Hazen and Sawyer
4011 WestChase Blvd, Ste 500
Raleigh, NC 27607 • 919.833.7152

Eastgate Park Dam
Preliminary Engineering Report

Final Report
Hazen #32511-005
October 31, 2023

NORTH CAROLINA PROFESSIONAL SEAL 040733 ENGINEER WILLIAM D. PRICE

NORTH CAROLINA PROFESSIONAL SEAL 14110 ENGINEER EVERETTE HAWKS KNIGHT

Table of Contents

Executive Summary 1
1. Project Background 1
2. Condition Assessment 5
2.1 Dam Embankment 5
2.2 Spillways 7
2.2.1 Principal Spillway 7
2.2.2 South Auxiliary Overflow Spillway 10
2.2.3 North Auxiliary Overflow Spillway 11
2.2.4 North Auxiliary Pipe Spillway 12
2.3 Downstream of Dam 13
3. Hydrologic and Hydraulic Analyses 16
3.1 Approach 16
3.2 Hydrologic Analyses 16
3.2.1 Precipitation 16
3.2.2 Hydrologic Characterization 17
3.2.3 Hydrologic Response 20
3.3 Hydraulic Analyses 21
3.3.1 Model Development 21
3.3.2 Hydraulic Performance 22
3.3.3 Dam Breach Inundation Hazards 22
4. Geotechnical Investigation 24
4.1 Field and Laboratory Testing 24
4.2 Stability Analysis 24
5. Alternatives Analysis 25
5.1 Hydraulic Capacity Improvement 25
5.2 Overtopping Protection 28
5.3 Embankment Stabilization 29
6. Conclusions and Recommendations 31

List of Tables

Table 1: Updated Dam Inventory Data 4
Table 2: Hydrologic Parameters 18
Table 3: Hydrologic Response 20

List of Figures

Figure 1: Vicinity Map 2
Figure 2: Site Map 3
Figure 3: Principal Spillway (Excerpt from 1974 Drawing #4) 8
Figure 4: North Auxiliary Pipe Spillway (1990 Drawing #2 Excerpt) 12
Figure 5: Rainfall Distributions 17
Figure 6: Drainage Map 19
Figure 7: Hydrologic Response 20
Figure 8: Piped Spillway Rating Curves 21
Figure 9: Hydraulic Performance 23
Figure 10: Schematic Views of Riprap-Lined Plunge Pool Basin (HEC-14, FHWA, USDOT) .. 26
Figure 11: Schematic Views of Contra Costa Basin (HEC-14, FHWA, USDOT) 27

List of Pictures

Picture 1: Upstream Face of Dam 6
Picture 2: Downstream Face of Dam 6
Picture 3: Top of Dam 7
Picture 4: Principal Spillway Inlet Structure 8
Picture 5: Principal Spillway Outlet Structure 9
Picture 6: Upstream Segment of Outlet Channel 9
Picture 7: Downstream Segment of Outlet Channel and Quail Hollow Drive Culvert 10
Picture 8: Seepage Emanating from Below Outlet Channel Concrete Lining 10
Picture 9: South Auxiliary Overflow Spillway 11
Picture 10: North Auxiliary Overflow Spillway 11
Picture 11: North Auxiliary Pipe Spillway Inlet Structure 12
Picture 12: Area Downstream of Dam (At Principal Spillway Outlet) 13
Picture 13: Area Downstream of Dam (Near Quail Hollow Drive Culvert) 14
Picture 14: Area Downstream of Dam (North of Near Quail Hollow Drive Culvert) 14
Picture 15: Area Downstream of Dam (South of Near Quail Hollow Drive Culvert) 14
Picture 16: Upstream End of Quail Hollow Drive Culvert 15
Picture 17: Downstream End of Quail Hollow Drive Culvert 15
Picture 18: Installation of Overtopping Protection at Lake Johnson Dam 28
Picture 19: Grass Cover and Asphalt Pedestrian Path at Lake Johnson Dam 29

List of Appendices

Appendix A: Reference Documents
Appendix B: Condition Assessment
Appendix C: Hydrologic Analysis
Appendix D: Hydraulic Analysis
Appendix E: Geotechnical Investigation
Appendix F: Conceptual Grading Plan

List of Abbreviations and Acronyms

Abbreviation	Definition	Abbreviation	Definition
1D	One Dimensional	NAD83	North American Datum of 1983
2D	Two Dimensional	NAVD88	North American Vertical Datum of 1988
AC	Acre	NC	North Carolina
ACB	Articulated Concrete Block	NCAC	North Carolina Administrative Code
CADD	Computed Aided Design Drawing	NCDS	North Carolina Dam Safety
CFS	Cubic Feet per Second	NCEM	North Carolina Emergency Management
DBIZ	Dam Breach Inundation Zone	NCFMP	North Carolina Floodplain Mapping Program
DEM	Digital Elevation Model	NEH	National Engineering Handbook
EAP	Emergency Action Plan	NLCD	National Land Cover Dataset
FIS	Flood Insurance Study	NOAA	National Oceanic and Atmospheric Administration
FPS	Feet Per Second	NRCS	Natural Resources Conservation Service
FS	Factor of Safety	PER	Preliminary Engineering Report
FT	Feet	PMP	Probable Maximum Precipitation
GIS	Geographical Information System	PRCR	Parks, Recreation, and Cultural Resources
H&H	Hydrology and Hydraulics	PWR	Partially Weathered Rock
HEC	Hydrologic Engineering Center	RAS	River Analysis System
HMGP	Hazard Mitigation Grant Program	RCN	Runoff Curve Number
HMR	Hydrometeorological Report	SDB	Sunny Day Breach
HMS	Hydrologic Modeling System	SDF	Spillway Design Flood
HR	Hour	SPT	Standard Penetration Test
HSG	Hydrologic Soil Group	TC	Time of Concentration
IN	Inch	US	United States
LLO	Low Level Outlet	USACE	United States Army Corps of Engineers
MI	Mile	USCS	Unified Soil Classification System
MIN	Minute		

Executive Summary

Eastgate Park, opened in 1972, is a 25-acre public facility with numerous amenities, including a neighborhood center, playground, picnic shelters, athletic fields and courts, as well as a 2.4-acre pond (Cooper's Pond) impounded by Eastgate Park Dam. The Park is located at 4200 Quail Hollow Drive within the North Hills area of Raleigh. Substantial improvements were made to the Dam in the 1970s involving an extension of the embankment to the south and construction of a new principal spillway. Three auxiliary spillways were improved in the 1990s, including armoring of the overflows sections and replacement of the pipe spillway system through the embankment near the northern abutment. Given these improvements, the Dam is approximately 455-feet long and 20-feet high with a maximum impoundment volume of 30-acre feet. The facility is currently regulated by North Carolina Dam Safety (NCDS) as a small, Class C (High Hazard) dam per Dam Safety Code 15A NCAC 02K (Code).

The City of Raleigh Stormwater Services (City) contracted Hazen and Sawyer (Hazen) to provide an evaluation of Eastgate Park Dam that included a condition assessment, hydrologic and hydraulic (H&H) analyses, and a geotechnical investigation. Overall, the Dam is in unsatisfactory condition with several adverse conditions noted, including active and potential erosion concerns, embankment stability hazards due to trees, and active seepage downstream of the outlet works. The downstream slope is relatively steep at a grade of 2H:1V. The top of dam is lined with concrete slabs that have embedded area inlets, small drains that discharge into the pond, and the presence of large diameter trees. The top of dam also exhibits signs of downward displacement, multiple fractures, and open joints. Auxiliary spillway sections or portions thereof appear to be partially located on embankment fill.

The analytical studies included both a geotechnical and hydraulic program. For the geotechnical investigation, a total of eight borings were performed: four standard penetration test borings along the top of dam and four hand augers in the embankment and downstream of the dam. The geotechnical investigation determined a low Factor of Safety (FS) of approximately 1.17; significantly less than the required FS of 1.5 by NCDS.

The H&H analysis included a detailed assessment of hydrology (watershed area, surface cover, and timing), field surveys of the normal pool area to verify the stage-storage relationship, and measurements to verify outlet works. Results of the H & H analysis found that overtopping of the dam was as much as 0.8-feet, significantly higher than presented in the previous NCDS EAP. In summary, overtopping at the dam abutments is predicted in the 25-year event, and approximately 800-cfs of additional spillway capacity is required to safely pass the Spillway Design Flood (SDF) in accordance with NCDS regulations.

The evaluation of Eastgate Park Dam identified several maintenance and repair needs, and most importantly, regulatory deficiencies with respect to slope stability and spillway capacity. Our recommendation is to flatten the downstream slope and provide for overtopping protection of the dam utilizing an open cell articulated block system. This alternative provides the means to make the necessary embankment improvements while also providing sufficient hydraulic capacity to pass the required NCDS SDF. In summary, this approach is not only the most economical approach but will have the least amount of temporary and permanent impacts to the Eastgate Park.

1. Project Background

Eastgate Park, opened in 1972, is a 25-acre public facility with numerous amenities, including a neighborhood center, playground, picnic shelters, athletic fields and courts, as well as a 2.4-acre pond (Cooper's Pond) impounded by Eastgate Park Dam. The Park is located at 4200 Quail Hollow Drive within the North Hills area of Raleigh (**Figure 1**). The contributing drainage area is largely residential, though the western and northern extents of the watershed contain limited commercial and business zones. The Pond and Dam are situated at the northern end of the Park along Quail Hollow Drive.

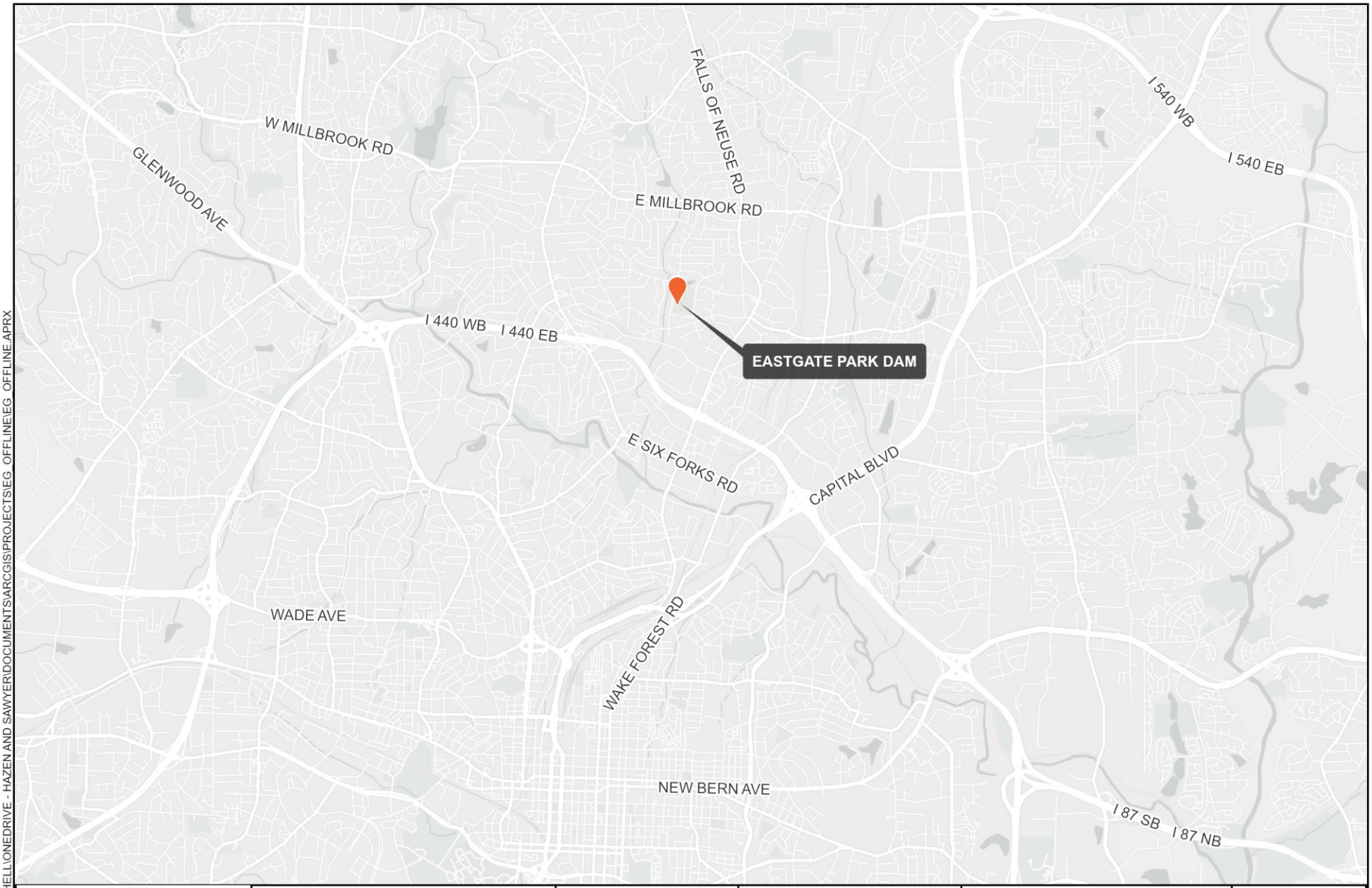
The Dam consists of five primary components: an earthen embankment, a principal spillway, and three auxiliary spillways (two overflow sections on the top of dam and one piped outlet) (**Figure 2**). The northern portion of the embankment is aligned from northwest to southeast and the southern portion is aligned towards the south. The dam ties into natural high ground on each end. The principal spillway inlet is located within the Pond, closer to the southern abutment. The spillway barrel passes through the embankment to the downstream toe where an outlet box discharges flow into a riprap-lined receiving channel. Two auxiliary overflow spillways are located on top of the dam, near the northern and southern abutment contacts. Both are relatively shallow depressions and discharge directly onto the downstream face or downstream groin. The third auxiliary spillway is also located near the northern abutment and consists of a small weir inside the Pond and a discharge pipe system that is routed downstream of the dam to the receiving channel.

Substantial improvements were made to the Dam in the 1970s involving an extension of the embankment to the south and construction of a new principal spillway.[1] The three auxiliary spillways were improved in the 1990s, including armoring of the overflows sections and replacement of the pipe spillway system through the embankment near the northern abutment.[2] Given these improvements, the Dam is approximately 455-feet long and 20-feet high with a maximum impoundment volume of 30-acre feet. The facility is currently regulated by North Carolina Dam Safety (NCDS) as a *small*, *Class C* (*High Hazard*) dam per Dam Safety Code 15A NCAC 02K (Code). Per the Code, the Dam must be capable of safely passing the spillway design flood (SDF) generated from the 1/3 probable maximum precipitation (PMP). Additional information regarding the facility is shown in the dam inventory data, provided as **Table 1**, which has been updated based upon data collection and evaluations performed under this study.

The City of Raleigh Stormwater Services (City) contracted Hazen and Sawyer (Hazen) to provide an evaluation of Eastgate Park Dam, including: a condition assessment, hydrologic and hydraulic analyses, and a geotechnical investigation. The following Preliminary Engineering Report (PER) summarizes the findings of this evaluation.

[1] Improvements as shown on drawings from: *Eastgate Park Phase 1 Development*, Geoffrey McLean, 1974 (See Reference Documents, **Appendix A**)

[2] Improvements as shown on drawings from: *Eastgate Park Lake Emergency Spillway Modifications and Intermediate Spillway*, Geoffrey McClean and Company / Charles A. Thomas, 1990. (See Reference Documents, **Appendix A**)



PATH: C:\USERS\GMITCHELL\ONEDRIVE - HAZEN AND SAWYER\DOCUMENTS\ARCGIS\PROJECTS\EG_OFFLINE\EG_OFFLINE.APRX

N

0 6,000 FEET

1" = 6,000'

Hazen

4011 WESTCHASE BLVD
SUITE 500
RALEIGH, NC 27607

Raleigh

ENGINEERING SERVICES

EASTGATE DAM PER

VICINITY MAP

DATE: JUNE 2023

FIGURE:

REFERENCE: ROAD NAMES (WAKE COUNTY), BASEMAP (CITY OF RALEIGH, WAKE COUNTY, STATE OF NORTH CAROLINA DOT, ESRI, HERE, GARMIN, SAFEGRAPH, GEOTECHNOLOGIES, INC, METI/NASA, USGS, EPA, NPS, USDA)

Hazen
4011 WESTCHASE BLVD
SUITE 500
RALEIGH, NC 27607
ENGINEERING SERVICES

EASTGATE DAM PER

SITE MAP

DATE: JUNE 2023

FIGURE:

PATH: C:\USERS\GMITCHELL\ONEDRIVE - HAZEN AND SAWYER\DOCUMENTS\ARCGIS\PROJECTS\EG_OFFLINE\EG_OFFLINE.APRX

REFERENCE: ROAD NAMES (WAKE COUNTY), PROJECT SURVEY, AERIAL IMAGERY (NC ONEMAP (2019). NORTH CAROLINA DEPARTMENT OF INFORMATION TECHNOLOGY, GOVERNMENT DATA ANALYTICS CENTER, CENTER FOR GEOGRAPHIC INFORMATION AND ANALYSIS. AVAILABLE AT WWW.NCONEMAP.GOV.)

Table 1: Updated Dam Inventory Data

Dam Name	Eastgate Park Dam
State Identification Number	WAKE-156
National Identification Number	NC04490
County	Wake
Jurisdictional Status	Jurisdictional
Dam Status	Impounding
Hazard Class	High
Dam Type	Earth
Dam Purpose	Recreation
Regional Office	Raleigh
Quadrangle	Raleigh East
Latitude and Longitude	35.8405, -78.6243
River or Stream	Big Branch-Tributary
River Basin	Neuse
Nearest Downstream City	Raleigh (0-mi)
Structural Height (ft)	20.0
Normal Freeboard (ft)	4.2
Hydraulic Height (ft)	17.0
Upstream Slope	3H:1V
Downstream Slope	2H:1V
Low Flow Requirement (cfs)	N/A
Maximum Spillway Capacity (cfs)	406
Normal Pool Elevation (ft, NAVD88)	257.6
Drainage Area (ac)	189
Surface Area (ac)	2.4
Normal Pool Capacity (ac-ft)	15.9
Maximum Impoundment Capacity (ac-ft)	29.7
Presence of Bottom Drain	Unknown
Last Inspection	03/01/2023
Primary Spillway	4.3’SQ Riser and 42” RCP Barrel
Auxiliary Spillway	South: 15’W x 1.7’H Overflow North: 18’W x 1.6’H Overflow North: 30” RCP Outlet

2. Condition Assessment

Engineers from Hazen visited the site multiple times between 2022 and 2023 to observe conditions at the dam during the course of the project. Overall, the dam is in unsatisfactory condition, with several adverse conditions noted, including active and potential erosion concerns, embankment stability hazards due to trees, and active seepage downstream of the outlet works. A summary of the Dam components, overall conditions, and general photographs are presented below. Descriptions and photographs of additional concerns are presented in **Appendix B**.

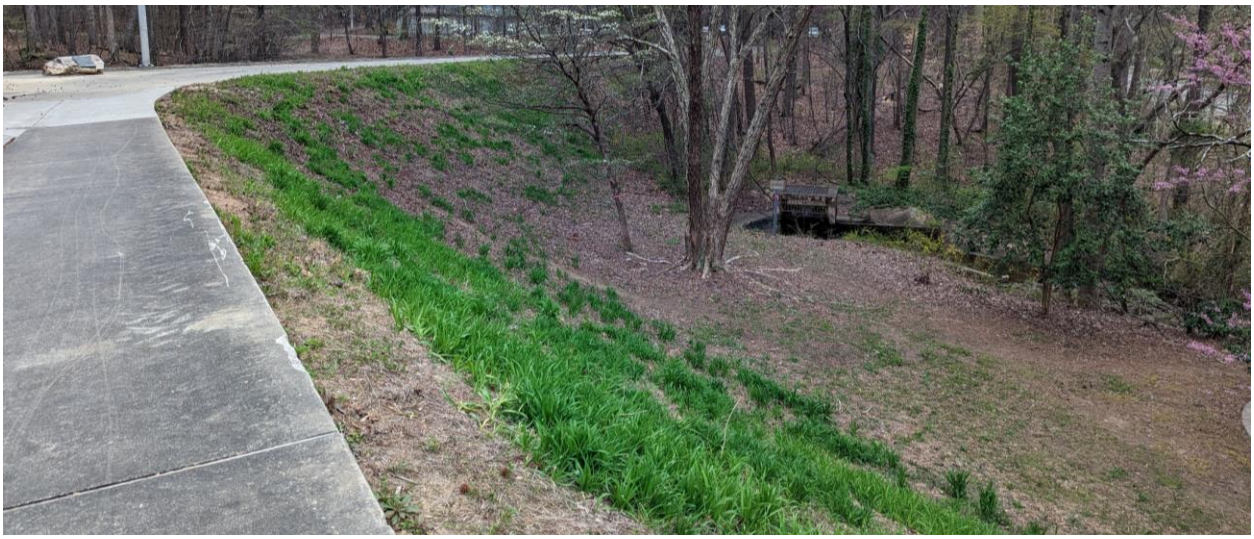
2.1 Dam Embankment

Eastgate Park Dam is an earthen embankment with a grouted curtain wall, grouted riprap on the upstream face, and vegetation on the downstream face (**Picture 1** and **Picture 2**). The downstream slope is relatively steep at a grade of 2H:1V, while the upstream is around 3H:1V. The top of dam is lined with concrete slabs that have embedded area inlets and small-diameter drains that discharge into the Pond (**Picture 3**). Slabs on the top of dam exhibit signs of downward displacement, multiple fractures, and open joints. Overflow spillway sections, discussed later, are present on each end of the embankment. The spillways appear to be situated on or immediately adjacent to the abutment contacts, and thus may be at least partially located on embankment fill.

The downstream slope surface appears stable; however, multiple bare spots and large diameter trees are present. Additionally, vegetation on the slope consists various weeds, with a mix of ground covering and low-height species. There is also a substantial drop in elevation between the top of dam slabs and downstream face which is highly susceptible to erosion should the embankment overtop. The grout lining on the upstream slope is largely intact; however, numerous voids and fractures were observed in the vicinity of the drains and surrounding the large diameter trees growing through the lining. Large trees are also present within 15-feet[3] of the downstream toe and both abutments. Lastly, embankment toe drains, presumably installed in accordance with the 1974 improvements (see Drawing #3), were unable to be located during site visits. As such, the presence and condition of the drains is unknown. A drain from the north is shown on the 1974 drawings as discharging into the riprap-lined portion of the outlet channel. A second drain, from the southern portion of the embankment, is shown discharging into a large, riprapped area downstream of the outlet structure; however, such an area does not appear to exist today.

[3] NCDS states trees, brush, and woody vegetation shall be removed from all parts of a dam and within 15-feet of any part of the dam (Dam Safety Guidance Regarding Trees on Dams, 05/23/2022, https://deq.nc.gov/media/29323/download?attachment)

Picture 1: Upstream Face of Dam



Picture 2: Downstream Face of Dam

Picture 3: Top of Dam

2.2 Spillways

2.2.1 Principal Spillway

The principal spillway (**Figure 3**) is composed of an inlet structure within the Pond, a pipe barrel, and an outlet structure at the toe of the downstream slope. The inlet structure consists of a 4-sided weir with embedded vertical trashracks (**Picture 4**). The top of the structure is also open with a similar, but horizontal, trashrack. The inlet sits directly on top of an approximately 7-foot high riser which has a formed invert to smoothly transition vertical dropping flow into the barrel. The barrel is 42-inches in diameter, approximately 95-feet long, and has an elevation fall of 12.5-feet. The barrel terminates at the outlet structure which is of the same design as the inlet structure, except for being three-sided (**Picture 5**). The outlet structure sits approximately 2.5-feet above adjacent grade which consists of a concrete-lined spiral that protects the area from erosion and directs flows into the receiving channel. The receiving channel is concrete-lined nearest the outlet then transitions to a riprap-lined channel (**Picture 6**). The first half of the channel has relatively small riprap while the second half, adjacent to Quail Hollow Drive, contains larger stone and appears to have been recently reconstructed during a nearby water main replacement project (**Picture 7**).

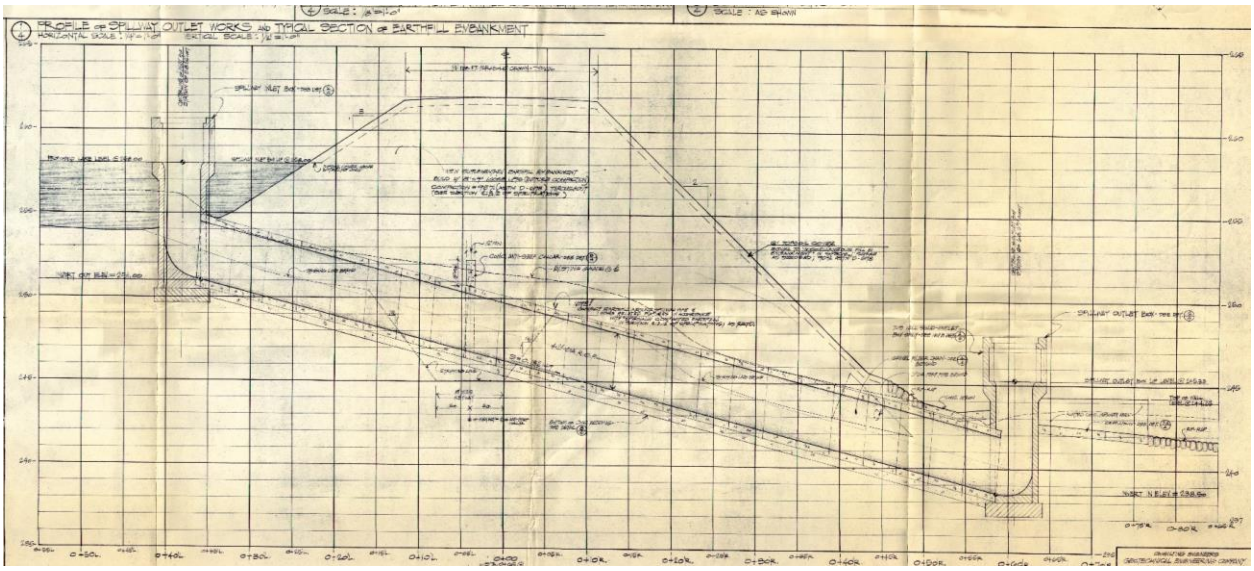


Figure 3: Principal Spillway (Excerpt from 1974 Drawing #4)

The inlet and outlet structure are in fair condition with typical concrete weathering and deterioration. Only minor debris accumulation was observed on the trashracks. Vertical displacement of the concrete outlet apron / channel was observed, as well as open joints and fractures. A substantial amount of seepage was also seen emanating from below / behind the concrete slope protection on the left (southern) side of the concrete-lined channel where the concrete lining terminates (**Picture 8**). A large-diameter tree is present at this same location and may be contributing to uplifting of the concrete lining.

Picture 4: Principal Spillway Inlet Structure

Picture 5: Principal Spillway Outlet Structure

Picture 6: Upstream Segment of Outlet Channel

Picture 7: Downstream Segment of Outlet Channel and Quail Hollow Drive Culvert

Picture 8: Seepage Emanating from Below Outlet Channel Concrete Lining

2.2.2 South Auxiliary Overflow Spillway

The first auxiliary spillway is a shallow overflow section located immediately south of the embankment and consists of reinforced concrete slabs on the upstream approach and top of dam. The downstream slope is not protected from erosion and contains several large-diameter trees with no formal ground cover. The control elevation of this section is approximately 1.7-feet below the top of dam and existing topography indicates overflows would be at least partially directed into the downstream groin which is also unprotected and has several trees present. While the concrete slabs are in relatively good condition, the lack of erosion protection and large trees downstream of the Dam present concerns should the spillway be activated. Additionally, with flows being directed into the unprotected groin, the Dam embankment may also be at risk of erosion. Further, given the relatively shallow nature of the spillway, the hydraulic capacity is fairly limited at Pond levels that do not overtop higher portions of the Dam. Combining risk of erosion with low hydraulic capacity, reliance on this spillway as part of safely passing the SDF may not be warranted.

Picture 9: South Auxiliary Overflow Spillway

2.2.3 North Auxiliary Overflow Spillway

A second overflow section is located on the northern end of the Dam where the final 20-feet of the embankment is nearly 1.6-feet lower in elevation as compared to the dam. Should this section be activated, flows would travel down the steep embankment slope and downstream groin. While the downstream slope is lined with grass and weeds, the upstream slope and top of dam consist of concrete slabs. Similar to the southern auxiliary spillway, the shallow nature of this section provides little hydraulic capacity but presents erosion concerns if the spillway experiences flow.

Picture 10: North Auxiliary Overflow Spillway

2.2.4 North Auxiliary Pipe Spillway

The third auxiliary spillway consists of small concrete u-shaped weir inside the Pond, and a 30-inch piped drainage system that connects to a 24-inch pipe located near the toe of dam. The weir structure was designed with the crest approximately 0.9-feet above normal pool, a depressed section immediately downstream of the weir, and the outflow pipe invert set at normal pool. The original design appears to imply unobstructed flow on three sides; however, existing grade partially blinds both the left and right portions of the crest. Further, the area round the weir and depressed inlet section are full of debris and sediment. Additionally, the weir and pipe appear to have been constructed slightly lower than designed (by 0.1 and 0.3-feet, respectively), though in a maintained condition, these likely function as intended. The pipe system runs from the inlet structure, downstream through the embankment to beyond the toe of dam, where a manhole realigns the system roughly parallel to the embankment. This manhole is also the location where the pipe diameter decreases to 24-inches. The pipe continues southward, ultimately discharging into the downstream receiving channel, approximately 35-feet upstream of the Quail Hollow Drive culvert.

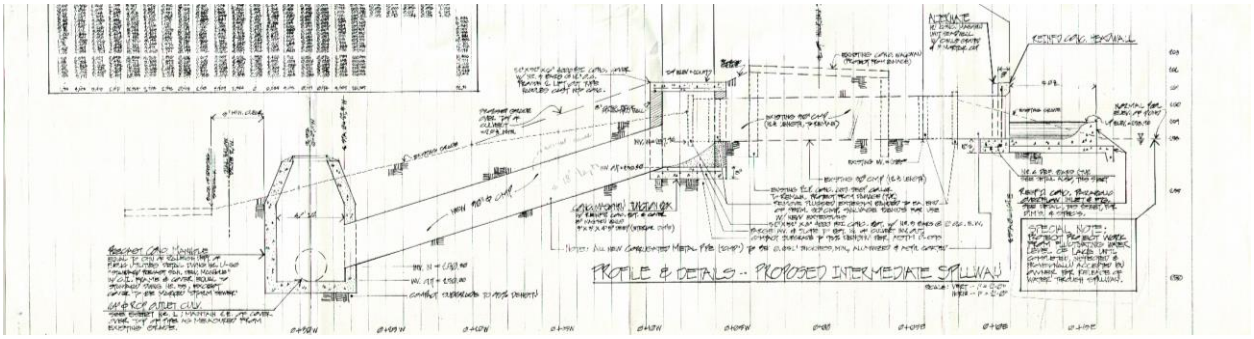


Figure 4: North Auxiliary Pipe Spillway (1990 Drawing #2 Excerpt)

Picture 11: North Auxiliary Pipe Spillway Inlet Structure

2.3 Downstream of Dam

Immediately downstream of the Dam is a wooded area in the vicinity of the principal spillway outlet and southern end of the embankment (**Picture 12**). Downstream areas along the middle portions of the Dam are grass-lined, with a few trees present (**Picture 13**). The northern end contains both grass-lining and a concrete sidewalk (**Picture 14**). Downstream (west) of these vegetated areas is Quail Hollow Drive, which is asphalt paved (**Picture 15**). A 72-inch culvert beneath the roadway conveys flow from the Dam's outlet channel to another reach of channel before connecting to Big Branch located behind homes located on the western side of Quail Hollow Drive.

Though not directly related to dam safety, the culvert and downstream channel were also inspected since these are located immediately downstream of the Dam and may impact inundation if not performing properly. The culvert appears to be in fair condition; however, during a March 2023 inspection, the amount of flow entering the pipe appeared to be less than the amount exiting, potentially indicating that water was running below the culvert. Additionally, the upstream endwall is in poor condition with both large vertical fractures and horizontal displacement (**Picture 16**). The channel reach downstream of the culvert is also in poor condition, with a downed tree across the waterway immediately downstream of the outlet and extreme erosion on both banks, including vertical slopes and undercuts (**Picture 17**).

Picture 12: Area Downstream of Dam (At Principal Spillway Outlet)

Picture 13: Area Downstream of Dam (Near Quail Hollow Drive Culvert)

Picture 14: Area Downstream of Dam (North of Near Quail Hollow Drive Culvert)

Picture 15: Area Downstream of Dam (South of Near Quail Hollow Drive Culvert)

Picture 16: Upstream End of Quail Hollow Drive Culvert

Picture 17: Downstream End of Quail Hollow Drive Culvert

3. Hydrologic and Hydraulic Analyses

3.1 Approach

Hydrologic and hydraulic analyses were conducted based upon standard engineering methods and practices. In general, hydrologic parameter computations followed the runoff curve number and times of concentration methodologies presented in the National Engineering Handbook (NEH) Part 630. Analyses (modeling) were performed utilizing United States' Army Corps of Engineers' (USACE), Hydrologic Engineering Center (HEC) software packages Hydrologic Modeling System (HMS) version 4.10 and River Analysis System (RAS) version 6.3.1.

The models were newly developed based upon project field survey data, available geographical information system (GIS) data, and the *effective* hydraulic model from the North Carolina Floodplain Mapping Program (NCFMP). Project survey included detailed topography of the dam embankment (from edge of water in the Pond to across Quail Hollow Drive, the principal and piped auxiliary spillways, the outlet structure and receiving channel, as well as the Quail Hollow Drive culvert and a portion of the downstream channel. A computer aided design drawing (CADD) of the survey is provided electronically in **Appendix A**.

The hydrologic model included subbasin routing (runoff curve number method), one-dimensional (1D) channel routing (Muskingum-Cunge), and reservoir routing (level pool). Areas contributing to the Dam and to Big Branch were included to establish inflow hydrographs to both. The hydraulic model was developed using a two-dimensional (2D) computational mesh. Available digital elevation models (DEMs) from North Carolina Emergency Management (NCEM) were used as the base topographic layer, with data from the project topographic survey being layered above the DEMs. To improve model stability and results, the Big Branch channel, as depicted in the NCFMP model, was also incorporated into the RAS terrain. Culvert and bridge crossing data were also obtained from the NCFMP model, except the Quail Hollow Drive culvert, which was included in the project survey. Spatial data were referenced to the North Carolina State Plane, North American Datum of 1983 (NAD83), in US feet. Vertical data were referenced to the North American Vertical Datum of 1988 (NAVD88).

3.2 Hydrologic Analyses

3.2.1 Precipitation

Evaluations included two rainfall distribution, with standard design storms (2- through 100-year, 24-hour) based upon National Oceanic and Atmospheric Administration (NOAA) Type B distribution and the probable maximum precipitation (PMP) based upon Hydrometeorological Report (HMR) number 51 (**Figure 5**). Standard design storm depths were obtained for the Dam's watershed from NOAA Atlas 14. PMP depths were generated from depth-area-duration relationships presented in HMR-51, after which the controlling storm was determined.

