ADDENDUM NUMBER TWO

DUPONT PUMP STATION AND BASIN IMPROVEMENTS – PHASE 2 (Contract A)
W-12-026-202

CITY OF CHATTANOOGA, TENNESSEE

The following changes shall be made to the Contract Documents, Specifications, and Drawings:

I. CONTRACT DOCUMENT

- Add “Attachment A to 13 60 13 Pre-Fabricated Restroom” to Section 13 60 13 Pre-Fabricated Restroom.
- Geotechnical Report prepared by CDM Smith is attached, appendices are included in Addendum No. 1. Contractors may rely on the data presented in this report. However, reliance on any interpretations of such data are at the Contractor’s sole risk.

December 3, 2019

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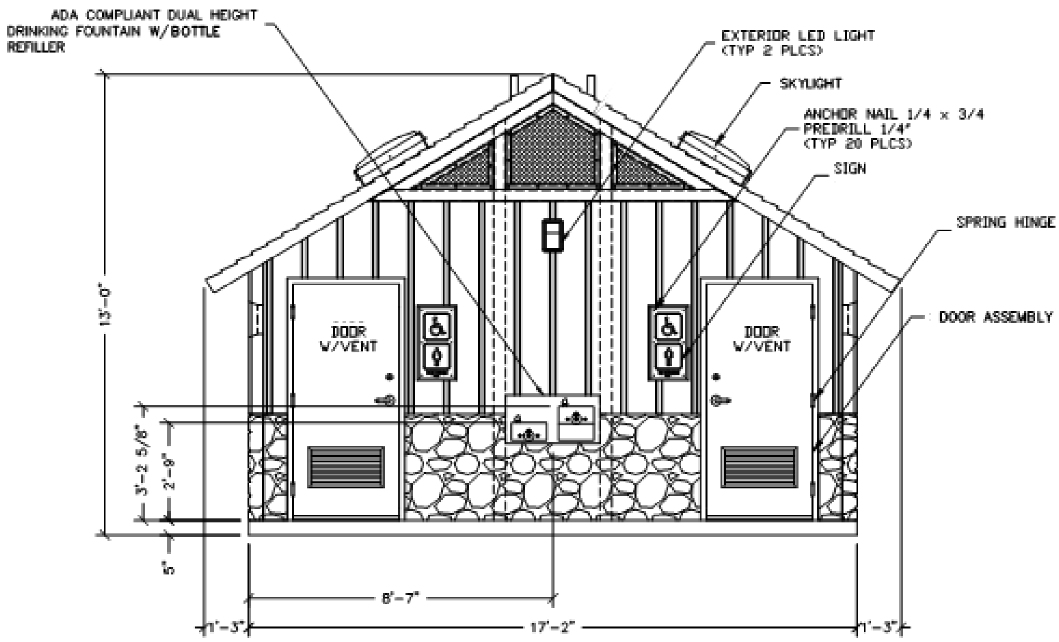
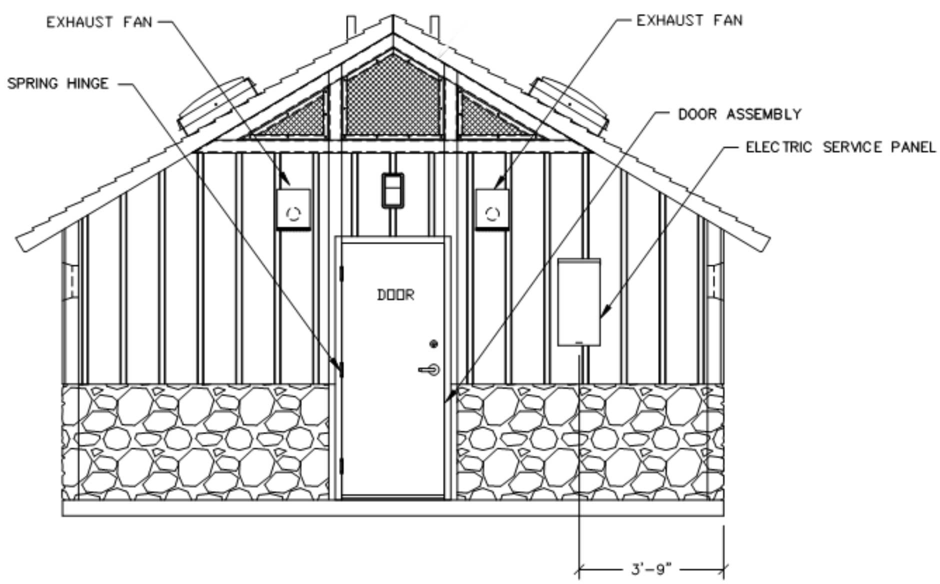
Justin C Holland, Administrator
City of Chattanooga



FRONT ELEVATION



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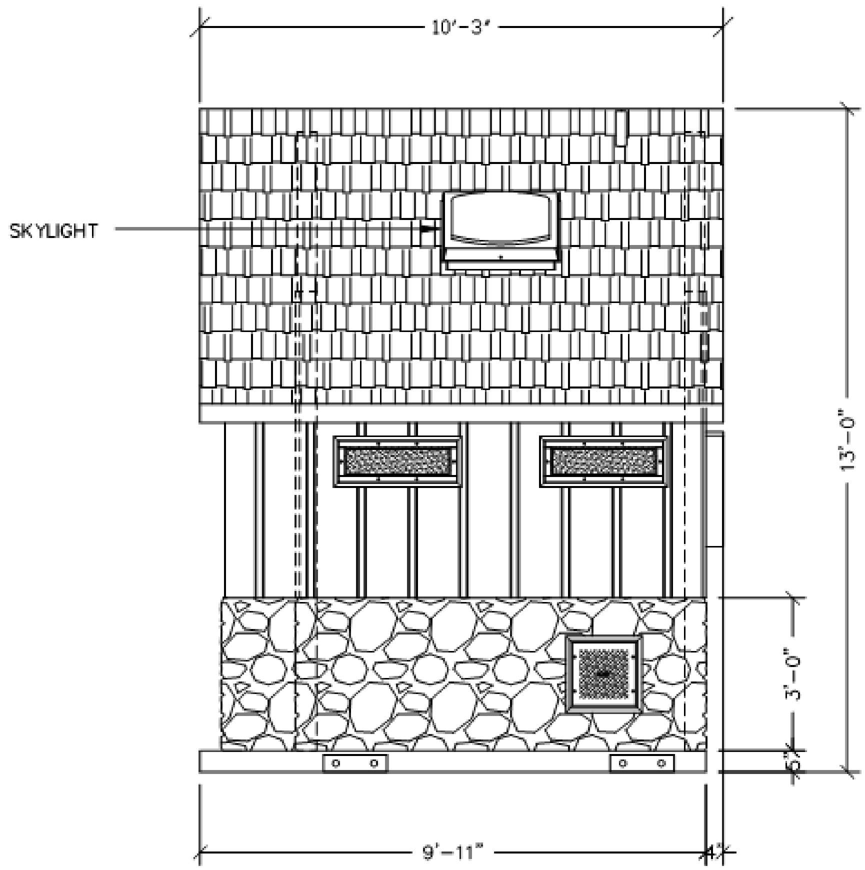
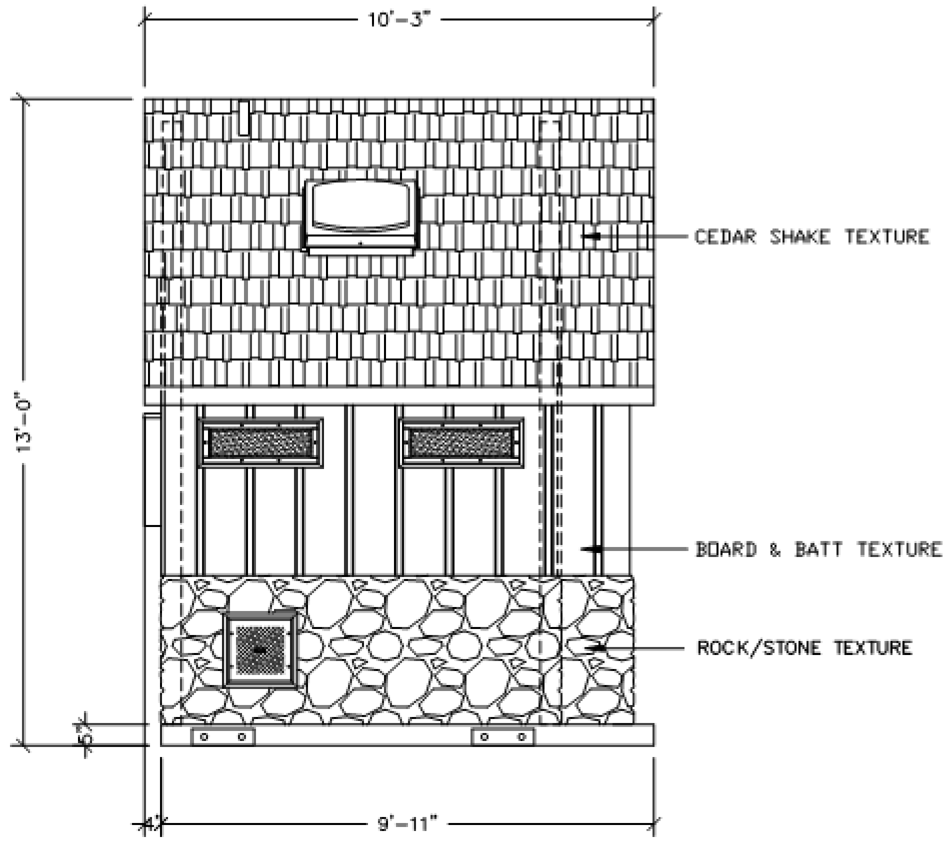


RIGHT SIDE ELEVATION



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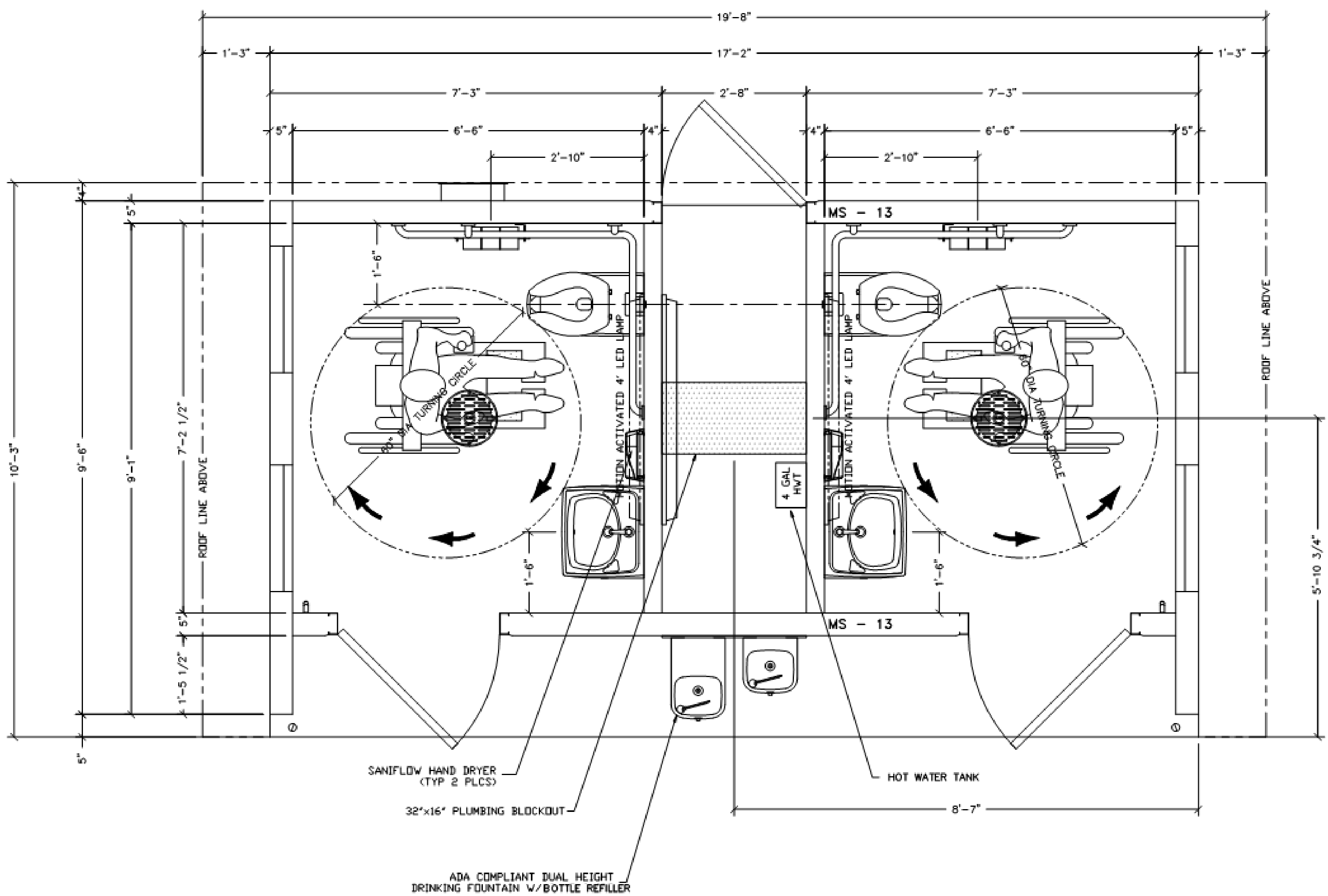
19'-8"
1'-3"
17'-2"
1'-3"
7'-3"
2'-8"
7'-3"
5"
6'-6"
4"
4"
6'-6"
5"
2'-10"
2'-10"
MS - 13
MS - 13
1'-6"
1'-6"
1'-6"
60" DIA TURNING CIRCLE
60" DIA TURNING CIRCLE
MOTION ACTIVATED 4' LED LAMP
MOTION ACTIVATED 4' LED LAMP
4 GAL HWT
ROOF LINE ABOVE
ROOF LINE ABOVE
10'-3"
9'-6"
9'-1"
7'-2 1/2"
5"
1'-5 1/2"
5"
5'-10 3/4"
8'-7"
SANIFLOW HAND DRYER
(TYP 2 PLCS)
32"x16" PLUMBING BLOCKOUT
HOT WATER TANK
ADA COMPLIANT DUAL HEIGHT
DRINKING FOUNTAIN W/BOTTLE REFILLER

