Subsurface Soil Exploration and
Geotechnical Engineering Evaluation
38-Acre Spaceport Commerce Park
Armstrong Drive
Titusville, Florida

Ardaman & Associates, Inc.

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Materials Consultants

June 16, 2015
File No. 15-23-5239

North Brevard Economic Development Zone
400 South Street, Suite A-1
Titusville, FL 32780

Attention: Mr. Troy Post, CEcD

Subject: Subsurface Soil Exploration and
Geotechnical Engineering Evaluation
38-Acre Spaceport Commerce Park
Armstrong Drive
Titusville, Florida

Dear Mr. Post:

As requested and authorized by you, we have completed a shallow subsurface soil exploration for the subject project. The purposes of performing this exploration were to evaluate the general subsurface conditions within the building, parking/drive, and retention pond areas and to provide recommendations for site preparation, foundation support, and pavement design. In addition, we have estimated the normal seasonal high groundwater level at the boring locations. This report documents our findings and presents our engineering recommendations.

SITE LOCATION AND SITE DESCRIPTION

The site for the proposed Space Commerce Park improvements is located just west of Armstrong Drive in Titusville, Florida (Section 4, Township 23 South, Range 35 East). The general site location is shown superimposed on the Titusville, Florida USGS quadrangle map presented on Figure 1.

The site is currently undeveloped and densely wooded and vegetated with thick brush. No structures are located on the site. Paths were mowed on the site to provide access for our ATV-mounted drilling equipment and our personnel. We note that significant topography changes were noted on the site during our field exploration. Based on review of a topographical survey provided by Honeycutt & Associates, Inc., the ground surface elevations range from +35 feet on the southwest portion of the site to +15 feet on the northeast portion.

REVIEW OF SOIL SURVEY MAPS

Based on the 1974 Soil Survey for Brevard County, Florida, as prepared by the U.S. Department of Agriculture Soil Conservation Service, the project site is located in an area mapped as the "Immokalee sand" and “Pomello sand” soil series. A description of these soil types, as obtained from the Soil Survey, is provided below.

Immokalee sand (Im):

"Immokalee sand" is a nearly level, poorly drained sandy soil in broad areas in the flatwoods, on low ridges between sloughs, and in low narrow areas between sand ridges and lakes and ponds. In a representative profile the surface layer, about 11 inches thick, is sand that is dark gray to very dark gray. It is underlain by a layer of gray to light gray sand 22 inches thick. The subsoil extends to a depth of 65 inches and is black to dark-reddish brown sand that includes some weakly cemented sand. Below this is yellowish-brown sand that extends to a depth of 80 inches. In most years, the water table is within a depth of 10 inches for 1 to 2 months. It is between 10 and 40 inches more than half the time, and during short dry periods, it is below 40 inches. The soil is flooded for 2 to 7 days once in 1 to 5 years.

Pomello sand (Ps):

"Pomello sand" is a nearly level, moderately well drained sandy soil on broad low ridges and low knolls. In a representative soil profile the surface layer is about 3 inches thick. Below this is light gray sand to a depth of 50 inches. Various weakly cemented sands are typically present from 50 to 70 inches, underlain by brown sand below 70 inches. The water table is 30 to 40 inches below the surface for 2 to 4 months in most years and between 40 and 60 inches for more than 6 months. During dry periods, it is below 60 inches for short periods.

PROPOSED CONSTRUCTION AND GRADING

It is our understanding that the proposed development includes a one-story, 130,000-square foot building; asphalt paved parking and drive areas; and several dry stormwater ponds. The proposed building will consist of load bearing masonry walls and interior columns with slab-on-grade floors. For the purposes of our analysis, we have assumed the maximum loading conditions for the structure to be on the order of 4 to 5 kips per linear foot for wall foundations, 100 to 150 kips for individual column foundations, and 100 pounds per square foot (psf) for slab-on-grade floors.

Grading plans are not complete at this time, therefore we have assumed that 1 to 2 feet of fill is required to raise the building and parking/drive areas to final elevations. If actual building loads or fill height exceed our assumptions, then the recommendations in this report may not be valid.

FIELD EXPLORATION PROGRAM

SPT and Auger Borings

The field exploration program included performing 11 Standard Penetration Test (SPT) borings and 15 auger borings. The SPT borings were performed within the proposed "footprint" of the building. The borings were advanced to depths of 20 and 25 feet below the ground surface using the methodology outlined in ASTM D-1586. A summary of this field procedure is included in Appendix I. Split-spoon soil samples recovered during performance of the borings were visually classified in the field and representative portions of the samples were transported to our laboratory in sealed sample jars.

The auger borings were performed in the proposed parking/drive and stormwater pond areas. They were drilled using a truck-mounted, 4-inch diameter, continuous flight auger or a 3-inch diameter, hand-held bucket auger to depths ranging from 5 to 15 feet below the ground surface. A summary of this field procedure is included in Appendix I. Representative soil samples were recovered from the auger borings and transported to our laboratory for further analysis.

The groundwater level at each of the boring locations was measured during drilling. Upon completion, the borings were backfilled with soil cuttings.

Permeability Test Sampling

Pits were excavated by hand to depths ranging from 1 to 3 feet below existing ground surface adjacent to Borings AB-11, AB-13, and AB-14. One 3-inch diameter Shelby-tube sample of soil was obtained in a vertical orientation in the bottom of each excavation. The Shelby-tube samples were capped and transported to our laboratory for soil permeability testing.

Test Locations

The approximate locations of the borings and test pits are schematically illustrated on a site plan shown on Figure 2. These locations were determined in the field by wheel measuring/estimating distances from existing site features and should be considered accurate only to the degree implied by the method of measurement used.

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 in Appendix II

Constant head permeability tests were performed on the Shelby-tube samples obtained from the test pits near the location of Borings AB-11, AB-13, and AB-14. The results of the laboratory permeability tests are tabulated in the “Soil Permeability” section of this report.

GENERAL SUBSURFACE CONDITIONS

General Soil Profile

The results of the field exploration and laboratory programs are graphically summarized on the soil boring profiles presented in Appendix II. 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.

The results of the SPT borings indicate the following general soil profile in the proposed building area:

Depth Below Ground Surface (feet)	Description (Unified Soil Classification)
0 to 5	Very loose to loose fine sand (SP)
5 to 25	Very loose to medium dense fine sand (SP), fine sand with silt (SP-SM), and silty fine sand (SM) with varying amounts of shell below 7 feet.

