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July 13, 2021

Cindy Mulkey
Program Administrator
Florida Department of Environmental Protection
Siting Coordination Office
2600 Blair Stone Road, MS 5500
Tallahassee, Florida 32399-2400

RE: Pasco County Public Infrastructure
West Pasco County Class I Landfill Ash Monofill
Slope Modification and Vertical Expansion
PA87-23G
Jones Edmunds Project No.: 16002-009-01

Dear Ms. Mulkey:

Pasco County (County) proposes to modify the Conditions of Certification that were issued under the Florida Electrical Power Plant Siting Act (PPSA) for the construction and operation of the Pasco County Resource Recovery Facility. This letter and the attached documents support the County's request to modify the Conditions with respect to the West Pasco County Landfill Class I Ash Monofill Cells A-1 through A-4.

The County proposes to modify the side slopes to three horizontals to one vertical (3H:1V) and to vertically expand Cells A-1, A-2 and A-3 by 21 feet to a final buildout elevation of 122 National Geodetic Vertical Datum (NAVD). The additional volume gained from these modifications will provide approximately 19 years of capacity for the County based on waste generation rates over the last several years.

These modifications will require that Subsection H in Section 3 of Attachment E in the Conditions of Certification be altered to reflect the final elevations proposed. Consequently, the Operation Plan provided as Appendix I to Attachment E of the Conditions of Certification will also be amended to reflect these changes.

If you have any questions or comments, please feel free to contact me at (352) 377-5821 or TMcKnight@JonesEdmunds.com.

Sincerely,

Tobin McKnight, PE, BCEE
Chief Engineer/Vice President
730 NE Waldo Road
Gainesville, Florida 32641

XC: Justin Roessler, PhD, PE, jroessler@pascocountyfl.net
El Kromhout, FDEP Tallahassee Solid Waste Section, elizabeth.kromhout@floridadep.gov

DISTRIBUTION LIST

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Florida Department of Environmental Protection
Southwest District Office
13051 N. Telecom Parkway
Temple Terrace, FL 33637

Florida Department of Environmental Protection
Solid Waste, MS 4565
2600 Blair Stone Rd.
Tallahassee, FL 32399-3000

Florida Department of Economic Development
Bureau of Community Planning and Growth
107 East Madison St.
Caldwell Building, MSC 160
Tallahassee, FL 32399-4120

Florida Fish & Wildlife Conservation Commission
Conservation Planning Services
620 South Meridian Street, MS 5B5
Tallahassee, FL 32399-1600
FWCConservationPlanningServices@myfwc.com

Florida Department of Transportation
District Administration
605 Suwannee Street
Tallahassee, FL 32399-0450

Florida Department of Agriculture and Consumer Services
Office of General Counsel
407 South Calhoun Street
Tallahassee, FL 32399-0800

Southwest Florida Water Management District
Office of General Counsel
2379 Broad Street
Brooksville, FL 34064-6899

Florida Department of State
Division of Historical Resources
500 S. Bronough Street, #402
Tallahassee, FL 32399-0250

PETITION FOR MODIFICATION OF THE

CONDITIONS OF CERTIFICATION

FOR THE

PASCO COUNTY

RESOURCE RECOVERY FACILITY

CONDITIONS OF CERTIFICATION NO. PA 87-23G

Submitted to:

Florida Department of Environmental Protection

Siting Coordination Office

2600 Blair Stone Road, MS #5500

Tallahassee, Florida 32399-3000

Prepared for:

Pasco County Public Infrastructure

Solid Waste & Resource Recovery

14230 Hays Road

Spring Hill, Florida 34610

Prepared by:

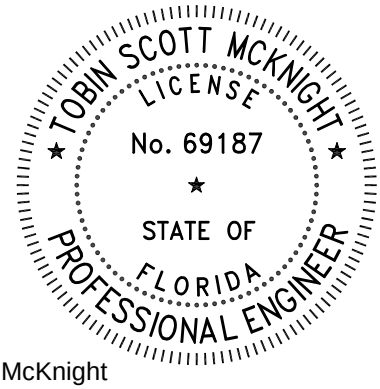
Jones Edmunds & Associates, Inc.

730 NE Waldo Road

Gainesville, FL 32641

Professional Engineering Certificate of Authorization #1841

July 2021



Tobin S McKnight
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Tobin S. McKnight, P.E., BCEE
Florida P.E. No. 69187

This item has been digitally signed and sealed by Tobin S. McKnight, PE on the date adjacent to the seal.

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1 INTRODUCTION

On August 24, 1988, the Florida Department of Environmental Protection (FDEP) Siting Board issued a Final Order of Certification (Case No. PA87-23) that authorized Pasco County to construct and operate the West Pasco County Solid Waste and Resource Recovery Facility. The Siting Board's approval was granted pursuant to the Florida Electrical Power Plant Siting Act (PPSA) and subject to Conditions of Certification (COC).

The County is petitioning to modify the COC for the Class I Landfill as described in the following sections of this document. This document constitutes the County's petition for a modification of the Facility's COC. This Petition is being submitted pursuant to Section 403.516(1)(c), Florida Statues, and Rule 62-17.211(1)(b), Florida Administrative Code (FAC).

2 THE WEST PASCO COUNTY SOLID WASTE AND RESOURCE RECOVERY FACILITY

The West Pasco County Solid Waste and Resource Recovery Facility is owned and operated by Pasco County Public Infrastructure and is at 14230 Hays Road in Spring Hill, Pasco County, Florida.

The 751-acre property has various facilities for processing and disposing of waste, such as a waste-to-energy (WTE) facility consisting of three municipal waste combustors, a Class I Landfill consisting of two solid waste cells, and an Ash Monofill with four disposal cells.

The Ash Monofill (Cells A-1, A-2, A-3, and A-4) is dedicated to ash disposal and receives only combustion residue (ash) from the on-site WTE facility. Ash that is generated from the combustion process is directed to the Ash Storage Building at the WTE facility, where it is periodically loaded within the building by a front-end loader into top-loading trucks. Once loaded, the trucks travel to the active ash disposal cell, where it is then off loaded.

The Class I Landfill receives non-processible or bypass solid waste when the WTE facility is either at capacity or is unavailable during maintenance. The Class I Landfill consists of two lined 10-acre cells – SW-1 and SW-2. Cell SW-1 was constructed in 1991 and was active through 2000; Cell SW-2 construction was completed in 2002 and is currently active.

3 PASCO COUNTY'S PROPOSED MODIFICATIONS

The County is currently filling ash in the northern half of Cell A-4, while A-1, A-2, A-3, and part of A-4 have temporary rain covers. The County's WTE operator, Covanta, is processing ash for metals recovery in portions of A-4. Cells A-1, A-2, and A-3 were previously filled in such a manner that there are unfilled valleys between these cells. The County is looking for more operational flexibility, to fill the valleys between the ash cells and to delay filling in the area where metals are being processed.

Currently, the final buildout of the ash cells is permitted with various side slopes from 3 horizontal to 1 vertical (3H:1V) to 5H:1V. To add additional capacity and increase the lifespan of the existing landfill, the County proposes to modify the side slopes to a 3H:1V

and to vertically expand Cells A-1, A-2, and A-3 by 21 feet to a final buildout elevation of 122 feet NAVD. The additional volume gained from these modifications would result in approximately 19 years of capacity for the County based on waste generation rates over the last several years.

4 ENVIRONMENTAL EFFECTS OF PASCO COUNTY'S PROPOSED MODIFCATIONS

The modifications proposed by the County will be conducted in compliance with all local, state, and federal laws that are applicable to the County's proposed activities. Consequently, the proposed modifications will not have any significant adverse impacts on the environment or the public as demonstrated in the Supporting Information of the Solid Waste Permit Application. Exhibit 1 provides the FDEP form *Application to Construct, Operate, Modify, or Close a Solid Waste Management Facility* and supporting information.

The proposed modifications to the Ash Monofill are minor revisions to the vertical extents and filling in valleys between existing ash cells. Since these changes do not increase the footprint of the landfill, no wetlands, environmentally sensitive land, or undeveloped land will be affected. Accordingly, no revisions are proposed to the Groundwater Quality Monitoring Requirements in Specific Condition I – *Department of Environmental Protection,* Subsection B – *Groundwater Quality Monitoring Plan,* of the COC.

The proposed vertical expansion of 21 feet to a final elevation of 122 feet NAVD is below the federal obstruction standards pursuant to 14 CFR Part 77, Subpart C; therefore, regarding Specific Condition I – *Department of Environmental Protection* subsection F – *Use of Air Space* of the COC, the applicable non-procedural requirements do not apply. Additionally, pursuant to Rule 62-701.320(13), FAC, affected airports within 5 miles of a lateral expansion of an existing landfill will be notified when the *Application to Construct, Operate, Modify, or Close a Solid Waste Management Facility* is filed with FDEP.

5 ENVIRONMENTAL BENEFITS OF PASCO COUNTY'S PROPOSED MODIFICATIONS

The County's proposed modifications to the Facility will aid in reducing the need to transport the bypass waste out of the county. These modifications will provide several environmental and public benefits by reducing the truck traffic, which will (a) eliminate the use of fossil fuels for hauling waste off site, (b) eliminate the greenhouse gas and other airborne emissions associated with the truck traffic, (c) eliminate the wear and tear on local roads associated with the truck traffic, and (d) eliminate the traffic congestions and increased risk of highway collisions associated with the truck traffic. This expansion will also increase the useful lifespan of this Facility resulting is decreased need to develop new areas for landfilling preserving these natural areas.

6 MODIFICATIONS TO PASCO COUNTY'S COC

The County proposes the following modifications to the Conditions of Certification, PA 87-23F, dated May 3, 2021:

Attachment E – *Solid Waste Construction and Operation Requirements, Section 3 – Operation, Subsection H – Landfill Elevation,* of the COC states:

> *"The final elevation for Ash Monofill A-4 shall not exceed +122 feet as shown on Sheets C-15 and C-15A of the plan set titled West Pasco Class I Landfill Cell A-4 Construction dated November 2008. The Final elevation of Solid Waste Cell SW-2 shall not exceed +101 feet as shown on Figure 7-1, Phasing Plan for Cells SW1 and SW2, of the approved Operation Plan (Appendix I)."*

The County proposes to alter the above referenced text from the COC to read:

> *"The final elevation for Ash Monofill* ***Cells A-1, A-2, A-3, and*** *A-4 shall not exceed +122 feet as shown* ***in*** ~~*on Sheets C-15 and C-15A of the plan set titled West Pasco Class I Landfill Cell A-4 Construction dated November 2008*~~ ***the approved Operation Plan (Appendix I)****. The Final elevation of Solid Waste Cell****s*** ~~*SW-2*~~ ***SW-1 and SW-2*** *shall not exceed* ~~*+101*~~ ***+121*** *feet as shown on Figure 7-1, Phasing Plan for Cells SW1 and SW2, of the approved Operation Plan (Appendix I)."*

Concurrently, the Operation Plan provided as Appendix I to Attachment E of the COC will be amended. The proposed alterations can be found in Appendix B of Exhibit 1.

7 CONCLUSION

The County respectfully requests that FDEP modify the COC for the Facility and thereby authorize the proposed vertical expansion, slope modification, and filling in the valleys of the Ash Monofill Solid Waste Cells A-1, A-2, A-3, and A-4.

Exhibit 1

Solid Waste Substantial Permit Modification Application

WEST PASCO RESOURCE RECOVERY FACILITY

ASH MONOFILL OPERATION MODIFICATION

SUBSTANTIAL MODIFICATION PERMIT APPLICATION

CONDITIONS OF CERTIFICATION NO. PA 87-23G

FDEP WACS NO. 45799

Submitted to:

Florida Department of Environmental Protection

Solid Waste Section

2600 Blair Stone Road, MS #4565

Tallahassee, Florida 32399

Prepared for:

Pasco County Public Infrastructure

Solid Waste & Resource Recovery Division

14230 Hays Road

Spring Hill, Florida 34610

Prepared by:

Jones Edmunds & Associates, Inc.

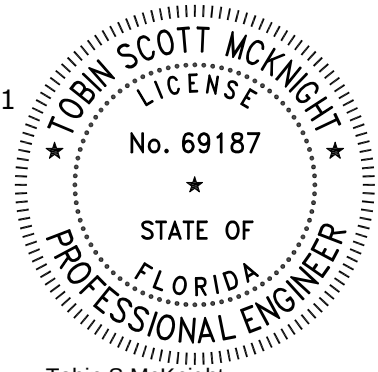
730 NE Waldo Road

Gainesville, FL 32641

Engineering Certificate of Authorization #1841

Geological Business #133

July 2021



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Tobin McKnight, PE, BCEE
Florida PE No. 69187

This item has been digitally signed and sealed by Tobin S. McKnight, PE on the date adjacent to the seal.

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Introduction

1 INTRODUCTION

This Petition for Modification and supporting information are being submitted on behalf of Pasco County Public Infrastructure's Solid Waste & Resource Recovery Division regarding the West Pasco County Landfill (WPCL) Ash Monofill cells. WPCL is at 14230 Hays Road, Spring Hill, Florida, just west of State Road 589 in Pasco County, in Section 25, Township 24S and Range 17E.

The Ash Monofill cells receive only combustion residue (ash) from the Resource Recovery Facility (RRF). Ash that is generated from the combustion process is directed to the dedicated ash storage building at the RRF, where it is periodically loaded into top-loading trucks. The loaded trucks travel to the active ash disposal cells, where they are off-loaded. The Ash Monofill is an essential component of the 751-acre site owned by Pasco County Public Infrastructure.

The Class I Landfill is permitted under the Florida Electrical Power Plant Siting Act and was originally designed to include a total of 16 cells – eight solid waste cells and eight ash monofill cells. Currently, four of the ash monofill cells (A-1, A-2, A-3, and A-4) have been constructed. Cells A-1 through A-3 are permitted to a final buildout of 101 feet National Geodetic Vertical Datum (ft-NGVD) with varying side slopes (3H:1V to 5H:1V). Cell A-4 is permitted to a final buildout of 122 ft-NGVD with varying side slopes (3H:1V to 5H:1V).

This petition is to modify the side slopes of WPCL Ash Monofill Cells A-1 through A-4 and to vertically expand Cells A-1 through A-3 to increase the overall capacity of the Class I Landfill. This application proposes a modification to the Conditions of Certification to operate the ash monofill, Florida Department of Environmental Protection (FDEP) Certification No. PA 87-23G. This permit application includes the *Application to Construct, Operate, Modify, or Close a Solid Waste Management Facility* (FDEP Form 62-701.900(1), FAC) and supporting information. The appendices include updated Engineering Plans, Operation Plan, and Closure and Long-Term Care Plan.

Application to Construct, Operate, Modify, or Close a Solid Waste Management Facility

FDEP Form 62-701.900(1)

Florida Department of Environmental Protection

Bob Martinez Center
2600 Blair Stone Road
Tallahassee, Florida 32399-2400

DEP Form #: 62-701.900(1), F.A.C.

Form Title: Application to Construct, Operate, Modify, or Close a Solid Waste Management Facility

Effective Date: February 15, 2015

Incorporated in Rule: 62-701.330(3), F.A.C.

STATE OF FLORIDA
DEPARTMENT OF ENVIRONMENTAL PROTECTION

APPLICATION TO CONSTRUCT, OPERATE, MODIFY, OR CLOSE A SOLID WASTE MANAGEMENT FACILITY

APPLICATION INSTRUCTIONS AND FORMS

Northwest District
160 Governmental Street
Suite 308
Pensacola, FL 32502-5794
850-595-8300

Northeast District
7777 Baymeadows Way West
Suite 100
Jacksonville, FL 32256-7590
904-256-1700

Central District
3319 Maguire Boulevard
Suite 232
Orlando, FL 32803-3767
407-897-4100

Southwest District
13051 North Telecom Pkwy
Temple Terrace, FL 33637
813-470-5700

South District
2295 Victoria Ave, Suite 364
P.O. Box 2549
Fort Myers, FL 33901-3881
239-344-5600

Southeast District
3301 Gun Club Road
MSC 7210-1
West Palm Beach, FL 33406
561-681-6600

INSTRUCTIONS TO APPLY FOR A SOLID WASTE MANAGEMENT FACILITY PERMIT

I. General

Solid Waste Management Facilities shall be permitted pursuant to Section 403.707, Florida Statutes (FS) and in accordance with Florida Administrative Code (FAC) Chapter 62-701. A permit application shall be submitted in accordance with the requirements of Rule 62-701.320(5)(a), F.A.C., to the appropriate Department office having jurisdiction over the facility. The appropriate fee in accordance with Rule 62-701.315, FAC, shall be submitted with the application by check made payable to the Department of Environmental Protection (DEP).

Complete appropriate sections for the type of facility for which application is made. Entries shall be typed or printed in ink. All blanks shall be filled in or marked "Not Applicable" or "No Substantial Change". Information provided in support of the application shall be marked "Submitted" and the location of this information in the application package indicated. The application shall include all information, drawings, and reports necessary to evaluate the facility. Information required to complete the application is listed on the attached pages of this form.

II. Application Parts Required for Construction and Operation Permits

A. Landfills and Ash Monofills - Submit Parts A through S
B. Asbestos Monofills - Submit Parts A, B, C, D, E, F, I, K, M, O through S
C. Industrial Solid Waste Disposal Facilities - Submit Parts A through S

NOTE: Portions of some Parts may not be applicable.

NOTE: For facilities that have been satisfactorily constructed in accordance with their construction permit, the information required for A, B and C type facilities does not have to be resubmitted for an operation permit if the information has not substantially changed during the construction period. The appropriate portion of the form should be marked "no substantial change".

III. Application Parts Required for Closure Permits

A. Landfills and Ash Monofills - Submit Parts A, B, L, N through S
B. Asbestos Monofills - Submit Parts A, B, M, O through S
C. Industrial Solid Waste Disposal Facilities - Submit Parts A, B, L through S

NOTE: Portions of some Parts may not be applicable.

IV. Permit Renewals

The above information shall be submitted at time of permit renewal in support of the new permit. However, facility information that was submitted to the Department to support the expiring permit, and which is still valid, does not need to be re-submitted for permit renewal. Portions of the application not re-submitted shall be marked "no substantial change" on the application form.

V. Application Codes

S	-	Submitted
LOCATION	-	Physical location of information in application
N/A	-	Not Applicable
N/C	-	No Substantial Change

VI. Listing of Application Parts

PART A: GENERAL INFORMATION

PART B: DISPOSAL FACILITY GENERAL INFORMATION

PART C: PROHIBITIONS

PART D: SOLID WASTE MANAGEMENT FACILITY PERMIT REQUIREMENTS, GENERAL

PART E: LANDFILL PERMIT REQUIREMENTS

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PART G: LANDFILL CONSTRUCTION REQUIREMENTS

PART H: HYDROGEOLOGICAL INVESTIGATION REQUIREMENTS

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PART J: VERTICAL EXPANSION OF LANDFILLS

PART K: LANDFILL OPERATION REQUIREMENTS

PART L: WATER QUALITY AND LEACHATE MONITORING REQUIREMENTS

PART M: SPECIAL WASTE HANDLING REQUIREMENTS

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PART O: LANDFILL CLOSURE REQUIREMENTS

PART P: OTHER CLOSURE PROCEDURES

PART Q: LONG-TERM CARE

PART R: FINANCIAL ASSURANCE

PART S: CERTIFICATION BY APPLICANT AND ENGINEER OR PUBLIC OFFICER

STATE OF FLORIDA
DEPARTMENT OF ENVIRONMENTAL PROTECTION
APPLICATION FOR A PERMIT TO CONSTRUCT, OPERATE, MODIFY OR CLOSE A SOLID WASTE MANAGEMENT FACILITY

Please Type or Print

PART A. GENERAL INFORMATION

1. Type of disposal facility (check all that apply):
   - ☑ Class I Landfill
   - ☑ Class III Landfill
   - ☐ Industrial Solid Waste
   - ☐ Other (describe):
   - ☑ Ash Monofill
   - ☐ Asbestos Monofill

NOTE: Waste Processing Facilities should apply on Form 62-701.900(4), FAC;
Yard Trash Disposal Facilities should notify on Form 62-701.900(3), FAC;
Compost Facilities should apply on Form 62-709.901(1), FAC; and
C&D Disposal Facilities should apply on Form 62-701.900(6), FAC

2. Type of application:
   - ☐ Construction
   - ☑ Operation
   - ☐ Construction/Operation
   - ☐ Closure
   - ☐ Long-term Care Only

3. Classification of application:
   - ☐ New
   - ☐ Renewal
   - ☑ Substantial Modification
   - ☐ Intermediate Modification
   - ☐ Minor Modification

4. Facility name: West Pasco County Class I Landfill

5. DEP ID number: 45799 County: Pasco

6. Facility location (main entrance):
   14230 Hays Road, Spring Hill, FL 34610

7. Location coordinates:
   Section: 24 Township: 24S Range: 17E
   Latitude: 28 ° 22 ' 42.23 " Longitude: 82 ° 33 ' 29.46 "
   Datum: WGS 84 Coordinate method: Google Earth
   Collected by: Thomas Le Blanc Company/Affiliation: Jones Edmunds

8. Applicant name (operating authority): Pasco County Public Infrastructure

Mailing address: 14230 Hays Road, Spring Hill, Florida 34610
Street or P.O. Box City State Zip

Contact person: Justin Roessler Telephone: (727) 857-2780

Title: Interim Solid Waste Director

JRoessler@PascoCountyFL.net
E-Mail address (if available)

9. Authorized agent/Consultant: Jones Edmunds & Associates, Inc.

Mailing address: 730 NE Waldo Road, Gainesville, FL 32641
Street or P.O. Box City State Zip

Contact person: Tobin McKnight, PE, BCEE Telephone: (352) 377-5821

Title: Chief Engineer/Vice President

TMcKnight@JonesEdmunds.com
E-Mail address (if available)

10. Landowner (if different than applicant): Same as applicant

Mailing address: _____
Street or P.O. Box City State Zip

Contact person: _____ Telephone: (____) _____

_____
E-Mail address (if available)

11. Cities, towns, and areas to be served:
Pasco County

12. Population to be served:
Current: 552,356 Five-Year Projection: 601,304

13. Date site will be ready to be inspected for completion: N/A

14. Expected life of the facility: 18.7 years

15. Estimated costs:
Total Construction: $ N/A Closing Costs: $ 12,672,293

16. Anticipated construction starting and completion dates:
From: N/A To: N/A

17. Expected volume or weight of waste to be received:
_____ yds^3/day 200 tons/day _____ gallons/day

PART B. DISPOSAL FACILITY GENERAL INFORMATION

1. Provide brief description of disposal facility design and operations planned under this application:

Provide the County with more operational flexibility for the ash cells to delay filling in the area where metals are being processed by filling the valleys between A-1, A-2, and A-3 and vertically expanding Cells A-1 through A-3 to match the final permitted elevation of Cell A-4.

2. Facility site supervisor: Justin Roessler

Title: Interim Solid Waste Director Telephone: (727) 857-2780

JRoessler@PascoCountyFL.net
E-Mail address (if available)

3. Disposal area: Total acres: 50 Used acres: ~45 Available acres: ~5

4. Weighing scales used: ☑ Yes ☐ No

5. Security to prevent unauthorized use: ☑ Yes ☐ No

6. Charge for waste received: _____ $/yds^3 72.08 $/ton

7. Surrounding land use, zoning:

- ☑ Residential
- ☐ Agricultural
- ☐ Commercial
- ☐ Industrial
- ☐ None
- ☑ Other (describe): Municipal

8. Types of waste received:

- ☑ Household
- ☑ Commercial
- ☑ Incinerator/WTE ash
- ☐ Treated biomedical
- ☐ Water treatment sludge
- ☐ Air treatment sludge
- ☐ Agricultural
- ☐ Asbestos
- ☑ C & D debris
- ☐ Shredded/cut tires
- ☐ Yard trash
- ☐ Septic tank
- ☐ Industrial
- ☐ Industrial sludge
- ☐ Domestic sludge
- ☑ Other (describe): Wood Chips, Biological Wastes

9. Salvaging permitted: ☐ Yes ☑ No

10. Attendant: ☑ Yes ☐ No Trained operator: ☑ Yes ☐ No

11. Trained spotters: ☑ Yes ☐ No Number of spotters used: 2

12. Site located in: ☐ Floodplain ☐ Wetlands ☐ Other (describe):
Uplands

13. Days of operation: Monday - Saturday

14. Hours of operation: 7:00AM - 5:00PM

15. Days working face covered: Daily

16. Elevation of water table: 33 ft. Datum Used: NGVD 29

17. Number of monitoring wells: 18

18. Number of surface monitoring points: 11

19. Gas controls used: ☐ Yes ☑ No Type controls: ☐ Active ☑ Passive

Gas flaring: ☐ Yes ☑ No Gas recovery: ☐ Yes ☑ No

20. Landfill unit liner type:

- ☐ Natural soils
- ☐ Single clay liner
- ☐ Single geomembrane
- ☐ Single composite
- ☐ Slurry wall
- ☑ Double geomembrane
- ☑ Geomembrane & composite
- ☐ Double composite
- ☐ None
- ☐ Other (describe):

A-1 - Double geomembrane liner

A-2 - Double Geomembrane liner with Geonet

A-3 & A-4 - Double geomembrane liner with geocomposite

21. Leachate collection method:

- ☑ Collection pipes
- ☑ Geonets
- ☐ Well points
- ☐ Perimeter ditch
- ☐ Other (describe):
- ☐ Double geomembrane
- ☐ Gravel layer
- ☐ Interceptor trench
- ☐ None

22. Leachate storage method:

☑ Tanks ☐ Surface impoundments

☐ Other (describe):

23. Leachate treatment method:

☐ Oxidation ☐ Chemical treatment

☐ Secondary ☐ Settling

☐ Advanced ☑ None

☐ Other (describe):

24. Leachate disposal method:

☐ Recirculated ☐ Pumped to WWTP

☑ Transported to WWTP ☐ Discharged to surface water/wetland

☐ Injection well ☐ Percolation ponds

☐ Evaporation ☐ Spray irrigation

☐ Other (describe):

25. For leachate discharged to surface waters:

Name and Class of receiving water:

N/A

26. Storm Water:

Collected: ☑ Yes ☐ No

Type of treatment:

Stormwater Retention Ponds

Name and Class of receiving water:

N/A

27. Environmental Resources Permit (ERP) number or status:

0320166-003-SA

PART C. PROHIBITIONS (62-701.300, FAC)

	LOCATION			
S ☑	Part C.1	N/A ☐	N/C ☐	1. Provide documentation that each of the siting criteria will be satisfied for the facility; (62-701.300(2), FAC)
S ☑	Part C.2	N/A ☐	N/C ☐	2. If the facility qualifies for any of the exemptions contained in Rules 62-701.300(12), (13) and (16) through (18), FAC, then document this qualification(s);
S ☑	Part C.3	N/A ☐	N/C ☐	3. Provide documentation that the facility will be in compliance with the burning restrictions; (62-701.300(3), FAC)
S ☑	Part C.4	N/A ☐	N/C ☐	4. Provide documentation that the facility will be in compliance with the hazardous waste restrictions; (62-701.300(4), FAC)
S ☑	Part C.5	N/A ☐	N/C ☐	5. Provide documentation that the facility will be in compliance with the PCB disposal restrictions; (62-701.300(5), FAC)
S ☑	Part C.6	N/A ☐	N/C ☐	6. Provide documentation that the facility will be in compliance with the biomedical waste restrictions; (62-701.300(6), FAC)
S ☑	Part C.7	N/A ☐	N/C ☐	7. Provide documentation that the facility will be in compliance with the Class I surface water restrictions; (62-701.300(7), FAC)
S ☑	Part C.8	N/A ☐	N/C ☐	8. Provide documentation that the facility will be in compliance with the special waste for landfills restrictions; (62-701.300(8), FAC)
S ☑	Part C.9	N/A ☐	N/C ☐	9. Provide documentation that the facility will be in compliance with the liquid restrictions; (62-701.300(10), FAC)
S ☑	Part C.10	N/A ☐	N/C ☐	10. Provide documentation that the facility will be in compliance with the used oil and oily waste restrictions; (62-701.300(11), FAC)
S ☑	Part C.11	N/A ☐	N/C ☐	11. Provide documentation that the facility will be in compliance with the CCA treated wood restrictions; (62-701.300(14), FAC)
S ☑	Part C.12	N/A ☐	N/C ☐	12. Provide documentation that the facility will be in compliance with the dust control restrictions; (62-701.300(15), FAC)

PART D. SOLID WASTE MANAGEMENT FACILITY PERMIT REQUIREMENTS, GENERAL (62-701.320, FAC)

	LOCATION		
S ☑	Part D.1	N/A ☐ N/C ☐	1. A minimum of one completed electronic application form, all supporting data and reports; (62-701.320(5)(a), FAC)
S ☑	Part D.2	N/A ☐ N/C ☐	2. Engineering and/or professional certification (signature, date, and seal) provided on the applications and all engineering plans, reports, and supporting information for the application; (62-701.320(6), FAC)
S ☑	Part D.3	N/A ☐ N/C ☐	3. A letter of transmittal to the Department; (62-701.320(7)(a), FAC)
S ☑	Part D.4	N/A ☐ N/C ☐	4. A completed application form dated and signed by the applicant; (62-701.320(7)(b), FAC)
S ☑	Part D.5	N/A ☐ N/C ☐	5. Permit fee specified in Rule 62-701.315, FAC in check or money order, payable to the Department; (62-701.320(7)(c), FAC)
S ☑	Part D.6	N/A ☐ N/C ☐	6. An engineering report addressing the requirements of this rule and with the following format: a cover sheet, text printed on 8 ½ inch by 11 inch consecutively numbered pages, a table of contents or index, the body of the report and all appendices including an operation plan, contingency plan, illustrative charts and graphs, records or logs of tests and investigations, engineering calculations; (62-701.320(7)(d), FAC)
S ☑	Part D.7	N/A ☐ N/C ☐	7. Operation Plan and Closure Plan; (62-701.320(7)(e)1, FAC)
S ☑	Part D.8	N/A ☐ N/C ☐	8. Contingency Plan; (62-701.320(7)(e)2, FAC)
S ☑	Part D.9	N/A ☐ N/C ☐	9. Plans or drawings for the solid waste management facilities in appropriate format (including sheet size restrictions, cover sheet, legends, north arrow, horizontal and vertical scales, elevations referenced to NGVD 1929) showing: (62-701.320(7)(f), FAC)
S ☑	Part D.9.a	N/A ☐ N/C ☐	a. A regional map or plan with the project location in relation to major roadways and population centers;
S ☑	Part D.9.b	N/A ☐ N/C ☐	b. A vicinity map or aerial photograph no more than one year old showing the facility site and relevant surface features located within 1000 feet of the facility;
S ☑	Part D.9.c	N/A ☐ N/C ☐	c. A site plan showing all property boundaries certified by a Florida Licensed Professional Surveyor and Mapper;
S ☑	Part D.9.d	N/A ☐ N/C ☐	d. Other necessary details to support the engineering report, including referencing elevations to a consistent, nationally recognized datum, and identifying the method used for collecting latitude and longitude data;

S ☐ Part D.10 N/A ☐ N/C ☑ 10. Documentation that the applicant either owns the property or has legal authority from the property owner to use the site; (62-701.320(7)(g), FAC)

S ☑ Part D.11 N/A ☐ N/C ☐ 11. For facilities owned or operated by a county, provide a description of how, if any, the facilities covered in this application will contribute to the county's achievement of the waste reduction and recycling goals contained in Section 403.706, FS; (62-701.320(7)(h), FAC)

S ☑ Part D.12 N/A ☐ N/C ☐ 12. Provide a history and description of any enforcement actions taken by the Department against the applicant for violations of applicable statutes, rules, orders, or permit conditions relating to the operation of any solid waste management facility in the state; (62-701.320(7)(i), FAC)

S ☑ Part D.13 N/A ☐ N/C ☐ 13. Proof of publication in a newspaper of general circulation of notice of application for a permit to construct or substantially modify a solid waste management facility; (62-701.320(8), FAC)

S ☑ Part D.14 N/A ☐ N/C ☐ 14. Provide a description of how the requirements for airport safety will be achieved, including proof of required notices if applicable. If exempt, explain how the exemption applies; (62-701.320(13), FAC)

S ☑ Part D.15 N/A ☐ N/C ☐ 15. Explain how the operator and spotter training requirements and special criteria will be satisfied for the facility; (62-701.320(15), FAC)

PART E. LANDFILL PERMIT REQUIREMENTS (62-701.330, FAC)

LOCATION

S ☑ Part E.1 N/A ☐ N/C ☐ 1. Regional map or aerial photograph no more than five years old showing all airports that are located within five miles of the proposed landfill; (62-701.330(3)(a), FAC)

S ☑ Part E.2 N/A ☐ N/C ☐ 2. Plot plan with a scale not greater than 200 feet to the inch showing: (62-701.330(3)(b), FAC)

S ☑ Part E.2.a N/A ☐ N/C ☐ a. Dimensions;

S ☑ Part E.2.b N/A ☐ N/C ☐ b. Locations of proposed and existing water quality monitoring wells;

S ☑ Part E.2.c N/A ☐ N/C ☐ c. Locations of soil borings;

S ☑ Part E.2.d N/A ☐ N/C ☐ d. Proposed plan of trenching or disposal areas;

S ☑ Part E.2.e N/A ☐ N/C ☐ e. Cross sections showing original elevations and proposed final contours which shall be included either on the plot plan or on separate sheets;

LOCATION		PART E CONTINUED
S ☑	Part E.2.f N/A ☐ N/C ☐	f. Any previously filled waste disposal areas;
S ☑	Part E.2.g N/A ☐ N/C ☐	g. Fencing or other measures to restrict access;
S ☑	Part E.3 N/A ☐ N/C ☐	3. Topographic maps with a scale not greater than 200 feet to the inch with five foot contour intervals showing: (62-701.330(3)(c), FAC)
S ☑	Part E.3.a N/A ☐ N/C ☐	a. Proposed fill areas;
S ☑	Part E.3.b N/A ☐ N/C ☐	b. Borrow areas;
S ☑	Part E.3.c N/A ☐ N/C ☐	c. Access roads;
S ☑	Part E.3.d N/A ☐ N/C ☐	d. Grades required for proper drainage;
S ☑	Part E.3.e N/A ☐ N/C ☐	e. Cross sections of lifts;
S ☑	Part E.3.f N/A ☐ N/C ☐	f. Special drainage devices if necessary;
S ☑	Part E.3.g N/A ☐ N/C ☐	g. Fencing;
S ☑	Part E.3.h N/A ☐ N/C ☐	h. Equipment facilities;
S ☑	Part E.4 N/A ☐ N/C ☐	4. A report on the landfill describing the following: (62-701.330(3)(d), FAC)
S ☑	Part E.4.a N/A ☐ N/C ☐	a. The current and projected population and area to be served by the proposed site;
S ☑	Part E.4.b N/A ☐ N/C ☐	b. The anticipated type, annual quantity, and source of solid waste expressed in tons;
S ☑	Part E.4.c N/A ☐ N/C ☐	c. Planned active life of the facility, the final design height of the facility, and the maximum height of the facility during its operation;
S ☑	Part E.4.d N/A ☐ N/C ☐	d. The source and type of cover material used for the landfill;
S ☑	Part E.5 N/A ☐ N/C ☐	5. Provide evidence that an approved laboratory shall conduct water quality monitoring for the facility in accordance with Chapter 62-160, FAC; (62-701.330(3)(g), FAC
S ☑	Part E.6 N/A ☐ N/C ☐	6. Provide a statement of how the applicant will demonstrate financial responsibility for the closing and long-term care of the landfill; (62-701.330(3)(h), FAC)

PART F. GENERAL CRITERIA FOR LANDFILLS (62-701.340, FAC)

	LOCATION		
S ☑	Part F.1	N/A ☐ N/C ☐	1. Describe (and show on a Federal Insurance Administration flood map, if available) how the landfill or solid waste disposal unit shall not be located in the 100 year floodplain where it will restrict the flow of the 100 year flood, reduce the temporary water storage capacity of the floodplain unless compensating storage is provided, or result in a washout of solid waste; (62-701.340(3)(b), FAC)
S ☑	Part F.2	N/A ☐ N/C ☐	2. Describe how the minimum horizontal separation between waste deposits in the landfill and the landfill property boundary shall be 100 feet, measured from the toe of the proposed final cover slope; (62-701.340(3)(c), FAC)

PART G. LANDFILL CONSTRUCTION REQUIREMENTS (62-701.400, FAC)

	LOCATION		
S ☐	Part G	N/A ☑ N/C ☐	1. Describe how the landfill shall be designed so the solid waste disposal units will be constructed and closed at planned intervals throughout the design period of the landfill, and shall be designed to achieve a minimum factor of safety of 1.5 using peak strength values to prevent failures of side slopes and deep-seated failures; (62-701.400(2), FAC)
S ☐	Part G	N/A ☑ N/C ☐	2. Landfill liner requirements; (62-701.400(3), FAC)
S ☐	Part G	N/A ☑ N/C ☐	a. General construction requirements; (62-701.400(3)(a), FAC)
S ☐	Part G	N/A ☑ N/C ☐	(1) Provide test information and documentation to ensure the liner will be constructed of materials that have appropriate physical, chemical, and mechanical properties to prevent failure;
S ☐	Part G	N/A ☑ N/C ☐	(2) Document foundation is adequate to prevent liner failure;
S ☐	Part G	N/A ☑ N/C ☐	(3) Constructed so bottom liner will not be adversely impacted by fluctuations of the ground water;
S ☐	Part G	N/A ☑ N/C ☐	(4) Designed to resist hydrostatic uplift if bottom liner located below seasonal high ground water table;
S ☐	Part G	N/A ☑ N/C ☐	(5) Installed to cover all surrounding earth which could come into contact with the waste or leachate;

LOCATION		PART G CONTINUED	
S ☐	Part G N/A ☑ N/C ☐	b. Composite liners; (62-701.400(3)(b), FAC)	
S ☐	Part G N/A ☑ N/C ☐	(1)	Upper geomembrane thickness and properties;
S ☐	Part G N/A ☑ N/C ☐	(2)	Design leachate head for primary leachate collection and removal system (LCRS) including leachate recirculation if appropriate;
S ☐	Part G N/A ☑ N/C ☐	(3)	Design thickness in accordance with Table A and number of lifts planned for lower soil component;
S ☐	Part G N/A ☑ N/C ☐	c. Double liners; (62-701.400(3)(c), FAC)	
S ☐	Part G N/A ☑ N/C ☐	(1)	Upper and lower geomembrane thickness and properties;
S ☐	Part G N/A ☑ N/C ☐	(2)	Design leachate head for primary LCRS to limit the head to one foot above the liner;
S ☐	Part G N/A ☑ N/C ☐	(3)	Lower geomembrane sub-base design;
S ☐	Part G N/A ☑ N/C ☐	(4)	Leak detection and secondary leachate collection system minimum design criteria (k ≥ 10 cm/sec, head on lower liner ≤ 1 inch, head not to exceed thickness of drainage layer);
S ☐	Part G N/A ☑ N/C ☐	d. Standards for geosynthetic components; (62-701.400(3)(d), FAC)	
S ☐	Part G N/A ☑ N/C ☐	(1)	Factory and field seam test methods to ensure all geomembrane seams achieve the minimum specifications;
S ☐	Part G N/A ☑ N/C ☐	(2)	Geomembranes to be used shall pass a continuous spark test by the manufacturer;
S ☐	Part G N/A ☑ N/C ☐	(3)	Design of 24-inch-thick protective layer above upper geomembrane liner;
S ☐	Part G N/A ☑ N/C ☐	(4)	Describe operational plans to protect the liner and leachate collection system when placing the first layer of waste above a 24-inch-thick protective layer;
S ☐	Part G N/A ☑ N/C ☐	(5)	HDPE geomembranes, if used, meet the specifications in GRI GM13, and LLDPE geomembranes, if used, meet the specifications in GRI GM17;
S ☐	Part G N/A ☑ N/C ☐	(6)	PVC geomembranes, if used, meet the specifications in PGI 1104;

LOCATION		PART G CONTINUED
S ☐ Part G N/A ☑ N/C ☐	(7)	Interface shear strength testing results of the actual components which will be used in the liner system;
S ☐ Part G N/A ☑ N/C ☐	(8)	Transmissivity testing results of geonets if they are used in the liner system;
S ☐ Part G N/A ☑ N/C ☐	(9)	Hydraulic conductivity testing results of geosynthetic clay liners if they are used in the liner system;
S ☐ Part G N/A ☑ N/C ☐	e. Geosynthetic specification requirements; (62-701.400(3)(e), FAC)	
S ☐ Part G N/A ☑ N/C ☐	(1)	Definition and qualifications of the designer, manufacturer, installer, QA consultant and laboratory, and QA program;
S ☐ Part G N/A ☑ N/C ☐	(2)	Material specifications for geomembranes, geocomposites, geotextiles, geogrids, and geonets;
S ☐ Part G N/A ☑ N/C ☐	(3)	Manufacturing and fabrication specifications including geomembrane raw material and roll QA, fabrication personnel qualifications, seaming equipment and procedures, overlaps, trial seams, destructive and non-destructive seam testing, seam testing location, frequency, procedure, sample size, and geomembrane repairs;
S ☐ Part G N/A ☑ N/C ☐	(4)	Geomembrane installation specifications including earthwork, conformance testing, geomembrane placement, installation personnel qualifications, field seaming and testing, overlapping and repairs, materials in contact with geomembranes, and procedures for lining system acceptance;
S ☐ Part G N/A ☑ N/C ☐	(5)	Geotextile and geogrids specifications including handling and placement, conformance testing, seams and overlaps, repair, and placement of soil materials and any overlying materials;
S ☐ Part G N/A ☑ N/C ☐	(6)	Geonet and geocomposites specifications including handling and placement, conformance testing, stacking and joining, repair, and placement of soil materials and any overlying materials;
S ☐ Part G N/A ☑ N/C ☐	(7)	Geosynthetic clay liner specifications including handling and placement, conformance testing, seams and overlaps, repair, and placement of soil materials and any overlying materials;

LOCATION		PART G CONTINUED
S ☐ Part G	N/A ☑ N/C ☐	f. Standards for soil liner components; (62-701.400(3)(f), FAC)
S ☐ Part G	N/A ☑ N/C ☐	(1) Description of construction procedures including over-excavation and backfilling to preclude structural inconsistencies and procedures for placing and compacting soil components in layers;
S ☐ Part G	N/A ☑ N/C ☐	(2) Demonstration of compatibility of the soil component with actual or simulated leachate in accordance with EPA Test Method 9100, or an equivalent test method;
S ☐ Part G	N/A ☑ N/C ☐	(3) Procedures for testing in situ soils to demonstrate they meet the specifications for soil liners;
S ☐ Part G	N/A ☑ N/C ☐	(4) Specifications for soil component of liner including at a minimum:
S ☐ Part G	N/A ☑ N/C ☐	(a) Allowable particle size distribution, and Atterberg limits including shrinkage limit;
S ☐ Part G	N/A ☑ N/C ☐	(b) Placement moisture and dry density criteria;
S ☐ Part G	N/A ☑ N/C ☐	(c) Maximum laboratory-determined saturated hydraulic conductivity using simulated leachate;
S ☐ Part G	N/A ☑ N/C ☐	(d) Minimum thickness of soil liner;
S ☐ Part G	N/A ☑ N/C ☐	(e) Lift thickness;
S ☐ Part G	N/A ☑ N/C ☐	(f) Surface preparation (scarification);
S ☐ Part G	N/A ☑ N/C ☐	(g) Type and percentage of clay mineral within the soil component;
S ☐ Part G	N/A ☑ N/C ☐	(5) Procedures for constructing and using a field test section to document the desired saturated hydraulic conductivity and thickness can be achieved in the field;
S ☐ Part G	N/A ☑ N/C ☐	g. If a Class III landfill is to be constructed with a bottom liner system, provide a description of how the minimum requirements for the liner will be achieved;

LOCATION		PART G CONTINUED
S ☐ Part G	N/A ☑ N/C ☐	3. Leachate collection and removal system (LCRS); (62-701.400(4), FAC)
S ☐ Part G	N/A ☑ N/C ☐	a. The primary and secondary LCRS requirements; (62-701.400(4)(a), FAC)
S ☐ Part G	N/A ☑ N/C ☐	(1) Constructed of materials chemically resistant to the waste and leachate;
S ☐ Part G	N/A ☑ N/C ☐	(2) Have sufficient mechanical properties to prevent collapse under pressure;
S ☐ Part G	N/A ☑ N/C ☐	(3) Have granular material or synthetic geotextile to prevent clogging;
S ☐ Part G	N/A ☑ N/C ☐	(4) Have a method for testing and cleaning clogged pipes or contingent designs for reducing leachate around failed areas;
S ☐ Part G	N/A ☑ N/C ☐	b. Other LCRS requirements; (62-701.400(4)(b), (c) and (d), FAC
S ☐ Part G	N/A ☑ N/C ☐	(1) Bottom 12 inches having hydraulic conductivity ≥ 1 x 10^{-3} cm/sec;
S ☐ Part G	N/A ☑ N/C ☐	(2) Total thickness of 24 inches of material chemically resistant to the waste and leachate;
S ☐ Part G	N/A ☑ N/C ☐	(3) Bottom slope design to accommodate for predicted settlement and still meet minimum slope requirements;
S ☐ Part G	N/A ☑ N/C ☐	(4) Demonstration that synthetic drainage material, if used, is equivalent or better than granular material in chemical compatibility, flow under load, and protection of geomembranes liner;
S ☐ Part G	N/A ☑ N/C ☐	(5) Schedule provided for routine maintenance of LCRS.
S ☐ Part G	N/A ☑ N/C ☐	4. Leachate recirculation; (62-701.400(5), FAC)
S ☐ Part G	N/A ☑ N/C ☐	a. Describe general procedures for recirculating leachate;
S ☐ Part G	N/A ☑ N/C ☐	b. Describe procedures for controlling leachate runoff and minimizing mixing of leachate runoff with storm water;
S ☐ Part G	N/A ☑ N/C ☐	c. Describe procedures for preventing perched water conditions and gas buildup;

LOCATION		PART G CONTINUED
S ☐ Part G	N/A ☑ N/C ☐	d. Describe alternate methods for leachate management when it cannot be recirculated due to weather or runoff conditions, surface seeps, wind-blown spray, or elevated levels of leachate head on the liner;
S ☐ Part G	N/A ☑ N/C ☐	e. Describe methods of gas management in accordance with Rule 62-701.530, FAC;
S ☐ Part G	N/A ☑ N/C ☐	f. If leachate irrigation is proposed, describe treatment methods and standards for leachate treatment prior to irrigation over final cover, and provide documentation that irrigation does not contribute significantly to leachate generation;
S ☐ Part G	N/A ☑ N/C ☐	5. Leachate storage tanks and leachate surface impoundments; (62-701.400(6), FAC)
S ☐ Part G	N/A ☑ N/C ☐	a. Surface impoundment requirements; (62-701.400(6)(b), FAC)
S ☐ Part G	N/A ☑ N/C ☐	(1) Documentation that the design of the bottom liner will not be adversely impacted by fluctuations of the ground water;
S ☐ Part G	N/A ☑ N/C ☐	(2) Designed in segments to allow for inspection and repair, as needed, without interruption of service;
S ☐ Part G	N/A ☑ N/C ☐	(3) General design requirements;
S ☐ Part G	N/A ☑ N/C ☐	(a) Double liner system consisting of an upper and lower 60-mil minimum thickness geomembrane;
S ☐ Part G	N/A ☑ N/C ☐	(b) Leak detection and collection system with hydraulic conductivity ≥ 1 cm/sec;
S ☐ Part G	N/A ☑ N/C ☐	(c) Lower geomembrane place on subbase ≥ 6 inches thick with $k \leq 1 \times 10^{-5}$ cm/sec or on an approved geosynthetic clay liner with $k \leq 1 \times 10^{-7}$ cm/sec;
S ☐ Part G	N/A ☑ N/C ☐	(d) Design calculation to predict potential leakage through the upper liner;
S ☐ Part G	N/A ☑ N/C ☐	(e) Daily inspection requirements, and notification and corrective action requirements if leakage rates exceed that predicted by design calculations;
S ☐ Part G	N/A ☑ N/C ☐	(4) Description of procedures to prevent uplift, if applicable;

LOCATION		PART G CONTINUED
S ☐ Part G	N/A ☑ N/C ☐	(5) Design calculations to demonstrate minimum two feet of freeboard will be maintained;
S ☐ Part G	N/A ☑ N/C ☐	(6) Procedures for controlling vectors and off-site odors;
S ☐ Part G	N/A ☑ N/C ☐	b. Above-ground leachate storage tanks; (62-701.400(6)(c), FAC)
S ☐ Part G	N/A ☑ N/C ☐	(1) Describe tank materials of construction and ensure foundation is sufficient to support tank;
S ☐ Part G	N/A ☑ N/C ☐	(2) Describe procedures for cathodic protection for the tank, if needed;
S ☐ Part G	N/A ☑ N/C ☐	(3) Describe exterior painting and interior lining of the tank to protect it from the weather and the leachate stored;
S ☐ Part G	N/A ☑ N/C ☐	(4) Describe secondary containment design to ensure adequate capacity will be provided and compatibility of materials of construction;
S ☐ Part G	N/A ☑ N/C ☐	(5) Describe design to remove and dispose of stormwater from the secondary containment system;
S ☐ Part G	N/A ☑ N/C ☐	(6) Describe an overfill prevention system, such as level sensors, gauges, alarms, and shutoff controls to prevent overfilling;
S ☐ Part G	N/A ☑ N/C ☐	(7) Inspections, corrective action, and reporting requirements;
S ☐ Part G	N/A ☑ N/C ☐	(a) Weekly inspection of overfill prevention system;
S ☐ Part G	N/A ☑ N/C ☐	(b) Weekly inspection of exposed tank exteriors;
S ☐ Part G	N/A ☑ N/C ☐	(c) Inspection of tank interiors when tank is drained, or at least every three years;
S ☐ Part G	N/A ☑ N/C ☐	(d) Procedures for immediate corrective action if failures detected;
S ☐ Part G	N/A ☑ N/C ☐	(e) Inspection reports available for Department review;
S ☐ Part G	N/A ☑ N/C ☐	c. Underground leachate storage tanks; (62-701.400(6)(d), FAC)

S ☐ Part G N/A ☑ N/C ☐ (1) Describe materials of construction;

S ☐ Part G N/A ☑ N/C ☐ (2) A double-walled tank design system to be used with the following requirements:

S ☐ Part G N/A ☑ N/C ☐ (a) Interstitial space monitoring at least weekly;

S ☐ Part G N/A ☑ N/C ☐ (b) Corrosion protection provided for primary tank interior and external surface of outer shell;

S ☐ Part G N/A ☑ N/C ☐ (c) Interior tank coatings compatible with stored leachate;

S ☐ Part G N/A ☑ N/C ☐ (d) Cathodic protection inspected weekly and repaired as needed;

S ☐ Part G N/A ☑ N/C ☐ (3) Describe an overfill prevention system, such as level sensors, gauges, alarms, and shutoff controls to prevent overfilling, and provide for weekly inspections;

S ☐ Part G N/A ☑ N/C ☐ (4) Inspection reports available for Department review;

S ☐ Part G N/A ☑ N/C ☐ 6. Liner systems construction quality assurance (CQA); (62-701.400(7), FAC)

S ☐ Part G N/A ☑ N/C ☐ a. Provide CQA Plan including:

S ☐ Part G N/A ☑ N/C ☐ (1) Specifications and construction requirements for liner system;

S ☐ Part G N/A ☑ N/C ☐ (2) Detailed description of quality control testing procedures and frequencies;

S ☐ Part G N/A ☑ N/C ☐ (3) Identification of supervising professional engineer;

S ☐ Part G N/A ☑ N/C ☐ (4) Identify responsibility and authority of all appropriate organizations and key personnel involved in the construction project;

S ☐ Part G N/A ☑ N/C ☐ (5) State qualifications of CQA professional engineer and support personnel;

S ☐ Part G N/A ☑ N/C ☐ (6) Description of CQA reporting forms and documents;

S ☐ Part G N/A ☑ N/C ☐ b. An independent laboratory experienced in the testing of geosynthetics to perform required testing;

S ☐ Part G N/A ☑ N/C ☐ 7. Soil liner CQA; (62-701.400(8), FAC)

S ☐ Part G N/A ☑ N/C ☐ a. Documentation that an adequate borrow source has been located with test results, or description of the field exploration and laboratory testing program to define a suitable borrow source;

S ☐ Part G N/A ☑ N/C ☐ b. Description of field test section construction and test methods to be implemented prior to liner installation;

S ☐ Part G N/A ☑ N/C ☐ c. Description of field test methods, including rejection criteria and corrective measures to insure proper liner installation;

S ☐ Part G N/A ☑ N/C ☐ 8. For surface water management systems at aboveground disposal units, provide documentation showing the design of any features intended to convey stormwater to a permitted or exempted treatment system; (62-701.400(9), FAC)

S ☐ Part G N/A ☑ N/C ☐ 9. Gas control systems; (62-701.400(10), FAC)

S ☐ Part G N/A ☑ N/C ☐ a. Provide documentation that if the landfill is receiving degradable wastes, it will have a gas control system complying with the requirements of Rule 62-701.530, FAC;

S ☐ Part G N/A ☑ N/C ☐ 10. For landfills designed in ground water, provide documentation that the landfill will provide a degree of protection equivalent to landfills designed with bottom liners not in contact with ground water; (62-701.400(11), FAC)

PART H. HYDROGEOLOGICAL INVESTIGATION REQUIREMENTS (62-701.410(2), FAC)

LOCATION

S ☐ Part H N/A ☑ N/C ☐ 1. Submit a hydrogeological investigation and site report including at least the following information:

S ☐ Part H N/A ☑ N/C ☐ a. Regional and site specific geology and hydrology;

S ☐ Part H N/A ☑ N/C ☐ b. Direction and rate of ground water and surface water flow including seasonal variations;

LOCATION		PART H CONTINUED
S ☐ Part H	N/A ☑ N/C ☐	c. Background quality of ground water and surface water;
S ☐ Part H	N/A ☑ N/C ☐	d. Any on-site hydraulic connections between aquifers;
S ☐ Part H	N/A ☑ N/C ☐	e. Site stratigraphy and aquifer characteristics for confining layers, semi-confining layers, and all aquifers below the site that may be affected by the disposal facility;
S ☐ Part H	N/A ☑ N/C ☐	f. Description of topography, soil types, and surface water drainage systems;
S ☐ Part H	N/A ☑ N/C ☐	g. Inventory of all public and private water wells within a one mile radius of the site including, where available, well top of casing and bottom elevations, name of owner, age and usage of each well, stratigraphic unit screened, well construction technique, and static water level;
S ☐ Part H	N/A ☑ N/C ☐	h. Identify and locate any existing contaminated areas on the site;
S ☐ Part H	N/A ☑ N/C ☐	i. Include a map showing the locations of all potable wells within 500 feet of the waste storage and disposal areas;
S ☐ Part H	N/A ☑ N/C ☐	2. Report signed, sealed, and dated by P.E. and/or P.G.

PART I. GEOTECHNICAL INVESTIGATION REQUIREMENTS (62-701.410(3) and (4), FAC)

LOCATION		
S ☑ Part I.1	N/A ☐ N/C ☐	1. Submit a geotechnical site investigation report defining the engineering properties of the site including at least the following:
S ☑ Part I.1.a	N/A ☐ N/C ☐	a. Description of subsurface conditions including soil stratigraphy and ground water table conditions;
S ☑ Part I.1.b	N/A ☐ N/C ☐	b. Investigate for the presence of muck, previously filled areas, soft ground, and lineaments;
S ☑ Part I.1.c	N/A ☐ N/C ☐	c. Estimates of average and maximum high water table across the site;
S ☑ Part I.1.d	N/A ☐ N/C ☐	d. Evaluation of potential for fault areas and seismic impact zones;
S ☑ Part I.1.e	N/A ☐ N/C ☐	e. Foundation analysis including:

S ☑	Part I.1.e(1)	N/A ☐ N/C ☐	(1) Foundation bearing capacity analysis;
S ☑	Part I.1.e(2)	N/A ☐ N/C ☐	(2) Total and differential subgrade settlement analysis;
S ☑	Part I.1.e(3)	N/A ☐ N/C ☐	(3) Slope stability analysis;
S ☑	Part I.1.f	N/A ☐ N/C ☐	f. Evaluation of potential for sinkholes and sinkhole activity at the site that is based upon the investigations required in Rule 62-701.410(3)(f), F.A.C.;
S ☑	Part I.1.g	N/A ☐ N/C ☐	g. A geotechnical report providing a description of methods used in the investigation, and includes soil boring logs, laboratory results, analytical calculations, cross sections, interpretations, conclusions, and a description of any engineering measures proposed for the site;
S ☑	Part I.2	N/A ☐ N/C ☐	2. Report signed, sealed, and dated by P.E. and/or P.G.

PART J. VERTICAL EXPANSION OF LANDFILLS (62-701.430, FAC)

	LOCATION		
S ☑	Part J.1	N/A ☐ N/C ☐	1. Describe how the vertical expansion shall not cause or contribute to any violations of water quality standards or criteria, shall not cause objectionable odors, or adversely affect the closure design of the existing landfill;
S ☐	Part J.2	N/A ☑ N/C ☐	2. Describe how the vertical expansion over unlined landfills will meet the requirements of Rule 62-701.400, FAC with the exceptions of Rule 62-701.430(1)(c), FAC;
S ☑	Part J.3	N/A ☐ N/C ☐	3. Provide foundation and settlement analysis for the vertical expansion;
S ☑	Part J.4	N/A ☐ N/C ☐	4. Provide total settlement calculations demonstrating that the final elevations of the lining system, gravity drainage, and no other component of the design will be adversely affected;
S ☑	Part J.5	N/A ☐ N/C ☐	5. Minimum stability factor of safety of 1.5 for the lining system component interface stability and for deep stability;
S ☐	Part J.6	N/A ☑ N/C ☐	6. Provide documentation to show the surface water management system will not be adversely affected by the vertical expansion;
S ☐	Part J.7	N/A ☑ N/C ☐	7. Provide gas control designs to prevent accumulation of gas under the new liner for the vertical expansion;

PART K. LANDFILL OPERATION REQUIREMENTS (62-701.500, FAC)

LOCATION		
S ☐ Part K	N/A ☐ N/C ☑	1. Provide documentation that the landfill will have at least one trained operator during operation and at least one trained spotter at each working face; (62-701.500(1), FAC)
S ☐ Part K	N/A ☐ N/C ☑	2. Provide a landfill operation plan including procedures for: (62-701.500(2), FAC)
S ☐ Part K	N/A ☐ N/C ☑	a. Designating responsible operating and maintenance personnel;
S ☐ Part K	N/A ☐ N/C ☑	b. Emergency preparedness and response, as required in subsection 62-701.320(16), FAC;
S ☐ Part K	N/A ☐ N/C ☑	c. Controlling types of waste received at the landfill;
S ☐ Part K	N/A ☐ N/C ☑	d. Weighing incoming waste;
S ☐ Part K	N/A ☐ N/C ☑	e. Vehicle traffic control and unloading;
S ☐ Part K	N/A ☐ N/C ☑	f. Method and sequence of filling waste;
S ☐ Part K	N/A ☐ N/C ☑	g. Waste compaction and application of cover;
S ☐ Part K	N/A ☐ N/C ☑	h. Operations of gas, leachate, and stormwater controls;
S ☐ Part K	N/A ☐ N/C ☑	i. Water quality monitoring;
S ☐ Part K	N/A ☐ N/C ☑	j. Maintaining and cleaning the leachate collection system;
S ☐ Part K	N/A ☐ N/C ☑	3. Provide a description of the landfill operation record to be used at the landfill, details as to location of where various operational records will be kept (i.e. DEP permit, engineering drawings, water quality records, etc.); (62-701.500(3), FAC)
S ☐ Part K	N/A ☐ N/C ☑	4. Describe the waste records that will be compiled monthly and provided to the Department annually; (62-701.500(4), FAC)
S ☐ Part K	N/A ☐ N/C ☑	5. Describe methods of access control; (62-701.500(5), FAC)
S ☐ Part K	N/A ☐ N/C ☑	6. Describe load checking program to be implemented at the landfill to discourage disposal of unauthorized waste at the landfill; (62-701.500(6), FAC)

LOCATION		PART K CONTINUED
S ☐ Part K N/A ☐ N/C ☑		7. Describe procedures for spreading and compacting waste at the landfill that include: (62-701.500(7), FAC)
S ☐ Part K N/A ☐ N/C ☑		a. Waste layer thickness and compaction frequencies;
S ☐ Part K N/A ☐ N/C ☑		b. Special considerations for first layer of waste placed above the liner and leachate collection system;
S ☐ Part K N/A ☐ N/C ☑		c. Slopes of cell working face and side grades above land surface, and planned lift depths during operation;
S ☐ Part K N/A ☐ N/C ☑		d. Maximum width of working face;
S ☐ Part K N/A ☐ N/C ☑		e. Description of type of initial cover to be used at the facility that controls:
S ☐ Part K N/A ☐ N/C ☑		(1) Vector breeding/animal attraction;
S ☐ Part K N/A ☐ N/C ☑		(2) Fires;
S ☐ Part K N/A ☐ N/C ☑		(3) Odors;
S ☐ Part K N/A ☐ N/C ☑		(4) Blowing litter;
S ☐ Part K N/A ☐ N/C ☑		(5) Moisture infiltration;
S ☐ Part K N/A ☐ N/C ☑		f. Procedures for applying initial cover, including minimum cover frequencies;
S ☐ Part K N/A ☐ N/C ☑		g. Procedures for applying intermediate cover;
S ☐ Part K N/A ☐ N/C ☑		h. Time frames for applying final cover;
S ☐ Part K N/A ☐ N/C ☑		i. Procedures for controlling scavenging and salvaging;
S ☐ Part K N/A ☐ N/C ☑		j. Description of litter policing methods;
S ☐ Part K N/A ☐ N/C ☑		k. Erosion control procedures;

LOCATION		PART K CONTINUED
S ☐ Part K N/A ☐ N/C ☑		8. Describe operational procedures for leachate management including: (62-701.500(8), FAC)
S ☐ Part K N/A ☐ N/C ☑		a. Leachate level monitoring;
S ☐ Part K N/A ☐ N/C ☑		b. Operation and maintenance of leachate collection and removal system, and treatment as required;
S ☐ Part K N/A ☐ N/C ☑		c. Procedures for managing leachate if it becomes regulated as a hazardous waste;
S ☐ Part K N/A ☐ N/C ☑		d. Identification of treatment or disposal facilities that may be used for off-site discharge and treatment of leachate;
S ☐ Part K N/A ☐ N/C ☑		e. Contingency plan for managing leachate during emergencies or equipment problems;
S ☐ Part K N/A ☐ N/C ☑		f. Procedures for recording quantities of leachate generated in gal/day and including this in the operating record;
S ☐ Part K N/A ☐ N/C ☑		g. Procedures for comparing precipitation experienced at the landfill with leachate generation rates and including this information in the operating record;
S ☐ Part K N/A ☐ N/C ☑		h. Procedures for water pressure cleaning or video inspecting leachate collection systems;
S ☐ Part K N/A ☐ N/C ☑		9. Describe how the landfill receiving degradable wastes shall implement a gas management system meeting the requirements of Rule 62-701.530, FAC; (62-701.500(9), FAC)
S ☐ Part K N/A ☐ N/C ☑		10. Describe procedures for operating and maintaining the landfill stormwater management system to comply with the requirements of Rule 62-701.400(9), FAC; (62-701.500(10), FAC)
S ☐ Part K N/A ☐ N/C ☑		11. Equipment and operation feature requirements; (62-701.500(11), FAC)
S ☐ Part K N/A ☐ N/C ☑		a. Sufficient equipment for excavating, spreading, compacting, and covering waste;
S ☐ Part K N/A ☐ N/C ☑		b. Reserve equipment or arrangements to obtain additional equipment within 24 hours of breakdown;
S ☐ Part K N/A ☐ N/C ☑		c. Communications equipment;

LOCATION		PART K CONTINUED
S ☐ Part K N/A ☐ N/C ☑		d. Dust control methods;
S ☐ Part K N/A ☐ N/C ☑		e. Fire protection capabilities and procedures for notifying local fire department authorities in emergencies;
S ☐ Part K N/A ☐ N/C ☑		f. Litter control devices;
S ☐ Part K N/A ☐ N/C ☑		g. Signs indicating operating authority, traffic flow, hours of operation, and disposal restrictions;
S ☐ Part K N/A ☐ N/C ☑	12. Provide a description of all-weather access road, inside perimeter road, and other on-site roads necessary for access at the landfill; (62-701.500(12), FAC)	
S ☐ Part K N/A ☐ N/C ☑	13. Additional record keeping and reporting requirements; (62-701.500(13), FAC)	
S ☐ Part K N/A ☐ N/C ☑		a. Records used for developing permit applications and supplemental information maintained for the design period of the landfill;
S ☐ Part K N/A ☐ N/C ☑		b. Monitoring information, calibration and maintenance records, and copies of reports required by permit maintained for at least 10 years;
S ☐ Part K N/A ☐ N/C ☑		c. Maintain annual estimates of the remaining life of constructed landfills, and of other permitted areas not yet constructed, and submit this estimate annually to the Department;
S ☐ Part K N/A ☐ N/C ☑		d. Procedures for archiving and retrieving records which are more than five years old;

PART L. WATER QUALITY MONITORING REQUIREMENTS (62-701.510, FAC)

LOCATION		
S ☐ Part L N/A ☐ N/C ☑	1. A water quality monitoring plan shall be submitted describing the proposed ground water and surface water monitoring systems, and shall meet at least the following requirements:	
S ☐ Part L N/A ☐ N/C ☑		a. Based on the information obtained in the hydrogeological investigation and signed, dated, and sealed by the P.G. or P.E. who prepared it; (62-701.510(2)(a), FAC)

LOCATION		PART L CONTINUED
S ☐ Part L N/A ☐ N/C ☑		b. All sampling and analysis performed in accordance with Chapter 62-160, FAC; (62-701.510(2)(b), FAC)
S ☐ Part L N/A ☐ N/C ☑		c. Ground water monitoring requirements; (62-701.510(3), FAC)
S ☐ Part L N/A ☐ N/C ☑	(1)	Detection wells located downgradient from and within 50 feet of disposal units;
S ☐ Part L N/A ☐ N/C ☑	(2)	Downgradient compliance wells as required;
S ☐ Part L N/A ☐ N/C ☑	(3)	Background wells screened in all aquifers below the landfill that may be affected by the landfill;
S ☐ Part L N/A ☐ N/C ☑	(4)	Location information for each monitoring well;
S ☐ Part L N/A ☐ N/C ☑	(5)	Well spacing no greater than 500 feet apart for downgradient wells and no greater than 1500 feet apart for upgradient wells, unless site specific conditions justify alternate well spacings;
S ☐ Part L N/A ☐ N/C ☑	(6)	Properly selected well screen locations;
S ☐ Part L N/A ☐ N/C ☑	(7)	Monitoring wells constructed to provide representative ground water samples;
S ☐ Part L N/A ☐ N/C ☑	(8)	Procedures for properly abandoning monitoring wells;
S ☐ Part L N/A ☐ N/C ☑	(9)	Detailed description of detection sensors, if proposed;
S ☐ Part L N/A ☐ N/C ☑		d. Surface water monitoring requirements; (62-701.510(4), FAC)
S ☐ Part L N/A ☐ N/C ☑	(1)	Location of and justification for all proposed surface water monitoring points;
S ☐ Part L N/A ☐ N/C ☑	(2)	Each monitoring location to be marked and its position determined by a registered Florida land surveyor;
S ☐ Part L N/A ☐ N/C ☑		e. Initial and routine sampling frequency and requirements; (62-701.510(5), FAC)
S ☐ Part L N/A ☐ N/C ☑	(1)	Initial background ground water and surface water sampling and analysis requirements;

LOCATION		PART L CONTINUED
S ☐ Part L	N/A ☐ N/C ☑	(2) Routine monitoring well sampling and analysis requirements;
S ☐ Part L	N/A ☐ N/C ☑	(3) Routine surface water sampling and analysis requirements;
S ☐ Part L	N/A ☐ N/C ☑	f. Describe procedures for implementing evaluation monitoring, prevention measures, and corrective action as required; (62-701.510(6), FAC)
S ☐ Part L	N/A ☐ N/C ☑	g. Water quality monitoring report requirements; (62-701.510(8), FAC)
S ☐ Part L	N/A ☐ N/C ☑	(1) Semi-annual report requirements; (see paragraphs 62-701.510(5)(c) and (d), FAC for sampling frequencies)
S ☐ Part L	N/A ☐ N/C ☑	(2) Documentation that the water quality data shall be provided to the Department in an electronic format consistent with requirements for importing into Department databases, unless an alternate form of submittal is specified in the permit;
S ☐ Part L	N/A ☐ N/C ☑	(3) Two and one-half year, or annual, report requirements, or every five years if in long-term care, signed dated, and sealed by P.G. or P.E.;

PART M. SPECIAL WASTE HANDLING REQUIREMENTS (62-701.520, FAC)

LOCATION		
S ☐ Part M	N/A ☐ N/C ☑	1. Describe procedures for managing motor vehicles; (62-701.520(1), FAC)
S ☐ Part M	N/A ☐ N/C ☑	2. Describe procedures for landfilling shredded waste; (62-701.520(2), FAC)
S ☐ Part M	N/A ☐ N/C ☑	3. Describe procedures for asbestos waste disposal; (62-701.520(3), FAC)
S ☐ Part M	N/A ☐ N/C ☑	4. Describe procedures for disposal or management of contaminated soil; (62-701.520(4), FAC)
S ☐ Part M	N/A ☐ N/C ☑	5. Describe procedures for disposal of biological wastes; (62-701.520(5), FAC)

PART N. GAS MANAGEMENT SYSTEM REQUIREMENTS (62-701.530, FAC)

	LOCATION		
S ☐	Part N	N/A ☐ N/C ☑	1. Provide documentation for a gas management system that will: (62-701.530(1), FAC)
S ☐	Part N	N/A ☐ N/C ☑	a. Be designed to prevent concentrations of combustible gases from exceeding 25% the LEL in structures and 100% the LEL at the property boundary;
S ☐	Part N	N/A ☐ N/C ☑	b. Be designed for site specific conditions;
S ☐	Part N	N/A ☐ N/C ☑	c. Be designed to reduce gas pressure in the interior of the landfill;
S ☐	Part N	N/A ☐ N/C ☑	d. Be designed to not interfere with the liner, leachate control system, or final cover;
S ☐	Part N	N/A ☐ N/C ☑	2. Provide documentation that will describe locations, construction details, and procedures for monitoring gas at ambient monitoring points and with soil monitoring probes; (62-701.530(2), FAC)
S ☐	Part N	N/A ☐ N/C ☑	3. Provide documentation describing how the gas remediation plan and odor remediation plan will be implemented; (62-701.530(3), FAC)
S ☐	Part N	N/A ☐ N/C ☑	4. Landfill gas recovery facilities; (62-701.530(5), FAC)
S ☐	Part N	N/A ☐ N/C ☑	a. Provide information required in Rules 62-701.320(7) and 62-701.330(3), FAC;
S ☐	Part N	N/A ☐ N/C ☑	b. Provide information required in Rule 62-701.600(4), FAC, where relevant and practical;
S ☐	Part N	N/A ☐ N/C ☑	c. Provide estimates of current and expected gas generation rates and description of condensate disposal methods;
S ☐	Part N	N/A ☐ N/C ☑	d. Provide description of procedures for condensate sampling, analyzing, and data reporting;
S ☐	Part N	N/A ☐ N/C ☑	e. Provide closure plan describing methods to control gas after recovery facility ceases operation, and any other requirements contained in Rule 62-701.400(10), FAC;

PART O. LANDFILL FINAL CLOSURE REQUIREMENTS (62-701.600, FAC)

	LOCATION		
S ☐	Part O	N/A ☐ N/C ☑	1. Closure permit requirements; (62-701.600(2), FAC)
S ☐	Part O	N/A ☐ N/C ☑	a. Application submitted to the Department at least 90 days prior to final receipt of wastes;
S ☐	Part O	N/A ☐ N/C ☑	b. Closure plan shall include the following:
S ☐	Part O	N/A ☐ N/C ☑	(1) Closure design plan;
S ☐	Part O	N/A ☐ N/C ☑	(2) Closure operation plan;
S ☐	Part O	N/A ☐ N/C ☑	(3) Plan for long-term care;
S ☐	Part O	N/A ☐ N/C ☑	(4) A demonstration that proof of financial assurance for long-term care will be provided;
S ☐	Part O	N/A ☐ N/C ☑	2. Closure design plan including the following requirements: (62-701.600(3), FAC)
S ☐	Part O	N/A ☐ N/C ☑	a. Plan sheet showing phases of site closing;
S ☐	Part O	N/A ☐ N/C ☑	b. Drawings showing existing topography and proposed final grades;
S ☐	Part O	N/A ☐ N/C ☑	c. Provisions to close units when they reach approved design dimensions;
S ☐	Part O	N/A ☐ N/C ☑	d. Final elevations before settlement;
S ☐	Part O	N/A ☐ N/C ☑	e. Side slope design including benches, terraces, down slope drainage ways, energy dissipaters, and description of expected precipitation effects;
S ☐	Part O	N/A ☐ N/C ☑	f. Final cover installation plans including:
S ☐	Part O	N/A ☐ N/C ☑	(1) CQA plan for installing and testing final cover;
S ☐	Part O	N/A ☐ N/C ☑	(2) Schedule for installing final cover after final receipt of waste;
S ☐	Part O	N/A ☐ N/C ☑	(3) Description of drought resistant species to be used in the vegetative cover;

S ☐ Part O N/A ☐ N/C ☑	(4) Top gradient design to maximize runoff and minimize erosion;
S ☐ Part O N/A ☐ N/C ☑	(5) Provisions for cover material to be used for final cover maintenance;
S ☐ Part O N/A ☐ N/C ☑	g. Final cover design requirements;
S ☐ Part O N/A ☐ N/C ☑	(1) Protective soil layer design;
S ☐ Part O N/A ☐ N/C ☑	(2) Barrier soil layer design;
S ☐ Part O N/A ☐ N/C ☑	(3) Erosion control vegetation;
S ☐ Part O N/A ☐ N/C ☑	(4) Geomembrane barrier layer design;
S ☐ Part O N/A ☐ N/C ☑	(5) Geosynthetic clay liner design, if used;
S ☐ Part O N/A ☐ N/C ☑	(6) Stability analysis of the cover system and the disposed waste;
S ☐ Part O N/A ☐ N/C ☑	h. Proposed method of stormwater control;
S ☐ Part O N/A ☐ N/C ☑	i. Proposed method of access control;
S ☐ Part O N/A ☐ N/C ☑	j. Description of the proposed or existing gas management system which complies with Rule 62-701.530, FAC;
S ☐ Part O N/A ☐ N/C ☑	3. Closure operation plan shall include: (62-701.600(4), FAC)
S ☐ Part O N/A ☐ N/C ☑	a. Detailed description of actions which will be taken to close the landfill;
S ☐ Part O N/A ☐ N/C ☑	b. Time schedule for completion of closing and long-term care;
S ☐ Part O N/A ☐ N/C ☑	c. Describe proposed method for demonstrating financial assurance for long-term care;
S ☐ Part O N/A ☐ N/C ☑	d. Operation of the water quality monitoring plan required in Rule 62-701.510, FAC;
S ☐ Part O N/A ☐ N/C ☑	e. Development and implementation of gas management system required in Rule 62-701.530, FAC;

LOCATION **PART O CONTINUED**

S ☐ Part O	N/A ☐ N/C ☑	4. Certification of closure construction completion and final reports including: (62-701.600(6), FAC)
S ☐ Part O	N/A ☐ N/C ☑	a. Survey monuments; (62-701.600(6)(a), FAC)
S ☐ Part O	N/A ☐ N/C ☑	b. Final survey report; (62-701.600(6)(b), FAC)
S ☐ Part O	N/A ☐ N/C ☑	c. Closure construction quality assurance report; (62-701.400(7), FAC)
S ☐ Part O	N/A ☐ N/C ☑	5. Declaration to the public; (62-701.600(7), FAC)
S ☐ Part O	N/A ☐ N/C ☑	6. Official date of closing; (62-701.600(8), FAC)
S ☐ Part O	N/A ☐ N/C ☑	7. Justification for and detailed description of procedures to be followed for temporary closure of the landfill, if desired; (62-701.600(9), FAC)

PART P. OTHER CLOSURE PROCEDURES (62-701.610, FAC)

LOCATION

S ☐ Part P	N/A ☐ N/C ☑	1. Describe how the requirements for use of closed solid waste disposal areas will be achieved; (62-701.610(1), FAC)
S ☐ Part P	N/A ☐ N/C ☑	2. Describe how the requirements for relocation of wastes will be achieved; (62-701.610(2), FAC)

PART Q. LONG-TERM CARE (62-701.620, FAC)

LOCATION

S ☐ Part Q	N/A ☐ N/C ☑	1. Maintaining the gas collection and monitoring system; (62-701.620(5), FAC)
S ☐ Part Q	N/A ☐ N/C ☑	2. Stabilization report requirements; (62-701.620(6), FAC)
S ☐ Part Q	N/A ☐ N/C ☑	3. Right of access; (62-701.620(7), FAC)
S ☐ Part Q	N/A ☐ N/C ☑	4. Requirements for replacement of monitoring devices; (62-701.620(8), FAC)
S ☐ Part Q	N/A ☐ N/C ☑	5. Completion of long-term care signed and sealed by professional engineer; (62-701.620(9), FAC)

PART R. FINANCIAL ASSURANCE (62-701.630, FAC)

LOCATION		
S ☑ Part R.1 N/A ☐ N/C ☐	1. Provide cost estimates for closing, long-term care, and corrective action costs estimated by a P.E. for a third party performing the work, on a per unit basis, with the source of estimates indicated; (62-701.630(3) & (7), FAC)
S ☑ Part R.2 N/A ☐ N/C ☐	2. Describe procedures for providing annual cost adjustments to the Department based on inflation and changes in the closing, long-term care, and corrective action plans; (62-701.630(4) & (8), FAC)
S ☑ Part R.3 N/A ☐ N/C ☐	3. Describe funding mechanisms for providing proof of financial assurance and include appropriate financial assurance forms. (62-701.630(5), (6), & (9), FAC)

PART S. CERTIFICATION BY APPLICANT AND ENGINEER OR PUBLIC OFFICER

1. Applicant:

The undersigned applicant or authorized representative of Pasco County Public Infrastructure Department is aware that statements made in this form and attached information are an application for a Operation modification permit from the Florida Department of Environmental Protection, and certifies that the information in this application is true, correct, and complete to the best of his/her knowledge and belief. Further, the undersigned agrees to comply with the provisions of Chapter 403, Florida Statutes, and all rules and regulations of the Department. It is understood that the Permit is not transferable, and the Department will be notified prior to the sale or legal transfer of the permitted facility.

Signature of Applicant or Agent

Justin Roessler, Interim Solid Waste Director
Name and Title (please type)

JRoessler@PascoCountyFL.net
E-Mail Address (if available)

14230 Hays Road
Mailing Address

Spring Hill, FL 34610
City, State, Zip Code

(727) 857-2780
Telephone Number

Date: 7/2/21

Attach letter of authorization if agent is not a government official, owner, or corporate officer.

2. Professional Engineer registered in Florida (or Public Officer if authorized under Sections 403.707 and 403.7075, Florida Statutes):

This is to certify that the engineering features of this solid waste management facility have been designed/examined by me and found to conform to engineering principles applicable to such facilities. In my professional judgment, this facility, when properly maintained and operated, will comply with all applicable statutes of the State of Florida and rules of the Department. It is agreed that the undersigned will provide the applicant with a set of instructions of proper maintenance and operation of the facility.

Tobin S McKnight
2021.07.21 11:42:04-04'00'
Signature

Tobin S. McKnight, PE, BCEE, Chief Engineer/Vice President
Name and Title (please type)

69187
Florida Registration Number (please affix seal)

TOBIN SCOTT MCKNIGHT LICENSE No. 69187 STATE OF PROFESSIONAL ENGINEER

730 N.E. Waldo Road
Mailing Address

Gainesville, FL 32641
City, State, Zip Code

TMcKnight@JonesEdmunds.com
E-Mail Address (if available)

(352) 377-5821
Telephone Number

Date:

This item has been digitally signed and sealed by Tobin S. McKnight, PE on the date adjacent to the seal.

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

Supporting Documentation

PART C

PART C – PROHIBITIONS

1 SITING CRITERIA

The proposed substantial modifications of the West Pasco County Landfill (WPCL) ash monofill cells are completely within the boundaries of the WPCL, as shown on Sheet C1, *Site Plan,* in the *West Pasco County Landfill Ash Monofill Operations Modifications* Engineering Plans provided in Appendix A. The siting prohibitions of Rule 62-701.300, Florida Administrative Code (FAC), for ash monofill cells are addressed below.

Rule 62.701.300(2), FAC:

- Solid waste will not be stored or disposed of in an area where geological formations or other subsurface features will not provide support for the solid waste. Part I of this application provides geotechnical analysis demonstrating the stability of the underlying geology.
- Solid waste will not be stored and/or disposed of within 500 feet of any existing or approved potable water wells. Figure C.1.b of this application provides a map of the potable well locations in the vicinity of WPCL.
- Solid waste will not be stored and/or disposed of in a dewatered pit.
- Solid waste will not be stored and/or disposed of in any natural or artificial body of water including groundwater and wetlands. Figure C.1.d identifies surface water bodies and wetlands near the property boundary.
- Solid waste will not be stored or disposed of within 200 feet of any natural or artificial body of water except for impoundments and conveyances that are part of the facility's permitted stormwater management system. The ash monofill uses leachate control methods that will result in compliance with FDEP's water quality standards and criteria.
- Solid waste will not be stored and/or disposed of on the right-of-way of any public highway, road, or alley.

2 EXEMPTIONS

The provisions in Rules 62-701.300(12), (13), and (16) through (18), FAC, below are not applicable to WPCL:

62-701.300(12): Yard trash: Yard trash will not be disposed of in the ash monofill cells. Yard trash will not be stored, processed, or disposed of within 100 feet of off-site potable wells (no setback required for on-site potable wells), within 50 feet from water bodies, or within 500 feet from wells serving community water supplies. No exemption is requested.

62-701.300(13): Tanks: Solid waste will not be stored or treated in tanks. No exemption is requested.

62-701.300(16): Indoor storage/treatment: Solid waste will not be stored or processed indoors. No exemption is requested

62-701.300(17): Storage in vehicles or containers: Solid waste will not be stored in vehicle or containers. No exemption is requested.

62-701.300(18): Existing Facilities: The ash monofill is an existing facility; therefore, it remains subject to the prohibitions that were in effect at the time the permit was issued authorizing construction. No exemption is requested.

3 BURNING RESTRICTION

In accordance with Rule 62-701.300(3), FAC, open burning of solid waste will not be permitted at WPCL.

4 HAZARDOUS WASTE DISPOSAL RESTRICTIONS

In accordance with Rule 62-701.300(4), FAC, hazardous waste will not knowingly be disposed of at WPCL. Refer to Section 8.0 of the Operation Plan in Appendix B for hazardous waste disposal restrictions.

5 PCB DISPOSAL RESTRICTIONS

In accordance with Rule 62-701.300(5), FAC, polychlorinated biphenyls (PCBs); liquids containing a PCB concentration of 50 parts per million or greater; or non-liquid PCBs at concentrations of 50 parts per million or greater in the form of contaminated soil, rags, or other debris will not knowingly be disposed of at WPCL.

6 BIOMEDICAL WASTE DISPOSAL RESTRICTIONS

In accordance with Rule 62-701.300(6), FAC, untreated biomedical waste will not knowingly be accepted or disposed of at WPCL. Refer to Section 8.10 of the Operation Plan in Appendix B for biomedical waste disposal restrictions.

7 CLASS I SURFACE WATER RESTRICTIONS

In accordance with Rule 62-701.300(7), FAC, Figure C.1.d shows that WPCL is not within 3,000 feet of Class I surface waters.

8 SPECIAL WASTE FOR LANDFILLS RESTRICTIONS

In accordance with Rule 62-701.300(8), FAC, WPCL will not knowingly accept the following materials for disposal in the ash monofil:

- Lead-acid batteries.
- Used oil.
- Yard waste.
- White goods.

- Whole waste tires.

Refer to Section 8.0 of the Operation Plan in Appendix B.

In accordance with Rule 62-701.300(9), FAC, WPCL will not knowingly accept lead-acid batteries, mercury-containing devices, or spent mercury-containing lamps at the Pasco County Resource Recovery Facility.

9 LIQUID RESTRICTIONS

In accordance with Rule 62-701.300(10), FAC, no liquid waste except for leachate or household waste as described in Rule 62-701.300(10)(a) and (b), FAC, will be accepted at WPCL. Refer to Section 8 of the Operation Plan in Appendix B for liquid and container disposal restrictions.

10 USED OIL RESTRICTIONS

In accordance with Rule 62-701.300(11), FAC, used oil or used oil mixed or comingled with solid waste will not be knowingly accepted for disposal at WPCL. Oily wastes, sorbents, or other materials used for maintenance or to clean up or contain leaks, spills, or accidental releases of used oil, and soils contaminated with used oil as a result of spills or accidental releases are not subject to these prohibitions. Refer to Section 8.3 of the Operation Plan in Appendix B for used oil restrictions.

11 TREATED WOOD RESTRICTIONS

In accordance with Rule 62-701.300(14), FAC, chromated copper arsenate (CCA)-treated wood shall not be incorporated into compost or made into mulch, decorative landscape chips, or any other wood product that is applied as a ground cover, soil, or soil amendment. CCA-treated wood may be ground and used as initial cover on interior slopes of lined solid waste disposal facilities provided it meets the criteria of Rule 62-701.200(53), FAC. CCA-treated wood will not be disposed of through open burning or through combustion in an air curtain incinerator.

12 DUST CONTROL RESTRICTIONS

In accordance with Rule 62-701.300(15) and Rule 62-296.320(4)(c), FAC, WPCL will not allow the unconfined emissions of particulate matter.

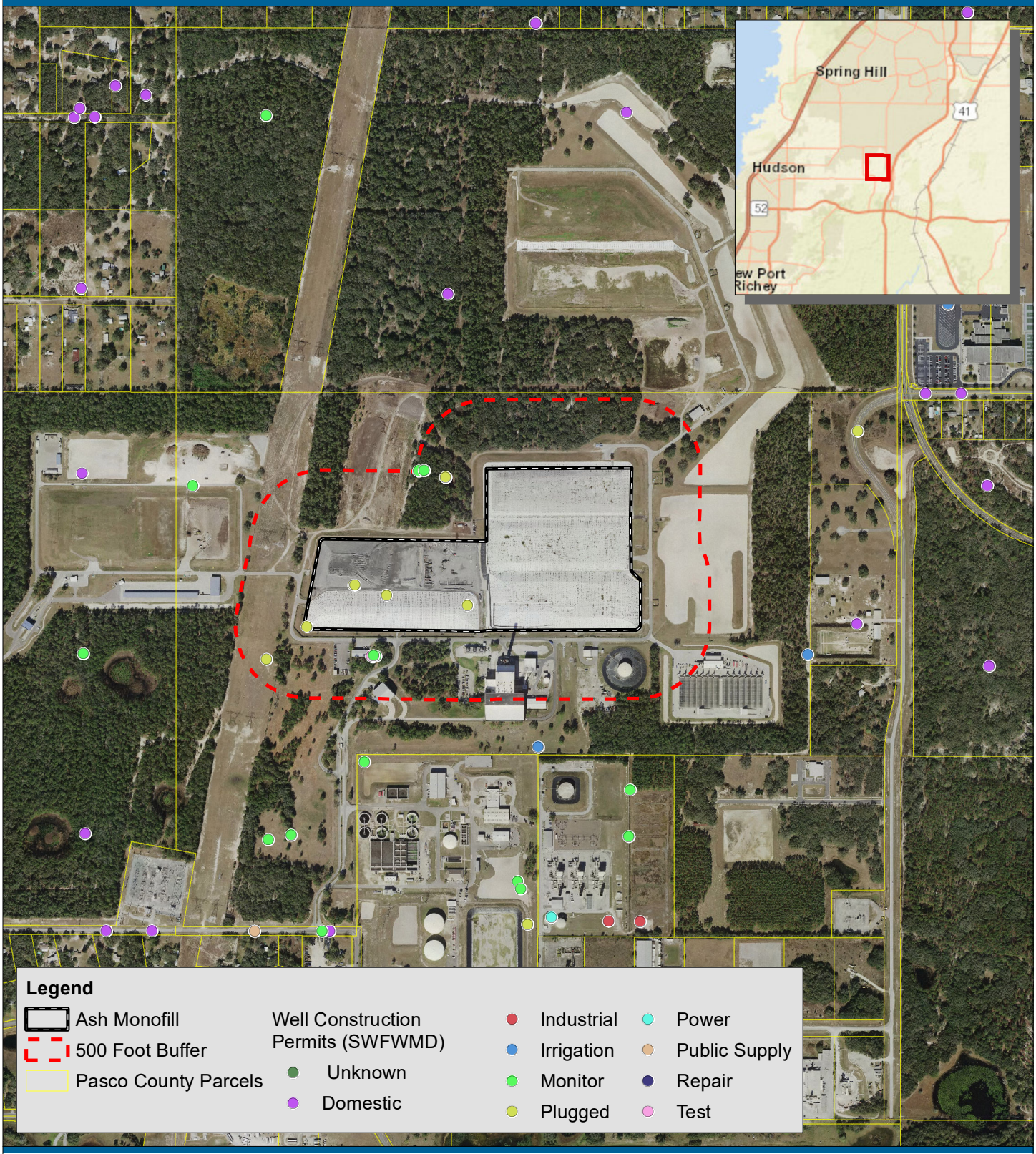
Figure C.1.b

Permitted Water Wells

Figure C.1.b

Permitted Water Wells

West Pasco County Landfill - Ash Monofill



Well Construction Permits (SWFWMD)
Filename: WCP_SITES
Publisher: Southwest Florida Water Management District
Date: March 2021
Source: http://data-swfwmd.opendata.arcgis.com/

JonesEdmunds

For Informational Purposes Only Y:\16002-PascoCounty\Projects\009-01-AshMonofillOperationsModification\GIS\mxd\FigC1b_WaterWells.mxd JReynolds 3/19/2021

Figure C.1.d

FEMA Flood Hazard Map

Figure C.1.d

FEMA Flood Hazard Map

West Pasco County Landfill - Ash Monofill

FEMA Flood Hazard
Filename: DFIRM_FLDHAZ_NOV20
Publisher: Federal Emergency Management Agency
Date: November 2020
Source: www.fgdl.org

JonesEdmunds

For Informational Purposes Only Y:\16002-PascoCounty\Projects\009-01-AshMonofillOperationsModification\GIS\mxd\FigC1d_FloodHazard.mxd JReynolds 3/19/2021

PART D

PART D – SOLID WASTE MANAGEMENT FACILITY PERMIT REQUIREMENTS, GENERAL

1. APPLICATION FORM AND ALL SUPPORTING DATA AND REPORTS

Rule 62-701.320(5)(a), Florida Administrative Code (FAC), requires one electronic copy of the application, engineering plans and reports, and all supporting information for the proposed construction, substantial modification, operation, or closure of a facility.

2. PROFESSIONAL CERTIFICATIONS

Jones Edmunds has prepared this application and is an authorized engineering firm in Florida (Certificate of Authorization #1841). The application plans, reports, and supporting information have been signed and sealed by Tobin S. McKnight, PE, BCEE, a registered Professional Engineer in Florida (Florida Professional Engineer No. 69187), as the Engineer of Record. Geotechnical portions of this application have been signed and sealed by Carol G. Sawyer, PE, a registered Professional Engineer in Florida (Professional Engineer No. 55214).

3. LETTER OF TRANSMITTAL

A letter of transmittal is included with this permit application.

4. APPLICATION FORM

Florida Department of Environmental Protection (FDEP) Form No. 62-701.900(1), effective date of February 15, 2015, has been completed and is signed and dated by the applicant.

5. PERMIT FEE

In accordance with Rule 62-701.315(1), FAC, a check for $10,000, payable to FDEP, will be submitted by the County as the permit application fee for the WPCL ash monofill operations modification.

6. ENGINEERING REPORT

This application and supporting documentation meet the requirements of an engineering report required by Rule 62-701.320(7)(d), FAC.

7. OPERATIONS PLAN AND CLOSURE PLAN

This application is for a substantial modification of WPCL's ash monofill operations. The Operation Plan in Appendix B identifies the policies, procedures, and guidelines for operating the Class I Landfill. The Closure and Long-Term-Care (CLTC) Plan in Appendix C outlines the closure and long-term care policies, procedures, and guidelines for the ash monofill.

8. CONTINGENCY PLAN

Contingency Plans for operations are outlined in Section 12.0 of the Operation Plan, in Appendix B.

9. PLANS OR DRAWINGS

The *West Pasco County Landfill Operations Modification* Plans are in Appendix A. The engineering plans provide cross sections and details of the proposed modifications of the ash monofill as described below.

a. Regional Map or Plan

The regional map is provided on Sheet G1, *Cover*, of the engineering plans in Appendix A.

b. Vicinity Map or Aerial Photograph

The vicinity map is provided on the Cover Sheet of the *West Pasco County Landfill Class I Slope Modification and Vertical Expansion* Engineering Plans in Appendix A.

An aerial photograph is provided on the *Site Plan,* Sheet C1, of the *West Pasco County Landfill Class I Slope Modification and Vertical Expansion* Engineering Plans in Appendix A.

c. Site Plan

The site property boundary is provided on the *Site Plan,* Sheet C1, of the *West Pasco County Landfill Class I Slope Modification and Vertical Expansion* Engineering Plans in Appendix A. No changes in the property boundary have occurred since the last permit application submission. The previously submitted boundary survey is not being resubmitted.

d. Supporting Details

The engineering plans in Appendix A of this application provide other necessary details to support the Engineering Report.

10. DOCUMENTATION FROM PROPERTY OWNER

Pasco County owns the property within the limits of the proposed project. No changes in property ownership have occurred since the last permit application submission. Copies of the previously submitted warranty deeds and boundary surveys are not being resubmitted.

11. WASTE REDUCTION AND RECYCLING GOALS

No changes to the facility's waste reduction and recycling goals are proposed in this permit application. Attachment D.11 includes the 2018 recycling rates for Pasco County.

12. HISTORY AND DESCRIPTION OF ENFORCEMENT ACTIONS

Attachment D.12 provides a history of enforcement actions.

13. PROOF OF PUBLICATION

In accordance with Rule 62-17.281, FAC, a Notice of Request for Modification for the Certification will be published in *The Tampa Bay Times*, a newspaper of general circulation where the facility is located, within 21 days after the filing of a request for modification. Proof of Publication will be submitted to FDEP's Siting Coordination Office within 7 days upon receipt.

In accordance with 62-701.320(8)(b), FAC, the Notice will be mailed to the highest-ranking elected official of Pasco County and each State Senator and Representative serving the jurisdiction in which the project is located within 14 days of submittal of the application to FDEP:

- Ron Oakley, Chairman of the Pasco County Board of County Commissioners.
- Representative Adrian Zika, Florida House of Representatives, District 37.
- Wilton Simpson, Florida Senate, District 10.

Attachments D.13.a and D.13.b include copies of the notice of application and proof of mailed letters, respectively.

14. AIRPORT SAFETY

In accordance with Rule 62-701.320(13), FAC, and as shown in Figure E.1 of Part E, airports within 10,000 feet and 5 miles of the WPCL have been identified. One airport, as defined in Rule 62-701.320(13), FAC, is within the 5-mile radius of the landfill. In accordance with Rule 62-701.320(13)(c), FAC, notification of the permit application submittal will be sent to the airport, to the Federal Aviation Administration, and the Florida Department of Transportation within 14 days of application submittal. FDEP will be copied on this correspondence, and Attachment D.14 provides copies of the letters.

15. OPERATOR AND SPOTTER TRAINING REQUIREMENTS

Operators and spotters employed at WPCL are trained in accordance with Rule 62-701.320(15), FAC. Personnel are required to successfully complete FDEP-approved training courses. The training plan and records documenting successful completion of the courses are kept on file at WPCL. Section 5 of the Operation Plan in Appendix B identifies the spotters' and operators' training requirements for WPCL.

Attachment D.11

Pasco County Recycling Rates

Attachment D.11

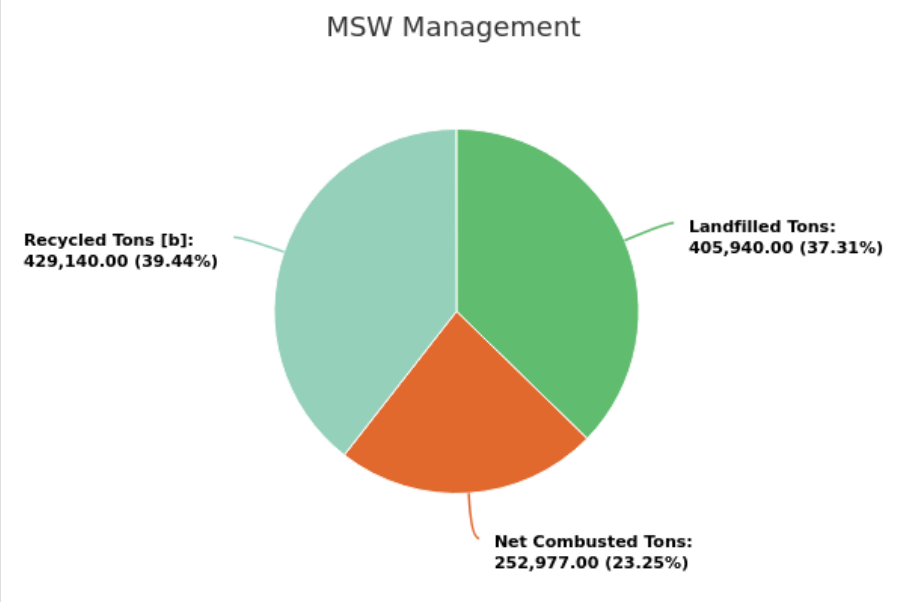
Florida DEP County Reports County Overview Report

Population

Member: Pasco County

Displaying 1 result

MEMBER	POPULATION
Recycling	515,077

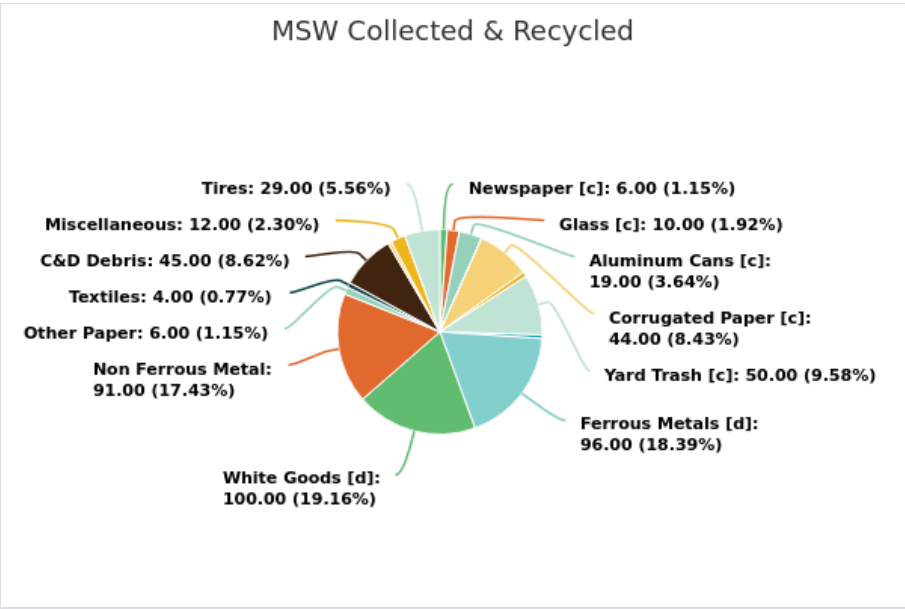


MSW Management

Member: Pasco County

Displaying 1 result

LANDFILLED TONS	NET COMBUSTED TONS	RECYCLED TONS [B]	STOCKPILED TONS	TOTAL	TOTAL POUNDS PER CAPITA PER DAY
405,940.00	252,977.00	429,140.00		1,088,057.00	11.57



Attachment D.11

MSW Collected & Recycled

Member: Pasco County

Displaying 18 results

MATERIAL TYPE	TONS OF MSW COLLECTED	TONS OF MSW RECYCLED	% MSW RECYCLED
Newspaper [c]	17,804.00	1,140.00	6%
Glass [c]	12,538.00	1,268.00	10%
Aluminum Cans [c]	3,614.00	694.00	19%
Plastic Bottles [c]	17,007.00	26.00	0%
Steel Cans [c]	6,058.00	6.00	0%
Corrugated Paper [c]	40,420.00	17,791.00	44%
Office Paper [c]	16,767.00	639.00	4%
Yard Trash [c]	64,892.00	32,215.00	50%
Other Plastics	54,286.00	1,527.00	3%
Ferrous Metals [d]	74,399.00	71,490.00	96%
White Goods [d]	11,805.00	11,805.00	100%
Non Ferrous Metal	25,107.00	22,841.00	91%
Other Paper	74,599.00	4,565.00	6%
Textiles	14,623.00	513.00	4%
C&D Debris	562,475.00	255,251.00	45%
Food	44,678.00	1,330.00	3%
Miscellaneous	44,220.00	5,251.00	12%
Tires	2,765.00	788.00	29%
Total	1,088,057.00	429,140.00	39.44

Attachment D.11

Traditional Recycling Rate

Member: Pasco County

Displaying 10 results

	RECYCLING RATE (%)	UNIT
A. Adjusted County Recycling Rate Calculation (excluding renewable energy credits)		
i. Unadjusted County Recycling Rate by means other than renewable energy	39.44	%
ii. Total tons of waste used for process fuel. (Process Fuel only)	0	tons
iii. Process Fuel	0	%
iv. Line A(i) minus line A(iii)	39.44	%
Adjusted Recycling Rate	39.44	%
B. Adjusted County Recycling Rate Calculation		
i. Unadjusted County Recycling Rate (including renewable energy)	59.69	%
ii. Process Fuel	0	%
iii. Line B(i) minus line B(ii)	59.69	%
Adjusted Recycling Rate	59.69	%
C. Overall County Adjusted Recycling Rate		
	59.69	%

Renewable Energy Recycling

Member: Pasco County

Displaying 1 result

	TONS	RECYCLING CREDIT (%)
Waste to Energy [jj]	220,365.00	34%

Participation Rates

Member: Pasco County

Displaying 3 results

	# UNITS	PARTICIPATION RATE (%)
Residential Single Family (curbside only)	153,026.00	29%
Commerical		
Scheduled Recycling Service Collection	1.00	100%
On Call Recycling Service Collection	1.00	100%

Attachment D.12

History of Enforcement Actions

Attachment D.12 Summary of Enforcement Actions

Landfill Site	Action	Result
WPCL	Warning Letter #WL96-0006SW51 SWD issued on November 7, 1996, related to operation of the West Pasco Class I Landfill.	Issue was resolved through entry into a Short Form Consent Order.
WPCL	Warning Letter #WL97-008SW51SWD issued on November 28, 1997, related to leachate management at the West Pasco County Class I Landfill.	Issue was resolved through entry into a Short Form Consent Order.
WPCL	Warning Letter #WL08-0014SW51SWD issued on July 29, 2008, related to ash management at the Pasco County Resource Recovery Facility.	Issue was resolved through entry into a Short Form Consent Order.

Attachments D.13.a and b

Notice of Application Letters

Florida Department of Environmental Protection

Notice of Request for Modification of the Certification

Notice of Request for Modification

Pasco County Public Infrastructure is requesting a modification for the Pasco County Resource Recovery Facility, PA 87-23, 87-1587 located in Pasco County, Florida. The request for modification was filed with the Department of Environmental Protection on 07/09/2021. The request is for the modification of the Ash Monofill cells. The County proposes to modify the side slopes to three horizontal to one vertical (3H:1V) and to vertically expand the landfill to a final elevation of 122 feet NAVD.

The petition for modification is available for public inspection electronically here https://fldeploc.dep.state.fl.us/WWW_WACS/Reports/SW_Facility_Docs.asp?wacsid=45799. Copies of the modification request may also be obtained by contacting the Department of Environmental Protection's Siting Coordination Office at 2600 Blair Stone Road, MS 5500, Tallahassee, FL 32399-2400, or by email at SCO@dep.state.fl.us.

JonesEdmunds
Integrity • Knowledge • Service

July 9, 2021

Ron Oakley, Chairman
Pasco County Board of County Commissioners
8731 Citizens Drive
New Port Richey, FL 34654

RE: West Pasco County Landfill Class I Slope Modification and Vertical Expansion
Notice of Application for Construction/Operation Permit Modification
for the Class I Landfill (Ash Monofill)
Jones Edmunds Project No.: 16002-009-01

Dear Mr. Oakley:

On behalf of Pasco County Solid Waste and Resource Recovery, Jones Edmunds is submitting this letter as notification of a solid waste permit application filed with the Florida Department of Environmental Protection (FDEP) to modify a Class I Landfill (Ash Monofill) at the West Pasco County Landfill (WPCL). The WPCL is off Hays Road, just west of State Road 589.

The proposed project is to modify the side slopes and to vertically expand the Ash Monofill. Solid Waste and Resource Recovery facilities must obtain approval from FDEP before modifying the Class I Landfill.

In accordance with Rule 62-701.320(8)(b), FAC, this notice has been mailed to the Chairman of the Board of the Pasco County Commissioners, the Pasco County State Representative, and the State Senator serving the jurisdiction where the landfill is located. This letter is intended only for your information and no response is needed.

In accordance with FDEP rules, we have enclosed for your file a copy of the Notice of Application that will be published in the *Tampa Bay Times* for this project.

If you have any questions concerning this letter or the Notice of Application, please contact me at (352) 377-5821.

Sincerely,

Tobin McKnight, PE, BCEE
Chief Engineer/Vice President
730 NE Waldo Road, Gainesville, FL 32641

Enclosure

XC: El Kromhout, Florida Department of Environmental Protection
Justin Roessler, Pasco County Solid Waste and Resource Recovery

JonesEdmunds
Integrity • Knowledge • Service

July 9, 2021

Ardian Zika
Florida House of Representatives
Building #4
20430 Gator Lane
Land O' Lakes, FL 34638-2804

RE: West Pasco County Landfill Class I Slope Modification and Vertical Expansion
Notice of Application for Construction/Operation Permit Modification
for the Class I Landfill (Ash Monofill)
Jones Edmunds Project No.: 16002-009-01

Dear Mr. Zika:

On behalf of Pasco County Solid Waste and Resource Recovery, Jones Edmunds is submitting this letter as notification of a solid waste permit application filed with the Florida Department of Environmental Protection (FDEP) to modify a Class I Landfill (Ash Monofill) at the West Pasco County Landfill (WPCL). The WPCL is off Hays Road, just west of State Road 589.

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If you have any questions concerning this letter or the Notice of Application, please contact me at (352) 377-5821.

Sincerely,

Tobin McKnight, PE, BCEE
Chief Engineer/Vice President
730 NE Waldo Road, Gainesville, FL 32641

Enclosure

XC: El Kromhout, Florida Department of Environmental Protection
Justin Roessler, Pasco County Solid Waste and Resource Recovery

JonesEdmunds
Integrity • Knowledge • Service

July 9, 2021

Wilton Simpson
The Florida Senate
4076 Commercial Way
Spring Hill, FL 34606

RE: West Pasco County Landfill Class I Slope Modification and Vertical Expansion
Notice of Application for Construction/Operation Permit Modification
for the Class I Landfill (Ash Monofill)
Jones Edmunds Project No.: 16002-009-01

Dear Mr. Simpson:

On behalf of Pasco County Solid Waste and Resource Recovery, Jones Edmunds is submitting this letter as notification of a solid waste permit application filed with the Florida Department of Environmental Protection (FDEP) to modify a Class I Landfill (Ash Monofill) at the West Pasco County Landfill (WPCL). The WPCL is off Hays Road, just west of State Road 589.

The proposed project is to modify the side slopes and to vertically expand the Ash Monofill. Solid Waste and Resource Recovery facilities must obtain approval from FDEP before modifying the Class I landfill.

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Sincerely,

Tobin McKnight, PE, BCEE
Chief Engineer/Vice President
730 NE Waldo Road, Gainesville, FL 32641

Enclosure

XC: El Kromhout, Florida Department of Environmental Protection
Justin Roessler, Pasco County Solid Waste and Resource Recovery

Shipment Receipt

Transaction Date: 09 Jul 2021 **Tracking Number:** 1ZF626R20390962748

① Address Information

Ship To:	Ship From:	Return Address:
The Florida Senate Wilton Simpson 4076 Commercial Way SPRING HILL FL 346062397	Jones Edmunds & Associates Joanne Talamo 730 NE Waldo Rd Gainesville FL 32641 Telephone:352-376-0661 x1282 email:jtalamo@jonesedmunds.com	Jones Edmunds & Associates Joanne Talamo 730 NE Waldo Rd Gainesville FL 32641 Telephone:352-376-0661 x1282 email:jtalamo@jonesedmunds.com

② Package Information

	Weight	Dimensions / Packaging	Declared Value	Reference Numbers
1.	0.3 lbs (1.0 lbs billable)	Other Packaging		Project #/task # - 16002-009-01-3000 Employee # & or name - CSpreadbury package contents - Document (NOA)

③ UPS Shipping Service and Shipping Options

Service: UPS Ground Service

Delivered By: End of Day Monday, Jul 12, 2021

Shipping Fees Subtotal: 8.93 USD

Transportation 8.23 USD

Fuel Surcharge 0.70 USD

Additional Shipping Options

Quantum View Notify E-mail Notifications: No Charge

1 jtalamo@jonesedmunds.com: Exception, Delivery

2 cspreadbury@jonesedmunds.com: Ship, Exception, Delivery

④ Payment Information

Bill Shipping Charges to: Shipper's Account F626R2

Shipping Charges:	**8.93 USD**
Subtotal Shipping Charges:	**8.93 USD**
Total Charged:	**8.93 USD**

Note: This document is not an invoice. Your final invoice may vary from the displayed reference rates.

* For delivery and guarantee information, see the UPS Service Guide ({0}). To speak to a customer service representative, call 1-800-PICK-UPS for domestic services and 1-800-782-7892 for international services.

Shipment Receipt

Transaction Date: 09 Jul 2021 **Tracking Number:** 1ZF626R20391364937

① Address Information

Ship To:	Ship From:	Return Address:
Pasco Co BOCC Ron Oakley, Chairman 8731 Citizens Drive NEW PORT RICHEY FL 346545572	Jones Edmunds & Associates Joanne Talamo 730 NE Waldo Rd Gainesville FL 32641 Telephone:352-376-0661 x1282 email:jtalamo@jonesedmunds.com	Jones Edmunds & Associates Joanne Talamo 730 NE Waldo Rd Gainesville FL 32641 Telephone:352-376-0661 x1282 email:jtalamo@jonesedmunds.com

② Package Information

	Weight	Dimensions / Packaging	Declared Value	Reference Numbers
1.	0.3 lbs (1.0 lbs billable)	Other Packaging		Project #/task # - 16002-009-01-3000 Employee # & or name - CSpreadbury package contents - Documents (NOA)

③ UPS Shipping Service and Shipping Options

Service: UPS Ground Service

Delivered By: End of Day Monday, Jul 12, 2021

Shipping Fees Subtotal:	12.13 USD	
Transportation	8.23 USD	
Fuel Surcharge	0.95 USD	
Delivery Area Surcharge	2.95 USD	
Package 1		

Additional Shipping Options

Quantum View Notify E-mail Notifications: No Charge

1 jtalamo@jonesedmunds.com: Exception, Delivery
2 cspreadbury@jonesedmunds.com: Ship, Exception, Delivery

④ Payment Information

Bill Shipping Charges to: Shipper's Account F626R2

Shipping Charges:	**12.13 USD**
Subtotal Shipping Charges:	**12.13 USD**
Total Charged:	**12.13 USD**

Note: This document is not an invoice. Your final invoice may vary from the displayed reference rates.

* For delivery and guarantee information, see the UPS Service Guide ({0}). To speak to a customer service representative, call 1-800-PICK-UPS for domestic services and 1-800-782-7892 for international services.

Shipment Receipt

Transaction Date: 09 Jul 2021 **Tracking Number:** 1ZF626R20394423528

① Address Information

Ship To:	Ship From:	Return Address:
Florida House of Representatives Ardian Zika 20430 Gator Lane Building 4 LAND O LAKES FL 346382804	Jones Edmunds & Associates Joanne Talamo 730 NE Waldo Rd Gainesville FL 32641 Telephone:352-376-0661 x1282 email:jtalamo@jonesedmunds.com	Jones Edmunds & Associates Joanne Talamo 730 NE Waldo Rd Gainesville FL 32641 Telephone:352-376-0661 x1282 email:jtalamo@jonesedmunds.com

② Package Information

	Weight	Dimensions / Packaging	Declared Value	Reference Numbers
1.	0.3 lbs (1.0 lbs billable)	Other Packaging		Project #/task # - 16002-009-01-3000 Employee # & or name - CSpreadbury package contents - Documents (NOA)

③ UPS Shipping Service and Shipping Options

Service: UPS Ground Service

Delivered By: End of Day Monday, Jul 12, 2021

Shipping Fees Subtotal: 8.93 USD

Transportation 8.23 USD

Fuel Surcharge 0.70 USD

Additional Shipping Options

Quantum View Notify E-mail Notifications: No Charge

1 jtalamo@jonesedmunds.com: Exception, Delivery
2 cspreadbury@jonesedmunds.com: Ship, Exception, Delivery

④ Payment Information

Bill Shipping Charges to: Shipper's Account F626R2

Shipping Charges:	**8.93 USD**
Subtotal Shipping Charges:	**8.93 USD**
Total Charged:	**8.93 USD**

Note: This document is not an invoice. Your final invoice may vary from the displayed reference rates.

* For delivery and guarantee information, see the UPS Service Guide ({0}). To speak to a customer service representative, call 1-800-PICK-UPS for domestic services and 1-800-782-7892 for international services.

Attachment D.14

Airport Notices

JonesEdmunds

Integrity • Knowledge • Service

July 12, 2021

Aleda Budd, Airport General Manager
Pilot Country
11500 Pilot Country Drive
Spring Hill, Florida 34610

RE: West Pasco County Landfill (WPCL) Ash Monofill Operations Modification
Notice of Application for Construction/Operation Permit Modification
for the Ash Monofill
Jones Edmunds Project No.: 16002-009-01

Dear Ms. Budd:

In accordance with Rule 62-701.320(13)(c), FAC, of the Florida Department of Environmental Protection (FDEP), this letter serves as formal notice that Pasco County Solid Waste and Resource Recovery filed a permit application with FDEP to substantially modify the operations of the Ash Monofill at 14230 Hays Road, Spring Hill, Florida 34610.

Rule 62-701.320(13)(c), FAC, requires Pasco County Solid Waste and Resource Recovery to notify all owners of airports/heliports within 5 miles of WPCL of this permit application, not including facilities used solely for helicopters or other aircraft that take off and land vertically. The Pilot Country Airport is within 5 miles of the landfill as illustrated on Figure E.1. Pasco County Public Infrastructure has notified the Pilot Country Airport in accordance with Rule 62-701.320(13)(c), FAC.

If you have any comments or concerns, please feel free to contact me at (352) 377-5821.

Sincerely,

Tobin McKnight, PE, BCEE
Chief Engineer/Vice President
730 NE Waldo Road
Gainesville, Florida 32641

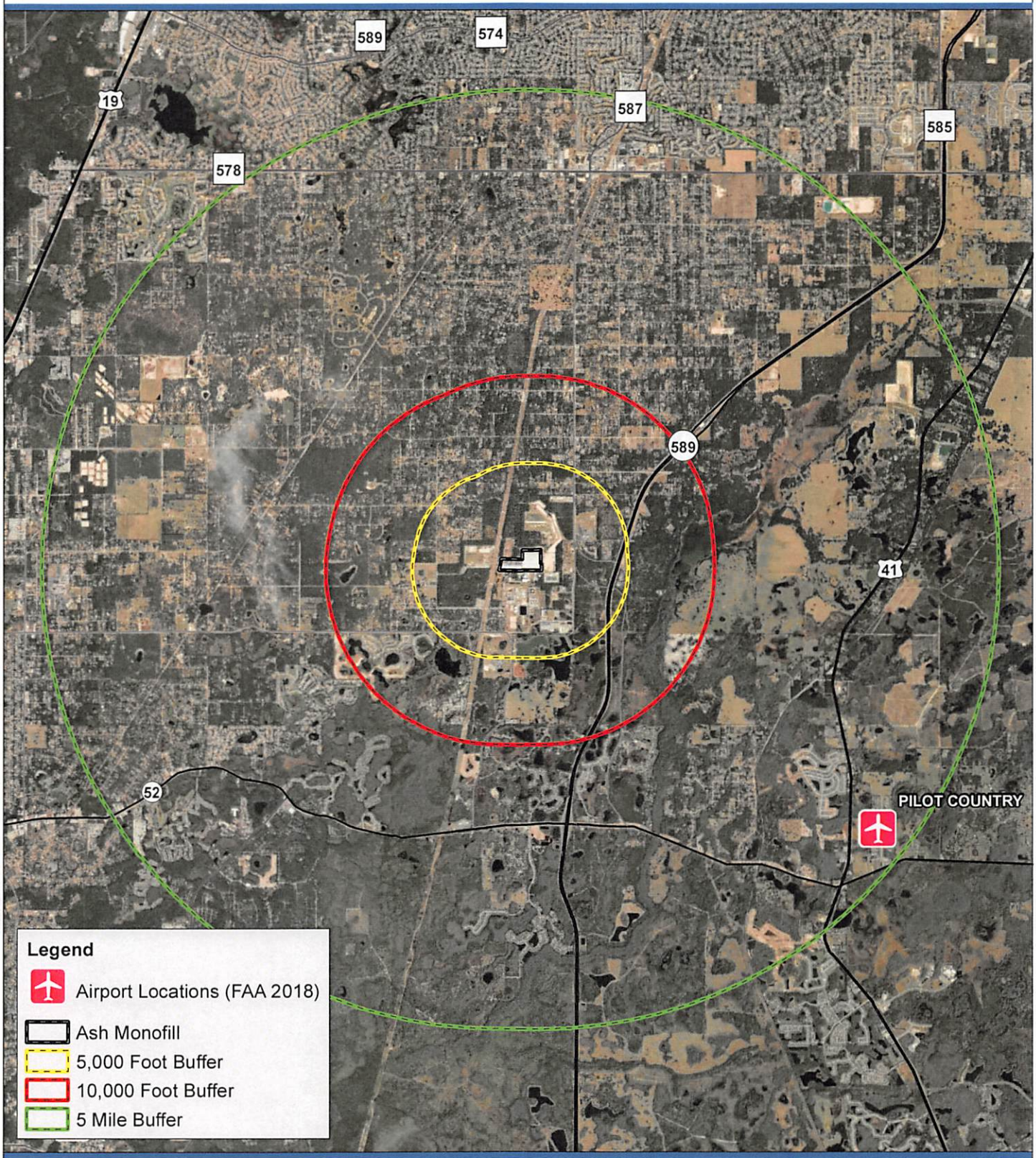
Enclosure

XC: Justin Roessler, Pasco County Solid Waste & Resource Recovery
El Kromhout, Florida Department of Environmental Protection

Figure E.1

Airport Location

West Pasco County Landfill - Ash Monofill



Airport Locations (FAA 2018)
Filename: AIRPORTS_2018
Publisher: Federal Aviation Administration
Date: June 2018
Source: www.fgdl.org

JonesEdmunds

For Informational Purposes Only Y:\16002-PascoCounty\Projects\009-01-AshMonofillOperationsModification\GIS\mxd\FigE1_AirportLocation.mxd JReynolds 3/19/2021

Shipment Receipt

Transaction Date: 12 Jul 2021 **Tracking Number:** 1ZF626R20394472958

1 Address Information

Ship To:
Pilot Country
Aleda Budd, Airport General Manager
11500 Pilot Country Drive
SPRING HILL FL 346107914

Ship From:
Jones Edmunds & Associates
Jodie McGregor
730 NE Waldo Rd
Gainesville FL 32641
Telephone:352-377-5821 x1257
email:jmcgregor@jonesedmunds.com

Return Address:
Jones Edmunds & Associates
Jodie McGregor
730 NE Waldo Rd
Gainesville FL 32641
Telephone:352-377-5821 x1257 email:jmcgregor@jonesedmunds.com

2 Package Information

	Weight	Dimensions / Packaging	Declared Value	Reference Numbers
1.	1.0 lbs (1.0 lbs billable)	Other Packaging		Project #/task # - 16002-009-01-3000 Employee # & or name - jmcgregor for Chad Spreadbury package contents - Notification-Ash Monofill Modify

3 UPS Shipping Service and Shipping Options

Service: UPS Ground Service

Guaranteed By: End of Day Tuesday, Jul 13, 2021

Shipping Fees Subtotal: 12.13 USD

Transportation	8.23 USD
Fuel Surcharge	0.95 USD
Delivery Area Surcharge	2.95 USD
Package 1	

Additional Shipping Options

Quantum View Notify E-mail Notifications: No Charge

1 jmcgregor@jonesedmunds.com: Exception, Delivery
2 cspreadbury@jonesedmund.com: Delivery
3 tmcknight@jonesedmunds.com: Delivery
E-mail Failure Notification: jmcgregor@jonesedmunds.com

4 Payment Information

Bill Shipping Charges to: Shipper's Account F626R2

Shipping Charges:	**12.13 USD**
Subtotal Shipping Charges:	**12.13 USD**
Total Charged:	**12.13 USD**

Note: This document is not an invoice. Your final invoice may vary from the displayed reference rates.

* For delivery and guarantee information, see the UPS Service Guide ({0}). To speak to a customer service representative, call 1-800-PICK-UPS for domestic services and 1-800-782-7892 for international services.

PART E

PART E – LANDFILL PERMIT REQUIREMENTS

1 VICINITY MAP SHOWING AIRPORTS

Figure E.1 shows the airports within 5 miles of the West Pasco County Landfill (WPCL). One airport, Pilot Country Airport of Spring Hill, Florida, is within the 5-mile radius. The airport location information is based on the 2018 Federal Aviation Administration dataset.

2 PLOT PLAN

The Engineering Plans for the *West Pasco County Landfill Ash Monofill Operations Modification* in Appendix A meet the requirements of Rule 62-701.330(3)(b), Florida Administrative Code (FAC), and include a plot plan of the site showing dimensions, proposed and existing water quality monitoring wells, soil borings, proposed plan of disposal areas, original elevations, proposed final contours, previously filled waste areas, and fencing.

a. Dimensions

The Engineering Plans in Appendix A are drawn to scale unless otherwise noted. Dimensions are provided on all plan views and cross sections. Some typical details are not drawn to scale but have dimension callouts.

b. Proposed and Existing Monitoring Wells

No additional monitoring wells are proposed in this permit application. Sheet C2, *Existing Well and Boring Site Plan*, shows the existing monitoring wells.

c. Soil Borings

Additional borings were required to further characterize the subsurface area along the perimeter of the landfill. Sheet C2, *Existing Well and Boring Site Plan*, shows the historical and new soil borings.

d. Disposal Areas

Sheet C1, *Site Plan,* shows the existing Ash Monofill disposal areas. Sheet C4, *Conceptual Final Cover Plan* shows the proposed modifications to the Ash Monofill.

e. Cross Sections

Sheets C5 and C6, *Conceptual Final Cover Sections,* shows the cross sections of the Ash Monofill including the original elevations, proposed side slopes, and final closure contours.

f. Previously Filled Waste Disposal Areas

Sheet C1, *Site Plan,* shows the existing Ash Monofill disposal areas.

g. Fencing or Other Measures to Restrict Access

Sheet C1, *Site Plan*, shows the access to and from the site is controlled by a perimeter chain-link or barbed-wired fence that surrounds the facility along the property boundary. Public access is limited to two gates, both of which are monitored by Pasco County staff from the scalehouses adjacent to the gates. After-hours access to the primary entrance gate off Hays Road is monitored and controlled by the control room operator at the Resource Recovery Facility.

3 TOPOGRAPHIC MAPS

As required by Rule 62-701.330(3)(c), FAC, the Engineering Plans in Appendix A contain topographic maps showing the information below.

a. Proposed Fill Areas

Sheet C1, *Site Plan*, shows the fill area for the Ash Monofil.

b. Borrow Areas

Sheet C1, *Site Plan*, shows the existing permitted borrow area.

c. Access Roads

Sheet C1, *Site Plan*, shows the access roads throughout the site.

d. Grades Required for Proper Drainage

Sheet C4, *Conceptual Final Cover Plan*, and Sheets C5 and C6, *Conceptual Final Cover Sections,* shows the drainage grades for the Ash Monofill.

e. Cross Sections of Lifts

Sheets C5 and C6, *Conceptual Final Cover Sections,* shows the cross sections of the Ash Monofill.

f. Special Drainage Devices

Sheet C7, *Closure Details,* shows the locations, material types, and number of drainage-control devices.

g. Fencing

Access to and from the site is controlled by a perimeter chain-link or barbed-wired fence that surrounds the property. Public access is limited to two gates, both of which are monitored by Pasco County staff from the scalehouses adjacent to the gates. After-hours access to the primary entrance gate off Hays Road is monitored and controlled by the control room operator at the Resource Recovery Facility.

h. Equipment Facilities

Sheet C1, *Site Plan,* shows the existing facilities (e.g., office, scalehouse, maintenance building, tire storage, and white goods storage).

4 LANDFILL REPORT

a. Current and Projected Population

Attachment E.4.a provides the annual population projections for Pasco, which were based on the Bureau of Economic and Business Research (BEBR) *Projections of Florida Population by County, 2020–2045*, Volume 54, Bulletin 189, April 2021, University of Florida.

b. Type, Annual Quantity, and Source of Solid Waste

The expected types and sources of solid waste are described in Section 4 of the Operation Plan in Appendix B.

c. Projected Facility Life

The design volume of the proposed modifications to the ash monofill were calculated based on the current waste surface and proposed final closure design in Appendix A. The design life was calculated using projected solid waste acceptance rates based on waste generation rates for WPCL for the last several years. The calculations show that the modified ash monofill has approximately 18.7 years of airspace available for waste disposal as of January 2021. Attachment E.4.c provides the remaining life and closure estimate calculations.

d. Source and Type of Cover Material

Section 7.5 of the Operation Plan in Appendix B includes information on the source and type of the cover material.

5 WATER QUALITY MONITORING

Water quality monitoring for WPCL is performed by the Pasco County Environmental Laboratory, which is a Florida Department of Environmental Protection (FDEP)-approved laboratory, in accordance with Rule 62-160, FAC. Attachment E.5 provides copies of the laboratories' sampling and testing certifications from the Florida Department of Health for the examination of environmental samples.

6 FINANCIAL RESPONSIBILITY

WPCL has an established management escrow agreement for the closure and long-term care of its facility. A full closure and long-term-care cost estimate was prepared for the ash monofill. Part R provides the calculations and supporting documents.

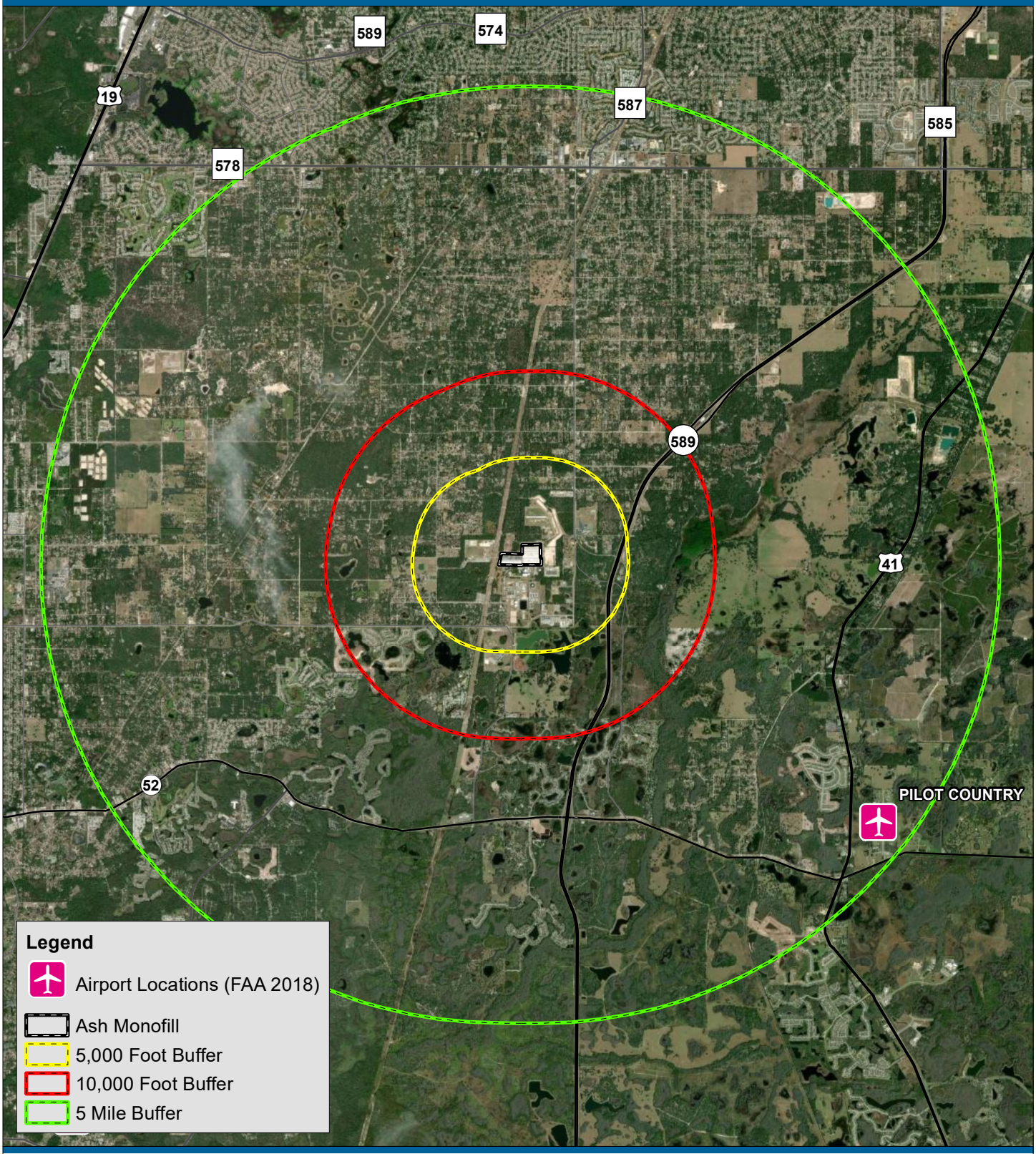
Figure E.1

Airport Location

Figure E.1

Airport Location

West Pasco County Landfill - Ash Monofill



Airport Locations (FAA 2018)
Filename: AIRPORTS_2018
Publisher: Federal Aviation Administration
Date: June 2018
Source: www.fgdl.org

JonesEdmunds

For Informational Purposes Only Y:\16002-PascoCounty\Projects\009-01-AshMonofillOperationsModification\GIS\mxd\FigE1_AirportLocation.mxd JReynolds 3/19/2021

Attachment E.4.a

Population Projections

JonesEdmunds

PROJECT NUMBER: 16002-009-01
PROJECT NAME: West Pasco County Landfill
SUBJECT: Attachment E.4.a - Projected Population Calculations
BY: TLeBlanc DATE: 12/31/2020
CHECKED BY: C.Spreadbury DATE: 6/2/2021

Data:

Bureau of Economic and Business Research - Projections of Florida Population by County[1]						
County	**2020**	**2025**	**2030**	**2035**	**2040**	**2045**
Pasco	542,638					
Low	542,638	543,900	565,700	578,700	586,200	589,400
Medium	542,638	593,000	635,700	668,800	696,400	720,500
High	542,638	637,100	703,000	758,500	809,700	857,600

(1) Reference: Projections of Florida Population by County, 2020-2045, with estimates for 2020, Bureau of Economic and Business Research, Volume 54, Bulletin 189, April 2021

Projected Population:

Projected Population Data			
Year	**Population**		
	Low	**Med**	**High**
2020	542,638	542,638	542,638
2021	542,890	552,356	560,337
2022	543,142	562,248	578,614
2023	543,395	572,317	597,487
2024	543,647	582,567	616,976
2025	543,900	593,000	637,100
2026	548,192	601,304	649,766
2027	552,517	609,725	662,684
2028	556,877	618,263	675,859
2029	561,271	626,921	689,296
2030	565,700	635,700	703,000
2031	568,276	642,186	713,765
2032	570,865	648,739	724,695
2033	573,465	655,358	735,793
2034	576,076	662,045	747,060
2035	578,700	668,800	758,500
2036	580,192	674,231	768,474
2037	581,688	679,706	778,580
2038	583,188	685,226	788,818
2039	584,692	690,790	799,191
2040	586,200	696,400	809,700
2041	586,839	701,155	819,061
2042	587,478	705,942	828,530
2043	588,118	710,762	838,109
2044	588,759	715,614	847,799
2045	589,400	720,500	857,600

Reference 1

Reference #1

College of Liberal Arts and Sciences
Bureau of Economic and Business Research

Florida Population Studies

BEBR

Volume 54, Bulletin 189, April 2021

Projections of Florida Population by County, 2025–2045, with Estimates for 2020

Stefan Rayer, Population Program Director
Ying Wang, Research Demographer

The Bureau of Economic and Business Research (BEBR) has been making population projections for Florida and its counties since the 1970s. This report presents our most recent set of projections and describes the methodology used to construct those projections. To account for uncertainty regarding future population growth, we publish three series of projections. We believe the medium series is the most likely to provide accurate forecasts in most circumstances, but the low and high series provide an indication of the uncertainty surrounding the medium series. It should be noted that these projections refer solely to permanent residents of Florida; they do not include tourists or seasonal residents. Furthermore, we note that this set of projections is still based on the Census 2010 counts and the BEBR population estimates since then. The next set of BEBR county projections, scheduled for release in early 2022, will incorporate the Census 2020 counts.

State projections

The starting point for the state-level projections was the April 1, 2010 census population count by age, sex, race, and Hispanic origin, as adjusted by the National Center for Health Statistics (NCHS) in the Vintage 2017 bridged race population estimates. Projections were made in one-year intervals using a cohort-component methodology in which births, deaths, and migration are projected separately for each age-sex cohort in Florida for non-Hispanic whites, non-Hispanic nonwhites, and Hispanics. We applied three different sets of assumptions to provide low, medium, and high series of projections. Although the low and high series do not provide absolute bounds on future population change, they provide a reasonable range in which Florida's future population is likely to fall.

Survival rates were applied by single year of age, sex, race, and Hispanic origin to project future deaths in the population. These rates were based on Florida Life Tables for 2012–2018, using mortality data published by the Office of Vital Statistics in the Florida Department of Health. The survival rates were adjusted upward each year until 2044 to account for projected increases in life expectancy. These adjustments were based on projected increases in survival rates released by the U.S. Census Bureau. We used the same mortality assumptions for all three series of projections because there is less uncertainty regarding future changes in mortality rates than is true for migration and fertility rates.

Domestic migration rates by age and sex were based on Public Use Microdata Sample (PUMS) files from the 2010–2019 American Community Survey (ACS) 1-year estimates, and the 2014–2018 and 2015–2019 ACS 5-year estimates. We first calculated an average of the 2010–2018 1-year estimates and the 2014–2018 5-year estimates. Next, we calculated an average of the 2011–2019 1-year estimates and the 2015–2019 5-year estimates. Our final domestic migration rates were based on an average of these two

sets of migration estimates. Projections based on input data from more than one time period tend to be more accurate than those based on a single time period. By combining 1-year ACS estimates, which are more current, with 5-year ACS estimates, which are more stable, we make use of the different strengths of each type of ACS data.

For all three racial/ethnic groups, we applied smoothing techniques to the age/sex-specific migration rates to adjust for data irregularities caused by small sample size. The smoothed in- and out-migration rates were weighted to account for recent changes in Florida's population growth rates. Projections of domestic in-migration were made by applying weighted in-migration rates to the projected population of the United States (minus Florida), using the most recent set of national projections produced by the U.S. Census Bureau. Projections of out-migration were made by applying weighted out-migration rates to the Florida population. In both instances, rates were calculated separately for males and females by race and ethnicity for each age up to 90 and over.

For the medium projection series, in-migration weights for non-Hispanic whites varied from 1.26 to 1.02, and out-migration weights varied from 0.96 to 0.98; for non-Hispanic nonwhites and for Hispanics, in-migration weights varied from 1.23 to 1.02, and out-migration weights varied from 0.99 to 0.98. For the low projection series, the in-migration weights described above were lowered for all three racial/ethnic groups over time – from 7% in 2021 to 11% in 2045; the out-migration weights were raised by the same margins. For the high projection series, the in-migration weights described above were raised for all three racial/ethnic groups over time – from 7% in 2021 to 11% in 2045; the out-migration weights were lowered by the same margins.

The distribution of foreign immigrants for the three racial/ethnic groups by age and sex was also based on averages of the patterns observed over the same time periods using the same ACS data sets as for domestic migration. Again, we smoothed the estimates to account for irregularities in the age/sex distribution of immigrants. For the medium projection series, we held foreign immigration at an average of the observed levels, with some short-term adjustments based on recent trends. In addition, we made minor adjustments to the racial/ethnic distribution of those migrants based on recent trends. For the low series, foreign immigration was projected to decrease by 1,250 per year from the average of the observed levels; for the high series, foreign immigration was projected to increase by 1,000 per year. Foreign emigration was assumed to equal 25% of foreign immigration for each series of projections.

Projections were made in one-year intervals, with each projection serving as the base for the following projection. Projected in-migration for each one-year interval was added to the survived Florida population at the end of the interval and projected out-migration was subtracted, giving a projection of the population age one and older.

Births were projected by applying age-specific birth rates (adjusted for child mortality) to the projected female population of each racial/ethnic group. These birth rates were based on Florida birth data for 2012–2018 published by the Office of Vital Statistics in the Florida Department of Health. They imply a total fertility rate (TFR) of 1.64 births per woman for non-Hispanic whites, 1.91 births per woman for non-Hispanic nonwhites, 1.78 births per woman for Hispanics, and 1.75 births per woman for total population. These rates were adjusted in the short-term projections to make them consistent with recent fertility trends. We also raised them long-term, though to a lesser extent than in previous years. We still expect fertility rates to increase, but more slowly and to a lower level than previously projected. We made this downward adjustment because recorded resident births in Florida, after having increased each year from 2012 through 2016, have trended downward again over the past four years (the birth data for 2020 are still provisional). By 2030, the adjusted rates imply a total fertility rate of 1.68 births per woman for non-Hispanic whites, 1.97 births per woman for non-Hispanic nonwhites, 1.84 births per

woman for Hispanics, and 1.80 births per woman for total population.

As a final step, projections for non-Hispanic whites, non-Hispanic nonwhites, and Hispanics were added together to provide projections of the total population. The medium projections of total population for 2021–2025 were adjusted to be consistent with the state population forecasts for those years produced by the State of Florida's Demographic Estimating Conference (DEC) held March 3, 2021. None of the projections after 2025 had any further adjustments. In this publication, we provide projections for 2025, 2030, 2035, 2040, and 2045. State projections for other years are available by request.

County projections

The cohort-component method is a good way to make population projections at the state level but is not necessarily the best way to make projections at the county level. Many counties in Florida are so small that the number of persons in each age-sex category is inadequate for making reliable cohort-component projections, given the lack of detailed small-area data. Even more important, county growth patterns are so volatile that a single technique based on data from a single time period may provide misleading results. We believe more useful projections of total population can be made by using several different techniques and historical base periods.

For counties, we started with the population estimate constructed by BEBR for April 1, 2020. We made projections for each county using five different techniques in five-year increments. The five techniques were:

1. Linear – the population will change by the same number of persons in each future year as the average annual change during the base period.

2. Exponential – the population will change at the same percentage rate in each future year as the average annual rate during the base period.

3. Share-of-growth – each county's share of state population growth in the future will be the same as its share during the base period.

4. Shift-share – each county's share of the state population will change by the same annual amount in the future as the average annual change during the base period.

5. Constant-share – each county's share of the state population will remain constant at its 2020 level.

For the linear and share-of-growth techniques we used base periods of two, ten, and twenty years (2018–2020, 2010–2020, and 2000–2020), yielding three sets of projections for each technique. For the exponential and shift-share techniques we used base periods of five and fifteen years (2015–2020 and 2005–2020), yielding two sets of projections for each technique. The constant-share method was based on data for a single year (2020).

This methodology produced eleven projections for each county for each projection year (2025, 2030, 2035, 2040, and 2045). From these, we calculated five averages: one using all eleven projections (AVE-11), one that excluded the highest and lowest projections (AVE-9), one that excluded the two highest and two lowest projections (AVE-7), one that excluded the three highest and three lowest projections (AVE-5), and one that excluded the four highest and four lowest projections (AVE-3). Based on the results of previous research, we designated the average that excluded the three highest and three lowest projections (AVE-5) as the default technique for each county. We evaluated the resulting projections by comparing them with historical population trends and with the level of population growth projected for the state as a whole. For counties in which AVE-5 did not provide reasonable projections, we selected the technique producing projections that fit most closely with our evaluation criteria.

For 52 counties we selected AVE-5, the average in which the three highest and three lowest projections

were excluded. In the remaining 15 counties, we selected projections made from an individual technique or calculated a custom average (e.g., an average of two or more of the individual techniques, or an average of one or more of the individual techniques and one of the above described averages). These include Bay, Calhoun, Franklin, Gadsden, Gulf, Hardee, Holmes, Jackson, Jefferson, Liberty, Madison, Monroe, Pinellas, Sumter, and Union counties.