City of Chattanooga

Waste Resources Division
DuPont Gravity Sewer and Pump Station

Geotechnical Interpretive Report

Chattanooga, Tennessee

September 2019

CDM Smith

City of Chattanooga

Waste Resources Division
DuPont Gravity Sewer and Pump Station

Geotechnical Interpretive Report

September 2019

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STATE OF TENNESSEE
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CDM Smith Project No. 129699-109746

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Section 1

Introduction

1.1 Project Description

The DuPont Pump Station and Basin Improvements – Phase 2 project scope consists of the design and construction of approximately 7,000 LF of 48-inch-diameter gravity sewer line from the existing DuPont Pump Station to Rivermont Park. It also includes the design and construction of a new wet-weather diversion structure and pump station in Rivermont Park. The new pump station will discharge into the existing DuPont Pump Station force main and will maximize its capacity. The project also involves the demolition of the existing Dupont Pump Station and existing diversion structure. The primary objective of this project is to reduce sanitary sewer overflows (SSOs) in the DuPont Parkway Pump Station drainage area and the Lupton drainage area through the construction of new wet-weather flow management facilities.

The location of the proposed structures and the alignment of the gravity sewer are shown on **Figure 1-1.** Existing site elevation at the pump station site varies between El. 652 feet and El. 655 feet. The final site grade will be at El. 660 feet to protect against 100-yr flood level of El. 659 feet. The pump station and diversion structure will be founded on mat foundations at approximately 26- feet below ground surface (ft-bgs). The electrical building will be founded on a strip foundation at approximately 5 ft-bgs, while the generator slab will be founded at approximately 3 ft-bgs. All depths indicate bottom of foundation.

The new 48-inch-diameter finished gravity sewer will be ductile iron (DIP) and constructed using mainly open-cut and pipe jacking techniques. Pipe jacking will be used under the railroad crossing as indicated on Figure 1-1.

The location for the pump station and associated structures was initially intended to be at the location about 447 feet west of the current site (**Figure 1-2**). This initial site was found to be underlain by large karstic voids and cavities and therefore was abandoned.

This report summarizes previous field investigations, recent field investigation, and laboratory testing programs for design of the proposed new pump station, structures, and finished sewer line.

1.2 Elevation Datum

All elevations noted herein are reported in feet in reference to the North American Vertical Datum of 1988 (NAVD88).

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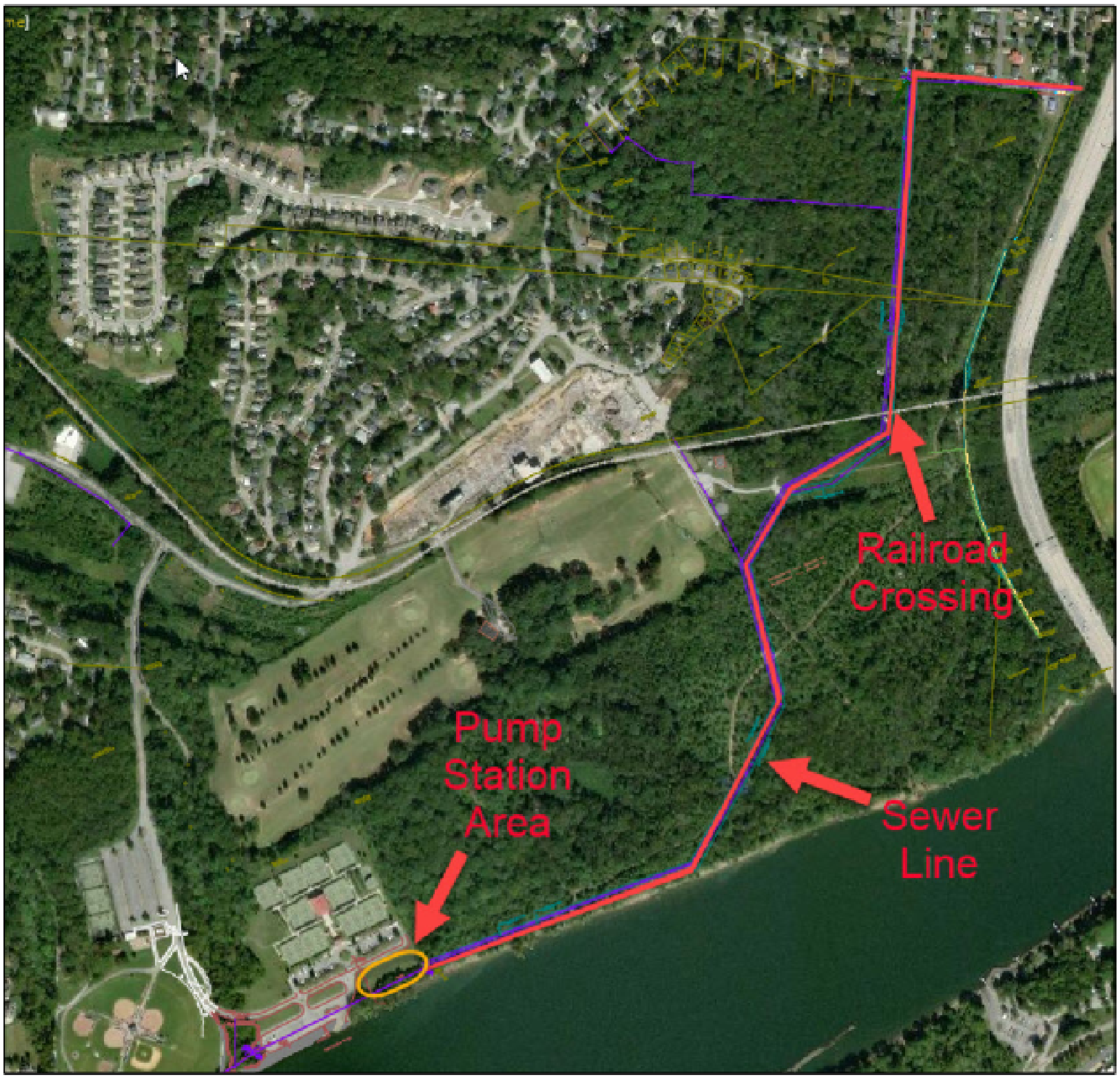

CDM Smith

Figure No. 1-1
Site Locus Plan
JULY 2019

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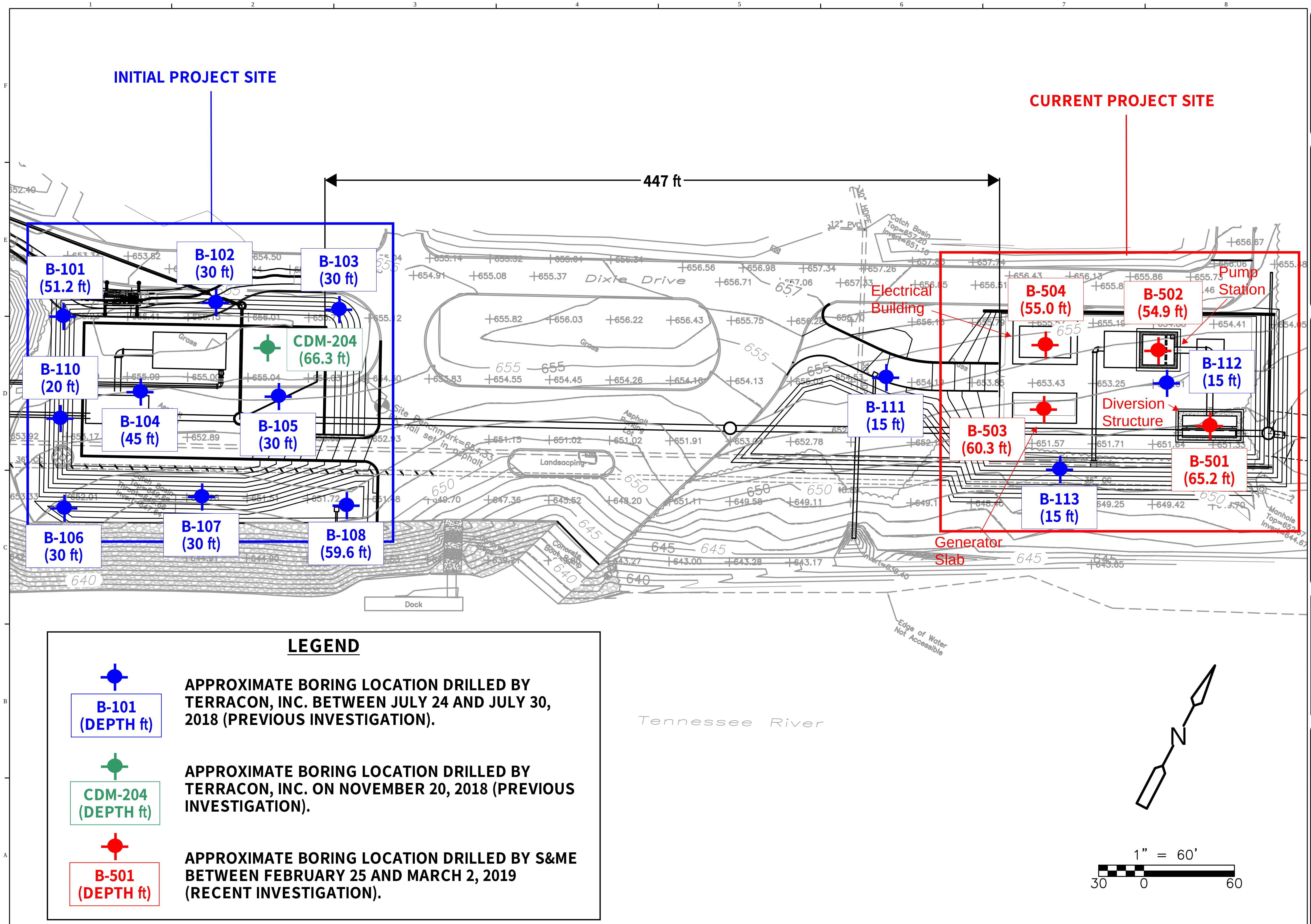

CDM Smith

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Chattanooga, TN 37403
Tel: (423) 771-4495

DUPONT PUMP STATION AND BASIN IMPROVEMENTS - PHASE 2
CITY OF CHATTANOOGA, TN
CONSENT DECREE PROGRAM

REV	DATE	REVISION DESCRIPTION

THIS LINE IS ONE INCH LONG WHEN PLOTTED FULL SCALE

THIS DRAWING MUST BE USED IN CONJUNCTION WITH THE APPLICABLE OR GOVERNING TECHNICAL SPECIFICATIONS AND OTHER CONTRACT DOCUMENTS.

PROJECT NO: 129699-109746

DATE: FEBRUARY 2019

DISC. LEAD: | DESIGNER: | CHECKER:

SHEET TITLE

FIGURE 1-2 SITE LOCATION PLAN

GEOTECHNICAL DATA REPORT

SHEET

60% SUBMITTAL - NOT FOR CONSTRUCTION

PLOT DATE: 3/21/2019
BY: CHERCHIAMD
LAST SAVED: 3/21/2019
CREATED: 3/11/2019

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1.3 Purpose and Scope

The purpose of this report is to provide geotechnical engineering recommendations for design and construction. Specifically, the scope of work included the following:

- Review subsurface information within the vicinity of the project site as collected during the preliminary and secondary field investigations;
- Drill four (4) test borings for the proposed structures and pipeline gravity sewer pipeline;
- Conduct geotechnical laboratory testing on select soil and rock samples to assist with classification and estimate the engineering properties of the materials;
- Perform geotechnical analyses and develop geotechnical engineering recommendations for design and construction of the proposed structures and gravity sewer pipeline; and
- Prepare this report presenting CDM Smith's recommendations and the data collected as part of the field investigations.

1.4 Report Limitations

The recommendations in this report have been prepared for the design of the Dupont Pump Station and Basin Improvements – Phase 2 project located in Chattanooga, Tennessee as understood at this time and described in this report. This report has been prepared in accordance with generally accepted engineering practices. No other warranty, express or implied, is made. In the event that changes in design or location of the proposed improvements occur, the conclusions and recommendations contained herein should not be considered valid unless verified in writing by CDM Smith.

