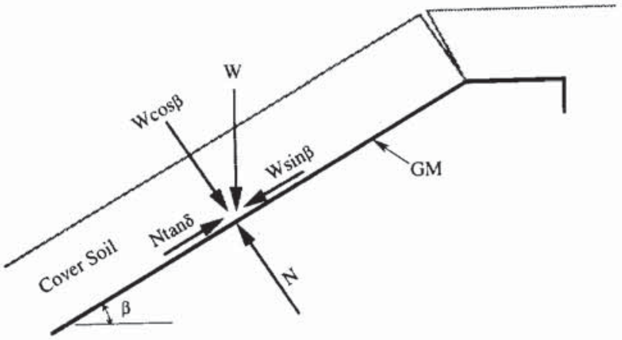


Figure 1. Limit equilibrium forces involved in an infinite slope analysis for a uniformly thick cohesionless cover soil.

By taking force summation parallel to the slope and comparing the resisting force to the driving or mobilizing force, a global factor of safety (FS) results;

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

$$= \frac{N \tan\delta}{W \sin\beta} = \frac{W \cos\beta \tan\delta}{W \sin\beta}$$

hence:

$$FS = \frac{\tan\delta}{\tan\beta} \tag{1}$$

Here it is seen that the FS-value is the ratio of tangents of the interface friction angle of the cover soil against the upper surface of the geomembrane (δ), and the slope angle of the soil beneath the geomembrane (β). As simple as this analysis is, its teachings are very significant, for example:

- To obtain an accurate FS-value, an accurately determined laboratory δ-value is absolutely critical. The accuracy of the final analysis is only as good as the accuracy of the laboratory obtained δ-value.
- For low δ-values, the resulting soil slope angle will be proportionately low. For example, for a δ-value of 20 deg., and a required FS-value of 1.5, the maximum slope angle is 14 deg. This is equivalent to a 4(H) on 1(V) slope which is relatively low. Furthermore, many geosynthetics have even lower δ-values than 20 deg.
- This simple formula has driven geosynthetic manufacturers to develop products with high δ-values, e.g., textured geomembranes, thermally bonded drainage geocomposites, internally reinforced GCLs, etc.

Unfortunately, the above analysis is too simplistic to use in most realistic situations. For example, the following situations cannot be accommodated:

- A finite length slope with the incorporation of a passive soil wedge at the toe of the slope
- The consideration of equipment loads on the slope
- Consideration of seepage forces within the cover soil
- Consideration of seismic forces acting on the cover soil
- The use of soil masses acting as toe berms
- The use of tapered covered soil thicknesses
- Reinforcement of the cover soil using geogrids or high strength geotextiles

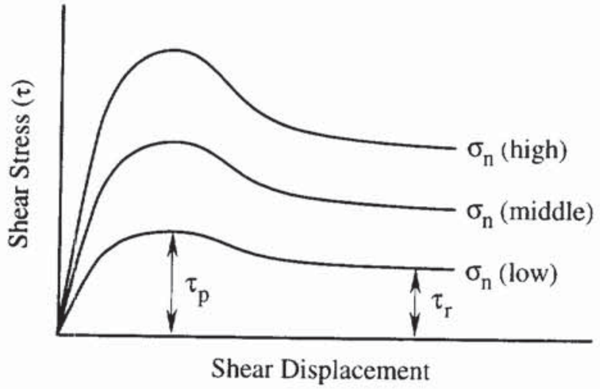
These specific situations will be treated in subsequent sections. For each situation, the essence of the theory will be presented, followed by the necessary design equations. This will be followed, in each case, with a design graph and a numeric example. First, however, the important issue of interface shear testing will be discussed.

2.2 Interface Shear Testing

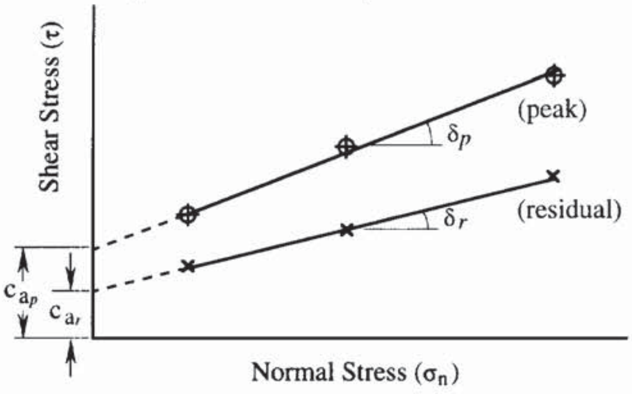
The interface shear strength of a cover soil with respect to the underlying material (often a geomembrane) is critical so as to properly analyze the stability of the cover soil. This value of interface shear strength is obtained by laboratory testing of the project specific materials at the site specific conditions. By project specific materials, we mean sampling of the candidate geosynthetics to be used at the site, as well as the cover soil at its targeted density and moisture conditions. By site specific conditions we mean normal stresses, strain rates, peak or residual shear strengths and temperature extremes (high and/or low). Note that it is completely inappropriate to use values of interface shear strengths from the literature for final design.

While the above list of items is formidable, at least the type of test is established. It is the direct shear test which has been utilized in geotechnical engineering testing for many years. The test has been adapted to evaluate geosynthetics in the USA as ASTM D5321 and in Germany as DIN 60500.

In conducting a direct shear test on a specific interface, one typically performs three replicate tests with the only variable being different values of normal stress. The middle value is usually targeted to the site specific condition, with a lower and higher value of normal stress covering the range of possible values. These three tests result in a set of shear displacement versus shear stress curves, see Figure 2a. From each curve, a peak shear strength (τ_p) and a residual shear strength (τ_r) are obtained. As a next step, these shear strength values, together with their respective normal stress values, are plotted on Mohr-Coulomb stress space to obtain the shear strength parameters of friction and adhesion, see Figure 2b.



(a) Direct shear test experimental data



(b) Resulting behavior on Mohr - Coulomb stress space

Figure 2. Direct shear test results and analysis procedure to obtain shear strength parameters.

The points are then connected (usually with a straight line), and the two fundamental shear strength parameters are obtained. These shear strength parameters are:

δ = the angle of shearing resistance, peak and/or residual, of the two opposing surfaces (often called the interface friction angle)

c_a = the adhesion of the two opposing surfaces, peak and/or residual (synonymous with cohesion when testing fine grained soils against one another)

Each set of parameters constitute the equation of a straight line which is the Mohr-Coulomb failure criterion common to geotechnical engineering. The concept is readily adaptable to geosynthetic materials in the following form:

$$\tau_p = c_{ap} + \sigma_n \tan \delta_p \tag{2a}$$

$$\tau_r = c_{ar} + \sigma_n \tan \delta_r \tag{2b}$$

The upper limit of "δ" when soil is involved as one of the interfaces is "ϕ", the angle of shearing resistance of the soil component. The upper limit of the "c_a" value is "c", the cohesion of the soil component. In the slope stability analyses to follow, the "c_a" term will be included for the sake of completeness, but then it will be neglected (as being a conservative assumption) in the design graphs and numeric examples. To utilize an adhesion value, there must be a clear physical justification for use of such values when geosynthetics are involved. Some unique situations such as textured geomembranes with physical interlocking of soils having cohesion, or the bentonite component of a GCL are valid reasons for including such a term.

Note that residual strengths are equal, or lower, than peak strengths. The amount of difference is very dependent on the material and no general guidelines can be given. Clearly, material specific and site specific direct shear tests must be performed to determine the appropriate values. Further, each direct shear test must be conducted to a relatively large displacement to determine the residual behavior, see Stark and Poeppel (1994). The decision as to the use of peak or residual strengths in the subsequent analysis is a very subjective one. It is both a materials specific and site specific issue which is left up to the designer and/or regulator. Even further, the use of peak values at the crest of a slope and residual values at the toe may be justified. As such, the analyses to follow will use an interface δ-value with no subscript thereby concentrating on the computational procedures rather than this particular detail. However, the importance of an appropriate and accurate δ-value should not be minimized.

Due to the physical structure of many geosynthetics, the size of the recommended shear box is quite large. It must be at least 300 mm by 300 mm unless it can be shown that data generated by a smaller device contains no scale or edge effects, i.e., that no bias exists with a smaller shear box. The implications of such a large shear box should not be taken lightly. Some issues which should receive particular attention are the following:

- Unless it can be justified otherwise, the interface will usually be tested in a saturated state. Thus complete and uniform saturation over the entire specimen area must be achieved. This is particularly necessary for CCLs and GCLs, Daniel, et al. (1993). Hydration takes relatively long in comparison to soils in conventional (smaller) testing shear boxes.
- Consolidation of soils (including CCLs and GCLs) in larger shear boxes is similarly affected.
- Uniformity of normal stress over the entire area must be maintained during consolidation and shearing so as to avoid stress concentrations from occurring.
- The application of relatively low normal stresses, e.g., 10, to 30 kPa simulating typical cover soil thicknesses, challenges the accuracy of some commercially available shear box setups and monitoring systems, particularly the accuracy of pressure gages.

- The issue of appropriate normal stress is greatly complicated if gas pressures are generated in the underlying waste. These gas pressures will counteract some (or all) of the gravitational stress of the cover soil. The resulting shear strength, and subsequent stability, can be significantly decreased. See Liu et al (1997) for insight into this possibility.
- Shear rates necessary to attain drained conditions (if this is the desired situation) are extremely slow, requiring long testing times.
- Deformations necessary to attain residual strengths require large relative movement of the two respective halves of the shear box. So as not to travel over the edges of the opposing shear box sections, devices should have the lower shear box significantly longer than 300 mm. However, with a lower shear box longer than the upper traveling section, new surface is constantly being added to the shearing plane. This influence is not clear in the material's response or in the subsequent behavior.
- The attainment of a true residual strength is difficult to achieve. ASTM D5321 states that one should "run the test until the applied shear force remains constant with increasing displacement". Many commercially available shear boxes have insufficient travel to reach this condition.
- The ring torsion shearing apparatus is an alternative device to determine true residual strength values, but is not without its own problems. Some outstanding issues are the small specimen size, nonuniform shear rates along the width of the specimen, anisotropic shearing with some geosynthetics and no standardized testing protocol. See Stark and Poeppel (1994) for information and data using this alternative test method.

2.3 Various Types of Loadings

There are a large variety of slope stability problems that may be encountered in analyzing and/or designing final covers of engineered landfills, abandoned dumps and remediation sites as well as leachate collection soils covering geomembranes beneath the waste. Perhaps the most common situation is a uniformly thick cover soil on a geomembrane placed over the soil subgrade at a given and constant slope angle. This "standard" problem will be analyzed in the next section. A variation of this problem will include equipment loads used during placement of cover soil on the geomembrane. This problem will be solved with equipment moving up the slope and then moving down the slope.

Unfortunately, cover soil slides have occurred and it is felt that the majority of the slides have been associated with seepage forces. Indeed, drainage above a geomembrane (or other barrier material) in the cover soil cross section must be accommodated to avoid the possibility of seepage forces. A section will be devoted to this class of slope stability problems.

Lastly, the possibility of seismic forces exists in earthquake prone locations. If an earthquake occurs in the vicinity of an engineered landfill, abandoned dump or remediation site, the seismic wave travels through the solid waste mass reaching the upper surface of the cover. It then decouples from the cover soil materials, producing a horizontal force which must be appropriately analyzed. A section will be devoted to the seismic aspects of cover soil slope analysis as well.

All of the above actions are destabilizing forces tending to cause slope instability. Fortunately, there are a number of actions that can be taken to increase the stability of slopes.

Other than geometrically redesigning the slope with a flatter slope angle or shorter slope length, a designer can add soil mass at the toe of the slope thereby enhancing stability. Both toe berms and tapered soil covers are available options and will be analyzed accordingly. Alternatively, the designer can always use geogrids or high strength geotextiles within the cover soil acting as reinforcement materials. This technique is usually referred to as veneer reinforcement. Cases of both intentional and nonintentional veneer reinforcement will be presented.

Thus it is seen that a number of strategies influence slope stability. Each will be described in the sections to follow. First, the basic gravitational problem will be presented followed by those additional loading situations which tend to decrease slope stability. Second, various actions that can be taken by the designer to increase slope stability will be presented. The summary will contrast the FS-values obtained in the similarly crafted numeric examples.

3 SITUATIONS CAUSING DESTABILIZATION OF SLOPES

This section treats the standard veneer slope stability problem and then superimposes upon it a number of situations, all of which tend to destabilize slopes. Included are gravitational, construction equipment, seepage and seismic forces. Each will be illustrated by a design graph and a numeric example.

3.1 Cover Soil (Gravitational) Forces

Figure 3 illustrates the common situation of a *finite* length, uniformly thick cover soil placed over a liner material at a slope angle "β". It includes a passive wedge at the toe and has a tension crack of the crest. The analysis that follows is after Koerner and Hwu (1991), but comparable analyses are available from Giroud and Beech (1989), McKelvey and Deutsch (1991) , Ling and Leshchinsky (1997) and others.

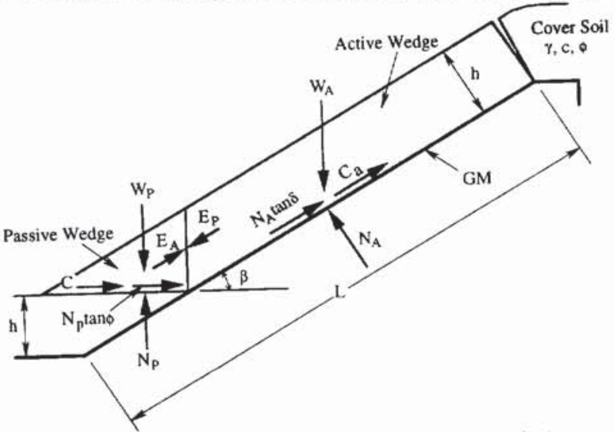


Figure 3. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil.

The symbols used in Figure 3 are defined below.

W_A = total weight of the active wedge
W_P = total weight of the passive wedge
N_A = effective force normal to the failure plane of the active wedge
N_P = effective force normal to the failure plane of the passive wedge
γ = unit weight of the cover soil
h = thickness of the cover soil
L = length of slope measured along the geomembrane
β = soil slope angle beneath the geomembrane
ϕ = friction angle of the cover soil
δ = interface friction angle between cover soil and geomembrane
C_a = adhesive force between cover soil of the active wedge and the geomembrane
c_a = adhesion between cover soil of the active wedge and the geomembrane
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the cover soil
E_A = interwedge force acting on the active wedge from the passive wedge
E_P = interwedge force acting on the passive wedge from the active wedge
FS = factor of safety against cover soil sliding on the geomembrane

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge,

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) \tag{3}$$

$$N_A = W_A \cos\beta \tag{4}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) \tag{5}$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin\beta = W_A - N_A \cos\beta - \frac{N_A \tan\delta + C_a}{FS}\sin\beta \tag{6}$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} \tag{7}$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} \tag{8}$$

$$N_p = W_P + E_P \sin\beta \tag{9}$$

$$C = \frac{(c)(h)}{\sin\beta} \tag{10}$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cos\beta = \frac{C + N_P \tan\phi}{FS} \tag{11}$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \tag{12}$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$ which in our case, using FS-values, is:

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$\begin{aligned} a &= (W_A - N_A\cos\beta)\cos\beta \\ b &= -[(W_A - N_A\cos\beta)\sin\beta\tan\phi \\ &\quad + (N_A\tan\delta + C_a)\sin\beta\cos\beta \\ &\quad + \sin\beta(C + W_P\tan\phi)] \\ c &= (N_A\tan\delta + C_a)\sin^2\beta\tan\phi \end{aligned} \tag{14}$$

The resulting FS-value is then obtained from the solution of the quadratic equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{15}$$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated. Thus a value of greater than 1.0 must be targeted as being the minimum factor of safety. How much greater than 1.0 the FS-value should be, is a design and/or regulatory issue. The issue of minimum allowable FS-values under different conditions will be assessed at the end of the paper. In order to better illustrate the implications of Eqs. 13, 14 and 15, typical design curves for various FS-values as a function of slope angle and interface friction angle are given in Figure 4. Note that the curves are developed specifically for the variables stated in the legend of the figure. Example 1 illustrates the use of the curves in what will be the standard example to which other examples will be compared.

Example 1:

Given a 30 m long slope with a uniformly thick 300 mm cover soil at a unit weight of 18 kN/m³. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is placed directly on a geomembrane as shown in Figure 3. Direct shear testing has resulted in a interface friction angle between the cover soil and geomembrane of 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg?

Solution:

Substituting Eq. 14 into Eq. 15 and solving for the FS-value results in the following which is seen to be in agreement with the curves of Figure 4.

$$\left.\begin{array}{l} a = 14.7\,\mathrm{kN/m} \\ b = -21.3\,\mathrm{kN/m} \\ c = 3.5\,\mathrm{kN/m} \end{array}\right\} FS = 1.25$$

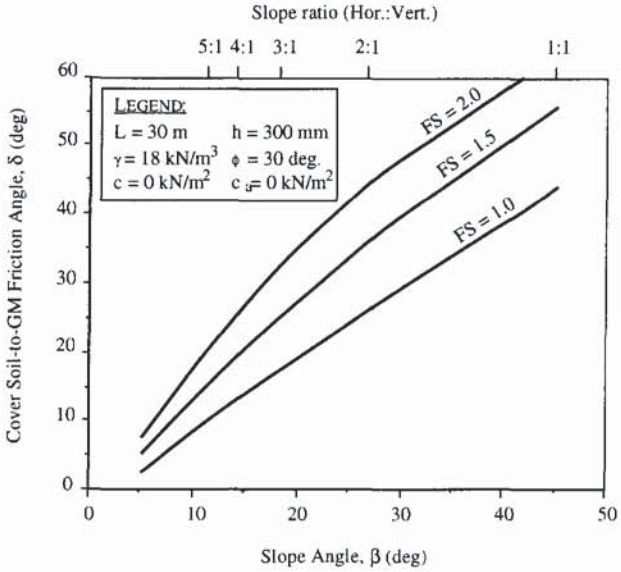


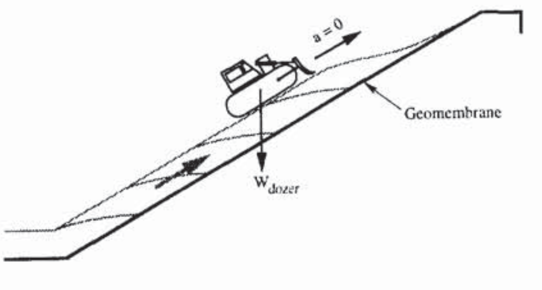
Figure 4. Design curves for stability of uniform thickness cohesionless cover soils on linear failure planes for various global factors-of-safety.

Comment:
In general, this is too low of a value for a final cover soil factor-of-safety and a redesign is necessary. While there are many possible options of changing the geometry of the situation, the example will be revisited later in this section using toe berms, tapered cover soil thickness and veneer reinforcement. Furthermore, this general problem will be used throughout the main body of this paper for comparison purposes to other cover soil slope stability situations.

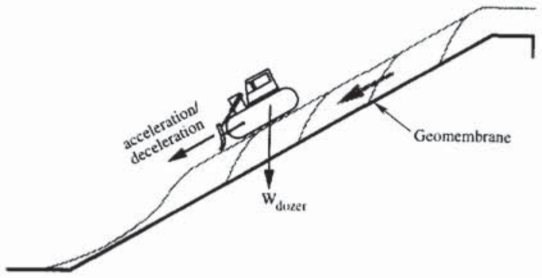
3.2 Tracked Construction Equipment Forces

The placement of cover soil on a slope with a relatively low shear strength inclusion (like a geomembrane) should always be from the toe upward to the crest. Figure 5a shows the recommended method. In so doing, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever present passive wedge and stable lower-portion beneath the active wedge. While it is necessary to specify low ground pressure equipment to place the soil, the reduction of the FS-value for this situation of equipment working up the slope will be seen to be relatively small.

For soil placement down the slope, however, a stability analysis cannot rely on toe buttressing and also a dynamic stress should be included in the calculation. These conditions decrease the FS-value and in some cases to a great extent. Figure 5b shows this procedure. Unless absolutely necessary, it is not recommended to place cover soil on a slope in this manner. If it is necessary, the design must consider the unsupported soil mass and the dynamic force of the specific type of construction equipment and its manner of operation.



(a) Equipment backfilling up slope
(the recommended method)



(b) Equipment backfilling down slope
(method is not recommended)

Figure 5. Construction equipment placing cover soil on slopes containing geosynthetics.

For the <u>first case</u> of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 6a. The analysis uses a specific piece of tracked construction equipment (like a bulldozer characterized by its ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the geomembrane. A Boussinesq analysis is used, see Poulos and Davis (1974). This results in an equipment force per unit width as follows:

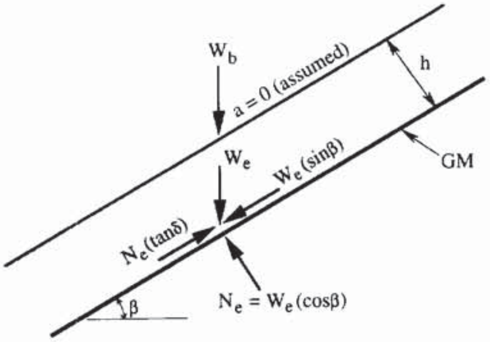
$$W_e = q w I \tag{16}$$

where

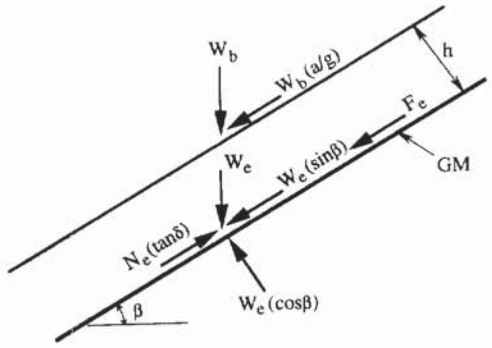
W_e = equivalent equipment force per unit width at the geomembrane interface

q = $W_b / (2 \times w \times b)$

W_b = actual weight of equipment (e.g., a bulldozer)
w = length of equipment track
b = width of equipment track
I = influence factor at the geomembrane interface see Figure 7



(a) Equipment moving up slope
(load with no assumed acceleration)



(b) Equipment moving down slope
(load plus acceleration or deceleration)

Figure 6. Additional (to gravitational forces) limit equilibrium forces due to construction equipment moving on cover soil (see Figure 3 for the gravitational soil force to which the above forces are added).

Upon determining the additional equipment force at the cover soil-to-geomembrane interface, the analysis proceeds as described in Section 3.1 for gravitational forces only. In essence, the equipment moving up the slope adds an additional term, W_e, to the W_A-force in Eq. 3. Note, however, that this involves the generation of a resisting force as well. Thus, the net effect of increasing the driving force as well as the resisting force is somewhat neutralized insofar as the resulting FS-value is concerned. It should also be noted that no acceleration/deceleration forces are included in this analysis which is somewhat optimistic. Using these concepts (the same equations used in Section 3.1 are used here), typical design curves for various FS-values as a function of equivalent ground contact equipment pressures and cover soil thicknesses are given in Figure 8. Note that the curves are developed specifically for the variables stated in the legend. Example 2a illustrates the use of the formulation.

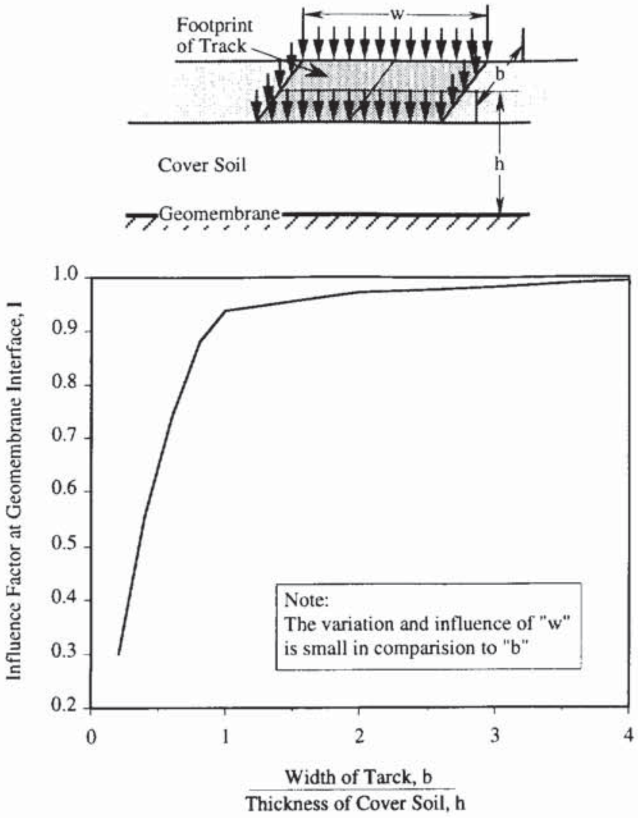


Figure 7. Values of influence factor, "I", for use in Eq. 16 to dissipate surface force of tracked equipment through the cover soil to the geomembrane interface, after Poulos and Davis (1974).

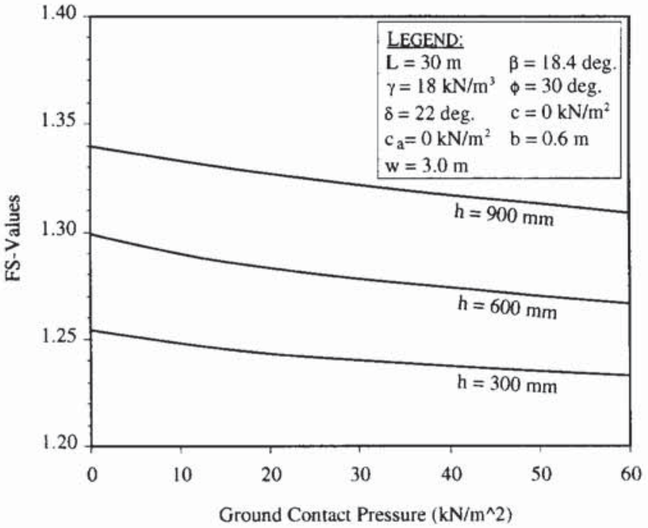


Figure 8. Design curves for stability of different thickness of cover soil for various values of tracked ground contact pressure construction equipment.

Example 2a:

Given 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the toe of the slope up to the crest. The bulldozer has a ground pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The cover soil to geomembrane friction angle is 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:
This problem follows Example 1 exactly except for the addition of the bulldozer moving up the slope. Using the additional equipment load Eq. 16, substituted into Eqs. 14 and 15 results in the following.

$$\left.\begin{array}{l} a = 73.1\,\text{kN / m} \\ b = -104.3\,\text{kN / m} \\ c = 17.0\,\text{kN / m} \end{array}\right\} \text{FS} = 1.24$$

Comment:
While the resulting FS-value is low, the result is best assessed by comparing it to Example 1, i.e., the same problem except without the bulldozer. It is seen that the FS-value has only decreased from 1.25 to 1.24. Thus, in general, a low ground contact pressure bulldozer placing cover soil up the slope with negligible acceleration/ deceleration forces does not significantly decrease the factor-of-safety.

For the second case of a bulldozer pushing cover soil down from the crest of the slope to the toe as shown in Figure 5b, the analysis uses the force diagram of Figure 6b. While the weight of the equipment is treated as just described, the lack of a passive wedge along with an additional force due to acceleration (or deceleration) of the equipment significantly changes the resulting FS-values. This analysis again uses a specific piece of construction equipment operated in a specific manner. It produces a force parallel to the slope equivalent to W_b (a/g), where W_b = the weight of the bulldozer, a = acceleration of the bulldozer and g = acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed, see Figure 9. A similar behavior will be seen for deceleration.

The acceleration of the bulldozer, coupled with an influence factor "I" from Figure 7, results in the dynamic force per unit width at the cover soil to geomembrane interface, "F_e". The relationship is as follows:

$$F_e = W_e\left(\frac{a}{g}\right) \tag{17}$$

where

F_e = dynamic force per unit width parallel to the slope at the geomembrane interface,

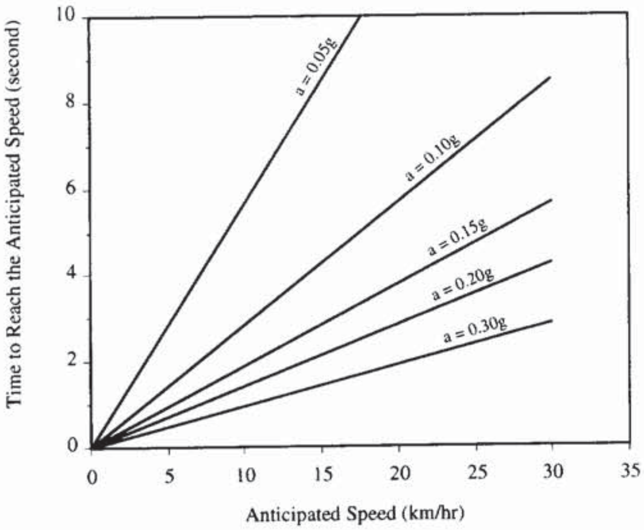


Figure 9. Graphic relationship of construction equipment speed and rise time to obtain equipment acceleration.

W_e = equivalent equipment (bulldozer) force per unit width at geomembrane interface, recall Eq. 16.
β = soil slope angle beneath geomembrane
a = acceleration of the bulldozer
g = acceleration due to gravity

Using these concepts, the new force parallel to the cover soil surface is dissipated through the thickness of the cover soil to the interface of the geomembrane. Again, a Boussinesq analysis is used, see Poulos and Davis (1974). The expression for determining the FS-value can now be derived as follows:

Considering the active wedge, and balancing the forces in the direction parallel to the slope, the following formulation results:

$$E_A + \frac{(N_e + N_A)\tan\delta + C_a}{FS} = (W_A + W_e)\sin\beta + F_e \tag{18}$$

where

N_e = effective equipment force normal to the failure plane of the active wedge
= $W_e \cos\beta$ (19)

Note that all the other symbols have been previously defined.

The interwedge force acting on the active wedge can down be expressed as:

$$E_A = \frac{(FS)\left[(W_A + W_e)\sin\beta + F_e\right]}{FS} - \frac{\left[(N_e + N_A)\tan\delta + C_a\right]}{FS} \tag{20}$$

The passive wedge can be treated in a similar manner. The following formulation of the interwedge force acting on the passive wedge results:

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} \tag{21}$$

By setting $E_A = E_P$, the following equation can be arranged in the form of Eq. 13 in which the "a", "b" and "c" terms are as follows:

$$a = \left[\left(W_A + W_e\right)\sin\beta + F_e\right]\cos\beta$$

$$\begin{aligned} b = -\Big\{&\left[\left(N_e + N_A\right)\tan\delta + C_a\right]\cos\beta \\ &+ \left[\left(W_A + W_e\right)\sin\beta + F_e\right]\sin\beta\tan\phi \\ &+ \left(C + W_P \tan\phi\right)\Big\} \end{aligned}$$

$$c = \left[\left(N_e + N_A\right)\tan\delta + C_a\right]\sin\beta\tan\phi \tag{22}$$

Finally, the resulting FS-value can be obtained using Eq. 15. Using these concepts, typical design curves for various FS-values as a function of equipment ground contact pressure and equipment acceleration can be developed, see Figure 10. Note that the curves are developed specifically for the variables stated in the legend. Example 2b illustrates the use of the formulation.

Example 2b:

Given a 30 m long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. It is placed on the slope using a bulldozer moving from the crest of the slope down to the toe. The bulldozer has a ground contact pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The estimated equipment speed is 20 km/hr and the time to reach this speed is 3.0 sec. The cover soil to geomembrane friction angle is 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:

Using the design curves of Figure 10 along with Eqs. 22 substituted into Eq. 15 the solution can be obtained:

- From Figure 9 at 20 km/hr and 3.0 sec. the bulldozer's acceleration is 0.19g.
- From Eq. 22 substituted into Eq. 15 we obtain

$$\left.\begin{aligned} a &= 88.8\,\mathrm{kN/m} \\ b &= -107.3\,\mathrm{kN/m} \\ c &= 17.0\,\mathrm{kN/m} \end{aligned}\right\} \quad FS = 1.03$$

Comment:

This problem solution can now be compared to the previous two examples:

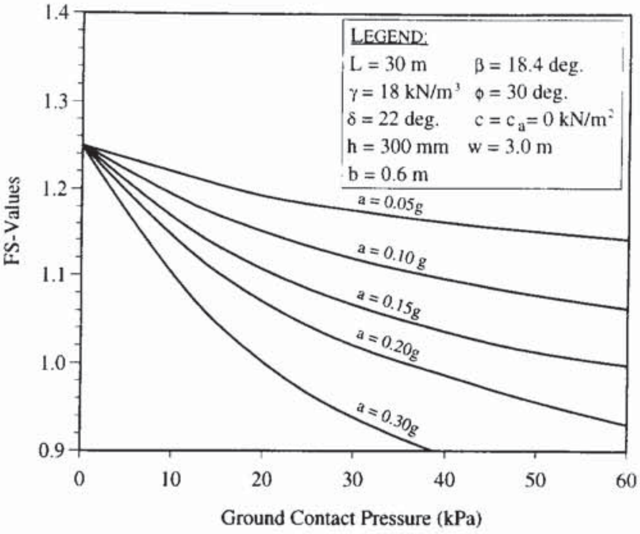


Figure 10. Design curves for stability of different construction equipment ground contact pressure for various equipment accelerations.

Ex. 1:	cover soil alone with no bulldozer loading	FS = 1.25
Ex. 2a:	cover soil plus bulldozer moving up slope	FS = 1.24
Ex. 2b:	cover soil plus bulldozer moving down slope	FS = 1.03

The inherent danger of a bulldozer moving down the slope is readily apparent. Note, that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguable the more severe condition due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should the cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the exact conditions made in the design. The maximum allowable weight and ground contact pressure of the equipment should be stated along with suggested operator movement of the cover soil placement operations. Truck traffic on the slopes can also give as high, or even higher, stresses and should be avoided unless adequately designed. Additional detail is given in McKelvey (1994). The issue of access ramps is a unique subset of this example and one which deserves focused attention due to the high loads and decelerations that often occur.

3.3 Consideration of Seepage Forces

The previous sections presented the general problem of slope stability analysis of cover soils placed on slopes under different conditions. The tacit assumption throughout was that either permeable soil or a drainage layer was placed above the barrier layer with adequate flow capacity to efficiently remove permeating water safely way from the cross section. The amount of water to be removed is obviously a site specific situation. Note that in extremely arid areas, or with very low permeability cover soils drainage may not be required although this is generally the exception.

Unfortunately, adequate drainage of final covers has sometimes not been available and seepage induced slope stability problems have occurred. The following situations have resulted in seepage induced slides:

- Drainage soils with hydraulic conductivity (permeability) too low for site specific conditions.
- Inadequate drainage capacity at the toe of long slopes where seepage quantities accumulate and are at their maximum.
- Fines from quarried drainage stone either clogging the drainage layer or accumulating at the toe of the slope thereby decreasing the as-constructed permeability over time.
- Fine, cohesionless, cover soil particles migrating through the filter (if one is present) either clogging the drainage layer, or accumulating at the toe of the slope thereby decreasing the as-constructed outlet permeability over time.
- Freezing of the drainage layer at the toe of the slope, while the soil covered top of the slope thaws, thereby mobilizing seepage forces against the ice wedge at the toe.

If seepage forces of the types described occur, a variation in slope stability design methodology is required. Such an analysis is the focus of this subsection. Note that additional discussion is given in Cancelli and Rimoldi (1989), Thiel and Stewart (1993) and Soong and Koerner (1996).

Consider a cover soil of uniform thickness placed directly above a geomembrane at a slope angle of "β" as shown in Figure 11. Different from previous examples, however, is that within the cover soil exists a saturated soil zone for part or all of the thickness. The saturated boundary is shown as two possibly different phreatic surface orientations. This is because seepage can be built-up in the cover soil in two different ways: a horizontal buildup from the toe upward or a parallel-to-slope buildup outward. These two hypotheses are defined and quantified as a horizontal submergence ratio (HSR) and a parallel submergence ratio (PSR). The dimensional definitions of both ratios are given in Figure 11.

When analyzing the stability of slopes using the limit equilibrium method, free body diagrams of the passive and active wedges are taken with the appropriate forces (now including pore water pressures) being applied. The formulation for the resulting factor-of-safety, for horizontal seepage buildup and then for parallel-to-slope seepage buildup, follows.

The Case of the Horizontal Seepage Buildup. Figure 12 shows the free body diagram of both the active and passive wedge assuming horizontal seepage. Horizontal seepage buildup can occur when toe blockage occurs due to inadequate outlet capacity, contamination or physical blocking of outlets, or freezing conditions at the outlets.

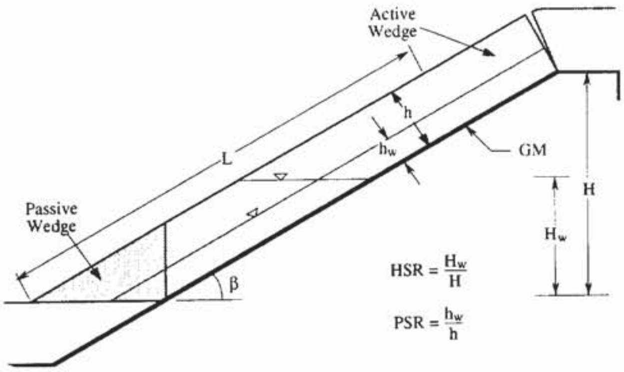


Figure 11. Cross section of a uniform thickness cover soil on a geomembrane illustrating different submergence assumptions and related definitions, Soong and Koerner (1996).

All symbols used in Figure 12 were previously defined except the following:

$\gamma_{sat'd}$ = saturated unit weight of the cover soil
γ_t = total (moist) unit weight of the cover soil
γ_w = unit weight of water
H = vertical height of the slope measured from the toe
H_w = vertical height of the free water surface measured from the toe
U_h = resultant of the pore pressures acting on the interwedge surfaces
U_n = resultant of the pore pressures acting perpendicular to the slope
U_v = resultant of the vertical pore pressures acting on the passive wedge

The expression for finding the factor-of-safety can be derived as follows:

Considering the active wedge,

$$W_A = \left(\frac{\gamma_{sat'd}(h)(2H_w \cos\beta - h)}{\sin 2\beta} \right) + \left(\frac{\gamma_t(h)(H - H_w)}{\sin\beta} \right) \tag{23}$$

$$U_n = \frac{\gamma_w(h)(\cos\beta)(2H_w \cos\beta - h)}{\sin 2\beta} \tag{24}$$

$$U_h = \frac{\gamma_w h^2}{2} \tag{25}$$

$$N_A = W_A(\cos\beta) + U_h(\sin\beta) - U_n \tag{26}$$

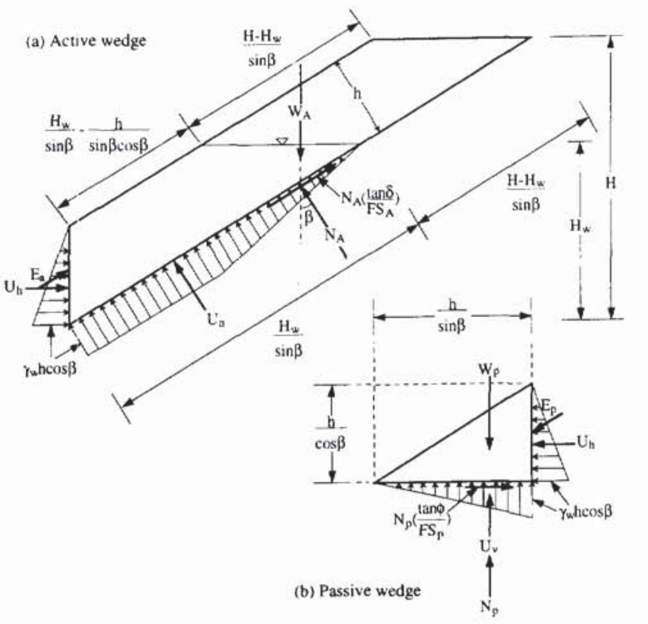


Figure 12. Limit equilibrium forces involved in a finite length slope of uniform cover soil with horizontal seepage buildup.

The interwedge force acting on the active wedge can then be expressed as:

$$E_A = W_A \sin\beta - U_h \cos\beta - \frac{N_A \tan\delta}{FS} \qquad (27)$$

The passive wedge can be considered in a similar manner and the following expressions result:

$$W_P = \frac{\gamma_{sat'd} h^2}{\sin 2\beta} \qquad (28)$$

$$U_V = U_h \cot\beta \qquad (29)$$

The interwedge force acting on the passive wedge can then be expressed as:

$$E_P = \frac{U_h(FS) - (W_P - U_V)\tan\phi}{\sin\beta\tan\phi - \cos\beta(FS)} \qquad (30)$$

By setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2 + bx + c = 0$ which in this case is:

$$a(FS)^2 + b(FS) + c = 0 \qquad (13)$$

where

$$\begin{aligned} a &= W_A \sin\beta\cos\beta - U_h \cos^2\beta + U_h \\ b &= -W_A \sin^2\beta \tan\phi + U_h \sin\beta\cos\beta\tan\phi \\ &\quad - N_A \cos\beta\tan\delta - (W_P - U_V)\tan\phi \\ c &= N_A \sin\beta\tan\delta\tan\phi \end{aligned} \qquad (31)$$

As with previous solution, the resulting FS-value is obtained using Eq. 15.

The Case of Parallel-to-Slope Seepage Buildup. Figure 13 shows the free body diagrams of both the active and passive wedges with seepage buildup in the direction parallel to the slope. Parallel seepage buildup can occur when soils placed above a geomembrane are initially too low in their hydraulic conductivity, or become too low due to long-term clogging from overlying soils which do not have a filter. Identical symbols as defined in the previous cases are used here with an additional definition of h_w equal to the height of free water surface measured in the direction perpendicular to the slope.

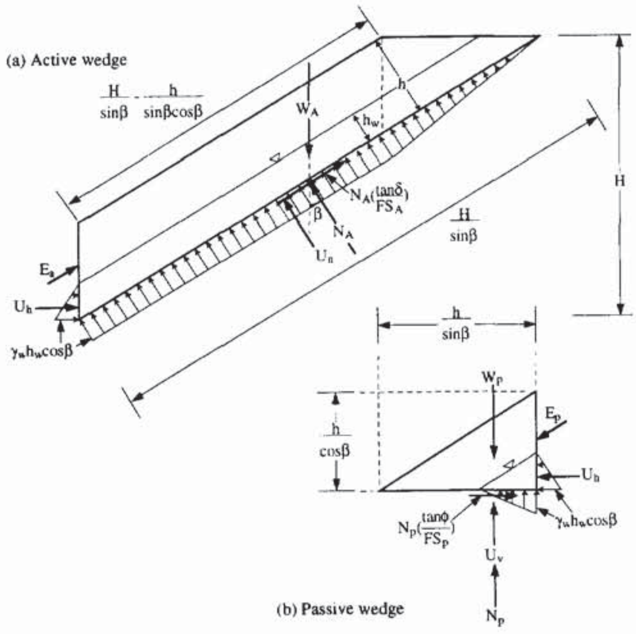


Figure 13. Limit equilibrium forces involved in a finite length slope of uniform cover soil with parallel-to-slope seepage buildup.

Note that the general expression of factor-of-safety shown in Eq. 15 is still valid. However, the a, b and c terms given in Eq. 31 have different definitions in this case owing to the new definitions of the following terms:

$$W_A = \frac{\gamma_t(h-h_w)(2H\cos\beta-(h+h_w))}{\sin 2\beta} + \frac{\gamma_{sat'd}(h_w)(2H\cos\beta-h_w)}{\sin 2\beta} \qquad (32)$$

$$U_n = \frac{\gamma_w h_w \cos\beta(2H\cos\beta-h_w)}{\sin 2\beta} \qquad (33)$$

$$U_h = \frac{\gamma_w(h_w)^2}{2} \qquad (34)$$

$$W_P = \frac{\gamma_t(h^2-h_w^2)+\gamma_{sat'd}(h_w^2)}{\sin 2\beta} \qquad (35)$$

In order to illustrate the behavior of these equations, the design curves of Figure 14 have been developed. They show the decrease in FS-value with increasing submergence ratio for all values of interface friction. Furthermore, the differences in response curves for the parallel and horizontal submergence ratio assumptions are seen to be very small. Note that the curves are developed specifically for variables stated in the legend. Example 3 illustrates the use of the design curves.

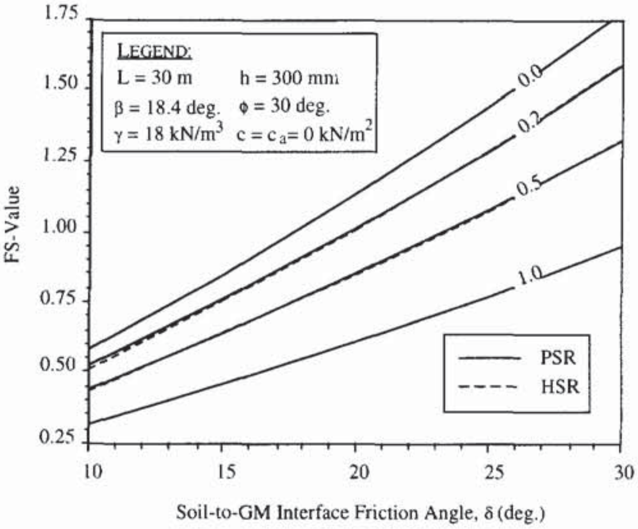


Figure 14. Design curves for stability of cohesionless, uniform thickness, cover soils for different submergence ratios.

Example 3:
Given a 30 m long slope with a uniform thickness cover soil of 300 mm at a dry unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The soil becomes saturated through 50% of its thickness, i.e., it is a parallel seepage problem with PSR = 0.5, and its saturated unit weight increases to 21 kN/m^3. Direct shear testing has resulted in an interface friction angle of 22 deg. with zero adhesion. What is the factor-of-safety at a slope of 3(H)-to-1(V), i.e., 18.4 deg.

Solution:

Solving Eqs. 31 with the values of Eqs. 32 to 35 for the a, b and c terms and then substituting them into Eq. 15 results in the following.

$$\left.\begin{aligned} a &= 51.7\,\text{kN / m} \\ b &= -57.8\,\text{kN / m} \\ c &= 9.0\,\text{kN / m} \end{aligned}\right\} \quad FS = 0.93$$

Comment:
The seriousness of seepage forces in a slope of this type are immediately obvious. Had the saturation been 100% of the drainage layer thickness, the FS-value would have been even lower. Furthermore, the result using a horizontal assumption of saturated cover soil with the same saturation ratio will give identically low FS-values. Clearly, the teaching of this example problem is that adequate <u>long-term</u> drainage above the barrier layer in cover soil slopes must be provided to avoid seepage forces from occurring.

3.4 Consideration of Seismic Forces

In areas of anticipated earthquake activity, the slope stability analysis of a final cover soil over an engineered landfill, abandoned dump or remediated site must consider seismic forces. In the United States, the Environmental Protection Agency (EPA) regulations require such an analysis for sites that have a probability of ≥ 10% of experiencing a 0.10 g peak horizontal acceleration within the past 250 years. For the continental USA this includes not only the western states, but major sections of the midwest and northeast states, as well. If practiced worldwide, such a criterion would have huge implications.

The seismic analysis of cover soils of the type under consideration in this paper is a two-part process:

- The calculation of a FS-value using a pseudo-static analysis via the addition of a horizontal force acting at the centroid of the cover soil cross section.
- If the FS-value in the above calculation is less than 1.0, a permanent deformation analysis is required. The calculated deformation is then assessed in light of the potential damage to the cover soil section and is either accepted, or the slope requires an appropriate redesign. The redesign is then analyzed until the situation becomes acceptable.

The <u>first part</u> of the analysis is a pseudo-static approach which follows the previous examples except for the addition of a horizontal force at the centroid of the cover soil in proportion to the anticipated seismic activity. It is first necessary to obtain an average seismic coefficient (C_S). The bedrock acceleration can be estimated from a seismic zone map, e.g., Algermissen (1991), using the procedures embodied in Richardson, et al (1995). Such maps are available on a worldwide basis. The value of C_s is nondimensional and is a ratio of the bedrock acceleration to gravitational acceleration. This value of C_s is modified using available computer codes such as "SHAKE", see

Schnabel, et al. (1972), for propagation to the site and then to the landfill cover. The computational process within such programs is quite intricate. For detailed discussion see Seed and Idriss (1982) and Idriss (1990). The analysis is then typical to those previously presented.

Using Figure 15, the additional seismic force is seen to be $C_S W_A$ acting horizontally on the active wedge. All additional symbols used in Figure 15 have been previously defined and the expression for finding the FS-value can be derived as follows:

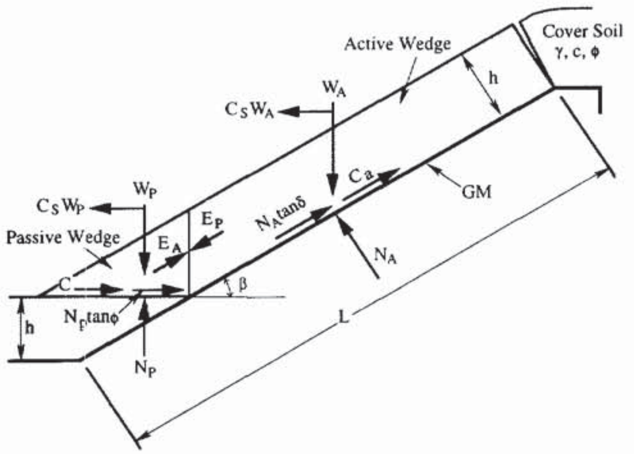


Figure 15. Limit equilibrium forces involved in pseudo-static analysis including use of an average seismic coefficient

Considering the active wedge, by balancing the forces in the horizontal direction, the following formulation results:

$$E_A \cos\beta + \frac{\left(N_A \tan\delta + C_a\right)\cos\beta}{FS} = C_S W_A + N_A \sin\beta \tag{36}$$

Hence the interwedge force acting on the active wedge results:

$$E_A = \frac{(FS)(C_S W_A + N_A \sin\beta)}{(FS)\cos\beta} - \frac{\left(N_A \tan\delta + C_a\right)\cos\beta}{(FS)\cos\beta} \tag{37}$$

The passive wedge can be considered in a similar manner and the following formulation results:

$$E_P \cos\beta + C_S W_P = \frac{C + N_P \tan\phi}{FS} \tag{38}$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi - C_S W_P (FS)}{(FS)\cos\beta - \sin\beta\tan\phi} \tag{39}$$

Again, by setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2 + bx + c = 0$ which in this case is:

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$\begin{aligned} a &= \left(C_S W_A + N_A \sin\beta\right)\cos\beta + C_S W_P \cos\beta \\ b &= -\Big[\left(C_S W_A + N_A \sin\beta\right)\sin\beta\tan\phi \\ &\quad + \left(N_A \tan\delta + C_a\right)\cos^2\beta \\ &\quad + \left(C + W_P \tan\phi\right)\cos\beta\Big] \\ c &= \left(N_A \tan\delta + C_a\right)\cos\beta\sin\beta\tan\phi \end{aligned} \tag{40}$$

The resulting FS-value is then obtained from the following equation:

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{15}$$

Using these concepts, a design curve for the general problem under consideration as a function of seismic coefficient can be developed, see Figure 16. Note that the curve is developed specifically for the variables stated in the legend. Example 4a illustrates the use of the curve.

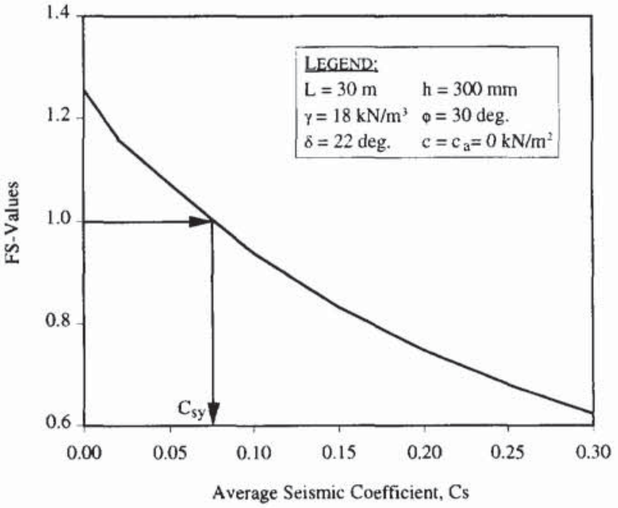


Figure 16. Design curve for a uniformly thick cover soil pseudo-static seismic analysis with varying average seismic coefficients.

Example 4a:

Given a 30 m long slope with uniform thickness cover soil of 300 mm at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is on a geomembrane as shown in Figure 15. Direct shear testing has resulted in an interface friction angle of 22 deg. with zero adhesion. The slope angle is 3(H)-to-1(V), i.e., 18.4 deg. A design earthquake appropriately transferred to the site's cover soil results in an average seismic coefficient of 0.10. What is the FS-value?

Solution:
Solving Eqs. 40 for the values given in the example and substituting into Eq. 15 results in the following FS-value.

$$\left.\begin{array}{l} a = 59.6\,\text{kN / m} \\ b = -66.9\,\text{kN / m} \\ c = 10.4\,\text{kN / m} \end{array}\right\} \quad FS = 0.94$$

Note that the value of FS = 0.94 agrees with the design curve of Figure 16 at a seismic coefficient of 0.10.

Comment:
Had the above FS-value been greater than 1.0, the analysis would be complete. The assumption being that cover soil stability can withstand the short-term excitation of an earthquake and still not slide. However, since the value in this example is less than 1.0, a second part of the analysis is required.

The second part of the analysis is directed toward calculating the estimated deformation of the lowest shear strength interface in the cross section under consideration. The deformation is then assessed in light of the potential damage that may be imposed on the system.

To begin the permanent deformation analysis, a yield acceleration, "C_{sy}", is obtained from a pseudo-static analysis under an assumed FS = 1.0. Figure 16 illustrates this procedure for the assumptions stated in the legend. It results in a value of $C_{sy} = 0.075$. Coupling this value with the time history response obtained for the actual site location and cross section, results in a comparison as shown in Figure 17a. If the earthquake time history response never exceeds the value of C_{sy}, there is no anticipated permanent deformation. However, whenever any part of the time history curve exceeds the value of C_{sy}, permanent deformation is expected. By double integration of the time history curve (which is acceleration), to velocity (Figure 17b) and then to displacement (Figure 17c), the anticipated value of deformation can be obtained. This value is considered to be permanent deformation and is then assessed based on the site-specific implications of damage to the final cover system. Empirical charts, e.g., Makdisi and Seed (1978) can also be used to estimate the permanent deformation. Example 4b continues the previous pseudo-static analysis into the deformation calculation.

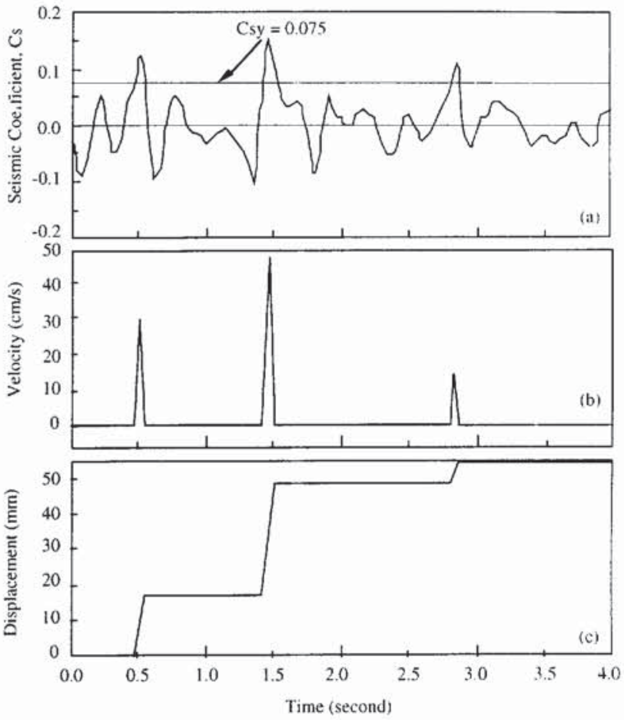


Figure 17. Hypothetical design curves to obtain permanent deformation utilizing (a) acceleration, (b) velocity and (c) displacement curves.

Example 4b:
Continue Example 4a and determine the anticipated permanent deformation of the weakest interface in the cover soil system. The site-specific seismic time-history diagram is given in Figure 17a.

Solution:
The interface of concern is the cover soil-to-geomembrane for this particular example. With a yield acceleration of 0.075 from Figure 16 and the site-specific (design) time history shown in Figures 17a, integration produces Figure 17b and then 17c. The three peaks exceeding the yield acceleration value of 0.075, produce a cumulative deformation of approximately 54 mm. This value is now viewed in light of the deformation capability of the cover soil above the particular interface used at the site. Note that current practice limits such deformation to either 100 or 300 mm depending on site-specific situations, see Richardson et al (1995).

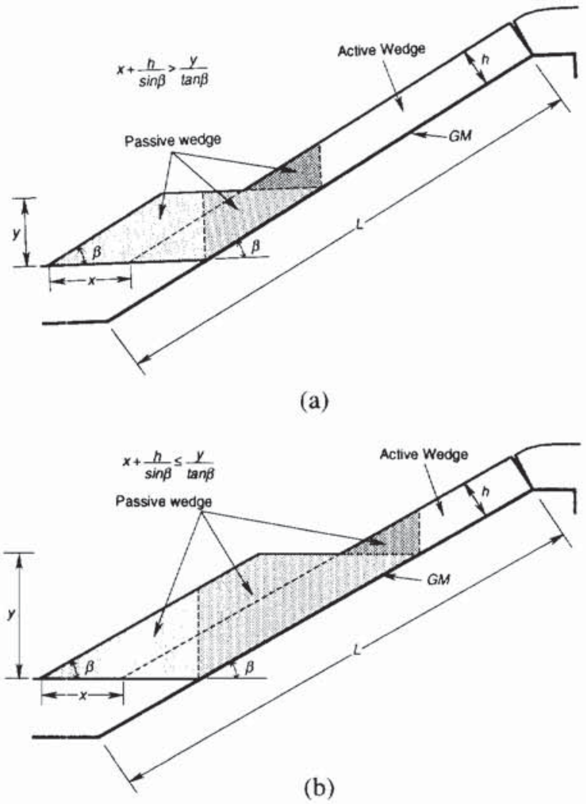
Comments:
An assessment of the implications of deformation (in this example it is 54 mm) is very subjective. For example, this problem could easily have been framed to produce much higher permanent deformation. Such deformation can readily be envisioned in high seismic-prone areas. In addition to an assessment of cover soil stability, the concerns for appurtenances and ancillary piping must also be addressed.

4 SITUATIONS CAUSING THE ENHANCED STABILIZATION OF SLOPES

This section represents a counterpoint to the previous section on slope destabilization situations, in that all situations presented here tend to increase the stability of the slopes. Thus they represent methods to increase the cover soil FS-value. Included are toe berms, tapered cover soils and veneer reinforcement (both intentional and nonintentional). Not included, but very practical in site-specific situations, is to simply decrease the slope angle and/or decrease the slope length. These solutions, however, do not incorporate new design techniques and are therefore not illustrated. They are, however, very viable alternatives for the design engineer.

4.1 Toe (Buttress) Berm

A common method of stabilizing highway slopes and earth dams is to place a soil mass, i.e., a berm, at the toe of the slope. In so doing one provides a soil buttress, acting in a passive state thereby providing a stabilizing force. Figure 18 illustrates the two geometric cases necessary to provide the requisite equations. While the force equilibrium is performed as previously described, i.e., equilibrium along the slope with abutting interwedge forces aligned with the slope angle or horizontal, the equations are extremely long. Due to space limitations (and the resulting trends in FS-value improvement) they are not presented.



(a)

(b)

Figure 18. Dimensions of toe (buttress) berms acting as passive wedges to enhance stability.

Example 5:

Given a 30 m long slope with a uniform cover soil thickness of 300 mm and a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The cover soil is on a geomembrane as shown in Figure 18. Direct shear testing has resulted in a interface friction angle between the cover soil and geomembrane of 22 deg. and zero adhesion. The FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg., was shown in Section 3.1 to be 1.25. What is the increase in FS-value using different sized toe berms with values of x = 1, 2 and 3 m, and gradually increasing y-values?

Solution:

The FS-value response to this type of toe berm stabilization is given in two parts, see Figure 19. Using thickness values of x = 1, 2 and 3 m, the lower berm section by itself is seen to have high FS-values initially, which decrease rapidly as the height of the toe berm increases. This is a predictable response for this passive wedge zone. Unfortunately, the upper layer of soil above the toe berm (the active zone) is only nominally increasing in its FS-value. Note that at the crossover points of the upper and lower FS-values (which is the optimum solution for each set of conditions), the following occurs:

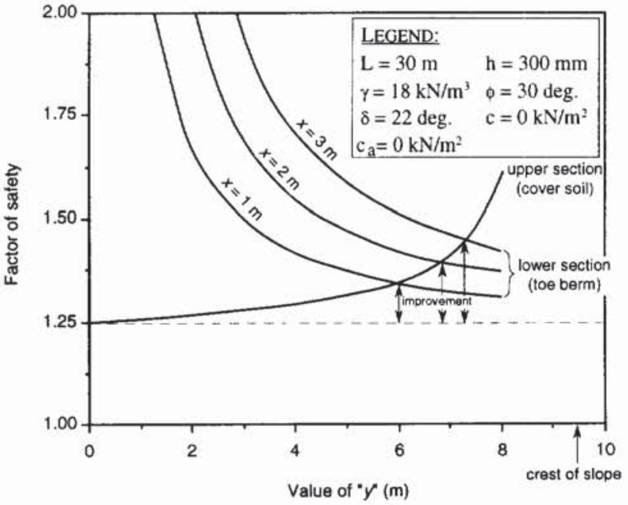


Figure 19. Design curves for FS-values using toe (buttress) berms of different dimensions.

