Geotechnical Engineering Report

Emmanuel Temple Seventh Day Adventist Church
Buffalo, NY
October 3, 2019
Terracon Project No. J5195120

Prepared for:
Foit Albert Associates
Buffalo, NY

Prepared by:
Terracon Consultants-NY, Inc.
Buffalo, New York

October 3, 2019

Foit Albert Associates
295 Main St, Suite 200
Buffalo, NY 14203

Attn: Mr. Michael R. Bray, AIA – Project Architect
P: (716) 856-3933
E: mbray@foit-albert.com

Re: Geotechnical Engineering Report
Emmanuel Temple Seventh Day Adventist Church
Corner of Genesee Street and Jefferson Avenue
Buffalo, NY
Terracon Project No. J5195120

Dear Mr. Bray:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PJ5195120 dated July 15, 2019. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants-NY, Inc.

STATE OF NEW YORK
MICHELE A. FIORILLO
085888
LICENSED PROFESSIONAL ENGINEER

Charles B. Guzzetta
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REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Note: Refer to each individual Attachment for a listing of contents.

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INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed Emmanuel Temple SDA Church to be located at Corner of Genesee Street and Jefferson Avenue in Buffalo, NY. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil and rock conditions
- Groundwater conditions
- Site preparation and earthwork
- Demolition considerations
- Excavation considerations
- Dewatering considerations
- Infiltration testing
- Foundation design and construction
- Floor slab design and construction
- Seismic site classification per IBC
- Lateral earth pressures
- Pavement design and construction
- Frost considerations

The geotechnical engineering Scope of Services for this project included the advancement of seven test borings (B-1 through B-7) to depths ranging from approximately 6 to 19 feet below existing site grades. Infiltration testing were also performed in proximity to borings B-5, B-6, and B-7.

Maps showing the site and boring locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The boring logs are included in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located at the Corner of Genesee Street and Jefferson Avenue in Buffalo, NY. See **Site Location**. Based upon our review of Sanborn Maps, it appears that the site was in the past developed with commercial and residential structures, which have been demolished.
Existing Improvements	Site is currently vacant.
Current Ground Cover	Portions of the site are gravel/asphalt covered and portions are grass covered.
Existing Topography	Relatively flat; slope gently to the south.
Geology	The project is located within the Erie-Ontario Lowlands physiographic province. This province consists of relatively low, flat areas to the south of Lake Ontario and Lake Erie. It rises to the Portage Escarpment in the south where it borders the Allegheny Plateau Province. The soil deposits within this province generally consist of both glacially-derived deposits, such as glacial till (i.e. terminal moraines and ground moraine), granular deposits (i.e. kame, glacial outwash, and beach ridges) and glacio-lacustrine deposits (i.e. varved silts, clay, and fine sand deposits). Mapping of surficial materials by the *Surficial Geologic Map of New York, Niagara Sheet* identifies surficial deposits at the project site as lacustrine silt and clay. Mapping of the bedrock by the *Geologic Map of New York, Niagara Sheet,* indicates that the bedrock underlying the project area may consist of Middle Devonian Onondaga Limestone.

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	RFP emailed to Terracon on July 3, 2019. Emails from October 3, 2019 with information on proposed structure and structural loads.
Project Description	The project is comprised of multiple parcels to be developed for a new Church.
Proposed Structure	The project includes a new 18,000 sf church building and its surrounding sitework.
Building Construction	We anticipate the structure to have load-bearing masonry walls, steel or wood frame, and a slab-on-grade (no basement).
Finished Floor Elevation	Finished floor elevation is expected to be at approximately same elevation of existing grades to minimize earthwork cute and fill operations.
Maximum Loads (estimate)	■ Columns: 50 kips ■ Walls: 2 kips per linear foot (klf) ■ Slabs: 125 pounds per square foot (psf)
Below-Grade Structures	We understand that the new structure will have a slab-on-grade, and basements are not planned.
Pavements	We assume both rigid (concrete) and flexible (asphalt) pavement sections should be considered. Anticipated traffic is as follows: ■ Autos/light trucks: 200 vehicles per day ■ Light delivery and trash collection vehicles: 5 vehicles per week ■ Tractor-trailer trucks: <1 vehicle per week The pavement design period is 20 years.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of site preparation and foundation options. Conditions encountered at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** section and the GeoModel can be found in the **Figures** section of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	Fill[1]	Silty Sand with Gravel; concrete rubble; numerous brick fragments, trace wood fragments; black and brown
2	Native Soil	Silty Clay and Silty Sand (CL-ML; SM); trace gravel and rock fragments; medium stiff (cohesive) and medium dense (non-cohesive)
3	Bedrock	Limestone: fine-grained; slightly fractured; moderate spacing; thin bedding; unweathered; strong rock

1. Fill was encountered in the borings to depths of 3 to 10 feet.
2. At the locations of B-1, the boring was extended deeper by means of coring methods in order to investigate the nature and quality of the underlying bedrock. The rock cores obtained at B-1 from 10 to 19 feet had recoveries ranging from 75 to 92 percent and RQD values ranging from 75 to 80 percent, indicating rock of good quality

Boring B-1 was noted to have a petroleum odor within samples recovered from a depth of about 6 to 8 feet below ground surface.

Groundwater Conditions

The boreholes were observed while drilling and after completion for the presence and level of groundwater. The water levels observed in the boreholes can be found on the boring logs in **Exploration Results**. Groundwater was observed in B-2 at a depth of 10 feet upon completion of sampling. In B-1 ground water was not encountered upon completion of drilling in the overburden soil; after rock coring in B-1, groundwater was noted at a depth of 7.5 feet; this is likely not indicative of the actual groundwater as water from external source was added to the borehole to facilitate coring.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Water may also become temporarily perched over low permeability layers. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GEOTECHNICAL OVERVIEW

The key geotechnical consideration impacting the proposed construction is the presence of nearly ten feet of uncontrolled fill at the site. Boring B-4 encountered a void space below a depth of 3 feet. We recommend that additional test pits be completed in proximity to B-4 to further investigate this portion of the site.

In our opinion, these fill materials are unsuitable in their current condition for direct support of the proposed building and floor slab due to the risk of greater than normal settlements associated with inherent variations in the nature and consistency of the fill. The underlying native materials are, in our opinion, suitable for foundation support.

To allow for the building to be constructed using conventional shallow foundations and slab-on-grade without the risk described above, it would be necessary to remove and replace the uncontrolled fill from the proposed building area or employ ground improvement methods such as aggregate piers or controlled-modulus columns (CMCs) that reinforce the fill layer in-place. Another option would be to support the entire building (foundations and structural floor slab) on a drilled shaft foundation system that derives its support from the bedrock underlying the native soil. This report provides recommendations for those three approaches.

Please note that ground improvement is normally conducted on a design-build basis by an experienced geotechnical specialty contractor because the designs, methods and/or equipment usually involve proprietary considerations. If this approach is selected, we recommend this report be provided to at least two specialty contractors and then meet with them to discuss ground improvement options and likely cost ranges. While we have specifically discussed with you the aggregate pier/stone column concept and believe that would be a technically suitable approach for this site, the specialty contractors may be able to identify other ground improvement options that would be more cost effective. Subsequently, if it is determined to proceed with the ground improvement option, we would be pleased to assist with evaluating and selecting the design-build contractor, including preparation of a performance-based design-build specification that would allow for specialty contractors to bid the project using their proprietary systems most effectively and efficiently.

Monitoring of the construction operations discussed herein will be critical in achieving the design subgrade support. We recommend Terracon be retained to evaluate soil bearing subgrades exposed after excavation to confirm they are suitable for footing, or slab support. Subsurface conditions in the explorations have been reviewed and evaluated with respect to the proposed construction plans known to us now.

The **General Comments** section provides an understanding of the report limitations.

EARTHWORK

Earthwork will include removal of topsoil or asphalt, removal of unsuitable fill (if encountered), excavations, and fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria as necessary to render the site in the state considered in our geotechnical engineering evaluation for foundations, and floor slabs.

