City of Myrtle Beach
SOUTH CAROLINA

PURCHASING AND
MATERIALS MANAGEMENT

(843) 918-2170
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ADDENDUM 001

19-B0095

Grand Park Linear Trail Project

April 3, 2019

A Pre-Bid Meeting was held and the following were in attendance:

Lou Almonte	Palmetto Corp.
Kipp Sorawns	Superior Blacktop
Tray Floyd	Coastal Asphalt
Kia Ford	Stalvey Construction
Amos Green	Greenwall Construction

Please find attached a list of questions that have been asked with the appropriate response. Also, attached is the Geotechnical Survey that was performed by S&ME.

The Bid opening remains the same, Tuesday April 9, 2019 at 2:00 PM.

Thank you for your interest in this project,

Ruth Burleson
Ruth Burleson/Buyer
City of Myrtle Beach

Grand Park Linear Trail Project Questions and Reponses:

1. Would the City of MB consider adding a pay item for Construction Stakes, Lines and Grades?
   Response: No, please include this cost in a related bid item.

2. Will the city hire and pay for a third party firm to perform testing (concrete, compaction, density, geotechnical, etc…)? Is the contractor responsible for any payment for testing?
   Response: Third party testing will be handled by the City.

3. Will the aggregate base course need to primed prior to asphalt paving?
   Response: No, unless required by SC DOT specifications. Base course should be adequately compacted.

4. Are we to assume that all excavated soils will be suitable for re-use on the project?
   Response: No.

5. Will the RCP pipe joints be O-Ring or Tongue & Groove?
   Response: Tongue and Groove with Mastic.

6. SCHEDULE: As discussed at Pre-Bid Meeting, it is our opinion that it is imperative, from a cost standpoint, that this Project be completed during the "summer/drier" months. Current Bid Documents call for 98% Modified Compaction at the existing clayey material. Correct moisture content is absolutely imperative to achieve this type of specified density.
   a. Based on the above, can we expect that this Project will be awarded in less than thirty (30) days or so?

   Response: Bid Proposals must be reviewed by the SCDOT LPA Section. If bidders have done a good job of submitting all required information and meeting the DBE requirement, we would expect 30 days would be sufficient to award the Project.

7. CONSTRUCTION ENTRANCES: Bid Documents indicate three (3) Construction Entrances. We feel that an additional construction entrance will be required at the south end of the NW Trail. If so, will this be paid at the construction entrance unit price?
   Response: The Bid Form will be adjusted to reflect four (4)

   a. Three (3) of the construction entrances will require access for trucks, equipment, etc. and will, more than likely, cause damage to existing sidewalk, curb and gutter, etc. We assume that the GC will be responsible for all costs required to repair any damage to these type items. Please confirm.

   Response: That is correct.

8. BID ITEM #2 - IMPORTED FILL (ASSUMES 1/3 UNDERCUT) - 1,500: SY at Bid Form vs. CY?
   Response: Bid Form will be changed to reflect CY.
   a. We do not understand the above note "Assumes 1/3 undercut". Also, it would appear that the "1,500" as noted above would be CY vs. SY. Please clarify.
   b. If there is going to be undercut, where do we place the costs for muck/haul off/etc.?

   Response: All cost will be paid under this bid item.

9. BID ITEM #11 - STAINLESS STEEL SAFETY RAILING:
   a. Confirm extent of stainless steel safety rail at south end of NW trail at Airpark Road.
      Response: From Sta. 0+00 to Sta. 3+77.97

b. Confirm extent of stainless safety rail at north end of SE trail at Airpark Road
   Response: From Sta. 10+52.92 to Sta. 3+77.97
c. 1/2" diameter stainless steel cable is extremely large. Based on IT research, the largest stainless steel cable rail that we saw was 3/8". It seems to me that 3/16" - 1/4" cable would be large enough.
   - On Drawing C1 .0 - Safety Rail Detail - indicates "up to%" SS cable rail ".

   Response: Use 3/8".
d. What is the specification/thickness/finish/etc. at 2" x 2" posts?
   - Assume thickness such as 0.12.
   - Assume specification for stainless steel such as 316, etc. 316 is correct
   - Finish would be such as polished or satin. We would assume satin. Satin
   - Please confirm *I* clarify above items.

   Response: Response to all bullet points of this question:
   - www.wagnercompanies.com
e. Confirm the stainless steel plate 3" x 1" is 1" thick and is welded on top of 2" x 2" posts. Since all this plate will require field welding at top of each post, as well as field welding to present a smooth transition where each plate end will require field welding, grinding, etc., there would probably be a reasonable cost savings if this top/cap plate were something thinner than 1" thick.
   Response: www.wagnercompanies.com
f. Please be aware that the Details for the Stainless Steel Safety Rail will require major on site fabrication/installation due to vertical changes in elevation and horizontal curves at the trail. Any particular details required at connections at ends of Safety Rail?
   Response: Shop Drawing Review
g. Any particular details required at connections at turnbuckles.
   Response: Shop Drawings Review
h. Please identify the approximate distance from edge of pavement to safety rail. This distance will be important to ensure that there is room for paving equipment after installation of safety rail.
   Response: Maintain 3' shoulder on trail
i. As discussed at Pre-Bid, City indicated that they believe that the details on the Bid Drawings were based on a particular manufacturer/supplier. If so, please provide that information. That information may answer a lot of the questions/concerns as outlined above.
   Response: Yes - www.wagnercompanies.com
j. Drawing C1 .9 - Various Details - indicates "steel posts" and "steel cable". Need to verify that the Safety Railing is 100% stainless steel.
   Response: Yes

10. BID ITEM #12 - 4" CONDUIT - 400 LF: We did not see where this 4" conduit would be located at the project. If the 4" conduit is to be provided for items such as electrical, future irrigation, etc., please provide the depth that will be required to install 4" conduit.
    Response: Eliminate this bid item

11. BID ITEM #5 AT PAVING BID SCHEDULE: Item #5 indicates 1 each replace ADA ramp. The specified ADA Ramp note is shown at south end of NE trail. This ADA Ramp is severely damaged and will need to be removed. Please confirm that we are to place removal costs at Bid Item #5 as indicated above.
    Response: Yes

12. MOBILIZATION / GENERAL CONDITION PAY ITEM: As suggested at the Pre-Bid Meeting, the addition of a Mobilization/General Conditions Pay Item would allow the GC to place fixed costs such as Insurance, Bonds, Business License, etc. In our opinion, this would provide more accurate unit prices and would be in the best interest of the City and the GC.
    Response: Another pay item will not be added.

13. UNCLASSIFED / CUT / FILL: Where will the costs for grading, such as cut/fill/unclassified/etc., b~ priced? If a Pay Item is added to the Unit Price Bid Form, what will be the estimated quantity?
    Response: Use Line Item No. 3

    a. Reference is made to Bid Documents that state sub-grade compaction will be 98% Modified Proctor. Considering that the existing material is very clayey, the existing material may prove to be unsuitable to achieve 98% Modified Proctor. In that case, we would expect to be paid by the CY for imported fill. Please clarify.
       Response: Will be paid in Item No. 2

14. SANTEE COOPER WORK: When will Santee Cooper conduit / light bases / poles / etc. be installed? Will this cause interference/coordination with GC work? At Pre-Bid Meeting, the City stated that it would the GC's responsibility for coordination with Santee Cooper. In our opinion, the most opportune time for Santee Cooper to install their work would be after the MLBC material is in place and compacted. With that in mind, given the fact that the GC will have no control over when and/or how long Santee Cooper takes to install their work, the 120 day current schedule may become a problem.
    Response: City will be responsible for coordinating with Santee Cooper and Construction.

15. CLEARING & GRUBBING: Bid Schedule indicates one (1) acre of clearing and grubbing. Site visit did not clarify how much actual clearing will be needed. It appears that almost 100% of the trail will require grubbing / removal of grass, topsoil, etc.
    a. Can the Engineer/Owner clarify how much actual clearing of trees, etc. is required?
    b. Also, can Engineer/Owner clarify where grubbing/topsoil//etc. costs will be placed at Bid Items?
    c. In summary, will we be paid for the grubbing/topsoil/etc. at the clearing and grubbing unit price item?

    Response: City and Contractor will decide quantities on site.

