Gainesville Regional Utilities

Deerhaven Generating Station

Coal Combustion Residuals Landfill Run-on and Run-off Control System Plan
(Version 2.0)

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

Prepared by:

Innovative Technical Solutions
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Gainesville, Florida 32606

September 2020

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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 modified according to the design presented in the CCR landfill closure and post-closure care plan before elevating the eastern berm of Cell 4 above its current elevations and depositing CCR in Cell 4 above the current elevations of the eastern berm.

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 22 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 documents that the CCR landfill is 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

The south and west sides of the landfill were regraded in 2019 to expand the capacity of the southern and western stormwater channels to handle run-off from a 24-hour, 25-year storm as the pre-existing channels were not adequately sized to handle run-off from this storm event. The current stormwater channels encompass a large fraction of the south and west side areas. The regraded stormwater run-off channels were seeded to control erosion. An additional culvert was also added on the southeast corner of the landfill. This report presents a revision of the first version of the run-on and run-off control plan prepared in 2016, incorporating the recent changes in the stormwater run-off management system of the landfill.

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 system modifications needs 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 §257.81.

2.0 Run-on and Run-off Management

Run-on and run-off are defined in the CCR rule (§257.53) as:

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

2.1 Exclusion of Run-on

The existing site topography surrounding the landfill precludes the possibility of run-on from the surrounding areas into the landfill. As depicted in drawings 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 would preclude run-on from the north of the landfill. The exterior extents of the modified stormwater channels on the south and the west sides of the landfill appear to be at a slightly higher elevation than the surrounding forested areas on these sides 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 to drain away from the landfill to the east-southeast. These grades around the landfill would preclude run-on from the land areas adjacent to the east, west, or the south of the landfill.

2.2 Run-off Classification and Management

The run-off from the landfill can be classified into CCR contact water and stormwater run-off. Contact water is the precipitation that has come in contact with CCR. Stormwater run-off 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 side and slope of the landfill 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 running off the eastern, western, and southern soil-covered side slopes will be intercepted and routed to the southeast corner of the landfill by stormwater channels located on these sides and eventually discharged to a stormwater pond located to the south of the landfill by means of corrugated high-density polyethylene culverts located on the southeast corners of the landfill. 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. The stormwater run-off from the northern slope will be intercepted by the northern drainage ditch and managed as contact water until stormwater channels are installed on this side of the landfill to divert the stormwater run-off from the northern drainage ditch. The stormwater run-off intercepted by the stormwater channel located north of the northern drainage ditch is routed to a culvert located on the northeast corner for eventual discharge into the forested area on the west of the landfill.

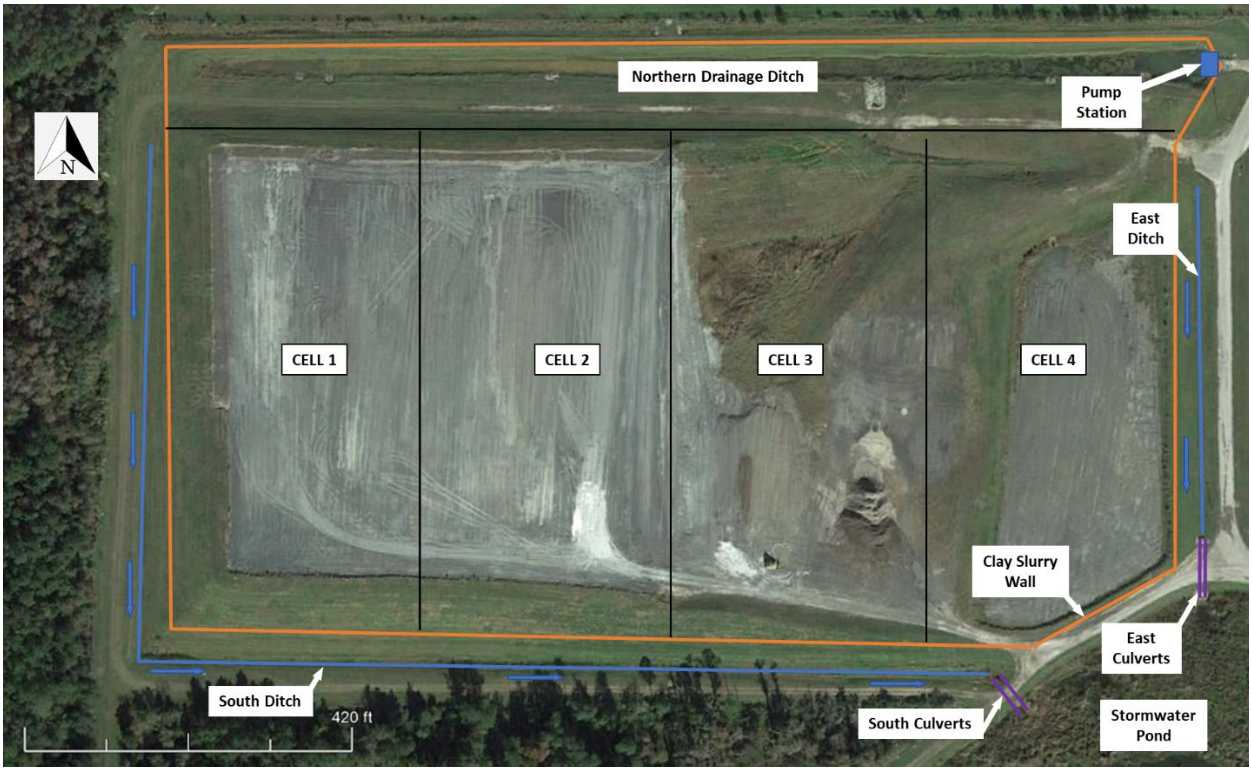


Figure 2-1. CCR Landfill Layout with Run-off Control Infrastructure (image from Google Earth, 12/05/2018)

2.2.1 CCR Contact Water - Downdrains

A conceptual landfill phasing plan was developed as part of the best management practices guidance document (IWCS 2020), which includes information on the size, number, arrangement, and location of downdrain pipes. The phasing plan includes the progressive filling of the landfill with active areas sloped at 2% to provide drainage of the contact water towards the north. Five (12)-inch diameter, high-density polyethylene (HDPE) downdrain pipes will collect the contact water accumulated along the inside edge of the northern peripheral containment berm and route it to the northern drainage ditch; each pipe has an inlet located at the inside toe of the berm. The pipe then protrudes through the containment berm, and runs down the northern side slope below grade, and underneath the unpaved access road (located between the landfill and the northern drainage ditch), and daylights into the northern drainage ditch. Appendix A includes downdrain cross-sections from IWCS (2020).

2.2.1.1 Critical Area

The critical area considered for CCR contact water generation volume and rate used to evaluate the capacity of the downdrain system was the bottom-most active area during initial filling. The best management practices guidance document (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, as presented in Appendix 2.

2.2.1.2 Capacity Evaluation

The capacity of the downdrain system was evaluated by 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 landfill filling areas, 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. It was assumed that the northern drainage ditch will be pumped down to contain minimal liquids before the onset of a 24 hour, 25-year storm event. However, an existing basin located in Cell 4 of the landfill has sufficient capacity to temporarily store 150,000 ft^3 of run-off corresponding to the Cell 4 drainage basin. Procedures for managing the run-off from this area are detailed in Section 3.2.

It should be noted that temporary storage of the run-off within Cell 4 is needed only in the event that GRU does not have the capability to actively pump liquids from the northern drainage ditch to the CCR impoundment during a 24-hour, 25-year storm event. It should also be noted that the temporary storage of liquids in Cell 4 is not a preferred approach for managing the liquids. The contact water volume estimation based on the current area represents the maximum contact water volume that would drain to the northern drainage ditch over the remaining life of the landfill as this area would decrease over time as the landfill expands vertically. As the CCR disposal continues in the landfill, the area of landfill contributing to contact water generation would decrease, and the area of the side slopes increase. Based on the current disposal rate, it is estimated that enough landfill area, which currently has exposed CCR, would become parts of the western and southern exterior slopes to route an additional 79,100 ft^3 of stormwater as run-off instead of as contact water. The northern drainage ditch at that point (representative of filling Phase 15 as shown in Appendix A of IWCS (2020)) would have adequate capacity to contain the contact water from the remaining working face area of the landfill at that point.

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 Channel and Culvert** - located along the western and southern sides of the landfill, this channel collects stormwater from the landfill's western and southern side slopes. The channel terminates at two 36-inch HDPE culvert pipes that discharge to the stormwater pond located to the south of the landfill.
- **Eastern Ditch and Culvert** - located along the eastern side of the landfill, this ditch collects stormwater from the landfill's eastern side slope. The ditch terminates at dual 24-inch HDPE culvert pipes, which discharge to the stormwater pond located to the southeast of the landfill.