Demolition

We understand that the site has been developed over the years. Although the borings did not encounter remains of buried structures (i.e. slabs; foundations; pavements; sidewalks; abandoned underground utilities; etc.), it is possible that such elements may be found during construction. Boring B-4 encountered 5 feet void space from 3 to 8 feet. The borings were completed at discrete locations and significant spacing (i.e. greater than 100 feet). This report does not reflect conditions that may occur between the borings investigated, or between sampling intervals in test borings. The nature and extent of variations between test borings and sampling intervals may not become evident until the course of construction. If encountered, removed buried structures should be backfilled with approved imported Structural Fill, which is placed and compacted in accordance with recommendations presented in this report. We recommend the following:

- Existing structures and utilities should be removed from beneath proposed foundations and floor slabs.
- Existing structures should be removed from proposed pavement areas to a minimum depth of 2 feet below of the pavement subgrade. Existing floor slabs (if left at a minimum depth of 2 feet below the bottom of pavement subgrade) should be broken up to promote drainage and minimize the potential for trapped water.
- Existing underground pipes may remain in-place if the top of the pipe is at least 2 ft below pavement subgrade or floor slab bottom and filled with Flowable Fill with a minimum compressive strength of 500 psi. Existing piping/structures should be disconnected from other existing piping intended to be left in place and functioning, and properly capped prior to placing Flowable Fill.

Site Preparation

Complete stripping of the topsoil (if encountered) should be performed in the proposed building and parking/driveway areas. Subgrades should be proof-rolled with a minimum 10-ton (static weight) smooth drum roller compactor. We recommend a minimum of two overlapping passes in one direction, followed by two overlapping passes in a direction perpendicular to the first passes. The intent is to compact areas with relatively loose surficial soil, to re-compact areas loosened by stripping operations, and to identify unacceptable subgrade areas. As an alternative, proof-rolling can also be performed with an adequately loaded vehicle such as a fully loaded tandem axle dump truck or other heavy, rubber-tired construction equipment weighing at least 20 tons.

Areas which excessively deflect under the proof-roll should be delineated and subsequently addressed by the Geotechnical Engineer. Unstable subgrades, as identified by the Geotechnical Engineer, should be over-excavated from the building footprint, footing bearing zones, and pavement areas to competent material and replaced with compacted Structural Fill.

Fill Material Types

Fill required to achieve design grade should be classified as Structural Fill and General Fill. Structural Fill is material used below, or within 10 feet of structures, pavements or constructed slopes. General Fill is material used to achieve grade outside of these areas. Earthen materials used for Structural and General Fill should meet the following material property requirements:

Fill Type[1]	USCS Classification	Acceptable Parameters
Structural Fill	GW, GW-GM, SW, SW-SM, SP, SM, GP	Liquid Limit less than 40, Plasticity Index less than 20, and less than 10% retained on No. 200 sieve
Aggregate Base/Subbase Course	GW, GW-GM, SW, SW-SM, SP, GP	NYSDOT, Subbase Course, Type 2. Less than 10% passing No. 200 sieve.
General Fill	GW, GP, GM, SW, SP, SM, CL, ML	General Fill may be used for general site grading; General Fill should not be used under settlement or frost-sensitive structures. General Fill should have a maximum particle size of 12 inches and no more than 40 percent by weight passing the No. 200 sieve.
Non-Frost Susceptible (NFS) Fill	GW, GP, SW, SP	NFS Fill should contain less than 5 percent material passing No. 200 sieve size.
Crushed Stone	GP	May be used to level subgrades at the bottom of trenches and to facilitate dewatering. Should be uniform ¾-inch angular Crushed Stone wrapped in a geotextile separation fabric (Mirafi 140N, or similar).
Lean Concrete	Not applicable	Lean Concrete should be flowable, self-compacting concrete with a compressive strength between 750 and 2,000 psi.

1. Compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to use on this site.

Excavated on-site soils are anticipated to consist primarily of existing fill with construction debris. We do not recommend reusing excavated on-site soils as Structural Fill.

It is our opinion that excavated non-organic soils (free of roots, oversized particles, large fragments of debris, and vegetation and other deleterious materials) may be suitable for reuse as General Fill to attain proposed subgrade elevation, provided material larger than 6 inches in size is removed, and that during construction proper compaction and optimum moisture content can be achieved. If construction is performed during the wet season, it is possible the moisture content of the excavated soils is in excess of the optimum moisture content required to achieve proper compaction, and that proper compaction of the on-site soils may be very difficult to achieve.

Saturated soils which cannot achieve compaction should be removed or used in non-structural areas where significant post construction settlement is acceptable. The contractor is ultimately responsible for moisture conditioning of fill/backfill materials to achieve proper compaction.

Fill Compaction Requirements

Structural and General fill should meet the following compaction requirements.

Item	Structural Fill	General Fill
Maximum Lift Thickness	12 inches or less in loose thickness when heavy, self-propelled compaction equipment is used. 6 to 8 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used.	Same as Structural fill
Minimum Compaction Requirements [1, 2, 3]	95% of maximum dry density below foundations and within 1 foot of finished pavement subgrade 92% of max. above foundations, below floor slabs, and more than 1 foot below finished pavement subgrade	90% of max.
Water Content Range [1]	Workable moisture levels	As required to achieve min. compaction requirements

1. Maximum density and optimum water content as determined by the modified Proctor test (ASTM D 1557).
2. We recommend testing fill for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested, as required, until the specified moisture and compaction requirements are achieved. The zone of fill compacted to meet this criterion should extend at least 5 feet horizontally beyond the building footprint.
3. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D 4253 and D 4254).

Utility Trench Backfill

Trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. Trenches should be backfilled with material that approximately matches the permeability characteristics of the surrounding soil to reduce the infiltration and preferential conveyance of surface water through the trench backfill. Fill placed as backfill for utilities located below the slab should consist of compacted Structural Fill or suitable bedding material.

Utility trenches are a common source of water infiltration and migration. All utility trenches that penetrate beneath the building should be effectively sealed to restrict water intrusion and flow through the trenches, which could migrate below the building. The trench should provide an effective trench plug that extends at least 5 feet out from the face of the building exterior. The plug material should consist of cementitious flowable fill or low permeability clay. The trench plug material should be placed to surround the utility line. If used, the clay trench plug material should be placed and compacted to comply with the water content and compaction recommendations for Structural Fill stated previously in this report.

Grading and Drainage

All grades must provide effective drainage away from the building during and after construction and should be maintained throughout the life of the structure. Water retained next to the building can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge onto splash blocks at a distance of at least 10 feet from the building.

Exposed ground should be sloped and maintained at a minimum 5 percent away from the building for at least 10 feet beyond the perimeter of the building. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structure should also be periodically inspected and adjusted, as necessary, as part of the structure’s maintenance program. Where paving or flatwork abuts the structure, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Earthwork Construction Considerations

Shallow excavations for the proposed structure are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of floor slabs. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to floor slab construction.

The groundwater table could affect over-excavation efforts, especially for over-excavation and replacement of lower strength soils. Dewatering may be required during excavation for the building foundations and for utility construction based on the conditions encountered at the time of drilling. The contractor should select a dewatering method to lower groundwater at least 2 feet below the excavation subgrade in order to minimize bearing surface disturbance during construction of footings and utilities. It is our opinion conventional sump pumping techniques could be implemented to minimize water ponding in the excavation area.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, “Excavations” and its appendices, and in accordance with any applicable local, and/or state regulations. The contractor should be aware that slope height, slope inclination, and excavation depth should in no instance exceed OSHA guidelines. OSHA guidelines are strictly enforced and if they are not followed, the owner, contractor, and/or earthwork and utility subcontractor could be liable and subject to substantial penalties.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation of adequate removal of vegetation and topsoil, proofrolling, and mitigation of areas delineated by the proofroll to require mitigation.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 2,500 square feet of compacted fill in the building areas and 5,000 square feet in pavement areas. One density and water content test should be performed for every 50 linear feet of compacted utility trench backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated under the direction of the Geotechnical Engineer. If unanticipated conditions are encountered, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer’s evaluation of subsurface conditions, including assessing variations and associated design changes.

SHALLOW FOUNDATIONS

The proposed structures may be supported on conventional spread footings bearing on imported Structural Fill placed upon native soils or on improved soil, as discussed in section **Ground Improvement** of this report.

