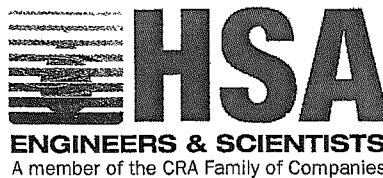


Report of
Subsurface Exploration and Geotechnical Engineering
Services to Support Civil and Foundation Design
Apollo Beach Condominium
Hillsborough County, Florida 33572
HSA Project No. 502-0376-00



June 20, 2006

Apollo Beach Property Holdings, LLC
C/o Andrews Asset Management, Inc.
7402 N. 56th Street
Suite 480
Tampa, FL 33617

Via email and Regular Mail

Attention: *Ed Andrews*

Subject: Report of Subsurface Exploration
and Geotechnical Engineering Services
to Support Civil and Foundation Design
Apollo Beach Condominium
Hillsborough County, Florida 33572
HSA Project No. 502-0376-00

Dear Mr. Andrews:

Pursuant to your request, **HSA Engineers & Scientists (HSA)** has completed the Geotechnical Engineering Services to support the civil engineering and foundation design for the condominium proposed for the former Ramada Inn property in Apollo Beach, Hillsborough County, Florida. This report provides the results of the field studies and discusses geotechnical considerations for the planned redevelopment. Included are recommendations for foundation, stormwater drainage and pavement design, as well as associated subgrade preparation for the structure and pavement areas.

SUMMARY

The geotechnical study revealed suitable subsurface conditions that will adequately support the anticipated four-story structure above parking. Generally, sandy soils with variable silt and shell content were encountered to a depth of about 50 feet. Sands with clayey fines occurred beneath followed by hard calcareous strata around the boring completion depth at about 80 to 85 feet. The borings indicated a relatively shallow, tidal, surficial water table within a depth of about 5 to 6 feet of the existing ground surface. Routine site preparation should include clearing, stripping,

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grubbing and proof rolling prior to filling to develop design grades. The anticipated loading conditions of 25 kips per lineal foot for wall loads and 300 kips for column loads were utilized in the evaluations of shallow as well as deep foundation alternatives for this project.

Please refer to the attached report for a more detailed description of the methods and results of our geotechnical exploration, as well as more detailed conclusions and other important details not included in the summary. The report should be read in its entirety to obtain a more complete understanding of the information provided, and to aid in any decisions made, or actions taken, based on this information.

PROJECT INFORMATION

Site Location and Project Description

The project site is located at 6502 Surfside Boulevard, at the intersection of Apollo Beach Boulevard and Marbella Boulevard, and borders Tampa Bay, within Section 17, Township 31 South and Range 19 East in Apollo Beach, Hillsborough County, Florida. The approximate location of the site is shown on **Figure 1**.

In 2004, **HSA** had conducted a preliminary geotechnical evaluation of the site and submitted a report to Kendar Corporation (HSA project number 50-15-3221-00). The report described deep foundation (pile) alternatives for consideration and presented preliminary recommendations for general site and pavement subgrade preparation and design.

In November 2005, **HSA** submitted a proposal to Apollo Beach Property Holdings, LLC, to provide geotechnical engineering services for the site (HSA proposal number 501-6786-98). The proposal included geotechnical services to provide information for foundation design of the buildings, retention area design criteria and to check shallow soil types for pavement and utility construction and design.



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Recent project geotechnical services carried out are based on our understanding of the anticipated development from our meeting with Mr. Ron Weisser, Mr. Ed. Andrews and Mr. James A. Scarola, P.E. on March 20, 2006.

Review of the previous boring data indicates a dense sand layer from about 30 or 35 to 45 feet deep that could be used to support moderate capacity piles. A very stiff to hard clay was also penetrated near the preliminary boring termination depth of 70 feet that, if sufficiently thick, could offer increased pile capacity. Accordingly, **HSA** submitted a new proposal (HSA proposal number 502-0376-98) and was proposed that the borings for foundation design for this study extend to around 90 feet deep.

Authorization to proceed with this project was received from Mr. Andrews on April 7, 2006, after accepting **HSA's** proposal No. 502-0376-98 (dated April 5, 2006). Correspondence and project information was also coordinated through Mr. Andrews.

The property formerly housed a Ramada Inn, which has been razed. As we understand, the site is located just outside a flood velocity zone and the proposed structure will be 4 story condominiums above garage parking.

The preliminary structural aspect of the building, based on the conceptual plan, was furnished by O'Donnell, Naccarato, Mignogna & Jackson, Inc. (ONM&J). The structural systems under consideration were the Filigree system, that imposes wall loads of 25 kips/ft and a column load of 300kips, and the PT system that has column load of 700 kips.

Digital Raster Graphics (scanned topographic maps) projections of Apollo beach, Florida provided by the USGS were reviewed for topographic information in the vicinity of the project sites. According to this map, the ground surface elevation of the site may have occurred around El. 0 feet based on the National Geodetic Vertical Datum (NGVD) of 1929; however, the property was raised 5 feet above sea level as part of previous construction.



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PURPOSE AND SCOPE OF SERVICES

The purpose of our study was to evaluate general subsurface soil and groundwater conditions within the depth of significant load influence for the anticipated building type and to assess pavement subgrade conditions based on borings spaced throughout each site. The subsurface materials encountered were reviewed with respect to the available project characteristics. For this geotechnical evaluation, engineering assessments of the following items have been formed:

1. Feasibility of utilizing shallow and deep foundation systems for support of the proposed structures. General design parameters required for the structure foundation system including, for shallow foundations (footings), allowable soil bearing pressure and minimum footing depths, types, dimensions and, for deep foundations, an estimated installed depth of piles, predicted capacity, and alternative types;
2. General design parameters required for the proposed stormwater management areas;
3. General pavement design considerations and section options;
4. Geotechnical impacts and construction considerations based on the information gathered during this study and the proposed construction;
5. Anticipated soil subgrade preparation, including stripping, grubbing and compaction. Engineering criteria for placement and compaction of approved structure and pavement embankment fill materials.

The following services have been provided in order to achieve the preceding objectives:

1. Conducted a reconnaissance to observe the existing site conditions;
2. Reviewed readily available published shallow soils information obtained from the *Soil Survey of Hillsborough County, Florida* published by the United States Department of Agriculture (USDA) Soil Conservation Service (now the Natural Resources Conservation Service);



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3. Selected boring locations and, based on scaled distances from a map, positioned and staked the boring locations in the field using a tape to measure distances and a compass to determine directions;
4. Conducted a total of ten (10) hand auger borings throughout the sites to a depth of up to 5 feet to check shallow soil conditions;
5. Performed four (4) Standard Penetration Test (SPT) borings to a nominal depth of between 80 to 90 feet within the proposed building areas, and one (1) SPT boring to a nominal depth of 30 feet within the proposed stormwater management area;
6. Performed one (1) Double Ring Infiltrometer (DRI) test within the proposed stormwater management area within a depth of 2 feet below the ground surface
7. Visually classified representative soil samples in the laboratory using the Unified Soil Classification System in general accordance with ASTM D2487/2488;
8. Measured and documented stabilized groundwater levels in the borings;
9. Identified soil conditions at each boring location and formed an opinion of the site soil stratigraphy. Provided a soil profile for each boring location.

