GEOTECHNICAL EVALUATION
PROPOSED APARTMENT BUILDING
1211 WESTERN AVENUE
ALBANY, NEW YORK

DENTE FILE NO. JB185131

Prepared For:

GSX VENTURES
7 Old Solomons Island Road, Suite 200
Annapolis, MD 21401

Prepared By:

DENTE GROUP
Watervliet, New York

January 16, 2019

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

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TABLE OF CONTENTS

GEOTECHNICAL EVALUATION
PROPOSED APARTMENT BUILDING
1211 WESTERN AVENUE
ALBANY, NEW YORK

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I. INTRODUCTION

This report presents the results of a geotechnical evaluation completed for a new apartment building planned for construction in Albany, New York. The evaluation was completed in general accord with our proposal number PFDE-18-141 which was authorized by GSX Ventures of Annapolis, Maryland.

In general, our scope of services for this project consisted of the following:

- Site reconnaissance by a Geotechnical Engineer,
- Layout and completion of four deep structural test borings,
- Layout and completion of four shallow site borings with infiltration testing at the boring locations,
- Layout and completion of penetrometer and shear wave velocity testing by ConeTec, Inc. at two locations,
- Preparation of this report, which summarizes the results of the site explorations and presents our recommendations to assist in planning for the geotechnical related aspects of the project.

This report and the recommendations contained within it were developed for specific application to the site and construction planned, as we currently understand it. Corrections in our understanding, changes in the structure locations, their grades, loads, etc. should be brought to our attention so that we may evaluate their effect upon the recommendations offered in this report.

It should be understood this report was prepared, in part, on the basis of a limited field exploration. The borings were advanced at discrete locations and the overburden soils sampled at specific depths. Conditions are only known at the locations and through the depths investigated. Conditions at other locations and depths may be different, and these differences may impact upon the conclusions reached and the recommendations offered. For this reason, we strongly recommend that we be retained to provide site observation services during construction.

A sheet entitled "Important Information about your Geotechnical Engineering Report" prepared by the Geotechnical Business Council is attached. This sheet should never be separated from this report and be carefully reviewed as it sets the only context within which this report should be used.

This report was prepared for informational purposes only and should not be considered part of the contract documents. It should be made available to interested parties in its entirety only. Should the data contained in this report not be adequate for the contractors' bidding purposes, the contractors may make their own investigations, tests, and analyses for use in bid preparation.

The recommendations offered in this report concerning the control of surface and subsurface waters, moisture, or vapor membranes address only conventional Geotechnical Engineering aspects and are not to be construed as recommendations for controlling or providing an environment that would prohibit or control infestations of the structure or its surroundings with mold or other biological agents.

II. SITE AND PROJECT DESCRIPTION

The project site is located at 1211 Western Avenue in Albany, New York as shown on aerial photograph presented in Appendix A. The site is bordered to the south by Western Avenue, west by a multi-story office building, north and east by paved parking lots. At the time this report was prepared, the site was occupied by a multi-story office building and paved parking lot with a few trees on the north side. The ground surface at the site was relatively level with surface elevations in the range of 200 to 203 feet.

It is our understanding that the new building will consist of upper and lower parking levels with five apartment building levels above. The parking ground floor elevations will range between 197 and 202 feet, or about 1 to 6 feet below the existing ground surface. The lower floor levels will be either cast-in-place concrete or steel framed with concrete slabs and the upper levels will consist of wood products. Based upon estimates made by the project's structural engineer, maximum column loads should be less than 500 kips and maximum wall loads less than 15 kips per foot.

III. SITE INVESTIGATION

The subsurface conditions at the site were investigated through the completion of four deep structural test borings, four shallow site borings with infiltration testing adjacent to the borings, and cone penetrometer and shear wave velocity testing at two locations. The approximate test locations shown on the Subsurface Investigation Plan in Appendix B. The ground surface elevations for the borings were estimated by us based upon our interpolation between topographic contours shown on the site plans.

The test borings were completed by us using a standard rotary drill rig equipped using hollow stem augers. As the boreholes were advanced, the overburden soils were sampled, and their relative density determined using split-spoon sampling techniques in general accord with ASTM D1586 procedures. Representative portions of the recovered split-spoon soil samples were transported to our office for visual classification by a Geotechnical Engineer. Individual subsurface logs which were prepared on this basis are presented in Appendix C.

The cone penetrometer and shear wave velocity testing were completed by ConeTec, Inc. of West Berlin, New Jersey. A summary report prepared by ConeTec titled "Presentation of Site Investigation Results" is provided in Appendix D.

The infiltration testing was conducted in accord with the guidelines in Appendix D of the NYS Stormwater Management Design Manual. In general, this entailed the installation of a four-inch diameter PVC pipe at the test depth and filling the pipe with 24 inches of water to presoak the soils. The pipes were then refilled to the 24-inch depth and the drop in the water level was recorded over four 1-hour time periods.

Subsurface Profile

The test borings first penetrated through a few inches of topsoil or asphaltic concrete pavement. Fill materials were present beneath these surfaces on the south side of the site. The fill extended to depths of about 5 to 7 feet and it was composed of relatively loose mixtures of silt and fine sand. In two locations, the assumed remnants of the original topsoil layer were present beneath the fill.

Below the surface materials and fills were native deposits composed of alluvial fine sand and silt. These soils were of a loose to firm relative density and they extended to depths of about 10 to 15 feet where they changed to glacio-lacustrine silt and clay of a loose relative density or soft to very soft consistency. The silt and clay soils extended to the maximum test boring termination depth of 52 feet and in the cone penetrometer test holes they were found to at least 100 feet below the ground surface. Based upon bedrock topography mapping contained in NYS Museum Map and Chart Series Number 37, the surface of rock may be deeper than 150 feet below grade in the general project area.

Although the glacio-lacustrine soils are soft to very soft, they are known to be pre-consolidated to pressures greater than the existing overburden stress. The cone penetrometer and shear wave velocity testing confirmed this and indicated that the soils are pre-consolidated to at least 1500 pounds per square foot above the existing overburden stress.

Groundwater Conditions

Groundwater measurements were obtained at completion of drilling and sampling and the results are noted on the individual subsurface logs. It should be understood these measurements may not accurately reflect the actual groundwater depths because adequate time did not pass after completion of drilling for water to enter and achieve a static level in the augers.

Based upon our interpretation of the subsurface conditions and the depths where the soils changed from “moist” to “wet”, it appears that groundwater was present at depths in the range of 3 to 8 feet below grade at the time of our investigation as tabulated below. This corresponds to groundwater surface elevation in the range of 197 to 198 feet on the north side of the site and 194 to 195 feet on the south side.

