STORMWATER MANAGEMENT
PERMIT APPLICATION
FOR
Second Street Station II
Fort Pierce, Florida
January 2018

John H. Blum, P.E.
Florida Reg. No. 45813

PREPARED BY

CARTER ASSOCIATES, INC.
CONSULTING ENGINEERS AND LAND SURVEYORS
C.O.A. #205
1708 21st Street
Vero Beach, FL 32960
(772) 562-4191

TABLE OF CONTENTS

TITLE	PAGE

I. INTRODUCTION

The proposed project location is 100 Avenue A Fort Pierce, Florida. Currently the site is a marl parking lot used as over flow parking for the adjacent properties. The existing topography of the site allows for runoff to flow to the southeast of the site.

II. OBJECTIVE / METHODOLOGY

The proposed project, consists of the construction of a 4-unit business/mercantile building (4,896 sf). The construction will include a patio seating area and reconstruction of the existing sidewalk pavers to the south and west of the proposed building. The total proposed parcel will be 6,000.67 sf, of that area 5,308 sf (88.5%) will be impervious. In the City of Ft. Pierce Florida Ordinance No. J-216 Sec. 17-28. (e)(5) when using an exfiltration trench without overflow capabilities the volume must provide the runoff difference of the Pre and Post 25-year-3-day storm event.

III. PROCEDURE

The stormwater system for the proposed construction will consist of roof gutters conveying the runoff that will tie into one Type ‘C’ inlet structure. The Type ‘C’ inlet will capture all of the runoff to the north of the building and tie into an underground exfiltration trench. The exfiltration trench has been sized to hold a volume greater than ±676 cubic feet of storage, using the Pre and Post 25-year-3-day event volume difference.

IV. CONCLUSION

As shown in the exfiltration trench calculations in Appendix C, the total storage volume meets the requirement stated in the City of Ft. Pierce Florida Ordinance No. J-216 Sec. 17-28. (e)(5). The volume in the proposed structure can hold ±697 ft^3, while the difference in volume of the Pre and Post 25-year-3-day event is ±676 ft^3. Thus, the exfiltration trench volume satisfies the required volume storage needed.

APPENDIX A

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==========================================================================================
==== Basins ==============================================================================
==========================================================================================

         Name: Site                     Node: Site Pre              Status: Onsite
        Group: 2nd Street Pre           Type: SCS Unit Hydrograph CN

        Unit Hydrograph: Uh484                   Peaking Factor: 484.0
          Rainfall File:                     Storm Duration(hrs): 0.00
    Rainfall Amount(in): 0.000                 Time of Conc(min): 5.00
               Area(ac): 0.138                   Time Shift(hrs): 0.00
           Curve Number: 85.00               Max Allowable Q(cfs): 999999.000
                DCIA(%): 0.00

Second Street - Pre

----------------------------------------------------------------------------------------------------
         Name: Site                     Node: Exfiltration          Status: Onsite
        Group: 2nd Street Post          Type: SCS Unit Hydrograph CN

        Unit Hydrograph: Uh484                   Peaking Factor: 484.0
          Rainfall File:                     Storm Duration(hrs): 0.00
    Rainfall Amount(in): 0.000                 Time of Conc(min): 5.00
               Area(ac): 0.138                   Time Shift(hrs): 0.00
           Curve Number: 96.00               Max Allowable Q(cfs): 999999.000
                DCIA(%): 0.00

Second Street - Post

==========================================================================================
==== Hydrology Simulations ===============================================================
==========================================================================================

         Name: Post25-3
     Filename: N:\JHB\P\17-86E 2nd Street Station II\Calcs\Storm-ICPR\Model\Post25-3.R32

      Override Defaults: Yes
    Storm Duration(hrs): 72.00
          Rainfall File: Sfwmd72
    Rainfall Amount(in): 9.50

Time(hrs)       Print Inc(min)
--------------- ---------------
72.000          15.00

----------------------------------------------------------------------------------------------------
         Name: Pre25-3
     Filename: N:\JHB\P\17-86E 2nd Street Station II\Calcs\Storm-ICPR\Model\Pre25-3.R32

      Override Defaults: Yes
    Storm Duration(hrs): 72.00
          Rainfall File: Sfwmd72
    Rainfall Amount(in): 9.50

Time(hrs)       Print Inc(min)
--------------- ---------------
72.000          15.00

==========================================================================================
==== Routing Simulations =================================================================
==========================================================================================

         Name: Post25-96           Hydrology Sim: Pst25-96
     Filename: N:\JHB\P\17-76E JHL Partners\Model\Post25-96.I32

      Execute: No          Restart: No            Patch: No
  Alternative: No

        Max Delta Z(ft): 1.00                     Delta Z Factor: 0.00500
    Time Step Optimizer: 10.000
        Start Time(hrs): 0.000                     End Time(hrs): 96.00
     Min Calc Time(sec): 0.5000              Max Calc Time(sec): 60.0000
        Boundary Stages:                          Boundary Flows:


Time(hrs)       Print Inc(min)
--------------- ---------------
96.000          15.000

Group           Run
--------------- -----
2nd Street Post Yes
```

```
--------------------------------------------------------------------------------------------------
         Name: Pre25-96            Hydrology Sim: Pre25-96
     Filename: N:\JHB\P\17-76E JHL Partners\Model\Pre25-96.I32

      Execute: No          Restart: No            Patch: No
  Alternative: No

        Max Delta Z(ft): 1.00                     Delta Z Factor: 0.00500
    Time Step Optimizer: 10.000
        Start Time(hrs): 0.000                     End Time(hrs): 96.00
      Min Calc Time(sec): 0.5000              Max Calc Time(sec): 60.0000
        Boundary Stages:                          Boundary Flows:


Time(hrs)       Print Inc(min)
--------------- ---------------
96.000          15.000

