EXHIBIT 21

GEOTECHNICAL BASELINE REPORT

[*See attached*]

Virginia Department of Transportation

FINAL GEOTECHNICAL BASELINE REPORT

I-64 HAMPTON ROADS BRIDGE-TUNNEL EXPANSION PROJECT

UNDER THE
VIRGINIA PUBLIC-PRIVATE
TRANSPORTATION ACT OF 1995
(AS AMENDED)

STATE PROJECT NO. 0064-M06-032
FEDERAL PROJECT NO. [●]

ISSUANCE OF FINAL GBR:
NOVEMBER 28, 2018

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1.0 Introduction

The purpose of this Geotechnical Baseline Report (GBR) is to establish contractual baselines describing the anticipated ground conditions to be encountered during construction of the following elements of the I-64 Hampton Roads Bridge-Tunnel Expansion (HRBT) project: the Tunnel Improvements.

Capitalized terms used in this GBR but not otherwise defined herein shall have the meanings given to such terms in the Comprehensive Agreement.

1.1 Purpose and Scope

The baselines established in this report shall be used by the Design-Builders in proposal preparation and will also be used to evaluate potential Differing Tunnel Improvements Site Conditions encountered during construction. To evaluate design alternatives and approaches, construction means and methods, selecting equipment, and developing construction plans, Design-Builders are required to read and consider the GBR, the Geotechnical Data Report (GDR), and all other Request for Proposal (RFP) Documents in their entirety.

The geotechnical baseline conditions described in this report reflect the Department's judgment of anticipated subsurface conditions and ground behavior based on construction means and methods commonly employed to complete the work indicated by the RFP Documents. In establishing these baselines, the Department considered available geologic and geotechnical data and construction experience in similar ground conditions. Actual conditions encountered in the field are expected to be within the range of conditions discussed herein for purposes of determining whether such conditions constitute Differing Tunnel Improvements Site Conditions pursuant to Section 4.3.2 of the General Conditions of Contract. However, the geotechnical baseline conditions presented in this report are not a warranty that these conditions will, in fact, be encountered. Ground behavior will be influenced by, among other factors, the construction sequence and methods employed by the Design-Builder, as well as the Design-Builder's equipment, materials and workmanship. The issuance of the baselines shall not relieve the Design-Builder of its responsibility for the adequacy of the subsurface investigation program with respect to the design and alignment of the permanent works, the design and implementation of construction means and methods and safety precautions and programs.

1.2 Project Elements Covered by GBR

This baseline report pertains only to the tunnel, the boat section and cut-and-cover tunnel approach structures on the portal islands, and island expansions. This baseline report does not represent conditions for other project construction elements, including the Roadway and Bridge Improvements. The Department will not issue a GBR covering other project construction activities. The baseline report does not represent environmental baseline conditions such as contaminated or hazardous soils or other materials.

1.3 Sources of Geotechnical Information

Geotechnical Investigations have been performed in phases for this project, and the applicable data for the tunnel and tunnel approach structures is presented in the following two Geotechnical Data Reports, which are included as part of the Contract:

- "Preliminary Geotechnical Data Report – Hampton Roads Bridge Tunnel – North and South Islands", prepared by ECS Mid-Atlantic LLC and dated June 15, 2018.
- "Geotechnical Data Report – Marine", prepared by Jacobs Engineering and dated July 2018.

Additional geotechnical information from within and near the HRBT location can be found in the historic reports, records and published references listed in Section 8.0. The additional geotechnical information includes boring logs from the original HRBT tunnel construction in the 1950s and the construction of the second HRBT crossing in the 1970s. In addition to the geotechnical information, an archaeological survey report (AECOM, 2017) includes the results of geophysical investigations performed along the tunnel alignment. These reports, records and published references listed in Section 8.0 are provided as Disclosed Information to the Contract.

1.4 Stationing and Datum

All stations referred to in this report are based on the stationing established for the second bridge tunnel crossing in the 1970s, with the end of the northernmost tube at Station 862+57, and the end of the southernmost tube at Station 931+54, as shown on Drawing S-1 of the original plans for Contract T-2 (Commonwealth of Virginia, 1975).

The project datum for horizontal control is NAD83, and all elevations referred to in this report are based on NAVD88.

1.5 Disclosed Information and Scope Validation on Portal Islands

The Disclosed Information included in the Comprehensive Agreement provides supplemental data on subsurface conditions beyond the baselines described in this report, such as but not limited to, the presence of utilities, building foundations, and other site improvements. Should the Disclosed Information within the perimeters of the existing North and South Islands have a material impact on the Design-Builder's means and methods, Article 2.2 of Part 4, General Conditions of Contract relating to scope validation and identification of scope issues shall apply.

2.0 Project Description

The Project consists of improvements along approximately nine miles of the I-64 corridor between Settlers Landing Road in Hampton (Exit 267) and I-564 in Norfolk (Exit 276). On land, the Project will add a third lane to I-64 in each direction, with a roadway section sufficient to accommodate a part-time median shoulder lane.

The Project will include a new bridge-tunnel crossing of the Hampton Roads waterway, approximately 3.5 miles long and generally parallel to the existing bridge-tunnel crossings. The new crossing will provide four additional lanes of traffic for a total of eight lanes of capacity across the waterway.

The bridge-tunnel section of the project includes the following main components:

- A new approximate 3.5-mile-long bridge-tunnel parallel to the existing bridge-tunnels;
- The tunnel may be constructed as either an immersed tube tunnel (ITT) or a bored tunnel, designed to carry four lanes of eastbound traffic (the existing two-lane eastbound tunnel will be subsequently converted to carry westbound traffic, thereby providing four lanes of westbound traffic);
- Widening of the existing north and south portal islands to the west as required to accommodate the new portals and approach structures for the parallel tunnel;
- New trestle bridges connecting the mainland to each of the portal islands;
- Repairs and improvements to the existing trestle bridges; and
- Support systems, services and utilities for the new tunnel including but not limited to: tunnel ventilation buildings; tunnel ventilation systems and equipment; tunnel and approach lighting; power for lighting and other tunnel electrical needs; tunnel drainage; fire detection and alarm; communications (phones and radio system); fire protection systems; intrusion detection, supervisory control and data acquisition (SCADA) and information technology systems (ITS).

3.0 Previous Construction Experience

The existing bridge-tunnel was built in two separate projects: one in the 1950s and the second in the 1970s. Both projects were built using ITT method of construction. In addition to the original design and record drawings for both projects, several technical journal articles describe the construction of the original and second crossings in some detail; these articles are included in the list of references in Section 8.0 of this report. In addition, project documents included as Supplemental Information to the contract contain several hundred photos taken during construction of each project.

There are also several other similar transportation tunnel projects in the vicinity of the Hampton Roads Bridge-Tunnel Expansion with similar construction methodologies and subsurface conditions. Section 3.3 provides a list of those projects.

3.1 Hampton Roads Project

The original Hampton Roads Project was completed in 1957 and included construction of a two-lane bridge and tunnel connecting Norfolk, Virginia with Hampton, Virginia across the Hampton Roads waterway. The tunnel has a length of 7,479 feet between portals and consists of 23 steel immersed tube tunnel sections, each approximately 300 feet long. The tunnel sections were placed in a dredged trench between two man-made portal islands, while the tunnel approach structures on the islands were built by cut and cover methods. Trestle bridge connections from the portal islands to the mainland on both the north and south sides provide a total water crossing length of approximately 3.5 miles.

The North Island was founded on sands and silty sands which provided a suitable foundation. However, at the South Island location, up to 80 feet of normally consolidated clay and organic layers overlying sandy soils were present, which were expected to cause settlement or stability issues during island construction. To mitigate these issues, the soft layers were dredged out over the width of the tunnel approach structures and vent buildings on the South Island with 2H:1V slopes, and the excavation was then hydraulically backfilled with soils dredged from nearby areas. This hydraulic fill consisted of poorly graded, fine sand, which was densified by vibro-flotation. The perimeter slopes of the islands are protected by rip rap and armor stone.

The tunnel approach structures on the islands were built by cut and cover methods with steel sheeting or soldier pile and lagging with cross-lot struts used for temporary support of excavation. Dewatering systems were employed to build the cut and cover structures "in-the-dry." Ventilation buildings and other support facilities were constructed on the islands.

3.2 Second Hampton Roads Bridge-Tunnel Crossing

The second Hampton Roads Bridge-Tunnel Crossing was completed in 1975, and was built parallel to, and 250 feet to the west of, the original tunnel. Similar to the original tunnel, it consists of a 6,900-ft long two-lane steel immersed tube tunnel between the two portal islands, which were enlarged to accommodate the new tunnel and approach structures. Two additional trestle bridges connect the islands with the mainland.

Design and construction of the South Island expansion faced similar issues to the original construction. As opposed to over-excavating the underlying normally consolidated clay and organic layers, sand drains were installed through the clay and organics, and a surcharge was placed to pre-consolidate these layers prior to tunnel construction. An initial contract to build the South Island expansion included placement of fill up to 25 feet higher than the finished island elevation, as well as the installation of the sand drains and monitoring systems. The consolidation of the clay layers achieved 90% consolidation approximately 15 months after completion of the surcharge placement, with a maximum measured settlement under the highest surcharge of approximately 13 feet.

A second contract included the removal of the South Island surcharge, the construction of the North Island expansion, and the immersed tube tunnel. The tunnel approach structures were again built by cut and cover methods using temporary support of excavation systems including sheeting and soldier pile and lagging, however this time with tiebacks instead of cross-lot struts. Dewatering systems were again employed to build the cut and cover structures "in the dry". Follow-on contracts included construction of the ventilation buildings and other facilities.

3.3 Other Projects

Other similar tunnel projects in the vicinity of the Hampton Roads Bridge-Tunnel Expansion with similar subsurface conditions include the following:

- Midtown Tunnel;
- Downtown Tunnel;
- I-664 Monitor-Merrimack Memorial Bridge-Tunnel; and
- Chesapeake Bay Bridge and Tunnel (original Thimble Shoal Channel and Chesapeake Channel tunnels constructed in the 1960s, as well as the Parallel Thimble Shoal Tunnel, which is currently under construction).

Transportation tunnel projects completed in the vicinity to date were constructed exclusively using the immersed tube tunneling method. The proposed Parallel Thimble Shoal Tunnel for the Chesapeake Bay Bridge-Tunnel crossing, which is expected to commence tunneling operations in 2019, will be constructed using the bored tunneling method.

4.0 Ground Characterization

The project site is located within the Coastal Plain of Virginia where the James River meets the Chesapeake Bay. The Coastal Plain comprises a thick wedge of sediments underlain by Precambrian to early Mesozoic basement bedrock at depths greater than 2,000 feet. The majority of the sediments consist of late Jurassic and Cretaceous clays, sands and gravels which were transported from the Appalachian Mountains and deposited in the Atlantic Ocean basin.

The subsurface materials anticipated to be encountered within the tunnel alignment overlie these sediments, and comprise younger Tertiary and Quaternary layers that can generally be classified into the following four geological strata, from youngest to oldest:

- Island Artificial Fill (af)
- Alluvial Marine Sediments (Qc, Qf, Qo)
- Yorktown Formation (Tys, Tyf)
- Eastover Formation (Tye)

The Yorktown and Eastover formations are fossiliferous marine sands and clays that were deposited during interglacial highstands of the sea under similar conditions to the current site environment. The Eastover is thought to be late Miocene-aged, while the Yorktown is considered to be early Pliocene-aged. The Alluvial Marine Sediments are Quaternary aged, geologically unconsolidated layers that were likely deposited during the Holocene and late Pleistocene epochs.

Each stratum is discussed in the following sections of the report, and the baseline properties of these layers are presented in Tables 4.1 and 4.2. A baseline subsurface profile projected along the alignment of the second tunnel built in the 1970s is shown in Figure 4.1. In addition, in Appendix A, the baseline subsurface profile is shown on full size sheets with the boring location plan. This baseline profile is applicable for new tunnel alignments between the existing tunnels, and for alignments up to 500 feet to the west of the second 1970s tunnel alignment. Although the subsurface profile is drawn along the 1970s tunnel alignment, for clarity, the existing tunnel structure and associated backfill materials are not included in the profile. The limits of existing structures, and the associated dredging limits and backfill and scour protection materials are shown on the record drawings (Commonwealth of Virginia, 1956, 1975). Qualifying statements related to interpretation of the subsurface profile for different elements of the proposed construction are presented in Sections 5.1.1, 5.2.1 and 5.3.1 of this report.

In Appendix B, the Tunnelman's Classification System (after Heuer, 1974) is presented. This classification system was developed to describe soft ground behavior during tunnel excavation and each stratum that could be encountered in the bored tunnel alignment will be classified in accordance with this system. Although this classification system was created before the widespread use of pressurized-face TBMs and therefore assumes an open-face excavation method, it is nevertheless useful for conveying how soils are anticipated to behave if not properly controlled during tunneling or other excavation.

4.1 Island Artificial Fill (af)

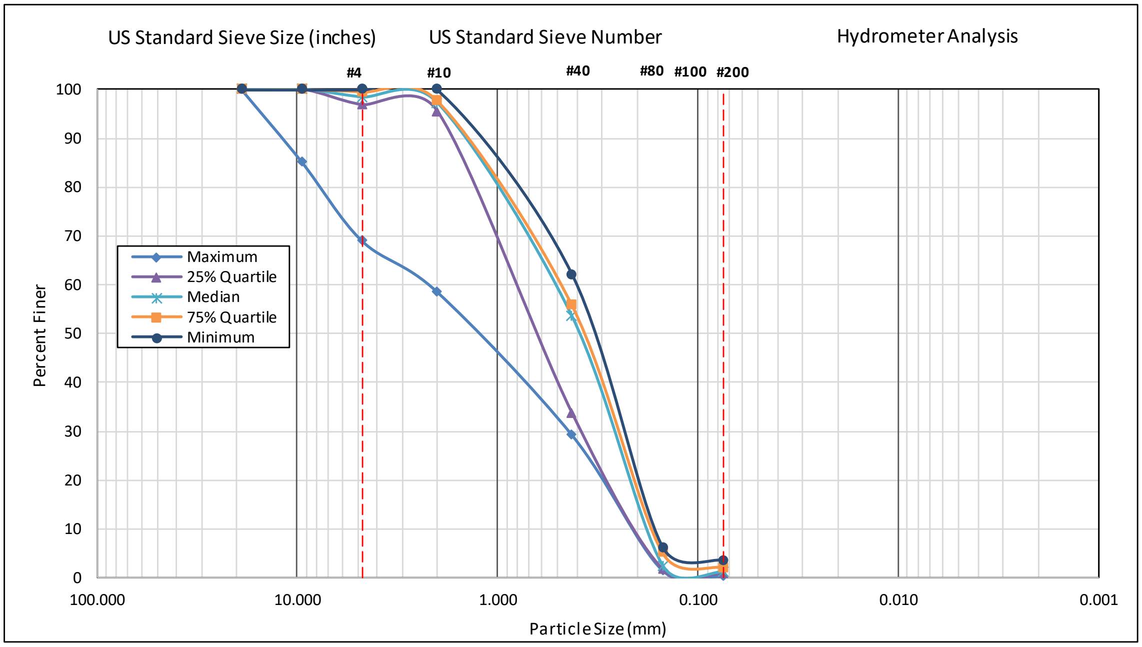
The North Island and the South Island were constructed using hydraulically placed fill to a finished surface elevation of approximately Elevation +11 feet. The hydraulic fill is predominantly composed of clean fine sands and typically classifies in the Unified Soil Classification System (USCS) as poorly graded sand (SP), silty sand (SM) and poorly graded sand with silt (SP-SM).