Boring	Ground Elevation	Groundwater Depth	Groundwater Elevation
B-1 / I-1	202.0	7	195.0
B-2	201.5	4	197.5
B-3	202.5	8	194.5
B-4	202.0	5	197.0
I-2	201.0	7	194.0
I-3	201.0	3	198.0
I-4	201.0	4	197.0

NOTE: Elevations and Depths are in feet.

Infiltration Test Results

The soils at the infiltration test depths were visually classified as variable mixtures of silt and fine sand. In test locations I-1 and I-3 it appeared that groundwater was present at or near the test depths. After presoaking the test holes with a 24-inch depth of water, between 8 and 24 inches of water remained in the test pipes after 24 hours. As detailed on the test reports in Appendix E, the measured infiltration rates were between 0 and 1 inch per hour.

IV. CONCLUSIONS AND RECOMMENDATIONS

A. General Site Evaluation

The soils present at the project site are relative loose density or soft to very soft consistency and, given the relatively high building loads, it is our opinion that a mat type foundation system should be used to support the structure on these soils. Piles are not recommended for this site because a suitable bearing stratum was not found within the 100-foot maximum depths explored and, based on published geologic information, the bearing stratum may be deeper than 150 feet.

With the ground floor elevations at the planned 1 to 6 feet below existing site grades, a minimum two feet thick base of crushed stone will be required to serve as a stabilizing base and to facilitate dewatering. The mat excavation will require shoring in some areas to protect adjoining properties. It is expected this shoring will be of a height which will not require anchorage.

The estimated high groundwater is about one foot above the lowest floor elevation 197 feet for the below grade parking level. It should be understood this groundwater level can be impacted and possibly raised by activities on adjoining sites, such as building and pavement construction and storm water management practices. The mat and below grade walls should be water-proofed and vapor membranes applied accordingly assuming a groundwater surface elevation at 198 feet. If water proofing is not provided, underdrains should be installed in the stone base to prevent groundwater from rising above a selected grade, these drains would be connected by gravity flow to the site's storm water system. It should also be understood that temporary construction dewatering will be required to construct the lower portions of the mat. The quantity of dewatering will depend on seasonal variations in precipitation and their impact on the local groundwater levels.

The following report sections provide additional preliminary recommendations to assist in planning for design and construction. We should review plans and specifications as they are developed and prior to their release for bidding to allow us to refine our recommendations, if required, and confirm that our recommendations were properly interpreted and applied.

B. Seismic Design Considerations

For seismic design purposes, we evaluated the site conditions in accord with Section 1613 of the New York State Building Code (2015). On this basis, it was determined that Seismic Site Class "D - Stiff Profile" is applicable to this project. This determination was made based on shear wave testing at the site which yielded an average shear wave velocity in the top 100 feet equal to about 690 feet per second.

Using the general building code procedures and applying the Site Class "D" designation, we obtained the following spectral response parameters.

Short Period Spectral Response Acceleration:	$S_S = 0.182$
1-Second Period Spectral Response Acceleration:	$S_1 = 0.070$

Short Period Site Coefficient:	$F_a = 1.6$
1-Second Period Site Coefficient:	$F_v = 2.4$

Short Period Design Spectral Response Parameter:	$S_{DS} = 0.195$
1-Second Period Design Spectral Response Parameter:	$S_{D1} = 0.112$

Assuming occupancy Category's I-III, building code Tables 1613.5.6 (1) and (2) define the project as Seismic Design Category "B".

C. Temporary Shoring and Excavations

Temporary excavation side slopes should be made no steeper than 1 vertical to 1.5 horizontal (1V:1.5H) as required by OSHA for a Type C soil.

All excavations should be completed so as not to undermine adjacent foundations or utilities. In general, excavations should not encroach within an existing foundation or utilities zone of influence defined by a line extending out and down from the existing foundation, grade beam, or utility at an inclination of 1V:1.5H horizontal. Excavations which encroach within this zone should be sheeted, shored, and braced as required to support the soil and adjacent structure loads, or the foundation or utility should be underpinned to establish bearing at a deeper level.

All shoring should be designed by a NYS registered Professional Engineer. For design purposes, the silt and fine sand soils in the upper 10 to 15 feet can be assumed to have a total unit weight equal to 120 pounds per cubic foot (pcf) and friction angle equal to 30 degrees. For the underlying silt and clay soils, a total unit weight equal to 114 pcf and friction angle equal to 28 degrees may be assumed.

D. Site Preparation and Earthwork

Site preparation should begin with installation of required temporary shoring. Excavation may then proceed to establish subgrade elevation at the elevation required for the installation of the crushed stone base. The final few feet of excavation should be completed using a backhoe equipped with a smooth-edged bucket to limit disturbance of the subgrade soils.

For planning purposes, it should be assumed that the stone base must be at least 24-inches thick. The thickness of the stone base may be reconsidered based upon the conditions encountered during construction. Dewatering from the stone base should be conducted on a continuous basis during construction as required.

Portions of the excavation may extend near to or below groundwater in soils composed of fine sand and/or silt. These subgrades will be sensitive to disturbance and the excavation should thus be completed incrementally to restrict all construction equipment traffic from traveling over the surfaces until the stone is placed. As the excavation is performed, a geotextile stabilization fabric should be placed over the surface followed by a minimum 24-inch thick base of crushed stone. If existing fills are encountered at the plan subgrade elevation, they should be removed completely and replaced with crushed stone.

The crushed stone should consist of ASTM C33 Blend 57 aggregate. It should be placed over a geotextile stabilization fabric such as Mirafi 500X or equivalent. The stone should be placed as a single lift and the surface chinked together using a vibratory roller with a maximum 5-ton static weight.

For planning purposes, it should be assumed that all excavated materials must be wasted off-site as they will generally be either too fine grained or wet for reuse as backfill. All fill and backfill of the building walls and beneath pavements should be completed using an Imported Structural Fill composed of well graded sand and gravel or crusher-run stone. The imported material should have 100% of its particles finer than the 3" sieve, between 35 and 65% passing the No.4 sieve, and less than 15% passing the No. 200 sieve. The fill should not contain recycled asphalt, bricks, glass, pyritic shale or recycled concrete, unless the recycled concrete is from a NYSDOT approved stockpile and even then, only with the owner’s specific consent.

The Structural Fill should be placed in uniform loose layers no more than about one-foot thick where heavy vibratory compaction equipment is used. Smaller lifts should be used where hand operated equipment is required for compaction. Each lift should be compacted to no less than 95 percent of the maximum dry density for the soil established by the Modified Proctor Compaction Test, ASTM D1557. For fill or backfill placed in landscape areas, the compaction standard may be reduced to 90 percent.

