Gainesville Regional Utilities

Deerhaven Generating Station

Coal Combustion Residuals Landfill Run-on and Runoff Control System Plan
(Version 1.0)

Prepared for:
Gainesville Regional Utilities
Deerhaven Generating Station
10001 NW 13th Street
Gainesville, Florida 32653

Prepared by:

Innovative Waste Consulting Services, LLC
6628 NW 9th Blvd.
Ste. 3
Gainesville, Florida 32605

October 2016

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1.0 Introduction

1.1 Important Note

Based on the calculations conducted as part of the development of this run-on and run-off control system plan, the current stormwater ditch located to the east of Cell 4 of the landfill will need to be reconfigured according to the design presented in the CCR landfill closure and post-closure care plan prior to depositing CCR in Cell 4.

1.2 Overview and Site Description

Gainesville Regional Utilities (GRU) owns and operates a coal combustion residuals (CCR) landfill according to the requirements of Title 40 of the Code of Federal Regulations, Part 257, Subpart D (CCR rule). The CCR landfill is approximately 23 acres in size and was constructed with a slurry wall containment system that was keyed into an existent natural clay layer underlying the landfill; the landfill does not have an engineered bottom liner system. The landfill is organized into four approximately equal-area (5.5 acre) cells, sequenced from Cell 1 (west) to Cell 4 (east).

§257.81(c) of the CCR rule requires the development of a run-on and run-off control system plan that shows how the CCR landfill will be designed, constructed, operated, and maintained with:

- a run-on control system to prevent flow onto the active portion of the landfill during the peak discharge from a 24-hour, 25-year storm
- a run-off control system from the active portion of the landfill to collect and control at least the water volume resulting from a 24-hour, 25-year storm

1.3 Report Organization

This Run-on and Run-off Control System Plan is organized into eight sections. Section 1 presents an overview of the plan and a description of the CCR landfill. Section 2 discusses the preclusion of site run-on and provides a summary of the capacity evaluation of run-off management infrastructure. Section 3 describes the steps GRU will take to prepare the run-off control system for a major storm event. Section 4 describes the steps GRU will take to maintain the run-off control system following a major storm event. Section 5 discusses plan update requirements and provisions for amendment of the plan. Section 6 discusses record keeping, notification and publicly-accessible internet site requirements. Section 7 lists the references used in the development of this plan. Section 8 includes a certification from a qualified professional engineer stating that this run-on and run-off control system plan meets the requirements of the CCR rule.

2.0 Run-on and Run-off Management

Run-on and run-off are defined in the CCR rule as:

- <u>Run-on</u> - any rainwater, leachate or other liquid that drains over land onto any part of a CCR landfill or lateral expansion of a CCR landfill
- <u>Run-off</u> - any rainwater, leachate or other liquid that drains over land from any part of a CCR LF

2.1 Exclusion of Run-on

The existing site topography surrounding the landfill precludes the possibility of landfill run-on. As depicted in drawings Y65-3, Y67-3 and Y81-2 of B&M (1981), a berm exists on the northern side of the

Northern Drainage Ditch with a V-shaped drainage ditch at its toe; run-on is not possible from land located to the north of the landfill. An unpaved perimeter access road bounds the landfill on its western and southern sides. This perimeter road is at a higher elevation than the low-lying forested areas on the outside of the road; run-on is not possible from the land areas adjacent to the west or the south of the landfill. An open field lies directly east of the paved access road that borders the landfill on its eastern side. As presented in drawing Y67-3 of B&M (1981), this field was graded at a 0.4% slope so that it drains away from the landfill to the east-southeast; run-on is not possible from the field located to the east of the landfill.

2.2 Run-off Classification and Management

Landfill run-off can be classified into CCR contact water and stormwater. Contact water (i.e., water that has come in contact with CCR) consists of liquid that has run over the surface of exposed CCR material. Stormwater includes precipitation that has not come in contact with CCR. CCR contact water will be routed through a series of downdrain pipes that will be incrementally installed along the northern periphery of the landfill mound as filling progresses. The downdrain pipes will discharge to a large ditch in the northern portion of the landfill (i.e., Northern Drainage Ditch) that collects and temporarily stores water that has come into contact with CCR.

Stormwater will be collected from landfill side slopes by means of ditches located along the landfill's western, southern and eastern sides and discharged to a stormwater pond located to the southeast of the landfill by means of culvert pipes located at each ditch's terminus. Figure 2-1 presents a layout of the landfill with important features of the run-off control system that will be referred to throughout this plan.

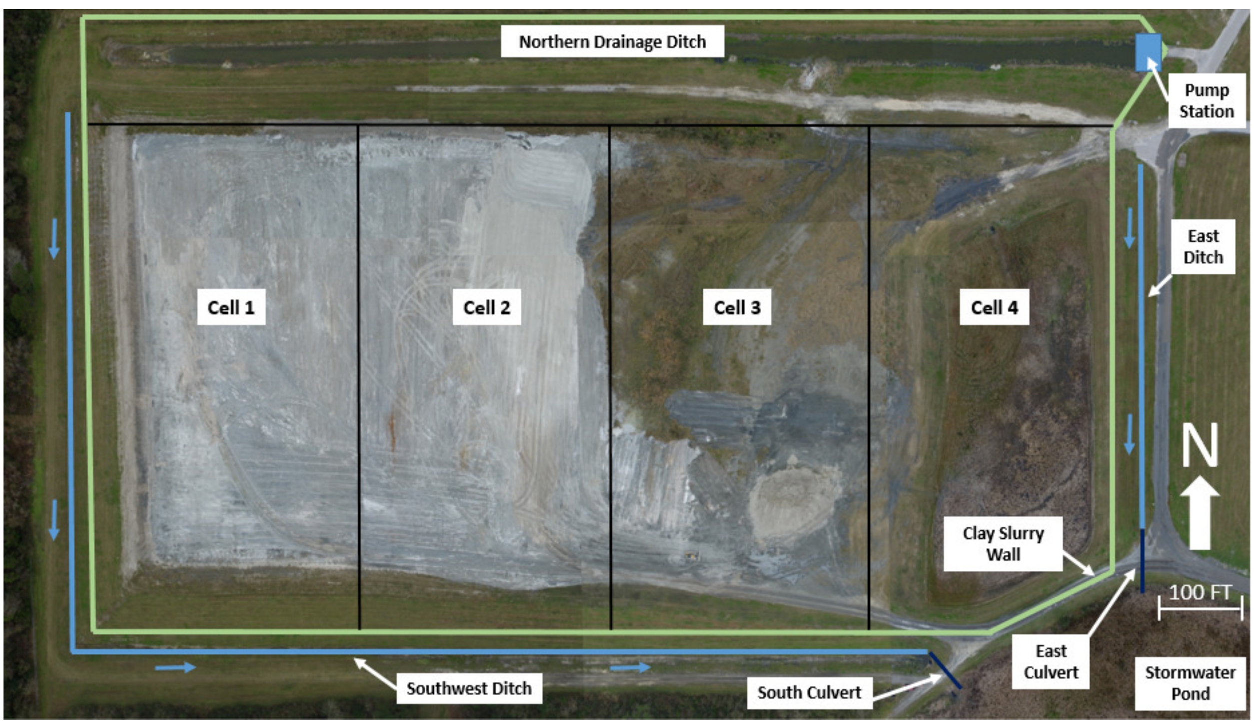


Figure 2-1. CCR Landfill Layout with Run-off Control Infrastructure (Image from IWCS (2016))

2.2.1 CCR Contact Water - Downdrains

A landfill phasing plan exists as part of a best management practices guide (GRU 2015) which includes information on the size, number, arrangement and location of downdrain pipes. The phasing plan includes the progressive fill of the landfill with active areas sloped at 2% to provide drainage towards the north. Twelve (12)-inch diameter, high density polyethylene (HDPE) downdrain pipes collect and route CCR contact water that collects along the inside edge of the northern peripheral containment berm; each pipe has an inlet located at the inside toe of the berm. The pipe then protrudes through the containment berm, daylights on the northern side slope, runs down the slope and underneath the unpaved access road (located between the landfill and the Northern Drainage Ditch), and then discharges to the Northern Drainage Ditch. Appendix A includes downdrain cross sections from GRU (2014).

2.2.1.1 Critical Area

The critical area considered for CCR contact water generation used to evaluate the capacity of the downdrain system was the bottom-most active area during initial filling. The best management practices guide (BMP) calls for the filling of Cells 1 and 2 (i.e., Basin 1) and then for the filling of Cells 3 and 4 (i.e., Basin 2). The largest, bottom-most active area of the initial fill phase for Basin 2 is larger than Basin 1 and is approximately 342,000 square feet.

2.2.1.2 Capacity Evaluation

The capacity of the downdrain system was evaluated assuming a worst-case scenario involving the complete obstruction of both downdrain pipes used for Basin 2 during its initial fill phase. In this event, CCR contact water will accumulate along the inside toe of the northern containment berm. Based on the geometry of the peripheral containment berms included in the phasing plan of the BMP, and as shown in Appendix C, there is a sufficient volume to contain the accumulated liquid associated with the design storm in the area inside the northern containment berm. Once the obstructions were removed, and as shown in Appendix C, it is estimated that the pipes would discharge the complete volume of accumulated liquid in approximately 5.3 hours.

2.2.2 CCR Contact Water – Northern Drainage Ditch

2.2.2.1 Critical Area

The maximum area discharging to the northern drainage ditch includes the initial filling phase of the Basin 1 and Basin 2 areas (as discussed in the previous section), the area of the landfill northern side slope, the area of the access road that lies between the landfill mound and the northern drainage ditch, and the area of the ditch itself.