The results of the exploration have been used in the geotechnical engineering analysis and the formulation of recommendations. The results of the subsurface exploration, including the recommendations and the data on which they are based, are presented in this written report prepared under the direction of a Florida licensed engineer specializing in geotechnical engineering who is familiar with the local soil conditions.



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SUBSURFACE EXPLORATION

Soil Borings

The subsurface exploration program for this study included ten (10) hand auger borings and five (5) Standard Penetration Test (SPT) borings as shown on the boring location map, **Figure 2**. Test boring locations from the previous study are also plotted on the **Figure 2**. Our personnel selected the boring locations by reference to a conceptual plan prepared by Scarola Associates. The boring locations were then laid out in the field based on tape measurements and compass directions from features shown on the map and identified in the field. Accordingly, the boring locations should be considered approximate.

The hand auger borings were advanced to a nominal depth of between 1-1/2 feet to 5 feet below the ground surface. The hand auger borings were performed in general accordance with ASTM D 1452 (Standard Practice for Soil Investigation and Sampling by Auger Borings). The hand auger borings were conducted by manually pushing and rotating a bucket auger into the ground in approximately 6-inch increments. As each soil type was penetrated, the technician/geologist recorded its depth interval and secured a representative sample for laboratory review and possible testing.

The SPT borings extended to a nominal depth of between 80 to 90 feet and were drilled using a truck-mounted drill rig. The SPT borings were conducted in general accordance with ASTM D 1586 (Standard Test Method for Penetration Test and Split Barrel Sampling of Soils) using the rotary wash method, where a clay slurry ("drill mud" or "drill fluid") was used to flush and stabilize the borehole. Standard Penetration Test sampling was performed at closely spaced intervals in the upper 10 feet and at 5-foot intervals thereafter. After seating the sampler 6 inches into the bottom of the borehole, the number of blows required to drive the sampler one foot further with a 140 pound hammer dropped 30 inches is known as the "N" value or blowcount. The blowcount has been empirically correlated to soil properties. The recovered samples were placed into containers and returned to our office for visual classification.



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Double Ring Infiltrometer Test

One (1) Double Ring Infiltrometer test (DRI-1), was performed in general accordance with the procedure outlined in ASTM D-3385. The test utilized a 24-inch outer ring and 12-inch inner ring with a 6-inch head of water, and was conducted at a depth of about 2 feet below the ground surface. The test was continued until stabilized readings were recorded within the inner ring. Water was periodically added to maintain the 6-inch head in the inner ring and annular space. The test was carried out on the May 1, 2006. The test could not conduct at the originally planned location, near boring SPT-5, due to the presence of buried concrete at shallow depth. Accordingly, the test was moved to new position and carried out near hand auger boring AB-3 location.

SUBSURFACE CONDITIONS

Soil Conservation Service Data

The U.S. Department of Agriculture - Soil Conservation Service, now known as Natural Resources Conservation Service, has mapped the shallow soils in this area of Hillsborough County. This information was outlined in a report titled *The Soil Survey of Hillsborough County, Florida* dated May 1989. The aerial photographs used in the mapping were prepared in 1982.

The USDA Soil Survey is not necessarily an exact representation of the soils on the site. The mapping is based on interpretation of aerial maps with scattered shallow borings for confirmation. The transition between different soil types may be gradual and the indicated boundary approximate. Differences may also occur from the typical stratigraphy and small areas of other similar and dissimilar soils may occur within the mapping unit. As such, there may be differences in the mapped description and the boring descriptions obtained for this report. The survey is, however, a good basis for evaluating the shallow soil conditions of the area.

The soil survey indicates the site is covered by St. Augustine fine sand (mapping unit 44). A soil mapping unit is an area predominated by a particular soil type. St. Augustine fine sand is nearly level and somewhat poorly drained. It occurs on flats and ridges bordering Tampa Bay. It is subject to flooding for very brief periods during hurricanes.



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Typically, this soil has a surface layer of dark gray fine sand about 3 inches thick. The upper part of the underlying material, to a depth of about 12 inches, is light brownish gray fine sand. The middle part, to a depth of about 30 inches, is light gray, mottled fine sand containing balls of sandy clay. The lower part to a depth of about 80 inches is gray fine sand. Similar soils included in mapping, in some areas, have a surface layer of sandy loam or loamy sand. Other similar soils, in some places, have an underlying material that consists of stratified lenses of sandy clay loam, clay loam, or loamy sand.

In most years, the seasonal high water table is at a depth of 20 to 30 inches. Permeability is moderately rapid or rapid. The available water capacity is low. If this soil is used for building site development, the main management concerns are excessive wetness and instability of cutbanks. Population growth has resulted in increased construction of houses on this soil. In most areas, this soil is artificially drained by surface drains and ditches. Cutbanks are not stable and are subject to slumping.

Soil Boring Results

Soil profiles of the hand auger boring and SPT test boring results are illustrated on **Figures 3 and 4**. Previous soil boring results are included in **Appendix A**. The stratification information was developed from the field boring logs, visual review/manual classification of the recovered soil samples in general accordance with ASTM D2487/2488. The stratification lines represent the boundary between soil types at the boring locations. The transition between strata may be gradual and the indicated boundary approximate. Soil strata boundaries were estimated when they occurred between sample intervals. Small variations not considered important to our engineering evaluation may have been omitted or abbreviated for clarity. The boring logs include the SPT "N" values (SPT logs) and soil descriptions. Following are the generalized subsurface conditions encountered in the borings. Please refer to the soil boring profiles for additional information not included in this discussion.



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Briefly, the borings indicate that the site is covered by about 8 feet to 17 feet of light gray, light brown to brown, slightly silty fine sand to fine sand (Stratum 1). Trace amount of some shells were evident within Stratum 1 sands. All the hand auger borings terminated in Stratum 1 at nominal depth of 5 feet except for hand auger locations AB-3, AB-6 and AB-8, which terminated at 4 feet, 3-1/2 feet and 1-1/2 feet, respectively, due to the presence of shallow buried objects, apparently concrete.

Below Stratum 1, an approximately 8 to 10 foot thick layer of dark olive brown to olive brown, silty fine sand with some shells (Stratum 2) was intercepted by the borings. Due to the presence of some loose zones in the Stratum 2 sands at boring location SPT-3, casing was set to the depth of 20 feet to prevent the borehole from caving-in.

Next, some borings penetrated light gray to olive gray, slightly silty fine sand to fine sand (Stratum 3) with interbedded lenses of dark olive brown silty, slightly clayey fine sand (Stratum 4) continued to depths of about 47 to 67 feet below grade. Boring SPT-5, located in the storm water retention area, terminated in Stratum 3 at a nominal depth of 30 feet.

Next, three (3) of the remaining four (4) SPT borings penetrated bluish gray to light gray, clayey to very clayey fine sand (Stratum 5) that extended to depths of 67 to 72 feet below the existing ground surface. Stratum 5 contained trace amount of calcareous rock fragments. Stratum 5 was absent at boring location SPT-3. Next, about 10 to 15 feet of bluish green slightly to partially-indurated sandy clay (Stratum 6) was penetrated in the four (4) deeper SPT borings. Boring SPT-2 was completed in Stratum 6 at a nominal depth of 82 feet.