E. Mat Foundation

The mat foundation may be seated on the recommended crushed stone base layer. The mat should be water-proofed and, if moisture sensitive coatings to the floor are planned, vapor membranes should be applied accordingly. The water-proofing should be designed assuming a groundwater surface elevation at 198 feet. Underdrains are not required if the mat and below grade walls are water-proofed. If water proofing is not provided, underdrains may be installed in the stone base to prevent groundwater from rising above a selected grade, these drains would be connected by gravity flow to the site's storm water system.

For preliminary planning purposes, the mat can be designed assuming a maximum net allowable bearing pressure equal to 1500 pounds per square foot and vertical modulus of subgrade reaction equal to 100 pounds per cubic inch. It should be understood the modulus is for a standard 12-inch diameter plate and it must be adjusted accordingly based upon the size of the mat.

Settlement of the mat should be less than 1.5 inches. This settlement should occur within a few weeks after construction is completed and each load increment is applied.

F. Retaining Walls

Building walls that retain earth should be designed to resist lateral earth pressures together with any applicable surcharge loads. Active earth pressures may be assumed for walls that are free to deflect as the backfill is placed and surcharge loads applied. At-rest earth pressures should be assumed for walls that are braced prior to backfilling or applying surcharge loads. The following design parameters are provided to assist in determining the lateral wall loads, whichever apply. The parameters include no factor of safety and they assume that the walls are backfilled with Structural Fill.

STRUCTURAL BACKFILL

Soils Angle of Internal Friction = 30 degrees
Coefficient of At-Rest Earth Pressure = 0.50
Coefficient of Active Earth Pressure = 0.33
Coefficient of Passive Earth Pressure = 3.00
Total Unit Weight of Compacted Backfill = 120 pcf
Coefficient of Sliding Friction on Stone Base = 0.55

Below grade walls should be water-proofed assuming a groundwater surface elevation at 198 feet. Water-proofing is not required if a standard perimeter foundation drain can be installed and outlet by gravity flow to the site's storm water management system. The drain may consist of a nominal 4-inch diameter perforated PVC or slotted HDPE pipe embedded at the base of a minimum 12-inch wide column of clean crushed stone (NYSDOT No. 1 and 2 size aggregate). The stone should be wrapped in a filter fabric (Mirafi 140N or equivalent) to provide separation from the surrounding soils.

G. Pavements

Assuming new pavements adjoining the building are subject to automobile traffic with occasional delivery type truck, the following section may be considered.

PAVEMENT SECTION	NYSDOT SPECIFICATION	THICKNESS (inches)
Wearing Course – Asphaltic Concrete	Section 403 – Type 6	1.5
Binder – Asphaltic Concrete	Section 403 – Type 3	2.5
Base – Crusher-Run Stone	Section 304 – Type 2	12
Fabric – Mirafi 500X or Eq.	-	Single Ply

New pavement areas should be stripped of existing asphalt and/or topsoil. The subgrade surfaces should be proof-compacted using a smooth drum roller with a static weight of at least 10-tons. The roller should operate in its vibratory mode unless directed otherwise by the Geotechnical Engineer observing the work. The roller should complete at least four overlapping passes over all surfaces. If soft areas are encountered, they should be investigated to determine the cause and stabilized accordingly. All pavement base course materials should be compacted to 95 percent of the material's maximum dry density as established through the Modified Proctor Test, ASTM D-1557.

Sidewalks and pavements constructed upon the site's soils will heave as frost seasonally penetrates the subgrades. The magnitude of the seasonal heave will vary with many factors and result in differential movements. As the frost leaves the ground, the sidewalks and pavements will settle back, but not entirely in all areas, and this may accentuate the differential movements across the pavement areas. Where curbs, walks, and storm drains meet these pavements, these differential heave and settlements may result in undesirable movements and create trip hazards. To limit the magnitude of heave and the creation of these uneven joints to generally tolerable magnitudes for most winters, a 16-inch thick crushed stone base course, composed of NYSDOT No. 1 and 2 size aggregate, may be placed beneath the sensitive sidewalk, drive, etc. areas. The stone layer must have an underdrain placed within it.

It should be understood the recommended pavement sections were not designed to support heavy construction equipment loads which would require an augmented section. The contractor should construct temporary haul and construction roadways and routes about the site as appropriate for the specific weather conditions and construction equipment he intends to employ at the site, and the overburden soil conditions encountered in the specific areas. Construction period traffic should not be routed across the recommended pavement sections unless augmented.

Finally, all pavements require routine maintenance and occasional repairs. Failure to provide maintenance and complete the required repairs in a timely manner will result in a shortened pavement service life.

H. Recommended Additional Services

We should review final plans and specifications prior to their release to bidders to confirm that our recommendations have been understood an implemented.

We should also be retained to monitor earthwork and bearing grade preparations for foundations and pavements. It should be understood the actual subsurface conditions that exist across this site will only be known when the site is excavated. Our presence during the earthwork and foundation construction phases will allow validation of the subsurface conditions assumed to exist for this study and the design recommended in this report.

V. CLOSURE

This report was prepared for specific application to the project site and the construction planned using methods and practices common to Geotechnical Engineering in the area and at the time of its preparation. No other warranty, either expressed or implied, is made.

We appreciate the opportunity to be of service. Should questions arise or if we may be of any other service, please contact us at your convenience.

Prepared By,

Edward Gravelle

1/16/19

Edward C. Gravelle, P.E.

Senior Engineer

1/16/19

Fred A. Dente, P.E.

Principal

APPENDIX A

AERIAL PHOTOGRAPH

Proposed Apartment Building
Albany, New York

PROPOSED APARTMENT BUILDING
1211 WESTERN AVENUE
ALBANY, NEW YORK

APPENDIX B

SUBSURFACE INVESTIGATION PLAN

Proposed Apartment Building
Albany, New York

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PLAN NOTES:

DEMOLITION SEQUENCING TO BE COORDINATED WITH CITY OF ALBANY.

SEE EROSION & SEDIMENT CONTROL PLAN FOR SEQUENCE OF CONSTRUCTION NOTES AND EROSION & SEDIMENT CONTROL REQUIREMENTS.

