COUNTY OF CHARLOTTE
PURCHASING DIVISION

Charlotte County Administration Center
18500 Murdock Circle
Port Charlotte, Florida 33948-1094
(941) 743-1378

via fax

TO: **PROSPECTIVE BIDDERS**

DATE: **November 6, 2018**

RE: **ADDENDUM #1 BID NO. 2019000063 – STORMWATER CONTROL STRUCTURE HAVERHILL 4.84 WATERWAY**

BID OPENING DATE: 2:00 p.m. (EST) NOVEMBER 28, 2018

Bidders are hereby notified that this Addendum shall be made a part of the documents. The intent is to add to, modify, or clarify the documents. Should any of these items have an effect on price, such changes shall be included in the price bid. These items have the same force and effect as if contained in the original.

QUESTION #1: Can the County supply a GeoTech report?
ANSWER: Attached is the GeoTech report that was requested during the pre-bid.

This addendum is binding and is to be considered as if contained within the original bid documents of Bid No. 2019000063. Bidders are required to acknowledge receipt of this addendum on their bid forms.

Cheryl Martin for
Kimberly A. Corbett, C.P.M., CPPB
Senior Division Manager Purchasing

KAC/vs

cc: Karen Bliss, Project Manager
File

GEOTECHNICAL REPORT
FOR REPLACEMENT OF
WATER CONTROL STRUCTURE HAV 4.84,
PEACHLAND BOULEVARD
AT HAVERHILL WATERWAY
PORT CHARLOTTE,
CHARLOTTE COUNTY, FLORIDA

Ardaman & Associates, Inc.

CORPORATE HEADQUARTERS

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Florida: Bartow, Cocoa, Fort Myers, Miami, Orlando, Port St. Lucie, Sarasota, Tallahassee, Tampa, West Palm Beach
Louisiana: Baton Rouge, Monroe, New Orleans, Shreveport

Ardaman & Associates, Inc.

Geotechnical, Environmental and
Materials Consultants

August 15, 2018
File No. 17-7043

TO: Johnson Engineering, Inc.
18501 Murdock Circle, Suite 404
Port Charlotte, Florida 33948

Attention: Mr. Christopher D. Beers, P.E., P.S.M.
Email: cdb@johnsoneng.com

SUBJECT: Geotechnical Report for Replacement of Water Control Structure HAV 4.84,
Peachland Boulevard at Haverhill Waterway
Port Charlotte, Charlotte County, Florida

Dear Mr. Beers:

Ardaman & Associates, Inc. is pleased to submit this report of our geotechnical exploration and engineering services for the above-referenced project. Our services were provided in general accordance with those outlined in our agreement dated February 7, 2017. The purpose of this exploration was to evaluate the general stratification and engineering properties of the subsurface soils at the subject site; and to provide recommendations related to foundation design and earthwork; and to assess the need for subsurface seepage cut-off at the water control structure location.

This geotechnical study covers foundation soils well within the influence of foundation loads, but does not cover deep soil or bedrock strata. The assessment of site environmental conditions for the presence of pollutants in the soil, rock, or groundwater at this site was beyond the scope of this exploration.

PROJECT INFORMATION

The subject site is located at the Haverhill Waterway crossing of Peachland Boulevard, which is between the Peachland Boulevard intersection with Waterside Street and Harbor Boulevard in Port Charlotte, Florida.

We have also been provided with the “90% Submittal” construction plans for the structure. These included Johnson Engineering sheet Nos. C1 to C10, C10.1 to C10.5, C11, C11.1, C12 to C15, S1 and S2 (project No. 20118682-024, dated January 2018).

We understand that the existing corrugated metal pipes will be replaced with two (2) reinforced concrete box culverts with headwalls and a water control weir, all supported on shallow foundation systems. A review of the design plans indicates the following:

Structure ID	Culvert Configuration	Culvert Length	Upstream Invert Elev. (NGVD29)	Downstream Invert Elev. (NGVD29)
HAV 4.84	2 – 10’ span x 9’ rise box culverts	±77 feet	2.25 feet	2.20 feet

FIELD EXPLORATION

Our field exploration program included conducting three (3) Standard Penetration Test (SPT) borings and one (1) hand auger boring at the locations shown on the attached Figure 1. The SPT borings were performed to determine the nature and condition of the subsurface soils to a maximum depth of 40 feet below the existing ground surface. The SPT soil borings were initially drilled to a depth of 4.5 feet with a hand auger at each boring location, in order to avoid damaging possible underground utilities. The location of boring No. 4 was not accessible to our SPT boring drill rig, so only a hand auger boring to a depth of 6 feet below the ground surface was performed at this location. The equipment and procedures used in the borings are described in Appendix I of this report.