Below Stratum 6 in borings SPT-1, SPT-3 and SPT-4, a layer of light gray, calcareous cemented sand and/or clay with weathered sandy limestone (Stratum 7) was penetrated. These SPT borings terminated at a nominal depth of 90 feet below the existing ground surface in Stratum 7.



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Based on the SPT 'N' values or blowcounts, the relative density of the Stratum 1 soils can be deduced as generally medium dense with some loose zones, although it is noted the shell fragments may have increased the blowcounts. The Stratum 2 soils were relatively loose with some medium dense zones, the Stratum 3 soils were mostly dense and the Stratum 4 soils were generally medium dense with some loose as well as dense zones. The relative density/consistency of the Stratum 5 soils can be depicted as generally medium dense/very stiff with some loose/firm zones. The consistency of the Stratum 6 soils can be concluded as stiff at the top and becoming hard with refusal conditions as the depth increases. The penetration resistance of the Stratum 7 calcareous layer can be deduced as medium-hard to hard.

Double Ring Infiltrometer Test Result

The stabilized infiltration rate at the DRI -1 location was calculated for the inner ring at a rate of about 32 ft/day.

Groundwater Information

The measured ground water levels are presented on the soil boring profiles. Water levels were recorded in the borings after several hours of stabilization time. At the time of our exploration, mid-April 2006, groundwater was measured in the borings at depths of approximately 5 feet below the existing ground surface. The measured ground water levels are presented on the soil boring profiles. The tidal variations in the Tampa Bay will influence the ground water depth in this site.

GEOTECHNICAL EVALUATION

General

The result of the recent and previous exploratory borings revealed suitable subsurface conditions for support of the anticipated four-story structure above parking on shallow as well as deep foundation alternatives, provided proper site preparation. The borings also encountered suitable subgrade condition for support of flexible or rigid pavements without the need for underdrains to control normal groundwater levels. While the majority of the site appears relatively clear of



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surficial or buried rubble and debris, some borings encountered apparent concrete remnants (buried rubble, slabs, etc.) at depths varying from 1-1/2 feet to 4 feet. Complete removal of the buried rubble and concrete from below building footings will be required. Foundation recommendations of this report should be considered as superceding the previous report (HSA Project No. 50-15-3221-00).

Seasonal High Groundwater Estimates

Based upon the results of our borings, our review of the SCS Soil Survey and rainfall patterns, it is our opinion that the seasonal high groundwater levels at the site would normally be about 4 feet below the existing ground surface. However, heavy or prolonged rainfall or high tide conditions could also result in groundwater levels being temporarily higher than the seasonal high groundwater estimates.

Site Preparation

Existing Utilities: Underground piping to be abandoned should be removed and the excavations backfilled with compacted clean fill, as described subsequently. Improperly abandoned piping can serve as conduits for soil erosion, leading to ground subsidence and removing support of pavements and structures above.

Stripping and Grubbing: The site should be cleared of surface materials, pavements, structures, foundations, debris from the demolishing of the existing building, etc. It is recommended that following clearing, the site be checked by an engineer or his representative for satisfactory stripping and grubbing conditions. As a minimum, it is recommended that the clearing operations extend at least 5 feet beyond the development perimeters. Any excavations or cavities formed by the removal of unsuitable materials or abandoned utilities from the existing building should be filled with clean structural fill placed and compacted in lifts. Root raking the site is recommended to locate near surface debris. The use of individual shallow foundations or strip footings may require hand probing each foundation area and/or test pits to check for the presence of buried debris or other unsuitable material associated with the previous development that could impact the performance of the structure. This will necessitate survey staking the column/foundation locations for field identification. If unsuitable material is found, the material



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needs to be completely removed from the foundation area, plus a margin of at least 5 feet outside the foundation perimeter as well as possibly from the area supporting slabs-on-grade. In particular, the location of the former Ramada Inn pool will likely require overexcavation and backfilling in compacted lifts to develop suitable subgrade support.

Subgrade Proofrolling and Compaction: Following the clearing operations, the exposed subgrade should be evaluated and proof-rolled to check for the presence of near-surface unsuitable materials that will require removal and to improve the density and uniformity of the near surface soils. First, any cut areas should be brought to grade; the proof-rolling should take place on the stripped or final grade, whichever is lower. The proof-rolling should consist of compaction with a heavy vibratory drum roller with a minimum static drum weight of 10 tons. A minimum of ten (10) overlapping passes should be made by the vibratory roller over the building and pavement areas and 5 foot margin, with the successive passes aligned perpendicular. The compactor should be operated at a slow walking pace. Careful observations should be made during proof-rolling to help identify any areas of soft yielding soils that may require over excavation and replacement. The Geotechnical Engineer, based on his or his representative's observations, can recommend the nature and extent of any remedial work. Aside from stripping and backfilling activities and the utility removal previously described, no other remedial work is anticipated at this site.

After the minimum number of passes is made, it is recommended that compaction continue as needed within the structure and pavement areas (this also includes sidewalks and exterior slabs) to develop a dry density of at least 95% of the modified Proctor maximum dry density (ASTM D-1557) to a minimum depth of 1 foot below stripped or cut grade, whichever is lower. Test frequencies of one test per 2,500 square feet of structural area proof rolled, and one test per 5,000 square feet of pavement area proof rolled, are recommended.



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Care should be utilized when operating the compactor near existing structures, use of a smaller compactor or static rolling may be needed within about 75 feet of existing buildings. It should be noted that vibrations from the compaction equipment may be felt in nearby structures at levels below those that may cause damage. Accordingly, the contractor is advised to observe and document any distress on nearby structures prior to beginning construction. Vibration monitoring should also be considered.

Fill Placement

After the site has been compacted and accepted by the Geotechnical Engineer, fill required to bring the site to final grade may be placed and properly compacted. Fill should be inorganic, non-plastic granular soil (clean to slightly silty sands). The fill should be placed in level lifts not to exceed 12 inches loose thickness. Each fill lift should be compacted to a minimum of 95% of the soil's modified Proctor maximum dry density as determined by ASTM Specification D-1557. In-place density tests should be performed on each lift by an experienced engineering technician working under the direction of a licensed geotechnical engineer to verify that the recommended degree of compaction has been achieved. Each lift should be tested and accepted prior to placing the next lift. We suggest the following minimum testing frequency per 12 inch layer of fill placed: one test per 2500 square feet of building area and one test per 5000 square feet of pavement area. Fill used to elevate the building and pavement areas should extend a minimum of 5 feet beyond the structure or pavement limits to control possible erosion or undermining of footing bearing soils and to provide lateral support. Furthermore, fill slopes should not exceed 2 horizontal to 1 vertical. All fill placed in utility line trenches and adjacent to footings beneath slabs-on-grade should also be properly placed and compacted to a minimum of 95% of the soil's modified Proctor maximum dry density as determined by ASTM Specification D-1557. However, in these restricted working areas, compaction should be accomplished with lightweight, hand-guided compaction equipment, and lift thicknesses should be limited to a maximum of 6 inches loose thickness. The minimum testing frequency should be one test per 6-inch fill layer per 100 feet of utility trench or building wall.