16. BID ITEM #4 - SEEDING: Bid Form indicates 18,000 SY of Seeding. Is this to include temporary and permanent seeding? If so, will the square yardage be measured for temporary and permanent seeding?
    Response: All permanent seeding.

17. CONCRETE AT SAFETY RAIL: Detail at Safety Rail/Retaining Wall at Drawing C1 .9 shows concrete "infill". Where will the costs for this item be priced?
    Response: Should be included in price for Safety Rail.

18. GEOGRID AT LF 13 DRAWING C1 .9: Geotextile is indicated at Bid Item #1 under Paving Schedule. We assume that this is the same item and will be paid at Bid Item #1 . Please confirm.
    Response: Yes

19. FILL / GRADING / ETC. AT POND BANK: See Pond Bank Detail at Drawing C1 .9, See Cross Sections for extent of blue highlight areas from site visit last Sunday for extent that the Engineer/Owner intends for grading down slope of pond bank.
    Response: Pond Banks 5 to 1 slope

    a. If temporary/permanent grassing is to be included at these areas, will we be paid under the grassing/SY Pay Item?
    Response: Yes

20. CADD DRAWINGS: Verify that GADD Drawings will be available from the Engineer.
    Response: Prepare Bid on what you have. An effort will be made to provide these files.

21. NOISE: Spec. Section 0300 - Page 1; Item #3 indicates "Residents re not disturbed by noise ... ". Question is - if one resident complains of noise for no good reason, what is the Contractor supposed to do?
    Response: City Noise Regulations will apply.

22. SURVEYING / STAKING / LAYOUT / ETC.: Spec. Section 0550-17; Article IV; paragraph A - indicates that the Owner will provide general layout and control grade for construction work.
    a. Spec. Section 0650-1 ; Article 1.05 - Construction Stake Out indicates that alignment and control - Engineer will provide a baseline for construction alignment and benchmark for elevation.
    b. Paragraph 1.05B indicates that Contractor shall perform all construction stake out, etc.
    c. As discussed at the Pre-Bid Meeting, GC will be responsible for all field engineering/construction stake out/etc. Please confirm.

    Response: Yes

23. SWPPP: Drawing C0.1 - General Note 19A indicates SWPPP might be by GC.
    Response: This refers to Contractor
    a. See Drawing EC1 .1; XII , with reference to SWPPP by GC.
    b. Please clarify whether responsibility for SWPPP will be by the Owner or the GC.
       Response: These handled by owner.

24. GEOTECHNICAL REPORT - DRAWING C1 .09: Drawing C1 .9 - Typical Pavement Section has a note "Compacted Subgrade per Geotechnical Report". Is there supposed to be a Geotechnical Report included with Bid Documents?
    Response: Report provided.

25. DAVIS BACON WAGE DECISION: Wage Decisions for **two categories** are included in the Project Manual – "Highway" and "Heavy". Based on our experience and interpretation of the work for this project, the applicable Decision would be the "Heavy" category versus the "Highway" category. Please confirm.
    Response: Yes, the appropriate Davis Bacon Wage Decision category for the project is "Heavy". The "Highway" category listed with the DOT specs used as a reference. Please continue with referring ONLY to the "Heavy" classification for all Wage Rates on this project.

Report of Geotechnical Exploration
Trail System Grand Linear Park
Myrtle Beach, South Carolina
S&ME Project No. 1463-17-018

S&ME

Prepared for:
Thomas & Hutton
611 Burroughs and Chapin Blvd., Suite 202
Myrtle Beach, South Carolina 29577

Prepared by:
S&ME, Inc.
1330 Highway 501 Business
Conway, SC 29526

June 16, 2017

S&ME

June 16, 2017

Thomas & Hutton
611 Burroughs and Chapin Blvd., Suite 202
Myrtle Beach, South Carolina 29577

Attention: Mr. Walter Warren

Reference: **Report of Geotechnical Exploration**
Trail System Grand Linear Park
Myrtle Beach, South Carolina
S&ME Project No. 1463-17-018

Dear Mr. Warren:

S&ME, Inc. has completed the geotechnical exploration for proposed construction of Grand Linear Park at Market Commons in Myrtle Beach, South Carolina. Our services were performed pursuant to S&ME proposal No. 14-1600750-R1 dated January 24, 2017, accepted on May 16, 2017.

The purpose of this exploration was to evaluate subsurface conditions within the construction footprint as they relate to site preparation, earthwork, and structural support of bridges and retaining walls. This report represents our understanding of the proposed construction, the site and subsurface conditions encountered, and our conclusions and recommendations.

Sincerely,

S&ME, Inc.

Kara Fugate
Staff Professional

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION
S&ME, INC.
No. C00478

Ronald P. Forest, Jr., P.E.
Senior Engineer

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER
No. 21248
6/16/17
RON FOREST JR.

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Executive Summary

For your convenience, this report is summarized in outline form below. This brief summary should not be used for design or construction purposes without reviewing the more detailed information presented in the remainder of this report.

1. **Soil Conditions:** The soil borings encountered 1 to 5 inches of topsoil. One boring was performed in a dirt area with no topsoil. The topsoil was underlain by approximately 7.5 to 16 feet of interbedded sands and clays (Stratum I). Underlying these interbedded sands and clays, a thick layer of very soft, high plasticity (fat) clays and clayey sands with shell fragments extended to depths ranging from 30 to 66.5 feet (Stratum II). Below the very soft fat clay, a combination of poorly graded sand, clayey sand, and sand with silt extended to the termination of our borings at depths ranging from 30 to 70 feet (Stratum III).

   - Some of the soil test borings had to be extended well beyond their originally planned termination depths due to the presence of very soft, high plasticity clays to great depth.
   - These very soft clays are much thicker in the northern portion of the site, near Bridge 1.

2. **Subsurface Water:** Subsurface water was observed to range from 6.25 feet to 10.4 feet below ground surface. The water levels that we measured may have been impacted by the heavy rainfall that occurred at the time of drilling.
3. **Driven Timber Pile Foundations:** We were not provided with any structural loading information for the proposed bridges. Based upon assumed load values, the estimated static total and differential settlements may exceed typically acceptable tolerances if shallow foundations are used; therefore, a deep foundation system consisting of driven 8-inch tip diameter, naturally tapered timber piles is recommended for support of Bridge 2 and Optional Bridge 3. Working capacities and anticipated pile lengths are discussed later in this report.
4. **Helical Pier Supported Foundations:** At Bridge 1, the estimated bearing depth for deep foundation support is about 60 to 70 feet below the existing ground surface. Timber piles 70 feet in length may not be readily available or cost prohibitive. Therefore, helical steel piers, which can be assembled on-site in 5 to 10 ft. long sections, may be may be a better alternative than very long timber piles to support the foundations at Bridge 1. Helical steel piers could also be used to support Bridges 2 and 3, if desired.
5. **Pavements:** Along the northern portion of the trail, in the vicinity of test locations T-1, T-7 and T-8, soft to firm, sandy fat clay was observed in the upper 3 ½ to 4 ½ feet. The California Bearing Ratio (CBR) value of this material was measured to be 1.5 percent; therefore, the soil subgrade in the northern portion of the trail should be built-up or undercut and replaced with at least 2 feet of sandy backfill to bridge over the soft clay subgrade soils. In the remainder of the trail, south of test locations T-1 and T-8, the near surface soils consist mostly of clayey sands, sandy lean clays, and sands, and CBR testing of samples recovered from test locations T-3 and T-6 ranged from 16 to 23 percent, indicating relatively good subgrade support characteristics. Once surface stabilization of the soils has been performed, we recommend construction of a pavement section that includes at least 6 inches of graded aggregate base course overlain by at least 2 inches of Type C hot mix asphalt surface course.

1.0 Introduction

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning earthwork, foundations, and other related construction issues. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations. The scope of our geotechnical services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials.

A site plan showing the approximate exploration locations is included in Appendix I. The boring logs, profile views, discussion of the field exploration procedures, and legends of soil classification and symbols are included in Appendix II. Appendix III contains the results of the laboratory testing and our laboratory test procedures.