2.2.2.2 Capacity Evaluation

The capacity of the Northern Drainage Ditch was evaluated on a volumetric basis under the scenario where existing pump infrastructure (i.e., a pump station located at the eastern end of the ditch) was offline for the duration of the design storm event. Based on the geometry of the northern drainage ditch as presented in drawings from B&M (1981) and the calculations presented in Appendix D, the volume of the northern drainage ditch is not sufficient to handle the total run-off expected from a 24-hour, 25-year design storm. However, an existing basin located in Cell 4 of the landfill has sufficient capacity to handle the excess run-off volume associated with the design storm. Procedures for pumping to this area are detailed in Section 3.2.

2.2.3 Stormwater Perimeter Ditches and Culverts

There are two ditch and culvert pairs which collect and divert stormwater away from the landfill:

- **Southwest Ditch and Culvert** - located along the western and southern sides of the landfill, this ditch collects stormwater from the landfill's western and southern side slopes. The ditch terminates at a 36-inch HDPE culvert pipe which discharges to the stormwater pond located to the southeast of the landfill.
- **Eastern Ditch and Culvert** - located along the eastern side of the landfill, the ditch will collect stormwater from the landfill's eastern side slope. The ditch will terminate at a dual 24-inch HDPE culvert pipe which discharges to the stormwater pond located to the southeast of the landfill.

2.2.3.1 Critical Areas

The maximum stormwater generation rate that will occur during the active life of the landfill will occur when the landfill reaches final grades. Therefore, the final grading plan included in the landfill's closure and post-closure care plan (GRU 2016) was used to calculate the maximum areas and corresponding maximum discharge rates to each stormwater and culvert pair. Calculations estimating the maximum discharge rates to these ditches and culverts can be found in Appendix B.

2.2.3.2 Capacity Evaluation

An evaluation of the ditches and culverts to handle the maximum stormwater discharge is presented in Appendix D and E, respectively. Based on the calculations and the configuration of the stormwater infrastructure, it was estimated that the southwestern ditch, southern culvert pipe and eastern culvert pipe appear to have sufficient capacity to handle the maximum stormwater run-off flows expected during the active life of the landfill.

However, based on the calculations, the eastern drainage ditch will need to be expanded according to the geometry presented in the GRU (2016) CCR Landfill Closure and Post-Closure Care Plan prior to starting the fill of Cell 4.

3.0 Preparation for Major Storm Events

3.1 Inspection of Run-off Control Features

At least 48-hours prior to a major storm event (e.g., tropical storm, hurricane), GRU will inspect and (as necessary) repair/maintain the following run-off control infrastructure:

- **Downdrains/Culverts** – ensure downdrain/culvert inlets and outlets are free of obstruction and that there is no evidence of pipe damage along the entire pipe lengths
- **Ditches** – ensure ditches are free of vegetation or sediment obstruction and that vegetation height is minimized

3.2 Northern Drainage Ditch Management

To prevent overtopping of the northern drainage ditch, GRU will take the following steps:

1. Pump out/drain all existing water in the Cell 4 basin area. All water in the Cell 4 basin area will be managed as CCR contact water.
2. Pump out all existing water in the northern drainage ditch

3. Plug the existing internal culvert pipe located in the northern section of Cell 4 using an inflatable pipe plug
4. Verify the operation of and position mobile centrifugal pumps capable of a combined total pump rate of at least 800 gallons per minute to pump from the northern drainage ditch to the Cell 4 basin area.
5. As soon as there is sufficient water in the Northern Drainage Ditch, GRU will start pumping from the ditch to the Cell 4 basin area.
6. During the storm event, GRU will monitor the liquid levels in the Northern Drainage Ditch and the Cell 4 basin area and adjust the pump rate to ensure that liquid does not overtop/overflow either the ditch or berm surrounding the Cell 4 basin area.

4.0 Run-off Control Following a Major Storm Event

4.1 Inspection of Run-off Control Features

Following completion of the storm event, GRU will inspect and repair any damage to the run-off control infrastructure included in the pre-storm inspection.

4.2 Northern Drainage Ditch Management

Following the storm event, GRU will pump out all accumulated liquid from the northern drainage ditch and Cell 4 basin area to the ash ponds of the CCR surface impoundment system. GRU will then remove the inflatable plug from the internal culvert pipe located in the northern section of Cell 4.

5.0 Run-on and Run-off Control System Plan Updates and Amendments

Per §257.81(c)(4), GRU will revise the Run-on and Run-off control system plan every 5 years. The 5-year interval will begin at the point the initial plan is placed in the operating record. As required by §257.81(c)(2), GRU will amend this plan whenever there is a change in conditions that would substantially impact the plan in effect.

6.0 Record Keeping, Notifications, Publicly-Accessible Website Requirements

GRU will place a copy of this and any updated/amended Run-on and Run-off Control System Plans in the operating record as it becomes available (per §257.105(g)(3)) and within 30 days of placement in the operating record, will send a notification of the availability of the plan to the Florida Department of Environmental Protection (per §257.106(g)(3)) and will post a copy of the plan to its publicly-accessible website (per §257.107(g)(3)).

7.0 References

B&M (1981). Construction Drawings. Deerhaven Generating Station Unit 2. City of Gainesville/Gainesville-Alachua County Regional Utilities Board. Prepared by Burns & McDonnell, Kansas City, Missouri.

GRU (2015). Best Management Practices Guide for Managing Coal Combustion Residuals at the Deerhaven Generating Station. Version 3.0. Report prepared by Innovative Waste Consulting Services, LLC, Gainesville, Florida for Gainesville Regional Utilities, Gainesville, Florida.

GRU (2016). Gainesville Regional Utilities Deerhaven Generating Station - Coal Combustion Residuals Landfill Closure and Post-Closure Care Plan (Version 1.0). Prepared for Gainesville Regional Utilities Deerhaven Generating Station by Innovative Waste Consulting Services, LLC, October 2016.

IWCS (2016). CCR Landfill. Aerial Imagery. Deerhaven Generating Station, Gainesville, Florida. Photograph taken 14 January 2016.

8.0 Professional Engineer Certification

This plan was prepared under the supervision, direction and control of the undersigned, registered professional engineer (PE). The undersigned PE is familiar with the requirements of 40 CFR 257.81 and certifies that this CCR Landfill Run-on and Run-off Control System Plan meets the requirements of 40 CFR 257.81.