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Section 2

Site and Subsurface Conditions

2.1 Site Conditions

2.1.1 General

The new 48-inch-diameter finished gravity sewer line will extend approximately 7,000 linear feet from DuPont Parkway to Dixie Drive in Chattanooga, Tennessee. The pump station will be just south of Dixie Drive, adjacent to the Champions Tennis Club. To the south of the site is the Tennessee River and to the west is Rivermont Park. A public easement runs through a heavily wooded area to the east, and bends to the north, crossing a railroad line and terminating at a residential neighborhood on the corner of Atlanta Drive and Elm Street. The plan view of the project extent is shown on Figure 1-1.

The existing site grades at the proposed pump station, electrical building, emergency generator building, and diversion structure range from about El. 652 to El. 655. Along the gravity sewer alignment, the existing grade ranges from El. 654 at the pump station site to El. 664 at Elm Street.

The finished gravity sewer alignment crosses under one (1) railroad as shown on Figure 1-1. The railroad crossing cannot be constructed using open-cut trenching, so trenchless construction techniques will be required.

2.2 Regional Geology

The project site is located within the Valley and Ridge Province. Subsurface conditions are characterized by parallel valleys and ridges oriented southwest-northeast consisting of Paleozoic sedimentary deposits. The bedrock in this region typically consists of sandstone underlain by limestone, dolomite, and shale. The limestone and dolomite are susceptible to dissolution along joints and bedding planes that results in weathering within the bedrock and near the overburden-bedrock interface. Cavities and large voids can develop as the weathering progresses. This geologic phenomenon is referred to as a Karstic condition. Soil or rock overlying voids can be stable due to arching; however, an unstable arch can develop as the void grows resulting in a sinkhole.

Based on the United States Geological Survey, the project site consists of the upper Knox Group, including Newala Formation, Mascot Dolomite, Kingsport Formation, Longview Dolomite, and Chepultepec Dolomite. Rocks are light gray, fine-grained dolomite with interbeds of blueish-gray limestone.

2.3 Subsurface Investigation Programs

2.3.1 General

Under subcontract to CDM Smith, Terracon, Inc., and S&ME, Inc. conducted subsurface investigation programs to provide site-specific information in the vicinity of the pump station and associated structures, and along the alignment of the gravity sewer. As shown on Figure 1-2, the initial site location was about 450 feet east of the current site. The general sequence of the field investigation activities was as follows:

1) Preliminary field investigation at the initial site location for the pump station and associated structures as well as the test borings along the sewer main.

2) Geophysical survey at the initial site location, after finding voids during preliminary field investigation.

3) Changing the layout at the initial site and drilling another test boring at the initial site.

4) After finding voids again following the layout change at the initial site, a geophysical field investigation at three alternative sites (Alternative Sites A, B and D).

5) Establishing the location of the current site, and final field investigation with four test borings at the current site location (Alternative Site B). The site was selected based on the results of the secondary geophysical surveys.

The investigations discussed above consisted of the following:

- A preliminary field investigation including twenty-five (25) test borings drilled by Terracon, Inc. was performed between July 24 and August 8, 2018 at the initial project site and along the gravity sewer alignment. The test boring logs and laboratory data are in the Geotechnical Data Report prepared by Terracon Consultants, Inc. (provided in **Appendix A**);

- A geophysical field investigation including three (3) electrical resistivity tomography (ERT) survey lines was performed by S&ME, Inc. on October 3, 2018. The interpreted ERT profiles are in the Revised Report for Geophysical Services prepared by S&ME, Inc. (provided in **Appendix B**);

- A secondary field investigation including one (1) test boring drilled by Terracon, Inc., with oversight from a CDM Smith representative, was performed on November 20, 2018 at the initial project site. The test boring log is provided in **Appendix C**;

- A secondary geophysical field investigation including nine (9) electrical resistivity tomography (ERT) survey lines was performed by S&ME, Inc. on October January 17, 2019 through January 18. The interpreted ERT profiles are in the Revised Report for Geophysical Services prepared by S&ME, Inc. (provided in Appendix B); and

- A final field investigation including four (4) test borings drilled by S&ME with oversight from a CDM Smith representative was performed between February 25 and March 2, 2019. The test boring logs are provided in Appendix C, and the laboratory data are available in the S&ME Laboratory Report provided in **Appendix D**.

Subsurface information from each investigation was reviewed and utilized to provide information regarding soil, bedrock, and groundwater conditions at the site.

2.3.2 Preliminary Field Investigation

2.3.2.1 Preliminary Geotechnical Investigation

A preliminary geotechnical investigation was performed by Terracon, Inc. between July 24 and August 8, 2018 at the initial project site for the pump station facility and along the proposed gravity sewer alignment. The exploration consisted of twenty-five (25) test borings with depths ranging from 15 feet to 60 ft-bgs using a track or truck-mounted drill rig equipped with an automatic Standard Penetration Test (SPT) hammer system and continuous-flight hollow stem auger drilling techniques. Thirteen (13) of the test borings were drilled at the initial proposed site of the pump station and associated buildings (100-Series), and twelve (12) of the test borings were drilled along the gravity sewer alignment (200-Series). Two (2) test borings (B-215 and B-216) were drilled at the railroad crossing where pipe jacking is anticipated.

Split spoon sampling was conducted at the test borings, and the number of blows required to advance a standard 2-inch outer diameter (OD) split-barrel sampler the last 12-inches of a typical 18-inch penetration with a 140-pound hammer falling 30-inches was recorded to determine the standard penetration resistance value (SPT-N). Auger refusal was encountered at test borings B-101, B-104, and B-108. At these locations, rock coring was performed. Rock cores were generally obtained in 5-foot runs using an NQ2-size wireline diamond-bit core barrel system. The percent recovery and Rock Quality Designation (RQD) were recorded. The RQD is defined as the sum, in inches, of all pieces of sound core, four inches in length or longer, divided by the length in inches of the entire core run, expressed as a percentage. The final boring logs were prepared from field logs and represent interpretations by a geotechnical engineer.

Laboratory testing was performed based upon assignments made by CDM Smith and included: moisture contents (ASTM D2216), Atterberg limits (ASTM D4318), grain size analysis (ASTM D422), one-dimensional consolidation testing (ASTM D2435/D2435M), consolidated-undrained triaxial compression 3-point testing (ASTM D4767), unconfined compressive strength testing of rock (ASTM D7012 – Method C), and flexible wall permeameter hydraulic conductivity testing. A Geotechnical Data Report was provided by Terracon, Inc. and is included in Appendix A.

All test borings were backfilled with grout to the ground surface upon completion.

2.3.2.2 Preliminary Geophysical Field Investigation Results

A large void was observed in test boring B-108 near the Tennessee River between 44.1 ft-bgs and 53.7 ft-bgs. Voids were not encountered in the other test borings around the site, so a geophysical field investigation was conducted to evaluate the extent of the karst feature. The geophysical investigation consisted of three (3) ERT survey lines oriented parallel to the Tennessee River at the initial pump station site.

ERT is an active geophysical technique that introduces a known amount of electrical current into the ground and measures the response to map electrical potentials in the subsurface material. Typically, clayey and moist soils conduct electricity more efficiently than dry sands, gravels, chert, and competent limestone/dolomite, i.e. clayey and moist soils exhibit a lower resistivity. The electrical resistivity also depends on the material within the pore or void space. If a cavity is filled with air, a high resistivity anomaly within the limestone/dolomite layer is expected. If a cavity is filled with water or clay, a low-resistivity anomaly within the limestone/dolomite layer is expected.

The results of the geophysical investigation indicated two (2) low-resistivity anomalies, as indicated in the geophysical report presented in **Appendix B**. The locations of the pump station and associated structures were adjusted to avoid the potential anomalies.

2.3.3 Secondary Field Investigation

2.3.3.1 Secondary Geotechnical Investigation

A secondary field investigation was conducted at the initial pump station facility to investigate the subsurface conditions beneath the relocated building footprints. The secondary field investigation consisted of one (1) test boring location (CDM-204) drilled by Terracon, Inc. on November 20, 2018. CDM-204 was drilled to a depth of 66.3 ft-bgs using an Acker drill rig equipped with an automatic SPT hammer system and continuous flight hollow stem auger drilling techniques.

Split-spoon sampling was conducted continuously from the ground surface to the depth of 15 feet and at 5-foot intervals thereafter to auger refusal. Representative soil samples from the test borings were collected and stored in glass jars for later review and laboratory testing. A CDM Smith representative visually classified the soil samples recovered in the field in general accordance ft-bgs, and rock coring was performed. Rock cores were generally obtained in 5-foot runs using an NQ2-size wireline diamond-bit core barrel system. The recovered rock cores were logged in the field by the CDM Smith representative and were stored in core boxes. The percent recovery and rock quality designation (RQD) were recorded.

The water level in the test boring was measured within the borehole and represents a 24-hour water level reading.

The test boring was backfilled with grout to the ground surface upon completion. The test boring log, prepared by CDM Smith, is included in Appendix C, and the rock core photographs are included in **Appendix E**.

Four (4) test borings were proposed for the secondary field investigation, but a large void from 45.1 feet bgs to 64.4 feet bgs was observed in the first test boring (CDM-204) conducted in this phase. Due to the void observed in the initial field investigation, the anomalies observed in the initial geophysical survey, and the void observed in test boring CDM-204, the secondary field investigation was terminated after completing test boring CDM-204.

2.3.3.2 Secondary Geophysical Field Investigation

A secondary geophysical field investigation was conducted to explore alternate pump station facility sites. The secondary geophysical investigation consisted of nine (9) ERT survey lines distributed throughout three (3) alternative sites: Alternative Site A, Alternative Site B, and Alternative Site D (**Figure 2-1**). Each alternative site had three (3) parallel ERT survey lines distributed throughout the site, as shown in the geophysical data report presented in Appendix B. Please note Alternative Site C was initially considered but was eliminated before the geophysical surveys. Thus, Alternative Site C is not shown on Figure 2-1.

CDM Smith

Figure No. 2-1
Original (Initial) Site Location and Alternative Sites Considered
JULY 2019

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The results of the geophysical investigation indicated one (1) low-resistivity anomaly on the southwest portion of Alternative Site B, two (2) low-resistivity anomalies at Alternative Site D, and three (3) low-resistivity anomalies at Alternative Site A, as indicated in the geophysical report presented in Appendix B. Based on the results of the geophysical investigation, Alternative Site B was selected for further field investigation and potential relocation of the proposed pump station facility.

2.3.4 Final Subsurface Investigation

A final geotechnical field investigation was performed at Alternative Site B by S&ME, Inc. between February 25, 2019 and March 2, 2019. The exploration consisted of four (4) test borings with depths ranging from 54.9 to 65.2 ft-bgs using a truck-mounted CME-550X drill rig equipped with an automatic SPT hammer system and continuous flight hollow stem auger drilling techniques.

Split-spoon sampling was either conducted continuously from the ground surface to the depth of 20 feet and at 5-foot intervals thereafter to auger refusal or at 5-foot intervals from the ground surface to the depth of 20 feet and continuously thereafter to auger refusal. Representative soil samples from the test borings were collected and stored in plastic bags for later review and laboratory testing. A CDM Smith representative visually classified the soil samples recovered in the field in general accordance with the Burmister classification system. In addition to the split-spoon samples, four (4) Shelby tube samples were collected using 3-inch-outer-diameter, 16-gauge wall thickness, 24-inch-long samplers with a sharp cutting edge. Shelby tube samples produce a relatively undisturbed soil sample for laboratory testing.

Auger refusal was encountered in all four (4) test borings at depths ranging from 28.6 to 36.0 ft-bgs, and rock coring was performed. Rock cores were generally obtained in 5-foot runs using an NQ2-size wireline diamond-bit core barrel system. The recovered rock cores were logged in the field by the CDM Smith representative and were stored in core boxes. The percent recovery and rock quality RQD were recorded. Select rock core samples were transported to the S&ME Inc for geotechnical laboratory testing.

Laboratory testing was performed based upon assignments made by CDM Smith and included Atterberg limits, grain size analysis, unconsolidated-undrained triaxial compression testing, unconfined compressive strength testing of rock, and soil corrosivity tests. A geotechnical laboratory testing was provided by S&ME, Inc. and is included in **Appendix D**.

Water levels in the test borings, where recorded, were measured within the boring and represent 24-hour water level readings.

All test borings were backfilled with grout to the ground surface upon completion. The test boring logs, prepared by CDM Smith, are included in Appendix C, and the rock core photographs are included in Appendix E.

2.3.5 Geotechnical Laboratory Testing

Geotechnical laboratory tests were performed on select soil samples and rock cores based on assignments made by CDM Smith. Laboratory testing conducted for the preliminary investigation was performed by Terracon, Inc., and laboratory testing conducted for the final investigation was performed by S&ME, Inc.

