Geotechnical Engineering Evaluation
Proposed Parking Garage on
South Lakeview Avenue at Tremaine Street
Winter Garden, Florida

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Geotechnical, Environmental and
Materials Consultants

December 22, 2014
File No. 14-6481

City of Winter Garden
300 West Plant Street
Winter Garden, Florida 34787

Attention: Mr. Arthur Miller

Subject: Geotechnical Engineering Evaluation
Proposed Parking Garage on
South Lakeview Avenue at Tremaine Street
Winter Garden, Florida

Gentlemen:

As requested and authorized by you, we have completed a geotechnical engineering evaluation for the subject project. We note that Ardaman & Associates conducted a Subsurface Soil Exploration for the subject site in November, 2014. The results of the field exploration as documented in our data report dated November 26, 2014 (Ardaman & Associates File No. 14-6481) were used in the geotechnical engineering evaluation presented herein. The purposes of performing this geotechnical engineering evaluation were to evaluate the general subsurface conditions within the building area and to provide recommendations for site preparation and foundation support. This report documents our findings and presents our engineering recommendations.

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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VAG/CTJ/jj
14-6481 additional sse parking garage

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1.0 SITE LOCATION AND SITE DESCRIPTION

The site for the proposed parking garage is located on the southeast corner of the intersection of South Lakeview Avenue and Tremaine Street in Winter Garden, Florida (Section 23, Township 22 South, Range 27 East). The general site location is shown superimposed on the Winter Garden, Florida U.S.G.S. quadrangle map presented on Figure 1.

The site is currently developed with an existing asphalt covered parking lot.

2.0 PROPOSED CONSTRUCTION AND GRADING

It is our understanding that the proposed development includes a 3-story, 4-level, parking garage of either precast or cast in place concrete construction and built using the design-build delivery method. Preliminary column spacing and foundation loads were provided to us by Walker Parking Consultants via electronic mail on December 2, 2014. Based on this correspondence, typical column spacing is anticipated to be 62 feet in the north-south direction and 36 feet in the east-west direction. Column loads for the parking garage are on the order of 670 kips for exterior columns, and 500 kips for interior columns. Further, we understand that the interior wall loads will be on the order of 30 kips per linear foot. Floor loads are assumed to be less than 250 pounds per square foot. Grading plans are not complete at this time, therefore we have assumed that 0 to 2 feet of fill are required to raise the building area to final elevations. If actual building loads or fill height exceed our assumptions, then the recommendations in this report may not be valid.

3.0 REVIEW OF SOIL SURVEY MAPS

Based on the 1989 Soil Survey for Orange County, Florida, as prepared by the U.S. Department of Agriculture Soil Conservation Service, the site is located in an area mapped as the "Milhopper-Urban land complex, 0 to 5 percent slopes" soil series and the “Urban land” soil series.

The southern portion of the property is in an area mapped as the "Milhopper-Urban land complex, 0 to 5 percent slopes" soil series, which consists of sandy Milhopper soil that is gently sloping and areas of Urban land. The internal drainage of the "Milhopper-Urban land complex, 0 to 5 percent slopes" is described as moderately well drained and the soil permeability is rapid in the surface and subsurface layers, and it is slow to moderate in the subsoil. According to the Soil Survey, the seasonal high water table for the "Milhopper-Urban land complex, 0 to 5 percent slopes" soil series is typically within 40 to 60 inches of the natural ground surface.

The northern portion of the property is in an area mapped as the “Urban land” soil series, which consists of urban facilities such as shopping centers, parking lots, industrial buildings, houses, streets, sidewalks, airports, and related urban structures. The natural soil cannot be observed. According to the Soil Survey, the seasonal high water table for the “Urban land” soil series is dependent upon the functioning of the drainage systems established in the area.

4.0 FIELD EXPLORATION PROGRAM

4.1 SPT Borings

The field exploration program included performing five Standard Penetration Test (SPT) borings. The SPT borings were performed at locations within the proposed "footprint" of the parking garage. The borings were advanced to depths ranging from 75 to 90 feet below the ground surface using the methodology outlined in ASTM D-1586. A summary of this field procedure is included in the Appendix. 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 groundwater level at each of the boring locations was measured during drilling. The boreholes were grouted with cement-bentonite slurry upon completion. The holes in the existing parking lot were filled with cold-patch asphalt.

4.2 Test Locations

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

5.0 LABORATORY PROGRAM

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

In addition, we conducted nine percent fines analyses (ASTM D1140) on selected soil samples obtained from the borings. The results of these tests are presented adjacent to the sample depth on the boring profiles on Figures 3 and 4.

6.0 GENERAL SUBSURFACE CONDITIONS

6.1 General Soil Profile

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

The results of the borings indicate the following general soil profile:

Depth Below Ground Surface (feet)	Description
0 - 4	Asphaltic concrete and base underlain by fine sand (SP) or fine sand with silt (SP-SM)
4 - 13.5	Loose to medium dense clayey fine sand (SC)
13.5 - 65	Very loose to loose fine sand with clay (SP-SC) or clayey fine sand (SC)
65 - 75	Medium dense to dense clayey fine sand (SC)
75 - 90	Medium dense to very dense clayey fine sand (SC) or claystone

We note that complete and/or partial losses of drilling fluid circulation occurred in Borings TH-1, TH-2 and TH-4 between a depth of approximately 60 and 65 feet below the existing ground surface.

We note that asphaltic concrete and base were present at all of the boring locations as shown on the boring profiles on Figures 3 and 4. The thickness of the asphaltic concrete and base as shown on the boring profiles should be considered a rough approximation. Coring of the pavement section would be required to provide accurate thickness measurements.

The above soil profile is outlined in general terms only. Please refer to Figures 3 and 4 for soil profile details.

6.2 Groundwater Level

An attempt was made to measure the groundwater level in the boreholes during drilling. Fluctuations in groundwater levels should be anticipated throughout the year primarily due to seasonal variations in rainfall and other factors that may vary from the time the borings were

conducted.

The absence of groundwater data at the boring locations indicates that groundwater was not encountered within the top 6 feet and could not be measured below a depth of 6 feet due to the mudded condition of the borehole and/or the slow recovery rates in the clayey soils (referenced “GNM” on Figures 3 and 4). However, this does not necessarily mean that groundwater would not be encountered within the top 6 feet of the borings referenced “GNM” at some other time.