Name of Professional Engineer: Justin Lamar Smith

Company: Innovative Waste Consulting Services, LLC

Signature: Justin L. Smith

Date: 10/14/16

PE Registration State: Florida

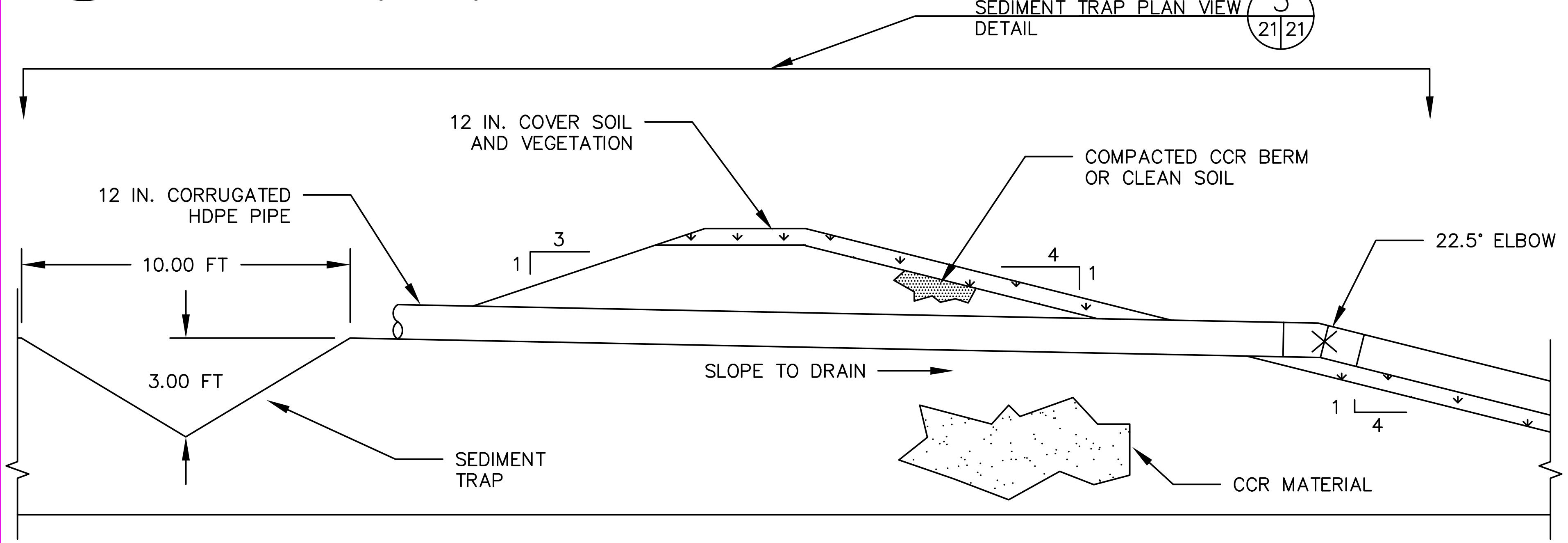
PE License No.: 80463

JUSTIN L. SMITH
LICENSE
No. 80463
STATE OF FLORIDA
PROFESSIONAL ENGINEER

APPENDIX A. PLAN-REFERENCED DRAWINGS

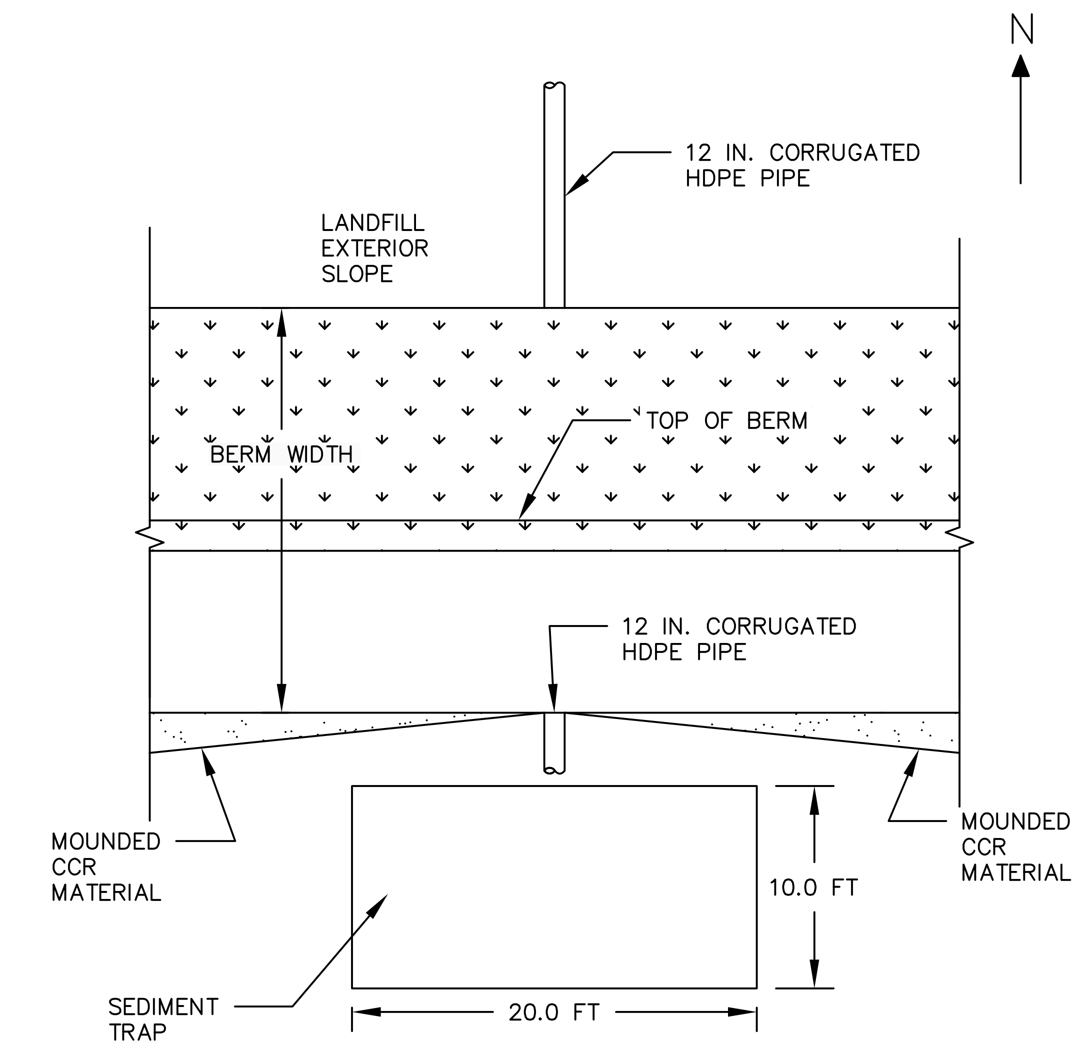
1 | 5 | 21

STORMWATER DOWN DRAIN PIPE DETAIL (NTS)



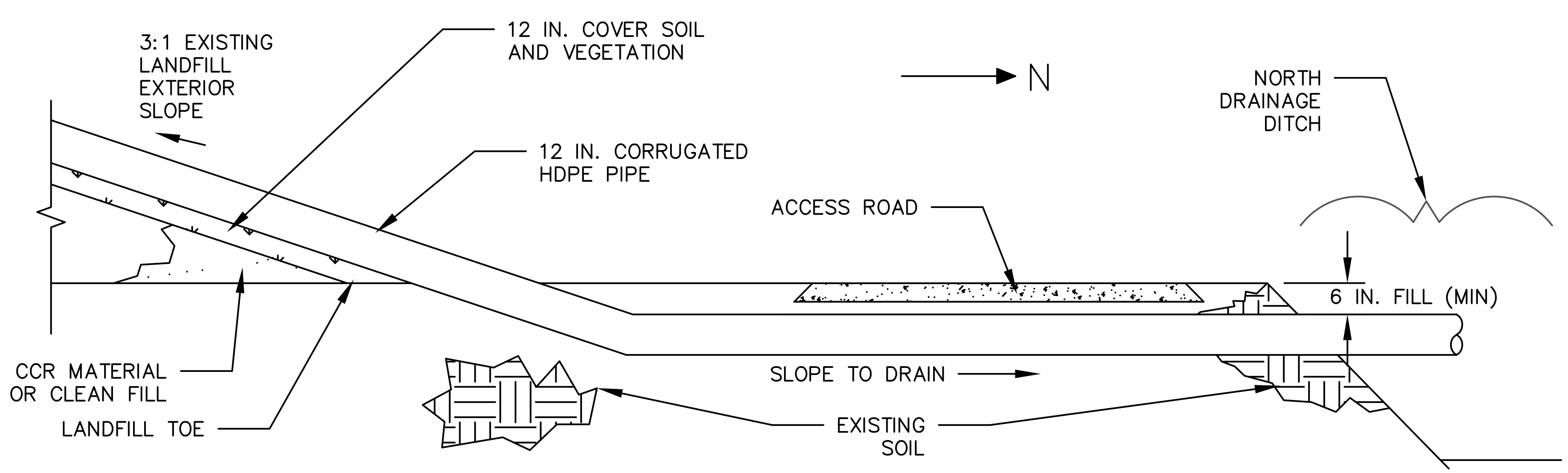
3 | 21 | 21

SEDIMENT TRAP PLAN VIEW DETAIL (NTS)



2 | 5 | 21

DOWN DRAIN ROAD DETAIL (NTS)



4/22/2013 10:04:18 AM

LTR	DATE	REVISIONS	BY	APPRD.

DESIGNED JP, JS, AB
DRAWN AB, JS
CHECKED JP, PJ

IWCS innovative waste consulting services, llc

INNOVATIVE WASTE CONSULTING SERVICES, LLC
6628 NW 9th BLVD. SUITE 3
GAINESVILLE, FL 32605-4282
PHONE: (352) 331-4828
FAX: (352) 331-4842

GRU

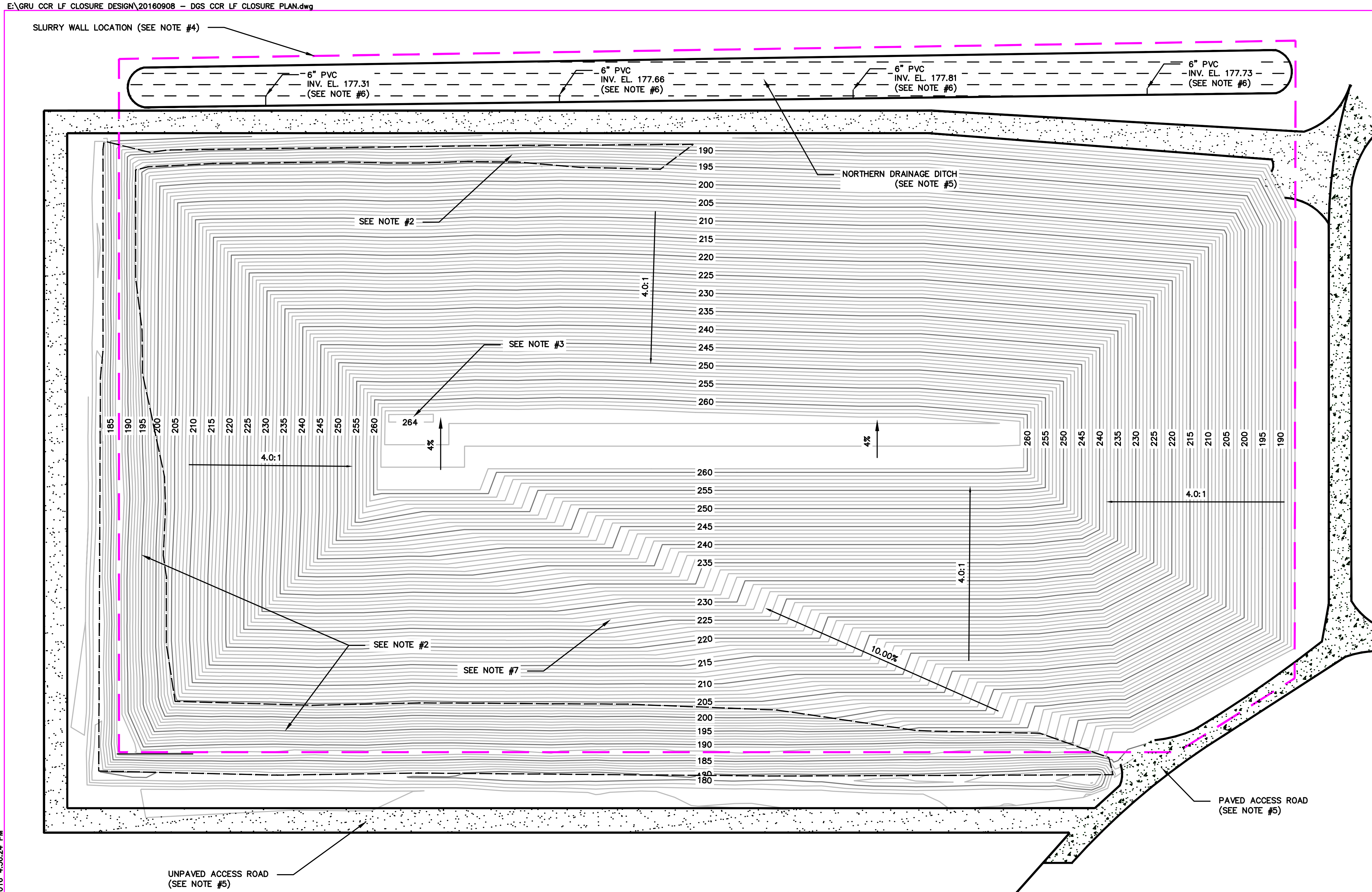
GAINESVILLE REGIONAL UTILITIES
10001 NW 13TH STREET
GAINESVILLE, FL 32653