Based on the mudline elevation shown on the drawings for the original tunnel construction contracts, at the North Island, the artificial fill thickness varies between 20 and 25 feet, with the base of the layer between Elevation -9 and -14 feet. At the South Island, the artificial fill thickness to the west of the existing tunnels appears to vary between 33 and 48 feet below ground surface with the base of the layer encountered between Elevations -22 and -37 feet. Although not reflected by the most recent site investigations as documented in the GDRs, the fill thickness is greater at locations beneath the original tunnels and tunnel approach structures constructed in the 1950s, with the over-excavation and replacement of the soft clay layers employed at that time resulting in the fill extending to the top of the Yorktown formation, up to Elevation -105, as shown in the 1950s record drawings and documented by Steuerman and Murphy (1957).

Over 13 feet of settlement was measured during construction of the South Island expansion in the 1970s so it is reasonable that the fill thickness would be greater than for the North Island. Based on analysis of the N60 SPT test results, the hydraulic fill is very loose to medium dense. Grain size analyses were performed on several samples, and the range of particle size distribution observed in the test samples from the recent geotechnical investigations is shown in Figure 4.2.

Baseline index and shear strength properties for this layer are provided in Table 4.1 and the range for the hydraulic conductivity is included in Table 4.2. When encountered above the groundwater level, this layer when unsupported and unimproved would classify in the Tunnelman's Classification System as "Running", while below groundwater would classify as "Flowing".

Armor stone, rip rap, and crushed rock filter layers were placed around the island perimeters to contain the hydraulic fill placement, and to provide scour protection. When the islands were expanded in the 1970s construction, it is understood that the existing scour protection placed in the 1950s was not removed but rather was buried under the new hydraulic fill; for example, a note on Sheet 7 (of 127) of the record drawing set for Contract T-2 (Commonwealth of Virginia, 1975) states: "Existing rip rap and bedding along the entire west side of North Island to be removed to El +7.0."



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Figure 4.2 Range of Particle Size Distribution - Island Artificial Fill (af)

4.2 Alluvial Marine Sediments

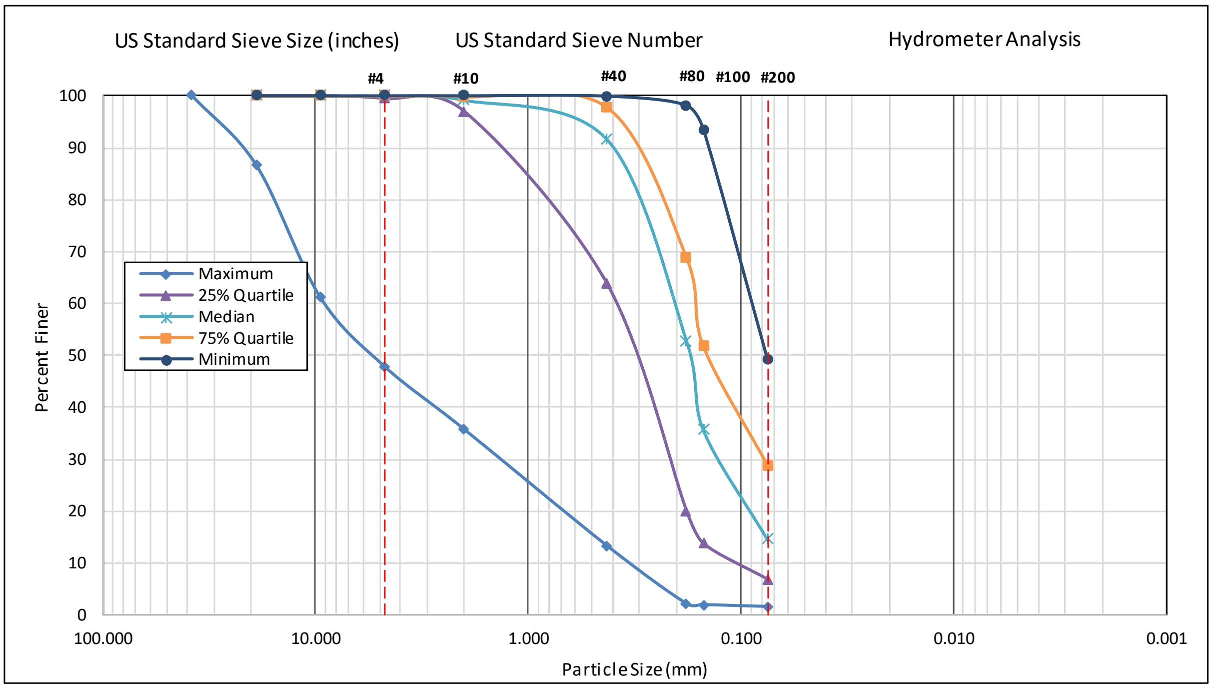
Below the island fill, and below the mudline outside of the island footprints, a layer of alluvial marine sediments is present, and consists of interlayered sands, inorganic and organic silts and clays, and peat. Based on the properties of the materials in this layer, it can be sub-divided into the following three sub-layers:

- Alluvial Marine Sediments – Coarse-Grained (Qc)
- Alluvial Marine Sediments – Fine-Grained (Qf)
- Alluvial Marine Sediments – Organic/Fat Clay (Qo)

The distribution of these three sub-layers is quite variable, particularly below the South Island. The sub-layers may be individually present or present in any combination. Furthermore, the relative order of appearance of the sub-layers, vertically, is also variable.

4.2.1 Alluvial Marine Sediments – Coarse-Grained (Qc)

The coarse-grained alluvial marine sediments were encountered below the fill of both islands and below the mudline over most of the tunnel alignment. Below the North Island and north of Station 878+00, the fine-grained and organic/fat clay marine sediments were not encountered, and the coarse-grained deposits extend to the top of the Yorktown formation, which was encountered between Elevations -85 and -32 feet. South of Station 878+00, the coarse-grained sediments vary between 5 and 30 feet thick and are typically encountered above the fine-grained alluvial marine sediment layers, but in some locations granular layers and lenses are also encountered within and below the cohesive alluvial deposits. The coarse-grained alluvial marine sediments are very loose to dense, and generally classify in the USCS as poorly graded sand (SP), silty sand (SM), and clayey sand (SC). Grain size analyses were performed on several samples, and the range of particle size distribution observed in the test samples is shown in Figure 4.3.



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Figure 4.3 Range of Particle Size Distribution-Alluvial Marine Sediments–Coarse-Grained (Qc)

Baseline index and shear strength properties for the Qc layer are provided in Table 4.1 and the baseline hydraulic conductivity range is listed in Table 4.2. If left unsupported or not modified with ground improvement, depending on the fines content and water pressure, this layer would classify as “Fast Raveling” or “Flowing” in the Tunnelman’s Classification System, as detailed in Appendix B.

4.2.2 Alluvial Marine Sediments – Fine-grained (Qf)

From approximately Station 878+00 to 945+00, the fine-grained marine sediments were encountered below the coarse-grained marine sediments. The fine-grained marine sediments predominantly consist of lean clay (CL) and fat clay (CH). A plasticity chart showing the liquid limit plotted versus the plasticity index for laboratory tests on this layer is shown in Figure 4.4. The thickness of the Qf layer is quite variable, and it can be difficult to visually distinguish between the underlying organic/fat clay sub-layer (i.e. the Qo layer described below). However, the distinction between the two sub-layers is clear from laboratory test data.

Below the channel between Stations 878+00 and 928+00, the top of the fine-grained Alluvial Marine Sediment layer was encountered below the coarse-grained marine sediments between Elevations -45 and -90 feet and varied between a few feet to over 35 feet thick. Below the South Island, between Stations 928+00 and 945+00, the top of the fine-grained Alluvial Marine Sediment layer was encountered between Elevation -25 and -50 feet, and was up to 30 feet thick.

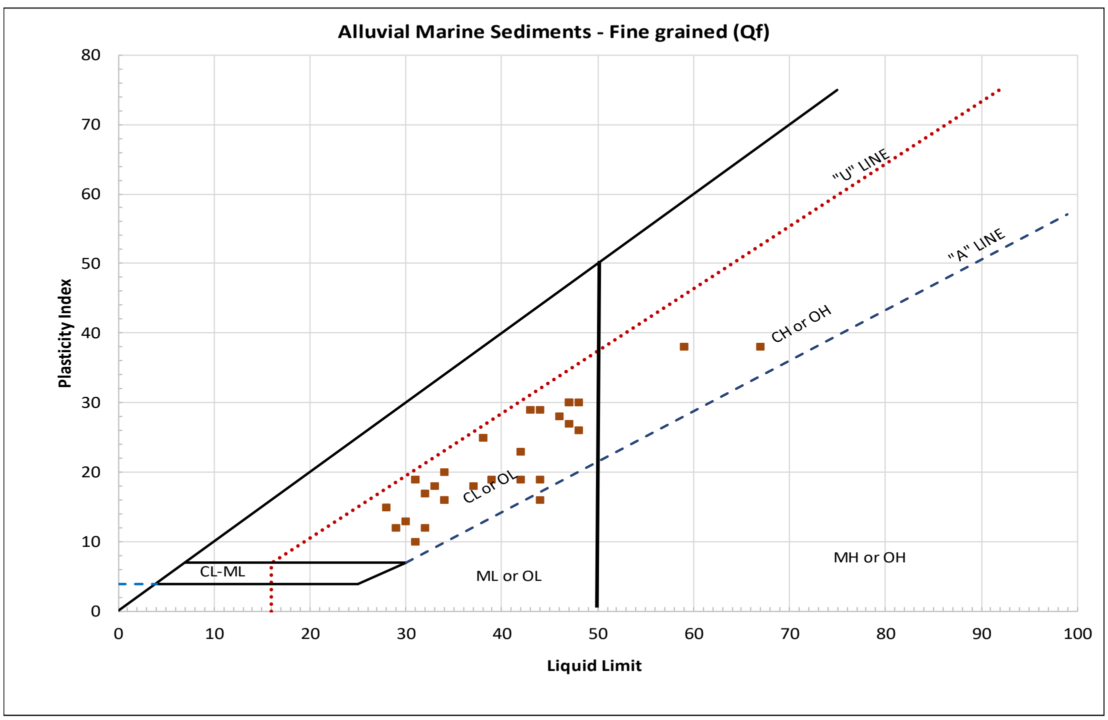


Figure 4.4 Plasticity Chart for Alluvial Marine Sediments – Fine-grained (Qf)

Extensive in situ shear strength testing was performed during the geotechnical investigation programs, including CPT probes, field vane shear tests, and flat plate dilatometer tests. In addition, an extensive laboratory testing program was performed. The results of these tests were highly variable. Figure 4.5 shows the measured undrained shear strength from the various tests versus elevation. The plot has been split between the results from the marine geotechnical program performed by Jacobs Engineering (Figure 4.5a) and the South Island geotechnical investigation by ECS Mid-Atlantic LLC (Figure 4.5b). The data in Figure 4.5 includes results from both the Qf sub-layer and the Qo sub-layer.

On the South Island, between Stations 928+00 and 944+00, the 1970s tunnel construction placed a surcharge up to 26 feet over the current island elevation, which pre-consolidated this layer and increased the shear strength below the island. The increase in strength is apparent from a comparison of the field and laboratory test results between the two GDRs. Figure 4.5 shows the overall range of the test results from the marine and South Island investigations, which demonstrates the increased shear strength below the South Island. Typical ranges of shear strength are also highlighted, which shows an increase in strength with depth. Two baseline ranges of undrained shear strength are provided in Table 4.1; one for locations outside the South Island surcharge from the 1970s construction, and the second for pre-consolidated layers below the South Island surcharge.

The consolidation properties of the Qf sub-layer were investigated by performing pore pressure dissipation tests during the CPT probing, and by an extensive laboratory test program consisting of one-dimensional consolidations tests. In addition, the South Island pre-consolidation works from the 1970s were instrumented and well-documented (Kuesel et al, 1973), which effectively provides a field load test of in situ compressibility properties.

Baseline index and shear strength properties for the Qf sub-layer are provided in Table 4.1, and the baseline range for parameters relating to compressibility of the layer is presented in Table 4.2. This layer would classify as “Squeezing” in the Tunnelman’s Classification System.

4.2.3 Alluvial Marine Sediments – Organic/Fat Clay (Qo)

Below the channel between Stations 878+00 and 906+00, the Alluvial Marine Sediment-Organic/Fat Clay layer was encountered between Elevations -60 and -110 feet, and generally varied between 10 and 20 feet thick. Between Stations 906+00 and 928+00 the thickness increased to approximately 50 feet. Below the northern section of the South Island, between Stations 928+00 and 932+00, the layer is as much as 60 feet thick and between Station 932+00 and 942+00 decreases to about 20 feet thick.

The index properties and classification of this layer were quite variable; it was variously classified as fat clay (CH), organic clay (OL), fat organic clay (OH) and peat (PT). A plasticity chart showing the liquid limit plotted versus the plasticity index for laboratory tests on this layer is shown in Figure 4.6.