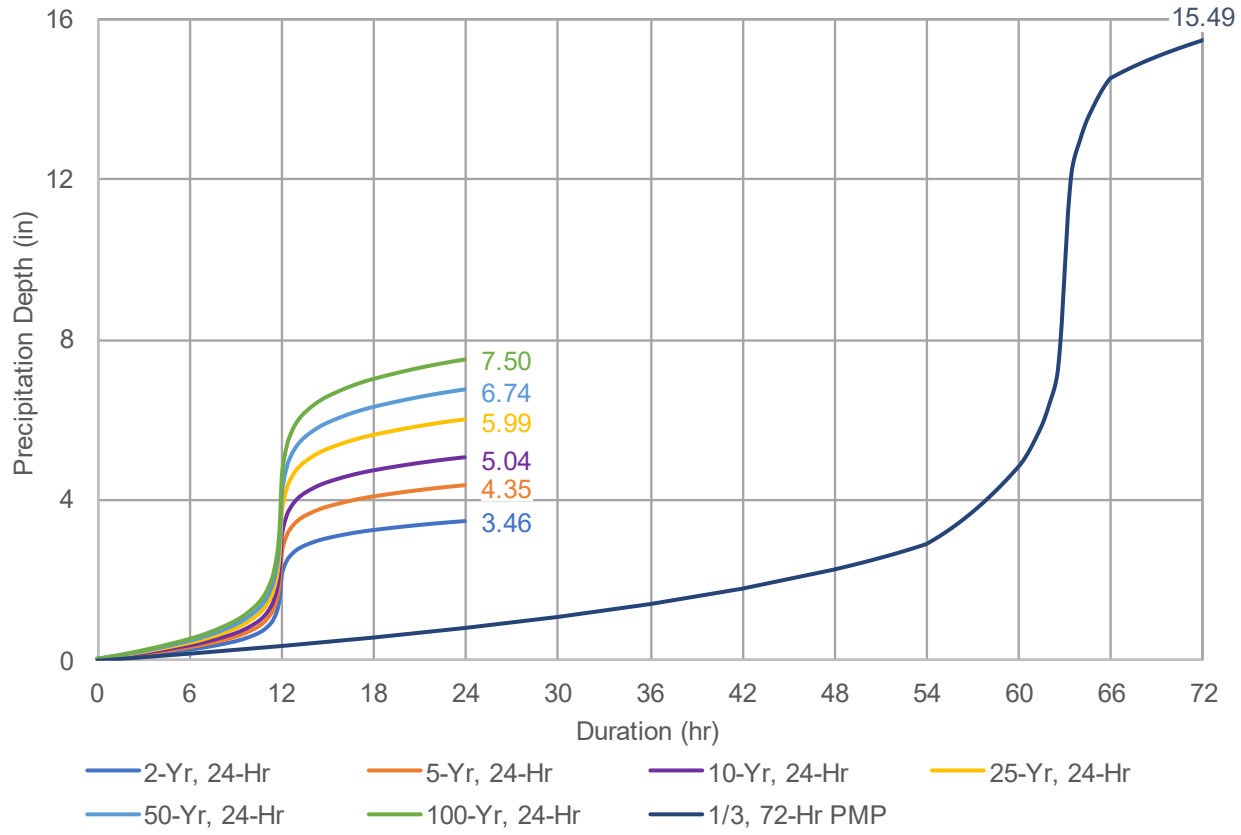


Figure 5: Rainfall Distributions

The controlling storm is the PMP duration that produces the highest peak discharge from the dam and is then used in determining the spillway design storm (referred to as the spillway design flood (SDF)). To establish the controlling storm, the 6-, 12-, 24-, 48-, and 72-hour PMPs were simulated in HMS. This analysis identified the 72-hour PMP duration as being the controlling storm. From there, the SDF was determined based upon North Carolina Administrative Code (NCAC), Subchapter 2K – Dam Safety (15A NCAC 02K). Per article .0205 of this code, Eastgate Park Dam is regulated as a *small* facility (less than 750-acre feet of storage and less than 35-feet in height) and is considered *high hazard (Class C)* due to downstream inundation impacts to roadways and habitable structures. Given the size and hazard class, the code specifies an SDF equal to the 1/3rd PMP.

3.2.2 Hydrologic Characterization

Hydrologic parameterization and runoff generation were based upon the runoff curve number (RCN) method outlined in NEH Part 630. Drainage basins were delineated using NCEM DEMs and manually revised to ensure better agreement with stormwater conveyance systems depicted on Raleigh iMaps. Overall, seven subbasins contributing to the Pond were delineated, ranging in area from 17- to 39-acres, with a total watershed area of 188-acres (**Table 2** and **Figure 6**).

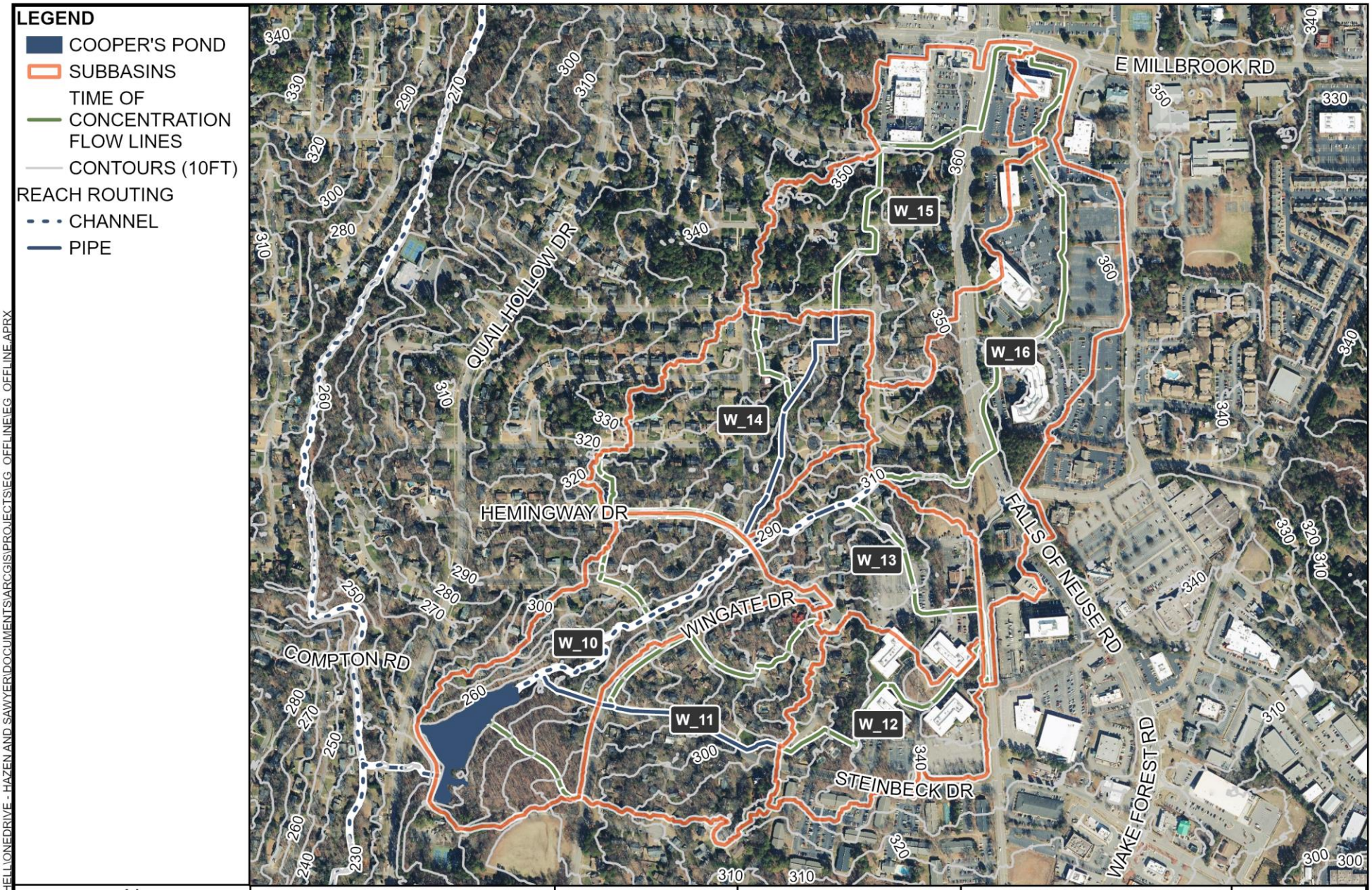
Table 2: Hydrologic Parameters

Subbasin	Area (ac)	Runoff Curve Number		Time of Concentration (min)	
		Existing	Future	Existing	Future
W_10	30	62	72	17	14
W_11	24	57	65	10	8
W_12	17	80	84	8	7
W_13	19	76	79	7	7
W_14	25	58	63	11	9
W_15	34	71	73	7	7
W_16	39	86	86	6	6
Overall	**188**	**70**	**75**	**27**	**24**

Evaluation of landuses included both existing conditions, via the National Land Cover Dataset (NLCD), and future development, via City zoning. All dam safety analyses (e.g. PMP events) used future conditions landuse per 15A NCAC 02K .0204. Hydrologic soil groups (HSG) were obtained from Natural Resources Conservation Service (NRCS) soils mapping. The combination of landuse and HSG were then used to assign RCNs from NEH. Overall, existing landuse was composed mostly of developed space (NLCD classes: open space, low intensity, and medium intensity) and predominate future landuses consisted of 1/4-acre residential as well as office mixed used. HSGs were approximately 50-percent HSG A, 20-percent HSG B, and 30-percent HSG C.

Times of concentration (TCs) were computed via velocity and watershed lag methods as outlined in NEH. Flowpaths were visually identified in each subbasin and then delineated using the NCEM DEMs. Each velocity flowpath was separated into sheet flow, shallow concentrated flow, and concentrated flow segments, with further subdivisions for changes in landcover and / or substantial changes in ground slope. Sheet flow was limited to a maximum length of 100-feet, per NEH. Travel times for concentrated flow were based upon velocities computed via Manning's equation. Prior to computation, flowpaths were compared to Raleigh iMaps to determine the presence of stormwater conveyance systems, and where encountered, pertinent diameters and lengths were recorded. To compute TCs, pipes were assumed to be flowing half-full and slopes were based on average ground path slopes. Open channel segments were computed using dimensions from Raleigh iMaps and slopes from the NCEM DEMs. In general, computed TCs were relatively short 6- to 17-minutes, which was expected given the heavily developed watershed and presence of long conveyance systems. Additional maps and tables presenting hydrologic parameterization are provided in **Appendix C**.

These same hydrologic analyses were also conducted for basins draining to Big Branch in order to assess inflows to the creek downstream of the Dam. This also permitted validation of the hydrologic analysis by simulating 2-year flows via RAS in Big Branch as well as comparing to the detailed NCFMP HMS model. No other definitive data were available for calibration of the model. The computed 2-year flow in the stream was shown to largely coincide with bankfull flow except one area immediately upstream of the I-440 culvert crossing. Peak discharges for the 100-year storm were also similar to those in the Flood Insurance Study (FIS).



PATH: C:\USERS\GMITCHELL\ONEDRIVE - HAZEN AND SAWYER\DOCUMENTS\ARCGIS\PROJECTS\EG_OFFLINE\EG_OFFLINE.APRX

N

0 350 700 FEET

1" = 700'

Hazen

4011 WESTCHASE BLVD
SUITE 500
RALEIGH, NC 27607

Raleigh

ENGINEERING SERVICES

EASTGATE DAM PER

SUBBASIN MAP

DATE: JUNE 2023

FIGURE:

REFERENCE: CONTOURS (GENERATED FROM NCEM DEM), ROAD NAMES (WAKE COUNTY), AERIAL IMAGERY (NC ONEMAP (2019). NORTH CAROLINA DEPARTMENT OF INFORMATION TECHNOLOGY, GOVERNMENT DATA ANALYTICS CENTER, CENTER FOR GEOGRAPHIC INFORMATION AND ANALYSIS. AVAILABLE AT WWW.NCONEMAP.GOV.)

3.2.3 Hydrologic Response

Simulations of 24-hour storms (2- through 100-year) for both existing and future landuse conditions, as well as the SDF (1/3 PMP) were conducted in HMS to evaluate the hydrologic response of the watershed. Pond inflows for 24-hour events ranged between 240- and 1,100-cubic feet per second (cfs) and the SDF produced a flow of 1,180-cfs. Further, according to these results, the overflow spillways are activated in events slightly larger than the 5-year storm and the dam embankment overtops in events larger than the 25-year storm (**Table 3** and **Figure 7**).

Table 3: Hydrologic Response

Event	Inflow (cfs)		Water Surface Elevation (ft)	
	Existing	Future	Existing	Future
2-Yr, 24-Hr	241	303	259.1	259.4
5-Yr, 24-Hr	386	469	259.9	260.4
10-Yr, 24-Hr	507	606	260.6	260.9
25-Yr, 24-Hr	677	794	261.2	261.5
50-Yr, 24-Hr	815	942	261.5	261.8
100-Yr, 24-Hr	958	1,094	261.8	262.0
1/3 PMP	-	1,176		262.2

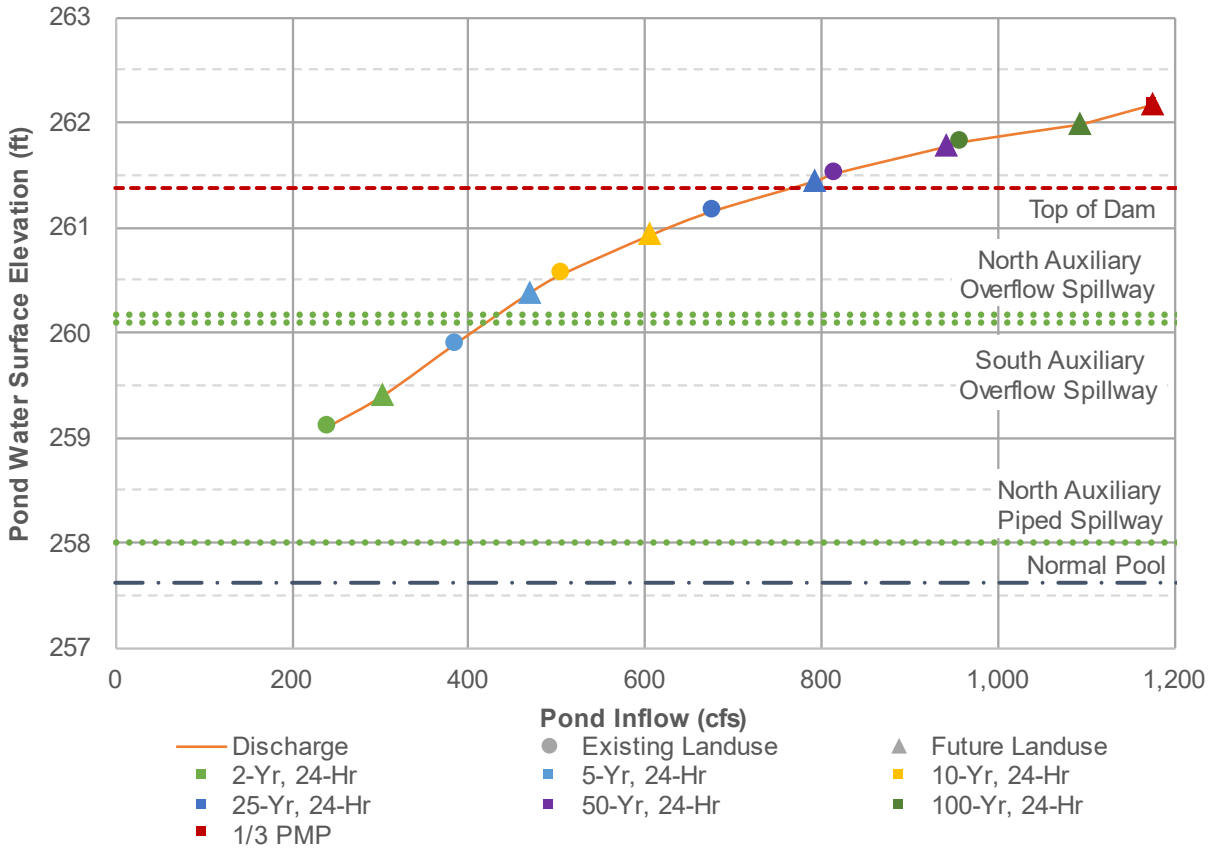


Figure 7: Hydrologic Response

3.3 Hydraulic Analyses

3.3.1 Model Development

Hydraulic analyses were conducted in RAS to assess the performance of the Dam, as well as to identify potential downstream inundation hazards to habitable structures and roadways. The 2D model encompassed the Pond (represented as a storage area with a stage-storage relationship), the Dam (represented as a weir with rating curves for the piped outlets (**Figure 8**)), the outflow channel between the Dam and Big Branch, as well as Big Branch from downstream of Compton Road to approximately 700-feet downstream of Calibre Chase Road, inclusive of roadway crossings represented as 2D overflow areas and culverts. Regions for Manning's values were manually delineated based upon best-available aerial imagery and included channel, paved, and forest zones. The default Manning's roughness coefficient for the computational mesh was set to represent open space, thus such areas were not manually delineated. Lastly, building footprints were also included within the Manning's regions and assigned a high value so as to represent areas of storage with little to no conveyance (values are provided in **Appendix D**).

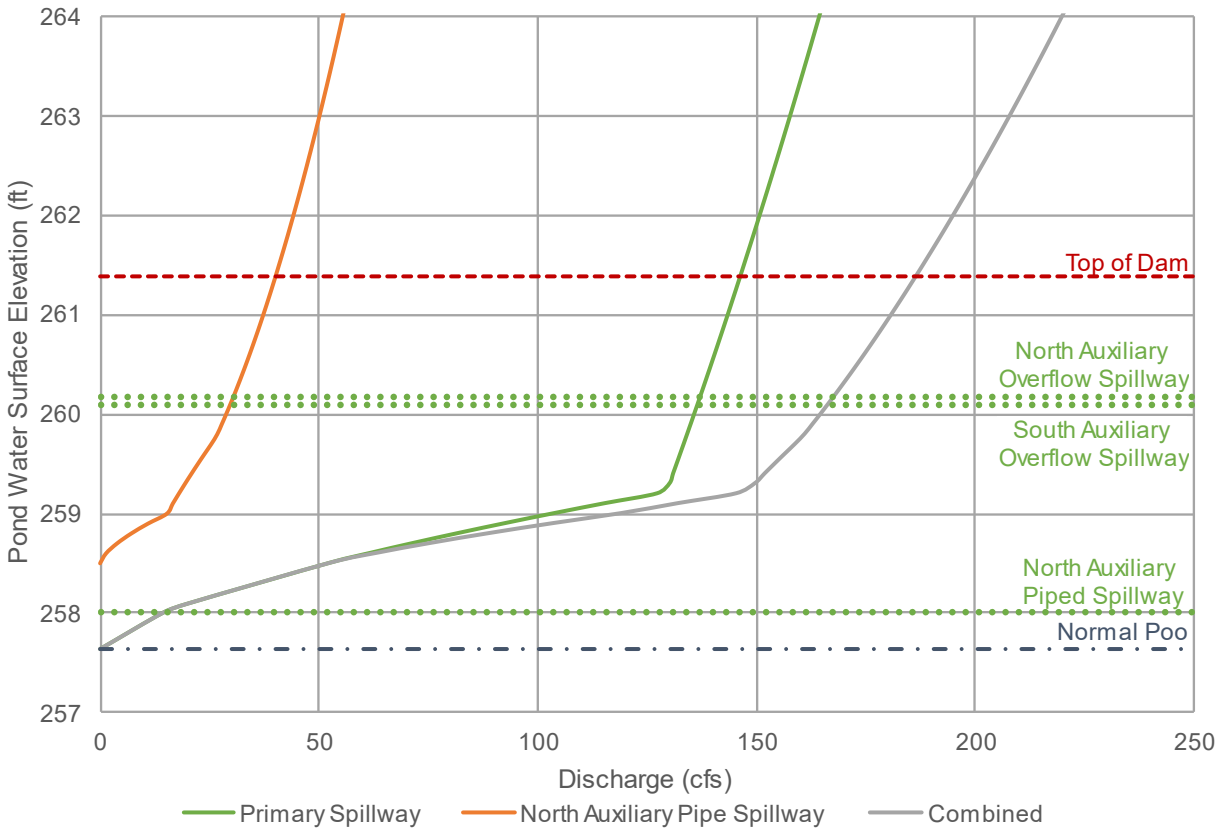


Figure 8: Piped Spillway Rating Curves

The three primary simulations were the SDF, SDF with assumed dam failure (SDF Breach), and the non-hydrologic event failure (e.g. sunny day breach (SDB)). The combination of these runs establishes the

downstream extent of Emergency Action Plan (EAP) dam breach inundation zone (DBIZ) mapping required by NCDS. Further, the SDF Breach and SDB simulations are used to produce DBIZ mapping, as well as identify downstream impacts to habitable structures and roads. Assumed dam failures were represented in RAS via computations and tools built into the software. Physical dam parameters and hydrologic responses (water surface elevations) were input to RAS for computation of dam breach parameters. These parameters were then input to HMS to simulate up to four breach scenarios for both the SDF breach and SDB. The breach parameters producing the highest discharge for each event were selected as the controlling set and input to RAS for simulation and mapping of the SDF Breach and SDF. The breach parameters and inputs are provided in **Appendix D**.

3.3.2 Hydraulic Performance

As observed in the hydrologic analysis, the dam was shown to overtop in approximately the 25-year and larger events, with a maximum depth of 0.8-feet in the SDF (**Figure 9**). The overflow spillways were activated in approximately the 5-year event. Further investigation determined that the facility may pass only the 1/7 PMP (7-inch rainfall, 380-cfs dam discharge) without overtopping the dam (e.g. not overtopping the non-overflow sections) and the 1/10 PMP (4.7 inch rainfall, 165-cfs dam discharge) without activating the overflow spillways. Overall, the hydraulic capacity of the facility (380-cfs) is only about 1/3 of the required capacity (1,175-cfs) to safely pass the SDF. To ensure compliance with dam safety regulations, improvements to the facility are required.

3.3.3 Dam Breach Inundation Hazards

The hydraulic analyses were also used to generate DBIZ maps and identify downstream flood hazards. Overall, impacts may occur as far downstream as near East Six Forks Road, where one residential structure is located within the inundation zone. Multiple habitable structures between this home and Compton Road are also at risk of flooding if the dam were to fail during the SDF, as shown on the revised EAP, provided under separate cover. Structure impacts during the SDB are limited, with only two at-risk buildings, both of which are located immediately downstream of the Dam across Quail Hollow Drive.

Impacted roadways include Quail Hollow Drive in both the SDB and SDF Breach, as well as Hardimont Road, Saint Albans Drive, and Cheswick Drive in the SDF Breach. Further, while not crossing Big Branch, Anderson Drive, Belvin Road, and Converse Drive are located within or along the fringe of inundation and thus may also be at-risk of flooding in the SDF Breach. Of particular concern are Quail Hollow Drive and Cheswick Drive, both of which may be inundated such that emergency access / evacuation routes are restricted due to flooding[4]. For Cheswick Drive, 13 properties may be inaccessible due to overtopping of the roadway. The portion of Quail Hollow Drive between Eastgate Park and Hardimont Road (including Emory Lane) may also be inaccessible during the SDF Breach due to roadway overtopping at the Dam and roadway flooding at the intersection of the two roadways. As a result, 25 properties (including Eastgate Park) may be at risk of being isolated without means of vehicular ingress or egress.

[4] Similarly, at least one lane of Belvin Road is located within the fringe of inundation; if the roadway becomes unpassable, access / evacuation of two properties (with one owner) may be restricted.

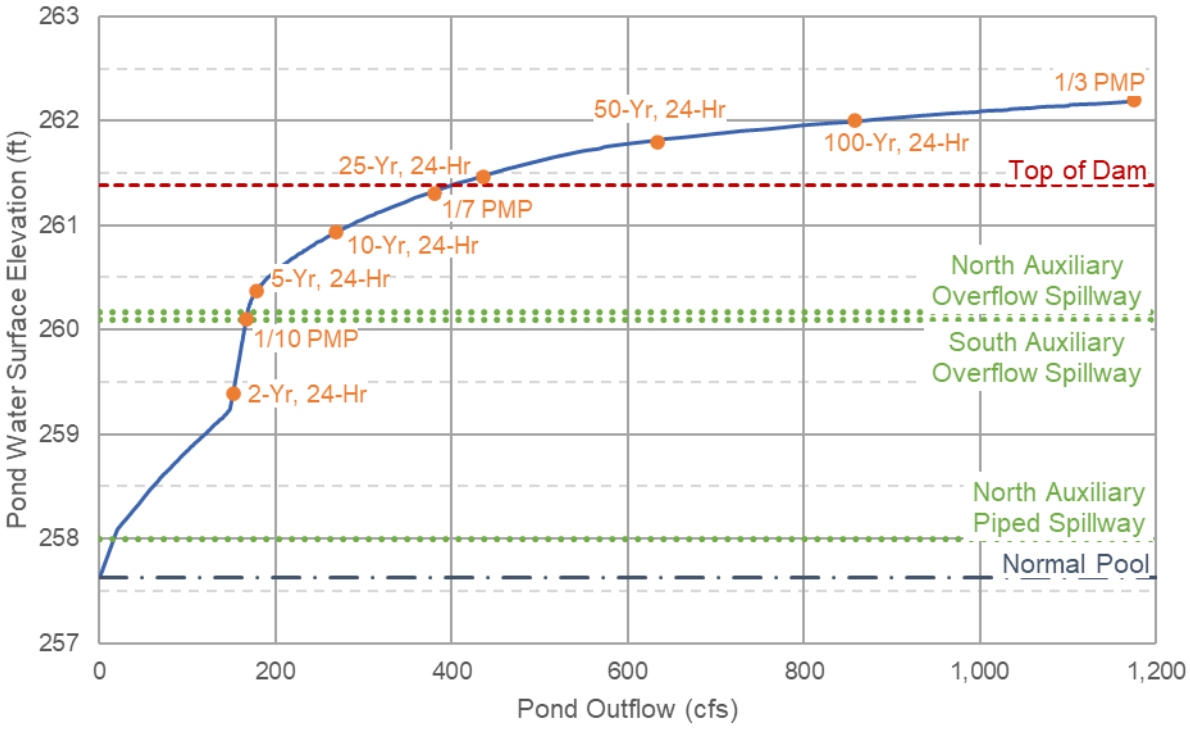


Figure 9: Hydraulic Performance

Impacts to critical facilities and other areas of public interest were investigated using best available GIS data and aerial imagery. No critical facilities were identified within the DBIZ; however, other areas of public interest are within the hazard zone. In addition to Eastgate Park, GoRaleigh bus stop #9667 (Hardimont Road at Quail Hollow Drive) and several properties acquired via the Hazard Mitigation Grant Program (HMGP) may be at risk of flooding should a dam failure occur. The HMGP properties are managed by the City's Parks, Recreation, and Cultural Resources Department (PRCR) and in some cases, used as public parks or open spaces. The properties are located at the following addresses:

- 3824 Quail Hollow Dr
- 517 Oakland Dr
- 3827 Quail Hollow Dr
- 4205 Quail Hollow Dr
- 4213 Quail Hollow Dr
- 4104 Converse Dr
- 4104 Converse Dr

Lastly, Crabtree Creek Trail (Middle Crabtree Creek Segment #110) crosses Big Branch immediately upstream of the confluence with Crabtree Creek. The crossing is downstream of this Study's analysis extent and modeling results indicate flow is confined to the channel along this reach of Big Branch. As such, the trail may not be impacted; however, should debris obstruct flow in the stream or trail bridge crossing, the pedestrian path or bridge may be overtopped.

4. Geotechnical Investigation

A geotechnical investigation was performed by GeoTechnologies, Inc. to ascertain the materials of construction, pertinent material properties, and to conduct stability analyses. A copy of the investigation report is included in **Appendix E** and a summary of the program and findings is provided below.

4.1 Field and Laboratory Testing

A total of eight borings were performed: four standard penetration test (SPT) borings along the top of dam and four hand augers (two along the downstream toe of dam, one on the downstream face, and one near the outlet structure). SPT borings were advanced to a depth of 35- to 40 feet below top of dam and soils were sampled at continuous intervals within fill material and selected intervals within residual soils. Personnel attempted to collect Shelby tube samples between 12- and 14-feet as well as 18- and 20-feet below top of dam; though none were recovered due to softness / wetness of the samples. However, samples at 26- to 28-feet below the top of dam were obtained. Hand auger borings were made to a depth of 2- to 6-feet below grade.

Laboratory testing of the SPT samples yielded low organic content (1.8- to 2.0-percent by weight) and dual Unified Soil Classification System (USCS) designations of clayey sand – silty sand (SC-SM) and elastic silt – fat clay (MH-CH). Concrete slabs on the top of dam were determined to be 3- to 4.5-inches thick, below which was a fill layer of silty to clayey sands and sandy low to high plasticity clays / silts ranging in depth between 12.5- and 23.5-feet. Compaction was estimated to range between poor- and well-compacted, with SPT resistances between 2- and-14 blows per foot (bpf). Loose to dense residual soils (clean to silty sands) were found below the fill material with SPT resistances of 8- to 35-bpf. Partially weathered rock (PWR) was encountered in three of the four SPT boring locations at 26.5 to 36.5-feet below the top of dam.

Hand auger borings indicated around 0- to 6-inches of topsoil at the ground surface, underlaid by silty sand to sandy silt fill material at three locations (one location encountered residual silty sand). The materials were estimated to be poorly- to well-compacted. The two borings at the toe of dam encountered refusal at approximately 2-feet below grade.

4.2 Stability Analysis

A stability analysis was performed to ascertain the factor of safety against slope failure. The location analyzed was approximately midway along the original embankment, which is the highest point of the dam and where encountered materials were generally soft / loose fill material. This analysis yielded a safety factor against slope failure of 1.17, below the NCDS required factor of 1.5 (15A NCAC 02K .0208.b). A second slope stability analysis for rapid drawdown was also conducted for this same location. This indicated a safety factor of 1.47, above the NCDS required factor of 1.25; however, this analysis assumed that the upstream slope below the water level was consistent with the exposed slope (3H:1V). Further investigation is required to confirm this assumption. If the upstream face below normal pool is steeper than 3H:1V then the analysis will require updating and the slope potentially reduced to 3H:1V.

5. Alternatives Analysis

As presented above, the evaluation of Eastgate Park Dam identified several maintenance and repair needs, but more importantly, brought to light two concerns regarding long-term safety as well as regulatory compliance: the facility does not safely pass the SDF and (2) the embankment does not meet slope stability requirements. The former may be addressed in one of two ways: (1) replace the existing spillways with one of sufficient size or (2) provide overtopping protection on the dam. The latter concern, embankment stability, may be resolved by lessening the slope of the downstream face. Each of these potential solutions is presented in the following sections.

5.1 Hydraulic Capacity Improvement

Based on the hydraulic assessment, nearly 800-cfs of additional capacity is required to safely pass the SDF. Options to provide this increase include replacement of the existing piped spillways (principal and north auxiliary) with either a traditional weir-chute style or a new riser-barrel spillway. A preliminary evaluation was therefore conducted to estimate the spillway size, with the following constraints:

- The top of dam was assumed to be reconstructed at a level elevation of 261.85-feet, which is the current high point elevation.
- A minimum freeboard of 3-feet is required for the SDF.
- The 100-year storm discharge from the dam may not increase compared to existing conditions

Several weir lengths and elevations, as well as new permanent normal pool elevations, were assessed to determine which configuration met the above criteria. Ultimately, a 56-foot long compound weir (multiple elevations) met these conditions, but required lowering normal pool by 5-feet. Other scenarios with higher normal pool elevations were unable to both maintain the 100-year peak discharge and provide 3-feet of freeboard in the SDF. More specifically, a 21-long weir at elevation 252.63-feet controlled the 100-year storm while an additional 35-feet of weir at elevation 258.00 added sufficient additional capacity for the SDF to safely pass. Overall, this is a moderate increase compared to the total weir length (43-feet) of the existing riser (4-sided box with an open top); however, the slightly raised dam top and lowered normal pool provide an additional 2.5-feet of driving head, thus also contributing to the increased capacity.

With the spillway size estimated, two general options for new spillway configurations were deemed well suited for EGD: a riser-barrel system (e.g. an enlarged version of the existing) or a labyrinth weir-chute system. The riser-barrel system would replace the existing principal spillway and is recommended to be fitted with trashracks and a low level outlet (LLO) to provide Pond drawdown capability. A new outlet pipe, in the form of a double barrel 9-foot wide by 6-foot high box culvert, is also necessary to convey flow downstream of the Dam.

Alternatively, a new labyrinth weir structure through the dam may be constructed instead of a riser-barrel system. A labyrinth is recommended as such weirs generally have a span 3- to 5-times shorter than standard weirs, thus the total span may be on the order of 20-feet or less. This configuration requires construction of training walls on either side of the weir, as well as an open chute on the downstream face. A bridge over the spillway is also necessary to maintain pedestrian access.

Along with spillway improvements, a new energy dissipator will also be required at the outlet that is designed for the SDF. Two common approaches include a riprap-lined plunge pool or a reinforced concrete dissipator structure; or possibly a combination of the two. Plunge pools (**Figure 10**) are generally less expensive and often easier to construct; however, also tend to be larger than structural measures. A preliminary design estimated the pool size to be 75-feet long by 70-feet wide by 2.5-feet deep. For perspective, this length is approximately 15' less than the distance between the existing outlet and Quail Hollow Drive culvert.