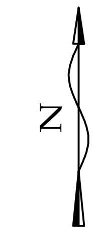
CCR LANDFILL SEQUENCING PLAN

DETAIL – STORMWATER DOWN DRAIN PIPES

DATE: 4/22/13
PRJ No: 13016
SCALE: AS SHOWN
DWG No: D-21

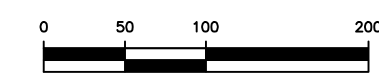
E:\GRU CCR LF CLOSURE DESIGN\20160908 - DGS CCR LF CLOSURE PLAN.dwg





NOTES:

1. FINAL GRADES REPRESENT THE PROPOSED CCR LANDFILL SURFACE AT FINAL BUILD OUT, INCLUDING A 1-FOOT INTERMEDIATE SOIL COVER CAP. THE PRECISE LOCATION OF GRADE BREAKS AND ELEVATIONS WILL DEPEND ON THE FIELD CONDITIONS AT THE TIME OF CLOSURE.
2. AREAS NEAR THE TOE OF THE NORTHERN, WESTERN AND SOUTHERN SIDE SLOPES WILL BE STEEPER THAN 4:1 (H:V) BASED ON EXISTING CONDITIONS. THE APPROXIMATE EXTENTS OF THESE AREAS ARE DELINEATED.
3. THE MAXIMUM ELEVATION OF THE CCR LANDFILL WITH THE INTERMEDIATE COVER SOIL LAYER IS DESIGNED TO BE APPROXIMATELY 264 FT.
4. THE NORTHERN AND EASTERN EXTENT OF THE SLURRY WALL IS APPROXIMATE AND BASED ON A 1981 DRAWING SET FROM BURNS AND MCDONNELL CONFORMING TO CONSTRUCTION RECORDS.
5. THE LOCATION OF THE NORTHERN DRAINAGE DITCH AND ACCESS ROADS IS APPROXIMATE.
6. THESE PIPE INVERTS REPRESENT THE OUTLETS OF AN EXISTING UNDERDRAIN SYSTEM FOR CCR CONTACT WATER.
7. AS NECESSARY, SLOPES MILDER THAN 4:1 MAY BE USED TO TIE-IN PROPOSED FINAL GRADES WITH EXISTING GRADES.



GRAPHIC SCALE
(FEET)

NOT FOR CONSTRUCTION

9/8/2016 4:56:24 PM

LTR	DATE	REVISIONS	BY	APPRD.

DESIGNED JS
DRAWN JS
CHECKED PJ

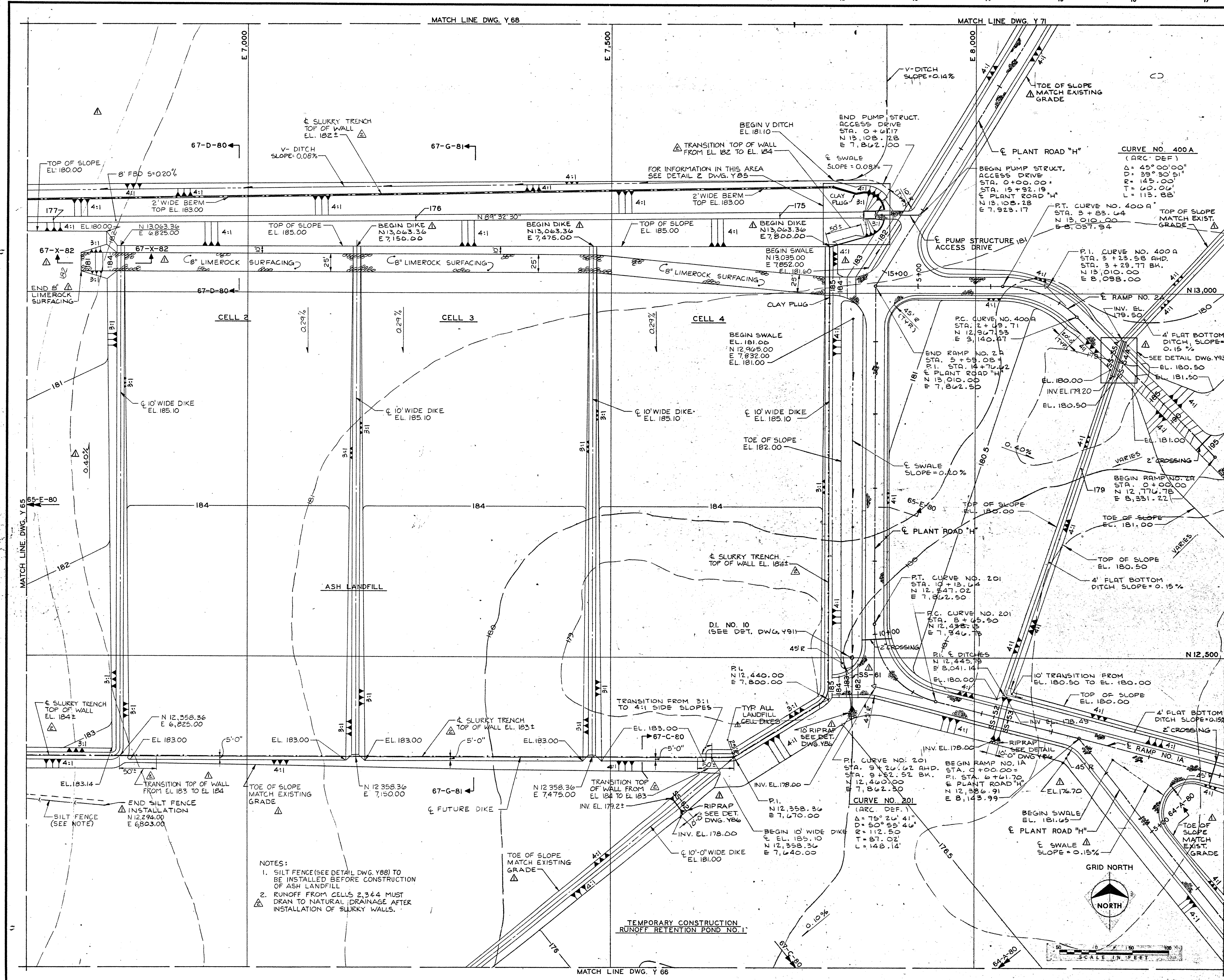
INNOVATIVE WASTE CONSULTING SERVICES, LLC
6628 NW 9th BLVD. SUITE 3
GAINESVILLE, FL 32605-4282
PHONE: (352) 331-4828
FAX: (352) 331-4842

GAINESVILLE REGIONAL UTILITIES
DEERHAVEN GENERATING STATION
10001 NW 13TH ST.
GAINESVILLE, FL 32653

CCR LANDFILL CLOSURE PLAN

PROPOSED FINAL GRADES

DATE: 8 SEPTEMBER 2016
PRJ No:
SCALE: AS SHOWN
DWG No: 5 OF 20



NO.	DATE	BY	REVISION
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	3-21-80	DMZ	ISSUED WITH ADDENDUM NO. 3
	4-22-80	DMZ	ISSUED AS BID
5	6-10-80	GWB	ADDED SLURRY TRENCH
	6-24-80	DMZ	ISSUED
6	6-30-81	DJB	CONFORMING TO CONSTRUCTION RECORDS