7.0 ENGINEERING EVALUATION AND RECOMMENDATIONS

7.1 General

The results of our exploration indicate that, with proper site preparation as recommended in this report, the existing soils are suitable for supporting the proposed parking garage on conventional shallow foundations, provided that the calculated settlements presented in the "Foundation Support by Spread Footings and Foundation Compaction Criteria" section of this report are deemed acceptable (including for precast versus a cast-in place type structure). If so, spread footings should provide an adequate support system for the structure. If the calculated settlements are deemed unacceptable, alternative recommendations for foundation support by driven piles are also presented in the following report sections. The choice of the most appropriate foundation system should be based on structure performance and economic analysis.

We note that because of the proximity of nearby structures, the project budget should allow for a pre-construction survey of the structures, monitoring during construction, and some repair of any damage that may occur.

The following are our recommendations for overall site preparation and foundation support which we feel best suited for the proposed parking garage 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.

7.2 Stripping and Grubbing

The "footprint" of the proposed structure, plus a minimum margin of five feet, should be stripped of all surface vegetation, stumps, debris, organic topsoil or other deleterious materials, as encountered.

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, asphalt, and any other underground structures should be removed from the proposed construction area. If pipes or any collapsible or leak prone utilities are not removed or completely filled (with grout or concrete), they 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. This excavation must not undermine the existing building foundations. Provide shoring, bracing, and/or underpinning of existing footings as necessary to protect from failure.

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

7.3 Proof-rolling

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

A density equivalent to or greater than 95 percent of the modified Proctor (ASTM D-1557) maximum dry density value for a depth of 2 feet in the building area 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.

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). Compaction should cease if deemed detrimental to adjacent structures, and Ardaman & Associates should be notified immediately. <u>Due to the proximity of existing structures, heavy vibratory compaction should not be used.</u>

7.4 Suitable Fill Material and the Compaction of Fill Soils

All fill materials 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, fine sand with silt and fine sand with clay (Strata Nos. 1, 2 and 3 without roots, as shown on Figures 3 and 4) are suitable for use as fill materials and, with proper moisture control, should densify using conventional compaction methods.

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.

7.5 Foundation Support by Spread Footings and Foundation Compaction Criteria

The following recommendations are contingent upon the magnitude of calculated settlement being considered acceptable to the design team.

Excavate the foundations to the proposed bottom of footing elevations and, thereafter, verify the in-place compaction for a depth of 2 feet below the footing bottoms. If necessary, compact the soils at the bottom of the excavations to at least 95 percent of the modified Proctor maximum dry density (ASTM D-1557) for a depth of 2 feet below the footing bottoms. Based on the existing soil conditions and, assuming the below outlined proof-rolling and compaction criteria is implemented, an allowable soil bearing pressure of 4,000 pounds per square foot (psf) may be used in the foundation design.

To use a 4,000 psf allowable soil bearing pressure, all bearing foundations should be a minimum of 42 inches wide. A minimum soil cover of 24 inches should be maintained from the bottom of the foundations to the adjacent finished grades.

A preliminary settlement analysis was undertaken for the proposed building using the approximate loads and preliminary column spacing provided by Walker Parker Consultants. We note that at the time that this report was issued, a foundation arrangement plan showing the column and interior wall layout (including locations of proposed wall loads) was not available. Therefore the interior walls were not included in the analysis. The settlement analysis was undertaken using a foundation bearing pressure of 4,000 psf, with three rows of interior type columns between two rows of exterior columns at the provided typical spacing. In addition and for the purpose of providing an option for lower calculated settlement, we conducted a second analysis using an allowable soil bearing pressure of 2,000 psf. The computer program SETTLE3D was used together with the results of the SPT borings and laboratory testing. The SETTLE3D program calculates stresses and settlements under uniformly loaded rectangular areas. The results of our preliminary calculations are summarized in the below table.

Foundation Description	Maximum Load (Combined Dead and Live)	Calculated Total Settlement (inches)	
		@ 4,000 psf	@ 2,000 psf
Interior Column	500	1.8	1.3
Exterior Column	670	2.0	1.5

Settlement may be higher than indicated in the above table in areas where foundations are in closer proximity to each other than noted in the typical column spacing provided. In addition, settlement may be higher depending on the placement of the interior wall loads. Additional settlement analysis should be conducted once a finalized foundation arrangement plan is available.

We note that for the option of a maximum allowable soil bearing pressure of 2,000 psf the minimum foundation width may be reduced to 18 inches and minimum soil cover may be reduced to 18 inches.

7.6 Deep Foundation Support Alternatives

The following report sections are intended to provide an alternative to shallow foundation support (i.e. spread footings) in the event that the magnitude of calculated settlement presented above is considered unacceptable.

7.6.1 Foundation Support by Driven Piles

Pile capacities have been estimated for one type of low-displacement pile (HP 14x73 steel H-pile) and one type of displacement pile (14-inch square concrete pile). We note, however, that only one of the borings (designated TH-3) encountered what we would consider to be a high yielding type pile bearing type layer. Although we have provided calculated capacity based on this boring (i.e. Boring TH-3), this capacity and length information should be considered preliminary pending the drilling of additional soils borings to further explore the nature of the pile bearing conditions, and our analyses updated accordingly.

Based on our experience and analyses, low displacement piles (i.e. steel H-piles) will typically require greater penetration to achieve the required design loads, and are sometimes higher in cost per unit of load capacity when compared to displacement piles. However, a potentially offsetting advantage of these piles is that they are easier to drive and, for this reason, are often used over displacement piles in urban settings. Driven displacement piles generally develop more capacity than low-displacement piles but are often harder to drive and create more vibrations during construction.

Static pile capacities were preliminarily estimated using SPT N-values from the borings. Specifically, the FDOT computer software, "FB-Deep" was used for the SPT borings, which is based on procedures developed by Dr. Schmertman, originally outlined in Research Bulletin No. 121-B, dated September, 1967. The calculations incorporate a factor of safety of 2 for both the side friction and mobilized end bearing. The results of the FB-Deep analyses are included in Appendix II.

Based on the calculations, we preliminarily recommend the following design capacities:

Pile	**Preliminary Maximum Allowable Axial Load (tons)***
HP 14x89	90
14-inch Square Concrete	115
*Based on Boring TH-3 only	

Our calculations indicate that the above capacities can likely be achieved at a depth of on the order of 80 to 85 feet or greater below the existing ground surface (at the location of Boring TH-3) for both types of piles. *Actual pile lengths and capacities may be higher or lower than estimated and will be determined during pile driving as well as following additional subsurface soil exploration (i.e. drilling borings to greater depth).*

We note that significantly greater capacity can likely be achieved by driving the HP 14x89 steel H piles to greater depths than explored in the soil borings. If steel H piles or square concrete piles are deemed a desirable foundation choice, then we suggest that the remaining borings be extended deeper than 80 feet to further evaluate recommended design capacities and pile lengths.

