NBC Phase III CSO Program
OF-210/213/214
Facilities
Contract IIIA-4

Geotechnical Design Summary Report

Report Status (60% Design)
Revision No. 1

McMILLEN JACOBS ASSOCIATES

July 28, 2021

This page intentionally left blank.

Table of Contents

List of Tables

List of Figures

Appendices

Distribution

To:	Christopher Cronin, PE BETA Group, Inc
From:	Tennyson M. Muindi, PE McMillen Jacobs Associates
Prepared By:	Katherine R. O’Hara McMillen Jacobs Associates
Reviewed By:	Tennyson M. Muindi, PE McMillen Jacobs Associates

Revision Log

Revision No.	Date	Revision Description
0	December 7, 2020	Internal Draft – Design Team
1	July 28, 2021	60% Design Submittal

1.0 Introduction

1.1 General

This Geotechnical Design Summary Report (GDSR) presents the results of engineering evaluations of the subsurface information collected for design of the consolidation conduit and associated near surface structures for the Narraganset Bay Commission (NBC) Consolidation Conduit Contract IIIA-4 (the Project). This GDSR presents interpretations and conclusions with respect to generalized soil, rock, and groundwater conditions, significant engineering properties of soil and rock; and geotechnical design and construction considerations for the proposed facilities.

The Project is one component of the NBC Phase III Combined Sewer Overflow (CSO) Program. The Phase III CSO Program is being managed by Stantec, along with Pare Corporation (PM/CM) as a subconsultant under contract with NBC. BETA Group, Inc. (BETA) is the Project Design Consultant. McMillen Jacobs Associates (McMillen Jacobs) is providing geotechnical and structural engineering services under contract with BETA.

1.2 Purpose and Scope

The purpose of this report is to present the following:

- Interpretation of geotechnical data collected to date, and characterization of the subsurface soil, rock, and groundwater conditions along the consolidation conduit alignment as related to the proposed construction;
- Evaluations of the significant engineering properties of the subsurface materials with respect to the proposed structures;
- Geotechnical related design and construction considerations for the Project;

The data that provided the basis for these interpretations and considerations are described in Section 2.

1.3 Project Description

This Project is part of the NBC Phase III CSO Program located in Rhode Island, which began in 2016 and is focused primarily on the Bucklin Point Service Area (BPSA) in the communities of Pawtucket (the City) and Central Falls. The overall NBC CSO Program is aimed at lowering annual CSO volumes and reducing annual shellfish bed closures in accordance with a 1992 Consent Agreement with the Rhode Island Department of Environmental Management (RIDEM). Phases I and II of this program, which focused on the Field’s Point Service Area (FPSA) in Providence, were completed in 2008 and 2015, respectively.

This Project includes the design and construction of a consolidation conduit, a junction chamber, a gate and screening structure, an approach channel, a diversion structure, manholes, and other ancillary facilities necessary to convey flow from outfall structures OF-210/211, OF-213, and OF-214 to the future Pawtucket Tunnel via Drop Shaft 213 and connecting adit set to be constructed under separate contract. Refer to Figure 1 for a Project Locus and Figures 2 through 5 for locations of the various project elements.

The selection of the horizontal and vertical alignment for the Project was developed jointly by the BETA design team and the PM/CM. Several alignment alternatives were evaluated due to the changed conditions requested by the PM/CM and NBC with the preferred alignment being as indicated herein. A partial list of evaluation criteria used in selecting the preferred alignment included:

- Maintenance of hydraulically acceptable gradients on the vertical consolidation conduit alignment and keep the near surface structures depths as shallow as practical.
- Locating the consolidation conduit alignment and near surface structures in areas that reduce the potential for impact to adjacent structures and utilities.
- Locate the gate and screening structure and approach channel with sufficient work space and for the convenient future location of DS-213 for connection to the Pawtucket Tunnel.
- Location of the structures in public right-of ways and easements where possible.

All elevations indicated in this GDSR are referenced to National Geodetic Vertical Datum (NVGD) 1929, in feet. Horizontal control for the project is based on Rhode Island Mainland State Plane, North American Datum (NAD) 1983. All stationing referred to in this GDSR is referenced as shown on the 60 percent Design Drawings, and Figures 2 through 4 herein.

1.4 Limitations

This GDSR has been prepared for specific application to the proposed facilities for the NBC Phase III Consolidation Conduit Contract IIIA-4 project in Pawtucket, Rhode Island, configured as referenced herein, in accordance with generally accepted geotechnical engineering practice. In the event that changes in the nature, design, or location of the project elements are made, the interpretations, conclusions, and recommendations presented herein should not be considered valid unless the changes are reviewed and the conclusions of this GDSR are modified or verified in writing by McMillen Jacobs.

The analysis and recommendations presented in this GDSR are based in part on the data obtained from the referenced explorations. The nature and extent of variations between explorations may not become evident until or during final design or construction. If variations then appear, the conclusions of this GDSR should not be considered valid unless the conclusions are re-evaluated and modified or verified in writing by McMillen Jacobs.

2.0 Field and Laboratory Investigations

2.1 General

This section presents a high level summary of the data that provided the basis for the interpretations and recommendations presented in this GDSR. It is beyond the scope of this GDSR to provide a detailed summary of all the field and laboratory investigations that have been conducted for the Project. Such information can be found in various other project reports as referenced herein.

2.2 Final Design Phase

A design phase field and laboratory investigation program was undertaken for the Project in 2019 and 2020 by McMillen Jacobs. The program consisted of drilling 13 widely spaced test borings, five of which are located along this Project alignment. In addition, groundwater monitoring wells were installed, and soil laboratory testing was conducted. The results of the investigation program are presented in the report entitled “Geotechnical Data Report, NBC Phase III CSO Program, Consolidation Conduits IIIA-4 and IIIA-5”, dated June 2020, hereafter referred to as the GDR. This GDSR was prepared utilizing data from that report.

2.3 Planning and Design Phase

A planning and design phase for the overall NBC CSO Phase III Program was performed by the PM/CM. The following information was provided and reviewed:

- Report entitled “NBC CSO Phase IIIA Contract No. 308.01 C, Request for Proposal, Appendix C, Geotechnical Data Report”, prepared by and Stantec and Pare Corporation., dated December 23, 2019.
- Supplemental B21-series test borings at the Drop Shaft 213 area (Masonic Temple) performed by the PM/CM and provided to McMillen Jacobs by BETA via email on April 16, 2021.

The borings that were judged to be representative of conditions along the Project alignment included test borings B17-8, B17-39, B17-40, B17-41, B17-45, B17-46/46A, B21-1, B21-2, and B21-3. Data from these test borings were utilized in the evaluations made in this GDSR.

2.4 Historic Borings

Geotechnical subsurface explorations have been performed along or adjacent to the Project alignment for various purposes. The following information was reviewed:

- Drawing set entitled “Plan, Profile and Sections of Proposed State Highway, Division St. Project, Contract Three” prepared by State of Rhode Island, Department of Public Works, Division of Roads and Bridges, dated 1957.
- Drawing set entitled “State of Rhode Island, Department of Transportation, Plans, Profiles, and Section of Proposed Bridge Replacement, Pawtucket Bridge No. 550, I-95 Over the Seekonk River, Volume 3 Bridge Plans” prepared by Commonwealth Engineers & Consultants Inc., dated April 2010.

The borings that were judged to be representative of conditions along the Project alignment included test borings BH-10, BH-11, BH-12, BH-13, BH-14, BH-32/32A, BH-33, BH-34, BH-35, BH-36, BH-37, and BH-102. Data from these test borings were used in preparation of this GDSR.

3.0 Site Geology

3.1 General

This section describes the general geology, site topography and land use in the vicinity of the proposed Project. Technical terminology used in this section and throughout this GDSR to describe soil units, rock types, and geologic features are defined in the Glossary of Technical Terminology provided in Appendix A. The Subsurface Investigation Key used for McMillen Jacobs test borings is provided in Appendix B.

3.2 Geologic Setting

The Project is in the New England physiographic province of the Appalachian Highland physiographic division, lying within the Seaboard lowland section (Denny 1982). The physiographic area is referred to as the Narragansett Basin, the result of a complex sequence of a geosynclinal sedimentation, volcanism, plutonism, and erosion (Quinn 1971). The basin is made up of several thousand feet of non-marine sedimentary rock that has been folded, faulted, and slightly to moderately metamorphosed.

The geologic history of the proposed project area is one of weathering, erosion, and deposition. Periods of glaciation have shaped much of the visible landscape and the Project area is characterized by the adjacent Blackstone/Seekonk River valley. Glacial and post glacial deposits dominate the landscape and generally consist of stratified layers of sand, silt, gravel, cobbles, and boulders.

The soil stratigraphy in the overall area from the ground surface downward is anticipated to consist of Fill material, Glaciofluvial deposits, and Glacial Till deposits overlying the bedrock. The bedrock is the Rhode Island Formation, a Pennsylvania-age sandstone with lessor amounts of conglomerate sandstone and siltstone. Quartz filled fractures are common. Evidence of faulting in the wider area is present but not expected along this Project alignment.

3.3 Site Topography and Land Use

The topography in the area is the result of a long and complex history of glaciation and site filling, which has had an influence on the current site and subsurface conditions. The topography is generally rolling to flat, with less than 200 feet of relief, sloping downwards towards the east. The ground surface along the Project alignment varies from approximately elevation El. 32 to El. 12, sloping gradually downward towards the south. The bedrock surface topography is irregular and is expected to range from about 10 feet to 30 feet below existing grade with the highest rock in the area beneath the Interstate (I-95) Highway.

Land use along the Project alignment appears to be generally commercial. It is anticipated that the structures along the Project alignment are founded on spread footings bearing on the Glacial deposits.

4.0 Engineering Properties of Soil and Rock

4.1 General

This section presents a characterization of the subsurface conditions encountered within the immediate vicinity along the Project alignment, including a brief description of these deposits and selected engineering properties.

4.2 Soil

The soil stratigraphy from the existing ground surface downward includes the following:

- Unit 1 Fill
- Unit 3 Glaciofluvial
- Unit 4 Glacial Till

All geologic units may not be encountered at all locations along the Project alignment.

Histograms of Standard Penetration Test (SPT) N-values and composite grain size distribution curves for each geologic unit are provided in Appendix C.

4.2.1 Unit 1 – Fill

Fill material is present surficially along the Project alignment from the existing ground surface to depths ranging from about 5 to 20 feet. The Fill consists of variable composition, uncontrolled man-made materials, and other construction debris.

Based on the test borings, the Fill encountered consists of loose to very dense, brown to black, coarse to fine sand with varying amounts of silt and gravel and contains fragments of glass, brick and concrete. SPT N-values ranged from about 3 to greater than 50 blows per foot (bpf), with values typically greater than 10 bpf.

Obstructions in the form of concrete were encountered in test borings B-4 and B-6 on Taft Street and Roosevelt Avenue Ext. at depths ranging from about 4 to 10 feet below the existing ground surface that necessitated resetting of these test borings multiple times. Based on review of readily available historic documents, remnant foundations from previous structures may be present in this area.

Upon conduct of test borings B-4 and B-6, vacuum excavation was performed in the Upper 6 feet of test borings dilled along this alignment. Therefore, the nature and quality of the Fill may not be fully described herein.

4.2.2 Unit 2 – Glaciofluvial

Glaciofluvial deposits were encountered in some test borings at the ground surface or below the Fill and above the Glacial Till or bedrock and ranged in thickness from about 5 to 10 feet. These Glaciofluvial sediments were produced by water flowing on, in, or under a retreating glacier and are characterized by sorted and stratified sand and gravel. The sediments are the result of erosion and have formed terraces from being trapped between the glacier ice and the sides of the river valley walls. Occurrences of cobbles and boulders are also common.

Based on the test borings performed, the Glaciofluvial deposits encountered generally consist of loose to medium dense, brown, coarse to fine sand or coarse to fine gravel. SPT N-values ranged from about 7 to greater than 50 bpf.

4.2.3 Unit 3 – Glacial Till

Glacial Till deposits were encountered beneath the Fill or Glaciofluvial deposits directly over the bedrock The Glacial Till is variable in nature due to the complex process of deposition beneath the moving glacial ice (lodgment till).

Based on the test borings, the Glacial Till generally consists of an unsorted mix of very dense, sand and gravel with lessor amounts of silt and clay and includes occurrences of cobbles, boulders, and rock fragments from the underlying bedrock. SPT N-values ranged from 10 to greater than 50 bpf, with values typically greater than 30 bpf.

4.2.4 Engineering Properties for Soil

Table 1 summarizes the engineering properties for each soil unit encountered. The location of the contacts between the overburden soils within borings along the Project alignment are shown on Figures 2 through 5. No attempt has been made to interpolate the location of the contacts between the borings.

Table 1 Summary of Engineering Properties for Soil

Property/Parameter	Fill	Glaciofluvial	Glacial Till
Saturated Unit Weight (pcf)	125	125	135
Buoyant Unit Weight (pcf)	63	63	73
Friction Angle (Φ)	32	32	34
At-Rest Earth Pressure Coefficient (K_o)	0.47	0.47	0.44
Active Earth Pressure Coefficient (Ka)	0.31	0.31	0.28
Passive Earth Pressure Coefficient (Kp)	3.26	3.26	3.54

4.3 Bedrock

The bedrock consists of the Rhode Island Formation of the Narragansett Bay Group. The Rhode Island Formation consists of predominantly sandstone with lessor amounts of conglomerate sandstone and siltstone.

The top of bedrock along the Project alignment is anticipated to vary as shown in the individual boring sticks on Figures 2 through 5. As indicated, the top of bedrock represents the surface of material that has been geologically classified as bedrock on the test boring logs, regardless of the quality the bedrock. The estimate top of bedrock line shown on Figure 3 and Figure 5 is based on a top of bedrock developed from the geophysical evaluation conducted by the PM/CM as part of the conceptual design of the Phase III Project. Typically, there is some variation from the top of bedrock estimated from geophysical methods to the top of bedrock determined from conducting test borings.

The bedrock is generally described on the boring logs as strong, slightly weathered to fresh, fine-grained siltstone and sandstone with occasional occurrences of conglomerate interlayers. A layer described as weathered bedrock ranging in thickness from about 2 to 3 feet was noted on tests boring logs for B17-40, B21-1, B21-2, and B21-3.

In the bedrock cored, bedding plane joints dipping 20 to 50 degrees were observed frequently and were described as smooth to slightly rough and slightly weathered to fresh.

About 75 percent of all the bedrock cored had an RQD of greater than 50 percent, indicating a fair to excellent quality rock as defined by Deere (1988). Refer to Appendix C for a RQD histogram.

5.0 Groundwater Conditions

5.1 General

Regional groundwater flow in the vicinity of the Project is anticipated to be north-south, along the Blackstone River/Seekonk River. The Blackstone River flows south from Worcester, Massachusetts to the Main Street Dam in Pawtucket, RI. At this point, it becomes the headwater for the Seekonk River, which is a tidal estuary that flows south before combining with the Providence River. The Blackstone River is the second largest source of freshwater to Narragansett Bay. The average annual precipitation is approximately 47 inches per year (Frimpter and Maevsky 1979).

The principal aquifer in the Project area is in the Glaciofluvial Deposits, comprised mainly of sand and gravel. These materials were deposited in the stream valley, are irregularly shaped, and form an unconfined aquifer. The Glaciofluvial aquifer is separated from the bedrock aquifer by the Glacial Till. The Glacial Till layer is of variable thickness that does not yield significant quantities of water. Specific yield of the Glaciofluvial aquifer is estimated to be approximately 20 percent (Frimpter and Maevsky 1979).

The bedrock aquifer is partially confined. Specific yield is greatest near top of rock and decreases to a negligible amount at depths over 300 feet. Specific yield of the upper 300 feet of bedrock is estimated to be less than 0.5 percent (Frimpter and Maevsky 1979).

Municipal and industrial water wells in the Project area are generally located at depths ranging from about 70 to 550 feet in bedrock. Yield from water wells located in bedrock are estimated to range from about 5 to 200 gallons per minute (gpm), with a median yield of about 30 gpm (Halberg, Knox, and Pauszek 1961).

