Report of Geotechnical Engineering Evaluation

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

Orange County, Florida

GEC Project No. 4406G

Geotechnical and Environmental Consultants, Inc.

At the very foundation of our community

July 18, 2019
Revised December 19, 2019

Silver Hills Development, Inc.
23500 Shelburne Road
Shaker Heights, OH 44122

Attention: Mr. Ryan Solow and Mr. Seth Mendelsohn
cc: Mr. Don Bieger, P.E. / DRMP, Inc. (dbieger@drmp.com)

Subject: Report of Geotechnical Engineering Evaluation
LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT
Orange County, Florida
GEC Project No. 4406G

Dear Mr. Solow and Mr. Mendelsohn:

Geotechnical and Environmental Consultants, Inc. (GEC) has completed our geotechnical engineering evaluation for the Lakehurst Drive / Carrier Drive Development project and prepared this report in accordance with our Proposal No. 9792G dated May 7, 2019. The purpose of this study was to develop recommendations to guide site preparation and design of structure foundations and pavements. This report describes our methodology, documents our field and laboratory test results and presents our recommendations regarding the geotechnical engineering aspects of the project.

GEC appreciates the opportunity to be of service to you on this project. If you have any questions concerning the contents of this report, or if we may be of further assistance, please contact us.

Sincerely,

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS, INC.
Certificate of Authorization No. 00005882

CHRISTOPHER P. MEYER
LICENSE
No 49328
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

Not available for signing.

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This Report has been digitally signed and sealed by Christopher P. Meyer, P.E. on the date adjacent to the seal. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

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1.0 SITE LOCATION AND DESCRIPTION

> The project site consists of an approximately 9.1-acre parcel… in Orange County, Florida…

The project site consists of an approximately 9.1-acre parcel that is located east of Universal Boulevard, 0.75 miles northeast of the I-4 and West Sand Lake Road interchange in Orange County, Florida as shown on **Figure 1**. The site was historically utilized as a commercial facility prior to being demolished and cleared in 2011. The parcel is now undeveloped with grass and sparse tree coverage.

The United States Geological Survey (USGS) Lake Jessamine, Florida Quadrangle map, reproduced on **Figure 1**, and topographic survey data of the Subject Property, provided by NV5, indicates the natural ground surface elevation in the vicinity of the project site ranges from approximately +99 to +103 feet NGVD (1929).

2.0 PROJECT DESCRIPTION

We understand that project plans, provided by DRMP, Inc. include the construction of 3- to 4-story apartments with associated paved parking/drive areas, a 4-story parking garage, and a stormwater pond. Final site grades are anticipated to be 0 to 2 feet above existing grades. The stormwater pond is proposed near the southwest quadrant of the property.

Preliminary maximum structural loads provided by Select Structural on November 3, 2019 are as follows:

• 3- to 4-story apartments	Column footings:	50 kips
	Wall footings:	5 klf
• 4-story garage	Column footings:	1,000 kips
	Wall footings:	50 klf

If any of the above project information is incorrect, please notify us so that we can evaluate whether the changes in design affect our recommendations.

3.0 NRCS SOIL SURVEY

The Natural Resources Conservation Service (NRCS) Soil Survey of Orange County, Florida was reviewed for near-surface soil and groundwater information. The NRCS Soil Survey map is shown on **Figure 1**. The soils near the project site are summarized in **Table 1**.

Table 1
Summary of NRCS Soil Survey Soil Units

Soil Unit No.	Soil Name	Depth (in)	Soil Description	Unified Classification Symbol	Depth to Seasonal High Groundwater (ft)	Hydrologic Group
20	Immokalee fine sand	0 – 5 5 – 35 35 – 67 67 – 80	Fine sand Fine sand, sand Fine sand, sand Fine sand, sand	SP-SM, SP SP-SM, SP SP-SM, SM SP-SM, SP	0.5 – 1.0	B/D
50	Urban land, 0 to 2 percent slopes	0 – 6 6 – 36 36 – 80	Cemented material Paragravelly sand, sand Paragravelly sand, paragravelly fine sand, fine sand, sand	--- SP-SM SM, SP-SM	+2.0 – 0.0	---

The NRCS soil survey depicts Urban land (Soil Unit No. 50) as the predominant soil type within the study area. Urban Land refers to areas where most of the ground surface is covered by asphalt, concrete, buildings and other impervious surfaces that modify surface/subsurface drainage and obscure or alter the soils so that their identification is not possible. The depth to seasonal high groundwater level for this soil unit is typically dependent upon the functioning of existing drainage systems.

Information contained in the NRCS Soil Survey is very general and may be outdated. Therefore, it may not be reflective of actual soil and groundwater conditions, particularly if recent development in the site vicinity has modified soil conditions or surface/subsurface drainage. The information obtained from recent soil borings provides a better characterization of actual site conditions.

4.0 CENTRAL FLORIDA GEOLOGY

Central Florida geology is conducive to sinkhole activity, which is referred to as *karst* geology. The relative risk of sinkhole activity increases in areas of high recharge to the Floridan aquifer. The USGS map entitled *Recharge and Discharge Areas of the Floridan Aquifer in the St. Johns River Water Management District and Vicinity, Florida, 1984* indicates the site lies in an area of low to moderate recharge; therefore, we can conclude that the risk of sinkhole formation at the site is relatively low compared to the rest of Central Florida. A site-specific evaluation of sinkhole risk, including deep soil borings, was not included in this investigation. Please refer to **Appendix A** for a more detailed discussion of Central Florida geology and sinkhole risk.

Artesian groundwater conditions can be predicted based on comparison of the Floridan aquifer potentiometric surface and ground surface elevations. The Florida Geological Survey map entitled *The Upper Floridan Aquifer Potentiometric Surface Contours, September, 2016* (the most recent

map available) indicates the potentiometric surface of the Floridan aquifer at the site is approximately +55 to +60 feet NGVD. Since the existing ground surface at the site ranges from +99 to +103 feet NGVD, artesian flow conditions are not anticipated. Additionally, artesian conditions were not encountered in our soil borings.

5.0 SUBSURFACE EXPLORATION

In addition to consulting published sources, GEC conducted an exploration of the project site to evaluate subsurface conditions. Our field exploration is summarized in **Table 2**. Please refer to **Appendix B** for a description of the field exploration methods used for this investigation.

Table 2
Site Exploration Summary

Project Element	[1]Exploration Method	Quantity	Depth Explored (feet)	Figure No.
Structures	SPT Borings (SPT-3 to SPT-14)	12	40	4
	Deep SPT Borings (3A, 7A and 10A)	3	75 – 85	6
	DCP Borings (DCP-1 and DCP-2)	2	3.5 - 6	3
Roadway	HA Borings (HA-1 to HA-15)	15	3.5 - 7	3
	Manual Muck Probes (50-ft Grid Pattern)	9	---	2
Pond/Roadway	MA Borings (AB-1 to AB-5)	5	5 – 10	7

1. SPT = Standard Penetration Test; HA = Hand Auger; MA = Machine Auger; DCP = Dynamic Cone Penetrometer

The locations of the field activities listed in **Table 2** are shown on the site plan in **Figure 2**. These locations were not surveyed; they were estimated by using Global Positioning System (GPS) coordinates provided by a handheld GPS unit. The approximate method used to locate them is sufficient to meet the intent of our study.

6.0 LABORATORY TESTING

Selected soil samples retrieved from the borings were tested in accordance with Florida Standard Testing Methods (FM), AASHTO (American Association of Highway and Transportation Officials) and American Standard Testing Methods (ASTM). Our laboratory testing program is summarized on the following table:

Table 3
Summary of Laboratory Testing Program

Type of Test	Test No.
Percent Fines	AASHTO T 88
Atterberg Limits	AASHTO T 89/90
Organic Content	FM 1-T 267
Natural Moisture Content	AASHTO T 265
Consolidation	ASTM D2435
Unit Weight	ASTM D7263
Specific Gravity	ASTM D854

The results of our laboratory tests are shown adjacent to the soil profiles on the Boring Results sheets (**Figures 3 - 7**).

6.1 Consolidation Testing

A portion of the undisturbed samples obtained at the boring locations were sampled for one-dimensional consolidation testing in general accordance with ASTM-D2435. Undisturbed samples selected for testing are carefully trimmed and placed in the fixed ring consolidometer. A seating pressure of about 100 psf is applied and the sample is inundated in water. The sample submergence is maintained throughout the test.

The sample is then incrementally loaded and deflections are monitored. Each incremental load is maintained until internal pore pressures are dissipated as indicated by a flattening of the time-deflection curve. A rebound of the sample is provided at a selected load increment when the sample is unloaded and reloaded to obtain further details of the loading characteristics of the soil.

The data obtained during incremental loading is reduced and a semi-log plot of sample void ratio versus applied stress is created. A copy of this curve is included in **Appendix C** of this report. This curve is utilized to estimate the magnitude of settlement that will be induced by anticipated site loadings. The curve is also used to estimate the pre-consolidation pressure (P_C) and the over-consolidation ratio (OCR) of the soils tested. The results of our consolidation tests and associated laboratory soil classification tests are summarized in **Appendix C**.

7.0 SUBSURFACE CONDITIONS

The soil and groundwater conditions encountered are summarized in this section. Please refer to **Figures 3 - 7** for a detailed description of the subsurface profile at each boring location shown on **Figure 2**. The results of selected laboratory tests are shown adjacent to the subsurface profiles at

the depth the samples were obtained.

7.1 Subsurface Profile

Table 4 summarizes the subsurface conditions encountered in the SPT and DCP borings performed within and adjacent to the proposed building footprints:

Table 4
General Subsurface Profile – Structures

Layer Depth (feet)	Soil Description	SPT Resistance (blows/foot)
0 to 6-13	Medium dense to very dense fine sand (SP), fine sand with silt (SP-SM) and silty fine sand (SM)	[1]HA to 58
6-13 to 28-33	Firm to hard sandy lean clay (CL) to fat clay (CH)	7 to 47
28-33 to 48-60	Loose to medium dense silty to clayey find sand (SM, SC)	8 to 26
48-60 to 75-85	Dense to very dense silty to clayey fine sand (SM, SC) to hard sandy clay (CL)	31 to 50/0”[2]

1. Hand augered for utility clearance.
2. 50 blows for 0 inches of penetration

The following notable exceptions were encountered:

- Limerock (likely former limerock base material) was encountered at SPT-4 from ground surface to 2 feet below existing grade.
- A layer of mucky fine sand was encountered at DCP-1 from ground surface to 1 foot below existing grade.

The auger borings (HA-1 to HA-15 and AB-1 to AB-5) generally encountered fine sand with varying silt content (SP, SP-SM and SM) from the existing ground surface to the boring termination depths of 3.5 to 10 feet below the existing ground surface. The following exceptions where encountered:

- A layer of mucky fine sand was encountered at HA-2, HA-4, HA-6 and HA-9 at various depths ranging from ground surface to 3 feet below existing grade.
- A layer of clayey fine sand was encountered at HA-1, HA-6, HA-13, HA-14, AB-1, AB-2, and AB-5 at depths ranging from 4 to 10 feet below existing grade.
- Limerock (likely former limerock base material) was encountered at HA-5,HA-6, AB-1 and AB-4 at depths ranging from ground surface to 2 feet below existing grade.

Manual muck probes were performed within the wetland area on the southern portion of the site at approximately 50-foot horizontal intervals. A summary of the manual muck probe results is included in the following table:

Table 5
Summary of Manual Muck Probes Results

Location	Standing Water Depth (June 2019) (ft)	Range of Estimated Surficial Organic Soil Thickness (ft)
Wetland	0.0	0.1 - 0.4

Specific muck probe results at individual probe locations are presented on **Figure 2.**

7.2 Groundwater Levels

Table 6 summarizes the measured and estimated seasonal high groundwater levels in building and pavement locations.

Table 6
Groundwater Levels

Location	Boring ID Nos.	Measured Depths (feet)	Estimated Seasonal High Depths (feet)
Structures	SPT-3 to SPT-14, DCP-1 and DCP-2	0.6 to 6.6	AGS[1] to 3.6
Roadway	HA-1 to HA-15, AB-3, AB-4, AB-5	1.1 to 6.4	AGS[1] to 3.5
Pond	AB-1 and AB-2	3.6 to 4.3	2.0 to 2.3

1. AGS = Above Ground Surface

Seasonal high groundwater levels are estimated to range from above the existing ground surface (indicated by "AGS" shown adjacent to the boring profile) in the southern "wetland" portion of the site and in the vicinity of SPT-3 and SPT-11 on the west side of the property down to 3.6 feet below the existing grade in the remaining areas of the property. Standing water was observed in the immediate vicinity of SPT-11.

Please refer to **Figures 3 - 7** for measured and estimated seasonal high groundwater levels at each boring location shown on **Figure 2**.

8.0 DESIGN RECOMMENDATIONS

GEC developed the following recommendations based on the results of our field and laboratory testing program and the provided structural loads. These recommendations are only applicable if the site is prepared in accordance with the construction requirements provided in **Section 9.0** of this report and **Appendix E**.

8.1 Foundation Design and Settlement for Parking Garage

> The site is underlain by an approximately 20-foot thick… clay layer…

The site is underlain by an approximately 20-foot thick sandy clay to fat clay layer (CL, CH) from about 13 feet deep to 33 feet deep at the site. Clays, as opposed to fine sands that are more typical in Central Florida, can result in excessive long-term soil settlements after structural loads are applied (during construction).

> …this clay layer exhibits… settlements of 2.5 to 4 inches… for the parking garage structure…

Based on our soil settlement analyses, this clay layer exhibits long-term consolidation settlements of 2.5 to 4 inches when subjected to the proposed high structural loads (1,000 kips and 50 klf) for the parking garage structure, which exceeds the typically allowed settlement tolerance of 1 inch. Example calculations of our settlement analyses are provided in **Appendix D**. GEC also performed a soil surcharge analyses to determine if the clay layer could be surcharged to reduce the clay settlement to less than 1 inch; however, even with using wick drains to accelerate consolidation, due to the slow drainage characteristic properties of the clay layer, the surcharge would take about 1 year to bring the long-term settlements down to less than 1 inch. We understand that this schedule delay was not acceptable and that an alternate intermediate or deep foundation system was needed for the parking garage structure.

> We recommend that an intermediate foundation system… be used for the parking garage structure.

We recommend that an intermediate foundation system (such as rigid inclusions or stone columns) be used for the parking garage structure. This foundation system should be designed to transfer the structural loads to below the 20-foot clay thick layer and to increase the net bearing capacity used for foundation design. This system should be designed for tolerable settlements for the garage structure. Rigid inclusion and stone column foundations are normally designed by specialty foundation contractors due to the proprietary nature of their individual foundation systems. We understand that you have contacted several foundation contractors to obtain foundation designs and cost estimates for the garage.

8.2 Foundation Design and Settlement for Apartment Buildings

> We recommend using… net soil bearing pressure of 4,000 psf in footing design for the apartment buildings.

Based on the provided lighter apartment building structural loads, we anticipate that the site will suitable for support of the proposed 3 to 4-story apartment buildings on conventional shallow isolated spread footings, continuous strip footings, or monolithic slab with thickened edges. We recommend using a maximum net soil bearing pressure of 4,000 psf in footing design for the apartment buildings.

Footing dimensions should be a minimum of 24 inches for isolated spread footings and 18 inches for strip footings. Footings should bear a minimum of 18 inches below the adjacent finished exterior grade. Design monolithic slabs so that the bottom of the thickened slab sections bear at least 12 inches below the adjacent finished exterior grade.

Our settlement analysis indicates that the apartment shallow foundations… will experience total settlements of less than 1 inch.

Our settlement analysis indicates that the apartment shallow foundations designed in accordance with the above recommendations, and assuming allowable bearing pressures of 4,000 psf, will experience total settlements of less than 1 inch. Approximately one half of this settlement is anticipated to occur over a period of about 1 year due to the long-term nature of clay consolidation settlement. Example calculations of our settlement analyses are provided in **Appendix D**.

8.3 Pavement Design

The results of the subsurface investigation indicate that the site can be made suitable for support of conventional flexible (asphalt) or rigid (concrete) pavement sections. **Table 7** presents our recommended pavement sections. These pavement sections are typical of similar projects in Central Florida and are not based on traffic loading information, which was not available when this report was prepared.

Table 7
Recommended Pavement Designs

Traffic Type	Surface Course	Base Course	Stabilized Subgrade
Light Duty (parking stalls)	1.5 inches Asphalt Surface Course	6 inches limerock (LBR 100 min)	12 inches (LBR=40)
		7 inches reclaimed concrete aggregate (LBR 150 min)	12 inches (LBR=40)
		8 inches soil-cement (300 psi min)	Stabilization not required*
Medium Duty (parking lot drive lanes and perimeter roads)	2.0 inches Asphalt Surface Course	8 inches limerock (LBR 100 min)	12 inches (LBR=40)
		9 inches reclaimed concrete aggregate (LBR 150 min)	12 inches (LBR=40)
		10 inches soil-cement (300 psi min)	Stabilization not required*
Heavy Duty (truck turning movements, loading areas)	6.0 inches Concrete (4,000 psi 28-day min)	Base course not required	12 inches (95% Modified Proctor Compaction only) Stabilization not required*

* Free draining subgrade soils (e.g. sands with <12% fines) are recommended to promote drainage of infiltrating water.

If traffic loading information becomes available, we should review it to confirm the suitability of these typical pavement sections.

Additional recommendations for using reclaimed concrete aggregate (RCA) are provided in **Appendix E**.

8.4 Pavement Base Clearance

Vertical separation of the base from the water table is required to prevent pavement damage created by inundation of the base course during periods of heavy rainfall. We recommend a minimum of 24 inches of separation between the bottom of the pavement base and the estimated seasonal high groundwater level. If this vertical clearance cannot be provided, limerock should not be used for the base material. A soil-cement or asphalt base course can be constructed with a minimum vertical base clearance of 12 inches. Underdrains will be required to artificially lower the water table where the seasonal high groundwater level is within 12 inches of the bottom of the base.

These conclusions are contingent upon preparation of proposed pavement areas in accordance with the requirements of **Appendix E.**

8.5 Wall/Lateral Earth Pressure Design Parameters

Lateral earth pressure for design of walls and other below-grade structures should be calculated using the following soil parameters:

- Friction Angle (degrees) = 30
- Moist Unit Weight (pcf) = 110
- Saturated Unit Weight (pcf) = 115
- Buoyant Unit Weight (pcf) = 55
- Angle of Wall Friction (degrees) = 15
- Base Friction Coefficient = 0.50

The active, passive and at-rest earth pressures can be calculated using these parameters. The earth pressure condition will be dependent on wall design. Frictional resistance at the foundation base can be calculated by multiplying the base contact pressure by the friction coefficient. The above soil parameters do not include a factor of safety; appropriate safety factors should be applied in the wall design calculations.

The estimated seasonal high groundwater levels included in this report can be used for guidance when determining the design high water level for structure design. If drainage through the wall

cannot be assured, we recommend using the ground surface elevation at the top of the structure for the design high water level.

GEC should review the final wall analysis and plans to verify that our recommendations have been correctly applied in the wall design.

8.6 Uplift Resistance for Submerged Structures

Structures below the water table, such as a pump station wet well, will be subjected to uplift (buoyant) forces. The components resisting buoyancy include the following:

- Weight of the structure
- Weight of the soil above the structure
- Shear forces on planes that radiate vertically from the perimeter of the structure to the ground surface

The allowable unit shearing resistance (F_a) in psf may be determined by the following formula:

- $F_a = K_o \gamma_m h (⅔ \tan \Phi) /S.F.$ (above water table)

- $F_a = K_o (\gamma_s - \gamma_W) h (⅔ \tan \Phi) /S.F.$ (below water table)

Where,

K_o = coefficient of earth pressure at rest = 0.5
γ_m = unit weight of moist soil = 110 pcf
γ_s = saturated unit weight of soil = 115 pcf
γ_W = unit weight of water = 62.4 pcf
h = vertical depth below grade at which shearing resistance is calculated
Φ = soil internal friction angle = 30 degrees
S.F. = safety factor = 2

These design parameters are based on the assumption that fill above and/or surrounding the structure will achieve the construction requirements provided in **Appendix E**.

8.7 Stormwater Ponds

Pond sites with relatively shallow surficial groundwater levels typically necessitate wet detention pond systems. However, dry retention pond systems may be feasible on pond sites with relatively deep groundwater, i.e., deeper than 3 to 4 feet.

The soil conditions encountered in the stormwater pond location are summarized in **Table 8.**

Table 8
General Subsurface Profile
Based on Pond Borings

Layer Depth (feet)	Soil Description
0 to 3-5	Fine sand (SP) to fine sand with silt (SP-SM)
3-5 to 5.5-7	Silty fine sand (SM)
5.5-7 to 10	Clayey fine sand (SC) to sand clay (CH)

The measured and estimated groundwater levels in the pond borings are summarized in **Table 9.**

Table 9
Groundwater Levels in Pond Borings

Location	Boring ID Nos.	Measured Depths (ft) (Nov, 2019)	Estimated Seasonal High Depths (ft)
Pond	AB-1	3.6	2.0
Pond	AB-2	4.3	2.3

Our estimated horizontal permeability values are provided in **Table 10**.

Table 10
Permeability Test Results

Soil Type	Estimated Horizontal Permeability (ft/day)
SP	10
SP-SM	3
SM	1
SC to CH	<0.1

For unconfined aquifer parameters we recommend using the information provided above and a fillable soil porosity of 0.25 ft/ft.

*The pond design recommendations presented above are provided assuming that all pond construction and maintenance recommendations presented below and in **Appendix E** are properly implemented.*

8.8 Suitability of Soil Excavated From Pond for Use as Fill

The following are our recommendations for the encountered soils used a fill material:

- Soils indicated as fine sand and fine sand with silt (SP, SP-SM) are suitable for use as engineered fill in structure and pavement areas.

- Soils indicated as silty sand (SM) and clayey sand (SC), with fines less than 20% and plastic index less than 10, are adaptable for use as fill, but will require drying to near optimum moisture content prior to compaction. Special procedures such as aeriating or mixing with fine sand (SP) may be required to achieve minimum compaction criteria.

- Soil with fines content greater than 20% and/or a plastic index greater than 10 (ML, MH, SC, CL, CH) are not suitable for use as engineered fill and should be placed in landscaped areas or removed from the site.

- Soils indicated as mucky fine sand or muck (PT) with an organic content greater than 5% (PT) are not suitable as structural fill. However, it may be possible to use these soils in landscape areas where structural or vehicle loads will not be anticipated. If utilized in landscaped areas, these soils may need to have clean sands mixed with them to meet the project specifications or will need to be removed from the site.

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 General Site Preparation

Our recommendations for general construction activities are presented in **Appendix E**, which addresses the following requirements:

Site Dewatering
Site Preparation
Fill Selection, Placement and Compaction
Foundation Subgrade Preparation

Pavement Subgrade Preparation
Excavation Safety
Stormwater Pond Construction and Maintenance
Reclaimed Concrete Aggregate Base Material

9.2 Unsuitable Soil Excavation

> The surficial muck… should be completely removed beneath structure and pavement locations…

The surficial muck depicted on the soil boring profiles (DCP-2, HA-2 and HA-9) should be completely removed beneath structure and pavement locations to a minimum of 5 feet horizontally beyond the proposed structure and pavement limits. The mucky sands encountered at Boring HA-4 from 2.5 to 3 feet deep can remain in-place. Muck excavation should be performed "in the dry" so that removal can be verified and to facilitate excavation backfill compaction. It will be critical to adhere to the site dewatering, site preparation and fill selection, placement and compaction requirements in **Appendix E** to ensure adequate muck removal.

In addition, full-time inspection of demucking operations by an engineering technician working under the supervision of a Florida-licensed geotechnical engineer is strongly recommended. Full-time inspection is needed to verify complete removal, and to prevent the removal of suitable soil that would unnecessarily increase construction costs. Allowing construction equipment operators to determine muck removal limits commonly results in cost overruns that could be avoided if a trained inspector is on-site to properly classify excavated soils.

9.3 Construction Materials Quality Control Testing

A comprehensive quality control program is required to verify that site preparation, pavement and building construction is conducted in accordance with the recommendations in this report including **Appendix E**. Materials testing and inspection services should be provided by **Geotechnical and Environmental Consultants, Inc.** due to our knowledge of the site conditions and understanding of the intent of our recommendations.

10.0 REPORT USE AND LIMITATIONS

This section of the report presents important information regarding the proper use of this report, our investigative methods and the limitations of this study. The test data, conclusions and recommendations presented in this report should be reviewed and applied with these limitations in mind.

Subsurface Variability. The analyses and recommendations contained in this report are based in part on the data obtained from a limited number of soil samples and groundwater measurements obtained from widely-spaced borings/piezometers. The sampling methods used indicate subsurface conditions only at the specific boring locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Borings cannot be relied upon to accurately reflect the variations that usually exist between boring locations and these variations may not become evident until construction. Conditions at the boring locations can change over time. Groundwater levels fluctuate seasonally, and soil conditions can be altered by earthmoving operations.