Group           Run
--------------- -----
2nd Street Pre  Yes
```

APPENDIX B

2nd Street
Max Volume
1/22/18

Simulation	Basin	Group	Time Max hrs	Flow Max cfs	Volume in	Volume ft3
Post25-3	Site2nd Street	Post	59.99	0.657	9.006	4511.227
Pre25-3	Site 2nd Street	Pre	59.99	0.628	7.657	3835.906

APPENDIX C

EXFILTRATION TRENCH CALCULATION

Pipe		
	pi	3.14
	Radius	1 ft
	length	19 ft
Volume (For 2 Pipes)		119.38 ft^3
Trench	width	13.5 ft
	depth	4.5 ft
	length	21 ft
Volume		578.18 ft^3
Total Volume		697.57 ft^3

Volume Comparison

25 Year 3 Day - Volume

Pre:	3835.906 ft^3
Post:	4511.227 ft^3
Difference:	675.321 ft^3

Exfiltration Trench Volume		25 Year 3 Day Volume Difference
697.57 ft^3	>	675.321 ft^3

APPENDIX D

Subsurface Soil Exploration
Historic Cobb Corner
Property Improvements
Indian River Drive
Fort Pierce, Florida

Ardaman & Associates, Inc.

OFFICES

Orlando – 8008 S. Orange Avenue, Orlando Florida 328098 – Phone (407) 855-3860

Alexandria – 3609 Mac Lee Drive, Alexandria, Louisiana 71302 – Phone (318) 443-2888

Bartow – 1525 Centennial Drive, Bartow, Florida 33830 – Phone (863) 533-0858

Baton Rouge – 316 Highlandia Drive, Baton Rouge, Louisiana 70884 – Phone (225) 752-4790

Cocoa – 1300 N. Cocoa Blvd., Cocoa, Florida 32922 – Phone (321) 632-2503

Fort Myers – 9970 Bavaria Road, Fort Myers, Florida 33913 – Phone (239) 768-6600

Miami – 2608 W. 84th Street, Hialeah, Florida 33016 – Phone (305) 825-2683

Monroe – 1122 Hayes Street, West Monroe, Louisiana 71292 – Phone (318) 387-4103

New Orleans – 1305 Distributors Row, Suite I, Jefferson, Louisiana 70123 – Phone (504) 835-2593

Port St. Lucie – 460 Concourse Place NW, Unit 1, Port St. Lucie, Florida 34986 – Phone (772) 878-0072

Sarasota – 78 Sarasota Center Blvd., Sarasota, Florida 34240 – Phone (941) 922-3526

Shreveport – 7222 Greenwood Road, Shreveport, Louisiana 71119 – Phone (318) 636-3673

Tallahassee – 3175 West Tharpe Street, Tallahassee, Florida 32303 – Phone (850) 576-6131

Tampa – 3925 Coconut Palm Drive, Suite 115, Tampa, Florida 33619 – Phone (813) 620-3389

West Palm Beach – 2200 North Florida Mango Road, Suite 101, West Palm Beach, Florida 33409 – Phone (561) 687-8200

MEMBERS:
A.S.F.E.
American Concrete Institute
ASTM International
Florida Institute of Consulting Engineers

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

September 27, 2017
File No. 17-5464

Kraaz and Kraaz Finance, Inc.
201 South 2nd Street, Unit 206
Fort Pierce, Florida 34950

Attention: Mr. Brian Stone

Subject: **Subsurface Soil Exploration**
Historic Cobb Corner Property Improvements
Indian River Drive
Fort Pierce, Florida

Mr. Stone:

As requested and authorized, we have completed a shallow subsurface soil exploration and geotechnical engineering evaluation for the subject project. The purposes of performing this exploration were to evaluate the general subsurface conditions within the proposed two-story or three-story building and associated parking/driving areas, and to provide recommendations for site preparation and foundation support. This report documents our findings and presents our engineering recommendations.

SITE LOCATION AND SITE DESCRIPTION

The Historic Cobb Corner Property is located at the northwest corner of the intersection on Indian River Drive and Avenue A in Fort Pierce, St. Lucie County, Florida (Section 10, Township 35 South, Range 40 East). The general site location is shown superimposed on the Fort Pierce, Florida USGS quadrangle map presented on Figure 1. The subject area for the proposed building is currently undeveloped, sparsely wooded and unimproved parking areas.

PROPOSED CONSTRUCTION AND GRADING

Based on the information provided, it is our understanding that the project entails the design and construction of a two-story or three-story building and associated parking/dive areas to the north of the historical P.P. Cobb Building. For the purpose of our analysis, we assumed that the proposed building will consist of load bearing walls and interior columns with a slab-on-grade floor. Typical anticipated loading conditions for the two-story or three-story structure were not provided, but have been assumed to be on the order of 3 to 5 kips per linear foot for wall foundations and 50 to 70 kips for individual column foundations. We have assumed less than 4 feet of fill is anticipated to bring the building and parking areas to final elevation. If actual building loads or fill heights exceed our assumptions, then the recommendations in this report may not be valid.

REVIEW OF SOIL SURVEY MAPS

The Soil Survey of St. Lucie County, Florida, which was issued by the U.S. Department of Agriculture, Soil Conservation Service in 1980, states that the predominant surficial soil type in the area where the site is located is *Urban land*. A brief description of this soil type, as taken from the Soil Survey, is presented below.

