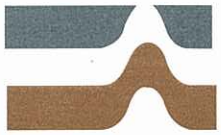
Subsurface Soil Exploration and
Geotechnical Engineering Evaluation
KSC Central Campus Design – Phase I
Kennedy Space Center, Florida



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January 22, 2013
File No. 12-23-5260
Revised January 25, 2013

Jones Edmunds & Associates, Inc.
730 Northeast Waldo Road
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Attention: Mr. Walter A. Nickel, P.E.

Subject: Subsurface Soil Exploration and
Geotechnical Engineering Evaluation
KSC Central Campus Design – Phase I
Kennedy Space Center, Florida
JEA Project No. 08930-001-02

Gentlemen:

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 and parking/drive 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 Central Campus Phase I project site includes the proposed NASA Headquarters Building (Headquarters Building) site located immediately south of NASA Parkway on the west and east sides of D Avenue Southeast on Kennedy Space Center, Florida. In addition, the project site includes the proposed Kennedy Data Center Building (KDC Building) site located just northeast of the intersection of C Avenue Southeast and 3rd Street on Kennedy Space Center.

Both of these proposed building sites are located in Section 5, Township 23 South, Range 37 East. The general project site location is shown superimposed on the Orsino, Florida USGS quadrangle map presented on Figure 1. The ground surface elevation in the project site vicinity is estimated to be approximately +12 feet NGVD based on the topographical map.

The majority of the Headquarters Building site and the entire KDC Building site are currently grass covered and sparsely wooded. Several improvements including asphalt roadways and parking areas and stormwater drainage ditches are located on portions of the proposed Headquarters Building site. The existing improvements will be removed from the project site prior to the new construction.

SOIL SURVEY REVIEW

The 1974 Soil Survey for Brevard County, Florida, as prepared by the U.S. Department of Agriculture (USDA) Soil Conservation Service, was reviewed during our evaluation. In addition, we reviewed the USDA's Web Soil Survey on December 26, 2012, to determine the soil types mapped for the project site. Based on these references, the majority of the north portion of the Headquarters Building site and the entire KDC Building site are mapped as the "Immokalee sand" soil series. The south portion of the Headquarters Building site is located in an area mapped as "Urban land". Smaller areas of "Pomello sand" and the "Riviera and Winder soils, depressional" soil series were also noted on the northeast portion of the Headquarters Building site. A description of the primary soil types identified on the two building sites, as obtained from the USDA, is presented below.

Immokalee sand (Im):

"Immokalee sand" is a nearly level to gently sloping, poorly drained sandy soil in broad areas in the flatwoods, in low areas between sand ridges, or in slightly elevated areas between ponds and sloughs. Included with this soil in mapping are small areas of Basinger, Myakka, Placid, Daytona, St. Johns, Satellite and Smyrna soils. The included soils generally make up less than 25 percent of any one mapped area. The general soil profile consists of sands to a depth of at least 80 inches.

In undrained areas, the water table is within 6 to 10 inches of the surface for 1 to 2 months in most years and between 10 and 40 inches more than half the time. Occasionally in very wet seasons it rises above the surface for a few days. Permeability is moderate or moderately rapid in the subsoil and rapid in the other horizons.

Urban land (Ur):

The "Urban land" (Ur) soil consists of areas that are 60 to more than 75 percent covered with streets, buildings, large parking lots, shopping centers, industrial parks, airports, and related facilities. Unoccupied areas, mostly lawns, parks, vacant lots, and playgrounds; are Astatula, Paola, Myakka, St. Lucie, Immokalee, Pomello, Cocoa, and Canaveral soils in tracts too small to be mapped separately.

PROPOSED CONSTRUCTION AND GRADING

Headquarters Building

It is our understanding that the proposed development includes the Phase I portion of the Headquarters Building. This building will include a 7-story central tower with one story sections on the west and south sides of the tower. The proposed footprint of the Phase I portion of the building encompasses an area of approximately 325 feet by 160 feet based on review of the preliminary site plan provided by Jones Edmunds & Associates (JEA). The Phase II portion of the building is anticipated to be a similar configuration and size to Phase I and be situated just east of Phase I.

We have assumed that the proposed one-story portions of the building will consist of load bearing masonry walls and interior columns with slab-on-grade floors. Information provided by WPM

indicates that column loads for the one-story portions of the building will range from 50 to 100 kips. The floor slab loads are anticipated to be on the order of 100 psf.

The first floor of the 7-story central tower is anticipated to be ground supported. The 7-story section of the building will reportedly consist of a poured-in-place concrete system with pan-formed beams and girders that will use shear walls in several locations for the lateral load resisting system. Elevated slabs will also be poured-in-place. A foundation plan with anticipated column and shear wall loads was provided by Walter P. Moore (WPM), the project structural engineers on, December 21, 2012. The foundation plan provided by WPM and used for our analyses was dated October 26, 2012. Based on our review of the plan and on discussion with WPM personnel, column foundation and mat bearing pressures (if shallow foundations are utilized) will not exceed 3,000 pounds per square foot (psf), with a maximum column load on the order of 1,500 kips and a maximum shear wall load on the order of 4,230 kips.

In addition to the building; asphalt passenger vehicle parking and drive areas are proposed for areas northwest, north, and northeast of the building area. An asphalt paved circular driveway is also planned for an area south of the building. Concrete paved truck delivery areas are planned for an area on the west side of the building. It is our understanding that 2 feet of fill is required to raise the building area and 1 to 2 feet of fill is required to raise the parking/drive areas to final elevations. If actual building loads, cut depth or fill height exceed our assumptions, then the recommendations in this report may not be valid.

KDC Building

The KDC Building will be a one-story building with footprint plan dimensions of approximately 198 feet by 100 feet. The proposed building will reportedly consist of load bearing tilt up concrete walls and interior steel columns with slab-on-grade floors. Information provided by WPM indicates that column loads for this building will be on the order of 110 kips and that wall loads are on the order of 2 kips per linear foot. The floor slab loads are anticipated to range from 80 psf in the general office areas to 250 psf in the areas where the high density computer racks will be situated.

It is our understanding that 4 to 5 feet of fill is required to raise the building area to final elevation. If actual building loads, cut depth 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 14 auger borings. SPT Borings B-1 through B-8 were performed within the proposed footprint of the Headquarters Building. Borings B-23 through B-25 were performed within the proposed footprint of the KDC Building. The borings were advanced to depths of 45 to 100 feet below ground surface (bgs) 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 areas at the Headquarters Building site. They were drilled using a truck-mounted, 4-inch diameter, continuous flight auger to a depth of 10 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 SPT borings were grouted with neat cement grout and the auger borings were backfilled with soil cuttings.

Test Locations

The approximate locations of the borings are schematically illustrated on site plans shown on Figures 2 and 3. These locations were determined in the field by hand-held GPS measurements and 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.

In addition, we conducted 10 organic content tests (ASTM D2974-87), 12 natural moisture content tests (ASTM D2216), 11 percent fines analyses (ASTM D1140), and two Atterberg limits tests (ASTM D4318) on selected soil samples obtained from the borings. The results of these tests are presented adjacent to the sample depth on the soil boring profiles in Appendix II.

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 borings indicate the following general soil profile in the proposed Headquarters Building area:

Depth Below Ground Surface (feet)	Description
0 to 5	Fine sand (Unified Soil Classification SP), fine sand with silt (SP-SM), and clayey fine sand (SC) with varying amounts of shell. Fill.
5 to 8.5	Very loose to very dense fine sand with silt (SP-SM), silty fine sand (SM), and sandy organic muck (OH, Pt). Some with significant root content.
8.5 to 12.5	Medium dense to very dense fine sand (SP) and hardpan-type soil.
12.5 to 28	Vey loose to medium dense fine sand (SP) and fine sand with silt (SP-SM) with varying amounts of shell.
28 to 48	Loose to medium dense fine sand (SP) and fine sand with silt (SP-SM) with varying amounts of shell.
48 to 80	Loose to medium dense fine sand with silt (SP-SM) and silty fine sand (SM) with varying amounts of shell.
80 to 98	Loose to medium dense silty fine sand (SM) and clayey fine sand (SC), and medium stiff to stiff clay (CL,CH).
98 to 100	Medium dense silty fine sand (SM).

Soils encountered in the proposed pavement areas at the Headquarters Building site included fine sand (SP), fine sand with silt (SP-SM), silty fine sand (SM), and clayey fine sand (SC). In addition, sandy organic muck was encountered in pavement Borings B-21 and B-22. A significant root concentration was encountered in pavement Boring B-14 from 1.5 to 2 feet bgs.

The results of the borings indicate the following general soil profile in the proposed KDC Building area:

Depth Below Ground Surface (feet)	Description
0 to 4	Fine sand (SP). Fill.
4 to 5	Fine sand with silt (SP-SM) with significant root content and hardpan-type soil.
5 to 6	Medium dense fine sand (SP).
6 to 13.5	Medium dense to very dense fine sand (SP) and fine sand with silt (SP-SM).
13.5 to 33.5	Very loose fine sand (SP).
33.5 to 45	Loose to medium dense fine sand (SP) and shell.

The above soil profiles are outlined in general terms only. Please refer to the soil boring profiles in Appendix II for soil profile details.

Groundwater Level

The groundwater level was measured in the boreholes on the day drilled after stabilization of the downhole water level. As shown on the soil boring profiles in Appendix II, groundwater was encountered at depths that ranged from 3.6 to 6 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.

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 rainfall to date in Brevard County upon completion of the field exploration program is approximately 39.4 inches, which is approximately 11.2 inches below 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 2 feet above the groundwater levels measured at the time of our field exploration.

ENGINEERING EVALUATION AND RECOMMENDATIONS

Recommendations for the Headquarters Building and KDC Building facilities are presented in the following sections.