Soil Stratification. The depths and thicknesses of the subsurface strata indicated on the boring logs were interpolated between samples obtained at different depths in the borings. The actual transition between soil layers may be different than indicated. *These stratification lines were used for our analytical purposes. Earthwork quantity estimates based on the results of the borings will vary from the actual quantities measured during construction.*

Groundwater Measurements. Groundwater levels can vary seasonally and with changes in subsurface conditions between boring and/or piezometer locations. Alterations in surface and/or subsurface drainage brought about by site development can also affect groundwater levels. *Therefore, groundwater depths measured at different times or at different locations on the site can be expected to vary from those measured by GEC during this investigation.*

Groundwater Predictions. For purposes of this report, estimated seasonal high groundwater levels are defined as groundwater levels that are anticipated at the end of the wet season during a "normal rainfall" year under pre-development site conditions. We define a "normal rainfall" year as a year in which rainfall quantity and distribution were at or near historical averages.

Construction Variations. If variations from the subsurface conditions described in this report do become evident during construction, GEC should be retained to reevaluate this report's conclusions and recommendations, and modify the recommendations included in this report, if needed, in light of such changes.

Plans/Specifications Review. GEC should be provided with the construction plans and specifications prior to bidding so that we can verify that the recommendations presented in this report were correctly interpreted and incorporated into the plans and specifications. The recommendations in this report were not written in specification language and are not intended to be used verbatim as a part of the plans and specifications. This report should not be wholly incorporated into the project contract documents.

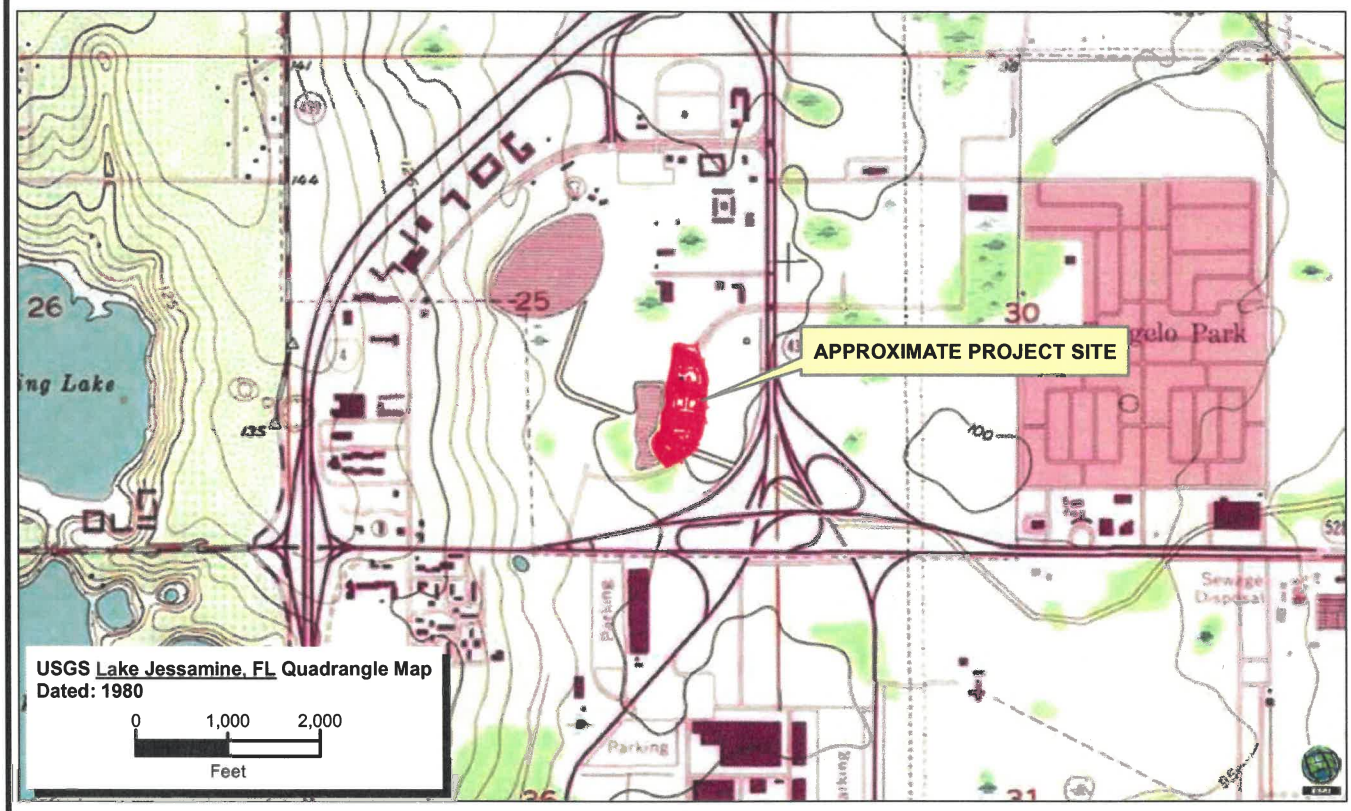
Design Changes. The conclusions or recommendations of this report should be disregarded if the nature, design, or location of the facilities is changed. If such changes are contemplated, GEC should be retained to review the new plans to assess the applicability of this report in light of the proposed changes.

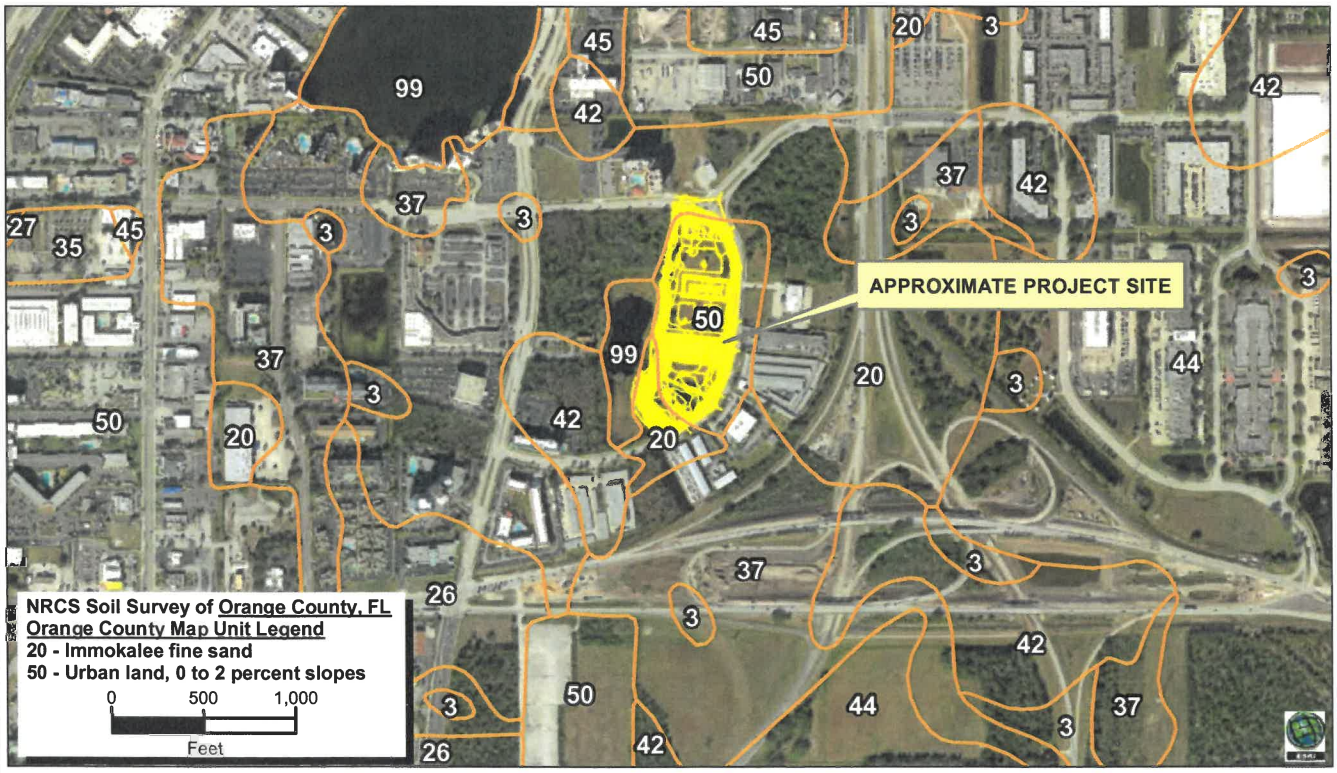
Contamination Exclusion. The sole purpose of the borings performed by GEC at this site was to obtain indications of subsurface conditions as part of a geotechnical exploration program. GEC has not evaluated the site for the potential presence of contaminated soil or groundwater, nor have we subjected any soil samples to analysis for contaminants.

Report Reliance. GEC has prepared this report for the exclusive use of our clients, Silver Hills Development, Inc. and DRMP, Inc., and for specific application to this project. GEC is not responsible for any third party's interpretation or use of this report's subsurface data, engineering analysis or recommendations without our written authorization.

Standard of Care. GEC has performed the services described in this report in a manner consistent with that level of care and skill ordinarily exercised by members of our profession currently practicing in Central Florida. No other representation is made or implied in this document.

FIGURES





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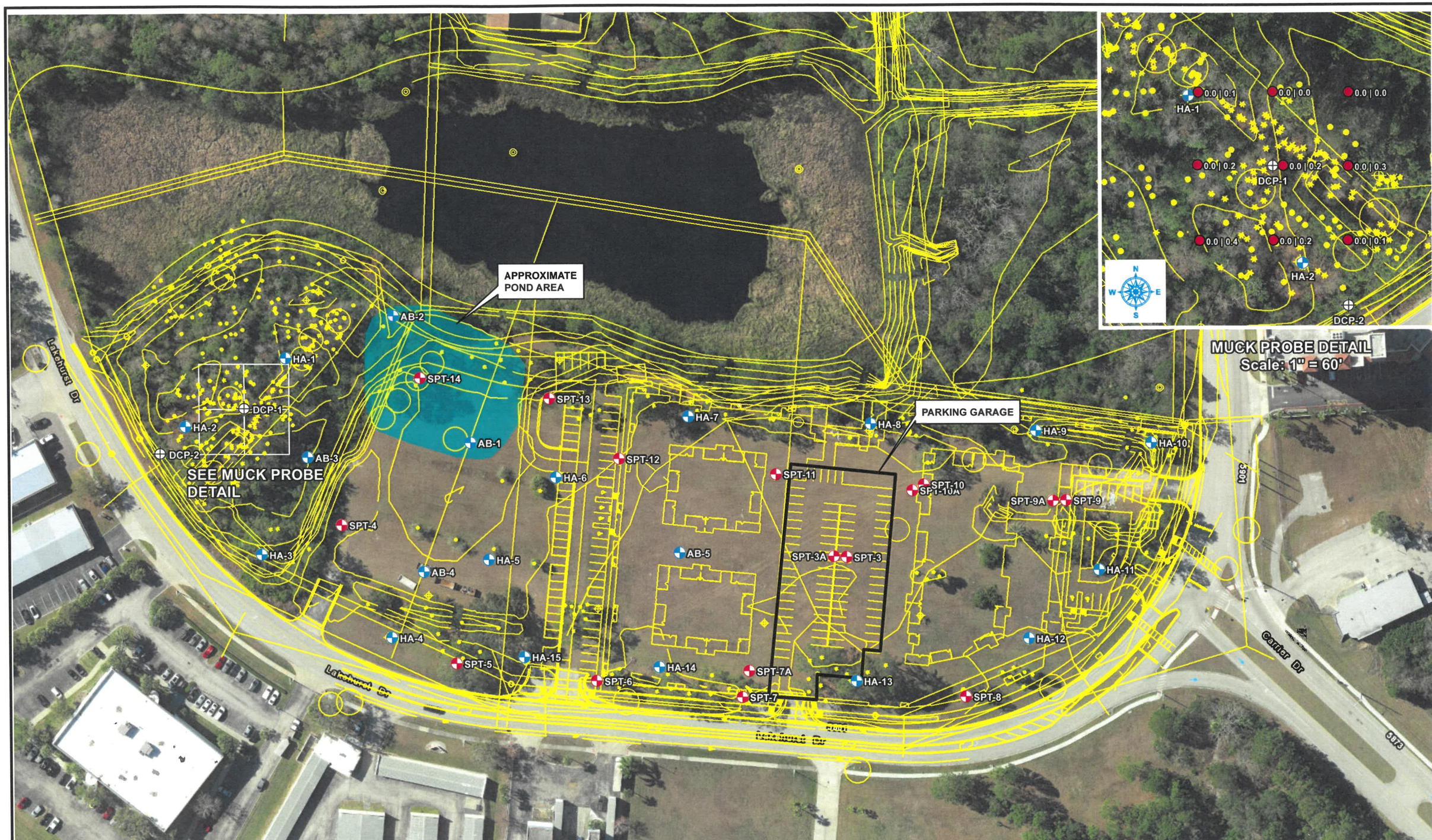
PROJECT NO.	4406G
DATE	7/16/2019
DRAWN BY	SKR
CHECKED BY	BMM
CHECKED BY	CPM

USGS QUADRANGLE AND NRCS SOIL SURVEY MAPS

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

FIGURE NO. 1

J:\Jobs - Drafting\4406G\4406Gquad.mxd 7/16/2019



APPROXIMATE SPT BORING LOCATION

APPROXIMATE AUGER BORING LOCATION

APPROXIMATE DCP LOCATION

SURFICIAL MUCK THICKNESS (FT.)

STANDING WATER DEPTH (FT.)

0.0 | 0.1 APPROXIMATE MUCK PROBE LOCATION

0 50 100
Feet

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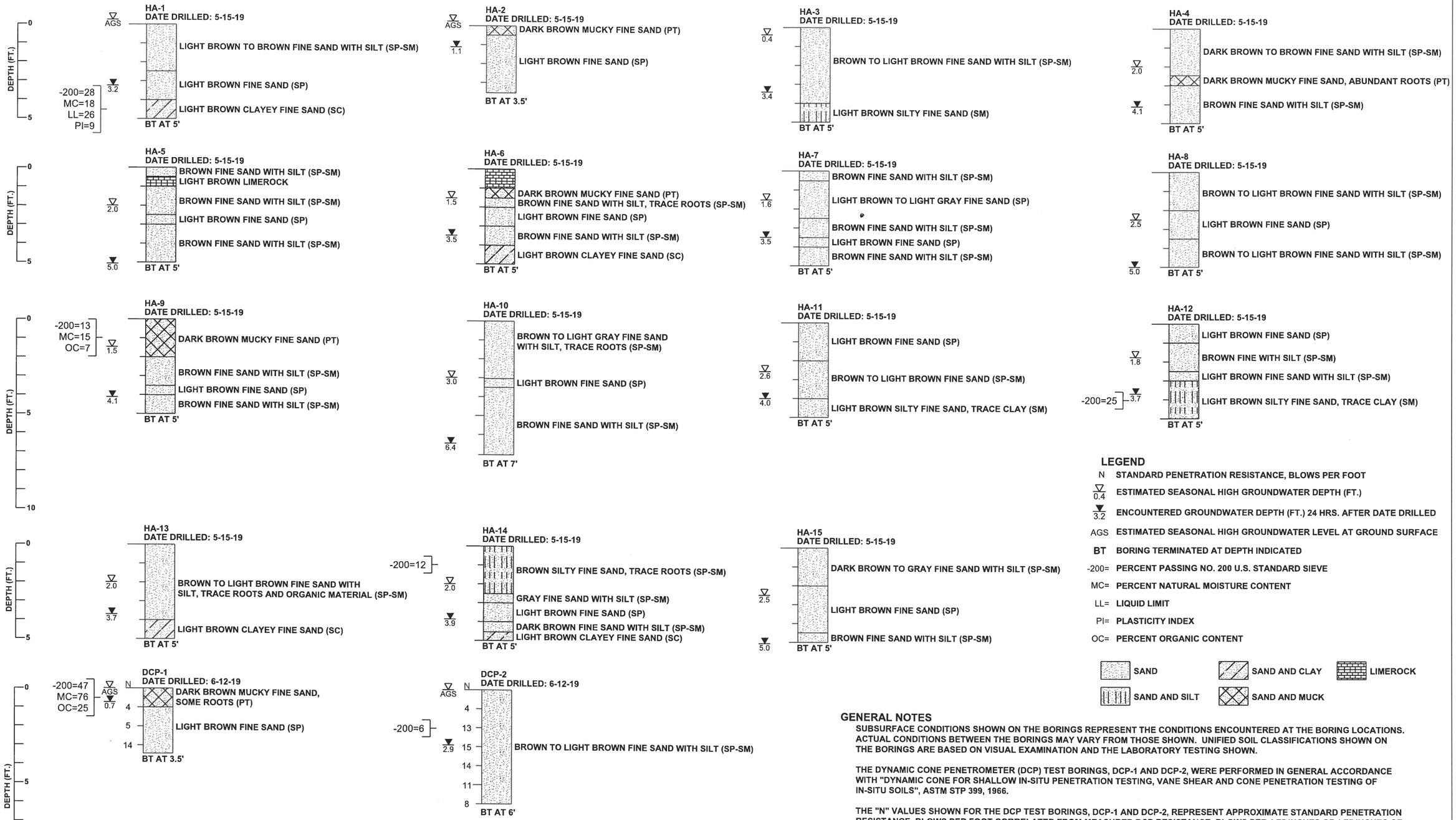
BORING LOCATION PLAN

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

FIGURE NO. 2

J:\Jobs - Drafting\4406G\4406G siteplan2.mxd 12/18/2019



LEGEND

N	STANDARD PENETRATION RESISTANCE, BLOWS PER FOOT
▽ 0.4	ESTIMATED SEASONAL HIGH GROUNDWATER DEPTH (FT.)
▼ 3.2	ENCOUNTERED GROUNDWATER DEPTH (FT.) 24 HRS. AFTER DATE DRILLED
AGS	ESTIMATED SEASONAL HIGH GROUNDWATER LEVEL AT GROUND SURFACE
BT	BORING TERMINATED AT DEPTH INDICATED
-200=	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
MC=	PERCENT NATURAL MOISTURE CONTENT
LL=	LIQUID LIMIT
PI=	PLASTICITY INDEX
OC=	PERCENT ORGANIC CONTENT

SAND — SAND AND CLAY — LIMEROCK — SAND AND SILT — SAND AND MUCK

GENERAL NOTES

SUBSURFACE CONDITIONS SHOWN ON THE BORINGS REPRESENT THE CONDITIONS ENCOUNTERED AT THE BORING LOCATIONS. ACTUAL CONDITIONS BETWEEN THE BORINGS MAY VARY FROM THOSE SHOWN. UNIFIED SOIL CLASSIFICATIONS SHOWN ON THE BORINGS ARE BASED ON VISUAL EXAMINATION AND THE LABORATORY TESTING SHOWN.

THE DYNAMIC CONE PENETROMETER (DCP) TEST BORINGS, DCP-1 AND DCP-2, WERE PERFORMED IN GENERAL ACCORDANCE WITH "DYNAMIC CONE FOR SHALLOW IN-SITU PENETRATION TESTING, VANE SHEAR AND CONE PENETRATION TESTING OF IN-SITU SOILS", ASTM STP 399, 1966.

THE "N" VALUES SHOWN FOR THE DCP TEST BORINGS, DCP-1 AND DCP-2, REPRESENT APPROXIMATE STANDARD PENETRATION RESISTANCE, BLOWS PER FOOT CORRELATED FROM MEASURED DCP RESISTANCE, BLOWS PER 1.75 INCHES OF 1.75 INCHES OF PENETRATION. THE CORRELATION OF DCP TO SPT "N" VALUES IS BASED ON THE CALIBRATION CURVE FOR COASTAL PLAINS SOILS, ASTM SPT 399, 1966.

BORING LOCATIONS WERE NOT SURVEYED. BORING LOCATIONS WERE ESTABLISHED IN THE FIELD USING A SUB-METER ACCURACY GPS UNIT (TRIMBLE GEO 7X).

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	RELATIVE DENSITY
SANDS	0-4	VERY LOOSE
	4-10	LOOSE
	10-30	MEDIUM DENSE
	30-50	DENSE
	OVER 50	VERY DENSE

NON-GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	CONSISTENCY
SILTS, CLAYS, MUCK, PEAT	0-2	VERY SOFT
	2-4	SOFT
	4-8	FIRM
	8-15	STIFF
	15-30	VERY STIFF
	OVER 30	HARD

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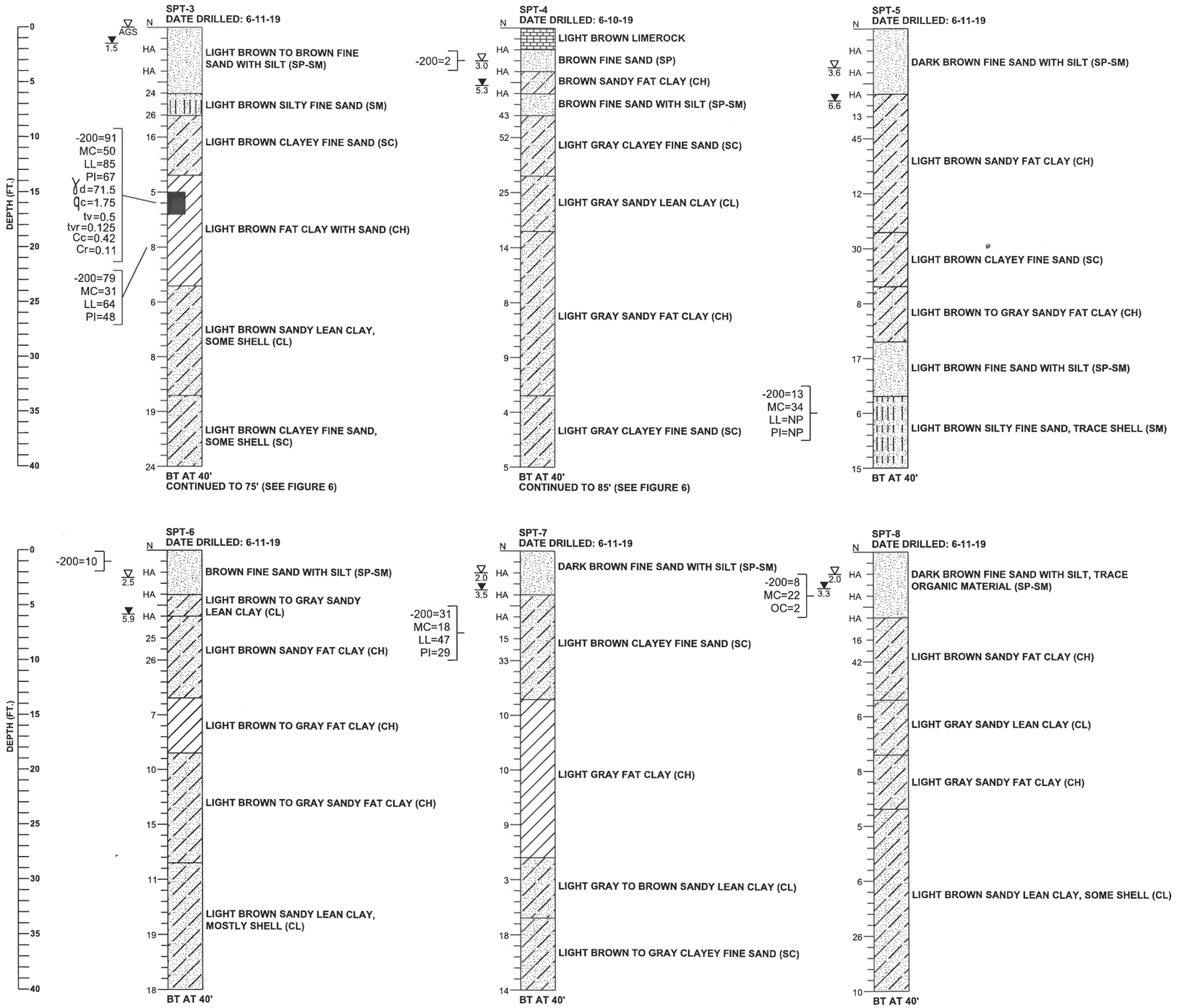
CHECKED BY	CPM

AUGER AND DCP BORING RESULTS

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

FIGURE NO. 3



LEGEND

N	STANDARD PENETRATION RESISTANCE, BLOWS PER FOOT
HA	HAND AUGERED FOR UTILITY CLEARANCE
▽ 3.0	ESTIMATED SEASONAL HIGH GROUNDWATER DEPTH (FT.)
▼ 5.3	ENCOUNTERED GROUNDWATER DEPTH (FT.) 24 HRS. AFTER DATE DRILLED
AGS	ESTIMATED SEASONAL HIGH GROUNDWATER LEVEL AT GROUND SURFACE
▬	UNDISTURBED SAMPLE (SHELBY TUBE)
BT	BORING TERMINATED AT DEPTH INDICATED
-200=	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
MC=	PERCENT NATURAL MOISTURE CONTENT
LL=	LIQUID LIMIT
PI=	PLASTICITY INDEX
OC=	PERCENT ORGANIC CONTENT
NP=	NON-PLASTIC
γd=	DRY UNIT WEIGHT (pcf)
qc=	UNCONFINED COMPRESSION STRENGTH (LAB) (tsf)
tv=	TORVANE SHEAR STRENGTH (LAB) (psf)
tvr=	REMOLDED TORVANE SHEAR STRENGTH (LAB) (psf)
Cc=	COMPRESSION INDEX (tsf)
Cr=	RECOMPRESSION INDEX (tsf)

SAND — SAND AND CLAY — LIMEROCK — SAND AND SILT — CLAY

GENERAL NOTES

SUBSURFACE CONDITIONS SHOWN ON THE BORINGS REPRESENT THE CONDITIONS ENCOUNTERED AT THE BORING LOCATIONS. ACTUAL CONDITIONS BETWEEN THE BORINGS MAY VARY FROM THOSE SHOWN. UNIFIED SOIL CLASSIFICATIONS SHOWN ON THE BORINGS ARE BASED ON VISUAL EXAMINATION AND THE LABORATORY TESTING SHOWN.