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Shallow Foundation Alternatives

The ability to use shallow foundations will be highly dependent upon final loading conditions and tolerances to potential settlement. In evaluating shallow foundations, an allowable soil bearing pressure of 3000 pounds per square foot (psf) and limiting settlement magnitudes of 3/4 to 1 inch for total settlement and 1/4 to 1/2 inch for differential settlement were utilized. If the structure can tolerate greater settlement, then other foundation and subgrade improvement options can be considered. We assumed removal of larger buried rubble and concrete slabs within residential and parking building foundation footprints. We also assumed vibratory compaction at the bottom of footing elevation or deeper (e.g., stone columns) to produce the greatest depth of improvement or densification of the shallow sandy soils. Shallow exploratory borings or test pit excavations to check for deposits of buried debris immediately beneath each foundation should be planned during site preparation activities.

Conventional Shallow Foundations - Assuming the column layout is favorable; the use of strip footings to structurally tie together the individual columns appears to be a viable option where the column loads are about 250 kips or interior and exterior wall loads are anticipated to be on the order of 20 kips per lineal foot. A continuous strip footing designed as a relatively rigid member would enhance the capability to distribute structural loads and reduce settlements. To aid in the design of the strip footings, a coefficient of subgrade reaction of 60 psi/in (pounds per square inches per inch) should be utilized. The foundations should be embedded (depth from ground surface to the bottom of foundation) a minimum of 24 inches. An allowable soil bearing pressure of 3000 psf should be achieved following compaction of the soils at the bottom of foundation elevations with a heavy vibratory roller (5 to 10 ton static drum weight).

Shallow Foundations With Ground Improvement – Individual column pads could also be utilized to support column loads up to approximately 500 kips and shear wall elements supporting loads up to 25 kips per lineal foot, provided the strength characteristics of intermediate loose sand deposits within a depth of about 30 feet are improved through vibratory compaction or stone columns (e.g., Vibro-replacement).



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Stone columns could be utilized to densify the intermediate, loose sand deposits below the depth of improvement provided by surficial, vibratory compaction. Based on the borings, the stone columns will need to be constructed below each of these foundations to a preliminary depth of 30 feet below the existing ground surface.

The stone columns are constructed through the use of vibrating probe (vibroflot) that is typically jetted to allow advancement of the probe to the required depth. A "dry" installation method is also available that minimizes the generation of water and spoil washed from the probe location; spoil and water control can be a problem in some instances. As the probe is advanced and retrieved, a durable aggregate is placed in the probe hole and subsequently compacted. This sequence of advancement, retrieval and compaction of the aggregate is continued up to the ground surface and is utilized to enhance the relative density and bearing capacity of the sandy soils within the zone of influence of the stone columns. The use of stone columns should increase the soil bearing capacity to 6000 psf, which can be used in the development of foundation sizes. Further, the vibro-replacement should improve the density of the sandy soils to produce Standard Penetration Test N-values in the range of 15 to 20 blows per foot.

The stone column construction should take place after stripping and grubbing of the site. Following the deep vibratory compaction process, the site will need to be re-leveled and compacted as described in the Subgrade Proofrolling and Compaction section of this report.

The stone columns are constructed by a specialty contractor, who along with the geotechnical staff, will participate in the final determination of depth requirements and number of stone columns needed to achieve the desired improvement.

Other ground improvement techniques, such as deep soil mixing (mixed in place soil-cement columns) or CSV (small, displacement type sand-cement columns installed by downward rotation of an auger through a container of cement-sand) may also be feasible, but do not seem as applicable as the above discussed technique.



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Deep Foundation Alternatives

The borings typically encountered the dense fine sand at about 30 to 45 feet deep (Stratum 3), underlain by loose/firm to medium-dense/very stiff sandy and clayey soils (Strata 4 and 5) to a depth of about 70 feet and followed by hard layers of clay and calcareous materials (Strata 5 and 6). Piles could achieve moderate capacity in the dense sands indicated at about 30 to 45 feet deep and higher capacity in the deeper hard soils. Accordingly, deep foundation alternatives are listed below.

Driven Pile Foundations – Square-shaped, pre-cast concrete, nominal 14-inch and 18-inch, driven pile foundations could be considered as a foundation support alternative; short, moderately loaded piling should be cost competitive with deep vibratory compaction. The following capacities for an individual pile are recommended, based on an embedded depth range of 30 to 35 and 75 to 85 feet.

Size (inches)	Driven Depth Below Existing Grade (Ft)	Allowable Capacity in Compression (Kips)	Allowable Uplift (Kips)
14	30-35	80	20
14	75-85	140	45
18	30-35	100	22
18	75-85	200	60

Some variation in pile lengths and capacity will due to the variations in the dense layer elevation and density between the SPT boring locations; we select somewhat conservative values for preliminary design. Steel pipe or H-piles are generally not recommended due to the potential for significant corrosion in this saltwater environment, as well as the possibility of excessive penetration/reduced end bearing of the thin section.

Quality assurance and testing should also be planned during the pile installation. A load test program should be used to confirm the design capacities of the driven piling and the capability of the contractor's installation equipment to install the piling as specified. In this regard, a series of pilot piles should be driven in the building area prior to production driving. The pilot piles will



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help establish production pile lengths and check the driving system. The pilot piles should be instrumented and tested with a Pile Driving Analyzer (PDA). The PDA is a dynamic pile load test that allows the estimation of pile capacity with depth, driving stresses within the piling (to check for potential compression or tension stress damage) and hammer efficiency. Although not as accurate, this type of pile load test allows for multiple tests per day or building at a reduced cost as compared to a static load test, which only typically tests one pile over several days. Without a static load test, a higher factor of safety should be used in design; adding a static load test may lead to increased pile capacity, but the delay and cost should be weighed against possible savings in pile installation.

The driven piling should also be installed under the continuous monitoring of a qualified geotechnician or geotechnical engineer. The purpose of the observations will be to monitor the installation depths and driving resistance for conformance to the criteria established during the pilot pile program.

A primary disadvantage with driven displacement piling is ground vibrations produced during driving, which could, at least, annoy people nearby. In some cases, this has caused owners of nearby structures to inspect them for distress, and to allege that any distress discovered was caused by the pile driving. Accordingly, vibration monitoring during pile installation and distress surveys of nearby structures prior to and after pile installation should be considered. Furthermore, the pile driving operations are relatively noisy and may be objectionable.

Auger-Cast Piling – Auger-cast piles can essentially be installed with negligible vibration, but require an experienced contractor to avoid installation defects and may be limited in tension or lateral capacity by the reinforcement that can be introduced into the pile top. Further, spoils can be generated by the process. The auger-cast piles are installed utilizing a continuous flight auger or drilling tool that is advanced to the required penetration depth, depending upon the desired capacity in tension or compression. Upon achieving the design depth, the auger or tool is withdrawn slowly as the grout is injected in the hollow auger stem under pressure, thus producing a cast-in-place pile.



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Based on our geotechnical studies, the following capacities for an individual pile are recommended, based on an embedded depth range of 30 to 35 and 75 to 85 feet, for nominal 16 inch diameter and 18-inch diameter, auger-cast piles.