Design Parameters – Compressive Loads

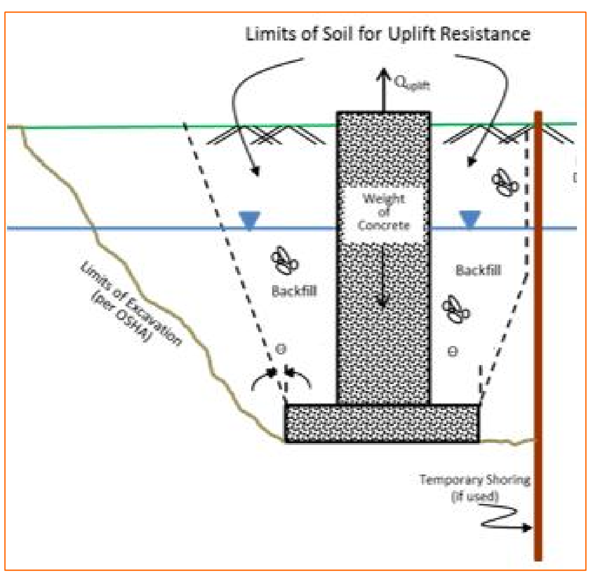
Item	Description
Maximum Net Allowable Bearing Pressure [1, 2]	■ A value of 3,000 psf may be used for shallow foundation installed upon a minimum of 12 inches of compacted Structural Fill placed upon native soil. ■ A value of 4,000 to 6,000 psf may likely be used for shallow foundation installed upon improved soil. The final design bearing capacity for the improved soils should be determined by the design-build ground improvement contractor.
Required Bearing Stratum [3]	■ Minimum 12 inches compacted Aggregate Base placed upon proof-rolled/compacted native soil. The Aggregate Base should extend a minimum lateral distance of 12 inches from the edge of the foundations. ■ *In-situ* improved soil or structural fill placed above improved soil
Minimum Foundation Dimensions	Columns: 30 inches Continuous: 18 inches
Ultimate Passive Resistance [4] **(equivalent fluid pressures)**	390 pcf (compacted Structural Fill)
Ultimate Coefficient of Sliding Friction [5]	0.50 (footing on compacted Structural Fill)
Minimum Embedment below Finished Grade [6]	Exterior footings in unheated areas: 48 inches Exterior footings in heated areas: 48 inches Interior footings in heated areas: 18 inches
Estimated Total Settlement from Structural Loads [2]	■ Less than about 1 inch for foundations bearing upon imported Structural Fill placed upon native soil ■ Ground improvement should be designed to allow for a maximum of 1 inch of total settlements. The final design total settlements for foundations placed upon improved soils should be determined by the design-build ground improvement contractor
Estimated Differential Settlement [2, 7]	■ About 2/3 of total settlement for foundations bearing upon imported Structural Fill placed upon native soil ■ Ground improvement should be designed to allow for a maximum of 0.5 inch of differential settlements. The final design differential settlements for foundations placed upon improved soils should be determined by the design-build ground improvement contractor

Item	Description

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. An appropriate factor of safety has been applied. These bearing pressures can be increased by 1/3 for transient loads unless those loads have been factored to account for transient conditions. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.
2. Values provided are for maximum loads noted in **Project Description**.
3. Unsuitable or soft soils should be over-excavated and replaced per the recommendations presented in the **Earthwork**.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted Structural Fill be placed against the vertical footing face. The Structural Fill must extend out and up from the base of the foundation at an angle of at least 60 degrees from vertical for the passive case.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Should be neglected for foundations subject to net uplift conditions.
6. Embedment necessary to minimize the effects of frost and/or seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.
7. Differential settlements are as measured over a span of 50 feet.

Design Parameters - Uplift Loads

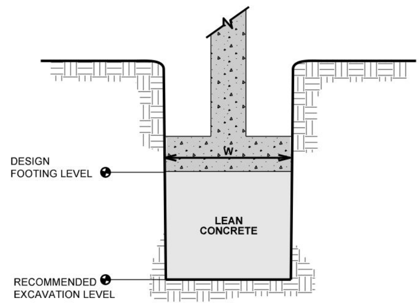
Uplift resistance of spread footings can be developed from the effective weight of the footing and the overlying soils. As illustrated on the subsequent figure, the effective weight of the soil prism defined by diagonal planes extending up from the top of the perimeter of the foundation to the ground surface at an angle, q, of 20 degrees from the vertical can be included in uplift resistance. The maximum allowable uplift capacity should be taken as a sum of the effective weight of soil plus the dead weight of the foundation, divided by an appropriate factor of safety. A maximum total unit weight of 120 pcf should be used for the backfill. This unit weight should be reduced to 60 pcf for portions of the backfill or natural soils below the groundwater elevation.



Foundation Construction Considerations

As noted in **Earthwork**, the footing excavations should be evaluated under the direction of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

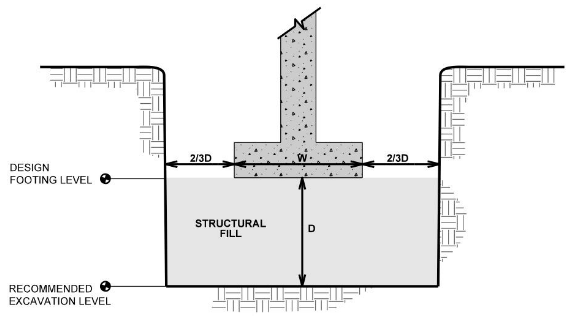
If unsuitable bearing soils are encountered at the base of the planned footing excavation, the excavation should be extended deeper to suitable soils, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. This is illustrated on the sketch below.



LEAN CONCRETE BACKFILL

NOTE: EXCAVATIONS ARE SHOWN VERTICAL; HOWEVER, THE SIDEWALLS SHOULD BE SLOPED AS NECESSARY FOR SAFETY

Over-excavation for structural fill placement below footings should be conducted as shown below. The over-excavation should be backfilled up to the footing base elevation, with imported Aggregate Base placed, as recommended in the **Earthwork** section.



OVER-EXCAVATION / BACKFILL ZONE

NOTE: EXCAVATIONS ARE SHOWN VERTICAL; HOWEVER, THE SIDEWALLS SHOULD BE SLOPED AS NECESSARY FOR SAFETY

GROUND IMPROVEMENT

As an alternative to supporting the structure on deep foundations or replacing the fill, the shallow foundations and floor slabs could be supported on existing fill and lower strength/lower density native soils after ground improvement has been performed to densify the existing fill and loose natural soils at the site. Ground improvement methods are proprietary systems designed by licensed contractors who design and implement the ground improvement system to meet the project needs.

Ground improvement systems are normally proprietary in the design and installation and are installed by geotechnical specialty contractors who retain their own engineer to produce, sign, and seal the design, which effectively results in a design-build scenario for the ground improvement portion of the project. Accordingly, the design-builder should be provided a copy of this report and the settlement tolerances of the building to be able to produce a cost-effective design for the project. Installation of the ground improvement system is usually conducted following the stripping and utility relocation, but prior to subgrade preparation activities and placement and compaction of structural fill below the building.

Following installation of the ground improvement system, the slab and foundation subgrades should be prepared in accordance with the ground improvement designer's requirements. This may include placement of a compacted fill layer between the slab and the ground improvement elements.

The ground improvement should extend at least 10 feet beyond the building footprint. Shallow spread footings founded on the improved ground may be typically designed with allowable bearing pressures of 4,000 psf to 6,000 psf.

Ground Improvement Systems

Several ground improvement systems could be considered for the proposed construction and are discussed in the following paragraphs.

Dense Aggregate Piers

DAPs consist of installing a compacted stone pier that is typically on the order of 24 to 36 inches in diameter. Layers of ¾-inch clean crushed stone or well graded aggregate, approximately 24 inches thick, are typically installed and compacted with a high energy ram. The design of the DAP system, including the element diameter, element spacing, and the lateral extent of the element grid beyond the limits of the overlying structure are all interdependent design factors and will need to be determined by the design-build contractor installing the piers. At a minimum, we recommend that the elements be advanced a minimum of 2 times the diameter of the element (i.e. 4 feet for a 24-inch diameter element or 6 feet for a 36-inch diameter element) into the native soil to ensure that they are firmly embedded in a suitable bearing stratum or a total length of 10 feet, whichever is deeper. However, the contractor's engineer will need to evaluate as part of the design whether the elements need to extend further below this level to obtain the required settlement performance. Following construction of the piers, the structure could then be supported via conventional spread foundations.

Based upon our review of the data collected for this geotechnical investigation, it is our opinion that this approach to ground improvement would be a viable solution for the proposed construction for this site, based on the anticipated loads of the structure and the subsurface conditions encountered at the site. Final design of the DAP system should be reviewed by Terracon for consistency with our understanding of subsurface conditions at the site.

Controlled Modulus Columns

Controlled-modulus columns (CMCs) or rigid inclusion elements (RIEs) are another ground improvement alternative. These inclusions are installed on a grid pattern utilizing a specialized boring tool and extend to an adequate deep layer. Once at the specified tip elevation within the bearing soils, the elements are formed with or without soil displacement by low-pressure injection (generally up to 30 psi) of a grout or concrete through the hollow core of the drilling tool. Concrete or grout is pumped through the auger stem and discharges from the auger tip as it is raised. The auger is extracted while a positive concrete head is maintained. The concrete/grout fills the void created by the auger during extraction. These RIEs and CMCs generally have a diameter between 12 and 18 inches.