- For x = 1 m; y = 6.0 m (63% of the slope height) and FS = 1.35 (only an 8% improvement in stability)
- For x = 2 m; y = 6.8 m (72% of the slope height) and FS = 1.37 (only a 12% improvement in stability)
- For x = 3 m; y = 7.3 m (77% of the slope height) and FS = 1.40 (only a 16% improvement in stability)

Comment:

Readily seen is that construction of a toe berm is <u>not</u> a viable strategy to stabilize relatively thin layers of sloped cover soil of the type under investigation. Essentially what is happening is that the upper section of the cover soil (the active wedge) above the berm is sliding off of the top of the toe berm. While the upper slope length is becoming shorter (as evidenced by the slight improvement in FS-values), it is only doing so with the addition of a tremendous amount of soil fill. Thus this toe berm concept is a poor strategy for the stabilization of forces oriented in the slope's direction. Conversely, it is an excellent strategy for embankments and dams where the necessary resisting force for the toe berm is horizontal thereby counteracting a horizontal thrust by the potentially unstable soil and/or water mass.

4.2 Slopes with Tapered Thickness Cover Soil

An alternative method available to the designer to increase the FS-value of a given slope is to uniformly taper the cover soil thickness from thick at the toe, to thin at the crest, see Figure 20. The FS-value will increase in approximate proportion to the thickness of soil at the toe. The analysis for tapered cover soils includes the design assumptions of a tension crack at the top of the slope, the upper surface of the cover soil tapered at a constant angle "ω", and the earth pressure forces on the respective wedges oriented at the average of the surface and slope angles, i.e., the E-forces are at an angle of (ω + β)/2. The procedure follows that of the uniform cover soil thickness analysis. Again, the resulting equation is not an explicit solution for the FS, and must be solved indirectly.

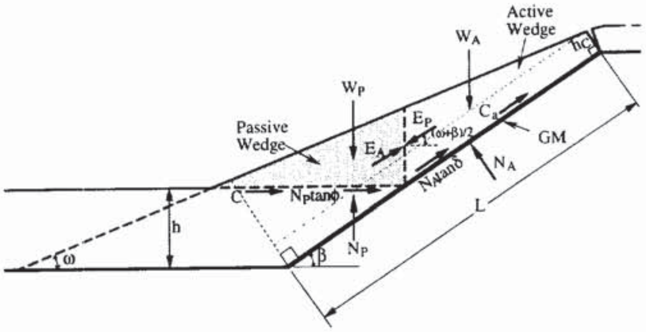


Figure 20. Limit equilibrium forces involved in a finite length slope analysis with tapered thickness cover soil from toe to crest.

All symbols used in Figure 20 were previously defined (see Section 3.1) except the following:

h = thickness of cover soil at bottom of the landfill, measured perpendicular to the base liner

h_C = thickness of cover soil at crest of the slope, measured perpendicular to the slope

y = see Figure 20

$$= \left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega)$$

ω = finished slope angle of cover soil, note that ω < β

The expression for determining the FS-value can be derived as follows:

Considering the active wedge,

$$W_A = \gamma\left[\left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)\left(\frac{y\cos\beta}{2} + h_c\right) + \frac{h_C^2 \tan\beta}{2}\right] \tag{41}$$

$$N_A = W_A \cos\beta \tag{42}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) \tag{43}$$

By balancing the forces in the vertical direction, the following formulations result:

$$E_A \sin\left(\frac{\omega+\beta}{2}\right) = W_A - N_A \cos\beta - \frac{N_A \tan\delta + C_a}{FS}(\sin\beta) \tag{44}$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A \tan\delta + C_a)\sin\beta}{\sin\left(\frac{\omega+\beta}{2}\right)(FS)} \tag{45}$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma}{2\tan\omega}\left[\left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_C}{\cos\beta}\right]^2 \tag{46}$$

$$N_P = W_P + E_P \sin\left(\frac{\omega+\beta}{2}\right) \tag{47}$$

$$C = \frac{\gamma}{\tan\omega}\left[\left(L - \frac{h}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_C}{\cos\beta}\right] \tag{48}$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cos\left(\frac{\omega+\beta}{2}\right) = \frac{C + N_P \tan\phi}{FS} \tag{49}$$

Hence the interwedge force acting on the passive wedge is:

$$E_P = \frac{C + W_P \tan\phi}{\cos\left(\frac{\omega + \beta}{2}\right)(FS) - \sin\left(\frac{\omega + \beta}{2}\right)\tan\phi} \tag{50}$$

By setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2 + bx + c = 0$ which in our case is

$$a(FS)^2 + b(FS) + c = 0 \tag{13}$$

where

$$\begin{aligned} a &= \left(W_A - N_A \cos\beta\right)\cos\left(\frac{\omega + \beta}{2}\right) \\ b &= -\left[\left(W_A - N_A \cos\beta\right)\sin\left(\frac{\omega + \beta}{2}\right)\tan\phi \right. \\ &\quad + \left(N_A \tan\delta + C_a\right)\sin\beta\cos\left(\frac{\omega + \beta}{2}\right) \\ &\quad \left. + \sin\left(\frac{\omega + \beta}{2}\right)\left(C + W_P \tan\phi\right)\right] \\ c &= \left(N_A \tan\delta + C_a\right)\sin\beta\sin\left(\frac{\omega + \beta}{2}\right)\tan\phi \end{aligned} \tag{51}$$

As usual, the resulting FS-value can then be obtained using Eq. 15. To illustrate the use of the above developed equations, the design curves of Figure 21 are offered. They show that the FS-value increases in proportion to greater cover soil thicknesses at the toe of the slope with respect to the thickness at the crest. This is evidenced by a shallower surface slope angle than that of the slope of the geomembrane and the soil beneath, i.e., the value of "ω" being less than "β". Note that the curves are developed specifically for the variables stated in the legend. Example 6 illustrates the use of the curves.

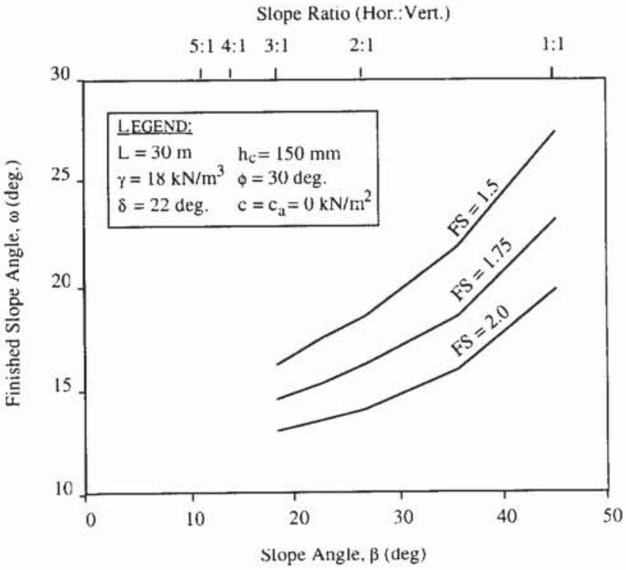


Figure 21. Design curves for FS-values of tapered cover soil thickness.

Example 6:
Given a 30 m long slope with a tapered thickness cover soil of 150 mm at the crest extending at an angle "ω" of 16 deg. to the intersection of the cover soil at the toe. The unit weight of the cover soil is 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The interface friction angle with the underlying geomembrane is 22 deg. with zero adhesion. What is the FS-value at an underlying soil slope angle "β" of 3(H)-to-1(V), i.e., 18.4 deg.?

Solution:
Using Eqs. 51, substituted into Eq. 15 yields the following:

$$\left.\begin{aligned} a &= 37.0\,\text{kN/m} \\ b &= -63.6\,\text{kN/m} \\ c &= 8.6\,\text{kN/m} \end{aligned}\right\} \quad FS = 1.57$$

Comment:
The result of this problem (with tapered thickness cover soil) is FS = 1.57, versus Example 1 (with a uniform thickness cover soil) which was FS = 1.25. Thus the increase in FS-value is 24%. Note, however, that at ω = 16 deg. the thickness of the cover soil normal to the slope at the toe is approximately 1.4 m. Thus the increase in cover soil volume used over Example 1 is from 8.9 to 24.1 m^3/m (≈170%) and the increase in necessary toe space distance is from 1.0 to 4.8 m (≈380%). The trade-offs between these issues should be considered when using the strategy of tapered cover soil thickness to increase the FS-value of a particular cover soil slope.

4.3 Veneer Reinforcement - Intentional

A fundamentally different way of increasing a given slope's factor of safety is to reinforce it with a geosynthetic material. Such reinforcement can be either intentional or non-intentional. By *intentional*, we mean to include a geogrid or high strength geotextile within the cover soil to purposely reinforce the system against instability, see Figure 22. Depending on the type and amount of reinforcement, the majority, or even all, of the driving, or mobilizing, stresses can be supported resulting in major increase in FS-value. By *non-intentional,* we refer to multi-component liner systems where a low shear strength interface is located beneath an overlying geosynthetic(s). In this case, the overlying geosynthetic(s) is inadvertently acting as veneer reinforcement to the composite system. In some cases, the designer may not realize that such geosynthetic(s) are being stressed in an identical manner as a geogrid or high strength geotextile, but they are. The situation where a relatively low strength protection geotextile is placed over a smooth geomembrane and beneath the cover soil is a case in point. Intentional, or non-intentional, the stability analysis is identical. The difference is that the geogrids and/or high strength geotextiles give a major increase in the FS-value, while a protection geotextile (or other lower strength geosynthetics) only nominally increases the FS-value.

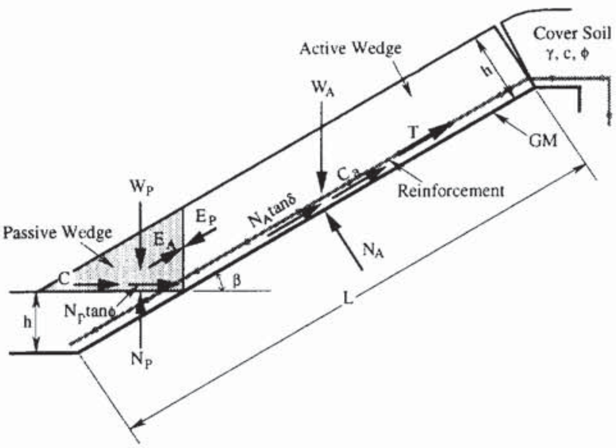


Figure 22. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil including the use of veneer reinforcement.

Seen in Figure 22 is that the analysis follows Section 3.1, but a force from the reinforcement "T", acting parallel to the slope, provides additional stability. This force "T", acts only within the active wedge. By taking free body force diagrams of the active and passive wedges, the following formulation for the factor of safety results. All symbols used in Figure 22 were previously defined (see Section 3.1) except the following:

T = T_{allow}, the allowable (long-term) strength of the geosynthetic reinforcement inclusion

Consider the active wedge and by balancing the forces in the vertical direction, the following formulation results:

$$E_A \sin\beta = W_A - N_A \cos\beta - \left(\frac{N_A \tan\delta + C_a}{FS} + T \right) \sin\beta \qquad (52)$$

Hence the interwedge force acting on the active wedge is:

$$E_A = \frac{(FS)(W_A - N_A \cos\beta - T\sin\beta)}{\sin\beta(FS)} - \frac{(N_A \tan\delta + C_a)\sin\beta}{\sin\beta(FS)} \qquad (53)$$

Again, by setting $E_A = E_P$ (see Eq. 12 for the expression of E_P), the following equation can be arranged in the usual form in which the "a", "b" and "c" terms are defined as follows:

$$a = (W_A - N_A \cos\beta - T\sin\beta)\cos\beta$$

$$b = -[(W_A - N_A \cos\beta - T\sin\beta)\sin\beta\tan\phi + (N_A \tan\delta + C_A)\sin\beta\cos\beta + \sin\beta(C + W_P \tan\phi)]$$

$$c = (N_A \tan\delta + C_a)\sin^2\beta\tan\phi \qquad (54)$$

Again, the resulting FS-value can be obtained using Eq. 15.

As noted, the value of T in the design formulation is T_{allow} which is invariably less than the as-manufactured strength of the geosynthetic reinforcement material. Considering the as-manufactured strength as being T_{ult}, the value should be reduced by such factors as installation damage, creep and long-term degradation. Note that if seams are involved in the reinforcement, a reduction factor should be added accordingly. See Koerner, 1998 (among others), for recommended numeric values.

$$T_{allow} = T_{ult} \left(\frac{1}{RF_{ID} \times RF_{CR} \times RF_{CBD}} \right) \qquad (55)$$

where

T_{allow} = allowable value of reinforcement strength
T_{ult} = ultimate (as-manufactured) value of reinforcement strength
RF_{ID} = reduction factor for installation damage
RF_{CR} = reduction factor for creep
RF_{CBD} = reduction factor for long term chemical/biological degradation

To illustrate the use of the above developed equations, the design curves of Figure 23 have been developed. The reinforcement strength can come from either geogrids or high strength geotextiles. If geogrids are used, the friction angle is the cover soil to the underlying geomembrane, under the assumption that the apertures are large enough to allow for cover soil strike-through. If geotextiles are used, this is not the case and the friction angle is the geotextile to the geomembrane. Also note that this value under discussion is the required reinforcement strength which is essentially T_{allow} in Eq. 55. The curves of Figure 23 clearly show the improvement of FS-values with increasing strength of the reinforcement. Note that the curves are developed specifically for the variables stated in the legend. Example 7 illustrates the use of the design curves.

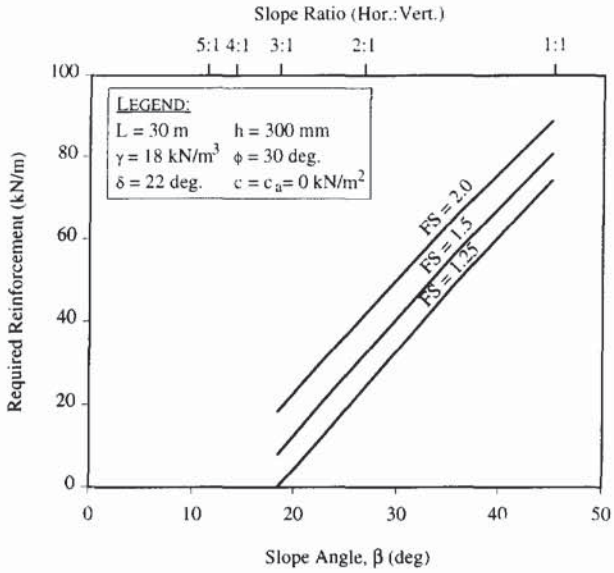


Figure 23. Design curves for FS-values for different slope angles and veneer reinforcement strengths for uniform thickness cohesionless cover soils.

Example 7:

Given a 30 m long slope with a uniform thickness cover soil of 300 mm and a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The proposed reinforcement is a geogrid with an allowable wide width tensile strength of 10 kN/m. Thus reduction factors in Eq. 55 have already been included. The geogrid apertures are large enough that the cover soil will strike-through and provide an interface friction angle with the underlying geomembrane of 22 deg. with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V), i.e., 18.4 deg.?

Solution:

Solving Eqs. 54 and substituting into Eq. 15 produces the following:

$$\left.\begin{aligned} a &= 11.8\,\text{kN / m} \\ b &= -20.7\,\text{kN / m} \\ c &= 3.5\,\text{kN / m} \end{aligned}\right\} \quad \text{FS} = 1.57$$

Comments:

Note that the use of T_{allow} = 10 kN/m in the analysis will require a significantly higher T_{ult} value of the geogrid per Eq. 55. For example, if the summation of the reduction factors in Eq. 55 were 4.0, the ultimate (as-manufactured) strength of the geogrid would have to be 40 kN/m. Also, note that this same type of analysis could also be used for high strength geotextile reinforcement. The analysis follows along the same general lines as presented here.

4.4 Veneer Reinforcement - Nonintentional

It should be emphasized that the preceding analysis is focused on intentionally improving the FS-value by the inclusion of geosynthetic reinforcement. This is provided by geogrids or high strength geotextiles being placed above the upper surface of the low strength interface material. The reinforcement is usually placed directly above the geomembrane or other geosynthetic material.

Interestingly, some amount of veneer reinforcement is often nonintentionally provided by a geosynthetic(s) material placed over an interface with a lower shear strength. Several situations are possible in this regard.

- Geotextile protection layer placed over a geomembrane
- Geomembrane placed over an underlying geotextile protection layer
- Geotextile/geomembrane placed over a compacted clay liner or geosynthetic clay liner
- Multilayered geosynthetics placed over a compacted clay liner or a geosynthetic clay liner

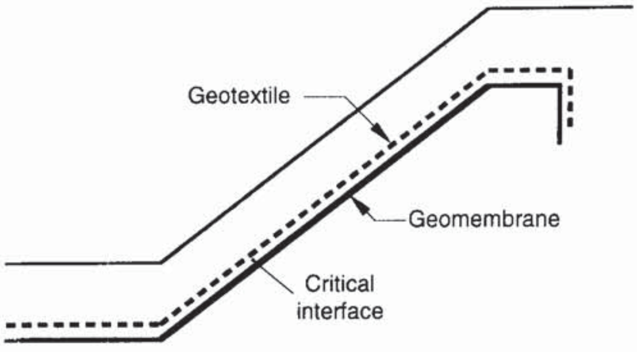
Each of these four situations are illustrated in Figure 24. They represent precisely the formulation of Section 4.3 which is based on Figure 22. On the condition that the geosynthetics above the weakest interface are held in their respective anchor trenches, the overlying geosynthetics provide veneer reinforcement, albeit of a nonintentional type. In the general case, such designs are not recommended although they can indeed provide increased resistance to slope instability of the weakest interface.

In performing calculations of the situations shown in Figure 24, the issue of strain compatibility must be considered. For the slopes shown in Figure 24 a and b, the issue is not important and the full wide width strength of the geotextile and geomembrane, respectively, can be used in the analysis. For the slopes shown in Figure 24 c and d, however, the complete stress vs. strain curves of each geosynthetic layer over the weak interface are necessary. The lowest value of failure strain of any one material dictates the strain at which the other geosynthetics will act. This will invariably be less than the full strength of the other geosynthetics. At this value of strain, however, the allowable strengths are additive and can be used in the analysis. Some detail on this issue is available in Corcoran and McKelvey (1995).

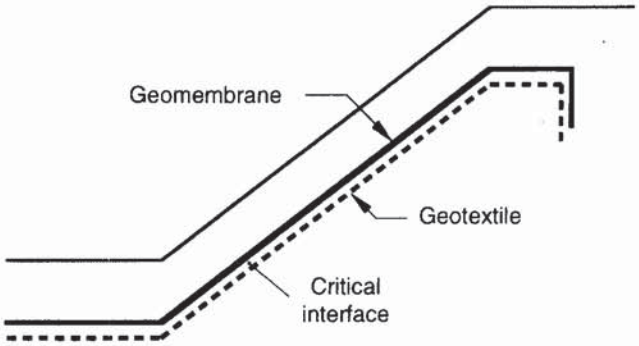
To illustrate the use of the above concepts, examples are given for the four situations shown in Figure 24.

Example 8:

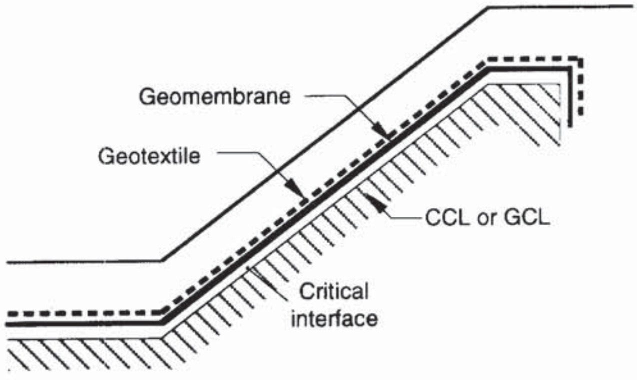
Given four 3(H)-to-1(V), i.e., 18.4 deg. slopes with cover soils as shown in Figures 24 a to d. In each case, the slope is 30 m long with 300 mm of uniformly thick cover soil at a unit weight of 18 kN/m^3. The soil has a friction angle of 30 deg. and zero cohesion, i.e., it is a sand. The friction angle of the critical interface is 10 deg. What are the FS-values using the geosynthetic tensile strength data provided in the following table?



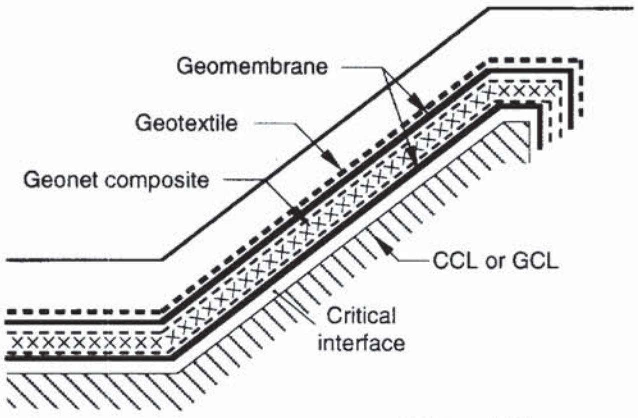
(a) Geotextile sliding on geomembrane



(b) Geomembrane sliding on geotextile



(c) Geotextile and geomembrane sliding on CCL or GCL



(d) Double liner system sliding on CCL or GCL

Figure 24. Various situations illustrating veneer reinforcement, albeit of an nonintentional type.

Values used for numeric examples of nonintentional veneer reinforcement.[1]

Slope type (figure)	GT strength[2] (kN/m)	GM strength[3] (kN/m)	GC strength[4] (kN/m)
24a	25	n/a	n/a
24b	n/a	15	n/a
24c	25	13	n/a
24d	25	13+13	36

Notes:

1. Strengths are product-specific and have been adjusted for strain compatibility.
2. Nonwoven needle punched geotextile of 540 g/m^2
3. Very flexible polyethylene geomembrane 1.0 mm thick
4. Biaxial geonet with two 200 g/m^2 nonwoven needle punched geotextiles thermally bonded to each side

Solution:

Substituting Eqs. 54 into Eq. 15 results in the following data and respective FS-values.

Slope type (figure)	a (kN/m)	b (kN/m)	c (kN/m)	FS-value
24a	7.3	-9.7	1.5	1.15
24b	10.3	-10.3	1.5	0.82
24c	3.4	-9.0	1.5	2.45
24d	-11.0	-6.2	1.5	>10.0

Comments:
While the practice illustrated in these examples of using the overlying geosynthetics as nonintentional veneer reinforcement is not recommended, it is seen to be quite effective when a number of geosynthetics overlying the weak interface are present. On a cumulative basis, they can represent a substantial force as shown in Figure 24d. If one were to rely on such strength, however, it would be prudent to apply suitable reduction factors to each material, and to inform the parties involved of the design situation.

5 SUMMARY

This paper has focused on the mechanics of analyzing slopes as part of final cover systems on engineered landfills, abandoned dumps and remediated waste piles. It also applies to drainage soils placed on geomembrane lined slopes beneath the waste, at least until solid waste is placed against the slope. Numeric examples in all of the sections have resulted in global FS-values. Each section was presented from a designer's perspective in transitioning from the simplest to the most advanced. It should be clearly recognized that there are other approaches to the analyses illustrated in the various examples. References available in the literature by Giroud and Beech (1989), McKelvey and Deutsch (1991), Koerner and Hwu (1991), Giroud et al (1995a), Giroud et al (1995b), Liu et al (1997), and Ling and Leshchinsky (1997) are relevant in this regard. All are based on the concept of limit equilibrium with different assumptions involving particular details, e.g.,

- Existence of a tension crack at the top of slope (filled or unfilled with water)
- Orientation of the failure plane beneath the passive wedge (horizontal or inclined)
- Specific details of construction equipment movement on the slopes in placing the cover soil, particularly the acceleration or deceleration, and the type of equipment itself (e.g., tracked versus wheel equipment)
- Specific details on seepage forces within the drainage layer, including the amount and its orientation
- Specific details on seismic forces, particularly the magnitude and the selection of interface strengths
- Specific details on the geometry of the toe berms or tapered cover soils
- Specific details on the strength and reduction factors used for intentional veneer reinforcement
- Specific details on the strain compatibility issues used with nonintentional veneer reinforcement.

When considering all of these site-specific details, it is readily seen that veneer cover soil analysis and design is a daunting, yet quite tractable, task. For example, one of the reviewers of this paper reanalyzed one of the examples presented herein and another reviewer reanalyzed all of the examples. Both used the analyses of Giroud et al (1995a) and (1995b). They found good agreement in all cases except the nonintentional veneer reinforcement with multiple geosynthetic layers, i.e., the last example presented. It is likely in this regard that different values of mobilized composite strength were being used.

Table 1 summarizes the FS-values of the similarly framed numeric examples presented herein so that insight can be gained from each of the conditions analyzed. Throughout the paper, however, the inherent danger of building a relatively steep slope on a potentially weak interface material, oriented in the exact direction of a potential slide, should have been apparent.

The standard example was purposely made to have a relatively low factor of safety, i.e., FS = 1.25. This FS-value was seen to moderately decrease for construction equipment moving up the slope, but seriously decrease with equipment moving down the slope, i.e., 1.24 to 1.03. It should be noted, however, that the example problems were hypothetical , particularly the equipment examples in the selection of acceleration /deceleration factors. There are an innumerable number of choices to select from, and we have selected values to make the point of proper construction practice. Also, drastically decreasing the FS-value were the influences of seepage and seismicity. The former is felt to be most serious in light of a number of slides occurring after heavy precipitation. The latter is known to be a concern at one landfill in an area of active seismicity.

The sequence of design situations shifted to scenarios where the FS-values were increased over the standard example. Adding soil either in the form of a toe berm or tapered cover soil both increase the FS-value depending on the mass of soil involved. The tapered situation was seen to be more efficient and preferred over the toe berm. Both designs, however, require physical space at the toe of the slope which is often not available. Thus the use of geosynthetic reinforcement was illustrated. By intentional veneer reinforcement it is meant that geogrids or high strength geotextiles are included to resist some, or all, of the driving forces that are involved. The numeric example illustrated an increase in FS-value from 1.25 to 1.57, but this is completely dependent on the type and amount of reinforcement. It was also shown that whenever the weakest interface is located beneath overlying geosynthetics they also act as veneer reinforcement albeit nonintentionally in most cases. The overlying geosynthetic layers must physical fail (or pull out of their respective anchor trenches, see Hullings, 1996) in order for the slope to mobilize the weakest interface strength layer and slide. While this is not a recommended design situation, it does have the effect of increasing the FS-value. The extent of increase varies from a flexible geomembrane to a nonwoven needle punched protection geotextile (both with relatively low strengths) to a multilayered geosynthetic system with 2 to 8 layers of geosynthetics (with very high cumulative strengths).

Table 1. Summary of numeric examples given in this paper for different slope stability scenarios.

Example No.	Situation or condition	Control FS-value	Scenarios decreasing FS-values	Scenarios increasing FS-values
1	standard example*	1.25		
2a	equipment up-slope		1.24	
2b	equipment down-slope		1.03	
3	seepage forces		0.93	
4	seismic forces		0.94	
5	toe (buttress) berm			1.35-1.40
6	tapered cover soil			1.57
7	veneer reinforcement (intentional)			1.57
8	veneer reinforcement (non intentional)			varies

* 30 m long slope at a slope angle of 18.4 deg. with sandy cover soil of 18.4 kN/m^3 dry unit weight with $\phi = 30$ deg. and thickness 300 mm placed on an underlying geosynthetic with a friction angle $\delta = 22$ deg.

6 CONCLUSION

We conclude with a discussion on factor of safety (FS) values for cover soil situations. Note that we are referring to the global FS-value, not reduction factors which necessarily must be placed on geosynthetic reinforcement materials when they are present. In general, one can consider global FS-values to vary in accordance with the site specific issue of required service time (i.e., the anticipated lifetime) and the implication of a slope failure (i.e., the concern). Table 2 gives the general concept in qualitative terms.

Table 2. Qualitative rankings for global factor-of-safety values in performing stability analysis of final cover systems, after Bonaparte and Berg (1987).

Duration→ ↓Concern	Temporary	Permanent
Noncritical	Low	Moderate
Critical	Moderate	High

Using the above as a conceptual guide, the authors recommend the use of the minimum global factor-of-safety values listed in Table 3, as a function of the type of underlying waste for *static* conditions.

Table 3. Recommended global factor-of-safety values for static conditions in performing stability analyses of final cover systems.

Type of Waste→ ↓Ranking	Hazardous waste	Non-hazardous waste	Abandonded dumps	Waste piles and leach pads
Low	1.4	1.3	1.4	1.2
Moderate	1.5	1.4	1.5	1.3
High	1.6	1.5	1.6	1.4

It is hoped that the above values give reasonable guidance in final cover slope stability decisions, but it should be emphasized that engineering judgment and (oftentimes) regulatory agreement is needed in many, if not all, situations.

ACKNOWLEDGMENTS

The opportunity of writing and presenting this paper on behalf of IGS and its Nominating Committee as the 1998 Giroud Lecture is sincerely appreciated. Financial assistance in its preparation by the consortium of Geosynthetic Institute members is gratefully acknowledged.

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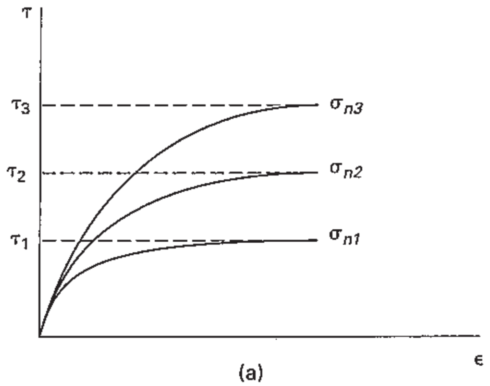
Attachment 2-3

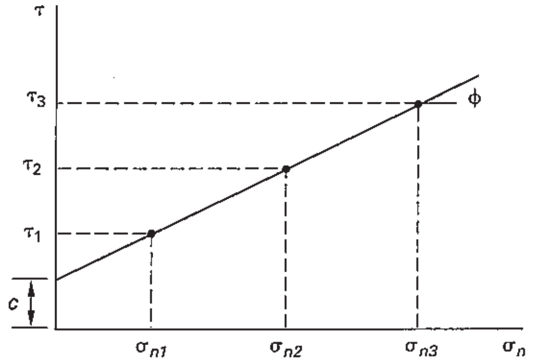
Characteristics of Cover Soil Material

into the direct shear box. The box has a top half and a bottom half that can slide laterally with respect to each other. A normal stress, σ_n, is applied vertically, and then one half of the box is moved laterally relative to the other at a constant rate. Measurements of vertical and horizontal displacement, δ, and horizontal shear load, P_h, are taken. The test is usually repeated at three different vertical normal stresses.

Because of the box configuration, failure is forced to occur on a horizontal plane. Results from each test are plotted as horizontal displacement versus horizontal stress, τ_h (horizontal force divided by the nominal area). *Failure* is determined as the maximum value of horizontal stress achieved. The vertical normal stress and failure stress from each test are then plotted in Mohr's circle space of normal stress versus shear stress.



(a)
stress-strain curves



(b)
Mohr's failure envelope

Figure 35.13 *Graphing Direct-Shear Test Results*

A line drawn through all of the test values is called the *failure envelope* (*failure line* or *rupture line*). The equation for the failure envelope is given by *Coulomb's equation*, which relates the strength of the soil, S, to the normal stress on the failure plane.[10,11,12]

$$S = \tau = c + \sigma \tan \phi \qquad 35.37$$

ϕ is known as the *angle of internal friction*.[13] c is the *cohesion intercept*, a characteristic of cohesive soils. Representative values of ϕ and c are given in Table 35.12.

Table 35.12 *Typical Strength Characteristics*

30 degrees used

group symbol	cohesion (as compacted) c lbf/ft^2 (kPa)	cohesion (saturated) c_{sat} lbf/ft^2 (kPa)	effective stress friction angle ϕ
GW	0	0	> 38°
GP	0	0	> 37°
GM	–	–	> 34°
GC	–	–	> 31°
SW	0	0	38°
SP	0	0	37°
SM	1050 (50)	420 (20)	34°
SM-SC	1050 (50)	300 (14)	33°
SC	1550 (74)	230 (11)	31°
ML	1400 (67)	190 (9)	32°
ML-CL	1350 (65)	460 (22)	32°
CL	1800 (86)	270 (13)	28°
OL	–	–	–
MH	1500 (72)	420 (20)	25°
CH	2150 (100)	230 (11)	19°
OH	–	–	–

(Multiply lbf/ft^2 by 0.04788 to obtain kPa.)

18. TRIAXIAL STRESS TEST

The *triaxial test* is a more sophisticated method than the direct shear test for determining the strength of soils. In the triaxial test apparatus, a cylindrical sample is stressed completely around its peripheral surface by pressurizing the sample chamber. This pressure is referred to as the *confining stress*. Then, the soil is loaded vertically to failure through a top piston. The confining stress is kept constant while the axial stress is varied. The radial component of the confining stress is called the *radial stress*, σ_R, and represents the minor principal stress, σ_3. The combined stresses at the ends of the sample (confining stress plus applied vertical stress) are called the *axial stress*, σ_A, and represent the major principal stress, σ_1.[14]

Results of a triaxial test at a given chamber pressure are plotted as a stress-strain curve. Two such examples are illustrated in Fig. 35.14. The axial component of

---

[10] Equation 35.37 is also known as the Mohr-Coulomb equation.

[11] The ultimate shear strength may be given the symbol S in some soils books.

[12] τ and σ in Coulomb's equation are the shear stress and normal stress, respectively, on the failure plane at failure.

[13] In a physical sense, the angle of internal friction for cohesionless soils is the angle from the horizontal naturally formed by a pile. For example, a uniform fine sand makes a pile with a slope of approximately 30°. For most soils, the natural angle of repose will not be the same as the angle of internal friction, due to the effects of cohesion.

[14] In reality, the triaxial test apparatus is a "biaxial" device because it controls stresses in only two directions: radial and axial.

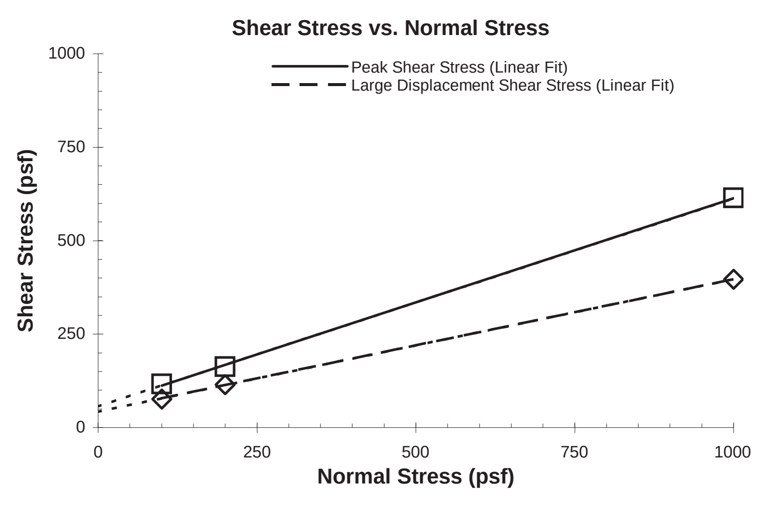
Interface Friction Test Report

Client:
Project:
Test Date:

TRI Log#:
Test Method: ASTM D 5321

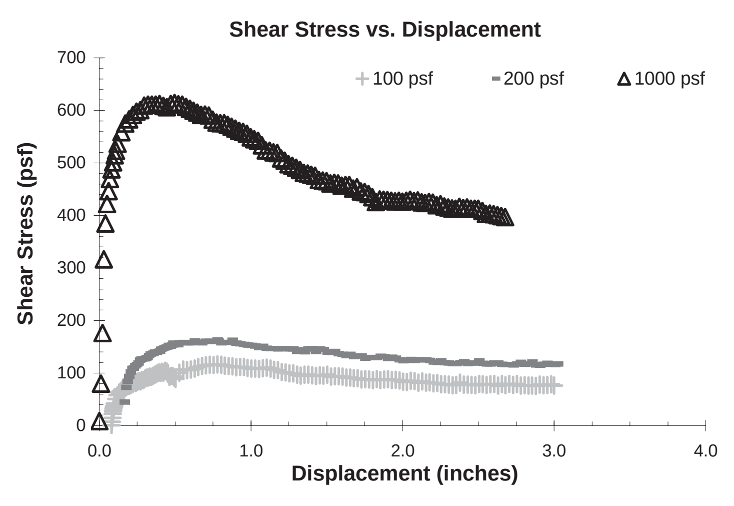
Quality Review/Date

Tested Interface: Double-sided Geocomposite vs. GSE 40 mil LLDPE Texture Geomembrane (104150610)



Test Results		
	Peak	Large Displacement (@ 3.0 in.)
Friction Angle (degrees):	29.1	19.5
Y-intercept or Adhesion (psf):	56	42

Shearing occurred at the interface.



Test Conditions

Upper Box & Double-sided Geocomposite

Lower Box GSE 40 mil LLDPE Textured Geomembrane

Box Dimensions: 12"x12"x4"

Interface Conditioning: Interface soaked and loading applied for a minimum of 1 hour prior to shear.

Test Condition: Wet

Shearing Rate: 0.04 inches/minute

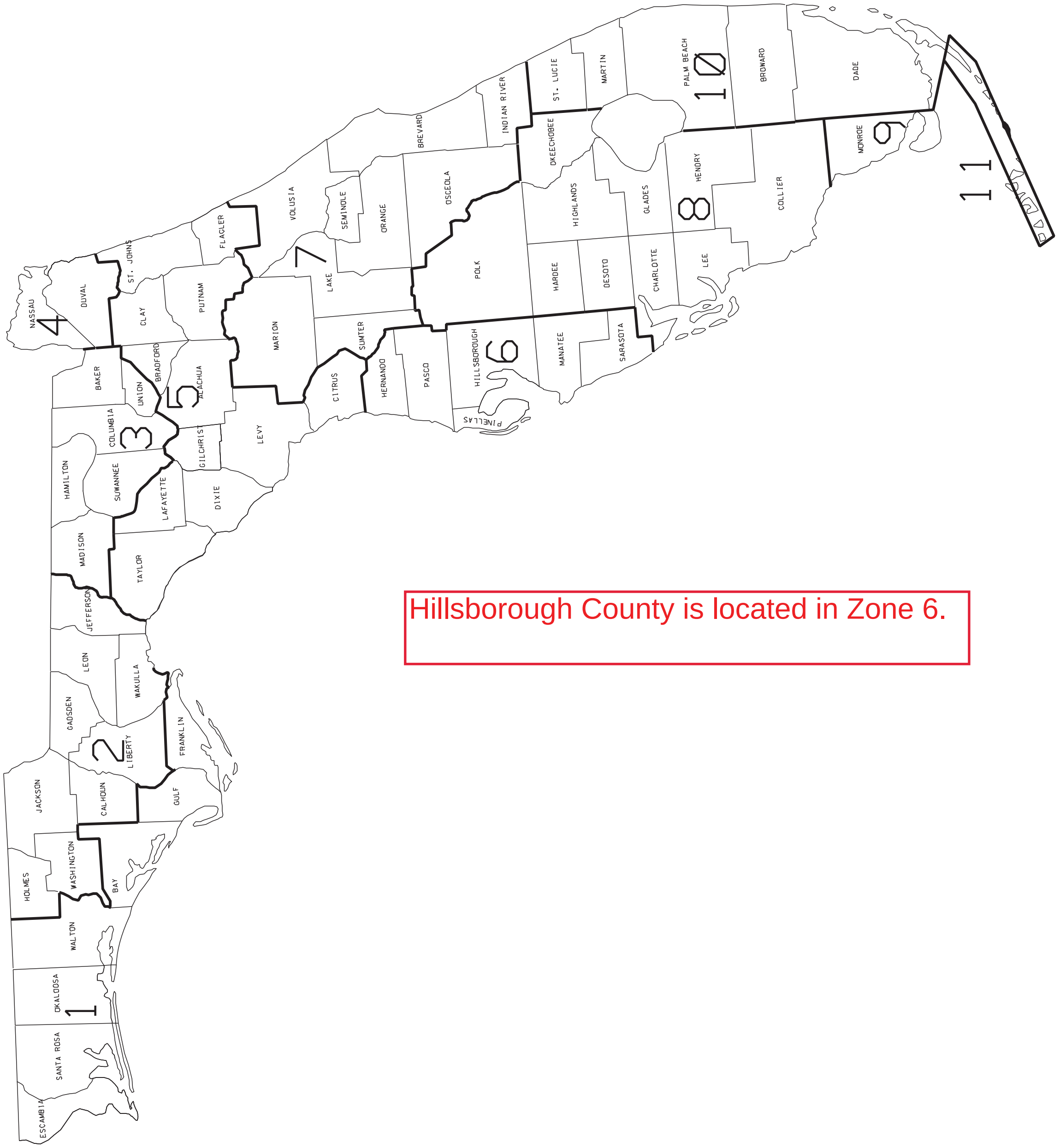
Test Data

Specimen No.	1	2	3
Bearing Slide Resistance (lbs)	3	3	18
Normal Stress (psf)	100	200	1000
Corrected Peak Shear Stress (psf)	116	163	614
Corrected Large Displacement Shear Stress (psf)	76	115	397
Peak Secant Angle (degrees)	49.3	39.1	31.6
Large Displacement Secant Angle (degrees)	37.2	29.9	21.7
Asperity (mils)	19.8	23.2	23.0

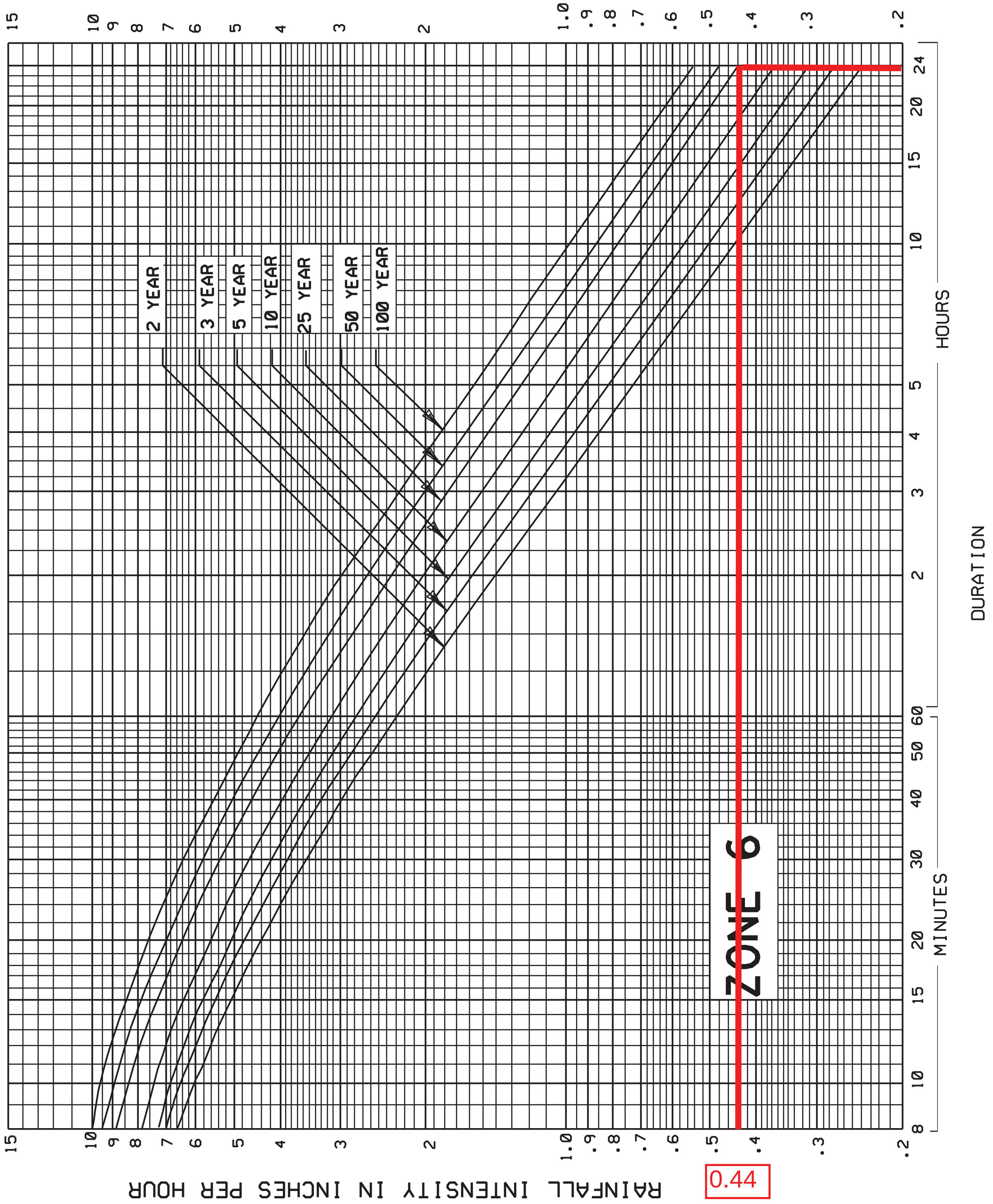
Attachment 2-4

IDF Curve

ZONES FOR PRECIPITATION IDF CURVES DEVELOPED BY THE DEPARTMENT



Hillsborough County is located in Zone 6.



RAINFALL INTENSITY-DURATION-FREQUENCY CURVES
ZONE 6

Attachment 2-5

300 MIL GSE FabriNet UF Transmissivity Calculations

SCS ENGINEERS

SHEET 1 OF 5

CLIENT	PROJECT	JOB NO.
Hillsborough County	Southeast County Landfill	09215600.03

SUBJECT	BY	DATE
Final Closure System Bi-planar Geocomposite Transmissivity/Hydraulic Conductivity Calculations	SRF	6/2/2017
	CHECKED	DATE
	KLS	6/6/2017

Objective:

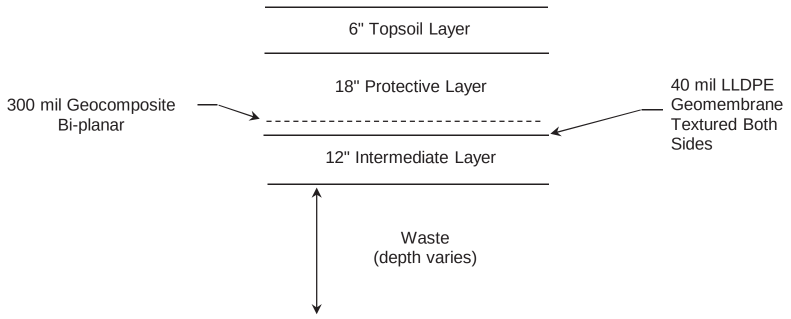
To determine the design hydraulic conductivity, design thickness, and porosity of the geocomposite selected for use in the final cover system using manufacturer's testing data.
The calculations for the long-term transmissivity of the geocomposite are based on 100 hour transmissivity values.

References:

1. Attachment 1 - GRI Standard - GC8 Technical Release, April 17, 2001.
2. Attachment 2 - GSE Drainage Design Manual data for bi-planar 100 hour transmissivity values (FabriNet UF Geocomposite).
3. Attachment 3 - Soil properties

Procedure:

1. Geocomposite properties are dependent on landfill load, rainfall and other conditions. Determine loads on geocomposite.
2. GRI Standard - GC8 is a way to determine geocomposite allowable flow rates based on site specific landfill conditions.
3. Calculate the hydraulic conductivity (k) at final landfill conditions.



SCS ENGINEERS

SHEET 2 OF 5

CLIENT Hillsborough County	PROJECT Southeast County Landfill	JOB NO. 09215600.03

SUBJECT Final Closure System Bi-planar Geocomposite Transmissivity/Hydraulic Conductivity Calculations	BY SRF CHECKED KLS	DATE 6/2/2017 DATE 6/6/2017

Equations:

Refer to Attachment 1 developed from Equation (1) and Equation (2) pg GC8-3

$$\theta_{allow} = \frac{\theta_{ultimate}}{RF_{IN} * RF_{CC} * RF_{BC} * RF_{CR} * FS}$$

Where:

θ_{allow} = Allowable transmissivity

$\theta_{ultimate}$ = Ultimate transmissivity (manufacturer's) under simulated conditions for 100 hours

RF_{IN} = Reduction Factor for elastic deformation, or intrusion of the adjacent geotextiles into the drainage channel

RF_{CC} = Reduction Factor for Chemical Clogging and/or precipitation of chemicals in the drainage core space

RF_{BC} = Reduction Factor for Biological Clogging in the drainage core space

RF_{CR} = Reduction Factor for Creep deformation of the drainage core and/or adjacent geotextile into the drainage channel

FS = Factor of Safety

$$RF_{CR} = \left[\frac{(t'/t) - (1 - n_{original})}{(t_{CR}/t) - (1 - n_{original})}\right]^3$$

Refer to Attachment 1 Equation (6) pg GC8-7

Where:

t' = Thickness at 100 hours

t = Virgin thickness

t_{CR} = Thickness at > 100 hours

$n_{original}$ = Original porosity

$$= 1 - \left[\frac{\mu}{\rho \times t}\right]$$

Refer to Attachment 1 Equation (7) pg GC8-7

μ = mass unit area

ρ = density

$$k = \frac{\theta_{allow}}{t'}$$

Where:

k = Hydraulic conductivity, cm/sec

Note:

RF_{IN} accounts for the geotextile encroaching on the geonet under a constant loading. A 100-hour transmissivity test accounts for intrusion. After the 100-hour seat time, the geotextile has already begun to intrude into the geonet, therefore, the transmissivity value reflects the intrusion. The transmissivity values for these calculations are all based on the 100-hour test, therefore, RF_{IN} = 1.0.

SCS ENGINEERS

SHEET 3 OF 5

CLIENT Hillsborough County	PROJECT Southeast County Landfill	JOB NO. 09215600.03

SUBJECT Final Closure System Bi-planar Geocomposite Transmissivity/Hydraulic Conductivity Calculations	BY SRF	DATE 6/2/2017
	CHECKED KLS	DATE 6/6/2017

Objective: To determine the load on the final cover geocomposite under final conditions.

Known: Landfill cross-section

Topsoil cover = 100 pcf Refer to Attachment 3

Protective cover = 100 pcf Refer to Attachment 3

Final Cover - 6-inch top soil cover + 18-inch protective cover

Material	Density	Material (ft)	(psf)		
Topsoil cover	100.0	0.5	50.0		
Protective cover	100.0	1.5	150.0		
Total		**2.0**	**200.0**	**=>**	**1,000**

SCS ENGINEERS

SHEET 4 OF 5

CLIENT	PROJECT	JOB NO.
Hillsborough County	Southeast County Landfill	09215600.03

SUBJECT	BY	DATE
Final Closure System Bi-planar Geocomposite Transmissivity/Hydraulic Conductivity Calculations	SRF	6/2/2017
	CHECKED	DATE
	KLS	6/6/2017

BI-PLANAR (FINAL COVER SYSTEM)

Purpose:

Calculate the design transmissivity, k, of a 300-mil bi-planar geocomposite under boundary conditions for final closure loading conditions.

From the GSE technical department, the following Transmissivity (θ) values are known:
(Based on FabriNet UF geocomposite specifications).

FabriNet 300-6-6	soil/geocomposite/geomembrane
@ 25% Gradient	Manufacturer's 100 hour θ_{100} Data
Load (psf)	(m^2/sec)
1,000	1.70E-03

Reduction Factors

RF - Intrusion, RF_{IN}

RF - Chemical Clogging, RF_{CC}

RF - Biological Clogging, RF_{BC}

RF - Creep, RF_{CR}

FS - Factor of Safety

thickness, t = 300 mil
0.30 inches
0.762 cm

Equations

$$\theta_{allow} = \frac{\theta_{ultimate}}{RF_{IN} * RF_{CC} * RF_{BC} * RF_{CR} * FS}$$

$$t' = \frac{t}{RF_{CR}}$$

$$k = \frac{\theta_{allow}}{t'}$$

Landfill Caps

Chemical Clogging RF_{CC} = 1.0 to 1.2 Refer to Attachment 1 pg GC8-9

Biological Clogging RF_{BC} = 1.2 to 3.5 Refer to Attachment 1 pg GC8-9

SCS ENGINEERS

SHEET 5 OF 5

CLIENT	PROJECT	JOB NO.
Hillsborough County	Southeast County Landfill	09215600.03

SUBJECT	BY	DATE
Final Closure System Bi-planar Geocomposite Transmissivity/Hydraulic Conductivity Calculations	SRF	6/2/2017
	CHECKED KLS	DATE 6/6/2017

BI-PLANAR (FINAL COVER SYSTEM)

Final Cover - 6-inch top soil cover + 18-inch protective cover

Reduction Factors

RF_{IN} = 1.0

RF_{CC} = 1.0

RF_{BC} = 1.2

RF_{CR} = 1.0

FS = 2.0

Refer to Attachment 1 pg GC8-9

thickness, t = 300 mil

0.30 inches

0.762 cm

@ 25% Gradient					
(psf)	(m^2/sec)	(m^2/sec)	(cm^2/sec)	(cm)	(cm/sec)
1,000	1.70E-03	7.08E-04	7.1	0.762	9.3

t' = 0.300 inches

Attachment 2-5-1

GRI Standard - GC8 Technical Release, April 17, 2001

Geosynthetic Institute
475 Kedron Avenue
Folsom, PA 19033-1208 USA
TEL (610) 522-8440
FAX (610) 522-8441

Original: April 17, 2001

GRI Standard GC8*

Standard Guide for

Determination of the Allowable Flow Rate of a Drainage Geocomposite

This specification was developed by the Geosynthetic Research Institute (GRI), with the cooperation of the member organizations for general use by the public. It is completely optional in this regard and can be superseded by other existing or new specifications on the subject matter in whole or in part. Neither GRI, the Geosynthetic Institute, nor any of its related institutes, warrant or indemnifies any materials produced according to this specification either at this time or in the future.

1. Scope

1.1 This guide presents a methodology for determining the allowable flow rate of a candidate drainage geocomposite. The resulting value can be used directly in a hydraulics-related design to arrive at a site-specific factor of safety.

1.2 The procedure is to first determine the candidate drainage composite's flow rate for 100-hours under site-specific conditions, and then modify this value by means of creep reduction and clogging reduction factors.

1.3 For aggressive liquids, a "go-no go" chemical resistance procedure is suggested. This is a product-specific verification test for both drainage core and geotextile covering.

1.4 The type of drainage geocomposites under consideration necessarily consists of a drainage core whose purpose it is to convey liquid within its manufactured plane. The drainage core can be a geonet, 3-D mesh, built-up columns, single or double cuspations, etc.

1.5 The drainage core usually consists of a geotextile on its upper and/or lower surface. In some cases, the drainage core is used by itself. The guide addresses all of these variations.

1.6 The guide is also applicable to thick nonwoven geotextiles when they are utilized for their drainage capability.

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*This GRI standard is developed by the Geosynthetic Research Institute through consultation and review by the member organizations. This specification will be reviewed at least every 2-years, or on an as-required basis. In this regard it is subject to change at any time. The most recent revision date is the effective version.

1.7 All types of polymers are under consideration in this guide.

1.8 The guide does not address the required (or design) flow rate to which a comparison is made for the final factor of safety value. This is clearly a site-specific issue.

2. Referenced Documents

2.1 ASTM Standards

D1987 – "Test Method for Biological Clogging of Geotextile or Soil/Geotextile Filters"

D2240 – "The Method for Rubber Property – Durometer Hardness"

D4716 – "Test Method for Constant Head Hydraulic Transmissivity (In Plane Flow) of Geotextiles and Geotextile Related Products"

D5322 – "Standard Practice for Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids"

D6364 – "Test Method for Determining the Short-Term Compression Behavior of Geosynthetics"

D6388 – "Standard Practice for Tests to Evaluate the Chemical Resistance of Geonets to Liquids"

D6389 – "Standard Practice for Tests to Evaluate the Chemical Resistance of Geotextiles to Liquids"

2.2 GRI Standards

GS 4 Test Method for Time Dependent (Creep) Deformation Under Normal Pressure

2.3 Literature

Giroud, J.-P., Zhao, A. and Richardson, G. N. (2000), "Effect of Thickness Reduction on Geosynthetic Hydraulic Transmissivity," Geosynthetics International, Vol. 7, Nos. 4-6, pp. 433-452.

Koerner, R. M. (1998), Designing with Geosynthetics, Prentice Hall Publishing Co., Englewood Cliffs, NJ, 761 pgs.

3. Summary of Guide

3.1 This guide presents the necessary procedure to be used in obtaining an allowable flow rate of a candidate drainage geocomposite. The resulting value is then compared to a required (or design) flow rate for a product-specific and site-specific factor of safety. The guide does not address the required (or design) flow rate value, nor the subsequent factor of safety value.

3.2 The procedures recommended in this guide use either ASTM or GRI test methods.

3.3 The guide is applicable to all types of drainage geocomposites regardless of their core configuration or geotextile type. It can also be used to evaluate thick nonwoven geotextiles.

4. Significance and Use

4.1 The guide is meant to establish uniform test methods and procedures in order for a designer to determine the allowable flow rate of a candidate drainage geocomposite for site-specific conditions.

4.2 The guide requires communication between the designer, testing organization and manufacturer in setting site-specific control variables such as product orientation, stress level, stress duration, type of permeating liquid and materials below/above the geocomposite test specimen.

4.3 The guide is useful to testing laboratories in that a prescribed guide is at hand to provide appropriate data for both designer and manufacturer clients.

5. Structure of the Guide

5.1 Basic Formulation – This guide is focused on determination of a "q_{allow}" value using the following formula:

$$q_{\text{allow}} = q_{100}\left[\frac{1}{RF_{CR} \times RF_{CC} \times RF_{BC}}\right] \quad (1)$$

where

q_{allow} = allowable flow rate
q_{100} = initial flow rate determined under simulated conditions for 100-hour duration
RF_{CR} = reduction factor for creep to account for long-term behavior
RF_{CC} = reduction factor for chemical clogging
RF_{BC} = reduction factor for biological clogging

Note 1: By simulating site-specific conditions (except for load duration beyond 100 hours and chemical/biological clogging), additional reduction factors such as intrusion need not be explicitly accounted for.

Note 2: The value of q_{allow} is typically used to determine the product-specific and site-specific flow rate factor of safety as follows:

$$FS = \frac{q_{\text{allow}}}{q_{\text{reqd}}} \quad (2)$$

The value of "q_{reqd}" is a design issue and is not addressed in this guide. Likewise, the numeric value of the factor-of-safety is not addressed in this guide. Suffice it to say that, depending on the duration and criticality of the situation, FS-values should be conservative unless experience allows otherwise.

5.2 Upon selecting the candidate drainage geocomposite product, one must obtain the 100-hour duration flow rate according to the ASTM D4716 transmissivity test. This establishes the base value to which drainage core creep beyond 100-hours and clogging from chemicals and biological matter must be accounted for.

> Note 3: It is recognized that the default duration listed in ASTM D4716 is 15-minutes. This guide purposely requires that the test conditions be maintained for 100-hours.

5.3 Reduction Factor for Creep – This is a long-term (typically 10,000 hours) compressive load test focused on the stability and/or deformation of the drainage core without the covering geotextiles. Stress orientation can be perpendicular or at an angle to the test specimen depending upon site-specific conditions.

5.4 Chemical and/or Biological Clogging – The issue of long term reduction factors to account for clogging within the core space is a site-specific issue. The issue is essentially impractical to simulate in the laboratory, hence a table is provided for consideration by the designer.

5.5 Chemical Resistance/Durability – This procedure results in a "go-no go" decision as to potential chemical reactions between the permeating liquid and the polymers comprising the drainage core and geotextiles. The issue will be addressed in this guide but is not a reduction factor, per se.

6. Determination of the Base Line Flow Rate (q_{100})

6.1 Using the ASTM D4716 transmissivity test with the conditions stated below (unless otherwise agreed upon by the parties involved), determine the 100-hour flow rate of the drainage geocomposite under consideration.

6.1.1 <u>The test specimen shall be the entire geocomposite</u>. If geotextiles are bonded to the drainage core, they shall not be removed and the entire geocomposite shall be tested as a unit. A minimum of three replicate samples in the site-specific orientation shall be tested and the results averaged for the reported value.

6.1.2 Specimen size shall be 300 × 300 mm (12 × 12 in.) within the stressed area.

6.1.3 The specimen orientation is to be agreed upon by the designer, testing laboratory and manufacturer. In this regard, it should be recognized that the specimen orientation during testing has to match the proposed installation orientation. Thus the site-specific design governs both the testing orientation and subsequent field installation orientation.

6.1.4 Specimen substratum shall be one of the following four options. The decision of which is made by the project designer, testing organization and manufacturer. The options are (i) rigid platen, (ii) foam, (iii) sand or (iv) site-specific soil or other material.

6.1.4.1 If a rigid platen is used the choices are usually wood, plastic or metal. The testing laboratory must identify the specifics of the material used.

6.1.4.2 If closed cell foam is used, it shall be 12 mm (0.5 in.) thick and a maximum durometer of 2.0 as measured in ASTM D2240, Type D.

6.1.4.3 If sand is used it shall be Ottawa test sand at a relative density of 85%, water content of 10% and compacted thickness of 25 mm (1.0 in.).

6.1.4.4 If site-specific soil or other material is used it must be carefully considered and agreed upon between the parties involved. Size, gradation, moisture content, density, etc., are all important considerations.

6.1.5 Specimen superstratum shall also be one of the four same options as mentioned in § 6.1.3 above. It need not be the same as the substratum.

6.1.6 The applied stress level is at the discretion of the designer, testing organization and manufacturer. Unless stated otherwise, the orientation shall be normal to the test specimen.