DEMOLITION NOTES:

1. INSTALL CONSTRUCTION FENCE AROUND PROPERTY BOUNDARY.
2. COORDINATE WITH UTILITY PROVIDER TO REMOVE UNDERGROUND WIRES.
3. DEMOLISH EXISTING BUILDING. CONTRACTOR SHALL OBTAIN APPROPRIATE PERMITS FOR DEMOLITION AND TERMINATION OF ALL EXISTING UTILITY SERVICES SERVING THE SITE.
4. REMOVE ALL EXISTING ASPHALT DOWN TO SUBBASE AND DISPOSE OF ASPHALT OFF-SITE.
5. REMOVE ALL EXISTING VEGETATION.

LEGEND (Added by Dente Group)

- B- Approximate Test Boring Location
- I- Approximate Test Boring and Infiltration Test Location
- CPT- Approximate Cone Penetrometer Test Location

0 32.808465.6168 131.2336Feet

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1211 Western Avenue

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No.	Revision	Date	Appvd.
3	Revised Building	5/1/2018	MBT
2	Revised Building	3/6/2018	MBT
1	Revised Building	12/28/2017	MBT

Designed by AWK Checked by MBT

Issued for Site Plan Review Date May 1, 2018

Not Approved for Construction

Drawing Title
Demolition Plan

Drawing Number
C-2

Sheet 2 of 8

Project Number
26138.00

Saved Monday, April 30, 2018 10:46:01 AM AKARL Plotted Monday, April 30, 2018 2:31:25 PM Moliel, Andrew

APPENDIX C

TEST BORING LOGS AND KEY

Proposed Apartment Building
Albany, New York

INTERPRETATION OF SUBSURFACE LOGS

The Subsurface Logs present observations and the results of tests performed in the field by the Driller, Technicians, Geologists and Geotechnical Engineers as noted. Soil/Rock Classifications are made visually, unless otherwise noted, on a portion of the materials recovered through the sampling process and may not necessarily be representative of the materials between sampling intervals or locations.

The following defines some of the terms utilized in the preparation of the Subsurface Logs.

SOIL CLASSIFICATIONS

Soil Classifications are visual descriptions on the basis of the Unified Soil Classification ASTM D-2487 and USBR, 1973 with additional comments by weight of constituents by BUHRMASTER. The soil density or consistency is based on the penetration resistance determined by ASTM METHOD D1586. Soil Moisture of the recovered materials is described as DRY, MOIST, WET or SATURATED.

SIZE DESCRIPTION		RELATIVE DENSITY/CONSISTENCY (basis ASTM D1586)			
SOIL TYPE	PARTICLE SIZE	GRANULAR SOIL		COHESIVE SOIL	
BOULDER	> 12	DENSITY	BLOWS/FT.	CONSISTENCY	BLOWS/FT.
COBBLE	3" - 12"	LOOSE	< 10	VERY SOFT	< 3
GRAVEL-COARSE	3" - 3/4"	FIRM	11 - 30	SOFT	4 - 5
GRAVEL - FINE	3/4" - #4	COMPACT	31 - 50	MEDIUM	6 - 15
SAND - COARSE	#4 - #10	VERY COMPACT	50 +	STIFF	16 - 25
SAND - MEDIUM	#10 - #40			HARD	25 +
SAND - FINE	#40 - #200				
SILT/NONPLASTIC	< #200				
CLAY/PLASTIC	< #200				

SOIL STRUCTURE		RELATIVE PROPORTION OF SOIL TYPES	
STRUCTURE	DESCRIPTION	DESCRIPTION	% OF SAMPLE BY WEIGHT
LAYER	6" THICK OR GREATER	AND	35 - 50
SEAM	6" THICK OR LESS	SOME	20 - 35
PARTING	LESS THAN 1/4" THICK	LITTLE	10 - 20
VARVED	UNIFORM HORIZONTAL PARTINGS OR SEAMS	TRACE	LESS THAN 10

Note that the classification of soils or soil like materials is subject to the limitations imposed by the size of the sampler, the size of the sample and its degree of disturbance and moisture.

ROCK CLASSIFICATIONS

Rock Classifications are visual descriptions on the basis of the Driller's, Technician's, Geologist's or Geotechnical Engineer's observations of the coring activity and the recovered samples applying the following classifications.

CLASSIFICATION TERM	DESCRIPTION
VERY HARD	NOT SCRATCHED BY KNIFE
HARD	SCRATCHED WITH DIFFICULTY
MEDIUM HARD	SCRATCHED EASILY
SOFT	SCRATCHED WITH FINGERNAIL
VERY WEATHERED	DISINTEGRATED WITH NUMEROUS SOIL SEAM
WEATHERED	SLIGHT DISINTEGRATION, STAINING, NO SEAMS
SOUND	NO EVIDENCE OF ABOVE
MASSIVE	ROCK LAYER GREATER THAN 36" THICK
THICK BEDDED	ROCK LAYER 12" - 36"
BEDDED	ROCK LAYER 4" - 12"
THIN BEDDED	ROCK LAYER 1" - 4"
LAMINATED	ROCK LAYER LESS THAN 1"
FRACTURES	NATURAL BREAKS AT SOME ANGLE TO BEDS

Core sample recovery is expressed as percent recovered of total sampled. The ROCK QUALITY DESIGNATION (RQD) is the total length of core sample pieces exceeding 4" length divided by the total core sample length for N size cored.

GENERAL

- Soil and Rock classifications are made visually on samples recovered. The presence of Gravel, Cobbles and Boulders will influence sample recovery classification density/consistency determination.
- Groundwater, if encountered, was measured and its depth recorded at the time and under the conditions as noted.
- Topsoil or pavements, if present, were measured and recorded at the time and under the conditions as noted.
- Stratification Lines are approximate boundaries between soil types. These transitions may be gradual or distinct and are approximated.

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **B-1.1**

PROJECT: Apartment Building	**DATE**	START: 8/31/18	FINISH: 8/31/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 202.0’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		BLOWS ON SAMPLER					CLASSIFICATION / OBSERVATIONS
DEPTH	#	6"	12"	18"	24"	N	
	1	1	2				**FILL:** ± 4” Topsoil over Brown Fine SAND and
				2	2	4	SILT, Moist
	2	1	1				Similar, Becomes Wet
				2	1	3	
5'							**(MOIST TO WET, LOOSE)**
	3	1	4				**TOPSOIL:** Dark Gray SILT, trace organics
				4	7	8	Lt. Grayish Brown SILT and Fine SAND, Moist
	4	3	4				Grades Grayish Brown Fine SAND, Some
				4	7	8	Silt, Wet
10'	5	3	2				Grades Little Silt
				2	4	4	
	6	8	8				Similar
				8	5	16	
15'							**(MOIST TO WET, LOOSE TO FIRM)**
	7	1	2				Gray Varved SILT and CLAY, Wet
				3	2	5	
20'							
	8	WH	1				Similar
				2	2	3	
25'							
	9	WH	WH				Similar
				2	2	2	

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **B-1.2**

PROJECT: Apartment Building	**DATE**	START: 8/31/18	FINISH: 8/31/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 202.0’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		**BLOWS ON SAMPLER**					**CLASSIFICATION / OBSERVATIONS**
DEPTH	#	6"	12"	18"	24"	N	
	10	WH	WH				Gray Varved SILT and CLAY, Wet
				1	2	1	
35'	11	WH	WH				Similar
				WH	WH	WH	
40'	12	WH	WH				Similar
				WH	WH	WH	
45'	13	WH	WH				Similar
				WH	1	WH	
50'	14	WH	WH				Similar
				WH	1	WH	**(WET, SOFT TO VERY SOFT)**
							Boring Ended at 52.0’
							Groundwater in augers at 7.4’ below grade at
55'							completion of drilling and sampling.