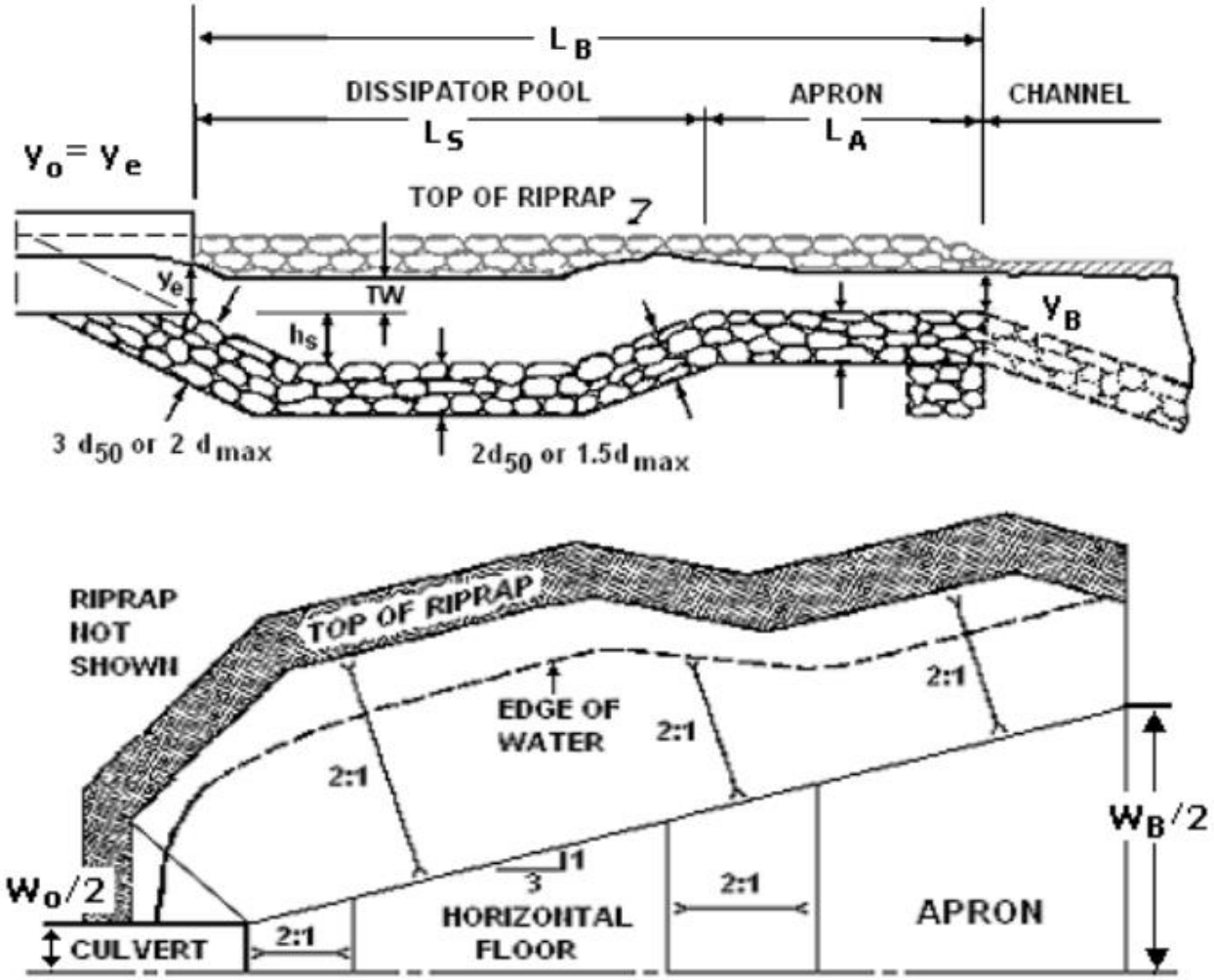


Figure 10: Schematic Views of Riprap-Lined Plunge Pool Basin (HEC-14, FHWA, USDOT)

Another option is a structural dissipator, of which there are numerous designs. Most involve a concrete-lined chute or channel and either concrete blocks or baffle walls. Selection and design are best suited for detailed design; however, a preliminary sizing of a Contra Costa Basin (**Figure 11**) estimated the structure to be 35-feet long by 20-feet wide by 10-feet high. This type of dissipator consists of a trapezoidal-shaped concrete-lined channel, two baffle walls, and end sill.

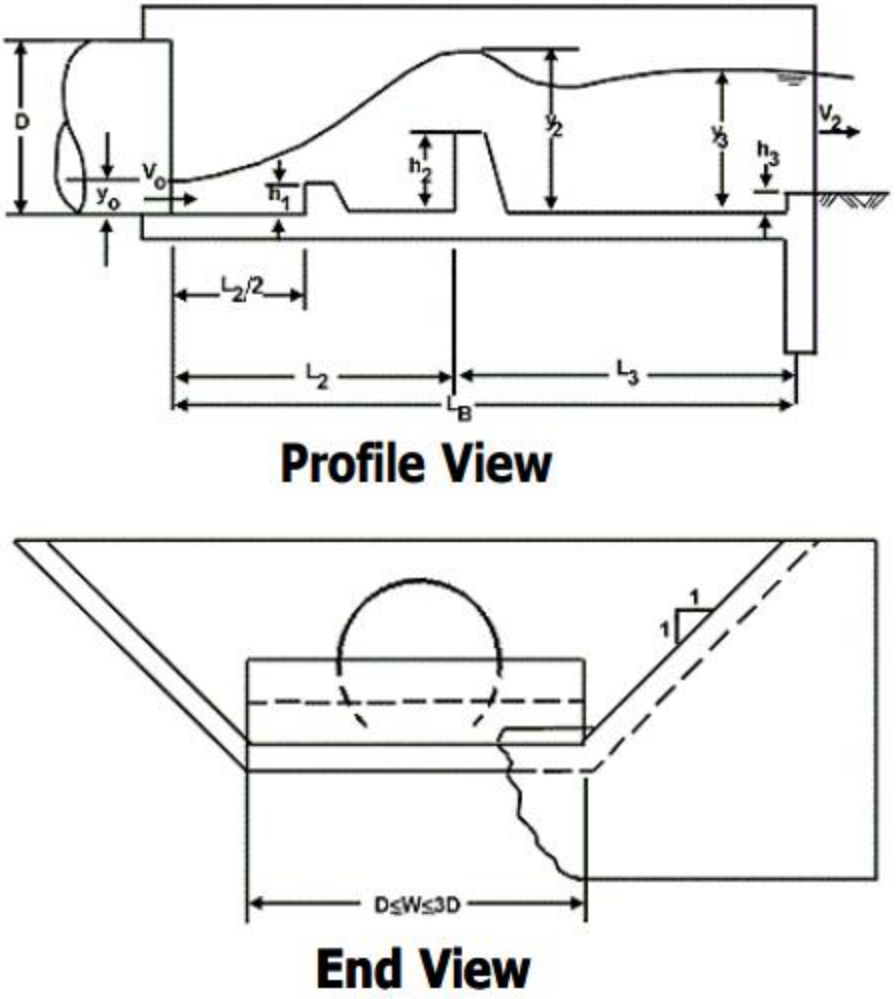


Figure 11: Schematic Views of Contra Costa Basin (HEC-14, FHWA, USDOT)

Lastly in regards to increasing hydraulic capacity, the most substantial impact is likely the required lowering of Cooper's Pond by 5-feet. This would reduce the normal pool area by nearly 1-acre, exposing both shallow and intermediate depth areas currently below water. Such an impact is not ideal as this changes the aesthetics and nature of the Pond. However, mitigating this change is possible via installation of a perimeter wall to approximately maintain the current normal pool footprint. Dredging of the shallow and intermediate depth areas is also necessary to ensure a minimum depth of water throughout the pool. All or a portion of the dredged materials may be placed along the current perimeter, behind the new wall to avoid or decrease costly hauling and disposal. The wall system must be designed to accommodate the likely poor-quality material as well as existing subgrade conditions. Such a project entails a robust geotechnical and testing program to evaluate subsurface conditions in regards to bearing capacity, hydrostatic forces, and presence of pollutants in the Pond sediments.

Given the intrinsic high cost and substantial adverse impacts of the above, such improvements were deemed unfeasible and not as cost-effective as a second, more suitable option involving installation of overtopping protection. Further, replacement of the spillway addresses only one concern (hydraulic

capacity); thus correcting the slope stability and maintenance / repair items would come as additional costs. However, adding overtopping protection, discussed in the following section, provides the necessary hydraulic capacity, does not result in permanent pool impacts, and inherently addresses most of the maintenance / repair recommendations, as well as the slope stability deficiency.

5.2 Overtopping Protection

Installation of overtopping protection on the Dam provides the necessary hydraulic capacity to pass the SDF while protecting the embankment from failure due to overflow. Recent examples of such an improvement are present within the City, including Lake Johnson Dam and Lake Benson Dam. Further, an overtopping protection system for Lake Wheeler Dam is currently being designed as well. Hazen is or was involved in each of these projects and thus is very familiar with the design and construction of these systems.

Specific to Eastgate Park Dam, an open-cell articulated concrete block (ACB) system is recommended to cover the entire embankment, including the currently grouted riprap upstream face. The top of dam is used for pedestrian access and can include asphalt pavement on top of the ACB to provide a smooth walking surface, similar to Lake Johnson Dam. A photograph of ACB installation at Lake Johnson Dam is shown in **Picture 18**. One or both of the embankment slopes may also be planted to improve aesthetics, as well as providing other environmental benefits. Planting the faces can be accomplished in one of two ways: (1) placing soil and seeds / plants in the openings of the blocks or (2) adding a topsoil layer above the ACB and seeding / planting. The former is generally recommended from a cost perspective, as the soil and plantings are sacrificial and prone to mobilization during an overflow event. Establishment of grass cover on Lake Johnson Dam is shown in **Picture 19**.

Picture 18: Installation of Overtopping Protection at Lake Johnson Dam

Picture 19: Grass Cover and Asphalt Pedestrian Path at Lake Johnson Dam

Overall, this alternative is likely to result in the least amount of permanent and temporary impacts to the facility. Further, overtopping protection helps to ensure regulatory compliance as well as protection from erosion during overflow events. This work may also be accomplished in conjunction with necessary embankment stability improvements which involve placement of fill on the downstream face, as discussed in the following section. Given the related nature of these two improvements and associated cost savings, construction of overtopping protection appears to be a better alternative as compared to spillway improvements.

5.3 Embankment Stabilization

Geotechnical stability analyses indicated that the embankment does not meet the NCDS safety factor against slope failure. Two options are available to address this issue: (1) flatten the slope of the downstream face or (2) chemical stabilization. The second option is often substantially more expensive than the first but does maintain the current geometry of the embankment. However, if overtopping protection is selected as the preferred hydraulic capacity improvement a few geometric changes, at a minimum, will still be required (leveling top of dam, smoothing slopes, etc.). Further, overtopping protection and flattening the downstream slope both require excavation and removal of topsoil and unsuitable subsoils, fine grading, and placement of suitable materials. As such, these two improvements are complimentary and will address both concerns (stability and overtopping / hydraulic capacity) without need of an expensive chemical stabilization program.

Flattening the downstream slope involves clearing and grubbing vegetation, stripping topsoil, removal of unsuitable materials, placement and compaction of suitable material to existing grade, placement of C-33 sand (minimum 12-inches thick), and placement and compaction of low permeability material above the sand (minimum 12-inches thick) to achieve a maximum slope of 2.75H:1V. The sand layer will extend to the existing downstream toe and continue outwards to the newly established toe, where a new toe drain is

to be installed. Above these layers includes the ACB system, which is composed of a gravel drainage layer and the concrete block. As mentioned previously, the block cells can be filled or the entire surface covered with topsoil and seeded to provide a vegetative cover.

One complication of flattening the downstream slope is avoiding impacts to the principal spillway outlet. The recommended maximum slope of 2.75H:1V causes the embankment slope and toe to extend beyond the outlet, thus a retaining wall will be required to avoid such impacts. A conceptual grading plan showing this wall is provided in **Appendix F**. Construction of the wall is expected to require at least four-feet of excavation below existing grade, thus temporary shoring may be necessary to protect the existing embankment during construction. Additionally, given the depth of excavation and likelihood of groundwater, dewatering is also likely to be necessary.

Overall, combining overtopping protection with flattening of the downstream slope into a single, comprehensive improvement present a solution with fewer permanent impacts, addresses the two primary dam safety concerns, and also corrects most of the maintenance / repair needs. The estimated cost for such a project was estimated to be $5,200,000, including construction, design, permitting, and contingencies.

6. Conclusions and Recommendations

Evaluations of Eastgate Park Dam identified two safety / regulatory concerns and several conditions requiring repair / maintenance. Overall, the facility is considered to be in unsatisfactory condition given these concerns and is in need of substantial improvements to ensure continued safe operation and regulatory compliance. The safety / regulatory concerns are that the embankment does not meet the NCDS required safety factor against slope failure and the outlet systems do not provide sufficient hydraulic capacity to safely pass the SDF. The improvements to rectify these two issues may be addressed as a single project by flattening the downstream slope and installing overtopping protection in the form of ACB matting. Construction of these improvements will also inherently address most of the maintenance and repair concerns as well. A summary of all recommended actions is provided below:

Investigations:

- Temporarily lower Pond to investigate condition and slope of upstream embankment face
- Inspect principal spillway structure, pipe, and bottom drain; anticipate replacement of bottom drain gate and appurtenances
- Investigate source of seepage emanating from below outlet channel concrete lining
- Perform additional geotechnical soil borings on and downstream of embankment to support design and construction of overtopping protection and flattening of downstream face
- Investigate Quail Hollow Drive culvert to ascertain if flow is occurring below pipe

General Repair / Maintenance:

- Remove trees, brush, and woody vegetation from the embankment and within 15-feet of any part of the dam (left and right abutments, downstream toe, and overflow areas)
- Remove bench from south auxiliary overflow spillway
- Clear and regrade area around north auxiliary pipe spillway
- Replace Quail Hollow Drive culvert endwalls
- Remove downed tree across channel downstream of Quail Hollow Drive culvert
- Stabilize channel and banks downstream of Quail Hollow Drive culvert

Stability and Overtopping Improvements:

- Remove grouted riprap on upstream face and concrete slabs on top of dam
- Eliminate area drains and associated pipes on top of dam
- Locate and remove embankment toe drains
- Flatten downstream embankment slope to no steeper than 2.75H:1V
- Install embankment blanket and toe drains
- Construct retaining wall around principal spillway outlet
- Install ACB overtopping protection

Appendix A: Reference Documents

1974 Improvements

EASTGATE PARK

RALEIGH PARKS AND RECREATION DEPARTMENT - RALEIGH, N.C.

PHASE ONE DEVELOPMENT

LEGEND

- EXISTING CONTOURS
- EXISTING SPOT ELEVATIONS
- PROPERTY LINE
- EXISTING CURB LINE W/ VANICLE (AS NOTED)
- EXISTING WATER/SEWER SERVICE LINES TO COMMUNITY CENTER
- EXISTING ELECTRICAL SERVICE (OVERHEAD CABLE)
- LIMIT OF REPORTED AREAS
- 3:1 OR SIMILAR ARE EXISTING OR PROPOSED GRADING STATED AS RATIO & WALL KIND THEREOF
- PROPOSED CONTOURS
- PROPOSED SPOT ELEVATIONS
- PROPOSED OR EXISTING STORM DRAINAGE LINES W/ RESPECTIVE, NO NOTED (EXISTING NOTED AS "EXIST. NO." WHERE APPLICABLE).
- PROPOSED EROSION BLANKET, AS NOTED
- PROPOSED TEMPORARY FENCING, HEIGHT & LENGTH AS NOTED.
- PROPOSED TEMPORARY PROTECTIVE FENCING, AS SPECIFIED.
- PROPOSED LIMIT OF GRADING OR LIMIT OF CONSTRUCTION LINES
- PROPOSED "LIMIT OF ALTERNATE LINES", ALTERNATE NO. AS NOTED

SPECIAL NOTES

"A" GRADE AROUND SELECTED TREES TO REMAIN AS DIRECTED BY LANDSCAPE ARCHITECT.

"B" UNCLASSIFIED EXCAVATED EARTH MATERIAL FROM GRADING OF WEST ENTRANCE DRIVE (BOX CUT) TRANSPORTED TO TEMPORARY TOPSOIL STOCKPILE AREA & DUMPED ON GRADE; INCORPORATE DUMPED MATERIAL INTO FINAL GRADING OF BORROW AREA.

"C" FENCE: 6'H., 10" DIAMETER TREE WELLS (1.0' CHANGE IN GRADE) AS SHOWN ON DETAIL 10 & 11 SHEET No. 6. SIX TREES TO BE WELLED SHALL BE DESIGNATED BY LANDSCAPE ARCHITECT AND GRADING OPERATORS SHALL NOT BE PERMITTED WITHIN 100' OF DESIGNATED SIX TREES UNTIL TREE WELLS ARE BUILT. TO BE RELOCATED IN ALTERNATE "A" & "B" AS PART "A".

"D" PLAY AREA NO. 6: PROVIDE 6" SAND BED AS INDICATED; PROVIDE 3/4" TREATED TIMBER BOARDS BETWEEN SAND BED & ADJACENT GRADES, PROVIDE 6" DIA. TIMBER POST OR REINFORCED CONCRETE PIPE 8' LONG ON GRADE AS DIRECTED BY LANDSCAPE ARCHITECT.

"E" CONTRACTOR LOCATE ALL UNDERGROUND UTILITY LINES PRIOR TO BEGINNING ANY CLEARING OR GRADING WORK IN IMMEDIATE AREA. LOCATIONS INDICATED ARE APPROXIMATE. WHERE CONFLICTS ARISE BETWEEN THE WORK & EXISTING OVERHEAD UTILITY LINES, PROPOSED GRADING WILL BE ADJUSTED AS APPROVED BY THE LANDSCAPE ARCHITECT.

GENERAL NOTES

SEE "STAKING & LAYOUT PLAN" SHT. NO. 1, FOR DIMENSIONED LAYOUT OF WORK.

EXCESS CUT OR FILL RESULTING FROM THE GENERAL GRADING OPERATION SHALL BE COMPENSATED FOR BY ADJUSTING THE FINAL GRADES IN THE OTHER COMPLETION TOPSOILED PLAYFIELD AREAS AFTER THE ROUGH GRADING OPERATION IS COMPLETED. GRADES SHALL BE ADJUSTED ACROSS THE FIELD AS APPROVED BY THE LANDSCAPE ARCHITECT.

PROVIDE SILT STRAW BALES & TYPE D CONTINUOUSLY ALONG TOE OF ALL SLOPES WITHIN LIMIT OF GRADING UNDER WHERE SLOPES ARE EQUAL TO OR GREATER THAN 3:1. BALES SHALL BE STANDARD SIZE & SHALL REMAIN IN PLACE UNTIL GRADES IS ESTABLISHED IN ACCORDANCE WITH TEMPERATION NOTES.

ALL GRADED SURFACES WITHIN LIMITS OF GRADING SHALL RECEIVE 4" MINIMUM TOPSOIL & PERMANENT GRASSING IN ACCORDANCE WITH SPECIAL AREA CONSTRUCTION OF ALL OTHER WORK AND UTILITIES OTHERWISE COVERED BY PAVING, WATER IMPROVEMENTS, ETC.

OLD MAN'S LAKE

QUAIL HOLLOW DRIVE

DRAWN	
CHECKED	GWL
APPROVED	

GRADING & SITE PLAN

REVISIONS

2

PROFILE SECTION ALONG ℄ OF MAIN PEDESTRIAN CONCOURSE SCALE: VERT. 1"=10.0' HOR. 1"=20.0'

PROFILE ELEVATION OF EXISTING EMBANKMENT w/ SECTION OF NEW EMBANKMENT
SCALE: VERT. 1"=10.0' HOR. 1"=20.0'

PROFILE SECTION @ CENTERLINE OF NEW SUPPLEMENTARY EARTHFILL EMBANKMENT
SCALE: VERT. 1"=10.0' HOR. 1"=20.0'

EASTGATE PARK

RALEIGH PARKS AND RECREATION DEPARTMENT - RALEIGH, N.C.

PHASE ONE DEVELOPMENT

GEOFFREY McLEAN
landscape architect
(919) 787-6691 ● 2517 fairview rd.
raleigh ● north carolina ● 27608

DRAWN BWL-SPA
CHECKED GML
APPROVED

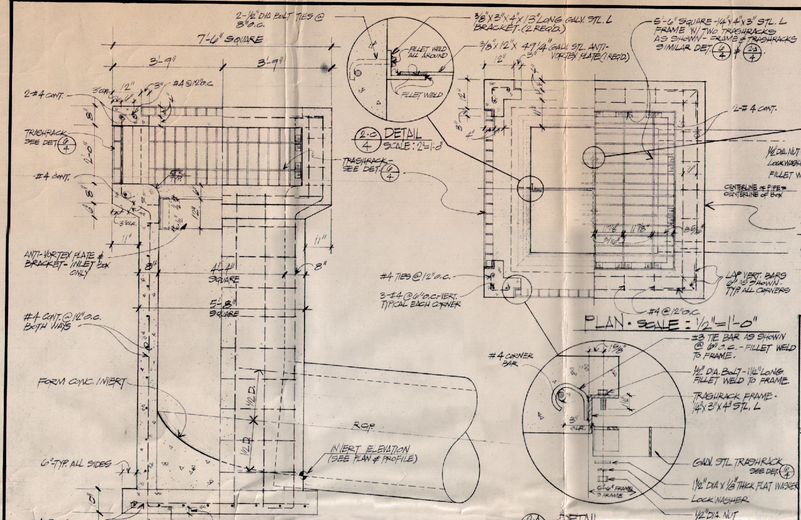
EMBANKMENT, PLAN & PROFILES

REVISIONS

CONSULTING ENGINEERS
GEOTECHNICAL ENGINEERING COMPANY
ULMSTEAD INDUSTRIAL PARK
RALEIGH, N.C.

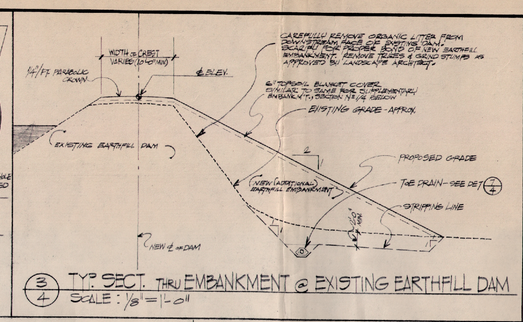
SHEET NO. 3 OF SIX

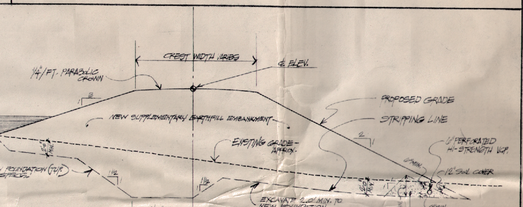
NAV 15/74

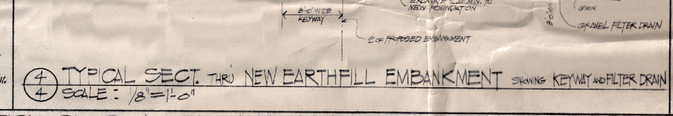


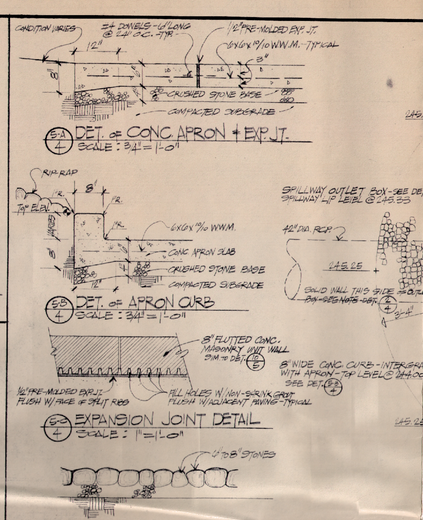
2/4 DETAIL OF SPILLWAY INLET/OUTLET BOX
SCALE: AS SHOWN

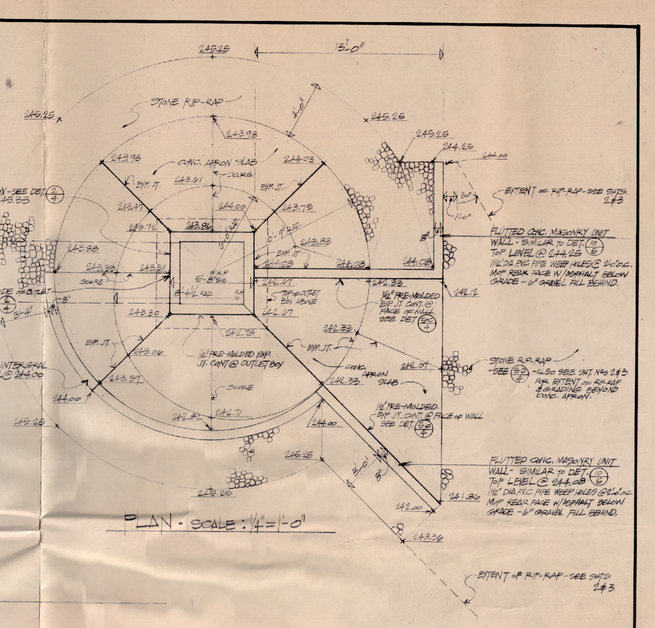
NOTES:
- ALL EXPOSED EDGES OF INLET & OUTLET BOX SHALL HAVE 1/2" CHAMFER UNLESS OTHERWISE NOTED.
- SUBMIT SHOP DRAWINGS FOR CONNECTIONS & INSTALL.

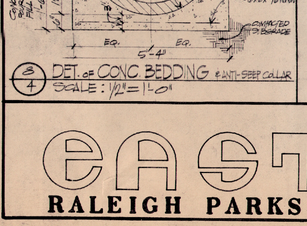






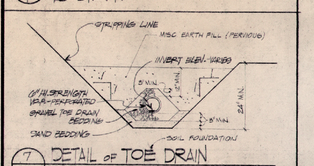




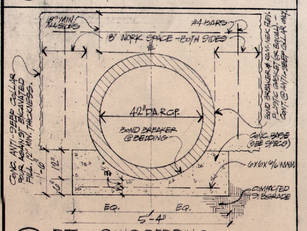


NOTES:
- BLOCK, GRAND AND LUGS ON TRASHRACKS TO BE GALVANIZED STEEL AS PER SPECIFICATIONS
- NOTCHING IN VERTICAL STRAPS TO HOLD HOOKS ON INLET BOX AND THREAD OUTSIDE OF OUTLET BOX.

4/4 GALV. TRASHRACK DETAIL
AS SHOWN



7/4 DETAIL OF TOE DRAIN
SCALE: 1/2"=1'-0"



8/4 DET. OF CONC BEDDING & ANTI-SEEP COLLAR
SCALE: 1/2"=1'-0"

1/4 PROFILE OF SPILLWAY OUTLET WORKS AND TYPICAL SECTION OF EARTHFILL EMBANKMENT
HORIZONTAL SCALE: 1/4"=1'-0" VERTICAL SCALE: 1/2"=1'-0"

EASTGATE PARK

RALEIGH PARKS AND RECREATION DEPARTMENT - RALEIGH, N.C.

PHASE ONE DEVELOPMENT

GEOFFREY McLEAN
landscape architect
(919) 787-6691 ● 2517 fairview rd.
raleigh ● north carolina ● 27608

DRAWN JPL
CHECKED JPL
APPROVED

EMBANKMENT SECTIONS & DETAILS

REVISIONS

CONSULTING ENGINEERS
GEOTECHNICAL ENGINEERING COMPANY
LUMSTEAD INDUSTRIAL PARK
RALEIGH, N.C.

SHEET NO: 4 OF SIX
MAY 15, 1974

℄ WINGATE DRIVE
(60' RIGHT of WAY)

LOT 29

EXISTING 30" RCP
& PIPE = ℄

LOT 30

BLOCK H
(HICKORY HILLS SUBDIVISION)

PROPOSED NEW TEMPORARY WIRE FENCE

EXISTING 20' WIDE
DRAINAGE EASEMENT
(PERMANENT)

LOT 31

LOT 32

PROPOSED NEW ACCESS EASEMENT
(TEMPORARY)

15' TYP.

PROP. OF CITY OF RALEIGH

SEDIMENT TRAP & LAGOON

GRID NORTH

EXISTING CHAIN LINK FENCE
INSTALLED BY OWNER

LIMIT OF ALTERNATE No. 1

TEMPORARY ACCESS ROAD
FOR ALTERNATE No. 1
(ABANDON AS PER CHANGE ORDER)

(ABANDON AS PER CHANGE ORDER)
TEMPORARY WIRE FENCE

ED MAN'S LAKE

NOTE:

- TREES WITHIN TEMPORARY ACCESS EASEMENT HAVING A TRUNK DIAMETER OF 6 INCHES OR LARGER WILL BE REMOVED ONLY UPON THE PRIOR INSPECTION & APPROVAL OF THE LANDSCAPE ARCHITECT.

EASTGATE PARK
PHASE ONE DEVELOPMENT

REVISED ACCESS EASEMENT TO SEDIMENT TRAP & LAGOON AREA

SCALE:	DRAWN BY:	CHECKED:	REVISED:	DATE:	SHEET No:
1" = 40'	SS. /JFA			6 AUG. 74	WO-1a

GENERAL NOTES:

1. TYP. SECT THRU EMBANKMENT @ EXISTING EARTHFILL DAM SHOWING PROPOSED KEYWAY AND FILTER DRAIN
2. THE ABOVE KEYWAY WILL RUN ALONG ENTIRE LENGTH OF DOWNSTREAM FACE OF EXISTING DAM AND BE TIED TO THE KEYWAY OF NEW EARTHFILL EMBANKMENT AS SHOWN ON SHEET No 4 OF 6 IN PLANS
3. SEE REQUIREMENTS OF ACCOMPANYING WRITTEN ORDER No. 1
4. ALL EARTHFILL-IN-EMBANKMENT & EARTH REFILL SHALL BE COMPACTED IN ACCORDANCE WITH ASTM D-698 FOR 95% COMPACTIVE STRENGTH

GEOFFRET McLEAN & Co.
LANDSCAPE ARCHITECTS
257 FAIRVIEW ROAD · RALEIGH, N.C.

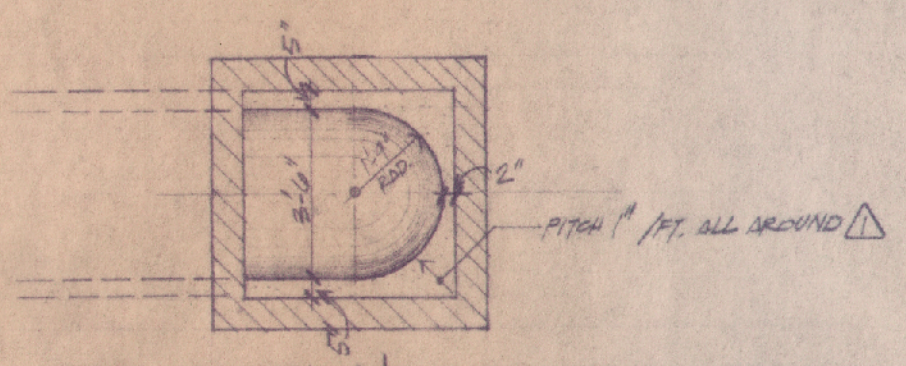
SCALE NONE | DRN. KEH/JEA | CHKD | DATE JUNE 74 | REV.

SHEET WO-1b

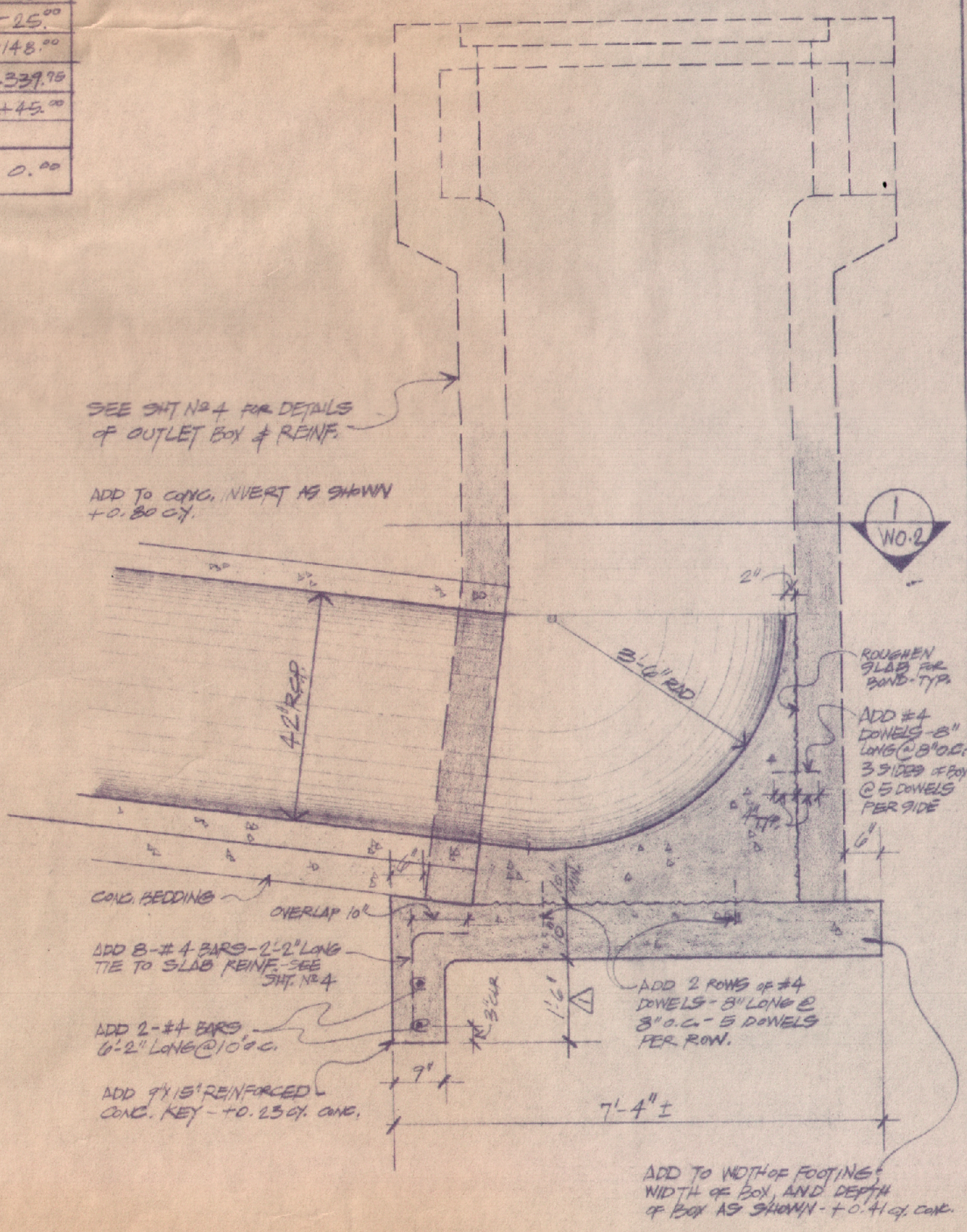
EASTGATE PARK
PHASE ONE DEVELOPMENT

REVISED SECTION (3/4) SHOWING NEW KEYWAY AS PROPOSED BY CONTRACTOR

OULE		
NET CHANGE	UNIT PRICE	COST ADJUST.
31 BLK	200.00/100 BLK	- 62.00
38 BLK	175.00/100 BLK	- 68.50
30 BLK	175.00/100 BLK	- 52.50
BLK.	150.00/100 BLK	- 22.50
C.Y.	200.00/C.Y.	- 8.00
11.75 C.Y.	6.00/C.Y.	+ 70.50
25 C.Y.	2.75/C.Y.	- 68.75
10 C.Y.	2.50/C.Y.	- 25.00
37.0 L.F.	4.00/L.F.	- 148.00
45.1 S.Y.	2.25/S.Y.	+ 339.75
2.0 S.Y.	2.25/S.Y.	+ 45.00
TOTAL		0.00



1 / WO-2 SECTION SHOWING PLAN OF CONC. INVERT
SCALE : 1/4" = 1'-0"



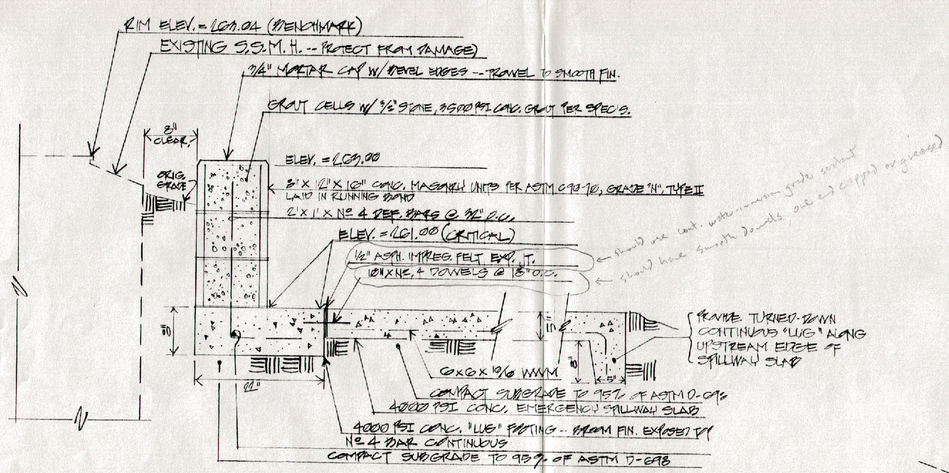
WO-2a ADJUSTMENTS TO SPILLWAY OUTLET BOX
SCALE : 1/2" = 1'-0"

EASTGATE PARK
PHASE ONE DEVELOPMENT

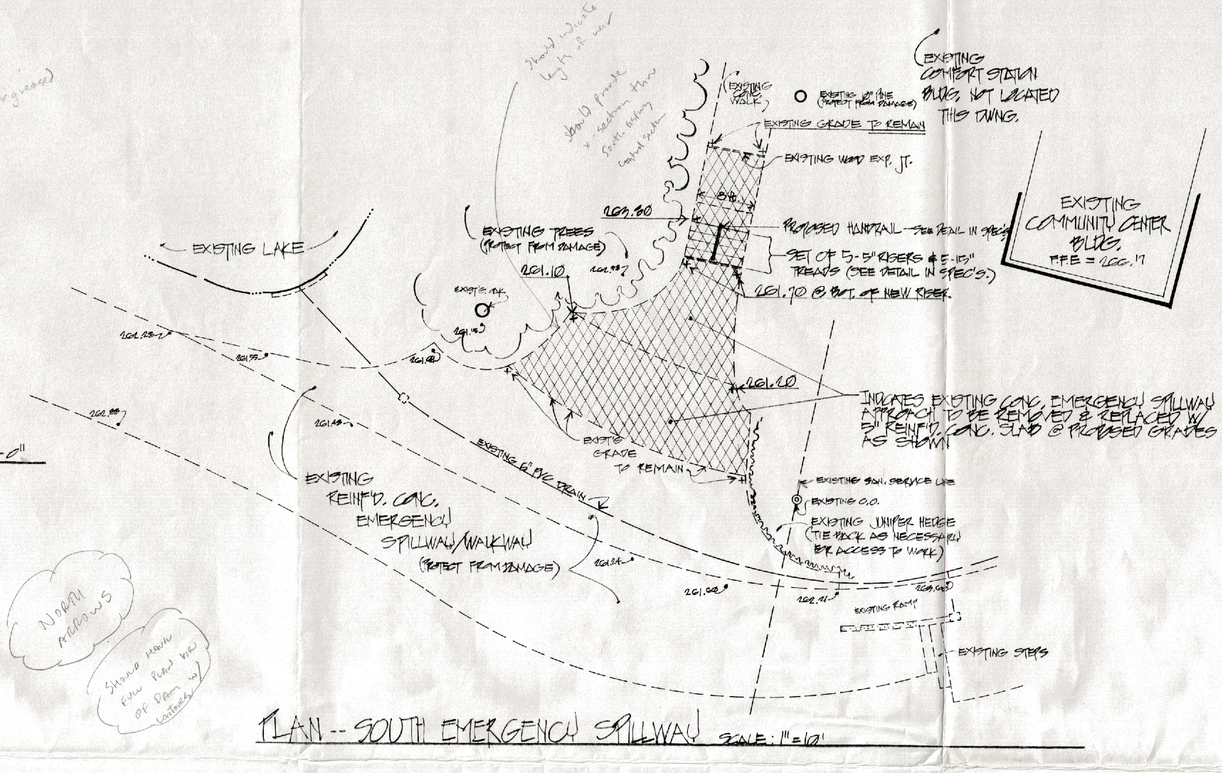
NORTH CAROLINA REGISTERED LANDSCAPE ARCHITECT 21 GEOFFREY D. McLEAN

DRAWN BY: JFA	SCALE: AS SHOWN	REVISIONS: 10/23/74 EXTEND CONC. KEY ON OUTLET BOX TO 1'-6" GEN. REV. ⚠	GEOFFREY McLEAN landscape architect RALEIGH	SHEET No WO-2
CHECKED BY:	DATE: 14 OCT 74	CONSULTING ENGINEERS GEOTECHNICAL ENGINEERING CO. UMSTEAD INDUSTRIAL PARK RALEIGH, N.C.		