6.1.7 The duration of the loading shall be for 100 hours. A single site-specific data point is obtained at that time, i.e., it is not necessary to perform intermediate flow rate testing, unless otherwise specified by the various parties involved.

6.1.8 The hydraulic gradient at which the above data point is taken (or a range of hydraulic gradients) is at the discretion of the designer, testing organization and manufacturer.

6.1.9 The permeating liquid is to be tap water, unless agreed upon otherwise by the designer, testing organization, and manufacturer.

6.1.10 Calculations

$$Q = kiA \tag{3}$$

$$Q = ki(Wt)$$

$$Q/W = \theta i \tag{4}$$

$$q = \theta i \tag{5}$$

where

Q = flow rate per unit time (m^3/sec)
k = permeability (m/sec)
i = hydraulic gradient (= H/L)
H = head loss across specimen (m)
L = length of specimen (m)
A = cross sectional area of specimen (m^2)
W = width of specimen (m)
t = thickness of specimen (m)
θ = transmissivity (m^3/sec-m or m^2/sec)
q = flow rate per unit width (m^2/sec)

The results can be presented as flow rate per unit width (Q/W), or as transmissivity (θ), as agreed upon by the parties involved.

7. Reduction Factor for Creep

7.1 Using the GRI GS4 test method or ASTM D6364 (mod.) for time dependent (creep) deformation, the candidate drainage core is placed under compressive stress and its decrease in thickness (deformation) is monitored over time.

> Note 4: This is not a flow rate test, although the test specimen can be immersed in a liquid to be agreed upon by the designer, testing organization, and manufacturer. However, it is usually a test conducted without liquid.

7.1.1 The test specimen shall be the drainage core only. If geotextiles are bonded to the drainage core they should be carefully removed. Alternatively, a sample of the drainage core can be obtained from the manufacturer before the geotextiles are attached. A minimum of three replicate tests shall be performed and the results averaged for the reported value.

7.1.2 Specimen size should be 150 × 150 mm (6.0 × 6.0 in.) and placed in a rigid box made from a steel base and sides. The steel load plate above the test specimen shall be used to transmit a constant stress over time. Deformation of the upper plate is measured by at least two dial gauges and the results averaged accordingly.

> Note 5: For high stress conditions requiring a large size and number of weights with respect to laboratory testing and safety, the specimen size can be reduced to 100 × 100 mm (4.0 × 4.0 in.).

7.1.3 Specimen substratum and superstratum shall be rigid platens. Alternatively, a 1.5 mm (60 mil) thick HDPE geomembrane can be placed against the drainage core with the steel plates as back-ups.

7.1.4 The test specimen shall be dry unless water or a simulated or site-specific leachate is agreed upon by the parties involved.

7.1.5 The normal stress magnitude(s) shall be the same as applied in the transmissivity test described in Section 6.0. Alternatively, it can be as agreed upon by the designer, testing organization, and manufacturer.

7.1.6 The load inclination shall be normal to the test specimen. If there exists a tendency for the core structure to deform laterally, separate tests at the agreed upon load inclinations shall also be performed at the discretion of the parties involved.

7.1.7 The dwell time shall be 10,000 hours. If, however, this is a confirmation test (or if a substantial data base exists on similar products of the same type), the dwell time can be reduced to 1000 hours. This decision must be made with agreement between the designer, testing organization, and manufacturer.

> Note 6: Alternative procedures to arrive at an acceptable value for the creep reduction factor based on shorter test times (e.g., the use of time-temperature superposition or stepped isothermal method) may be acceptable if agreed upon by the various parties involved.

7.1.8 The above process results in a set of creep curves similar to Figure 1(a). The curves are to be interpreted as shown in Figure 1(b). The reduction factor for creep of the core is interpreted according to the following formulas, after Giroud, Zhao and Richardson (2000).

$$RF_{CR} = \left[\frac{(t_{CO}/t_{original}) - (1 - n_{original})}{(t_{CR}/t_{original}) - (1 - n_{original})}\right]^3 \qquad (6)$$

where

RF_{CR} = reduction factor for creep
$t_{original}$ = original thickness (m)
t_{CO} = thickness at 100-hours (m)
t_{CR} = thickness at >>100-hours, e.g., at 10,000 hours (m)
$n_{original}$ = original porosity (see Equation 7)

$$n_{original} = 1 - \frac{\mu}{\rho t_{original}} \qquad (7)$$

where

μ = mass per unit area (kg/m^2)
ρ = density of the formulation (kg/m^3)

7.1.9 The above illustrated numeric procedure is <u>not applicable</u> to drainage geocomposites which include geotextiles. It is for the drainage core only.

<u>Example</u>: A HDPE geonet has the following properties: mass per unit area $\mu = 1216$ g/m^2 (or 1.216 kg/m^2); density $\rho = 950$ kg/m^2 and original thickness of 8.55 mm.

Test specimens were evaluated according to ASTM D4716 for 100 hours and the average thickness decreased to 7.14 mm. A 10,000 hour creep test was then performed on a representative specimen according to GRI-GS4 and the resulting thickness further decreased to 6.30 mm. Thus Δy in Figure 1(b) is 7.14 – 6.30 = 0.84 mm. Determine the creep reduction factor "RF_{CR}".

<u>Solution</u>: The porosity n, is calculated according to Eq. (7) as follows

$$n_{original} = 1 - \frac{\mu}{\rho t_{original}}$$

$$= 1 - \frac{1.216}{(950)(0.00855)}$$

$$= 1 - 0.150$$

$$n_{original} = 0.850$$

The reduction factor for creep is calculated according to Eq. (6) as follows:

$$RF_{CR} = \left[\frac{(t_{CO} / t_{original}) - (1 - n_{original})}{(t_{CR} / t_{original}) - (1 - n_{original})}\right]^3$$

$$= \left[\frac{(7.14 / 8.55) - (1 - 0.850)}{(6.30 / 8.55) - (1 - 0.850)}\right]^3$$

$$= \left[\frac{0.835 - 0.150}{0.737 - 0.150}\right]^3$$

$$= \left[\frac{0.685}{0.587}\right]^3$$

$$RF_{CR} = 1.59$$

Note 7: Other calculation methods to arrive at the above numeric value of creep reduction factor may be considered if agreed upon by the various parties involved.

8. Reduction Factors for Core Clogging

There are two general types of core clogging that might occur over a long time period. They are chemical clogging and biological clogging. Both are site-specific and both are essentially impractical to simulate in the laboratory.

8.1 Chemical clogging within the drainage core space can occur with precipitates deposited from high alkalinity soils, typically calcium and magnesium. Other precipitates can also be envisioned such as fines from turbid liquids although this is less likely since the turbid liquid must typically pass through a geotextile filter. It is obviously a site-specific situation.

8.2 Biological clogging within the drainage core space can occur by the growth of biological organisms or by roots growing through the overlying soil and extending downward, through the geotextile filter, and into the drainage core. It is a site-specific situation and depends on the local, or anticipated, vegetation, cover soil, hydrology, etc.

8.3 Default tables for the above two potential clogging mechanisms (chemical and biological) are very subjective and by necessity broad in their upper and lower limits. The following table is offered as a guide.

Range of Clogging Reduction Factors (modified from Koerner, 1998)

Application	Chemical Clogging (RF_{CC})	Biological Clogging (RF_{BC})
Sport fields	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.0 to 1.2	1.1 to 1.3
Roof and plaza decks	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock and soil slopes	1.1 to 1.5	1.0 to 1.2
Drainage blankets	1.0 to 1.2	1.0 to 1.2
Landfill caps	1.0 to 1.2	1.2 to 3.5
Landfill leak detection	1.1 to 1.5	1.1 to 1.3
Landfill leachate collection	1.5 to 2.0	1.1 to 1.3

9. Polymer Degradation

9.1 Degradation of the materials from which the drainage geocomposite are made, with respect to the site-specific liquid being transmitted, is a polymer issue. Most geocomposite drainage cores are made from polyethylene, polypropylene, polyamide or polystyrene. Most geotextile filter/separators covering the drainage cores are made from polypropylene, polyester or polyethylene.

Note 8: It is completely inappropriate to strip the factory bonded geotextile off of the drainage core and then test one or the other component. The properties of both the geotextile and drainage core will be altered in the lamination process from their original values.

9.2 If polymer degradation testing is recommended, the drainage core and the geotextile should be tested separately in their as-received condition before lamination and bonding.

9.3 The incubation of the drainage cores and/or geotextile coupons is to be done according to the ASTM D5322 immersion procedure.

9.4 The testing of the incubated drainage cores is to be done according to ASTM D6388 which stipulates various test methods for evaluation of incubated geonets.

Note 9: For drainage cores other than geonets, e.g., columnar, cuspated, meshes, etc., it may be necessary to conduct additional tests than appear in ASTM D6388. These tests, and their procedures, should be discussed and agreed upon by the project designer, testing organization, and manufacturer.

9.5 The testing of the incubated geotextiles is to be done according to ASTM D6389 which stipulates various test methods for evaluation of incubated geotextiles.

Note 10: The information obtained in testing the drainage core (Section 9.4) and the geotextile (Section 9.5) result in a "go-no go" situation and not in a reduction factor, per se. If an adverse chemical reaction is indicated, one must select a different type of geocomposite material (drainage core and/or geotextile).

10. Summary

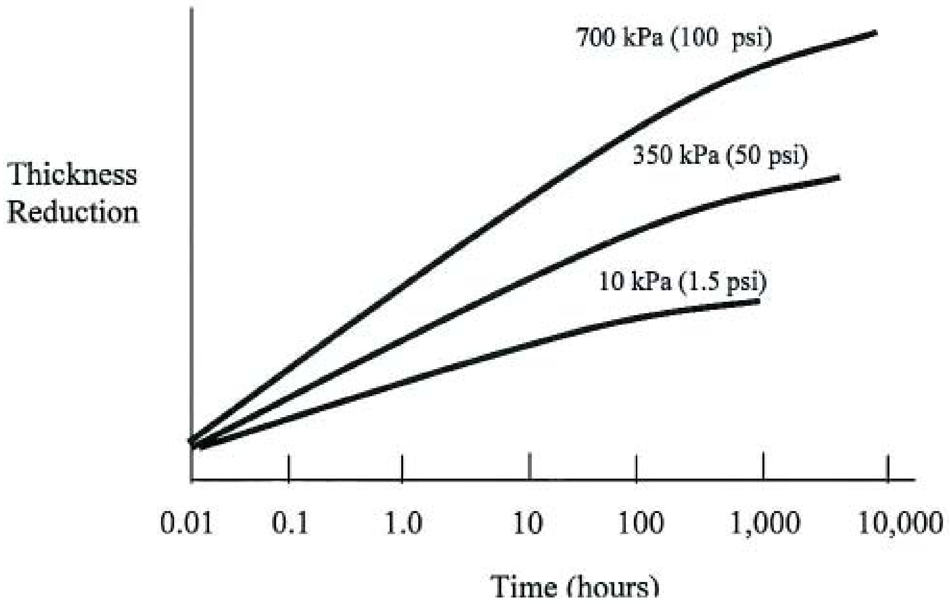
10.1 For a candidate drainage geocomposite, the 100-hour flow rate behavior under the site-specific set of variables, e.g., specimen orientation, stress level, hydraulic gradient, and permeating liquid is to be obtained per ASTM D4716 following procedures of Section 6.0.

10.2 A reduction factor for long term creep of the drainage core following Section 7.0 per GRI GS4 or ASTM D6364 (mod.) is then obtained. The result is usually a unique value for a given set of conditions.

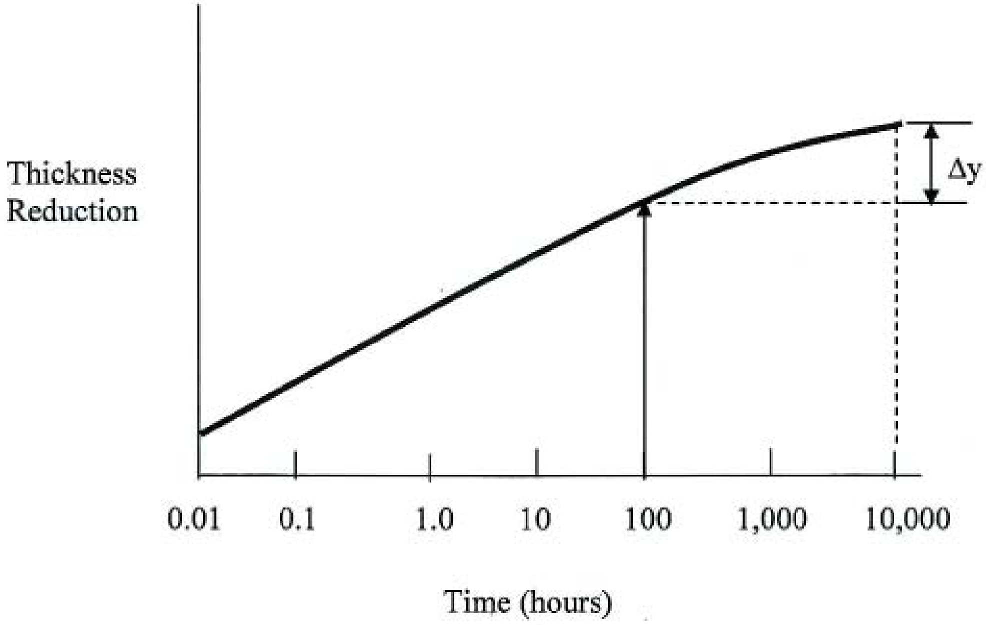
10.3 A reduction factor for chemical and/or biological clogging, as discussed in Section 8.0 can be included. It is very much a site-specific situation at the discretion of the parties involved.

10.4 Polymer degradation to aggressive liquids is covered in separate immersion and test protocols, e.g., ASTM D5322 (immersion), ASTM D6388 (geonets) and ASTM D6389 (geotextiles) as discussed in Section 9.0. The procedure does not result in a reduction factor, rather in a "go-no go" decision with the product under consideration.

10.5 Other possible flow rate reductions and/or concerns such as flow in overlap regions, effect of high or low temperatures, etc., are site-specific and cannot readily be generalized in a guide such as this.



(a) Hypothetical data from creep testing illustrating effect of normal load magnitude



(b) Interpretation of project specific normal load curve to obtain creep reduction factor

Figure 1 – Hypothetical example of creep test data and data interpretation to obtain creep reduction factor

Attachment 2-5-2

Manufacturer's Transmissivity Testing Data for GSE FabriNet UF Geocomposite

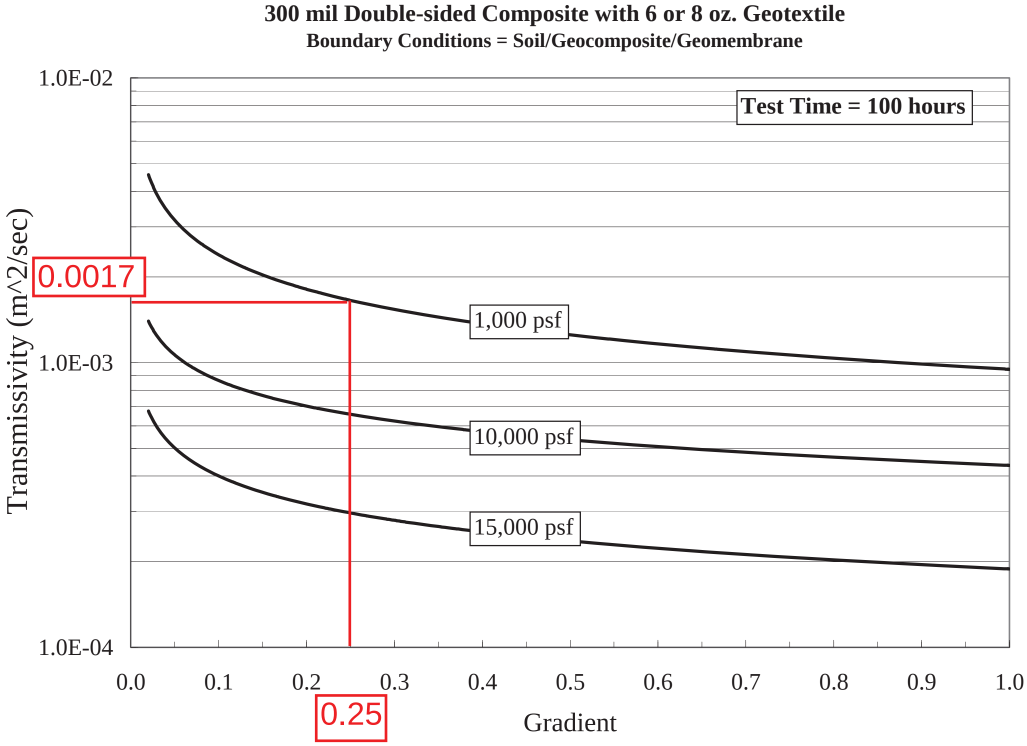


Figure A-9. Performance Transmissivity of a 300 mil GSE FabriNet UF geocomposite under Soil.

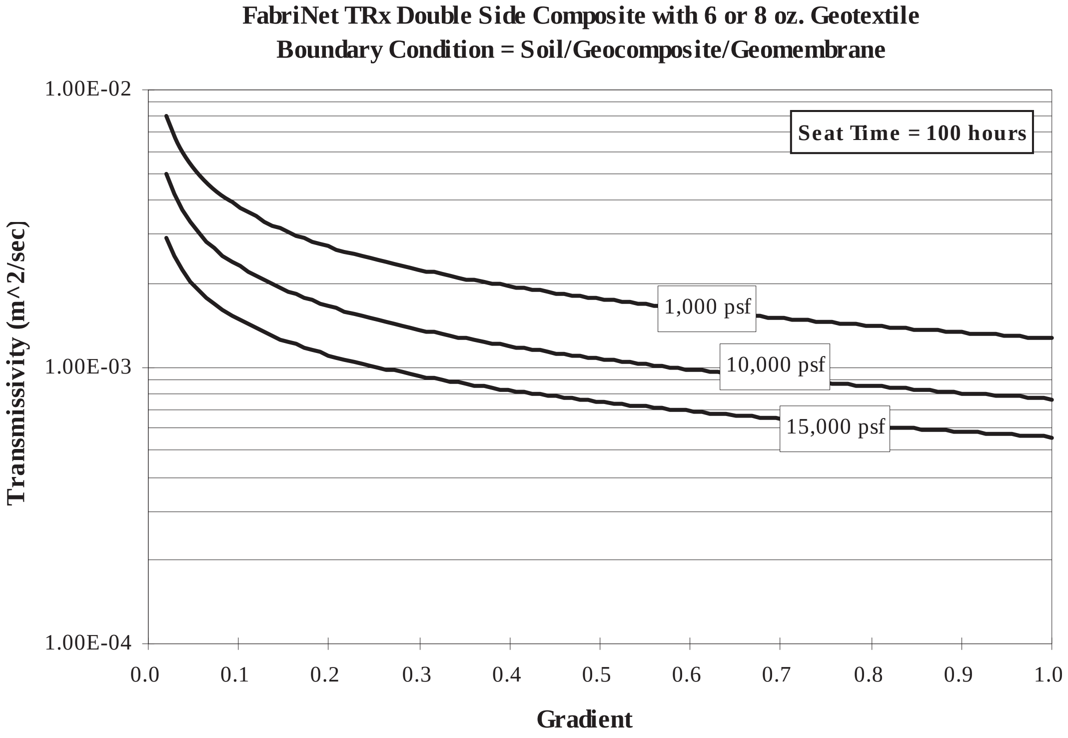


Figure A-10. Performance Transmissivity of GSE FabriNet TRx geocomposite under Soil.

Attachment 2-5-3

Characteristics of Soil

Soil Mechanics

T. William Lambe • Robert V. Whitman

Massachusetts Institute of Technology

1969

JOHN WILEY & SONS, New York • Chichester • Brisbane • Toronto • Singapore

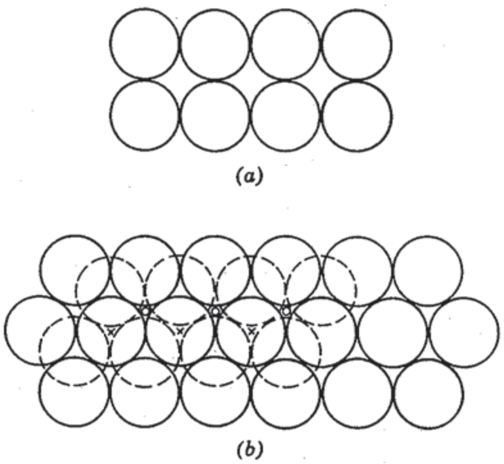


Fig. 3.2 Arrangements of uniform spheres. (*a*) Plan and elevation view: simple cubic packing. (*b*) Plan view: dense packing. Solid circles, first layer; dashed circles, second layer; ∘, location of sphere centers in third layer: face-centered cubic array; ×, location of sphere centers in third layer: close-packed hexagonal array. (From Deresiewicz, 1958.)

these simple packings can be computed from the geometry of the packings, and the results are given in Table 3.2.

This table also gives densities for some typical granular soils in both the "dense" and "loose" states. A variety of tests have been proposed to measure the maximum and minimum void ratios (Kolbuszewski, 1948). The test to determine the maximum density usually involves some form of vibration. The test to determine minimum density usually involves pouring oven-dried soil into a container. Unfortunately, the details of these tests have not been entirely standardized, and values of the maximum density and minimum density for a given granular soil depend on the procedure used to determine them. By using special measures, one can obtain densities greater than the so-called maximum density. Densities considerably less than the so-called minimum density can be obtained, especially with very fine sands and silts, by slowly sedimenting the soil into water or by fluffing the soil with just a little moisture present.

Table 3.2 Maximum and Minimum Densities for Granular Soils

	Void Ratio		Porosity (%)		Dry Unit Weight (pcf)	
Description	$e_{\max}$	$e_{\min}$	$n_{\max}$	$n_{\min}$	$\gamma_{d\min}$	$\gamma_{d\max}$
Uniform spheres	0.92	0.35	47.6	26.0	—	—
Standard Ottawa sand	0.80	0.50	44	33	92	110
Clean uniform sand	1.0	0.40	50	29	83	118
Uniform inorganic silt	1.1	0.40	52	29	80	118
Silty sand	0.90	0.30	47	23	87	127
Fine to coarse sand	0.95	0.20	49	17	85	138
Micaceous sand	1.2	0.40	55	29	76	120
Silty sand and gravel	0.85	0.14	46	12	89	146

100 pcf

B. K. Hough, *Basic Soils Engineering*. Copyright © 1957, The Ronald Press Company, New York.

The smaller the range of particle sizes present (i.e., the more nearly uniform the soil), the smaller the particles, and the more angular the particles, the smaller the minimum density (i.e., the greater the opportunity for building a loose arrangement of particles). The greater the range of particle sizes present, the greater the maximum density (i.e., the voids among the larger particles can be filled with smaller particles).

A useful way to characterize the density of a natural granular soil is with *relative density* D_r, defined as

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100\%$$

$$= \frac{\gamma_{d\max}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \times 100\% \qquad (3.1)$$

where

$e_{\min}$ = void ratio of soil in densest condition
$e_{\max}$ = void ratio of soil in loosest condition
e = in-place void ratio
$\gamma_{d\max}$ = dry unit weight of soil in densest condition
$\gamma_{d\min}$ = dry unit weight of soil in loosest condition
γ_d = in-place dry unit weight

Table 3.3 characterizes the density of granular soils on the basis of relative density.

Table 3.3 Density Description

Relative Density (%)	Descriptive Term
0–15	Very loose
15–35	Loose
35–65	Medium
65–85	Dense
85–100	Very dense

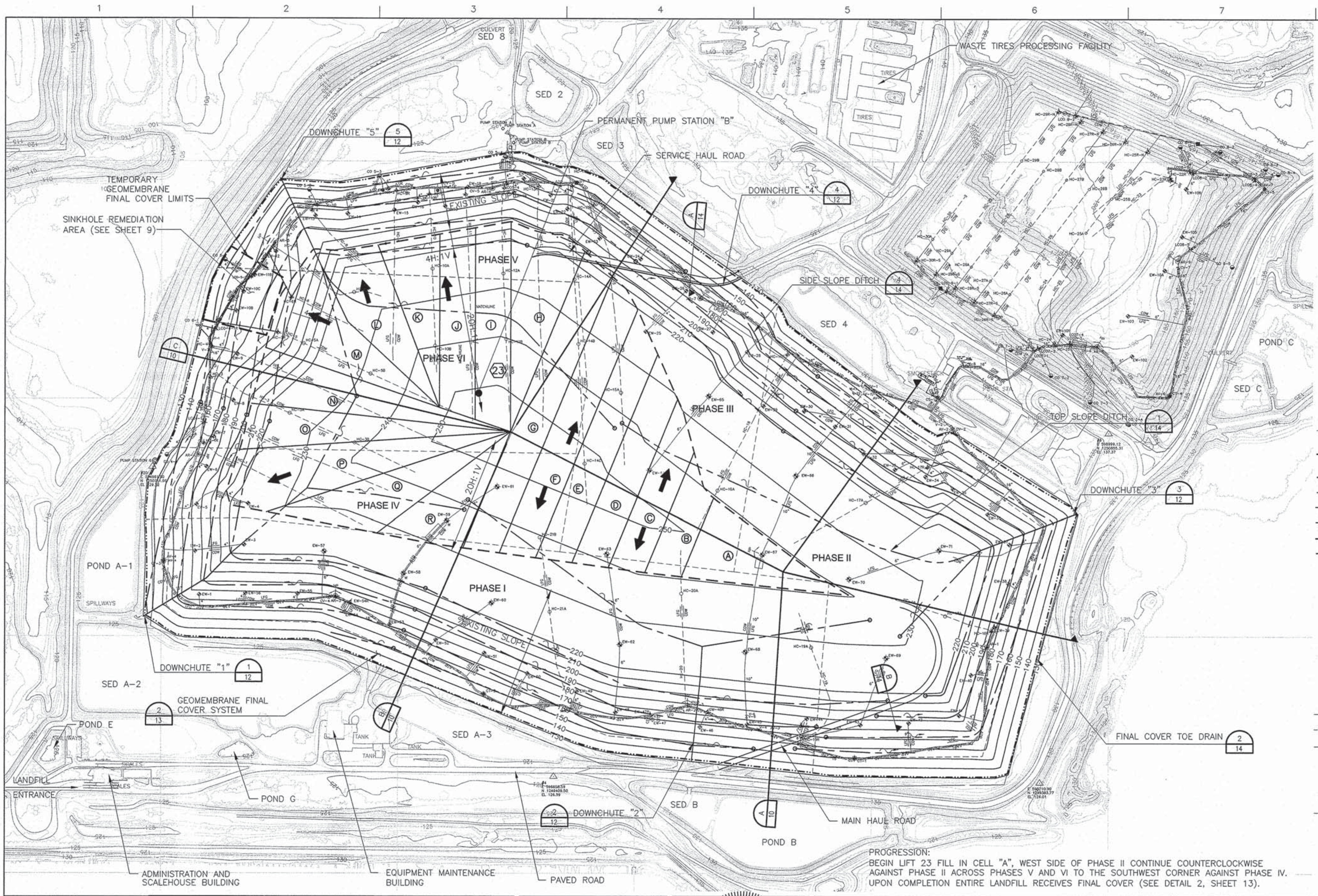
Values of water content for natural granular soils vary from less than 0.1% for air-dry sands to more than 40% for saturated, loose sand.

Typical Values of Phase Relationships for Cohesive Soils

The range of values of phase relationships for cohesive soils is much larger than for granular soils. Saturated sodium montmorillonite at low confining pressure can exist at a void ratio of more than 25; saturated clays

Attachment 2-6

Final Buildout Site Plan and Final Closure Detail



PROGRESSION:
BEGIN LIFT 23 FILL IN CELL "A", WEST SIDE OF PHASE II CONTINUE COUNTERCLOCKWISE AGAINST PHASE II ACROSS PHASES V AND VI TO THE SOUTHWEST CORNER AGAINST PHASE IV. UPON COMPLETION ENTIRE LANDFILL RECEIVES FINAL COVER (SEE DETAIL 2, SHEET 13).

LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
∼→	DRAINAGE FLOW DIRECTION
- - - - - - - -	APPROXIMATE PHASE FOOTPRINT
—··—··—	APPROXIMATE LANDFILL LIMITS
— — — —	APPROXIMATE LIFT LIMITS
—·—·—·—	EXISTING SWALE
——— - ———	SIDE SLOPE DITCH
——— - - - ———	TOP SLOPE DITCH
—140—	PROPOSED CONTOUR
————	PROPOSED DOWNCHUTE
EW-10	LFG EXTRACTION WELL
⊕EW-46	DOWNSLOPE LFG EXTRACTION WELL
EW-88	CAISSON LFG EXTRACTION WELL
■ UT-2	ON-GRADE CONDENSATE U-TRAP
CS-1	CONDENSATE SUMP WITH PUMP
CT-5	SELF-DRAINING CONDENSATE TRAP
◉ AR-24	HEADER ACCESS RISER
————	HEADER/LATERAL PIPE
------------	AIR SUPPLY LINE
—·—·—·—	CONDENSATE DRAIN LINE/LEACHATE DEWATERING LINE
CO 4-1	EXISTING LEACHATE COLLECTION SYSTEM CLEANOUT
— 4" FM —	EXISTING LEACHATE FORCE MAIN
CDW →	DIRECTION OF CONDENSATE /DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)

Hillsborough County
Florida

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES V AND VI - LIFT 23 (FINAL LIFT)

0 1" 2"

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DWG No.	8

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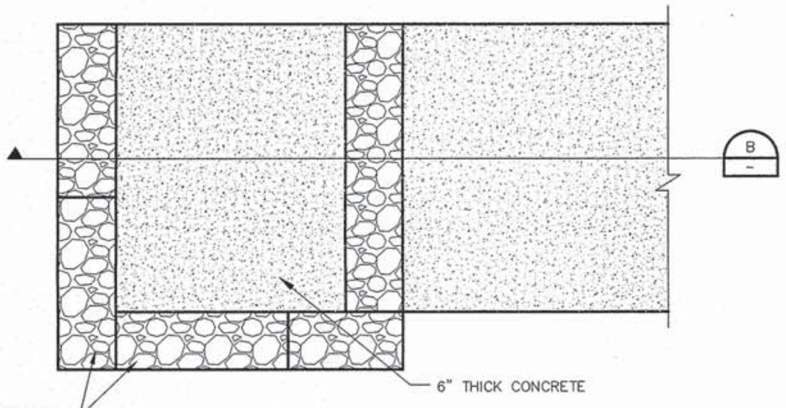
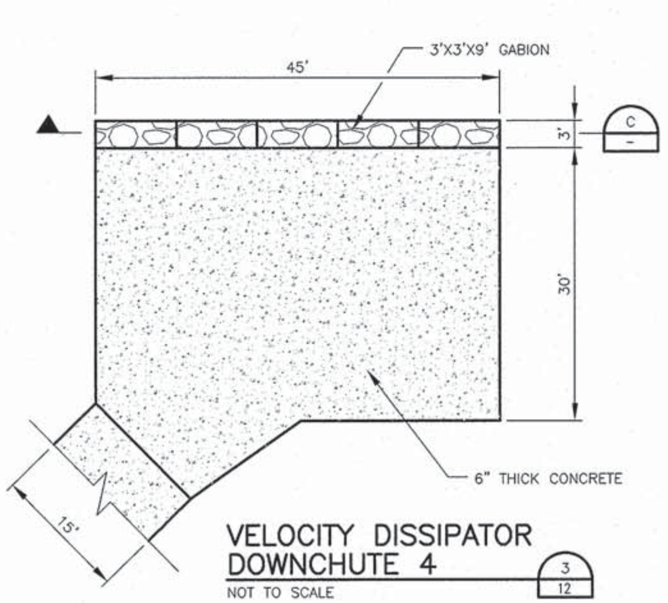


VELOCITY DISSIPATOR DOWNCHUTES 1, 3 & 5

NOT TO SCALE

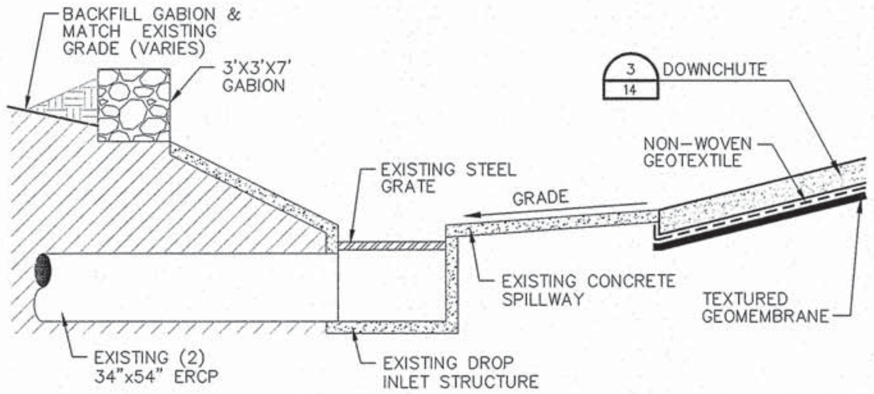
1 / 12



VELOCITY DISSIPATOR DOWNCHUTE 4

NOT TO SCALE

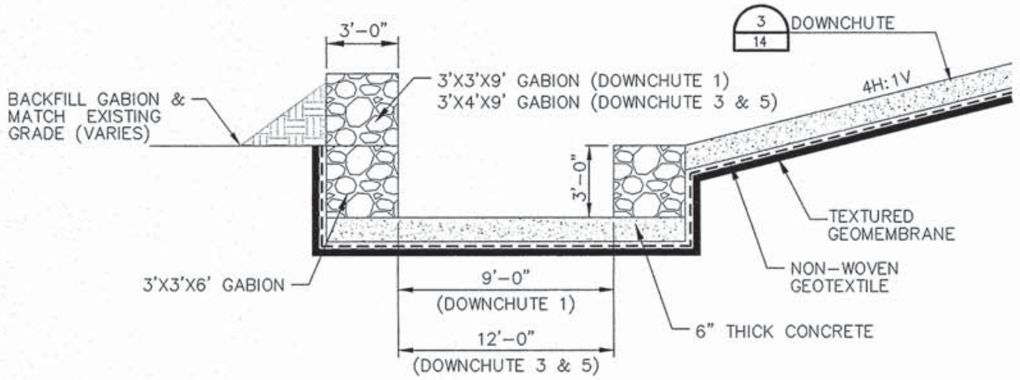
3 / 12



VELOCITY DISSIPATOR SECTION

NOT TO SCALE

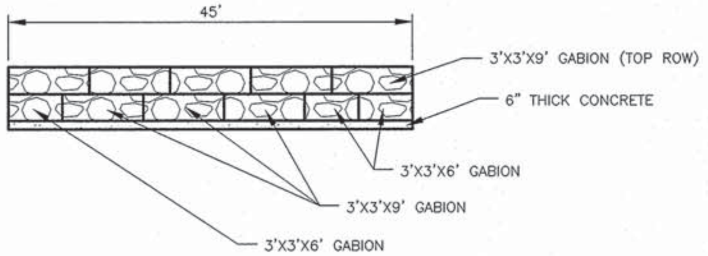
A / 12



SECTION

NOT TO SCALE

B / –



SECTION

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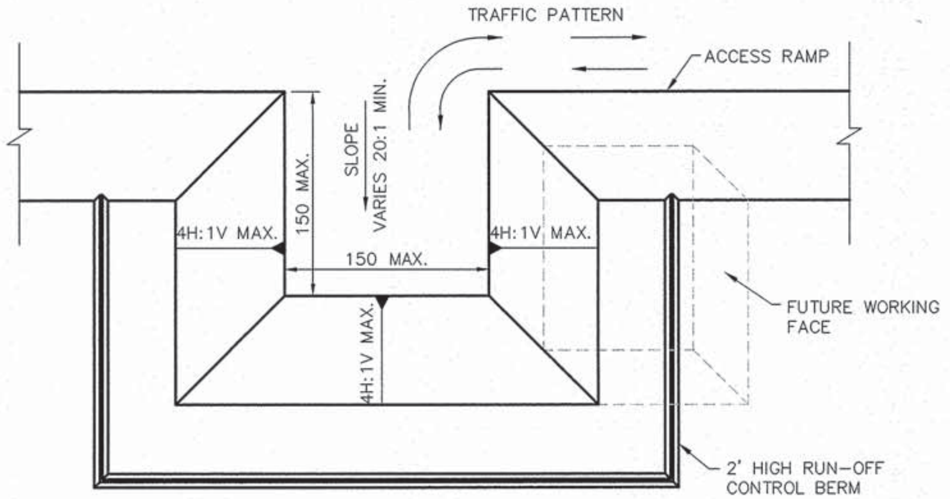
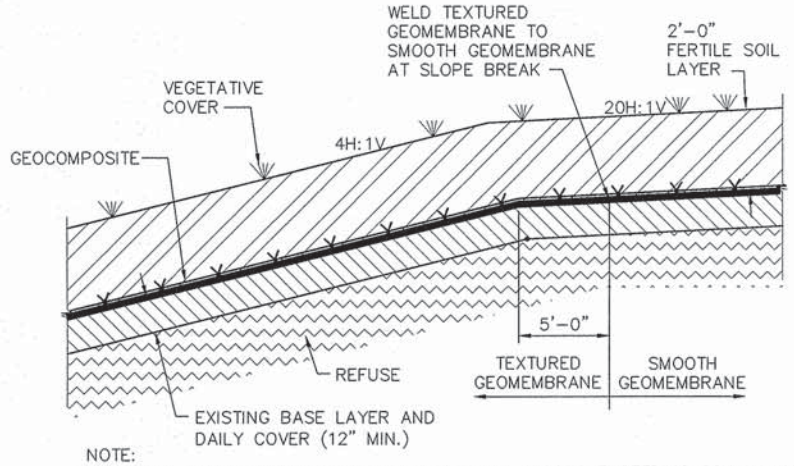
C / –



TYPICAL WORKING FACE CELL PLAN

NOT TO SCALE

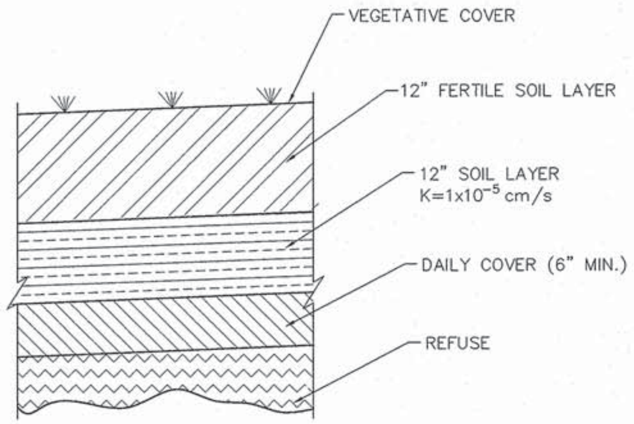


NOTE:
TEXTURED GEOMEMBRANE TO BE INSTALLED ON SLOPES EXCEEDING 20H:1V AND SMOOTH GEOMEMBRANE TO BE INSTALLED ON SLOPES LESS THAN OR EQUAL TO 20H:1V

TYPICAL GEOMEMBRANE FINAL COVER SYSTEM DETAIL

NOT TO SCALE

2 / 8 / 14



TYPICAL TEMPORARY FINAL COVER DETAIL

NOT TO SCALE

4 / 4 / 5

Hillsborough County
Florida

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096–193806–001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 2

0 1" 2"

FILENAME	00C–13.DWG
SCALE	AS SHOWN

DWG No.
13

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ATTACHMENT 3-A

EXAMPLE SETTLEMENT CALCULATION FOR TH-48

Given:

1. 1983 top of clay, bottom of clay, and initial thickness are based on field observations from 1983 investigation

1983 Top of Clay (NGVD)	1983 Bottom of Clay (NGVD)	Initial Clay Thickness (ft)
123.0	111.0	12.0

2. Refer to 2017 topographic map at TH-48 for surface elevation (See Attachment 3-B)

2017 Ground Surface (NGDV)
190.5

Assumptions:

3. Estimated cover soil to be 1.5 feet in thickness. Waste depth calculated from topographic map elevation minus 1983 clay elevation, cover thickness, and drainage layer thickness

Intermediate Cover	
Thickness (ft)	Density (lb/ft^3)
1.5	120

Waste	
Thickness (ft)	Density (lb/ft^3)
63.0	74

Drainage	
Thickness (ft)	Density (lb/ft^3)
3.0	120

Calculation:

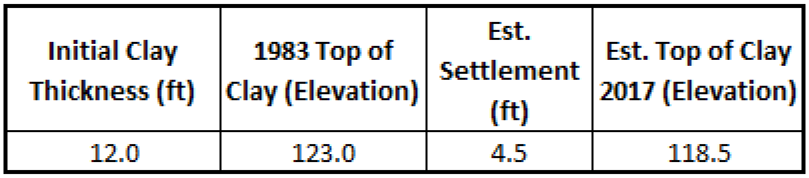
4. Stress imparted to the clay layer in tons/ft^2 calculated by multiplying depth by density of each layer and adding each component

(1.5 ft*120 lb/ft^3) + (63 ft*74 lb/ft^3) + (3 ft*120 lb/ft^3) =

Stress	
(lb/ft^2)	(tons/ft^2)
5,202.0	2.6

<u>Estimated Settlement:</u>

5. Use figure titled “Southeast County Landfill Variation of Consolidation Settlement with Load Intensity for Waste Phosphatic Clay Layers of Various Thickness" (See Attachment 3-C) to determine consolidation settlement in feet. This figure was developed by Ardaman & Associates, Inc. specifically for the phosphatic clay at the Southeast County Landfill based on empirical data derived from soil samples. Thickness of the clay layer was based on 1983 investigation findings (See step 1)



Initial Clay Thickness (ft)	1983 Top of Clay (Elevation)	Est. Settlement (ft)	Est. Top of Clay 2017 (Elevation)
12.0	123.0	4.5	118.5

Therefore, the estimated top of clay elevation at TH-48 equals 118.5. This is in agreement with boring elevation observations made in 2016 and 2017 at nearby SB-02 and SB-17D (117.9 and 119.6, respectively).

ATTACHMENT 3-B

2017 TOPOGRAPHIC MAP

LEGEND:

⊕ SB-11D SCS BORING LOCATIONS 2016–2017

● AB-37 HISTORICAL BORINGS COMPLETED BY OTHERS

2017 TOPOGRAPHIC CONTOUR (5–FOOT INTERVALS)

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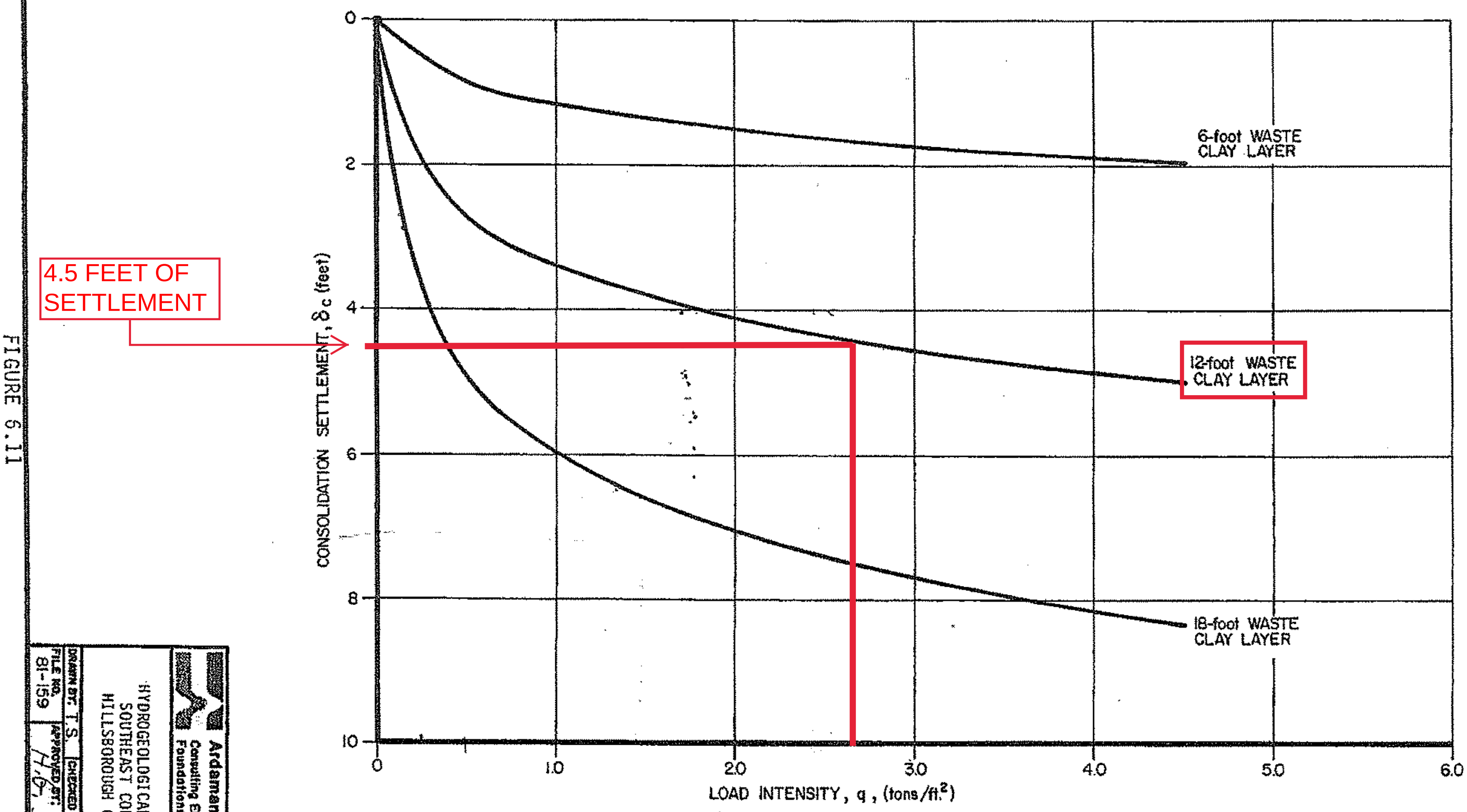
SCS ENGINEERS

2017 TOPOGRAPHIC MAP WITH BORING LOCATIONS
SOUTHEAST COUNTY LANDFILL
JUNE 2017

ATTACHMENT 3-C

SOUTHEAST COUNTY LANDFILL VARIATION OF CONSOLIDATION SETTLEMENT WITH LOAD INTENSITY FOR WASTE PHOSPHATIC CLAY LAYERS OF VARIOUS THICKNESS

FIGURE 6.11



4.5 FEET OF SETTLEMENT

VARIATION OF CONSOLIDATION SETTLEMENT WITH LOAD INTENSITY FOR WASTE PHOSPHATIC CLAY LAYERS OF VARIOUS THICKNESS

Ardaman & Associates, Inc.
Consulting Engineers in Soil Mechanics,
Foundations, and Material Testing

HYDROGEOLOGICAL INVESTIGATION
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: T.S.
CHECKED BY: D.J.C.
DATE: 1/20/83
FILE NO. 81-159
APPROVED BY:

K&E 19 1153 12-80 MC1983a

ATTACHMENT 4-A

ARDAMAN AND ASSOCIATES, INC. RESPONSE TO FIRST REQUEST FOR ADDITIONAL INFORMATION BY FDEP, APPLICATION FOR MINOR MODIFICATIONS, SOUTHEAST COUNTY LANDFILL, HILLSBOROUGH COUNTY, FLORIDA

Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

June 13, 2017
File Number 17-13-0061

SCS Engineers
4041 Park Oaks Blvd
Tampa, FL 33610

Attention: Mr. Robert B. Curtis, P.E.
Project Manager

Subject: Response to Comment No. 4 in First Request for Additional Information by Florida Department of Environmental Protection, Application for Minor Modifications, Southeast County Landfill, Hillsborough County, Florida

Gentlemen/Ladies:

As requested and authorized by SCS Engineers (SCS), Ardaman & Associates, Inc., (Ardaman) has reviewed Comment No. 4 by the Florida Department of Environmental Protection (FDEP) that relates to the application for minor modifications for the Southeast County Landfill, in Hillsborough County, Florida. The FDEP comment was documented in a letter dated May 8, 2017, and is repeated below in underlined font followed by our response. In preparing the response to the FDEP comment, Ardaman has reviewed the information provided by SCS as well as some historical data in our project files. However, no field exploration or data collection effort was performed by Ardaman to address this particular FDEP comment. We also made no effort to review and address other FDEP comments, which we understand will be handled by SCS.

4. Please provide time rate of settlement calculations that reflect both the compaction of the fill material and the continuous adjustment of the surcharge load. We are interested in time to both 50% and 90% settlement and, also, developing cross sections that shows the bottom expected elevation of two LCS lines at 5 year intervals. These cross sections should be consistent with your time rate of settlement calculations. Please use the most critical LCS paths for your cross drawings. Also, Chapter 3 of Qian and Koerner ("Geotechnical Aspects of Landfill Design and Construction") provides a discussion of compacted clay liner design and performance. Selected portions of this chapter might be relevant to the current application.

The magnitude of settlement of a clay layer under load is a function of the magnitude of the applied load, the *in situ* stress state of the clay layer, the thickness of the clay layer, and the compressibility of the clay material. The rate of consolidation or settlement of a clay layer under load is a function of the permeability (i.e., hydraulic conductivity) and compressibility of the clay material, and the maximum drainage distance for dissipation of excess pore water pressure induced by the applied load.

The time required to achieve 50 and 90 percent consolidation for varying thicknesses of waste phosphatic clay in the Phase I through VI areas of the Southeast County Landfill, based on a measured coefficient of consolidation[1] of 1.5×10^{-4} cm^2/sec and two-way drainage with a maximum drainage distance equals to half the waste phosphatic clay thickness, is presented in Table 1. A 7-year waiting period between placements of successive waste lifts was originally recommended in 1983 for landfill design considering a coefficient of consolidation of 1.5×10^{-4} cm^2/sec, a waste phosphatic clay thickness of 12 feet, and 95 percent consolidation. Based on the latest known information, the waste phosphatic clay thickness beneath the entire Phase I through VI areas had settled to 8 feet or less. At these thicknesses, the time to achieve 95 percent consolidation under a new landfill load increment or waste lift should be less than 4 years.

Prior to construction of the landfill, the original top surface of the waste phosphatic clay within the former settling area was relatively flat, with typical elevations ranging from +122 to +124 feet (NGVD). The original waste phosphatic clay thickness ranged from 4 feet along the northern, eastern, and southern boundaries to approximately 18 feet within the Phase VI area, where the leachate collection sump is located. The western boundary had a waste phosphatic clay thickness that varied between 4 and 14 feet. As shown in Figure 1, the current top elevations of the waste phosphatic clay range from approximately +122 feet (NGVD) along the northern, eastern, and southern boundaries to approximately +113 feet (NGVD) around the sump area, which corresponds to a waste phosphatic clay settlement of less than 2 feet along the perimeter and approximately 10 feet near the sump location. Accordingly, the waste phosphatic clay is estimated to have a current thickness of approximately 8 feet near the sump location. The elevation contours shown in Figure 1 were developed by SCS based on a combination of observations from recent field explorations and predictions of waste phosphatic clay settlements using the settlement curves developed by Ardaman in 1983[2].

Ardaman has evaluated whether the leachate currently drains to and will continue to drain to the sump that was installed beneath the waste materials and above the waste phosphatic clay in the Phase VI area, at a location where the waste phosphatic clay had a maximum initial thickness of 18 feet. As requested in the FDEP comment, Ardaman has selected two landfill cross sections (designated Sections A and B) that closely align with two leachate collection system (LCS) lines for evaluation of the leachate drainage grade (i.e., the grade of the top surface of the waste phosphatic clay) under current and future landfill loading conditions. The locations of the two selected landfill cross sections are shown in Figure 1. As shown, Section A is oriented approximately in the north-south direction and lies on the south side of the sump, whereas Section B is oriented approximately in the east-west direction and lies on the east side of the sump. Because these two landfill cross sections have the longest distances in the north-south and east-west directions for approximately the same elevation change from the landfill perimeter to the sump location, the leachate drainage slopes are flatter and, therefore, represent the worst case conditions for leachate drainage along the north-south and east-west directions.

Based on recent information provided by SCS and historical data in our project files, the current, intermediate (i.e., showing waste lifts between the current and buildout conditions), and buildout profiles of the landfill, the original top and bottom elevations of the waste

---

[1] Coefficient of consolidation is a parameter that can be derived from the permeability and compressibility of the material.

[2] See Ardaman report titled “Hydrogeological Investigation, Southeast County Landfill, Hillsborough County, Florida,” dated February 22, 1983.

phosphatic clay, and the current top elevations of the waste phosphatic clay along Sections A and B are displayed in Figures 2 and 3, respectively.

The filling schedule for the Phase I through VI areas since March 2003, as provided by SCS, is presented in Table 2. As shown, the most recent landfilling operation was conducted in Lift No. 13 within the Phase I area, after a waiting period of 7 years from the previous waste lift, and was completed in April 2017, when the landfilling operation was moved to the Capacity Expansion Area. The upcoming landfilling operation will be conducted in Lift No. 16A within the Phase IV area from June 2017 through June 2022, after a waiting period of 15 years since placement of the previous waste lift in this area. The landfilling schedule indicates that, except for the last waste lift in Phases V and VI, a waiting period of at least 5.5 years will be provided for placements of successive waste lifts until Phases I through VI reach the buildout condition in early 2043. The last lift in Phases V and VI will be placed after a waiting period of 3.8 years. Accordingly, except for the waste phosphatic clay that lies directly beneath the active landfill phase, the landfilling schedule should allow complete consolidation of the waste phosphatic clay before placement of a new waste lift. We understand that waste materials have been placed in lifts with a typical thickness of approximately 20 feet.

Assuming that the properties of the waste phosphatic clay material are relatively uniform, the magnitude of any future settlement will be directly proportional to the additional landfill load and the waste phosphatic clay thicknesses where and when the additional landfill load is applied. As indicated previously, the rate of settlement will depend on the coefficient of consolidation of the waste phosphatic clay and the maximum distance for dissipation of any excess pore water pressure within the waste phosphatic clay. Landfill load is a function of waste height and density. Based on landfill operation data provided by SCS, a waste density of 74 lb/ft^3 is considered to be representative of the refuse and ash mixture in the landfill.

Although the FDEP comment asked for settlement profiles at 5-year intervals, it is more practical and meaningful to compute the settlement profiles and evaluate the leachate drainage grade after the waste phosphatic clay has completely consolidated and fully settled under the weight of each future waste lift. It should be noted that, unlike a laboratory consolidation test in which static loads are applied incrementally and the 50 and 90 percent consolidation levels are commonly used in the deformation versus log-time and square-root-of-time plots to estimate the end of consolidation for each load increment, the applied landfill load in the Phase I through VI areas is not static and the thickness of the waste phosphatic clay varies throughout the entire Phase I through VI areas. Therefore, at any given time after a new landfill load increment has been applied, the percent consolidation will vary from locations to locations, with the locations having a thinner waste phosphatic clay reaching complete consolidation sooner. A landfill phase is considered to have achieved full consolidation only after the locations with the thickest waste phosphatic clay have reached 95 percent consolidation. It should further be noted that the original landfilling schedule was established considering that consolidation of the waste phosphatic clay would begin at the end of placement of each waste lift (i.e., any consolidation that occurs during placement of a waste lift was ignored, which is conservative), and would essentially be completed after a waiting period of 7 years. The time required to achieve 50 and 90 percent consolidation after placement of a waste lift for varying thicknesses of waste phosphatic clay was provided in Table 1.

To develop the settlement profiles at the completion of consolidation for each future waste lift, Ardaman has selected three locations each along Sections A and B for settlement computations. Based on a waste density of 74 lb/ft^3 and the heights of the waste lifts, the computed settlements at the selected locations are presented in Table 3 and the settlement profiles of the waste phosphatic clay are displayed in Figure 4. The computed settlements were based on the consolidation settlement curves developed by Ardaman in 1983, including an adjustment factor of 1.25 to take into consideration findings from a 2007 study that showed observed settlements to be, on average, 25 percent higher than the predicted settlements from the 1983 study at eight locations in the Phase I, II, III, IV, and VI areas. In our calculations, we assumed that settlement of waste phosphatic clay along the toe of the landfill slope is minimal and that settlement of the natural foundation soils beneath the waste phosphatic clay is negligible in comparison to compression of the waste phosphatic clay. As shown in Figure 4, the future waste phosphatic clay surface at some locations on the cross section was higher than the current waste phosphatic clay surface shown in Figure 1. This apparent inconsistency could be attributed to insufficient data or local variations of the waste phosphatic clay.

Based on the elevation contours shown in Figure 1, the average slopes of the waste phosphatic clay surface (i.e., the leachate drainage grade) from below the landfill crest to the sump along Sections A and B were computed to be approximately 0.7 and 0.2 percent, respectively. The slope of the waste phosphatic clay surface from below the landfill crest to the landfill toe was estimated to be 1 percent for both cross sections. The steeper leachate drainage slope beneath the landfill slope could be attributed to increasing waste phosphatic clay thickness and increasing landfill load towards the interior portion of the landfill away from the toe of slope. With a combination of the greater waste phosphatic clay thickness and greater landfill load near the sump area, the leachate drainage slope on top of the waste phosphatic clay is expected to continue to increase at decreasing rates as landfill height increases. On this basis, there should be no concern for reversal of the leachate drainage slopes along Sections A and B or any other landfill sections.

Although the waste phosphatic clay beneath the Phase I through VI areas was intended to serve as a leakage control liner to preclude migration of landfill leachate, it is not a compacted clay liner. A compacted clay liner does not compress significantly over time like a waste phosphatic clay will, and is required to have a thickness of only 1 to 3 feet in accordance with 62-701, F.A.C., depending on the design leachate head above the liner and the design hydraulic conductivity of the soil liner.

We trust that the above response adequately addresses the FDEP Comment No. 4. If you have any questions or need additional information, please let us know.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.
Certificate of Authorization No. 5950

Jeyisanker Mathiyaparanam, P.E.
Project Engineer

Francis K. Cheung, P.E.
Principal Engineer 06/13/17
Florida License No. 36382

FRANCIS K. CHEUNG
No 36382

Enclosures

Table 1

Time to Achieve 50 and 90 Percent Consolidation

Waste Phosphatic Clay Thickness* (feet)	Time to Achieve 50 Percent Consolidation** (years)	Time to Achieve 90 Percent Consolidation** (years)
4	0.2	0.7
6	0.3	1.5
8	0.6	2.7
10	1.0	4.1
12	1.4	6.0
14	1.9	8.1
16	2.5	11
18	3.1	13

---

* Prior to landfill construction, the waste phosphatic clay in the former settling area beneath Phases I through VI was documented to have a thickness of up to 18 feet. Under current condition, the waste phosphatic clay is expected to have a thickness of no more than 10 feet.

** Considering a coefficient of consolidation of 1.5×10^{-4} cm^2/sec and assuming two-sided drainage with the maximum drainage distance equals to half the waste phosphatic clay thickness.

Table 2

Landfilling Schedule

	Phase/Waste Lift	Month of Waste Placement		Years from Previous Filling
		Commencement	Completion	
Past Landfilling	Phase I - Lift 8	Mar 2003	Aug 2004	7.9
	Phase II - Lift 9	Aug 2004	Aug 2005	7.0
	Phase III - Lift 10	Aug 2005	Aug 2006	14.5
	Phase IV - Lift 11	Aug 2006	Jun 2007	12.0
	Phase V to VI - Lift 12	Jun 2007	Mar 2010	8.0
Active and Future Landfilling	Phase I - Lift 13	May 2010	Apr 2017	7.0
	Phase IV – Lift 16A	Jun 2017	Jun 2022	15.0
	Phase V to VI – Lift 17A	Jun 2022	Feb 2026	15.0
	Phase II - Lift 14	Feb 2026	May 2028	21.5
	Phase III - Lift 15	May 2028	Dec 2029	22.8
	Phase I - Lift 18	Dec 2029	Aug 2031	19.6
	Phase II - Lift 19	Aug 2031	Feb 2034	5.5
	Phase III - Lift 20	Feb 2034	Jun 2035	5.8
	Phase IV - Lift 21	Jun 2035	Mar 2036	17.8
	Phase V to VI - Lift 22	Mar 2036	Dec 2039	15.8
	Phase V to VI – Lift 23	Dec 2039	Jan 2043	3.8

Table 3

Waste Phosphatic Clay Settlements under Future Waste Lifts

	Sequence /Lift	Station (feet)	Original Clay Thickness (feet)	Waste Thickness (feet)	Estimated Applied Load on Top of Clay* (tons/ft^2)	Predicted Consolidation Settlement (feet)	Predicted Thickness Upon Completion of Consolidation (feet)	Adjusted Consolidation Settlement** (feet)	Adjusted Thickness Upon Completion of Consolidation (feet)
Section A	14/15	02+29	14.3	116	4.4	5.9	8.4	7.4	6.9
		06+00	13.0	96	3.7	5.3	7.7	6.6	6.4
		08+95	9.6	80	3.1	3.5	6.1	4.4	5.2
	16A	01+36	15.3	125	4.7	6.9	8.4	8.6	6.7
		06+00	13.0	96	3.7	5.3	7.7	6.6	6.4
		08+95	9.6	80	3.1	3.5	6.1	4.4	5.2
	Final Buildout	00+83	15.8	140	5.3	7.4	8.4	9.3	6.6
		04+79	13.9	119	4.5	6.2	7.7	7.8	6.1
		08+11	10.7	101	3.9	4.4	6.3	5.5	5.2
Section B	14/15	00+00	16.6	117	4.4	7.6	9.0	9.5	7.1
		12+00	12.3	111	4.2	5.1	7.2	6.4	5.9
		23+00	10.1	80	3.1	3.6	6.5	4.5	5.6
	16A	00+00	16.6	125	4.7	7.8	8.8	9.8	6.8
		12+00	12.3	111	4.2	5.1	7.2	6.4	5.9
		23+00	10.1	80	3.1	3.6	6.5	4.5	5.6
	Final Buildout	00+00	16.6	140	5.3	7.9	8.7	9.9	6.7
		12+00	12.3	128	4.9	5.2	7.1	6.5	5.8
		22+00	9.9	100	3.8	3.8	6.1	4.8	5.1

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* Including the weight of 2 feet of sand cover below the landfill and above the waste phosphatic clay.