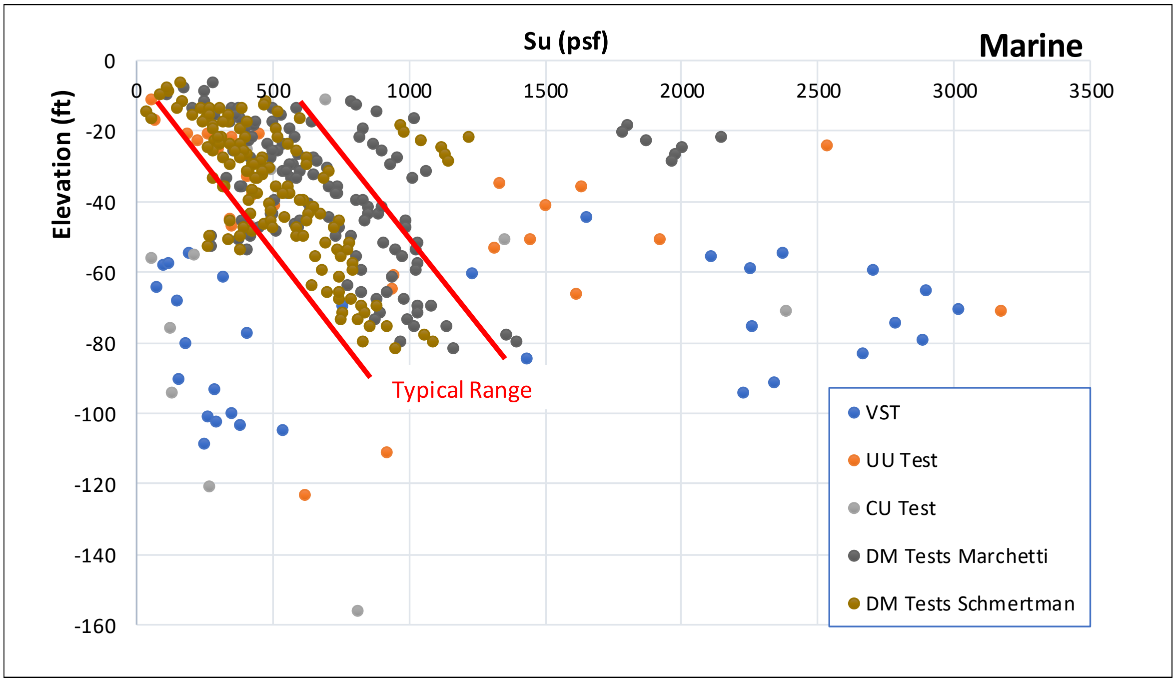


Figure 4.5a Outside South Island Surcharge Area from 1970s Construction

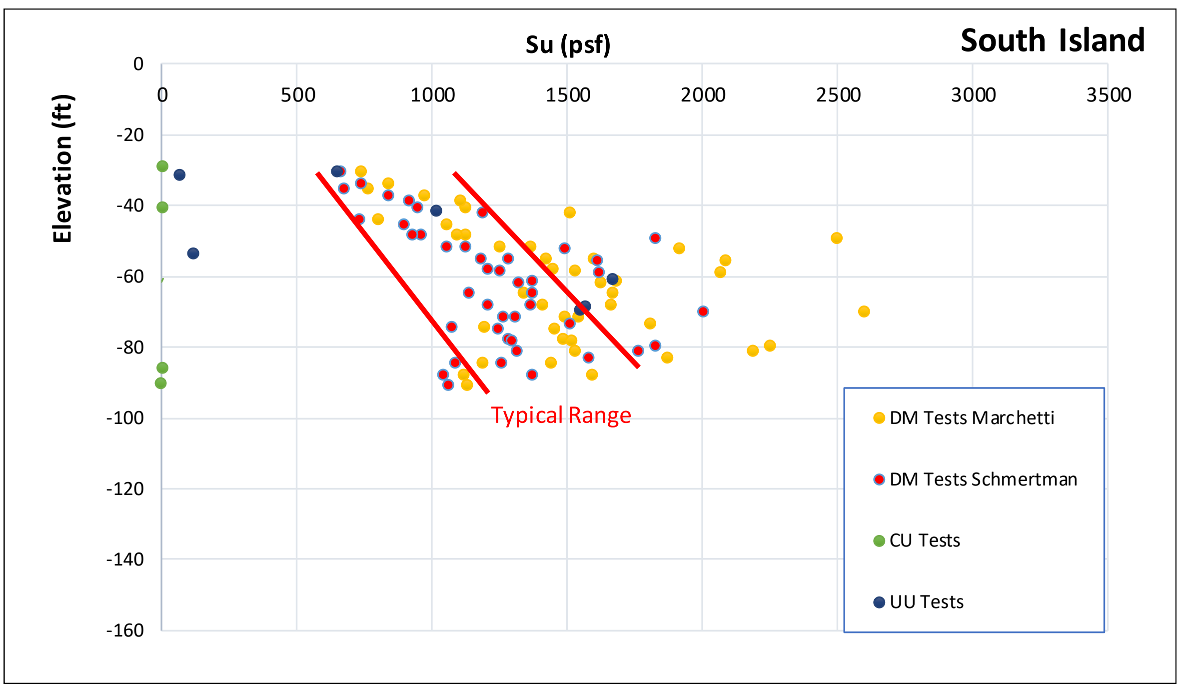


Figure 4.5b: Below South Island Surcharge Area from 1970s Construction

Figure 4.5 Undrained Shear Strength v. Elevation, Fine-Grained Alluvial Marine Sediments

(Note: The test results shown include both Qf and Qo sub-layers)

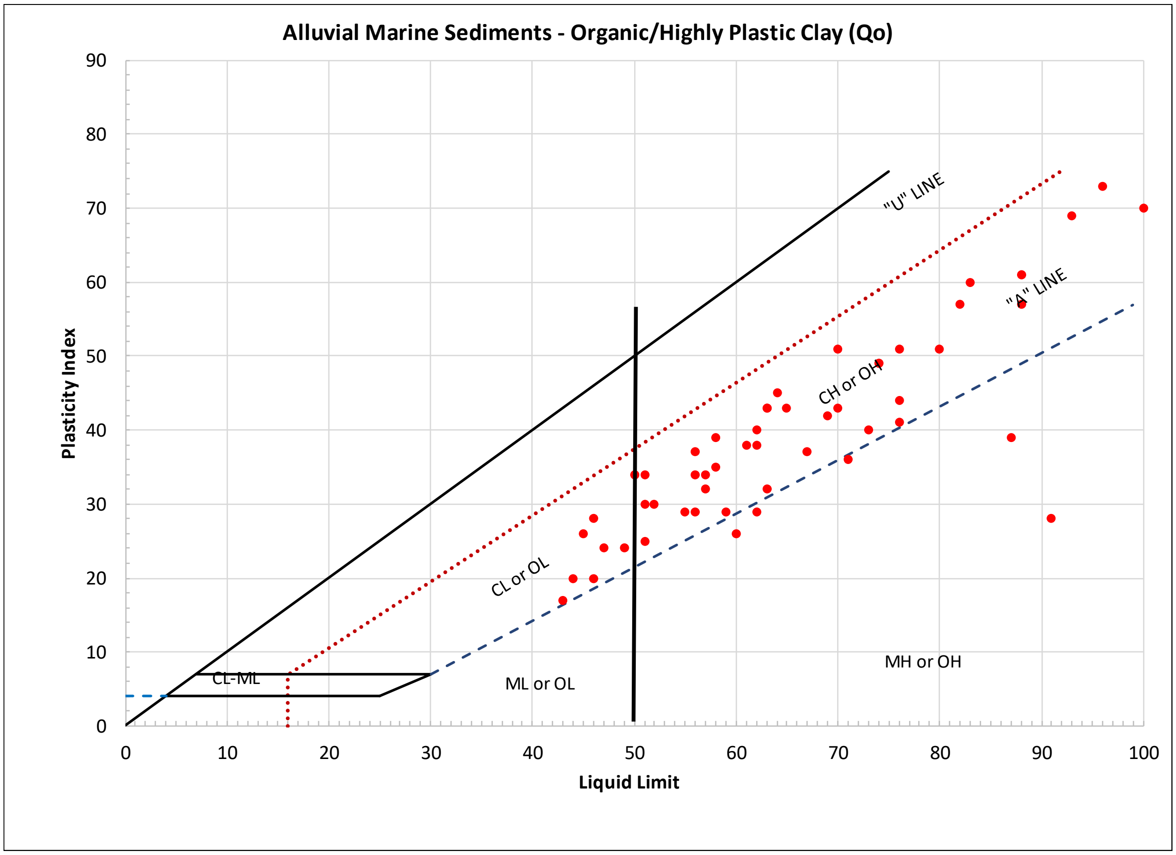


Figure 4.6 Plasticity Chart for Alluvial Marine Sediments – Organic/Fat Clay (Qo)

Extensive in situ and laboratory shear strength testing was also performed for this layer during the geotechnical investigation programs, and the measured undrained shear strength versus elevation from the various tests is shown in Figure 4.5. As discussed in the previous section of the report, the plot has been split between the results from the marine geotechnical program (Figure 4.5a) and the geotechnical investigation from the South Island (Figure 4.5b).

The consolidation properties of the Qo sub-layer were also investigated by performing pore-pressure dissipation tests during the CPT probing, and by an extensive laboratory test program consisting of one-dimensional consolidations tests. As discussed in the previous section, the South Island pre-consolidation works from the 1970s were instrumented and well-documented (Kuesel et al, 1973), which effectively provides a field load test of in situ compressibility properties.

Baseline index and shear strength properties for the Qo layer are provided in Table 4.1, and the baseline range for parameters relating to compressibility of the layer is presented in Table 4.2. This layer would classify as “Squeezing” in the Tunnelman’s Classification System, as detailed in Appendix B.

4.3 Yorktown Formation (Tys, Tyf)

The Yorktown Formation lies beneath the Alluvial Marine Sediments over the entire tunnel alignment and can generally be distinguished by the change to a green to blue gray color. The top of the formation was encountered between Elevations -32 and -52 feet below the North Island, between Elevations -60 and -140 feet between the islands, and between Elevations -40 and -120 feet below the South Island. The Yorktown formation is predominantly a granular layer, but throughout, there are numerous interbedded layers of fine-grained deposits; it can therefore be subdivided into the following two sub-layers:

- Yorktown Formation – Predominantly Coarse-Grained (Tys)
- Yorktown Formation – Predominantly Fine-Grained (Tyf)

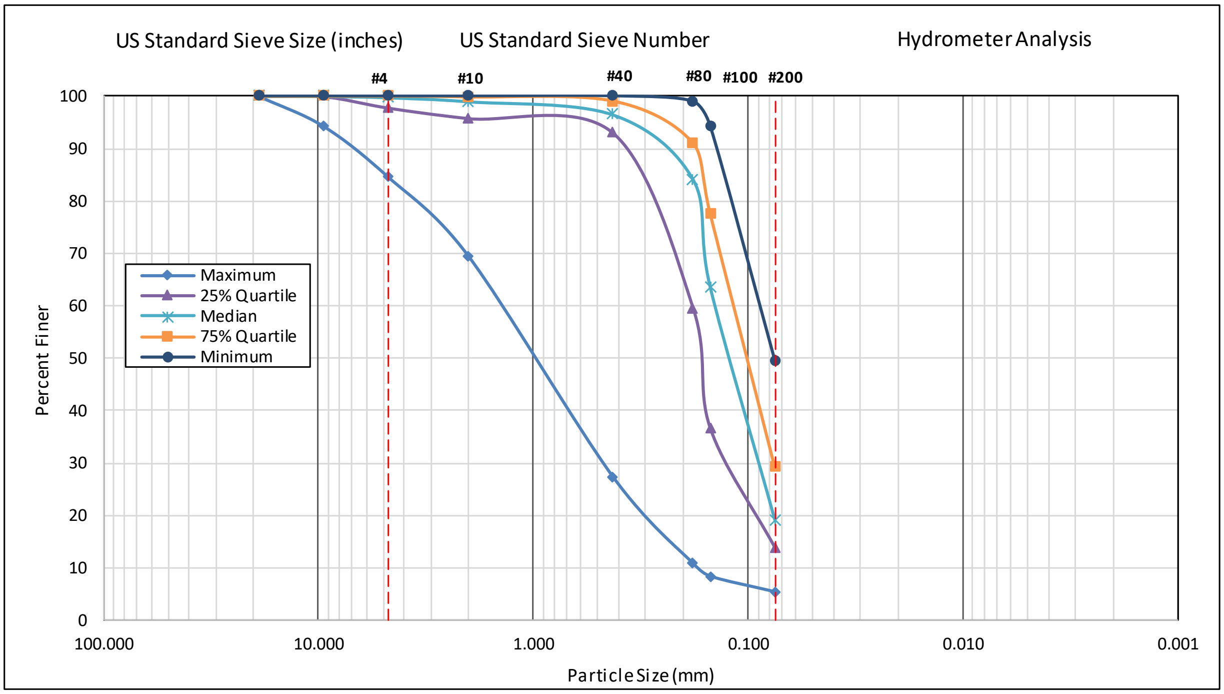
4.3.1 Yorktown Formation – Coarse-Grained (Tys)

The coarse-grained Yorktown formation layer generally consists of green to blue-gray, over-consolidated, silty fine sand with varying amounts of fine-grained fractions and marine shell fragments, typically classified as silty sand (SM) or clayey sand (SC), and sometimes classified as poorly-graded sand (SP), well-graded sand (SW), poorly-graded gravel (GP) or well-graded gravel (GW). Based on analysis of the N60 SPT test results, the layer relative density varies from very loose to very dense.

In several zones, the SPT test recorded “weight of rod” resistance at depths of over 100 feet below the mudline, which was unexpected within this over-consolidated layer; these SPT results may not be truly representative of the in situ relative density and may have been caused by sample disturbance in the difficult drilling conditions involving fine sand layers under high hydrostatic pressures.

Grain size analyses were performed on several samples, and the range of particle size distribution observed in the test samples is shown in Figure 4.7.

Baseline index and shear strength properties for the Tys layer are provided in Table 4.1 and the baseline hydraulic conductivity range is listed in Table 4.2. When unsupported and unimproved, and depending on the fines content and water pressure, this layer would classify as “Fast Raveling” or “Flowing” in the Tunnelman’s Classification System, as detailed in Appendix B.



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Figure 4.7 Range of Particle Size Distribution - Yorktown Formation – Coarse-Grained (Tys)

4.3.2 Yorktown Formation – Predominantly Fine-grained (Tyf)

Fine-grained layers comprising very stiff to hard sandy, shelly clays, were encountered at various elevations within the Yorktown formation. These layers are more prevalent towards the base of the formation, between Elevations -150 and -170 feet, where a 5 to 20 feet thick layer was observed along most of the tunnel alignment. This layer generally classifies as either fat or lean clay (CH or CL) with sand. A plasticity chart showing the liquid limit plotted versus the plasticity index for laboratory tests on this layer is shown in Figure 4.8.

Baseline index and shear strength properties for the Tyf layer are provided in Table 4.1. This layer would classify as “Squeezing” in the Tunnelman’s Classification System, as detailed in Appendix B.