2.2.3.1 Critical Areas

The maximum stormwater generation rate for these stormwater ditches/channels would occur when the landfill reaches final grades. Therefore, the final grading plan included in the landfill's closure and post-closure care plan (IWCS 2016) was used to calculate the maximum areas and corresponding maximum discharge rates to each stormwater ditch/channel 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 capacities to handle the stormwater run-off associated with a 24-hour, 25-year storm 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 channel, southeastern culverts, and eastern culvert pipes 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 IWCS (2016) CCR Landfill Closure and Post-Closure Care Plan before depositing CCRs in Cell 4 above the current elevations of the eastern berm.

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 optimal.

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 before the storm event to contain the contact water and stormwater, which would drain to Cell 4, within Cell 4.

4. During the storm event, GRU will monitor the liquid levels in the northern drainage ditch and the Cell 4 basin area 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 the completion of a 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 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. GRU will then pump out all accumulated liquid from Cell 4 to the northern drainage ditch and eventually to the ash ponds of the CCR surface impoundment system.

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 five 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.

IWCS (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 (2020). Best Management Practices Guidance Document for Managing Coal Combustion Residuals at the Deerhaven Generating Station- Version 6.0. September 2020.

8.0 Professional Engineer Certification

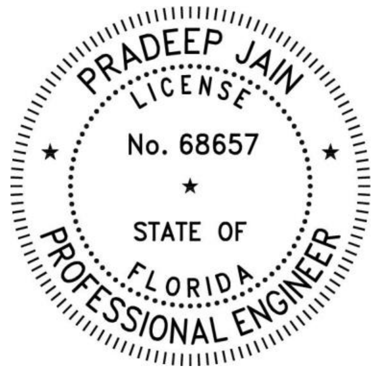
This plan was prepared under the supervision, direction, and control of the undersigned, registered professional engineer (PE). The undersigned PE is familiar with the requirements of 40 CFR 257.81 and certifies that this CCR Landfill Run-on and Run-off Control System Plan meets the requirements of 40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

Name of Professional Engineer: Pradeep Jain

Company: Innovative Waste Consulting Services, LLC

Signature: Pradeep Jain

Digitally signed by Pradeep Jain
Date: 2020.09.23 15:22:06 -04'00'



Date: 09/23/2020

PE Registration State: Florida

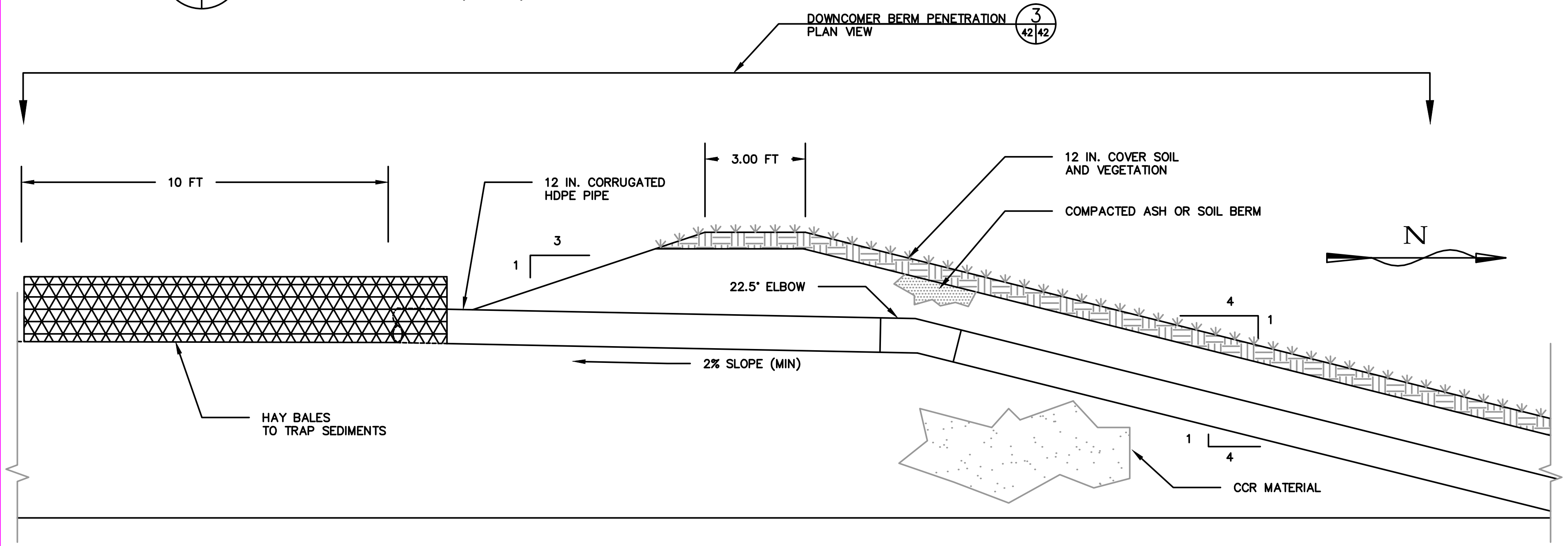
PE License No.: 68657

This item has been digitally signed and sealed by Pradeep Jain, 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.

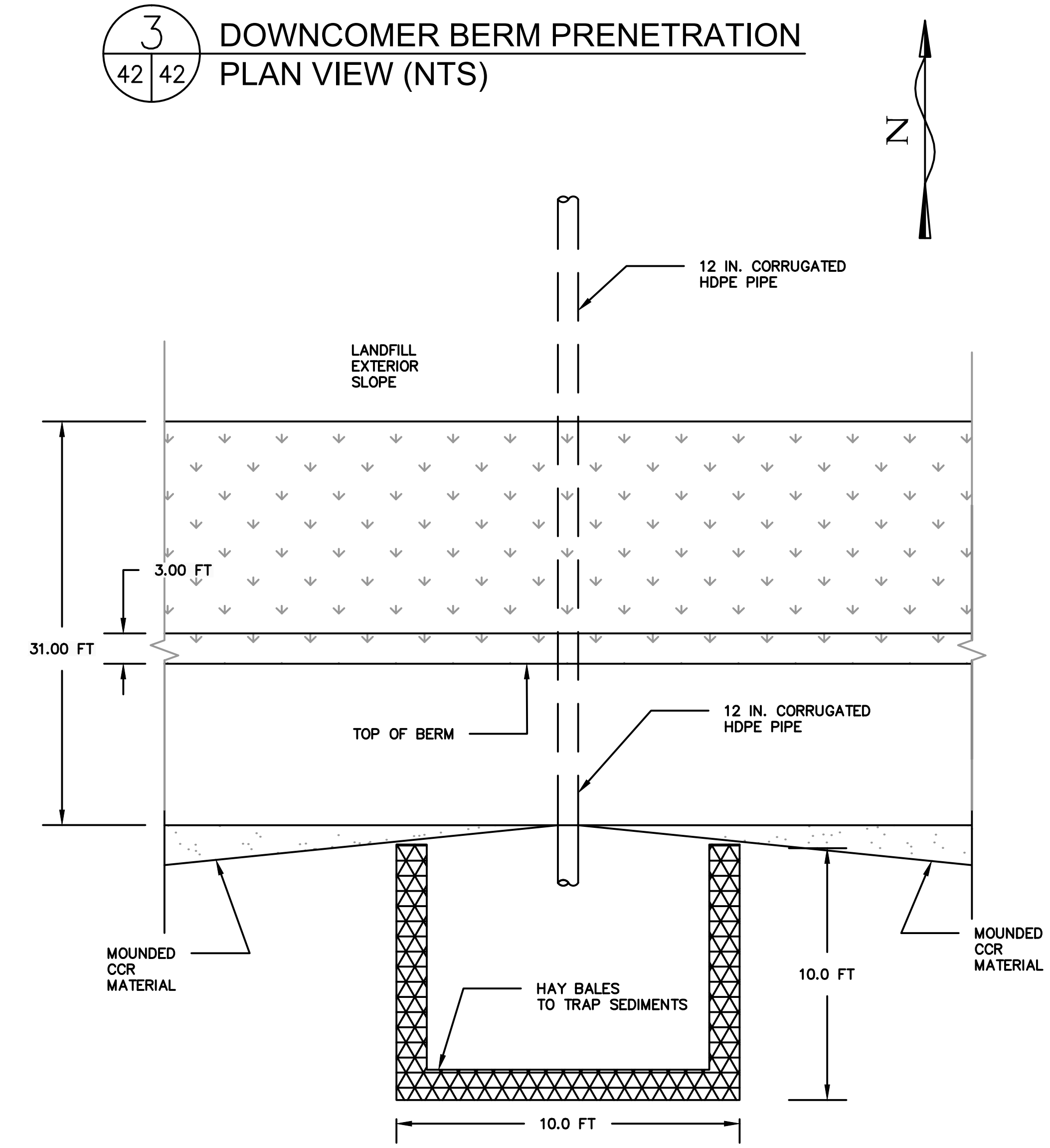
APPENDIX A. PLAN-REFERENCED DRAWINGS

C:\Users\Justin\Innovative Waste Consulting Services\IWCSERVER – Documents\Projects\2017\GRU – CCR LF Phasing Plan Update\Drawings\AutoCAD\IWCS Drawings\20200902 – GRU DGS CCR LF PHASING PLAN.dwg