1990 Improvements

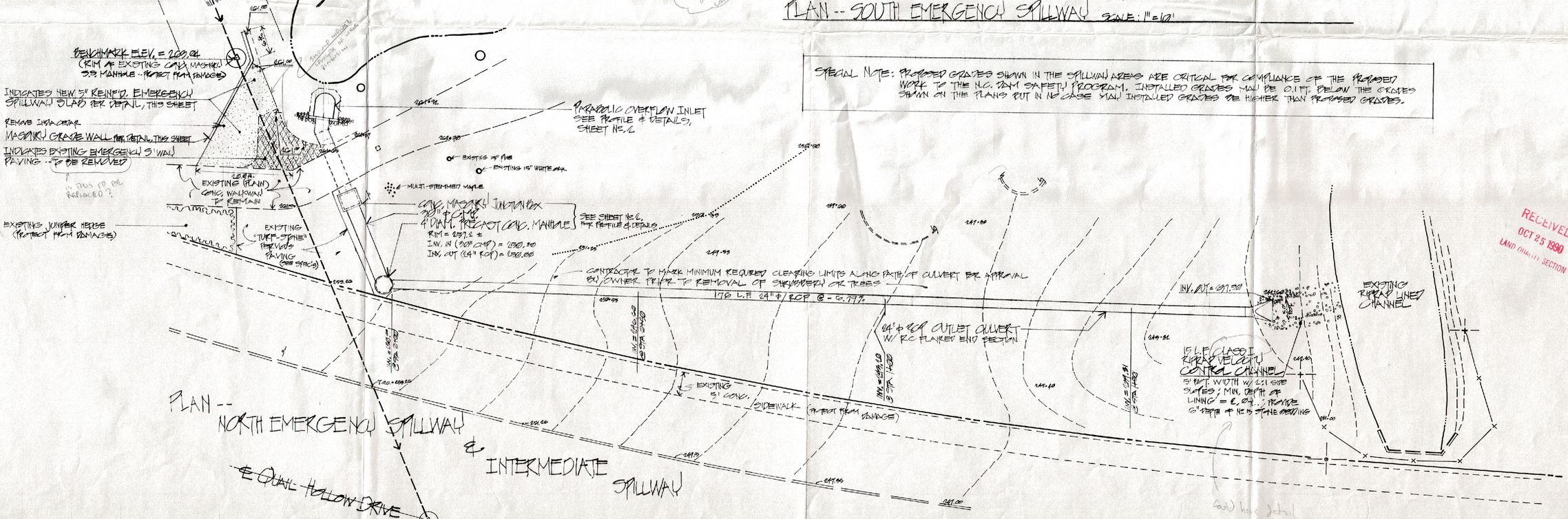


DETAIL of EMERGENCY SPILLWAY SLAB & MASONRY GRADE WALL SCALE 1" = 1'-0"



PLAN -- SOUTH EMERGENCY SPILLWAY SCALE: 1" = 10'

SPECIAL NOTE: PROPOSED GRADES SHOWN IN THE SPILLWAY AREAS ARE CRITICAL FOR COMPLIANCE OF THE PROPOSED WORK TO THE N.C. DAM SAFETY PROGRAM. INSTALLED GRADES MAY BE 0.1 FT. BELOW THE GRADES SHOWN ON THE PLANS BUT IN NO CASE MAY INSTALLED GRADES BE HIGHER THAN PROPOSED GRADES.



PLAN -- NORTH EMERGENCY SPILLWAY & INTERMEDIATE SPILLWAY

DATE: SEPT. 4, 1990

EMERGENCY SPILLWAY MODIFICATIONS & INTERMEDIATE SPILLWAY

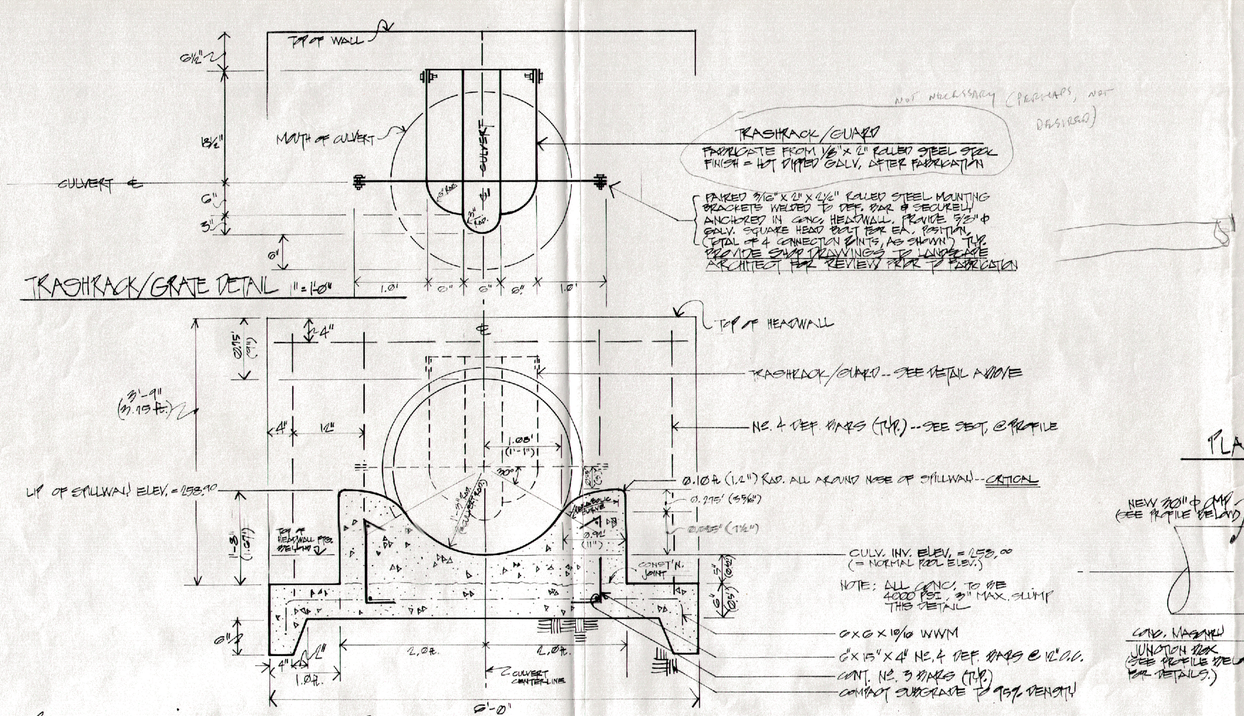
EASTGATE PARK LAKE

CITY OF RALEIGH PARKS & RECREATION DEPT., OWNER

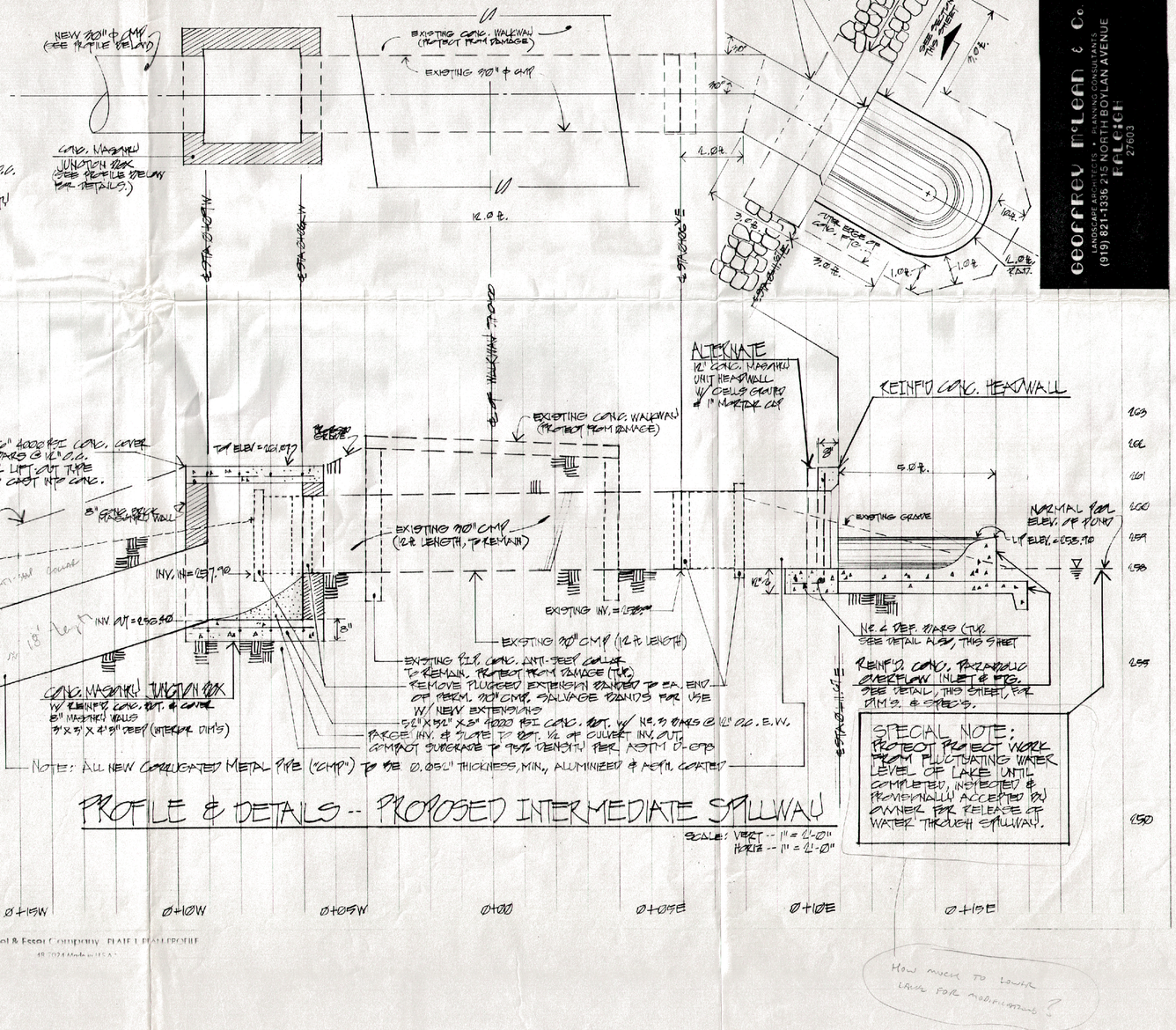
Sheet No. 1

CONSULTING ENGINEER
Charles A. Thomas, PE
3109 MONTICELLO DRIVE
RALEIGH, NORTH CAROLINA 27612
(919) 781-6824 or 821-1336

geoffrey McLean & Co.
LANDSCAPE ARCHITECTS • PLANNING CONSULTANTS
(919) 821-1336 215 NORTH BOYLAN AVENUE
RALEIGH 27603



COMPUTED STAGE - DISCHARGE VALUES for OVERFLOW INLET & CULVERT



EMERGENCY SPILLWAY MODIFICATIONS & INTERMEDIATE SPILLWAY
EASTGATE PARK LAKE
CITY OF RALEIGH PARKS & RECREATION DEPT., OWNER
DATE: SEPT. 4, 1990
SHEET NO. 2

CONSULTING ENGINEER
Charles A. Thomas, PE
3109 MONTICELLO DRIVE
RALEIGH, NORTH CAROLINA 27612
(919) 781-6824 or 821-1336

geoffrey mclean & Co.
LANDSCAPE ARCHITECTURE & CONSULTANTS
1410 BOYLAN AVENUE
(919) 821-1335 215 NORTH BOYLAN AVENUE
RALEIGH
27603

Knuffel & Esser Company FLAT FILE PROFILE
49-7024 Made in U.S.A.

Project Survey

(Provided as electronic copy in DWG format)

Appendix B: Condition Assessment

Location: South Abutment **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Abutment Contact	Large-Diameter Trees	Ground Stability

.

Location: South Auxiliary Overflow Spillway (Upstream Approach) **Direction:** Northeast

Area(s) / Component(s):	Observation(s):	Concern(s):
Approach	Bench	Obstruction to Flow
	Lack of Ground Cover, Sheet Erosion	Erosion
Concrete Slab	Improper Leading Edge	Slab Uplift / Failure, Obstruction to Flow
	Improper Joint	Slab Uplift / Failure
	Possible Abandoned Area Drain	Slab Uplift / Failure

Location: South Auxiliary Overflow Spillway (Crest / Top of Dam) **Direction:** Northwest

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joint	Slab Uplift / Failure
	Transverse Fracture	Slab Uplift / Failure
Downstream Groin Downstream Area	Large-Diameter Trees	Obstruction to Flow, Ground Stability
	Lack of Ground Cover	Erosion

Location: Top of Dam near South Auxiliary Spillway **Direction:** North/West

Area(s) / Component(s):	Observation(s):	Concern(s):
Area Drain (Typical of Multiple)	Drain in Dam Top	Slab Uplift / Failure
	Drain Outlet through Upstream Slope	Soil Piping / Stability
	Grouted Riprap Deterioration	Slope Stability, Material Mobilization

Location: Top of Dam near Midpoint of Added Embankment (1974 Improvements) **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Transverse, Longitudinal, and Radial Fractures	Slab Uplift / Failure

Location: Top of Dam near Midpoint of Added Embankment (1974 Improvements) **Direction:** North/South

Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face, Toe, and Groin	Trees / Woody Vegetation	Ground Stability, Soil Piping
	Weedy Vegetation	Growth Competition, Erosion
	Bare Areas	Erosion

Location: Top of Dam near Midpoint of Added Embankment (1974 Improvements) **Direction:** East

Area(s) / Component(s):	Observation(s):	Concern(s):
Primary Spillway Trash Rack	Minor Vegetation and Debris	Reduced Hydraulic Capacity
Gate	Operability, Type, and Condition Unknown	Emergency Drawdown Capability

Location: Top of Dam near Midpoint of Embankment **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Transverse, Longitudinal, and Radial Fractures	Slab Uplift / Failure
Upstream Face	Large-Diameter Trees	Slope Stability, Soil Piping
	Light Pole	Slope Stability

Location: Top of Dam at Midpoint of Embankment **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Transverse, Longitudinal, and Radial Fractures	Slab Uplift / Failure
	Settlement	Soil Stability / Mobilization
Upstream Face	Large-Diameter Trees	Slope Stability, Soil Piping

Location: Upstream Face at Midpoint of Embankment **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Grouted Riprap	Fractures	Uplift, Material Mobilization, Slope Stability
	Voids	Uplift, Material Mobilization, Slope Stability
	Large-Diameter Trees	Slope Stability, Soil Piping, Grout Damage

Location: Top of Dam at Midpoint of Embankment **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face and Toe	Bare Areas	Erosion
	Weedy Vegetation	Growth Competition, Erosion
	Large-Diameter Trees	Slope Stability, Soil Piping
Concrete Slab	Improper Trailing edge	Erosion

Location: Top of Dam near Midpoint of Embankment **Direction:** Southwest

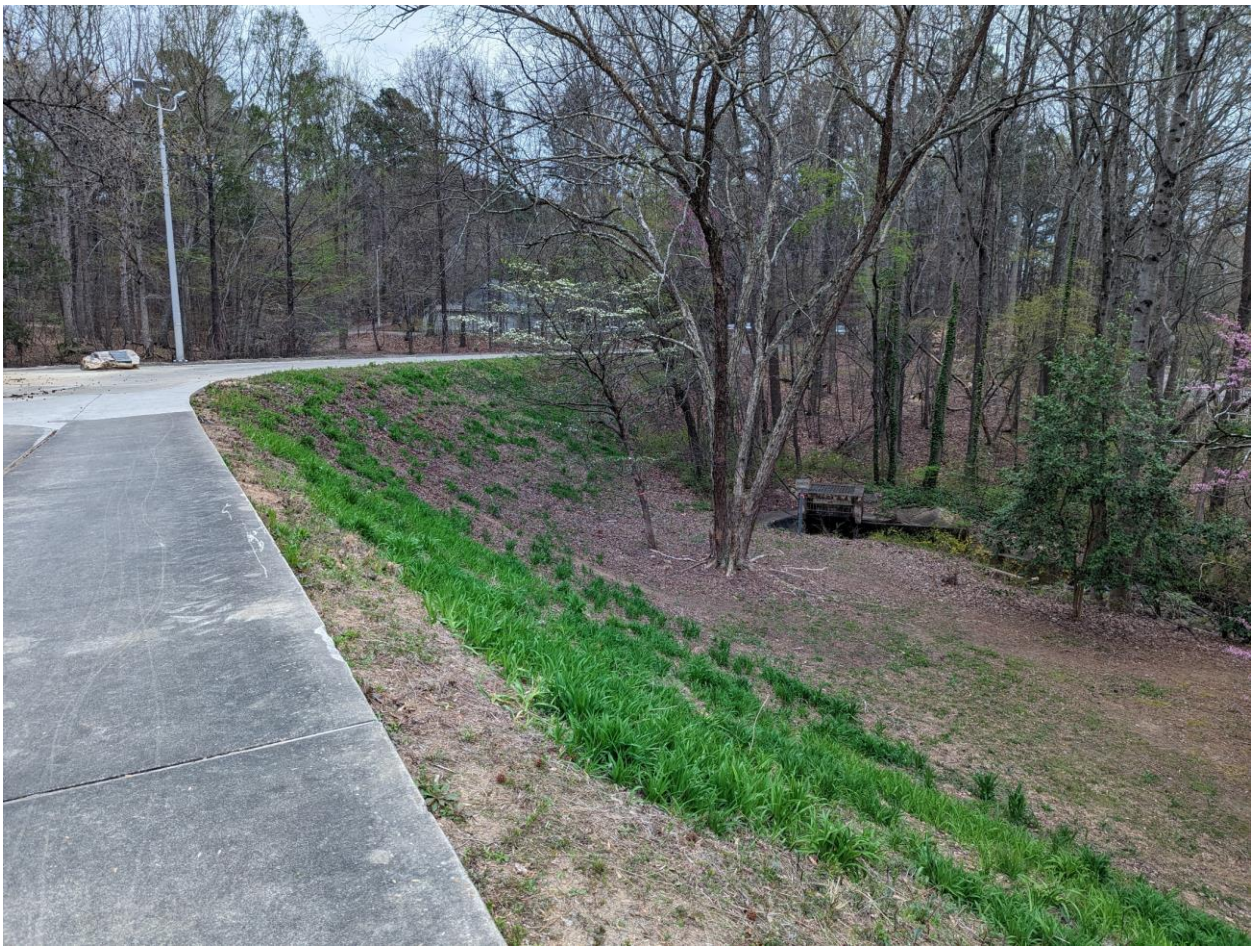
Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face and Toe, and Outlet Structure	Bare Areas	Erosion
	Weedy Vegetation	Growth Competition, Erosion
	Large-Diameter Trees	Slope Stability, Soil Piping

Location: Top of Dam near Midpoint of Embankment **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure

Location: Top of Dam near Scenic Overlook **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Longitudinal Fractures	Slab Uplift / Failure



Location: Top of Dam near Scenic Overlook **Direction:** South

Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face and Toe, and Outlet Structure	Bare Areas	Erosion
	Weedy Vegetation	Growth Competition, Erosion
	Large-Diameter Trees	Slope Stability, Soil Piping

Location: Top of Dam at Scenic Overlook **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Longitudinal and Radial Fractures	Slab Uplift / Failure

Location: Top of Dam at Scenic Overlook **Direction:** West

Area(s) / Component(s):	Observation(s):	Concern(s):
Block Wall	Fracture	Wall Failure, Material Loss

Location: Top of Dam near Scenic Overlook / North Auxiliary Overflow Spillway **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Longitudinal, Transverse, and Radial Fractures	Slab Uplift / Failure

Location: Top of Dam near Scenic Overlook / North Auxiliary Overflow Spillway **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Improper Joints	Slab Uplift / Failure
	Longitudinal, Transverse, and Radial Fractures	Slab Uplift / Failure
Upstream Face	Large-Diameter Trees	Slope Stability, Soil Piping, Grout Damage

Location: North Auxiliary Overflow Spillway / Downstream Face **Direction:** Southwest

Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face	Bare Area, Sheet Erosion	Erosion
	Improper Armoring	Erosion
Concrete Slab	Improper Trailing Edge	Erosion

Location: North Abutment / North Auxiliary Pipe Spillway **Direction:** South

Area(s) / Component(s):	Observation(s):	Concern(s):
Spillway Inlet	Debris Accumulation	Reduced Hydraulic Capacity
	Trees / Woody Vegetation	Slope Stability, Soil Piping

Location: Downstream Toe near North Abutment **Direction:** South

Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face and Toe	Debris Accumulation	Growth Competition, Erosion
	Trees	Slope Stability, Soil Piping

Location: Outlet Structure **Direction:** South

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Slab	Slab Displacement	Slab Uplift /Failure
	Fractures	Slab Uplift / Failure
Adjacent Area	Trees	Ground Stability, Soil

Location: Outlet Structure **Direction:** South

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Apron	Slab Displacement	Slab Uplift /Failure
	Fractures	Slab Uplift / Failure

Location: Outlet Structure **Direction:** West

Area(s) / Component(s):	Observation(s):	Concern(s):
Adjacent Area	Trees / Woody Vegetation	Ground Stability
	Lack of Ground Cover	Erosion

Location: Outlet Channel near Outlet Structure **Direction:** East

Area(s) / Component(s):	Observation(s):	Concern(s):
Adjacent Area	Trees / Woody Vegetation	Ground Stability
	Lack of Ground Cover	Erosion

Location: Outlet Channel at end of Concrete Apron **Direction:** South

Area(s) / Component(s):	Observation(s):	Concern(s):
Concrete Apron	Heavy, Clear Seepage	Unknown Origin, Soil Piping, Material Loss
	Trees	Slab Uplift, Ground Stability

Location: Downstream of Dam near Outlet Channel **Direction:** East

Area(s) / Component(s):	Observation(s):	Concern(s):
Downstream Face, Groin, and Toe	Lack of Ground Cover	Erosion
	Trees	Slope Stability, Soil Piping
	Weedy Vegetation	Growth Competition, Erosion

Location: Outlet Channel near Quail Hollow Drive Culvert **Direction:** North

Area(s) / Component(s):	Observation(s):	Concern(s):
North Auxiliary Pipe Spillway	None	None

Location: Quail Hollow Drive Culvert (Upstream) **Direction:** West

Area(s) / Component(s):	Observation(s):	Concern(s):
Endwall	Fractures	Wall Failure
	Horizontal Displacement	Wall Failure

Location: Quail Hollow Drive Culvert (Downstream) **Direction:** Northeast

Area(s) / Component(s):	Observation(s):	Concern(s):
Endwall	Fractures	Wall Failure
	Horizontal Displacement	Wall Failure
Adjacent Area	Lack of Ground Cover	Erosion
	Trees / Woody Vegetation	Ground and Wall Stability, Root Intrusion
	Downed Tree	Reduced Hydraulic Capacity

Location: Quail Hollow Drive Culvert **Direction:** West / East

Area(s) / Component(s):	Observation(s):	Concern(s):
Culvert	Outflow Appears to Exceed Inflow Visible Change in Flow inside Pipe	Inspectors noted that culvert outflow appeared to exceed outflow, possibly indicating flow may be occurring below the pipe. Additionally, there is a substantial change in culvert flow velocity and depth between the inlet and outlet (however, this may be due to a change in slope or debris obstructing flow)

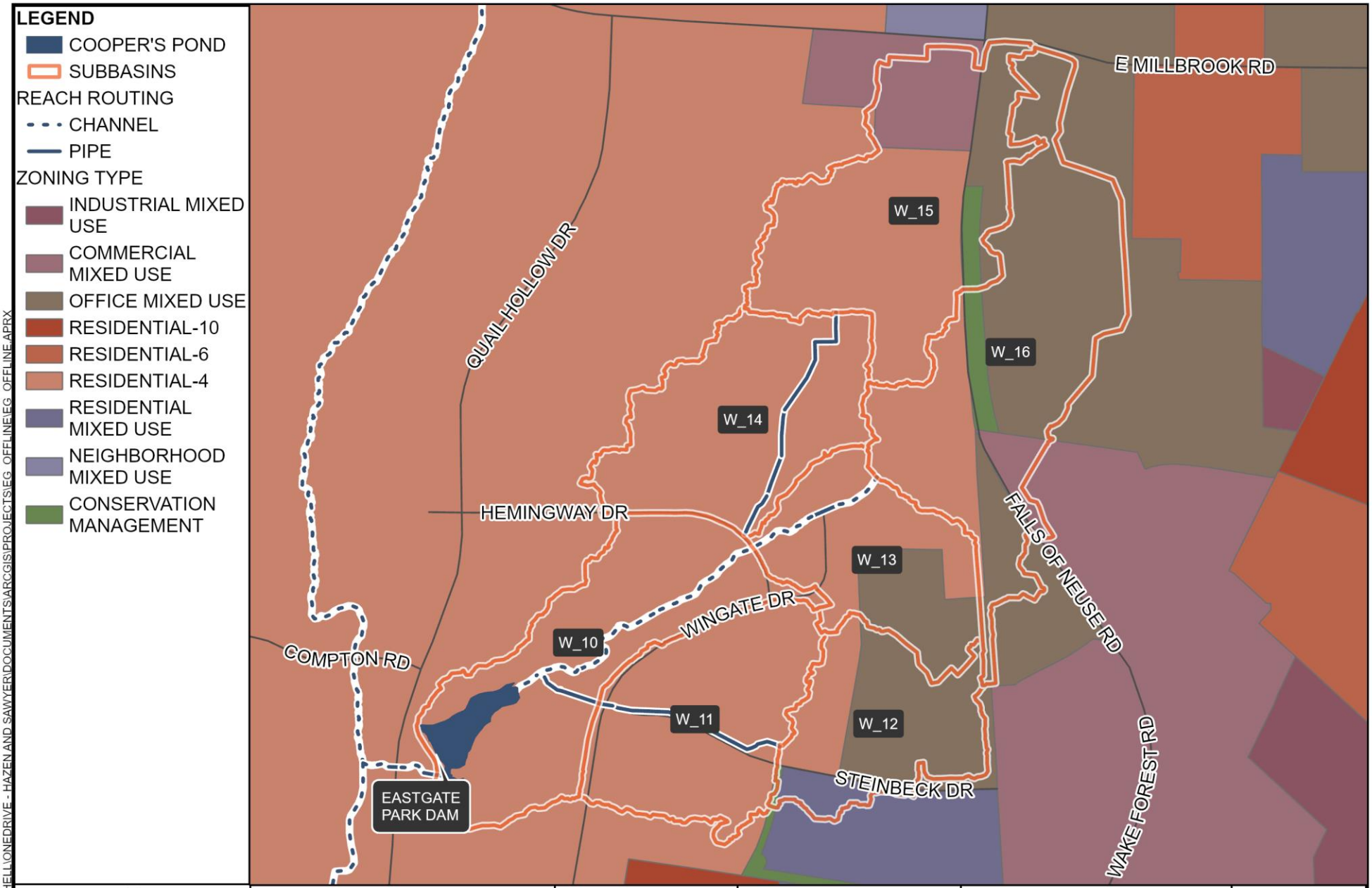
Location: Downstream of Quail Hollow Drive Culvert **Direction:** Northwest

Area(s) / Component(s):	Observation(s):	Concern(s):
Channel	Bank Erosion	Erosion, Property Damage

Appendix C: Hydrologic Analysis

Future Landuse Runoff Curve Numbers

Zone	NEH Landuse	CNs for Hydrologic Soil Group			
		A	B	C	D
Conservation Management	**Open Space (Good)**	39	61	74	80
Commercial Mixed Use	**Urban (Commercial)**	89	92	94	95
Industrial Mixed Use	**Urban (Industrial)**	81	88	91	93
Office Park	**Urban (Industrial)**	81	88	91	93
Office Mixed Use	**Urban (Industrial)**	81	88	91	93
Planned Development	**Urban (Industrial)**	81	88	91	93
Neighborhood Mixed Use	**Residential - 1/8 acre**	77	85	90	92
Residential-10	**Residential - 1/8 acre**	77	85	90	92
Residential-6	**Residential - 1/8 acre**	77	85	90	92
Residential Mixed Use	**Residential - 1/8 acre**	77	85	90	92
Residential-4	**Residential - 1/4 acre**	61	75	83	87
Residential-2	**Residential - 1/2 acre**	54	70	80	85



N

0 350 700 FEET

1" = 700'

Hazen

4011 WESTCHASE BLVD
SUITE 500
RALEIGH, NC 27607

Raleigh

ENGINEERING SERVICES

EASTGATE DAM PER

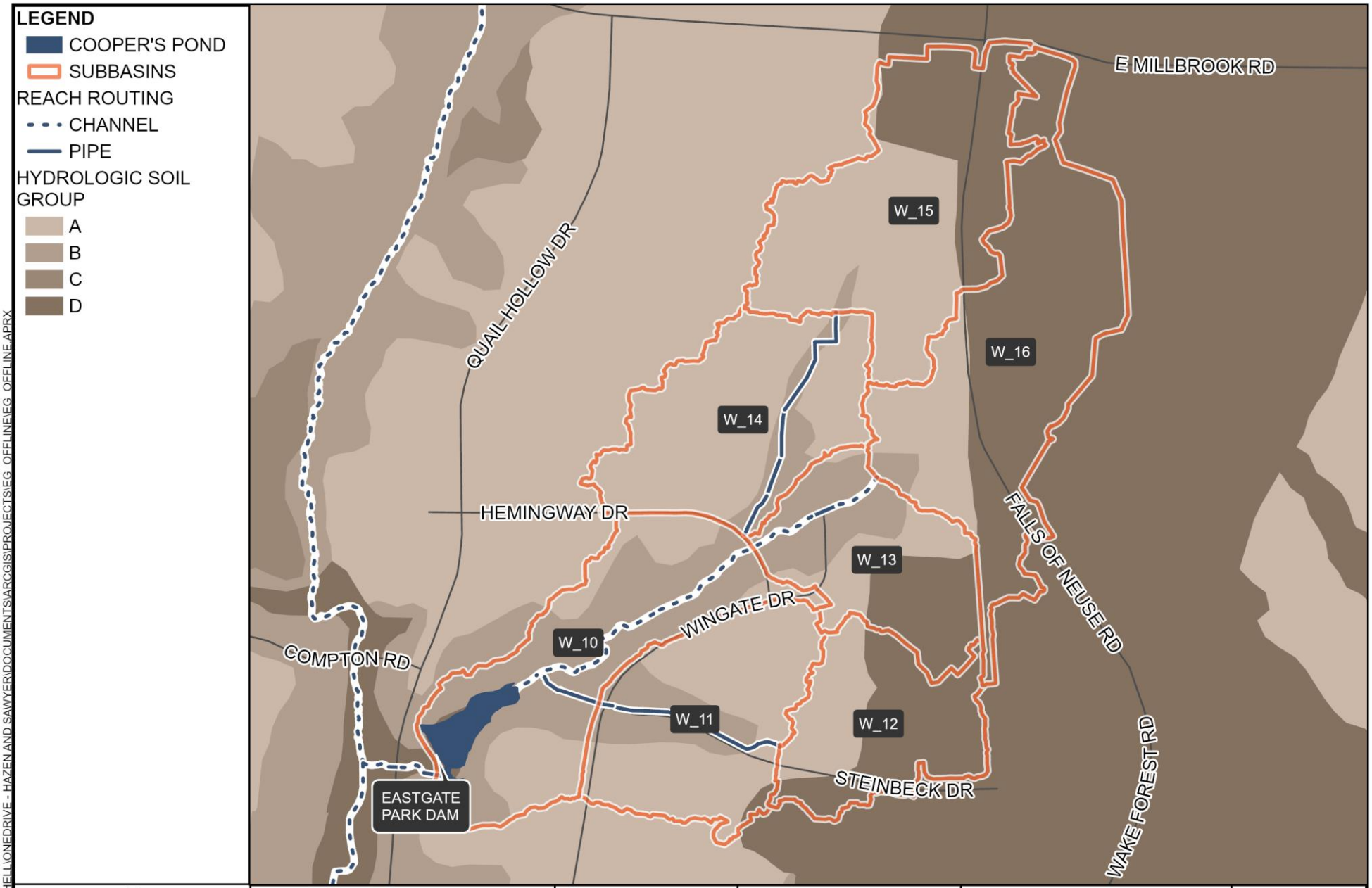
ZONING MAP

DATE: JUNE 2023

FIGURE:

PATH: C:\USERS\GMITCHELL\ONEDRIVE - HAZEN AND SAWYER\DOCUMENTS\ARCGIS\PROJECTS\EG_OFFLINE\EG_OFFLINE.APRX

REFERENCE: ROAD NAMES, ZONING (WAKE COUNTY)



PATH: C:\USERS\GMITCHELL\ONEDRIVE - HAZEN AND SAWYER\DOCUMENTS\ARCGIS\PROJECTS\EG_OFFLINE\EG_OFFLINE.APRX

N

0 350 700 FEET

1" = 700'

Hazen

4011 WESTCHASE BLVD
SUITE 500
RALEIGH, NC 27607

Raleigh

ENGINEERING SERVICES

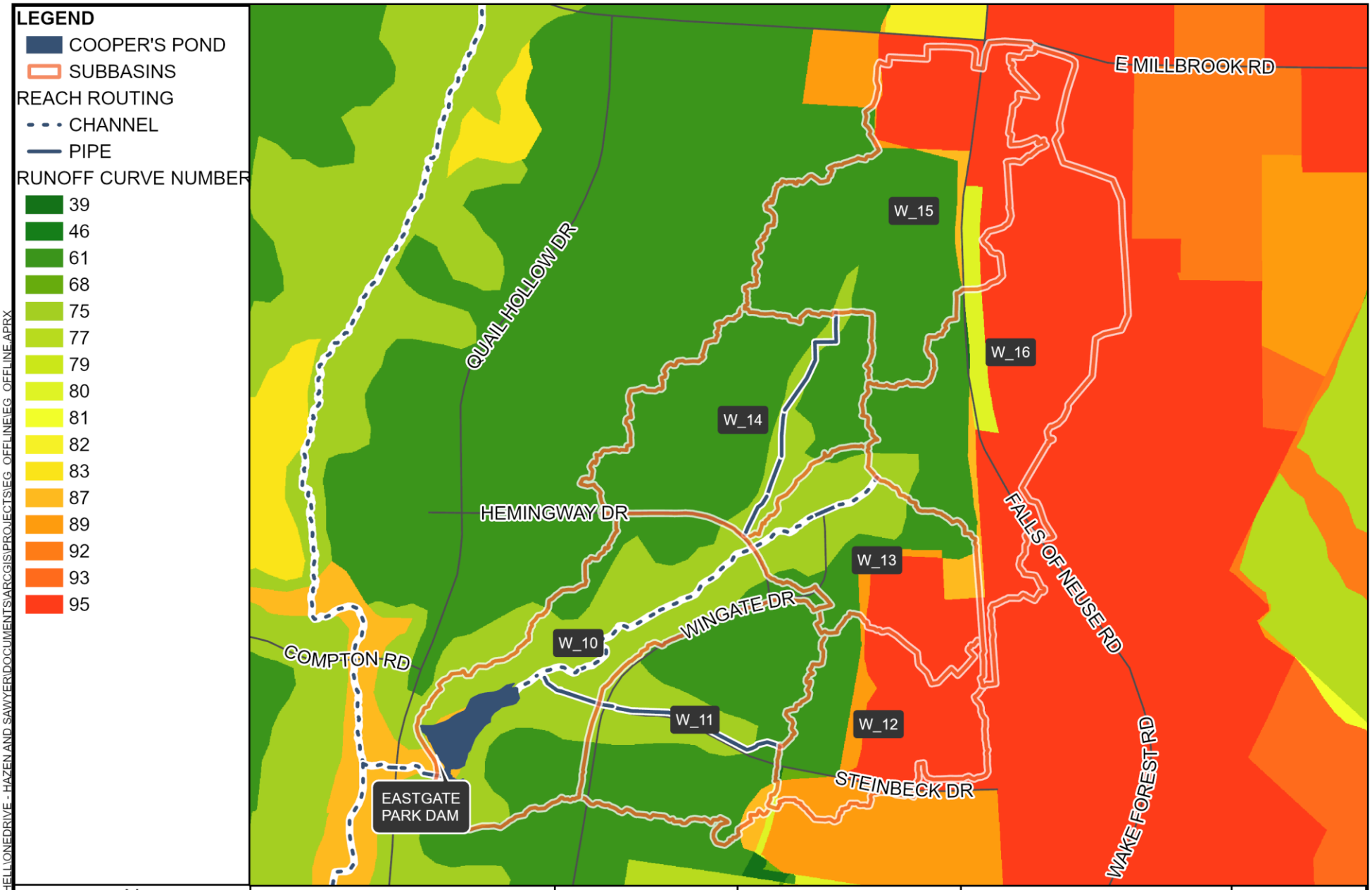
EASTGATE DAM PER

HYDROLOGIC SOIL GROUP MAP

DATE: JUNE 2023

FIGURE:

REFERENCE: ROAD NAMES (WAKE COUNTY), SOIL TYPE (SOIL SURVEY STAFF, NATURAL RESOURCES CONSERVATION SERVICE, UNITED STATES DEPARTMENT OF AGRICULTURE. WEB SOIL SURVEY. AVAILABLE ONLINE AT THE FOLLOWING LINK: HTTP://WEBSOILSURVEY.SC.EGOV.USDA.GOV/)



PATH: C:\USERS\GMITCHELL\ONEDRIVE - HAZEN AND SAWYER\DOCUMENTS\ARCGIS\PROJECTS\EG_OFFLINE\EG_OFFLINE.APRX

N

0 350 700 FEET

1" = 700'

Hazen

4011 WESTCHASE BLVD
SUITE 500
RALEIGH, NC 27607

Raleigh

ENGINEERING SERVICES

EASTGATE DAM PER

HYDROLOGIC SOIL GROUP MAP

DATE: JUNE 2023

FIGURE:

REFERENCE: NEH PART 630, ROAD NAMES, ZONING (WAKE COUNTY), SOIL TYPE (SOIL SURVEY STAFF, NATURAL RESOURCES CONSERVATION SERVICE, UNITED STATES DEPARTMENT OF AGRICULTURE. WEB SOIL SURVEY. AVAILABLE ONLINE AT THE FOLLOWING LINK: HTTP://WEBSOILSURVEY.SC.EGOV.USDA.GOV/)

Appendix D: Hydraulic Analysis

Manning's Roughness Coefficients

Land Cover	Manning's Coefficient
Paved	0.015
Channel	0.045
Open Space (Mesh Default)	0.050
Forest	0.140
Building	10.000

Dam Breach Parameter Inputs

Event	SDF Breach	SDB
Top of Dam Elevation (ft)	261.38	261.38
Breach Bottom Elevation (ft)	247	247
Pool Elevation at Failure (ft)	262.2[1]	257.6[2]
Pool Volume at Failure (ac-ft)	31	15.2
Failure Mode	Overtopping[3]	Piping
Dam Top Width (ft)	13	13
Upstream Face Slope (H:V)	3	3
Downstream Face Slope (H:V)	2	2
Earth Fill Type[4]	Fine Homogeneous	Fine Homogeneous

Notes:

1. Peak elevation during SDF
2. Normal pool elevation
3. Dam embankment overtops in SDF, thus mode of failure is overtopping
4. Fill type unknown; selected fill type that produced higher breach discharges

Spillway Design Flood Dam Breach Parameters

Parameter Set	MacDonald et al	Froehlich (1995)	Froehlich (2008)	Von Thun & Gillette
Center Station (ft)	225	**225**	225	225
Final Bottom Width (ft)	9	**12**	21	51
Final Bottom Elevation (ft)	247	**247**	247	247
Left Side Slope (H:V)	0.5	**1.4**	1	0.5
Right Side Slope (H:V)	0.5	**1.4**	1	0.5
Breach Weir Coefficient	2.6	**2.6**	2.6	2.6
Breach Formation Time (hr)	0.14	**0.18**	0.25	0.34
Failure Mode	Overtopping	**Overtopping**	Overtopping	Overtopping
Piping Coefficient	N/A	**N/A**	N/A	N/A
Initial Piping Elevation (ft)	N/A	**N/A**	N/A	N/A
Failure Trigger	Set Time	**Set Time**	Set Time	Set Time
Start Date	01 Jan 2023	**01 Jan 2023**	01 Jan 2023	01 Jan 2023
Start Time	15:18	**15:18**	15:18	15:18
Peak Breach Discharge (cfs)	2,438	**3,219**	2,892	2,671

Sunny Day Breach Dam Breach Parameters

Parameter Set	MacDonald et al	Froehlich (1995)	Froehlich (2008)	Von Thun & Gillette
Center Station (ft)	225	225	225	225
Final Bottom Width (ft)	1	5	12	39
Final Bottom Elevation (ft)	247	247	247	247
Left Side Slope (H:V)	0.5	0.9	0.7	0.5
Right Side Slope (H:V)	0.5	0.9	0.7	0.5
Breach Weir Coefficient	2.6	2.6	2.6	2.6
Breach Formation Time (hr)	0.10	0.12	0.18	0.31
Failure Mode	Piping	Piping	Piping	Piping
Piping Coefficient	0.5	0.5	0.5	0.5
Initial Piping Elevation (ft)	252	252	252	252
Failure Trigger	Set Time	Set Time	Set Time	Set Time
Start Date	01 Jan 2023	01 Jan 2023	01 Jan 2023	01 Jan 2023
Start Time	0:01	0:01	0:01	0:01
Peak Breach Discharge (cfs)	931	988	988	1,009

Appendix E: Geotechnical Investigation

GeoTechnologies, Inc.
Geotechnical and Construction Materials Testing Services

March 20, 2023

Everette Knight, P.E.
HAZEN & SAWYER
4011 Westchase Boulevard, Suite 500
Raleigh, NC 27607

Re: Subsurface Investigation
Eastgate Park Dam – Stability Analysis
Raleigh, North Carolina
GeoTechnologies Proposal No. 1-22-1005-EA

Mr. Knight:

GeoTechnologies was requested to evaluate the stability of the existing dam at Eastgate Park in Raleigh, NC. It is our understanding that alterations may be planned for the structure, and the stability of the dam is of interest. The dam of interest is displayed on the attached Figures 1A and 1B. This report details our investigation of the dam, the results of our stability analyses, and our recommendations.

SITE INFORMATION AND EXISTING CONDITIONS

According to historical engineering plans provided to GeoTechnologies, the existing dam consists of an old dam of unknown age with geometric modifications constructed around 1974. The northern portion of the dam consists of a combination of the original dam and the 1974 modifications. The southern portion of the dam (where the embankment angles west) was entirely constructed in the 1974 modifications. The existing lake currently utilizes a system of weirs and pipes to direct water above a certain pond elevation into a nearby downstream drainage channel running east to west.

The 1974 construction plans outlined a gravel drain with an outlet pipe at the toe of the dam. However, no outlet pipe was observed at the toe of the dam, so it is assumed that if a toe drain was constructed, it is no longer functional. The downstream slope face has vegetation present, which serves to mitigate soil erosion. The upper portion of the upstream slope face that leads to the lake is armored with a thin layer of concrete towards the crest. No visible slope failure or sloughing was observed on the downstream face of the dam. Due to the lake pool, the upstream face of the dam is largely not visible.

Recent surveying data was provided for the dam, which is displayed in the attached Figure 1B. The height of the dam ranges from about 6 to 16 feet. The downstream face of the dam has a slope inclination of approximately 2H:1V. The exposed upper portion of the upstream face has a slope inclination of approximately 3H:1V. However, below the water level, the topography of the upstream face is unknown. The crest of the dam ranges in width from about 12 to 30 feet. The geometry of the dam used for analysis is displayed in the attached Figures 3A through 3C. The 1974 historical construction drawings provided to GeoTechnologies are also attached to this report.

INVESTIGATION PROCEDURES

Four standard penetration test (SPT) borings (labeled “B” borings) were performed at approximately even spacing at the crest of the dam. The borings began with removing concrete cores from the concrete walkway at the crest of the dam in each of the four boring locations, utilizing a mobile coring rig with a diamond-impregnated coring bit. The borings extended to a depth of approximately 35 to 40 feet below existing ground surface. The four borings performed were completed utilizing an all-terrain vehicle mounted drill rig utilizing wash boring techniques. Soils were

sampled at continuous intervals within the fill soils, and selected intervals within residual soils, using SPT procedures designated in ASTM D-1586. Boring B-1 was sampled utilizing a manual SPT hammer, and borings B-2 through B-4 were sampled utilizing an automatic SPT hammer.

Undisturbed Shelby tube samples were attempted to be collected from 12-14 feet below existing grade in boring B-2, and from 18 to 20 feet below existing grade in boring B-3. However, no sample was able to be retained in the tubes due to the softness/wetness of the material. An undisturbed Shelby tube was attempted to be advanced from 26 to 28 feet below existing grade in boring B-3. This sample was able to recover approximately 6 to 8 inches of undisturbed material.

Four supplementary borings (labeled "H" borings) were advanced approximately 2 to 6 feet below existing grade with a hand auger. These borings were performed on or past the toe of the downstream slope of the dam. These borings were performed by an engineer and the soils were classified in the field, with the approximate consistency of the borings estimated using a ½" steel probe rod. The purpose of the borings was to provide additional information on soil type and consistency, and to determine the depth of the groundwater table at various points across the dam profile.

LABORATORY TESTING

Laboratory testing included grain size analysis (ASTM D-1140), hydrometer testing (ASTM-D-7928), Atterberg limits testing (ASTM D-4318), organic content testing (ASTM D-2974), and direct shear testing (ASTM D-3080). Our original proposal had called for triaxial and permeability tests to be performed on undisturbed samples. However, the attempts to advance an undisturbed Shelby tube into the fill soils failed, and only a short section of undisturbed residual soil was able to be collected. Consequently, only one direct shear test was performed on the sample of undisturbed residual soils for the sample from 26 to 27 feet below existing grade in boring B-3.

The results of the drained direct shear testing yielded a friction angle of approximately 32.5 degrees with a cohesion of approximately 100 psf. The SPT samples which exhibited any conspicuous amount of organic material were testing for organic content. The results of the organic content tests yielded 1.8 to 2.1 percent by weight, indicating a low organic content for the samples tested. The classification tests performed indicated the tested samples had USCS classifications of SC-SM and MH-CH. It should be noted though that several of the samples were very close to 50 % fines content and had very similar percentages of clay and silt. Therefore, many of the samples were given dual classifications. The results of all of the laboratory testing are attached to this report.

SUBSURFACE CONDITIONS

A generalized subsurface profile prepared from the test boring data is attached to this report as Figure 2 to graphically illustrate subsurface conditions encountered at this site. More detailed descriptions of the conditions encountered at the individual test boring locations attached to this report.

SPT Borings. The "B" borings encountered a 3 to 4.5 inches thick layer of concrete at the surface. Below this layer of concrete at the crest of the dam, fill soils were encountered 12.5 to 23.5 feet below existing grade. The fill soils consisted of silty to clayey sands and sandy low to high plasticity clays/silts. The plasticity and fines content of the fill soils were highly variable. SPT resistances within the fill soils varied from 2 to 14 blows per foot (bpf), which is indicative of poorly to well-compacted soil. Boring B-2 encountered the softest/loosest fill soils.

Below the fill soils, residual soils were encountered. The residual soils consisted of loose to dense clean to silty sands. SPT resistances within the residual soils varied from 8 to 35 blows per foot (bpf). Three of the four borings encountered partially weathered rock (PWR) prior to the termination point of the borings. PWR is defined as material which could be penetrated with soil drilling augers, but which exhibited penetration resistances in excess of 100 bpf.

PWR was encountered at 26.5 to 36.5 feet below existing grade in borings B-2, B-3 and B-4. SPT resistances within the PWR varied from 50/0.5" to 50/4".

Once borings were completed, the drilling fluid was removed with a pump and then allowed to re-fill with groundwater. Groundwater measurements were taken approximately 1 hour and 24 hours after pumping the holes dry. It should be noted that boring B-4 did not get a 24-hour reading on the groundwater table. Once groundwater measurements were taken, the borings were tremmie grouted and concrete patched. Groundwater was encountered in the borings at depths ranging from about 13 to 24 feet below the crest of the embankment at 1 hour after the time of boring completion. Groundwater was encountered in the borings at depths ranging from about 16.5 to 22 feet below the crest of the embankment 24 hours after time of boring completion. It should be noted that groundwater levels will fluctuate during different periods of the year and based upon the pool elevation of the dam.

Hand Auger Borings. The "H" borings were performed with a hand auger and 1/2 inch diameter steel probe rod. These borings encountered 0 to 6 inches of topsoil at the surface. Boring H-1 through H-3 encountered fill soils that consisted of silty sand to sandy silt. Probing of the fill soils indicated poorly to well-compacted material. Boring H-4 encountered residual soils consisting of silty sand. All of the hand auger borings performed were dry at the time of boring. The borings performed at the toe of the dam (H-1 and H-2) encountered hand auger refusal approximately 2 feet below existing grade. Both appeared to encounter large rocks that prevented advancement of the hand auger. These hand auger borings were offset several times, and the same conditions were encountered. It is unclear what was encountered, but it could have possibly been remnants of an old toe drain or rip rap armoring.

STABILITY ANALYSIS

Based on the conditions encountered in the SPT and hand auger borings and the laboratory data, stability analyses were performed for the dam. As previously stated, the geometry of the dam was based upon the provided survey data displayed in Figure 1B. The analysis was based upon boring B-2 because it was performed at the peak height of the embankment and encountered the most soft/loose fill material. The analysis was run assuming that the lake pooled at 4 feet below the crest of the dam. The analyses were performed in a downstream steady-state condition and an upstream rapid drawdown condition utilizing SLIDE engineering software.

It should be noted that the topography of the upstream face of the slope below the water level is unknown. The stability analysis for rapid drawdown was performed assuming that the 3H:1V slope continued down to an elevation 8 feet below the crest of the dam. To properly evaluate the stability of the dam in a rapid drawdown situation, it is necessary to obtain further surveying data on the entire topography of the upstream face of the dam.

For fill soils within the analysis, soil strength parameters used in the analysis were separated into poorly and well-compacted fill soils. The poorly compacted fill soils were assumed to have a friction angle of 26 degrees and a cohesion of 5 psf. The well-compacted fill soils were assumed to have a friction angle of 30 degrees and a cohesion of 25 psf. Loose residual soils were assumed to have a friction angle of 30 degrees and a cohesion of 100 psf. Medium dense residual soils were assumed to have a friction angle of 32 degrees and a cohesion of 100 psf. PWR was assumed to have a friction angle of 45 degrees and a cohesion of 0 psf.

The attached Figure 3A visualizes the parameters and output of the SLIDE analysis for the existing dam conditions. Our analysis found the factor of safety against dam failure to be approximately 1.15. The required steady-state factor of safety against failure for a high-hazard dam is 1.5. Based on our analysis, we recommend that modifications be made to improve long-term stability of the dam. Modifications options for the dam will be discussed in the following section.

RECOMMENDATIONS FOR DAM MODIFICATION

As previously mentioned, based on our analysis, the existing dam will require modifications in order to achieve the minimum required steady-state factor of safety against failure of 1.5. The simplest means of increasing the factor of safety against failure would be to flatten the downstream slope. This would involve placement of well-compacted soil or rip-rap to extend the toe of the downstream side of the dam. This option would add significant weight to the side of the dam resisting failure, and would thus increase the factor of safety against failure. It is recommended to incorporate a blanket and toe drain into any additions to the downstream side of the dam, in order to maintain a safe groundwater level across the dam profile and handle any potential seepage.

Based upon discussion with Everette Knight of Hazen & Sawyer, it is unlikely that rip rap would be selected as a material to flatten the slope for aesthetic reasons. Therefore, primary consideration will be given to placement of properly compacted soil to flatten the existing downstream slope of the dam. It should be noted that extending a portion of the toe will be constrained by the concrete spillway mechanism to the west of the dam. In this area, a properly designed and constructed wall will be necessary to flatten the slope to the desired angle.

Proposed Modifications to Upstream Face of the Dam. As previously noted, the topography of the upstream face of the slope below the water level is unknown. The stability of the dam in a rapid drawdown situation is sensitive to the topography of the existing upstream face. For this reason, the analysis for rapid drawdown of the dam should be considered preliminary, and the remaining topography of the upstream face of the dam should be surveyed, and that survey information should be provided to GeoTechnologies.

Based upon our stability analysis (displayed in the attached Figure 3C), if the remaining slope of the upstream face is 3H:1V or flatter, the dam has a factor of safety against failure due to rapid drawdown of at least 1.25. This meets the minimum standard for Dam Safety for high hazard dams, and thus no modifications to the upstream face of the dam would be recommended. However, if any portion of the upstream face of the dam is steeper than 3H:1V, it is recommended to place rip rap in order to flatten the slope to an inclination of 3H:1V or flatter. Well-compacted soil could also be used to flatten the slope, however this would involve removal and replacement of any soft soils present at the face and/or toe of the slope. Therefore, flattening the slope with soil is likely to be more logistically difficult than flattening the slope with rip-rap.

Proposed Modifications to Downstream Face of the Dam. Any recommended dam modifications should be designed based upon recent surveying plans and any constraints for construction. The intent of modifications to the embankment is to control seepage while improving the steady state safety factor to at least 1.5 (which is in-compliance for Dam Safety requirement for a high-hazard dam). Our analysis (displayed in the attached Figure 3B) indicates that flattening the slope to 2.75H:1V or flatter will increase steady state safety factor to at least 1.5. Therefore, for the downstream slope, we recommend the installation of a blanket drain and the extension of the toe about 12 feet to flatten the slope to about 2.75H:1V.

Dam modification should begin with the striping of any vegetation. After the slope face has been stripped, a minimum 12 inches (or as shown on Figure 4) of C-33 sand should be placed on the approved subgrade to construct a blanket drain. A minimum of 12 inches of compacted soil (95% standard Proctor maximum dry density) with a maximum permeability of 1 $x10^{-5}$ cm/sec should be placed over the drain. The gradation of the fill should be in compliance with filter criteria relative to the C-33 sand. The C-33 sand should be brought to the existing slope toe and then extended under the new slope toe, as outlined in the attached Figures 4 and 5. The drainage pipe for the toe drain should be installed as outlined in Figures 4 and 5, with solid outlet pipes installed at a minimum of every 100 linear feet. Based on the existing site topography, the best location to discharge the drainage pipe would be the nearby downstream drainage channel west of the dam. Once (or as) the drain is installed, the new slope geometry can be constructed from compacted soils with a maximum permeability of 1 $x10^{-5}$ cm/sec. The drain should be protected from

contamination during the construction process. Newly placed fill on the face of the existing slope should be placed using proper benching techniques as outlined in the attached Figure 7.

The foundation area for the extended slope should be evaluated by a geotechnical engineer to determine the depth and extent of any undercut necessary for stability of the extended slope. Based on a probe rod inspection, the contractor should be prepared for undercut depths of 1 to 2 feet. Undercut and backfilling should be performed in small sections which allow for the operation to be completed daily. No undercut excavations should be left open overnight. Additionally, the contractor should maintain on-site at least one roll of nonwoven geotextile fabric equal to Mirafi 160N and 100 cubic yards of C-33 sand in the event that undercut operations expose pressurized seepage which needs to be countered with a weighted filter as an emergency measure.

The grain size analysis of the embankment soils tested indicates that typical C-33 sand can be utilized as a filter material for the blanket drain, as previously outlined in the report (and in Figure 4). However, it is possible that more coarse base soils could be encountered where the blanket drain is proposed to be constructed. If more coarse base soils are encountered, it may be necessary to modify the filter material for the blanket drain. Alternatively, approximately 12 inches of the base material could be replaced with well-compacted soils with a fines content greater than 40 percent. We recommend that verification grain size testing be performed on the base soils, the borrow soils and the actual C-33 filter sand which will be used. This grain size testing should be performed prior to importing any material on-site.

Issues with Flattening the Downstream Slope. The concrete principal spillway structure will interfere with flattening the existing slope. In order to flatten the slope without affecting this site feature, a retaining wall will likely have to be constructed in this area. Based upon preliminary analysis, this wall will have to be about 3 to 6 feet in height and have a significant embedment of 3 to 4 feet below existing grade. There are many options for types of retaining walls, including mechanically stabilized earth wall (MSE), gravity walls, cantilever walls, post and panel walls.

Construction of the wall may involve significant excavation at the toe of the existing slope. In order to safely construct this retaining wall, shoring and/or dewatering may be necessary. Additionally, the soils surrounding the existing 42” RCP pipe in the area may be disturbed. It is difficult to properly compact soils below an existing concrete pipe. Therefore, if the soils surrounding this pipe are disturbed, it is recommended to construct a concrete cradle surrounding the pipe, as outlined in the attached Figure 6. The design will need to incorporate the recommended blanket drain and provide proper drainage.

Shoring and dewatering are the responsibility of the contractor performing the dam modifications. Once the location and type of retaining wall desired has been determined, GeoTechnologies can be contacted to make recommendations with regards to the logistics of construction. GeoTechnologies can also provide wall designs for gravity or MSE walls, if requested.

Borrow Materials. New fill needed to achieve the design geometry should consist of low to moderately plastic clays and silts with Unified Soil Classifications of CL, CL-CH, ML, and MH, although silty to clayey sands (SM-SC) may be considered as well. The maximum permeability of the borrow should not exceed 1x10E-5 cm/sec unless otherwise approved by the geotechnical engineer. The borrow should also comply with filter criteria relative to C-33 sand as appropriate.

During construction, density tests should be performed on a full-time basis to identify any potential problem areas. The fill should be placed in loose lifts of no more than 8 inches in thickness. All fill soils should be compacted to a minimum of 95% of the standard Proctor maximum dry density. Assuming the specified density can be achieved, it is our recommendation that fill materials be placed at moisture contents of -1% dry of optimum to +3% wet of optimum unless otherwise directed by the geotechnical engineer.

Dam Outlet Pipes. It should be noted that this report contains no assessment or recommendations for the internal or external conditions of the outlet pipes/structures for the dam. In order to properly evaluate these site features, a separate evaluation would need to be performed by a qualified engineer.

Alternative Stabilization Option. An alternative option for improving stability of the existing dam is to improve the soils within the dam using chemical stabilization. This would involve working with a specialty contractor to grout weak fill layers to improve the overall strength of the soil profile. This would allow the existing geometry of the dam to remain the same, while significantly improving the factor of safety against failure. It is recommended to explore chemical stabilization as an option, although it should be noted it is often expensive compared to alternative stabilization options. If chemical stabilization is to be pursued, please contact GeoTechnologies for additional recommendations regarding chemical stabilization of the existing dam.

CLOSING

GeoTechnologies, Inc. appreciates the opportunity to be of service on this phase of the project. Please contact us if you have any questions concerning this letter or if we may be of additional service on this or other projects.

Sincerely,
GeoTechnologies, Inc.

Sean A. Corcoran, P.E.
NC License No. 46382

NORTH CAROLINA PROFESSIONAL SEAL 046382 ENGINEER SEAN A. CORCORAN

SAC/pr-els

Attachments

Table 1 - Boring Coordinates

Eastgate Park Dam Stability Analysis

Job # 1-22-1005EA

Boring	Latitude	Longitude
B-1	35.840713	-78.624417
B-2	35.840508	-78.624280
H-1	35.840477	-78.624393
B-3	35.840337	-78.624133
H-2	35.840297	-78.624291
H-3	35.840323	-78.624192
H-4	35.840279	-78.62436
B-4	35.840033	-78.624150

Figure 1A - Boring Location Map

Job # 122-1005EA

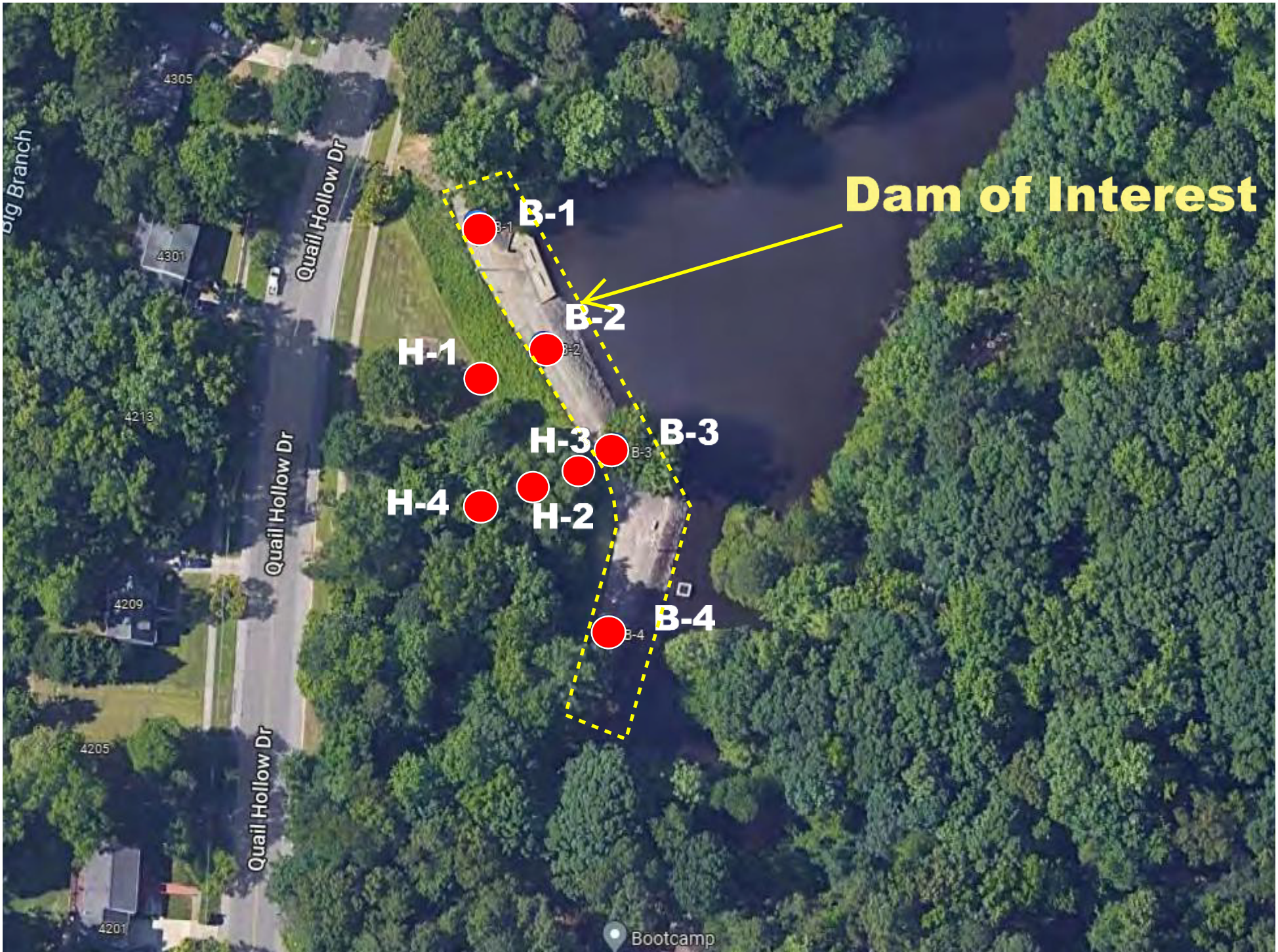
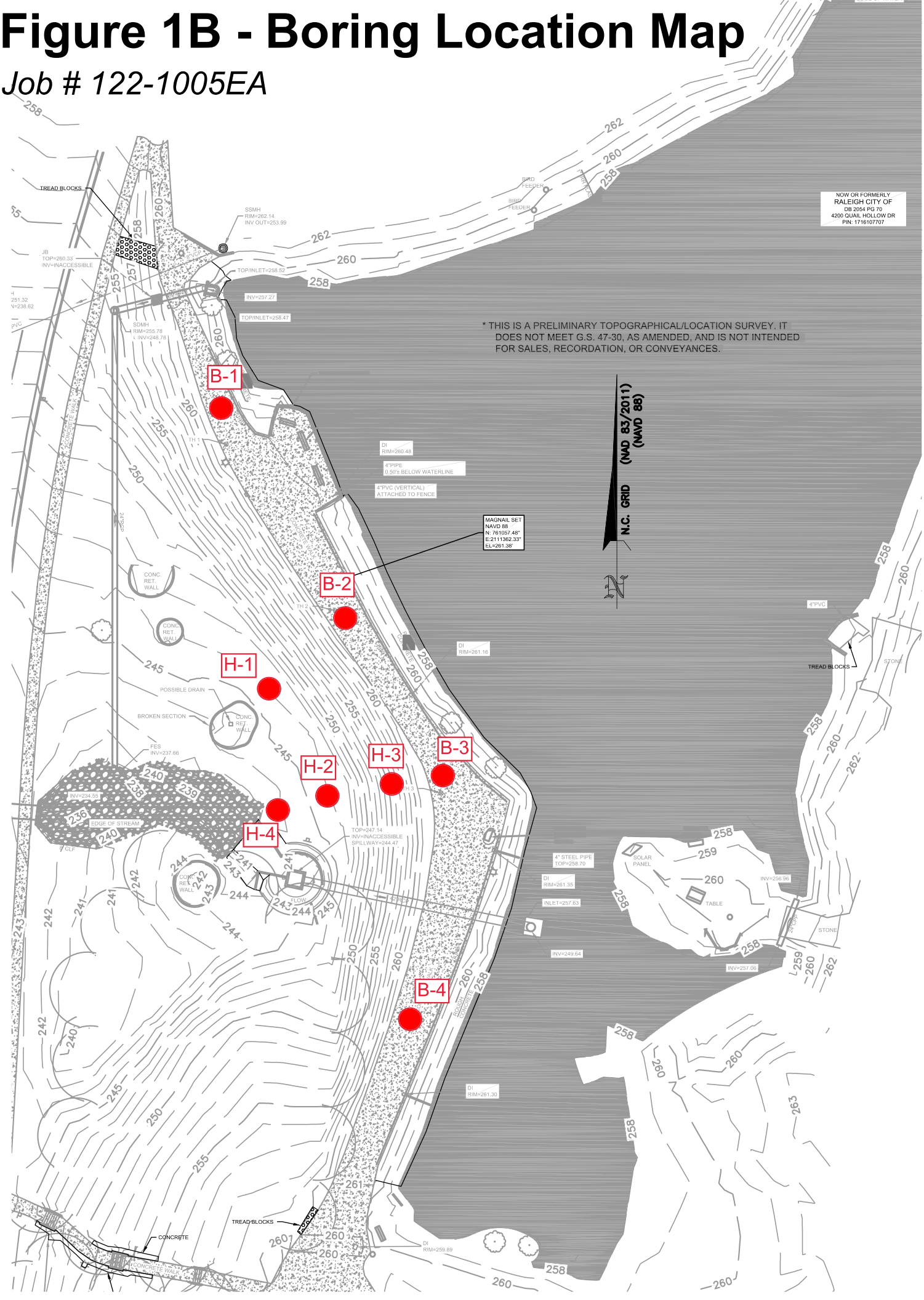
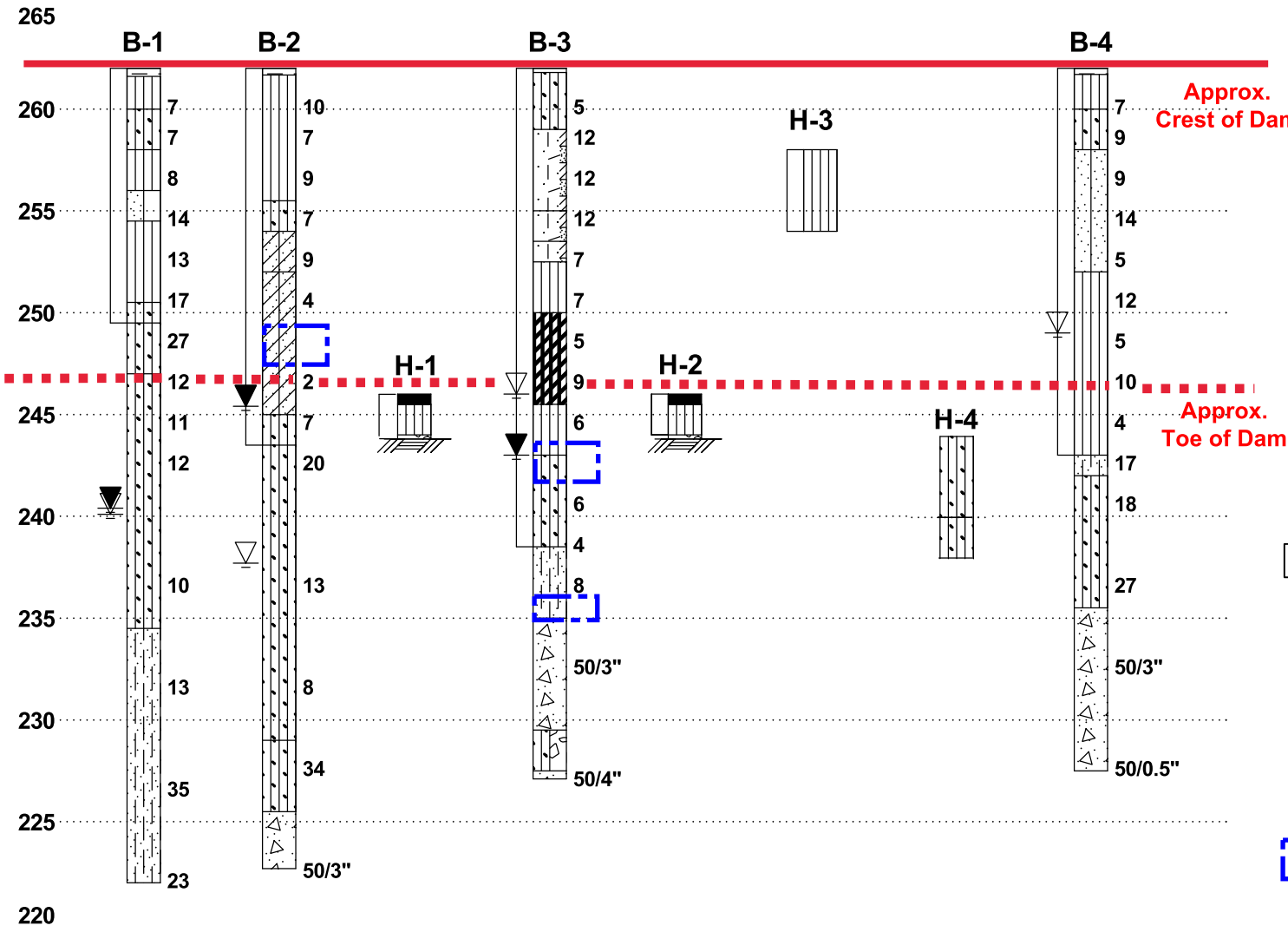


Figure 1B - Boring Location Map

Job # 122-1005EA



GENERALIZED SUBSURFACE PROFILE



LEGEND

- Concrete
- Low Plasticity Silt
- Silty Sand
- Silty Sand to High Plasticity Silt
- Silty to Clean Sand
- Silty to Clayey Fine to Medium SAND
- Partially Weathered Rock
- Topsoil
- Stone and Rocks
- High Plasticity Clay to Clayey Sand
- High Plasticity Silt to Clay
- Silty Sand w/ Rocks
- Fill
- Standard Penetration Resistance
- * Dynamic Cone Penetration
- Groundwater at Time of Boring
- Groundwater after Time of Boring
- Auger Refusal
- = Undisturbed Shelby Tube Attempted to be Collected

PROJECT:

Eastgate Park Dam
Raleigh, North Carolina

GeoTechnologies, Inc.