Required tension forces have not been supplied to us at this time. However, allowable uplift capacities are anticipated to be at least 50 percent of the allowable compressive pile capacities presented in the above table.

To minimize the reduction of allowable compressive capacities for individual piles within a pile group, we recommend that the horizontal spacing of the piles in the individual pile cap be a minimum of 3 times the pile diameter. Should the pile spacing be less than 3 times the pile diameter, group reduction factors may become necessary. If this becomes apparent during design, we should be given the opportunity to review the pile arrangement to determine if group reduction factors are necessary.

It is noted that during pile driving, vibrations may cause settlement/displacement in existing nearby structures. This may be particularly problematic in the event the concrete (i.e. displacement) pile choice is selected. Care should be exercised to avoid damaging any neighboring structures while the pile driving operation is underway. Prior to commencing pile driving, the existing condition (i.e.

cracks) of nearby structures should be documented with photographs and survey. We also recommend that vibration monitoring be performed during pile driving. Monitoring can be stopped if data from the initial phases of pile driving indicate that vibration intensities are not an issue or at least well within tolerable limits.

The pile capacities presented above are estimated based on commonly used methods of analysis. The actual pile capacities may be higher or lower than those estimated. A test pile program will need to be implemented prior to driving the production piles. The test pile program will be used to assess pile driveability and refine foundation design parameters, final pile capacities, pile lengths and pile elevations at the subject site.

We recommend that a minimum of four piles be instrumented with a Pile Driving Analyzer (PDA) and driven at selected locations. Dynamic pile testing should also be performed during pile re-striking to evaluate time dependent soil strength changes (i.e. setup) during the installation of the test piles. Re-strikes of test piles may be required at 15 minute, 2-hour and/or 72-hour intervals after initial driving. The data will be used to evaluate hammer-driving system performance, pile stresses during driving, pile structural integrity, and calculation of pile static compressive bearing capacity. The dynamic testing results will then be used in wave equation analysis to establish an optimum pile driving criteria to be employed in driving the production piles. Prior to test pile driving, hammer specification should be available to assess pile driving system suitability. If deemed prudent during construction, a static load test(s) should be performed.

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

1. Manufacturer's specifications of the proposed hammer including all modifications.
2. Description and dimensions of the capblock material.
3. Description and dimensions of all cushion material.
4. Plans showing dimensions and details of construction of any followers to be used.
5. Any additional pertinent information necessary to perform 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.

Piles should be driven with a steam, air or diesel hammer capable of delivering the energy specified in the Florida Department of Transportation Standard Specifications for Road and Bridge Construction, Section 455. The hammer should be operated at the chamber pressure and speed recommended by the manufacturer at all times. Due to variable soil conditions, it may be necessary to vary the hammer energy during driving. Pile driving should be as continuous an operation as possible and should proceed without stopping over the last 10 feet of penetration.

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 Ardaman & Associates.

Because of the variable soil conditions encountered, some piles could be driven to a satisfactory depth, while nearby piles could be driven to a deeper depth and in a manner which could suggest that a void has been encountered near the tip elevation of the previously driven piles. If such a condition occurs during construction, it is recommended that the previously driven piles be restruck to verify capacity.

7.6.2 Foundation Support by Augered Cast-In-Place Piles (APG)

Augered Cast-in-Place Piles are made by rotating a continuous-flight, hollow-shaft auger into the ground to the specified pile depth, or until the specified refusal criteria is satisfied. Grout is then injected throughout the auger shaft, as the auger is being withdrawn, in such a way as to exert a positive upward grout pressure on the earth filled auger flights as well as a positive lateral pressure on the soil surrounding the grout filled pile hole. A spacer attached to a steel reinforcing rod is placed through the fresh concrete to verify that the hole has been maintained. 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.

Due to the complete and/or partial losses of drilling fluid circulation that were encountered in three out of the five borings, together with the relatively weak nature of the soils above this depth, we do not recommend the use of auger cast piles for this project. Use of auger cast piles may result in pile integrity problems (e.g. due to loss of pile grout into the zones where losses of drilling fluid circulation were encountered) or unacceptable settlement if the piles are tipped in the loose soils encountered above the zone of drilling fluid loss. As such, auger cast piles have been given no further consideration in this report.

7.7 Pile Lateral Load Analysis (If Required)

Input parameters for the FB-Pier program are included in Table 1. These recommended parameters are provided for use by the designer in performing a pile lateral load analysis. We note that the parameters were based on the soil profile encountered in Boring TH-4. Parameters can be provided for other borings if requested or deemed prudent.

7.8 Lateral Earth Pressures

Lateral loads acting on the embedded structure 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 depth below ground surface and the soil unit weight (submerged or moist) plus hydrostatic

pressure (if applicable). The following equations can be used to determine the lateral at-rest earth pressure:

$\sigma_h = K_o \gamma_m h$ (above groundwater table)
$\sigma_h = K_o [\gamma_m h_w + \gamma_b (h - h_w)]$ (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 backfill is lightly compacted yet not overcompacted)
γ_m = effective moist unit weight of soil = 110 pcf for compacted moist soil above the water table.
γ_b = buoyant unit weight of soil = 58 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.

7.9 Retaining Wall Design

Retaining walls should be designed to withstand the lateral forces exerted by the soils and surcharge loads developed behind the wall. We suggest using a coefficient of lateral earth pressure of 0.33 for walls which are able to deflect slightly and 0.5 for walls unable to deflect. A moist soil weight of 115 pcf should be multiplied by the appropriate coefficient of lateral earth pressure to determine the equivalent fluid pressure acting on the retaining wall. Also, the surcharge loads should be multiplied by the appropriate coefficient to determine the resulting additional uniform horizontal load.

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 be compacted with hand-held or light weight compactors to between 95 and 100 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. Heavy equipment should be kept at least 5 feet away from the wall. To prevent the building of water pressure, weep holes should be installed at the base of the wall.

The above design does not take into account horizontal forces which may develop as a result of water pressure behind the walls. Suitable safety factors should be used for design.

7.10 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.

7.11 Dewatering

The control of 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 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.

7.12 Discussion Relative to Sinkhole Conditions

Although the complete and/or partial losses of drilling fluid circulation that occurred in three of the five borings that were conducted for this study can sometimes be considered an indicator of sinkhole activity, it is our opinion that at this site they are related to the erosional and depositional processes that resulted in the formation of the land mass over geologic time, and do not directly indicate that a sinkhole(s) is forming at the boring locations (which were spaced over 100 feet apart from each other). The presence of persistent clayey soils over depth will also greatly retard the downward flow of water, thereby reducing the potential for sinkhole related erosional processes to develop. As such, we consider the risk for a sinkhole to develop at this site to be no greater than for many other developed sites located in this part of Orange County. As for any development of this type and in this region of Florida, we recommend that sinkhole insurance coverage be purchased and maintained.