Size (inches)	Install Depth Below Existing Grade (Ft)	Allowable Capacity in Compression (Kips)	Allowable Uplift (Kips)*
16	30-35	60	30
16	75-85	110	50
18	30-35	70	40
18	75-85	200	70

*Also dependent upon suitable pile reinforcement

An alternative method for cast-in-place piling, particularly where increased capacity is needed for compression or tension loading, is drilled (screwed) displacement piling. The drilled displacement pile system utilizes a drilling tool that displaces soil into the sides of the borehole rather than removing it, and as a result, the displaced soils densify the surrounding soils. This increases the capacity of the pile by approximately 25% over the traditional auger-cast-in-place pile. The drilled displacement system also has the advantage of having minimal drilling spoils. This system is ideally suited to homogenous granular soils of up to a medium dense state of compaction, but may not be able to penetrate the sands encountered around a depth of 35 to 35 feet unless pre-drilled, and thus would be more favorable for the shorter pile alternative.

It should be noted that the intermediate zone of very loose to loose sandy soils (Stratum 2) that could impact grout intakes.

As previously mentioned, these types of piles can also withstand tension or uplift loads when appropriately reinforced. Tension piles typically include limited reinforcement, which usually consists of a single reinforcing rod in the center of the pile. It is also feasible to penetrate a reinforcement cage into the upper portion of the pile in order to withstand some nominal shear forces and bending moments. All reinforcement needs to be installed in the wet grout immediately following withdrawal of the continuous flight augers, and should be centered with centralizers.



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Quality assurance testing is also warranted to confirm the design capacity and ability of the contractor's installation equipment. In this regard, it is recommended that profile piles be augured throughout the building limits with the intent of assessing the general uniformity of subsurface conditions and to evaluate the ability of the equipment to properly install the piles to the required penetration and depth. Based on the results of the profile piles, at least two (2) piles location should be selected for compression load testing. Depending upon anticipated uplift loads, a second location may be required for a tension load test. The test piles should be tested to a minimum of twice the design load in accordance with ASTM D-1143 (compression) and ASTM D-3689 (tension).

The auger-cast piles should be installed under the continuous monitoring of a qualified geotechnical engineer or geotechnician. The purpose of the monitoring will be to check the drilling depths and drilling resistance to confirm that the required auger-cast pile penetration is being achieved, and to monitor grout pressures and volumes. Variations observed in drilling consistencies may warrant modification in auger-cast penetration requirements by the geotechnical engineer. In addition, grout samples must also be secured on a daily basis to check the compressive strength of the grout for comparison to specification requirements. The auger-cast piling is highly dependent upon construction techniques; therefore, for higher loaded piles and/or those at critical locations such as shear walls, pile integrity testing should be considered.

Vibrations

It should be noted that vibrations from construction equipment and compaction equipment may be felt in nearby structures. Although peak particle velocities of vibrations from the construction equipment will generally be below the threshold of damage to structures, these vibrations can be perceived by individuals and may prompt complaints from those individuals. Accordingly, the contractor is advised to observe and document interior and exterior distress in nearby structures prior to beginning construction. Monitoring vibrations during construction activities with portable seismographs is also a suggested means of mitigating the potential for vibration damage disputes.



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Consideration of Impact of Construction Upon Existing Concrete Seawall

As mentioned above, care must be taken when vibrations from construction, in particular from driven piles, which could cause some distress to the existing concrete seawall. However, shallow foundation alternatives could impart more distress on the seawall than driving piles. The total force per unit length caused by the strip loading to the seawall is calculated as 1.2 kips/ft near the pool area and 0.4 kips/ft near the beach area, assuming the structure was built on a 10 ft wide strip footing with using the footing pressure of 3 ksf. Once the preliminary foundation design is complete, we could provide further input into this issue.

Pavement Design Guidelines

It is anticipated that an asphaltic concrete pavement will be utilized for the delivery/loading area and that the parking garage floor will incorporate concrete paving. In general, the natural shallow sandy soils encountered should be acceptable for construction and support of flexible (limerock, crushed concrete, or shell base with and asphaltic concrete surface), semi-flexible (soil cement base) and rigid (Portland cement concrete) type pavement sections for the delivery/loading area after nominal subgrade preparation. If fill soils are utilized to develop pavement grades, the fill should consist of clean to slightly silty fine sand with less than 12% passing the No. 200 sieve (Unified Soil Classification of SP to SP-SM). Experience suggests that the existing sands and recommended fill will possess a Limerock Bearing Ratio (LBR) of around 20 to 30.

Flexible Pavement: The choice of a flexible pavement base type will depend on the depth of the watertable and economic considerations related to the availability and cost of aggregate. Typically, limestone or shell is the most economical and available to the Apollo beach area, but due to the limited size of the delivery and loading area, crushed concrete could be a less expensive alternative. At a minimum, the base thickness should be no less than 6 to 8 inches and compacted to a density of no less than 98% of the modified Proctor value. Limerock, shell and processed concrete base material should meet a minimum LBR of 100 and comply with FDOT specifications for quality. Processed concrete should also be graded in accordance with FDOT Standard Specification Section 204. The groundwater level should be controlled to at least 12 inches below soil cement or crushed concrete base. Stabilization of the subgrade soils to a minimum LBR of 40 is recommended for uncemented base course materials (e.g., crushed



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concrete or limerock). Subgrade stabilization would not be required for a soil cement base. Preliminarily, a pavement section consisting of 1-1/2 inches of FDOT type S-I or S-III asphaltic concrete over 6 inches of compacted base material could be utilized for light automobile traffic. For more frequent traffic and occasional light trucks, the design could consist of 2 inches of type S-I or S-III asphaltic concrete over 8 inches of compacted base material. The current FDOT Standard Specifications and/or Hillsborough County requirements may be utilized for design and construction. Providing for heavy traffic, including emergency vehicles, may require a more substantial pavement section.

Concrete Pavement - It is anticipated that a concrete pavement will be utilized for the ground level/parking garage. Pavement areas associated with trash collector receptacles or other heavy duty vehicular loading areas may also necessitate the use of a rigid, concrete pavement. Rigid (Portland cement concrete) pavement design should follow Hillsborough County specifications for minimum compressive strength. In the absence of specific requirements, the concrete should have a minimum 28 day compressive strength of 4,500 psi. Based on our experience, a minimum thickness of 5 to 6 inches should be utilized for standard duty automobile applications and a minimum thickness of 7 inches should be utilized for moderate to heavy-duty applications. The project civil or pavement engineer should design any steel reinforcement within the concrete pavement. The subgrade should be prepared to achieve a minimum LBR of 30 to a depth of 12 inches below the concrete base elevation. The subgrade soils should be compacted to a minimum density of 98% of the modified Proctor maximum dry density. To aid in the design of the anticipated concrete pavement sections, a modulus of subgrade reaction of 120 psi/in should be utilized for the low bearing sands unless stabilization is added to increase the bearing value.

The concrete pavement should be constructed in accordance with Portland Cement Association recommendations. In particular, the concrete should be cured through use of a curing compound or by covering with wet burlap. Light traffic should be restricted from the pavement for 7 days and heavy traffic for 30 days.