1.1 Site and Project Description

Project information was initially provided in an email received by Ron Forest (S&ME) from Walter Warren (Thomas Hutton) on September 26, 2016. Additional information was provided in email exchanges between Walter Warren, Ron Forest and Chelsea Jones (S&ME) on September 18, 2016 and September 19, 2016. In the original email from Walter Warren, we received a parcels map dated September 26, 2016. In a later email from Mr. Warren, we reviewed a Boundary Survey Map dated October 22, 2015 and a "Preliminary Trail Plan" which was undated. We were requested to revise our proposal to include additional borings at potential sites for two retaining walls and deeper borings at the sites for bridges after discussions with the City of Myrtle Beach. Final approval to begin work was received on May 16, 2017 from Walter Warren.

1.2 Site Description

The site is located off of Farrow Parkway, between U.S. Highway 17 Bypass and The Market Common, nearest to the intersection with Airpark Drive, in Myrtle Beach, South Carolina. The property is now used as a grass pathway for walkers or bikers. A site vicinity plan is included in Appendix I as Figure 1.

1.3 Project Description

It is our understanding that the City of Myrtle Beach plans to construct a walking and bicycle path on site. The proposed walkway is approximately 6,000 feet in length and will follow along the bank of an existing pond. The walkway may also include up to three prefabricated timber pedestrian bridges. We also understand that the pavement sections should be able to accommodate the occasional lift bucket truck operated by Santee Cooper for servicing of the overhead lighting along the path, as well as, the occasional lawn maintenance vehicle.

Our exploration included the pavement areas for the walkway, bridge locations, and retaining wall locations in order to evaluate the near-surface soils along the path. The purpose of this evaluation was to provide foundation support recommendations for the prefabricated timber bridges, and to recommend pavement sections for the future paved path and lateral earth pressure coefficients for retaining walls to be designed by others. We were not provided with any structural loading information prior to preparing our report, and therefore our conclusions are based upon some assumed loading parameters.

2.0 Exploration Procedures

2.1 Field Exploration

On May 26, 2017, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

- We performed a site walkover, observing general features of topography, existing structures, ground cover, and surface soils at the project site.
- We established 18 soil penetration test (SPT) boring locations at the project site. A test location sketch is attached in Appendix I as Figure 2. There were five borings marked at future bridge locations (B-1 through B-3). There were three borings marked at retaining wall locations (R-1 through R-3). There were an additional nine borings placed along the proposed path (T-1 through T-9). There was one boring located near the intersection of Farrow Parkway and Forbus Court, for the sidewalk widening (T-10).
- We also made contact with SC 811 for clearance to dig.

On June 7 through 9, and 12, 2017 representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

- We advanced five SPT borings at the bridges (B-1 through B-3) to depths ranging from 40 to 70 feet each. We originally proposed to advance each of the bridge borings to a depth of 40 feet; however, due to the very soft soil conditions encountered at the planned termination depth, several of these borings were extended until firm bearing materials were encountered. The client (Walter Warren) was informed of these changes via telephone and email at the time that they occurred. A subtotal of 60 additional linear feet of drilling was performed at the bridge borings.
- We advanced three SPT borings at the retaining walls (R-1 through R-3) to depths ranging from 35 to 45 feet each. We originally proposed to advance each of these borings to a depth of 25 feet; however, due to the very soft soil conditions encountered at the planned termination depth, several of these borings were extended until firm bearing materials were encountered. The client (Walter Warren) was informed of these changes via telephone and email at the time that they occurred. A subtotal of 50 additional linear feet of drilling was performed at the retaining walls.
- We advanced ten SPT borings to a depth of 5 feet each (T-1 through T-10). Boring T-10 was not originally proposed, but was requested to be performed by Walter Warren in an email to Worth King of S&ME dated May 30, 2017.
- Soil samples were collected at regular depth intervals of every 2 ½ feet in the upper 10 feet and every 5 feet thereafter with a standard 1.4 inch I.D., two-inch O.D., split barrel sampler.
- We collected three bulk samples from the boreholes at three different locations (T-3, T-6, and T-7).
- The subsurface water level at each boring was measured in the field at the time of drilling and again at least 24 hours after drilling.

A description of the field exploration procedures performed, as well as, the SPT logs are attached in Appendix II.

2.2 Laboratory Testing

Soil samples that we obtained were transported to our laboratory and classified in general accordance with the visual-manual method described in ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Method)". The following ASTM standardized laboratory tests were performed upon three bulk samples:

- Natural Moisture Content (ASTM D 2216)
- Fines Content Testing to measure the percent passing the No. 200 sieve (ASTM D 1140)
- Atterberg Plasticity Limits (ASTM D 4318)
- Modified Proctor Moisture-Density Relationship (ASTM D 1557)
- California Bearing Ratio (CBR) (ASTM D 1883)

One additional Atterberg Limits plasticity test was also performed upon one of the recovered split-spoon samples, to further characterize the plastic behavior of the soft marine clay layer. A summary of the laboratory procedures used to perform these tests is presented in Appendix III. The individual test results are also included in Appendix III.

3.0 Site and Surface Conditions

This section of the report describes the general site and surface conditions observed at the time of our exploration.

3.1 Topography

We observed that the proposed development area appears to be relatively level, sloping downward around the perimeter of the water features. Ground surface elevations were not directly surveyed, and no site specific topographic plan was made available to us; therefore, for the purpose of our boring logs and bridge location profiles (Appendix II), the ground surface level was set to zero.

3.2 Site Conditions

The subject site is located off on Farrow Parkway, crossing over Airpark Drive and extending around the existing pond. At the time of our exploration, site cover at each test location consisted of lawn grasses measuring a few inches in height and some natural vegetation or fallen trees. Topsoil was measured to a maximum of 5 inches in depth, varying across the site.

3.2.1 Existing Landfill

Between our test locations T-5 and B-3, we observed that the trail alignment proceeds along the edge of a land-use restricted former landfill. The landfill is discussed in more detail in our environmental services report, which is being issued separately. However, because there were signs posted at each corner of the landfill stating "soil disturbance prohibited", we did not advance any boreholes within the landfill zone. Therefore, the soil conditions within the landfill area are not known at this time.

3.3 Local Geology

The site is located in the Coastal Plain Physiographic Region of South Carolina. This area is dominated topographically by a series of relic beach terraces, which progressively increase in surface altitude as they proceed inland. These terraces have been extensively mapped and correlated over wide areas. A review of local geologic mapping indicates that surface soils penetrated in our borings represent a part of the Socastee Formation, consisting of recent marine deposits laid down approximately 200,000 years ago. Beneath the upper formation, geologic mapping and soils encountered within borings indicate the Penholoway Formation, which is characterized by sands, silts and clays often with shell fragments and cemented materials, laid down during the Lower Pleistocene Epoch approximately 760,000 years ago.

4.0 Subsurface Conditions

The generalized subsurface conditions at the site are described below. For more detailed descriptions and stratifications at test locations, the respective boring logs should be reviewed in Appendix II.

4.1 Interpreted Subsurface Profile

Two subsurface cross-sectional profiles of the site soils are attached in Appendix I to illustrate a general representation of the subsurface conditions at the proposed construction areas of Bridges 1 and 2. Profile B-1A – B-1B (Figure 3) depicts the subsurface conditions within the "Bridge 1" area at the northern end of the site. Profile B-2A – B-2B (Figure 4) depicts the subsurface conditions within the "Bridge 2" area near the southern end of the site.

The strata indicated in the profiles are characterized in the following section. Note that the profiles are not to scale, and were prepared for illustrative purposes only. Subsurface stratifications may be more gradual than indicated, and conditions may vary between test locations. Soils presented on the profiles were grouped into several general strata based on estimated physical properties derived from the borings and the recovered samples. The strata encountered are labeled I through III on the soil profile to allow their properties to be systematically described.

4.2 Description of Subsurface Soils

This section describes subsurface soil conditions observed at the site, as shown on the profiles in figures 3 and 4 of Appendix I.

4.2.1 Stratum I: Interbedded Clayey Sands, Sandy Clays, and Sands

Underlying the topsoil, an interbedded zone of firm to stiff sandy lean clay (USCS Classification "CL"), very soft fat clay (CH), soft to firm sandy fat clay (CH), very loose to medium dense clayey sand (SC), loose to medium dense silty sand (SM), very loose to loose poorly graded sand with silt (SP-SM), very loose poorly graded sand with clay (SP-SC), and very loose to medium dense poorly graded sand (SP) was encountered to depths of approximately 7.5 to 16 feet. These soils were typically a combination of grey and orange in color. The SPT N-values in this stratum range from 2 to 19 blows per foot (bpf), but typically ranged from 6 to 8 bpf, indicating a typically loose relative density in the sandy soils and firm consistency in the clayey soils. Borings T-1 through T-10 each terminated in this stratum at a depth of 5 feet.