Construction Considerations

- The ground improvement installer should have a minimum of 5 years of experience installing the proposed ground improvement system for projects of similar size and complexity.
- The installer of the ground system should provide detailed design calculations sealed by a professional engineer licensed in the New York. The design calculations should demonstrate the ground improvement system is estimated to control long-term total and differential settlements to that required for the project. The installer should warrant their work as well as the maximum total and differential settlements they predict.
- A detailed work plan should be prepared by the ground improvement installer and should be reviewed and approved by Terracon prior to implementation of ground improvement operations.
- As part of the ground improvement installation, an in-depth QA/QC program should be implemented. Terracon should be provided the opportunity to assist with the review of the specifications for this work to confirm that appropriate quality control steps are implemented.

DEEP FOUNDATIONS

Drilled Shaft Design Parameters

The building structure and floor slab (structural slab) may also be supported by drilled shafts bearing upon competent bedrock, generally encountered in the borings at depths of 10 to 14 feet below existing ground surface. We recommend a minimum shaft diameter of 30 inches be used in order to permit cleaning and observation of the shaft bottoms. The design parameters provided in the table below should be used for the design of drilled shaft foundations.

Description	Value
Net Allowable Bearing Capacity (≥ 10 feet deep) [1] Bedrock	 30 ksf
Estimated Total Settlement	negligible
Estimated Soil Total Unit Weight Native Soil/Fill Bedrock	 120 pcf 150 pcf
Allowable Side Friction Resistance Fill/Native Soil Bedrock	 neglect 6.0 ksf

1. Contribution to shaft capacity from soils within the upper 4 feet should be ignored. Side friction should be applied to the surface area of the designed shaft from the 4-foot depth to the bottom of the shaft. The values presented for allowable end bearing include a factor of safety of approximately 3.

Tensile reinforcement should extend to the bottom of shafts subjected to uplift loading. Buoyant unit weights of the soil and concrete should be used in the calculations below the highest anticipated groundwater elevation.

Drilled shaft should have a minimum (center-to-center) spacing of three diameters. Closer spacing may require a reduction in axial load capacity. Axial capacity reduction can be determined by comparing the allowable axial capacity determined from the sum of individual piles in a group versus the capacity calculated using the perimeter and base of the pile group acting as a unit. The lesser of the two capacities should be used in design.

Drilled Shaft Construction Considerations

We recommend the following guidelines be incorporated into the technical specifications for drilled shaft installation:

- The contractor should take measures to prevent collapse of the shaft during excavation and prior to concreting. Full-depth temporary casing should be placed for the shaft excavation into the overburden soil. If casing is removed during concrete placement, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures present on a casing exterior.

- The conditions of the bearing surface at the drilled shaft location should be observed by the geotechnical engineer or his representative.

1. Since the shaft is designed to bear on bedrock, confirmation that the shaft is supported on competent bedrock and not just a boulder or shelf rock over soil is important. The Contractor shall be prepared to use a core barrel to extend the shaft a few feet deeper into bedrock when unusual conditions are encountered or when doubt exists. Direction regarding advancing the drilled shaft deeper to verify bedrock has been encountered shall be given by the geotechnical engineer during drilling activities.
2. The bottom of the shaft should be prepared with a cleanout bucket; no more than 1-inch of spoil material over the entire base shall be allowed.

- Reinforcing steel cage placement must be done in a controlled manner to ensure accurate placement and appropriate concrete cover is achieved. The reinforcing cage shall be placed and centered in the hole for the shaft prior to concreting. Centralizers should be installed at the bottom and along the axial length of the steel reinforcing at sufficient spacing to maintain at least 3 inches of concrete cover, but at a spacing that does not exceed 10 feet. Bottom support (i.e. cylindrical feet) made of a material that is not detrimental to the reinforcement or concrete, should also be placed at the bottom of the cage to ensure the bottom of the cage is maintained at the proper elevation.
- Reinforcing cage and concrete should be placed within 2 hours after the drilled shaft has been excavated, cleaned out, inspected, and accepted by the geotechnical engineering representative. Concreting should be one continuous operation to avoid cold joints.
- Concrete may be dropped into the drilled shaft without segregation, provided there is less than 3 inches of water in the hole, and concrete is not allowed to bounce off the side of the shaft hole. The impact of the falling concrete should be adequate to provide densification, and vibration is generally not required, with the exception of approximately the last 5 feet, where the height of fall is limited.
- The shaft hole should be maintained relatively dry (i.e. less than 3 inches of water in the hole) prior to placement of the concrete. If this is difficult, concrete may be placed using tremie methods. If a tremie is used, the end of the tremie must be at least 5 feet below the surface of the concrete to prevent the water from contaminating the fresh concrete. For the concrete to pass freely through the tremie, the minimum diameter of the tremie shall be 10 inches. The tremie should be clean, smooth, and free of built-up concrete and other foreign material.
- Construction operations which may cause soil movement immediately adjacent (within 5 feet) to the drilled shaft shall be avoided for a minimum of 24 hours after completing the shaft concrete pour.
- Terracon should be retained to observe the drilled shaft excavation to evaluate the suitability of the bearing materials and to verify conditions in the drilled shaft excavation

are consistent with those encountered in the test borings. If unsuitable materials are encountered at planned depths, it may be necessary to deepen the shaft.

SEISMIC CONSIDERATIONS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the soil/bedrock properties encountered at the site and as described on the exploration logs and results, it is our professional opinion that the **Seismic Site Classification is D**. Subsurface explorations at this site were extended to a maximum depth of 19 feet. The site properties below the boring depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. Additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring depth.

FLOOR SLABS

If the Owner elects to support the floor slab on drilled shafts, we recommend that the floor slab be designed as a structural slab, supported by the deep foundation system. We recommend that the structural floor slab bear on a minimum of 6 inches of Aggregate Base material in order to provide short term support of the floor slab until sufficient setting (and strength) of the concrete has occurred. After installation of the drilled shafts and/or utilities, ruts created by construction traffic in the Aggregate Base course material should be regraded prior to forming for the structural slab. It is highly recommended that concrete for the structural slab be pumped in-place from outside the building perimeter.

If the owner elects to construct floor slabs on imported Structural Fill placed upon native soils, exposed subgrades beneath proposed floor slab areas should be proof-rolled as discussed in **Site Preparation**.

Design parameters for floor slabs assume the requirements for **Earthwork** and/or **Ground Improvement** have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the floor slab.

Floor Slab-on-Grade Design Parameters

Item	Description
Floor Slab Support [1]	■ Minimum 12 inches of Aggregate Base material compacted to at least 95% of Modified Proctor (ASTM D 1557) placed directly upon proofrolled stable on-site native soils. ■ Minimum 12 inches[3] of imported Slab Aggregate Base compacted to at least 90% of Modified Proctor (ASTM D 1557) placed directly upon proofrolled improved soils.
Estimated Modulus of Subgrade Reaction [2,4]	100 pounds per square inch per inch (psi/in) for point loads

1. Floor slabs should be structurally independent of building footings or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
2. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the floor slab support as noted in this table. It is provided for point loads. For large area loads the modulus of subgrade reaction would be lower.
3. Based upon the ground improvement system, the minimum Slab Aggregate Base thickness may be revisited and potentially reduced. The final aggregate base thickness for the floor slab bearing upon improved soils should be determined by the design-builder ground improvement contractor. For example, the material used for the load transfer platform (if required) may contribute toward the aggregate base required beneath slab.
4. Based upon the ground improvement system, the Modulus of Subgrade Reaction may be revisited and potentially increased. The final Modulus of Subgrade Reaction for the floor slab bearing upon improved soils should be determined by the design-builder ground improvement contractor.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or cracks should be sealed with a water-proof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

Floor Slab Construction Considerations

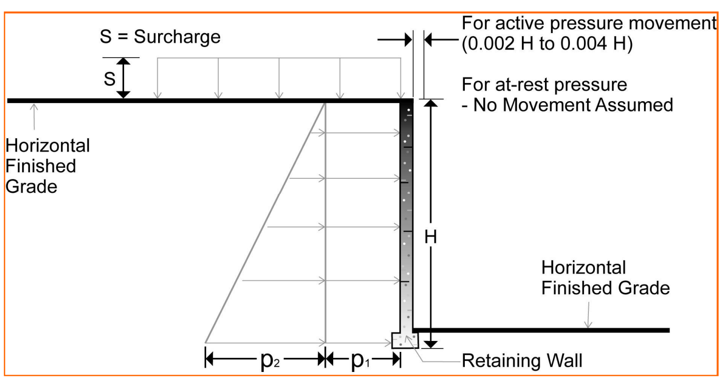
Finished subgrade within and for at least 10 feet beyond the floor slab should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed, and Structural Fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