STANDARD PENETRATION TEST BORINGS WERE PERFORMED IN ACCORDANCE WITH ASTM D-1586. STANDARD PENETRATION RESISTANCES ARE SHOWN ON THE BORINGS AT THE TEST DEPTHS IN IN BLOWS PER FOOT UNLESS OTHERWISE NOTED.

BORING LOCATIONS WERE NOT SURVEYED. BORING LOCATIONS WERE ESTABLISHED IN THE FIELD USING A SUB-METER ACCURACY GPS UNIT (TRIMBLE GEO 7X).

SPLIT SPOON SAMPLER:
INSIDE DIAMETER: 1.375 IN.
OUTSIDE DIAMETER: 2.0 IN.
AVERAGE HAMMER DROP: 30 IN.
HAMMER WEIGHT: 140 LBS.

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	RELATIVE DENSITY
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	4-10	LOOSE
	10-30	MEDIUM DENSE
	30-50	DENSE
	OVER 50	VERY DENSE

NON-GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	CONSISTENCY
SILTS, CLAYS, MUCK, PEAT	0-2	VERY SOFT
	2-4	SOFT
	4-8	FIRM
	8-15	STIFF
	15-30	VERY STIFF
	OVER 30	HARD

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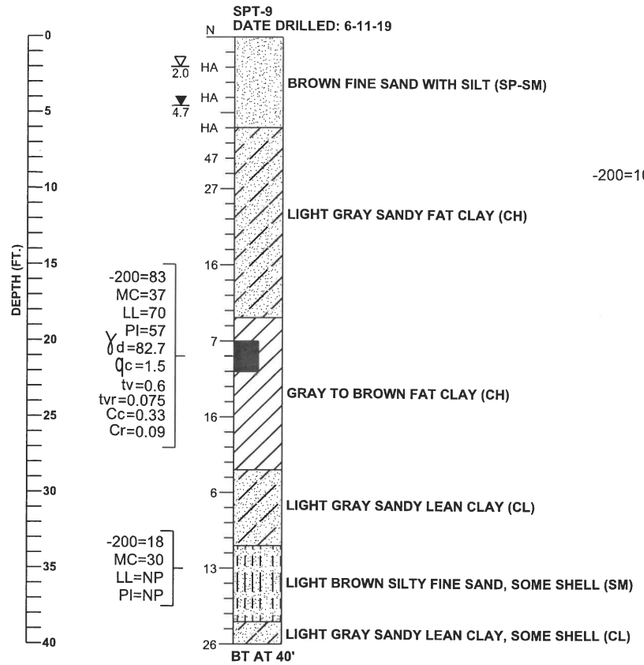
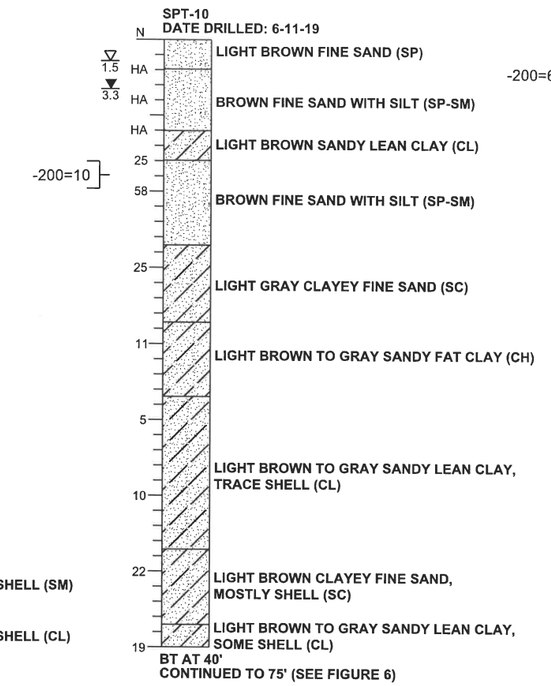
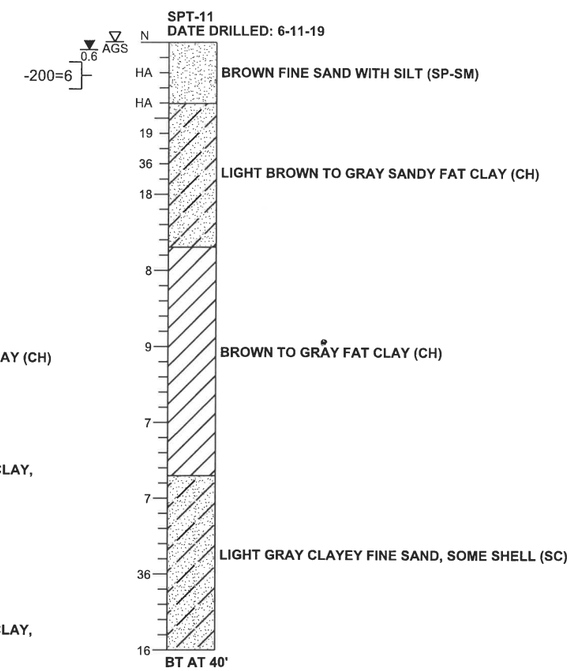
SPT BORING RESULTS

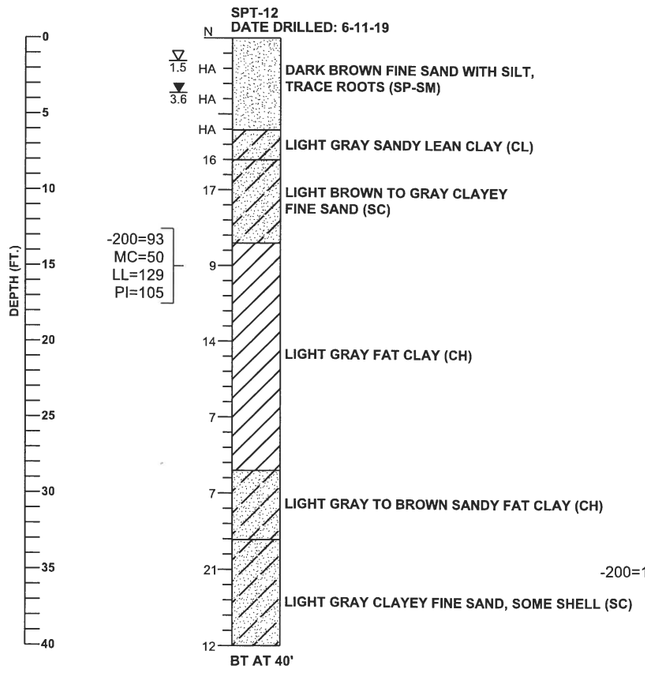
LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

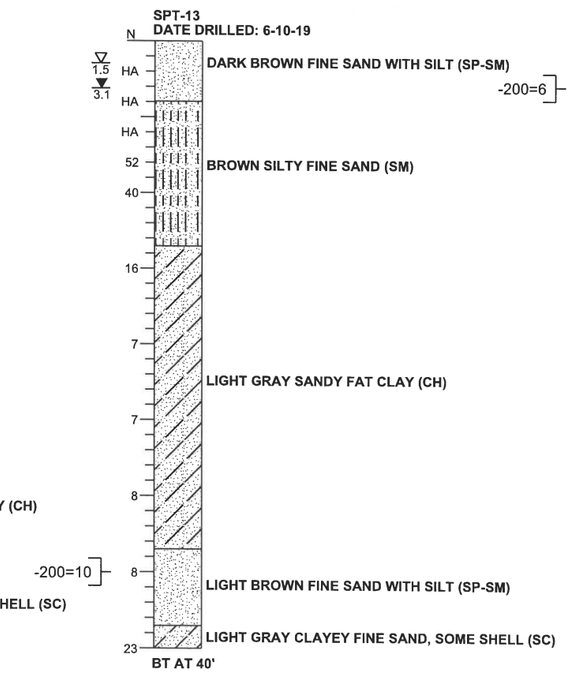
FIGURE NO. 4

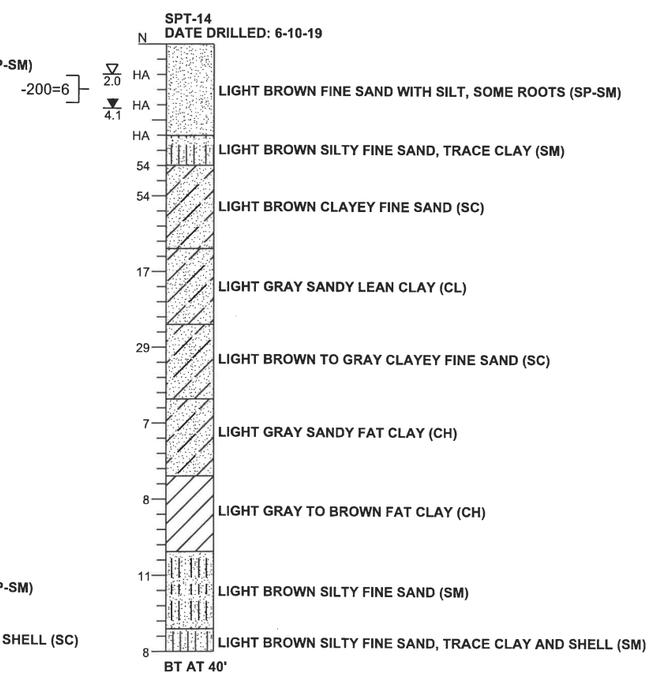












LEGEND

N	STANDARD PENETRATION RESISTANCE, BLOWS PER FOOT
HA	HAND AUGERED FOR UTILITY CLEARANCE
▽ 2.0	ESTIMATED SEASONAL HIGH GROUNDWATER DEPTH (FT.)
▼ 4.7	ENCOUNTERED GROUNDWATER DEPTH (FT.) 24 HRS. AFTER DATE DRILLED
AGS	ESTIMATED SEASONAL HIGH GROUNDWATER LEVEL AT GROUND SURFACE
▬	UNDISTURBED SAMPLE (SHELBY TUBE)
BT	BORING TERMINATED AT DEPTH INDICATED
-200=	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
MC=	PERCENT NATURAL MOISTURE CONTENT
LL=	LIQUID LIMIT
PI=	PLASTICITY INDEX
OC=	PERCENT ORGANIC CONTENT
NP=	NON-PLASTIC
γd=	DRY UNIT WEIGHT (pcf)
qc=	UNCONFINED COMPRESSION STRENGTH (LAB) (tsf)
tv=	TORVANE SHEAR STRENGTH (LAB) (psf)
tvr=	REMOLDED TORVANE SHEAR STRENGTH (LAB) (psf)
Cc=	COMPRESSION INDEX (tsf)
Cr=	RECOMPRESSION INDEX (tsf)

SAND — SAND AND CLAY
SAND AND SILT — CLAY

GENERAL NOTES

SUBSURFACE CONDITIONS SHOWN ON THE BORINGS REPRESENT THE CONDITIONS ENCOUNTERED AT THE BORING LOCATIONS. ACTUAL CONDITIONS BETWEEN THE BORINGS MAY VARY FROM THOSE SHOWN. UNIFIED SOIL CLASSIFICATIONS SHOWN ON THE BORINGS ARE BASED ON VISUAL EXAMINATION AND THE LABORATORY TESTING SHOWN.

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SPLIT SPOON SAMPLER:
INSIDE DIAMETER: 1.375 IN.
OUTSIDE DIAMETER: 2.0 IN.
AVERAGE HAMMER DROP: 30 IN.
HAMMER WEIGHT: 140 LBS.

CORRELATION OF STANDARD PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY OF SOIL

GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	RELATIVE DENSITY
SANDS	0-4	VERY LOOSE
	4-10	LOOSE
	10-30	MEDIUM DENSE
	30-50	DENSE
	OVER 50	VERY DENSE

NON-GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	CONSISTENCY
SILTS, CLAYS, MUCK, PEAT	0-2	VERY SOFT
	2-4	SOFT
	4-8	FIRM
	8-15	STIFF
	15-30	VERY STIFF
	OVER 30	HARD

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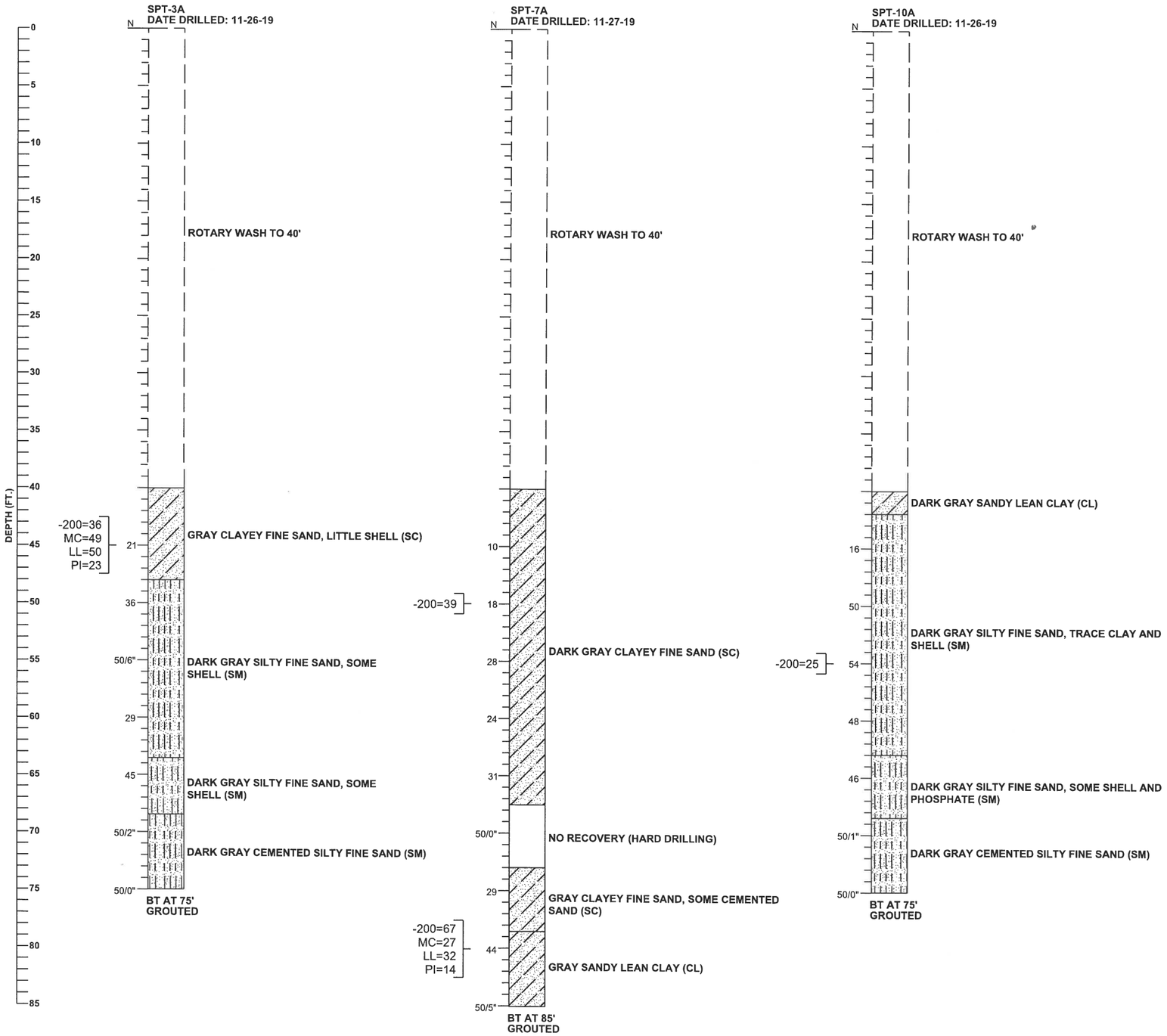
SPT BORING RESULTS

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

FIGURE NO. 5



LEGEND

N	STANDARD PENETRATION RESISTANCE, BLOWS PER FOOT
50/1"	NUMBER OF BLOWS FOR 1 INCH OF PENETRATION
BT	BORING TERMINATED AT DEPTH INDICATED
-200=	PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
MC=	PERCENT NATURAL MOISTURE CONTENT
LL=	LIQUID LIMIT
PI=	PLASTICITY INDEX

SAND

SAND AND CLAY

SAND AND SILT

GENERAL NOTES

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	4-10	LOOSE
	10-30	MEDIUM DENSE
	30-50	DENSE
	OVER 50	VERY DENSE

NON-GRANULAR SOILS	MANUAL HAMMER (SAFETY) N VALUE (blows per foot)	CONSISTENCY
SILTS, CLAYS, MUCK, PEAT	0-2	VERY SOFT
	2-4	SOFT
	4-8	FIRM
	8-15	STIFF
	15-30	VERY STIFF
	OVER 30	HARD

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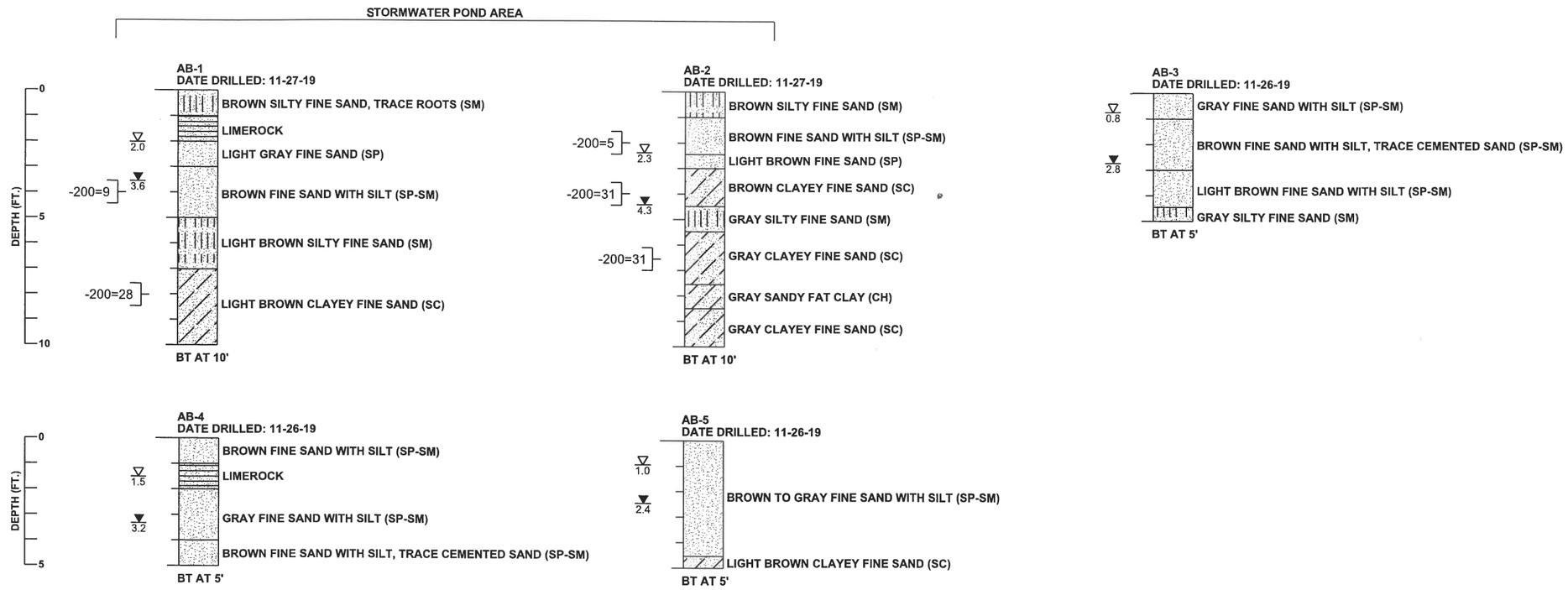
CHECKED BY	CPM

SPT BORING RESULTS

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

FIGURE NO. 6



LEGEND

- ▽ 2.0 — ESTIMATED SEASONAL HIGH GROUNDWATER DEPTH (FT.)
- ▼ 3.6 — ENCOUNTERED GROUNDWATER DEPTH (FT.) 24 HRS. AFTER DATE DRILLED
- BT — BORING TERMINATED AT DEPTH INDICATED
- -200= — PERCENT PASSING NO. 200 U.S. STANDARD SIEVE
- SAND
- SAND AND CLAY
- SAND AND SILT
- LIMEROCK

GENERAL NOTES

SUBSURFACE CONDITIONS SHOWN ON THE BORINGS REPRESENT THE CONDITIONS ENCOUNTERED AT THE BORING LOCATIONS. ACTUAL CONDITIONS BETWEEN THE BORINGS MAY VARY FROM THOSE SHOWN. UNIFIED SOIL CLASSIFICATIONS SHOWN ON THE BORINGS ARE BASED ON VISUAL EXAMINATION AND THE LABORATORY TESTING SHOWN.

BORING LOCATIONS WERE NOT SURVEYED. BORING LOCATIONS WERE ESTABLISHED IN THE FIELD USING A SUB-METER ACCURACY GPS UNIT (TRIMBLE GEO 7X).

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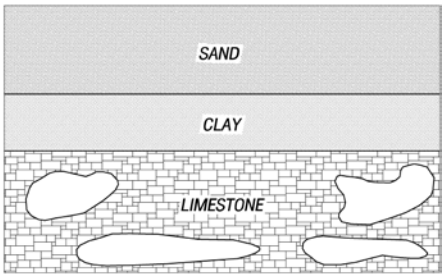
ADDITIONAL AUGER BORING RESULTS

LAKEHURST DRIVE / CARRIER DRIVE DEVELOPMENT

FIGURE NO. 7

APPENDIX A
CENTRAL FLORIDA GEOLOGY

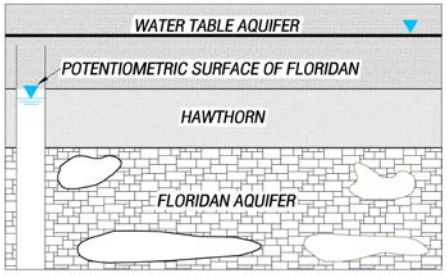
A. CENTRAL FLORIDA GEOLOGY



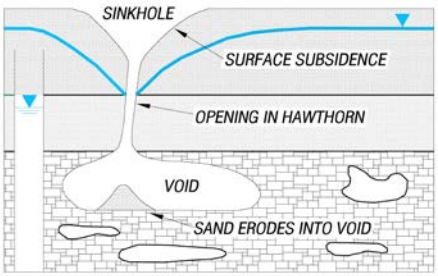
KARST GEOLOGY OF CENTRAL FLORIDA

Central Florida geology is defined by the nature and relationship of three sedimentary layers. The deepest layer is cavernous limestone known as the Floridan aquifer (Floridan). The Floridan is overlain by the clayey sand to sandy clay of the Hawthorn Group (Hawthorn). The Hawthorn is in turn overlain by a surface layer of sand, which comprises the water table aquifer.

Central Florida geology is classified as "karst" or sinkhole-prone. Where the Hawthorn is absent, water and sand can flow downward to cavities within Floridan aquifer, like sand through an hourglass, *recharging* the Floridan aquifer, and sometimes causing the formation of surface depressions, or sinkholes. This process of subsurface erosion caused by recharge is known as *raveling*. Thus, areas of groundwater *recharge* to the Floridan aquifer are more likely to experience sinkhole activity.



CENTRAL FLORIDA AQUIFER SYSTEMS



SINKHOLE FORMATION MECHANISM

No field exploration method can accurately predict the occurrence of sinkholes. It is common geotechnical practice in Central Florida to assess sinkhole risk at a site based on published geology and recent sinkhole history. Further evaluation can be made by performing deep soil borings. The purpose of the borings is to explore the Hawthorn and overlying sands for indicators of sinkhole activity, including extensive zones of loose, *raveled* soil and losses of drilling fluid circulation. Evaluation of sinkhole risk by deep borings is typically performed for significant structures, but is not usually performed for horizontal facilities such as highways. It is generally considered that the cost of performing such an evaluation for that type of facility would not be cost-effective when compared to the benefit derived.

In some instances, sinkholes formed in wetlands, or by their formation the sinkholes created wetlands. These sinkholes often filled with highly compressible organic soil (peat and/or muck), sometimes to depths greater than 100 feet. Although these relic sinkholes do not typically reactivate, the highly compressible nature of the organic soil can cause excessive settlement of roadways, structures or other facilities that may be constructed over them.