8.0 QUALITY ASSURANCE

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

As a minimum, an on-site engineering technician should monitor all stripping and grubbing to verify that all deleterious materials have been removed and should observe the proof-rolling operation to verify that the appropriate number of passes are applied to the subgrade. In-situ density tests should be conducted during backfilling activities and below all footings and floor slabs 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.

Any pile driving or installation operations should be continuously monitored by a representative of Ardaman & Associates.

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

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

Representative samples of the various natural ground and fill soils 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.

10.0 CLOSURE

The analyses and recommendations submitted herein are based on the data obtained from the soil borings presented on Figures 3 and 4 and 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 is based on a relatively shallow exploration and is not intended to be an evaluation for sinkhole potential. This study does not include an evaluation of the environmental (ecological or hazardous/toxic material related) condition of the site and subsurface.

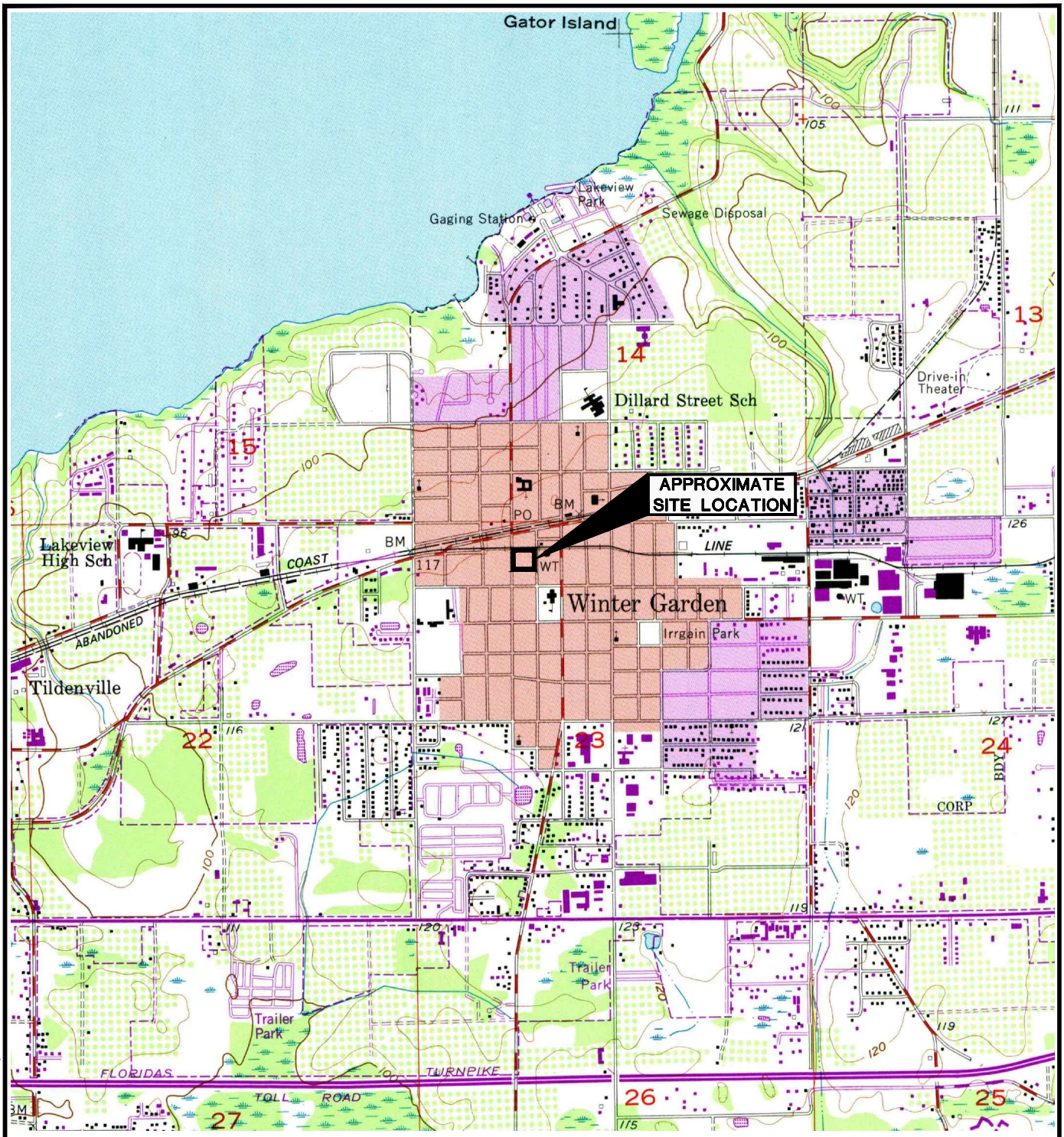
This report has been prepared for the exclusive use of the City of Winter Garden in accordance with generally accepted geotechnical engineering practices for the purpose of assisting with the development of the proposed parking garage. In the event any changes occur in the design, nature, or location 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.

TABLE 1
FB-Pier Input Parameters
Proposed Parking Garage on
South Lakeview Avenue at Tremaine Street
Winter Garden, Florida

Ground Surface Elevation (Feet NAVD)	Typical Material Description	μ	Φ (degrees)	γ_T (pcf)	C (psf)	ε_{50}	τ (conc.) (tsf)	τ (steel-H) (tsf)	G (ksf)	E (ksf)	k (pci)
0 – 4	Fine sand with silt (SP-SM) or fine sand with clay (SP-SC)	0.3	29	112	---	---	0.18	0.07	115	300	15
4 – 13.5	Medium dense clayey fine sand (SC)	0.3	32	117	---	---	0.6	0.4	200	500	40
13.5 – 28.5	Loose to medium dense fine sand with silt (SP-SM) to silty fine sand (SM) or fine sand with clay (SP-SC)	0.3	30	112	---	---	0.35	0.2	150	400	22
28.5 – 57	Very loose to loose fine sand with clay (SP-SC) to clayey fine sand (SC)	0.3	29	111	---	---	0.2	0.15	100	250	15
57 – 68.5	Very loose clayey fine sand (SC)	0.3	28	105	---	---	0.07	0.05	75	150	5
68.5 – 80	Medium dense clayey fine sand (SC) or claystone	0.3	32	118	---	---	0.75	0.5	300	600	55

Note: Parameters based on upper portion of soil profile encountered in Boring TH-4.