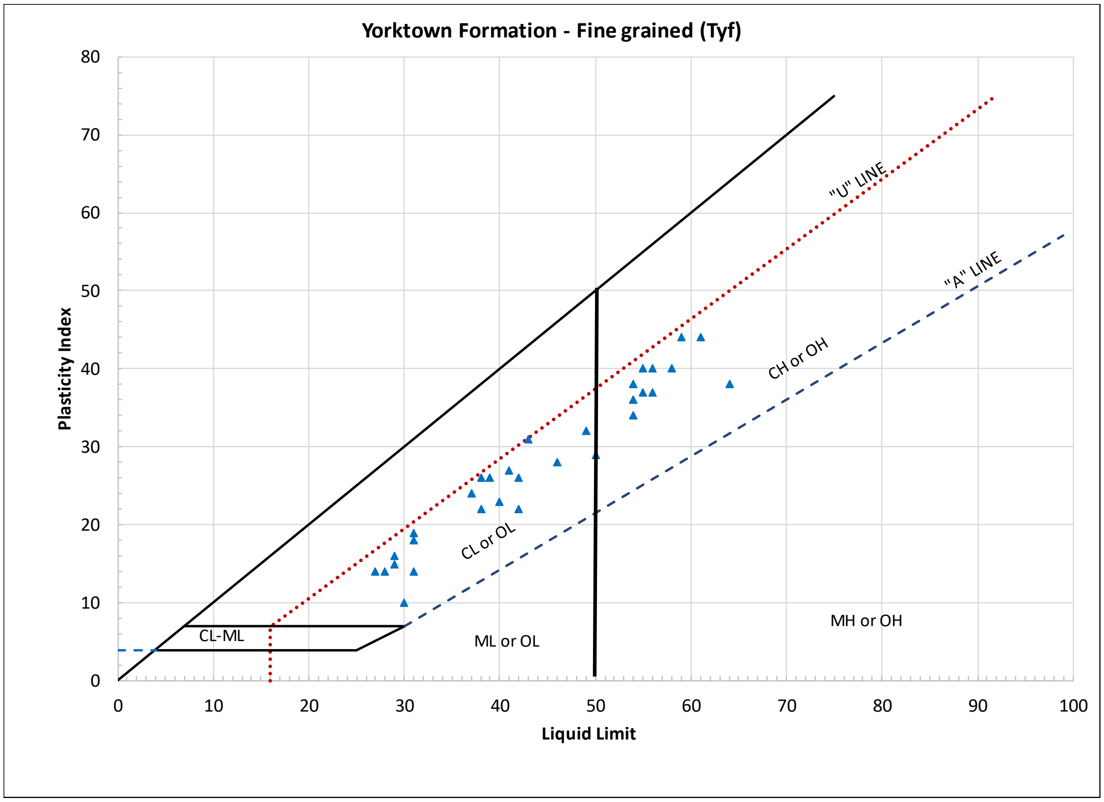


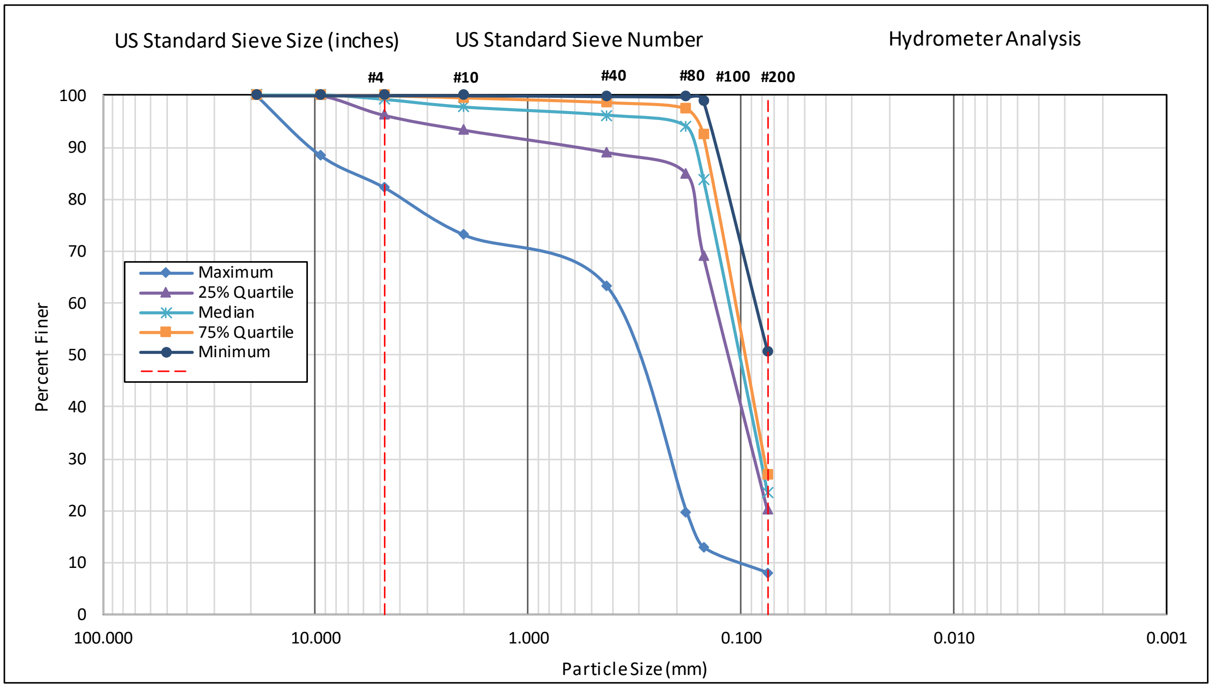
Figure 4.8 Plasticity Chart for Yorktown Formation – Predominantly Fine-Grained (Tyf)

4.4 Eastover Formation (Tye)

The Eastover formation underlies the Yorktown formation along the entire tunnel alignment and was first encountered at elevations varying between Elevation -158 feet to Elevation -175. The formation extended to Elevation -275 feet where the deeper borings were terminated. The Eastover formation generally consists of green-gray, over-consolidated, fine to coarse sand with varying amounts of fine-grained material and marine shell fragments. The formation is comprised of poorly graded sand (SP), silty sand (SM), clayey sand (SC) and poorly graded sand with clay (SP-SC). A dense marker bed consisting of cemented shell fragments and coarse sand was encountered overlying the Eastover formation. Based on analysis of the N60 SPT test results, the layer's relative density varies from very loose to very dense.

In several zones, the SPT test recorded "weight of rod" resistance at depths of over 100 feet below the mudline, which was unexpected within this over-consolidated layer; these SPT results may not be truly representative of the in situ relative density and may have been caused by sample disturbance in the relatively difficult drilling conditions involving fine sand layers under high hydrostatic pressures. Some SPT samples also encountered refusal within this layer which is considered to be representative of denser, more cemented zones.

Grain size analyses were performed on several samples, and the range of particle size distribution observed in the test samples is shown in Figure 4.9.



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Figure 4.9 Range of Particle Size Distribution - Eastover Formation (Tye)

Baseline index and shear strength properties for the Tye layer are provided in Table 4.1. This layer when unsupported and unimproved would classify as “Flowing” in the Tunnelman’s Classification System, as detailed in Appendix B.

4.5 Groundwater

Groundwater levels at the site are based on tidal fluctuations and are expected to vary from Elevation +4 feet at high tide to Elevation -1 feet at low tide.

Artesian groundwater conditions were not observed during the project geotechnical investigation but have been reported present in the Yorktown and Eastover formations in the technical literature (e.g. Hamilton and Larson, 1988).

For baseline purposes, the Design-Builder shall consider that the groundwater pressure in all subsurface layers may vary by ±5 feet from the hydrostatic pressure due to the ambient sea level in the shipping channel.

Stratum / Sub-Layer	Typical USCS Classification	Total Unit Weight (pcf)	Water Content (%)	Liquid Limit (%)	Plasticity Index	Effective Friction Angle (°)	Undrained Shear Strength (psf)
		Baseline Range	Baseline Range	Baseline Range	Baseline Range	Baseline Range	Baseline Range
Island Artificial Fill (af)(2)	SP, SM, SP-SM	105-125	N/A	Non-Plastic	N/A	30-34	N/A
Alluvial Marine Sediments – Coarse-Grained (Qc)	SP, SM, SC	105-125	N/A	Non-Plastic	N/A	30-40	N/A
Alluvial Marine Sediments – Fine-Grained (Qf)	CL, CH	100-120	20-60	30-60	15-30	N/A	225-750(3) 525-1400
Alluvial Marine Sediments - Organic/Fat Clay (Qo)	OL, OH, CH, PT	90-110	35-75	30-110	25-60	N/A	125-650 (3) 250-1500
Yorktown Formation – Coarse-Grained (Tys)	SM, SC	115-130	N/A	Non-Plastic	N/A	30-42	N/A(4)
Yorktown Formation – Fine-Grained (Tyf)	CL, CH	110-130	25-40	30-55	15-40	N/A	750-6000(4)
Eastover Formation (Tye)	SP, SM, SP-SC	120-130	N/A	Non-Plastic	N/A	30-42	N/A(4)

Notes:

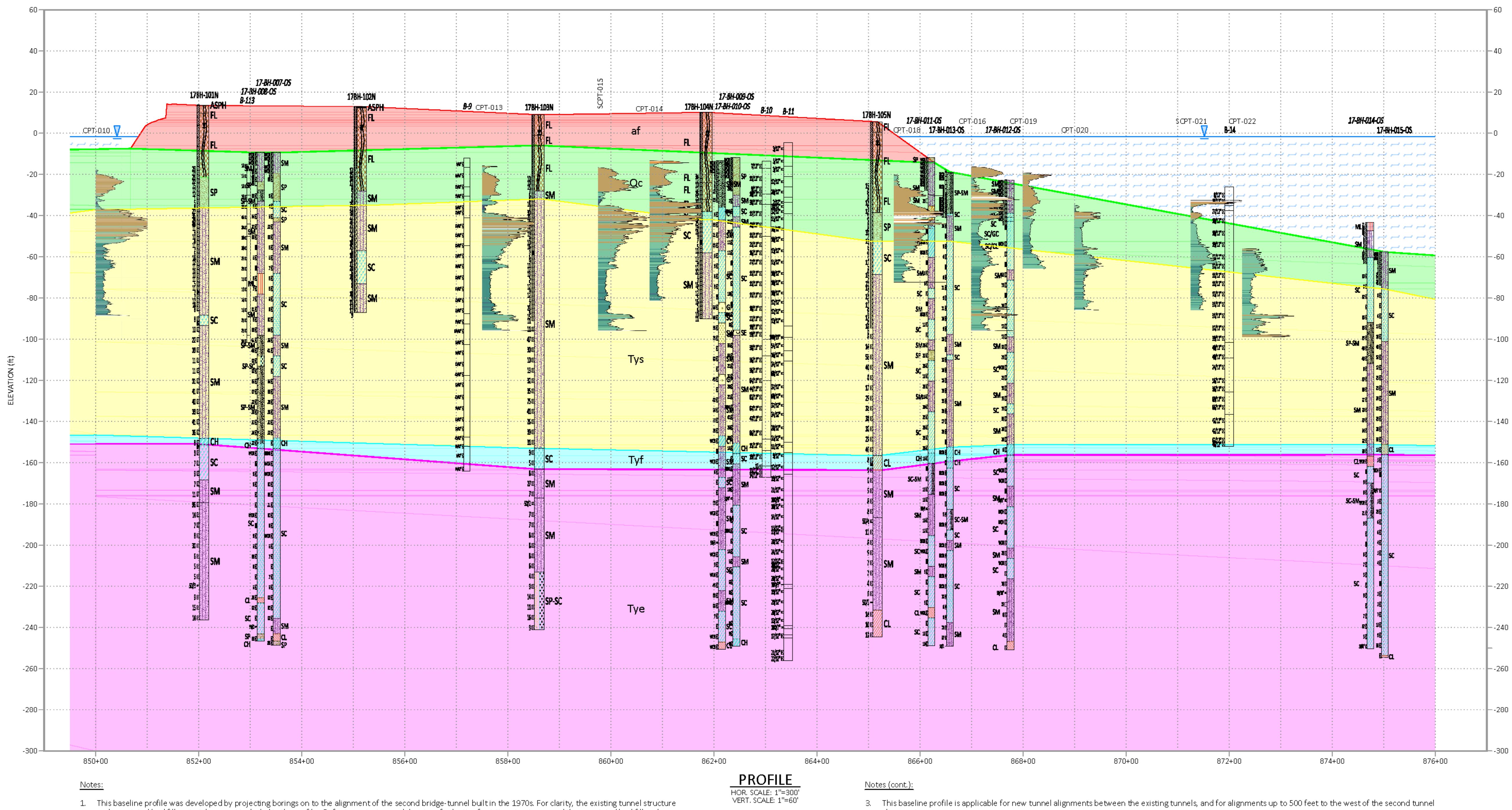
1. The baseline values shown represent the range of material properties that the Design-Builder must consider when selecting construction equipment and materials, means and methods, and evaluating the potential impacts of construction on existing structures. Soil properties for design of the temporary and permanent project elements shall be derived by the Design-Builder in accordance with Section 24, Geotechnical - Islands and Tunnels of the Technical Requirements of the Contract.
2. Baselines for artificial fill consider the hydraulically placed sand only; the baselines do not consider the oversized material (i.e. containment dikes material, scour protection material, etc.).
3. The first range of undrained shear strength is for locations outside the 1970s South Island surcharge area, while the second range is for pre-consolidated layers below the South Island surcharge. The extents of the South Island surcharge area shall be defined to be as shown on Drawing No S-6 of Contract T-2 of the 1970s construction record drawings (Commonwealth of Virginia, 1975).
4. Cemented sand and shell layers up to 5 feet thick within the Yorktown and Eastover Formations may have unconfined compressive strength up to 1000 psi.
5. For the granular layers (af, Qc, Tys, and Tye), the baseline range of the layer's particle size distribution shall be between the maximum and minimum limits shown on Figures 4.2, 4.3, 4.7 and 4.9.
6. For table entries designated as "N/A", a baseline range is not provided.

Table 4.1 – Baseline Values for Soil Index Properties and Shear Strength

Stratum / Sub-Layer	Typical USCS Classification	Hydraulic Conductivity (cm/sec)	In Situ Void Ratio	Over Consolidation Ratio (OCR)	Compression Index (C_c)	Recompression Index (C_r)	Coefficient of Secondary Compression (C_α)	Coefficient of Consolidation[3] (C_v) (cm^2/sec)
		Baseline Range	Baseline Range	Baseline Range	Baseline Range	Baseline Range	Baseline Range	Baseline Range
Island Artificial Fill (af)	SP, SM, SP-SM	10^{-3}- 10^{-1}	N/A	N/A	N/A	N/A	N/A	N/A
Alluvial Marine Sediments - Coarse Grained (Qc)	SP, SM, SC	10^{-3} – 10^{-1}	N/A	N/A	N/A	N/A	N/A	N/A
Alluvial Marine Sediments – Fine-Grained (Qf)	CL, CH	10^{-7} to 10^{-5}	0.6-1.0	1.0 – 1.5[2] 1.0 – 2.0	0.30-0.50	0.02-0.05	0.001-0.005	0.003-0.030[3] 0.001-0.020
Alluvial Marine Sediments - Organic/Fat Clay (Qo)	OL, OH, CH, PT	10^{-7} to 10^{-5}	1.3-2.0	1.0 – 1.5[2] 1.0 – 2.0	0.60-1.00	0.05-0.16	0.001-0.010	0.001-0.015[3] 0.0005-0.010
Yorktown Formation – Coarse-Grained (Tys)	SM, SC	10^{-3} – 10^{-1}	N/A	N/A	N/A	N/A	N/A	N/A
Yorktown Formation - Fine-Grained (Tyf)	CL, CH	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Eastover Formation (Tye)	SP, SM, SP-SC	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:
1. The baseline values shown represent the range of material properties that the Design-Builder must consider when selecting construction equipment and materials, means and methods, and evaluating the potential impacts of construction on existing structures. Soil properties for design of the temporary and permanent project elements shall be derived by the Design-Builder in accordance with Section 24, Geotechnical – Islands and Tunnels of the Technical Requirements of the Contract.
2. The first range of OCR is for locations outside the 1970s South Island surcharge area, while the second range is for pre-consolidated layers below the South Island surcharge area. The extents of the South Island surcharge area shall be defined to be as shown on Drawing No S-6 of Contract T-2 of the 1970s construction record drawings (Commonwealth of Virginia, 1975).
3. Horizontal and vertical coefficient of consolidation shall be considered equal. The first range of C_v is representative for virgin compression, while the second range is for recompression.
4. For table entries designated as “N/A”, a baseline range is not provided.

Table 4.2 – Baseline Values for Soil Permeability and Compressibility



Notes:

1. This baseline profile was developed by projecting borings on to the alignment of the second bridge-tunnel built in the 1970s. For clarity, the existing tunnel structure and associated backfill materials are not included in the profile. Refer to project record drawings for limits of existing structures, and the associated backfill and scour protection materials.
2. For clarity, some CPT's are not shown and some have been moved slightly from their projected location.