General

The results of our exploration indicate that, with proper site preparation as recommended in this report, the existing soils, with the exception of the organic muck and soil containing significant roots, are suitable for supporting the proposed buildings on a conventional shallow foundation system (i.e. spread footings and/or a mat foundation). These recommendations for shallow foundations are applicable provided that the calculated settlements, as presented in the following report sections, are deemed acceptable. If so, spread footings and/or a mat foundation should provide an adequate support system for the structures. If the calculated settlements are deemed

unacceptable, then an alternative deep foundation system (i.e. piles) or deep soil improvement may be required. Recommendations for deep pile foundations have been included in subsequent sections of this report.

Preliminary indications from WPM are that foundation settlement for the structures will be limited to 1 inch of total settlement and up to ¾ inch of differential settlement. Settlements calculated for the different foundation scenarios evaluated during this project are presented in the following sections. In several of the foundation scenarios, the preliminary settlement limitations presented by WPM were exceeded based on our calculations.

Sandy organic muck and/or soil containing significant roots were encountered in building Borings B-2, B-3, B-4, B-6, B-7, B-23, B-24, and B-25 at depths ranging from 4 to 9 feet bgs. These soils are not suitable for structural support and should be removed in their entirety from beneath the proposed building areas as outlined in the "Demucking and Removal of Soil Containing Significant Roots in Building Areas" section presented below.

We note that sandy organic muck was encountered in pavement Borings B-21 and B-22 at depths ranging from 2.5 to 7 feet bgs, and in thicknesses of 0.5 and 2 feet, respectively. Also, significant roots were encountered in pavement Boring B-14 at depths of 1.5 to 2 feet bgs. In order to construct pavement areas that require normal long term maintenance, it may be prudent to remove the sandy muck and the soil containing significant roots in the pavement areas. However, consideration could be given to leaving these materials in place in the pavement areas due to the relatively light additional loading anticipated, the relatively low organic content of the muck in these areas, and the fact that the layers were not very thick at the locations encountered within the pavement areas. If the sandy muck and soil containing significant roots are left in place, higher than normal frequencies of pavement maintenance could be experienced in the areas where these materials are present.

The following are our recommendations for overall site preparation, foundation support, and pavement construction which we feel are best suited for the proposed facilities 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 "footprints" of the proposed buildings and the parking/drive areas, plus a minimum margin of five feet, should be stripped of all surface vegetation, stumps, debris, asphalt, concrete, 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.

All existing foundations, slabs and any other underground structures should be removed from the proposed construction areas. If pipes or any collapsible or leak prone utilities are not removed or completely filled (with grout or concrete), they might 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.

It has been our experience that soils surrounding existing buildings and pavement 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 by Ardaman & Associates, Inc.

Demucking and Removal of Soil Containing Significant Roots in Building Areas

The sandy organic muck (Soil Stratum No. 6 on the soil boring profiles in Appendix II) and soil containing significant roots are not suitable for providing structural support and must be removed ("demucked") to their entire vertical limits and to a minimum horizontal margin outside the structural areas equivalent to the depth to the bottom of the muck and/or soil containing significant roots, but no less than 5 feet if the depth to the bottom of the unsuitable soils is less than 5 feet.

The excavated unsuitable soil must not be used as structural fill material and should be disposed of as directed by the owner. The excavations should be sloped or braced to prevent slope failure, as required. Means and methods of preventing slope failure and providing a safe work zone relative to excavations is the responsibility of the contractor.

Demucking and removal of the soil containing significant roots will likely extend to depths below the groundwater table. Demucking and removal of the soil containing significant roots should be performed "in the dry". The use of well points and/or sheet piles may be required to help control groundwater during excavation and backfilling. Regardless of the dewatering method used, we recommend that the groundwater table be maintained at least 24 inches below all earthwork and compaction surfaces. Adjacent building foundations must be adequately supported during the excavation, if the foundations may be affected.

Prior to backfilling of the excavation, the bottom of the excavation must be inspected by Ardaman & Associates to verify the complete removal of all deleterious materials deemed unsuitable.

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 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 areas 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.

Due to the relatively high groundwater level at this site, proof-rolling may cause upward movement or "pumping" of the groundwater. However, 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. Depending on the time of year, a 12- to 18-inch layer of clean fine sand (SP) fill may be required prior to proof-rolling.

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). If requested, Ardaman & Associates could assist with pre- and post-construction surveys of existing structures and with vibration monitoring during construction. 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 presented 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 can be used in some applications, but will be more difficult to compact due to their inherent nature to retain soil moisture. The organic muck and soil containing significant roots should not be used for structural fill.

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.

We note that the partially cemented hardpan-type soils were encountered in Borings B-7 and B-21. Hardpan-type soils can be problematic for several reasons. First, hardpan can be difficult to excavate, often requiring special equipment; especially in confined excavations such as utility trenches, footings, etc. Excavated hardpan-type soils are often boulder-size chunks of cemented soils which are not easily broken down for re-use as structural fill. In addition, when pulverized into fragments that can be compacted to an adequately dense matrix, the in-place soil often fails the relative compaction test because the laboratory test continues to pulverize the soil into smaller particles resulting in a denser laboratory matrix than that which occurs in the field. Alternative acceptance criteria may need to be used for hardpan-type soils used as fill. This criteria would need to be developed on a site specific basis after observing the contractor's earthwork methodology and the nature and condition of the compacted hardpan-type soils.

Foundation Support by Shallow Spread Footings and Foundation Compaction Criteria

The following recommendations are contingent upon the magnitude of calculated settlement being acceptable to the design team. If not acceptable, then pile foundations or deep soil improvement may be required.

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.

KDC Building and One-Story Portions of Headquarters Building:

Based on the existing soil conditions and, assuming the above outlined demucking and/or removal of soil containing significant roots, 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 for the KDC Building and for the one-story portions of the Headquarters Building. This bearing pressure should result in foundation settlement within tolerable limits (i.e., a maximum of around 1 inch) for the KDC Building and for the one-story portions of the Headquarters Building.

All bearing wall foundations for the KDC Building and the one-story portions of the Headquarters Building 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.

Central Tower Portion of Headquarters Building:

The following recommendations for shallow foundations assume that the above outlined demucking and/or removal of soil containing significant roots, proof-rolling and compaction criteria are implemented. Based on information provided by WPM, the bearing pressures for the shallow spread column and shear wall foundations for the tower portion of the Headquarters Building will be limited to 3,000 psf. Based on our evaluation, this maximum bearing pressure is suitable for design of the foundations (not considering settlement which is discussed below). In addition, a temporary increase of 30% may be used for transient wind loads.

Using the column loads provided by WPM and a maximum bearing pressure of 3,000 psf for the column and shear wall foundations, an evaluation of foundation settlement the tower portion of the Headquarters Building was performed using the computer program Settle 3D. A maximum fill thickness of 2 feet and a depth to the bottom of the foundations of 5.2 feet were used in our analyses. For the sandy soils encountered, published correlations relying on SPT N-values were used to estimate the elastic moduli. The Westergard stress distribution method was used for calculating the stress changes caused by the estimated weight of the proposed structure in the underlying foundation soils. The computer program output for the settlement analyses is included in Appendix III. As shown in Appendix III, settlement calculated under the estimated loading conditions consisted of a little over 3 inches for the large interior mat foundation with up to around 2 inches of differential settlement between this mat and adjacent columns. It is estimated that the

majority (approximately 70 percent) of the calculated settlement will occur during construction as the loads are applied.

Output from the Settle 3D computer program is included in Appendix III for both the KDC and Headquarters buildings. Please refer to Appendix III for settlement calculated at individual column locations. Settlement at locations not included in the analysis can be provided if requested

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.

Mat Foundation Support and Foundation Compaction Criteria – Option for Headquarters Building

Foundation Recommendations

It is our understanding that consideration is being given to founding the central tower section of the Headquarters Building on a mat foundation. The following recommendations are contingent upon the magnitude of calculated settlement being acceptable to the design team. If not acceptable, then pile foundations or deep soil improvement may be required.

Excavate the building foundation to the proposed bottom of mat elevation and, thereafter, verify the in-place compaction for a depth of 2 feet below the bottom of the mat. If necessary, compact the bottom of the excavation to achieve a minimum dry density equivalent to 95 percent of the modified Proctor maximum dry density (ASTM D-1557) for a depth of 2 feet below the bottom of the mat.

The dimensions of the proposed mat foundation were provided by a WPM representative on January 16, 2013. Settlement analyses for the proposed mat foundation were performed using the provided mat dimensions, an average mat bearing pressure of 1,350 psf, and the results of our field exploration. For the sandy soils encountered, published correlations relying on the SPT N-values were used to estimate the elastic moduli. The Westergard stress distribution method was used for calculating the stress changes caused by the estimated weight of the proposed structure in the underlying foundation soils.

Based on our analyses, settlements calculated under the estimated loading conditions consisted of approximately 2.5 inches of total settlement at the center of the mat foundation and approximately 1 inch of differential settlement. It is estimated that the majority (approximately 70 percent) of the calculated settlement will occur during construction as the loads are applied.

If the one-story sections of the Headquarters Building surrounding the central tower are structurally connected to the tower, than we recommend that these connections be made as late in the construction sequence as possible in order to accommodate the anticipated settlements.

Foundation Support by Deep Pile Foundations – Option for Headquarters Building

The relatively loose soil conditions encountered in the upper levels of the soil profile create the potential for excessive total and differential settlements for the relatively heavily loaded 7-story central tower portion of the Headquarters Building. Due to these conditions, several deep foundation alternatives have been considered by us at this site.

Foundation Support by Driven Piles

We have analyzed allowable compressive capacities for 12-, 14-, and 18-inch prestressed, precast concrete (PPC) piles for the support of the Headquarters Building. Static pile capacities were estimated utilizing SPT N-values from Borings B-1, B-4 and B-7 for this study. The calculations incorporate a factor of safety of 2 for both the side friction and mobilized end bearing. A summary of the results of the axial pile capacities analyses versus tip depths are presented in the table below. These capacities are based on a minimum pile spacing of 3.0 times the width of the pile measured center-to-center. If pile spacing is less, pile capacity will be reduced.