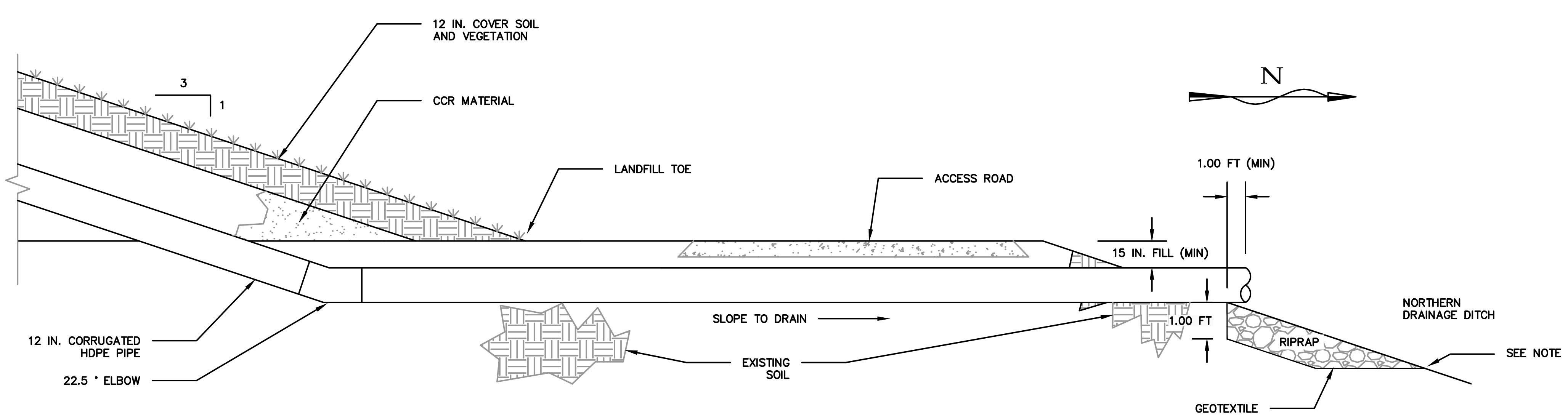
1 / 4 | 42 DOWNCOMER BERM PENETRATION SECTION VIEW (NTS)



3 / 42 | 42 DOWNCOMER BERM PRENETRATION PLAN VIEW (NTS)



2 / 4 | 42 DOWNCOMER ROAD CROSSING SECTION VIEW (NTS)



NOTES:

1. RIPRAP SHALL BE PLACED DOWN TO WATER'S EDGE

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LTR	DATE	REVISIONS	BY	APPRD.
A	9/1/2020	REPLACED SEDIMENT TRAPS WITH HAY BALES	AK	PJ

DESIGNED JS
DRAWN JS, AK
CHECKED PJ

INNOVATIVE WASTE CONSULTING SERVICES, LLC
3720 NW 43rd ST. SUITE 103
GAINESVILLE, FL 32605–4282
PHONE: (352) 331–4828
FAX: (352) 331–4842

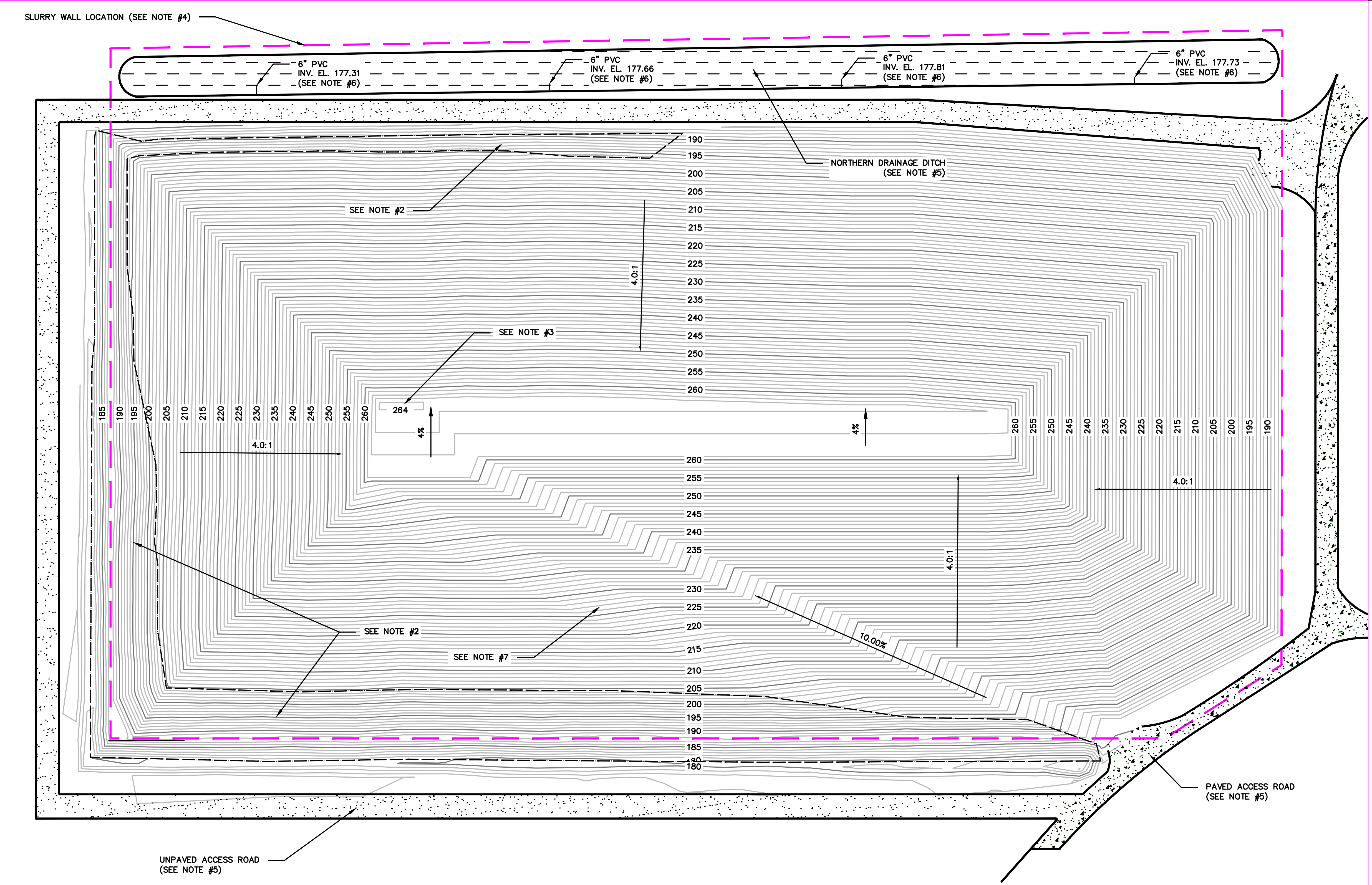
GAINESVILLE REGIONAL UTILITIES
DEERHAVEN GENERATING STATION
1001 NW 13TH ST.,
GAINESVILLE, FL 32653

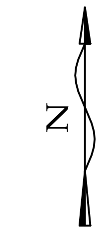
CCR LANDFILL FILLING PLAN

DETAILS 1–3

DATE: AUGUST 2017
SCALE: AS SHOWN
DWG No: 42 OF 45

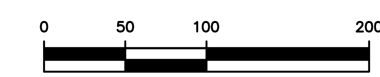
C:\Users\Justin\Innovative Waste Consulting Services\IWCSERVER - Documents\Projects\2017\GRU - CCR LF Phasing Plan Update\Drawings\AutoCAD\IWCS Drawings\20200923 - DGS CCR LF CLOSURE PLAN.dwg





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.