5.2 Groundwater Levels

Based on the groundwater monitoring performed for the Project from September 2019 to February 2020, measured maximum groundwater levels along the Project alignment ranged in elevation from approximately El. 17 to El. 1.5, corresponding to a depth below the existing ground surface of about 8 to 15 feet. Groundwater levels shown on Figures 2 through 5 indicate the groundwater level recorded at individual test boring locations at the date indicated. Groundwater levels shown at test borings drilled as part of the planning and design phase for the Project were recorded during drilling within the open borehole. Groundwater monitoring within the recently installed groundwater monitoring wells has not been conducted for a sufficient duration to provide seasonal water level fluctuations at these locations.

Groundwater levels in the Project area are expected to be influenced by the Blackstone River/Seekonk River, which varies due to the presence of three historic dams. The Seekonk River water level average is anticipated at approximately El. 2. The Seekonk River is tidal, with an anticipated normal daily tidal range of about 2.3 feet above and below the average.

Groundwater levels are also expected to vary seasonally, as a result of changing weather patterns and precipitation. In addition, groundwater levels in the Project area may be influenced by local construction activities and drainage into and out of existing underground utilities.

5.3 Hydraulic Conductivity of Rock

The hydraulic conductivity of the bedrock aquifer is mainly due to the fractures present within the rock mass. Based on the results of the hydraulic conductivity testing (packer testing) in bedrock completed for the Pawtucket Tunnel Project, estimated hydraulic conductivities at the interface between the soil and bedrock are on the order of 1×10^{-4} to 6×10^{-3} cm/sec. In addition, the data suggests that the hydraulic conductivities estimated in testing performed in boreholes drilled near the river were generally higher than testing performed in boreholes located some distance away from the river.

5.4 Groundwater Impacts

Detrimental impacts to the ground and existing structures from construction related groundwater management is not anticipated. Impacts from groundwater drawdown generally occur due to long-term groundwater lowering away from excavations. The most significant of such impacts typically include ground and existing structure settlements as a result of consolidation of compressible soil deposits. The soil deposits anticipated to be encountered during construction of the Project elements include Fill materials underlain by dense to very dense Glacial deposits followed by bedrock. These deposits are generally granular and are not anticipated to consolidate or respond significantly due to groundwater lowering.

Construction considerations relative to groundwater control are discussed in Section 8.

6.0 Geotechnical Design Considerations for Shafts and Permanent Structures

6.1 General

The primary purpose of this section is to discuss how geotechnical issues may affect design of the shafts and permanent structures. The term shafts used herein refers to cut and cover excavations that will be used to conduct trenchless and open cut installation operations, and construct the permanent structures. The locations of the structures are shown on Figures 2 through 5. Approximate location, dimensions, and depth are described below.

The gate and screening structure (GSS) is located at 50 Taft Street within the former Masonic Temple property immediately adjacent to proposed Drop Shaft 213. The GSS serves to reduce the volume of debris and floatables from the combined sewerage prior to discharge to the tunnel. The GSS is about 26 feet by 10 feet and is about 32 feet deep. The GSS includes two additional structures designated GSS-1 and GSS-2. GSS-1 is a 4 foot diameter shaft and is about 22 feet deep located along the approach channel. GSS-2 is a 12-foot diameter shaft and is about 32 feet deep located between the GSS and the 72-inch pipe that connects to the Junction Chamber.

The approach channel will convey flow from the GSS to the future Drop Shaft 213. The approach channel is about 6 feet by 6 feet and is about 30 feet deep.

The Junction Chamber (JC) is located in the northbound travel lane of Taft Street and will receive flow from the consolidation conduits to the north and south and convey that flow to the GSS. The JC is about 10 feet by 12 feet and is about 30 feet deep.

OF-214 Diversion Structure (DS) is located at the intersection of Taft Street, Jenk’s Way, and Roosevelt Ave Extension. The OF-214 DS serves to divert flow from OF-214 and the consolidation conduits from the north to the tunnel via the downstream facilities. The OF-214 DS is about 11 feet by 14 feet and is about 27 feet deep.

OF-213 DS is located east of Roosevelt Avenue Extension immediately south of the parking lot entrance for the Pawtucket Substation No. 2. The OF-213 DS serves to divert flow from OF-213 and the consolidation conduits from the north to the tunnel via the downstream facilities. The OF-213 DS is about 8 feet by 18 feet and is about 20 feet deep.

OF-210 DS is located within the westbound travel lane of Main Street. The OF-210 DS serves to divert flow from OF-210 and the consolidation conduits from the north to the tunnel via the downstream facilities. The OF-210 DS is a 10 foot diameter manhole and is about 16 feet deep. MH210-1 is located to the west of OF-210 DS and is an 8 foot diameter manhole about 12 feet in deep.

Three manholes are located along the consolidation conduit to the north of OF-213 DS and are designated MH213-3, MH213-2, and MH213-1. Three additional manholes are located along the consolidation conduit to the south of the JC and are designated MH217-1, MH217-2, MH217-3. The manholes are approximately 8-foot diameter precast reinforced concrete structures with installation depths slightly below the consolidation conduit at each discrete manhole location.

6.2 Excavation and Initial Support

6.2.1 Excavation

Excavations ranging from about 15 to 40 feet will be required to construct the permanent structures. Refer to Figures 2 through 5 and the individual boring sticks, which indicate the soil units anticipated to be encountered at each structure location. Excavation of bedrock will be required at the GSS(s) and may be encountered at the invert excavation depths at OF-214 DS, MH213-1, MH217-1, and MH217-2. In addition, boulders were encountered in test boring B21-3 located adjacent to GSS-2 from approximately El. 9 to El. 4. Descriptions and engineering properties of the soil units and bedrock are provided above in Section 4. Boulders or weathered bedrock should be expected at the top of rock surface.

Excavation of bedrock will likely be accomplished using mechanical excavation equipment, such as an excavator or a clam shell bucket. Due to the limited amount of bedrock to be removed and anticipated quality of rock at the top of rock surface, its anticipated that rock removal can be accomplished by mechanical means using hydraulic hammers and rippers.

6.2.2 Initial Support

Excavation support will be designed by the Contractor subject to the performance criteria included in the contract documents. At the GSS(s) and the JC, rigid, impermeable temporary support will be required. Likely feasible systems include secant pile walls or slurry diaphragm walls drilled-into rock with internal bracing consisting of steel struts and walers. Where circular shafts are used, lateral load are resisted through ring compression and may be designed as unreinforced without any internal bracing. Maximum installation tolerances of 1 inch within in-plan design location and 1 percent out-of-verticality are recommended to ensure contact between adjacent piles for water cut-off.

At the remaining shaft locations, flexible, impermeable temporary support may be used. Likely feasible methods include, but are not limited to, steel soldier piles and timber lagging with internal bracing consisting of steel struts and walers. Bracing levels should be kept to a minimum to maximize efficiency when constructing the structures. The Contractors design will need to consider installation tolerances to ensure build-out of the permanent structures can be accomplished.

For planning purposes, where excavations penetrate bedrock, it would be reasonable to assume the use of rock dowels and mesh/shotcrete for temporary rock support.

6.3 Permanent Structure Design

6.3.1 Uplift

It is anticipated that the structures will be empty after overflow storm events and that maximum seasonal groundwater levels could occur when empty. It is noted that during storm events the structures will be filled to varying levels. The structures should therefore be designed for buoyant uplift conditions reflecting the highest seasonal groundwater level that could occur under non-flood conditions.

For design purposes, it is recommended the uplift pressure for each structure be calculated as a function of the footprint area and depth below groundwater table. Specifically, the uplift pressure can be calculated as the product of the invert depth below the groundwater table, the unit weight of water, and the surface area of the structure footprint.

For design purposes, the design water table depth should be assumed to be at the existing ground surface.

The methods used to resist uplift pressures generally includes dead weight or post-tensioned tie-down anchors extending into bedrock. Post-tensioned tie-down rock anchors are not recommended given the relatively small footprint areas of the proposed structures and the likely presence of cobbles and boulders within the Glacial deposits. Given these anticipated difficulties, it is recommended that the uplift forces be resisted by dead weight, if practical.

Dead weight would generally include the total weight of the structure. In addition, the side frictional force of the soil against the walls, or soil against soil if a "lip" is formed, could be used. Additional dead weight can be engaged by extending the bottom of the foundation beyond the outside wall face forming a "lip" such that there is a column of soil directly on the "lip". The thickness of the walls and slab can also be increased to add more weight, but for every foot deeper that the foundation bottom is extended, the water pressure will increase by an amount equal to the unit weight of water.

A recommended friction angle of 22 degrees should be utilized for side frictional force in the computation of uplift resistance. A factor of safety against uplift of at least 1.1 is recommended.

6.3.2 Foundation Support

Based on the results of the test borings completed for the Project, it is anticipated that dense Glacial deposits or bedrock will be encountered at the planned invert elevations for the new structures. It is expected that these materials will provide adequate foundation support and that special preparation or treatment of subgrade bearing surfaces will not be required. An allowable bearing pressure of 2 tons per square foot (tsf) should be utilized for design.

Settlement of the structures founded in these materials is anticipated to be negligible assuming good construction workmanship. Refer to Section 8.4 for additional considerations with regard to subgrade preparation and backfill.

6.3.3 Walls

The structure walls should be designed for a combination of soil and groundwater pressure. The groundwater pressure should be calculated assuming the structure is empty and the groundwater level is at the exiting ground surface. It is anticipated that the walls will be fairly rigid and therefore not free to move sufficiently to mobilize active soil forces. An at-rest coefficient of lateral earth pressure is therefore recommended. Recommended engineering soil properties for computing lateral pressures are summarized in Table 1 in Section 4.

7.0 Geotechnical Design Considerations for the Consolidation Conduit

7.1 General

The primary purpose of this section is to discuss how geotechnical issues will affect design of the consolidation conduit.

The consolidation conduit will be a reinforced concrete pipe (RCP) constructed using a combination of trenchless and open cut methods. Segments (reaches) requiring trenchless installation were determined based on line and grade requirements, depth of installation, pipe size, installation lengths, presence of glacial soils and bedrock, groundwater table above pipe invert, ground cover, impacts to adjacent structures and managing surface disruptions. Conventional pipe jacking and utility tunneling were evaluated and considered as the preferred trenchless methods.

Table 2 provides a summary of the RCP pipe reaches for IIIA-4.

Table 2 Summary of Pipe Reaches

Pipe Reach	Installation Method	Nominal Diameter of Pipe (in)	Approximate Length (ft)	Approximate Depth Range to Invert (ft)
GSS to JC	Utility Tunneling	72	39	30
JC to OF-214 DS	Open Cut	60	151	22 to 26
OF-214 DS to OF-213 DS	Open Cut	54	203	19 to 26
OF-213 to MH213-3	Pipe Jacking	48	185	18 to 20
MH213-3 to MH213-1	Open Cut	48	285	12 to 18
MH213-1 to OF210 DS	Pipe Jacking	48	60	16 to 18
JC to MH217-1	Open Cut	48	275	19 to 29
MH217-1 to MH217-2	Pipe Jacking	48	19	18 to 19
MH217-2 to MH217-3	Pipe Jacking	48	214	16 to 19
MH217-3 to MH217-4	Open Cut	48	266	16

7.2 Trenchless Methods

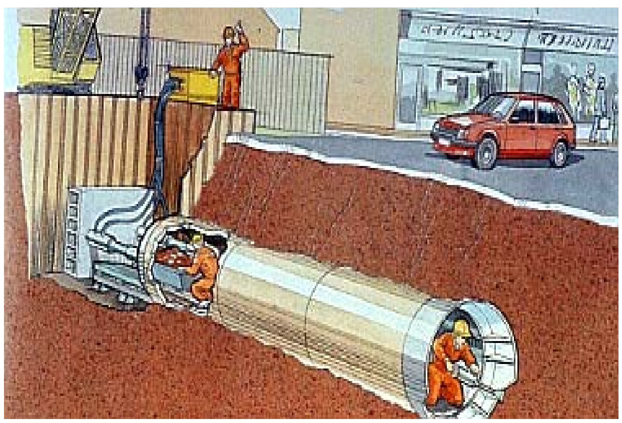
7.2.1 Conventional Pipe Jacking

Pipe jacking (PJ) consists of advancing pipe through the ground with thrust provided by hydraulic jacks and excavation of soil at the front of the pipe string either manually or mechanically. The process requires personnel entry into the pipe being jacked to operate excavation equipment. The excavation method can vary from a very basic process of hand excavation (such spades and shovels) to excavation using mechanical digging arms. Pipe jacking is typically limited to lengths less than 500 feet.

Regardless of the excavation method, it is usually accomplished inside of a shield attached to the leading section of pipe. The purpose of the shield is to provide a safe working environment for the workers and allow the bore to stay open during the excavation process for the pipe to be jacked into place. Typically, pipe jacking is done with pipes 48 inches in diameter and larger which accommodates worker entry. Pipe jacking is typically used in soil types with low permeability and good stand-up time, or in soils that have been dewatered or otherwise pre-stabilized by ground improvement methods such as grouting.

Control of line and grade is maintained with the use of an optical or laser guidance system and shield jacks for steering corrections.

The process requires a simple, cyclic procedure for installing individual sections of pipe, then jacking the pipe forward by utilizing hydraulic jacks reacting against the back wall of the jacking pit. In unstable ground conditions, the face is excavated simultaneously with the jacking operation. In stable ground conditions, excavation may be advanced slightly ahead of the jacking process The excavated spoil is removed through the inside of the pipe, typically with small carts, to the jacking pit. The typical components are shown below.



Conventional Pipe Jacking

The jacked pipe can either be a temporary casing or the product pipe. If the product pipe is jacked directly, it needs to be designed to handle the pipe jacking loads without damage. Directly jacking the product pipe is usually more economical than jacking a temporary casing pipe and installing the product pipe later due to the larger excavation size and annular backfill grouting requirement. It is recommended that direct jacking of the product pipe be considered for this project.

The friction force between the ground and the shield and pipe can be reduced by the injection of bentonite, or other lubricating fluid, into the annular space between the jacking pipe and the ground. If the shield and pipe annulus is sealed, the friction force can be reduced further by pressurizing the bentonite in the annulus to reduce the amount of closure due to unstable soils.

Face stabilization is used to minimize flowing ground behavior encountered below the groundwater table and raveling ground behavior above the groundwater table. Face stabilization for conventional pipe jacking typically consist of direct support of the face with breasting boards, doors, or sand shelves when needed. Obstructions are readily accessible due to the openness of the shield and mobility of the excavation tools.

PJ requires that a jacking pit and receiving pit be excavated at either end of the casing to be installed. Staging area requirements at the jacking pit are relatively limited, with plant and equipment generally consisting of a small crane, front end loader, spoil storage area and stockpiled casing pipe segments. The jacking pit for PJ is generally at least 15 feet long and is generally governed by the length of pipe (casing) sections to be jacked. Receiving pits can be somewhat smaller and must be large enough to remove the PJ shield.

7.2.2 Utility Tunneling

Utility tunneling (UT) is like conventional pipe jacking discussed above with the primary difference being the tunnel lining techniques. In all pipe jacking methods, the jacked pipe supports the ground during the excavation. In UT, ground supports consisting of prefabricated steel or concrete liner plates, or steel ribs and wood lagging systems are installed incrementally as the excavation advances. The lining is installed in the tunneling shield near the tunnel face as the soil is removed and the tunneling shield advances and forms a continuous support of the exposed soil. The typical components of utility tunneling are shown below.

Utility Tunneling Components

There are two basic lining system used for UT: two-pass and one-pass. A two-pass lining system is one in which an initial lining is placed as the tunnel is excavated and the final lining is placed later, typically after tunnel excavation is completed. For the ground conditions anticipated for this project, the most likely initial lining type would be steel ribs and lagging or steel liner plate. The final tunnel lining will consist of reinforced concrete pipe (RCP). A one-pass lining system is one in which the permanent lining is installed as the tunnel is excavated. The permanent lining typically consists of bolted and gasketed pre-cast concrete segments that are erected to form a ring within the tunneling shield. Use of one-pass lining systems is typically limited to tunnels 8 to 10 feet in diameter or larger.

The line and grade guidance systems to be used are like those for conventional pipe jacking. Steering control is accomplished during the soil excavation and shield advancing process. The tunneling shield is usually equipped with jacking cylinders at its rear portion, which propel the shield forward by jacking against the already erected liner sections as the face excavation proceeds. In addition to propelling the shield forward the jacks are used to steer the shield thereby enabling alignment change or correction if required.