** An adjustment factor of 1.25 was applied to take into consideration findings from a 2007 study that showed observed settlements to be, on average, 25 percent higher than the predicted settlements from the 1983 study at eight locations in the Phase I, II, III, IV, and VI areas.

0 200 400

SCALE: 1"=200'

TOPOGRAPHIC SURVEY PERFORMED BY
PICKETT AND ASSOCIATES, INC.

TOPOGRAPHIC SURVEY DATED
JANUARY 06, 2017

CONTOUR INTERVAL = 1' INCREMENT

LEGEND

—120— CURRENT TOP ELEVATIONS OF WASTE PHOSPHATIC CLAY

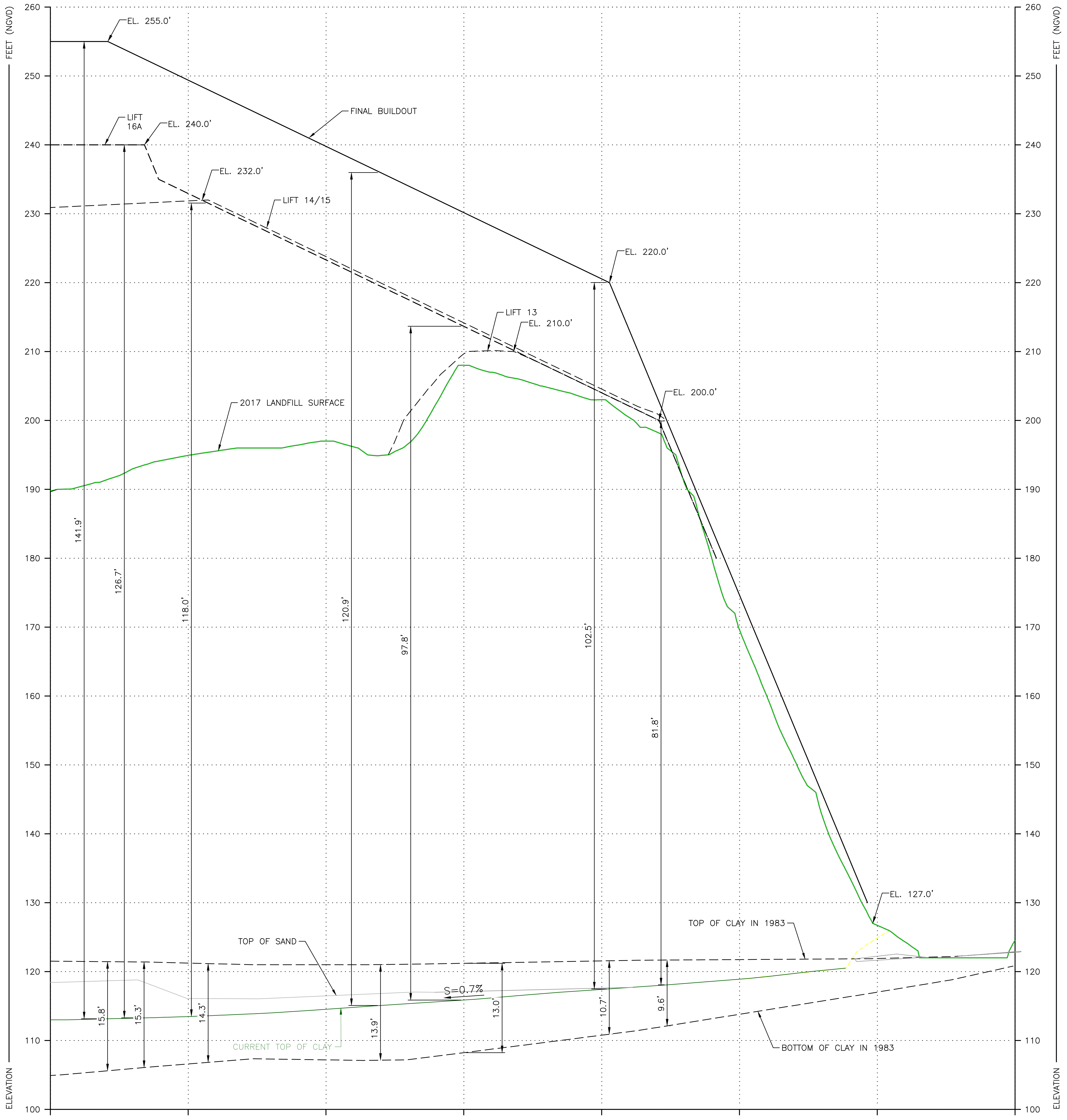
CURRENT TOP OF CLAY ELEVATIONS

Ardaman & Associates, Inc.
Geotechnical, Environmental and
Materials Consultants

SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	RDH	CHECKED BY:	JM	DATE:	06/06/17
FILE NO.	17-13-0061	APPROVED BY:	FKC	FIGURE:	1

R:\Corporate\17\17-13-0061_PROJECT DESCRIPTION_Current Sheet Set\1 Plan view.dwg 2017/06/13 10:24 AM, Hoffman, Ruben



A
-

CROSS SECTION

HORIZONTAL SCALE: 1" = 100'
VERTICAL SCALE: 1" = 10'

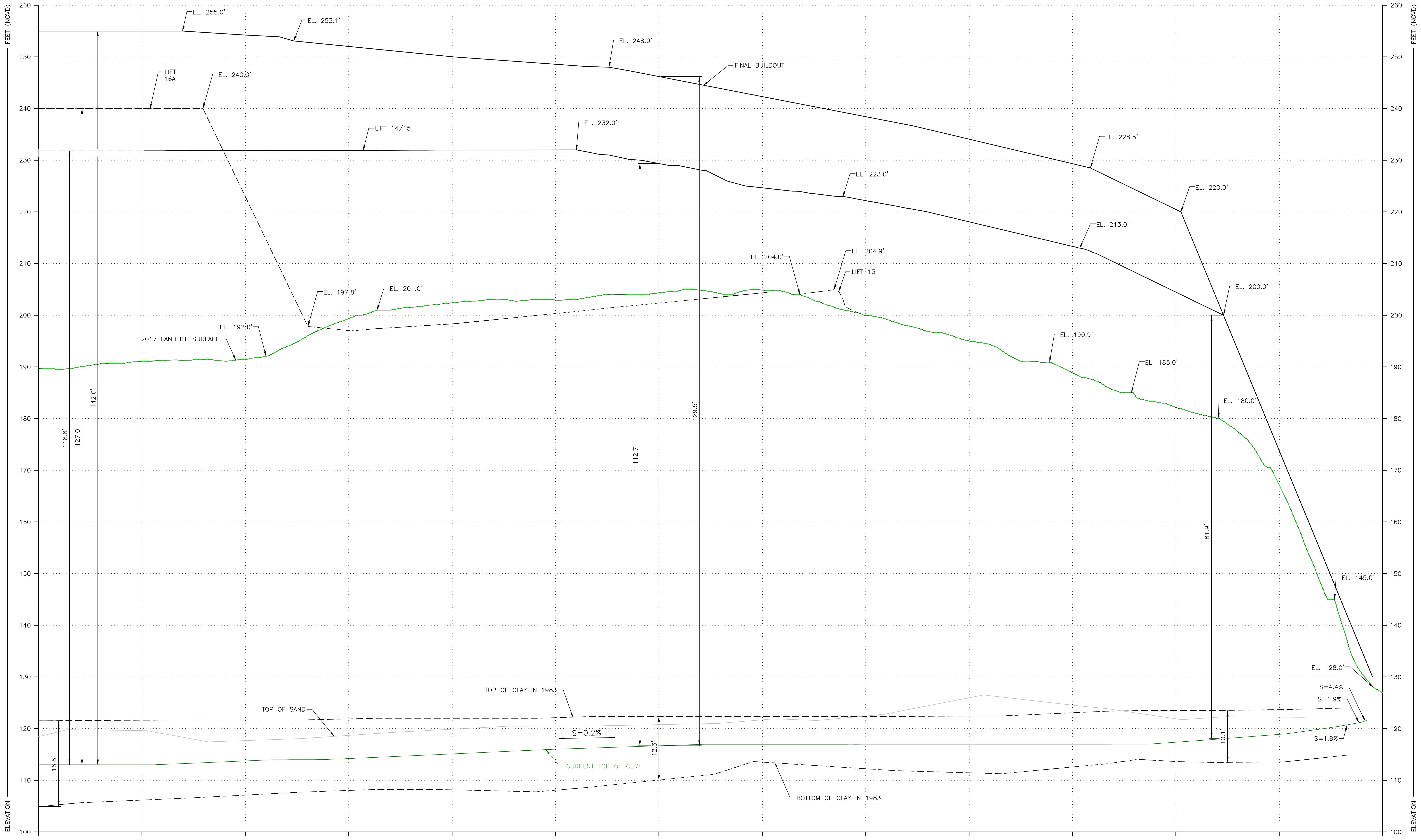
CROSS SECTION A

Ardaman & Associates, Inc.
Geotechnical, Environmental and
Materials Consultants

SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY:	RDH	CHECKED BY:	JM	DATE:	06/13/17
FILE NO. 17-13-0061	APPROVED BY: FKC			FIGURE:	2

R:\Corporate\17\17–13–0061_PROJECT DESCRIPTION_Current Sheet Set\2–3 Cross sections.dwg 2017/06/13 10:24 AM, Hoffman, Ruben



B / **-** **CROSS SECTION**

HORIZONTAL SCALE: 1" = 100'
VERTICAL SCALE: 1" = 10'

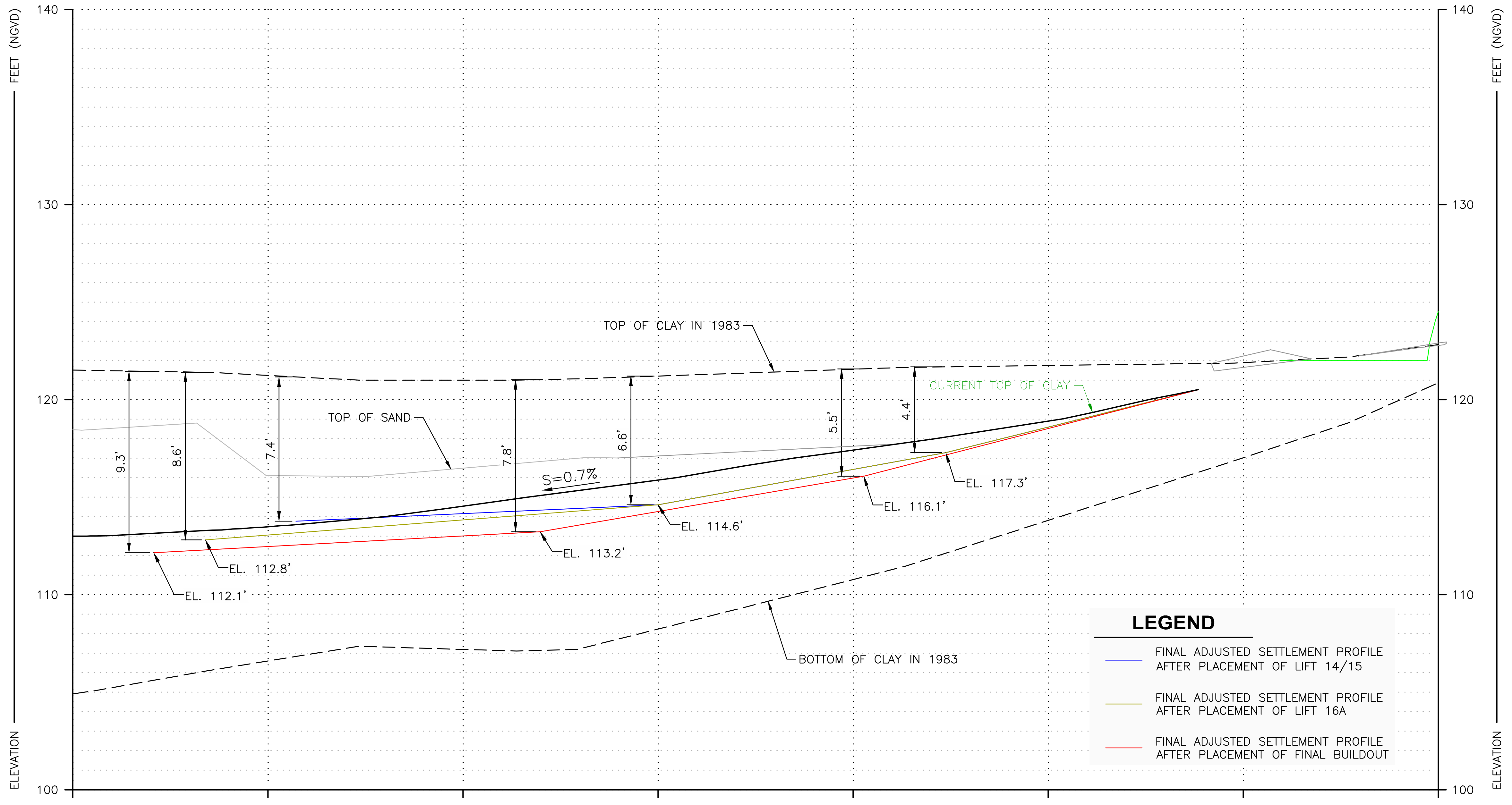
CROSS SECTION B

Ardaman & Associates, Inc.
Geotechnical, Environmental and
Materials Consultants

SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

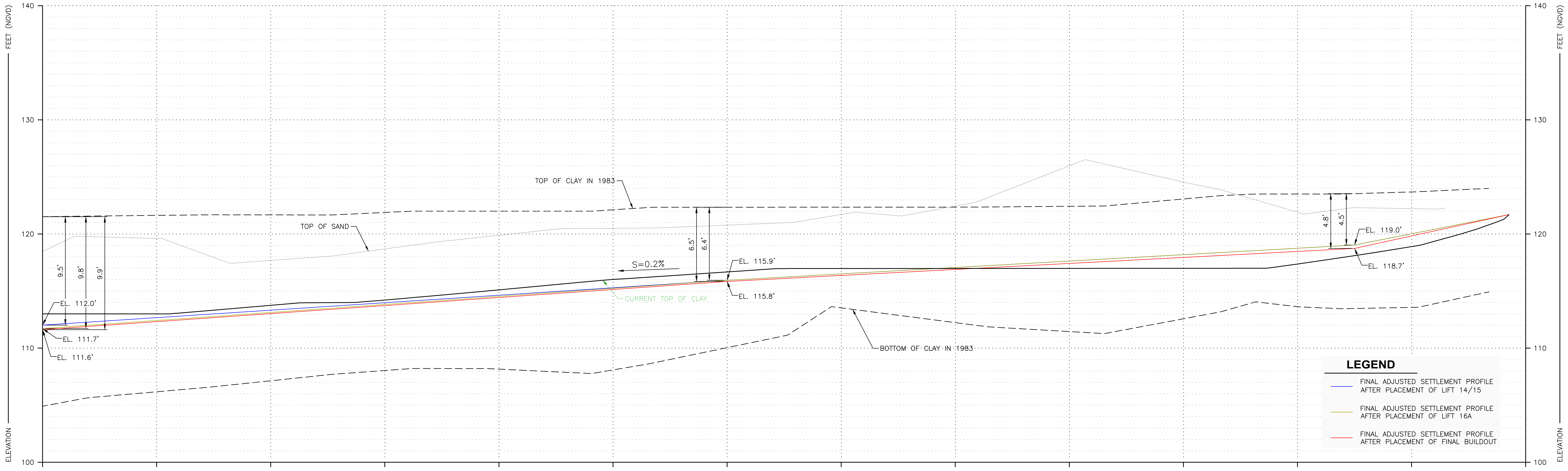
DRAWN BY: **RDH**	CHECKED BY: **JM**	DATE: **06/13/17**	
FILE NO. **17-13-0061**	APPROVED BY: **FKC**	FIGURE: **3**	

R:\Corporate\17\17-13-0061_PROJECT DESCRIPTION_Current Sheet Set\2-3 Cross sections.dwg 2017/06/13 10:24 AM, Hoffman, Ruben



A ADJUSTED SETTLEMENT PROFILE ALONG CROSS SECTION A

HORIZONTAL SCALE: 1" = 100'
VERTICAL SCALE: 1" = 5'



B ADJUSTED SETTLEMENT PROFILE ALONG CROSS SECTION B

HORIZONTAL SCALE: 1" = 100'
VERTICAL SCALE: 1" = 5'

PREDICTED SETTLEMENTS

Ardaman & Associates, Inc.
Geotechnical, Environmental and
Materials Consultants

SOUTHEAST LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DRAWN BY: **RDH** CHECKED BY: **JM** DATE: **06/13/17**
FILE NO. **17-13-0061** APPROVED BY: **FKC** FIGURE: **4**

R:\Corporate\17\17-13-0061_PROJECT DESCRIPTION_Current Sheet Set\4 Predicted Settlement Cross sections.dwg 2017/06/13 10:24 AM, Hoffman, Ruben

ATTACHMENT 5

COMPLETE UPDATED OPERATIONS PLAN

ATTACHMENT A

OPERATION PLAN
PHASES I-VI AND THE
CAPACITY EXPANSION AREA
(SECTIONS 7, 8, AND 9)
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

Prepared for:

HILLSBOROUGH COUNTY
PUBLIC UTILITIES DEPARTMENT
SOLID WASTE MANAGEMENT GROUP (SWMG)

332 N. Falkenburg Road
Tampa, FL 33619

Prepared by:

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5426 Bay Center Drive, Suite 400
Tampa, FL 33609

Certificate of Authorization #4213

Revised ~~September 2015~~
April 2017

TABLE OF CONTENTS

APPENDICES

PART K INTRODUCTION

The Southeast County Facility (Facility) includes the Southeast County Landfill (SCLF), which is permitted by the Florida Department of Environmental Protection (FDEP) as a Class I landfill for Phases I-VI and the Capacity Expansion Area. This Operation Plan includes Phases I-VI and Sections 7, 8, and 9 of the Capacity Expansion Area.

The Facility is the final depository for municipal solid waste (MSW) ash residues, non-processables, and bypass wastes from the Solid Waste Management System of Unincorporated Hillsborough County. The Facility also receives solid waste from the cities of Temple Terrace and Tampa, as well as MSW ash residues and bypass wastes from the Waste-to-Energy Incinerator Facilities of the City of Tampa and Hillsborough County. Hazardous waste will not be accepted at the Facility.

This operation plan was prepared in conjunction with an operation permit application; as such, the format follows the requirements of Part K of the Permit Application Form.

K.1 TRAINING

In accordance with Rule 62-701.320(15), Florida Administrative Code (FAC), key supervisory personnel at the Facility have received Landfill Operator Certification training. Operator training includes a 24-hour initial course and 16 hours of continuing education every three years. Spotter training includes an 8-hour initial course and four hours of continuing education every three years. Operator and Spotter training courses are offered by the University of Florida Center for Training, Research and Education for Environmental Occupations (TREEO) and through other FDEP-approved sources. Landfill personnel are encouraged to attend these courses after discussions with the Landfill Manager. The currently available TREEO training courses and schedule are listed in Appendix A. The listing is also available at www.treeo.ufl.edu. Documentation demonstrating that the facility operators and spotters have received the required continuing education is presented in Attachment D.15 of the Phases I-VI and Capacity Expansion Area (Sections 7, 8, and 9) Permit Renewal Application dated June 2013.

As required by Rule 62-701.500(1), FAC, a certified Landfill Operator will be on site when waste is received for disposal at the landfill, and a trained spotter will be on site during all times when waste is deposited at the landfill working face to detect any unauthorized wastes. In addition, the equipment operators have sufficient training and knowledge to move waste and soil and to develop the site in accordance with the design and operational standards described in the operation permit application.

K.2 LANDFILL OPERATION PLAN

K.2.a. SWMG Organization and Responsibilities

Hillsborough County (County) owns the Facility and is the applicant for the operation permit. A Landfill Contractor (Contractor), currently Waste Management, Inc. of Florida (WMIF), will operate and maintain the Facility in accordance with the permit conditions under the contract that exists between the County and the Contractor.

The following Hillsborough County Public Utilities Department, Solid Waste Management Group (SWMG) and Contractor personnel are currently responsible for the operations at this Facility:

- Larry E. Ruiz, Landfill Operations Manager (SWMG)
- Ernest Ely, District Landfill Manager (Contractor)

In addition, the following positions are maintained at the Facility: scale-house clerks (SWMG), waste monitors (SWMG), equipment operators (Contractor), spotters (Contractor), laborers (Contractor), security personnel (Contractor), and mechanic (Contractor). At least one trained operator familiar with the landfill operations will be on site at all times while the Facility is open in accordance with Rules 62-701.320(15) and 62-701.500(1), FAC.

K.2.b. Contingency Plan

The contingency plan for the Facility is based upon addressing two potential emergencies:

- Equipment failure.
- Large influx of material resulting from a natural disaster such as a hurricane, fire, or from a breakdown at local waste-to-energy facilities.

Sufficient backup equipment will be provided on site for equipment breakdowns and downtime for normal routine equipment maintenance. If primary and backup major equipment (i.e., landfill compactor or bulldozer) fail, one or both of the following contingency measures will be implemented:

- Use existing contracts with contractors and rental equipment dealers to furnish rental equipment on short notice (Appendix B).
- Establish arrangements with other County agencies to furnish equipment.

The Contractor will be responsible for providing equipment and a working force of adequate size and skill to maintain the landfill operation in compliance with all applicable federal, state, and local regulations. If sufficient local personnel are not available, the Contractor will relocate from other facilities sufficient personnel with the proper skills to maintain operations.

Given that a large volume of wastes requiring disposal from a natural disaster is non-putrescible, it can be stored on site temporarily (adjacent to the working face) and landfilled after the state of emergency has ended.

In the case of a large fire, bomb threat, or other unforeseen situation requiring specialized emergency response personnel, 911 will be called for the local Fire Department or Sheriff's Department. Waste handling will be suspended and the affected area will be evacuated, if necessary. The landfill will be temporarily closed until the responding Department determines that the landfill is safe for re-entry. If the Facility will remain closed for more than 48 hours, the incoming waste will be diverted to an alternate facility in an adjacent county.

In case of an accidental spill of oil, fuel, leachate, or chemicals, the spill will be minimized by controlling the source immediately (e.g., by closing the valve, turning-off switch, or taking any other necessary action). The affected area will be protected by diverting vehicular traffic. Building a berm, plugging a drain or ditch, or adding absorbent material will control runoff from the affected area. The affected area will be cleaned, and the effectiveness of the cleanup confirmed by sampling, as needed, depending on the nature of the spilled material. For spill countermeasures of secondary containment at the Leachate Treatment and Reclamation Facility (LTRF) and the effluent/leachate storage tank, refer to Section 11.0 of the Leachate Management Plan (LMP).

K.2.c. Waste Type Control

The automated accounting system, clerks at the scalehouse, and the site security fence help discourage unauthorized entry and uncontrolled disposal of unauthorized waste. A sign at the entrance states the general regulations including the types of prohibited solid waste.

A minimum of three random load inspections of solid waste per week will be conducted at the active landfill (See Part K.6 and Appendix C). As an additional control, the SWMG has one waste monitor and the Contractor has at least one trained spotter at the working face to visually inspect each load of waste as it is unloaded and deposited. If any unauthorized special waste (i.e., lead-acid batteries, used oil, yard trash, white goods, and whole tires) is found at the working face during the random inspection or as part of routine operations, the waste will be segregated and removed from the site for recycling or other processing in accordance with FDEP regulations. Items that may contain liquids or gases will be stored upright, undamaged, and in a container as appropriate. The maximum on-site storage will be as follows:

- 50 batteries in a secondary containment covered tray.
- 20 gallons of used oil placed upright in an undamaged container.
- 40 cubic yards (cy) yard trash in one 40-cy roll-off container.
- 75 white goods and lawnmowers placed upright (on the ground) until all liquids, chlorofluorocarbons (CFCs), and Freon are removed. After the metal recycling contractor removes all liquids, CFCs, and Freon, the white goods are marked with spray paint to indicate that they are ready to be placed in the scrap metal containers.
- Scrap metal in two 40-cy roll-off containers (including processed white goods).

These special wastes will be stored next to the working face and removed from the site within 30 days.

Whole tires will be stored and managed at the on-site Waste Tire Processing Facility (WTPF). Lead-acid batteries will be collected by the SWMG's contracted battery recycler. Scrap metal, including white goods and lawnmowers, will be collected and processed by the SWMG's metals recycling contractor. Propane tanks will be collected by the recycling contractor. Until the SWMG develops a beneficial use for landfill gas, yard trash will be rejected, required to be reloaded, and directed to be taken to the yard trash processing facility at the South County Transfer Station.

If unauthorized waste (i.e., hazardous, polychlorinated biphenyl’s (PCBs), untreated biomedical, or free liquid) is found at the working face, the waste will be isolated and the Landfill Manager will be immediately notified. The Landfill Manager is trained in the proper procedure to follow, including notifying the FDEP. Similarly, if suspect waste is found, the waste will be isolated and the Landfill Manager notified. The Landfill Manager will prepare a suspect waste report and ensure that the waste is properly managed (Appendix C). If hazardous wastes are found, the FDEP will be notified immediately and the waste will be isolated and restricted from access until it is removed from the landfill by a qualified hazardous waste contractor. Hazardous wastes will be removed from the Facility within 24 hours.

K.2.c.(1) Waste Profile Program

The Waste Profile Program, administered by the SWMG, establishes policies, procedures, and guidelines for managing waste to comply with federal, state, and local regulations for minimizing risks to the environment, public health, and employees posed by non-hazardous and unregulated waste. The Waste Profile Program includes an internal structured reporting format, guidelines, and procedures to assist customers to comply with waste disposal requirements. The SWMG does not accept unauthorized waste for disposal at the landfill. The following are the objectives of the waste profile program:

- Preclude the entry and disposal of hazardous waste into the Facility.
- Preclude leachate developing hazardous waste characteristics.
- Protect the landfill liner.
- Prevent objectionable odors from becoming a problem.
- Ensure that delivered materials can be handled safely.

K.2.c.(2) Motor Vehicles

Motor vehicles will not be accepted at the facility; however, mobile homes will be accepted for disposal in the landfill at the active working face if they cannot be recycled. Appliances (white goods) and waste tires from mobile homes must be removed before being accepted at the facility and processed as stated in Section K.2.c.

K.2.c.(3) Shredded Waste

The Facility will accept shredded tires. As provided by Chapter 62-711 FAC, the SWMG will use shredded tires for initial cover since shredded tires are an effective initial cover for controlling disease, vectors, odors, litter, and scavenging.

K.2.c.(4) Asbestos Waste

Asbestos waste will be accepted at the Facility. The entire footprint of Phases I-VI and the Capacity Expansion Area will be designated as an asbestos disposal area. Before landfilling, the material must be wetted and placed in a leak-tight wrapping. The bags will be placed in a prepared trench at the working face. Materials such as transite paneling and pipe insulation must be wrapped sufficiently to maintain their integrity during disposal. After placement, the bags will be immediately covered with 6 inches of asbestos-free material (i.e., soil or select waste without large or sharp objects that may damage the asbestos packaging). The location, quantity and source of asbestos containing material will be documented. Copies of the asbestos waste shipment records complying with 40 CFR 61-Subpart M will be maintained on site.

K.2.c.(5) Wastewater Treatment Biosolids

Biosolids (industrial and domestic sludge) from wastewater treatment systems are accepted for disposal in the landfill. Biosolids will be applied to the working face of the landfill and daily cover applied in accordance with Section K.2.g to control odors. Disposal operations of biosolids will not occur within 50 feet of exterior side slopes

Biosolids from the wastewater treatment facility (WWTF) will be required to pass the paint filter test which will be based on the percent solids of the biosolids produced by the WWTF.

A paint filter test will be initially performed on the biosolids to demonstrate the minimum percent solids content that will pass the paint filter test. Thereafter, the WWTF will be required to provide a report of the percent solids content of the biosolids delivered each day to the Facility. Biosolids from the WWTFs with percent solids content at or above the minimum solids content passing the paint filter test will be accepted at the Facility. In the event the percent solids content from a WWTF is below the minimum solids content, the WWTF must, before disposal at the SCLF, perform and provide documentation that the lower percent solids content passes the paint filter test.

In addition to landfilling, the County manages a solid waste composting operation at the SCLF. The operation co-composts together, a mix of dewatered biosolids received from local, Hillsborough County municipal wastewater treatment plants and yard waste received directly at the landfill from commercial and residential customers. The compost operation covers approximately 7 acres of an inactive area on top of the Capacity Expansion Area (CEA).

Yard waste is ground-up and mixed with biosolids at the facility and formed into windrows on an asphalt pad where it cures over a period of weeks. The material is periodically turned with a mechanical turner and after initial curing, is transferred to a final curing pile on the asphalt pad.

Following a few more months of curing the material is put through a mechanical screen for size control and moved to another area on the pad for temporary storage until it is taken away by the customer. The finished compost product is distributed to local farmers and the general public. A more complete description of the compost operation is included in the Composting Operation and Maintenance Plan.

K.2.d. Weighing Incoming Waste

All incoming waste will be weighed before disposal in the landfill. The existing scales are fully automated and computerized, with the capability for data storage and retrieval for daily record keeping and reporting. All customers are issued receipts upon exiting the Facility.

K.2.e. Traffic Control

The working face area is the most equipment-intensive area of operation for the Facility. In this area, solid waste transportation vehicles arrive, turn around, back up to the working face, and unload the solid waste. Landfill operation equipment will continually spread and compact the solid waste as it is received. During normal operating conditions, only one working face will be active at any given time, with the solid waste at all other areas within the landfill secured by a minimum of 6 inches of initial cover. The working face may alternate as needed between Phases I-VI to the CEA. It is intended that only one working face will be active at a time at either Phases I-VI or the CEA.

The approach to the working face will be maintained in an accessible condition so that two or more vehicles may safely unload simultaneously side by side. When unloading is complete, the vehicles will immediately leave the working face area. Entrance and exit haul roads will be provided (both temporary and permanent) and maintained to facilitate future unloading operations. Contractor personnel will direct traffic as necessary to expedite safe movement of vehicles and to ensure that all waste transport vehicles unload within the designated area.

K.2.f. Method and Sequence of Filling Waste

Each phase will be landfilled as shown in the Operating Sequence Plans provided ~~with the Phases I-VI and Capacity Expansion Area (Sections 7, 8, and 9) Permit Renewal Application and in Appendix E.~~in Appendix E. The lifts in each of the several phases are shown on one sheet to minimize the number of sheets, but each lift is independent of the others.

K.2.f.(1) Phases I-VI

One working face will be maintained for the anticipated traffic maneuvering during waste fill operations. Typical lifts consist of two lifts 8 to 10 feet high, to reach the maximum elevation shown on the operating sequence drawings including daily and intermediate cover. Because of the phosphatic clay liner stability in Phases I-VI, at no time shall a lift exceed the maximum height shown on the operating sequence drawings. The initial filling in Phases I-VI was completed in 2010. Waste filling will continue over the existing area as shown on the operating sequence plans.

Existing intermediate cover placed over the Phase I-VI area will be removed as landfilling progresses. The remaining air space in Phases I-VI is divided into eleven lifts (13-15, 16A, 17A, 18 - 23) as shown on the drawings.

The Contractor will prepare filling plans in accordance with the sequence drawings 45 days before the development of a new lift. Subsequently, grades for the new lift will be set on grade by a registered engineer, land-surveyor, or by an authorized agent.

Landfilling in Lifts 13~~-16~~ (Sheet ~~4~~4A) ~~begins~~ began on the west side of Phase I and ~~proceeds~~ proceeded ~~counter clockwise~~east over ~~Phases~~Phase I~~, II, III and IV~~.

Landfilling in ~~Lift 17~~Lift 16A (Sheet ~~5~~4B) begins on the ~~west~~east side of Phase ~~III~~IV and proceeds ~~from east to~~ west over Phases IV~~, V~~ and VI.

Landfilling in Lifts 17A (Sheet 4C) begins on the south side of Phase IV and proceeds clockwise over Phases IV, VI, and V until elevation 240 feet has been reached.

Landfilling in Lifts 14 and 15, (Sheet 5A) begins on the west side of Phase II and proceeds counterclockwise over Phases II and III.

Landfilling in Lifts 18-21 (Sheet 6) begins on the south side of Phase I and proceeds counter clockwise over Phases I, II, III and IV.

Landfilling in Lift 22 (Sheet 7) begins on the south side of Phase IV and proceeds from east to west over Phases IV, V and VI.

Landfilling in Lift 23 (Sheet 8) begins in the center of Phases I-VI, near Phase II and proceeds from east to west over Phases I through VI, to the permitted final grades (Elev 255) of the landfill. Upon completion of filling operations in Lift 23, final cover will be placed over the entire Phase I-VI area as described in Section K.7.h.

K.2.f.(2) Section 7 of the Capacity Expansion Area

The initial filling in Section 7 was complete as of May 2005. The outer sideslopes have not reached their final design 3H:1V slope. The temporary sideslopes of Section 7 will be filled to reach their maximum design slope of 3H:1V during waste filling operations in Section 9.

The east and south sideslopes as well as most of the top of Section 7 have received intermediate cover. Stormwater runoff from the top of Section 7 sheet flows to a downchute on the southeast corner that discharges to a culvert leading to sedimentation basin C (Sed C). Stormwater runoff from the sideslopes of Section 7 drains to the perimeter ditches, eventually flowing to the culvert to Sed C. Any stormwater that does not infiltrate into the ground at Sed C discharges to Pond C for additional attenuation prior to flowing through the on-site stormwater management system described in Section K.10.

K.2.f.(3) Section 8 of the Capacity Expansion Area

The initial filling in Section 8 was completed as of May 2007. Similar to Section 7, the outer sideslopes have not reached their final design slope of 3H:1V. The temporary sideslopes of Section 8 will be filled to reach their design slope during waste filling operations in Section 9.

The east and north sideslopes, as well as most of the top of Section 8 have received intermediate cover. Stormwater runoff from the top of Section 8 discharges to Sed C. Stormwater runoff on the east sideslope drains to perimeter ditches, eventually flowing to the culvert to Sed C. Stormwater runoff on the north sideslope of Section 8 flows easterly along perimeter ditches around the CEA eventually discharging through the culvert to Sed C. Any stormwater that does not infiltrate into the ground in Sed C discharges to Pond C for additional attenuation prior to flowing through the on-site stormwater management system described in Section K.10.

K.2.f.(4) Section 9 of the Capacity Expansion Area

One working face will be maintained for the anticipated traffic maneuvering during waste fill operations. Typical lifts consist of two lifts 8 to 10 feet high, to reach the maximum elevation shown on the operating sequence drawings including daily and intermediate cover.

The proposed filling sequence for Section 9 is presented in the drawings provided in Appendix E. The initial filling in Section 9 was completed as of July 2009.

Waste placement in Section 9 has proceeded against the west sideslopes of Sections 7 and 8 and landfilling of fill sequence 9-15 has been completed (CEA Sheet 6). Waste filling will continue incorporating areas of both Sections 7 and 8. As the Operations Fill Sequence Drawings show, filling will proceed to bring the sideslopes of Sections 7, 8, and 9 to their design slope of 3H:1V slopes as shown on fill sequence 16-18 (CEA Sheets 6 and 7). The filling of Section 7, 8, and 9 areas will bring the combined areas to an approximate elevation of 285 feet as shown on Sheet 8.

K.2.g. Waste Compaction and Application of Cover

Waste will be placed at the top or bottom of the working face and spread toward the bottom or top, respectively. Waste will be spread in approximately 2-foot-thick layers and compacted with a minimum of three to five passes of the landfill compactor. The spreading and compacting is intended to be a continuous operation. A minimum in-place waste density of 1,000 pounds/cubic yard (lb/cy) will be achieved.

A minimum of 6 inches of compacted initial cover or tarp will be placed over the waste at the end of each operation day in accordance with 62-701.500(7)(f)1. Auto shredder residue, alone or mixed with soil, recovered screen material street sweepings, screened ditch cleaning soil, and solid waste combustor ash residue may be used as initial cover as allowed by 62-701.500 (7)(e). Before the working face between landfills is moved, the area that will remain inactive will be covered with compacted initial cover, soil, or a mixture of 50 percent unscreened wood mulch and 50 percent soil (no ash), with sufficient thickness (minimum 6 inches) to prevent erosion and the mixing of leachate with stormwater. A minimum of 1 foot of intermediate cover, in addition to the 6-inch initial cover, will be applied and maintained within 7 days of cell completion if additional solid waste will not be deposited within 180 days of cell completion.

When landfilling operations begin again in areas with intermediate cover, the intermediate cover (free of waste) will be stripped from the surface (upper 12 inches) and reused over other areas needing intermediate cover. The stripped intermediate cover will be pushed ahead and used as perimeter berms around the active working face area. The intermediate areas are graded to promote drainage (minimum 2 percent slope) and seeded to prevent erosion.

K.2.h. Operation of Leachate, Gas and Stormwater Controls

See Sections K.8, K.9, and K.10 for leachate, gas, and stormwater controls, respectively.

K.2.i. Water Quality Monitoring

K.2.i.(1) Phases I-VI

Water quality monitoring for Phase I-VI is included in Section L of the Operation Permit Intermediate Modification Application, dated April 2015.

K.2.i.(2) Capacity Expansion Area

Water quality monitoring for Sections 7, 8, and 9 is included in Section L of the Operation Permit Intermediate Modification Application, dated April 2015.

K.2.j. Leachate Collection and Removal System Maintenance

Refer to the current LMP Report in Appendix C of the April 2015 Operation Permit Intermediate Modification Application.

K.3 OPERATING RECORD

The operating record will be maintained on site in the Administration Building or at the SWMG office. The operating record will be accessible to the Facility operation personnel and will be available for inspection by FDEP. The records include the following:

- Waste reports
- Operation permits
- Construction and closure permits including any modifications

- Monitoring results, such as water quality testing
- Notifications to FDEP
- Engineering drawings
- Training certifications as required by Chapter 62-701.320(15), FAC

K.4 WASTE RECORDS

K.4.a. Amount and Origin of Waste

The amount of solid waste received at the landfill will be weighed and recorded in tons per day in accordance with Rule 62-701.500(4), FAC. Waste reports, including the amount received and county of origin, for the waste types listed in Section K.4(b) will be compiled monthly and provided annually to the FDEP.

K.4.b. Waste Types

All reports will contain a minimum of the following waste types:

- Class I waste
- Class III waste
- Ash residue
- Other waste

K.4.c. Construction and Demolition Debris

If dedicated loads of construction and demolition debris (C&D) are received, an annual report will be submitted to the FDEP as required in subsection 62-701.730(12), FAC and form 62-701.900(7). This report will include tonnage of material types received and recovered based on county of origin.

K.5 ACCESS CONTROLS

The perimeter fence and berms around the Facility prevent the entry of livestock, protect the public from exposure to potential health and safety hazards, and discourage unauthorized entry or uncontrolled disposal of unauthorized materials. 'No trespassing' signs are also posted along the perimeter fence. The SWMG and Contractor personnel will inspect the premises daily. The gate at the Facility entrance and all other gates will be kept locked at all times the landfill is closed, and the Contractor will provide security personnel to guard the Facility during non-operating hours.

K.6 LOAD-CHECKING PROGRAM

The SWMG has established a random-load-checking program as referenced in Part K.2.c to detect and prevent disposal of unauthorized wastes into the landfill. In addition, site access control discourages the disposal of unauthorized and hazardous wastes. A sign at the entrance of the Facility explains the types of waste prohibited at the landfill.

In accordance with Rule 62-701.500(6)(a), FAC, a minimum of three random loads will be checked at the active working face(s) each week. The selected drivers will be directed to discharge their loads at a designated location next to the working face. If any unauthorized special waste (i.e., lead-acid batteries, used oil, yard trash, white goods, and whole tires) is found during the random inspection or as part of routine operations, the waste will be segregated and removed from the site for recycling as described in Part K.2.c. These special wastes will be stored next to the working face and removed from the site within 30 days.

If an unauthorized waste (i.e., hazardous, PCBs, untreated biomedical, or free liquid) is found, the generator of the waste, if known by the driver, will be contacted to determine the waste source. Either the hauling company or the generator of the waste will be directed to remove the unauthorized waste. The random load inspections will be documented on a report from which includes the date and time, name of the hauling company and the driver of the vehicle, the vehicle license number, the source of the waste or generator, and any observations or notes made by the inspector (Appendix C).

The inspector will identify and note all unauthorized waste found during the random load inspection, estimated quantity, and the action taken. The inspector will sign the inspection form that will be retained at the Facility.

If the waste owner cannot be identified, the waste will be evaluated by Contractor personnel in charge. The waste will be isolated and contained and will not be moved until the waste is determined to be acceptable. If it is determined that the waste is not suitable for disposal, the SWMG will be notified for additional assessment and testing of the waste. Subsequently, a record of the decision will be placed into the daily operations file for the Facility.

If any regulated hazardous waste is discovered in a random load check or is identified by an operator or spotter, the Landfill Manager and the FDEP will be notified immediately as well as the generator or hauler, if known. The Landfill Manager is trained in the proper procedure to follow including notifications. If the generator or hauler is not known, the SWMG will be responsible for disposing of the hazardous waste at a properly permitted Facility. The hazardous waste will be isolated and restricted from access until it is removed from the landfill by a qualified hazardous waste contractor. Hazardous wastes will be removed from the site within 24 hours.

As required in Rule 62-701.320(15), FAC and discussed in Part K.1, inspectors, scale-house attendants, equipment operators, and landfill spotters will be trained to identify unacceptable wastes and hazardous wastes.

K.7 SPREADING AND COMPACTING WASTE

All loads coming into the Facility, including small-volume containers, will be delivered to the working face daily. To preserve the prepared base area and to protect the leachate collection system, traffic will be prohibited to operate directly on the chipped tires overlying the drainage layer. Traffic will only be allowed to maneuver on top of the compacted and covered waste. Therefore, the initial lift of all new disposal areas will be accessed by vehicles from the top of the working face. The waste will be spread and compacted from the top, keeping all heavy equipment off the prepared base.

For all subsequent lifts, the waste placement will vary depending on field conditions. Some lifts will be built from the bottom of the active working face. At the discretion of the operator, waste will also be placed from the top of the active working face and spread toward the bottom. Waste will be placed against the covered working face of the previous day's waste. The first cell will act as a means of access and as a berm to guide the placement of waste for the remaining cells. See Part K.2.g for additional information on waste compaction.

The following guidelines will provide an efficient and environmentally sound method of operation for the Facility:

- Portable litter fencing will be placed at the working face where needed to reduce windblown litter.

- Cracks or eroded sections in the surface of any filled and covered area will be repaired and a regular maintenance program will be followed to eliminate pockets or depressions that may develop as waste settles.

- If 12 inches of intermediate cover (free of waste) has been placed over a partially filled area, it will be removed, reused, and stockpiled for later use before the placement of a new lift.

- Tire chips, ash residue from incinerated MSW, tarps, soil, or a 50/50 soil/mulch mix may be used for initial cover. Stormwater runoff will not be allowed from waste-filled areas covered with tire chips or ash. Runoff from outside the bermed working face area will be considered stormwater only if the flow passes over areas that have no exposed waste and have been adequately covered with a tarp or at least 6 inches of compacted soil (or a mixture of soil/mulch) which is free of waste and has been stabilized to control erosion.

- Sufficient cover material will be stockpiled near the working face to provide an adequate supply for initial cover operations. In some areas, daily stockpiling may not be necessary because of the proximity of the borrow area.

K.7.a. Waste Layer Thickness and Compaction Frequencies

Landfill personnel will direct all incoming waste to be unloaded at the toe or top of the working face. Waste will be spread in approximately 2-foot-thick layers and compacted with a minimum of three to five passes of the landfill compactors. The spreading and compacting is intended to be a continuous operation, and waste will not be placed in a layer until the previous layer is compacted.

K.7.b. First Layer Thickness

For Phases I-VI and Sections 7, 8, and 9, the initial waste layer has been placed. To protect the integrity of the leachate collection system of the landfill, traffic and heavy equipment were not allowed directly on the sand drainage layer.

The procedure for filling and compacting the first layer of waste for future permitted sections at the Capacity Expansion Area will protect the integrity of the liner and leachate collection system. Traffic directly on the protective layer will be prohibited, and the first lift will be accessed by vehicles from the top of the working face. An initial 4-feet-thick lift of selected waste will be placed over the protective layer. The selected waste will be MSW and ash not containing large rigid objects and will be spread and compacted from the top of the working face.

K.7.c. Slopes and Lift Depth

The working face slope will be maintained at a slope no steeper than 3H:1V. Each cell will be constructed in a horizontal lift to an approximate height of 8 to 12 feet, with the maximum height as shown on the Drawings provided separately with the Phases I-VI and the Capacity Expansion Area (Sections 7, 8, and 9) Operation Permit Renewal Application as shown in Appendix E.

K.7.d. Working Face

Cells will be constructed with slopes no steeper than 3H:1V, and a working face will be maintained to provide unhindered vehicle access to the working face while minimizing exposed areas and unnecessary use of cover material. The working face may alternate as needed between Phases I-VI to the CEA. The working face will be bermed with soil or a 50/50 soil/mulch mix (no ash). The berm will be constructed to prevent the mixing of leachate with stormwater.

K.7.e. Initial Cover Controls

At the end of each working day, the waste will be covered with a 6-inch lift of compacted cover material such as soil, a mixture of 50 percent wood mulch and 50 percent soil (or ash), ash, chipped tires, tarps or other materials as approved in 62-701.500(7)(e) FAC, in accordance with 62-701.500(7)(f)1. These cover materials will provide vector control, mitigate windblown litter, reduce the potential for fire, and reduce odors and moisture infiltration into the waste. The initial cover material will be spread over the exposed waste and, with the exception of tarps, compacted by the equipment used to spread the cover (i.e., bulldozer or scraper). The initial cover material will not be removed before placement of successive lifts of waste, with the exception of tarps, which will be removed before placement of successive lifts. Any remaining litter and cleanings from equipment will be placed at the bottom of the completed cell and covered.

Before the working face between landfills is moved, the area that will remain inactive will be covered with compacted cover (free of waste), soil, or a mixture of 50 percent unscreened wood mulch and 50 percent soil (no ash), with sufficient thickness (minimum 6 inches) to prevent erosion and the mixing of leachate with stormwater.

K.7.f. Initial Cover Frequency

At the end of each day's operation, the active landfill working face will be thoroughly compacted, and cover material will be spread and compacted to a depth of 6 inches over the day's entire working face and sideslopes in accordance with 62-701.500(7)(f)1. Initial cover material is discussed in Part K.7.e. If needed, the portable barriers that define the working face will be moved to the positions required to define the next day's operation.

The Facility is equipped to excavate and haul cover materials from on-site borrow areas to the working face. Additionally, an elevating scraper is used to excavate and haul cover material from the borrow area to the working face where it can be spread by a scraper or bulldozer.

When using a 50/50 mixture of soil and mulch the following process will be used:

1. The area to be excavated will be identified in advance. The area used for mulch mixing will not be larger than 15 acres.

2. A 4-foot layer of mulch will be placed over the designated excavation area.

3. As the area is excavated, the excavator will take bucket loads of the mulch layer plus 4 feet of soil, mixing the load as it is placed in the dump trucks.

4. The trucks will deliver the load to the working face. As the loads are deposited, additional mixing will occur.

5. The soil/mulch mixture will be spread over the working face using a bull dozer, causing additional mixing.

K.7.g. Intermediate Cover

Intermediate cover will be placed and maintained over cells which will not receive additional solid waste or final cover within 180 days as required in Rule 62-701.500(7)(g), FAC. Recovered screen material or a mixture of soil and ground or chipped yard trash provided that soil makes up at least 50 percent by volume of the mixture may be utilized as intermediate cover. The working face will be bermed to reduce stormwater impacts. Sideslopes will be well maintained to minimize erosion. Intermediate cover material will be placed over the landfill surface within 7 days of cell completion if additional waste will not be placed within 180 days. Intermediate cover will be placed to a minimum compacted thickness of 12 inches on top of the 6 inches of compacted initial cover. On-site material will be used for intermediate cover. Specifically, phosphatic waste clays available on site will be mixed with sand and used for intermediate cover.

To conserve the soil/clay mix, a portion of the intermediate cover will be removed immediately before placement of additional solid waste on top of the lift or before placement of additional waste. The soil/clay mix (free of waste) will be stripped and reused as initial or intermediate cover material. The stripped intermediate cover will be pushed ahead as needed for the perimeter interceptor berms constructed around the active working face area. The intermediate cover areas will be graded to promote drainage (minimum 2-percent slope) and seeded to prevent erosion.

K.7.h. Final Cover

K.7.h.(1) Temporary Final Cover

A temporary final cover consisting of a soil layer will be installed over cells in Phases I-VI and/or the CEA which will not receive additional solid waste. The temporary final cover will consist of a 12-inch layer of soil with a hydraulic conductivity of 1.0 x 10^{-5} cm/sec. Vegetative cover will be placed on areas which have reached interim final grade in Phases I-VI. These areas will not receive additional waste until the end of the consolidation period before waste can be filled on top of the area. In CEA Sections 7, 8, and 9, the temporary final cover will be installed on the south and east side slopes as shown on the drawings. As required, temporary drainage berms and downchutes will be placed at the working face to control and direct stormwater runoff away from disposal areas.

K.7.h.(2) Final Cover

When portions of the Facility are brought to design grades, final cover will be placed over the areas that have attained final elevation within 180 days in accordance with Rule 62-701.500(7)(h), FAC. Vegetative cover will be established. The final cover system and sequence for final cover placement will be submitted with the application for closure at least 90 days before the partial closure of the sideslopes.

K.7.i. Scavenging and Salvaging

Except for such operations that are conducted as part of a recycling program, scavenging and salvaging are not permitted at the Facility. If the volume of recyclable goods is sufficient, as determined by the Landfill Manager, those items may be separated from the waste which is to be disposed.

During waste placement on the landfill, recyclable items such as wood, concrete, metals, cardboard, and other recyclables may be manually pulled from the active face, segregated, and placed in the staging area/roll-off containers adjacent to the working face area. With the exception of clean concrete, the remaining materials will be transferred off-site for recycling. The clean concrete will be stored on site until sufficient quantity is stockpiled and used for on-site road base or other on-site uses.

After the recyclable materials have been removed, the remaining materials will be disposed in the active Class I waste disposal area of the landfill.

Any recycling method, other than manual extraction, will only be implemented following review and concurrence by the FDEP.

K.7.j. Litter Policing

If necessary, portable litter fences will be placed downwind of the immediate working area to confine most of the windblown material. Litter around the site and the entrance roadways will be collected regularly and picked up within 24 hours, in accordance with Rule 62-701.500(7)(j), FAC.

K.7.k. Erosion-Control Procedures

The Facility fill sequence and the drainage facilities have been designed to minimize erosion of landfill sideslopes and washout of adjacent areas. The landfill surface will be inspected daily for cracks, eroded areas, and depressions in the landfill surface. Corrective action will be implemented within 7 days of detection. In areas where standing water develops, the area will be filled, compacted, and graded to provide positive drainage. Where the standing water problem cannot be corrected by proper grading, temporary drainage ditches will be constructed to drain off the standing water. Intermediately covered areas or other areas that discharge to the stormwater management system and which exhibit significant erosion will be repaired as follows:

- If greater than 50 percent of the soil cover material has eroded, the area will be repaired within 7 days.
- If waste or liner is exposed, the area will be repaired by the end of the next working day.

K.8 LEACHATE MANAGEMENT

Please see the revised LMP (Appendix B of the Operation Permit Intermediate Modification dated September 2015).

K.9 GAS MONITORING AND MANAGEMENT PROGRAM

K.9.a. Gas Monitoring

SWMG personnel shall monitor and record landfill gas (LFG) readings quarterly at the perimeter LFG monitoring wells and in the Administration, LTRF, and Maintenance buildings. The locations of the existing LFG monitoring points are included in Appendix F. The ambient air and areas with slab penetration (areas with plumbing for water and drains) will be monitored inside these structures. The monitoring will be conducted for the Lower Explosive Limit (LEL) of methane using a GEM-500 Infrared Landfill Gas Analyzer (or equivalent). The probes will not be purged. Once the GEM is connected to the sampling port, the valve will be opened and the GEM pump will be started. The GEM reading will be observed and the value will be recorded.

When personnel must enter confined spaces or areas where dangerous gases may be present, the SWMG will follow the requirements in the "Code of Federal Regulations Title 29, Part 1910.146 OSHA" and the safety guidelines outlined in "A Compilation of Landfill Gas and Field Practices and Procedures" prepared by the SWANA Landfill Gas Division Health and Safety Task Force.

If methane is detected in concentrations greater than the regulatory limit (100 percent of the lower explosive limit at the property boundary or 25 percent of the lower explosive limit within structures), the SWMG will evaluate potential measures to correct the exceedances. If an unacceptable concentration of methane is detected in a monitoring location (i.e., a well or an on-site structure), the SWMG will immediately take appropriate actions to protect human health. The SWMG will notify FDEP and will re-monitor the location during each of the next 3 days. During this time the SWMG will evaluate potential causes of the exceedance and will implement procedures to remedy the situation if exceedances persist after the third day. Within 7 days of the initial exceedance, the SWMG will submit a remediation plan to FDEP in accordance with Rule 62-701.530(3)(a) FAC.

Landfill Gas Monitoring Points	
I.D.	Probe/Building Location
LFG-1	Property boundary probe: South property boundary
LFG-2	Property boundary probe: Southwest property boundary
LFG-3	Property boundary probe: Northwest property boundary
LFG-4	Property boundary probe: North property boundary
SP-1	Scalehouse/Administration Building
SP-2	Scalehouse/Administration Building
SP-3	Scalehouse/Administration Building
SP-4	Scalehouse/Administration Building
SP-5	Scalehouse/Administration Building
SP-6	Scalehouse/Administration Building
SP-7	Scalehouse/Administration Building
SP-8	Scalehouse/Administration Building
SP-9	Maintenance Building
SP-10	Maintenance Building
SP-11	Maintenance Building
SP-12	Maintenance Building
SP-13	Leachate Treatment Facility Building
SP-14	Leachate Treatment Facility Building
SP-15	Leachate Treatment Facility Building

As described in Part K.7, the SWMG has a program for the placement of cover, which is effective for controlling disease, vectors, objectionable odors, and litter. No objectionable odors have been detected or reported by adjacent property owners. At least quarterly, or more frequently if necessary, qualified personnel from the SWMG will assess the presence of ambient objectionable odors at the perimeter monitoring points shown in Appendix F. If objectionable odors are detected at the property line, the SWMG will implement an odor-monitoring program as required by Rule 62-701.530(3)(b) FAC.

K.9.b. Landfill Gas Collection System

The design of the Landfill Gas (LFG) collection system and the subsequent operation is in accordance with the federal New Source Performance Standards (NSPS) for municipal solid waste landfills (Subpart WWW) and Subpart AAAA of the National Emission Standards for Hazardous Air Pollutants (NESHAP), which dictates the operational procedures for the gas collection and control (GCCS).

Landfill gas that is generated in the landfill is currently collected by the system GCCS in Phases I-VI and Sections 7, 8, and 9. Permit No. 35435-016-SC/08 details the requirements of the GCCS. The SCLF continues to remain in compliance with the GCCS operation and Title V permit requirements. The repairs and upgrades to the GCCS in the area of the former sinkhole have been completed and were designed to provide landfill gas collection and extraction per the pre-sinkhole conditions and in accordance with the previously permitted GCCS design intent.

The facility maintains all operational and manufacturer procedural documentation for the blower, flare, control devices, and LFG system components on site in the “LFG Specialties User Manual for Utility Flare System Unit 2162”, dated September 2009.

For additional information on the GCCS operating and maintenance procedures and safety protocols, refer to the GCCS Design Plan, the Startup, Shutdown and Malfunction Report (SSM), and current Title V Air Operation Permit.

K.10 STORMWATER-MANAGEMENT SYSTEM

K.10.a. Leachate Reduction

K.10.a.(1) Stormwater Diversion

K.10.a.(1).1 Site Stormwater System

The stormwater system was designed to transport the maximum expected flows from a 24-hour, 25-year rainfall event and minimize the collection of standing water within the disposal areas. To efficiently collect and transport the stormwater runoff away from the disposal areas, the stormwater system will be maintained in good condition, with the proper slopes and free from obstructions. Erosion control measures and corrective action are described in Part K.7.k of the Operation Plan. In addition, the design maintains conformance with the site’s Southwest Florida Water Management District (SWFWMD) Stormwater Permit (a copy was submitted in Volume 3 of the Construction Permit Application for the Capacity Expansion Area, Section 7, September 2002). The major stormwater component designs and operations are as follows:

- Interior Stormwater Separation berms are generally designed to be 3 feet high and 3 feet wide across the top with sideslopes of 3H:1V. The separation berms divide the contributing runoff areas to facilitate the collection and handling of stormwater as well as providing separation from leachate.

- Sideslope swales were designed to convey stormwater flow from the sideslopes to the downchutes as shown on the drawings. Sideslope swales will be constructed where needed and as shown on the sequence drawings provided separately with the Phases I-VI and Capacity Expansion Area (Sections 7, 8, and 9).

- Downchutes constructed on the side slopes of the landfill will transport stormwater flow to the perimeter stormwater ditches.

- The perimeter stormwater ditches collect surface water runoff around the site, prevent offsite drainage from entering the landfill area, and drain runoff to the appropriate stormwater ponds and sedimentation basins located around the site.

K.10.a.(1).2 Phases I-VI

The Phases I-VI stormwater collection system directs stormwater runoff from the landfill and surrounding sub-shed areas and into stormwater sedimentation basins and detention ponds. The sedimentation basins are designated A-2, A-3, B, C, 2, 3, 4, and 8. The ponds are designated as Ponds A-1, B, C, D, and E, and an evaporation area. As the Phase I-VI areas are filled with waste, daily and intermediate cover (clean fill) is applied over the waste which promotes drainage away from the waste material. This minimizes the amount of water that is allowed to infiltrate into the waste. Stormwater that comes in contact with the waste in the active working area is considered leachate and will not be allowed to run off into the stormwater management system. The size of the working area will be kept to a minimum to minimize leachate and berms around the working area will separate stormwater from leachate. The runoff will be directed toward downchutes that will be conveyed to one of the basins.

K.10.a.(1).3 Capacity Expansion Area

The CEA stormwater collection system directs stormwater runoff from the landfill and surrounding sub-shed areas and into the existing stormwater sedimentation basins and detention ponds. The receiving basins are designated as Sed C and Seds 2, 3, 4, and 8, which flow into Ponds C and D, respectively. As the CEA, currently Sections 7, 8 and 9, is filled with waste, it will then be covered with daily and intermediate cover (clean fill) to allow drainage away from the waste. This minimizes the amount of water that is allowed to infiltrate into the waste. Stormwater that comes in contact with the waste (now considered leachate) in the active working area will not be allowed to run off into the stormwater management system. The size of the working area will be kept to a minimum to minimize leachate. Berms around the working area will separate stormwater from leachate. The runoff will be directed toward downchutes and transported via stormwater ditches to Sed C and Pond C. The undeveloped areas of the CEA will collect and drain stormwater runoff to sedimentation basin D (Sed D) and Pond D.

K.10.a.(1).4 Stormwater Management System Improvements

Improvements to the Stormwater management System (SWMS) at the SCLF were completed in March 2012, see figure in Appendix H. Improvements to the existing SWMS as part of the Stormwater Improvements Project consisted of the following:

1. Conversion of dry retention Basins A, B and C from underdrain systems to wet detention systems (Basin C was converted from dry retention with underdrain system to wet detention system as part of Section 9 construction in April 2008).

2. Restructuring of evaporation areas located north of the scale house and WMIF's maintenance building to increase attenuation with a wet pool design. New Ponds A-1, A-2 and A-3, and existing Basins F and G are interconnected and function as one system that ultimately discharges through modified control structures in

Pond B. New Ponds A-2 and A-3 increase retention times of runoff from Phases I-VI with treatment provided in Pond B.

3. Sedimentation ponds between Phases I-VI and the CEA, SED-2, SED-3, SED-4 and SED-8, were constructed provide additional settling areas and reduce sediment transport into Basin D. These sedimentation swales and ponds provide some treatment, but most of the treatment will continue to be provided by the existing Basin D.

K.10.a.(1).5 Other Site Stormwater Basins

Several other basins located around the site collect stormwater runoff; however, they do not collect runoff from disposal areas. The other basins are mentioned in this plan for informational purposes. Basins E, F and G collect runoff from the scalehouse. Stormwater Detention Basin H collects runoff from the LTRF.

K.10.a.(2) Rain Tarps

Rain tarps will be used to cover open areas (areas that have not received waste material yet but are connected to the leachate collection system) to keep stormwater out of the leachate collection system. Water that has collected on top of the rain tarp is considered stormwater and can be pumped to the appropriate stormwater basin that was designed for that area. Before placement of waste, all rain tarps will be removed.

K.10.a.(3) Stabilized Slopes

As filling progresses, the top and side slopes that will not receive additional solid waste for 2 or more months will be stabilized. First, compacted fill will be placed over the waste material to keep stormwater from infiltrating into the waste and to promote runoff. The slopes can then be stabilized with vegetative cover, seed, and mulch, or rain tarp covers. Exterior side slopes that are constructed to design grade and interior side slopes that will not receive waste for longer than 180 days will be covered with intermediate cover and either vegetative cover or hydroseed.