We collected three bulk samples from this stratum at three different test locations (T-3, T-6, and T-7). These samples were subjected to natural moisture content, grain size distribution, plasticity, moisture-density relationship testing, and California Bearing Ratio tests.

- The bulk sample taken from T-3 boring location was classified as a clayey sand (SC) with a fines content of 16.2 passing the No. 200 sieve, and was brown in color. The natural moisture content was measured to be 5.6 percent. The liquid limit was 21 percent and the plasticity index was 9 percent. The modified proctor maximum dry density was 109.1 pounds per cubic foot (pcf) at an optimum moisture content of 8.9 percent, indicating that the in place soil is approximately 3.3 percent drier than the optimum moisture content for compaction. The CBR value was measured to be 23.4 percent at 95 percent compaction (ASTM D 1557).

- The bulk sample taken from the T-6 boring location was classified as a clayey sand (SC) with a fines content of 36.2 passing the No. 200 sieve, and was brown in color. The natural moisture content was measured to be 14.4 percent. The liquid limit was 36 percent and the plasticity index was 21 percent. The modified proctor maximum dry density was 107.2 pounds per cubic foot (pcf) at an optimum moisture content of 10.9 percent, indicating that the in-situ soil is approximately 3.5 percent wet of the optimum moisture content for compaction. The CBR value was measured to be 16.3 percent at 95 percent compaction (ASTM D 1557).

- The bulk sample taken from the T-7 boring location was classified as a sandy fat clay (CH) with a fines content of 62.4 passing the No. 200 sieve, and was brown in color. The natural moisture content was measured to be 20.3 percent. The liquid limit was 53 percent and the plasticity index was 30 percent. The modified proctor maximum dry density was 103.3 pounds per cubic foot (pcf) at an optimum moisture content of 13.8 percent, indicating that the in-situ soil is approximately 6.5 percent wet for the optimum moisture content for compaction. The CBR value was measured to be 1.5 percent at 95 percent compaction (ASTM D 1557).

4.2.2 *Stratum II: Very Soft Fat Clay*

Underlying the interbedded sand and clay mixtures, a layer of very soft, dark grey, cohesive soils consisting primarily of high plasticity (fat) clays (CH), but also containing some clayey sands (SC) with a high percentage of clay fines extended to depths of approximately of 30 to 66.5 feet, and was observed in each of the deeper borings. There were trace amounts of shell present in several of the samples.

The soils in this stratum typically exhibited "weight of hammer" (WOH) penetration resistance measurements, which is defined as the sampler advancing its full length of 18 inches under only the dead weight of the hammer itself, without applying any blows or drops.

The thickness of this very soft clay layer varied across the site, but was greatest at the northern end of the trail near Bridge 1. See Table 4-1 below for a list of the very soft clay layer thicknesses at each of our deep boring test locations.

Table 4-1: Depth and Thickness of Very Soft Marine Clay Layer

Structure	Boring No.	Depth Range of Very Soft Fat Clay (Feet-Feet)	Thickness of Very Soft Fat Clay Layer (feet)
Bridge 1	B-1A	16 – 55	39
	B-1B	16 – 66.5	50.5
Bridge 2	B-2A	12 – 35	23
	B-2B	11 – 36	25
Bridge 3	B-3	7.5 – 34.5	27
Retaining Wall 1	R-1	10 – 35	25
Retaining Wall 2	R-2	11 – 30	19
Retaining Wall 3	R-3	13 – 33.5	20.5

4.2.3 *Stratum III: Loose to Dense Sands and Sand Mixtures*

Beneath the very soft clays of Stratum II, a stratum of loose to dense poorly graded sands (SP), clayey sands (SC), and silty sand (SM) was encountered to depths ranging from 30 to 70 feet. Borings R-1, R-2, R-3, B-1A, B-1B, B-2A, B-2B, and B-3 all terminated in this stratum at various depths. SPT N-values measured within this stratum ranged from 10 to 33 bpf, but averaged about 18 bpf overall, indicating typically medium dense relative density. Some distributed organics were encountered in this stratum in most of the borings, consisting of wood material believed to be relic roots.

4.3 Subsurface Water

Water levels measured in the boreholes at least 24 hours after drilling ranged from approximately 6.25 feet to 10.4 feet beneath the ground surface, and appeared to generally correspond with the water levels in the adjacent ponds and canals. Water was not encountered in any of the five foot deep borings at the time of drilling, and these borings were backfilled upon completion.

Subsurface water levels may fluctuate seasonally at the site, being influenced by rainfall variation and other factors. Site construction activities can also influence water elevations. During our field exploration, there was significant rainfall and this may be reflected in the water level measurements.

5.0 Conclusions and Recommendations

The conclusions and recommendations included in this section are based on the project information outlined previously and the data obtained during our exploration. If the construction scope is altered, the proposed trail or bridge locations are changed, or if conditions are encountered during construction that differ from those encountered in the SPT borings, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

5.1 Surface Preparation

Site preparation will include stripping of surface vegetation, removal of about 3 to 5 inches of topsoil in grassy areas, and removal of peat and organic matter, root mat, roots, stumps, etc. The following recommendations are provided regarding site preparation and earthwork.

1. Strip surface vegetation, topsoil, and any other organic or unsuitable materials, where encountered, and dispose of outside the future pavement footprints. Organic soils containing more than about 5 percent organics should be removed from the proposed construction areas. Do not locate burn piles or debris piles within the construction area.

    A. Within the landfill zone there may be a land use restriction regarding the removal of the topsoil prior to grading for the new trail. If the topsoil and rootmat cannot be removed due to these restrictions, then it may become necessary to improve this ground in other ways to stabilize it sufficiently to construct the pavement section on top of the existing ground. If this is the case, please contact us for additional ground stabilization recommendations.

2. Existing underground utilities or culverts that are to be demolished should be removed from the site. Areas where existing construction is to be removed should be backfilled in accordance with section 5.2 of this report.
3. Because the CBR value of the near-surface soils at test location T-7 was measured to be 1.5 percent, and since similar soft soils were observed at test locations T-1 and T-8, we therefore recommend that the native soil subgrade in this northern portion of the trail (between approximately T-1 and T-8) should either be bridged over or undercut and replaced with a sandy compacted backfill layer at least 2 feet in total thickness, in order to bridge over the soft clay subgrade soils.
4. After the stripping and undercut and replacement operations described in items 1 through 3 above are complete, the densified surface should be proofrolled under the observation of the geotechnical engineer (S&ME) by making repeated passes with a fully-loaded dump truck. The proofrolling should be conducted only during dry weather and after drainage has been implemented and allowed time to function in order to avoid deteriorating the surface. Although not expected to be widespread, isolated areas of rutting or pumping soils indicated by the proofroll may require selective undercutting or further stabilization prior to fill placement or pavement construction, and such areas should be addressed on a case by case basis at the time of construction.

5.2 Fill Placement and Compaction

Where new fill soils are to be placed on the site, the following recommendations apply:

1. Before beginning to place fill, sample and test each proposed fill material to determine if it is suitability for use, maximum dry density, optimum moisture content, and natural moisture content. It is recommended that any imported fill soils used to build up the embankment for the pavements meet the following minimum requirements: plasticity index of 10 percent or less; clay/silt fines content of not greater than 25 percent. Some of the upper layer of soils in the southern portion of this site, south of test locations T-1 and T-8, appear to meet these criteria,

and would likely be suitable to borrow for re-use as structural fill material. All other fill will likely need to be imported. The Stratum II fat clays are unsuitable for re-use as fill.

2. Where fill soil is required, structural fill should be compacted throughout to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557). Compacted soils must not exhibit pumping or rutting under equipment traffic. Loose lifts of fill should be no more than 8 inches in thickness prior to compaction. Structural fill should extend at least 5 feet beyond the edge of pavements before either sloping or being allowed to exhibit a lower level of compaction.
3. All fill placement should be witnessed by an experienced S&ME soils technician working under the guidance of the geotechnical engineer. In general, at least one field density test for every 5,000 square feet should be conducted for each lift of soil in large area fills, with a minimum of 2 tests per lift. At least one field density test should be conducted per each 150 cubic feet of fill placed in confined areas such as isolated undercuts and in trenches, with a minimum of 1 test per lift.