SCALE: As Shown

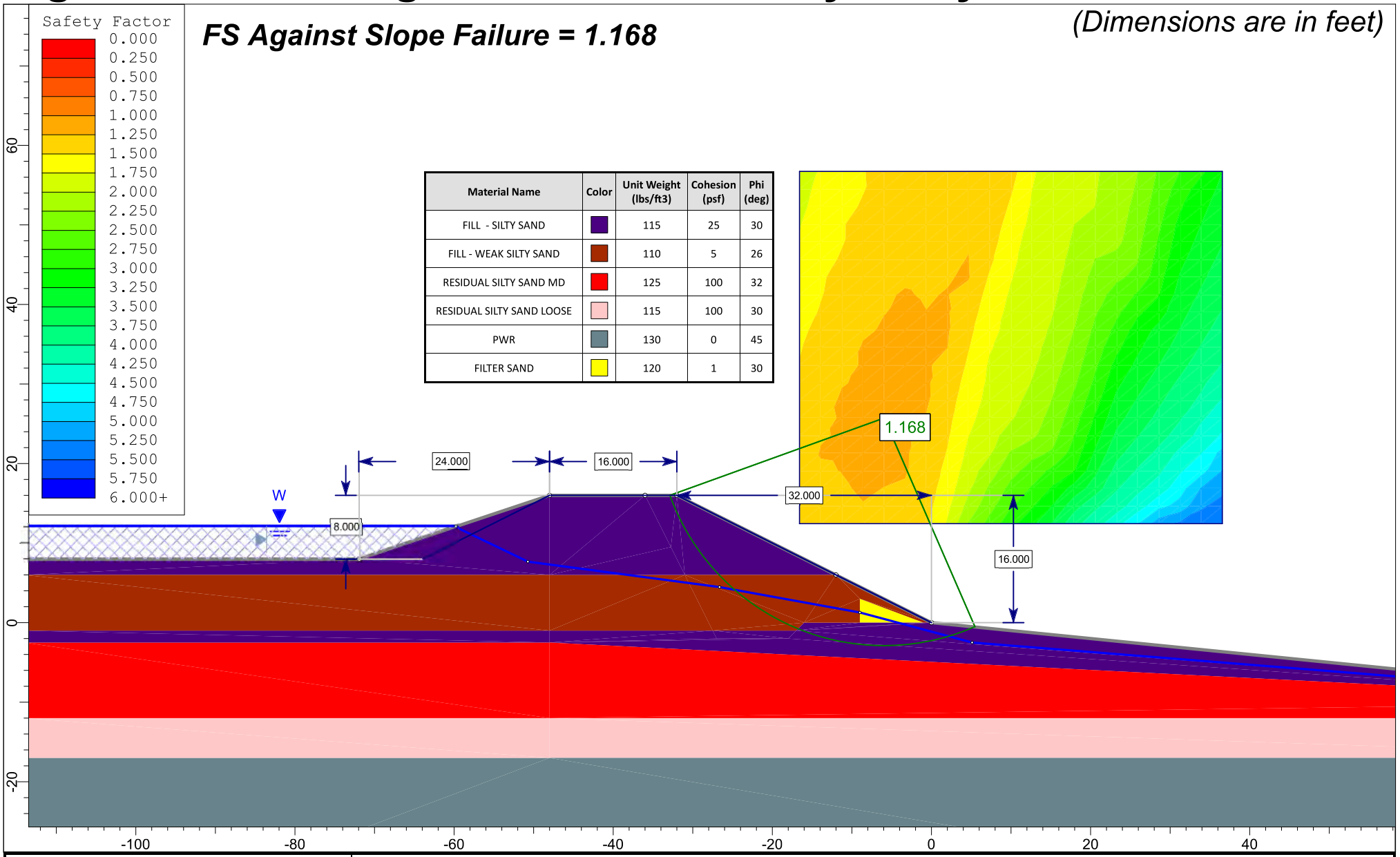
JOB No:122-1005EA

FIGURE No:2

Figure 3A - Existing Conditions Stability Analysis

Job # 122-1005EA

(Dimensions are in feet)



rocscience	Project: SLIDE - An Interactive Slope Stability Program		
	Analysis Description:		
	Drawn By:	Scale: 1:200	Company:
	Date:		File Name: 1221005EA Eastgate Dam - B-2.slim

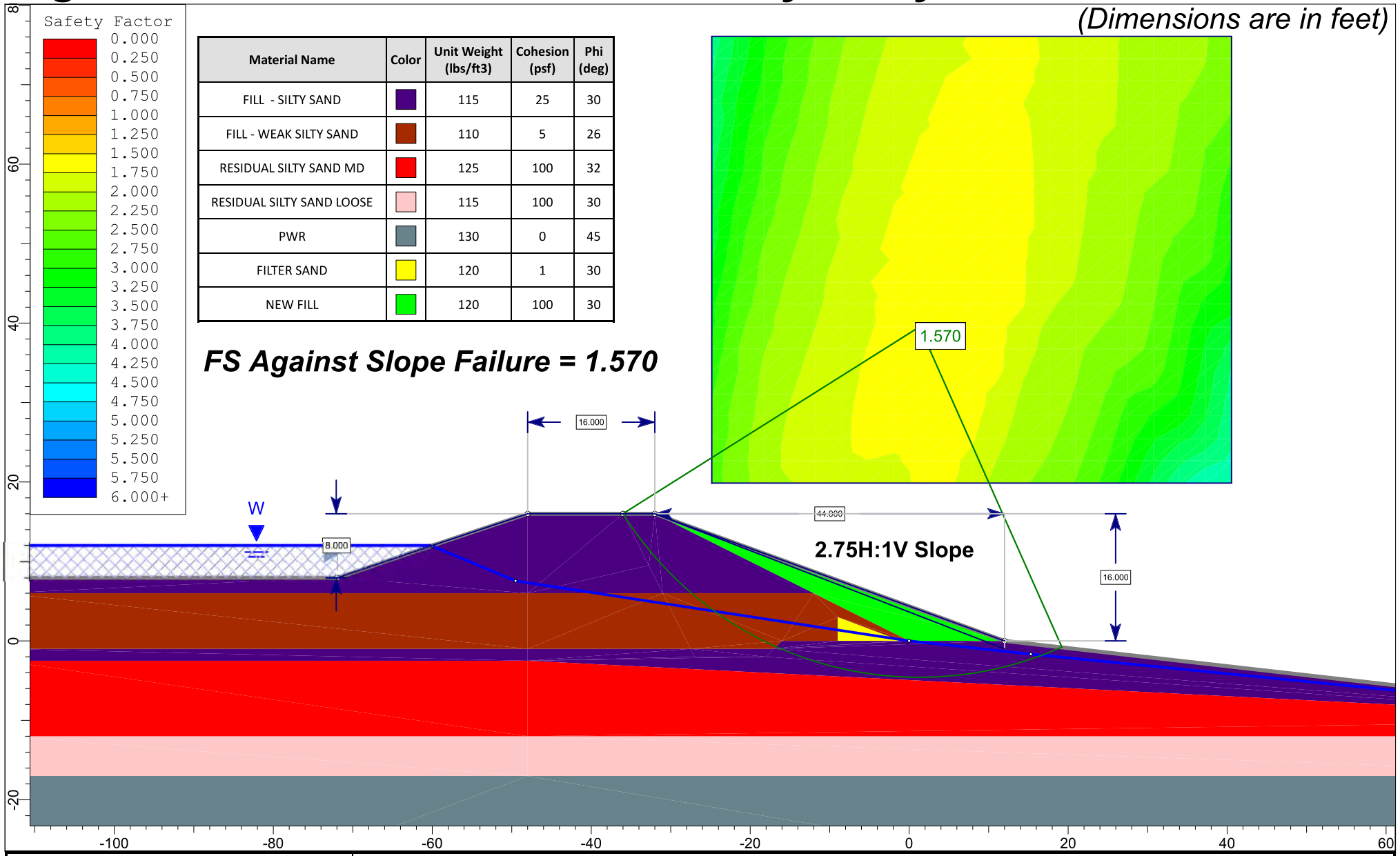
SLIDEINTERPRET 6.020

Profile Based upon Boring B-2

Figure 3B - Dam Modification Stability Analysis

Job # 122-1005EA

(Dimensions are in feet)



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
FILL - SILTY SAND		115	25	30
FILL - WEAK SILTY SAND		110	5	26
RESIDUAL SILTY SAND MD		125	100	32
RESIDUAL SILTY SAND LOOSE		115	100	30
PWR		130	0	45
FILTER SAND		120	1	30
NEW FILL		120	100	30

FS Against Slope Failure = 1.570

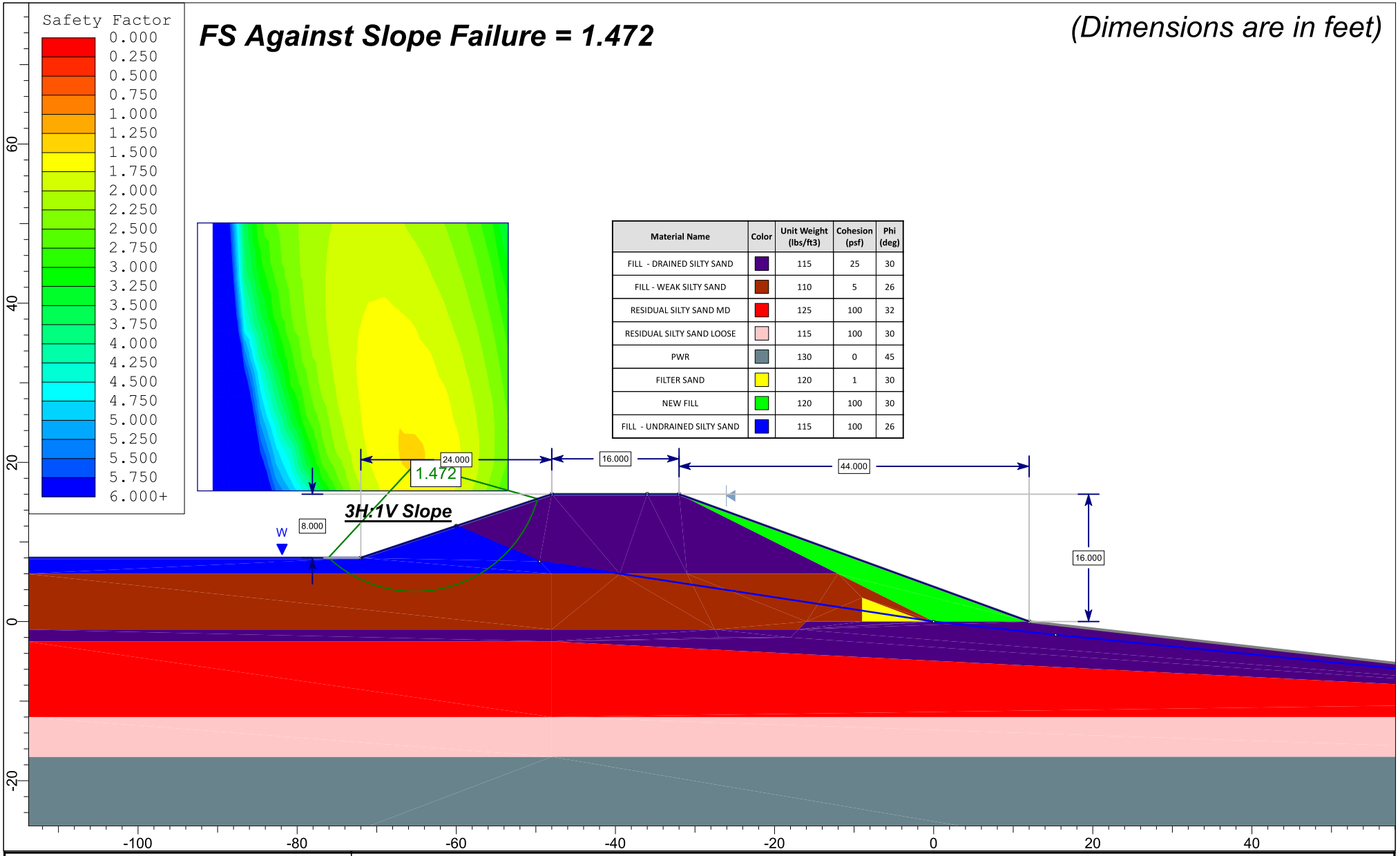
rocscience	Project: SLIDE - An Interactive Slope Stability Program		
	Analysis Description:		
	Drawn By:	Scale: 1:200	Company:
	Date:		File Name: 1221005EA Eastgate Dam - B-2 rip rap 2.75 to 1.slim

SLIDEINTERPRET 6.020

Profile Based upon Boring B-2

Figure 3C - Rapid Drawdown Stability Analysis

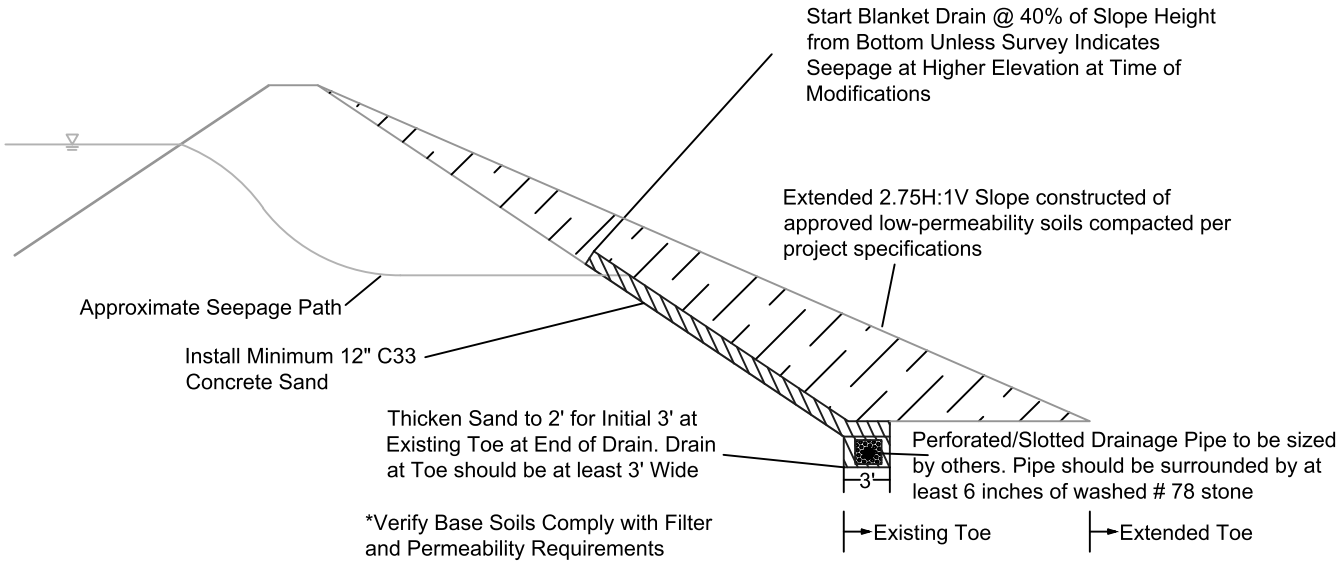
Job # 122-1005EA



rocscience	Project: SLIDE - An Interactive Slope Stability Program		
	Analysis Description:		
	Drawn By:	Scale: 1:200	Company:
SLIDEINTERPRET 6.020	Date:		File Name: 1221005EA Eastgate Dam - B-2 rip rap 2.75 to 1 rapid drawdown.slim

Profile Based upon Boring B-2

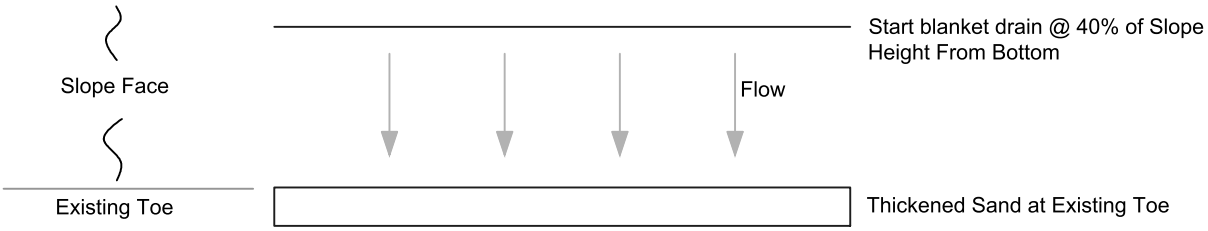
NOTE: Blanket Drain can be constructed on top of the existing embankment slope once vegetation has been cleared and the base soils have been approved by an engineer



BLANKET DRAIN CROSS SECTION

NTS

Note: See Figure 5 for a detailed cross section of the toe drain

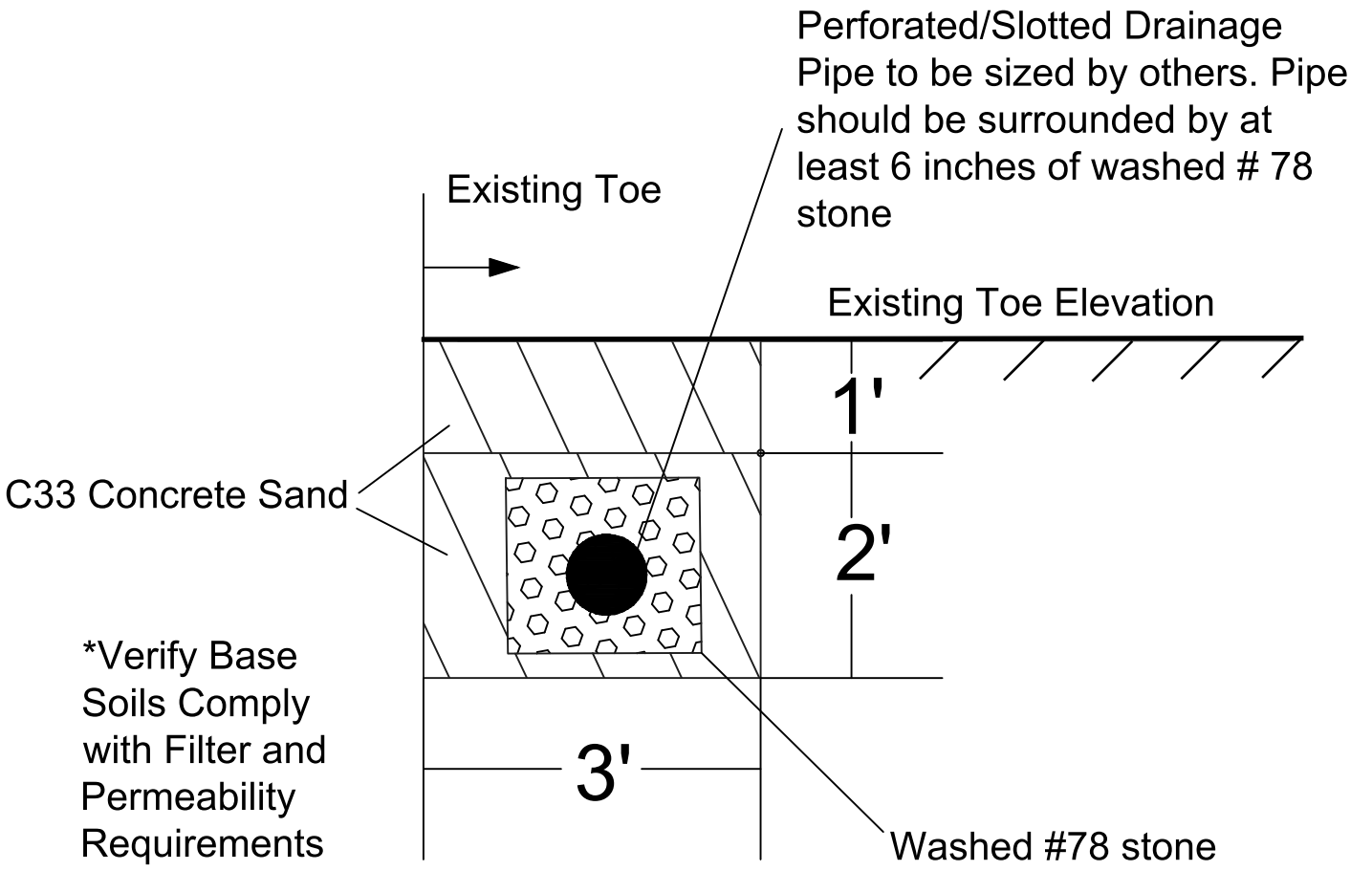


*Blanket Outlet to be Determined as Modification Plans Proceed

BLANKET DRAIN PLAN

NTS

PROJECT: Eastgate Park Dam Raleigh, North Carolina	GeoTechnologies, Inc. Geotechnical and Construction Materials Testing Services	SCALE: NTS JOB No: 1-22-1005-EA FIGURE No: 4



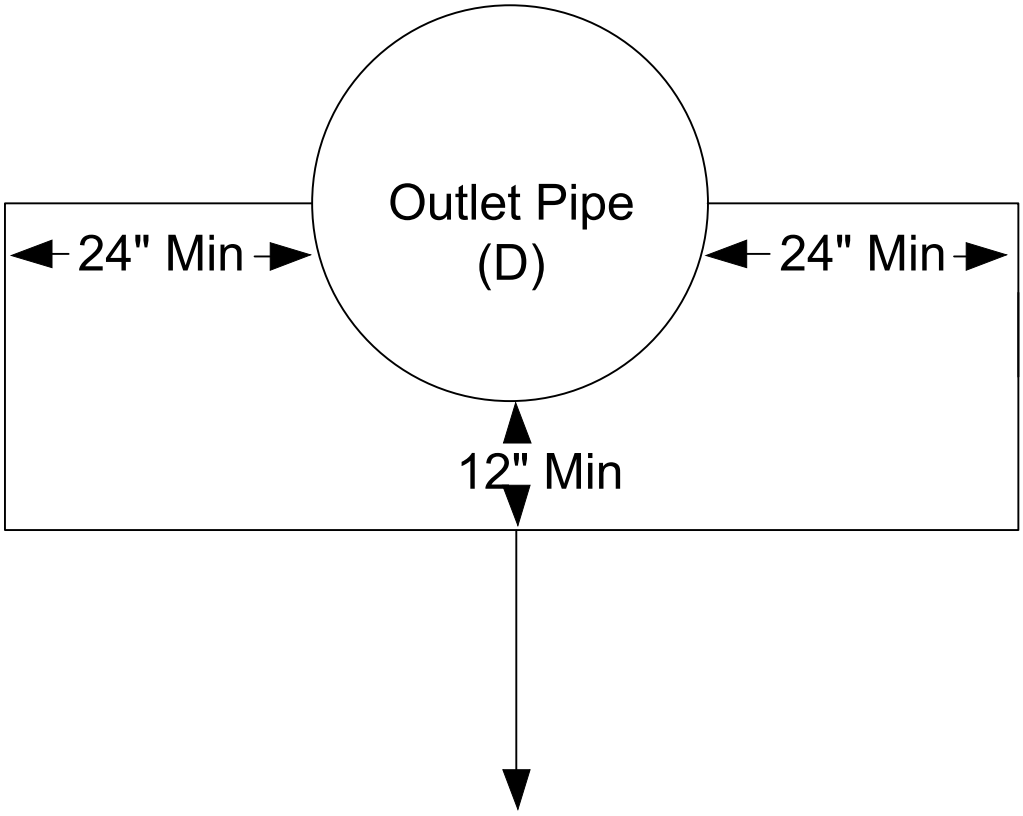
Note: Drainage pipe should have a solid pipe outlet installed at minimum every 100 linear feet. The drainage outlet should daylight at a lower elevation. The nearby downstream drainage channel may be an appropriate location to daylight the outlet pipes.

TOE DRAIN CROSS SECTION DETAILS

NTS

PROJECT: Eastgate Park Dam Raleigh, North Carolina	GeoTechnologies, Inc. Geotechnical and Construction Materials Testing Services	SCALE: NTS JOB No: 1-22-1005-EA FIGURE No: 5

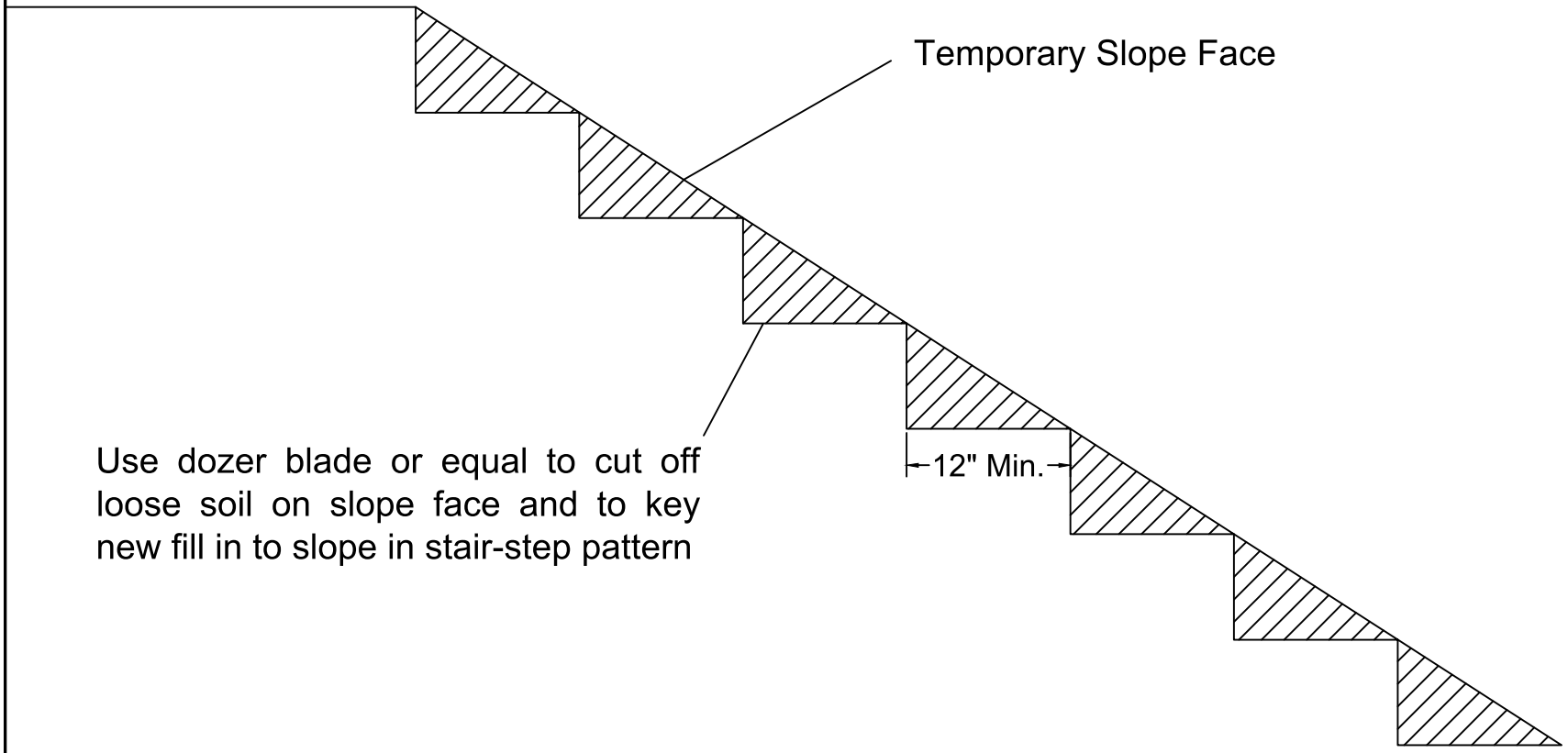
Concrete Cradle



Concrete cradle to at least midpoint ($\frac{1}{2}$D) of outlet pipe.