7.3 Trenchless Reaches

This section provides a brief description and discussion for each reach of the consolidation conduit that is recommended to be constructed using trenchless methods.

Anticipated ground behavior relative to soft ground tunnel construction is described according to the "Tunnelman's Ground Classification," as originally developed by Terzaghi's (1950) and modified by

Heuer (1974), and presented in Appendix E, Table E.1. The Tunnelman's Ground Classification is an empirical classification of behavior wherein general soil types are categorized into different groups based on typical behavior at the face of an open-face soft ground tunneling shield above and below the groundwater.

Ground behavior relative to rock tunneling is described in general accordance with the descriptive classification scheme developed by Terzaghi (1946) as presented in Table E.2 included in Appendix E. This classification scheme was originally developed to estimate rock loads for design of steel supports for 10 to 20-foot diameter tunnels excavated by drill and blast methods. While this classification scheme is not widely used for estimating loads for design of tunnel linings of the type anticipated for the consolidation conduit, the descriptive terminology is useful for describing dominant rock mass behavior, particularly for rock masses where gravity is the dominant driving force.

7.3.1 GSS-2 to the Junction Chamber

The consolidation conduit between GSS-2 and the JC is oriented east west perpendicular to Taft Street, just south of Jenks Way. The 72-inch diameter pipe is approximately 39 feet in length and sloping towards GSS-2 at 0.22 percent. Existing ground surface is at approximately El. 23 with depth to invert at about 30 feet.

Subsurface conditions along this reach are anticipated to consist of Fill materials over Glacial deposits and bedrock. At GSS-2, bedrock is anticipated and consists of strong, gray Sandstone. Towards the JC, bedrock is anticipated to transition to Glacial Deposits that consist of very dense, coarse to fine gravel with varying amounts of sand and silt. The groundwater level is anticipated at about 10 to 20 feet below the existing ground surface, ranging in from El. 17 to El. 1.5, sloping downward towards the JC and the Seekonk River.

Based on review of the 1884 historical plan information, waterfront buildings once occupied the area between the Division Street Bridge and Jenk’s Way on the east side of Taft Street. The information predates construction of Jenks Way, Roosevelt Avenue Extension, and I-95. There are currently no existing buildings in this area. However, it is uncertain if remnants of old foundations still exist.

During the advancement of test boring B-4, concrete was encountered at approximately 8 feet below the existing ground surface and the test boring was relocated two times. There was no evidence of the presence of existing utilities in the area based on review of the available utility plans. It appears that this test boring was located in the general footprint of a former building.

Open-cut and trenchless construction techniques were considered for the installation of the consolidation conduit between GSS-2 and JC with trenchless techniques being selected as the preferred installation method. Constructability challenges uniquely identified for conventional open-cut construction include the unknown condition of the existing underground electric utilities, the ability to adequately support the existing unreinforced concrete duct-banks, and the required excavation support given the anticipated depth of construction. Groundwater management is a constructability challenge for both open-cut and trenchless techniques.

Trenchless construction of the consolidation conduit mitigates potential impacts to the existing electrical utilities. Trenchless installation will be conducted from the temporary excavations prepared for GSS-2 and the JC structure.

Two trenchless construction techniques were considered; pipe jacking and utility tunneling: Utility tunneling is recommended given the relatively short reach (approximately 39 feet), ability to excavate mixed soil and bedrock conditions, and the versatility of conducting mining operations from within the GSS-2 and JC excavations.

Ground improvement methods, such as jet grouting, is proposed to manage groundwater along this relatively short reach. The modified ground would enable mining operations to be conducted in stable conditions. It is anticipated that the ground improvements will occur following the construction of the excavation support systems for GSS-2 and the JC. Ground improvement would take place from the ground surface. If deemed necessary, supplemented grouting could be conducted from within the face of the tunnel during mining.

7.3.2 OF-213 DS to MH213-3

The consolidation conduit between OF-213 DS and MH213-3 is located along Roosevelt Avenue Extension beginning on the east side of the roadway and crossing over to the west side of the roadway. The 48 inch diameter pipe is approximately 185 feet in length. Existing ground surface is at approximately El. 24, with a depth to invert at about 18 feet.

Subsurface conditions along this reach are anticipated to consist of Fill materials over Glaciofluvial deposits followed by Glacial Till deposits. The Fill consists of medium dense sand and gravel with trace amounts of silt. Wood fragments were noted in test boring B-7 performed along this reach at a depth of about 11 feet below the existing ground surface. The Glaciofluvial deposits are dense to very dense fine sand and with little silt. The groundwater level is anticipated at about El. 15, above the pipe crown.

A portion of the Pawtucket Substation No. 2 facility extends underground along this segment with multiple duct banks conveying the underground electrical distribution system. The ability to support these facilities is uncertain when considering open-cut construction techniques. Therefore, trenchless construction in the form of pipe jacking is recommended. Dewatering along the trenchless alignment will be required to allow for open face mining.

7.3.3 MH213-1 to DS-210

The consolidation conduit between MH213-1 and OF-210 DS is located from the end of Roosevelt Avenue Extension to the middle of the intersection with Main Street. The 48 inch diameter pipe is approximately 60 feet in length. Existing ground surface ranges from approximately El. 32 to El. 31 with a depth to invert of about 16 feet to 18 feet.

Subsurface conditions along this reach are anticipated to consist of Fill materials over Glacial deposits followed by bedrock. The Fill consists of medium dense sand with some gravel silt. The Glacial Deposits consist of loose to very dense, gravel and sand and may include fragments of fractured rock. Bedrock is anticipated along the planned invert and consists of strong, fresh Sandstone. The groundwater level is anticipated at about El. 22, above the consolidation conduit crown.

Open-cut and trenchless construction techniques were considered for the installation of the pipe along this reach. Numerous underground utilities exist within Main Street and include electric, communication, gas, water, drain, sewer, and gas lines. Due to the presence of these utilities, open-cut installation methods were discounted. Therefore, trenchless construction in the form of pipe jacking is recommended.

7.3.4 MH217-1 to MH217-3

The consolidation conduit between MH217-1 and MH217-3 is located along Taft Street beneath the I-95 Bridge and includes two sections of 48 inch diameter pipe totaling 233 feet in length. Existing ground surface ranges from approximately El. 16 to El. 12, sloping downward towards the south. Depth to invert ranges from about 16 feet to 19 feet. An additional manhole (MH217-2) was required to realign the alignment as it passed beneath the bridge to avoid remnant bridge foundations as discussed below.

The existing reinforced concrete foundation for the original I-95 bridge was left in place following construction of the new Bridge. The extent and composition including reinforcement are unknown. Partial or complete removal of the remnant foundation was considered but was rejected primarily due to its size and depth below ground surface potential environmental impacts, the construction impacts to the existing electrical vault, and the overall cost.

Subsurface conditions along this reach are anticipated to consist of Fill materials or Glacial deposits over bedrock. The Fill consists of loose to very dense sand and gravel with silt. The Glacial deposits consist of dense, coarse to fine, gravel and sand and may include fragments of fractured rock. Bedrock was encountered at elevations ranging from approximately El. 0 to El. -10 and was generally described as hard, slightly weathered to fresh, interbedded sandstone and siltstone. Based on the historic test borings, the bedrock is observed to be generally dipping to the east towards the river. The groundwater level is anticipated at about El. 0, at the consolidation conduit crown.

Due to the presence of the existing utilities, open-cut installation methods were discounted. Therefore, trenchless construction in the form of pipe jacking is recommended.

7.4 Line and Grade Control and Jacking Loads

Based on the conditions encountered in the test borings, special consideration relative to control of line and grade are not anticipated. The contract documents will include required submittals to be prepared by the Contractor addressing line and grade control measures and contingency plans that describe the operational changes that will be made to make the necessary corrections to the alignment.

It is anticipated that jacking loads can be managed to acceptable levels through the continuous use of lubricants along the pipe exterior during jacking. It is also anticipated that during stoppages temporary increases in startup jacking force loads will occur. If temporary increases occur during placement of pipe in the pipe string, this should be carefully monitored as it could be an indication of adverse ground behavior that may warrant 24/7 mining to prevent the pipe from locking up. Since the actual magnitude of jacking force is highly dependent on the Contractor’s means and methods, the estimated jacking loads is the responsibility of the Contractor.

7.5 Open Cut Reaches

Open cut methods will be used along the remaining reaches of the consolidation conduit alignment where the depth of the required excavation is generally about 25 feet and the existing utility conflicts are less restrictive. Refer to Figures 2 and 3 and the individual boring sticks, which indicate the soil units anticipated to be encountered along these reaches. Descriptions and engineering properties of the soil units are provided above in Section 4.

Excavation through Fill materials and naturally deposited Glacial soils will be required to install the pipe along these reaches. Man-placed obstructions including remnant foundation should be anticipated within the Fill materials. Cobbles and boulders should be anticipated within the Glacial deposits. The groundwater level is anticipated to range as discussed in Section 5. Excavation of the Fill materials and naturally deposited Glacial soils will likely be accomplished using mechanical excavation equipment, such as a long arm excavator. Refer to Section 8.3 for additional considerations with regard to groundwater management.

Excavation support will be selected and designed by the Contractor. Applicable excavation support systems include, but are not limited to, steel soldier piles and timber lagging with soldier piles set in predrilled-holes spaced from 6 to 10 feet on-center. Drill-holes should be backfilled with lean mix concrete to final excavation subgrade after each soldier pile is in-place. Above the excavation subgrade, the drill holes should be backfilled with excavatable flow fill to the existing site grade. The excavation support system should be internally braced with steel struts and walers. Bracing levels should be kept to a minimum to maximize efficiency.

As noted previously, remnant foundations and features of previous structures, including concrete and brick foundations, and other miscellaneous debris are anticipated to be encountered along some segments of the alignment. The use of trench boxes or slide rail systems for temporary support can be considered for locations where the excavation depth is about 20 feet or less. However, due to unknowns associated with man-made obstructions expected below grade, this may be prohibitive.

7.6 Foundation Support for Consolidation Conduits

Based on the results of the test borings completed for the Project, it is anticipated that dense Glacial deposits or bedrock will be encountered at the planned invert elevations for the consolidation conduit.

It is expected that the Glacial deposits and bedrock will provide adequate foundation support and that special preparation or treatment of subgrade bearing surfaces will not be required. Crushed stone encapsulated in geotextile fabric can be placed over bedrock to provide bedding and even out irregular bedrock features. Settlement of the consolidation conduit founded in these materials is not anticipated if they are properly placed on undisturbed soils and adequately backfilled.

Where Fills are encountered along the alignments at the proposed pipe invert, it is recommended that the trench be over-excavated a minimum of two feet below normal design subgrade and backfilled with compacted sand and gravel, crushed stone, or screened gravel. A geotextile filter fabric should completely encapsulate the over-excavated section if crushed stone or screened gravel is used. Refer to Section 8 for additional considerations with regard to subgrade preparation and backfill.

8.0 Geotechnical Construction Considerations

8.1 General

The purpose of this section is to provide recommendations and comments on geotechnical aspects of the proposed construction. While it is not possible to foresee all geotechnical-related problems that may arise during construction, several potential issues suggested by site and subsurface conditions, proposed construction, and the results of geotechnical field and laboratory investigations and studies are discussed.

Prospective Contractors for this Project should evaluate potential construction issues based on their own knowledge and experiences with similar ground conditions in the area, taking into account their own proposed construction methods.

8.2 Working Space

The Project is located in an urban area and restrictions on construction traffic routes and work hours around the sites may be imposed. The staging areas will be confined to areas identified on the contract drawings. It is anticipated that the former Masonic Temple property will be used for Contractor staging and for ingress/egress of construction vehicles. Traffic management plans should be developed with the goal of limiting impact to local traffic, residents, and businesses, as well cyclists and pedestrians.

Typical support surface area requirements need to accommodate the following:

Drill Rig for Temporary Support Elements	Groundwater Management Equipment and Piping
Shaft Crane	Crew Change Trailer
Pipe Storage	Lubrication System
Material and Parts Storage	Temporary Muck Storage
Generator and Backup Power Supply	Field Office
Truck Transport for Spoil off-site Disposal	Front End Loader

8.3 Groundwater Management

To construct the permanent structures and along the open cut reaches of the alignment, active site de-watering may be necessary. In addition, permeation grouting may be necessary in certain pipe jacking areas to cut-off groundwater inflows. In addition, groundwater will need to be removed from within the excavation limits by use of localized sumps as excavation proceeds. This ensures maintenance of a suitable subgrade to work from.

Based on the presence of gravel and weathered rock encountered in test borings located along the along the reach from GSS-2 to the JC, grouting may be required to create a plug to cut-off groundwater inflows.

8.4 Subgrade Preparation

Bedding and backfill requirements for the new structures and consolidation conduits may vary depending upon the method of construction. If the structures are precast, the bedding should consist of a material recommended by the precast manufacturer. Typically, compacted fill behind and beneath new structures and consolidation conduits consist of bank-run sand and gravel, crushed stone, or screened gravel free of organic material, snow, ice, or other unsuitable materials. Other materials could be acceptable for bedding and backfill and should be evaluated on a case-by-case basis if proposed by the Contractor. Backfill above the bedding should consist of suitable material excavated for construction of structures or trenches or off-site borrow.

Backfill should be placed in lift thicknesses not exceeding 12 inches loose measure and compacted to at least 95 percent of the maximum dry density. In confined areas, hand guided equipment such as a large vibratory plate compactor should be used, and the loose lift thickness should not exceed 6-inches. Cobbles or boulders having a size exceeding 2/3 of the loose lift thickness should be removed prior to compaction.

The subgrade surface should be firm, dry, and undisturbed to provide for adequate support. Until the subgrade is protected, incidental traffic of workers and equipment across the final subgrade should be prohibited. Disturbed subgrade surfaces should be excavated to undisturbed soils. In addition, a geotextile filter fabric should be used to provide a separation barrier between bedding materials and the natural subgrade soils. The filter fabric used should be fully encapsulating, extending up the sides of the excavation and over the top of the bedding.

Precautions should be taken if work takes place during any time temperatures fall below freezing. Soil bearing surfaces below new structure foundations or pipelines must be protected against freezing.

8.5 Re-use of Excavated Materials

It is anticipated that excavated soil from construction of the new structures and consolidation conduit will primarily consist of Fills and Glacial Deposits, and to a lesser degree, bedrock. Reuse of some of these materials may be possible for backfilling between the walls of the structures and the excavation support system, above structures, above open cut reaches of the consolidation conduit, or for site grading.

Fill soils are anticipated to contain considerable amounts of deleterious materials and are generally not considered suitable soils for backfill. Glaciofluvial and Glacial Till deposits can likely be reused as granular backfill. Excavated bedrock is generally not considered suitable for backfill.

If sheeting is used to support the excavation, backfill between the sheeting and structure wall face should be comprised of a suitable mixture of compacted sand and gravel with the percentage of fines by weight less than 10 percent. Backfill above structures and open cut reaches of the consolidation conduit should be comprised of a suitable mixture of compacted sand and gravel with the percentage of fines by weight of no more than 35 percent. Crushed stone could also be used to minimize the required compaction effort. However, if crushed stone is used, a filter fabric would be required to mitigate movement of fine-grained soil.

Laboratory grain size analyses should be performed on the stockpiled material to confirm whether it will meet the applicable pipe bedding and backfill criteria.

8.6 Protection of Structures

As indicated on Figures 2 through 5, utilities exist along the Project alignment. Some utilities are planned to be relocated and discussions are ongoing with the utility owners. Utilities not being relocated prior to excavation that cross the alignment will need to be supported in-place during construction.

The field stone masonry retaining wall is located along the Seekonk River and has been determined to be a gravity wall based on test pitting performed in December 2020. The stone making up the wall is bonded together with cement mortar and has an estimated batter of 1 foot horizontal to 3 feet vertical on the earth retaining side. Based on this information, it appears the footprint of the open cut for laying pipe will not directly interfere with the existing stone masonry retaining wall. Excessive vibration and load on the wall due to traffic and construction may cause damage and should be mitigated by design and managed by continuous monitoring..