According to the USDA Soil Survey, *Urban land* consists of areas that are more than 70 percent covered by airports, shopping centers, parking lots, large buildings, streets, and sidewalks. Other areas, for example, lawns, parks, vacant lots, and playgrounds are made up mostly of Ankona, Lawnwood, Nettles, Pendarvis, Pepper, Tantile, St. Lucie, Paola, and Waveland soils. The surface of these soils, to a depth of about 12 inches, has been covered with fill material consisting of sandy and loamy materials which contain limestone and shell fragments in places. These areas of soils are too small to be mapped separately.

FIELD EXPLORATION PROGRAM

The field exploration program included performing two (2) Standard Penetration Test (SPT) borings (B-1 and B-2 on Figure 2) in the vicinity of the proposed building area and two (2) auger borings (AB-1 and AB-2) within or in the vicinity of the surrounding parking/drive areas. The SPT borings were advanced to an approximate depth of 20 feet below the existing ground surface using the general methodology outlined in ASTM D-1586. The auger borings were conducted using a 4-inch diameter continuous flight auger to the depth of 10 feet below the existing ground surface. Descriptions of these field procedures are included in the Appendix.

Soil samples were recovered from the sampler or auger during performance of the borings. The samples were visually classified in the field and representative portions of the samples were transported to our laboratory in sealed sample jars. The groundwater level at each of the boring locations was measured during drilling. Upon completion, the borings were backfilled with soil cuttings. The approximate locations of the borings are shown on Figure 2. These locations were determined in the field by estimating distances from existing site features and should be considered accurate only to the degree implied by the method of measurement used.

In order to estimate the hydraulic conductivity of the shallow soils, two constant-head exfiltration tests (Exfil-1 and Exfil-2) were performed in the vicinity of the proposed stormwater treatment areas as shown on Figure 2. These tests were performed in general accordance with the methods described in the South Florida Water Management District (SFWMD) Permit Information Manual, Volume IV. In brief, the hydraulic conductivities obtained from exfiltration tests Exfil-1 and Exfil-2 are shown in the table below.

Test Number	Boring location	Hydraulic Conductivity ($cfs/ft^2 – ft$)
Exfil-1	Exfil-1	5.49×10^{-4}
Exfil-2	Exfil-2	5.92×10^{-4}

It is noted that a suitable factor of safety should be used with this value. In addition, for the type of soils tested, a transformation ratio of 1 horizontal to 1 vertical is appropriate (i.e; the estimated ratio of horizontal to vertical permeability).

LABORATORY PROGRAM

Representative soil samples obtained during our field sampling operation were packaged and transferred to our laboratory for further visual examination and classification. The soil samples were visually classified in general accordance with the Unified Soil Classification System (ASTM D-2488). The resulting soil descriptions are shown on the soil boring profiles presented on Figures 3 and 4.

GENERAL SUBSURFACE CONDITIONS

General Soil Profile

The results of the field exploration and laboratory programs are graphically summarized on the soil boring profiles presented on Figures 3 and 4. The stratification of the boring profiles represents our interpretation of the field boring logs and the results of laboratory examinations of the recovered samples. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

As shown on the soil boring profiles on Figures 3, the SPT borings typically encountered loose to dense, light brown to brown and light gray to gray fine sand (Unified Soil Classification SP), fine sand with silt (SP-SM), and fine sand with clay (SP-SC) to the boring termination depths of 20 feet. As shown on Figure 4, the auger borings encountered similar soils to a depth of 10 feet below the existing ground surface. These soil profiles are outlined in general terms only. Please refer to Figures 3 and 4 for soil profile details.

Groundwater Level

The groundwater level was measured in the boreholes on the day drilled. As shown on Figures 3 and 4, groundwater was encountered in the borings at approximate depths ranging from 7 to 9 feet below the existing ground surface on the date indicated. Fluctuations in groundwater levels should be anticipated throughout the year primarily due to seasonal variations in rainfall and other factors that may vary from the time the borings were conducted.

NORMAL SEASONAL HIGH GROUNDWATER LEVEL