The laboratory test program for the preliminary investigation was conducted by Terracon, Inc. and consisted of the following:

- Eighteen (18) grain size analyses performed in accordance with ASTM D422,
- Twenty (20) grain size analyses with hydrometers performed in accordance with ASTM D422 and D1140,
- Thirty (30) Atterberg limits tests performed in accordance with ASTM D4318,
- Seventy-three (73) moisture content analyses performed in accordance with ASTM D2216.
- Twenty-eight (28) USCS classifications made in accordance with ASTM D2187.
- Three (3) unconfined compressive strength (UCS) Tests performed on rock core samples in accordance with ASTM D2166.
- Two (2) one-dimensional consolidation tests performed in accordance with ASTM D2435/D2435M, and
- Three (3) flexible wall permeameter hydraulic conductivity tests performed in accordance with ASTM D5084.

All test results for the preliminary investigation are included in Appendix A. Summaries of the geotechnical laboratory test results for soil and rock are included in **Table 2-1** through **Table 2-4**.

The geotechnical laboratory test program for the final investigation was conducted by S&ME, Inc. This program consisted of the following:

- Five (5) grain size analyses performed in accordance with ASTM D6913.
- Four (4) grain size analyses with hydrometers performed in accordance with ASTM D6913 and D7928
- Eight (8) Atterberg limits tests performed in accordance with ASTM D4318.
- Eighteen (18) Moisture content analyses performed in accordance with ASTM D2216.
- Seven (7) USCS classifications made in accordance with ASTM D2187.
- Five (5) UCS Tests performed on rock core samples in accordance with ASTM D7012 Method C.
- Two (2) Corrosivity suite analyses performed in accordance with AASHTO T 289, ASTM D 512, and AWWA 4500-S D.
- One (1) three-point Unconsolidated Undrained (UU) test performed in accordance with ASTM D2850.

All test results are included in Appendix D. Summaries of the geotechnical laboratory test results for soil and rock are included in **Table 2-1** through **Table 2-4**.

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Table 2-1 Summary of Geotechnical Index Test Results

City of Chattanooga															
Dupont Pump Station and Gravity Sewer															
Chattanooga, TN															
Exploration Number	Sample Number	Sample Depth (ft)	Strata	USCS Classification (1)	Grain Size Analysis (2)							Atterberg Limits(3)			Moisture Content (%) (4)
					Gravel (%)		Sand (%)			Fines (%)		LL (%)	PL (%)	PI (%)	
					Coarse	Fine	Coarse	Medium	Fine	Silt	Clay				
Preliminary Subsurface Investigation - Terracon - 100 Series															
B-101	--	1	Upper Soils	CH	0.0		3.4			45.2	51.4	54	25	29	19.0
B-101	--	3.5	Upper Soils	--	--	--	--	--	--	--		--	--	--	20.0
B-101	--	8.5	Upper Soils	--	--	--	--	--	--	--		--	--	--	23.0
B-101	--	13.5	Upper Soils	--	--	--	--	--	--	--		--	--	--	25.0
B-101	--	23.5	Lower Soils	--	--	--	--	--	--	--		--	--	--	32.0
B-101	--	28.5	Lower Soils	ML	0.0		42.7			37.2	20.1	NV	NP	NP	41.0
B-102	--	20	Upper Soils	--	--	--	--	--	--	--		--	--	--	27.0
B-102	--	25	Upper Soils	CL	0.0		12.7			50.7	36.6	41	21	20	30.0
B-102	--	30	Upper Soils	--	0.1		23.2			48.0	28.7	--	--	--	42.0
B-103	--	2.5	Upper Soils	CH	0.0		3.3			43.2	53.5	52	24	28	20.0
B-103	--	6.5	Upper Soils	CL	0.0		4.2			95.8		47	23	24	24.0
B-103	--	10	Upper Soils	--	--	--	--	--	--	--		--	--	--	25.0
B-103	--	20	Lower Soils	--	--	--	--	--	--	--		--	--	--	28.0
B-103	--	25	Lower Soils	--	--	--	--	--	--	--		--	--	--	29.0
B-103	--	30	Lower Soils	ML	0.2		38.7			40.7	20.3	NV	NP	NP	44.0
B-104	--	2.5	Upper Soils	--	16.4		30.5			53.2		--	--	--	18.0
B-104	--	20	Upper Soils	CL	0.0		28.6			40.5	30.9	32	21	11	28.0
B-104	--	25	Lower Soils	ML	0.0		36.8			42.2	21.1	30	25	5	33.0
B-105	--	1	Upper Soils	--	0.0		13.6			86.4		--	--	--	--
B-105	--	5	Upper Soils	--	--	--	--	--	--	--		45	21	24	17.0
B-105	--	6.5	Upper Soils	--	27.5		29.0			22.0	21.4	--	--	--	26.0
B-105	--	15	Upper Soils	--	--	--	--	--	--	--		--	--	--	25.0
B-105	--	25	Upper Soils	CL	0.0		15.5			49.2	35.3	36	20	16	30.0
B-105	--	30	Upper Soils	--	--	--	--	--	--	--		--	--	--	44.0
B-106	--	2.5	Upper Soils	--	22.2		27.1			50.7		--	--	--	19.0
B-106	--	5	Upper Soils	--	--	--	--	--	--	--		--	--	--	18.0
B-106	--	6.5	Upper Soils	CH	--	--	--	--	--	--		--	--	--	27.0
B-106	--	10	Upper Soils	--	--	--	--	--	--	--		--	--	--	22.0

City of Chattanooga

Dupont Pump Station and Gravity Sewer

Chattanooga, TN

Exploration Number	Sample Number	Sample Depth (ft)	Strata	USCS Classification (1)	Grain Size Analysis (2)										Atterberg Limits(3)			Moisture Content (%) (4)
					Gravel (%)			Sand (%)				Fines (%)			LL (%)	PL (%)	PI (%)	
					Coarse	Fine		Coarse	Medium	Fine		Silt	Clay					
B-106	--	15	Upper Soils	--	--	--		--	--	--		--			--	--	--	23.0
B-106	--	20	Upper Soils	CL	0.0				13.2			46.6	40.2		39	23	16	27.0
B-106	--	25	Lower Soils	--	--	--		--	--	--		--						27.0
B-106	--	30	Lower Soils	SM	35.4				41.2			12.4	10.9		31	29	2	35.0
B-107	--	2.5	Upper Soils	--	--	--		--	--	--		--			--	--	--	16.0
B-107	--	5	Upper Soils	SC	21.7				28.5			17.9	31.9		43	19	24	16.0
B-107	--	10	Upper Soils	CH	8.7				12.3			38.6	40.4		50	24	26	36.0
B-107	--	20	Upper Soils	--	--	--		--	--	--		--			--	--	--	26.0
B-107	--	25	Lower Soils	ML	0.1				29.1			46.6	24.2		30	28	2	35.0
B-107	--	30	Lower Soils	--	8.6				78.5			12.9			--	--	--	15.0
B-108	--	3.5	Fill	--	--	--		--	--	--		--			49	20	29	17.0
B-108	--	6	Upper Soils	CH	--	--		--	--	--		--			--	--	--	27.0
B-108	--	8.5	Upper Soils	CL	0.0				5.5			94.5			48	25	23	35.0
B-108	--	13.5	Upper Soils	--	--	--		--	--	--		--			--	--	--	26.0
B-108	--	18.5	Upper Soils	--	--	--		--	--	--		--			38	21	17	22.0
B-108	--	23.5	Upper Soils	CL	0.1				15.9			49.6	34.4		37	24	13	38.0
B-108	--	28.5	Lower Soils	--	45.5				48.4			6.1			--	--	--	10.0
B-110	--	2.5	Upper Soils	--	--	--		--	--	--		--			--	--	--	15.0
B-110	--	5	Upper Soils	CL	11.7				24.2			27.0	37.2		40	21	19	19.0
B-110	--	6.5	Upper Soils	--	--	--		--	--	--		--			--	--	--	24.0
B-110	--	10	Upper Soils	--	--	--		--	--	--		--			--	--	--	25.0
B-110	--	15	Upper Soils	CL	0.0				14.3			85.7			41	20	21	26.0
B-110	--	20	Upper Soils	--	--	--		--	--	--		--			--	--	--	28.0
B-112	--	2.5	Upper Soils	CL	2.0				8.8			38.0	51.3		44	23	21	23.0
B-112	--	5	Upper Soils	--	--	--		--	--	--		--			--	--	--	24.0
B-112	--	10	Upper Soils	CH	0.0				2.2			97.8			51	25	26	24.0
B-112	--	15	Upper Soils	--	--	--		--	--	--		--			--	--	--	25.0
B-113	--	5	Upper Soils	CH	0.0				2.0			44.3	53.7		50	26	24	23.0
Preliminary Subsurface Investigation - Terracon - 200 Series																		
B-203	--	2.5	Upper Soils	--	--	--		--	--	--		--			--	--	--	24.0

City of Chattanooga																
Dupont Pump Station and Gravity Sewer																
Chattanooga, TN																
Exploration Number	Sample Number	Sample Depth (ft)	Strata	USCS Classification [1]	Grain Size Analysis [2]								Atterberg Limits[3]			Moisture Content (%) [4]
					Gravel (%)		Sand (%)			Fines (%)			LL (%)	PL (%)	PI (%)	
					Coarse	Fine	Coarse	Medium	Fine	Silt	Clay					
B-203	--	5	Upper Soils	--	--	--	--	--	--	--			--	--	--	17.0
B-203	--	7.5	Upper Soils	--	--	--	--	--	--	--			--	--	--	19.0
B-203	--	10	Upper Soils	--	--	--	--	--	--	--			--	--	--	22.0
B-203	--	15	Upper Soils	CL	0.0			10.9		89.1			39	21	18	24.0
B-203	--	20	Upper Soils	--	--	--	--	--	--	--			--	--	--	24.0
B-205	--	20	Upper Soils	CL	0.0			15.8		49.2	35.0		33	22	11	25.0
B-206	--	2.5	Fill	--	11.0			32.9		56.1			--	--	--	9.0
B-206	--	5	Upper Soils	--	--	--	--	--	--	--			--	--	--	20.0
B-206	--	7.5	Upper Soils	CL	11.3			21.9		66.8			32	20	12	21.0
B-206	--	10	Upper Soils	--	--	--	--	--	--	--			36	21	15	23.0
B-206	--	13.5	Upper Soils	--	--	--	--	--	--	--			--	--	--	21.0
B-207	--	15	Upper Soils	--	19.3			40.0		40.7			--	--	--	14.0
B-208	--	5	Fill	--	35.6			38.1		26.3			--	--	--	13.0
B-208	--	6.5	Upper Soils	--	2.9			24.9		72.2			--	--	--	28.0
B-208	--	10	Upper Soils	--	41.5			41.6		16.9			--	--	--	11.0
B-215	--	6.5	Upper Soils	CL	5.2			19.1		75.6			40	22	18	19.0
B-215	--	10	Upper Soils	SC	35.8			43.5		20.7			38	20	18	14.0
Final Subsurface Investigation - CDM Smith - 500 Series																
B-501	S-1	3.5-5	Upper Soils	CH	0.0		0.0	0.2	1.2	48.0	50.6		54	22	32	22.3
B-501	S-3	13.5-15	Upper Soils	CL									43	19	24	19.2
B-501	S-7	26-28	Upper Soils	--	0.0		1.0	2.0	40.0	58.0						
B-502	S-2	8-9.5	Upper Soils	CH	0.0		0.0	0.0	2.0	98.0			51	21	30	21.4
B-502	S-7	25.5-27.5	Upper Soils	--	0.0		0.3	2.4	45.6	29.4	22.2		NP	NP	NP	
B-503	S-2	2-4	Upper Soils	CL	0.0		1.7	1.0	4.3	38.5	54.5		47	21	26	21.2
B-503	ST-2	10-11	Upper Soils	CL	0.0		0.0	0.0	2.0	98.0			48	21	27	21.4
B-504	S-5	8-10	Upper Soils	CH	0.0		0.0	0.0	1.0	98.0			51	21	30	21.4
B-504	S-9	16-18	Upper Soils	CL	0.0		0.0	0.1	3.7	47.7	48.5		45	22	23	22.3

1 USCS classifications were performed in accordance with ASTM D-2487.
2 Grain size analysis tests performed in accordance with ASTM D-422 and ASTM D-1140.
3 Atterberg Limits analysis performed in accordance with ASTM D-4318.
4 Moisture content analysis performed in accordance with ASTM D-2216.

Abbreviations:

CH: Fat Clay
CL: Lean Clay
ML: Lean Silt
NP: Non-Plastic
SC: Clayey Sand
SM: Silty Sand

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Table 2-2 Summary of One-Dimensional (1-D) Consolidation Test Results

City of Chattanooga														
Dupont Pump Station and Gravity Sewer														
Chattanooga, TN														
Exploration Number	Sample Depth (ft)	Sample Elevation [(2)]	Moisture Content (%)		Void Ratio	Dry Density (pcf)	σ'_p	σ'_{vo}	OCR	Cc	Cr	Cv (ft^2/yr)	
			w_o		e_o		(tsf)	(tsf)				Min	Max
B-104	21	631	26.2		0.784	95.9	2.0	1.05	1.90	0.23	0.04	0.076	7.162
B-104	23	629	29.4		0.908	89.3	2	1.07	1.87	0.31	0.04	0.145	3.978

[1] 1-D Consolidation testing conducted in accordance with ASTM D2435.