To the east of the JC, an existing stone wall is located above the Seekonk River wall and appears to have been built to support earth fill associated with the construction of Jenks Way and Roosevelt Avenue Extension. The condition of the wall is variable and appears to have been repaired at least once at the base of the wall near its tallest point. Vibrations associated with construction of the pipeline may have impacts to the wall. Due to the proximity of the OF-214 DS and the required upgrades to OF-214, a portion of the wall is identified to be removed and replaced.

8.7 Geotechnical Instrumentation

A geotechnical instrumentation program is recommended to determine if ground movements are as anticipated. If unanticipated ground movements are observed, the data can be used to assist in evaluation of required modified construction methods to reduce the ground movement to acceptable levels. Presented below is a summary of general recommendations:

- Deformation monitoring points
- Inclinometers installed adjacent to the shaft walls to measure the horizontal deflections of the wall and soil;
- Structure monitoring points should be installed on all above ground structures within the construction zone of influence;
- Additional groundwater observation wells should be installed along reaches where dewatering is implemented to monitor the groundwater levels during construction;
- Seismographs to monitor ground vibrations adjacent to existing structures and utilities.

8.8 Construction Monitoring

It is recommended that an experienced geotechnical engineer or technician qualified by training and experience be present during pertinent construction phases, such as pipe jacking, utility tunneling, excavation, and preparation of near surface structure and open cut pipeline foundation subgrades. The general purpose of the on-site monitoring program is to provide accurate documentation of construction activities. This will allow an overall perspective of the construction progress and verify compliance with the Project Documents and the Building Code. It is recommended that a representative be on site to:

- Observe trenchless construction
- Observe and test, if necessary, the exposed soils at subgrade levels to confirm that in-situ conditions are consistent with those predicted for design, and to observe that the natural soils are not disturbed by construction activities

- Observe and document the installation of the consolidation conduit in open cut reaches

In order to clearly document potential changed conditions claims, a qualified field representative should prepare accurate day-to-day documentation of construction conditions and the work performed by the Contractor. This is particularly important during tunneling, excavation for near surface structures, and pipe installation in open cut reaches. This will enable the design team to observe compliance with the design concepts and specifications, help resolve construction problems, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

9.0 References

Chute, N.E. 1949. Surficial Geology of the Pawtucket Quadrangle, Rhode Island-Massachusetts, US Geological Survey.

Denny, C. 1982. Geomorphology of New England. U.S. Geological Survey Professional Paper 1208. Washington, D.C.: U.S. Govt. Printing Office.

Deere, D. U., and D. W. Deere. 1988. The Rock Quality Designation (RQD) Index in Practice. In Rock Classification Systems for Engineering Purposes. Edited by L. Kirkaldie. Philadelphia: American Society for Testing and Materials.

Frimpter, M. H., and A. Maevsky. 1979. Geohydrologic impacts of coal development in the Narragansett Basin, Massachusetts, and Rhode Island. Geological Survey Water-Supply Paper 2062. Washington, D.C.: U.S. Govt. Printing Office.

Haley & Aldrich report entitled "Geotechnical Data Report – Section 1; Main Spine and Seekonk Tunnels and Shafts, Vol. IV of IV of the Geotechnical Preliminary Design Report, Phase IB – Preliminary Design; CSO Control Facilities Program; Providence, Rhode Island, dated 1995.

Halberg, H. N., C. E. Knox, and F. H. Pauszek. 1961. Water resources of the Providence area, Rhode Island. Water resources of industrial areas. Geological Survey Water-Supply Paper 1499-A. Washington, D.C.: U.S. Govt. Printing Office.

Heuer, Ronald E. 1974, "Important Ground Parameters in Soft Ground Tunneling", Subsurface Explorations for Underground Excavation and Heavy Construction, ASCE Specialty conference, pp. 41-55.

Quinn, A. W. 1959. Bedrock Geology of the Providence Quadrangle, Rhode Island, U.S. Geological Survey.

Quinn, A.W., Ray,R.G., and Seymour,W.L. 1949, Bedrock Geology of the Pawtucket Quadrangle, Rhode Island-Massachusetts. U.S. Geological Survey.

Skehan, J. W., N. Rast, and S. Mosher. 1986. Paleoenvironmental and tectonic controls of sedimentation in coal-forming basins of southeastern New England. In Paleoenvironmental and tectonic controls in coal-forming basins of the United States. GSA Special Paper 210. Ed. P. C. Lyons and C. L. Rice; p. 9-30. Boulder, Colorado: Geological Society of America.

Smith, J. H. 1956. Surficial Geology of the Providence Quadrangle, Rhode Island, US Geological Survey.

Stantec and Pare Corporation, report entitled "NBC CSO Phase IIIA Contract No. 308.01 C, Request for Proposal, Appendix C, Geotechnical Data Report", dated December 23, 2019.

State of Rhode Island, Department of Public Works, Division of Roads and Bridges, Plan, Profile and Sections of Proposed State Highway, Division St. Project, Contract Three, Rhode Island FA Project No.1-01(11), Contract No. 5737, dated 1957.

State of Rhode Island, Department of Transportation, Plans, Profiles, and Section of Proposed Bridge Replacement, Pawtucket Bridge No. 550, I-95 Over the Seekonk River, Volume 3 Bridge Plans, Rhode Island Contract No. 2010-CB-004, FA Project Nos.BRO-0550(003), IM-0550(004), IMG-0550(005), prepared by Commonwealth Engineers & Consultants Inc., dated April 2010.

Terzaghi, K.1946. Rock Defects and Loads on Tunnel Supports. Edited by Proctor and White. In Rock Tunneling with Steel Supports. Youngstown, Ohio: Commercial Shearing and Stamping Company.

FIGURES

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

Narragansett Bay Commission

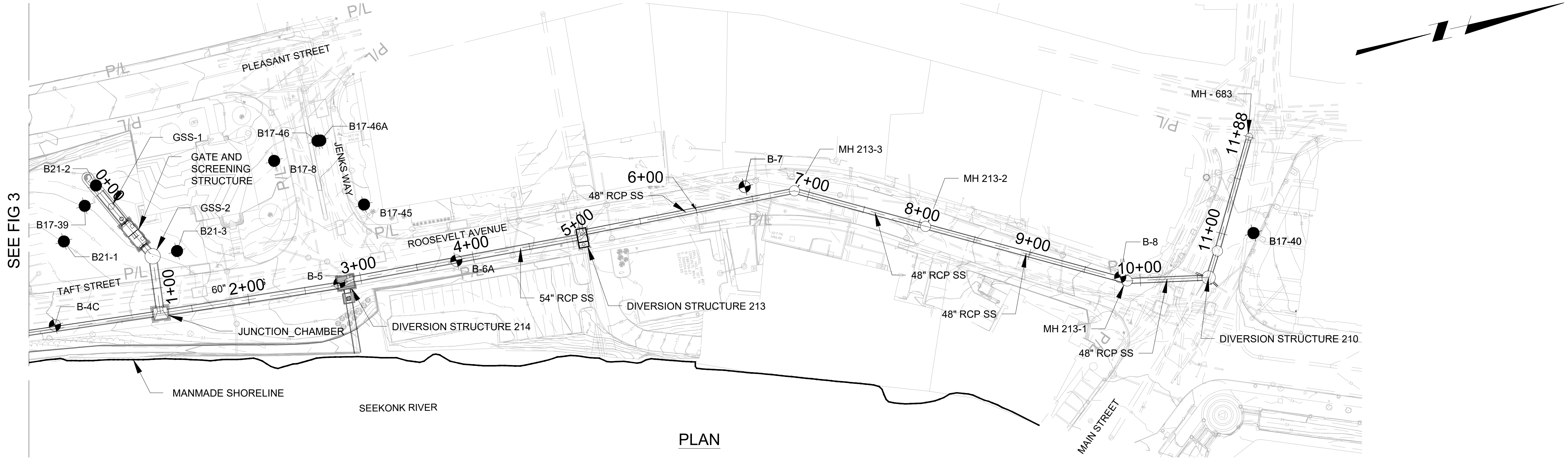
NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

Stantec PARE

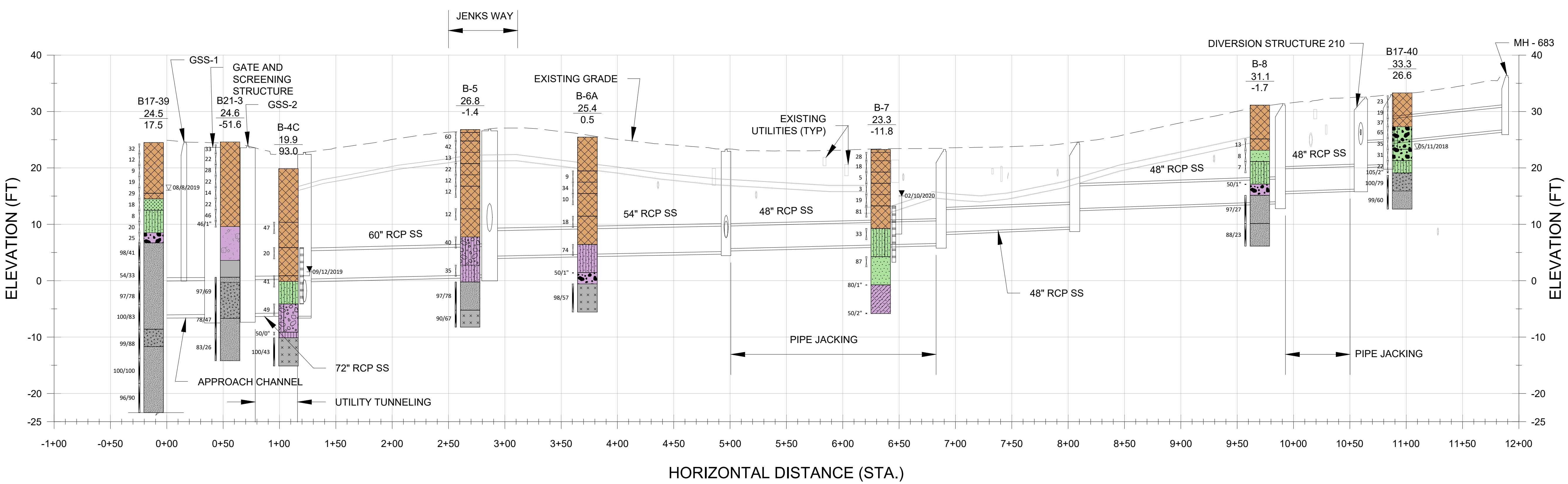
GEOTECHNICAL DESIGN SUMMARY REPORT
CONSOLIDATION CONDUIT IIIA-4
PROJECT LOCUS
PAWTUCKET, RHODE ISLAND

DATE
JULY, 2021

FIGURE
1



PLAN



PROFILE

NOTE:

SEE FIGURE 6 FOR GENERAL LEGEND AND NOTES.

VERT SCALE IN FEET
SCALE 1" = 20'

HORZ SCALE IN FEET
SCALE 1" = 100'

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

Stantec

PARE

GEOTECHNICAL DESIGN SUMMARY REPORT

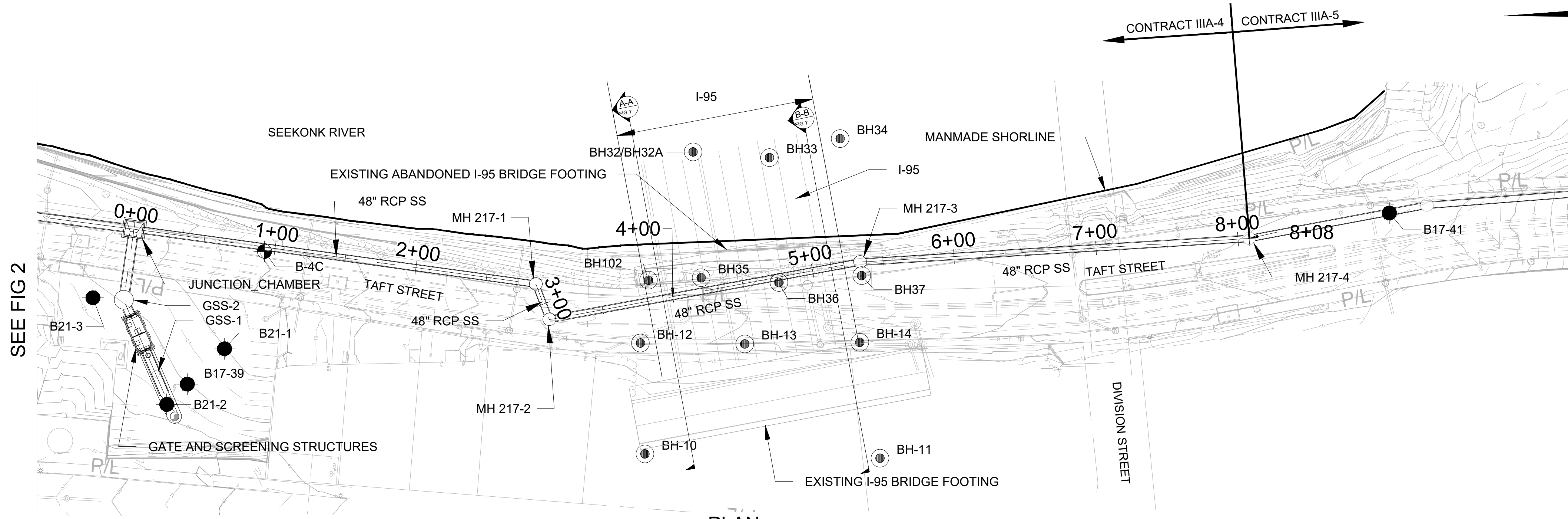
CONSOLIDATION CONDUIT IIIA-4
GEOTECHNICAL PLAN AND PROFILE STA 0+00 TO STA 11+88

DATE

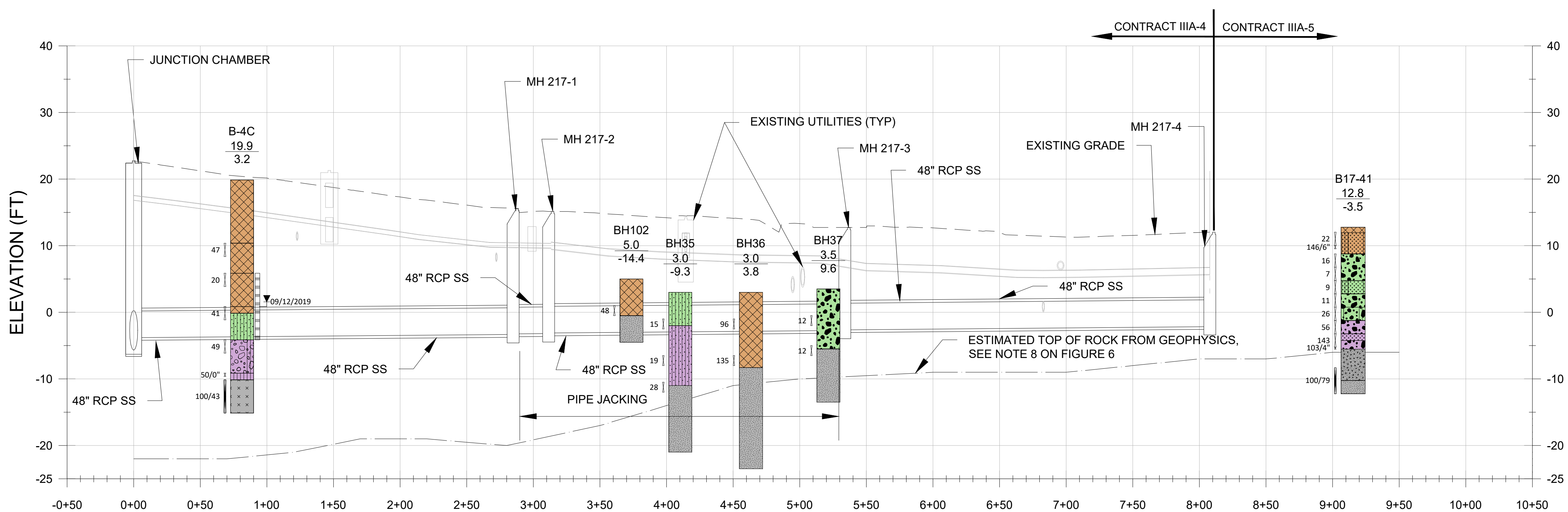
JULY, 2021

FIGURE

2



PLAN



PROFILE

VERT SCALE IN FEET
SCALE 1" = 20'

HORZ SCALE IN FEET
SCALE 1" = 100'

NOTE:

SEE FIGURE 6 FOR GENERAL LEGEND AND NOTES.