K.10.a.(4) Closure

As disposal areas reach final elevations as discussed in Part K.7.h, areas may have a final or temporary final cover placed over the waste material that will provide a low permeability cover over the waste and thus minimize long-term infiltration of stormwater into the waste materials as described in Section K.7.h.(1). As stormwater infiltration is cut off, water within the waste will drain to the leachate collection system within the lined area of the landfill. Since infiltration of stormwater will be minimal, the amount of leachate resulting from stormwater infiltration will reduce over time.

The methods described above represent the current plan; however, as operations continue, they may be modified if alternate methods prove more efficient or allow a higher percentage of stormwater runoff, thus resulting in greater leachate minimization.

K.11 EQUIPMENT AND OPERATION

Landfill operation was discussed in Part K.2.

K.11.a. Operating Equipment

The landfill is typically operated with the following on-site equipment:

- Steel-wheeled compactors.
- Bulldozers.
- Articulated dump truck.
- Water tank truck.
- Motor grader.
- Excavator.
- Several pickup trucks.
- Other miscellaneous construction and maintenance equipment.

Where appropriate, equipment is fitted with safety cabs and fire extinguishers. The Contractor is required to have back-up equipment available within 24 hours.

K.11.a.(1) Equipment Care

Routine preventive maintenance minimizes equipment downtime and increases equipment service life. Therefore, the appropriate operation and maintenance (owner's) manual should be consulted. However, applicable maintenance activities implemented at the site include:

- A routine inspection program;
- Routine lubrication; and,
- Maintenance records up-keep.

Minimal equipment washing using low-volume, high-pressure technique may be performed on lined areas of the landfill that do not have intermediate or final cover. The activity is exempt from industrial wastewater permitting since the wash water is collected by the leachate collection system. Washing will occur within, or adjacent to, the active working face. Runoff will be contained within the limits of the lined landfill and not allowed to comingle with stormwater runoff.

K.11.b. Reserve Equipment

Sufficient backup equipment will be provided on site for equipment breakdowns and downtime for normal routine equipment maintenance. Pre-arrangements with contractors and rental equipment dealers will be made to furnish equipment on short notice in the case of a major equipment failure. The Reserve Equipment Agreement is presented in Appendix B.

K.11.c.Communications Equipment and Personnel Facilities

Telephones are located at the Administrative and Maintenance Buildings for use in emergencies. Cellular telephones and two-way radios are also used. The Administration Building is equipped with water supply, toilet facilities, emergency first-aid supplies, and electricity. The building also provides shelter for employees in case of inclement weather. The Maintenance Building is equipped with spare parts, tools, equipment, and electrical services for operations and repair.

K.11.d. Dust Control

K.11.d.(1) Phases I-VI

Dust control outside of the landfill will be provided by applying water sprayed from a water tank truck and will be applied to the unpaved access roads as required to control dust generation. Dust control inside of the landfill will be provided by applying small quantities of leachate as described in Section 8.4 of the LMP.

K.11.d.(2) Capacity Expansion Area

Dust control outside of the landfill will be provided by applying water sprayed from a water tank truck and will be applied to the unpaved access roads as required to control dust.

Dust control inside the active waste disposal areas will be provided by applying small quantities of leachate from a spray bar mounted on the rear of a tank truck. Leachate will be sprayed onto the active fill areas of the CEA, including the working face, which includes a berm to prevent runoff, and areas with the required 6 inches of initial cover as required to control dust.

Leachate used as dust control reduces the amount of fresh pond water that would otherwise be sprayed from tanker trucks to control dust on the active fill areas and provides for leachate evaporation. Leachate quantities used for dust control will continue to be reported in the leachate balance report submitted to the FDEP.

The SWMG will monitor the rate of application, soil moisture conditions, and the specific landfill areas used so that this leachate disposal method does not generate runoff. Spray bar leachate spraying will be applied under the following conditions:

- Leachate will only be sprayed on active-fill areas, including the working face that includes a berm to prevent runoff and areas with the required 6 inches of compacted initial cover.

- Leachate will not be sprayed on areas with intermediate or final cover, seeded or unseeded, or on areas that do not have a berm to prevent runoff.

- The maximum grade leachate will be sprayed on is 10H:1V slope. Areas within 150 feet of a 4H:1V or steeper sideslope will not be sprayed. Areas receiving leachate will be controlled at all times to prevent leachate runoff from entering the stormwater system.

- Leachate will not be sprayed during a rainfall event.

- The tank truck spray bar method maximizes evaporation. The application rate of leachate will be such that leachate does not accumulate on the landfill surface nor infiltrate quickly into the covered refuse. The main goal of this leachate disposal method is evaporation rather than recirculation of leachate.

- Leachate will not be sprayed at the end of the day on the initial cover of the working face or other areas. Spraying should be done early in the morning after any dew evaporates and continue until early afternoon or until all available areas have been used.

K.11.e.Fire Protection and Chemical Fires

A charged fire extinguisher is kept at the scalehouse, Administration Building, Maintenance Building, and with all landfill equipment all times. Excavated soil will be used for fire control at the working face.

If a load of waste delivered to the site is smoking or on fire, landfill personnel direct the load to the "hot spot" area (an area within the landfill footprint with at least 12 inches of soil cover) where appropriate fire fighting procedures are followed.

Water for fire protection will be supplied from the fire hydrant and intake structure located east of Phase II. A second fire hydrant and intake structure is located south of the LTRF. If there is a small fire at the working face, waste handling will continue on an alternate working face until the fire is suppressed. If a fire cannot be controlled using materials and personnel already on site, the Fire Department will be immediately contacted and the emergency response plan described in Part K.2.b will be followed. See Part K.2.b for spills and containment of contaminated water such as from fire fighting.

No chemicals will be accepted at the landfill. All waste coming through the scale house will be observed to eliminate unwanted chemicals capable of starting a fire. If a chemical accident does occur, the following steps will be taken:

- Call the local Fire Department (911).
- Contain the fire in a small area until Fire Department arrives. To eliminate inhalation of potentially toxic fumes, fight fire from the upwind side.
- Take appropriate steps to contain and control the fire to the greatest extent possible while protecting human life and health.

K.11.f. Litter Control Devices

See Part K.7.j of this Operation Plan.

K.11.g. Signs

A sign indicating the hours of operation is located at the Facility entrance. Signs indicating the name of the operating authority, charges for disposal, and identifying the asbestos disposal site are located near the scalehouse area. Traffic flow and speed limit signs are located at various points along the landfill access road.

K.12 ALL-WEATHER ACCESS ROAD

The access roadway enters the site from CR 672. An asphalt paved road travels north from CR 672 and turns east into the Facility. The access road location was selected to minimize impacts to residential and agricultural areas along CR 672. There is a gate on the access roadway at CR 672 and fencing to prevent unauthorized access.

The main access road is a 40-foot-wide roadway with a 24-foot-wide asphalt paved section and 8-foot-wide shoulders constructed within the 100-foot-wide right-of-way. The main access road is paved and extends into the Facility through the property entrance, runs along the south side of the site, and turns north along the east side of the Facility area.

Other on-site roadways will be required on a temporary and permanent basis to serve the borrow area and for maintenance and services of on-site facilities. A stockpile of materials to construct and maintain all-weather roads to the active working face is available on site.

K.13 ADDITIONAL RECORDKEEPING

Operation records, such as permits, plans, inspections and others, are maintained at the Facility and at the SWMG office. The active area of Phases I-VI will be surveyed monthly and the active area of the CEA will be surveyed twice each year to calculate the volume used and to estimate the in-place density.

K.13.a. Permit Application Development

The SWMG keeps all information including site investigations, construction records, operation records, inspections, and permits.

K.13.b. Monitoring Information Records

The SWMG also keeps all monitoring records on groundwater, surface water, weather, and landfill gas. Copies are regularly submitted to the FDEP and the Environmental Protection Commission of Hillsborough County.

K.13.c. Remaining Site Life Estimates

An estimate of the remaining site life for the permitted area will be prepared annually for submission to the FDEP.

K.13.d. Archiving and Retrieving Records

Records of the landfill that are more than 3 years old will be available at the Facility.

APPENDIX A

TRAINING COURSES

Updated 4/3/2013

CEUS Currently Approved by the Florida SWMTC for 1/2013-12/2015 for Solid Waste Operators/Spotter

http://landfill.treeo.ufl.edu/Courses.aspx

Course #	Course Title	Course Provider	Landfill	Construction & Demolition Debris	Transfer Station	Materials Recovery Facility	Spotter	
203	8-Hour Initial Training Course for Spotters at Class I,II,III Facilities, Waste Processing Facilities, and C&D Sites	Kohl Consulting, Inc.	8	8	8	8	8	Initial
214	Spotter Training Plan for Land Clearing Debris Site	Wetland Solutions	8	8	8	8	8	Initial
219	8 Hour Initial Training for Spotter	Consolidated Resource Recovery, Inc.	8	8	8	8	8	Initial *Restricted*
248	Spotter Training for Solid Waste Facilities	University of Florida TREEO Center	8	8	8	8	8	Initial
442	24-Hour Initial Training Course for Landfill Operators of Class I, Class II, Class III, and C&D Sites	UF TREEO	16	16	8	8	4	Initial
443	16-Hour Initial Training Course for Operators of Transfer Stations and Material Recovery Facilities	UF TREEO	12	12	8	8	4	Initial
444	SWANA-Transfer Station Design & Operations	SWANA	8	8	8	0	8	Initial
462	8-hour Training Course for Spotters at Landfills, C&D Sites and Transfer Stations	UF TREEO	8	8	8	8	8	Initial
488	8-Hour Spotter Training Class I II III Landfill C&D Sites and Transfer Facilities	Safety Consulting and Training	8	8	8	8	8	Initial
582	16-Hour Initial Traiing Course for Transfer Station and MRF Operators	Kohl Consulting Inc	10	10	8	8	4	Initial
608	24-Hour Initial Training Course for Landfill Operators (Class I III and C&D Sites)	Kohl Consulting, Inc.	16	16	8	8	4	Initial
598	SWANA - Manager of Landfill Operations [MOLO] & Exam	SWANA	16	16	8	8	4	Initial
706	The SWM Combo Class: 24-Hour Initial Trainig Coruse for Landfill Opertors (Class I, II, III and C&D Sites) with 16-Hour Initial MRF/TS Opertor Class and 8-Hour Spotter Class [Initial Only]	Kohl Consulting Inc.	24	24	16	16	8	Initial
700	Construction and Demolition Debris Recycling and Management Workshop	FDEP & SWIX	4	4	4	4	4	
701	SWANA-FL 2012 Summer Conference	SWANA-FL	8	8	4	4	4	
702	2012 NAHMMA Florida Chapter HHW/SQG Workshop and General Session	NAHMMA-Florida Chapter	4	4	4	4	2	
703	16-hour Landfill Operator Refresher Course	Kohl Consulting Inc	16	16				
704	SWANA - WasteCon 2013	SWANA	8	8	7	5	2	
705	The Nitty Gritty of Native Bvegetation on Landfills - eCourse	SWANA	1	1				
706	The SWM Combo Class: 24-Hour Initial Trainig Coruse for Landfill Opertors (Class I, II, III and C&D Sites) with 16-Hour Initial MRF/TS Opertor Class and 8-Hour Spotter Class [Initial Only]	Kohl Consulting Inc.	24	24	16	16	8	
707	OSHA 1910.120 HazWoper Refresher	Burt McKee	4	4	4	4	4	
708	Train-the-Trainer: How to Design & Deliver Effective Training	University of Florida TREEO Center	7	7	7	7	2	
709	Fundamentals of Slope Stability and Settlement for Solid Waste Disposal Facilities	University of Florida TREEO Center	16	16				
710	Basic Water and Wastewater Pump Maintenance	University of Florida TREEO Center	4	4				
711	Pumping Systems Operation and Maintenance	University of Florida TREEO Center	4	4				
712	Basic Electricity for the Non Electrician	American Trainco	2	2	2	2		
713	24-hour HAZWOPER OSHA Training course - online	University of South Florida - OSHA Training Institute	6	6	6	6	3	
714	8-hour HAZWOPER Refresher Training course - Online	Safety Unlimited Inc	4	4	4	4	4	

Course #	Course Title	Course Provider	Landfill	Construction & Demolition Debris	Transfer Station	Materials Recovery Facility	Spotter	
715	8-hour HazWoper Refresher - Operations Level	American Compliance Technologies	4	4	4	4	4	
716	8-hr Hazwoper OSHA Refresher	FDEP	4	4	4	4	4	
717	4-hour OSHA Hazardous Materials Awareness Level Course	Local Environmental Planning Council - District 5 and Citrus County Solid Waste Dept	4	4	4	4	4	
718	4-Hour Refresher Course for Spotters at Landfills, C&D Sites and Transfer Stations	University of Florida TREEO Center	4	4	4	4	4	
719	Waste Screening Refresher	University of Florida TREEO Center	4	4	4	4	4	
720	Hazardous Waste Regulations in Solid Waste Operations and Recycling	University of Florida TREEO Center	8	8	8	8	4	
721	Hazardous Waste Regulations in Solid Waste Operations	University of Florida TREEO Center	4	4	4	4	4	
722	Health and Safety for Solid Waste Workers [am]	University of Florida TREEO Center	4	4	4	4	4	
723	Health and Safety for Solid Waste Workers [pm]	University of Florida TREEO Center	4	4	4	4	4	
724	Health and Safety for Solid Waste Workers [am+pm]	University of Florida TREEO Center	4	4	4	4	4	
725	Solid Waste Workplace Health and Safety Trianing - 4 hours	University of Florida TREEO Center	4	4	4	4	4	
726	IS-00340 Hazardous Materials Management	FEMA Emergency Management Institute	4	4	4	4	4	
727	Is-271.a Anticipating Hazardous Weather & Community Risk, 2nd Edition	FEMA Emergency Management Institute	2	2				
728	Managing Composting Operations	Solid Waste Association of North America [SWANA]	16	16				
729	Personal Protection Equipment (PPE) and Safety Procedures	University of Florida TREEO Center	4	4	4	4	4	
730	Heavy Equipment Safety	University of Florida TREEO Center	4	4	4	4	4	
731	Supervisor Safety Training for Solid Waste Operations Staff	University of Florida TREEO Center	4	4	4	4	4	
732	Permit Required Confined Space Awareness	University of Florida TREEO Center	4	4	4	4	4	
733	8-hour OSHA HazWoper Annual Refresher	University of Florida TREEO Center	4	4	4	4	4	
734	40-Hour OSHA HAZWOPER Training Course	University of Florida TREEO Center	8	8	8	8	4	
735	Hazardous Waste Regulations for Generators	University of Florida TREEO Center	4	4	4	4	4	
736	Exposure to Blooborne and Airborne Pathogens	University of Florida TREEO Center	6	6	6	6	4	
737	Bird and Wildlife Management for Ultiliites	University of Florida TREEO Center	4	4	4	4	2	
738	Beyond 40% - Florida's Pathway to Sustainability"	Solid Waste Association of North America [SWANA] + Recycle Florida Today [RFT]	6	6	6	6	2	
739	Getting Back to Basics with Landfill Gas	University of Florida TREEO Center	8	8			4	
740	Is-632.s Introduction to Debris Operation	Emergency Management Institute	2	2	2	2	2	
741	SI:300 Introduction to Air Pollution Toxicology (1994)	US EPA Air Pollution Training Institute (APTI)	4	4	4	4		
742	4-Hour Spotter Refresher Course for Spotters at Solid Waste Management Facilities in Florida	Kohl Consulting Inc	4	4	4	4	4	
743	Health & Safety Issues for Solid Waste Management Facilities	Kohl Consulting Inc.	8	8	8	8	4	

Course #	Course Title	Course Provider	Landfill	Construction & Demolition Debris	Transfer Station	Materials Recovery Facility	Spotter	
744	The Sense of Smell, Odor, Theory and Odor Control	Kohl Consulting Inc.	4	4	4	4	2	
745	Spotters at Landfills and Transfer Stations: Safety Awareness Review	Kohl Consulting Inc.	4	4	4	4	4	
746	Landfill and Transfer Station Operators: Waste Acceptability and Safety Issues Review	Kohl Consulting Inc.	4	4	4	4	4	
747	Improving Landfill Operations	Kohl Consulting Inc.	4	4				
748	Fires at Landfills and Other Solid Waste Management Facilities	Kohl Consulting Inc.	4	4	4	4	4	
749	Improving Transfer Station Efficiency	Kohl Consulting Inc.			4	4		
750	Landfill Gas Collection and Re-Use	Kohl Consulting Inc.	4	4				
751	Landfills: Past, Present and Future	Kohl Consulting Inc.	4	4			4	
752	Landfills and Transfer Stations: Past, Present and Future	Kohl Consulting Inc.	4	4	4		4	
753	Wet Weather Operations	Kohl Consulting Inc.	4	4	2	2	4	
754	Topics in Solid Waste Management for Landfill Operators, MRF Operators and Transfer Station Operators	Kohl Consulting Inc.	4	4	2	2	2	
755	Wildlife and Plants at Florida Solid Waste Management Facilities	Kohl Consulting Inc.	4	4	4	4	2	
756	Measurement and Improvement of Performance at Solid Waste Management Facilities ("If you Can't Measure it, You Can't Manage It")	Kohl Consulting Inc.	4	4	4	4		
757	CPR / AED	American Safety & Health Institute - American Health Association - American Red Cross	2	2	2	2	2	
758	First Aid	American Safety & Health Institute - American Health Association - American Red Cross	2	2	2	2	2	
759	Refresher Training Course for Experienced Solid Waste Operators - 16hrs	University of Florida TREEO Center	16	16				
760	Refresher Training Course for Experienced Solid Waste Operators - 8hrs	University of Florida TREEO Center	8	8	8	8		
761	Refresher Training Course for Experienced Solid Waste Operators - 4hrs	University of Florida TREEO Center	4	4	4	4	4	
762	U.S. DOT Hazardous Materials/Waste Transportation	University of Florida TREEO Center	6	6	6	6	4	
763	OSHA 10-hour General Industry Safety Outreach Training	Training Consultants Inc.	4	4	4	4	4	
764	NAHMMA 2013 Florida Chapter Annual Conference – General Sessions	North American Hazardous Materials Management Association	10	10	8	8	4	
765	Road-e-o: Heavy Equipment Safety Training	SWANA-FL	4	4	4	4	2	
766	North American Waste-To-Energy Conference NAWTEC 21st Annual	SWANA	4	4		4		
767	Food Waste Recycling Workshop	SWIX & FDEP	5		3		2	
768	Florida Stormwater, Erosion, and Sedimentation Control Inspector Training and Certification Program	FDEP	3	3				

APPENDIX B

RESERVE EQUIPMENT AGREEMENT

Ring Power CAT

Ring Power Corporation
10421 Fern Hill Drive
Riverview, FL 33578

Waste Management Inc. /Southeast Landfill
P.O. Box 627
Balm, FL 33503
Location: Hillsborough County Landfill

2/21/2013

Rental Rates effective through 12/31/13
Waste Management is responsible for maintenance and all damages to rental equipment.
Equipment rental is subject to availability.
Transportation cost quoted upon request.

Make	Model	Description	Day Rate	Week Rate	Month Rate	Cleaning Fee
CAT	D8T	Dozer(w/o waste handling arrangement)	$1,900.00	$5,800.00	$16,400.00	$ 2,400.00
CAT	D6T	Dozer(w/o waste handling arrangement)	$1,100.00	$3,300.00	$ 9,100.00	
CAT	D6N	Dozer(w/o waste handling arrangement)	$ 900.00	$2,700.00	$ 7,400.00	
CAT	D5K	Dozer(w/o waste handling arrangement)	$ 620.00	$1,760.00	$ 5,040.00	
CAT	725	Articulated dump truck 18.8 cyd capacity	$1,100.00	$3,200.00	$ 8,700.00	
CAT	329EL	Hydraulic Excavator 2.5 cyd bucket capacity	$ 900.00	$2,600.00	$ 6,900.00	
CAT	613	Scraper 11 cyd bowl capacity	$1,100.00	$3,200.00	$ 8,700.00	
CAT	12M	Motor Grader 14' mold board	$ 800.00	$2,300.00	$ 6,000.00	
CAT	938K	Wheel Loader 3.05 cyd bucket capacity	$ 700.00	$2,000.00	$ 5,000.00	
CAT	416E	Loader Backhoe	$ 200.00	$ 500.00	$ 1,500.00	
CAT	CS56	Single Drum Roller 84" wide drum	$ 500.00	$1,400.00	$ 3,400.00	

*Plus tax & Insurance

Ring Power guarantees Waste Management a suitable rental machine delivered to Hillsborough County Landfill within 24 hours of their request.

APPENDIX C

RANDOM INSPECTION AND VIOLATION REPORT

SOLID WASTE FACILITY INSPECTION / VIOLATION REPORT

REPORT TYPE: ☐ INSPECTION ☐ VIOLATION ☐ LF RANDOM INSPECTION

LOCATION: ____________ DATE: ________ TIME: ________

DELIVERING COMPANY: FRANCHISE COLLECTOR: ☐ WMI ☐ EB ☐ KR

OTHER: ____________

DRIVER NAME: ____________ VEHICLE #: ________

VEHICLE TYPE ☐ FEL ☐ RO ☐ RL ☐ SL ☐ SEMI ☐ DUMP

OTHER: ____________

CUSTOMER / GENERATOR: ____________ TRANSACTION #: ________

TYPE OF WASTE:

☐ YARD WASTE
☐ C & DD
☐ FURNITURE
☐ CARDBOARD
☐ COMMERCIAL WASTE
☐ OTHER: ____________

☐ INDUSTRIAL
☐ INSULATION
☐ AG WASTE
☐ FIELD PLASTIC
☐ HOUSEHOLD GARBAGE

☐ AUTO PARTS
☐ ASH RESIDUE
☐ ROOFING
☐ METALS

☐ BY PASS WASTE
☐ ANIMAL WASTE
☐ SPECIAL WASTE

TYPE OF VIOLATION: ☐ FACILITY ☐ LOAD ☐ SAFETY ☐ CONTAINER

DETAILS: ____________

DRIVER COMMENTS: ____________

RESULTS: ☐ ACCEPTED ☐ REJECTED ☐ RELOAD ☐ ALREADY IN PIT

INSPECTOR'S SIGNATURE: ____________

ADDITIONAL COMMENTS: ____________

inspect White Copy: Customer Yellow Copy: Inspector Pink Copy: Office

APPENDIX D

NOT USED

APPENDIX E

PHASES I-VI AND CAPACITY EXPANSION AREA FILL SEQUENCING PLANS

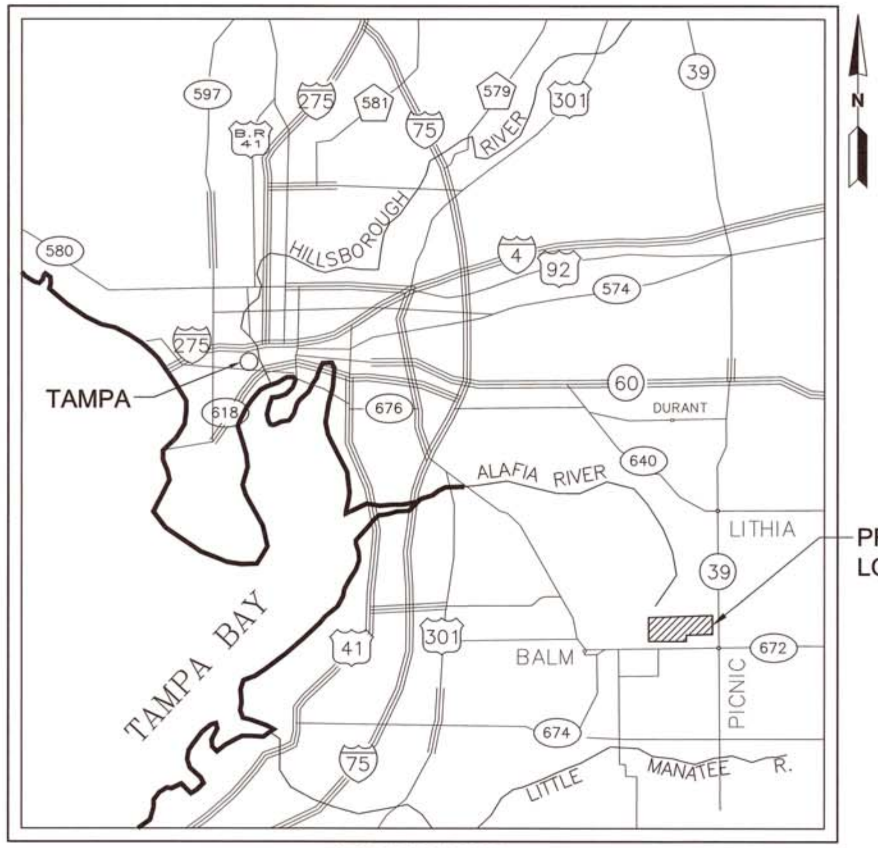
PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY

TAMPA, FLORIDA
APRIL 2017

Hillsborough County Florida

BOARD OF COUNTY COMMISSIONERS

SANDRA L. MURMAN - DISTRICT 1
VICTOR D. CRIST - DISTRICT 2
LESLEY MILLER, JR. - DISTRICT 3
STACY R. WHITE - DISTRICT 4
KEN HAGAN - DISTRICT 5
PAT KEMP - DISTRICT 6
AL HIGGINBOTHAM - DISTRICT 7



LOCATION MAP
NOT TO SCALE

INDEX OF DRAWINGS

SHEET	SHEET TITLE
1	COVER SHEET
2	INDEX, LEGENDS AND GENERAL NOTES
3	FACILITY SITE PLAN AND EXISTING TOPOGRAPHY
4	PHASES I TO IV - LIFTS 13 TO 16
4A	PHASES I - LIFT 13
4B	PHASES VI, IV, I - LIFT 16A
4C	PHASES IV, VI, V - LIFT 17A
5	PHASES V AND VI - LIFT 17
5A	PHASES I, III - LIFTS 14, 15
6	PHASES I TO IV - LIFTS 18 TO 21
7	PHASES V AND VI - LIFT 22
8	PHASES V AND VI - LIFT 23 (FINAL LIFT)
9	SINKHOLE REMEDIATION PLAN
10	LANDFILL CROSS SECTIONS
11	SINKHOLE REMEDIATION CROSS SECTION
12	DETAILS 1
13	DETAILS 2
14	DETAILS 3
15	DETAILS 4

NOTE:

THIS UPDATE TO PHASE I-VI OPERATING SEQUENCE DRAWINGS INCLUDE MODIFICATIONS TO LIFT SEQUENCES 13 THROUGH 17; AS SUCH, SCS ENGINEERS IS ONLY SIGNING AND SEALING SHEETS 4A, 4B, 4C, AND 5A. THE REMAINING LIFT SEQUENCES 18 THROUGH 23 (FINAL LIFT) WILL CONTINUE IN ACCORDANCE WITH THE CURRENTLY FDEP APPROVED OPERATING SEQUENCE DRAWINGS, DATED JUNE 2013, PREPARED, SIGNED AND SEALED BY HDR ENGINEERING, INC.

SCS ENGINEERS
STEARNS, CONRAD AND SCHMIDT
CONSULTING ENGINEERS, INC.
4041 PARK OAKS BLVD., SUITE 100
TAMPA, FLORIDA 33610
PH. (813) 621-0080 FAX. (813) 623-6757
FLORIDA CERTIFICATE OF AUTHORIZATION NO. 00004892
WWW.SCSENGINEERS.COM

SCS PROJECT NO. 09215600.03

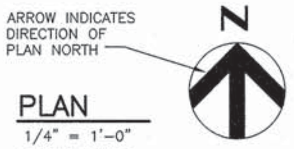
No. 73758
4-7-2017
ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

\\Tam-fs02\tampa\dwg\PROJECT\Hillsborough\09215600.03 - Solid Waste\Engineering Drawings\09215600.03 - C-00.dwg Apr 06, 2017 - 4:33pm Layout Name: COVER SHEET By: 3630ric

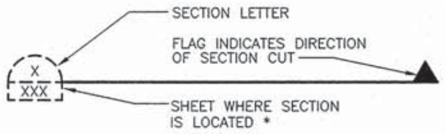
ENGINEERING SYMBOLOGY

⟨9A⟩	LIFT NUMBER
➡	DAILY PROGRESSION
——→	FILL PROGRESSION
∽←	DRAINAGE FLOW DIRECTION
- - - - - - - -	APPROXIMATE PHASE FOOTPRINT
—— · · ——	APPROXIMATE LANDFILL LIMITS
— — — —	APPROXIMATE LIFT LIMITS
— — · —	APPROXIMATE LIMITS OF BORROW AREA
▨	APPROXIMATE TEMPORARY FINAL COVER AREA AFTER EACH LIFT
–·–·–·–·–	EXISTING SWALE
—— — ——	SIDESLOPE DITCH
——140——	EXISTING CONTOUR
——**140**——	PROPOSED CONTOUR
————	PROPOSED DOWNCHUTE
BVC	BEGIN VERTICAL CURVE
CTRD	CENTERED
ERCP	ELLIPTICAL REINFORCED CONCRETE PIPE
EVC	END VERTICAL CURVE
∽150∽	EXIST. CONTOURS
EXP. JT.	EXPANSION JOINT
I.E.	INVERT ELEVATION
LF	LINEAR FEET
LT	LEFT
PC	POINT OF CURVATURE
PI	POINT OF INTERSECTION
PT	POINT OF TANGENCY
PVI	POINT OF VERTICAL INTERSECTION
RT	RIGHT
TYP.	TYPICAL
∽∽	VEGETATION
VC	VERTICAL CURVE
ø	DIAMETER
'	FOOT
"	INCH
Ⓐ	CELL DESIGNATION
(S-21)	EXISTING DRAINAGE STRUCTURE
[TS-4]	TEMPORARY DRAINAGE STRUCTURE
—— — — ——	PROPERTY LINE
>——<	STORMWATER STRUCTURE
CDW →	DIRECTION OF CONDENSATE/ DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)

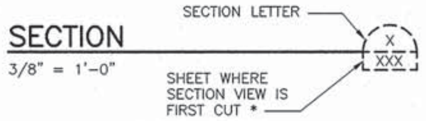
GENERAL SYMBOLOGY



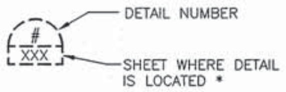
PLAN TITLE



SECTION CUT MARKER



SECTION TITLE

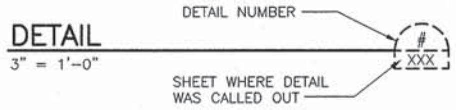


DETAIL MARKER

FOR REFERENCING DETAILS INCLUDED IN DRAWING SET.

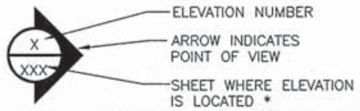
DETAIL MARKER

FOR REFERENCING DETAILS BOUND IN SPECIFICATIONS OR SEPARATE VOLUME.

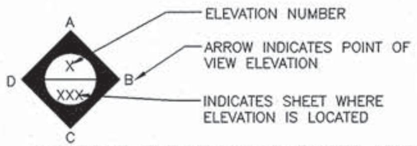


DETAIL TITLE

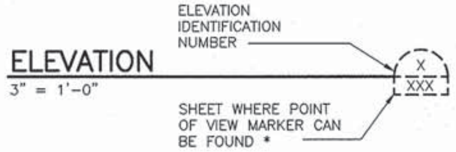
* FOR COMMON DETAILS AND SECTIONS OR DETAILS THAT ARE CUT OR CALLED OUT ON MULTIPLE SHEETS, THE SHEET NUMBER IS REPLACED BY A DASH (–).



SINGLE ELEVATION OR PHOTO MARKER



MULTIPLE ELEVATION OR PHOTO MARKER



ELEVATION TITLE

ABBREVIATIONS

APPROX – APPROXIMATE, APPROXIMATELY
BLDG – BUILDING
BTM – BOTTOM
CB – CATCH BASIN
CM – CONCRETE MONUMENT
CMP – CORRUGATED METAL PIPE
CONC – CONCRETE
CONT – CONTINUOUS
CORR – CORRUGATED
DET – DETAIL
DIA – DIAMETER
DIM – DIMENSION
DWG – DRAWING
EA – EACH
EOL – EDGE OF LINER
ETC – ET CETERA
ENCL – ENCLOSE, ENCLOSURE
EL – ELEVATION
EQUIP – EQUIPMENT
EXIST – EXISTING
FDEP – FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
FDOT – FLORIDA DEPARTMENT OF TRANSPORTATION
FES – FLARED END SECTION
FIN – FINISHED
FM – FORCE MAIN
GALV – GALVANIZED
GCL – GEOSYNTHETIC CLAY LINER
GFFR – GROUT FILLED FIBER REVETMENT
GR – GRADE
GDL – GEOSYNTHETIC DRAINAGE LINER
GM – GAS MONITORING LOCATION
GP – GAS PROBE
HCSWMG – HILLSBOROUGH COUNTY SOLID WASTE MANAGEMENT GROUP
HDPE – HIGH DENSITY POLYETHYLENE
HP – HIGH POINT
ID – INSIDE DIAMETER
IE – INVERT ELEVATION
LF – LINEAL FEET
LFG – LANDFILL GAS
LLDPE – LINEAR LOW DENSITY POLYETHYLENE
LP – LOW POINT
MES – MITRED END SECTION
MAX – MAXIMUM
MH – MANHOLE
MIN – MINIMUM
MISC – MISCELLANEOUS
MSL – (ABOVE) MEAN SEAL LEVEL
MT – MOUNT
MW – GROUNDWATER MONITORING WELL
N/A – NOT APPLICABLE
N/AVAIL – NOT AVAILABLE
NGVD – NATIONAL GEODETIC VERTICAL DATUM
NIC – NOT IN CONTRACT
No – NUMBER
NTS – NOT TO SCALE
OC – ON CENTER
OD – OUTSIDE DIAMETER
OSHA – OCCUPATIONAL SAFETY & HEALTH ADMINISTRATION
PLS – PROFESSIONAL LAND SURVEYOR
PS – PUMP STATION
R – RADIUS
RCP – REINFORCED CONCRETE PIPE
REF – REFERENCE
REQD – REQUIRED
SCH – SCHEDULE
SDR – STANDARD DIMENSION RATIO
SHT – SHEET
SIM – SIMILAR
SS – STAINLESS STEEL
STD – STANDARD
STL – STEEL
STW – STORMWATER MONITORING STATION
TPO – THERMOPLASTIC POLYOLEFIN
TYP – TYPICAL
USC&GS – UNITED STATES COASTAL AND GEODETIC SURVEY
USGS – UNITED STATES GEOLOGICAL SURVEY
WGT – WEIGHT
W.E. – WATER ELEVATION
WWTP – WASTE WATER TREATMENT PLANT

GENERAL NOTES

1. THE EXISTING TOPOGRAPHY, DATED 1/5/2013, WAS OBTAINED FROM DRAWINGS PROVIDED BY PICKETT & ASSOCIATES, INC.
2. THE PROPOSED OPERATING SEQUENCES (LIFTS 13 – 23) ARE BASED ON THE EXISTING TOPOGRAPHY SHOWN ON THE 1/5/2013 AERIAL SURVEY. ACTUAL OPERATING SEQUENCES MAY NEED TO BE MODIFIED IN THE FIELD TO ALLOW FOR LANDFILL SETTLEMENT. REVISED GRADES WILL BE DETERMINED BASED ON THE MAXIMUM DESIGNED 20–FOOT LIFT HEIGHT.
3. THE LANDFILL LINER AND EXISTING DRAINAGE STRUCTURES WERE SURVEYED BY WEIDENER SURVEY AND MAPPING PA ON 2/23/94.

Hillsborough County
Florida

HDR
HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096–193806–001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

INDEX, LEGENDS AND GENERAL NOTES

0 1" 2"

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SCALE	AS SHOWN

DWG No.
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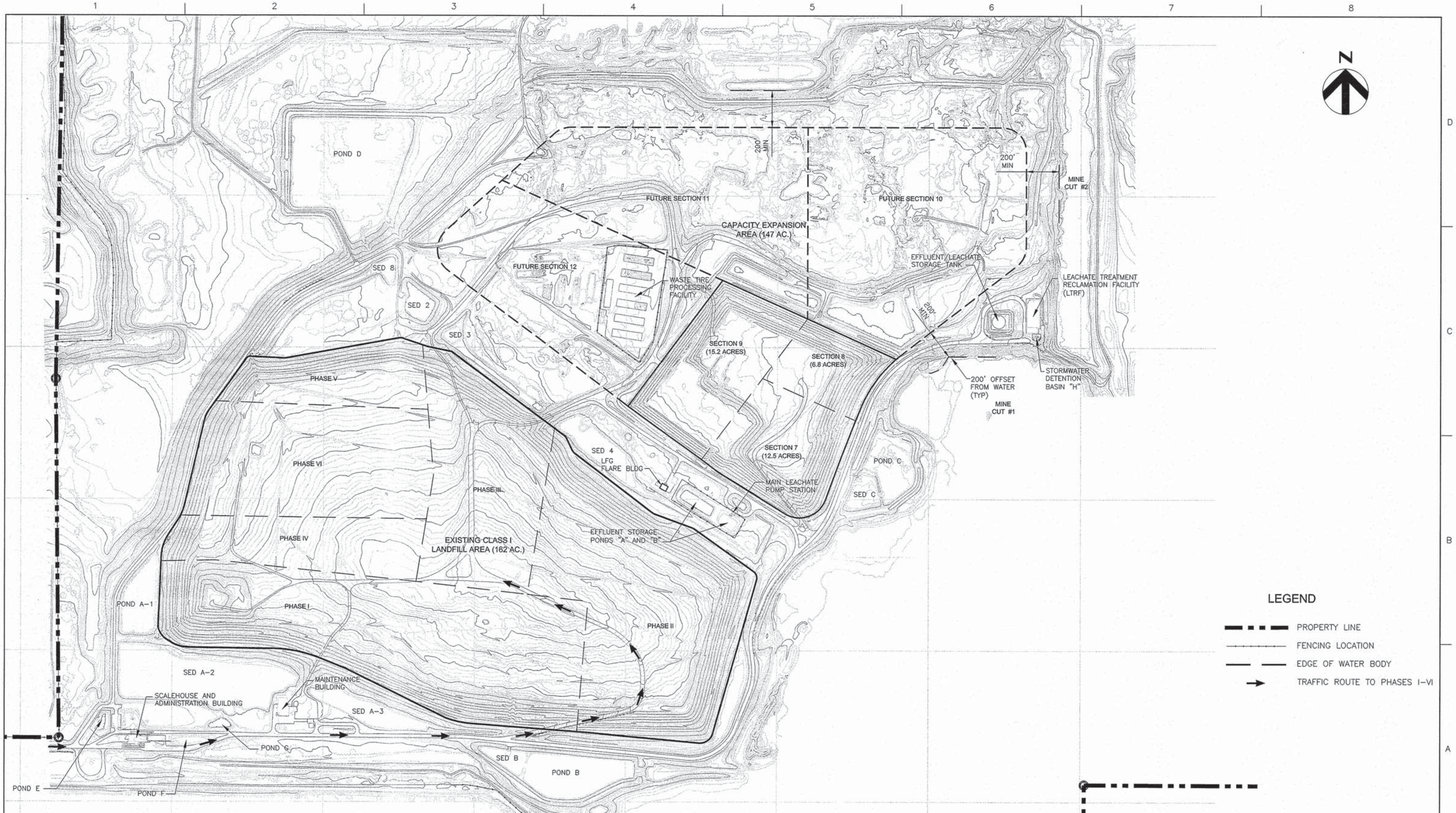

NOTE:
EXISTING TOPOGRAPHY FROM AERIAL PHOTOGRAPHY PROVIDED BY PICKETT AND ASSOCIATES, INC. DATED JANUARY 5, 2013.

Hillsborough County
Florida

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

FACILITY SITE PLAN AND EXISTING TOPOGRAPHY

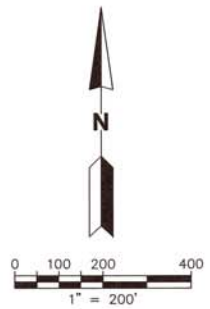
0 1" 2"

FILENAME	00C–03.DWG
SCALE	1" = 300'

DWG No.
3

C:\pwworking\tpa\d0445786\00C-03.dwg, 6/10/2013 1:14:51 PM

\\Tam-fs02\tampa\dwg\PROJECT\Hillsborough\09215600.03 – Solid Waste\Engineering Drawings\09215600.03 – C-4A.dwg Apr 07, 2017 – 9:49am Layout Name: Plot By: 3630rlc



LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
- - - - -	APPROXIMATE PHASE FOOTPRINT
—··—··—	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
	EXISTING SWALE
—— - ——	SIDESLOPE DITCH
—140—	PROPOSED CONTOUR
——	PROPOSED DOWNCHUTE
▭	TEMPORARY FINAL COVER

PROGRESSION:

1. BEGIN LIFT 13 FILL IN CELL "A", ON WEST SIDE OF PHASE I AND CONTINUE EASTERLY ACROSS PHASE I TO THE PHASE II BOUNDARY.
2. EXTEND DOWNCHUTES 1 AND 2 DURING PLACEMENT OF INTERMEDIATE COVER.

ROBERT B. CURTIS
No. 73758
STATE OF FLORIDA
4-7-17

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
△	4/7/17	FDEP SUBMITTAL	RBC
△			
△			
△			
△			
△			

SHEET TITLE
PHASE I - LIFT 13

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL
FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

SCS ENGINEERS
STEARNS, CONRAD AND SCHMIDT
CONSULTING ENGINEERS, INC.
4041 PARK OAKS BLVD., SUITE 100, TAMPA, FL 33610
PH. (813) 621-0080 FAX. (813) 623-6757
FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO. 09215600.03	DWN. BY: RLC	Q/A RVW BY: RBC
DSN. BY: CAR	CHK. BY: RBC	APP. BY: RBC

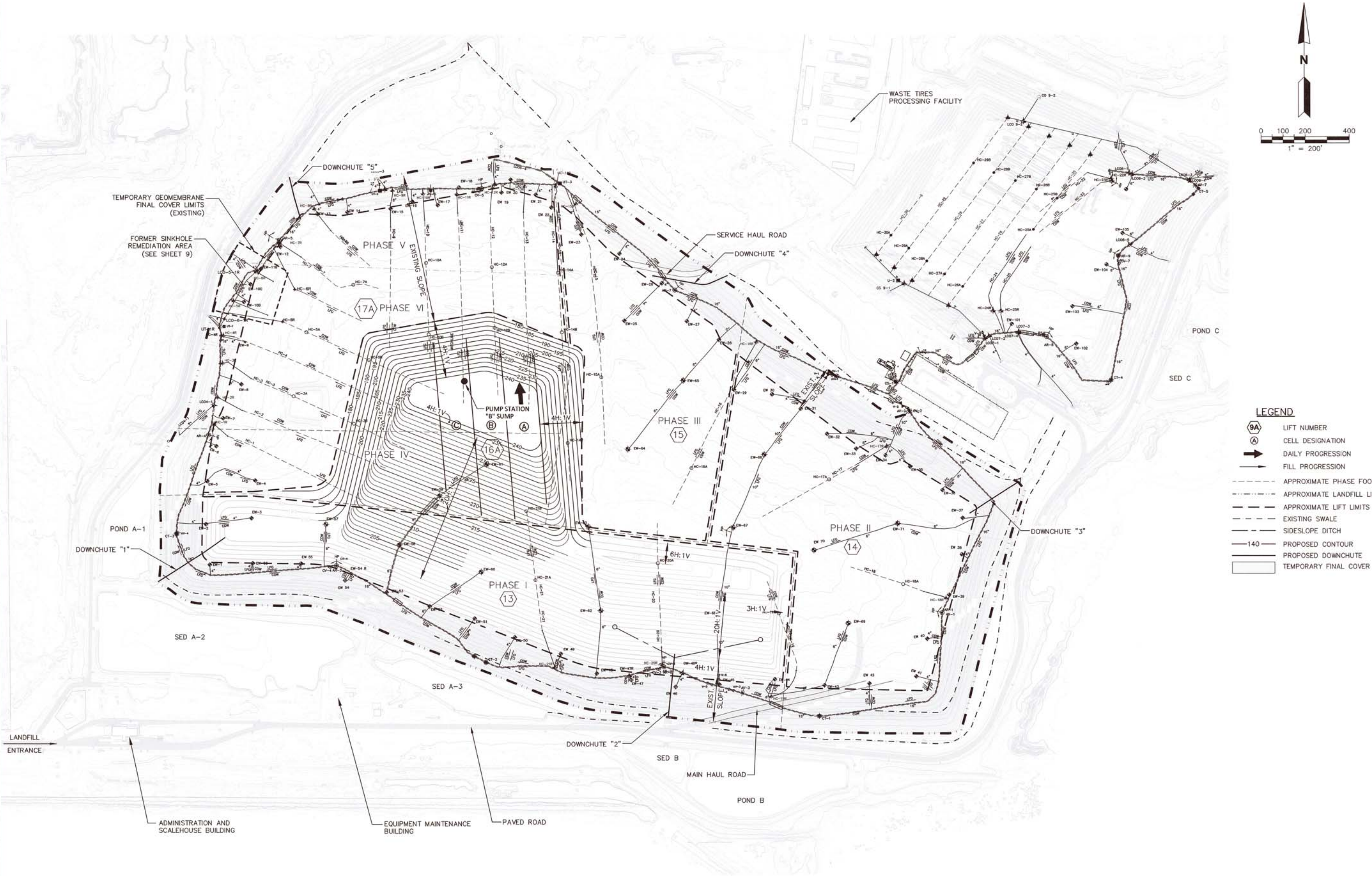
CADD FILE:
09215600.03 – C-4A

DATE:
APRIL 2017

SCALE:
AS SHOWN

DRAWING NO.
4A

SHEET **4A** of **15**



LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
- - - -	APPROXIMATE PHASE FOOTPRINT
— · · —	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
— - —	EXISTING SWALE
——	SIDESLOPE DITCH
—140—	PROPOSED CONTOUR
——	PROPOSED DOWNCHUTE
▭	TEMPORARY FINAL COVER

PROGRESSION:

BEGIN LIFT 16A FILL IN CELL "A", WEST SIDE OF PHASE III AND CONTINUE WEST-SOUTHWEST ACROSS PHASES VI, IV, AND I TO THE NORTHERN LIMIT OF LIFT 13.

\\Tam-fs02\tampa\dwg\PROJECT\Hillsborough\09215600.03 – Solid Waste\Engineering Drawings\09215600.03 – C-4B.dwg Apr 07, 2017 – 9:51am Layout Name: Plot By: 3630rlc

ROBERT B. CURTIS
No. 73758
4-7-17

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
1	4/7/17	FDEP SUBMITTAL	RBC

SHEET TITLE
PHASE VI, IV, I - LIFT 16A

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

SCS ENGINEERS
STEARNS, CONRAD AND SCHMIDT CONSULTING ENGINEERS, INC.
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PH. (813) 621-0080 FAX. (813) 623-6757
FL CERTIFICATE OF AUTHORIZATION NO. 00004892

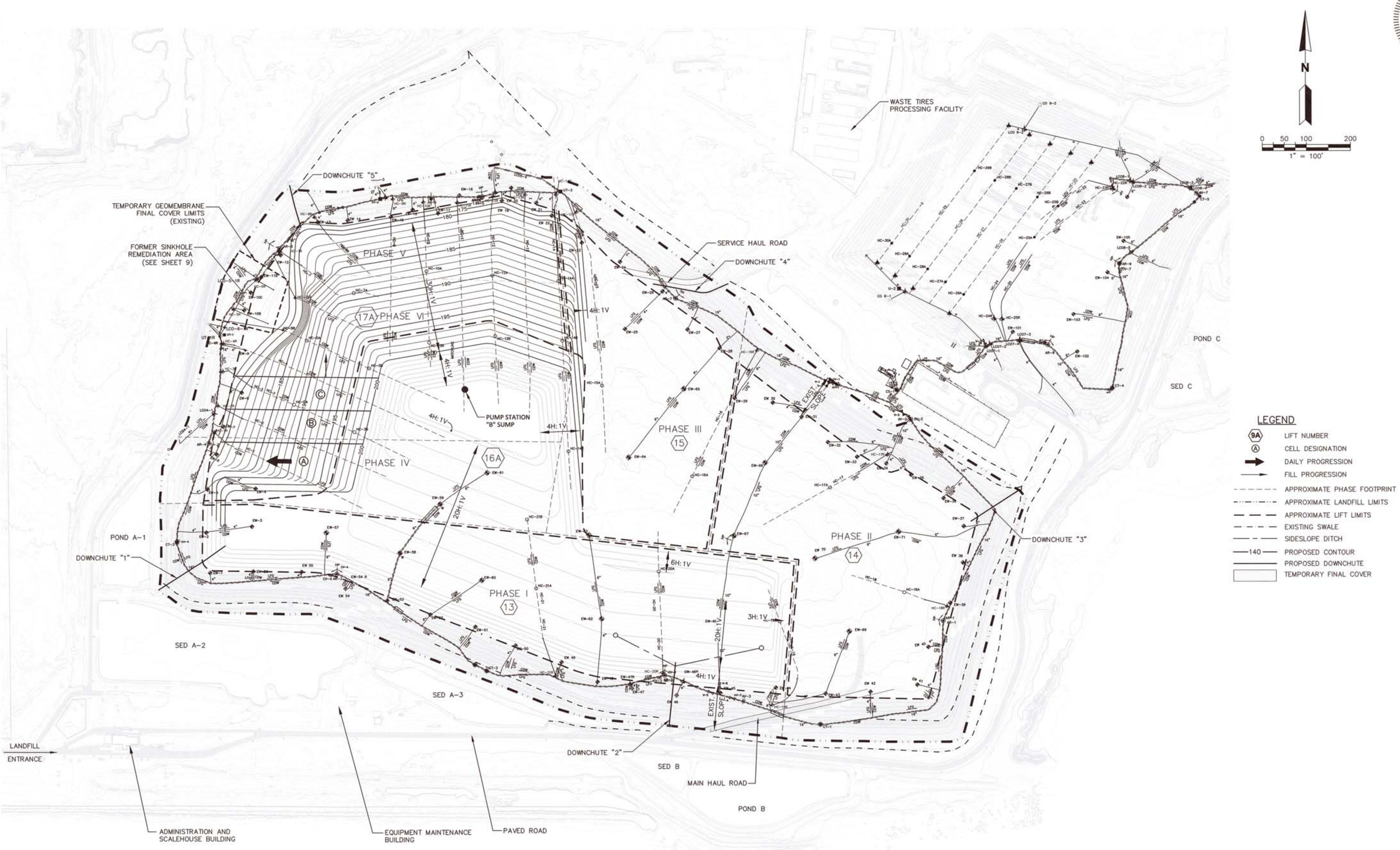
PROJ. NO. 09215600.03	DWN. BY: RLC	Q/A RVW BY: RBC
DSN. BY: CAR	CHK. BY: RBC	APP. BY: RBC

CADD FILE: 09215600.03 – C-4B

DATE: APRIL 2017

SCALE: AS SHOWN

DRAWING NO.
4B
SHEET **4B** of **15**



PROGRESSION:

BEGIN LIFT 17A FILL IN CELL "A", NORTH SIDE OF PHASE I AND CONTINUE CLOCKWISE ACROSS PHASES IV, VI, AND V TO THE WESTERN LIMIT OF PHASE III.

\\Tam-fs02\tampa\dwg\PROJECT\Hillsborough\09215600.03 – Solid Waste\Engineering Drawings\09215600.03 – C-4C.dwg Apr 07, 2017 – 9:46am Layout Name: Plot By: 3630rlc

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
1	4/7/17	FDEP SUBMITTAL	RBC

SHEET TITLE
PHASE IV, VI, V - LIFT 17A

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL
FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

SCS ENGINEERS
STEARNS, CONRAD AND SCHMIDT
CONSULTING ENGINEERS, INC.
4041 PARK OAKS BLVD., SUITE 100, TAMPA, FL 33610
PH. (813) 621-0080 FAX. (813) 623-6757
FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO. 09215600.03
DWN. BY: RLC
Q/A RVW BY: RBC
DSN. BY: CAR
CHK. BY: RBC
APP. BY: RBC

CADD FILE: 09215600.03 – C-4C

DATE: APRIL 2017

SCALE: AS SHOWN

DRAWING NO. **4C**

SHEET **4C** of **15**

PROGRESSION:

BEGIN LIFT 14 FILL IN CELL "A", WEST SIDE OF PHASE II AND CONTINUE COUNTERCLOCKWISE TO THE PHASE III BOUNDARY. BEGIN LIFT 15 ON THE NORTHERN SIDE SLOPES AND PREPARE THE PHASE III SIDE SLOPES AND THE SERVICE LANE ROAD FOR CLOSURE. THEN CONTINUE LIFT 15 FILL IN CELL "A", EAST SIDE PHASE III AND CONTINUE WEST ACROSS PHASE III TO THE WEST SIDE OF PHASE III.

ROBERT B. CURTIS
LICENSE No. 73758
STATE OF FLORIDA
PROFESSIONAL ENGINEER
4-7-17
ROBERT B. CURTIS, P.E.
LICENSE NO. 73758

REV	DATE	DESCRIPTION	CHK. BY
1	4/7/17	FDEP SUBMITTAL	RBC

SHEET TITLE
PHASES I, II, III - LIFTS 14, 15

PROJECT TITLE
SOUTHEAST COUNTY LANDFILL
FILL SEQUENCE REVISION

CLIENT
HILLSBOROUGH COUNTY
PUBLIC WORKS DEPARTMENT
SOLID WASTE MANAGEMENT DIVISION
TAMPA, FL 33619

SCS ENGINEERS
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CONSULTING ENGINEERS, INC.
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PH. (813) 621-0080 FAX. (813) 623-6757
FL CERTIFICATE OF AUTHORIZATION NO. 00004892

PROJ. NO.	DWN. BY:	Q/A RVW BY:
09215600.03	RLC	RBC
DSN. BY: CAR	CHK. BY: RBC	APP. BY: RBC

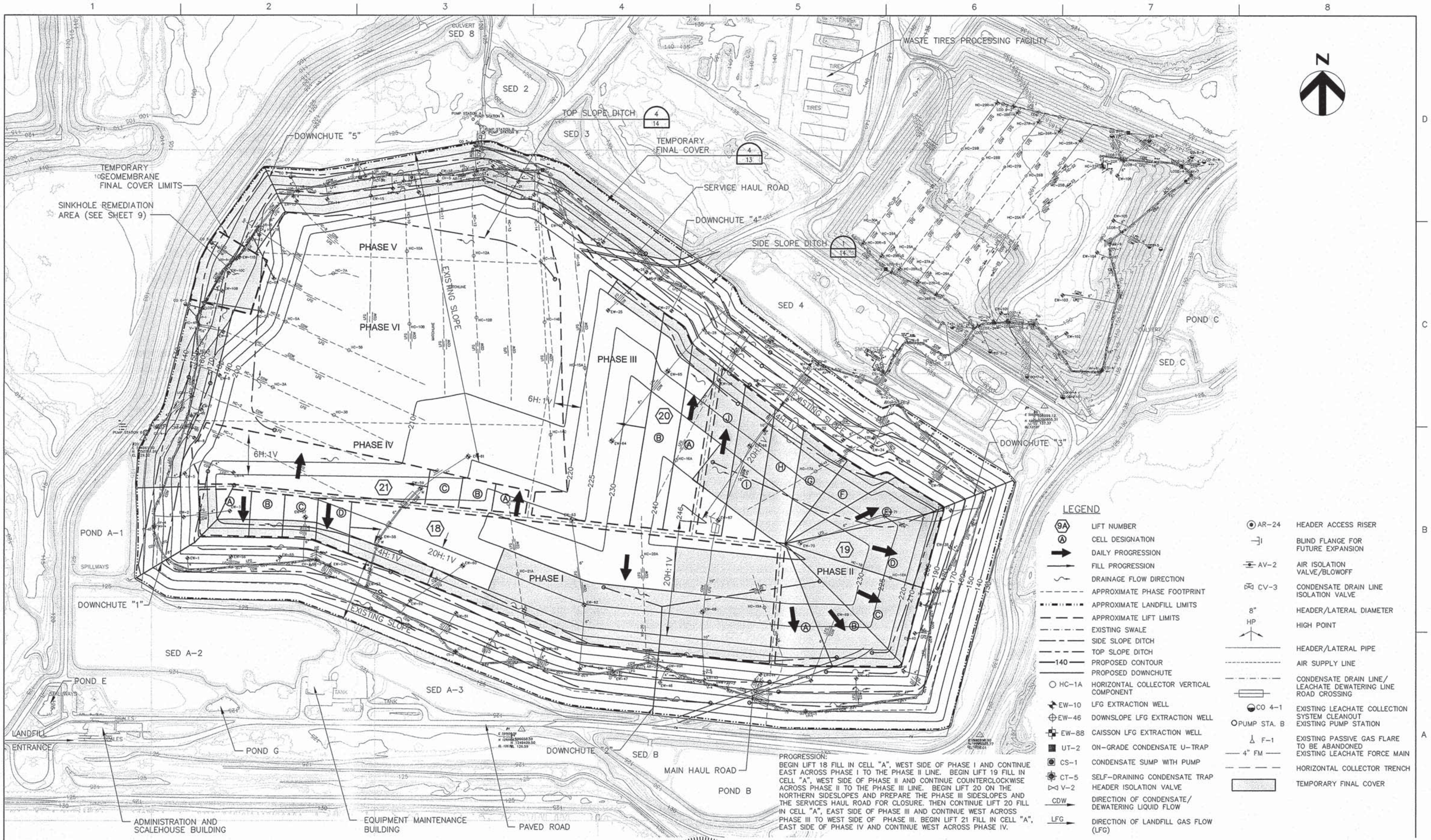
CADD FILE:
09215600.03 – C-5A

DATE:
APRIL 2017

SCALE:
AS SHOWN

DRAWING NO.
5A

SHEET **5A** of **15**



PROGRESSION:
BEGIN LIFT 18 FILL IN CELL "A", WEST SIDE OF PHASE I AND CONTINUE EAST ACROSS PHASE I TO THE PHASE II LINE. BEGIN LIFT 19 FILL IN CELL "A", WEST SIDE OF PHASE II AND CONTINUE COUNTERCLOCKWISE ACROSS PHASE II TO THE PHASE III LINE. BEGIN LIFT 20 ON THE NORTHERN SIDESLOPES AND PREPARE THE PHASE III SIDESLOPES AND THE SERVICES HAUL ROAD FOR CLOSURE. THEN CONTINUE LIFT 20 FILL IN CELL "A", EAST SIDE OF PHASE III AND CONTINUE WEST ACROSS PHASE III TO WEST SIDE OF PHASE III. BEGIN LIFT 21 FILL IN CELL "A", EAST SIDE OF PHASE IV AND CONTINUE WEST ACROSS PHASE IV.

LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
∿	DRAINAGE FLOW DIRECTION
- - - - - -	APPROXIMATE PHASE FOOTPRINT
··—··—··	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
—·—·—	EXISTING SWALE
—— - ——	SIDE SLOPE DITCH
— - - —	TOP SLOPE DITCH
—140—	PROPOSED CONTOUR
———	PROPOSED DOWNCHUTE
○ HC-1A	HORIZONTAL COLLECTOR VERTICAL COMPONENT
EW-10	LFG EXTRACTION WELL
⊕ EW-46	DOWNSLOPE LFG EXTRACTION WELL
EW-88	CAISSON LFG EXTRACTION WELL
■ UT-2	ON-GRADE CONDENSATE U-TRAP
◙ CS-1	CONDENSATE SUMP WITH PUMP
CT-5	SELF-DRAINING CONDENSATE TRAP
⋈ V-2	HEADER ISOLATION VALVE
CDW →	DIRECTION OF CONDENSATE/ DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)
◉ AR-24	HEADER ACCESS RISER
⊣I	BLIND FLANGE FOR FUTURE EXPANSION
AV-2	AIR ISOLATION VALVE/BLOWOFF
CV-3	CONDENSATE DRAIN LINE ISOLATION VALVE
8"	HEADER/LATERAL DIAMETER
HP	HIGH POINT
———	HEADER/LATERAL PIPE
------------	AIR SUPPLY LINE
—·—·—·—	CONDENSATE DRAIN LINE/ LEACHATE DEWATERING LINE
▭	ROAD CROSSING
◒ CO 4-1	EXISTING LEACHATE COLLECTION SYSTEM CLEANOUT
○ PUMP STA. B	EXISTING PUMP STATION
F-1	EXISTING PASSIVE GAS FLARE TO BE ABANDONED
— 4" FM —	EXISTING LEACHATE FORCE MAIN
— — — —	HORIZONTAL COLLECTOR TRENCH
▨	TEMPORARY FINAL COVER

Hillsborough County
Florida

HDR

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES I TO IV - LIFTS 18 TO 21

0 1" 2"

FILENAME	00C-06.DWG	DWG No.
SCALE	1"=200'	6

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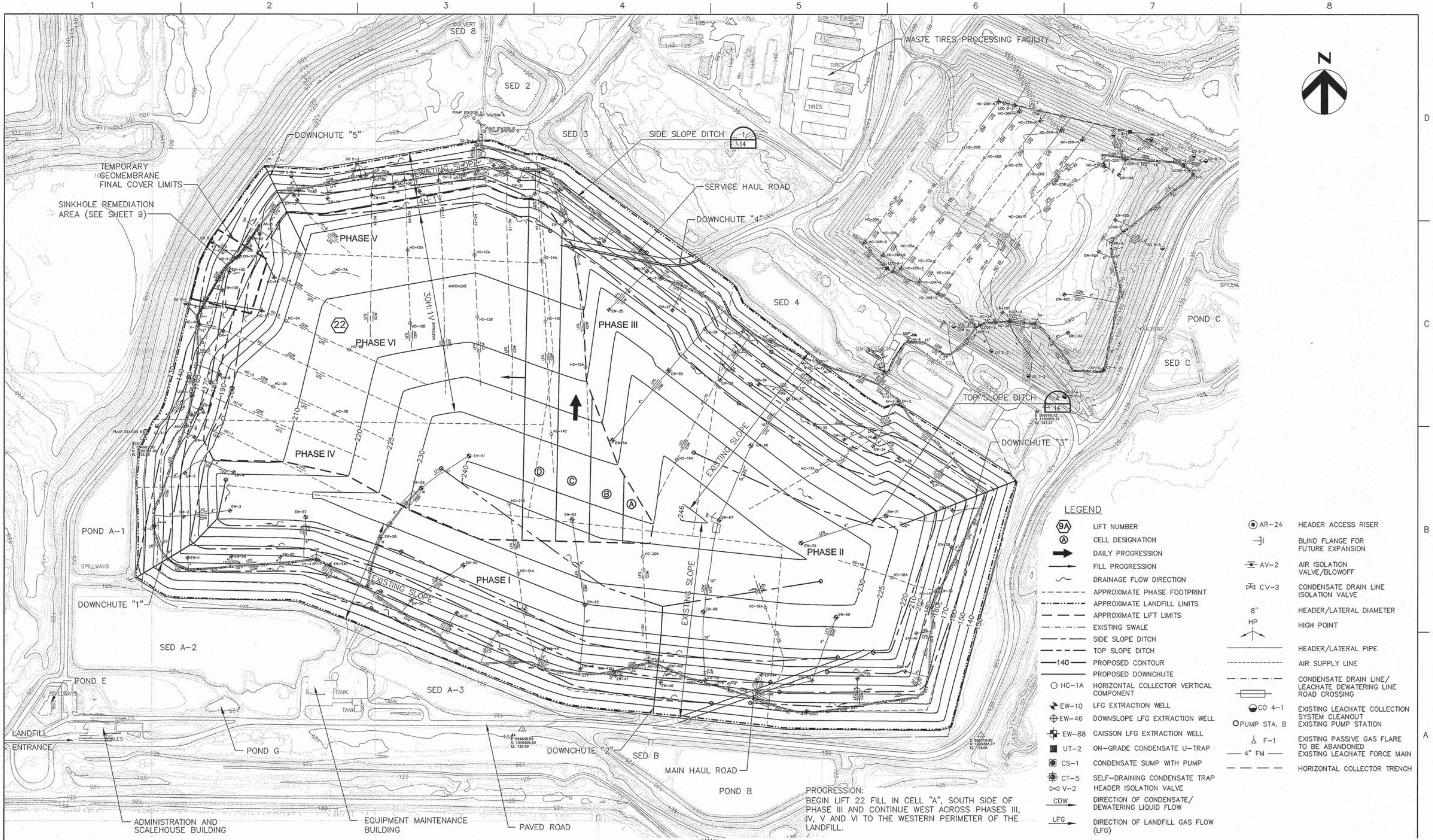

PROGRESSION:
BEGIN LIFT 22 FILL IN CELL "A", SOUTH SIDE OF PHASE III AND CONTINUE WEST ACROSS PHASES III, IV, V AND VI TO THE WESTERN PERIMETER OF THE LANDFILL.

LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
∼	DRAINAGE FLOW DIRECTION
- - - - - -	APPROXIMATE PHASE FOOTPRINT
-··-··-	APPROXIMATE LANDFILL LIMITS
— — —	APPROXIMATE LIFT LIMITS
-·-·-·-	EXISTING SWALE
—— - ——	SIDE SLOPE DITCH
—— - - ——	TOP SLOPE DITCH
—140—	PROPOSED CONTOUR
———	PROPOSED DOWNCHUTE
○ HC-1A	HORIZONTAL COLLECTOR VERTICAL COMPONENT
EW-10	LFG EXTRACTION WELL
⊕ EW-46	DOWNSLOPE LFG EXTRACTION WELL
EW-88	CAISSON LFG EXTRACTION WELL
■ UT-2	ON-GRADE CONDENSATE U-TRAP
◉ CS-1	CONDENSATE SUMP WITH PUMP
CT-5	SELF-DRAINING CONDENSATE TRAP
⋈ V-2	HEADER ISOLATION VALVE
CDW →	DIRECTION OF CONDENSATE/ DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)
◉ AR-24	HEADER ACCESS RISER
	BLIND FLANGE FOR FUTURE EXPANSION
AV-2	AIR ISOLATION VALVE/BLOWOFF
CV-3	CONDENSATE DRAIN LINE ISOLATION VALVE
8"	HEADER/LATERAL DIAMETER
HP	HIGH POINT
———	HEADER/LATERAL PIPE
- - - - - - -	AIR SUPPLY LINE
—-—-—-—	CONDENSATE DRAIN LINE/ LEACHATE DEWATERING LINE
	ROAD CROSSING
CO 4-1	EXISTING LEACHATE COLLECTION SYSTEM CLEANOUT
○ PUMP STA. B	EXISTING PUMP STATION
F-1	EXISTING PASSIVE GAS FLARE TO BE ABANDONED
— 4" FM —	EXISTING LEACHATE FORCE MAIN
— — — —	HORIZONTAL COLLECTOR TRENCH

Hillsborough County
Florida

HDR
HDR Engineering, Inc.
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Tampa, FL 33609-3444
HDR CA# 4213

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	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES V AND VI - LIFT 22

0 1" 2"

FILENAME	00C-07.DWG
SCALE	1"=200'

DWG No.
7

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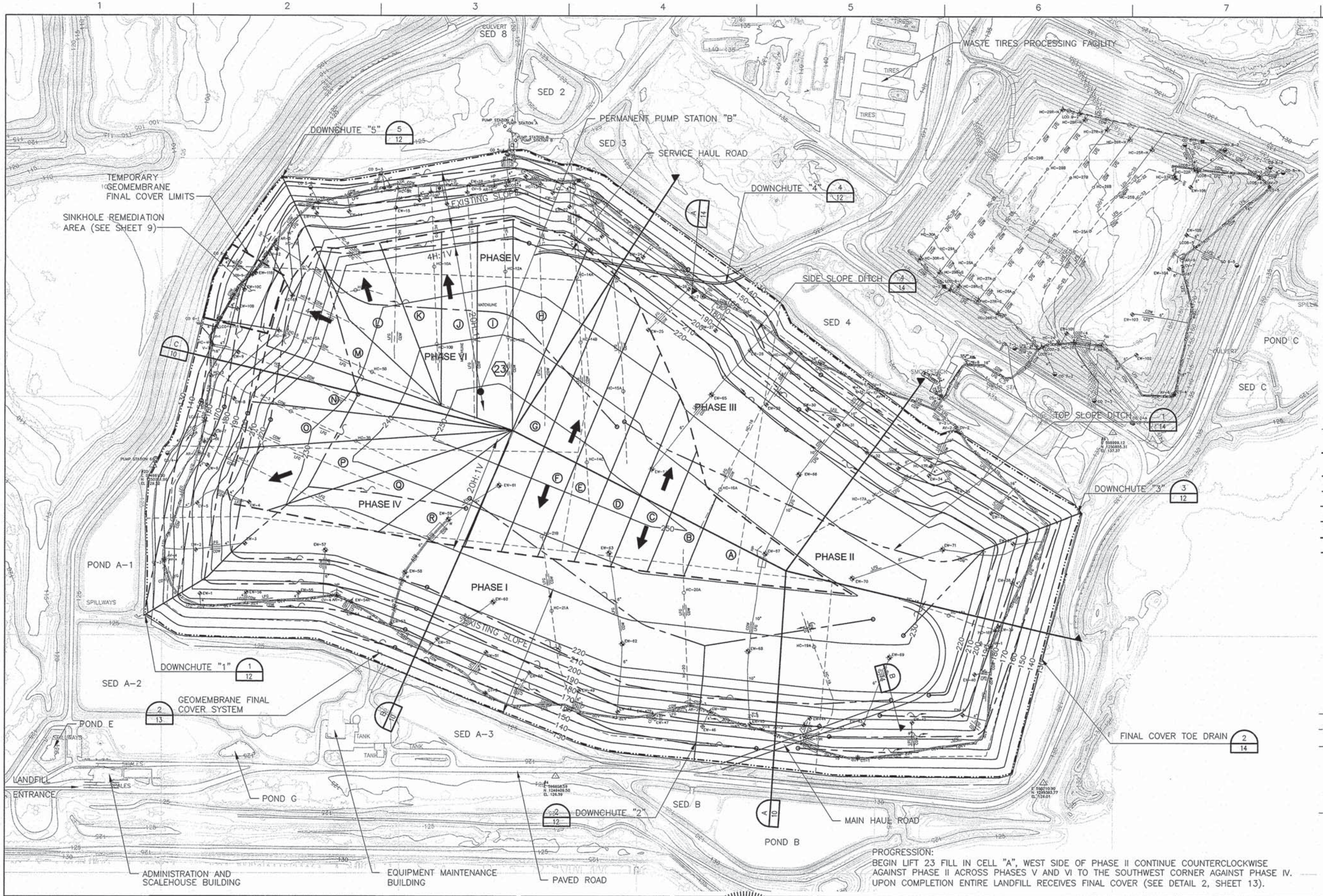

PROGRESSION:
BEGIN LIFT 23 FILL IN CELL "A", WEST SIDE OF PHASE II CONTINUE COUNTERCLOCKWISE AGAINST PHASE II ACROSS PHASES V AND VI TO THE SOUTHWEST CORNER AGAINST PHASE IV. UPON COMPLETION ENTIRE LANDFILL RECEIVES FINAL COVER (SEE DETAIL 2, SHEET 13).

LEGEND

9A	LIFT NUMBER
Ⓐ	CELL DESIGNATION
➡	DAILY PROGRESSION
→	FILL PROGRESSION
~→	DRAINAGE FLOW DIRECTION
- - - - - - - -	APPROXIMATE PHASE FOOTPRINT
—··—··—	APPROXIMATE LANDFILL LIMITS
— — — —	APPROXIMATE LIFT LIMITS
—·—·—·—	EXISTING SWALE
——— – ———	SIDE SLOPE DITCH
——— - - - ———	TOP SLOPE DITCH
—140—	PROPOSED CONTOUR
————	PROPOSED DOWNCHUTE
EW-10	LFG EXTRACTION WELL
⊕EW-46	DOWNSLOPE LFG EXTRACTION WELL
EW-88	CAISSON LFG EXTRACTION WELL
■ UT-2	ON-GRADE CONDENSATE U-TRAP
CS-1	CONDENSATE SUMP WITH PUMP
CT-5	SELF-DRAINING CONDENSATE TRAP
⊙ AR-24	HEADER ACCESS RISER
————	HEADER/LATERAL PIPE
------------	AIR SUPPLY LINE
—·—·—·—	CONDENSATE DRAIN LINE/LEACHATE DEWATERING LINE
CO 4-1	EXISTING LEACHATE COLLECTION SYSTEM CLEANOUT
— 4" FM —	EXISTING LEACHATE FORCE MAIN
CDW →	DIRECTION OF CONDENSATE /DEWATERING LIQUID FLOW
LFG →	DIRECTION OF LANDFILL GAS FLOW (LFG)

Hillsborough County
Florida

HDR Engineering, Inc.
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HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

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PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

PHASES V AND VI - LIFT 23 (FINAL LIFT)

0 1" 2"

FILENAME	00C-08.DWG
SCALE	1"=200'

DWG No.
8

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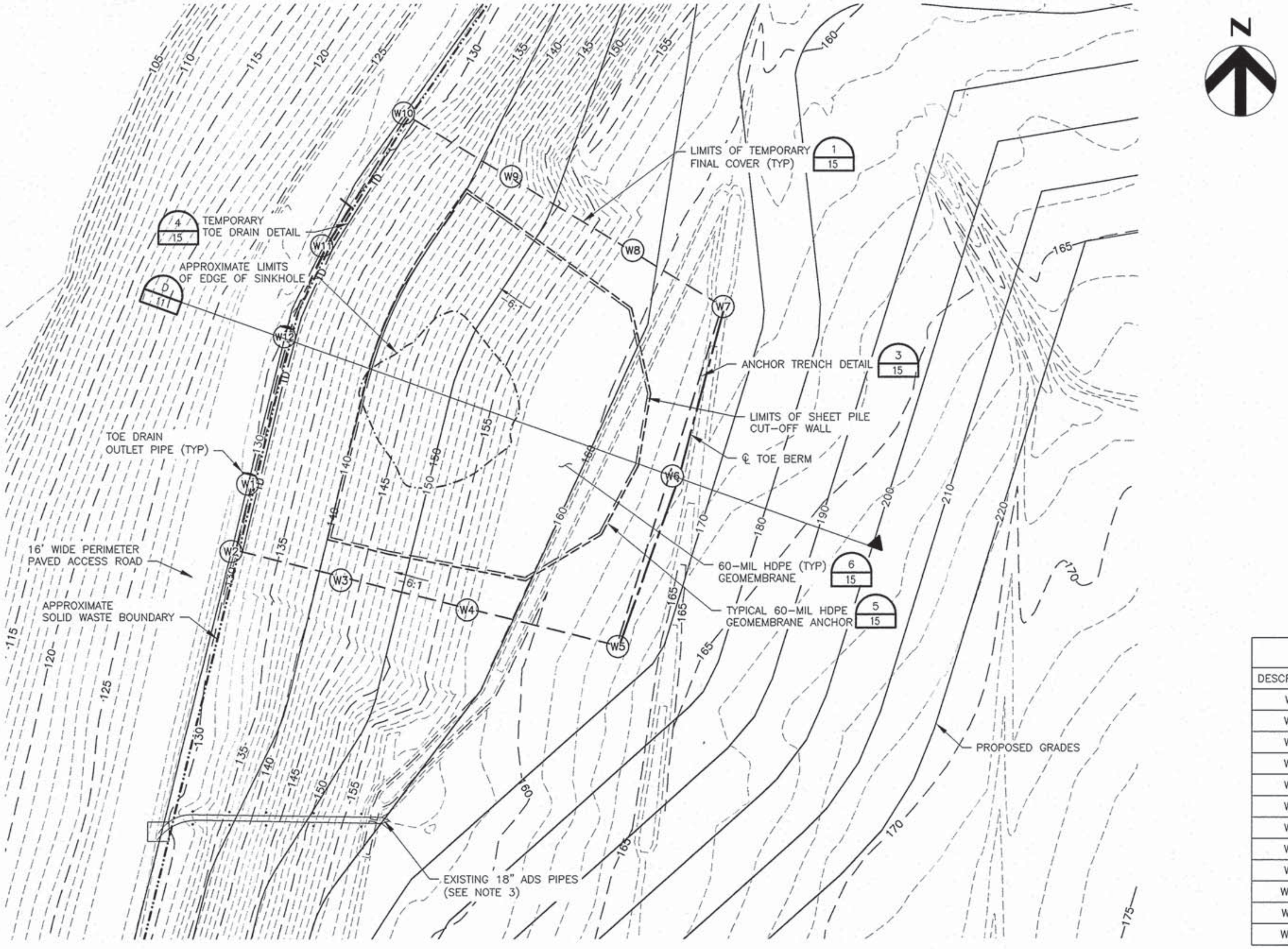

LIMITS OF FINAL COVER			
DESCRIPTION	NORTHING	EASTING	ELEVATION
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W2	1,251,332.6	595,169.56	129.21
W3	1,251,312.3	595,247.07	142.69
W4	1,251,291.4	595,336.55	158.01
W5	1,251,265.5	595,442.88	162.72
W6	1,251,385.8	595,481.43	162.31
W7	1,251,505.8	595,517.36	162.22
W8	1,251,545.4	595,453.35	159.23
W9	1,251,597.7	595,367.51	145.05
W10	1,251,642.4	595,291.38	126.82
W11	1,251,548.6	595,233.61	128.73
W12	1,251,484.4	595,207.36	129.31

LEGEND:

(W1) (W2)	WOODEN MARKER POST (LOCATIONS ON TABLE THIS SHEET)
—— TD ——	TOE DRAIN
— — 155 — —	EXISTING CONTOUR
——155——	PROPOSED CONTOUR

NOTES:

1. EXISTING TOPOGRAPHY PROVIDED BY PICKETT AND ASSOCIATES, INC. FROM AERIAL PHOTOGRAPHY DATED JANUARY 5, 2013.
2. LFG SYSTEM NOT SHOWN FOR CLARITY OF DRAWING.
3. EXISTING STORMWATER PIPES TO BE RELOCATED PER STORMWATER PLANS.

Hillsborough County
Florida

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	6/2013	FDEP SUBMITTAL

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SINKHOLE REMEDIATION PLAN

0 1" 2"

FILENAME	00C-09.dwg	DWG No.	9
SCALE	1"=50'		

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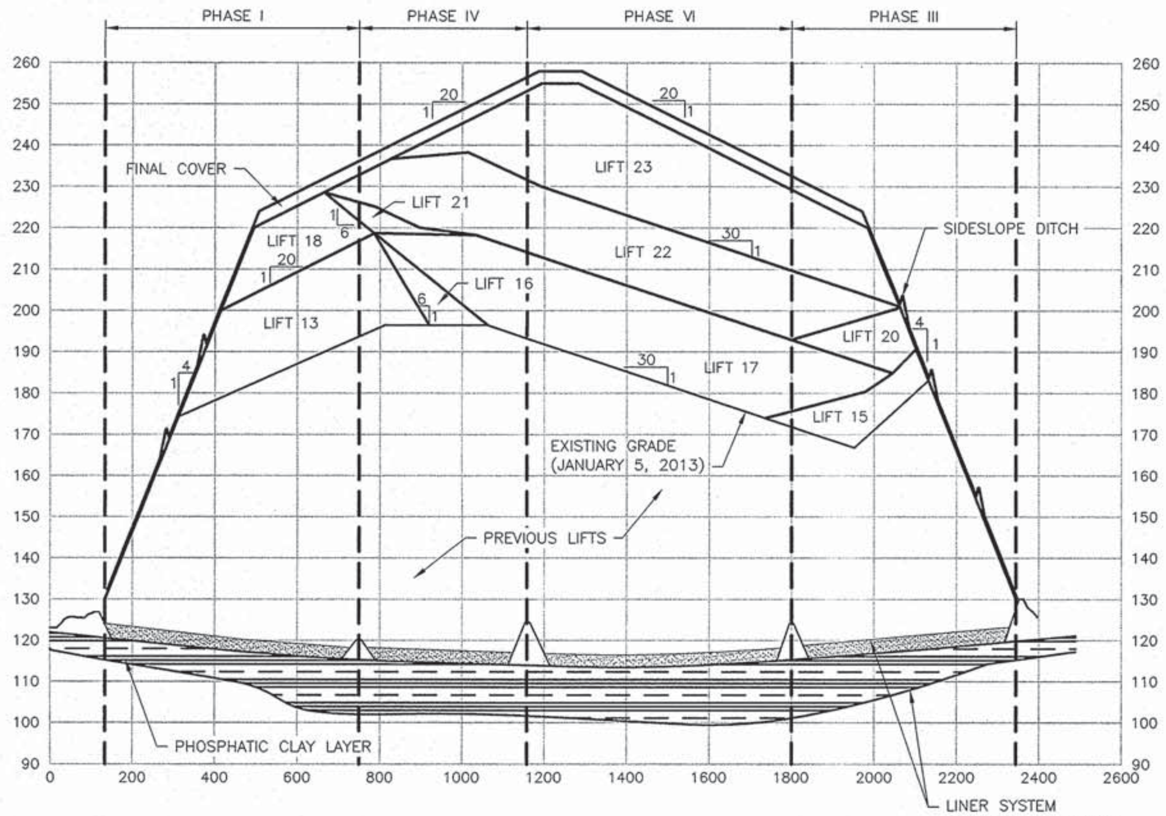
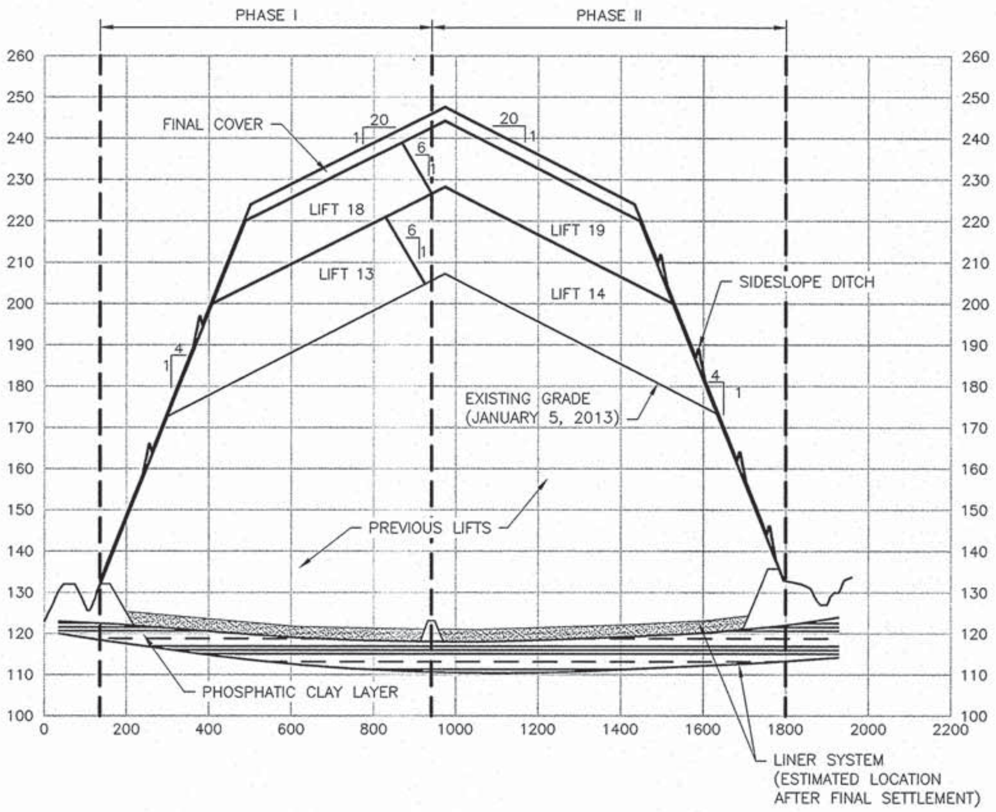
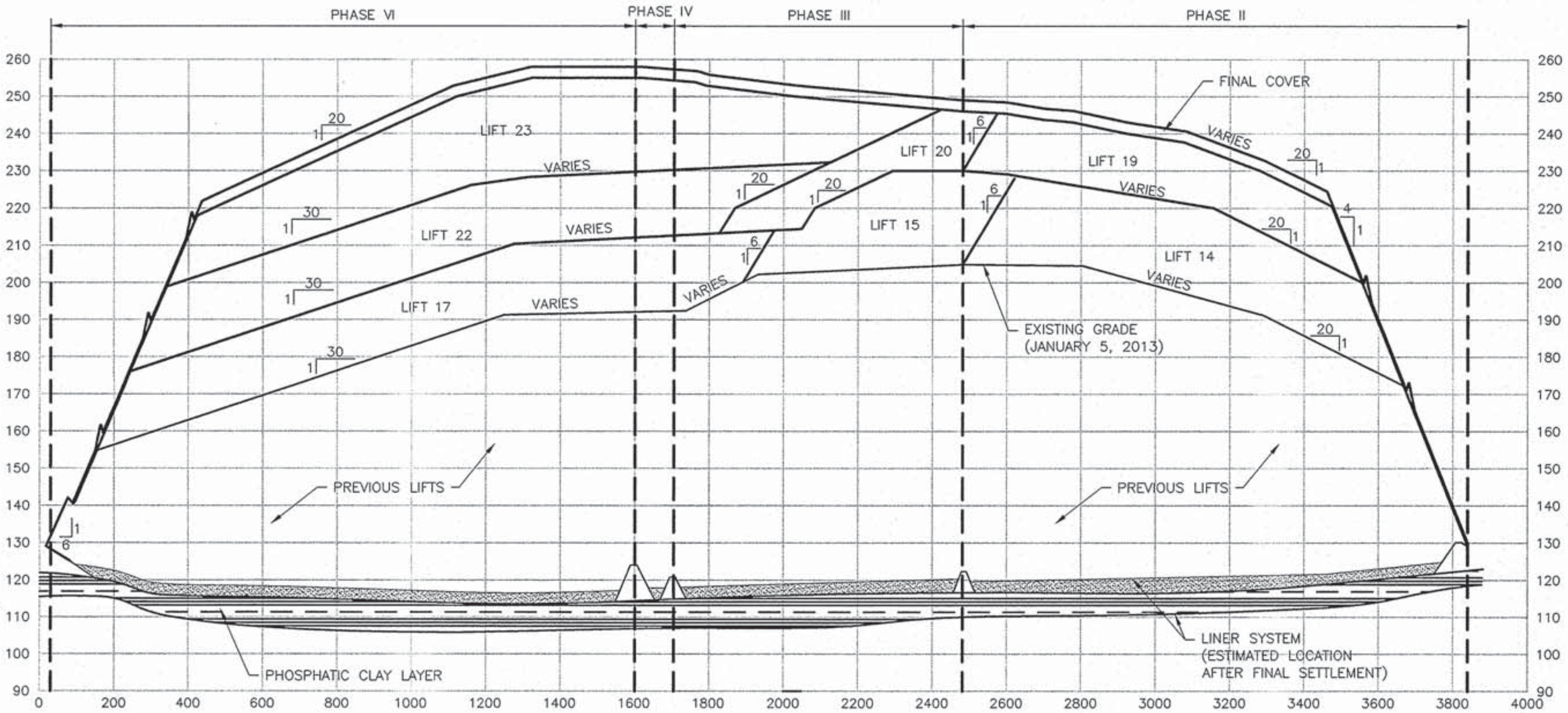


LANDFILL CROSS SECTION B/8



LANDFILL CROSS SECTION A/8



LANDFILL CROSS SECTION C/8

LEGEND:

▨ LINER SYSTEM

☰ PHOSPHATIC CLAY LAYER

NOTE:
SOME SLOPES AND LIFT SIZE VARY DUE TO SECTION CUT ORIENTATION. SEE SHEET C-08 FOR SECTION LOCATIONS.

Hillsborough County
Florida

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5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

ROBERT B. CURTIS
No. 73758
6/7/13

ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

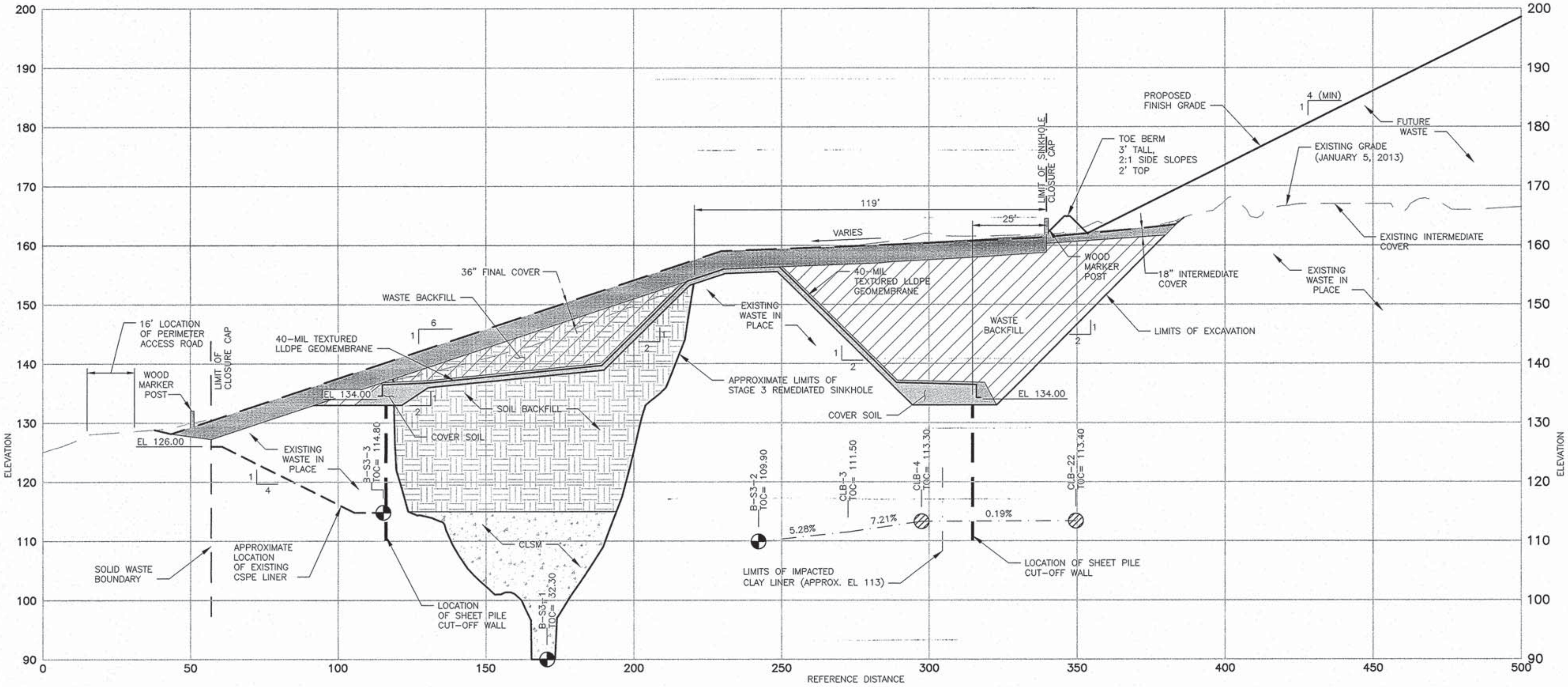
LANDFILL CROSS SECTIONS

0 1" 2"

FILENAME	00C-10.DWG
SCALE	NOT TO SCALE

DWG No.
10

C:\pwworking\tpa\d0445786\00C-10.dwg, 6/10/2013 1:29:04 PM



SECTION

SCALE: 1"=20' (HORIZONTAL)
1"=10' (VERTICAL)

D
9

LEGEND:

●	GEOTECHNICAL BORING LOCATION (SDII)
▨	GEOTECHNICAL BORING LOCATION (TIERRA)
TOC =	TOP OF CLAY ELEVATION
CLSM =	CONTROLLED LOW STRENGTH MATERIAL

Hillsborough County
Florida

HDR

HDR Engineering, Inc.
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HDR CA# 4213

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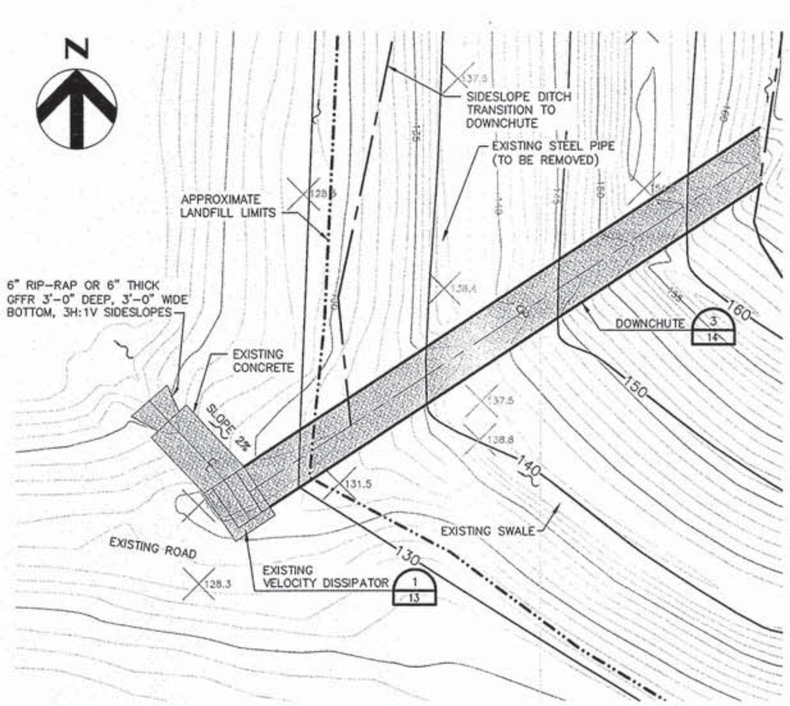
PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SINKHOLE REMEDIATION CROSS SECTION

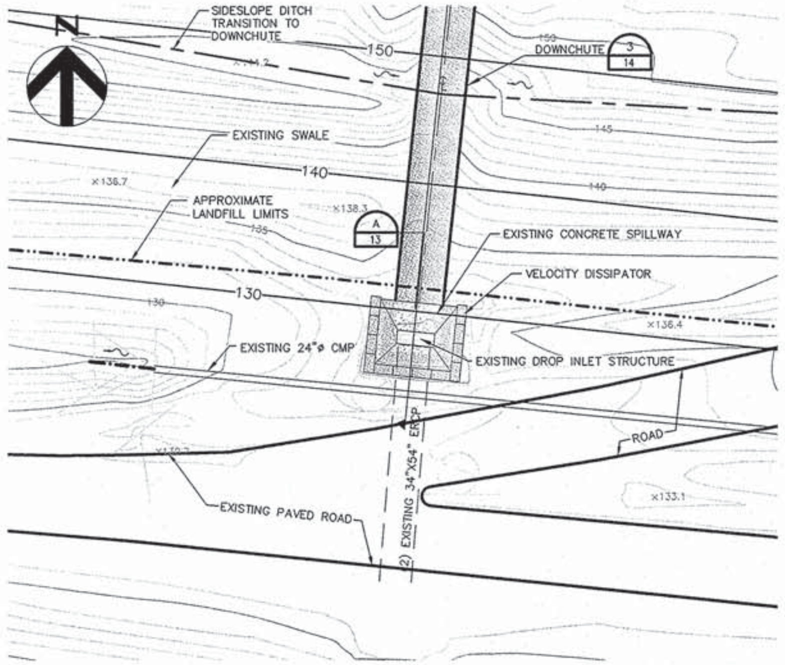
0 1" 2"

FILENAME	00C-11.DWG
SCALE	AS SHOWN

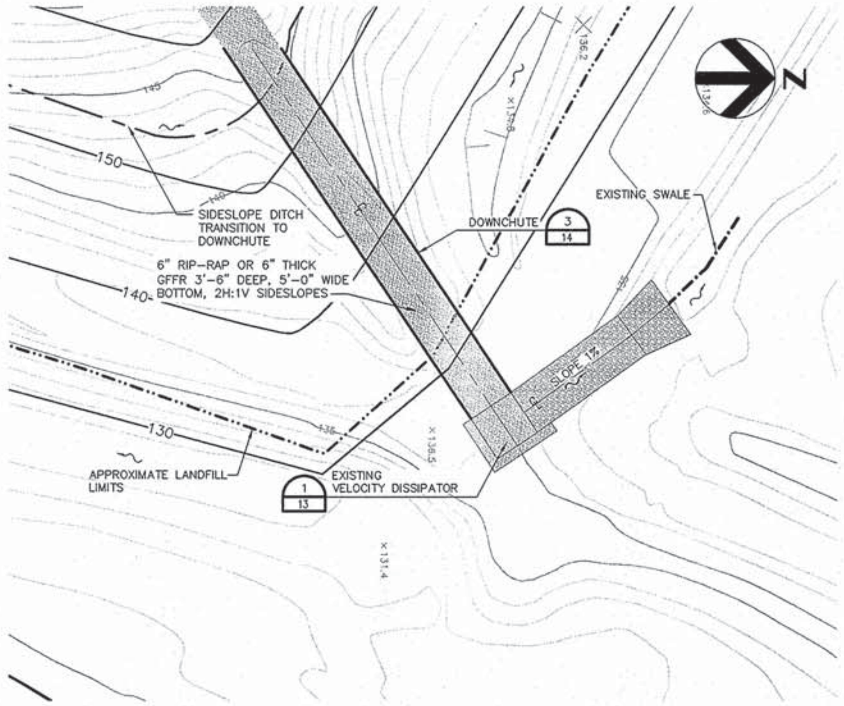
DWG No.
11



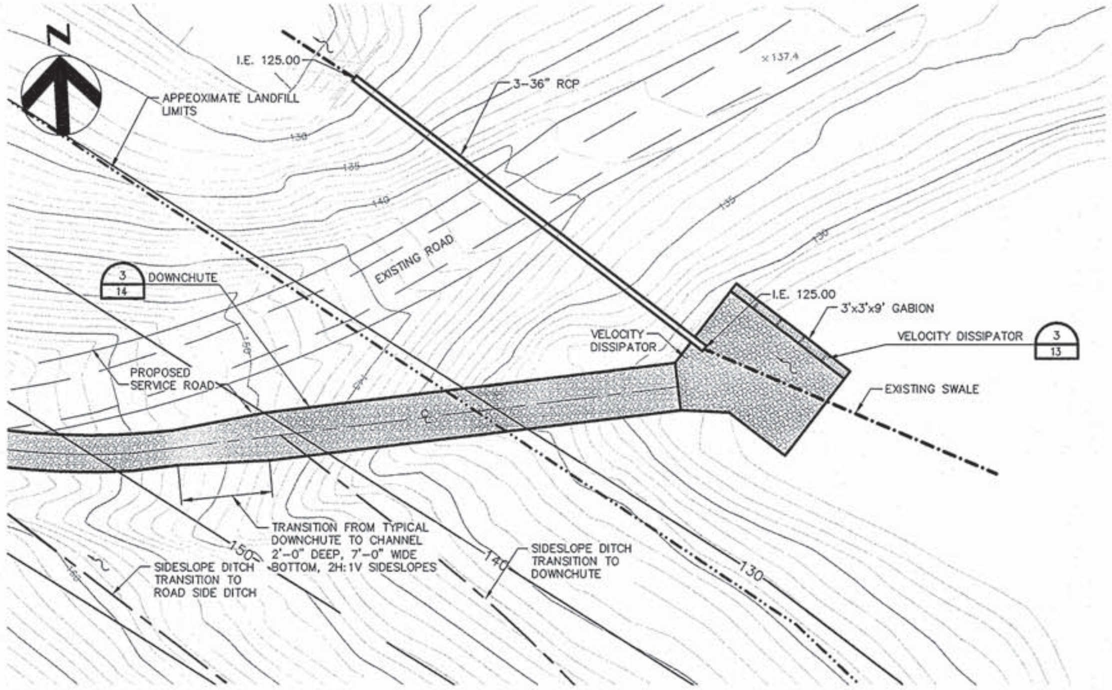
DOWNCHUTE "1" – PLAN DETAIL 1/8



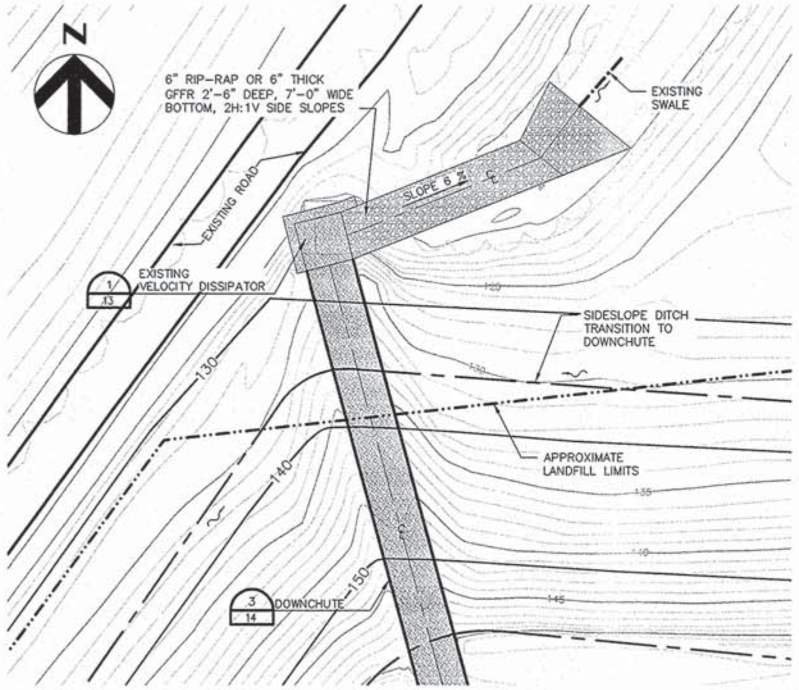
DOWNCHUTE "2" – PLAN DETAIL 2/8



DOWNCHUTE "3" – PLAN DETAIL 3/8



DOWNCHUTE "4" – PLAN DETAIL 4/8



DOWNCHUTE "5" – PLAN DETAIL 5/8

Hillsborough County Florida

HDR

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5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

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	6/2013	FDEP SUBMITTAL

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CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

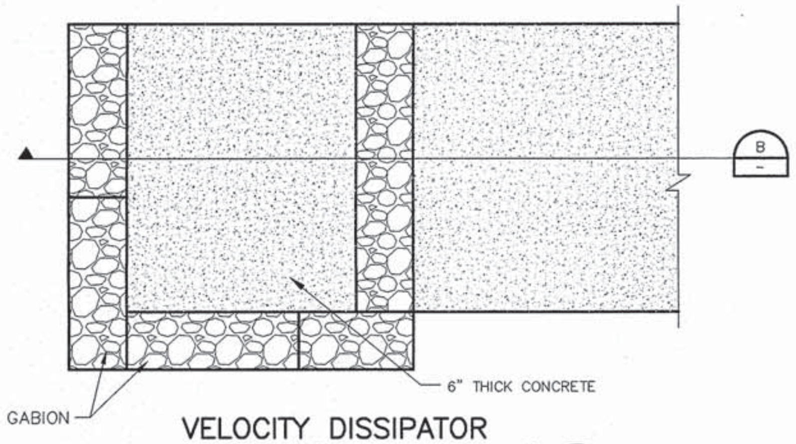
DETAILS 1

0 1" 2"

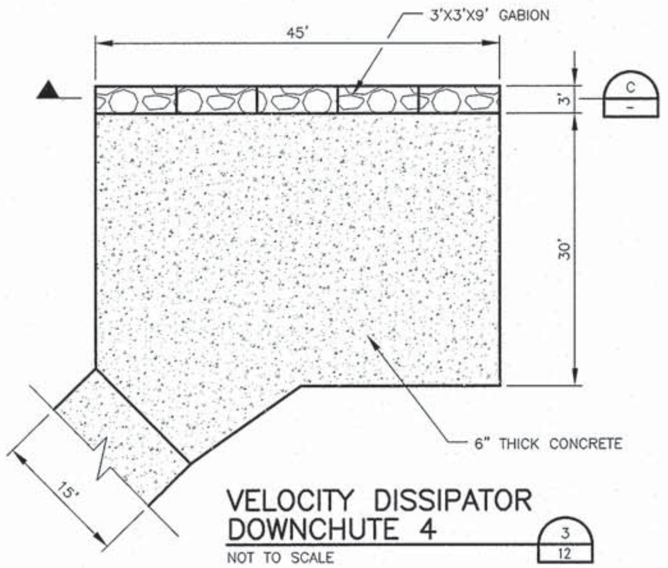
FILENAME 00C-12.DWG
SCALE 1"=30'

DWG No. 12

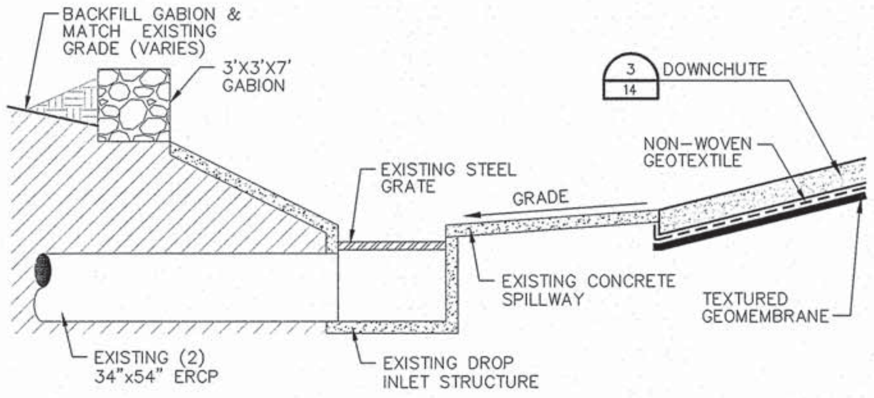
C:\pwworking\tpa\d0445788\00C-12.dwg, 6/10/2013 1:31:21 PM



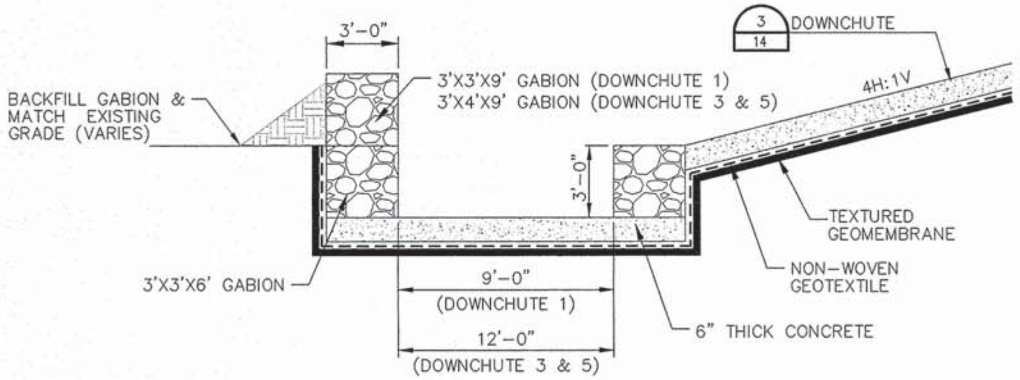
VELOCITY DISSIPATOR DOWNCHUTES 1, 3 & 5 (1/12)
NOT TO SCALE



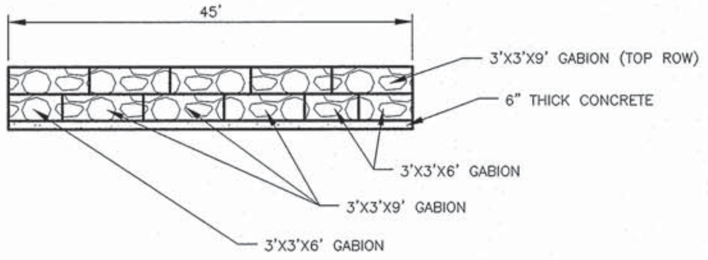
VELOCITY DISSIPATOR DOWNCHUTE 4 (3/12)
NOT TO SCALE



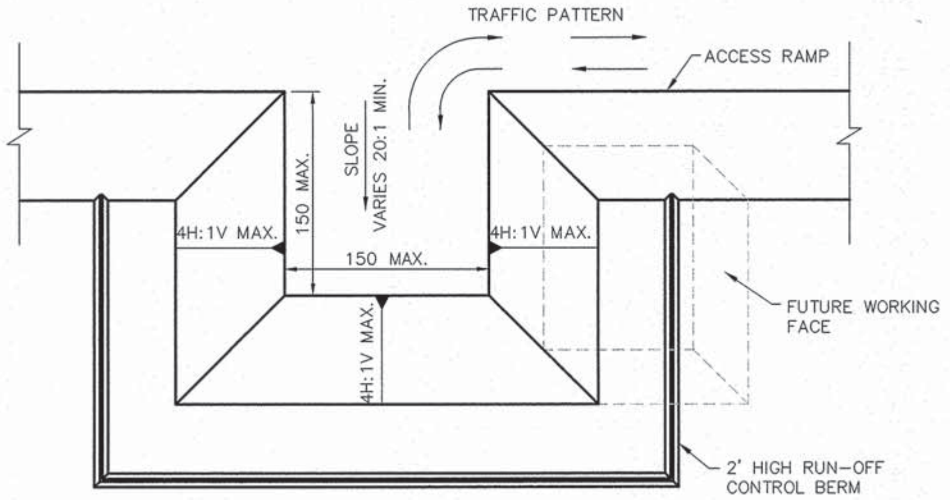
VELOCITY DISSIPATOR SECTION (A/12)
NOT TO SCALE



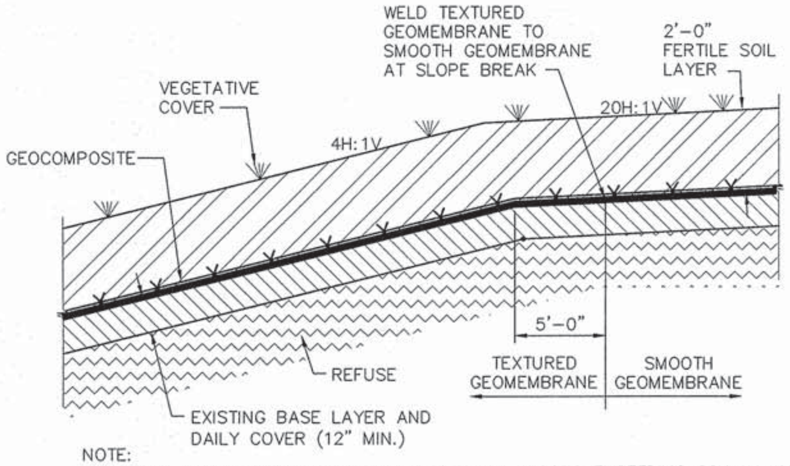
SECTION (B/–)
NOT TO SCALE



SECTION (C/–)
NOT TO SCALE

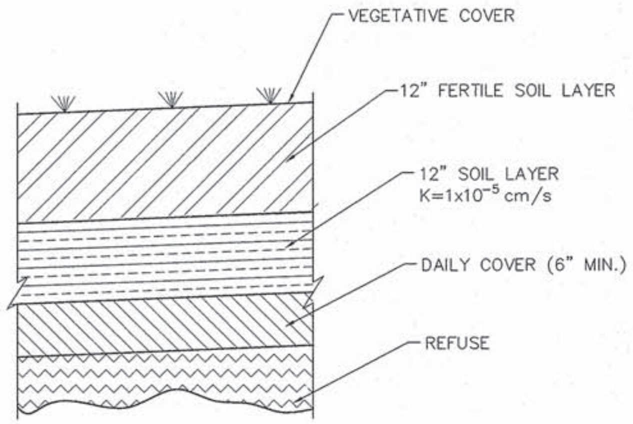


TYPICAL WORKING FACE CELL PLAN
NOT TO SCALE



NOTE:
TEXTURED GEOMEMBRANE TO BE INSTALLED ON SLOPES EXCEEDING 20H:1V AND SMOOTH GEOMEMBRANE TO BE INSTALLED ON SLOPES LESS THAN OR EQUAL TO 20H:1V

TYPICAL GEOMEMBRANE FINAL COVER SYSTEM DETAIL (2/8/14)
NOT TO SCALE



TYPICAL TEMPORARY FINAL COVER DETAIL (4/4/5)
NOT TO SCALE

Hillsborough County
Florida

HDR

HDR Engineering, Inc.
5426 Bay Center Drive
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Tampa, FL 33609-3444
HDR CA# 4213

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	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096–193806–001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

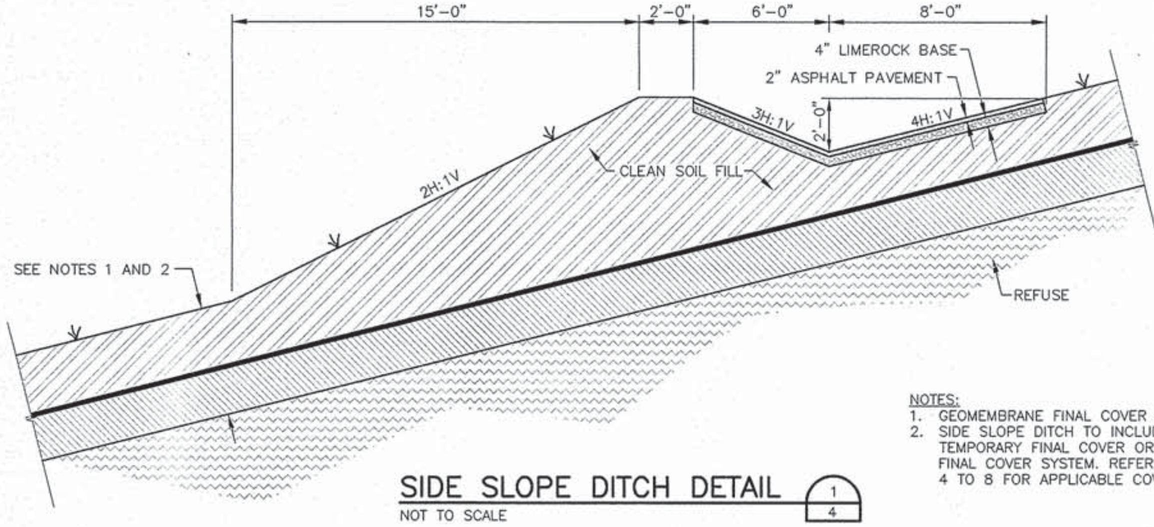
PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 2

0 1" 2"

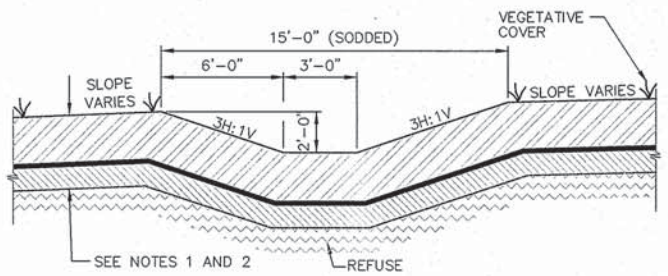
FILENAME	00C–13.DWG
SCALE	AS SHOWN

DWG No.
13



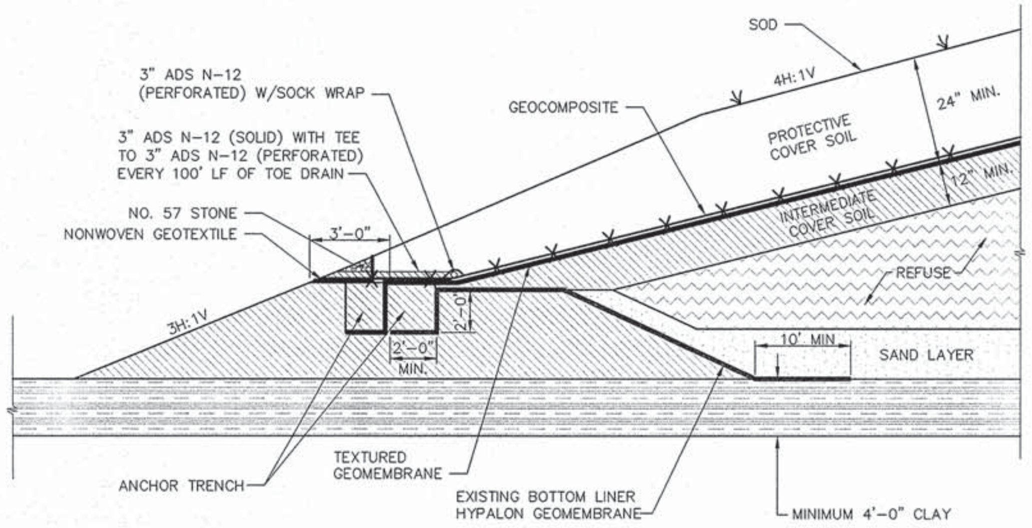
NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. SIDE SLOPE DITCH TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.

SIDE SLOPE DITCH DETAIL 1/4
NOT TO SCALE

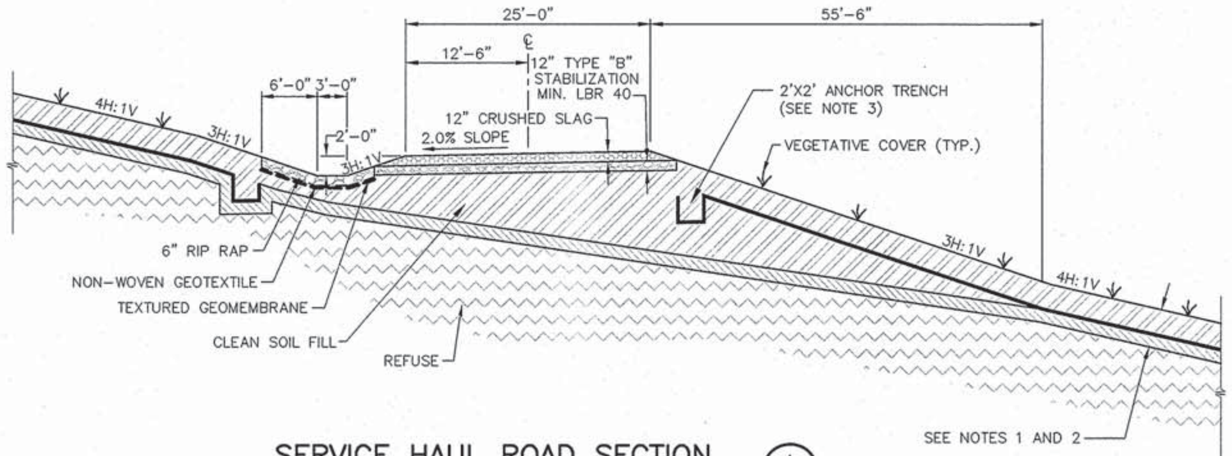


NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. TOP SLOPE DITCH TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.

TOP SLOPE DITCH DETAIL 4/4
NOT TO SCALE

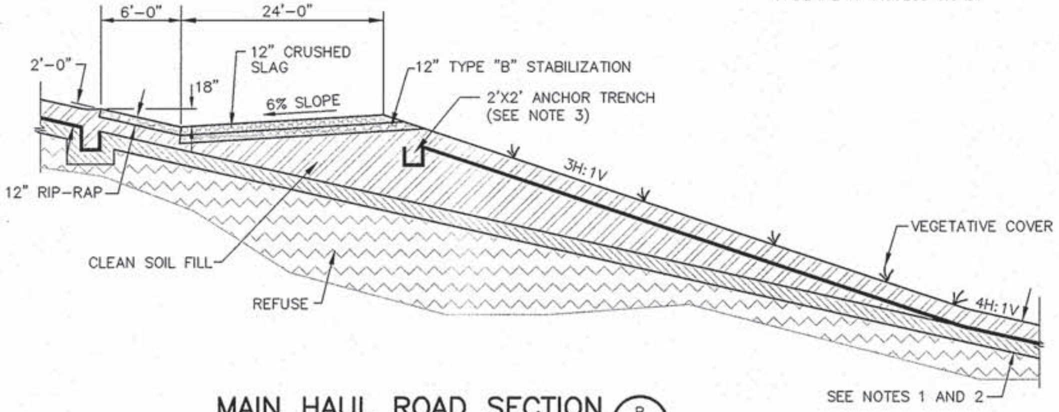


FINAL COVER TOE DRAIN DETAIL 2/8
NOT TO SCALE



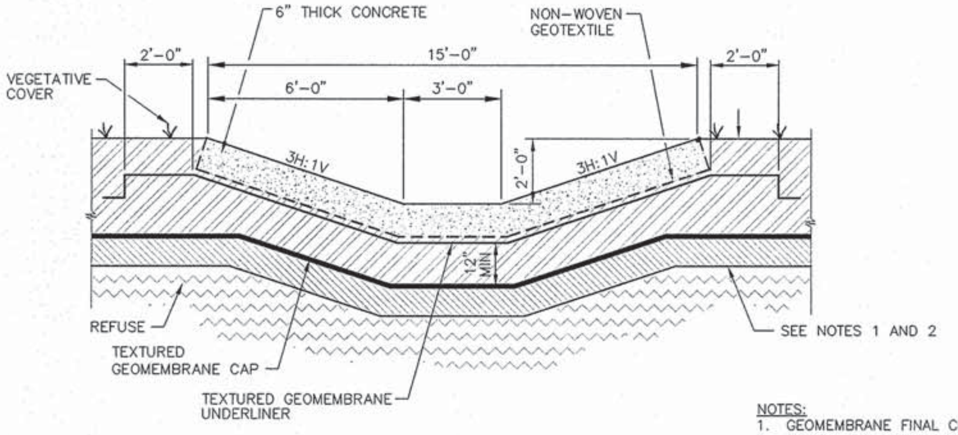
SERVICE HAUL ROAD SECTION A/8
NOT TO SCALE

NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. SERVICE HAUL ROAD TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.
3. GEOMEMBRANE LINER NOT INSTALLED UNDERNEATH ACCESS ROAD.



MAIN HAUL ROAD SECTION B/8
NOT TO SCALE

NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. MAIN HAUL ROAD TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.
3. GEOMEMBRANE LINER NOT INSTALLED UNDERNEATH ACCESS ROAD.



TYPICAL DOWNCHUTE DETAIL 3/12
NOT TO SCALE

NOTES:
1. GEOMEMBRANE FINAL COVER SYSTEM SHOWN.
2. TYPICAL DOWNCHUTE TO INCLUDE EITHER A TEMPORARY FINAL COVER OR GEOMEMBRANE FINAL COVER SYSTEM. REFER TO DRAWINGS 4 TO 8 FOR APPLICABLE COVER SYSTEM.