5.3 Driven Pile Foundations

Although we were not provided with any actual structural loads for the proposed bridges, it is our conclusion that shallow foundations may not be suitable for support of the bridges at this site, due to the thick zone of very soft soil, and the excessive static settlements that may result from consolidation of the Stratum II clays under even relatively light loads.

Timber piles are relatively light-duty piles that can be more cost effective than other higher capacity deep foundation types, provided that they can accommodate the applied static and lateral structural loads, and are properly installed. Timber piles may be a feasible foundation support alternative for Bridge 2 and Optional Bridge 3; however, due to the great depth required to reach suitable end bearing soils that we observed at Bridge 1, timber piles may not be the best alternative for support of Bridge 1. Recommendations for helical steel piers are provided in Section 5.4 of this report, which may be considered a more suitable alternative for support of Bridge 1. Helical piers may also be considered for support of Bridges 2 and 3, if desired.

5.3.1 Axial Capacity of Timber Piles

We estimated static capacity for timber piles using the method outlined in the NAVFAC Design Manual 7.2 (1984), based upon information obtained from the soil test borings performed at the referenced site. Our pile calculations were modeled to a bearing depth of **40 to 45 feet** below the existing ground surface, because based upon the SPT N-values measured in borings B-2A, B-2B, and B-3, it is anticipated that driven timber piles are unlikely to advance beyond 45 feet.

Performance of index (test) piles prior to ordering the production piles will be important on this site, to determine the depth at which the piles are expected to terminate driving and to confirm that our assumptions about the refusal depth are correct.

Based on our exploration and analysis, an individual 8-inch (minimum) tip diameter naturally tapered timber pile, pre-augered to a depth of 5 feet and then driven to a depth of **40 to 45 feet** is anticipated to provide an *ultimate* axial compressive capacity of 45 tons, which must be factored for safety. The estimated *allowable* (design) axial compressive and uplift resistance capacity values are provided in Table

5-2 below. These values assume a factor of safety of 3 against the ultimate capacity for this embedment depth under static conditions.

Table 5-1: Timber Pile Vertical Capacities at Bridges 2 and 3 Only

Pile Type	Allowable Axial Capacity[A] (tons)	Allowable Uplift Capacity[B] (tons)	Modeled Total Pile Length (feet)	Modeled Embedment Depth[C] (feet)
8-inch Tip Diameter Natural Taper Timber Pile (Bridges 2 & 3 Only)	15	5	40 to 45	40 to 45

A. Allowable compressive capacity assumes a factor of safety of 3 applied to the ultimate axial compressive capacity.
B. Allowable uplift capacity assumes a factor of safety of 3 applied to the estimated ultimate skin friction capacity.
C. Assumes pile is advanced to refusal using the proper impact (non-vibratory) hammer.

For the purpose of developing our axial capacity recommendations, the upper 5 feet of existing soils surrounding the piles was not considered to contribute to the skin friction support of the piles. We also assumed that the piles would not be installed using either a vibratory hammer or water jetting techniques, which may reduce the available long-term skin friction and end bearing capacity of the piles used to support the structure. For this project, these installation methods should be prohibited.

The ultimate pile capacity values are for a single, isolated foundation, and assume a spacing of at least 3 feet between piles, center-to-center. A pile spacing of less than 3 feet is not recommended; a greater spacing may be used.

If the capacities used in Table 5-2 are used for design, then a static load test is not required. If it is desired to increase the design axial (download) capacity above the values given in Table 5-2, then an axial static load test would be required to be performed under the observation of the Geotechnical Engineer. Please contact us for more information if this alternative is desired.

5.3.2 *Lateral Stability of Piles*

We analyzed the geotechnical response of a laterally loaded, 10-inch butt diameter, 8-inch tip diameter, naturally tapered timber pile using L-PILE 6.0 and a generalized subsurface profile based upon the soil conditions observed at borings B-2A, B-2B, and B-3. The L-PILE program performs a beam-column analysis of single piles subjected to given lateral and axial loading and assuming a non-linear soil response. The pile was modeled using fixed head pile top boundary conditions, with an embedment depth of 40 feet beneath the ground surface. Only static loading conditions were considered for this analysis; please contact us to analyze lateral reactions under seismic loading conditions if needed.

In the analysis, the pile was assigned a constant modulus of 1,500 ksi. At the top of the pile, an axial load of 15 tons was applied and lateral loads of 5.3, 8.8, 11.4, and 13.5 kips were applied under a fixed head condition to result in horizontal deflections ranging from ¼ inch to 1 inch. The resulting graphic plots of pile deflection, shear, and bending moment as a function of depth along the pile are attached in Appendix IV for your review.

The structural capacity and integrity of the piles under the applied shear forces and moment at each structural connection has not been considered in our analysis and must be evaluated by the Structural Engineer. The Structural Engineer should also review the boundary condition assumptions to confirm that these assumptions are compatible with the foundation design.

5.3.3 *Pile Settlement*

Settlements of pile supported foundations are anticipated to be ½-inch or less under static loading due to elastic shortening of the piles. Settlements contributed by consolidation of the bearing layer under the axial loads applied are anticipated to be insignificant for a single pile bearing in the dense soil conditions observed at 40 to 45 feet.

Settlement of pile groups may be greater than for individual piles. Group settlements may be estimated using the equivalent footing method, assuming the enclosed area by the group to act similar to a spread footing that bears at an elevation equal to two-thirds the pile length below the surface. To use this method requires that the size of the pile group, number and spacing of piles, and axial load on the group be known. We should be retained to estimate the total group settlements as well as check the differential settlement between adjacent dissimilar groups (if applicable) once the actual pile loads and the configurations of the pile groups have been finally determined.

5.3.4 *Installation Depth of Piles*

Foundation piles for the project should be driven to refusal or to driving criteria that are established at the time of construction based upon the energy of the hammer being used. Our boring logs indicate that refusal may occur at 40 to 45 feet beneath the existing ground surface, using a hammer of adequate size with piles driven in a continuous operation to the extent possible.

In areas where piles refuse at less than 40 feet in depth, the full allowable capacity may or may not be available depending upon the conditions at termination of driving, and extra piles may become necessary to accommodate the axial, lateral, or uplift loads of the structure.

The anticipated pile embedment depths may also need to be adjusted if any grade changes occur due to removal of soils or if new fill placement is performed prior to pile driving operations.

5.3.5 *Pile Driving Equipment*

Timber piles should be driven with an impact hammer delivering at least 5,000 foot-pounds of driving energy per stroke and not more than 15,000 foot-pounds of energy per stroke. Ram weight should not exceed 5,000 pounds, and a minimum ram weight of 2,000 pounds is recommended. A project specific driving criteria should be developed by the engineer based upon the contractor's proposed hammer specifications and observations made during driving of indicator piles.

Do not use vibratory hammers or water jetting to advance the piles.

If any individual pile achieves the driving criteria that indicates the desired capacity has been achieved prior to reaching an embedment depth of 40 feet, then that pile may be terminated early; however, in no case shall termination above an embedment depth of 18 feet be accepted.

Driving hammers may be diesel, steam, or air operated. The use of a gravity-drop hammer to drive piles is discouraged, since this type of hammer can produce excessively high and damaging stresses and may not be capable of advancing the pile to the designed depth. Timber piles should be driven with the aid of a metal casting that is designed to securely hold the piles in position during driving, and will distribute the load on the head of the pile to reduce splitting or brooming. All timber piles should be clean peeled and pressure treated in accordance with the requirements of AWPA C3. Timber pile design stresses should be established in accordance with ASTM D-2899.

To help assess the variability to driving refusal depth across the site, we recommend that at least one, 55 ft. long indicator pile be driven at each bridge at the commencement of pile construction, under the observation of the Geotechnical Engineer. These piles will indicate the pile installation depth that can be anticipated for the production piles in that area, and confirm that the contractor's proposed hammer energy is sufficient to advance the pile to the designed penetration depth. Splicing of timber piles can be time-consuming and difficult, so it is important to have a reasonable expectation of the required installation depth prior to the contractor ordering production piles, and prior to commencement of general production pile driving. Pile driving refusal criteria should be established for the selected hammer prior to the installation activities to avoid structural damage to the foundation piles.