We also made adjustments in several counties to account for changes in institutional populations such as university students and prison inmates. Adjustments were made only in counties in which institutional populations account for a large proportion of total population or where changes in the institutional population have been substantially different than changes in the rest of the population. In the present set of projections, adjustments were made for Alachua, Baker, Bradford, Calhoun, Columbia, DeSoto, Dixie, Franklin, Gadsden, Gilchrist, Glades, Gulf, Hamilton, Hardee, Hendry, Holmes, Jackson, Jefferson, Lafayette, Leon, Liberty, Madison, Okeechobee, Santa Rosa, Sumter, Suwannee, Taylor, Union, Wakulla, Walton, and Washington counties.

We did not make a specific adjustment to these county projections to account for any potential impacts of the COVID-19 pandemic. Pandemic-related population effects are highly uncertain at the local level, and there are no good data available yet to assess them. Furthermore, the latest state projections from the March 3, 2021 DEC, to which these county projections are controlled, show a similar state total for April 1, 2025 as the state projections adopted at the December 3, 2019 DEC before the pandemic. Thus, while there may be some short-term effects, we currently expect no significant impacts from the pandemic on the projections for 2025 and beyond.

Range of county projections

The techniques described in the previous section were used to construct the medium series of county projections. This is the series we believe will generally provide the most accurate forecasts of future population change. We also constructed low and high projections to provide an indication of the uncertainty surrounding the medium county projections. The low and high projections were based on analyses of past population forecast errors for counties in Florida, broken down by population size and growth rate. They indicate the range into which approximately three-quarters of future county populations will fall, if the future distribution of forecast errors is similar to the past distribution.

The range between the low and high projections varies according to a county's population size in 2020 (less than 30,000; 30,000 to 199,999; and 200,000 or more), rate of population growth between 2010 and 2020 (less than 7.5%; 7.5–15%; 15–30%; and 30% or more), and the length of the projection horizon (on average, projection errors grow with the length of the projection horizon). Our studies have found that the distribution of absolute percent errors tends to remain fairly stable over time, leading us to believe that the low and high projections provide a reasonable range of errors for most counties. It must be emphasized, however, that the actual future population of any given county could be below the low projection or above the high projection.

For the medium series of projections, the sum of the county projections equals the state projection for each year (except for slight differences due to rounding). For the low and high series, however, the sum of the county projections does not equal the state projection. The sum of the low projections for counties is lower than the state's low projection and the sum of the high projections for counties is higher than the state's high projection. This occurs because potential variation around the medium projection is greater for counties than for the state as a whole.

Acknowledgement

Funding for these projections was provided by the Florida Legislature.

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Reference #1

Projections of Florida Population by County, 2025–2045, with Estimates for 2020 (continued)

County and State	Estimates April 1, 2020	2025	2030	2035	2040	2045
MIAMI-DADE	2,832,794					
Low		2,791,500	2,844,300	2,871,900	2,880,900	2,876,900
Medium		2,992,700	3,128,300	3,234,600	3,322,200	3,398,200
High		3,186,900	3,423,300	3,621,000	3,799,900	3,964,600
MONROE	77,823					
Low		73,600	72,200	70,500	68,700	66,700
Medium		78,800	79,400	79,800	80,000	80,200
High		84,300	87,500	90,100	92,200	94,200
NASSAU	89,258					
Low		89,500	93,600	96,800	99,300	100,800
Medium		99,200	107,500	114,600	121,100	126,900
High		108,200	121,600	134,900	148,200	161,400
OKALOOSA	203,951					
Low		200,200	202,800	204,000	204,500	203,800
Medium		214,600	223,200	230,000	236,000	241,100
High		228,500	244,000	257,200	269,700	280,900
OKEECHOBEE	42,112					
Low		40,600	40,500	40,300	39,900	39,500
Medium		43,400	44,500	45,300	46,000	46,700
High		46,500	49,100	51,500	53,700	55,900
ORANGE	1,415,260					
Low		1,429,800	1,494,500	1,542,300	1,577,800	1,602,500
Medium		1,558,700	1,678,400	1,777,900	1,864,300	1,941,800
High		1,674,800	1,857,100	2,021,600	2,179,700	2,331,800
OSCEOLA	387,055					
Low		409,200	447,500	476,000	499,500	518,300
Medium		453,600	512,500	560,700	603,600	643,100
High		491,900	574,500	649,300	723,900	798,500
PALM BEACH	1,466,494					
Low		1,440,900	1,465,500	1,482,000	1,491,200	1,491,500
Medium		1,544,900	1,612,200	1,668,600	1,717,000	1,758,500
High		1,645,000	1,763,800	1,868,600	1,966,800	2,055,400
PASCO	542,638					
Low		543,900	565,700	578,700	586,200	589,400
Medium		593,000	635,700	668,800	696,400	720,500
High		637,100	703,000	758,500	809,700	857,600
PINELLAS	984,054					
Low		960,600	959,500	953,100	943,000	930,500
Medium		1,011,800	1,031,400	1,045,200	1,055,500	1,063,800
High		1,067,900	1,116,700	1,153,300	1,184,100	1,210,200
POLK	715,090					
Low		718,300	747,800	770,200	785,000	794,600
Medium		783,100	840,200	888,400	929,300	965,800
High		841,400	929,200	1,009,500	1,084,500	1,156,200
PUTNAM	73,723					
Low		69,300	67,900	66,400	64,800	63,300
Medium		74,200	74,700	75,100	75,500	75,800
High		79,400	82,200	84,900	87,100	89,400
ST. JOHNS	261,900					
Low		274,700	297,000	314,400	328,600	339,400
Medium		304,600	340,500	370,900	398,000	422,800
High		330,200	381,300	428,800	476,200	522,900
ST. LUCIE	322,265					
Low		326,300	342,800	353,400	360,300	364,400
Medium		355,800	384,800	407,500	426,400	443,100
High		382,300	426,000	463,200	497,800	530,300

Attachment E.4.c

Projected Facility Life

JonesEdmunds

PROJECT NUMBER: 16002-009-01
PROJECT NAME: West Pasco County Landfill Ash Monofill Operations Modification
SUBJECT: Attachment E.4.c - Projected Facility Life
BY: C.SPREADBURY DATE: 3/15/2021
CHECKED BY: T.LEBLANC DATE: 6/8/2021

Objective: To determine the remaining life from present day of the Class I Ash Monofill Cells (A-1 through A-4) at West Pasco County Landfill

A. Calculate the average waste generation growth rate
B. Calculate the expected lifespan

Data:

Starting Annual Waste Disposal Rate of MSW to WtE (FY2021):		331,000	Tons MSW	*(Reference 2)*
MSW-to-Ash Conversion		0.23	Ton Ash/Ton MSW	*(Reference 2)*
Starting Annual Waste Disposal Rate of Ash (FY2021):		76,130	Tons Ash	
Remaining Volume with Slope Modification and Vertical Expansion (as of 1/1/2021):	V_{mod} =	1,790,000	CY	*(Reference 3)*
Estimated lifespan with expansion volume (assuming constant WTE rate)		25.9	Years	*(starting from 1/1/2021)*
Estimated lifespan with expansion volume (assume 520,000 TDY increase after 2027)		18.7	Years	*(starting from 1/1/2021)*

Assumptions:

1. Waste-to-energy (WtE) Facility operates at capacity resulting in 331,000 tons of waste processed per year *(Reference 2)*
2. Waste (ash) apparent density is 2,200 lb/CY *(Reference 2)*
3. Future waste generation rates are represented by the average waste generation growth rate from the last five years
4. Facility expansion is anticipated to be complete starting the 2027 Calendar Year which will increase ash generation rate. *(Reference 2)*

Waste density:

$$\rho_l = \frac{Class\ I\ Materials\ Received\ (tons)}{Volume\ Consumed\ (CY)} * (2000 \frac{\text{lbs}}{\text{ton}})$$

2,200 lb/CY
1.10 tons/CY

A. Projected Waste Generation Growth Rate

Waste Generation Growth Rates

Fiscal Year[1]	Waste Generation[2] (tons)	Waste Generation Growth Rate
2015	338,197	
2016	354,273	4.75%
2017	364,947	3.01%
2018	378,733	3.78%
2019	386,807	2.13%
	Average Growth Rate	3.42%

Notes:
(1) Fiscal year, October 1 to September 30; however, remaining life is measured from January 1, 2021
(2) Waste generation data based off of Interoffice Memorandum *(Reference 1)*

B. Lifespan Calculations

Lifespan Calculations

Fiscal Year[1]	Average Waste Generation Growth Rate (%/yr)	Total Class I Waste Generated (tons/yr)	Total Class I Waste Processed by WtE[3] (tons/yr)	Total Ash Generated by WtE Facility (tons/yr)	Total Ash Disposed of in Ash Monofill[5] (tons/yr)	Volume Consumed, Ash Monofill (CY/yr)	Remaining Volume, Ash Monofill[6] (CY)
2019[2]	3.42%	386,807	331,000	76,130	76,130	69,209	
2020	3.42%	400,032	331,000	76,130	76,130	69,209	1,790,000
2021	3.42%	413,709	331,000	76,130	76,130	69,209	1,720,791
2022	3.42%	427,853	331,000	76,130	76,130	69,209	1,651,582
2023	3.42%	442,481	331,000	76,130	76,130	69,209	1,582,373
2024	3.42%	457,610	331,000	76,130	76,130	69,209	1,513,164
2025	3.42%	473,255	331,000	76,130	76,130	69,209	1,443,955
2026	3.42%	489,436	331,000	76,130	76,130	69,209	1,374,745
2027[4]	3.42%	506,169	506,169	116,419	116,419	105,835	1,268,910
2028	3.42%	523,475	520,000	119,600	119,600	108,727	1,160,183
2029	3.42%	541,373	520,000	119,600	119,600	108,727	1,051,455
2030	3.42%	559,882	520,000	119,600	119,600	108,727	942,728
2031	3.42%	579,024	520,000	119,600	119,600	108,727	834,001
2032	3.42%	598,821	520,000	119,600	119,600	108,727	725,274
2033	3.42%	619,294	520,000	119,600	119,600	108,727	616,546
2034	3.42%	640,468	520,000	119,600	119,600	108,727	507,819
2035	3.42%	662,365	520,000	119,600	119,600	108,727	399,092
2036	3.42%	685,011	520,000	119,600	119,600	108,727	290,365
2037	3.42%	708,431	520,000	119,600	119,600	108,727	181,637
2038	3.42%	732,652	520,000	119,600	119,600	108,727	72,910
2039	3.42%	757,701	520,000	119,600	119,600	108,727	-35,817
				Remaining Life Starting from January 1, 2021			**18.7**
				Anticipated Closure (Rounded to the nearest month)			**August-2039**

Notes:
(1) Fiscal year, October 1 to September 30; however, remaining life is measured from January 1, 2021.
(2) Values from 2019 are actual values as reported by Interoffice Memorandum *(Reference 1)* .
(3) Assumes that Waste-to-energy Facility operates at capacity resulting in 331,000 tons of waste processed per year, until capacity increase in 2027 *(Reference 2)* .
(4) Proposed start date of WtE capacity increase (520,000 TPY anticipated maximum capacity) *(Reference 2, Table 1)* .
(5) Assumes all ash generated from WtE is landfilled at ash monofill (i.e., no significant beneficial reuse activities).
(6) Calculated as remaining volume as of December 31 of that year.

References:

1. *Interoffice Memorandum from Tim Treshler to Michael Carballa, January 24, 2020*
2. *Remaining Disposal Capacity and Site Life Calculations, JMG Engineering, Inc., February 2021*
3. *Additional Capacity from Modifications Calculation, Jones Edmunds Associates*

Reference 1

Reference 1 - Tonnage Data

PASCO COUNTY FLORIDA

PUBLIC INFRASTRUCTURE BRANCH
Solid Waste and Resource Recovery

INTEROFFICE MEMORANDUM

DATE: January 24, 2020

TO: Michael J. Carballa, P. E., BCEE, Assistant County Administrator (Public Infrastructure)

THRU: John Power, Public Infrastructure, Solid Waste Director 1/28/2020

FROM: Tim Treshler, Public Infrastructure, Sr. Project Manager Solid Waste 1/28/2020

FILE: UTSW20-0230

SUBJECT: Quarterly Tonnage Report, Waste-to-Energy Facility, December, 2019

REFERENCES:

Attached are the latest monthly totals for Solid Waste and Ash processed at the Pasco County Waste-To-Energy Facility. This report is provided quarterly, but presented by month, with daily averages.

	Oct.	Nov.	Dec.	
Current Quarter's ratio of ash to waste by month:	24%	28%	26%	
This Fiscal Year's monthly average ratio ash to waste =				26%
Current Quarter's processable waste by month:	27,090	28,612	30,933	Tons
Current Quarter's Recycling totals by month:	691.25	627.57	582.16	** Tons

** as of October 2011, this figure is based on Recycling hauled out

FY 2020 WASTE SUMMARY:

86,635	Tons of MSW delivered to the Waste-to-Energy Facility to date
0	Tons Bypassed to SW Cells (Not Included in WTE Report Tonnage)
11,711	Tons Hauled Out of County (Not Included in WTE Tonnage)
98,346	Total MSW to date for Fiscal Year

PAST MSW ACTIVITY:

FY 15	FY 16	FY 17	FY 18	FY 19	
338,197	364,273	364,947	378,733	366,907	*Total MSW for Fiscal Year*
337,622	342,646	339,815	341,122	342,994	*Tons of MSW delivered to the Waste-to-Energy Facility during FY*
0	3,156	0	15,093	10,306	*Tons of MSW Bypassed to the SW Cells during FY*
575	8,472	25,333	22,518	33,507	*Tons of MSW Hauled out of the County during FY*

cc: Justin G. Grant, Public Infrastructure Fiscal and Business Administration Director
Joanne Chamberlain, Public Infrastructure Fiscal and Business Administration, Accountant II
Justin Roessler, P.E., Assistant Solid Waste Director
Rachelle Dobbs, Recycling Supervisor

Mission: Providing our community, customers, and future generations with high quality, efficient, and innovative water, wastewater, and solid waste services

Reference 2

Smith, George

From: Jason Gorrie <jason@jmg-eng.com>
Sent: Friday, February 26, 2021 10:42 AM
To: SWD_Waste
Cc: Timothy J. Treshler; Carrie S. Kimble
Subject: Pasco County Class I Landfill - 2020 Remaining Useful Life Report
Attachments: Class I Landfill Remaining Useful Life Report - Reporting Year 2020.pdf

Jason M. Gorrie, P.E., BCEE
President

JMG Engineering, Inc.
3825 Henderson Blvd.
Suite 604
Tampa, Florida 33629
(813) 605-0706 (office)
(813) 220-0653 (cell)

Pasco County Utilities

Remaining Disposal Capacity and Site Life Calculations
Pasco County Resource Recovery Facility

Reporting Year 2020

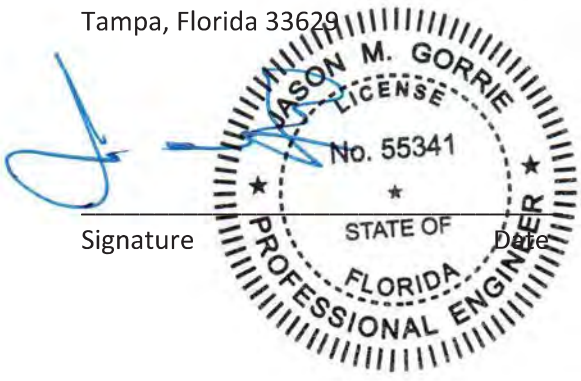
February 2021

Prepared For:

Pasco County Utilities

Prepared By:

Jason Gorrie, P.E.
JMG Engineering, Inc.
3825 Henderson Blvd.
Suite 604
Tampa, Florida 33629



Signature Date

1.0 Purpose of Report

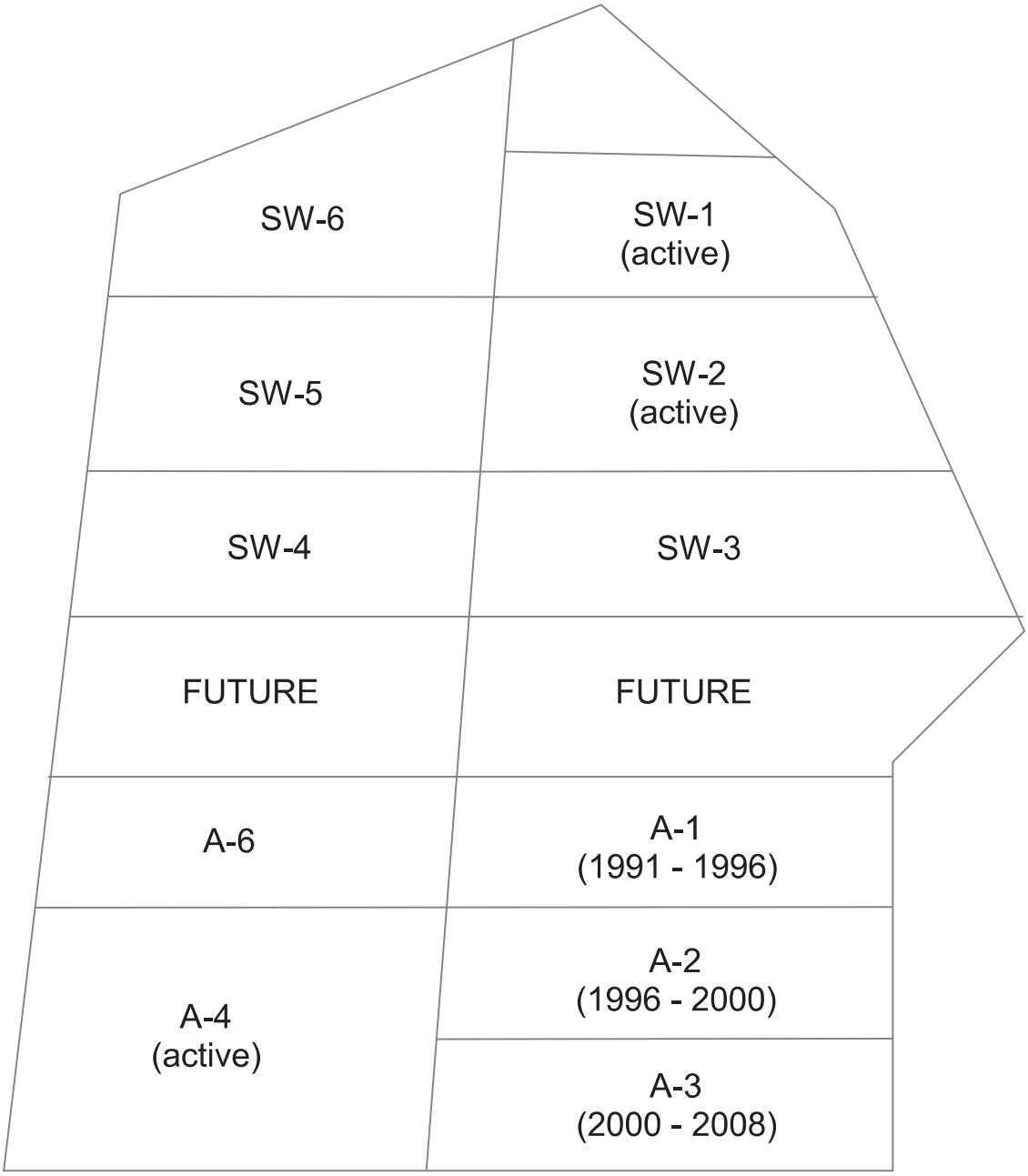
Rule 62-701.500(13)(c) of the Florida Administrative Code (FAC) requires landfill owners and operators to annually prepare an estimate of the remaining life and capacity (in cubic yards) of the existing, constructed landfill and an annual estimate of the life and capacity of other permitted areas not yet constructed. For the purposes of this Report, the "existing, constructed landfill" is considered to be the currently active portions of the site designated as A-4 (for combustion ash) and SW-1/SW-2 (for unprocessed municipal solid waste.) The remaining useful life of disposal cell A-4 is presented in **Section 3.0**. The remaining useful life of disposal cell SW-1/SW-2 is presented in **Section 4.0**. Finally, the remaining useful life of future disposal cells permitted by Power Plant Siting Act Conditions of Certification PA 87-23C are presented in **Section 5**.

2.0 Assumptions Used in Report

- The primary disposal mechanism for municipal solid waste generated within Pasco County is the Pasco County Resource Recovery Facility. The Resource Recovery Facility is permitted to combust approximately 1,050 tons per day of municipal solid waste. Residue from the combustion of the municipal solid waste (hereinafter referred to as "ash") is disposed in those areas identified in **Figure 1** as "Ash Disposal Cells." As of the writing of this Report, disposal cells A-1, A-2, and A-3 are temporarily inactive and not receiving ash. While these cells are not currently receiving ash and are temporarily capped with a tarp to minimize leachate generation, they are still available for additional disposal in the future. Disposal cell A-4 is the currently active disposal cell for ash residue and receives all ash generated by the Resource Recovery Facility[1].

- The actual mass value of waste that can be processed by the Resource Recovery Facility is variable due to the heating value of the waste. Drier waste and waste containing larger amounts of plastics has a higher heating value, resulting in less mass throughput through the Resource Recovery Facility. Wetter waste and waste containing less plastics has a lower heating value, resulting in more mass throughput through the Resource Recovery Facility. The heating value of the waste (measured in British thermal units per pound of waste, or Btu/lb) has been observed to be gradually increasing over the course of several years, resulting in a slightly lower mass throughput capability of the Resource Recovery Facility. For purposes of this Report, it is assumed that the annual solid waste mass throughput capacity of the Resource Recovery Facility will remain at 1,050 tons per day for the foreseeable future. Assuming no changes in facility availability, the Resource Recovery Facility is capable of processing approximately 331,000 tons per years given the 1,050 tons per day design capacity, 90% availability and a standard higher heating value of 4,800 Btu per pound. (CDM Smith, 2015)

[1] The Florida Department of Environmental Protection has authorized Pasco County to beneficially utilize a significant portion of the ash generated by the Resource Recovery Facility. For purposes of this Report, it is conservatively assumed that all ash will be placed in the Ash Disposal Cells and not beneficially utilized. It should be noted that future beneficial reuse projects undertaken in accordance with the FDEP's approval could substantially reduce the amount of ash placed in the Ash Disposal Cells in future years.



LEGEND

A-x Ash Disposal Cells
SW-x Solid Waste Disposal Cells

JMG ENGINEERING INC

FIGURE 1

Class I Landfill Footprint

2019 Useful Life
Estimate Pasco County
Class I Landfill

- The quantity of ash generated by the combustion process at the Resource Recovery Facility is relatively consistent at 23% (by mass) of the municipal solid waste combusted. (Pasco County Utilities, 2018) Previous studies by Covanta and others have demonstrated that the density of the ash is also relatively consistent over protracted periods at 2,200 pounds per cubic yard. Municipal solid waste that is bypassed from the Resource Recovery Facility and placed in the solid waste disposal cells is assumed to have a compacted density of approximately 1,100 pounds per cubic yard.

- It is widely recognized that the population of Pasco County is growing. Future waste generation rates were estimated in 2015 based on population projections generated by the University of Florida's Bureau of Economic and Business Research. These numbers are relied upon in this Report and presented in Table 1.

- The projections presented in this Report rely on multiple assumptions (such as population growth, WTE Facility combustion efficiency, landfill operation efficiency, planned future combustion capacity, etc.) The predicted dates are considered to be accurate to ± 3 years.

2.1 Permitted Disposal Capacity

Construction and operation of the disposal cells at the site is governed by Power Plant Siting Act Conditions of Certification PA87-23C. The permitted maximum elevation of the currently active Ash Disposal Cell (A-4) is 120 feet NGVD. In 2015, CDM-Smith estimated the total volume available for disposal in cell A-4 to be 1,254,269 cubic yards (CDM-Smith, 2015).

In February of 2020, the FDEP approved a modification to the Landfill Operations Plan that allows for a new fill sequence that combines existing disposal cells SW-1 and SW-2 into a common disposal cell. The modification also increased the maximum elevation of the combined cell from 101 feet NGVD to 121 feet NGVD. This new fill sequence and maximum permitted height result in an estimated total volume available for disposal of approximately 1.6 million cubic yards.

3.0 Useful Remaining Life of Cell A-4

The currently permitted capacity of Ash Disposal Cell A-4 is considered to be 1,254,269 cubic yards. As of January 2021, all six subcells are considered to be "active", however, only subcells 4 and 5 are receiving ash for disposal. Subcell 6 is being utilized for ash and metals processing only. In order to estimate the current volume of waste in place in Cell A-4, D.C. Johnson Associates conducted a topographic survey of the active cell in December 2020. The elevation contours of the A4 disposal cell are depicted at **Attachment 1**.

Using the elevations reported on the topographic survey in conjunction with the known elevations of the liner system, it is possible to estimate the volume of ash in place in cell A-4. Mathematically, the volume under a function is defined by the double integral:

$$\int_{x-min}^{x-max} \int_{y-min}^{y-max} f(x,y)dx\,dy$$

To determine the function f(x, y) defining the shape of the landfill surfaces, JMG Engineering relied upon the volume calculation algorithms of the surface mapping software program Surfer® 13. The total volume of ash calculated to be currently in place in A-4 is approximately 500,948 cubic yards.

Given the permitted volumetric capacity of 1,254,269 cubic yards and subtracting the 527,702 cubic yards calculated from the 2020 survey to be currently in place, approximately 726,567 cubic yards of airspace remains available for disposal. Municipal waste combustor ash has an approximate density of 2,200 pounds per cubic yard. Using the assumed density, the 726,567 remaining cubic yards can accommodate approximately 799,224 tons of ash.

Table 1 below shows the projected ash generation rate as a function of anticipated Resource Recovery Facility capacity. Because the waste-to-energy facility is currently operating at its maximum capacity, no changes in the ash generation rate are expected in the immediate future. However, it is anticipated that the planned Facility expansion will be complete by the year 2026 and will result in an increased ash generation rate of approximately 87,000 cubic yards beginning in Calendar Year 2027. As shown in Table 1, this increased ash generation rate **will consume the remaining available airspace of A4 by approximately 2029.**

4.0 Useful Remaining Life of Cell SW-2

The calculation procedures used to estimate the remaining useful life for disposal cell SW-1/SW-2 are identical to those described above for disposal cell A-4. Utilizing the surveyed elevations provided by Pasco County Engineering Services Department (see **Attachment 2**), the Surfer 13® algorithms determined that the volume of solid waste present in SW-1/SW-2 at the beginning of the fiscal year is approximately 790,780 cubic yards (see **Attachment 4**).

The new permitted volumetric capacity of SW-1/SW-2 is assumed to be 1.6 million cubic yards (see **Attachment 6**). Subtracting the 790,780 cubic yards calculated to be currently in place, approximately 809,220 cubic yards of airspace remains available for disposal.

The intent of the solid waste cells (inclusive of SW-1/SW-2) as stated in the original application for Power Plant Siting Certification is to accommodate solid waste deliveries in excess of the Resource Recovery Facility's capacity. As stated previously, the Resource Recovery Facility is operating at capacity. This necessitates periodic (and temporary) diversions during times when the Resource Recovery Facility must operate at reduced capacity to facilitate maintenance activities. The rate of waste placement in SW-1/SW-2 is also heavily dependent upon year-to-year budget decisions about the amount of waste to haul out of County for disposal.

Table 1 below shows the projected waste diversion volume as a function of anticipated Resource Recovery Facility capacity and future population growth[2]. Using an assumed density of 1,100 pounds per cubic yard for municipal solid waste compacted in-place (USEPA, 2016), and further assuming that daily cover equivalent to 15% (by volume) per year will be utilized, it is estimated that remaining airspace will be consumed at the rate identified in Table 1. Accordingly, **the estimated remaining useful life of cell**

[2] The diversion rates listed in Table 1 are conservative. The actual diversion rate is expected to be less than that presented in Table 1 because the Resource Recovery Facility has demonstrated an ability to process waste in excess of the 331,000 tons per year listed in the 2015 Solid Waste Master Plan.

SW-1/SW-2 (as currently designed and permitted) is at least 10 years using a combination of exporting waste out-of-county and disposal in SW-1/SW-2[3].

5.0 Useful Remaining Life of Future Disposal Cells

As shown in Figure 1, several acres of permitted disposal space remain available for future use by the County. At present, liner and leachate collection systems have been constructed for the following disposal cells: SW-1, SW-2, A-1, A-2, A-3, and A-4. Because these lined cells are adjacent to each other, it is possible to maximize the airspace over existing lined areas by utilizing the interstitial space (or valley) between the existing cells. Such a practice will delay the need for construction of additional liner and leachate collection systems.

Finally, those cells identified in Figure 1 as SW-3, SW-4, SW-5, and SW-6 are sited and permitted as disposal areas but are yet to be constructed. Assuming that the ultimate capacity of these yet-to-be constructed cells will have an approximate capacity of 750,000 cubic yards each[4], an additional 3 million cubic yards of disposal volume for MSW remains available to Pasco County. This equates to roughly 13 years of disposal capacity at the reasonably foreseeable MSW disposal rates anticipated in the future, as depicted in **Table 2**. Use of these permitted cells for disposal of ash (rather than uncombusted MSW) from the Resource Recovery Facility) will prolong the remaining useful life even further.

[3] Rapid population growth has necessitated increased reliance on out-of-county hauling. It is assumed that out-of-county hauling will continue to be heavily relied upon until the waste-to-energy expansion is completed in 2025.
[4] It is assumed that landfill operators will be able to achieve greater compaction efficiencies and lower daily cover application rates if it becomes necessary to operate the solid waste disposal cells in a permanent (rather than intermittent) fashion, resulting in a larger disposal capacity than is currently achieved in SW-1/SW-2.

Table 1

Table 1
Estimated Waste Generation Rates, Waste Disposal Rates, and Remaining Disposal Capacity

	Resource Recovery Facility				A-4 Disposal Cell			SW-1/SW-2 Disposal Cell					Hauled Out-of County	
Fiscal Year (Oct - Sep)	Projected Total Waste Received[1] (tons)	Estimated Waste Processed[1a] (tons)	Estimated Ash Generation Rate (tons)	(yds^3)[5]	Estimated Ash In Place at Beginning of Fiscal Year (yds^3)	Estimated Waste Placement Rate (yds^3)	Estimated Remaining Disposal Capacity at End of Fiscal Year[3] (yds^3)	Estimated Waste In Place at Beginning of Fiscal Year[9] (yds^3)	Estimated Waste Placement Rate (tons)	Estimated Waste Placement Rate (yds^3)[6]	Estimated Daily Cover Application Rate (yds^3)	Estimated Remaining Disposal Capacity at End of Fiscal Year[4] (yds^3)	Estimated Waste Transferred[8] (tons)	Estimated Waste Transferred (yds^3)[2]
2018[7]	378,733	341,122	85,875	78,068	346,700	78,068	836,000	219,261	15,093	27,442	4,116	403,000	22,518	150,120
2019[7a]	386,807	342,994	83,477	75,889	418,000	75,889	760,380	250,819	10,306	18,738	2,811	381,451	33,507	223,380
2020[7b]	411,616	352,875	81,161	73,783	500,948	73,783	726,567	768,591	9,639	17,525	2,629	809,220	49,102	327,347
2021	450,000	331,000	76,130	69,209	527,702	69,209	657,358	790,780	24,000	43,636	6,545	740,598	95,000	633,333
2022	460,000	331,000	76,130	69,209	596,911	69,209	588,149	840,962	29,000	52,727	7,909	679,962	100,000	666,667
2023	475,000	331,000	76,130	69,209	666,120	69,209	518,940	901,598	44,000	80,000	12,000	587,962	100,000	666,667
2024	485,000	331,000	76,130	69,209	735,329	69,209	449,731	993,598	54,000	98,182	14,727	475,053	100,000	666,667
2025	495,000	331,000	76,130	69,209	804,538	69,209	380,522	1,106,507	64,000	116,364	17,455	341,235	100,000	666,667
2026	505,000	331,000	76,130	69,209	873,747	69,209	311,312	1,240,325	74,000	134,545	20,182	186,507	100,000	666,667
2027	515,000	515,000	118,450	107,682	942,957	107,682	203,631	1,395,053	0	0	0	186,507	0	0
2028	525,000	520,000	119,600	108,727	1,050,638	108,727	94,903	1,395,053	5,000	9,091	1,364	176,053	0	0
2029	530,000	520,000	119,600	108,727	1,159,366	47,874	47,029	1,405,507	10,000	18,182	2,727	155,144	0	0
2030	545,000	520,000	119,600	108,727				1,426,416	25,000	45,455	6,818	102,871	0	0
2031	550,000	520,000	119,600	108,727				1,478,689	30,000	54,545	8,182	40,144	0	0

[1] Source: Integrated Solid Waste Management System Master Plan Update, Figure 3-2, CDM-Smith,2019, except where noted

[1a] Source: Integrated Solid Waste Management System Master Plan Update, Section 3, CDM-Smith,2019

[2] assume density of uncompacted mixed MSW to be 300 lbs/cu. yd

[3] assume total capacity of A4 is 1,254,269 cu. yds.

[4] assume total capacity of SW1/SW2 is 900,000 cu. yds. prior to 2020, 1.6 million after 2020 (following FDEP Modification)

[5] assume density of ash to be 2,200 lbs/cu. yd.

[6] assume density of compacted mixed MSW to be 1,100 lbs/cu. yd.

[7] 2018 Values are actual values as reported by Pasco County, 1/17/19 Memorandum from J. Chamberlain to M. Carballa

[7a] 2019 Values are actual values as reported by Pasco County, 1/24/20 Memorandum from T. Treshler to M. Carballa

[7b] 2020 Values are actual values as reported by Pasco County, 1/29/2021 Memorandum from J. Chamerlain to M. Carballa

[8] Assumes that favorable pricing will result in decision to haul out of county through 2025

[9] pre-2020 values are for SW2 only. 2020 values and beyond are combined SW1 and SW2

Reference 3

JonesEdmunds

PROJECT NUMBER: 16002-009-01
PROJECT NAME: West Pasco County Landfill (Ash Monofill)
SUBJECT: Additional Volume from Modifications

Volumes

Index	Base Surface	Comparison Surface	Cut	Fill	Net
3	EG-DTM-CEN	PG-COVR-101	18478.54 Cu. Yd.	1246256.19 Cu. Yd.	1227777.65 Cu. Yd.<Fill>
4	PG-COVR-101	PG-COVR-INT	2656.83 Cu. Yd.	554284.84 Cu. Yd.	551628.02 Cu. Yd.<Fill>

Airspace Capacity Estimate		
	1246257 CY	- Airspace Gained From Current to Cells A-1/2/3 to EL. 101, Cell A-4 to EL. 122.
+	554285 CY	- Airspace Gained from EL. 101 Cells A-1/2/3 to EL. 122.
	1790000 CY	- Conservative Estimate.

Attachment E.5

Laboratory Certifications

State of Florida
Department of Health, Bureau of Public Health Laboratories
This is to certify that

E44123

PASCO COUNTY UTILITIES ENVIRONMENTAL LABORATORY
8864 GOVERNMENT DRIVE
NEW PORT RICHEY, FL 34654

has complied with Florida Administrative Code 64E-1,
for the examination of environmental samples in the following categories

DRINKING WATER - MICROBIOLOGY, DRINKING WATER - PRIMARY INORGANIC CONTAMINANTS, DRINKING WATER - SECONDARY INORGANIC CONTAMINANTS, NON-POTABLE WATER - GENERAL CHEMISTRY, NON-POTABLE WATER - METALS, NON-POTABLE WATER - MICROBIOLOGY

Continued certification is contingent upon successful on-going compliance with the NELAC Standards and FAC Rule 64E-1 regulations. Specific methods and analytes certified are cited on the Laboratory Scope of Accreditation for this laboratory and are on file at the Bureau of Public Health Laboratories, P. O. Box 210, Jacksonville, Florida 32231. Clients and customers are urged to verify with this agency the laboratory's certification status in Florida for particular methods and analytes.

Date Issued: July 01, 2021 Expiration Date: June 30, 2022

Patty A. Lewandowski, MBA, MT(ASCP)
Chief Bureau of Public Health Laboratories
DH Form 1697, 7/04
NON-TRANSFERABLE E44123-41-07/01/2021
Supersedes all previously issued certificates

Laboratory Scope of Accreditation

Attachment to Certificate #: E44123-41, expiration date June 30, 2022. This listing of accredited analytes should be used only when associated with a valid certificate.

State Laboratory ID: **E44123** EPA Lab Code: **FL00137** **(727) 847-8902**

E44123
Pasco County Utilities Environmental Laboratory
8864 Government Drive
New Port Richey, FL 34654

Matrix: **Drinking Water**

Analyte	Method/Tech	Category	Certification Type	Effective Date
Aluminum	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019
Antimony	SM 3113 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Arsenic	SM 3113 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Barium	EPA 200.7	Primary Inorganic Contaminants	NELAP	10/9/2019
Beryllium	SM 3113 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Cadmium	SM 3113 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Calcium	EPA 200.7	Primary Inorganic Contaminants	NELAP	10/9/2019
Chloride	EPA 300.0	Secondary Inorganic Contaminants	NELAP	12/17/2008
Chromium	EPA 200.7	Primary Inorganic Contaminants	NELAP	10/9/2019
Color	SM 2120 B	Secondary Inorganic Contaminants	NELAP	6/4/2009
Copper	EPA 200.7	Primary Inorganic Contaminants,Secondary Inorganic Contaminants	NELAP	10/9/2019
Escherichia coli	SM 9223 B	Microbiology	NELAP	4/27/2004
Fluoride	EPA 300.0	Primary Inorganic Contaminants	NELAP	3/24/2014
Hardness	SM 2340 B	Secondary Inorganic Contaminants	NELAP	10/9/2019
Heterotrophic plate count	SM 9215 B	Microbiology	NELAP	5/26/2016
Iron	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019
Lead	SM 3113 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Magnesium	EPA 200.7	Primary Inorganic Contaminants	NELAP	10/9/2019
Manganese	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019
Mercury	SM 3112 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Molybdenum	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019
Nickel	EPA 200.7	Primary Inorganic Contaminants	NELAP	10/9/2019
Nitrate	EPA 300.0	Primary Inorganic Contaminants	NELAP	12/17/2008
Nitrate	SM 4500-NO3 F	Primary Inorganic Contaminants	NELAP	4/27/2004
Nitrite	EPA 300.0	Primary Inorganic Contaminants	NELAP	12/17/2008
Nitrite	SM 4500-NO2-B	Primary Inorganic Contaminants	NELAP	12/17/2008
Odor	SM 2150 B	Secondary Inorganic Contaminants	NELAP	6/4/2009
Potassium	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019
Residue-filterable (TDS)	SM 2540 C	Secondary Inorganic Contaminants	NELAP	4/27/2004
Selenium	SM 3113 B	Primary Inorganic Contaminants	NELAP	10/9/2019
Silver	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019
Sodium	EPA 200.7	Primary Inorganic Contaminants	NELAP	6/4/2009
Sulfate	EPA 300.0	Primary Inorganic Contaminants	NELAP	12/17/2008
Total coliforms	SM 9223 B	Microbiology	NELAP	4/27/2004
Total nitrate-nitrite	EPA 300.0	Primary Inorganic Contaminants	NELAP	12/17/2008

Clients and Customers are urged to verify the laboratory's current certification status with the Environmental Laboratory Certification Program.

Laboratory Scope of Accreditation

Attachment to Certificate #: E44123-41, expiration date June 30, 2022. This listing of accredited analytes should be used only when associated with a valid certificate.

State Laboratory ID: **E44123** EPA Lab Code: **FL00137** **(727) 847-8902**

E44123
Pasco County Utilities Environmental Laboratory
8864 Government Drive
New Port Richey, FL 34654

Matrix: **Drinking Water**

Analyte	Method/Tech	Category	Certification Type	Effective Date
Total nitrate-nitrite	SM 4500-NO3 F	Primary Inorganic Contaminants	NELAP	4/27/2004
Zinc	EPA 200.7	Secondary Inorganic Contaminants	NELAP	10/9/2019

Laboratory Scope of Accreditation

Attachment to Certificate #: E44123-41, expiration date June 30, 2022. This listing of accredited analytes should be used only when associated with a valid certificate.

State Laboratory ID: **E44123** EPA Lab Code: **FL00137** **(727) 847-8902**

E44123
Pasco County Utilities Environmental Laboratory
8864 Government Drive
New Port Richey, FL 34654

Matrix: **Non-Potable Water**

Analyte	Method/Tech	Category	Certification Type	Effective Date
Alkalinity as CaCO3	SM 2320 B	General Chemistry	NELAP	4/27/2004
Aluminum	EPA 200.7	Metals	NELAP	10/9/2019
Ammonia as N	SM 4500-NH3 G	General Chemistry	NELAP	4/27/2004
Antimony	EPA 200.7	Metals	NELAP	10/9/2019
Arsenic	EPA 200.7	Metals	NELAP	10/9/2019
Barium	EPA 200.7	Metals	NELAP	10/9/2019
Beryllium	EPA 200.7	Metals	NELAP	10/9/2019
Biochemical oxygen demand	SM 5210 B	General Chemistry	NELAP	7/29/2008
Cadmium	EPA 200.7	Metals	NELAP	10/9/2019
Calcium	EPA 200.7	Metals	NELAP	10/9/2019
Carbonaceous BOD (CBOD)	SM 5210 B	General Chemistry	NELAP	4/27/2004
Chloride	EPA 300.0	General Chemistry	NELAP	12/17/2008
Chromium	EPA 200.7	Metals	NELAP	10/9/2019
Color	SM 2120 B	General Chemistry	NELAP	6/4/2009
Copper	EPA 200.7	Metals	NELAP	10/9/2019
Enterococci	SM 9230 D	Microbiology	NELAP	10/9/2019
Escherichia coli	SM 9223 B /QUANTI-TRAY	Microbiology	NELAP	10/9/2019
Fecal coliforms	COLILERT®-18 (Fecal Coliforms)	Microbiology	NELAP	10/9/2019
Fecal coliforms	SM 9222 D	Microbiology	NELAP	4/27/2004
Iron	EPA 200.7	Metals	NELAP	10/9/2019
Lead	EPA 200.7	Metals	NELAP	10/9/2019
Magnesium	EPA 200.7	Metals	NELAP	10/9/2019
Manganese	EPA 200.7	Metals	NELAP	10/9/2019
Mercury	SM 3112 B	Metals	NELAP	10/9/2019
Molybdenum	EPA 200.7	Metals	NELAP	10/9/2019
Nickel	EPA 200.7	Metals	NELAP	10/9/2019
Nitrate	EPA 300.0	General Chemistry	NELAP	12/17/2008
Nitrate as N	SM 4500-NO3 F	General Chemistry	NELAP	4/27/2004
Nitrate-nitrite	SM 4500-NO3 F	General Chemistry	NELAP	4/27/2004
Nitrite	EPA 300.0	General Chemistry	NELAP	12/17/2008
Nitrite as N	SM 4500-NO2-B	General Chemistry	NELAP	4/27/2004
Oil & Grease	EPA 1664A	General Chemistry	NELAP	3/24/2014
Orthophosphate as P	SM 4500-P E	General Chemistry	NELAP	3/24/2014
Phosphorus, total	SM 4500-P E	General Chemistry	NELAP	3/24/2014
Potassium	EPA 200.7	Metals	NELAP	10/9/2019

Laboratory Scope of Accreditation

Attachment to Certificate #: E44123-41, expiration date June 30, 2022. This listing of accredited analytes should be used only when associated with a valid certificate.

State Laboratory ID: **E44123** EPA Lab Code: **FL00137** **(727) 847-8902**

E44123
Pasco County Utilities Environmental Laboratory
8864 Government Drive
New Port Richey, FL 34654

Matrix: **Non-Potable Water**

Analyte	Method/Tech	Category	Certification Type	Effective Date
Residue-filterable (TDS)	SM 2540 C	General Chemistry	NELAP	4/27/2004
Residue-nonfilterable (TSS)	SM 2540 D	General Chemistry	NELAP	4/27/2004
Residue-total	SM 2540 B	General Chemistry	NELAP	12/17/2008
Selenium	EPA 200.7	Metals	NELAP	10/9/2019
Silver	EPA 200.7	Metals	NELAP	10/9/2019
Sodium	EPA 200.7	Metals	NELAP	10/9/2019
Sulfate	EPA 300.0	General Chemistry	NELAP	12/17/2008
Total coliforms	SM 9222 B	Microbiology	NELAP	3/24/2014
Total coliforms	SM 9223 B /QUANTI-TRAY	Microbiology	NELAP	10/9/2019
Total hardness as CaCO3	SM 2340 B	General Chemistry	NELAP	10/9/2019
Total nitrate-nitrite	EPA 300.0	General Chemistry	NELAP	12/17/2008
Zinc	EPA 200.7	Metals	NELAP	10/9/2019

PART F

PART F – GENERAL CRITERIA FOR LANDFILLS

1. LOCATION

See Part C.1 for information on the 100-year floodplain determination.

2. HORIZONTAL SEPARATION

Appendix A provides the engineering plans of the *West Pasco County Landfill Ash Monofill Operations Modification*, showing at least 100 feet of horizontal separation exists between the nearest property boundary and the ash monofill's edge of liner. Therefore, the distance between the property boundary and the toe of the proposed final cover slope exceeds 100 feet, meeting the minimum separation requirement in Rule 62-701.340(4)(c), Florida Administrative Code (FAC).

PART I

PART I – GEOTECHNICAL INVESTIGATION REQUIREMENTS

BACKGROUND

Pasco County Public Infrastructure owns and operates the West Pasco County Solid Waste and Resource Recovery Facility. The 751-acre property has various facilities for solid waste processing and disposal including a mass burn waste-to-energy (WTE) facility, ash monofill, a Class I Landfill that operates as part of the County's integrated solid waste management system, and a Class III Landfill. WTE ash is disposed of in the ash monofill cell (Cells A-1 through A-4). Waste that is diverted from the WTE facility because it is at capacity is disposed of in the Class I Landfill (Cells SW-1 and SW-2).

Ash filling is currently occurring in the north half of Cell A-4. Cells A-1, A-2, A-3, and part of A-4 have temporary rain covers. The WTE operator, Covanta, is processing ash for metals recovery in portions of Cell A-4. The cells were previously filled in such a manner that unfilled valleys are between the cells.

The final buildout of the ash cells is permitted with various side slopes from 3 horizontal to 1 vertical (3H:1V) to 5H:1V. The permitted final buildout for ash Cells A-1 through A-3 is to elevation 101 feet National Geodetic Vertical Datum (NAVD); Cell A-4 is permitted to elevation 122 feet NAVD.

The County desires more operational flexibility, would like to fill the valleys between the ash cells to delay filling in the area where metals are being processed, and would like to modify the side slopes and peak buildout in an effort to gain capacity. Therefore, the County is seeking a vertical expansion and side slope modification of the ash monofill.

The proposed modifications are as follows:

- Slope modification: modify all side slopes to 3H:1V.
- Vertical expansion: modify the final buildout of Cells A-1 through A-3 to elevation 122 ft NAVD.

PURPOSE

The geotechnical investigation is to gather information regarding the existing subsurface conditions and to characterize the soil engineering properties and groundwater conditions in accordance with the Florida Department of Environmental Protection (FDEP) Rule 62-701.410(3), Florida Administrative Code (FAC). Information obtained from the recent subsurface investigation, historical geotechnical investigations, and experience with similar soils and conditions are the basis for the foundation analysis that evaluates the suitability of the foundation soils for the proposed Class I Ash Monofill vertical expansion and side slope modification.

STATE REGULATORY AUTHORITY

In accordance with Rule 62-701.410(3), FAC, a subsurface investigation and geotechnical foundation analysis was conducted for the proposed Class I side slope and vertical modification. This Report provides descriptions of the subsurface conditions, details of the foundation analyses, and conclusions regarding the suitability of the foundation for construction of the Class I modification.

The outline of this report follows FDEP Form 62-701.900(1), Part I, Geotechnical Investigation Requirements (effective date June 11, 2015).

1 GEOTECHNICAL INVESTIGATION REPORT

In accordance with Rule 62-701.410(3), FAC, Jones Edmunds conducted a geotechnical investigation and foundation analysis for the Class I Ash Monofill modification. The geotechnical investigation included reviewing historical subsurface investigations and drilling new borings to confirm information. This Report summarizes the historical and current subsurface investigations, discusses the new borings drilled on the perimeter of the Ash Monofill, describes the foundation soils, provides details of the foundation analyses, evaluates sinkhole potential, describes faults and seismic zones, and provides conclusions. Attachment I.1 provides figures showing Ash Monofill final cover contours, cross-sections, soil boring locations, and settlement point locations.

The following figures are included in Attachment I.1:

- Figure I-1: Ash Monofill Modification – Boring Plan
  - Includes aerial image and boring locations.
- Figure I-2: Ash Monofill Modification – Buildout
  - Includes conceptual final cover contours and cross-section locations.
- Figure I-3: Ash Monofill Modification – Sections
  - Includes Cross-Sections A-A’ and B-B’ and marked settlement points.
- Figure I-4: Ash Monofill Modification – Sections
  - Includes Cross-Sections C-C’ and D-D’ and marked settlement points.

Appendix A, Engineering Plans, of this permit modification application provides the conceptual final cover design and details used for the foundation analyses.

a. Description of Subsurface Conditions

Jones Edmunds reviewed historical geotechnical reports and subsurface investigation boring logs from the site. The reports and logs reviewed were generally from 2000 and were prepared by Camp Dresser & McKee, Driggers Engineering Services Incorporated, and Qore Property Sciences. We also reviewed publicly available data such as the US Geological Survey (USGS) Soil Survey and the Florida Geological Society Sinkhole

Database. To supplement the historical investigations, an additional subsurface investigation was completed by Professional Service Industries, Inc. (PSI) in 2021.

Discussions of the regional site conditions, published soil surveys data, subsurface investigations, subsurface profiles, water table conditions, and material parameters used in the foundation analyses are presented below:

(1) Regional Site Conditions

Regional site conditions are summarized below:

(a) Topography

The West Pasco County Landfill (WPCL) site is relatively flat with a ground surface elevation of approximately 48 ft NAVD in the Ash Monofill area. The WPCL site is within the Gulf Coastal Lowlands physiographic province along the west coast of Florida. Surface drainage is to the south.

(b) Regional and Site Geology

The stratigraphy of Pasco County is characterized by undifferentiated surficial sediments made up of sand and clayey sand near land surface, overlying clays generally assigned to the Hawthorn formation. Underlying the clay is weathered limestone and competent limestone. Although not encountered in all the borings, competent limestone is estimated to be greater than 50 feet below grade.

(c) Hydrostratigraphy

Two distinct water-bearing units or aquifers occur within the WPCL site. The surficial (upper-most) aquifer is comprised of the permeable portions of the surficial sediments. This aquifer defines the surficial water table. The seasonal high-water table (SHWT) is approximately at elevation 36 feet NAVD in this aquifer in the footprint of the Ash Monofill. The competent limestone underlying the surficial deposits of sand, clayey sand, and clay is considered the upper Floridan aquifer system. The Floridan aquifer is semi-confined by the overlying clays associated with the Hawthorn Formation. Groundwater flow in the Floridan aquifer system is generally west toward the Gulf of Mexico.

(2) Soil Survey

The *Soil Survey of Pasco County, Florida*, published by the US Department of Agriculture, Natural Resources Conservation Service, gives information for soils up to 80 inches deep. In general, the Ash Monofill area has two major soil types up to 80 inches: Candler fine sand, excessively drained, and Tavares sand, moderately well drained. A soils map and supporting descriptions are provided in Attachment I.1.a(2).

(3) Subsurface Investigations

(a) Historical Investigations

Jones Edmunds reviewed historical geotechnical reports and subsurface investigation boring logs from the site. The reports and logs reviewed were generally from 1994 and 2005 and were prepared by PSI and Law Engineering. We also reviewed publicly available data such as the USGS Soil Survey and the Florida Geological Society Sinkhole Database.

Our review of historical subsurface information found the following:

- Cell A-2: Five shallow piezometers (drilled 12 to 30 feet deep), three augers (drilled 6 to 21 feet deep), and five standard penetration test (SPT) borings (drilled 14 to 26 feet deep).
- Cell A-3: Three piezometers (drilled to 21.5 feet).
- Cell A-4: Six STP borings (drilled to 50 feet deep in 2005).

Boring locations are shown in Figure I.1. The historical boring logs that we reviewed are provided in Attachment I.1.a(3).

(b) Current Subsurface Investigation

To supplement the historical data, seven SPT borings, A-1 through A-7, were drilled in March 2021 by PSI. The borings were drilled to a depth of 30 to 60 feet deep along the perimeter of the Ash Monofill. Based on the historical and most recent borings, the general subsurface profile consists of varying thicknesses (2 to 60 feet) of loose-to-medium-dense sand, overlying varying thickness (5 to 10 feet) of primarily medium-to-stiff clayey sand, sandy/silty clay, and clay over weathered limestone. The top of the limestone varied from approximately 20 to 45 feet below grade. Based on the deep borings in Cell A-4 from 2005 and the 2021 borings along the perimeter of the Ash Monofill, the southwest corner of Cell A-4 has limestone at a depth of more than 50 feet. Several of the deep boring logs noted very loose and/or very soft zones as reported by the designation "weight of hammer" or "weight of rod" at 45 to 50 feet (Cell A-4: B-6 [2005] and A-7 [2021]). The logs also reported instances of loss of circulation, which occurs when the drilling fluid pressure exceeds the formation pressure. This can occur through permeable materials such as sand or gravel. The logs reviewed did not report any voids. The *Report of Soil Borings* dated March 26, 2021, by PSI is provided in Attachment I.1.a(3).

(4) Subsurface Profile

The soil stratification boundary lines shown on the soil boring logs are approximate and represent the best estimate of the soil transitions.

Because soils vary from location to location with depth, subsurface conditions at other locations may differ from those shown in the borings. As generally accepted practice, the subsurface conditions are interpolated between borings to create a general subsurface profile.

The subsurface soil profile observed in the 2021 subsurface investigation is largely consistent with the historical borings drilled at the site. The general subsurface profile consists of approximately 10 to 25 feet of loose to medium-dense sand overlying varying thicknesses of sandy clay and clay over weathered limestone. No voids were reported in the boring logs. Several of the deep boring logs noted very loose and/or very soft zones as reported by the designation "weight of hammer" or "weight of rod" as noted above. The logs also reported instances of loss of circulation.

The general subsurface profile used in the foundation analyses consists of the following:

- Soil 1 – Compacted Subgrade (as part of construction): Sand, variably clayey, medium dense.
- Soil 2 – Sands: Slightly clayey or silty, loose to medium dense.
- Soil 3 – Clayey Sand/Sandy Clay: Variably silty, soft to firm.
- Soil 4 – Clay: Variably sandy, with limestone fragments, firm to stiff.
- Soil 5 – Weathered Limestone: Variably clayey/sandy, medium dense.

Soil classifications were performed during drilling based on visual identification. Engineering properties of the foundation soils were based on information reported on the boring logs, review of historical laboratory test results, and literature data.

(5) Groundwater Table Conditions

Based on the March 2021 subsurface investigation, groundwater was not apparent in the SPT borings; however, groundwater levels from adjacent wells were reviewed to estimate the groundwater table conditions. Refer to the discussion in Section I.1.c. on the seasonal high-groundwater table (SHGWT).

(6) Material Classification and Strength Parameters

Jones Edmunds estimated the values for the various soil, waste, and material parameters required for the foundation analyses based on laboratory testing, empirical correlations, and experience with soils similar to those encountered. Attachment I.1.a(6) summarizes the subsurface engineering parameters and provides references for the material parameters.

b. Unsuitable Foundation Conditions

Unsuitable foundation conditions addressed here include the presence of muck, previously filled areas, soft ground, lineaments, and sinkholes.

(1) Presence of Muck

No muck was reported in the historical boring logs or in the 2021 borings drilled as part of the proposed expansion project.

(2) Previously Filled Areas

The proposed slope modification and vertical expansion is within the limits of the permitted and constructed footprint of the Ash Monofill cells. Based on our review of historical data, the existing cells were not constructed over waste or previously filled areas.

(3) Soft Ground

The relative density of soils is correlated to the SPT blow count (N-value). Very loose granular soils are defined as having N-values less than 4 and very soft cohesive soils (silts and clays) as having N-values less than 2.

The historical and current subsurface investigations did encounter varying thickness of soils that are characterized as loose granular material (N-values between 4 and 10) or soft cohesive material (N-values between 2 and 4). Of the borings in the footprint of the Ash Monofill, a few very loose and/or very soft zones were reported by the designation "weight of hammer" or "weight of rod." The logs also reported instances of loss of circulation, which occurs when the drilling fluid pressure exceeds the formation pressure. This can occur through permeable materials such as sand or gravel. The logs reviewed did not report any voids.

The proposed slope modification and vertical expansion will be constructed primarily over existing waste that has loaded the foundation soils over time and settled. The soft or loose soils were taken into consideration in the settlement analysis.

(4) Lineaments

Lineament studies are subjective observations of topographic features (such as straightened stream channels, elongated ponds, or alignments of low areas) that may indicate subsurface rock fracturing or dissolution, and thus the potential for sinkhole activity. Lineament features are not conclusive until subsequent exploration has been performed to confirm

the presence of karst activity. Previous reports have discussed sinkhole potential at the site[1,2].

c. Seasonal High-Groundwater Table

The SHGWT is used in the foundation analysis for determining the level at which the soils are saturated. Jones Edmunds reviewed historical reports and the groundwater level data from 2015 through 2017[3] from the existing groundwater monitoring wells near the Ash Monofill. The water levels from the wells closest to the ash monofill ranged from elevation 26.6 (4MW-13D) to 38.5 feet (2MW-2) . The average water level is elevation 33.1 feet NAVD. For foundation analysis purposes, we assumed an SHGWT at elevation 36 feet NAVD.

The existing Ash Monofill cells were designed to be above the SHGWT.

d. Evaluation of Fault Areas, Seismic Impact Zones, and Unstable Areas

In accordance with Subsection 62-701.410(3)(c), FAC, and 40 CFR 258.13, lateral expansions of Ash Monofills/Class I Landfills shall not be located within 200 feet of a fault that has had displacement in Holocene time. The WPCL is not within 200 feet of a fault line based on our review of the USGS and US Quaternary Faults and Folds Database.

In accordance with 62-701.410(3)(c), FAC, and 40 CFR 258.14, lateral expansions of Ash Monofills/Class I Landfills shall not be located in seismic impact zones, which is defined as an area with 10 percent or greater probability that the maximum horizontal ground acceleration will exceed 0.10g (gravity) in 250 years. The site has an estimated ground acceleration of less than 0.10g with a 10-percent chance of exceedance in 250 years as shown in the USGS Map of Seismic Zones provided in Attachment I.1.d. The WPCL is in a seismically stable area.

e. Foundation Analysis

In accordance with Subsection 62-701.410(3)(e), FAC, the foundation analyses shall include foundation bearing capacity, subgrade settlement (total and differential), and slope stability (global and veneer). These analyses are conservatively based on subsurface soil engineering properties (as outlined in Attachment I.1.a(6)) estimated from field and laboratory investigations and literature data. Subsurface conditions in the area of the proposed landfill expansion are generalized into profiles for foundation analysis.

---

[1] May 16, 1987, Jammal & Associates Inc., *Summary Report, Geotechnical/Hydrogeological Investigation Proposal Class III Sanitary Landfill*.

[2] February 25, 2009, CD, Response to FDEP RAI Letter dated February 12, 2009.

[3] May 2018, *Water Quality Monitoring Plan Evaluation Report, Semester II 2015 through Semester II 2017*, prepared by Pasco County.

The foundation analyses for the Ash Monofill modification were based on a build-out elevation to 122 feet NAVD, using 3H:1V final cover side slopes.

(1) Foundation Bearing Capacity Analysis

The bearing capacity analysis is provided in Attachment I.1.e(1).

The resulting analysis shows that the proposed modified Ash Monofill has a factor of safety against bearing capacity failure greater than 1.5, showing the ability of the foundation soil to safely carry a structure without undergoing a shear failure. This meets the requirements of Rule 62-701.400(2), FAC, "designs to prevent failures of side slopes, and designs to prevent deep-seated failures through the waste, along liner systems, and through foundation soils, shall achieve a minimum factor of safety of 1.5 using peak strength values."

(2) Subgrade Settlement Analysis

The subgrade settlement analysis is provided in Attachment I.1.e(2).

Foundation settlement is the vertical deflection of the foundation soil as a result of the landfill overburden load. We calculated the settlement of the foundation soil to evaluate the adequacy of the foundation to support the proposed loads while maintaining appropriate leachate collection system drainage after settlement. Rule 62-701.400(4)(c), FAC, requires the following:

> *"The minimum slope for the leachate collection system, in areas which drain to lateral collection pipes and header pipes, shall be 1.0 percent after predicted settlement. The minimum slopes for the collection pipes of the leachate collection system, i.e., lateral and header pipes, shall be 0.3 percent after predicted settlement. Minimum slopes shall be measured from the peak grade to the lowest grade along the design flow path."*

The Ash Monofill was designed, permitted, and constructed before the above Rule requirement. Attachment I.1.e(2) provides the total and differential settlement calculations at critical points along the bottom liner. The calculations show that the differential settlements will not adversely affect the slope of the bottom liner design. The slopes after settlement are positive and allow leachate to flow to the perimeter leachate collection sumps.

(3) Slope Stability Analysis

The slope stability analyses evaluate the stability of the modified Ash Monofill at the complete build-out elevation, when the load on the foundation is greatest. The two landfill slope stability issues are (1) through the final cover system (veneer failure) and (2) potential failure through the waste mass and along the bottom liner (global block failure). This section presents the veneer and the global slope stability analyses.

Attachment I.1.e(3)(a), Veneer Slope Stability Calculations, and Attachment I.1.e(3)(b), Global Slope Stability Analysis, include the slope stability analyses.

These two types of failures may occur with very different consequences. The failure of the final cover soils caused by erosion, called *veneer failure*, has localized impacts and can be repaired at a reasonable cost with minimal or no impact on the environment. Foundation failures beneath the waste and failure within the waste mass, called *global slope stability*, have more severe consequences. The minimum factor of safety for veneer and global slope stability is 1.5 in accordance with Rule 62-701.400(2), FAC.

(a) Veneer Slope Stability

The veneer slope stability calculations and references are provided in Attachment I.1.e(3)(a).

A veneer slope stability analysis was performed for the proposed final cover system. The final cover system will consist of 2 feet of soil over a geocomposite and textured geomembrane on a 3H:1V slope. This analysis determines a minimum interface friction angle (δ) between the cover soil and geomembrane needed to achieve a factor of safety of 1.5 (as required by Rule 62-701.400(2), FAC).

(b) Global Slope Stability

The global slope stability calculations and references are provided in Attachment I.1.e(3)(b).

The purpose of the global slope stability analysis is to evaluate the potential for failure through the waste mass and along the bottom liner system. This requires determining the minimum required bottom liner friction angle based on the minimum required factor of safety for waste stability of 1.5 (using peak strength values) in accordance with Rule 62-701.400(2), FAC.

At the time the cells were constructed, interface friction/direct shear testing was not performed on the bottom liner systems. This was not the practice at the time. Current design and construction practices, however, include direct shear testing on the actual liner materials before they are installed to verify that they meet the slope stability design parameters of the system. Based on historical drawings, Cells A-1 and A-2 were constructed with smooth high-density polyethylene (HDPE) geomembrane bottom liners, which was common practice at the time; and Cells A-3 and A-4 were constructed with textured HDPE geomembrane bottom liners.

As part of the Class I Landfill expansion in 2019, Jones Edmunds obtained geosynthetic and soil samples from the anchor trenches of SW-1 and SW-2 – outside the limits of waste. The interface shear testing results from the Class I Landfill can be expected to be similar to those materials at the Ash Monofill. Samples were gathered on March 15, 2019, and subsequently tested by TRI Environmental. Direct shear testing was performed on the following interfaces:

1. Smooth geomembrane vs. soil (SW-1).
2. Geosynthetic clay liner (GCL) vs. textured geomembrane (SW-2).
3. GCL vs. soil (SW-2).

The results of the testing are provided in Attachment I.1.e(3)(b) and were used to determine the acceptable modified slope and vertical expansion heights.

The TRI test results showed that the minimum bottom liner interface friction angle at peak strength is 19.3 degrees between the underlying soil and the smooth geomembrane liner. Our analyses determined the minimum bottom liner interface friction angle for the proposed expansion to be 10. Based on the direct shear testing data, the existing bottom liner systems exceed this minimum requirement; thus resulting in a global slope stability factor of safety greater than 1.5.

f. Sinkholes

Based on *The Sinkhole Type, Development, and Distribution Map of Florida*, Map Series No. 110 (Attachment I.1.f), prepared by the USGS and dated 1985, the Ash Monofill is in Area III. The area is defined as having a cover of 30 to 200 feet thick consisting of cohesive clayey sediments of low permeability; sinkholes are numerous, of varying size, and develop abruptly. Cover-collapse sinkholes dominate.

Figure I.1.f, Sinkhole Map, in Attachment I.1.f includes reported subsidence features from the Florida Geological Survey (FGS)/FDEP database. The database includes many reported subsidence features within 5 miles of the facility. Karst features are typical in west-central Florida. No indication of sinkhole activity was discovered during the subsurface investigations in the area of the Ash Monofill. Previous reports have discussed sinkhole potential at the site[4,5].

g. Summary of Methods and Conclusions

The historical and recent subsurface investigations completed across the WPCL included progressing SPT borings and performing laboratory testing on representative samples to further identify the foundation soils. Based on the available data, generalized subsurface cross-sections were developed and used for the foundation analyses. Results of the analyses show that the Ash Monofill foundation soils are adequate to support the proposed side slope modification and vertical expansion. The proposed side slopes of 3H:1V meet the minimum required factor of safety against slope stability failure. The proposed vertical expansion does not adversely affect the existing leachate collection system. Detailed calculation spreadsheets and printouts are provided in attachments as noted in this text.

[4] May 16, 1987, Jammal & Associates Inc., *Summary Report, Geotechnical/Hydrogeological Investigation Proposal Class III Sanitary Landfill*.

[5] February 25, 2009, CDM, Response to FDEP RAI Letter dated February 12, 2009

REPORT SIGNED, SEALED, AND DATED BY A PROFESSIONAL ENGINEER

This document is signed and sealed as part of the permit application package:

ENGINEERING CERTIFICATION

This Geotechnical Report was prepared specifically for the permit application prepared for the proposed Class I Landfill (Ash Monofill) side slope and vertical expansion in Pasco County, Florida. The geotechnical subsurface investigation was based on investigations completed by others; Jones Edmunds' interpretations and design recommendations are based on these reports. This Report has been prepared in accordance with accepted professional engineering practices.

The recommendations submitted in this Report are based on the available subsurface information contained within this Report. If there are revisions to the plans and further recommendations for this project or if deviations from the subsurface conditions noted in this Report are encountered, Jones Edmunds should be notified immediately to determine if changes in the project recommendation are required.

Signature Date

7/12/2021

CAROL G SAWYER LICENSE No. 55214 STATE OF FLORIDA PROFESSIONAL ENGINEER

Carol G. Sawyer, PE
Florida PE Registration No. 55214
Jones Edmunds & Associates, Inc.
Florida Certificate of Authorization No. 1841

Attachment I.1

Figures

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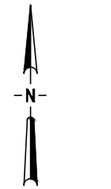

GRAPHIC SCALE

0 150 300 600

SCALE IN FEET

LEGEND

	EXISTING ASH MONOFILL LIMITS
	LIMITS OF ASH MONOFILL CELL (APPROX)
	PROPERTY BOUNDARY
	LIMITS OF LANDFILL CELL (APPROX)
B-3	PSI BORINGS–ASH MONOFILL (2005)
LB3	LAW ENGINEERING BORINGS (1994)
C-8	QORE BORINGS (2000)
PT3	DRIGGERS ENGINEERING SERVICES BORINGS (2000)
B-1	PSI BORING–SOLID WASTE (2019)
A-1	PSI BORING (2021)
4MW-11D	GROUNDWATER MONITORING WELL
GMP-2 GMW-5	GAS MONITORING PROBE

FIGURE I-1 - BORING PLAN
ASH MONOFILL MODIFICATION
WEST PASCO COUNTY LANDFILL AND RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

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LEGEND

- EXISTING ASH MONOFILL LIMITS
- LIMITS OF ASH MONOFILL CELL (APPROX)
- 240 PROPOSED VERTICAL EXPANSION CONTOURS
- 120 EXISTING BOTTOM CONTOURS
- B–3 PSI BORINGS–ASH MONOFILL (2005)
- B–1 PSI BORING–SOLID WASTE (2019)
- LB3 LAW ENGINEERING BORINGS (1994)
- PT6 DRIGGERS ENGINEERING SERVICES BORINGS (2000)
- A–1 PSI BORING (2021)
- 2MW18D GROUNDWATER MONITORING WELL
- 2% BOTTOM LINER SLOPE
- +122.0 FINAL COVER SPOT ELEVATION

N

GRAPHIC SCALE
0 40 80 160
SCALE IN FEET

ASH CELL A-1
ASH CELL A-2
ASH CELL A-3
ASH CELL A-4

C1 C2 C3 C4 C5 C6 C7 C8
D1 D2
A1 A2 A3 A4

3% MIN 4% MAX (TYP)
0.5% (TYP)
2% (TYP)
0.4 (TYP)
0.5 (TYP)
3:1 (TYP)

A-1 A-2 A-3 A-4 A-5 A-6 A-7 185'
A-4 166'

A A' B B' C C' D D'

2MW-25D 2MW-25S 2MW-24S 2MW-24D 2MW-26S 2MW-26D 2MW-27S 2MW-27D 2MW-20D
2MW-13D 4MW-13D 4MW-12D 4MW-11D 4MW-14D 4MW-15D 4MW-16D
B-1 B-2 B-3 B-4 B-5 B-6
LB1 LB3 LB5 LB6 LB7 LB8 LB9 LB10 LB12
PT-4 PT-5 PT-6

FIGURE I-2
ASH MONOFILL MODIFICATION BUILDOUT
WEST PASCO COUNTY LANDFILL AND RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

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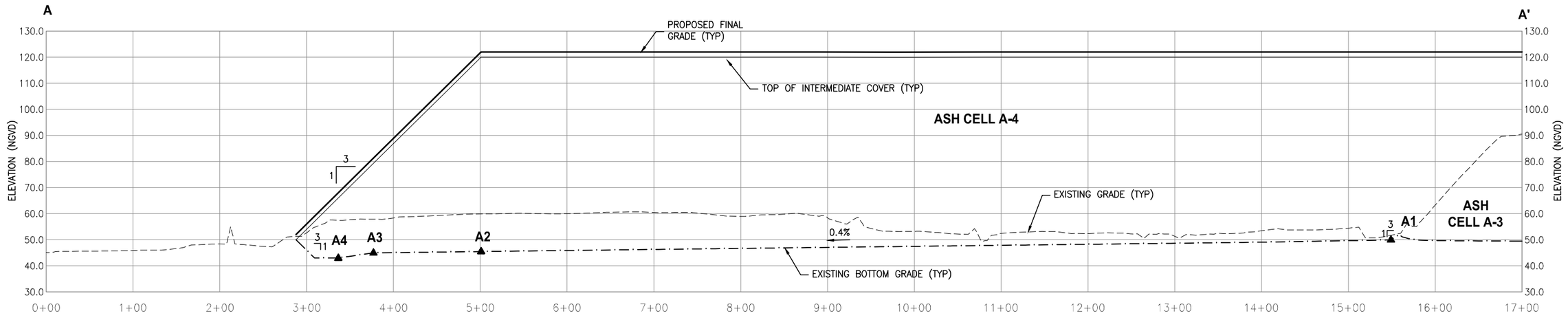
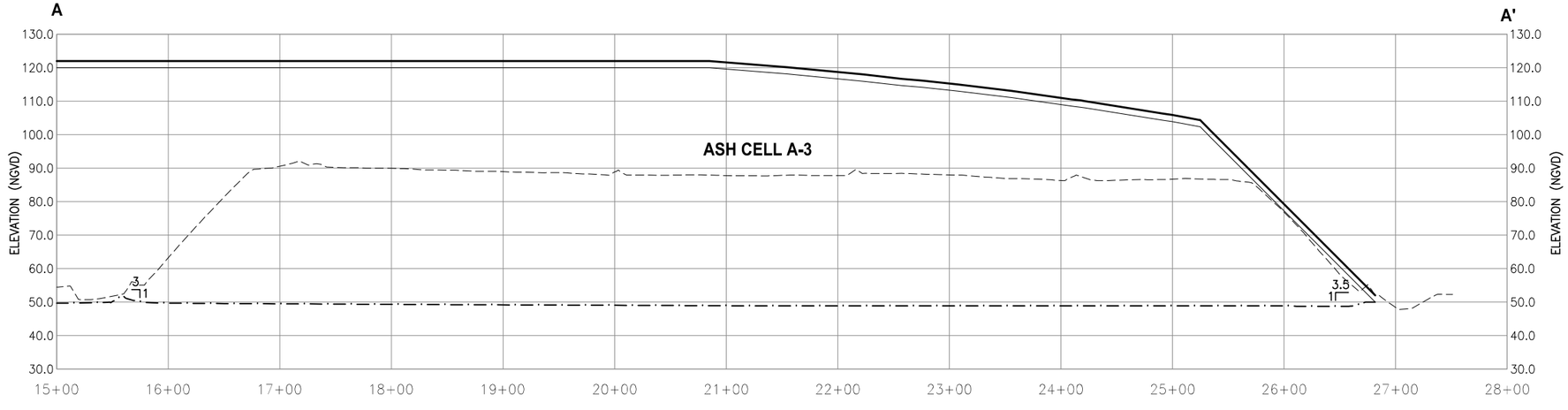


NOTE:
1. EXISTING WASTE GRADE ELEVATION IS BASED ON TOPOGRAPHIC SURVEY DATED 12-8-2020.



LEGEND

- A1 ▲ SETTLEMENT POINT
- EXISTING GRADE
- PROPOSED FINAL COVER
- BOTTOM LINER

GRAPHIC SCALE
0 30 60 120
SCALE IN FEET
HORIZONTAL

GRAPHIC SCALE
0 10 20 40
SCALE IN FEET
VERTICAL

FIGURE I-3 - SECTIONS
ASH MONOFILL MODIFICATION
WEST PASCO COUNTY LANDFILL AND RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

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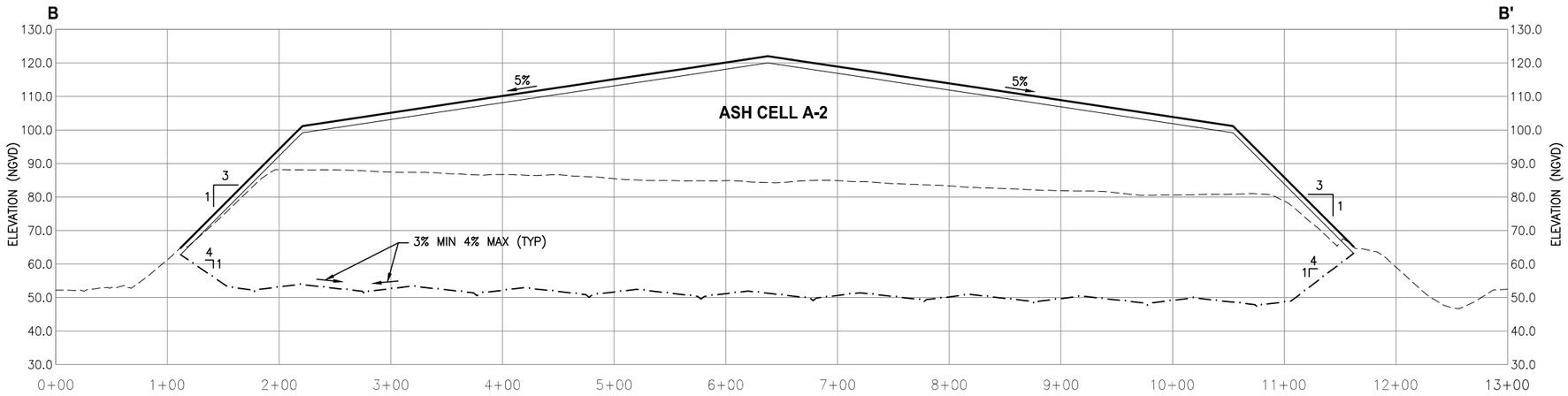

LEGEND

- EXISTING GRADE
- PROPOSED FINAL COVER
- BOTTOM LINER

GRAPHIC SCALE
0 30 60 120
SCALE IN FEET
HORIZONTAL

GRAPHIC SCALE
0 10 20 40
SCALE IN FEET
VERTICAL

FIGURE I-4 - SECTIONS
ASH MONOFILL MODIFICATION
WEST PASCO COUNTY LANDFILL AND
RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

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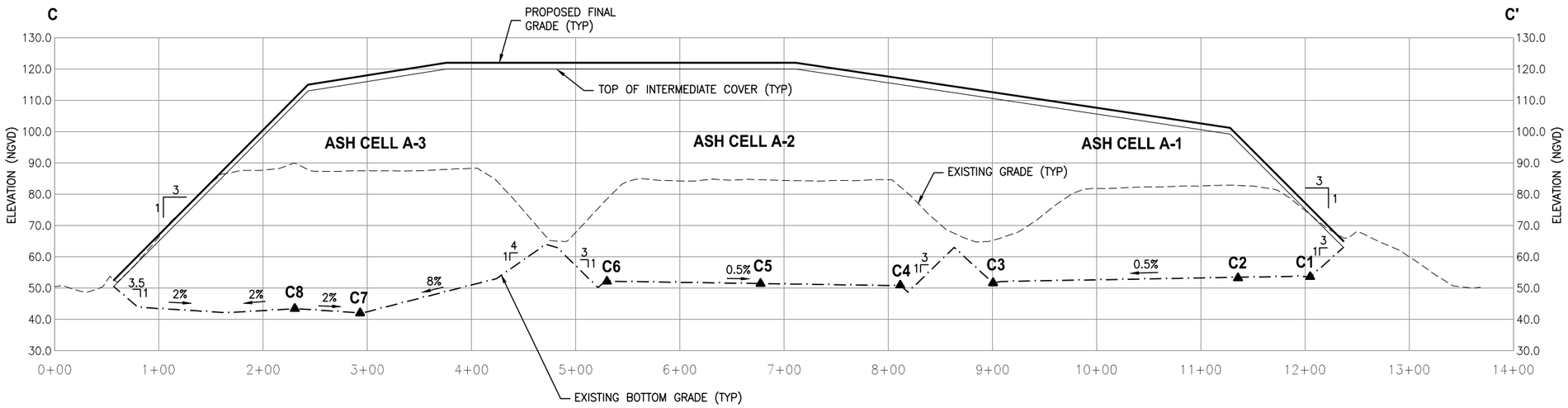

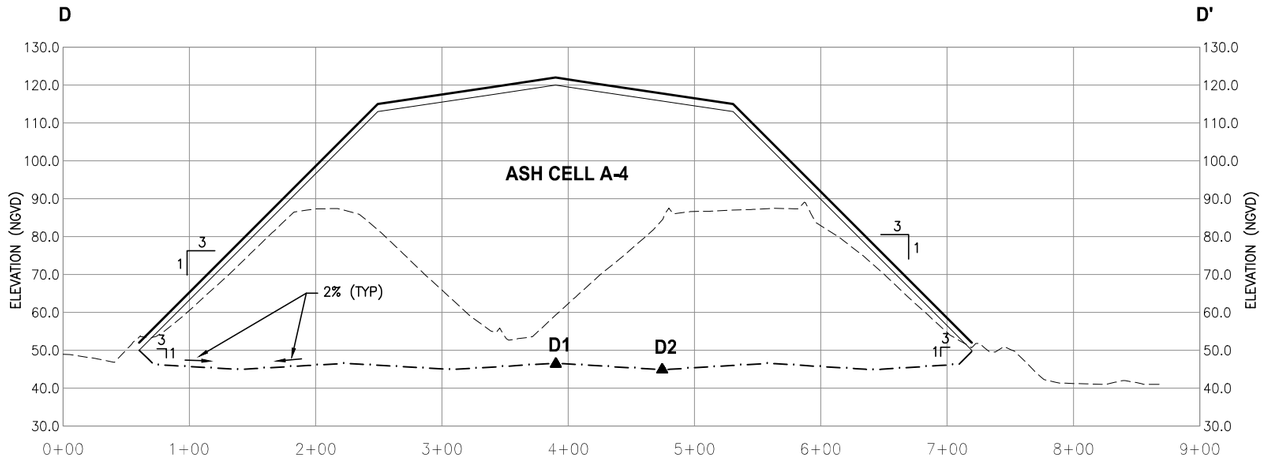

LEGEND

- D1 ▲ SETTLEMENT POINT
- EXISTING GRADE
- PROPOSED FINAL COVER
- BOTTOM LINER

GRAPHIC SCALE
0 30 60 120
SCALE IN FEET
HORIZONTAL

GRAPHIC SCALE
0 10 20 40
SCALE IN FEET
VERTICAL

FIGURE I-5 - SECTIONS
ASH MOFILL MODIFICATION
WEST PASCO COUNTY LANDFILL AND RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

JonesEdmunds

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Attachment I.1.a(2)

Soil Survey

Map Scale: 1:12,400 if printed on A landscape (11" x 8.5") sheet.

N

Meters
0 150 300 600 900

Feet
0 500 1000 2000 3000

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 17N WGS84

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features

- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Pasco County, Florida
Survey Area Data: Version 19, Jun 9, 2020

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jan 7, 2020—Jan 22, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
6	Tavares sand, 0 to 5 percent slopes	170.6	31.4%
7	Sparr fine sand, 0 to 5 percent slopes	17.8	3.3%
11	Adamsville fine sand, 0 to 2 percent slopes	2.8	0.5%
13	Candler fine sand, 0 to 5 percent slopes	351.9	64.8%
Totals for Area of Interest		**543.1**	**100.0%**

Pasco County, Florida

6—Tavares sand, 0 to 5 percent slopes

Map Unit Setting

National map unit symbol: 2v173
Elevation: 0 to 180 feet
Mean annual precipitation: 44 to 56 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 300 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Tavares and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Tavares

Setting

Landform: Flats on marine terraces, ridges on marine terraces, knolls on marine terraces
Landform position (two-dimensional): Shoulder, backslope
Landform position (three-dimensional): Interfluve, base slope
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Eolian or sandy marine deposits

Typical profile

A - 0 to 7 inches: sand
C - 7 to 80 inches: sand

Properties and qualities

Slope: 0 to 5 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Moderately well drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): High to very high (6.00 to 50.02 in/hr)
Depth to water table: About 42 to 72 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 1.9 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3s
Hydrologic Soil Group: A

Forage suitability group: Sandy soils on rises, knolls, and ridges of mesic uplands (G154XB121FL)
Other vegetative classification: Longleaf Pine-Turkey Oak Hills (R154XY002FL)
Hydric soil rating: No

Minor Components

Apopka

Percent of map unit: 6 percent
Landform: Knolls on marine terraces, ridges on marine terraces
Landform position (two-dimensional): Shoulder, summit, footslope
Landform position (three-dimensional): Nose slope, side slope, crest
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Longleaf Pine-Turkey Oak Hills (R154XY002FL)
Hydric soil rating: No

Candler

Percent of map unit: 4 percent
Landform: Ridges on marine terraces, knolls on marine terraces
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Interfluve, side slope, tread
Down-slope shape: Convex
Across-slope shape: Convex
Other vegetative classification: Longleaf Pine-Turkey Oak Hills (R155XY002FL), Longleaf Pine-Turkey Oak Hills (R154XY002FL)
Hydric soil rating: No

Adamsville

Percent of map unit: 3 percent
Landform: Knolls on flatwoods, rises on flatwoods
Landform position (two-dimensional): Summit
Landform position (three-dimensional): Interfluve, talf, rise
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Upland Hardwood Hammock (R154XY008FL), Upland Hardwood Hammock (R155XY008FL)
Hydric soil rating: No

Zolfo

Percent of map unit: 2 percent
Landform: Flats on marine terraces
Landform position (two-dimensional): Footslope
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear

Hydric soil rating: No

Data Source Information

Soil Survey Area: Pasco County, Florida
Survey Area Data: Version 17, Sep 13, 2018

Pasco County, Florida

13—Candler fine sand, 0 to 5 percent slopes

Map Unit Setting

National map unit symbol: 2shkf
Elevation: 10 to 260 feet
Mean annual precipitation: 47 to 56 inches
Mean annual air temperature: 68 to 77 degrees F
Frost-free period: 280 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Candler and similar soils: 90 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Candler

Setting

Landform: Knolls on marine terraces, ridges on marine terraces
Landform position (two-dimensional): Summit
Landform position (three-dimensional): Interfluve, tread
Down-slope shape: Linear, convex
Across-slope shape: Convex, linear, concave
Parent material: Eolian deposits and/or sandy and loamy marine deposits

Typical profile

Ap - 0 to 5 inches: fine sand
E - 5 to 74 inches: fine sand
E and Bt - 74 to 80 inches: fine sand

Properties and qualities

Slope: 0 to 5 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Excessively drained
Runoff class: Very low
Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 19.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very low (about 2.5 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4s

Hydrologic Soil Group: A
Forage suitability group: Sandy soils on ridges and dunes of xeric uplands (G154XB111FL)
Other vegetative classification: Longleaf Pine-Turkey Oak Hills (R154XY002FL)
Hydric soil rating: No

Minor Components

Tavares

Percent of map unit: 4 percent
Landform: Ridges on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex, concave
Across-slope shape: Linear, concave
Other vegetative classification: Longleaf Pine-Turkey Oak Hills (R154XY002FL)
Hydric soil rating: No

Adamsville

Percent of map unit: 3 percent
Landform: Rises on marine terraces, flats on marine terraces
Landform position (three-dimensional): Interfluve, talf
Down-slope shape: Convex, concave
Across-slope shape: Linear, concave
Other vegetative classification: South Florida Flatwoods (R154XY003FL)
Hydric soil rating: No

Millhopper

Percent of map unit: 3 percent
Landform: Rises on marine terraces, flats on marine terraces
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear, convex
Other vegetative classification: Longleaf Pine-Turkey Oak Hills (R154XY002FL)
Hydric soil rating: No

Data Source Information

Soil Survey Area: Pasco County, Florida
Survey Area Data: Version 17, Sep 13, 2018

Attachment I.1.a(3)

2021 PSI Report

2005 CDM/PSI Boring Logs

1994 Law Engineering Report

GEOTECHNICAL ENGINEERING SERVICES REPORT

For

Pasco County Ash Monofill Facility
West Pasco Recycling Center and Landfill
Hays Road, Pasco County, Florida

Prepared for

Pasco County Utilities – Solid Waste
14230 Hays Road
Spring Hill, Florida 34610

Prepared by

PROFESSIONAL SERVICE INDUSTRIES, INC.
5801 BENJAMIN CENTER DRIVE
SUITE 112
TAMPA, FLORIDA 33634
TELEPHONE (813) 886-1075
FAX (813) 249-4916

PSI PROJECT NO. 0390-1933

March 26, 2021

5801 Benjamin Center Drive, Suite 112
Tampa, Florida 33634
phone: 813.886.1075
fax: 813.249.4916
intertek.com/building
psiusa.com
Engineering Certificate of Authorization 3684

March 26, 2021

Pasco County Utilities – Solid Waste
14230 Hays Road
Spring Hill, Florida 34610

ATT: Mr. Justin Roessler, PhD, PE
Assistant Solid Waste Director

Re: Report of Soil Borings
Pasco County Ash Monofill Facility
West Pasco Recycling Center and Landfill
Hays Road, Pasco County, Florida
PSI Project Number 0390-1933

Dear Mr. Roessler :

Professional Service Industries, Inc. (PSI), an Intertek Company is pleased to submit our subsurface exploration report for the above-referenced project. This report presents the results and evaluation of the encountered subsurface conditions.

Should there be any questions, please do not hesitate to contact our office at (813) 886-1075. PSI would be pleased to continue providing geotechnical services or construction materials testing throughout the implementation of the project. We look forward to working with you and your organization on this and future projects.

Respectfully submitted,

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

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BORING LOCATION PLAN AND SOIL PROFILES ..... SHEET 1 & SHEET 2

1.0 PROJECT INFORMATION

We have been contacted by Jones Edmunds to perform soil borings to support additional development at the West Pasco Recycling Center and Landfill located off of Hays Road in Pasco County, Florida. Information regarding the proposed development(s) planned for this location have not been provided, however; a boring plan requesting seven (7) soil borings and boring locations was presented in the RFP.

If any of the noted information is incorrect or has changed, please notify PSI at your earliest convenience so that we may amend the recommendations presented in this report before construction starts.

2.0 PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to obtain information on the subsurface soil and groundwater conditions at requested locations. In order to achieve this objective, seven Standard Penetration Test (SPT) boring A-1 to A-7 were performed to an approximate depth of 60 feet.

The borings were located in the field by estimating distances from existing features identified from the schematic site plan provided for this project, therefore; the Boring Location Plan presented on **Sheet 1** should be considered approximate.

The SPT borings were advanced using mud rotary drilling methods. In the SPT boring, samples were collected and SPT resistances were measured virtually continuously from 4 to 10 feet and on intervals of 5 feet thereafter to boring termination depths. The top 4 feet of the SPT borings were manually augured to check for underground utilities or other obstructions. Drilling and sampling techniques were accomplished in general accordance with ASTM standards. As called for by local regulations, the SPT borings were grouted upon completion.

Representative portions of each sample were returned to our Tampa, Florida office where they were visually classified by a geotechnical engineer. Classifications were performed in general accordance with the Unified Soil Classification System (USCS). Soil profiles were prepared for the boreholes indicating lithological materials encountered and any additional pertinent information.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 SITE LOCATION AND DESCRIPTION

The proposed project is located at the existing West Pasco Recycling Sewer and Class Landfill. At the time of our subsurface exploration, the boring locations were observed to be relatively flat and in grass areas. Expect A-6 was located inside a trench.

3.2 SUBSURFACE CONDITIONS

In A-1 to A-6, the borings revealed very loose to medium dense relatively clean to slightly silty sands from the existing ground surface to approximate depths ranging from 6 to 22 feet below the surface. Below the upper relatively clean sands intermittent layers of mixed clayey sands (SC), high plastic fat clays (CH), and

a final layer of highly weathered limestone and calcareous clay material were then observed until boring termination.

In A-7, the boring encountered loose to medium dense relatively clean to slightly silty sands from the existing ground surface to approximate depths 60 feet below the existing ground surface. A layer of very soft high plastic fat clays (CH) was encountered between the approximate depth 48 and 52 feet below the ground surface. Overall, SPT N-Values ranged from 0 to 50 blows per foot (bpf) in the borings.

The soil profiles presented on **Sheet 2** include soil descriptions and stratifications. Variations may occur and should be expected between the widely spaced boring locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual.

3.3 GROUNDWATER INFORMATION

Groundwater was not apparent in the soil borings. It should be noted that groundwater levels tend to fluctuate during periods of prolonged drought and extended rainfall and may be affected by man-made influences. We would expect groundwater to perch above the clayey sand and clay soils following heavy or prolonged precipitation events. These soils were encountered at depths of approximately 4 to 6 feet in about half of the soil borings performed. In addition, a seasonal effect will also occur in which higher groundwater levels are normally recorded during rainy seasons.

4.0 REPORT LIMITATIONS

The Geotechnical Engineer warrants that the findings, recommendations, specifications or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

The services provided were conventional in nature and did not include any special services that may lessen the risk of conditions that can contribute to moisture, mold or other microbial contaminant growth in buildings. You may be aware that mold is abundant throughout nature and is comprised of a wide variety of microscopic fungi. Due to its nature, the potential for mold infestations cannot be completely eliminated.

The scope of services also does not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

Florida is underlain by a soluble limestone formation, which can dissolve and result in surface subsidence and the formation of sinkholes. A comprehensive assessment of the potential for sinkhole development at this site was not included in our scope of services. Typically, such an assessment would include geophysical surveying methods.

The recommendations submitted are based on the available subsurface information obtained by PSI and available design information furnished by the Client for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the recommendations are required.

APPENDIX

Reference: Aerial obtained from Google

LEGEND

⌖ Approximate SPT boring location

BORING LOCATION PLAN

0 250'

PLAN SCALE

N

intertek psi

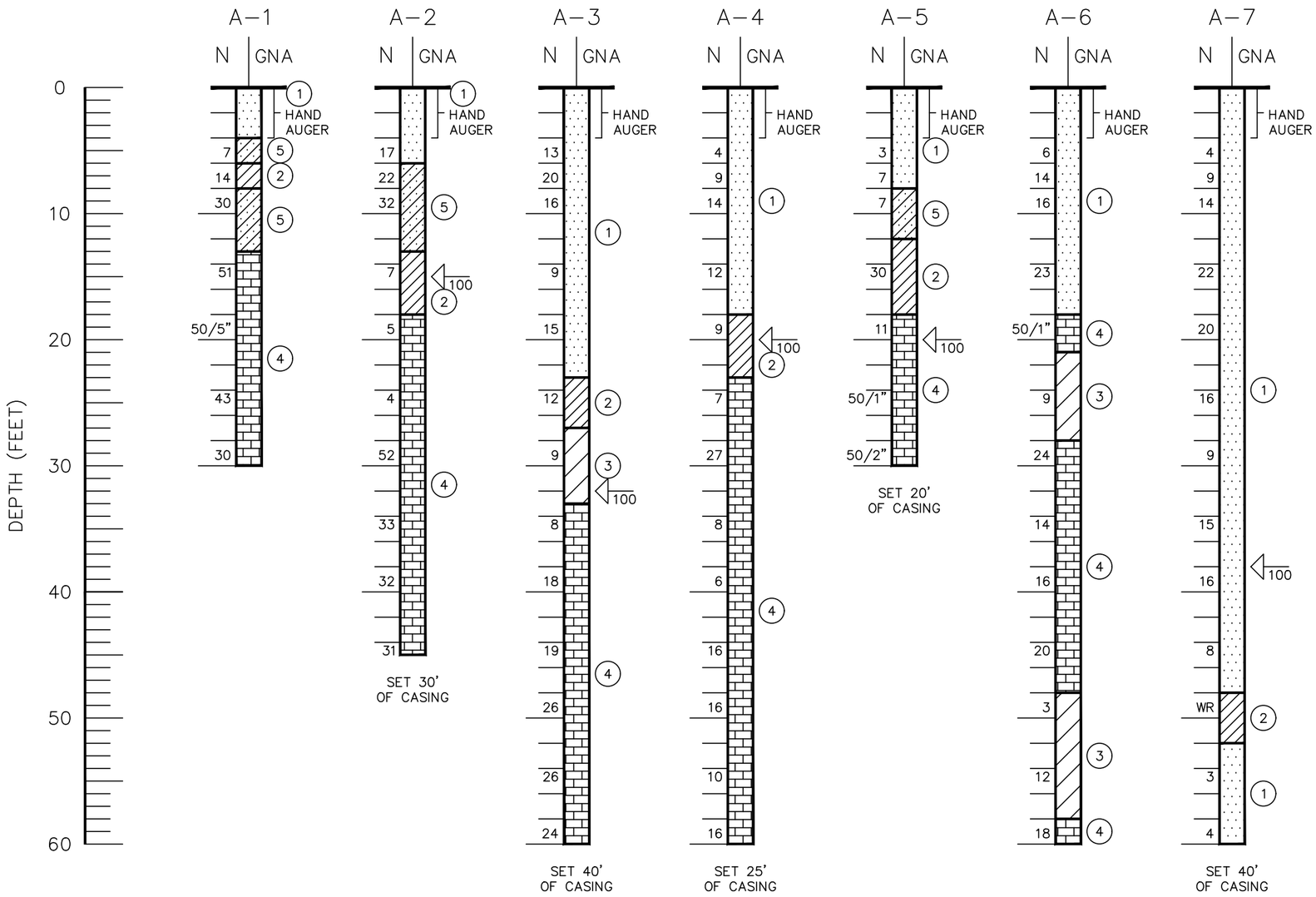
DRAWN	DJG
CHECKED	CA
APPROVED	LTL
SCALE	NOTED

GEOTECHNICAL SERVICES

ASH MONOFILL

PASCO COUNTY, FLORIDA

DATE	PROJ. NO.	SHEET
MAR 21	03901933	1



LEGEND

1. Tan/gray/brown fine SAND to slightly silty sand (SP/SP–SM)
2. Light gray high plastic CLAY with some sand (CH)
3. Orange/gray high plastic CLAY with highly weathered limestone fragments and calcareous clay (CH)
4. Light gray highly weathered LIMESTONE with calcareous clay
5. Gray/orange/light brown clayey SAND (SC)

SP — Unified Soil Classification System (ASTM D 2487) group symbol as determined by visual review

GNA — Groundwater level not apparent

N — SPT N–value in blows/foot

WR — Fell under weight of rod

50/6" — Fifty blows for six inches

← — Loss of circulation (%)

SOIL PROFILES

0 10'

VERTICAL SCALE

intertek psi

DRAWN	DJG
CHECKED	CA
APPROVED	LTL
SCALE	NOTED

GEOTECHNICAL SERVICES

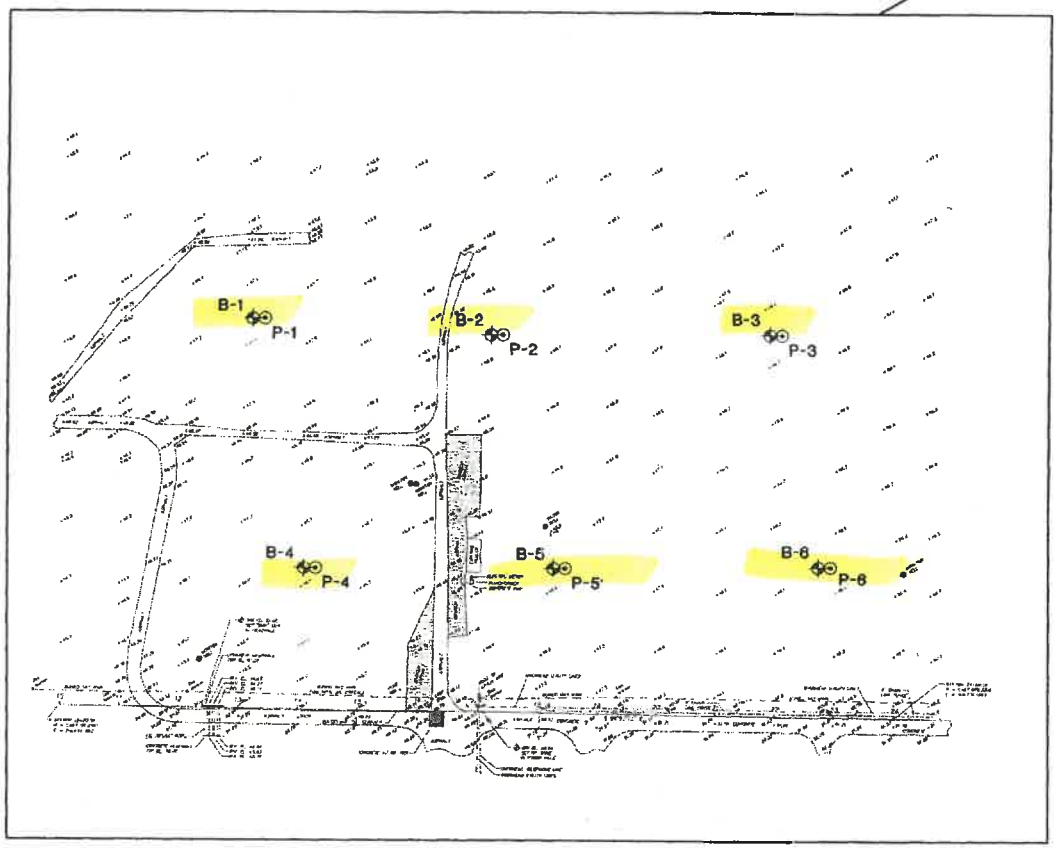
ASH MONOFILL

PASCO COUNTY, FLORIDA

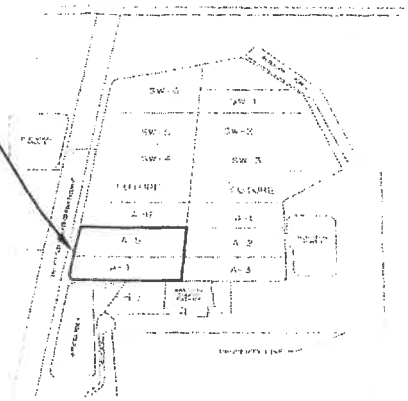
DATE	PROJ. NO.	SHEET
MAR 21	03901933	2

Cell A-4

50' deep



"DETAIL AREA"



LEGEND

- Approximate SPT boring location (Performed September 2005)
- Approximate Piezometer location (Performed January 2006)

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
DEC 10 2008
SOUTHWEST DISTRICT TAMPA

BORING LOCATION PLAN

0 200'

PLAN SCALE

DRAWN	AN
CHECKED	DEV
APPROVED	MEM
SCALE	NOTED

GEOTECHNICAL SERVICES

ASH CELL A-4 EXPANSION

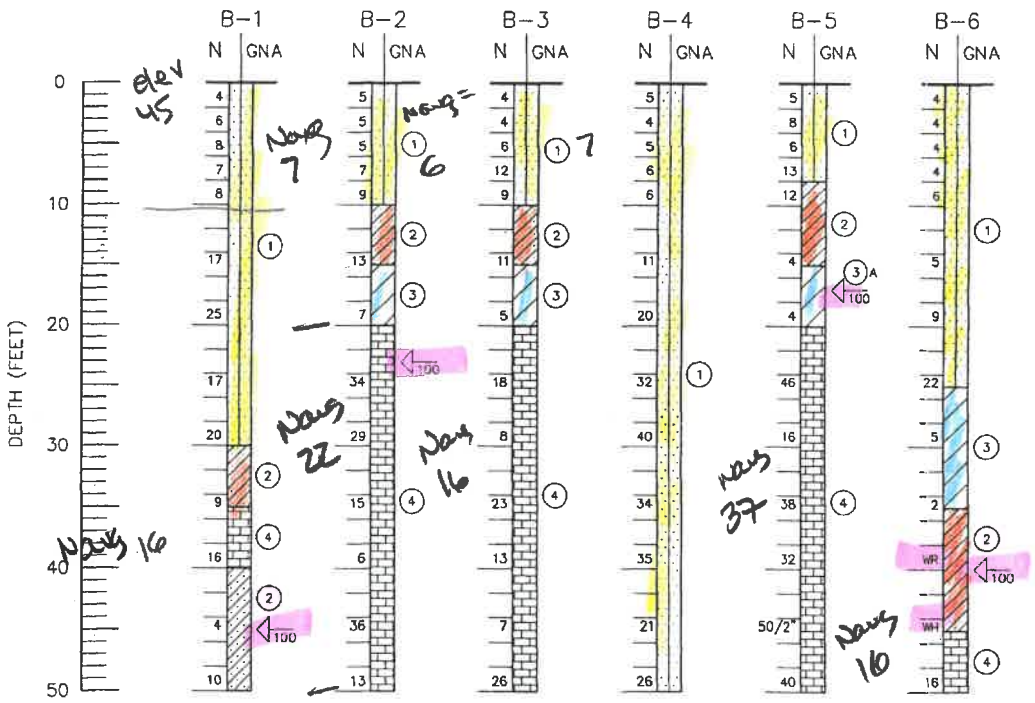
SPRING HILL, FLORIDA

PSI Information To Build On
Engineering • Consulting • Testing

DATE	PROJ. NO.	SHEET
SEP 05	787-65018	SHEET 2

P:\787 Geo Comm\...\65018 Ash Cell 4\65018a2

2005



SOIL PROFILES

0 5'

VERTICAL SCALE

LEGEND

1	White to yellowish brown to brown fine SAND to slightly silty fine SAND (SP/SP-SM)
2	Green to gray to orange sandy clay to clayey SAND (SC/CL)
3	Green CLAY with orange mottling (CH)
4	Weathered LIMESTONE
A	With limestone fragments
SP	Unified Soil Classification System (ASTM D 2487) group symbol as determined by visual review
N	SPT N-value in blows/foot
GNA	Groundwater level not apparent
←	Loss of circulation (%)
WR	Fell under weight of rod
WH	Fell under weight of rod & hammer
50/6"	Fifty blows for six inches

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
DEC 10 2008
SOUTHWEST DISTRICT TAMPA

DRAWN	AN
CHECKED	DEV
APPROVED	MEM
SCALE	NOTED

GEOTECHNICAL SERVICES

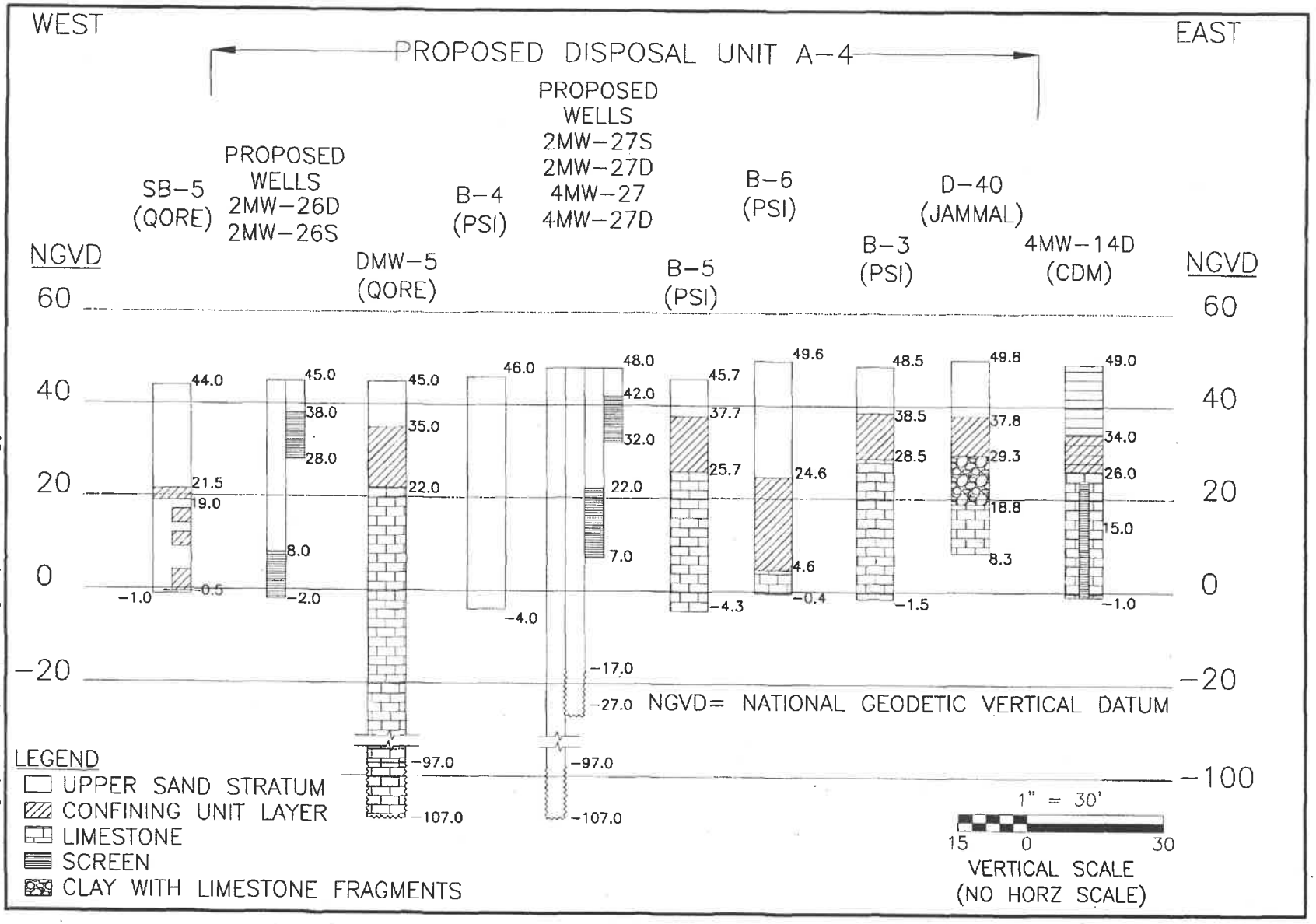
ASH CELL A-4 EXPANSION

SPRING HILL, FLORIDA

PSI Information To Build On
Engineering • Consulting • Testing

DATE	PROJ. NO.	SHEET
SEP 05	787-65018	SHEET 3

P:\787 Geo Summary\65018 Ash Cell 4\65018s3



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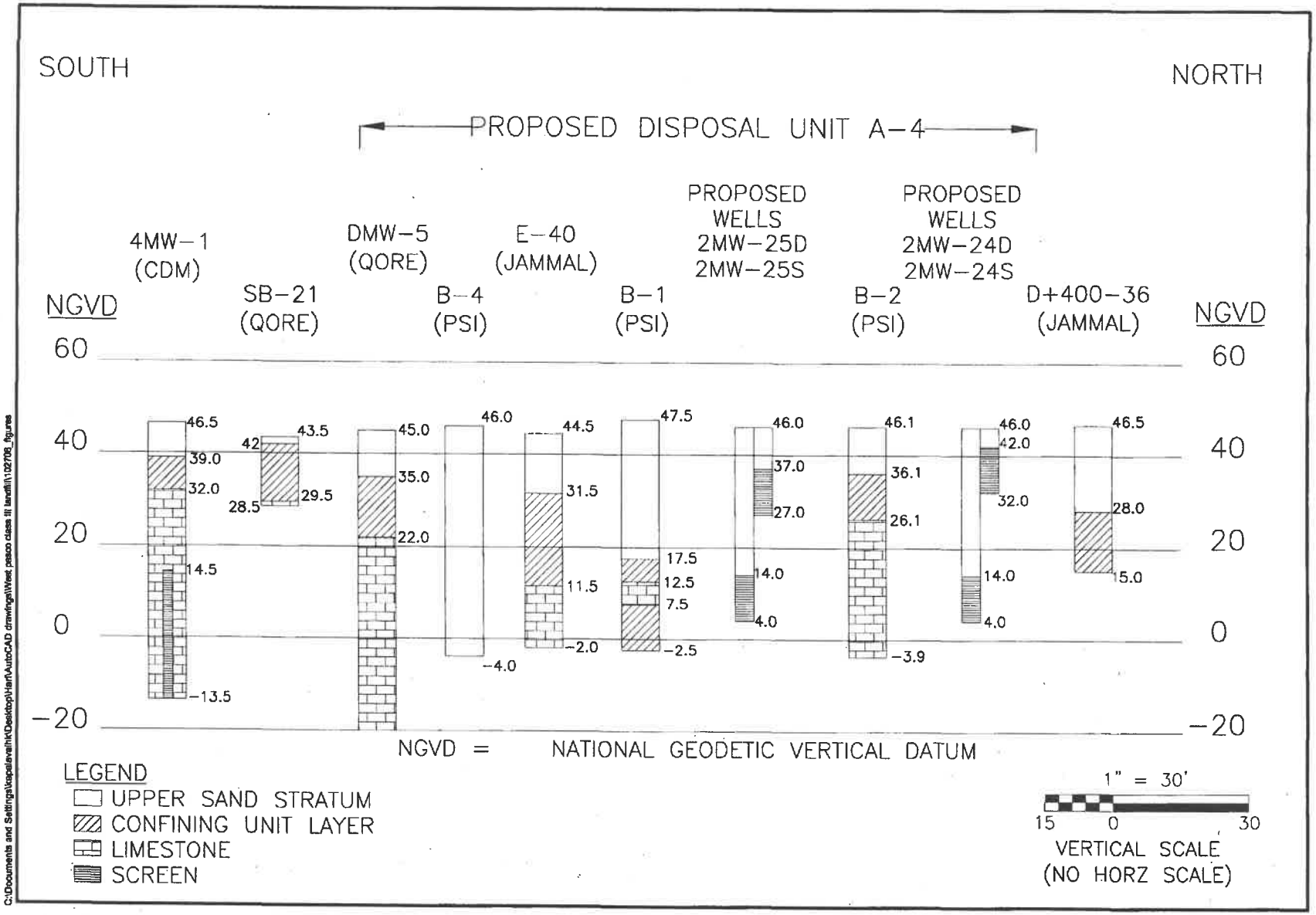
CDM
consulting • engineering • construction • operations

FIGURE NO. 3
GEOLOGICAL CROSS-SECTION OF WELLS AND BORINGS IN AND AROUND PROPOSED DISPOSAL CELL A-4 OF THE WEST PASCO CLASS I LANDFILL



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CDM
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FIGURE NO. 4
GEOLOGICAL CROSS-SECTION OF WELLS AND BORINGS IN AND AROUND PROPOSED DISPOSAL CELL A-4 OF THE WEST PASCO CLASS I LANDFILL

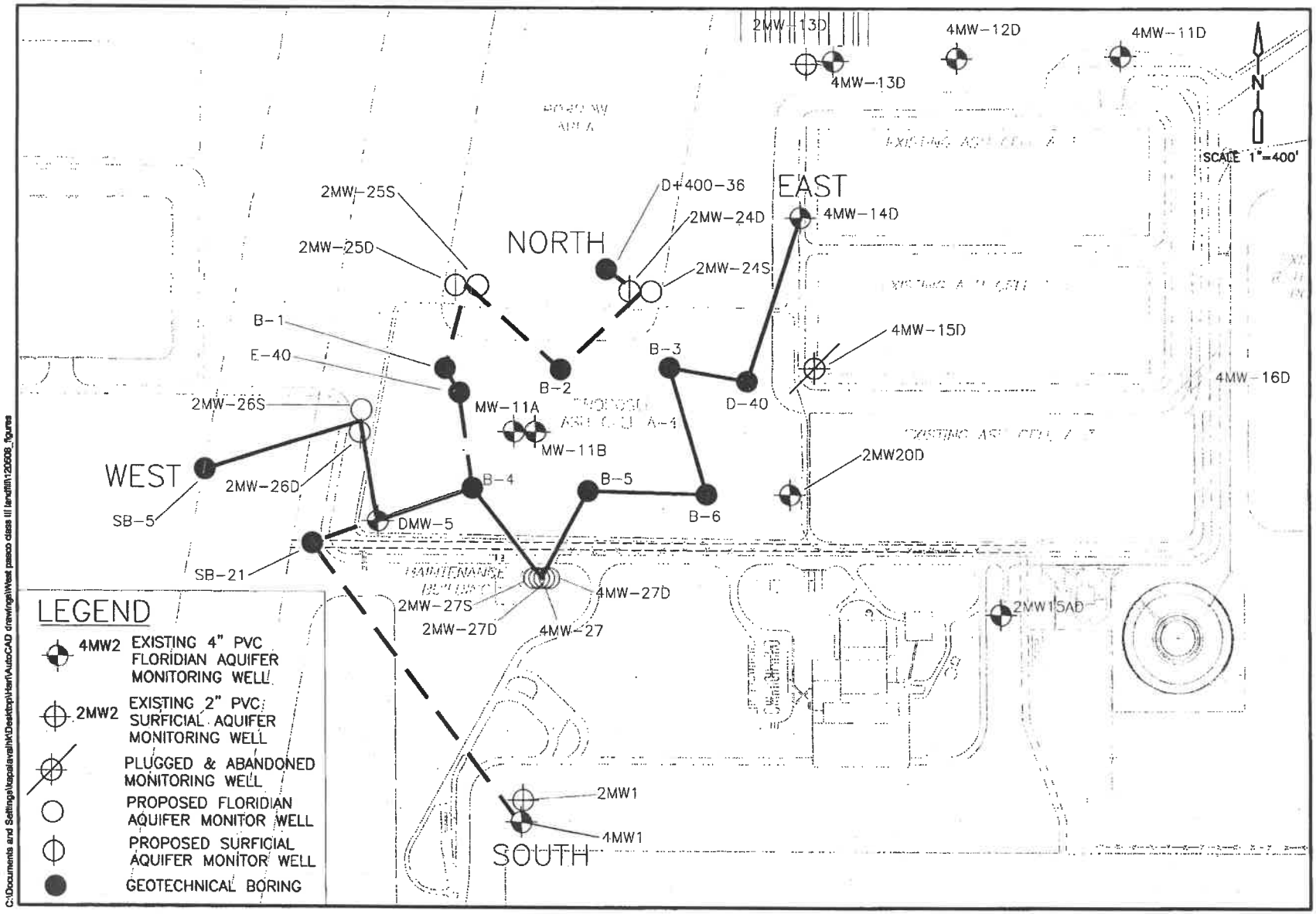

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FIGURE NO. 5
CROSS-SECTION LOCATION MAP OF SOUTHERN PORTION OF WEST PASCO COUNTY CLASS I LANDFILL

LAW
ENGINEERING AND ENVIRONMENTAL SERVICES

March 30, 1994

Ash - A-2

Mr. Richard Mayer
Law Environmental, Inc.
1715 N. Westshore Blvd., Suite 875
Tampa, Florida 33607

Subject: **Results of Geotechnical Exploration**
Pasco County Resource Recovery
Pasco County, Florida
Law Engineering Project No. 461-06530.27

Dear Mr. Mayer:

Law Engineering is pleased to submit the results of our geotechnical exploration for the subject project. Our services were provided in accordance with the Intercompany Subconsulting Agreement which was signed by Mr. Richard Mayer of Law Environmental and Mr. John Phillips of Law Engineering.

The scope of work for this project was provided by Law Environmental and consisted of the following:

- Set 5 piezometers;
- Drill 5 soil test borings;
- Take 3 undisturbed samples;
- Perform 3 consolidation tests;
- Perform 5 grain size tests.

The geotechnical exploration was performed at the Pasco County Resource Recovery site immediately south of the existing cell A-1. = A-2

The borings performed encountered a relatively uniform subsurface profile consisting of 3 strata. Beneath the ground surface, the borings encountered clean fine sands to depths ranging from 6 feet to 20 feet below grade. In boring LB-2, the fine sand was encountered to the termination depth of 26 feet. The sandy soil encountered was very loose to firm with standard penetration resistance values (N-values) ranging from 1 to 22 blows per foot.

The borings then encountered a more cohesive clayey soil ranging in thickness from 2 to 16 feet in thickness. The clay layer was firm to very stiff with N-values ranging from 6 to 23 blows per foot. Three undisturbed samples were obtained from this clay layer and consolidation tests were performed.

In two of the five borings, the limestone formation was encountered at depths of 13.5 and 16 feet below existing grade. The formation consisted of calcareous silty clay with limestone fragments. The limestone encountered was very stiff to very hard with N-values ranging from 29 blows per foot to 50 blows for 3 inches of penetration.

The subsurface conditions encountered are shown on the Test Boring Records in the Appendix. These records represent our interpretation of the subsurface conditions based on the field logs, visual examination of field samples by an engineer, and tests of the field samples. The lines designating the interface between various strata on the Test Boring Records represent the approximate interface location. In addition, the transition between strata may be gradual.

In addition to the soil test borings performed, the following 5 piezometers were installed to the following depths:

Piezometer	Depth
LB-1	14 feet
LB-3	29 feet
LB-5	22 feet
LB-7	14 feet
LB-11	12 feet

At test locations LB-6, LB-9, and LB-12 an auger boring was drilled to 21, 21, and 6 feet, respectively, however, no piezometers were set at these locations.

The results of the three consolidation tests are presented in the Appendix. The samples tested were classified as slightly sandy clay soils. Also, five grain size tests were performed on samples obtained from each soil test boring. As requested, the grain size samples tested were from the surficial sandy stratum. The grain size curves are also presented in the Appendix.

We were informed that the landfill cell would likely be 75 to 100 feet in height. We were also informed that the landfill material weighs about 85 pcf. This corresponds to a stress in the range of 6.4 to 8.5 ksf. Using this stress range together with the soil conditions found in the soil test borings together with the results of our laboratory testing, we calculated that settlement should range from 1.5 to 2.0 feet. Ultimate bearing capacities over 15 ksf should be available at this site.

We wish to remind you that our exploration services include storing the samples collected and making them available for inspection for 60 days. The samples are then discarded unless you request otherwise.

We appreciate the opportunity to work on this project. If you have any questions, please give us a call.

Sincerely,

LAW ENGINEERING, INC.

John C. Phillips, P.E.
Senior Geotechnical Engineer
Registered Florida No. 47586

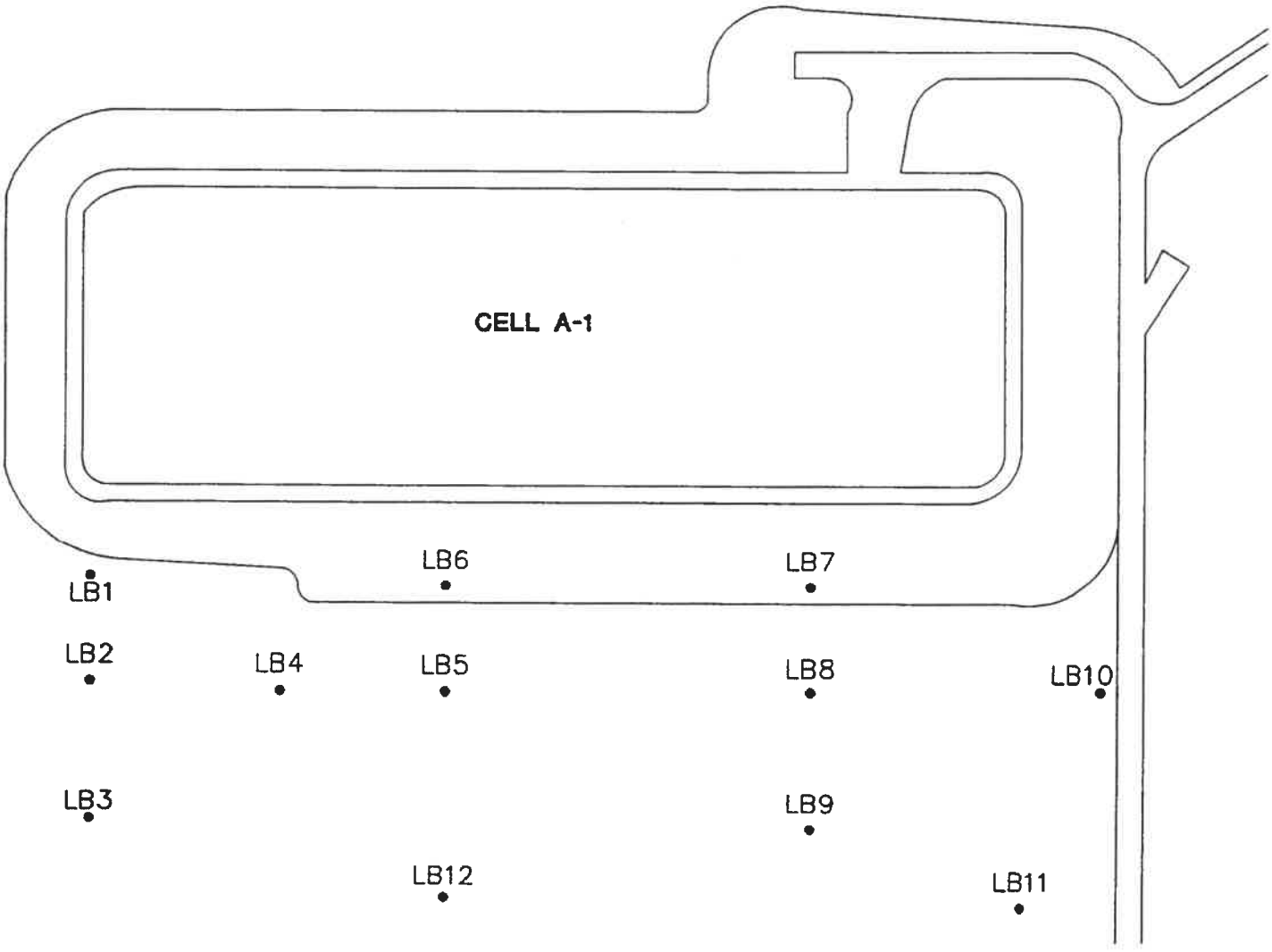
Curtis J. Roos, P.E.
Chief Engineer
Registered Florida No. 27570

JCP/CJR:sjm/1653027.RGE

Distribution: 2 - Addressee

Appendix:
Field Exploration Plan
Key to Classifications and Symbols
Test Boring Records
Grain Size Curves
Consolidation Test Results
Field Testing Procedures
Laboratory Testing Procedures

APPENDIX



0 100 200 300

APPROXIMATE SCALE IN FEET

REFERENCE: Site Plan Provided By Law Environmental

Prepared/Date: JLD 3/4/94
Checked/Date: JCP 3/4/94

LAW ENVIRONMENTAL – TAMPA
PASCO COUNTY RESOURCE RECOVERY
PASCO COUNTY, FLORIDA

LAW
ENGINEERING AND ENVIRONMENTAL SERVICES

FIELD EXPLORATION PLAN

Project 461-06530.27 Figure

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	NO. OF BLOWS, N	RELATIVE DENSITY
SANDS:	0-4	Very Loose
	5-10	Loose
	11-30	Firm
	31-50	Dense
	OVER 50	Very Dense

		CONSISTENCY
	0-2	Very Soft
	3-4	Soft
SILTS	5-8	Firm
&	9-15	Stiff
CLAYS:	16-30	Very stiff
	31-50	Hard
	OVER 50	Very Hard

PARTICAL SIZE IDENTIFICATION

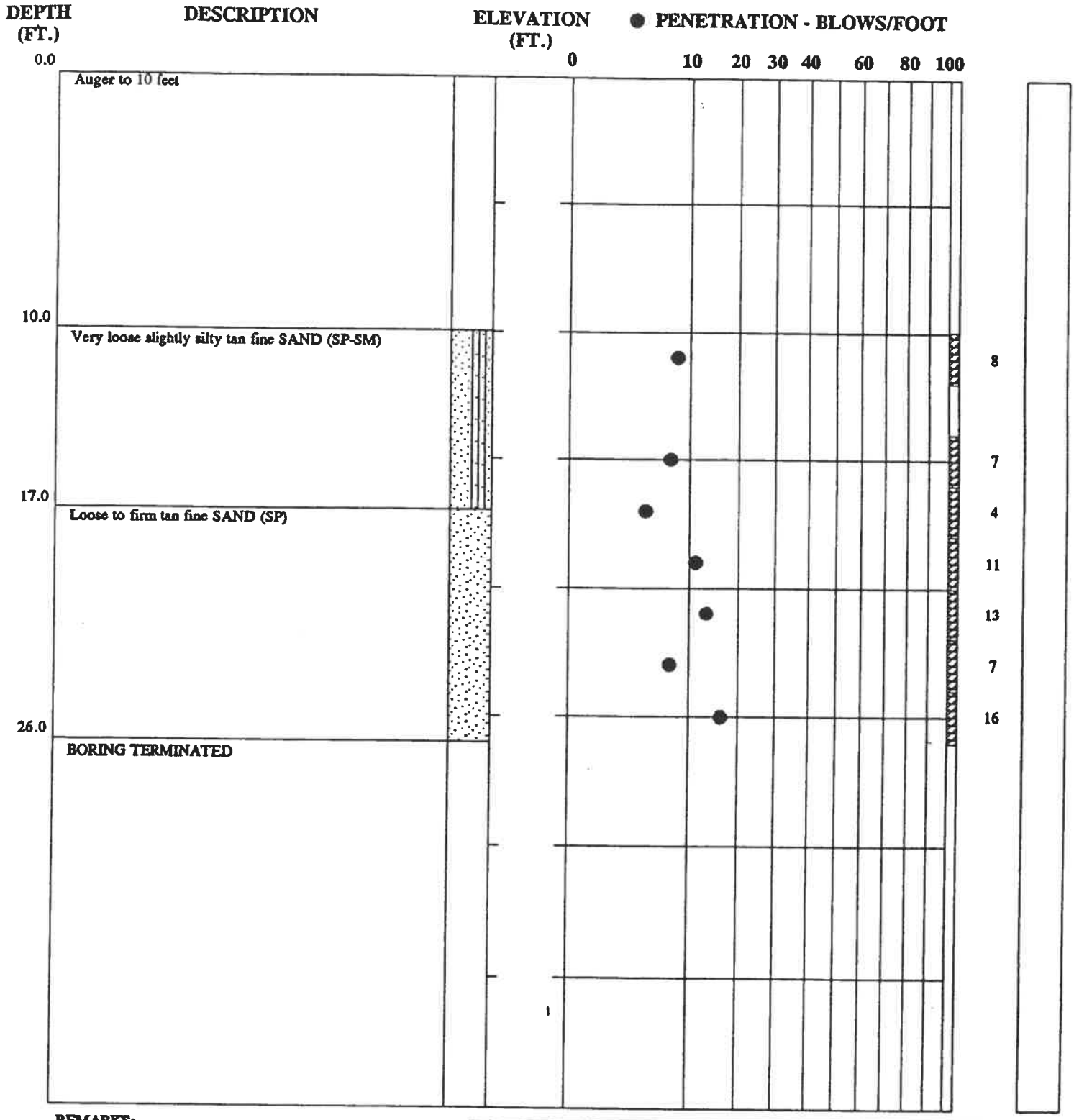
BOULDERS:		Greater than 300 mm
COBBLES:		75 mm to 300 mm
GRAVEL:	Coarse -	19.0 mm to 75 mm
	Fine -	4.75 mm to 19.0 mm
SANDS:	Coarse -	2.00 mm to 4.75 mm
	Medium -	0.425 mm to 2.00 mm
	Fine -	0.075 mm to 0.425 mm
SILTS & CLAYS:		Less than 0.075 mm

KEY TO DRILLING SYMBOLS

- Undisturbed Sample
- Split Spoon Sample
- Water Table 24 HR.
- Water Table at Time of Drilling
- M=82% Moisture Content
- Loss of Drilling Water

KEY TO SOIL CLASSIFICATIONS

- ASPHALT
- SP - Poorly graded sands
- SP-SM - Slightly silty sands
- SM - Silty sands
- SP-SC - Slightly clayey sands
- SC - Clayey sands
- SW - Well graded sands
- CL - Low plasticity inorganic clays
- CH - High plasticity inorganic clays
- MH - High plasticity inorganic silts
- ML - Low plasticity inorganic silts
- OH - High plasticity organic silts and clays
- OL - Low plasticity organic silts and clays
- LIMESTONE FORMATION - Weathered limestone
- LIMESTONE FORMATION - Limestone
- PEAT - Peat
- FILL - Sand fill



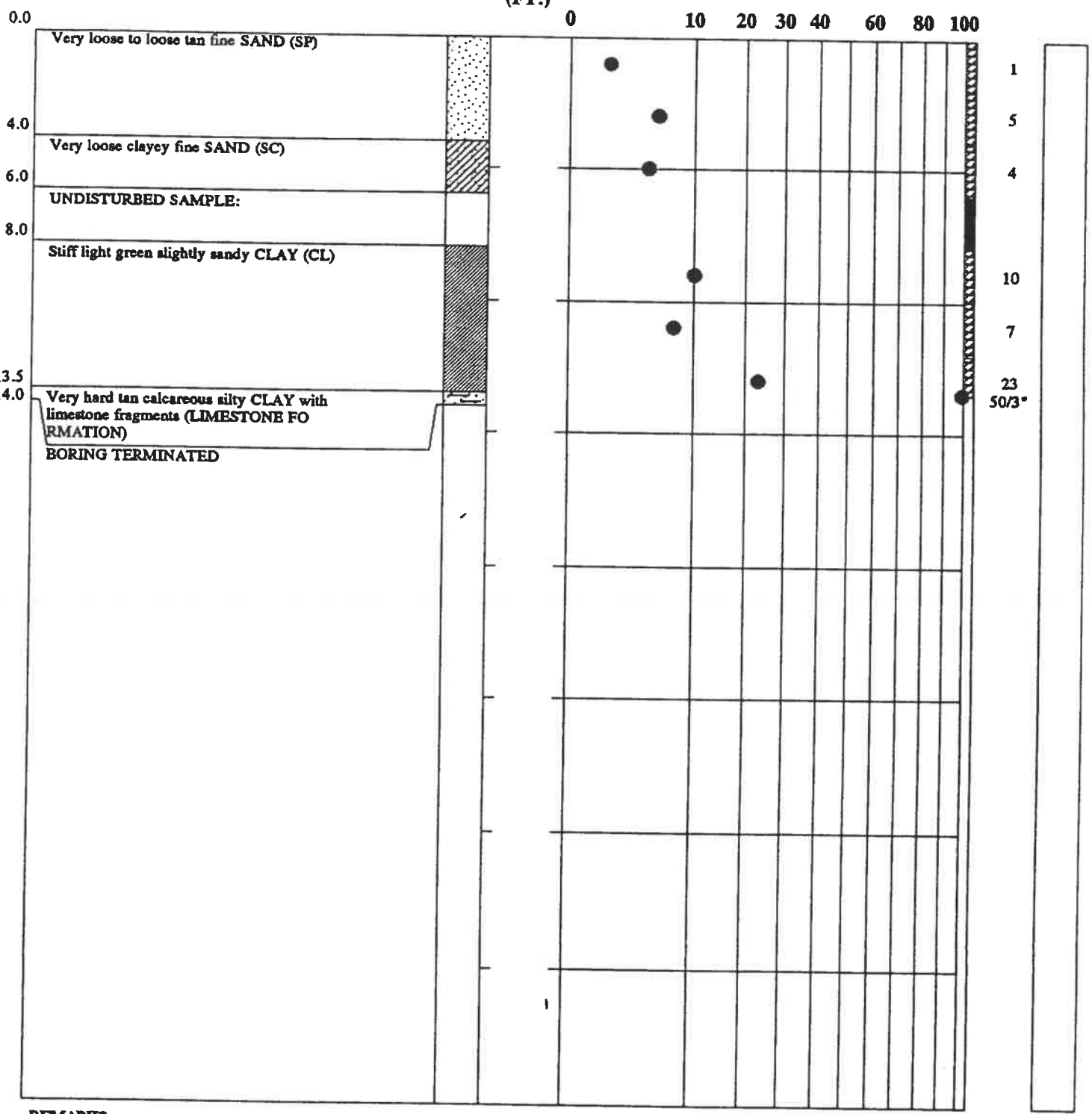
REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	LB-2
DATE DRILLED	November 23, 1993
PROJECT NUMBER	461-06530.27
PROJECT	PASCO COUNTY RESOURCE RECOV
PAGE 1 OF 1	

LAW ENGINEERING



REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	LB-4
DATE DRILLED	November 23, 1993
PROJECT NUMBER	461-06530.27
PROJECT	PASCO COUNTY RESOURCE RECOV
PAGE 1 OF 1	

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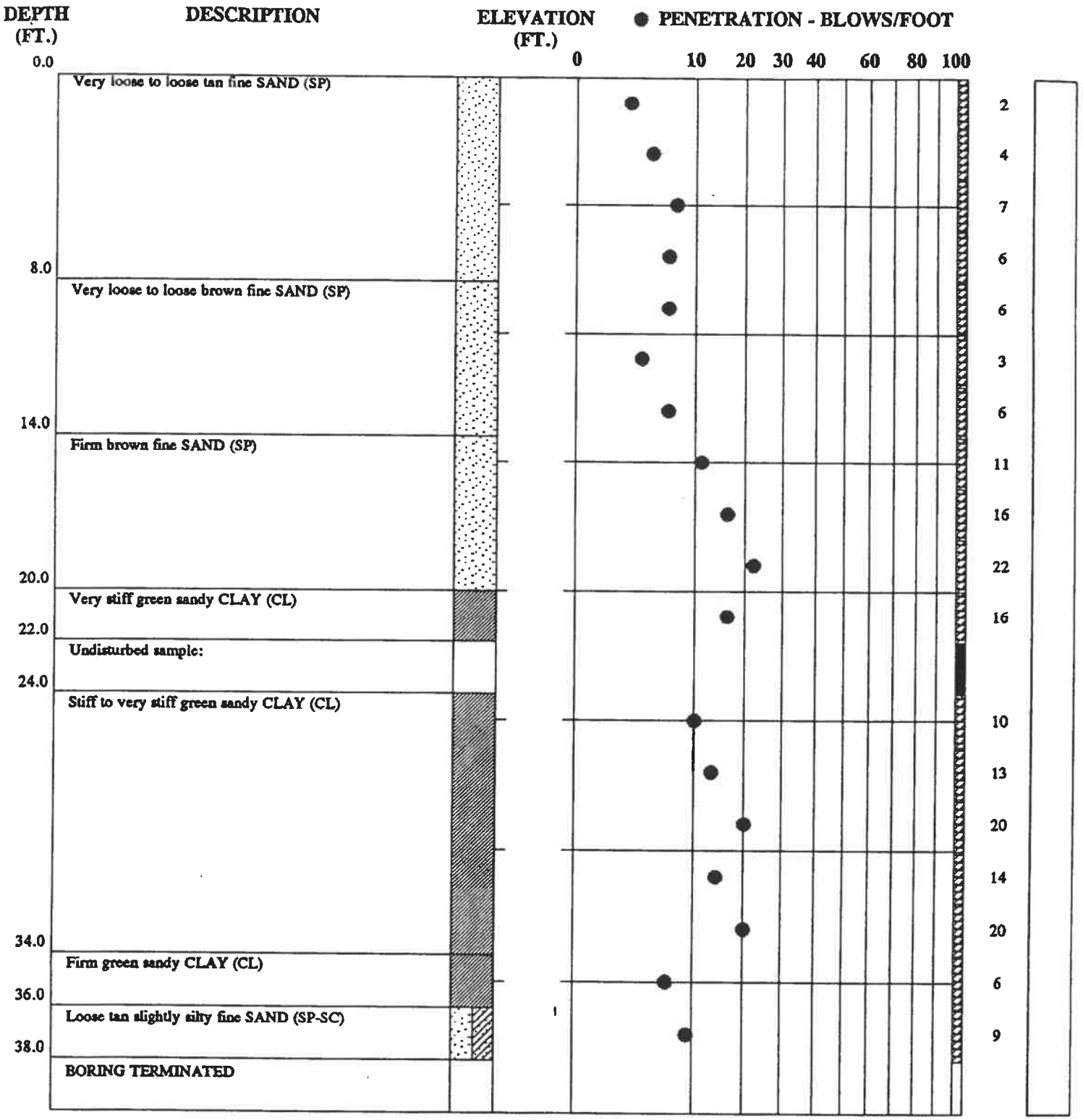


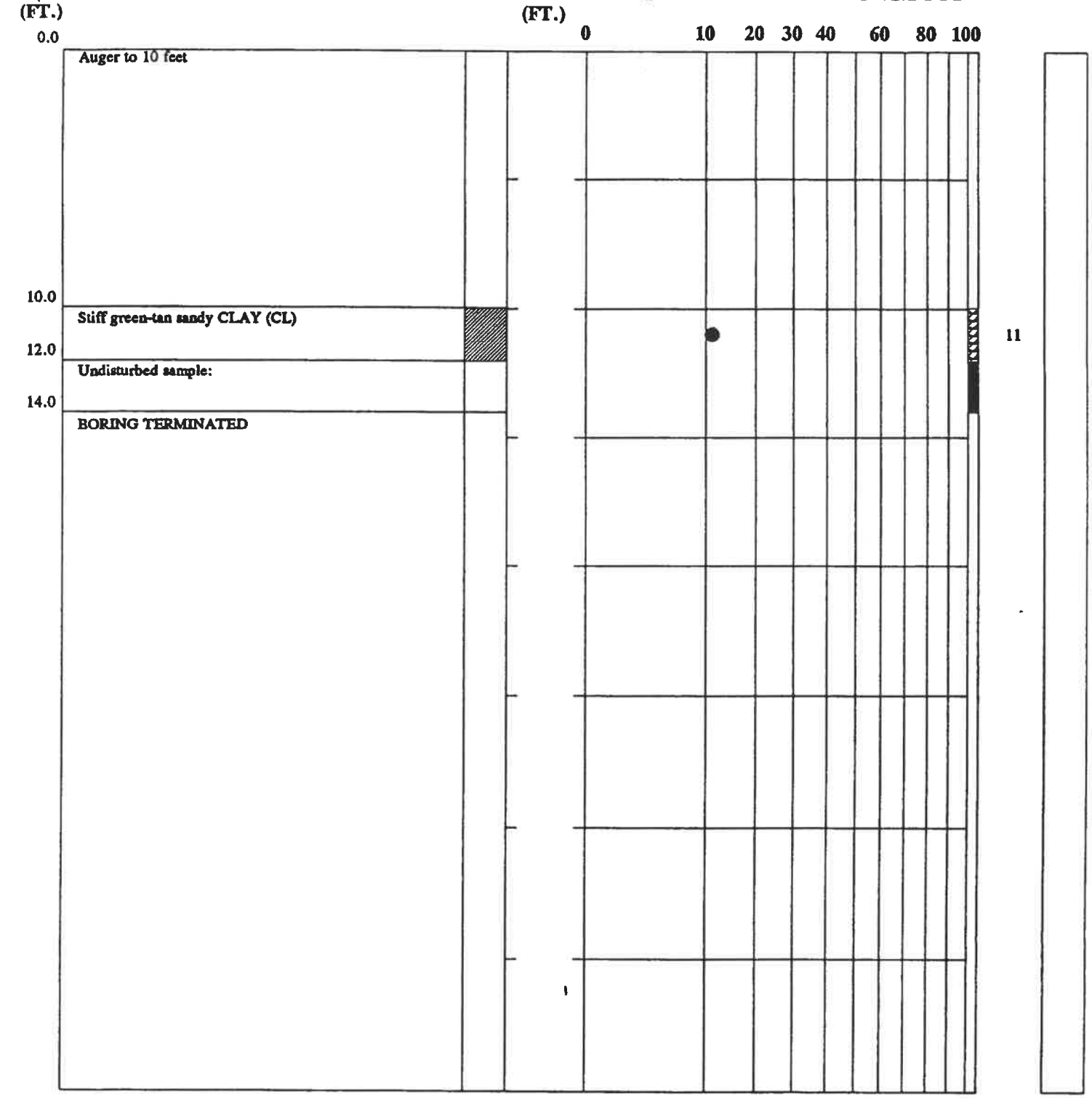
REMARKS:

TEST BORING RECORD

BORING NUMBER	LB-5
DATE DRILLED	November 22, 1993
PROJECT NUMBER	461-06530.27
PROJECT	PASCO COUNTY RESOURCE RECOV
PAGE 1 OF 1	

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

LAW ENGINEERING



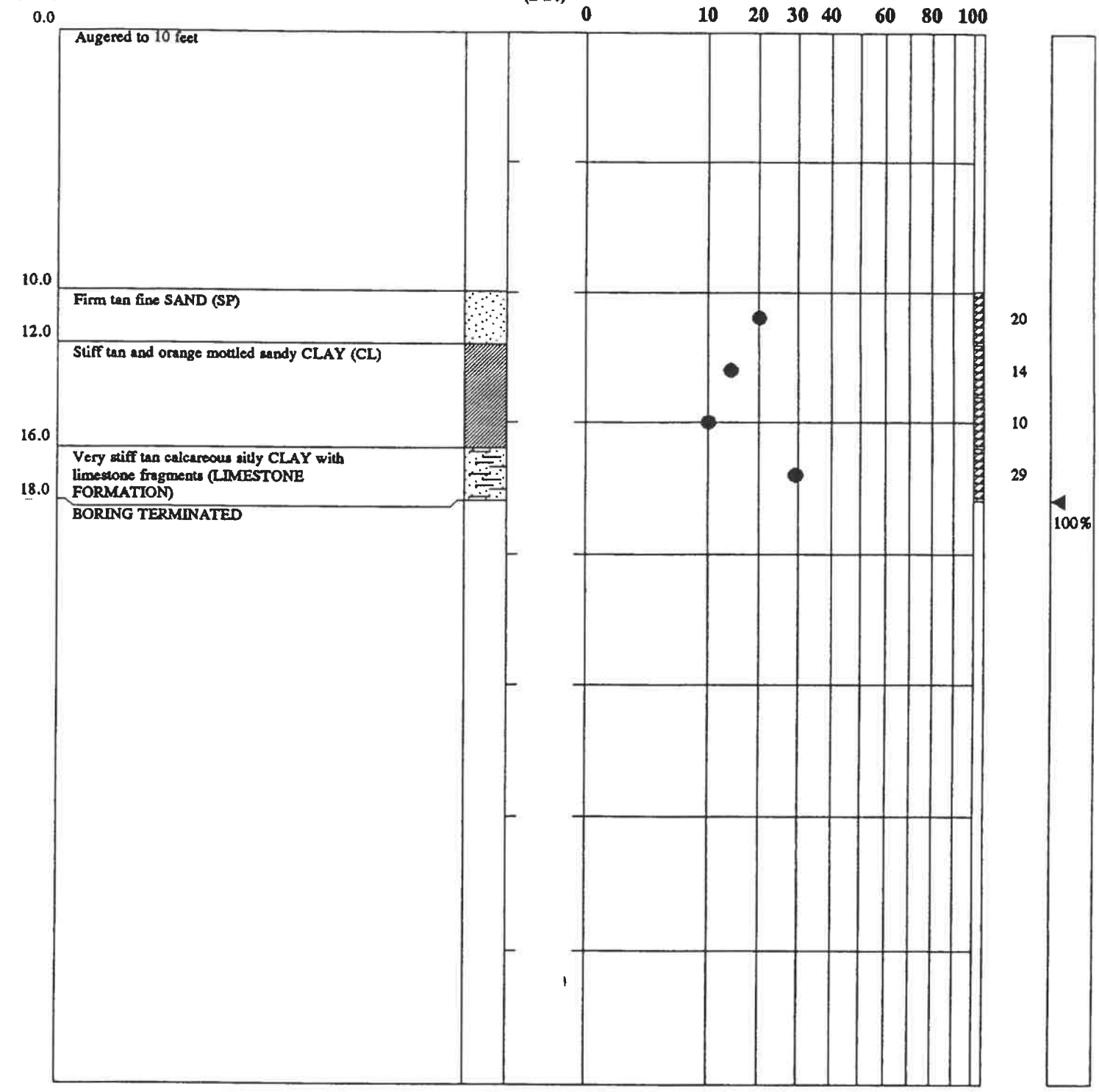
REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	LB-8
DATE DRILLED	November 23, 1993
PROJECT NUMBER	461-06530.27
PROJECT	PASCO COUNTY RESOURCE RECOV
PAGE 1 OF 1	

LAW ENGINEERING



REMARKS:

TEST BORING RECORD	
BORING NUMBER	LB-10
DATE DRILLED	November 22, 1993
PROJECT NUMBER	461-06530.27
PROJECT	PASCO COUNTY RESOURCE RECOV
PAGE 1 OF 1	

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

LAW ENGINEERING

Attachment I.1.a(6)

Material Engineering Parameters and References

Reference 1	**US Department of the Interior (1987) Design of Small Dams - Table 5-1**
Reference 2	**Ardaman & Associates (2007) Test Data from Landfilled WTE Ash**
Reference 3	**Duncan and Wright (2005) Soil Strength and Slope Stability, Fig. 5.21**
Reference 4	**Bowles (1996) Foundation Analysis and Design - Table 5-6**
Reference 5	**Terzaghi, Peck, Mesri (1996) Soil Mechanics in Engineering Practice**

TECHNICAL MEMORANDUM

West Pasco County Class I Ash Monofill Slope Modification and Vertical Expansion

TO: Project File

FROM: Chad Spreadbury, PhD, EI

XC: Carol Sawyer, PE

DATE: June 11, 2021

SUBJECT: Attachment I.1.a(6) – Material Parameters
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

This Technical Memorandum summarizes the material classifications and engineering parameters used in the settlement, bearing capacity, and slope stability analyses. The attached calculations present the materials and the material parameters.