The Geotechnical Engineer should approve the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

LATERAL EARTH PRESSURES

Design Parameters

Structures with unbalanced backfill levels on opposite sides (i.e. loading dock walls) should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



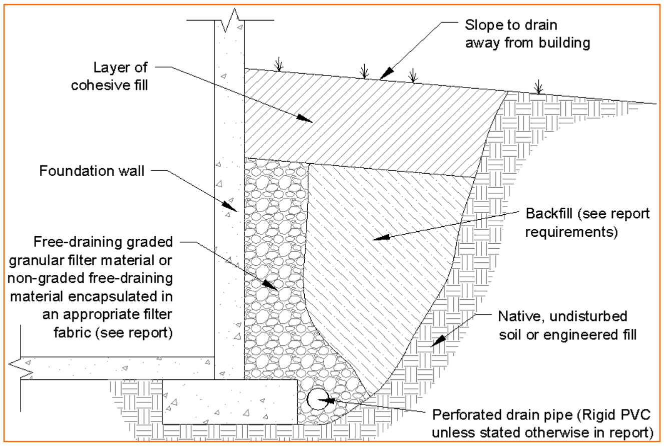
Lateral Earth Pressure Design Parameters			
Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3, 4, 5] p_1 (psf)	Effective Fluid Pressures (psf) [2, 4, 5] Drained
Active (Ka)	Granular - 0.31	(0.31)S	(37)H
At-Rest (Ko)	Granular - 0.47	(0.47)S	(56)H
Passive (Kp)	Granular - 3.25	--	(390)H

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance.
2. Uniform, horizontal backfill, compacted to at least 95 percent of the ASTM D 1557 maximum dry density, rendering a maximum unit weight of 120 pcf and friction angle of 32 degree.
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included. Hand operated equipment should be used within 4 feet of back of wall.
5. No safety factor is included in these values.

Backfill placed against structures should consist of granular soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active and passive cases, respectively.

Subsurface Drainage for Below Grade Walls

A perforated rigid plastic drain line installed behind the base of loading dock walls and extends below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around a below-grade building area or exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material such as NYSDOT Item 203.21 Select Structural Fill; however, we recommend less than 5% passing the No. 200 sieve. The Select Structural Fill should be encapsulated in a filter fabric. The granular fill should extend to within 2 feet of final grade, where it should be capped with rigid or flexible pavement or low permeable fill to reduce infiltration of surface water into the drain system.



As an alternative to Select Structural Fill, a pre-fabricated drainage structure may be used. A pre-fabricated drainage structure is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

PAVEMENTS

General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in **Project Description** and in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs, noted in this section, must be applied to the site, which has been prepared as recommended in the **Earthwork** section.

Pavement Design Parameters

Pavement designs were based on *AASHTO Guide for Design of Pavement Structures (1993)* and our experience with similar projects. The thickness of each course is a function of subgrade strength, traffic, design life, serviceability factors, and frost susceptibility.

A subgrade CBR of 3 was used for the AC pavement designs, and a modulus of subgrade reaction of 100 pci was use for the PCC pavement designs. The values were empirically derived based upon our experience with the on-site soils and our understanding of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**. A modulus of rupture of 600 psi was used for pavement concrete.

Pavement Section Thicknesses

Frost susceptibility is a major factor in the overall pavement section thickness. The total pavement structural sections presented in this report are based also upon the expected depth of freeze, which for the project site is anticipated at 48 inches. Based on local field data and experience, and provided positive pavement drainage is maintained, we anticipate that the minimum pavement structural sections presented in the following sections are required to minimize pavement heave and keep cracking within tolerable amounts.

As a result of the fine-grained nature of the in-situ soil, and possible variations across the site of subgrade material (i.e. existing fill; native soils; compacted Structural Fill), we recommend a separation high-strength woven geotextile be placed upon all new approved flexible pavement subgrades prior to placing the subbase course materials. A Mirafi RS280i or HP370 or approved equivalent may be used. These geotextiles should provide separation (i.e. mitigate migration of fines into the overlying subbase course material, which may contribute to its degradation and loss of strength), filtration (i.e. allows for movement of water across the plane of the geotextile with limited soil loss), confinement (i.e. will restrain lateral movement of the aggregate), and reinforcement.

The following table provides options for Asphaltic Concrete and for Portland Cement Sections:

Asphaltic Concrete Design		
Layer	**Thickness (inches)**	
	Light Duty [1]	**Heavy Duty** [1]
Asphalt Top Course [2]	1.5	1.5
Asphalt Binder Course [2]	2.0	3.5
Aggregate Base Course [2]	12.0	12.0

1. See **Project Description** for more specifics regarding pavement type.
2. All materials should meet the current NYSDOT Department of Transportation (NYSDOT) Standard Specifications.
   - Asphalt Top Course – NYSDOT Section 402 for Type 12.5 F2 Top Course HMA, Item No. 402.127202
   - Asphalt Binder Course – NYSDOT Section 402 for Type 19 F9 Binder Course HMA, Item No. 402.197902
   - Aggregate Base Course – NYSDOT Section 304 for Type 2 Subbase Course, Item No. 304.12

Portland Cement Concrete Design		
Layer	**Thickness (inches)**	
	Light Duty [1]	**Heavy Duty** [3]
PCC [2]	6.0	8.0
Aggregate Base [3]	12.0	12.0

1. See **Project Description** for more specifics regarding traffic classifications.
2. All materials should meet the current State, County, and City Department of Transportation (NYSDOT) Standard Specifications for Highway and Bridge Construction.
   - Concrete Pavement - NYSDOT Portland Cement Concrete Section 502, with a minimum compressive strength of 4,000 psi at 28 days.
   - Aggregate Base Course – NYSDOT Section 304 for Type 2 Subbase Course, Item No. 304.12
3. In areas of anticipated heavy traffic, fire trucks, delivery trucks, or concentrated loads (e.g. dumpster pads), and areas with repeated turning or maneuvering of heavy vehicles.

The estimated pavement sections provided in this report are minimums for the assumed design criteria, and as such, periodic maintenance should be expected. Areas for parking of heavy vehicles, concentrated turn areas, and start/stop maneuvers could require thicker pavement sections. Edge restraints (i.e. concrete curbs or aggregate shoulders) should be planned along curves and areas of maneuvering vehicles. A maintenance program that includes surface sealing, joint cleaning and sealing, and timely repair of cracks and deteriorated areas will increase the pavement's service life. As an option, thicker sections could be constructed to decrease future maintenance.

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase. Subdrains should be sloped to provide positive gravity drainage to reliable discharge points. Periodic maintenance of subdrains is required for long-term proper performance.

Openings in pavements, such as decorative landscaped areas, are sources for water infiltration into surrounding pavement systems. Water can collect in the islands and migrate into the surrounding subgrade soils thereby degrading support of the pavement. This is especially applicable for islands with raised concrete curbs, irrigated foliage, and low permeability near-surface soils. The civil design for the pavements with these conditions should include features to restrict or to collect and discharge excess water from the islands. Examples of features are edge drains connected to the storm water collection system, longitudinal subdrains, or other suitable outlet and impermeable barriers preventing lateral migration of water such as a cutoff wall installed to a depth below the pavement structure.

Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic maintenance should be anticipated. Therefore, preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Maintenance consists of both localized maintenance (e.g. crack and joint sealing and patching) and global maintenance (e.g. surface sealing). Preventive maintenance is usually the priority when implementing a pavement maintenance program. Additional engineering observation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur and repairs may be required.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Final grade adjacent to paved areas should slope down from the edges at a minimum 2%
- Subgrade and pavement surfaces should be properly sloped to promote proper surface drainage
- Drainage systems should be installed below pavements where surrounding areas are anticipated to be wet frequently
- Joint sealant and seal cracks should be installed immediately
- Seal all landscaped areas in or adjacent to pavements to reduce moisture migration to subgrade soils

FROST CONSIDERATIONS

The soils on this site are frost susceptible, and small amounts of water can affect the performance of the slabs on-grade, sidewalks, and pavements. Exterior slabs should be anticipated to heave during winter months. If frost action needs to be eliminated in critical areas, we recommend the use of non-frost susceptible (NFS) fill or structural slabs (for instance, structural stoops in front of building doors). Placement of NFS material in large areas may not be feasible; however, the following recommendations are provided to help reduce potential frost heave:

- Provide surface drainage away from the building and slabs, and toward the site storm drainage system.
- Install drains around the perimeter of the building, stoops, below exterior slabs, and connect them to the storm drainage system.