Test borings were located in the field utilizing an aerial photograph of the site and visual reckoning to available landmarks. The locations should be considered accurate only to the degree implied by the method used. Should more accurate locations be required, a registered land surveyor should be retained.

The existing ground surface elevations provided on the topographic survey and approximate boring locations were used to estimate the existing ground surface elevation (NGVD29) at each boring location. These elevations should be considered approximations, only. The boring profile elevations shown on Figure 1 are based upon these estimates.

LABORATORY TESTING

The field soil boring logs and recovered soil samples were transported to our Sarasota office following the completion of the field exploration activities. Each representative sample was examined by a geotechnical engineer in our laboratory for visual classification and assignment of laboratory tests.

The laboratory tests performed included determining the fines (silt and clay) content and water (natural moisture) content of selected samples. The test results are presented on the graphic soil profiles on Figure 1, at the depth from which the respective sample was recovered.

The tests were performed in accordance with the applicable ASTM standards, which are listed in Appendix I. The soil descriptions shown on the soil profiles are based on the laboratory test results and a visual classification procedure in general accordance with the Unified Soil Classification System (ASTM D-2487 or D-2488).

SUBSURFACE CONDITIONS

The delineation of the vertical extent and a description of each soil stratum discovered in the course of this geotechnical study are given in the soil boring logs (graphic soil profiles) on Figure 1 of this report. The soil boring logs were prepared by a geotechnical engineer based upon a combination of technical review of the field soil boring logs, the laboratory test results and visual classification of the recovered soil samples.

It should be noted that the stratification lines shown on the logs indicate a transition from one soil type to another. The actual boundary between the soil strata may be gradual or indistinct. Consequently, the stratification boundary lines shown on the soil boring profiles represent our estimate of the location of the transition between distinct soil strata. They are in no way intended to designate a depth of exact geologic change.

Furthermore, the recommendations contained in this report are based on the soil profiles encountered in the borings. While a boring is representative of subsurface conditions at its respective location and vertical reach, local variations which are characteristic of the subsurface materials of the region, or which may be due to man-made alteration, may be encountered.

A generalized description of the subsurface soil profile encountered is provided below. The elevations are based upon and referenced to the approximate ground surface elevation at the boring locations.

Elevation (feet NGVD29)	Soil Description
Surface to +13	Fine sand (SP) and fine sand with clay (SP-SC); some trace shell or organics
+13 to +10	Loose to medium dense fine sand (SP), fine sand with silt (SP-SM) and silty fine sand with organics (SM)
+10 to -9	Loose to medium dense fine sand (SP) and fine sand with silt (SP-SM)
-9 to -14	Loose fine sand with silt (SP-SM) and very silty fine sand (SM)
-14 to -19	Loose very silty fine sand (SM) and soft clay (CL-CH)
-19 to -22	Loose to very loose fine sand with silt (SP-SM) to silty fine sand (SM)

As indicated on the soil boring logs, the water levels in the boreholes were measured to be approximately 6½ to 8½ feet below the existing ground surface at the time of the field exploration. This corresponds to an elevation of approximately +9.5 to +10.5 feet NGVD29. This water level reading may differ from the actual stable groundwater table due to variations in the permeability of soil layers. The degree of accuracy of the reported water level is also related to the time allowed for the borehole water level to come to equilibrium. It should be noted that fluctuations in the groundwater level may occur due to variations in rainfall and other environmental or physical factors at the time measurements are made.

EVALUATION AND RECOMMENDATIONS

The following evaluations and recommendations are based on the project information provided and the subsurface soil conditions encountered during this geotechnical study.

<u>Soil Evaluation</u>

The SPT borings generally encountered sand with varying amounts of silt and/or clay from the ground surface to a depth of approximately 27 to 32 feet below the existing ground surface (to approximately elevation -9 to -15 feet NGVD29). These soils included mostly fine sand (SP) and fine sand with silt (SP-SM), with some layers of fine sand with clay (SP-SC) and silty fine sand (SM), having fines (silt and clay) contents ranging from less than 5 percent to as much at 11 percent, based upon visual classification and laboratory tests. Some strata also contained a trace amount of shell fragments or trace to some organics. These soils were generally in a loose to medium dense state.

These soils were underlain by an approximately 2 to 5 feet thick layer of loose, very silty fine sand (SM), having a fines (primarily silt) content of approximately 44 percent. The physical properties of this layer are likely more similar to that of a firm to stiff sandy silt, than to a typical silty sand, due to the relatively high fines content.