2 METHODOLOGY

Determination of the soil, waste, and bottom liner material parameters are based on laboratory testing, published data, empirical correlations, subsurface investigations, and experience with similar materials.

3 PARAMETERS

3.1 Soil Classification

The soil classifications are used to define the material types and differentiate the soil layers used for the geotechnical analyses. Soil classifications are based on visual identification during drilling and subsequent laboratory testing of representative Standard Penetration Test (SPT) split-spoon samples. Soil classifications are based on the Unified Soil Classification System (USCS) standards (ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes [Unified Soil Classification System]).

3.2 Settlement Analysis Parameters

Settlement of the foundation soils is calculated based on determining the changes in stress imposed on the soil due to excavation of the existing soil and addition of the proposed landfill loads. Settlement of generally granular soils is calculated based on elastic settlement

equations; settlement of generally cohesive soils is calculated based on consolidation equations. Listed below are the material parameters required for the settlement analyses:

1. Elastic settlement equations incorporate the following material parameters:

    a. Young's Elastic Modulus (Es): measure of soil stiffness; ratio of stress over strain; calculated based on the soil's SPT N-value.
    b. Poisson's ratio (υ): ratio of axial compression to lateral expansion; it is determined based on the soil type.

2. The consolidation settlement parameters are determined based on empirical correlations with Liquid Limit (LL) and Plastic Index (PI) determined by laboratory testing and published values for similar soils. The consolidation settlement parameters are defined as follows: P_C

    a. Primary compression index (Cc).
    b. Recompression index (Cr). For normally consolidated soils (where the initial stress, σ_0, is equal to the pre-consolidation pressure, P_C) Cr is modeled as 0. The soils at this site are over-consolidated; therefore, Cr is used in the settlement calculations.
    c. Initial void ratio (e_0).
    d. Pre-consolidation pressure (P_C) is determined based on the results of the consolidation tests for similar soils. Expressed in pounds per square foot (psf), this is the maximum past pressure seen by the soils. As is typical, the pre-consolidation pressure is consistent for this site. However, there are conditions where upper layers can have a lower pre-consolidation pressure than the lower layers. For normally consolidated soils, P_C is modeled as 0 psf.

3.3 MATERIAL STRENGTH PARAMETERS

The material strength parameters used in the foundation analyses are defined as the following:

1. Unit Weight or density (γ) of the material expressed in terms of pounds per cubic foot (pcf). Unit weight is used to calculate overburden pressures. Unit weight is estimated based on SPT N-values and soil type.

2. Cohesion (c) as defined by the Mohr-Coulomb relationship of shear stress to principal stress is expressed in psf. This is determined based on laboratory testing, comparison to empirical calculation methods, and published values for similar soils. Because the landfill is filled slowly over time, using the effective stress (drained) parameters is appropriate. However, the total stress (undrained) parameters can be used as a conservative estimate.

3. Angle of internal friction (φ) of the soil, or interface friction angle (δ) as defined by the Mohr-Coulomb relationship of shear stress to principal stress expressed in terms of degrees (°). This is determined based on laboratory testing, comparison to empirical calculation methods, and published values for similar soils. Because the landfill is filled slowly over time – using the effective stress (drained) parameters is appropriate. Using the total stress (undrained) parameters can be used as a conservative estimate.

JonesEdmunds	PROJECT NUMBER:	16002-009-01	Attachment I.1.a(6)-1
	PROJECT NAME:	West Pasco Ash Monofill Slope Mod & Vertical Expansion	
	SUBJECT:	Material Parameters Summary	
	BY:	C.Spreadbury	Date: 6/21/21
	CHECKED BY:	C.Sawyer	Date: 6/28/21

Objective:

Summarize the material settlement and strength parameters for use in the foundation analyses.

Parameters:

Settlement Parameters:

Young's Elastic Modulus (Es)	Reference 4 - Bowles (1996) Foundation Analysis and Design - Table 5-6
Poisson's Ratio (υs)	Reference 4 - Bowles (1996) Foundation Analysis and Design - Table 2-7
Primary Compression index (Cc)	Published correlations with Liquid Limit and Plastic Index
Recompression index (Cr)	Published correlations with Liquid Limit and Plastic Index
Initial void ratio (e_0)	Published correlations with Liquid Limit and Plastic Index
Pre-consolidation pressure (Pc)	Comparison to similar soils in Polk County

Strength Parameters:

Unit Weight (γ_T) (pcf)	Reference 1 - US Department of the Interior (1987) Design of Small Dams
	Reference 2 - Ardaman & Associates (2007) Test Data from Landfilled WTE Ash
Cohesion (c) (psf)	Reference 1 - US Department of the Interior (1987) Design of Small Dams
Friction (Φ or δ) (°)	References vary - see below

Landfill:

Final Cover Material

Classification:

The final cover components are modeled as one layer. The total thickness of the final cover soil is 3 feet (modeled as SP-SC, slightly clayey sand, which includes (listed from top to bottom):

- 2-foot-thick final cover soil.
- Double-sided nonwoven geocomposite.
- Textured geomembrane.
- 1-foot-thick intermediate cover soil

Engineering Parameters:

γ_T	120	pcf	Reference 1 - US Department of the Interior (1987) , Table 5-1
			The unit weight of the final cover material is dominated by the unit weight of the soil.
c	0	psf	Assume cohesion is zero for granular soils.
Φ	30	deg	Reference 1 - US Department of the Interior (1987) , Table 5-1
δ	30	deg	Interface Friction Angle. Attachment I.1.e(3)(a) Veneer Slope Stability

Class I Waste (WTE Ash)

Classification:

Waste (WTE ash) properties are based on literature values for well compacted waste (ash).
Newer waste (ash) is not expected to act as a cohesive material - conservative assumption.

Engineering Parameters:

γ_T	110	pcf	Reference 2 - Ardaman & Associates, Inc., 2007.
			Ash specimens from Southwest FL.
c	0	psf	Assume zero for non-cohesive material.
Φ	35	deg	Reference 2 - Ardaman & Associates, Inc., 2007. (noted 44 degrees but we used lower value)

Trench Gravel

Classification:

Trench gravel is located in the leachate collection trench and the material properties are used for pipe calculations.

Engineering Parameters:

γ_T	125	pcf	Reference 1 - US Department of the Interior (1987), Table 5-1
			90% of total average density using maximum unit weight and optimum moisture content.
c	0	psf	Assume zero cohesion for granular soil.
Φ	35	deg	

JonesEdmunds	PROJECT NUMBER:	16002-009-01	Attachment I.1.a(6)-1
	PROJECT NAME:	West Pasco Ash Monofill Slope Mod & Vertical Expansion	
	SUBJECT:	Material Parameters Summary	
	BY:	C.Spreadbury	Date: 6/21/21
	CHECKED BY:	C.Sawyer	Date: 6/28/21

Bottom Liner System

Classification:

The bottom liner system is modeled as one layer.The LCRS consists of the following materials (listed from top to bottom):

- 2-foot layer of drainage sand.
- A-1 smooth 60-mil HDPE geomembrane over compacted subgrade.
- A-2 textured 60-mil HDPE geomembrane over GCL over compacted subgrade.
- A-3 textured 60-mil HDPE geomembrane over GCL over compacted subgrade.
- A-4 textured 60-mil HDPE geomembrane over GCL over compacted subgrade.

Engineering Parameters:

γ_T	115	pcf	Reference 1 - US Department of the Interior (1987), Table 5-1
			The unit weight of the bottom liner system is estimated based on the unit weight of the protective soil and drainage sand (both will be the same material), which makes up the greatest mass of the bottom liner system.
c	0	psf	Assume zero cohesion (or adhesion) of the bottom liner.
Φ	10	deg	Attachment I.1.e(3) - Slope Stability Analysis
			Interface friction tests were performed in April 2019 on actual bottom liner material from both cells, SW-1 and SW-2. The interface test results exceed the minimum required as determined by the slope stability analysis.

Foundations Soils:

Soil 1: Compacted Subgrade (Sand): fill soil below the bottom liner - 1-foot minimum thickness

Classification:
Modeled as a granular soil with elastic settlement.

Settlement Parameters:

Es	500(N+15)/50	Reference 4 - Bowles (1996), Table 5-6
ν_s	0.3	Reference 4 - Bowles (1996), Table 2-7

Strength Parameters:

γ_T	115	pcf	Reference 1 - US Department of the Interior (1987) Design of Small Dams - Table 5-1
c	0	psf	Assume cohesion is zero for granular soils.
Φ	32	deg	Reference 1 - US Department of the Interior (1987), Table 5-1
δ	10	deg	Attachment I.1.e(3) - Slope Stability Analysis

Soil 2: Sand (SP, SP-SM)

Classification:
Modeled as a granular soil with elastic settlement.

Settlement Parameters:

Es	500(N+15)/50	Reference 4 - Bowles (1996), Table 5-6
Es	250(N+15)/50	Reference 4 - Bowles (1996), Table 5-6 (Saturated)
ν_s	0.3	Reference 4 - Bowles (1996), Table 2-7

Strength Parameters:

γ_T	110	pcf	Reference 1 - US Department of the Interior (1987), Table 5-1
c	0	psf	Assume cohesion is zero for granular soils.
Φ	30	deg	Reference 1 - US Department of the Interior (1987), Table 5-1

JonesEdmunds	PROJECT NUMBER:	16002-009-01	Attachment I.1.a(6)-1
	PROJECT NAME:	West Pasco Ash Monofill Slope Mod & Vertical Expansion	
	SUBJECT:	Material Parameters Summary	
	BY:	C.Spreadbury	Date: 6/21/21
	CHECKED BY:	C.Sawyer	Date: 6/28/21

Soil 3: Sandy Clay or Clay (SC, CL, CH)

Classification:

Modeled as cohesive material with consolidation settlement.

Settlement Parameters:

Cc	0.5		Estimated based on empirical correlations with Atterberg Limits.
Cr	0.02		Reference 5 - Estimated based on correlation with Cc (Cr = Cc *0.046) see Figure 16.9 Terzaghi, Peck and Mesri (1996)
e_0	1.00		Reference 5 - Extrapolated from Figure 12.9 Terzaghi, Peck and Mesri (1996) based on Natural Moisture Content
Pc	4,000	psf	Based on published consolidation test results on Hawthorn clays.

Strength Parameters:

γ_T	120	pcf	Reference 1 - US Department of the Interior (1987), Table 5-1
c	2000	psf	Based on published consolidation test results on Hawthorn clays.
Φ	0	deg	Assume Friction angle is zero for cohesive soils.

Soil 4: Clay and Sandy Clay (CH, CL)

Classification:

Modeled as cohesive material with consolidation settlement.

Settlement Parameters:

Cc	0.88		Estimated based published information for soft sandy clays.
Cr	0.04		Reference 5 - Estimated based on correlation with Cc (Cr = Cc *0.046) see Figure 16.9 Terzaghi, Peck and Mesri (1996)
e_0	1.2		Reference 5 - Extrapolated from Figure 12.9 Terzaghi, Peck and Mesri (1996) based on Natural Moisture Content
Pc	4,000	psf	Based on published consolidation test results on Hawthorn clays.

Strength Parameters:

γ_T	120	pcf	Reference 1 - US Department of the Interior (1987), Table 5-1
c	2000	psf	Reference 1 - US Department of the Interior (1987), Table 5-1
Φ	0	deg	Assume Friction angle is zero for cohesive soils.

Soil 5: Weathered Limestone with Sandy Clay and Clay

Classification:

Modeled as a granular soil with elastic settlement.

Settlement Parameters:

Es	1200(N+6)/50		Reference 4 - Bowles (1996), Table 5-6
ν_s	0.4		Reference 4 - Bowles (1996), Table 2-7

Strength Parameters:

γ_T	125	pcf	Reference 1 - US Department of the Interior (1987), Table 5-1
c	0	psf	Assume cohesion is zero for granular soils
Φ	40	deg	Reference 1 - US Department of the Interior (1987), Table 5-1

References:

Reference 1 - US Department of the Interior (1987) Design of Small Dams - Table 5-1
Reference 2 - Ardaman & Associates (2007) Test Data from Landfilled WTE Ash
Reference 3 - Duncan and Wright (2005) Soil Strength and Slope Stability, Fig. 5.21
Reference 4 - Bowles (1996) Foundation Analysis and Design -Table 5-6
Reference 5 - Terzaghi, Peck, and Mesri (1996) Soil Mechanics in Engineering Practice

Reference 1

US Department of the Interior (1987) Design of Small Dams - Table 5-1

Reference 1

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

A Water Resources Technical Publication

First Edition, 1960
Second Edition, 1973
Revised Reprint, 1977
Third Edition, 1987

Reference 1

Table 5-1.—Average engineering properties of compacted soils. From the Western United States. Last updated October 6, 1982.

USCS soil type	Specific gravity		Compaction: Laboratory		Compaction: Index unit weight		Shear strength: Avg. placement		Shear strength: Effective stress		Values listed
	No. 4 minus	No. 4 plus	Maximum unit weight, lb/ft³	Optimum moisture content, %	Max., lb/ft³	Min., lb/ft³	Unit weight, lb/ft³	Moisture, content, %	c′ lb/in²	φ′, degrees	
	2.69	2.58	124.2	11.4	133.6	108.8	—	—	—	—	Average of all values
	0.02	0.08	3.2	1.2	10.4	10.2	—	—	—	—	Standard deviation
GW	2.65	2.39	119.1	9.9	113.0	88.5	—	—	—	—	Minimum value
	2.75	2.67	127.5	13.3	145.6	132.9	—	—	—	—	Maximum value
	16	9	5		16				0		Total number of tests
	2.68	2.57	121.7	11.2	137.2	112.5	127.5	6.5	5.9	41.4	Average of all values
	0.03	0.07	5.9	2.2	6.3	8.3	7.2	1.2	—	2.5	Standard deviation
GP	2.61	2.42	104.9	9.1	118.3	85.9	117.4	5.3	5.9	38.0	Minimum value
	2.76	2.65	127.7	17.7	148.8	123.7	133.9	8.0	5.9	43.7	Maximum value
	35	12	15		34				3		Total number of tests
	2.73	2.43	113.3	15.8	132.0	108.0	125.9	10.3	13.4	34.0	Average of all values
	0.07	0.18	11.5	5.8	3.1	0.2	0.9	1.2	3.7	2.6	Standard deviation
GM	2.65	2.19	87.0	5.8	128.9	107.8	125.0	9.1	9.7	31.4	Minimum value
	2.92	2.92	133.0	29.5	135.1	108.1	126.9	11.5	17.0	36.5	Maximum value
	34	17	36		2				2		Total number of tests
	2.73	2.57	116.6	13.9	—	—	111.1	15.9	10.2	27.5	Average of all values
	0.08	0.21	7.8	3.8	—	—	10.4	1.6	1.5	7.2	Standard deviation
GC	2.67	2.38	96.0	6.0	—	—	96.8	11.2	5.0	17.7	Minimum value
	3.11	2.94	129.0	23.6	—	—	120.9	22.2	16.0	35.0	Maximum value
	34	6	37		0				3		Total number of tests
	2.67	2.57	126.1	9.1	125.0	99.5	—	—	—	—	Average of all values
	0.03	0.03	6.0	1.7	6.0	7.1	—	—	—	—	Standard deviation
SW	2.61	2.51	118.1	7.4	116.7	87.4	—	—	—	—	Minimum value
	2.72	2.59	135.0	11.2	137.8	109.8	—	—	—	—	Maximum value
	13	2	1		12				0		Total number of tests
	2.65	2.62	115.6	10.8	115.1	93.4	103.4	5.4	5.5	37.4	Average of all values
	0.03	0.10	9.7	2.0	7.2	8.8	14.6	—	3.0	2.0	Standard deviation
SP	2.60	2.52	106.5	7.8	105.9	78.2	88.8	5.4	2.5	35.4	Minimum value
	2.77	2.75	134.8	13.4	137.3	122.4	118.1	5.4	8.4	39.4	Maximum value
	36	3	7		39				2		Total number of tests
	2.68	2.18	116.6	12.5	110.1	84.9	112.0	12.7	6.6	33.6	Average of all values
	0.06	0.11	8.9	3.4	8.7	7.9	11.1	5.4	5.6	5.7	Standard deviation
SM	2.51	2.24	92.9	6.8	88.5	61.6	91.1	1.6	0.2	23.3	Minimum value
	3.11	2.63	132.6	25.5	122.9	97.1	132.5	25.0	21.2	45.0	Maximum value
	149	9	123		21				17		Total number of tests
	2.69	2.17	118.9	12.4	—	—	115.6	14.2	5.0	33.9	Average of all values
	0.04	0.18	5.9	2.3	—	—	14.1	5.7	2.5	2.9	Standard deviation
SC	2.56	2.17	104.3	6.7	—	—	91.1	7.5	0.7	28.4	Minimum value
	2.81	2.59	131.7	18.2	—	—	131.8	22.7	8.5	38.3	Maximum value
	88	4	73		0				10		Total number of tests
	2.69	—	103.3	19.7	—	—	98.9	22.1	3.6	34.0	Average of all values
	0.09	—	10.4	5.7	—	—	11.5	8.9	4.3	3.1	Standard deviation
ML	2.52	—	81.6	10.6	—	—	80.7	11.1	0.1	25.2	Minimum value
	3.10	—	126.0	34.6	—	—	119.3	40.3	11.9	37.7	Maximum value
	65	0	39		0				14		Total number of tests
	2.71	2.59	109.3	16.7	—	—	106.5	17.7	10.3	25.1	Average of all values
	0.05	0.13	5.5	2.9	—	—	7.8	5.1	7.6	7.0	Standard deviation
CL	2.56	2.42	90.0	6.4	—	—	85.6	11.6	0.9	8.0	Minimum value
	2.87	2.75	121.4	29.2	—	—	118.7	35.0	23.8	33.8	Maximum value
	270	3	221		0				31		Total number of tests
	2.79	—	85.1	33.6	—	—	—	—	—	—	Average of all values
	0.25	—	2.3	1.6	—	—	—	—	—	—	Standard deviation
MH	2.47	—	82.9	31.5	—	—	—	—	—	—	Minimum value
	3.50	—	89.0	35.5	—	—	—	—	—	—	Maximum value
	10	0	5		0				0		Total number of tests

Reference 1

Table 5-1.—Average engineering properties of compacted soils. From the Western United States. Last updated October 6, 1982. —Continued.

USCS soil type	Specific gravity		Compaction				Shear strength				Values listed
			Laboratory		Index unit weight		Avg. placement		Effective stress		
	No. 4 minus	No. 4 plus	Maximum unit weight, lb/ft³	Optimum moisture content, %	Max., lb/ft³	Min., lb/ft³	Unit weight, lb/ft³	Moisture, content, %	c′ lb/in²	φ′, degrees	
	2.73	—	95.3	25.0	—	—	93.6	25.7	11.5	16.8	Average of all values
	0.06	—	6.6	5.4	—	—	8.1	5.7	7.4	7.2	Standard deviation
CH	2.51	—	82.3	16.6	—	—	79.3	17.9	1.5	4.0	Minimum value
	2.89	—	107.3	41.8	—	—	104.9	35.3	21.5	27.5	Maximum value
	74	0	36		0			12			Total number of tests

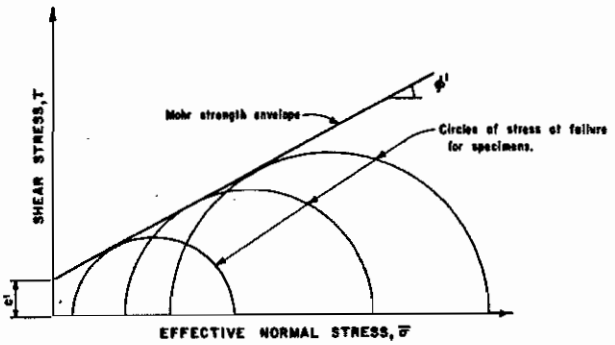


Figure 5-13.—Shear strength of compacted soils. 288-D-2474.

the soil mass can be determined under controlled conditions that will represent a property of the soil.

In 1856, H. Darcy showed experimentally that the rate of flow of water, q, through a soil specimen of cross-sectional area A was directly proportional to the imposed hydraulic gradient ($i = \Delta h/L$) or $q = kiA$. The coefficient of proportionality, k, has been called "Darcy's coefficient of permeability," "coefficient of permeability" (also referred to as hydraulic conductivity) or "permeability." Permeability is the soil property that indicates the ease with which water will flow through the soil. The use of k in estimating flow through soils is discussed in section 6.9(b). Many units of measurement are commonly used for expressing the coefficient of permeability. The units used on figure 5-14 are feet per year (or cubic feet per square foot per year at unit gradient). One foot per year is virtually equal to 10^{-6} cm/s.

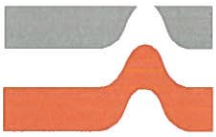
Permeability in some soils is very sensitive to small changes in unit weight, water content, or gradation. Because of the possible wide variation in permeability, the numerical value of k should be considered only as an order of magnitude. It is customary in the Bureau of Reclamation to describe soils with permeabilities less than 1 ft/yr as impervious; those with permeabilities between 1 and 100 ft/yr as semipervious; and soils with permeabilities greater than 100 ft/yr as pervious. These values, however, are not absolute for the design of dams. Successful structures have been built whose various zones were constructed of soils with permeabilities not within these respective ranges.

Reference 2

Excerpts from Ardaman & Associates Report (2007)

Excerpts from Ardaman & Associates, Inc. Report (2007)

SUBSURFACE SOIL EXPLORATION
LEE/HENDRY LANDFILL
HENDRY COUNTY, FLORIDA



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American Concrete Institute
American Society for Testing and Materials
Florida Institute of Consulting Engineers

Excerpts from Ardaman & Associates, Inc. Report (2007)

LABORATORY ANALYSIS

One-dimensional consolidation tests were performed on plastic clay specimens obtained from the Shelby tube samples retrieved from the test borings. The tests were performed in accordance with ASTM D 2435. This test method subjects a soil specimen to increasing loads and incrementally monitoring the change in height. The data is presented on an e-log p curve, which plots void ratio (e) as a function of the log of time pressure (p). The parameters necessary for settlement calculation can then be derived from these curves: compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c) and initial void ratio (e_o). A separate plot of the change in sample height versus log time for each load increment allows calculation of the coefficient of consolidation (c_v) and the coefficient of secondary compression (c_α) so that the ratio of primary settlement and amount of secondary compression may be predetermined. A summary of the data is presented below and on the e-log p curve in the Appendix.

Dry Unit Weight and Moisture Content Used to Calculate "Wet Unit Density"

SPT Boring Location	Sample No.	Depth (ft.)	Dry Unit Weight, (pcf)	Moisture Content, %	Percent Fines	Liquid Limit	Plasticity Index	USCS Classification	Remarks
Ash	U-1	24 - 26	93.0	27.4	-	-	-	SC	-
Ash	U-2	34 - 36	51.0	85.4	-	-	-	CL	-
Ash	U-3	40 - 42	88.8	32.4	35.9	-	-	SC	-
C&D	U-1	19 - 21	102.8	23.4	49.4	-	-	SC	-
C&D	U-2	30 - 32	53.4	79.8	91.5	168	124	CH	Consol
C&D	U-3	45 - 47	46.7	95.6	99.1	173	122	CH	Consol

Boring Location	
SPT C&	
SPT C&	

2007 Ardaman & Associates, Inc. Test Results

Sample	Depth (feet)	Dry Unit Weight (pcf)	MC (%)	Wet Unit Density (pcf)
Ash-1	24-26	93	27.4	118
Ash-2	34-36	51	85.4	94.6
Ash-3	40-42	88.8	32.4	118
Avg.		77.6	48.4	110

Ash sp

Undrained (CU) triaxial shear strength tests. The tests were performed in accordance with ASTM D 4767. In this test the specimen is allowed to consolidate under the confining pressure prior to shear, but no drainage is permitted during shear. A minimum of three tests at different confining pressures is required to derive the stress parameters. Pore pressure measurements are taken during testing so that the effective stress parameters can be derived. The results of the CU tests as derived from the p'-q plots are summarized below.

Excerpts from Ardaman & Associates, Inc. Report (2007)

Sample	Sample No.	Depth (ft.)	Friction Angle, Ø (deg)	Cohesion, c (tsf)
Clay	SPT C&D U-2	30 - 32	21.0	0.64
Clay	SPT C&D U-3	45 - 47	20.3	0.52
ASH	Bulk	-	44.3	0

GENERAL COMMENTS

The analysis in this report are based on the data obtained from the twenty-two (22) boring locations as directed by Jones Edmunds and Associates, Inc. personnel.

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and may be encountered. The boring logs and related information are based on the driller's logs and visual examination of selected sample in the laboratory.

The delineation between soil types shown on the logs is approximate and the description represents our interpretation of subsurface conditions at the designated boring locations and on the particular date drilled.

If you have any questions about this report, please contact this office.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.

Martin A. Call, P.E.
Senior Project Engineer

Gary A. Drew, P.E.
Branch Manager/Vice President

Reference 3

Duncan and Wright (2005)
Soil Strength and Slope Stability,
Figure 5.21

Reference 3

Recapitulation

- The drained shear strengths of sands, gravels, and rockfill materials can be expressed as $s = \sigma' \tan \phi'$.
- Values of ϕ' for these materials are controlled by density, gradation, and confining pressure.
- The variation of ϕ' with confining pressure can be represented by

$$\phi' = \phi_0 - \Delta\phi \log_{10} \frac{\sigma_3'}{p_a}$$

  where σ_3' is the confining pressure and p_a is atmospheric pressure.
- When large particles are removed to prepare specimens for laboratory tests, the test specimens should be prepared at the same relative density as the original material, not the same void ratio.
- Values of ϕ' for granular materials can be estimated based on the Unified Soil Classification, relative density, and confining pressure.
- Values of ϕ' for granular materials can also be estimated based on results of standard penetration tests or cone penetration tests.

Table 5.2 Values of ϕ_0 and $\Delta\phi$ for Sands and Gravels

Unified classification	Standard Proctor RC[a] (%)	Relative density, D_r[b] (%)	ϕ_0[c] (deg)	$\Delta\phi$ (deg)
GW, SW	105	100	46	10
	100	75	43	8
	95	50	40	6
	90	25	37	4
GP, SP	105	100	42	9
	100	75	39	7
	95	50	36	5
	90	25	33	3
SM	100	—	36	8
	95	—	34	6
	90	—	32	4
	85	—	30	2

Drainage sand

Soil 1&2

Source: Wong and Duncan (1974).

[a]RC = relative compaction = $\gamma_d / \gamma_{d\,\max} \times 100\%$.

[b]$D_r = (e_{\max} - e)/e_{\max} - e_{\min}) \times 100\%$.

[c]$\phi' = \phi_0 - \Delta\phi \log_{10} \sigma_3'/p_a$ where p_a is atmospheric pressure.

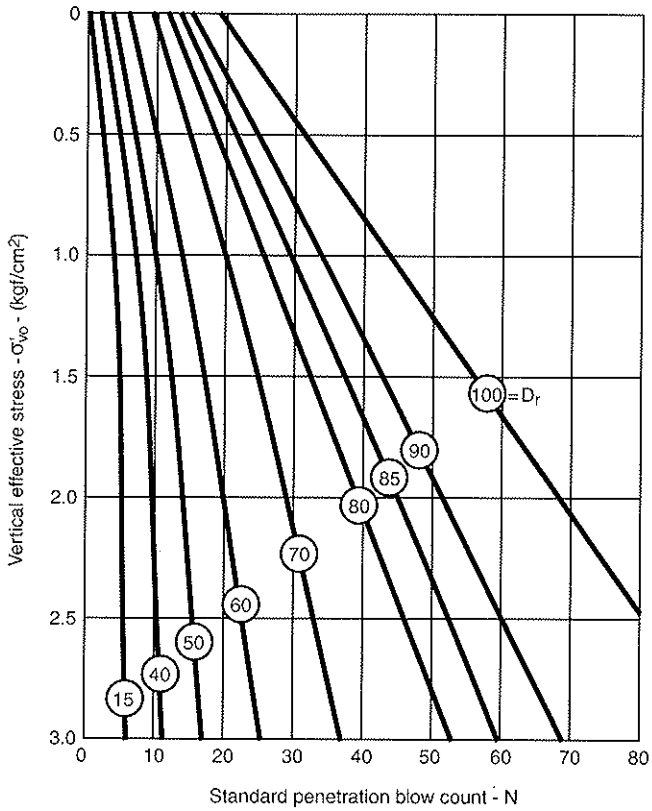

Figure 5.5 Relationship among SPT blow count, overburden pressure, and relative density for sands. (After Gibbs and Holtz, 1957, and U.S. Dept. Interior, 1974.)

SILTS

The shear strength of silts in terms of effective stress can be expressed by the Mohr–Coulomb strength criterion as

$$s = c' + \sigma' \tan \phi' \qquad (5.5)$$

where s is the shear strength, c' the effective stress cohesion intercept, and ϕ' the effective stress angle of internal friction.

The behavior of silts has not been studied as extensively and is not as well understood as the behavior of granular materials or clays. Although the strengths of silts are governed by the same principles as the strengths of other soils, the range of their behavior is wide, and sufficient data are not available to anticipate or estimate their properties with the same degree of reliability as is possible in the case of granular soils or clays.

Silts encompass a broad range of behavior, from behavior that is very similar to the behavior of fine sands

Source: Soil Strength and Slope Stability, 2005, Duncan and Wright, John Wiley & Sons, Inc., Hoboken, New Jersey

Reference 3

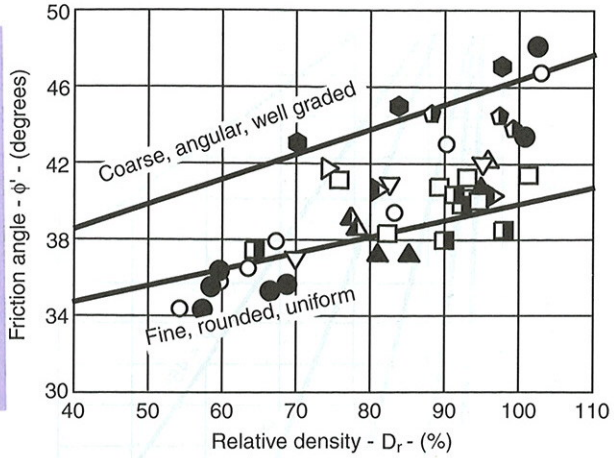

Figure 5.9 Correlation between friction angle and relative density for sands. (Data from Schmertmann, 1975, and Lunne and Kleven, 1982.)

Table 5.3 Relationship Among Relative Density, SPT Blow Count, and Angle of Internal Friction for Clean Sands

State of packing	Relative density, D_r (%)	SPT blow count, N^a (blows/ft)	Angle of internal friction ϕ'^b (deg)
Very loose	< 20	< 4	< 30
Loose	20–40	4–10	30–35
Compact	40–60	10–30	35–40
Dense	60–80	30–50	40–45
Very dense	> 80	> 50	> 45

Source: Meyerhof (1956).

[a]$N = 15 + (N' - 15)/2$ for $N' > 15$ in saturated very fine or silty sand, where N is the blow count corrected for dynamic pore pressure effects during the SPT, and N' is the measured blow count.

[b]Reduce ϕ' by 5° for clayey sand; increase ϕ' by 5° for gravelly sand.

= undisturbed strength/remoulded strength), they are very easily disturbed. In a study of a silt from the Alaskan arctic (Fleming and Duncan, 1990), it was found that disturbance reduced the undrained strengths measured in unconsolidated–undrained tests by as much as 40%, and increased the undrained strengths measured in consolidated–undrained tests by as much as 40%. Although silts can usually be sampled using the same

Table 5.4 Correlation Among Relative Density, CPT Cone Resistance, and Angle of Internal Friction for Clean Sands

State of packing	Relative density, D_r (%)	q_c (tons/ft² or kgf/cm²)	ϕ' (deg)
Very loose	< 20	< 20	< 32
Loose	20–40	20–50	32–35
Medium	40–60	50–150	35–38
Dense	60–80	150–250	38–41
Very dense	> 80	250–400	41–45

Source: Meyerhof (1976).

techniques as those used for clays, the quality of samples should not be expected to be as good.

Cavitation

Unlike clays, nonplastic silts almost always tend to dilate when sheared, even if they are normally consolidated. In undrained tests, pore pressures decrease as a result of this tendency to dilate, and pore pressures can become negative. When pore pressures are negative, dissolved air or gas may come out of solution, forming bubbles within test specimens that greatly affect their behavior.

Figure 5.10 shows stress–strain and pore pressure–strain curves for consolidated–undrained triaxial tests on nonplastic silt from the Yazoo River valley. As the specimens were loaded, they tended to dilate, and the pore pressures decreased. As the pore pressures decreased, the effective confining pressures increased. The effective stresses stopped increasing when cavitation occurred, because from that point on the volume of the specimens increased as the cavitation bubbles expanded. The value of the maximum deviator stress for each sample was determined by the initial pore pressure (the back pressure), which determined how much negative change in pore pressure took place before cavitation occurred. The higher the back pressure, the greater was the undrained strength. These effects can be noted in Figure 5.10.

Drained or Undrained Strength?

Values of c_v for nonplastic silts are often in the range 100 to 10,000 cm²/h (1000 to 100,000 ft²/yr). It is often difficult to determine whether silts will be drained or undrained under field loading conditions,

Source: Soil Strength and Slope Stability, 2005, Duncan and Wright, John Wiley & Sons, Inc., Hoboken, New Jersey

Reference 3

Undrained strengths of compacted clays. Values of c and ϕ (total stress shear strength parameters) for the as-compacted condition can be determined by performing UU triaxial tests on specimens at their compaction water contents. Undrained strength envelopes for compacted, partially saturated clays tested are curved, as discussed in Chapter 3. Over a given range of stresses, however, a curved strength envelope can be approximated by a straight line and can be characterized in terms of c and ϕ. When this is done, it is especially important that the range of pressures used in the tests correspond to the range of pressures in the field conditions being evaluated. Alternatively, if the computer program used accommodates nonlinear strength envelopes, the strength test data can be represented directly.

Values of total stress c and ϕ for compacted clays vary with compaction water content and density. An example is shown in Figure 5.20 for compacted Pittsburgh sandy clay. The range of confining pressures used in these tests was 1.0 to 6.0 tons/ft^2. The value of c, the total stress cohesion intercept from UU tests, increases with dry density but is not much affected by compaction water content. The value of ϕ, the total stress friction angle, decreases as compaction water content increases, but is not so strongly affected by dry density.

If compacted clays are allowed to age prior to testing, they become stronger, apparently due to thixotropic effects. Therefore, undrained strengths measured using freshly compacted laboratory test specimens provide a conservative estimate of the strength of the fill a few weeks or months after compaction.

MUNICIPAL SOLID WASTE

Waste materials have strengths comparable to the strengths of soils. Strengths of waste materials vary depending on the amounts of soil and sludge in the waste, as compared to the amounts of plastic and other materials that tend to interlock and provide tensile strength (Eid et al., 2000). Larger amounts of materials that interlock increase the strength of the waste. Although solid waste tends to decompose or degrade with time, Kavazanjian (2001) indicates that the strength after degradation is similar to the strength before degradation.

Kavazanjian et al. (1995) used laboratory test data and back analysis of stable slopes to develop the lower-bound strength envelope for municipal solid waste shown in Figure 5.21. The envelope is horizontal with a constant strength $c = 24$ kPa, $\phi = 0$ at normal pressures less than 37 kPa. At pressures greater than 37 kPa, the envelope is inclined at $\phi = 33°$ with $c = 0$.

Eid et al. (2000) used results of large-scale direct shear tests (300 to 1500 mm shear boxes) and back analysis of failed slopes in waste to develop the range of strength envelopes show in Figure 5.22. All three envelopes (lower bound, average, and upper bound) are inclined at $\phi = 35°$. The average envelope shown in Figure 5.22 corresponds to $c = 25$ kPa, and the lowest of the envelopes corresponds to $c = 0$.

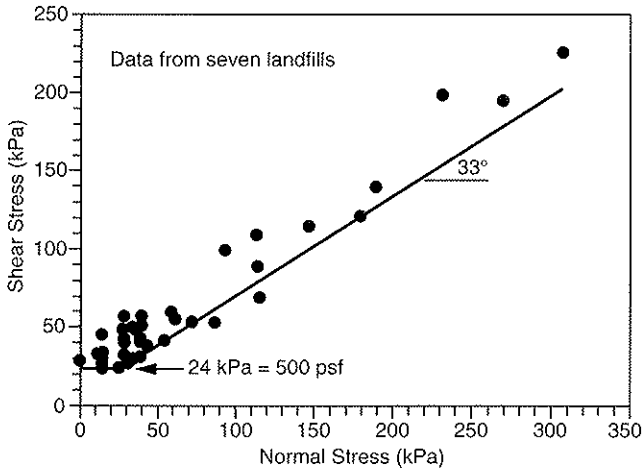

Figure 5.21 Shear strength envelope for municipal solid waste based on large-scale direct shear tests and back analysis of stable slopes. (After Kavazanjian et al., 1995.)

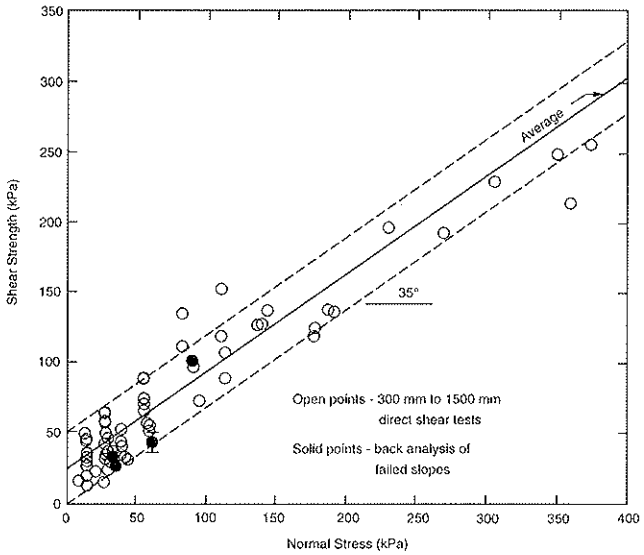

Figure 5.22 Range of shear strength envelopes for municipal solid waste based on large-scale direct shear tests and back analysis of failed slopes. (After Eid et al., 2000.)

Source: Soil Strength and Slope Stability, 2005, Duncan and Wright, John Wiley & Sons, Inc., Hoboken, New Jersey

Reference 3

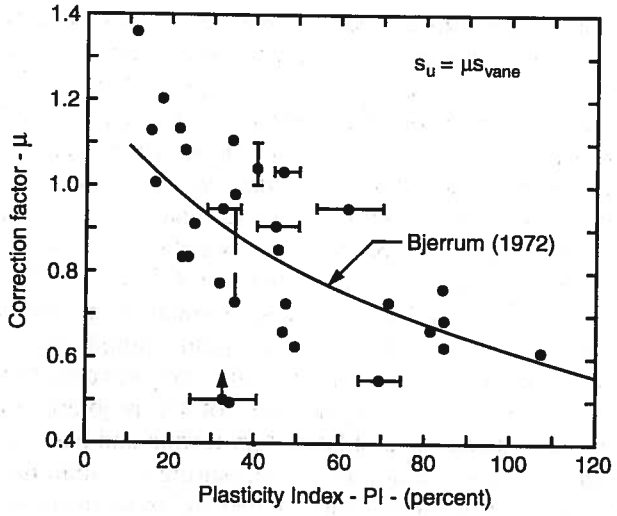

Figure 5.14 Variation of vane shear correction factor and plasticity index. (After Ladd et al., 1977.)

Cone penetration tests. Cone penetration tests (CPTs) are attractive as a means of evaluating undrained strengths of clays in situ because they can be performed quickly and at lower cost than field vane shear tests. The relationship between undrained strength and cone tip resistance is

$$s_u = \frac{q_c - \sigma_{vo}}{N_k^*} \qquad (5.9)$$

where s_u is the undrained shear strength, q_c the CPT tip resistance, σ_v the total overburden pressure at the test depth, and N_k^* the cone factor. The units for s_u, q_c, and σ_v in Eq. (5.9) must be the same.

Values of the cone factor N_k^* for a number of different clays are shown in Figure 5.15. These values were developed by comparing *corrected* vane strengths with cone penetration resistance. Therefore Eq. (5.9) provides values of s_u comparable to values determined from field vane shear tests *after correction*. It can be seen that there is little systematic variation of N_k^* with the plasticity index. A value of $N_k^* = 14 \pm 5$ is applicable to clays with any PI value.

A combination of field vane shear and CPT tests can often be used to good advantage to evaluate undrained strengths at soft clay sites. A few vane shear tests are performed close to CPT test locations, and a site-specific value of N_k^* is determined by comparing the results. The cone test is then used for production testing.

Standard Penetration Tests. Undrained strengths can be estimated very crudely based on the results of Standard Penetration Tests. Figure 5.16, which shows

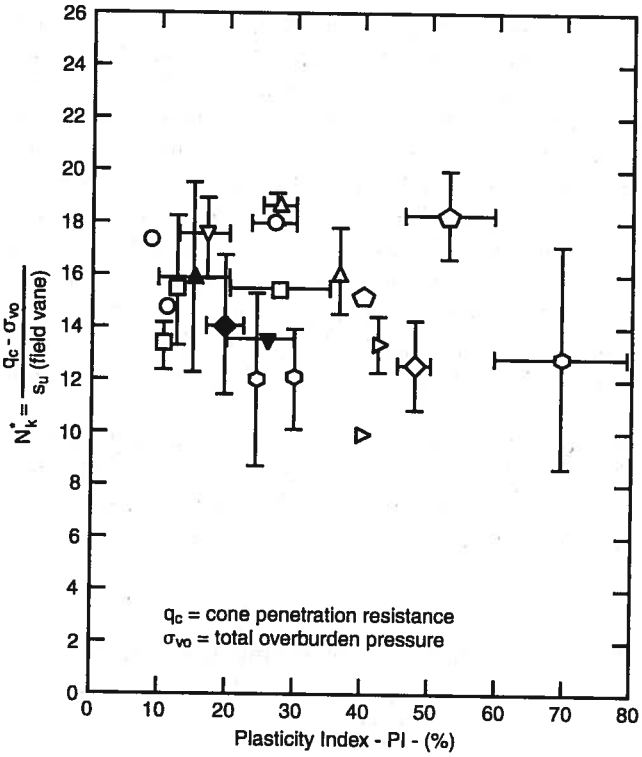

Figure 5.15 Variation of the ratio of net cone resistance ($q_c - \sigma_{vo}$) divided by vane shear strength (s_u vane) with plasticity index for clays. (After Lunne and Klevan, 1982.)

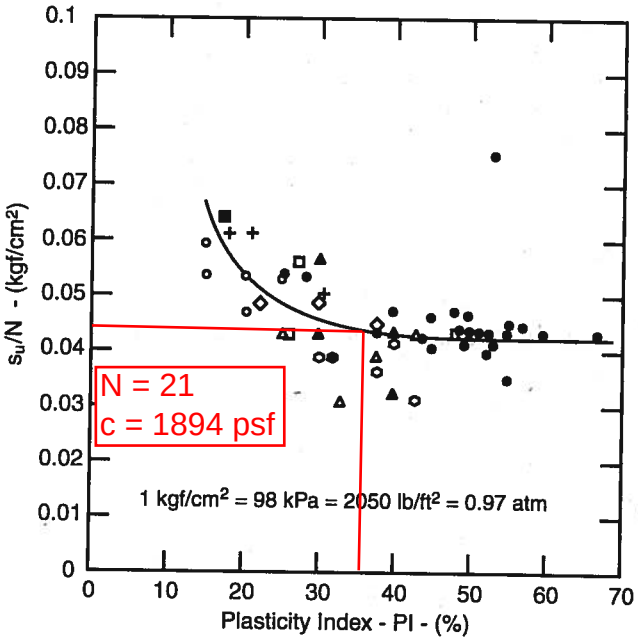

Figure 5.16 Variation of the ratio of undrained shear strength (s_u) divided by SPT blow count (N), with plasticity index for clay. (After Terzaghi et al., 1996.)

Source: Soil Strength and Slope Stability, 2005, Duncan and Wright, John Wiley & Sons, Inc., Hoboken, New Jersey

Reference 4

Bowles (1996)
Foundation Analysis and Design - Table 5-6

Reference 4

FOUNDATION ANALYSIS AND DESIGN

Fifth Edition

Joseph E. Bowles, P.E., S.E.
Consulting Engineer/Software Consultant
Engineering Computer Software
Peoria, Illinois

The McGraw-Hill Companies, Inc.

New York St. Louis San Francisco Auckland Bogotá Caracas
Lisbon London Madrid Mexico City Milan Montreal New Delhi
San Juan Singapore Sydney Tokyo Toronto

TABLE 2-7
Values or value ranges for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	−0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15
Steel	0.33

0.30 Soil 1, 2
0.045 Soil 3, 4
0.40 Soil 5 (weathered LS)

Another material property concept is the *bulk modulus* E_b, which is defined as the ratio of hydrostatic stress to the volumetric strain ϵ_v and is given as

$$E_b = \frac{2}{3}G'\frac{1+\mu}{1-2\mu} = \frac{E_s}{3(1-2\mu)} \qquad (f)$$

For an *elastic* material the shear modulus G' cannot be (−), so Eq. (*a*) sets the lower limit of $\mu > -1$. Equation (*f*) sets the upper limit at $\mu < 0.5$. It appears that the range of μ for soils (that are not "elastic") is from about −0.1 to 1.00. Table 2-7 gives a range of values for select materials. It is very common to use the following values for soils:

μ	Soil type
0.4–0.5	Most clay soils
0.45–0.50	Saturated clay soils
0.3–0.4	Cohesionless—medium and dense
0.2–0.35	Cohesionless—loose to medium

Although it is common to use $\mu = 0.5$ for saturated clay soils, the reader should be aware that this represents a condition of no volume change under the applied stress σ_z. Over time, however, volume change does occur as the pore fluid drains. Equation (*e*) defines the Poisson's ratio that develops initially ($\epsilon_v = 0$) and also later when $\epsilon_v > 0$. Since the strain is produced from stress and Fig. 1-1 indicates a vertical variation, it necessarily follows that μ is stress-dependent from Eq. (*e*).

A special case in geotechnical work is that of *plane strain.* This arises where strains occur parallel to two of the coordinate axes (say the x and z) but the strain is zero perpendicular to the x-z plane (along the y axis). If we set $\epsilon_y = 0$ in the set of equations for Hooke's law [(Eqs. (2-64)] and solve for the resulting values of E_s and μ, we obtain the following:

$$E_s' = \frac{E_s}{1-\mu^2} \qquad \mu' = \frac{\mu}{1-\mu} \qquad (2\text{-}65)$$

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$	$E_s = (2 \text{ to } 4)q_u$
	$= 7000\sqrt{N}$	$= 8000\sqrt{q_c}$
	$= 6000N$	— — —
	— — —	$E_s = 1.2(3D_r^2 + 2)q_c$
	‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	*$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$
		$e = 1.0 \quad F = 3.5$
		$e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$	$E_s = (6 \text{ to } 30)q_c$
	$E_{s(\text{OCR})} \approx E_{s,\text{nc}}\sqrt{\text{OCR}}$	
Gravelly sand	$E_s = 1200(N + 6)$	
	$= 600(N + 6) \quad N \le 15$	
	$= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use	$^{\$}E_s' = 2.5q_c$
	$2500 < q_c < 5000$ use	$E_s' = 4q_c + 5000$
	where	
	E_s' = constrained modulus $= \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

4. It is not easy to determine if a cohesionless deposit is overconsolidated or what the OCR might be. Cementation may be less difficult to discover, particularly if during drilling or excavation sand "lumps" are present. Carefully done consolidation tests will aid in obtaining the OCR of cohesive deposits as noted in Chap. 2.

In general, with an OCR > 1 you should carefully ascertain the site conditions that will prevail at the time settlement becomes the design concern. This evaluation is, of course, true for any site, but particularly so if OCR > 1.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem

Reference 5

Terzaghi, Peck, and Mesri (1996)
Soil Mechanics in Engineering Practice

Reference 5

Soil Mechanics in Engineering Practice

Third Edition

Karl Terzaghi
Late Professor of the Practice of Civil Engineering
Harvard University
Lecturer and Research Consultant in Civil Engineering
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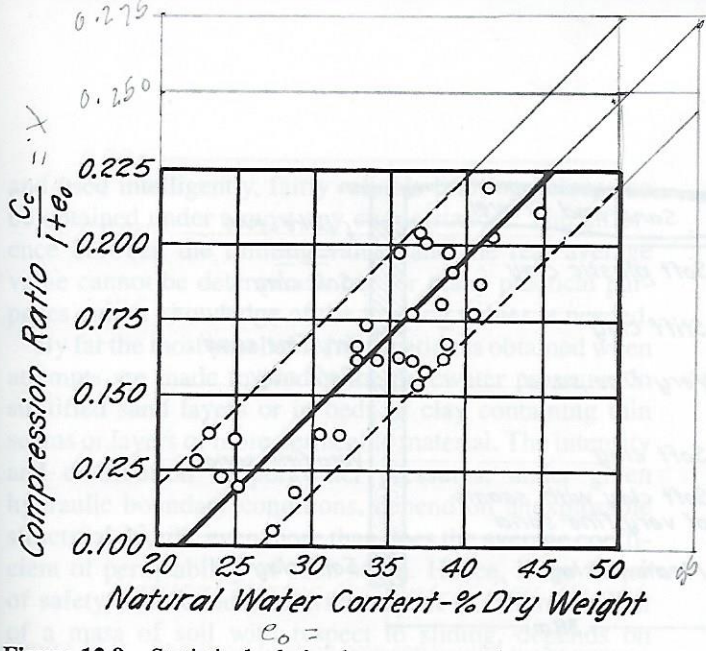


Figure 12.9 Statistical relation between natural water content and compression ratio for samples of clay from boring in Boston, MA (after Fadum 1941).

one representative boring is selected. Near this boring an undisturbed sample boring is made that furnishes samples of at least *B* quality to be submitted to consolidation tests.

Because of the great amount of time and labor involved in the performance of consolidation tests (Article 16.9), these tests cannot be made on most jobs on more than 10 or 15 samples without undue delay. Yet, even in relatively homogeneous clay strata the physical properties of the clay are likely to change to a considerable extent from point to point. Consequently, the compressibility characteristics of a clay can be determined at a reasonable expense only on the basis of statistical relationships between the compressibility and the index properties of the clay.

Finally, on very large and expensive jobs, such as the construction of airport runways on deep fills over thick compressible sediments in bays or estuaries, or the construction on sensitive clays of dams to retain reservoirs at a given elevation, reliable forecasts of long-term settlements are required. The supplementary information to be obtained for such forecasts is described in Article 16.

Figure 52.15 illustrates the degree of accuracy that can be achieved in computing the distribution of the settlement over the base area of a building located above fairly homogeneous clay strata. The real distribution is shown on the left-hand side, and the computed on the right. The structure itself is complex but symmetrical. No such results can be obtained if the profile of the subsoil of a building is erratic, because the settlement of buildings on such subsoils depends not only on the intensity and the distribution of the loads but also on the variations in the compressibility of the subsoil in horizontal directions. In addition, the rate of settlement can change from place to place, because it depends on the degree of continuity of layers and pockets of cohesionless material present in

the subsoil. Figure 12.10 illustrates such a profile. It represents the results of test borings in a composite shore deposit located at the south shore of Lake Erie. More than 100 borings were made, and the spacing between drill holes did not exceed 30 m. Yet the borings records did not indicate whether or not the layers of clay encountered in the drill holes were continuous.

If the profile of the subsoil of a proposed structure is erratic an extensive program of undisturbed sample borings and elaborate soil investigations is entirely out of place. Information of far more practical value can be obtained by means of numerous subsurface soundings supplemented by exploratory drill holes. The results of such investigations inform the designer of the location of the softest and the most resistant spots beneath the base of the building. At the location of the softest clay, an undisturbed sample boring may be made to obtain information on the details of stratification and on the compressibility of the clay that was encountered in the exploratory holes. The maximum settlement is estimated on the basis of the procedures described in Article 41. In judging whether the proposed structure can stand the estimated settlement, the spacing between the hardest and the softest spots of the subsoil is taken into consideration. The results of even very elaborate soil investigations would hardly add anything to the information obtained by the recommended procedure.

12.8 Summary of Procedures in Subsoil Reconnaissance

According to the preceding discussions, a subsoil reconnaissance involves several successive stages. The first step is to decide on the depth and spacing of the exploratory borings.

If the proposed structure is a building, current practice calls for about one drill hole for every 250 m^2 of the area covered by the building. If a retaining wall is to be constructed or an open cut to be made, it is customary to make at least one drill hole for every 30 m of length of the wall or the cut. However, these rules are based on convention rather than rational considerations. If the subsoil is erratic, more useful information can usually be obtained in less time and at a smaller expense by combining the exploratory borings with subsurface soundings.

The depth to which the exploratory drill holes should be made depends on whether the subsoil may contain layers of soft clay. If the local geological conditions or the conditions revealed by earlier test borings in the vicinity exclude this possibility, the drill holes do not need to be made to a depth of more than 5 or 10 m below subgrade. On the other hand, if the subsoil may contain layers of soft clay at an unknown depth, a sound decision regarding the minimum depth of the test borings can be made only on the basis of the results of a rough estimate of the maximum depth at which the presence of clay strata may

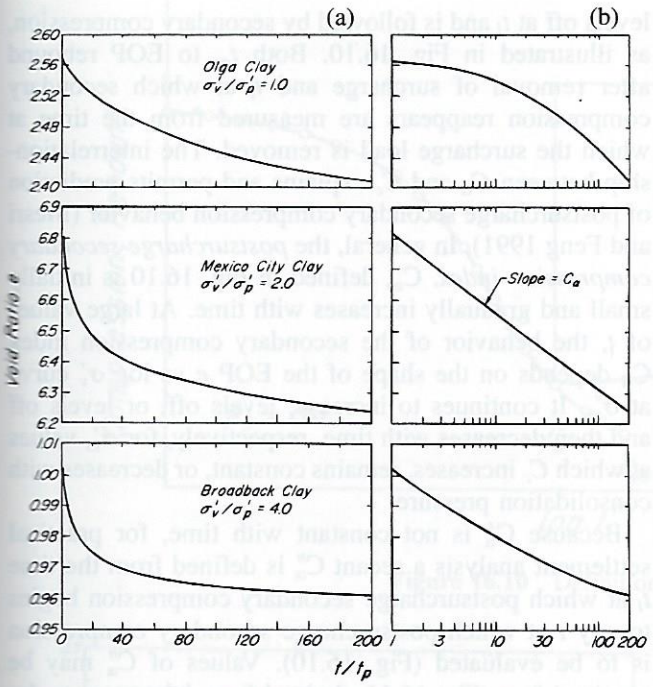


Figure 16.8 Examples of secondary compression for natural clay.

$$S = \frac{\Delta e}{1 + e_o} L_o \tag{16.10}$$

The value of Δe between $t/t_p = 1$ and any required time can be read directly from the e vs t/t_p curve corresponding to the particular final pressure σ'_{vf}. Often, however, it is preferable to evaluate the secondary settlement in terms of compressibility parameters. It is then convenient to plot secondary compression against the logarithm of time, as in Fig. 16.8*b*. The *secondary compression index*, $C_\alpha = \Delta e/\Delta \log t$, defines the slope of the e vs log t curve in the secondary compression range. For a constant C_α between t_p and t, Eq. 16.10 is rewritten as

$$S = \frac{C_\alpha}{1 + e_o} L_o \log \frac{t}{t_p} \tag{16.11}$$

The magnitude of secondary settlement depends not only on the compressibility parameter C_α, but also on the value of t/t_p. In the laboratory the value of t/t_p can be very large, mainly because the duration of the primary consolidation stage for laboratory samples is generally small. On the other hand, in the field the values of t_p for layers of soil can range from several months to many years, and for the typical useful life of a structure the value of t/t_p rarely exceeds 100 and is often less than 10. Exceptions are encountered, however, in clays containing permeable layers, in peats, in certain residual soils with high initial permeability, and in deposits in which vertical drains are installed (Article 25.5) to accelerate primary consolidation. In any event, as t/t_p decreases, the secondary settlement becomes less and less significant.

Because the interparticle mechanisms responsible for compression over time are the continuation of the response of the soil to an increase in effective stress, it is reasonable to expect the magnitudes of the compressibilities with respect to effective stress and to time to be related. Indeed, for any soil, it is found that at any (σ'_v, t) during secondary compression the ratio C_α/C_c is a constant in both the recompression and compression ranges (Mesri and Godlewski 1977). This is illustrated in Fig. 16.9 for undisturbed specimens of Brown Mexico City clay with w_o = 313 to 340%, w_l = 361%, w_p = 91%, and σ'_p/σ'_{vo} = 1.4. The value of $C_c = \Delta e/\Delta \log \sigma'_v$ in the ratio C_α/C_c expresses the variable slope of the e vs log σ'_v curve in both recompression and compression; moreover, in general, $C_\alpha = \Delta e/\Delta \log t$ does not remain constant with time. Hence, the secondary compression index C_α may remain constant, decrease, or increase with time in the range of consolidation pressure in which C_c remains constant, decreases, or increases with σ'_v, respectively (Mesri and Godlewski 1977, Mesri and Castro 1987). For a fundamental understanding of the compressibility of soils, which is essential for developing theories of consolidation and for interpreting field settlement observations and laboratory consolidation data, an appreciation of the behavior of C_α with time is needed. However, because in the field the value of t/t_p is often small, C_α can often be assumed to be independent of time for settlement analysis.

Values of C_α/C_c are shown in Table 16.1. For all geotechnical materials the total range is 0.01 to 0.07; the midpoint of the range, 0.04, is also the most common value for inorganic clays and silts. The value of C_α in the early stages of secondary compression, for example

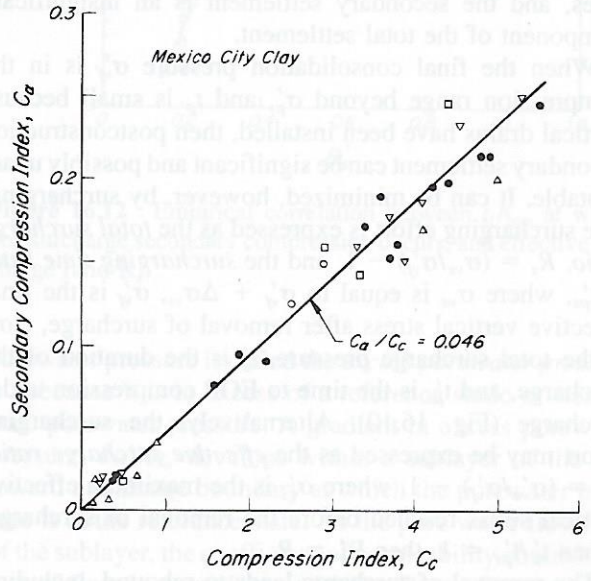
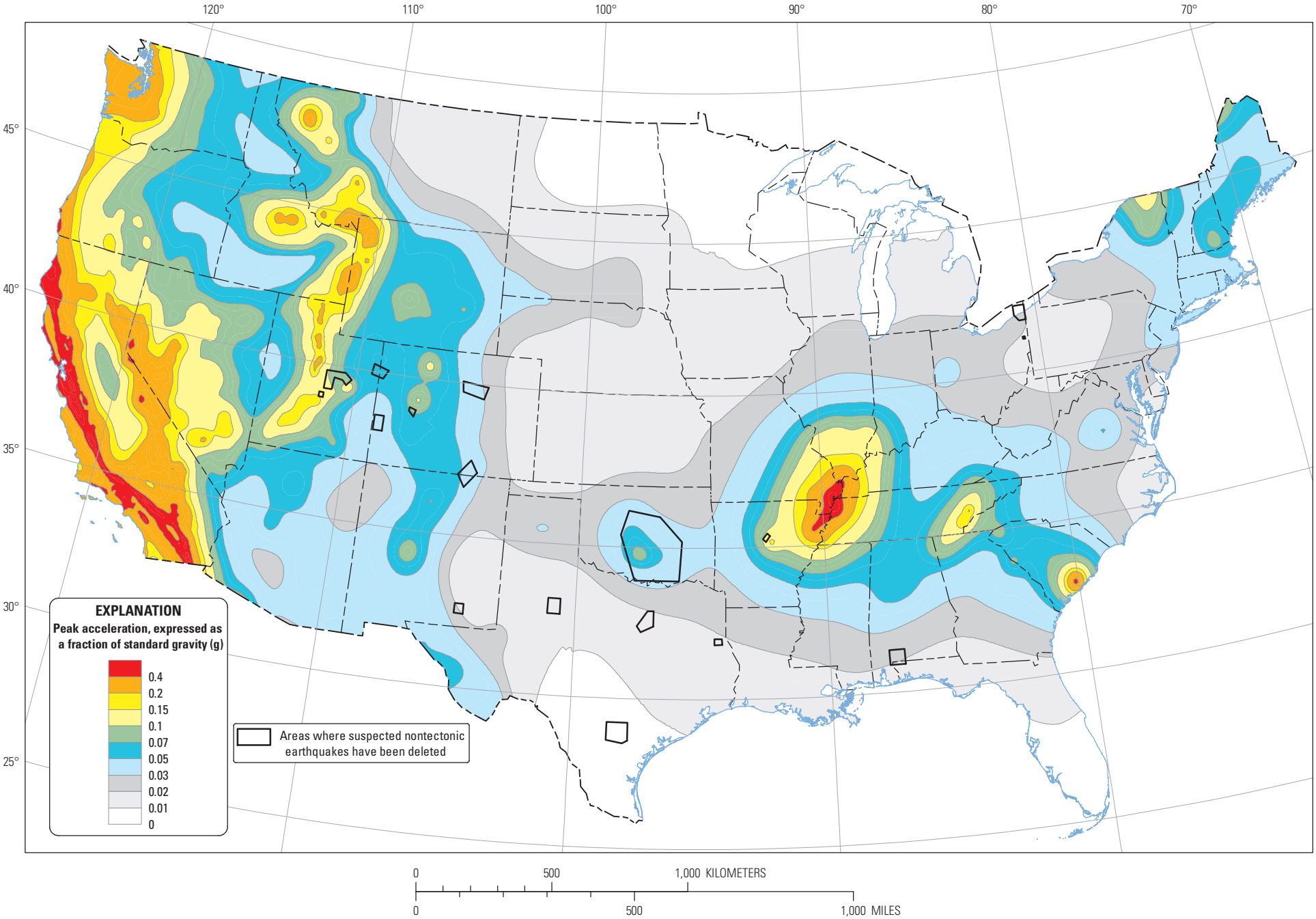


Figure 16.9 An example of relation between C_α and C_c.

Attachment I.1.d

USGS Seismic Zones Map



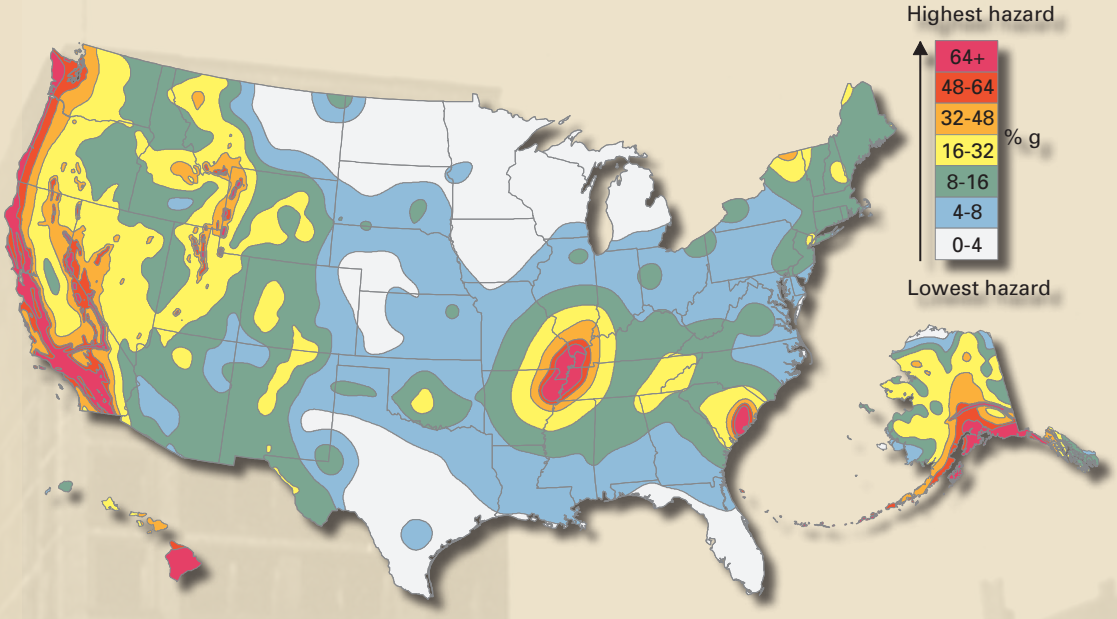
Ten-percent probability of exceedance in 50 years map of peak ground acceleration

USGS
science for a changing world

nehrp

United States National Seismic Hazard Maps

The U.S. Geological Survey's maps of earthquake shaking hazards provide information essential to creating and updating the seismic design provisions of building codes and insurance rates used in the United States. Periodic revisions of these maps incorporate the results of new research. Buildings, bridges, highways, and utilities built to meet modern seismic design provisions are better able to withstand earthquakes, not only saving lives but also enabling critical activities to continue with less disruption. These maps can also help people assess the hazard to their homes or places of work and can also inform insurance rates.



*The 2008 U.S. Geological Survey National Seismic Hazard Maps for the United States are based on current information about the rate at which earthquakes occur in different areas and on how far strong shaking extends from earthquake sources. Colors on this particular map show the levels of horizontal shaking that have a 2-in-100 chance of being exceeded in a 50-year period. Shaking is expressed as a percentage of **g** (**g** is the acceleration of a falling object due to gravity).*

America's first line of defense against earthquakes is the construction of buildings that can withstand severe shaking. Cities and counties rely on the seismic design provisions in building codes to ensure that structures can resist the earthquakes that are probable in their region. The variations in the seismic threat across the country are depicted on maps in building codes as zones of different risk levels. These building code maps are based on more detailed maps, collectively known as the National Seismic Hazard Maps, prepared by the U.S. Geological Survey (USGS) with input from scientists and engineers at universities, State geological surveys, and the private sector.

The National Seismic Hazard Maps incorporate a wealth of geologic and seismologic information used to estimate the shaking, or ground motion, from earthquakes. In particular, the maps incorporate estimates of the magnitudes and locations of all likely earthquakes, how often these earthquakes occur, and the strength of ground shaking that they cause. The maps are time-independent, as they portray a long-term average hazard and are not affected by the time of the last earthquake rupture. This attribute makes the maps well suited for use in building codes, where the long-term life of a structure and its resilience to earthquake shaking must be considered.

The National Seismic Hazard Maps are revised periodically by the USGS to include new research results on earthquake faults, crustal deformation, and earthquake ground shaking. The timing of the updates is designed to coincide with the schedule for updates of model building codes. During the update process, USGS solicits recommendations for improvements from the earth science and engineering communities, prepares draft maps that are vetted regionally, and solicits both public and expert opinion in review of the maps before they are finalized. Producing these maps is a key responsibility of the USGS in the four-agency, congressionally established National Earthquake Hazards Reduction Program (NEHRP). The NEHRP agencies work cooperatively with appropriate national building codes and standards organizations to ensure that the most up to date and technically sound earthquake engineering design technology is available for use around the Nation. The four agencies—Federal Emergency Management Agency (FEMA), National Institute of Standards and Technology (NIST), National Science Foundation (NSF), and USGS—view the implementation of building codes as a central purpose of this partnership and the best means to ensure that buildings and other structures in local communities are adequately protected against natural hazards.

U.S. Department of the Interior
U.S. Geological Survey

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Fact Sheet 2008–3017
April 2008

Seismic hazard maps have many other applications. For example, the 2002 USGS National Seismic Hazard Maps are used as the basis of seismic design requirements for highway bridges nationwide. Such maps are also used by:

- Insurance companies to set property rates in various areas of the country.
- Federal Emergency Management Agency (FEMA) to plan the allocation of assistance funds for education and preparedness and improve risk assessments.
- Risk and loss modeling companies to estimate potential losses to a region or to a portfolio of assets.
- Civil engineers to estimate the stability and landslide potential of hillsides, and the liquefaction potential of strata underlying communities.
- U.S. Environmental Protection Agency to set construction standards that help ensure the safety of waste-disposal facilities.

A new series of National Seismic Hazard Maps was publicly released by the USGS in 2008. Each of these maps shows the severity of expected earthquake shaking for a particular level of probability. For example, the map on the front page shows levels of earthquake shaking that have a 2-in-100 chance of being exceeded in a 50-year period. The time period of 50 years is commonly used because it represents a typical building lifetime, while the 2-percent probability level is usually considered an acceptable hazard level for the building codes. This new series of maps also depicts shaking using a number of different measures that engineers can readily apply to designing earthquake-resistant buildings of different heights, which respond to different frequencies of ground motion.

The updated National Seismic Hazard Maps incorporate newly published seismic, geologic, and geodetic models that define the best available science on earthquake rates and associated ground shaking. These ground-motion models show the decay in ground shaking with distance from the earthquake source (attenuation), and incorporate the effects of varying soil type and faulting style. The updated attenuation models incorporate many recent earthquake strong-motion records and generally result in lower estimates of ground shaking than previous models. New fault studies, geologic maps, reflection and refraction surveys, and geodetic strain rate models provide insights into earthquake sizes and recurrence rates for crustal and subduction sources. This information was discussed with working groups, State geological surveys, Federal agencies, and other earth-science experts from industry and academia in revising the input models. The new ground-motion and earthquake source models result in similar or lower predicted ground motion across most of the United States compared to motion calculated in earlier versions of the hazard maps.

Through the continuing process of refining estimates of the shaking and earthquake hazard across the Nation and observing successes and failures in building design, scientists and engineers are laying the groundwork for communities to remain safer during future earthquakes. By improving the resilience to earthquake shaking of homes, office buildings, hospitals, highways, dams, and utilities, scientists and engineers also help to ensure that those communities are able to recover rapidly following a major temblor.

Charleston, South Carolina, Earthquake August 31, 1886. The worst earthquake wreck in Charleston. (USGS photo library). Photograph by J.K. Hillers.

Building the Model Building Codes: The Role of Seismic Hazard Maps

There is no single mandatory national building code or standard for designing and constructing buildings and other structures in the United States. Building codes are adopted and enforced by individual States and by local jurisdictions within the States. State or local building codes are usually based on a "model" building code or other voluntary consensus standards. State and local governments may either directly adopt the model code or adopt the code with amendments. Today, the model building code that is predominantly adopted in the United States is the International Building Code (IBC), promulgated by the International Code Council (ICC), a non-governmental organization. The ICC also promulgates other model codes that impact the building process, such as the International Residential Code (IRC). Some localities have also adopted the National Fire Protection Association (NFPA) 5000 Building Construction and Safety Code.

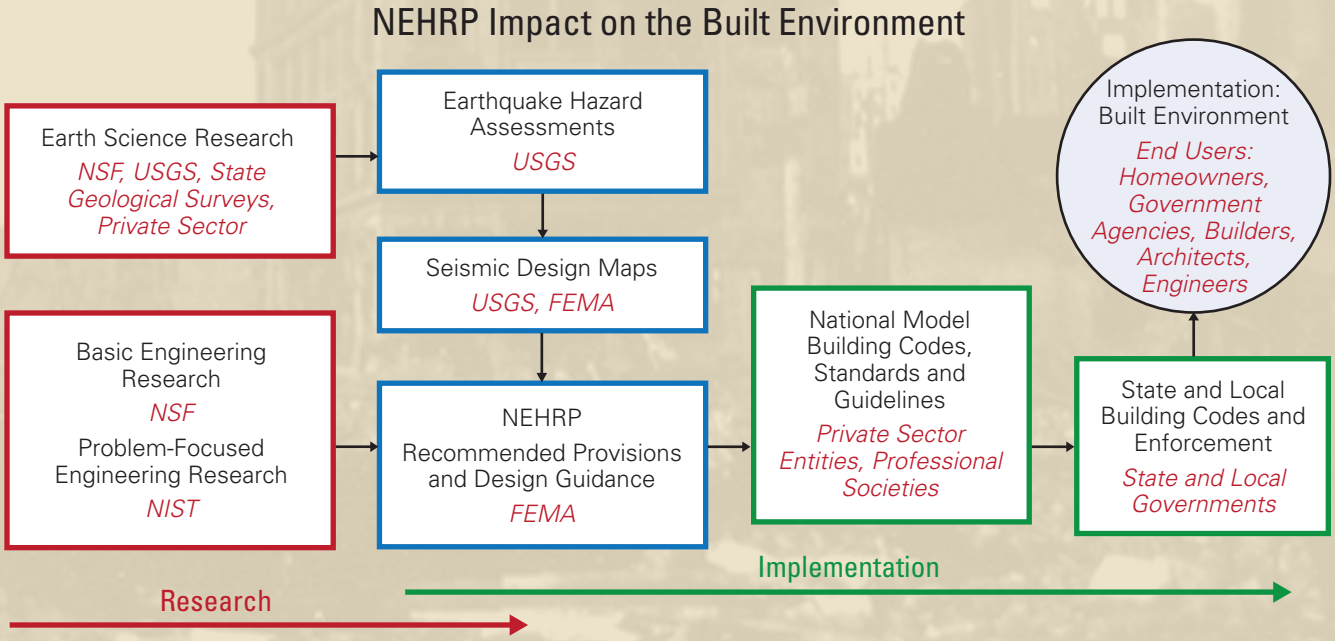
The IBC and its companion ICC documents form the basis of the building codes in 46 States and have been adopted by local governments within the remaining 4 States; thus, State and local building codes adopted throughout the United States share a common origin. As States and localities have based their codes upon the model codes, the performance of buildings constructed in accordance with the newer building codes has improved.

The four-agency National Earthquake Hazards Reduction Program (NEHRP) supports the development of seismic provisions in building codes by maintaining the quality of four documents that build on one another:

- The *NEHRP Recommended Provision for Seismic Regulations for New Buildings and Other Structures*. This document presents the state of the art of earthquake engineering research and practice in a form usable by the engineering community, and provides a nationally applicable resource document for all model codes and standards. FEMA leads the effort to maintain this document, which since its inception in 1985 has been updated on a 3- to 5-year recurring cycle. USGS directly supports the update process by providing appropriate updated seismic ground motion maps and soil/site condition parameters that incorporate the latest seismological findings. USGS also prepares seismic design maps based on the hazard levels and other modifications recommended by technical committees participating in the update process. Appropriate basic (NSF-sponsored) and applied (NIST-sponsored) engineering research results are included in the updates. The document also serves the needs of the Federal Government as a baseline for executive orders related to Federal building design and construction.
- ASCE 7. The American Society of Civil Engineers (ASCE) supports the development on a 5-year cycle of the *Minimum Design Loads for Buildings and Other Structures*, known as ASCE 7. FEMA and USGS work with ASCE to ensure that ASCE 7 adequately addresses seismic loads and design in a manner that is "substantially equivalent" to guidelines recommended by the *NEHRP Recommended Provisions*.
- The International Codes and the NFPA 5000. Most State and local jurisdictions use the IBC and the IRC as the basis for their building code laws and regulations; a few local jurisdictions have adopted NFPA 5000. The IBC and NFPA 5000 adopt ASCE 7 by reference for most seismic design requirements, and the IRC incorporates NEHRP-based seismic requirements directly.

Cooperating Agencies and Organizations

American Assoc. of State Hwy. Transportation Officials
American National Standards Institute
American Society of Civil Engineers
Applied Technology Council
Building Officials and Code Administrators International
Building Seismic Safety Council
State Geological Surveys
California Earthquake Authority
Environmental Protection Agency
Federal Emergency Management Agency
International Conference of Building Officials
Mid-America Earthquake Center
National Science Foundation
National Center for Earthquake Engineering Research
National Institute of Standards and Technology
National Oceanic and Atmospheric Administration
Pacific Earthquake Engineering Research Center
Southern Building Code Congress
Southern California Earthquake Center
Structural Engineers Association of California
Western States Seismic Policy Council



Flow chart of the Building Code Process. USGS provides both basic research and hazard information that is critical to development of seismic provisions in building codes. USGS works with other NEHRP agencies (FEMA, NIST, NSF) to ensure the most up to date and technically sound earthquake engineering design technology is available for use around the Nation.

USGS Earthquake Hazard Assessment Products and Tools

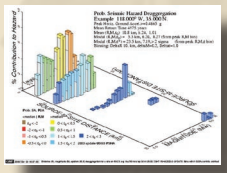
U.S. National Probabilistic Ground-Motion Maps

This Web site contains maps and associated input/output data and documentation for probabilistic hazard maps for the 48 Conterminous States, Alaska, Hawaii, and Puerto Rico.

Seismic Design Values for Buildings

This Web site allows users to determine design ground motion at a site for various building codes, using latitude/longitude or Zip code. In addition, one can display and download a hazard curve or uniform hazard spectrum for a site.

Hazard Mapping and Analysis Tools



USGS offers a number of on-line Web tools that allow an individual to assess which sources pose the greatest hazard in a particular region, look up hazard values using latitude/longitude or Zip code, make customized USGS probabilistic hazard maps for an area of interest, map probability of a given magnitude within a certain distance from a site, and access computer software for seismic hazard analysis.

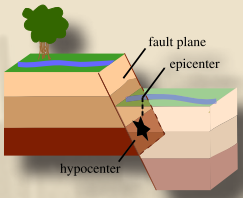
U.S. Quaternary Fault and Fold Database

This Web site contains information on faults and associated folds in the United States that are believed to be sources of M>6 earthquakes during the Quaternary (the past 1,600,000 years). Maps of these geologic structures are linked to detailed descriptions and references.

Urban Probabilistic Ground-Motion Maps

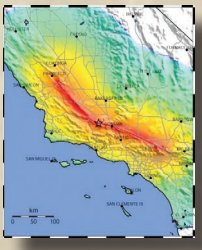
These maps show more detailed probabilistic seismic hazard maps for urban areas that take into account near-surface geologic conditions, sedimentary basin structure, and directivity effects.

Earthquake Hazards 101



USGS provides a wealth of explanatory materials for the layperson, including information on concepts behind earthquake maps, the use of probability, what the maps mean, how they are made, and answers to frequently asked questions.

Deterministic or Scenario Ground-Motion Maps



Maps predict the median level of ground shaking from a particular "scenario" event. They do not take into account the likelihood of that scenario occurring, but they are helpful when assessing the potential impact of a particular event.

Time-Dependent Earthquake Probability Maps

These maps forecast the likelihood of an earthquake rupture occurring during an interval of time in the future. Time-dependent maps are considered research tools and are not currently applied in building codes.

For Further Information

To access these products and tools and for further information, see the Web site for the U.S. Geological Survey Earth Hazards Program at URL *http://earthquake.usgs.gov/hazmaps/,* and visit *http://www.nehrp.gov* or e-mail info@nehrp.gov. You may also contact Mark Petersen: mpetersen@usgs.gov.

Attachment I.1.e(1)

Bearing Capacity Analysis

TECHNICAL MEMORANDUM

West Pasco County Class I Ash Monofill Landfill Slope Modification and Vertical Expansion

TO: Project File

FROM: Carol Sawyer, PE

DATE: June 22, 2021

SUBJECT: Attachment I.1.e(1) – Bearing Capacity Analysis
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

This Technical Memorandum summarizes the bearing capacity analysis for the West Pasco Class I Landfill (Ash Monofill) Slope Modification. This memorandum documents the methodology, design parameters, and analysis results to demonstrate that the foundation soils are capable to support the proposed expansion and meet the requirements of Rules 62-701.400(2) and 62-701.410(3)(e), FAC.

2 METHODOLOGY

The ultimate bearing capacity of the soil is calculated to evaluate the adequacy of the foundation to support the proposed loads. Typical bearing capacity calculations based on Terzaghi's equations are applicable to small footings or mat foundations for buildings. These equations have been modified for modeling large embankment loads. Such equations are, however, typically not applicable to large area loading such as stockpiling or landfilling.

A more accurate analysis for the foundation bearing capacity of a large embankment (as in the case of a landfill) is an evaluation of deep rotational foundation failure through the peak elevation of the landfill. A slope stability analysis was performed that modeled the rotational failure plane through the foundation soils. This model is comparable to a bearing capacity failure.

The bearing capacity analysis was performed using Geo-Slope International's SLOPE/W software, which uses limit equilibrium methods to compute the factor of safety (FS) of earth and rock slopes from sliding down under the influence of gravity. Our analysis uses the Spencer method, which formulates moment and force equilibrium factors of safety.

2.1 PARAMETERS

Attachment I.1.a(6) outlines and describes how the bearing capacity analysis was performed using soil and waste strength parameters. The analysis is based on undrained conditions. The groundwater table is modeled at the SHGWT at elevation 36 feet.

2.2 BEARING CAPACITY ANALYSIS

One critical cross-section was chosen based on its peak elevation and steepest bottom liner slope (Attachment I.1, Figure I-3 – Section A-A').

A circular (rotational) failure surface analysis was performed with the failure surface through the foundation soils. This analysis determines the stability of the waste mass and foundation soils and identifies the failure plane with the lowest factor of safety. The resulting failure surface is through the weathered and fragmented limestone layer. This analysis provides a conservative analysis of the potential global foundation stability at the maximum landfill height and 3H:1V side slopes.

The analysis results are attached to this memorandum.

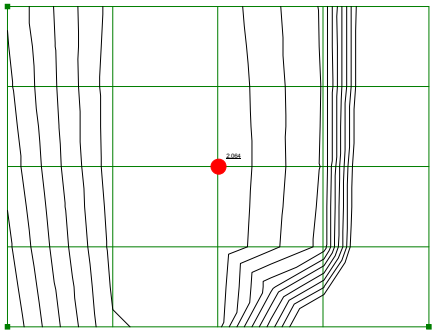
3 RESULTS

The results of the bearing capacity analysis and analysis report are attached. The results indicate that the landfill design is sufficient for providing foundation bearing capacity stability to meet the required minimum factors of safety (FS >1.5). The final closure analysis through the critical cross section and based on the given material strength parameters meets the required minimum factor of safety of 1.5 for landfill slopes. This meets the requirements of Rule 62-701.400(2), FAC, "designs to prevent failures of side slopes, and designs to prevent deep-seated failures through the waste, along liner systems, and through foundation soils, shall achieve a minimum factor of safety of 1.5 using peak strength values."

Date: 05/03/2021
Title: Pasco - Ash Monofill Vertical Exp
File Name: CellA-4-SlopeStability.gsz

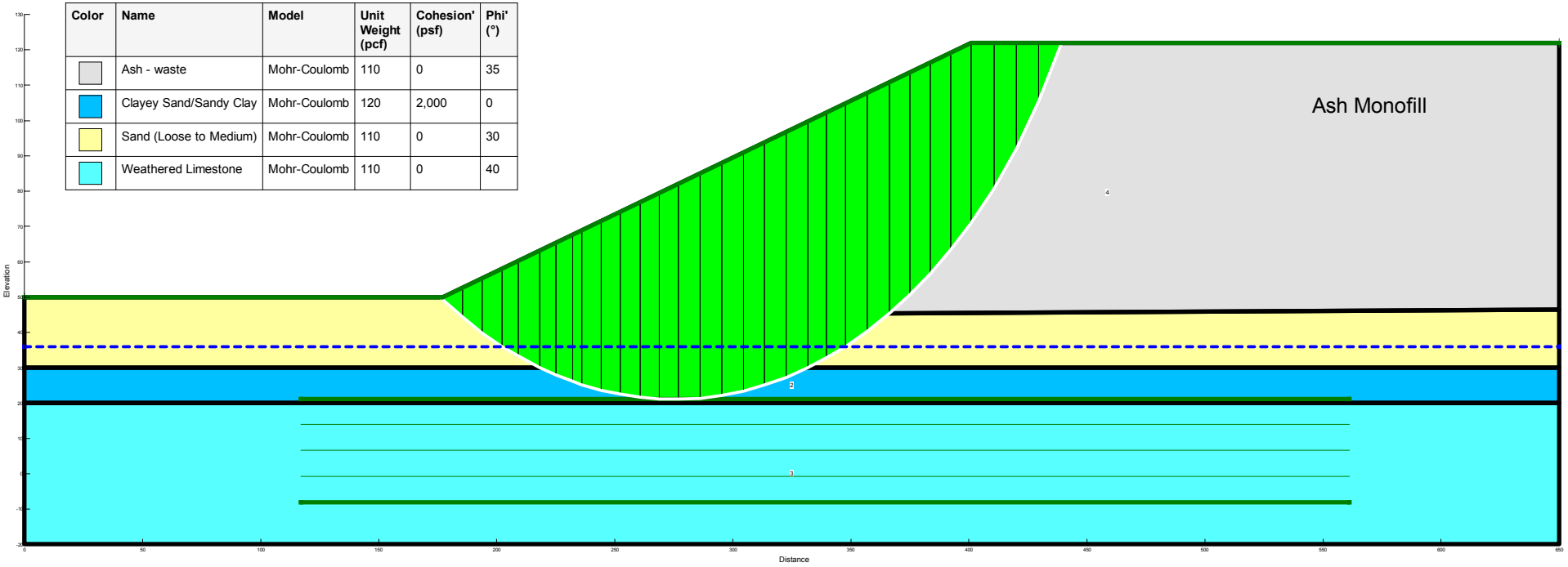
JE Project No.: 16002-009-01

Notes:
1. Prpoed vertical expansion to elevation 122 feet with modified slopes of 3H:1V.
2. Section through Cell A-4 (west side).
3. SHWT at elevation 36 feet.
4. Subsurface profile based on 2021 borings by PSI and historical borings.



FS= 2.06

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Ash - waste	Mohr-Coulomb	110	0	35
	Clayey Sand/Sandy Clay	Mohr-Coulomb	120	2,000	0
	Sand (Loose to Medium)	Mohr-Coulomb	110	0	30
	Weathered Limestone	Mohr-Coulomb	110	0	40



SLOPE/W Analysis

Report generated using GeoStudio 2019. Copyright © 1991-2018 GEOSLOPE International Ltd.

File Information

File Version: 10.00
Title: Pasco - Ash Monofill Vertical Exp
Created By: Carol G. Sawyer, PE
Last Edited By: Carol G. Sawyer, PE
Revision Number: 25
Date: 05/03/2021
Time: 01:12:53 PM
Tool Version: 10.0.0.17401
File Name: CellA-4-SlopeStability.gsz
Directory: Y:\16002-PascoCounty\Projects\009-01-AshMonofillOperationsModification\Reports\Task 2 - Preliminary Analysis\Slope Stability\
Last Solved Date: 05/04/2021
Last Solved Time: 01:20:45 PM

Project Settings

Unit System: U.S. Customary Units

Analysis Settings

SLOPE/W Analysis

Description: Pasco - Ash Monofill Vertical Expansion
Kind: SLOPE/W
Method: Morgenstern-Price
Settings
- Side Function
  - Interslice force function option: Half-Sine
- PWP Conditions from: Piezometric Line
  - Apply Phreatic Correction: No
  - Use Staged Rapid Drawdown: No
- Unit Weight of Water: 62.430189 pcf

Slip Surface
- Direction of movement: Right to Left
- Use Passive Mode: No
- Slip Surface Option: Grid and Radius
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack Option: (none)

Distribution
- F of S Calculation Option: Constant

Advanced
- Geometry Settings
  - Minimum Slip Surface Depth: 0.1 ft
  - Number of Slices: 30
- Factor of Safety Convergence Settings
  - Maximum Number of Iterations: 100

Tolerable difference in F of S: 0.001
Solution Settings
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Ash - waste

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 35 °
Phi-B: 0 °

Sand (Loose to Medium)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 30 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Clayey Sand/Sandy Clay

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 2,000 psf
Phi': 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Weathered Limestone

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 40 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Grid

Upper Left: (201.9806, 242.02827) ft
Lower Left: (201.9806, 167.93158) ft
Lower Right: (348.01348, 167.93158) ft
Grid Horizontal Increment: 4
Grid Vertical Increment: 4

Slip Surface Radius

Upper Left Coordinate: (117, 21.03832) ft
Upper Right Coordinate: (561, 21.03832) ft
Lower Left Coordinate: (117, -8.08311) ft
Lower Right Coordinate: (561, -8.08311) ft
Number of Increments: 4
Use Left Projection: No
Left Projection Angle: 135 °
Use Right Projection: No
Right Projection Angle: 45 °

Slip Surface Limits

Left Coordinate: (0, 50) ft
Right Coordinate: (650, 122) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X	Y
Coordinate 1	0 ft	36 ft
Coordinate 2	650 ft	36 ft

Points

	X	Y
Point 1	0 ft	50 ft
Point 2	177 ft	50 ft
Point 3	401 ft	122 ft
Point 4	650 ft	122 ft
Point 5	209 ft	43 ft
Point 6	236 ft	43 ft
Point 7	277 ft	45 ft
Point 8	650 ft	46.5 ft
Point 9	0 ft	30 ft
Point 10	650 ft	30 ft
Point 11	0 ft	20 ft
Point 12	650 ft	20 ft
Point 13	0 ft	-20 ft
Point 14	650 ft	-20 ft
Point 15	232 ft	43 ft

Regions

	Material	Points	Area
Region 1	Sand (Loose to Medium)	1,2,5,15,6,7,8,10,9	10,868 ft²
Region 2	Clayey Sand/Sandy Clay	9,10,12,11	6,500 ft²

Region 3	Weathered Limestone	11,12,14,13	26,000 ft²
Region 4	Ash - waste	2,3,4,8,7,15,5	28,120 ft²

Slip Results

Slip Surfaces Analysed: 110 of 125 converged

Current Slip Surface

Slip Surface: 61
Factor of Safety: 2.064
Volume: 12,764.579 ft³
Weight: 1,410,436.1 lbf
Resisting Moment: 1.2129369e+08 lbf·ft
Activating Moment: 58,749,021 lbf·ft
Resisting Force: 579,938.02 lbf
Activating Force: 281,006.87 lbf
Slip Rank: 1 of 125 slip surfaces
Exit: (175.92264, 50) ft
Entry: (439.15808, 122) ft
Radius: 183.94161 ft
Center: (274.99704, 204.97993) ft

Slip Slices

	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength	Suction Strength	Base Material
Slice 1	176.46132 ft	49.658262 ft	-852.68789 psf	45.870655 psf	26.483435 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 2	181.2219 ft	46.81341 ft	-675.08324 psf	618.81672 psf	357.274 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 3	189.66569 ft	42.098502 ft	-380.73066 psf	1,647.7486 psf	951.3281 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 4	198.10948 ft	37.943354 ft	-121.32399 psf	2,617.6847 psf	1,511.321 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 5	205.66569 ft	34.642853 ft	84.726936 psf	3,415.1741 psf	1,922.8346 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 6	213.64125 ft	31.642853 ft	272.0175 psf	4,166.9387 psf	2,248.7338 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 7	221.71187 ft	28.96184 ft	439.39365 psf	4,747.31 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 8	228.57062 ft	27.028975 ft	560.06281 psf	5,244.3603 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay

Slice 9	234 ft	25.676976 ft	644.46836 psf	5,592.8507 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 10	240.1 ft	24.427238 ft	722.4897 psf	5,937.6771 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 11	248.3 ft	23.033202 ft	809.51968 psf	6,354.4499 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 12	256.5 ft	22.01711 ft	872.95447 psf	6,692.9837 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 13	264.7 ft	21.372675 ft	913.18664 psf	6,955.8369 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 14	272.9 ft	21.095983 ft	930.46062 psf	7,146.9956 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 15	281.5593 ft	21.212036 ft	923.21539 psf	7,272.9992 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 16	290.6779 ft	21.765064 ft	888.68974 psf	7,332.2548 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 17	299.79649 ft	22.775839 ft	825.58684 psf	7,324.6258 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 18	308.91509 ft	24.252062 ft	733.42597 psf	7,257.9124 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 19	318.03369 ft	26.205293 ft	611.48538 psf	7,139.1397 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 20	327.15229 ft	28.65143 ft	458.77263 psf	6,974.138 psf	0 psf	2,000 psf	0 psf	Clayey Sand/Sandy Clay
Slice 21	335.69937 ft	31.394515 ft	287.52128 psf	6,717.2512 psf	3,712.2063 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 22	343.67493 ft	34.394515 ft	100.23072 psf	6,479.7179 psf	3,683.1986 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 23	352.34887 ft	38.173074 ft	-135.6654 psf	6,191.9962 psf	3,574.9506 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 24	361.72121 ft	42.852847 ft	-427.82455 psf	5,857.7066 psf	3,381.9485 psf	0 psf	0 psf	Sand (Loose to Medium)
Slice 25	370.73145 ft	47.996534 ft	0 psf	5,397.1591 psf	3,779.1315 psf	0 psf	0 psf	Ash - waste
Slice 26	379.37961 ft	53.615451 ft	0 psf	5,011.3859 psf	3,509.0102 psf	0 psf	0 psf	Ash - waste
			0 psf			0 psf	0 psf	Ash - waste

Slice 27	388.02776 ft	59.967816 ft		4,593.5217 psf	3,216.4185 psf			
Slice 28	396.67592 ft	67.155789 ft	0 psf	4,136.6986 psf	2,896.5475 psf	0 psf	0 psf	Ash - waste
Slice 29	405.76976 ft	75.801972 ft	0 psf	3,480.3382 psf	2,436.959 psf	0 psf	0 psf	Ash - waste
Slice 30	415.30928 ft	86.268275 ft	0 psf	2,612.9005 psf	1,829.5726 psf	0 psf	0 psf	Ash - waste
Slice 31	424.8488 ft	98.626515 ft	0 psf	1,652.1374 psf	1,156.839 psf	0 psf	0 psf	Ash - waste
Slice 32	434.38832 ft	113.67355 ft	0 psf	560.38929 psf	392.38881 psf	0 psf	0 psf	Ash - waste

Attachment I.1.e(2)

Settlement Analysis Calculations

Reference 1	**Geotechnical Aspects of Landfill Design & Construction, Qian, Koerner, & Gray, 2002**
Reference 2	**Foundation Analysis and Design, Bowles, 5th Edition, 1996**
Reference 3	**Test Data from Landfilled WTE Ash, Ardaman & Associates, 2007**
Reference 4	**FDOT Soils and Foundation Handbook, 2018 (Table 1)**

TECHNICAL MEMORANDUM

Pasco County West Pasco Ash Monofill Slope Modification and Vertical Expansion

TO: Project File

FROM: Chad Spreadbury, PhD, EI

XC: Carol Sawyer, PE

DATE: June 22, 2021

SUBJECT: Attachment I.1.e(2) – Subgrade Settlement Analysis
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

This Technical Memorandum summarizes the subgrade settlement analysis for the Ash Monofill slope modification and vertical expansion. This memorandum documents the methodology, design parameters, and analysis results to demonstrate that the bottom liner slopes after settlement will meet the requirements of FDEP Rule 62-701.400(4)(c), FAC, after closure of the landfill.

2 METHODOLOGY

Foundation settlement is the vertical deflection of the foundation soil as a result of the landfill overburden load. The settlement of the foundation soil is calculated to evaluate the adequacy of the foundation to support the proposed loads while maintaining appropriate leachate collection system drainage. Typical settlement calculations based on elastic and consolidation equations are applicable for modeling large area loading such as stockpiling or landfilling.

Total settlement of the foundation soils was calculated based on determining the changes in stress imposed on the subsurface soil due to the Ash Monofill. The bottom liner grades were determined from as-built documents. Construction of the landfill includes installing the bottom liner system, filling the waste, and placing the final cover. The final cover was designed with 3H:1V side slopes.

The analysis included calculating the total settlement at key points within the landfill footprint (i.e., along the leachate collection drainage paths) to determine final slope of the bottom liner after settlement and thus the effectiveness of the leachate collection system for long-term operation. The attached calculations provide the foundation settlement analysis

including the stress conditions, total settlement, differential settlement, and bottom liner slopes after settlement.

The following steps were taken to calculate the total settlement within the landfill footprint and the slope of the bottom liner after settlement:

- Establish cross-sections along and perpendicular to the leachate collection trenches.
- Establish critical settlement point locations along each cross-section.
- Establish the subsurface profile at each settlement point based on the closest boring.
- Establish the initial stresses at each settlement point due to existing soils.
- Establish the final stress at each settlement point due to the landfill construction/vertical expansion.
- Calculate the total settlement at each settlement point.
- Calculate the change in bottom liner slope after settlement.

Based on the historical and most recent borings, the general subsurface profile consists of varying thicknesses (2 to 60 feet) of loose-to-medium-dense sand, overlying varying thickness (5 to 10 feet) of primarily medium-to-stiff clayey sand, sandy/silty clay, and clay over weathered limestone. The top of the limestone varied from approximately 20 to 45 feet below grade.

2.1 Establish Cross Sections

Three cross-sections were used for this analysis: A-A', C-C', and D-D'. Section A-A' (west-east) runs along the primary leachate collection trench in Cell A-4; Section C-C' (south-north) runs through Cells A-3, A-2, and A-1, and Section D-D' (south-north) runs through the peak elevation of the landfill in Cell A-4. The cross-sections are included in Attachment I.1.

2.2 Establish Settlement Points

The settlement point locations were chosen along each cross-section and correspond to a peak cover elevation or a change in the bottom liner grade. The settlement points are shown on the figures in Attachment I-1.

2.3 Establish Subsurface Profile at Each Settlement Point

The soil profiles at each settlement point are based on the nearest boring information representative of the soil lithology. The soil layers were established based on the soil classification type, SPT blow count (N) data, and laboratory analysis.

2.4 Establish the Initial Stress Due to Existing Soils

The initial stresses (σ_o) on the soils before construction of the landfill were calculated at the mid-point of each soil layer by multiplying the unit weight of the soil layer (γ_T) by the thickness of the soil layer (H_0) above the water table and using the buoyant unit weight ($\gamma_T - \gamma_{water}$) below the water table. The stresses of each consecutive soil layer are added to calculate a cumulative stress at the center point of each layer. To calculate the potential

maximum settlement, the seasonal high water table (SHWT) is assumed to be at elevation 36 feet National Geodetic Vertical Datum (NGVD). This value is determined from the May 2018 water quality report, with the surficial aquifer ranging from 27 to 36 feet NGVD across the site.

The foundation soils of the ash cells have been loaded to some degree by the mass of the existing waste; therefore, the foundation soils already have experienced some degree of settlement. This settlement analysis did not account for the preloading of the foundation soils in the cells, which is conservative. The analysis was based on subsurface information from perimeter borings that have not been subject to the waste-loading conditions.

2.5 Establish the Final Stress Due to the Landfill

The final stress ($\sigma_{final} = \sigma_o + \Delta\sigma$) at the mid-point of each soil layer was calculated based on the development conditions. The applied or change in stress ($\Delta\sigma$) due to the landfill load was established at each settlement point based on the final elevation of the landfill and the bottom liner elevation at that point. The total applied stress at the bottom liner elevation is the sum of the applied stress for each component. The final soil stress (σ_{final}) at the midpoint of each soil layer was calculated by adding the initial midpoint stress (σ_o) to the applied stress ($\Delta\sigma$).

This analysis is for the total settlement of the waste due to the existing waste and proposed vertical expansion. The final stress calculated for this analysis includes the existing plus the proposed waste fill.

2.6 Calculate the Total Settlement at Each Settlement Point

The total settlement was calculated at each settlement point as the sum of the settlement of each soil layer. Settlement calculations depend on the type of subgrade soil. Settlement of generally non-cohesive soils is modeled by elastic settlement equations; settlement of generally cohesive soils is modeled by consolidation equations. Non-cohesive soils tend to settle more immediately, whereas cohesive soils consolidate and settle over time. The settlement equations used are based on Qian, et al. (2002). The soil layer and type, soil parameters, settlement calculation equations, and references used are attached to this memorandum.

2.6.1 Elastic Settlement

Elastic settlement is calculated based on the initial thickness of the soil layer (H_0), the change in stress ($\Delta\sigma$), a calculated elastic modulus (Es), the established Poisson's ratio (υ), and the calculated constrained modulus (Ms). The method of calculating the elastic modulus is based on soil type and average SPT N-values. The higher the SPT N value, the more compact the soil, and the lower the settlement.

2.6.2 Consolidation Settlement

Consolidation settlement is calculated based on the initial thickness of the soil layer (H_0), the change in stress ($\Delta\sigma$), a compression index value (Cc), a recompression index value (Cr), an initial void ratio (e_0), and the pre-consolidation stress (p_c). The compression and

recompression index, initial void ratio, and pre-consolidation stress values are based on a consolidation tests performed on clay samples.

The clayey soils above the limestone are defined as over-consolidated because the pre-consolidation stresses are greater than the existing stresses. The soils at this site, therefore, have experienced a higher load than their current state.

3 RESULTS

The attached calculations summarize the settlement equations used, the material parameters, the stresses at each settlement point, the calculated total settlement at each settlement point, the differential settlement between settlement points, and the slopes of the leachate collection lines after settlement.

The proposed vertical expansion design is for a maximum build-out elevation of 122 feet NAVD; about 21 feet higher than the current permitted design.

The maximum total settlement in the Ash Monofill vertical expansion is estimated to be approximately 1.0 foot at settlement point A2. The maximum differential settlement is estimated to be approximately 0.5 foot over 120feet between settlement points A2 and A3.

The attached settlement and bottom liner slope calculations demonstrate that the estimated differential settlements will not adversely affect the slope of the bottom liner. The settlement calculations for the proposed modification indicate that the existing bottom liner will have adequate positive drainage after settlement of the landfill.

The calculations show that the proposed modification meets the requirements of FDEP Rule 62-701.400(4)(c), FAC stating that: "The leachate collection and removal system shall be designed with a bottom slope to achieve the required leachate head after the predicted settlement determined by the foundation analysis. The minimum slope for the leachate collection system, in areas which drain to lateral collection pipes and header pipes, shall be 1.0 percent after predicted settlement. The minimum slopes for the collection pipes of the leachate collection system, i.e., lateral and header pipes, shall be 0.3 percent after predicted settlement.

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	PROJECT NUMBER:	16002-009-01	
	PROJECT NAME:	Pasco County - West Pasco Ash Monofill Slope Mod & Vertical Exp	
	SUBJECT:	Settlement Analysis	
	BY:	C.Sawyer	Date: 6/28/21
	CHECKED BY:	C.Spreadbury	Date: 6/29/21

Objective:

Calculate settlement of the existing Ash Monofill (A-1 through A-4) based on proposed vertical expansion.

Data:

Cross-sections and Settlement Point Locations:	Refer to Figures I.1 - I.4	
Subsurface investigation and lab testing performed by:	PSI Geotechnical Services Engr Report (dated 3/26/2021)	
Estimated Seasonal High Water Table elevation:	36 feet	(based on May 2018 GW report)
Landfill Type:	Traditional	(no leachate recirculation)

Young's Elastic Modulus (Es)
Bowles 1996

Material	Unsaturated (ksf)	Saturated (ksf)
Normally Consolidated Sand	Es = 500(N+15)/50	Es = 250(N+15)/50
Gravelly Sand	Es = 1200(N+6)/50	--
Clayey Sand	Es = 320(N+15)/50	--
Silts, sandy silts, clayey silts	Es = 300(N+6)/50	--

Relative Densities (Safety Hammer)
FDOT Soils and Foundations Handbook

Granular Materials		Cohesive Materials	
"N" Value	Relative Density	"N" Value	Relative Density
<4	Very Loose	<2	Very Soft
4-10	Loose	2-4	Soft
10-30	Medium Dense	4-8	Firm
30-50	Dense	8-15	Stiff
>50	Very Dense	15-30	Very Stiff
>50	Refusal	>30	Hard

Poisson's Ratio (u)
"Bowles" 1996

Type of Soil	Poisson's Ratio
Clay, saturated	0.4 - 0.5
Clay, unsaturated	0.1-0.3
Sandy Clay	0.2-0.3
Silt	0.3-0.35
Sand, commonly used	0.3-0.4
Rock	0.1-0.4

Calculations:

Elastic Settlement (ES) Equation (Granular Soil)

$$Z_e = \left(\frac{\Delta\sigma}{M_s}\right) \times H_i$$ *(equ. 12.20, Qian, Koerner, & Gray)*

Where:

Z_e = elastic settlement of soil layer (ft)
$\Delta\sigma$ = increment of vertical effective stress (psf) = (σ_f - σ_o)
σ_o = initial vertical effective stress due to overlying soil (at mid-point of layer) (psf)
σ_f = initial vertical effective stress due to loading (at mid-point of layer) (psf)
M_s = constrained modulus of soil (psf) = (E_s x (1 - u_s)) / ((1 + u_s) x (1-2u_s)) *(equ. 12.21, Qian, Koerner, & Gray)*
E_s = Young's Elastic Modulus (psf) *(refer to Table 5-6, Bowles)*
u_s = Poisson's Ratio *(refer to Table 2-7, Bowles)*
H_i = initial thickness of soil layer (ft)

Consolidation Settlement Equation (Cohesive Soil) *(equ. 12.22, Qian, Koerner, & Gray)*

Normally Consolidated (NC)
For: $\sigma_o \geq p_c$

$$Z_c = \frac{C_c}{(1+e_o)} \times H_i \times \log\left(\frac{\sigma_o + \Delta\sigma}{\sigma_o}\right)$$

Overconsolidated Case I (OCI)
For: $\sigma_o < p_c$ and $\sigma_o + \Delta\sigma \leq p_c$

$$Z_c = \frac{C_r}{(1+e_o)} \times H_i \times \log\left(\frac{\sigma_o + \Delta\sigma}{\sigma_o}\right)$$

Overconsolidated Case II (OCII)
For: $\sigma_o < p_c$ and $\sigma_o + \Delta\sigma > p_c$

$$Z_c = \left[\frac{C_r}{(1+e_o)} \times H_i \times \log\left(\frac{p_c}{\sigma_o}\right)\right] + \left[\frac{C_c}{(1+e_o)} \times H_i \times \log\left(\frac{\sigma_o + \Delta\sigma}{p_c}\right)\right]$$

Where:

Z_c = primary consolidation settlement of clay layer (ft)
C_c = primary compression index
C_r = recompression index (For $\sigma_o = p_c$, C_r is modeled as 0)
H_i = initial thickness of clay layer (ft)
e_{oi} = initial void ratio of clay layer
$\Delta\sigma$ = increment of vertical effective stress (psf) = (σ_f - σ_o)
σ_f = initial vertical effective stress due to loading (at mid-point of layer) (psf)
σ_o = initial vertical effective stress due to overlying soil (at mid-point of layer) (psf)
p_c = preconsolidation pressure (psf) (if normally consolidated pc = 0 psf)

References:

1. "*Geotechnical Aspects of Landfill Design & Construction*", Qian, Koerner, & Gray, 2002.
2. "*Foundation Analysis and Design",* Bowles, 5th Edition, 1996.
3. "*Test Data from Landfilled WTE Ash* ", Ardaman & Associates, Inc., 2007.
4. "*FDOT Soils and Foundation Handbook",* 2018. (Table 1)

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	CHECKED BY:	C.Spreadbury	Date: 6/29/21

SOIL PROPERTIES

							Consolidation Parameters (if Applicable)			
Soil Layer ID	**Description**	**Material**	**Poisson's Ratio μ_s**	**Friction Angle φ (degree)**	**Cohesion c (psf)**	**Unit Weight γ (pcf)**	**Compression Index C_c**	**Recompression Index C_r**	**Void Ratio e_o**	**Preconsolidation Pressure P_c (psf)**
1	Compacted Subgrade	Sand	0.30	32	0	115				
2	SP, SP-SM,SP-SC, SM	Sand	0.30	30	0	110				
3	SC,CL,CH	Clay	0.45		2000	120	0.50	0.02	1.0	4,000
4	CH, SC	Clay	0.45		2000	120	0.88	0.04	1.2	4,000
5	Weathered LS	Limestone	0.40	40	0	125				

Notes: 1. Compacted subgrade = minimum 1 feet of compacted soil beneath the bottom liner; fill material under liner.

2. Compression Index (Cc), Recompression Index (Cr), Void Ratio (e) and Preconsolidation Pressure (Pc) are estimated based on Qore LL and PL test results.

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PROJECT NAME: Pasco County - West Pasco Ash Monofill Slope Mod & Vertical Exp
SUBJECT: Settlement Analysis
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CHECKED BY: C.Spreadbury Date: 6/29/21

SOIL STRESS CONDITIONS AT THE BOTTOM LINER ELEVATION

Settlement Point	**A1 (Cell A4)**	**A2 (Cell A4)**	**A3 (Cell A4)**	**A4 (Cell A4)**	**C1 (Cell A1)**	**C2 (Cell A1)**	**C3 (Cell A1)**	**C4 (Cell A2)**	**C5 (Cell A2)**	**C6 (Cell A2)**	**C7 (Cell A3)**	**C8 (Cell A3)**	**D1 (Cell A4)**	**D2 (Cell A4)**
Cross-Section	**A-A**	**A-A**	**A-A**	**A-A**	**C-C**	**C-C**	**C-C**	**C-C**	**C-C**	**C-C**	**C-C**	**C-C**	**D-D**	**D-D**
Boring	general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings	general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings	general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings	general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based 2021 (A-3) PSI borings	general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings	general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings
Final closure elevation (ft)	122.0	122.0	80.0	68.0	79.0	101.2	113.0	117.0	122.0	120.0	110.0	105.0	122.0	117.0
Top of waste elevation (ft)	119.0	119.0	77.0	65.0	76.0	98.2	110.0	114.0	119.0	117.0	107.0	102.0	119.0	114.0
Bottom liner elevation (ft)	50.0	46.0	45.0	43.0	53.5	53.3	51.7	50.0	51.5	52.0	42.1	43.4	46.6	44.9
Existing grade elev (ft) (before construction)	50.0	46.0	45.0	43.0	53.5	53.3	51.7	50.0	51.5	52.0	42.1	43.4	46.6	44.9
Seasonal high water table (SHWT) elev (ft)	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0
Thickness (feet)														
Cover soil thickness (T_c)	3	3	3	3	3	3	3	3	3	3	3	3	3	3
Class I Waste thickness (T_{waste})	67	71	30	20	21	43	56	62	66	63	63	57	70	67
Drainage sand thickness (T_s)	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Unit Weights (pcf)														
Cover Soil (γ_c)	120	120	120	120	120	120	120	120	120	120	120	120	120	120
Waste (γ_{waste}) ASH Waste	110	110	110	110	110	110	110	110	110	110	110	110	110	110
Protective Cover (γ_s)	120	120	120	120	120	120	120	120	120	120	120	120	120	120
In-situ soil, unsaturated (existing grade above proposed liner) ($\gamma_{existing}$)	115	115	115	115	115	115	115	115	115	115	115	115	115	115
In-situ soil, saturated (existing grade above proposed liner) (γ_{sat})	120	120	120	120	120	120	120	120	120	120	120	120	120	120
γ_w = *unit weight of water* =	62.4													
Stress Increment ($\Delta\sigma$) (psf)														
σ_f = *stress from waste and cover soil load*														
$\sigma_f = (\gamma_c \times T_c) + (\gamma_{waste} \times T_{waste}) + (\gamma_s \times T_s) =$	7,970	8,410	3,900	2,800	2,910	5,330	6,760	7,420	7,860	7,530	7,530	6,870	8,300	7,970
σ_i = *initial stress at the bottom liner elevation*														
If existing grade and SHWT are at or above bottom liner elevation: $\sigma_2 = (\gamma_{existing} \times$ (existing grade elev - SHWT)) + ((γ_{sat} - 62.4) x (SHWT - bottom liner))=														
If existing grade is at or below the SHWT and above bottom liner elevation: $\sigma_i = ((\gamma_{sat}$ - 62.4) x (existing grade elev - bottom liner))=														
If the existing grade is above the bottom liner and both are above the SHWT: $\sigma_2 = \gamma_{existing}$ x (existing grade elev - bottom liner)=														
If bottom liner is above or equal to the existing grade: $\sigma_i = 0$	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total $\Delta\sigma = \sigma_f - \sigma_i$	7,970	8,410	3,900	2,800	2,910	5,330	6,760	7,420	7,860	7,530	7,530	6,870	8,300	7,970

Note: 2 feet of drainage sand on top of bottom liner therefore: waste thickness = top of waste - bottom liner - sand thickness

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SUBJECT: Settlement Analysis
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CHECKED BY: C.Spreadbury Date: 6/29/21

SETTLEMENT OF FOUNDATION SOILS DUE TO PROPOSED WASTE LOADING

Notes:
1. Refer to Figures I-1 through I-4.

Settlement Point: A1 Surface Elevation: 50 feet Bottom Liner Elevation: 50.0 feet SHWT: 36 feet

general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings

Elev	Soil ID	Soil	Hoi (feet)	φ (degree)	γ (pcf)	Navg	Calc Type	Elastic Settlement (ES) Equation	(ksf)	(psf)	Consolidation (NC, OCI, OCII) Cc	Cr	eo	Pc	us	Ms (psf)	σo (mid-point) (psf)	Δσ (psf)	σtotal = σo + Δσ (psf)	Z (ft)	Z (in)
50 - 48	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	7,970	8,085	0.03	0.41
48 - 36	2	SP, SP-SM,SP-SC, SM	12.0	30	110	7	ES	Es = 500(N+15)/50	220	220,000					0.30	296,154	890	7,970	8,860	0.32	3.88
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	10	ES	Es = 250(N+15)/50	125	125,000					0.30	168,269	1,693	7,970	9,663	0.28	3.41
30 - 20	3	SC,CL,CH	10.0	0	120	7	OCI				0.50	0.02	1.00	4,000	0.45		2,124	7,970	10,094	0.08	0.93
20 - -20	5	Weathered LS	40.0	40	125	16	ES	Es = 1200(N+6)/50	528	528,000					0.40	1,131,429	3,664	7,970	11,634	0.28	3.38
																		Total Settlement =		**1.00**	**12.0**

Settlement Point: A2 Surface Elevation: 46 feet Bottom Liner Elevation: 46 feet SHWT: 36 feet

general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings

Elev	Soil ID	Soil	Hoi (feet)	φ (degree)	γ (pcf)	Navg	Calc Type	Elastic Settlement (ES) Equation	(ksf)	(psf)	Consolidation (NC, OCI, OCII) Cc	Cr	eo	Pc	us	Ms (psf)	σo (at midpoint) (psf)	Δσ (psf)	σtotal = σo + Δσ (psf)	Z (ft)	Z (in)
46 - 44	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	8,410	8,525	0.04	0.43
44 - 36	2	SP, SP-SM,SP-SC, SM	8.0	30	110	10	ES	Es = 500(N+15)/50	250	250,000					0.30	336,538	670	8,410	9,080	0.20	2.40
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	10	ES	Es = 250(N+15)/50	125	125,000					0.30	168,269	1,253	8,410	9,663	0.30	3.60
30 - 20	3	SC,CL,CH	10.0	0	120	7	OCI				0.50	0.02	1.00	4,000	0.45		1,684	8,410	10,094	0.09	1.07
20 - -20	5	Weathered LS	40.0	40	125	11	ES	Es = 1200(N+6)/50	408	408,000					0.40	874,286	3,224	8,410	11,634	0.38	4.62
																		Total Settlement =		**1.01**	**12.1**

Settlement Point: A3 Surface Elevation: 45 feet Bottom Liner Elevation: 45.0 feet SHWT: 36 feet

general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings

Elev	Soil ID	Soil	Hoi (feet)	φ (degree)	γ (pcf)	Navg	Calc Type	Elastic Settlement (ES) Equation	(ksf)	(psf)	Consolidation (NC, OCI, OCII) Cc	Cr	eo	Pc	us	Ms (psf)	σo (at midpoint) (psf)	Δσ (psf)	σtotal = σo + Δσ (psf)	Z (ft)	Z (in)
45.0 - 43	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	3,900	4,015	0.02	0.20
43 - 36	2	SP, SP-SM,SP-SC, SM	7.0	30	110	10	ES	Es = 500(N+15)/50	250	250,000					0.30	336,538	615	3,900	4,515	0.08	0.97
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	10	ES	Es = 250(N+15)/50	125	125,000					0.30	168,269	1,143	3,900	5,043	0.14	1.67
30 - 20	3	SC,CL,CH	10.0	0	120	7	OCI				0.50	0.02	1.00	4,000	0.45		1,574	3,900	5,474	0.06	0.75
20 - -20	5	Weathered LS	40.0	40	125	11	ES	Es = 1200(N+6)/50	408	408,000					0.40	874,286	3,114	3,900	7,014	0.18	2.14
																		Total Settlement =		**0.48**	**5.7**

Settlement Point: A4 Surface Elevation: 43 feet Bottom Liner Elevation: 43.0 feet SHWT: 36 feet

general profile based on 2005 (B1- B6) and 2021 (A-1)PSI borings

Elev	Soil ID	Soil	Hoi (feet)	φ (degree)	γ (pcf)	Navg	Calc Type	Elastic Settlement (ES) Equation	(ksf)	(psf)	Consolidation (NC, OCI, OCII) Cc	Cr	eo	Pc	us	Ms (psf)	σo (at midpoint) (psf)	Δσ (psf)	σtotal = σo + Δσ (psf)	Z (ft)	Z (in)
43.0 - 41	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	2,800	2,915	0.01	0.14
41 - 36	2	SP, SP-SM,SP-SC, SM	5.0	30	110	10	ES	Es = 500(N+15)/50	250	250,000					0.30	336,538	505	2,800	3,305	0.04	0.50
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	10	ES	Es = 250(N+15)/50	125	125,000					0.30	168,269	923	2,800	3,723	0.10	1.20
30 - 20	3	SC,CL,CH	10.0	0	120	7	OCI				0.50	0.02	1.00	4,000	0.45		1,354	2,800	4,154	0.06	0.67
20 - -20	5	Weathered LS	40.0	40	125	8	ES	Es = 1200(N+6)/50	336	336,000					0.40	720,000	2,894	2,800	5,694	0.16	1.87
																		Total Settlement =		**0.36**	**4.4**

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Settlement Point: C1 Surface Elevation: 53.5 feet Bottom Liner Elevation: 53.5 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
53.5 - 51.5	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	2,910	3,025	0.01	0.15
51.5 - 36	2	SP, SP-SM,SP-SC, SM	15.5	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	1,198	2,910	4,108	0.12	1.39
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	2,193	2,910	5,103	0.09	1.07
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		2,624	2,910	5,534	0.04	0.45
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	4,164	2,910	7,074	0.09	1.04
																			Total Settlement =	**0.34**	**4.1**

Settlement Point: C2 Surface Elevation: 53.3 feet Bottom Liner Elevation: 53.3 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
53.3 - 51.3	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	5,330	5,445	0.02	0.27
51.3 - 36	2	SP, SP-SM,SP-SC, SM	15.3	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	1,072	5,330	6,402	0.21	2.51
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	2,056	5,330	7,386	0.16	1.97
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		2,487	5,330	7,817	0.06	0.69
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	4,027	5,330	9,357	0.16	1.91
																			Total Settlement =	**0.61**	**7.3**

Settlement Point: C3 Surface Elevation: 51.7 feet Bottom Liner Elevation: 51.7 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
51.7 - 49.7	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	6,760	6,875	0.03	0.34
49.7 - 36	2	SP, SP-SM,SP-SC, SM	13.7	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	984	6,760	7,744	0.24	2.85
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	1,880	6,760	8,640	0.21	2.49
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		2,311	6,760	9,071	0.07	0.82
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	3,851	6,760	10,611	0.20	2.43
																			Total Settlement =	**0.74**	**8.9**

Settlement Point: C4 Surface Elevation: 50 feet Bottom Liner Elevation: 50.0 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
50.0 - 48.0	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	7,420	7,535	0.03	0.38
48 - 36	2	SP, SP-SM,SP-SC, SM	12.0	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	890	7,420	8,310	0.23	2.74
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	1,693	7,420	9,113	0.23	2.74
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		2,124	7,420	9,544	0.08	0.90
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	3,664	7,420	11,084	0.22	2.66
																			Total Settlement =	**0.78**	**9.4**

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PROJECT NUMBER: 16002-009-01
PROJECT NAME: Pasco County - West Pasco Ash Monofill Slope Mod & Vertical Exp
SUBJECT: Settlement Analysis
BY: C.Sawyer Date: 6/28/21
CHECKED BY: C.Spreadbury Date: 6/29/21

Settlement Point: C5 Surface Elevation: 51.5 feet Bottom Liner Elevation: 51.5 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
51.5 - 49.5	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	7,860	7,975	0.03	0.40
49.5 - 36	2	SP, SP-SM,SP-SC, SM	13.5	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	973	7,860	8,833	0.27	3.26
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	1,858	7,860	9,718	0.24	2.90
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		2,289	7,860	10,149	0.07	0.89
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	3,829	7,860	11,689	0.24	2.82
																			Total Settlement =	**0.86**	**10.3**

Settlement Point: C6 Surface Elevation: 52.0 feet Bottom Liner Elevation: 52.0 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
52.0 - 50.0	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	7,530	7,645	0.03	0.38
50 - 36	2	SP, SP-SM,SP-SC, SM	14.0	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	1,000	7,530	8,530	0.27	3.24
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	1,913	7,530	9,443	0.23	2.78
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		2,344	7,530	9,874	0.07	0.86
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	3,884	7,530	11,414	0.23	2.70
																			Total Settlement =	**0.83**	**10.0**

Settlement Point: C7 Surface Elevation: 42.1 feet Bottom Liner Elevation: 42.1 feet SHWT: 36 feet

general profile based 2021 (A-3) PSI borings

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
42.1 - 40.1	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	7,530	7,645	0.03	0.38
40.1 - 36	2	SP, SP-SM,SP-SC, SM	4.1	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	456	7,530	7,986	0.08	0.95
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	824	7,530	8,354	0.23	2.78
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		1,255	7,530	8,785	0.10	1.17
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	2,795	7,530	10,325	0.23	2.70
																			Total Settlement =	**0.66**	**8.0**

Settlement Point: C8 Surface Elevation: 43.4 feet Bottom Liner Elevation: 43.4 feet SHWT: 36 feet

Elev	Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
			(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
43.4 - 41.4	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	6,870	6,985	0.03	0.35
41.4 - 36	2	SP, SP-SM,SP-SC, SM	5.4	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	527	6,870	7,397	0.10	1.14
36 - 30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	967	6,870	7,837	0.21	2.53
30 - 20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		1,398	6,870	8,268	0.09	1.07
20 - -20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	2,938	6,870	9,808	0.21	2.47
																			Total Settlement =	**0.63**	**7.6**

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PROJECT NAME: Pasco County - West Pasco Ash Monofill Slope Mod & Vertical Exp
SUBJECT: Settlement Analysis
BY: C.Sawyer Date: 6/28/21
CHECKED BY: C.Spreadbury Date: 6/29/21

Settlement Point: D1 Surface Elevation: 46.6 feet Bottom Liner Elevation: 46.6 feet SHWT: 36 feet

Elev		Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
				(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
46.6 -	44.6	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	8,300	8,415	0.04	0.42
44.6 -	36	2	SP, SP-SM,SP-SC, SM	8.6	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	703	8,300	9,003	0.18	2.19
36 -	30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	1,319	8,300	9,619	0.26	3.06
30 -	20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		1,750	8,300	10,050	0.09	1.05
20 -	-20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	3,290	8,300	11,590	0.25	2.98
																				Total Settlement =	**0.81**	**9.7**

Settlement Point: D2 Surface Elevation: 44.9 feet Bottom Liner Elevation: 44.9 feet SHWT: 36 feet

0

Elev		Soil ID	Soil	Hoi	φ	γ	Navg	Calc Type	Elastic Settlement (ES)			Consolidation (NC, OCI, OCII)				us	Ms	σo (at midpoint)	Δσ	σtotal = σo + Δσ	Z	
				(feet)	(degree)	(pcf)			Equation	(ksf)	(psf)	Cc	Cr	eo	Pc		(psf)	(psf)	(psf)	(psf)	(ft)	(in)
44.9 -	42.9	1	Sand	2.0	32	115	20	ES	Es = 500(N+15)/50	350	350,000					0.30	471,154	115	7,970	8,085	0.03	0.41
42.9 -	36	2	SP, SP-SM,SP-SC, SM	6.9	30	110	14	ES	Es = 500(N+15)/50	290	290,000					0.30	390,385	610	7,970	8,580	0.14	1.69
36 -	30	2	SP, SP-SM,SP-SC, SM	6.0	30	110	14	ES	Es = 250(N+15)/50	145	145,000					0.30	195,192	1,132	7,970	9,102	0.24	2.94
30 -	20	3	SC,CL,CH	10.0	0	120	11	OCI				0.50	0.02	1.00	4,000	0.45		1,563	7,970	9,533	0.09	1.08
20 -	-20	5	Weathered LS	40.0	40	125	20	ES	Es = 1200(N+6)/50	624	624,000					0.40	1,337,143	3,103	7,970	11,073	0.24	2.86
																				Total Settlement =	**0.75**	**9.0**

JonesEdmunds	PROJECT NUMBER:	16002-009-01	
	PROJECT NAME:	Pasco County - West Pasco Ash Monofill Slope Mod & Vertical Exp	
	SUBJECT:	Settlement Analysis	
	BY:	C.Sawyer	DATE: 6/28/2021
	CHECKED BY:	C.Spreadbury	DATE: 6/29/2021

BOTTOM LINER SLOPE CALCULATIONS

Settlement Point	Boring	Elev. Before Settlement	Total Potential Settlement (feet)	Elevation After Settlement	Slope Point-Point	Differential Settlement (feet)	Distance to Adjacent Point (feet)	Slope Before Settlement	Slope After Settlement
Section A-A'					**Cell A-4 Constructed in 2008. Main leachate trench flowing east to west.**				
					0.3% after settlement for leachate collection trench				
A1	A-A	50.0	1.00	49.0					
A2	A-A	46.0	1.01	45.0	A1→A2	0.01	1056	0.4%	0.4%
A3	A-A	45.0	0.48	44.5	A2→A3	0.53	120	0.8%	0.4%
A4	A-A	43.0	0.36	42.6	A3→A4	0.11	36	5.6%	5.2%
				(High to low)	**A1→A3**	**0.5**	**1176**	**0.4%**	**0.4%**

Section C-C'	Cell A-2 constructed in 1996. Sawtooth design flowing north. Cell A-1 construcuted 1991; sawtooth design flowing south.								
0.3% after settlement for leachate collection trench									
C1	C-C	53.5	0.34	53.2					
C2	C-C	53.3	0.61	52.7	C1→C2	0.27	72	0.3%	0.7%
C3	C-C	51.7	0.74	51.0	C2→C3	0.13	240	0.7%	0.7%
				(High to low)	**C1→C3**	0.40	312	0.6%	0.7%
C4	C-C	50.0	0.78	49.2					
C5	C-C	51.5	0.86	50.6	C5→C4	0.07	150	1.0%	1.0%
C6	C-C	52.0	0.83	51.2	C6→C5	0.03	140	0.4%	0.4%
				(High to low)	**C6→C4**	0.05	**289**	0.7%	0.7%
1.0% after settlement for slopes to the leachate collection trench									
C7	C-C	42.1	0.66	41.4					
C8	C-C	43.4	0.63	42.8		0.04			
				(High to low)	**C8→C7**	0.04	**64**	2.0%	2.1%
1.0% after settlement for slopes to the leachate collection trench									
D1	D-D	46.6	0.81	45.8					
D2	D-D	44.9	0.75	44.2		0.06			
				(High to low)	**D1→D2**	0.06	**84**	2.0%	1.9%

Reference 1

Geotechnical Aspects of Landfill Design & Construction, Qian, Koerner, & Gray, 2002

Reference 1

GEOTECHNICAL ASPECTS OF LANDFILL DESIGN AND CONSTRUCTION

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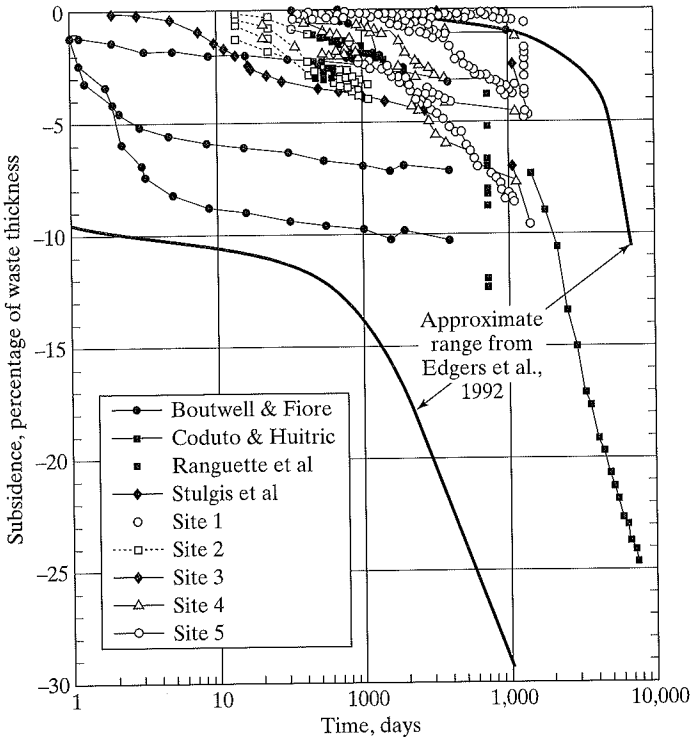


FIGURE 12.1 Landfill Subsidence, New and Previously Published Data (after Spikula, 1997)

drains. The latter outcome may increase the amount of moisture entering the landfill, which, in turn, will produce more leachate.

Settlement of municipal solid waste often begins rapidly as load is placed and continues to occur for long periods thereafter. The mechanisms of refuse settlement are complex, even more so than for soil, because of the extreme heterogeneity of waste fill and the presence of large voids. The main mechanisms involved in waste settlement are the following (Sowers, 1973; Murphy and Gilbert, 1985; Edil et al., 1990; Edgers et al., 1992):

(i) *Mechanical Compression:* Densification, distortion, bending, crushing, and reorientation; similar to consolidation of organic soils. Compression caused by the self-weight of the landfill and imposed loads, occurs in the from of initial, and/or primary consolidation, and/or secondary (delayed) compression.

(ii) *Raveling:* The movement of finer particles into larger voids or cavities within the fill. It is usually difficult to distinguish this mechanism from others.

(iii) *Physical-Chemical Change:* The deterioration and volume loss of waste products by corrosion, oxidation, and combustion.

(iv) *Bio-Chemical Decomposition:* The reduction of waste mass by fermentation and decay, both aerobic and anaerobic processes. This mechanism will be discussed at length in Chapter 15 on bioreactor landfills.

Reference 1

(e.g., temperature within landfill and oxygen reaching the waste) still is not entirely clear. These functions should be used with caution in engineering practice and should be supported by additional testing data and research.

12.7 ESTIMATION OF LANDFILL FOUNDATION SETTLEMENT

If the landfill is underlain by a soil layer, particularly a thick layer of soft, fine-grained soil, consolidation settlements may be large. In these cases, design analyses should consider settlement of the foundation clay layer. Both primary consolidation and long-term secondary settlement should be considered. Calculations are performed using conventional equations from soil mechanics theory and a time frame at least equal to the active life and postclosure care period of the landfill.

Excessive settlement of an underlying foundation clay layer will affect the performance of a landfill liner and leachate collection system. The purposes of analyzing the settlement of a foundation clay layer and overlying landfill liner and leachate collection/removal system are as follows:

(i) Tensile strain induced in the liner system and leachate collection and removal system must be limited to a minimum allowable tensile strain for the components of these two systems. The compacted clay liner usually has the smallest allowable tensile strain value between 0.1% and 1.0% and an average allowable tensile strain of 0.5%.

(ii) Post-settlement grades of the landfill cell subbase and the leachate collection pipes must be sufficient to maintain leachate performance to prevent grade reversal and leachate ponding in accordance with the rule requirements.

12.7.1 Total Settlement of Landfill Foundation

The total settlement of landfill foundation soil can be divided into three portions: elastic settlement, primary consolidation settlement, and secondary consolidation settlement. The settlement of sandy soils includes only elastic settlement. The settlement of clayey soils includes all three types of settlements. The total settlement of clayey soil is equal to the sum of the elastic settlement and the primary and secondary settlements. Because the permeability of clay is quite low, it takes a long time to complete the whole process of consolidation settlement. The settlement of clayey soil is usually much larger than the settlement of sandy soils.

Because the settlement of sandy soils includes only elastic settlement, the settlement of sand layer can be calculated from the Elastic Settlement equation, which is

Elastic Settlement

$$Z_e = (\Delta\sigma/M_s)H_o \tag{12.20}$$

where Z_e = elastic settlement of soil layer, ft or m;
H_o = initial thickness of soil layer, ft or m;
$\Delta\sigma$ = increment of vertical effective stress, lb/ft^2 or kN/m^2;
M_s = constrained modulus of soil, lb/ft^2 or kN/m^2.

Reference 1

The constrained modulus is given by

$$M_s = \frac{E_s \cdot (1 - v_s)}{(1 + v_s)(1 - 2 \cdot v_s)} \tag{12.21}$$

where M_s = constrained modulus of soil, lb/ft^2 or kN/m^2;
E_s = elastic modulus of soil, see Table 9.5, lb/ft^2 or kN/m^2;
v_s = Poisson's ratio of soil, see Table 9.5.

The *primary consolidation settlement* is given by

Consolidation Settlement

$$Z_c = C_r \cdot \frac{H_{oi}}{1 + e_{oi}} \cdot \log \frac{p_c}{\sigma_o} + C_c \cdot \frac{H_o}{1 + e_{oi}} \cdot \log \frac{\sigma_o + \Delta\sigma}{p_c} \tag{12.22}$$

where Z_c = primary consolidation settlement of clay layer, ft or m;
H_o = initial thickness of clay layer, ft or m;
e_{oi} = initial void ratio of clay layer;
C_r = recompression index;
C_c = primary compression index.
σ_o = initial vertical effective stress, lb/ft^2 or kN/m^2;
p_c = preconsolidation pressure, lb/ft^2 or kN/m^2;
$\Delta\sigma$ = increment of vertical effective stress, lb/ft^2 or kN/m^2.

The *secondary compression settlement* is given by

$$Z_\alpha = C_\alpha \cdot \frac{H_{os}}{1 + e_{os}} \cdot \log \frac{t_2}{t_1} \tag{12.23}$$

where Z_α = long-term secondary compression settlement, ft or m;
e_{os} = initial void ratio of clay layer before starting secondary consolidation settlement;
C_α = secondary consolidation compression index;
H_{os} = initial thickness of clay layer before starting secondary consolidation settlement, ft or m;
t_1 = starting time of the time period for which long-term settlement of the layer is desired;
t_2 = ending time of the time period for which long-term settlement of the layer is desired.

The total settlement of clay layer includes three portions: elastic settlement, primary consolidation settlement, and secondary consolidation settlement. These three types of settlement for clayey soil layers can be calculated from Equations 12.20, 12.22, and 12.23, respectively. The total settlement of clayey soil at point *i* can be determined from the equation

$$Z_i = (Z_e)_i + (Z_c)_i + (Z_\alpha)_i \tag{12.24}$$

where Z_i = total settlement of points *i*;
$(Z_e)_i$ = elastic settlement of point *i*;
$(Z_c)_i$ = primary consolidation settlement of point *i*;
$(Z_\alpha)_i$ = secondary consolidation settlement of point *i*.

Reference 1

The preceding settlement equations (Equations 12.20 through 12.24) provide a framework and means to account for different types of settlement. Not all foundations on soil settle exactly in the manner previously described. In addition to soil type, the amount of settlement in each category also depends on the degree of saturation, load duration, and load distribution.

The settlement calculations should be performed at discrete points along several selected settlement lines, such as Lines 1, 2, 3, and 4 for a municipal solid waste landfill as shown in Figure 12.16. The following principles should guide the arrangement of the settlement lines for landfill foundation settlement calculations:

(i) Some settlement lines should be set along the leachate collection pipelines (usually 1% slope) to check the grade changes and tensile strains of the leachate collection pipes due to settlement.