PROJECT: Eastgate Park Dam Raleigh, North Carolina	GeoTechnologies, Inc. Geotechnical and Construction Materials Testing Services	SCALE: NTS JOB No: 122-1005-EA FIGURE No: 6

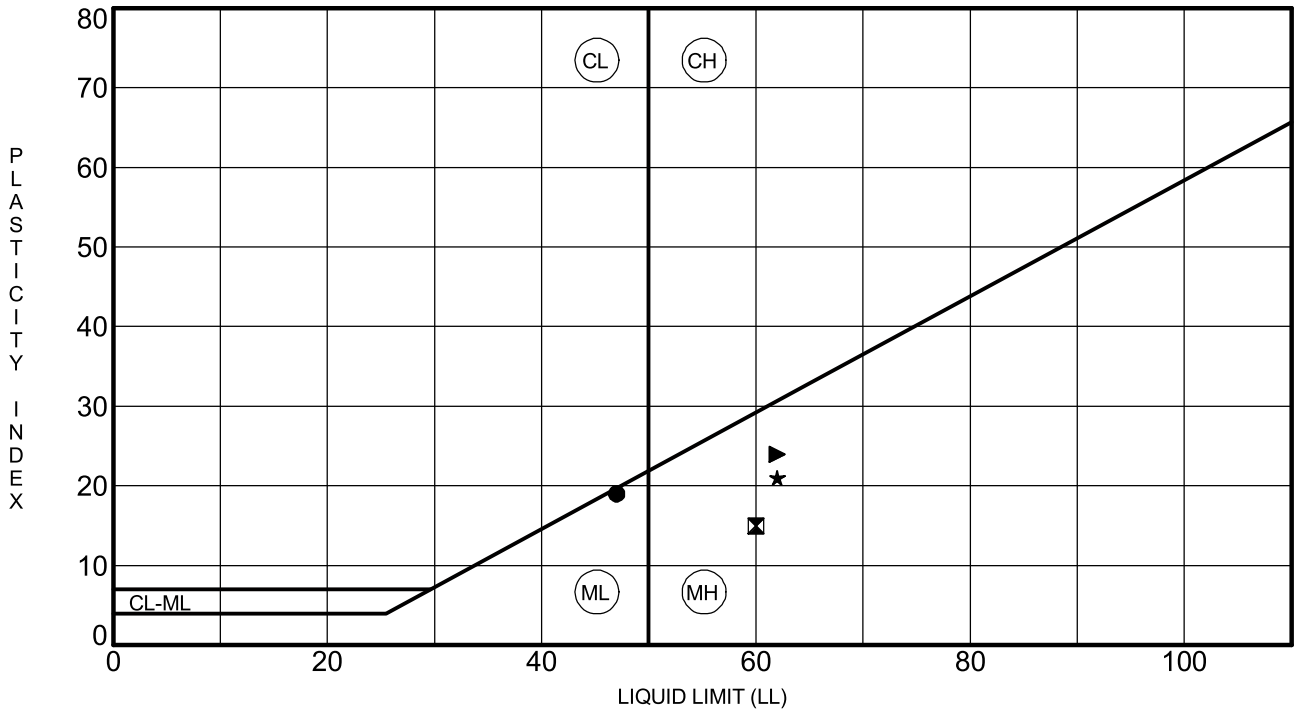
Slope Benching Detail



PROJECT: Eastgate Park Dam Raleigh, North Carolina	GeoTechnologies, Inc.	SCALE: NTS JOB No: 122-0952-EA FIGURE No: 7

Laboratory Testing

ASTM D-4318



Specimen Identification				LL	PL	PI	Fines	Classification
●	B-2	S-7	14.0-15.0'	47	28	19	37.2	Brown Clayey Silty Fine to Course Sand
⊠	B-3	S-5	8.0-9.0'	60	45	15	48.6	Brown Clayey Silty Medium to Fine Sand
▶	B-3	S-7	12.0-13.0'	62	38	24	62.1	Brown Fine to Medium Sandy Silty Clay
★	B-4	S-5	8.0-9.0'	62	41	21	47.5	Brown Clayey Silty Fine to Medium Sand

PROJECT **Eastgate Park Dam - Raleigh, NC**

JOB NO. **1-22-1005-EA**

DATE **12/13/22**

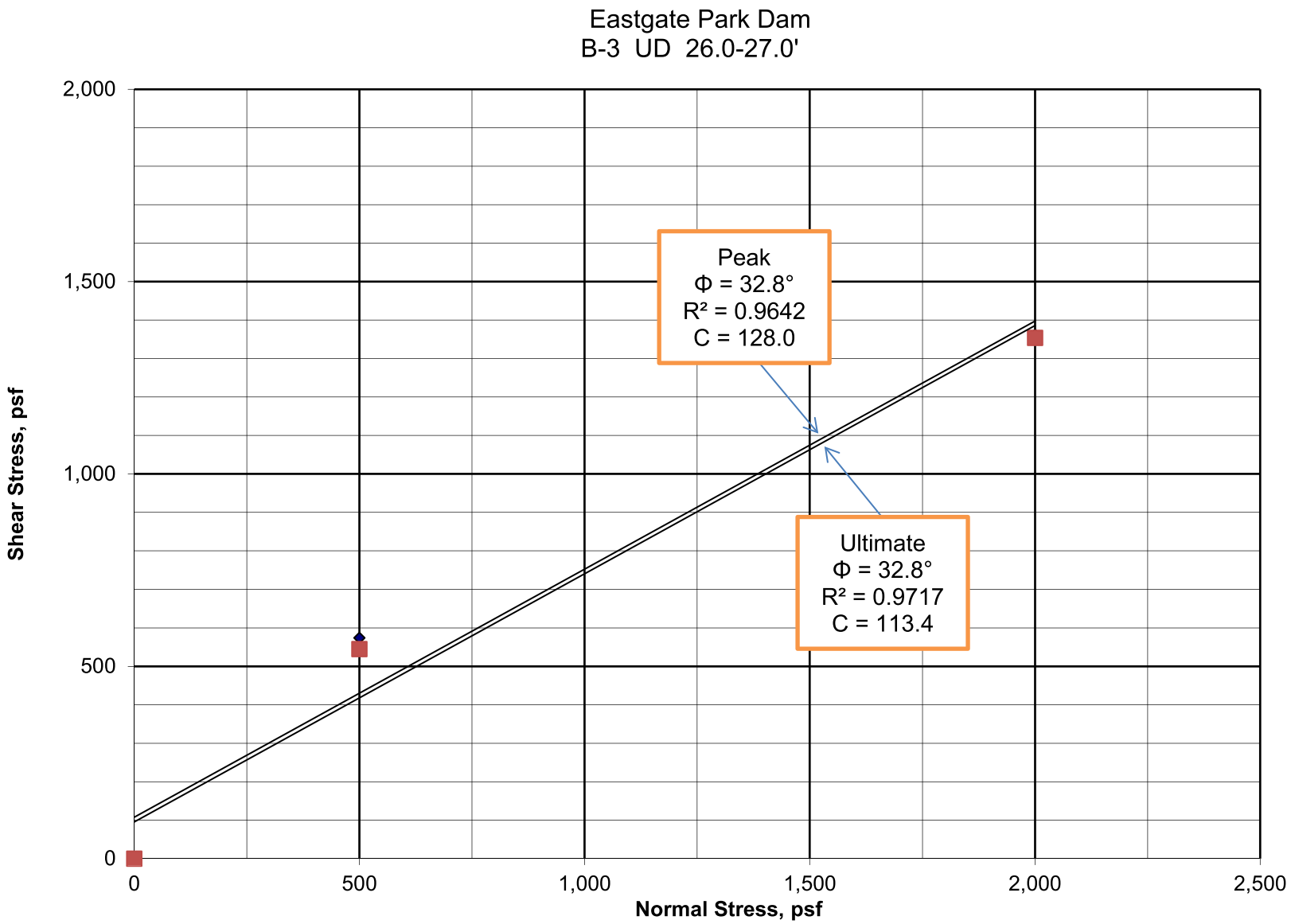
Date Recieved: 11/21/2022

Dates Tested: 11/21-12/12/2022

ATTERBERG LIMITS' RESULTS

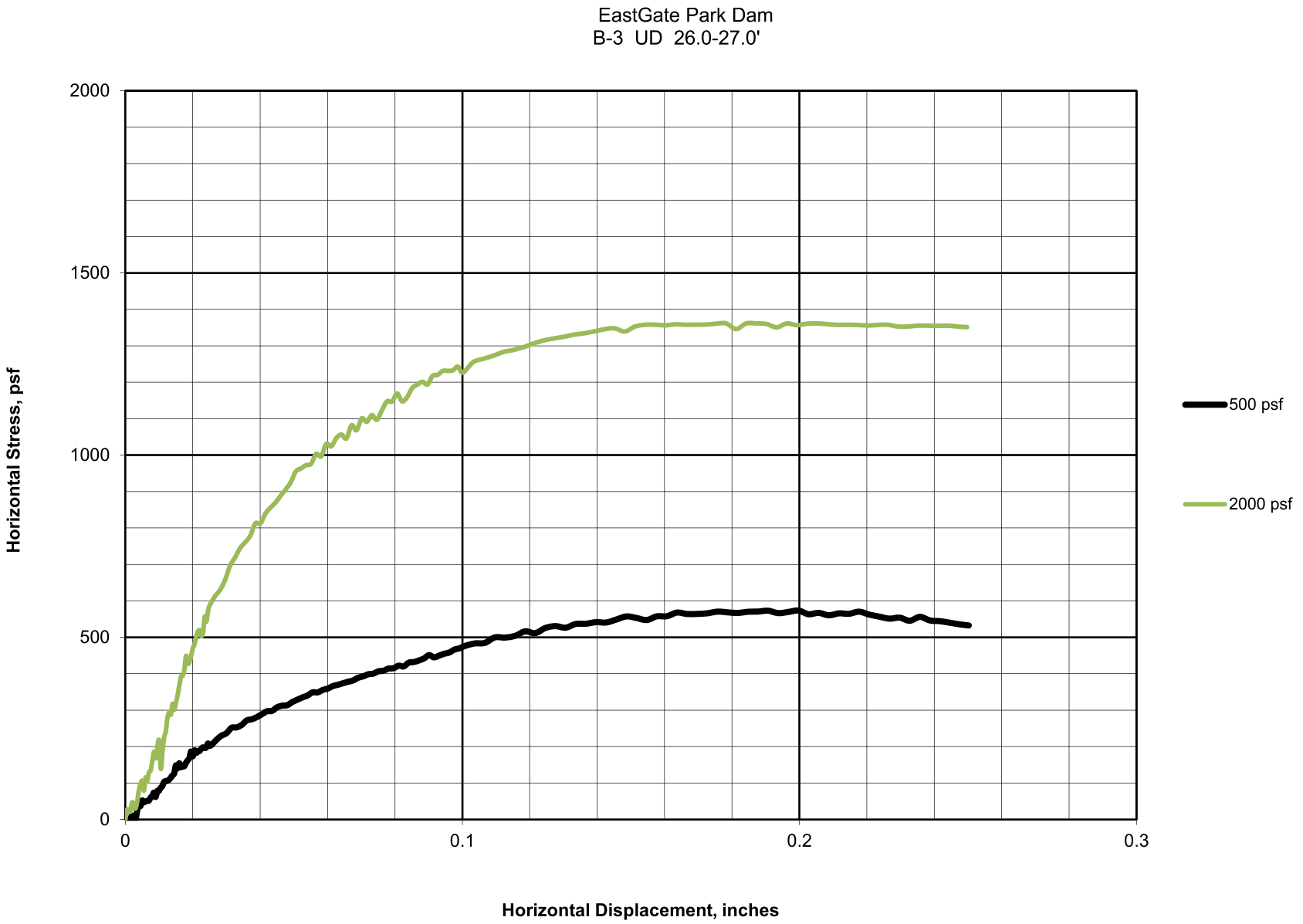
3200 Wellington Ct
Raleigh, NC 27615

Drained Direct Shear Testing Results



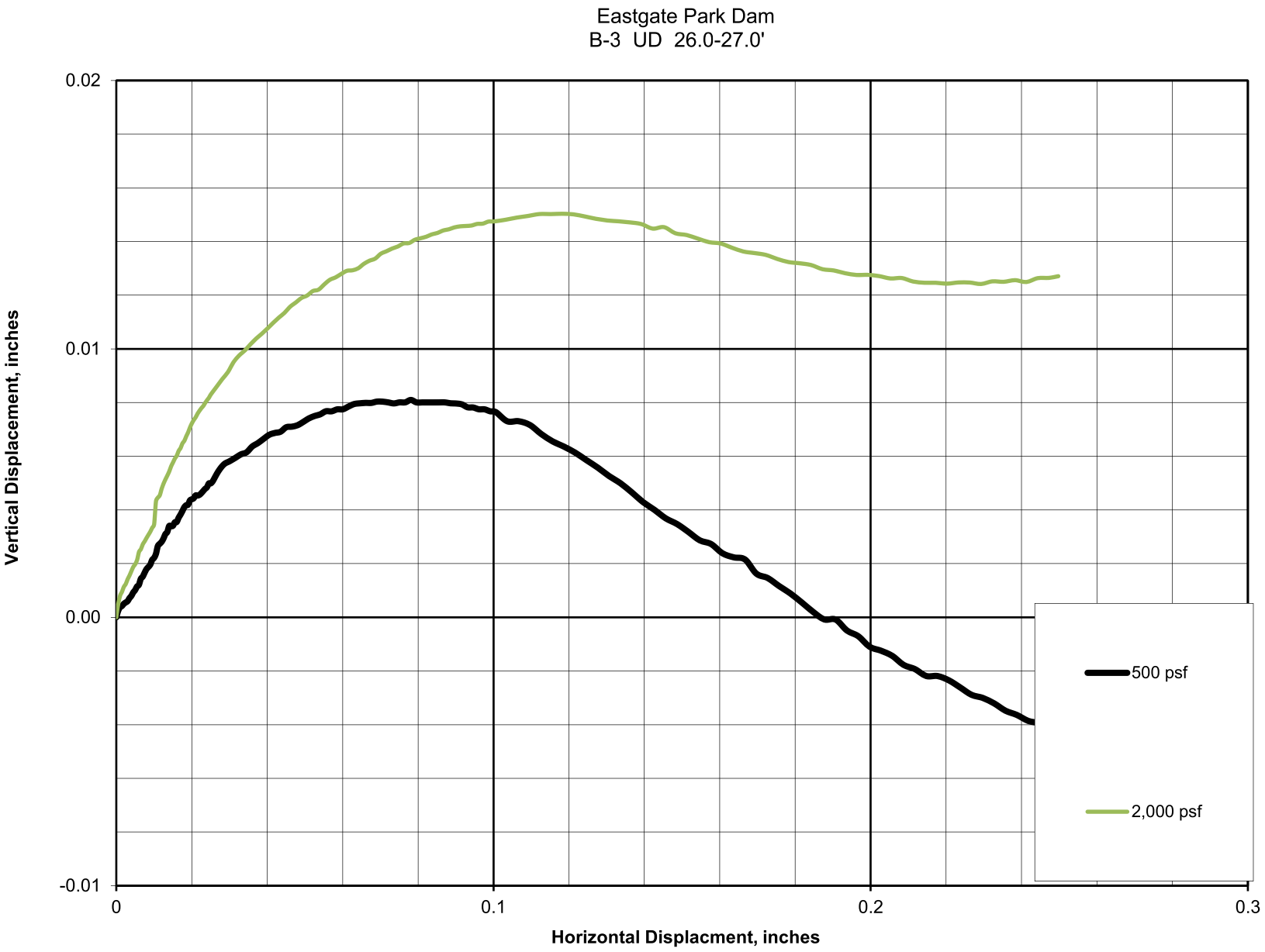
Job # 122-1005EA

Drained Direct Shear Testing Results



Job # 122-1005EA

Drained Direct Shear Testing Results



Job # 122-1005EA

GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

Boring No.	Elev./Depth	Nat. W.C.	L.L.	P.L.	P.I.	Soil Description or Classification
B-2 S-7	14.0-15.0'		47.0	28.0	19.0	Brown Clayey Silty Fine to Course Sand

Project:

Eastgate Park Dam
Raleigh, NC

Job No.: 1-22-1005-EA

Date: 12/13/22

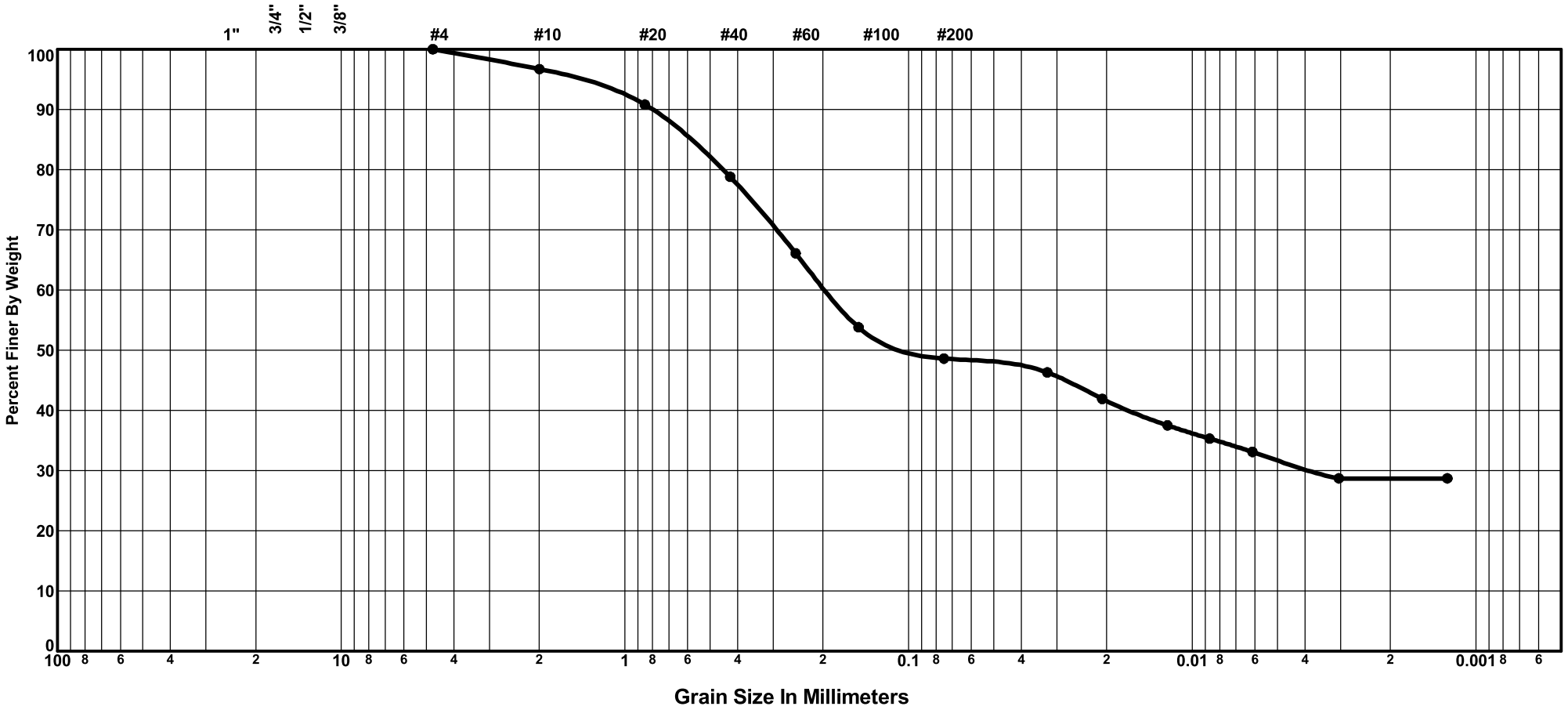
Date Recieved: 11/21/2022

Dates Tested: 11/21-12/12/2022

GRAIN SIZE DISTRIBUTION

GeoTechnologies, Inc.
3200 Wellington Ct
Raleigh, NC 27615

U.S. Standard Sieve Sizes



GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

Boring No.	Elev./Depth	Nat. W.C.	L.L.	P.L.	P.I.	Soil Description or Classification
B-3 S-5	8.0-9.0'		60.0	45.0	15.0	Brown Clayey Silty Medium to Fine Sand

Project:

Eastgate Park Dam
Raleigh, NC

Job No.: 1-22-1005-EA

Date: 12/13/22

Date Recieved: 11/21/2022

Dates Tested: 11/21-12/12/2022

GRAIN SIZE DISTRIBUTION

GeoTechnologies, Inc.

3200 Wellington Ct
Raleigh, NC 27615

GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

Boring No.	Elev./Depth	Nat. W.C.	L.L.	P.L.	P.I.	Soil Description or Classification
B-3 S-7	12.0-13.0'		62.0	38.0	24.0	Brown Fine to Medium Sandy Silty Clay

Project:

Eastgate Park Dam
Raleigh, NC

Job No.: 1-22-1005-EA

Date: 12/13/22

Date Recieved: 11/21/2022

Dates Tested: 11/21-12/12/2022

GRAIN SIZE DISTRIBUTION

GeoTechnologies, Inc.
3200 Wellington Ct
Raleigh, NC 27615

GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

Boring No.	Elev./Depth	Nat. W.C.	L.L.	P.L.	P.I.	Soil Description or Classification
B-4 S-5	8.0-9.0'		62.0	41.0	21.0	Brown Clayey Silty Fine to Medium Sand

Project:

Eastgate Park Dam
Raleigh, NC

Job No.: 1-22-1005-EA

Date: 12/13/22

Date Recieved: 11/21/2022

Dates Tested: 11/21-12/12/2022

GRAIN SIZE DISTRIBUTION

GeoTechnologies, Inc.
3200 Wellington Ct
Raleigh, NC 27615

GeoTechnologies, Inc.

LF-8

MOISTURE, ASH & ORGANIC MATTER OF PEAT & ORGANIC SOILS

ASTM D-2974

JOB NAME: Eastgate **JOB NO.:** 1-22-1005-EA

PERFORMED BY: CAS **DATE:** 12/22/2022

BORING: B-3 **SAMPLE:** S-3 **DEPTH:** 4-5'

SAMPLE DESCRIPTION: Brown Sandy Silt

MOISTURE CONTENT

	PAN NUMBER		15
A	**PAN WEIGHT**		121.71
B	**WET SOIL & PAN**		293.20
C	**DRY SOIL & PAN**		265.11
D	**WATER WEIGHT**	**B - C**	28.09
E	**DRY SOIL WEIGHT**	**C - A**	143.40
	MOISTURE CONTENT	**(D/E) X 100**	19.6

ASH & ORGANIC MATTER

	TIME IN FURNACE		8:00 AM
	TIME OUT OF FURNACE		3:00 PM
	TEMPERATURE, C		450 C
	CRUCIBLE NUMBER		2
A	**CRUCIBLE WEIGHT**		51.32
B	**DRY SOIL + CRUCIBLE**		102.05
C	**FURNACE FIRED SOIL + CRUCIBLE**		100.96
D	**ASH**	**C - A**	49.64
E	**OVEN DRIED SOIL (B-A)**	**B - A**	50.73
F	**ASH CONTENT, %**	**(D X 100)/E**	97.85
	ORGANIC MATTER, %	**100 - F**	2.1

GeoTechnologies, Inc.

LF-8

MOISTURE, ASH & ORGANIC MATTER OF PEAT & ORGANIC SOILS

ASTM D-2974

JOB NAME: Eastgate **JOB NO.:** 1-22-1005-EA

PERFORMED BY: CAS **DATE:** 12/22/2022

BORING: B-3 **SAMPLE:** S-9 **DEPTH:** 16-17'

SAMPLE DESCRIPTION: Brown Sandy Silt

MOISTURE CONTENT

	PAN NUMBER		9
A	**PAN WEIGHT**		119.21
B	**WET SOIL & PAN**		262.58
C	**DRY SOIL & PAN**		226.41
D	**WATER WEIGHT**	**B - C**	36.17
E	**DRY SOIL WEIGHT**	**C - A**	107.20
	MOISTURE CONTENT	**(D/E) X 100**	33.7

ASH & ORGANIC MATTER

	TIME IN FURNACE		8:00 AM
	TIME OUT OF FURNACE		3:00 PM
	TEMPERATURE, C		450 C
	CRUCIBLE NUMBER		3
A	**CRUCIBLE WEIGHT**		104.54
B	**DRY SOIL + CRUCIBLE**		192.26
C	**FURNACE FIRED SOIL + CRUCIBLE**		190.42
D	**ASH**	**C - A**	85.88
E	**OVEN DRIED SOIL (B-A)**	**B - A**	87.72
F	**ASH CONTENT, %**	**(D X 100)/E**	97.90
	ORGANIC MATTER, %	**100 - F**	2.1

GeoTechnologies, Inc.

LF-8

MOISTURE, ASH & ORGANIC MATTER OF PEAT & ORGANIC SOILS

ASTM D-2974

JOB NAME: Eastgate **JOB NO.:** 1-22-1005-EA

PERFORMED BY: CAS **DATE:** 12/22/2022

BORING: B-3 **SAMPLE:** S-10 **DEPTH:** 20-21'

SAMPLE DESCRIPTION: Gray Silty Sand

MOISTURE CONTENT

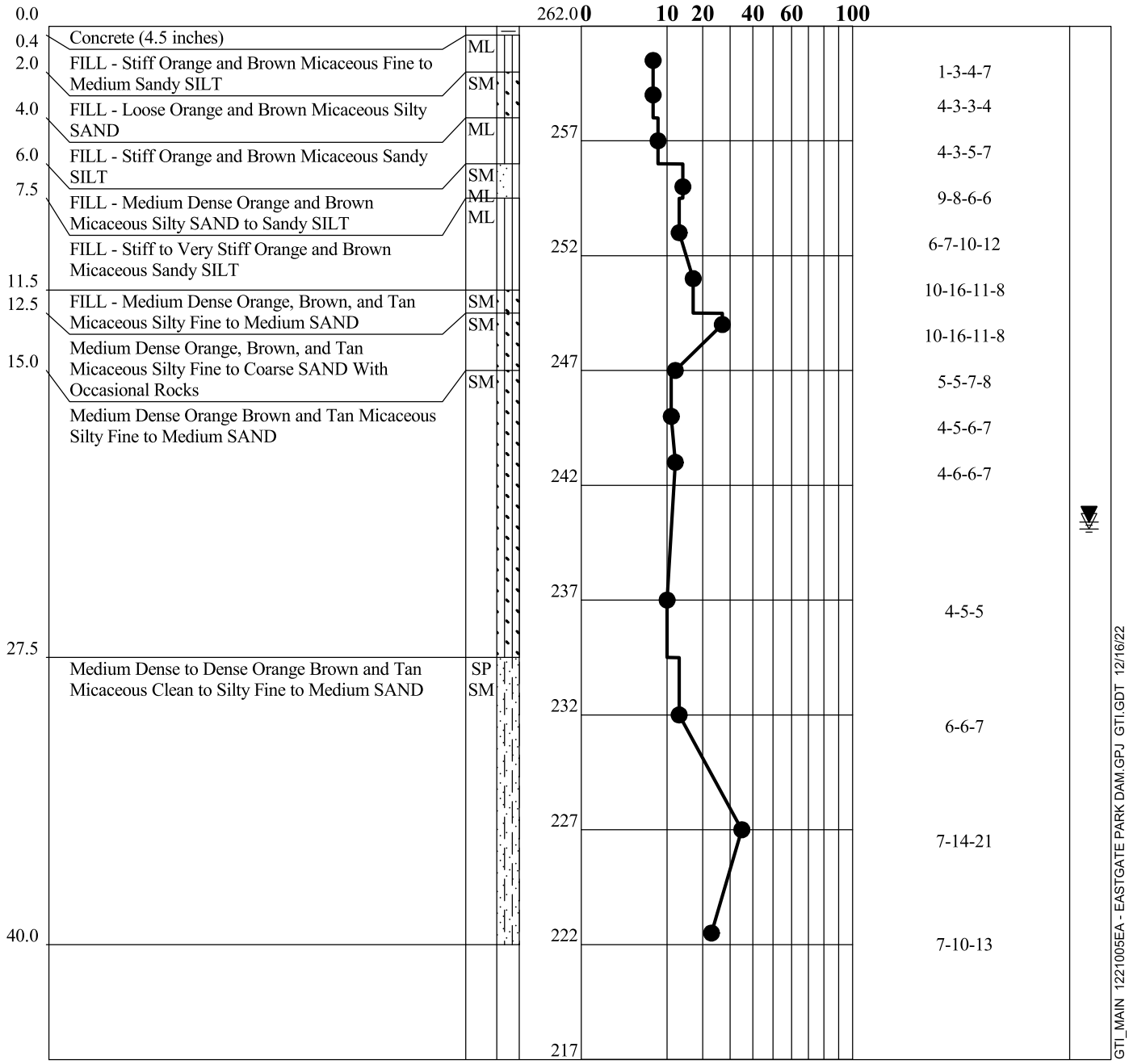
	PAN NUMBER		11
A	**PAN WEIGHT**		119.27
B	**WET SOIL & PAN**		304.73
C	**DRY SOIL & PAN**		272.36
D	**WATER WEIGHT**	**B - C**	32.37
E	**DRY SOIL WEIGHT**	**C - A**	153.09
	MOISTURE CONTENT	**(D/E) X 100**	21.1

ASH & ORGANIC MATTER

	TIME IN FURNACE		8:00 AM
	TIME OUT OF FURNACE		3:00 PM
	TEMPERATURE, C		450 C
	CRUCIBLE NUMBER		1
A	**CRUCIBLE WEIGHT**		50.91
B	**DRY SOIL + CRUCIBLE**		108.36
C	**FURNACE FIRED SOIL + CRUCIBLE**		107.31
D	**ASH**	**C - A**	56.40
E	**OVEN DRIED SOIL (B-A)**	**B - A**	57.45
F	**ASH CONTENT, %**	**(D X 100)/E**	98.17
	ORGANIC MATTER, %	**100 - F**	1.8

Boring Logs

TEST BORING RECORD

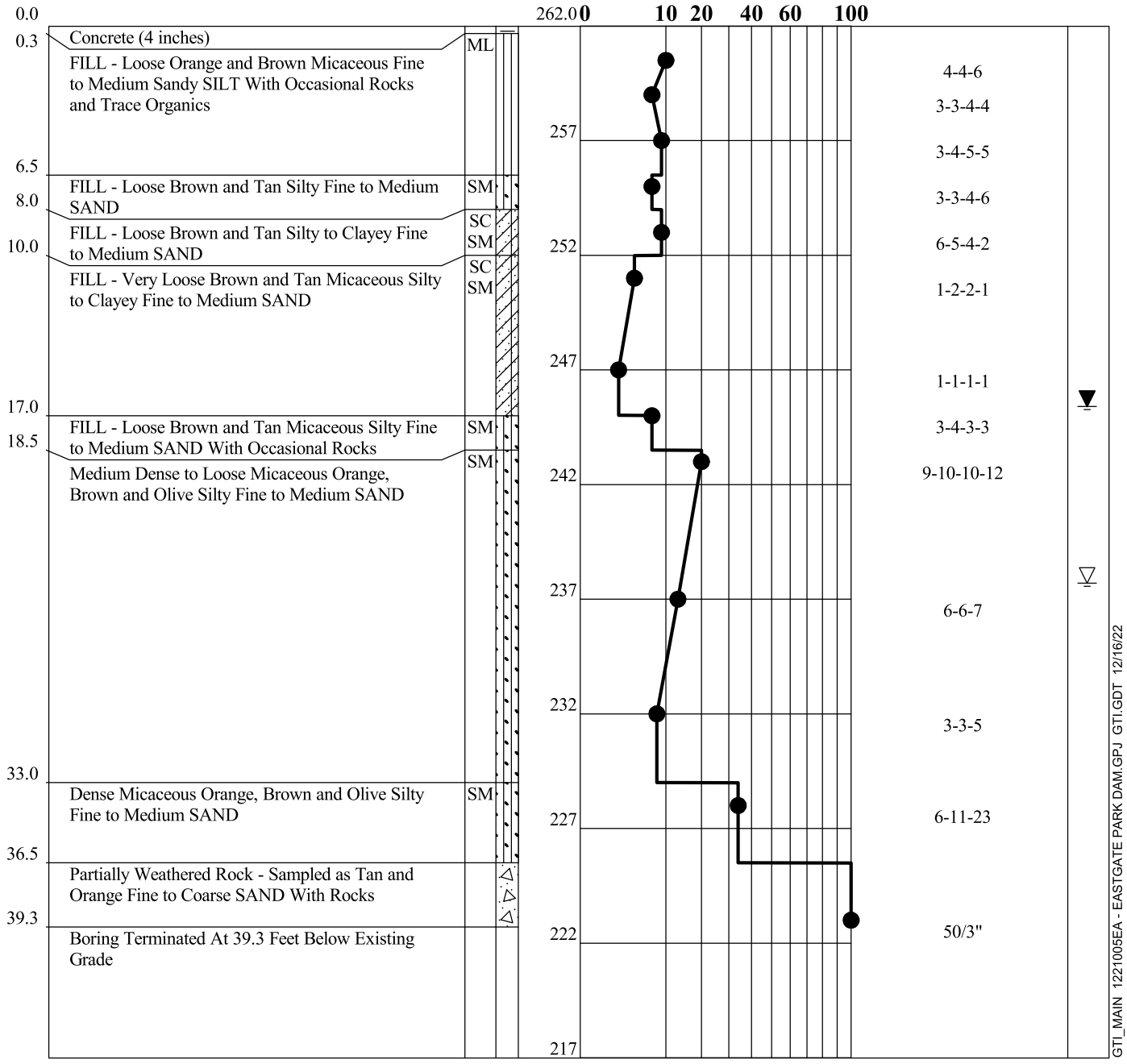


Groundwater Encountered At 21.9 ft Below Existing Grade At The Time of Boring. Groundwater Encountered At 21.6 ft Below Existing Grade 24 Hours After At The Time of Boring.

JOB NUMBER 122-1005EA
BORING NUMBER B-1
DATE 11-15-22

GeoTechnologies, Inc.

TEST BORING RECORD



Groundwater Encountered At 24.3 ft Below Existing Grade At The Time of Boring. Groundwater Encountered At 16.6 ft Below Existing Grade 24 Hours After At The Time of Boring.

JOB NUMBER 122-1005EA
BORING NUMBER B-2
DATE 11-16-22

GeoTechnologies, Inc.

DYNAMIC HAND CONE PENETROMETER RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	PENETRATION PER INCREMENT	BLOWS PER 1 3/4"
0.0	Topsoil	246.0	0 10 20 40 60 100	
0.5	FILL - Stiff Orange and Brown Micaceous Sandy SILT With Rocks (N est. = 15+)	ML		
2.0	Stone and Rocks			
2.2	Hand Auger Refusal at 2.2 Feet Below Existing Grade	241		

Dry At Time of Boring

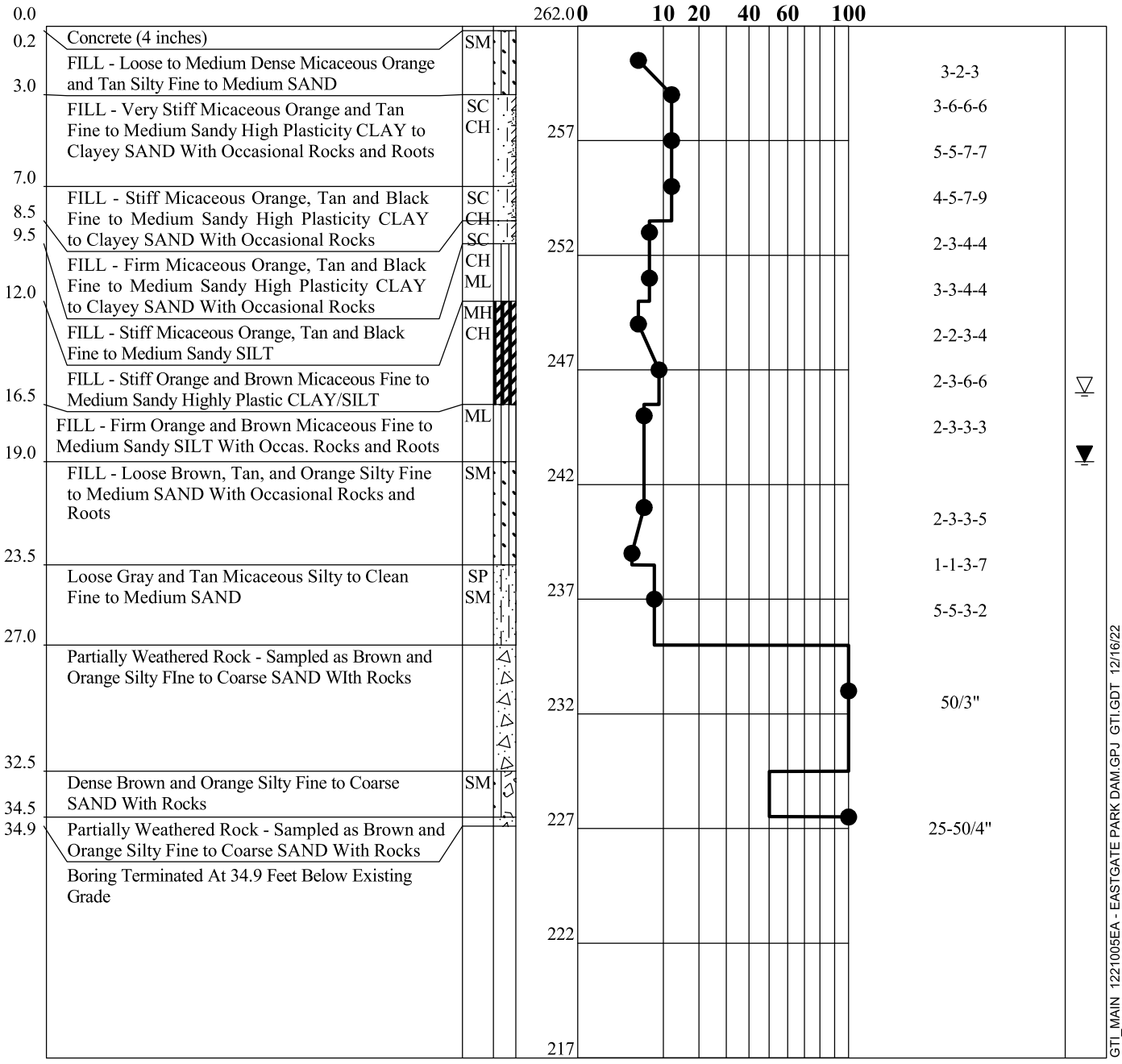
GTI_MAIN 1221005EA - EASTGATE PARK DAM.GPJ GTI.GDT 12/16/22

JOB NUMBER 122-1005EA
BORING NUMBER H-1
DATE 11-16-22

PAGE 1 OF 1

GeoTechnologies, Inc.