μ Poissons Ratio
φ Internal angle of friction
γ_T Saturated unit weight
C Undrained shear strength
τ Ultimate vertical shear stress
G Shear Modulus
E Youngs Modulus
k Soil Modulus



SECTION 23
TOWNSHIP 22 SOUTH
RANGE 27 EAST

OBTAINED FROM U.S.G.S. QUAD MAP: WINTER GARDEN, FLORIDA 1984

N

0 1000 2000

APPROXIMATE SCALE: 1"=2000'

QUADRANGLE LOCATION

SITE LOCATION MAP

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

SUBSURFACE SOIL EXPLORATION
PROPOSED PARKING GARAGE
ON SOUTH LAKEVIEW AVENUE AT
TERMAINE STREET
WINTER GARDEN, FLORIDA

DRAWN BY: CD	CHECKED BY:	DATE: 11/21/14
FILE NO. 14-6481	APPROVED BY:	FIGURE: 1

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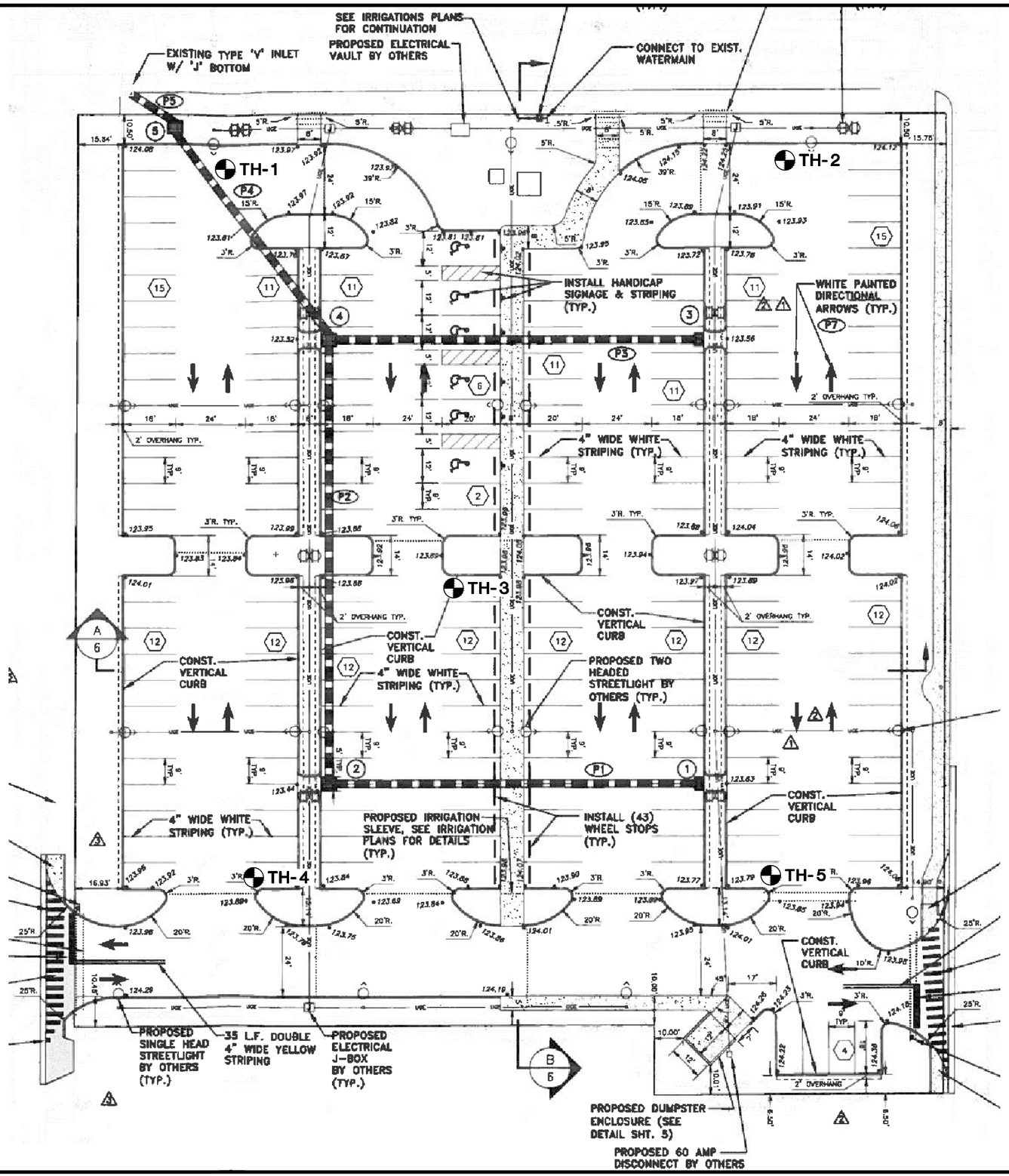

LEGEND

TH — STANDARD PENETRATION TEST (SPT) BORING LOCATION

NOTE: THE BASE MAP FOR THE BORING LOCATION PLAN IS A SITE PLAN BY AR MILLER ENGINEERING, INC., DATED 08/22/08.

N

0 20 40

APPROXIMATE SCALE: 1"=40'

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BORING LOCATION PLAN

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

SUBSURFACE SOIL EXPLORATION
PROPOSED PARKING GARAGE
ON SOUTH LAKEVIEW AVENUE AT
TERMAINE STREET
WINTER GARDEN, FLORIDA

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LEGEND

SOIL DESCRIPTIONS

1. FINE SAND (SP)
2. FINE SAND WITH SILT (SP-SM)
3. FINE SAND WITH CLAY (SP-SC)
4. SILTY FINE SAND (SM)
5. CLAYEY FINE SAND (SC)
6. ASPHALTIC CONCRETE AND BASE
7. CLAYSTONE (MOSTLY CEMENTED SAND, SILT AND CLAY)

COLORS

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

TH	STANDARD PENETRATION TEST (SPT) BORING
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT
-200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES)(ASTM D-1140)
50/3	50 BLOWS FOR 3-INCHES PENETRATION INTO SOIL
←	COMPLETE LOSS OF DRILLING FLUID CIRCULATION
⇦	PARTIAL LOSS OF DRILLING FLUID CIRCULATION
GNM	GROUNDWATER NOT MEASURED (i.e., NOT ENCOUNTERED IN THE TOP 6 FEET AND NOT MEASURED BELOW 6 FEET DUE TO THE MUDDED CONDITION OF THE BOREHOLE AND/OR SLOW RECOVERY RATES IN THE CLAYEY SOILS)
SP,SP-SM SM,SC,CH	UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

NOTES:

1. UPON COMPLETION OF EACH SPT BORING, THE BOREHOLE WAS GROUTED WITH CEMENT-BENTONITE SLURRY.
2. THICKNESS OF ASPHALTIC CONCRETE AND BASE SHOULD BE CONSIDERED A ROUGH APPROXIMATION ONLY.
3. ALL SPT BORINGS WERE PERFORMED USING AN AUTOMATIC HAMMER TO THE BORING TERMINATION DEPTH. AUTOMATIC HAMMER N-VALUES MAY BE CONVERTED TO EQUIVALENT SAFETY HAMMER N-VALUES BY MULTIPLYING BY 1.24.