Proper jointing of the pavement is critical in controlling cracking. Crack control joints should be formed into the pavement during placement. Alternatively, crack control joints may be saw cut into the concrete as soon as the new concrete can support the cutting equipment (generally 4 to 6



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hours); sufficient saw cutting equipment and personnel should be available to complete the saw cutting within 4 to 6 hours of the final concrete placement. Joints should be made to about 1/4 the thickness of the pavement. Joint spacing should not exceed 12 feet and a 1.25:1 length-to-width ratio. Construction joints should be keyed to provide load transfer between adjacent slab sections. Isolation joints should be constructed where the pavement abuts non-yielding structures (buildings, storm inlets, and curbs). All joints should be patterned to give maximum distance between the joint and flow lines, especially in inverted crown drainage schemes. Joints should be sealed with elastomeric caulking or formed with a "cold-key" type product that inhibits seepage/piping action that could lead to loss of subgrade.

On-Site Soil Suitability

The fine sand sands slightly silty fine sands (non-organic and non-organic laden portion of Stratum 1) discussed in the previous paragraphs can be categorized as SP-SM to SP, or slightly silty to relatively clean fine sands based on the Unified Soil Classification System (USCS) are most suitable for use as fill. The relatively clean, sandy soil types (SP to SP-SM) are certainly considered acceptable for more select structure and pavement subgrade, and will possess improved permeability or drainage characteristics. These fine sands should require minimal processing in order to properly place and compact. Moisture contents will probably require adjustment in order to effect maximum densification, depending upon specification requirements.

LIMITATIONS

Our professional services have been performed, our findings obtained, and our opinions prepared in accordance with generally accepted geotechnical engineering principles and practices. HSA is not responsible for the conclusions, opinions or recommendations made by others based on these data. Please note that HSA reserves the right to revise the comments and/or recommendations above as conditions change or additional information becomes available.



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The scope of this study was intended to evaluate generalized soil conditions within the planned development areas based on the available survey data. The analysis and opinions submitted in this report are based upon the data obtained from the shallow soil borings performed at the locations indicated. If any subsurface variations become evident, a re-evaluation of the opinions contained in this report will be necessary after we have had an opportunity to observe the characteristics of the conditions encountered.

The scope of our services does not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the site studied. Any statements in this report regarding odors, staining of soils, or other unusual conditions observed are strictly for the information of our client.

CLOSURE

HSA Engineers & Scientists appreciates the opportunity to be of service to you on this project. We look forward to working with you during construction as your materials testing laboratory. If you have any questions, please do not hesitate to contact us.

Sincerely,

HSA Engineers & Scientists

Engineering Business No. 00007098

A handwritten signature in black ink, reading "Andrew T. Sway".

Andrew T. Sway, PhD, E.I.
Engineer
Project Manager

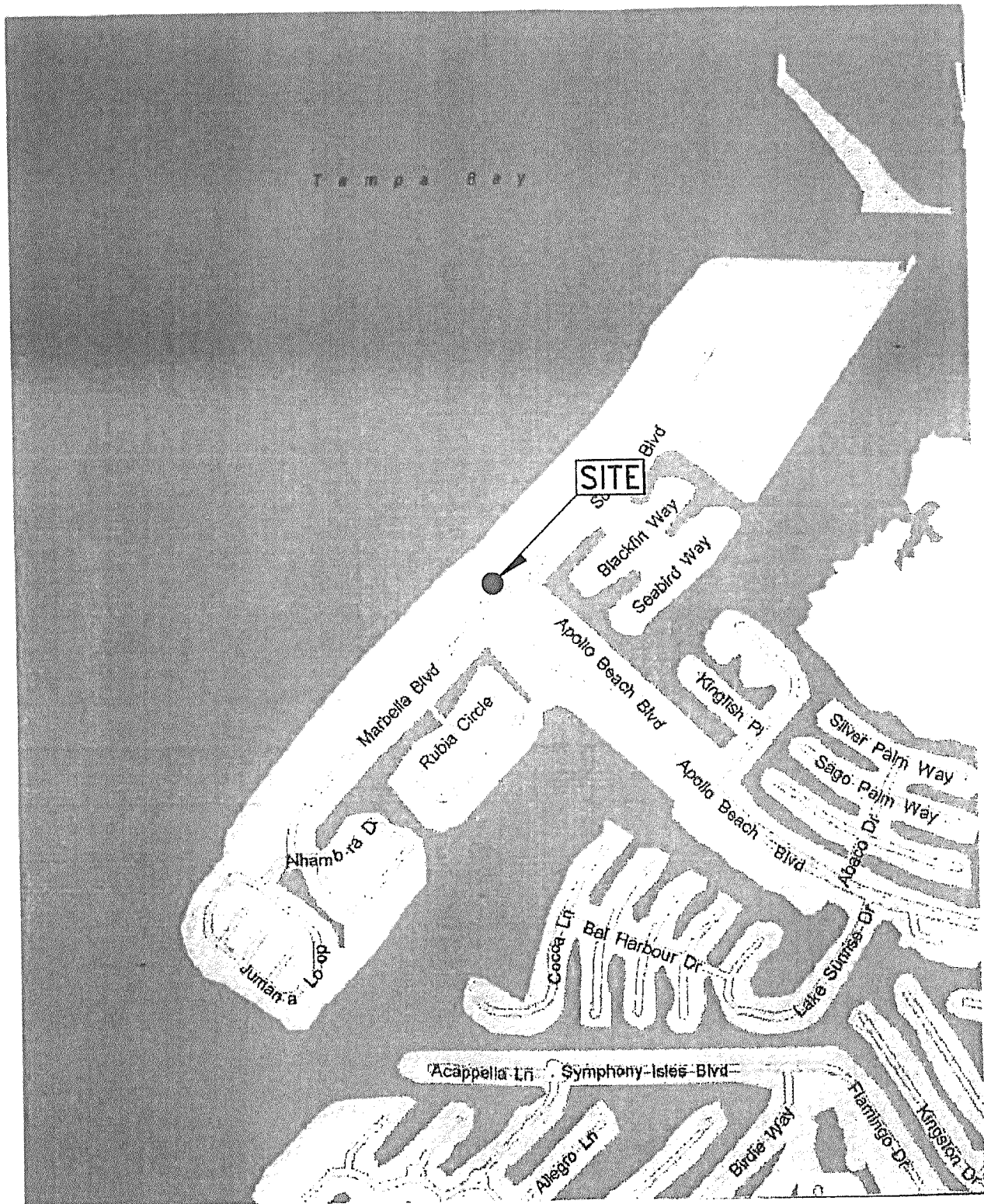
A handwritten signature in black ink, reading "John R. Marquardt".