Similar soils were encountered in the auger borings, with the exception of Borings AB-11, AB-12, and AB-13 where approximately 1.5 to 2 feet of clayey fine sand (SC) was encountered at the ground surface. The above soil profiles are outlined in general terms only. Please refer to Appendix II for soil profile details.

Groundwater Level

The groundwater level was measured in the boreholes on the day drilled. As shown in Appendix II, groundwater was encountered at depths that ranged from 3.8 to 18 feet below the existing ground surface on the dates 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.

The absence of groundwater data at some of the boring locations indicates that groundwater was not encountered within the vertical reach of the borings on the date drilled. However, this does not necessarily mean that groundwater would not be encountered at some other time.

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.

In addition to evaluating the conditions above, we have reviewed annual precipitation data available from the Melbourne Office of the National Weather Service. Based on this data, the annual rainfall to date in Brevard County is approximately 15 inches, which is approximately 0.8 inches above normal for this time of year.

Based on our interpretation of the site conditions using our boring logs, we estimate the normal seasonal high groundwater level at the boring locations to be approximately 3 feet above the groundwater levels measured at the time of our field exploration.

ENGINEERING EVALUATION AND RECOMMENDATIONS

General

The results of our exploration indicate that, with proper site preparation as recommended in this report, the existing soils are suitable for supporting the proposed building on a conventional shallow foundation system and for support of the asphalt parking/drive areas. Spread footings should provide an adequate support system for the structure.

We note that clayey fine sand was encountered between the ground surface and a depth of up to 2 feet below the ground surface at the locations of Auger Borings AB-11 through AB-13. Clayey soil can be difficult to moisture condition and compact and can also be problematic because its relatively low permeability promotes ponding of water during and following rainfall. This will require particular consideration if construction occurs during a rainy period. Ponding of water may be mitigated by sloping the excavations to promote positive site drainage. Use of a kneading type compactor (eg; Sheepsfoot roller) may assist with compaction of clayey soil. Care should be taken to ensure that the contractor is aware that clayey soil may be exposed on the site, and be prepared to handle the aforementioned issues. The contractor may elect to over-excavate to a depth of 1 or 2 feet (or deeper) where the clayey soil is present and backfill with sand if the contractor decides that removal and replacement is more feasible than compacting the clayey soil.

The following are our recommendations for overall site preparation, foundation support, and pavement construction which we feel are best suited for the proposed facility 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 and the parking/drive areas, plus a minimum margin of 5 feet, should be stripped of all surface vegetation, stumps, debris, organic topsoil or other deleterious materials, as encountered. Specifically, the organic topsoil as encountered in the borings to depths ranging from 2 to 4 inches should be stripped. 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.

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 area 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 proposed 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 2 feet in the building area and 1 foot in the parking/drive areas must be achieved beneath the stripped and grubbed ground surface. Additional passes and/or overexcavation and recompaction may be required if these minimum density requirements are not achieved. The soil moisture should be adjusted as necessary during compaction.

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.

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. The fine sand and fine sand with silt (Strata Nos. 1 and 2 without roots, as shown on the soil boring profiles in Appendix II) are suitable for use as fill soil and, with proper moisture control, should densify using conventional compaction methods. Soils with more than 12 percent passing the No. 200 sieve (Strata Nos. 3 and 4 in Appendix II) 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.

The use of soils with relatively high fines content (i.e.; silty and clayey soils) as fill should be avoided near the ground surface in green-space areas since these relatively low permeability soils promote ponding of water during and following rainfall. Also, in high groundwater areas, silty and clayey soils may cause a rise in the water table elevation due to capillary action. Additionally, these relatively low permeability soils should not be used directly beneath any pavement section as they may trap water within the pavement section leading to premature pavement failure.

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 2 feet 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 2 feet 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 18 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.

Dewatering

If the control of groundwater is required to achieve the necessary stripping, excavation, proof-rolling, filling, compaction, and any other earthwork, sitework, and/or foundation subgrade preparation operations required for the project, the actual method(s) of dewatering should be determined by the contractor. Dewatering should be performed to lower the groundwater level to depths that are adequately below excavations and compaction surfaces. Adequate groundwater level depths below excavations and compaction surfaces vary depending on soil type and construction method, and are usually 2 feet or more. Dewatering solely with sump pumps may not achieve the desired results.

Typical Asphaltic Concrete Surface Pavement Section

Site Preparation

All areas to be paved should be prepared as previously outlined. Prior to pavement base installation, the subgrade soil compaction should be verified for a depth of 12 inches (i.e.; compacted to at least 95 percent of the modified Proctor (ASTM D-1557, AASHTO T-180) maximum dry density value).

Limerock or Cemented Coquina Base

A limerock or cemented coquina base course 8 inches thick overlying a 10-inch thick stabilized subbase 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 cemented coquina should have a minimum Limerock Bearing Ratio (LBR) value of 100 and should be compacted to at least 98 percent of the modified Proctor (ASTM D-1557, AASHTO T-180) maximum density value.

A 10-inch thick subbase having a minimum Limerock Bearing Ratio (LBR) value of 40 must be achieved beneath the limerock or cemented coquina base. The natural soils may have to be stabilized with suitable clayey soil in order to achieve the required LBR value. The stabilized subbase must be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D-1557, AASHTO T-180).

Crushed Concrete Base (Optional)

An 8-inch thick crushed concrete base may be used. A minimum clearance of 12 inches should be maintained between the bottom of the crushed concrete base and the seasonal high groundwater table.

The crushed concrete base should have a minimum Limerock Bearing Ratio (LBR) value of 100 and should be compacted to at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557, AASHTO T-180). The crushed concrete should meet Graded Aggregate Base gradation requirements according to Section 204, of the Florida Department of Transportation Standard Specifications for Road and Bridge Construction, latest edition. The subgrade beneath the crushed concrete base should consist of free draining sand compacted to at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557, AASHTO T-180).

We note that if the contractor’s means and methods include stabilizing soils beneath the crushed concrete base, then the stabilizing material should be coarse material (e.g.; gravel). Low permeability soils (e.g.; silt and/or clay) should not be used as stabilizing material beneath crushed concrete base.