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LTR	DATE	REVISIONS	BY	APPRD.

DESIGNED JS
DRAWN JS
CHECKED PJ

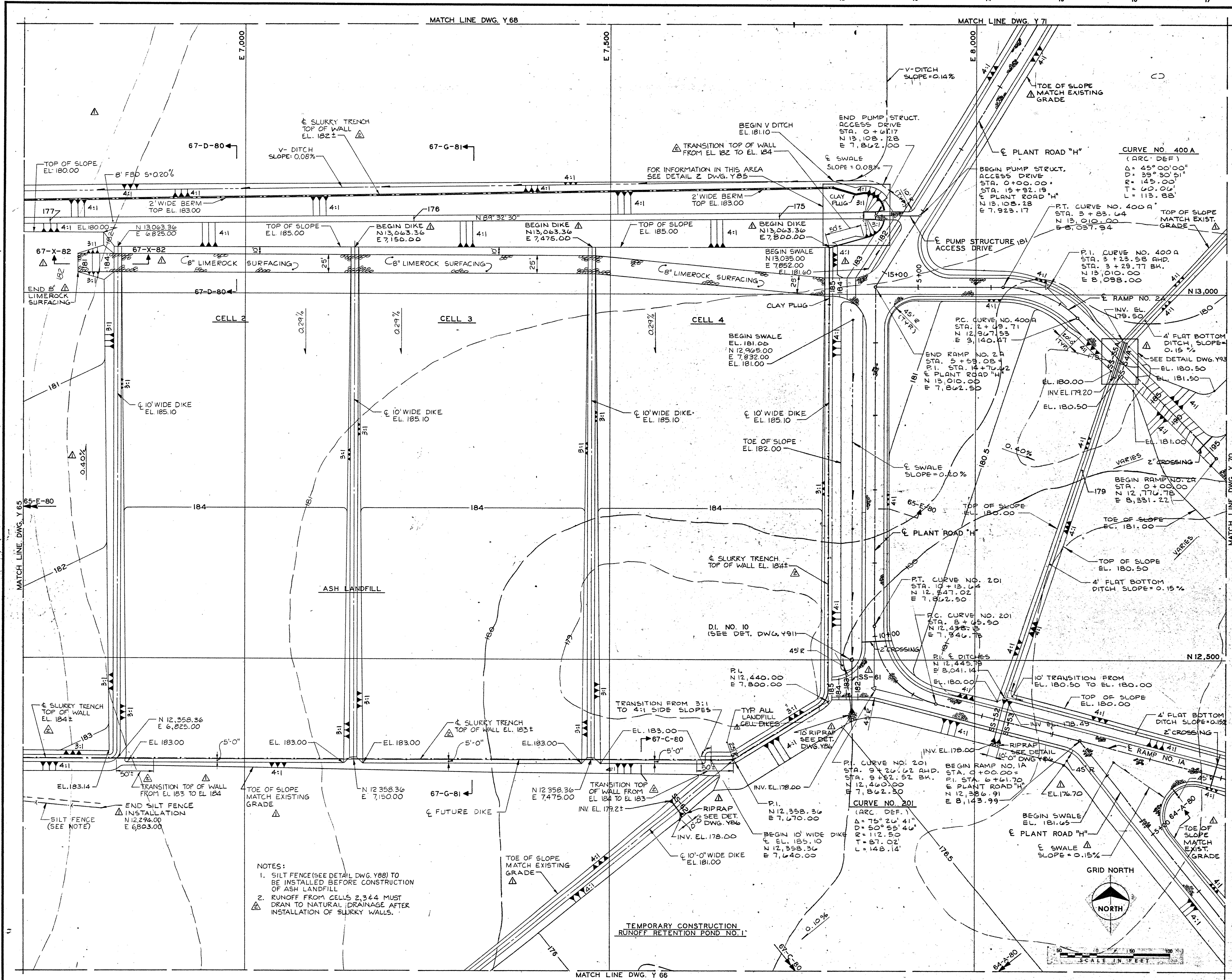
INNOVATIVE WASTE CONSULTING SERVICES, LLC
3720 NW 43rd ST. SUITE 103
GAINESVILLE, FL 32605-4282
PHONE: (352) 331-4828
FAX: (352) 331-4842

GAINESVILLE REGIONAL UTILITIES
DEERHAVEN GENERATING STATION
1001 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



NOTES:
1. SILT FENCE (SEE DETAIL DWG. Y88) TO BE INSTALLED BEFORE CONSTRUCTION OF ASH LANDFILL
2. RUNOFF FROM CELLS 2,3 & 4 MUST DRAIN TO NATURAL DRAINAGE AFTER INSTALLATION OF SLURRY WALLS.

NO.	DATE	BY	REVISION
1	3-10-80	DJB	ADDED AND REVISED NOTES FOR CLARIFICATION. (L-10) (J-12) (J-14) ADDED RIPRAP. (J-12)(J-14) REVISED STORM SEWERS.
	3-21-80	DMZ	ISSUED WITH ADDENDUM NO.3
	4-22-80	DMZ	ISSUED AS BID
2	6-10-80	GWB	ADDED SLURRY TRENCH
	6-24-80	DMZ	ISSUED
3	6-30-81	DJB	CONFORMING TO CONSTRUCTION RECORDS

CONTRACT NO. 29C
YARD STRUCTURES III

DEERHAVEN GENERATING STATION
UNIT 2

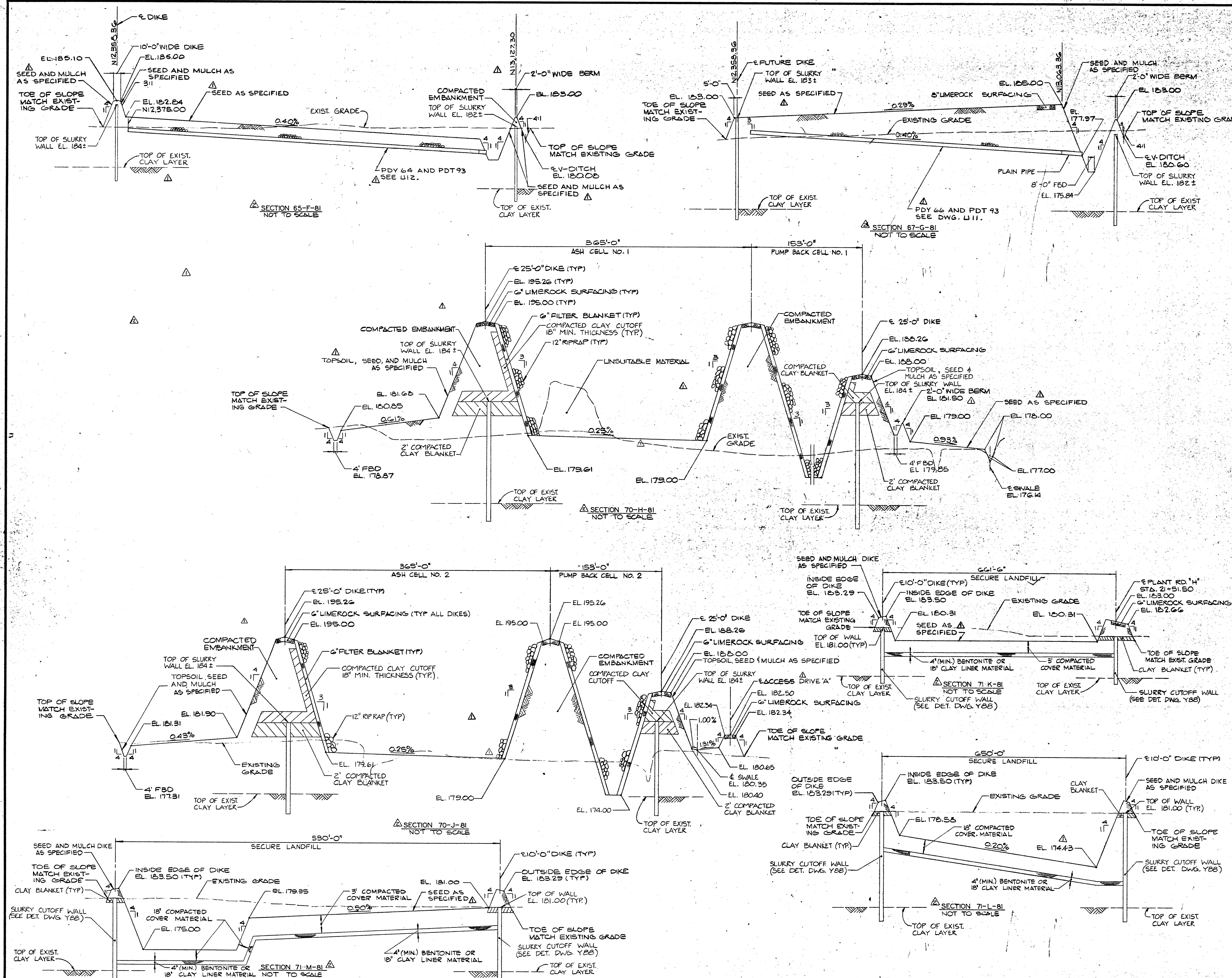
CITY OF GAINESVILLE/
GAINESVILLE-ALACHUA COUNTY
REGIONAL UTILITIES BOARD

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	TPW	(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