Beneath the very silty fine sand layer (i.e. below an elevation of approximately -14 to -17 feet NGVD29), the soils consisted of an approximately 3 to 5 feet thick layer of soft clay, underlain by very loose fine sand with silt (SP-SM) to silty fine sand (SM) to our deepest end of boring depth (at approximately elevation -22.5 feet NGVD29).

Based on our analyses, the soils are be capable of supporting the proposed reinforced concrete box culverts (hereafter referred to as “culverts”), headwalls and weir structures, if the soils are properly prepared per the recommendations of this report. The following soil preparation and other recommendations are made to minimize settlement and to better control seepage.

Site Preparation

Culvert construction activities require that a stable soil contact elevation be available to support the culverts. Surface water and groundwater control procedures (i.e. dewatering during construction) will be necessary to provide the aforementioned stable soil contact elevation. Positive site drainage should be established early during construction in order to reduce ponding of surface water during heavy or prolonged rainfall. Means and methods of groundwater and surface water control should be the responsibility of the contractor.

Typical methods for surface water control include damming off the channels and diverting the water through a temporary channel, or pumping the water around construction activities. We anticipate that, even if the stream is diverted, water will seep into the channel from the stream bed and sides of the ditch, potentially creating a loosening of foundation contact soils. If this is the case, well points will need to be provided in the construction area to maintain groundwater levels at least 2 feet below the working area surface, to allow for a stable excavation, proper material placement and adequate compaction.

The results of our borings and the provided topographic survey indicate that the elevation of the proposed culvert, headwall and weir foundations (approximately 0 to +1 feet NGVD29) will likely be within soil strata consisting of loose to medium dense fine sand with silt (SP-SM). These soils are relatively permeable, due to their relatively low fines (silt and clay) content. This will need to be considered in the design of the dewatering system.

Drying these soils to near their optimum moisture content (as determined by Modified Proctor compaction tests prior to or during construction) will be necessary in order to be adequately compacted.

The “footprint” of the proposed culvert, headwall and weir foundations, plus a margin of at least 5 feet outside the foundation perimeters, should be stripped of all surface vegetation, stumps, debris, or other deleterious organic soils encountered during earthwork activities. Any encountered soils that are not select material must be removed or “demucked” to their entire vertical limits and to a minimum horizontal distance of 5 feet beyond the edges of the structure. Any soft or organic sediment in the stream or ditch channels should be removed from beneath the culvert foundations and at a 1:1 downward slope beyond the culvert foundations. Any excavated deleterious soils, if encountered, must not be used as structural fill material and should be disposed of as directed by the engineer.

If it is necessary to stabilize excessively wet or yielding materials at the bottom of the excavation (at the foundation elevation) we recommend that the foundation subgrade be undercut a minimum of 12-inches below the foundation bottom and backfilled with clean, compacted select sand fill. The select sand fill shall consist of fine sand containing less than 12 percent (by weight) passing the No. 200 sieve (ASTM D1140) and classifying as “SP” or “SP-SM” per the Unified Soil Classification System (ASTM D2487). The fill shall be compacted to at least 95 percent of Modified Proctor (ASTM D1557) maximum density.

Where fill is required to raise the existing surface beneath the proposed culvert system to final grade, or as backfill surrounding and overlying the culverts, This fill shall consist of fine sand containing less than 12 percent (by weight) passing the No. 200 sieve and classifying as “SP” or “SP-SM” per the Unified Soil Classification System (ASTM D2487). The fill should be placed in lifts not exceeding 12 inches in loose thickness. Each lift should be compacted by repeated passes with appropriate equipment to achieve a minimum of 95 percent of the Modified Proctor maximum dry density (ASTM D1557). The engineer may specify greater compaction, such as 98 percent of Modified Proctor, to a certain depth beneath pavements, however. Density tests to confirm compaction should be performed in each lift of fill before the next lift is placed. The placement of structural fill and compaction operations should continue until the desired elevation is achieved.

Care should be exercised when using compaction equipment adjacent to the culvert walls or headwalls to avoid overstressing or causing damage to the structure. Areas that do not require undercutting or filling should also be compacted with appropriate equipment, to achieve this same density (95 percent of the Modified Proctor) to a depth of at least 1 foot below the compacted surface.

Foundation Recommendations

Following preparation of the subgrade soils as described above, the shallow foundations for support of headwalls, concentrated (point) loads, or similar may be designed for an allowable soil contact pressure no greater than 2,000 pounds per square foot, including design dead load plus live load.

Culvert Settlement

For the purposes of our analyses, we modeled the box culvert base as a 24 feet wide by 77 feet long mat foundation.