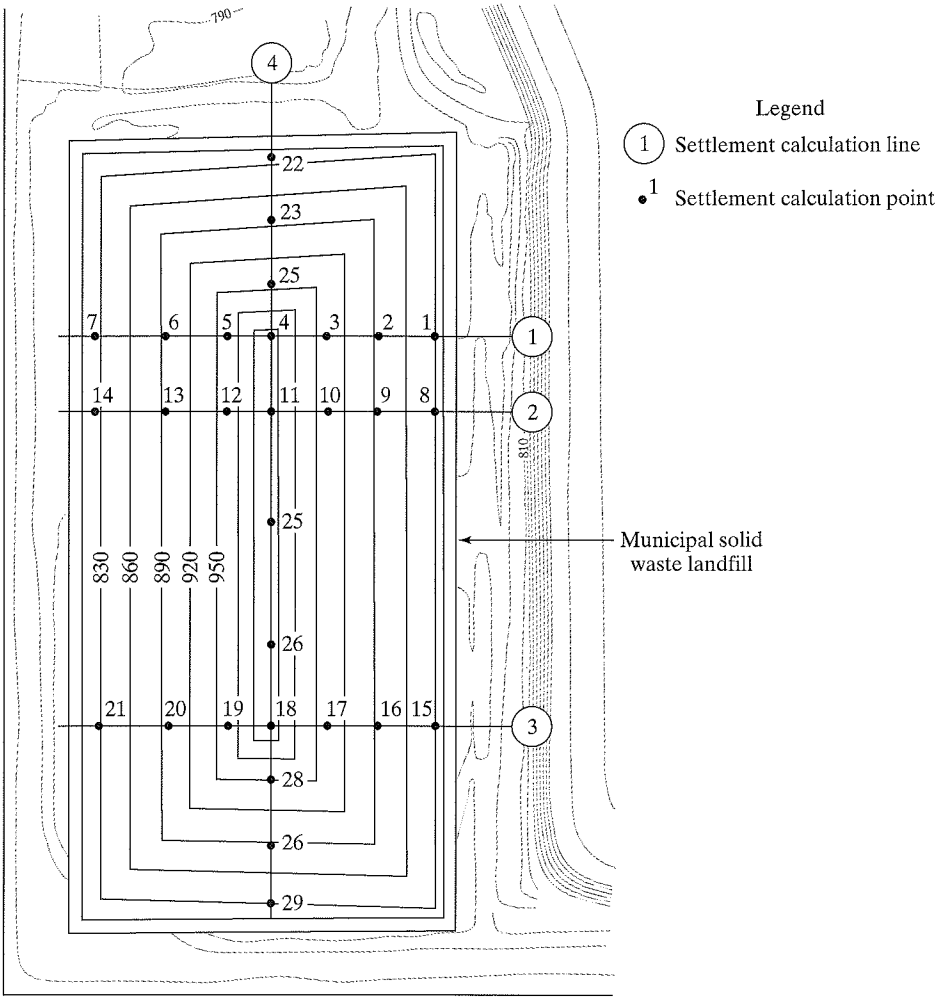


FIGURE 12.16 Settlement Calculation Locations for Landfill Liner System Foundation

Reference 1

(ii) Some settlement lines should be set perpendicular to the leachate collection pipes (i.e., along the direction of leachate flow in the leachate drainage layer—usually 2% slope), to check the slope change of leachate drainage layer.

(iii) Settlement lines are usually set locations where there are the large changes of overburden pressures, which may cause large differential settlements of the subgrade.

At each settlement point shown in Figure 12.16 (i.e., Point 1 to Point 29), the thickness of the various soil units at each point and the thickness of the waste to be placed (i.e., overburden pressure) can be estimated from the engineering plans or cross-sections for the specific projects. The value of the total settlement at each point depends on both the engineering properties of soils and the load due to the waste fill.

12.7.2 Differential Settlement of Landfill Foundation

The differential settlements, tensile strains of liner system materials and leachate collection pipes, and changes of final grades between adjacent settlement points after settlement can be evaluated from the calculated values of the total settlements at various settlement points along each settlement line ranged on the landfill subgrade.

The differential settlement between adjacent points can be calculated using the equation

$$\Delta Z_{i,i+1} = Z_{i+1} - Z_i \tag{12.25}$$

where $\Delta Z_{i,i+1}$ = differential settlement between points i and $i + 1$;
Z_i = total settlement of point i;
Z_{i+1} = total settlement of point $i + 1$.

The final slope angle between adjacent points after settlement can be calculated using the equation

$$\tan\beta_{Fnl} = \frac{X_{i,i+1} \cdot \tan\beta_{Int} - \Delta Z_{i,i+1}}{X_{i,i+1}} \tag{12.26}$$

where $X_{i,i+1}$ = horizontal distance between points i and $i + 1$;
$\Delta Z_{i,i+1}$ = differential settlement between points i and $i + 1$;
β_{Int} = initial slope angle between points i and $i + 1$;
β_{Fnl} = final slope angle between points i and $i + 1$ after settlement.

The landfill subgrade changes along each settlement line due to different settlement can be calculated from the Equation 12.26. Figure 12.17 presents the slope changes due to differential settlement along a settlement line. The differential settlement will result in grade reversal between points 3 and 4 as shown in Figure 12.17. As a result, leachate will pond on the liner at this area.

The tensile strains of a liner system and a leachate collection system resulting from the settlements can be estimated using the equation

$$\varepsilon_{i,i+1} = \frac{(L_{i,i+1})_{Fnl} - (L_{i,i+1})_{Int}}{(L_{i,i+1})_{Int}} \times 100\% \tag{12.27}$$

where $\varepsilon_{i,i+1}$ = tensile strain in liner system between points i and $i + 1$;

Reference 1

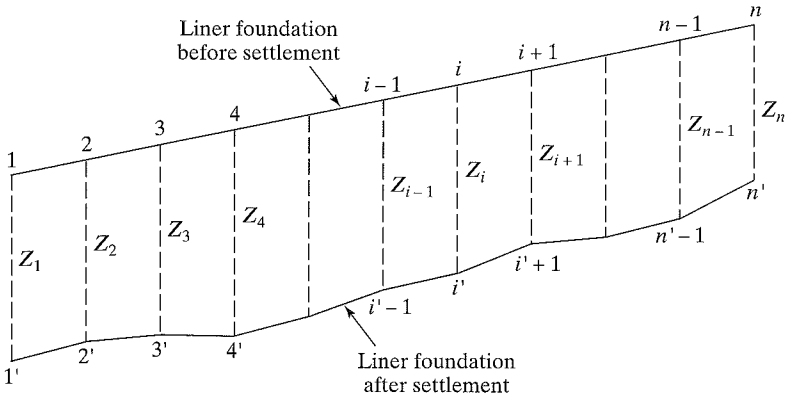


FIGURE 12.17 Subgrade Changes along a Settlement Line due to Differential Settlement

$(L_{i,i+1})_{Int}$ = distance between points i and $i + 1$ in their initial positions;
$(L_{i,i+1})_{Fnl}$ = distance between points i and $i + 1$ in their post-settlement positions.

The distance between points i and $i + 1$ in their initial positions can be calculated using the equation

$$(L_{i,i+1})_{Int} = [(X_{i,i+1})^2 + (X_{i,i+1} \cdot \tan\beta_{Int})^2]^{1/2} \tag{12.28}$$

The distance between points i and $i + 1$ in their post-settlement positions can be calculated using the equation

$$(L_{i,i+1})_{Fnl} = [(X_{i,i+1})^2 + (X_{i,i+1} \cdot \tan\beta_{Int} - \Delta Z_{i,i+1})^2]^{1/2} \tag{12.29}$$

The maximum acceptable tensile strains (i.e., the elongations at yield) of various liner system and leachate collection system components must be obtained from material specific laboratory testing.

PROBLEMS

12.1 What benefit does a landfill operation gain from some settlement in the landfill waste?

12.2 Describe the impact of landfill settlement on landfill design and performance.

12.3 What are the main mechanisms of solid waste settlement?

12.4 List the main factors that influence the amount of landfill settlement.

12.5 What are the main differences of the settlement of solid waste compared with the settlement of clay?

12.6 What are the main advantages and disadvantages of the calculation method for landfill settlement described in Section 12.4?

Reference 2

Foundation Analysis and Design, Bowles, 5th Edition, 1996

Reference 2

FOUNDATION ANALYSIS AND DESIGN

Fifth Edition

Joseph E. Bowles, P.E., S.E.
Consulting Engineer/Software Consultant
Engineering Computer Software
Peoria, Illinois

The McGraw-Hill Companies, Inc.

New York St. Louis San Francisco Auckland Bogotá Caracas
Lisbon London Madrid Mexico City Milan Montreal New Delhi
San Juan Singapore Sydney Tokyo Toronto

Reference 2

TABLE 2-7
Values or value ranges for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	−0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15
Steel	0.33

0.30 Soil 1, 2
0.045 Soil 3, 4
0.40 Soil 5 (weathered LS)

Another material property concept is the *bulk modulus* E_b, which is defined as the ratio of hydrostatic stress to the volumetric strain ϵ_v and is given as

$$E_b = \frac{2}{3}G'\frac{1+\mu}{1-2\mu} = \frac{E_s}{3(1-2\mu)} \qquad (f)$$

For an *elastic* material the shear modulus G' cannot be (−), so Eq. (*a*) sets the lower limit of $\mu > -1$. Equation (*f*) sets the upper limit at $\mu < 0.5$. It appears that the range of μ for soils (that are not "elastic") is from about −0.1 to 1.00. Table 2-7 gives a range of values for select materials. It is very common to use the following values for soils:

μ	Soil type
0.4–0.5	Most clay soils
0.45–0.50	Saturated clay soils
0.3–0.4	Cohesionless—medium and dense
0.2–0.35	Cohesionless—loose to medium

Although it is common to use $\mu = 0.5$ for saturated clay soils, the reader should be aware that this represents a condition of no volume change under the applied stress σ_z. Over time, however, volume change does occur as the pore fluid drains. Equation (*e*) defines the Poisson's ratio that develops initially ($\epsilon_v = 0$) and also later when $\epsilon_v > 0$. Since the strain is produced from stress and Fig. 1-1 indicates a vertical variation, it necessarily follows that μ is stress-dependent from Eq. (*e*).

A special case in geotechnical work is that of *plane strain.* This arises where strains occur parallel to two of the coordinate axes (say the x and z) but the strain is zero perpendicular to the x-z plane (along the y axis). If we set $\epsilon_y = 0$ in the set of equations for Hooke's law [(Eqs. (2-64)] and solve for the resulting values of E_s and μ, we obtain the following:

$$E_s' = \frac{E_s}{1-\mu^2} \qquad \mu' = \frac{\mu}{1-\mu} \qquad (2\text{-}65)$$

Reference 2

TABLE 5-6
Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}. Refer also to Tables 2-7 and 2-8.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — —	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$
	‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	*$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(\text{OCR})} \approx E_{s,\text{nc}}\sqrt{\text{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $\quad$ $^{\$}E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E_s' = 4q_c + 5000$ where E_s' = constrained modulus $= \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

4. It is not easy to determine if a cohesionless deposit is overconsolidated or what the OCR might be. Cementation may be less difficult to discover, particularly if during drilling or excavation sand "lumps" are present. Carefully done consolidation tests will aid in obtaining the OCR of cohesive deposits as noted in Chap. 2.

In general, with an OCR > 1 you should carefully ascertain the site conditions that will prevail at the time settlement becomes the design concern. This evaluation is, of course, true for any site, but particularly so if OCR > 1.

5-9 SIZE EFFECTS ON SETTLEMENTS AND BEARING CAPACITY

5-9.1 Effects on Settlements

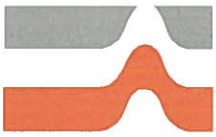
A major problem in foundation design is to proportion the footings and/or contact pressure so that settlements between adjacent footings are nearly equal. Figure 5-9 illustrates the problem

Reference 3

WTE Ash Testing Results, Ardaman & Associates, 2007

Excerpts from Ardaman & Associates, Inc. Report (2007)

SUBSURFACE SOIL EXPLORATION
LEE/HENDRY LANDFILL
HENDRY COUNTY, FLORIDA



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American Concrete Institute
American Society for Testing and Materials
Florida Institute of Consulting Engineers

Excerpts from Ardaman & Associates, Inc. Report (2007)

LABORATORY ANALYSIS

One-dimensional consolidation tests were performed on plastic clay specimens obtained from the Shelby tube samples retrieved from the test borings. The tests were performed in accordance with ASTM D 2435. This test method subjects a soil specimen to increasing loads and incrementally monitoring the change in height. The data is presented on an e-log p curve, which plots void ratio (e) as a function of the log of time pressure (p). The parameters necessary for settlement calculation can then be derived from these curves: compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c) and initial void ratio (e_o). A separate plot of the change in sample height versus log time for each load increment allows calculation of the coefficient of consolidation (c_v) and the coefficient of secondary compression (c_α) so that the ratio of primary settlement and amount of secondary compression may be predetermined. A summary of the data is presented below and on the e-log p curve in the Appendix.

Dry Unit Weight and Moisture Content Used to Calculate "Wet Unit Density"

SPT Boring Location	Sample No.	Depth (ft.)	Dry Unit Weight, (pcf)	Moisture Content, %	Percent Fines	Liquid Limit	Plasticity Index	USCS Classification	Remarks
Ash	U-1	24 - 26	93.0	27.4	-	-	-	SC	-
Ash	U-2	34 - 36	51.0	85.4	-	-	-	CL	-
Ash	U-3	40 - 42	88.8	32.4	35.9	-	-	SC	-
C&D	U-1	19 - 21	102.8	23.4	49.4	-	-	SC	-
C&D	U-2	30 - 32	53.4	79.8	91.5	168	124	CH	Consol
C&D	U-3	45 - 47	46.7	95.6	99.1	173	122	CH	Consol

Boring Location	
SPT C&	
SPT C&	

2007 Ardaman & Associates, Inc. Test Results

Sample	Depth (feet)	Dry Unit Weight (pcf)	MC (%)	Wet Unit Density (pcf)
Ash-1	24-26	93	27.4	118
Ash-2	34-36	51	85.4	94.6
Ash-3	40-42	88.8	32.4	118
Avg.		77.6	48.4	110

Ash sp

Undrained (CU) triaxial shear strength tests. The tests were performed in accordance with ASTM D 4767. In this test the specimen is allowed to consolidate under the confining pressure prior to shear, but no drainage is permitted during shear. A minimum of three tests at different confining pressures is required to derive the stress parameters. Pore pressure measurements are taken during testing so that the effective stress parameters can be derived. The results of the CU tests as derived from the p'-q plots are summarized below.

Excerpts from Ardaman & Associates, Inc. Report (2007)

Sample	Sample No.	Depth (ft.)	Friction Angle, Ø (deg)	Cohesion, c (tsf)
Clay	SPT C&D U-2	30 - 32	21.0	0.64
Clay	SPT C&D U-3	45 - 47	20.3	0.52
ASH	Bulk	-	44.3	0

GENERAL COMMENTS

The analysis in this report are based on the data obtained from the twenty-two (22) boring locations as directed by Jones Edmunds and Associates, Inc. personnel.

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and may be encountered. The boring logs and related information are based on the driller's logs and visual examination of selected sample in the laboratory.

The delineation between soil types shown on the logs is approximate and the description represents our interpretation of subsurface conditions at the designated boring locations and on the particular date drilled.

If you have any questions about this report, please contact this office.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.

Martin A. Call, P.E.
Senior Project Engineer

Gary A. Drew, P.E.
Branch Manager/Vice President

Reference 4

FDOT Soils and Foundation Handbook, 2018 (Table 1)

Soils and Foundations Handbook
2018

State Materials Office
Gainesville, Florida

Table 1, Relative Density or Consistency

Granular Materials		
Relative Density	**Safety Hammer SPT N-Value (Blow/Foot)**	**Automatic Hammer SPT N-Value (Blow/Foot)**
Very Loose	Less than 4	Less than 3
Loose	4 – 10	3 – 8
Medium Dense	10 – 30	8 – 24
Dense	30 – 50	24 – 40
Very Dense	Greater than 50	Greater than 40
Silts and Clays		
Consistency	**Safety Hammer SPT N-Value (Blow/Foot)**	**Automatic Hammer SPT N-Value (Blow/Foot)**
Very Soft	Less than 2	Less than 1
Soft	2 – 4	1 – 3
Firm	4 – 8	3 – 6
Stiff	8 – 15	6 – 12
Very Stiff	15 – 30	12 – 24
Hard	Greater than 30	Greater than 24

If SPT data is not available, consistency can be estimated in the field based on visual-manual examination of the material. Refer to ASTM D 2488 for consistency criteria.

The pocket penetrometer and torvane devices may be used in the field as an index of the remolded undrained shear strength of clay samples. See Section 5.15.4.

6.1.1.5 Friction Angle vs. SPT-N

Various published correlations estimate the angle of internal friction, φ, of cohesionless soils based on SPT-N values and effective overburden pressure. Some of these correlations are widely accepted whereas, others are more likely to overestimate triaxial test data. In the absence of laboratory shear strength testing, φ estimates for cohesionless soils, based on SPT-N, shall not exceed the values proposed by Peck, 1974 (see **Figure 16**). These values are based on SPT-N values obtained at an effective overburden pressure of one ton per square foot. The correction factor, C_N, proposed by Peck, 1974 (see **Figure 17**) may be used to "correct" N values obtained at overburden pressures other than 1 tsf.

Attachment I.1.e(3)(a)

Veneer Slope Stability Calculations

Reference 1	**Geotechnical Aspects of Landfill Design & Construction, Qian, Koerner, & Gray, 2002**
Reference 2	**Soil Mechanics, Lambe & Whitman, 1969**
Reference 3	**Caterpillar – D7R Series 2 Track-Type Tractor Specifications**

TECHNICAL MEMORANDUM

West Pasco County Ash Monofill Slope Modification and Vertical Expansion

TO: Project File

FROM: Chad Spreadbury, PhD, EI

XC: Carol Sawyer, PE

DATE: June 29, 2021

SUBJECT: Attachment I.1.e(3)(a) – Veneer Slope Stability Analysis
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

This Technical Memorandum summarizes the veneer slope stability analysis of the final cover system of the Ash Monofill slope modification and vertical expansion. This Technical Memorandum documents the methodology, design parameters, and analysis results. The final cover system design consists of 2 feet of soil over a geocomposite and a textured geomembrane on a 3H:1V side slope. This analysis determines a minimum interface friction angle (δ) between the interfaces of the cover system: soil vs geomembrane, soil vs geocomposite, and geocomposite vs geomembrane.

The final cover design meets the requirements of the Florida Department of Environmental Protection (FDEP) Rule 62-701.400(2), Florida Administrative Code (FAC).

2 METHODOLOGY

The veneer slope stability analysis is based on the limit equilibrium analysis for a finite length slope presented by Qian et al 2002 (Reference 1). This analysis balances forces for a finite-length uniformly thick cover soil over a liner on a constant slope. This analysis accounts for the wedge of soil at the bottom of the slope providing a counteractive force against the sliding veneer of soil. This analysis also accounts for the seepage forces in the cover soil. This is applicable to conditions in Florida where some stormwater is expected to infiltrate the cover soil and a potential for the cover soil exists to retain some of the permeated water. This condition is called parallel seepage.

Parallel seepage is defined as the condition when stormwater builds up parallel to the slope and may occur if the cover soils are not permeable enough to allow stormwater to flow down the slope. The analysis is based on assuming that 50 percent of the cover soil is saturated (parallel submergence ratio = 0.50).

This analysis also assumes an equipment load on the final cover surface. The equipment is assumed to be a tracked-type dozer.

The analysis included parallel seepage with an equipment load on the face of the slope. This is a worst-case condition and should not be considered as representative of the cover system conditions.

The analysis was performed using iterations based on varying the soil/geomembrane interface friction angles to determine the minimum interface friction angle to achieve a factor-of-safety against veneer slope failure of 1.5.

3 PARAMETERS

The veneer slope stability analyses were performed using soil strength parameters from testing results and literature data for the cover soils. The submergence ratio is modeled as 0.50 (50 percent of the total soil height is saturated). The final cover is designed with 3H:1V side slopes.

4 RESULTS

The veneer slope stability calculations and references are attached. The calculations show a minimum interface friction angle of 30 degrees is required to achieve a factor-of-safety of 1.5 against veneer slope failure. Based on experience with other final cover system interface friction testing, the materials will likely achieve interface friction angles greater than 30 degrees, resulting in a veneer slope stability factor-of-safety greater than 1.5.

The results indicate that the final cover design is sufficient to meet the required minimum factors of safety. This meets the requirements of Rule 62-701.400(2), FAC, "designs to prevent failures of side slopes, and designs to prevent deep-seated failures through the waste, along liner systems, and through foundation soils, shall achieve a minimum factor of safety of 1.5 using peak strength values."

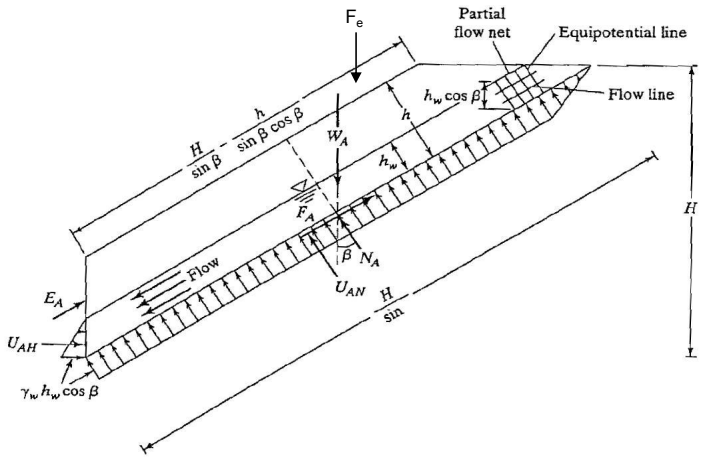
The final closure application specifications will require interface shear tests for each final cover interface. The tests will be performed on the actual materials proposed for the project. The tests will be performed before the material arrives to the project site to confirm that each interface meets the minimum required shear-strength parameters (based on the minimum interface friction angle calculated herein).

JonesEdmunds	PROJECT NUMBER:	16002-009-01		**Attachment I.1.e(3)(a)**
	PROJECT NAME:	West Pasco Ash Monofill Slope Mod & Vertical Exp		
	SUBJECT:	Veneer Slope Stability		
	BY:	C.Spreadbury	Date:	3/23/21
	CHECKED BY:	C.Sawyer	Date:	6/25/21

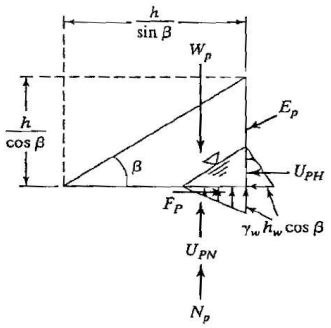
Objective: Determine the minimum interface friction angle required for a factor of safety against a veneer slope failure of 1.5.

Data: Veneer Slope Stability Active and Passive Wedges *(Reference 1)*

Active Wedge



Passive Wedge



Parallel Submergence Ratio (h_w / h)=	**50%**			
w = Natural moisture content of soil layer =	8%			*(Assuming SP or SP-SC)*
h = Thickness of the soil layer =	2.0	ft		*(Permit Drawings)*
γ_w = unit weight of water =	62.4	pcf		
G = Soil specific gravity =	2.65			*(Reference 2)*
e = Soil void ratio =	0.5			*(Reference 2)*
H = Maximum height between benches =	65	ft (vertical)		*(Permit Drawings)*
β = soil slope angle =	3	to 1 =	0.32 radians	*(Permit Drawings)*
φ = friction angle of soil =	32	degrees =	0.56 radians	*(Reference 2)*
Equipment Type =	Track-Type Tractor			*(Reference 3)*
W_e = equipment operating weight =	57,500	lbs		*(Reference 3)*
A = equipment contact area =	5,700	in^2 =	40 ft^2	*(Reference 3)*

Calculations:

1. Calculate the Soil Layer Unit Weights

$\gamma_m = (G\gamma_w)(1+w)/(1+e)$

where:		
G = Soil specific gravity =	2.65	
γ_w = unit weight of water =	62.4	pcf
w = Natural moisture content of soil layer =	8%	
e = Soil void ratio =	0.5	

γ_m = moisture unit weight of soil layer at natural moisture content = 119 pcf

$\gamma_{sat} = (G+Se)(\gamma_w)/(1+e)$

where:		
G = Soil specific gravity =	2.65	
γ_w = unit weight of water =	62.4	pcf
S = Degree of saturation =	50%	
e = Soil void ratio =	0.5	

γ_{sat} = unit weight of soil layer at specified saturation = 121 pcf

2. Calculate the Equivalent Landfill Equipment Force per Unit Width at the Geomembrane Interface

$F_e = (W_e/A) \cos \beta$

where:	W_e = equipment operating weight =	57,500	lbs
	A = equipment contact area =	40	ft^2
	β = soil slope angle =	0.32	radians
	F_e = Equipment Force per Unit Width =	1,400	lb/ft

3. Calculate the Weight of the Wedges

3a) Active Wedge

$W_A = 0.5 [(\gamma_m (h - h_w) (2H \cos\beta - h - h_w)) + (\gamma_{sat} h_w (2H \cos\beta - h_w))]/(\sin\beta \cos\beta) + F_e$

where:	γ_m = mositure unit weight of soil layer =	119	pcf
	h = Thickness of the soil layer =	2.0	ft
	h_w = Depth of the seepage water in the soil layer =	1.0	ft
	H = Maximum height between benches =	65	ft (vertical)
	β = soil slope angle	0.32	radians
	γ_{sat} = Saturated unit weight of the soil layer =	121	pcf
	F_e = Equipment Force per Unit Width =	1,400	lb/ft
W_A = weight of active wedge =		74,458	lb/ft

3b) Passive Wedge

$W_P = 0.5 [\gamma_m (h^2 - h_w^2) + \gamma_{sat} (h_w^2)] / (\sin\beta \cos\beta)$

where:	γ_m = moist unit weight of soil layer =	119	pcf
	h = Thickness of the soil layer =	2.0	ft
	h_w = Depth of the seepage water in the soil layer =	1.0	ft
	γ_{sat} = unit weight of the soil layer at 100% saturation =	121	pcf
	β = soil slope angle	0.32	radians
W_P = weight of passive wedge =		796	lb/ft

4. Calculate the Resultant Pore Water Pressures

4a) Resultant of Pore Water Pressures Acting on Bottom of Active Wedge (Perpendicular to Slope)

$U_{AN} = \gamma_w h_w (H - 0.5 h_w \cos\beta)/\tan\beta$

where:	γ_w = unit weight of water =	62.4	pcf
	h_w = Depth of the seepage water in the soil layer =	1.0	ft
	H = Maximum height between benches =	65	ft (vertical)
	β = soil slope angle	0.32	radians
U_{AN} = resultant acting on bottom of active wedge =		12,079	lb/ft

4b) Resultant of Pore Water Pressures Acting on Bottom of Passive Wedge (Perpendicular to Bottom of Passive Wedge)

$U_{PN} = 0.5 \gamma_w h_w^2 / \tan\beta$

where:	γ_w = unit weight of water =	62.4	pcf
	h_w = Depth of the seepage water in the soil layer =	1.0	ft
	β = soil slope angle	0.32	radians
U_{PN} = resultant acting on bottom of passive wedge =		94	lb/ft

4c) Resultant of Pore Water Pressures Acting on Lateral Side of Active or Passive Wedge (Perpendicular to Lateral Side)

$U_H = 0.5\ \gamma_w\ h_w^2$

where: γ_w = unit weight of water = 62.4 pcf

h_w = Depth of the seepage water in the soil layer = 1.0 ft

U_H = resultant acting on lateral side of active or passive wedge = 31 lb/ft

5. Iteratively solve for the minimum interface friction angle to meet the minimum Factor of Safety against Veneer Slope Failure

Iteratively solve for δ so that FS = 1.5:

δ = interface friction angle = 30 degrees = 0.52 radians

$$FS = \frac{-b + (b^2 - 4ac)^{0.5}}{2a}$$

where: a = 22,340

$= W_A\ sin\beta\ cos\beta + U_H\ (1 - cos^2\beta\ )$

b = -37,165

$= -[W_P\ tan\varphi + W_A\ (sin^2\beta\ tan\varphi + cos^2\beta\ tan\delta) - U_{AN}\ cos\beta\ tan\delta - U_{PN}\ tan\varphi + U_H\ sin\beta\ cos\beta\ (tan\varphi - tan\delta)]$

c = 6,681

$= (W_A\ cos\beta - U_{AN} + U_H\ sin\beta) * sin\beta\ tan\delta\ tan\varphi$

FS = Factor of Safety against Veneer Slope Failure = 1.5

Conclusion: **The design requires a minimum interface friction angle (δ) of 30 degrees to meet the minimum factor of safety against veneer slope failure of 1.5 required by Rule 62-701.400(2).**

References:

1. *Geotechnical Aspects of Landfill Design and Construction* by Qian, Koerner, Gray, 2002.
2. *Soil Mechanics,* by Lambe & Whitman, 1969.
3. *Caterpillar - D7E TRACK-TYPE TRACTOR - Specifications*

Reference 1

Geotechnical Aspects of Landfill Design & Construction, Qian, Koerner, & Gray, 2002

Reference 1

GEOTECHNICAL ASPECTS OF LANDFILL DESIGN AND CONSTRUCTION

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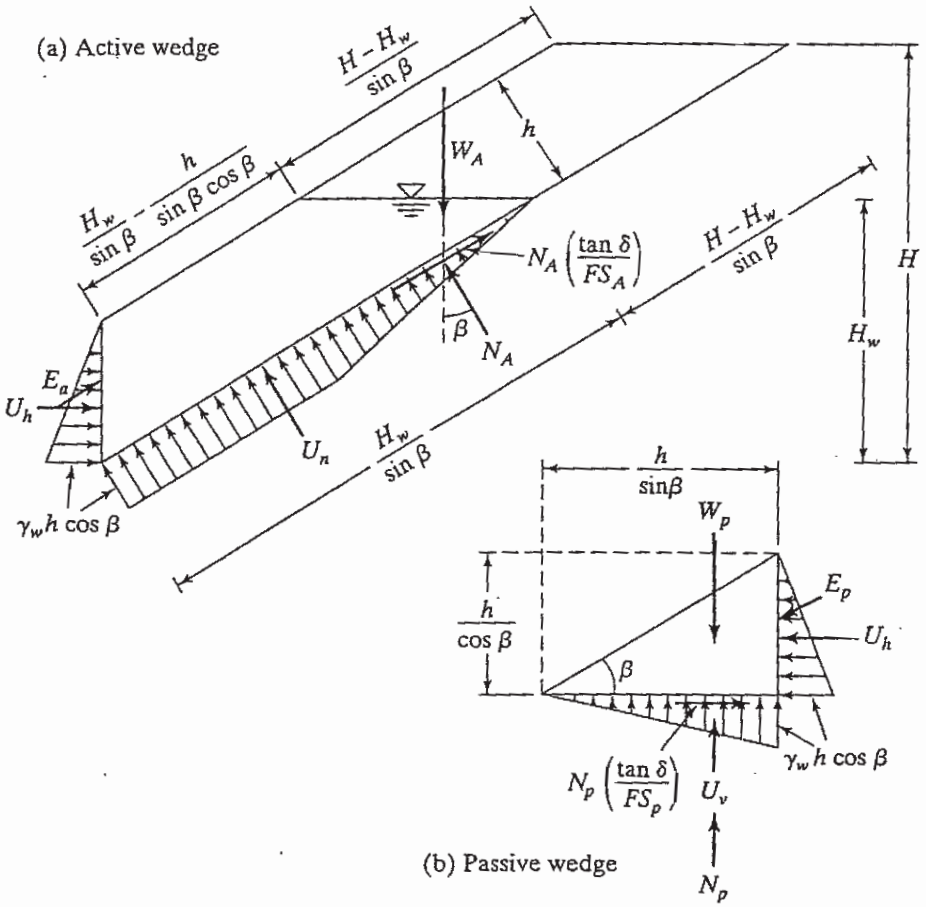


FIGURE 13.13 Limit Equilibrium Forces Involved in a Finite Length Slope of Uniform Cover Soil with Horizontal Seepage Buildup

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

$$E_A = W_A \cdot \sin\beta + U_h \cdot \cos\beta - \frac{N_A \cdot \tan\delta}{FS}$$

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

$$W_P = \frac{\gamma_{sat} \cdot h^2}{\sin 2\beta} \tag{13.18}$$

$$U_v = U_h \cdot \cot\beta \tag{13.19}$$

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

$$E_P = \frac{U_h \cdot (FS) - (W_P - U_v) \cdot \tan\phi}{\sin\beta \cdot \tan\phi - \cos\beta \cdot (FS)}$$

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

$$a \cdot FS^2 + b \cdot FS + c = 0$$

Reference 1

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

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.20}$$

where $a = W_A \cdot \sin\beta \cdot \cos\beta - U_h \cdot \cos^2\beta + U_h$

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

$c = N_A \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$

13.4.3.2 The Case of Parallel-to-Slope Seepage Buildup. Figure 13.14 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 that are not filtered. The individual forces, friction angles, and slope angles involved in Figure 13.14 are listed as follows:

W_A = weight of the active wedge (area times unit weight), lb/ft or kN/m;
W_P = weight of the passive wedge (area times unit weight), lb/ft or kN/m;
β = angle of the slope, degree;
H = height of the cover soil slope from the toe of the cover soil to the top of the slope (see Figure 13.14), ft or m;
h = thickness of the soil layer (perpendicular to the slope), ft or m;
h_w = depth of seepage water in the soil layer (perpendicular to the slope), ft or m;
γ = moisture unit weight of the soil layer, lb/ft^3 or kN/m^3;
γ_{sat} = saturated unit weight of the soil layer, lb/ft^3 or kN/m^3;
γ_w = unit weight of water, 62.4 lb/ft^3 or 9.81 kN/m^3;
ϕ = friction angle of the cover soil, degree;
δ = interface friction angle between the soil layer and geomembrane, degree;
N_A = normal force acting on bottom of the active wedge, lb/ft or kN/m;
F_A = frictional force acting on bottom of the active wedge, lb/ft;
U_{AN} = resultant of the pore water pressures acting on bottom of the active wedge (perpendicular to the slope), lb/ft or kN/m;
U_{AH} = resultant of the pore water pressures acting on lower lateral side of the active wedge (perpendicular to the interface between the active and passive wedges), lb/ft or kN/m;
E_A = force from passive wedge acting on active wedge (unknown in magnitude but assumed direction parallel to the slope), lb/ft or kN/m;
N_P = normal force acting on the bottom of passive wedge, lb/ft or kN/m;
F_P = frictional force acting on the bottom of passive wedge, lb/ft or kN/m;
U_{PH} = resultant of the pore water pressures acting on lateral side of the passive wedge (perpendicular to the lateral side), lb/ft or kN/m;

Reference 1

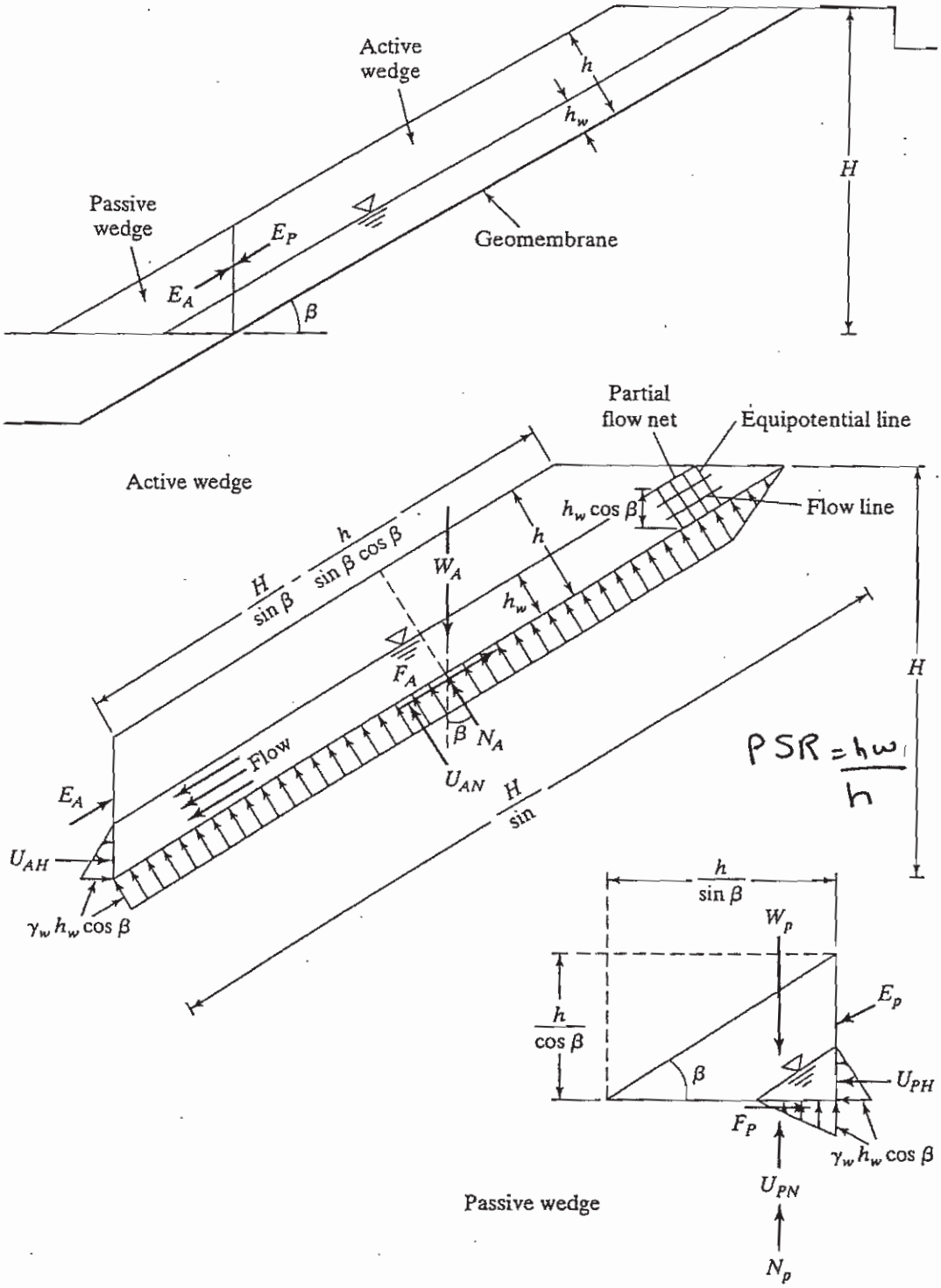


FIGURE 13.14 Cross Section of Sand Layer over Geomembrane on Side Slope with Seepage Parallel to Slope.

Reference 1

$$FS = \frac{N_P \cdot \tan\phi}{F_P} \tag{13.33}$$

Substituting Equations 13.29 and 13.32 into Equation 13.33 gives

$$FS = \frac{\begin{array}{c}(W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \cdot \tan\phi \\ - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi / FS\end{array}}{\begin{array}{c}U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta \\ - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \cdot \cos\beta \cdot \tan\delta / FS\end{array}}$$

$$\begin{aligned}&(U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta) \cdot FS - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \\ &\quad \cdot \cos\beta \cdot \tan\delta = (W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \\ &\quad \cdot \tan\phi - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi / FS\end{aligned}$$

$$\begin{aligned}&(W_A \cdot \sin\beta \cdot \cos\beta + U_{PH} - U_{AH} \cdot \cos^2\beta) \cdot FS^2 - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \\ &\quad \cdot \cos\beta \cdot \tan\delta \cdot FS = (W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \\ &\quad \cdot \tan\phi \cdot FS - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi\end{aligned}$$

$$\begin{aligned}&(W_A \cdot \sin\beta \cdot \cos\beta + U_{PH} - U_{AH} \cdot \cos^2\beta) \cdot FS^2 - [W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi \\ &+ \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_{AH} \cdot \sin\beta \cdot \cos\beta \\ &\cdot (\tan\phi - \tan\delta)] \cdot FS + (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi = 0\end{aligned} \tag{13.34}$$

Because $U_H = U_{PH} = U_{AH}$,

$$\begin{aligned}&[W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta)] \cdot FS^2 - [W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi \\ &+ \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \\ &\cdot (\tan\phi - \tan\delta)] \cdot FS + (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi = 0\end{aligned} \tag{13.35}$$

Using $a \cdot x^2 + b \cdot x + c = 0$

The resulting FS can be expressed as

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.36}$$

where

Parallel

$$\begin{aligned}a &= W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta) \\ b &= -[W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta \\ &\quad - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)] \\ c &= (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi\end{aligned}$$

$$U_{AN} = \gamma_w \cdot h_w \cdot (H - 0.5\, h_w \cdot \cos\beta)/\tan\beta \tag{13.37}$$

$$U_H = 0.5 \cdot \gamma_w \cdot h_w^2 \tag{13.38}$$

$$U_{PN} = 0.5 \cdot \gamma_w \cdot h_w^2/\tan\beta \tag{13.39}$$

$$\begin{aligned}W_A = 0.5 \cdot [&\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) \\ &+ \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta)\end{aligned} \tag{13.40}$$

$$W_P = 0.5 \cdot [\gamma \cdot (h^2 - h_w^2) + \gamma_{sat} \cdot h_w^2]/(\sin\beta \cdot \cos\beta) \tag{13.41}$$

Reference 1

U_H = resultant of the pore water pressures acting on lateral side of the active wedge or passive wedge (perpendicular to the lateral side), lb/ft or kN/m, $U_H = U_{AH} = U_{PH}$;

U_{PN} = resultant of the pore water pressures acting on bottom of the passive wedge (perpendicular to bottom of the passive wedge), lb/ft or kN/m;

E_P = force from active wedge acting on passive wedge (unknown in magnitude but assumed direction parallel to the slope), lb/ft or kN/m, $E_A = E_P$;

FS = factor of safety for stability of the cover soil mass.

Considering the force equilibrium of the active wedge (Figure 13.14), we obtain

$\Sigma F_Y = 0$:
$$N_A + U_{AN} = W_A \cdot \cos\beta + U_{AH} \cdot \sin\beta$$
$$N_A = W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta \quad (13.21)$$

$\Sigma F_X = 0$:
$$F_A + E_A + U_{AH} \cdot \cos\beta = W_A \cdot \sin\beta$$
$$E_A = W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - F_A \quad (13.22)$$
$$F_A = N_A \cdot \tan\delta / FS \quad (13.23)$$

Substituting Equation 13.21 into Equation 13.23 gives

$$F_A = (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta / FS \quad (13.24)$$

Substituting Equation 13.24 into Equation 13.22 gives

$$E_A = W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta / FS \quad (13.25)$$

Considering the force equilibrium of the passive wedge (Figure 13.14) yields

$$E_P = E_A \quad (13.26)$$

$\Sigma F_Y = 0$:
$$N_P + U_{PN} = W_P + E_P \cdot \sin\beta \quad (13.27)$$

Substituting Equation 13.26 into Equation 13.27 gives

$$N_P = W_P + E_A \cdot \sin\beta - U_{PN} \quad (13.28)$$

Substituting Equation 13.25 into Equation 13.28 gives

$$N_P = W_P - U_{PN} + [W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta / FS] \cdot \sin\beta$$

$$N_P = W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta / FS \quad (13.29)$$

$\Sigma F_X = 0$:
$$F_P = U_{PH} + E_P \cdot \cos\beta \quad (13.30)$$

Substituting Equation 13.26 into Equation 13.30 gives

$$F_P = U_{PH} + E_A \cdot \cos\beta \quad (13.31)$$

Substituting Equation 13.25 into Equation 13.31 gives

$$F_P = U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \cdot \cos\beta \quad (13.32)$$

Reference 2

Soil Mechanics, Lambe & Whitman, 1969

Reference 2

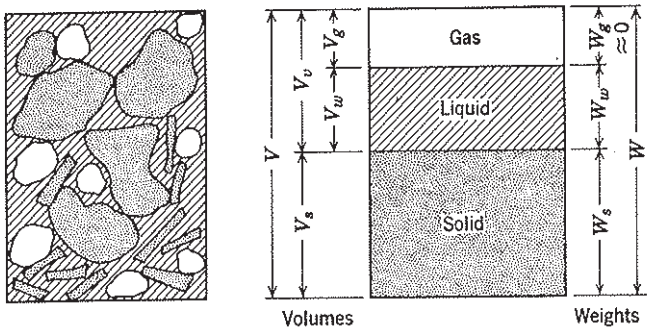


Fig. 3.1 Relationships among soil phases. (*a*) Element of natural soil. (*b*) Element separated into phases.

Volume

Porosity:

$$n = \frac{V_v}{V}$$

Void ratio:

$$e = \frac{V_v}{V_s}$$

Degree of saturation:

$$S = \frac{V_w}{V_v}$$

$$n = \frac{e}{1 + e}; \qquad e = \frac{n}{1 - n}$$

Weight

Water content:

$$w = \frac{W_w}{W_s}$$

Specific Gravity

Mass:

$$G_m = \frac{\gamma_t}{\gamma_0}$$

Water:

$$G_w = \frac{\gamma_w}{\gamma_0}$$

Solids:

$$G = \frac{\gamma_s}{\gamma_0}$$

γ_0 = Unit weight of water at 4°C $\approx \gamma_w$

Note that $Gw = Se$

Unit Weight

Total:

$$\gamma_t = \frac{W}{V} = \frac{G + Se}{1 + e}\gamma_w = \frac{1 + w}{1 + e}G\gamma_w$$

Solids:

$$\gamma_s = \frac{W_s}{V_s}$$

Water:

$$\gamma_w = \frac{W_w}{V_w}$$

$e = \left(\frac{G}{\gamma_d}\gamma_w\right) - 1$

Dry:

$$\gamma_d = \frac{W_s}{V} = \frac{G}{1 + e}\gamma_w = \frac{G\gamma_w}{1 + wG/S} = \frac{\gamma_t}{1 + w}$$

Submerged (buoyant):

$$\gamma_b = \gamma_t - \gamma_w = \frac{G - 1 - e(1 - S)}{1 + e}\gamma_w$$

Submerged (saturated soil):

$$\gamma_b = \gamma_t - \gamma_w = \frac{G - 1}{1 + e}\gamma_w$$

Specific gravity is the unit weight divided by the unit weight of water. Values of specific gravity of solids G for a selected group of minerals[4] are given in Table 3.1.

Table 3.1 Specific Gravities of Minerals

Quartz	2.65
K-Feldspars	2.54–2.57
Na–Ca-Feldspars	2.62–2.76
Calcite	2.72
Dolomite	2.85
Muscovite	2.7–3.1
Biotite	2.8–3.2
Chlorite	2.6–2.9
Pyrophyllite	2.84
Serpentine	2.2–2.7
Kaolinite	2.61[a]
	2.64 ± 0.02
Halloysite (2 H_2O)	2.55
Illite	2.84[a]
	2.60–2.86
Montmorillonite	2.74[a]
	2.75–2.78
Attapulgite	2.30

[a] Calculated from crystal structure.

The expression $Gw = Se$ is useful to check computations of the various relationships.

The student in soil mechanics must understand the meanings of the relationships in Fig. 3.1, convince himself once and for all that they are correct, and add these terms to his active vocabulary. These relationships are basic to most computations in soil mechanics and thus are an essential part of soil mechanics.

Typical Values of Phase Relationships for Granular Soils

Figure 3.2 shows two of the many possible ways that a system of equal-sized spheres can be packed. The dense packings represent the densest possible state for such a system. Looser systems than the simple cubic packing can be obtained by carefully constructing arches within the packing, but the simple cubic packing is the loosest of the stable arrangements. The void ratio and porosity of

[4] Chapter 4 discusses the common soil minerals.

Source: T. William Lambe and Robert V. Whitman (1979) Soil Mechanics, John Wiley & Sons, New York.

Reference 2

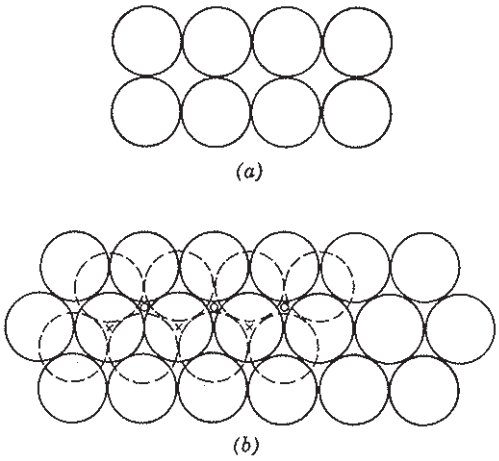


Fig. 3.2 Arrangements of uniform spheres. (*a*) Plan and elevation view: simple cubic packing. (*b*) Plan view: dense packing. Solid circles, first layer; dashed circles, second layer; ∘, location of sphere centers in third layer: face-centered cubic array; ×, location of sphere centers in third layer: 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

Description	Void Ratio		Porosity (%)		Dry Unit Weight (kN/m³)	
	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	14.5	17.3
Clean uniform sand	1.0	0.40	50	29	13.0	18.5
Uniform inorganic silt	1.1	0.40	52	29	12.6	18.5
Silty sand	0.90	0.30	47	23	13.7	20.0
Fine to coarse sand	0.95	0.20	49	17	13.4	21.7
Micaceous sand	1.2	0.40	55	29	11.9	18.9
Silty sand and gravel	0.85	0.14	46	12	14.0	22.9

After B. K. Hough, *Basic Soils Engineering*. Copyright © 1957, The Ronald Press Company, New York.

Mining Sand
Cemented Sand
Variable Sand

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

Source: T. William Lambe and Robert V. Whitman (1979) Soil Mechanics, John Wiley & Sons, New York.

Reference 3

Caterpillar – D7E
Track-Type Tractor Specifications

D7E

Track-Type Tractor

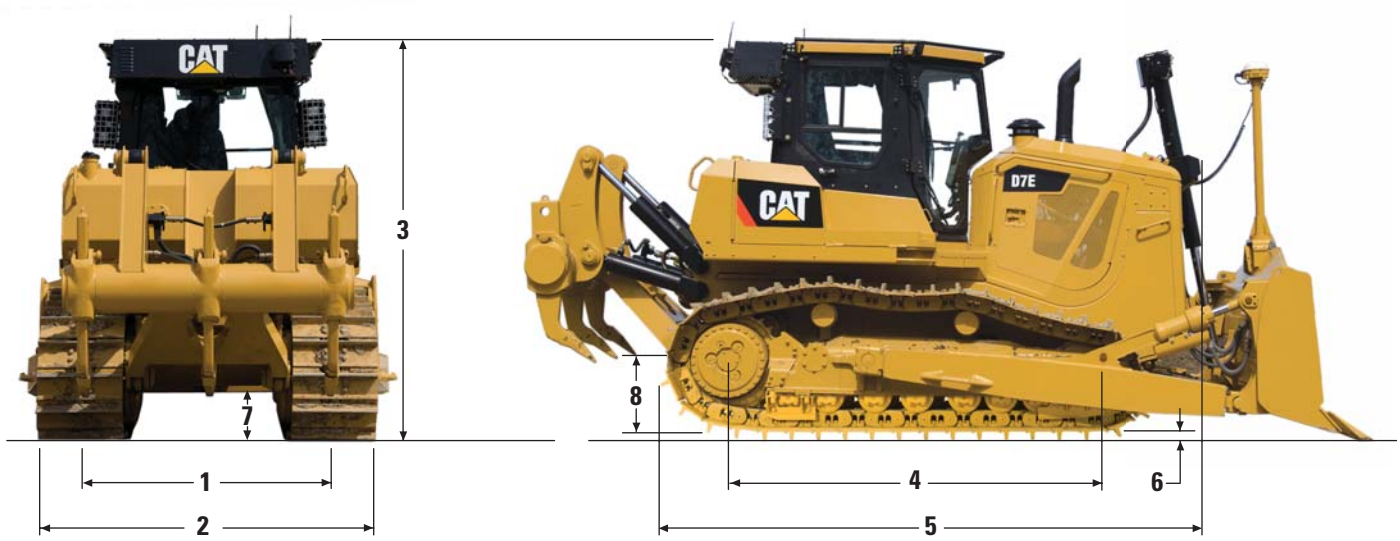
Reference 3

Engine		
Engine Model	Cat® C9.3 ACERT™	
Global Emissions	U.S. Tier 4 Interim/EU Stage IIIB/ Japan MLIT Step 4	
Engine Power (Maximum)		
ISO 14396	198 kW	266 hp
ISO 14396 (DIN)		270 hp
Net Power (Rated)		
ISO 9249/SAE J1349	175 kW	235 hp
ISO 9249/SAE J1349 (DIN)		238 hp
Net Power (Maximum)		
ISO 9249/SAE J1349	187 kW	251 hp
ISO 9249/SAE J1349 (DIN)		254 hp

Drive Train		
Type	Electric Drive	
Weights		
Operating Weight – STD SU	26 055 kg	57,441 lb
Operating Weight – LGP S	28 525 kg	62,886 lb
Shipping Weight – STD	21 955 kg	48,402 lb
Shipping Weight – LGP	24 335 kg	53,649 lb

Dimensions

All dimensions are approximate



		STD		LGP	
1	Track Gauge	1981 mm	78 in	2286 mm	90 in
2	Width of Tractor over Trunnions	2880 mm	113 in	3423 mm	135 in
	Width of Tractor without Trunnions (std. shoes)	2591 mm	102 in	3200 mm	126 in
3	Machine Height from Tip of Grouser				
	Top of Stack	3365 mm	132 in	3365 mm	132 in
	Top of Standard Cab	3392 mm	134 in	3392 mm	134 in
	From Ground Face of Shoe	3322 mm	131 in	3322 mm	131 in
4	Length of Track on Ground	3016 mm	119 in	3450 mm	136 in
5	Length of Basic Tractor	4608 mm	181 in	4608 mm	181 in
	With the following attachments add to basic tractor length:				
	Ripper (with tip at ground line)	1391 mm	55 in	N/A	
	Ripper (with tip fully raised)	1222 mm	48 in	N/A	
	Winch	1032 mm	41 in	1032 mm	41 in
	Drawbar	270 mm	10.6 in	270 mm	10.6 in
	S Blade	977 mm	38 in	N/A	
	SU Blade	1187 mm	47 in	N/A	
	U Blade	1425 mm	56 in	N/A	
	A Blade	1230 mm	48 in	1230 mm	48 in
6	Height of Grouser	70 mm	2.75 in	70 mm	2.75 in
7	Ground Clearance	472 mm	18.6 in	472 mm	18.6 in
	Ground Contact Area (std. shoes)	3.68 m^2	5,698 in^2	6.31 m^2	9,792 in^2
	Number of Shoes per Side	40		44	
	Standard Shoe Width and Type	610 mm	24 in	915 mm	36 in
		MS		MS	
	Ground Pressure	69.5 kPa	10.1 psi	44.3 kPa	6.4 psi
	Pitch	215.9 mm	8.5 in	215.9 mm	8.5 in
	Track Rollers/Side	7		8	
	Number of Carrier Rollers	2		2	
8	Drawbar Height (grouser tip to center of clevis)	719 mm	28 in	719 mm	28 in

Attachment I.1.e(3)(b)

Global Slope Stability

TECHNICAL MEMORANDUM

West Pasco County Ash Monofill Slope Modification and Vertical Expansion

TO: Project File

FROM: Chad Spreadbury, PhD, EI

XC: Carol Sawyer, PE

DATE: June 29, 2021

SUBJECT: Attachment I.1.e(3)(b) – Global Slope Stability Analysis
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

This Technical Memorandum summarizes the global slope stability analysis of the proposed Ash Monofill Slope modification and vertical expansion. This analysis evaluates the potential for failure through the waste mass and along the bottom liner system. This memorandum describes the methodology, design parameters, and analysis results to demonstrate that the global slope stability of the system will meet the minimum required factor of safety for waste and foundation stability of 1.5, as required by FDEP Rule 62-701.400(2), FAC, after closure of the landfill.

2 METHODOLOGY

The global slope stability analysis is performed to determine the minimum required bottom liner interface friction angle to meet the minimum required factor of safety against slope failure of 1.5.

The following steps were taken to calculate the slope stability of the Class I expansion:

- Establish a software model and methodology for calculating the factor of safety against waste mass and bottom liner failure.
- Establish the critical cross-section to be analyzed based on the steepest slopes and peak landfill elevations.
- Analyze the factor of safety of global slope stability based on the existing bottom liner interface friction angle for the proposed slope modification and vertical expansion. The minimum factor of safety should be 1.5.

2.1 Software Model and Methodology

The slope stability analyses were performed using Geo-Slope International's SLOPE/W software. SLOPE/W is a software product that uses limit equilibrium theory to compute the factor of safety of earth and rock slopes. Our analyses used the Spencer method, which formulates moment and force equilibrium factors of safety.

A sliding block failure surface was analyzed through the bottom liner system. The sliding block failure analysis is typical of masses with weak planes, and provides the most critical failure scenario. SLOPE/W uses a left block and right block of failure centers to model the slip surface with the lowest factor of safety. The sliding block failure surface is modeled by specifying a range of points at the two lower corners of the slip surface (grid blocks).

Numerous iterations of the model are performed until a stable and repeatable minimum factor of safety is determined. A factor of safety of 1.0 indicates impending instability, and a factor of safety of 1.5 is generally accepted as adequate for global stability of landfill slopes.

2.2 Critical Cross-Section

One critical cross-section was chosen based on its peak elevation, greatest bottom liner depth, and nearness to the site stormwater features (Attachment I.1, Figure I-3 – Section A-A', using a constant 3H:1V final cover slope).

2.3 Bottom Liner Stability

To determine the minimum interface friction angle (δ) of the bottom liner system, a global slope stability analysis was done at the critical cross section for waste peak height using the sliding block analysis. When using this analysis, a failure plane through the bottom liner system is defined. Using an iterative process that varies the locations of the failure nodes, the minimum factor of safety of 1.5 is determined. The results of the analysis are attached.

Laboratory shear tests will be performed on actual landfill bottom liner materials to ensure the minimum interface friction angle is achieved.

3 PARAMETERS

The slope stability analyses were performed using values of soil strength parameters from testing results and literature for the foundation soils, liner system material, waste, and cover soils. The groundwater table is modeled at the SHGWT elevation.

The material strength parameters used in the analyses are summarized in Attachment I.1.a(6) and are shown on slope stability analysis report printout. The geometry is based on Attachment I.1, Figure I-3 (Section A-A').

4 RESULTS

The results of the global slope stability analysis and analysis report are attached. Also attached are the interface friction shear tests performed on samples from the bottom liner materials from the Class I (MSW) Cells SW-1 and SW-2, which can be expected to be similar

to those used for the existing ash cells on-site. These bottom liner materials were sampled by excavating the cover soil at the bottom liner berm edge, outside the limits of waste. The minimum bottom liner interface friction angle at peak strength is 19.3 degrees, between the underlying soil and the smooth geomembrane liner (most conservative liner design).

The slope stability analysis is for the modified final closure through the critical cross-section meets the required minimum factor of safety of 1.5 for landfill slopes based on the given material strength parameters. This meets the requirements of Rule 62-701.400(2), FAC, "designs to prevent failures of side slopes, and designs to prevent deep-seated failures through the waste, along liner systems, and through foundation soils, shall achieve a minimum factor of safety of 1.5 using peak strength values."

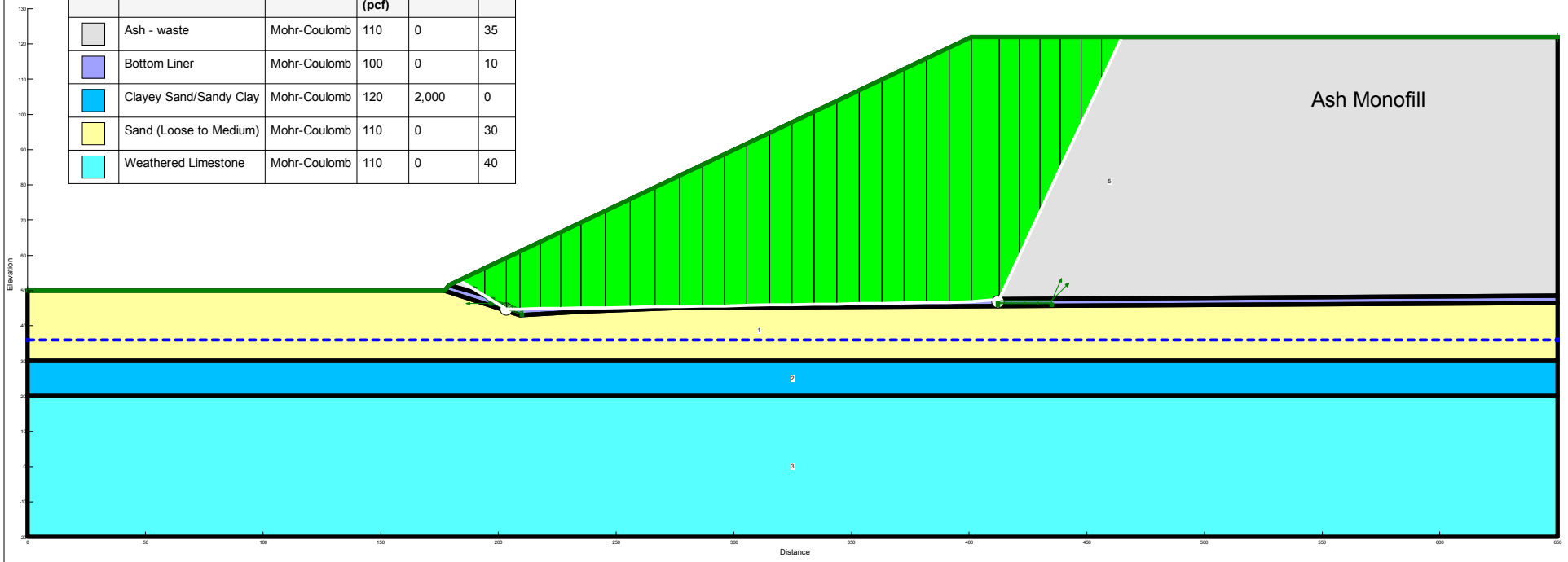
Date: 05/04/2021
Title: Pasco - Ash Monofill Vertical Exp
File Name: CellA-4-SlopeStability-block.gsz

JE Project No.: 16002-009-01

Notes:
1. Prpoed vertical expansion to elevation 122 feet with modified slopes of 3H:1V.
2. Section through Cell A-4 (west side).
3. SHWT at elevation 36 feet.
4. Subsurface profile based on 2021 borings by PSI and historical borings.
5. This is a block analysis assuming failure through the bottom liner.

FS= 1.5

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Ash - waste	Mohr-Coulomb	110	0	35
	Bottom Liner	Mohr-Coulomb	100	0	10
	Clayey Sand/Sandy Clay	Mohr-Coulomb	120	2,000	0
	Sand (Loose to Medium)	Mohr-Coulomb	110	0	30
	Weathered Limestone	Mohr-Coulomb	110	0	40



SLOPE/W Analysis

Report generated using GeoStudio 2019. Copyright © 1991-2018 GEOSLOPE International Ltd.

File Information

File Version: 10.00
Title: Pasco - Ash Monofill Vertical Exp
Created By: Carol G. Sawyer, PE
Last Edited By: Carol G. Sawyer, PE
Revision Number: 39
Date: 05/04/2021
Time: 01:31:20 PM
Tool Version: 10.0.0.17401
File Name: CellA-4-SlopeStability-block.gsz
Directory: Y:\16002-PascoCounty\Projects\009-01-AshMonofillOperationsModification\Reports\Task 2 - Preliminary Analysis\Slope Stability\

Project Settings

Unit System: U.S. Customary Units

Analysis Settings

SLOPE/W Analysis

Description: Pasco - Ash Monofill Vertical Expansion
Kind: SLOPE/W
Method: Morgenstern-Price
Settings
- Side Function
  - Interslice force function option: Half-Sine
- PWP Conditions from: Piezometric Line
  - Apply Phreatic Correction: No
  - Use Staged Rapid Drawdown: No
- Unit Weight of Water: 62.430189 pcf

Slip Surface
- Direction of movement: Right to Left
- Use Passive Mode: No
- Slip Surface Option: Block
- Critical slip surfaces saved: 1
- Restrict Block Crossing: No
- Optimize Critical Slip Surface Location: No
- Tension Crack Option: (none)

Distribution
- F of S Calculation Option: Constant

Advanced
- Geometry Settings

Minimum Slip Surface Depth: 0.1 ft
Number of Slices: 30
Factor of Safety Convergence Settings
Maximum Number of Iterations: 100
Tolerable difference in F of S: 0.001
Solution Settings
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Ash - waste

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 35 °
Phi-B: 0 °

Sand (Loose to Medium)

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 30 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Clayey Sand/Sandy Clay

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 2,000 psf
Phi': 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Weathered Limestone

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 40 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Bottom Liner

Model: Mohr-Coulomb

Unit Weight: 100 pcf
Cohesion': 0 psf
Phi': 10 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (0, 50) ft
Right Coordinate: (650, 122) ft

Slip Surface Block

Left Grid
Upper Left: (197.47434, 47.51318) ft
Lower Left: (196.76219, 46.22452) ft
Lower Right: (209.91154, 43.17245) ft
X Increments: 4
Y Increments: 4
Starting Angle: 135 °
Ending Angle: 180 °
Angle Increments: 2
Right Grid
Upper Left: (412.28244, 47.19523) ft
Lower Left: (412.28244, 45.93371) ft
Lower Right: (435.04282, 45.93371) ft
X Increments: 4
Y Increments: 4
Angle Increments: 2

Piezometric Lines

Piezometric Line 1

Coordinates

	X	Y
Coordinate 1	0 ft	36 ft
Coordinate 2	650 ft	36 ft

Points

	X	Y
Point 1	0 ft	50 ft
Point 2	177 ft	50 ft
Point 3	401 ft	122 ft

Point 4	650 ft	122 ft
Point 5	209 ft	43 ft
Point 6	236 ft	44 ft
Point 7	277 ft	45 ft
Point 8	650 ft	46.5 ft
Point 9	0 ft	30 ft
Point 10	650 ft	30 ft
Point 11	0 ft	20 ft
Point 12	650 ft	20 ft
Point 13	0 ft	-20 ft
Point 14	650 ft	-20 ft
Point 15	179 ft	51.5 ft
Point 16	192 ft	49 ft
Point 17	209 ft	45 ft
Point 18	235 ft	46 ft
Point 19	277 ft	47 ft
Point 20	650 ft	48.5 ft

Regions

	Material	Points	Area
Region 1	Sand (Loose to Medium)	1,2,5,6,7,8,10,9	10,902 ft²
Region 2	Clayey Sand/Sandy Clay	9,10,12,11	6,500 ft²
Region 3	Weathered Limestone	11,12,14,13	26,000 ft²
Region 4	Bottom Liner	2,15,16,17,18,19,20,8,7,6,5	948.75 ft²
Region 5	Ash - waste	15,3,4,20,19,18,17,16	27,238 ft²

Attachment I.1.f

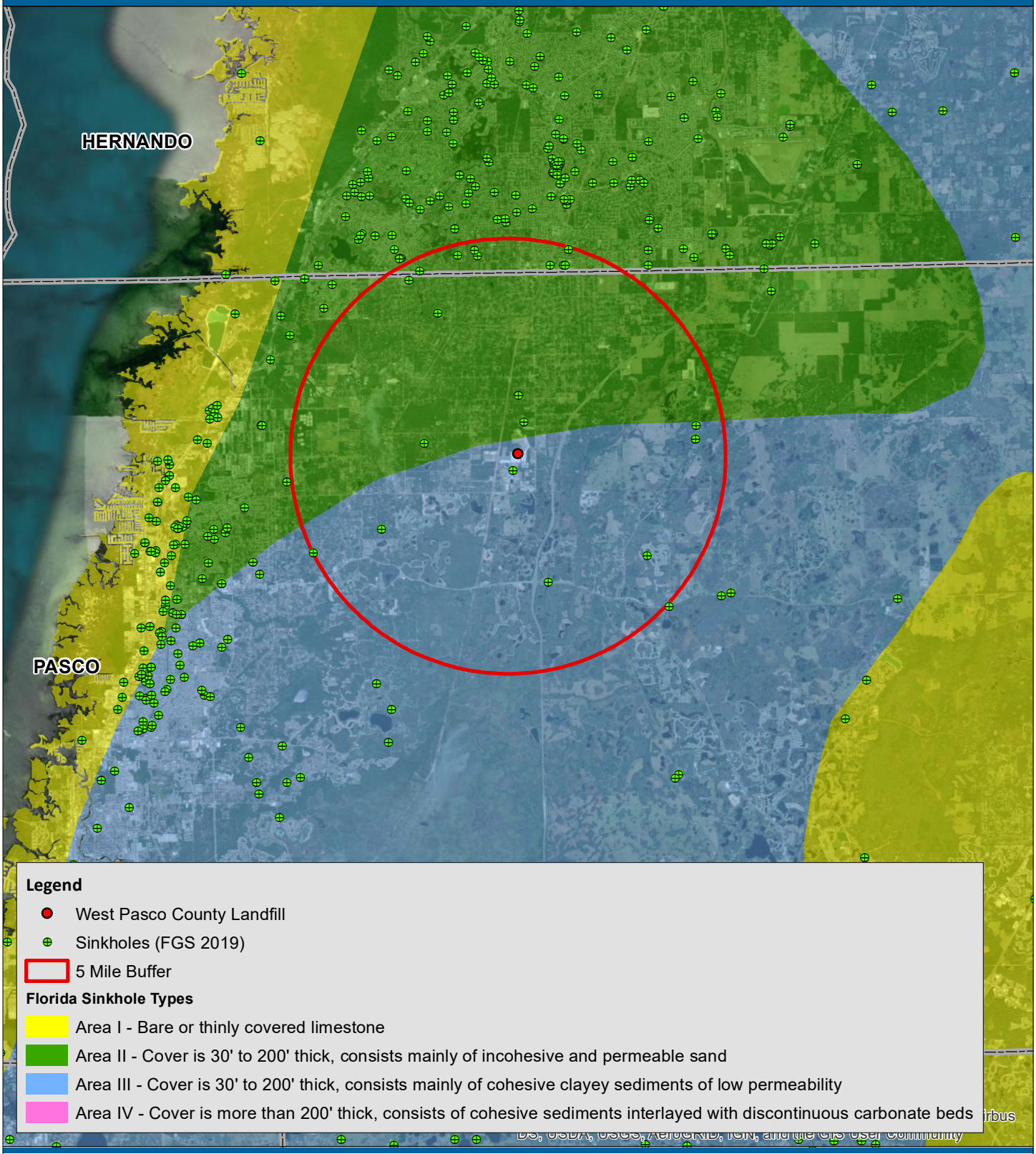
Figure I.1.f, Sinkhole Map

and

The Sinkhole Type, Development, and Distribution Map of Florida
USGS, 1985

Sinkhole Map

West Pasco County Landfill



JonesEdmunds

For Informational Purposes Only Y:\16002-PascoCounty\Projects\002-01-ClassISlopeMod\GIS\mxd\Pasco LF Sinkhole Map MMorse 2018-12-21.mxd JReynolds 5/6/2021

MAP SERIES NO. 110

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

FLORIDA DEPARTMENT OF NATURAL RESOURCES
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SINKHOLE TYPE, DEVELOPMENT, AND DISTRIBUTION IN FLORIDA

By
William C. Sinclair and J.W. Stewart

Prepared by the
U.S. GEOLOGICAL SURVEY

in cooperation with the
FLORIDA DEPARTMENT OF ENVIRONMENTAL REGULATION
BUREAU OF WATER RESOURCES MANAGEMENT
and the
FLORIDA DEPARTMENT OF NATURAL RESOURCES
BUREAU OF GEOLOGY

Tallahassee, Florida
1985

DEPARTMENT OF NATURAL RESOURCES
BUREAU OF GEOLOGY

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INTRODUCTION

Sinkholes are a natural and common geologic feature in areas underlain by limestone and other rock types that are soluble in natural water. The term sinkhole is used for closed depressions in the land surface that are formed by surficial solution or by subsidence or collapse of surficial materials owing to the solution of near-surface limestone or other soluble rocks. Discussion here refers to sinkhole occurrence in limestone and dolomite, the most common rock types in Florida.

Sinkholes occur in a variety of shapes from steep-walled "natural wells" to funnel-shaped or bowl-shaped depressions. Sinkholes are a product of solution-erosion processes and are analogous to valleys carved by rivers in areas underlain by insoluble clastic rocks. Just as rivers constantly erode the land surface by carrying away particles of rocks a grain at a time, limestone is carried slowly away in chemical solution an ion at a time. The ions are carried by water that percolates through cavities and conduits that were developed by solution of the limestone along fracture systems, bedding planes, and other permeable zones in the rock. Surface erosion by rivers is well understood and subject to some control by man, but the detection of cavities at depth and prediction of potential sinkhole collapse is inherently difficult.

Sinkholes are of interest in Florida because they are one of the predominant landform features of the State; because their development may be sudden, resulting in possible loss of life and property; because they may cause flooding during storms when the drainage capacity of natural subsurface conduits is exceeded; and because they may provide an avenue for pollutants on the land surface to move rapidly into the underlying limestone and dolomite—the source of more than 80 percent of the drinking water in the State. The purpose of this report is to describe the geologic and hydrologic features controlling the development of sinkholes so that sinkholes and their related problems may be better understood.

FACTORS THAT AFFECT SOLUTION OF LIMESTONE

The ultimate cause of collapse or subsidence that forms virtually all sinkholes in Florida is solution of soluble rock by acidic water. As rain falls through the atmosphere, it dissolves carbon dioxide and other gases and becomes slightly acidic. Rainwater percolating through the soil zone continues to dissolve carbon dioxide that is released by bacterial action and decomposition of organic material. The amount of carbon dioxide available in soils is 10 to 100 times greater than that available in the atmosphere (Heen, 1960, p. 35).

Water percolating through the soil zone becomes a weak carbonic acid that reacts readily with limestone and dolomite as it moves downward. The reactions are:

$$CO_2 + H_2O \rightleftharpoons H_2CO_3$$
(carbon dioxide) (water) (carbonic acid)

$$H_2CO_3 + CaCO_3 \rightleftharpoons Ca^{++} + 2HCO_3^-$$
(carbonic acid) (limestone) (calcium ions) (bicarbonate ions)

$$2H_2CO_3 + (CaMg(CO_3)_2) \rightleftharpoons Ca^{++} + Mg^{++} +$$
(carbonic acid) (dolomite) (calcium ions) (magnesium ions)

$$4HCO_3^-$$
(bicarbonate ions)

As the reaction proceeds, water gradually assimilates calcium and bicarbonate ions until it is saturated and further solution of limestone is not possible. The reaction is reversible depending on such variables as temperature and carbon dioxide pressure. Thus, the same reaction that dissolves limestone may deposit a scale of calcium carbonate (limestone) in a teapot when water is boiled. Under natural conditions, the reaction can reverse, resulting in deposition, such as the formation of stalactites and the sealing of voids and fractures by precipitation of carbonate minerals.

The chemistry of rocks is important in controlling the development of solution cavities. Pure limestone ($CaCO_3$) is commonly more easily dissolved by natural water than is dolomite ($CaMgCO_3)_2$), the second most prevalent rock in Florida. The presence of impurities, such as sand and clay, within the rock will reduce the rate of carbonate solution. The insoluble clay particles commonly form an impermeable residuum that mantles the limestone and retards infiltration of water, thus reducing further solution.

The reaction between water and rocks is controlled by the rate of circulation of water and this, in turn, is controlled by the rate of recharge and by the rate of ground-water flow. Solution is most intense where the corrosive water first contacts the limestone surface and along joints or vertical fracture zones within the limestone. Solution of limestone is also enhanced where ground-water flow is concentrated at depths below the limestone surface. For example, in areas where the water table is below the limestone surface, ground water is chemically most active in the zone of fluctuation of the water table. Consequently, solution of limestone is usually accelerated in this zone, causing still more vigorous ground-water circulation. Solution may also be accelerated along unconformities, bedding planes, and other changes in rock structure and texture where flow is concentrated if the water is chemically active.

The relation of sea level to land surface has been the dominant factor in the geologic history of Florida. Periods of submergence of the land and deposition of limestone have alternated with periods of exposure of the land and dissolution of the carbonate rocks. Table 1 summarizes the major geologic units deposited over the last 60 million years.

Fluctuations in sea level have had an important effect on hydrologic conditions and on development of sinkholes throughout the State. The central ridges that parallel Florida's east coast are capped by a series of relict beaches that formed from about middle to late Miocene time. These beaches have stood above the sea for 5 to 10 million years. The many large sinkhole lakes and internally drained depressions in the central ridges attest to a long, uninterrupted period of weathering, solution, and subsidence of the underlying limestone bedrock.

More recent (Pleistocene) fluctuations of sea level, associated with the advance and retreat of glaciers, have ranged from 400 feet below to about 100 feet above the present sea level. In response, ground-water levels have also fluctuated, and factors that affect solution of the limestone and development of sinkholes have varied.

During the warm interglacial period prior to the last major glaciation, the sea rose to about 100 feet above its present level. Wave action and coastal currents swept surficial sand and clay into land-surface depressions and produced a relatively subdued topography that consisted of beaches, dunes, and terraces that mantled an irregular bedrock surface. Many sinkholes, lakes, and springs were effectively buried, but major topographic features, such as bedrock ridges and river valleys, are discernible through the surficial mantle.

As the last glacial expansion kept progressively more of the world's water frozen on the land, the sea declined 300 to 400 feet below its present level. Florida's land area increased by more than a factor of two as the coasts moved seaward and gulfward to the edges of the continental shelf. This change in sea level lowered the base level toward which surface runoff and ground water would flow and affected rates of recharge, levels of the water table, and even direction of ground-water flow in some areas. As the glaciers waned, the sea gradually rose to its present level, and Florida's hydrologic system developed into its present state. However, the system is controlled to some extent by topography and solution conduits that were formed under varied conditions over geologic time.

Although Florida is underlain by thousands of feet of limestone and dolomite that are susceptible to solution by ground water, most of the State is covered by a mantle of insoluble clastic rock material, such as sand and clay, that control, to a great extent, the ultimate route of water that falls as rain. In areas where limestone is exposed at land surface, or is thinly covered, rainfall infiltrates directly through a network of conduits that have been opened and enlarged by solution of the rock. Where the limestone is covered by permeable sand, however thick, the rainwater is largely held where it falls and infiltrates downward through the sand into the limestone below.

Surface runoff is uncommon where rainfall moves directly into the aquifers. Streams and rivers are widely separated, and valleys that were developed by occasional flood runoff are poorly defined. Conversely, in areas where limestone is covered by thick and impermeable clay layers, downward movement of water is retarded or impeded, and well-defined stream channels receive surface runoff and drain a high water table through a closely spaced network of tributaries.

TYPES OF SINKHOLES

Three major types of sinkholes common to Florida are solution sinkholes, cover-collapse sinkholes, and cover-subsidence sinkholes. These sinkhole types are distinguished by their mode of formation; the type of sinkhole that may develop in a given area is controlled largely by the geology and hydrology.

Solution Sinkholes

Solution sinkholes occur in areas where limestone is exposed at land surface or is covered by thin layers of soil and permeable sand. Solution is most active at the limestone surface and along joints, fractures, or other openings in the rock that permit water to move easily into the subsurface. Dissolved limestone and some insoluble residue are carried downward by the percolating water along enlarged openings as solution of the limestone progresses. Large voids commonly do not form because subsidence of the soil layer occurs as the limestone surface dissolves. The result is a gradual downward movement of the land surface and development of a depression that collects increasing amounts of surface runoff as its perimeter expands. This type of sinkhole usually forms as a bowl-shaped depression with the slope of its sides determined by the rate of subsidence relative to the rate of erosion of the walls of the depression from surface runoff. Surface runoff may also carry sand and clay particles into the depression, which may form a relatively impermeable seal in the bottom. A marsh or lake forms when water is ponded because infiltration is restricted by the clayey seal. The gently rolling hills and shallow depressions typical of solution-subsidence topography are common over large parts of Florida.

Cover-Collapse Sinkholes

Cover-collapse sinkholes occur where a solution cavity develops in the limestone to a size such that the overlying cover material can no longer support its own weight. Collapse is generally abrupt when this occurs and is sometimes catastrophic. Collapse sinkholes provide dramatic local changes in topography. They may occur in any area of soluble rock; however, they are less likely to occur in areas of deeply buried rocks. Collapse sinkholes generally occur in areas where the limestone is near land surface and the limestone aquifer is under water-table conditions. Ground-water circulation is most vigorous at and just below the water table where solution of the limestone is accelerated. Accelerated solution also may occur at certain depths where bedding planes in the limestone or changes in rock composition concentrate the flow of ground water.

Limestone is commonly exposed in the vertical or overhanging walls of collapse sinkholes shortly after they form. The sinkholes generally are circular in shape and the walls may be round and smooth, but mostly they are irregular in shape because of the influence of joints and fractures in the rock. Surface drainage, erosion, and deposition of debris in collapse sinkholes will eventually smooth the sides and reduce their slopes until they may become indistinguishable from other types of sinkholes.

Cover-Subsidence Sinkholes

Limestone, like most bedrock, generally lies beneath a cover of soil and other unconsolidated material, such as sand and clay. Coastal sediments and beach deposits left during periods of high sea level occur throughout Florida at various altitudes. The variable thickness and composition of this cover is important in sinkhole development.

Cover-subsidence sinkholes occur where the cover material is relatively incohesive and permeable, and individual grains of sand move downward in sequence to replace grains that have themselves moved downward to occupy space formerly held by the dissolved limestone. In areas where the sand cover is 50 to 100 feet thick, subsidence sinkholes generally are only a few feet in diameter and depth.

Where the limestone is buried beneath a sufficient thickness of unconsolidated material, few sinkholes generally occur. Spalling of sand into solution cavities that have developed along joints in the limestone may cause subsidence due to upward migration of the cavities (a process known as piping) to form cylindrical holes at the land surface. If the overburden is incohesive sand, the upward-migrating cavity is dissipated by a general lessening of density over a large area, and the result will be a relatively broad and extensive subsidence of the land surface that occurs over a period of time. Generally, subsidence of this type may go unnoticed for several years. If the overburden is a dense plastic clay, its low permeability may impede downward movement of ground water and retard the development of solution cavities in the underlying limestone. For this reason, areas underlain by relatively impermeable clay generally are not affected by sinkholes.

TYPES OF LIMESTONE COVER

Four areas of sinkhole occurrence in Florida, based on the type and thickness of cover material overlying the limestone, are described in this section and are delineated on the map.

Area I.—Bare or thinly covered limestone. Sinkholes are few. Generally shallow and broad, and develop gradually. Solution sinkholes dominate.

Throughout large areas of the State, the cover material ranges in thickness from less than a foot to about 25 feet (Area I on map). Generally, the cover material is very permeable, and the effect, in terms of solution development of the limestone, is that of bare limestone exposed to weathering. Solution at the limestone surface and in joints near the surface decreases with depth, but the solution of limestone is the dominant process in landscape development throughout Area I. Area I reportedly has very few collapse sinkholes, and those that occur generally are very shallow and broad and develop slowly.

Area II.—Cover is 30 to 200 feet thick and consists mainly of incohesive and permeable sand. Sinkholes are few, shallow, of small diameter, and develop gradually. Cover-subsidence sinkholes dominate.

In areas where limestone is covered by materials that are relatively incohesive and permeable, sinkholes develop by gradual subsidence (Area II). In some areas, particularly where the sand cover is relatively thick, consolidation and compaction of the sand grains lend a certain cohesiveness. Generally, cover-subsidence sinkholes are only a few feet in diameter and depth. Their small size and innocuous mode of occurrence evolve because cavities in the limestone surface cannot develop to an appreciable size before they are filled with sand. The cover sand in Area II is relatively permeable; water infiltrates directly to the water table, which may be in the sand or limestone, depending on the topography. Few sinkholes develop in Area II. Most sinkholes are less than 10 feet deep and average 5 to 15 feet in diameter.

Area III.—Cover is 30 to 200 feet thick and consists mainly of cohesive clayey sediments of low permeability. Sinkholes are most numerous, of varying size, and develop abruptly. Cover-collapse sinkholes predominate.

Throughout much of Florida, the sand cover becomes increasingly clayey with depth, and a layer of low-permeability clay generally overlies the limestone surface. The clay component provides a degree of cohesiveness to the cover material that allows it to bridge a developing cavity in the limestone. Failure of the bridge results in a cover-collapse sinkhole, typical of Area III, whose dimensions are related to the size of the cavity and the bearing strength of the clay. The mechanism of solution erosion of limestone that precedes a cover-collapse sinkhole is similar to that of a cover-subsidence sinkhole. The distinction between the two categories of sinkholes is one of degree of cohesion or bearing strength of the cover material. Cohesion or strength controls whether the cover material subsides slowly or collapses abruptly.

A factor equally as important as the cohesion or bearing strength of the clay is its low permeability. In areas where the cover contains a clay layer of sufficiently low permeability to greatly retard downward infiltration of water, a difference in head develops across the confining clay between water perched in the surficial material and the water level in the underlying carbonate aquifer. Fluctuations in head difference between the surficial aquifer and the carbonate aquifer tend to weaken the overlying clay bridge. An increase in head difference, either because of flooding at land surface or a decline in head in the lower aquifer, tends to load the clay bridge with an increased hydrostatic force that, in conjunction with the forces of gravity already at work on the cover material, may stress the clay beyond its bearing strength. Pumping of water from the lower aquifer, with consequent drawdown of the water level, is possibly the most common mechanism by which sinkholes are induced by man.

Throughout Area III, the thickness and composition of unconsolidated material that covers the limestone has an important effect on the shape and size of land-surface collapse. A thick layer of dense clay may have sufficient strength to bridge a large-diameter cavity whose roof crumbles and spalls into a solution-enlarged joint in the limestone. When the clay layer finally fails, the resulting sinkhole will be relatively large and will probably form abruptly. The width of the cavity at the limestone surface may not be as great as that in the clay, or as great as the diameter of the subsequent cover-collapse sinkhole. The limestone cavity may be a deep, small-diameter conduit through which debris is transported by gravity and ground-water flow. The size of the sinkhole is proportional to the thickness and bearing strength of the cover material and the volume of the underlying cavity. Area III reportedly has the greatest number of sinkhole occurrences in the State. Most of the sinkholes are of the cover collapse type, such as the sinkhole that occurred in Winter Park, May 8-13, 1981.

Area IV.—Cover is more than 200 feet thick and consists of cohesive sediments interlayered with discontinuous carbonate beds. Sinkholes are very few, but several large diameter, deep sinkholes occur. Cover-collapse sinkholes dominate.

Area IV is defined, principally, on the occurrence of a thick section of clastic sediments that generally overlie the carbonate rock. The sediments limit ground-water circulation in the underlying carbonate rocks and the development of solution cavities. Carbonate layers, however, are common within the section of clastic sediments in some areas, and silty, sandy, phosphatic limestone and dolomite may be the principal rock types. In areas where the shallow carbonate rocks are permeable enough to permit ground-water circulation, they too are subject to solution and subsequent collapse of overburden. These carbonate rocks are discontinuous within the sand-clay cover of Area IV, and their occurrence and extent are not well known. Area IV reportedly has very few sinkhole occurrences, although several large diameter, deep sinkholes are present in the area.

Table 1.—*Geologic framework*
[Modified from Wilson and Gerhart, 1980, table 1]

System	Series	Stratigraphic unit	General lithology	Major lithologic unit	Geologic process	Age estimates of boundaries in million years[1]
Quaternary	Holocene Pleistocene	Surficial sand, terrace sand, phosphorite	Predominantly fine sand; interbedded clay, marl, shell, limestone, phosphorite	Sand	Fluctuations of sea level with consequent high water tables and deposition in low-lying areas alternating with low water tables and accelerated weathering of soluble rocks	2
Tertiary	Pliocene	Undifferentiated deposits[2]	Clayey and pebbly sand; clay, marl, shell, phosphatic	Clastic		5
	Miocene	Hawthorn Formation	Dolomite, sand, clay, and limestone, silty, phosphatic	Carbonate and clastic		
		Tampa Limestone	Limestone, sandy, phosphatic, fossiliferous; sand and clay in lower part in some areas			
	Oligocene	Suwannee Limestone	Limestone, sandy limestone, fossiliferous		Exposure and weathering Carbonate deposition	24
	Eocene	Ocala Limestone	Limestone, chalky, locarsonitical, dolomitic near bottom	Carbonate	Exposure and weathering Carbonate deposition	38
		Avon Park Formation	Limestone and hard brown dolomite, intergranular evaporite in lower part in some areas		Exposure and weathering Carbonate deposition	55
		Oldsmar Formation	Dolomite and limestone, containing intergranular gypsum in most areas	Carbonate containing evaporites	Exposure and weathering Carbonate deposition	63
	Paleocene	Cedar Keys Limestone	Dolomite and limestone containing beds of anhydrite			

[1]Geologic Names Committee, 1983, Major geochronologic and chronostratigraphic units: U.S. Geological Survey.
[2]Includes all or parts of Caloosahatchee Marl, Bone Valley Formation, Alachua Formation, and Tamiami Formation.

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_____1984, Sinkholes in Florida: Florida Sinkhole Research Institute pamphlet [in press].

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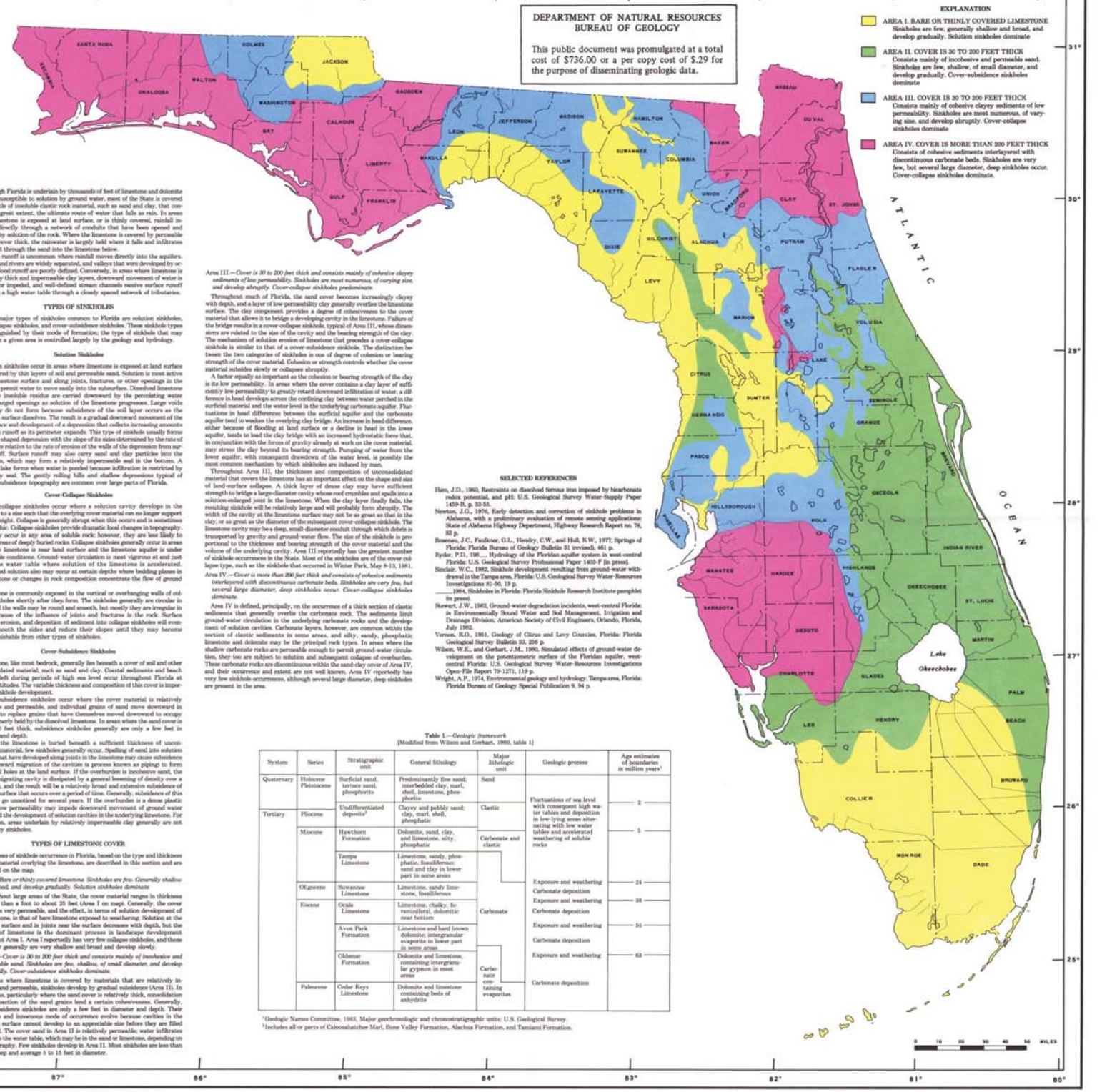

PART J

PART J – VERTICAL EXPANSION OF LANDFILLS

BACKGROUND

The proposed West Pasco County Landfill (WPCL) Ash Monofill slope modification and vertical expansion project includes placing additional incinerator ash on the side slopes and to a peak elevation in excess of what has been previously permitted with the intent to increase life of the existing Ash Monofill; therefore, in accordance with Rule 62-701.430, Florida Administrative Code (FAC), the proposed modification is classified as a vertical expansion. In accordance with Sections 62-701.430(1)(b) and 62-701.430(1)(c)1, FAC, an interface liner is not required because the Class I Landfill being expanded has a bottom liner system.

The outline of this report follows FDEP Form 62-701.900(1), Part J, Vertical Expansion of Landfills (effective date February 15, 2015).

1 EFFECTS OF VERTICAL EXPANSION

The proposed vertical expansion will not contribute to leachate leakage or objectionable odors from the existing Class I Ash Monofill. The conclusions, with respect to the leachate collection system, are based on the following evaluations:

1. Jones Edmunds reviewed the report titled *Pasco County Solid Waste West Landfill – Cell SW-2 Leachate Collection Pipe Jetcleaning* by Florida Jetclean (May 2021), which in addition to solid waste cell SW-2 reports cleaning done by Jetclean for the ash cells (A-1, A-2, A-3) since May 2020. This report indicated that all leachate collection lines were open and were cleaned throughout the entire length . Attachment J.1-1 provides a copy of the above-referenced report.
2. Due to the increased load on the geocomposite as a result of the vertical expansion, Jones Edmunds performed calculations to demonstrate that the material has sufficient transmissivity to handle the expected leachate flow to maintain a maximum head on the liner of 12 inches. Attachment J.1-2 provides the Geocomposite Transmissivity Technical Memorandum and supporting calculations.
3. Jones Edmunds performed calculations to evaluate pipe crushing due to the increased loads on the leachate collection pipes from the vertical expansion to verify continued acceptable performance after the expansion. Attachment J.1-3 provides these calculations.

Leachate collection and disposal operations and odor management will be consistent with procedures detailed in the Operation Plan in Appendix B. The proposed expansion will not adversely affect the closure design of the landfill since the final closure system is not currently installed. Attachment J.1-4 provides updated stormwater calculations for the revised closure design.

2 VERTICAL EXPANSION OVER UNLINED LANDFILLS

The proposed vertical expansion is over a lined landfill. Therefore, this section is not applicable.

3 FOUNDATION AND SETTLEMENT ANALYSIS

Please refer to Part I for foundation and settlement analyses for the vertical expansion.

4 TOTAL SETTLEMENT CALCULATIONS

Please refer to Part I for total settlement calculations demonstrating that the final elevations of the lining system, gravity drainage, and other components of the design will not be adversely affected.

5 SLOPE STABILITY

In accordance with Rule 62-701.410(3)(e), FAC, slope stability analyses were performed at critical cross-sections. The stability analyses results meet and exceed the recommended minimum factor of safety of 1.5 (Rule 62-701.400(2), FAC). Please refer to Part I for additional details.

6 SURFACE WATER MANAGEMENT (RULE 62-701.430(2)(C), FAC)

No modifications are proposed to the site's stormwater management system outside the edge of liner nor are any additional impacts expected.

7 GAS CONTROL (RULE 62-701.430(2)(D), FAC)

An interface liner is not required nor is one being installed because of this vertical expansion. Therefore, this section is not applicable.

Attachment J.1-1

Pasco County Solid Waste West Landfill – Cell SW-2 Leachate Collection Pipe Jetcleaning

by Florida Jetclean (May 2021)

Attach. J.1-1 - Leachate Collection System Cleanout Report

Johnson, Sabrina O

From: Justin G. Roessler <jroessler@pascocountyfl.net>
Sent: Friday, May 21, 2021 1:32 PM
To: Morgan, Steve
Cc: Madden, Melissa; Jason Gorrie; Timothy J. Treshler
Subject: Leachate Collection System Cleaning Report
Attachments: Leachate System Cleaning Report.pdf

Steve,
Please see the attached letter outlining the leachate cleaning that has been done on our leachate collection system.
Thank you,

Justin G. Roessler, PhD, PE
Interim Solid Waste Director
Solid Waste and Resource Recovery Department
Pasco County
P 727-847-2411 Ext. 2084
14230 Hays Rd.
Spring Hill, FL 34610
jroessler@pascocountyfl.net
www.mypasco.net

"Serving our community to create a better future."

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CUSA18A7

Attach. J.1-1 - Leachate Collection System Cleanout Report

PASCO COUNTY FLORIDA
OPEN SPACES. VIBRANT PLACES.

Date: May 20, 2021

Mr. Steven Morgan
Air and Solid Waste Permitting Manager
Florida Department of Environmental Protection
Southwest District Office
13051 North Telecom Parkway
Temple Terrance, FL 33637-0926

Re: Pasco County Resource Recovery Facility – WACS-45799
May 2021 Report – Leachate Collection and Removal System Sufficiency Evaluation

Dear Mr. Morgan,

The Pasco County Resource Recovery Facility, per its Conditions of Certification, is required to provide the Florida Department of Environmental Protection (FDEP) with an evaluation of the effectiveness of the Leachate Collection and Removal System (LCS) for its Class I Landfill. Since 2020 the Pasco County Solid Waste Department has implemented an annual preventative LCS maintenance program to ensure that leachate is being effectively collected and removed in accordance with its FDEP-approved Operations Plan. Please find as attachments the results of LCS cleaning and maintenance for all Class I Landfill. The attached documents detail the date of maintenance, leachate lines cleaned, methods employed, and condition of lines after maintenance.

Should you have any questions or require additional information, please do not hesitate to contact me by email at jroessler@pascocountyfl.net or phone at (727) 857-2780.

Sincerely,

Justin Roessler

Justin Roessler, PhD, P.E,
Solid Waste Interim Director

JGR/fdep

Cc: Tim Treshler, Sr. Project Manager, Solid Waste Department

Attach. J.1-1 - Leachate Collection System Cleanout Report

FLORIDA JETCLEAN

HIGH PRESSURE WATER JETTING
EXPLOSION PROOF VIDEO INSPECTION
VACUUM TRUCK SERVICES
WWW.FLORIDAJETCLEAN.COM

7538 DUNBRIDGE DRIVE
ODESSA, FL 33556
T: 800-226-8013 / F: 813-926-4616
FLORIDAJETCLEAN@YAHOO.COM

Pasco County Solid Waste
West Landfill – Cell SW-2
Leachate Collection Pipe Jetcleaning

Work Performed
May 2021

Conducted By:
Florida Jetclean
800-226-8013

Attach. J.1-1 - Leachate Collection System Cleanout Report

FLORIDA JETCLEAN

HIGH PRESSURE WATER JETTING
EXPLOSION PROOF VIDEO INSPECTION
VACUUM TRUCK SERVICES
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7538 DUNBRIDGE DRIVE
ODESSA, FL 33556
T: 800-226-8013 / F: 813-926-4616
FLORIDAJETCLEAN@YAHOO.COM

REPORT

DATE : 5/10/2021
TO : Timothy Treshler – Pasco County Solid Waste Management
FROM : Ralph Calistri (floridajetclean@yahoo.com)
SUBJECT : 2021 West Pasco County Landfill – Cell SW-2 - Pipe Jetcleaning Project

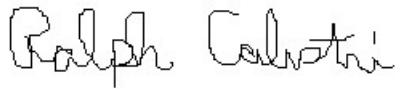
Florida Jetclean completed the high-pressure water-jetting of the SW2 leachate collection piping at the above landfill on 5/10/2021. The jetting log (below) documents the pipes that were addressed, as well as the jetting distances that were achieved in each pipe.

CELL SW-2 (8" HDPE)

NE Lateral Cleanout	1,000’	Entire Pipe Cleaned Through Overlap
SE Lateral Cleanout	1,000’	Entire Pipe Cleaned Through Overlap
NE Header Cleanout	375’	Entire Pipe Cleaned
SE Header Cleanout	375’	Entire Pipe Cleaned
NW Lateral Cleanout	1,000’	Entire Pipe Cleaned Through Overlap
SW Lateral Cleanout	1,000’	Entire Pipe Cleaned Through Overlap

Please call us with questions or concerns.

Thank you,



Ralph Calistri - Florida Jetclean - 800-226-8013

Attach. J.1-1 - Leachate Collection System Cleanout Report

PROGRESSIVE
ENVIRONMENTAL SERVICES

May 12, 2021

Tim Treshler
Pasco County
14230 Hays Rd.,
Spring Hill, FL 34610

Re: Report for Chemical Cleaning in Leachate Collection System at Pasco County Landfill.

Dear Mr. Treshler:

Attached is the Report on completed chemical cleaning work on the leachate collection system (LCS) piping and Headers in Cells A-1, A-2, A-3 and SW-1 over the last two years.

Completed chemical cleaning was achieved by flushing all the pipes with the chemical solution until chemical activity was measured at the end point of each piping system. When this occurs scale as been dissolved in the pipe and in the area of the gravel pack around each perforated pipe.

We appreciate the opportunity to work with Pasco County and help with keeping the systems open and operating as originally designed. Please let us know if you have any questions or if we can help in any other way.

Best regards,

S. Steven Carl, President

Cc: Dr. Justin Proessler
Travis Carl
Dale Hall
Linda Lewis

Steven Carl, President
steven.carl@progressive.us.com
4425 Portsmouth Blvd., Ste 120
Chesapeake, VA 23321

Attach. J.1-1 - Leachate Collection System Cleanout Report

Cell A-1 (6" HDPE Primary)	Date Cleaned	Condition After Cleaning
Lateral 1 (furthest East)	21-Apr	Clear
Lateral 2	21-Apr	Clear
Lateral 3	21-Apr	Clear
Lateral 4	21-Apr	Clear
Lateral 5	21-Apr	Clear
Lateral 6	21-Apr	Clear
Lateral 7	21-Apr	Clear
Lateral 8	21-Apr	Clear
Lateral 9	21-Apr	Clear
Lateral 10	21-Apr	Clear
Header to Lift Station (12")	21-Apr	Clear

Cell A-2 (6" HDPE Primary)	Date Cleaned	Condition After Cleaning
Lateral 10 (furthest West)	20-May	Clear
Header to Lift Station (12")	20-May	Clear

Cell A-3 (6" HDPE Primary)	Date Cleaned	Condition After Cleaning
North Lateral	20-May	Clear
South Lateral	20-May	Clear
Header to Lift Station (12")	20-May	Clear
FM to Leachate Tank (12")	20-May	Clear

Cell SW-1 (6" HDPE Primary)	Date Cleaned	Condition After Cleaning
Lateral 1 (furthest East)	21-Apr	Clear
Lateral 2	21-Apr	Clear
Lateral 3	21-Apr	Clear
Lateral 4	21-Apr	Clear
Lateral 5	21-Apr	Clear
Lateral 6	21-Apr	Clear
Lateral 7	21-Apr	Clear
Lateral 8	21-Apr	Clear
Lateral 9	21-Apr	Clear
Lateral 10	21-Apr	Clear
Header to Lift Station (12")	21-Apr	Clear

Attachment J.1-2

Geocomposite Transmissivity Design

TECHNICAL MEMORANDUM

WEST PASCO COUNTY LANDFILL ASH MONOFILL VERTICAL EXPANSION

TO: Project File

FROM: Chad Spreadbury, PhD, EI

XC: George Reinhart, PhD, PE

DATE: June 29, 2021

SUBJECT: Attachment J.1-2, Geocomposite Transmissivity Design

1 PURPOSE

The design transmissivity of the geocomposite used in the leachate collection system was calculated to demonstrate that the design transmissivity will be maintained for the design period of the facility in accordance with Rule 62-701.400(3)(d)8, Florida Administrative Code (FAC), and to determine whether the proposed vertical expansion above the current ash cells (A-1, A-2, and A-3) may compress the geocomposite such that the head on liner increases above 12 inches. The geocomposite design transmissivity and thickness were determined using manufacturer performance data, geocomposite loads, and reduction factors. The geocomposite design transmissivity and thickness were then used to derive the geocomposite hydraulic conductivities and used to evaluate whether the liner head exceeds the compressed geomembrane's thickness.

Due to the overall length of the flow path, the peak loads for the Closed scenario over the entire flow path was used to be conservative. As leachate flows from the center of the landfill to the outer edges, the peak load applied to the geocomposite decreases and thus the transmissivity of the geocomposite increases incrementally as the leachate flow rate increases. The leachate flow rate used in this analysis was calculated using impingement rates based on leachate data from the County within the last year (July 2020-March 2021, or Quarter 3, 2020, through Quarter 1, 2021) and the overall position of the geocomposite along the flow path. These data were used to verify that the maximum leachate head would be contained within the thickness of the geocomposite at all locations along the bottom liner flow path.

2 METHODOLOGY

The geocomposite hydraulic conductivities were determined using the transmissivity and thickness data obtained during construction quality assurance testing and by applying various reduction factors and for site-specific loading. The geocomposite properties are

adjusted based on the manufacturer's recommendations and GRI GC-8 (2001, Reference 3) to determine the geocomposite properties at the peak load. The design transmissivity and thickness are determined from the following procedure:

1. Calculate 100-hour transmissivity, Θ100, at the peak load using manufacturer's data. In this case, data were only available for a 15,000-pound-per-square-foot (psf) load, which was used as a conservative estimate of the expected loading after the vertical expansion.

2. Calculate geocomposite thickness at the loads of interest using the manufacturer's data for a similar geocomposite product.

3. Calculate the geocomposite transmissivity after applying the following reduction factors:

    a. Creep deformation.
    b. Geotextile intrusion.
    c. Chemical degradation.
    d. Particulate clogging.
    e. Chemical clogging.
    f. Biological clogging.

4. Calculate the design transmissivity after applying a factor of safety of 2.

5. Calculate the design hydraulic conductivity from the design transmissivity and design thickness.

This process was used to evaluate the geocomposite transmissivity under Cell A-3, which is the only cell that contains a leachate collection and removal system (LCRS) geocomposite. Cells A-1 and A-2 do not contain LCRS geocomposites; rather, the designs for these cells consist of drainage sand installed directly above the bottom liner. The hydraulic conductivity of this design is not expected to be impacted by the increased loading. The leachate head was evaluated along the flow path from the center of the landfill to the perimeter leachate pump stations. Giroud's equation (2003) for leachate head on the liner where the hydraulic head is small (Equation 35, Reference 2) and leachate generation rates were used to calculate the maximum head on the liner along this path under the Closed scenario.

3 PARAMETERS

The following reduction factors were selected based on recommendations from the manufacturer and GRI GC-8:

- Creep deformation (load dependent).
- Geotextile intrusion.
- Chemical degradation.
- Particulate clogging.
- Chemical clogging.
- Biological clogging.

The geocomposite loading parameters are based on the loading used for the geocomposite testing, 15,000 psf, which is higher than the expected loading at the maximum height of waste (ash) of approximately 80 feet (or approximately 9,000 psf). The maximum peak flow rates for the leachate collection system are based on leachate-generation reporting documents (monthly) from July 2020 to March 2021.

4 CONCLUSIONS

The geocomposite transmissivity and head on the liner analyses were performed to confirm that the proposed vertical expansion and subsequent increase in loading will not cause an increase in head on the liner above 12 inches. The results from evaluating the transmissivity analysis along the flow path at the greatest depth of waste (within Cell A-3) demonstrate that under current leachate generation rates, the leachate head will be contained within the geocomposite.

Exhibit 1

Geocomposite Transmissivity

JonesEdmunds

PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 6
PROJECT NAME: PASCO COUNTY ASH MONOFILL VERTICAL EXPANSION
SUBJECT: ATTACH. J.1-2 - GEOCOMP. TRANS. DESIGN
BY: C.SPREADBURY Date: 6/24/2021
CHECKED BY: G.Reinhart Date: 6/25/2021

Objective: Determine the geocomposite hydraulic conductivity and thickness from manufacturer's transmissivity and loading testing data for use in the demonstration that the head on the liner is retained within the geocomposite over the duration of its length under the closed condition.

Discussion: The geocomposite transmissivity analysis presents a series of calculations to determine the long-term transmissivity of the geosynthetic drainage materials (geocomposites) based on minimum 100 hour transmissivity values. This is done to verify that the existing geocomposite installed will not be impacted by the proposed vertical expansion.

Calculation:

1. Calculate the 100-hour transmissivity, Θ_{100}, at the loads of interest based on manufacturer's data using linear interpolation.

Manufacturer's Θ_{100} Data

Applied Load [psf]	Θ_{100} [m^2/s]
15,000	2.5E-03

Interpolated Θ_{100} Data at Loads of Interest

Applied Load [psf]	Θ_{100} [m^2/s]	
15,000	0.00254	Position 1

(Reference 4)

2. Calculate geocomposite thickness, t_{dsg}, using linear interpolation for the loads of interest based on the manufacturer's 10,000-hour testing data (based on a similar product).

Justin,

We only have data on 15,000psf loading for TRx300. The creep reduction factor is 1.2 @15,000psf. % Retained at 10,000 hours @15,000psf is 80%.

Thanks,

Clay Reichert, P.E.
TECHNICAL MANAGER
[O] 404.596.1838

Manufacturer's t_{dsg} Data

Applied Load [psf]	Percent Retained Thickness	$t_{10,000}$ [mil]
0	100%	300
15,000	80%	240

(Reference 1)

3. Calculate the long-term-in-soil hydraulic transmissivity, Θ_{LTIS}, of the geocomposite at the various loads of interest. This is the minimum hydraulic transmissivity calculated for the geocomposite subjected to the maximum stress anticipated in the soil during the design life of the leachate collection layer and subjected to all mechanisms likely to reduce its hydraulic transmissivity.

$$\Theta_{LTIS} = \frac{\Theta_{100}}{RF_{CR} \times RF_{IN} \times RF_{CD} \times RF_{PC} \times RF_{CC} \times RF_{BC}}$$

(Reference 2, Equation 14, p 294)

where:
Θ_{LTIS} = long-term-in-soil transmissivity [cm^2/s]
Θ_{100} = transmissivity after 100 hours under expected load [cm^2/s]
RF_{CR} = reduction factor for creep deformation
RF_{IN} = reduction factor for geotextile intrusion
RF_{CD} = chemical degradation reduction factor
RF_{PC} = reduction factor for particulate clogging
RF_{CC} = reduction factor for chemical clogging
RF_{BC} = reduction factor for biological clogging

Creep Deformation Reduction Factor

The reduction factor for creep deformation, RF_{CR}, is dependent upon the load applied and increases as the load increases. This value is calculated using the following data from the manufacturer interpolated at loads of interest.

Justin,

We only have data on 15,000psf loading for TRx300. The creep reduction factor is 1.2 @15,000psf. % Retained at 10,000 hours @15,000psf is 80%.

Thanks,

Clay Reichert, P.E.
TECHNICAL MANAGER
[o] 404.596.1838

Manufacturer's RF_{CR} Data	
Applied Load [psf]	RF_{CR} [-]
15,000	1.20

Interpolated RF_{CR} Data at Loads of Interest		
Applied Load [psf]	RF_{CR} [-]	
15,000	1.20	Position 1

RF_{CR} Values Used in Analysis at Loads of Interest		
Applied Load [psf]	RF_{CR} [-]	
15,000	1.20	Position 1

(Reference 1)

Geotextile Intrusion Reduction Factor

A minimal geotextile intrusion reduction factor, RF_{IN}, of 1.1 was selected for the collection system because the hydraulic transmissivity testing includes intrusion. In the leak detection system, no intrusion is expected because the secondary geocomposite is overlain by a geomembrane, not soil; therefore the RF_{IN} is selected to be 1.0.

Chemical Degradation Reduction Factor

The materials used for the geocomposite are not expected to chemically degrade in a landfill; therefore, the reduction factor for chemical degradation, RF_{CD}, is selected to be 1.0.

Particulate Clogging Reduction Factor

The geotextile component of the geocomposite has been designed to retain the soil drainage layer above. Therefore, the reduction factor for particulate clogging, RF_{PC}, is selected to be 1.0.

Chemical Clogging Reduction Factor

The chemical clogging reduction factor, RF_{CC}, will vary throughout the life of the facility. Initially, flow reduction due to chemical clogging of the geocomposites will be low for both leachate collection and leak detection systems. As the system ages, some flow reduction is expected within the typical range of reduction factors. Lower reduction factors are used for the leak detection system because of the significantly lower leachate flow. Chemical clogging reduction factors were selected based on criteria from GRI GC8. The intermediate/final scenario was used for all locations.

Age	RF_{CC}	
	Collection [-]	Detection [-]
Initial	1.5	1.1
Intermediate/Final	2.0	1.5

(Reference 3, p 9)

Biological Clogging Reduction Factor

The biological clogging reduction factor, RF_{BC}, will vary throughout the life of the facility. Initially, flow reduction due to biological clogging of the geocomposites will be low for both leachate collection and leak detection systems. As the system ages, some flow reduction is expected within the typical range of reduction factors. Lower reduction factors are used for the leak detection system because of the significantly lower leachate flow. Biological clogging reduction factors were selected based on criteria from GRI GC8. The intermediate/final scenario was used for all locations.

Age	RF_{BC}	
	Collection [-]	Detection [-]
Initial	1.1	1.1
Intermediate/Final	1.3	1.3

(Reference 3, p 9)

Θ_{LTIS} Calculation

Parameter	Units	Geocomposite Properties	
		Collection	Detection
Load	psf	15,000	
Θ_{100}	cm^2/sec	25.4	25.4
RF_{CR}	[-]	1.20	1.20
RF_{IN}	[-]	1.1	1.0
RF_{CD}	[-]	1.0	1.0
RF_{PC}	[-]	1.0	1.0
RF_{CC}	[-]	2.0	1.5
RF_{BC}	[-]	1.3	1.3
ΠRF	[-]	3.4	2.3
Θ_{LTIS}	**cm^2/sec**	**7.4**	**10.9**

4. Calculate the design transmissivity, Θ_{dsg}, of the geocomposite at the various loads of interest by applying a factor of safety of 2 to the Q_{LTIS}.

$\Theta_{dsg} = \Theta_{LTIS} / FS$

where: Θ_{LTIS} = long-term-in-soil transmissivity [cm^2/s]
FS = factor of safety = 2 - *(Reference 2, p 299)*
Θ_{dsg} = design transmissivity [cm^2/s]

Parameter	Units	Geocomposite Properties	
		Collection	Detection
Load	psf	15,000	
Θ_{LTIS}	cm^2/sec	7.4	10.9
Θ_{dsg}	**cm^2/sec**	**3.7**	**5.4**

5. Calculate the design hydraulic conductivity, k_{dsg}, of the geocomposite at the various loads of interest.

$k_{dsg} = \Theta_{dsg} / t_{dsg}$

where: k_{dsg} = design hydraulic conductivity [cm/s]
Θ_{dsg} = design transmissivity [cm^2/s]
t_{dsg} = design thickness [cm]

Parameter	Units	Geocomposite	
		Collection	Detection
Load	psf	15,000	
Θ_{dsg}	cm^2/sec	3.7	5.4
t_{dsg}	mil	240	240
	cm	0.61	0.61
k_{dsg}	**cm/sec**	**6.1**	**8.9**

References:
1. Email Correspondence from Steve Mayes Dated October 3, 2017. http://www.gseworld.com/Online-Drainage-Design-Manual/
2. Giroud, J. P., Zornberg, J. G., Zhao, A. (2000) "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers." Geosynthetics International, Vol. 7, Nos. 4-6, p. 285-380.
3. Geosynthetics Institute. (2001) "GRI Standard GC8 - Determination of the Allowable Flow Rate of a Drainage Geocomposite." Folsom, p 1-11.
4. COMANCO. (2003) "Certification of Construction Completion and Quality Assurance Documentation. Section 9.

Exhibit 2

Head on Liner Calculations and Transmissivity: Closed Condition

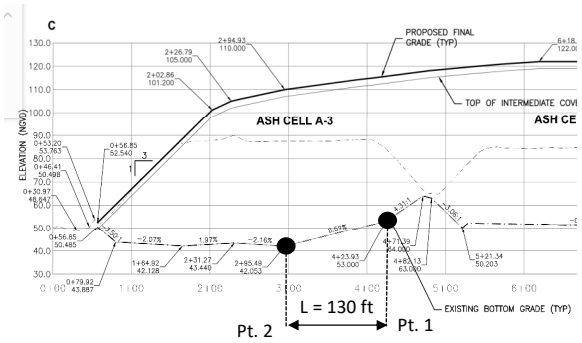
JonesEdmunds

PROJECT NUMBER:	16002-009-01	SHEET: 5 OF 6	
PROJECT NAME:	PASCO COUNTY ASH MONOFILL VERTICAL EXPANSION		
SUBJECT:	ATTACH. J.1-2 - GEOCOMP. TRANS. DESIGN		
BY:	C.SPREADBURY	Date:	6/23/2021
CHECKED BY:	G.Reinhart	Date:	6/25/2021

Objective: Ensure the calculated head on the liner based on the geocomposite transmissivity is less than the thickness of the compressed geocomposite thickness under closure conditions and peak flow between evaluation points 1 and 2. This evaluation was selected for the identified points (1 and 2) for Cell A-3, as this area of landfill will have the greatest load (height of waste) above it once the vertical expansion is completed.

Discussion: The geocomposite head on the liner analysis presents a series of calculations to demonstrate that the transmissivity of the geocompsite is sufficient to contain the maximum drainage within the geocompsite. Maximum geocomposite loads are used at points 1 and 2 within Cell A-3 to evaluate maximum liner head over the length of the cell.

Figure 1: Positions within cell A-3 for transmissitivity evaluation under closure condition.



Geocompsite type:	Tenax				
Geocompsite thickness:	300 mil				
Geonet thickness:	0.240	in	Position 1	Attach. J.1-2	
Segment:	Points 1 to 2		Figure 1		
Peak Load:	15,000	psi	Based on test data, exceeds expected loading at vert. expansion		(Attach. J.1-2)
Length of Section:	130	ft	Figure 1 & Permit Drawings		
Condition:	Closed				

Calculation:

1. Calculate the liquid thickness on the liner for the geonet length from position 1 to position 2.
a. Use Giroud's equation for liquid thickness on the liner where liquid head is at low level.

$$t_max = q_(h * L)/(k \sin[\![(\beta)]\!])$$ Ref. 2, Eqn 35

Where:

t_{max} =	maximum head on the liner (inches)	Calculated
qh =	horizontal flow through the geonet	Calculated
L =	the length of bottom liner section from position 1 to positon 2	Permit Drawings (See Figure 1)
k_{dsg} =	the design hydraulic conductivity at position one	Attach. J.1-2
β =	the angle corresponding to the slope of the liner after settlement	Min. Allowed Grade as per 62-701.400(4)(c)

b. The horizontal flow through the geonet (q_h) is dependent on the leachate impingement rate, provided by HELP model and the length liner section being evaluated.

q_h = Imp * L

Where:

Imp = peak impingement rate from HELP model (ft/sec)
L = the length of bottom liner section from position 1 to position 2

Peak Daily Leachate Collected from Bottom Liner	606	ft^3/day	Peak Daily Flow Pasco Leachate Data Q3, 2020 to Q1, 2021
Area of Landfill	10	acres	Area of Cell A-3
Area of Landfill	435600	ft^2	
q_h =	1.61E-08	ft/sec	Calculated

c. The angle corresponding to the minimum slope of the liner after settlement (β) is dependent on the minimum required landfil gradient as calculated in Part I

β = 0.3 ° (min.)

Where:

i = minimum gradient after settlement 0.3 ° (min.) from 62-701.400(4)(c)
β = 0.005 rad

a. Use Giroud's equation for liquid thickness on the liner where liquid head is at low level

$$t_max = q_(h * L)/(k \sin[\![(\beta)]\!])$$

Where:

t_{max} = maximum head on the liner (inches)

q_h =	horizontal flow through the geonet	1.61E-08	ft/sec
L =	the length of bottom liner section from position 1 to position 2	130	ft
k_{dsg} =	the design hydraulic conductivity at position one	6.07	cm/sec
		0.199	ft/sec
β =	the angle corresponding to the slope of the liner after settlement	0.005	rad
	t_{max} =	0.002	ft
d. Is the maximum liner head smaller than the geonet thickness ?		0.024	in

t_{net} =	thickness of geonet under loading from position 1 =	0.240 inches		
t_{max} =	maximum head on liner between positions 1 and 2 =	0.024 inches	**Is t_{max} > t_{net} ?**	**yes**

Conclusion:
The leachate head will be contained within the geocomposite between positions 1 and 2.

References:

1. Email Correspondence from Steve Mayes Dated October 3, 2017. http://www.gseworld.com/Online-Drainage-Design-Manual/
2. Giroud, J. P., Zornberg, J. G., Zhao, A. (2000) "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers." Geosynthetics International, Vol. 7, Nos. 4-6, p. 285-380.
3. Geosynthetics Institute. (2001) "GRI Standard GC8 - Determination of the Allowable Flow Rate of a Drainage Geocomposite." Folsom, p 1-11.
4. COMANCO. (2003) "Certification of Construction Completion and Quality Assurance Documentation. Section 9.

Exhibit 3

West Pasco County Landfill Leachate Generation Data (July 2020-March 2021)

Leachate Generation for Ash Monofill Cells at West Pasco County Landfill - Summary Table

Leachate Generation - Summary Table

Month	Cells A-1-A-3 Primary (gal)	Cells A-4 Primary (gal)	Total Primary (gal)	Cells A-1-A-3 Secondary (gal)	Cell A-4 Secondary (gal)	Total Secondary (gal)
Jan-20			339,359	207	37	244
Feb-20			221,460	100	220	320
Mar-20			107,525	81	122	203
Apr-20			225,644	100	700	800
May-20			656,360	102	2,200	2,302
Jun-20			1,135,196	97	14,100	14,197
Jul-20	613,354	271,435	884,789	100	10,075	10,175
Aug-20	276,198	365,670	641,868	99	5,684	5,783
Sep-20	372,645	874,970	1,247,615	100	29,007	29,107
Oct-20	280,547	211,714	492,261	101	43,753	43,854
Nov-20	191,460	437,148	628,608	89	4,746	4,835
Dec-20	223,634	110,855	334,489	112	5,943	6,055
Jan-21	541,995	62,291	604,286	70	199	269
Feb-21	238,547	79,928	318,475	100	361	461
Mar-21	935,229	143,000	1,078,229	102	15	117
Total	3,673,609		8,916,164			118,722
Average Daily Flow**	13,606		19,814			264

* Leachate Generation Data provided by Pasco County.

** Assumes each month is 30 days.

Leachate Report - Quarter 3, 2020

PASCO COUNTY: UTILITIES SERVICES BRANCH
LEACHATE REPORT
QUARTER ENDING: September 30, 2020

WEST PASCO CLASS I LANDFILL (WACS# 45799)
TOTAL LEACHATE:

			Acres: 50		Acres: 20
MONTH	**MEASURED RAINFALL (INCHES)**	**TOTAL LEACHATE (Gallons) ASH CELLS**	**Gallons / acres / days ASH**	**TOTAL LEACHATE (Gallons) SW CELLS**	**Gallons / acres / days SW**
Jul-20	4.81	894,864	577.33	399,579	644.48
Aug-20	7.50	647,552	417.78	728,460	1174.94
Sep-20	10.50	1,276,622	851.08	2,035,728	3392.88
TOTAL:	22.81	2,819,038		3,163,767	
		AVERAGE:	612.83		1719.44

SECONDARY LEACHATE and ACTION LEAKAGE RATE (ALR):
ALR Formula = Gallons / Acres / Days (g/a/d)

Acreage:	**10**	**10**	**10**	**20**	**10**	**10**
MONTH	**CELL A1**	**CELL A2**	**CELL A3**	**CELL A4**	**CELL SW1**	**CELL SW2**
	gallons - ALR	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**
Jul-20	100g - 0.32g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	10075g - 16.25g/a/d	116g - 0.37g/a/d	0g - 0.00g/a/d
Aug-20	99g - 0.32g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	5684g - 9.17g/a/d	160g - 0.52g/a/d	1g - 0.00g/a/d
Sep-20	100g - 0.33g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	29007g - 48.34g/a/d	170g - 0.57g/a/d	450g - 1.50g/a/d
Quarter:	299g - 0.33g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	44766g - 24.33g/a/d	446g - 0.48g/a/d	451g - 0.49g/a/d

Leachate Report - Quarter 3, 2020

PASCO COUNTY: UTILITIES SERVICES BRANCH
LEACHATE REPORT
QUARTER ENDING: September 30, 2020

WEST PASCO CLASS I LANDFILL (WACS# 45799)
LEACHATE DETAIL: (All units in gallons)

MONTH	CELLS A1, A2, and A3 (Combined) PRIMARY LEACHATE	CELL A1 SECONDARY LEACHATE	CELL A2 SECONDARY LEACHATE	CELL A3 SECONDARY LEACHATE	CELL A4 PRIMARY LEACHATE	CELL A4 SECONDARY LEACHATE
Jul-20	613,354	100	0	0	271,435	10,075
Aug-20	276,198	99	0	0	365,670	5,684
Sep-20	372,645	100	0	0	874,970	29,007
TOTAL:	**1,262,197**	**299**	**0**	**0**	**1,512,075**	**44,766**

MONTH	CELLS SW1 and SW2 (Combined) PRIMARY LEACHATE	CELL SW1 SECONDARY LEACHATE	CELL SW2 SECONDARY LEACHATE
Jul-20	399,579	116	0
Aug-20	728,460	160	1
Sep-20	2,035,728	170	450
TOTAL:	**3,163,767**	**446**	**451**

Leachate Report - Quarter 4, 2020

PASCO COUNTY: UTILITIES SERVICES BRANCH
LEACHATE REPORT
QUARTER ENDING: December 31, 2020

WEST PASCO CLASS I LANDFILL (WACS# 45799)
TOTAL LEACHATE:

			Acres: 50		**Acres:** 20
MONTH	**MEASURED RAINFALL (INCHES)**	**TOTAL LEACHATE (Gallons) ASH CELLS**	**Gallons / acres / days ASH**	**TOTAL LEACHATE (Gallons) SW CELLS**	**Gallons / acres / days SW**
Oct-20	2.50	536,014	345.82	690,944	1114.43
Nov-20	5.00	633,354	422.24	884,823	1474.71
Dec-20	2.13	340,432	219.63	406,850	656.21
TOTAL:	9.63	1,509,800		1,982,617	
		AVERAGE:	328.22		1077.51

SECONDARY LEACHATE and ACTION LEAKAGE RATE (ALR):
ALR Formula = Gallons / Acres / Days (g/a/d)

Acreage:	**10**	**10**	**10**	**20**	**10**	**10**
MONTH	**CELL A1**	**CELL A2**	**CELL A3**	**CELL A4**	**CELL SW1**	**CELL SW2**
	gallons - ALR	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**
Oct-20	101g - 0.33g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	43753g - 70.57g/a/d	235g - 0.76g/a/d	0g - 0.00g/a/d
Nov-20	89g - 0.30g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	4746g - 7.91g/a/d	157g - 0.52g/a/d	0g - 0.00g/a/d
Dec-20	112g - 0.36g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	5943g - 9.59g/a/d	200g - 0.65g/a/d	0g - 0.00g/a/d
Quarter:	302g - 0.33g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	54442g - 29.59g/a/d	592g - 0.64g/a/d	0g - 0.00g/a/d

Leachate Report - Quarter 4, 2020

PASCO COUNTY: UTILITIES SERVICES BRANCH
LEACHATE REPORT
QUARTER ENDING: December 31, 2020

WEST PASCO CLASS I LANDFILL (WACS# 45799)
LEACHATE DETAIL: (All units in gallons)

MONTH	CELLS A1, A2, and A3 (Combined) PRIMARY LEACHATE	CELL A1 SECONDARY LEACHATE	CELL A2 SECONDARY LEACHATE	CELL A3 SECONDARY LEACHATE	CELL A4 PRIMARY LEACHATE	CELL A4 SECONDARY LEACHATE
Oct-20	280,547	101	0	0	211,714	43,753
Nov-20	191,460	89	0	0	437,148	4,746
Dec-20	223,634	112	0	0	110,855	5,943
TOTAL:	**695,641**	**302**	**0**	**0**	**759,717**	**54,442**

MONTH	CELLS SW1 and SW2 (Combined) PRIMARY LEACHATE	CELL SW1 SECONDARY LEACHATE	CELL SW2 SECONDARY LEACHATE
Oct-20	690,944	235	0
Nov-20	884,823	157	0
Dec-20	406,850	200	0
TOTAL:	**1,982,617**	**592**	**0**

PASCO COUNTY: UTILITIES SERVICES BRANCH
LEACHATE REPORT
QUARTER ENDING: March 31, 2021

Leachate Report - Quarter 1, 2021

WEST PASCO CLASS I LANDFILL (WACS# 45799)
TOTAL LEACHATE:

			Acres: 50		**Acres:** 20
MONTH	**MEASURED RAINFALL (INCHES)**	**TOTAL LEACHATE (Gallons) ASH CELLS**	**Gallons / acres / days ASH**	**TOTAL LEACHATE (Gallons) SW CELLS**	**Gallons / acres / days SW**
Jan-21	0.25	604,485	389.99	258,730	417.31
Feb-21	2.50	318,836	227.74	383,899	685.53
Mar-21	1.50	1,078,244	695.64	288,343	465.07
TOTAL:	4.25	2,001,565		930,972	
		AVERAGE:	444.79		517.21

SECONDARY LEACHATE and ACTION LEAKAGE RATE (ALR):
ALR Formula = Gallons / Acres / Days (g/a/d)

Acreage:	**10**	**10**	**10**	**20**	**10**	**10**
MONTH	**CELL A1**	**CELL A2**	**CELL A3**	**CELL A4**	**CELL SW1**	**CELL SW2**
	gallons - ALR	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**	**gallons - ALR**
Jan-21	70g - 0.23g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	199g - 0.32g/a/d	122g - 0.39g/a/d	0g - 0.00g/a/d
Feb-21	100g - 0.36g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	361g - 0.64g/a/d	125g - 0.45g/a/d	0g - 0.00g/a/d
Mar-21	102g - 0.33g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	15g - 0.02g/a/d	166g - 0.54g/a/d	0g - 0.00g/a/d
Quarter:	272g - 0.30g/a/d	0g - 0.00g/a/d	0g - 0.00g/a/d	575g - 0.32g/a/d	413g - 0.46g/a/d	0g - 0.00g/a/d

Leachate Report - Quarter 1, 2021

PASCO COUNTY: UTILITIES SERVICES BRANCH
LEACHATE REPORT
QUARTER ENDING: March 31, 2021

WEST PASCO CLASS I LANDFILL (WACS# 45799)
LEACHATE DETAIL: (All units in gallons)

MONTH	CELLS A1, A2, and A3 (Combined) PRIMARY LEACHATE	CELL A1 SECONDARY LEACHATE	CELL A2 SECONDARY LEACHATE	CELL A3 SECONDARY LEACHATE	CELL A4 PRIMARY LEACHATE	CELL A4 SECONDARY LEACHATE
Jan-21	541,995	70	0	0	62,291	199
Feb-21	238,547	100	0	0	79,928	361
Mar-21	935,229	102	0	0	143,000	15
TOTAL:	**1,715,771**	**272**	**0**	**0**	**285,219**	**575**

MONTH	CELLS SW1 and SW2 (Combined) PRIMARY LEACHATE	CELL SW1 SECONDARY LEACHATE	CELL SW2 SECONDARY LEACHATE
Jan-21	258,730	122	0
Feb-21	383,899	125	0
Mar-21	288,343	166	0
TOTAL:	**930,972**	**413**	**0**

Reference 1

Justin Roessler

From: Clay Reichert <creichert@gseworld.com>
Sent: Tuesday, October 03, 2017 2:59 PM
To: Justin Roessler; George Reinhart
Cc: Steve Mayes
Subject: RE: New River: Landfill Expansion Geocomposite Data

Justin,

We only have data on 15,000psf loading for TRx300. The creep reduction factor is 1.2 @15,000psf. % Retained at 10,000 hours @15,000psf is 80%.

Thanks,

Clay Reichert, P.E.
TECHNICAL MANAGER
[O] 404.596.1838

GSE ENVIRONMENTAL™
GSE Environmental 4898 Tarry Glen Drive, Suwanee, GA 30024
gseworld.com

From: Justin Roessler [mailto:JRoessler@jonesedmunds.com]
Sent: Tuesday, October 3, 2017 1:55 PM
To: Clay Reichert <creichert@gseworld.com>; George Reinhart <GReinhart@jonesedmunds.com>
Cc: Steve Mayes <smayes@gseworld.com>
Subject: RE: New River: Landfill Expansion Geocomposite Data

Thanks Clay,

One last request. Do you have the retained thickness and creep reduction factors for the TRx 300 mil?

Justin Roessler, PhD
Engineer Intern
Jones Edmunds & Associates, Inc.
352.377.5821 ext. 1360

The information contained in this message including any attachment is confidential. It is intended for the private use of the intended addressee only. If you are not the intended addressee, please immediately notify the sender by reply email and destroy the original transmission and any attachment. E-mail transmission of information cannot be guaranteed to be free of error or other defect. If verification is required, request a hard-copy version. The sender disclaims liability for any defects in this message caused by e-mail transmission including a virus. It is the responsibility of the recipient to ensure that this message is virus free.

From: Clay Reichert [mailto:creichert@gseworld.com]
Sent: Tuesday, October 03, 2017 12:29 PM
To: Justin Roessler <JRoessler@jonesedmunds.com>; George Reinhart <GReinhart@jonesedmunds.com>
Cc: Steve Mayes <smayes@gseworld.com>
Subject: RE: New River: Landfill Expansion Geocomposite Data

Justin,

Here are the 100 hours trans values:

TRx 300 mil double sided 6 or 8oz/sy composite. Boundary Condition: sand/GC/GM

Loading	Gradient=0.02	Gradient = 0.1
(psf)	Transmissivity (m^2/s)	
1,000	$5x10^{-3}$	$3x10^{-3}$
10,000	$3x10^{-3}$	$1.7x10^{-3}$
15,000	$1.8x10^{-3}$	$1x10^{-3}$
20,000	$8x10^{-4}$	$4.2x10^{-4}$

If you need at other gradients or boundary conditions, please let me know.

Thanks,

Clay Reichert, P.E.

TECHNICAL MANAGER

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From: Justin Roessler [mailto:JRoessler@jonesedmunds.com]
Sent: Wednesday, September 27, 2017 3:26 PM
To: Steve Mayes <smayes@gseworld.com>; George Reinhart <GReinhart@jonesedmunds.com>; Clay Reichert <creichert@gseworld.com>
Subject: RE: New River: Landfill Expansion Geocomposite Data

Thanks Steve,

We are working up a couple of different design scenarios and need some additional information? Would you be able to provide us with the 100-hour transmissivity for the TrxNet for whatever values you have available: 1,10,15,20k would be fine. Would you be able to provide us with preliminary costing information for the FabriNet 300 and the Txr or at a minimum the differential cost between the two materials? I appreciate your assistance in this matter.

Best Regards,

Justin Roessler, PhD
Engineer Intern
Jones Edmunds & Associates, Inc.
352.377.5821 ext. 1360

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From: Steve Mayes [mailto:smayes@gseworld.com]
Sent: Monday, September 25, 2017 3:50 PM
To: Justin Roessler <JRoessler@jonesedmunds.com>; George Reinhart <GReinhart@jonesedmunds.com>; Clay Reichert

Reference 2

Technical Paper by J.P. Giroud, J.G. Zornberg, and A. Zhao

HYDRAULIC DESIGN OF GEOSYNTHETIC AND GRANULAR LIQUID COLLECTION LAYERS

ABSTRACT: The present paper provides equations for the hydraulic design of liquid collection layers. A first series of equations gives the maximum thickness of the liquid collected in a liquid collection layer. These equations are used in design to check that the maximum liquid thickness is less than an allowable thickness. Some of the equations make it possible to rigorously calculate the liquid thickness, whereas other equations, which are simpler, give an approximate value of the liquid thickness. A second series of equations makes it possible to calculate the required hydraulic conductivity of the liquid collection layer material and the required hydraulic transmissivity of the liquid collection layer. These equations are useful for selecting the material used to construct the liqnid collection layer. The equations provided in the present paper include reduction factors to quantify the decrease in flow capacity of liquid collection layers dne to thickness reduction (caused by the applied stresses) and hydraulic conductivity rednction (cansed by clogging). Practical recommendations and design examples are presented for both geosynthetic and granular liquid collection layers.

KEYWORDS: Liquid collection layer, Leachate collection layer, Leakage detection and collection layer, Drainage layer, Thickness, Granular, Geosynthetic.

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REFERENCE: Giroud, J.P., Zornberg, J.G., and Zhao, A., 2000, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers", *Geosynthetics International*, Special Issue on Liquid Collection Systems, Vol. 7, Nos. 4-6, pp. 285-380.

$$k_{LTIS} = \frac{k_{measured}}{\prod(RF)} = \frac{k_{measured}}{RF_{PC} \times RF_{CC} \times RF_{BC}} \quad (15)$$

where: k_{LTIS} = long-term-in-soil hydraulic conductivity of the granular material, i.e. hydraulic conductivity of the granular material located in the soil and subjected to conditions that can cause the development of clogging during the design life of the liquid collection layer; and $k_{measured}$ = hydraulic conductivity of a specimen of granular material representative of the granular material as installed, measured in a hydraulic conductivity test performed with water during a short period of time so that clogging does not develop.

1.7.4 *Factor of Safety*

In addition to the reduction factors described in Sections 1.7.2 and 1.7.3, a factor of safety, *FS*, is used in all calculations to take into account possible uncertainties, such as the fact that the measurement of hydraulic characteristics (i.e. hydraulic conductivity and hydraulic transmissivity) is generally delicate and prone to errors. Values such as 2 or 3, or sometimes greater values, are typically recommended for the factor of safety.

In the equations provided in the present paper, there are two ways of using a factor of safety. The factor of safety can be applied to the maximum liquid thickness, FS_T, or to the relevant hydraulic characteristic, FS_H, i.e. the hydraulic transmissivity in the case of a geosynthetic liquid collection layer or the hydraulic conductivity in the case of a granular liquid collection layer. The two ways (factor of safety on the maximum liquid thickness and factor of safety on the hydraulic characteristic) will be compared.

It is important to note that FS_T and FS_H are not partial factors of safety to be used simultaneously. They are two ways of expressing the factor of safety of the liquid collection layer.

1.7.5 *Use of Reduction Factors and Factor of Safety*

As indicated in Section 1.3, there are two design approaches: the thickness approach (described in Section 3) that consists of calculating the maximum liquid thickness, and the hydraulic characteristic approach (described in Section 4) that consists of calculating the required hydraulic conductivity of the liquid collection layer material or the hydraulic transmissivity of the liquid collection layer. Use of the reduction factors in these two approaches is described in Section 3 for the thickness approach and in Section 4 for the hydraulic characteristic approach.

1.8 Design Options

The flow capacity of a liquid collection layer depends on two sets of characteristics: the intrinsic characteristics of the liquid collection layer and the characteristics of the slope on which the liquid collection layer is installed. The intrinsic characteristics are the thickness of the liquid collection layer and the hydraulic conductivity of the liquid collection layer material (or the hydraulic transmissivity of the liquid collection layer, which is the product of the thickness and hydraulic conductivity). The characteristics

$$\lambda x \approx \frac{t \cos^2\beta}{\sin\beta} \tag{32}$$

hence:

$$t \approx \frac{\lambda x \sin\beta}{\cos^2\beta} = \frac{\lambda x \tan\beta}{\cos\beta} \tag{33}$$

Combining Equations 9 and 33 gives:

$$t = \frac{q_h x}{k \sin\beta} \tag{34}$$

Equation 34 shows that, when λ is very small, the liquid thickness, t, varies linearly with the abscissa, x. Therefore, the maximum value of t occurs for $x = L$, hence:

$$t_{max} \approx \frac{\lambda L \sin\beta}{\cos^2\beta} = \frac{\lambda L \tan\beta}{\cos\beta} = \frac{q_h L}{k \sin\beta} \tag{35}$$

Equation 35 is well known because it can easily be obtained using Darcy's equation, as shown in Section 2.8. Further discussion of Equation 35 can be found in Sections 2.8 and 2.11.

2.3.3 *Numerical Solution*

In 1992, the governing differential equation was solved numerically (Giroud et al. 1992). To solve the differential equation, the boundary condition used to represent free drainage at the toe of the liquid collection layer (in accordance with an assumption presented in Section 1.4.1) was zero liquid thickness and infinite hydraulic gradient. The validity of this boundary condition is discussed in Appendix B. The numerical solution was deemed accurate because it was consistent with values obtained analytically for the two limit cases, i.e. when λ tends toward infinity (Equation 31) and toward zero (Equation 35).

2.4 Empirical Solution Based on Numerical Results

It was noted by Giroud et al. (1992) that the values of the maximum liquid thickness calculated using Equation 16 (or Equation 17, which is equivalent) were very close to the accurate values obtained by numerically solving the governing differential equation. The difference between the values calculated using Equation 16 or 17 and the accurate values is shown in Table 2. It appears in Table 2 that, for typical slopes, the difference between the values calculated using Equation 16 or 17 and the accurate values is less than 13%, the values calculated using Equation 16 or 17 being greater than the accurate values obtained numerically. A dimensionless modifying factor, j, was then added to Equation 16 to improve its accuracy, hence:

$$t_{max} = j \frac{\sqrt{\tan^2\beta + 4 q_h/k} - \tan\beta}{2\cos\beta} L \tag{36}$$

Reference 3

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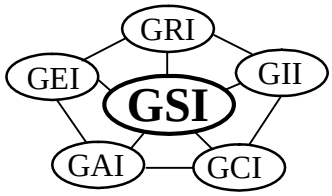

Original: April 17, 2001

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.

*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.