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **B-2**

PROJECT: Apartment Building	**DATE**	START: 9/05/18	FINISH: 9/05/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 201.5’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		BLOWS ON SAMPLER					CLASSIFICATION / OBSERVATIONS
DEPTH	#	6"	12"	18"	24"	N	
	1	5	5				± 1” Asphalt and ± 3” Base Material
				5	3	10	**TOPSOIL:** Dk. Brown SILT and Fine SAND
	2	1	3				Light Gray/Brown Mottled SILT, trace fine
				7	11	10	sand, Moist
	3	6	5				Grades Brown SILT, Little Fine Sand, Wet
5'				6	5	11	
	4	3	5				Grades Grayish Brown
				4	3	9	
							(MOIST TO WET, LOOSE TO FIRM)
10'	5	1	2				Gray Varved SILT and CLAY, Wet
				3	1	5	
15'	6	1	1				Similar
				2	1	3	
20'	7	WH	WH				Similar
				1	2	1	**(WET, SOFT TO VERY SOFT)**
							Boring Ended at 22.0’
							Groundwater in augers at 21.1’ below grade at
25'							completion of drilling and sampling.

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **B-3**

PROJECT: Apartment Building	**DATE**	START: 8/31/18	FINISH: 8/31/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 202.5’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		**BLOWS ON SAMPLER**					**CLASSIFICATION / OBSERVATIONS**
DEPTH	#	6"	12"	18"	24"	N	
							± 5” Asphalt and ± 7” Base Material
	1	1	1				**FILL:** Dark Brown/Gray SILT and Fine SAND
				2	1	3	
	2	WH	1				Similar
				1	2	2	
5'	3	1	1				Similar
				1	3	2	
	4	7	5				**(MOIST, LOOSE)**
				6	7	11	Grayish Brown Fine SAND, Little Silt, Wet
10'	5	2	3				Similar
				4	7	7	
							(WET, FIRM TO LOOSE)
15'	6	1	2				Gray Varved SILT and CLAY, Wet
				1	2	3	
20'	7	1	2				Similar
				2	2	4	**(WET, SOFT)**
							Boring Ended at 22.0’
							Groundwater in augers at 9.6’ below grade at
25'							completion of drilling and sampling.

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **B-4**

PROJECT: Apartment Building	**DATE**	START: 9/05/18	FINISH: 9/05/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 202.0’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		**BLOWS ON SAMPLER**					**CLASSIFICATION / OBSERVATIONS**
DEPTH	#	6"	12"	18"	24"	N	
							± 3.5” Asphalt and ± 4” Base Material
	1	3	4				Light Brown Mottled SILT, Moist
				5	6	9	
	2	3	3				Grades Grayish Brown Fine SAND and SILT
				3	4	6	
5'	3	2	2				Grades Grayish Brown SILT, Wet
				1	2	3	
	4	WH	1				Similar
				3	3	4	
							(MOIST TO WET, LOOSE)
10'	5	1	2				Gray and Brown Varved SILT, Wet
				2	1	4	
15'	6	WH	1				Gray Varved SILT and CLAY, Wet
				1	2	2	
20'	7	1	2				Similar
				2	2	4	**(WET, LOOSE / SOFT TO VERY SOFT)**
							Boring Ended at 22.0’
							No measurable groundwater in augers at
25'							completion of drilling and sampling.

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **I-1**

PROJECT: Apartment Building	**DATE**	START: 8/31/18	FINISH: 8/31/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 202.0’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		**BLOWS ON SAMPLER**					**CLASSIFICATION / OBSERVATIONS**
DEPTH	#	6"	12"	18"	24"	N	
							Augered to 4.5’ depth with no sampling and installed infiltration test pipe. Refer to log for test boring B-1 for soil data.
5'							Boring Ended at 4.5’
10'							
15'							
20'							
25'							

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **I-2**

PROJECT: Apartment Building	**DATE**	START: 9/05/18	FINISH: 9/05/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 201.0'
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		BLOWS ON SAMPLER					CLASSIFICATION / OBSERVATIONS
DEPTH	#	6"	12"	18"	24"	N	
	1	1	4				**POSSIBLE FILL:** ± 2" Topsoil over Brown SILT
				3	3	7	trace fine sand, Moist
	2	2	3				Similar with seam Black SILT, trace organics
				4	4	7	
	3	1	1				Grades Dark Gray SILT, Little Fine Sand, Moist
5'				1	2	2	
	4	1	3				**(MOIST, LOOSE)**
				3	8	6	Gray Fine SAND, Little Silt, Wet
	5	3	4				
				4	4	8	**(WET, LOOSE)**
10'							Boring Ended at 10.0'
							Groundwater in augers at 6.9' below grade at
							completion of drilling and sampling.
15'							
20'							
25'							

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **I-3**

PROJECT: Apartment Building	**DATE**	START: 9/05/18	FINISH: 9/05/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 201.0’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		**BLOWS ON SAMPLER**					**CLASSIFICATION / OBSERVATIONS**
DEPTH	#	6"	12"	18"	24"	N	
	1	5	5				Light Brown Mottled SILT, Moist
				5	4	10	
	2	2	5				Grades Light Gray/Brown Mottled SILT, Little
				5	5	10	Fine Sand, Wet
	3	1	1				Grades Gray SILT, Wet
5'				1	1	2	
	4	1	2				Grades Light Grayish Brown
				3	3	5	
	5	3	3				Grades Light Gray
				4	3	7	**(MOIST TO WET, LOOSE)**
10'							Boring Ended at 10.0’
							No measurable groundwater in augers at
							completion of drilling and sampling.
15'							
20'							
25'							