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

Narragansett Bay Commission

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

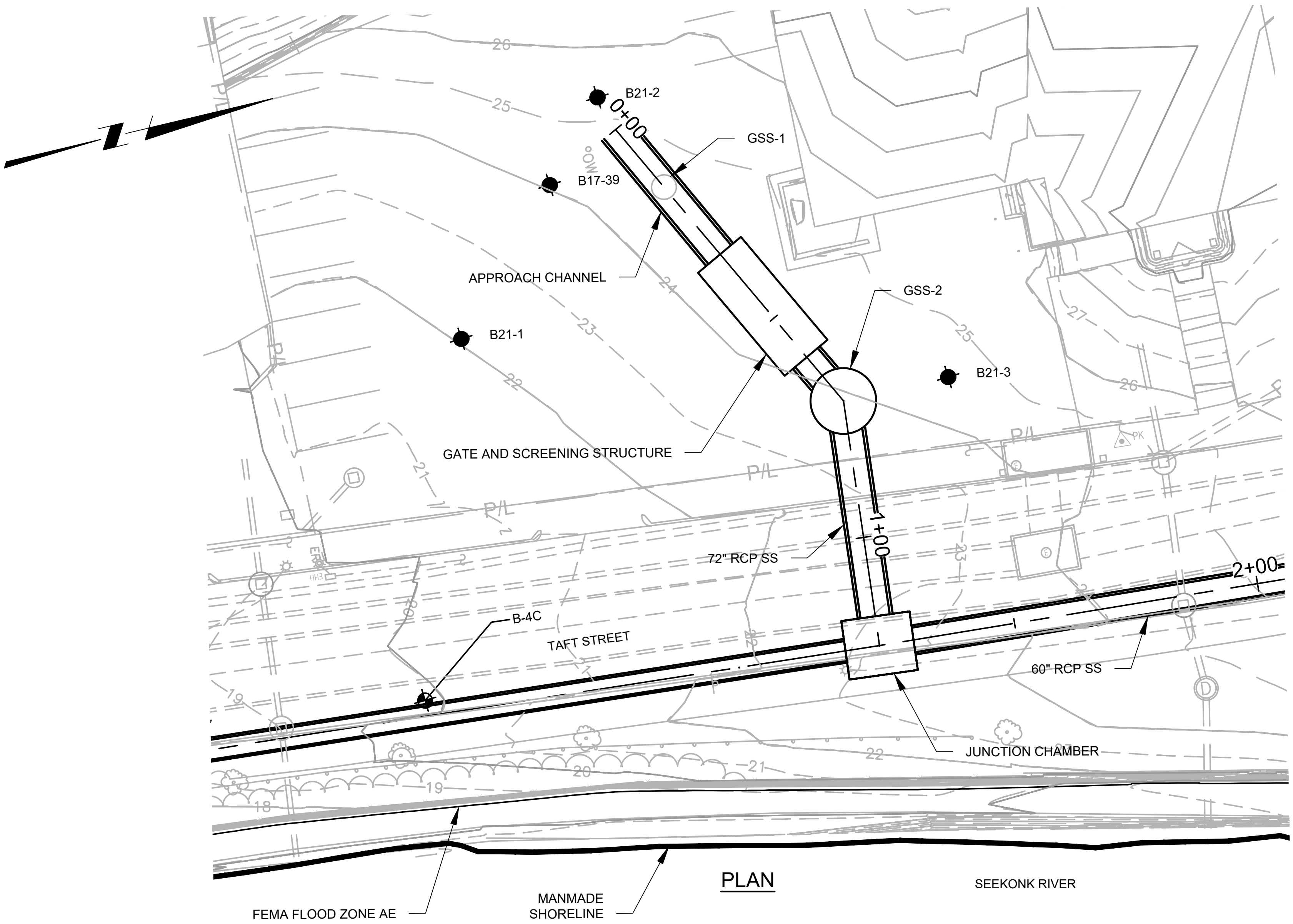
Stantec PARE

GEOTECHNICAL DESIGN SUMMARY REPORT

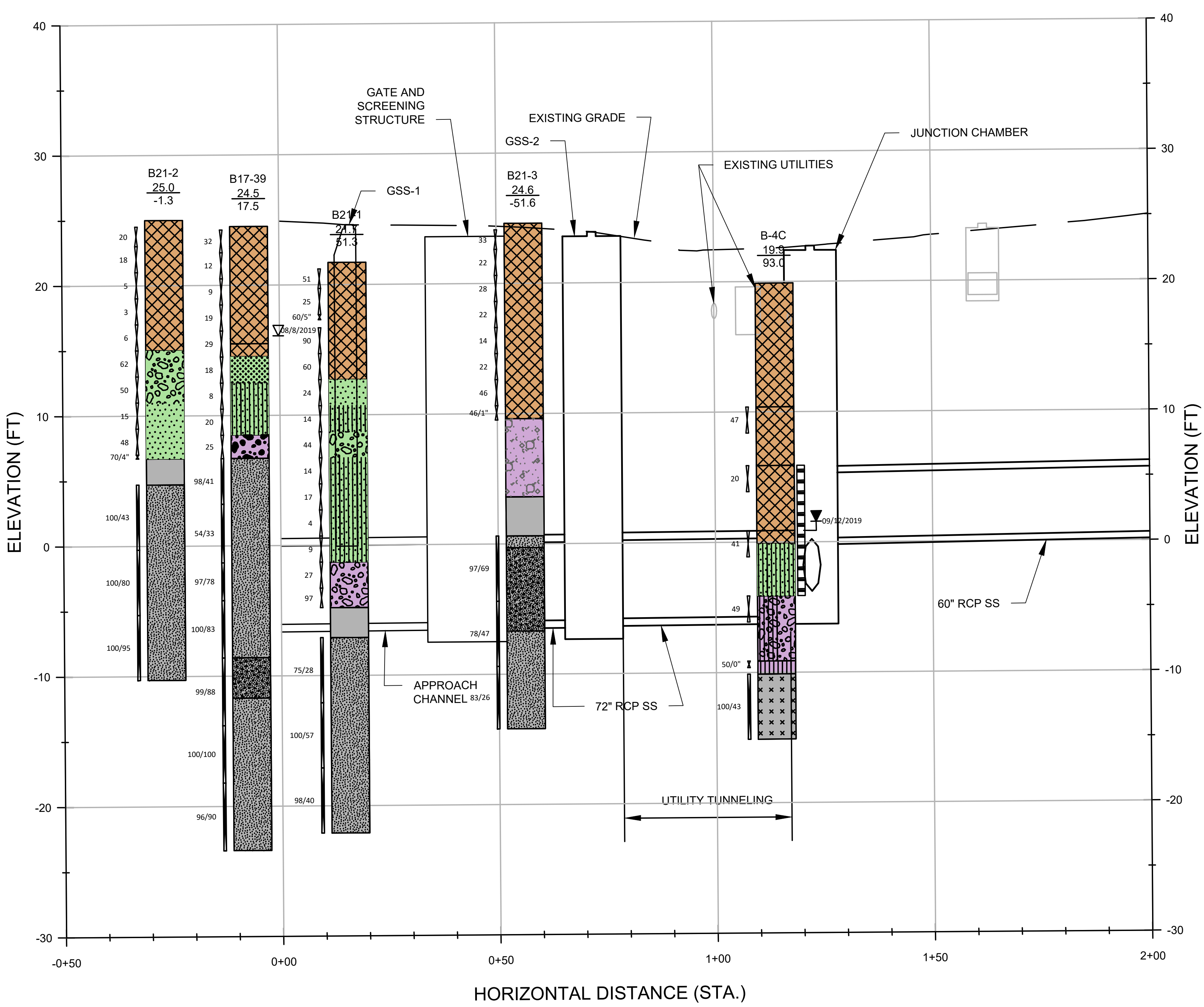
CONSOLIDATION CONDUIT IIIA-4
GEOTECHNICAL PLAN AND PROFILE STA 0+00 TO STA 8+08

DATE
JULY, 2021

FIGURE
3



PLAN



PROFILE

30 0 30
HORZ SCALE IN FEET
SCALE 1" = 30'

10 0 10
VERT SCALE IN FEET
SCALE 1" = 10'

NOTE:

1. SEE FIGURE 6 FOR GENERAL LEGEND AND NOTES.

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

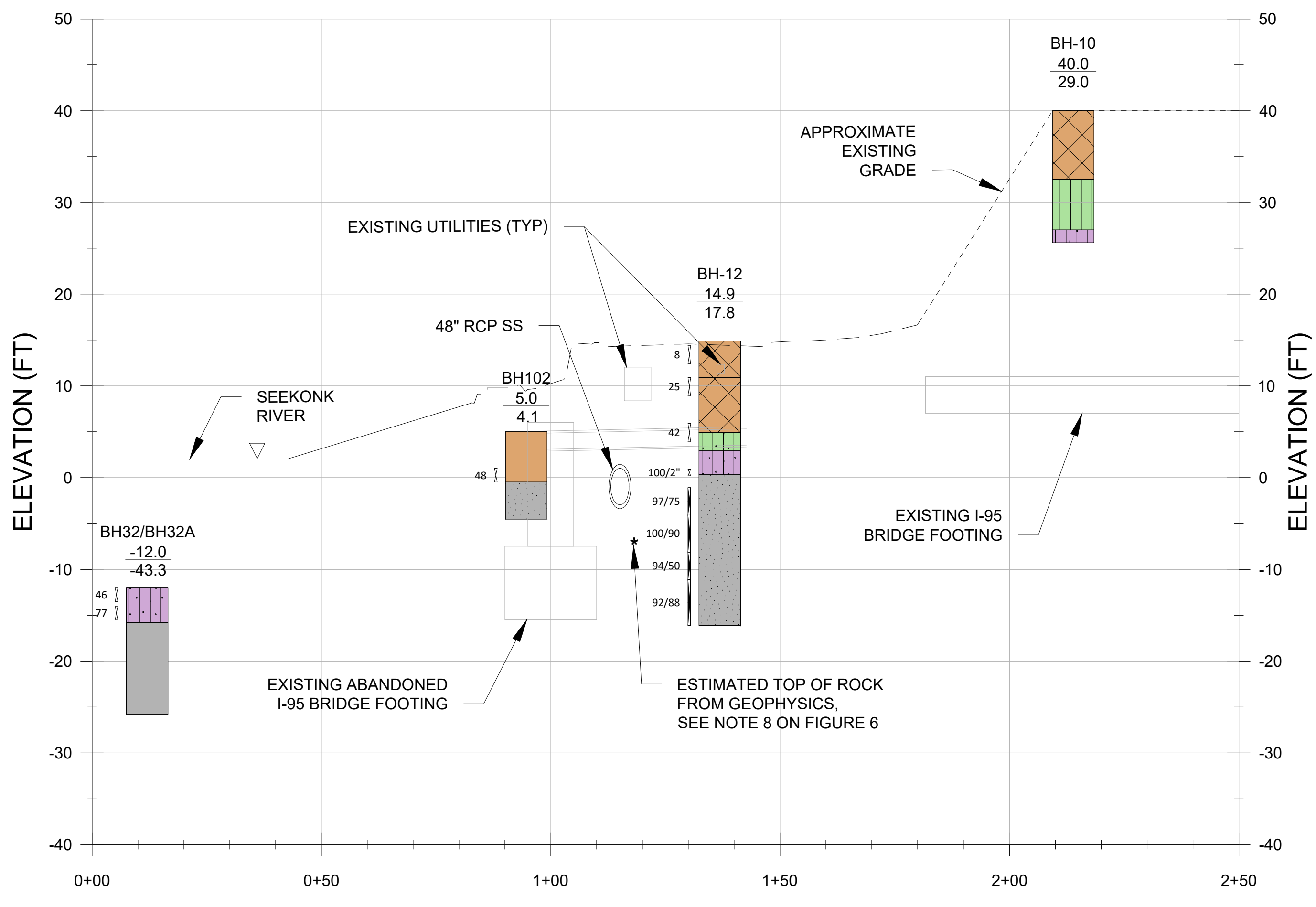
Stantec PARE

GEOTECHNICAL DESIGN SUMMARY REPORT

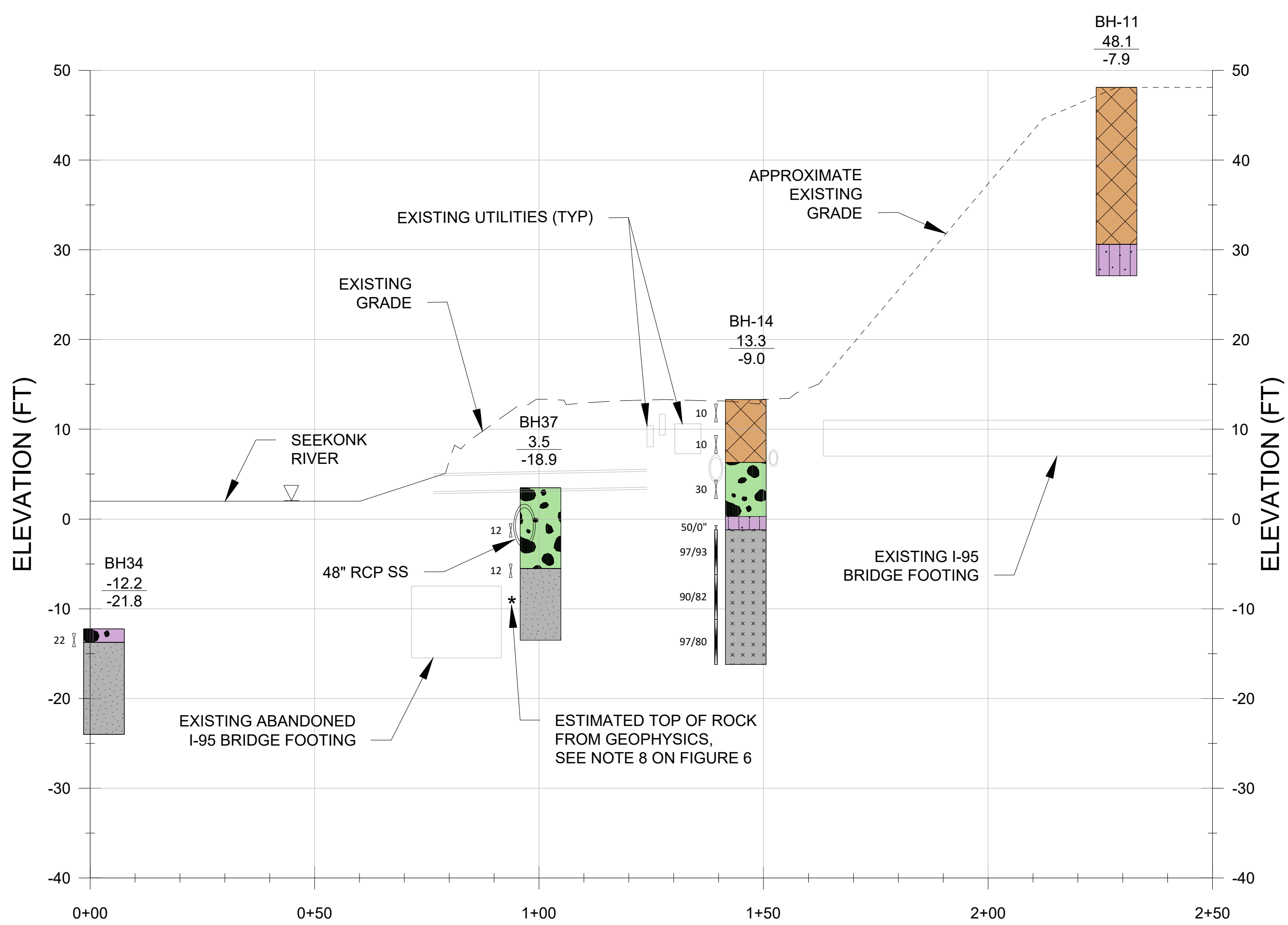
CONSOLIDATION CONDUIT IIIA-4
GEOTECHNICAL PLAN AND PROFILE STA. 0+00 TO STA. 1+22.19

DATE JULY, 2021

FIGURE 4



SECTION A-A



SECTION B-B

NOTE:

SEE FIGURE 6 FOR GENERAL LEGEND AND NOTES.