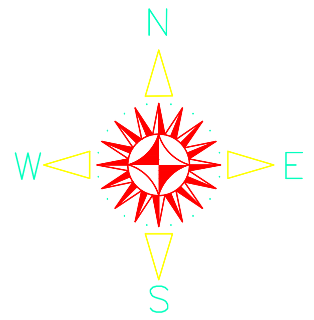
GRADING SECTIONS 2

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

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

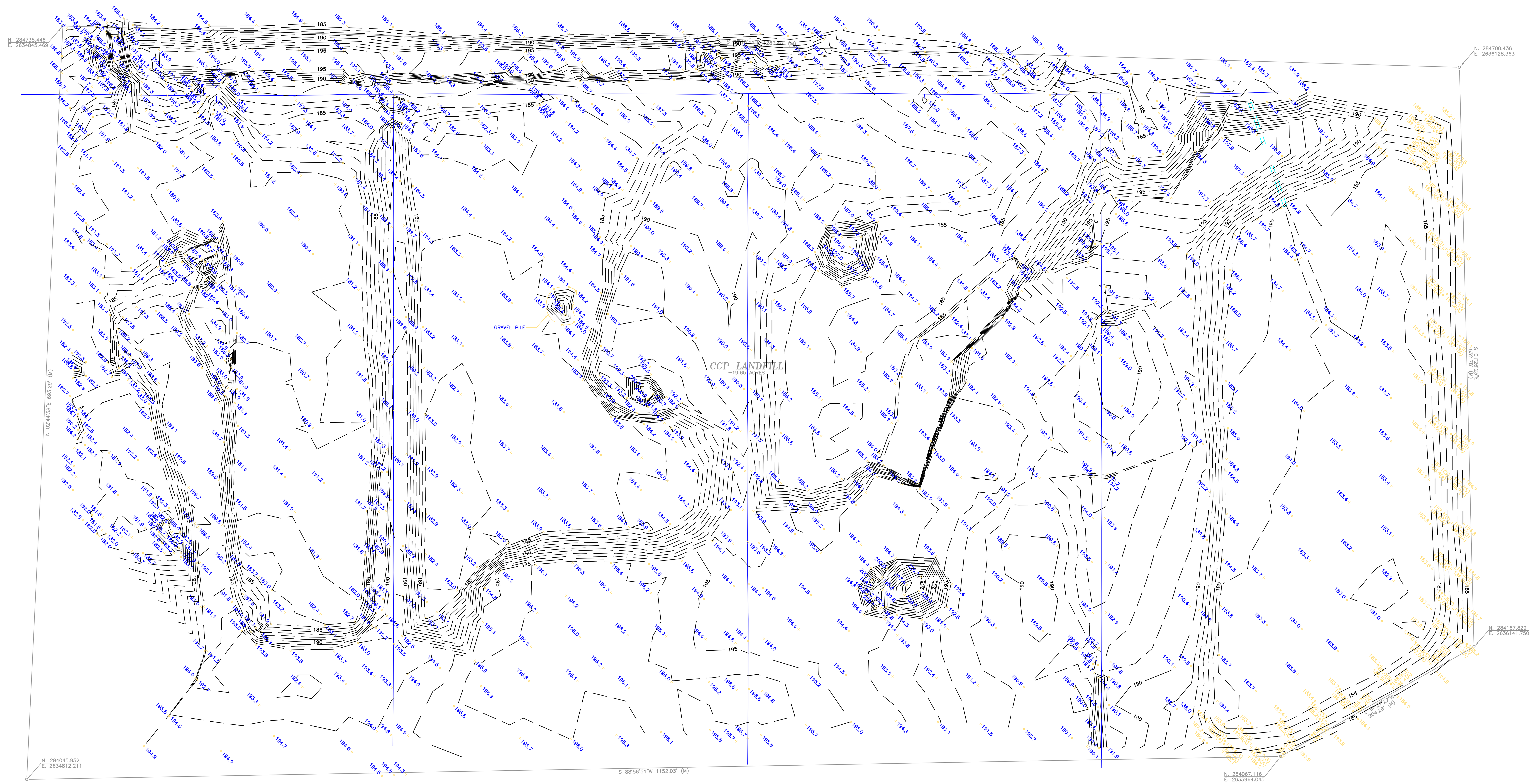
SPECIFIC PURPOSE & TOPOGRAPHIC SURVEY

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	

C:\Users\Justin\Innovative Waste Consulting Services\IWCSERVER - Documents\Projects\2018\GRU - CCR LF Field Services\Drawings\AutoCAD\2019\AutoCAD\IWCS Drawings\2019 CCR LF annual survey volume.dwg

8/21/2020 11:14:54 AM

LTR	DATE	REVISIONS	BY	APPRD.

DESIGNED

DRAWN AK

CHECKED PJ

INNOVATIVE WASTE CONSULTING SERVICES, LLC
3720 NW 43rd St. Suite 103
GAINESVILLE, FL 32605
PHONE: (352) 331-4828
FAX: (352) 331-4842

GAINESVILLE REGIONAL UTILITIES
DEERHAVEN GENERATING STATION

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

GRU CCR LANDFILL TOPOGRAPHY
DECEMBER 2019

DATE: AUGUST 2020

SCALE: AS SHOWN

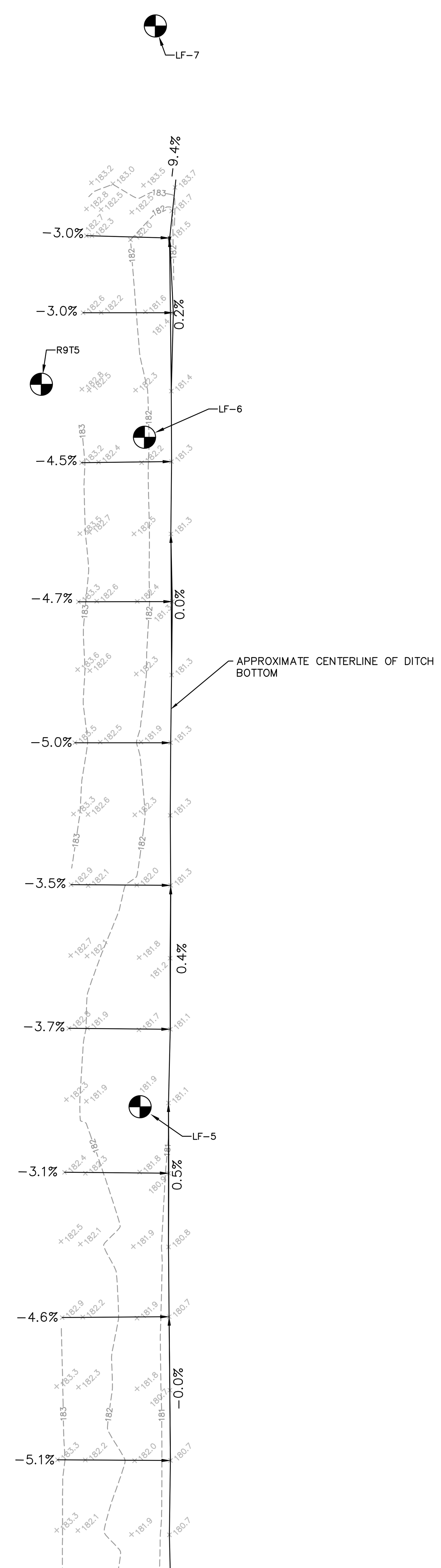
DWG No: OF


N
420 ft
Imagery from Google Earth Pro
Imagery date: 12/5/2018
Area Draining to the Northern Drainage Ditch (832,000 ft²)
Area with Less than 75% Grass Cover (513,000 ft²)
Area Draining to Cell 4 (272,000 ft²)