Wearing Surface

A minimum 2-inch layer of Type SP-9.5 or SP-12.5 asphaltic concrete should be used for a wearing surface in the parking/drive areas.

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. Equivalent Type S asphaltic concrete may be substituted for Type SP-9.5 or SP-12.5; however, we recommend a minimum Marshall stability of 2,200 pounds if Type S is used.

The latest specifications of Florida Department of Transportation shall govern the 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 are planned. For this study, soil conditions were explored in the proposed pond areas with five auger borings (AB-11 through AB-15) to a depth of 15 feet.

Soil Profile and Soil Permeability

The fine sand and fine sand with silt (Strata Nos. 1 and 2 in Appendix II) are generally considered to be relatively permeable. The silty fine sand and clayey fine sand (Strata Nos. 3 and 4 in Appendix II) are likely less permeable than the fine sand and fine sand with silt, and should be considered to be aquitards for retention pond drawdown evaluation. For areas where dry bottom retention ponds are planned, the clayey fine sand (Stratum No. 4) present at the ground surface should be excavated completely from the pond footprint plus a horizontal margin of at least 10 feet in all directions.

The results of the constant-head laboratory permeability tests are presented in the following table:

Test Location	Sample Depth (feet)	Measured Permeability (inches/hour)
TP-1/AB-11	3 - 3.5	2
TP-2/AB-13	2.5 - 3	6
TP-3/AB-14	1 - 2	10

It is noted that a suitable factor of safety should be used with these values. 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).

Other Retention Pond Considerations

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. Because of the high groundwater level conditions at this site, underdrains may be necessary to prevent growth of aquatic vegetation and for pond volume recovery. An outfall (i.e., stormwater drain, ditch, canal, natural low areas, etc.) will be necessary to utilize underdrains.

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* Reference: "Flexible Pavement Design Manual", Florida Department of Transportation. (2005)

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.

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 number 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. We recommend that Ardaman & Associates be retained to review the asphalt pavement mix design proposed for use on the project prior to pavement placement. During asphalt pavement construction, samples of the asphaltic concrete should be obtained and tested in the laboratory to verify compliance with the mix design, including testing Marshall Stability (Type S asphalt), flow, asphalt content, and aggregate gradation. We also recommend full-time monitoring/testing in the batch plant and on the site during pavement placement. 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 Central 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 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 linear feet of pipe. Additional tests should be performed in backfill for manholes, inlets, etc.

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

Representative samples of the various natural ground and fill soils, as well as stabilized subgrade (where applicable) and base materials 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 Figure 2 and in Appendix II, 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 North Brevard Economic Development Zone 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.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.
Certificate of Authorization No. 5950

Colin T. Jewsbury, P.E.
Senior Engineer
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CTJ/JPM/ljh

Cc: Mr. Rodney Honeycutt, P.E. - Honeycutt & Associates, Inc.

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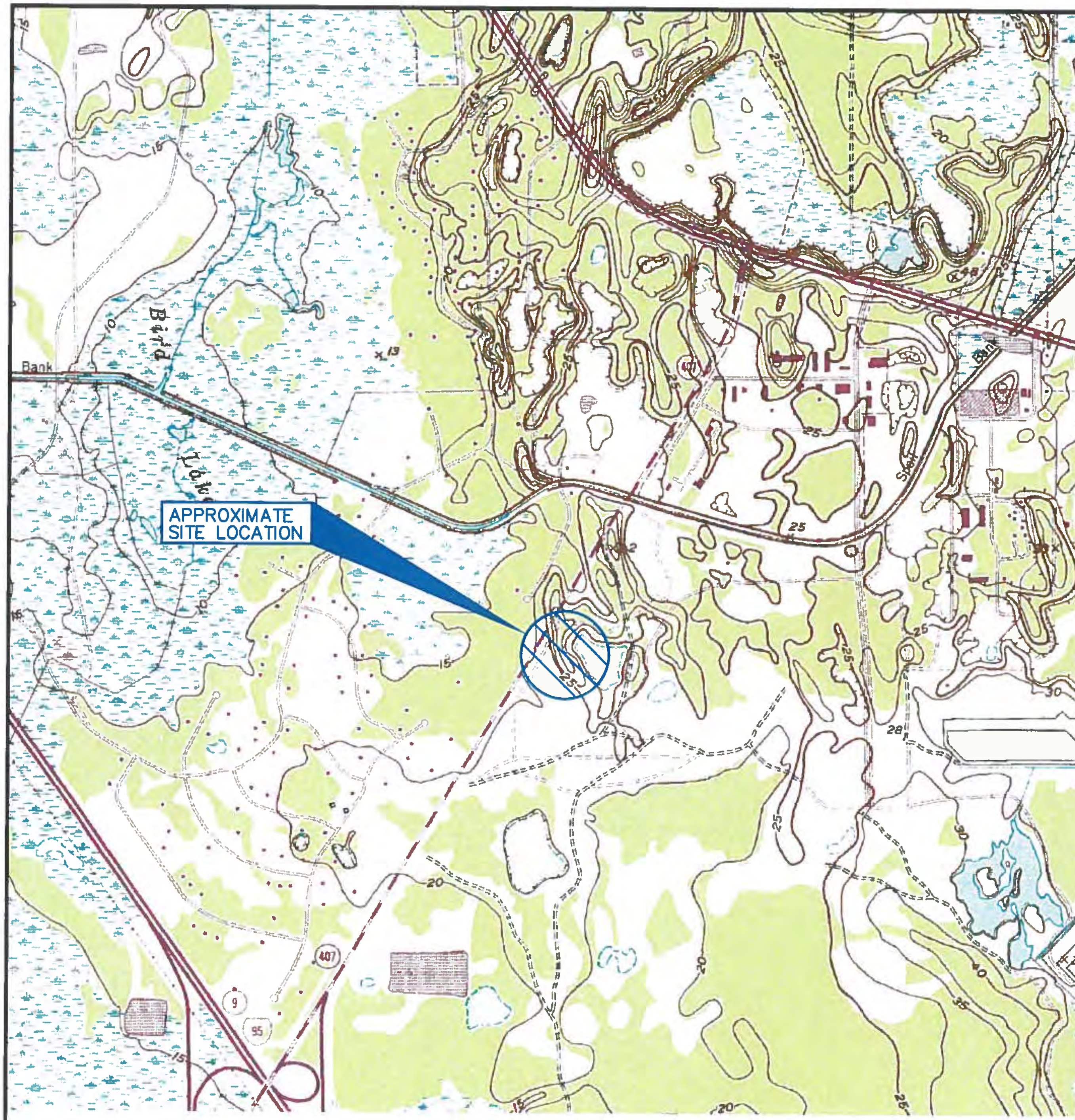