- Grade clayey subgrades, so groundwater potentially perched in overlying more permeable subgrades, such as sand or aggregate base, slope toward a site drainage system.
- Place NFS fill as backfill beneath slabs and pavements critical to the project.
- Place a 3 horizontal to 1 vertical (3H:1V) transition zone between NFS fill and other soils.
- Place NFS materials in critical sidewalk areas.

As an alternative to extending NFS fill to the full frost depth, consideration can be made to placing extruded polystyrene or cellular concrete under a buffer of at least 2 feet of NFS material.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

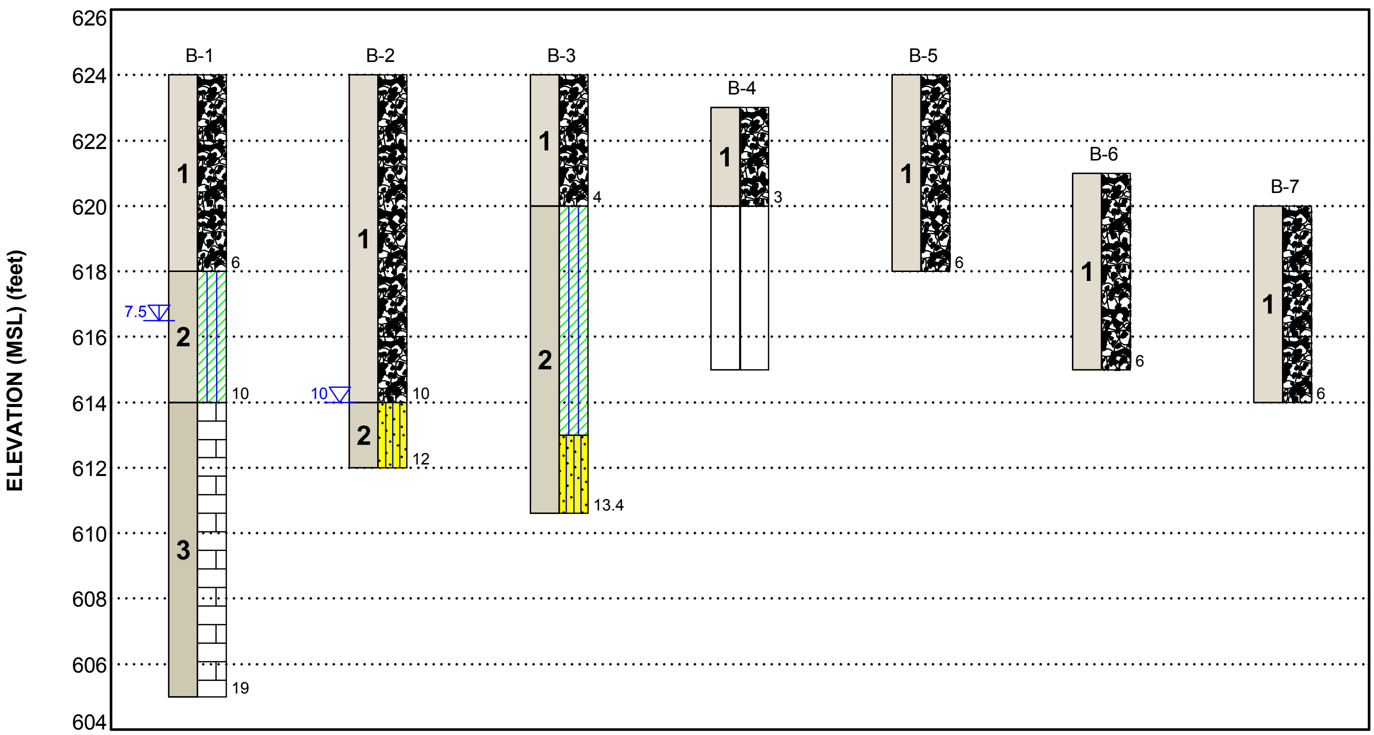
FIGURES

Contents:

GeoModel

GEOMODEL

Emmanuel Temple SDA Church ■ Buffalo, NY
Terracon Project No. J5195120



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Fill	Silty Sand with Gravel; concrete rubble; numerous brick fragments, trace wood fragments; black and brown
2	Native Soil	Silty Clay and Silty Sand (CL-ML; SM); trace gravel and rock fragments; medium stiff (cohesive) and medium dense (non-cohesive)
3	Bedrock	Limestone: fine-grained; slightly fractured; moderate spacing; thin bedding; unweathered; strong rock

LEGEND

Fill

Silty Sand

Silty Clay

Limestone

First Water Observation

Second Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

Number of Borings	Boring Depth (feet)	Planned Location
4	8 to 19	Planned building area
3	6	Planned pavement

Boring Layout and Elevations: Unless otherwise noted, Terracon personnel provided the boring layout. Coordinates were obtained with a handheld GPS unit (estimated horizontal accuracy of about ±10 feet) and approximate elevations were obtained from Google Earth. If elevations and a more precise boring layout are desired, we recommend borings be surveyed following completion of fieldwork.

Subsurface Exploration Procedures: We advanced the borings with a ATV-mounted rotary drill rig using continuous hollow stem flight augers. Samples were obtained continuously using a split-spoon sampler to the terminus of each borings. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the middle 12 inches of a normal 24-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. We observed and recorded groundwater levels during drilling and sampling. For safety purposes, all borings were backfilled with auger cuttings after their completion.

When auger refusal was encountered upon bedrock, rock cores were obtained at B-1 to investigate the nature and quality of the underlying bedrock. The percent recovery and the Rock Quality Designation (RQD) for the recovered sample were recorded. The percent recovery is the ratio of the length of rock recovered over the length of coring. The RQD is the ratio of the sum of the length of recovered rock core 4 inches or greater in length, over the length of rock core recovered. The RQD is useful is providing a qualitative and quantitative evaluation of the engineering quality of bedrock.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory. Based on the material's texture and plasticity, we described and classified the soil samples in accordance with the Unified Soil Classification System.

Rock classification was conducted using locally accepted practices for engineering purposes; petrographic analysis may reveal other rock types. Rock core samples typically provide an improved specimen for this classification. Boring log rock classification was determined using the Description of Rock Properties.

Infiltration Testing

Three boreholes were advanced to a depth of approximately 40 to 46 inches below the existing ground surface. IT-01 was advanced near B-5, IT-02 near B-6, and IT-03 near B-7. A 4-inch diameter PCV pipe was then installed at each location for infiltration testing. The tests were performed in general accordance with NYDEC Stormwater Management Design Manual - Appendix D. The results of the infiltration testing are included in **Exploration Results**.

PHOTOGRAPHY LOG

	
Fig.1 Facing East	Fig. 2: Facing North
	
Fig. 3: Facing South	Fig. 4: Facing North -West

SITE LOCATION AND EXPLORATION PLANS

Contents:

Note: All attachments are one page unless noted above.