Based upon our review of the construction plans for the proposed structure, the weight of the proposed construction (culvert and fill) is approximately the same as or less than the effective weight of the existing condition (pipes and fill). The net increase in pressure transmitted to the subgrade soils is, therefore, small to negligible and should result in only minor settlement, if any. This assumes proper dewatering, preparation and compaction of the subgrade soils as described above.

Settlement at the headwalls may be greater, however, due to the placement of fill significantly above the existing ground surface. Settlement of the headwalls is not expected to exceed 1 inch, however. Due to the sandy nature of the underlying soils, this settlement should occur primarily during placement of the fill.

At-Rest Lateral Earth Pressures Acting on Culverts

At-rest earth pressures acting on the culverts side-walls include lateral loading due to soil and water. The lateral earth pressure will be a function of both the submerged soil unit weight and the depth below the ground surface. For clean, well compacted, granular backfill, the coefficient of lateral earth pressure at rest is 0.5. Care should be exercised to avoid over-compacting the soils close to the walls to avoid increasing the pressure against the wall above this coefficient value.

Hydrostatic pressure on the culvert should also be considered in structural design of the culverts, in addition to the lateral earth pressure.

Earth Pressure on Shoring and Bracing

If temporary shoring and bracing is required for any excavations, the system should be designed to resist lateral earth pressure. The design earth pressure will be a function of the flexibility of the shoring and bracing system. For a flexible system restrained by braces placed as the excavation proceeds, the design earth pressure for shoring and bracing can be computed using a uniform earth pressure distribution with width. It is recommended that well-points be used to dewater the soils around the excavations, since the addition of hydrostatic loads can more than double the pressure on the shoring and bracing. For such dewatered excavations, we recommend the following uniform pressure distribution over the full braced height as follows:

Uniform Soil Pressure Distribution, $p = 0.65\ K_a \gamma_s H$

where: p = uniform pressure distribution for design of braced excavation
K_a = coefficient of active earth pressure = 0.33
γ_s = unit weight of saturated soils = 120 pcf
H = depth of excavation

An appropriate factor of safety should be applied for the design of braced excavations.

Lateral pressure distributions determined in accordance with the above do not consider hydrostatic pressures or surcharge loads. To the extent that such pressures and forces may act on the wall, they should be included in the design.

Construction equipment and excavated soils should be kept a minimum distance of 5 feet from the edge of the braced or shored excavation. Soil placed adjacent to (maintaining a minimum 5-

foot horizontal clearance) the braced or shored excavation should have a slope no steeper than 2.0H to 1.0V.

Means and methods of excavation and bracing should be the responsibility of the contractor; however, excavation and/or bracing should, at a minimum, adhere to the requirements of the Occupational Safety Health Administration (OSHA).

Uplift Resistance

Permanent structures submerged below the water table will be subjected to uplift forces caused by buoyancy. The components resisting this buoyancy include: 1) the total unit weight of the structure divided by an appropriate factor of safety; 2) the buoyant unit weight of the soil overlying the structure; and 3) the shear forces acting on the shear planes that radiate vertically upward from the perimeter of the edges of the structure to the ground surface. The allowable unit shearing resistance may be determined by the following formula:

Allowable Unit Shear Resistance

Above the Groundwater Table, $F = K_o \tan(\tfrac{2}{3}\phi)\gamma_m h$

Below the Groundwater Table, $F = \dfrac{K_o \tan(\tfrac{2}{3}\phi)\left[(h-h_w)\gamma_w + h_w\gamma_b\right]}{SF}$

where: F = unit shear resistance (psf)
K_o = coefficient of earth pressure at rest = 0.5
γ_m = unit weight of moist soil = 110 pcf
γ_b = buoyant unit weight of soil = 52 pcf
h = vertical depth below grade at which shearing resistance is determined
h_w = height of groundwater level above depth at which shearing resistance is determined
ϕ = angle of internal friction of the soil
SF = Safety Factor = 2.0

The values given for the above parameters assume that the permanent structures are surrounded by clean, well compacted, granular backfill that extends horizontally to at least 2 feet behind the structures.

The factor of safety against uplift (F_{su}) is the total of the resisting forces divided by the hydrostatic uplift force. Based upon the design drawings provided, the F_{su} is at least 1.5 for the box culverts and also at least 1.5 for the weir structure. This assumes that the hydraulic head difference across the weir is never greater than 5.5 feet and that the ditch invert upstream of the weir is no lower than 8.0 feet NGVD29. If the head difference may be greater than 5.5 feet or if the ditch upstream may erode to lower than 8.0 feet NGVD29, a seepage cutoff wall (such as a sheet pile wall) beneath the weir may be necessary to decrease hydrostatic uplift pressures and increase the resistance of the weir structure to hydrostatic uplift.