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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:

$$q_{allow} = q_{100}\left[\frac{1}{RF_{CR} \times RF_{CC} \times RF_{BC}}\right] \quad (1)$$

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:

$$FS = \frac{q_{allow}}{q_{reqd}} \quad (2)$$

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 The test specimen shall be the entire geocomposite. 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).

$$RF_{CR} = \left[\frac{\left(t_{CO} / t_{original}\right) - \left(1 - n_{original}\right)}{\left(t_{CR} / t_{original}\right) - \left(1 - n_{original}\right)} \right]^3 \qquad (6)$$

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 μ = 1216 g/m^2 (or 1.216 kg/m^2); density ρ = 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:

$$RF_{CR} = \left[\frac{\left(t_{CO} / t_{original}\right) - \left(1 - n_{original}\right)}{\left(t_{CR} / t_{original}\right) - \left(1 - n_{original}\right)} \right]^3$$

$$= \left[\frac{(7.14 / 8.55) - (1 - 0.850)}{(6.30 / 8.55) - (1 - 0.850)} \right]^3$$

$$= \left[\frac{0.835 - 0.150}{0.737 - 0.150} \right]^3$$

$$= \left[\frac{0.685}{0.587} \right]^3$$

$$RF_{CR} = 1.59$$

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

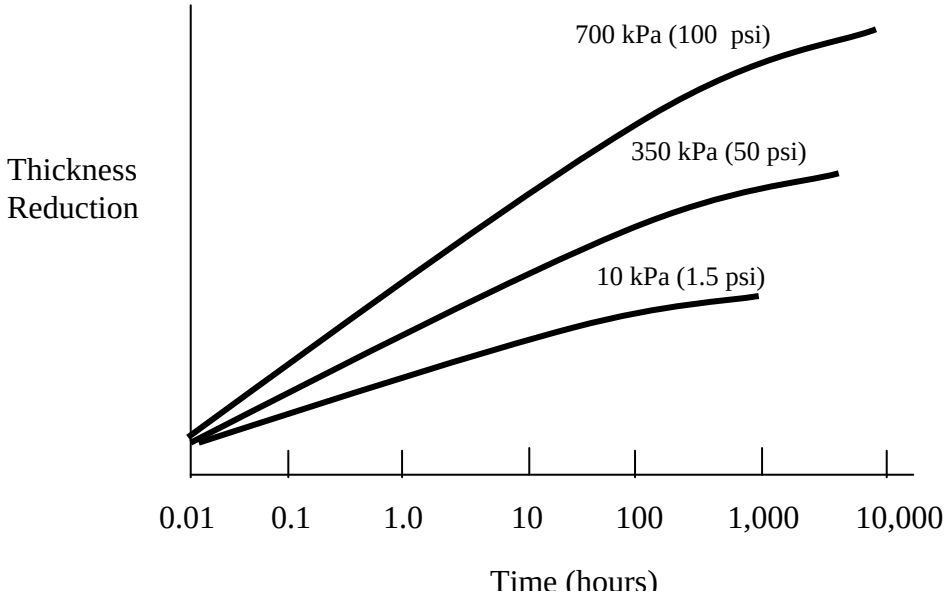
10.1 For a candidate drainage geocomposite, the 100-hour flow rate behavior under the site-specific set of variables, e.g., specimen orientation, stress level, hydraulic gradient, and permeating liquid is to be obtained per ASTM D4716 following procedures of Section 6.0.

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.

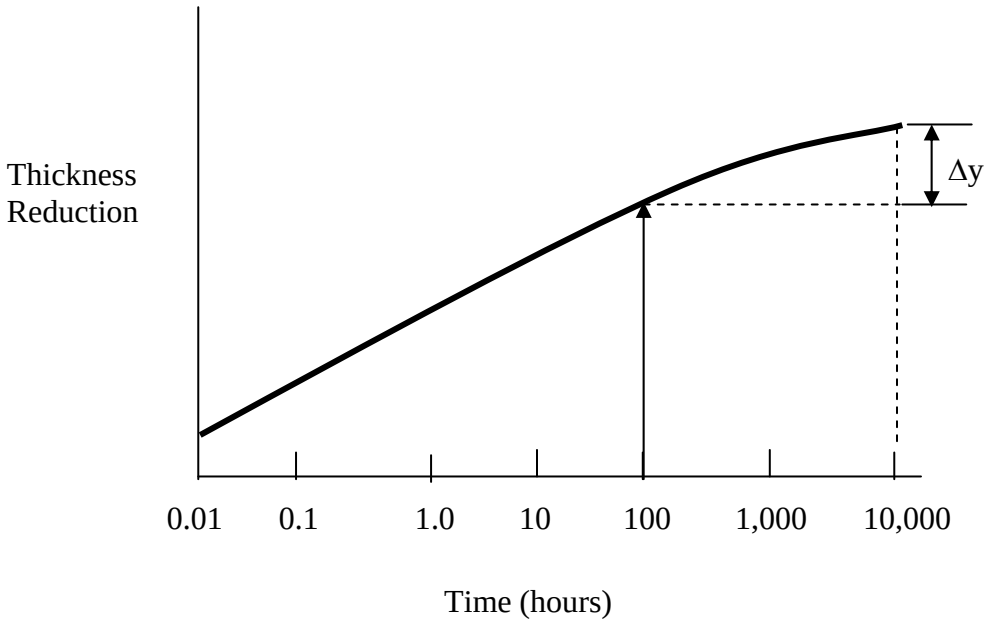
10.3 A reduction factor for chemical and/or biological clogging, as discussed in Section 8.0 can be included. It is very much a site-specific situation at the discretion of the parties involved.

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

Reference 4

Reference 4

Pasco County, Florida

West Pasco County Landfill Expansion
Cells A-3 and SW-2

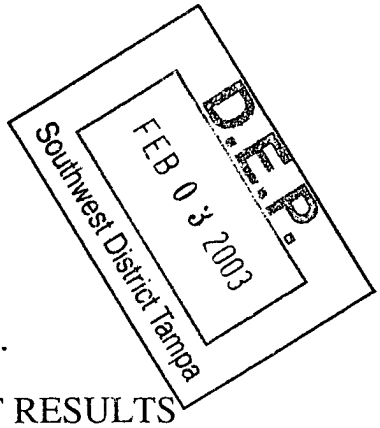
Certification of Construction Completion and Quality Assurance Documentation

January 2003

Volume 2 of 4

Reference 4

West Pasco County Landfill Expansion
Cells A-3 and SW-2
Certification of Construction Completion



VOLUME 2 OF 4

SECTION 6 GEOMEMBRANE CONFORMANCE TEST RESULTS SETS 3-7 OF 7

SECTION 7 DESTRUCTIVE SAMPLE RESULTS A-3

SECTION 8 DESTRUCTIVE SAMPLE RESULTS SW-2

SECTION 9 GEOCOMPOSITE NET CONFORMANCE TEST RESULTS

TILE CONFORMANCE TEST RESULTS

UCTION PROGRESS PHOTOS

LOT	T (m2/s) (10^-3)
1	2.23
2	2.49
3	3.29
4	2.87
5	1.80
6	2.41
7	2.58
8	1.8
9	2.75
10	1.82
11	3.90
12	2.32
13	2.01
14	1.8
15	5.08
16	2.25
17	2.36
18	1.80
19	2.31
20	2.94
21	3.38
22	2.48
23	2.03
24	1.93
25	3.23
26	2.24
AVG	2.54 x 10^-3 m2/s

VOLUME 2

Compiled Test Data (26 LOTS), avg. transmissivity for 100 hr seating time, 0.1 gradient, 15,000 psf load presented

Attachment J.1-3

Structural Pipe Design

TECHNICAL MEMORANDUM

WEST PASCO COUNTY LANDFILL ASH MONOFILL SLOPE MODIFICATION AND VERTICAL EXPANSION

TO: Project File

FROM: Thomas Le Blanc, EI

XC: George Reinhart, PhD, PE

DATE: June 29 2021

SUBJECT: Attachment J.1-3 – Structural Pipe Analysis
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

Florida Department of Environmental Protection (FDEP) Rule 62-701.400(4)(a)2, Florida Administrative Code (FAC), requires that leachate pipes within the landfill have sufficient mechanical properties to prevent collapse under pressures exerted by the overlying wastes and cover materials. This calculation demonstrates the structural integrity of the Leachate Collection and Removal System (LCRS) pipes for post-construction conditions under the maximum imposed load of the landfill.

2 METHODOLOGY

The structural integrity of the pipes is evaluated in terms of (1) pipe deflection, (2) ring compressive stress, (3) pipe wall buckling, and (4) ring-bending strain.

Pipe deflection calculations show that the pipe deflection is within allowable standards for the standard dimension ratio (SDR) of the pipe. The pipes are designed to deflect, but over-deflection can result in reversal of curvature and failure in the pipe.

Ring compressive stress (also called *wall crushing*) is the radially directed soil pressure creating a ring compressive stress, like squeezing a pipe. For these calculations, the soil pressure is assumed to be uniform around the pipe. We have included a correction for the perforated pipe to evaluate if the perforated pipe will resist compressive stress. The correction for the perforations is a very conservative estimate and is neither based on physical properties of the pipe nor on an empirical relationship of the pipe strength and applied stress.

Wall-buckling calculations show that the pipe wall thickness is sufficient to resist buckling. The required SDR (ratio of pipe diameter to wall thickness) is determined so that the pipe's

allowable constrained buckling pressure is greater than the effective pressure on the pipe due to the maximum landfill load.

Ring-bending strain calculations show that the pipe wall strain (when the pipe is deformed to an elliptical shape) is less than the allowable strain for the pipe material. The strain is calculated for the maximum pipe deflection (calculated in the ring-deflection calculations) and compared to the allowable strain to determine a factor of safety against ring-bending strain failure.

The maximum imposed load is conservatively based on the maximum load of the bottom liner, independent of the pipe location. The maximum imposed load is calculated based on the modified arching load, which is considered a conservative approach for flexible pipe. The approach is to add 40 percent of the difference between the prism load and the Marston load to the Marston load. Prism load is the sum of material unit weight multiplied by material thickness above the pipe. The prism load calculation is conservative because it neglects soil arching, which occurs as the load is transferred from the prism load to the bedding sidewalls as the load settles. As the load is transferred to the sidewalls, the bedding compresses and stiffens and provides an arching support for the soil load. Marston load takes into account the reduced load on the pipe due to the soil arching. Thus, the modified arching load is a conservative estimate of the vertical load on the pipe and includes an inherent factor of safety against failure.

3 PARAMETERS

Important parameters are the landfill and cover load, bedding material strength, pipe strength and elasticity, and pipe perforation size and density (holes per row per foot). The maximum landfill load is taken from the Settlement Analysis (Attachment I.1.e(2)). The landfill geometry is shown on the Engineering Plans in Appendix A. The pipe strength is given by the manufacturer. The pipe size and pipe perforations are determined by the design.

4 CONCLUSIONS

The leachate collection systems are varied between the two pre-existing cells, A-1, A-2, A-3, and A-4. The primary collection system for A-1 and A-2 consists of 6-inch-diameter SDR-17 high-density polyethylene (HDPE) perforated LCRS pipes, and the secondary collection system consists of 4-inch-diameter and 6-inch-diameter perforated LCRS pipes, for A-1 and A-2, respectively. The primary and secondary collection systems for A-3 and A-4 consist of 8-inch-diameter SDR-17 HDPE perforated LCRS pipes.

Based on the pipe structural integrity calculations, the perforated LCRS pipes encased in gravel bedding will be structurally stable under the maximum and long-term landfill loads for cells A-1, A-2, A-3, and A-4.

JonesEdmunds

PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 4 INCH SDR 17 HDPE A-1 LEACHATE DETECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads
Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o)
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain

Data:

1. Soil and waste parameters

γ =	average specific weight of landfilled material (conservative estimate) =	110 pcf	*(Reference 1)*
φ =	internal soil friction angle [º] for gravel trench =	35 °	*(Ref. 2, Table 11.3)*
E' =	soil reaction modulus =	3,000 psi	*(Ref. 3, Table 3.4)*
	(gravel trench crushed rock, moderate compaction)		

2. Pipe dimensions and material properties

D_o =	outside diameter of pipe (the same value as B_c)	4.5 in	*(Reference 4)*
		0.375 ft	*Conversion from in to ft*
t =	minimum wall thickness	0.265 in	*(Reference 4)*
		0.02 ft	*Conversion from in to ft*
E =	elastic modulus of the pipe material (50 year,100°F)	23,000 psi	*(Ref. 5, Table 5-1)*
	Pipe perforation diameter =	0.375 in	*(Reference 6)*
	Pipe hole spacing between rows (x) =	3 in (on center)	*(Reference 6)*
	Number of holes/row =	2	*(Reference 6)*
	4 -inch diameter SDR 17.00 HDPE		*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e =	distance from top of pipe to plane of equal settlement =	3.0 ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H =	distance from top of pipe to top of landfill at max load =	73.0 ft	*(Part I)*
W_p =	Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001
5 CPCHEM, *The Performance Pipe Engineering Manual*. Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities*. SW-870, March 1983 Revised Edition, p. 382

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2$$ *(Ref. 3, Eqn 2.5)*

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)}$$ *(Ref. 3, Eqn 2.7)*

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	11	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.375	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	3.0	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	73	ft	*(Part I)*
W_c =	**11 x 110 x 0.38^2 =**	**177**	**lb/ft =**	**15 lb/in**

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c$$ *(Ref 5, Eqn 7-5)*

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	177	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*(Part I)*
W_{mod} =	**0.4 x (9000 - 177) + 177) =**	**3,706**	**lb/ft =**	**309 lb/in**
	Actual Estimated Load on Pipe			

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$ *(Ref 3, Eqn 3.5)*

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	309	lb/in	*Calculated*
r =	mean radius of the pipe = (D_o - 2 t)/2 =	1.985	in	*Calculated*
D_o =	outside diameter of pipe =	4.5	in	*(Reference 4)*
t =	minimum wall thickness	0.265	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress) *(50-year lifespan at 100 degrees F)*	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0016	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*
ΔX =	**(1.5x0.1x309x1.99^3) / (23000x0.0016 + 0.061x3000x1.99^3) =**	**0.25**	**in**	

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

$\Delta X / D_o$ = 5.5%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = 7.5% *(Ref 5, Page 112)*

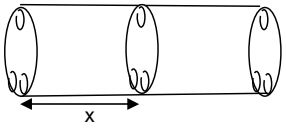
5.5% < 7.5%

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12-L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o =	9,884	psf	*(Equals W_{mod} / D_o)*
	69	psi	*Conversion, psf to psi*
L_P = total accumulated length of perforations in one foot of pipe			
Perforation diameter =	0.375	in	*(Reference 6)*
Hole spacing between rows (x) =	3	in (on center)	*(Reference 6)*
Lp =	1.5	in	*Calculated*
P_{EFF} = (9884 x 12) / (12 - 1.5) =	**11,296**	**psf**	*Calculated*
	78	**psi**	*Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]			
P_T = effective vertical load applied to pipe = P_{EFF} =	78	psi	*Calculated*
D_o = Pipe diameter	4.500	in	*(Reference 4)*
A = cross-sectional area per unit length = t =	0.265	in^2/in	
S_A = 78 x 4.5 / (2 x 0.265) =	**666**	**psi**	*Calculated*

c. Compare the calculated compressive strength to the recommended (maximum allowable stress) long-term compressive strength given by the manufacturer.

The recommended, long-term (maximum allowable stress) compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		**Y_s [psi]**	
LCS Pipe	**666**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.2

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.00	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	73	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.97		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0016	in^4/in	*Calculated*
D_o =	pipe outer diameter =	4.500	in	*(Reference 4)*
t =	pipe wall thickness =	0.265	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	3.97	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.97 x 3000 x 23000 / 12(17 - 1)^3] =**	**104**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**3**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**69**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**31.7**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**1.5**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **1.5**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	4.22	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.140	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.25		*Calculated*
ε =		**2.3%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**1.8**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe

Factor of Safety = 1.8

JonesEdmunds

PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 6 INCH SDR 17 HDPE A-1 LEACHATE COLLECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads. Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain.

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling.
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o)
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain

Data:

1. Soil and waste parameters

γ = average specific weight of landfilled material (conservative estimate) =	110	pcf	*(Reference 1)*
φ = internal soil friction angle [º] for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
E' = soil reaction modulus =	3,000	psi	*(Ref. 3, Table 3.4)*
(gravel trench crushed rock, moderate compaction)			

2. Pipe dimensions and material properties

D_o = outside diameter of pipe (the same value as B_c)	6.625	in	*(Reference 4)*
	0.552	ft	*Conversion from in to ft*
t = minimum wall thickness	0.39	in	*(Reference 4)*
	0.03	ft	*Conversion from in to ft*
E = elastic modulus of the pipe material (50 year,100°F)	23,000	psi	*(Ref. 5, Table 5-1)*
Pipe perforation diameter =	0.375	in	*(Reference 6)*
Pipe hole spacing between rows (x) =	3	in (on center)	*(Reference 6)*
Number of holes/row =	2		*(Reference 6)*
6 -inch diameter SDR 17.00 HDPE			*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings. *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e = distance from top of pipe to plane of equal settlement =	1.3	ft	*(Reference 6)*
(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H = distance from top of pipe to top of landfill at max load =	71.3	ft	*(Part I)*
W_p = Maximum prism load on bottom liner (rounded up for conservatism) =	9,000	psf	*(Part I)*

References:

1 WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001.
5 CPCHEM, *The Performance Pipe Engineering Manual*. Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities.* SW-870, March 1983 Revised Edition, p. 382

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2$$ *(Ref. 3, Eqn 2.5)*

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)}$$ *(Ref. 3, Eqn 2.7)*

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	52	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.552	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	1.3	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	71.3	ft	*Part I*

W_c = 52 x 110 x 0.55^2 = 1,753 lb/ft = 146 lb/in

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c$$ *(Ref 5, Eqn 7-5)*

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	1,753	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*Part I*

W_{mod} = 0.4 x (9000 - 1753) + 1753) = 4,652 lb/ft = 388 lb/in
Actual Estimated Load on Pipe

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$ *(Ref 3, Eqn 3.5)*

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	388	lb/in	*Calculated*
r =	mean radius of the pipe = (D_o - 2 t)/2 =	2.923	in	*Calculated*
D_o =	outside diameter of pipe =	6.625	in	*(Reference 4)*
t =	minimum wall thickness	0.39	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress)	23,000	psi	*(Ref 5, Table 5-1)*
	(50-year lifespan at 100 degrees F)			
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0049	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*

ΔX = (1.5x0.1x388x2.92^3) / (23000x0.0049 + 0.061x3000x2.92^3) = 0.31 in

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

$\Delta X / D_o$ = 4.7%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = **7.5%** *(Ref 5, Page 112)*

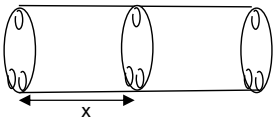
4.7% < **7.5%**

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12\text{-}L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o =	8,426	psf	*(Equals W_{mod} / D_o)*
	59	psi	*Conversion, psf to psi*
L_P = total accumulated length of perforations in one foot of pipe			
Perforation diameter =	0.375	in	*(Reference 6)*
Hole spacing between rows (x) =	3	in (on center)	*(Reference 6)*
Lp =	1.5	in	*Calculated*
P_{EFF} = (8426 x 12) / (12 - 1.5) =	**9,630**	**psf**	*Calculated*
	67	**psi**	*Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]			
P_T = effective vertical load applied to pipe = P_{EFF} =	67	psi	*Calculated*
D_o = Pipe diameter	6.625	in	*(Reference 4)*
A = cross-sectional area per unit length = t =	0.39	in^2/in	
S_A = 67 x 6.625 / (2 x 0.39) =	**568**	**psi**	*Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		Y_s [psi]	
LCS Pipe	**568**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.4

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.45	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	71.3	ft max	*(Part I)*
B' =	elastic support factor = (1 + 4*e$^{-0.065H}$)$^{-1}$ =	0.96		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = t^3/12 =	0.0049	in^4/in	*Calculated*
D_o =	pipe outer diameter =	6.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.390	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	5.845	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.96 x 3000 x 23000 / 12(17 - 1)^3] =**	**104**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**22**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**59**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = P_{wc} / (W_c/D_o) =	**4.7**
Factor of Safety for Modified Arching Load = P_{wc} / (W_{mod}/D_o) =	**1.8**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **1.8**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	6.21	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.207	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.31		*Calculated*
ε =		**2.0%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.1**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe.

Factor of Safety = 2.1

JonesEdmunds

PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 6 INCH SDR 17 HDPE A-2 LEACHATE DETECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads
Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o)
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain

Data:

1. Soil and waste parameters

γ =	average specific weight of landfilled material (conservative estimate) =	110 pcf	*(Reference 1)*
φ =	internal soil friction angle [º] for gravel trench =	35 °	*(Ref. 2, Table 11.3)*
E' =	soil reaction modulus =	3,000 psi	*(Ref. 3, Table 3.4)*
	(gravel trench crushed rock, moderate compaction)		

2. Pipe dimensions and material properties

D_o =	outside diameter of pipe (the same value as B_c)	6.625 in	*(Reference 4)*
		0.552 ft	*Conversion from in to ft*
t =	minimum wall thickness	0.39 in	*(Reference 4)*
		0.03 ft	*Conversion from in to ft*
E =	elastic modulus of the pipe material (50 year,100°F)	23,000 psi	*(Ref. 5, Table 5-1)*
	Pipe perforation diameter =	0.375 in	*(Reference 6)*
	Pipe hole spacing between rows (x) =	2.5 in (on center)	*(Reference 6)*
	Number of holes/row =	2	*(Reference 6)*
	6 -inch diameter SDR 17.00 HDPE		*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e =	distance from top of pipe to plane of equal settlement =	2.5 ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H =	distance from top of pipe to top of landfill at max load =	75.0 ft	*(Part I)*
W_p =	Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001
5 CPCHEM, *The Performance Pipe Engineering Manual*. Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities*. SW-870, March 1983 Revised Edition, p. 382

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2$$ *(Ref. 3, Eqn 2.5)*

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)}$$ *(Ref. 3, Eqn 2.7)*

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]		
C_c =	Coefficient for Marston Load calculations	26 -	*Calculated*
γ =	average specific weight of landfilled material =	110 pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.552 ft	*(Reference 4)*
e =	natural log base number		
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27	*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35 °	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70	*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	2.5 ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H =	distance from top of pipe to top of embankment	75.0 ft	*(Part I)*

W_c = 26 x 110 x 0.55^2 = 862 lb/ft = 72 lb/in

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c$$ *(Ref 5, Eqn 7-5)*

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]		
W_c =	Vertical load per unit length of pipe	862 lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000 lb/ft	*(Part I)*

W_{mod} = 0.4 x (9000 - 862) + 862) = 4,117 lb/ft = 343 lb/in

Actual Estimated Load on Pipe

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$ *(Ref 3, Eqn 3.5)*

where:

ΔX =	horizontal deflection [in]		
D_L =	deflection lag factor =	1.5	*(Reference 3, Page 52)*
K =	bedding constant =	0.1	*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	343 lb/in	*Calculated*
r =	mean radius of the pipe = (D_o - 2 t)/2 =	2.923 in	*Calculated*
D_o =	outside diameter of pipe =	6.625 in	*(Reference 4)*
t =	minimum wall thickness	0.39 in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress) *(50-year lifespan at 100 degrees F)*	23,000 psi	*(Ref 5, Table 5-1)*
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0049 in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000 lb/in^2	*(Ref 3, Table 3.4)*

ΔX = (1.5x0.1x343x2.92^3) / (23000x0.0049 + 0.061x3000x2.92^3) = 0.27 in

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

ΔX / D_o = 4.1%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = 7.5% *(Ref 5, Page 112)*

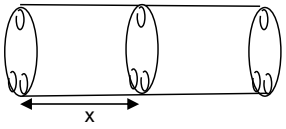
4.1% < 7.5%

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12-L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o =	7,458 psf	*(Equals W_{mod} / D_o)*
	52 psi	*Conversion, psf to psi*
L_P = total accumulated length of perforations in one foot of pipe		
Perforation diameter =	0.375 in	*(Reference 6)*
Hole spacing between rows (x) =	2.5 in (on center)	*(Reference 6)*
Lp =	1.8 in	*Calculated*
P_{EFF} = (7458 x 12) / (12 - 1.8) =	**8,774 psf**	*Calculated*
	61 psi	*Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]		
P_T = effective vertical load applied to pipe = P_{EFF} =	61 psi	*Calculated*
D_o = Pipe diameter	6.625 in	*(Reference 4)*
A = cross-sectional area per unit length = t =	0.39 in^2/in	
S_A = 61 x 6.625 / (2 x 0.39) =	**518 psi**	*Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		Y_s [psi]	
LCS Pipe	**518**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.5

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.00	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	75.0	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.97		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0049	in^4/in	*Calculated*
D_o =	pipe outer diameter =	6.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.390	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	5.845	in	*Calculated*
P_{wc} =	**5.65 / 2 x SQRT[1 x 0.97 x 3000 x 23000 / (17 - 1)^3] =**	**104**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**11**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**52**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**9.6**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**2.0**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **2.0**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	6.21	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.207	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.27		*Calculated*
ε =		**1.8%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.4**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe

Factor of Safety = 2.4

PART I - 6 INCH SDR 17 HDPE A-2 LEACHATE COLLECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads. Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain.

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling.
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o).
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain.

Data:

1. Soil and waste parameters

γ =	average specific weight of landfilled material (conservative estimate) =	110 pcf	*(Reference 1)*
φ =	internal soil friction angle [°] for gravel trench =	35 °	*(Ref. 2, Table 11.3)*
E' =	soil reaction modulus =	3,000 psi	*(Ref. 3, Table 3.4)*
	(gravel trench crushed rock, moderate compaction)		

2. Pipe dimensions and material properties

D_o =	outside diameter of pipe (the same value as B_c)	6.625 in	*(Reference 4)*
		0.552 ft	*Conversion from in to ft*
t =	minimum wall thickness	0.39 in	*(Reference 4)*
		0.03 ft	*Conversion from in to ft*
E =	elastic modulus of the pipe material (50 year,100°F)	23,000 psi	*(Ref. 5, Table 5-1)*
	Pipe perforation diameter =	0.375 in	*(Reference 6)*
	Pipe hole spacing between rows (x) =	2.5 in (on center)	*(Reference 6)*
	Number of holes/row =	2	*(Reference 6)*
	6 -inch diameter SDR 17.00 HDPE		*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings. *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e =	distance from top of pipe to plane of equal settlement =	1.5 ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H =	distance from top of pipe to top of landfill at max load =	74.0 ft	*(Part I)*
W_p =	Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001.
5 CPCHEM, *The Performance Pipe Engineering Manual* . Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities.* SW-870, March 1983 Revised Edition, p. 382

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2 \qquad \textit{(Ref. 3, Eqn 2.5)}$$

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)} \qquad \textit{(Ref. 3, Eqn 2.7)}$$

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	49	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.552	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	1.5	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	74.0	ft	*Part I*
W_c =	**49 x 110 x 0.55^2 =**	**1,627**	**lb/ft =**	**136 lb/in**

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c \qquad \textit{(Ref 5, Eqn 7-5)}$$

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	1,627	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*Part I*
W_{mod} =	**0.4 x (9000 - 1627) + 1627) =**	**4,576**	**lb/ft =**	**381 lb/in**
	Actual Estimated Load on Pipe			

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3} \qquad \textit{(Ref 3, Eqn 3.5)}$$

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	381	lb/in	*Calculated*
r =	mean radius of the pipe = (D_o - 2 t)/2 =	2.923	in	*Calculated*
D_o =	outside diameter of pipe =	6.625	in	*(Reference 4)*
t =	minimum wall thickness	0.39	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress)	23,000	psi	*(Ref 5, Table 5-1)*
	(50-year lifespan at 100 degrees F)			
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0049	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*
ΔX =	**(1.5x0.1x381x2.92^3) / (23000x0.0049 + 0.061x3000x2.92^3) =**		**0.30**	**in**

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

$\Delta X / D_o$ = 4.6%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = **7.5%** *(Ref 5, Page 112)*

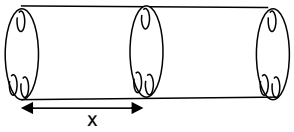
4.6% < **7.5%**

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12\text{-}L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o =	8,289	psf	*(Equals W_{mod} / D_o)*
	58	psi	*Conversion, psf to psi*
L_P = total accumulated length of perforations in one foot of pipe			
Perforation diameter =	0.375	in	*(Reference 6)*
Hole spacing between rows (x) =	2.5	in (on center)	*(Reference 6)*
Lp =	1.8	in	*Calculated*
P_{EFF} = (8289 x 12) / (12 - 1.8) =	**9,752**	**psf**	*Calculated*
	68	**psi**	*Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]			
P_T = effective vertical load applied to pipe = P_{EFF} =	68	psi	*Calculated*
D_o = Pipe diameter	6.625	in	*(Reference 4)*
A = cross-sectional area per unit length = t =	0.39	in^2/in	
S_A = 68 x 6.625 / (2 x 0.39) =	**575**	**psi**	*Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		Y_s [psi]	
LCS Pipe	**575**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.4

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.45	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	74.0	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.97		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0049	in^4/in	*Calculated*
D_o =	pipe outer diameter =	6.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.390	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	5.845	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.97 x 3000 x 23000 / (17 - 1)^3] =**	**104**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**20**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**58**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**5.1**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**1.8**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **1.8**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	6.21	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.207	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.30		*Calculated*
ε =		**2.0%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.1**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe.

Factor of Safety = 2.1

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PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 8 INCH SDR 17 HDPE A-3 LEACHATE DETECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads. Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain.

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling.
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o).
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain.

Data:

1. Soil and waste parameters

γ = average specific weight of landfilled material (conservative estimate) =	110 pcf		*(Reference 1)*
φ = internal soil friction angle [º] for gravel trench =	35 °		*(Ref. 2, Table 11.3)*
E' = soil reaction modulus =	3,000 psi		*(Ref. 3, Table 3.4)*
(gravel trench crushed rock, moderate compaction)			

2. Pipe dimensions and material properties

D_o = outside diameter of pipe (the same value as B_c)	8.625 in		*(Reference 4)*
	0.719 ft		*Conversion from in to ft*
t = minimum wall thickness	0.507 in		*(Reference 4)*
	0.04 ft		*Conversion from in to ft*
E = elastic modulus of the pipe material (50 year,100°F)	23,000 psi		*(Ref. 5, Table 5-1)*
Pipe perforation diameter =	0.500 in		*(Reference 6)*
Pipe hole spacing between rows (x) =	3 in	(on center)	*(Reference 6)*
Number of holes/row =	2		*(Reference 6)*
8 -inch diameter SDR 17.00 HDPE			*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings. *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e = distance from top of pipe to plane of equal settlement =	2.3 ft	*(Reference 6)*
(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H = distance from top of pipe to top of landfill at max load =	81.0 ft	*(Part I)*
W_p = Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001.
5 CPCHEM, *The Performance Pipe Engineering Manual* . Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities.* SW-870, March 1983 Revised Edition, p. 382

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PROJECT NUMBER: 16002-009-01 SHEET: 2 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2$$ *(Ref. 3, Eqn 2.5)*

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)}$$ *(Ref. 3, Eqn 2.7)*

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	34	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.719	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	2.3	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	81.0	ft	*(Part I)*
W_c =	**34 x 110 x 0.72^2 =**	**1,920**	**lb/ft =**	**160 lb/in**

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c$$ *(Ref 5, Eqn 7-5)*

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	1,920	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*(Part I)*
W_{mod} =	**0.4 x (9000 - 1920) + 1920) =**	**4,752**	**lb/ft =**	**396 lb/in**
	Actual Estimated Load on Pipe			

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$ *(Ref 3, Eqn 3.5)*

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	396	lb/in	*Calculated*
r =	mean radius of the pipe = $(D_o - 2t)/2$ =	3.806	in	*Calculated*
D_o =	outside diameter of pipe =	8.625	in	*(Reference 4)*
t =	minimum wall thickness	0.507	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress)	23,000	psi	*(Ref 5, Table 5-1)*
	(50-year lifespan at 100 degrees F)			
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0109	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*
ΔX =	**(1.5x0.1x396x3.81^3) / (23000x0.0109 + 0.061x3000x3.81^3) =**			**0.32 in**

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

ΔX / D_o = 3.7%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = 7.5% *(Ref 5, Page 112)*

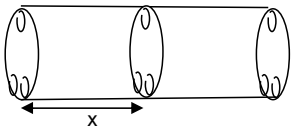
3.7% < **7.5%**

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12-L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o = 6,612 psf *(Equals W_{mod} / D_o)*

46 psi *Conversion, psf to psi*

L_P = total accumulated length of perforations in one foot of pipe

Perforation diameter = 0.500 in *(Reference 6)*

Hole spacing between rows (x) = 3 in (on center) *(Reference 6)*

Lp = 2 in *Calculated*

P_{EFF} = (6612 x 12) / (12 - 2) = 7,934 psf *Calculated*

55 psi *Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]

P_T = effective vertical load applied to pipe = P_{EFF} = 55 psi *Calculated*

D_o = Pipe diameter 8.625 in *(Reference 4)*

A = cross-sectional area per unit length = t = 0.507 in^2/in

S_A = 55 x 8.625 / (2 x 0.507) = 469 psi *Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		Y_s [psi]	
LCS Pipe	**469**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.7

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.00	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	81.0	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.98		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0109	in^4/in	*Calculated*
D_o =	pipe outer diameter =	8.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.507	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	7.611	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.98 x 3000 x 23000 / (17 - 1)^3] =**	**105**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**19**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**46**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**5.6**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**2.3**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **2.3**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	8.09	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.269	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.32		*Calculated*
ε =		**1.6%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.7**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe.

Factor of Safety = 2.7

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PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 8-INCH SDR 17 HDPE A-3 LEACHATE COLLECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads. Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain.

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling.
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o)
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain

Data:

1. Soil and waste parameters

γ = average specific weight of landfilled material (conservative estimate) =	110 pcf		*(Reference 1)*
φ = internal soil friction angle [°] for gravel trench =	35 °		*(Ref. 2, Table 11.3)*
E' = soil reaction modulus =	3,000 psi		*(Ref. 3, Table 3.4)*
(gravel trench crushed rock, moderate compaction)			

2. Pipe dimensions and material properties

D_o = outside diameter of pipe (the same value as B_c)	8.625 in		*(Reference 4)*
	0.719 ft		*Conversion from in to ft*
t = minimum wall thickness	0.507 in		*(Reference 4)*
	0.04 ft		*Conversion from in to ft*
E = elastic modulus of the pipe material (50 year,100°F)	23,000 psi		*(Ref. 5, Table 5-1)*
Pipe perforation diameter =	0.500 in		*(Reference 6)*
Pipe hole spacing between rows (x) =	3 in	(on center)	*(Reference 6)*
Number of holes/row =	2		*(Reference 6)*
8 -inch diameter SDR 17.00 HDPE			*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings. *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e = distance from top of pipe to plane of equal settlement =	1.2 ft	*(Reference 6)*
(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H = distance from top of pipe to top of landfill at max load =	80.0 ft	*(Part I)*
W_p = Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001.
5 CPCHEM, *The Performance Pipe Engineering Manual* . Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities.* SW-870, March 1983 Revised Edition, p. 382

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2 \qquad \textit{(Ref. 3, Eqn 2.5)}$$

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)} \qquad \textit{(Ref. 3, Eqn 2.7)}$$

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	60	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.719	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi/2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	1.2	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	80.0	ft	*(Part I)*
W_c =	**60 x 110 x 0.72^2 =**	**3,435**	**lb/ft =**	**286 lb/in**

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\,W_C + 0.4\,W_P = 0.4\,(W_p - W_c) + W_c \qquad \textit{(Ref 5, Eqn 7-5)}$$

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	3,435	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*(Part I)*
W_{mod} =	**0.4 x (9000 - 3435) + 3435) =**	**5,661**	**lb/ft =**	**472 lb/in**
	Actual Estimated Load on Pipe			

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3} \qquad \textit{(Ref 3, Eqn 3.5)}$$

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	472	lb/in	*Calculated*
r =	mean radius of the pipe = (D_o - 2 t)/2 =	3.806	in	*Calculated*
D_o =	outside diameter of pipe =	8.625	in	*(Reference 4)*
t =	minimum wall thickness	0.507	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress)	23,000	psi	*(Ref 5, Table 5-1)*
	(50-year lifespan at 100 degrees F)			
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0109	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*
ΔX =	**(1.5x0.1x472x3.81^3) / (23000x0.0109 + 0.061x3000x3.81^3) =**		**0.38**	**in**

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

$\Delta X / D_o$ = 4.4%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = 7.5% *(Ref 5, Page 112)*

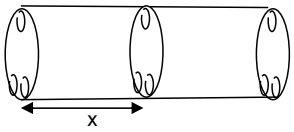
4.4% < **7.5%**

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12-L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o =	7,876	psf	*(Equals W_{mod} / D_o)*
	55	psi	*Conversion, psf to psi*
L_P = total accumulated length of perforations in one foot of pipe			
Perforation diameter =	0.500	in	*(Reference 6)*
Hole spacing between rows (x) =	3	in (on center)	*(Reference 6)*
Lp =	2	in	*Calculated*
P_{EFF} = (7876 x 12) / (12 - 2) =	**9,452**	**psf**	*Calculated*
	66	**psi**	*Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]			
P_T = effective vertical load applied to pipe = P_{EFF} =	66	psi	*Calculated*
D_o = Pipe diameter	8.625	in	*(Reference 4)*
A = cross-sectional area per unit length = t =	0.507	in^2/in	
S_A = 66 x 8.625 / (2 x 0.507) =	**558**	**psi**	*Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		**Y_s [psi]**	
LCS Pipe	**558**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.4

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.28	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	80.0	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.98		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0109	in^4/in	*Calculated*
D_o =	pipe outer diameter =	8.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.507	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	7.611	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.98 x 3000 x 23000 / (17 - 1)^3] =**	**105**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**33**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**55**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**3.1**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**1.9**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **1.9**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	8.09	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.269	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.38		*Calculated*
ε =		**1.9%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.3**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe.

Factor of Safety = 2.3

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PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 8 INCH SDR 17 HDPE A-4 LEACHATE DETECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads. Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain.

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling.
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o).
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain.

Data:

1. Soil and waste parameters

γ =	average specific weight of landfilled material (conservative estimate) =	110 pcf	*(Reference 1)*
φ =	internal soil friction angle [º] for gravel trench =	35 °	*(Ref. 2, Table 11.3)*
E' =	soil reaction modulus =	3,000 psi	*(Ref. 3, Table 3.4)*
	(gravel trench crushed rock, moderate compaction)		

2. Pipe dimensions and material properties

D_o =	outside diameter of pipe (the same value as B_c)	8.625 in	*(Reference 4)*
		0.719 ft	*Conversion from in to ft*
t =	minimum wall thickness	0.507 in	*(Reference 4)*
		0.04 ft	*Conversion from in to ft*
E =	elastic modulus of the pipe material (50 year,100°F)	23,000 psi	*(Ref. 5, Table 5-1)*
	Pipe perforation diameter =	0.500 in	*(Reference 6)*
	Pipe hole spacing between rows (x) =	3 in (on center)	*(Reference 6)*
	Number of holes/row =	2	*(Reference 6)*
	8 -inch diameter SDR 17.00 HDPE		*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings. *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e =	distance from top of pipe to plane of equal settlement =	2.3 ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H =	distance from top of pipe to top of landfill at max load =	79.3 ft	*(Part I)*
W_p =	Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001.
5 CPCHEM, *The Performance Pipe Engineering Manual*. Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities.* SW-870, March 1983 Revised Edition, p. 382

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PROJECT NUMBER: 16002-009-01 SHEET: 2 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2$$ *(Ref. 3, Eqn 2.5)*

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)}$$ *(Ref. 3, Eqn 2.7)*

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	33	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.719	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	2.3	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	79.3	ft	*(Part I)*
W_c =	**33 x 110 x 0.72^2 =**	**1,881**	**lb/ft =**	**157 lb/in**

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c$$ *(Ref 5, Eqn 7-5)*

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	1,881	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*(Part I)*
W_{mod} =	**0.4 x (9000 - 1881) + 1881) =**	**4,729**	**lb/ft =**	**394 lb/in**
	Actual Estimated Load on Pipe			

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$ *(Ref 3, Eqn 3.5)*

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	394	lb/in	*Calculated*
r =	mean radius of the pipe = $(D_o - 2t)/2$ =	3.806	in	*Calculated*
D_o =	outside diameter of pipe =	8.625	in	*(Reference 4)*
t =	minimum wall thickness	0.507	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress)	23,000	psi	*(Ref 5, Table 5-1)*
	(50-year lifespan at 100 degrees F)			
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0109	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*
ΔX =	**(1.5x0.1x394x3.81^3) / (23000x0.0109 + 0.061x3000x3.81^3) =**			**0.32 in**

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

ΔX / D_o = 3.7%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = 7.5% *(Ref 5, Page 112)*

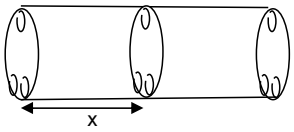
3.7% < **7.5%**

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12-L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o = 6,579 psf *(Equals W_{mod} / D_o)*

46 psi *Conversion, psf to psi*

L_P = total accumulated length of perforations in one foot of pipe

Perforation diameter = 0.500 in *(Reference 6)*

Hole spacing between rows (x) = 3 in (on center) *(Reference 6)*

Lp = 2 in *Calculated*

P_{EFF} = (6579 x 12) / (12 - 2) = 7,895 psf *Calculated*

55 psi *Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]

P_T = effective vertical load applied to pipe = P_{EFF} = 55 psi *Calculated*

D_o = Pipe diameter 8.625 in *(Reference 4)*

A = cross-sectional area per unit length = t = 0.507 in^2/in

S_A = 55 x 8.625 / (2 x 0.507) = 466 psi *Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		Y_s [psi]	
LCS Pipe	**466**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.7

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.00	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	79.3	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.98		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0109	in^4/in	*Calculated*
D_o =	pipe outer diameter =	8.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.507	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	7.611	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.98 x 3000 x 23000 / (17 - 1)^3] =**	**105**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**18**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**46**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**5.8**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**2.3**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **2.3**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	8.09	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.269	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.32		*Calculated*
ε =		**1.6%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.7**	

Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe.

Factor of Safety = 2.7

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PROJECT NUMBER: 16002-009-01 SHEET: 1 OF 4
PROJECT NAME: WEST PASCO COUNTY LANDFILL CELLS A-1 - A-4
SUBJECT: ATTACHMENT J.1-3 - STRUCTURAL PIPE DESIGN
BY: TLEBLANC Date: 3/24/2021
CHECKED BY: C.SPREADBURY Date: 6/25/2021

PART I - 8-INCH SDR 17 HDPE A-4 LEACHATE COLLECTION PIPE STRUCTURAL ANALYSIS

Objective: Evaluate the design of the leachate collection system (LCS) pipe for structural integrity under expected loads. Determine factors of safety against failure due to ring deflection, ring compression, wall buckling, and ring bending strain.

Method:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.
   a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.
   b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).
   c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.
   d. Calculate the deflection ratio and compare to the allowable deflection ratio.
2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.
   a. Calculate the effective compressive pressure for a perforated pipe.
   b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).
   c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.
3. Calculate the factor of safety against constrained pipe wall buckling.
   a. Calculate the constrained pipe wall buckling stress.
   b. Compare the constrained pipe wall buckling stress to the vertical stress (W_c / D_o)
4. Calculate the factor of safety for safe ring bending strain.
   a. Calculate the ring bending strain.
   b. Compare to safe bending strain

Data:

1. Soil and waste parameters

γ = average specific weight of landfilled material (conservative estimate) =	110 pcf		*(Reference 1)*
φ = internal soil friction angle [°] for gravel trench =	35 °		*(Ref. 2, Table 11.3)*
E' = soil reaction modulus =	3,000 psi		*(Ref. 3, Table 3.4)*
(gravel trench crushed rock, moderate compaction)			

2. Pipe dimensions and material properties

D_o = outside diameter of pipe (the same value as B_c)	8.625 in		*(Reference 4)*
	0.719 ft		*Conversion from in to ft*
t = minimum wall thickness	0.507 in		*(Reference 4)*
	0.04 ft		*Conversion from in to ft*
E = elastic modulus of the pipe material (50 year,100°F)	23,000 psi		*(Ref. 5, Table 5-1)*
Pipe perforation diameter =	0.500 in		*(Reference 6)*
Pipe hole spacing between rows (x) =	3 in	(on center)	*(Reference 6)*
Number of holes/row =	2		*(Reference 6)*
8 -inch diameter SDR 17.00 HDPE			*(References 6 and 7)*

3. LCS design cross-section is provided in the Permit Drawings. *(Reference 6)*

4. Landfill geometry - thickness of material (soil and waste).

H_e = distance from top of pipe to plane of equal settlement =	1.2 ft	*(Reference 6)*
(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)		
H = distance from top of pipe to top of landfill at max load =	78.3 ft	*(Part I)*
W_p = Maximum prism load on bottom liner (rounded up for conservatism) =	9,000 psf	*(Part I)*

References:

1 *WTE Ash Data from Southwest FL Landfill, Ardaman & Associates, Inc., 2007.*
2 Soil Mechanics. Lambe and Whitman, 1969.
3 *Buried Pipe Design.* A.P. Moser, 1990, McGraw-Hill.
4 Performance Pipe, Municipal & Industrial Series/IPS Pipe Data, Bulletin: PP 152, 2001.
5 CPCHEM, *The Performance Pipe Engineering Manual*. Book 2, Chapters 5&7 : Buried Pipe Design
6 Pages of Engineering Plans by Others
7 Engineering Report and Application for Permit to Construct Disposal Unit A-2 at Pasco County Resource Recovery Facility Class I Landfill. Law Environmental, Inc. October 1994
8 EPA, *Lining of Waste Impoundment and Disposal Facilities.* SW-870, March 1983 Revised Edition, p. 382

Calculations:

1. Calculate the pipe deflection ratio and compare to the allowable deflection ratio.

a. Calculate the vertical load per unit length of pipe (W_c) using Marston's load equation for a pipe under an embankment.

$$W_c = C_c \gamma B_c^2$$ *(Ref. 3, Eqn 2.5)*

$$C_c = \frac{e^{-2K\mu(H_e/B_c)} - 1}{-2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{-2K\mu(H_e/B_c)}$$ *(Ref. 3, Eqn 2.7)*

where:

W_c =	Marston load, vertical load per unit length of pipe [lb/ft]			
C_c =	Coefficient for Marston Load calculations	59	-	*Calculated*
γ =	average specific weight of landfilled material =	110	pcf	*(Reference 1)*
$B_c = D_o$ =	outside diameter of the pipe	0.719	ft	*(Reference 4)*
e =	natural log base number			
K =	Rankine earth pressure coefficient = $\tan^2(45 - \varphi / 2)$	0.27		*Calculated based on φ*
φ =	internal soil friction angle for gravel trench =	35	°	*(Ref. 2, Table 11.3)*
μ =	friction coefficient between backfill and trench sides = tan φ	0.70		*(tan φ)*
H_e =	distance from top of pipe to plane of equal settlement	1.2	ft	*(Reference 6)*
	(assuming waste will settle as a mass this is the distance from the top of pipe to the top of the sand drainage layer)			
H =	distance from top of pipe to top of embankment	78.3	ft	*(Part I)*
W_c =	**59 x 110 x 0.72^2 =**	**3,364**	**lb/ft =**	**280 lb/in**

b. Calculate the modified arching load per unit length of pipe (W_{mod}) based on Marston's load (W_c) and prism load (W_p).

$$W_{mod} = 0.6\, W_C + 0.4\, W_P = 0.4\,(W_p - W_c) + W_c$$ *(Ref 5, Eqn 7-5)*

where:

W_{mod} =	modified arching vertical soil pressure [lb/ft]			
W_c =	Vertical load per unit length of pipe	3,364	lb/ft	*Calculated*
W_p =	Prism load per unit length of pipe = γ_i x H_i =	9,000	lb/ft	*(Part I)*
W_{mod} =	**0.4 x (9000 - 3364) + 3364) =**	**5,618**	**lb/ft =**	**468 lb/in**
	Actual Estimated Load on Pipe			

c. Calculate the horizontal deflection in the pipe due to the overlying landfill load using modified Iowa formula.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3}$$ *(Ref 3, Eqn 3.5)*

where:

ΔX =	horizontal deflection [in]			
D_L =	deflection lag factor =	1.5		*(Reference 3, Page 52)*
K =	bedding constant =	0.1		*(Reference 3, Page 51)*
W_{mod} =	Modified arching vertical load per unit length of pipe =	468	lb/in	*Calculated*
r =	mean radius of the pipe = (D_o - 2 t)/2 =	3.806	in	*Calculated*
D_o =	outside diameter of pipe =	8.625	in	*(Reference 4)*
t =	minimum wall thickness	0.507	in	*(Reference 4)*
E =	elastic modulus of the pipe material (no tensile stress)	23,000	psi	*(Ref 5, Table 5-1)*
	(50-year lifespan at 100 degrees F)			
I =	moment of inertia of pipe wall per unit length = $t^3/12$ =	0.0109	in^4/in	*Calculated*
E' =	soil reaction modulus (gravel trench crushed rock, moderate compaction) =	3,000	lb/in^2	*(Ref 3, Table 3.4)*
ΔX =	**(1.5x0.1x468x3.81^3) / (23000x0.0109 + 0.061x3000x3.81^3) =**		**0.37**	**in**

d. Calculate the deflection ratio and compare to the allowable deflection ratio.

ΔX / D_o = 4.3%

Conclusion: Safe deflection ratio for non-pressure HDPE pipe = 7.5% *(Ref 5, Page 112)*

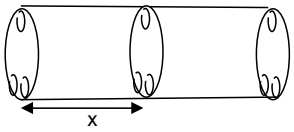
4.3% **<** **7.5%**

Safe deflection ratio includes an inherent factor of safety.
Deflection ratios of 7.5% have been shown to perform reasonably *(Ref 5, pg 112)*
LCS pipe meets allowable deflection limits for maximum applied load.

2. Calculate the actual compressive pressure (also called wall crushing) using effective vertical load due to perforations and compare to the recommended long-term compressive strength design value for Driscoplex pipe.

a. Calculate the effective compressive pressure for a perforated pipe.

$$P_{EFF} = \frac{P_T \times 12}{(12-L_P)}$$



(Reference 8)

where:

P_{EFF} = effective compressive pressure applied to pipe

P_T = vertical load applied to pipe (w/o perf.) = W_{mod}/D_o =	7,817	psf	*(Equals W_{mod} / D_o)*
	54	psi	*Conversion, psf to psi*
L_P = total accumulated length of perforations in one foot of pipe			
Perforation diameter =	0.500	in	*(Reference 6)*
Hole spacing between rows (x) =	3	in (on center)	*(Reference 6)*
Lp =	2	in	*Calculated*
P_{EFF} = (7817 x 12) / (12 - 2) =	**9,380**	**psf**	*Calculated*
	65	**psi**	*Conversion, psf to psi*

This is a very conservative calculation - observations indicate perforations have no effect on the strength of the pipe, and thus should not affect the load calculation.

b. Calculate the actual long-term compressive strength for a perforated pipe (wall crushing).

$$S_A = P_T D_o / 2A$$

(Ref 3, Eqn 3.11)

where:

S_A = actual long-term compressive strength [psi]			
P_T = effective vertical load applied to pipe = P_{EFF} =	65	psi	*Calculated*
D_o = Pipe diameter	8.625	in	*(Reference 4)*
A = cross-sectional area per unit length = t =	0.507	in^2/in	
S_A = 65 x 8.625 / (2 x 0.507) =	**554**	**psi**	*Calculated*

c. Compare the calculated compressive strength to the recommended long-term compressive strength given by the manufacturer.

The recommended, long-term compressive strength (Ys) design value for Driscoplex polyethylene pipe = 800 psi

	S_A [psi]		Y_s [psi]	
LCS Pipe	**554**	**<**	**800**	*(Ref 5, page 103)*

Conclusion: LCS pipe passes ring compressive stress including perforation calculations.

Factor of Safety = (Y_s/S_A) = 1.4

3. Calculate the factor of safety against constrained pipe wall buckling.

a. Calculate the constrained pipe wall buckling stress.

$$P_{WC} = \frac{5.65}{N}\sqrt{RB'E'\frac{E}{12(DR-1)^3}}$$ *(Ref 5, Eqn 7-32)*

where:

P_{WC} =	allowable constrained buckling pressure [lb/in^2]			
N =	safety factor	2		*(Ref 5, page 107)*
R =	buoyancy reduction factor = 1 - 0.33 x (H'/H) =	1.00		*(Ref 5, Eqn 7-34)*
	H' = groundwater height above pipe (ft)	0.28	ft	*Max head on liner (1 - Do)*
	H = cover above pipe (ft)	78.3	ft max	*(Part I)*
B' =	elastic support factor = $(1 + 4*e^{-0.065H})^{-1}$ =	0.98		*(Ref 5, Eqn 7-35)*
E' =	soil reaction modulus for layer around pipe=	3,000	psi	*(Ref 3, Table 3.4)*
E =	pipe elastic modulus (50 year, 100ºF) =	23,000	psi	*(Ref 5, Table 5-1)*
I =	moment of inertia = $t^3/12$ =	0.0109	in^4/in	*Calculated*
D_o =	pipe outer diameter =	8.625	in	*(Reference 4)*
t =	pipe wall thickness =	0.507	in	*(Reference 4)*
DR =	pipe standard dimension ratio = D_o/t =	17		*Calculated*
DI =	pipe inner diameter = D_o - 2t =	7.611	in	*Calculated*
P_{WC} =	**5.65 / 2 x SQRT[1 x 0.98 x 3000 x 23000 / (17 - 1)^3] =**	**104**	**psi**	*Calculated*
W_c / D_o =	Marston vertical pipe pressure =	**33**	psi	*Calculated*
W_{mod}/D_o =	modified arching vertical pressure =	**54**	psi	*Calculated*

b. Compare the constrained pipe wall buckling stress to the vertical stress (Wc / Do)

Factor of Safety for Marston Load = $P_{wc} / (W_c/D_o)$ =	**3.2**
Factor of Safety for Modified Arching Load = $P_{wc} / (W_{mod}/D_o)$ =	**1.9**

Conclusion: Pipe passes constrained wall buckling calculations.

Factor of Safety = $P_{wc}/(W_{mod}/D_o)$ = **1.9**

4. Calculate the factor of safety for safe ring bending strain.

a. Calculate the ring bending strain.

$$\varepsilon = \frac{f_D \times \Delta X \times 2C}{D_m^2} \times 100\%$$ *(Ref 5, Eqn 7-39)*

where:

ε =	ring bending strain (%)			
f_D =	deformation shape factor (non-elliptical shape) =	6		*(Ref 5, page 112)*
D_m =	pipe mean diameter = D_o - 1.06t =	8.09	in	*(Ref 5, Eqn 7-27)*
C =	outer fiber wall centroid = 0.5 (1.06t) =	0.269	in	*(Ref 5, Eqn 7-41)*
ΔX =	ring deflection =	0.37		*Calculated*
ε =		**1.8%**		*Calculated*

b. Compare to safe bending strain

	Safe bending strain =	4.2%	*(Ref 5, page 112)*
	Factor of safety = safe bending strain / ring bending strain =	**2.3**	

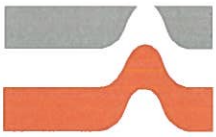
Conclusion: The pipe passes the ring bending strain for high performance polyethylene non-pressure pipe.

Factor of Safety = 2.3

Reference 1

Excerpts from Ardaman & Associates, Inc. Report (2007)

SUBSURFACE SOIL EXPLORATION
LEE/HENDRY LANDFILL
HENDRY COUNTY, FLORIDA



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Excerpts from Ardaman & Associates, Inc. Report (2007)

LABORATORY ANALYSIS

One-dimensional consolidation tests were performed on plastic clay specimens obtained from the Shelby tube samples retrieved from the test borings. The tests were performed in accordance with ASTM D 2435. This test method subjects a soil specimen to increasing loads and incrementally monitoring the change in height. The data is presented on an e-log p curve, which plots void ratio (e) as a function of the log of time pressure (p). The parameters necessary for settlement calculation can then be derived from these curves: compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c) and initial void ratio (e_o). A separate plot of the change in sample height versus log time for each load increment allows calculation of the coefficient of consolidation (c_v) and the coefficient of secondary compression (c_α) so that the ratio of primary settlement and amount of secondary compression may be predetermined. A summary of the data is presented below and on the e-log p curve in the Appendix.

Dry Unit Weight and Moisture Content Used to Calculate "Wet Unit Density"

SPT Boring Location	Sample No.	Depth (ft.)	Dry Unit Weight, (pcf)	Moisture Content, %	Percent Fines	Liquid Limit	Plasticity Index	USCS Classification	Remarks
Ash	U-1	24 - 26	93.0	27.4	-	-	-	SC	-
Ash	U-2	34 - 36	51.0	85.4	-	-	-	CL	-
Ash	U-3	40 - 42	88.8	32.4	35.9	-	-	SC	-
C&D	U-1	19 - 21	102.8	23.4	49.4	-	-	SC	-
C&D	U-2	30 - 32	53.4	79.8	91.5	168	124	CH	Consol
C&D	U-3	45 - 47	46.7	95.6	99.1	173	122	CH	Consol

Boring Location	
SPT C&	
SPT C&	

2007 Ardaman & Associates, Inc. Test Results

Sample	Depth (feet)	Dry Unit Weight (pcf)	MC (%)	Wet Unit Density (pcf)
Ash-1	24-26	93	27.4	118
Ash-2	34-36	51	85.4	94.6
Ash-3	40-42	88.8	32.4	118
Avg.		77.6	48.4	110

Ash sp

Undrained (CU) triaxial shear strength tests. The tests were performed in accordance with ASTM D 4767. In this test the specimen is allowed to consolidate under the confining pressure prior to shear, but no drainage is permitted during shear. A minimum of three tests at different confining pressures is required to derive the stress parameters. Pore pressure measurements are taken during testing so that the effective stress parameters can be derived. The results of the CU tests as derived from the p'-q plots are summarized below.

Excerpts from Ardaman & Associates, Inc. Report (2007)

Sample	Sample No.	Depth (ft.)	Friction Angle, Ø (deg)	Cohesion, c (tsf)
Clay	SPT C&D U-2	30 - 32	21.0	0.64
Clay	SPT C&D U-3	45 - 47	20.3	0.52
ASH	Bulk	-	44.3	0

GENERAL COMMENTS

The analysis in this report are based on the data obtained from the twenty-two (22) boring locations as directed by Jones Edmunds and Associates, Inc. personnel.

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and may be encountered. The boring logs and related information are based on the driller's logs and visual examination of selected sample in the laboratory.

The delineation between soil types shown on the logs is approximate and the description represents our interpretation of subsurface conditions at the designated boring locations and on the particular date drilled.

If you have any questions about this report, please contact this office.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.

Martin A. Call, P.E.
Senior Project Engineer

Gary A. Drew, P.E.
Branch Manager/Vice President

Reference 2

Soil Mechanics

T. William Lambe • *Robert V. Whitman*

Massachusetts Institute of Technology

1969

JOHN WILEY & SONS, New York • Chichester • Brisbane • Toronto • Singapore

Table 11.3 Summary of Friction Angle Data for Use in Preliminary Design

	Friction Angles							
	Slope Angle of Repose		At Ultimate Strength		At Peak Strength			
					Medium Dense		Dense	
Classification	i(°)	Slope (vert. to hor.)	ϕ_{cv}(°)	tan ϕ_{cv}	ϕ(°)	tan ϕ	ϕ(°)	tan ϕ
Silt (nonplastic)	26 to 30	1 on 2 to 1 on 1.75	26 to 30	0.488 to 0.577	28 to 32	0.532 to 0.625	30 to 34	0.577 to 0.675
Uniform fine to medium sand	26 to 30	1 on 2 to 1 on 1.75	26 to 30	0.488 to 0.577	30 to 34	0.577 to 0.675	32 to 36	0.675 to 0.726
Well-graded sand	30 to 34	1 on 1.75 to 1 on 1.50	30 to 34	0.577 to 0.675	34 to 40	0.675 to 0.839	38 to 46	0.839 to 1.030
Sand and gravel	32 to 36	1 on 1.60 to 1 on 1.40	32 to 36	0.625 to 0.726	36 to 42	0.726 to 0.900	40 to 48	0.900 to 1.110

From B. K. Hough, *Basic Soils Engineering*. Copyright © 1957, The Ronald Press Company, New York.
Note. Within each range, assign lower values if particles are well rounded or if there is significant soft shale or mica content, higher values for hard, angular particles. Use lower values for high normal pressures than for moderate normal pressure.

problems involving man-made fills, it is difficult to either measure or estimate the friction angle of a sand on the basis of laboratory tests alone. For these reasons, extensive use is made in practice of correlations between the friction angle of a sand and the resistance of the natural sand deposit to penetration.

Figure 11.14 shows an empirical correlation between the resistance offered to the standard penetration spoon (Chapter 7) and the friction angle. Inevitably, any such correlation is crude. The actual friction angle may deviate by $\pm 3°$ or more from the value given by the curve. The given relation is intended to apply for depths of overburden up to 40 ft, and is conservative for greater depths.

11.6 SUMMARY OF MAIN POINTS

1. The strength of soil can be represented by a Mohr envelope, which is a plot of τ_{ff} versus σ_{ff}. Generally the Mohr envelope of a granular soil is curved. For stresses less than 100 psi, the envelope usually is almost straight so that
$$\tau_{ff} = \sigma_{ff} \tan \phi$$
where ϕ is the friction angle corresponding to the peak point of the stress-strain curve.
2. The value of ϕ for any soil depends upon ϕ_μ and upon the amount of interlocking; i.e., the initial void ratio and σ_{ff}.
3. Where sand is being subjected to very large strains, ϕ_{cv} should be used in the failure law. Unless the sand is very loose, ϕ_{cv} will be less than ϕ. Where the sand is sliding over the surface of a structure, the friction angle will vary from ϕ_μ to ϕ_{cv}, depending on the smoothness of the surface.
4. A knowledge of the effect of composition helps guide the selection of materials to be used in man-made fills
5. Materials to be used in man-made fills should be tested using the actual range of confining pressures which will be encountered in the fill.
6. For many practical problems, the friction angle of an *in situ* sand deposit can be determined by indirect means, such as the standard penetration test.

PROBLEMS

11.1 Given the following triaxial test data, plot the results (*a*) in a Mohr diagram and (*b*) in a p–q diagram, and determine ϕ by each method.

Reference 3

11.1
M

Buried Pipe Design

A. P. Moser, Ph.D.
Mechanical Engineering
Utah State University
Logan, Utah

McGraw-Hill
New York St. Louis San Francisco Auckland Bogotá
Caracas Hamburg Lisbon London Madrid Mexico
Milan Montreal New Delhi Paris
San Juan São Paulo Singapore
Sydney Tokyo Toronto

Reference 1 - Pipe Structural Design - Reference 1

Embankment conditions. Not all pipes are installed in ditches (trenches) and it is necessary to treat the problem of pipes buried in embankments. An embankment is where the top of the pipe is above the natural ground. Marston defined this type of installation as a positive projecting conduit. Typical examples are railway and highway culverts. Figure 2.3 shows two cases of positive projecting conduits as proposed by Marston. In case I, the ground at the sides of the pipe settles more than the top of the pipe. In case II, the top of the pipe settles more than the soil at the sides of the pipe. Case I was called the projection condition by Marston and is characterized by a positive settlement ratio, r_{sd}, as defined in Fig. 2.4. The shear forces are downward and cause a greater load on the buried pipe for this case. Case II is called the ditch condition and is characterized by a negative settlement ratio, r_{sd}. The shear forces are directed upward in this case and result in a reduced load on the pipe.

In conjunction with positive projecting conduits, Marston determined the existence of a horizontal plane above the pipe, where the shearing forces are zero. This plane is called the plane of equal settlement. Above this plane, the interior and exterior prisms of soil settle equally. The condition where the plane of equal settlement is real (it is located within the embankment) is called an incomplete projection or an incomplete ditch condition. If the plane of equal settlement is imaginary (the shear forces extend all the way to the top of the embankment), it is called a complete ditch or complete projection condition.

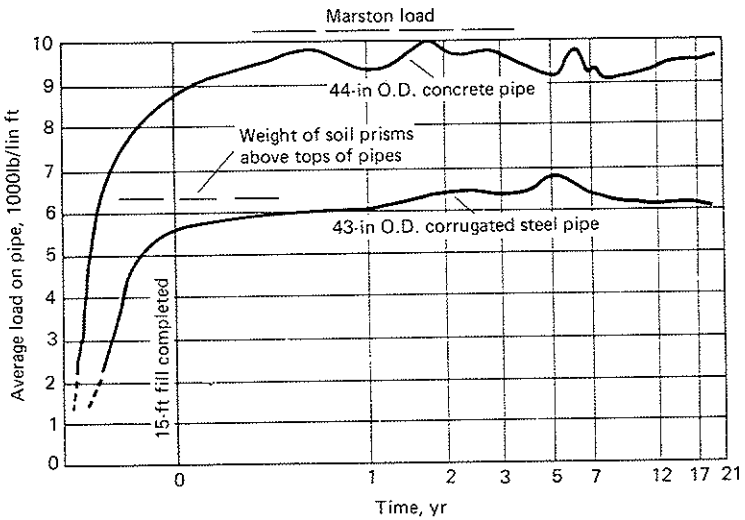

Figure 2.3 Measured loads on rigid and flexible pipe over a period of 21 years. *(Reprinted from Spangler and Handy, Soil Engineering, Harper & Row, 1973, by permission of the publisher.)*

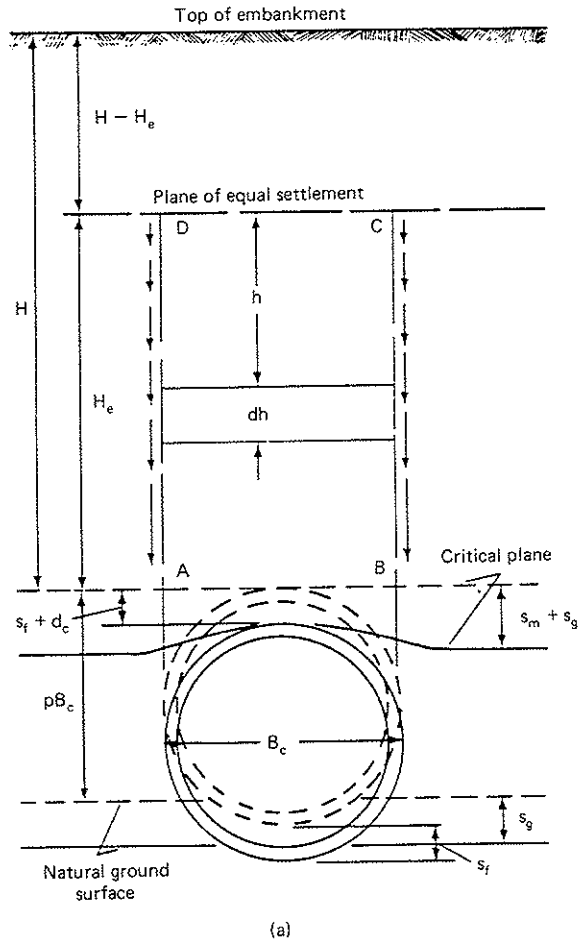

Figure 2.4 Comparison of positive projecting conduits: (*a*) Projection condition; (*b*) ditch condition. (r_{sc} = $[(S_m + S_g) - (S_f + d_c)]/S_m$; r_{sd} = settlement ratio; s_m = compression of soil at sides of pipe; s_g = settlement of natural ground surface at sides of pipe; s_f = settlement of foundation underneath pipe; d_c = deflection of the top of pipe.) *(Reprinted from Spangler and Handy, Soil Engineering, Harper & Row, 1973, by permission of the publisher.)*

All of the above discussed parameters affect the load on the pipe and are incorporated in Marston's load equation for positive projecting (embankment) conduits.

$$W_c = C_c \gamma B_c^2 \qquad (2.5)$$

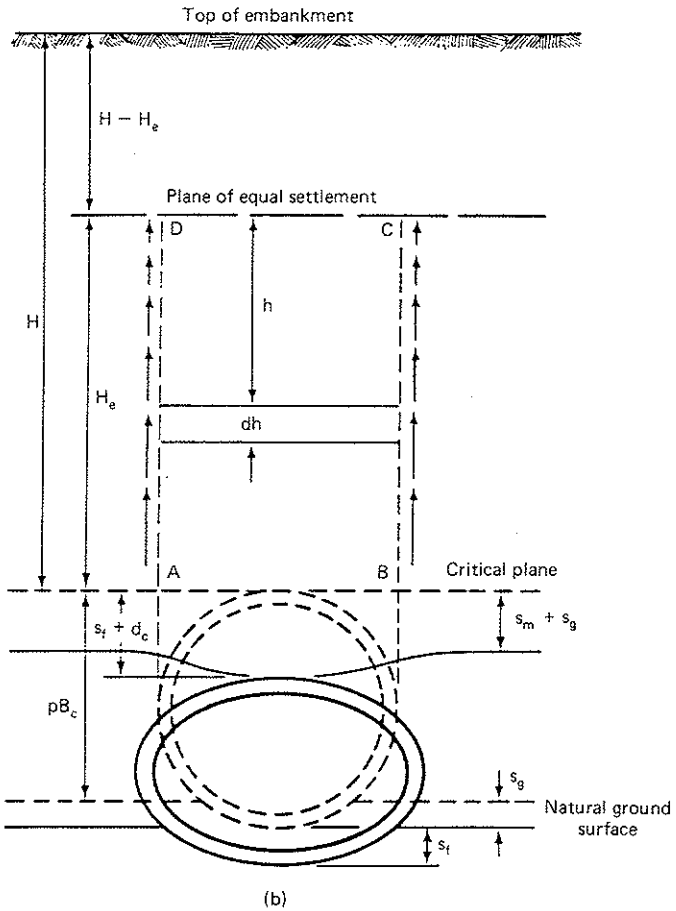

Figure 2.4 (*Continued*).

where

$$C_c = \frac{e^{\pm 2K\mu(H/B_c)} - 1}{\pm 2K\mu} \tag{2.6}$$

or

$$C_c = \frac{e^{\pm 2K\mu(H_e/B_c)} - 1}{\pm 2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2K\mu(H_e/B_c)} \tag{2.7}$$

Equation (2.6) is for the complete condition. The negative signs are for the complete ditch and the positive signs are for the complete projection condition.

Equation (2.7) is for the incomplete condition, where the negative signs are for the incomplete ditch and the positive signs are for the incomplete projection condition. H_e is the height of the plane of equal settlement. Note that if $H_e = H$, the incomplete case [Eq. (2.7)] becomes the complete case and Eq. (2.6) applies for C_c.

Although the above equations are difficult and cumbersome, they have been simplified and can be found in graphical form in many references.

Note that value C_c is a function of the ratio of height of cover to pipe-diameter ratio, the product of the settlement ratio and projection ratio, Rankine's constant and the coefficient of friction.

$$C_c = f\left(\frac{H}{B_c}, r_{sd}p, K, \mu\right)$$

The value of the product $K\mu$ is generally taken as 0.19 for the projection condition and 0.13 for the ditch condition. Figure 2.5 is a typ-

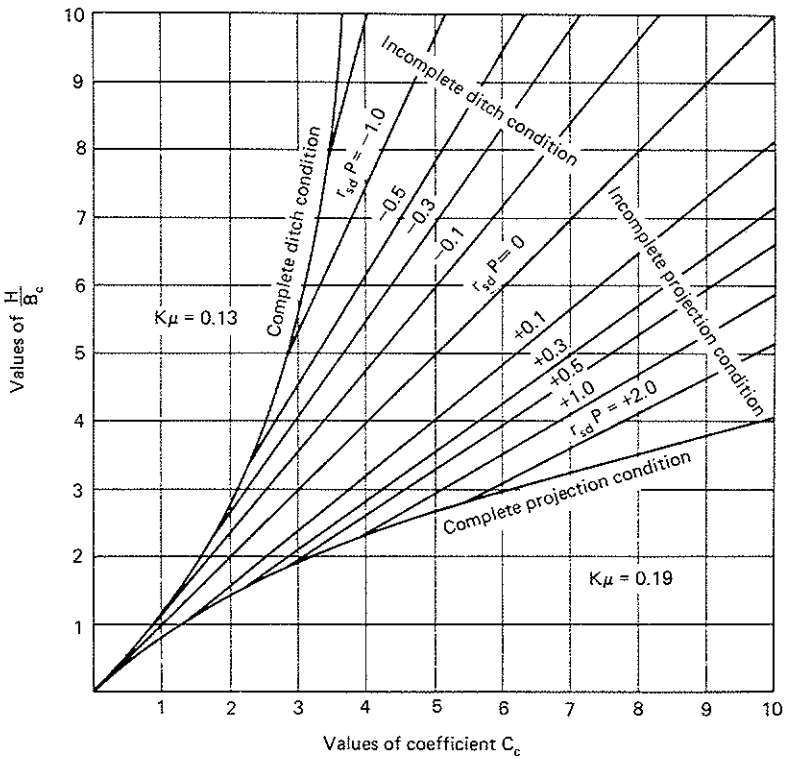

Figure 2.5 Diagram for coefficient C_c for positive projecting conduits. (*Reprinted from Spangler and Handy, Soil Engineering, Harper & Row, 1973,*[11] *by permission of the publisher.*)

Attachment G.3.aa(2)-2 - Pipe Structural Design - Reference 1

At the transition width, $W_c \approx W_d$

$$W_{3\text{-edge}} = 3131 \frac{\text{SF}}{\text{LF}}$$

$$W_{D\text{-load}} = 2505 \frac{\text{SF}}{\text{LF}}$$

Minimum Required Strength for SF = 1.5

Bedding class	Three-edge (lb/ft)	*D*-load [lb/(ft)/(ft)]
B	2472	1977
C	3131	2505
D	4270	3416

Flexible Pipe Analysis

Installation design

Three parameters are most essential in the design or the analysis of any flexible conduit installation. They are

1. Load (depth of burial)
2. Soil stiffness in pipe zone
3. Pipe stiffness

The design load on the pipe is easily calculated using the prism-load theory as discussed in Chap. 2. This load is simply the product of the soil unit weight and the height of cover. Research has shown that the long-term load on a flexible pipe approaches the prism load. Thus, if this load is used in design, the deflection lag factor should be taken as unity.

The soil stiffness is usually expressed in terms of E' (effective soil modulus, lb/in^2). E′ is dimensionally load per square inch and is a soil modulus term. The soil modulus (E′) is a function of soil properties such as soil density, soil type, and moisture content. Experience has shown that soil density is the most important parameter influencing soil stiffness.

For flexible pipes, pipe stiffness rather than crush strength is usually the controlling pipe material property. Pipe stiffness may be expressed in terms of various parameters as follows:

Pipe Stiffness Terminology

Stiffness factor = EI

Ring stiffness = EI/r^3 (or sometimes EI/D^3)

Pipe stiffness = $F/\Delta y = 6.7\ EI/r^3$

where E = modulus of elasticity, lb/in^2
I = moment of inertia of the wall cross-section per unit length of pipe, in^4/in = in^3
r = mean radius of pipe, in
D = mean diameter of pipe, in
F = force, lb/in
Δy = vertical deflection, in

The most commonly used terminology is pipe stiffness ($F/\Delta y$). For a given pipe product, this term is readily determined in the laboratory by a parallel plate loading test. In this test, a pipe sample is placed between two horizontal parallel plates in a testing machine. A compressive load is applied and increased until the vertical deflection (Δy) reaches 5 percent of the diameter. $F/\Delta y$ is the load at 5 percent divided by the sample length and divided by the vertical deflection Δy. Typical units for $F/\Delta y$ are lb/in^2. This is evident from the third equation in the above table as it is clear that $F/\Delta y$ has the same units as the modulus of elasticity (E).

In summary, the three most important parameters for flexible pipe analysis and design are (1) load, (2) soil stiffness, and (3) pipe stiffness. Any design method that does not include a consideration of these three parameters is incomplete.

For a flexible pipe, vertical deflection is the variable that must be controlled by proper installation design. This deflection is a function of the three parameters discussed above.

Spangler's Iowa formula

M. G. Spangler, a student of Anson Marston, observed that the Marston theory for calculating loads on buried pipe was not adequate for flexible pipe design. Spangler noted that flexible pipes provide little inherent stiffness in comparison to rigid pipes, yet they perform remarkably well when buried in soil. This significant ability of a flexible pipe to support vertical soil loads is derived from (1) the redistribution of loads around the pipe, and (2) the passive pressures induced as the sides of the pipe move outward against the surrounding earth. These considerations, coupled with the idea that the ring deflection may form a basis for flexible pipe design, prompted M. G. Spangler to study flexible pipe behavior to determine an adequate design procedure. His research and testing led to the derivation of the Iowa formula which he published in 1941.[22]

Spangler incorporated the effects of the surrounding soil on the pipe's deflection. This was accomplished by assuming that Marston's theory of loads applied, and that this load would be uniformly distrib-

Attachment G.3.(2)-2 - Pipe Structural Design - Reference 1

Attachment G.3.a(2)-2 - Pipe Structural Design - Reference 1

uted at the plane at the top of the pipe. He also assumed a uniform pressure over part of the bottom, depending upon the bedding angle. On the sides, he assumed the horizontal pressure h on each side would be proportional to the deflection of the pipe into the soil. The constant of proportionality was defined as shown in Fig. 3.5 and was called the modulus of passive resistance of the soil. The modulus would presumably be a constant for a given soil and could be measured in a simple lab test. Through analysis he derived the Iowa formula as follows:

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 e r^4} \tag{3.4}$$

where D_L = deflection lag factor
K = bedding constant
W_c = Marston's load per unit length of pipe, lb/in
r = mean radius of the pipe, in
E = modulus of elasticity of the pipe material, lb/in^2
I = moment of inertia of the pipe wall per unit length, in^4/in = in^3
e = modulus of passive resistance of the side fill, lb/(in^2)(in)
ΔX = horizontal deflection or change in diameter, in

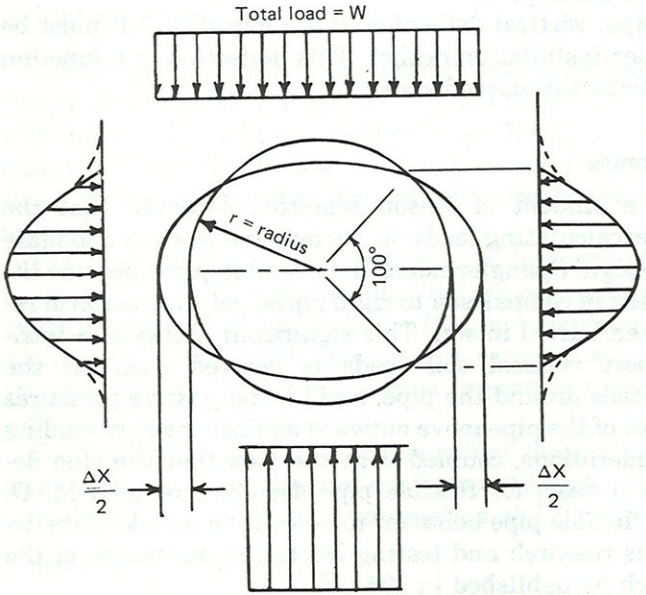

Figure 3.5 Basis of Spangler's derivation of the Iowa formula for deflection of buried pipes. [$\Delta X = D_L K W_c r^3/(EI + 0.061 e r^4)$ (the Iowa formula); where $e = 2h/\Delta X$; $2r = D$ = pipe diameter; K = Bedding constant; D_L = Deflection lag factor; and EI = Stiffness factor (related to pipe stiffness).] *(Reprinted with permission of Utah State University.)*

Equation (3.4) can be used to predict deflections of buried pipe if the three empirical constants K, D_L, and e are known. The bedding constant, K, accommodates the response of the buried flexible pipe to the opposite and equal reaction to the load force derived from the bedding under the pipe. The bedding constant varies with the width and angle of the bedding achieved in the installation. The bedding angle is shown in Fig. 3.6. Table 3.3 contains a list of bedding factors, K, dependent upon the bedding angle. These were determined theoretically by Spangler and published in 1941. As a general rule, a value of $K = 0.1$ is assumed.

In 1958, Reynold K. Watkins, a graduate student of Spangler, was investigating the modulus of passive resistance through model studies and examined the Iowa formula dimensionally.[31] The analysis determined that e could not possibly be a true property of the soil in that its dimensions are not those of a true modulus. As a result of Watkins' effort, another soil parameter was defined. This was the modulus of soil reaction, $E' = er$. Consequently, a new formula called the modified Iowa formula was proposed.

$$\Delta X = \frac{D_L K W_c r^3}{EI + 0.061 E' r^3} \tag{3.5}$$

Two other observations from Watkins' work are of particular note. (1) There is little point in evaluating E' by a model test and then using this modulus to predict ring deflection as the model gives ring deflection directly. (2) Ring deflection may not be the only performance limit.

Another parameter in the Iowa formula, needed to calculate deflections, is the deflection lag factor D_L. Spangler recognized that in soil-pipe systems, as with all engineering systems involving soil, the soil consolidation at the sides of the pipe continues with time after installation of the pipe. His experience had shown that deflections could increase by as much as 30 percent over a period of 40 years. For this reason, he recommended the incorporation of a deflection lag factor of

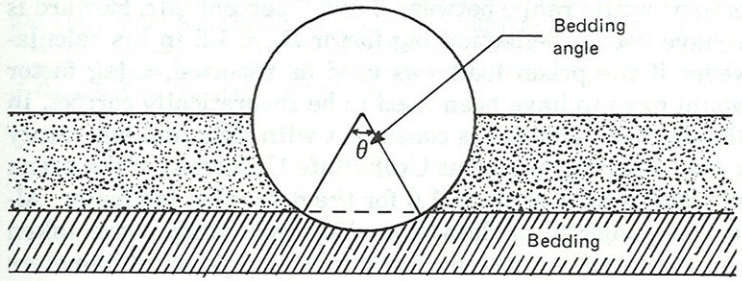

Figure 3.6 Bedding angle.

TABLE 3.3 Values of Bedding Constant, *K*

Bedding angle, degrees	*K*
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

1.5 as a conservative design procedure. However, recall that the load proposed by Spangler was the Marston load for a flexible pipe. For most sewer pipe installations the prism load is at least 1.5 times greater than the Marston load (see Chap. 2 for soil loads on pipe). If the prism load is used for design, a design deflection lag factor $D_L = 1.0$ should be used.

The remaining parameter in the modified Iowa formula is the soil modulus E'. Many research efforts have attempted to measure E' without success. The most useful method has involved the measurement of deflections of a buried pipe for which installation conditions are known, followed by a back calculation through the Iowa formula to determine the effective value of E'. This requires assumed values for the load, the bedding factor, and the deflection lag factor. Inconsistent assumptions have led to a variation in reported values of E'.

One attempt to acquire information on values of E' was conducted by Amster K. Howard of the United States Bureau of Reclamation.[5] Howard used data from laboratory and field tests to compile a table of average E' values for various soil types and densities (see Table 3.4). He assigned values to E', K, and W_c and then used the Iowa formula to calculate a theoretical value of deflection. This theoretical deflection was then compared with actual measurements. By assuming the E' values of Table 3.4 and a bedding constant $K = 0.1$, Howard was able to correlate the theoretical and empirical results to within ± 2 percent deflection when he used the prism soil load. This means that if theoretical deflections using Table 3.4 were approximately 5 percent, measured deflections would range between 3 and 7 percent. Mr. Howard is reported to have used a deflection lag factor $D_L = 1.5$ in his calculations. However if the prism load was used as reported, a lag factor $D_L = 1.0$ would have to have been used to be theoretically correct. In any case, the data in Table 3.4 is consistent with field and laboratory data taken over a 20-year period at Utah State University if the prism load is used along with a value of 1.0 for the deflection lag factor. Although the vast majority of data from Howard's study were taken from tests on steel and reinforced plastic mortar pipe with diameters greater than 24 in, it does provide some useful information to guide

TABLE 3.4 Average Values of Modulus of Soil Reaction, *E'* (For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System*)	*E'* for degree of compaction of bedding, lb/in²			
	Dumped	Slight, < 85% proctor, < 40% relative density	Moderate, 85%–95% proctor, 40%–70% relative density	High, >95% proctor, > 70% relative density
Fine-grained soils (LL > 50)† Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent soils engineer; Otherwise use $E' = 0$			
Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1000
Fine-grained soils (LL < 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained soils with fines GM, GC, SM, SC contains more than 12% fines	100	400	1000	2000
Coarse-grained soils with little or no fines GW, GP, SW, SP‡ contains less than 12% fines	200	1000	2000	3000
Crushed rock	1000	3000	3000	3000
Accuracy in terms of percentage deflection§	± 2	± 2	± 1	± 0.5

(2000+3000)/2 = 2500 lb/in2

*ASTM Designation D2487, USBR Designation E-3
†LL = liquid Limit
‡Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC)
§For ± 1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.

NOTE: Values applicable only for fills less than 50 ft (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate deflection lag factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/ft³ (598,000 J/m¹) (ASTM D698, AASHO T-99, USBR Designation E-11). 1 lb/in² = 6.9 kN/m².

SOURCE: Amster K. Howard, "Soil Reaction for Buried Flexible Pipe," U.S. Bureau of Reclamation, Denver, Colo. Reprinted with Permission from American Society of Civil Engineers *J. Geotech. Eng. Div.*, January 1977, pp. 33–43.

Attachment G.3.a(2)-2 - Pipe Structural Design - Ref 1

Attachment G.3.a(2)-2 - Pipe Structural Design - Reference 1

plied that stress, strain, deflection, and buckling are independent, but only convenient parameters on which to focus ones attention. For a particular product, certain performance limits are not considered because others will always occur first. The following is a list of performance limits which are often considered in design and could be thought of as possible responses to soil pressure:

Wall crushing (stress)
Wall buckling
Reversal of curvature (deflection)
Overdeflection
Strain limit
Longitudinal stresses
Shear loadings
Fatigue
Delamination

Wall crushing. Wall crushing is the terminology used to describe the condition of localized yielding for a ductile material or cracking failure for brittle materials. This performance limit is reached when the in-wall stress reaches the yield stress or the ultimate stress of the pipe material. The ring-compression stress is the primary contributor to this performance limit. (See Fig. 3.14.)

$$\text{Ring compression} = \frac{P_v D}{(2A)} \tag{3.11}$$

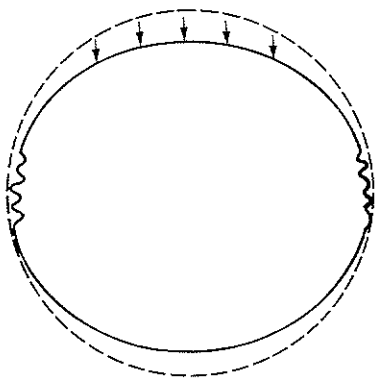

Figure 3.14 Wall crushing at the 3 and 9 o'clock positions.

where P_v = vertical soil pressure
D = diameter
A = cross-sectional area per unit length

However, wall crushing can also be influenced by the bending stress.

$$\text{Bending stress} = \frac{Mt/2}{I} \tag{3.12}$$

M = bending moment per unit length
t = wall thickness
I = moment of inertia of wall cross section per unit length

Wall crushing is the primary performance limit or design basis for most "rigid" or brittle pipe products. This performance limit may also be reached for stiffer flexible pipes installed in highly compacted backfill and subjected to very deep cover. A quick check for this performance limit can be made by comparing the ring compression stress with yield and/or ultimate strengths.

Wall buckling. Buckling is not a strength performance limit, but can occur because of insufficient stiffness. The buckling phenomenon may govern design of flexible pipes subjected to internal vacuum, external hydrostatic pressure, or high soil pressures in compacted soil. (See Fig. 3.15.)

The more flexible the conduit, the more unstable the wall structure will be in resisting buckling. For a circular ring in plane stress, subjected to a uniform external pressure, the critical buckling pressure is

$$P_{cr} = \frac{3EI}{R^3} \tag{3.13}$$

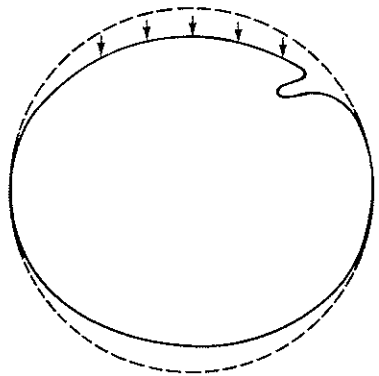

Figure 3.15 Localized wall buckling.

Reference 4

PERFORMANCE PIPE
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PERFORMANCE PIPE Municipal & Industrial Series/IPS Pipe Data

Pipe weights are calculated in accordance with PPI TR-7. Average inside diameter calculated using nominal OD and minimum wall plus 6% for use in estimating fluid flows. Actual ID will vary. When designing components to fit the pipe ID, refer to pipe dimensions and tolerances in applicable pipe specifications.

Pressure Ratings are for water at 73.4 °F. For other fluid and service temperature, ratings may differ. Refer to Engineering Manual for Chemical and Environmental Considerations.

Pressure Rating		255 psi DR 7.3			200 psi DR 9.0			160 psi DR 11.0			130 psi DR 13.5			
IPS Pipe Size	Nominal OD (in)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	IPS Pipe Size
1 1/4"	1.660	0.227	1.179	0.44	0.184	1.270	0.37	0.151	1.340	0.31	0.123	1.399	0.26	1 1/4"
1 1/2"	1.900	0.260	1.349	0.58	0.211	1.453	0.49	0.173	1.533	0.41	0.141	1.601	0.34	1 1/2"
2"	2.375	0.325	1.686	0.91	0.264	1.815	0.76	0.216	1.917	0.64	0.176	2.002	0.53	2"
3"	3.500	0.479	2.485	1.98	0.389	2.675	1.66	0.318	2.826	1.39	0.259	2.951	1.15	3"
4"	4.500	0.616	3.194	3.27	0.500	3.440	2.74	0.409	3.633	2.29	0.333	3.794	1.90	4"
5 3/8"	5.375	0.736	3.815	4.66	0.597	4.109	3.90	0.489	4.338	3.27	0.398	4.531	2.72	5 3/8"
5"	5.563	0.762	3.948	5.00	0.618	4.253	4.18	0.506	4.490	3.51	0.412	4.690	2.91	5"
6"	6.625	0.908	4.700	7.09	0.736	5.065	5.93	0.602	5.349	4.97	0.491	5.584	4.13	6"
7 1/8"	7.125	0.976	5.056	8.20	0.792	5.446	6.86	0.648	5.751	5.75	0.528	6.006	4.78	7 1/8"
8"	8.625	1.182	6.119	12.01	0.958	6.594	10.05	0.784	6.963	8.42	0.639	7.270	7.00	8"
10"	10.750	1.473	7.627	18.66	1.194	8.219	15.61	0.977	8.679	13.09	0.796	9.062	10.87	10"
12"	12.750	1.747	9.046	26.25	1.417	9.746	21.97	1.159	10.293	18.41	0.944	10.749	15.29	12"
13 3/8"	13.375	1.832	9.491	28.88	1.486	10.225	24.18	1.216	10.797	20.26	0.991	11.274	16.84	13 3/8"
14"	14.000	1.918	9.934	31.64	1.556	10.701	26.50	1.273	11.301	22.20	1.037	11.802	18.44	14"
16"	16.000	2.192	11.353	41.33	1.778	12.231	34.60	1.455	12.915	29.00	1.185	13.488	24.09	16"
18"	18.000	2.466	12.772	52.31	2.000	13.760	43.79	1.636	14.532	36.69	1.333	15.174	30.48	18"
20"	20.000	2.740	14.191	64.58	2.222	15.289	54.05	1.818	16.146	45.30	1.481	16.860	37.63	20"
22"	22.000	3.014	15.610	78.14	2.444	16.819	65.40	2.000	17.760	54.82	1.630	18.544	45.56	22"
24"	24.000	3.288	17.029	93.00	2.667	18.346	77.85	2.182	19.374	65.24	1.778	20.231	54.21	24"
26"	26.000				2.889	19.875	91.36	2.364	20.988	76.57	1.926	21.917	63.62	26"
28"	28.000				3.111	21.405	105.95	2.545	22.605	88.78	2.074	23.603	73.78	28"
30"	30.000				3.333	22.934	121.62	2.727	24.219	101.92	2.222	25.289	84.69	30"
32"	32.000							2.909	25.833	115.97	2.370	26.976	96.35	32"
34"	34.000							3.091	27.447	130.93	2.519	28.660	108.81	34"
36"	36.000							3.273	29.061	146.80	2.667	30.346	121.98	36"
42"	42.000										3.111	35.405	166.00	42"
48"	48.000													48"
54"	54.000													54"

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Pressure Ratings are for water at 73.4 °F. For other fluid and service temperature, ratings may differ. Refer to Engineering Manual for Chemical and Environmental Considerations.

Pressure Rating		100 psi DR 17.0			80 psi DR 21.0			65 psi DR 26.0			50 psi DR 32.5			
IPS Pipe Size	Nominal OD (in)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	Minimum Wall (in)	Average ID (in)	Weight (lbs/ft)	IPS Pipe Size
1 1/4"	1.660													1 1/4"
1 1/2"	1.900													1 1/2"
2"	2.375	0.140	2.078	0.43										2"
3"	3.500	0.206	3.063	0.93										3"
4"	4.500	0.265	3.938	1.54	0.214	4.046	1.26							4"
5 3/8"	5.375	0.316	4.705	2.20	0.256	4.832	1.80	0.207	4.936	1.47				5 3/8"
5"	5.563	0.327	4.870	2.35	0.265	5.001	1.93	0.214	5.109	1.57				5"
6"	6.625	0.390	5.798	3.34	0.315	5.957	2.73	0.255	6.084	2.23	0.204	6.193	1.80	6"
7 1/8"	7.125	0.419	6.237	3.86	0.339	6.406	3.16	0.274	6.544	2.58	0.219	6.661	2.08	7 1/8"
8"	8.625	0.507	7.550	5.65	0.411	7.754	4.64	0.332	7.921	3.79	0.265	8.063	3.05	8"
10"	10.750	0.632	9.410	8.78	0.512	9.665	7.21	0.413	9.874	5.87	0.331	10.048	4.75	10"
12"	12.750	0.750	11.160	12.36	0.607	11.463	10.13	0.490	11.711	8.26	0.392	11.919	6.67	12"
13 3/8"	13.375	0.787	11.707	13.61	0.637	12.025	11.15	0.514	12.285	9.09	0.412	12.502	7.35	13 3/8"
14"	14.000	0.824	12.253	14.91	0.667	12.586	12.22	0.538	12.859	9.96	0.431	13.086	8.05	14"
16"	16.000	0.941	14.005	19.46	0.762	14.385	15.96	0.615	14.696	13.01	0.492	14.957	10.50	16"
18"	18.000	1.059	15.755	24.64	0.857	16.183	20.19	0.692	16.533	16.47	0.554	16.826	13.30	18"
20"	20.000	1.176	17.507	30.41	0.952	17.982	24.93	0.769	18.370	20.34	0.615	18.696	16.41	20"
22"	22.000	1.294	19.257	36.80	1.048	19.778	30.18	0.846	20.206	24.61	0.677	20.565	19.86	22"
24"	24.000	1.412	21.007	43.81	1.143	21.577	35.91	0.923	22.043	29.30	0.738	22.435	23.62	24"
26"	26.000	1.529	22.759	51.39	1.238	23.375	42.14	1.000	23.880	34.39	0.800	24.304	27.74	26"
28"	28.000	1.647	24.508	59.62	1.333	25.174	48.86	1.077	25.717	39.88	0.862	26.173	32.19	28"
30"	30.000	1.765	26.258	68.45	1.429	26.971	56.12	1.154	27.554	45.78	0.923	28.043	36.93	30"
32"	32.000	1.882	28.010	77.86	1.524	28.769	63.84	1.231	29.390	52.10	0.985	29.912	42.04	32"
34"	34.000	2.000	29.760	87.91	1.619	30.568	72.06	1.308	31.227	58.81	1.046	31.782	47.43	34"
36"	36.000	2.118	31.510	98.57	1.714	32.366	80.78	1.385	33.064	65.94	1.108	33.651	53.20	36"
42"	42.000	2.471	36.761	134.16	2.000	37.760	109.97	1.615	38.576	89.71	1.292	39.261	72.37	42"
48"	48.000	2.824	42.013	175.23	2.286	43.154	143.65	1.846	44.086	117.18	1.477	44.869	94.56	48"
54"	54.000				2.571	48.549	181.75	2.077	49.597	148.33	1.662	50.477	119.70	54"

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Reference 5

Table 5-1 Typical Elastic Modulus for DriscoPlex® PE 3408

Load Duration	*Elastic Modulus†, 1000 psi (MPa), at Temperature, °F (°C)*							
	-20 (-29)	*0 (-18)*	*40 (4)*	*60 (16)*	*73 (23)*	*100 (38)*	*120 (49)*	*140 (60)*
Short-Term	300.0 (2069)	260.0 (1793)	170.0 (1172)	130.0 (896)	110.0 (758)	100.0 (690)	65.0 (448)	50.0 (345)
10 h	140.8 (971)	122.0 (841)	79.8 (550)	61.0 (421)	57.5 (396)	46.9 (323)	30.5 (210)	23.5 (162)
100 h	125.4 (865)	108.7 (749)	71.0 (490)	54.3 (374)	51.2 (353)	41.8 (288)	27.2 (188)	20.9 (144)
1000 h	107.0 (738)	92.8 (640)	60.7 (419)	46.4 (320)	43.7 (301)	35.7 (246)	23.2 (160)	17.8 (123)
1 y	93.0 (641)	80.6 (556)	52.7 (363)	40.3 (278)	38.0 (262)	31.0 (214)	20.2 (139)	15.5 (107)
10 y	77.4 (534)	67.1 (463)	43.9 (303)	33.5 (231)	31.6 (218)	25.8 (178)	16.8 (116)	12.9 (89)
50 y	69.1 (476)	59.9 (413)	39.1 (270)	29.9 (206)	28.2 (194)	23.0 (159)	15.0 (103)	11.5 (79)

† Typical values based on ASTM D 638 testing of molded plaque material specimens.

Controlling Expansion and Contraction

Black polyethylene pipe on the surface or above grade and exposed to the sun can absorb solar energy. The resulting pipe temperatures can be greater than the air temperature. To help reduce temperature changes resulting solar heating of a piping system, the pipe may be shaded or placed in a location that receives less direct sunlight.

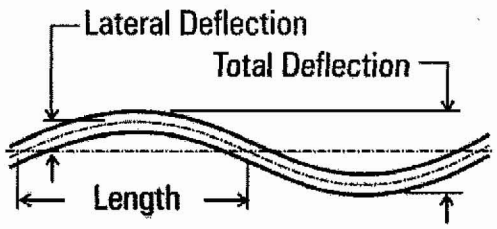
The effects of thermal expansion and contraction on a piping system can be controlled in several ways, including

- Lateral deflection expansion loops (snaking the pipe)
- Anchor and guide the pipe
- Conventional Expansion loops
- Expansion joints (non-pressures systems only)
- Burying pipes

Lateral Deflection Expansion Loops

The simplest installation involves stringing pipe between end point anchor structures. If the pipe is simply laid in a straight line between the end anchors the pipeline anchoring structures must be capable of handling potentially high thermal contraction thrust loads during temperature decrease, and during temperature increase, the maximum compressive thrust is the force required to cause lateral deflection at which time the compressive stress and end thrust will then

Figure 5-1 Lateral Deflection



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Modified Arching Load

For flexible pipe, a more conservative approach is to use a soil pressure load between the prism load and the Marston load. One approach is to add 40 percent of the difference between the prism load and the Marston load to the Marston load. Formula 7-5 may be used to obtain the modified arching load vertical soil pressure.

$$P_C = 0.6P_M + 0.4P_E \quad \textbf{(7-5)}$$

Where terms are previously defined and:

P_C = modified arching vertical soil pressure, lb/ft^2

In example 7-1, the modified arching vertical soil pressure from Formula 7-5 is:

$$P_C = 0.6(1512) + 0.4(2160) = 1771\ lb/ft^2$$

A value for the modified arching vertical soil pressure suitable for most soils may be determined from formula 7-6.

$$P_C = FwH \quad \textbf{(7-6)}$$

Where terms are previously defined and

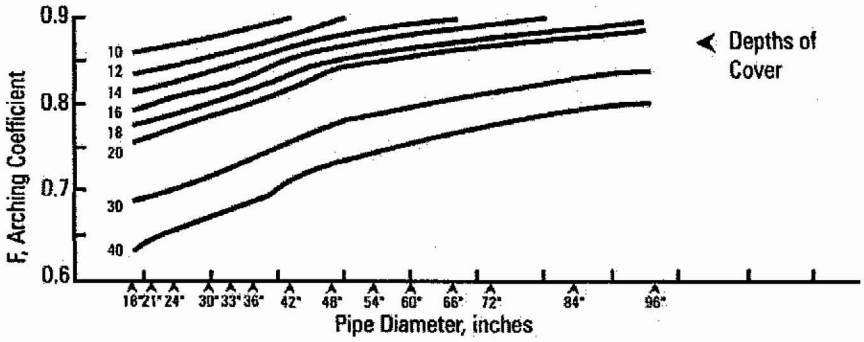
F = arching coefficient

$$F = \frac{P_M + 0.4(P_E - P_M)}{P_E} \quad \textbf{(7-7)}$$

Figure 7-3 is a graphical solution for the arching coefficient, F, based on the Marston load obtained with Ku′ = 0.130 for ordinary clay soil. Thus the Figure 7-3 arching coefficient is conservative for soils having a Ku′ value greater than 0.130. The arching coefficient should be used only where the trench width does not exceed 3 ft plus pipe OD for 42" and smaller pipe, and 4 ft plus pipe OD for 48" and larger pipe.

Figure 7-3 Arching Coefficient for Modified Arching Load

Based on clay soil, Ku' = 0.130, and trench widths of 3 ft plus pipe OD for 42" diameter and smaller pipe, and trench widths of 4 ft plus pipe OD for 48" diameter and larger pipe.



In Example 7-1, the arching coefficient, F, from Figure 7-3 is 0.82. Solving Formula 7-6 yields:

$$P_C = 0.82(120)(18) = 1771\ lb/ft^2$$

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Although net soil pressure is equal to the buoyant weight of the soil, ground water pressure is also acting on the pipe. Therefore the total pressure (water and earth load) can be found using the saturated unit weight of the soil.

$$P_C = (0.83)(120)(18) = 1793\ lb/in^2$$

Next, solve Formula 7-24 for the compressive stress. For DriscoPlex™ 2000 SPIROLITE® 36" Class 100 pipe, the wall cross-sectional area, A, and outside diameter, D_O are found in DriscoPlex™ 2000 SPIROLITE® pipe product literature. A is 0.470 in^2/in, and D_O is 36 plus twice the 2.02" wall height, or 40.04 in.

$$S = \frac{(1793)(40.04)}{(288)(0.470)} = 530\ lb/in^2$$

The application is within the 800 lb/in^2 allowable stress guideline.

Unconstrained Pipe Wall Buckling

Flexible pipe may be viewed as having the cross section of a long, slender column rolled into a cylinder. Compressive thrust, in combination with radially directed forces, may cause instability or buckling, that is, a large wrinkle or dimple in the pipe wall. This type of deflection can be compared to the Euler buckling of a column.

Unconstrained pipe wall buckling can be a consideration for low pressure and non-pressure pipes where the pipe is not externally supported by embedment, or when embedment provides little or no support. Compared to the capacity for tensile wall stress from internal pressure, unconstrained flexible pipe has less resistance to external, radially directed pressure. Some examples of external pressures on unconstrained pipe include: external atmospheric pressure from vacuum within the pipe; external hydrostatic load such as groundwater above a slipliner, or water above partially full underwater pipeline; a column separation in a liquid flow in a downhill pipeline; siphoning or a reduced internal pressure where a flow liquid in a pipeline crests a rise; and cavitation due to pump shut-off or start-up. If an unconstrained pipe will be subjected to an external pressure during service, the unconstrained buckling resistance should be checked.

For unconstrained pipe, the critical external pressure or negative pressure above which buckling can occur may be estimated by:

For DriscoPlex™ 2000 SPIROLITE® pipe:

$$P_{CR} = \frac{24 E I_P}{(1-\mu^2) D_M{}^3} \qquad \textbf{(7-25)}$$

Where

P_{CR}	=	vertical load applied to pipe, lb/ft^2
E	=	elastic modulus, lb/in^2 (Table 5-1)
I_P	=	profile wall moment of inertia, in^4/in
μ	=	Poisson's ratio
D_M	=	pipe mean diameter, in

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For DriscoPlex™ 2000 SPIROLITE® pipe:

$$D_M = D_I + 2Z \quad \textbf{(7-26)}$$

For DriscoPlex™ OD controlled pipe:

$$D_M = D_O - 1.06t \quad \textbf{(7-27)}$$

Where

D_I = pipe inside diameter, in
Z = profile wall centroid, in
D_O = pipe outside diameter
t = pipe minimum wall thickness

Poisson's ratio, μ, for polyethylene pipe is 0.45 for long-term loading and 0.35 for short-term loading. Expressing critical external buckling pressure in terms of DR for OD controlled pipe, Formula 7-25 becomes

$$P_{CR} = \frac{2E}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \quad \textbf{(7-28)}$$

Where

DR = pipe dimension ratio

$$DR = \frac{D_O}{t} \quad \textbf{(7-29)}$$

Ovality Effects

Ovality or deflection of the pipe diameter reduces buckling resistance because the bending moment in the pipe wall increases.

$$P = f_O\, P_{CR} \quad \textbf{(7-30)}$$

Where

P = buckling pressure, lb/in^2
f_O = ovality compensation factor, Figure 7-14

Initially deflected unconstrained pipe generally assumes an oval shape. The percent deflection (ovality) of pipe is determined by

$$\%\,Deflection = 100\left(\frac{D - D_O}{D}\right) \quad \textbf{(7-31)}$$

Where

D = pipe average diameter, in
D_O = pipe minimum diameter, in

See Table 5-1 for elastic modulus values for determining critical buckling pressure in Formulas 7-25 and 7-28. The modulus selected should account for the temperature of the pipe and the duration of the applied load. When unconstrained pipes are installed on or above the surface, sunlight heating can increase pipe temperature and reduce buckling resistance.

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For a constant water table above the pipe, Table 5-1 indicates a 50-year, 73° F modulus of 28,200 lb/in^2, thus Formula 7-28 yields

$$P_{CR} = \frac{(24)(28{,}200)(0.124)}{(1-0.45^2)(25^3)} = 6.79\ lb/in^2$$

Assuming 5% ovality and a 2 to 1 safety factor, f_O from Figure 7-14 is 0.64. Formula 7-30 yields

$$P = \frac{(0.64)(6.79)}{2} = 2.17\ lb/in^2 = 5.0\ ft\ H_2O$$

Flooding conditions are occasional happenings, usually lasting a few days to a week or so. From Table 5-1, 1000 hours (41.6 days) is about twice the expected flood duration, so a value of 43,700 lb/in^2 provides about a 2 to 1 safety margin. Solving as above,

$$P_{CR} = \frac{(24)(43{,}700)(0.124)}{(1-0.45^2)(25^3)} = 10.44\ lb/in^2$$

$$P = (0.64)(10.44) = 6.68\ lb/in^2 = 15.4\ ft\ H_2O$$

Constrained Pipe Wall Buckling

Buckling resistance is increased when flexible pipe is embedded in soil. The soil and pipe couple together to resist buckling forces. A vertically applied thrust force causes the pipe to widen horizontally, but horizontal pipe deflection is restrained by the embedment soil, thus the pipes critical buckling pressure increases. A pipe/soil interaction occurs when the depth of cover is sufficient to mobilize soil support. A publication by the American Water Works Association, AWWA C-950, indicates that at least four feet of cover is needed to mobilize soil support.

AWWA C-950 provides a design equation for buckling of a buried plastic pipe. The following constrained pipe buckling equation is applicable to DriscoPlex™ OD controlled and DriscoPlex™ 2000 SPIROLITE® pipe.

For OD Controlled Pipe

$$P_{WC} = \frac{5.65}{N}\sqrt{R\,B'\,E'\,\frac{E}{12(DR-1)^3}} \qquad \textbf{(7-32)}$$

For DriscoPlex™ 2000 SPIROLITE® Pipe

$$P_{WC} = \frac{5.65}{N}\sqrt{R\,B'\,E'\,\frac{E\,I}{D_M{}^3}} \qquad \textbf{(7-33)}$$

Where terms are previously defined and

P_{WC} = allowable constrained buckling pressure, lb/in^2
N = safety factor
R = buoyancy reduction factor

$$R = 1 - 0.33\frac{H'}{H} \qquad \textbf{(7-34)}$$

H' = groundwater height above pipe, ft
H = cover above pipe, ft

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B' = elastic support factor

$$B' = \frac{1}{1 + 4e^{(-0.065H)}} \qquad \textbf{(7-35)}$$

E' = soil reaction modulus, lb/in^2 (Table 7-7)

The designer should apply a safety factor appropriate to the application. A safety factor of 2.0 has been used for thermoplastic pipe.

The allowable constrained buckling pressure should be compared to the total vertical stress acting on the pipe crown from the combined load of soil and groundwater or floodwater. It is prudent to check buckling resistance against a groundwater level for a 100-year-flood. In this calculation the total vertical stress is typically taken as the prism load pressure for saturated soil, plus the fluid pressure of any floodwater above the ground surface.

Example 7-11

Find the allowable buckling pressure for a DriscoPlex™ 2000 SPIROLITE® 36" Class 100 36" pipe, installed in compacted soil embedment having an E´ of 2000 lb/in^2. Determine if Class 100 pipe is sufficient for an applied load from 18 feet of cover and ground water to the surface.

Solution: Solve Formula 7-33 using Formulas 7-26, 7-35, 7-34 and Table 5-1. DriscoPlex™ 2000 SPIROLITE® pipe dimensions and properties are published in Bulletin PP-401. For DriscoPlex™ 2000 SPIROLITE® 36" Class 100 pipe, I is 0.171 in^4/in, and Z is 0.58 in. Solve for terms D_M, B', and R.

$$D_M = 36 + 2(0.58) = 37.16\,in$$

$$B' = \frac{1}{1 + 4e^{(-0.065(18))}} = 0.446$$

$$R = 1 - 0.33\frac{18}{18} = 0.67$$

Under a 100-year-flood condition, soil cover, H, and floodwater height, H´, are both 18 feet.

From Table 5-1, E is 28,200 lb/in^2 for 50 years at 73° F. A common practice is a safety factor of 2. Solving Formula 7-32,

$$P_{WC} = \frac{5.65}{2}\sqrt{\frac{(0.67)(0.446)(2000)(28{,}800)(0.171)}{(37.16)^3}}$$

$$P_{WC} = 21.17\,lb/in^2 = 3051\,lb/ft^2$$

The load applied to the pipe is found using the prism load, Formula 7-1.

(In this example, the specified soil reaction modulus, E´, is an empirical value that was developed using prism load rather than arching load methods. Therefore, the prism soil load must be used. If a soil reaction modulus value is developed using arching or modified arching methods, then soil loads should be calculated using the appropriate method. See discussions on Soil Reaction Modulus and Vertical Soil Pressure.)

$$P_E = (120)(18) = 2160\,lb/ft^2$$

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DriscoPlex™ 2000 SPIROLITE® pipe is manufactured to ASTM F 894, which states that profile pipe designed for 7.5% deflection will perform satisfactorily when installed in accordance with ASTM D 2321. Deflection is measured at least 30 days after installation.

Manufacturing processes for DriscoPlex™ 2000 SPIROLITE® and DriscoPlex™ OD controlled pipe differ. Deflection limitations for OD controlled pipe are controlled by long-term material strain.

Ring Bending Strain

As pipe deflects, bending strains occur in the pipe wall. For an elliptically deformed pipe, the pipe wall ring bending strain, ε, can be related to deflection:

$$\varepsilon = f_D \frac{\Delta X}{D_M} \frac{2C}{D_M} \qquad \textbf{(7-39)}$$

Where

ε	=	wall strain
f_D	=	deformation shape factor
ΔX	=	deflection, in
D_M	=	mean diameter, in
C	=	distance from outer fiber to wall centroid, in

For DriscoPlex™ 2000 SPIROLITE® pipe

$$C = h - z \qquad \textbf{(7-40)}$$

For DriscoPlex™ OD Controlled pipe

$$C = 0.5(1.06t) \qquad \textbf{(7-41)}$$

Where

h	=	pipe wall height, in
z	=	pipe wall centroid, in
t	=	pipe minimum wall thickness, in

For elliptical deformation, f_D = 4.28. However, buried pipe rarely has a perfectly elliptical shape. Irregular deformation can occur from installation forces such as compaction variation alongside the pipe. To account for the non-elliptical shape many designers use f_D = 6.0.

Lytton and Chua report that for high performance polyethylene materials such as those used by Performance Pipe, 4.2% ring bending strain is a conservative value for non-pressure pipe. Jansen reports that high performance polyethylene material at an 8% strain level has a life expectancy of at least 50 years.

When designing non-pressure heavy wall OD controlled pipe (DR less than 17), and high RSC (above 200) DriscoPlex™ 2000 SPIROLITE® pipe, the ring bending strain at the predicted deflection should be calculated and compared to the allowable strain.

In pressure pipe, the combined stress from deflection and internal pressure should not exceed the material's long-term design stress rating. Combined stresses are incorporated into Table 7-9 values, which presumes deflected pipe at full pressure. At reduced pressure, greater deflection is allowable.

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Table 7-9 Safe Pressure Pipe Deflection

DR	*Safe Deflection as % of Diameter*
32.5	8.5
26	7.0
21	6.0
17	5.0
13.5	4.0
11	3.0
9	2.5

Example 7-13

Find the ring bending strain in the wall of the DriscoPlex™ 2000 SPIROLITE® 36" Class 100 pipe in Example 7-12.

Solution: Use Formula 7-40 and f_D = 6.0. From Bulletin PP-401, h = 2.02 in., and z = 0.58 in.

$$\varepsilon = 6(0.0237)\frac{(2.02-0.58)}{(36+2(0.58))} = 0.0055 = 0.55\%$$

The strain is well below the permissible strain for ASTM F 894 profile pipe.

Design Considerations For Shallow Cover Pipe

Pipe installed under shallow cover does not completely develop a the interaction between pipe and soil structure interaction; therefore, design methods must be modified. The designer should consider the following three cases: (1) flotation due to insufficient soil cover, (2) ring bending due to live load, and (3) upward buckling due to flooding or high groundwater levels.

The exact depth of cover required to fully develop pipe-soil structure interaction depends on the particular installation conditions.

Shallow Cover Surcharge Load

The preceding design methods assume that the pipe behaves primarily as a membrane structure, that is, the pipe is almost perfectly flexible with little ability to resist bending.

At depths of cover less than one pipe diameter, this membrane action may not develop fully, thus a surcharge load or live load places a bending load on the pipe crown. For this reason, flexible pipe manufacturers often recommend that pipe be buried at least one pipe diameter below a live load. If this cannot be accomplished, the designer should perform a special analysis to determine if the pipe has adequate beam bending strength.

R. Watkins in *Minimum Soil Cover Required Over Buried Flexible Cylinders* provides a design equation for determining the pipe cross section for shallow cover live load applications. Watkins' method presumes that a combination of pipe flexural strength and the ring resistance of the soil surrounding the pipe resist the live load at shallow cover. The maximum bending stress

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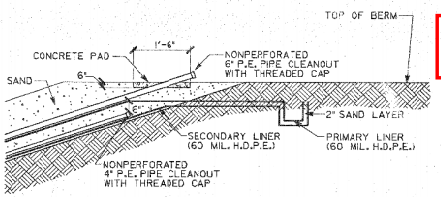
Reference 6

A-1



CLEANOUT DETAIL (A, C-10)
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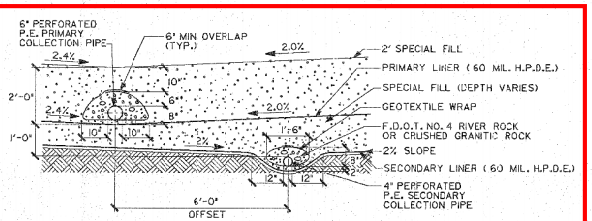

LEACHATE COLLECTION PIPE DETAIL (B, C-10)
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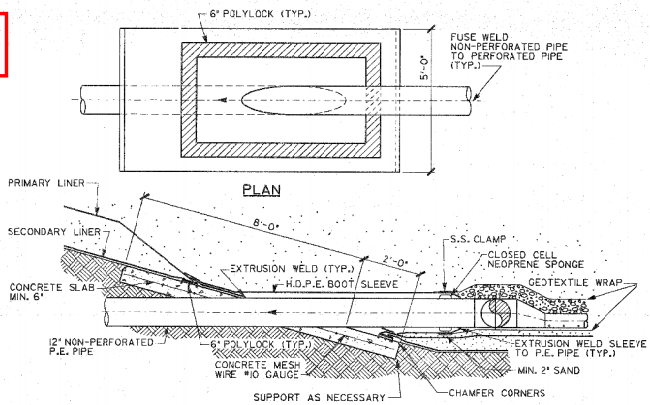

SINGLE PIPE PENETRATION DETAIL (D)
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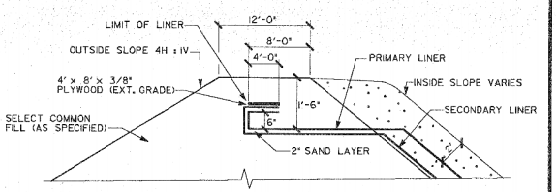

NOTE: TO BE INSTALLED IN INTERIOR BERMS ONLY. SEE SHEET 6 FOR INTERIOR BERM LOCATIONS.

INTERIOR LINER ANCHOR DETAIL (F, C-10)
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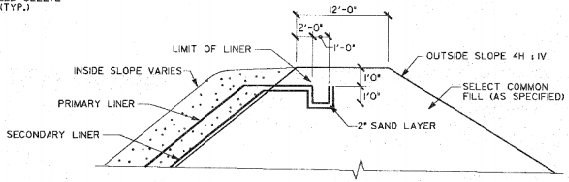

NOTE: TO BE INSTALLED IN EXTERIOR BERMS ONLY. SEE SHEET 6 FOR EXTERIOR BERM LOCATIONS.

EXTERIOR LINER ANCHOR DETAIL (G, C-10)
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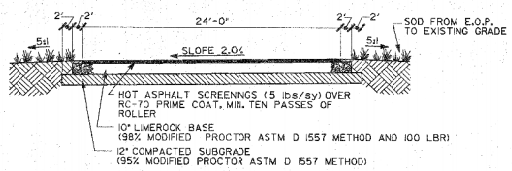
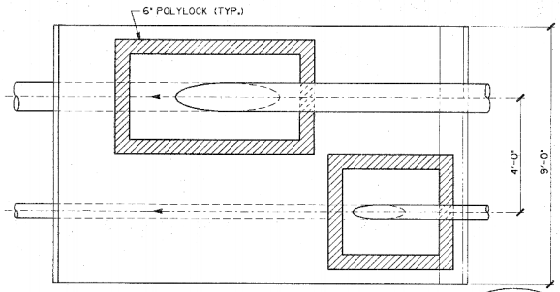
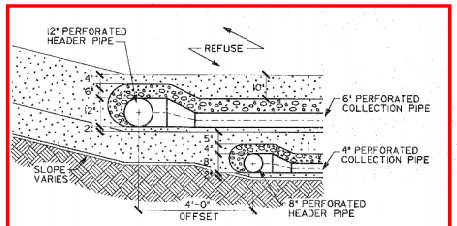


SECTION (1, C-11)
N.T.S.





SECTION (2)
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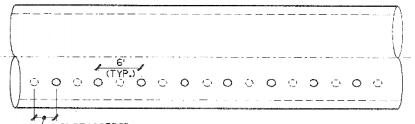

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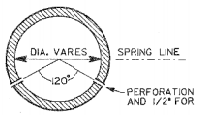

SECTION

PIPE PERFORATION DETAIL (C)
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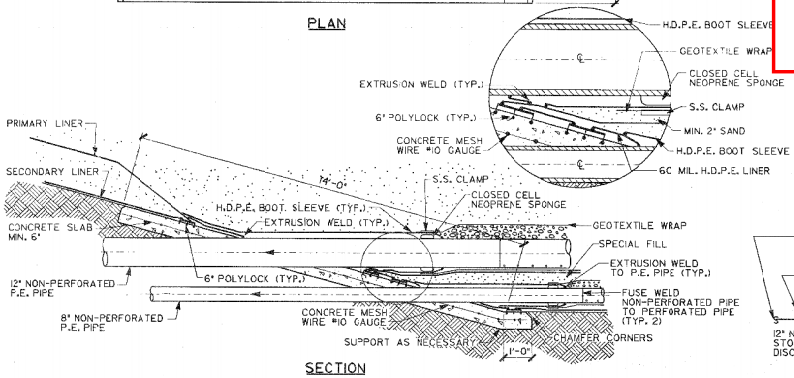

DOUBLE PIPE PENETRATION DETAIL (E)
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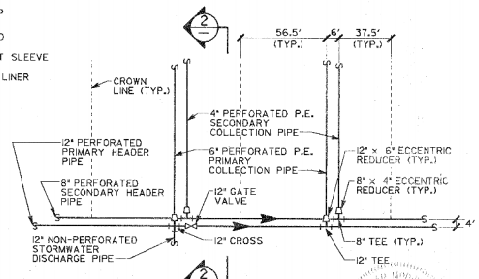


HEADER PIPE DETAIL (H)
N.T.S.

REV. NO.	DATE	DRWN	CHKD	REMARKS
1	3/8/99	SRB	DM	ADDM.#2-DETAILS MOVED TO SHT. C-16

DESIGNED BY: C.G. SRB
DRAWN BY: LKS
CHECKED BY: DM
APPROVED BY:
DATE: AUG. 1989

CAMP DRESSER & McKEE INC.
environmental engineers, scientists, planners & management consultants

WEST PASCO COUNTY CLASS I LANDFILL - PHASE I CONSTRUCTION DRAWINGS

MISCELLANEOUS DETAILS AND SECTIONS

PASCO COUNTY, FLORIDA

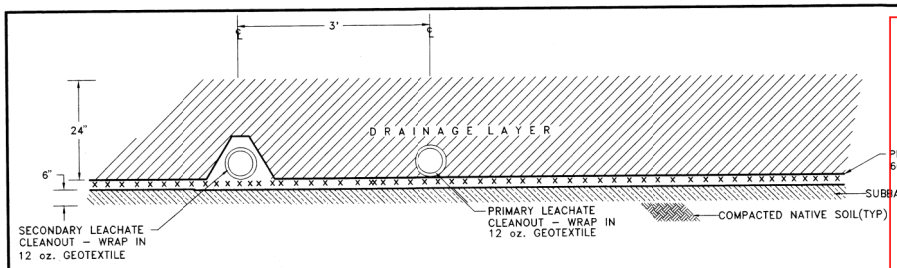
PROJECT NO. 6104-03

SHEET NO. C-14

DATE OF PLOT IS 9-AUG-1989 22:04
NAME OF FILE IS P:\PIP\DTL37\PIPDTL03.C4.DGN

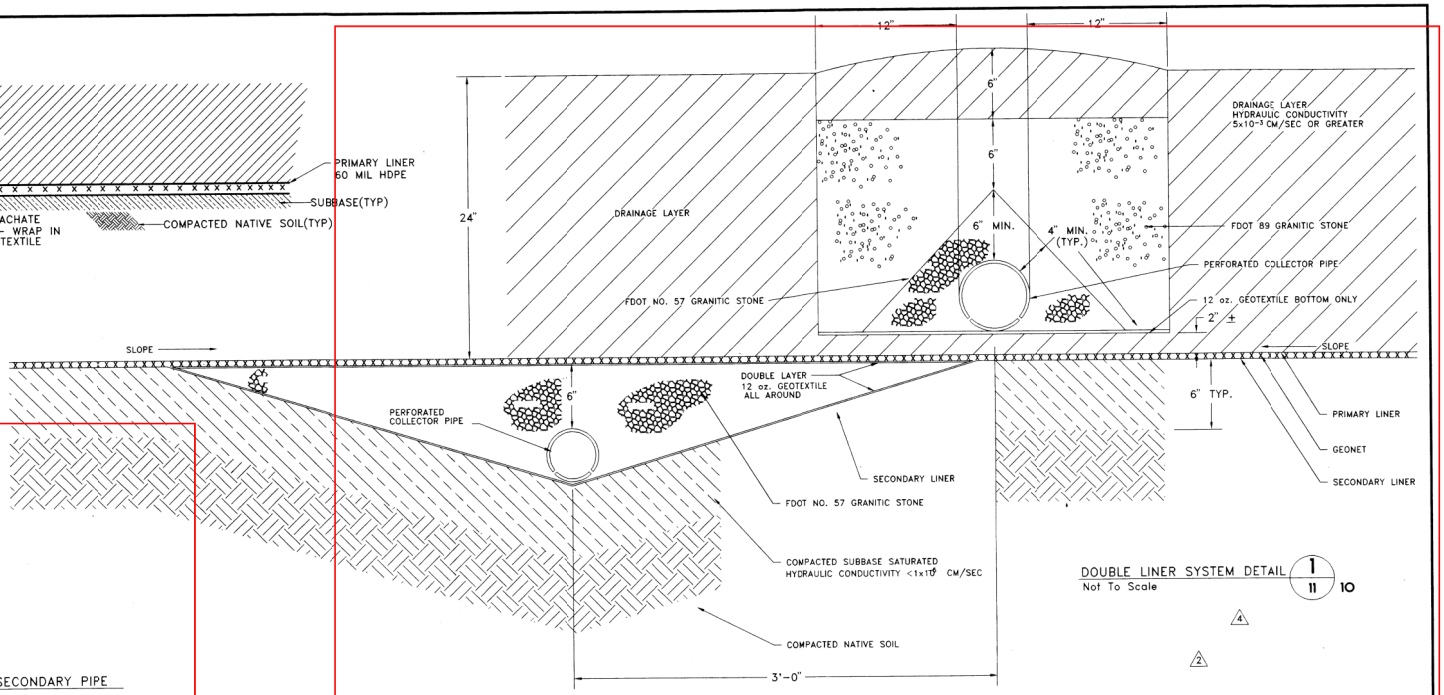
RECORD DRAWINGS MAY 1 10, 1991

A-2

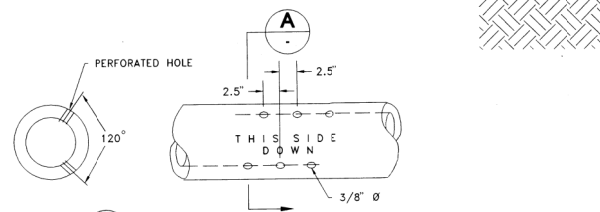


SECTION F / 11 8 — Not To Scale

SECTION THROUGH BERM SHOWING CLEANOUT PLACEMENT IN BERM

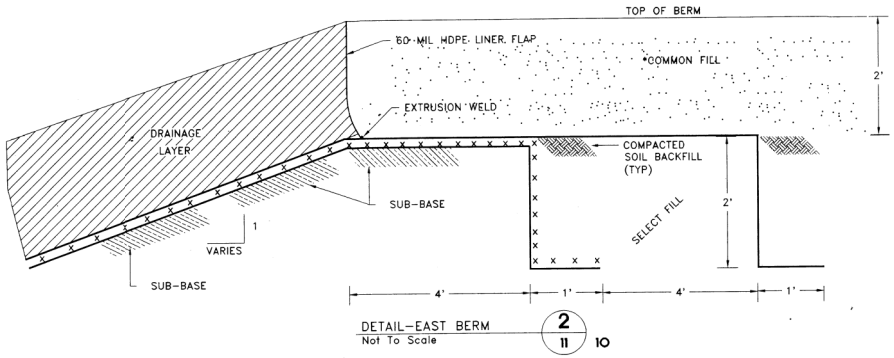


DOUBLE LINER SYSTEM DETAIL 1 / 11 10 — Not To Scale

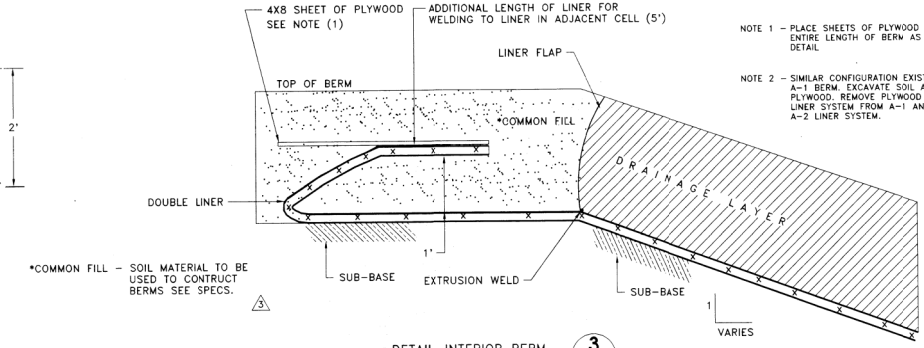


SECTION A / - TYPICAL PLAN OF PERFORATED PRIMARY AND SECONDARY PIPE — Not To Scale

NOTE: COLLECTION AND HEADER PIPES ARE PERFORATED, CLEANOUT PIPES ARE SOLID



DETAIL–EAST BERM 2 / 11 10 — Not To Scale



*COMMON FILL – SOIL MATERIAL TO BE USED TO CONTRUCT BERMS SEE SPECS. 3

NOTE 1 – PLACE SHEETS OF PLYWOOD 4' WIDE END TO END ENTIRE LENGTH OF BERM AS SHOWN IN THIS DETAIL

NOTE 2 – SIMILAR CONFIGURATION EXISTS AT THE SOUTH A-1 BERM. EXCAVATE SOIL ABOVE EXISTING PLYWOOD. REMOVE PLYWOOD AND EXTEND EXISTING LINER SYSTEM FROM A-1 AND WELD PARTS TO NEW A-2 LINER SYSTEM.

DETAIL–INTERIOR BERM 3 / 11 10 — Not To Scale

ACAD=48356511

REV	DATE	BY	SUB	APP	DESCRIPTION
1	9/94	REM		REM	DRAFTING CHANGES
2	11/94	REM		REM	DRAFTING CHANGES
3	12/94	REM		REM	ADD NOTES
4	2/95	REM		REM	MODIFY SECTION

DESIGNED GWE
DRAWN RDD
CHECKED REM
IN CHARGE REM
DATE 6/94

PASCO COUNTY BOARD of COUNTY COMMISSIONERS
UTILITY SERVICES BRANCH

SUBMITTED

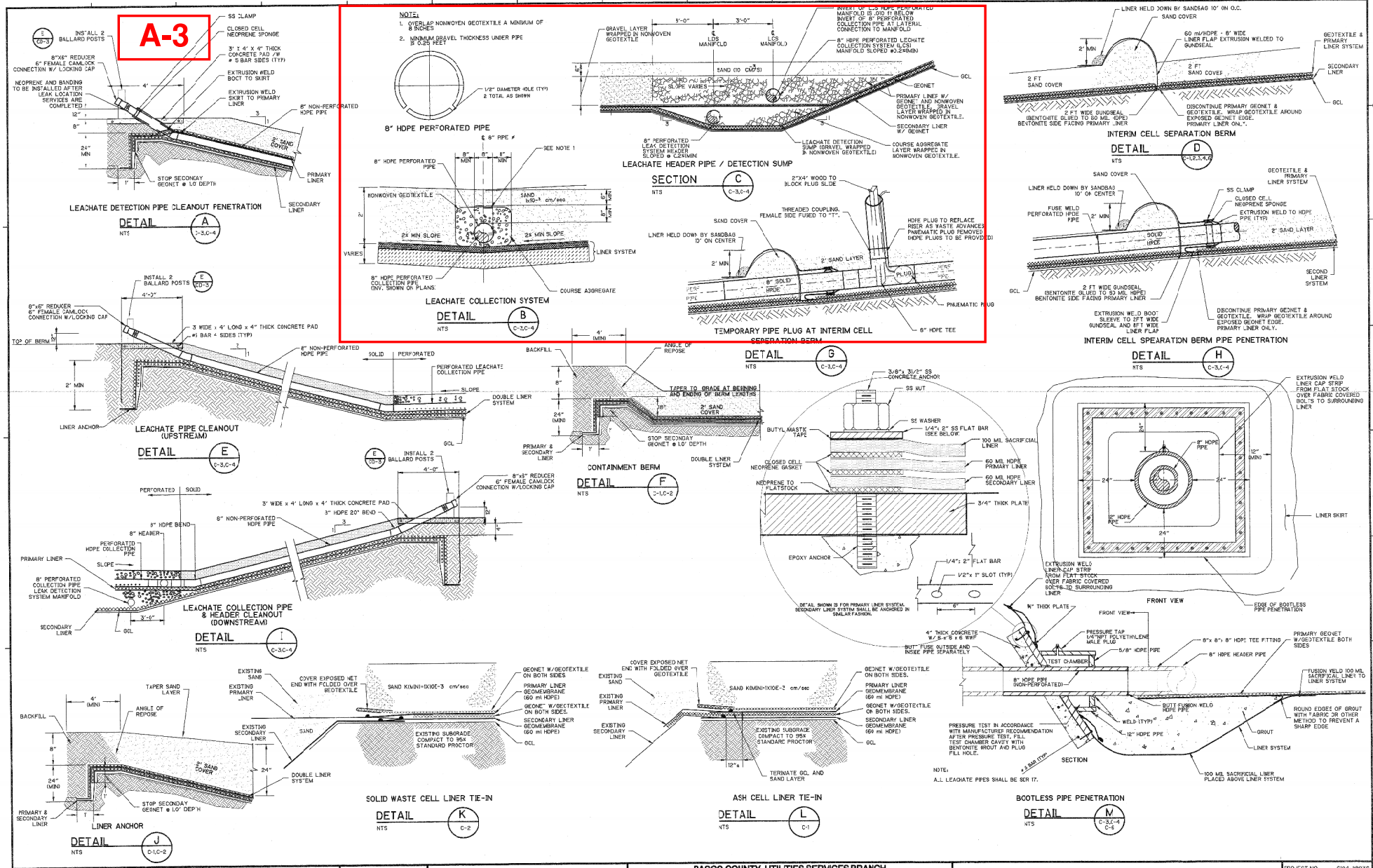
LAW ENGINEERING AND ENVIRONMENTAL SERVICES
APPROVED

RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

DOUBLE LINER SYSTEM
SECTIONS and DETAILS

SCALE AS NOTED
CONTRACT 573565
DWG NO 11 | REV | PAGE NO

A-3



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: D. EL-HAJJI
DRAWN BY: L.SICORA
SHEET CHK'D BY: J. McHUGH
CROSS CHK'D BY: D. STROBRIDGE
APPROVED BY: A.MILLA-SALEH
DATE: APRIL 2001

CDM Camp Dresser & McKee Inc.
consulting engineering construction operations
1715 North Westshore Blvd. Suite 875
Tampa, Florida 33607
Tel: 813-281-2900
License no. EB-0000020

PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

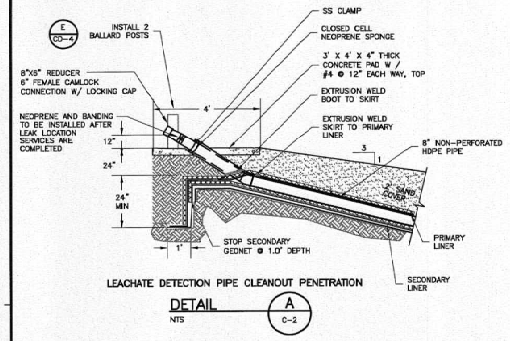
WEST PASCO COUNTY LANDFILL EXPANSION CELLS A-3 AND SW-2

MISCELLANEOUS DETAILS

PROJECT NO. 6104-28936
FILE NAME: cetdtcdl.dwg

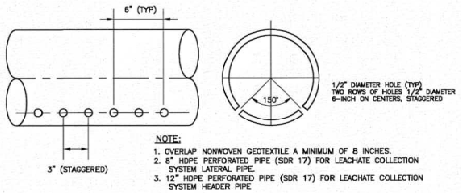
SHEET NO.
CD-1

A-4

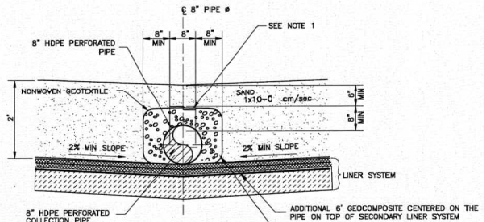


LEACHATE DETECTION PIPE CLEANOUT PENETRATION

DETAIL A C-2 NTS

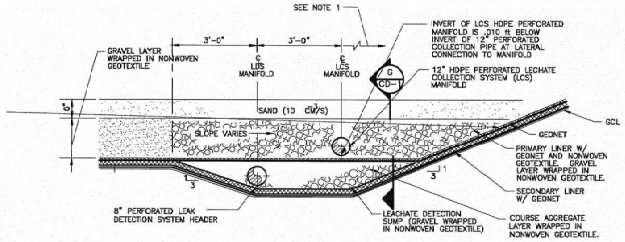


NOTE:
1. OVERLAP NONWOVEN GEOTEXTILE A MINIMUM OF 6 INCHES.
2. 8" HDPE PERFORATED PIPE (SDR 17) FOR LEACHATE COLLECTION SYSTEM LATERAL PIPE.
3. 12" HDPE PERFORATED PIPE (SDR 17) FOR LEACHATE COLLECTION SYSTEM HEADER PIPE



LEACHATE COLLECTION SYSTEM

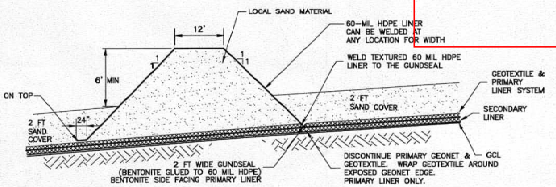
DETAIL B C-2 NTS



NOTE:
1. THE ℄ OF LCS MANIFOLD IS 34' OFF THE INSIDE EDGE OF LINER ANCHOR BERM.

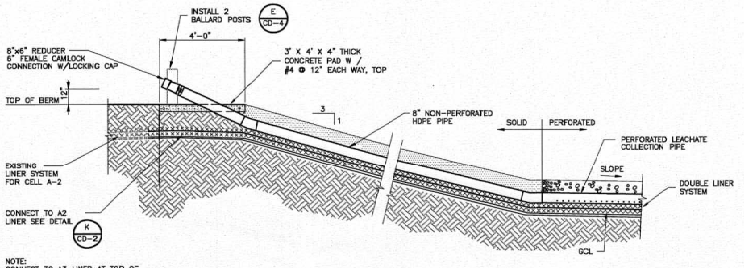
LEACHATE HEADER PIPE / DETECTION SUMP

SECTION C C-2 NTS



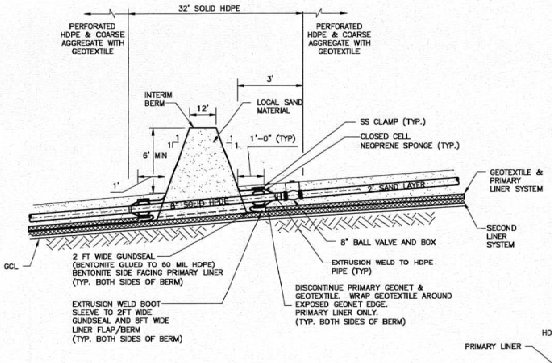
INTERIM CELL SEPARATION BERM (RUNNING NORTH-SOUTH)

DETAIL D C-2 NTS



LEACHATE COLLECTION PIPE CLEANOUT (UPSTREAM)

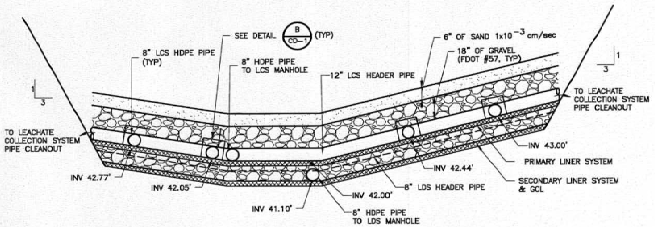
DETAIL E C-2



NOTE:
FOR CONTROL OF STORMWATER AND OPERATION OF 8" BALL VALVES, SEE DRAWINGS C-6 TO C-11 AND OPERATIONS PLAN 2.5.1.