ENGINEERING CLASSIFICATION

I COHESIONLESS SOILS

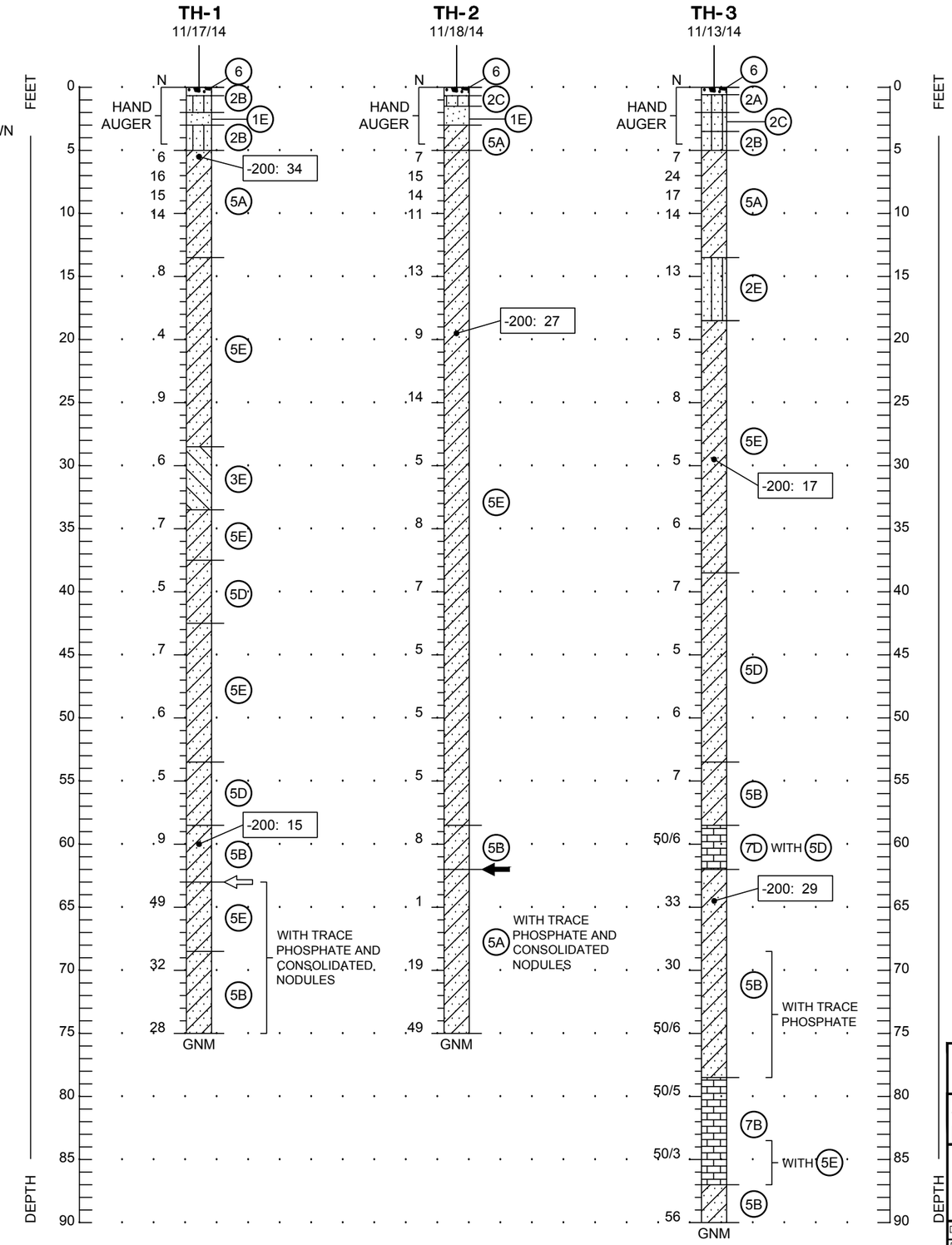
DESCRIPTION	BLOW COUNT "N"
VERY LOOSE	<4
LOOSE	4 TO 10
MEDIUM DENSE	10 TO 30
DENSE	30 TO 50
VERY DENSE	>50

II COHESIVE SOILS

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

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

GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR. ABSENCE OF WATER SURFACE DATA IN THE BORING IMPLIES THAT NO GROUNDWATER DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THIS LOCATION OR WITHIN THE VERTICAL REACHES OF THIS BORING IN THE FUTURE.



SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
PROPOSED PARKING GARAGE
ON SOUTH LAKEVIEW AVENUE AT
TERMAINE STREET
WINTER GARDEN, FLORIDA

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LEGEND

SOIL DESCRIPTIONS

1. FINE SAND (SP)
2. FINE SAND WITH SILT (SP-SM)
3. FINE SAND WITH CLAY (SP-SC)
4. SILTY FINE SAND (SM)
5. CLAYEY FINE SAND (SC)
6. ASPHALTIC CONCRETE AND BASE
7. CLAYSTONE (MOSTLY CEMENTED SAND, SILT AND CLAY)

COLORS

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TH	STANDARD PENETRATION TEST (SPT) BORING
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT
-200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES)(ASTM D-1140)
50/3	50 BLOWS FOR 3-INCHES PENETRATION INTO SOIL
WOR	SAMPLER ADVANCED BY STATIC WEIGHT OF RODS ONLY
←	COMPLETE LOSS OF DRILLING FLUID CIRCULATION
GNM	GROUNDWATER NOT MEASURED (i.e., NOT ENCOUNTERED IN THE TOP 6 FEET AND NOT MEASURED BELOW 6 FEET DUE TO THE MUDDED CONDITION OF THE BOREHOLE AND/OR SLOW RECOVERY RATES IN THE CLAYEY SOILS)
‖	3 1/2-INCH DIAMETER TEMPORARY STEEL CASING
SP,SP-SM SM,SC,CH	UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