SECTION 4
TOWNSHIP 23 SOUTH
RANGE 35 EAST

OBTAINED FROM U.S.G.S. QUAD MAP: TITUSVILLE, FLORIDA

N

0 1000 2000

SCALE: — 1"=2000'

QUADRANGLE LOCATION

SITE LOCATION MAP

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

DRAWN BY: TAT	CHECKED BY:	DATE: 6/8/15
FILE NO. 15-5239	APPROVED BY:	FIGURE: 1

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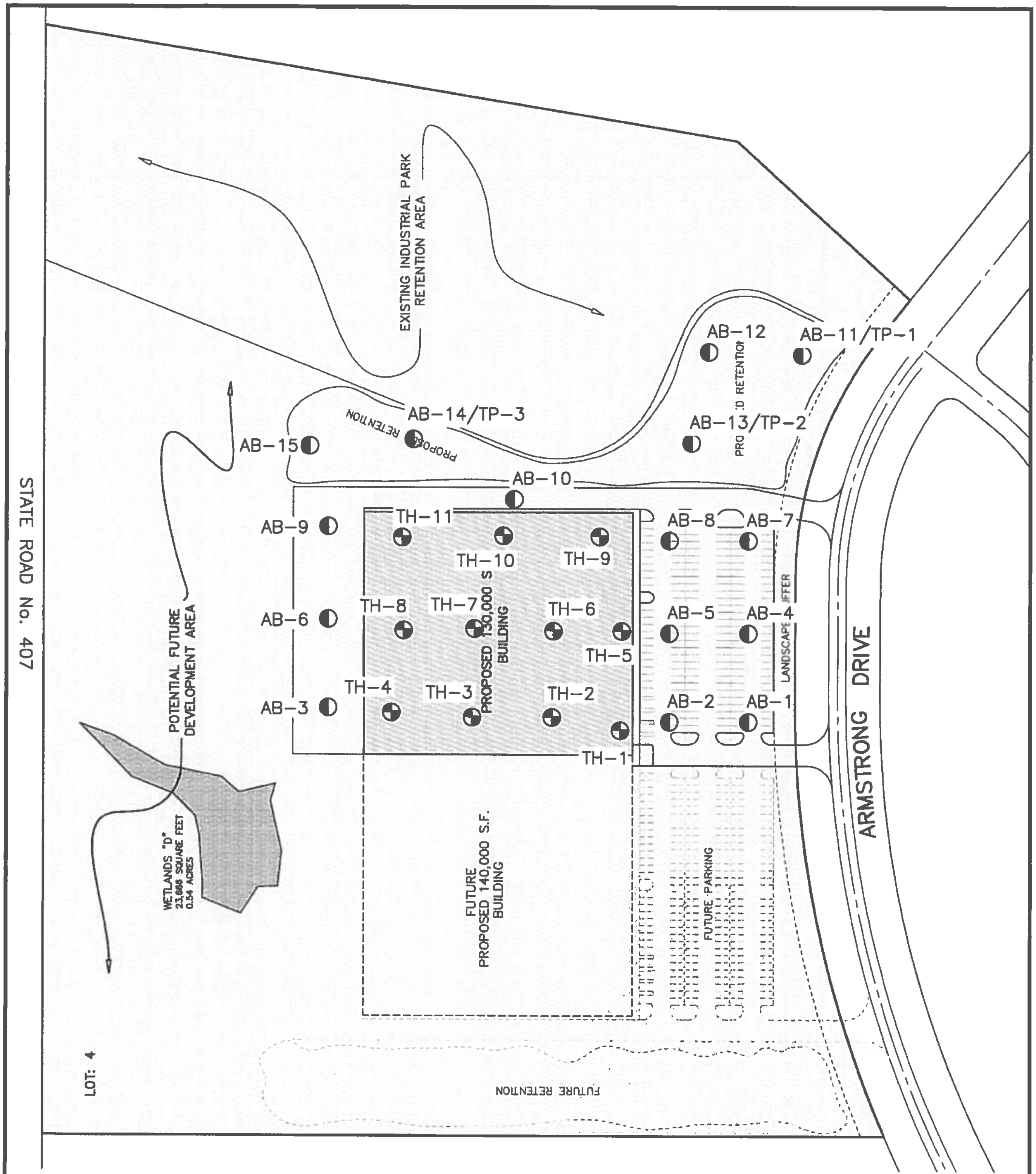

LEGEND

- TH STANDARD PENETRATION TEST (SPT) BORING LOCATION
- AB AUGER BORING LOCATION
- TP TEST PIT LOCATION

BORING LOCATION PLAN

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE PARK PARCEL
TITUSVILLE, FLORIDA

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APPENDIX I

Standard Penetration Test and Auger Boring Procedures

<u>STANDARD PENETRATION TEST</u>

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.

AUGER BORINGS

Auger borings are used when a relatively large, continuous sampling of soil strata close to ground surface is desired. A 4-inch diameter, continuous flite, helical auger with a cutting head at its end is screwed into the ground in 5-foot sections. It is powered by the rotating action of the Kelly bar of a rotary drill rig. The sample is recovered by withdrawing the auger out of the ground without rotating it. The soil sample so obtained, is classified and representative samples put in bags or jars and brought back to the laboratory for classification testing.

APPENDIX II

Soil Boring Profiles

LEGEND

SOIL DESCRIPTIONS

1. FINE SAND (SP)
2. FINE SAND WITH SILT (SP-SM)
3. SILTY FINE SAND (SM)
4. CLAYEY FINE SAND (SC)

COLORS

- A GRAYISH BROWN
- B DARK GRAY OR DARK BROWN
- C YELLOWISH BROWN OR ORANGE-BROWN
- D LIGHT BROWN TO BROWN
- E LIGHT GRAY TO GRAY

TH STANDARD PENETRATION TEST (SPT) BORING

AB AUGER BORING

N STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

▽ GROUNDWATER LEVEL MEASURED ON DATE DRILLED

SP,SP-SM SM,SC,CH UNIFIED SOIL CLASSIFICATION SYSTEM

NOTE: ALL SPT BORINGS WERE PERFORMED USING A SAFETY HAMMER IN THE UPPER 10 FEET AND AN AUTOMATIC HAMMER BELOW 10 FEET TO THE BORING TERMINATION DEPTH. AUTOMATIC HAMMER N-VALUES MAY BE CONVERTED TO EQUIVALENT SAFETY HAMMER N-VALUES BY MULTIPLYING BY 1.24. ALL REPORTED N-VAULES ARE SAFETY HAMMER.