5.3.6 *Pile Installation Observations during Production*

Due to the relatively light loads (IBC only requires static load testing when piles support loads of 40 tons or more), load testing is not required, provided that the recommendations presented in this report are followed and the allowable design capacity values provided in Table 5-2 are used; however, it is strongly recommended that the Geotechnical Engineer, or a qualified representative under his direction, should observe the pile driving operations during production. This allows us to maintain driving records, detect variations in pile installation if they occur, and assess the pile driving operations for variability from the design assumptions so that adjustments can be made in the field at the time of construction, if warranted.

5.4 Helical Pier Support Recommendations

This section provides geotechnical information that may be used by others to design a helical pier support system. Because suitable end bearing soils were not encountered until a depth of 60 to 70 feet at Bridge 1, it may be more economical or feasible to support Bridge 1 on helical steel piers, rather than driven timber piles. Helical piers could also be used to support Bridges 2 and 3, if desired, as an alternative to the timber piles discussed previously.

Based upon the soil profile at Bridge 1 (see also Figure 3 in Appendix I), we anticipate that helical piers embedded to depths of between 60 and 70 feet below the ground surface should provide sufficient axial capacity to support the bridge. We recommend that pier capacities be confirmed with field load testing prior to production pile installation.

The helical piers can likely be cast directly into pile caps constructed at the surface, which will also act as a footing for the bridge. The structural engineer will need to design this connection. The pier embed plate is typically recommended by the pier vendor since these systems are often of proprietary design and construction, and may vary from vendor to vendor.

5.4.1 Helical Steel Piers

Helical steel piers, often called "screw anchors", have been used for remedial underpinning and building foundation support for many years. The anchor consists of a plate or plates formed into the shape of a helix. The plate or plates are attached (usually welded) to a central shaft. The anchors are installed by applying a specified torque to the anchor and screwing it into the soil. There are varying sizes (typically 6 to 14 inch diameter) and number of helices which are selected based upon the soil parameters, loads, depth of embedment, etc. Torque capacities of the installation equipment can range from 12,000 to 50,000 ft-lbs.

The use of helical piers as a supplemental foundation support system has several advantages, as follows:

1. The ease and speed of the installation. Piers are assembled on site in short sections.
2. No removal of soil, since the installation causes displacement of the soil which produces little to no spoils.
3. Installation equipment comes in various sizes and types and may be mounted to a Bobcat® or track-hoe, or even installed by hand for low-torque applications.
4. The installation of the piers is practically vibration free, which is advantageous to nearby vibration sensitive buildings.
5. The installation of piers in the vicinity of existing foundations typically do not cause problems related to the performance of the existing foundations.

The design of the helical piers is based on bearing capacity and assumes that the capacity of the pier is equal to the sum of the capacities of the individual helices. The capacity of the helix is based on the unit bearing capacity at the depth of the helix applied to the helix area. Friction along the shaft is not accounted for in determining the pier capacity. The capacity of the pier is also based on the soil parameters above or below the helices. The helices are required to be spaced so that the stress zones for each helix do not overlap.

Based on the soil profile encountered at the site, we anticipate that the helical piers will need to be embedded to depths ranging from 60 to 70 feet below the existing ground surface at Bridge 1 to mitigate the potential for excessive post-construction settlement.

We have not provided design specifications for the helical piers in this report, because it was beyond the scope of this exploration. The design of helical piers is often vendor-specific, due to the variability between manufacturers. Typically, each vendor can provide design guidance on their own specific product. We can also assist in the provision of design recommendations, pier capacities, and specifications for the helical pier foundation systems, but this would need to be performed in conjunction with the pier vendor selected, because many of the capacity-related design characteristics of these systems are proprietary to specific products and equipment.

Due to the proprietary nature of helical piers, the pier lateral capacity calculations cannot be performed until a specific helical pier design is selected. Lateral pier capacity and lateral deflection estimation was beyond the scope of this report, and may be provided by the selected pier vendor at a later date.

Static settlements of properly designed piers are anticipated to be about ½ inch or less under static loading due to elastic shortening of the piers. Settlements contributed by consolidation of the bearing layer under the axial loads applied are anticipated to be insignificant for a properly installed single pier. Total post-construction static settlements of the combined footing and pier support system should be 1 inch or less.

Settlement of pier groups may be greater than for individual piers. Group settlements may be estimated using the equivalent footing method, assuming the enclosed area by the group to act similar to a spread footing that bears at an elevation equal to two-thirds the pile length below the surface. To use this method requires that the size of the group, number and spacing of piers, and axial load on the group be known.

5.4.2 Correlation of Soil Strength Parameters

It is important to recognize that the values of cohesion, friction angle, and unit weight that we provide herein are correlated from the standard penetration test (SPT) data collected during the exploration, and were not directly measured. For example, this exploration did not include direct measurement of cohesion and internal friction angle through triaxial shear testing. To provide directly measured values, additional (Shelby tube) samples would need to be collected and tested in the laboratory, which was not requested and is not provided as part of this report. The client therefore recognizes and assumes all risks associated with using these correlated values versus directly measured soil strength parameters.

5.4.3 Deep Foundation Soil Strength Parameters

We have estimated the soil parameters that may be assumed for helical pier foundation design, which may be performed by others. The estimated soil parameters are approximated for borings B-1A and B-1B at Bridge 1, and are provided in Table 5-3 below.

Table 5-2: Estimated Soil Strength Parameters at Bridge 1

Stratum No.	Depth (feet)	Correlated Soil Types	Unit Weight (pcf)		Friction Angle – ϕ (deg)	Minimum Estimated Cohesion – c (psf)
			Moist	Buoyant		
I	0 to 16	Upper Interbedded Clayey Sands, Sandy Clays, and Sands	120	58	25	0
II	16 to 55 (B-1A); 16 to 66 (B-1B)	Very Soft Fat Clays	105	43	0*	185
III	55 to 60 (B-1A); 66 to 70 (B-1B)	Loose to Dense Sands and Sand Mixtures	122	60	37	0

*assumes saturated clays below the water table

5.5 Lateral Earth Pressures

The lateral earth pressure coefficients given below may be used to design the pile caps and other earth retaining structures.

The values given in the following table assume placement and compaction of backfill around and behind these structures in accordance with the compaction recommendations given in the next section of this report. These values assume backfill generally classified as SM or SC soils according to the Unified Soil Classification system. These assumptions were made based upon the use of the on-site near surface clayey and silty sands of Stratum I (or similar imported fill materials) being used as the backfill material.

Table 5-3: Lateral Earth Pressure Coefficients

Support Condition	Angle of Internal Friction (ϕ')	Cohesion (lbs./sq.ft.)	Moist Unit Weight (γ)	Drained Static Earth Pressure Coefficient (K)
Active Condition (K_a)	25	0	120	0.41
At-Rest (K_o)	25	0	120	0.58
Passive (K_p)	25	0	120	2.5

A. The above values represent a fully-drained soil condition at or near the optimum moisture content. Where backfill soils are not fully drained, the lateral soil pressure must consider hydrostatic forces below the water level, and submerged soil unit weight.
B. A coefficient of sliding friction (tan δ) of 0.30 may be used in computation of the lateral sliding resistance.

For driven sheet pile walls that may be advanced into the deeper soil strata at this site at test locations R-1, R-2, and/or R-3, the following general soil strength parameters may apply. These values have been correlated from SPT data, and were not directly measured. The soil stratum depth ranges for each stratum may vary between the retaining wall locations. Please see the individual boring logs R-1, R-2, and R-3 for indications of the depths at which each stratum was observed.

Table 5-4: Estimated Soil Strength Parameters for Retaining Walls

Soil Stratum	USCS Soil Classification(s)	Moist Unit Weight (lbs./cu.ft.)	Buoyant Unit Weight (lbs./cu.ft.)	Effective Angle of Internal Friction (ϕ')	Cohesion (lbs./sq.ft.)
I	SP, SC, Sandy CL, Sandy CH	120	58	25	0
II	CH	105	43	0	185
III	SP, SC	122	60	37	0

Earth pressures should be calculated by the designer assuming the moist soil unit weight above the water table. Buoyant unit weights should be used in computations for soils below the water level. The designer shall consider all unbalanced water forces along with any surcharge or building loads. We note that the water levels can fluctuate and may vary at the time of construction.