The normal seasonal high groundwater level each year is the level in the August-September period at the end of the rainy season during a year of normal (average) rainfall. The water table elevations associated with a higher than normal rainfall and in the extreme case, flood, would be higher to much higher than the normal seasonal high groundwater level. The normal high water levels would more approximate the normal seasonal high groundwater levels.

The seasonal high groundwater level is affected by a number of factors. The drainage characteristics of the soils, the land surface elevation, relief points such as drainage ditches, lakes, rivers, swamp areas, etc., and distance to relief points are some of the more important factors influencing the seasonal high groundwater level.

Based on our interpretation of the site conditions using the soil borings and the Soil Survey, we preliminarily estimate the normal seasonal high groundwater level at the boring locations to be approximately 1 to 2 feet above the groundwater level measured in the boreholes at the time of our field exploration.

ENGINEERING EVALUATION AND RECOMMENDATIONS

General

The results of our exploration and analysis indicate that, with proper site preparation as recommended in this report, the existing soils are suitable for supporting the proposed two-story or three-story building on a conventional shallow foundation system. Spread footings should provide an adequate support system for the structure. The soil borings typically encountered suitable sandy soils at the locations and depths explored.

The following are our recommendations for overall site preparation and foundation support which we feel are best suited for the proposed buildings and existing soil conditions. The recommendations are made as a guide for the design engineer and/or architect, parts of which should be incorporated into the project's specifications.

Stripping and Grubbing

The "footprint" of the proposed building area, plus a minimum margin of five feet, should be stripped of all surface vegetation, stumps, debris, concrete, asphalt, organic topsoil or other deleterious materials, as encountered. Buried utilities should be removed or plugged to eliminate conduits into which surrounding soils could erode.

After stripping, the site should be grubbed or root-raked such that roots with a diameter greater than ½ inch, stumps, or small roots in a dense state, are completely removed. The actual depth(s) of stripping and grubbing must be determined by visual observation and judgment during the earthwork operation.

Special care should be taken to ascertain that all existing foundations, slabs, and any other underground structures are removed from the proposed construction area. If pipes or any collapsible or leak prone utilities are not removed or completely filled (with grout or concrete), they may serve as conduits for subsurface erosion resulting in excessive settlements. Over-excavated areas resulting from the removal of underground structures and unsuitable materials should be backfilled in accordance with the fill soils section of this report. This excavation must not undermine the existing facilities and/or building foundations. Provide shoring, bracing, and/or underpinning of existing footings as necessary to protect from failure.

It has been our experience that soils surrounding existing buildings sometimes contain pockets of construction debris or other deleterious materials requiring removal and replacement with compacted clean fine sands. Therefore, we strongly recommend that the stripped surface be inspected and approved by Ardaman & Associates, Inc. prior to filling the site.

Proof-rolling

We recommend proof-rolling the cleared surface to locate any unforeseen soft areas or unsuitable surface or near-surface soils, to increase the density of the upper soils, and to prepare the existing surface for the addition of the fill soils (as required). Proof-rolling of the building areas should consist of at least 10 passes of a compactor capable of achieving the density requirements described in the next paragraph. Each pass should overlap the preceding pass by 30 percent to achieve complete coverage. If deemed necessary, in areas that continue to "yield", remove all deleterious material and replace with clean, compacted sand backfill. The proof-rolling should occur after cutting and before filling. The number of passes can be reduced to three within the parking/drive areas.

A density equivalent to or greater than 95 percent of the modified Proctor (ASTM D-1557) maximum dry density value for a depth of 1 foot must be achieved beneath the stripped and grubbed ground surface. Additional passes and/or over-excavation and recompaction may be required if these minimum density requirements are not achieved. The soil moisture should be adjusted as necessary during compaction.