CONTRACT NO. 29C
YARD STRUCTURES III

DEERHAVEN GENERATING STATION UNIT 2

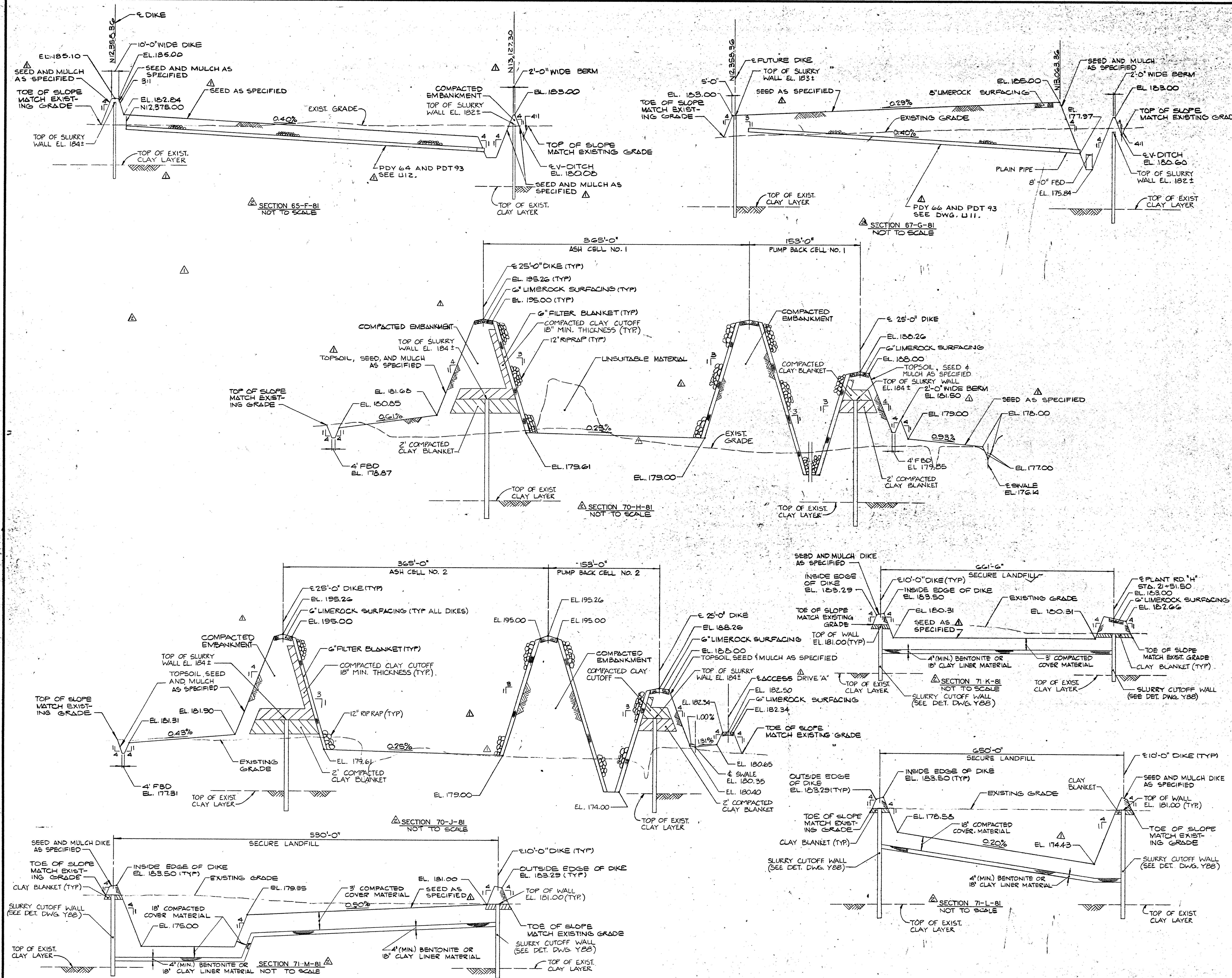
CITY OF GAINESVILLE/
GAINESVILLE-ALACHUA COUNTY
REGIONAL UTILITIES BOARD

FLORIDA

GRADING PLAN 4

Burns & McDonnell
Engineers-Architects-Consultants
KANSAS CITY, MISSOURI

DATE FEB. 18, 1980	DRAWING NO. Y67	REV. 3	
DESIGNED HUTCHESON			
DETAILED BAKER	PROJECT 76-077-1		
CHECKED DMZ	SHEET	OF	SHEETS



NO.	DATE	BY	REVISION
1	3-10-80	DJB	REVISED & ADDED NOTES FOR CLARIFICATION.
	3-12-80	TDW	(F-9, J-6) REVISED TIMBER POLE LINER PROTECTION
	3-21-80	DMZ	ISSUED WITH ADDENDUM NO. 3
	4-22-80	DMZ	ISSUED AS BID
2	6-13-80	GWB	REVISED PER ALTERNATE BIDS NO. 3 & NO. 4
	6-24-80	DMZ	ISSUED
	7-1-81	DJB	CONFORMING TO CONSTRUCTION RECORDS

CONTRACT NO. 29C
YARD STRUCTURES III

DEERHAVEN GENERATING STATION UNIT 2

CITY OF GAINESVILLE/ GAINESVILLE-ALACHUA COUNTY REGIONAL UTILITIES BOARD

FLORIDA

GRADING SECTIONS 2

Burns & McDonnell
Engineers–Architects–Consultants
KANSAS CITY, MISSOURI

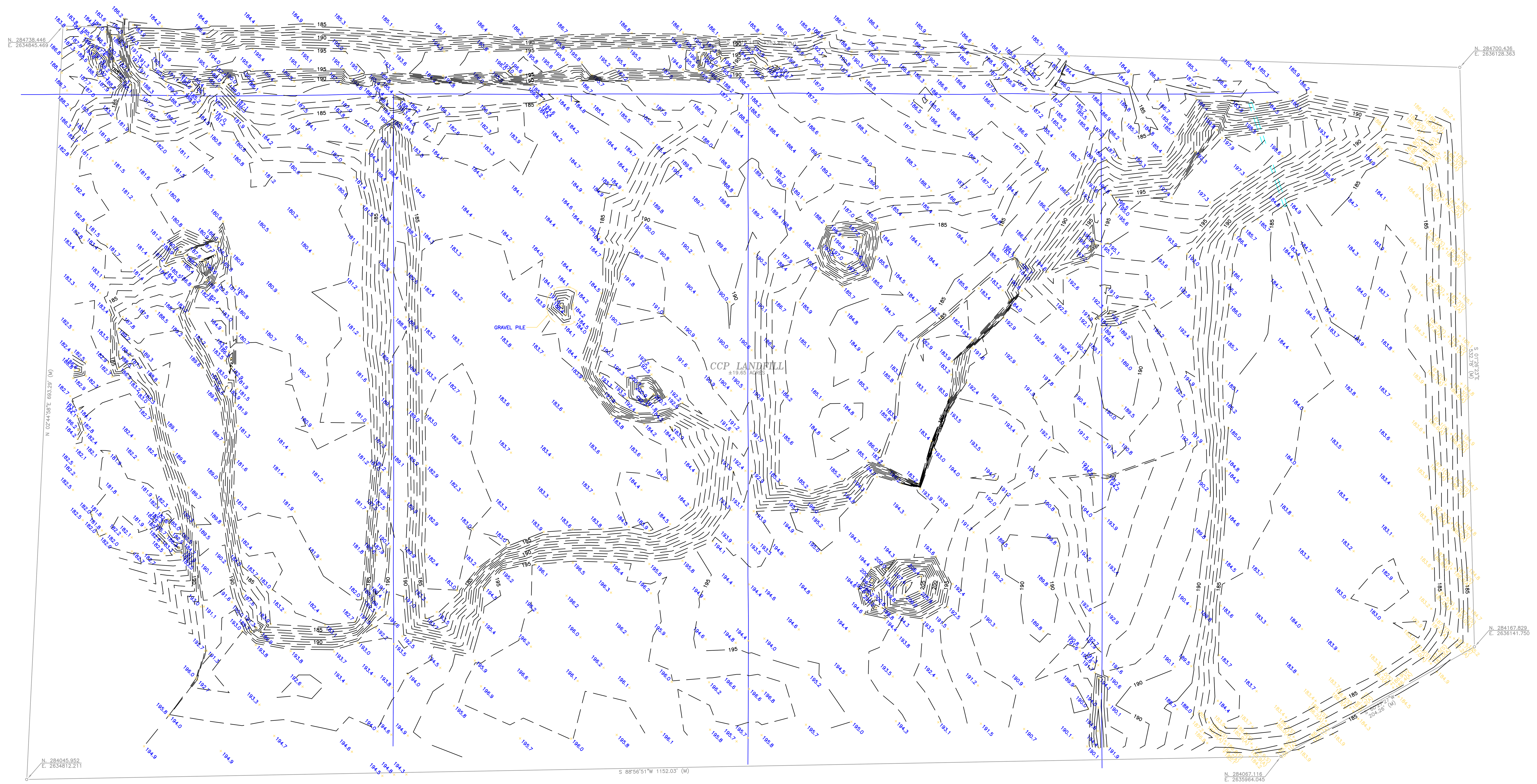
DATE FEB. 18, 1980	DRAWING NO. Y81 REV. 2
DESIGNED HUTCHESON	
DETAILED MADDOCK	PROJECT 76-077-1
CHECKED DMZ	SHEET OF SHEETS

SPECIFIC PURPOSE & TOPOGRAPHIC SURVEY

GAINESVILLE REGIONAL UTILITIES – DEERHAVEN GENERATING STATION CCP LANDFILL
IN SECTION 27, TOWNSHIP 8 SOUTH, RANGE 19 EAST, ALACHUA COUNTY, FLORIDA

50 0 50 100 150
1" = 50'

BENCHMARK
NAIL & DISK IN S.E. CORNER
OF CONC PUMP STATION
ELEV. = 183.40



CERTIFIED TO:

1) MORETRENCH

LEGEND:

= FOUND 1/2" REBAR & CAP L.B. 6894

ABBREVIATIONS:

(A) = ASH ELEVATION
CCP = COAL COMBUSTION PRODUCT
CPP = CORRUGATED PLASTIC PIPE
CONC = CONCRETE
LB = LICENSED SURVEYOR BUSINESS
(M) = FIELD MEASURED
PSM = PROFESSIONAL SURVEYOR & MAPPER
(S) = TOP SOIL ELEVATION
TBM = TEMPORARY BENCHMARK

SURVEYOR NOTES:

1) ALL ELEVATIONS SHOWN HEREON ARE IN FEET. THE ELEVATION DATUM IS UNKNOWN TO THIS SURVEYOR. THIS SURVEYOR HAS USED BENCHMARK AND PLANT GRID COORDINATE INFORMATION PROVIDED BY GRU IN DEERHAVEN PLANT GENERAL SITEWORK DRAWINGS BY EBAUGH & GOETHE, INC. DATED 5/30/74.
2) ALL UTILITIES AND OR IMPROVEMENTS, IF ANY, MAY NOT BE SHOWN ON THIS DRAWING.
3) THE PURPOSE OF THIS SURVEY IS TO SHOW AS-BUILT CONDITIONS OF THE BERM (CCP AND TOP SOIL), NEW HEADER AND UNDERDRAIN PIPES AND THEIR COVERAGE.
4) THE BOUNDARY OF THE CCP LANDFILL WAS DESIGNED AND DETERMINED BY INNOVATIVE WASTE CONSULTING SERVICES AND EXTRAPOLATED FROM A SURVEY PERFORMED BY THIS OFFICE DATED 4-19-2009.
5) THIS MAP OF SURVEY REFLECTS CONDITIONS LOCATED AS OF THE DATE OF FIELD WORK COMPLETION (SEE TITLE BLOCK).
6) AREAS OF ENVIRONMENTAL CONCERN HAVE NOT BEEN LOCATED BY THIS SURVEYOR, UNLESS OTHERWISE DEPICTED HEREON.

CERTIFICATE OF SURVEYOR:

NOT VALID WITHOUT THE SIGNATURE AND THE ORIGINAL RAISED SEAL OF A FLORIDA LICENSED SURVEYOR AND MAPPER. ADDITIONS OR DELETIONS TO THIS MAP BY ANYONE OTHER THAN THIS SURVEYOR IS PROHIBITED.