APPENDIX B
FIELD EXPLORATION METHODS

B. FIELD EXPLORATION METHODS

B.1 SPT Borings

Standard Penetration Test (SPT) borings were drilled in general accordance with ASTM Procedure D-1586. The boreholes were advanced by the rotary wash method with bentonite-based mud used as the circulating fluid to help remove cuttings and to help stabilize the borehole. Casing was used as necessary to further stabilize the borehole and prevent loose surficial sands from raveling into the lower more stable portions of the borehole. GEC's field crew obtained SPT samples continuously in the borings to a depth of 10 feet and at 5-foot depth intervals thereafter. A GEC engineering technician supervised the drilling operation, and collected, examined and visually classified each sample. Our technician collected representative samples for further visual examination and classification in our laboratory. SPT borings that penetrate the Hawthorn Group are grout-sealed upon completion.

B.2 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) tests were performed in manual auger borings in general accordance with "Dynamic Cone for Shallow In-Situ Penetration Testing," *Vane Shear and Cone Penetration Testing of In-Situ Soils, ASTM STP 399*, 1966. The DCP tests were performed at 2-foot intervals until the boring termination depth was reached.

B.3 Machine Auger Borings

Machine auger borings were performed in general accordance with ASTM Procedure D-1452. Machine auger borings were performed by hydraulically turning a continuous flight, solid-stem, auger into the ground in 5-foot increments until the desired boring termination depth was achieved. The auger flights were retrieved in 5-foot increments, without further rotation of the auger, and the retrieved soil was examined by our technician prior to collection of representative samples. Representative samples were collected for further visual examination and classification in the GEC laboratory.

B.4 Manual Auger Borings

A GEC engineering technician performed standard barrel manual auger borings in general accordance with ASTM D-1452, by manually turning a 3-inch diameter, 6-inch long sampler into the soil until it was full. He then retrieved the sampler and visually examined and classified the soil. This procedure was repeated until the desired termination depth was achieved.

Representative samples were collected for further visual examination and classification in the GEC laboratory.

B.5 Groundwater Measurement

A GEC engineering technician measured the depth to groundwater in the boreholes at the time of drilling and again after approximately 24 hours. Once the 24-hour groundwater measurement was recorded, the boreholes were then backfilled with soil cuttings to prevailing ground surface.

B.6 Manual Muck Probes

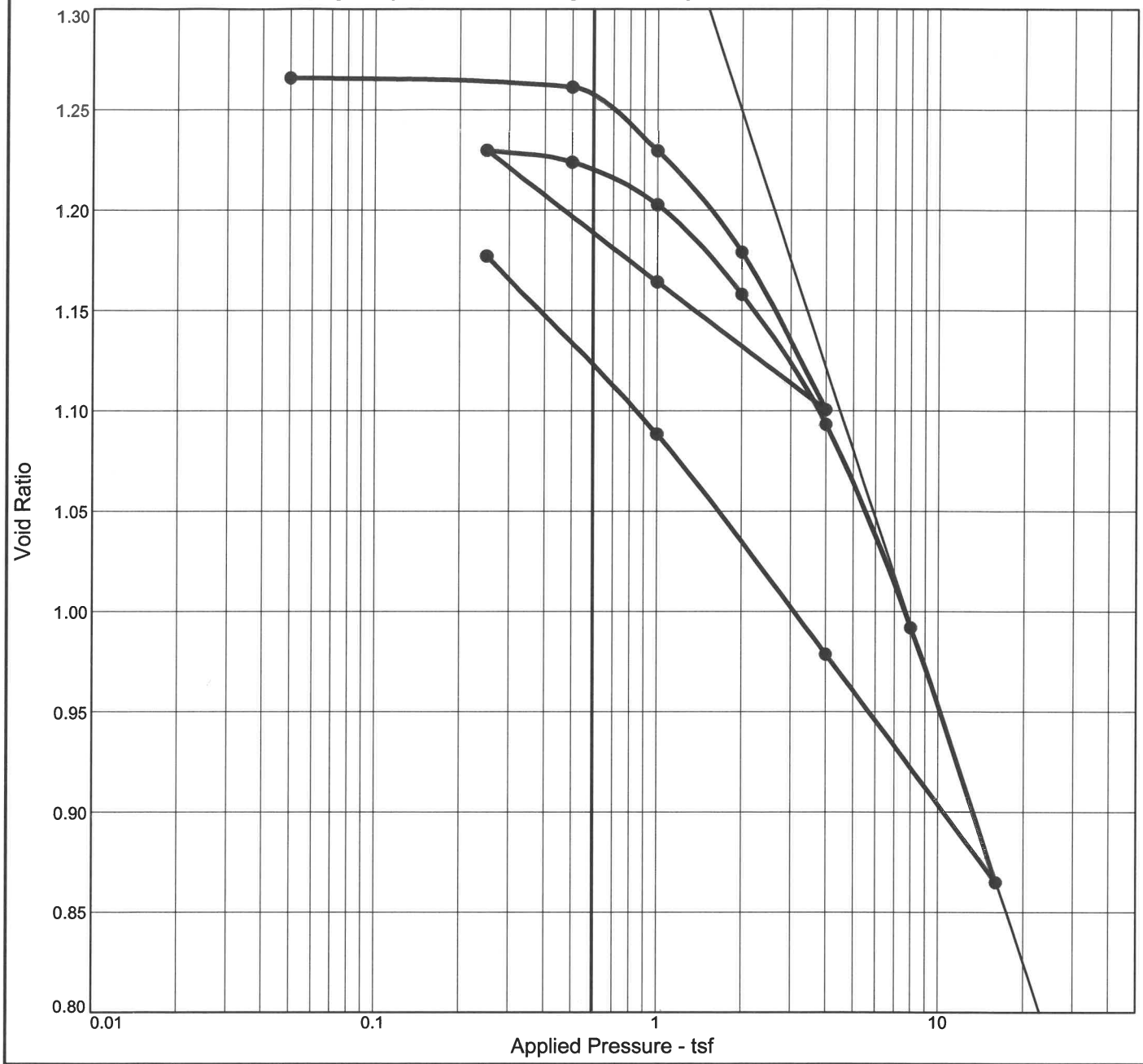
Manual muck probes were performed by pushing a slender metal rod into the surficial soil and evaluating the relative resistance of the soil to manual penetration. Highly organic soils, such as muck and/or peat, are characteristically very soft and will easily yield to the manual probe. Manual probes, however, cannot detect peat or muck layers which are present beneath layers of sand or dense soils which cannot be penetrated by the probe. The probes can also penetrate to some extent in very loose sands which may be present beneath peat or muck layers. No soil samples are obtained for visual examination or laboratory testing when using this exploratory technique. The soil type being penetrated is inferred solely by evaluating the relative resistance of the soil to penetration. These limitations can lead to some under-estimation or over-estimation of peat or muck layer thicknesses. The probe data presented in this report should be evaluated with these limitations in mind.

APPENDIX C
LABORATORY TEST RESULTS

APPENDIX C-1
CONSOLIDATION

Garage

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
98.0 %	49.9 %	71.5	85	67	2.50	0.44	2.2	0.42	0.11			1.274

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Fat Clay	CH	

Project No. 4406G **Client:**

Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A **Depth:** 15'-17'

Remarks:
Fines Content= 91.4%

Geotechnical and Environmental Consultants, Inc.

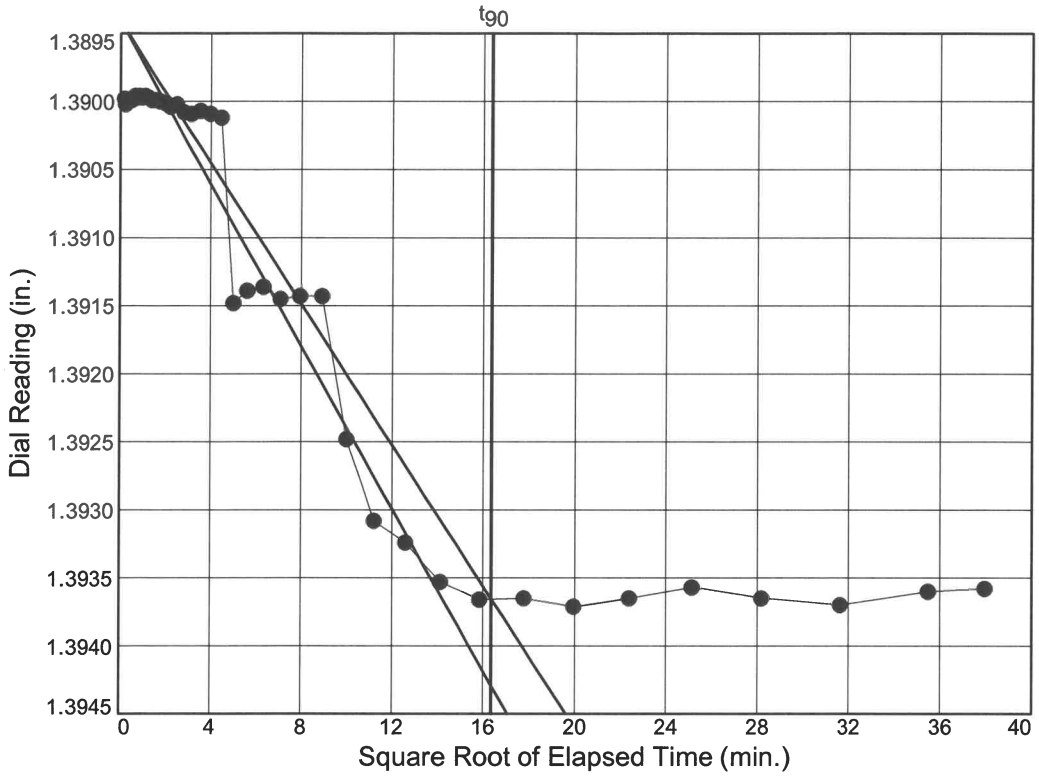
Figure

Tested By: TJR **Checked By:** BMM

Dial Reading vs. Time

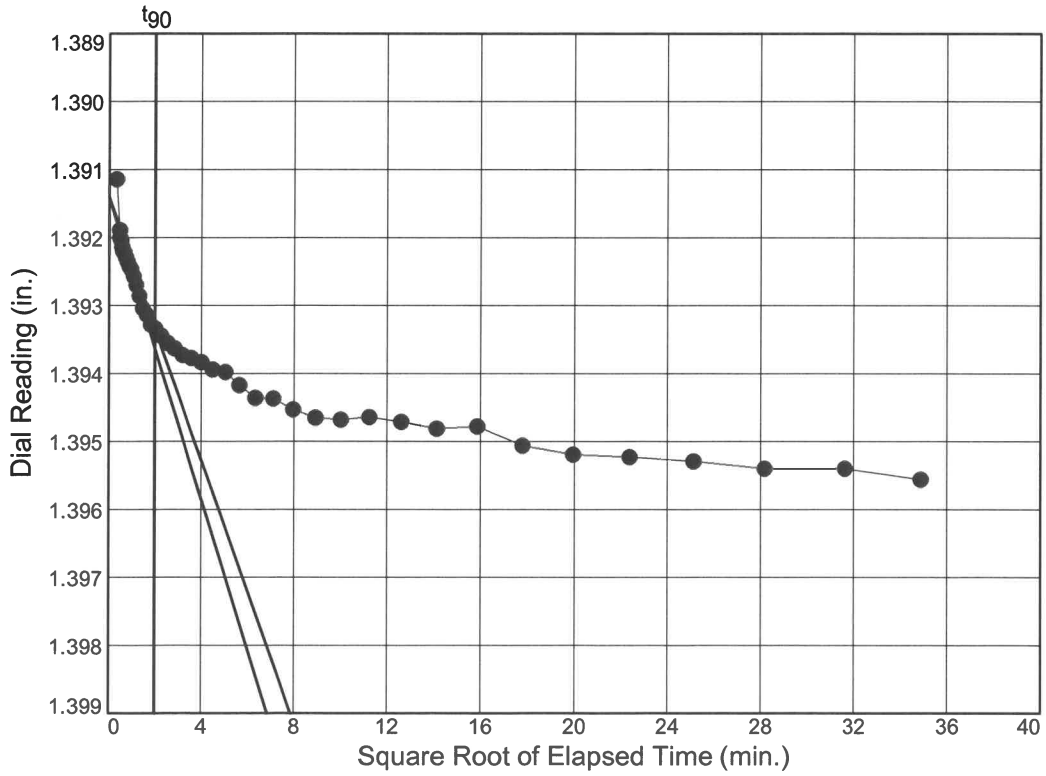
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 1
Load= 0.05 tsf
D_0 = 1.3894
D_{90} = 1.3937
D_{100} = 1.3941
T_{90} = 267.54 min.

C_v @ T_{90}
0.008 ft.^{2}/day



Load No.= 2
Load= 0.50 tsf
D_0 = 1.3914
D_{90} = 1.3933
D_{100} = 1.3935
T_{90} = 3.99 min.

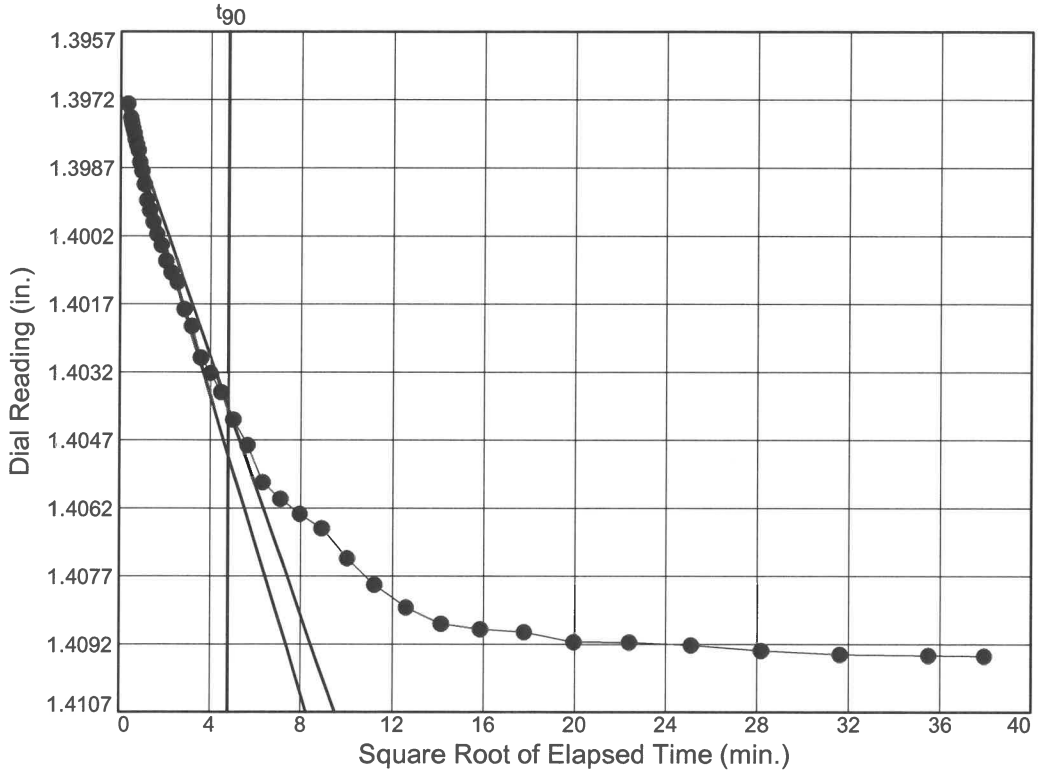
C_v @ T_{90}
0.529 ft.^{2}/day

Figure

Dial Reading vs. Time

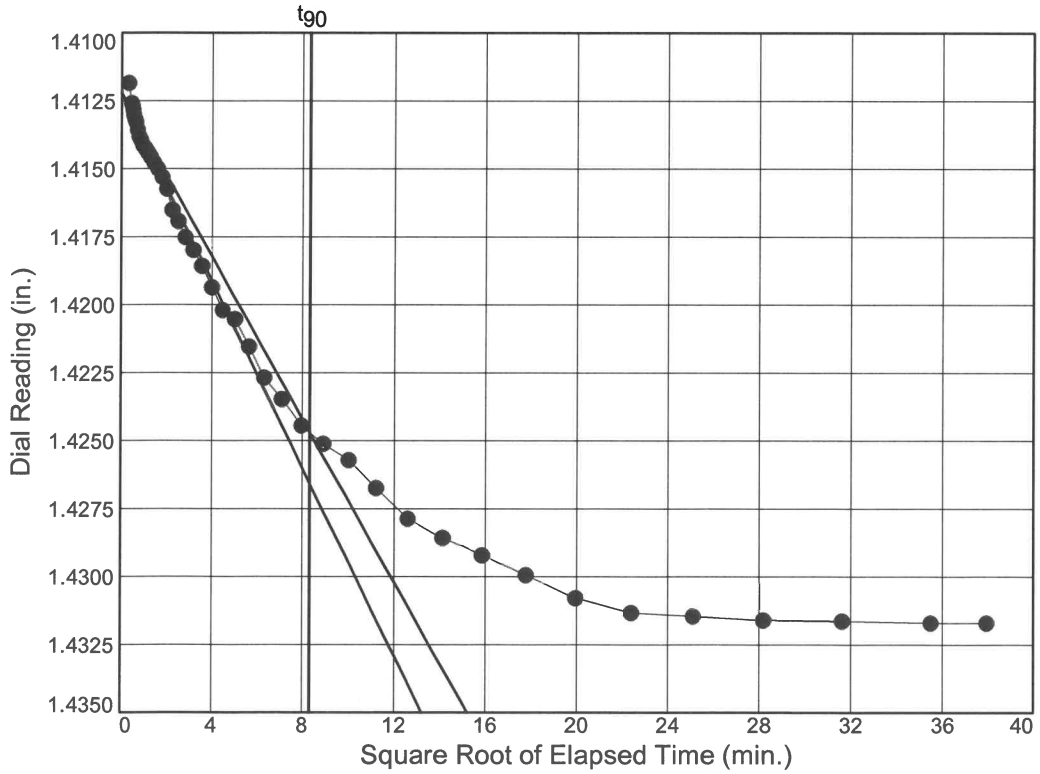
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 3
Load= 1.00 tsf
D_0 = 1.3971
D_{90} = 1.4040
D_{100} = 1.4048
T_{90} = 23.25 min.

C_v @ T_{90}
0.089 ft.^{2}/day



Load No.= 4
Load= 2.00 tsf
D_0 = 1.4122
D_{90} = 1.4247
D_{100} = 1.4261
T_{90} = 69.36 min.

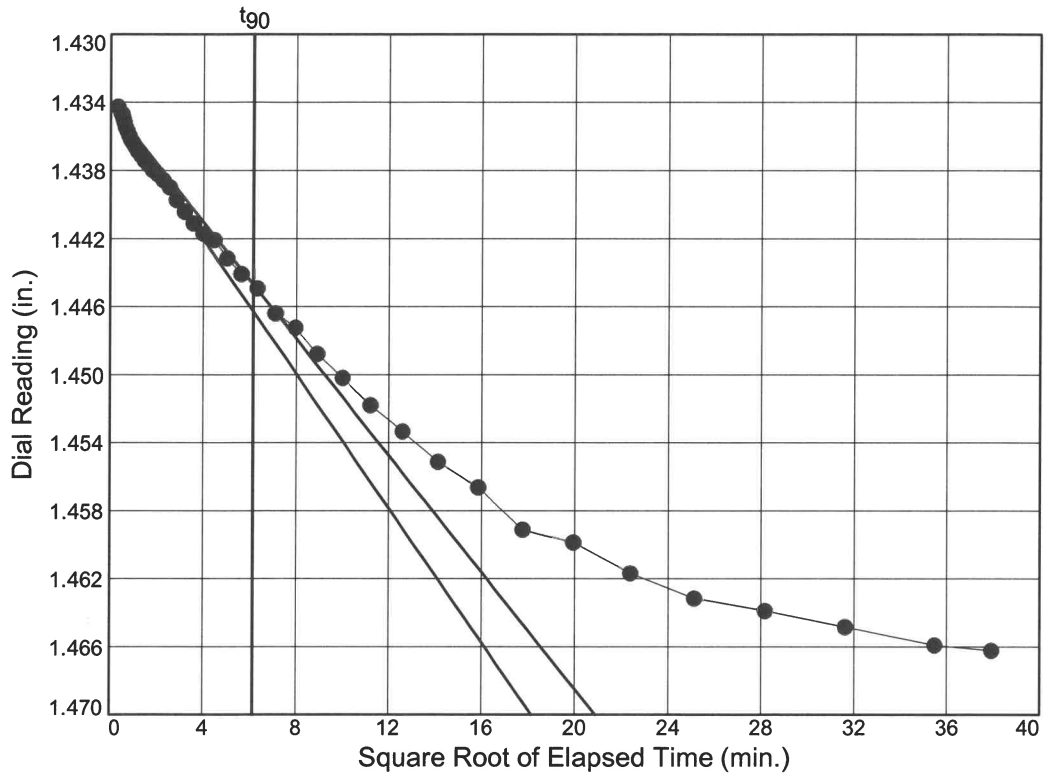
C_v @ T_{90}
0.029 ft.^{2}/day

Figure

Dial Reading vs. Time

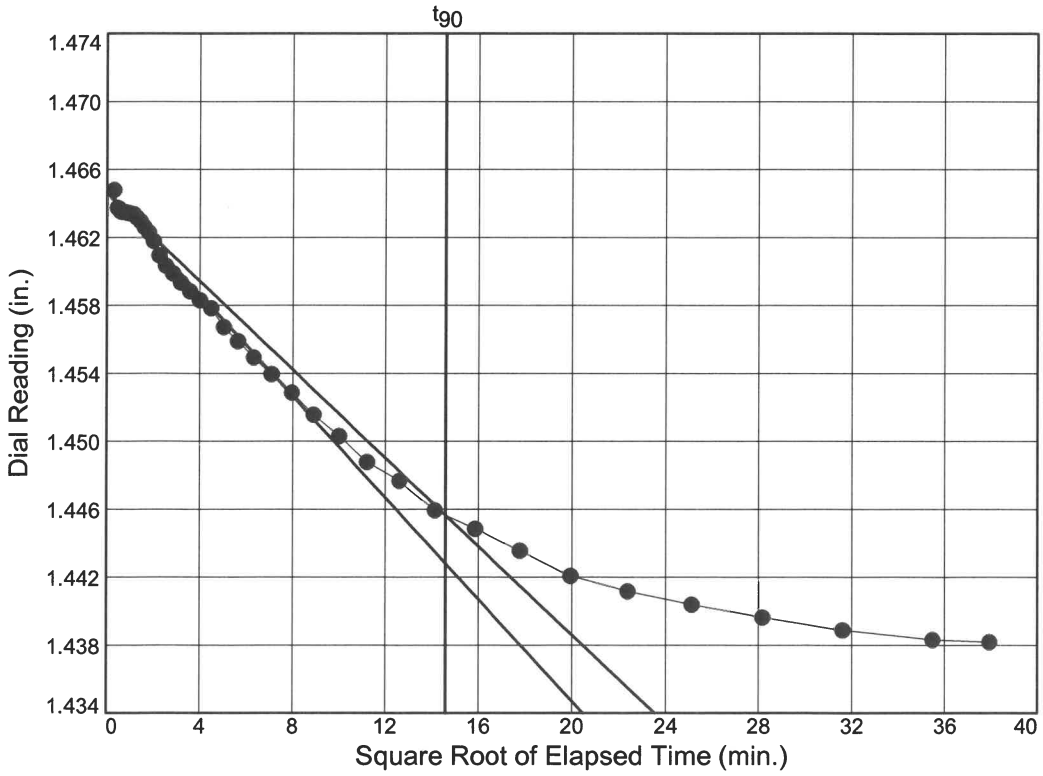
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 5
Load= 4.00 tsf
D_0 = 1.4342
D_{90} = 1.4447
D_{100} = 1.4459
T_{90} = 37.72 min.

C_v @ T_{90}
0.050 ft.^{2}/day



Load No.= 6
Load= 1.00 tsf
D_0 = 1.4646
D_{90} = 1.4456
D_{100} = 1.4435
T_{90} = 212.73 min.