We recommend that the existing surface be level and firm prior to the addition of fill soils. Proof-rolling with a front-end loader may help achieve the desired surface and compaction condition before adding the fill soils. The site should be dewatered as necessary.

Care should be exercised to avoid damaging any neighboring structures while the compaction operation is underway. Prior to commencing compaction, occupants of adjacent structures should be notified and the existing condition (i.e. cracks) of the structures documented with photographs and survey (if deemed necessary). Compaction should cease if deemed detrimental to adjacent structures, and Ardaman & Associates should be notified immediately.

Vibrations

We recommend that the site preparation contractor closely monitor the vibrations produced during the compaction operations so that they do not adversely affect the adjacent structures. A seismograph with a suitable indicator range should be arranged on the site during the compaction operations to verify that ground vibrations are maintained within acceptable levels, and special attention given to limiting ground vibrations with acceptable levels adjacent to nearby existing structures. Additionally, the contractor and design engineer should determine if existing structures will be especially susceptible to vibrations caused during this phase of the project. A pre-condition survey should also be performed prior to construction to document pre-existing cracks or other signs of existing damage to the structures. Ardaman and Associates would be pleased to provide these services, if requested.

Suitable Fill Material and the Compaction of Fill Soils

All fill soil should be free of organic materials, such as roots and vegetation. We recommend using fill with less than 12 percent by dry weight of material passing the U.S. Standard No. 200 sieve size. Soils with more than 12 percent passing the No. 200 sieve can be used in some applications, but will be more difficult to compact due to their inherent nature to retain soil moisture.

All structural fill should be placed in level lifts not to exceed 12 inches in uncompacted thickness. Each lift should be compacted to at least 95 percent of the modified Proctor (ASTM D-1557) maximum dry density value. The filling and compaction operations should continue in lifts until the desired elevation(s) is achieved. If hand-held compaction equipment is used, the lift thickness should be reduced to no more than 6 inches.

Foundation Support by Spread Footings and Foundation Compaction Criteria

Excavate the foundations to the proposed bottom of footing elevations and, thereafter, verify the in-place compaction for a depth of 1 foot below the footing bottoms. If necessary, compact the soils at the bottom of the excavations to at least 95 percent of the modified Proctor maximum dry density (ASTM D-1557) for a depth of 1 foot below the footing bottoms. Based on the existing soil conditions and, assuming the above outlined proof-rolling and compaction criteria are implemented, an allowable soil bearing pressure of 2,500 pounds per square foot (psf) may be used in the foundation design. This bearing pressure should result in foundation settlement within tolerable limits (i.e., 1 inch or less).

All bearing wall foundations should be a minimum of 18 inches wide and column foundations 24 inches wide. A minimum soil cover of 24 inches should be maintained from the bottom of the foundations to the adjacent finished grades.

Floor Slab Moisture Reducer and Slab Compaction Requirements

Compaction beneath all floor slabs should be verified for a depth of 12 inches and meet the 95 percent criteria (modified Proctor, ASTM D-1557).

Precautions should be taken during the slab construction to reduce moisture entry from the underlying subgrade soils. Moisture entry can be reduced by installing a membrane between the subgrade soils and floor slab. Care should be exercised when placing the reinforcing steel (or mesh) and slab concrete such that the membrane is not punctured. We note that the membrane alone does not prevent moisture from occurring beneath or on top of the slab.

If interior columns are isolated from the floor slab, an expansion joint should be provided around the columns and sealed with a water-proof sealant. We note that the site has a relatively high groundwater table level which needs to be considered if any recessed slabs or sump pits (i.e., loading docks, etc.) are being planned for the site.

Typical Asphaltic Concrete Surface Pavement Section

Site Preparation

All areas to be paved should be prepared as previously outlined. Prior to stabilized subgrade and pavement base installation, the subgrade soil compaction should be verified for a depth of 2 feet (i.e., compacted to at least 95 percent of the modified Proctor (ASTM D-1557, AASHTO T-180) maximum dry density value).

Limerock/Coquina Base