Pile Type	Estimated Tip Depth* (feet)	Allowable Axial Compressive Capacity (tons)
12" PPC	60	45
	70	50
	80	60
	90	60
14" PPC	60	55
	70	60
	80	65
	90	70
18" PPC	60	70
	70	75
	80	85
	90	90
* Estimated Tip Depth is in feet below ground surface at the boring locations.		

Pile tip depths presented above are estimates based on the results of our analyses. The actual pile lengths and capacities may be higher or lower than estimated and will be determined during driving. Uplift capacities of the piles are anticipated to be at least 50 percent of the compressive capacities presented above. We note that if driven piles are deemed a desirable foundation alternative, it may be beneficial to extend some of the soil borings to deeper depths where significantly higher capacities than indicated above may be achievable.

Settlement of the pile groups due to the application of the building loads is anticipated to be minimal, on the order of ¼ to ½ inch.

We recommend a test pile program at the beginning of pile construction consisting of a minimum of two piles instrumented with a Pile Driving Analyzer (PDA). The data will be used to evaluate hammer-driving system performance, pile stresses during driving, pile structural integrity, and finalization of allowable axial pile capacity. This test pile data will also be useful for estimating production pile lengths.

At least 10 days prior to initiation of pile driving, the pile driving contractor will need to provide the Geotechnical Engineer (Ardaman & Associates, Inc.) with the following information regarding the proposed piles and pile driving system:

A. Manufacturer's specifications of the proposed hammer including all modifications.

B. Description and dimensions of the capblock material.

C. Description and dimensions of all cushion material.

D. Plans showing dimensions and details of construction of any followers to be used.

E. Any additional pertinent information necessary for wave equation analysis.

The Contractor should perform his own wave equation analysis to determine the validity of the proposed pile driving equipment and should include the results of this analysis along with the above information.

As specified in Revised FDOT Specification Section 455, piles should not be driven beyond practical refusal (20 blows per inch). The piles used on this project should conform to the latest FDOT Standard Specification regarding precast, prestressed concrete piles. Piles should be driven with a steam, air, hydraulic, or diesel hammer. At all times, the hammer should be operated at the chamber pressure and speed recommended by the manufacturer. Pile driving should be as continuous an operation as possible and should proceed without stopping over the last 10 feet of penetration. Pile redrives and/or set checks should be performed as necessary or deemed prudent. Should the contractor change hammers or procedures such that the energy delivered to the production pile is different than that of the test piles, the engineer should be contacted. Additional PDA tests may be required and the driving criteria modified.

During driving, pile driving records should be kept for each pile detailing pertinent information such as the pile type, pile length, date driven, and blow count per foot. The capacity of each pile should be reviewed based on its final tip elevation and driving record. We recommend that pile driving operations be continuously monitored by an Ardaman & Associates' geotechnical engineer or his representative.

It is noted that during pile driving, vibration may cause settlement/displacement in existing nearby structures. Care should be exercised to avoid damaging any neighboring structures while the compaction operation is underway. Pile driving may also create excessive noise that could be problematic for activities at the subject site. 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). Pre-construction condition surveys of nearby structures should be considered due to the proximity of the nearby structures. Ardaman & Associates would be pleased to provide this service.

The soil profile encountered at the project site is generally loose to medium dense with the exception of some dense to very dense soils encountered in the top 10 to 20 feet. Therefore, hard driving conditions are not anticipated, particularly once the top 10 to 20 feet of soil is penetrated. However, it is difficult to quantify vibration levels prior to the start of pile driving. We recommend that Ardaman & Associates be retained to perform vibration monitoring during pile driving at the site. If vibration levels during pile driving are found to be excessive, the contractor should be prepared to preform/predrill the top 10 to 20 feet at the pile locations prior to driving in order to reduce vibration levels. Underpinning of the adjacent buildings may be required to accommodate pile driving.

Foundation Support by Auger Cast Piles (Alternative)

When considering the installation of auger cast piles, the designer should realize that the load capacity of the completed pile is dependent upon the skill and care exercised by the Contractor. We recommend a detailed specification be developed for their installation.

We have analyzed allowable compressive capacities for 12-, 14-, and 18-inch augered cast-in-place piles (ACIP). Static pile capacities were estimated utilizing SPT N-values from Borings B-1, B-4 and B-7 for this study. A summary of the results of the axial pile capacities analyses versus tip depths are presented in the table below.

Pile Type	Estimated Tip Depth* (feet)	Allowable Axial Compressive Capacity (tons)
12" ACIP	55	40
	60	45
	65	50
	70	55
	75	60
	80	65
14" ACIP	55	50
	60	55
	65	60
	70	65
	75	70
	80	75
18" ACIP	55	65
	60	70
	65	75
	70	85
	75	90
	80	95

The calculated allowable capacities presented in the table above are based on a factor of safety of 2, and are developed by skin friction along the length of the pile plus end bearing. A factor of safety of 2 is typically used when static load compression testing is performed to verify capacity. Minimum pile spacing should be 3.0 pile diameters from center to center of the piles in a group to prevent group interaction which could result in a loss of some of the capacity of an individual pile. Capacities must be verified in the field by actual load tests, and pile size and length adjusted, if

necessary. Allowable tension capacities per individual pile are calculated to be at least 35 percent of the calculated axial capacities presented in the preceding table.

Settlement of the pile groups due to the application of the building loads is anticipated to be minimal, on the order of ¼ to ½ inch.

Care should be taken to maintain the vertical and horizontal alignment of the piles. We recommend that the concrete grout used to form the piles reach a compressive strength of at least 5,000 psi in 28 days.

The lower five feet of the pile borehole should be double-reamed and double-pumped. That is, once the auger reaches the specified depth, the grout pumping should commence, and the auger should be retrieved a height of 3 to 5 feet, and then reinserted to the bottom of the pile while the grout is being pumped. Once the auger is reinserted to the bottom of the pile, it may be retrieved slowly to the ground surface as the grout is being pumped.

The amount of concrete grout used to form each pile should be at least 1.2 times the theoretical pile volume. At a minimum, this calculated volume of grout is to be pumped per foot of pile as the auger is retrieved. The Contractor will need to install each pile such that an adequate cross section is maintained.

Piles should not be installed within 10 pile diameters, center to center, of a pile filled with grout that is less than 24 hours old. If grout heave occurs at a nearby pile, this distance must be increased to a sufficient distance so that the installation of adjacent piles does not affect grout set or pile geometry.

If grout pumping and/or auger retrieval operations are stopped at any time during the formation of a given pile, the borehole is to be reaugered and the pile formed anew. If the concrete level in any completed pile drops, the pile shall be rejected and replaced. If there is difficulty in placing the reinforcement steel in any pile, the pile shall be redrilled and replaced. All reinforcement steel should be fitted with spacers including a spacer at its lower tip to allow easier installation into the pile and assure its centering. Reinforcement must extend the full depth of the pile through which tension resistance is needed. However, deep reinforcement should consist of 1 or 2 bars clustered in the center of the pile. Steel reinforcement cages, if needed, should be limited in size and depth.

Any modification to these procedures is to be approved by the Geotechnical Engineer based on observations during pile installation.

We recommend conducting a minimum of one static load compression test in accordance with ASTM D-1143 (Quick Load Test Method) at a non-production pile location prior to commencing production pile installation to finalize the load capacity and pile length requirement. The test load should be at least twice the design load. The contractor should supply the load test set up and jacking equipment. Ardaman & Associates, Inc. will provide the gauges and reference beams and monitor pile vertical movements. The contractor's jack and pump must be calibrated just prior to the load test, by a certified laboratory and the results of the calibration must be provided to Ardaman prior to the pile load test.

The results of the load test along with the load test pile installation methodology will be used to establish the methodology of pile installation for production piles. Actual production pile lengths

may be longer than those presented in the axial capacity table presented above pending the results of the load test.

In addition, we recommend that low strain pile integrity testing be conducted on some of the auger cast piles prior to acceptance. The number of piles to be tested will depend on the contractor's installation and site operation procedures, but at a minimum, 20 percent of the piles should be tested.

Dewatering

Based on the groundwater conditions encountered, the control of the groundwater may be required to achieve the necessary stripping and subsequent construction, backfilling, and compaction requirements presented in the preceding sections. The requirement for control of groundwater should particularly be anticipated for demucking, removal of soil containing significant roots, and footing and utility excavations. The actual method(s) of dewatering should be determined by the contractor. However, regardless of the method(s) used, we suggest drawing down the water table sufficiently, say 2 to 3 feet, below the bottom of any excavation or compaction surface to preclude "pumping" and/or compaction-related problems with the foundation soils.

Dewatering should be accomplished with the knowledge that the permeability of soil tends to decrease with an increasing silt and clay content. Therefore, silty fine sand is typically less permeable than fine sand. The SP, SP-SM, and some SM type soils can usually be dewatered by well pointing or ditch/sump methods.

At Rest Earth Pressures Acting on Retaining Walls

The following recommendations are for retaining walls that will be founded no deeper than 6 feet below existing grade at the Headquarters Building and KDC Building sites. Lateral loads acting on retaining walls and embedded structures will include at-rest earth pressures as well as hydrostatic pressures and surcharge loads. The lateral earth pressure will be a function of both the soil unit weight (submerged or moist) and the depth below ground surface. The following equation can be used to determine the lateral at-rest earth pressure:

$$\sigma_h = K_o \gamma_m h \text{ (above groundwater table)}$$
$$\sigma_h = K_o [\gamma_m h_w + \gamma_b (h - h_w)] \text{ (below groundwater table)}$$

Where:

- σ_h = lateral earth pressure (psf)
- K_o = coefficient of at rest earth pressure (0.5) (this value assumes that the soil backfill is lightly compacted yet not overcompacted)
- γ_m = moist unit weight of soil = 120 pcf for compacted moist soil above the water table.
- γ_b = buoyant unit weight of soil = 65 pcf for compacted saturated soil below the water table.
- h = vertical depth (feet) below grade at which lateral earth pressure is determined
- h_w = vertical depth (feet) below grade to groundwater table.