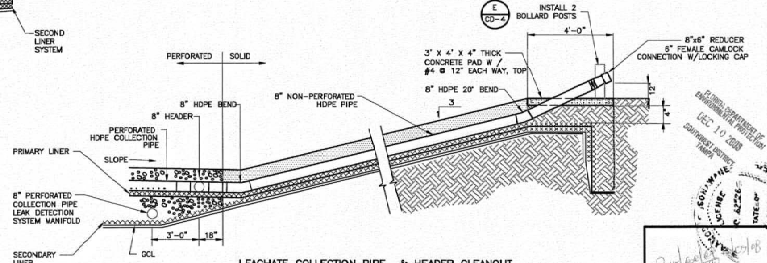
INTERIM CELL SEPARATION BERM PIPE PENETRATION (RUNNING EAST-WEST)

DETAIL H C-2 NTS



LEACHATE COLLECTION / DETECTION SYSTEM SUMP

SECTION G C-2 NTS



LEACHATE COLLECTION PIPE & HEADER CLEANOUT (DOWNSTREAM)

DETAIL I C-2 NTS

DATE:
AAMOD S. SONAWANE
P.E NO. 62526

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: A. SONAWANE
DRAWN BY: R. PETERMAN
SHEET CHK'D BY: J. LADNER
CROSS CHK'D BY: R. STROBRIDGE
APPROVED BY: A. SONAWANE
DATE: NOVEMBER 2009

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consulting • engineering • construction • operations

PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

WEST PASCO CLASS I LANDFILL
CELL A-4 CONSTRUCTION

LINER SYSTEM DETAILS

PROJECT NO. 6104-4613B
FILE NAME: CD01STDT

SHEET NO.
CD-1

Xref's: □ Images: □
Filename: L:\6104\4613B\Mod\CIVIL\DETAILS\CD01stdt.dwg Last saved by: MOHLEDP Time: 10/29/2009 2:29:14 PM

Reference 7

LAW COMPANIES

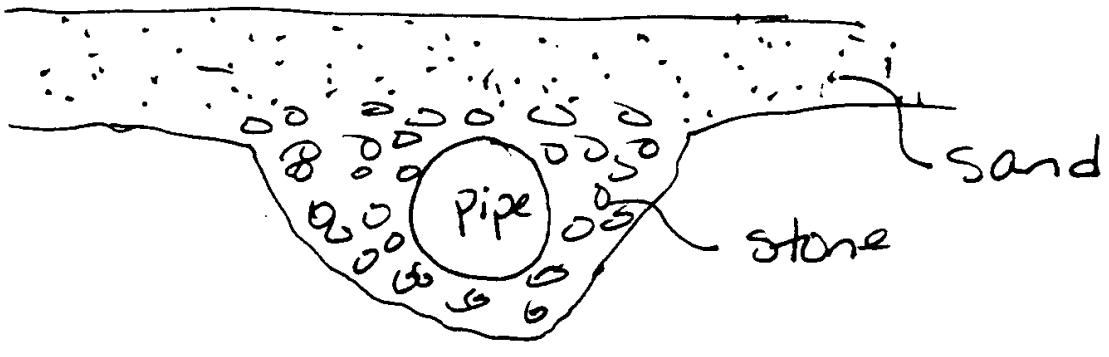
GEOTECHNICAL, ENVIRONMENTAL
& CONSTRUCTION MATERIALS
CONSULTANTS

JOB NO. _____ SHEET 1 OF 2

JOB NAME Pasco County Landfill

BY LAM DATE 7/7/94

CHECKED BY CJR DATE 7/7/94



stone ↙ sand ↙

D_{15} (filter) $\leq 5 D_{85}$ (soil) ✓

D_o (pipe) $\leq$ D_{85} (soil) ✓

↑ stone

D_{85} of silica sand (FDOT Silica Sand section 902-2)

$\simeq$ 0.9mm to 2.36 mm

$5 D_{85}$ = 4.5mm to 11.8 mm

D_{15} stone (No 89) — FDOT

$\simeq$ 1.2 mm to 4 mm

D_{15} (filter) $\leq$ $5 D_{85}$ (soil) YES

1.2 mm to 4mm $\leq$ 4.5mm to 11.8 mm

D_{85} stone (No 89) — FDOT ✓

$\simeq$ 7mm to 9mm

$D_o \leq D_{85}$ (soil)

$\therefore$ pipe openings must be less than 7mm ✓

LAW COMPANIES

GEOTECHNICAL, ENVIRONMENTAL
& CONSTRUCTION MATERIALS
CONSULTANTS

JOB NO. _____ SHEET 2 OF 2

JOB NAME Pasco County Landfill

BY LAM DATE 7/7/94

CHECKED BY CJL DATE 7/7/94

7mm = 0.7cm

1inch = 2.54cm

∴ 7mm = 0.276 inches

pipe openings must be
less than 0.276 inches

use 0.25 inches or less ✓

LAW COMPANIES

GEOTECHNICAL, ENVIRONMENTAL
& CONSTRUCTION MATERIALS
CONSULTANTS

JOB NO. 464-3565.01 SHEET 1 OF ____

JOB NAME Resource Recovery Class I

BY REM DATE 7/6/94

CHECKED BY RB DATE 7/7/94

Pipe Strength / Loading

NON-pressure

Pipes - Collection & header - Burial elevation 125'
- 47
78'

- Sides Slope Riser 30'

pressure

- Force main 4" dinde & 5" containment
may use 6" ϕ if spacers are
required

Preliminary Design for permit based on Driscopipe Eng Manual - a copy of some pages is attached.

1st Buried collection / header pipe using Chart 24 Soil Type II slight to moderate Compaction Soil modulus - 1500
Then for Max Burial 78' - use SDR 17

2ND Riser - use a stiffer pipe check chart
SDR 13.5 OK

Reference 7

LAW COMPANIES

GEOTECHNICAL, ENVIRONMENTAL
& CONSTRUCTION MATERIALS
CONSULTANTS

JOB NO. 464-3565.01 SHEET 2 OF ____

JOB NAME Pressure Recovery Class I

BY REM DATE 7/6/94

CHECKED BY ____ DATE ____

FINALLY for pressure pipe force main

SDR - 17 - 100 psi, which is more than adequate.

Reference 8

be small compared to the pressure due to the fill, the vertical pressure on the top of the pipe can be assumed to be equal to the unit weight of the refuse fill multiplied by the distance from top of fill to top of pipe, thus:

$$\sigma_v = (\omega_f)(H_f).$$

V.2.2.3 Perforated Pipe

Perforations will reduce the effective length of pipe available to carry loads and resist deflection. The effect of perforations can be taken into account by using an increased load per nominal unit length of the pipe. If l_p equals the cumulative length in inches of perforations per foot of pipe, the increased vertical stress to be used equals:

$$(\sigma_v)_{design} = \frac{12}{12-l_p} \times (\sigma_v)_{actual}$$

V.2.3 Deflection

A well accepted formula for calculating flexible pipe deflection under earth loading is that developed by Spangler. This equation, also known as the Iowa formula, is presented together with suggested values for its various constants in the 1970 edition of the American Society of Civil Engineers (ASCE) Manual of Practice, No. 37, Chapter 9, Section E, Subsection 1, and is as follows:

$$\Delta y = D_e \frac{KWr^3}{EI + 0.061\, E'r^3}$$

where:

Δy = horizontal and vertical deflection of the pipe (in)

D_e = a factor, generally taken at a conservative value of 1.5, compensating for the lag or time dependent behavior of the soil/pipe systems (dimensionless)

W = vertical load acting on the pipe per unit of pipe length (lb/in)

r = mean radius of the pipe (in)

E = modulus of elasticity of the pipe materials (psi)

E' = modulus of passive soil resistance (psi) (normally estimated to be 300 psi for soils of Proctor density of 65%, and 700 psi for soils of Proctor density of at least 90%)

Attachment J.1-4

Stormwater Calculations

TECHNICAL MEMORANDUM

WEST PASCO COUNTY LANDFILL ASH MONOFILL SLOPE MODIFICATION AND VERTICAL EXPANSION

TO: Project File

FROM: Chad Spreadbury, PhD, EI

XC: George Reinhart, PhD, PE

DATE: June 29, 2021

SUBJECT: Attachment J.1-4 – Stormwater Design
Jones Edmunds Project No. 16002-009-01

1 PURPOSE

This Technical Memorandum provides the stormwater system design for the conceptual build-out of the West Pasco County Landfill (WPCL) Class I Landfill Cells A-1, A-2, A-3, and A-4 in accordance with Rule 62-701.400(9), Florida Administrative Code (FAC). The design includes terrace swales, drop inlets, and downcomers for the 25-year/24-hour design storm event.

2 METHODOLOGY

The proposed closure stormwater system includes terrace swales, drop inlets, and downcomers on the WPCL Class I ash monofill slope modification and vertical expansion to convey stormwater to the perimeter swales and permitted treatment systems at the time of closure. Sheet C6, *Conceptual Final Cover Plan,* of the Engineering Plans in Appendix A, provides the conceptual layout of the terraces, downcomers, and control structures proposed as part of the final cover system. Figure J.1-4 shows the delineation of the basins and basin areas.

Figure J.1-4 shows that the landfill expansion was divided into 14 basins (A through N), and each basin was subdivided into sub-basins (numbered 1 through 2) by the stormwater berms.

The hydrologic calculations were performed using the rational method, and the hydraulic design was based on Manning's equation. The design storm is the 25-year/24-hour storm. Design calculations include the following:

1. Determining the peak runoff from each basin using the rational method:

   $Q = CiA$

   Where:

   Q = peak discharge (cubic feet per second [cfs]).
   C = runoff coefficient [-].
   i = rainfall intensity (inches per hour [in/hr]).
   A = basin area (acres [ac]).

2. Calculating the downcomers pipe and tack-on berm capacities using Manning's equation for open channel flow:

   $Q=\frac{1.49}{n} R^{\frac{2}{3}} A \sqrt{s}$

   Where:

   Q = flow rate [cfs].
   n = Manning roughness coefficient [-].
   R = hydraulic radius (feet [ft]) = A/P.
   A = inside cross-section area (square feet [ft^2]).
   P = wetted perimeter [ft].
   s = slope [ft/ft].

3. Calculating the drop inlet weir capacity based on the orifice weir equation:

   $Q= C_1 A\sqrt{2gH}$

   Where:

   Q = weir flow rate [cfs].
   C_1 = weir coefficient [-].
   A = weir area [ft^2].
   g = acceleration due to gravity (foot per second squared [ft/sec^2]).
   H = total hydraulic head [ft].

3 PARAMETERS

Table 1 gives the parameters used for this stormwater design. The references are included with this Technical Memorandum.

Table 1 Stormwater Design Parameters

Parameter	Value	Reference
Design Storm	25-year/24-hour	Rule 62-701, FAC
Rainfall	9 inches	FDOT (Reference 1)
Runoff Coefficient	0.95	(Reference 2)
Manning's Roughness Coefficients		
HDPE (single wall)	n = 0.024	ADS Pipe (Reference 4)
HDPE (double wall/N-12)	n = 0.012	ADS Pipe (Reference 4)
Grass Swale	n = 0.030	(Reference 2)
Weir Coefficient	0.61	(Reference 2)

4 RESULTS

The hydrologic and hydraulic analyses performed for the stormwater system and the results are summarized below:

- Peak Runoff – The peak runoff as calculated using the rational method yielded values of 0.56 to 36.5 cfs during the peak-intensity design storm.
- Berms – The berms have a maximum flow depth of 1.1 feet, which is sufficient to maintain a minimum freeboard of 1.4 feet during the peak-intensity design storm.
- Drop Inlets – The drop inlets specified are sufficient to handle the flow of the peak-intensity storm with a minimum factor-of-safety of 1.8.
- Downcomer Pipes – The double-walled high-density polyethylene pipe of various diameters is sufficient to convey the peak discharge from the peak-intensity storm at a maximum of 80-percent pipe capacity for a minimum factor of safety of 1.3.

Figure J.1-4

Ash Monofill Stormwater Basins

© Jones Edmunds 2234

7/7/2021 9:25 AM BY: PJPSTILL PATH= Y:\16002-PASCOCOUNTY\PROJECTS\009-01-ASHMONOFILLOPERATIONSMODIFICATION\CAD\FIGURES PERMIT\16002009-FIG-J.1-4.DWG 7/7/2021 09:30 AM BY: PAUL E. IJPSTILL

GRAPHIC SCALE
0 40 80 160
SCALE IN FEET

N

ASH CELL A-1
ASH CELL A-2
ASH CELL A-4
ASH CELL A-3

A-1 B-1 A-2 B-2
N-1 N-2
C-2 C-1
D-2 D-1
M-1 M-2
J-1 K-1 L-1 J-2 K-2 L-2
I-1 I-2 H-2 G-2 F-2 E-2
H-1 G-1 F-1 E-1

EXISTING EDGE OF LINER

FIGURE J.1-4
ASH MONOFILL STORMWATER BASINS
WEST PASCO COUNTY LANDFILL AND
RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

JonesEdmunds

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PROJECT NUMBER: 16002-009-01 WORKSHEET: 1 OF 4
PROJECT NAME: Pasco Class I Ash Monofill Modifications
SUBJECT: STORMWATER DESIGN
BY: CSPREADBURY Date: 5/18/2021
CHECKED BY: MMORSE Date: 6/21/2021

Objective: Determine if the final cover stormwater management system is sufficient for the 25-year/24-hour design storm. Calculate the peak discharge using the Rational Method for the 25-year/24-hour storm and compare it to the flow capacity of the sideslope terrace swales, drop inlets, downcomer pipes, and perimeter swales using the Manning Equation and the Orifice Weir Equation.

Data: See Figure J.1-4 for basin designations and areas.

Calculation: Determine peak discharge for each subbasin using the Rational Method.

$Q = CiA$ *(Reference 2, p. 20-13)*

where: Q = peak discharge [cfs]

C = runoff coefficient = 0.95 - *(Reference 2, p. A-44, Concrete/Asphalt)*

I_{25} = rainfall intensity 25 yr = 9 in/hr *(Reference 1, FDOT IDF Curves)*

A = basin area [ac]

Down-comer System	Basin	Basin Area (from CAD)	Basin Area	Peak Discharge	Downcomer Total Level 2 ONLY
		(ft^2)	[ac]	[cfs]	[cfs]
A	A-1	49,115	1.13	9.64	
	A-2	88,352	2.03	17.34	17.34
B	B-1	49,171	1.13	9.65	
	B-2	89,963	2.07	17.66	17.66
C	C-1	47,568	1.09	9.34	
	C-2	90,922	2.09	17.85	17.85
D	D-1	23,022	0.53	4.52	
	D-2	107,983	2.48	21.20	21.20
E	E-1	189,246	4.34	37.15	
	E-2	186,104	4.27	36.53	36.53
F	F-1	76,547	1.76	15.02	
	F-2	58,610	1.35	11.50	11.50
G	G-1	83,784	1.92	16.45	
	G-2	59,901	1.38	11.76	11.76
H	H-1	112,745	2.59	22.13	
	H-2	60,369	1.39	11.85	11.85
I	I-1	88,186	2.02	17.31	
	I-2	2,871	0.07	0.56	0.56
J	J-1	82,542	1.89	16.20	
	J-2	50,956	1.17	10.00	10.00
K	K-1	66,192	1.52	12.99	
	K-2	49,577	1.14	9.73	9.73
L	L-1	64,196	1.47	12.60	
	L-2	48,936	1.12	9.61	9.61
M	M-1	13,803	0.32	2.71	
	M-2	127,422	2.93	25.01	25.01
N	N-1	47,047	1.08	9.23	
	N-2	87,481	2.01	17.17	17.17
				Maximum =	36.5
				Minimum =	0.6

Note: Level 1 basins are assumed to drain directly to perimeter swale.

References:

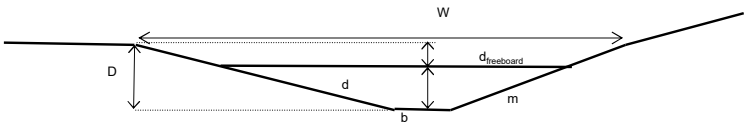
1) FDOT. (2001). Zones for Precipitation IDF Curves Developed by the Department. Drainage Manual - Appendix B - IDF Curves.
2) Lindeburg, M. R. (2008). Civil Engineering Reference Manual for the PE Exam, 11th Ed. Professional Publications, Inc. Belmont, CA.
3) U.S. Dept. of the Interior, Bureau of Reclamation. (2001). Water Measurement Manual. Water Resources Research Laboratory.
4) ADS, Inc. (2007). Drainage Handbook (Hydraulics).

JonesEdmunds

PROJECT NUMBER: 16002-009-01 WORKSHEET: 2 OF 4
PROJECT NAME: Pasco Class I Ash Monofill Modifications
SUBJECT: STORMWATER DESIGN
BY: CSPREADBURY Date: 5/26/2021
CHECKED BY: MMORSE Date: 6/21/2021

Sideslope Terrace Swale Sizing

Data: Sideslope terrace swale design based on Sideslope Terrace Detail Conceptual Final Cover Details



where: W = swale width = 15.0 ft
D = swale depth (critical section) = 2.5 ft
b = swale bottom width = 5.0 ft
$d_{freeboard}$ = freeboard depth = D - d ft
d = flow depth ft
m = channel side slopes = 2 :1 (max)

Note: Critical Sections are the sections that have the maximum flow in the swale and are located adjacent to each drop structure.

Calculation: Determine the depth of freeboard for each terrace swale at the critical section adjacent to the drop structures. Use the Manning Eq. to iteratively solve for the flow depth (d) of the critical swale section to obtain the required peak discharge (Q). Determine if the minimum freeboard depth is greater than 6 inches by subtracting the swale depth from the flow depth (d).

$$Q = \frac{1.49}{n} R^{2/3} A \sqrt{s}$$ *(Reference 2, p 19-4)*

where: Q = flow rate cfs
n = Manning roughness coefficient = 0.03 - *(Reference 2, p A-36)*
R = hydraulic radius = A/P ft *(Average of Natural Channels - Good Condition,*
A = flow area = (b + md) d ft^2 *and Natural Channels w/ stones and weeds)*
P = wetted perimeter = $b+2dV(1+m^2)$ ft
s = slope of swale ft/ft
b, d, m = geometric parameters from data section above

Downcomer System	Basin	Basin Area	Peak Discharge - Level 2 ONLY, Q=CIA	Solver Peak Discharge - Level 2 ONLY, Q	Slope, S_o, 1.00%	Flow Area, A	Wetted Perimeter, P	Hydraulic Radius, R	SOLVER Flow Depth, d	Freeboard Depth, $d_{freeboard}$
		[ac]	[cfs]	[cfs]	[ft/ft]	[ft^2]	[ft]	[ft]	[ft]	[ft]
A	A-1	1.13	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	A-2	2.03	17.3	17.3	1.00%	4.96	8.40	0.59	**0.76**	**1.7**
B	B-1	1.13	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	B-2	2.07	17.7	17.7	1.00%	5.02	8.44	0.60	**0.77**	**1.7**
C	C-1	1.09	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	C-2	2.09	17.8	17.8	1.00%	5.06	8.46	0.60	**0.77**	**1.7**
D	D-1	0.53	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	D-2	2.48	21.2	21.2	1.00%	5.70	8.80	0.65	**0.85**	**1.6**
E	E-1	4.34	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	E-2	4.27	36.5	36.5	1.00%	8.36	10.13	0.83	**1.15**	**1.4**
F	F-1	1.76	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	F-2	1.35	11.5	11.5	1.00%	3.74	7.70	0.49	**0.60**	**1.9**
G	G-1	1.92	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	G-2	1.38	11.8	11.8	1.00%	3.80	7.73	0.49	**0.61**	**1.9**
H	H-1	2.59	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	H-2	1.39	11.8	11.8	1.00%	3.82	7.74	0.49	**0.61**	**1.9**
I	I-1	2.02	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	I-2	0.07	0.6	0.6	1.00%	0.53	5.46	0.10	**0.10**	**2.4**
J	J-1	1.89	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	J-2	1.17	10.0	10.0	1.00%	3.41	7.49	0.45	**0.56**	**1.9**
K	K-1	1.52	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	K-2	1.14	9.7	9.7	1.00%	3.34	7.45	0.45	**0.55**	**2.0**
L	L-1	1.47	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	L-2	1.12	9.6	9.6	1.00%	3.31	7.43	0.45	**0.54**	**2.0**
M	M-1	0.32	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	M-2	2.93	25.0	25.0	1.00%	6.40	9.17	0.70	**0.93**	**1.6**
N	N-1	1.08	0.0	0.0	1.00%	0.00	5.00	0.00	0.00	2.5
	N-2	2.01	17.2	17.2	1.00%	4.93	8.38	0.59	**0.76**	**1.7**

Note: Only Level 2 basin flow was sized for swale.

Maximum Flow Depth = 1.15 ft
Minimum Freeboard = 1.4 ft

JonesEdmunds

PROJECT NUMBER: 16002-009-01 WORKSHEET: 3 OF 4
PROJECT NAME: Pasco Class I Ash Monofill Modifications
SUBJECT: STORMWATER DESIGN
BY: CSPREADBURY Date: 5/26/2021
CHECKED BY: MMORSE Date: 6/21/2021

Drop Inlet Weir Capacity

Calculation: Determine the flow rate through the terrace swale drop inlets using the Orifice Weir Equation.
Compare the ratio of the Weir Flow Rate to the Peak Discharge rate of the 25 yr event to determine the safety factor.
A safety factor greater than 1 is acceptable.

$$Q = C_1 A\sqrt{(2gH)}$$ *(Reference 3, p. 2-6)*

where: Q = flow rate [cfs]

C_1 = weir coefficient	0.61	-	*(Reference 3, p. 2-6)*
A = weir area	12.6	ft^2	
Where: D_{inlet} = Inlet Diameter	4.0	ft	
g = acceleration due to gravity =	32.2	ft/sec^2	
H = total hydraulic head [ft]			

Downcomer System	Basin	Peak Discharge, Q [cfs]	Total Hydraulic Head, H [ft]	Weir Flow Rate, Q_{max} [cfs]	FS_{weir}
A	A-1	-	-	-	-
	A-2	17.3	0.76	53.7	3.1
B	B-1	-	-	-	-
	B-2	17.7	0.77	53.9	3.1
C	C-1	-	-	-	-
	C-2	17.8	0.77	54.1	3.0
D	D-1	-	-	-	-
	D-2	21.2	0.85	56.7	2.7
E	E-1	-	-	-	-
	E-2	36.5	1.15	65.9	1.8
F	F-1	-	-	-	-
	F-2	11.5	0.60	47.8	4.2
G	G-1	-	-	-	-
	G-2	11.8	0.61	48.1	4.1
H	H-1	-	-	-	-
	H-2	11.8	0.61	48.2	4.1
I	I-1	-	-	-	-
	I-2	0.6	0.10	19.7	35.0
J	J-1	-	-	-	-
	J-2	10.0	0.56	45.9	4.6
K	K-1	-	-	-	-
	K-2	9.7	0.55	45.6	4.7
L	L-1	-	-	-	-
	L-2	9.6	0.54	45.4	4.7
M	M-1	-	-	-	-
	M-2	25.0	0.93	59.4	2.4
N	N-1	-	-	-	-
	N-2	17.2	0.76	53.5	3.1

Note: Only Level 2 basin flow was sized for inlet weir.

JonesEdmunds	PROJECT NUMBER: 16002-009-01	WORKSHEET: 4 OF 4
	PROJECT NAME: Pasco Class I Ash Monofill Modifications	
	SUBJECT: STORMWATER DESIGN	
	BY: CSPREADBURY	Date: 5/26/2021
	CHECKED BY: MMORSE	Date: 6/21/2021

Downcomer Pipe Sizing

Calculation: Determine the maximum flow capacity through HDPE downcomer pipe using Manning's Equation.
A Factor of Safety (FS) > 1.0 means the pipe is large enough to handle the maximum flow.

$$Q = \frac{1.49}{n} R^{2/3} A \sqrt{s}$$ *(Reference 2, p 19-4)*

where: Q = flow rate — cfs

n = Manning roughness coefficient = 0.012 - *(Reference 4, Smooth interior)*

R = hydraulic radius = A/P — ft

A = inside cross-section area = $\pi(D_{pipe}/2)^2$ — ft^2

P = wetted perimeter = $\pi(D_{pipe})$ — ft

Where: D_{pipe} = Pipe Diameter — ft

s = slope 0.333 ft/ft (assuming 3H:1V sideslope)

1. Table of pipe sizes (diameters) and corresponding hydraulic parameters.

Diameter	Inside Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Slope, S	Flow, Q_{max}
[in]	[ft^2]	[ft]	[ft]	[ft/ft]	[cfs]
6	0.20	1.57	0.13	0.333	3.5
12	0.79	3.14	0.25	0.333	22.3
15	1.23	3.93	0.31	0.333	40.5
18	1.77	4.71	0.38	0.333	65.9
24	3.14	6.28	0.50	0.333	141.9
30	4.91	7.85	0.63	0.333	257.2
36	7.07	9.4	0.75	0.333	418.3
42	9.6	11.0	0.88	0.333	631.0
48	12.6	12.6	1.00	0.333	900.9

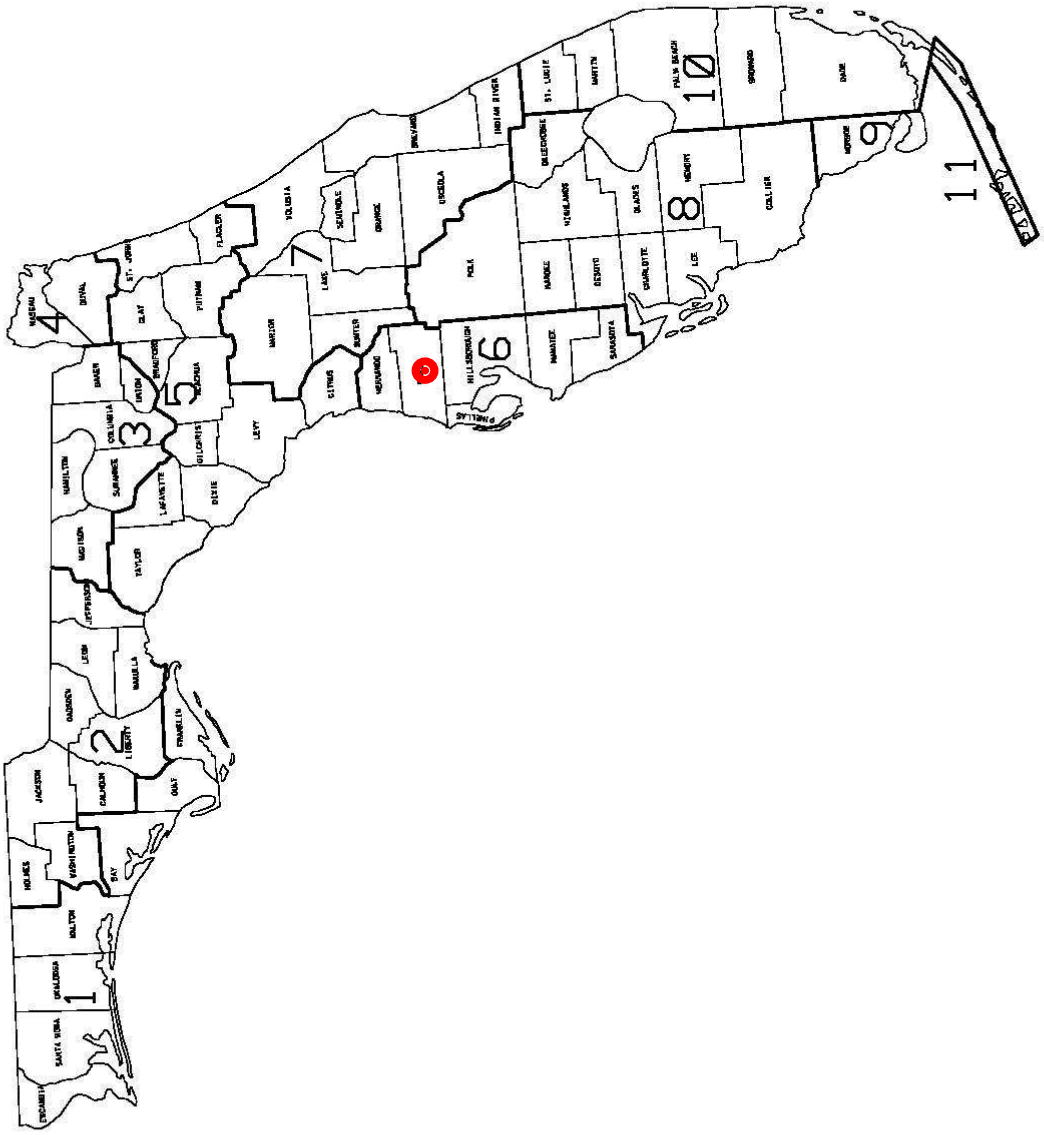
2. Determine pipe size based on capacity and required flow, Q.

Downcomer System	Basin	Peak Discharge 25 yr	Level 2 Downcomer Required Flow, Q_{req}	Pipe Diameter	Pipe Flow, Q_{max}	Q_{req}/Q_{max}	FS_{pipe}
		[cfs]	[cfs]	[in]	[cfs]	[%]	[-]
A	A-1	9.6	-	12	22.3	-	-
	A-2	17.3	17.3	12	22.3	78%	1.3
B	B-1	9.7	-	12	22.3	-	-
	B-2	17.7	17.7	12	22.3	79%	1.3
C	C-1	9.3	-	12	22.3	-	-
	C-2	17.8	17.8	12	22.3	80%	1.3
D	D-1	4.5	-	12	22.3	-	-
	D-2	21.2	21.2	18	65.9	32%	3.1
E	E-1	37.1	-	12	22.3	-	-
	E-2	36.5	36.5	18	65.9	55%	1.8
F	F-1	15.0	-	12	22.3	-	-
	F-2	11.5	11.5	12	22.3	51%	1.9
G	G-1	16.4	-	12	22.3	-	-
	G-2	11.8	11.8	12	22.3	53%	1.9
H	H-1	22.1	-	12	22.3	-	-
	H-2	11.8	11.8	12	22.3	53%	1.9
I	I-1	17.3	-	12	22.3	-	-
	I-2	0.6	0.6	12	22.3	3%	39.7
J	J-1	16.2	-	12	22.3	-	-
	J-2	10.0	10.0	12	22.3	45%	2.2
K	K-1	13.0	-	12	22.3	-	-
	K-2	9.7	9.7	12	22.3	44%	2.3
L	L-1	12.6	-	12	22.3	-	-
	L-2	9.6	9.6	12	22.3	43%	2.3
M	M-1	2.7	-	12	22.3	-	-
	M-2	25.0	25.0	18	65.9	38%	2.6
N	N-1	9.2	-	12	22.3	-	-
	N-2	17.2	17.2	12	22.3	77%	1.3

Note: Only Level 2 basin flow was sized for downcomer pipe. min. FS = 1.3

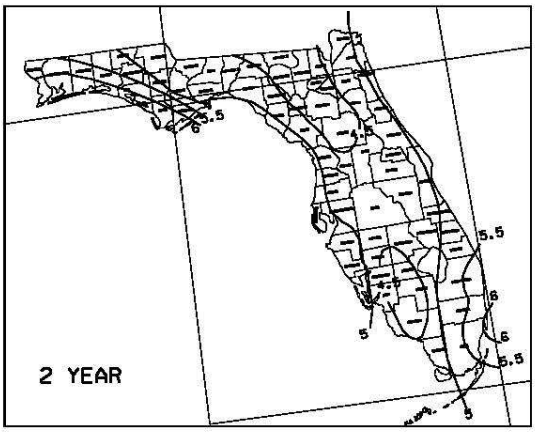
Reference 1

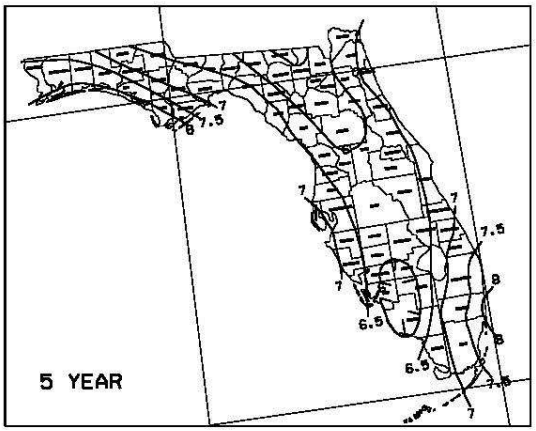
ZONES FOR PRECIPITATION IDF CURVES DEVELOPED BY THE DEPARTMENT

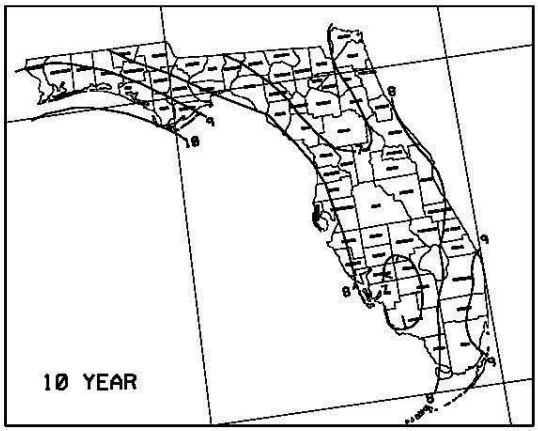


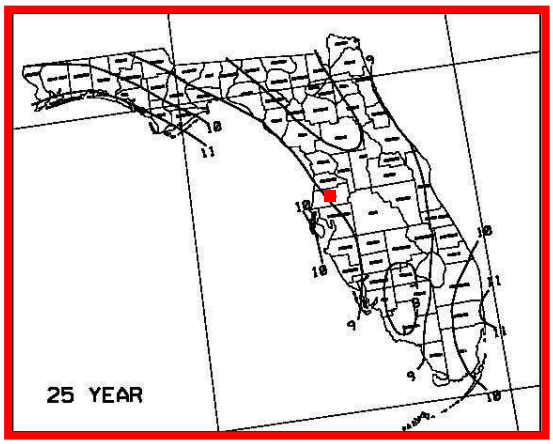
PRECIPITATION DEPTH DATA FOR 2-,5-,10-,25-, 50-, AND 100-YEAR FREQUENCIES

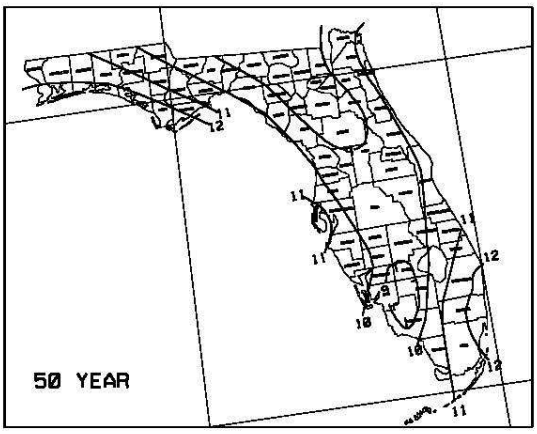
1 DAY

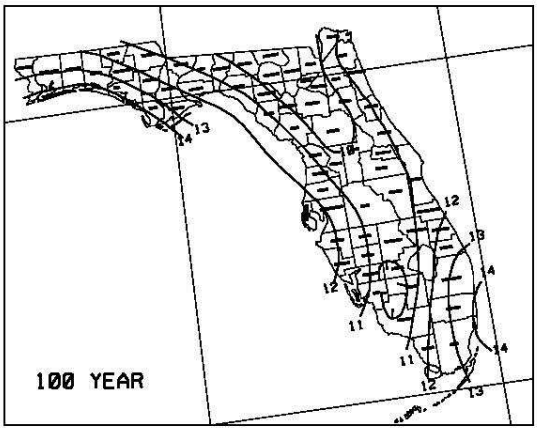












ALL DEPTH CONTOURS IN INCHES

Reference 2

Support Material

Civil Engineering Reference Manual

for the PE Exam

Eleventh Edition

Michael R. Lindeburg, PE

Professional Publications, Inc. • Belmont, CA

6. GOVERNING EQUATIONS FOR UNIFORM FLOW

Since water is incompressible, the continuity equation is

$$A_1 v_1 = A_2 v_2 \qquad 19.8$$

The most common equation used to calculate the flow velocity in open channels is the 1768 *Chezy equation*.[3]

$$v = C\sqrt{RS} \qquad 19.9$$

Various methods for evaluating the *Chezy coefficient*, C, or "Chezy's C," have been proposed.[4] If the channel is small and very smooth, Chezy's own formula can be used. The friction factor, f, is dependent on the Reynolds number and can be found in the usual manner from the Moody diagram.

$$C = \sqrt{\frac{8g}{f}} \qquad 19.10$$

If the channel is large and the flow is fully turbulent, then the friction loss will not depend so much on the Reynolds number as on the channel roughness. The 1888 Manning formula is frequently used to evaluate the constant C.[5] Notice that the value of C depends only on the channel roughness and geometry. (The conversion constant 1.49 in Eq. 19.11(b) is reported as 1.486 by some authorities. 1.486 is the correct SI-to-English conversion, but it is doubtful whether this equation warrants four significant digits.)

$$C = \left(\frac{1.00}{n}\right) R^{1/6} \qquad \text{[SI]} \qquad 19.11(a)$$

$$C = \left(\frac{1.49}{n}\right) R^{1/6} \qquad \text{[U.S.]} \qquad 19.11(b)$$

n is the *Manning roughness coefficient* (*Manning constant*). Typical values of Manning's n are given in App. 19.A. Judgment is needed in selecting values since tabulated values often differ by as much as 30%. More important to recognize for sewer work is the layer of slime that often coats the sewer walls. Since the slime characteristics can change with location in the sewer, there can be variations in Manning's roughness coefficient along the sewer length.

Independent of these factors, the value of n also depends on the depth of flow, leading to a value (n_{full}) specifically intended for use with full flow. (It is seldom clear from tabulations such as App. 19.A whether the values are for full flow or general use.) The variation in n can be taken into consideration using *Camp's correction*, shown in App. 19.C. However, this degree of sophistication cannot be incorporated into an analysis problem unless a specific value of n is known for a specific depth of flow.

Combining Eqs. 19.9 and 19.11 produces the *Manning equation*, also known as the *Chezy-Manning equation*.

$$v = \left(\frac{1.00}{n}\right) R^{2/3}\sqrt{S} \qquad \text{[SI]} \qquad 19.12(a)$$

$$v = \left(\frac{1.49}{n}\right) R^{2/3}\sqrt{S} \qquad \text{[U.S.]} \qquad 19.12(b)$$

All of the coefficients and constants in the Manning equation may be combined into the *conveyance*, K.

$$Q = vA = \left(\frac{1.00}{n}\right) AR^{2/3}\sqrt{S} = K\sqrt{S} \qquad \text{[SI]} \qquad 19.13(a)$$

$$Q = vA = \left(\frac{1.49}{n}\right) AR^{2/3}\sqrt{S} = K\sqrt{S} \qquad \text{[U.S.]} \qquad 19.13(b)$$

Example 19.1

A rectangular channel on a 0.002 slope is constructed of finished concrete. The channel is 8 ft (2.4 m) wide. Water flows at a depth of 5 ft (1.5 m). What is the flow rate?

SI Solution

The hydraulic radius is

$$R = \frac{A}{P} = \frac{(2.4\text{ m})(1.5\text{ m})}{1.5\text{ m} + 2.4\text{ m} + 1.5\text{ m}} = 0.67\text{ m}$$

From App. 19.A, the roughness coefficient for finished concrete is 0.012. The Manning coefficient is determined by Eq. 19.11(a).

$$C = \left(\frac{1.00}{n}\right) R^{1/6} = \left(\frac{1.00}{0.012}\right)(0.67\text{ m})^{1/6} = 77.9$$

The discharge is

$$Q = vA = C\sqrt{RS}A = \left(77.9\,\frac{\sqrt{\text{m}}}{\text{s}}\right)\left(\sqrt{(0.67\text{ m})(0.002)}\right)(1.5\text{ m})(2.4\text{ m}) = 10.3\text{ m}^3/\text{s}$$

Customary U.S. Solution

The hydraulic radius is

$$R = \frac{A}{P} = \frac{(8\text{ ft})(5\text{ ft})}{5\text{ ft} + 8\text{ ft} + 5\text{ ft}} = 2.22\text{ ft}$$

[3] Pronounced "Shay'-zee." This equation does not appear to be dimensionally consistent. However, the coefficient C is not a pure number. Rather, it has units of (length)$^{1/2}$/time (i.e., (acceleration)$^{1/2}$).

[4] Other methods of evaluating C include the *Kutter equation* (also known as the *G.K. formula*) and the *Bazin formula*. These methods are interesting from a historical viewpoint, but both have been replaced by the Manning equation.

[5] This equation was originally proposed in 1868 by Gaukler and again in 1881 by Hagen, both working independently. For some reason, the Frenchman Flamant attributed the equation to an Irishman, R. Manning. In Europe and many other places, the Manning equation may be known as the *Strickler equation*.

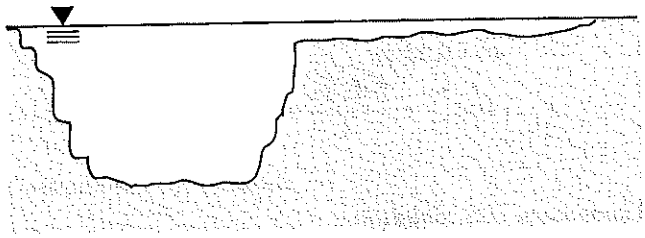
13. ANALYSIS OF NATURAL WATERCOURSES

Natural watercourses do not have uniform paths or cross sections. This complicates their analysis considerably. Frequently, analyzing the flow from a river is a matter of making the most logical assumptions. Many evaluations can be solved with a reasonable amount of error.

As was seen in Eq. 19.30, the friction loss (and hence the hydraulic gradient) depends on the square of the roughness coefficient. Therefore, an attempt must be made to evaluate the roughness constant as accurately as possible. If the channel consists of a river with flood plains, it should be treated as parallel channels. The flow from each subdivision can be calculated independently and the separate values added to obtain the total flow. (The common interface between adjacent subdivisions is not included in the wetted perimeter.) Alternatively, a composite value of the roughness coefficient, n_c, can be approximated from the individual values of n and the corresponding wetted perimeters.

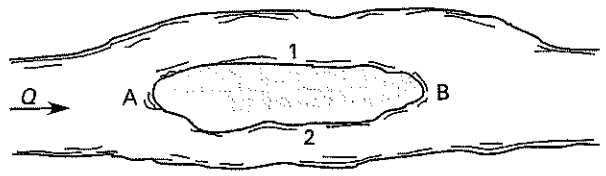
$$n_c = \left(\frac{\sum P_i (n_i)^{3/2}}{\sum P_i}\right)^{2/3} \qquad 19.43$$

Figure 19.4 *River with Flood Plain*



If the channel is divided by an island into two channels, some combination of flows will usually be known. For example, if the total flow, Q, is known, Q_1 and Q_2 may be unknown. If the slope is known, Q_1 and Q_2 may be known. Iterative trial-and-error solutions are often required.

Figure 19.5 *Divided Channel*



Since the elevation drop $(z_A - z_B)$ between points A and B is the same regardless of flow path,

$$S_1 = \frac{z_A - z_B}{L_1} \qquad 19.44$$

$$S_2 = \frac{z_A - z_B}{L_2} \qquad 19.45$$

Once the slopes are known, initial estimates Q_1 and Q_2 can be calculated from Eq. 19.13. The sum of Q_1 and Q_2 will probably not be the same as the given flow quantity, Q. In that case, Q should be prorated according to the ratios of Q_1 and Q_2 to $Q_1 + Q_2$.

If the lengths L_1 and L_2 are the same or almost so, the Manning equation may be solved for the slope by writing Eq. 19.46.

$$Q = Q_1 + Q_2 = \left(\left(\frac{A_1}{n_1}\right)(R_1)^{2/3} + \left(\frac{A_2}{n_2}\right)(R_2)^{2/3}\right)\sqrt{S} \qquad \text{[SI]} \quad 19.46(a)$$

$$Q = Q_1 + Q_2 = 1.49\left(\left(\frac{A_1}{n_1}\right)(R_1)^{2/3} + \left(\frac{A_2}{n_2}\right)(R_2)^{2/3}\right)\sqrt{S} \qquad \text{[U.S.]} \quad 19.46(b)$$

Equation 19.46 yields only a rough estimate of the flow quantity, as the geometry and roughness of a natural channel changes considerably along its course.

14. FLOW MEASUREMENT WITH WEIRS

A *weir* is an obstruction in an open channel over which flow occurs. Although a dam spillway is a specific type of weir, most weirs are intended specifically for flow measurement.

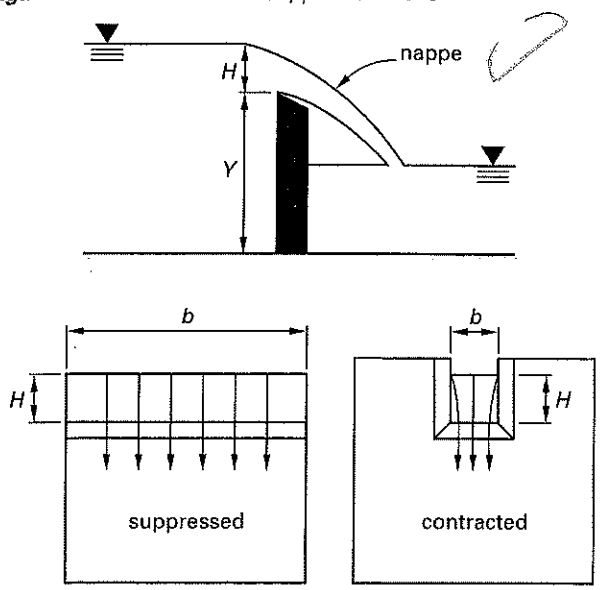
Measurement weirs consist of a vertical flat plate with sharp edges. Because of their construction, they are called *sharp-crested weirs*. Sharp-crested weirs are most frequently rectangular, consisting of a straight, horizontal crest. However, weirs may also have trapezoidal and triangular openings.

For any given width of weir opening (referred to as the *weir length*), the discharge will be a function of the head over the weir. The head (or sometimes surface elevation) can be determined by a standard *staff gauge* mounted adjacent to the weir.

The full channel flow usually goes over the weir. However, it is also possible to divert a small portion of the total flow through a measurement channel. The full channel flow rate can be extrapolated from a knowledge of the split fractions.

If a rectangular weir is constructed with an opening width less than the channel width, the falling liquid sheet (called the *nappe*) decreases in width as it falls. Because of this *contraction* of the nappe, these weirs are known as *contracted weirs*, although it is the nappe that is actually contracted, not the weir. If the opening of the weir extends the full channel width, the weir is known as a *suppressed weir*, since the contractions are suppressed.

Figure 19.6 *Contracted and Suppressed Weirs*



The derivation of an expression for the quantity flowing over a weir is dependent on many simplifying assumptions. The basic weir equation (Eq. 19.47 or 19.48) is, therefore, an approximate result requiring correction by experimental coefficients.

If it is assumed that the contractions are suppressed, upstream velocity is uniform, flow is laminar over the crest, nappe pressure is zero, the nappe is fully ventilated, and viscosity, turbulence, and surface tension effects are negligible, then the following equation may be derived from the Bernoulli equation.

$$Q = \tfrac{2}{3} b\sqrt{2g}\left(\left(H + \frac{v_1^2}{2g}\right)^{3/2} - \left(\frac{v_1^2}{2g}\right)^{3/2}\right) \qquad 19.47$$

If the velocity of approach, v_1, is negligible, then

$$Q = \tfrac{2}{3} b\sqrt{2g}H^{3/2} \qquad 19.48$$

Equation 19.48 must be corrected for all of the assumptions made, primarily for a nonuniform velocity distribution. This is done by introducing an empirical discharge coefficient, C_1. Equation 19.49 is known as the *Francis weir equation.*

$$Q = \tfrac{2}{3} C_1 b\sqrt{2g}H^{3/2} \qquad 19.49$$

Many investigations have been done to evaluate C_1 analytically. Perhaps the most widely known is the coefficient formula developed by *Rehbock.*[7]

[7]There is much variation in how different investigators calculate the discharge coefficient, C_1. For ratios of H/b less than 5, $C_1 = 0.622$ gives a reasonable value. With the questionable accuracy of some of the other variables used in open channel flow problems, the pursuit of greater accuracy is of dubious value.

$$C_1 = \left(0.6035 + 0.0813\left(\frac{H}{Y}\right) + \frac{0.000295}{Y}\right) \times \left(1 + \frac{0.00361}{H}\right)^{3/2} \quad \text{[U.S. only]}$$

$$\approx 0.602 + 0.083\left(\frac{H}{Y}\right) \quad \text{[U.S. and SI]} \qquad 19.50$$

When $H/Y < 0.2$, C_1 approaches 0.61 to 0.62. In most cases, a value in this range is adequate. Other constants (i.e., $^2/_3$ and $\sqrt{2g}$, can be taken out of Eq. 19.49. In that case,

$$Q \approx 1.84bh^{3/2} \quad \text{[SI]} \qquad 19.51(a)$$

$$Q \approx 3.33bh^{3/2} \quad \text{[U.S.]} \qquad 19.51(b)$$

If the contractions are not suppressed (i.e., one or both sides do not extend to the channel sides) then the actual width, b, should be replaced with the *effective width.* In Eq. 19.52, N is 1 if one side is contracted and N is 2 if there are two end contractions.

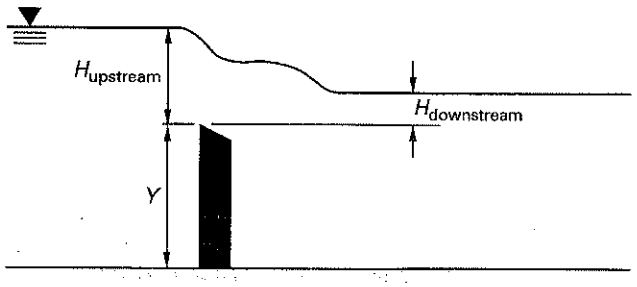
$$b_{\text{effective}} = b_{\text{actual}} - 0.1NH \qquad 19.52$$

A *submerged rectangular weir* requires a more complex analysis because of the difficulty in measuring H and because the discharge depends on both the upstream and downstream depths. The following equation, however, may be used with little difficulty.

$$Q_{\text{submerged}} = Q_{\text{free flow}}\left(1 - \left(\frac{H_{\text{downstream}}}{H_{\text{upstream}}}\right)^{3/2}\right)^{0.385} \qquad 19.53$$

Equation 19.53 is used by first finding the flow rate, Q, from Eq. 19.49 and then correcting it with the bracketed quantity.

Figure 19.7 *Submerged Weir*



Example 19.6

The crest of a sharp-crested, rectangular weir with two contractions is 2.5 ft (1.0 m) high above the channel bottom. The crest is 4 ft (1.6 m) long. A 4 in (100 mm) head exists over the weir. What is the velocity of approach?

14. HYDROGRAPH SYNTHESIS

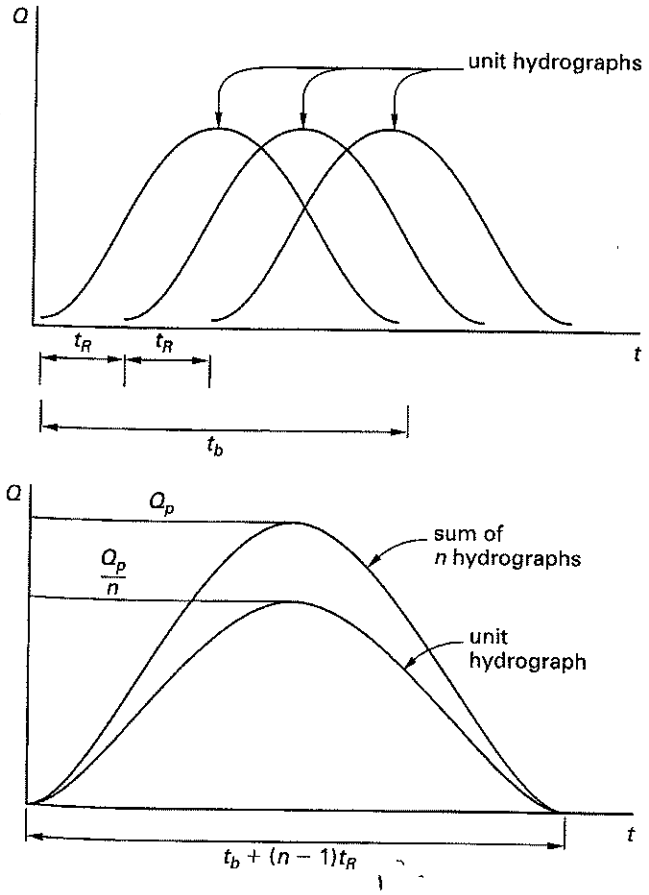
If a storm's duration is not the same or close to the hydrograph base length, the unit hydrograph cannot be used to predict runoff. For example, the runoff from a 6 hr storm cannot be predicted from a unit hydrograph derived from a 2 hr storm. However, the technique of hydrograph synthesis can be used to construct the hydrograph of the longer storm from the unit hydrograph of a shorter storm.

A. Lagging Storm Method

If a unit hydrograph for a storm of duration t_R is available, the *lagging storm method* can be used to construct the hydrograph of a storm whose duration is a whole multiple of t_R. For example, a 6 hr storm hydrograph can be constructed from a 2 hr unit hydrograph.

Let the whole multiple number be n. To construct the longer hydrograph, draw n unit hydrographs, each separated by time t_R. Then add the ordinates to obtain a hydrograph for an nt_R duration storm. Since the total rainfall from this new hydrograph is n inches (having been constructed from n unit hydrographs), the curve will have to be reduced (i.e., divided) by n everywhere to produce a unit hydrograph.

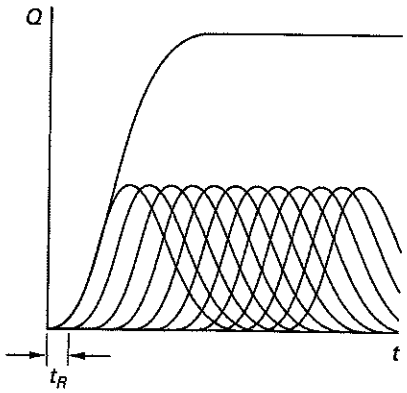
Figure 20.17 *Lagging Storm Method*



B. S-Curve Method

The *S-curve method* can be used to construct hydrographs from unit hydrographs with longer or shorter durations, even when the storm durations are not multiples. This method begins by adding the ordinates of many unit hydrographs, each lagging the other by time t_R, the duration of the storm that produced the unit hydrograph. After a sufficient number of lagging unit hydrographs have been added together, the accumulation will level off and remain constant. At that point, the lagging can be stopped. The resulting accumulation is known as an S-curve.

Figure 20.18 *Constructing the S-Curve*



If two S-curves are drawn, one lagging the other by time $t_{R'}$, the area between the two curves represents a hydrograph area for a storm of duration t'_R. The differences between the two curves can be plotted and scaled to a unit hydrograph by multiplying by the ratio of (t_R/t'_R).

15. PEAK RUNOFF FROM THE RATIONAL METHOD

Although total runoff volume is required for reservoir and dam design, the instantaneous peak runoff is needed to size culverts and storm drains.

The *rational formula* ("method," "equation," etc.), Eq. 20.36, for peak discharge has been in widespread use in the United States since the early 1900s.[3] It is applicable to small areas (i.e., less than several hundred acres or so), but is seldom used for areas greater than 1 to 2 mi^2. The intensity used in Eq. 20.36 depends on the time of concentration and the degree of protection desired (i.e., the recurrence interval).[4]

$$Q_p = CIA_d \qquad 20.36$$

[3] In Great Britain, the rational equation is known as the *Lloyd-Davies equation.*

[4] When using intensity-duration-frequency curves to size storm sewers, culverts, and other channels, it is assumed that the frequencies and probabilities of flood damage and storms are identical. This is not generally true, but the assumption is usually made anyway.

APPENDIX 19.A

Manning's Roughness Coefficient[a,b]
(design use)

channel material	n
plastic (PVC and ABS)	0.009
clean, uncoated cast iron	0.013–0.015
clean, coated cast iron	0.012–0.014
dirty, tuberculated cast iron	0.015–0.035
riveted steel	0.015–0.017
lock-bar and welded steel pipe	0.012–0.013
galvanized iron	0.015–0.017
brass and glass	0.009–0.013
wood stave	
small diameter	0.011–0.012
large diameter	0.012–0.013
concrete	
average value used	0.013
typical commercial, ball and spigot rubber gasketed end connections	
– full (pressurized and wet)	0.010
– partially full	0.0085
with rough joints	0.016–0.017
dry mix, rough forms	0.015–0.016
wet mix, steel forms	0.012–0.014
very smooth, finished	0.011–0.012
vitrified sewer	0.013–0.015
common-clay drainage tile	0.012–0.014
asbestos	0.011
planed timber (flume)	0.012 (0.010–0.014)
canvas	0.012
unplaned timber (flume)	0.013 (0.011–0.015)
brick	0.016
rubble masonry	0.017
smooth earth	0.018
firm gravel	0.023
corrugated metal pipe (CMP)	0.024 (see App. 17.F)
natural channels, good condition	0.025
rip rap	0.035
natural channels with stones and weeds	0.035
very poor natural channels	0.060

[a] Compiled from various sources.
[b] Values outside these ranges have been observed, but these values are typical.

Average of Natural Channels, Good Condition and Natural Channels with Stone and Weeds = 0.030

APPENDIX 20.A
Rational Method Runoff C-Coefficients

categorized by surface	
forested	0.059–0.2
asphalt	0.7–0.95
brick	0.7–0.85
concrete	0.8–0.95
shingle roof	0.75–0.95
lawns, well-drained (sandy soil)	
up to 2% slope	0.05–0.1
2% to 7% slope	0.10–0.15
over 7% slope	0.15–0.2
lawns, poor drainage (clay soil)	
up to 2% slope	0.13–0.17
2% to 7% slope	0.18–0.22
over 7% slope	0.25–0.35
driveways, walkways	0.75–0.85
categorized by use	
farmland	0.05–0.3
pasture	0.05–0.3
unimproved	0.1–0.3
parks	0.1–0.25
cemeteries	0.1–0.25
railroad yards	0.2–0.35
playgrounds (except asphalt or concrete)	0.2–0.35
business districts	
neighborhood	0.5–0.7
city (downtown)	0.7–0.95
residential	
single family	0.3–0.5
multiplexes, detached	0.4–0.6
multiplexes, attached	0.6–0.75
suburban	0.25–0.4
apartments, condominiums	0.5–0.7
industrial	
light	0.5–0.8
heavy	0.6–0.9

Assumed worst-case scenario of Asphalt/Concrete: 0.95

Reference 3

Water Resources
Research Laboratory

U. S. Department of the Interior
Bureau of Reclamation

WATER MEASUREMENT MANUAL

A WATER RESOURCES TECHNICAL PUBLICATION

Water Measurement web page

A guide to effective water measurement practices for better water management

U.S. Department of the Interior

Bureau of Reclamation

In cooperation with

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As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering sound use of our land and water resources; protecting our fish, wildlife, and biological diversity; preserving the environmental and cultural values of our national parks and historical places; and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to ensure that their development is in the best interests of all our people by encouraging stewardship and citizen participation in their care. The Department also has a major responsibility for American Indian reservation communities and for people who live in island territories under U.S. Administration.

CHAPTER 2 - BASIC CONCEPTS RELATED TO FLOWING WATER AND MEASUREMENT

1. Introduction

Experiences with the Bureau of Reclamation's Water Management Workshops, held each year in Denver, Colorado, have indicated a need to explain fundamental concepts of flowing water and its measurement. The workshops have also demonstrated the need to present concepts in simple terms using step-by-step development (Schuster, 1970). Because of more recent water measurement developments and the new chapters and sections added to this edition, this chapter has expanded the previous edition's appendix material into a more complete form. Thus, many more equations are included to maintain step-by-step development of the new material. Readers who have difficulties with algebra or the technical writing level should skim the text to provide exposure to concepts and terminology related to water measurement. More experienced water providers and users can use this chapter as a quick review of hydraulic principles related to water measurement.

Eventually, operators may wish to further investigate and seek more advanced references in hydraulics and fluid mechanics. Streeter (1951) has a chapter on flow measurement that covers tube-type flow meters. Bean (1971) has full information on fluid meter theory and provides detailed material for determining coefficients for tube-type meters. King and Brater (1963) have a thorough discussion of general critical depth relations and detailed relationships for most common hydraulic flow section shapes. Bos (1989) covers the entire field of open channel water measurement devices.

2. Kinds of Flow

Flow is classified into open channel flow and closed conduit flow. Open channel flow conditions occur whenever the flowing stream has a free or unconstrained surface that is open to the atmosphere. Flows in canals or in vented pipelines which are not flowing full are typical examples. The presence of the free water surface prevents transmission of pressure from one end of the conveyance channel to another as in fully flowing pipelines. Thus, in open channels, the only force that can cause flow is the force of gravity on the fluid. As a result, with steady uniform flow under free discharge conditions, a progressive fall or decrease in the water surface elevation always occurs as the flow moves downstream.

In hydraulics, a pipe is any closed conduit that carries water under pressure. The filled conduit may be square, rectangular, or any other shape, but is usually round. If flow is occurring in a conduit but does not completely fill it, the flow is not considered pipe or closed conduit flow, but is classified as open channel flow.

Flow occurs in a pipeline when a pressure or head difference exists between ends. The rate or discharge that occurs depends mainly upon (1) the amount of pressure or head difference that exists from the inlet to the outlet; (2) the friction or resistance to flow caused by pipe length, pipe roughness, bends, restrictions, changes in conduit shape and size, and the nature of the fluid flowing; and (3) the cross-sectional area of the pipe.

3. Basic Principles of Water Measurement

Most devices measure flow indirectly. Flow measuring devices are commonly classified into those that sense or measure velocity and those that measure pressure or head. The head or velocity is measured, and then charts, tables, or equations are used to obtain the discharge. Some water measuring devices that use measurement of head, h, or pressure, p, to determine discharge, Q, are:

(1) Weirs
(2) Flumes
(3) Orifices
(4) Venturi meters
(5) Runup measurement on a flat "weir stick"

Head, h, or depth commonly is used for the open channel devices such as flumes and weirs. Either pressure, p, or head, h, is used with tube-type flowmeters such as a venturi.

Pressure, p, is the force per unit area as shown on figure 2-1 that acts in every direction normal to containing or submerged object boundaries. If an open vertical tube is inserted through and flush with the wall of a pipe under pressure, water will rise to a height, h, until the weight, W, of water in the tube balances the pressure force, F_p, on the wall opening area, a, at the wall connection. These tubes are called piezometers. The volume of water in the piezometer tube is designated ha. The volume times the unit weight of water, γha, is the weight, W.

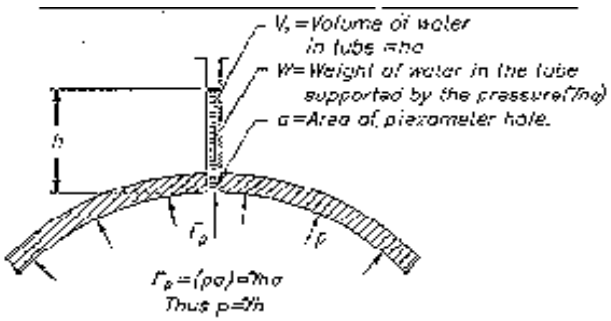

Figure 2-1. – Pressure definition

The pressure force, F_p, on the tap connection area is designated *pa*. The weight and pressure force are equal, and dividing both by the area, *a*, gives the unit pressure on the wall of the pipe in terms of head, *h*, written as:

$$p = \gamma h \quad (2\text{-}1)$$

or:

$$h = \frac{p}{\gamma} \quad (2\text{-}2)$$

Thus, head is pressure, *p*, divided by unit weight of water, γ, or 62.4 pounds per cubic foot (lb/ft^3). Pressure is often expressed in **psi** or pounds per square inch (lb/in^2), which may be converted to feet of water by multiplying the (lb/in^2) value by 2.31. For example, 30 lb/in^2 is produced by 69.3 feet of water. When the head principle is used, the discharge, *Q*, is computed from an equation such as the one used for a sharp-crested rectangular weir of length, *L*:

$$Q = CLh^{3/2} \quad (2\text{-}3)$$

A coefficient, *C*, is included that accounts for simplifying assumptions and other deficiencies in deriving the equation. The coefficient can vary widely in nonstandard installations, but is well defined for standard installations or is constant over a specified range of discharge.

The flow cross-sectional area, *A*, does not appear directly in the equation, but an area can be extracted by rewriting this equation:

$$Q = CLhh^{1/2} \quad (2\text{-}4)$$

in which:

$$A = Lh \quad (2\text{-}5)$$

In this form, *C* also contains a hidden square root of 2*g*, which, when multiplied by $(h)^{1/2}$, is the theoretical velocity. This velocity does not need to be directly measured or sensed. Because the weir equation computes velocity from a measuring head, a weir is classified as a head measuring device.

Some devices that actually sample or sense velocities, *v*, are:

(1) Float and stopwatch
(2) Current and propeller meters
(3) Vane deflection meters

These devices generally do not measure the average velocity, *V*, for an entire flow cross section. Thus, the relationship between sampled velocities, *v*, and the mean velocity, *V*, must be known as well as the flow section area, *A*, to which the mean velocity applies. Then, the discharge, *Q*, sometimes called the flow rate, is the product, *AV*.

Discharge or rate of flow has units of volume divided by unit time. Thus, discharge can be accurately determined by measuring the time, t, to fill a known volume, V_o:

$$Q = \frac{V_o}{t} \quad (2\text{-}6)$$

Water measurement devices can be calibrated using very accurate volumetric tanks and clocks. More commonly, weight of water in the tanks is used by converting the weight of water per unit volume. The weight of water per cubic foot, called unit weight or specific weight, γ, is 62.4 lb/ft^3 at standard atmospheric conditions.

4. Discharge-Area-Velocity Relationships

Flow rate or discharge, Q, is the volume of water in cubic feet passing a flow section per unit time, usually measured in cubic feet per second (ft^3/s). The distance, d_v, in feet that water will travel at a given velocity in a pipe of constant diameter is velocity, V, in feet per second (ft/s) multiplied by time, t, in seconds, or:

$$d_v = Vt \quad (2\text{-}7)$$

The volume, V_o, in cubic feet passing from the upstream to the downstream ends of this distance is the distance, d_v, in feet times area, A, in square feet of the flow section. Thus:

$$V_o = d_v A = AVt \quad (2\text{-}8)$$

To get the time rate of flow or discharge, Q, in cubic feet per second, divide the right and left sides of equation 2-8 by time, t, in seconds, resulting in:

$$Q = AV \quad (2\text{-}9)$$

Flow in open channels of rectangular cross section is often expressed in terms of unit discharge, q, in cubic feet per second per foot of width which is discharge, Q, in cubic feet per second divided by cross-sectional width, L_b, in feet or:

$$q = \frac{Q}{L_b} = \frac{VA}{L_b} = VD \quad (2\text{-}10)$$

The area, A, is L_bD, where D is the depth of flow. The continuity concept is an important extension of equation 2-9. On the basis that water is incompressible and none is lost from a flowing system, then as the cross-sectional area changes, the velocity must adjust itself such that the values of Q or VA are constant:

$$Q = A_1V_1 = A_2V_2 = \ldots\ldots = A_nV_n \quad (2\text{-}11)$$

where the subscript denotes any number of arbitrarily selected positions along the flowing system. This principle, known as continuity, is especially useful in the analysis of tube flow measurement devices such as the venturi meter.

5. Flow Totalization

Water is sold and measured in terms of total volume consumed, say cubic feet, over some convenient time period, perhaps for billing each month. Many flowmeters have built in capability to sum or totalize volume continually. Thus, the volume consumed is obtained by taking the difference of two sequential monthly readings. To aid irrigation operation and management, most meters provide instantaneous rate of flow or discharge displayed in units such as cubic feet per second. These flow rates are used to set flow and predict the volume of water that will be consumed for intervals of time after flow setting.

6. Other Examples of Velocity Flow Measurement Devices

Measuring devices not previously mentioned are dilution in the concentration of tracers, such as salts and dyes; acoustic or magnetic meters; pitot tubes; rotameters, which are tapered tubes with suspended flow indicators; and many others that are not commonly used. In the dilution method, discharge is calculated by determining the quantity of water necessary to dilute a known quantity of concentrated chemical or dye solution. Chemical analysis or color comparison is used to determine the degree of dilution of the injected or mixed samples. In transit time acoustic meters, the velocity of sound pulses in the direction of flow is compared to the velocity of sound pulses opposite to the direction of flow to determine the mean velocity and, thus, discharge. With Doppler acoustic meters, sound pulses are reflected from moving particles within the water mass, similar to radar. In the magnetic meter, the flowing water acts like a moving electrical conductor passing through a magnetic field to produce a voltage that is proportional to discharge. Pitot tubes relate velocity head, $V^2/2g$, to discharge.

7. Velocity Head Concept

A dropped rock or other object will gain speed rapidly as it falls. Measurements show that an object dropping 1 foot (ft) will reach a velocity of 8.02 feet per second (ft/s). An object dropping 4 ft will reach a velocity of 16.04 ft/s. After an 8ft drop, the velocity attained is 22.70 ft/s. This gain in speed or acceleration is caused by the force of gravity, which is equal to 32.2 feet per second per second (ft/s^2). This acceleration caused by gravity is referred to as g.

If water is stored in a tank and a small opening is made in the tank wall 1 ft below the water surface, the water will spout from the opening with a velocity of 8.02 ft/s. This velocity has the same magnitude that a freely falling rock attains after falling 1 ft. Similarly, at openings 4 ft and 8 ft below the water surface, the velocity of the spouting water will be 16.04 and 22.68 ft/s, respectively. Thus, the velocity of water leaving an opening under a given head, h, is the same as the velocity that would be attained by a body falling that same distance.

The equation that shows how velocity changes with h and defines velocity head is:

$$V = \sqrt{2gh} \quad (2\text{-}12)$$

which may also be written in velocity head form as:

$$h = \frac{V^2}{2g} \quad (2\text{-}13)$$

8. Orifice Relationships

Equations 2-9 and 2-13 can be used to develop an equation for flow through an orifice, which is a sharp-edged hole in the side or bottom of a container of water (figure 2-2a). To find the velocity of flow in the orifice, use equation 2-13, then multiply by area to get AV, or discharge, Q, resulting in:

$$Q_t = A\sqrt{2gh} \quad (2\text{-}14)$$

The subscript t denotes theoretical discharge through an orifice. This equation assumes that the water is frictionless and is an ideal fluid. A correction must be made because water is not an ideal fluid. Most of the approaching flow has to curve toward the orifice opening. The water, after passing through the orifice, continues to contract or curve from the sharp orifice edge. If the orifice edges are sharp, the jet will appear as shown on figure 2-2. The maximum jet contraction occurs at a distance of one-half the orifice diameter (d/2) downstream from the sharp edge. The cross-sectional area of the jet is about six-tenths of the area of the orifice. Thus, equation 2-14 must be corrected using a contraction coefficient, C_c, to produce the actual discharge of water being delivered. Thus, the actual discharge equation is written as:

$$Q_a = C_c A\sqrt{2gh} \quad (2\text{-}15)$$

For a sharp-edged rectangular slot orifice where full contraction occurs, the contraction coefficient is about 0.61, and the equation becomes:

$$Q_a = 0.61A\sqrt{2gh} \quad (2\text{-}16)$$

A nonstandard installation will require further calibration tests to establish the proper contraction coefficient because the coefficient actually varies with the proximity to the orifice edge with respect to the approach and exit boundaries and approach velocity.

9. Thin Plate Weir Relationships

Most investigators derive the equation for sharp-crested rectangular weirs by mathematical integration of elemental orifice strips over the nappe (Bos, 1989). Each strip is considered an orifice with a different head on it.

The resulting rectangular weir equation for theoretical discharge is:

$$Q_t = \frac{2}{3}(2g)^{1/2} L_b h^{3/2} \quad (2\text{-}17)$$

A correction factor is needed to account for simplifications and assumptions. Thus, a discharge coefficient, C_d, is added to obtain actual discharge, expressed as:

$$Q_a = C_d \frac{2}{3} \sqrt{2g}\, L_b h^{3/2} \quad (2\text{-}18)$$

This relationship is the basic weir equation and can be modified to account for weir blade shape and approach velocity. However, C_d must be determined by analysis and calibration tests. For standard weirs, C_d is well defined or constant for measuring within specified head ranges.

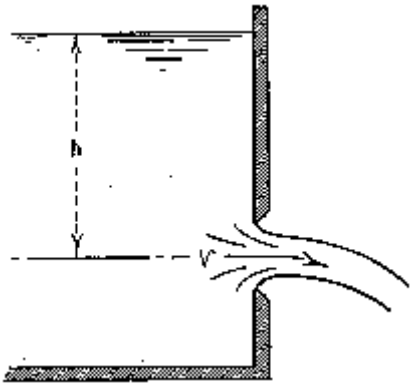

Figure 2-2a -- Orifice flow.

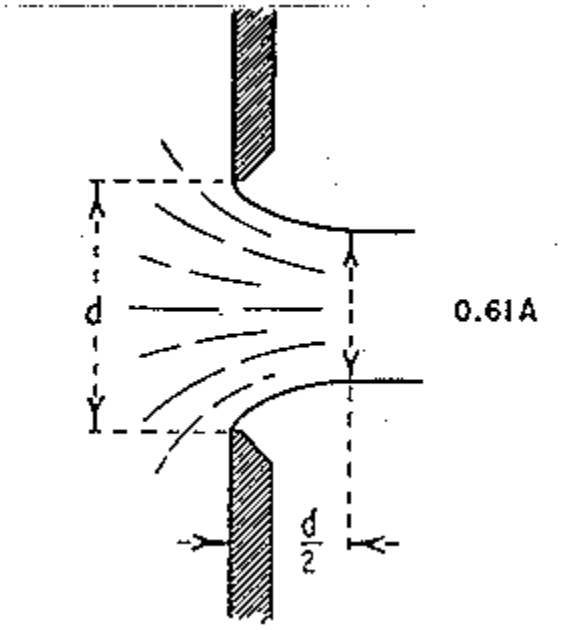

Figure 2-2b -- Contraction at an orifice.

10. Energy Balance Flow Relationships

Hydraulic problems concerning fluid flow are generally handled by accounting in terms of energy per pound of flowing water. Energy measured in this form has units of feet of water. The total amount of energy is that caused by motion, or velocity head, $V^2/2g$, which has units of feet, plus the potential energy head, Z, in feet, caused by elevation referenced to an arbitrary datum selected as reference zero elevation, plus the pressure energy head, h, in feet. The head, h, is depth of flow for the open channel flow case and p/γ defined by equation 2-2 for the closed conduit case. This summation of energy is shown for three cases on figure 2-3.

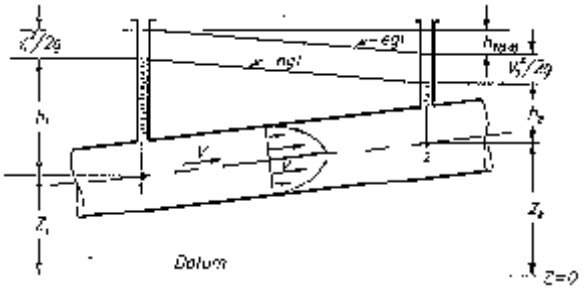

Figure 2-3a -- Energy balance in pipe flow.

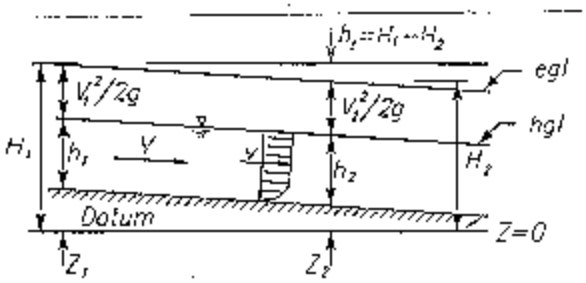

Figure 2-3b -- Energy balance in open channel flow.

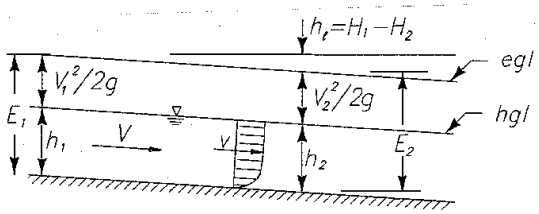

Figure 2-3c -- Specific energy balance.

Figures 2-3a and 2-3b show the total energy head, H_1; for example, at point 1, in a pipe and an open channel, which can be written as:

$$H_1 = h_1 + \frac{V_1^2}{2g} + Z_1 \quad (2\text{-}19)$$

At another downstream location, point 2:

$$H_2 = h_2 + \frac{V_2^2}{2g} + Z_2 \quad (2\text{-}20)$$

Energy has been lost because of friction between points 1 and 2, so the downstream point 2 has less energy than point 1. The energy balance is retained by adding a head loss, $h_{f(1\text{-}2)}$. The total energy balance is written as:

$$h_1 = \frac{V_1^2}{2g} + Z_1 = h_2 + \frac{V_2^2}{2g} + Z_2 + h_{f(1\text{-}2)} \quad (2\text{-}21)$$

The upper sloping line drawn between the total head elevations is the energy gradeline, *egl*. The next lower sloping solid line for both the pipe and open channel cases shown on figure 2-3 is the hydraulic grade line, *hgl*, which is also the water surface for open channel flow, or the height to which water would rise in piezometer taps for pipe flow.

A special energy form is commonly used in hydraulics in which the channel invert is selected as the reference *Z* elevation (figure 2-3c). Thus, *Z* drops out, and energy is the sum of depth, *h*, and velocity head only. Energy above the invert expressed this way is called specific energy, *E*. This simplified form of energy equation is written as:

$$\textit{Specific energy} = E = \frac{V^2}{2g} + h \quad (2\text{-}22)$$

Equations 2-21 and 2-11 lead to several interesting conclusions. In a fairly short pipe that has little or insignificant friction loss, total energy at one point is essentially equal to the total energy at another point. If the size of the pipeline decreases from the first point to the second, the velocity of flow must increase from the first point to the second. This increase occurs because with steady flow, the quantity of flow passing any point in the completely filled pipeline remains the same. From the continuity equation (equation 2-11), when the flow area decreases, the flow velocity must increase.

The second interesting point is that when the velocity increases in the smaller section of the pipeline, the pressure head, *h*, decreases. At first, this decrease may seem strange, but equation 2-21 shows that when $V^2/2g$ increases, *h* must decrease proportionately because the total energy from one point to another in the system remains constant, neglecting friction loss. The fact that the pressure does decrease when the velocity in a given system increases is the basis for tube-type flow measuring devices.

In open channel flow where the flow accelerates, more of its supply of energy becomes velocity head, and depth must decrease. On the other hand, when the flow slows down, the depth must increase.

An example of accelerating flow with corresponding decreasing depth is found at the approach to weirs. The drop in the water surface is called drawdown. Another example occurs at the entrance to inverted siphons or conduits where the flow accelerates as it passes from the canal, through a

contracting transition, and into the siphon barrel. An example of decelerating flow with a rising water surface is found at the outlet of an inverted siphon, where the water loses velocity as it expands in a transition back into canal flow.

Flumes are excellent examples of measuring devices that take advantage of the fact that changes in depth occur with changes in velocity. When water enters a flume, it accelerates in a converging section. The acceleration of the flow causes the water surface to drop a significant amount. This change in depth is directly related to the rate of flow.

11. Hydraulic Mean Depth and Hydraulic Radius

Figure 2-4 shows an irregular flow cross section with different methods for defining depth of flow. In terms of frictional head losses, the perimeter is important. Hydraulic radius, R_h, is defined as the area of the flow section divided by the wetted perimeter, P_w, which is shown on figure 2-4 and is written as:

$$R_h = \frac{A}{P_w} \quad (2\text{-}23)$$

Thus, wetted perimeter times the hydraulic radius is equal to the area of irregular section flow as shown on figures 2-4a and 2-4c.

For use in Froude number and energy relationships in open channel flow hydraulics, mean depth, h_m, is defined as the depth which, when multiplied by the top water surface width, T, is equal to the irregular section area, A, shown on figures 2-4a and 2-4b, of the flow section and is commonly used for critical flow relationships. The equation for hydraulic mean depth, h_m, is:

$$h_m = \frac{A}{T} \quad (2\text{-}24)$$

In rectangular channels, hydraulic radius, R_h, does not equal depth, but approaches depth as the channel becomes very wide. However, the hydraulic mean depth, h_m, is the same as the depth of the rectangular flow section.

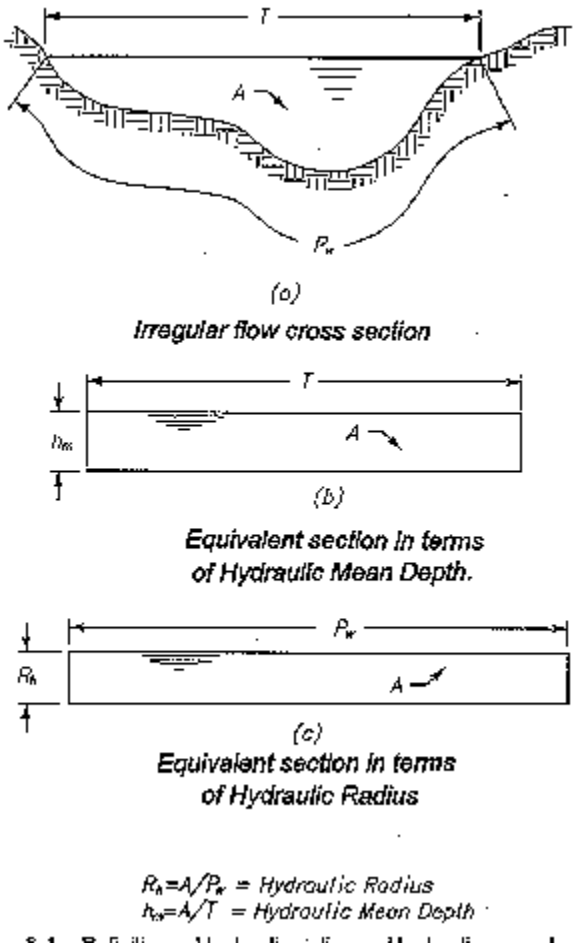

Figure 2-4 -- Definitions of hydraulic radius and hydraulic mean depth (area is the same for all three cases).

12. Froude Number, Critical Flow Relationships

In open channel hydraulics, the Froude number is a very important non-dimensional parameter. The Froude number, *F*, is the ratio of inertia force to gravity force, which simplifies to:

$$F = \frac{V}{\sqrt{gh_m}} \quad (2\text{-}25)$$

where the subscript *m* denotes hydraulic mean depth as defined previously in section 11 of this chapter.

For open channel modeling, the Froude number of a model is made equal to the Froude number of the actual full size device. The length ratio is set and the scale ratios for velocity and discharge are determined from the equality. However, the modeler must make sure that differences in friction loss between the model and the actual device are insignificant or accounted for in some way.

Open channel flow water measurement generally requires that the Froude number, F, of the approach flow be less than 0.5 to prevent wave action that would hinder or possibly prevent an accurate head reading.

When the Froude number is 1, the velocity is equal to the velocity of wave propagation, or celerity. When this condition is attained, downstream wave or pressure disturbances cannot travel upstream. A Froude number of 1 also defines a very special hydraulic condition. This flow condition is called critical and defines the critical mean depth and critical velocity relationship as:

$$F_c = \frac{V_c}{\sqrt{g h_{cm}}} \quad (2\text{-}26)$$

The subscript c denotes critical flow condition. The critical hydraulic mean depth, h_{cm}, is the depth at which total specific energy is minimum for a given discharge. Conversely, h_{cm} is the depth at which the discharge is maximum for a given total specific energy. When depth is greater than critical, the resulting velocity is considered streaming or tranquil and is called subcritical velocity. Conversely, when the depth is less than critical, the flow is rapid or shooting and is called super-critical velocity.

Water measurement flumes function best by forcing flow to pass through critical depth; then discharge can be measured using one head measurement station upstream. Also, for weirs and flumes, one unique head value exists for each discharge, simplifying calibration. This flow condition is called free flow. However, if the downstream depth submerges critical depth, then separate calibrations at many levels of submergence are required, and two head measurements are needed to measure flow.

Designing flumes for submerged flow will always decrease accuracy of flow measurement. Flumes and weirs can be submerged unintentionally by poor design, construction errors, structural settling, attempts to supply increased delivery needs by increasing downstream heads, accumulated sediment deposits, or weed growths.

Important critical flow relationships can be derived using equation 2-26 and rewriting in the form:

$$V_c = \sqrt{g h_{cm}} \quad (2\text{-}27)$$

Solving for head in equation 2-27 results in:

$$h_{cm} = \frac{V_c^{\,2}}{g} \quad (2\text{-}28)$$

Dividing both sides of this equation by 2 gives critical velocity head in terms of critical mean depth written as:

$$\frac{V_c^2}{2g} = \frac{h_{cm}}{2} \quad (2\text{-}29)$$

The total energy head with Z equal to zero for critical flow using equation 2-19 is:

$$H_c = h_c + \frac{V_c^2}{2g} \quad (2\text{-}30)$$

Squaring both sides of equation 2-27 and replacing velocity with Q/A and h_{cm} with A/T according to equation 2-24 and rearranging results in:

$$\frac{Q_c^2}{g} = \frac{A_c^3}{T_c} \quad (2\text{-}31)$$

This equation and the specific energy equation 2-22 are the basic critical flow relationships for any channel shape.

13. Discharge Equation for Broad-Crested Rectangular Weirs

The discharge equation for the rectangular broad-crested weir will now be derived similar to Bos (1989). The width, L_b, of a rectangular flow section is the same as T, the top water surface width. Also, h_c is the same as h_{cm}, and using equation 2-29 for velocity head, equation 2-30 can be rewritten as:

$$H_c = h_c + \frac{h_c}{2} \quad (2\text{-}32)$$

or:

$$H_c = \frac{3}{2} h_c \quad (2\text{-}33)$$

Conversely:

$$h_c = \frac{2}{3} H_c \quad (2\text{-}34)$$

Multiplying both sides of equation 2-27 by the area, A_c, of the flow section, which is $L_b h_c$, results in discharge expressed as:

$$Q = L_b h_c \sqrt{g h_c} = L_b \sqrt{g h_c}^{3/2} \quad (2\text{-}35)$$

To get unit discharge, q, this equation is divided by the width of flow, L_b, resulting in:

$$q = \frac{Q}{L_b} = \sqrt{g h_c}^{3/2} \quad (2\text{-}36)$$

Solving for h_c:

$$h_c = \sqrt[3]{\frac{q^2}{g}} \quad (2\text{-}37)$$

Using equation 2-34 to replace h_c with H_c in equation 2-35 results in theoretical discharge, Q_t:

$$Q_1 = L_b \sqrt{g} \left(\frac{2}{3} H_c\right)^{3/2} \quad (2\text{-}38)$$

Discharges in equations 2-35 through 2-38 are usually considered actual, assuming uniform velocity throughout the critical depth cross section and assuming that no correction of velocity distribution is needed.

Because specific energy is constant in a fairly short measuring structure with insignificant friction losses, specific energy, H_c, at the critical location can be replaced with specific energy, H_1, at a head measuring station a short distance upstream. However, some friction loss, possible flow curvature, and non-uniform velocity distribution occur. Thus, a coefficient of C_d must be added to correct for these effects, resulting in an expression for actual discharge:

$$Q_a = C_d L_b \frac{2}{3} \sqrt{\frac{2}{3}} g H_1^{3/2} \quad (2\text{-}39)$$

For measurement convenience, the total head, H_1, is replaced with the depth, h_1. To correct for neglecting the velocity head at the measuring station, a velocity coefficient, C_v, must be added, resulting in:

$$Q_a = C_d C_v L_b \frac{2}{3} \sqrt{\frac{2}{3}} g h_1^{3/2} \quad (2\text{-}40)$$

This equation applies to both long-throated flumes or broad-crested weirs and can be modified for any shape by analyses using the energy balance with equation 2-31.

These equations differ only in numerical constants that are derived from assumptions and selection of basic relationships used in their derivation. However, experimental determination of the coefficient values for C and C_v would compensate, making each equation produce the same discharge for the same measuring head. Either equation could be used.

The examples given above show that traditional discharge equations are often a mixture of rational analysis and experimental coefficient evaluation.

However, recent development of computer modeling of long-throated flumes (Clemmens et al. [1991]) precludes the need for experimental determination of coefficients. These long-throated flumes are covered in chapter 8.

14. Application of Energy Principle to Tube-Type Flowmeters

The energy equation can be used to derive the venturi meter (figure 2-5) equation by assuming that the centerline of the meter is horizontal ($Z_1 = Z_2$); and due to its short length, there is no head loss, $h_f = 0$. Although these assumptions were made to simplify the derivation, the final results will be identical for any orientation of the venturi meter.

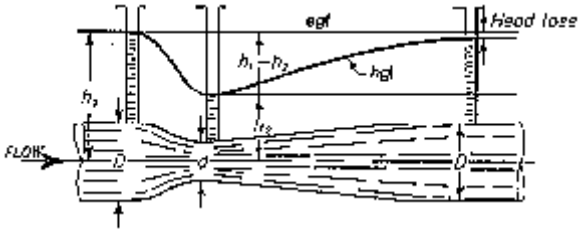

Figure 2-5 -- Venturi meter.

Thus:

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_2^2}{2g} \quad (2\text{-}41)$$

By the continuity equation for the approach and throat sections:

$$V_1 A_1 = V_2 A_2 \quad (2\text{-}42)$$

Either V_1 *or* V_2 can be solved for in terms of the other; for example:

$$V_2 = V_1 \left(\frac{A_1}{A_2}\right) \quad (2\text{-}43)$$

Substituting this result into the energy equation results in:

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_1^2}{2g}\left(\frac{A_1}{A_2}\right)^2 \quad (2\text{-}44)$$

Solving for the head difference gives:

$$h_1 - h_2 = \frac{V_1^2}{2g}\left(\frac{A_1}{A_2}\right)^2 - \frac{V_1^2}{2g} = \frac{V_1^2}{2g}\left[\left(\frac{A_1}{A_2}\right)^2 - 1\right] \quad (2\text{-}45)$$

Solving for V_1^2:

$$V_1^2 = \frac{(h_1 - h_2)2g}{(\frac{A_1}{A_2})^2 - 1} \quad (2\text{-}46)$$

Taking the square root of both sides and multiplying both sides by A_1 results in the theoretical discharge equation:

$$Q_t = V_1 A_1 = A_1 \sqrt{\frac{2g(h_1 - h_2)}{(\frac{A_1}{A_2})^2 - 1}} \quad (2\text{-}47)$$

To obtain actual discharge, a coefficient, C_d, added to compensate for velocity distribution and for minor losses not accounted for in the energy equation yields:

$$Q_a = C_d A_1 \sqrt{\frac{2g(h_1 - h_2)}{(\frac{A_1}{A_2})^2 - 1}} \quad (2\text{-}48a)$$

Some investigators solve for discharge using throat area and velocity, resulting in:

$$Q_a = C_d A_2 \sqrt{\frac{2g(h_1 - h_2)}{1 - (\frac{A_2}{A_1})^2}} \quad (2\text{-}48b)$$

However, equations 2-48a and 2-48b are identical and can be converted to:

$$Q_a = C_d A_1 A_2 \sqrt{\frac{2g(h_1 - h_2)}{A_1^2 - A_2^2}} \quad (2\text{-}49)$$

Equations 2-48b and 2-49 also apply to nozzles and orifices in pipes. On figure 2-5, the hydraulic grade line, *hgl*, represents the pressure that acts on the walls of the venturi meter. An appreciable drop will be noticed at the narrow throat, and a gradual pressure rise is seen as the flow leaves the throat and smoothly spreads and slows in the expanding portion of the meter.

Figure 2-6 shows the conditions that occur in a pipe orifice meter. As the flow approaches the orifice plate, the water near the pipe walls is slowed and stopped in the corners formed by the plate and the pipe walls. As a result, the pressure just ahead of the orifice at point B is a little greater than in the pipeline farther upstream at A. As the flow accelerates and passes through the orifice, the pressure drops and is lowest just downstream from the plate where the jet is smallest, and the velocity is highest at point C.

Farther down-stream, the flow begins to spread out and slow down, and a rise in pressure occurs at points D and E.

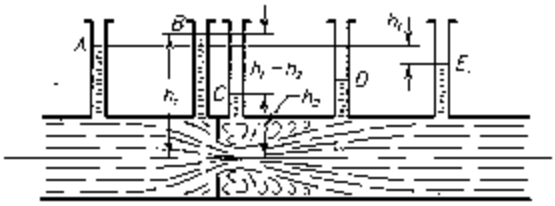

Figure 2-6 -- Pipe orifice meter.

In both venturi meters and orifice meters, the pressure difference between the inlet tap and the throat or minimum pressure tap is related to discharge tables or curves using the suitable coefficients with the proper equation. An example discharge curve is shown for an 8-inch (in) venturi meter on figure 2-7. Thus, the meters may serve as reliable flow measuring devices.

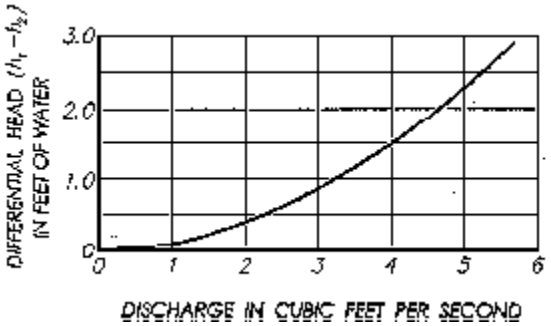

Figure 2-7 -- Typical calibration curve for an 8-in venturi meter.

15. Equation Coefficients

The previous examples show that coefficients are used in water measurement to correct for factors which are not fully accounted for using simplifying assumptions during derivations of equations. For the convenience of using a measured water head, h_1, instead of the more complex total head, H_1, C_v is used because velocity head is often ignored in equations.

Orifices require an area correction to account for jet contraction in an orifice, the flow is forced to curve around and spring from the sharp edge, forming a contracted jet or vena contracta. Thus, the contracted area of flow, A_c, should be used in hydraulic relationships. Thus, the area, A_o, of the orifice must be corrected by a coefficient of contraction defined as:

$$C_c = \frac{A_c}{A_o} \quad (2\text{-}50)$$

Properly designed venturi meters and nozzles have no contraction, which makes C_c unity because of the smooth transitions that allow the water to flow parallel to the meter boundary surfaces. Ultimately, the actual discharge must be measured experimentally by calibration tests, and the theoretical discharge must be corrected.

A common misconception is that coefficients are constant. They may indeed be constant for a range of discharge, which is the case for many standard measuring devices. Complying with structural and operational limits for standard devices will prevent measurement error caused by using coefficients outside of the proper ranges. Some water measuring devices cover wider ranges using variable coefficients of discharge by means of plots and tables of values with respect to head and geometry parameters.

Coefficients also vary with measuring station head or pressure tap location. Therefore, users should make sure that the coefficients used match pressure or head measurement locations. Water measurement equations generally require use of some to all of these coefficients to produce accurate results.

Often, composite numerical coefficients are given that are product combinations of area or a dimension factored from the area, acceleration of gravity, integration constants, and the correction coefficients. However, geometry dimensions and physical constants, such as acceleration of gravity, are better kept separate from the nondimensional coefficients that account for the difference between theoretical and actual conditions. Otherwise, converting equations from English to metric units is more difficult.

Equation 2-49 also applies to orifices and nozzles. The coefficient of discharge for venturi meters ranges from 0.9 to about unity in the turbulent flow range and varies with the diameter ratio of throat to pipe. The coefficient of discharge for orifices in pipes varies from 0.60 to 0.80 and varies with the diameter ratio. For flow nozzles in pipelines, the coefficient varies from 0.96 to 1.2 for turbulent flow and varies with the diameter ratio. ASME (1983) and ISO (1991) have a detailed treatment of pipeline meter theory, coefficients, and instruction in their use.

16. Normal Flow Equations and Friction Head Loss

Many measuring devices, such as flumes, weirs, and submerged orifices, are sensitive to exit flow conditions. Flumes and weirs can be drowned out by too much downstream submergence depth, and submerged orifices can have too little downstream water above the top orifice edge. Therefore, reliable knowledge of exit depth conditions is needed to properly set the elevation of crests and orifices so as to not compromise accuracy. Inaccurate assessment of downstream depth has even made some measuring device installations useless. Good operation and flow depth forecasts are needed to ensure the design effectiveness of new irrigation measurement systems. Designing for the insertion of a new device into an existing system provides a good opportunity to obtain actual field measurements for investigating possible submergence problems.

The use of actual discharge water surface measurements is recommended. In the absence of actual measurements, normal flow equations are often used to predict flow depths.

Normal flow occurs when the water surface slope, S_{ws}, is the same as the invert or bottom slope, S_o. When normal flow is approached, the velocity equations of Chezy, Manning, and Darcy-Weisbach are used to compute depth of flow. However, these equations are in terms of hydraulic radius, R_h, and depth must be determined on the basis of the definition of R_h, which is the cross-sectional area, A, divided by its wetted perimeter, P_w.

Chezy developed the earliest velocity equation, expressed as:

$$V = C\sqrt{R_h S} \quad (2\text{-}51)$$

Manning's equation is more frequently used and is expressed as:

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2} \quad (2\text{-}52)$$

The Darcy-Weisbach equation is a more rigorous relationship, written as:

$$V = \sqrt{\frac{8gR_h S}{f}} \quad (2\text{-}53)$$

The coefficients C, n, and f are friction factors. The Darcy-Weisbach friction factor, f, is nondimensional and is a function of Reynolds number, $4R_hV/L$, and relative roughness, $k/4R_h$, in which L is kinematic viscosity, and k is a linear measure of boundary roughness size. The Reynolds number accounts for variation of viscosity. This function is given in the form of plots in any fluid mechanics textbook; for example, Streeter (1951), Rouse (1950), and Chow (1959). These plots are generally in terms of pipe diameter, D, which should be replaced with $4R_h$ for open channel flow. Values of k have been determined empirically and are constant for a given flow boundary material as long as the roughness can be considered a homogenous texture rather than large roughness elements relative to the depth.

Solving equations 2-51, 2-52, and 2-53 for $V/(R_hS)^{1/2}$ results in a combined flow equation and relationship between the three friction factors, C, n, and f, written as:

$$\frac{1.49R_h^{1/6}}{n} = \sqrt{\frac{8g}{f}} = C = \frac{V}{\sqrt{R_h S}} \quad (2\text{-}54)$$

Solving for velocity using equation 2-54 and multiplying by area produces a discharge equation and can be used in the slope area method of determining discharge as discussed in chapter 13.

All three of these friction factors have been determined empirically, computed from measurement of equation variables. The Chezy factor, C, varies with hydraulic radius, slope, and physical boundary roughness. The Chezy factor varies from 22 to 220.

Manning's friction factor, n, varies from 0.02 for fine earth lined channels to 0.035 for gravel. If the channel beds are strewn with rocks or are 1/3 full of vegetation, the n value can be as much as 0.06. The n values for concrete vary from 0.011 to 0.016 as finish gets rougher. Values of k can be found in hydraulic and fluid mechanics textbooks such as Streeter (1951), Rouse (1950), and Chow (1959). The value of k for concrete varies from 0.01 to 0.0001 ft depending on condition and quality of finishing.

Because Chezy and Manning equations and their friction factors have been determined for ordinary channel flows, they do not accurately apply to shallow flow, nor can these two equations be corrected for temperature viscosity effects. Values of k are constant for given material surfaces for $k/4R_h$ equal or less than 1/10 and when $4R_hV/\nu$ is greater than 200,000.

Flow depths downstream are more likely the result of intentional structural restriction or water delivery head requirements downstream. Therefore, in designing and setting the elevation of flumes and weirs, the flow conditions just downstream need to be carefully assessed or specified in terms of required downstream operations and limits of measuring devices. More advanced hydraulic analyses are needed where normal flow is not established. For gradually varied flow, the friction equations can be used as trial and error computations applied to average end section hydraulic variables for relatively short reach lengths. The design and setting of crest elevations in an existing system permit the establishment of operation needs and downstream depths by actual field measurement.

17. Approach Flow Conditions

Water measurement devices are generally calibrated with certain approach flow conditions. The same approach conditions must be attained in field applications of measuring devices.

Poor flow conditions in the area just upstream from the measuring device can cause large discharge indication errors. In general, the approaching flow should be subcritical. The flow should be fully developed, mild in slope, and free of curves, projections, and waves. Pipeline meters commonly require 10 diameters of straight pipe approach. Fittings and combinations of fittings, such as valves and bends, located upstream from a flowmeter can increase the number of required approach diameters. *Fluid Meters* (American Society of Mechanical Engineers, 1983) and International Organization for Standardization (1991) give requirements for many pipeline configurations. By analogy and using a minimum of 10 pipe diameters of straight approach, open channel flow would require 40 hydraulic radii of straight, unobstructed, unaltered approach.

A typical example approach criteria as specified by Bos (1989) follows:

- If the control width is greater than 50 percent of the approach channel width, then 10 average approach flow widths of straight, unobstructed approach are required.
- If the control width is less than 50 percent of the approach width, then 20 control widths of straight, unobstructed approach are required.
- If upstream flow is below critical depth, a jump should be forced to occur. In this case, 30 measuring heads of straight, unobstructed approach after the jump should be provided.
- If baffles are used to correct and smooth out approach flow, then 10 measuring heads ($10h_1$) should be placed between the baffles and the measuring station.

Approach flow conditions should be continually checked for deviation from these conditions as described in chapter 8 of this manual.

18. Bibliography

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Reference 4

Table 3-1
Conveyance Factors (Standard Units)

Design Manning's Values for HDPE Pipe *		
Product	**Diameter**	**Design Manning's "n"**
N-12®, N-12® ST, and N-12® WT	4" - 60"	"n" = 0.012
AASHTO and Single Wall	18" - 24"	"n" = 0.024
	12" - 15"	"n" = 0.022
	10"	"n" = 0.019
	8"	"n" = 0.019
	3" - 6"	"n" = 0.017
Smoothwall	3" - 6"	"n" = 0.009 **
Conveyance Equations: k = Q/(s^0.5) Q = k s^0.5		

Conveyance Factors for Circular Pipe Flowing Full																		
Manning's "n" Values																		
Dia. (in.)	**Area (sq. ft.)**	**0.009**	**0.010**	**0.011**	**0.012**	**0.013**	**0.014**	**0.015**	**0.016**	**0.017**	**0.018**	**0.019**	**0.020**	**0.021**	**0.022**	**0.023**	**0.024**	**0.025**
3	0.05	1.3	1.1	1.0	1.0	0.9	0.8	0.8	0.7	0.7	0.6	0.6	0.6	0.5	0.5	0.5	0.5	0.5
4	0.09	2.7	2.5	2.2	2.1	1.9	1.8	1.6	1.5	1.5	1.4	1.3	1.2	1.2	1.1	1.1	1.0	1.0
6	0.20	8.1	7.3	6.6	6.1	5.6	5.2	4.9	4.6	4.3	4.1	3.8	3.6	3.5	3.3	3.2	3.0	2.9
8	0.35	17.5	15.7	14.3	13.1	12.1	11.2	10.5	9.8	9.2	8.7	8.3	7.9	7.5	7.1	6.8	6.5	6.3
10	0.55	31.6	28.5	25.9	23.7	21.9	20.3	19.0	17.8	16.8	15.8	15.0	14.2	13.6	12.9	12.4	11.9	11.4
12	0.79	51.5	46.3	42.1	38.6	35.6	33.1	30.9	28.9	27.2	25.7	24.4	23.2	22.1	21.1	20.1	19.3	18.5
15	1.23	93.3	84.0	76.3	70.0	64.6	60.0	56.0	52.5	49.4	46.7	44.2	42.0	40.0	38.2	36.5	35.0	33.6
18	1.77	151.7	136.6	124.1	113.8	105.0	97.5	91.0	85.3	80.3	75.9	71.9	68.3	65.0	62.1	59.4	56.9	54.6
21	2.41	228.9	206.0	187.3	171.6	158.4	147.1	137.3	128.7	121.2	114.4	108.4	103.0	98.1	93.6	89.6	85.8	82.4
24	3.14	326.8	294.1	267.3	245.1	226.2	210.1	196.1	183.8	173.0	163.4	154.8	147.0	140.0	133.7	127.9	122.5	117.6
27	3.98	447.3	402.6	366.0	335.5	309.7	287.6	268.4	251.6	236.8	223.7	211.9	201.3	191.7	183.0	175.0	167.8	161.0
30	4.91	592.5	533.2	484.7	444.3	410.2	380.9	355.5	333.3	313.7	296.2	280.6	266.6	253.9	242.4	231.8	222.2	213.3
33	5.94	763.9	687.5	625.0	572.9	528.9	491.1	458.3	429.7	404.4	382.0	361.9	343.8	327.4	312.5	298.9	286.5	275.0
36	7.07	963.4	867.1	788.2	722.6	667.0	619.3	578.0	541.9	510.0	481.7	456.4	433.5	412.9	394.1	377.0	361.3	346.8
42	9.62	1453.2	1307.9	1189.0	1089.9	1006.1	934.2	871.9	817.5	769.4	726.6	688.4	654.0	622.8	594.5	568.7	545.0	523.2
45	11.04	1746.8	1572.1	1429.2	1310.1	1209.3	1122.9	1048.1	982.6	924.8	873.4	827.4	786.1	748.6	714.6	683.5	655.0	628.8
48	12.57	2074.8	1867.4	1697.6	1556.1	1436.4	1333.8	1244.9	1167.1	1098.4	1037.4	982.8	933.7	889.2	848.8	811.9	778.1	746.9
54	15.90	2840.5	2556.4	2324.0	2130.4	1966.5	1826.0	1704.3	1597.8	1503.8	1420.2	1345.5	1278.2	1217.4	1162.0	1111.5	1065.2	1022.6
60	19.63	3762.0	3385.8	3078.0	2821.5	2604.4	2418.4	2257.2	2116.1	1991.6	1881.0	1782.0	1692.9	1612.3	1539.0	1472.1	1410.7	1354.3
72	28.27	6117.3	5505.6	5005.1	4588.0	4235.1	3932.6	3670.4	3441.0	3238.6	3058.7	2897.7	2752.8	2621.7	2502.5	2393.7	2294.0	2202.2

* Utah Water Research Laboratory, "Manning Friction Coefficient Testing of 4-, 10-, 12- and 15-inch Corrugated Plastic Pipe"[3]
** "Lingedburg, Michael, "Civil Engineer Reference Manual"[4]

© ADS, Inc., August, 200

PART K

PART K – LANDFILL OPERATION REQUIREMENTS – (62-701.500, FAC)

K.1 INTRODUCTION

Pasco County previously submitted a request to the Florida Department of Environmental Protection (FDEP) to modify the Conditions of Certifications (PA 87-23D) for the West Pasco County Landfill Class I solid waste cells vertical expansion. On February 3, 2020, the updated Conditions of Certification (PA 87-23E) and its accompanying Operation Plan were put into effect. Since then, an updated Conditions of Certification (PA 87-23F) has been put into effect as of May 3, 2021. Minor modifications are being proposed to the May 3, 2021 Operation Plan, and it is provided as Appendix B of this Solid Waste Permit Application.

PART R

PART R – FINANCIAL RESPONSIBILITY REQUIREMENTS

R.1 COST ESTIMATES

Please see the Financial Assurance Cost Estimate provided in Attachment R.1.

R.2 PROCEDURES FOR PROVIDING ANNUAL COST ADJUSTMENTS

Cost estimates for closure and long-term care are adjusted and provided to the Florida Department of Environmental Protection (FDEP) before September 1 of each year. Costs will be listed separately for closure and for long-term care. Pasco County Public Infrastructure uses and plans to continue using an escrow account to demonstrate financial responsibility. The cost estimates will be based on one of the two following methods as appropriate:

- Prepared and certified by a professional engineer for a third party performing the work on a per-unit basis with the source of estimates indicated.
- Inflating the previous year's estimate by the appropriate inflation factor as required by Chapter 62-701.630(4), Florida Administrative Code (FAC).

R.3 FUNDING MECHANISMS

Pasco County Public Infrastructure uses and plans to continue using an escrow account to demonstrate financial responsibility. Appropriate audits and forms will be provided as required by Rule 62-701.630, FAC.

Attachment R.1

Financial Assurance and Long-Term-Care Cost Estimates

FINANCIAL ASSURANCE CLOSURE AND LONG-TERM-CARE COST ESTIMATE PASCO COUNTY RESOURCE RECOVERY FACILITY

CONDITIONS OF CERTIFICATION NO.: PA 87-23G

FACILITY ID NO.: 45799

Submitted to:

Florida Department of Environmental Protection

Division of Waste Management

2600 Blair Stone Road, MS #4565

Tallahassee, Florida 32399-2400

Prepared for:

Pasco County Public Infrastructure

14230 Hays Road

Spring Hill, Florida 34610

Prepared by:

Jones Edmunds & Associates, Inc.

730 NE Waldo Road

Gainesville, Florida 32641

Certificate of Engineering Authorization #1841

Jones Edmunds Project No.: 16002-009-01

July 2021

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Exhibit 2 Backup of Closure and Long-Term Care Unit Costs

INTRODUCTION

This document presents the estimated closure and long-term-care costs for the West Pasco County Landfill (WPCL) as required by Florida Department of Environmental Protection (FDEP) Rules 62-701.630 and 62-701.730, Florida Administrative Code (FAC). The landfill cells included in this cost estimate are the Class I Ash Monofill Cells A-1 through A-4. Exhibit 1 provides Financial Assurance Cost Estimate Form 62-701.900(28) showing the detailed cost estimates for closure and long-term care (LTC) of the Class I Landfill Ash Monofill cells. Exhibit 2 provides backup for the costs.

GENERAL INFORMATION AND ASSUMPTIONS

The following information provides justification for quantities and unit prices for the closure and LTC cost estimates in accordance with Rule 62-701.630(3)(a), FAC. This information is provided in support of the cost estimate provided in the *Closure Cost Estimating Form for Solid Waste Facilities*, FDEP Form 62-701.900(28). The cost estimates are based on a third party performing the work on a per-unit basis with the source of the estimates indicated. "Number of Units" (quantities) and "Costs/Unit" (unit costs) are explained for each line item where applicable based on the final cover design presented in the following Engineering Plans and details provided in Appendix A of this permit application:

- Sheet C4, *Conceptual Final Cover Plan*
- Sheet C5 and C6, *Conceptual Final Cover Sections*
- Sheet C7, *Closure Details*

The final cover will consist of a 12-inch intermediate bedding layer over the Class I ash, a 40-mil textured geomembrane, a drainage geocomposite, 24 inches of soil final cover, and vegetative cover. The WPCL closure system consists of the following, from top to bottom:

- Soil Fill: 24 inches of soil.
- Geocomposite: 200 mil.
- Barrier Layer: 40-mil textured geomembrane.
- Intermediate Bedding Layer: 12 inches of sandy soil.

A soil-loss factor is used to adjust for a reduction in volume due to soil compaction. The soil-loss factor is included in the quantity calculation because quantities of soil purchased will be greater than the quantity of soil in-place after compaction.

Table 1 presents the general information and assumptions used to calculate the closure and LTC cost estimates.

Table 1 General Information and Assumptions

Facility Name:	West Pasco County Landfill
Facility Type:	Class I Landfill
Phase/Cell:	Cells A-1 – A-4
Acres (footprint):	50 acres
Closure Surface Area (three dimensional [3D])*:	2,162,301 SF
Soil Loss Factor:	20 percent
Synthetics Loss Factor:	10 percent
LTC Period:	30 years (minimum)

*Assumes no future ash monofill cells beyond A-4.

Table 2 presents calculations for soil volumes used in the closure cost estimates.

Table 2 Soil Volume Calculations

Thickness (feet)	Surface Area (SF)	Loss Factor (%)	Soil Volume (CF)	Soil Volume (CY)
0.5	2,162,301	20	1,297,380	48,051
1.0	2,162,301	20	2,594,761	96,102
1.5	2,162,301	20	3,892,141	144,153
2.0	2,162,301	20	5,189,522	192,205

Table 3 presents calculations for surface area units such as geosynthetics and maintenance.

Table 3 Geosynthetic Area Calculations

3D Surface Area	Unit	Loss Factor	Surface Area
2,162,301	SF	10%	2,378,531
240,256	SY	10%	264,281
49.6	acres	10%	54.6

Table 4 defines the abbreviations used in the cost estimate.

Table 4 List of Abbreviations

AC	Acre	LF	Linear foot
CF	Cubic foot	LS	Lump sum
CY	Cubic yard	MSF	1,000 square feet
DY	Day	SF	Square foot
EA	Each	SY	Square yard
HR	Hour	YR	Year

The detailed closure and LTC cost estimates are based on the following assumptions:

- The unit costs provided with each item are costs from recent bids, County Contracts, Florida Department of Transportation (FDOT) Pay Item 12-Month Moving Averages, and RS Means Heavy Construction Cost Data.
- Item costs that were estimated using the RS Means Heavy Construction Cost Data are for Tampa, Florida, which is the closest location available in the online edition and is representative of costs for Spring Hill, Florida.

IV. EXPLANATION OF CLOSURE COST ESTIMATES

1. PROPOSED MONITORING WELLS

No additional monitoring wells are required at the time of closure. As part of the operation of the landfill, the permit-required wells are installed.

2. SLOPE AND FILL

This item assumes grading and surface preparation (6 inches of excavation) will be required throughout the closure area and 12 inches of off-site material will be spread over the closure surface and compacted to serve as intermediate cover. The cost estimate is based on the estimates shown below:

Description	Unit	Number of Units	Cost/Unit	Cost Reference
Excavation	CY	48,051	$5.45/CY	FDOT Item: 0120 1 *Regular Excavation*
Placement and Spreading	CY	96,102	$2.10/CY	RS Means Item: 312323170020 *Fill, dumped material, spread, by dozer, excludes compaction.*
Compaction	CY	96,102	$0.43/CY	RS Means Item: 312323235100 *Compaction, roller, 12 inches, four passes.*
Off-Site Material	CY	96,102	$21.88/CY	FDOT Item: 0120 2 2 *Borrow Excavation, Truck Measure*

3. COVER MATERIAL (BARRIER LAYER)

The final cover of the landfill will consist of a layer of textured geomembrane and an overlying geocomposite. The cost estimate is based on the estimates shown below:

Description	Unit	Number of Units	Cost/Unit	Cost Reference
Geomembrane	SY	264,282	$4.50/SY	*ARGU America Quote*
Geocomposite	SY	264,282	$5.40/SY	*ARGU America Quote*

4. Top Soil Cover

The quantity for this item is based on 24 inches of final cover soil over the landfill closure area (including soil loss factor). The cost estimate figures are based on the following:

Description	Unit	Number of Units	Cost/Unit	Cost Reference
Off-Site Material	CY	192,205	$21.88/CY	FDOT Item: 0120 2 2 *Borrow Excavation, Truck Measure*
Placement and Spreading	CY	192,205	$2.10/CY	RS Means Item: 312323170020 *Fill, dumped material, spread, by dozer, excludes compaction.*

5. Vegetative Layer

Jones Edmunds assumes that the top of the landfill and the side slopes will be sodded.

Description	Unit	Number of Units	Cost/Unit	Cost Reference
Sodding	SY	264,282	$2.87/SY	FDOT Item: 0570 1 2, *Performance Turf, Sod*

6. Stormwater Control System

The stormwater system for Cells A-1, A-2, A-3, and A-4 consists of approximately 1,700 feet of 12-inch and 370 feet of 18-inch-diameter downchute piping and 14 inlet and outlet structures.

Description	Unit	Number of Units	Cost/Unit	Cost Reference
Piping – 12-inch	LF	1,700	$85.23/LF	FDOT Items: 0430175112 *Pipe Culv, 12 inches*
Piping – 18-inch	LF	370	$74.54/LF	FDOT Items: 0430175118 *Pipe Culv, 18 inches*
	Piping (Total Cost)		$172,470.80/LS	
Control Structures	EA	14	$4,355.00/EA	FDOT Item: 0425 1521 *Inlets, Type C, <10 feet*

7. Passive Gas Controls

Because no putrescible waste will be placed in the ash cells, negligible landfill gases will be generated, and no gas-control systems are expected to be necessary.

8. Active Gas Extraction Controls

An active gas system will not be constructed at this facility.

9. Security System

Security System costs include the repair of 100 feet of fencing and the replacement of one gate and sign at the time of closure. The cost estimate figures are based on the following:

Description	Unit	Total Units	Cost/Unit	Cost Reference
Fence repairs	LF	100	$25.00/LF	FDOT Item: 0550 10251 *Fencing, Type B, 8.1-10.0 feet with Barb Attmt*
Gate repairs	EA	1	$2,900/EA	FDOT Item: 0550 60214 *Fence Gate, Type B, Single, 18.1 to 20.0-foot Opening*
Sign repairs	EA	1	$340.94/EA	FDOT Item: 0700 1 11 *Single Post Sign, <12 SF*

10. Engineering

Engineering costs for designing and permitting were estimated based on experience with previous landfill closure projects in Florida and were estimated at 5 percent of the total construction cost.

11. Professional Services

This item consists of project management for the bidding, construction, construction quality assurance, and construction completion certification phases of the project and uses Jones Edmunds' standard rates.

Construction quality control (CQC) responsibilities belong to the general contractor and geomembrane installation subcontractor. Costs shown in Quality Assurance Testing are the third-party Construction Quality Assurance (CQA) activities, which are assumed to be administered by the consulting engineer. These costs were estimated from experience with previous landfill closure projects in Florida.

Position	Hours	Cost/Hour	Hours	Cost/Hour
	Contract Management		Quality Assurance	
PE Supervisor	240	$215	190	$215
On-Site Engineer	110	$170	360	$170
Office Engineer	530	$130	460	$130
On-Site Technician	210	$85	2,770	$85
Other: Administrative	210	$85	50	$85

QUALITY ASSURANCE TESTING

Jones Edmunds estimated quality assurance testing based on a cost of $2,400 per acre. This includes CQA for the liner and soil. The estimate was based on a recent EGC closure project, but it is assumed to be a reasonable approximation for the quality assurance testing unit cost for a traditional soil closure.

Description	Unit	Number of Units	Cost/Unit	Cost Reference
Quality Assurance Testing	AC	50	$2,400	*Costs from Bids for New River Phase I EGC Closure (2019)*

12. CONTINGENCY

A contingency amount of 5 percent of the total construction cost is estimated to cover unforeseen costs that may be incurred during construction and price fluctuations in the construction industry.

13. SITE SPECIFIC COSTS

There are no closure Site-Specific Costs identified for WPCL Cells A-1 through A-4 at this time.

V. LONG-TERM CARE EXPLANATION OF COST ESTIMATES

1. GROUNDWATER MONITORING

The groundwater monitoring system for Cells A-1 through A-4 consists of 20 monitoring wells that must be sampled semi-annually. Based on Jones Edmunds' estimates for the annual sampling, laboratory analyses, and biannual reporting, the cost breakdown is presented below:

Description	Sampling Frequency (Events/Year)	Number of Wells	(Cost/Well)/Event	Cost Reference
Semiannual Well Sampling	2	20	$1,086.33	Jones Edmunds Estimate

2. SURFACE WATER MONITORING

The existing stormwater system will not discharge from the site; therefore, no cost was assumed for surface water monitoring regarding Cells A-1 through A-4.

3. GAS MONITORING

Because negligible gas emissions are expected from the ash cells, no gas monitoring is expected at this time.

4. LEACHATE MONITORING

Leachate monitoring is not required at the WPCL by FDEP permit. However, the collected leachate from the ash cells is periodically hauled off site for disposal. The disposal site requires an annual demonstration that the leachate does not exhibit the toxicity characteristic defined in 40 CFR 261.24. The annual cost to conduct a Toxicity Characteristic Leaching Procedure (TCLP) analysis is approximately $350.

Description	Sampling Frequency (Events/Year)	Cost/Event	Cost Reference
Annual TCLP Testing	1	$350	*Financial Cost Estimate from JMG Engineering (dated 06/2018) (Reference 1)*

5. LEACHATE COLLECTION/TREATMENT SYSTEMS

The total annual leachate generation rate is estimated based on the generation rates for the closed East Pasco County Class I Landfill as determined in the previous Financial Assurance Cost Estimate by JMG Engineering, attached as Reference 1, and assumes that Cells A-1 through A-4 will generate a similar amount annually.

Maintenance of the leachate collection and treatment system includes jet cleaning the collection pipes every 5 years. The cost for cleaning the facility's sumps annually includes

the use of a vacuum truck and technician for an estimated 2 days. Leachate from the ash cells is currently collected in a 2-million-gallon aboveground storage tank and periodically hauled off site for disposal at a permitted domestic wastewater treatment facility.

Description	Unit	Total Units	Frequency (Event/Years)	Number of Units/Year	Cost/Unit	Cost Reference
Collection Pipes	LF	22,443	1/5YR	4,489	$0.75/LF	*Florida Jet Clean Quote*
Disposal	KGAL	1,286	1/YR	1	$70/ KGAL	*Aqua Clean Environmental Quote (Reference 1)*
Sumps, Traps	DY	2	1/5YR	0.40	$2,500/ DY	*Florida Jet Clean Quote*

6. Groundwater Monitoring Wells Maintenance

The cost for this item is based on replacing one groundwater well every 5 years. This includes abandoning and drilling a new well. The average depth of each well is assumed to be 60 feet.

Description	Unit	Total Units	Frequency (Event/Year)	Cost/Unit	Cost Reference
Replacement Well	EA	1	1/5YR	$3,915/EA	*Preferred Drilling Solutions, Inc. Quote*
Abandonment	EA	1	1/5YR	$680/EA	*Preferred Drilling Solutions, Inc. Quote*

7. Gas System Maintenance

Since there are no gas wells, there is no need for gas system maintenance.

8. Landscape Maintenance

The cost for this item is based on mowing the Landfill approximately eight times per year. The area to be mowed is based on the 3D surface area of Cell's closure surface.

Description	Unit	Total Units	Cost/Unit	Cost Reference
Mowing	AC	54.6	$48.60/AC	FDOT Item: 0107 2 *Mowing*

9. Erosion Control and Cover Maintenance

For erosion control and cover maintenance, Jones Edmunds assumed that the entire closure surface would be regraded and sodded over the 30-year closure period; this is approximately 8,470 square yards per year. The repair of 1,000 square yards of liner and underlying geocomposite is assumed per year.

Description	Unit	Total Units	Cost/Unit	Cost Reference
Sodding	SY	8,470	$2.87/SY	FDOT Item: 0570 1 2, *Performance Turf, Sod*
Regrading	SY	8,470	$0.28/SY ($30.88/MSF)	RS Means Item: 312216103312 *Fine grading, slopes, steep, large quantities, finish grading*
Liner Repair	SY	1,000	$12.96/SY	*Thalle Schedule of Values*

10. Stormwater Management System Maintenance

Jones Edmunds assumed that the perimeter ditch system will be cleaned annually. Approximately 6 inches of sediment is assumed to be removed from the bottom of a 2-foot-wide ditch.

Description	Unit	Total Units	Frequency (Event/Years)	Cost/Unit	Cost Reference
Conveyance Maintenance	CY	263	1/YR	$5.45/CY	FDOT Item: 0120 1 *Regular Excavation*

11. Security System Maintenance

Security system maintenance assumes 100 feet of fencing repairs annually and a new sign and gate replacement annually.

Description	Unit	Total Units	Cost/Unit	Cost Reference
Fence repairs	LF	100	$25.00/LF	FDOT Item: 0550 10251 *Fencing, Type B, 8.1 to 10.0 feet with Barb Attmt*
Gate repairs	EA	1	$2,900/EA	FDOT Item: 0550 60214 *Fence Gate, Type B, Single, 18.1- to 20.0-foot Opening*
Sign repairs	EA	1	$340.94/EA	FDOT Item: 0700 1 11 *Single Post Sign, <12 SF*

12. Utilities

This item is estimated to be $1,000 annually to run the leachate pumps.

13. Leachate Collection/Treatment Systems Operation

This item includes operation labor costs for managing the system and the current Jones Edmunds' billing rates for the personnel. The estimate includes 8 hours per week for on-site personnel.

14. ADMINISTRATIVE

This item includes administration labor costs for managing the permitting and reporting requirements, inspections, and maintenance. The cost estimate is consistent with current Jones Edmunds' labor rates and experience with similar facilities.

Position	Hours/Year	Cost/Hour
PE Supervisor	52	$215
Office Engineer	104	$130
On-Site Technician	520	$85

15. CONTINGENCY

The contingency is calculated as 5 percent of the total costs of Items 1 through 14. This number is based on Jones Edmunds' engineering judgment and consideration of estimation of expected annual costs.

16. SITE-SPECIFIC COSTS

There are no LTC Site-Specific Costs identified for WPCL Cells A-1 through A-4.

Reference 1

Reference 1

LONG TERM CARE COST ESTIMATE

(Note: These estimates are for the entire Class I Landfill, inclusive of A1, A2, A3, A4, SW1, and SW2)

June 2018

1. **Groundwater Monitoring [62-701.510 (6), and (8)(a)]**

   The West Pasco Class I Landfill has 29 groundwater monitoring wells (20 associated with the ash cells and 9 associated with the solid waste cells) that are sampled semi-annually. Sampling and analysis is conducted by Pasco County Environmental Services. Included in Part 4 are the unit costs estimates provided by Pasco County Environmental Laboratory to obtain the required groundwater samples and to analyze them for the required constituents. Annual groundwater sampling and analysis is estimated to be **$20,300.**

2. **Surface Water Monitoring [62-701.510(4), and (8)(b)]**

   It is not anticipated that the existing stormwater system will discharge from the site. Accordingly, there is no cost associated with surface water monitoring.

3. **Gas Monitoring [62-701.400(10)]**

   Landfill gas from the solid waste cells is monitored at 6 gas monitoring locations on a quarterly basis. Sample analysis is conducted in-situ, so the only costs associated with the gas monitoring program are employee time and mileage. Annual costs associated with the gas monitoring program are:

   - Quarterly samples (four hours on site at $20/hr, four 30-mile trips @ $0.575/mile) = $389 (add $100 for conservancy)

   Annual gas monitoring is estimated to be approximately **$480.**

V. 4 - Leachate Monitoring

4. **Leachate Monitoring [62-701.510(5),(6)(b) and 62-701.510(8)(c).**

   Currently, leachate is collected and periodically hauled off-site for disposal. The disposal site requires an annual demonstration that the leachate does not exhibit the toxicity characteristic defined at 40 CFR 261.24. The annual cost to conduct a TCLP analysis is approximately $350.

   Annual leachate monitoring is estimated to be **$350.**

5. **Leachate Collection/Treatment Systems Maintenance**

   Routine maintenance of the leachate collection system is a high-pressure cleaning of all laterals and collection mains every five years. A third-party contractor recently conducted this maintenance at a cost of $9,120 (see **Part 4**). Though not deemed necessary following the last routine pressure cleaning, it is possible that additional video-inspection *could* become necessary

Reference 1

in the future. Therefore, for purposes of estimating long-term care costs, Pasco County will apply a safety factor to this estimate and assume an annual cost of **$9,000** per year for the ash cells and **$9,000** per year for the solid waste cells.

Leachate from the ash cells is currently collected in a 2 million gallon above ground storage tank and periodically hauled off site for disposal at a permitted domestic wastewater treatment facility. Leachate from the solid waste cells is pumped directly to the adjacent Shady Hills Wastewater Treatment Facility. The current contract price for transportation and disposal of the ash leachate is approximately $0.07/gallon. For consistency, it is conservatively assumed that leachate from the solid waste cells will also be hauled off-site once the cells are closed.

Once the landfill (both the ash cells and the solid waste cells) is in long term care, the amount of leachate generated will be minimal because of the landfill cover. To approximate the amount of leachate that will be generated following installation of the final cover systems, leachate generation rates for the closed East Pasco Class I landfill were reviewed. The East Pasco Landfill was used for this analysis because the closure design is similar to that anticipated for the West Pasco landfill. The portion of the East Pasco Landfill that incorporates a leachate collection system is approximately 80 acres in size, which is just slightly larger than the 70 acres envisioned in this Long Term Care Cost estimate for the West Pasco site. Monthly leachate generation rates for East Pasco show that the average monthly volume of leachate collected in the capped and closed landfill is approximately 150,000 gallons per month. Extrapolating this value out over a 12-month period results in an estimated annual leachate generation rate of 1.8 million gallons for the 80 acre closed landfill (which is conservatively larger than the 70 acre West Pasco landfill.) At $0.07/gallon for transportation and disposal, this equates to approximately **$126,000** per year in leachate disposal costs ($90,020 for the ash cells and $35,980 for the solid waste cells).The previous [illegible] that the leachate generatio[illegible]uring operation.

(1,800,000 gal.)*(50 acre-ash cells/70 acres-total) =

= 1,286,000 gal./year from 50-acre ash monofill area

6. Groundwater Monitoring Well Maintenance

It is estimated that the construction of a new well, installed to a depth of approximately 30 feet (the average depth of a surficial aquifer monitoring well at the site) is approximately $4,500. Assuming that all of the existing monitoring wells will at some point during the 30-year long term care period be replaced, total replacement cost will be $130,500 ($4,500 x 29 wells). Dividing this value by the 30-year long term care period results in an estimated annual cost of **$4,350**.

7. Gas System Maintenance

The gas monitoring system at SW-1 and SW-2 consists of passive vents designed to reduce the potential for lateral gas migration beyond the property boundary. The estimated cost to design, permit, and construct a passive vent is assumed to be $12,500 (based on 2015 installation of passive vents at the East Pasco Class I Landfill). Assuming that routine maintenance over the

Exhibit 1

Financial Assurance Cost Estimate
Form 62-701.900(28), FAC

Florida Department of Environmental Protection

Bob Martinez Center
2600 Blair Stone Road
Tallahassee, Florida 32399-2400

DEP Form # 62-701.900(28), F.A.C.
Form Title: Closure Cost Estimating Form For Solid Waste Facilities
Effective Date: January 6, 2010
Incorporated in Rule 62-701.630(3), F.A.C.

CLOSURE COST ESTIMATING FORM FOR SOLID WASTE FACILITIES

Date of DEP Approval: _____

I. GENERAL INFORMATION:

Facility Name: West Pasco County Landfill (Class I Cells A-1, A-2, A-3, A-4) WACS ID: 45799

Permit Application or Consent Order No.: PA87-23 Expiration Date: _____

Facility Address: 14230 Hays Road, Spring Hill, FL 34610

Permittee or Owner/Operator: Pasco County Public Infrastructure

Mailing Address: 14230 Hays Road, Spring Hill, FL 34610

Latitude: 28° 22' 42.23" Longitude: 82° 33' 29.46"

Coordinate Method: Google Earth Datum: WGS 84

Collected by: Chad Spreadbury Company/Affiliation: Jones Edmunds

Solid Waste Disposal Units Included in Estimate:

Phase / Cell	Acres	Date Unit Began Accepting Waste	Active Life of Unit From Date of Initial Receipt of Waste	If active: Remaining life of unit	If closed: Date last waste received	If closed: Official date of closing
Cell A-1	10	Feb. 1991	5.75	18.7	N/A	N/A
Cell A-2	10	Dec. 1996	6.5			
Cell A-3	10	May 2003	8			
Cell A-4	20	Jul. 2011	6.8			

Total disposal unit acreage included in this estimate: Closure: 50 Long-Term Care: 50

Facility type: (check all that apply) ☑ Class I ☐ Class III ☐ C&D Debris Disposal ☐ Other: _____

II. TYPE OF FINANCIAL ASSURANCE DOCUMENT (Check type)

- ☐ Letter of Credit*
- ☐ Performance Bond*
- ☐ Guarantee Bond*
- ☐ Insurance Certificate
- ☐ Financial Test
- ☐ Trust Fund Agreement
- ☑ Escrow Account
- ☐ Form 29 (FA Deferral)

* - Indicates mechanisms that require the use of a Standby Trust Fund Agreement

Northwest District
160 Government Center
Pensacola, FL 32502-5794
850-595-8360

Northeast District
7825 Baymeadows Way, Ste. B200
Jacksonville, FL 32256-7590
904-807-3300

Central District
3319 Maguire Blvd., Ste. 232
Orlando, FL 32803-3767
407-894-7555

Southwest District
10351 N. Telecom Pky.
Temple Terrace, FL 33637
813-632-7600

South District
2295 Victoria Ave., Ste.364
Fort Myers, FL 33901-3881
239-332-6975

Southeast District
400 N. Congress Ave., Ste 200
West Palm Beach, FL 33401
561-681-6600

II. ESTIMATE ADJUSTMENT

40 CFR Part 264 Subpart H as adopted by reference in Rule 62-701.630, Florida Administrative Code, (F.A.C.) sets forth the method of annual cost estimate adjustment. Cost estimates may be adjusted by using an inflation factor or by recalculating the maximum costs of closure in current dollars. Select one of the methods of cost estimate ajustment below.

☐ **(a) Inflation Factor Adjustment** ☑ **(b) Recalculated or New Cost Estimates**

Inflation adjustment using an inflation factor may only be made when a Department approved closure cost estimate exists and no changes have occurred in the facility operation which would necessitate modification to the closure plan. The inflation factor is derived from the most recent Implicit Price Deflator for Gross National Product published by the U.S. Department of Commerce in its survey of Current Business. The inflation factor is the result of dividing the latest published annual Deflatory by the Deflator for the previous year. The inflation factor may also be obtained from the Solid Waste website www.dep.state.fl.us/waste/categories/swfr or call the Financial Coordinator at (850) 245-8706.

This adjustment is based on the Department approved closing cost estimate dated: _____

Latest Department Approved **Closing Cost** Estimate:		Current Year Inflation Factor, **e.g. 1.02**		Inflation Adjusted Closing Cost Estimate:
_____	×	_____	=	_____

This adjustment is based on the Department approved long-term care cost estimate dated: _____

Latest Department Approved Annual **Long-Term Care** Cost Estimate:		Current Year Inflation Factor, **e.g. 1.02**		Inflation Adjusted Annual Long-Term Care Cost Estimate:
_____	×	_____	=	_____
Number of Years of Long Term Care Remaining:			×	_____
Inflation Adjusted Long-Term Care Cost Estimate:			=	_____

Signature by: **Owner/Operator** **Engineer** (check what applies)

_____ Signature

_____ Name & Title

_____ Date

_____ Telephone Number

_____ Address

_____ City, State, Zip Code

_____ E-Mail Address

IV. ESTIMATED CLOSING COST (check what applies)

☑ **Recalculated Cost Estimate** ☐ **New Facility Cost Estimate**

Notes:
1. Cost estimates for the time period when the extent and manner of landfill operation makes closing most expensive.
2. Cost estimate must be certified by a professional engineer.
3. Cost estimates based on third party suppliers of material, equipment and labor at fir market value.
4. In some cases, a price quote in support of individual

Description	Unit	Number of Units	Cost / Unit	Total Cost
1. Proposed Monitoring Wells	**(Do not include wells already in existence.)**			
	EA		$ -	$ -
			Subtotal Proposed Monitoring Wells:	$ -
2. Slope and Fill (bedding layer between waste and barrier layer):				
Excavation	CY	48,051	$ 5.45	$ 261,879
Placement and Spreading	CY	96,102	$ 2.10	$ 201,815
Compaction	CY	96,102	$ 0.43	$ 41,324
Off-Site Material	CY	96,102	$ 21.88	$ 2,102,717
Delivery	CY		$ -	$ -
			Subtotal Slope and Fill:	$ 2,607,735
3. Cover Material (Barrier Layer):				
Off-Site Clay	CY		$ -	$ -
Synthetics - 40 mil	SY	264,282	$ 4.50	$ 1,189,267
Synthetics - GCL	SY		$ -	$ -
Synthetics - Geonet	SY		$ -	$ -
Synthetics - Other (explain) Geocomposite	SY	264,282	$ 5.40	$ 1,427,121
			Subtotal Cover Material:	$ 2,616,388
4. Top Soil Cover:				
Off-Site Material	CY	192,205	$ 21.88	$ 4,205,435
Delivery	CY			$ -
Spread	CY	192,205	$ 2.10	$ 403,629
			Subtotal Top Soil Cover:	$ 4,609,064
5. Vegetative Layer:				
Sodding	SY	264,282	$ 2.87	$ 758,488
Hydroseeding	SY		$ -	$ -
Fertilizer	AC		$ -	$ -
Mulch	AC		$ -	$ -
Other (explain)			$ -	$ -
			Subtotal Vegetative Layer:	$ 758,488
6. Stormwater Control System:				
Earthwork	CY			$ -
Grading	SY		$ -	$ -
Piping	LS	1	$ 172,470.80	$ 172,471
Ditches	LF		$ -	$ -
Berms	LF		$ -	$ -
Control Structures	EA	14	$ 4,355	$ 60,970
Other (explain)	LF			$ -
			Subtotal Stormwater Control System:	$ 233,441

Description	Unit	Number of Units	Cost / Unit	Total Cost
7. **Passive Gas Control:**				
Wells	EA		$ -	$ -
Pipe and Fittings	LF		$ -	$ -
Monitoring Probes	EA		$ -	$ -
NSPS/Title V Requirements	LS		$ -	$ -
			Subtotal Passive Gas Control:	$ -
8. Active Gas Extraction Control:				
Traps	EA		$ -	$ -
Sumps	EA		$ -	$ -
Flare Assembly	EA		$ -	$ -
Flame Arrestor	EA		$ -	$ -
Mist Eliminator	EA		$ -	$ -
Flow Meter	EA		$ -	$ -
Blowers	EA		$ -	$ -
Collection System	LS		$ -	$ -
Other (explain)	EA		$ -	$ -
			Subtotal Active Gas Extraction:	$ -
9. Security System:				
Fencing	LF	100	$ 25.00	$ 2,500
Gate(s)	EA	1	$ 2,900.00	$ 2,900
Sign(s)	EA	1	$ 340.94	$ 341
			Subtotal Security System:	$ 5,741
10. Engineering:				
Closure Plan Report	LS		$ -	$ -
Certified Engineering Drawings	LS		$ -	$ -
NSPS/Title V Air Permit	LS		$ -	$ -
Final Survey	LS		$ -	$ -
Certification of Closure	LS		$ -	$ -
Other (explain)	LS	1	$ 541,543	$ 541,543
7.5% Total Construction			Subtotal Engineering:	$ 541,543

Description	Hours	Cost / Hour	Hours	Cost / Hour	Total Cost
11. Professional Services:					
	Contract Management		Quality Assurance		
P.E. Supervisor	240	$ 215	190	$ 215	$ 92,450
On-Site Engineer	110	$ 170	360	$ 170	$ 79,900
Office Engineer	530	$ 130	460	$ 130	$ 128,700
On-Site Technician	210	$ 85	2,770	$ 85	$ 253,300
Other (explain)	210	$ 85	50	$ 85	$ 22,100
Administrative Assistant					
				Subtotal Professional Services:	$ 576,450

Description	Unit	Number of Units	Cost / Unit	Total Cost
Quality Assurance Testing	AC	50	$ 2,400	$ 120,000
			Subtotal Professional Services:	$ 696,450
			Subtotal of 1-11 Above:	$ 12,068,850
12. Contingency:	5	% of Subtotal of 1-11 Above		$ 603,443
			Subtotal Contingency:	$ 603,443
			Estimated Closing Cost Subtotal:	$ 12,672,293

Description	Total Cost
13. Site-Specific Costs:	
Mobilization and Bonds and insurance	
Waste Tire Facility	$ -
Materials Recovery Facility	$ -
Special Wastes	$ -
Leachate Management System Modification - Annual Disposal	$ -
Other (explain)	$ -
Subtotal Site Specific Costs:	$ -
TOTAL ESTIMATED CLOSING COSTS ($):	$ 12,672,293

V. ANNUAL COST FOR LONG-TERM CARE

See 62-701.600(1)a.1., 62-701.620(1), 62-701.630(3)a. and 62-701.730(11)b. F.A.C. for required term length. For landfills certified closed and Department accepted, enter the remaining long-term care length as "Other" and provide years remaining.

(Check Term Length) ☐ 5 Years ☐ 20 Years ☑ 30 Years ☐ Other, ____ Years

Notes: 1. Cost estimates must be certified by a professional engineer.

2. Cost estimates based on third party suppliers of material, equipment and labor at fair market value.

3. In some cases, a price quote in support of individual item estimates may be required.

All items must be addressed. Attach a detailed explanation for all items marked not applicable (N/A)

Description	Sampling Frequency (Events / Year)	Number of Wells	(Cost / Well) / Event	Annual Cost
1. Groundwater Monitoring [62-701.510(6), and (8)(a)]				
Monthly	12		$ -	$ -
Quarterly	4		$ -	$ -
Semi-Annual	2	20	$ 1,086.33	$ 43,453
Annual	1		$ -	$ -
			Subtotal Groundwater Monitoring:	$ 43,453
2. Surface Water Monitoring [62-701.510(4), and (8)(b)]				
Monthly	12		$ -	$ -
Quarterly	4		$ -	$ -
Semi-Annual	2		$ -	$ -
Annual	1		$ -	$ -
			Subtotal Surface Water Monitoring:	$ -
3. Gas Monitoring [62-701.400(10)]				
Monthly	12		$ -	$ -
Quarterly	4		$ -	$ -
Semi-Annual	2		$ -	$ -
Annual	1		$ -	$ -
			Subtotal Gas Monitoring:	$ -
4. Leachate Monitoring [62-701.510(5), (6)(b) and 62-701.510(8)c]				
Monthly	12		$ -	$ -
Quarterly	4		$ -	$ -
Semi-Annual	2		$ -	$ -
Annual	1	1	$ 350.00	$ 350
Other (explain) _____			$ -	$ -
			Subtotal Leachate Monitoring:	$ 350

Description	Unit	Number of Units / Year	Cost / Unit	Annual Cost
5. Leachate Collection/Treatment Systems Maintenance				
Maintenance				
Collection Pipes	LF	4,489	$ 0.75	$ 3,367
Sumps, Traps	DY	0.4	$ 2,500.00	$ 1,000
Lift Stations	EA		$ -	$ -
Cleaning	LS		$ -	$ -
Tanks	DY		$ -	$ -

Description	Unit	Number of Units / Year	Cost / Unit	Annual Cost
5. (continued)				
Impoundments				
Liner Repair	SY		$ -	$ -
Sludge Removal	CY		$ -	$ -
Aeration Systems				
Floating Aerators	~~EA~~ DAY			$ -
Spray Aerators	EA		$ -	$ -
Disposal				
Off-site (Includes transportation and disposal)	1,000 gallon	1,286	$ 70.00	$ 90,020
		Subtotal Leachate Collection / Treatment Systems Maintenance:		$ 94,387
6. Groundwater Monitoring Wells Maintenance				
Monitoring Wells	LF		$ -	$ -
Replacement	EA	0.2	$ 3,915.00	$ 783
Abandonment	EA	0.2	$ 680.00	$ 136
		Subtotal Groundwater Monitoring Well Maintenance:		$ 919
7. Gas System Maintenance				
Piping, Vents	EA		$ -	$ -
Blowers	EA		$ -	$ -
Flaring Units	EA		$ -	$ -
Meters, Valves	EA		$ -	$ -
Compressors	EA		$ -	$ -
Flame Arrestors	EA		$ -	$ -
Operation	EA		$ -	$ -
			Subtotal Gas System:	$ -
8. Landscape Maintenance				
Mowing 8 /yr	AC	54.6	$ 48.60	$ 21,230
Fertilizer /yr	AC		$ -	$ -
			Subtotal Landscape Maintenance:	$ 21,230
9. Erosion Control and Cover Maintenance				
Sodding	SY	8,470	$ 2.87	$ 24,309
Regrading	SY	8,470	$ 0.28	$ 2,372
Liner Repair	SY	1,000	$ 12.96	$ 12,960
Clay	CY		$ -	$ -
			Subtotal Erosion Control and Cover Maintenance:	$ 39,641
10. Storm Water Management System Maintenance				
Conveyance Maintenance	CY	263	$ 5.45	$ 1,433
			Subtotal Storm Water System Maintenance:	$ 1,433
11. Security System Maintenance				
Fences	LF	100	$ 25.00	$ 2,500
Gate(s)	EA	1	$ 2,900.00	$ 2,900
Sign(s)	EA	1	$ 340.94	$ 341
			Subtotal Security System:	$ 5,741

Description	Unit		Number of Units / Year	Cost / Unit	Annual Cost
12. Utilities		LS	1	$ 1,000	$ 1,000
13. Leachate Collection/Treatment Systems Operation					
Operation					
P.E. Supervisor		HR		$ -	$ -
On-Site Engineer		HR		$ -	$ -
Office Engineer		HR		$ -	$ -
On-Site Technician		HR	416	$ 85	$ 35,360
Materials		LS			
		Subtotal Leachate Collection/Treatment System Operation:			$ 35,360
14. Administrative			Hours	$/Hour	Total
P.E. Supervisor		HR	52	$ 215	$ 11,180
On-Site Engineer		HR		$ -	$ -
Office Engineer		HR	104	$ 130	$ 13,520
Onsite Technician		HR	520	$ 85	$ 44,200
Other		HR			$ -
				Subtotal Administrative:	$ 68,900
				Subtotal of 1-14 Above:	$ 312,414
15. Contingency	5%	% of Subtotal of 1-14 Above			$ 15,621
				Subtotal Contingency:	$ 15,621

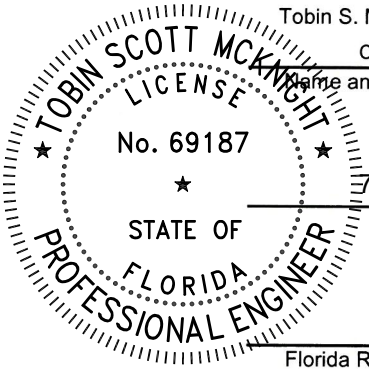
Description	Unit	Number of Units / Year	Cost / Unit	Annual Cost
16 Site-Specific Costs (explain)				
				$ -
				$ -
				$ -
				$ -
			Subtotal Site Specific Costs:	$ -
			ANNUAL LONG-TERM CARE COST ($ / Year):	**$ 328,035**
			Number of Years of Long-Term Care:	30
			TOTAL LONG-TERM CARE COST ($):	**$ 9,841,050**

VI. CERTIFICATION BY ENGINEER

This is to certify that the Cost Estimates pertaining to the engineering features of this solid waste management facility have been examined by me and found to conform to engineering principles applicable to such facilities. In my professional judgment, the Cost Estimates are a true, correct and complete representation of the financial liabilities for closing and/or long-term care of the facility and comply with the requirements of Rule 62-701.630 F.A.C. and all other Department of Environmental Protection rules, and statutes of the State of Florida. It is understood that the Cost Estimates shall be submitted to the Department annually, revised or adjusted as required by Rule 62-701.630(4), F.A.C.