An 8-inch thick limerock or coquina base course having a minimum Limerock Bearing Ratio (LBR) value of 100, overlying a 12-inch thick stabilized subgrade can be used provided that grading and drainage plans preclude periodic saturation of the base material. The periodic saturation of a limerock/coquina base material could lead to premature pavement distress. A minimum clearance of 18 inches must be maintained between the bottom of the limerock/coquina base and the seasonal high groundwater table.

The limerock or coquina should be compacted to at least 98 percent of the modified Proctor (ASTM D-1557, AASHTO T-180) maximum density value. For bus and/or truck parking and drive areas, the base thickness should be increased to a minimum of 10 inches.

A minimum 12-inch thick stabilized subgrade having a minimum Limerock Bearing Ratio (LBR) value of 40 must be achieved beneath the base. The natural soils will have to be stabilized with suitable clayey soil or another approved stabilization material in order to achieve the required LBR value. The stabilized subgrade must be compacted to at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557, AASHTO T-180). The stabilized subgrade must be firm and unyielding immediately prior to placement of the base material.

Wearing Surface

A minimum 1½ inch layer of Type SP-9.5 or SP-12.5 asphaltic concrete should be used for a wearing surface in automobile parking/drive areas. For bus and/or truck parking/drive areas, at least 2½ inches of Type SP-9.5 or SP-12.5 asphaltic concrete should be used. The asphalt wearing surface must be placed on an adequately compacted and unyielding base course. Specific requirements for the Type-SP asphaltic concrete wearing surface are outlined in Section 334 in the Florida Department of Transportation, Standard Specifications for Road and Bridge Construction, Latest Edition.

The latest specifications of Florida Department of Transportation shall govern the design and placement of the base and asphaltic concrete wearing surface. The above minimum requirements will satisfactorily support Traffic Level A*. If a heavier traffic pattern is anticipated, the design section should be increased accordingly.

Retention Ponds

We understand that dry bottom retention ponds or exfiltration trenches are planned for the site. For this study, soil conditions were explored in the proposed stormwater treatment areas with two auger boring to a depth of 5 feet in conjunction with field exfiltration tests (Exfil – 1 and Exfil – 2). The fine sand, fine sand with silt and fine sand with clay soils (Strata Nos. 1, 2 and 3 on Figure 3) encountered in the borings are considered to be relatively permeable.

For dry bottom retention ponds, pond performance will be significantly influenced by the soil permeability and the vertical separation between the pond bottom and the seasonal high groundwater level. Ardaman & Associates, Inc. would be pleased to assist in evaluating the design exfiltration rates, underdrains and/or groundwater baseflow as pond geometry and stormwater volume requirements become available.

QUALITY ASSURANCE

We recommend establishing a comprehensive quality assurance program to verify that all site preparation and foundation and pavement construction is conducted in accordance with the appropriate plans and specifications. Materials testing and inspection services should be provided by Ardaman & Associates, Inc.

As a minimum, an on-site engineering technician should monitor all stripping and grubbing to verify that all deleterious materials have been removed and should observe the proof-rolling operation to verify that the appropriate numbers of passes are applied to the subgrade. In-situ density tests should be conducted during filling activities and below all footings, floor slabs and pavement areas to verify that the required densities have been achieved. In-situ density values should be compared to laboratory Proctor moisture-density results for each of the different natural and fill soils encountered.

Additionally for the pavements, Limerock Bearing Ratio tests should be performed. The base course(s) should be tested for density and thickness. Samples of the asphaltic concrete should

* Reference: "Flexible Pavement Design Manual", Florida Department of Transportation. (Latest Edition)

be obtained and tested in the laboratory for Marshall stability, flow, asphalt content, and aggregate gradation. Also, the asphaltic concrete thickness should be verified in the field.

Finally, we recommend inspecting and testing the construction materials for the foundations and other structural components.

IN-PLACE DENSITY TESTING FREQUENCY