For design, an appropriate factor of safety should be applied to the lateral earth pressure calculated using the above equation. Lateral pressure distributions determined in accordance with the above do not include hydrostatic pressures or surcharge loads. Where applicable, they should be incorporated in the design.

Backfill placed behind the retaining walls should consist of granular soils that are free draining and relatively free of fines. The backfill within 5 feet of the retaining wall should be placed in thin lifts and should be compacted with hand-held or light-weight compactors to 93 percent of the modified Proctor maximum dry density (ASTM D-1557). Overcompaction of the backfill should be avoided since it could cause excessively large earth pressures to develop against the walls. Dewatering should be performed during the construction, if necessary, to perform the compaction "in the dry". Heavy equipment should be kept at least 5 feet away from the wall. To reduce the potential for the build up of water pressure, weep holes should be installed at the base of the wall.

An active earth pressure coefficient value of 0.33 may be used for design. In addition, a coefficient of friction against sliding between the subgrade soils and the concrete foundation of 0.4 should be appropriate for well compacted sandy subgrade.

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).

Soil-Cement Base

Install a minimum 6-inch thick soil-cement base compacted to at least 95 percent of the standard Proctor maximum dry density (AASHTO T-134). The soil cement should have a minimum laboratory compressive strength of 300 pounds per square inch (psi) in seven days. For truck parking and drive areas, increase the minimum thickness to 8 inches. A minimum clearance of 12 inches should be maintained between the bottom of the soil cement base and the normal seasonal high groundwater level.

The term soil-cement applies to a type of base material that utilizes cement mixed with a select soil to develop its strength. We recommend conducting a laboratory soil-cement design to determine the actual cement content necessary. Shrinkage cracking normally occurs during the hydration of a soil-cement base and is reflected through the asphaltic concrete surface. We strongly recommend a curing time period of at least seven days before applying the overlying asphaltic concrete wearing surface. This waiting period will not eliminate the cracks in the pavement surface, but will help reduce the severity of the cracks. An approved tack coat should be applied on a clean (swept) soil-cement surface to develop a sufficient bond before the paving begins. The base should be inspected and sounded for hardness prior to paving operations. Any soft or otherwise unsuitable areas should be repaired as recommended by Ardaman and Associates.

Cracks in the pavement surface that occur when soil-cement base is used are sometimes unappealing to owners. We recommend that an owner unaccustomed to these cracks observe examples of pavement cracks in actual soil cement applications to verify that cracks are acceptable.

Free-draining subgrade soils (i.e; soils with less than 12 percent fines by dry weight) are recommended beneath soil-cement base to promote drainage of infiltrating water. If the contractor's means and methods include stabilizing soils beneath the soil-cement 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 soil cement base.

Limerock Base (Optional)

In lieu of soil cement, a limerock base course 6 inches thick overlying an 8-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 base material could lead to premature pavement distress. A minimum clearance of 18 inches must be maintained between the bottom of the limerock base and the seasonal high groundwater table.

The limerock 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. For truck parking and drive areas, the base thickness should be a minimum of 8 inches.

An 8-inch thick subbase having a minimum Limerock Bearing Ratio (LBR) value of 40 must be achieved beneath the limerock 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)

Crushed concrete base may be used. We recommend that the crushed concrete base course be supported by a free-draining subgrade. Six inches of base should be used in automobile parking areas while 8 inches of crushed concrete base should be used in truck parking and drive areas. 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, 2013 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 1½-inch layer of Type SP-9.5 or SP-12.5 asphaltic concrete should be used for a wearing surface in automobile parking areas. For truck parking and drive areas, 2 inches of Type SP-9.5 or SP-12.5 asphaltic concrete should be used.

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, 2013 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.

Typical Concrete Pavement Section

This typical pavement section is provided if a rigid, Portland cement concrete pavement is desired for the proposed parking/drive areas. We have assumed that the subject pavement will be subjected to truck loads typical of normal street traffic (i.e: maximum 18-kip axle loads). In addition, the average daily truck traffic is assumed to be fewer than 100 per day. The following site preparation and design recommendations should be utilized.

All areas to be paved should be stripped and grubbed of surficial debris including vegetation, roots and organic matter. All existing asphalt and concrete should be removed. The subgrade should then be compacted to achieve a density equivalent to 95 percent of the modified Proctor (ASTM D-1557, AASHTO T-180) maximum dry density value for a depth of 12 inches. This will provide a modulus of subgrade reaction of at least 150 pounds per cubic inch. All fill beneath pavement areas should comply with the recommendations (material types and compaction requirements) presented in the "Suitable Fill Material and the Compaction of Fill Soils" section of this report. Well drained soils (Unified Classification SP) must be utilized beneath the concrete pavement. A minimum clearance of 2 feet must be maintained between the bottom of concrete pavement and the normal seasonal high groundwater table level.

The concrete pavement should be a minimum of 6 inches in thickness and have a minimum compressive strength of 4,000 pounds per square inch.

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* Reference: "Flexible Pavement Design Manual", Florida Department of Transportation. (2008)

This pavement design is based on the "Guide for Design and Construction of Concrete Parking Lots" published by the American Concrete Institute (ACI) in 1988. Jointing of the pavement should comply with the specifications of the Portland Cement Association (PCA).

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. A representative of Ardaman & Associates should be present during the demucking and removal of the soil containing significant roots to ensure that all unsuitable soils are removed. 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. Soil cement base should be inspected and sounded for hardness after a minimum seven-day wearing period. Areas determined during the seven-day inspection to be soft or otherwise insufficient should be removed and replaced or otherwise repaired according to the geotechnical engineer's recommendations. 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.

The installation of test and production piles should be monitored full-time by a representative of Ardaman & Associates. 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 spread 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.

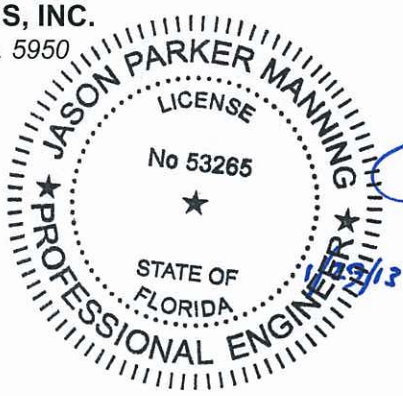
CLOSURE

The analyses and recommendations submitted herein are based on the data obtained from the soil borings presented on Figures 2 and 3, and in Appendix II, as well as the provided 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 Jones Edmunds & Associates, 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.

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



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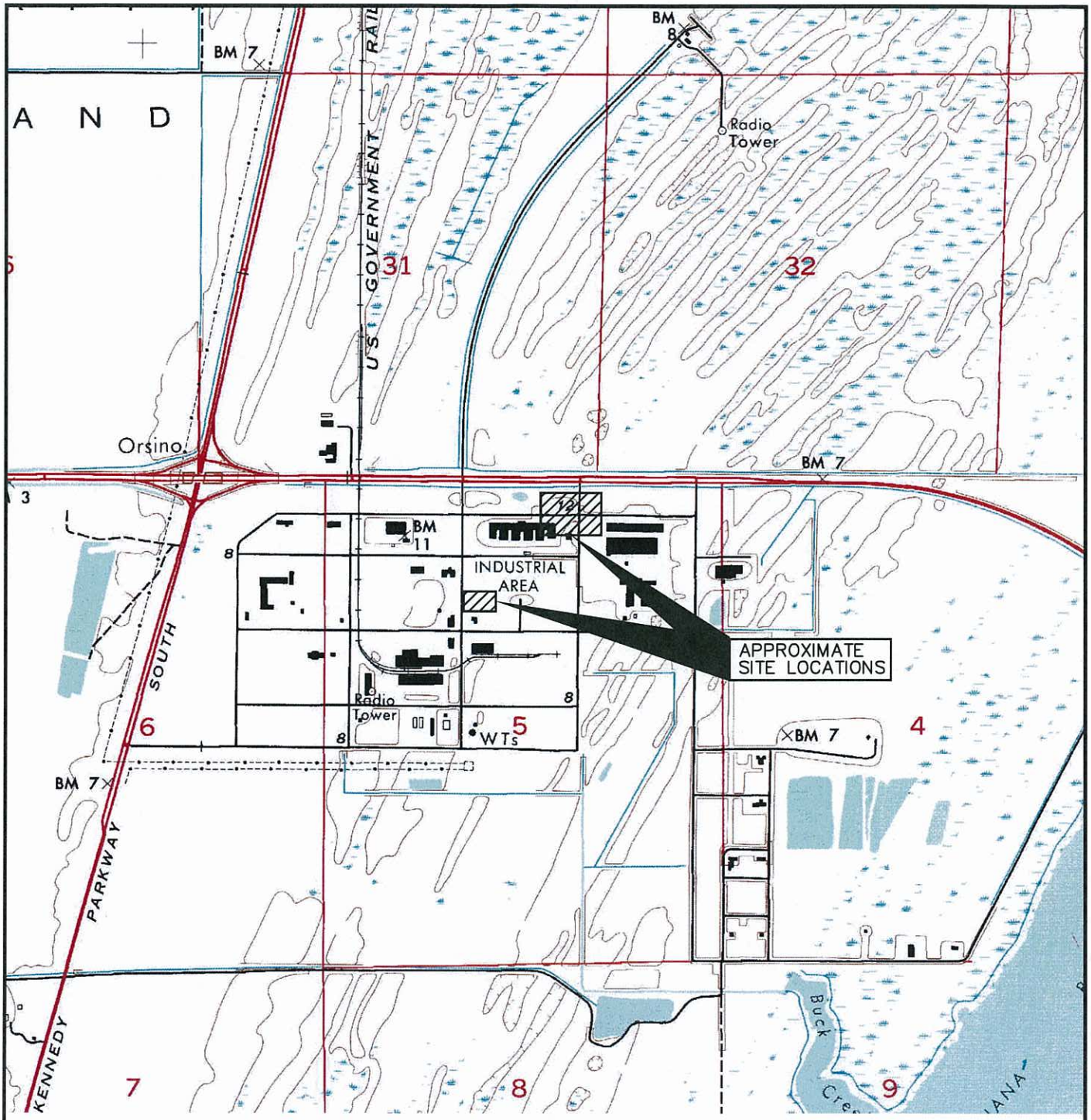