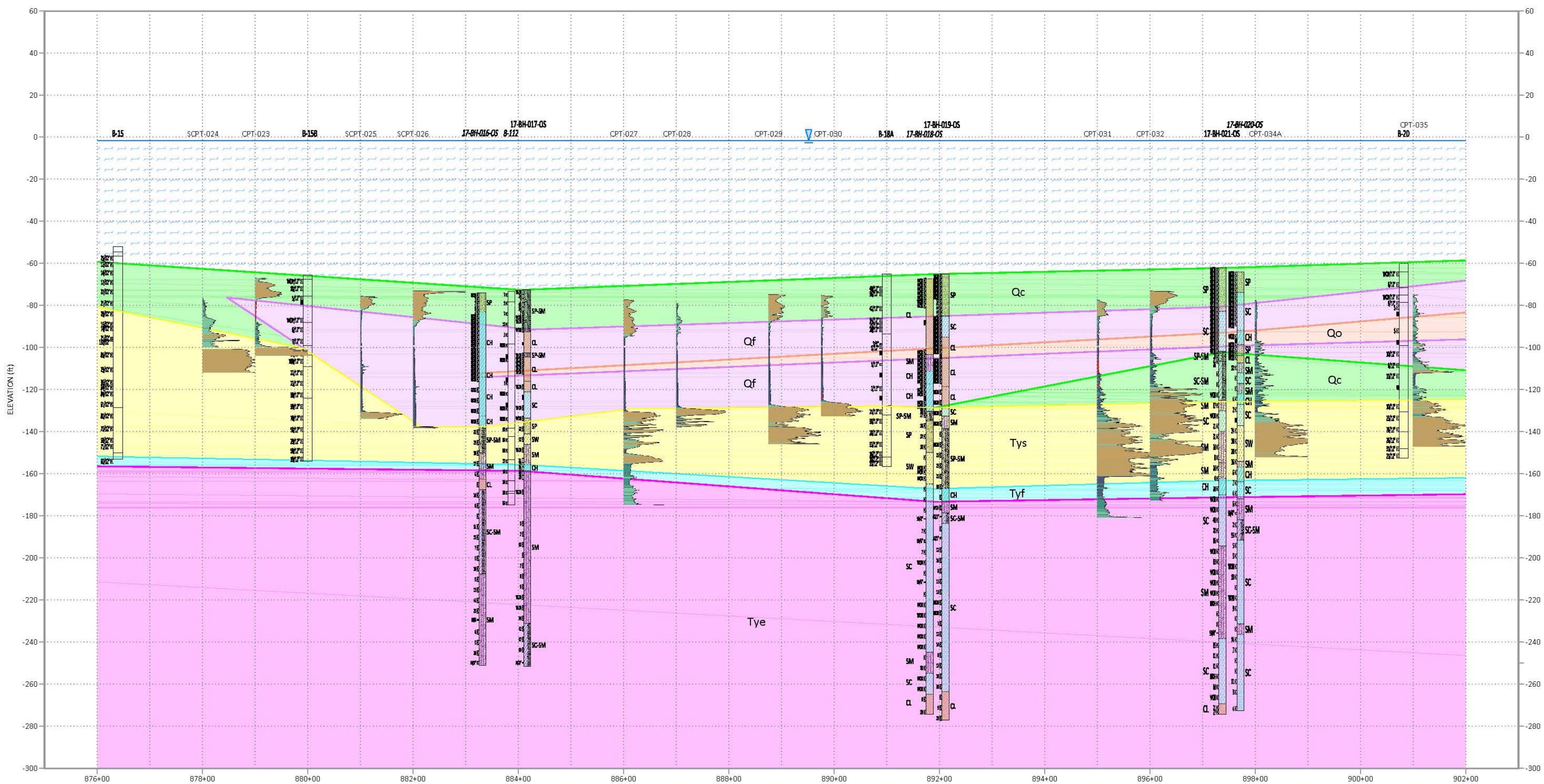
Notes (cont.):

3. This baseline profile is applicable for new tunnel alignments between the existing tunnels, and for alignments up to 500 feet to the west of the second tunnel alignment.
4. Qualifying statements related to interpretation of the subsurface profile for different elements of the proposed construction are presented in Sections 5.1.1, 5.2.1 and 5.3.1 of this report.

Figure 4.1 – Baseline Subsurface Profile 1/4

Legend

- Tye - Eastover Formation; Predominantly Coarse-grained (SC, SM, SP, SC-SM, SP-SC with minor lenses of CL)
- Tyf - Yorktown Formation; Predominantly Fine-grained (CH, mainly fat clay with some lenses of CL, MH and SC)
- Tys - Yorktown Formation; Predominantly Coarse-grained (SM, SP-SM, SP, SW, GP, SC, SW-SM, SC-SM)
- Qc - Alluvial Marine Sediments; Predominantly Coarse-grained (SP, SC, SM, SP-SM)
- Qf - Alluvial Marine Sediments; Predominantly Fine-grained (CL, CH)
- Qo - Alluvial Marine Sediments; Organic/Fat Clay (OL, OH, CH, PT)
- af - Coarse-grained fill material (Fill) (SP, SM, SP-SM)
- Water



PROFILE
HOR. SCALE: 1"=300'
VERT. SCALE: 1"=60'

Notes:

1. This baseline profile was developed by projecting borings on to the alignment of the second bridge-tunnel built in the 1970s. For clarity, the existing tunnel structure and associated backfill materials are not included in the profile. Refer to project record drawings for limits of existing structures, and the associated backfill and scour protection materials.
2. For clarity, some CPT's are not shown and some have been moved slightly from their projected location.

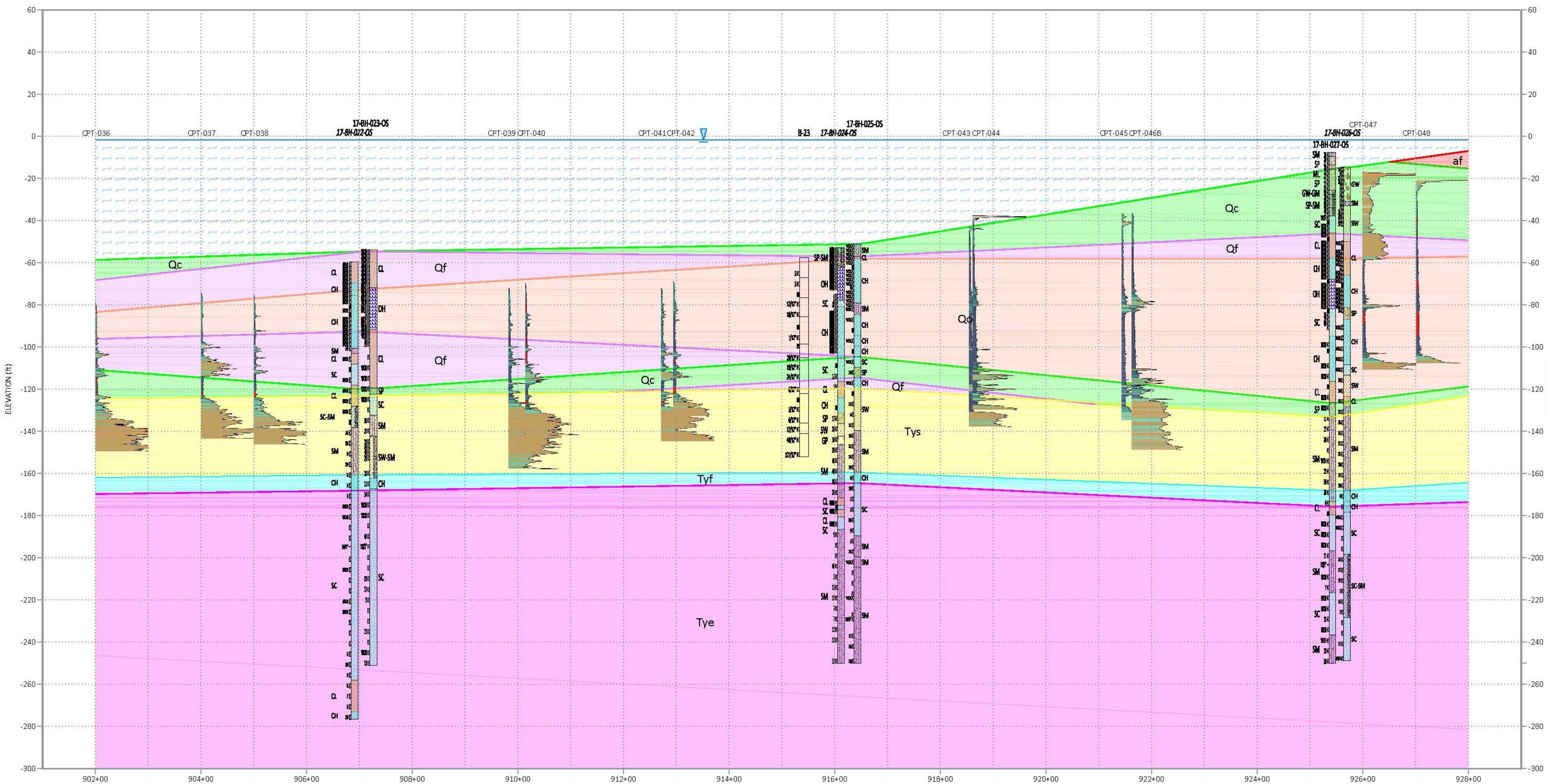
Notes (cont.):

3. This baseline profile is applicable for new tunnel alignments between the existing tunnels, and for alignments up to 500 feet to the west of the second tunnel alignment.
4. Qualifying statements related to interpretation of the subsurface profile for different elements of the proposed construction are presented in Sections 5.1.1, 5.2.1 and 5.3.1 of this report.

Figure 4.1 – Baseline Subsurface Profile 2/4

Legend

- Tye - Eastover Formation; Predominantly Coarse-grained (SC, SM, SP, SC-SM, SP-SC with minor lenses of CL)
- Tyf - Yorktown Formation; Predominantly Fine-grained (CH, mainly fat clay with some lenses of CL, MH and SC)
- Tys - Yorktown Formation; Predominantly Coarse-grained (SM, SP-SM, SP, SW, GP, SC, SW-SM, SC-SM)
- Qc - Alluvial Marine Sediments; Predominantly Coarse-grained (SP, SC, SM, SP-SM)
- Qf - Alluvial Marine Sediments; Predominantly Fine-grained (CL, CH)
- Qo - Alluvial Marine Sediments; Organic/Fat Clay (OL, OH, CH, PT)
- af - Coarse-grained fill material (Fill) (SP, SM, SP-SM)
- Water



PROFILE
HOR. SCALE: 1"=300'
VERT. SCALE: 1"=60'

Notes:

1. This baseline profile was developed by projecting borings on to the alignment of the second bridge-tunnel built in the 1970s. For clarity, the existing tunnel structure and associated backfill materials are not included in the profile. Refer to project record drawings for limits of existing structures, and the associated backfill and scour protection materials.
2. For clarity, some CPT's are not shown and some have been moved slightly from their projected location.

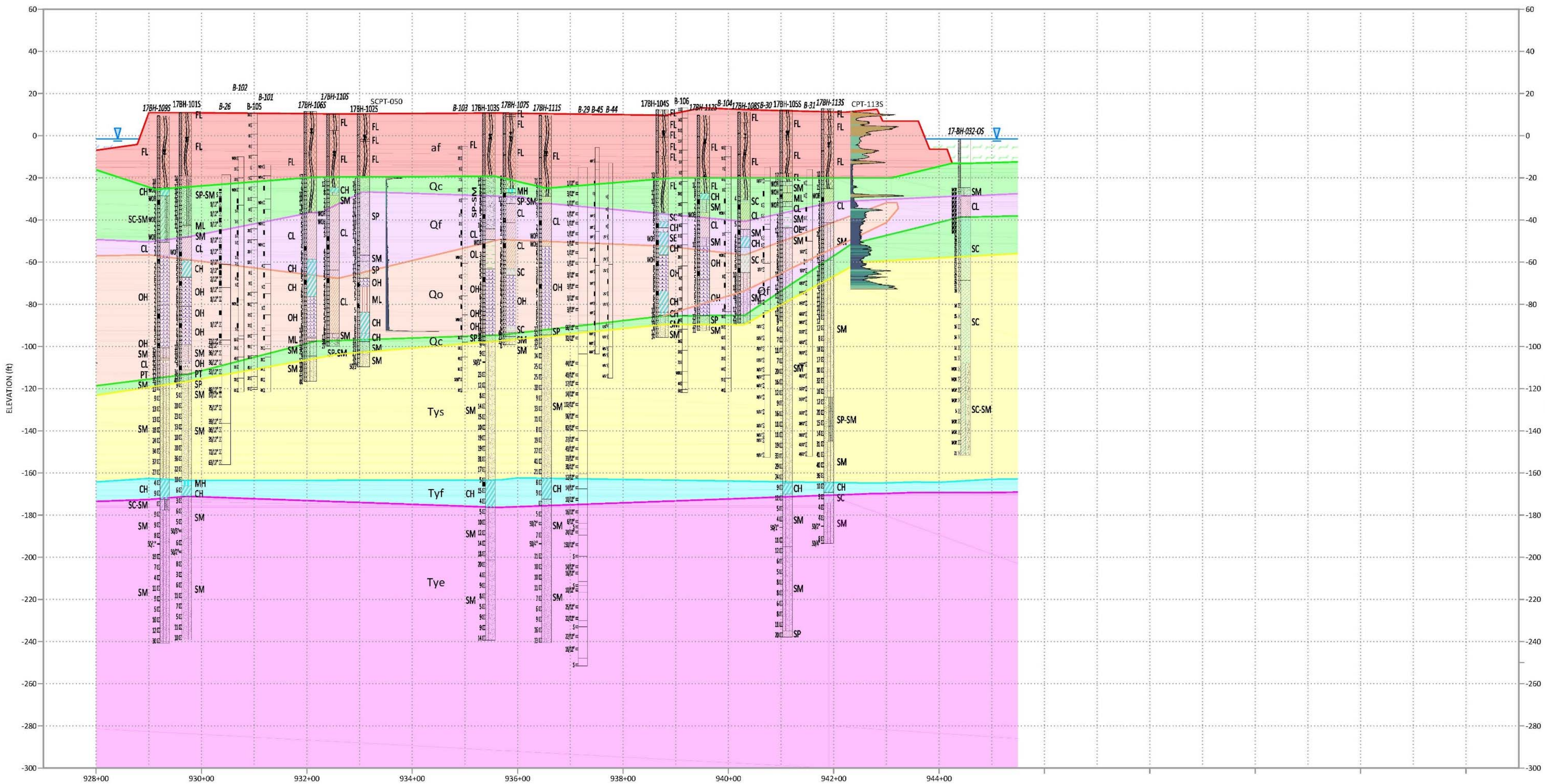
Notes (cont.):

3. This baseline profile is applicable for new tunnel alignments between the existing tunnels, and for alignments up to 500 feet to the west of the second tunnel alignment.
4. Qualifying statements related to interpretation of the subsurface profile for different elements of the proposed construction are presented in Sections 5.1.1, 5.2.1 and 5.3.1 of this report.

Figure 4.1 – Baseline Subsurface Profile 3/4

Legend

- Tye - Eastover Formation; Predominantly Coarse-grained (SC, SM, SP, SC-SM, SP-SC with minor lenses of CL)
- Tys - Yorktown Formation; Predominantly Coarse-grained (SM, SP-SM, SP, SW, GP, SC, SW-SM, SC-SM)
- Qf - Alluvial Marine Sediments; Predominantly Fine-grained (CL, CH)
- af - Coarse-grained fill material (Fill) (SP, SM, SP-SM)
- Tyf - Yorktown Formation; Predominantly Fine-grained (CH, mainly fat clay with some lenses of CL, MH and SC)
- Qc - Alluvial Marine Sediments; Predominantly Coarse-grained (SP, SC, SM, SP-SM)
- Qo - Alluvial Marine Sediments; Organic/Fat Clay (OL, OH, CH, PT)
- Water



Notes:

1. This baseline profile was developed by projecting borings on to the alignment of the second bridge-tunnel built in the 1970s. For clarity, the existing tunnel structure and associated backfill materials are not included in the profile. Refer to project record drawings for limits of existing structures, and the associated backfill and scour protection materials.
2. For clarity, some CPT's are not shown and some have been moved slightly from their projected location.

Notes (cont.):

3. This baseline profile is applicable for new tunnel alignments between the existing tunnels, and for alignments up to 500 feet to the west of the second tunnel alignment.
4. Qualifying statements related to interpretation of the subsurface profile for different elements of the proposed construction are presented in Sections 5.1.1, 5.2.1 and 5.3.1 of this report.

Figure 4.1 – Baseline Subsurface Profile 4/4

Legend

- Tye - Eastover Formation; Predominantly Coarse-grained (SC, SM, SP, SC-SM, SP-SC with minor lenses of CL)
- Tyf - Yorktown Formation; Predominantly Fine-grained (CH, mainly fat clay with some lenses of CL, MH and SC)
- Tys - Yorktown Formation; Predominantly Coarse-grained (SM, SP-SM, SP, SW, GP, SC, SW-SM, SC-SM)
- Qc - Alluvial Marine Sediments; Predominantly Coarse-grained (SP, SC, SM, SP-SM)
- Qf - Alluvial Marine Sediments; Predominantly Fine-grained (CL, CH)
- Qo - Alluvial Marine Sediments; Organic/Fat Clay (OL, OH, CH, PT)
- af - Coarse-grained fill material (Fill) (SP, SM, SP-SM)
- Water

5.0 Design and Construction Considerations

This section will discuss design and construction considerations as well as anticipated ground behavior during construction of the Tunnel Improvements, including:

- Below-grade tunnel approach structures
- Bored tunnel
- Island expansions

5.1 Tunnel Approach Structures

It is anticipated that the below-grade approach structure to the bored tunnel on both portal islands will be similar to the existing boat sections and cut and cover tunnels. This will involve installation of support of excavation systems, excavation to the design subgrade level, and construction of the reinforced concrete structures.