In Southeast Florida, earthwork testing is typically performed on an on-call basis when the contractor has completed a portion of the work. The test result from a specific location is only representative of a larger area if the contractor has used consistent means and methods and the soils are practically uniform throughout. The frequency of testing can be increased and full-time construction inspection can be provided to account for variations. We recommend that the following minimum testing frequencies be utilized.

In proposed structural areas, the minimum frequency of in-place density testing should be one test for each 2,500 square feet of structural area (minimum of five test locations per building). In-place density testing should be performed at this minimum frequency for a depth of 1 foot below natural ground and for every 1-foot lift of fill placed in the structural areas. In addition, density tests should be performed in each column footing for a depth of 1 foot below the bearing surface. For continuous or wall footings, density tests should be performed at a minimum frequency of one test for every 50 lineal feet of footing, and for a depth of 1 foot below the bearing surface.

In proposed parking areas, a minimum frequency of one In-place density test for each 5,000 square feet of area should be used. The existing, natural ground should be tested to a depth of 12 inches at the prescribed frequency. Each 12-inch lift of fill, as well as the stabilized subgrade (where applicable) and base should be tested at this frequency. Utility backfill should be tested at a minimum frequency of one In-place density test for each 12-inch lift for each 200 lineal feet of pipe. Additional tests should be performed in backfill for manholes, inlets, etc.

Representative samples of the various natural ground and fill soils should be obtained and transported to our laboratory for Proctor compaction tests. These tests will determine the maximum dry density and optimum moisture content for the materials tested and will be used in conjunction with the results of the in-place density tests to determine the degree of compaction achieved.

CLOSURE

The analyses and recommendations submitted herein are based on the data obtained from the soil borings presented on Figures 3 and 4, and on the assumed loading conditions. This report does not reflect any variations which may occur adjacent to or between the borings. The nature and extent of the variations between the borings may not become evident until during construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented in this report after performing on-site observations during the construction period and noting the characteristics of the variations. This study does not include an evaluation of the environmental (ecological or hazardous/toxic material related) condition of the site and subsurface.

This report has been prepared for the exclusive use of Kraaz and Kraaz Finance, Inc. in accordance with generally accepted geotechnical engineering practices. In the event any changes occur in the design, nature, or location of the proposed facility, we should review the applicability of conclusions and recommendations in this report. We recommend a general review of final design and specifications by our office to verify that earthwork and foundation recommendations are properly interpreted and implemented in the design specifications. Ardaman and Associates should attend the pre-bid and preconstruction meetings to verify that the bidders/contractor understand the recommendations contained in this report.

We are pleased to be of assistance to you on this phase of the project. When we may be of further service to you or should you have any questions, please contact us.

Best regards,

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

9/27/17

Dan J. Zrallack, P.E.
Branch Manager
Florida License No. 63911

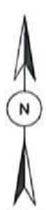
9/27/17

Sharmila Pant
Assistant Project Engineer

DANIEL J. ZRALLACK
LICENSE
No. 63911
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

SECTION 10
TOWNSHIP 35 SOUTH
RANGE 40 EAST

OBTAINED FROM U.S.G.S. QUAD MAPS: FORT PIERCE, FLORIDA 1949
(PHOTOREVISED 1970)



NOT TO SCALE

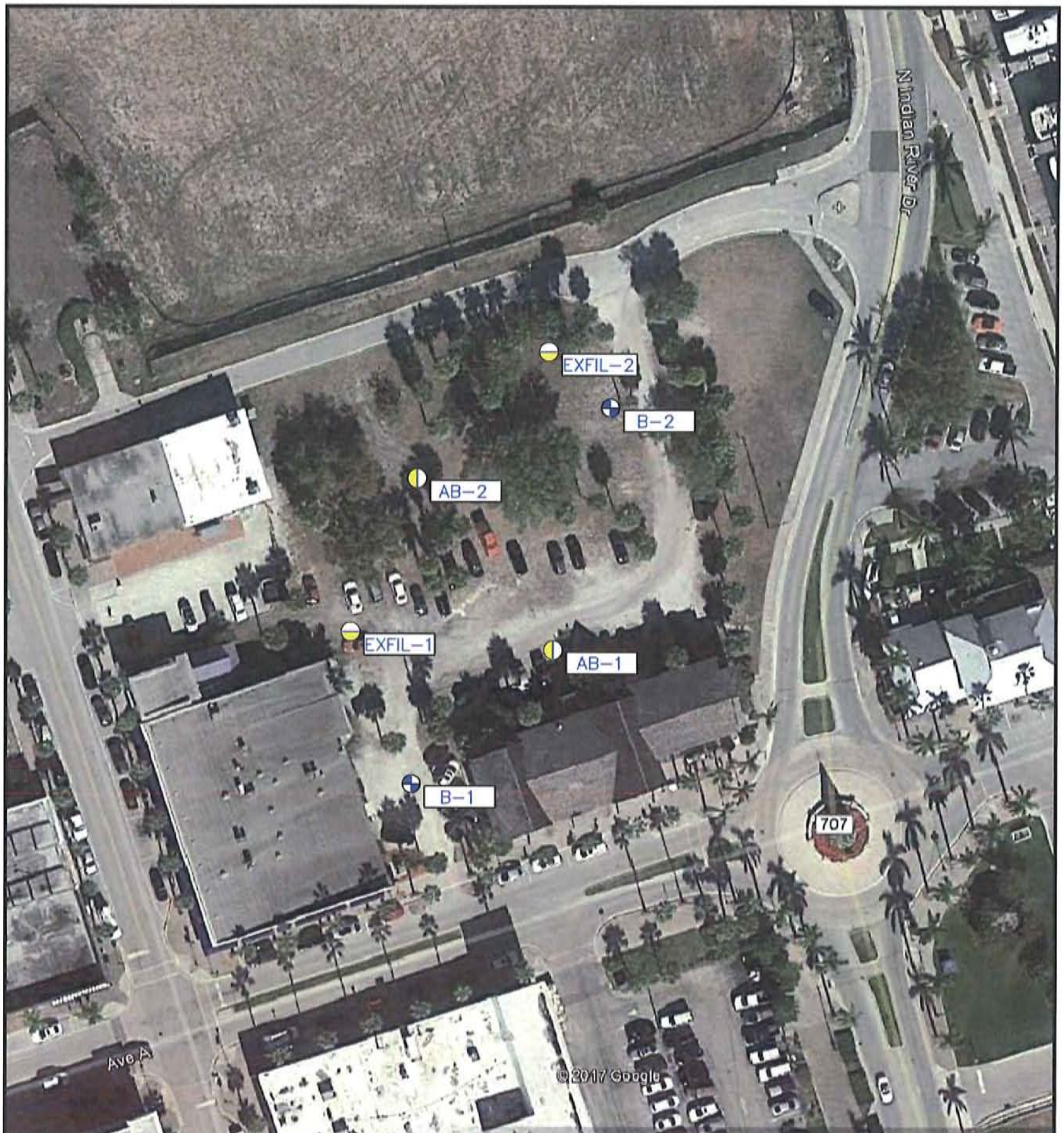
QUADRANGLE LOCATION

SITE LOCATION MAP

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

Subsurface Soil Exploration
Historic Cobb Corner Property Improvements
Indian River Drive
Fort Pierce, Florida

DRAWN BY: SP	CHECKED BY:	DATE: 09/25/17
FILE NO. 17-5464	APPROVED BY:	FIGURE: 1



N

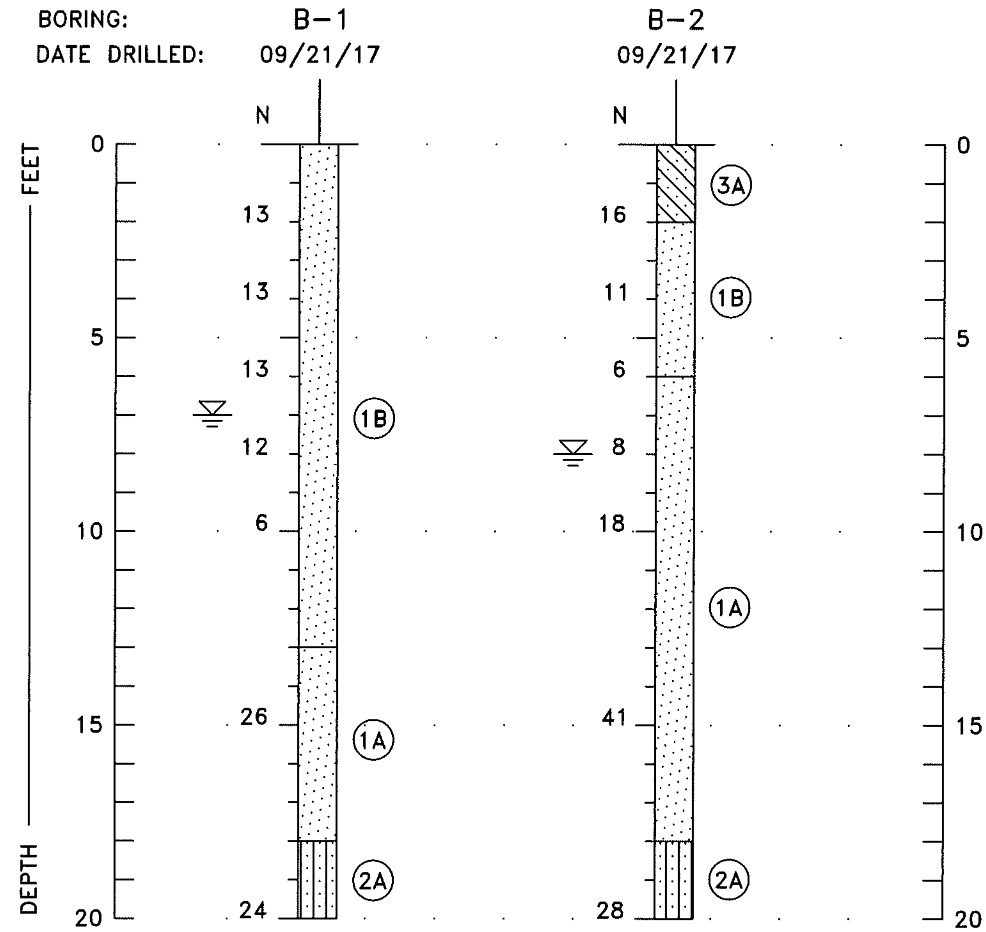
NOT TO SCALE

AB AUGER BORING LOCATION

B STANDARD PENETRATION TEST (SPT) BORING LOCATION

EXFIL EXFILTRATION TEST LOCATION

SOIL BORING LOCATIONS		
Ardaman & Associates, Inc. Geotechnical, Environmental and Materials Consultants		
Subsurface Soil Exploration Historic Cobb Corner Property Improvements Indian River Drive Fort Pierce, Florida		
DRAWN BY: SP	CHECKED BY:	DATE: 09/25/17
FILE NO. 17-5464	APPROVED BY:	FIGURE: 2



LEGEND

SOIL DESCRIPTIONS