SECTION 5
TOWNSHIP 23 SOUTH
RANGE 37 EAST

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

N

0 1000 2000

SCALE: — 1"=2000'

QUADRANGLE LOCATION

SITE LOCATION MAP

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

SUBSURFACE SOIL EXPLORATION
CENTRAL CAMPUS DESIGN–PHASE I
KENNEDY SPACE CENTER, FLORIDA

DRAWN BY: TAT	CHECKED BY:	DATE: 11/30/12
FILE NO. 12–5260	APPROVED BY:	FIGURE: 1

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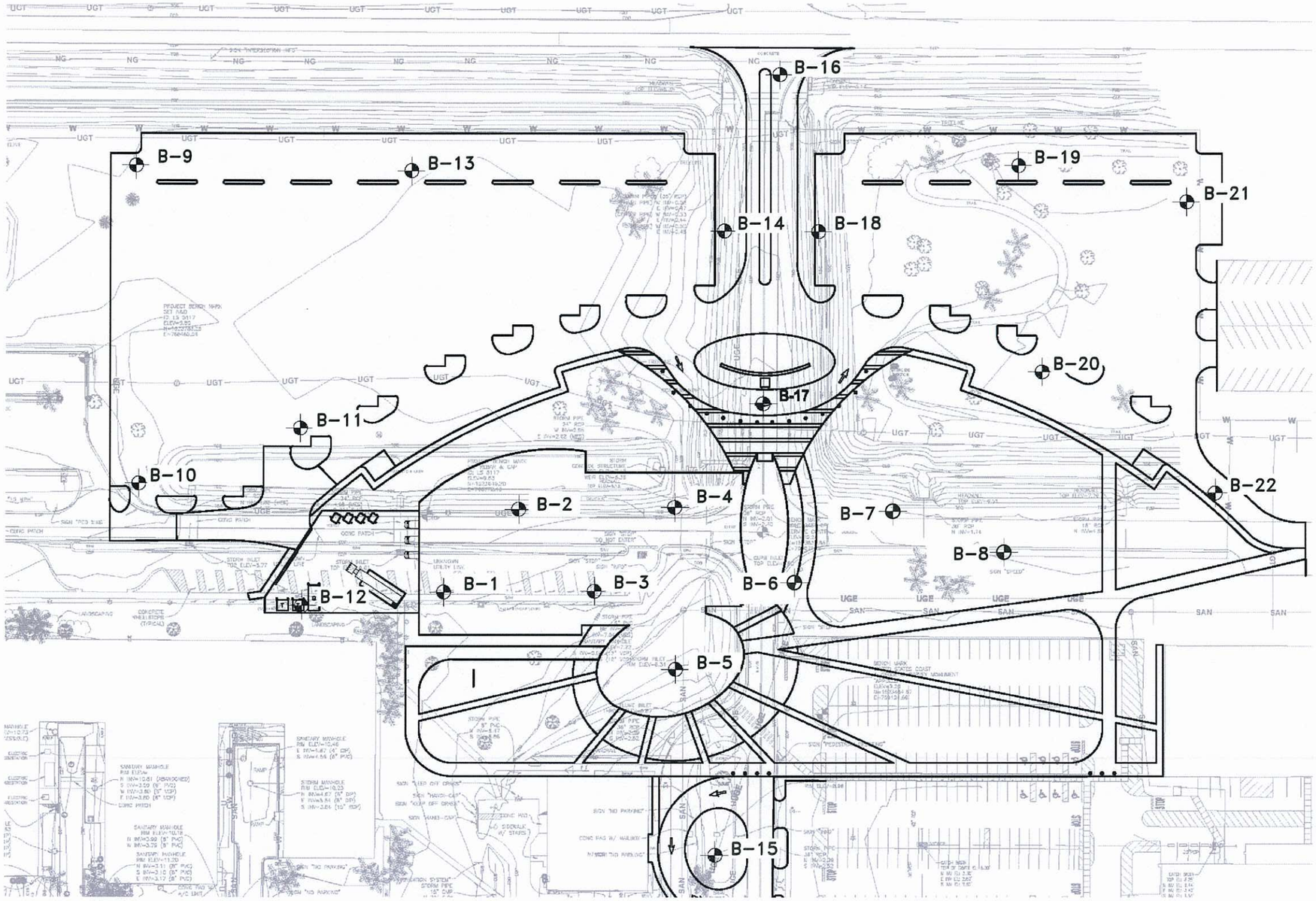


LEGEND

B SOIL BORING LOCATION

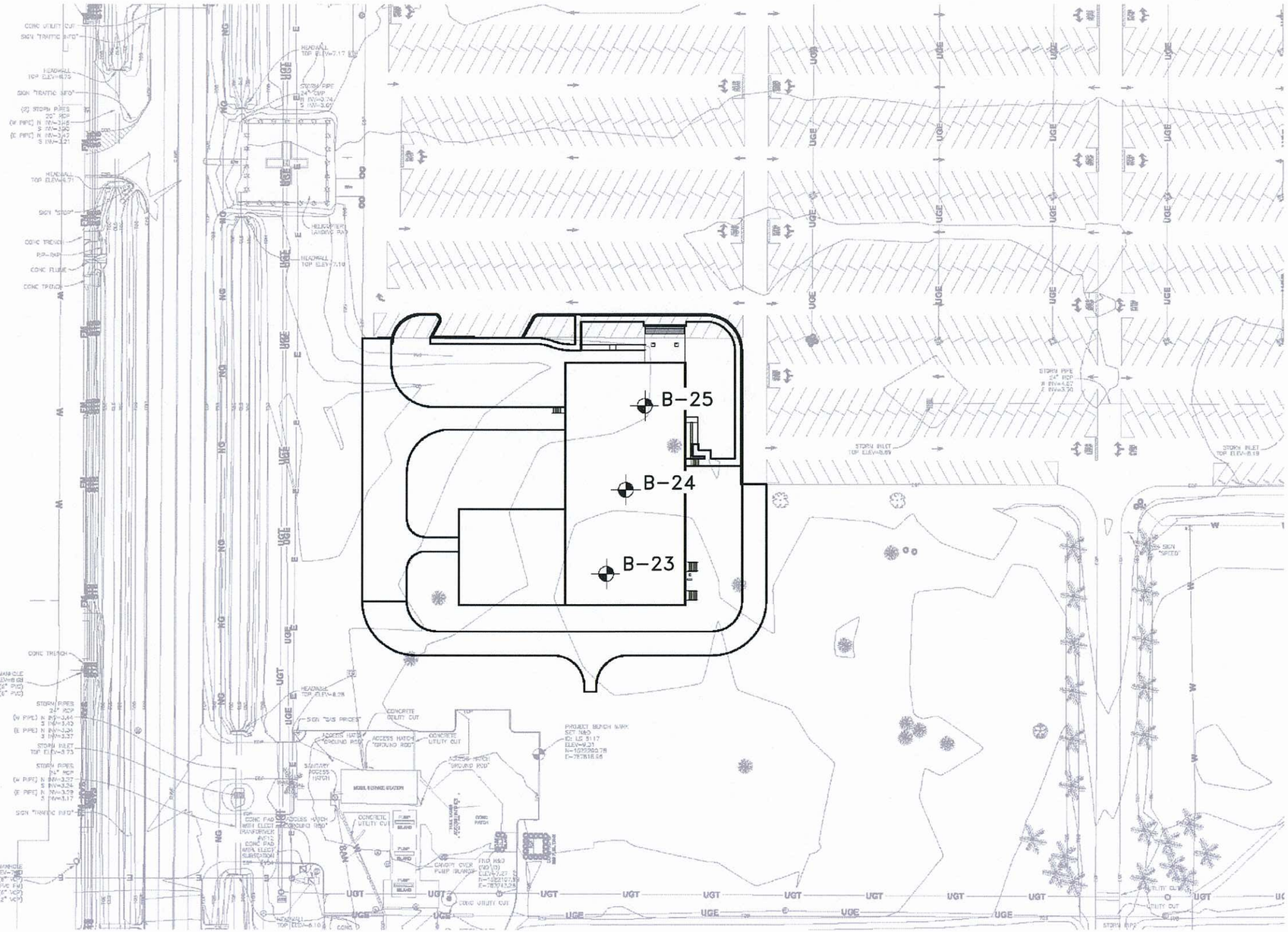
NOTE: BORING LOCATION PLAN PROVIDED BY JONES EDMUNDS

**BORING LOCATION PLAN-
CENTRAL CAMPUS**

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

SUBSURFACE SOIL EXPLORATION
CENTRAL CAMPUS DESIGN–PHASE I
KENNEDY SPACE CENTER, FLORIDA

DRAWN BY: TT	CHECKED BY:	DATE: 11/30/12
FILE NO. 12–5260	APPROVED BY:	FIGURE: 2



LEGEND

⊕ B SOIL BORING LOCATION

NOTE: BORING LOCATION PLAN PROVIDED BY JONES EDMUNDS

**BORING LOCATION PLAN-
DATA CENTER**

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

SUBSURFACE SOIL EXPLORATION
CENTRAL CAMPUS DESIGN–PHASE I
KENNEDY SPACE CENTER, FLORIDA

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APPENDIX I

Standard Penetration Test and Auger Boring 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.