Tobin S McKnight
2021.07.21 11:43:05-04'00'
Signature

730 NE Waldo Road
Mailing Address



Tobin S. McKnight, PE, BCEE
Chief Engineer
Name and Title (please type)

Gainesville, Florida 32641
City, State, Zip Code

7/13/2021
Date

tmcknight@jonesedmunds.com
E-Mail address (if available)

69178
Florida Registration Number
(please affix seal)

(352) 377-5821
Telephone Number

This item has been digitally signed and sealed by Tobin S. McKnight, PE on the date adjacent to the seal.

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

VII. SIGNATURE BY OWNER/OPERATOR

E-Mail address (if available) Telephone Number

Signature

14230 Hays Road
Mailing Address

Justin Roessler, PhD, PE,
Solid Waste Director
Name and Title (please type)

Spring Hill, FL 34610
City, State, Zip Code

jroessler@PascoCountyFL.net
E-Mail address (if available)

(727) 856-0119
Telephone Number

Exhibit 2

Backup of Closure and Long-Term Care Unit Costs

IV. 2.1 & LTC V.10 - Excavation

Florida Department of Transportation
Item Average Unit Cost
From 2020/04/01 to 2021/03/31
Statewide

Market Area: 07
Contract Type: CC
Displaying: VALID ITEMS WITH HITS
From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0108 2	1	$42,765.00	$42,765.00	1.000	EA	N	MONITOR EXISTING STRUCTURES- VIBRATION MONITORING
0108 3	1	$15,000.00	$15,000.00	1.000	EA	N	MONITOR EXISTING STRUCTURES- GROUNDWATER MONITORING
0110 1 1	13	$12,990.44	$3,275,020.82	252.110	AC	N	CLEARING & GRUBBING
0110 2 2	1	$17,000.00	$17,170.00	1.010	AC	N	SELECTIVE CLEARING AND GRUBBING, AREAS WITH TREES TO REMAIN
0110 4 10	12	$11.34	$447,605.01	39,472.000	SY	N	REMOVAL OF EXISTING CONCRETE
0110 7 1	7	$203.47	$41,101.80	202.000	EA	N	MAILBOX, F&I SINGLE
0120 1	11	$5.45	$1,730,848.31	317,337.000	CY	N	REGULAR EXCAVATION
0120 2 2	1	$37.72	$34,362.92	911.000	CY	N	BORROW EXCAVATION, TRUCK MEASURE
0120 4	4	$15.28	$12,757.00	835.000	CY	N	SUBSOIL EXCAVATION
0120 6	11	$6.98	$1,708,621.69	244,784.000	CY	N	EMBANKMENT
0120 71	1	$19,339.84	$19,339.84	1.000	LS	N	REGULAR EXCAVATION (3-R PROJECTS ONLY)
0120 74	1	$5.50	$3,657.50	665.000	CY	N	SURCHARGE EMBANKMENT
0141 70	1	$2,500.00	$7,500.00	3.000	AS	N	SETTLEMENT PLATE ASSEMBLY
0160 4	9	$7.03	$2,517,687.92	357,886.000	SY	N	TYPE B STABILIZATION
0285701	8	$14.33	$2,229,545.06	155,552.000	SY	N	OPTIONAL BASE, BASE GROUP 01
0285702	3	$23.81	$71,189.50	2,990.000	SY	N	OPTIONAL BASE, BASE GROUP 02
0285703	1	$55.00	$14,355.00	261.000	SY	N	OPTIONAL BASE, BASE GROUP 03
0285704	2	$25.11	$56,676.25	2,257.000	SY	N	OPTIONAL BASE, BASE GROUP 04
0285708	1	$36.84	$11,088.84	301.000	SY	N	OPTIONAL BASE, BASE GROUP 08
0285709	5	$21.89	$595,861.04	27,224.000	SY	N	OPTIONAL BASE, BASE GROUP 09

Cost Estimate Report

Date: 06/01/2021

IV: 2.2 & 2.3 & 4 - Placement & Spread and Compaction

Citrus County Central Landfill FACE 2021

Year 2021

Unit Detail Report

Prepared By: JoAnne Talamo Jones Edmunds & Associates, Inc.

LineNumber	Description	Quantity	Unit	Total Incl. O&P	Ext. Total Incl. O&P
Division 31 Earthwork					
312323170020	Fill, dumped material, spread, by dozer, excludes compaction	0.00	L.C.Y.	$2.10	$0.00
312323235100	Compaction, riding, vibrating roller, 4 passes, 12" lifts	0.00	E.C.Y.	$0.43	$0.00
Division 31 Earthwork Subtotal					**$0.00**
Subtotal					**$0.00**
General Contractor's Markup on Subs				**0.00%**	**$0.00**
Subtotal					**$0.00**
General Conditions				**0.00%**	**$0.00**
Subtotal					**$0.00**
General Contractor's Overhead and Profit				**0.00%**	**$0.00**
Grand Total					**$0.00**

IV. Closure - 2 & 4 - Off-Site Topsoil

Florida Department of Transportation
Item Average Unit Cost
From 2020/05/01 to 2021/04/30
Statewide

Contract Type: CC
Displaying: VALID ITEMS WITH HITS
From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0110 4 10	139	$20.72	$5,214,463.86	251,708.000	SY	N	REMOVAL OF EXISTING CONCRETE
0110 7 1	32	$188.49	$118,748.95	630.000	EA	N	MAILBOX, F&I SINGLE
0110 21	11	$3.78	$162,932.95	43,088.000	LF	N	TREE PROTECTION BARRIER
0110 22	4	$242.08	$121,039.33	500.000	EA	N	TREE ROOT AND BRANCH PRUNING
0110 23	7	$425.40	$54,876.62	129.000	EA	N	TREE REMOVAL
0110 71 1	1	$200.00	$168,800.00	844.000	LF	N	BRIDGE FENDER SYSTEM, REMOVAL & DISPOSAL
0110 73	2	$180.65	$63,408.00	351.000	LF	N	REMOVE EXISTING BULKHEAD
0110 82	2	$1,841.25	$129,440.00	70.300	MB	N	REMOVE & DISPOSE OF STRUCTURAL TIMBER
0120 1	102	$7.18	$9,660,963.33	1,345,162.700	CY	N	REGULAR EXCAVATION
0120 2 2	41	$21.88	$945,144.61	43,197.900	CY	N	BORROW EXCAVATION, TRUCK MEASURE
0120 3	1	$500.00	$6,650.00	13.300	CY	N	LATERAL DITCH EXCAVATION
0120 4	19	$16.94	$884,403.74	52,220.100	CY	N	SUBSOIL EXCAVATION
0120 5	2	$29.28	$257,079.90	8,779.700	CY	N	CHANNEL EXCAVATION
0120 6	88	$8.90	$8,411,196.96	945,209.700	CY	N	EMBANKMENT
0120 6103	1	$56.91	$2,333,247.40	40,998.900	CY	N	EMBANKMENT, CHANNEL EMBANKMENT, PROJECT 443155-1-52-01
0120 71	33	$31,696.56	$1,109,379.65	35.000	LS	N	REGULAR EXCAVATION (3-R PROJECTS ONLY)
0120 74	2	$24.52	$244,372.56	9,966.200	CY	N	SURCHARGE EMBANKMENT
0125 1	3	$17.57	$13,090.16	745.000	CY	N	EXCAVATION FOR STRUCTURES
0141 70	2	$2,038.46	$26,500.00	13.000	AS	N	SETTLEMENT PLATE ASSEMBLY
0144 1 1	1	$100.00	$11,000.00	110.000	LF	N	DIGITAL INCLINOMETER CASING, VERTICAL
0145 2	11	$5.40	$550,907.94	101,960.000	SY	N	GEOSYNTHETIC REINFORCED FOUNDATION OVER SOFT SOIL

Closure Item IV.3 - Geomembrane

All Bids were received by 3:00 p.m on January 7, 2019

Bid Tabulation

BID FORM ITEMS				Solmax GSE Varennes, Quebec Michael Winterbourne mwinterbourne@solmax.com			National Lining Systems Inc. Fort Myers, FL Joe Irwin jirwin@nationalliningsystems.com			Agru Georgetown, SC Chris Eichelberger ceichelberger@arguamerica.com
Item		Est. Quantity	Units	Est. Quantity	Unit Price	Total Price	Est. Quantity	Unit Price	Total Price	
1	Geomembrane	820,000	SF	830,898	$ 0.354	$ 294,138	820,000	$ 0.4057	$ 332,674	
2	Geotextile	31,500	SF	31,500	$ -	$ -	31,500	$ 0.1457	$ 4,590	
TOTAL BASE BID						**$ 294,138**			**$ 337,264**	
	Notes:			*20 lb spools of welding rod at $63 each; final cost depends on contractor's estimate* *Only bidding on geomembrane.*			*weld rod does not include any tie-ins as none are described in the bid documents* *Installer teaming with Atarfil (EGC) and Skapps (geotextile).*			*email 12/12/18: not able to meet the project schedule so will not be providing a bid*
Bid Requirements:										
Signed Bid Form				Y			Y			
Sample warranty - 20 yrs from time of installation				Y	*Bid: 20 years from effective date; upon folow-up vendor noted that this is the first day after install.*		N	*Bid: 20 years from date of sale*		
Manufacturer Prequalification Info				Y			Y			

All Bids were received by 3:00 p.m on September 25, 2020

Bid Tabulation

The IFB was emailed to four vendors and only one responded with a bid.

BID FORM ITEMS				Solmax Geosynthetics Houston, TX Michael Winterbourne mwinterbourne@solmax.com Bid Date: 9/17/2020		
Item		Est. Quantity	Units	Est. Quantity	Unit Price	Total Price
1A	Geomembrane [60-mil textured HDPE black]	1,661,400	SF	1,661,400	$ 0.264	$ 438,609.60
1B	Geomembrane [60-mil textured HDPE white]	1,661,400	SF	1,661,400	$ 0.283	$ 470,176.20

GMB	(15 acres)				
Solmax	2019	$	0.354	/SF	material plus delivery
Solmax	2020	$	0.283	/SF	material plus delivery
Agru	2019	$	0.406	/SF	material plus delivery
	avg	$	0.348	/SF	material plus delivery
		$	0.150	/SF	install
	TOTAL	$	0.50	/SF	
		$	4.48	/SY	
	rounded	$	4.50	/SY	

Closure Item IV.3 - Cover Material (Barrier Layer) Geocomposite

Matthew Morse

From: Matthew Morse
Sent: Tuesday, July 14, 2020 10:13 AM
To: Matthew Morse
Subject: FW: Landfill Closure Budget Pricing Update
Attachments: CT_ClosureOptions_Brochure CAE1.pdf

From: Chris Eichelberger <CEichelberger@AgruAmerica.com>
Sent: Tuesday, July 14, 2020 9:55 AM
To: Matthew Morse <MMorse@jonesedmunds.com>
Cc: Mike Gnau <MGnau@AgruAmerica.com>
Subject: RE: Landfill Closure Budget Pricing Update

CAUTION: This email originated from outside of the organization. Do not click links or open attachments unless you recognize the sender and know the content is safe.

Good morning Matthew,

You can use $0.60/SF for Supply / Install of Geocomposite and $0.25 / SF for the 8 oz NW fabric.

I have attached our closure solutions offered, if your team would be interested in discussing these further with the client. Thank you

($0.60 / SF) X (9 SF / SY) = **$5.40 / SY**

Best regards,

Chris Eichelberger
VP – Technical Marketing
AGRU America, Inc.

Mobile: (843) 630-4160
500 Garrison Road
Georgetown, SC 29440 USA
agruamerica.com

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Florida Department of Transportation

Item Average Unit Cost

From 2020/03/01 to 2021/02/28

Statewide

Market Area: 07

Contract Type: CC

Displaying: VALID ITEMS WITH HITS

From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0536 7 2	1	$250.00	$2,500.00	10.000	EA	N	SPECIAL GUARDRAIL POST- SPECIAL STEEL POST FOR CONCRETE STRUCTURE MOUNT
0536 7 3	2	$165.44	$992.64	6.000	EA	N	SPECIAL GUARDRAIL POST- ENCASED POST FOR SHALLOW MOUNT
0536 73	7	$2.83	$26,663.51	9,416.000	LF	N	GUARDRAIL REMOVAL
0536 85 20	6	$1,117.87	$11,178.74	10.000	EA	N	GUARDRAIL END TREATMENT- TRAILING ANCHORAGE
0536 85 24	6	$2,999.83	$32,998.11	11.000	EA	N	GUARDRAIL END TREATMENT- PARALLEL APPROACH TERMINAL
0536 85 26	1	$3,020.00	$3,020.00	1.000	EA	N	GUARDRAIL END TREATMENT- TYPE CRT
0546 72 3	4	$2,279.70	$90,677.35	39.776	GM	N	GROUND-IN RUMBLE STRIPS, 8" SINUSOIDAL
0548 12	1	$66.00	$3,260,730.00	49,405.000	SF	N	RETAINING WALL SYSTEM, PERMANENT, EXCLUDING BARRIER
0548 13	1	$9.23	$264,170.00	28,635.000	SF	N	RETAINING WALL SYSTEM,TEMPORARY, EXCLUDING BARRIER
0550 10110	1	$12.00	$18,564.00	1,547.000	LF	N	FENCING, TYPE A, 0.0-5.0', STANDARD
0550 10220	2	$16.23	$36,622.75	2,257.000	LF	N	FENCING, TYPE B, 5.1-6.0', STANDARD
0550 10228	1	$25.25	$505.00	20.000	LF	N	FENCING, TYPE B, 5.1-6.0, RESET EXISTING
0550 60214	1	$2,900.00	$2,900.00	1.000	EA	N	FENCE GATE, TYPE B, SINGLE, 18.1-20.0' OPENING
0550 60222	1	$1,500.00	$1,500.00	1.000	EA	N	FENCE GATE, TYPE B, DOUBLE, 6.1-12.0' OPENING
0570 1 2	14	$2.87	$2,905,973.19	1,013,576.000	SY	N	PERFORMANCE TURF, SOD
0570 1 3	2	$3.08	$173,402.90	56,327.000	SY	N	PERFORMANCE TURF, SOD AND SOIL- SHOULDER TREATMENT INDEX 570-010
0571 1 11	4	$4.52	$145,475.40	32,175.000	SY	N	PLASTIC EROSION MAT, TURF REINFORCED MAT, TYPE 1
0571 1 13	1	$5.00	$12,065.00	2,413.000	SY	N	PLASTIC EROSION MAT, TURF REINFORCED MAT, TYPE 3

Florida Department of Transportation

Item Average Unit Cost

From 2020/05/01 to 2021/04/30

Statewide

Market Area: 07

Contract Type: CC

Displaying: VALID ITEMS WITH HITS

From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0425 1355	1	$4,369.58	$17,478.32	4.000	EA	N	INLETS, CURB, TYPE P-5, PARTIAL
0425 1361	1	$5,400.00	$10,800.00	2.000	EA	N	INLETS, CURB, TYPE P-6, <10'
0425 1421	1	$13,400.00	$13,400.00	1.000	EA	N	INLETS, CURB, TYPE J-2, <10'
0425 1429	1	$13,500.00	$13,500.00	1.000	EA	N	INLETS, CURB, TYPE J-2, MODIFY
0425 1455	1	$4,369.58	$17,478.32	4.000	EA	N	INLETS, CURB, TYPE J-5, PARTIAL
0425 1465	1	$4,970.06	$9,940.12	2.000	EA	N	INLETS, CURB, TYPE J-6, PARTIAL
0425 1471	1	$9,200.00	$9,200.00	1.000	EA	N	INLETS, CURB, TYPE 7, <10'
0425 1501	1	$3,650.00	$7,300.00	2.000	EA	N	INLETS, DT BOT, TYPE A, <10'
0425 1505	1	$3,350.00	$3,350.00	1.000	EA	N	INLETS, DT BOT, TYPE A, PARTIAL
0425 1511	2	$6,835.00	$34,175.00	5.000	EA	N	INLETS, DT BOT, TYPE B, <10'
0425 1515	2	$6,187.50	$24,750.00	4.000	EA	N	INLETS, DT BOT, TYPE B, PARTIAL
0425 1521	2	$4,355.00	$87,100.00	20.000	EA	N	INLETS, DT BOT, TYPE C,<10'
0425 1525	1	$3,400.00	$3,400.00	1.000	EA	N	INLETS, DITCH BOTTOM, TYPE C, PARTIAL
0425 1531	3	$3,185.41	$79,635.32	25.000	EA	N	INLETS, DITCH BOTTOM, TYPE C MODIFIED- BACK OF SIDEWALK, <10'
0425 1535	1	$4,867.17	$4,867.17	1.000	EA	N	INLETS, DITCH BOTTOM TYPE C MODIFIED- BACK OF SIDEWALK, PARTIAL
0425 1541	2	$3,196.83	$262,140.00	82.000	EA	N	INLETS, DT BOT, TYPE D, <10'
0425 1543	2	$5,757.33	$86,360.00	15.000	EA	N	INLETS, DITCH BOTTOM, TYPE D, J BOT, <10'
0425 1545	1	$5,800.00	$5,800.00	1.000	EA	N	INLETS, DITCH BOTTOM, TYPE D, PARTIAL
0425 1549	2	$3,854.00	$57,810.00	15.000	EA	N	INLETS, DT BOT, TYPE D, MODIFY

Closure IV. 6 - Control Structure

Florida Department of Transportation
Item Average Unit Cost
From 2020/05/01 to 2021/04/30
Statewide

Contract Type: CC
Displaying: VALID ITEMS WITH HITS
From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0430174115	8	$172.55	$25,019.43	145.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 15"SD
0430174118	30	$71.69	$764,694.17	10,666.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 18"SD
0430174124	19	$91.76	$299,147.74	3,260.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 24"SD
0430174130	7	$125.06	$103,550.29	828.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 30"SD
0430174136	6	$111.75	$112,528.20	1,007.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 36"SD
0430174142	2	$122.10	$55,676.00	456.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 42"SD
0430174148	1	$233.21	$6,996.30	30.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 48"SD
0430174215	2	$119.79	$9,103.92	76.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 15"SD
0430174218	20	$84.06	$325,903.87	3,877.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 18"SD
0430174224	9	$98.68	$273,152.15	2,768.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 24"SD
0430174230	3	$115.38	$110,076.70	954.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 30"SD
0430174236	2	$136.49	$30,845.85	226.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 36"SD
0430174242	1	$280.00	$37,520.00	134.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ELLIP/OTHER, 42"SD
0430175112	8	$85.23	$46,452.47	545.000	LF	N	PIPE CULVERT,OPTIONAL MATERIAL,ROUND, 12"S/CD
0430175115	35	$84.21	$595,691.48	7,074.000	LF	N	PIPE CULVERT,OPTIONAL MATERIAL,ROUND, 15"S/CD
0430175118	73	$76.05	$5,263,454.12	69,207.000	LF	N	PIPE CULVERT,OPTIONAL MATERIAL,ROUND, 18"S/CD

IV. Closure 6 - 12" Piping

Florida Department of Transportation

Item Average Unit Cost

From 2020/05/01 to 2021/04/30

Statewide

Market Area: 07

Contract Type: CC

Displaying: VALID ITEMS WITH HITS

From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0425 7	1	$500.47	$500.47	1.000	EA	N	MANHOLE COVER- REPLACE
0425 10	1	$2,100.00	$10,500.00	5.000	EA	N	YARD DRAIN
0425 15 41	1	$6,832.66	$6,832.66	1.000	EA	N	INLET TOP, REPAIR, CURB INLET
0425 82	1	$1,300.00	$1,300.00	1.000	EA	N	REPLACE GRATE
0430174115	2	$121.38	$4,490.88	37.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 15"SD
0430174118	5	$74.54	$264,974.86	3,555.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 18"SD
0430174124	3	$86.61	$91,025.56	1,051.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 24"SD
0430174130	2	$120.74	$62,180.00	515.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 30"SD
0430174136	1	$150.00	$33,900.00	226.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, ROUND, 36"SD
0430174218	3	$71.80	$166,586.00	2,320.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 18"SD
0430174224	3	$82.59	$102,575.00	1,242.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 24"SD
0430174230	1	$120.00	$70,560.00	588.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 30"SD
0430174236	1	$150.00	$19,050.00	127.000	LF	N	PIPE CULVERT, OPTIONAL MATERIAL, OTHER SHAPE - ELLIP/ARCH, 36"SD
0430175112	1	$500.00	$2,000.00	4.000	LF	N	PIPE CULVERT,OPTIONAL MATERIAL,ROUND, 12"S/CD
0430175115	5	$64.78	$197,454.65	3,048.000	LF	N	PIPE CULVERT,OPTIONAL MATERIAL,ROUND, 15"S/CD
0430175118	7	$70.84	$558,820.66	7,889.000	LF	N	PIPE CULVERT,OPTIONAL MATERIAL,ROUND, 18"S/CD

Closure IV. 6 - 18" Piping

Florida Department of Transportation

Item Average Unit Cost

From 2020/01/01 to 2020/12/31

Statewide

Contract Type: CC

Displaying: VALID ITEMS WITH HITS

From: 0102 1 To: 9999999

CL IV.9.1. & LTC V.11.1.

CL IV.9.2. & LTC V.11.2.

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0550 10220	13	$19.40	$660,489.27	34,044.000	LF	N	FENCING, TYPE B, 5.1-6.0', STANDARD
0550 10222	1	$20.99	$28,315.51	1,349.000	LF	N	FENCING, TYPE B, 5.1-6.0, W/ VINYL COATING
0550 10228	1	$25.25	$505.00	20.000	LF	N	FENCING, TYPE B, 5.1-6.0, RESET EXISTING
0550 10251	1	$25.00	$1,000.00	40.000	LF	N	FENCING, TYPE B, 8.1-10.0', WITH BARBED WIRE ATTMT
0550 10354	1	$99.00	$29,799.00	301.000	LF	N	FENCING, TYPE R, 8.1-10.0', WITH PARTIAL ENCLOSURE, TYPE R ONLY
0550 10410	1	$17.00	$4,080.00	240.000	LF	N	FENCING, WOOD FENCE, 0.0-5.0'
0550 10420	1	$60.00	$2,760.00	46.000	LF	N	FENCING, WOOD FENCE, 5.1-6.0'
0550 10918	1	$15.46	$3,756.78	243.000	LF	N	FENCING, SPECIAL TYPE, 0.0-5.0', RESET EXISTING
0550 60112	1	$2,925.00	$2,925.00	1.000	EA	N	FENCE GATE, TYPE A, SINGLE, 6.1-12.0' OPENING
0550 60122	1	$2,986.14	$2,986.14	1.000	EA	N	FENCE GATE, TYPE A, DOUBLE, 6.1-12.0' OPENING
0550 60211	2	$1,791.36	$3,582.71	2.000	EA	N	FENCE GATE, TYPE B, SINGLE, 0- 6.0' OPENING
0550 60212	1	$513.40	$513.40	1.000	EA	N	FENCE GATE, TYPE B, SINGLE, 6.1 - 12.0' OPENING
0550 60214	1	$2,900.00	$2,900.00	1.000	EA	N	FENCE GATE, TYPE B, SINGLE, 18.1-20.0' OPENING
0550 60222	4	$1,588.57	$7,942.86	5.000	EA	N	FENCE GATE, TYPE B, DOUBLE, 6.1-12.0' OPENING
0550 60223	1	$1,500.00	$3,000.00	2.000	EA	N	FENCE GATE, TYPE B, DOUBLE, 12.1-18.0' OPENING
0550 60232	1	$4,000.00	$8,000.00	2.000	EA	N	FENCE GATE, TYPE B, SLIDING/CANTILEVER, 6.1-12' OPENING
0550 60234	1	$4,119.15	$8,238.30	2.000	EA	N	FENCE GATE, TYPE B, SLIDING/CANTILEVER, 18.1-20.0' OPENING
0550 60422	1	$800.00	$800.00	1.000	EA	N	FENCE GATE, WOOD, DOUBLE, 6.1-12.0 FOOT OPENING
0550 60424	1	$2,650.00	$2,650.00	1.000	EA	N	FENCE GATE, WOOD, DOUBLE, 18.1-20.0 FOOT OPENING
0561 1	9	$1,837.87	$7,019,387.00	3,819.300	TN	N	COATING EXISTING STRUCTURAL STEEL
0561 2	5	$93.73	$670,365.00	7,152.000	SF	N	COATING EXISTING STRUCTURAL STEEL

Florida Department of Transportation
Item Average Unit Cost
From 2020/05/01 to 2021/04/30
Statewide

CL IV.9.3. & LTC V.11.3.

Contract Type: CC
Displaying: VALID ITEMS WITH HITS
From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0695 7131	2	$4,516.87	$22,584.36	5.000	EA	N	TRAFFIC MONITORING SITE CABINET, FURNISH & INSTALL, TYPE 3, BASE MOUNT
0695 7132	7	$4,672.36	$74,757.83	16.000	EA	N	TRAFFIC MONITORING SITE CABINET, FURNISH & INSTALL, TYPE 3, PEDESTAL MOUNT
0695 7141	2	$5,762.90	$11,525.79	2.000	EA	N	TRAFFIC MONITORING SITE CABINET, FURNISH & INSTALL, TYPE 4, BASE MOUNT
0695 7162	13	$5,830.95	$110,788.10	19.000	EA	N	TRAFFIC MONITORING SITE CABINET, F&I, TYPE 3, 2 PANE BACK, PEDESTAL MOUNT
0695 7163	1	$7,233.00	$7,233.00	1.000	EA	N	TRAFFIC MONITORING SITE CABINET, FURNISH & INSTALL, TYPE 3, 2 PLANE BACK, POLEMOUNT
0695 7171	1	$4,750.00	$9,500.00	2.000	EA	N	TRAFFIC MONITORING SITE CABINET, FURNISH & INSTALL, TYPE 4, 2 PLANE BACK, BASE MOUNT
0695 7600	23	$646.72	$31,689.12	49.000	EA	N	TRAFFIC MONITORING SITE CABINET, REMOVE EXISTING CABINET
0695 8 11	1	$3,500.00	$3,500.00	1.000	EA	N	TRAFFIC MONITORING SITE COMMUNICATIONS MODEM FURNISH & INSTALL
0700 1 11	135	$340.94	$1,627,283.39	4,773.000	AS	N	SINGLE POST SIGN, F&I GROUND MOUNT, UP TO 12 SF
0700 1 12	98	$1,132.00	$1,644,800.35	1,453.000	AS	N	SINGLE POST SIGN, F&I GROUND MOUNT, 12-20 SF
0700 1 13	51	$1,567.73	$561,247.10	358.000	AS	N	SINGLE POST SIGN, F&I GROUND MOUNT, 21-30 SF
0700 1 14	12	$2,158.37	$60,434.39	28.000	AS	N	SINGLE POST SIGN, F&I GROUND MOUNT, 31+ SF
0700 1 21	5	$1,672.03	$66,881.08	40.000	AS	N	SINGLE POST SIGN, F&I BARRIER MOUNT INDEX 11871/700-013 UP TO 12 SF

Closure Item IV.11 - Liner CQA

All quotes received by 3:00 PM on July 16, 2019

Quote Tabulation

Quote Item	Item	ASTM Method	Terracon Consultants, Inc. Jacksonville, FL Shane Whitter (904) 900-6494 Quantity	Unit Price	Total Price	GSE Engineering & Consulting, Inc. Gainesville, FL Kenneth Hill, P.E. (352) 377-3233 Quantity	Unit Price	Total Price	Cal-Tech Testing, Inc. Jacksonville, FL Mike Stalvey, Jr. (386) 755-3633 Quantity	Unit Price	Total Price
A. Soil and Gravel Tests											
1	**Modified Proctor**	ASTM D1557	3	$ 125.00	$ 375.00	3	$ 90.00	$ 270.00	3	$ 115.00	$ 345.00
2	**Permeability**	ASTM D2434	1	$ 250.00	$ 250.00	1	$ 200.00	$ 200.00	1	$ 300.00	$ 300.00
3	**USCS Soil Classification**	ASTM D2487	3	$ 50.00	$ 150.00	3	$ 135.00	$ 405.00	3	$ 50.00	$ 150.00
4	**Organic Content**	ASTM D2974	4	$ 60.00	$ 240.00	4	$ 50.00	$ 200.00	4	$ 50.00	$ 200.00
5	**Gradation**	ASTM D488	1	$ 75.00	$ 75.00	1	$ 65.00	$ 65.00	1	$ 75.00	$ 75.00
6	**Carbonate Content**	ASTM D4373	4	$ 300.00	$ 1,200.00	4	$ 125.00	$ 500.00	4	$ 125.00	$ 500.00
7	**Density and Moisture Content**	ASTM D6938	170	$ 12.00	$ 2,040.00	170	$ 22.00	$ 3,740.00	170	$ 28.00	$ 4,760.00
8	**Synthetic Precipitation Leaching Procedure**	SW-846 Test Method 1312	1	$ 240.00	$ 240.00	1	$ 500.00	$ 500.00	1	$ 300.00	$ 300.00
9	**Thickness**	-	84	$ 10.00	$ 840.00	84	$ 15.00	$ 1,260.00	84	$ 20.00	$ 1,680.00
	SUBTOTAL TESTING				$ 5,410.00			$ 7,140.00			$ 8,310.00
B. Other Direct Costs											
	Travel Cost		34	$ 70.00	$ 2,380.00	70	$ 150.00	$ 10,500.00	165	$ 100.00	$ 16,500.00
	Engineering Review		5	$ 135.00	$ 675.00	0	$ -	$ -	60	$ 110.00	$ 6,600.00
	Adminstrative/Reporting		13	$ 75.00	$ 975.00	0	$ -	$ -	80	$ 48.00	$ 3,840.00
	Hourly Rate for Standby time Greater than 1/2 Hour		15	$ 55.00	$ 825.00	10	$ 45.00	$ 450.00	165	$ 50.00	$ 8,250.00
	SUBTOTAL OTHER DIRECT COSTS				$ 4,855.00			$ 10,950.00			$ 35,190.00
C. Allowance for Retesting											
	Allowance		-	-	$ 1,000.00	-	-	$ 1,000.00	-	-	$ 1,000.00
	TOTAL BASE BID				**$ 11,265.00**			**$ 19,090.00**			**$ 44,500.00**

Total QA Testing per Acre:
$590.84 + $1663.44 = $2,254.28
Inflation factor July 2019 to April 2021 = 1.041
$2,254.28 * 1.041 = $2346.71

Rounded to $2,400

Liner CQA	(15 acres)		
TRI	2019	$	9,800.00
Groupe CTT	2019	$	11,108.00
Geotesting	2019	$	5,680.00
	avg	$	8,862.667
	per acre	$	590.84

Closure Item IV. 11 - Soil CQA

New River Solid Waste Association
New River Regional Landfill
EGC Closure - Soil Materials Testing Services

All quotes received by 3:00 PM on July 16, 2019

Quote Tabulation

Quote Item	Item	ASTM Method	Terracon Consultants, Inc. Jacksonville, FL Shane Whitter (904) 900-6494 Quantity	Unit Price	Total Price	GSE Engineering & Consulting, Inc. Gainesville, FL Kenneth Hill, P.E. (352) 377-3233 Quantity	Unit Price	Total Price	Cal-Tech Testing, Inc. Jacksonville, FL Mike Stalvey, Jr. (386) 755-3633 Quantity	Unit Price	Total Price
A. Soil and Gravel Tests											
1	**Modified Proctor**	ASTM D1557	3	$ 125.00	$ 375.00	3	$ 90.00	$ 270.00	3	$ 115.00	$ 345.00
2	**Permeability**	ASTM D2434	1	$ 250.00	$ 250.00	1	$ 200.00	$ 200.00	1	$ 300.00	$ 300.00
3	**USCS Soil Classification**	ASTM D2487	3	$ 50.00	$ 150.00	3	$ 135.00	$ 405.00	3	$ 50.00	$ 150.00
4	**Organic Content**	ASTM D2974	4	$ 60.00	$ 240.00	4	$ 50.00	$ 200.00	4	$ 50.00	$ 200.00
5	**Gradation**	ASTM D488	1	$ 75.00	$ 75.00	1	$ 65.00	$ 65.00	1	$ 75.00	$ 75.00
6	**Carbonate Content**	ASTM D4373	4	$ 300.00	$ 1,200.00	4	$ 125.00	$ 500.00	4	$ 125.00	$ 500.00
7	**Density and Moisture Content**	ASTM D6938	170	$ 12.00	$ 2,040.00	170	$ 22.00	$ 3,740.00	170	$ 28.00	$ 4,760.00
8	**Synthetic Precipitation Leaching Procedure**	SW-846 Test Method 1312	1	$ 240.00	$ 240.00	1	$ 500.00	$ 500.00	1	$ 300.00	$ 300.00
9	**Thickness**	-	84	$ 10.00	$ 840.00	84	$ 15.00	$ 1,260.00	84	$ 20.00	$ 1,680.00
	SUBTOTAL TESTING				$ 5,410.00			$ 7,140.00			$ 8,310.00
B. Other Direct Costs											
	Travel Cost		34	$ 70.00	$ 2,380.00	70	$ 150.00	$ 10,500.00	165	$ 100.00	$ 16,500.00
	Engineering Review		5	$ 135.00	$ 675.00	0	$ -	$ -	60	$ 110.00	$ 6,600.00
	Adminstrative/Reporting		13	$ 75.00	$ 975.00	0	$ -	$ -	80	$ 48.00	$ 3,840.00
	Hourly Rate for Standby time Greater than 1/2 Hour		15	$ 55.00	$ 825.00	10	$ 45.00	$ 450.00	165	$ 50.00	$ 8,250.00
	SUBTOTAL OTHER DIRECT COSTS				$ 4,855.00			$ 10,950.00			$ 35,190.00
C. Allowance for Retesting											
	Allowance		-	-	$ 1,000.00	-	-	$ 1,000.00	-	-	$ 1,000.00
	TOTAL BASE BID				**$ 11,265.00**			**$ 19,090.00**			**$ 44,500.00**

Soil CQA	(15 acres)		
Terracon	2019	$	11,265.00
GSE	2019	$	19,090.00
Cal-TECH	2019	$	44,500.00
	avg	$	24,951.67
	per acre	$	1,663.44

FLORIDA JETCLEAN

Long-Term Care V.5.1 and V.5.2
Collection Pipes and Sumps/Traps Maintenance

HIGH PRESSURE WATER JETTING – EXPLOSION PROOF INSPECTION
PIPE LOCATING – NO DIG REPAIRS – VACUUM TRUCK SERVICES

7538 Dunbridge Drive
Odessa, FL 33556
www.floridajetclean.com

TEL : 800-226-8013
FAX : 813-926-4616

PROPOSAL

DATE : 12/6/2019
TO : Matthew Morse - Jones Edmunds
FROM : Ralph Calistri (floridajetclean@yahoo.com)
SUBJECT : 2019 - Lee/Hendry Landfill - Financial Assurance Jetcleaning Estimate

Thank you for your inquiry. We confirm our capability and interest in performing these jetcleaning services for Jones Edmunds at the Lee/Hendry Landfill.

Based on your email and our prior work at this location, we quote as follows:

High-pressure water-jetting of an estimated 10,387 LF of existing leachate collection piping $ 7,790.25

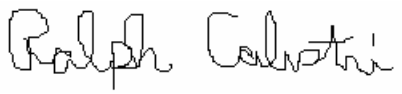
Vacuum Truck for Vacuum Removal of Silting and Solids From Sump Areas and/or Pump Stations $ 2,500.00 / day

Maintenance/Cleaning: $7,790.25 / 10,387 ft = $0.75/ft

Subject to:

- An adequate no-charge on site water supply for jetcleaning via hydrant or other high flow source.
- 2 wheel drive access to within 10’ of all system access points.
- Exposed and opened cleanouts at ground level.
- Pricing is unrelated to actual or achieved footages but on the number of setups required and the time we anticipate being on site.
- Pipes affected by heavy non-routine silting or hardened blockages will require additional equipment or extended jetting/vacuum truck time, at additional expense.
- Day-rate pricing for jet-vac truck covers up to 8 hours, and includes all drive time, standby time, water fill time, and disposal time.
- No charge on site dumping of liquids and solids loaded onto vacuum truck.
- Payment : net 30 days

Regards,



Ralph Calistri – Florida Jetclean - 800-226-8013

Preferred Drilling Solutions, Inc.
11747 87th St. North, La
Ph: 727-561-7477 Fa
www.pdsflorida.com

LTC Item V.6 - GW Well Maintenance
Replacement Well

Contractor Name: JonesEdmunds

Site Name & Location: Walton County Central Landfill
Near Defuniak Springs, FL

Date: 8/20/19

FAC ID#:

PROPOSED SCOPE OF WORK:

(1) 2" x 60' MW with 10' .010 screen, no soil sampling, soread cuttings, pad and AGP

4 Concrete filled bollards painted safety yellow

PDS Assumes location is truck accessible

**Note: It is against PDS policy to drill in Pea Gravel. Borings in Pea Gravel will be terminated or can proceed solely at the risk of the consultant, DEP or Property Owner*

DRILLING	Unit	Unit Rate	Number of Units	Extended Price
Rig Type: Auger/Mud Rotary_____ Sonic_____ Other_____				
Split Spoon Collection (continuous or 5' intervals) (can be used in conjunction with well installation) (includes decon)				
<50 foot boring depth	per foot			$0.00
50 foot to 100 foot boring depth	per foot			$0.00
>100 foot boring depth	per foot			$0.00
Borehole Grouting				
4 - inch borehole diameter	per foot			$0.00
6 - inch borehole diameter	per foot			$0.00
8 - inch borehole diameter	per foot			$0.00
1" - 2" Well Installation (includes steamcleaning decon, screen, riser, sand pack, seal and grout)				
<50 foot boring depth	per foot	$34.75	60	$2,085.00
50 foot to 100 foot boring depth	per foot			$0.00
>100 foot boring depth	per foot			$0.00
4" Well Installation (includes steamcleaning decon, screen, riser, sand pack, seal and grout)				
<50 foot boring depth	per foot			$0.00
50 foot to 100 foot boring depth	per foot			$0.00
>100 foot boring depth	per foot			$0.00
Recovery Well Diameter: 4-6"	per foot			$0.00
Double Cased Wells				$0.00
6" Surface Casing	per foot			$0.00
8" Surface Casing	per foot			$0.00
Well Completion (includes AGP w/ cover, concrete pad, locking well cap, and saw/jackhammer prep.)	per well	$250.00	1	$250.00
1"- 2" Well Abandonment (includes grouting)	per foot			$0.00
3"- 4" Well Abandonment (includes grouting)	per foot			$0.00
5"- 6" Well Abandonment (includes grouting)	per foot			$0.00
2' x 2' Well Pad Removal and Patch	each			$0.00
MISCELLANEOUS				
Mobilization	roundtrip	$775.00	1	$775.00
Per Diem	per crew	$300.00		$0.00
DOT Approved 55-gal Drum	each	$50.00		$0.00
Permits (Drilling or abandonment only)	each	$30.00		$0.00
Well Development Time (30 minutes per well)	each	$75.00		$0.00
Concrete filled 4" bollard painted yellow	each	$100.00	4	$400.00
				$0.00
Additional Development Time	per hour	$150.00		$0.00
Additional Decontamination Time	per hour			$0.00
Standby/Delay/Difficult Access Time	per hour	$300.00		$0.00

Days to Complete Scope of Work: 1

Signature and Title of Person Submitting Quote: _____

Cost per Well: *(without mobilization)*
($34.75/LF)(60 LF)+($250)+($30)+($75)+($100)(4) = $2,840/Well

Cost per Day: *(assumes 1 day of work for 1 crew)*
($775/roundtrip)+($300/crew) = $1,075/Day

Cost per Well: *(including mobilization)*
[($2,840/Well)(1 Well)+($1,075)]/(1 Well) = $3,915/Well

Preferred Drilling Solutions, Inc.
11747 87th St. North, Larg
Ph: 727-561-7477 Fax:
www.pdsflorida.

LTC Item V.6 - GW Well Maintenance Abandonment

Contractor Name: JonesEdmunds

Site Name & Location: Walton County Central Landfill
Near Defuniak Springs, FL

Date: 8/22/19

FAC ID#:

PROPOSED SCOPE OF WORK:

Abandon (2) 4" x 20' gas vent wells with bentonite hole plug, cap with grout
Abandon (1) 2" x 60' MW with cement bentonite grout from the bottom up'
Pad and manhole removal as needed

**Note: It is against PDS policy to drill in Pea Gravel. Borings in Pea Gravel will be terminated or can proceed solely at the risk of the consultant, DEP or Property Owner*

DRILLING	Unit	Unit Rate	Number of Units	Extended Price
Rig Type: Auger/Mud Rotary_____ Sonic_____ Other_____				
Split Spoon Collection (continuous or 5' intervals) (can be used in conjunction with well installation) (includes decon)				
<50 foot boring depth	per foot			$0.00
50 foot to 100 foot boring depth	per foot			$0.00
>100 foot boring depth	per foot			$0.00
Borehole Grouting				
4 - inch borehole diameter	per foot			$0.00
6 - inch borehole diameter	per foot			$0.00
8 - inch borehole diameter	per foot			$0.00
1" - 2" Well Installation (includes steamcleaning decon, screen, riser, sand pack, seal and grout)				
<50 foot boring depth	per foot			$0.00
50 foot to 100 foot boring depth	per foot			$0.00
>100 foot boring depth	per foot			$0.00
4" Well Installation (includes steamcleaning decon, screen, riser, sand pack, seal and grout)				
<50 foot boring depth	per foot			$0.00
50 foot to 100 foot boring depth	per foot			$0.00
>100 foot boring depth	per foot			$0.00
Recovery Well Diameter: 4-6"	per foot			$0.00
Double Cased Wells				$0.00
6" Surface Casing	per foot			$0.00
8" Surface Casing	per foot			$0.00
Well Completion (includes AGP w/ cover, concrete pad, locking well cap, and saw/jackhammer prep.)	per well			$0.00
1"- 2" Well Abandonment (includes grouting)	per foot	$8.00	60	$480.00
3"- 4" Well Abandonment (includes grouting)	per foot	$20.00	40	$800.00
5"- 6" Well Abandonment (includes grouting)	per foot			$0.00
2' x 2' Well Pad Removal and Patch	each	$150.00	1	$150.00
MISCELLANEOUS				
Mobilization	roundtrip	$575.00	1	$575.00
Per Diem	per crew	$300.00		$0.00
DOT Approved 55-gal Drum	each	$50.00		$0.00
Permits (Drilling or abandonment only)	each	$50.00	1	$50.00
Well Development Time (30 minutes per well)	each	$75.00		$0.00
Concrete filled 4" bollard painted yellow	each	$100.00		$0.00
				$0.00
Additional Development Time	per hour	$150.00		$0.00
Additional Decontamination Time	per hour			$0.00
Standby/Delay/Difficult Access Time	per hour	$300.00		$0.00

Assume mobilization is covered in Well Installation Quote
Cost per Well: *(without mobilization)*
($8/LF)(60LF)+($150)+($50) = $680/Well

Days to Complete Scope of Work: 1

Signature and Title of Person Submitting Quote: _____

Florida Department of Transportation

Item Average Unit Cost

From 2020/03/01 to 2021/02/28

Statewide

Market Area: 07

Contract Type: CC

Displaying: VALID ITEMS WITH HITS

From: 0102 1 To: 9999999

Item	No. of Conts	Weighted Average	Total Amount	Total Quantity	Unit Meas	Obs?	Description
0102 99	14	$10.86	$166,016.90	15,294.000	ED	N	PORTABLE CHANGEABLE MESSAGE SIGN, TEMPORARY
0102104	10	$19.80	$325,132.60	16,421.000	ED	N	TEMPORARY SIGNALIZATION AND MAINTENANCE, INTERSECTION
0102107 1	10	$8.76	$145,623.10	16,628.000	ED	N	TEMPORARY TRAFFIC DETECTION AND MAINTENANCE, INTERSECTION
0102115	4	$.37	$11,171.82	30,094.000	ED	N	TYPE III BARRICADE
0102150 1	4	$5.45	$17,670.00	3,240.000	ED	N	PORTABLE REGULATORY, SIGN
0102150 2	6	$5.36	$20,270.00	3,784.000	ED	N	RADAR SPEED DISPLAY UNIT
0102909	3	$23.75	$13,893.00	585.000	DA	N	TEMPORARY RAISED RUMBLE STRIPS- PER DAY, INCLUDES ALL SETS AND RELOCATIONS
0104 1	3	$2.54	$19,407.50	7,628.000	SY	N	ARTIFICIAL COVERINGS /ROLLED EROSION CONTROL PRODUCTS
0104 7	1	$225.00	$900.00	4.000	EA	N	SEDIMENT BASIN / CONTAINMENT SYSTEM
0104 9	1	$1,125.00	$4,500.00	4.000	EA	N	SEDIMENT BASIN / CONTAINMENT SYSTEM- CLEANOUT
0104 10 3	12	$1.60	$245,189.85	153,221.000	LF	N	SEDIMENT BARRIER
0104 11	3	$19.47	$21,905.50	1,125.000	LF	N	FLOATING TURBIDITY BARRIER
0104 12	5	$11.92	$120,543.00	10,111.000	LF	N	STAKED TURBIDITY BARRIER- NYLON REINFORCED PVC
0104 15	7	$2,643.14	$76,650.98	29.000	EA	N	SOIL TRACKING PREVENTION DEVICE
0104 18	13	$81.57	$94,702.84	1,161.000	EA	N	INLET PROTECTION SYSTEM
0104 19	3	$1.65	$12,593.75	7,628.000	SY	N	CHEMICAL TREATMENT- POWDERED, FOR EROSION CONTROL
0107 1	10	$39.36	$338,348.87	8,597.300	AC	N	LITTER REMOVAL
0107 2	10	$48.60	$367,223.64	7,556.170	AC	N	MOWING

LTC V.8. - Mowing

Cost Estimate Report

Date: 03/30/2021

Pasco County Ash Cells Vert Expansion

Year 2021

Unit Detail Report

Prepared By: JoAnne Talamo Jones Edmunds & Associates, Inc.

LTC V.9 - Regrading

LineNumber	Description	Quantity	Unit	Total Incl. O&P	Ext. Total Incl. O&P
Division 31 Earthwork					
312216103312	Fine grading, slopes, steep, large quantities, finish grading	0.00	M.S.F.	$30.88	$0.00
Division 31 Earthwork Subtotal					**$0.00**
Subtotal					**$0.00**
General Contractor's Markup on Subs					**$0.00**
Subtotal					**$0.00**
General Conditions					**$0.00**
Subtotal					**$0.00**
General Contractor's Overhead and Profit				**0.00%**	**$0.00**
Grand Total					**$0.00**

Cost = ($30.88/MSF)*(9SF/SY)*(1/1000)

= $0.28/SY

LTC Item V.9.3 - Liner Repair

Schedule of Values

Slurry Wall Landfill Phase VI Sequential Closure (B2-19-01)

Brevard County Central Disposal Facility *Cost Items based on costs associated with a nearby landfill

	Description	Total Schedule of Values									
		Unit	Quantity	Unit Price ($)	Value ($)	Quantity	Unit Price ($)	Quantity	Unit Price ($)	Quantity	Unit Price ($)
1. General Conditions and Indemnification											
A	Mobilization	LS	1	$ 500,000.00	$ 412,500.00		$ -		$ -	0	$ -
B	Demobilization	LS	1	$ 50,000.00	$ 137,500.00		$ -		$ -	0	$ -
C	Bonds and Insurance	LS	1	$ 127,010.00	$ 127,010.00		$ -		$ -	0	$ -
D	Indemnification	LS	1	$ 1,000.00	$ 1,000.00				$ -	0	$ -
	Subtotal - General Conditions and Indemnification				**$ 678,010.00**				**$ -**		**$ -**
2. Closure Cover System Installation											
A	Survey during construction (Layout, Layer Survey, As-built & Certification)	LS	1	$ 98,551.00	$ 98,551.00				$ -	0	$ -
B	Erosion Protection during construction on landfill	LS	1	$ 125,000.00	$ 125,000.00		$ -		$ -	0	$ -
C	Site preparation and clearing	LS	1	$ 300,000.00	$ 300,000.00		$ -		$ -	0	$ -
D	Exposing existing anchor trench	LS	1	$ 186,800.00	$ 186,800.00				$ -	0	$ -
E	40-mil LLDPE geomembrane	SF	1,408,335	$ 0.62	$ 873,167.70				$ -	0	$ -
F	Composite Drainage Net	SF	1,408,335	$ 0.82	$ 1,154,834.70				$ -	0	$ -
G	Sodding	SY	156,482	$ 2.10	$ 328,612.20				$ -	0	$ -
H	Toe Drains and Discharge Pipes, Barrier Layer Transition	LS	1	$ 130,000.00	$ 130,000.00				$ -	0	$ -
UNIT PRICE ITEMS										0	$ -
I	Solid waste grading/waste contouring/sub-base preparation	CY	65,000	$ 4.25	$ 276,250.00		$ -		$ -	0	$ -
J	Clean Granular Fill (6-inch Leveling Course)	CY	30,000	$ 25.00	$ 750,000.00		$ -		$ -	0	$ -
K	Clean Granular Fill (24-inch Protective Cover)	CY	170,000	$ 25.00	$ 4,250,000.00		$ -		$ -	0	$ -
L	Repair and Replacement of existing GCL at stormwater inlet transition and LFG tie-ins (Contractor providing GCL Material)	SF	10,000	$ 5.50	$ 55,000.00		$ -		$ -	0	$ -
M	Replacement of existing GCL at Southwest sideslope (County providing GCL material)	SF	12,000	$ 3.20	$ 38,400.00		$ -		$ -	0	$ -
	Subtotal - Closure Cover System				**$ 8,566,615.60**		**$ -**		**$ -**		**$ -**
3. Stormwater Management System Piping and Structures											
A	HDPE piping-36 inch piping pipe trench & backfill, connect to structure	LF	1,410	$ 108.00	$ 152,280.00		$ -		$ -	0	$ -
B	HDPE piping 30 inch piping, pipe trench & backfill, connect to structure	LF	1,325	$ 100.00	$ 132,500.00		$ -		$ -	0	$ -
C	HDPE piping 24 inch piping, pipe trench & backfill, connect to structure	LF	560	$ 84.90	$ 47,544.00		$ -		$ -	0	$ -
D	Seepage flap and seepage barrier near existing inlets	EA	10	$ 900.00	$ 9,000.00		$ -		$ -	0	$ -
E	36-inch dia. HDPE inlets structures installed complete with FFCRS	EA	18	$ 10,500.00	$ 189,000.00		$ -		$ -	0	$ -
F	Terrace Underdrain System	LF	7,760	$ 38.00	$ 294,880.00		$ -		$ -	0	$ -
G	Restoration of Interface 30-ft wide terrace, swale shaping, grading & sodding	LS	1	$ 75,000.00	$ 75,000.00		$ -		$ -	0	$ -
	Subtotal-Secondary Stormwater Management System				**$ 900,204.00**		**$ -**		**$ -**		**$ -**
4. LFG Mains and Appurtenances LFG Wells											
A	LFG wellhead with all piping, fernco coupling & fittings	EA	32	$ 2,750.00	$ 88,000.00		$ -		$ -	0	$ -
B	LFG header pipe 16-inch diameter HDPE SDR 17	LF	2,730	$ 50.00	$ 136,500.00		$ -		$ -	0	$ -
C	LFG header pipe-12-inch diameter HDPE SDR 11	LF	80	$ 115.00	$ 9,200.00		$ -		$ -	0	$ -
D	LFG header pipe-10-inch diameter HDPE SDR 12	LF	2600	$ 45.00	$ 117,000.00		$ -		$ -	0	$ -
E	LFG header pipe-8-inch diameter HDPE SDR 11	LF	200	$ 35.00	$ 7,000.00		$ -		$ -	0	$ -
F	LFG header or lateral pipe-6-inch diameter HDPE SDR 11	LF	994	$ 25.00	$ 24,850.00		$ -		$ -	0	$ -
G	LFG lateral-4-inch diameter HDPE SDR 11	LF	3,260	$ 20.00	$ 65,200.00		$ -		$ -	0	$ -
H	Tie-in to existing Active LFG System	EA	6	$ 3,000.00	$ 18,000.00		$ -		$ -	0	$ -
I	LFG 16-inch valves	EA	2	$ 10,000.00	$ 20,000.00		$ -		$ -	0	$ -
J	LFG 10-inch valves	EA	2	$ 9,000.00	$ 18,000.00		$ -		$ -	0	$ -
K	Air supply piping-2 inch diameter HDPE Tube	LF	6,525	$ 2.16	$ 14,094.00		$ -		$ -	0	$ -
L	Air supply Valves	EA	5	$ 740.00	$ 3,700.00		$ -		$ -	0	$ -
M	Existing LFG Header Relocation, Repair and Replumbing Including Valves - SW sideslope	LS	1	$ 130,000.00	$ 130,000.00		$ -		$ -	0	$ -
UNIT PRICE ITEMS											
N	LFG Wells	LF	1,850	$ 165.00	$ 305,250.00		$ -		$ -	0	$ -
O	LFG Well Borehead Abandonment (allowance)	LF	350	$ 55.00	$ 19,250.00		$ -		$ -	0	$ -
P	Wellhead Extension (Allowance)	EA	20	$ 400.00	$ 8,000.00		$ -		$ -	0	$ -
Q	Wellhead Replacement (Allowance)	EA	10	$ 1,800.00	$ 18,000.00		$ -		$ -	0	$ -
	Subtotal-LFG Well Field, Headers and Collection Piping Systems				**$ 1,002,044.00**		**$ -**		**$ -**		**$ -**
5. Miscellaneous (provide a list on additional sheet if necessary)											
6. Owner's Contingency					**$ 100,000.00**						
Total- Bid Amount (for parts 1 through 6)					**$ 11,246,873.60**		$ -		$ -		$ -

LTC Item V.9.3
$0.62/SF x 9SF/SY = $5.58/SY
$0.82/SF x 9SF/SY = ~~$7.38/SY~~
$12.96/SY

NS:
Mobilization and Demobilization Costs complies with the 5% of the project bid cost less owners Contingency. Twenty-five percent of this line item must be allocated to demobilization activities. Please revise items 1(A) and 1(B). -

Thalle:
Revised.

NS:
Schedule of Values does not include cost for four (4) valves that are required in the Southwest LFG header repair/replacement portion of the work. Please revise Item 4(I) and any other affected items resulting from this change.

Thalle:
The Value for the Southwest LFG header repair/replacement portion of the work is under item 4M including the valves, fittings, and other components.

Appendix A

Engineering Plans

PATH=Y:\16002-PASCOCOUNTY\PROJECTS\009-01-ASHMONOFILLOPERATIONSMODIFICATION\CAD\DWGS\GENERAL\16002009-G01.DWG

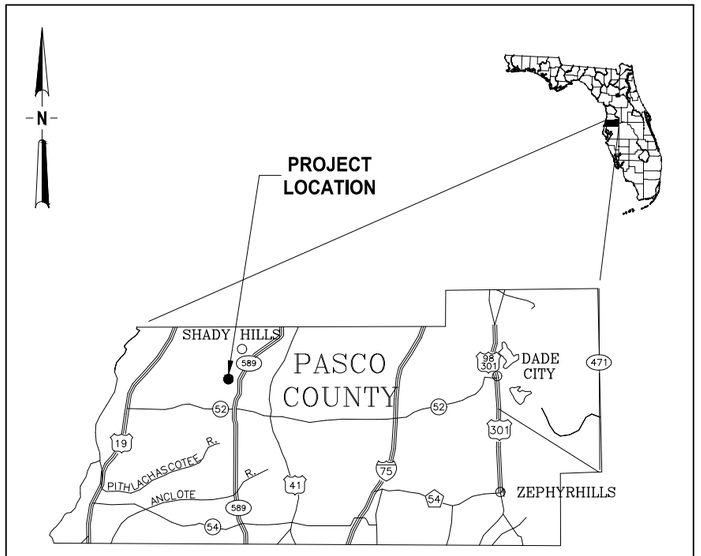
© Jones Edmunds 2234

22x34

WEST PASCO COUNTY LANDFILL

ASH MONOFILL OPERATIONS MODIFICATIONS

PASCO COUNTY, FLORIDA

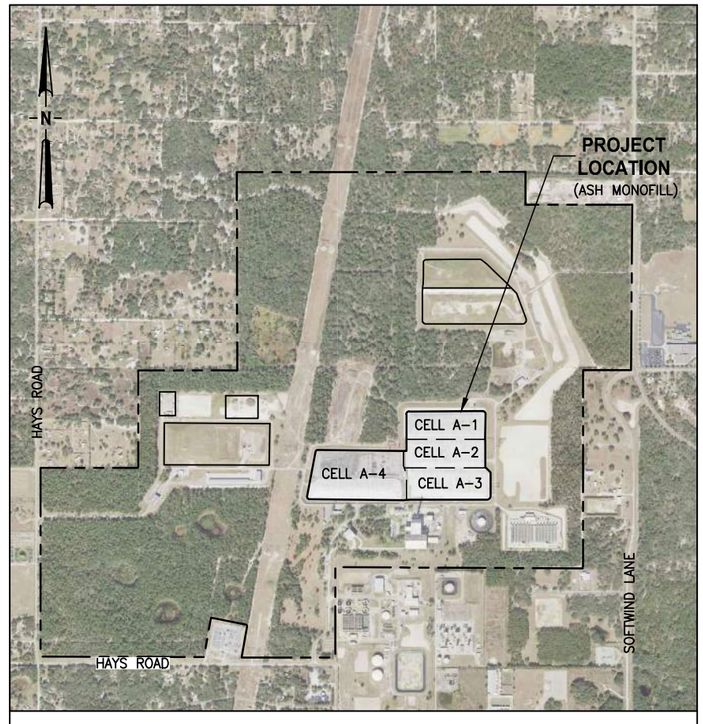


VICINITY MAP

SITE ADDRESS: 14230 HAYS ROAD, SPRING HILL, FLORIDA 34610
MAILING ADDRESS:

PREPARED BY:

JonesEdmunds
CERTIFICATE OF AUTHORIZATION #1841
730 NE WALDO ROAD, GAINESVILLE, FLORIDA 32641 / (352) 377-5821



LOCATION MAP

LAST SAVED: 7/9/2021 8:44 AMBY:PUPSTILL PLOTTED:7/9/2021 11:56 AMBY:PAUL E. UPSTILL

PROJECT No: 16002-009-01

Tobin S McKnight
2021.07.21 11:44:15-04'00'

TOBIN S. McKNIGHT
P.E. # 69187

THIS ITEM HAS BEEN DIGITALLY SIGNED AND SEALED BY TOBIN S. MCKNIGHT, PE. ON THE DATE ADJACENT TO THE SEAL.

PRINTED COPIES OF THIS DOCUMENT ARE NOT CONSIDERED SIGNED AND SEALED AND THE SIGNATURE MUST BE VERIFIED ON ANY ELECTRONIC COPIES.

TOBIN S. McKNIGHT LICENSE No. 69187 STATE OF FLORIDA PROFESSIONAL ENGINEER

JULY 2021

PERMIT

LEGEND

- PROPERTY BOUNDARY
- LIMITS OF LANDFILL CELL (APPROX)
- EXISTING ASH MONOFILL LIMITS
- LIMITS OF ASH MONOFILL CELL (APPROX)
- 107 — EXISTING CONTOUR
- × 108.25 — EXISTING GRADE SPOT ELEVATION
- X — EXISTING FENCE
- 107 — PROPOSED CONTOUR
- DROP INLET
- GMW-1 — EXISTING GAS MONITORING WELL
- 2MW5 — EXISTING GROUNDWATER MONITORING WELL
- B-3 — BORING
- GMP-2 / GMW-5 — EXISTING GAS MONITORING PROBE
- 102.0 * — GRADE SPOT ELEVATION
- STORMWATER/SURFACE WATER FLOW

GENERAL NOTES

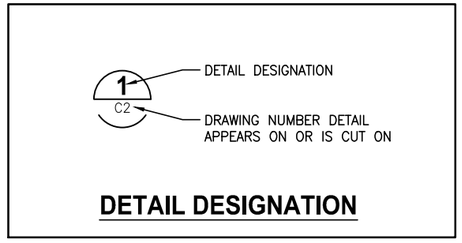
1. ALL ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM 1988 (NAVD 88), UNLESS OTHERWISE NOTED.
2. ELEVATIONS SURVEYED WITHIN THE LIMITS OF ASH CELL A-1 THROUGH A-4 WERE PROVIDED BY DC JOHNSON AND ASSOCIATES, DATED 1-8-2021.
3. ———, C/L, & ● ARE EXAMPLES OF DRAWING ELEMENTS WHICH HAVE BEEN SCREENED/SHADOWED TO INDICATE EXISTING CONDITIONS THAT WHERE PREVIOUSLY PERMITTED BY FDEP.

ABBREVIATIONS

ASTM	AMERICAN SOCIETY FOR TESTING AND MATERIALS
ø, DIA	DIAMETER
DWG	DRAWING
EL	ELEVATION
ELB/ELL	ELBOW
ETC	ET CETERA
EXIST	EXISTING
FDEP	FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
FDOT	FLORIDA DEPARTMENT OF TRANSPORTATION
GCCS	GAS COLLECTION AND CONTROL SYSTEM
GFFR	GROUT FILLED FIBER REVETMENT
HDPE	HIGH DENSITY POLYETHYLENE
HP	HIGH POINT
LCS	LEACHATE COLLECTION SYSTEM
LDS	LEACHATE DETECTION SYSTEM
LFG	LANDFILL GAS HEADER
LLDPE	LINEAR LOW DENSITY POLYETHYLENE
MAX	MAXIMUM
MIL	THOUSAND'S OF AN INCH
MIN	MINIMUM
NAVD	NORTH AMERICAN VERTICAL DATUM
NGVD	NATIONAL GEODETIC VERTICAL DATUM
No	NUMBER
NPT	AMERICAN STANDARD TAPER PIPE THREAD
NTS	NOT TO SCALE
OC	ON CENTER
OSHA	OCCUPATIONAL SAFETY & HEALTH ADMINISTRATION
PE	PLAIN END
PVC	POLYVINYL CHLORIDE
SCH	SCHEDULE
SDR	STANDARD DIMENSION RATIO
SPT	STANDARD PENETRATION TEST
SS	STAINLESS STEEL
TYP	TYPICAL
W/	WITH

DRAWING INDEX

DWG	DESCRIPTION
GENERAL	
G1	COVER
G2	LEGEND, NOTES, ABBREVIATIONS, AND DRAWING INDEX
CIVIL	
C1	SITE PLAN
C2	EXISTING WELL AND BORING SITE PLAN
C3	TOPOGRAPHIC SURVEY
C4	CONCEPTUAL FINAL COVER PLAN
C5	CONCEPTUAL FINAL COVER SECTIONS
C6	CONCEPTUAL FINAL COVER SECTIONS
C7	CONCEPTUAL FINAL COVER SECTIONS
C8	CONCEPTUAL FINAL COVER DETAILS



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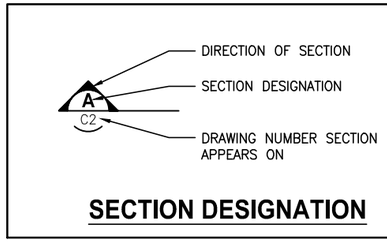

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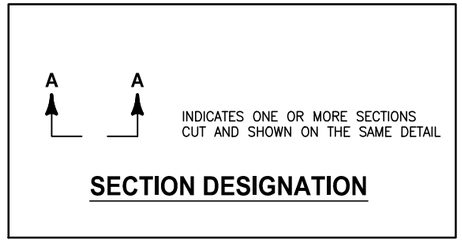

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WEST PASCO COUNTY LANDFILL
ASH MONOFILL OPERATIONS MODIFICATIONS
PASCO COUNTY, FLORIDA

LEGEND, NOTES, ABBREVIATIONS, AND DRAWING INDEX

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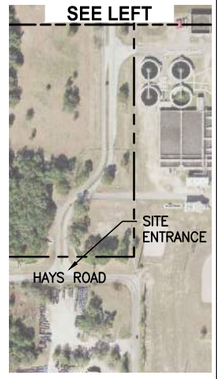

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SITE PLAN

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DWG NO: C1

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PASCO COUNTY, FLORIDA

EXISTING WELL AND BORING SITE PLAN

APPROVED BY
TOBIN S. MCKNIGHT
P.E. #69187

PROJECT NO: 16002-009-01
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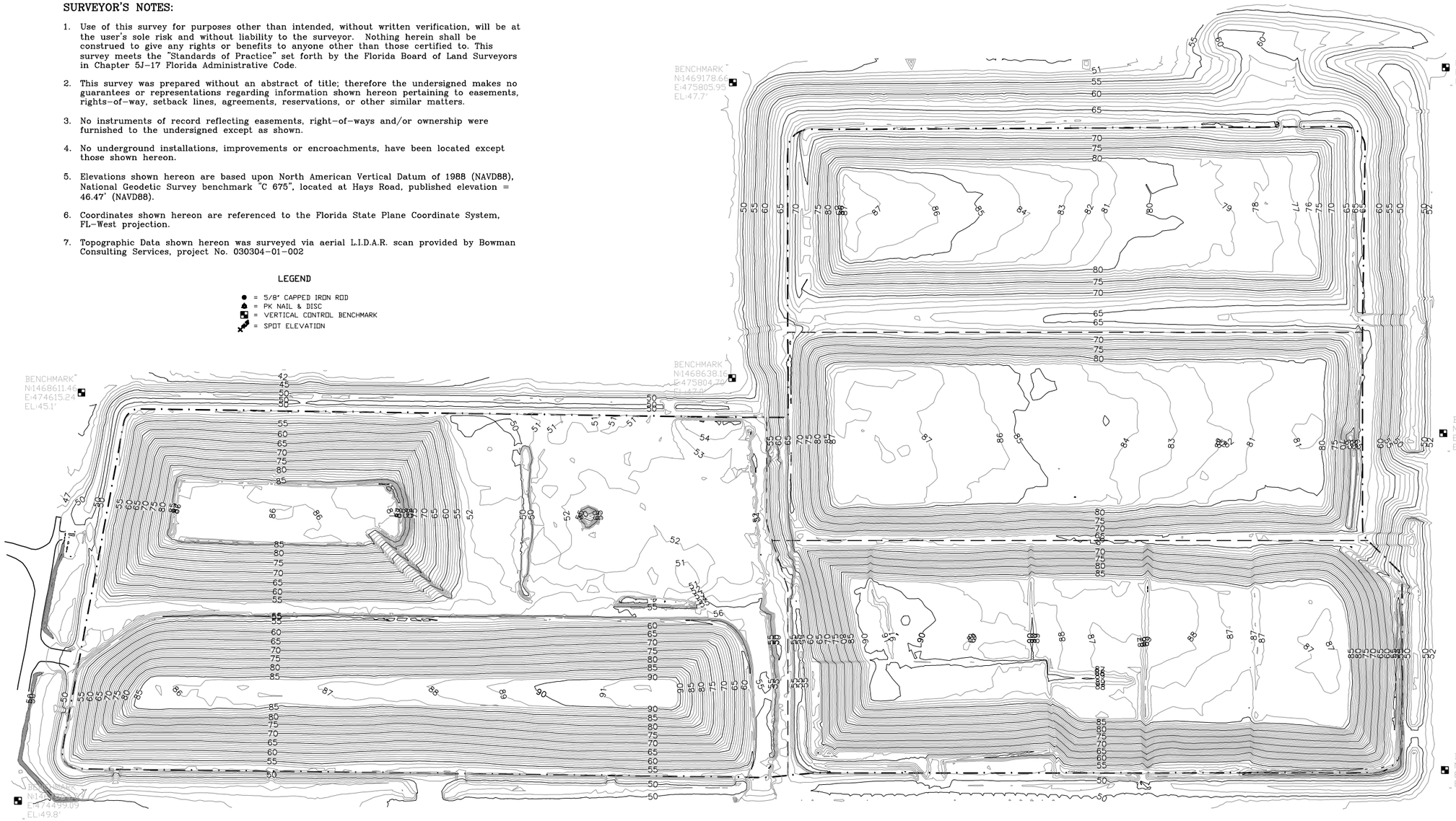
DWG NO: C2

SURVEYOR'S NOTES:

1. Use of this survey for purposes other than intended, without written verification, will be at the user's sole risk and without liability to the surveyor. Nothing herein shall be construed to give any rights or benefits to anyone other than those certified to. This survey meets the "Standards of Practice" set forth by the Florida Board of Land Surveyors in Chapter 5J-17 Florida Administrative Code.
2. This survey was prepared without an abstract of title; therefore the undersigned makes no guarantees or representations regarding information shown hereon pertaining to easements, rights-of-way, setback lines, agreements, reservations, or other similar matters.
3. No instruments of record reflecting easements, right-of-ways and/or ownership were furnished to the undersigned except as shown.
4. No underground installations, improvements or encroachments, have been located except those shown hereon.
5. Elevations shown hereon are based upon North American Vertical Datum of 1988 (NAVD88), National Geodetic Survey benchmark "C 675", located at Hays Road, published elevation = 46.47' (NAVD88).
6. Coordinates shown hereon are referenced to the Florida State Plane Coordinate System, FL-West projection.
7. Topographic Data shown hereon was surveyed via aerial L.I.D.A.R. scan provided by Bowman Consulting Services, project No. 030304-01-002

LEGEND

- = 5/8" CAPPED IRON ROD
- = PK NAIL & DISC
- = VERTICAL CONTROL BENCHMARK
- = SPOT ELEVATION



SHEET 2 and 3 of 4

Topographic Survey

PREPARED FOR: Pasco County Solid Waste Department

Pasco Waste, Area 2

PROJECT NO: 2019-430A01.BG00001

SEC: 25 | TWP: 24 S | RNG: 17 E | COUNTY: Pasco

JOHNSON ASSOCIATES
SURVEYING AND MAPPING
Licensed Business No. 4514
11911 S. Curley Street
San Antonio, FL 33576
Phone: (352) 588-2768
Email: survey@dc_johnson.com
www.dc_johnson.com

SEAL

Daniel C. Johnson
P.S./PSM License Number 3653
This survey conforms to the "Standards of Practice" set forth by the Florida Board of Land Surveyors in Chapter 5J-17 Florida Administrative Code.

Not valid without the signature and original raised seal of a Florida Licensed Surveyor and Mapper.

DRAWN DLJ | DATE: 01/08/2021
DECED DCJ | DATE: 01/08/2021

DATE	REVISION	BY

DATE OF FIELD SURVEY: 12/08/2020

LTR.	DATE	REVISIONS	BY	APPRD.

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WEST PASCO COUNTY LANDFILL
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PASCO COUNTY, FLORIDA

TOPOGRAPHIC SURVEY

PROJECT NO: 16002-009-01
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-N-

ASH CELL A-1

ASH CELL A-2

ASH CELL A-3

ASH CELL A-4

1 C8 CLASS I FINAL COVER SEE DETAIL (TYP)

A C8 DOWNCOMER SEE SECTION (TYP)

B C6

A C5

D C7

C C5

3:1 (TYP)

5%

+122.0

2MW-13D 4MW-13D 4MW-12D 4MW-11D 4MW-14D 2MW-25D 2MW-25S 2MW-24S 2MW-24D 2MW-15D 4MW-16D 2MW-20D

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WEST PASCO COUNTY LANDFILL
ASH MONOFILL OPERATIONS MODIFICATIONS
PASCO COUNTY, FLORIDA

CONCEPTUAL FINAL COVER PLAN

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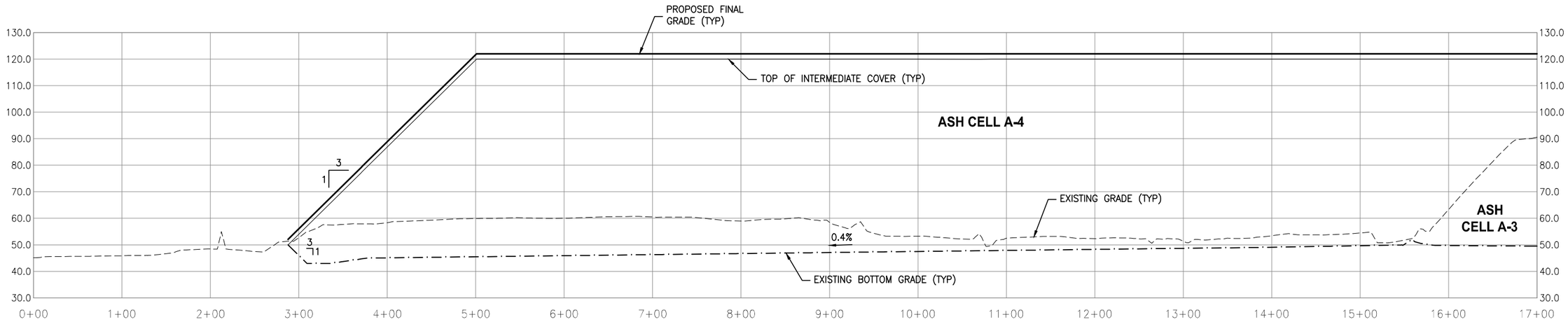

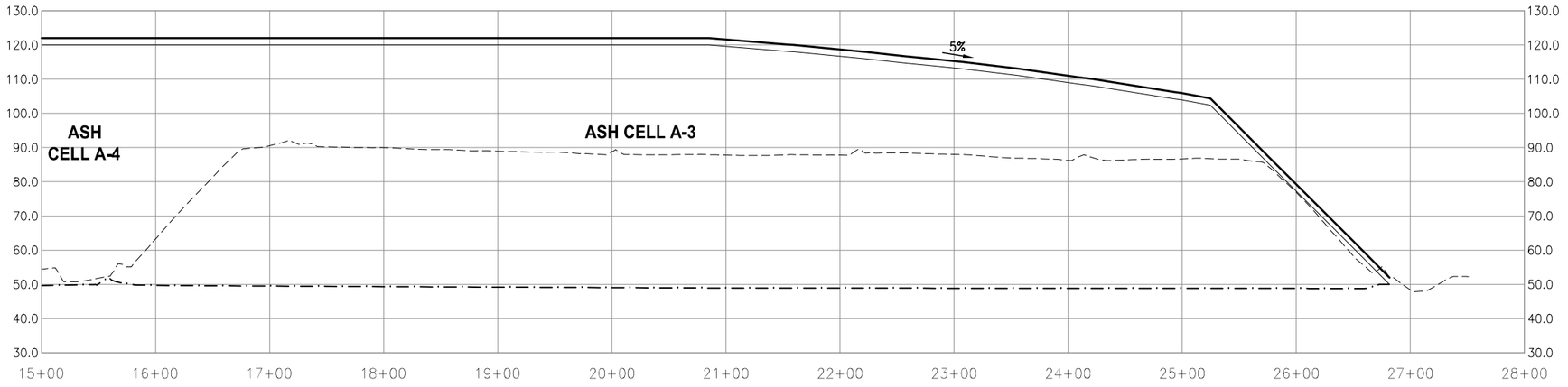

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SCALE IN FEET
HORIZONTAL

GRAPHIC SCALE
0 10 20 40
SCALE IN FEET
VERTICAL

SECTION A (C4)

NOTES

1. EXISTING WASTE GRADE ELEVATIONS IN SUBCELL A–4 ARE APPROXIMATELY AT ELEVATION 88'.

PERMIT

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WEST PASCO COUNTY LANDFILL
ASH MONOFILL OPERATIONS MODIFICATIONS
PASCO COUNTY, FLORIDA

CONCEPTUAL FINAL COVER SECTIONS

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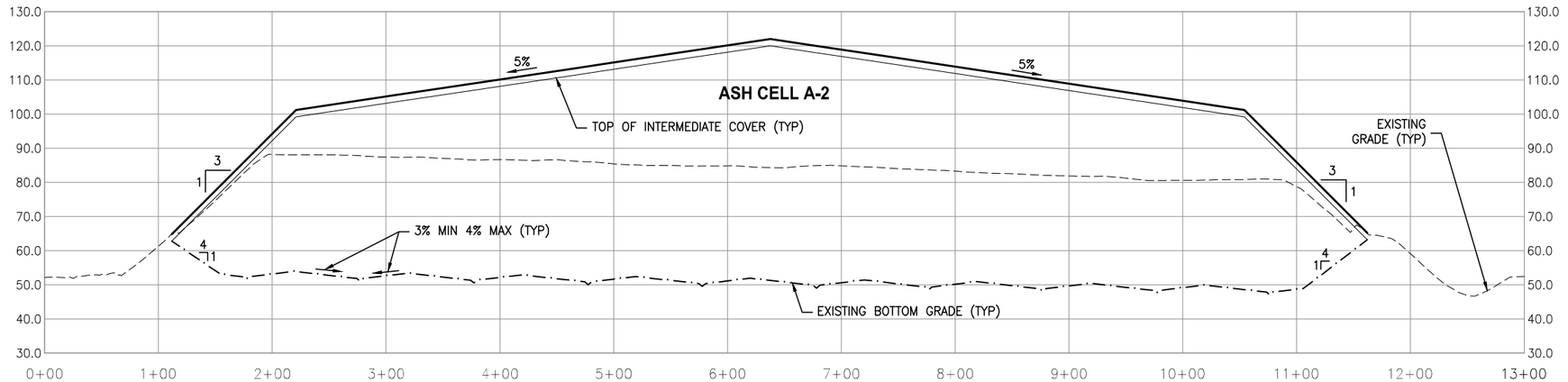

SECTION B (C4)

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HORIZONTAL

GRAPHIC SCALE
0 10 20 40
SCALE IN FEET
VERTICAL

PERMIT

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WEST PASCO COUNTY LANDFILL
ASH MONOFILL OPERATIONS MODIFICATIONS
PASCO COUNTY, FLORIDA

CONCEPTUAL FINAL COVER SECTIONS

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DWG NO: C6

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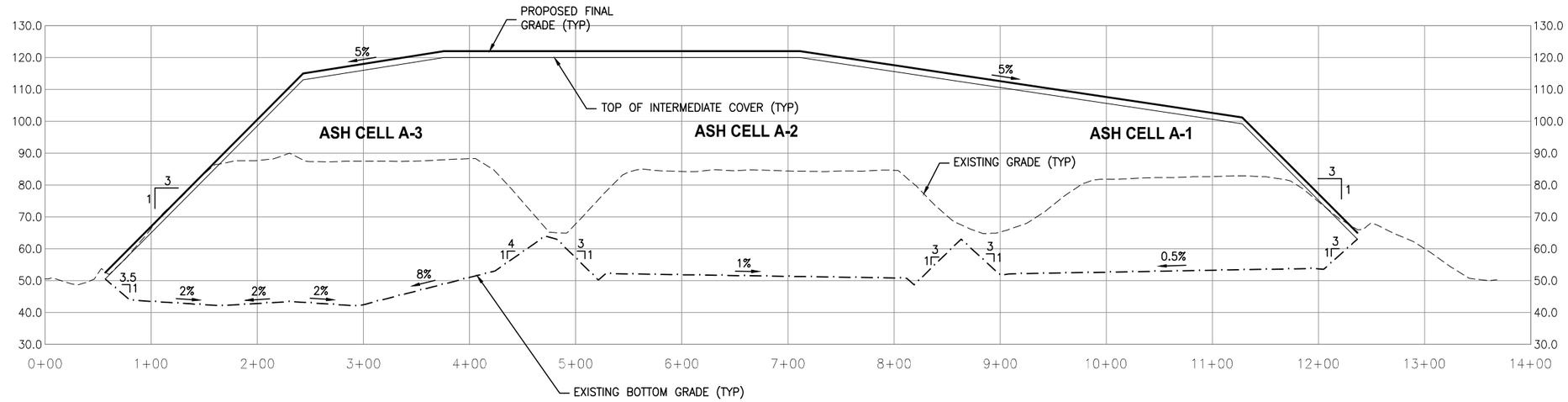
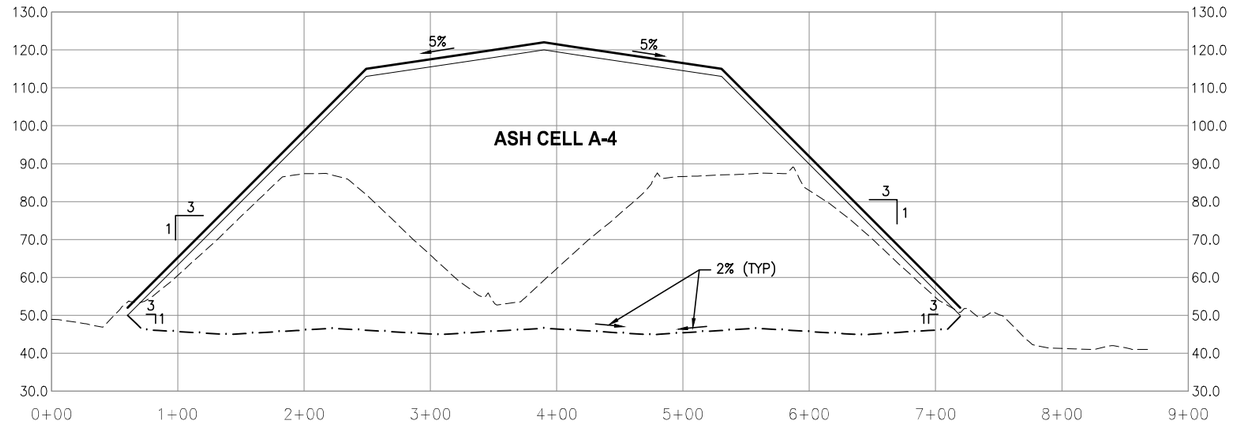


SECTION C (C4)



SECTION D (C4)

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SCALE IN FEET
HORIZONTAL

GRAPHIC SCALE
0 10 20 40
SCALE IN FEET
VERTICAL

PERMIT

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WEST PASCO COUNTY LANDFILL
ASH MONOFILL OPERATIONS MODIFICATIONS
PASCO COUNTY, FLORIDA

CONCEPTUAL FINAL COVER SECTIONS

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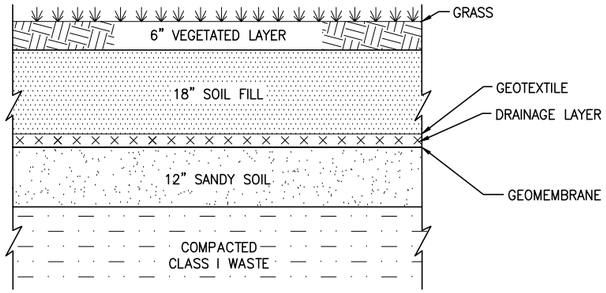

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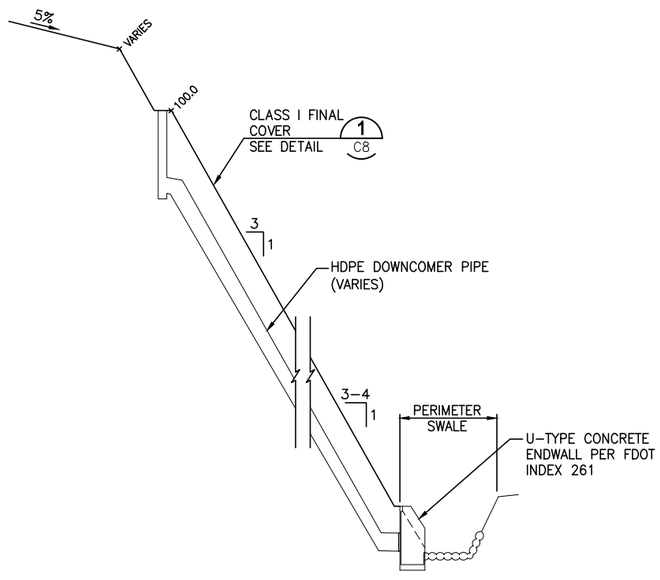

TYPICAL DOWNCOMER SECTION A
NTS C4

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WEST PASCO COUNTY LANDFILL
ASH MONOFILL OPERATIONS MODIFICATIONS
PASCO COUNTY, FLORIDA

CONCEPTUAL FINAL COVER DETAILS

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P.E. #69187

PROJECT NO: 16002-009-01
DATE: JUL 2021
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DWG NO: C8

Appendix B

Operation Plan

OPERATION PLAN
FOR THE
PASCO COUNTY
WEST PASCO COUNTY CLASS I LANDFILL

Prepared for:

Pasco County Public Infrastructure

Solid Waste & Resource Recovery

14230 Hays Road

Spring Hill, Florida 34610

Prepared by:

Jones Edmunds & Associates, Inc.

730 NE Waldo Road

Gainesville, FL 32641

Certificate of Engineering Authorization #1841

Geology Business #133

July 2021

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Attachments

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1.0 GENERAL DESCRIPTION OF THE SOLID WASTE COMPLEX

The West Pasco County Class I Landfill (WPCL) (for which this Operation Plan has been prepared) is an integral component of the larger West Pasco Solid Waste Complex. The overall 800-acre Complex is comprised of the following components:

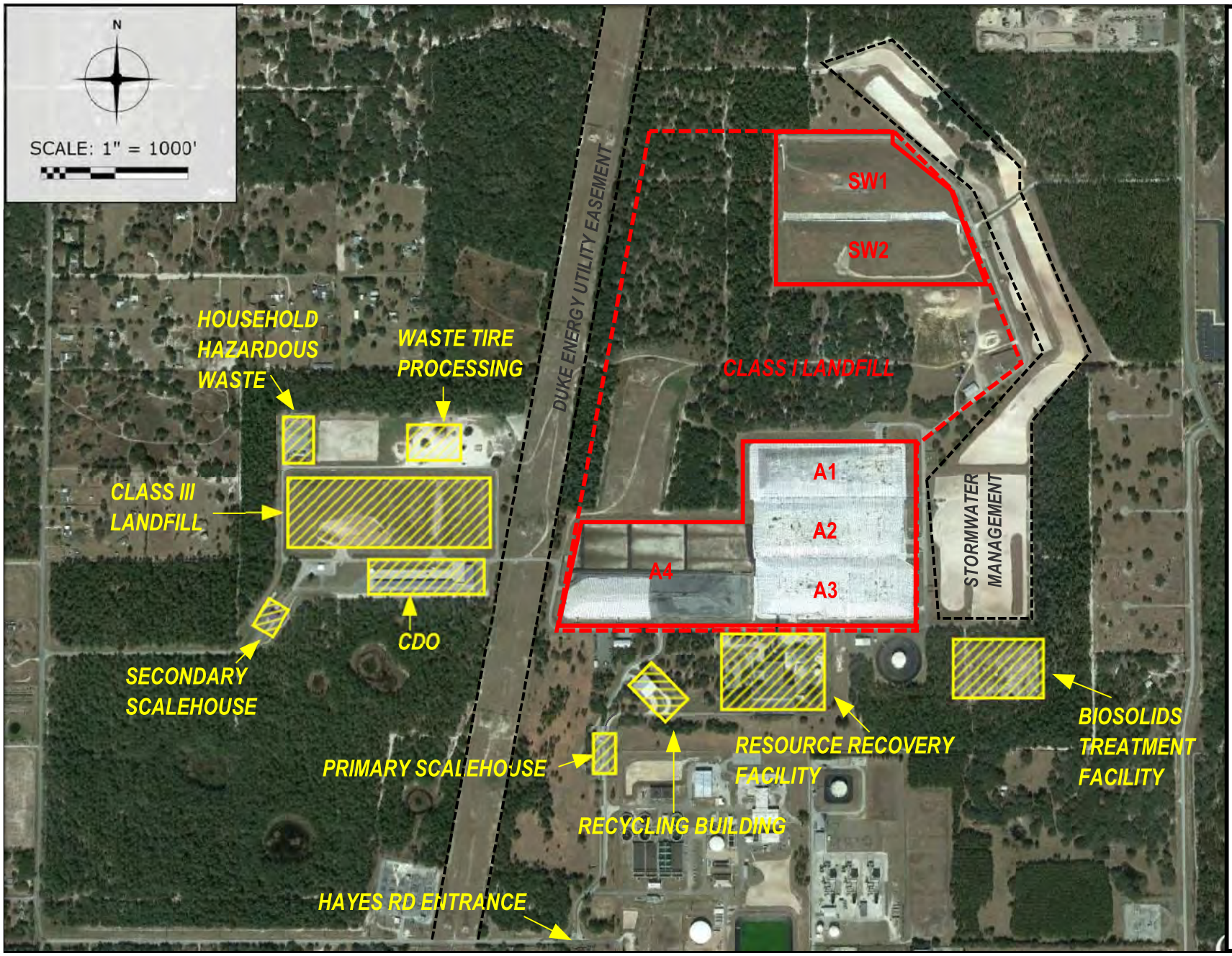
- The Pasco County Resource Recovery Facility;
- The West Pasco Class I Landfill (the component addressed by this Operation Plan);
- The West Pasco Class III Landfill;
- The West Pasco Materials Recovery Facility;
- Citizen's Drop Off Facility;
- The Pasco County Biosolids Treatment Plant, and;
- Various support structures, including two scalehouses, vehicle maintenance buildings, etc.

The main entrance of the West Pasco Solid Waste Complex is at 14230 Hays Road in Spring Hill and is depicted on **Figure 1-1**. The majority of the complex is permitted under the Florida Electrical Power Plant Siting Act through Conditions of Certification No. PA 87-23F; however, the Class III Landfill and the Biosolids Treatment Plant are permitted through individual permits outside of PA 87-23F. The following solid waste management activities occur at the Complex:

- Scalehouse operations.
- Solid Waste combustion at the Resource Recovery Facility.
- Metals recovery in the Ash Storage Building of the Resource Recovery Facility.
- Ash disposal in the Ash Monofill (currently Cell A-4).
- Engineered Aggregate processing within the Ash Monofill (currently limited to Cell A-4).
- Maintenance of temporary cover on the inactive cells of Ash Monofill (Cells A-1, A-2, and A-3).
- Solid Waste Disposal in the Solid Waste Landfill (currently Cell SW-2).
- Maintenance of temporary cover on the inactive cell of the Solid Waste Landfill (Cell SW-1).
- Construction and demolition (C&D) disposal within the Class III Landfill.

- Yard waste processing.
- White goods collection and recycling.
- Household hazardous waste (HHW) collection.
- Recycled materials processing.
- Stormwater collection and treatment.
- Landfill gas monitoring.
- Groundwater monitoring.
- Landfill leachate collection and transport.
- Heavy equipment and vehicle maintenance.
- Waste tire collection and processing.

The entire 800-acre site is enclosed by a chain link fence or barbed wired fence to limit access. Public access is limited to two gates, both of which are monitored by Pasco County staff in the scalehouses adjacent to the gates. After-hours access to the primary entrance gate off Hays Road is monitored and controlled by the control room operator at the Resource Recovery Facility.



Site Plan

West Pasco Solid Waste Facility

Figure 1-1

2.0 SAFETY

2.1 EMERGENCY CONTACTS AND REPORTING

Dr. Justin Roessler, PhD, PE	Office	(727) 856-0119
Interim Solid Waste Director	Cell	(352) 270-1454
Mr. Tim Treshler	Office	(727) 856-0119
Sr. Project Manager	Cell	(727) 856-0119

The emergency contact individuals are subject to change. The Operation Plan will be updated if any changes occur. All accidents should be immediately reported to the Interim Solid Waste Director. After immediately reacting to the accident (first aid or equipment repair), the Interim Solid Waste Director or others designated by the Interim Solid Waste Director will investigate the cause of the accident. A full accident report will be drafted and submitted to Director. The accident report will include all facts involved in the incident as applicable including, but not limited to, date, time of day, weather conditions, hauler vehicle traffic conditions, location on the landfill, equipment and personnel involved, circumstances leading to cause, and response. Photographs will be attached when applicable.

2.2 OPERATOR AND HAULER REQUIREMENTS

Smoking is prohibited on and near the landfill cells and the working face. A spark from a lighter, cigarette, cigar, or pipe could ignite landfill gas (LFG). A sign at the scalehouse notifies anyone entering the landfill of the No-Smoking Policy. All Pasco County employees will enforce this policy in the field. "No Smoking" signs are posted at all entrances, roadways, and active areas.

Welding and oxy-acetylene cutting are prohibited near the landfill or working face. If a vehicle requires this type of maintenance, it shall be towed to an area away from LFG venting. If this is not possible, an employee trained in using a combustible gas meter will take a reading around the area to determine whether methane/combustible gases are absent and their concentrations if they are present.

2.3 TRAINING, EQUIPMENT, AND MATERIALS

Safety materials and equipment include fire extinguishers at landfill sites and first aid kits that are easily accessible for field use. Other equipment provided by the County may include various types of Personal Protective Equipment (PPE) and protective disposable coveralls. Landfill field employees are required to wear appropriate clothing on the work site, such as steel-toed boots, long pants, and a shirt.

Pasco County Public Infrastructure maintains a pro-active approach to training by requiring key operating personnel attend the University of Florida's Training, Research, and Education of Environmental Operations (TREEO) certification courses. All Operators and Spotters must attend and pass the Initial Training for Operators of Landfills and Waste Processing Facilities

course. Within three years of passing the initial training course and every three years thereafter, each operator must complete an additional 16-hour refresher course. Additionally, all spotters must attend and pass the Initial Training Course for Spotters at Landfills, C&D Sites, and Transfer Stations. Within three years of passing the initial training course and every three years thereafter, each spotter must complete an additional 8-hour refresher course.

Copies of course completion certificates are kept on file within the Administration offices. The landfill has at least one trained operator on site during all times when the landfill receives waste. At least one trained spotter is at each working face at all times to detect unauthorized wastes when the landfill receives waste other than ash.

3.0 ORGANIZATION

Pasco County Solid Waste is organized as follows:

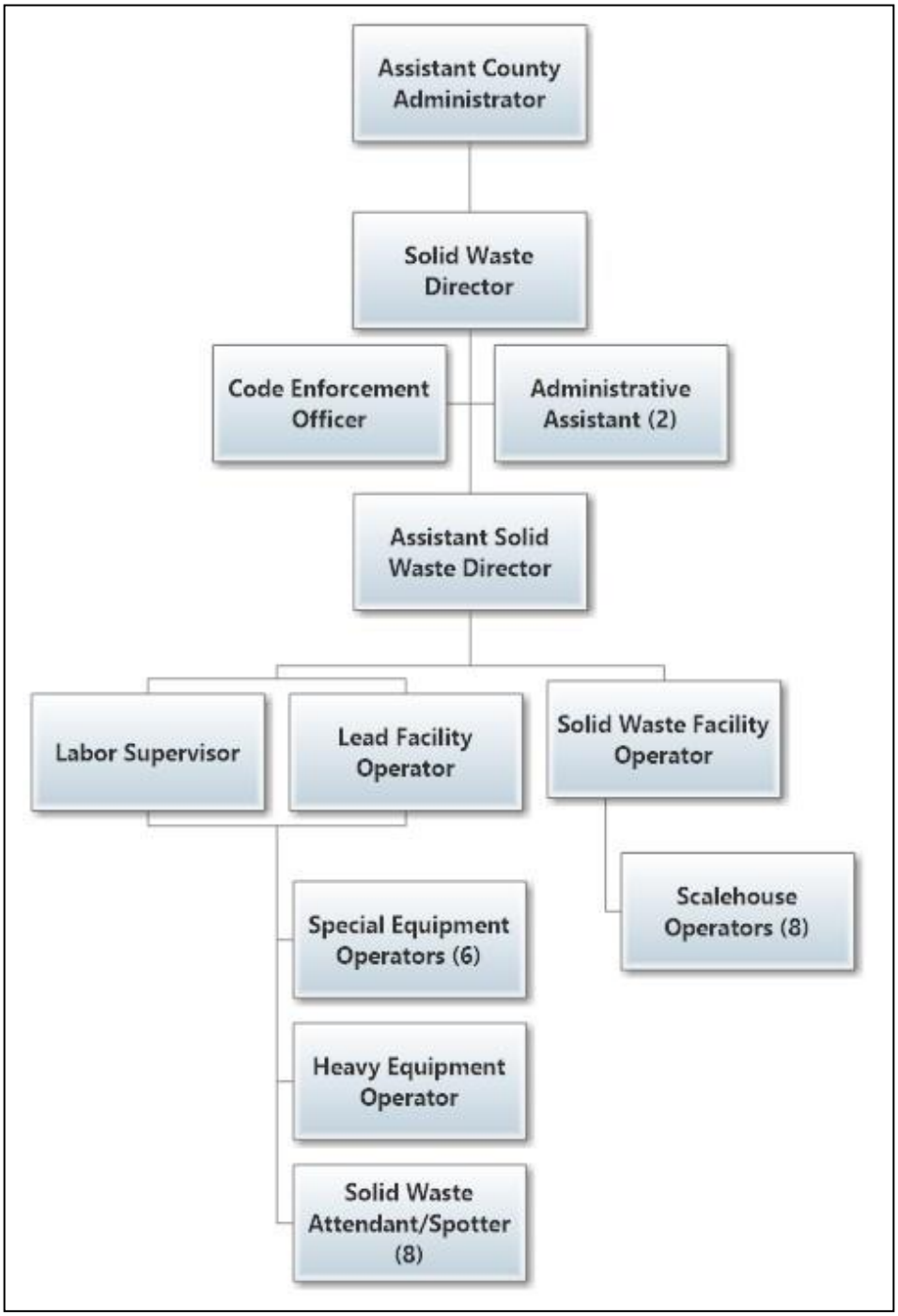


Figure 3-1
Organizational Chart

4.0 SCALEHOUSE OPERATIONS

The scale operation is critical to the efficiency of the landfill facility. This operation serves several important functions including security, initial load checking, waste identification and segregation, traffic control, waste quantity recording, and financial documentation.

The primary scalehouse is the original to Facility, was constructed in 1990, and is approximately 1,000 feet from the entrance gate on the main facility access road. The scale facility has a scalehouse, two inbound scales, one outbound scale, one inbound bypass lane, one outbound bypass lane, and a video surveillance system. Monitoring the movement of haul trucks and other vehicles is part of the duties of the Scalehouse Attendant. All traffic entering the facility must check in with the Scalehouse Attendant. Waste delivery vehicles are required to weigh in, and all visitors must first report in and state the purpose of their visit. A Scalehouse Attendant is maintained on duty during all periods when the facility is open.

A secondary scalehouse, constructed in 2011, is utilized primarily for citizens that choose to self-haul to the Citizens' Drop-Off facility. The secondary scalehouse has a scalehouse, one inbound scale, one outbound scale, one inbound bypass lane, one outbound bypass lane, and a video surveillance system.

Administration maintains communication with landfill staff via hand-held radios and cell phones. A sign at both entrances to the facility identifies the operating authority, hours and days of operation, waste disposal restrictions, and other information. Speed limit signs are posted along the main road leading to the active filling areas. Signs identifying roads leading to the Class I filling area are posted at the intersection of each road. Other miscellaneous signs direct customers to the other public facilities, including recycling drop-off containers, mulch, and tire disposal area.

4.1 LOAD SCREENING

The Scalehouse Attendant screens all loads as they are received at the scale facility. The Scalehouse Attendant performs two types of inspections on incoming waste loads per transaction: hauler interview, and visual inspections when visible from inside the scale facility.

The initial inspection of the incoming waste is conducted by the Scalehouse Attendant. This inspection includes identification from the driver of the type of waste and random visual inspections of the vehicle. The random visual inspections include looking for suspicious containers that may contain prohibited wastes and smoke rising from the payload area of the vehicle.

The types of businesses in the area that may generate prohibited wastes include automotive repair, painting, dry cleaning, fiberglass fabrication, and marine service. All incoming loads

of waste should be checked for visual and olfactory indications. Indications of suspicious loads include the following:

- Hazardous placards or markings.
- Drums.
- Containerized liquids.
- Powders or dusts.
- Sludges.
- Bright or unusual colors.
- Chemical odors.

4.2 Prohibited Wastes

The Scalehouse Attendant should also scan incoming loads for components that contain prohibited materials. Waste materials prohibited from disposal at the West Pasco Facility are listed below. Refer to Section 7.6 for handling and removal of prohibited wastes from the waste stream. References to sections where disposal is addressed are shown parenthetically. Please note that waste tires, white goods, yard waste, and other selected items are accepted at the Complex but diverted from the Class I cell.

- Lead Acid Batteries (8.7).
- Paint (8.4 and 8.5).
- Biomedical wastes (8.10).
- Whole tires (8.11).
- Used motor oil and oil filters (8.3).
- White goods (8.12).
- Household Hazardous Waste (8.1).
- Unknown Wastes (8.2).
- Septic/sewage/sludge except that bound for the Biosolids Processing Facility or meeting the definition of a solid waste in accordance with 62-701(107) and not prohibited by Rule 62-701.300, FAC (8.10).
- Segregated Electronics (8.8).
- Contaminated Soils (8.9).

4.3 Waste Categories

Pasco County utilizes defined categories of types of waste received at the scalehouse. These categories of waste are designed to provide required reporting to FDEP as well as to properly manage the incoming waste. Each waste category is managed at a specific area of the Complex. The Scalehouse Attendant is responsible for directing the driver of the haul vehicle to the proper location in the landfill for unloading.

Table 4-1 lists all waste categories received at the Complex and the location for unloading within the Complex.

The tonnage of all waste categories is compiled monthly. The waste quantity reports are submitted annually to FDEP.

Table 4-1
PC Scales Waste Category Codes and Disposal Area

Code	Description	Disposal Area
AC	Air Conditioner	HHW
AR	Adopt a Road	RRF, Class I, Class III, Waste Tire
CCT	Coastal Cleanup	RRF, Class I, Class III, Waste Tire
CD	Construction Debris	Class III
CEE	Commercial Electronic Equipment	HHW
CM	Computer Monitor	HHW
COM	Computer without Monitor	HHW
EE	Electronic Equipment	HHW
FCY	Fuel Cylinder	HHW
HEL	Helium Tank	HHW
MET	Metal	Class III
MSW	Municipal Solid Waste – Residential	RRF, Class I
MSWC	Municipal Solid Waste – Commercial	RRF, Class I
OXY	Oxygen Tank	HHW
PT20	Propane Tank up to 20 gallons	HHW, Class III
PT50	Propane Tank up to 50 gallons	HHW, Class III
REF	Refrigeration	HHW

RIMS	Outbound Tire Rims	N/A
TC	Tire Chips	Waste Tire
TCOTR	Off road Tire Chips	Waste Tire
TLT	Toilet Recycling Program	Class III
TR	Tires	Waste Tire
TRE1	Tires (each) car or pickup	Waste Tire
TRE2	Tires (each) large truck or semi	Waste Tire
TVP	TV Projection and Console	HHW
TV1	TV up to 36" screen	HHW
WC	Wood Chips	RRF, Class I, Class III
WG	Outbound metal and white goods	N/A
YWC	Yard Waste Commercial	Yard Waste
YWR	Yard Waste Residential	Yard Waste

4.4 Data Recording and Reporting

Vehicles are first weighed at the scale with the gross weight of the vehicle recorded. Pasco County utilizes the software PC Scale to categorize and track inbound and outbounds loads. Following assignment of the appropriate commodity code into the PC Scale program, the Scalehouse Attendant then directs the vehicle to the appropriate area of the Complex to unload. The vehicle returns to the scale where the tare weight is recorded to determine the net weight of the material. Materials such as white goods, tires, and bulk yard waste are dropped off at the designated areas as shown on **Figure 1-1**. These areas are clearly indicated by signs.

5.0 LANDFILL OPERATOR AND SPOTTER TRAINING PLAN

Landfill Operator and Spotter training will comply with Rule 62-701.320(15), FAC.

Training courses, whether public or in-house, meet the requirements of Rule 62-701.320(15), FAC and shall be certified by the FDEP Solid Waste Management Training Committee (SWMTC). This training plan, along with documents that record training plan implementation, are kept on site and will be made available to FDEP's inspection staff upon request. Training records are also be kept by University of Florida Center for Training, Research, and Education for Environmental Occupations (TREEO).

5.1 OPERATORS

New Supervisors hired by Pasco County Solid Waste will participate in 24 hours of initial training provided by an entity that has been pre-approved by the Department pursuant to Section 403.716, Florida Statutes. Within 3 years after passing the exam and every 3 years thereafter, Supervisors will participate in continuing education courses totaling 16 hours conducted by an approved provider.

5.2 SPOTTERS

New spotters will participate in 8 hours of initial training provided by entities meeting the requirements of Rule 62-701.320(15), FAC. Every 3 years after initial training, landfill Spotters will participate in continuing education courses provided by approved entities totaling 4 hours.

6.0 ASH MONOFILL OPERATIONS

The Ash Monofill portion of the Class I Landfill receives only combustion residue (ash) from the Pasco County Resource Recovery Facility. Ash that is generated from the combustion process is directed to the dedicated Ash Storage Building at the Resource Recovery Facility, where it is periodically loaded within the building by a front-end loader into top loading trucks. The loaded trucks travel to the active ash disposal cell, where they are off-loaded. Ash hauling operations occur between 7 AM and 6 PM, Monday through Friday, or more frequently as needed.

The liner and leachate collection system have been constructed, one disposal unit at a time (A-1, A-2, A-3, A-4) with temporary roads and swales for access and surface-water management. The phasing plan for the currently active ash disposal cell (A-4) is depicted in the drawing set dated July 2008 by CDM sheets C-6 through C-11. The proposed modifications to the side slopes and vertical expansion of Cells A-1, A-2, and A-3 to a final buildout elevation of 122 feet North American Vertical Datum of 1988 (ft NAVD) requires an update to the preexisting phasing plan. As of June 2021, Cell A-4 is the active disposal cell for the Resource Recovery Facility's ash; however, activities such as advanced metals recovery exercises by Covanta and periodical ash-derived aggregate processing partially occupy the available disposal area, encouraging the County to explore revised phasing plans to increase operational flexibility.

In general, waste filling is done in conjunction with using a rain tarp as intermediate cover to reduce leachate generation. As of May 2021, Cells A-1, A-2, and A-3 are covered by such intermediate cover. The rain tarp is a 20-mil geomembrane that serves as intermediate cover for areas that are not expected to be used for disposal within 180 days. All precipitation on the landfill that comes in contact with the rain tarp (and not in contact with ash) is managed as non-contact stormwater.

Cell A-4 is divided into six subcells. The subcells are numbered from one to six, starting at the southwest corner of the overall cell progressing eastward (subcells 1, 2, and 3), then from the northwest corner progressing eastward (subcells 4, 5, and 6). The sequence of filling is depicted on drawings dated July 2008 by CDM, sheets C-6 through C-11.

The general filling sequence is accomplished by filling in each subcell to an approximate elevation of 85 feet, with a side-slope of 3:1. A secondary berm (inside the subcell divider berms) around the Phase I fill area has been constructed to divert contaminated stormwater (leachate) to low areas as shown on sheet C-6. This overall plan is accompanied by a modified phasing plan for the entire ash monofill, which accounts for the vertical expansion and valley fills between Cells A-1/A-2, Cells A-2/A-3, and Cells A-3/A-4. Unless otherwise noted, all ash beneficial reuse activities (described in detail in Section 6.6) shall be relocated or ceased as necessary as this phasing plan proceeds.

6.1 CURRENT FILLING SEQUENCE OF A-4

This section describes the current filling sequence for Cell A-4 starting with the individual subcells, after which the modified filling sequence will be discussed.

6.1.1 PHASE I - FILLING IN SUBCELL 1

1. Flag the edge of the liner at the top of the berm or mark with traffic cones. Ash will not be placed within 2 feet of the flagged or marked line.

2. Close the ball valves in subcells 2 and 4. All other ball valves will be in open position. Install stormwater pumps in subcells 2 and 4. Stormwater from all subcells except subcell 1 will be pumped to the surface drainage swales.

3. Place a berm/swale around the active phase of the cell area. All waste and water in contact with waste will be contained within the bermed area. (See sheet C-6.)

4. Ash will be unloaded from the entrance ramp to the cell and spread over the 2 feet of protective sand cover, constructing a platform. A minimum of 5 feet of initial lift will be placed and rolled to cover subcell 1. The ash for the initial lift will be free from large, sharp objects that may damage the liner system.

5. Continue normal filling operation, maintaining the perimeter berm/ swale around the sub-cell 1. Filling the sub-cell 1 will be accomplished by constructing approximately 10-foot lifts, working in a general direction from south to north, and from west to east.

6. As the filling operation progresses, construct an access road using ash from the west of subcell 1 as shown in sheet C-6, and provide drainage berms/swales on both sides of the access road and around the perimeter of the subcell 1 area. Only the portion of the access road within the landfill footprint shall be constructed of ash.

7. Continue the operation until the elevation of 85 feet is achieved as indicated in sheet C-6.

6.1.2 PHASE II - FILLING IN SUBCELL 2

1. Close the ball valves in subcells 3 and 4. All other ball valves will be in open position. Remove stormwater pumps for subcell 2 and install them in subcell 3. Stormwater from all subcells except subcells 1 and 2 will be pumped to the surface drainage swales.

2. Begin the filling operation in the Phase II area in a similar manner as in Phase I. Please see sheet C-7 for additional details.

6.1.3 PHASE III - FILLING IN SUBCELL 3

1. Close the ball valves in subcell 4. All other ball valves will be in open position. Remove stormwater pumps for subcell 3. Stormwater from all sub-cells except subcells 1, 2, and 3 will be pumped to the surface drainage swales.

2. Begin the filling operation in the Phase III area in a similar manner as in Phases I and II. Please see sheet C-8 for additional details.

6.1.4 Phase IV - Filling in Subcell 4

1. Close the ball valves in subcell 5. All other ball valves will be in open position. Remove stormwater pumps for subcell 4 and install them in subcell 5. Stormwater from all subcells except subcells 1, 2, 3, and 4 will be pumped to the surface drainage swales.

2. Begin the filling operation in the Phase IV area in a similar manner as in Phases I, II, and III. Please see sheet C-9 for additional details.

6.1.5 Phase V - Filling in subcell 5

1. Close the ball valves in subcell 6. All other ball valves will be in open position. Remove stormwater pumps for subcell 5 and install them in subcell 6. Stormwater from subcell 6 will be pumped to the surface drainage swales.

2. Begin the filling operation in the Phase V area in a similar manner as in Phases I, II, III, and IV. Please see sheet C-10 for additional details.

3. The Aggregate Processing Operation described in Section 6.5 will take place within subcell 5. Once the subcell has been floored with ash, equipment will be placed in the approximate configuration shown on **Figure 6-1**. Please refer to Section 6.5 of this Operation Plan for a description of the Aggregate Processing operation.

6.1.6 Phase VI - Filling in subcell 6

1. All ball valves will be in open position. Remove stormwater pumps for subcell 6.

2. Begin the filling operation in the subcell 6 area in a similar manner as in Phases I, II, III, IV, and V. Please see sheet C-11 for additional details.

6.2 Valley Filling and Vertical Expansion

This section discusses the valley fills proposed between Cells A-1, A-2, A-3, and A-4, as well as the vertical expansion for filling all four cells to a final elevation of 122 ft NAVD. Filling in the existing valleys between Cells A-1/A-2, A-2/A-3, A-3/A-4, and within A-4 may occur after filling in the remainder of area in Cell A-4 or upon completion of the current phase in A-4.

6.2.1 Valley Fill Between Cell A-1/A-2

1. The temporary rain cover should be removed within the valley starting west to east. The valley should be filled in progressing west to east by constructing approximately 10-foot lifts. Before filling in the valley area, the existing ash surface shall be prepared to limit the potential for surface water short-circuiting between the "aged" and "fresh" ash contact surface. Continue the filling operation until approximate elevation of 85 ft NAVD.

2. Leachate collection system cleanout points exist at several points in this valley. To accommodate valley filling, these existing cleanouts will be extended/abandoned as necessary.

6.2.2 Valley Fill Between Cell A-2/A-3

1. The temporary rain cover should be removed within the valley starting west to east. The valley should be filled in progressing west to east by constructing approximately 10-foot lifts. Continue the filling operation until approximate elevation of 85 ft NAVD. Leachate collection system cleanout points exist at several points in this valley. To accommodate valley filling, these existing cleanouts will be extended/abandoned as necessary.

6.2.3 Filling in Valley within Cell A-4

1. Begin filling the valley area between subcells from east to west. Fill the area by constructing approximately 10-foot lifts working in the general direction from south to north and toward the west. Continue the filling operation until approximate elevation of 85 ft NAVD.

6.2.4 Valley Fill Between Cell A-3/A-4

1. Begin filling the valley area between cells from east to west. Fill the area by constructing approximately 10-foot lifts working in the general direction from south to north and toward the west.

6.2.5 Final Buildout of Cells A-1, A-2, A-3, and A-4 to Elevation 122 ft NAVD

1. These lifts will be an area fill over Cells A-1, A-2, A-3, and A-4 to the final elevation of 122 ft NAVD. Slopes will be maintained to meet the modified 3H:1V. These lifts will be approximately 10 feet thick.

6.3 Ash Monofill Waste Inspection

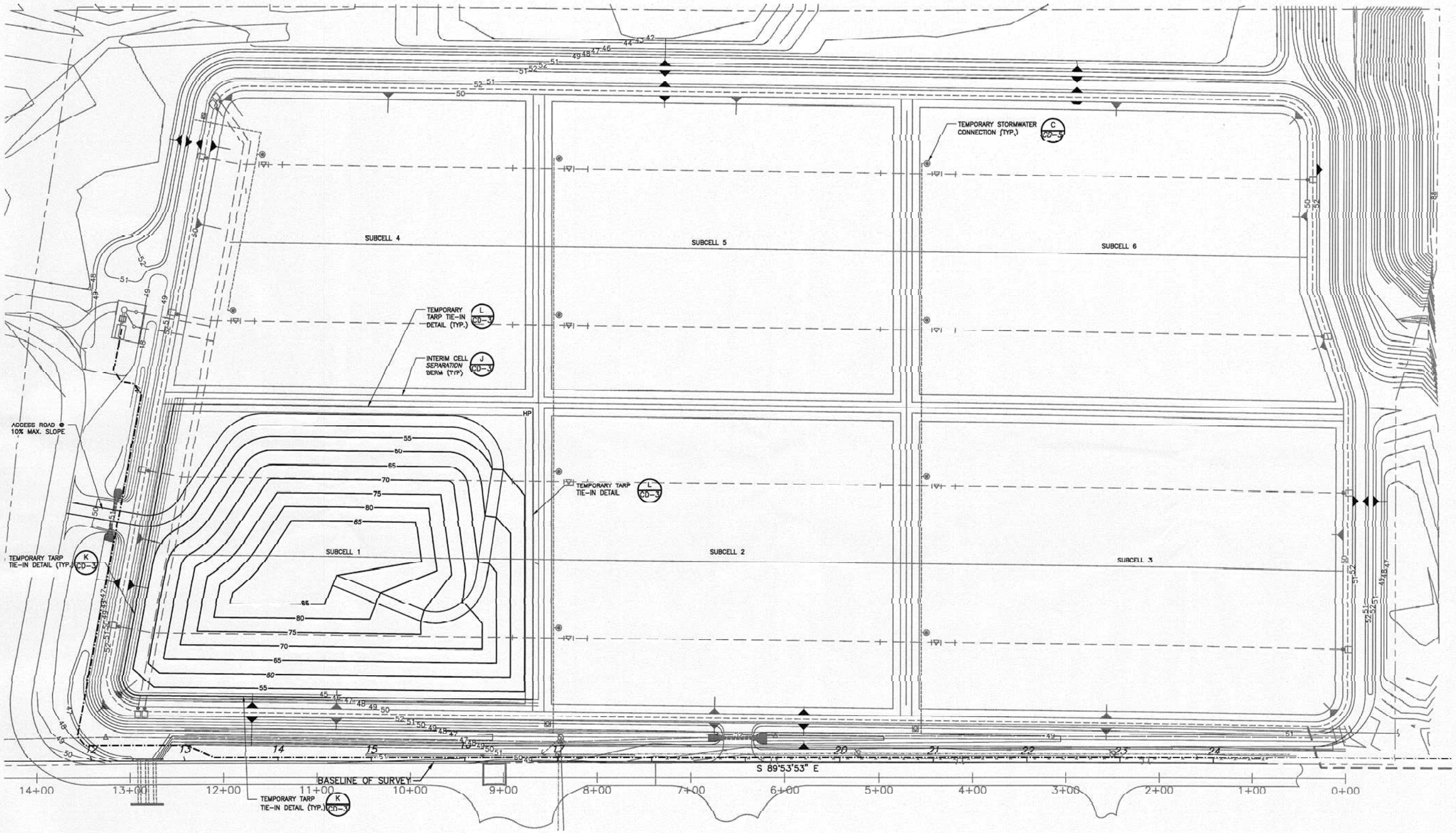
Because the ash monofill receives only combustion residue (ash) from the Resource Recovery Facility, there is no need to deploy spotters to the working face. All Operators working within the ash monofill are trained to identify materials that may have inadvertently been loaded into the ash transport trucks and to segregate such materials if necessary. Such materials could be oversized material (such as white goods) that were segregated at the tipping floor of the Resource Recovery Facility. If such non-ash materials are identified at the monofill, they will be segregated and taken to the appropriate location within the larger Solid Waste Complex.

6.4 Ash Monofill Hauler Queuing

Private haulers are not authorized to utilize the ash monofill. All hauling of ash residue from the Ash Storage Building to the active cell is accomplished by a single contractor employed by the Resource Recovery Facility Operator (Covanta Pasco, Inc.). Hauling takes place Monday through Friday, or more frequently if needed. The Ash Storage Building is sufficiently sized to store several day's worth of ash.

6.5 Waste Compaction and Application of Cover

The ash is spread and compacted as necessary by a front-end loader. Because it contains no putrescible material, daily cover is not applied. On areas where no activity is expected for 180 days or more, a 20-mil geomembrane rain tarp may be installed to minimize leachate generation.



NOTE:

1. TEMPORARY TARP DETAIL WILL BE AS DETAILS K & L, SHEET CD-3, TO BE USED AS NECESSARY.
2. SEE LANDFILL OPERATIONS PLAN 3.6.1, PHASE 1.

FLORIDA STATEWIDE ENGINEERING AUTHORIZATION DEC 10 2008 SOUTHWEST DISTRICT TAMPA

DATE:
AAMOD S. SONAWANE
P.E NO. 62526

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: A. SONAWANE
DRAWN BY: P. ANDUZE
SHEET CHK'D BY: J. LADNER
CROSS CHK'D BY: M. SINUSHKUR
APPROVED BY: A. SONAWANE
DATE: NOVEMBER 2008

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PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

WEST PASCO CLASS I LANDFILL
CELL A-4 CONSTRUCTION

PHASE 1 PLAN FOR
CELL A-4

PROJECT NO. 6104-46136
FILE NAME: C006STPL

SHEET NO.
C-6

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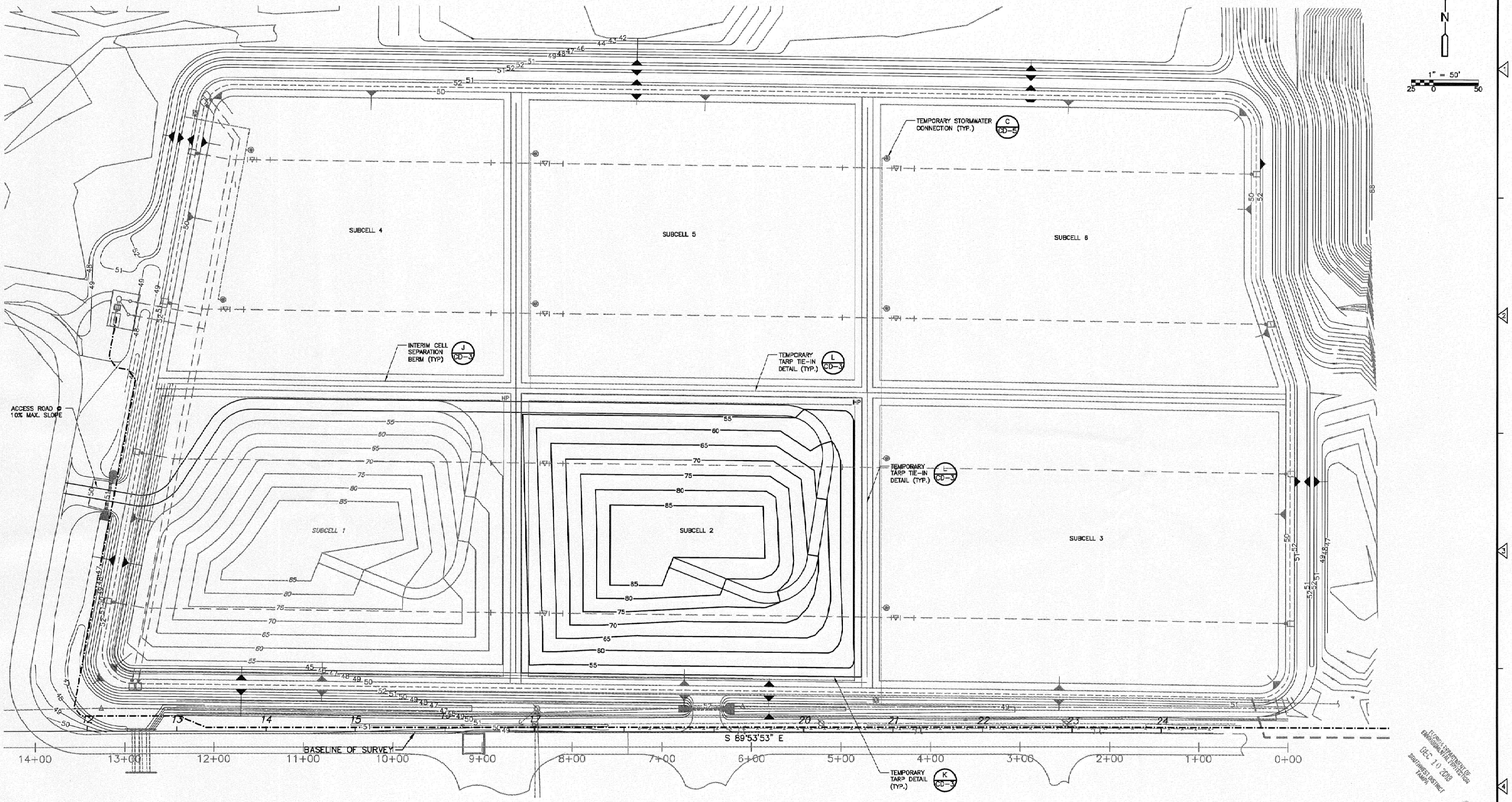

NOTE:

1. TEMPORARY TARP DETAIL WILL BE AS DETAILS K & L, SHEET CD-3, TO BE USED AS NECESSARY.
2. SEE LANDFILL OPERATIONS PLAN 3.6.1, PHASE 2.

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION DEC 10 2008 SOUTHWEST DISTRICT TAMPA

DATE:
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PASCO COUNTY FLORIDA

WEST PASCO CLASS I LANDFILL
CELL A-4 CONSTRUCTION

PHASE 2 PLAN FOR
CELL A-4

PROJECT NO. 5104-46136
FILE NAME: CC07STPL

SHEET NO.
C-7

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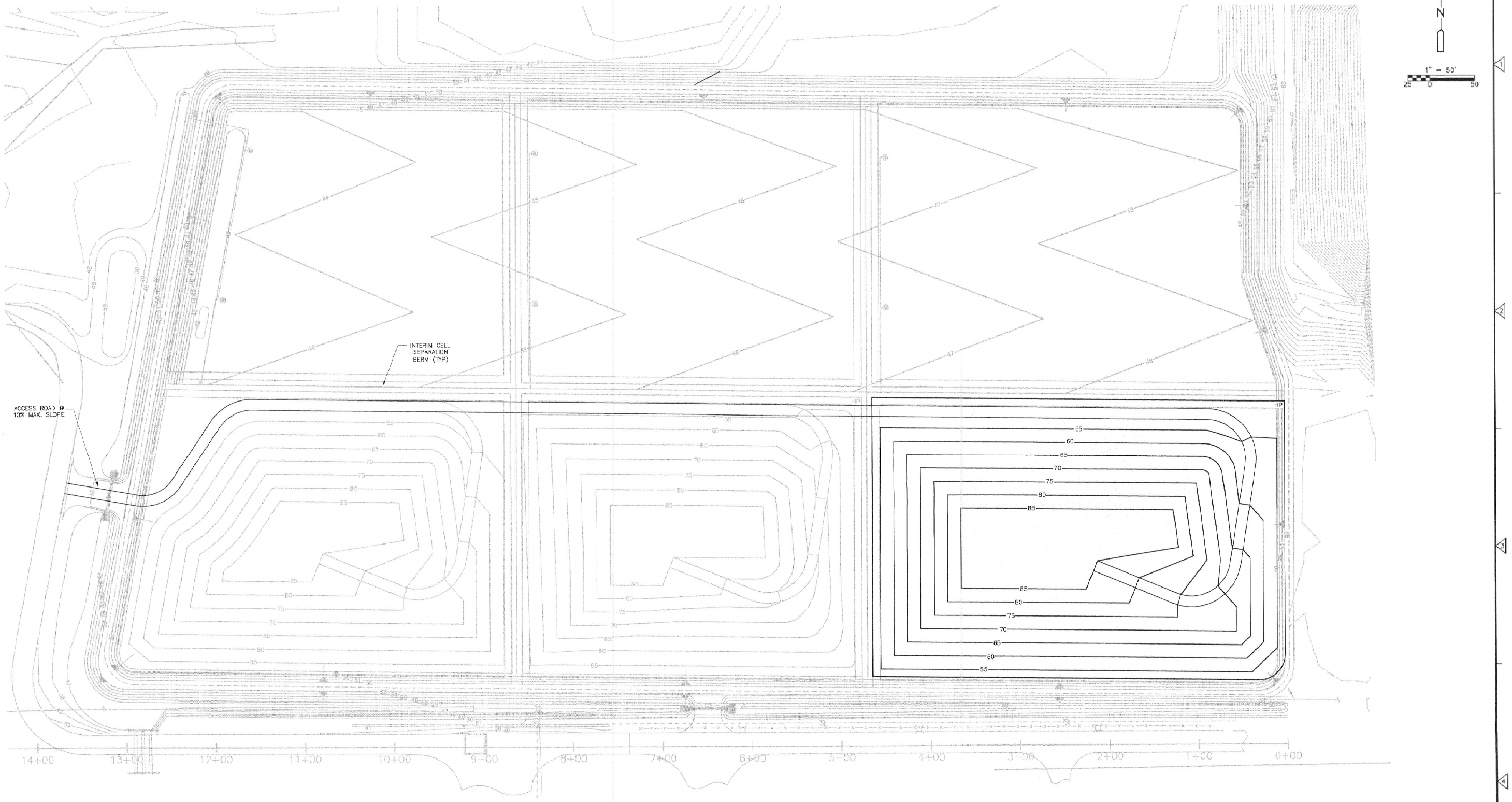

NOTE:
1. TEMPORARY TARP DETAIL WILL BE AS DETAIL K, SHEET CC-3, TO BE USED AS NECESSARY.

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
JUL 28 2008
SOUTHWEST DISTRICT TAMPA

DATE:
AAMOD S. SONAWANE
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PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

WEST PASCO COUNTY LANDFILL
EXPANSION CELL A-4

PHASE 3 PLAN FOR
CELL A-4

PROJECT NO. 6104-46136
FILE NAME: C008STPL
SHEET NO.
C-8

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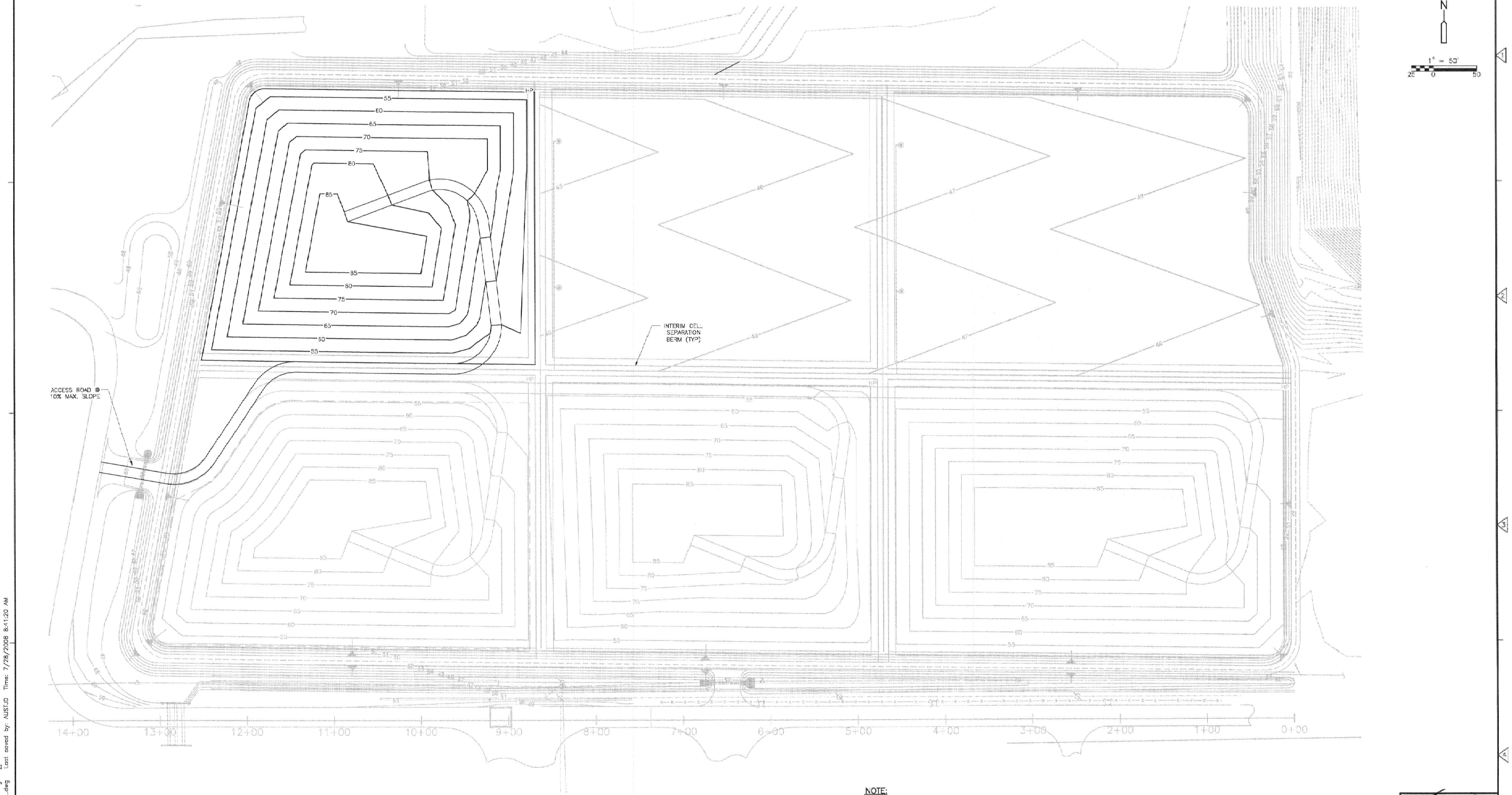

NOTE:
1. TEMPORARY TARP DETAIL WILL BE AS DETAIL K, SHEET CD-3, TO BE USED AS NECESSARY.

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
JUL 28 2008
SOUTHWEST DISTRICT TAMPA

DATE:
AAMOD S. SONAWANE
P.E NO. 62526

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: A. SONAWANE
DRAWN BY: P. ANDUZE
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PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

WEST PASCO COUNTY LANDFILL
EXPANSION CELL A-4

PHASE 4 PLAN FOR
CELL A-4

PROJECT NO. 6104-46136
FILE NAME: C009STPL

SHEET NO.
C-9

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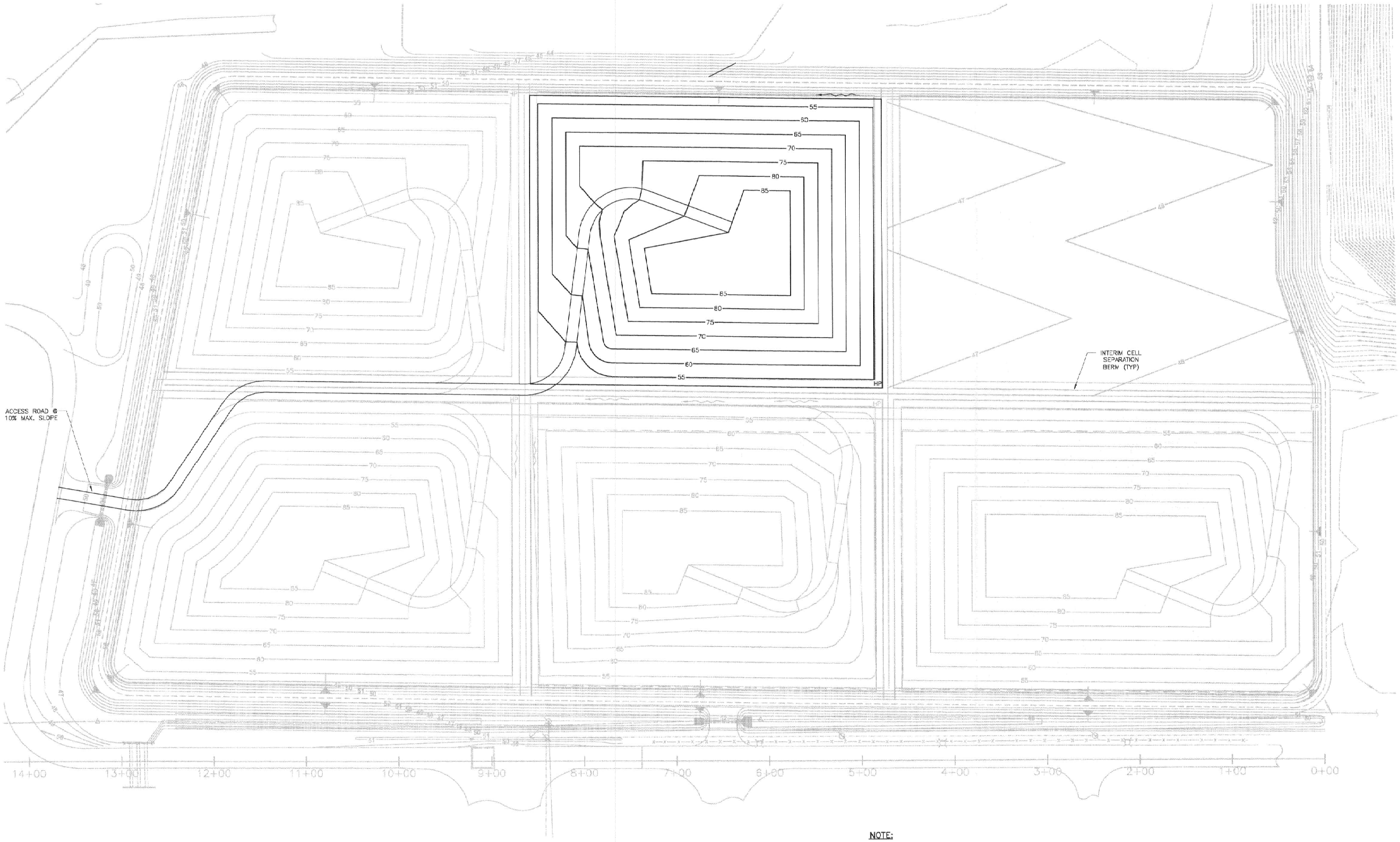

NOTE:
1. TEMPORARY TARP DETAIL WILL BE AS DETAIL K, SHEET CD-3, TO BE USED AS NECESSARY.