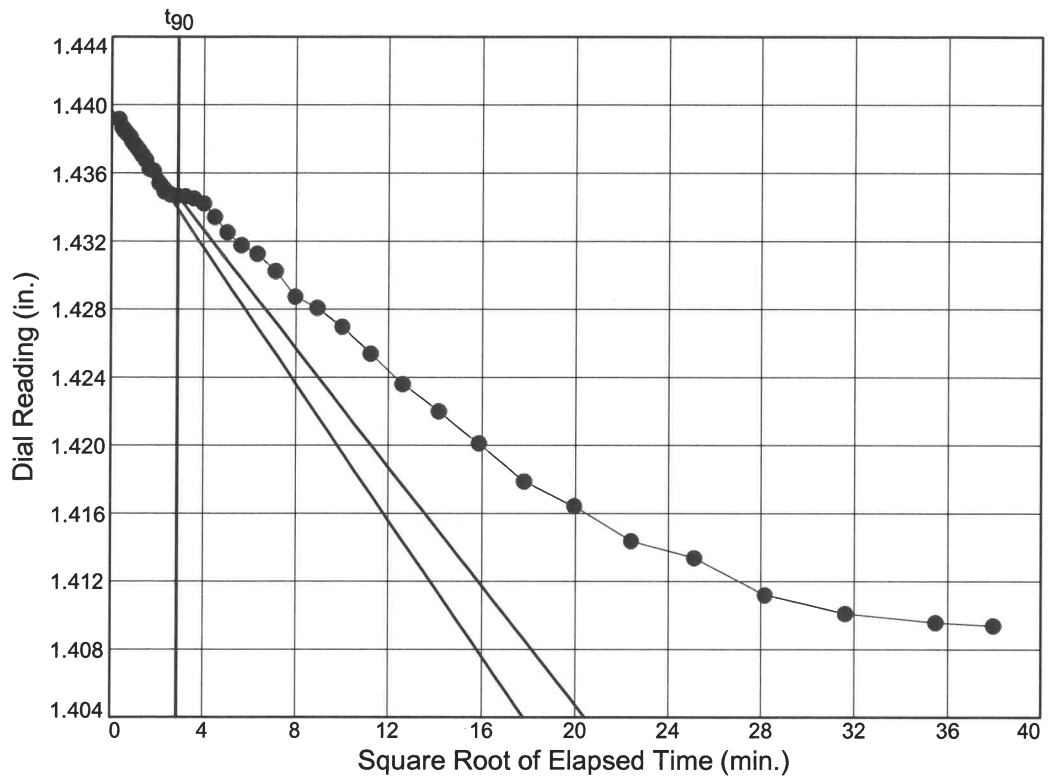
C_v @ T_{90}
0.009 ft.^{2}/day

Figure

Dial Reading vs. Time

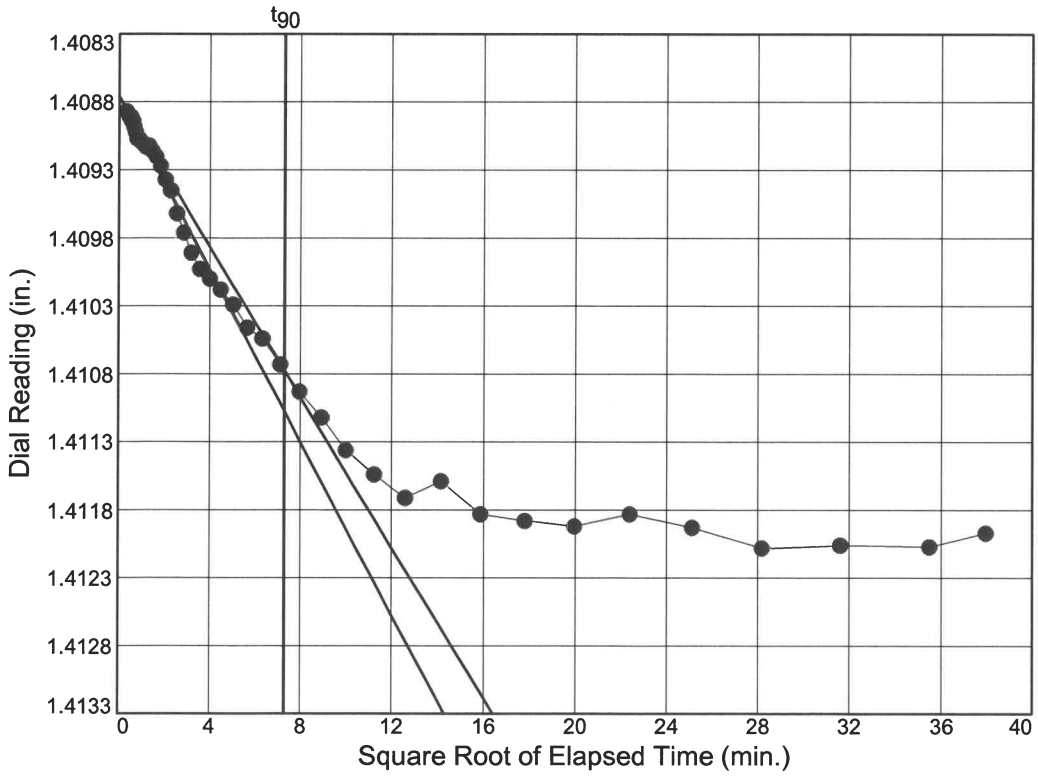
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 7
Load= 0.25 tsf
D_0 = 1.4396
D_{90} = 1.4346
D_{100} = 1.4341
T_{90} = 8.18 min.

C_v @ T_{90}
0.241 ft.^{2}/day



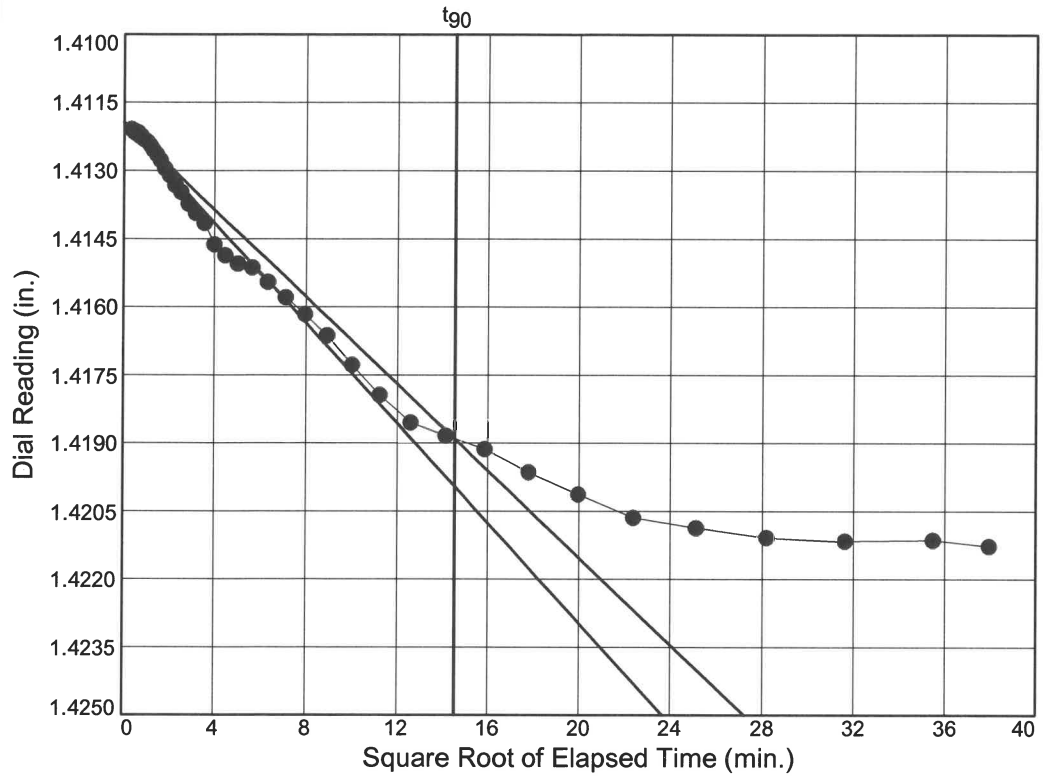
Load No.= 8
Load= 0.50 tsf
D_0 = 1.4088
D_{90} = 1.4108
D_{100} = 1.4110
T_{90} = 52.75 min.

C_v @ T_{90}
0.039 ft.^{2}/day

Dial Reading vs. Time

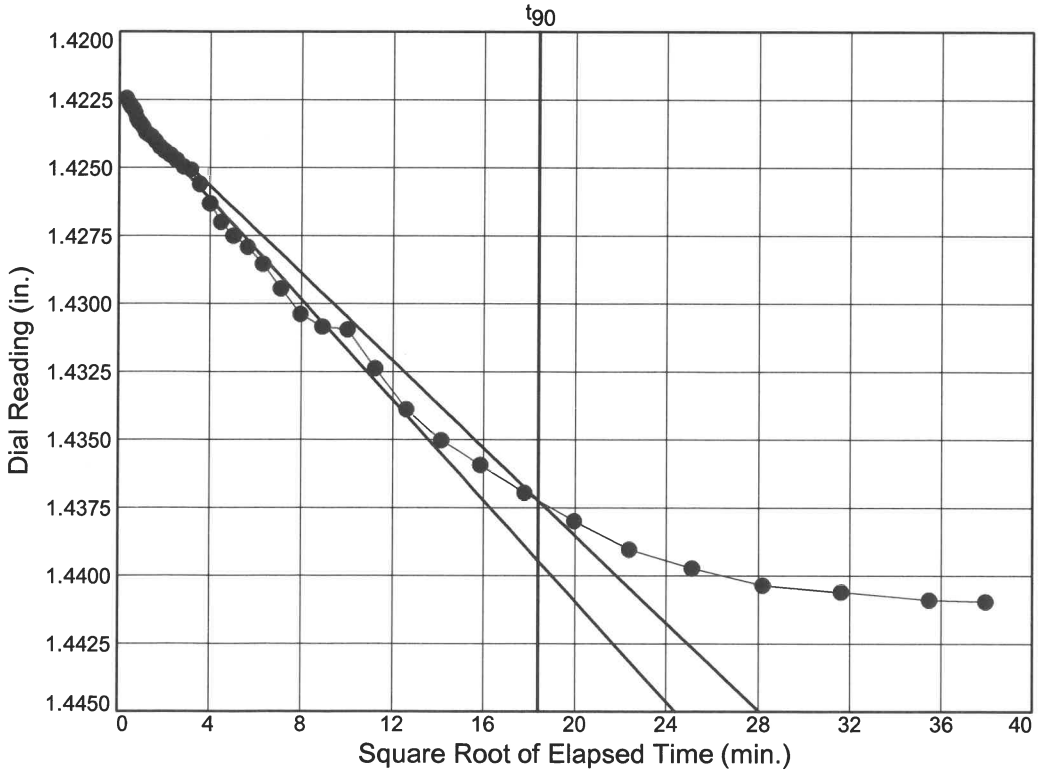
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 9
Load= 1.00 tsf
D_0 = 1.4119
D_{90} = 1.4189
D_{100} = 1.4197
T_{90} = 210.88 min.

C_v @ T_{90}
0.010 ft.^{2}/day



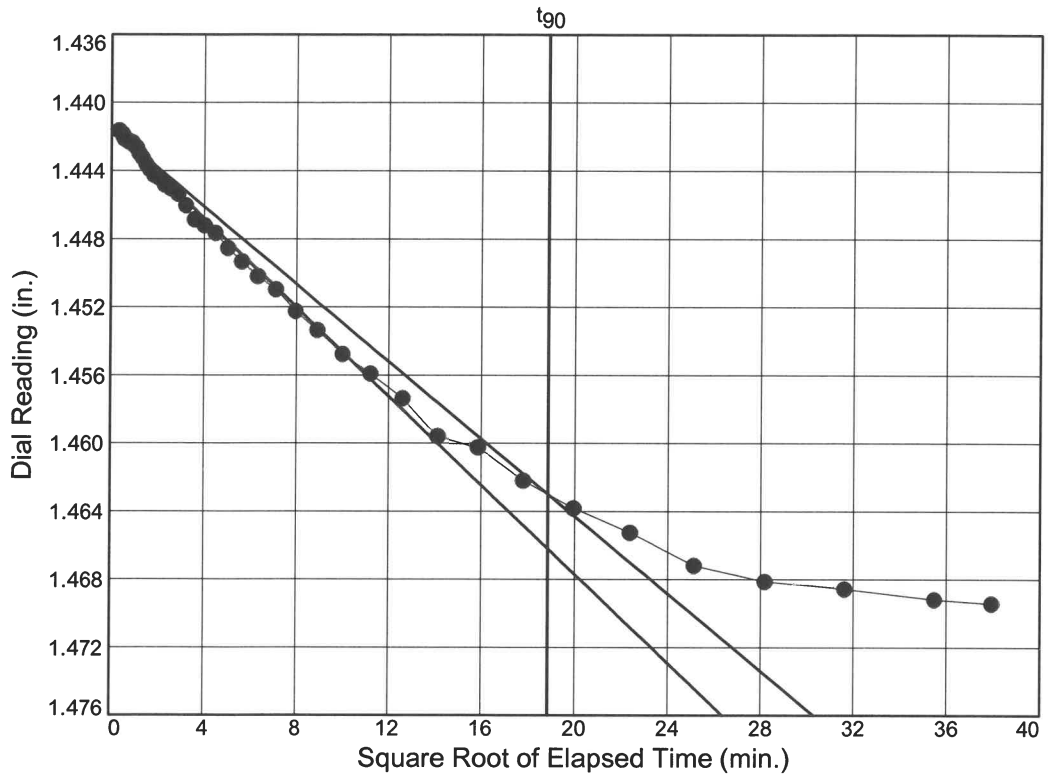
Load No.= 10
Load= 2.00 tsf
D_0 = 1.4224
D_{90} = 1.4372
D_{100} = 1.4389
T_{90} = 338.58 min.

C_v @ T_{90}
0.006 ft.^{2}/day

Dial Reading vs. Time

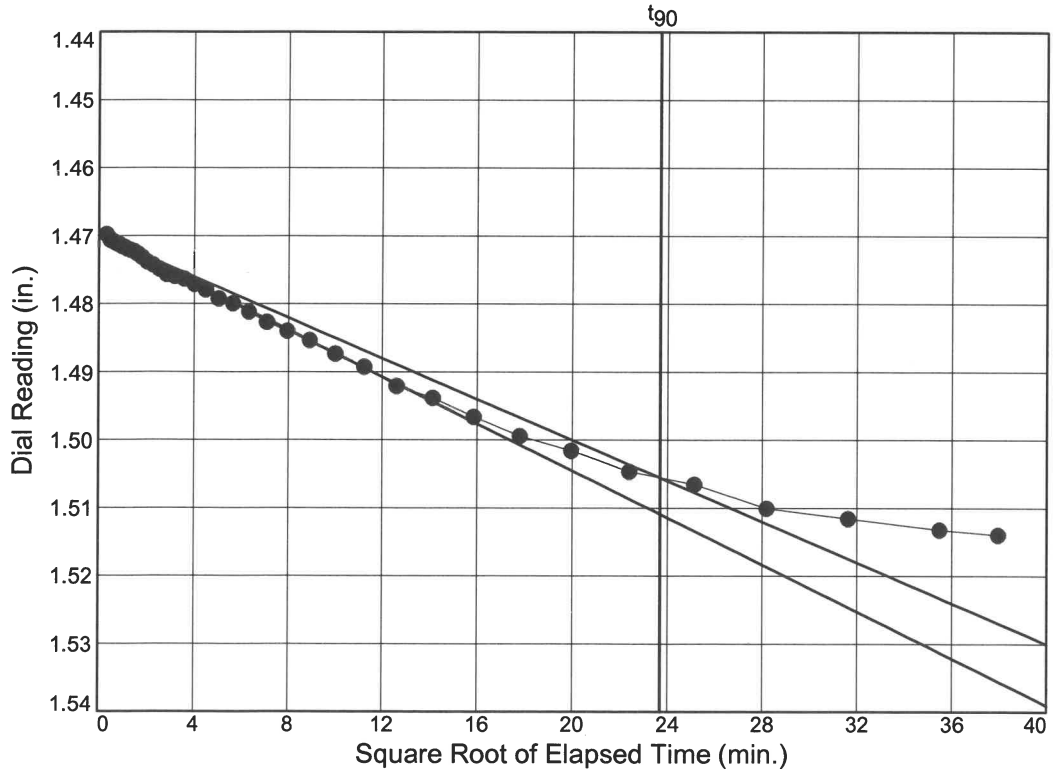
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 11
Load= 4.00 tsf
D_0 = 1.4416
D_{90} = 1.4630
D_{100} = 1.4654
T_{90} = 355.45 min.

C_v @ T_{90}
0.005 ft.^{2}/day



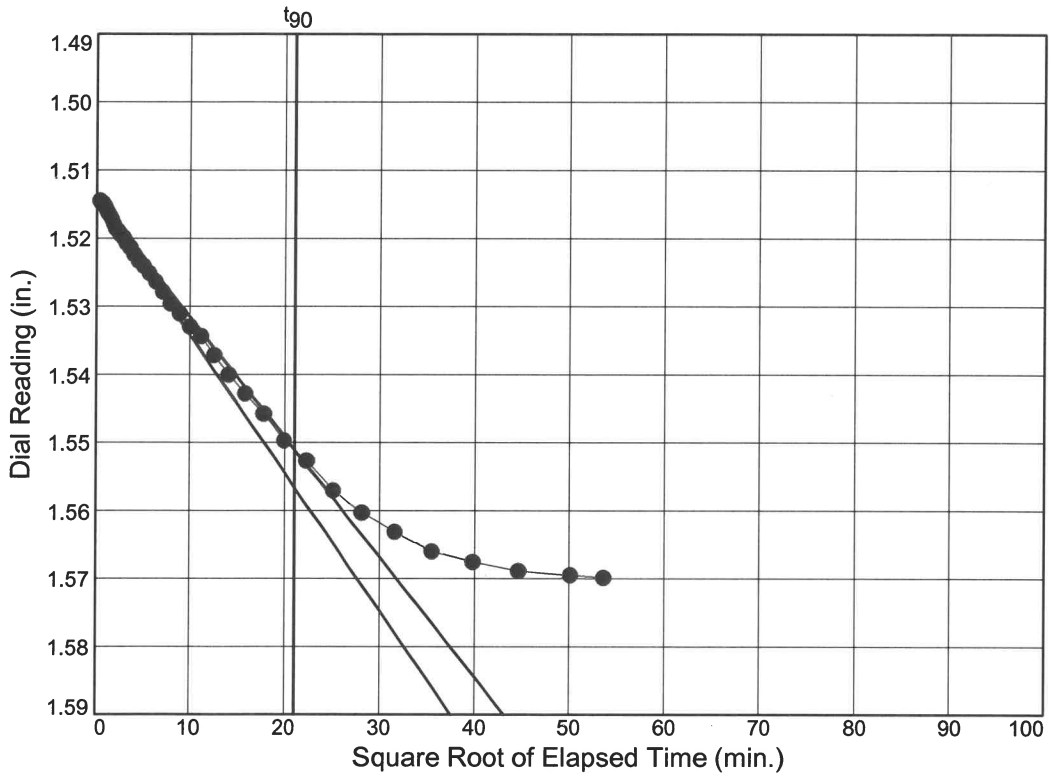
Load No.= 12
Load= 8.00 tsf
D_0 = 1.4700
D_{90} = 1.5055
D_{100} = 1.5095
T_{90} = 560.53 min.

C_v @ T_{90}
0.003 ft.^{2}/day

Dial Reading vs. Time

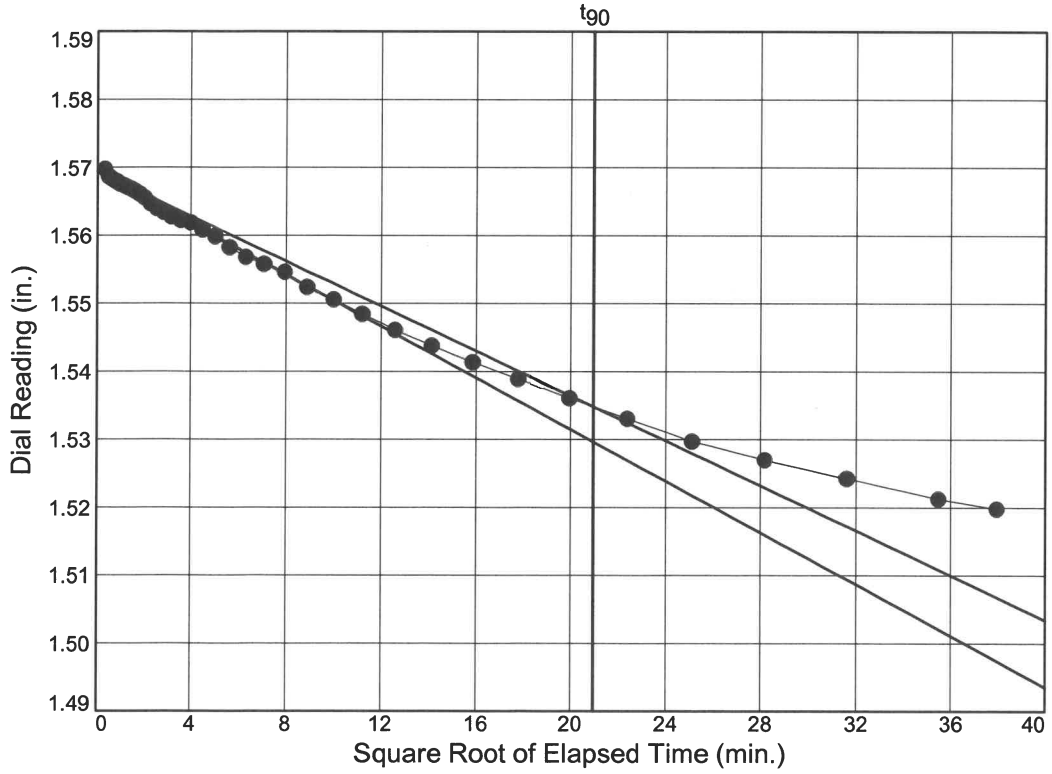
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 13
Load= 16.00 tsf
D_0 = 1.5138
D_{90} = 1.5511
D_{100} = 1.5553
T_{90} = 446.77 min.

C_v @ T_{90}
0.003 ft.^{2}/day



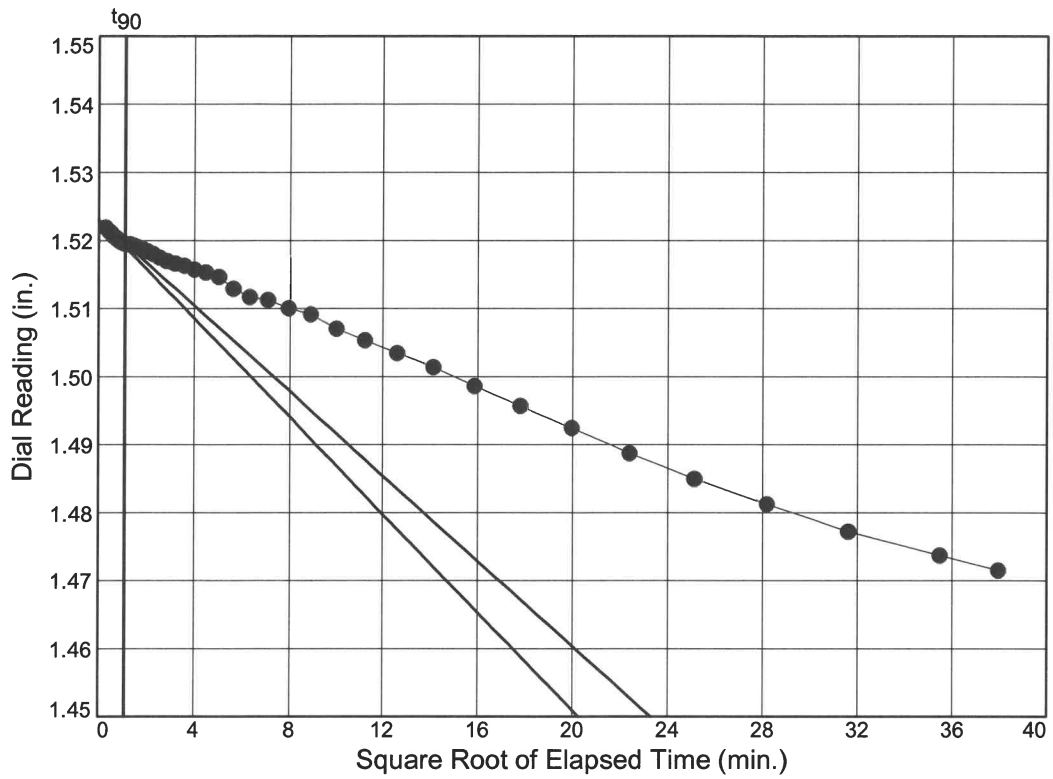
Load No.= 14
Load= 4.00 tsf
D_0 = 1.5694
D_{90} = 1.5348
D_{100} = 1.5310
T_{90} = 439.42 min.