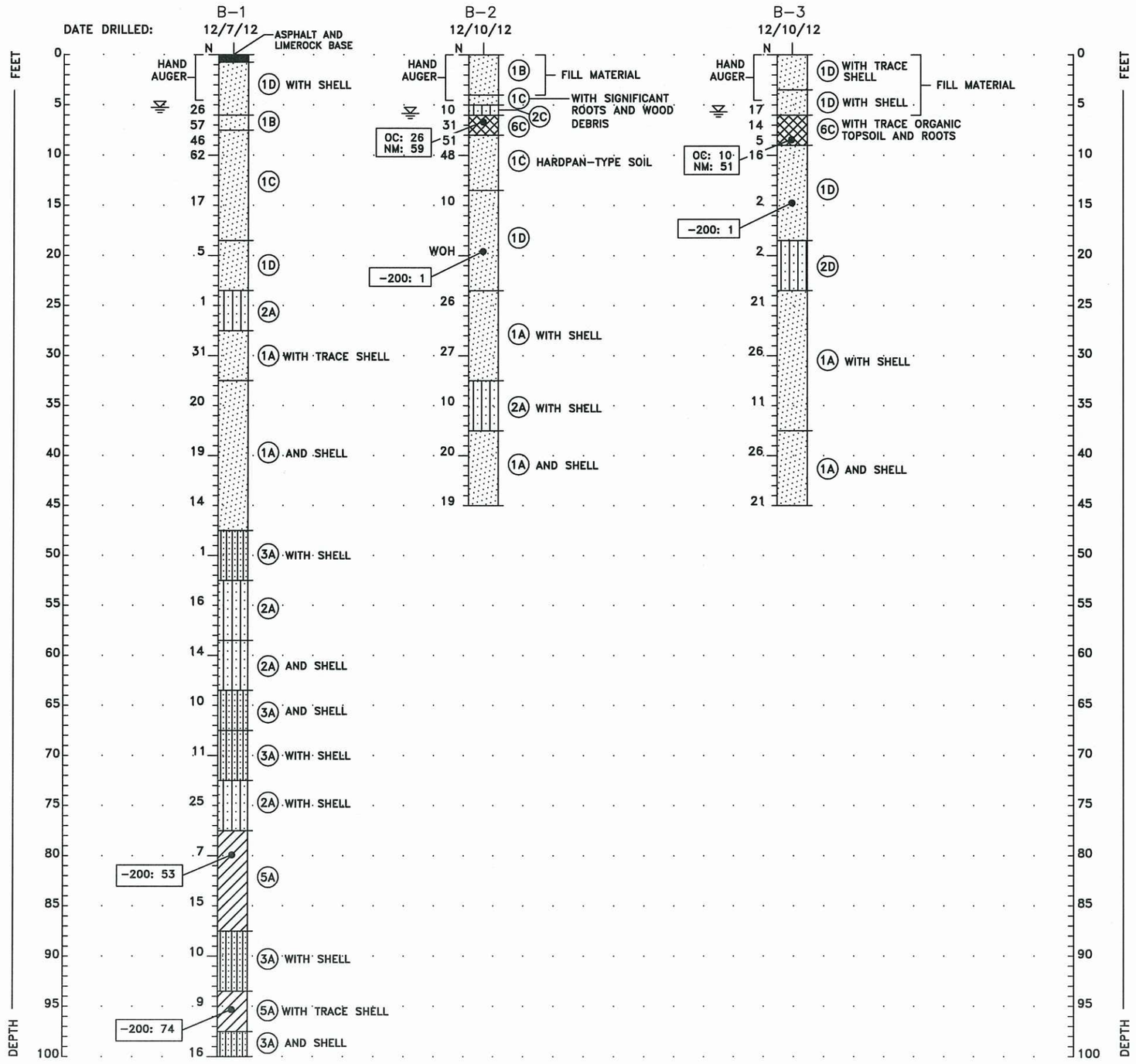
Representative split-spoon samples from the soils at every 5 feet of drilled depth and from every different stratum are brought to our laboratory in air-tight jars for further evaluation and testing, if necessary. Samples not used in testing are stored for 30 days prior to being discarded. After completion 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.

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)
5. CLAY (CL,CH)
6. MUCK/PEAT (OH,Pt)
7. SILT (ML)

COLORS

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

B	SOIL BORING
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT
WOH	SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER AND RODS ONLY
⏚	GROUNDWATER LEVEL MEASURED ON DATE DRILLED
NM	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D-2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES)(ASTM D-1140)
OC	ORGANIC CONTENT IN PERCENT (ASTM D-2974)
LL	LIQUID LIMIT IN PERCENT (ASTM D-423)
PL	PLASTIC LIMIT IN PERCENT (ASTM D-424)
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 WERE CONVERTED TO EQUIVALENT SAFETY HAMMER N-VALUES BY MULTIPLYING BY 1.24. ALL REPORTED N-VAULES ARE SAFETY HAMMER OR EQUIVALENT.

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.

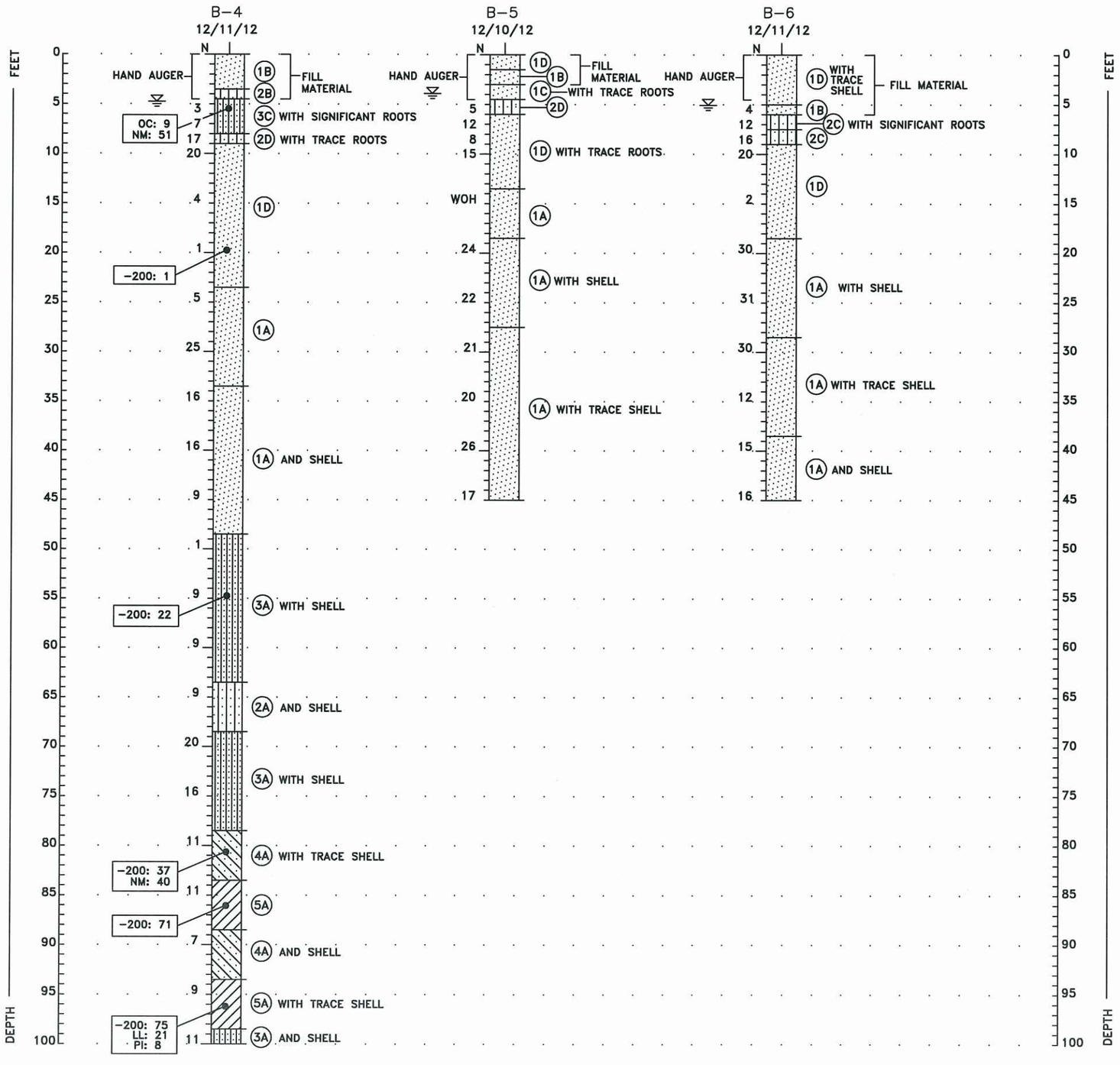
GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR.

SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
CENTRAL CAMPUS DESIGN-PHASE I
KENNEDY SPACE CENTER, FLORIDA

DRAWN BY: TT	CHECKED BY:	DATE: 1/11/12	
FILE NO. 12-5260	APPROVED BY:	FIGURE:	



LEGEND

SOIL DESCRIPTIONS

1. FINE SAND (SP)
2. FINE SAND WITH SILT (SP-SM)
3. SILTY FINE SAND (SM)
4. CLAYEY FINE SAND (SC)
5. CLAY (CL,CH)
6. MUCK/PEAT (OH,Pt)
7. SILT (ML)

COLORS

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

B	SOIL BORING
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT
WOH	SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER AND RODS ONLY
▽	GROUNDWATER LEVEL MEASURED ON DATE DRILLED
NM	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D-2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES)(ASTM D-1140)
OC	ORGANIC CONTENT IN PERCENT (ASTM D-2974)
LL	LIQUID LIMIT IN PERCENT (ASTM D-423)
PL	PLASTIC LIMIT IN PERCENT (ASTM D-424)
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 WERE CONVERTED TO EQUIVALENT SAFETY HAMMER N-VALUES BY MULTIPLYING BY 1.24. ALL REPORTED N-VAULES ARE SAFETY HAMMER OR EQUIVALENT.

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.