VERT SCALE IN FEET
SCALE 1" = 20'

HORZ SCALE IN FEET
SCALE 1" = 40'

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER OVERFLOW PROGRAM

Stantec PARE

GEOTECHNICAL DESIGN SUMMARY REPORT

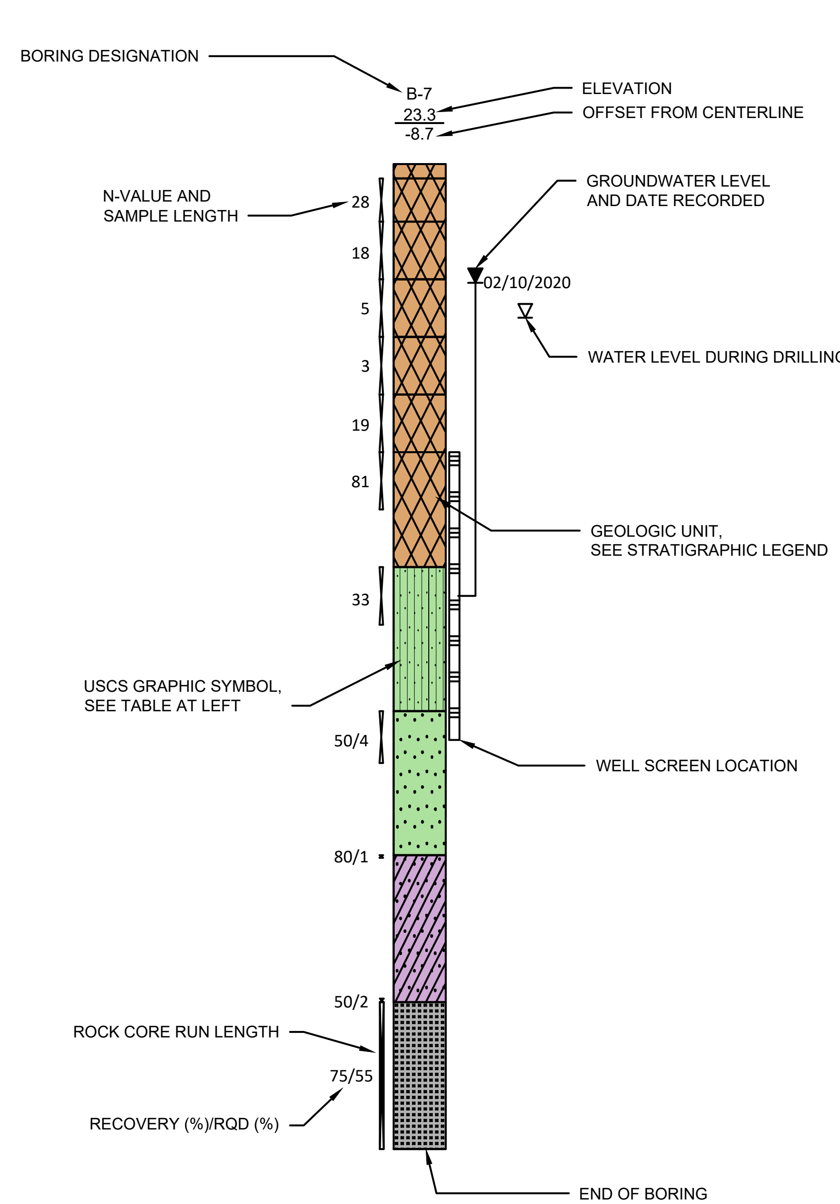
CONSOLIDATION CONDUIT IIIA-4
I-95 CROSS SECTIONS

DATE JULY, 2021

FIGURE 5

UNIFIED SOIL CLASSIFICATION SYSTEM (Based on ASTM D2488 & D2487)					
MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (50% or more retained on No. 200 sieve)	GRAVELS (more than 50% retained on No. 4 sieve)	CLEAN GRAVELS (less than 5% fines)	GW		WELL-GRADED GRAVEL
			GP		POORLY GRADED GRAVEL
		GRAVELS (with 5 to 12% fines)	GW-GM		WELL-GRADED GRAVEL WITH SILT
			GW-GC		WELL-GRADED GRAVEL WITH CLAY
			GP-GM		POORLY GRADED GRAVEL WITH SILT
			GP-GC		POORLY GRADED GRAVEL WITH CLAY
		GRAVELS WITH FINES (more than 12% fines)	GM		SILTY GRAVEL
			GC		CLAYEY GRAVEL
			GC-GM		SILTY CLAYEY GRAVEL
	SANDS (less than 50% retained on No. 4 sieve)	CLEAN SANDS (less than 5% fines)	SW		WELL-GRADED SAND
			SP		POORLY GRADED SAND
		SANDS (with 5 to 12% fines)	SW-SM		WELL-GRADED SAND WITH SILT
			SW-SC		WELL-GRADED SAND WITH CLAY
			SP-SM		POORLY GRADED SAND WITH SILT
			SP-SC		POORLY GRADED SAND WITH CLAY
		SANDS WITH FINES (more than 12% fines)	SM		SILTY SAND
			SC		CLAYEY SAND
			SC-SM		CLAYEY SAND WITH SILT
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS & CLAYS (liquid limit less than 50)	INORGANIC	ML		SILT
			CL		LEAN CLAY
			CL-ML		CLAY WITH SILT
		ORGANIC	OL		LOW PLASTICTIY ORGANIC CLAY
	SILTS & CLAYS (liquid limit greater than 50)	INORGANIC	MH		ELASTIC SILT
			CH		FAT CLAY
		ORGANIC	OH		HIGH PLASTICTIY ORGANIC CLAY
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER		PT		PEAT

BORING LEGEND:



NOTES:

1. BASE PLAN USED FOR THIS FIGURE WAS A CAD-GENERATED DRAWING TITLED "PAWT_SITE_PLAN_&_PROFILE_IIIA-4" PREPARED BY BETA GROUP, INC. AND PROVIDED ON 2/24/2021.
2. ALL ELEVATIONS ARE IN FEET AND REFER TO THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NVGD29).
3. NORTH ARROW ALIGNED WITH GRID NORTH, RHODE ISLAND STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).
4. POSITIVE OFFSET = RIGHT OF CENTERLINE, LOOKING UP STATION.
5. NEGATIVE OFFSET = LEFT OF CENTERLINE, LOOKING UP STATION.
6. THE SOIL STRATIGRAPHY SHOWN IS GENERALIZED INTERPRETATION BASED ON THE SAMPLES COLLECTED WITHIN EACH BORING. NO ATTEMPT WAS MADE TO INTERPOLATE SOIL STRATIGRAPHY BETWEEN BORINGS AS THE DISTRIBUTION OF MATERIALS IS VARIABLE AND NON-UNIFORM IN BOTH VERTICAL AND HORIZONTAL DIRECTIONS.
7. AS DRILLED LOCATIONS OF THE B-SERIES TEST BORINGS CONDUCTED IN 2019 AND 2020 WERE SURVEYED BY BRYANT AND ASSOCIATES, INC. IN NOVEMBER 2019 AND FEBRUARY 2020 AND PROVIDED TO MCMILLEN JACOBS ASSOCIATES BY BETA GROUP, INC.
8. GEOPHYSICS INFORMATION OBTAINED FROM THE NBC PHASE IIIA, PAWTUCKET TUNNEL GDR, PREPARED BY STANTEC/PARE, DATED DECEMBER 23, 2019. ACTUAL GEOPHYSICS STATIONING AND ESTIMATED TOP OF ROCK OBTAINED FROM APPENDIX C, LAND GEOPHYSICS SURVEY, SEISMIC REFRACTION SURVEY FOR THE NARRAGANSETT BAY COMMISSION, COMBINED SEWER OVERFLOW PROJECT, PAWTUCKET MAIN TUNNEL SECTION, WEST LINE SCAN, PLATES 1 AND 3, PREPARED BY HAGER-RICHTER GEOSCIENCE, INC., DATED NOVEMBER 2017.
9. FLOOD PLAIN INFORMATION IS FROM FEMA, PANEL NO. 44007C0194J. FLOOD PLAIN ELEVATIONS CONVERTED FROM VERTICAL DATUM NAVD 1988 TO NGVD 1929 AND ARE APPROXIMATELY:
   - NORTH OF DIVISION STREET BRIDGE: AE ELEVATION 12.8
   - SOUTH OF DIVISION STREET BIRDGE: VE ELEVATION 13.8

STRATIGRAPHIC LEGEND:

COLOR	GEOLOGIC UNITS	GRAPHIC SYMBOL	DESCRIPTION
	FILL		SILTSTONE
	GLACIOFLUVIAL DEPOSITS		SANDSTONE
	GLACIAL TILL DEPOSITS		CONGLOMERATE
	BEDROCK		

BORING SYMBOL KEY:

B-7	DESIGNATION AND LOCATION OF TEST BORING PERFORMED BY GEOLOGIC EARTH EXPLORATION, INC. DURING THE PERIODS 27 AUGUST THROUGH 18 SEPTEMBER 2019 AND 04 FEBRUARY THROUGH 12 FEBRUARY 2020 UNDER THE SUPERVISION OF MCMILLEN JACOBS ASSOCIATES.
BH-35	DESIGNATION AND LOCATION OF TEST BORING PERFORMED FOR THE PAWTUCKET BRIDGE AND PAWTUCKET BRIDGE REPLACEMENT PERFORMED IN 1955 AND 2010.
B17-39	DESIGNATION AND LOCATION OF TEST BORING PERFORMED BY NEW ENGLAND BORING CONTRACTORS, INC. PERFORMED BETWEEN 04 MAY 2018 AND 08 AUGUST 2019 AND IN MARCH 2021 UNDER THE SUPERVISION OF PARE CORPORATION.

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

Narragansett Bay Commission

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

Stantec
PARE

GEOTECHNICAL DESIGN SUMMARY REPORT
CONSOLIDATION CONDUIT IIIA-4
GENERAL LEGEND AND NOTES FOR
GEOTECHNICAL PLANS AND PROFILES

DATE
JULY, 2021

FIGURE
6

APPENDICES

APPENDIX A
Glossary of Technical Terminology

Glossary of Technical Terminology

Abrasivity: Characteristic of rock that contributes to the wearing down of tool or equipment surfaces that are in frictional contact with the rock.

Alluvium. Material, such as sand, silt, or clay, deposited on land by streams.

Aquifer: A saturated stratum, formation, or group of formations of soil or rock capable of storing and transmitting water in economically significant quantities to wells and/or springs.

Bedrock. The solid rock that underlies the soil and other unconsolidated material or that is exposed at the surface.

Boulders. Rock fragments larger than 2 feet (60 centimeters) in diameter.

Dip: The angle at which any planar feature is inclined from the horizontal; always perpendicular to strike.

Discontinuity: A general term for any naturally occurring fracture in a rock mass having zero to low tensile strength. It is the collective term for most types of joints (bedding joints, foliation joints) and faults.

Flood plain. A nearly level alluvial plain that borders a stream and is subject to flooding unless protected artificially.

Formation: The basic or fundamental rock stratigraphic unit in the local classification of rocks consisting of a body of rock generally characterized by some degree of internal lithologic homogeneity or distinctive lithologic features by mappability at the surface or near subsurface.

Fracture: Any break in rock along which no significant movement has occurred.

Glaciofluvial deposits. Material moved by glaciers and subsequently sorted and deposited by streams flowing from the melting ice. The deposits are stratified and occur as kames, eskers, deltas, and outwash plains.

Glacial Till: (Hardpan) Unsorted, nonstratified glacial drift consisting of clay, silt, sand, and boulders transported and deposited by glacial ice.

Gravel. Rounded or angular fragments of rock up to 3 inches (2 millimeters to 7.5 centimeters) in diameter. An individual piece is a pebble.

Ground behavior: Reactions or manifestations of the ground as it is excavated and exposed.

Groundwater. Water filling all the unblocked pores of underlying material below the water table.

Hydraulic Conductivity: The potential rate of groundwater flow through a unit area of saturated soil or rock under a unit hydraulic gradient, measured at right angles to the groundwater flow direction.

N-Value: Standard penetration resistance defined as the number of blows required to drive the standard 1-3/8 in. I.D. split-spoon sampler with a 140-lb. hammer falling freely through a distance of 30 in. for a distance of 12 in., typically counted from the 6^{th} to the 18^{th} inch that the sampler is driven

Outwash, glacial. Stratified sand and gravel produced by glaciers and carried, sorted, and deposited by glacial melting.

Physiographic Province: A region of which all parts are similar in geologic structure and climate and which has consequently had a unified geomorphic history.

Reach: A single continuous portion of tunnel alignment.

Relief. The elevations or inequalities of a land surface, considered collectively.

Rock: Naturally occurring, coherent aggregate of one of more minerals; for example, limestone composed mainly of the mineral calcite.

Rock fragments. Rock or mineral fragments having a diameter of 2 millimeters or more; for example, pebbles, cobbles, stones, and boulders,

Sand: As a soil separate, individual rock or mineral fragments from 0.05 millimeter to 2.0 millimeters in diameter.

Silt: As a soil separate, individual mineral particles that range in diameter from the upper limit of clay (0.002 millimeter) to the lower limit of very fine sand (0.05 millimeter As a soil textural class, soil that is 80 per cent or more silt and less than 12 percent clay.

Specific Yield: The ratio of the volume of water drained by gravity for a material to the volume of the material.

Stratigraphy: A branch of geology dealing with the classification, nomenclature, correlation, and interpretation of stratified rocks.

Weathering: The physical disintegration and chemical decomposition of rock in situ.

APPENDIX B
Subsurface Exploration Key

SOIL

Soil description on logs of subsurface explorations are based on Standard Penetration Test (SPT) results, visual-manual examination of exposed soil samples, and the results of laboratory tests on selected samples. The criteria, descriptive terms, and definitions are presented herein.
The natural soils are identified and described by visual-manual procedures (ASTM D2488) and in accordance with the United Soil Classification System (USCS) (ASTM D2487) as practiced by McMillen Jacobs Associates. Fill materials may not be classified by USCS criteria.

PENETRATION RESISTANCE

Standard penetration resistance (SPT) (ASTM D1586) - Number of blows required to drive a standard 2 in. O.D. split spoon sampler one foot with a 140 lb. weight falling 30 inches freely downward.

DENSITY / CONSISTENCY

Coarse - Grained Soils	
Apparent Density	SPT Resistance, N (BPF)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine - Grained Soils	
Apparent Consistency	SPT Resistance, N (BPF)
Very Soft	0 - 2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	>30

Notes: BPF = Blows Per Foot (uncorrected)

WOR = Weight of Rod

COLOR

Basic colors (black, brown, gray, olive, red, and yellow) and combinations (i.e. gray-brown, olive-brown, olive-gray, red-gray, red-brown, yellow-brown, and red-yellow). Modifiers such as light and dark may be used.