C_v @ T_{90}
0.003 ft.^{2}/day

Dial Reading vs. Time

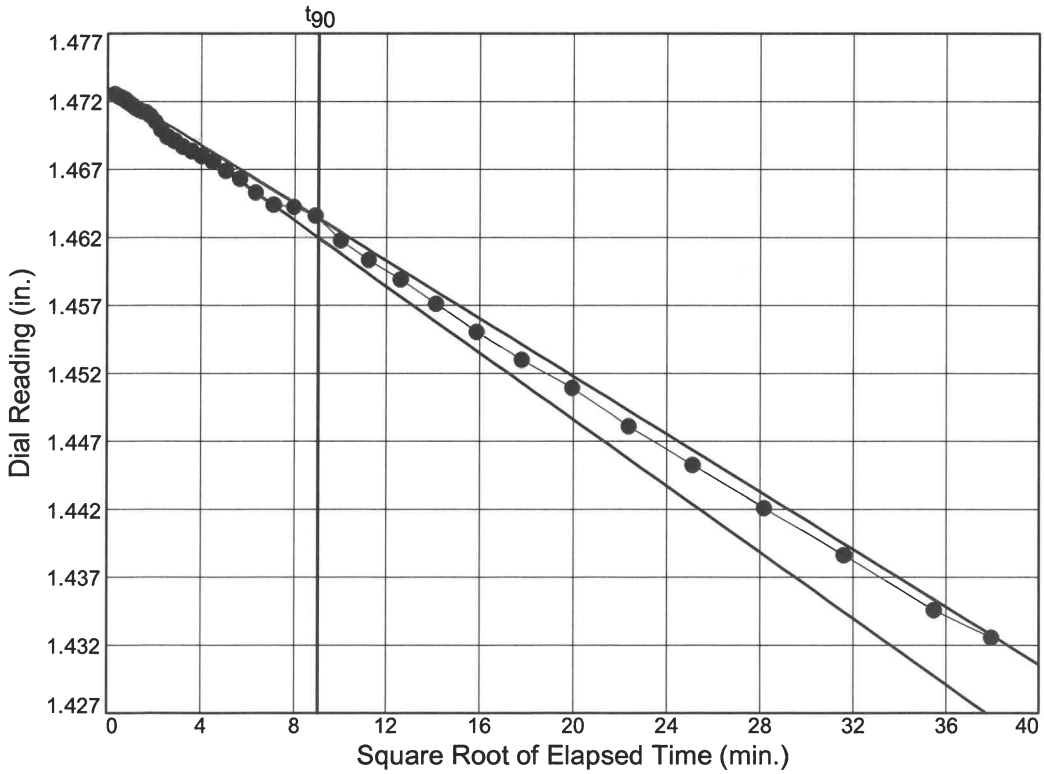
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-3A Depth: 15'-17'



Load No.= 15
Load= 1.00 tsf
D_0 = 1.5230
D_{90} = 1.5195
D_{100} = 1.5192
T_{90} = 1.20 min.

C_v @ T_{90}
1.408 ft.^{2}/day



Load No.= 16
Load= 0.25 tsf
D_0 = 1.4730
D_{90} = 1.4634
D_{100} = 1.4623
T_{90} = 81.99 min.

C_v @ T_{90}
0.023 ft.^{2}/day

Figure

UNDISTURBED SAMPLE CLASSIFICATION

Job Name: Lake Hurst
Date: 8/7/19
Job Number: 44066
Tech: TJR
Boring Number: SPT-3A
Proj Mgr.: CPM
Station:
Offset:
Depth: 15-17

From/To

Description: 0"-24" Gray fat clay (CH)
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:
Penetrometer = 1.75 tsf
Torvane = 0.5 tsf
Remold = 0.125 tsf

Description:
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:

Description:
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:

Description:
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:

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UNIT WEIGHT

Project Name:	Lakehurst Drive
Project Number:	4406G
Date:	8-27-19

Boring Number:	SPT-3A
Depth:	15'-17'

Section Depth:	18"-24"			
	1	2	3	Avg.
Length (in.)	6.03	5.93	5.99	5.98
Diameter (in.)	2.83	2.84	2.84	2.84
Area of Sample (sq. inches)				6.32
Volume of Sample (cu. Inches)				37.79
Volume of Sample (cu. feet)				0.02
Wet Weight of Sample (lbs)				2.35
Wet Density (pcf)				107.49
Moisture Content (%)				49.90
Dry Density (pcf)				71.71

Section Depth:				
	1	2	3	Avg.
Length (in.)				
Diameter (in.)				
Area of Sample (sq. inches)				
Volume of Sample (cu. Inches)				
Volume of Sample (cu. feet)				
Wet Weight of Sample (lbs)				
Wet Density (pcf)				
Moisture Content (%)				
Dry Density (pcf)				

Section Depth:				
	1	2	3	Avg.
Length (in.)				
Diameter (in.)				
Area of Sample (sq. inches)				
Volume of Sample (cu. Inches)				
Volume of Sample (cu. feet)				
Wet Weight of Sample (lbs)				
Wet Density (pcf)				
Moisture Content (%)				
Dry Density (pcf)				

Section Depth:				
	1	2	3	Avg.
Length (in.)				
Diameter (in.)				
Area of Sample (sq. inches)				
Volume of Sample (cu. Inches)				
Volume of Sample (cu. feet)				
Wet Weight of Sample (lbs)				
Wet Density (pcf)				
Moisture Content (%)				
Dry Density (pcf)				

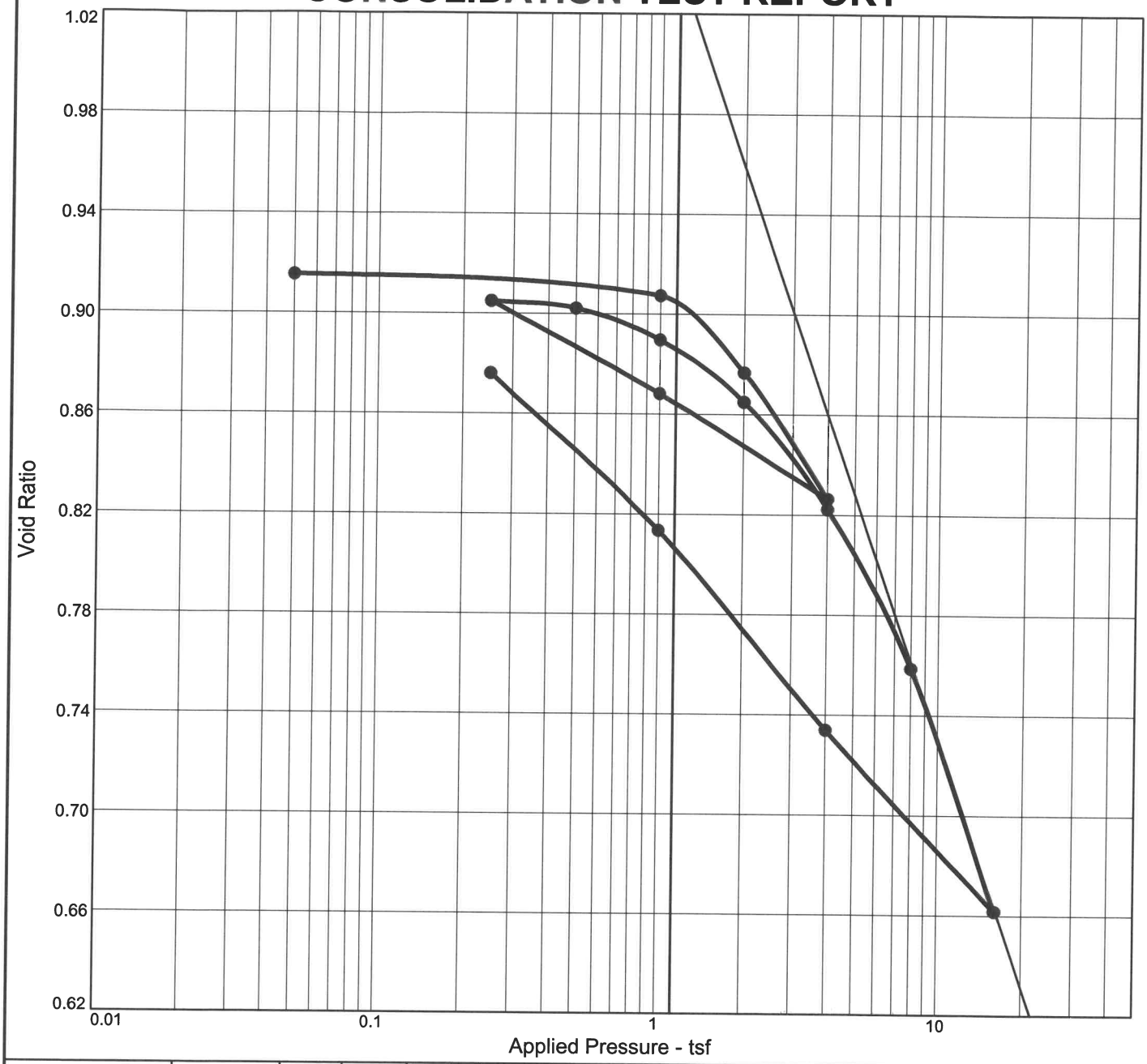
Unit Weight ASTM D7263 Method B, Intact Specimen

Moisture Content ASTM D2216

GEC Geotechnical and Environmental Consultants, Inc.

Apartments

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
98.5 %	36.7 %		70	57	2.48	0.73	3.4	0.33	0.09			0.924

MATERIAL DESCRIPTION	USCS	AASHTO
Light Brown Fat Clay w/ Sand	CH	

Project No. 4406G **Client:**

Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A **Depth:** 20'-22'

Remarks:
Fines Content= 82.5%

Geotechnical and Environmental Consultants, Inc.

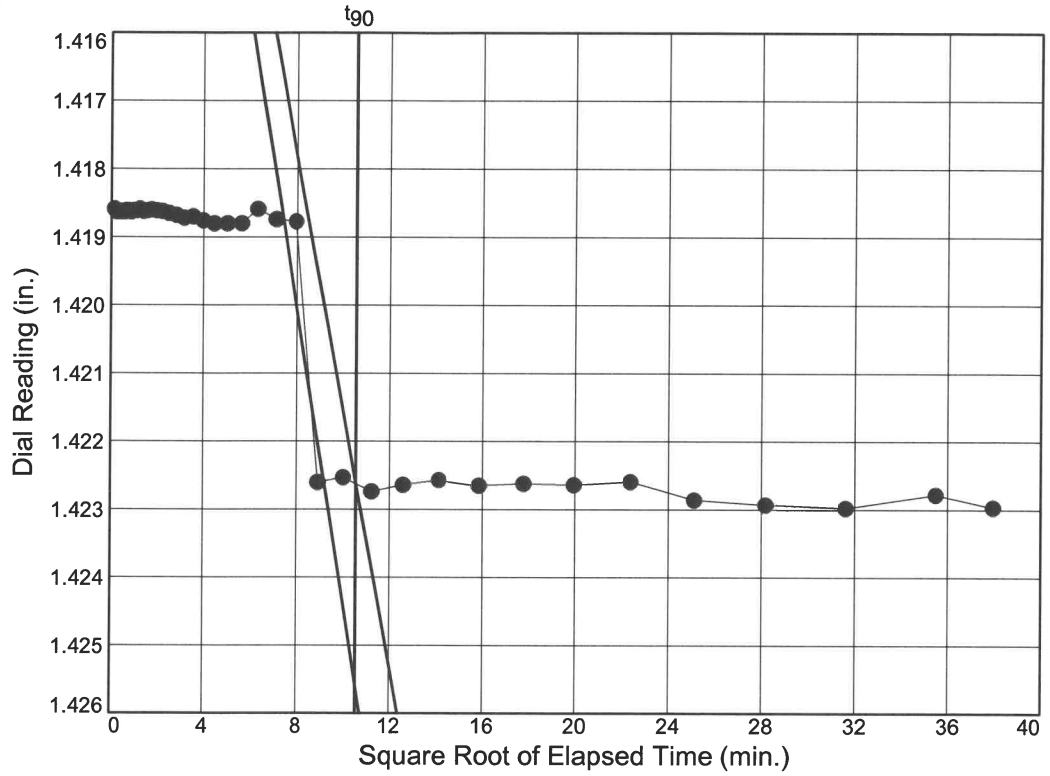
Figure

Tested By: TJR **Checked By:** BMM

Dial Reading vs. Time

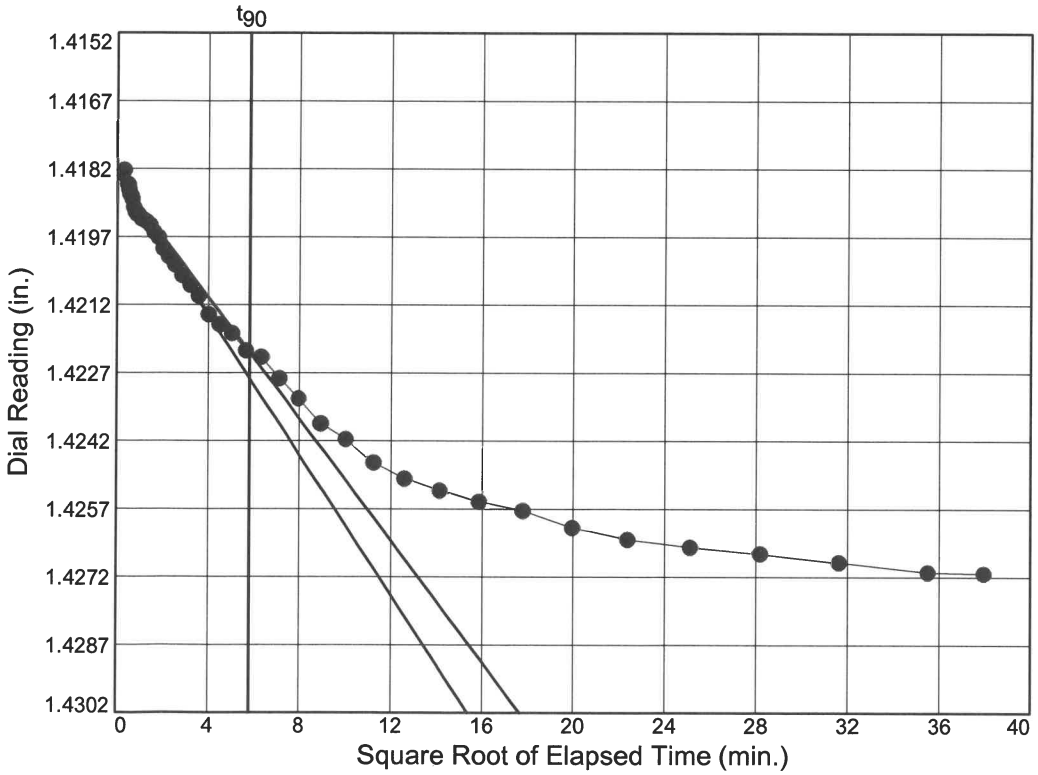
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 1
Load= 0.05 tsf
D_0 = 1.4029
D_{90} = 1.4226
D_{100} = 1.4248
T_{90} = 111.58 min.

C_v @ T_{90}
0.019 ft.^{2}/day



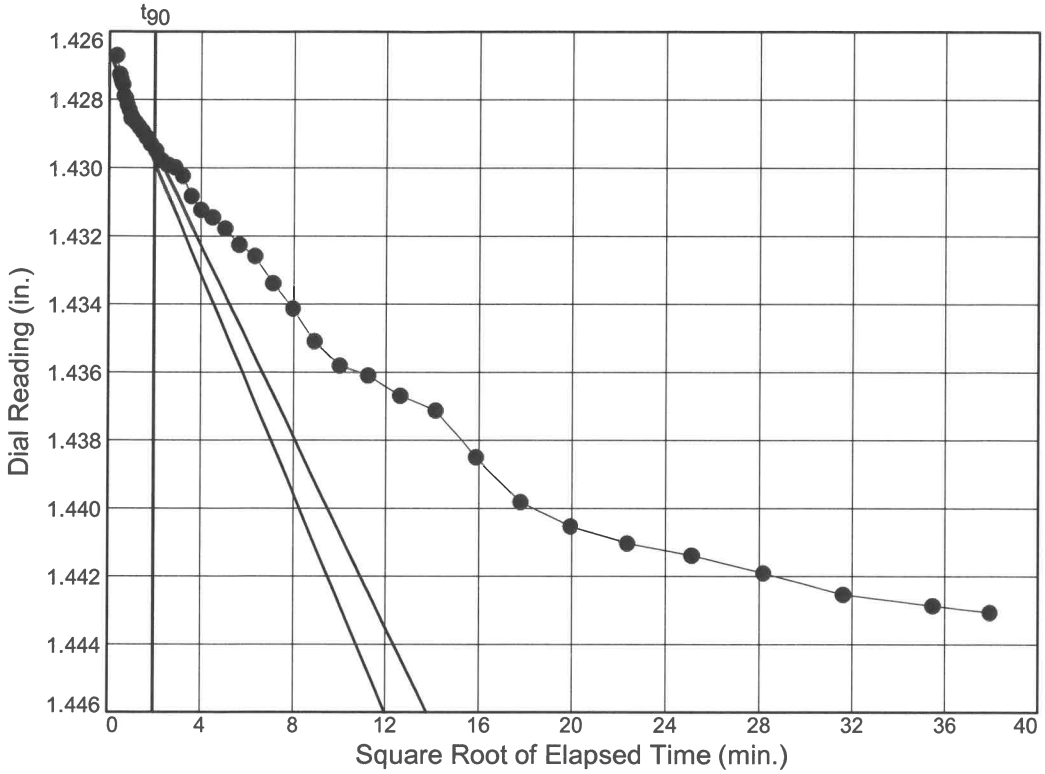
Load No.= 2
Load= 1.00 tsf
D_0 = 1.4184
D_{90} = 1.4222
D_{100} = 1.4227
T_{90} = 33.67 min.

C_v @ T_{90}
0.063 ft.^{2}/day

Dial Reading vs. Time

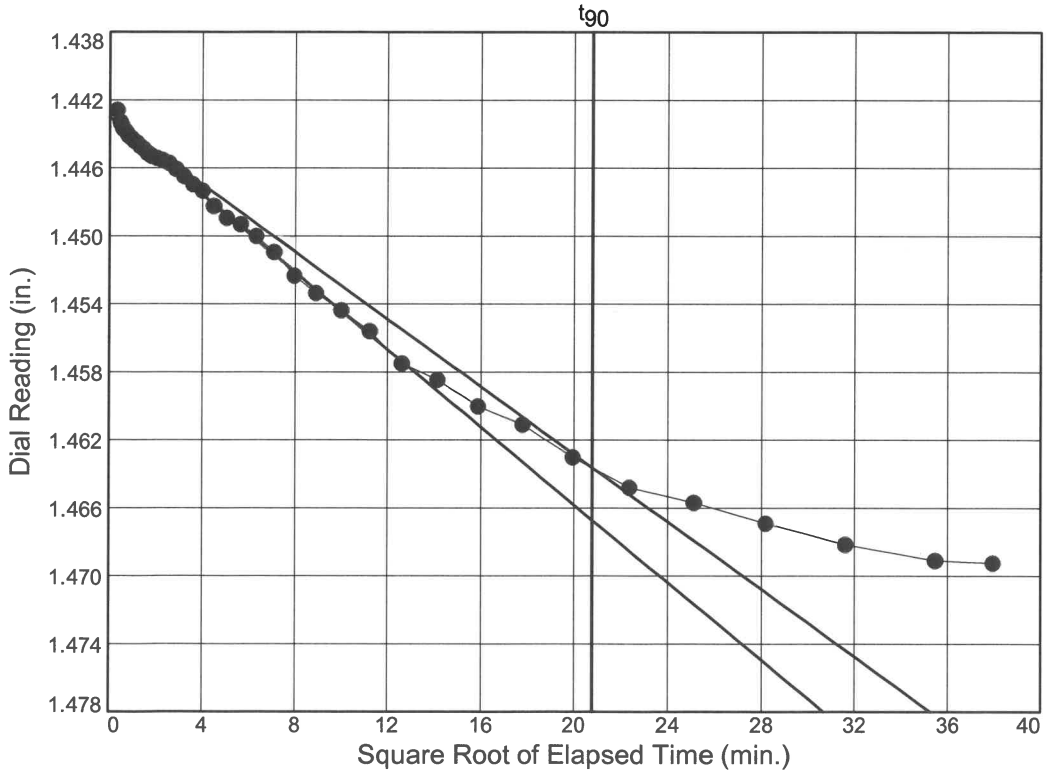
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 3
Load=2.00 tsf
D_0 = 1.4267
D_{90} = 1.4294
D_{100} = 1.4297
T_{90} = 3.88 min.

C_v @ T_{90}
0.530 ft.^{2}/day



Load No.= 4
Load=4.00 tsf
D_0 = 1.4430
D_{90} = 1.4636
D_{100} = 1.4659
T_{90} = 432.69 min.

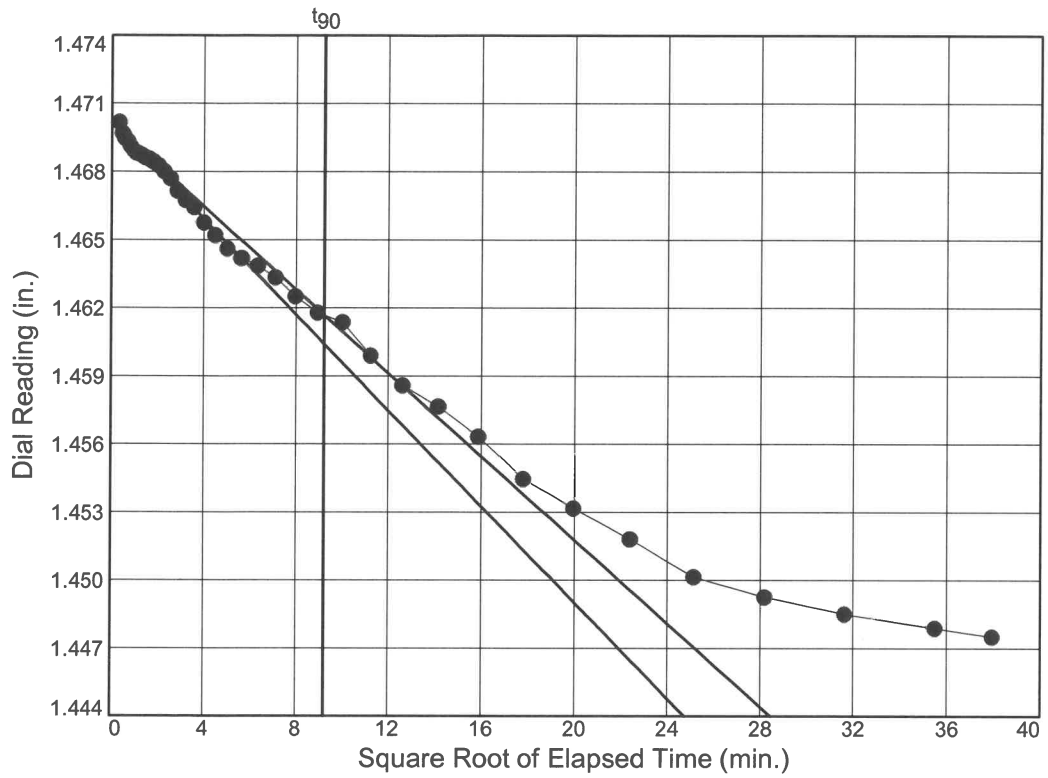
C_v @ T_{90}
0.005 ft.^{2}/day

Figure

Dial Reading vs. Time

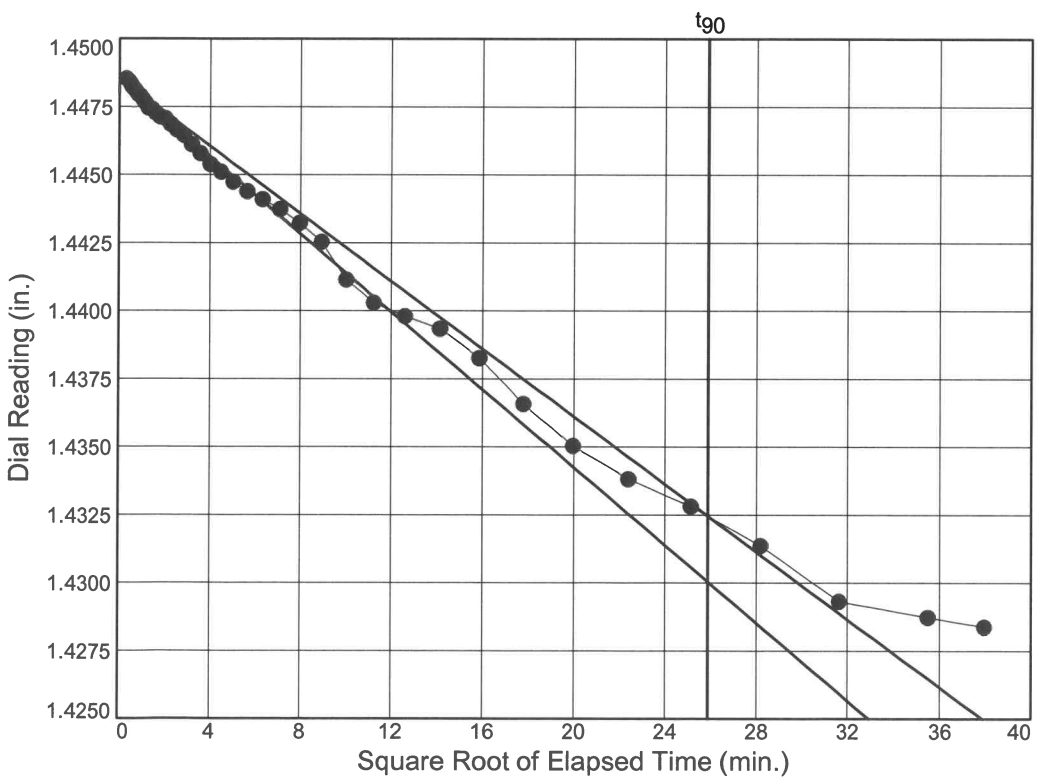
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 5
Load= 1.00 tsf
D_0 = 1.4701
D_{90} = 1.4617
D_{100} = 1.4607
T_{90} = 84.94 min.