I HEREBY CERTIFY THAT THE SURVEY DATA SHOWN HEREON, IS A TRUE AND CORRECT REPRESENTATION OF A SURVEY PERFORMED UNDER MY SUPERVISION OF THE HEREON DESCRIBED PROPERTY, AND IT MEETS THE MINIMUM TECHNICAL STANDARDS AS SET FORTH BY THE FLORIDA BOARD OF LAND SURVEYORS, PURSUANT TO SECTION 472.027, FLORIDA STATUTES, AND CHAPTER 61G17-6, FLORIDA ADMINISTRATIVE CODE.

BY:______________________
JAMES E. BRINKMAN, PSM – FLA. CERT# 5582

DATE:______________________

BRINKMAN SURVEYING & MAPPING INC.
4607 NW 6th STREET SUITE C, GAINESVILLE, FL. 32609
PHONE: (352) 374-7707 FAX: (352) 374-8757

SCALE: 1" = 50'	"THE BENCHMARK IN QUALITY SERVICE"	DRAWN BY: B.G.
DATE: 5/3/2010		CHECKED BY: J.B.
FIELD WORK COMPLETED ON 4/22/2010		
PREPARED FOR: MORETRENCH	DRAWING NUMBER 050-AS-BUILT-TOPO-09	

APPENDIX B. PEAK DISCHARGE CALCULATIONS

B STORMWATER PEAK DISCHARGE RATE

This calculation package estimates the peak run-off discharges that must be accepted by the downdrain pipes located along the northern portion of the landfill; the ditches located along the western, southern and eastern sides of the landfill; and the culvert pipes located at terminus of these ditches based on the precipitation expected from a 24-hr, 25-yr design storm. Based on the geographic location of the CCR landfill and the National Weather Service Hydrometeorological Design Studies Center website, the site-specific rainfall from a 24-hr, 25-yr storm was estimated to be 7.27 inches (NOAA 2015).

The landfill areas considered in this analysis include:

1. The initial phase active area for Cells 1 and 2 (i.e., Basin 1) and Cells 3 and 4 (i.e., Basin 2)
2. The final grade side slope areas and swale areas for the western and southern ditches
3. The intermediate side slope and swale area for the eastern ditch

The peak discharge for each basin is found according to the following (USDA 1986) Equation (1):

$$q_p = q_u * A_m * Q_r * F_p \qquad (1)$$

Where,

q_p = peak discharge (cfs)

q_u = unit peak discharge (csm/in)

A_m = drainage area (mi^2)

Q_r = runoff (in)

F_p = pond and swamp adjustment factor (= 1.00 for 0% pond and swamp area)

Runoff, Q_r, can be found using previously determined input variables through the following (USDA 1986) Equation (7):

$$Q_r = \frac{(P-0.2S)^2}{P+0.8S} \qquad (2)$$

It was assumed that the landfill areas are 0% swamp or pond. Therefore, F_p was assumed as 1.0.

The critical (or greatest) q_u is found by determining the critical (or shortest) time of concentration, T_c, by using the plot in Exhibit 4-II from USDA (1986). The appropriate curve used in this plot is found by solving the ratio of initial abstraction to precipitation, where the equation for initial abstraction has been generalized for agricultural watersheds and is represented as (USDA 1986) Equation (3):

$$I_a = 0.2 \times S \qquad (3)$$

Where,

I_a = Initial abstraction, or runoff loss (in)

S = Potential maximum retention after runoff begins (in)

S can be found by determining the curve number for the runoff area, as presented in the following (USDA 1986) Equation (4):

$$S = \frac{1000}{CN} - 10 \tag{4}$$

Where,

CN = curve number based on site surface soil conditions.

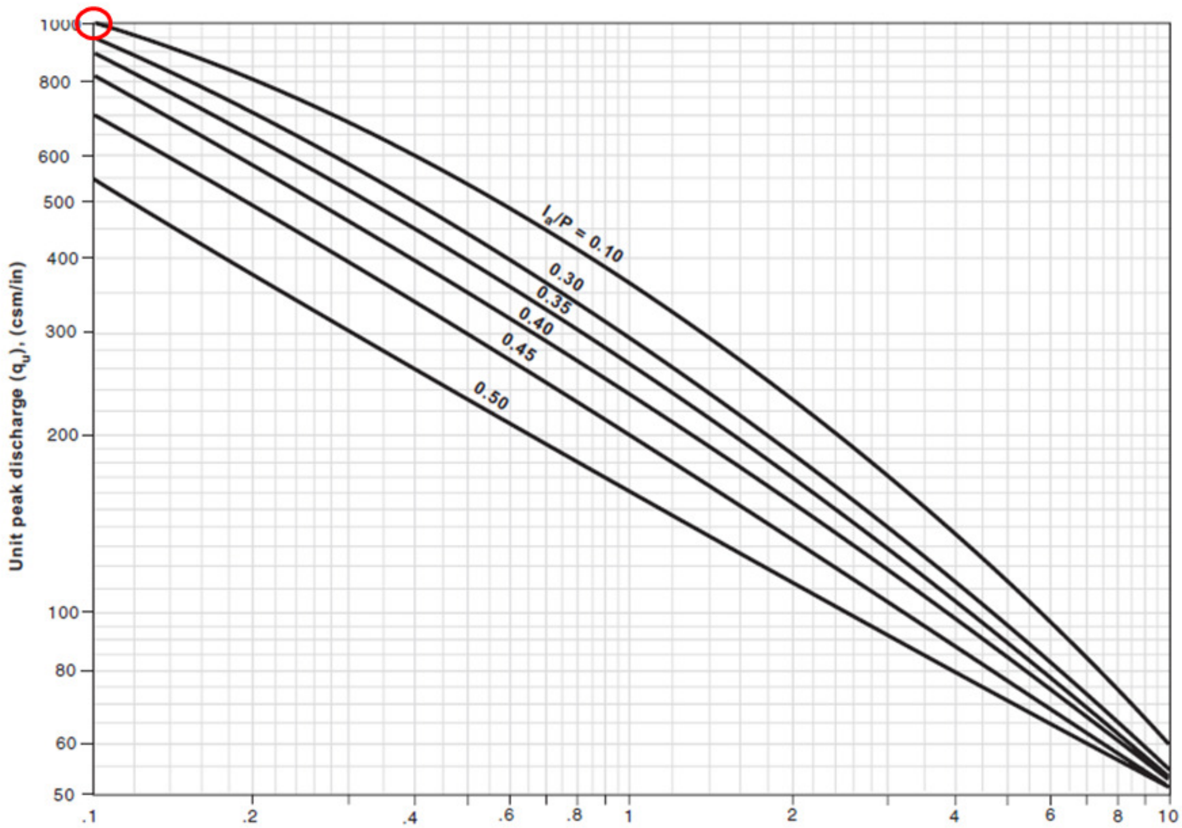
The soil type used for future cover at the site is unknown. Therefore, for the purpose of a conservative design, and based on a review of Appendix A of Technical Release 55 (USDA 1986), hydrologic soil group D (clay loam, silty clay loam, sandy clay, silty clay, or clay) was selected to estimate runoff from intermediate and final cover soils. Hydrologic soil group D (clay loam, silty clay loam, sandy clay, silty clay, or clay) was used to represent the CCR working faces of Basin 1 and Basin 2. For open spaces with "good condition" grass cover (grass cover >75%), the runoff curve number is estimated as 80 for hydrologic soil group D. For a newly-graded area with no vegetation, Table 2-2a (USDA 1986) provides a runoff curve number estimate of 94 for hydrologic soil group D. The potential maximum retention, initial abstraction, runoff, and ratio of initial abstraction to precipitation for each soil type are shown in the table below.

Table 1. Run-off Parameters of Hydrologic Soil Groups

Area	Hydrologic Soil Group	Curve Number (CN)	Potential Maximum Retention (in.)	Initial Abstraction (in.)	Runoff (in.)	I_a/P
Cover Soil	D	80	2.5	0.5	4.9	0.07
CCR Working Face	D	94	0.64	0.13	6.6	0.02

As estimated based on location and NOAA (2015), the P (i.e., 25-year frequency, 24-hour rainfall) for the site is 7.27 inches and the I_a/P for both areas is below the range of values listed in Exhibit 4-II of USDA (1986), and shown on the next page. Since I_a/P for both areas are below the range of values listed in Exhibit 4-II, it was assumed that the maximum unit peak discharge (q_u) for all the drainage basins in this analysis is 1000 csm/in (the maximum y-intercept of I_a/P in USDA (1986) Exhibit 4-II). This provides a conservative estimate of the unit peak discharge that could occur at the site.

Exhibit 4-II Unit peal discharge (q_u) for NRCS (SCS) type II rainfall distribution



Tables 3 and 4 present the estimated peak discharge rates for the swales and road-crossing culverts, respectively. It is important to note that the contributing areas outlined in the tables include the area of the swales themselves. The areas presented in the table were evaluated using AutoCAD Civil 3D 2012 and the site's phasing plan and closure plan design drawings.

Table 2. Downdrain Pipe Basins

Basin	Contributing Area (ft^2)	Area (mi^2)	Unit Peak Discharge, q_u (csm/in)	Peak Discharge, q_p (cfs)
Basin 1	301,000	0.01079	1000	70.8
Basin 2	342,000	0.01226	1000	80.4

Table 3. Ditch Basins

Basin	Contributing Area (ft^2)	Area (mi^2)	Unit Peak Discharge, q_u (csm/in)	Peak Discharge, q_p (cfs)
West (Final Grades)	170,000	0.00609	1000	30.1
South (Final Grades)	284,000	0.01017	1000	50.3
East (Final Grades)	107,000	0.00384	1000	19.0
East (@ 190 ft elevation)	25,900	0.00093	1000	4.6
Northern Drainage Ditch	883,000			

Table 4. Culvert Pipe Basins

Basin	Contributing Area (ft^2)	Area (mi^2)	Unit Peak Discharge, q_u (csm/in)	Peak Discharge, q_p (cfs)
West+South (Final Grades)	453,000	0.01627	1000	80.4
East (Final Grades)	107,000	0.00384	1000	19.0

References

NOAA (2015). NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: FL. http://bit.ly/1Lji8tK accessed 3 November 2015. Data found for site coordinates using the National Oceanic and Atmospheric Administration National Weather Service Hydrometeorological Design Studies Center Precipitation Frequency Data Server.