SITE LOCATION

Emmanuel Temple Seventh Day Adventist Church ■ Buffalo, NY

October 3, 2019 ■ Terracon Project No. J5195120

Terracon GeoReport

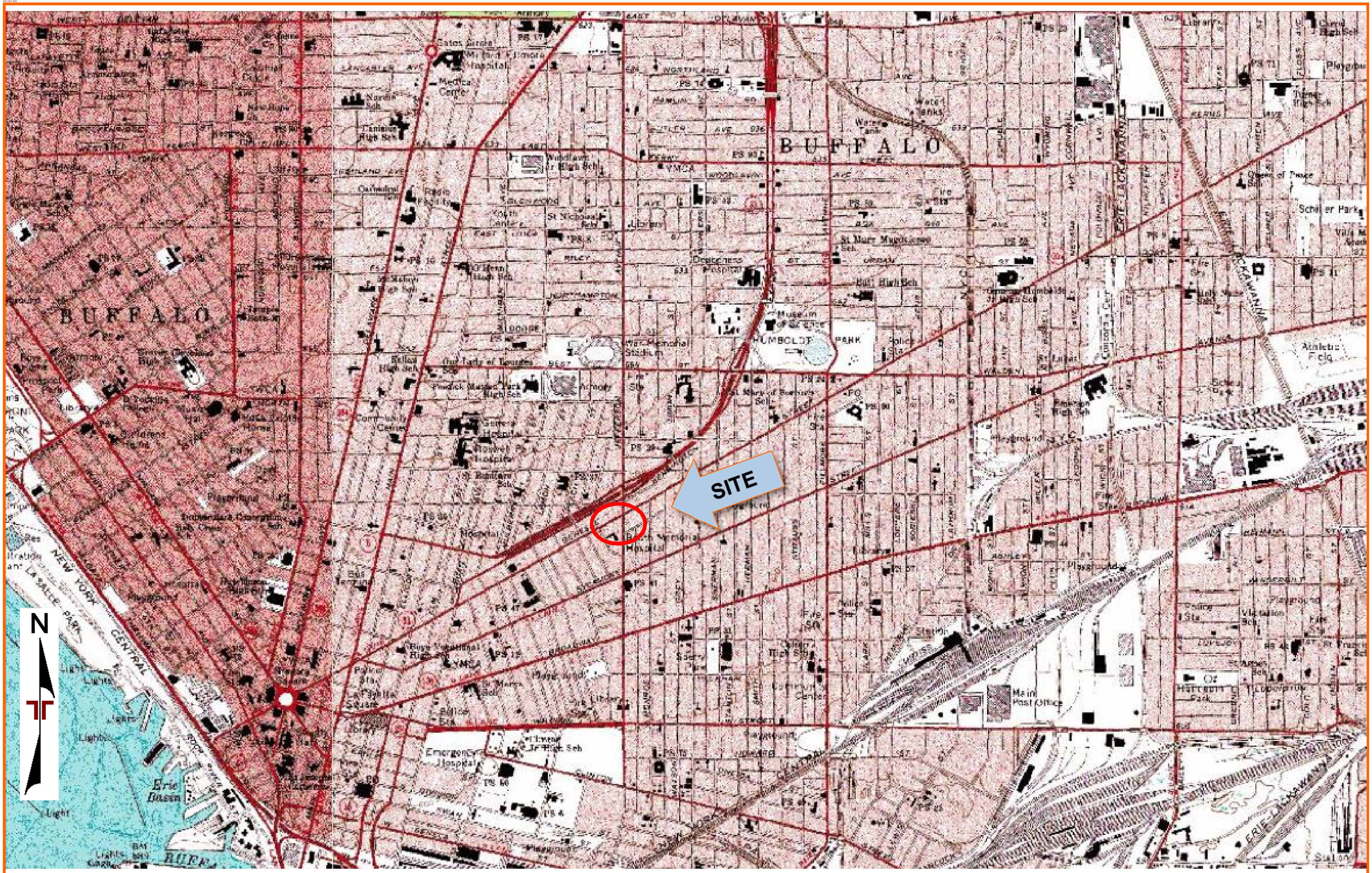


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

EXPLORATION PLAN

Emmanuel Temple Seventh Day Adventist Church ■ Buffalo, NY

October 3, 2019 ■ Terracon Project No. J5195120

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Boring Logs (B-1 through B-7)
Infiltration Test Results

Note: All attachments are one page unless noted above.

BORING LOG NO. B-1

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave Buffalo, NY

LOCATION See Exploration Plan

Latitude: 42.8957° Longitude: -78.8547°

Approximate Surface Elev.: 624 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
1		2.0	**FILL - SILTY SAND WITH GRAVEL**, numerous brick fragments, black	622+/-				10	3-6-9-5 N=15	
		6.0	**FILL - CONCRETE RUBBLE**, gray	618+/-	5			4	13-25-20-11 N=45	
								8	4-5-6-6 N=11	
2		10.0	**SILTY CLAY (CL-ML)**, trace sand, occasional silt partings, red brown, medium stiff, slight petroleum odors noted	614+/-	10	▽		18	4-3-5-5 N=8	
									6-3-5-50/1" N=8	
3		19.0	**LIMESTONE**, gray, fine-grained, slightly fractured, moderate spacing, thin bedding, unweathered, strong rock, Contains occasional chert nodules, fossiliferous	605+/-	15			55	RUN # 1 10.0' - 15.0'	80
								36	RUN # 2 15.0' - 19.0'	75
			Boring Terminated at 19 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

WATER LEVEL OBSERVATIONS

None encountered prior to coring

▽ *7.5' BGS after coring*

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

BORING LOG NO. B-2

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates
Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave
Buffalo, NY

LOCATION See Exploration Plan

Latitude: 42.8959° Longitude: -78.8543°

Approximate Surface Elev.: 624 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)
1	0.0 – 8.0	**FILL - SILTY SAND WITH GRAVEL**, black and brown Contains trace cinders, trace brick fragments, trace concrete fragments	616+/-
1	8.0 – 10.0	**FILL - SILTY SAND**, trace gravel, gray and black, saturated	614+/-
2	10.0 – 12.0	**SILTY SAND (SM)**, trace gravel, trace limestone fragments, gray, medium dense	612+/-

Sample Spoon Penetration Refusal Encountered at 11.6' BGS. Auger Penetration Refusal Encountered at 12 Feet

DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
0 – 2		Split Spoon	3	2-3-4-6 N=7	
2 – 3.5		Split Spoon	12	10-8-50/5"	
4 – 6		Split Spoon	8	7-2-4-7 N=6	
6 – 8		Split Spoon	6	3-4-13-11 N=17	
8 – 10	▽ (10.0')	Split Spoon	2	3-5-6-5 N=11	
10 – 11.6		Split Spoon	12	5-6-13-50/1" N=19	

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

WATER LEVEL OBSERVATIONS

▽ *10.0' at Completion of Sampling*

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

BORING LOG NO. B-3

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates
Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave
Buffalo, NY

LOCATION See Exploration Plan

Latitude: 42.8955° Longitude: -78.8545°

Approximate Surface Elev.: 624 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
1			**FILL - SILTY SAND WITH GRAVEL**, trace brick fragments, trace cinders, black				X	12	6-8-8-5 N=16	
			becomes red-brown and black, contains trace wood fragments				X	8	3-3-4-7 N=7	
		4.0		620+/-						
2			**SILTY CLAY (CL-ML)**, trace sand, numerous silt partings, red brown, stiff to very stiff		5		X	6	9-5-5-5 N=10	
							X	18	5-6-10-10 N=16	
							X	18	5-6-9-9 N=15	
		11.0		613+/-	10		X	18	10-7-8-8 N=15	
			SILTY SAND (SM), trace gravel, trace limestone fragments, gray							
		13.4		610.5+/-				0	50/0"	
			Sample Spoon and Auger Penetration Refusal Encountered at 13.4 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

WATER LEVEL OBSERVATIONS

None encountered at completion of sampling

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

BORING LOG NO. B-4

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave Buffalo, NY

LOCATION See Exploration Plan

Latitude: 42.8956° Longitude: -78.8542°

Approximate Surface Elev.: 623 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH / DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
1		**FILL - CONCRETE**, gray				X	3	25-40-50/2"	
		3.0	620+/-						
		VOID SPACE, Driller notes encountering 5' void from 3' to 8' BGS		5					
		8.0	615+/-						
		Boring Terminated at 8 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

3 additional attempts to advance borehole encountered concrete at depths from grade to 3' below grade.

WATER LEVEL OBSERVATIONS

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

BORING LOG NO. B-5

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates
Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave
Buffalo, NY

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Latitude: 42.8956° Longitude: -78.8551° Approximate Surface Elev.: 624 (Ft.) +/- DEPTH / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
1		**FILL - SILTY SAND WITH GRAVEL** , with brick fragments, gray and brown			X	12	3-3-5-7 N=8	
					X	2	3-6-8-10 N=14	
		Contains trace concrete fragments	5		X	2	16-6-3-3 N=9	
		6.0 / 618+/-						
		Boring Terminated at 6 Feet						

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

WATER LEVEL OBSERVATIONS

None encountered at completion of sampling

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

BORING LOG NO. B-6

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates
Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave
Buffalo, NY

LOCATION See Exploration Plan

Latitude: 42.8953° Longitude: -78.8544°

Approximate Surface Elev.: 621 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
1		**FILL - SILTY SAND**, with brick fragments, brown and black					3	3-4-10-8 N=14	
							0	9-7-10-6 N=17	
		Becomes brown, contains trace wood fragments		5			2	7-8-9-9 N=17	
		6.0	615+/-						
		Boring Terminated at 6 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

WATER LEVEL OBSERVATIONS

None encountered at completion of sampling

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

BORING LOG NO. B-7

PROJECT: Emmanuel Temple SDA Church

CLIENT: Foit-Albert Associates Buffalo, NY

SITE: Corner of Genesee and Jeffeson Ave Buffalo, NY

LOCATION See Exploration Plan

Latitude: 42.8949° Longitude: -78.8543°

Approximate Surface Elev.: 620 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH / DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)
1		**FILL - SILTY SAND**, with brock fragments, black and brown				X	5	3-4-7-21 N=11	
		2.0	618+/-						
		FILL - BRICK FRAGMENTS, red brown				X		7-7-12-7 N=19	
		4.0	616+/-						
		FILL - SILTY SAND, trace gravel, brown and black		5		X		3-5-6-7 N=11	
		6.0	614+/-						
		Boring Terminated at 6 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
3.25 inch ID Hollow Stem Augers and 2 inch OD Split Barrel Sampler

Abandonment Method:

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth.