SUPPLEMENTAL SOIL DESCRIPTIONS AND STRUCTURE:

Laminating	- 0 to 1/16 in. thick (cohesive)
Parting	- 0 to 1/16 in. thick (granular)
Seam	- 1/16 to 1/2 in. thick
Layer	- 1/2 to 12 in. thick
Stratum	- > 12 in. thick
Pocket	- Small, erratic deposit less than 12 in. size
Lens	- Lenticular deposit larger than a pocket
Occasional	- One or less per 12 in. of thickness
Frequent	- More than one per 12 in. of thickness
Interbedded	- Alternating soil layers of differing composition
Varved	- Alternating thin seams of silt and clay
Mottled	- Variation of color

SAMPLE SYMBOLS

⧗	SPT Sample 2 in. OD
X	SPT Sample 3 in. OD
▮	Shelby Tube Sample

ADDITIONAL GRAPHIC DESCRIPTIONS

Asphalt

Fill

SOIL IDENTIFICATION AND DESCRIPTION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS Based on ASTM D2488 & D2487)

MAJOR DIVISIONS			GROUP/SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (50% or more retained on No. 200 sieve)	GRAVELS (more than 50% retained on No. 4 sieve)	CLEAN GRAVELS (less than 5% fines)	GW		WELL-GRADED GRAVEL
			GP		POORLY GRADED GRAVEL
		GRAVELS (with 5 to 12% fines)	GW-GM		WELL-GRADED GRAVEL WITH SILT
			GW-GC		WELL-GRADED GRAVEL WITH CLAY
			GP-GM		POORLY GRADED GRAVEL WITH SILT
			GP-GC		POORLY GRADED GRAVEL WITH CLAY
		GRAVELS WITH FINES (more than 12% fines)	GM		SILTY GRAVEL
			GC		CLAYEY GRAVEL
	SANDS (less than 50% retained on No. 4 sieve)	CLEAN SANDS (less than 5% fines)	SW		WELL-GRADED SAND
			SP		POORLY GRADED SAND
		SANDS (with 5 to 12% fines)	SW-SM		WELL-GRADED SAND WITH SILT
			SW-SC		WELL-GRADED SAND WITH CLAY
			SP-SM		POORLY GRADED SAND WITH SILT
			SP-SC		POORLY GRADED SAND WITH CLAY
		SANDS WITH FINES (more than 12% fines)	SM		SILTY SAND
			SC		CLAYEY SAND
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS & CLAYS (liquid limit less than 50)	INORGANIC	ML		SILT
			CL		LEAN CLAY
		ORGANIC	OL		LOW PLASTICITY ORGANIC CLAY
	SILTS & CLAYS (liquid limit greater than 50)	INORGANIC	MH		ELASTIC SILT
			CH		FAT CLAY
		ORGANIC	OH		HIGH PLASTICITY ORGANIC CLAY
	SILT/CLAY (liquid limit between 12 and 25)	INORGANIC	CL-ML		CLAYEY SILT / SILTY CLAY
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER		PT		PEAT

Notes:

1. Dual symbols (symbols separated by a hyphen, e.g. SP-SM, slightly silty fine SAND) are used for soils between 5% and 12% fines or when liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

ROCK

Rock descriptions noted on logs of subsurface explorations are based on visual-manual examination of exposed rock outcrops and core samples. The criteria, descriptive terms and definitions used are as follows:

FIELD HARDNESS / STRENGTH

(after ISRM, 1978; CGS, 1985; Marinos & Hoek, 2001)

Extremely Strong	Cannot be scratched with a knife point or sharp pick; can only be chipped with repeated heavy hammer blows.
Very Strong	Cannot be scratched with a knife point or sharp pick; core breaks with repeated heavy hammer blows.
Strong	Can be scratched with a knife or pick; core breaks with heavy hammer blow.
Moderately Weak	Can be grooved 1/16 in. deep by knife or sharp pick; core breaks with light hammer blow.
Weak	Can be grooved easily with a knife or pick; can be scratched with fingernail; core breaks with light pressure.
Very Weak	Can be readily indented; grooved with fingernail or carved with a knife; core beaks with light pressure.

WEATHERING (after ISRM, 1978)

The action of organic and inorganic chemical and physical processes resulting in alteration of color, texture, and composition

Fresh	No visible sign of alteration, except perhaps slight discoloration on major discontinuity surfaces
Slight	Discoloration of rock material and discontinuity surfaces
Moderate	Less than half the rock material decomposed to soil. Some fresh rock; continuous "framework".
High	More than half the rock material decomposed and/or disintegrated to soil.
Completely	All rock material disintegrated to soil, but mass still intact
Residual Soil	All rock material converted to soil. Material has not been significantly transported.

COLOR:

Basic colors and combinations: gray, light gray, brown, red-brown

TEXTURE

Size, shape and arrangements of constituents

Aphantic	Individual grains invisible
Fine-grained	Grains barely visible to the unaided eye, up to 1/16 in. dia.
Medium Grained	Grains between 1/16 and 3/16 in. dia.
Coarse Grained	Grains between 3/16 and 1/4 in. dia.
Very Coarse Grained	Grains larger than 1/4 in. dia.

LITHOLOGY

Rock Classification and modifiers; accepted formation names

DISCONTINUITIES:

Type	Definition
Joint	A natural fracture along which no displacement has occurred. May occur in parallel groups called sets.
Shear	A natural fracture along which displacement has occurred. Surface may be slickensided or striated.
Fault	A natural fracture along which displacement has occurred. Usually lined with gouge and slickensides.
Shear or Fault Zone	Zone of fractured rock and gouge bordering the displacement plane.

ORIENTATION / ATTITUDE

Term	Angle (degrees)
Horizontal	0-5
Low Angle	6-35
Moderately Dipping	36-55
High Angle	56-85
Vertical	86-100

SPACING

Term	Inches
Extremely Close	<3/4
Very Close	3/4 - 2-1/2
Close	2-1/2 - 8
Moderate	8 - 24
Wide	24 - 80
Very Wide	80- 20 ft.
Extremely Wide	> 20 ft.

ROUGHNESS OF DISCONTINUITY SURFACE

Term	Abbreviation	Description
Very Rough	VR	Near-vertical steps and ridges
Rough	R	Ridges, side-steps, and asperities evident; abrasive to the touch
Slightly Rough	SR	Asperities can be felt.
Smooth	SM	Smooth to the touch
Slickensided	SL	Smooth glossy finish with visible striations

APERTURE/GAP

Term	MM
Very Tight	< .1
Tight	0.1 - 0.25
Partly Open	0.25 - 0.5
Open	0.5 - 2.5
Moderately Wide	2.5 - 10
Wide	> 10
Very Wide	10 - 100
Extremely Wide	100 - 1000
Cavernous	> 1000

INFILLING

Material	Abbreviation
Clay	CL
Silt	SI
Sand	SA
Serpentine	SE
Sulfide	SL
Calcite	CA
Pyrite	PY
Quartz	QZ
Chlorite	CH
Iron Oxide Staining	FE
Could not be determined	X

BEDDING

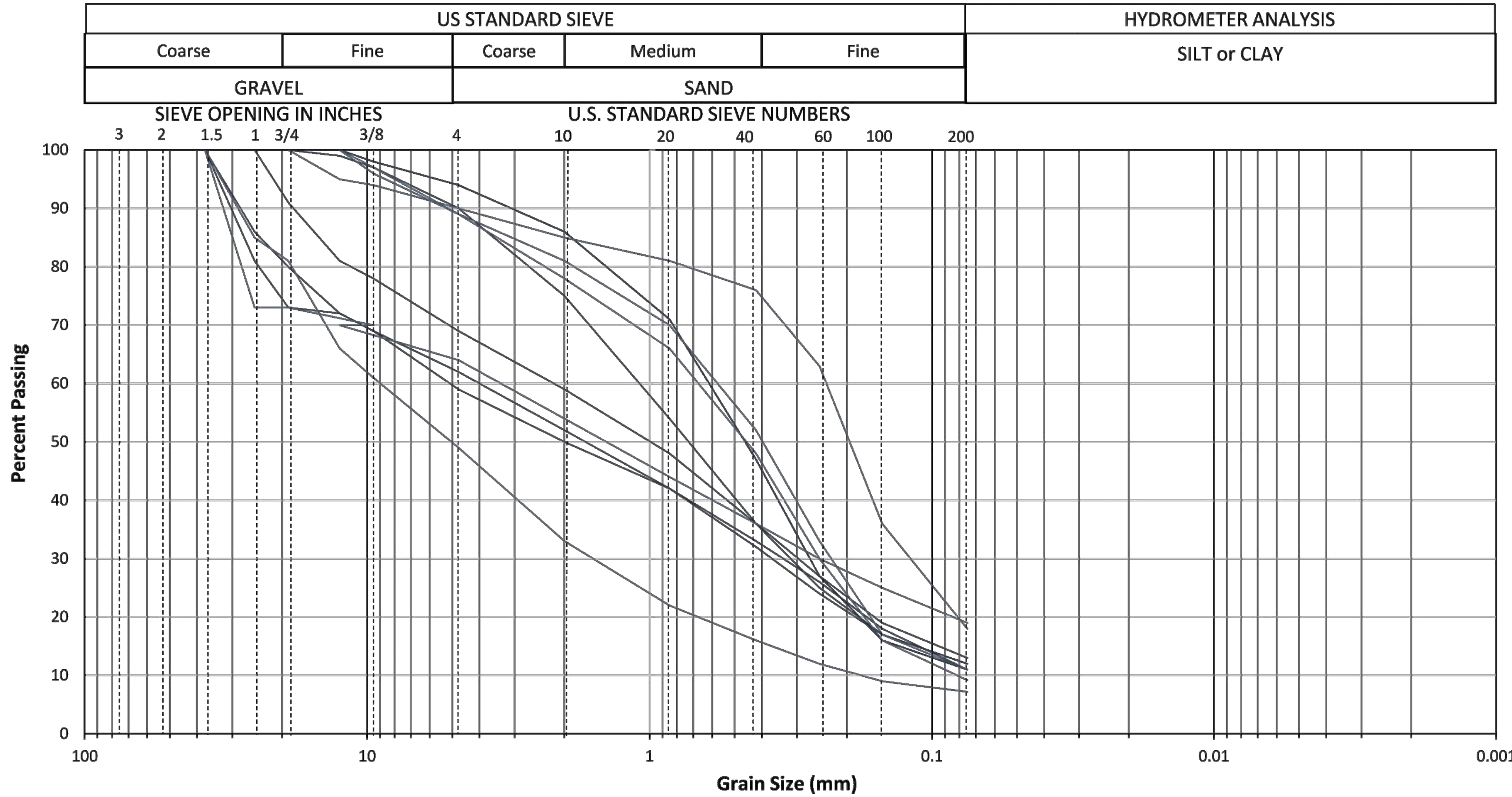
Term	Inches	Term	Inches
Very Thin	< 2.5	Thick	25-36
Thin	2.5 - 8	Very Thick	< 36
Medium	9 - 24		

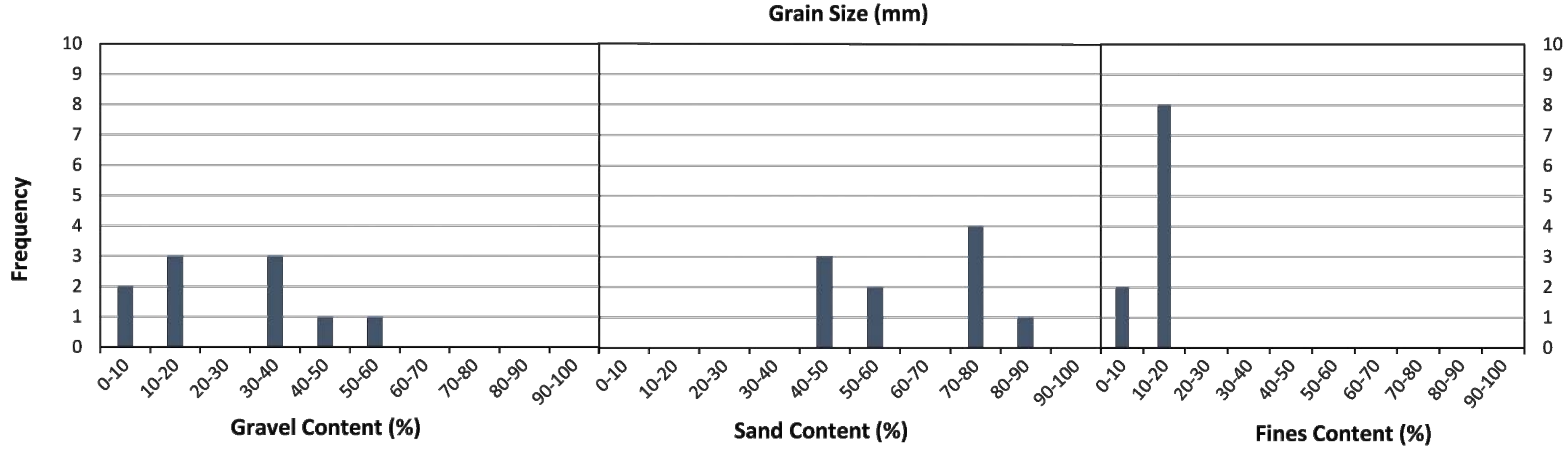
GENERAL NOTES:
1. Logs of subsurface exploration depict soil, rock and groundwater conditions only at the boring locations specified on the dates indicated. Subsurface conditions may vary at other locations and at other times.
2. Water levels, where noted on the logs, were measured at the times under the conditions indicated. During test boring drilling, these water levels could have been affected by the introduction of water in to the borehole, extraction of tools or other procedures and thus may not reflect actual groundwater levels at the test boring location. Groundwater level fluctuations may also occur as a results of variations in precipitation, temperature, season, tides, river stage, adjacent construction operations, construction dewatering systems, water supply well pumping, and other conditions.

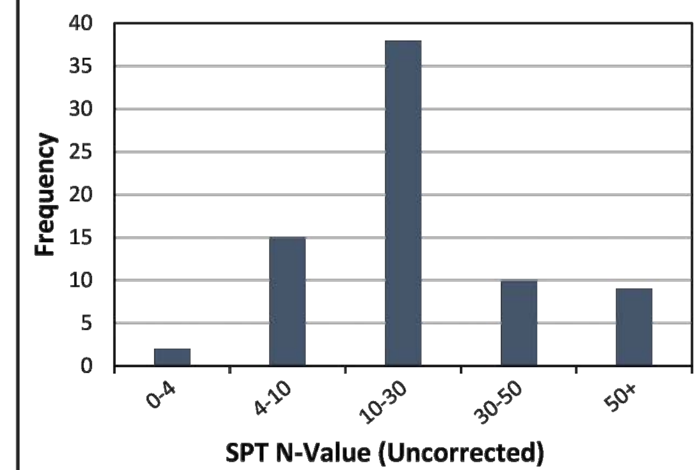
McMILLEN JACOBS ASSOCIATES

SUBSURFACE EXPLORATION KEY

APPENDIX C
Soil Data Summaries
(IIIA-4 Data)







Note:
1: Histogram prepared using data from the following borings: B-4C, B-5, B-6A, B-7, B-8, B17-8, B17-39, B17-40, B17-41, B17-45, B17-46A, BH-12, BH-13, BH-14, B21-1, B21-2, and B21-3.

Note:
1: Plot and histograms prepared using data from the following borings: B-4C, B-5, B-7, B17-8, B17-39, B17-40, B17-41, B17-45, and B17-46A.