John R. Marquardt, P.E. 6/20/06
Senior Geotechnical Engineer
Florida License No. 24205



FIGURES

SECTION 17, TOWNSHIP 31 SOUTH, RANGE 19 EAST
HILLSBOROUGH COUNTY, FLORIDA



JOB NO.: 502037600 CAD NO.: 037600-01 DATE: 05/02/08	 ENGINEERS & SCIENTISTS 4019 E. Fowler Avenue Tampa, Florida 33617 Tel: (813) 971-3882	APOLLO BEACH CONDOMINIUM 6502 SURFSIDE BOULEVARD APOLLO BEACH, FLORIDA	SHEET TITLE SITE LOCATION MAP FIGURE 1
--	--	---	--

TAMPA BAY

N45°37'51"E 388.53'(F)

BLUETOOTH LINE PER
PLAN BOOK 35, PAGE 18

EX. 0.5' WOOD
SEAWALL

EX. 1.2' CONC.
SEAWALL

6' IRON
FENCE

DECK

POOL

BEACH AREA

SPT-1

EXERCISE/SPA

ARCHITECTURAL
METAL FENCE, TYP.

1.5" x 6" CONCRETE
COLUMN
(TYP.)

300.03'(F)

1.0" SIZE
CONCRETE
WALL(TYP.)

N44°23'00"W

S44°22'36"E

300.00'(C)

5' CONC. S/W

EX. 60"

EX. ASPHALT PAVEMENT

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

EX. 60"

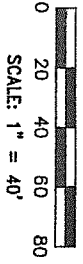
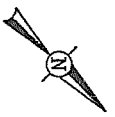
EX. 60"

EX. 60"

EX. 60"

SURF SIDE BOULEVARD
(60' N/W PER CONDO BOOK 6, PAGE 61-61)

EX. 6' W/ LINE
OF SURF SIDE BLVD



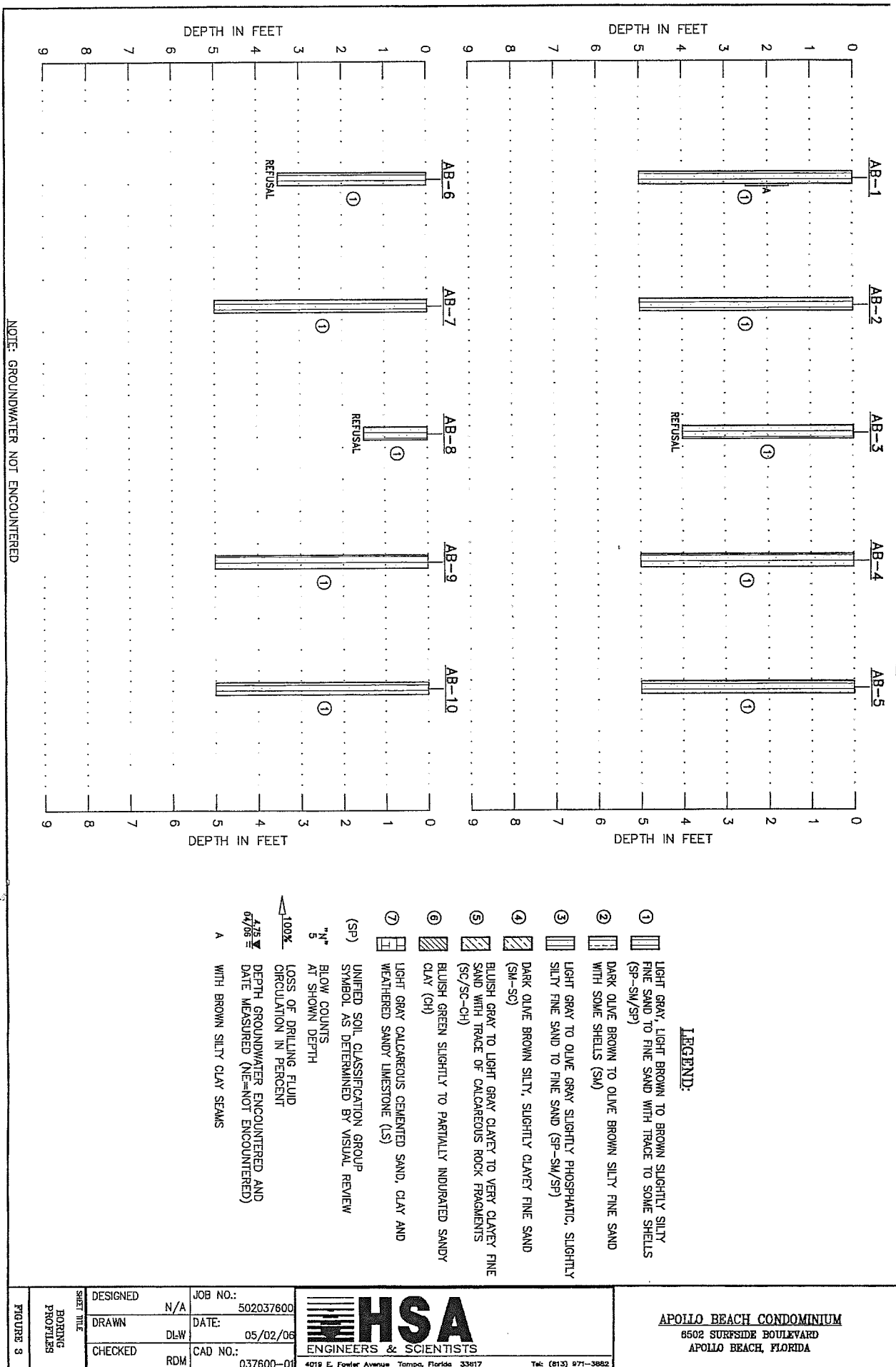
LEGEND

- SPT-1 STANDARD PENETRATION TEST BORING LOCATIONS (RECENT)
- AB-1 HAND AUGER BORING LOCATIONS (RECENT)
- DR-1 DOUBLE RING INFILTRATION TEST LOCATION (RECENT)
- SPT-1 STANDARD PENETRATION TEST BORING LOCATIONS (PREVIOUS)
- AB-1 HAND AUGER BORING LOCATIONS (PREVIOUS)

- NOTES:**
1. BASE MAP GENERATED FROM DRAWINGS PROVIDED BY SCAROLA ASSOCIATES, FIELD TAPE MEASUREMENTS AND ESTIMATED DIRECTIONS.
 2. BUILDING OUTLINE & LOCATION ARE APPROXIMATE.
 3. BORING & TEST PIT LOCATIONS ARE APPROXIMATE.