5.6 New Pavement Recommendations

The following recommendations apply only to new pavement areas. We assume that new pavement subgrades will be constructed atop compacted structural fill soils compacted to at least 95 percent of the modified Proctor maximum dry density. We have performed our evaluations assuming that a CBR value of at least 10 percent will be available using imported sandy fill soils compacted to 95 percent. If soils exhibiting a CBR value of less than 10 percent at 95 percent compaction are to be used on this project, these recommendations may require revision.

- This CBR assumption requires that the upper 2 feet of subgrade along the northern portion of the trail, roughly between test locations T-1 and T-8, be built-up or removed and replaced with at least 2 feet thick of compacted sandy fill having a CBR of at least 10 percent. The CBR value of the native soils in this portion of the trail was only 1.5 percent.

We understand that the pavement sections should be able to accommodate the occasional lift bucket truck operated by Santee Cooper for servicing of overhead lighting along the path, as well as the occasional lawn maintenance vehicle. The recommended pavement section components are provided in the table below. The pavement thickness computations for flexible pavements were made using the AASHTO method, assuming an initial serviceability of 4.2 and a terminal serviceability index of 2.0, and a reliability factor of 90 percent. Assuming that only SCDOT approved source materials will be used in flexible pavement section construction, we used a structural layer coefficient of 0.44 for the HMA layers and a coefficient of 0.18 for the graded aggregate base course (GABC).

Table 5-5: Recommended Minimum Pavement Sections(a)

Pavement Area	Theoretical Allowable Traffic Load (ESALs)	HMA Surface Course Type C (inches)	HMA Intermediate Course Type C (inches)	Compacted SCDOT Graded Aggregate Base Course [GABC] (inches)
Light Duty Flexible (Asphalt)	12,800	2.0	---	6.0

(a)Single-stage construction and soil compaction as recommended is assumed; S&ME, Inc. must observe pavement subgrade preparation and pavement installation operations.

5.6.1 *General Pavement Construction Recommendations*

The following recommendations are provided regarding pavement construction:

1. Fill placed in pavement areas should be compacted as recommended previously in this report. Prior to pavement section installation, all exposed pavement area subgrades should be methodically proofrolled at final subgrade elevation under the observation of S&ME, Inc., and any identified unstable areas should be repaired as directed.
2. The stone base course underlying pavements should consist of a graded aggregate base course (GABC) as specified by the SCDOT 2007 Standard Specifications for Highway Construction, Section 305. Proposed materials for use should be provided by a SCDOT-approved source.

3. As stated in the SCDOT Section 305 specification, we recommend that all new base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC T-140). Base courses should not exhibit pumping or rutting under equipment traffic. Heavy compaction equipment is likely to be required in order to achieve the required base course compaction, and the moisture content of the material will likely need to be maintained very near the optimum moisture content in order to facilitate proper compaction. S&ME, Inc. should be contacted to perform field density and thickness testing of the base course prior to paving.
4. Experience indicates that a thin surface overlay of asphalt pavement may be required in about 7 to 10 years due to normal wear and weathering of the surface. Such wear is typically visible in several forms of pavement distress, such as aggregate exposure and polishing, aggregate stripping, asphalt bleeding, and various types of cracking. There are means to methodically estimate the remaining pavement life based on a systematic statistical evaluation of pavement distress density and mode of failure. We recommend the pavement be evaluated in about 7 years to assess the pavement condition and remaining life.
5. Construct the HMA surface course in accordance with the specifications of Section 403 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007 edition).
6. It is important that the asphaltic concrete be properly compacted, as specified in Section 401.4 of the SCDOT specification. Asphaltic concrete that is insufficiently compacted will show wear much more rapidly than if it were properly compacted.
7. Sufficient testing should be performed during flexible pavement installation to confirm that the required thickness, density, and quality requirements of the pavement specifications are followed.

6.0 Limitations of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations of the soils at the site to those encountered at our boring locations may not become evident until construction. If variations appear evident, then we should be provided the opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structure are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers. Assessment of site environmental conditions; sampling of soils, ground water or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts were beyond the scope of this geotechnical exploration.

Appendices

Appendix I

Site Vicinity Map

Test Location Sketch (overlaid on Aerial Photo)

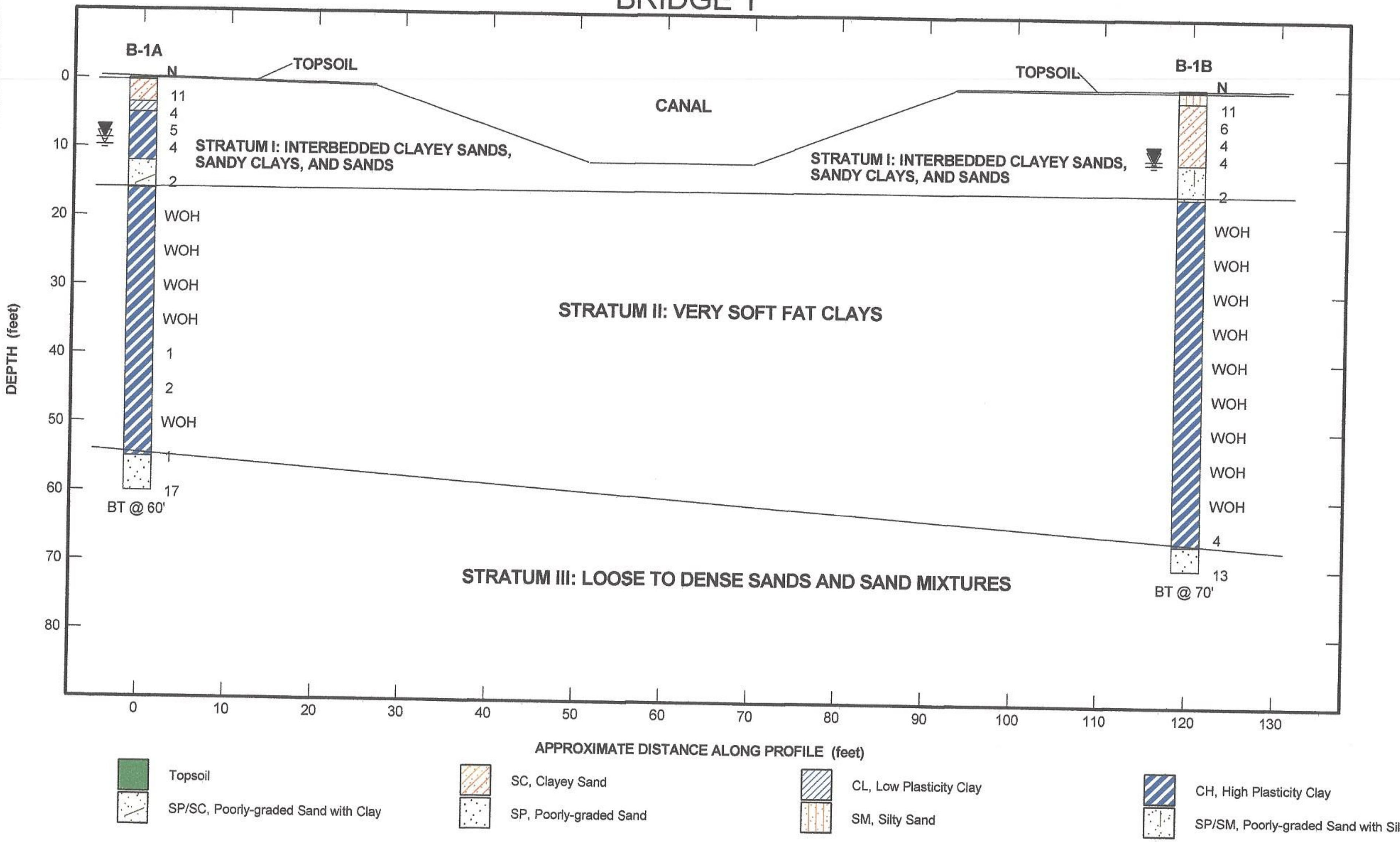
Interpreted Subsurface Soil Profile at Bridge 1
B-1A - B-1B