ENGINEERING CLASSIFICATION

I COHESIONLESS SOILS

DESCRIPTION	BLOW COUNT "N"
VERY LOOSE	0 TO 4
LOOSE	4 TO 10
MEDIUM DENSE	10 TO 30
DENSE	30 TO 50
VERY DENSE	>50

II COHESIVE SOILS

DESCRIPTION	UNCONFINED COMPRESSIVE STRENGTH, QU, TSF	BLOW COUNT "N"
VERY SOFT	<1/4	0 TO 2
SOFT	1/4 TO 1/2	2 TO 4
MEDIUM STIFF	1/2 TO 1	4 TO 8
STIFF	1 TO 2	8 TO 15
VERY STIFF	2 TO 4	15 TO 30
HARD	>4	>30

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. ABSENCE OF WATER SURFACE DATA ON CERTAIN BORINGS IMPLIES THAT NO GROUNDWATER DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THESE LOCATIONS OR WITHIN THE VERTICAL REACHES OF THESE BORINGS IN THE FUTURE.

SOIL PROFILES LEGEND

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

DRAWN BY: TAT	CHECKED BY:	DATE: 6/8/15
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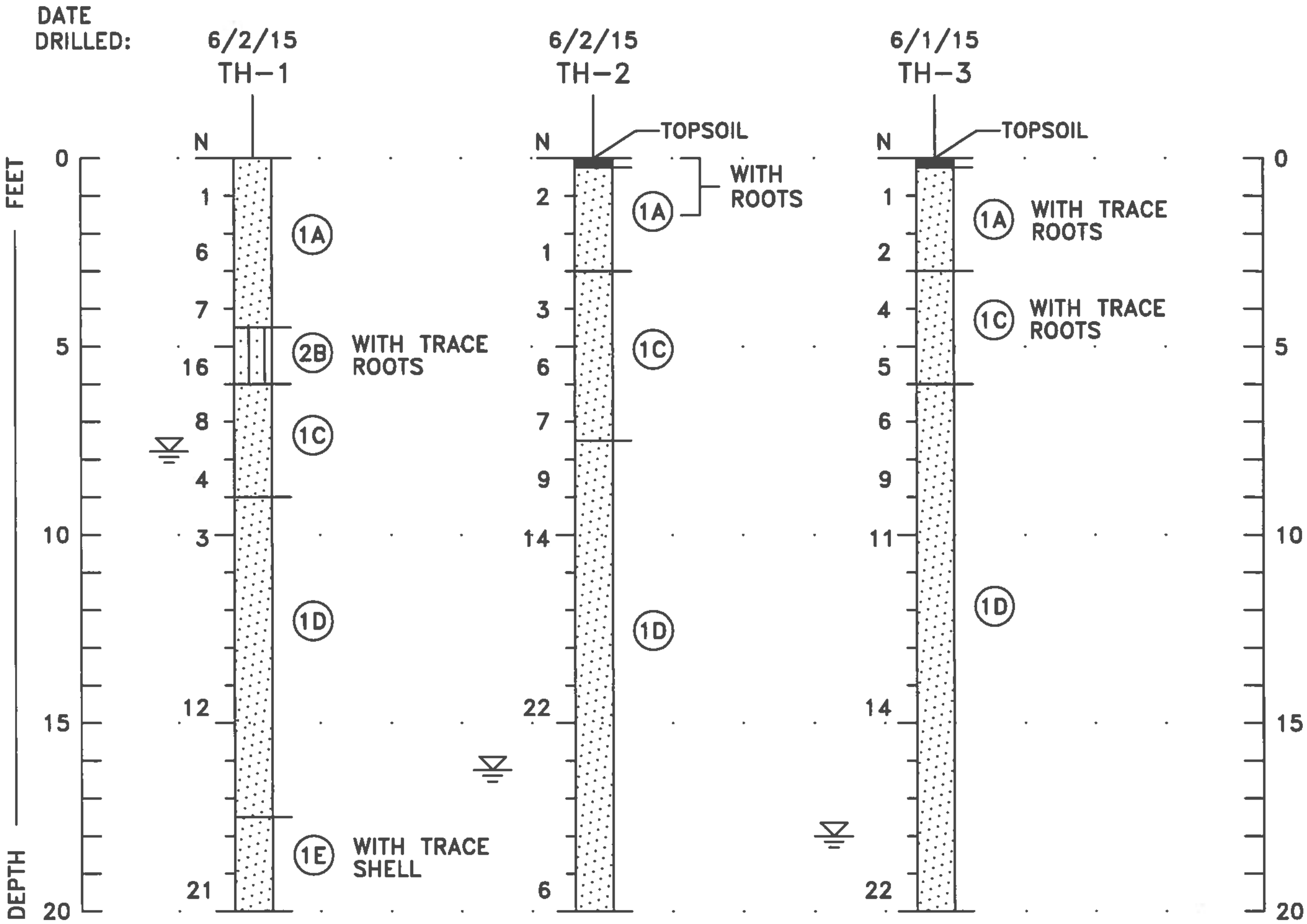