Seepage Cut-Off

Recommendations relative to seepage cut-offs for the box culverts and weir structure will be discusses separately, as follows. This section only discusses erosion caused by underground seepage. Resistance to surface erosion is not addressed and should be designed for by others.

Box Culverts

Since the subject culverts are open-ended, free-flowing culverts (no weir or other water level control feature, other than the proposed weir structure approximately 20 feet upstream from the culverts), there should normally be only a small hydraulic head difference from the upstream to the downstream ends of the culverts. Seepage-related stability issues, such as seepage induced subsurface erosion (soil piping) beneath the culverts should, therefore, not be a concern since the hydraulic gradient of seepage from one end to the other should be negligible. If the sustained head difference from the upstream end to the downstream end of the culvert may be greater than 3 feet, however, some form of seepage cut-off or seepage interceptor should be considered.

If a coarse aggregate (washed shell or gravel) bedding is used beneath the culverts, it would be prudent to incorporate measures in the design that discourages seepage flow through the bedding. This could consist of a “key” in the headwall footings that extends a few feet below the bedding, so that the bedding is not in direct or near-direct contact with the drainage canal.

Weir Structure

Based upon the design drawings provided, the proposed weir is to be a reinforced concrete structure supported on a 2 feet thick concrete mat that extends 6 feet beyond the upstream side of the weir wall. The weir wall is an approximately 1 foot thick concrete wall with a top elevation of +15.8 feet NGVD29 and a 44 feet wide weir notch at an elevation of +12.5 feet NGVD29.

As discussed previously in this report, there may be differential settlement between the weir structure and the culvert structure/headwall. It is imperative that the construction joint between the weir structure's mat foundation and the culvert structure/headwall foundations be water tight, so that groundwater seepage does not "spring up" through the joints, possibly eroding soil up through the joints and undermining the foundations.

If water-tight joints cannot be achieved, or the joints cannot be otherwise protected to prevent soil from moving through the joints, a seepage cutoff wall (such as a sheet pile wall) beneath the weir could be used to decrease seepage pressures beneath the foundations. To be most effective, the cutoff wall should be located near the up-gradient (north) edge of the weir foundation mat, extend around the east and west sides of the mat and continue to the east and west ends of the headwall. The top of the cutoff wall should be cast into the foundations, or otherwise attached in a water-tight manor. We recommend that the bottom of the cutoff wall be at or lower than an elevation of (-)19 feet NGVD29.

Temporary Sheet Pile Wall

We understand that temporary sheet pile walls for cofferdam construction may be used to seal off the upstream and downstream water flow during culvert replacement construction activities. A sheet-pile retaining wall generally consists of steel or aluminum sheet piles driven to sufficient depth to provide lateral stability and to serve as an effective seepage barrier. It should be noted that the joints between adjacent sheets are not 100 percent waterproof.

The required minimum depth of the sheet piles for lateral (rotational) stability would depend upon the height of the water impounded, the ground surface elevation at the wall location, the subsurface soil conditions and other factors. Cofferdam design is typically provided by the contractor, but we would be pleased to provide design assistance (suitable soil parameters), if desired.

The cofferdam sheet piles would be a more effective seepage barrier if they were driven a few feet into the very silty fine sand and clay strata that were encountered below an elevation of approximately (-)10 to (-)15 feet NGVD29, since these soils likely have very low permeability and would restrict seepage beneath the sheet piles. The sheet piles may need to be deeper than this, however, for lateral stability.

The sheet piles should be installed by a percussion or vibratory hammer that is compatible with the selected sheet pile section and length. We do not recommend jetting the sheet piles into place, as this may result in excessive lateral movements under load. The contractor should provide sheet pile driving equipment capable of installing the sheet pile walls through the materials encountered in our borings. We note that settlement monitoring and settlement surveys for structures within a certain zone of influence may be required depending upon hammer energy.

Field Observations

Site preparation operations, including preparation of foundation bearing surfaces and compaction of any structural fill, should be observed and tested by an Ardaman & Associates geotechnical engineer or his representative. Observations by our representative are necessary to verify that subsurface conditions (which are revealed during the site preparation operations) are consistent with those found during this geotechnical study; to confirm that the foundation design is being constructed as indicated in the approved construction documents; and to confirm that the earthwork procedures are completed in accordance with the recommendations contained in this report.

CLOSURE

The recommendations provided above are based in part on project information provided to us, and they only apply to this specific project and site. If any of the project information is incorrect or if additional information becomes available, the correct or additional data should be conveyed to us for review. We can then modify our recommendations if they are inappropriate for the proposed project.