DENTE GROUP, A TERRACON COMPANY	SUBSURFACE LOG **I-4**

PROJECT: Apartment Building	**DATE**	START: 9/05/18	FINISH: 9/05/18

LOCATION: 1211 Western Ave. - Albany, NY	**METHODS:** 3-1/4" Hollow Stem Augers
CLIENT: GSX Ventures, LLP	with ASTM D1586 Sampling
JOB NUMBER: JB185131	**SURFACE ELEVATION:** ± 201.0’
DRILL TYPE: CME 45 Trailer Mounted Rig	**CLASSIFICATION:** E. Gravelle, PE

SAMPLE		**BLOWS ON SAMPLER**					**CLASSIFICATION / OBSERVATIONS**
DEPTH	#	6"	12"	18"	24"	N	
	1	7	5				Brown Fine SAND, Some Silt, Moist
				3	2	8	
	2	1	2				Grades Light Grayish Brown
				3	7	5	**(MOIST, LOOSE)**
	3	3	3				Gray SILT, Wet
5'				2	2	5	
	4	3	4				Similar
				5	5	9	
	5	3	3				Similar
				3	2	6	**(WET, LOOSE)**
10'							Boring Ended at 10.0’
							No measurable groundwater in augers at completion of drilling and sampling.
15'							
20'							
25'							

APPENDIX D

CONE PENETROMETER TEST REPORT

Proposed Apartment Building
Albany, New York

PRESENTATION OF SITE INVESTIGATION RESULTS

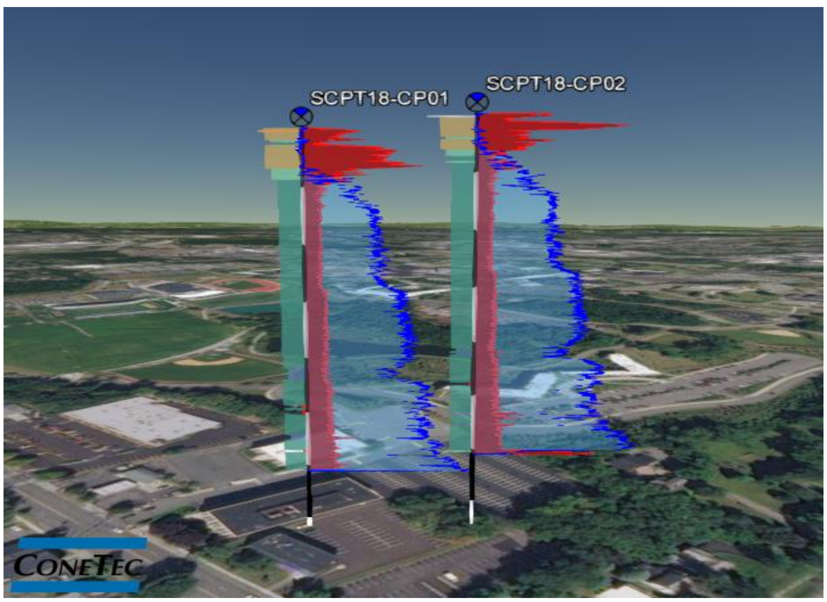
1211 Western Avenue
Albany, New York

Prepared for:

Dente Group

ConeTec Job No: 18-53103

Project Start Date: 31-Aug-2018
Project End Date: 31-Aug-2018
Report Date: 4-Sep-2018



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Introduction

The enclosed report presents the results of a seismic piezocone penetration testing (SCPTu or SCPT) program carried out at the 1211 Western Avenue site located in Albany, New York. The site investigation program was conducted by ConeTec Inc. (ConeTec), under contract to Dente Group (Dente) of Watervliet, New York.

A total of 2 seismic cone penetration tests were completed at 2 locations. The SCPT program was performed to evaluate the subsurface soil conditions. SCPT sounding locations were selected and numbered under supervision of Dente personnel (Mr. Dave Mineau).

Project Information

Project	
Client	Dente Group
Project	1211 Western Avenue, Albany, NY
ConeTec project number	18-53103

A map from CESIUM including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT Truck Rig	25 ton truck mounted (twin cylinders)	SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
SCPT	GPS (GlobalSat MR-350)	32618 (WGS 84 / UTM North)

Cone Penetration Test (CPT)	
Depth reference	Ground surface at the time of the investigation.
Tip and sleeve data offset	0.1 meter. This has been accounted for in the CPT data files.
Pore pressure dissipation (PPD) tests	One pore pressure dissipation test was completed to determine the phreatic surface.
Additional Comments	Shear wave velocity tests were conducted at five foot depth intervals at both locations.

Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
310:T1000F10U500	310	10	150	1000	10	500

Limitations

This report has been prepared for the exclusive use of Dente Group (Client) for the project titled "1211 Western Avenue, Albany, NY". The report's contents may not be relied upon by any other party without the express written permission of ConeTec. ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meet or exceed those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

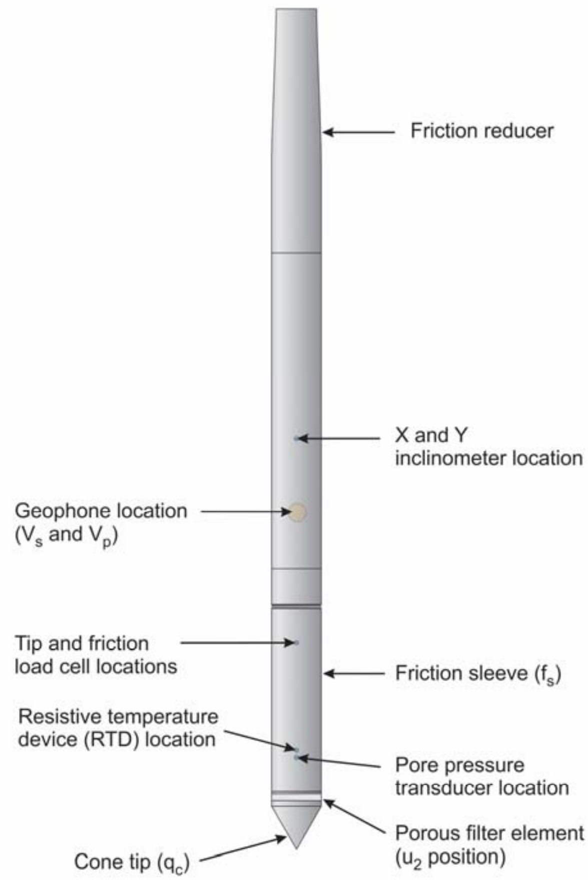

Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerin or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerin under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is included in an appendix.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

References

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

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Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

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Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355.