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APPENDIX C. DOWNDRAIN CAPACITY CALCULATIONS

C.1 DOWNDRAIN BASIN CAPACITY CALCULATIONS

Based on the current phasing plan for the CCR landfill, the initial phase of Basin 2 (i.e., Cells 3 & 4) has the largest area that can contribute to CCR contact water run-off. The purpose of the calculations in this section is to estimate whether the low-lying area at the inside toe of the northern peripheral containment berms is capable of temporarily retaining CCR contact water generated from a 24 hour, 25-year design storm under worst-case conditions where both underdrain inlets are obstructed. A general schematic of a cross section of Basin 2 is presented in Figure 1.

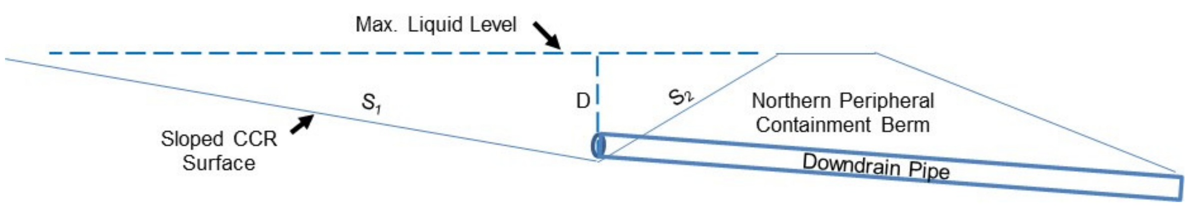

Figure 1. Basin 2 Downdrain Area Cross Section

The area of this cross section can be found as:

$$A = \frac{D^2}{2}\left(\frac{1}{S_1} + \frac{1}{S_2}\right) \quad (1)$$

Where,

S_1 = Slope of working face area (ft/ft)

S_2 = Slope of containment berm (ft/ft)

D = Maximum liquid depth (ft)

The capacity (volume) of the basin can then be calculated as:

$$Volume = A \times W \quad (2)$$

Where,

W = east-west width of the basin area

The total CCR contact water generated in Basin 2 was calculated by multiplying the total area of Basin 2 (see Appendix B) and multiplying it by the precipitation associated with the 24-hour, 25-year design storm. Table 1 provides a summary of the values used for the input variables in the Basin 2 downdrain basin capacity evaluation, and Table 2 provides a summary of the calculation results.

Table 1. Inputs for Basin 2 Downdrain Basin Capacity Calculation

Parameter (unit)	Value
Area contributing to flow (ft^2)	342,000
P (in)	7.27
S_1, slope of working face	0.02
S_2, slope of berm	0.33
Width of cross section (ft)	585
D, Design Depth (ft)	4.00

Table 2. Outputs for Basin 2 Downdrain Basin Capacity Calculation

Total CCR Contact Water Generated (ft3)	Basin Capacity (ft3)
207,000	248,000

As presented in Table 2, the downdrain basin area located in Basin 2 is estimated to be able to retain approximately 248,000 ft^3 of liquid while the total volume of CCR contact water that could be generated from the initial phase of the Basin 2 area (i.e. the working area during landfill phasing) during a 24-hour, 25-year storm is 207,000 ft^3. Therefore, the capacity of the downdrain basin areas are considered acceptable.

C.2 DOWNDRAIN PIPE DRAINAGE TIME CALCULATIONS

This section estimates the time it takes for the two downdrain pipes of the initial phase of the Basin 2 area to drain the volume of CCR contact water that would be retained under a worst-case scenario where the underdrain outlets were obstructed. The following assumptions were made for this calculation:

- It was assumed that the initial segment (i.e., the relatively horizontal leg of the pipe immediately following the inlet) of the downdrain pipe is sloped at a 2% grade.
- The pipe was assumed to flow full the entire duration when draining the filled basin

Manning's equation gives the pipe flow velocity (m/s) as Equation (1):

$$V = \frac{1.49R^{2/3}i^{1/2}}{n} \tag{1}$$

Where,

i = hydraulic gradient (ft/ft)

n = Manning's roughness coefficient

R = hydraulic radius (ft)

R for a full flowing pipe is given by the following Equation (2):

$$R = \frac{A_w}{P_w} = \frac{D}{4} \quad (2)$$

Where,

D = the inner diameter of the pipe (ft)

A_w = cross-sectional area of flow (ft^2) = $\frac{\pi}{4}D^2$

P_w = perimeter of the flow area = πD

The continuity equation gives the flow rate (ft^3/s) as Equation (3):

$$Q = VA_w$$

Which may be simplified to:

$$Q = 0.464\frac{D^{8/3}i^{1/2}}{n} \quad (3)$$

Table 3. Inputs for Downdrain Pipe Drain Time Calculations

Parameter (Unit)	Value
ID (ft)	1.00
i (-)	0.02
n (for HDPE)	0.012

Table 4. Outputs for Downdrain Pipe Drain Time Calculations

Parameter (Unit)	Value
Q, Flow in pipe (cfs)	5.47

The total amount of time it will take to drain the total volume of accumulated contact water (as estimated in the previous section) can be estimated by dividing the total volume by two times (i.e., there are two downdrains) the flow rate estimated from Equation 3. Based on the values calculated previously, it is estimated that it will take approximately **5.3 hours** to drain the total volume of CCR contact water that would be retained as a result of a 24-hour, 25-year design storm for the initial phase of Basin 2 for a worst case scenario where both underdrains were obstructed prior to the onset of the storm.

APPENDIX D. DITCH CAPACITY CALCULATIONS

D.1 V-SHAPED EAST DITCH CAPACITY CALCULATIONS

This section of the calculation package estimates the maximum elevation that Cell 4 can be filled to before it will be necessary to reconfigure the existent v-shaped eastern ditch. The eastern ditch will accept and transport runoff from the eastern side slope of the landfill. As the height of CCR in Cell 4 increases, the total area of the eastern side slope increases. To estimate the maximum area that the ditch can collect run-off from without overtopping, it is first necessary to evaluate the maximum flow that the ditch can accept without overtopping. Figure 1 presents a general cross-section schematic of the eastern ditch.

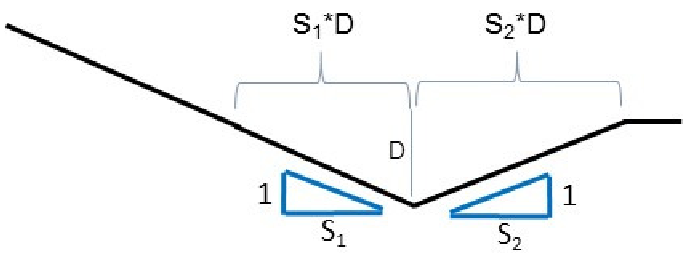

Figure 1. Current East Ditch Cross Section

As presented in the figure,

S_1 = the horizontal distance associated with each foot of vertical rise of the inside (i.e., landfill) slope of the v-shaped ditch (ft)

S_2 = the horizontal distance associated with each foot of vertical rise of the outside slope of the v-shaped ditch (ft)

D = the design liquid depth (ft)

The following additional design assumptions were used:

- The inside slopes of the v-shaped ditch are the same and are sloped at 4 horizontal to 1 vertical (4:1).
- The ditch is longitudinally sloped at 0.2%, towards the dual culvert pipe drain inlet.

Based on these assumptions, Manning's and the continuity equation were used to estimate the maximum flow that the eastern ditch can accept without overtopping. Manning's equation is presented below:

$$V = \frac{1.486}{n} R^{2/3} i^{1/2} \tag{1}$$

Where,

V = velocity (ft/s)

n = Manning's roughness coefficient (0.05 for excavated or dredged channel, channel not maintained, with weeds and brush uncut including dense weeds as high as the flow depth, normal value (Chow 1959))

i = hydraulic gradient, or longitudinal slope of the channel (ft/ft)

R = hydraulic radius (ft),

$$R = \frac{A}{P_W}$$

Where,

A = cross-sectional flow area (ft^2)

P_W = wetted perimeter (ft)

Because both inside slopes of the ditch have the same configuration (i.e., $S_1 = S_2$), $B_1 = B_2$ and

$$A = (S_1 D)$$

$$P_W = 2D(1 + (S_1)^2)^{0.5}$$

To calculate the max flow that the eastern drainage ditch can accept (Q_{max}), equation (2) was used:

$$Q_{max} = A\frac{1.486}{n}R^{2/3}i^{1/2}$$

$$Q_{max} = (S_1 D) * \frac{1.486}{n} * \left(\frac{S_1}{2\sqrt{1+S_1{}^2}}\right)^{2/3} i^{1/2} \qquad (2)$$

Where,

Q_{max} = maximum ditch design flow (ft^3/s)

A summary of the input values used to calculate the capacity of the Eastern Drainage Ditch is presented in Table 1.

Table 1. Inputs Parameters for Existing Eastern Drainage Ditch

Parameter		Value
S, side slope of ditch	Horiz component	4
	Vertical component	1
D, Ditch depth (ft)		1
i, longitudinal slope (ft/ft)		0.002
n, Manning's coefficient		0.05

Based on the calculations presented above, and using the input values presented in Table 1 that are representative of the current geometry of the eastern ditch, it is estimated that the current ditch can accept a maximum flow of:

3.28 ft^3/s

The total area that would contribute this flow rate was back-calculated from the equations presented in Appendix B as:

18,500 ft^2

Excluding the adjacent berm side slope and the adjacent paved access road, the total area that the existing eastern drainage ditch currently collects from is:

18,100 ft^2

Therefore, GRU will need to reconfigure the eastern drainage ditch according to the design presented in the site's closure and post-closure care plan prior to depositing CCR in Cell 4.