Interpreted Subsurface Soil Profile at Bridge 2
B-2A - B-2B

SCALE:	Not To Scale	S&ME	SITE VICINITY MAP Grand Linear Park Myrtle Beach, South Carolina	FIGURE NO 1
SOURCE:	Google Earth			
DATE:	June, 2017			
DRAWN BY:	KEF		JOB NO. 1463-17-018	

SCALE:	1"= 500'	AERIAL PHOTOGRAPH GRAND LINEAR PARK PROJECT MYRTLE BEACH, HORRY COUNTY, SC	S&ME WWW.SMEINC.COM	EXHIBIT #
SOURCE:	SCDNR GIS SITE			2
SOURCE DATE:	2006			
DATE:	MAY 2017	S & ME PROJECT # 1463-17-018		

BRIDGE 1



N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

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DATE:	6/15/17

S&ME

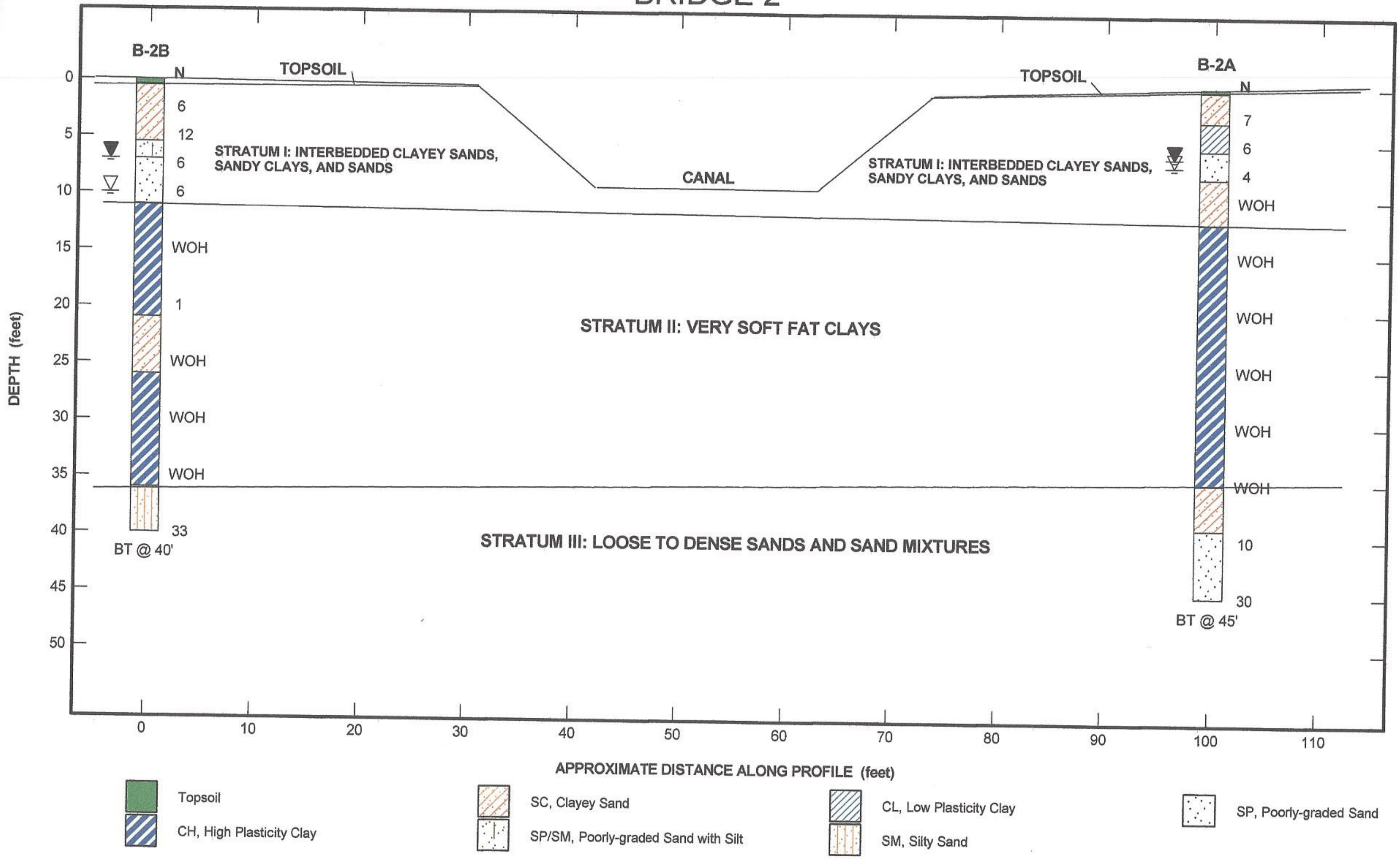
Diagram: Figure 3: Bridge 1 Interpreted Subsurface Profile

Project: Grand Linear Park

Location: Myrtle Beach, SC

Figure 3

BRIDGE 2



N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

JOB NO: 1463-17-018	S&ME	Diagram: Figure 4: Bridge 2 Interpreted Subsurface Profile	Figure 4
DATE: 6/15/17		Project: Grand Linear Park	
		Location: Myrtle Beach, SC	

Appendix II

Summary of Exploration Procedures

Soil Classification Chart

SPT Boring Logs

Summary of Exploration Procedures

The American Society for Testing and Materials (ASTM) publishes standard methods to explore soil, rock and ground water conditions in Practice D-420-98, *"Standard Guide to Site Characterization for Engineering Design and Construction Purposes."* The boring and sampling plan must consider the geologic or topographic setting. It must consider the proposed construction. It must also allow for the background, training, and experience of the geotechnical engineer. While the scope and extent of the exploration may vary with the objectives of the client, each exploration includes the following key tasks:

- Reconnaissance of the Project Area
- Preparation of Exploration Plan
- Layout and Access to Field Sampling Locations
- Field Sampling and Testing of Earth Materials
- Laboratory Evaluation of Recovered Field Samples
- Evaluation of Subsurface Conditions

The standard methods do not apply to all conditions or to every site. Nor do they replace education and experience, which together make up engineering judgment. Finally, ASTM D 420 does not apply to environmental investigations.

Reconnaissance of the Project Area

We walked over the site to note land use, topography, ground cover, and surface drainage. We observed general access to proposed sampling points and noted any existing structures.

Checks for Hazardous Conditions - State law requires that we notify the South Carolina (SC 811) before we drill or excavate at any site. SC 811 is operated by the major water, sewer, electrical, telephone, CATV, and natural gas suppliers of South Carolina. SC 811 forwarded our location request to the participating utilities. Location crews then marked buried lines with colored flags within 72 hours. They did not mark utility lines beyond junction boxes or meters. We checked proposed sampling points for conflicts with marked utilities, overhead power lines, tree limbs, or man-made structures during the site walkover.

Boring and Sampling

Standard Penetration Testing with Rotary Wash

A rotary drilling process was used to advance the hole and a heavy drilling fluid was circulated in the bore holes to stabilize the sides and flush the cuttings. Temporary casing was also used to stabilize the boreholes when needed.

Soil sampling and penetration testing were performed in general accordance with ASTM D 1586, *"Standard Test Method for Penetration Test and Split Barrel Sampling of Soils.* At regular intervals, drilling tools were removed and soil samples were obtained with a standard 1.4 inch I. D., two-inch O. D., split barrel sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional 12 to 18 inches (depending on whether am 18-inch or 24-inch split-spoon was being used) with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through the two final six inch increments was recorded as the penetration resistance (SPT N) value. The N-value, when properly interpreted by qualified professional staff, is an index of the soil strength and foundation support capability.

Bulk Samples

At selected locations and depths, representative bulk samples of the soils were obtained by augering into the embankment using an 8-inch O.D. hollow-stem auger, and collecting shovel loads from the cuttings or spoils brought to the surface, until a sample of 30 to 50 lbs was obtained. The sample was placed in a cloth or plastic sack marked with appropriate descriptive information. Samples were protected from freezing at all times.

Water Level Measurement

Subsurface water levels in the boreholes were measured during the onsite exploration and after a period of about 24 hours by measuring depths from the existing grade to the current water level using an electronic water-sensing tape.

Backfilling of Borings

Once subsurface water levels were obtained, boring spoils were backfilled into the open bore holes. Bore holes were backfilled to the existing ground surface.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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