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

DRAWN BY: TAT	CHECKED BY:	DATE: 6/8/15
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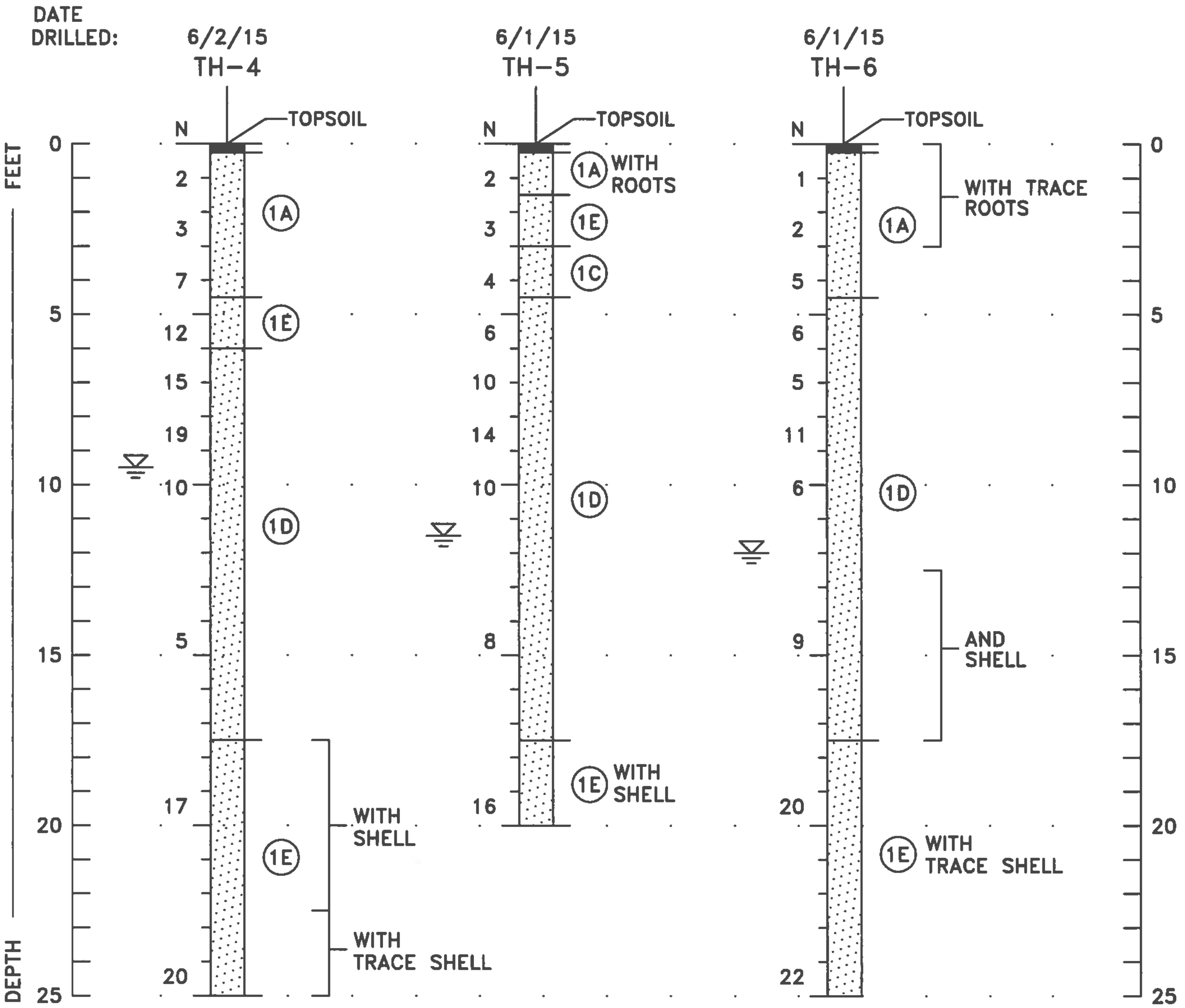

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

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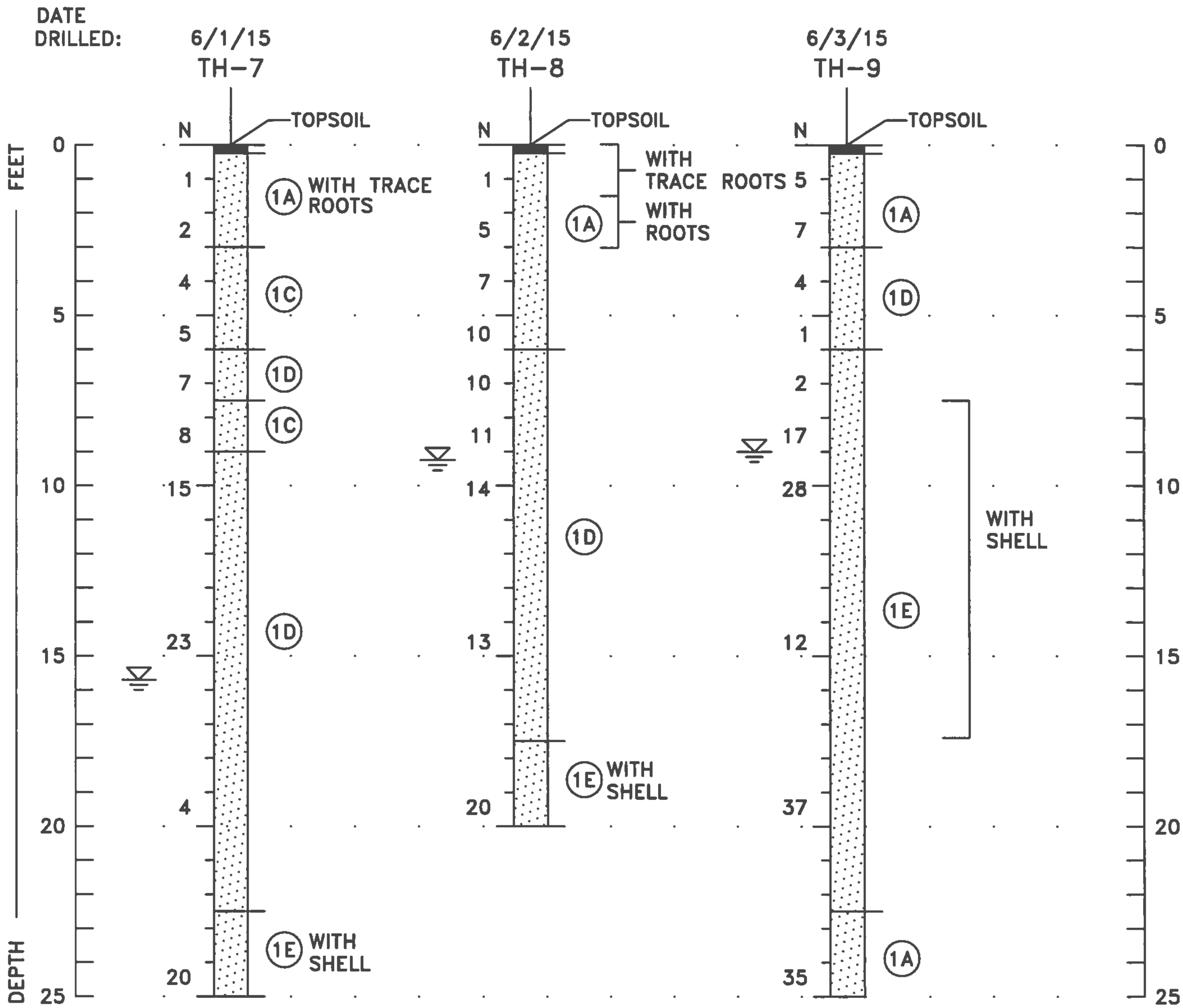