Hillsborough County
Florida

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CIVIL DESIGN BY	R. CURTIS
DRAWN BY	L. RODRIGUEZ
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6/7/13
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CERTIFICATE NO. 73758

PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 3

0 1" 2"

FILENAME	00C-14.DWG
SCALE	AS SHOWN

DWG No.
14

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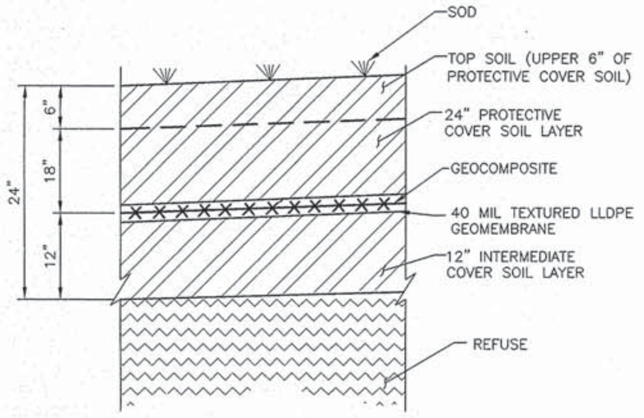
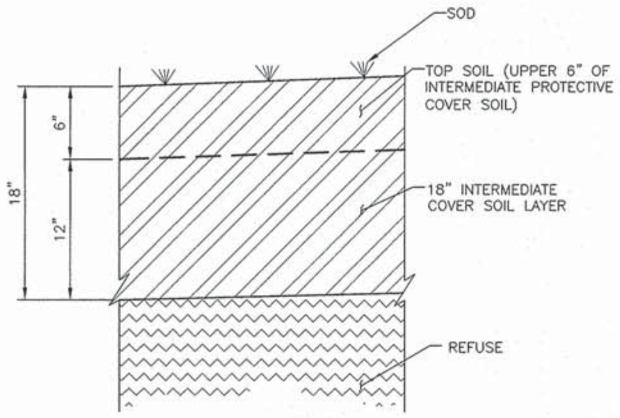
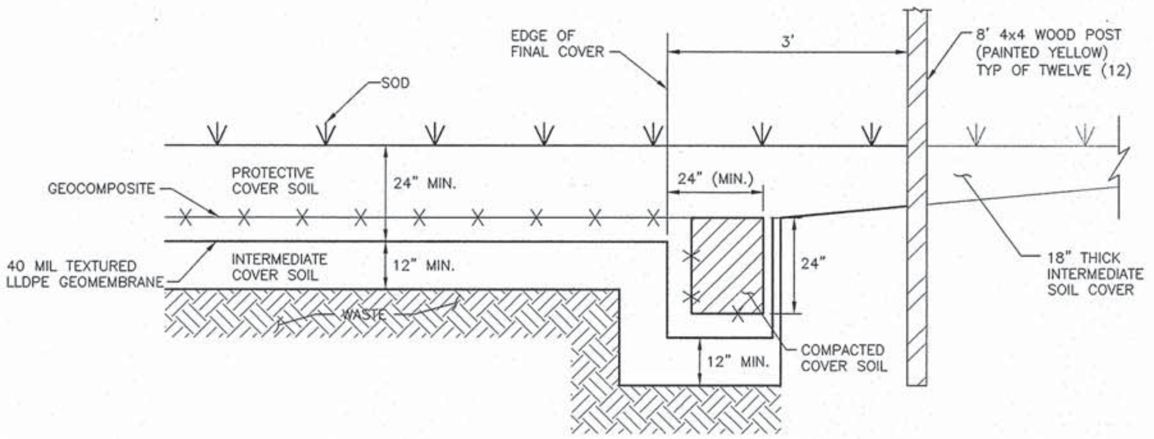


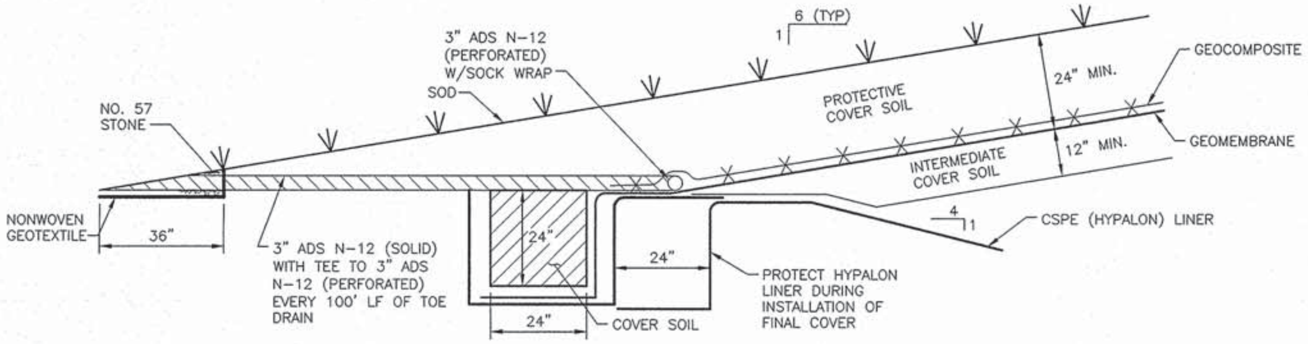
TEMPORARY GEOMEMBRANE FINAL COVER DETAIL 1/9
NOT TO SCALE



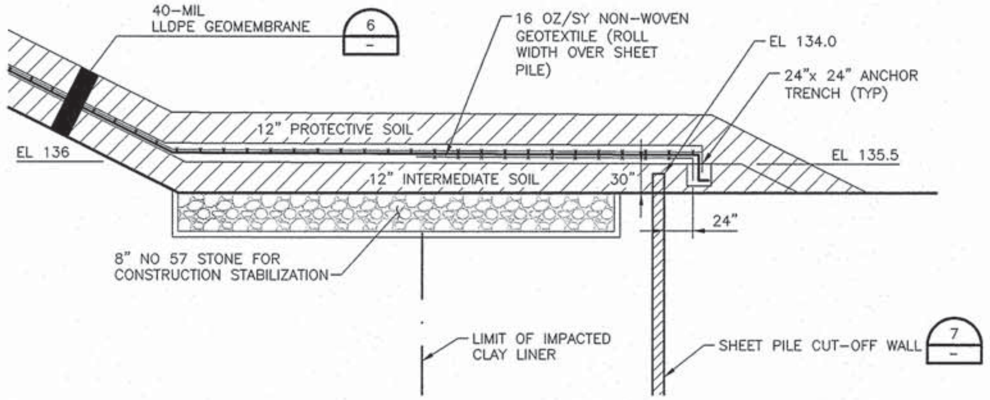
INTERMEDIATE COVER DETAIL 2/9
NOT TO SCALE



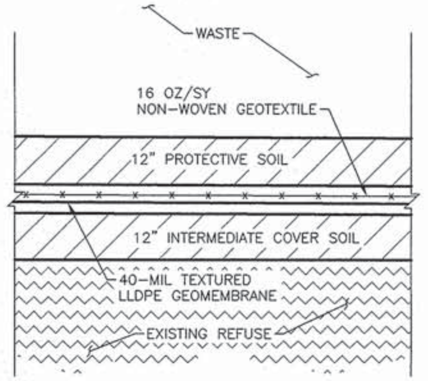
ANCHOR TRENCH DETAIL 3/9
NOT TO SCALE



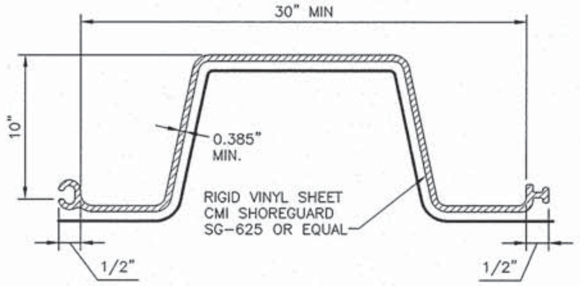
TEMPORARY TOE DRAIN DETAIL 4/9
NOT TO SCALE



40-MIL LLDPE GEOMEMBRANE ANCHOR DETAIL 5/9
NOT TO SCALE



40-MIL LLDPE GEOMEMBRANE DETAIL 6/9
NOT TO SCALE



* STEEL GUIDE METAL THICKNESS SUFFICIENT TO PENETRATE THROUGH MATERIALS SHOWN ON BORINGS.

TYPICAL SHEET PILE CUT-OFF WALL DETAIL 7/–
NOT TO SCALE

Hillsborough County
Florida

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PHASES I-VI OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

DETAILS 4

0 1" 2"
FILENAME 00C-15.dwg
SCALE AS SHOWN
DWG No. 15

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Hillsborough County
Florida

HDR Engineering, Inc.
5426 Bay Center Drive
Suite 400
Tampa, FL 33609-3444
HDR CA# 4213

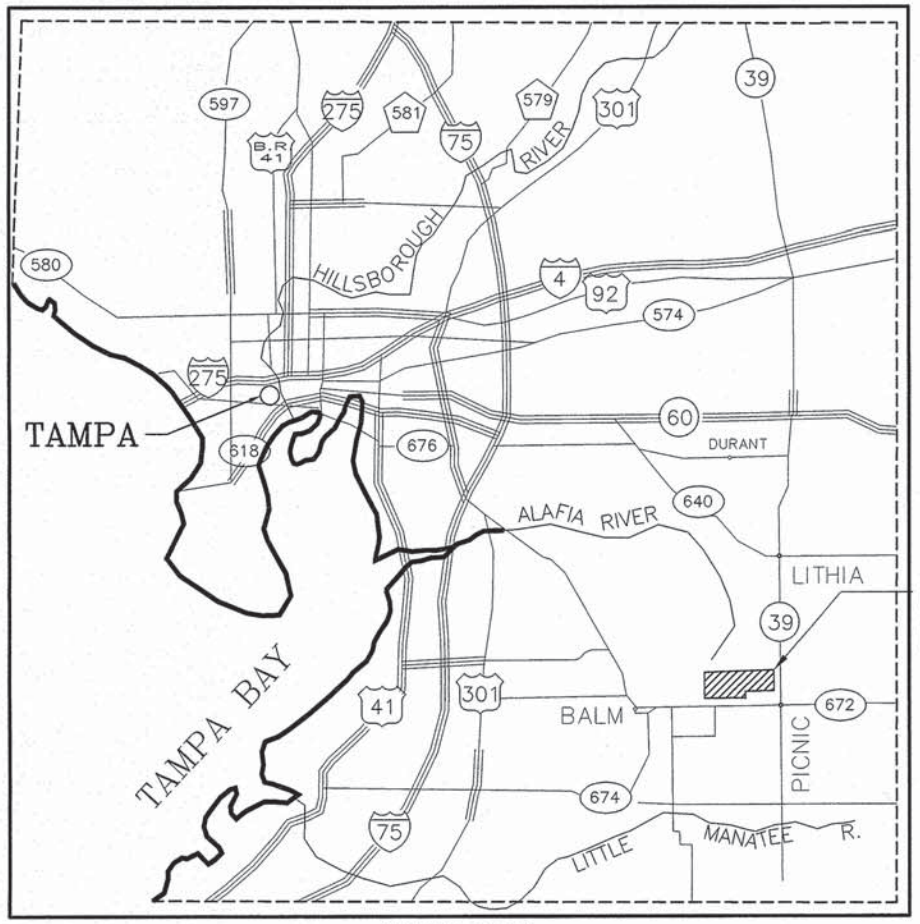
Drawings For

CAPACITY EXPANSION AREA SECTIONS 7, 8 AND 9 OPERATING SEQUENCE

SOUTHEAST COUNTY LANDFILL HILLSBOROUGH COUNTY, FLORIDA

Project No.
096-193806-001

LITHIA, FLORIDA
JUNE 2013



PROJECT LOCATION

NOT TO SCALE

DRAWING INDEX

Sheet Number	Sheet Title
1	COVER SHEET
2	INDEX, LEGENDS AND GENERAL NOTES
3	FACILITY SITE PLAN AND EXISTING TOPOGRAPHY
4	SECTIONS 7, 8 AND 9 STORMWATER PLAN
5	SECTIONS 7, 8 AND 9 OPERATING SEQUENCE FILL SEQUENCE 9 TO 12
6	SECTIONS 7, 8 AND 9 OPERATING SEQUENCE FILL SEQUENCE 13 TO 16
7	SECTIONS 7, 8 AND 9 OPERATING SEQUENCE FILL SEQUENCE 17 AND 18
8	SECTIONS 7, 8 AND 9 FINAL GRADING PLAN
9	SECTIONS 7, 8 AND 9 CROSS SECTIONS
10	SECTIONS 7, 8 AND 9 OPERATING SEQUENCE DETAILS
11	SECTIONS 7, 8 AND 9 OPERATING SEQUENCE DETAILS

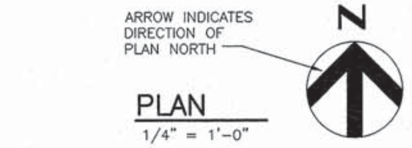
No. 73758
6/7/13

ROBERT B. CURTIS, P.E.
LICENSE NO. 73758
HDR CA# 4213

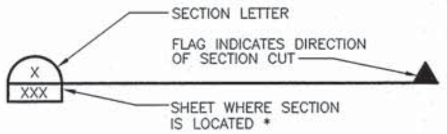
ENGINEERING SYMBOLOGY

⤳	DRAINAGE FLOW DIRECTION
— — — —	APPROXIMATE PHASE FOOTPRINT
———	APPROXIMATE LANDFILL LIMITS
—— ——	APPROXIMATE LIFT LIMITS
———	APPROXIMATE LIMITS OF BORROW AREA
▨	APPROXIMATE TEMPORARY COVER AREA AFTER EACH LIFT
▨	APPROXIMATE INTERMEDIATE FINAL COVER AREA
— · — · —	EXISTING SWALE
—140—	EXISTING CONTOUR
—140—	PROPOSED CONTOUR
▨	PROPOSED DOWNCHUTE
BVC	BEGIN VERTICAL CURVE
CTRD	CENTERED
ERCP	ELLIPTICAL REINFORCED CONCRETE PIPE
EVC	END VERTICAL CURVE
~150~	EXIST. CONTOURS
EXP. JT.	EXPANSION JOINT
I.E.	INVERT ELEVATION
LF	LINEAR FEET
LT	LEFT
PC	POINT OF CURVATURE
PI	POINT OF INTERSECTION
PT	POINT OF TANGENCY
PVI	POINT OF VERTICAL INTERSECTION
RT	RIGHT
TYP.	TYPICAL
〰	VEGETATION
VC	VERTICAL CURVE
ø	DIAMETER
'	FOOT
"	INCH
Ⓐ	CELL DESIGNATION
(S-21)	EXISTING DRAINAGE STRUCTURE
TS-4	TEMPORARY DRAINAGE STRUCTURE
▬ ▪ ▪ ▬	PROPERTY LINE
>—<	STORMWATER STRUCTURE
—FM—	FORCE MAIN PIPE
—LC—	LEACHATE COLLECTION PIPE

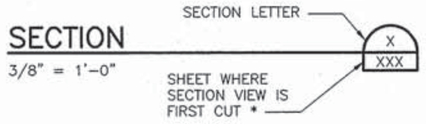
GENERAL SYMBOLOGY



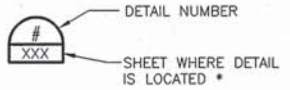
PLAN TITLE



SECTION CUT MARKER



SECTION TITLE

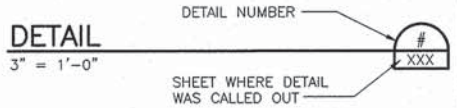


DETAIL MARKER

FOR REFERENCING DETAILS INCLUDED IN DRAWING SET.

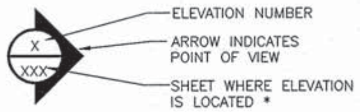
DETAIL MARKER

FOR REFERENCING DETAILS BOUND IN SPECIFICATIONS OR SEPARATE VOLUME.

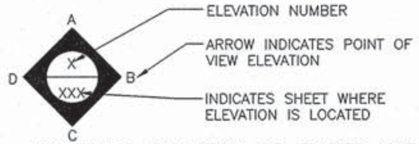


DETAIL TITLE

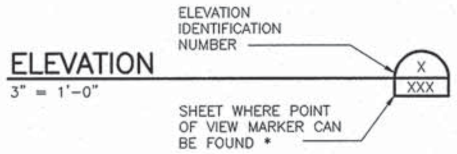
* FOR COMMON DETAILS AND SECTIONS OR DETAILS THAT ARE CUT OR CALLED OUT ON MULTIPLE SHEETS, THE SHEET NUMBER IS REPLACED BY A DASH (–).



SINGLE ELEVATION OR PHOTO MARKER



MULTIPLE ELEVATION OR PHOTO MARKER



ELEVATION TITLE

ABBREVIATIONS

APPROX – APPROXIMATE, APPROXIMATELY
BLDG – BUILDING
BTM – BOTTOM
CB – CATCH BASIN
CM – CONCRETE MONUMENT
CMP – CORRUGATED METAL PIPE
CONC – CONCRETE
CONT – CONTINUOUS
CORR – CORRUGATED
DET – DETAIL
DIA – DIAMETER
DIM – DIMENSION
DWG – DRAWING
EA – EACH
EOL – EDGE OF LINER
ETC – ET CETERA
ENCL – ENCLOSE, ENCLOSURE
EL – ELEVATION
EQUIP – EQUIPMENT
EXIST – EXISTING
FDEP – FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
FDOT – FLORIDA DEPARTMENT OF TRANSPORTATION
FES – FLARED END SECTION
FIN – FINISHED
FM – FORCE MAIN
GALV – GALVANIZED
GCL – GEOSYNTHETIC CLAY LINER
GFFR – GROUT FILLED FIBER REVETMENT
GR – GRADE
GDL – GEOSYNTHETIC DRAINAGE LINER
GM – GAS MONITORING LOCATION
GP – GAS PROBE
HCSWMG – HILLSBOROUGH COUNTY SOLID WASTE MANAGEMENT GROUP
HDPE – HIGH DENSITY POLYETHYLENE
HP – HIGH POINT
ID – INSIDE DIAMETER
IE – INVERT ELEVATION
LF – LINEAL FEET
LFG – LANDFILL GAS
LLDPE – LINEAR LOW DENSITY POLYETHYLENE
LP – LOW POINT
MES – MITRED END SECTION
MAX – MAXIMUM
MH – MANHOLE
MIN – MINIMUM
MISC – MISCELLANEOUS
MSL – (ABOVE) MEAN SEAL LEVEL
MT – MOUNT
MW – GROUNDWATER MONITORING WELL
N/A – NOT APPLICABLE
N/AVAIL – NOT AVAILABLE
NGVD – NATIONAL GEODETIC VERTICAL DATUM
NIC – NOT IN CONTRACT
No – NUMBER
NTS – NOT TO SCALE
OC – ON CENTER
OD – OUTSIDE DIAMETER
OSHA – OCCUPATIONAL SAFETY & HEALTH ADMINISTRATION
PLS – PROFESSIONAL LAND SURVEYOR
PS – PUMP STATION
R – RADIUS
RCP – REINFORCED CONCRETE PIPE
REF – REFERENCE
REQD – REQUIRED
SCH – SCHEDULE
SDR – STANDARD DIMENSION RATIO
SHT – SHEET
SIM – SIMILAR
SS – STAINLESS STEEL
STD – STANDARD
STL – STEEL
STW – STORMWATER MONITORING STATION
TPO – THERMOPLASTIC POLYOLEFIN
TYP – TYPICAL
USC&GS – UNITED STATES COASTAL AND GEODETIC SURVEY
USGS – UNITED STATES GEOLOGICAL SURVEY
WGT – WEIGHT
W.E. – WATER ELEVATION
WWTP – WASTE WATER TREATMENT PLANT

GENERAL NOTES

1. THE EXISTING TOPOGRAPHY, DATED 1/5/2013, WAS OBTAINED FROM DRAWINGS PROVIDED BY PICKETT & ASSOCIATES, INC.
2. THE PROPOSED OPERATING SEQUENCES (LIFTS 9 – 18) ARE BASED ON THE EXISTING TOPOGRAPHY SHOWN ON THE 1/5/2013 AERIAL SURVEY. ACTUAL OPERATING SEQUENCES MAY NEED TO BE MODIFIED IN THE FIELD TO ALLOW FOR LANDFILL SETTLEMENT. REVISED GRADES WILL BE DETERMINED BASED ON THE MAXIMUM DESIGNED 20-FOOT LIFT HEIGHT.

Hillsborough County
Florida

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HDR CA# 4213

ISSUE	DATE	DESCRIPTION
	6/2013	FDEP SUBMITTAL

PROJECT MANAGER	R. SIEMERING
REVIEWED BY	T. YANOSCHAK
CIVIL DESIGN BY	R. CURTIS RBC
DRAWN BY	L. RODRIGUEZ
PROJECT NUMBER	0096-193806-001

No. 73758
6/7/13
ROBERT B. CURTIS, P.E.
CERTIFICATE NO. 73758

CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

INDEX, LEGENDS AND GENERAL NOTES

0 1" 2"

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DWG No.
2

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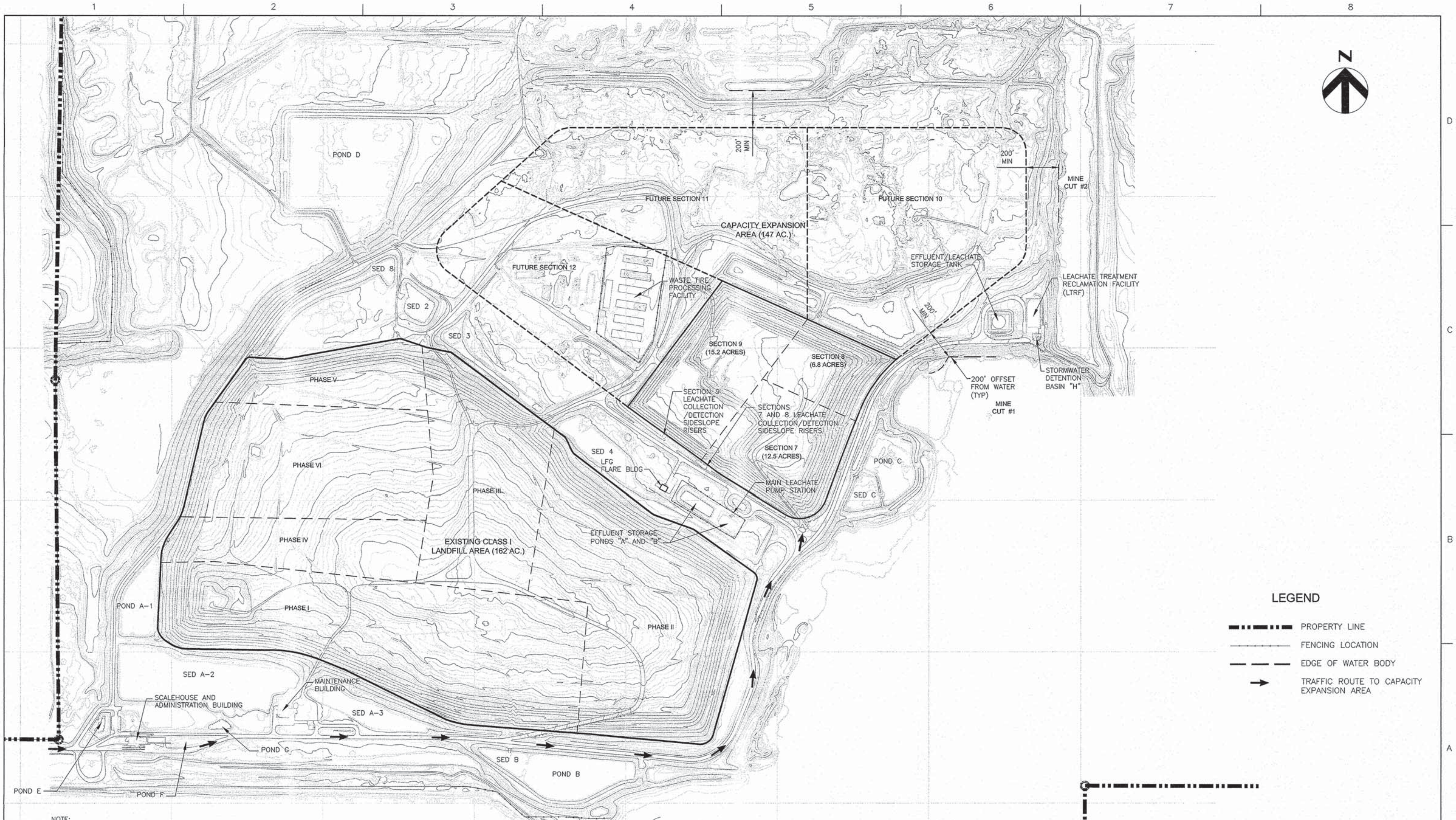

NOTE:
EXISTING TOPOGRAPHY FROM AERIAL PHOTOGRAPHY PROVIDED BY PICKETT AND ASSOCIATES, INC. DATED JANUARY 5, 2013.

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HILLSBOROUGH COUNTY, FLORIDA

FACILITY SITE PLAN AND EXISTING TOPOGRAPHY

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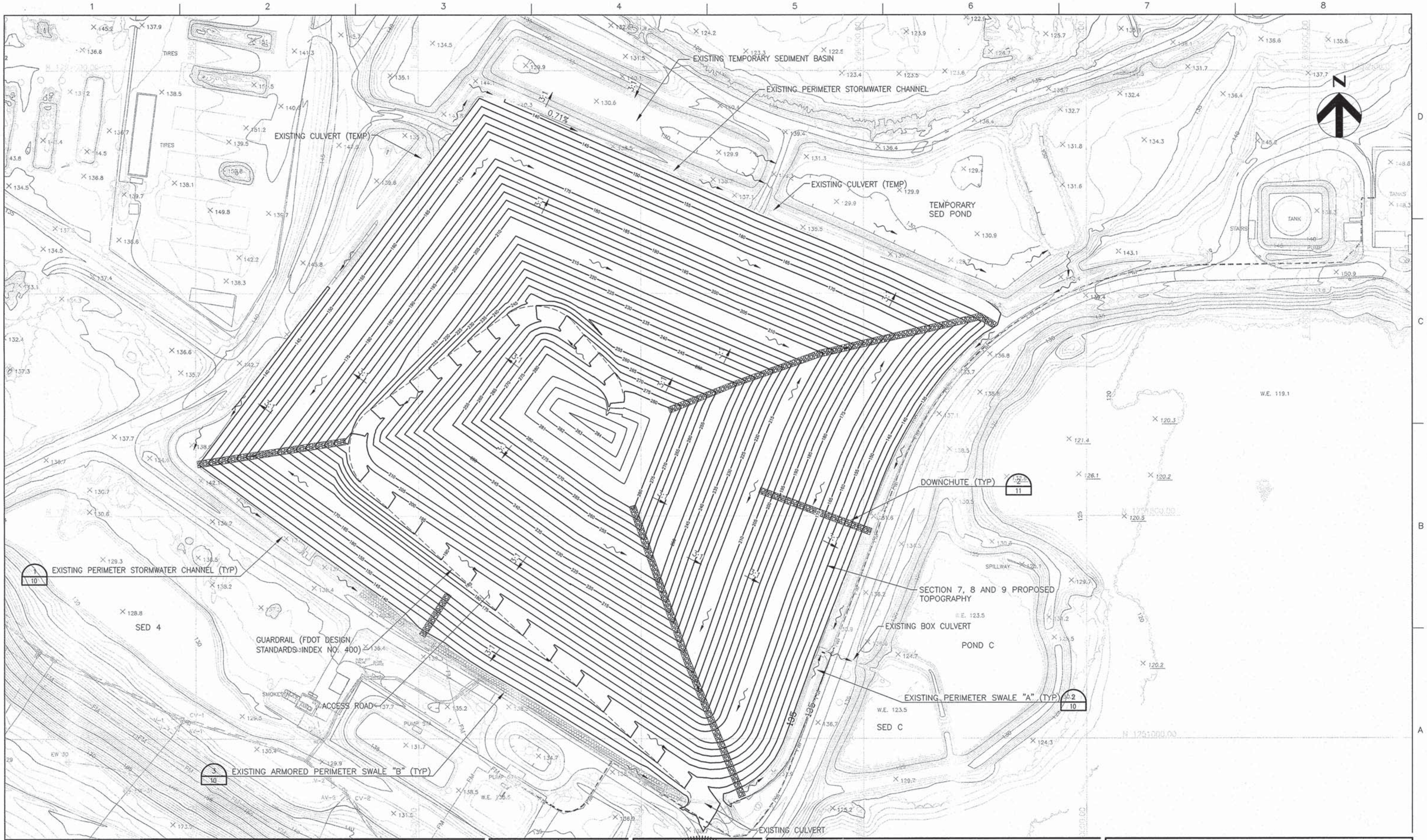

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 STORMWATER PLAN

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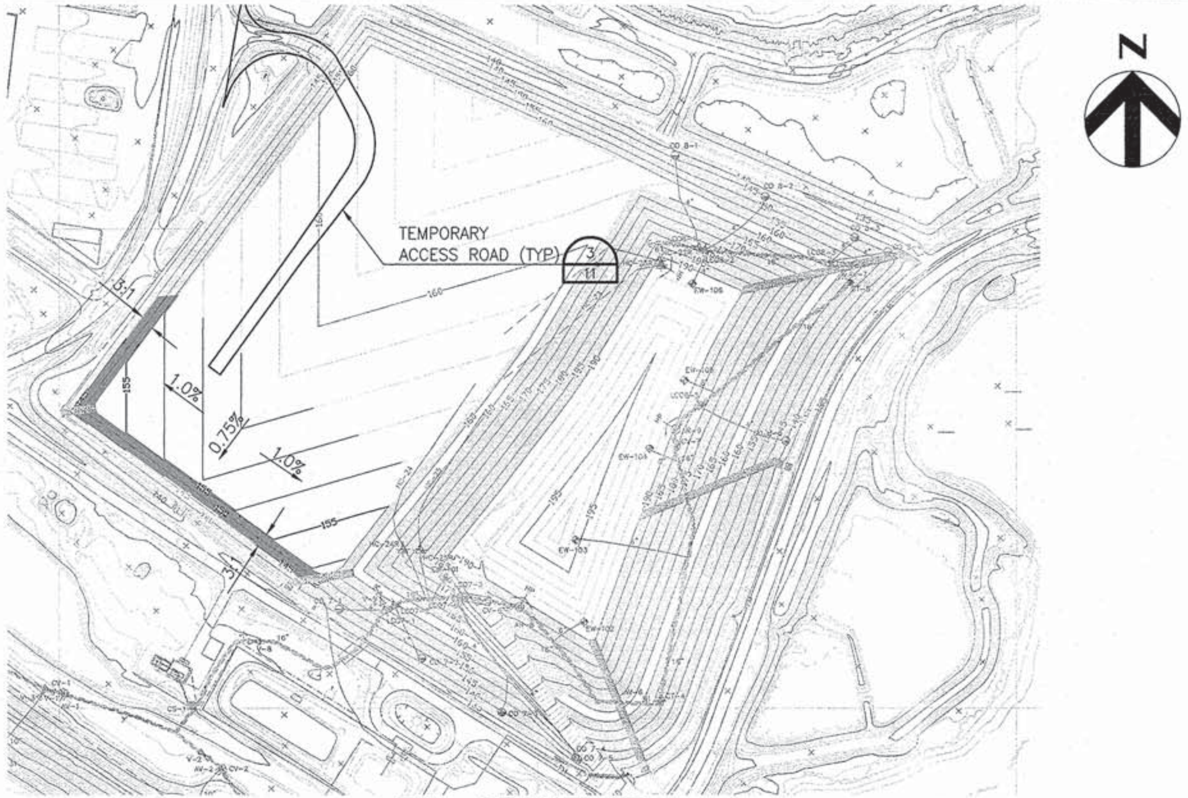

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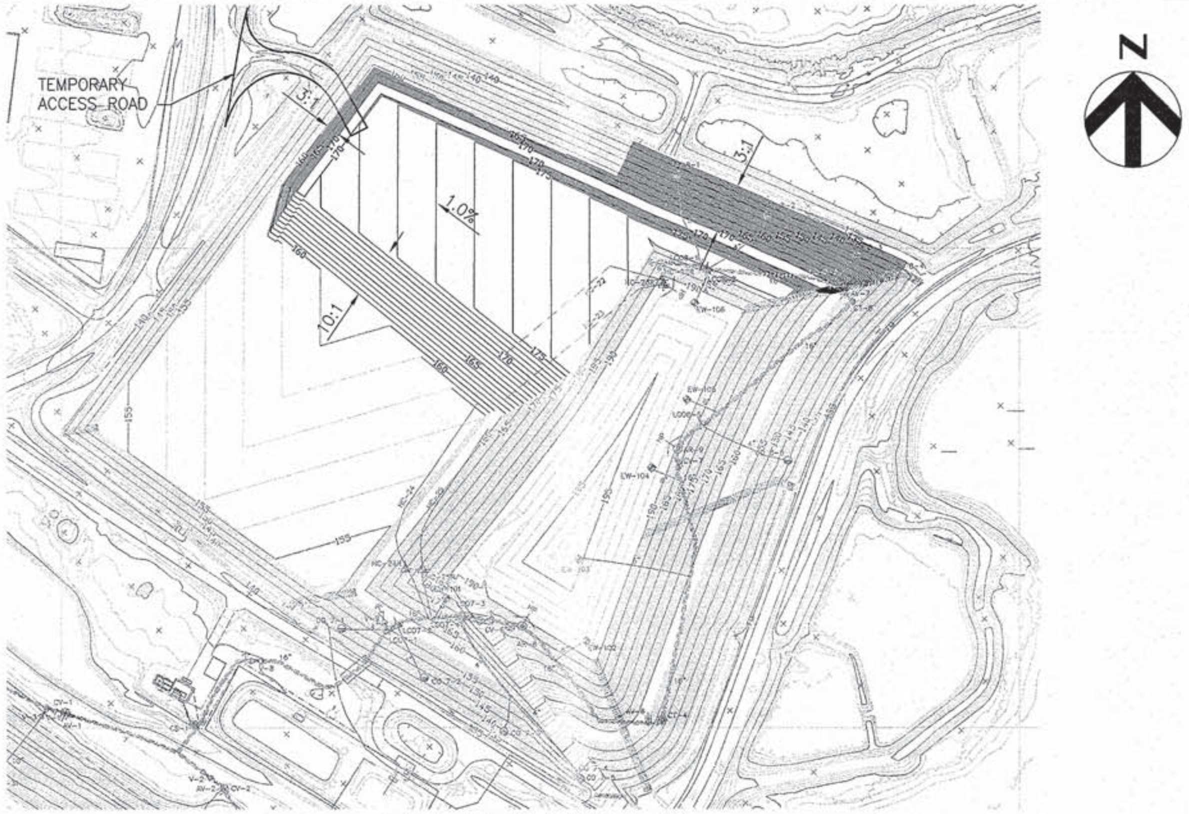

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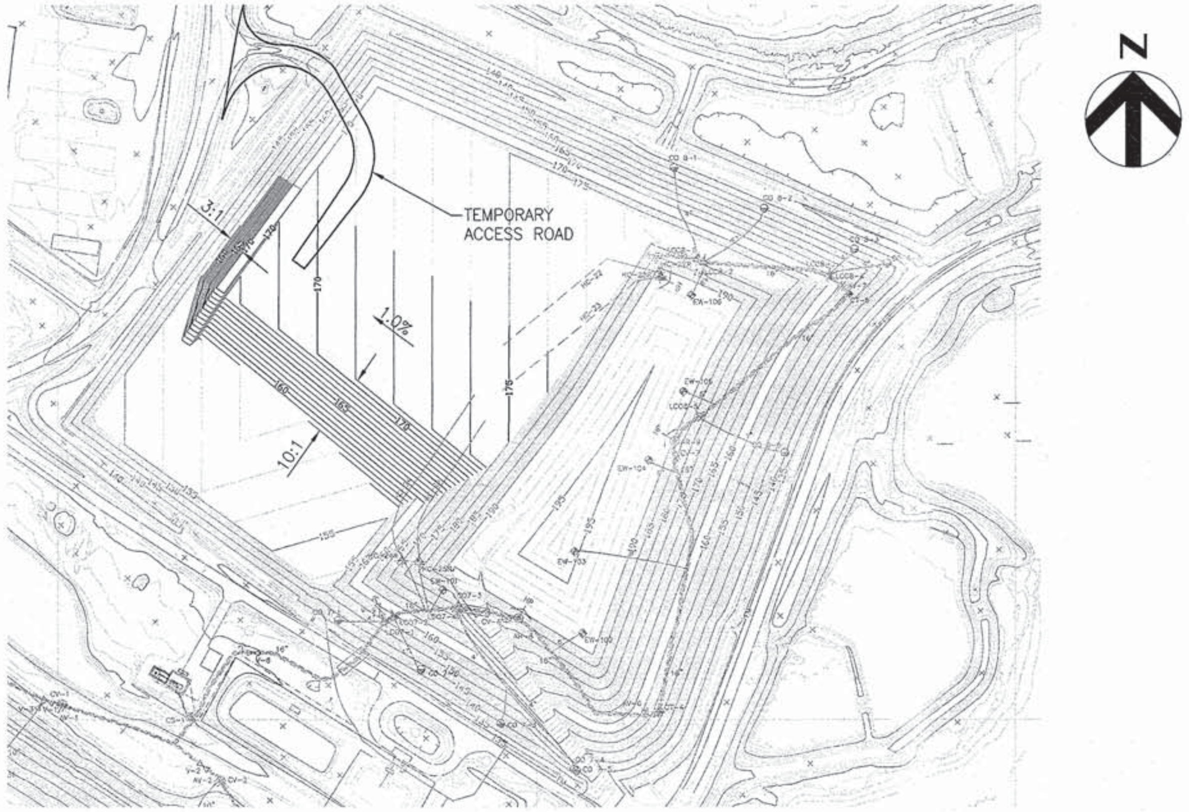

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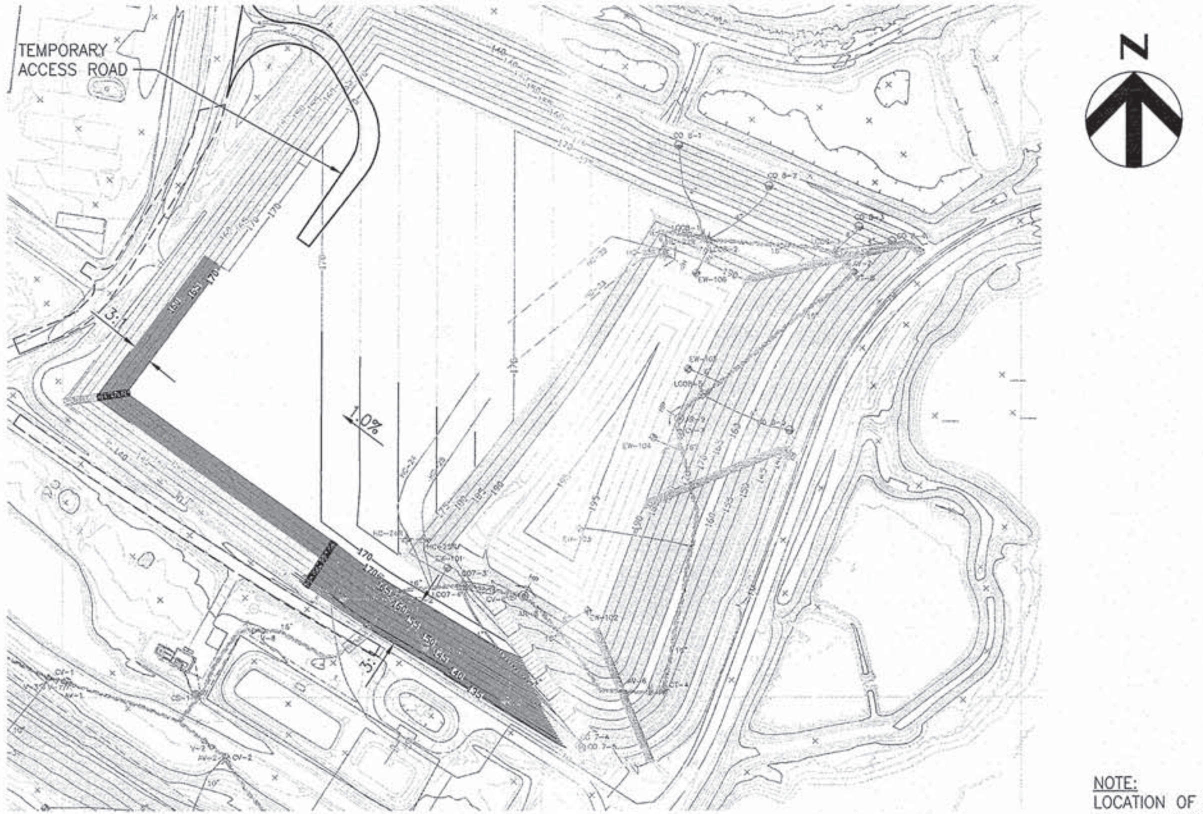

FILL SEQUENCE 12
SCALE 1"=200'

NOTE:
LOCATION OF ACCESS ROAD TO VARY WITH OPERATIONS

Hillsborough County Florida

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
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SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 OPERATING SEQUENCE
FILL SEQUENCE 9 TO 12

0 1" 2"

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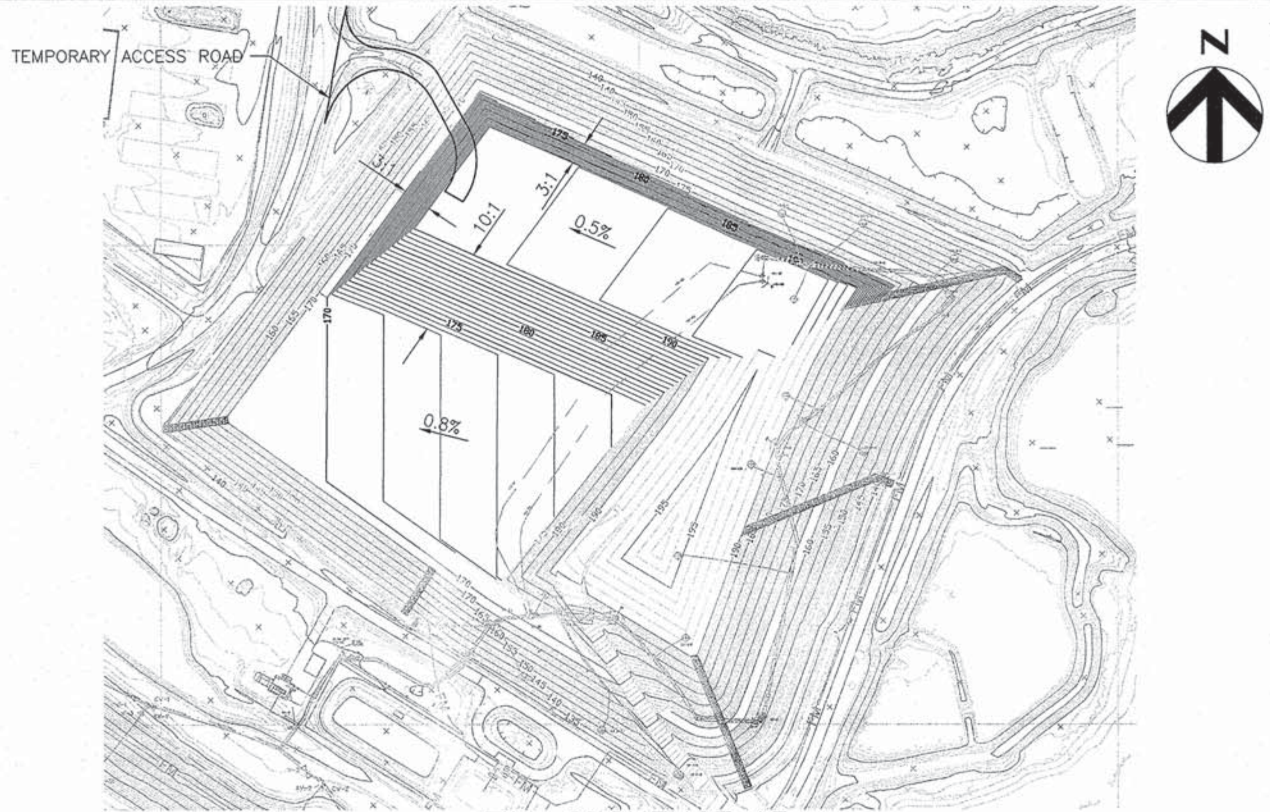

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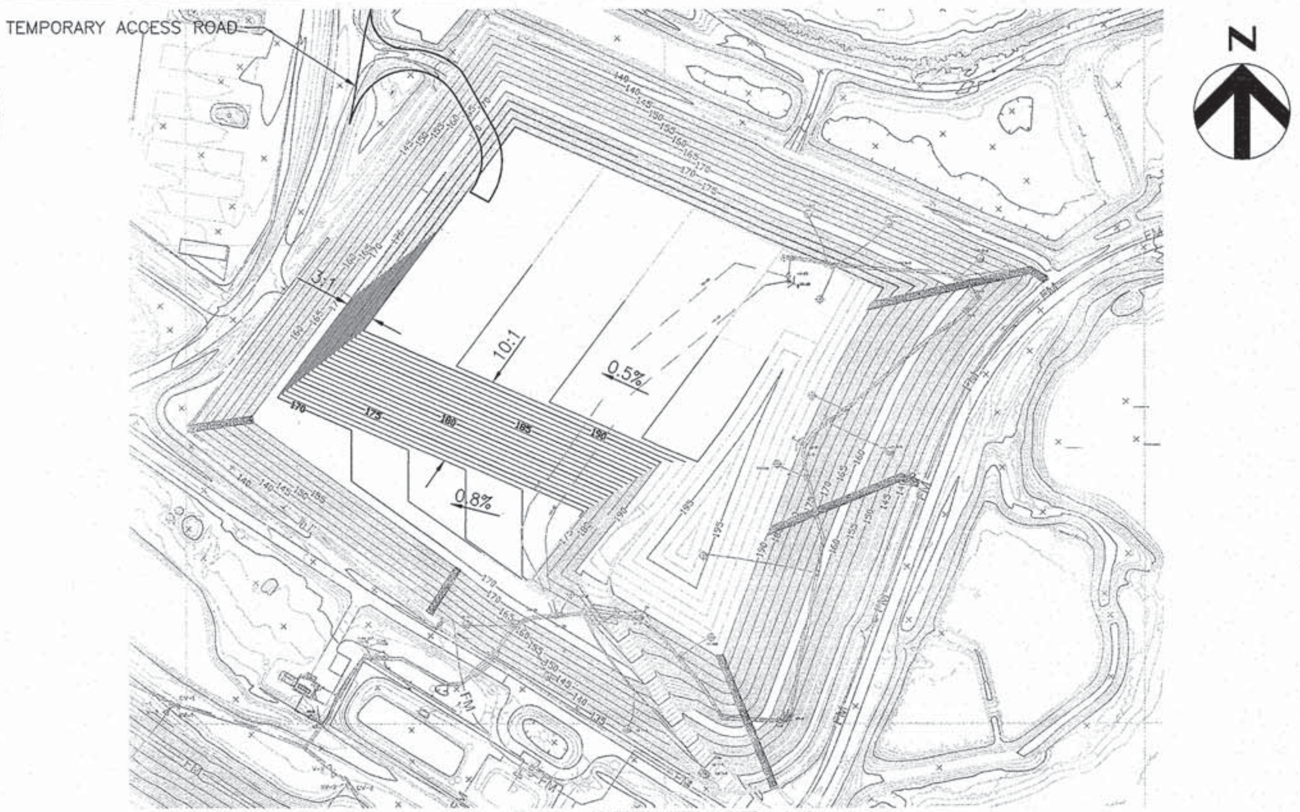

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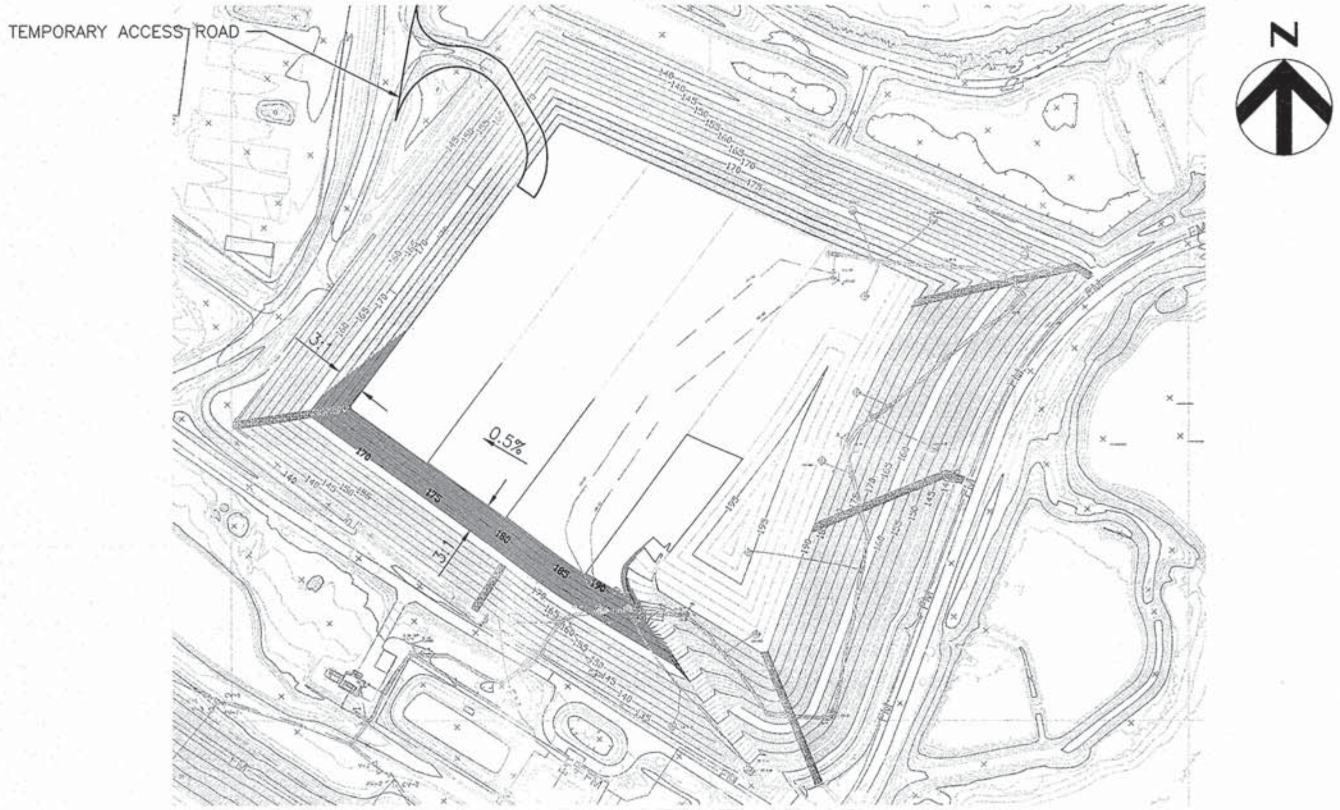

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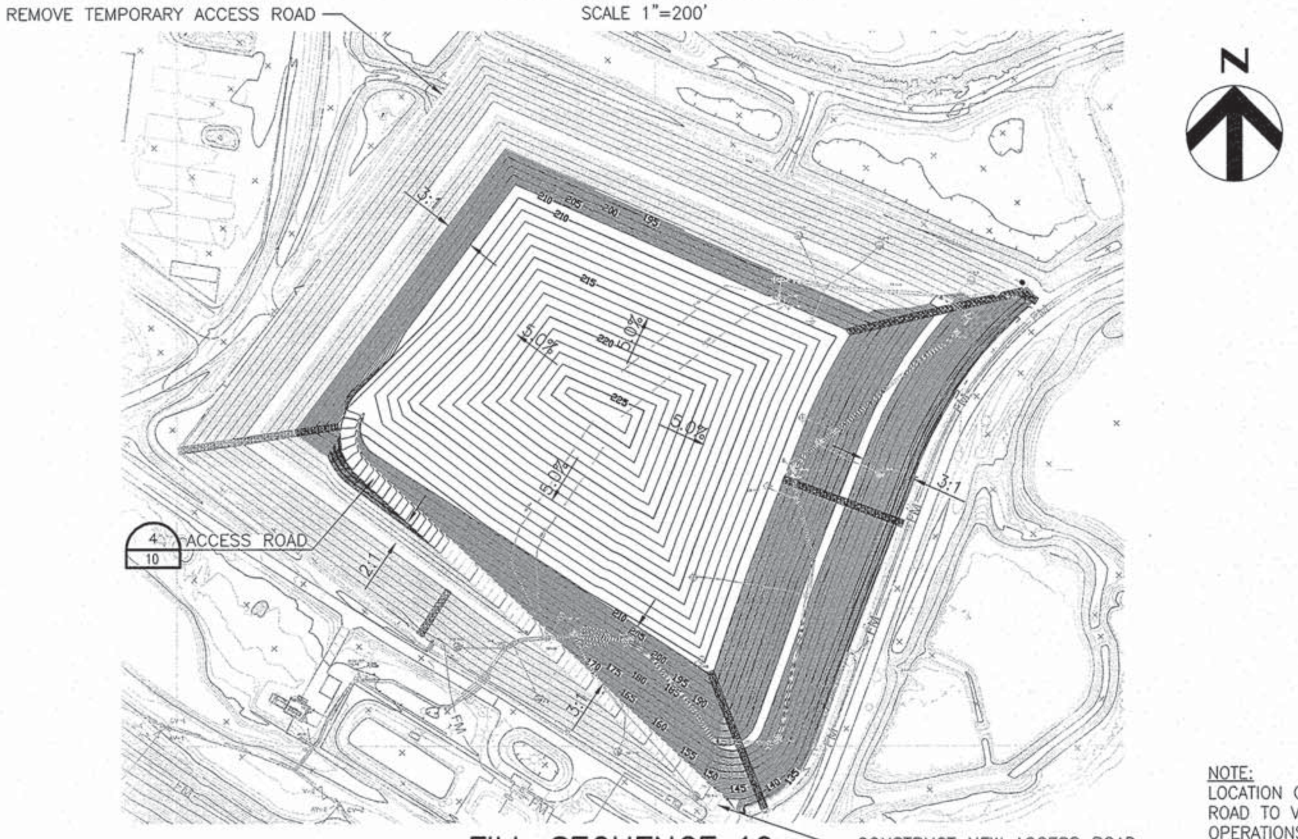

FILL SEQUENCE 16

SCALE 1"=200'

NOTE:
LOCATION OF ACCESS
ROAD TO VARY WITH
OPERATIONS

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 OPERATING SEQUENCE
FILL SEQUENCE 13 TO 16

0 1" 2"

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DWG No.
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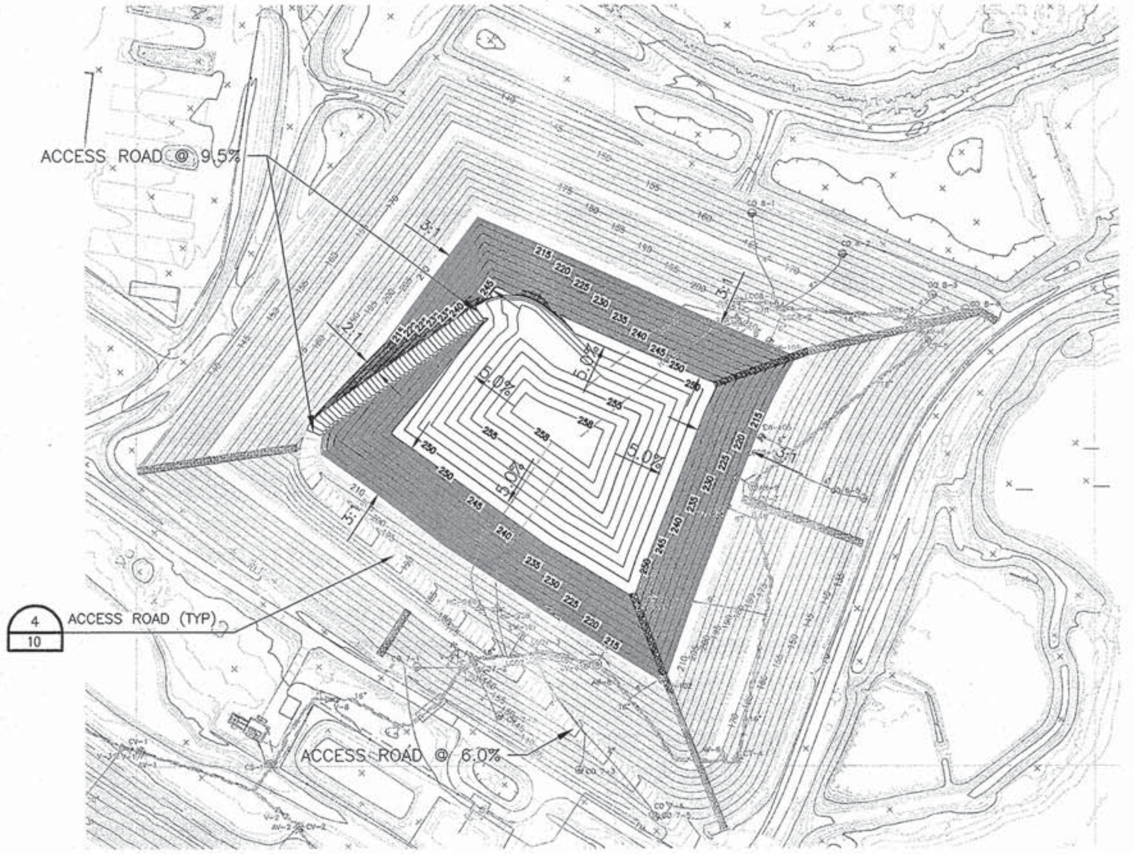

FILL SEQUENCE 17

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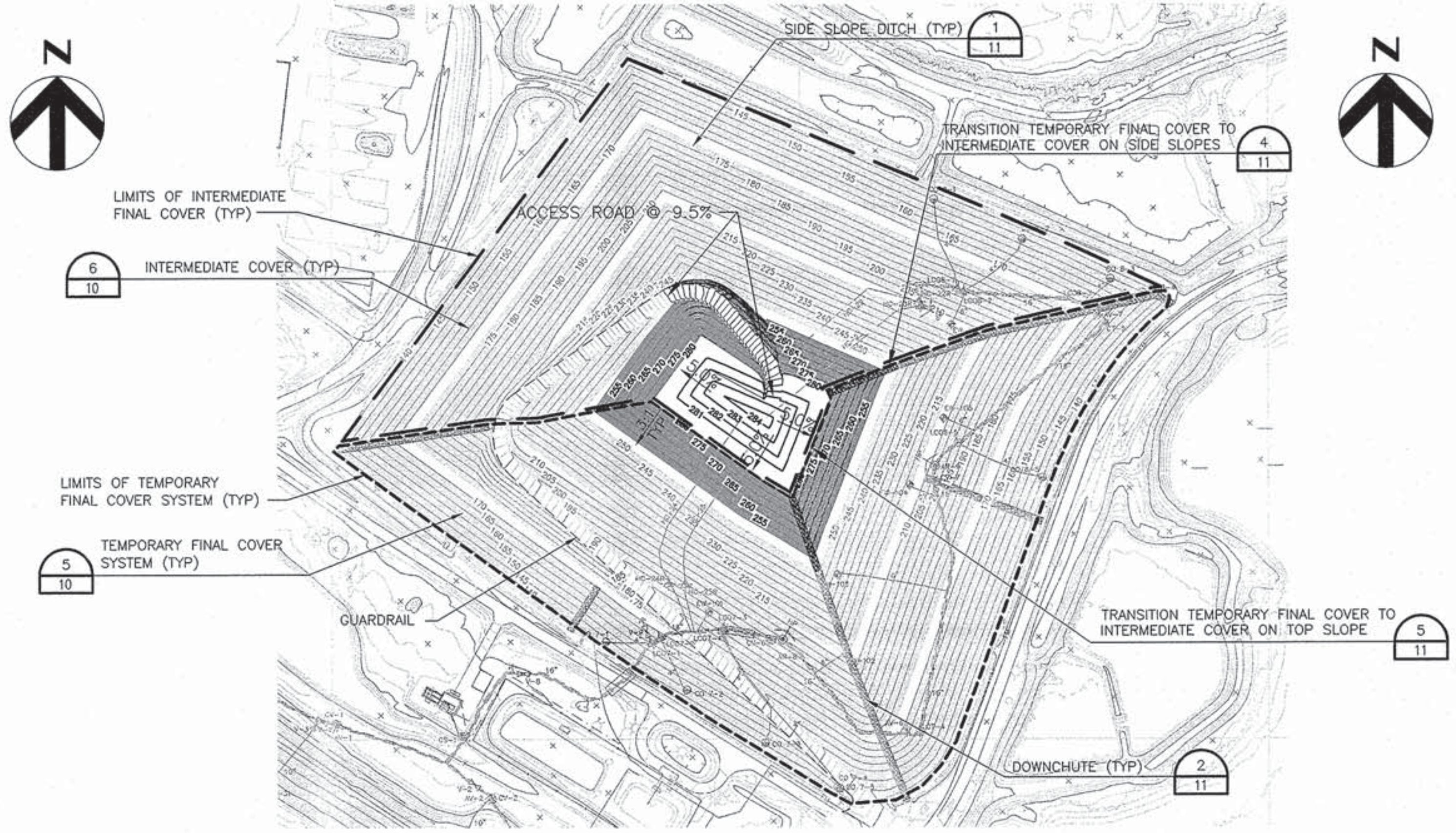

FILL SEQUENCE 18

SCALE 1"=200'

NOTES:
1. TEMPORARY FINAL COVER TO BE PLACED OVER SOUTH AND EAST SLOPES. INTERMEDIATE COVER TO BE PLACED ON WEST AND NORTH SLOPES TO ALLOW FOR EXPANSION.
2. LOCATION OF ACCESS ROAD TO VARY WITH OPERATIONS.
3. UPON COMPLETION OF CAPACITY EXPANSION AREA SECTION 13, A FINAL COVER WILL BE PLACED ON THE SOUTH AND EAST SLOPES. REFER TO DETAIL 7 SHEET 10.