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (Vp) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

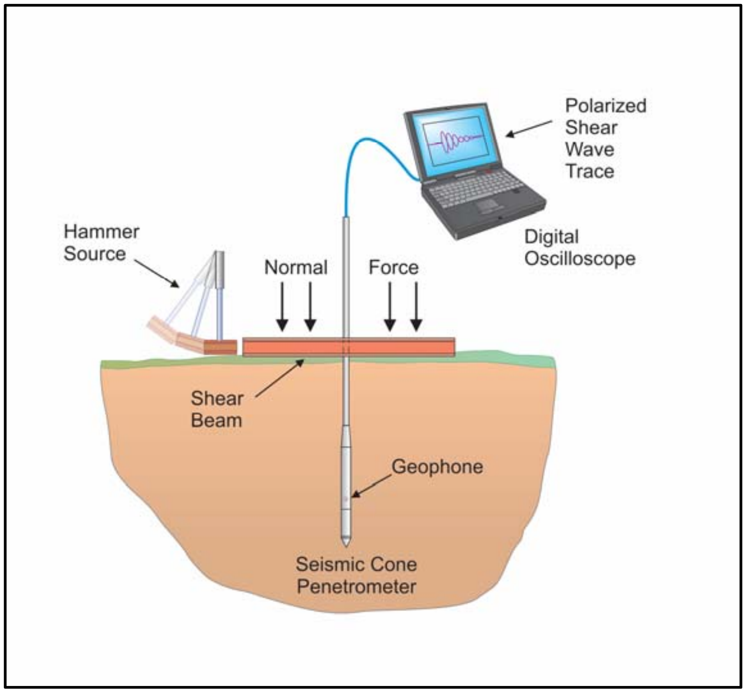


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

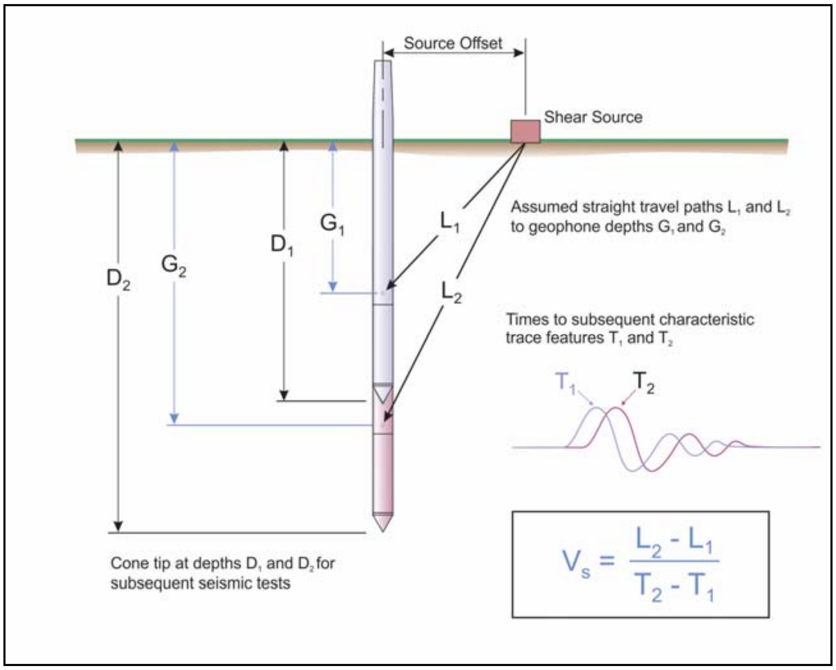


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 100 feet (30 meters) ($\bar{v}_s$) has been calculated and provided for all applicable soundings using the following equation presented in ASCE, 2010.

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

References

American Society of Civil Engineers (ASCE), 2010, “Minimum Design Loads for Buildings and Other Structures”, Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