TOPOGRAPHIC ASBUILT SURVEY

ALACHUA COUNTY, FLORIDA

GRU DEERHAVEN CCR LANDFILL PERIPHERAL STORMWATER CHANNEL



WELL/PIEZOMETER LOCATIONS

WELL/PIEZOMETER DESIGNATIONS	NORTHING (FEET) NAD83 FL NORTH	EASTING (FEET) NAD83 FL NORTH	LATITUDE (DEG°MIN'SEC")	LONGITUDE (DEG°MIN'SEC")	TOP OF CASING ELEVATION NAVD88 (FEET)	NAIL IN CONCRETE FOUNDATION NAVD88 (FEET)
LF-2 (REPLACEMENT)	284008.20	2635888.42	29°45'50.46"	-082°23'47.40"	182.33	179.06
LF-3 (REPLACEMENT)	283992.14	2635456.75	29°45'50.38"	-082°23'52.30"	183.70	180.69
LF-4 (REPLACEMENT)	283987.48	2634913.87	29°45'50.43"	-082°23'58.46"	184.83	182.12
R9T5	284643.14	2634741.61	29°45'56.95"	-082°24'00.27"	NOT PROVIDED	181.51
LF-5	284314.95	2634786.50	29°45'53.70"	-082°23'59.83"	184.33	181.43
LF-6	284619.10	2634788.53	29°45'56.71"	-082°23'59.75"	184.59	181.66
LF-7	284805.95	2634793.43	29°45'58.56"	-082°23'59.65"	185.74	182.88

ABBREVIATIONS

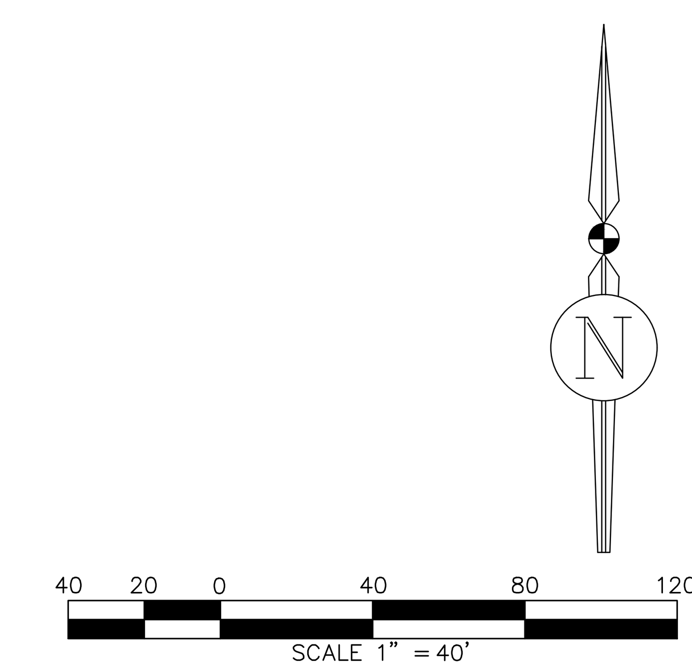
INV = INVERT
ELEV = ELEVATION
JBPRO = JBROWN PROFESSIONAL GROUP
LB = LICENSED SURVEYING BUSINESS
LS = LICENSED SURVEYOR
NL = NAIL& DISK
NAVD88 = NORTH AMERICAN VERTICAL DATUM OF 1988
NGVD29 = NATIONAL GEODETIC VERTICAL DATUM OF 1929
ST = STORM WATER PIPE
IRC = IRON ROD CAPPED

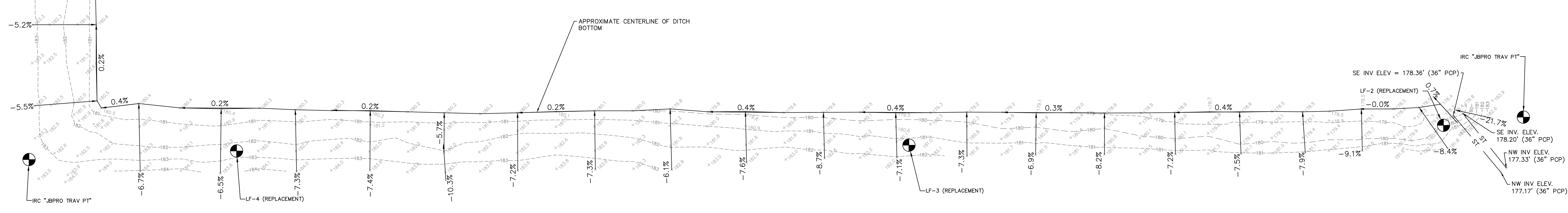
SYMBOL LEGEND

STORM WATER PIPE
BENCHMARK / WELL/PIEZOMETER LOCATION @ THE NAIL
× 132.2 SPOT ELEVATION – SOFT SURFACE
132 CONTOUR LINES

SURVEYOR'S NOTES

1. ELEVATIONS ARE BASED ON THE BENCHMARKS IN THE APPROVED CONSTRUCTION PLANS FOR THIS PROJECT ON NGVD29 DATUM AND ARE DEPICTED ON THIS MAP.
2. THIS SURVEY IS BASED ON MEASUREMENTS CONDUCTED BETWEEN THE 6TH AND 10TH OF MAY 2019 AND 22ND OF JUNE 2020.
3. NO UNDERGROUND UTILITIES WERE LOCATED IN THE COURSE OF THIS SURVEY.
4. ADDITIONAL ENCUMBRANCES MAY AFFECT THE SUBJECT PARCEL THAT DO NOT APPEAR ON THIS MAP.
5. NO EASEMENT RESEARCH WAS CONDUCTED DURING THE COURSE OF THIS SURVEY.
6. THIS IS AN ASBUILT OF FINAL CONSTRUCTION OF THE LOWER BERM AND DITCH AT THE GRU DEERHAVEN CCR LANDFILL AS OF THE DATE ABOVE.
7. ALL WELL/PIEZOMETER ELEVATIONS WERE CONVERTED FROM NGVD29 DATUM USING VERTCON SOFTWARE TO NAVD88 DATUM.
8. THIS IS NOT A BOUNDARY SURVEY
9. MEASURING POINT ELEVATIONS WERE OBTAINED ON THE TOP OF THE WELL/PIEZOMETER CASING AND ARE MARKED WITH A BLACK MARKER.
10. ELEVATIONS OF WELL/PIEZOMETER POINTS ARE BASED ON THE PROJECT BENCHMARKS IN THE APPROVED CONSTRUCTION PLANS BEING ON NGVD29 DATUM. THE RELATIVE ACCURACY OF THE NEW CONTROL POINTS BETWEEN ONE ANOTHER IS 0.01 FEET. THIS ACCURACY IS ONLY GUARANTEED AS LONG AS THERE IS NO DAMAGE TO THE FOUNDATION IN WHICH THE CONTROL POINTS ARE SET.
11. ALL WELL/PIEZOMETER LOCATIONS (NORTHING/EASTING AND LAT/LONG) ARE FOR THE NAIL AND DISK SET IN THE CONCRETE FOUNDATION AT THE BASE OF THE WELL/PIEZOMETER CASING.





TOPOGRAPHIC ASBUILT SURVEY

CERTIFIED TO:
1. COLEMAN CONSTRUCTION
2. GAINESVILLE REGIONAL UTILITIES
3. INNOVATIVE WASTE CONSULTING SERVICES, LLC

Scale:	1"=40'
Proj. No.	800-19-15
Drawn:	R.BOOKER
Checked:	T.Wright
Dwg. Name:	800-19-15-ST1
Dwg. Date:	06/18/2019
Field Book:	N/A
Pages:	N/A
Sheet:	1 of 1

THE MAP OF THE PROPERTY DESCRIBED HEREON WAS MADE UNDER MY SUPERVISION AND THIS MAP OF SURVEY FURTHER MEETS THE STANDARDS OF PRACTICE SET FORTH BY THE STATE OF FLORIDA BOARD OF PROFESSIONAL SURVEYORS & MAPPERS IN CHAPTER 5J-17.05, FLORIDA ADMINISTRATIVE CODE, PURSUANT TO SECTION 472.027, FLORIDA STATUTES, AND THE MAP OF SURVEY SHOWN HEREON IS A TRUE AND ACCURATE REPRESENTATION THEREOF TO THE BEST OF MY KNOWLEDGE, BEING SUBJECT TO NOTES AND NOTATIONS SHOWN HEREON.