TEST BORING RECORD



Groundwater Encountered At 19 ft Below Existing Grade At The Time of Boring. Groundwater Encountered At 19 ft Below Existing Grade 24 Hours After At The Time of Boring.

JOB NUMBER 122-1005EA
BORING NUMBER B-3
DATE 11-16-22

GeoTechnologies, Inc.

DYNAMIC HAND CONE PENETROMETER RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	PENETRATION PER INCREMENT	BLOWS PER 1 3/4"
0.0		258.0	0 10 20 40 60 100	
	FILL - Orange and Brown Medium Sandy SILT WIth Occasional Seams of Silty SAND (N est. = 4 - 8)	ML		
4.0	Boring Terminated At 4 Feet Below Existing Grade	253		

Dry At Time of Boring

GTI_MAIN 1221005EA - EASTGATE PARK DAM.GPJ GTI.GDT 12/16/22

JOB NUMBER 122-1005EA
BORING NUMBER H-2
DATE 11-17-22

GeoTechnologies, Inc.

DYNAMIC HAND CONE PENETROMETER RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	PENETRATION PER INCREMENT	BLOWS PER 1 3/4"
0.0	Topsoil	246.0	0 10 20 40 60 100	
0.5	FILL - Stiff Orange and Brown Micaceous Sandy SILT With Rocks (N est. = 15+)	ML		
2.0	Stone and Rocks			
2.2	Hand Auger Refusal at 2.2 Feet Below Existing Grade	241		

Dry At Time of Boring

GTI_MAIN 1221005EA - EASTGATE PARK DAM.GPJ GTI.GDT 12/16/22

JOB NUMBER 122-1005EA
BORING NUMBER H-3
DATE 11-16-22

PAGE 1 OF 1

GeoTechnologies, Inc.

DYNAMIC HAND CONE PENETROMETER RECORD

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	PENETRATION PER INCREMENT	BLOWS PER 1 3/4"
0.0		246.0	0 10 20 40 60 100	
	Medium Dense Orange, Brown, and Tan Micaceous Silty Fine to Coarse SAND With Occasional Rocks (Estimated N est. = 8)	SM		
		241		
6.0	Boring Terminated At 6 Feet Below Existing Grade			

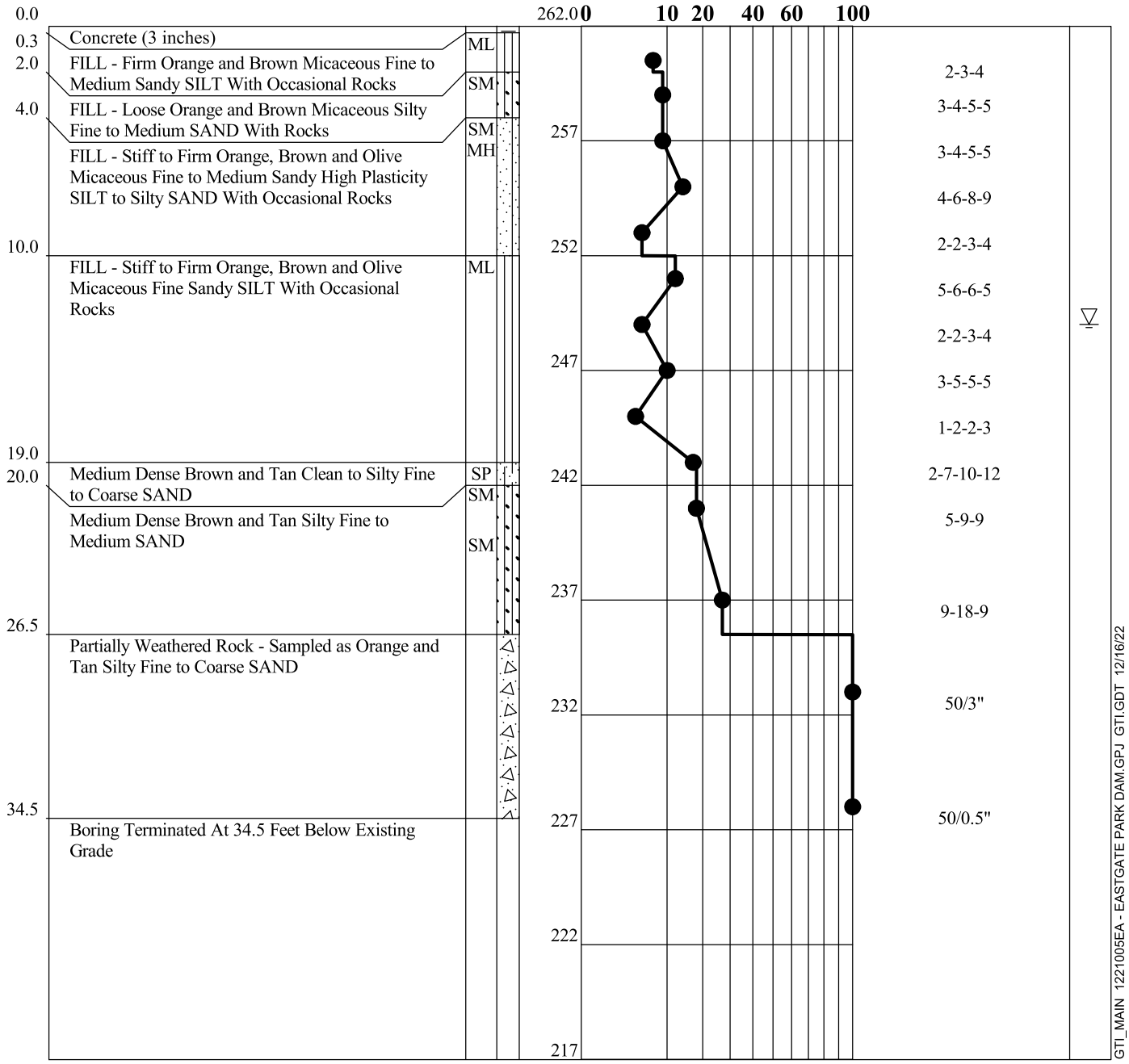
Dry At Time of Boring

GTI_MAIN 1221005EA - EASTGATE PARK DAM.GPJ GTI.GDT 12/16/22

JOB NUMBER 122-1005EA
BORING NUMBER H-4
DATE 11-17-22

GeoTechnologies, Inc.

TEST BORING RECORD

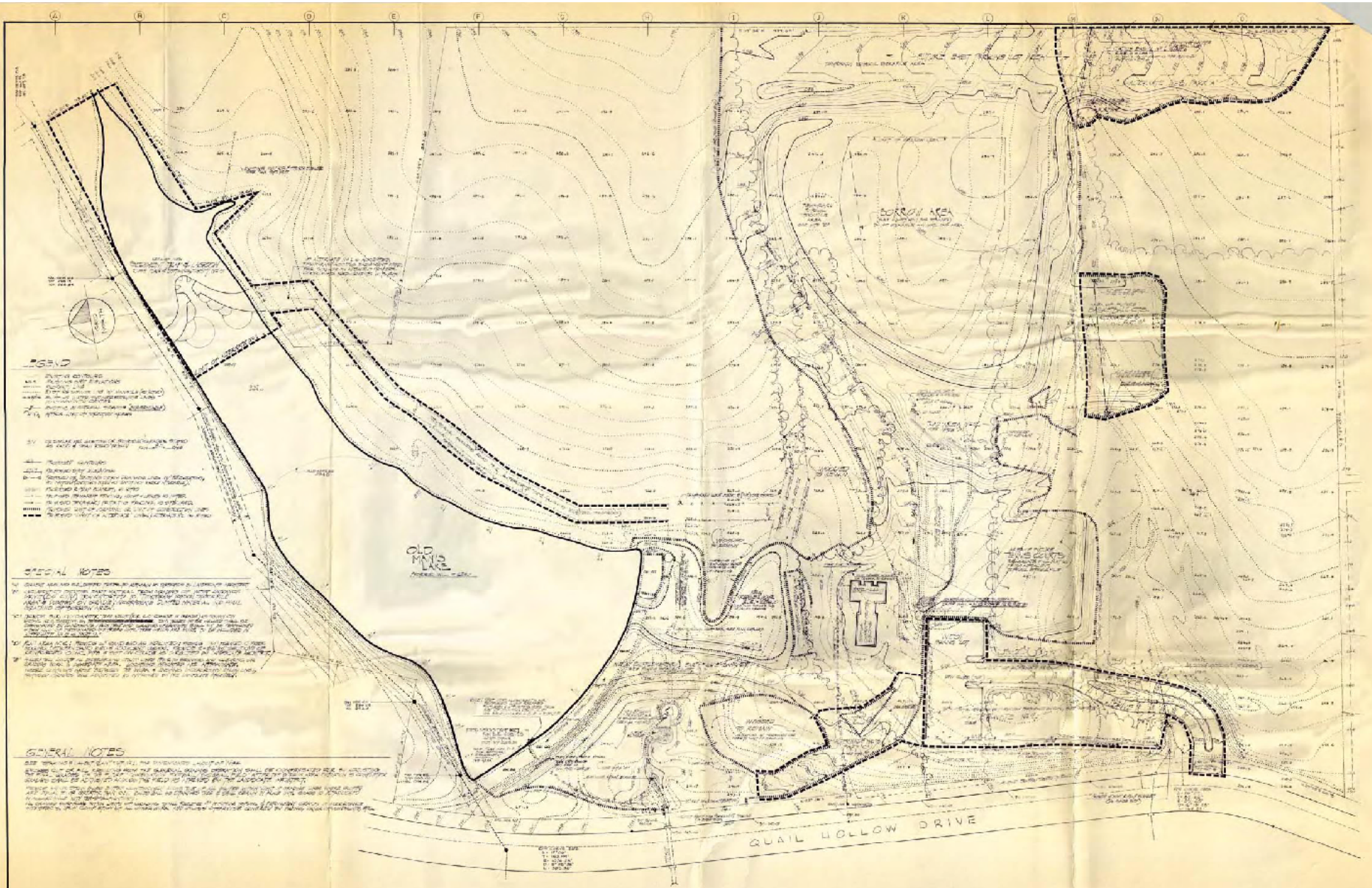


Groundwater Encountered At 13.0 ft Below Existing Grade At The Time of Boring.

JOB NUMBER 122-1005EA
BORING NUMBER B-4
DATE 11-17-22

GeoTechnologies, Inc.

Historical Construction Plans



EASTGATE PARK

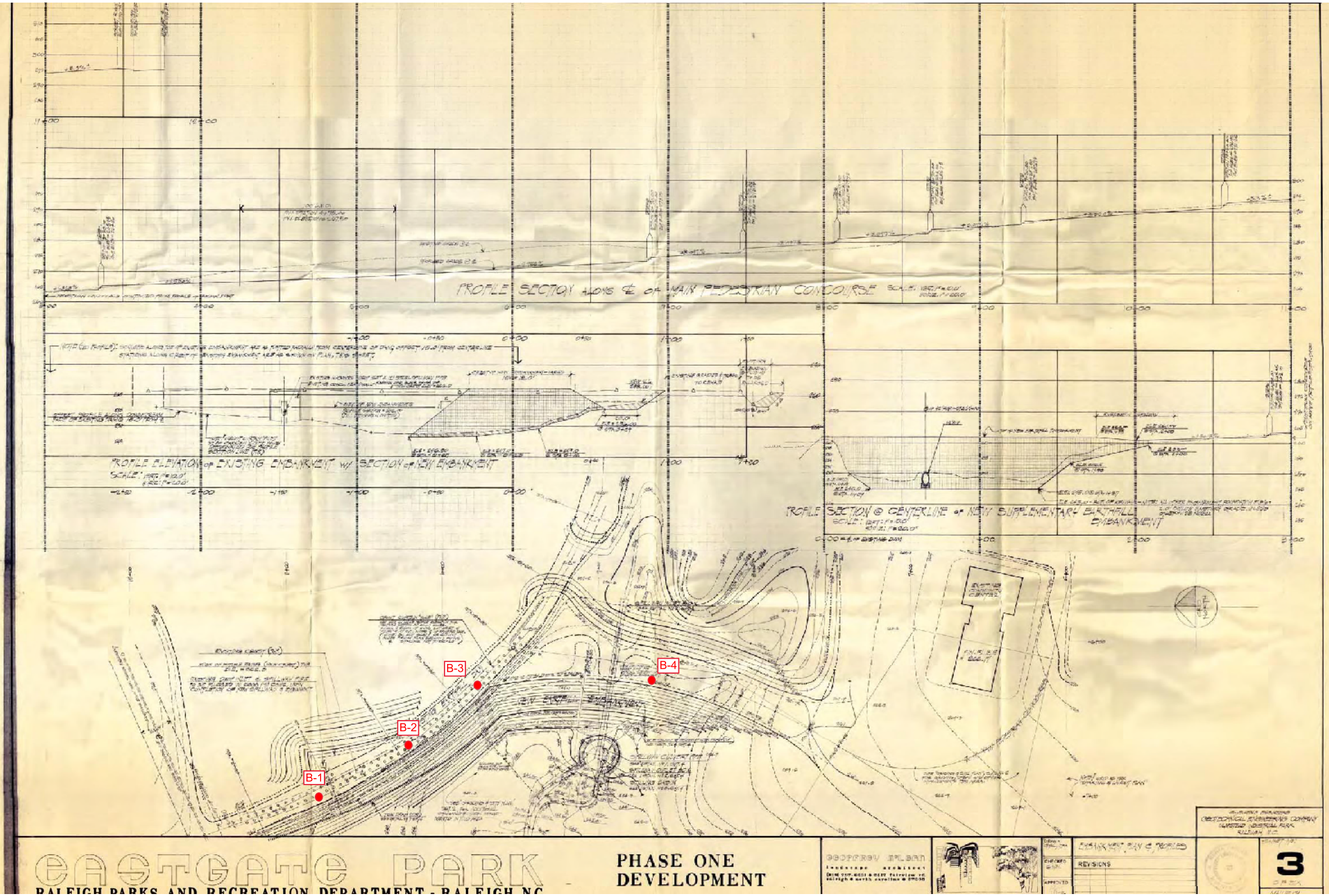
RALEIGH PARKS AND RECREATION DEPARTMENT - RALEIGH, N.C.

PHASE ONE DEVELOPMENT

GRADING & SITE PLAN

REVISIONS

2



EASTGATE PARK

RALEIGH PARKS AND RECREATION DEPARTMENT - RALEIGH, N.C.

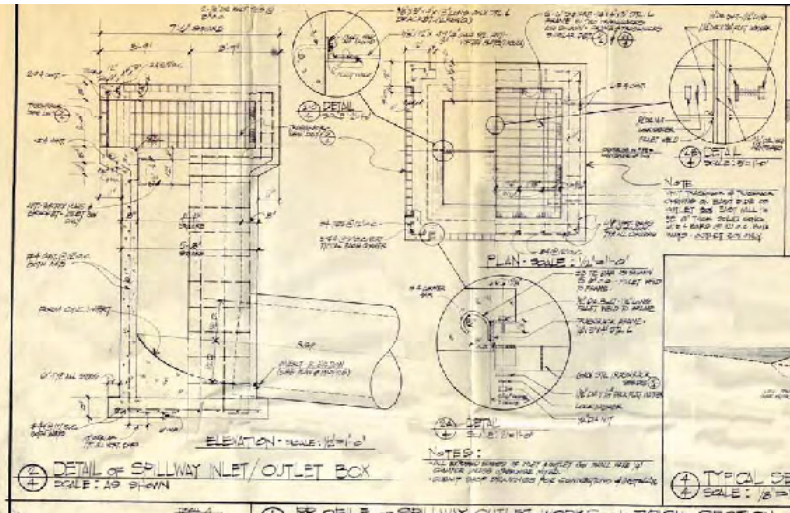
PHASE ONE DEVELOPMENT

GEOFFREY EPLER
landscape architect

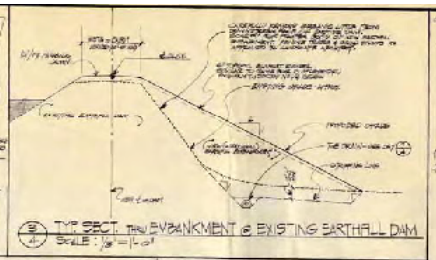
REVISIONS

EMBANKMENT PLAN & PROFILES

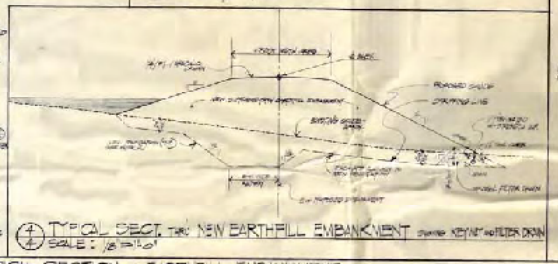
3



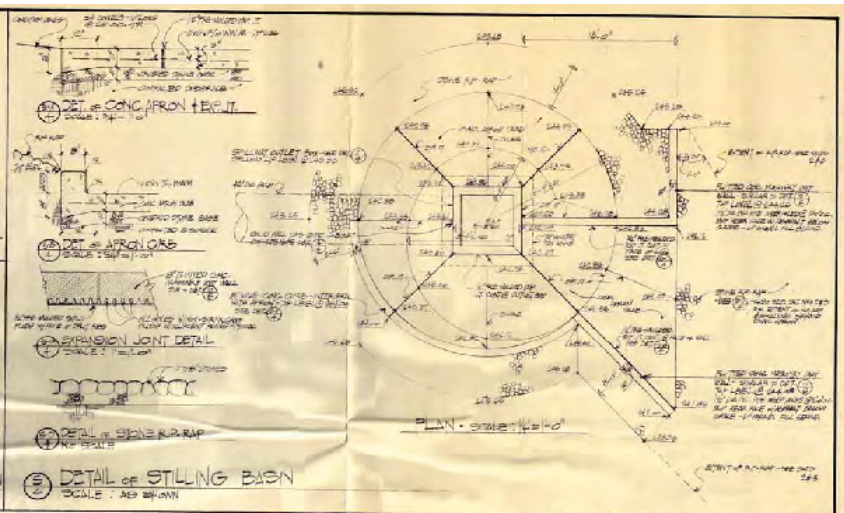
DETAIL OF SPILLWAY INLET/OUTLET BOX
SCALE : AS SHOWN



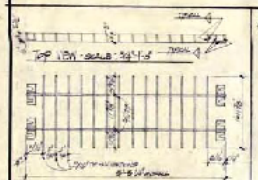
TYP. SECT. THRU EMBANKMENT @ EXISTING EARTHFILL DAM
SCALE : 1/8"=1'-0"



TYPICAL SECT. THRU NEW EARTHFILL EMBANKMENT
SCALE : 1/8"=1'-0"

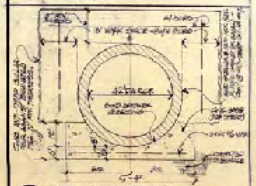


DETAIL OF STILLING BASIN
SCALE : AS SHOWN



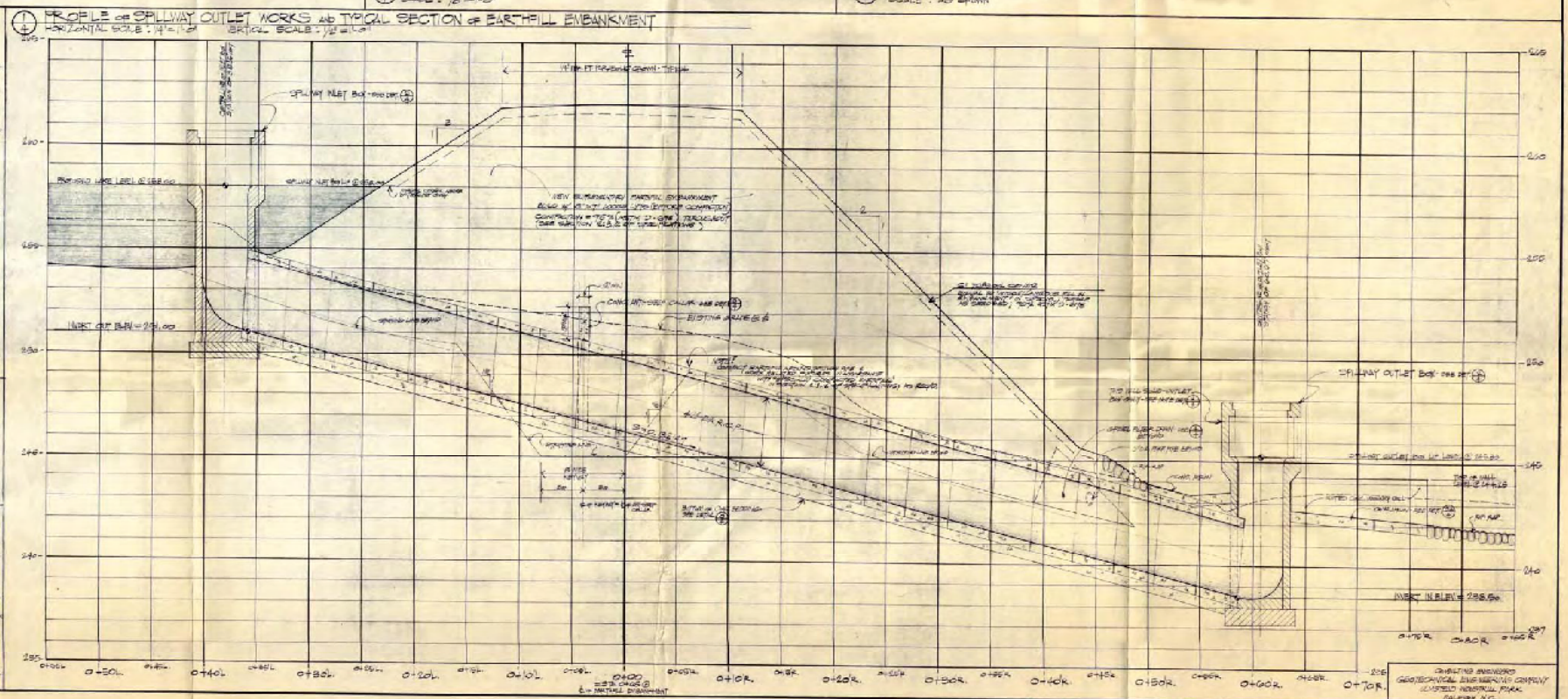
GAIN TRASHRACK DETAIL
AS SHOWN

DETAIL OF TOE DRAIN
SCALE : 1/2"=1'-0"



DET. OF CONC. BEDDING
SCALE : 1/2"=1'-0"

PROFILE OF SPILLWAY OUTLET WORKS AND TYPICAL SECTION OF EARTHFILL EMBANKMENT
HORIZONTAL SCALE : 1"=1'-0" VERTICAL SCALE : 1/2"=1'-0"



EASTGATE PARK

RALEIGH PARKS AND RECREATION DEPARTMENT

PHASE ONE DEVELOPMENT

BRADFORD McLEAN
landscape architect
8510 FALLS OF NEUSE RD. Fairview Rd
raleigh north carolina 27615

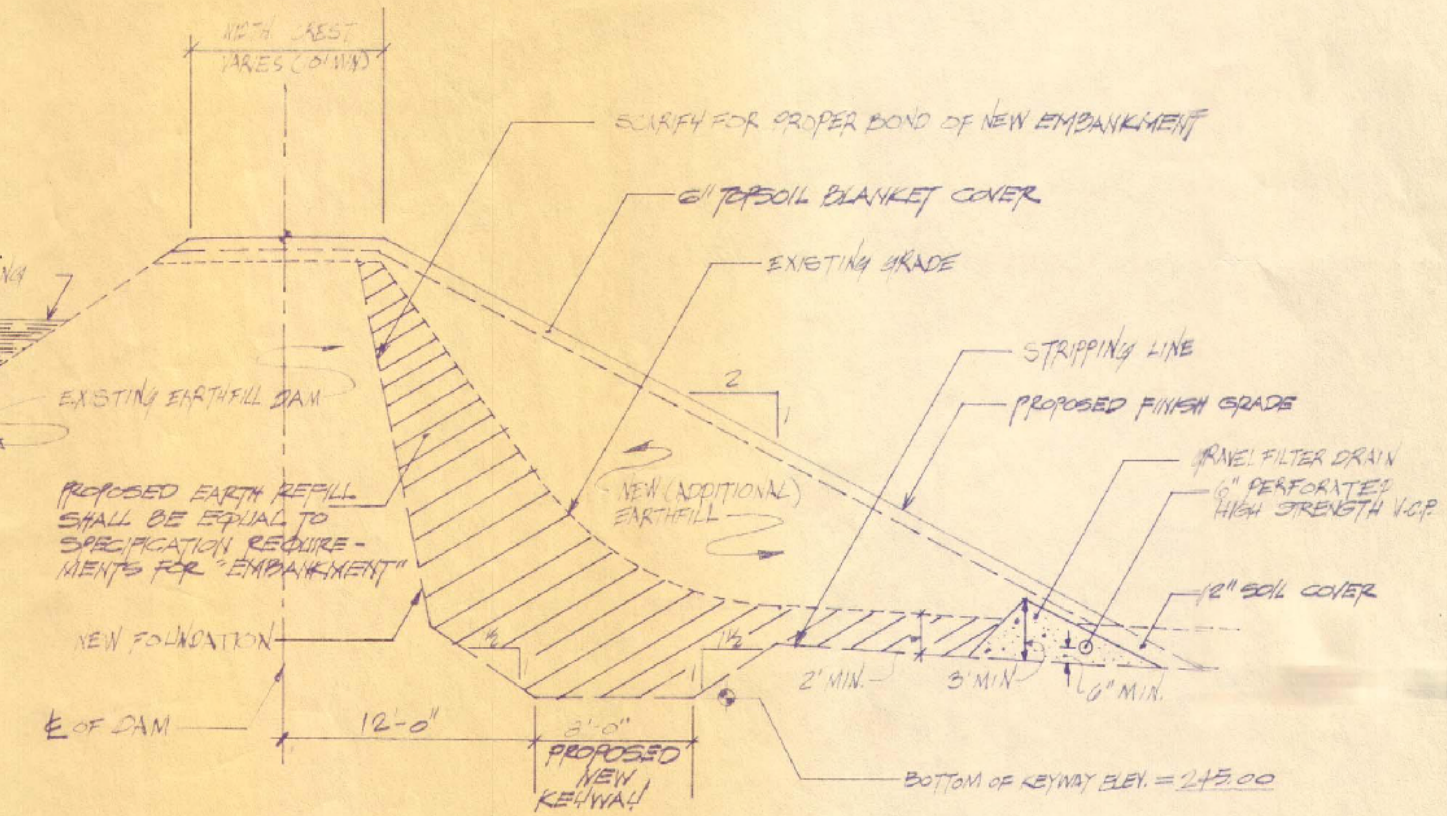
DRAWN JH
CHECKED JH
APPROVED

SWIM MOMENT SECTIONS & DETAILS

REVISIONS

CIVIL ENGINEERS
GEOTECHNICAL ENGINEERING COMPANY
CUSTELO INDUSTRIAL PARK
RALEIGH, N.C.

SHEET NO.
4



GENERAL NOTES:

1. TYP SECT THRU EMBANKMENT @ EXISTING EARTHFILL DAM SHOWING PROPOSED KEYWAY AND FILTER DRAIN
2. THE ABOVE KEYWAY WILL RUN ALONG ENTIRE LENGTH OF DOWNSTREAM FACE OF EXISTING DAM AND BE TIED TO THE KEYWAY OF NEW EARTHFILL EMBANKMENT AS SHOWN ON SHEET No. 4 OF 6 IN PLANS
3. SEE REQUIREMENTS OF ACCOMPANYING WRITTEN ORDER No. 1
4. ALL EARTHFILL-IN-EMBANKMENT & EARTH REFILL SHALL BE COMPACTED IN ACCORDANCE WITH ASTM D-698 FOR 95% COMPACTIVE STRENGTH

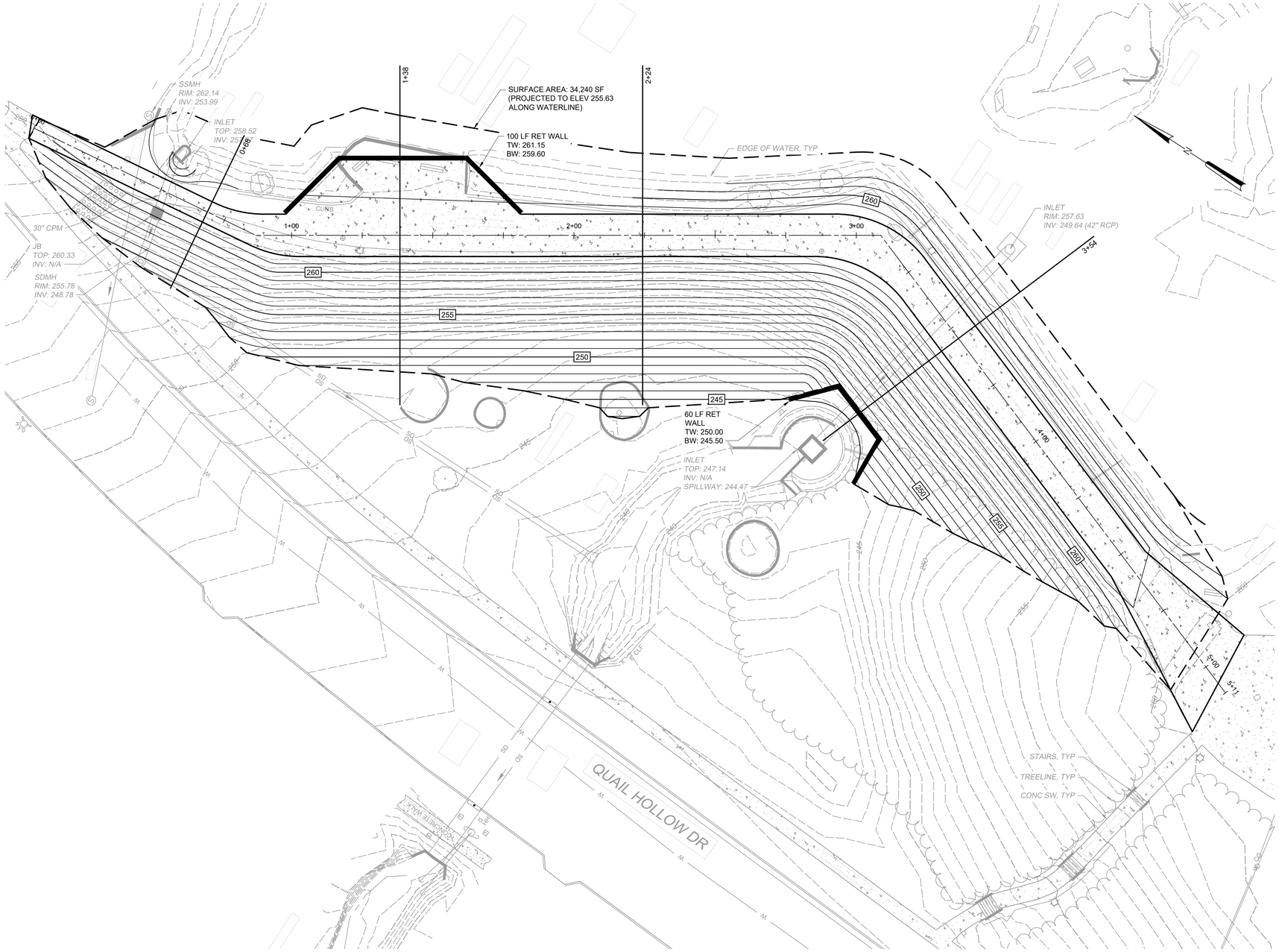
EASTGATE PARK
PHASE ONE DEVELOPMENT
REVISED SECTION (3/4) SHOWING NEW KEYWAY AS PROPOSED BY CONTRACTOR

GEORGE McLEAN & CO
LANDSCAPE ARCHITECTS
257 FAIRVIEW ROAD · RALEIGH, N.C.

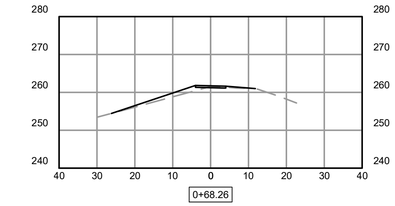
SCALE: NONE	DRN: KEH/JRH	CHKD	DATE 6/4/75	REV.

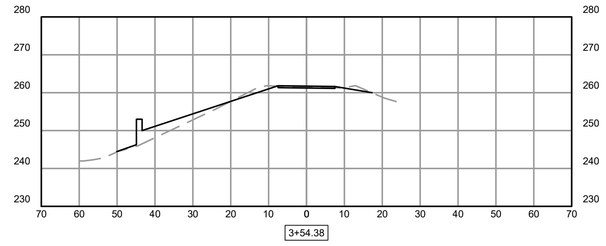
SHEET WO-1b

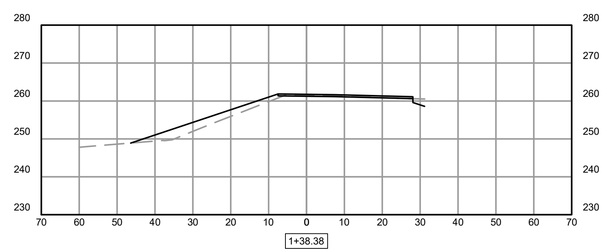
Appendix F: Conceptual Grading Plan

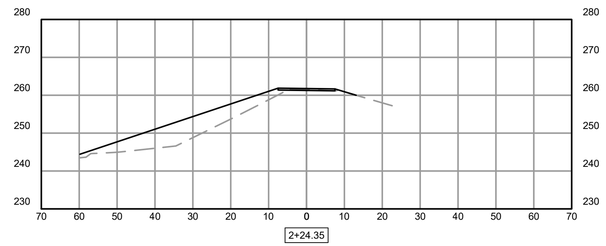


PLAN
SCALE: 1" = 20'









REV	ISSUED FOR	DATE	BY

PROJECT ENGINEER: EHK
DESIGNED BY:
DRAWN BY:
CHECKED BY:
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO FULL SCALE 0 1/2" 1"

PRELIMINARY DRAWING
DO NOT USE FOR
CONSTRUCTION

Hazen
HAZEN AND SAWYER
4011 WESTCHASE BLVD, SUITE 500
RALEIGH, NC 27607

CITY OF RALEIGH, NORTH CAROLINA
PUBLIC UTILITIES DEPARTMENT

EASTGATE PARK DAM
IMPROVEMENTS

DATE: MAY 2023
HAZEN NO.: 32511-005
CONTRACT: XXXX
DRAWING NUMBER:

File: O:\32511-RAL\32511-005\CAD_BIM\CIVIL\C1 Saved by JWALDRON Save date: 5/5/2023 8:21 AM
PLOT DATE: 5/30/2023 2:52 PM BY: JWALDRON