[2] Elevations are approximate and referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Abbreviations

w_o = initial water content
e_o = initial void ratio
σ'_p = Pre-consolidation Pressure
σ'_{vo} = Estimated Existing Effective Vertical Stress
OCR = Overconsolidation Ratio
Cc = Compression Index
Cr = Recompression Index
Cv=Coefficient of consolidation

Table 2-3 Summary of Triaxial Test Results

City of Chattanooga							
Dupont Pump Station and Gravity Sewer							
Chattanooga, TN							
Exploration Number	Sample Depth (ft)	Sample Elevation [2]	Calculated Void Ratio	Dry Density (pcf)	Strain at Failure (%)	Unconfined Compressive Strength (tsf)	Undrained Shear Strength (tsf)
B-104	8-10	643	0.61	105	15.0	1.8	
B-104	10-12	641	0.91	88	4.6	0.85	
B-104	22-24	629	0.95	87	6.0	1.42	
B-502	19.5-21.5	633	0.74	98.7	--		0.5
B-502	19.5-21.5	633	0.77	97.3	--		
B-502	19.5-21.5	633	0.75	98	--		

[1] B104 samples were tested in accordance with ASTM D2166. B502 samples were tested in accordance with ASTM D2850 (UU Test).

[2] Elevations are approximate and referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Table 2-4 Summary of Rock Core Test Results

City of Chattanooga					
Dupont Pump Station and Gravity Sewer					
Chattanooga, TN					
Exploration Number	Sample Number	Sample Depth (ft)	Wet Density (pcf)	Unconfined Compressive Strength (ksi)	Hydraulic Conductivity (ft/day)
B-101		--	145.0	18.2	--
B-104		--	156.0	18.9	--
B-104		--	160.7	18.1	--
B-101		36.1-41.1	168.1	--	1.83E-05
B-104		28.2-30.0	169.9	--	2.39E-05
B-108		33.6-39.6	168.6	--	6.69E-06
B-501	C-3	36.3-36.6	171.5	35.0	--
B-501	C-5	47.0-47.4	174.9	34.3	--
B-502	C-2	31.9-32.2	166.8	27.9	--
B-502	C-3	38.8-39.2	170.1	28.6	--
B-503	C-1	37.4-37.7	175.2	41.7	--

[1] Hydraulic Conductivity test performed using a flexible wall permeameter, ASTM D5084.

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2.4 Subsurface Conditions

The subsurface conditions encountered during the preliminary, secondary, and final field investigation phases, as interpreted from the test boring logs, are generally consistent with regional geologic data. The subsurface conditions at the proposed pump station facility and along the gravity sewer alignment consist of Surface Material, Miscellaneous Fill, Upper Soil, Lower Soil, and Bedrock. A summary of the subsurface conditions is included in **Table 2-5**.

2.4.1 Surficial Material

Surficial material consisting of topsoil or asphalt and aggregate base course was encountered in every test boring with thicknesses ranging from 0.3 feet to 0.8 feet.

2.4.2 Miscellaneous Fill

Fill was identified at four (4) test boring locations. All locations where Fill was encountered were part of the preliminary subsurface investigation (B-108, B-205 through B-206, and B-208). The Fill layer was encountered beneath surficial materials with thicknesses ranging from 2.7 feet to 5.7 feet. The Fill layer typically consisted of loose to medium dense, light brown and red or dark brown, lean CLAY, some fine to coarse gravel, some rock or chert fragments; or very loose, brown, fine to coarse SAND and fine to coarse GRAVEL, some clay. SPT N-Values range from 1 to 23 blows/foot (bl/ft) with an average value of 7.5 bl/ft at the test boring locations.

2.4.3 Upper Soils

Upper Soils were encountered beneath the surficial material or miscellaneous fill layers at all thirty (30) test boring locations. The upper soil layer consists of Fat Clay (CH), Lean Clay (CL), or Clayey Sand (SC/SC-SM). SPT N-Values in the Upper Soils at the preliminary investigation locations ranged from 0 to 42 bl/ft with an average of 11 bl/ft and at the final investigation locations ranged from 0 to greater than 50 bl/ft with an average of 11 bl/ft at the test boring locations. Clayey sand typically overlies the lean clay, but it sometimes is below the lean clay. As shown in **Table 2-5**, the low-blow count (<2) material can be observed immediately above the limestone. The sub-strata typically consisted of the following:

2.4.3.1 Fat Clay

Fat Clay ranged from 5.5 feet to 21.5 feet thick at the preliminary investigation borings (B-101 through B-103, B-107, B-112 through B-114, and CDM-204) and from 6.0 feet to 17.3 feet thick at the final investigation test borings (B-501 and B-503 through B-504). At the preliminary investigation locations, the Fat Clay typically consisted of medium stiff to stiff, dark brown, yellow and brown, or gray, high plasticity CLAY, trace mica. At the final investigation locations, the Fat Clay typically consisted of moist to wet, very soft to stiff, gray, dark gray, dark brown, or orange-brown, high plasticity CLAY, trace fine to coarse sand, trace mica.

2.4.3.2 Clayey Sand

Clayey Sand ranged from 5.7 feet to 14.5 feet thick at the preliminary investigation test borings (B-105, B-107, B-208, and B-215) and from 4.1 feet to 6.3 feet thick at the final investigation test borings (B-501 through B-502 and B-504). At the preliminary investigation locations, Clayey Sand typically consisted of loose to medium dense, brown or yellow to brown, fine to coarse SAND, some clay, little fine to coarse gravel, trace mica. At the final investigation locations, Clayey Sand typically consisted of wet, very loose to loose, dark gray, fine to coarse SAND, some clay, trace to little wood, trace mica.

2.4.3.3 Lean Clay

Lean Clay ranged from 3.0 feet to 22.5 feet thick at the preliminary investigation test borings (B-102 through B-106, B-108 through B-112, B-201 through B-216, and CDM-204) and from 4.0 feet to 24.5 feet thick at the final investigation test borings (B-501 and B-501 through B-504). At the preliminary investigation locations, Lean Clay typically consisted of very soft to stiff, gray, brown, or dark gray, low plasticity CLAY, “none” to little fine to coarse sand, trace mica. At the final investigation locations, Lean Clay typically consisted of moist to wet, very soft to stiff, brown, gray, tan, or dark gray, low plasticity CLAY, “none” to trace fine to coarse sand, trace mica.

2.4.4 Lower Soils

Lower Soils were encountered beneath Upper Soils at nine (9) test boring locations including seven (7) preliminary investigation locations and two (2) final investigation locations. Where encountered, Lower Soils ranged from 3.0 feet to 14.2 feet thick at the preliminary investigation locations (B-101, B-103 through B-104, B-106 through B-108, and CDM-204) and from 1.4 feet to 6.7 feet thick at the final investigation locations (B-503 through B-504). At the preliminary investigation locations, Lower Soils typically consisted of soft to medium stiff, dark brown, brown, or gray and brown, SILT, some fine to coarse sand, trace mica or loose to dense, dark gray, gray, or brown and gray, fine to coarse SAND, some silt, “none” to little fine to coarse gravel. At the final investigation locations, Lower Soils typically consisted of wet, dense, gray, fine to medium SAND or fine to coarse GRAVEL. SPT N-Values in the Lower Soils at the preliminary investigation locations ranged from 0 to greater than 50 bl/ft with an average of 18 bl/ft and at the final investigation locations ranged from 2 to 31 bl/ft with an average of 19 bl/ft at the test boring locations. As shown in **Table 2-5**, the low-blow count (<2) material can be observed immediately above the limestone.

2.4.5 Bedrock

Bedrock was cored where auger refusal was encountered at eight (8) test boring locations including four (4) preliminary investigation locations (B-101, B-104, B-108, and CDM-204) and four (4) the final investigation locations (B-501 through B-504). Bedrock consisted of regions of Voids and competent Limestone. Voids within the bedrock ranged from 0.1 ft to 15.7 ft thick and were often encountered as water-filled voids at various depths within a borehole. Competent rock encountered at the preliminary investigation locations typically consisted of gray or greenish gray, LIMESTONE, with shale parting and greenish gray dolomitic zones. Rock encountered at the final investigation locations typically consisted of moderately hard to very hard, slightly fractured to sound, fresh to slightly weathered, blue-gray or gray and white, LIMESTONE. Bedrock recovery values in the preliminary investigation locations ranged from 0 to 100 percent with an average of 72 percent, and the RQD values ranged from 0 to 88 percent with an average of 48 percent at the test boring locations. Bedrock recovery values in the final investigation locations ranged from 57 to 100 percent with an average of 93 percent, and the RQD values ranged from 21 to 100 percent with an average of 83 percent.

Table 2-5 Summary of Subsurface Explorations

City of Chattanooga												
Dupont Pump Station and Gravity Sewer												
Chattanooga, TN												
Exploration Number	Approximate Ground Surface El.(1) (ft)	Exploration Depth (ft)	Strata Thickness (ft)								Depth to Groundwater (ft)(2)	Groundwater Elevation (ft)(2)
			Surface	Fill	Upper Soils			Lower Soils	Bedrock			
					(CH)	(CL)	(SC/SC-SM)	(ML/SM/SP/GP)	Voids	Limestone		
Preliminary Subsurface Investigations - 100 Series												
B-101	654.0	51.2	0.5	--	21.5	--	--	14.2 (ML)	--	>15.0	31.0(NE)	623.0
B-102	657.0	30	0.5	--	21.5	>8.0	--	--	--	--	NE	--
B-103	657.0	30	0.5	--	5.5	11.0	--	>13.0 (ML)	--	--	NE	--
B-104	652.0	45	--	--	--	22.0	--	6.2 (ML) (3)	--	>16.8	NR	--
B-105	655.0	30	0.8	--	--	>14.7	14.5	--	--	--	NE	--
B-106	652.0	30	0.8	--	--	19.2	--	>10.0 (SM) (3)	--	--	27.0(NE)	625.0
B-107	652.0	30	0.8	--	15.5	--	5.7	>8.0 (ML) (3)	--	--	27.0(NE)	625.0
B-108	652.0	59.6	0.3	5.7	--	22.0 (3)	--	6.6 (SP)	9.6	>15.4	26.0(NE)	626.0
B-109	660.0	20	0.3	--	--	>19.7	--	--	--	--	NE	--
B-110	635.0	20	0.8	--	--	>19.2	--	--	--	--	NE	--
B-111	655.0	15	0.3	--	--	>14.7	--	--	--	--	NE	--
B-112	654.0	15	0.3	--	>7.0	7.7	--	--	--	--	NE	--
B-113	650.0	15	0.3	--	>14.7	--	--	--	--	--	NE	--
Preliminary Subsurface Investigations - 200 Series												
B-201	656.0	15	0.3	--	--	>14.7	--	--	--	--	NE	--
B-202	657.0	15	0.3	--	--	>14.7	--	--	--	--	NE	--
B-203	661.0	20	0.8	--	--	>19.2	--	--	--	--	NE	--
B-204	661.0	15	0.8	--	--	>14.2	--	--	--	--	NE	--
B-205	662.0	20	0.3	2.7	--	>17.0	--	--	--	--	NE	--
B-206	655.0	15	0.5	2.5	--	>12.0	--	--	--	--	NE	--
B-207	653.0	15	0.6	--	--	>14.4	--	--	--	--	NE	--
B-208	654.0	15	0.6	4.9	--	3.0	>6.5	--	--	--	NE	--
B-209	657.0	16	0.3	--	--	>15.7	--	--	--	--	NE	--
B-210	661.0	20	0.3	--	--	>19.7	--	--	--	--	NE	--
B-215	662.0	15	0.5	--	--	7.5	>7.0	--	--	--	NE	--
B-216	654.0	15	0.5	--	--	>14.5	--	--	--	--	NE	--
Preliminary Subsurface Investigations - CDM 200 Series												
CDM-204	655.5	66.3	0.5	--	9.0	31.0 (3)	--	3.0	18.7	>4.2	24.0	631.5

City of Chattanooga														
Dupont Pump Station and Gravity Sewer														
Chattanooga, TN														
Exploration Number	Approximate Ground Surface El.[1] (ft)	Exploration Depth (ft)	Strata Thickness (ft)										Depth to Groundwater (ft)[2]	Groundwater Elevation (ft)[2]
			Surface	Fill	Upper Soils				Lower Soils		Bedrock			
					(CH)	(CL)	(SC/SC-SM)		(ML/SM/SP/GP)		Voids	Limestone		
Final Subsurface Investigations - 500 Series														
B-501	651.9	65.2	--	--	--	22.5	6.3 [3]		--		1.2	>35.1	0.0	651.9
B-502	653.7	54.9	--	--	--	24.5	4.1 [3]		--		--	>26.3	0.2	653.5
B-503	652.8	60.3	--	--	17.3	12.0	--		6.7		0.3	>24.0	NR	--
B-504	654.6	55.0	--	--	6.0	18.0	5.0		1.4		0.3	>23.7	3.0	651.6

1 Elevations are approximate and referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

2 Groundwater level readings were taken during and upon completion of the test boring. Parenthetical values represent value after drilling if recorded as different than measurement during drilling.