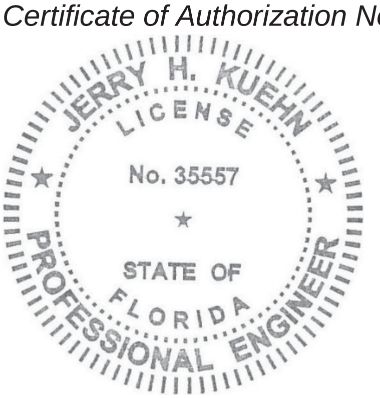
Regardless of the thoroughness of a geotechnical exploration, there is always a possibility that conditions between borings will be different from those at specific boring locations and that conditions will not be as anticipated by the designers or contractors. In addition, the construction process may alter the soil conditions. Therefore, experienced geotechnical personnel should observe and document the construction procedures used and the conditions encountered. Unanticipated conditions and inadequate procedures should be reported to the design team along with timely recommendations to solve any problems created. We recommend that the owner retain Ardaman & Associates to provide construction consulting and testing services based upon our familiarity with the project and the subsurface conditions.

When the final design drawings and specifications are completed, Ardaman & Associates should be engaged to review them to determine whether changes in the original design concept may have affected the validity of our recommendations, and to confirm that these recommendations have been implemented in the design drawings and specifications.

We appreciate the opportunity to be of service. Please contact our office if you should you have any questions in regards to this report, or if we can be of any further assistance.

Very truly yours,

ARDAMAN & ASSOCIATES, INC.
Certificate of Authorization No. 5950



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Jerry H Kuehn
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Jerry H. Kuehn, P.E.
Senior Project Engineer
Fl. License No. 35557

Gregory S. Stevens, P.E.
Project Engineer
Fl. License No. 71511

JHK/GSS:ly

APPENDIX I

Soil Boring, Sampling and Test Methods

SOIL BORING, SAMPLING AND TESTING METHODS

Standard Penetration Test

The Standard Penetration Test (SPT) is a widely accepted method of in situ testing of foundation soils (ASTM D-1586). A 2-foot long, 2-inch O.D. split-barrel sampler attached to the end of a string of drilling rods is driven 18 inches into the ground by successive blows of a 140-pound hammer freely dropping 30 inches. The number of blows needed for each 6 inches of penetration is recorded. The sum of the blows required for penetration of the second and third 6-inch increments of penetration constitutes the test result or N-value. After the test, the sampler is extracted from the ground and opened to allow visual examination and classification of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing a conservative estimate of the behavior of soils under load. The following tables relate N-values to a qualitative description of soil density and, for cohesive soils, an approximate unconfined compressive strength (Qu):

Cohesionless Soils:

N-Value	Description
0 to 4	Very loose
4 to 10	Loose
10 to 30	Medium dense
30 to 50	Dense
Above 50	Very dense

Cohesive Soils:

N-Value	Description	Qu (ton/ft^2)
0 to 2	Very soft	Below 1/4
2 to 4	Soft	1/4 to 1/2
4 to 8	Medium stiff	1/2 to 1
8 to 15	Stiff	1 to 2
15 to 30	Very stiff	2 to 4
Above 30	Hard	Above 4

The tests are usually performed at 5-foot intervals. However, more frequent or continuous testing is done by our firm through depths where a more accurate definition of the soils is required. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is a bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, NX-size flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid.

Representative split-spoon samples from each sampling interval and from every different stratum are brought to our laboratory in air-tight jars for further evaluation and testing, if necessary. After thorough examination and testing of the samples, the samples are discarded unless prior arrangements have been made. After completion of a test boring, the hole is kept open until a steady state groundwater level is recorded. The hole is then sealed, if necessary, and backfilled.

A hammer with an automatic drop release (auto-hammer) is sometimes used. In this case, a correction factor is applied to the raw blow counts, since the energy efficiency of the auto-hammer is greater than that of the safety hammer. Based upon calibration of the auto-hammer (per ASTM D4633) and standard practice, we use a multiplier of 1.24 to correct the auto-hammer blow counts to equivalent safety hammer "N" values.

Hand Auger Borings

Hand auger borings are used, if soil conditions are favorable, when the soil strata are to be determined within a shallow (approximately 5 to 9 feet) depth or when access is not available to power drilling equipment. A 3-inch diameter, hand bucket auger with a cutting head is simultaneously turned and pressed into the ground. The bucket auger is retrieved to the surface at approximately 6-inch intervals and its contents emptied for inspection. The soil sample so obtained is classified and representative samples put in bags or jars and transported to the laboratory for further classification and testing.