Notes:

WATER LEVEL OBSERVATIONS

None encountered at completion of sampling

Terracon
15 Marway Cir, Ste 2B
Rochester, NY

Boring Started: 08-26-2019	Boring Completed: 08-26-2019
Drill Rig: Mobile B-57	Driller: R. Brown
Project No.: J5195120	

INFILTRATION TEST DATA SUMMARY

Project: Emmanuel Temple Site
Weather: Overcast, light rain 60F
Presoak Date: 10/2/2019

Terracon Project No.: J5195120
Tester : J. Schneider
Test Date: 10/3/2019

Terracon GeoReport

Test Location	Test Depth	Soil Classification	Trial Number	Water Drop (inches)	Elapsed Time (hours)	Infiltration Rate (inches/hour)
IT-01	40"	Silty Sand with gravel, traces of clay, brick and wood fragments	1	24	1	24
			2	24	1	24
			3	24	1	24
			4	24	1	24
			Average infiltration rate for the four trials was 24 inches per hour. Infiltration rate of the final trial was 24 inches per hour.			
IT-02	46"	Silty Sand with gravel, traces of clay, brick and wood fragments	1	0	1	0
			2	1	1	1
			3	0.5	1	0.5
			4	0	1	0
			Average infiltration rate for the four trials was 0.37 inches per hour. Infiltration rate of the final trial was 0 inches per hour.			
IT-03	46"	Silty Sand with gravel, traces of clay, brick and wood fragments	1	10	1	10
			2	1	1	1
			3	6.5	1	6.5
			4	5.5	1	5.5
			Average infiltration rate for the four trials was 5.7 inches per hour. Infiltration rate of the final trial was 5.5 inches per hour.			
Testing was conducted in general accordance with Appendix D of the New York State Storm Water Management Design Manual.						

SUPPORTING INFORMATION

Contents:

Note: All attachments are one page unless noted above.

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Emmanuel Temple SDA Church ■ Buffalo, NY

Terracon Project No. J5195120

Terracon GeoReport

SAMPLING	WATER LEVEL	FIELD TESTS
Rock Core Standard Penetration Test	**Water Initially Encountered** **Water Level After a Specified Period of Time** **Water Level After a Specified Period of Time** Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	**N** Standard Penetration Test Resistance (Blows/Ft.) **(HP)** Hand Penetrometer **(T)** Torvane **(DCP)** Dynamic Cone Penetrometer **UC** Unconfined Compressive Strength **(PID)** Photo-Ionization Detector **(OVA)** Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (tsf)**	**Standard Penetration or N-Value Blows/Ft.**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

RELATIVE PROPORTIONS OF SAND AND GRAVEL		RELATIVE PROPORTIONS OF FINES	
Descriptive Term(s) of other constituents	Percent of Dry Weight	**Descriptive Term(s) of other constituents**	**Percent of Dry Weight**
Trace	<15	Trace	<5
With	15-29	With	5-12
Modifier	>30	Modifier	>12

GRAIN SIZE TERMINOLOGY		PLASTICITY DESCRIPTION	
Major Component of Sample	**Particle Size**	**Term**	**Plasticity Index**
Boulders	Over 12 in. (300 mm)	Non-plastic	0
Cobbles	12 in. to 3 in. (300mm to 75mm)	Low	1 - 10
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)	Medium	11 - 30
Sand	#4 to #200 sieve (4.75mm to 0.075mm	High	> 30
Silt or Clay	Passing #200 sieve (0.075mm)		

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu ³ 4 and 1 £ Cc £ 3 [E]	GW	Well-graded gravel [F]
			Cu < 4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu ³ 6 and 1 £ Cc £ 3 [E]	SW	Well-graded sand [I]
			Cu < 6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above “A”	CL	Lean clay [K, L, M]
			PI < 4 or plots below “A” line [J]	ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above “A” line	CH	Fat clay [K, L, M]
			PI plots below “A” line	MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.

[C] Gravels with 5 to 12% fines require dual symbols: 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.

[D] Sands with 5 to 12% fines require dual symbols: 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.

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ³ 15% sand, add “with sand” to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add “with organic fines” to group name.

[I] If soil contains ³ 15% gravel, add “with gravel” to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add “with sand” or “with gravel,” whichever is predominant.

[L] If soil contains ³ 30% plus No. 200 predominantly sand, add “sandy” to group name.

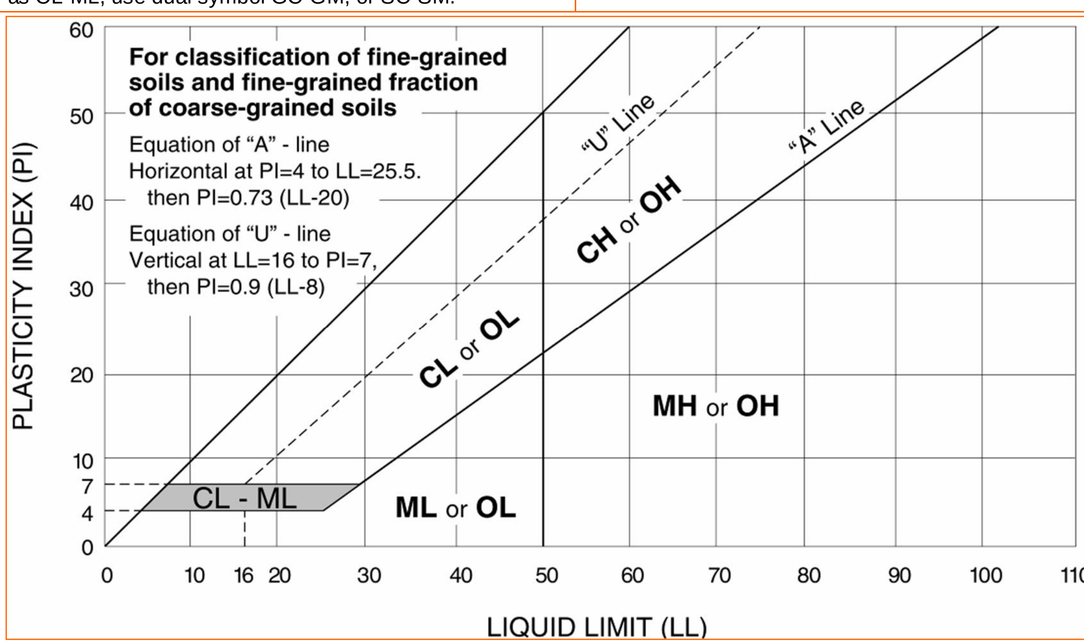
[M] If soil contains ³ 30% plus No. 200, predominantly gravel, add “gravelly” to group name.

[N] PI ³ 4 and plots on or above “A” line.

[O] PI < 4 or plots below “A” line.

[P] PI plots on or above “A” line.

[Q] PI plots below “A” line.



ROCK VERSION 1

WEATHERING	
Term	**Description**
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi (MPa)**
Extremely weak	Indented by thumbnail	40-150 (0.3-1)
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	150-700 (1-5)
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	700-4,000 (5-30)
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	4,000-7,000 (30-50)
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	7,000-15,000 (50-100)
Very strong	Specimen requires many blows of geological hammer to fracture it	15,000-36,000 (100-250)
Extremely strong	Specimen can only be chipped with geological hammer	>36,000 (>250)

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Extremely close	< ¾ in (<19 mm)	**Laminated**	< ½ in (<12 mm)
Very close	¾ in – 2-1/2 in (19 - 60 mm)	**Very thin**	½ in – 2 in (12 – 50 mm)
Close	2-1/2 in – 8 in (60 – 200 mm)	**Thin**	2 in – 1 ft. (50 – 300 mm)
Moderate	8 in – 2 ft. (200 – 600 mm)	**Medium**	1 ft. – 3 ft. (300 – 900 mm)
Wide	2 ft. – 6 ft. (600 mm – 2.0 m)	**Thick**	3 ft. – 10 ft. (900 mm – 3 m)
Very Wide	6 ft. – 20 ft. (2.0 – 6 m)	**Massive**	> 10 ft. (3 m)

Discontinuity Orientation (Angle): Measure the angle of discontinuity relative to a plane perpendicular to the longitudinal axis of the core. (For most cases, the core axis is vertical; therefore, the plane perpendicular to the core axis is horizontal.) For example, a horizontal bedding plane would have a 0-degree angle.

ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

Reference: U.S. Department of Transportation, Federal Highway Administration, Publication No FHWA-NHI-10-034, December 2009 Technical Manual for Design and Construction of Road Tunnels – Civil Elements