D.2 SOUTHWEST DITCH CAPACITY CALCULATIONS

As estimated in Appendix B, the maximum flow (from a 24-hour, 25-year design storm) that must be handled by the west and south ditches is 30.1 cfs and 50.3 cfs, respectively. Since the west ditch flows directly into the south ditch, the south ditch must be able to accommodate the max flow from both west and south contributing areas (i.e., 80.4 cfs). The maximum flow that each ditch can handle can be estimated by combining Manning's Equation and the Continuity Equation (presented previously) into Equation (3):

$$Q_{max} = A * \frac{1.486}{n} (\frac{A}{WP})^{2/3} i^{1/2} \qquad (3)$$

A representative cross section of the layout of the western and southern ditches is presented in Figure 2.

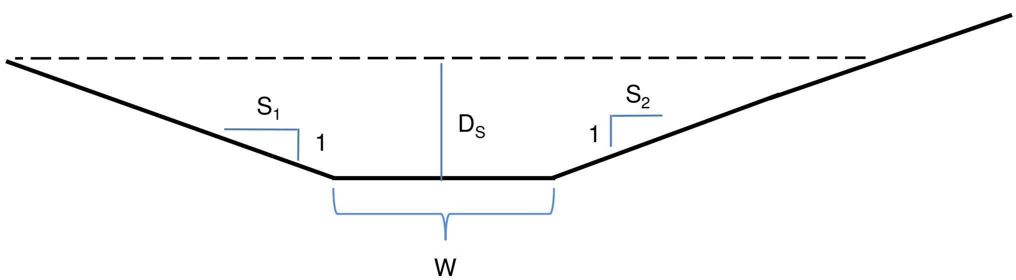

Figure 3. Drainage Ditch Cross Section showing variables in Equation (4) and Equation (5)

To estimate the maximum flow that can be handled by each ditch, it is necessary to calculate their maximum flow cross-sectional area and wetted perimeter, presented in Equations (4) and (5), respectively:

$$A = \frac{1}{2} S_2 (D_S)^2 + D_S W + \frac{1}{2} S_1 (D_S)^2 \qquad (4)$$

$$W_P = \sqrt{(D_S)^2 + (S_2 D_S)^2} + W + \sqrt{(D_S)^2 + (S_1 D_S)^2} \qquad (5)$$

Where,

D_S = the depth of the ditch (ft)

W = the width of the bottom of the ditch (ft)

S_1 = the incremental horizontal distance for each vertical foot of the outside (i.e., with respect to the landfill) slope of the ditch

S_2 = the incremental horizontal distance for each vertical foot of the inside (i.e., with respect to the landfill) slope of the ditch

Please see Figure 3 for a definition sketch of these variables. Table 3 presents a summary of the inputs used in the calculations and the resulting flow depth. A Manning's roughness coefficient of 0.05 was selected as the minimum value of the range presented by Chow (1959) for excavated or dredged channels, channels not maintained, weeds and brush uncut, dense weeds, high as flow depth. The minimum value of the range was selected because it is not anticipated that the entire ditch will be completely filled with vegetation as high as the flow depth.

Table 3. Inputs and Outputs for the West and South Drainage Ditch Capacity Calculations

Inputs		
Parameter	**West**	**South**
n	0.05	0.05
ditch slope	0.002	0.0034
D_s (ft)	3	2.5
S_1 (-)	3	4
S_2 (-)	3	2.6
W (ft)	6	8.2
Outputs		
Flow Capacity (ft^3/s)	88.6	98.1

The 88.6 and 98.1 ft^3/s flow capacity of the western and southern ditches, respectively, exceeds the anticipated 30.1 cfs and 50.3 cfs design flows that would be directed to these features in the event of a 24-hour, 25-year storm. Therefore, the existing design of the southwest ditch (comprised of the connected western and southern ditches) at the landfill is considered acceptable.

D.3 NORTHERN DRAINAGE DITCH CALCULATIONS

The northern drainage ditch follows the same geometry as the ditches shown in Figure 3. The calculations in this section estimate the maximum quantity of run-off that the northern drainage ditch can accept without overtopping. The maximum quantity of runoff that would be directed towards the northern drainage ditch would occur during the initial fill phases of the landfill. Based on a review of the landfill phasing plan and using AutoCAD Civil 3D, the maximum total area that would contribute runoff to the northern drainage ditch is approximately:

883,000 ft^2

The total volume of runoff from this area is estimated by multiplying the depth of runoff (conservatively assumed as 6.6 inches for CCR areas, as calculated in Appendix B) by the total area. This volume is estimated as:

486,000 ft^3

Based on a section and layout drawings included in the as-built construction drawings from B&M (1981) that are included in Appendix A, an apparent length of 1,320 feet, and using that same area calculation presented in Equation 4, the total volume of the northern drainage ditch is estimated as:

333,000 ft^3

Therefore, the expected volume of run-off that will be discharged to the northern drainage ditch exceeds the capacity of the northern drainage ditch by:

153,000 ft^3

A summary of the input values used in the volume calculation is presented in Table 4.

Table 4. Input Values for Northern Drainage Ditch Capacity Estimate

Inputs	
Ditch Length (ft)	1,320
Ds (ft)	7.00
S_1	4
S_2	4
W (ft)	8

The next section evaluates the ability of GRU to pump out the excess run-off collected by the northern drainage ditch into an existing low-lying basin in the Cell 4 area.

D.3 CELL 4 BASIN AREA CAPACITY CALCULATIONS

Using a topographic map from BSMI (2010) (refer to Drawing #6 in Appendix A), the approximate area of the low-lying basin area in Cell 4 is estimated to be 96,000 ft^2. As presented in BSMI (2010), there is at least, on average, 2 feet of headspace from the bottom of the basin to the top of the surrounding berm. With a 2-ft height to the top of the berm, the volume of this basin is:

192,000 ft^3

The capacity of the low-lying basin area in Cell 4 exceeds the excess volume of run-off calculated in the previous section.

Therefore, as long as GRU follows the procedures outlined in the Run-on and Run-off Control System Plan including the use of the low-lying basin in Cell 4 to accept excess run-off from the northern drainage ditch, GRU should be able to manage the total volume of rainfall associated with a 24-hour, 25-year storm without overtopping the northern drainage ditch. In order to pump the excess volume to the Cell 4 basin over the 24-hour storm duration, GRU would need to mobilize and operate pumps with a minimum combined flow rate of about 800 gpm, starting pump operation as soon as possible after the storm begins.

References

B&M (1981). Construction Drawings. Deerhaven Generating Station Unit 2. City of Gainesville/Gainesville-Alachua County Regional Utilities Board. Prepared by Burns & McDonnell, Kansas City, Missouri.

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APPENDIX E. CULVERT PIPE CAPACITY CALCULATIONS

E CULVERT PIPE FLOW CALCULATIONS

Determine the minimum culvert pipe inner diameter necessary to handle the peak stormwater discharge rate from the southern and eastern swales as determined in Appendix B.

Manning's equation gives the pipe flow velocity (m/s) as Equation (1):

$$V = \frac{R^{2/3} i^{1/2}}{n} \tag{1}$$

Where,

i = slope of the pipe (m/m)

n = Manning's roughness coefficient

R = hydraulic radius (m)

R for a full flowing pipe is given by the following Equation (2):

$$R = \frac{A_w}{P_w} = \frac{D}{4} \tag{2}$$

Where,

D = the inner diameter of the pipe (m)

A_w = cross-sectional area of flow (m^2) = $\frac{\pi}{4} D^2$

P_w = perimeter of the flow area = πD

The continuity equation gives the flow rate (m^3/s) as Equation (3):

$$Q = VA_w$$

Or can be solved for pipe flow velocity (m/s) by rearranging terms,

$$V = \frac{4Q}{\pi D^2} \tag{3}$$

Equation (1) can be rearranged to solve for R so that:

$$R = \left(\frac{Vn}{i^{1/2}}\right)^{\frac{3}{2}}$$

From equation (2), D = 4R

Therefore, the necessary inner diameter of a pipe can be found as Equation (4):

$$D = 4\left(\frac{Vn}{i^{1/2}}\right)^{\frac{3}{2}} \tag{4}$$

Substituting equation (3) into (4)

$$D = 4\left(\frac{4Qn}{\pi D^2 i^{1/2}}\right)^{\frac{3}{2}}$$

Or by factoring out D,

$$D = 4^{\frac{1}{4}}\left(\frac{4Qn}{\pi i^{1/2}}\right)^{\frac{3}{8}} \quad (5)$$

Therefore, using the peak flows estimated in Appendix B, the slopes of each of the culverts, and using a Manning's roughness coefficient of 0.012 for smooth-walled plastic pipe (each presented in Table 1 below) the minimum required inner pipe diameter for each of the culvert pipes is presented in Table 2:

Table 1. Inputs for Culvert Pipe Diameter Calculations

Culvert Pipe Location	South	East
Discharge (cfs)	80.4	19.0
Number of Culverts	1	2
Discharge into each Culvert (cfs)	80.4	9.5
n (for HDPE)	0.012	0.012
i (ft/ft)	0.022	0.013

Table 2. Outputs for Culvert Pipe Diameter Calculations

Culvert Pipe Location	South	East
Required ID (inches)	32.3	16.0
Existing/Designed[1] Pipe Size (inches)	36	24

Therefore, as shown in Table 2, the current design of the culverts is considered acceptable to handle the peak run-off associated with a 24-hour, 25-year storm.

[1] Installation of two 24-inch culverts to replace an existing 15-inch CMP is planned for 5 December 2015.