5.1.1 Stratigraphy

Figure 4.1 shows the baseline stratigraphy along the tunnel approaches based on the recent and historic geotechnical information. For design and construction of the tunnel approach structures, the interfaces between adjacent geological strata shown in the baseline profile are qualified by the following statements:

North Island (Station 850+00 to 867+00):

- The boundary shown between the Artificial Fill (af) and the Alluvial Marine Sediments Coarse-grained (Qc) is not baselined. The geotechnical properties of both materials are similar. Obstructions, that may be encountered more frequently in the fill layer, are addressed in Section 5.1.2 of this GBR; and
- The boundary between the Alluvial Marine Sediments (Qc) and the Yorktown formation (Tys) is baselined and may vary by ±5 feet from the boundary shown in Figure 4.1.

South Island (Station 924+00 to 945+00):

- The boundary shown between the Artificial Fill (af) and the Alluvial Marine Sediments-Coarse-grained (Qc) is not baselined. The geotechnical properties of both materials are similar. Obstructions that may be encountered more frequently in the fill layer are addressed in Section 5.1.2 of this GBR;

- The boundary between the Alluvial Marine Sediments-Coarse-grained sub-layer (Qc) and the underlying Alluvial Marine Sediments-Fine-grained sub-layer (Qf) is baselined and may vary by ±5 feet from the boundary shown in Figure 4.1;
- The boundary between the Alluvial Marine Sediments-Fine-grained sub-layer (Qf) and the underlying Alluvial Marine Sediments-Organic/Fat Clay sub-layer (Qo) is baselined and may vary by ±5 feet from the boundary shown in Figure 4.1; and
- The boundary between the Alluvial Marine Sediments (Qc, Qf, Qo) and the Yorktown formation is baselined and may vary by ±5 feet from the boundary shown in Figure 4.1.

5.1.2 Obstructions

There are numerous potential obstructions that may affect the construction of the tunnel approach structures and must be accounted for by the Design-Builder both in the design and selection of means and methods of construction. These obstructions fall into the following categories and are discussed in the subsequent paragraphs:

- Rock Containment Dikes and Scour Protection; and
- Temporary Works from Previous Construction.

5.1.2.1 Rock Containment Dikes and Scour Protection

As shown on the record drawings (Commonwealth of Virginia, 1956, 1975), the existing portal islands were constructed using a series of containment dikes, which contain armor stone with individual stone size varying up to a maximum dimension of 5 feet. Additionally, the portal islands, as well as the existing tunnels, are covered with various layers of scour protection including heavy rip rap, quarry run rock and gravel sized crushed rock. The materials that comprise the containment dikes and the scour protection were obtained from intact, crystalline, igneous or metamorphic rock sources. They could potentially cause obstructions to installation of support of excavations systems, and cause handling difficulties requiring special lifting equipment during mass excavation of the approaches.

These rip rap and stone layers were installed in both the 1950s and 1970s construction, and it is understood that the original stone layers from the 1950s were buried when the islands were expanded in the 1970s. The general location of the armor stone dikes and heavy rip rap placements are shown on the record drawings (Commonwealth of Virginia, 1956, 1975) and construction photographs included as Supplemental Information to the Contract.

It is also very likely that rip rap and stone layers were also placed, inadvertently or otherwise, in areas not shown on the record drawings. In addition, numerous major storms have occurred throughout the construction and operational history of the islands which have damaged the stone protection of both islands. This has resulted in the deposition and possible burial of random armor stone, construction materials and other miscellaneous debris within the limits of the proposed temporary and permanent construction.

For baseline purposes, the Design-Builder shall be prepared to encounter heavy rip rap and other large stone layers as follows:

- At the North Island, between Stations 850+00 and 867+00, the Design-Builder may encounter armor stone with individual stone size up to a maximum dimension of 5 feet and weighing up to 10 tons anywhere above Elevation -25 feet. This applies over a width of 500 feet to the east and 200 feet to the west of the second 1970s tunnel centerline.
- At the South Island, between Stations 924+00 and 945+00, the Design-Builder may encounter armor stone with individual stone size up to a maximum dimension of 5 feet and weighing up to 10 tons anywhere above Elevation -30 feet. This applies over a width of 500 feet to the east and west of the second 1970s tunnel centerline.
- The thickness of the various stone layers is shown on the record drawings (Commonwealth of Virginia, 1956, 1975) and for baseline purposes may be encountered in local areas up to twice the layer dimension shown on the drawings.

The Design-Builder’s means and methods and bid price shall account for accommodating any rip rap or stone layers encountered within the limits outlined in the previous three bulleted items. Heavy rip rap or armor stone is not expected to be encountered outside of these limits.

5.1.2.2 Temporary Works from Previous Construction

Based on review of construction photographs, both the 1950s and 1970s construction employed a combination of soldier pile and lagging and steel sheeting to support the tunnel approach excavations. In the 1950s, cross lot struts were placed between the two walls, whereas tiebacks were installed in the 1970s. To maintain a dry excavation, dewatering wells were also installed. Dewatering wells were likely to be more numerous in the vicinity of where excavations were deepest. Temporary piers supported on piles may also have been constructed around the perimeter of the islands for off-loading construction materials.

The Design-Builder's means and methods and bid price for the tunnel approach structures shall account for encountering any of the following temporary works from previous construction:

- Steel soldier pile and lagging or steel sheeting encountered up to 10 feet behind the face of the current finished walls of the tunnel approaches, and up to 20 feet below the base of the original excavation.
- Tieback tendons encountered up to 65 feet behind the face of the finished walls, and up to 20 feet below the full depth of the original excavation. Tieback tendons are baselined to have been de-tensioned;
- Steel dewatering well screens may be encountered to Elevation -140 feet and may be present within 50 feet of the periphery of the current approach structures. Dewatering wells may or may not have been grouted or backfilled when abandoned.

5.1.3 Groundwater Control

The Design-Builder shall not rely on any untreated in situ soil strata (or combination of soil strata) to provide stability and/or groundwater cutoff for any excavation required for the tunnel approach structures. The Design-Builder shall consider that dewatering or ground treatment, along with provision of sufficient penetration below the base of excavation with support of excavation walls, will be required to provide stability and limit groundwater inflows into the excavations.

5.1.4 Gas Conditions

The geotechnical investigations did not find any evidence of the presence of gas in the deposits that will be encountered during construction of the approach structures. However, the occurrence of methane due to decay of organic matter is expected. Historic records indicate that the organic layers present from Station 878+00 to 945+00 have previously been a source of gas. It is understood that during installation of the sand drains on the South Island during the 1970s construction, methane gas was encountered. The escaping gas was lit which fueled a flare that burned for months. In addition, in a 1970s era boring for I-64 on the Willoughby Spit, to the south of the South Island, methane gas was encountered in the organic layer, which led to relocation of a proposed bridge following an unsuccessful attempt to burn off the gas that lasted for three months without a significant drop in pressure.

Therefore, during construction of the below grade tunnel approach structures on the South Island, gas may be encountered during any construction activity that penetrates layers with organic content.

5.1.5 Soil pH

Soil pH testing was conducted on samples from all strata that will be encountered in the tunneling. For baseline purposes, the Design-Builder shall consider the soil pH value will range between a minimum of 7.0 and a maximum of 11.0.

5.2 Bored Tunnel

This section describes anticipated subsurface conditions along the bored tunnel alignment including soil stratigraphy, ground improvement required to stabilize very soft layers below the TBM, potential obstructions, and various geotechnical-related issues that could arise during construction of the tunnel.

Section 23, Bored Tunnel of the Technical Requirements (TRs) of the Contract Documents requires tunnel excavation methods that were established based on the need to control unstable ground conditions. The minimum requirements include the use of a pressurized face Tunnel Boring Machine (TBM) that can control groundwater and stabilize the tunnel face in the various strata that will be encountered. This machine must be able to apply a positive pressure to the tunnel face, resisting the earth and groundwater pressures to effectively control the ground and groundwater inflows.

5.2.1 Stratigraphy

Figure 4.1 shows the baseline stratigraphy along the tunnel alignment based on the recent and historic geotechnical information. For design and construction of the bored tunnel by TBM, the boundaries between adjacent geological strata shown in the baseline profile are qualified by the following statements:

- The boundary shown between the Artificial Island Fill (af) and the Alluvial Marine Sediments-Coarse-grained (Qc) is not baselined. The geotechnical properties of both materials are similar.
- The boundary shown between the Alluvial Marine Sediments-Coarse-grained sub-layer (Qc) and the Alluvial Marine Sediments- Fine-grained sub-layer (Qf) is baselined and may vary by ±5 feet.
- The boundary shown between the Alluvial Marine Sediments-Fine- grained sub-layer (Qf) and the Alluvial Marine Sediments- Organic/Fat Clay sub-layer (Qo) is not baselined.

- Except for the baseline for determining ground improvement limits provided in the next bullet, the boundary shown between the Alluvial Marine Sediments (Qc, Qf, Qo) and the Yorktown formation (Tys) is baselined and may vary by ±5 feet.
- For determining ground improvement limits required by the contract, the boundary shown between the Alluvial Marine Sediments (Qc, Qf, Qo) and the Yorktown formation (Tys) is baselined and does not vary from the location shown in Figure 4.1.
- The boundary shown between the two Yorktown formation sub-layers (Tys, Tyf), and between the Yorktown and the Eastover formations is not baselined. When tunneling in the Yorktown (Tys, Tyf), or the Eastover (Tye) formations, the TBM excavation will include very soft to hard cohesive deposits, very loose to very dense granular deposits, as well as cemented sands and shell layers; the Design-Builder shall be prepared to encounter the full range of the materials defined in this GBR for the Yorktown and Eastover Formations anywhere along the tunnel alignment below the Alluvial Marine Sediments stratum (Qc, Qf, Qo).

The Design-Builder shall anticipate unstable soil conditions (e.g., flowing soil as defined in Appendix B) at any location along the tunnel alignment and accordingly must maintain a positive face pressure at the heading at all times; therefore, methods to excavate the tunnel in a controlled manner (i.e., match the muck removal rate from the excavation chamber to the advance rate of the TBM while maintaining the desired pressure at the working face) will be necessary to maintain stability and control of over-excavation. Refer to TR Section 23, Bored Tunnel for minimum TBM requirements and requirements for compressed air interventions. The ground conditions along the entire alignment will not support free air interventions.

Although adjustments to TBM operating parameters (e.g., face pressures, type and volume of soil conditioning) are to be expected with the mixed soil conditions to be encountered at the face of the TBM, these adjustments will be more frequent within zones where soil groups with distinctly different index or engineering characteristics are encountered. Changes in the behavior of the excavated material (e.g. stickiness, abrasivity, strength, and hydraulic conductivity) will also occur as the proportion of the different soil groups encountered at the face changes. Therefore, continuous monitoring and adjustments of face pressures and conditioners will be necessary when tunneling throughout.

5.2.2 Break-in/out

Ground improvement will be necessary to allow for a stable excavation during break-out/break-in of the TBM from the tunnel approach structures at the portal islands (i.e. the soil at the break-out/break-in locations will be neither stable nor impervious). TR Section 23, Bored Tunnel specifies minimum requirements for the ground improvement.

5.2.3 Behavior of Soft Soils

Between approximately Stations 877+50 and 945+50, and between approximately Elevations -50 and -130 feet, the Alluvial Marine Sediments-Fine-grained sub-layer (Qf) and the underlying Alluvial Marine Sediments-Organic/Fat Clay sub-layer (Qo) consist of very soft, inorganic and organic silts and clays, and peat. Ground improvement will be required to strengthen these layers to provide sufficient support to the TBM to maintain alignment and grade.

TR Section 23, Bored Tunnel requires that between Stations 877+50 and 945+50, whenever any part of the tunnel below the springline is within the Alluvial Marine Sediments-Fine-grained sub-layer (Qf) or the Alluvial Marine Sediments-Organic/Fat Clay sub-layer (Qo), the Design-Builder shall provide ground improvement to the in situ soils in advance of tunneling. The horizontal and vertical limits of the ground improvement required by Section 23 are shown in Figure 5.1.

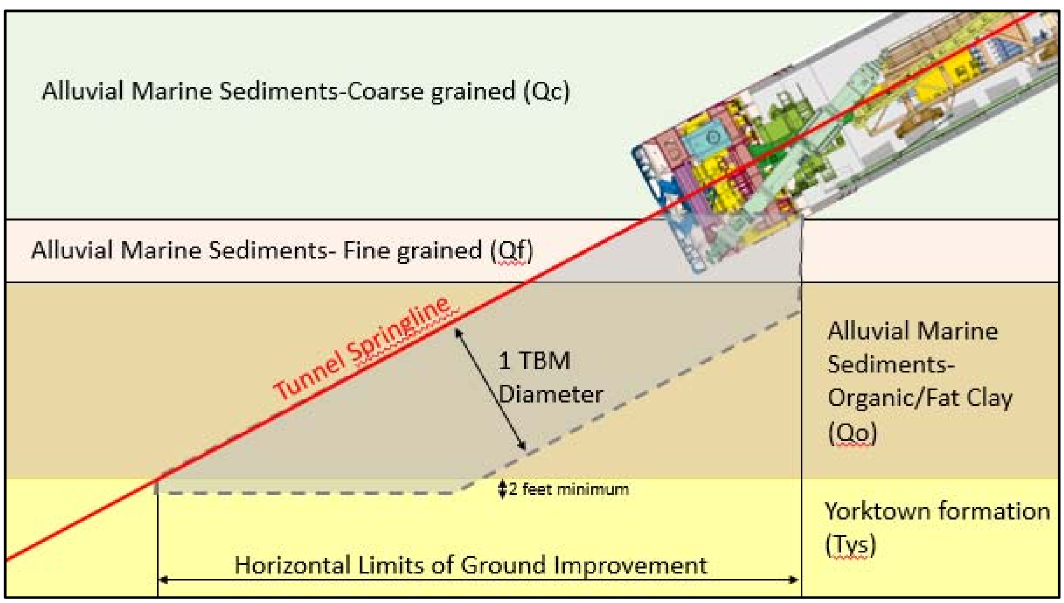


Figure 5.1 - Limits of Ground Improvement Required to Support TBM in Soft Soils

The Design-Builder's bid price for the required ground improvement should be based on the baseline profile shown in Figures 4.1 as modified by the qualifying statements in Section 5.2.1. Prior to performance of this ground improvement, TR Section 24, Geotechnical - Islands and Tunnels requires that a detailed geotechnical investigation be implemented to define the top and bottom of the Qf and Qo sub-layers.

The ground improvement method shall be selected by the Design-Builder to be compatible with their tunneling operations from the alternatives listed in TR Section 24, Geotechnical - Islands and Tunnels. Other ground improvement techniques may be considered as an Alternative Technical Concept (ATC). The design and performance requirements of the ground improvement is the responsibility of the Design-Builder. The improved ground limits and characteristics shall be able to support the TBM to maintain the Design-Builder's proposed tunnel alignment within reasonable driving tolerances.