3 A soft layer is present with blow counts less than or equal to 2 immediately above the limestone with occasional presence of stiff sand in between

Abbreviations:

> Indicates strata not fully penetrated

NE indicates not encountered

-- Indicates no value

NR Indicates not recorded

2.4.6 Groundwater Conditions

24-hour groundwater level measurements were recorded where encountered at each test boring location. When encountered at the preliminary investigation locations, groundwater was observed between 26 feet and 31 ft-bgs (approximately El. 623 to 626). When encountered at the final investigation locations, groundwater was observed between 0 feet and 3 feet bgs (approximately El. 651.6 to El. 653.5). Due to the proximity of the Tennessee River to the site, ground water levels will likely correspond to the river stage elevation. Flood conditions were active at the time of drilling for the 500-Series boring locations, which likely influenced the shallow groundwater readings.

2.5 Expected Variations in Subsurface Conditions

The interpretation of general subsurface conditions presented herein is based on soil, rock, and groundwater conditions observed at the test boring locations. However, subsurface conditions may vary between test boring locations. If conditions are found to be different from those described herein, recommendations contained in this report should be re-evaluated by CDM Smith and confirmed in writing.

Water levels measured in the test borings should not necessarily be considered to represent stabilized groundwater levels. In addition, water levels are expected to fluctuate with river level, season, temperature, climate, construction in the area, and other factors. Actual conditions during construction may be different from those observed at the time of the test borings.

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Section 3

Geotechnical Engineering Evaluation and Design Recommendations

3.1 General

Geotechnical engineering evaluations have been made as they relate to the Dupont Pump Station and Basin Improvements – Phase 2 project in Chattanooga, Tennessee. The locations of the structures are as shown on Figure 1-1 noted as current site. In general, these evaluations are based on the results of the subsurface investigations described in Section 2 of this report, published correlations with soil and rock properties and the minimum requirements of the International Building Code 2012 and Tennessee Building Code. In addition, recommended design criteria are based on performance tolerances, such as allowable settlement, as understood to relate to similar structures.

3.2 Geotechnical Considerations

A summary of the primary geotechnical considerations and evaluations related to the design of the proposed pump station, associated structures, and gravity sewer pipeline construction are described in the following sections.

3.2.1 Potential Karst Conditions within Bedrock

The site is considered susceptible to the typical carbonate dissolution hazards of karst topography, including sinkholes and caves. Several small and large voids have been documented in the area, as discussed in Section 2. Two (2) test borings encountered large voids in the bedrock including voids of 9.6 feet in test boring B-108, 15.7 feet in test boring CDM-204, and three test borings encountered minor voids of up to 0.8 feet in test boring B-501, 0.3 feet in test boring B-503, and up to 0.2 feet in test boring B-504. The large voids were encountered at the initial pump station facility site, approximately 447 feet west of the current project site. Much-smaller voids were encountered in the current project site. Pump station and diversion structures have below-grade foundations (approximately 26 feet below proposed grade), so any potential voids may threaten the structural integrity of these buildings. Given this, and the presence of soft soils immediately above the limestone, pump station and diversion structures are recommended to be founded on micropiles.

3.2.2 Site Development

As part of the site development, 4 to 9-feet of fill will be placed and compacted to elevate site grades above the 100-year flood level. Stability of the permanent slope adjacent to the diversion structure and settlement of the structures bearing on shallow foundations due to compression of the native soils under the new fill loads were considered in the design recommendations herein.

3.3 Pump Station Site Design Recommendations

3.3.1 Site Development

The global stability of the permanent embankment adjacent to the diversion structure was assessed for the end-of-construction condition and the 100-year flood stage condition. A river stage of El. 650 feet NAVD88 was used for the end-of-construction condition, and a river stage of El. 659 feet NAVD88 was used for the 100-year flood stage condition. A surcharge of 200 pounds per cubic foot was applied at the top of the slope in both analyses to account for maintenance vehicle traffic and potential equipment staging. An embankment with a 3H:1V slope, if constructed with good construction practices, is anticipated to have a factor of safety of approximately 2.4 at the end-of-construction and approximately 2.0 during a 100-year flood event. The factors of safety exceed the minimum criteria given in USACE EM1110-2-1902.

3.3.2 Pump Station and Diversion Structures

Based on the proposed project site layout, anticipated dimensions, depths and loadings of the proposed structures, subsurface soil conditions, and other design requirements, we recommend that the proposed pump station and diversion structures be supported on deep foundations consisting of micropiles bearing in the bedrock layer.

The micropiles are designed to derive their axial capacity through skin friction within the bedrock layer developed in accordance with procedures outlined in the Federal Highway Administration (FHWA) *Micropile Design and Construction Reference Manual* dated December 2005. The end bearing capacity of the drilled micropiles has not been considered in the socket design. Any skin friction within the Fill, Upper Soils, and Lower Soils layers has been neglected. All micropiles should be installed using a permanent casing above the bedrock layer to prevent loose, collapsible soils and weathered rock from caving in during installation and per Tennessee Building Code requirements.

The drilled micropiles are designed as Type A (gravity-grouted) micropiles with an allowable skin friction value of 21.6 kips per square foot (ksf) in the bedrock layer. For a 200-kip axial design capacity, a 7.5-inch-outside-diameter micropile requires about 9 feet of socket embedment length (i.e., bonded length) within the bedrock, and a 9.75-inch-outsidediameter micropile requires about 7 feet of socket embedment length. However, per Tennessee Building Code, 9.75-inch-oustide diameter is recommended. At least one (1) foot plunge depth into the limestone is required for the casing, where the permanent casing is embedded into the limestone by one foot. This depth should not be considered as part of the embedment length. To account for potential encounter of voids in the limestone, the following provisions should be followed during construction:

1. Less than 6-inch void, micropile bond zone length remains unchanged.
2. 6-inch void to 12-inch void, extend micropile bond zone length one foot.
3. Greater than 12-inch void, restart count of the micropile bond zone length from the bottom of the void.

A factor of safety of 2.0 was used to estimate the allowable axial capacity of the micropiles. The micropile axial capacity should be confirmed by static micropile load tests in accordance with

ASTM D1143 or tensile micropile load tests in accordance with ASTM D3689. A minimum of one micropile load test and one micropile proof test (i.e., micropile load test to 160% of the design load) should be conducted for the pump station and the diversion structure.

3.3.2.1 Micropile Spacing

Center-to-center spacing of the micropiles should be at least 3 micropile diameters to limit group interaction for the axial capacity. If a spacing of less than 3 diameters is used, micropile group effects should be considered for axial capacity.

3.3.2.2 Micropile Cap

Micropile caps that are exposed to freezing temperatures should extend at least 24 inches below any adjacent ground surface.

Micropiles should be embedded into the micropile cap or slab no less than 3 inches. Micropile connections into micropile caps or slab reinforcement shall be designed by the structural engineer in accordance with the Code.

3.3.2.3 Under-Slab Utilities

Under-slab utilities may be hung from the micropile-supported mat or grade beams. Connections should be designed to carry the weight of the soil over the utilities within a zone extending upward at 1H:2V from the springline of the utility. Flexible utility connections and oversized sleeves should be provided through foundation walls and grade beams where utilities transition from micropile-supported within the structure to soil supported outside the structure. These flexible connections and oversized sleeves should be designed to accommodate at least 0.5 inches of differential movement at the transition.

3.3.3 Electrical Building and Generator Structures

The electrical building will be supported on strip footings with a design width of 3 feet 4 inches, and the generator platform will be constructed on a slab-on-grade foundation with a thickened edge. The foundations may be designed for a maximum allowable bearing capacity of 3.2 ksf at the electrical building and 3.0 ksf at the generator building.

3.3.3.1 Foundation Depth

In accordance with the Code, all foundations supported on soil should bear below the frost depth. Unheated areas or areas adjacent to exterior ground surfaces should bear no less than 24 inches below any adjacent ground surface exposed to freezing.

3.3.3.2 Foundation Preparation

Foundation preparation shall consist of 12 inches of compacted structural fill or 12 inches of compacted crushed stone wrapped by non-woven geotextile, placed over fill. For any structure bearing upon structural fill or crushed stone, the extent of structural fill or crushed stone should be at a minimum of 2 feet horizontal distance from the edge of the foundation.

Foundation subgrade should be proof rolled by at least four passes of the appropriate compaction equipment prior to the placement of foundation preparation. If clay materials are encountered at subgrade, the final 6 inches of the excavation should be performed by a smooth-edge bucket.

3.3.3.3 Foundation Bearing Capacity

Based on our evaluation, allowable bearing capacity for the electrical building and generator platform is 3.2 ksf and 3.0 ksf, respectively. The allowable bearing capacities are sufficient to support the design structural pressures of 3.0 ksf and 1.5 ksf for the electrical building and generator platform, respectively.

3.3.3.4 Foundation Settlement

Based on our evaluation, settlement of the electrical building and generator platform, under the anticipated loads and designed as recommended above, are expected to be up to 2.0-inches of total settlement with an approximate differential settlement of 1-inch.

3.3.4 Design Groundwater

For the purpose of design, the groundwater level should be assumed to be at the 100-year flood level, which according to the FEMA Flood Map data is El. 659.

3.3.5 Lateral Loads on Below-Grade Walls

Below-grade portions of structures that are fixed against rotation at the top or will not sufficiently rotate enough should be designed for at-rest pressures from soil and groundwater based on equivalent fluid pressure of 60 pounds per cubic foot (pcf) above the design groundwater level and 90 pcf below the design groundwater level.

In addition to these pressures, a lateral pressure equal to 0.5 times surface vertical surcharge loads from building foundations, slabs, traffic or other loads should be applied over the full height of all walls. To eliminate the surcharge loading from adjacent building foundations on walls, the buildings should be separated such that a line extending at least 2.0 ft beyond the edge of the foundation, then outward and downward at a slope of 1H:1V does not intersect the adjacent structure. Walls to which vehicles can reasonably be expected to approach with in a distance equal to half the wall height should be designed for a minimum temporary uniform vertical surcharge of 300 psf. Earthquake-induced pressures developed in accordance with the Code should be included in the design of all below grade walls.

3.3.6 Resistance to Unbalanced Lateral Loads

Unbalanced lateral loads should be resisted by friction on the bottom of shallow foundations or micropile caps and grade beams. For purpose of design, a coefficient of 0.35 should be considered between the concrete and the underlying structural fill or crushed stone. However, should lateral loads exceed the friction available, the surplus loads may be resisted by passive pressures on the micropile caps and grade beams or mat foundations, provided the structure is appropriately designed for the pressure. Passive resistance up to a maximum equivalent fluid pressure of 150 pcf may be used provided the mat foundations, micropile caps and grade beams are backfilled with structural fill that is compacted to a density of at least 98 percent of the maximum dry density as determined by laboratory test ASTM D698. The resistance from the upper 2 feet of soil should be neglected due to the surface effects and the potential for settlement, disturbance, frost action and other factors. No frictional resistance may be assumed for micropile-supported structures.

3.3.7 Resistance to Buoyancy

Any structures that extend below the design groundwater level should be designed to resist hydrostatic pressures from the design groundwater level referenced above using the dead weight of the structure plus weight of fill placed directly over the structure and extension to the structure foundations. For purposes of design against uplift, the material used as backfill should be assumed to have a total unit weight, in place, of 120 pcf. In addition, for pile-supported structures, a tension capacity of up to 50 percent the design axial compression capacity of the piles may be used for design against uplift. A factor of safety of at least 1.25 should be used to evaluate uplift resistance under normal groundwater and 100-year flood conditions.

3.3.8 Earthquake Considerations

For purposes of determining design earthquake forces for the structures in accordance with the Code, the site should be considered as Site Class "D". Therefore, the spectral accelerations are modified for Site Class D when determining the design earthquake response accelerations and seismic design category for the seismic analysis at the site.

The sandy zone as part of lower soils layer immediately above the limestone bedrock could potentially liquefy under design accelerations. The resulting settlements are approximately 2 inches as obtained following the methodology proposed by Idriss and Boulanger (2008) under free field conditions. However, the pump station and diversion structures are founded on micropiles that are keyed into the bedrock, so these structures would not be subject to liquefaction settlement. The generator slab and electrical buildings will have at least 20 feet of clayey material in between the liquefiable zone and their foundations. This thick non-liquefiable zone is considered sufficient to reduce surface manifestation of liquefaction and reduce the impact on structural integrity based on the recommendations by Ishihara (1985).

3.4 Gravity Sewer Pipeline Recommendations

3.4.1 General

Cut-and-cover techniques are planned for the construction of the gravity sewer pipeline except where the alignment crosses a railroad, as shown in the Contract Drawings. Where the sewer crosses the railroad that cannot be open cut, trenchless construction technique, such as pipe jacking, should be used to mitigate disruption of the rail line.

3.4.2 Pipe Subgrade

The sewer pipeline will be installed by cut-and-cover methods in excavated trenches for most of the alignment. The existing soils, low plasticity clays, encountered along the pipeline are generally suitable for support of the proposed pipe.

If organic, loose, or otherwise unstable soils are encountered at subgrade level, these soils should be excavated to the top of the naturally deposited, suitable inorganic soils and replaced with compacted structural fill. Where compacted structural fill is placed for support of the sewer pipeline, the lateral limits of the fill should be defined as a line extending horizontally outward and downward at a 1H;1V slope from the springline of the pipe to a maximum depth of 4 feet.

3.4.3 Pipe Bedding

The pipe should be placed on a bedding of at least 6 inches of crushed stone, and the stone should wrap the pipe at least up to the elevation of the springline for effective material placement within the haunch area of the pipe. The stone will eliminate pipe contact with plastic clays that may be present in the subgrade at the bottom of the excavated trench.