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CENTRAL CAMPUS DESIGN-PHASE I
KENNEDY SPACE CENTER, FLORIDA

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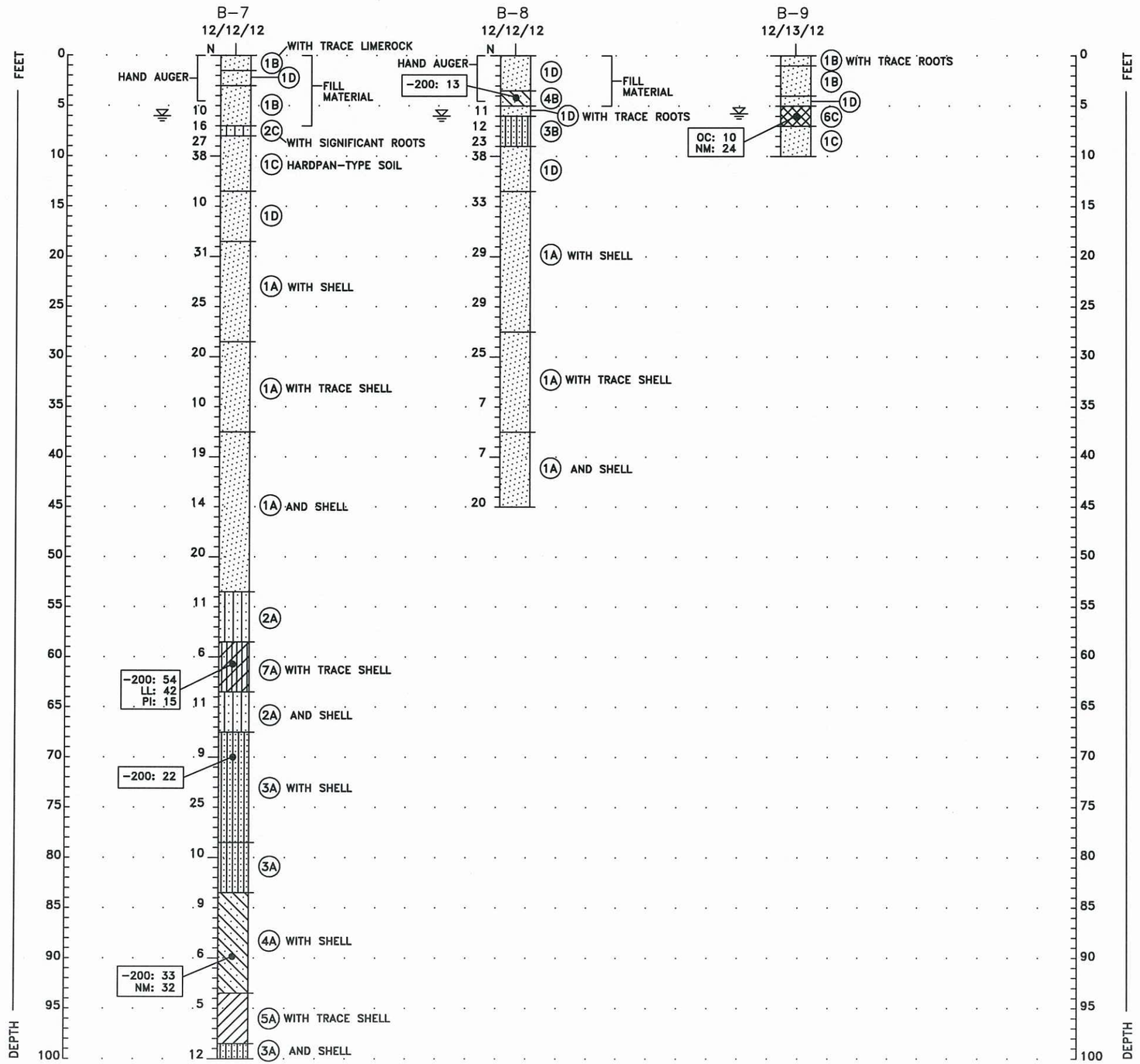

LEGEND

SOIL DESCRIPTIONS

1. FINE SAND (SP)
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3. SILTY FINE SAND (SM)
4. CLAYEY FINE SAND (SC)
5. CLAY (CL,CH)
6. MUCK/PEAT (OH,Pt)
7. SILT (ML)

COLORS

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

B	SOIL BORING
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT
WOH	SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER AND RODS ONLY
▽	GROUNDWATER LEVEL MEASURED ON DATE DRILLED
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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.

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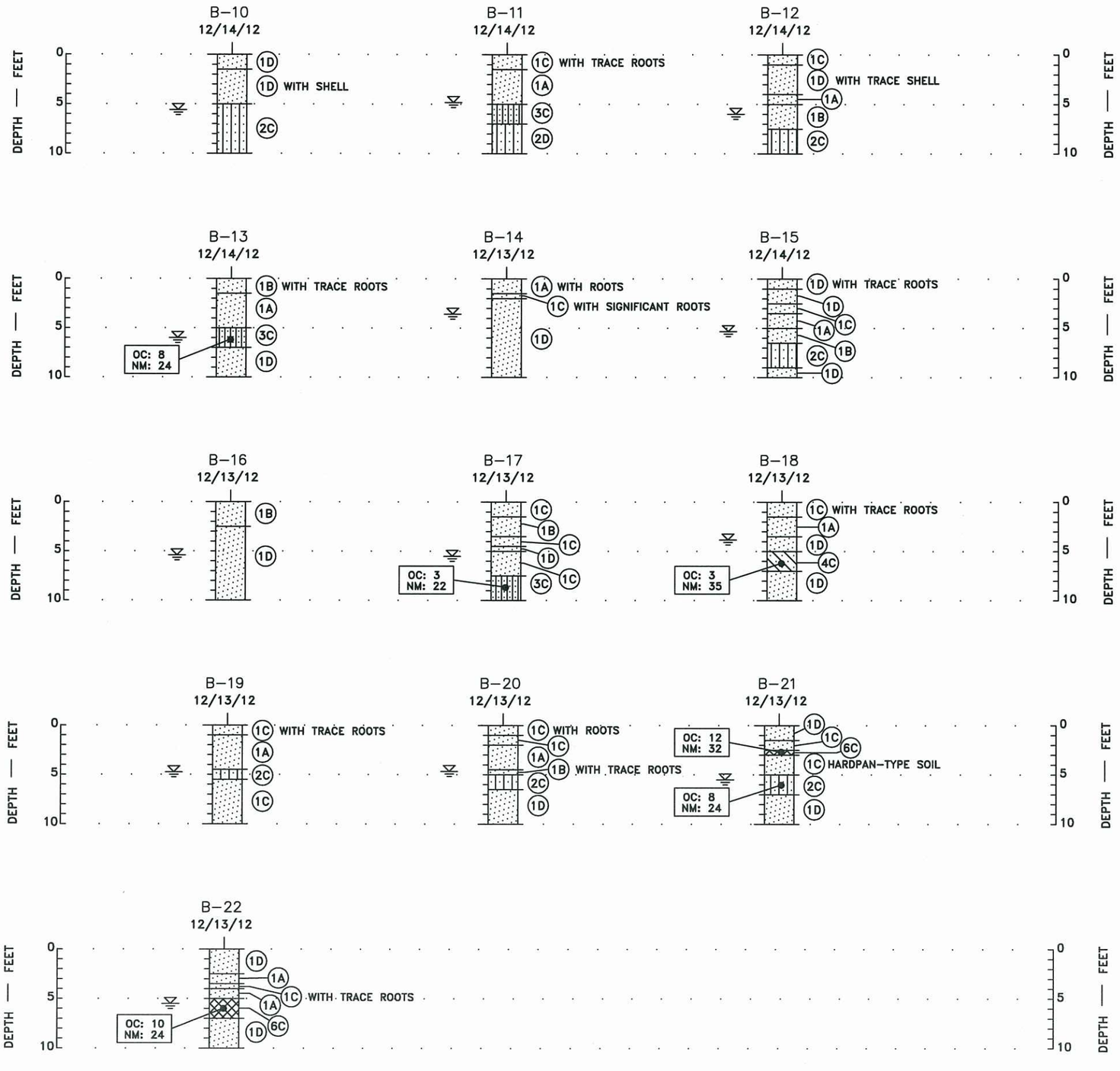

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ENGINEERING CLASSIFICATION

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

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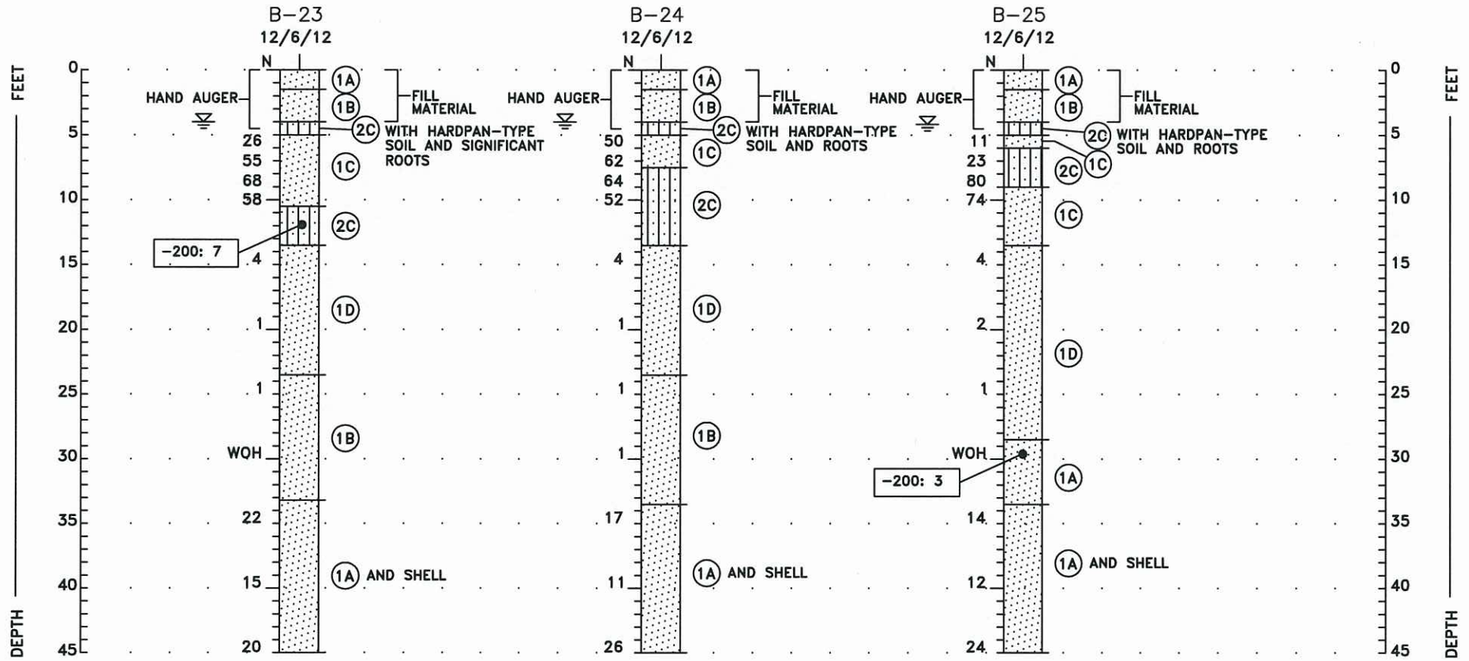