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 OPERATING SEQUENCE
FILL SEQUENCE 17 AND 18

0 1" 2"

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DWG No.
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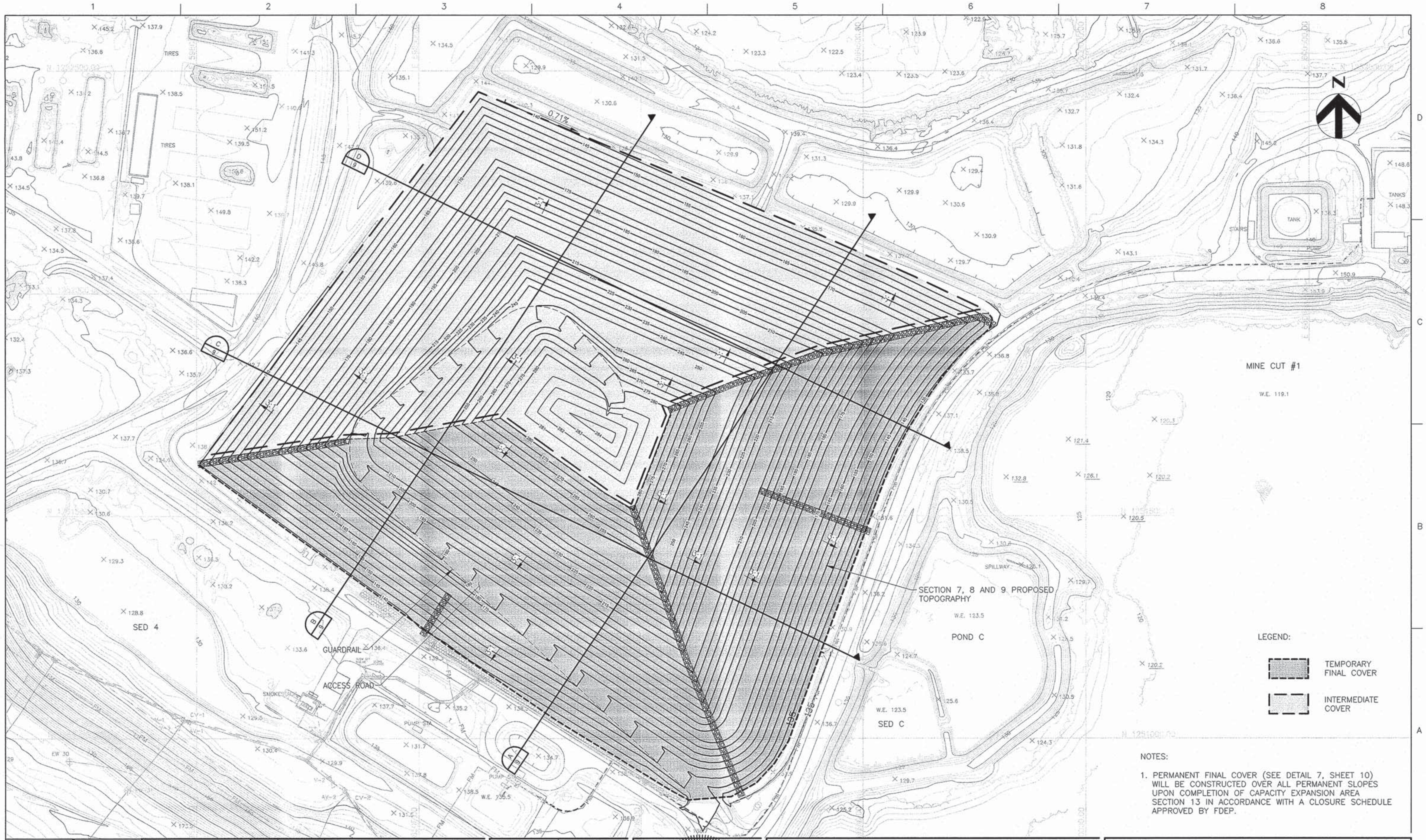

LEGEND:

▨ TEMPORARY FINAL COVER

▨ INTERMEDIATE COVER

NOTES:

1. PERMANENT FINAL COVER (SEE DETAIL 7, SHEET 10) WILL BE CONSTRUCTED OVER ALL PERMANENT SLOPES UPON COMPLETION OF CAPACITY EXPANSION AREA SECTION 13 IN ACCORDANCE WITH A CLOSURE SCHEDULE APPROVED BY FDEP.

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 FINAL GRADING PLAN

0 1" 2"

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DWG No.
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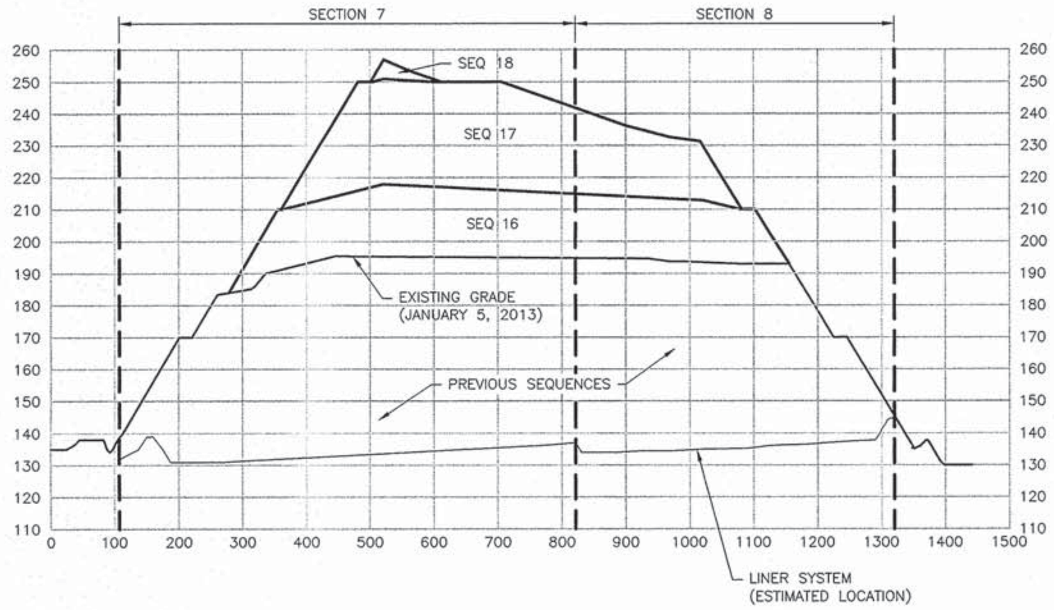
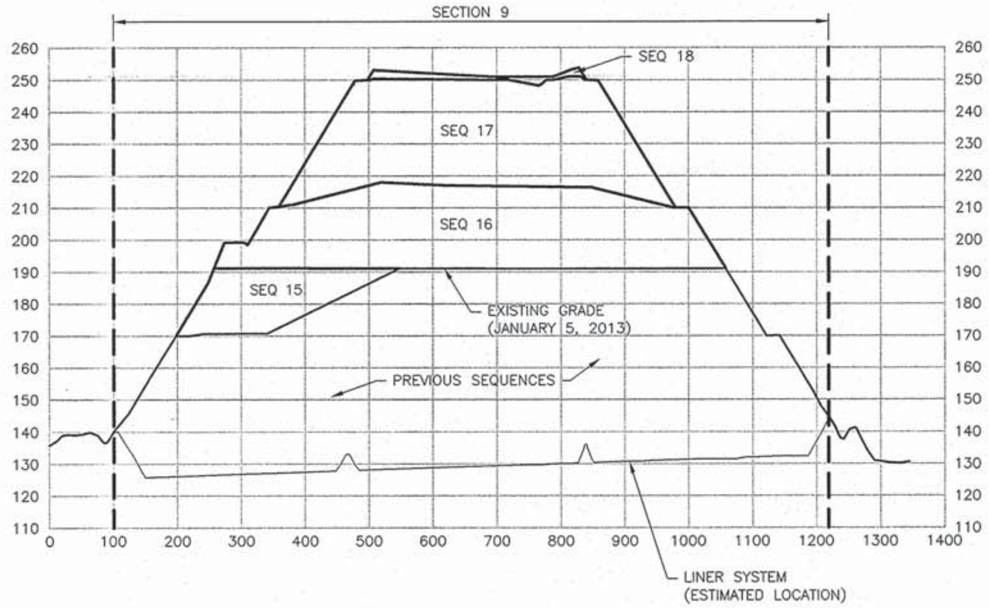
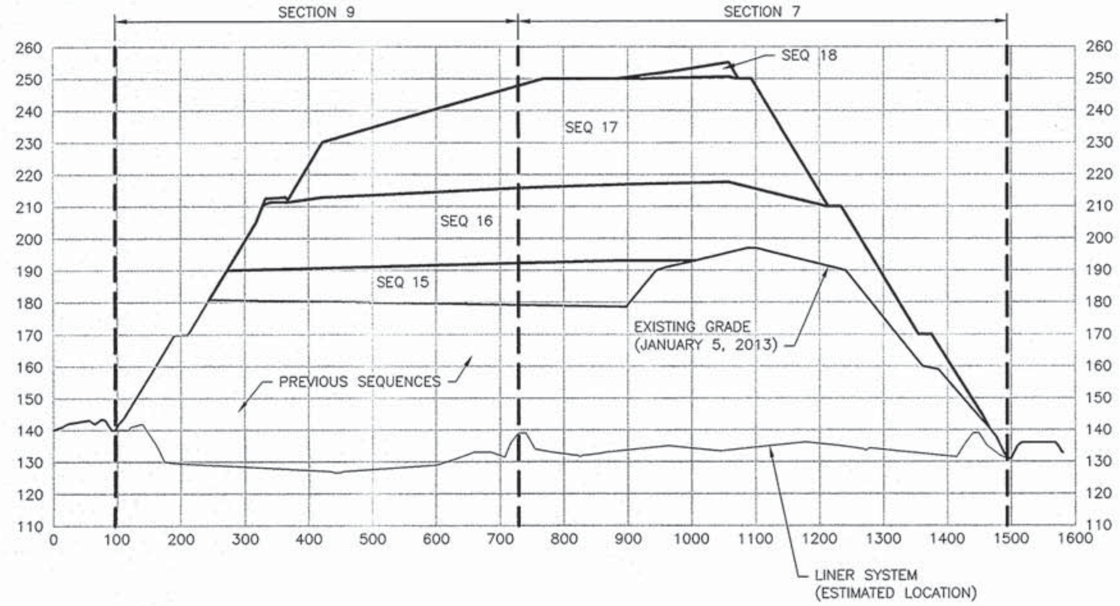


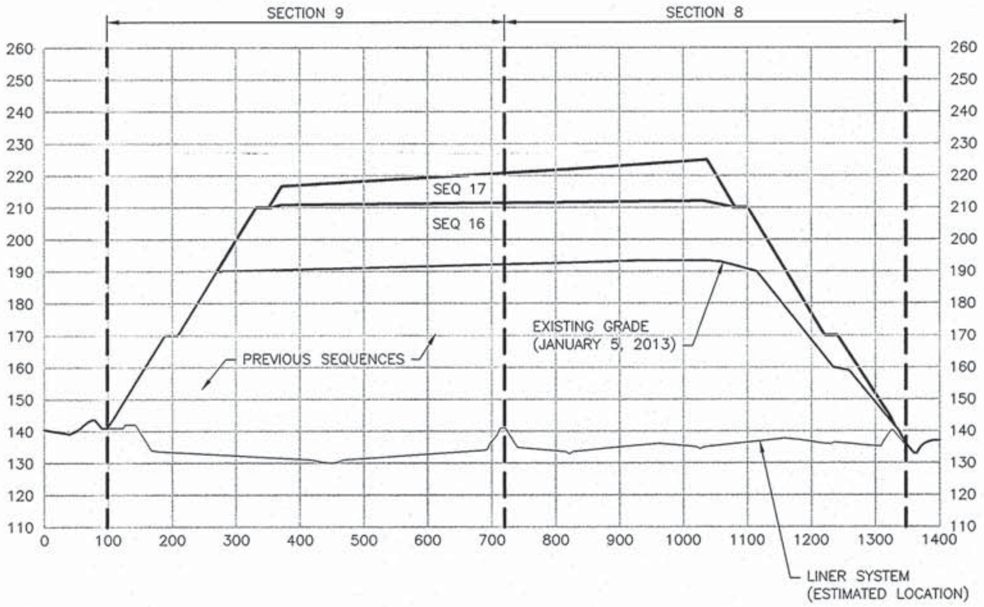
LANDFILL CROSS SECTION A 8



LANDFILL CROSS SECTION B 8



LANDFILL CROSS SECTION C 8



LANDFILL CROSS SECTION D 8

NOTE:
SOME SLOPES AND SEQUENCE SIZE VARY
DUE TO SECTION CUT ORIENTATION. SEE
SHEET C-08 FOR SECTION LOCATIONS.

Hillsborough County
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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 CROSS SECTIONS

0 1" 2"

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DWG No.
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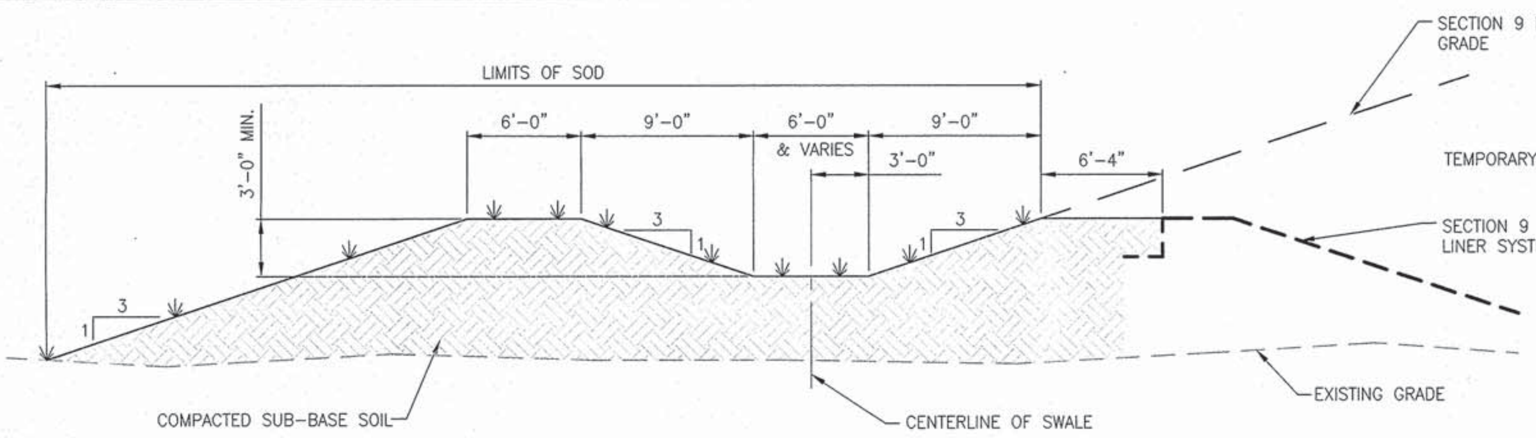
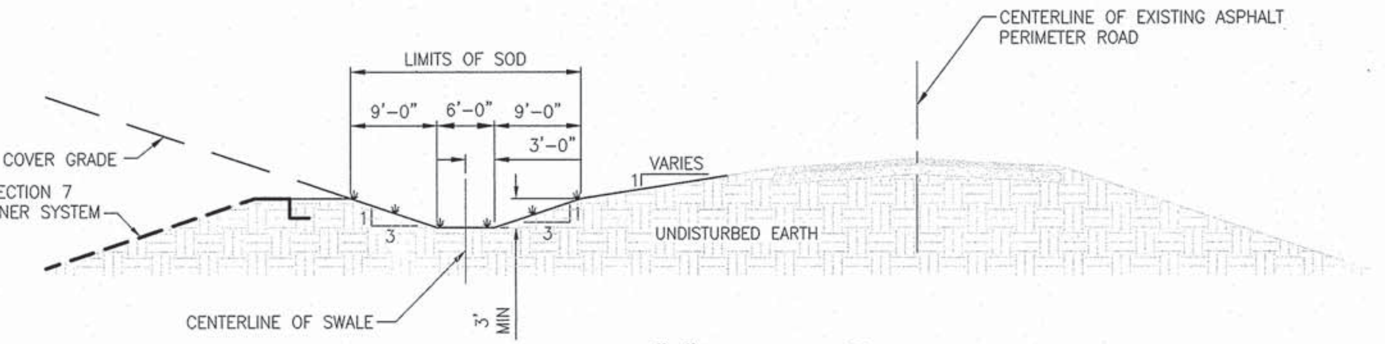


PERIMETER STORMWATER CHANNEL DETAIL

1
4

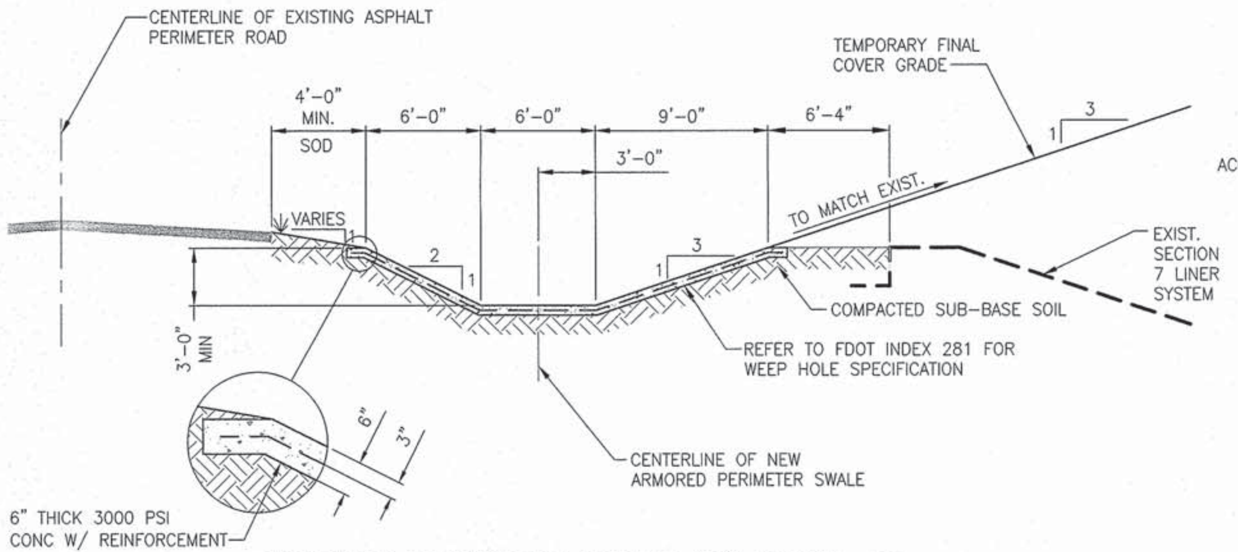
NOT TO SCALE



PERIMETER SWALE "A" DETAIL

2
4

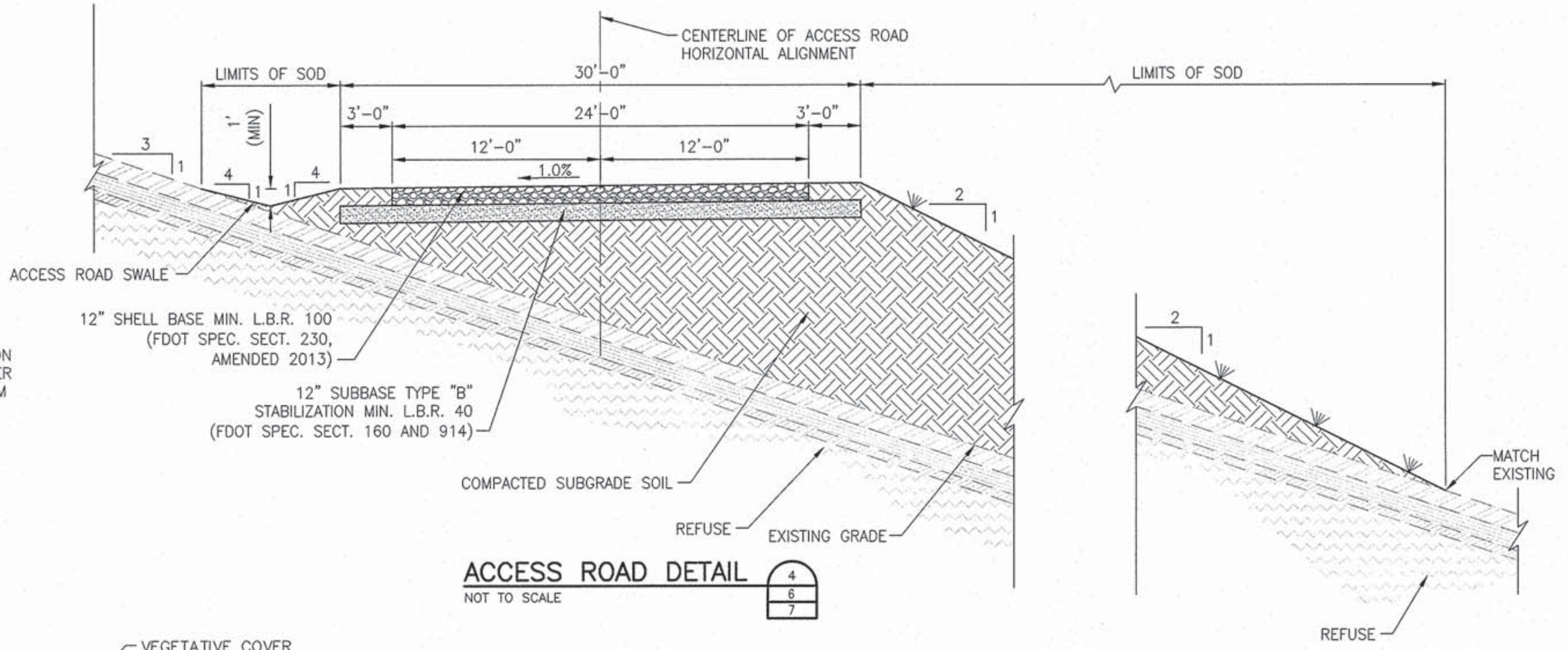
NOT TO SCALE



ARMORED PERIMETER SWALE "B" DETAIL

3
4

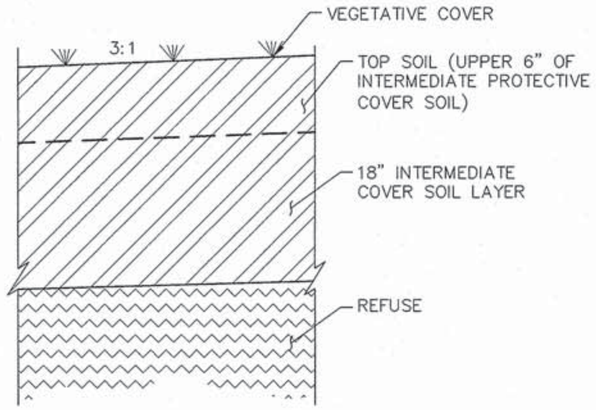
NOT TO SCALE

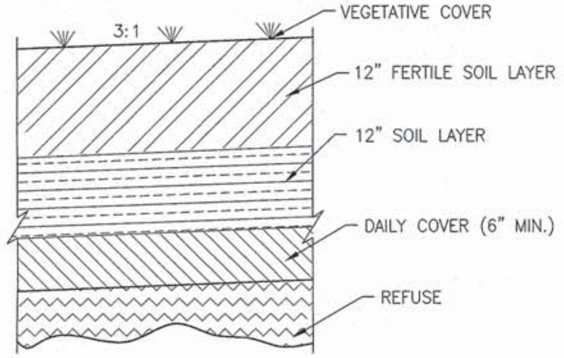


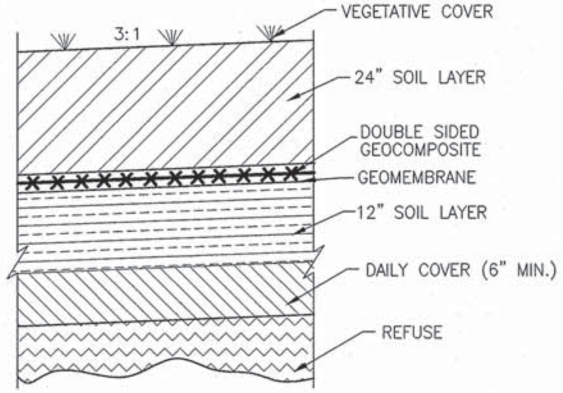
ACCESS ROAD DETAIL

4
6
7

NOT TO SCALE







TYPICAL TEMPORARY FINAL COVER DETAIL

5
7
11

NOT TO SCALE

TYPICAL INTERMEDIATE COVER DETAIL

6
7
11

NOT TO SCALE

FINAL COVER DETAIL

7
-

NOT TO SCALE

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
OPERATING SEQUENCE
SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 OPERATING SEQUENCE DETAILS

0 1" 2"

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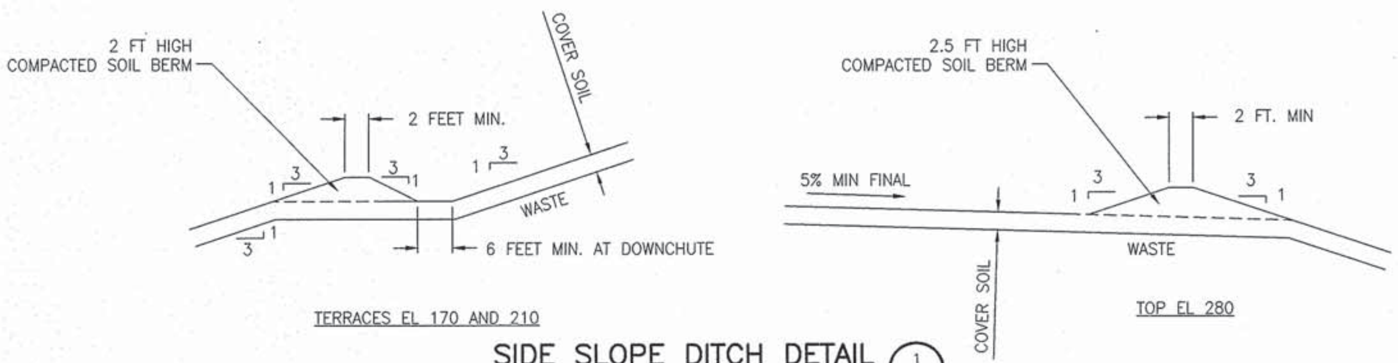
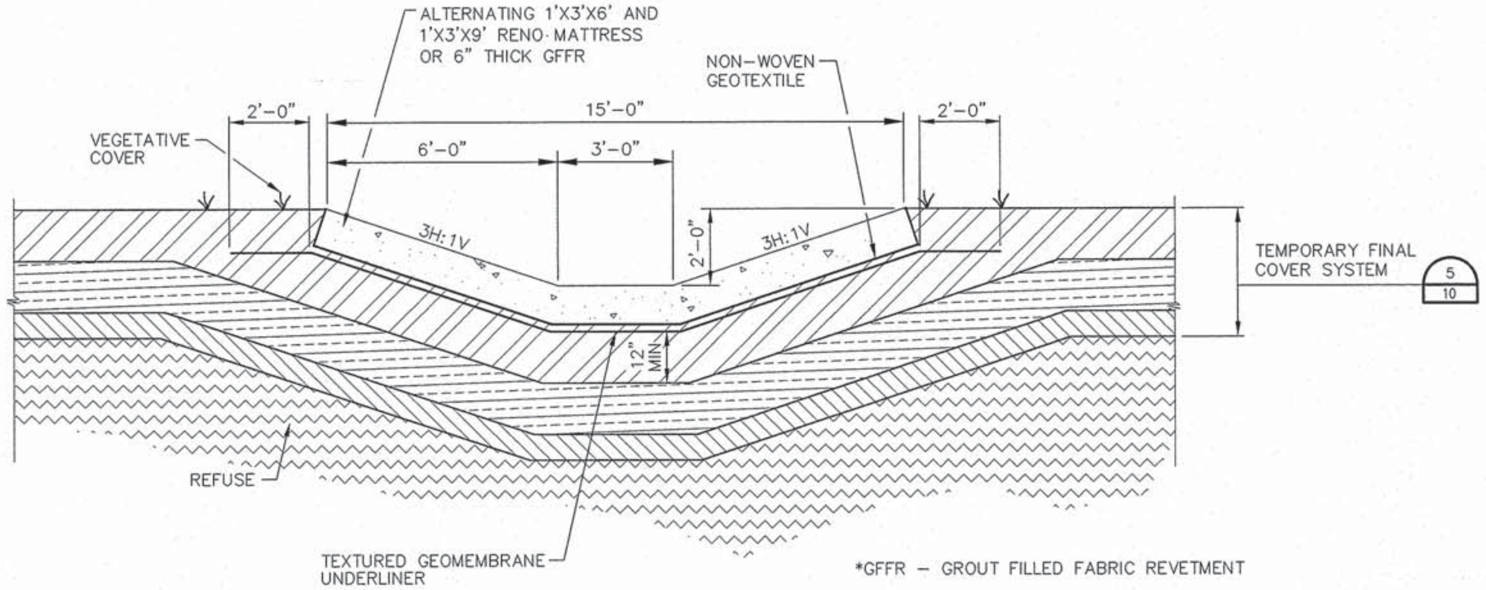


SIDE SLOPE DITCH DETAIL 1/7

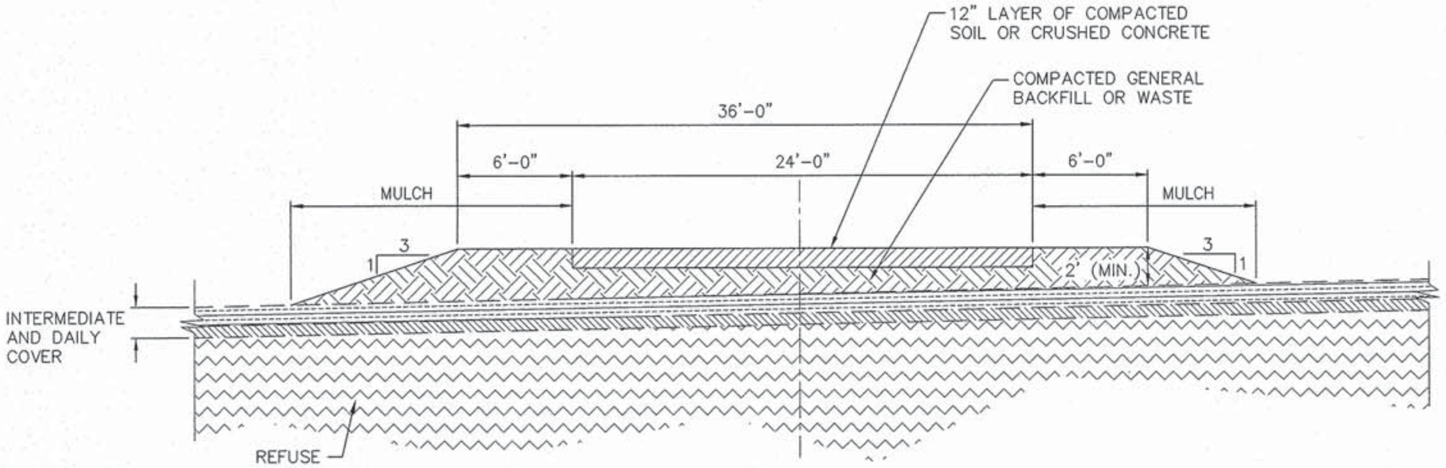
NOT TO SCALE



*GFFR – GROUT FILLED FABRIC REVETMENT

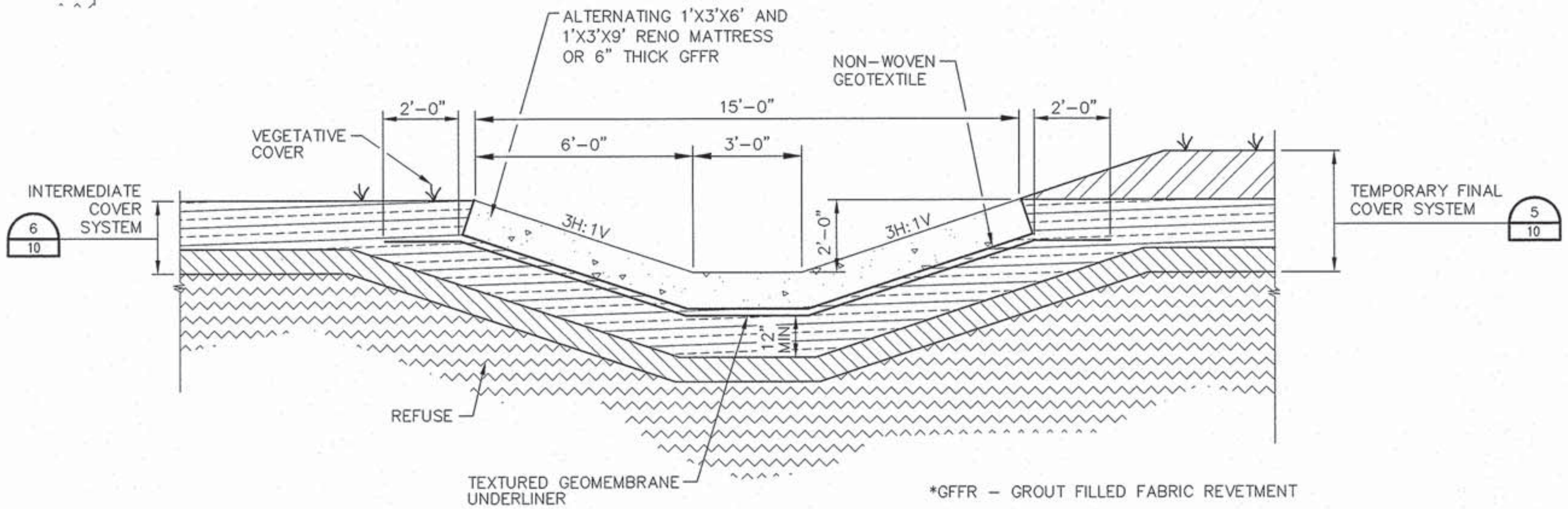
TYPICAL DOWNCHUTE DETAIL 2/4

NOT TO SCALE



TYPICAL TEMPORARY ACCESS ROAD DETAIL 3/5

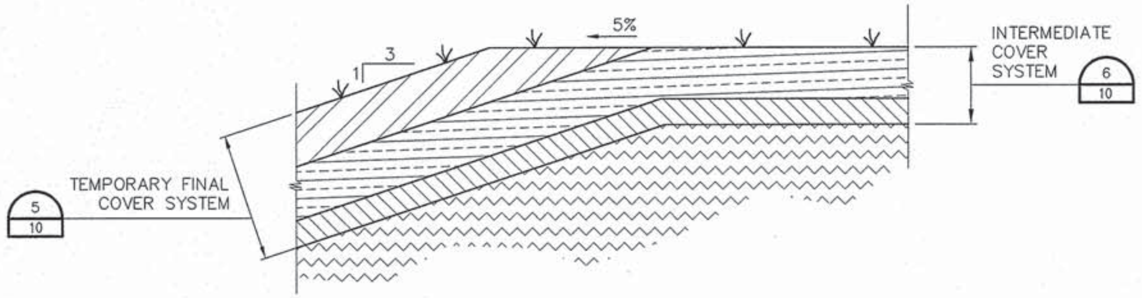
NOT TO SCALE



*GFFR – GROUT FILLED FABRIC REVETMENT

TRANSITION TEMPORARY FINAL COVER TO INTERMEDIATE COVER ON SIDE SLOPES 4/7

NOT TO SCALE



TRANSITION TEMPORARY FINAL COVER TO INTERMEDIATE FINAL COVER ON TOP SLOPE 5/7

NOT TO SCALE

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CAPACITY EXPANSION AREA
SECTIONS 7, 8 AND 9
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SOUTHEAST COUNTY LANDFILL
HILLSBOROUGH COUNTY, FLORIDA

SECTIONS 7, 8 AND 9 OPERATING SEQUENCE DETAILS

0 1" 2"

FILENAME	0844902101-C11-DET.DWG
SCALE	NOT TO SCALE
DWG No.	11

APPENDIX F

LANDFILL GAS MONITORING POINTS

HILLSBOROUGH COUNTY SOLID WASTE MANAGEMENT DEPARTMENT
SOUTHEAST COUNTY LANDFILL – LFG READINGS

ADMINISTRATION BUILDING

	Methane Gas	LEL	Carbon Dioxide	Oxygen	Balance Gas
SP-1					
SP-2					
SP-3					
SP-4					
SP-5					
SP-6					
SP-7					
SP-8					

MAINTENANCE BUILDING

	Methane Gas	LEL	Carbon Dioxide	Oxygen	Balance Gas
SP-9					
SP-10					
SP-11					
SP-12					

LEACHATE TREATMENT PLAN

	Methane Gas	LEL	Carbon Dioxide	Oxygen	Balance Gas
SP-13					
SP-14					
SP-15					

LANDFILL GAS PERIMETER MONITORING POINT

	Methane Gas	LEL	Carbon Dioxide	Oxygen	Balance Gas	Objectional Ambient Odor (Y/N)
LFG-1						Y/N
LFG-2						Y/N
LFG-3						Y/N
LFG-4						Y/N

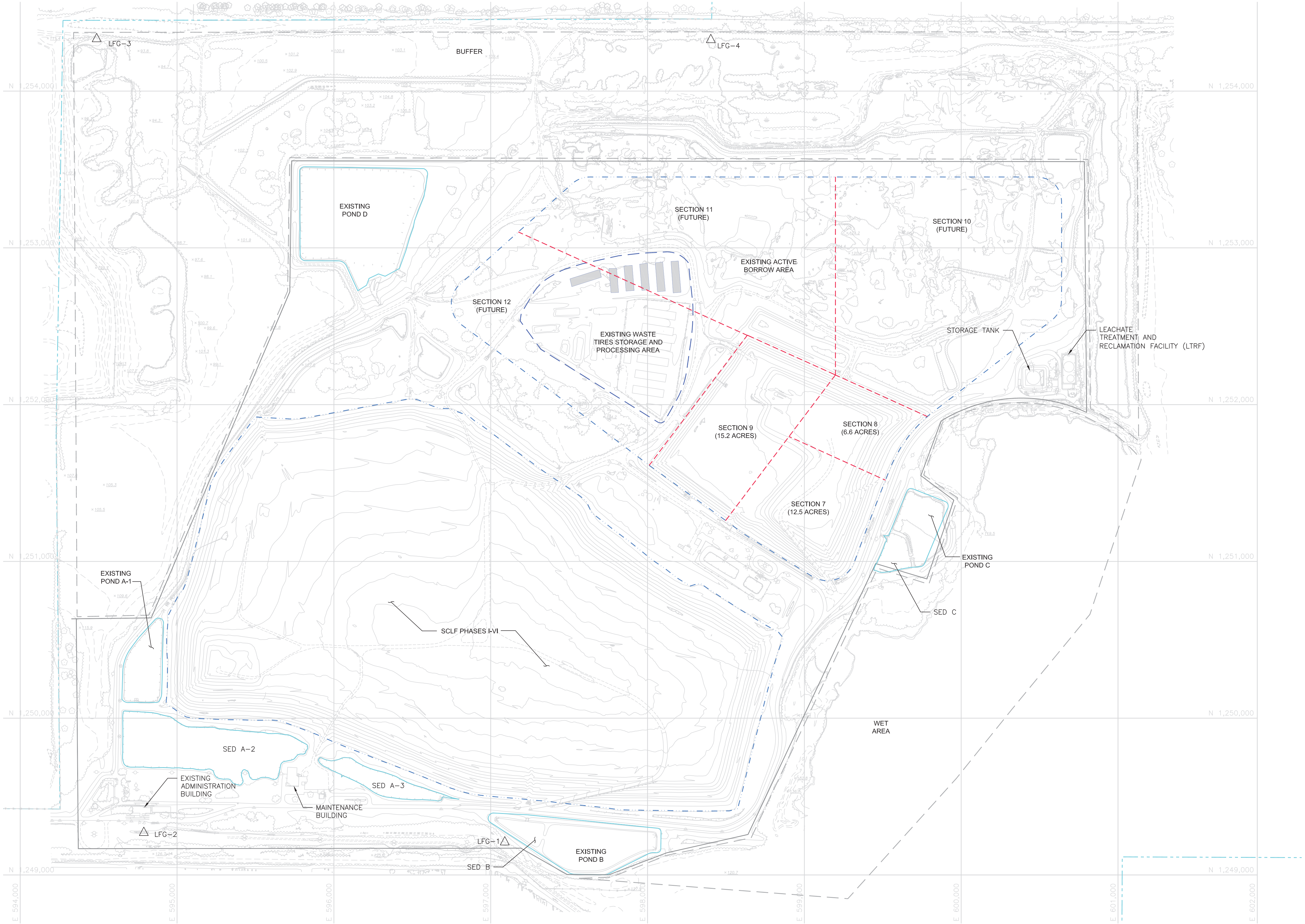
TECHNICIAN SIGNATURE: ________________________________

SUPERVISOR SIGNATURE: ________________________________

DATE: _________________________

COMMENTS: ____
____
____
____

Legend: SP = Ambient Sample Point



N

NOTE:

1. TOPOGRAPHICAL INFORMATION COMPILED FROM EXISTING CONDITIONS SURVEY PERFORMED BY PICKETT & ASSOCIATES DATE JULY 13, 2012.

LEGEND:

△ LFG−1	EXISTING LANDFILL PERIMETER GAS MONITORING WELL LOCATION
— · —	PROPERTY LINE
— ·· —	SOLID WASTE BOUNDARY
— — —	PHASE/SECTION LIMITS
— — —	EXISTING CLASS I LANDFILL FACILITIES
——	LANDFILL FOOTPRINT

300 150 0 300 600

SCALE: 1"=300'

HDR

LANDFILL GAS PERIMETER MONITORING WELLS

HILLSBOROUGH COUNTY, FLORIDA

FIGURE F-1

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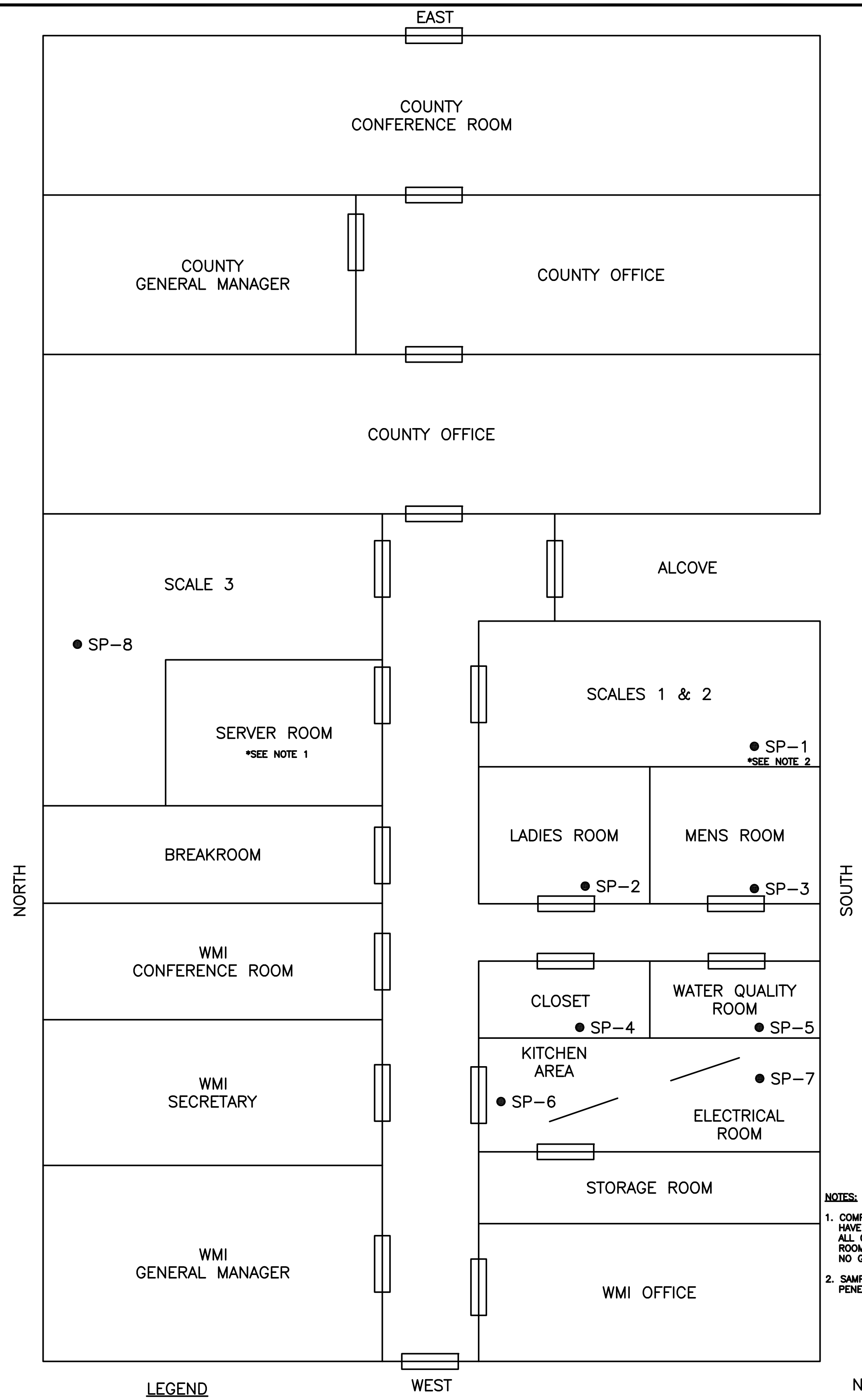

NOTES:

1. COMPUTER ROOM DOES NOT HAVE FLOOR PENETRATIONS. ALL CONDUITS ENTER THE ROOM THROUGH THE CEILING. NO GAS SAMPLING REQUIRED.
2. SAMPLE NEAR VACUUM TUBE PENETRATION IN FLOOR.

NOT TO SCALE

LEGEND

● SP-1 INDICATE GAS SAMPLE POINT

DOORWAY

ADMINISTRATION BUILDING LFG MONITORING POINTS

HILLSBOROUGH COUNTY, FLORIDA

FIGURE F-2

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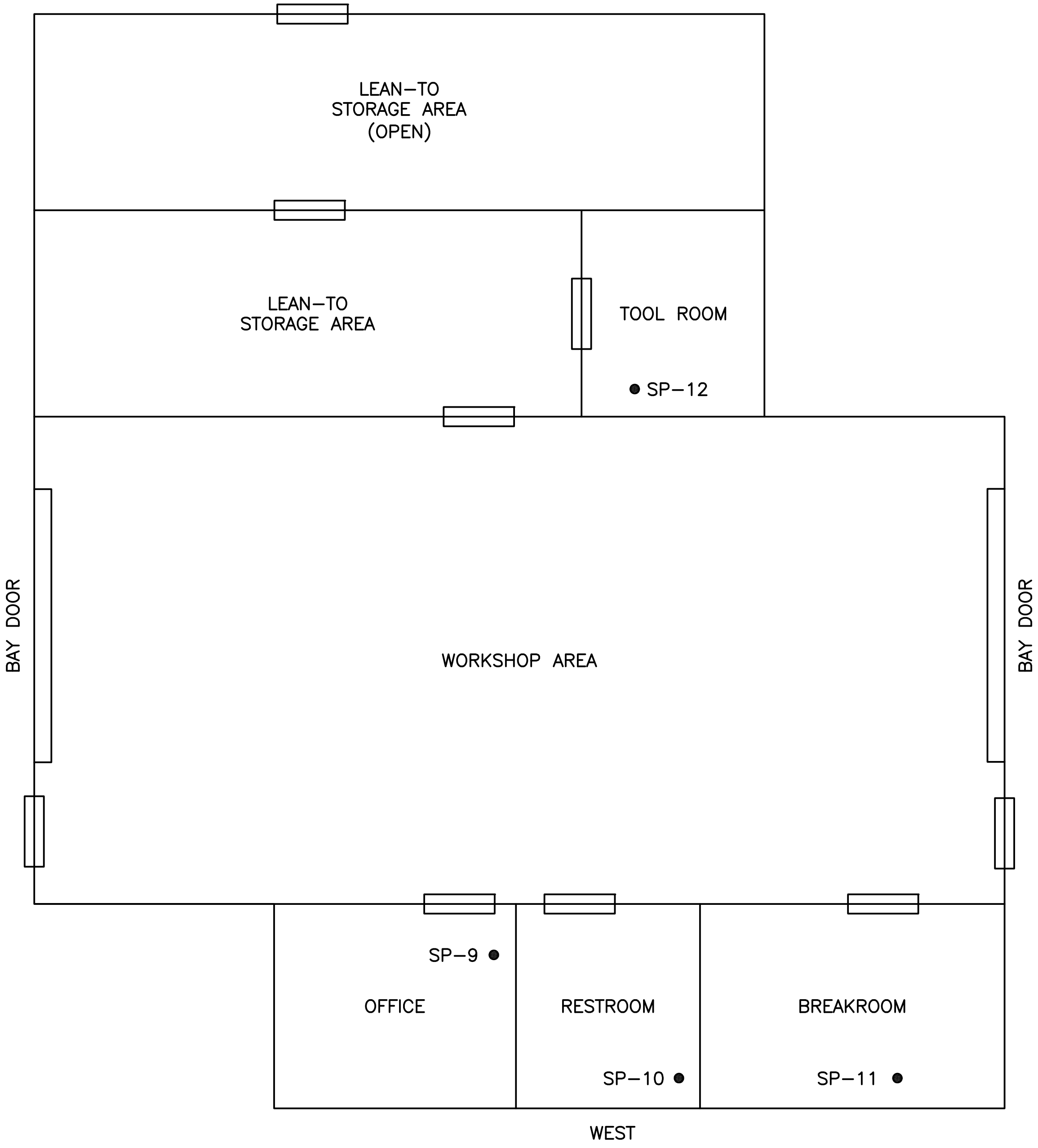

LEGEND

● SP−10 INDICATE GAS SAMPLE POINT

▭ DOORWAY

NOT TO SCALE

MAINTENANCE BUILDING
LFG MONITORING POINTS

HILLSBOROUGH COUNTY, FLORIDA

FIGURE
F-3

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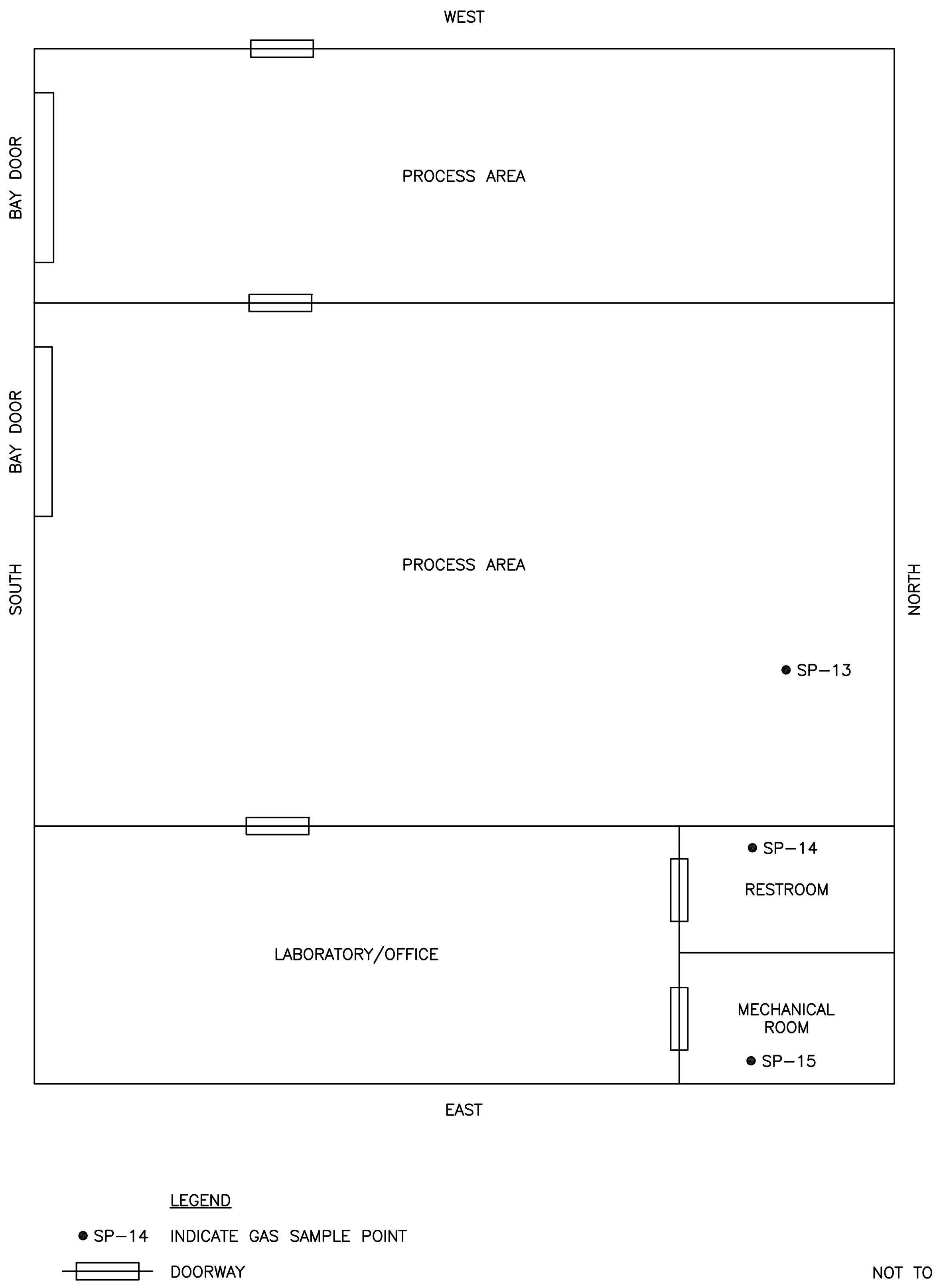

NOT TO SCALE

LTRF OFFICE
LFG MONITORING POINTS

HILLSBOROUGH COUNTY, FLORIDA

FIGURE F-4

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APPENDIX G

NOT USED

APPENDIX H

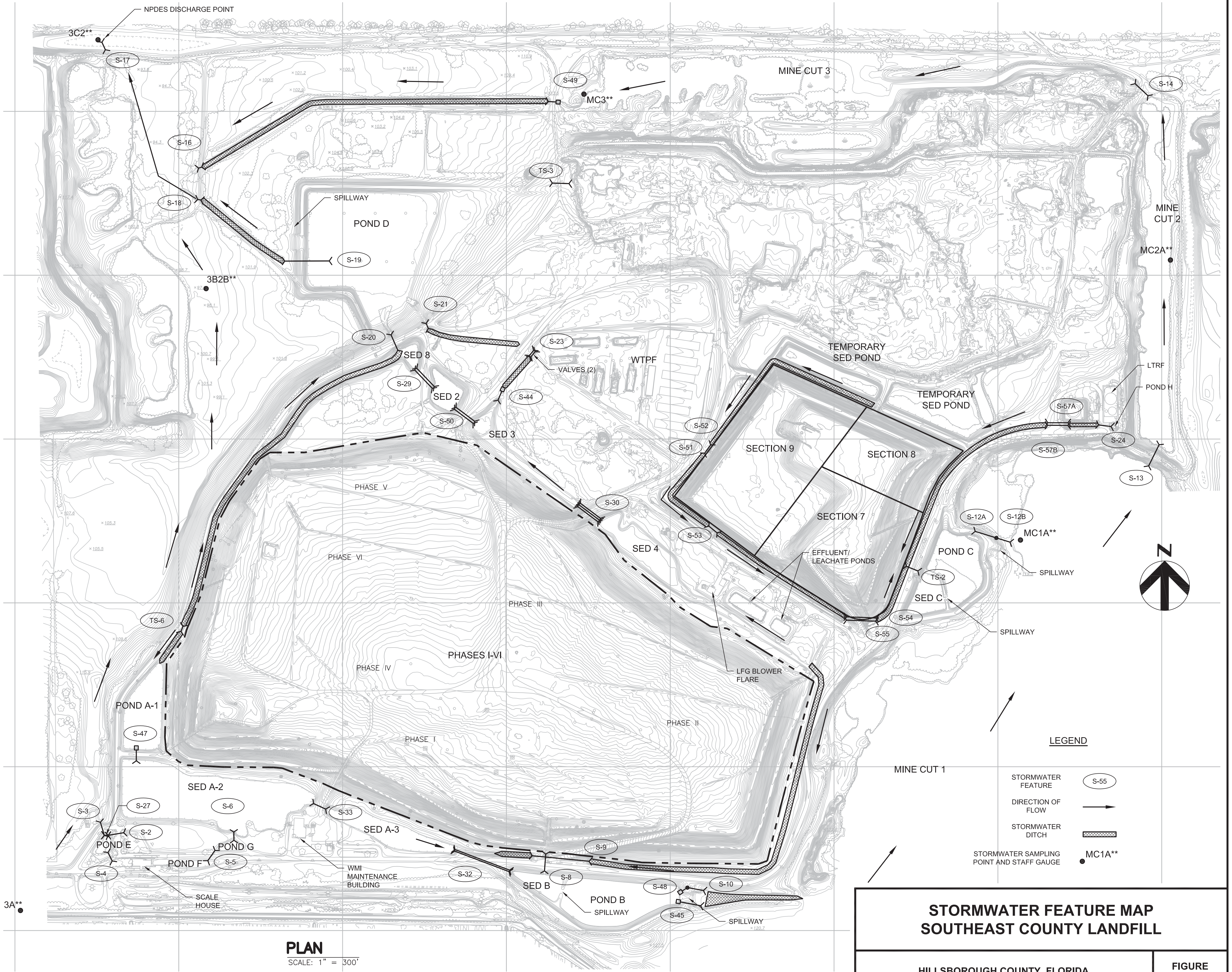
STORMWATER MANAGEMENT SYSTEM (SWMS) PLAN

EXISTING STORMWATER PIPE DATA TABLE

STRUCTURE NO	TYPE OF STRUCTURE	INVERT ELEVATION DOWNSTREAM	INVERT ELEVATION DOWNSTREAM	DIAMETER (IN)	LENGTH (FT)
S-2	ERCP	124.83 (E)	124.72 (W)	14x22	92.38
S-3	CMP	122.96 (S)	122.07 (N)	36.00	81.19
S-4	ERCP	124.98 (S)	124.91 (N)	14x22	47.87
S-5	ERCP	124.44 (N)	125.34 (S)	14x22	73.39
S-6	ERCP	124.63 (S)	124.08 (N)	14x22	50
S-8	ERCP	126.70 (S)	126.51 (N)	34x54	100.67
	ERCP	126.66 (S)	126.51 (N)	34x54	100.39
S-9	CMP	123.90 (W)	123.64 (E)	24.00	343.74
S-10	RCP	121.73 (E)	121.62 (W)	48.00	100.06
S-12A	RCP	121.79 (W)	121.35 (E)	30.00	169.40
S-12B	RCP	121.45 (W)	121.39 (E)	48.00	50.37
S-13	RCP	121.69 (S)	120.71 (N)	24.00	104.48
	RCP	121.75 (S)	120.86 (N)	24.00	104.56
S-14	RCP	120.35 (E)	118.806 (W)	24.00	104.90
	RCP	120.43 (E)	118.956 (W)	24.00	104.90
S-16	STEEL	94.87 (E)	94.62 (W)	24 (W)-21 (E)	22.04
	STEEL (E)-ECMP (W)	94.97 (E)	94.81 (W)	21 (E)-22x24 (W)	20.98
S-17	RCP	90.98 (N)	90.69 (S)	48.00	50.51
	RCP	90.87 (N)	90.62 (S)	48.00	50.71
S-18	CMP	95.47 (E)	95.09 (W)	18.00	19.89
S-19	RCP	101.16 (E)	100.91 (W)	48.00	161.35
S-20	CMP	115.32 (N)	114.60 (S)	48.00	90.98
	CMP	115.48 (N)	114.73 (S)	48.00	91.11
S-21	RCP	123.16 (N)	122.95 (S)	36.00	34.84
S-23	HDPE	130.20 (N)	130.00 (S)	8.00	41.00
	HDPE	130.20 (N)	130.00 (S)	8.00	41.00
S-24	ERCP	146.44 (E)	145.05 (W)	12x18	91.04
S-27	CMP	123.02 (E)	123.00 (W)	18.00	24.15
S-29	RCP	119.55 (E)	117.01 (W)	30.00	114.00
	RCP	119.55 (E)	117.01 (W)	30.00	114.00
S-30	RCP	124.96 (E)	125.02 (W)	36.00	119.00
	RCP	124.96 (E)	125.02 (W)	36.00	119.00
	RCP	124.96 (E)	125.02 (W)	36.00	119.00
S-32	ERCP	122.99 (W)	122.02 (E)	24x38	355.00
	ERCP	122.99 (W)	122.02 (E)	24x38	355.00
S-33	RCP	119.95 (W)	119.97 (E)	36.00	81.00
S-44	HDPE	127.11 (N)	125.10 (S)	8.00	60.00
	HDPE	127.11 (N)	125.10 (S)	8.00	60.00
S-45	RCP	121.99 (W)	121.94 (E)	36x60	75.00
S-47	RCP	120.94 (S)	120.01 (N)	30.00	66.00
S-48	RCP	121.67 (W)	121.68 (E)	48.00	29.00
S-49	RCP	107.00 (E)	106.83 (W)	42.00	48.00
S-50	RCP	122.10 (E)	120.07 (W)	30.00	108.00
	RCP	122.10 (E)	120.07 (W)	30.00	108.00
S-51	RCP	139.69 (N)	139.54 (S)	36.00	50
S-52	RCP	139.69 (N)	139.54 (S)	36.00	50
S-53	RCP	138.00 (W)	138.00 (E)	3x6 BOX	27
S-54	HDPE	132.17 (W)	131.41 (E)	30.00	175
S-55	HDPE	132.28 (W)	131.29 (E)	30.00	175
S-57A	RCP	143.23	142.23	24.00	136
S-57B	RCP	143.23	142.23	24.00	136
TS-2	BOX CULVERT	130.05 (W)	129.18 (E)	48x96	74.73
TS-3	RCP	129.007 (E)	128.157 (W)	18.00	98.07
TS-6	METAL	125.94 (N)	125.55 (S)	20.00	29.65
	CMP	125.90 (N)	125.68 (S)	36.00	19.59

300 150 0 300 600

SCALE: 1"=300'



PLAN

SCALE: 1" = 300'

STORMWATER FEATURE MAP
SOUTHEAST COUNTY LANDFILL

HILLSBOROUGH COUNTY, FLORIDA

FIGURE
1