C_v @ T_{90}
0.023 ft.^{2}/day



Load No.= 6
Load= 0.25 tsf
D_0 = 1.4485
D_{90} = 1.4325
D_{100} = 1.4307
T_{90} = 669.67 min.

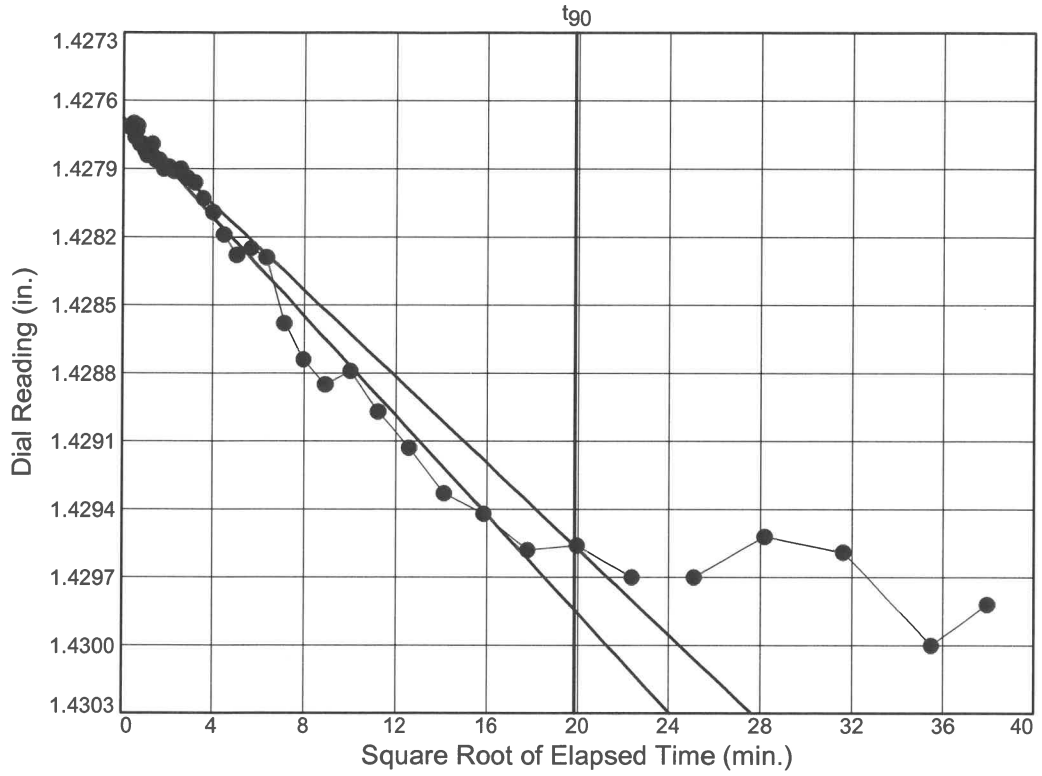
C_v @ T_{90}
0.003 ft.^{2}/day

Figure

Dial Reading vs. Time

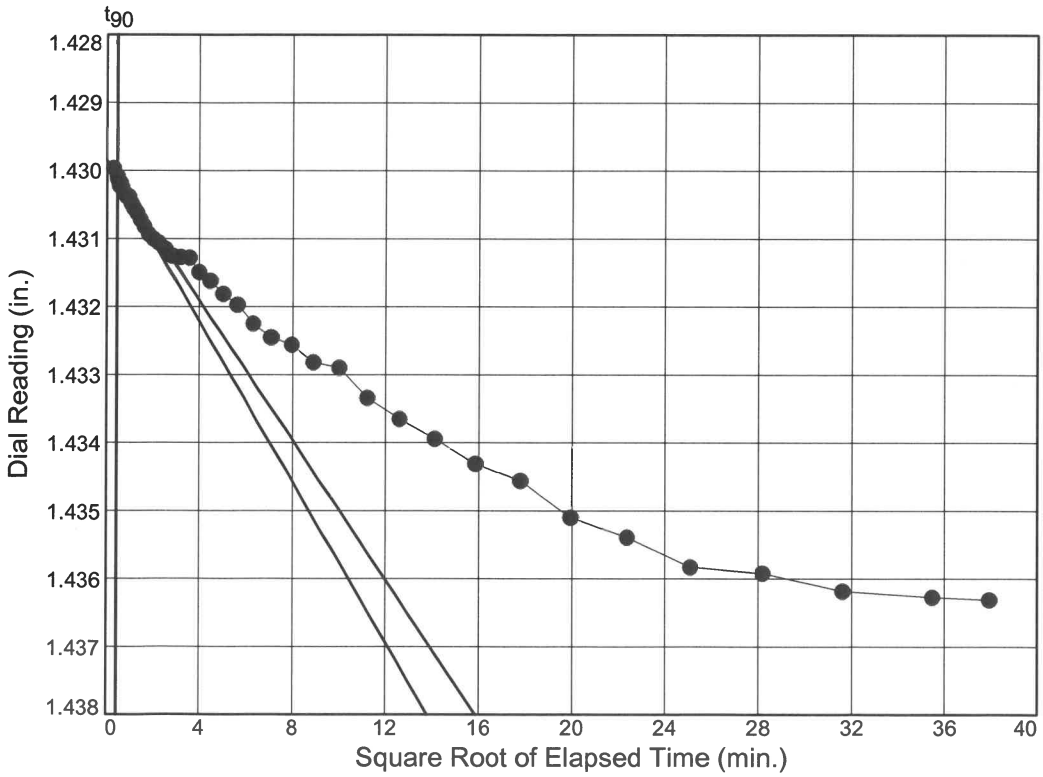
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 7
Load= 0.50 tsf
D_0 = 1.4277
D_{90} = 1.4296
D_{100} = 1.4298
T_{90} = 394.07 min.

C_v @ T_{90}
0.005 ft.^{2}/day



Load No.= 8
Load= 1.00 tsf
D_0 = 1.4299
D_{90} = 1.4301
D_{100} = 1.4301
T_{90} = 0.20 min.

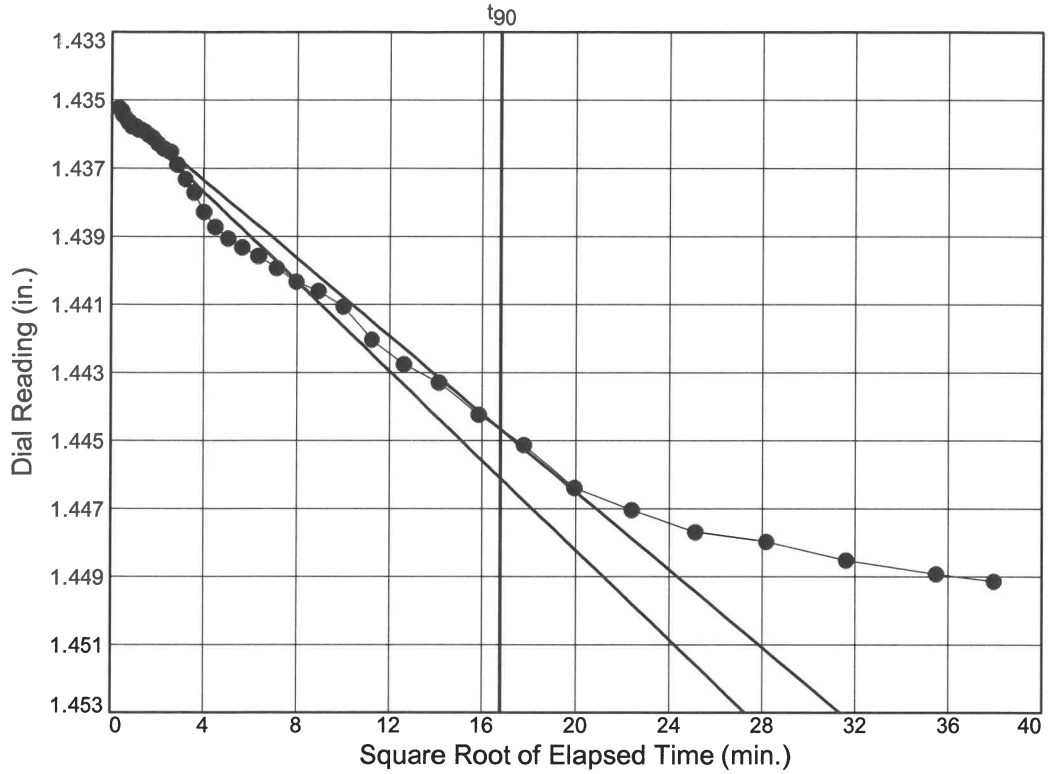
C_v @ T_{90}
10.307 ft.^{2}/day

Figure

Dial Reading vs. Time

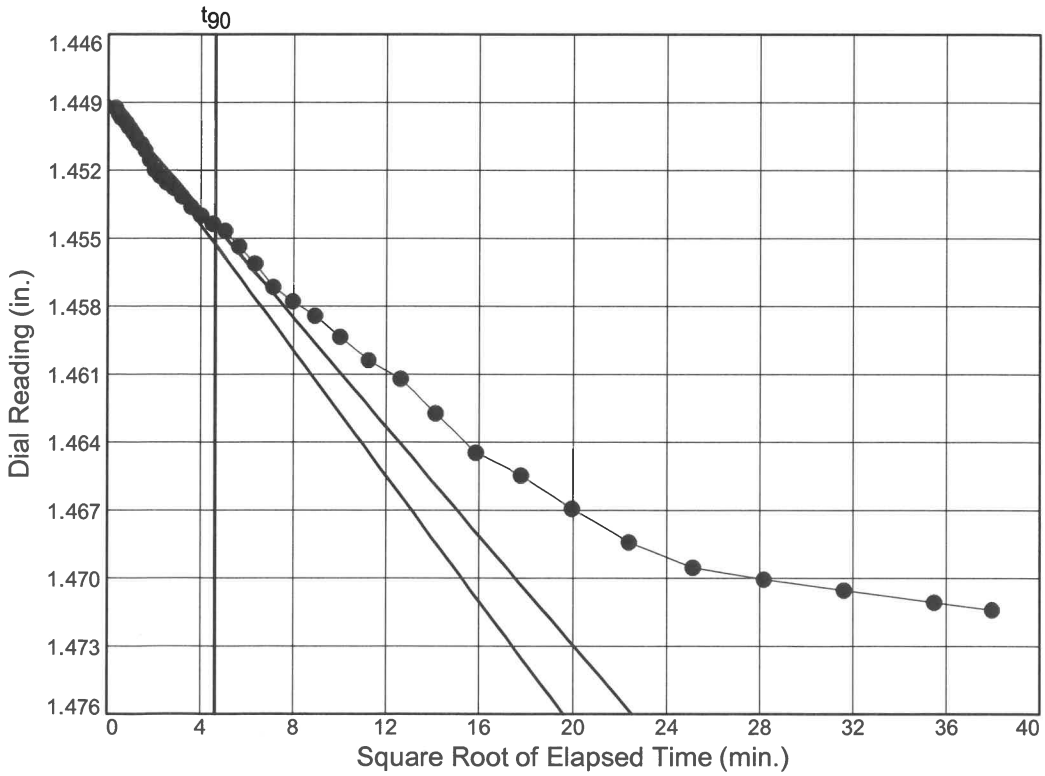
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 9
Load= 2.00 tsf
D_0 = 1.4351
D_{90} = 1.4447
D_{100} = 1.4457
T_{90} = 281.40 min.

C_v @ T_{90}
0.007 ft.^{2}/day



Load No.= 10
Load= 4.00 tsf
D_0 = 1.4489
D_{90} = 1.4544
D_{100} = 1.4550
T_{90} = 21.12 min.

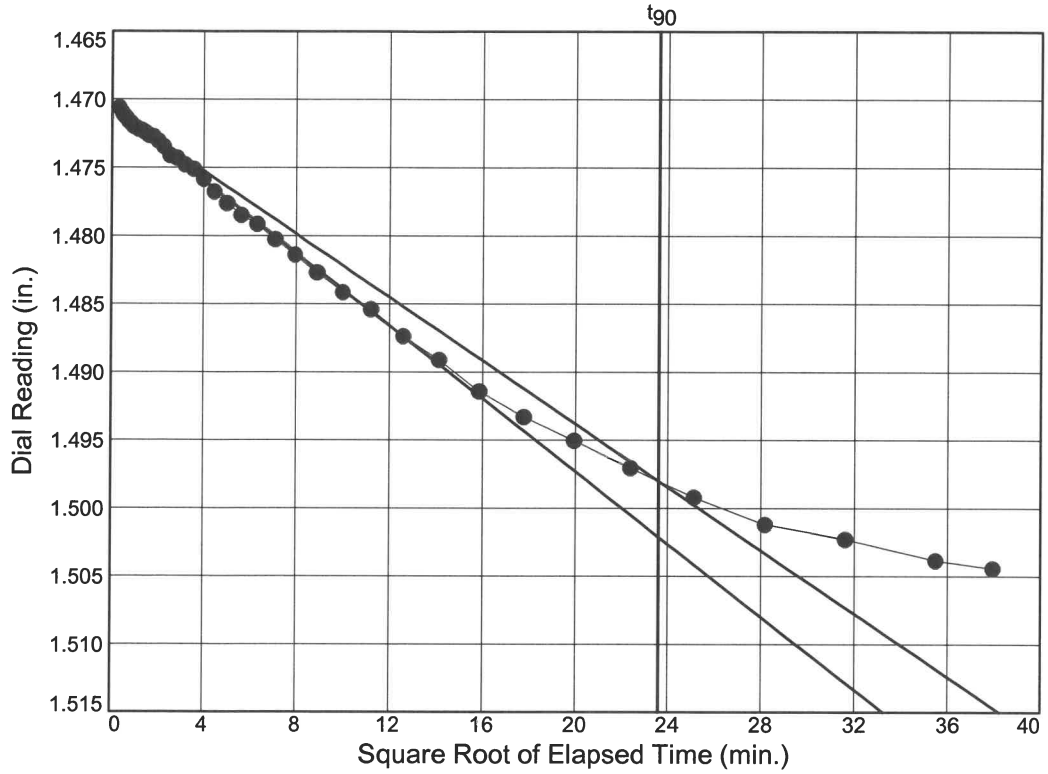
C_v @ T_{90}
0.092 ft.^{2}/day

Figure

Dial Reading vs. Time

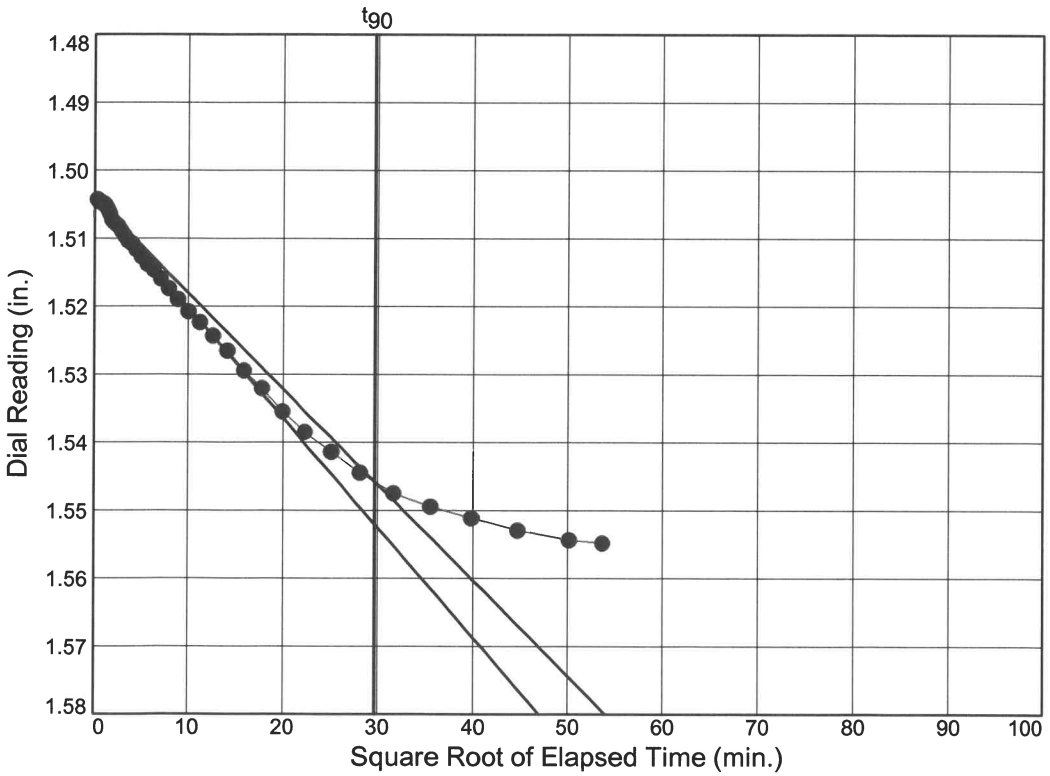
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 11
Load= 8.00 tsf
D_0 = 1.4705
D_{90} = 1.4980
D_{100} = 1.5011
T_{90} = 558.18 min.

C_v @ T_{90}
0.003 ft.^{2}/day



Load No.= 12
Load= 16.00 tsf
D_0 = 1.5039
D_{90} = 1.5458
D_{100} = 1.5505
T_{90} = 881.84 min.

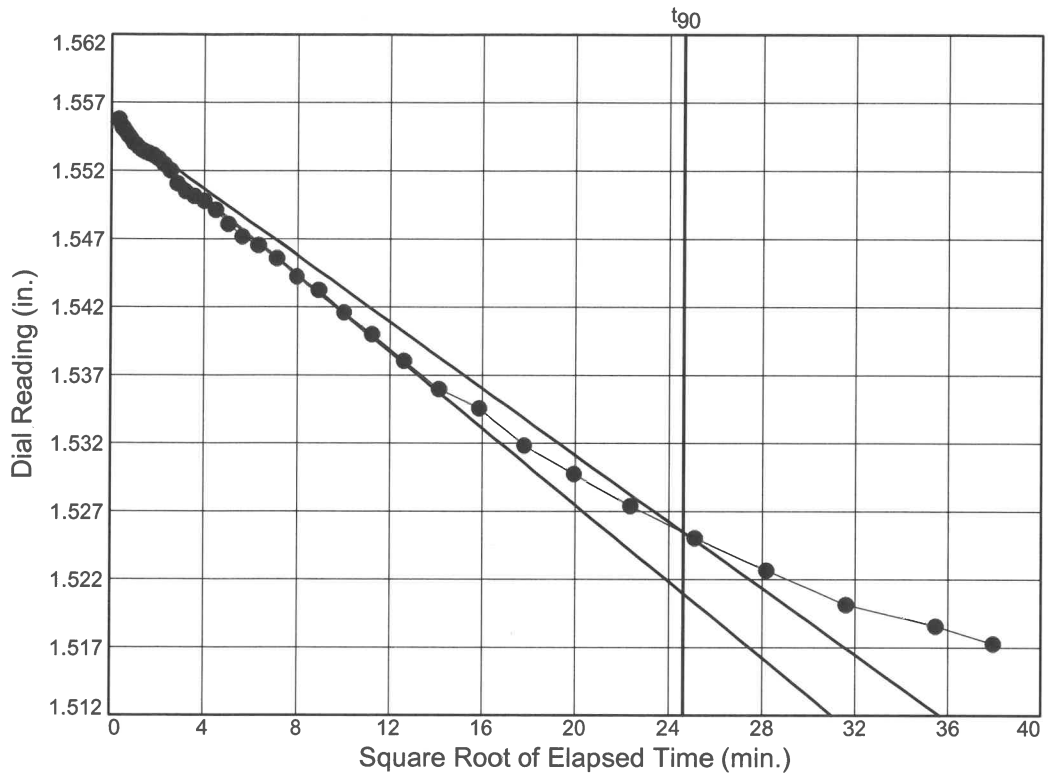
C_v @ T_{90}
0.002 ft.^{2}/day

Figure

Dial Reading vs. Time

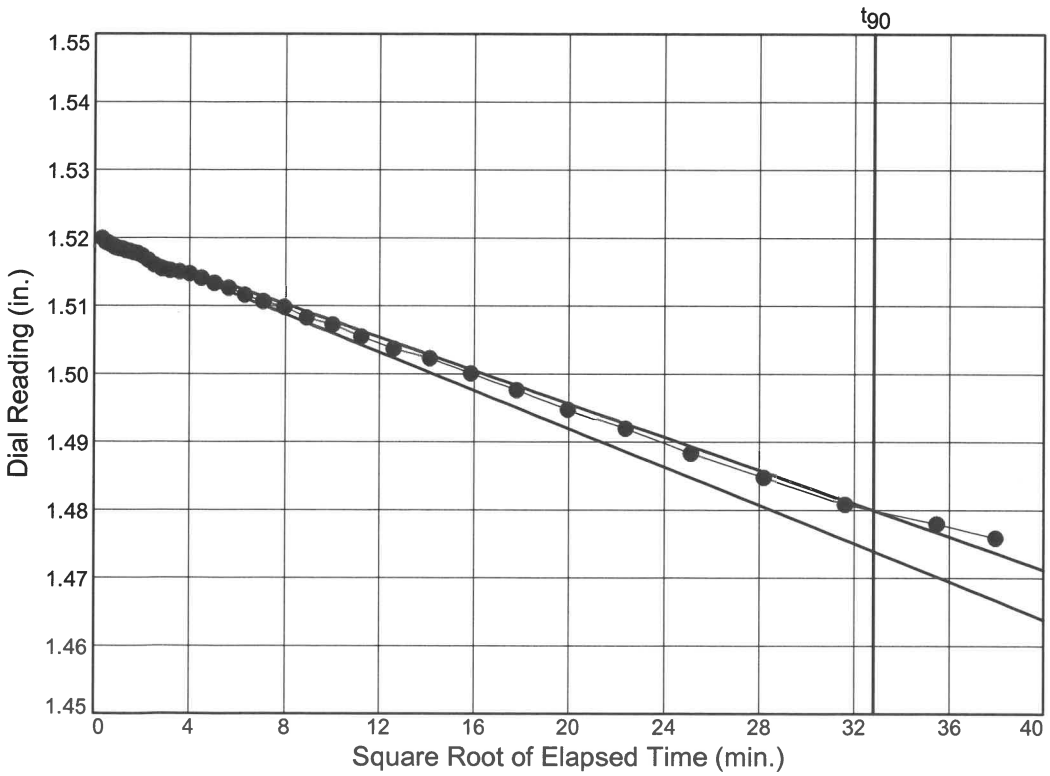
Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 13
Load= 4.00 tsf
D_0 = 1.5556
D_{90} = 1.5255
D_{100} = 1.5221
T_{90} = 607.00 min.

C_v @ T_{90}
0.003 ft.^{2}/day



Load No.= 14
Load= 1.00 tsf
D_0 = 1.5200
D_{90} = 1.4799
D_{100} = 1.4755
T_{90} = 1078.13 min.

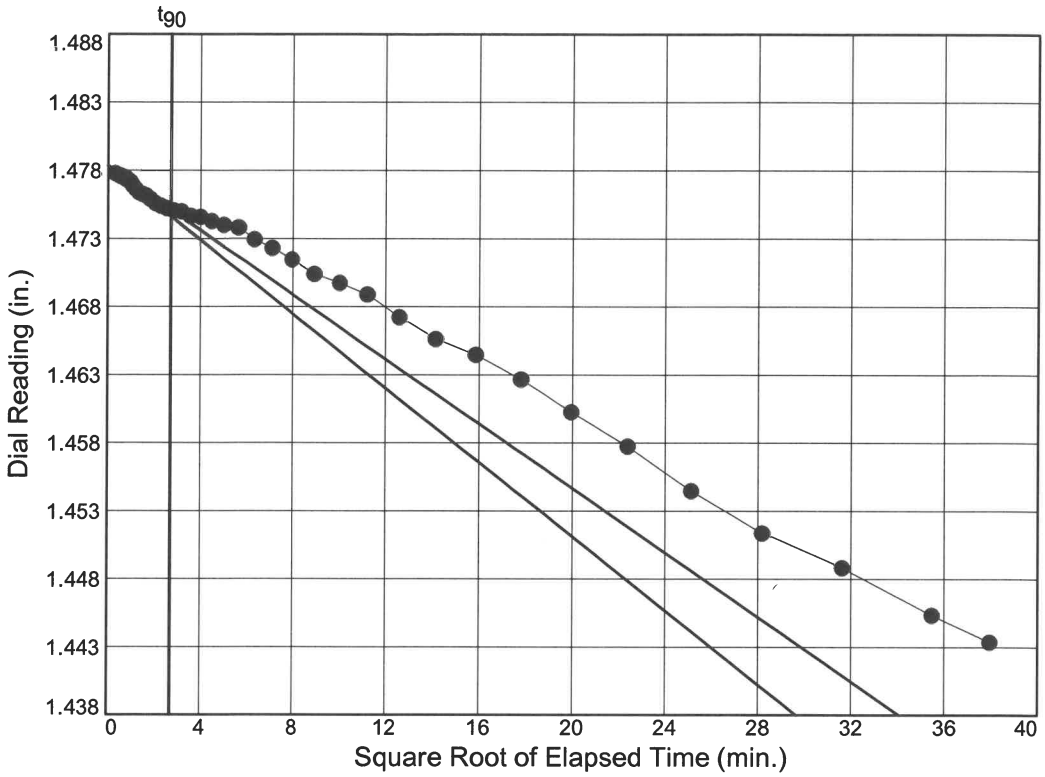
C_v @ T_{90}
0.002 ft.^{2}/day

Figure

Dial Reading vs. Time

Project No.: 4406G
Project: Lakehurst Drive-Carrier Drive Development

Location: SPT-9A Depth: 20'-22'



Load No.= 15

Load= 0.25 tsf

D_0 = 1.4783

D_{90} = 1.4751

D_{100} = 1.4748

T_{90} = 7.34 min.