Florida License No. LS7210
Certificate of Authorization No. LB8031

NOT VALID WITHOUT THE SIGNATURE AND ORIGINAL RAISED SEAL OF A FLORIDA LICENSED SURVEYOR & MAPPER OR VALID DIGITAL SIGNATURE IN ELECTRONIC FORM

TROY V WRIGHT, PSM
Professional Surveyor and Mapper

Prepared By:
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3530 NW 43rd Street • Gainesville, Florida 32606
PHONE: (352) 375-8999 • FAX: (352) 375-0833
E-MAIL: contact@jbprogroup.com

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-hour, 25-year storm event. 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-hour, 25-year storm was estimated to be 7.27 inches (NOAA 2020).

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 and final 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):

$$q_p = q_u * A_m * Q_r * F_p \tag{1}$$

Where,

q_p = peak discharge (cfs)

q_u = unit peak discharge (csm/in)

A_m = drainage area (mi^2)

Q_r = run-off (in)

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

Run-off, Q_r, was calculated using (USDA 1986) Equation (2):

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

Where,

P = Rainfall (inches)

S = Potential maximum retention after run-off begins (inches)

S can be found by determining the curve number for the run-off area, as presented in the following (USDA 1986) Equation (3):

$$S = \frac{1000}{CN} - 10 \tag{3}$$

Where,

CN = curve number based on site surface soil conditions.

The soil type used for future cover at the site is unknown. Therefore, for 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 run-off from intermediate and final cover soils. Hydrologic soil group D (clay loam, silty clay loam, sandy clay, silty clay, or clay) "newly graded area" was used to represent the CCR working faces of Basin 1 and Basin 2 without grass cover. For other areas, including CCR surfaces with "good condition" grass cover (grass cover >75%), the run-off 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 run-off curve number estimate of 94 for hydrologic soil group D for "newly graded area." 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 (4):

$$I_a = 0.2 \times S \tag{4}$$

Where,

I_a = Initial abstraction, or run-off loss (inches)

The potential maximum retention, initial abstraction, run-off, 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 (inches)	Initial Abstraction (inches)	Run-off (inches)	I_a/P
Cover Soil	D	80	2.5	0.5	4.9	0.07
CCR with more than 75% Grass Cover	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 (2020), 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

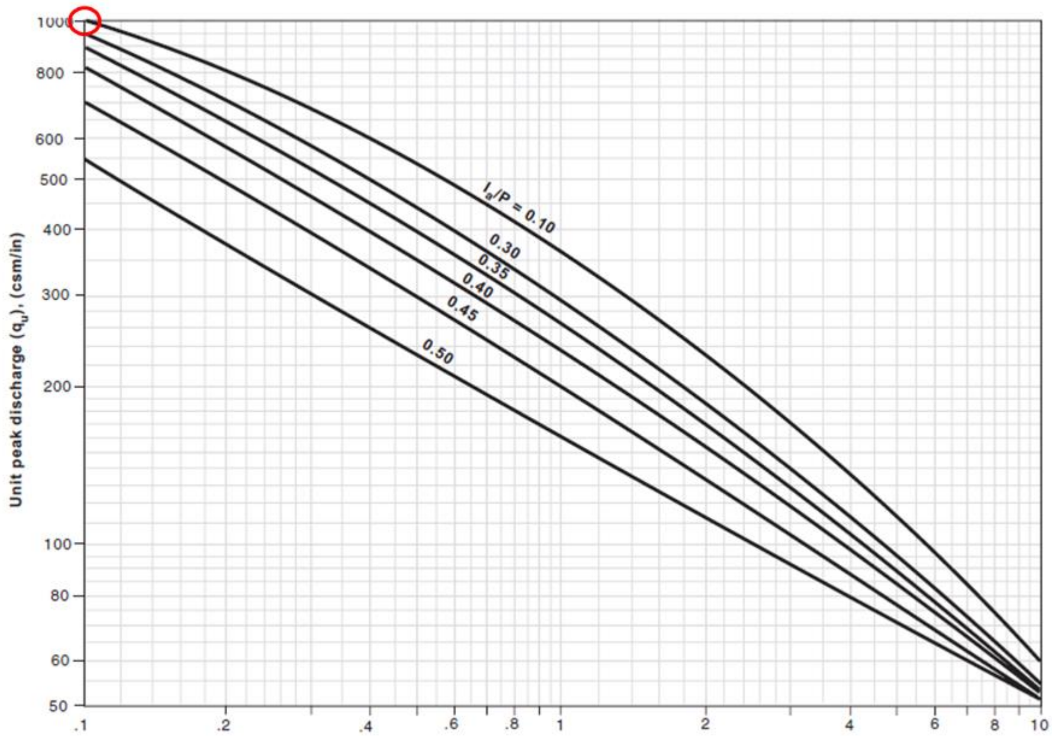

Table 2 presents the area of Basins 1 and 2 and the corresponding estimated peak discharge rates. Tables 3 and 4 present the estimated peak discharge rates for the swales and road-crossing culverts, respectively. It is important to note that the contributing areas outlined in the tables include the area of the swales themselves. The areas presented in the table were evaluated using AutoCAD Civil 3D 2013 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 Drainage 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	0.00610	1000	30.1
South (Final Grades)	284,000	0.01019	1000	50.4
East (Final Grades)	107,000	0.00384	1000	19.0
East (@ 190 ft elevation)	25,900	0.00093	1000	4.6
Northern Drainage Ditch-Based on December 2019 Topographic Conditions	832,000			

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)	454,000	0.01629	1000	80.5
East (Final Grades)	107,000	0.00384	1000	19.0

References

NOAA (2020). NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: FL. http://bit.ly/1Lji8tK accessed August 14, 2020. Data found for site coordinates using the National Oceanic and Atmospheric Administration National Weather Service Hydrometeorological Design Studies Center Precipitation Frequency Data Server.

USDA (1986). Urban Hydrology for Small Watersheds. Technical Release – 55. Published by the United States Department of Agriculture Natural Resources Conservation Service and Conservation Engineering Division, June 1986.

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 and 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 has the capacity to 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.

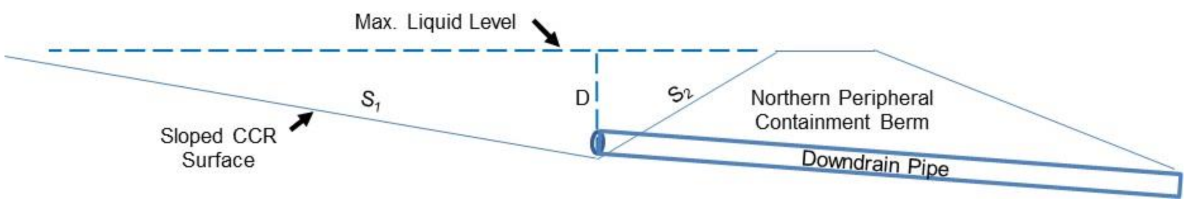

Figure 1. Basin 2 Downdrain Area Cross Section

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 (inches)	7.27
S_1, slope of the working face	0.02
S_2, slope of the 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 (ft^3)	Basin Capacity (ft^3)
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 is 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 during the storm event. 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):

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

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:

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

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, Pipe flow rate (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 run-off 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. In order 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.

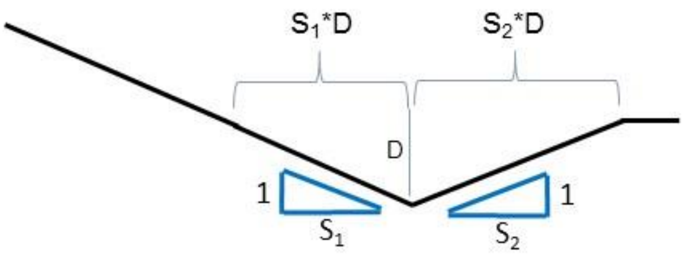

Figure 1. Current East Ditch Cross Section

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:

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

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., $S_1 = S_2$), and

$$A = S_1 D^2$$

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

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

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

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	Horizontal 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 convey stormwater at a maximum flow of:

3.29 ft^3/s

The total area that would contribute to 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 is:

18,100 ft^2

The eastern ditch is adequately sized to handle stormwater run-off from th current drainage basin area that contributes run-off to this ditch. However, GRU will need to expand the capacity of 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 above the current level of the berm.