If crushed stone is placed below the pre-construction groundwater level and over or against soils, a geotextile should be placed between the soils and the crushed stone to protect against the migration of fines into the pipe bedding.

3.4.4 Trench Backfill

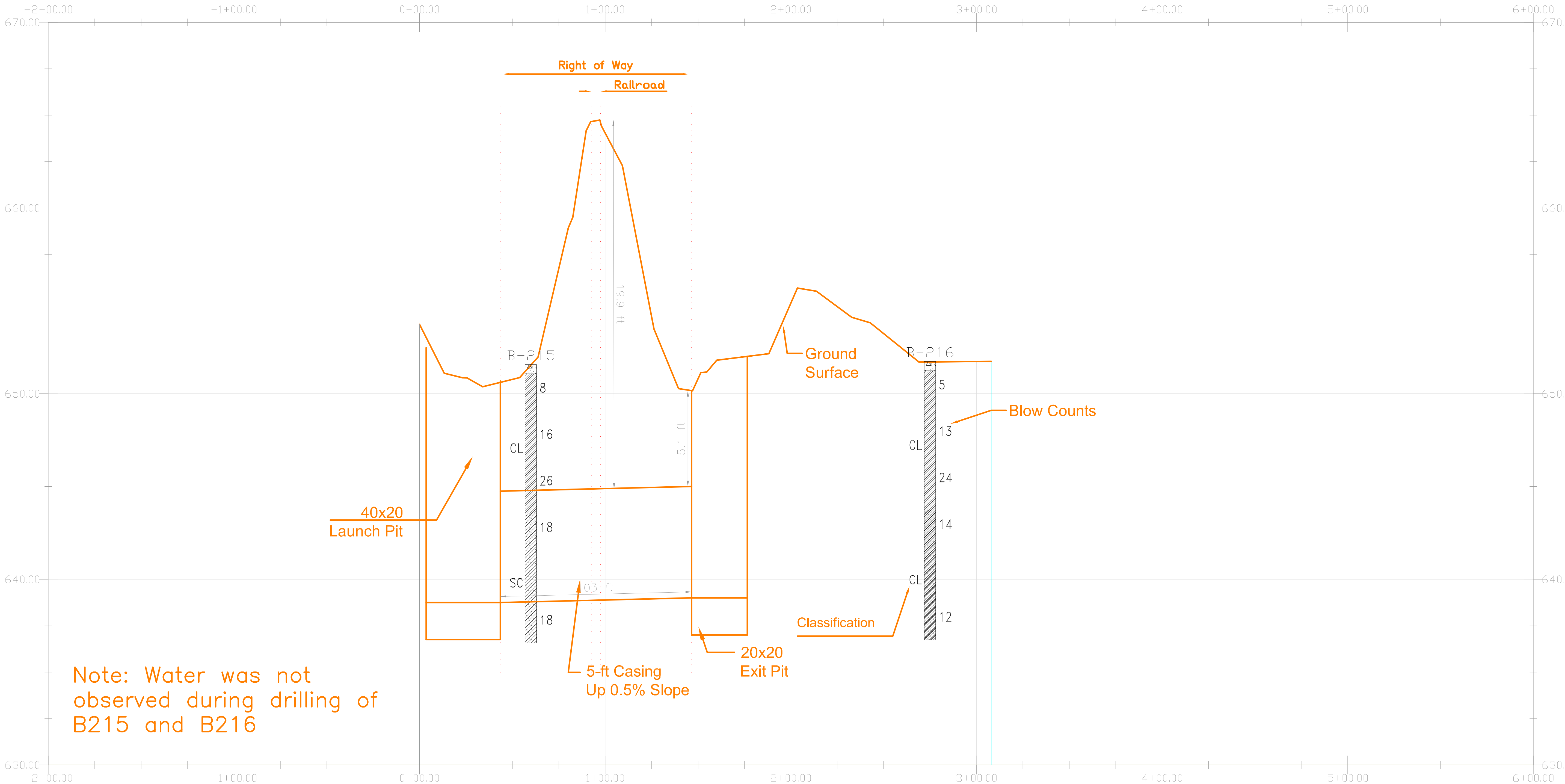
Select common fill should be brought to one foot above the crown of the pipe. Material meeting the criteria for common fill should be used above the select common fill. The remainder of the trench should be backfilled with common fill or select common fill. Refer to **Section 4** for a description of common/select common fill and compaction requirements.

3.5 Trenchless Crossing Recommendations

3.5.1 General

The gravity sewer alignment crosses a rail road as shown on Figure 1-1 and **Figure 3-2**. The railroad crossing will be constructed using trenchless techniques. The length of the railroad crossing is approximately 103 feet, and the depth of cover over the top of the casing is approximately 20 ft.

We recommend pipe jacking with steel casing for construction of the trenchless crossing and installation of the carrier pipe. Pipe jacking should consist of the installation of a minimum 60-inch diameter steel casing for the 48-inch diameter ductile iron pipe (DIP) as shown on Figure 3-1. The invert elevation of the pipeline is proposed to be at approximately El. 644, which provides a minimum soil cover of approximately 5 feet below the existing ground surface near the entry/exit pits at the toe of the railroad embankment. Immediately below the rail road tracks, the thickness of soil cover is approximately 20 feet.



Note: Water was not observed during drilling of B215 and B216

CDM Smith

CITY OF CHATTANOOGA
DUPONT PUMP STATION AND GRAVITY SEWER
CHATTANOOGA, TN

FIGURE 3-1
TRENCHLESS CROSSING
APRIL 2019

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3.5.2 Pipejacking

3.5.2.1 General

Pipejacking consists of pushing a steel casing pipe into the ground using hydraulic jacks at the jacking pit. The material at the heading is excavated from within the steel casing using a continuous flight auger or hand mining. The casing is advanced along with simultaneous excavation of material from the face. This method is considered to be a suitable trenchless construction method for the proposed alignment.

The steel casing pipe will form a temporary liner into which the carrier pipe can be installed and grouted. Use of a casing pipe provides a means to jack through the anticipated earth without damaging the carrier pipe and to allow for proper alignment of the carrier pipe following jacking.

We recommend that pipe jacking be performed on a continuous basis, 24 hours per day, 7 days per week. Pipe jacking methods shall be in accordance with the contract drawings and project specifications. The joints shall be fully closed by welding or mechanical means to ensure tightness.

3.5.2.2 Temporary Ground Support

Temporary ground support of the trenchless crossing should be provided by a steel casing pipe.

Design of the temporary ground support is the responsibility of the Contractor and should be designed by a professional engineer, experienced in pipe jacking and should be registered in the State of Tennessee. The ground support system should be designed to resist the full earth, water, surcharge, and jacking loads acting on it. Surcharge loads from the railroad crossing must be considered. The design should meet the requirements of the contract drawings and project specifications.

Jacking operations should be conducted with an auger that has nearly the same outside diameter of the casing pipe with minimal overcut. Once installed, any voids between the casing pipe and the earth should be grouted using a cement-bentonite grout. Grout should completely fill any voids.

Grouting should be conducted as soon as jacking is completed. Grout pressure should not exceed one-half of the existing overburden pressure. Grout holes must be provided at 4.5-foot maximum intervals placed 120 degrees on center along the entire length of the casing pipe. Grout holes through the casing pipe can be used to insert lubricant which may be required if excessive jacking loads are encountered.

After completion of installation of the carrier pipe, the annulus between the casing pipe and carrier pipe should be filled with a cement grout.

3.5.2.3 Steel Casing Pipe

Based on the anticipated steel casing pipe diameter (60 inches), total crossing length (approximately 103 feet), design surcharge loads, soil overburden, and estimated jacking forces, we anticipate that casing pipe for pipe jacking will have minimum 0.875-inch-thick minimum side walls.

Casing segments, each assumed to be approximately 20 feet long, will be jacked from the entry pit and will need to be welded together or connected using a mechanical connection such as Permalok. The finished casing pipe should be relatively watertight.

3.5.2.4 Ground Conditions and Face Stability

Ground conditions along the trenchless alignment are expected to consist of the Upper Soil materials. These soils are expected to be excavatable in a pipe jacking operation.

Based on the groundwater conditions observed at the time of explorations and during monitoring well readings, groundwater is not expected at the pipeline invert at the trenchless crossing. Should groundwater conditions vary, in order to provide a stable excavation face, groundwater would need to be lowered to below the invert of the tunnel construction.

3.5.2.5 Entry and Exit Pits

A jacking (entry) pit and a receiving (exit) pit will be required at the trenchless crossing. The jacking (entry) pit is expected to be approximately 40 feet by 20 feet in plan area in order to accommodate the anticipated jacking equipment. The receiving (exit) pit is expected to be approximately 20 feet by 20 feet in plan area. All pits should extend to about 2 feet below the proposed pipe invert.

Based on the recommended minimum soil cover of casing pipe and the size of the casing, the depth of the jacking and receiving pits are expected to be about 15 feet below the existing ground surface.

The jacking and receiving pits should have a concrete mat poured at the bottom of the excavation to serve as a working mat. This mat is expected to be about 6 inches thick. The actual thickness of the mat will be determined by the Contractor and will be based on their construction equipment and procedures.

The bottom of the jacking and receiving pits may extend below the groundwater level based on the groundwater condition observed at time of excavation. If groundwater is encountered above the bottom of the pit, dewatering is required to lower the groundwater 2-feet below the bottom of excavation. A drainage layer should be provided under the concrete mat in order to provide a means by which to maintain a dry and stable excavation subgrade. At least 12 inches of compacted, crushed stone should be used as the drainage layer. The stone should be separated from the underlying soils by a geotextile to protect against the migration of fines into the stone.

Requirements for excavation support at the jacking and receiving pits are provided under Construction Considerations. The detailed design and construction of the jacking and receiving pits is the responsibility of the Contractor.

3.5.2.6 Settlements

Ground surface settlement along the tunnel alignment is anticipated to be less than 0.5 inch for the railroad crossing, provided the Contractor conducts all excavation from within the casing, employs proper dewatering/stabilization along the casing, and conducts pipe jacking operations in accordance with the standard of care for that industry.

We recommend that a system of monitoring points be installed along the tunnel alignments to monitor ground deformation.

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Section 4

Construction Considerations

4.1 General

The purpose of this section is to discuss issues related to geotechnical aspects of construction as required for development of the contract drawings and project specifications. Included are anticipated methods of construction required to achieve the recommendations presented herein and identification of potential construction-related problems. The proposed structures and pipeline are near existing facilities, and the impact of construction on those facilities has also been considered herein.

The Contractor will be required to base his/her construction methods and cost estimates on an independent interpretation of the subsurface conditions.

4.2 Excavation and Excavation Support

Excavations for the proposed pipelines are anticipated to generally encounter fill and clay and extend up to 15 feet below existing grade. Undermining of existing foundations must not occur. Excavation should not extend into the zone of influence of any existing structures or utilities without an approved excavation support system. The zone of influence is defined as extending 2 feet beyond the bottom exterior edge of the existing foundation then down and away at a 1 horizontal to 1 vertical (1H:1V) slope or at a 1H:1V slope from the springline of the utility. No excavations are anticipated for the proposed structures as the structures will be constructed within the existing intermediate basins.

The Contractor will be responsible for conducting the excavation work in accordance with the applicable federal and state laws and regulations, including OSHA. Where open excavations are feasible, the side slopes should be designed in accordance with OSHA regulations. The Contractor should be responsible for selection and the design of the means and methods for excavation and excavation support such as open-cut with stable side slopes, trench box, soldier pile and lagging, etc.

Use of excavation support may limit the amount of excavation spoils and serve to protect adjacent structures, utilities and roadways. Selection of the excavation support systems will likely be dependent upon subsurface strata, groundwater conditions, adjacent structures, surcharge loading, etc. Trench box systems should not be permitted within the zone of influence of existing structures, utilities or roadways or jacking or receiving pits. The Contractor should develop an excavation plan, including excavation support systems designed by a Professional Engineer licensed in the State of Tennessee. Additional design considerations may be required based on the Contractor's planned construction methods.

4.3 Dewatering

As necessary, the Contractor will be responsible to design and implement a dewatering and drainage system that maintains a stable, undisturbed subgrade that is free from groundwater and surface water during all construction operations. Dewatering will be needed for the excavation for pump station and diversion structure building foundations. Dewatering may be needed for certain sections of the pipeline trench construction depending on the seasonal fluctuations.

The design of the dewatering system should be performed by a Professional Engineer registered in the State of Tennessee. To avoid disturbance of the subgrade, the water level in all excavations should be maintained at least 2 feet below the subgrade level during the entire period of excavation and fill placement.

Where applicable, the dewatering system should be designed in conjunction with the excavation support system selected by the Contractor. Depending on the depth of excavation and excavation support system selected, wells, well points and/or pumping from open sumps within the excavation may be required. Wells, well points and sumps must be adequately filtered to avoid loss of fines. The site should be graded to direct surface runoff away from the excavations.

The Contractor must be prepared to operate the dewatering system continuously, as required to complete the work and avoid floatation or uplift prior to completion of the facility. During periods where failure of the system would adversely impact work completed, the Contractor should provide a back-up system to ensure continuous operation.