Laboratory Test Methods

Soil samples returned to our laboratory are examined by a geotechnical engineer or geotechnician to obtain more accurate descriptions of the soil strata. Laboratory testing is performed on selected samples as deemed necessary to aid in soil classification and to further define engineering properties of the soils. The test results are presented on the soil boring logs at the depths at which the respective sample was recovered, except that grain size distributions or selected other test results may be presented on separate tables, figures or plates as described in this report. The soil descriptions shown on the logs are based upon a visual-manual classification procedure in general accordance with the Unified Soil Classification System (ASTM D-2488-84) and standard practice. Following is a list of abbreviations which may be used on the boring logs or elsewhere in this report.

-200	- Fines Content (percent passing the No. 200 sieve); ASTM D1140
DD	- Dry Density of Undisturbed Sample; ASTM D2937
Gs	- Specific Gravity of Soil; ASTM D854
k	- Hydraulic Conductivity (Coefficient of Permeability)
LL	- Liquid Limit; ASTM D423
OC	- Organic Content; ASTM D2974
pH	- pH of Soil; ASTM D2976
PI	- Plasticity Index (LL-PL); ASTM D424
PL	- Plastic Limit; ASTM D424
Qp	- Unconfined Compressive Strength by Pocket Penetrometer;
Qu	- Unconfined Compressive Strength; ASTM D2166 (soil), D7012 (rock)
SL	- Shrinkage Limit; ASTM D427
ST	- Splitting Tensile Strength; ASTM D3967 (rock)
USCS	- Unified Soil Classification System; ASTM D2487, D2488
w	- Water (Moisture) Content; ASTM D2216

Soil Classifications

The soil descriptions presented on the soil boring logs are based upon the Unified Soil Classification System (USCS), which is the generally accepted method (ASTM D-2487 and D-2488) for classifying soils for engineering purposes. The following modifiers are the most commonly used in the descriptions.

For Sands:	Modifier	Fines, Sand or Gravel Content*
	with silt or with clay	5% to 12% fines
	silty or clayey	12% to 50% fines
	with gravel or with shell	15% to 50% gravel or shell

For Silts or Clays:	Modifier	Fines, Sand or Gravel Content*
	with sand	15% to 30% sand and gravel; and % sand > % gravel
	sandy	30% to 50% sand and gravel; and % sand > % gravel
	with gravel	15% to 30% sand and gravel; and % sand < % gravel
	gravelly	30% to 50% sand and gravel; and % sand < % gravel

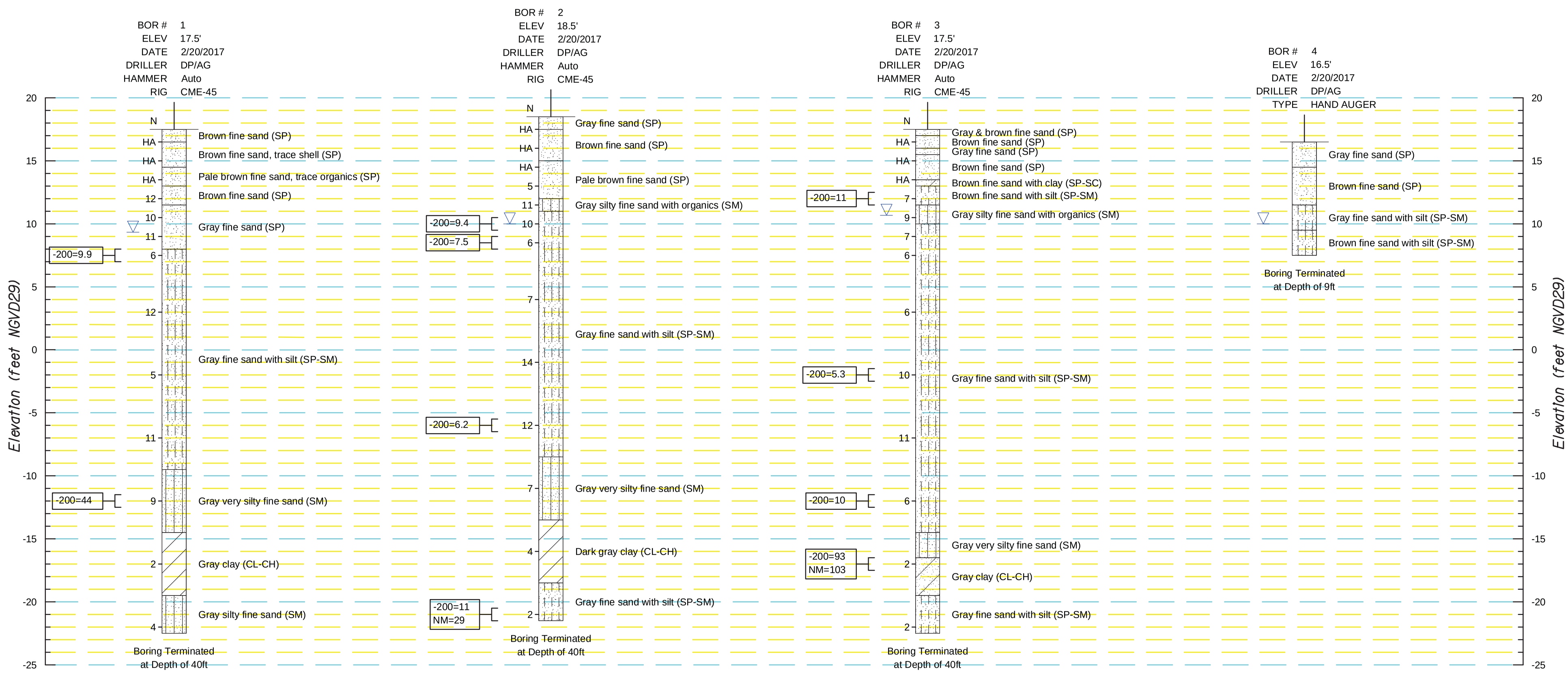
* may be determined by laboratory testing or estimated by visual/manual procedures. Fines content is the combined silt and clay content, or the percent passing the No. 200 sieve.