5.2.4 Obstructions

The geologic strata present along the tunnel alignment are not known to contain naturally occurring boulders that could impede the progress of the TBM, and the Design-Builder may consider that boulders will not be encountered outside the limits defined herein for man-made obstructions. Nevertheless, as a safeguard, and as required by TR Section 23, Bored Tunnel, the TBM shall be equipped to accommodate boulders.

There are numerous "man-made" obstructions that may impede the progress of the TBM and must be accounted for by the Design-Builder both in the design and selection of means and methods of construction. These obstructions fall into the following categories and are discussed in the following paragraphs:

- Rock Containment Dikes and Scour Protection;
- Temporary Works and Miscellaneous Debris from Previous Construction;
- Unrecovered Equipment from Geotechnical Drilling Program; and
- Foundations for Survey Towers.

5.2.4.1 Rock Containment Dikes and Scour Protection

Rock containment dikes and scour protection constructed in the 1950s and 1970s were described in Section 5.1.2.1. With respect to TBM tunneling, for baseline purposes, the Design- Builder shall be prepared to encounter heavy rip rap and stone layers as follows:

- At the North Island, between Stations 850+00 and 867+00, the Design-Builder may encounter armor stone the same as the size and limits detailed in Section 5.1.2.1;
- At the South Island, between Stations 924+00 and 945+00, the Design-Builder may encounter armor stone the same as the size and limits detailed in Section 5.1.2.1;
- Between the two islands, from Station 867+00 to 924+00, the Design-Builder may encounter armor stone with a maximum dimension up to 5 feet and weighing up to 10 tons anywhere within 10 feet below the mudline shown on Figure 4.1;

- Between the two islands, from Station 867+00 to 924+00, the Design-Builder may encounter scour protection, locking backfill, and other types of special fill materials within the backfilled trenches of the existing tunnels, as shown on the record drawings (Commonwealth of Virginia, 1956,1975). For baseline purposes, the backfilled trench limits may vary up ±20 feet horizontally and ±5 feet vertically from the limits shown on the record drawings; and
- The thickness of the various stone layers is shown on the record drawings (Commonwealth of Virginia, 1956, 1975) and for baseline purposes may be encountered in local areas up to twice the layer dimension shown on the drawings.

The Design-Builder's means and methods, and bid price, for TBM tunneling shall account for accommodating any rip rap or stone layers encountered within the limits outlined in the previous four bulleted items. However, the Design-Builder's bid price may consider that heavy rip rap or armor stone will not be encountered outside of these limits.

5.2.4.2 Temporary Works from Previous Construction

Temporary works from previous construction in the 1950s and 1970s were described in Section 5.1.2.2.

The Design-Builder's means and methods and bid price for TBM tunneling shall account for encountering any of the following temporary works:

- Soldier pile and lagging or sheeting encountered up to 10 feet behind the face of the current finished walls of the tunnel cut and cover approach structures, and up to 20 feet below the base of the original excavation;
- De-tensioned tieback tendons may be encountered up to 65 feet behind the face of the finished walls, and up to 20 feet below the full depth of the original excavation.
- Steel dewatering well screens may be encountered to Elevation -140 feet and may be present within 50 feet of the periphery of the current approach structures. The steel well casings may or may not have been backfilled with grout or other material.
- Miscellaneous debris from the previous construction may be present within the footprint of the work. For example, a steel cable was encountered in a Shelby tube sample in Boring 17-BH-022-OS at a depth between 15 and 20 feet below the mudline. For baseline purposes, the Design-Builder shall consider that miscellaneous construction debris may be encountered within ±25 feet horizontally, and ±10 feet vertically from the limits of the backfilled trench for the previous tunnels, as shown on the record drawings.

- Timber, steel or concrete piles left-in-place from temporary piers may be present up to 50 feet from the perimeter of the islands (both 1950s and 1970s perimeters) and may extend 10 feet below the top of the Yorktown Formation shown in Figure 4.1. For baseline purposes, the Design-Builder shall account for up to 4 such piles being encountered where the tunnel alignment crosses the island perimeter (or former perimeter).

5.2.4.3 Unrecovered Equipment from Geotechnical Drilling Program

During performance of the marine drilling program, equipment issues and failures led to pieces of steel being left buried below the mudline at three test locations, as outlined below.

B-018 After completion of the boring, a 40-foot-long section of 6-inch casing became de-coupled during removal and was left in the borehole;

CPT-034 After completion of the CPT probe, a 5-foot-long section of NQ casing became de-coupled during removal and was left in the borehole; and

VST-045 After completion of a vane shear test it was discovered that the connection between the vane and the rods sheared off leaving the vane, a coupler, and an extension rod in the borehole.

The surface coordinates and elevations of the buried equipment that could not be recovered is detailed in Table 5.1 and shown in Figure 5.2. The coordinates should be considered to be accurate within ±20 feet, and the elevation within ±30 feet.

Boring	**Northing**	**Easting**	**Elevation (ft NAVD88)**	**Description of Potential Obstruction**
B-018	36° 59’ 36.90773”N	76° 18’ 40.92049”W	-76.2 to -116.2	6” Casing
CPT-034	36° 59’ 30.56334”N	76° 18’ 40.06898”W	-73.6 to -90.3	NQ Casing
VST-045	36° 59’ 12.9209”N	76° 18’ 18.2663”W	-81.7 to -83.7	Shear Vane

Table 5.1 - Location of Unrecovered Equipment from Geotechnical Drilling Program

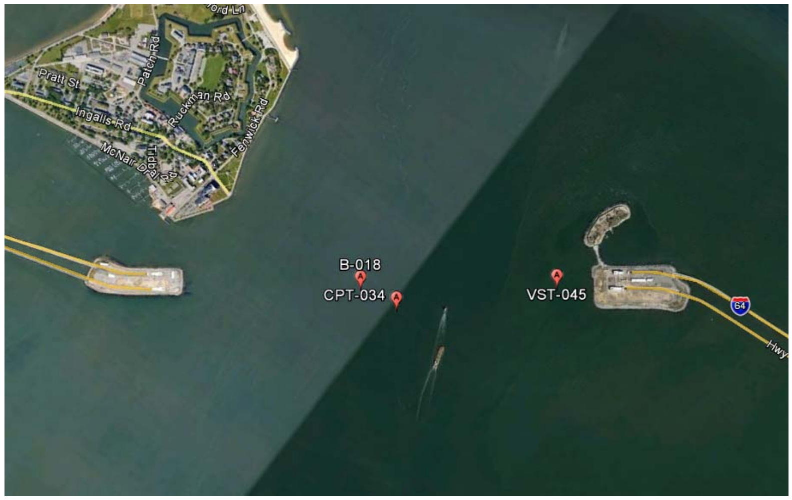


Figure 5.2 - Location of Unrecovered Equipment from Geotechnical Drilling Program

The Design-Builder's means and methods, and bid price, shall account for encountering these potential obstructions and removing as necessary. If encountered with the TBM, these items may create obstructions to inhibit TBM advance and or damage to the TBM and require special effort to remove them such as through interventions.

5.2.4.4 Foundations for Survey Towers

Horizontal and vertical control for the existing tunnels was established with the aid of survey towers located in the Hampton Roads waterway. While the superstructure of the towers was removed, the piles and other foundation elements were abandoned in place. The general locations of the survey towers, which have since been removed, are indicated on the original drawings (Commonwealth of Virginia, 1956, 1975). The Design-Builder's means and methods, and bid price, shall account for encountering the piled foundations of these structures within ten feet of the locations indicated on the original drawings, and removing as necessary. If encountered with the TBM, these items may create obstructions to inhibit TBM advance and or damage to the TBM and require special effort to remove them such as through interventions.

5.2.5 Sticky Clay Behavior

The TBM tunnel drive will encounter cohesive layers within the Alluvial Marine Sediments (Qc, Qf, Qo) and the Yorktown (Tyf, Tys) and Eastover (Tye) formations. The TBM design and implementation of soil conditioning shall consider the potential for sticky clay behavior. Selection of the specific materials, means, and methods for soil conditioning is the sole responsibility of the Design-Builder. The Design-Builder must continuously monitor and adjust the conditioning process to ensure these materials, means, and methods are appropriate for the ground conditions being encountered at the face. This will require constant supervision and well-defined working plans and instructions for all construction personnel involved in TBM operations. Accordingly, baselines for soil stickiness potential are not provided herein. The Design-Builder is solely responsible for determining how the soils, given the baseline soil properties as defined in Tables 4.1 will respond to the means and methods selected and implemented by the Design-Builder. Sticky clay behavior will not be considered grounds for a Differing Tunnel Improvements Site Condition for layers encountered with properties within the baseline values included in Table 4.1.

5.2.6 Cemented Sand and Shell Zones

The Yorktown (Tyf, Tys) and Eastover (Tye) formations contain cemented layers and shell beds that are classified as very dense or hard. The design and operation of the TBM should consider the occurrence of these layers.

An upper bound unconfined compressive strength for the Yorktown (Tyf, Tys) and Eastover (Tye) formations as indicated in Table 4.1 shall be a baseline for consideration of the potential resistance that very dense granular, hard cohesive, or cemented sands or shells will present to excavation equipment for the TBM tools and forward thrusting of the TBM.

5.2.7 Soil Abrasion Behavior

The abrasion potential of the various soil strata that may be encountered by bored tunneling was investigated by means of soil abrasion testing using a method based on Nilsen et al (2007). The results of the testing are provided in the GDRs (Jacobs Engineering, 2018, ECS Mid-Atlantic LLC, 2018).

Baseline values of soil abrasion for various soil strata (i.e., natural soil, without conditioners) are presented in Table 5.2. The baselines were developed based upon the soil abrasion test results included in the GDRs. As per Jakobsen et al. (2013), a soil with an SAT™ less than or equal to 7 is classified as a "low" abrasive material, a soil with an SAT™ between 7 and 22 is classified as a "medium" abrasive material and a soil with an SAT™ of greater than or equal to 22 is classified as being a "high" abrasive material. These qualifiers notwithstanding, it is the numerical values of the SAT™ that are baselined.

Stratum	USCS Classification	Soil Abrasion Test Value
		Baseline Range
Island Artificial Fill (af)	SP, SM, SP-SM	20-40
Alluvial Marine Sediments – Coarse-Grained (Qc)	SP, SM, SC	30-55
Alluvial Marine Sediments – Fine-Grained (Qf)	CL, CH	N/A
Alluvial Marine Sediments – Organic/Fat Clay (Qo)	OL, OH, CH, PT	N/A
Yorktown Formation – Coarse-Grained (Tys)	SM, SC	30-40
Yorktown Formation – Fine-Grained (Tyf)	CL, CH	35-50
Eastover Formation (Tye)	SP, SM, SP-SC	25-35

Table 5.2 – Baseline Values for Soil Abrasivity

5.2.8 Gas Conditions

Historic records indicating that the organic layers have previously been a source of methane gas were described in Section 5.1.4.

The classification of the underground work associated with the bored tunnel option in accordance with OSHA regulations (i.e., 29 CFR 1926.800 Subpart S) is provided in TR Section 23, Bored Tunnel.

5.2.9 Soil pH

Soil pH testing was conducted on samples from all strata that will be encountered in the tunneling. For baseline purposes, the Design-Builder shall consider the soil pH value will range between a minimum of 7.0 and a maximum of 11.0.

5.3 Island Expansions

Expansions of the existing North and South Islands may be required to provide portals for horizontal tunnel alignments located to the west of the existing tunnels, and also to provide sufficient cover for vertical alignments that would not allow the TBM to operate with existing island grades.

5.3.1 Stratigraphy

Figure 4.1 shows the baseline stratigraphy below the islands based on the recent and historic geotechnical information. For design and construction of the island expansions, the interfaces between adjacent geological strata shown in the baseline profile are qualified by the statements included in Section 5.1.1.

5.3.2 Obstructions

There are numerous potential obstructions that may affect the construction of the island expansions and must be accounted for by the Design-Builder both in the design and selection of means and methods of construction. For design and construction of the island expansions the potential impacts from obstructions shall be as presented in Section 5.1.2.

6.0 Instrumentation and Monitoring

The Contractor shall be responsible for the design, installation, monitoring, and data interpretation of the Instrumentation and Monitoring program in accordance with the Technical Requirements.

7.0 Limitations

The interpretations and assessments contained in this report are based upon the available information from borings, in situ tests and laboratory tests. The geologic environment of the Hampton Roads Bridge-Tunnel Expansion project is complex, and as such, no amount of pre-construction information will convey as detailed an understanding as will exist during and following excavation. The range of anticipated site and construction conditions presented in this report is established as a baseline only to be used by all Bidders in preparing their bids, and by the Department to evaluate claims relating to alleged Differing Tunnel Improvements Site Conditions. The range of anticipated conditions was developed with the standard of care commonly applied as the state of the practice in the profession. No warranty is included either expressed or implied that the actual conditions encountered will conform exactly to the baseline conditions described herein.

8.0 References

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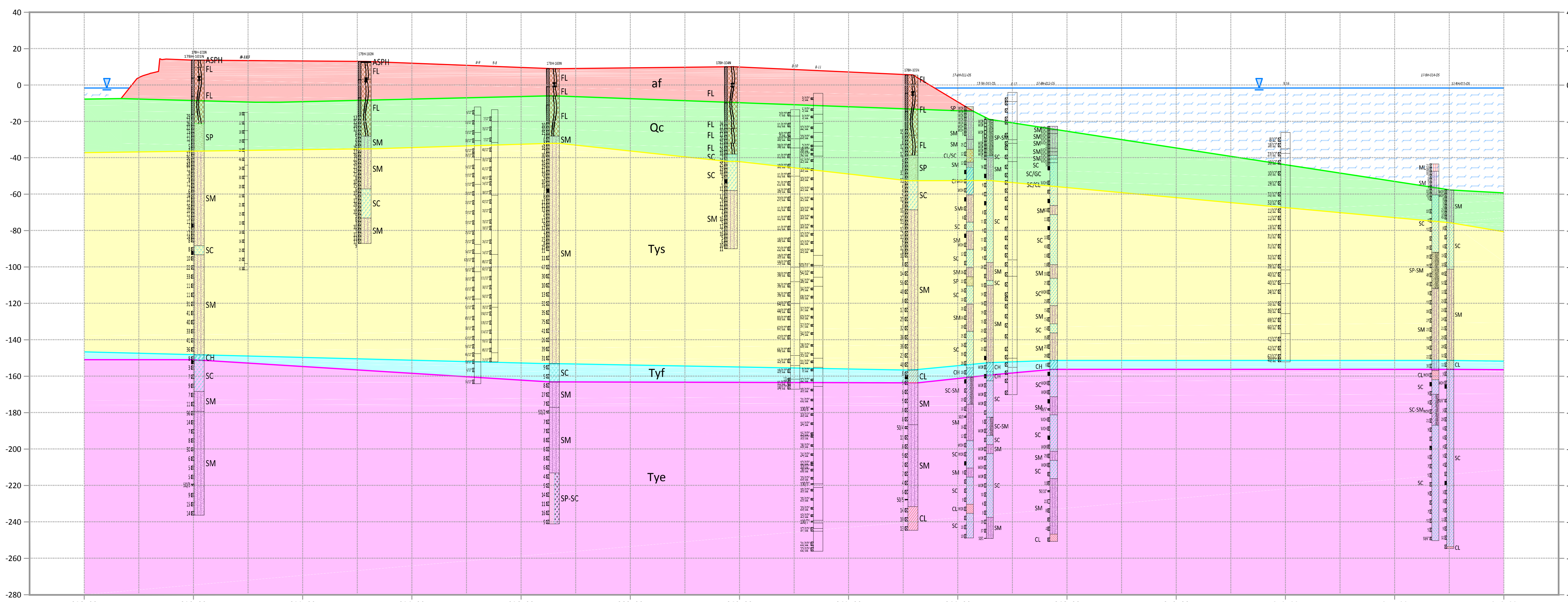
APPENDIX A - Boring Location Plan and Baseline Subsurface Profile