FIGURE 2	DESIGNED	N/A	JOB NO.: 502037600	 4019 E. Fowler Avenue Tampa, Florida 33617 Tel: (813) 971-3882
	DRAWN	DLW	DATE: 05/02/06	
	CHECKED	RDM	CAD NO.: 037600-01	
	SHEET TITLE: BORING LOCATION PLAN			

APOLLO BEACH CONDOMINIUM
6502 SURF SIDE BOULEVARD
APOLLO BEACH, FLORIDA



APOLLO BEACH CONDOMINIUM
 6502 SURFSIDE BOULEVARD
 APOLLO BEACH, FLORIDA

HSA
 ENGINEERS & SCIENTISTS

4019 E. Fowler Avenue Tampa, Florida 33617

Tel: (813) 871-3862

JOB NO.: 502037600
 DATE: 05/02/06
 CAD NO.: 037600-01

DESIGNED N/A
 DRAWN DLW
 CHECKED RDM

SHEET TITLE
 BORING PROFILES
 FIGURE 3

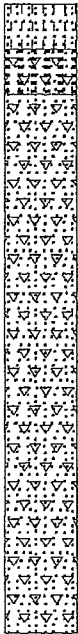


APPENDIX A

SOIL BORING LOG

AB-1

PROJECT Existing Ramada Inn	PROJECT NO. 50-15-3221-00
CLIENT Kendar Corporation	DATE 12-15-04
LOCATION see map	ELEV. not measured
EXCAVATION METHOD hand auger	LOGGER Jamie Elkins
DEPTH TO - Water: not enc. When checked: at completion Caving:	

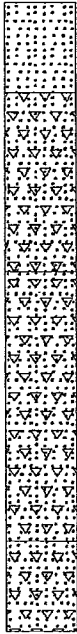
ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS			USCS	DESCRIPTION	% PASSING 200 SIEVE	MOISTURE CONTENT %
	GRAPHIC	BULK	DRIVEN				
0				SP-SM	Dark gray slightly silty fine sand		
				SP	Yellowish brown fine sand with trace roots and trace shell fragments		
				SP	Light yellowish gray brown fine sand, shell fragment content increasing with depth		
2							
4							
6							
8					End of boring at 7.0 feet		
10							

Notes:

SOIL BORING LOG

AB-2

PROJECT Existing Ramada Inn	PROJECT NO. 50-15-3221-00
CLIENT Kendar Corporation	DATE 12-15-04
LOCATION see map	ELEV. not measured
EXCAVATION METHOD hand auger	LOGGER Jamie Elkins
DEPTH TO - Water: not enc. When checked: at completion Caving:	


ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS			USCS	DESCRIPTION	% PASSING 200 SIEVE	MOISTURE CONTENT %
	GRAPHIC	BULK	DRIVEN				
0				SP	Brownish gray fine sand		
1				SP	Yellowish brown fine sand with some shell fragments		
2							
3				SP	Light yellowish gray fine sand with some shell fragments		
4							
5							
6				SP	Light gray fine sand with trace shell fragments		
7					End of boring at 7.0 feet		
8							
9							
10							

Notes:

SOIL BORING LOG

AB-3

PROJECT Existing Ramada Inn	PROJECT NO. 50-15-3221-00
CLIENT Kendar Corporation	DATE 12-15-04
LOCATION see map	ELEV. not measured
EXCAVATION METHOD hand auger	LOGGER Jamie Elkins
DEPTH TO - Water: not enc. When checked: at completion Caving:	

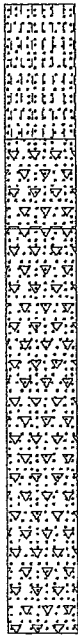
ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS			USCS	DESCRIPTION	% PASSING 200 SIEVE	MOISTURE CONTENT %
	GRAPHIC	BULK	DRIVEN				
0				SP-SM	Dark gray slightly silty fine sand		
				SP	Yellowish brown fine sand with trace shell fragments		
2				SP	Light yellowish gray fine sand with trace shell fragments		
					Seam of dark brown silty clay		
4							
6							
8					End of boring at 7.0 feet		
10							

Notes:

SOIL BORING LOG

AB-4

PROJECT <div style="text-align: center;">Existing Ramada Inn</div>	PROJECT NO. <div style="text-align: center;">50-15-3221-00</div>
CLIENT <div style="text-align: center;">Kendar Corporation</div>	DATE <div style="text-align: center;">12-15-04</div>
LOCATION <div style="text-align: center;">see map</div>	ELEV. <div style="text-align: center;">not measured</div>
EXCAVATION METHOD <div style="text-align: center;">hand auger</div>	LOGGER <div style="text-align: center;">Jamie Elkins</div>
DEPTH TO - Water: not enc. When checked: at completion Caving:	

ELEVATION/ DEPTH	SOIL SYMBOLS AND SAMPLERS			USCS	DESCRIPTION	% PASSING 200 SIEVE	MOISTURE CONTENT %
	GRAPHIC	BULK	DRIVEN				
0				SP-SM	Dark gray to dark brown slightly silty fine sand		
2				SP	Yellowish brown fine sand with some shell fragments with dark brown fine sand		
4				SP	Light yellowish gray fine sand with some shell fragments		
6					End of boring at 7.0 feet		
8							
10							

Notes:

PROJECT: <i>Existing Ramada Inn</i>				LOG OF BORING SPT-1						
PROJECT NO.: 50-15-3221-00										
DATE DRILLED: 12-09-04		ELEVATION:		Location: see map						
DRILER: Joe Erickson		BORING DEPTH: 70.00 Feet								
DRILLING METHOD: mud rotary		WATER LEVEL: not meas.								
ANALYSIS					Depth feet	SPT "N" Values	GROUND WATER	GRAPHIC LOG	USCS	GEOLOGIC DESCRIPTION
% Natural Moisture Content	% Passing 200 Sieve	Liquid Limit	Plastic Limit	Plasticity Index						
					0	18			SP- SM	Light gray fine sand to slightly silty fine sand with trace to some shell
					4	36				
						17				
					8	13				
						14				
					12					
					16	9			SM	Brownish gray silty fine sand with some shell
					20	8				
					24	WH			SP- SM	Light gray slightly silty fine sand with some shell
					28					
					32	65				
					36	43				
					40	54			SP- SM	Grayish brown slightly silty fine sand with some fine phosphate and shell
					44	27				
					48				CH	Cream colored and green calcareous clay
					52	5				
					56	15				
					60	31			SM- SC	Light cream colored and gray calcareous silty clayey fine sand with fine phosphate
					64	28			CL	Green clay
					68					
					72	33				End of boring at 70 feet
					76					
					80					
					84					
					88					
					92					

PROJECT: <i>Existing Ramada Inn</i>				LOG OF BORING SPT-2						
PROJECT NO.: 50-15-3221-00										
DATE DRILLED: 12-09-04		ELEVATION:		Location: see map						
DRILER: Joe Erickson		BORING DEPTH: 80.00 Feet								
DRILLING METHOD: mud rotary		WATER LEVEL: not meas.								
ANALYSIS					Depth feet	SPT "N" Values	GROUND WATER	GRAPHIC LOG	USCS	GEOLOGIC DESCRIPTION
% Natural Moisture Content	% Passing 200 Sieve	Liquid Limit	Plastic Limit	Plasticity Index						
					0	HA			SP-SM	Light gray fine sand to slightly silty fine sand with trace to some shell
					4	22				
					8	16				
					12	9				
					16	3				
					20	16				
					24	4				
					28	5				
					32	3			SM	Dark brownish gray silty slightly clayey fine sand with some shell
					36	25			SP-SM	Light gray slightly silty fine sand with some shell
					40	26			SP-SM	Dark brownish gray silty slightly clayey fine sand
					44	58			SP-SM	Light gray slightly silty fine sand with some shell and fine phosphate
					48	13			SM-SC	Light cream colored and gray calcareous silty clayey fine sand with fine phosphate
					52	7				
					56	4				
					60	16				
					64	12			CL	Cream colored and green calcareous clay
					68	60				
					72	50/0				
					76					
					80					End of boring at 80 feet
					84					
					88					
					92					