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

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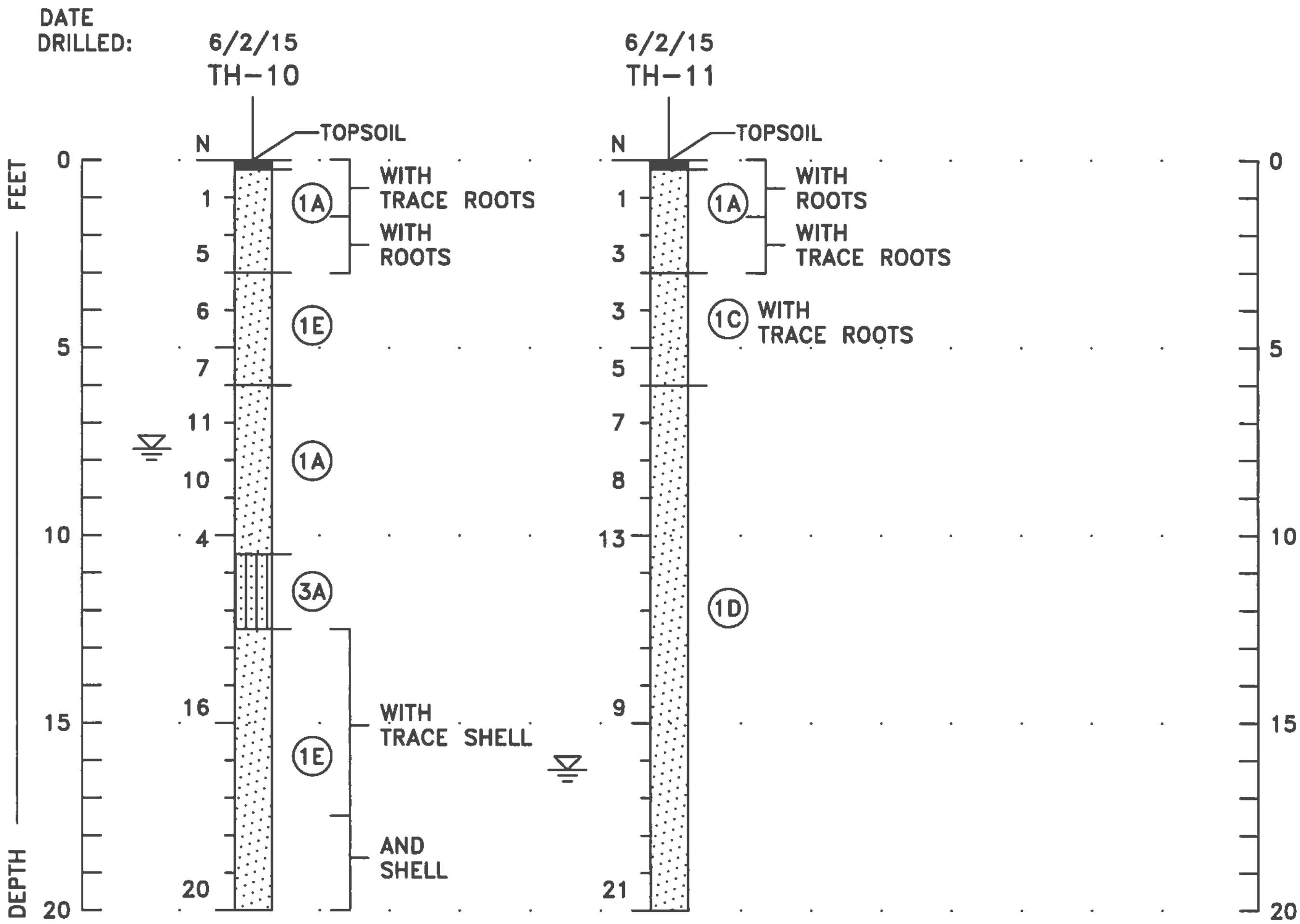