ENGINEERING CLASSIFICATION

I COHESIONLESS SOILS

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

II COHESIVE SOILS

DESCRIPTION	UNCONFINED COMPRESSIVE STRENGTH, QU, TSF	BLOW COUNT "N"
VERY SOFT	<1/4	<2
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HARD	>4	>30

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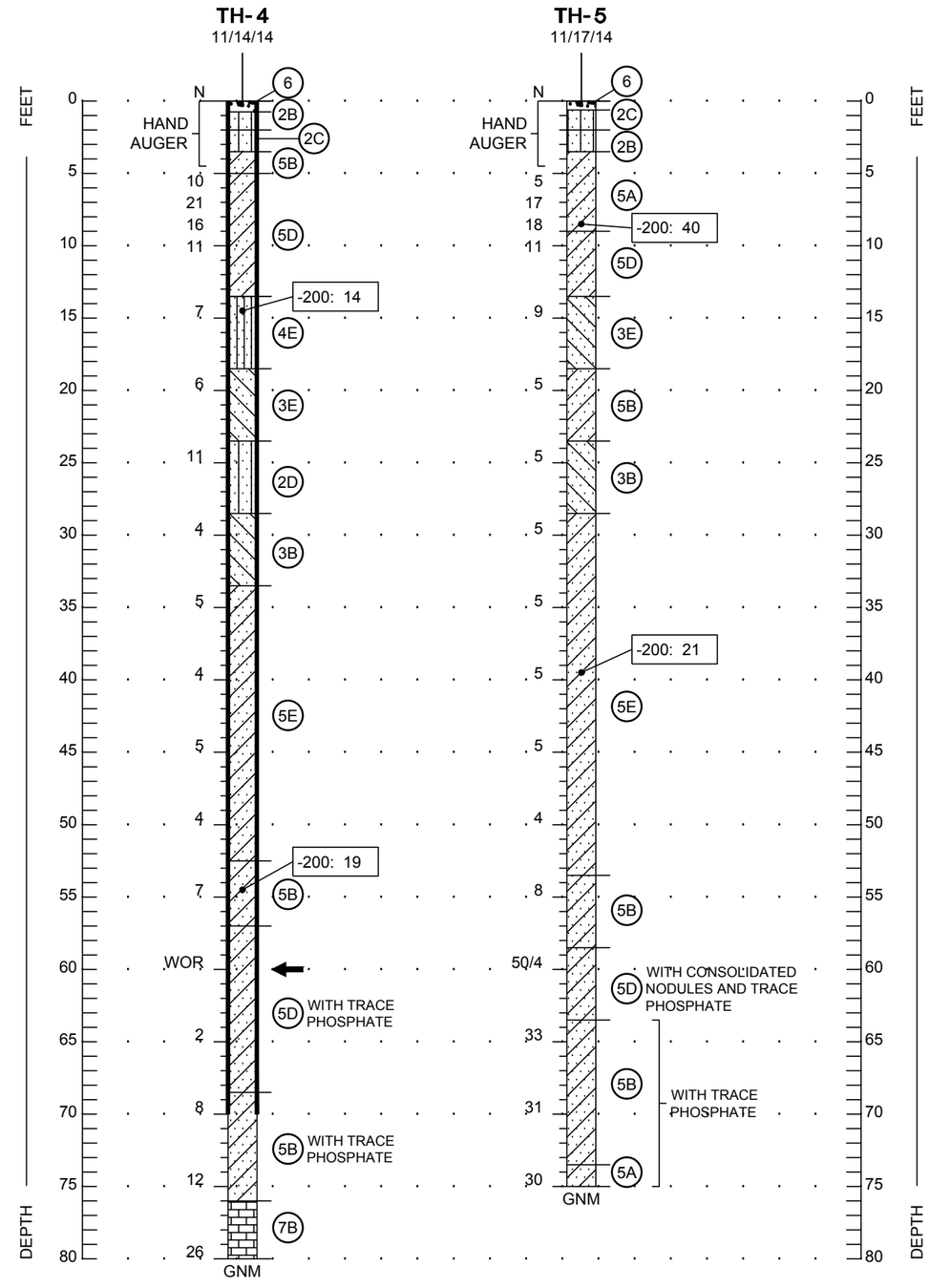

NOTES:

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SOIL BORING PROFILES

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

SUBSURFACE SOIL EXPLORATION
PROPOSED PARKING GARAGE
ON SOUTH LAKEVIEW AVENUE AT
TERMAINE STREET
WINTER GARDEN, FLORIDA

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APPENDIX I

Standard Penetration Test Boring Procedure

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. 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.