The Contractor must design the dewatering system to not adversely impact adjacent structures or site features. All dewatering, handling and disposal of pumped water and any special testing should be conducted in accordance with local regulations, permits and specified requirements.

4.4 Protection and Preparation of Subgrade Soils

Care should be taken to avoid excess traffic on the excavated subgrade prior to placement of the structural fill, crushed stone and screened gravel or concrete foundations. Final excavation should be made using a smooth-edged bucket where possible. The exposed subgrade should be protected against precipitation, and the subgrade should not be allowed to freeze. Under no circumstances should fill or foundation concrete be placed on a disturbed, wet, or frozen subgrade.

Granular soil subgrades should be proof rolled with a vibratory compactor for at least four passes for the structures and two passes prior to placement of fill or pipeline bedding. Any unsuitable material present at the subgrade level should be removed and replaced with compacted structural fill or crushed stone wrapped in geotextile as recommended herein. A working mat is required below all structures and it shall consist of structural fill (12-inch minimum) or crushed stone (12-inch minimum).

4.5 Protection of Adjacent Structures

4.5.1 General

Excavation for the proposed pipelines and jacking and receiving pits will be made within the zone of influence of existing structures, railroads and utilities. Protection of existing structures, roadways, railroads and utilities is the responsibility of the Contractor. The construction procedures undertaken must be performed in a manner that does not negatively affect the existing facilities.

4.5.2 Deformation Monitoring

We recommend that surface monitoring points (SMPs), deformation monitoring points (DMPs) and crack monitors be established on the existing structures and utilities within 50 feet of the excavations. The points should be monitored during support of excavation installation, trenchless installation, excavation, foundation pier installation, and backfilling work.

DMPs should be installed and formal initial readings taken prior to any support of excavation installation, excavation or dewatering activities within 50 feet of the instrument. Crack monitoring devices should be installed, and formal initial readings taken prior to any excavation, dewatering, or support of excavation installation within 50 feet of the instrument.

Survey of the monitoring points should be performed at a minimum weekly prior to installation of excavation support systems, trenchless installation, excavation, dewatering and/or demolition activities within a 50-foot radius of each instrument. During the active construction operations, the Contractor should monitor all instruments twice per week. The monitoring frequency should increase to daily if threshold values are exceeded. Monitoring should continue bi-weekly after these active construction operations (completion of backfilling and compaction) are completed within a 50-foot radius of each instrument.

The Contractor should be prepared to alter the construction and implement remedial actions if settlement reaches the threshold values. If settlements exceeding the limiting values are measured, the Contractor should suspense all construction operation at the location related to ground deformation, stabilize the excavation and revise the excavation and/or dewatering methods to prevent additional settlement. The threshold and limiting values as follows:

Monitoring Instrument	Threshold Values	Limiting Values
SMP	0.5 inch	1 inch
DMP	0.25 inch	0.5 inch

4.5.3 Vibration Monitoring

Ground vibrations due to demolition activities and excavation support installation can cause damage to adjacent structures, roadways, utilities and other facilities. To avoid or mitigate this potential damage, limits on ground vibrations in the form of ground displacement, velocity or acceleration at given frequencies are typically established. The Bureau of Mines has established criteria to limit ground vibrations using the peak particle velocity (PPV) and frequency parameters. These limits have been established using the cracking of plaster walls in a residential house as a model.

The maximum peak particle velocities associated with demolition and vibratory or impact excavation support installation methods at the ground surface at existing adjacent structures and utilities should be as follows:

Frequency (Hz)	Max. Peak Particle Velocity (in. per sec.)
Over 40	2.0
30 to 40	1.5
20 to 30	1.0
Less than 20	0.5

In no case should the maximum peak particle velocities caused by pile driving exceed 2.0 inches per second at the closest facility (structure or utility) to the work.

A minimum of two seismographs should be located at adjacent/nearby structures and utilities during all demolition and excavation support installation activities to confirm compliance with the recommendations herein and record actual impact vibrations.

In addition, a preconstruction survey should be conducted on structures located within 150 feet of areas of demolition and vibratory or impact excavation support installation. The preconstruction survey should consist of visual inspection and documentation (written, photographic, and/or video) of the existing facility. If damage to adjacent facilities is reported, a similar survey should be conducted at the end of the work and the conditions recorded in the two surveys should be compared for indications of construction-related damage to the existing facilities.

4.6 Backfill

4.6.1 Structural Fill

Granular fill used as structural fill below foundations should consist of a mineral soil free of organic material, loam, debris, frozen soil or other deleterious material which may be compressible, or which cannot be properly compacted. Structural fill should conform to the following gradation requirements:

U.S. Standard Sieve Size	Percent Passing by Weight
1.5 inches	100
No. 4	20-90
No. 40	5-75
No. 200	0-50

Structural fill should have a maximum liquid limit of 50 percent, a maximum plasticity index of 25 percent, and a maximum dry density of at least 95 pounds per cubic foot (pcf) as determined by ASTM D698.

Structural fill should be placed in 8-inch-thick lifts, as placed, and compacted with suitable equipment to at least 98 percent of maximum dry density as determined by ASTM D698. Lift thickness should be reduced to 4 inches in confined areas accessible only to hand-guided compaction equipment. Structural fill should be placed within two percent of its optimum moisture content.

4.6.2 Common Fill

Common fill should consist of soil free of roots, vegetative matter, organic material, topsoil, loam, waste, debris, highly micaceous silt, frozen soil, or other objectionable material. It should not contain stone blocks, broken concrete, masonry rubble, or other similar materials. It should have physical properties such that it can be readily spread and compacted. It should contain stones no larger than six inches, have a maximum of 75 percent passing the No. 200 sieve, a maximum liquid limit of 60 percent, a maximum plasticity index of 30 percent, and exhibit a dry density of at least 90 pcf as determined by ASTM D698. Select common fill should meet the criteria of common fill except it should contain stones no larger than 2 inches.

Common fill and select common fill should be placed in maximum 12-inch-thick lifts, as placed, and compacted with suitable compaction equipment to at least 95 percent of the maximum dry density as determined by ASTM D698. Lift thickness should be reduced to 6 inches in confined areas accessible only to hand-guided compaction equipment. Common fill should be placed within three percent of its optimum moisture content.

4.6.3 Crushed Stone

Crushed stone should consist of hard, durable, angular or subangular particles of proper size and gradation, and should be free of sand, loam, clay, excess fines, and other deleterious materials. The material should conform to the requirements for TDOT No. 57 stone.

Crushed stone should be placed in maximum 6-inch-thick lifts, as placed, and compacted with suitable compaction equipment to at least 98 percent of the maximum dry density as determined by AASHTO T180. Lift thickness should be reduced to 4 inches in confined areas accessible only to hand-guided compaction equipment. Crushed stone should be placed within two percent of its optimum moisture content.

4.6.4 Trench Backfill

Trenches may be backfilled with select fill, common fill, and/or material excavated from the trench provided it meets the criteria of common fill. Criteria on backfill placement in the trench are described in Section 3.

4.7 Geotextile

Except where screened gravel and crushed stone are placed above the design groundwater level and/or against bedrock, a nonwoven geotextile should be used to separate it from the underlying subgrade soils to protect against the migration of fines into the pipeline bedding. The geotextile fabric should be Mirafi 140N or equivalent.

4.8 Micropile Installation

4.8.1 General

A specialty geotechnical contractor (Micropile Contractor) will be required to install the drilled micropiles as recommended herein. The drilled micropile submittal should include the shop drawings showing the drilled micropile layout and a work plan that outlines the proposed installation equipment and proposed drilled micropile materials. The Micropile Contractor should provide equipment capable of constructing micropiles to a depth equal to the deepest anticipated micropile tip elevation plus 30 feet. The Micropile Contractor should provide special drilling equipment including, but not limited to, rock core barrels, rock tools, air tools, and other equipment as necessary to excavate the borehole to the size and depths required. Blasting shall not be used to advance the excavation.

Micropile drilling operations should be performed in a continuous manner using rotary drilling equipment, and drilling methods should employ sufficient fluid pressure to provide complete removal of the drill cuttings from the hole. Permanent steel casing is required to maintain wall stability of the drilled boreholes through the overburden soils and weathered rock fragments/gravel and socketed into 7 feet into bedrock (9.75-inch diameter micropile). Any inflow of groundwater through the pervious soil layers also should be controlled using permanent casing.

Competent bedrock (i.e., continuous and unweathered) should be confirmed by a qualified geotechnical engineer or representative under the direction of the Engineer at the time of construction. After achieving the embedment depth into bedrock, the bottom of the borehole should be cleaned to the extent practical and approved by the Engineer.

Reinforcing bar should be placed into the borehole immediately after grouting and while the grout is still fluid or prior to placing the grout. Reinforcing bar should be set in the borehole with appropriate spacers so the reinforcing will remain in the specified tolerances. Concrete centralizers or other approved non-corrosive centering devices should be used within two feet of the top and bottom of the micropile. Centralizers should also be used at intervals not exceeding ten feet along the length of one micropile.

Concrete should be poured using a tremie pipe starting from the bottom of the hole. Reinforcing bar should extend far enough above the concrete to ensure that a sound connection can be made between reinforcing steel and the structural element it supports. The reinforcing bar should meet the specifications shown on the drawings, and the elevation of the top of the reinforcing should be checked after concrete is placed.

No micropile shall be left partially completed overnight and must be completed, grouted, and protected at the termination of each day’s operation. Micropiles should not be installed within six times the diameter of a newly constructed micropile until the grout of the micropile has set for a minimum of 24 hours.

4.8.2 Obstructions and Differing Bedrock Conditions

Obstructions may be present in the fill and overburden layers at the site. The nature of the obstructions may include, but is not limited to, debris, abandoned foundations, cobbles or boulders. If the obstruction is located within the top 15 feet of the micropile which prevents micropile installation, pre-excavation may be used to remove the obstruction. Micropiles that encounter obstructions that cannot be removed may require that the micropile be relocated. The Contractor should be prepared to address potential difficulties associated with shallow voids in the bedrock or thin pinnacles/ledges of bedrock (over soil) that may be penetrated before obtaining satisfactory bedrock to construct the rock socket.

4.8.3 Micropile Load and Proof Tests

One (1) micropile load test should be conducted in accordance with ASTM D1143 or ASTM D3689 prior to installation of the production micropiles. Three sets of telltales or three pairs of strain gauges should be installed to measure and evaluate the loading/movement transferred to the bearing materials for the load-test micropile. The load test micropile should be cast with a minimum of three (3) ¾-inch diameter PVC Schedule 40 pipes, set to various depths within the micropile to allow for the installation of telltales to be used during the load testing, if that method is selected by the Contractor. The micropiles should not be load tested until the concrete strength has achieved the 28-day compressive strength. The micropiles should be loaded to at least 1.6 times the highest design load. During installation of the production micropiles, the Contractor should perform one proof testing on a micropile selected by the Engineer. Proof testing should not occur until the concrete strength has achieved the 28-day compressive strength. The proof-test micropile should be loaded to at least 160 percent of the design load either in compression or tension.

4.9 Trenchless Construction

The railroad crossing will be installed by pipe jacking as recommended in **Section 3** and specified in the Contract Documents to limit the impact of construction.

Excavation at the face should be conducted within the casing/shield to reduce the potential for disturbance outside the casing. As stated previously, a continuous flight auger or open face shield is expected to be adequate as long as proper dewatering can be employed to maintain groundwater levels at least 1 foot below the casing invert at all times during pipe jacking operations. The Contractor should anticipate the potential for obstructions and/or bedrock within the casing horizon and be equipped to hand-mine and remove such obstructions from the face of the excavation.

4.10 Construction Monitoring

It is recommended that a qualified Geotechnical Engineer or experienced technician under the direction of the Geotechnical Engineer be present during construction to confirm that the Contractor complies with the intent of these recommendations. Specifically, the field representative would undertake the following responsibilities:

- Observe the installation of the geotechnical instrumentation and review site monitoring data collected;

- Monitor the excavation and installation and performance of excavation support systems and observe for potential karstic activity or deformations;
- Confirm that appropriate dewatering and surface water control methods are employed;
- Confirm the removal of unsuitable materials present at foundation subgrade level and replacement with proper backfill material;
- Confirm that the subgrades are prepared, and conditions encountered are suitable for support of the proposed structures;
- Monitor drilled micropile load and proof test(s) and production drilled micropile installation;
- Observe, test and document placement and compaction of backfill material, where appropriate; and
- Monitor the pipe jacking operations including ground conditions encountered, face stability, excavation methods and rates and grouting operations.

In addition, the field representative would be present to identify and provide response should conditions encountered differ from those assumed during preparation of this report.

4.11 Closing

These recommendations have been prepared for the City of Chattanooga Dupont Pump Station and Gravity Sewer Line project located in Chattanooga, Tennessee as understood at this time and described in this report. These recommendations have been prepared in accordance with generally accepted engineering practices. No other warranty, express or implied, is made. In the event that changes in the design or location of the alignment occur, the conclusions and recommendations contained herein should not be considered valid unless verified in writing by CDM Smith.

Section 5
References

1. Ishihara, K. (1985) "Stability of natural deposits during earthquakes" Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering. Vol. I, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

2. Thomson, J. (1993) “Pipejacking and Microtunneling” Springer Science + Business Media Dordrecht, 1993

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