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
JUL 25 2008
SOUTHWEST DISTRICT TAMPA

AAMOD S. SONAWANE
STATE OF FLORIDA
DATE:
AAMOD S. SONAWANE
P.E NO. 62526

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: A. SONAWANE
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SHEET CHK'D BY: J. LADNER
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PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

WEST PASCO COUNTY LANDFILL
EXPANSION CELL A-4

PHASE 5 PLAN FOR
CELL A-4

PROJECT NO. 6104-48136
FILE NAME: C010STPL

SHEET NO.
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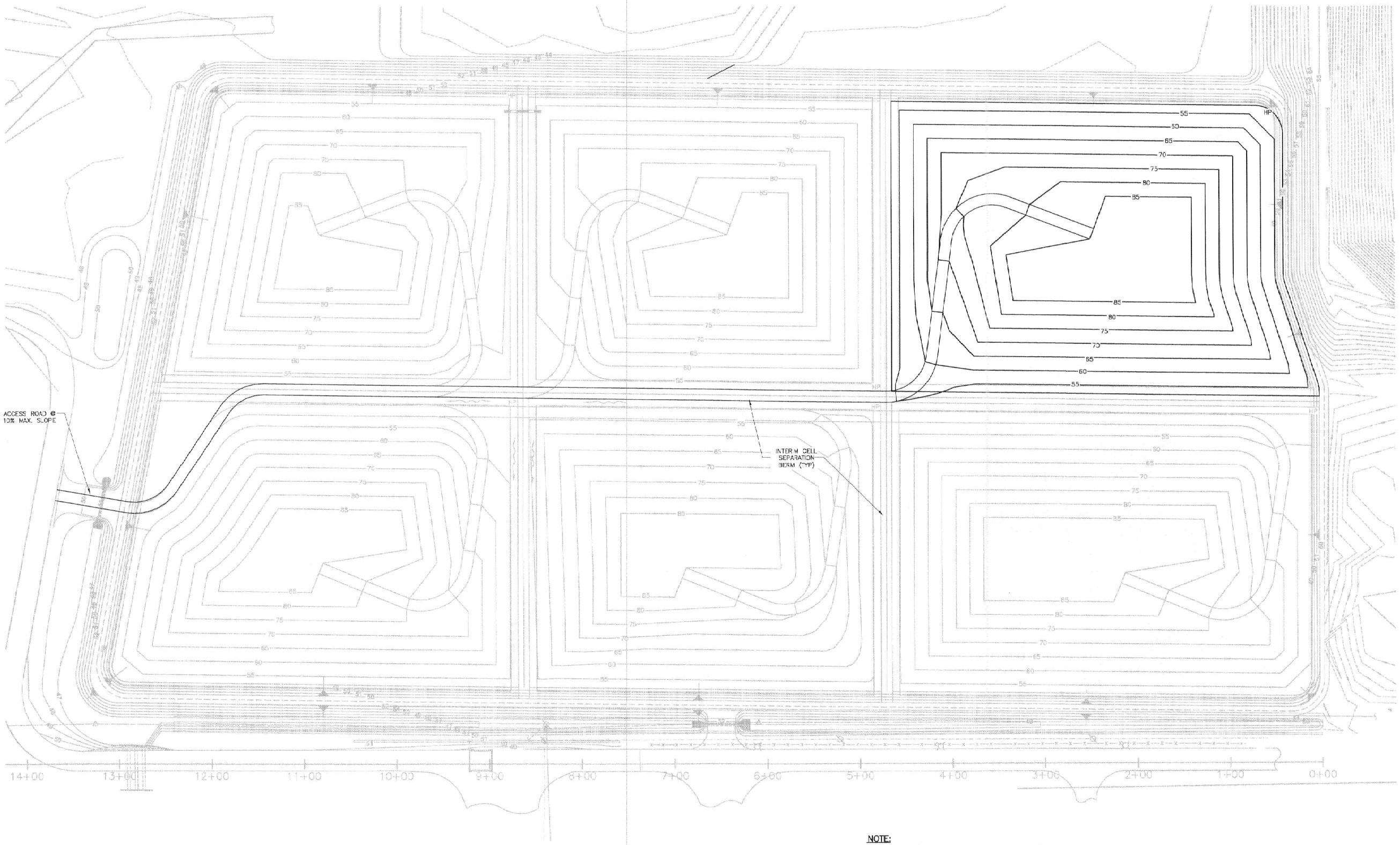

NOTE:

1. TEMPORARY TARP DETAIL WILL BE AS DETAIL K, SHEET CD-3, TO BE USED AS NECESSARY.

FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION
JUL 29 2008
SOUTHWEST DISTRICT TAMPA

DATE:
AAMOD S. SONAWANE
P.E NO. 62526

REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY: A. SONAWANE
DRAWN BY: P. ANDUZE
SHEET CHKD BY: J. LADNER
CROSS CHKD BY: D. STROBRIDGE
APPROVED BY: A. SONAWANE
DATE: JULY 2008

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FL COA No. EB-0000020
consulting • engineering • construction • operations

PASCO COUNTY, UTILITIES SERVICES BRANCH
PASCO COUNTY FLORIDA

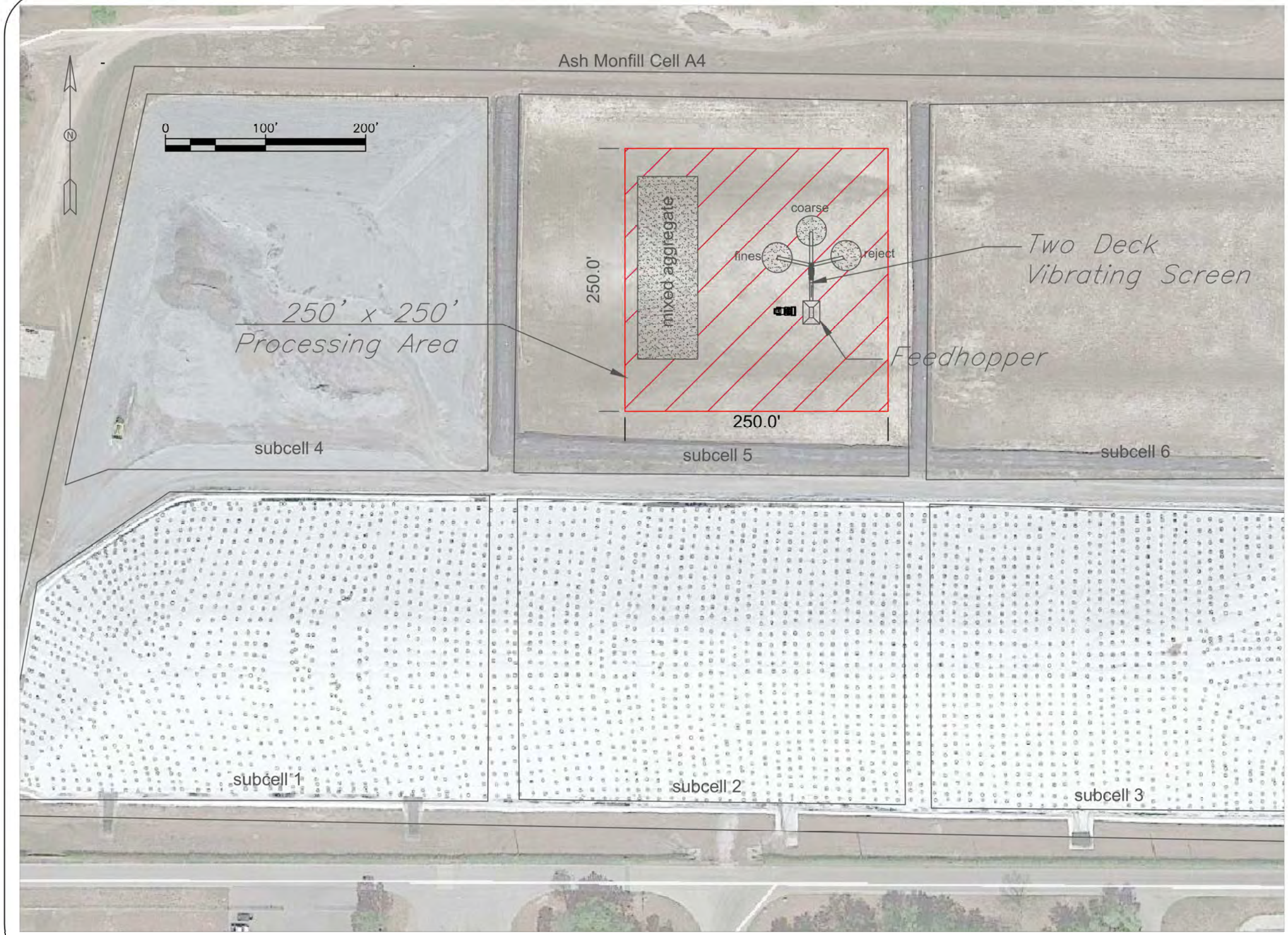
WEST PASCO COUNTY LANDFILL
EXPANSION CELL A-4

PHASE 6 PLAN FOR
CELL A-4

PROJECT NO. 6104-46136
FILE NAME: C011STPL

SHEET NO.
C-11

Xref's: [CDM_2436_CBASE_MODEL C-BASE,]
Filename: L:\2104\46136\Permit2\CIVIL\C011STPL.dwg Last saved by: AUSVD Time: 7/28/2008 8:59:07 AM



Generalized Layout
Ash Processing Equipment

JMG ENGINEERING

Project Name and Address

Pasco County Class I Landfill
14230 Hays Rd
Spring Hill, FL

Project	Sheet
Date 12/18/2018	6-1
Scale 1" = 100'	

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6.6 ENGINEERED AGGREGATE PROCESSING

In 2019, the Pasco County Resource Recovery Facility's Conditions of Certification (PA 87-23D) were updated to include beneficial ash reuse. These are codified in Attachment E Part 6 of the Conditions of Certification – Solid Waste Construction and Operation Requirements. This allows for the use of processed bottom ash as an engineered aggregate in certain roadway and building construction applications. A copy of the relevant section of the conditions of certification is included as **Attachment 1**. To enhance the properties of the bottom ash for use as a construction material, it is necessary to process the bottom ash to achieve a specified gradation. To accomplish this, a two-deck vibrating screen will periodically be leased and positioned on subcell 5 of A-4 as shown on **Figure 6-1**. The frequency of the screening operation is dependent upon the need for engineered aggregate destined for beneficial reuse. The screen will segregate raw bottom ash into three size fractions: > 1 1/4", 3/8" – 1 1/4", and < 3/8". The screening operation will occupy an approximate 250' x 250' area within the cell.

Raw bottom ash will be placed in the screening equipment feedhopper and processed into separate piles of the size fractions discussed above. Processing will continue until each pile is approximately 20-feet high as depicted on **Figure 6-2**. The material greater than 1 1/4" is unsuitable for beneficial reuse and will remain as waste in the ash cell. Depending upon the specified use, the two remaining piles (referred to as the coarse fraction and the fine fraction) will either be used "as-is" or blended together to achieve the desired gradation. In addition to screening of ash, the processing area in A-4 may also be utilized to remove residual ash from ferrous and non-ferrous metals *(additional information on this process is presented in Section 15).*

Figure 6-2
Segregated Aggregate Piles

7.0 MUNICIPAL SOLID WASTE LANDFILL OPERATIONS (SW-1 AND SW-2)

The Solid Waste Landfill portion of the Class I Landfill receives municipal solid waste delivered to the Complex that is unable to be processed by the Resource Recovery Facility because the Facility is either at capacity or is unavailable during maintenance periods. The Resource Recovery Facility is the primary disposal option for all of the municipal solid waste generated within the County, however, it becomes necessary from time to time to divert MSW from the RRF. The diverted waste is directed to disposal cells SW-1 and SW-2. Diversion periods typically last for up to one month before all maintenance at the Resource Recovery Facility is completed and all waste once again is processed at the Resource Recovery Facility.

Following is a detailed description of the fill sequence for the Solid Waste cells (SW-1 and SW-2)

7.1 VALLEY FILL CELLS SW-1 AND SW-2 FILL SEQUENCE

The Class I Landfill at the West Pasco Landfill and Resource Recovery Facility consists of two 10-acre cells – SW-1 and SW-2. Cell SW-1 is nearly at capacity and Cell SW-2 is the active cell. Cell SW-2 was designed with features and filled in such a manner to reduce leachate generation from stormwater. The filling of SW-1 and SW-2 to-date has been conducted as two separate cells, which resulted in a valley between the two cells. The general filling sequence will be accomplished by filling the valley between the cells and then area-filling SW-2 until the final build-out grades are achieved. Before valley filling begins, the temporary rain tarp will be removed, and protective cover soil will be placed in preparation for accepting waste. The filling sequence is as follows:

7.1.1 LIFTS 1 AND 2, VALLEY FILL, PART 1

The following liner preparations and valley filling may be progressed all at one time or in three or more phases depending upon site conditions and operational needs. Work will progress within the valley from west to east.

Liner Preparation

1. Remove temporary rain tarp and stormwater interceptor berm from side slope of SW-1 exposing geocomposite (geonet and geotextile), as shown in **Figure 7**, Stormwater Interceptor Control. Inspect geocomposite for damage. Geocomposite should not be exposed to sunlight for more than 21 days.

2. Install 2 feet of sand protective cover on the geocomposite side slope where the rain tarp has been removed. Verify sand depths on a 50-foot grid by potholing taking care not to damage the geocomposite or geomembrane.

3. Selected waste filling - Selected solid waste will be unloaded and spread over the two feet of protective sand cover. The waste will be inspected for large objects that shall be removed so there is no damage to the liner system. The initial selected waste lift shall be a minimum of 4 feet thick.

4. Fill Lift 1 of the Phase IV valley area between SW-1 and SW-2 from west to east. Fill the area by constructing approximately 10-foot lifts working in the general direction from south to north and west to east. Lift 1 may be progressed across the entire width of the cell or lifts may be placed in phases depending upon site conditions and operational needs. Continue the filling operation until the solid waste match waste elevation in Cell SW-2 as shown in **Figure 7-1** and **Figure 7-2**.

7.1.2 LIFTS 3 AND 4

1. Lifts 3 and 4 will be area fills over the SW-2 cell and the previous valley fill. Filling will generally begin in the northwest and will proceed from north to south in rows such that the lift is filled to the outer buildout grades. Once a row is complete, additional rows will be filled moving from west to east.

2. Lifts 3 and 4 are approximately 10 feet thick and will fill up to elevation 95 feet. Slopes will be modified to meet the final build out criterial of 3H:1V.

7.1.3 LIFT 5

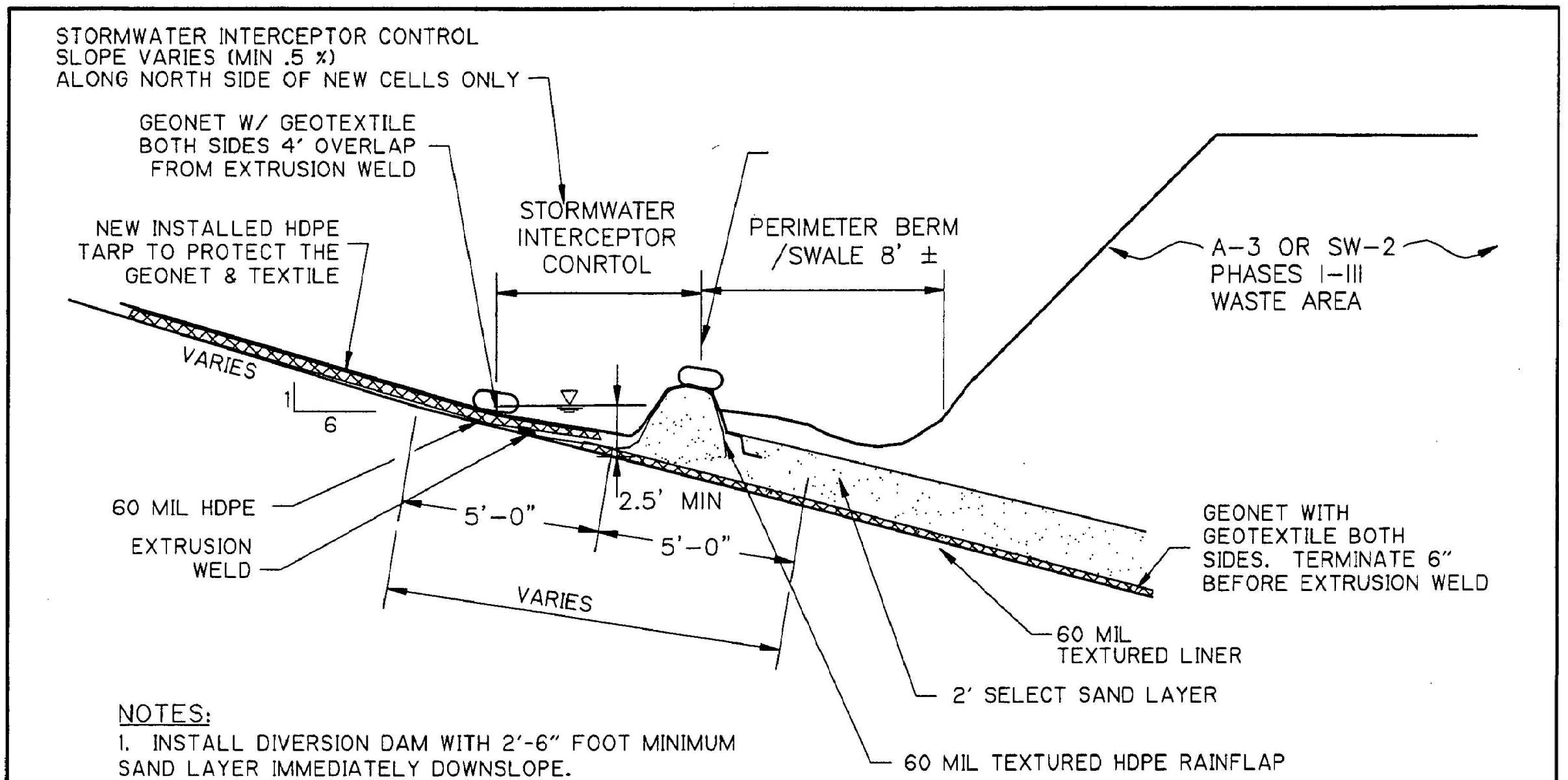
1. Lift 5 will be an area fill over the SW-1 and SW-2 cells. Lift 5 varies in thickness and may be up to 15 feet thick. Filling will generally begin in the northwest and will proceed from north to south in rows such that the lift is filled to the outer buildout grades. Once a row is complete, additional rows will be filled moving from west to east.

2. Lift 5 will be filled so that the outer slopes are at build out grades of 3H:1V.

7.1.4 LIFT 6

1. Lift 6 will be an area fill over the SW-1 and SW-2 cells. Filling will generally begin in the northwest and will proceed from north to south in rows such that the lift is filled to the outer buildout grades. Once a row is complete, additional rows will be filled moving from west to east.

2. Lift 6 is approximately 10 feet thick and will fill up to elevation 111 feet. Slopes will be modified to meet the final buildout criteria of 3H:1V.



NOTES:

1. INSTALL DIVERSION DAM WITH 2'-6" FOOT MINIMUM SAND LAYER IMMEDIATELY DOWNSLOPE.

2. PRIOR TO FILLING IN PHASES, IV, V, AND VI, THE RAINFLAP SHALL BE CUT AWAY, AND THE GEONET W/ GEOTEXTILE TO BE OVERLAPED BY 12" MIN ON TOP OF THE EXISTING GEONET W/ GEOTEXTILE.

3. CONTINUE 2' SAND LAYER ON SIDE SLOPE BEFORE FILLING WASTE/ASH

SCALE: NTS

Figure No. 7
Stormwater Interceptor Control

CDM

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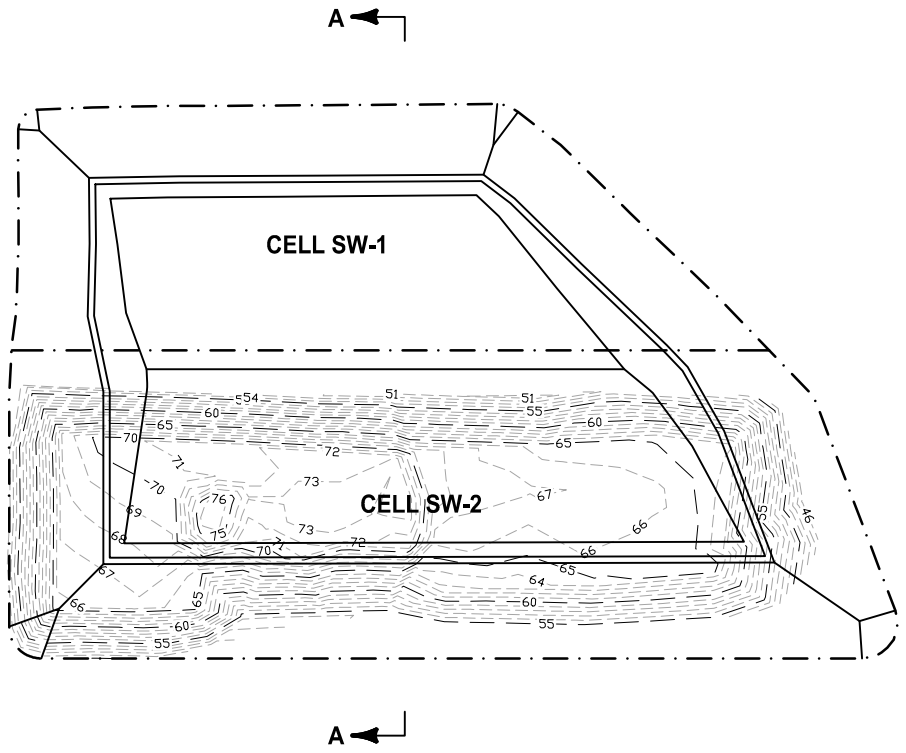
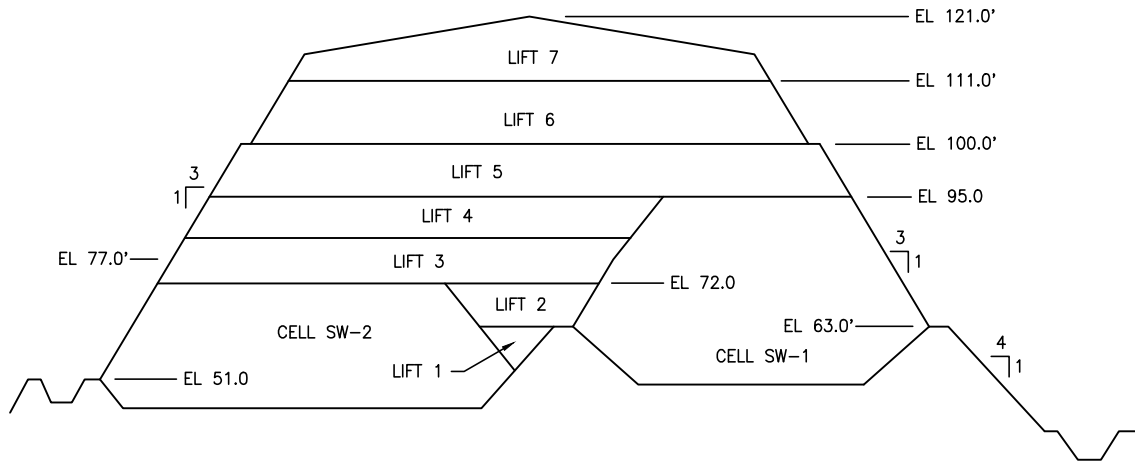


PLAN
NTS



SECTION A-A
NTS

FIGURE 7-1 PHASING PLAN
WEST PASCO LANDFILL AND
RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

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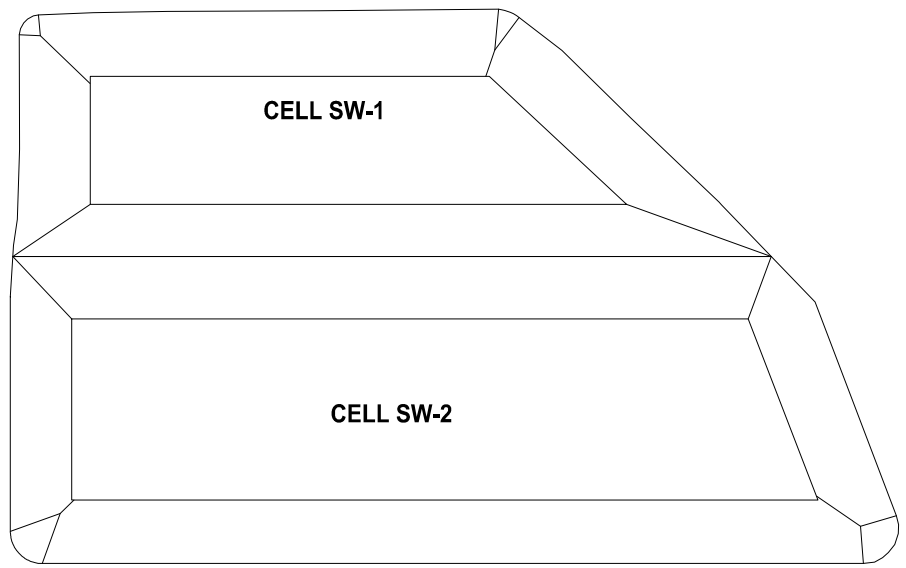
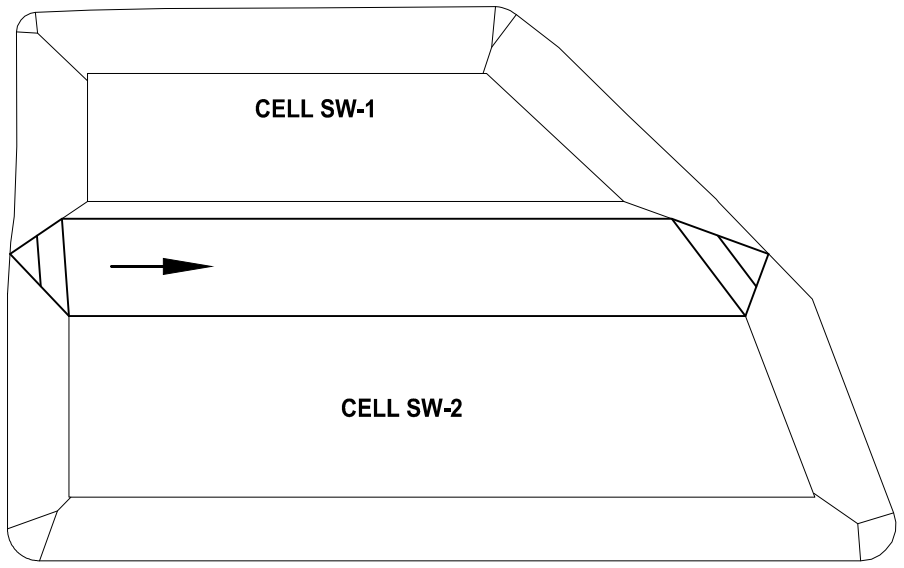


EXISTING CONDITIONS
NTS



LIFTS 1 AND 2
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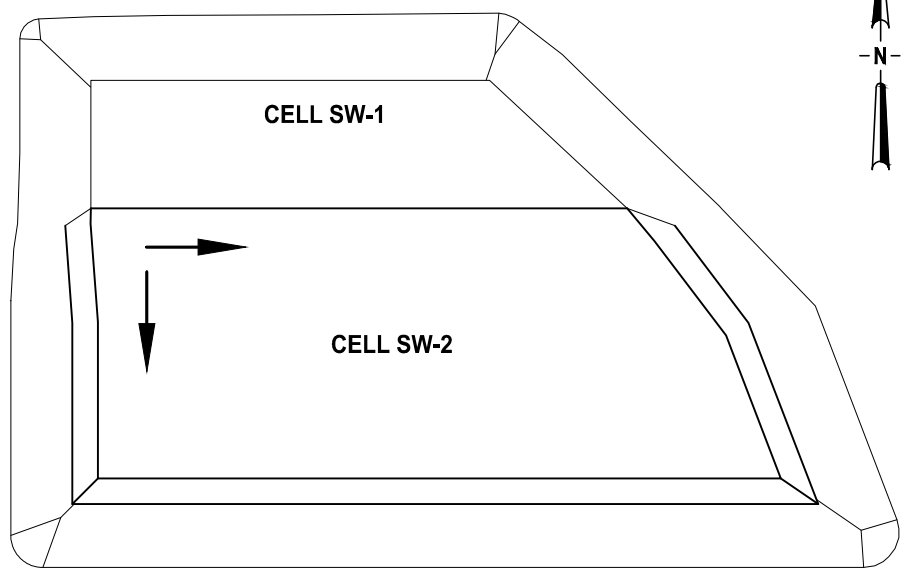
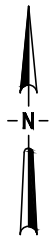
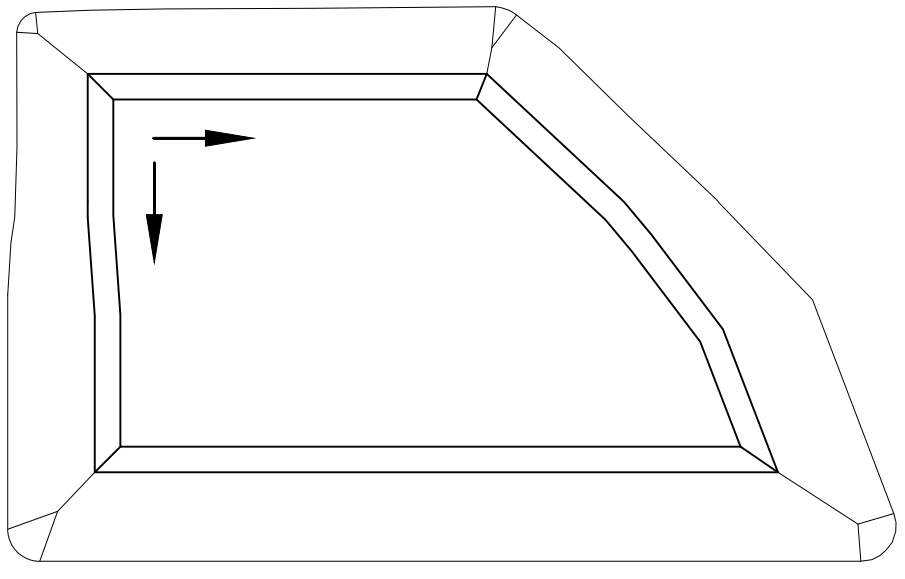
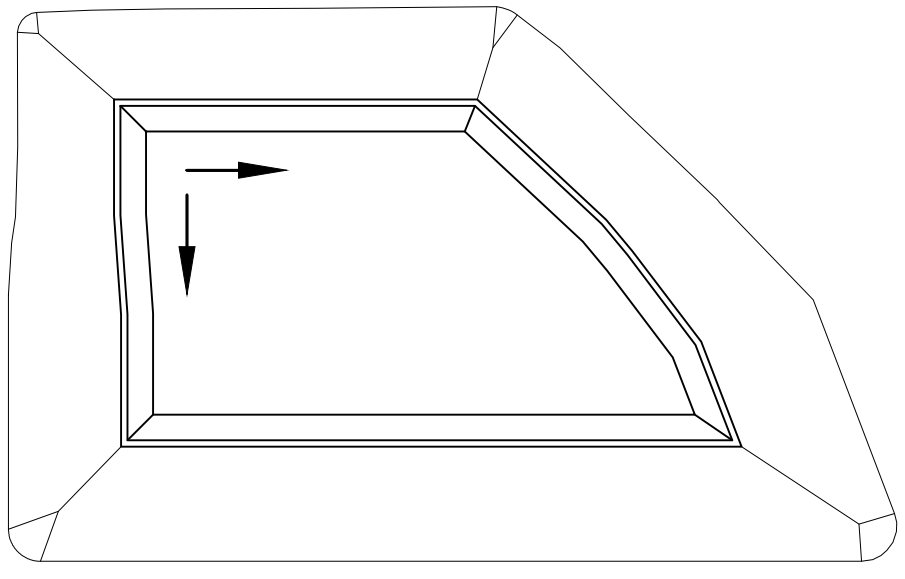


LIFTS 3 AND 4
NTS

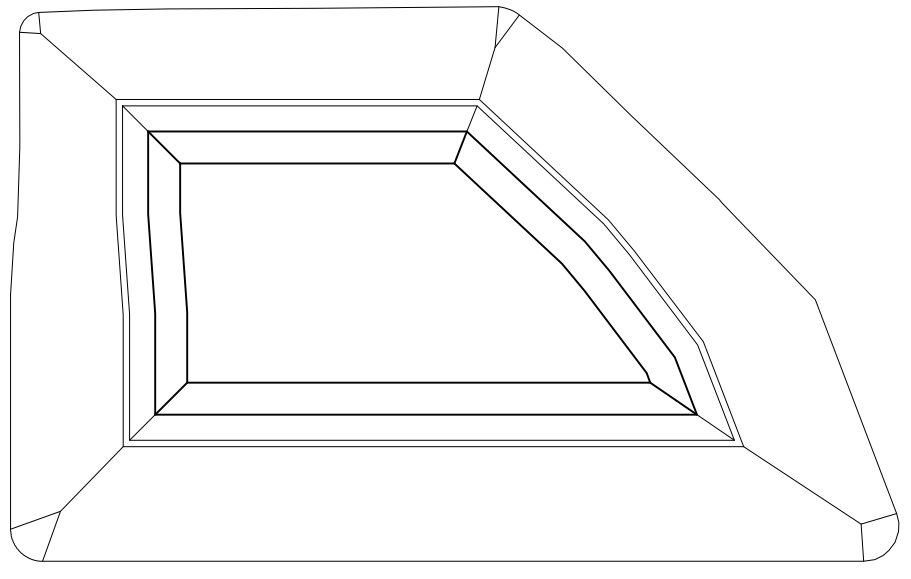




LIFT 5
NTS



LIFT 6
NTS



LIFT 7
NTS

FIGURE 7-2
CELLS SW-1 AND SW-2 FILLING SEQUENCE
WEST PASCO LANDFILL AND
RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

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7.1.5 LIFT 7

1. Lift 7 will be an area fill over the SW-1 and SW-2 cells. Filling will generally begin in the northwest and will proceed from north to south in rows such that the lift is filled to the outer buildout grades. Once a row is complete, additional rows will be filled moving from west to east.

2. Lift 7 is approximately 10 feet thick and will be fill up to an elevation of 118 feet to account for the 3 feet of soil cover resulting in the final elevation of 121 feet.

7.2 SOLID WASTE INSPECTION

All Operators and Spotters are trained according to the requirements of Section 5.0. Pasco County Solid Waste maintains at a minimum one trained Spotter at the active working face during waste disposal operations. The Spotter (trained) is stationed where they can inspect each shipment of waste for unauthorized/prohibited waste before being compacted. The Spotter (trained) will be located on heavy equipment spreading the waste at the working face. Although this section is written to generally address Spotters on heavy equipment, Spotters trained on the ground may be used periodically if Equipment Operators are used that are not trained Spotters. Additionally, trained Operators may also serve as Spotters. Generally, two Operators are onsite available to assist at the active working face.

A Pasco County employee will direct incoming traffic to the appropriate area at the working face. If the Equipment Operator identifies unauthorized/prohibited materials, they will do one of the following as allowed by the Florida Administrative Code:

- Move the unauthorized waste away from the active area for later removal and proper management. These wastes will be moved to temporary storage points as addressed below as soon as ground labor is available (same day).

- Stop operation and notify another person on the ground or on other equipment who will come to the active area and remove the unauthorized waste before operations are resumed.

A Spotter or Operator is inspecting waste at all times when a waste-hauling vehicle is unloading. Waste materials such as white goods, and tires are pulled from the working face and placed in pick-up trucks or to the side away from traffic if the pickup truck is not close to the Spotter. To ensure the health and safety of the spotters, if tires are observed in the waste while the waste is being pushed, the tires will be staged at the end of the slope and picked up at the end of the day. Other landfill staff have been trained to be aware of materials that are prohibited from disposal in a Class I Landfill and to help identify and remove these materials as required.

If suspicious wastes are identified, the waste load is cordoned off and the Landfill Operations Supervisor is contacted. They will examine the suspicious waste and determine whether to accept or reject the load. Loads are rejected if they contain hazardous waste or prohibited material that cannot feasibly be separated from the rest of the load or when

separation would cause possible contamination such as the presence of liquids or powders that cannot be removed from the landfill once the load is dumped. If the waste material is determined to be an unacceptable waste, the hauler is contacted and asked to remove the waste themselves from the waste stream. All incidents of attempted unauthorized waste disposal will be documented and maintained in the Administration Office.

Scavenging and salvaging by the employees or the public are strictly forbidden at the landfill for the safety of everyone at the landfill. Employees have been informed of this prohibition.

7.3 HAULER QUEUING

The Pasco County employees (who may be trained Spotters) at each working area shall control the number of vehicles allowed at that working face to afford comfortable maneuverability. Additional vehicles will be held at a distance from the working face to allow existing vehicles sufficient room to maneuver and unload.

Unloading vehicles should be spaced adequately to allow the crews sufficient space to work safely. For their own safety, vehicle crews are not permitted to wander from their vehicle for any reason. As vehicles leave the working face, the next vehicle is directed to a vacant unloading area. Pasco County employees move traffic into and out of the working face as quickly as possible while maintaining safe working conditions.

The trained Landfill Equipment Operators or Spotters inspect all loads discharged from the unloading vehicles, remove all unacceptable wastes, and spread the waste as soon as the vehicle moves. Another vehicle is permitted to unload its waste load in an area only after the previous waste load has been inspected, the prohibited wastes have been removed, and the load has been moved to the working face for compaction.

7.4 WASTE PLACEMENT AND COMPACTION

The active face width will be kept as small as practical to accommodate waste delivery vehicles. The actual working face width may vary depending on the daily vehicle traffic volume. The maximum width of the active face will be maintained at approximately 250 feet. Landfill Management may consider expanding the size of the working face if the backup of waste vehicles become too large.

For health and safety reasons and due to weather, fill-sequencing two working faces may be necessary to accommodate traffic in some situations. This second working face will be bermed and use the same traffic area. This area will be in the immediate vicinity of the Primary Active Area. Given the limited use of the Solid Waste cells, the use of two working faces is unlikely.

Solid waste shall be formed into horizontal lifts to construct cells. The working face of the cell and side grades above land surface shall be at a slope no greater than 3 feet horizontal to 1 foot vertical rise (3H:1V). Normal lift depths should not exceed 20 feet of compacted

solid waste. However, during initial filling of the bottom lift of waste or when completing the last lifts in a phase, larger depths may be necessary.

7.5 Cover Materials and Methods

All soil used for daily, intermediate, and final cover over the required impervious cover or synthetic liner required for closure consists of material excavated from the landfill property, other County-owned lands, or purchased from offsite sources.

7.5.1 Initial Daily Cover

At the end of each working day, a compacted 6-inch layer of soil (daily cover) shall be placed over the working face to control odor, litter, fires, and vector control. Pasco County will use Alternative Daily Cover (ADC) in place of soil for daily cover, such as plastic tarps, Posi-Shell or other FDEP-approved materials, to extend the life of onsite soil sources. This includes a 50% soil and mulch mixture. Posi-Shell or mulch may also be utilized to stabilize erosion of covered side slopes as necessary.

Daily cover material is applied in a manner that prevents wasting material or exposing wastes. In areas where the initial waste layer is being installed or the lift is near the side slope, daily cover shall be applied from the top down the face of the fill cell. Inner slopes or plateaus are covered from the top or bottom depending on working conditions.

Throughout the day, cover soil and mulch may be delivered to the active area and spread when possible to reduce the area needing cover at the end of the working day. Additional landfill equipment is used to transport cover soil to the working face area. A dozer is used to spread the cover soil and compact it over the working face. Pasco County maintains excavating equipment and off-road dump trucks to perform this work.

7.5.2 Intermediate Cover/Temporary Cover

Intermediate cover consisting of 12 inches of soil will be placed on areas that are not expected to receive additional wastes within 180 days. This intermediate cover is in addition to the 6-inch initial cover required. When an ADC is used in lieu of the 6-inch initial soil cover, the intermediate cover shall be 18 inches thick. Before additional wastes are placed in areas previously covered with intermediate cover, the intermediate cover will be removed and stockpiled adjacent to the active face to use as daily cover.

7.5.3 Final Cover

Cell closure will begin immediately after final receipt of waste, when all grades have reached their final elevations. Final cover and seeding or planting of vegetative cover will be fully described in a separate Application for Landfill Closure Permit.

7.6 Special Waste Recovery

Special wastes prohibited from disposal in the Class I Landfill are addressed in Section 4.2. Small amounts of special and hazardous wastes may be contained within a load of refuse

unloaded from a vehicle. While spreading the wastes onto the working face from a discharged load, the Spotters/Equipment Operators will be looking for special and hazardous wastes, which will be removed from the discharged load before being compacted into the landfilled waste. These prohibited waste materials will be placed to the side of the working face, in pick-up trucks, or 20-yard bins that are away from vehicle traffic. The materials collected in the bins and pick-up trucks will be removed from the site daily. Spotters/Equipment Operators will remove items such as white goods and tires more frequently throughout the day if a stockpile accumulates. These materials are taken to the white goods and tire storage areas within the Solid Waste Complex.

Operators are to be aware of containers that could contain prohibited hazardous wastes. These containers include the following:

- 55-gallon drums.
- 5-gallon chemical drums or paint containers.
- Marked medical wastes.
- Small containers of paints or pesticides and other HHWs.

As discussed in Section 4.2, these non-allowed wastes are isolated and the Landfill Operations Supervisor will be contacted for closer inspection. The vehicle will be detained until the material is identified for proper disposal. If the material is identified as a hazardous waste, it is loaded back into the hauler vehicle for proper disposal, if practical. Otherwise, the area should be cordoned off to prevent other loads from being dumped in the vicinity. The Solid Waste Director will contact a contracted hazardous waste hauler. The identified hazardous waste will be loaded onto this vehicle as soon as it arrives at the landfill. All identified hazardous wastes must be removed the same day they are identified or stored in the Household Hazardous Waste Area if transportation cannot occur until the next working day.

7.7 Litter Control

Litter fences will typically be used near the active cells and working face to collect and prevent the spread of litter. Due to the shifting waste operations, fences may periodically not be in place. Litter that escapes the landfill working area shall be picked up by the end of the working day but no later than the next working day. The Landfill Operations Manager or Landfill Construction/Lead Operator inspects the normal traffic areas surrounding SW-2 daily when the cell is in use. The operational boundaries of the entire Solid Waste Complex are inspected weekly.

Pasco County staff will collect litter continually on windy days. Temporary labor may be used as needed to support County staff for litter control if the volume of litter becomes too much for existing staff. The use of trustees and community service workers is also available as needed.

All open vehicles delivering waste to the Pasco County Solid Waste Complex must have a tarp or some type of enclosure to prevent litter on the site as well as all roads within the

County, as required by Florida law. All open-top vehicles entering the Complex without a tarp or enclosure will be informed by the Scalehouse Attendant that a tarp or enclosure is required to transport solid waste within the County.

8.0 SPECIAL WASTE MANAGEMENT

8.1 HOUSEHOLD HAZARDOUS WASTE

Household hazardous waste (HHW) is accepted for disposal at the HHWCF north of the Class III Landfill (see **Figure 1-1**). This facility is operated for Pasco County residents Monday through Saturday,
7:00 A.M. to 5:00 P.M.

The personnel at the facility manage a diverse range of HHWs according to the characteristics of each waste. All personnel at the HHW are 40-hour HAZWOPER-certified and receive 8-hour refresher courses annually. The facility is capable of accepting and managing nearly any waste produced in a household including oils, paints, solvents, fertilizers, pesticides, herbicides, fluorescent tubes, oxidizers, and propane. However, explosive, radioactive, and bio-hazardous materials like hypodermic needles, lancets, and wound dressings are not accepted for disposal at the HHHW.

At the HHW, personnel only accept materials for disposal from households within Pasco County. Wastes from Conditionally Exempt Small Quantity Generators (CESQGs) may be accepted under specific circumstances, such as a special business waste collection event. However, if a customer has an especially large amount of waste, if they have numerous containers of a particular brand of product, or if the waste they have brought for disposal is typically produced by a business, the customer will be asked additional questions. These questions may include, but are not limited to, what the product was used for.

Once collected, wastes are disposed of using a hazardous waste transporter and delivery to treatment, storage, and disposal facilities (TSDFs) operating under a permit issued pursuant to Subtitle C of the Federal Resource Conservation and Recovery Act (RCRA) and issued by the U.S. Environmental Protection Agency (USEPA) or an authorized state.

All packaged materials are shipped by a licensed hazardous waste transporter every 60 days to a fully permitted TSDF as defined in 40 CFR 264. No waste remains at the HHWCF for more than 120 days.

8.2 UNKNOWN AND UNMARKED WASTES

If containers accepted for disposal are unmarked, have illegible labels, or have contents that do not resemble their labels, the customer is questioned about the purchase and use of the material to determine its waste characteristics (i.e., flammability, reactivity [oxidizers], toxicity, or corrosivity). Many household products have uniquely shaped containers to help determine their use and waste characteristic. Materials are further grouped according to physical appearance (e.g., solid, liquid, color, viscosity etc.). The unknown material is then stored in chemical storage lockers with blast-proof doors. Ultimately, Pasco County uses their Hazardous Waste Contractor to identify waste types before bulking or lab packing the material for disposal.

Liquids suspected of being acidic or basic are tested using pH paper. Smelling a container to determine its contents is forbidden at the HHW. However, most poisons, petroleum products, and some cleaners have strong, distinct aromas that are detectable simply by opening their container. Any HHWs that are found to be potentially reactive by these tests are stored separately from antagonistic chemicals (e.g., acids are stored in the locker marked "Corrosive"). The caustic materials are stored in the "Corrosive" locker. Flammable liquids must be kept away from oxidizers, so they are stored in the "Flammable" locker while the oxidizers are stored in the "Corrosive" locker. Regardless of the results of these tests, all collected materials are stored at the HHW until they have been prepared for reuse or disposal. Any material determined to be reactive is stored in a locker with fire suppression and blast-proof doors. If material in unlabeled drums can be identified, it will be bulked with compatible material.

8.3 Used Oil

Used motor oils, hydraulic fluids, transmission fluids, and similar petroleum lubricants are consolidated into 55-gallon drums. The Used Oil Contractor is responsible for testing for the halogen content in these materials when they are collected. The drums are removed by a contracted Used Oil Contractor. Used oil is not permitted to be disposed of in the Class I Landfill.

8.4 Latex Paints

The HHW uses a wire rack to store 1- and 5-gallon paint cans. These racks are typically capable of holding about 60 cans of paint. Approximately two-thirds of the paint that arrives at the HHW is latex-based. Useable latex paint is opened and consolidated into 5-gallon buckets. This paint is given to citizens of Pasco County.

Paint that is not suitable for give-away is combusted at the Resource Recovery Facility.

8.5 Oil Based Paints

The HHW uses wire racks to store 1- and 5-gallon paint cans. These racks are typically capable of holding about 60 cans of paint. Approximately one-third of the paint that arrives at the HHWCF is oil-based. Useable oil-based paint is consolidated into 5-gallon buckets. This paint is given to citizens of Pasco County. Paint that is unusable is bulked into 55-gallon drums for disposal. A hazardous waste transporter moves the filled drums to a fully permitted TSDF for proper disposal.

8.6 Poisons

Herbicides and pesticides are the largest group of poisons that are accepted at the HHW. They are prepared as "combination packages" or "lab packs" for disposal by the Hazardous Waste Contractor. Small individual containers are placed with similar wastes into a larger container (5-gallon bucket or 55-gallon drum) with Vermiculite to provide shock absorption and hold any liquids that may escape during shipment. The material is moved by a

hazardous waste transporter to a fully permitted TSDF for proper disposal. A few specialized wastes such as pentachlorophenol and arsenic compounds are packed by and managed separately by the Hazardous Waste Contractor. These specialized wastes are shipped by a hazardous waste transporter to a fully permitted TSDF in the same way that the lab packs are. However, they are subject to different treatment standards.

8.7 Lead Acid Batteries

Lead-acid batteries are stacked on wooden pallets. The stacked batteries are then shrink wrapped onto the pallets and stored on a large containment pallet inside the HHW. The containment pallet has enough room to store three wooden pallets and have room for other products. The wooden pallets are picked up by a Lead-Acid Battery Contractor for recycling.

8.8 Electronics Recycling

Electronics waste, including televisions, VCRs, computer towers, monitors, digital cameras, and PDAs are collected and stored at the HHW for recycling. Electronics wastes are stored and managed in a manner that prevents breakage. The e-waste is stored in enclosed roll-off containers and transported to an electronics recycler.

8.9 Contaminated Soils

Pasco County will accept minor amount of contaminated soils for disposal in SW-2. Soil that has been contaminated with petroleum products or any other materials that are not hazardous wastes may be disposed of in permitted Class I landfills. Petroleum-contaminated soil that has been treated pursuant to Chapter 62-713, FAC may be disposed of at permitted disposal facilities and may, if it meets the criteria of Rules 62-701.200 (53) and (55), FAC, be used as initial or intermediate cover material at solid waste disposal facilities. Contaminated soil that has the potential to leach constituents in excess of FDEP groundwater standards or criteria may be used only at Class I landfills and only in those areas of the landfill where runoff or infiltration is captured by the leachate collection system.

8.10 Biological Wastes

Bodies of captive wildlife, as well as bodies of domestic animals that have not died due to disease, may be disposed of in the SW cells of the Class I Landfill.

Disposal of bodies of domestic animals, after death due to natural or accidental causes or euthanasia, shall be accomplished according to Section 823.041(1), FS. This statute requires the disposal of the bodies of domestic animals by burning or burying at least 2 feet below the ground surface. This provision does not prohibit the disposal of such animals in Class I landfills.

Disposal of dead poultry and hatchery residue shall be accomplished according to Section 583.181(2), FS, which provides for disposal of dead poultry and hatchery residue by every poultry producer, egg producer, and poultry hatchery in Florida. The statute includes disposal of the dead poultry and hatchery residue in a sanitary landfill as an acceptable

method of disposal. Furthermore, the transporter of the poultry or hatchery residue will transport the material in containers that are sufficiently sealed to prevent spillage.

Treated biomedical waste (i.e. autoclaved waste) is not accepted at the Class I landfill but is accepted at the Resource Recovery Facility. Biomedical waste that has been treated may be disposed of as solid waste that is not biomedical. Such treated waste must be in containers clearly labeled with the phrase "Treated Biomedical Waste." The local governments that are responsible for solid waste collection and disposal shall be notified that treated biomedical waste will be disposed of in their facility before such disposal. All transport vehicles transporting treated biomedical waste to a solid waste facility for disposal shall be fully enclosed and secured when unattended. Treated biomedical waste shall be disposed of only at permitted Class I landfills or incinerators used to combust solid waste.

8.11 TIRES

Segregated tires that are brought to the Facility are stored and processed in a Waste Tire Processing Area just north of the Class III Landfill. A chipping contractor is brought in approximately every six months to process the tires, which are then hauled to a permitted off-site disposal facility.

8.12 WHITE GOODS

White goods are removed from the site at least every 3 months. White goods containing chlorofluorocarbons (CFCs, such as Freon) are stored upright and managed carefully so that CFCs are not discharged to the atmosphere. White goods with the refrigerant removed are marked as such. Because Freon is extracted from all units, the compressors are not removed. Other scrap metals, such as lawnmowers, are processed (by removing tires, oil, and gasoline, as needed) and compacted with the white goods into 40-cubic-yard containers.

9.0 LEACHATE MANAGEMENT

As defined in Rule 62-701.200 (66), FAC, leachate is liquid that has passed through or emerged from solid waste and may contain dissolved, suspended, or mixed materials. Leachate must be contained and kept separate from any groundwater or surface waters. This section will provide Pasco County Solid Waste employees with a general understanding of the requirements for managing the leachate generated from the Class I (Ash and Solid Waste) Landfill operations. **Figure 9-1** identifies all locations for the leachate collection system. Rule 62-701.500(8) FAC establishes requirements for leachate management and provides the criteria for this section.

9.1 LEACHATE REDUCTION

Leachate is generated from rainfall that is absorbed into the landfill and water within the waste at the time of disposal. The leachate is collected by the Class I Landfill bottom liner system and treated as wastewater. One of the goals of the landfill design and daily operation is to minimize leachate production from the landfill to reduce the cost associated with leachate treatment and to minimize the potential environmental contamination risks. The methods described in this section can be used separately or simultaneously to achieve leachate reduction.

9.1.1 RAIN COVERS

In addition to the leachate/stormwater separation intrinsic to the design of the A-4 and SW-2 cells, Pasco County may use rain covers (or tarpaulins) to minimize leachate generation. Areas of the landfill that will not be utilized for at least 180 days may be covered with a 20-mil geomembrane and anchored using tires. Stormwater that collects on the rain cover in areas utilizing this technique is managed as non-contact stormwater, which discharges to the perimeter stormwater ditch.

9.1.2 STABILIZED SIDESLOPES

As filling progresses, side slopes that will not receive solid waste for 2 months or more will be stabilized with sod, seed and mulch, or rain covers. Exterior side slopes that are up to design grade and interior side slopes that will not be filled again for longer than 180 days will be covered with either intermediate cover and sod or a rain cover.

9.1.3 CLOSING

Pasco County Solid Waste plans to achieve an effective runoff factor of at least 30% and will work toward a goal of 50% or more upon final closure in the Class I Landfill. The methods described above represent the present plan; however, as operations continue, they may be modified if alternate methods prove more efficient or allow a higher percentage of stormwater runoff resulting in greater leachate minimization.

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Note: THIS FIGURE IS NOT TO SCALE AND THE LOCATIONS OF SOME ITEMS HAVE BEEN EXAGGERATED FOR CLARITY

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LEACHATE COLLECTION SYSTEM
WEST PASCO CLASS I LANDFILL

FIGURE
9-1

9.2 LEACHATE COLLECTION

The leachate collection system encompasses the Ash Monofill (Cells A-1, A-2, A-3, and A-4) and the Solid Waste Landfill (Cells SW-1 and SW-2). Monitoring the collection, pumping and storage components occurs daily, Monday through Friday. All of these phases have been included in this Operation Plan.

9.2.1 ASH MONOFILL CELL A-1

Cell A-1 of the Ash Monofill is the original cell of the landfill constructed in 1990. It is approximately 10 acres in size and received ash between approximately 1990 and 1995. The cell was constructed with a double liner system, with intrinsic leachate removal systems for both liners (primary and secondary).

Leachate in the cell is collected by perforated laterals running north-south which drain to a 12-inch PVC leachate header pipe that runs (east to west) on the southern perimeter of the cell. The leachate collection pipes (primary and secondary) penetrate the liner(s) and discharges into one of two leachate manholes on the east side of the cell, one for the primary leachate collection system and one for the secondary leachate collection system. Leachate from the secondary liner is allowed to accumulate in the metering manhole and is periodically pumped (and metered) into the primary manhole. Leachate drains from the metering manholes by gravity to a pump station adjacent to the metering manholes. From the pump station, leachate is pumped to a 2,000,000-gallon concrete storage tank for eventual disposal off-site.

9.2.2 ASH MONOFILL CELL A-2

Cell A-2 was constructed circa 1995 and is of a similar configuration to that of Cell A-1. The leachate collection system for Cell A-2 was modified circa 2000 from its original design by converting the leachate evacuation method from a pumped principle to a gravity principle. The original design and construction of the cell directed leachate from the primary liner system and the secondary liner system to headers running east-west along the northern perimeter of the cell. The headers discharged into a sump in the northeast corner of the cell, and the accumulated leachate was pumped through a side-slope riser pipe to the pump station located adjacent to cell A-1. In 2000, the side slope riser pumping system was abandoned, and new gravity pipes with liner penetrations (one for leachate collected by the primary liner and one for leachate collected by the secondary liner) were installed and directed to new leachate metering manholes. As with Cell A-1, leachate flows by gravity from the metering manholes to the master pump station adjacent to A-1. From the pump station, leachate is combined with the leachate from Cell A-1 and pumped to a 2,000,000-concrete storage tank for eventual disposal off-site.

In 2016, during a routine evaluation of the effectiveness of the landfill leachate collection and removal system, it was discovered that little to no leachate was flowing from the primary liner sump to the metering manhole. It has been determined that the most likely cause of this problem is calcification of the stone within the sump, thus preventing leachate from draining through the gravity pipes. To remedy the situation the County has been utilizing a diesel-powered pump to pump the primary leachate from the 8-inch HDPE

leachate clean-out pipe stub on the east side of the cell to the primary metering manhole of Cell A-2. The County is currently in the design process of replacing the temporary diesel pump with permanent infrastructure consisting of the following:

- One submersible pump of 1.0-HP, (Grundfos Model 30 SQE 05-40 or equal) located inside the existing 8-inch HDPE primary leachate collection clean-out pipe which connects to the leachate collection sump. The submersible pump is electrically powered and sits at the bottom of the clean-out pipe to pump out primary leachate through a half-inch or greater size discharge tubing into a new 3-inch leachate forcemain.
- A 3-inch HDPE pipe connection to the existing 8-inch HDPE leachate clean-out pipe stub on the east side of Cell A-2. This new 3-inch pipe is the new leachate forcemain and is buried at least 3 feet below grade from the east side of Cell A-2 and crosses the access road and discharges into the primary wet-well of Cell A-2. The road crossing is accomplished by using a 12-inch steel casing extended beyond the paved road limits by at least 5 feet.
- A control panel integral with the submersible pump which offers controls over the submersible pump and has an emergency hand 'ON/OFF' switch. A magnetic flow meter which is strapped on the 3-inch forcemain and measures leachate flows. Instrumentation with the control panel which transmits electronic signals for recorded flow through a fiber optic cable to provide output and record flow at the leachate building.
- An 8 feet x 12 feet concrete pad on the east side of Cell A-2 to house the above grade parts of the 3-inch primary leachate forcemain, flow meter and control panel.
- A new 120v single phase electrical power connect stub-out on the concrete pad/control panel as a potential measure to power a new submersible pump for pumping secondary leachate in case the secondary leachate cannot flow by gravity in future. The existing 3-inch leachate forcemain is capable to convey the additional flow for secondary leachate.

9.2.3 Ash Monofill Cell A-3

Cell A-3 was constructed circa 2003 and is of a similar configuration to cell A-1. It is approximately 10 acres in size and received ash between approximately 2003 and 2008. The cell was constructed with a double liner system, with intrinsic leachate removal systems for both liners (primary and secondary).

Leachate in the cell is collected by perforated 8-inch PVC leachate header pipes that run (east to west) on the interior of the cell. The leachate collection pipes (primary and secondary) penetrate the liner(s) and discharges into one of two leachate manholes on the east side of the cell, one for the primary leachate collection system and one for the secondary leachate collection system. Leachate from the secondary liner is allowed to accumulate in the metering manhole and is periodically pumped (and metered) into the primary manhole. Leachate drains from the metering manholes by gravity to a pump station adjacent to cell A-1. From the pump station, leachate is combined with the leachate from Cells A-1 and A-2 and pumped to a 2,000,000-gallon concrete storage tank for eventual disposal off-site.

9.3 ASH MONOFILL CELL A-4

Cell A-4 was constructed circa 2008 and currently serves as the active ash disposal cell for the landfill. It is approximately 20 acres in size and has been receiving ash since 2009. The cell was constructed with a double liner system, with intrinsic leachate removal systems for both liners (primary and secondary).

Leachate in the cell is collected by perforated laterals running north-south which drain to four 12-inch PVC leachate header pipes that run (east to west) on in the interior of the cell. The leachate collection pipes (primary and secondary) penetrate the liner(s) and discharges into one of two leachate holding tanks on the west side of the cell, one for the primary leachate collection system and one for the secondary leachate collection system. Leachate drains from the holding tanks by gravity to a pump station adjacent to the holding tanks. From the pump station, leachate is pumped to a 2,000,000-gallon concrete storage tank for eventual disposal off-site.

9.3.1 9.2.5 SOLID WASTE CELL SW-1

Cell SW-1 of the Solid Waste Landfill is the original cell of the landfill constructed in 1990. It is approximately 10 acres in size and received waste between approximately 1990 and 1998. The cell was constructed with a double liner system, with intrinsic leachate removal systems for both liners (primary and secondary).

Leachate in the cell is collected by perforated laterals running north-south which drain to an 8-inch PVC leachate header pipe that runs (east to west) on the southern perimeter of the cell. The leachate collection pipes (primary and secondary) penetrate the liner(s) and discharges into one of two leachate manholes on the east side of the cell, one for the primary leachate collection system and one for the secondary leachate collection system. Leachate from the secondary liner is allowed to accumulate in the metering manhole and is periodically pumped (and metered) into the primary manhole. Leachate drains from the metering manholes by gravity to a pump station adjacent to the metering manholes. From the pump station, leachate is pumped directly to the adjacent Shady Hills Wastewater Treatment Facility.

9.3.2 SOLID WASTE CELL SW-2

Cell SW-2 of the Solid Waste Landfill was constructed circa 2001. It is approximately 10 acres in size and has been receiving waste since 2002. The cell was constructed with a double liner system, with intrinsic leachate removal systems for both liners (primary and secondary).

Leachate in the cell is collected by perforated laterals running north-south which drain to an 8-inch PVC leachate header pipe that runs (east to west) on the southern perimeter of the cell. The leachate collection pipes (primary and secondary) penetrate the liner(s) and discharges into one of two leachate manholes on the east side of the cell, one for the primary leachate collection system and one for the secondary leachate collection system. Leachate from the secondary liner is allowed to accumulate in the metering manhole and is periodically pumped (and metered) into the primary manhole. Leachate drains from the

metering manholes by gravity to a pump station adjacent to the metering manholes. From the pump station, leachate is pumped directly to the adjacent Shady Hills Wastewater Treatment Facility.

9.4 ASH LEACHATE STORAGE TANK

As described above, leachate that is collected in the ash disposal cells is pumped to a 2-million gallon concrete storage tank located to the south of cell A-3. From the pump station, the leachate flows through a 6" PVC forcemain, through a totalizing meter, and into the storage tank. The tank has a 100-foot interior diameter and is 34 feet tall (excluding the domed roof). Each foot of depth in the tank holds approximately 60,000 gallons.

9.4.1 TANK INSPECTIONS

Pasco County staff visually inspect the exterior of the tank weekly. The inspector will look for any structural damage to the tank, damage to the coating system, loose connections, visible leaks, and maintenance deficiencies. The inspector also looks for any structural damage to the secondary containment system (described below) and visible leaks.

The interior of the tank is inspected at least once every 5 years. During an interior inspection, the inspector looks for any damage to the interior two-part epoxy coating system, structural damage or cracking of the tank, and/or visible leaks.

If inspections reveal any deficiencies with the interior and/or exterior of the tanks that could result in the system failing to contain leachate, Pasco County will take immediate action to remediate the situation. The tank manufacturer (Crom Corporation), coordinating with Pasco County, will handle failures or damage to the tank. Pasco County will immediately notify the manufacturer of the situation; the tank manufacturer will perform a detailed damage assessment report and remediation of the tank. FDEP will be immediately notified in writing by Pasco County of the situation and of the proposed corrective action for significant deficiencies that require more than 48 hours to repair.

9.4.2 SECONDARY CONTAINMENT SYSTEM

The secondary containment design includes a bermed area lined with 60-mil HDPE geomembrane. This area is designed to provide 2,200,000 gallons of storage capacity or 110 percent of the tank volume. Two feet of freeboard is provided at the maximum design capacity. The containment area liner material is high density polyethylene (HDPE), which is compatible with the leachate. The liner is continuous under the storage tank slab, separated by a soil drainage layer for protection of the liner.

The secondary containment area is designed with a 12-inch drain line which discharges to a stormwater swale leading to stormwater retention pond No. 1. The 12-inch discharge line is designed to discharge up to 5.4 cubic feet per second (cfs). This rate is equal to the accumulation rate of a 25-year/24-hour storm event at the point in time when 10 percent of the secondary containment volume has accumulated.

A valve is also provided on the gravity discharge line for the secondary containment system. The valve will normally remain closed. Within 24 hours of any significant rainfall accumulation, Stormwater collected in secondary containment will be visually inspected to determine if the stormwater has been contaminated. Signs of contamination include the following:

- An oily sheen on the surface of the liquid.
- A dark or nontransparent appearance of the liquid.
- An excess of suspended solids in the liquid.
- An odor coming from the liquid.

If no contamination is noted, the valve will be opened to discharge the accumulated stormwater. Once the stormwater is drained from the secondary containment area, the valve will be closed by the operator. If it is contaminated, the stormwater will be treated as leachate and pumped to the storage tank.

9.5 Ash Leachate Disposal

Once produced, leachate must be disposed of. Disposal of leachate generated by the ash disposal cells is achieved by loading leachate onto tanker trucks and hauling it to a WWTP for treatment. Leachate is normally loaded into tanker trucks from a dedicated pump station located adjacent to the 2-million-gallon storage tank, but can also be loaded from different areas (such as the metering manholes or the pump station sump) as needed based on the conditions at the site. The primary disposal mechanism for leachate generated from the ash monofill is the City of Tampa's Howard F. Curren Wastewater Treatment Facility. Pasco County maintains a contract with the City of Tampa for leachate disposal rights at the wastewater treatment plant. If changes in the facility receiving leachate occur, FDEP will be notified.

9.6 Solid Waste Leachate Disposal

Leachate collected by the leachate collection systems serving Cells SW-1 and SW-2 is pumped directly to the adjacent Shady Hills Wastewater Treatment Plant, owned by Pasco County, for disposal. An intra-Department account has been established to charge Solid Waste for the amount of leachate delivered to the wastewater plant.

9.7 Leachate Monitoring, Data Collection, and Reporting

Rule 62-701.500(8)(f), F.A.C. requires that the quantity of leachate collected by the leachate collection and removal system be recorded in gallons per day before on-site treatment and transport off-site. To accurately record the amount of leachate collected, Pasco County utilizes in-line magnetic flow meters on the discharge side of the three pump stations (the first serving A-1, A-2, and A-3; the second serving A-4; and the third serving SW-1 and SW-2). Each of the flow meters totalizes the volume of leachate (in gallons) passing through the respective leachate transmission pipeline. On a daily basis (Monday through Friday), an Operator records the totalized value and the time of day that

the reading was recorded. The records are compiled on monthly basis into a spreadsheet and submitted semi-annually to the FDEP.

Separate from the flow measuring requirements of 62-701.500(8)(f) is an obligation to monitor the effectiveness of the liner systems serving each of the individual cells. Pasco County accomplishes this by separately collecting leachate generated off of the secondary liner system(s). Anything but a trivial amount of leachate collected off of a secondary liner system could indicate a possible breach (or other problem) of the primary liner. On a daily basis, Pasco County operators inspect the metering manholes for all of the secondary liner systems. Anything more than a small amount of leachate collected in a secondary liner metering manhole is immediately reported to the Solid Waste Manager.

9.8 Leachate System Maintenance

Leachate flow rates from the pump stations are observed at least weekly. An extremely low (or high) flow rate, when compared to recent flow rates, may indicate a problem with the leachate-collection system. This problem could be a malfunction with pumps or its instrumentation controls. The problem could also be a blockage in or a collapse of the leachate-collection pipe. If a block is suspected, Pasco County will hire the services of a jet-cleaning and video-inspection company. An emergency purchase order for jet-cleaning and/or video-inspection of the leachate collection lines can be prepared as soon as an inspection is determined to be required. Pasco County expects that an emergency purchase order can be approved and a contractor can be hired within a maximum of 1 month. The jet-cleaning and video-inspection company will first jet-clean the pipes from the clean-outs and then video-inspect the pipes from the same clean outs. If major problems within the system are found, Pasco County will propose a remedial action plan and submit it to FDEP for approval before beginning the remedial work.

In accordance with 62-701.500(8)(h), FAC water pressure cleaning or video recording inspection is conducted at least once every five years.

9.9 Leachate Contingency Plan

9.9.1 Leachate Pumps, Hauling, and Flow Meters

If all pumps (onboard vehicle pump, storage tank transfer pumps, and leachate wet well pumps) fail, portable pumps are available from other County Departments. In addition, a rental transfer pump can be obtained from a local rental source.

If the sewer discharge pipeline is not functioning (for the Solid Waste cells) and the existing hauling contractor is unable to transport the leachate offsite (for the Ash Cells), an emergency can be declared to select another hauler so that unnecessary delays caused by bidding and selection can be prevented.

If the primary flow meter (the compliance meter) ceases to operate for either the Solid Waste cells or the Ash cells, contracted maintenance personnel will remove the instrument and insert a spare flow meter supplied by Pasco County Water & Wastewater. The faulty

instrument will be shipped to the service representative or manufacturer to repair or replace.

9.9.2 Electrical Power Failure

The leachate collection and leak detection metering manholes that receive leachate from all primary and secondary leachate collection systems are all gravity based and do not require electrical power. However, the pump stations that evacuate the manholes require electrical power. In the event of a prolonged electrical outage, back-up generators will be brought in to operate the leachate collection pump stations.

If electrical power is not available at the ash monofill tanker loadout station, leachate can be pumped from the Crom tank directly into tankers using the onboard vehicle pumps or a portable 3-inch pump or an electrical generator can be provided to supply backup power to the pump station. Backup generators are available from Pasco County Public Infrastructure.

9.9.3 Leachate Treatment Contingency

If the primary disposal facilities (City of Tampa for ash leachate and Shady Hills for solid waste leachate) are unable to accept the leachate, Pasco County will implement a contingency plan consisting of hauling the leachate to an industrial disposal facility or another treatment facility for disposal. Pasco County retains one or more vendors who can haul leachate off-site for disposal. If leachate is determined to be hazardous, it will be managed in accordance with the requirements of Rule 62-730, FAC, for hazardous waste generators and transporters.

10.0 SURFACE WATER MANAGEMENT

The Class I Landfill utilizes two surface water drainage areas as shown on **Figure 10-1**. The ponds have been sized to accommodate surface water runoff from the entire landfill at buildout and are at this time largely over-sized. Surface water flowing from the landfill drains to perimeter swales that discharge to the large retention ponds east of the landfill. Stormwater modeling indicates that any runoff that will overflow the ponds will travel essentially along the natural drainage paths that existed pre-development to undeveloped depressional areas west of the power line easement. However, because the retention pond capacity is largely underutilized at this time, overflow from the ponds is not expected to occur.

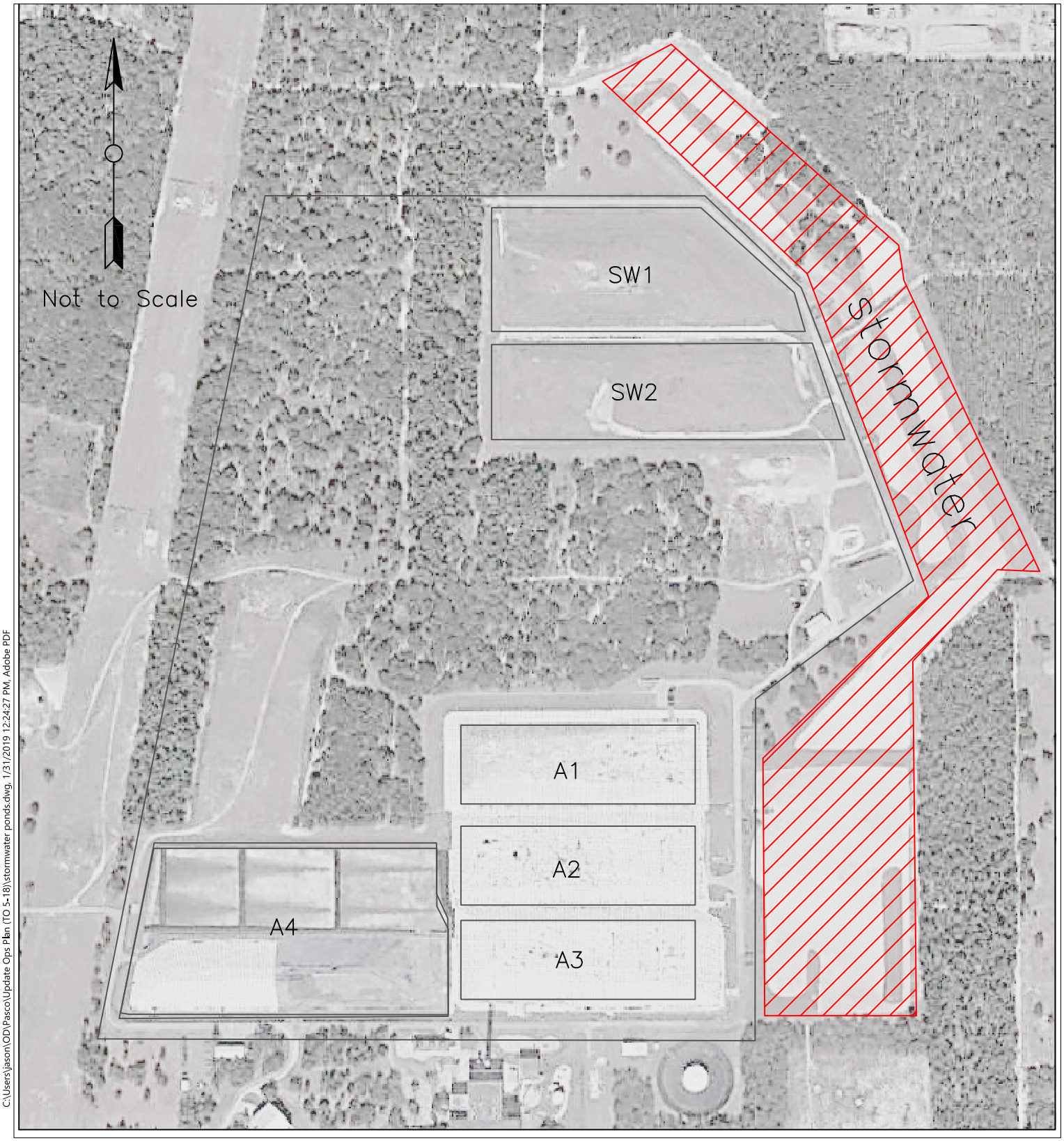
Drainage ditches have been constructed adjacent to the operating landfill cells and the closed landfill areas. These drainage ditches lead to the surface water ponds described above. These ditches are designed to receive the stormwater runoff from the landfill cells, paved areas and roads for collection and infiltration (and/or evapotranspiration) in the ponds.

10.1 STORMWATER SYSTEM MAINTENANCE

The following maintenance activities are performed as needed on an appropriate frequency to ensure the proper collection, conveyance, and disposition of surface waters associated with the Class I Landfill:

- All conveyance swales and ditches on the project site will be kept cleared of dense vegetation, debris, and trash that would impede the flow of stormwater runoff.
- All disturbed areas, swales, and basin side slopes that do not have living grass to prevent erosion will be sodded or seeded and mulched.
- All culverts that are damaged thereby inhibiting design flow rates will be cleaned, repaired, or replaced.
- The ditches, side slopes, and stormwater pond side slopes and berms will be mowed and kept clear of vegetation that would impede the discharge or receiving of stormwater.

In accordance with Rule 62-701.500(7)(k), F.A.C., the landfill will repair within 3 days erosion that causes waste to be exposed or the stormwater management system to malfunction. If the major erosion cannot be repaired within 7 days, the Pasco County will notify FDEP and propose an erosion correction schedule for the repairs.



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STORMWATER MANAGEMENT PONDS
WEST PASCO CLASS I LANDFILL

FIGURE
10-1

11.0 LANDFILL GAS MANAGEMENT

Because the solid waste disposal cells SW-1 and SW-2 are relatively small and are intended to be utilized infrequently, there is no active landfill gas management associated with them. In addition to a series of passive vents, Pasco County maintains a robust landfill gas monitoring program to ensure that landfill gas does not migrate laterally. The monitoring program is described below.

11.1 LANDFILL GAS MONITORING PROGRAM

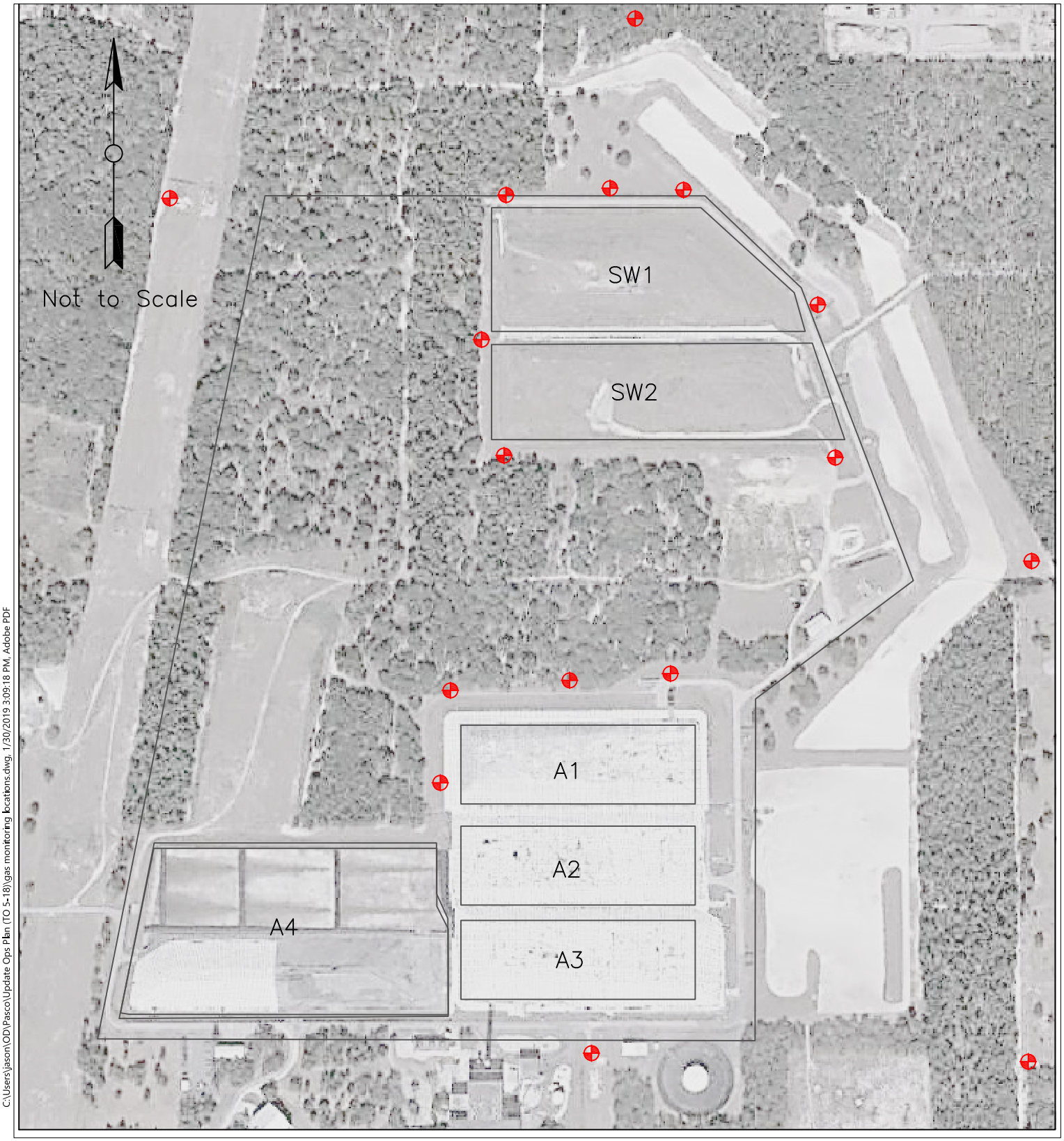
Pasco County Environmental Services is responsible for implementing the Gas Monitoring Program at the landfill. The LFG monitoring program consists of quarterly monitoring of six LFG monitoring wells, in addition to monitoring gas levels in nineteen groundwater monitoring wells. Gas monitoring is also conducted within the enclosed structures at the Class III scalehouse and the Class III Operations Building. **Figure 11-1** depicts the location of each of the gas monitoring locations. Gas monitoring is conducted in accordance with Rule 62-701.530(2)(c), F.A.C.

A portable landfill gas analyzer is used to measure the levels of combustible gases (primarily methane) at each of the monitoring locations. Part of the monitoring program is inspection of the monitoring wells for any symptoms of LFG leakage such as dead grass in the vicinity of the monitoring well or other locations. All field measurement data and observations are provided to FDEP no later than 15 days after the end of the quarter in which the monitoring occurred.

If results indicate that gas is present in excess of the 25% of the lower explosive limit (LEL) in a structure or 100% of the LEL at or beyond the landfill property boundary, a Gas Remediation Plan will be prepared in accordance with Rule 62-701.530(3)(a), F.A.C., and Pasco County Public Infrastructure will:

- Immediately take all necessary steps to ensure protection of human health and notify FDEP.

- Within 7 days of detection, submit to FDEP for approval a remediation plan for the methane gas releases. The plan shall describe the nature and extent of the problem and the proposed remedy.



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Note: THIS FIGURE IS NOT TO SCALE AND THE LOCATIONS OF SOME ITEMS HAVE BEEN EXAGGERATED FOR CLARITY

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GAS MONITORING LOCATIONS
WEST PASCO CLASS I LANDFILL

FIGURE
11-1

12.0 SITE CONTINGENCY PLAN

Emergency conditions that may constitute a special waste-handling event at the Solid Waste Complex may be created by a natural disaster (i.e., hurricane, tornado, and/or flooding), explosion, or fire. The Landfill Operations Manager is responsible for implementing the contingency operations.

12.1 LANDFILL FIRE

A fire extinguisher is maintained in all Pasco County landfill equipment. The quantity, type, and location of fire extinguishers located throughout the site is subject to change as conditions change on the site. Landfill fires can be very dangerous with the presence of LFG, which includes methane. Fire-fighting should only be attempted if the fire is relatively small and controllable. The area should be immediately evacuated of operating staff and outside personnel if a fire is ignited that cannot be easily and quickly controlled. The Solid Waste Manager should be contacted, and the fire should be reported to the Administration Office via field communications. The Administration office will immediately call the fire department. During a fire, incoming trucks will be prevented from using the landfill.

Once the fire is extinguished, appropriate cover will be applied to the waste and operations will continue at the original active face. If the fire is extensive and a temporary active face cannot be established, incoming trucks will be redirected to another landfill.

Landfill fires can be ignited from several sources or causes. These causes include the following:

- A gas pocket ignited from a spark generated from smoking.
- Sparks generated from pushing metal wastes.
- Chemical reactions.
- Sparks from operating equipment.
- Introduction of smoldering waste into the working face.

Onsite materials and equipment of fire protection consist of soil stockpiles near the working face, fire extinguishers carried on landfill equipment, and onsite water trucks.

The daily cover used in the landfill operation provides an effective firewall. Instructions in firefighting procedures will be routinely provided to site personnel through the prescribed TREEO training. If a fire occurs within the waste pile at the landfill, the fire department will be immediately notified, and additional soil cover will be applied to cut off the flow of oxygen to the burning area. If the fire cannot be extinguished or controlled within 1 hour, the landfill shall cease accepting waste for disposal in those areas of the facility impacted by the fire. The local fire department will again be notified via 911 of the conditions at the site and may be requested to assist site personnel and provide additional equipment, if necessary. Pasco County officials and FDEP will also be notified of conditions at the site and of the fire control plan belong implemented.

Collection vehicles entering the landfill with smoldering loads shall be directed to the Hot Load area at the Resource Recovery Facility. The truck should remain closed to minimize the amount of oxygen available to feed the fire. The local fire department shall be notified by dialing 911; Pasco County HHW staff shall be notified as well as landfill operations.

13.0 RECORDKEEPING

In addition to records and reporting required by other sections of this operation plan, Pasco County Solid Waste will maintain the following the following:

- Keep records of all information used to develop or support the permit applications and any supplemental information submitted to comply with FDEP requirements pertaining to the construction of the landfill throughout the design period. Records pertaining to the operation of the landfill, except for weigh tickets, shall be kept for the design period of the landfill. Weigh tickets shall be kept for a minimum of 5 years.

- Retain records of all monitoring information, including calibration and maintenance records and copies of all reports required by permit, for at least ten years. Background water quality records shall be kept for the design period of the landfill.

- Maintain an annual estimate of the remaining life and capacity in cubic yards of each active disposal facility for the Class I landfill. The annual estimate shall be based on a summary of the heights, lengths, and widths of the solid waste disposal units. The estimate shall be made and reported annually to FDEP.

All records 5 years or older may be archived offsite given that the records can be retrieved within 7 days for inspection.

14.0 OFF-SITE BENEFICIAL ASH REUSE

Section 403.7045(5) of the Florida Statutes authorizes the Department to allow beneficial reuse of ash residue when an applicant demonstrates that "no significant threat to public health will result and that applicable Department standards and criteria will not be violated." Beginning in 2012, Pasco County began working with the FDEP to investigate the recycling of bottom ash for beneficial reuse as a building construction material. A series of roadway test strips and associated groundwater monitoring wells were constructed under an FDEP research and development project (Permit No. 26254-004-SO-21). Through the research and development process, the County demonstrated that the requirements of Chapter 403.7045(5), F.S., could be achieved and the Department issued a Standing Authorization specifying the conditions under which bottom ash could be beneficially utilized within the County's geographic boundaries. In 2019, the Pasco County Resource Recovery Facility's Condition of Certification (PA 87-23D) was updated to include beneficial ash reuse. These are codified in Attachment E Part 6 of the Conditions of Certification – Solid Waste Construction and Operation Requirements. A copy of the relevant section of the conditions of certification is included as Attachment 1.

To render the bottom ash usable for targeted recycling projects, the County follows the procedures described in Section 6.6 of this Operation Plan. Following processing, the engineered material is beneficially reused consistent with the requirements in the attached conditions of certification.

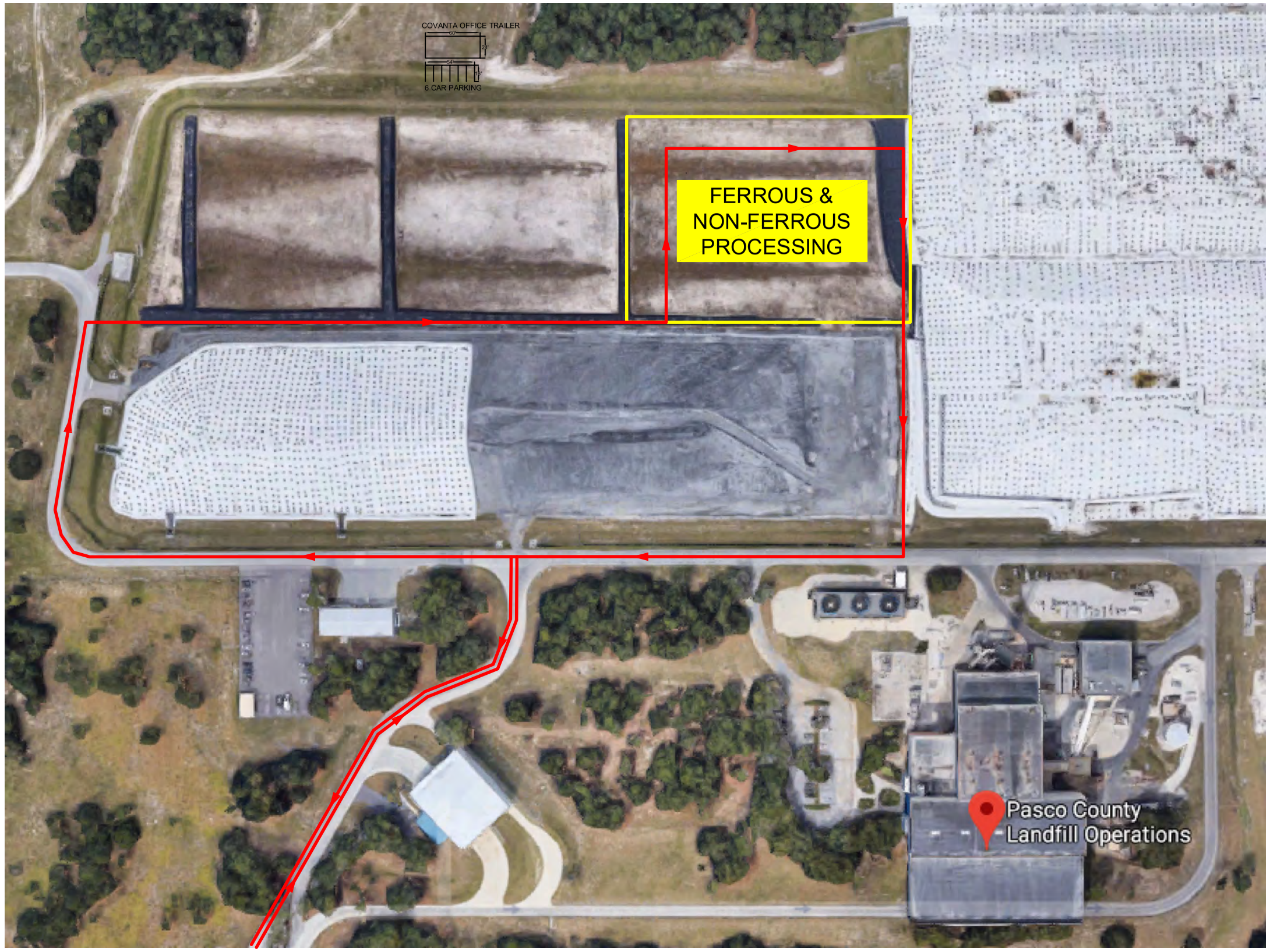
15.0 ASH MONOFILL FERROUS AND NON-FERROUS METALS PROCESSING

The 6th subcell of ash cell A-4 will be temporarily used to process ferrous and non-ferrous metal to remove residual ash prior to transport to a third-party recycling facility. This will include metal generated at the Pasco County Resource Recovery Facility as well as metal from other waste-to-energy (WTE) facilities within the state. Metal from all facilities will be brought over the County's scales and weighed. This metal will be stored in discrete storage bunkers and then processed through a series of screens and magnets to remove the residual ash. Following processing, the ash and metal will be loaded into separate bunkers. All ash not generated from Pasco County will be returned to the facility of origin. The mass of outgoing metal and ash for all facilities be weighed and recorded.

Figure 15-1 outlines the location of the processing area and a traffic pattern for the transfer truck operations. **Figure 15-2** provides an illustration of the equipment and storage bunkers which will be located within the ash cell. The processing area (Cell A-4 – subcell 6) will be floored with 4 feet of select waste (ash) before the processing equipment is brought on site. All equipment will be placed on top of the floored ash and will not be anchored into the ash or underlying leachate collection system in any way. The metals processing system is designed as a mobile operation.

To prevent the mixing of leachate and stormwater, the cell will be floored such that all rainwater that contacts the area will be introduced into the leachate collection and removal system. This will be accomplished by establishing a floored area no less than 10 feet inward from the extent of the liner. The aerial displayed in **Figure 15-1** is several years old; to reduce leachate generation the entire south 10 acres of the cell have been covered with a temporary rain cap and the metals processing area in operation in the north 10 acres of the cell will be open. Consistent with practices previously described in the Operation Plan, fugitive dust emissions within the processing area will be controlled by wetting on an as-needed basis.

Figure 15-1



NOTES:

1. ALL TRUCK MUST CHECK IN AND OUT AT THE SCALE HOUSE LOCATED AT THE ENTRANCE TO THE PASCO FACILITY.

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PROPOSED PASCO MOBILE NFE/FE PLANT

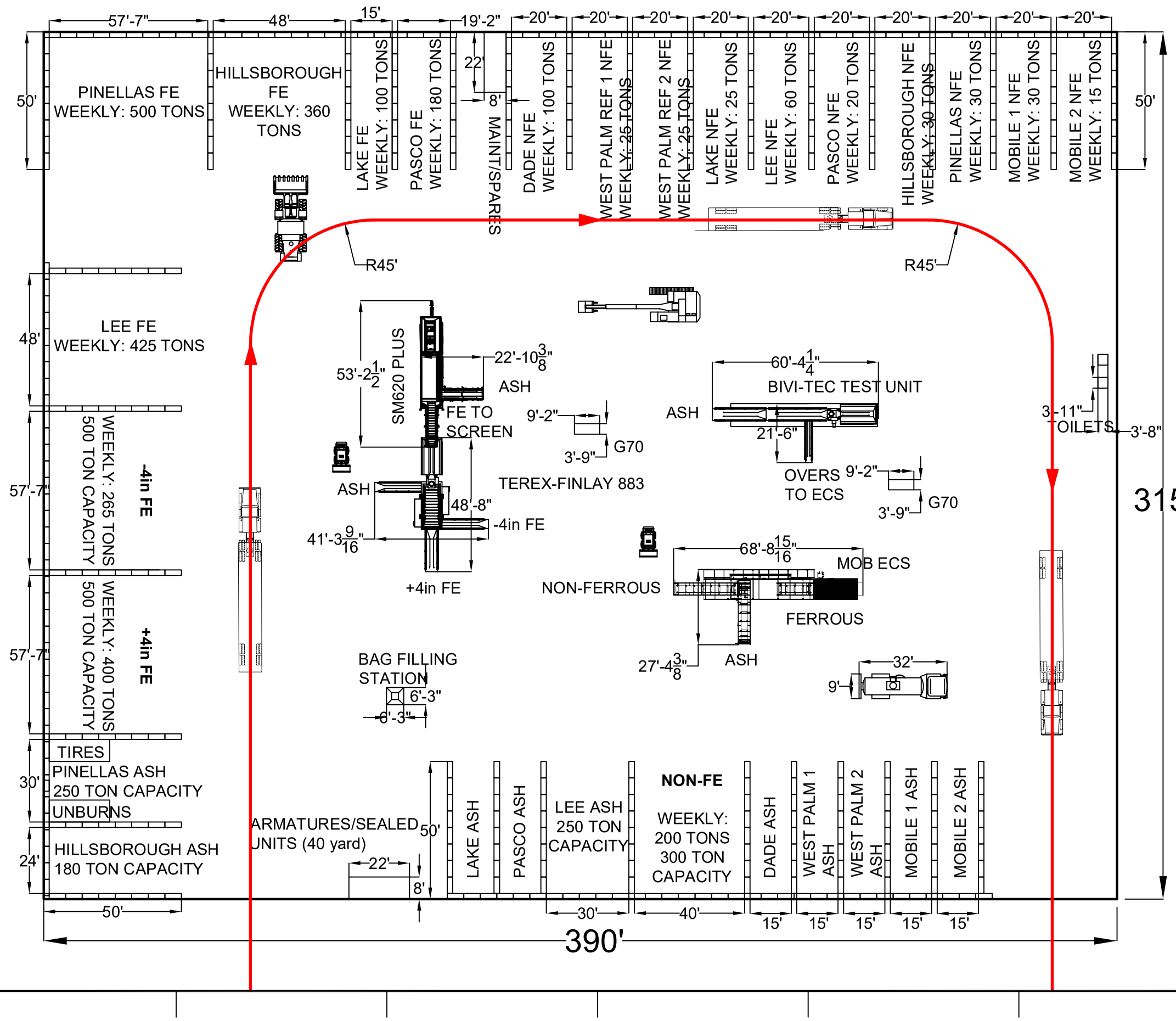
COVANTA

COVANTA ENGINEERING SERVICES, INC.
445 SOUTH STREET
MORRISTOWN, NEW JERSEY 07960

APPROVED	DATE	DRAWING NO.	SHT. NO.	REV.
	10/01/19	PASCO-1	1 OF 2	1

PASCO-1 10/01/19 PF: 1=1 "ANSI B"

Figure 15-2



NOTES:

1. ALL NFE BUNKERS HAVE 150 TON CAPACITY, CAN BE INCREASED IF NECESSARY.
2. ALL ASH BUNKERS HAVE 100 TON CAPACITY UNLESS OTHERWISE SPECIFIED.
3. ARMATURES/SEALED UNITS TO BE PICKED OUT AND STORED IN ROLL–OFF 40 YARD CONTAINER 8'X22'.
4. PINELLAS ASH: TIRES AND UNBURNS TO BE SEPARATED INTO TWO ROLL–OFF CONTAINERS 8'X22'.
5. TWO (2) G70 GENERATORS WILL BE USED TO POWER THE MOBILE PROCESSING EQUIPMENT. 9'–2" BY 3'–9".
6. SIX (6) PARKING SPOTS AVAILABLE FOR EMPLOYEES.
7. TWO (2) PORTABLE TOILETS AVAILABLE FOR EMPLOYEES IN ADDITION TO FACILITIES IN COVANTA OFFICE TRAILER.

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PROPOSED PASCO MOBILE NFE/FE PLANT

COVANTA

COVANTA ENGINEERING SERVICES, INC.
445 SOUTH STREET
MORRISTOWN, NEW JERSEY 07960

APPROVED	DATE	DRAWING NO.	SHT. NO.	REV.
	10/22/19	PASCO–2	2 OF 2	1

PASCO–2 10/22/19 PF: 1=1 "ANSI B"

ATTACHMENT 1

C. Closure Operation Plan. All closure shall be done in accordance with the approved closure operation plan.

D. Certification of Closure Construction Completion. After closure construction has been completed, the engineer of record shall certify to the Department on Form 62-701.900(2) that the closure is complete and that it was done in accordance with the plans submitted to the Department except where minor deviation was necessary. All deviations shall be described in detail and the reasons therefore enumerated.

5. Financial Assurance and Cost Estimates

A. Financial Assurance Mechanism

The licensee may not receive waste for disposal or storage in any disposal unit for which financial assurance has not been approved. Proof that the financial mechanisms are established and funded in accordance with Rule 62-701.630, F.A.C. shall be submitted to the Department at least sixty (60) days prior to the planned acceptance of solid waste in any disposal unit. When established, the licensee shall maintain, in good standing, the financial assurance mechanisms. Supporting documentation and evidence of increases associated with cost estimate increases shall be submitted within the time frames specified in Rule 62-701.630, F.A.C.

All submittals in response to this specific condition shall be sent to:
Florida Department of Environmental Protection
Financial Coordinator - Solid Waste Section
2600 Blair Stone Road, MS 4548
Tallahassee, Florida 32399-2400

B. Cost Estimates.

(1) The licensee shall submit closure cost estimates, including annual adjustments for inflation, in accordance with the requirements of Rule 62-701.630(3) and (4), F.A.C., and 40 CFR Part 264.142(a) using Form 62-701.900(28).

(2) An owner or operator using an escrow account shall submit the annual inflation adjusted estimate(s) between July 1 and September 1. An owner or operator using a letter of credit, guarantee bond, performance bond, financial test, corporate guarantee, trust fund or insurance shall submit the inflation adjusted cost estimate(s) between January 1 and March 1.

(3) A revised closure cost estimate, made by recalculating the total cost of closure or long- term care, in current dollars shall be submitted every 5 years in accordance with 62-701.630(3), F.A.C.

(4) All submittals in response to this specific condition shall be sent to the District Office and a copy to the address identified in this Attachment (Attachment E) 5. Financial Assurance and Cost Estimates, A., or to the following email address: Solid.Waste.Financial.Coordinator@dep.state.fl.us.

6. Beneficial Ash Use Authorization

A. Beneficial Use Authorization by the Department.

Section 403.7045(5) of the Florida Statutes authorizes the Department to approve the beneficial use of ash residue when an applicant demonstrates that "no significant threat to public health will result and that applicable Department standards and criteria will not be violated."

B. Documents Submitted in Support of Beneficial Use of Ash Residue.
The following documents have been submitted by the Licensee and reviewed by the Department regarding the beneficial use of bottom ash residue by the Licensee:

(1) "Request for Approval of Standing Beneficial Use Encapsulated Bottom Ash Generated by the Pasco County Resource Recovery Facility". dated August 14, 2014.

(2) "Request for Amendment to Approval of Standing Beneficial Use of Encapsulated Bottom Ash Generated by the Pasco County Resource Recovery Facility", dated October 20, 2016.

(3) "Request for Amendment to the Previous Approval of the Standing Beneficial Uses of Bottom Ash Generated by the Pasco County Resource Recovery Facility", which was included as part of the "Request for Modification to Conditions of Certification", dated February 6, 2019.

C. Approval of Beneficial Use of Bottom Ash Residue Generated by the Licensee.
Based on review of the documents listed in Paragraph 6.B. above, the Licensee is hereby authorized to beneficially use bottom ash residue in accordance with Section 403.7045(5), F.S., as follows:

(1) Specified Materials:

The following materials (Specified Materials) are allowed for recycling or reuse within Pasco County by, under the direction of the Licensee:

a. Bottom Ash Residue as defined in Rule 62-701.200(7)(a), F.A.C., from the Pasco County Resource Recovery Facility (Facility), a waste-to-energy facility as defined in Section 403.7061(4), F.S. (bottom ash), andb. Any product or construction material, including asphalt or concrete, which contains bottom ash residue as referenced herein.

(2) Required Treatment:

a. The Specified Materials for recycling and reuse shall be conditioned, by or under the direction of the Licensee, in accordance with the following procedure:

Prior to recycling and reuse, bottom ash residue will be aged for a minimum period of 3 months. Aging will be conducted on top of or in a lined Class I landfill cell. During the entire treatment period, ash piles shall be no greater than 20 feet in height.

(3) Specified Uses:

a. The recycling and reuse of the above specified materials are limited to, and only approved for, the following specified uses where the Specified Materials shall be as required herein: (i) Covered use as a road base course, where bottom ash residue from the Facility has been screened to remove large pieces of waste materials (>3/4 inch), and the road base course will be completely covered by a pavement layer; (ii) Encapsulated use as a replacement for up to 50%, by weight, of the customarily used aggregate in Portland cement concrete pavement, where bottom ash residue from the Facility has been screened to remove large pieces of waste materials (>3/4 inch) and the fine fraction of the ash (<3/8 inch);

(iii) Encapsulated as an aggregate in hot mix asphalt pavement, where bottom ash residue from the Facility has been screened to remove large pieces of waste materials (>3/4 inch) and the fine fraction of the ash (<3/8 inch);

(iv) As a partial coarse aggregate replacement in Portland cement concrete slabs beneath structures; and

(v) As a structural base course, underlying multi-use paths.

(4) Recycling and Reuse Conditions:

The Licensee shall implement the recycling and reuse controls specified

herein. The following controls and conditions are binding upon the Licensee and are enforceable under Chapter 403, F.S:

a. Bottom ash residue shall be treated by conditioning, prior to recycling and reuse, in accordance with this Authorization.

b. The Licensee shall not allow recycling, reuse, or disposal of the Specified Materials in any manner inconsistent with the requirements of Chapter 62-701, F.A.C., or this Authorization.

c. Bottom ash residue shall not be placed within 3 feet of groundwater, or 15 feet of wetlands or natural water bodies, or within 100 feet of a potable well that is being used or might be used for human or livestock water consumption.

d. When utilized as road base, the placement of the bottom ash residue shall not extend beyond the outside edge of the pavement*. Bottom ash residue remaining outside the edge of the pavement after construction shall be removed and recycled in a manner consistent with this Authorization or placed in a permitted Class I lined landfill.

e. When utilized as road base, placement of the pavement shall be completed as soon as practicable after placement of the bottom ash residue.

f. When bottom ash residue is utilized as a road base course, only dense-graded asphalt mixes shall be placed as the overlying pavement layer with a 4-inch or greater as-built thickness, except that for two lane roads designed for lower density traffic loads, an overlying pavement layer with a 2-inch as-built thickness may be used.

g. All roadways shall be placed and maintained following generally accepted road construction practices consistent with those utilized in Pasco County at the time of the issuance of the COC modification that incorporates this Authorization.

h. When used as structural base for multi-use paths, the placement of the bottom ash residue shall not extend more than six inches beyond the edge of the pavement and shall be completely covered by a compacted soil layer of a thickness of at least one inch, or equal to the thickness of the pavement, whichever is greater.

i. Bottom ash residue remaining outside this distance after construction shall be removed and recycled in a manner consistent with this Authorization or placed in a permitted Class I lined landfill.

j. When bottom ash residue is utilized as a structural base layer under a multi-use path, only asphalt or concrete mixes shall be placed as the overlying pavement layer with a one-inch or greater as-built thickness.

k. When removed from service, asphalt or concrete products containing bottom ash residue as aggregates shall be managed as construction and demolition debris, recycled or reused in a manner consistent with this Authorization, or placed in a permitted Class I lined landfill.

l. When removed from service, bottom ash residue that has been used as a base course shall be recycled or reused in a manner allowed under this Authorization or placed in a permitted Class I lined landfill.

m. Fugitive dust emissions from the storage, processing, transport or placement of bottom ash residue shall be controlled by wetting.

n. Bottom ash residue used as an aggregate in asphalt and concrete shall be stored on site at batch plants for a period of no more than 120 hours. Loads shall be covered with a tarp, or a similar protective cover, during transportation and ash will not be transported during periods of inclement weather. Following batching of the concrete or asphalt products, the unused

o. Bottom ash residue shall be recovered and either used in a subsequent allowed construction project or placed in a Class I lined landfill.

p. The Licensee shall comply with the following record keeping requirements:

(i) Whenever not the owner, the Licensee shall receive written notice, from the owner or duly authorized agent of the owner (Owner) of the property where the Specified Materials are to be placed, providing the Licensee express authorization for the placement of the Specified Materials, detailing the Owner's authorized placement dates, placement locations, and the maximum quantities of bottom ash residue that may be recycled or reused, or otherwise placed, on the Owner's property;

(ii) The Licensee may receive such notices from Owner's electronically;

(iii) The Licensee shall provide and maintain records detailing the actual placement dates, locations, quantities, the nature of recycling or reuse including the types of associated construction materials (i.e., road base course, concrete pavement, or asphalt pavement) where the Specified Materials were placed, and the dates and disposition whenever any Specified Materials are removed from service;

(iv) Notices and records shall be maintained in a centralized record storage system accessible from the Facility, and also in association with the particular construction project records for any related road construction project that utilizes the Specified Materials; and

(v) Such notices and records shall be maintained by the Licensee for a period of not less than 30 years after the removal of the Specified Materials from service.

q. Recycling and reuse of the Specified Materials, and associated construction activities, shall be conducted in a manner that does not cause or contribute to violations of state water quality standards. Performance-based erosion and sediment control best management practices shall be implemented and maintained immediately prior to, during, and after construction as needed to stabilize all disturbed areas, including material storage, staging, and processing areas, to prevent adverse impacts to the water resources and adjacent lands. Erosion and sediment control measures shall be installed and maintained in accordance with the State of Florida Erosion and Sediment Control Designer and Reviewer Manual (Florida Department of Environmental Protection and Florida Department of Transportation June 2007), available at www.dep.state.fl.us/water/wetlands/docs/erp/FLErosionSedimentManual_6_07.pdf, and the Florida Stormwater Erosion and Sedimentation Control Inspector's Manual (Florida Department of Environmental Protection, Nonpoint Source Management Section, Tallahassee, Florida, July 2008), available at www.dep.state.fl.us/water/nonpoint/docs/erosion/erosion-inspectors-manual.pdf.

r. The Licensee shall immediately notify the Department in writing upon, or at least within 5 business days of, becoming aware of any condition, test result, or other information indicating that recycling or reuse of the Specified Materials, in accordance with this Authorization, may cause or contribute to violations of state water quality standards.

s. Upon reasonable notice to the Licensee, Department staff or agents with proper identification shall have permission to enter, inspect, sample and test as the Department may claim needed to verify compliance with the requirements of Chapter 403, F.S., and this Authorization.

t. Uses other than as specified herein are not approved under this Authorization. Where bottom ash residue from the Facility, or related materials, may be used or placed in a manner that does not comply with this Authorization, such materials shall be managed as a solid waste subject to the requirements of Chapter 62-701, F.A.C.

*Pavement is defined as including the paved shoulders

Appendix C

Closure and Long-Term Care Plan

WEST PASCO COUNTY LANDFILL
CLOSURE AND LONG-TERM
CARE PLAN
FOR
CLASS I LANDFILL

CONDITIONS OF CERTIFICATION NO.: PA 87-23G

WACS: 45799

Prepared for:

Pasco County Public Infrastructure

14230 Hayes Road

Spring Hill, Florida 34610

Prepared by:

Jones Edmunds & Associates, Inc.

730 NE Waldo Road

Gainesville, Florida 32641

Certificate of Engineering Authorization #1844

July 2021

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1 INTRODUCTION

This Closure and Long-Term Care Plan provides guidelines and procedures for the closure requirements, closure construction, inspection, maintenance, repairs, monitoring, and record keeping of the West Pasco County Class I Landfill (Solid Waste Cells SW-1 and SW-2 and Ash Monofill Cells A-1, A-2, A-3, and A-4) including the stormwater system. This Plan combines recommendations and Florida Department of Environmental Protection (FDEP) requirements. Only those permit items pertinent to the closure and routine maintenance and/or operation of the closed landfills and the stormwater system have been identified and discussed. Figure 1 depicts the site plan of the West Pasco County Landfill.

This Plan contains a general discussion of the following: (1) Closure Requirements specified in Rule 62-701.600, Florida Administrative Code (FAC), and (2) Long-Term Care requirements specified in Rule 62-701.620(2), FAC, which includes an evaluation of the proposed landfill cover system and stormwater system and recommendations for continued closure performance.

1.1 CLOSURE PERMIT GENERAL REQUIREMENTS

A Closure Permit Application will be submitted to FDEP on Form 62-701.900(1) at least 90 days before final receipt of waste. The Plan submitted at that time will be in accordance with contemporary solid waste regulations and will address only the areas not closed under previous closure permits. The conceptual closure Engineering Plans submitted as part of the plan set prepared for this permit application will be updated to reflect actual site conditions at the time of full or partial closure. These plans will include sufficient detail to construct the closure.

The Closure Plan submitted as part of the Closure Permit Application will include the following items and are described in the indicated Sections:

1. Closure Design Plan (Section 2).
2. Closure Operation Plan (Section 3).
3. Plan for Long-Term Care (Section 4).
4. Demonstration of Proof of Financial Responsibility for Long-Term Care (Section 3).

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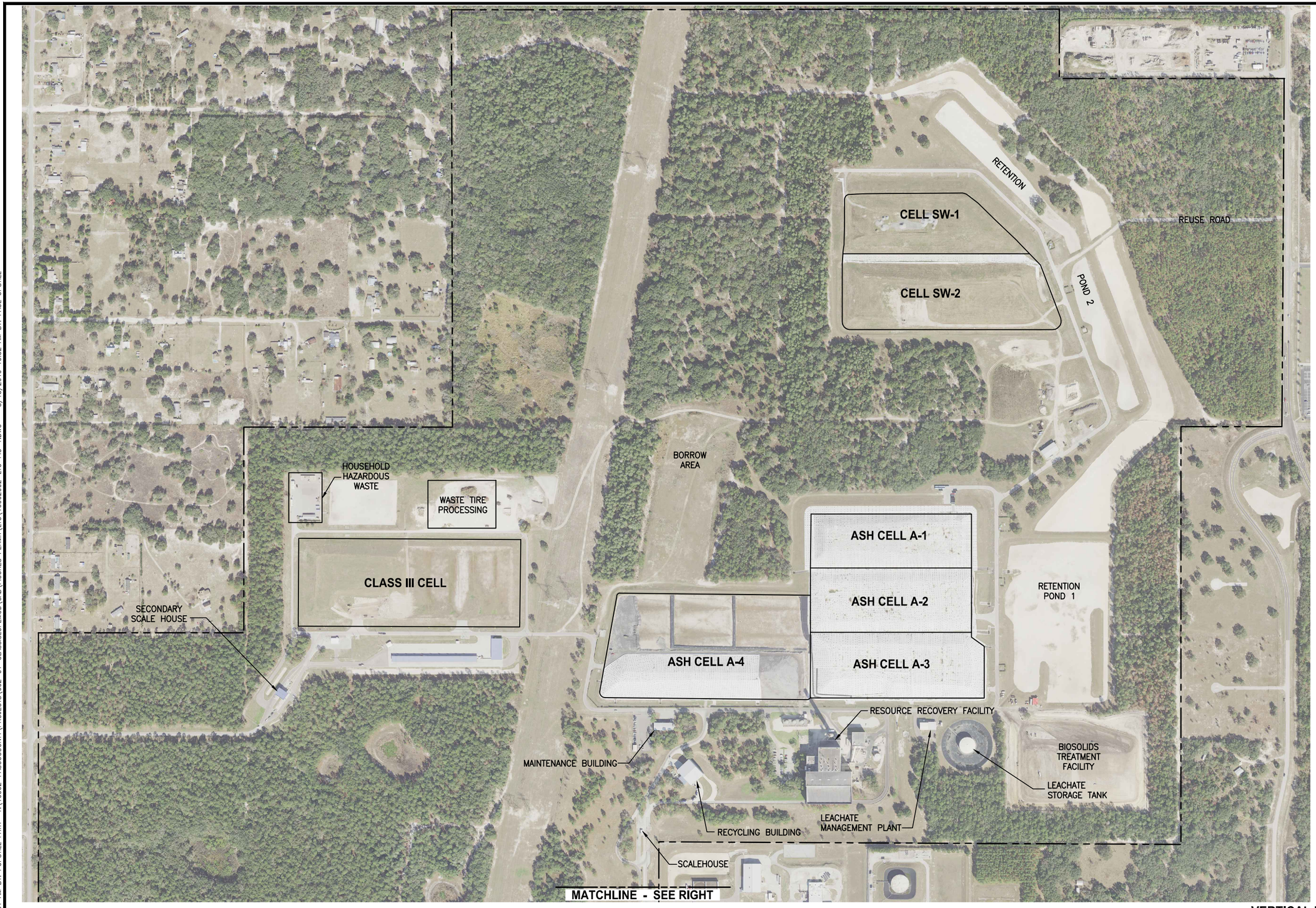

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GRAPHIC SCALE

0 150 300 600

SCALE IN FEET

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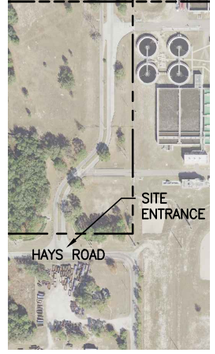

FIGURE 1
VERTICAL EXPANSION SITE PLAN
WEST PASCO COUNTY LANDFILL AND RESOURCE RECOVERY FACILITY
PASCO COUNTY, FLORIDA

JonesEdmunds

8/15/2019 10:51 AM BY: PJPSTILL PATH= Y:\16002-PASCOCOUNTY\PROJECTS\002-01-CLASSIILOPEMOD\CAD\FIGURES PERMIT\LTC\16002002-LTC-FIG-1.DWG 8/15/2019 10:52 AM BY: PAUL UPSTILL

2 CLOSURE DESIGN PLAN

The Closure Design Plan will consist of engineering plans and a report on closing procedures that apply to the final closing of solid waste disposal units during the operation of the landfill, the final closing of the landfill, and the monitoring and maintenance during the long-term-care period. The Closure Design Plan shall include the following information.

2.1 CLOSURE PHASING

A plan sheet illustrating the detailed closure phasing of the site will be submitted as part of the Closure Design Plan.

2.2 TOPOGRAPHIC MAPS

Updated Engineering Plans showing existing topography and proposed final grades will be submitted as part of the Closure Design Plan with the Closure Permit Application. The conceptual final cover design for Cells SW-1 and SW-2 is presented in the permit application Engineering Plans submitted with the "*West Pasco County Landfill Cells SW-1 and SW-2 Slope Modification and Vertical Expansion Substantial Modification Permit Application*" as part of the modification to the Conditions of Certification for the West Pasco County Landfill (August 2019). The conceptual final cover design for Cells A-1, A-2, A-3, and A-4 are presented in the permit application Engineering Plans submitted with the "*West Pasco County Ash Monofill Slope Modification and Vertical Expansion Substantial Modification Permit Application*" as part of the modification to the Conditions of Certification for the West Pasco County Landfill (June 2021).

2.3 APPROVED DESIGN DIMENSIONS

Solid waste disposal units will be finally closed when reaching approved design dimensions and elevations in accordance with the requirements of Rule 62-701.600(3), FAC. Temporary closure may be implemented and is explained in Section 3.

2.4 FINAL ELEVATIONS

Final elevations of the final cover are shown before settlement.

2.5 SIDE SLOPE DESIGN

The side slopes of the final cover design are shown to include benches, downslope drainage ways, and energy dissipaters. Side slopes shall not be steeper than 3 feet horizontal to 1 foot vertical rise to control erosion of the final cover material. The side slopes will be designed to control stormwater flow using techniques such as building benches and/or tack-on berms on the side slopes of the landfill. The side slopes will contain downslope drainage ways with water-flow energy dissipaters. Access for maintenance equipment will be provided. The design will address the susceptibility for erosion of the earthen material proposed for final cover relative to historical rainfall patterns for the area, the period between the patterns for the area, the period between the application of the final cover and establishment of vegetation, and maintenance procedures.

The Closure Design Plan will include an evaluation of the stability of the cover system and the disposed waste. The closure shall be designed to meet the minimum 1.5 factor of safety criteria in Subsection 62-701.400(2), FAC. This evaluation will include an analysis of the potential for slides along the weakest interface of the final cover system and for deep-seated rotational or translational failures through the waste and the final cover. The analysis will be based on the laboratory measurements of the interface friction angles of the cover system components.

2.6 FINAL COVER INSTALLATION

Final cover installation plans will be submitted at the same time as the Closure Permit Application, including a Construction Quality Assurance Plan that meets the requirements of Subsections 62-701.400(7) and (8), FAC. A schedule for installing closure components will be provided. The top-grade vent will be designed to minimize erosion, maximize runoff, and to prevent ponding or low spots while considering total fill height and expected subsidence caused by decomposing waste.

Final cover will be placed over the entire surface of each completed solid waste disposal unit or units within 180 days after the final waste deposit or within the time frame set forth in the approved Closure Plan. The final cover will be vegetated to control erosion and provide a moisture infiltration seal with species that are drought-resistant and have roots that will not penetrate the final cover. Provisions for cover material and maintenance of the cover during long-term care will be provided.

2.7 FINAL COVER DESIGN

The final cover system will be designed to minimize infiltration and erosion and will include a barrier layer meeting Chapter 62-701.600, FAC, requirements. The barrier layer will have chemical and physical resistance to materials that it may come in contact with and be capable of withstanding exposure to the natural environmental stresses and forces throughout the installation/seaming process, as well as settlement of the waste during the closure and long-term care period. All geosynthetic components used in the final cover will meet the standards and specifications contained in Subparagraphs 62-701.400(3)(d)1., 2., and (3)(d)5.-11. and Subparagraphs 62-701.400(3)(e) and (f), FAC. The barrier layer will have a permeability that is substantially equivalent to or less than the permeability of the bottom liner system.

A protective soil layer at least 24 inches thick will be placed on top of the geomembrane. Material specifications, installation methods, and compaction specifications will be designed to protect the barrier layer from root penetration, resist erosion, and remain stable on the final design slopes of the landfill. A stability analysis will be submitted at the time of permitting. This layer will include topsoil or soils that will sustain vegetative growth. The design may include a drainage layer between the geomembrane and the protective soil layer if determined necessary during the design and permitting process.

A conceptual final cover design for Cells SW-1 and SW-2 is presented in the permit application Engineering Plans submitted with the "*West Pasco County Landfill Cells SW-1 and SW-2 Slope Modification and Vertical Expansion Substantial Modification Permit Application*" as part of the modification to the Conditions of Certification for the West Pasco

County Landfill (August 2019). The conceptual final cover designs for Cells A-1, A-2, A-3, and A-4 are presented in the permit application Engineering Plans submitted with the "*West Pasco County Ash Monofill Slope Modification and Vertical Expansion Substantial Modification Permit Application*" as part of the modification to the Conditions of Certification for the West Pasco County Landfill (June 2021). These conceptual final cover designs include, from bottom to top, a soil cover, a geomembrane cap system, and a final soil cover able to support vegetation.

2.8 STORMWATER CONTROL

The stormwater management and control system for Cells SW-1 and SW-2 is presented in the permit application Engineering Plans submitted with the "*West Pasco County Landfill Cells SW-1 and SW-2 Slope Modification and Vertical Expansion Substantial Modification Permit Application*" as part of the modification to the Conditions of Certification for the West Pasco County Landfill (August 2019). The stormwater management and control systems for Cells A-1, A-2, A-3, and A-4 are presented in the permit application Engineering Plans submitted with the "*West Pasco County Ash Monofill Slope Modification and Vertical Expansion Substantial Modification Permit Application*" as part of the modification to the Conditions of Certification for the West Pasco County Landfill (June 2021).

The design provides perimeter drainage ditches before construction of the closure as part of the overall site construction. This stormwater system will be maintained throughout active operations at the landfill and will serve as the stormwater management system after closure. In general, benches and tack-on berms on the landfill sides control runoff. The side slope benches and tack-on berms will be graded to drain into multiple ditch bottom inlets. Pipes from the inlets channel stormwater down the slopes into the perimeter drainage ditches. The stormwater system shown on these plans is conceptual, and a detailed design will be provided as part of the permit application for construction of the closure. The Closure Design Plan will demonstrate how the stormwater management systems are operated and maintained as necessary to meet the requirements of Subsection 62-701.400(9), FAC.

2.9 ACCESS CONTROL

Access to the West Pasco Solid Waste Disposal Complex is controlled at a site entrance off Hays Road using gates. The entire facility is fenced, and control will be maintained after closure. The Closure Plan submitted as part of the Closure Permit Application will provide a detailed description of the access control to prevent future waste dumping or use of the facility by unauthorized persons will be provided in.

2.10 GAS MANAGEMENT SYSTEM

The proposed gas management system will include a passive gas vent system. The Closure Design Plan submitted with the Closure Permit Application will provide for the development and implementation of the gas management system required in Rule 62-701.530, FAC.

3 CLOSURE OPERATION PLAN

A Closure Operation Plan will be provided as part of the Closure Permit Application.

3.1 ACTIONS TO CLOSE

The Closure Operation Plan will include a detailed description of the actions that will be taken to close the landfill. Typical steps include placing cover; grading; constructing berms, ditches, roads, and retention-detention ponds; installing or closing wells and boreholes; installing fencing; seeding or installing sod vegetation; and protecting on-site utilities and easements as appropriate for the site.

3.2 TIME SCHEDULE

The Closure Operation Plan will also include a time schedule to complete the closure construction and long-term care.

3.3 FINANCIAL ASSURANCE AND LONG-TERM CARE MECHANISMS

The Closure Operation Plan will describe the mechanisms for financial assurance and long-term care.

Pasco County Public Infrastructure will provide adequate financial assurance for this facility in accordance with Rule 62-701.630, FAC. The County will review the long-term care costs annually and submit updates to the costs between July 1 and September 1 using the FDEP form 62-701.900 (28), FAC. The costs of compliance with Rule 62-701.620, FAC, are included.

Annual adjustments to long-term care will be made, in accordance with Rule 62-701.630(4) and (8), FAC, either by (1) recalculating the total cost in current dollars, or (2) using an inflation factor provided by FDEP.

The County funds an escrow account that finances the long-term-care expenses, in accordance with Rule 62-701.630(5), FAC. The County Auditor confirms annually with FDEP that the escrow account is appropriately funded and managed.

3.4 WATER QUALITY MONITORING PLAN

The Closure Operation Plan will describe the Water Quality Monitoring Plan required in Rule 62-701.510, FAC.

3.5 GAS MANAGEMENT SYSTEM

The Closure Operation Plan will provide for development and implementation of a gas management system as required in Rule 62-701.530, FAC.

4 CLOSURE COMPLETION

4.1 FINAL SURVEY

After closure is complete, a Florida-licensed Professional Surveyor and Mapper will conduct a final survey of the constructed closure in compliance with Section 62-701.600(6)(b), FAC, to verify that the final contours and elevations are in accordance with the plans as approved in the Closure Permit. The contours in the final survey will be shown at no greater than 5-foot intervals. A copy of the survey will be included with the Certification of Closure Construction Completion. In accordance with Section 62-701.600(6)(b), FAC, survey monuments are not required for landfills with a final elevation of 20 feet or higher above the natural land surface.

4.2 CLOSURE CONSTRUCTION CERTIFICATION

In accordance with Chapter 62-701.600(6), FAC, a signed, dated, and sealed Certification of Closure Construction Completion by the Engineer of Record will be submitted to FDEP on Form 62-701.900(2) when closure construction is complete to certify that the closure is complete and was performed in accordance with the plans submitted to FDEP except where minor deviation was necessary. This Certification will describe in detail any deviations from the permitted Closure Plan, explain the reasons for the deviations, and verify that the final survey requirements have been met.

5 DECLARATION TO THE PUBLIC

After closure operations are inspected and approved by FDEP, Pasco County Public Infrastructure will file a declaration to the public in the deed records of Pasco County. The declaration will include a legal description of the property on which the landfill is located and a site plan specifying the area actually filled with solid waste. The declaration will also include a notice that any future owner or user of the site should consult with FDEP before planning or initiating any activity that disturbs the landfill cover, monitoring system, or other control structures. A certified copy of the declaration will be filed with FDEP.

6 OFFICIAL DATE OF CLOSING

After FDEP determines that such documents comply with the Closure Plan and the requirements of Sections 3.1 through 3.3, FDEP will notify Pasco County Public Infrastructure that closing of the facility has been completed.

7 TEMPORARY CLOSURE

In accordance with Chapter 62-701.600(9), FAC, placement of final cover may be delayed if the landfill is temporarily closed in accordance with an approved Closure Plan. Pasco County Public Infrastructure's experience indicates advantages to delaying final cover and these meet the Rule reasons in Subsection 62-701.600(9)(a), FAC.

1. Biological degradation of waste is promoted.

2. Additional waste will be deposited on the Cells as a lateral expansion and/or vertical expansion.

In accordance with Subsection 62-701.600(9)(b) and (c), FAC, the conditions of temporary closure will be met.

1. The Cells will be constructed in compliance with their permit conditions and will have a liner and leachate control system.

2. A schedule for closure will be shown in the Closure Plan Application.

3. Final cover is installed on side slopes of each completed disposal unit that will not receive additional waste or that will not be excavated except if Pasco County Public Infrastructure can demonstrate that a different process will be adequate to control erosion and odors and will be expected to result in compliance with applicable stormwater and leachate management requirements.

4. Odors and vectors are controlled.

5. An intermediate cover is installed on the Cells.

6. The financial assurance requirements of Rule 62-701.630, FAC, are met, and the closure cost estimate accounts for the costs of temporary closure as well as the costs of the final closure.

7. Pasco County Public Infrastructure demonstrates that delaying placement of final cover will not cause or contribute to any significant increase in leachate escaping from the Cells into the environment.

8. If the Cells are excavated in the future, they will have a temporary final cover installed.

8 OTHER CLOSURE PROCEDURES

8.1 CLOSED LANDFILL USE

No use has been designated for the closed landfill area. In accordance with Chapter 62-701.610(1), FAC, Pasco County Public Infrastructure will consult with FDEP before conducting activities at the closed landfill. Pasco County Public Infrastructure acknowledges that FDEP retains regulatory control over any activities that may affect the integrity of the environmental protection measures of the landfill.

8.2 RELOCATION OF WASTES

If at any time after closure Pasco County Public Infrastructure intends to relocate waste within the footprint of the landfill, a permit modification application will be submitted to FDEP for approval.

9 LONG-TERM-CARE REQUIREMENTS

If disturbed, closed landfill areas are a potential hazard to public health, groundwater, and the environment. Therefore, FDEP retains regulatory control over any activities that may affect the integrity of the environmental protection measures, such as the landfill cover, drainage, monitoring system, or stormwater controls. This Section 9 is intended to comply with Rule 62-701.620, FAC, *Long Term Care*.

9.1 LONG-TERM-CARE PERIOD

Pasco County Public Infrastructure owns the West Pasco County Landfill and is responsible for monitoring and maintaining the facility in accordance with the FDEP-approved Closure Plan. The Long-Term Care Plan will remain in effect for a minimum of 30 years from the date of closing but may be extended if FDEP determines the plan to be ineffective as defined by Subsections 62-701.620(1)(a) through (d), FAC.

Pasco County Public Infrastructure will monitor and maintain the integrity and effectiveness of the final cover as well as other appurtenances of the facility, control erosion, and fill subsidence; comply with the groundwater monitoring plan; and maintain the stormwater system in accordance with the approved Closure Plan for 30 years from the date of closing. Before the long-term care monitoring and maintenance period expires, FDEP may extend the period if the Closure Design Plan or Closure Operation Plan is found to be ineffective or if Pasco County Public Infrastructure has not performed all required monitoring and maintenance. In accordance with Chapter 62-701.620(1), FAC, FDEP may determine that the Closure Design Plan or Closure Operation Plan is ineffective if:

a. *The groundwater monitoring system indicates that the landfill continues to impact groundwater at concentrations that may be expected to result in violations of FDEP water quality standards or criteria.*

b. *The gas monitoring system indicates that the landfill continues to produce gas in amounts that may be expected to exceed the concentrations of combustible gases allowed in Paragraph 62-701.530(1)(a), FAC.*

c. *Significant subsidence of waste has not ceased.*

d. *The final cover does not have well-established vegetation or is showing signs of* continuing significant erosion problems.

Pasco County Public Infrastructure will retain the right of entry and provisions for access as further explained in Section 9.7. Consultation with FDEP is required before conducting activities that may affect environmental protection measures at the closed landfill. Supervision of the closed landfill is the responsibility of a person experienced in the closure requirements of a solid waste management facility.

9.2 "LONG-TERM CARE ONLY" CLOSURE PERMITS

Closure permits involving only long-term care will be renewed every 10 years. After Sections 4 through 6 are complied with, FDEP allows long-term care to be incorporated into

the Operations Permit of an unclosed landfill cell. The closed area and operating area must be within the survey boundary of the same solid waste management facility. The consolidation of the long-term care into an Operations Permit eliminates the Long-Term Care Only Closure Permit, but the Operations Permit must be renewed every 5 years.

9.3 Reduced Long-Term Care Period

In accordance with Rule 62-701.620(3), FAC, Pasco County Public Infrastructure may apply to FDEP for a permit modification to reduce the long-term-care period or eliminate some aspects of long-term care. FDEP will grant such modification if reasonable assurance is provided to FDEP that there is no threat to human health or the environment and if the landfill:

a. *Has been constructed and operated in accordance with approved standards.*

b. *Was closed with appropriate final cover, vegetative cover has been established, and a monitoring system has been installed.*

c. *Has a 10-year history after closure of no violations of water quality standards or criteria detected in the monitoring system and no increases over background water for any monitoring parameters that may be expected to result in violations of water quality standards or criteria.*

d. *Has had no detrimental erosion of cover, and subsidence of waste has ceased.*

9.4 Modify Groundwater Monitoring Plan

In accordance with Subsection 62-701.620(4), FAC, Pasco County Public Infrastructure may apply to FDEP for a modification to the groundwater monitoring plan to remove a parameter from the list specified in Subsection 62-701.510(7), FAC. FDEP will grant such a modification if Pasco County Public Infrastructure demonstrates that leachate and groundwater have consistently been sampled and analyzed for the parameter and the parameter has never been detected in the leachate or in any groundwater well or surface water point during the active life of the landfill.

9.5 Modify Gas Monitoring

Pasco County Public Infrastructure, in accordance with Section 62-701.620(5), FAC, may apply to FDEP for a permit modification to reduce, or eliminate the frequency of, the long-term-care period for gas monitoring. FDEP will grant such a modification if Pasco County Public Infrastructure demonstrates that the landfill has stabilized to the point where no significant combustible gases or objectionable odors are produced.

9.6 Stabilization Report

Every 5 years after a permit for long-term care is issued, Pasco County Public Infrastructure will submit a report to FDEP that addresses landfill stabilization. The submittal shall include the technical report required in Paragraph 62-701.510(8)(b), FAC, and shall also address subsidence, barrier layer effectiveness, stormwater management, and gas production and

management. For lined landfills, the submittal will also address leachate collection and removal system effectiveness, leachate quality, and leachate quantity.

9.7 Landfill Access and Landfill Signs

Pasco County Public Infrastructure will retain the right of entry to the landfill property for the long-term-care period, after termination of solid waste operations, for inspection, monitoring, and maintenance of the site.

The boundary of the landfill property is fenced. Access to the site is to be restricted to prevent unauthorized entry and dumping. As part of the routine checklist procedure, all fencing and signage will be inspected for damage and repaired, repainted, and replaced if necessary to maintain the integrity of these items. If any landfill monuments are severely damaged or destroyed, corrective actions will be taken.

The site roadways will also be inspected as part of the routine checklist procedure. If any access roads are severely damaged, corrective actions will be taken to maintain passable and safe roads on the site.

9.8 Inspection, Maintenance, and Repair

Pasco County Public Infrastructure will inspect and maintain the landfill to minimize impacts to the function and/or integrity of the final cover systems. Pasco County Public Infrastructure currently provides, and during closure site will continue to provide, access control, erosion control, grass cover maintenance, and prevention of ponding. Primary focus during the inspection will be on the condition of the surface vegetation, landfill cap, gas collection and monitoring system, stormwater system, and monitor devices.

Table 1 is an inspection checklist that Pasco County Public Infrastructure proposes to use monthly during post-closure care to ensure compliance with the long-term-care requirements and provide a log of landfill inspection activities. The checklist form will be completed and signed by the individual conducting the monthly inspection. Items requiring attention are noted on the form and brought to the attention of the Pasco County Public Infrastructure Landfill Operations Manager.

Table 1 West Pasco County Landfill Inspection Log

Pasco County Public Infrastructure
Post-Closure Inspection Checklist
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Landfill Inspected: _____ Date of Inspection: _____

Field Personnel: _____

Conditions: _____

Section A: Fencing and Security	Yes	No	N/A
1. Damage to fences, gates, or locks			
2. Gates unlocked/locks missing			
3. Signs of forced entry detected			

Section B: Access Roads	Yes	No	N/A
1. Access and site roads in poor condition			
2. Signs need repair			

Section C: Final Cover System	Yes	No	N/A
1. Settlement of cover			
2. Evidence of erosion, cracks, gullies			
3. Inadequate growth of grass cover			
4. Excessive grass height (greater than 18 inches)			
5. Holes or damage to cover			
6. Growth of damaging weeds or saplings			
7. Evidence of leachate seeps			
8. Landfill marker damage			
9. Impacts due to settlement			
10. Ponding of water			

Section D: Gas Collection/Control System	Yes	No	N/A
1. Visible damage to system components			
2. Blockage in pipes			
3. Excessive release of odors			

Section E: Monitoring Devices	Yes	No	N/A
1. Damage to groundwater monitoring wells			
2. Damage to gas monitoring wells			
3. Locks missing			
4. Damage to gas monitor probe			

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Post-Closure Inspection Checklist
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Section F: Stormwater Management System	Yes	No	N/A
1. Ponding of water			
2. Areas of silting			
3. Insufficient slope to promote positive drainage			
4. Areas of erosion in ditches or areas leading to ditches			
5. Inlets repair required			
6. Piping repair required			
7. Retention pond damage			
8. Berm repair required			
9. Letdown pipe repair required			
10. Grout filled fabric repair required			
11. Litter or garbage problem			
12. Pollutants in drainage areas			
13. Water other than stormwater entering system			
14. Ditches/Culverts obstructed by vegetation or other			
15. Debris or weeds in perimeter ditch			

Section G: Site and surrounding area	Yes	No	N/A
1. Surface depressions			

Signature of Field Personnel:_____ Date: _____

Explanation of items marked Yes above:

_____

_____

_____

_____

_____

_____

_____

_____

Table 2 is a schedule for notification if corrective actions are required. Records of discovery will also be kept on the Inspection Checklist (Table 1).

Table 2 Schedule for Notification and Corrective Actions

Activity	Initial Notification	Written Notification/Corrective Action Plan	Corrective Action
Sinkhole within 500 feet	Within 24 hours of discovery	Within 7 days of discovery, including description, location, size shown on plan sheet, corrective action plan	Based on proposed schedule
Fire/Explosion	Within 24 hours of discovery	Within 7 days of discovery, including remedial measures and schedule of activities	Based on proposed schedule
Damage to Facilities/Failure of Systems	Within 24 hours of discovery with explanation	Within 7 days of discovery, including details of damage/failure, remedial measures, schedule of repairs	Based on proposed schedule
Damage to Groundwater Monitoring System	Within 24 hours of discovery with explanation	Within 7 days of discovery, including details of damage/failure, remedial measures, schedule of repairs	Based on proposed schedule
Damage to Stormwater System	Within 24 hours of discovery with explanation	Within 7 days of discovery, including details of damage/failure, remedial measures, schedule of repairs	Within 30 days of written notification
Erosion of Final Cover System >6-inches in depth	N/A	Description on Inspection Log	Within 72 hours of discovery
Leachate not accepted by Disposal Facility	Same as Written Notification	Within 3 days of cessation of leachate acceptance, including explanation of contingency measures and schedule of disposal	Within 7 days of cessation of acceptance
Landfill Cap Repairs and Replacement	Same as Written Notification	Within 3 days of discovery: including details of damage/failure, remedial measures, schedule of repairs	Based on proposed schedule

Note: Notification shall be submitted to FDEP Southwest District.

9.9 GRASS

Grass cover maintenance will include mowing, fertilizing, seeding, mulching, and filling areas of subsidence. Mowing, fertilizing, seeding, mulching, and filling will continue to be performed as needed. The following is a general schedule and description of grass maintenance activities:

- *Mowing* – The height of the grass will be observed during monthly inspections. If the grass is found to be approximately 18 inches high, mowing will be scheduled before the next inspection. Caution will be exercised while mowing to keep heavy equipment away from the gas wells, gas vents, and monitoring devices.

- *Fertilizing* – The general recommendations for commercial fertilizer are 12-8-2 formulation (nitrogen-phosphorus-potassium), of which 60% of the nitrogen is to be in the urea-formaldehyde form and to conform to state laws. Fertilizer should be applied in the early spring (March) and mid-summer (July) as needed. The spread rate should be 8 to 10 pounds per 1,000 square feet or as instructed on the package. The local US Department of Agriculture extension office should be contacted to verify these recommendations.

- *Seed and Sod* – Damaged areas or other areas where grass cover is sparse must be reseeded or sodded. Sod is generally recommended to use in all areas such as on steep slopes and in highly eroded or bare spots. Sod should be staked in place with sod pegs where necessary.

  Seeding, if done on relatively flat areas, should be performed in the early spring and late fall as needed in the following manner:

  - Early spring:
    - Scarified bahia with 20-percent bermuda seed.
    - Minimum percent pure seed—95.
    - Minimum percent germination and hard seed —80.
    - Bahia seed will not germinate until overnight temperatures stay above 70° Fahrenheit.
  - Late fall
    - Italian rye.
    - Minimum percent pure—95.
    - Minimum percent germination and hard seed—90.
    - Seed will not germinate until overnight temperatures stay below 70° Fahrenheit and above 40° Fahrenheit.
    - Bahia Sod—16-inch by 24-inch slabs with 1-1/2-inch root bed.

- *Seed Rates* – The following are general recommendations for maintenance and replacement growth. High erosion areas and bare patches should be seeded more heavily. Spread rates may vary for different grass seeds from different suppliers. The instructions on the seed bags should be followed. Maintenance seeding should be used where the grass is healthy and full. Replacement seed rates should be used on bare or thin grass-growth areas.

  - Maintenance Seed Rates
    - Spring 1/2 pound per 1,000 square feet.
    - Fall 1/2 pound per 1,000 square feet.
  - Replacement Seed Rates
    - Spring 3 to 5 pounds per 1,000 square feet.
    - Fall 3 to 5 pounds per 1,000 square feet.

- *Watering* – The Pasco County Public Infrastructure should water as required to maintain the health of the grass; daily watering should never be necessary. If the blades of the

grass begin to wilt and loose resiliency when walked on, water needs to be applied. The water that is applied should be clean and potable or from on-site stormwater ponds and ditches.

9.10 Erosion Control

Avoiding erosion is probably the most cost-effective means of protecting the closure cap. A relatively minor eroded area combined with a severe storm event can cause degradation of the final cover. The best way to avoid erosion is to maintain a healthy stand of grass and keep drainage swales free of silt and sediment. Cleaning the drainage swales will prevent overflow and backflow and reduce the risk of erosion from these causes. Large amounts of silt or sediment removed from the drainage swales may indicate damage to the closure cap.

9.11 Stormwater Structures

All stormwater structures should be clean of all silt or soil deposits. All soil settlement surrounding these items should be brought to the attention of the Solid Waste Facility Operator and then repaired in a manner consistent with the surrounding area. Grass should be maintained, replaced, reseeded, and mowed as indicated in the section on grassing. The drainage swales will be cleaned annually as needed.

9.12 Landfill Cap

Post-closure maintenance of the cover system will include inspecting the system in those landfill areas that have a differential settlement of 5 feet or more in a horizontal distance of 100 feet. The system shall be repaired in those areas as necessary. Any differential settlement at the landfill is to be corrected to allow drainage paths to remain intact. *Differential settlement* is defined as one area of the closure subsiding or settling faster than the surrounding area. Differential level checks will be performed if evidence of settlement is detected during routine site inspections. Differential level check information will be kept on file and will be made available for FDEP review.

If the final cover must be replaced, repair will follow the original design specifications. Repairs to the final cover will be done under the supervision of a professional engineer. Repairs to the final cover system and test results will be documented in Daily Observation Reports maintained by Pasco County Public Infrastructure. Repairs to any layer of the final cap system will be in accordance with the specifications and Construction Quality Assurance (CQA) Manual approved by FDEP in the closure permit at the time of closure.

9.13 Management of Monitoring Devices

This section describes procedures for maintaining and repairing groundwater and gas monitoring devices. Figure 1 shows the locations of the monitoring. Groundwater monitoring will continue at the West Pasco County Landfill with the potential for periodic revisions after laboratory reports are evaluated.

9.13.1 Gas and Groundwater Monitoring Wells

If a gas monitoring well, groundwater monitoring well, or other monitoring device required by the permit is destroyed or becomes inoperable, Pasco County Public Infrastructure will

notify FDEP in writing within 24 hours. Pasco County Public Infrastructure will submit a corrective plan within 7 days, if required, as shown in Table 2. The written notice will provide a detailed description of what problem has occurred and remedial actions that will be taken. All inoperative monitoring devices shall be repaired or replaced within 60 days of the discovery unless otherwise directed by FDEP.

9.13.2 Groundwater Monitoring

If deemed necessary, the damaged groundwater monitoring well will be properly abandoned and a new well will be constructed close to the abandoned well. Copies of the well abandonment permit and site inspection report will be provided to FDEP. Monitoring well design and replacement will be approved by FDEP before well abandonment and installation. After the construction of the monitoring well is completed, the following information will be provided to FDEP:

1. Well identification.
2. Driller's lithologic log.
3. Latitude/longitude.
4. Total well depth.
5. Aquifer monitored.
6. Casing diameter.
7. Screen type and slot size.
8. Casing type and length.
9. Elevation at top of pipe.
10. Southwest Florida Water Management District (SWFWMD) well construction permit number.
11. Elevation at land surface.

The newly constructed monitor well will be developed and included in the routine monitoring.

Groundwater monitoring will only be conducted by an individual trained in groundwater monitoring and reported to FDEP as required by the closure permit. Care should be taken when equipment is near any monitoring well so that no damage is done.

9.13.3 Gas Monitoring

Soil monitoring probes will be replaced and repaired in accordance with Chapter 62-701.530, FAC. FDEP will be notified of maintenance repair activities.

9.14 Record-Keeping Requirements

Records of information used to develop or support the permit applications and any supplemental information submitted to FDEP shall be kept for the design life of the landfill. Records of monitoring information, including calibration and maintenance records, all original chart recordings for continuous monitoring instrumentation, and copies of all reports required by permit, will be kept for at least 10 years. Background water quality records will be kept for the design life of the landfill. Annual estimates of the remaining life and capacity and site life will be maintained. Annual estimates will be based on a summary of the heights, lengths, and widths of the solid waste disposal units and will be submitted annually

to FDEP. Records more than 5 years old, which are required to be retained, may be archived provided they can be retrieved within 7 days.

9.14.1 Records of Monitoring

Records of water quality monitoring information are to include the following:

1. Facility name and identification number and the surface water and groundwater monitoring points' identification numbers.
2. The date, exact place, and time of sampling or measurements.
3. The person responsible for performing the sampling or measurements.
4. Water levels before sampling.
5. The dates analyses were performed.
6. The person responsible for performing the analyses.
7. The analytical techniques or methods used and method detection limits and applicable water quality standards.
8. STORET code numbers for parameters analyzed.
9. The results of such analyses.

Water quality monitoring reporting and evaluations, including routine sampling events, will be consistent with the pertinent requirements of Section 62-701.510(8), FAC.

Biennial reporting will be performed by a professional engineer or hydrogeologist experienced in hydrogeologic investigations. The biennial report will be updated at the time of permit renewals. The biennial report will meet requirements of Section 62-701.510(8), FAC, and will include the following:

1. Tabular displays of data.
2. Trend analyses.
3. Comparisons of shallow-, middle-, and deep-zone wells.
4. Correlation of parameters and discussions of data correlations.
5. Interpretations of groundwater contour maps and flow rates.
6. Evaluation of the adequacy of water quality monitoring frequency and sampling locations.

9.14.2 Inspection Forms

Inspections of the final cover and stormwater system will be documented and kept on file at the West Pasco County Landfill administration offices.

Deficiencies observed in the fencing and security, access roads, monitoring devices, stormwater system, or final cover system will be documented by Pasco County Public Infrastructure during landfill inspections. The extent of damaged areas, the extent of the areas repaired, and a detailed description of the repair work will be recorded.

9.15 Completion of Long-Term Care

After the long-term-care period for the permitted closure is completed, Pasco County Public Infrastructure will notify FDEP that a certification, signed and sealed by a professional engineer, verifying that long-term care has been completed in accordance with the Closure

Plan has been placed in the operating record. This action will comply with Section 62-701.620(9), FAC.