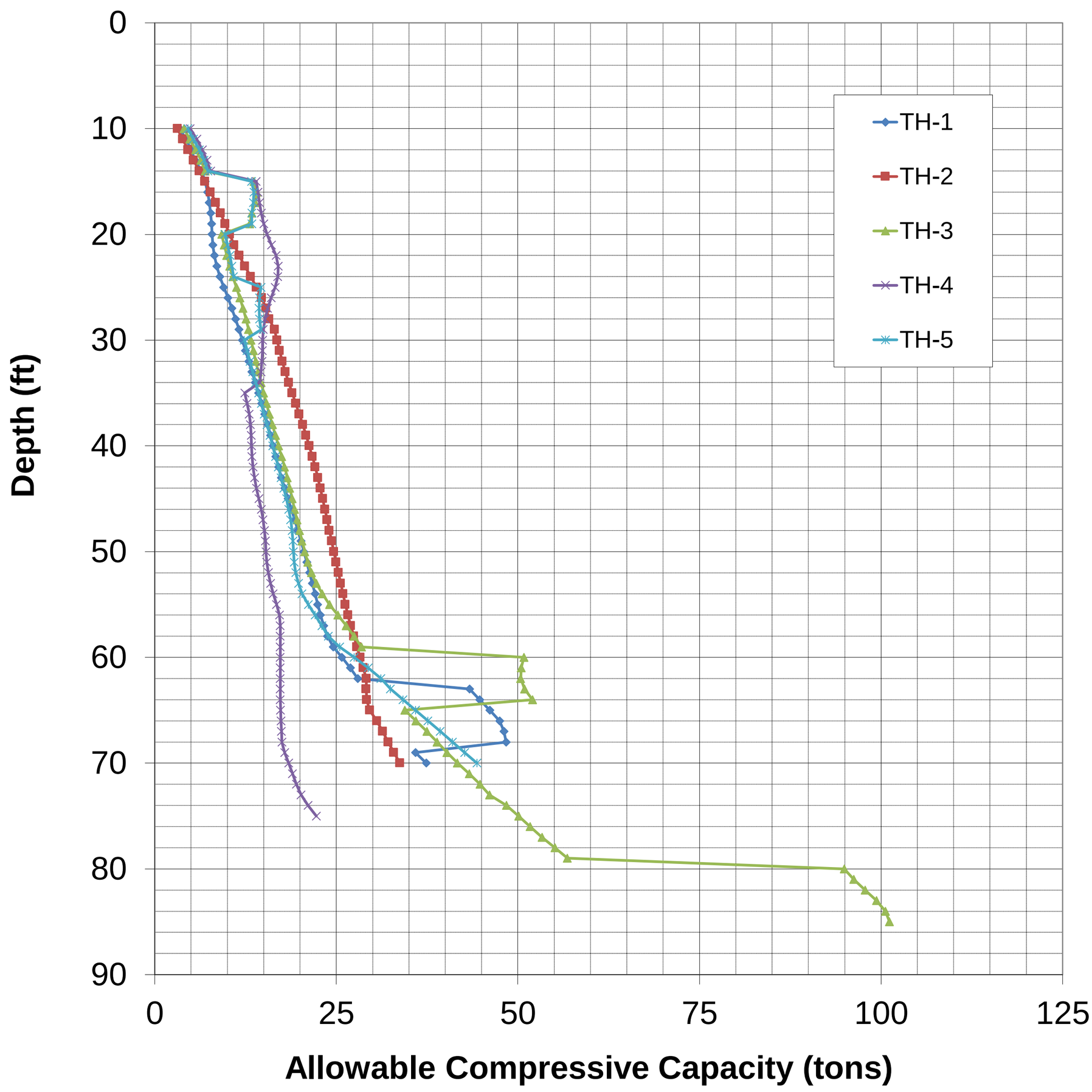
APPENDIX II

Allowable Compressive Pile Capacity for HP 14x89 Steel H Piles and

14-inch Square Concrete Piles

ALLOWABLE BEARING RESISTANCE

HP 14x89 Steel H-Piles



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GEOTECHNICAL ENGINEERING EVALUATION
PROPOSED PARKING GARAGE ON
SOUTH LAKEVIEW AT TREMAINE STREET
WINTER GARDEN, FLORIDA

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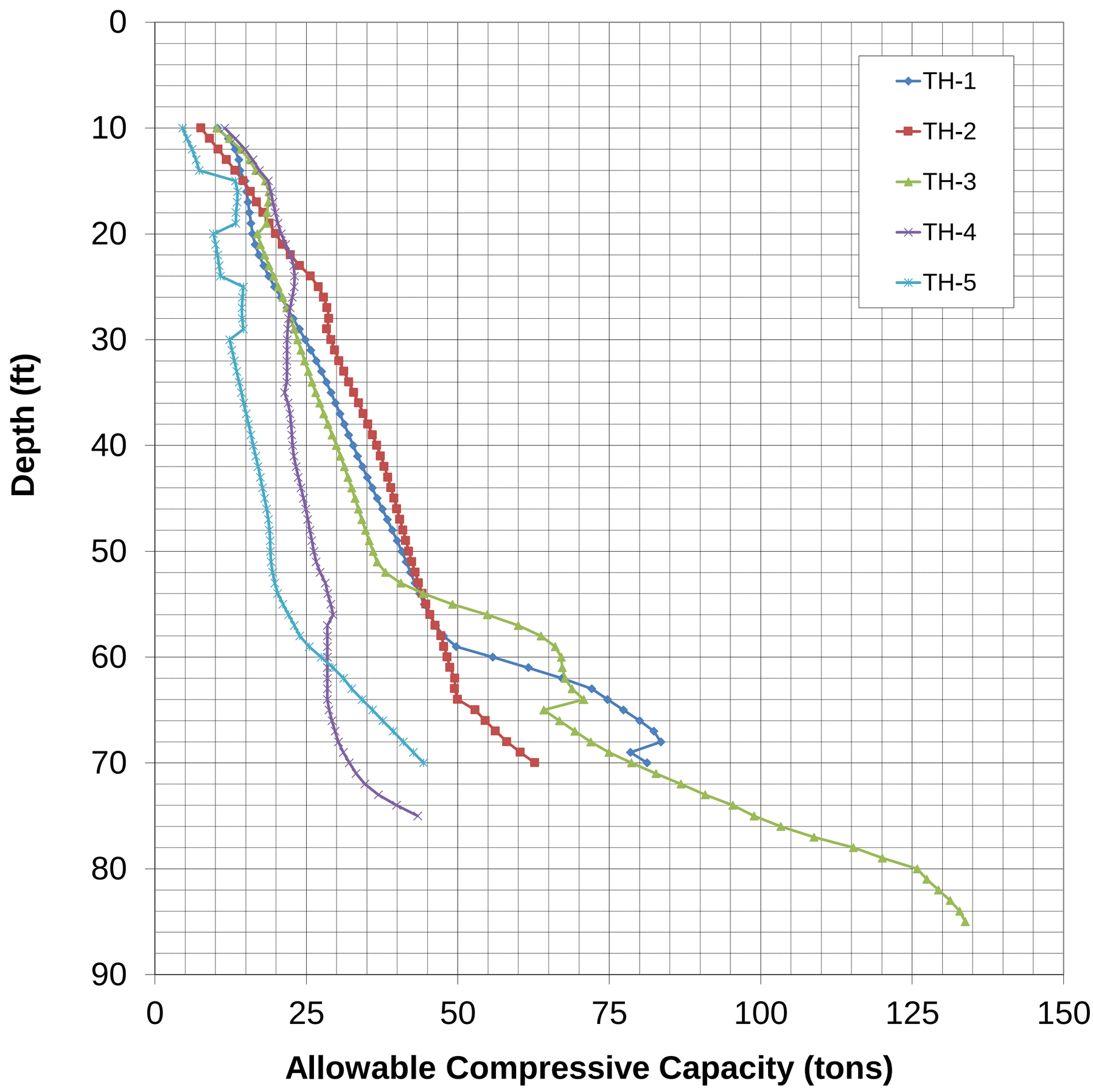
ALLOWABLE BEARING RESISTANCE

14-inch Square Concrete Piles



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Geotechnical, Environmental
and Material Consultants

GEOTECHNICAL ENGINEERING EVALUATION
PROPOSED PARKING GARAGE
SOUTH LAKEVIEW AVENUE AT TREMAINE STREET
WINTER GARDEN, FLORIDA

File No.	Date	Figure
14-6481	**December-14**	**---**