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

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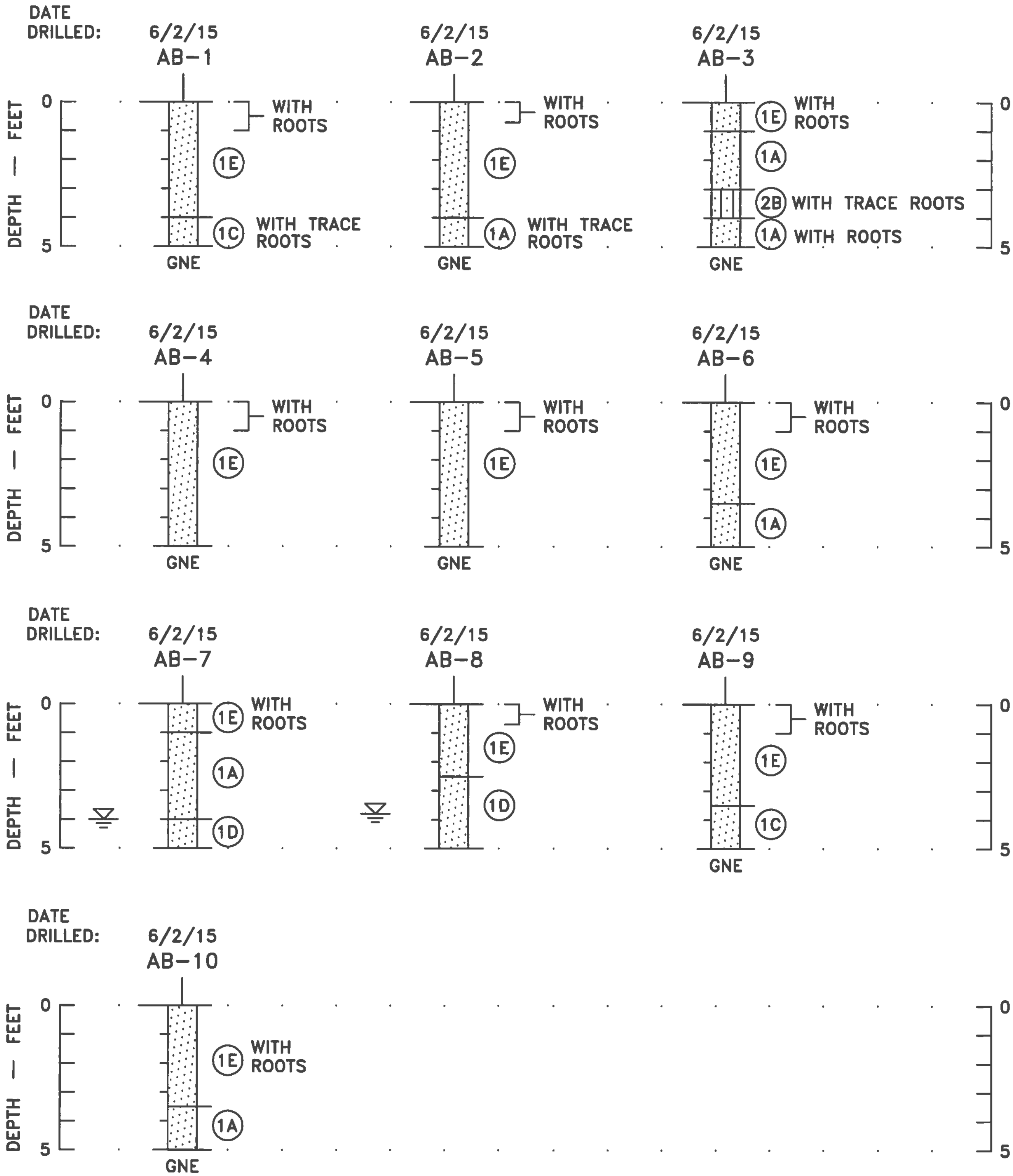

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE PARK PARCEL
TITUSVILLE, FLORIDA

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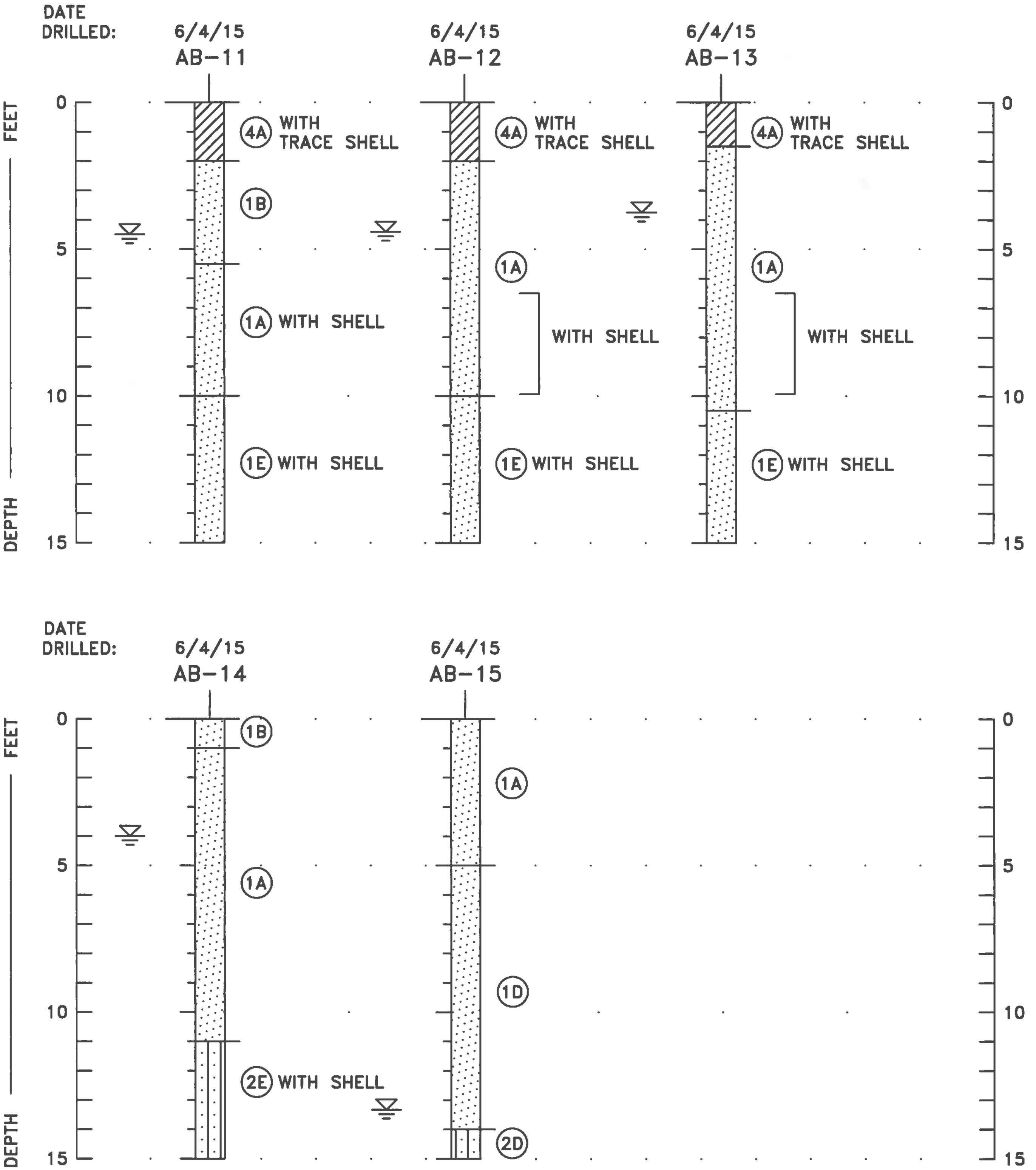

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
38-ACRE SPACEPORT COMMERCE
PARK PARCEL
TITUSVILLE, FLORIDA

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