DRAFT

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

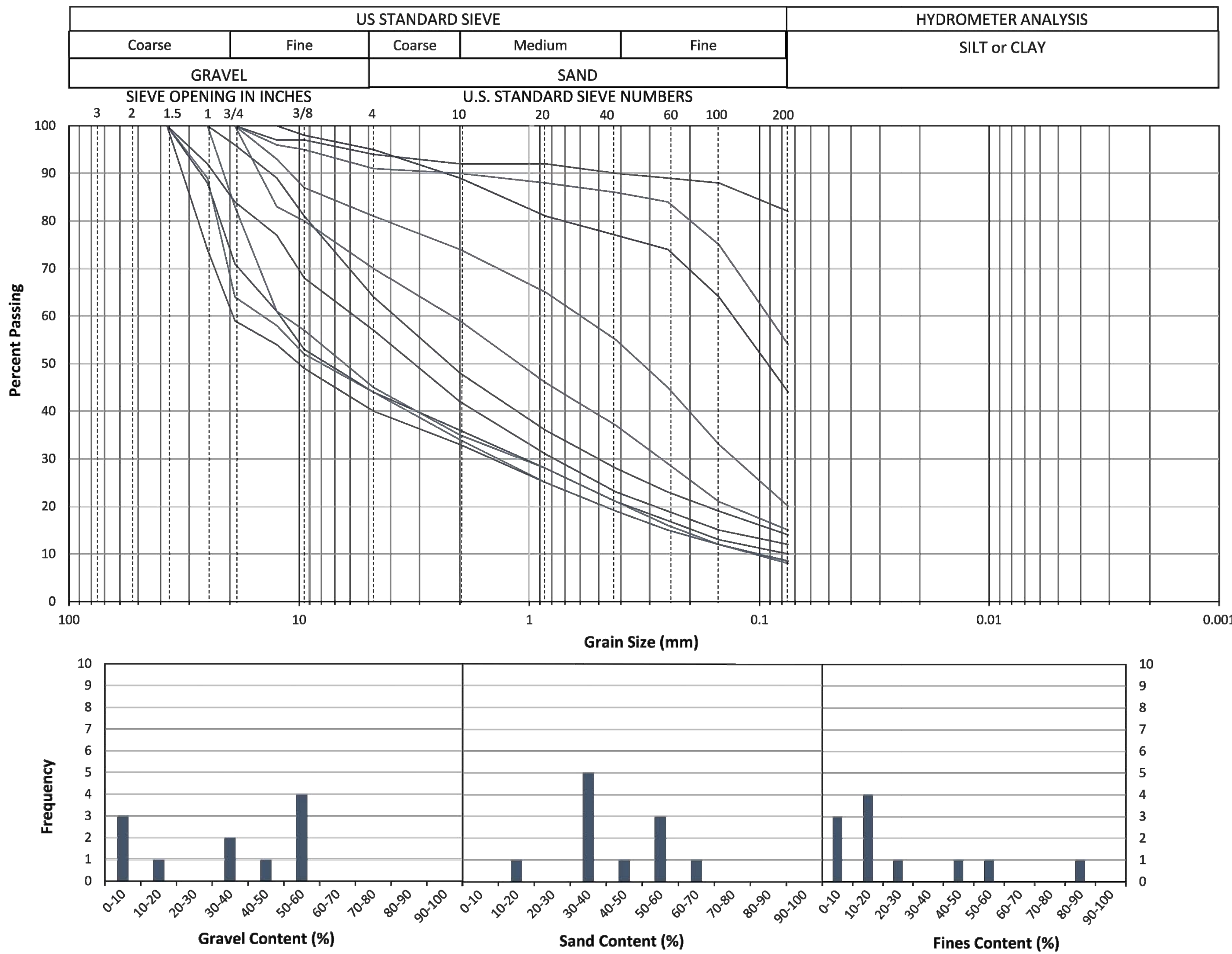
NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

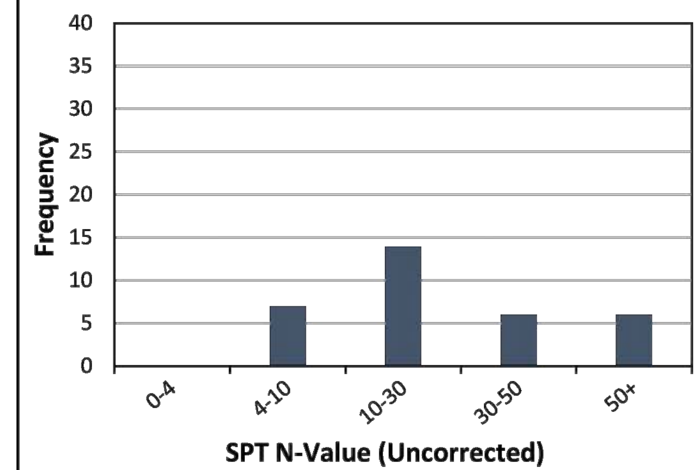
Stantec PARE

NBC PHASE III CSO PROGRAM
CONSOLIDATION CONDUIT IIIA-4
GEOTECHNICAL DESIGN SUMMARY REPORT
SOIL PROPERTY SUMMARY - FILL

DATE
MAY, 2021

FIGURE
C-1





Note:
1: Histogram prepared using data from the following borings: B-7, B-8, B17-39, B17-40, B17-41, B17-45, BH-14, B21-1, and B21-2.

Note:
1: Plot and histograms prepared using data from the following borings:B-4C, B-7, B-8, B17-39, B17-40, B17-41, and B17-45.

DRAFT

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

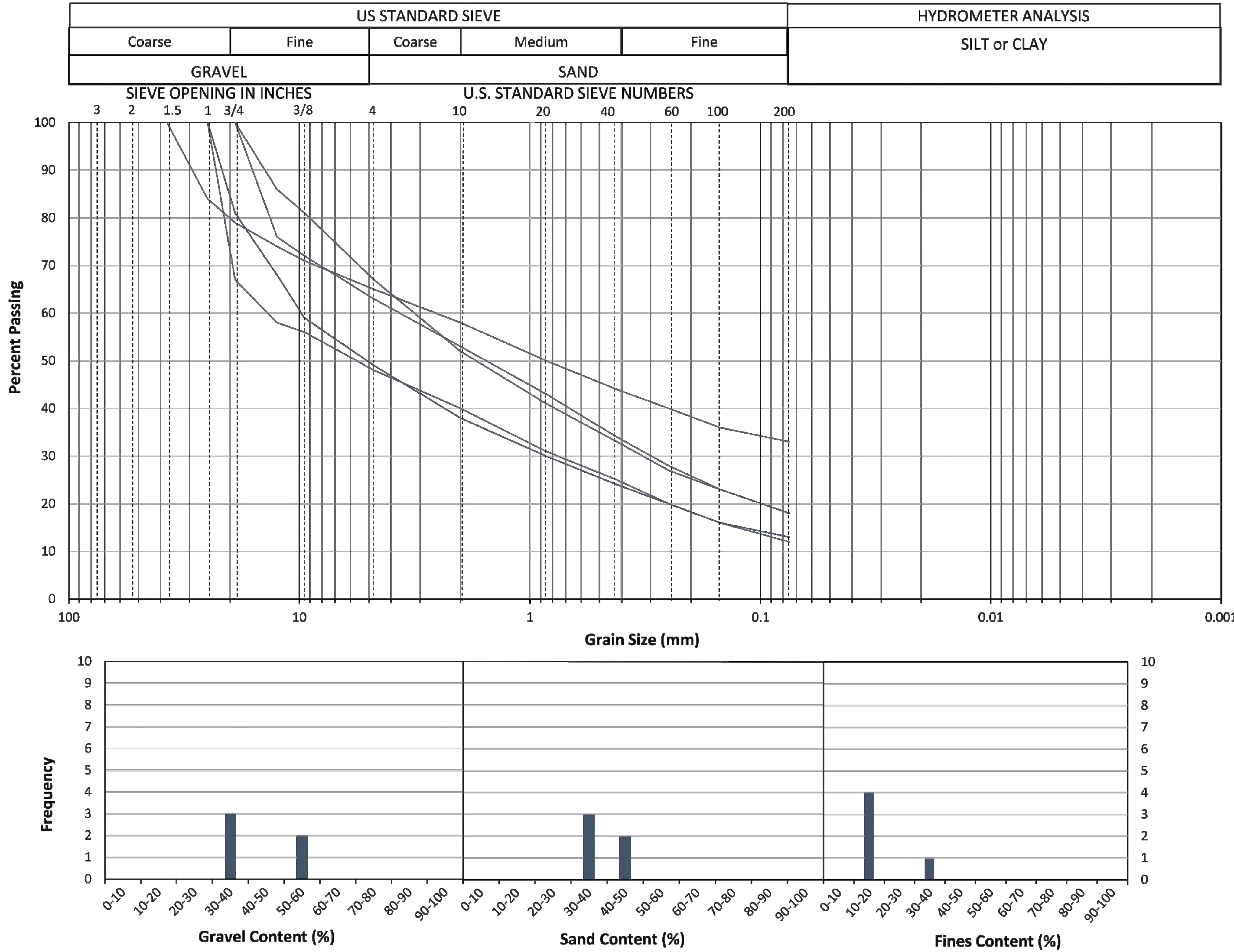
Stantec

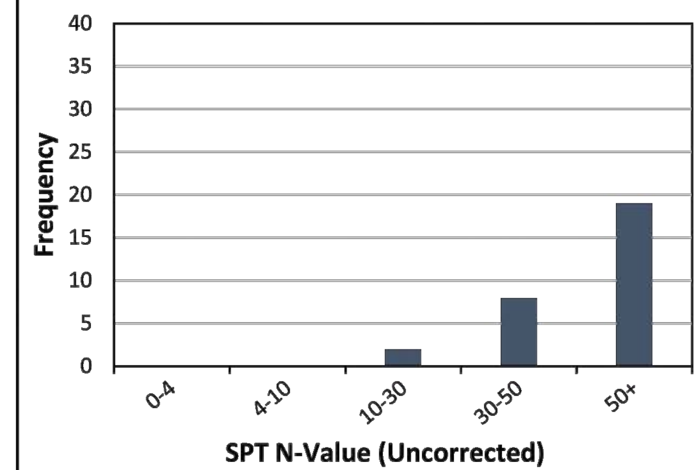
PARE

NBC PHASE III CSO PROGRAM
CONSOLIDATION CONDUIT IIIA-4
GEOTECHNICAL DESIGN SUMMARY REPORT
SOIL PROPERTY SUMMARY - GLACIOFLUVIAL

DATE
MAY, 2021

FIGURE
C-2





Note:
1: Histogram prepared using data from the following borings: B-4C, B-5, B-6A, B-7, B-8, BH-12, BH-14, B17-8, B17-39, B17-41, B17-45, B17-46A, and B21-1.

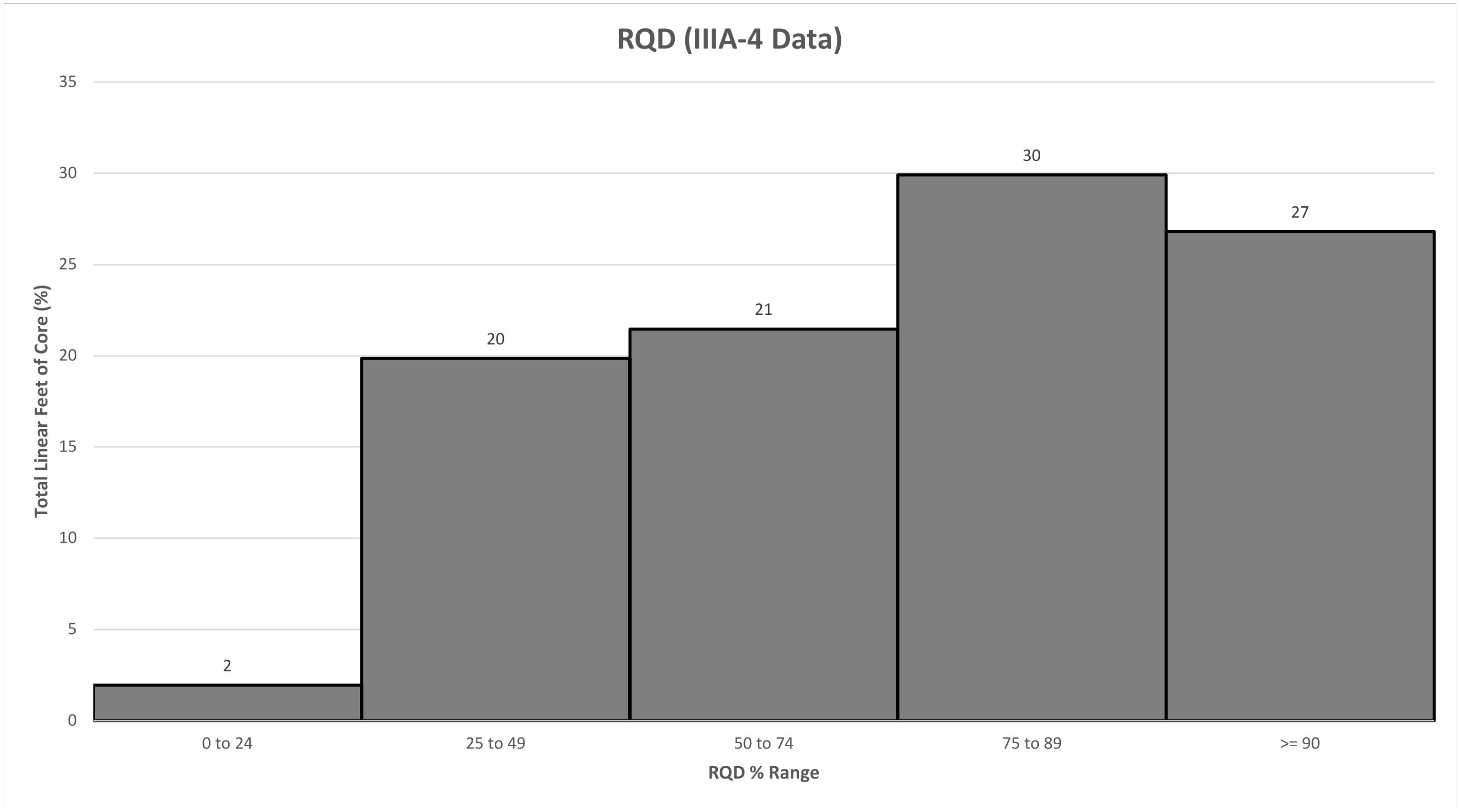
Note:
1: Plot and histograms prepared using data from the following borings: B-5, B-6A, B17-41, B17-45, and B17-46.

DRAFT

PROJECT 5980

McMILLEN JACOBS ASSOCIATES	NARRAGANSETT BAY COMMISSION PHASE III COMBINED SEWER OVERFLOW PROGRAM Stantec PARE	NBC PHASE III CSO PROGRAM CONSOLIDATION CONDUIT IIIA-4 GEOTECHNICAL DESIGN SUMMARY REPORT SOIL PROPERTY SUMMARY - GLACIAL TILL	DATE MAY, 2021 FIGURE C-3

APPENDIX D
RQD Histogram
(IIIA-5 Data)



Notes:

1: Histogram prepared using data from the following borings: B-4C, B-5, B-6A, B-8, B17-8, B17-39, B17-40, B17-41, B17-45, B17-46A, B21-1, B21-2, and B21-3.

2: For borings B17-8 and B17-39, only data from core runs C-1 through C-6 were used.

DRAFT

PROJECT 5980

McMILLEN JACOBS ASSOCIATES

NARRAGANSETT BAY COMMISSION
PHASE III COMBINED SEWER
OVERFLOW PROGRAM

Stantec PARE

NBC PHASE III CSO PROGRAM
CONSOLIDATION CONDUIT IIIA-4
GEOTECHNICAL DESIGN SUMMARY REPORT
ROCK QUALITY DESIGNATION (RQD) - HISTOGRAM

DATE MAY, 2021

FIGURE D-1

APPENDIX E
Ground Behavior Classification and Descriptions

Table E.1 Tunnelman’s Ground Classification

Table E.2 Rock Mass Descriptions

Table E.1 Tunnelman's Ground Classification

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move	Loess above water table; hard clay , marl, cemented sand and gravel when not highly overstressed.
Raveling	Slow Raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast depending upon degree of overstress
	Fast Raveling		
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.
Running	Cohesive Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (30-35 degrees.) When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak, cementation in any granular soil, may allow material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive running.
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances completely filling the tunnel in some cases.	Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.

(after Heuer, 1974)

Table E.2 Rock Mass Descriptions

Descriptive Terminology	Description and Associated Behavior
Intact	Rock contains neither joint nor hairline cracks. Therefore, if it breaks it breaks across intact solid rock. Due to rock damage resulting from blasting, spalls may drop from the roof several hours to days after blasting. This is known as spalling condition. Hard, intact rocks may also exhibit popping or slabbing conditions, involving the spontaneous, sudden, and violent detachment of rock slabs front he roof or sidewalls.
Stratified	Rock consists of individual strata with little or no resistance against separation along boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the spalling condition is common.
Moderately Jointed	Rock contains joints and hairline cracks, but the blocks between joints are locally grown together or intimately interlocked such that vertical walls do not require lateral support. In rock of this type, both the spalling and popping (or slabbing) conditions may be encountered.
Blocky and Steamy	Rock consists of chemically intact or nearly intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, walls may require lateral support.
Crushed	Rock consists of crushed, but chemically intact rock that has the character of a crusher run material. If most or all of the fragments are as small as fine sand grains and no remediation or support installation has been performed, crushed rock below the water table exhibits the ground behavioral response of water bearing sand.
Squeezing	Rock slowly advances into the tunnel excavation without perceptible volume increase. A pre-requisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.
Swelling	Rock advances into the tunnel excavation chiefly due to expansion. The capacity to swell appears to be limited to rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.

(after Terzaghi, 1946)