PLAN

SCALE: 1"=100'



PROFILE

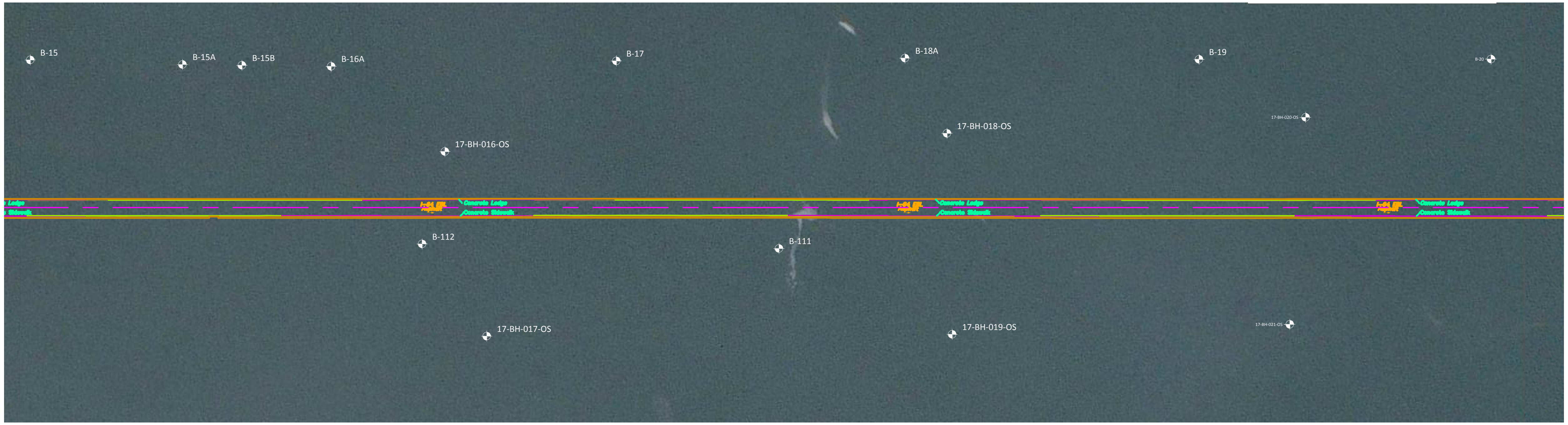
HOR. SCALE: 1"=100'
VERT. SCALE: 1"=30'

Legend

- Tye - Eastover Formation; Predominantly Coarse-grained (SC, SM, SP, SC-SM, SP-SC with minor lenses of CL)
- Tyf - Yorktown Formation; Predominantly Fine-grained (CH, mainly fat clay with some lenses of CL, MH and SC)
- Tys - Yorktown Formation; Predominantly Coarse-grained (SM, SP-SM, SP, SW, GP, SC, SW-SM, SC-SM)
- Qc - Alluvial Marine Sediments; Predominantly Coarse-grained (SP, SC, SM, SP-SM)
- Qf - Alluvial Marine Sediments; Predominantly Fine-grained (CL, CH)
- Qo - Alluvial Marine Sediments; Organic/Fat Clay (OL, OH, CH, PT)
- af - Coarse-grained fill material (Fill) (SP, SM, SP-SM)
- Water

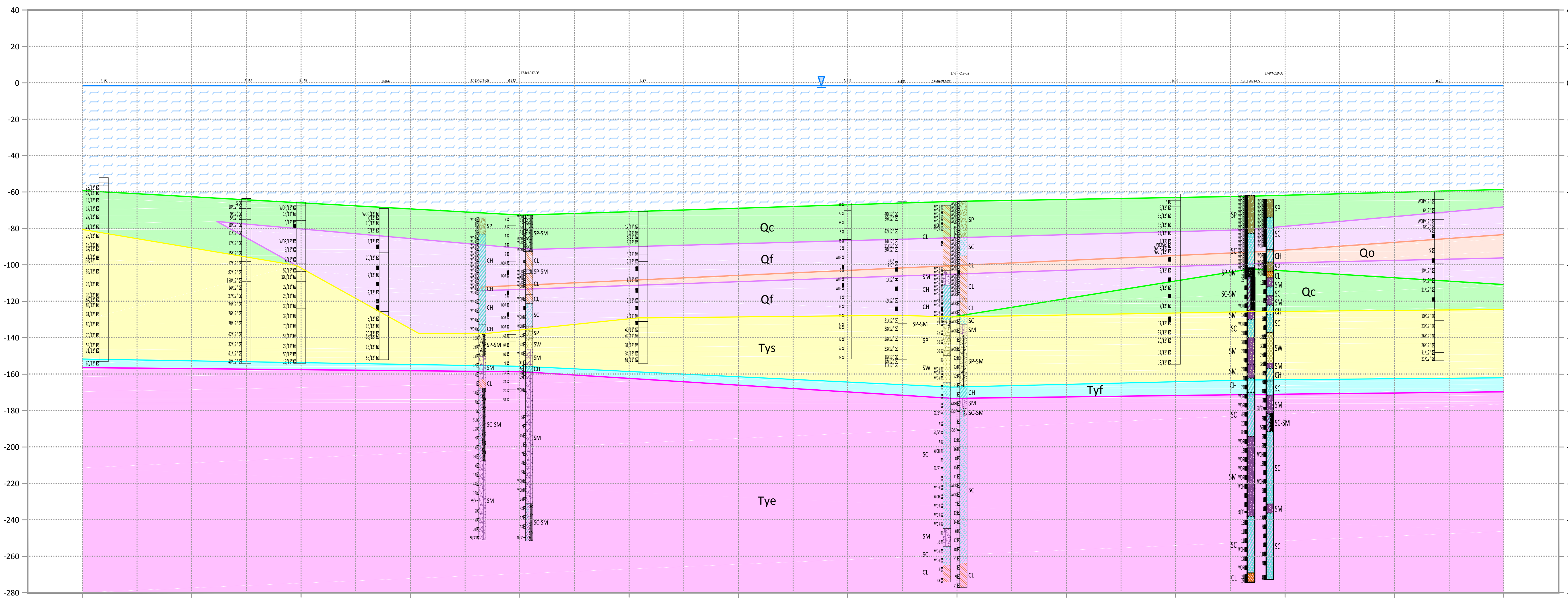
Notes:

1. This baseline profile was developed by projecting borings on to the alignment of the second bridge-tunnel built in the 1970s. For clarity, the existing tunnel structure and associated backfill materials are not included in the profile. Refer to project record drawings for limits of existing structures, and the associated backfill and scour protection materials.
2. For clarity, some CPT's are not shown and some have been moved slightly from their projected location.
3. This baseline profile is applicable for new tunnel alignments between the existing tunnels, and for alignments up to 500 feet to the west of the second tunnel alignment.
4. Qualifying statements related to interpretation of the subsurface profile for different elements of the proposed construction are presented in Sections 5.1.1, 5.2.1 and 5.3.1 of this report.



PLAN

SCALE: 1"=100'



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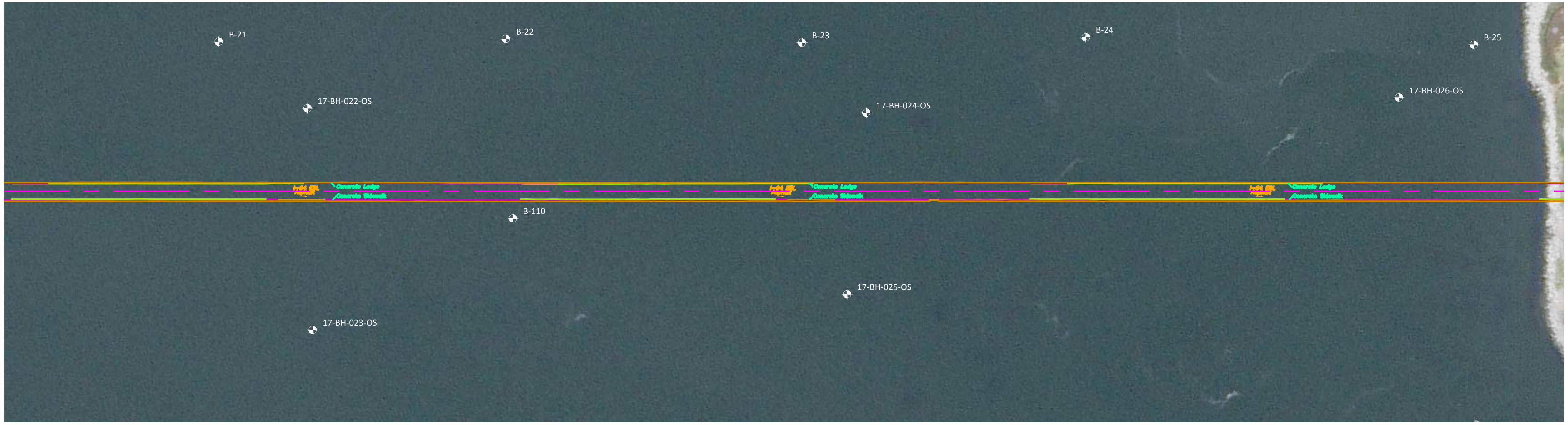

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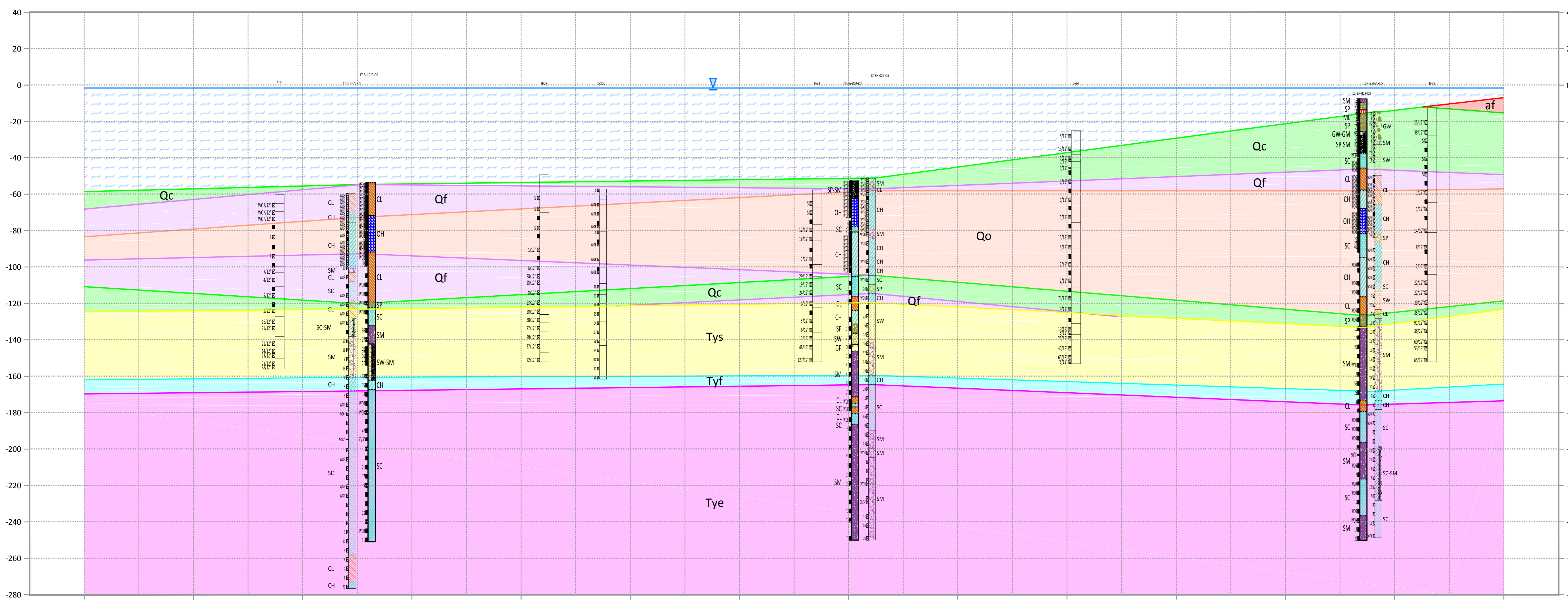

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- Water

Notes:

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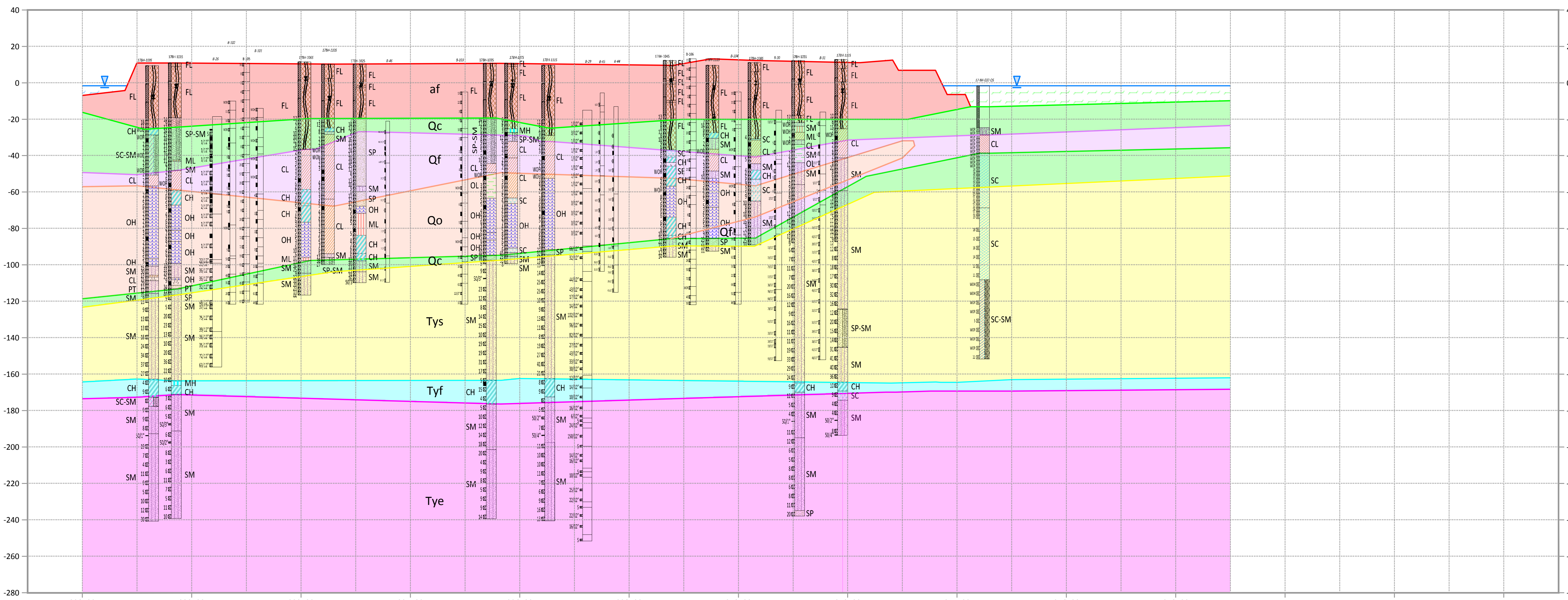
SCALE: 1"=100'



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APPENDIX B – Tunnelman's Classification System

Tunnelman's Classification System (After Heuer, 1974)

Classification		Behavior	Typical Soil Type
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to	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 raveling 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 execution 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. 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 the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive- raveling
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter from the invert as well as the face, crown, and walls, and can flow for great distances,	Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also occur in sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly pre-consolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.