D.2 SOUTHERN AND WESTERN DITCH CAPACITY CALCULATIONS

As estimated in Appendix B, the run-off from a 24-hour, 25-year design storm event that must be handled by the west and south ditches is 30.1 cfs and 50.4 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., 80.5 cfs). The maximum flow rate that the ditch can handle was estimated using Manning's and the continuity equations to assess whether the ditch is adequately sized to convey the stormwater run-off associated with a 24-hour, 25-year storm. A representative cross-section of the layout of the western and southern ditches is presented in Figure 2

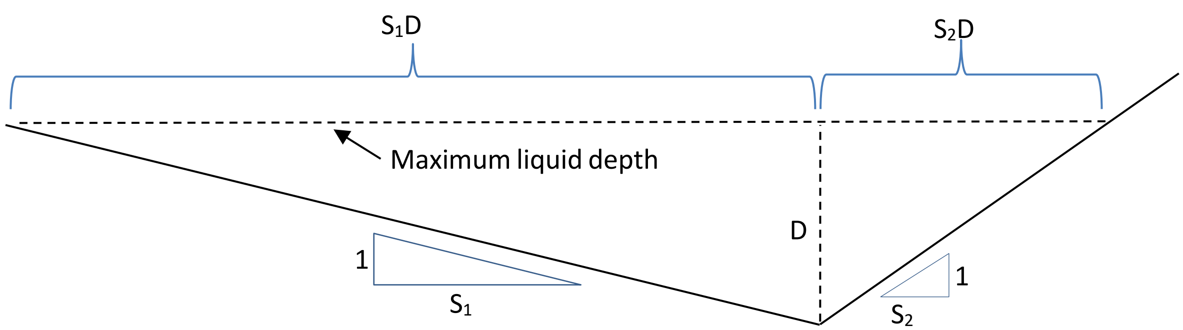

Figure 2. Drainage Ditch Cross Section showing variables in Equation (3) and Equation (4)

The ditch dimensions and longitudinal slope used to estimate the capacity of the south ditch were estimated from the as-built drawing (Appendix A). The estimated dimensions are presented as follows:

i. The south stormwater ditch is a v-shaped channel with slope of 4:1 (H:V) (S_2:1 as shown in Figure 2) on the landfill side and a slope of 12:1 (H:V) (S_1:1 as shown in Figure 2) on the south side.

ii. The average longitudinal slope of the south ditch is approximately 0.0018 (V:H) (0.18%).

iii. The minimum depth for the west and the south ditch was observed to be at the southeast corner and is approximately 2.6 ft.

Based on the above, Manning's and the continuity equation were used to estimate the maximum flow rate that the southern ditch can handle. Manning's equation is presented below:

$$V = \frac{1.486}{n} R^{2/3} s^{1/2} \qquad (3)$$

Where,

V = velocity (ft/s)

n = Manning's roughness coefficient

s = longitudinal slope of channel (ft/ft)

R = hydraulic radius (ft),

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

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

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

Where,

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

W_P = wetted perimeter (ft)

D = the design liquid depth (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

To calculate the max flow that the critical cross-section could handle, equation (6) was used

$$Q_{max} = A * \frac{1.486}{n} R^{2/3} s^{1/2} \qquad (6)$$

Where,

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

Table 3 presents a summary of the inputs used in the calculations and the resulting flow depth. A Manning's roughness coefficient of 0.033 was selected as the maximum value of the range presented by Chow (1959) for excavated or dredged channels (short straight grass and few weeds).

Table 2. Inputs for Additional V-Shaped Ditch Capacity Calculations

S_1, side slope of ditch	Horizontal component	4
	Vertical component	1
S_2, side slope of ditch	Horizontal component	12
	Vertical component	1
D, Ditch depth (ft)		2.6
s, longitudinal slope (ft/ft)		0.0018
n, manning's coefficient		0.033

Table 3. Outputs for Additional V-Shaped Ditch Capacity Calculations

P, wetted perimeter	42.03
A, Flow Area (ft^2)	54.08
R, hydraulic radius	1.29
Vmax, max velocity	2.27
Q_{max} (cfs)	122.84

The maximum flow capacity of the ditch (122.8 ft^3/s) exceeds the anticipated 80.5 ft^3/s peak run-off rate associated with 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 estimated to be adequately sized.

D.3 NORTHERN DRAINAGE DITCH CALCULATIONS

The calculations in this section estimate the maximum quantity of run-off that the northern drainage ditch can accept without overtopping. The northern drainage ditch is a trapezoidal-shaped ditch. Based on a section and layout drawings included in the as-built construction drawings from B&M (1981) that are included in Appendix A, the ditch bottom is 8 ft wide, side slopes are 4:1 (horizontal:vertical), and the depth is 7 ft. The average cross-sectional area of the ditch is approximately 252 ft^2. The ditch length is approximately 1320 ft. The storage capacity of the ditch is estimated to be 332,600 ft^3.

The maximum quantity of run-off that would be directed towards the northern drainage ditch was based on the current area of the landfill, excluding the southern, western and eastern side slopes; the run-off from these slopes would be routed to stormwater ditches and these sides and eventually to the stormwater pond located on the south side of the landfill.

Based on the topographic conditions of the landfill in December 2019, the total area that would contribute run-off to the northern drainage ditch is approximately 832,000 ft^2. Based on the most recent google imagery available (Imagery date 12/5/2018), approximately 513,000 ft^2 of this area appears to be without any grass cover, and the balance 319,000 ft^2appears to have more than 75% grass cover. The total volume of run-off from these areas is estimated by multiplying the depth of run-off (4.9 inches for landfill areas with 75% grass cover and 6.6 inches for areas without any grass cover as calculated in Appendix B). The total run-off volume is estimated to be approximately 411,700ft^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 **79,100 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
Ditch Depth (ft)	7.00
Side Slope	4:1
Width of the bottom (ft)	8

The next section evaluates the ability to temporarily retain the contact water and stormwater from the Cell 4 drainage basin within the Cell 4 area until the pumping of water accumulated in the northern drainage ditch to the CCR impoundment system is initiated to create the capacity to accept more liquids.

D.3 CELL 4 BASIN AREA CAPACITY CALCULATIONS

Based on the topographic conditions in December 2019, contact water/stormwater from approximately 272,000 ft^2 area of the landfill would run off to the Cell 4 area. Based on the estimated run-off of 4.9 to 6.6 inches, approximately 111,000 to 150,000 ft^3 of the liquids volume associated with a 24-hour, 25-year storm would run-off to Cell 4. The approximate area of the low-lying basin area in Cell 4 is estimated to be 90,000 ft^2. The average liquid depth associated with the run-off volume is estimated to range from 1.23 to 1.67 ft. Using a topographic map from BSMI (2010) (refer to Drawing #6 in Appendix A), the minimum elevation of the top of the Cell 4 peripheral berm is 188.5 ft, and the topographic survey conducted in December 2019 suggests that the maximum elevation of CCRs in Cell 4 is 186 ft. Therefore, there is at least 2 feet of headspace from the CCR elevation to the top of the surrounding Cell 4 berm. The estimate suggests that the contact water and stormwater run-off draining to Cell 4 can be temporarily contained within the cell footprint during the design storm event.

The liquids storage 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 temporarily store the run-off, GRU should be able to manage the total volume of run-off associated with a 24-hour, 25-year storm without overtopping the northern drainage ditch.

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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 ditches, as determined in Appendix B.

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

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

Where,

i = slope of the pipe (ft/ft)

n = Manning's roughness coefficient

R = hydraulic radius (ft)

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

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

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):

$$Q = V A_w$$

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:

$$R = \left(\frac{Vn}{1.486\, i^{1/2}}\right)^{\frac{3}{2}}$$

From equation (2), D = 4R

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

$$D = 4\left(\frac{Vn}{1.486\, i^{1/2}}\right)^{\frac{3}{2}} \tag{4}$$

Substituting equation (3) into (4)

$$D = 4\left(\frac{4Qn}{1.486\,\pi D^2 i^{1/2}}\right)^{\frac{3}{2}}$$

Or by factoring out D,

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

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

	South		
Culvert Pipe Location	**P-1**	**P-1a**	**East**
Discharge (cfs)	80.5		19.0
Number of Culverts	1	1	2
Discharge into each Culvert (cfs)	40.25	40.25	9.5
n (for HDPE)	0.012	0.012	0.012
i (ft/ft)	0.0145	0.0198	0.013

Table 2. Outputs for Culvert Pipe Diameter Calculations

	South		East
Culvert Pipe Location	**P-1**	**P-1a**	
Required ID (inches)	27.0	25.4	16.0
Existing/Designed Pipe Size (inches)	36	36	24

Therefore, as shown in Table 2, the current culverts are adequate to handle the peak run-off associated with a 24-hour, 25-year storm event.