- (1) FINE SAND (SP)
- (2) FINE SAND WITH SILT (SP-SM)
- (3) FINE SAND WITH CLAY (SP-SC)

COLORS

- (A) LIGHT BROWN TO BROWN
- (B) LIGHT GRAY TO GRAY

ENGINEERING CLASSIFICATION

I COHESIONLESS SOILS

DESCRIPTION	BLOW COUNT "N"
VERY LOOSE	<4
LOOSE	4 TO 10
MEDIUM DENSE	10 TO 30
DENSE	30 TO 50
VERY DENSE	>50

N STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

▽ GROUNDWATER LEVEL MEASURED ON DATE DRILLED

SP,SP-SM SM,SC,CH UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

NOTE: ALL SPT BORINGS WERE PERFORMED USING A HAND AUGER IN THE UPPER 4 FEET AND AN AUTOMATIC HAMMER BELOW 4 FEET TO THE BORING TERMINATION DEPTH. AUTOMATIC HAMMER N-VALUES MAY BE CONVERTED TO EQUIVALENT SAFETY HAMMER N-VALUES BY MULTIPLYING BY 1.24.

WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS AND FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL VARIATIONS CHARACTERISTIC OF THE SUBSURFACE MATERIALS OF THE REGION ARE ANTICIPATED AND MAY BE ENCOUNTERED. THE BORING LOGS AND RELATED INFORMATION ARE BASED ON THE DRILLER'S LOGS AND VISUAL EXAMINATION OF SELECTED SAMPLES IN THE LABORATORY. THE DELINEATION BETWEEN SOIL TYPES SHOWN ON THE LOGS IS APPROXIMATE AND THE DESCRIPTION REPRESENTS OUR INTERPRETATION OF SUBSURFACE CONDITIONS AT THE DESIGNATED BORING LOCATIONS ON THE PARTICULAR DATE DRILLED.

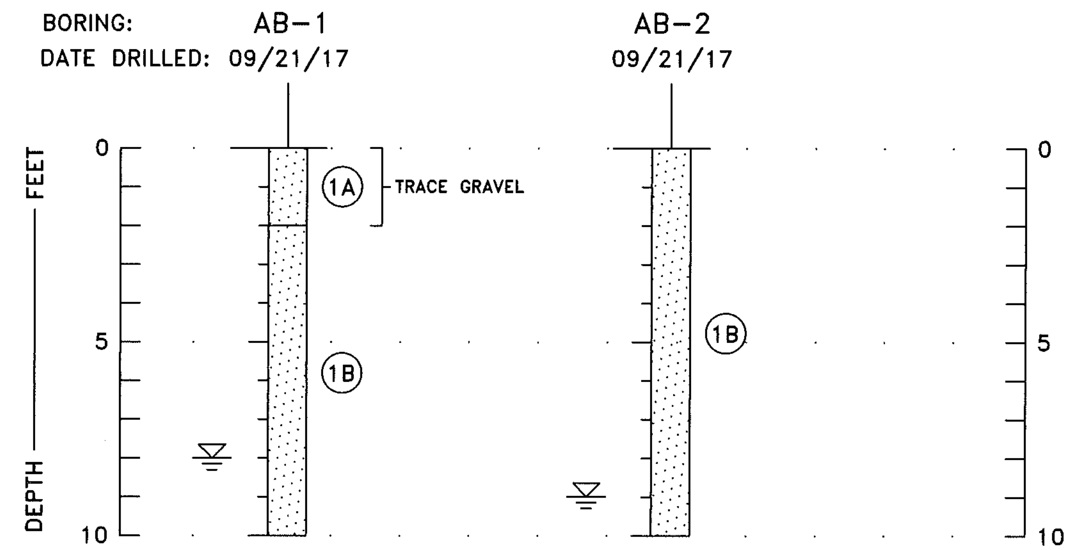
GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR.

SOIL BORING PROFILES

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

Subsurface Soil Exploration
Historic Cobb Corner Property Improvements
Indian River Drive
Fort Pierce, Florida

DRAWN BY: SP	CHECKED BY:	DATE: 09/25/17
FILE NO. 17-5464	APPROVED BY:	FIGURE: 3



LEGEND

SOIL DESCRIPTIONS

- ① FINE SAND (SP)
- ② FINE SAND WITH SILT (SP-SM)
- ③ FINE SAND WITH CLAY (SP-SC)

COLORS

- Ⓐ LIGHT BROWN TO BROWN
- Ⓑ LIGHT GRAY TO GRAY

GROUNDWATER LEVEL MEASURED ON DATE DRILLED

SP,SP-SM
SM,SC,CH — UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

NOTE:

WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS AND FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL VARIATIONS CHARACTERISTIC OF THE SUBSURFACE MATERIALS OF THE REGION ARE ANTICIPATED AND MAY BE ENCOUNTERED. THE BORING LOGS AND RELATED INFORMATION ARE BASED ON THE DRILLER'S LOGS AND VISUAL EXAMINATION OF SELECTED SAMPLES IN THE LABORATORY. THE DELINEATION BETWEEN SOIL TYPES SHOWN ON THE LOGS IS APPROXIMATE AND THE DESCRIPTION REPRESENTS OUR INTERPRETATION OF SUBSURFACE CONDITIONS AT THE DESIGNATED BORING LOCATIONS ON THE PARTICULAR DATE DRILLED.

GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR.

SOIL BORING PROFILES

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Historic Cobb Corner Property Improvements
Indian River Drive
Fort Pierce, Florida

DRAWN BY: SP	CHECKED BY:	DATE: 09/25/17
FILE NO. 17-5464	APPROVED BY:	FIGURE: 4

Ardaman & Associates, Inc.

HYDRAULIC CONDUCTIVITY TEST LOG
SFWMD USUAL OPEN-HOLE TEST
Exfil-1

PROJECT: Historic Cobb Corner Property Improvements

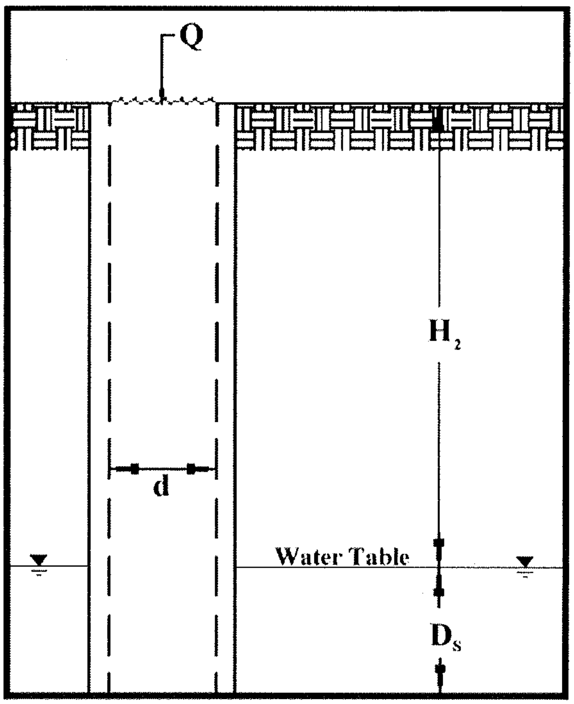
FILE No.: 17-5464

TEST LOCATION: Exfil-1

DRILL CREW: AR

GROUNDWATER OBSERVED AT DEPTH 7 ft.

TEST DATE: 08/29/2017



$$K = \frac{4Q}{\pi d(2H_2^2 + 4H_2 D_s + H_2 d)}$$

Q ["Stabilized" Flow Rate (cfs)] = 9.80×10^{-3}

K [Hydraulic Conductivity (cfs/sqf - ft head)] = 5.49×10^{-4}

d [Diameter of Test Hole (ft)] = 0.5

H_2 [Depth to Water Table (ft)] = 7

* D_s [Saturated Hole Depth (ft)] = -2

* By Groundwater

DEPTH	SYMBOLS	SOIL DESCRIPTION	SAMPLE No.
0		Brown fine sand (SP)	1
1		Light gray fine sand (SP)	2
2			
3			
4			
5		End of Boring	
6			

NOTES:

Ardaman & Associates, Inc.

HYDRAULIC CONDUCTIVITY TEST LOG
SFWMD USUAL OPEN-HOLE TEST
Exfil-2

PROJECT: Historic Cobb Corner Property Improvements

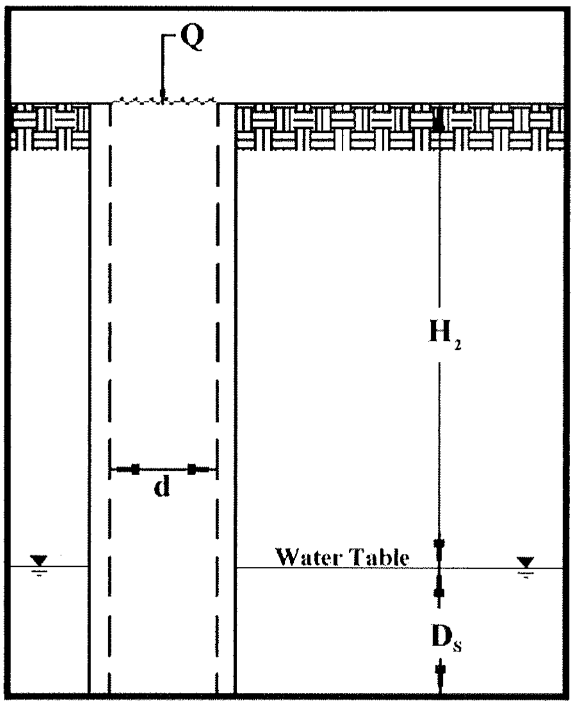
FILE No.: 17-5464

TEST LOCATION: Exfil-2

DRILL CREW: AR

GROUNDWATER OBSERVED AT DEPTH 7 ft.

TEST DATE: 08/29/2017



$$K = \frac{4Q}{\pi d(2H_2^2 + 4H_2 D_s + H_2 d)}$$

Q ["Stabilized" Flow Rate (cfs)] = 1.06×10^{-2}

K [Hydraulic Conductivity (cfs/sqf - ft head)] = 5.92 x 10^{-4}

d [Diameter of Test Hole (ft)] = 0.5

H_2 [Depth to Water Table (ft)] = 7

* D_s [Saturated Hole Depth (ft)] = -2

* By Groundwater

DEPTH	SYMBOLS	SOIL DESCRIPTION	SAMPLE No.
0		Brown fine sand (SP)	1
1.5		Light gray fine sand (SP)	2
5		End of boring	

NOTES:

APPENDIX

Drilling Procedures

STANDARD PENETRATION TEST

The standard penetration test is a widely accepted test 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 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 the soils at every 5 feet of drilled depth and from every different stratum are brought to our laboratory in air-tight jars for further evaluation and testing, if necessary. Samples not used in testing are stored for 30 days prior to being discarded. 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.