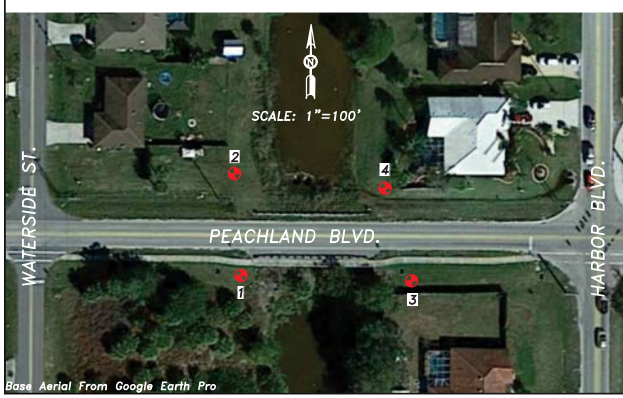
The USCS also uses a set of Group Symbols, which may also be listed on the soil boring logs. The following is a summary of these.

Group Symbol	General Group Name*	Group Symbol	General Group Name*
GW	Well-graded gravel	SW	Well-graded sand
GP	Poorly graded gravel	SP	Poorly graded sand
GW-GM	Well-graded gravel with silt	SW-SM	Well-graded sand with silt
GW-GC	Well-graded gravel with clay	SW-SC	Well-graded sand with clay
GP-GM	Poorly graded gravel with silt	SP-SM	Poorly graded sand with silt
GP-GC	Poorly graded gravel with clay	SP-SC	Poorly graded sand with clay
GM	Silty gravel	SM	Silty sand
GC	Clayey gravel	SC	Clayey sand
GC-GM	Silty, clayey gravel	SC-SM	Silty, clayey sand
CL	Lean clay	ML	Silt
CL-ML	Silty clay	MH	Elastic silt
CH	Fat clay	OL or OH	Organic silt or organic clay

* Group names may also include other modifiers, per standard or local practice.

Other soil classification standards may be used, depending on the project requirements. The AASHTO classification system is commonly used for highway design purposes and the USDA soil textural classifications are commonly used for septic (on-site sewage disposal) system design purposes.





TEST BORING LOCATIONS

LEGEND

∇	GROUNDWATER LEVEL MEASURED ON DATE DRILLED
N	SPT N-VALUE IN BLOWS PER FOOT (UNLESS OTHERWISE NOTED)
	SPT N VALUES CONVERTED TO EQUIVALENT SAFETY HAMMER
HA	HAND AUGER
-200	FINES PASSING THE NO. 200 SIEVE (%)
NM	NATURAL MOISTURE CONTENT

GRANULAR MATERIALS– RELATIVE DENSITY	SPT (BLOWS/FOOT)
VERY LOOSE	LESS THAN 4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FOOT)
VERY SOFT	LESS THAN 2
SOFT	2–4
FIRM	4–8
STIFF	8–15
VERY STIFF	15–30
HARD	GREATER THAN 30

Ardaman & Associates, Inc.
Geotechnical, Environmental and Materials Consultants

Test Locations/Soil Boring Logs
Stormwater Structure HAV 4.8
Peachland Blvd.
Between Waterside & Harbor Blvd.
Port Charlotte, Charlotte County, Florida

DRAWN BY: KGS	CHECKED BY:	DATE: 2/24/17
FILE NO. 17–7043	APPROVED BY:	FIGURE: 1