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DENSE	30 TO 50
VERY DENSE	>50

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CENTRAL CAMPUS DESIGN-PHASE I
KENNEDY SPACE CENTER, FLORIDA

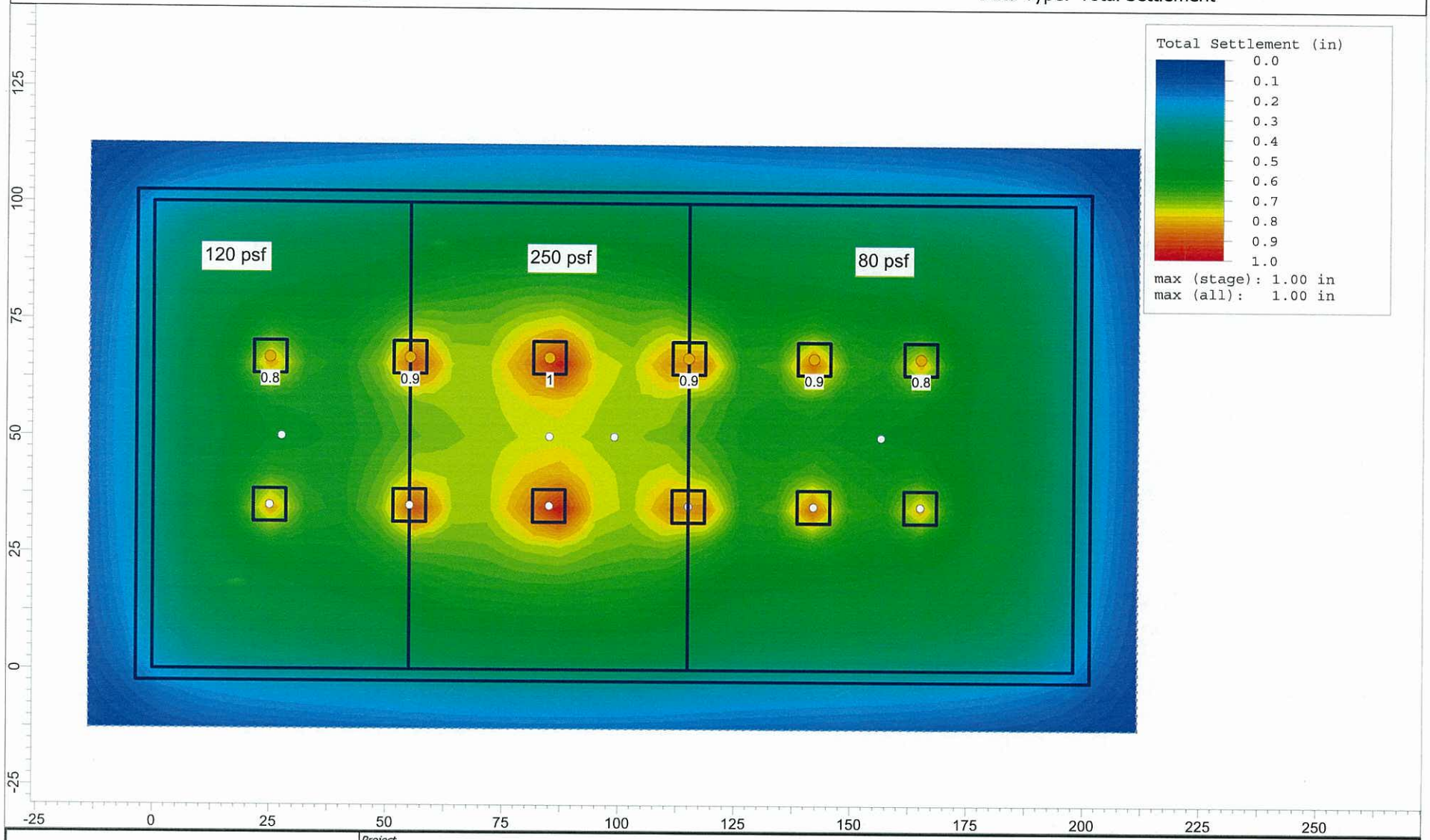
DRAWN BY: TT	CHECKED BY:	DATE: 12/18/12
FILE NO. 12-5260	APPROVED BY:	FIGURE:

APPENDIX III

Settle 3D Output (for Spread Footings Analysis)

Stage 1

Data Type: Total Settlement



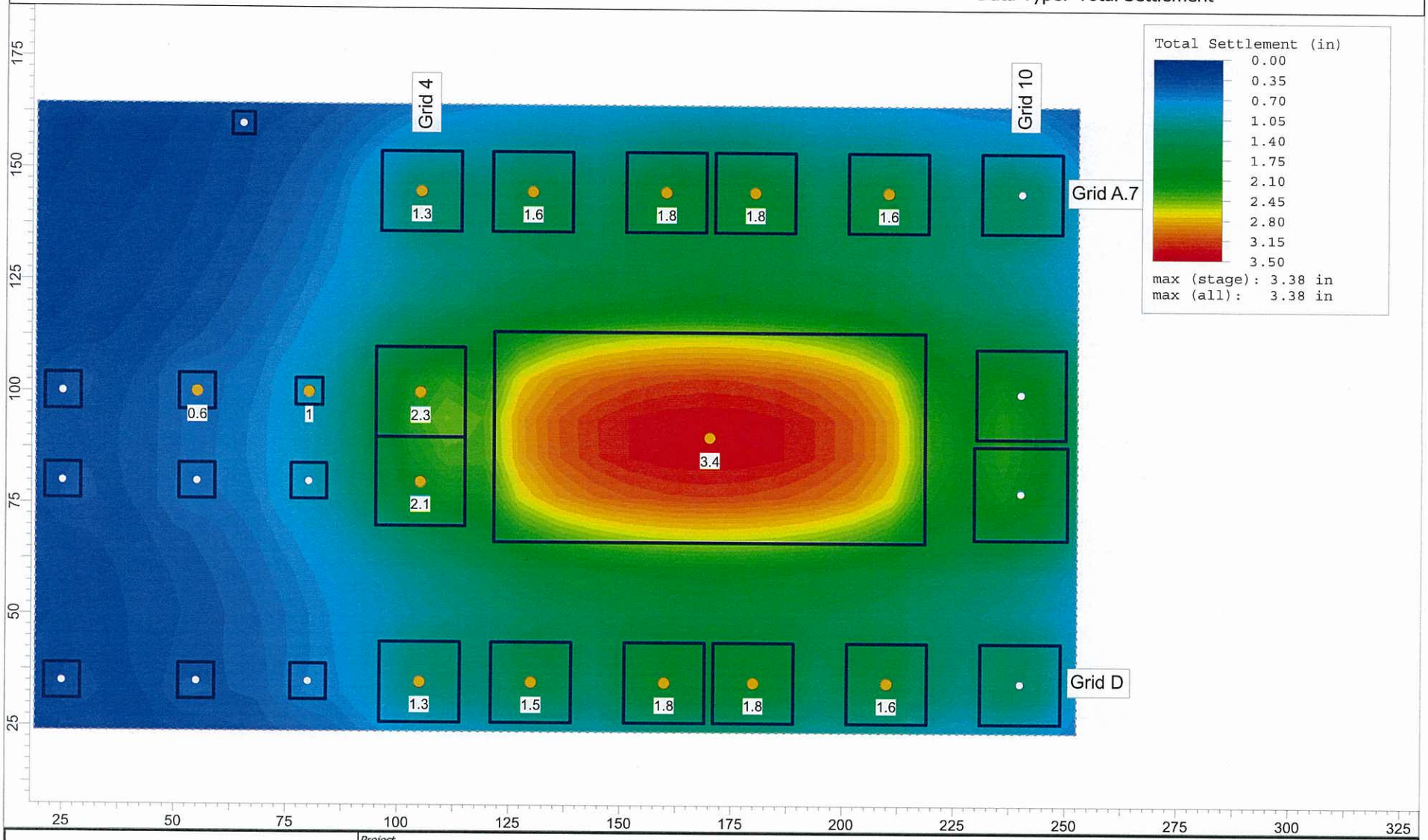
50 Years Ardaman & Associates, Inc.

SETTLE3D 2.016

Project	KSC Central Campus Design - Phase I		
Analysis Description	KDC Building with 5 Feet of Fill		
Drawn By	Jason P. Manning, P.E.	Company	Ardaman & Associates, Inc.
Date	12/20/2012, 4:56:14 PM	File Name	KDC Rev1.s3z

Stage 1

Data Type: Total Settlement



50 Years Ardaman & Associates, Inc.	Project	KSC Central Campus Design - Phase I	
	Analysis Description	7-Story Headquaters Building	
	Drawn By	Jason P. Manning, P.E.	Company: Ardaman & Associates, Inc.
	Date	12/20/2012, 3:00:22 PM	File Name: Nasa Rev 1.s3z

SETTLE3D 2.016