C_v @ T_{90}
0.265 ft.^{2}/day

Figure

UNDISTURBED SAMPLE CLASSIFICATION

Job Name: Lake Horst
Date: 8/7/9
Job Number: 44066
Tech: TJL
Boring Number: SPT-9A
Proj Mgr.: CPM
Station:
Offset:
Depth: 20.22

From/To

Description: 0"-12" Clay cuttings
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:

Description: 12"-24" Lt Brn. fat clay (CH)
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%: Penetrometer = 1.5 tsf
Torvane = 0.6 tsf
1 min hand = 0.075 tsf

Description:
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:

Description:
MC %: SG.: LL.: Consol:
-200 Unit Wt.: PI.: Triax.:
OC.%:

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UNIT WEIGHT

Project Name:	Lakehurst Drive
Project Number:	4406G
Date:	8-27-19

Boring Number:	SPT-9A
Depth:	20'-22'

Section Depth:	12"-18"			
	1	2	3	Avg.
Length (in.)	5.71	5.69	5.50	5.63
Diameter (in.)	2.86	2.86	2.84	2.85
Area of Sample (sq. inches)				6.39
Volume of Sample (cu. Inches)				36.00
Volume of Sample (cu. feet)				0.02
Wet Weight of Sample (lbs)				2.36
Wet Density (pcf)				113.05
Moisture Content (%)				36.70
Dry Density (pcf)				82.70

Section Depth:				
	1	2	3	Avg.
Length (in.)				
Diameter (in.)				
Area of Sample (sq. inches)				
Volume of Sample (cu. Inches)				
Volume of Sample (cu. feet)				
Wet Weight of Sample (lbs)				
Wet Density (pcf)				
Moisture Content (%)				
Dry Density (pcf)				

Section Depth:				
	1	2	3	Avg.
Length (in.)				
Diameter (in.)				
Area of Sample (sq. inches)				
Volume of Sample (cu. Inches)				
Volume of Sample (cu. feet)				
Wet Weight of Sample (lbs)				
Wet Density (pcf)				
Moisture Content (%)				
Dry Density (pcf)				

Section Depth:				
	1	2	3	Avg.
Length (in.)				
Diameter (in.)				
Area of Sample (sq. inches)				
Volume of Sample (cu. Inches)				
Volume of Sample (cu. feet)				
Wet Weight of Sample (lbs)				
Wet Density (pcf)				
Moisture Content (%)				
Dry Density (pcf)				

Unit Weight ASTM D7263 Method B, Intact Specimen

Moisture Content ASTM D2216

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APPENDIX D
DESIGN CALCULATIONS

APPENDIX D-1
FOUNDATION SETTLEMENT

RECTANGULAR LOADS

Project Name: Lakehurst Dr - Garage Col
Client : Silver Hills
Date : 11/12/2019

Project Number : 4406G
Project Manager: CPM
Computed by : CPM

Increment of stresses obtained using : Boussinesq

Settlement for X = 8.00 (ft) Y = 8.00 (ft)

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (psf)
1	0.00	0.00	16.00	16.00	4000.00

Foundation Elev. = -4.00 (ft) Ground Surface Elev.= 0.00 (ft)
Water table Elev. = -1.50 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Coefficient Comp.	Coefficient Recomp.	Coefficient Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	1.5				110.00	0.00	0.00
2	INCOMP.	11.5				110.00	0.00	0.00
3	COMP.	19.0	0.420	0.110	0.110	107.00	3.43	0.00
					Total Settlement =		3.43	0.00

N°.	Sublayer Thick. (ft)	Sublayer Elev. (ft)	Soil Stresses Initial (psf)	Soil Stresses Increment (psf)	Soil Stresses Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	INCOMP.					
3	1.90	-13.95	754.77	2181.35	6100.00	0.65
4	1.90	-15.85	839.51	1824.69	6300.00	0.55
5	1.90	-17.75	924.25	1528.85	6500.00	0.47
6	1.90	-19.65	1008.99	1288.04	6700.00	0.39
7	1.90	-21.55	1093.73	1093.18	6900.00	0.33
8	1.90	-23.45	1178.47	935.29	7100.00	0.28
9	1.90	-25.35	1263.21	806.72	7300.00	0.24
10	1.90	-27.25	1347.95	701.28	7500.00	0.20
11	1.90	-29.15	1432.69	614.15	7700.00	0.17
12	1.90	-31.05	1517.43	541.56	7900.00	0.15

Total Settlement = 3.43 (in.)

STRIP FOOTING UNIFORM VERTICAL LOADING

Project Name: Lakehurst Dr - Garage Wal
Client : Silver Hills
Date : 11/12/2019

Project Number : 4406G
Project Manager: CPM
Computed by : CPM

Increment of stresses obtained using : Boussinesq

Settlement for X = 6.25 (ft)

Width of strip b = 12.50 (ft)
Ground Surface Elev. = 0.00 (ft)
Water table Elev. = -1.50 (ft)

Foundation Elev. = -4.00 (ft)
p load/unit area = 4000.00 (psf)
Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Coefficient Comp.	Recomp.	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	1.5				110.00	0.00	0.00
2	INCOMP.	11.5				110.00	0.00	0.00
3	COMP.	19.0	0.420	0.110	0.110	107.00	3.71	0.00
						Total Settlement =	3.71	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	INCOMP.					
3	1.90	-13.95	754.77	1639.13	6100.00	0.55
4	1.90	-15.85	839.51	1551.14	6300.00	0.50
5	1.90	-17.75	924.25	1461.46	6500.00	0.45
6	1.90	-19.65	1008.99	1373.98	6700.00	0.41
7	1.90	-21.55	1093.73	1290.99	6900.00	0.37
8	1.90	-23.45	1178.47	1213.63	7100.00	0.34
9	1.90	-25.35	1263.21	1142.26	7300.00	0.31
10	1.90	-27.25	1347.95	1076.83	7500.00	0.28
11	1.90	-29.15	1432.69	1017.03	7700.00	0.26
12	1.90	-31.05	1517.43	962.44	7900.00	0.24

Total Settlement = 3.71 (in.)

RECTANGULAR LOADS

Clay only

Project Name: Lakehurst Dr - Apt Col
Client : Silver Hills
Date : 11/12/2019

Project Number : 4406G
Project Manager: CPM
Computed by : CPM

Increment of stresses obtained using : Boussinesq

Settlement for X = 2.50 (ft) Y = 2.50 (ft)

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (psf)
1	0.00	0.00	5.00	5.00	4000.00

Foundation Elev. = -2.00 (ft) Ground Surface Elev.= 0.00 (ft)
Water table Elev. = -1.50 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Coefficient Comp.	Coefficient Recomp.	Coefficient Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	1.5				110.00	0.00	0.00
2	INCOMP.	13.5				110.00	0.00	0.00
3	COMP.	15.0	0.330	0.090	0.090	113.00	0.39	0.00
					Total Settlement =		0.39	0.00

N°.	Sublayer Thick. (ft)	Sublayer Elev. (ft)	Soil Stresses Initial (psf)	Soil Stresses Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	INCOMP.					
3	1.50	-15.75	845.55	228.04	6100.00	0.09
4	1.50	-17.25	921.45	187.22	6300.00	0.07
5	1.50	-18.75	997.35	156.34	6500.00	0.05
6	1.50	-20.25	1073.25	132.44	6700.00	0.04
7	1.50	-21.75	1149.15	113.59	6900.00	0.03
8	1.50	-23.25	1225.05	98.47	7100.00	0.03
9	1.50	-24.75	1300.95	86.16	7300.00	0.02
10	1.50	-26.25	1376.85	76.01	7500.00	0.02
11	1.50	-27.75	1452.75	67.55	7700.00	0.02
12	1.50	-29.25	1528.65	60.42	7900.00	0.01

Total Settlement = 0.39 (in.)

Clay only
+
0.47" sand
= 0.86" Total

SET2018 - An Update of SET96

Stress History and Settlement Analysis
for Footings on Sand

Project Information

Job Name	Lakehurst Drive - Apartment Buildings
Job Number	4406G
Run Description	column footing 5 x 5

Load Application Data

Footing Shape	Square
Width (ft)	= 5
Length (ft)	= 0
Embedment Depth (ft)	= 3
Elapsed Time After Loading (yrs)	= 0.1
Bearing pressure 1 (psf)	= 4250

Soil and Groundwater Conditions

Moist Unit Weight of Soil (pcf)	= 110
Saturated Unit Weight of Soil (pcf)	= 115
Groundwater Depth (ft)	= 1.5
Phi Angle for Bearing Capacity (deg)	= 32
OC Sand Correction Factor	= 4.00
NC Sand Correction Factor	= 0.50

Calculated Footing Parameters

Initial Overburden Pressure (psf)	= 244
Elastic Modulus Conversion Factor	= 2.50
Iz at Foundation Base	= 0.10
Izp depth (ft)	= 5.5
Maximum Depth of Strain Influence (ft)	= 13
Creep Correction Factor	= 1.00
Ultimate Bearing Capacity (psf)	= 12659

SET2018 - An Update of SET96

Stress History and Settlement Analysis
for Footings on Sand

Results for Bearing Pressure = 4250 psf

Net Bearing Pressure (psf) = 4006
Izp = 0.83
Embedment Correction Factor = 0.97

Layer Base (ft)	qc (tsf)	P'vo (psf)	DeltaP (psf)	Iz	Phi Angle (deg)	Ko	OCR	Eoc (ksf)	DeltaH OC (in)	Enc (ksf)	DeltaH NC (in)
3.5	22	257	4003	0.17	43.5	1.22	11.3	440	0.006	55	0.0248
4	22	283	3934	0.32	43	1.15	10.3	440	0.0113	55	0.0445
4.5	22	310	3725	0.46	42.6	1.09	9.4	440	0.0171	55	0.0592
5	22	336	3395	0.61	42.1	1.04	8.7	440	0.0246	55	0.0614
5.5	25	362	3006	0.75	42.4	1.05	8.9	500	0.0335	63	0.0135
6	25	389	2615	0.8	42	1.01	8.3	500	0.0372	63	0
6.5	25	415	2255	0.74	41.7	0.98	7.8	500	0.0347	63	0
7	25	441	1940	0.69	41.4	0.94	7.3	500	0.0321	63	0
7.5	35	467	1671	0.63	42.8	1.05	8.9	700	0.0211	88	0
8	35	494	1445	0.58	42.5	1.02	8.5	700	0.0193	88	0
8.5	35	520	1256	0.52	42.3	0.98	8	700	0.0174	88	0
9	35	546	1098	0.47	42	0.95	7.6	700	0.0156	88	0
9.5	10	573	965	0.41	34.5	0	0	0	0	0	0
10	10	599	853	0.36	34.2	0	0	0	0	0	0
10.5	10	625	759	0.3	33.9	0	0	0	0	0	0
11	10	652	678	0.25	33.6	0	0	0	0	0	0
11.5	10	678	609	0.19	33.4	0	0	0	0	0	0
12	10	704	550	0.14	33.1	0	0	0	0	0	0
12.5	10	730	498	0.08	32.9	0	0	0	0	0	0
13	10	757	453	0.03	32.6	0	0	0	0	0	0

OC = 0.27 NC = 0.20

For Bearing Pressure = 4250 psf
Total Settlement = 0.47 in

Sand Only

SET2018 - An Update of SET96

Stress History and Settlement Analysis
for Footings on Sand

PENETRATION TEST DATA FILE

Layer No.	Layer Desc.	Base Depth (ft)	N (bpf) or C (tsf)	qc/N	Compressible
1	Shallow dense sands	5	22	1	Y
2	N/A	7	25	1	Y
3	N/A	9	35	1	Y
4	Clay	32	10	1	N

APPENDIX E
CONSTRUCTION RECOMMENDATIONS

E. CONSTRUCTION RECOMMENDATIONS

The following sections include comments on issues related to the geotechnical aspects of the proposed construction. *These recommendations are not intended to dictate construction methods or sequences.* Instead, they are furnished as an aid to design professionals and to identify important construction issues related to foundation and earthwork plans and specifications. These recommendations may also be useful to personnel who observe construction activity.

Prospective contractors for this project should evaluate potential construction problems on the basis of their review of the contract documents, their own knowledge and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed means and methods.

E.1 Groundwater and Surface Water Control

Depending on groundwater levels at the time of construction, excavation depths and final design grades, temporary dewatering may be required to facilitate stable excavations and placement and compaction of fill. The contractor should be required to provide a dewatering system which maintains groundwater levels at least 2 feet below compaction surfaces, including the bottom of all excavations. A system of ditches and sumps may be sufficient in some instances to achieve adequate dewatering, but the contractor should be prepared to install well-point, or other, dewatering systems as necessary.

We do not recommend allowing the contractor to use gravel (e.g.; #57 stone) in excavation bottoms when soils are too wet to achieve the required compaction. Gravel contains small voids that may allow soil raveling to occur, resulting in ground subsidence. Instead the contractor should be made aware, in the plans and again at the pre-construction meeting, that adequate dewatering will be required where groundwater levels are within 2 feet of compaction surfaces and that the use of gravel in excavation bottoms is not allowed.

Additionally, the contractor must provide positive site drainage during site preparation and fill placement. Surface runoff should not be allowed to accumulate on compaction surfaces. Temporary rim ditches may be required to facilitate site preparation by routing drainage away from active construction operations.

E.2 General Site Preparation

Our recommendations regarding routine site preparation of the structure and pavement areas can be summarized as follows:

- Remove all concrete, asphalt, vegetation and organic topsoil, major root systems, buried utilities, sprinkler systems and other deleterious materials from beneath the proposed structure and pavement limits for the cut sections of the site or when no fill is required. For areas where fill is required to achieve final grade, deleterious materials need to be removed from the entire filled area to the toe of the fill slope. Standard clearing, grubbing, and topsoil stripping procedures should be appropriate for most of this site.

- Perform temporary dewatering as required to achieve proper site preparation, fill placement and compaction.

- Allow a Geotechnical Engineer to inspect the site after it has been stripped to verify adequate topsoil and vegetation removal and also to observe subsequent proofrolling.

- In structure and pavement areas where fill is required, proofroll the stripped ground surface using a large vibratory roller (Dynapac CA-25 or equivalent). Proofroll cut areas after excavation to proposed grade to allow adequate compaction of the exposed subsoil.

- ***Exercise extreme caution when operating vibratory equipment near existing structures.*** Nearby structures may be adversely affected by vibratory rolling operations. Operate roller in the static mode if excessive vibrations are experienced by any structures or if the soil subgrade becomes unstable. Preconstruction surveys of nearby structures should be conducted to document baseline conditions. Sensitive structures should be monitored for vibrations during construction operations.

- Proofroll the structure and pavement areas with a minimum of 10 overlapping passes in each of two perpendicular directions. Allow a Geotechnical Engineer, or his representative, to observe proofrolling operations. The purposes of the proofrolling will be to detect unstable soils that yield when subjected to compaction and to densify the near-surface loose sands for support of shallow foundations, soil supported floor slabs, and new pavements.

- Remove material that yields excessively during proofrolling and replace with fill selected and compacted as described in the next section of this report. The Geotechnical Engineer,

based on his observations, should recommend the nature and extent of any remedial work. If the soil subgrade is saturated, or if the fill is at a moisture content over "optimum", then instability may occur and the contractor may be required to implement remedial measures to successfully place and compact the fill.

- Surficial organic soil (PT) as encountered at Borings DCP-1, HA-2 and HA-9 should be removed from all structure and pavement areas to a minimum of 5 feet beyond the proposed structure and pavement limits.

- Silty or clayey sands and clays (SM, SC, CL and CH) may be exposed at the compaction surface during site preparation. These soils can be unstable during proofrolling if they contain excess moisture. The contractor should be prepared to manipulate the moisture content of unstable subgrade soils as necessary to achieve stability and compaction requirements.

- Continue proofrolling until the soil at a depth of 12 inches below the compaction surface has attained a minimum of 95% of the soil's modified Proctor maximum dry density as determined by ASTM Standard D-1557.

- Sand excavated above the water table may have to be wetted to attain the moisture content needed to achieve the required degree of compaction.

- Allow an Engineering Technician, working under the direction of a Geotechnical Engineer registered in the State of Florida, to perform in-place density tests to verify that the required degree of compaction has been achieved.

E.3 Fill Selection, Placement and Compaction

After the contractor proofrolls the site in accordance with the above recommendations, the contractor should place and compact fill required to bring the site to final grade. We recommend that all fill be selected, placed and compacted as follows:

- Use fill material comprised of non-plastic sands with 12% or less fines content. The fill should not contain any significant amount of organic soil (less than 3% by weight) and should be substantially free from roots or other organic or deleterious materials.

- Sands excavated above the water table may have to be wetted to attain the moisture content needed to achieve the required degree of compaction.

- Place fill in level lifts no thicker than 12 inches. Thinner lifts may be needed to achieve compaction in silty or clayey sands.

- Compact fill to a minimum of 95% of the soil's modified Proctor maximum dry density as determined by ASTM Standard D-1557 for each lift of fill placed.

- Allow an Engineering Technician, working under the direction of a registered Geotechnical Engineer, to perform in-place density tests to verify that the recommended degree of compaction has been achieved.

- Extend fill a minimum of 10 feet beyond building limits to prevent possible erosion or undermining of footing bearing soils.

- Provide fill slopes no steeper than 2 horizontal to 1 vertical.

- Compact fill placed in utility trenches to the specifications stated above. However, in restricted working areas, where use of a large vibratory roller is not feasible, compact fill with lightweight, hand-guided compaction equipment and limit lift thicknesses to a maximum of 6 inches.

- All excavations including utility trenches, should comply with the recommendations included in the **Excavation Safety** section in **Appendix C**.

E.4 Foundation Subgrade Preparation

We recommend the following steps be taken during footing excavation and subgrade preparation:

- Excavate footings in accordance with the recommendations presented in the **Excavation Safety** section in **Appendix E.6.**

- Compact footing/slab subgrade soils to a depth of 12 inches below footing bearing elevations to a minimum of 95% of the soil's modified Proctor maximum dry density as determined by ASTM Standard D-1557.

- Perform in-place density tests at 12 inches below the footing/slab bearing elevation to verify footing subgrade density.

- Overexcavate excessively loose or disturbed soils encountered in footing/slab areas and replace with sands selected and compacted in accordance with these recommendations.

- Allow a Geotechnical Engineer, or his representative, to observe footing excavation conditions prior to placement of reinforcing steel or concrete.

- On the basis of the Geotechnical Engineer's observations, remove any unsuitable material encountered in the footing excavations and replace with sand selected and compacted in accordance with the **Fill Selection, Placement and Compaction** section of this report.

E.5 Pavement Subgrade Preparation

Our general recommendations for the pavement subgrade are as follows:

- Prepare pavement areas in accordance with the **General Site Preparation** and **Fill Selection, Placement and Compaction** sections of this report.

- Compact the 12-inch subgrade beneath the base to a minimum of 98% of ASTM D-1557 maximum density.

- Perform in-place density tests to verify pavement subgrade density.

- Stabilize the subgrade beneath a limerock base to a minimum Limerock Bearing Ratio (LBR) of 40.

- Stabilization is not required beneath a soil-cement base or rigid (concrete) pavement. However, the lack of subgrade stabilization should be considered in the pavement design.

E.6 Excavation Safety

The owner and the contractor should be familiar with local, state and federal safety regulations, including current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. Construction site safety is the responsibility of the contractor. The contractor should also be responsible for the means, methods, techniques, sequences, and operations of the construction.

The contractor should be aware that slope height, slope inclination, and excavation depths (including utility trench excavations) should not exceed those specified in local, state, or federal

safety regulations; e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926. *OSHA regulations are strictly enforced and, if not followed, the owner, contractor, earthwork subcontractor or utility subcontractor could be liable for substantial penalties.*

The soil encountered in the borings performed by GEC at this site is primarily sand with varying amounts of silt. We anticipate that OSHA will classify these materials as Type C. OSHA recommends a maximum temporary slope inclination of 1.5 horizontal to 1 vertical for this soil type. Soils encountered in the construction excavations may vary significantly across the site. Our soil classifications are based on the materials encountered in widely-spaced borings. The contractor should verify that similar conditions exist throughout the proposed excavation area. If different subsurface conditions are encountered at the time of construction, GEC should be contacted immediately to evaluate the conditions encountered.

E.7 Stormwater Pond Construction and Maintenance

The stormwater pond analysis and recommendations for soil permeability values included in this report are based upon field and/or laboratory testing of shallow site soils in their in-situ condition. Depending on the project construction schedule, the stormwater pond may be constructed early. This early construction of the pond can allow runoff from the site to enter the pond where excess silt or clay soil can be deposited upon the pond bottom and/or side slopes. This silt and/or clay deposition can effectively reduce the permeability of the exposed in-situ soils to the pond where the pond will no longer function as designed. This same condition can also occur during post-construction pond use but the silt or clay deposit typically takes a much longer time; usually 5 to 10 years of use or longer.

Therefore, to provide for the pond's long term performance as originally designed and to help prevent the deposition of silt or clay on the newly completed pond bottom or sides during and/or immediately after construction, we recommend the following:

- Leave the pond bottom areas slightly higher (6 to 8 inches) than the proposed final grade and do not cut the pond to final grade until construction of the site is essentially complete. The purpose of this recommendation is to help prevent silts, grease, oil or other materials from depositing on the final pond bottom during construction. The stormwater pond system is very susceptible to clogging by construction site-generated sediments. These sediments will inhibit the ability of the pond to function properly. The contractor should wait until the end of construction to remove the 6 to 8 inches of excess soil at the pond bottom and the contractor should scrape the pond bottom thoroughly prior to sodding or seeding.

- If the pond bottom and sides are to be sodded, use sand-grown rather than muck-grown sod. Muck-grown sod has a thick organic base which can greatly reduce infiltration. Seeding of the pond bottom and sides would also be acceptable if proper erosion control measures are used until the vegetation is established.

- Deposition of particulate matter from stormwater runoff can be expected to occur on the pond bottom and sides during normal pond operation. Therefore, a regular pond maintenance program to remove these accumulating deposits and allow the pond to function as designed should be established. This maintenance program should be included in the pond operation manual that is passed to the owner or property manager. GEC will be happy to assist you in creating this pond operation manual for an additional fee if you so desire.

E.8 Reclaimed Concrete Aggregate Base Material

Reclaimed concrete aggregate base material should meet the specifications of the state or local jurisdiction. If a specification for reclaimed concrete aggregate base is not available, GEC recommends the following minimum specifications be included in the project specifications:

- Reclaimed concrete aggregate base material should meet the following gradation requirement:

Sieve Size	Percent by Weight Passing
2 inch	100
3/4 inch	65 to 95
3/8 inch	40 to 85
No. 4	25 to 65
No. 10	20 to 50
No. 50	5 to 25
No. 200	0 to 10

- The reclaimed concrete aggregate base should consist of crushed concrete material derived from the crushing of hard Portland cement concrete.
- Reclaimed concrete aggregate base should not contain plastic soils (i.e.; the minus 0.425 mm (No. 40) sieve material should be non-plastic).

- Reclaimed concrete aggregate base should have a minimum limerock bearing ratio (LBR) of 150.
- Reclaimed concrete aggregate base should be free of all materials that fall under the category of solid waste or hazardous materials as defined by the state or local jurisdiction and should meet all Department of Environmental Protection (DEP) permit requirements which pertain to construction, demolition and recycling of these materials. Reclaimed concrete aggregate base should also be substantially free from other deleterious materials which are not classified as solid waste or hazardous materials and be asbestos free. The following limits should not be exceeded:

Deleterious Material	Percent by Weight
Bituminous Concrete	1
Bricks	1
Wood and other Organic Substances	0.1
Heavy Metals (except Lead)	0.1
Lead	5 ppm
Reinforcing Steel and Welded Wire Fabric	0.1
Plaster and Gypsum Board	0.1

- The reclaimed concrete aggregate base supplier should have DEP permit requirements section 62-701.730 or be qualified as a clean debris source under DEP rules.