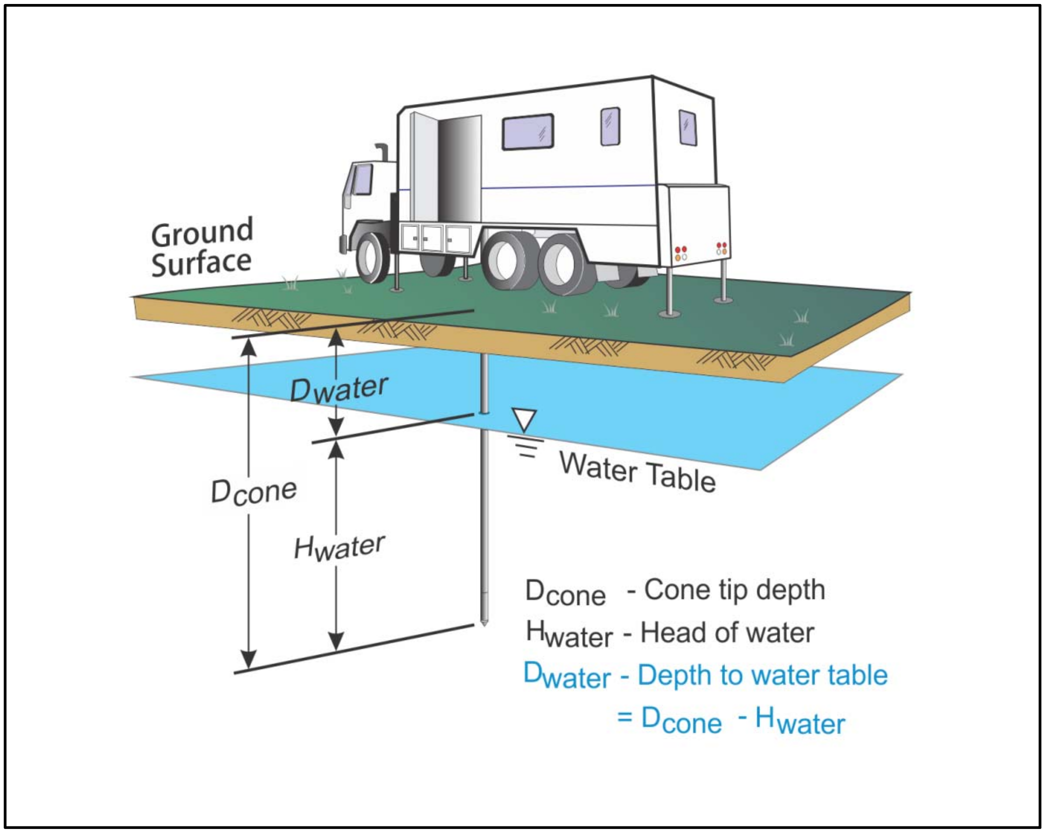


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

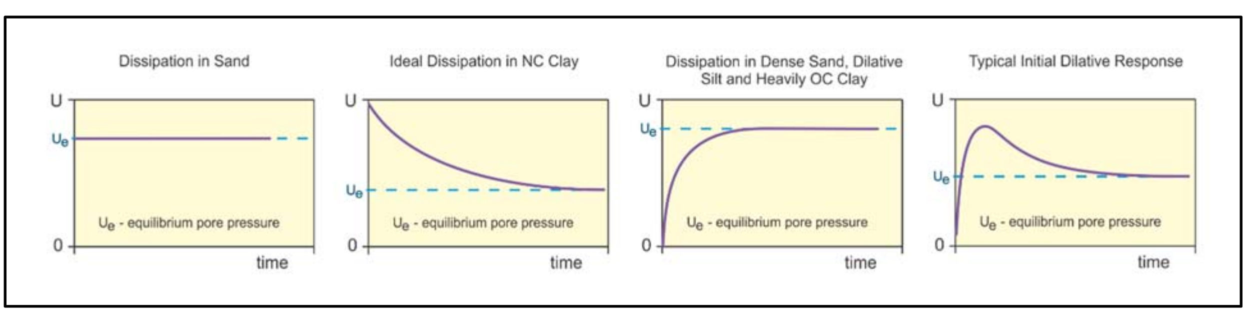


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T^* is the dimensionless time factor (Table Time Factor)

a is the radius of the cone

I_r is the rigidity index

t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

References

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Time Domain Traces
- Seismic Cone Penetration Test Tabular Results
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and

Standard Cone Penetration Test Plots

ConeTec

Job No:	18-53103
Client:	Dente Group
Project:	1211 Western Avenue, Albany, NY
Start Date:	31-Aug-2018
End Date:	31-Aug-2018

CONE PENETRATION TEST SUMMARY

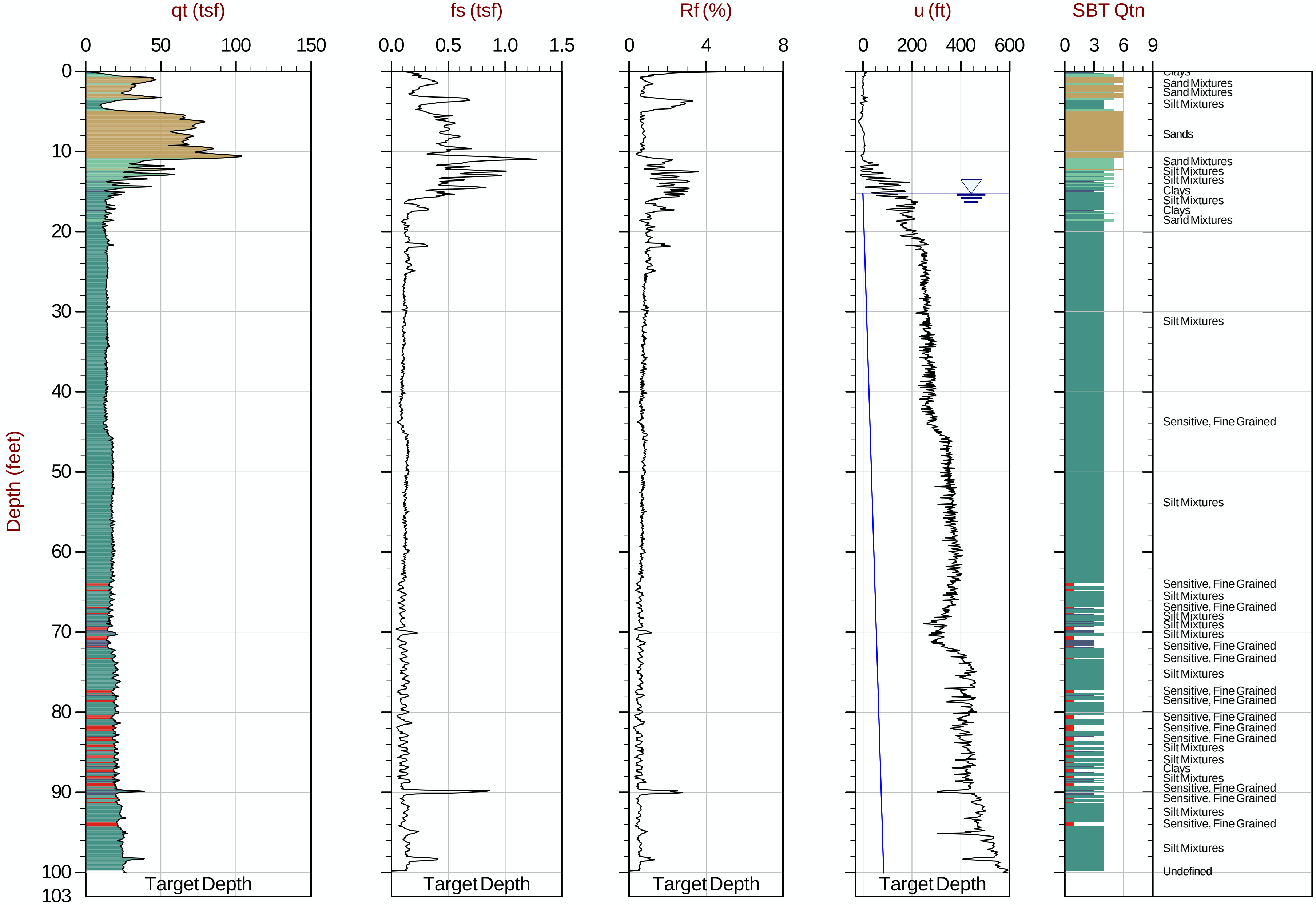
Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Shear Wave Velocity Tests	Northing[2] (m)	Easting (m)	Refer to Notation Number
SCPT18-CP01	18-53103_SP01	8/31/2018	310:T1000F10U500	15.3	100.06	20	4725551	596570	3
SCPT18-CP02	18-53103_SP02	8/31/2018	310:T1000F10U500	15.3	100.06	20	4725570	596623	
Totals	2 soundings				200.13	40			

1. Assumed phreatic surface depths were determined from the pore pressure data unless otherwise noted. Hydrostatic data were used for calculated parameters.
2. Coordinates are WGS 84 / UTM Zone 18 and were collected using a MR-350 GlobalSat GPS Receiver.
3. Assumed phreatic surface estimated from an adjacent CPT's pore pressure dissipation test.
4. No phreatic surface detected

CONETEC

Dente Group

Job No: 18-53103
Date: 2018-08-31 08:56
Site: 1211 Western Avenue, Albany, NY
Sounding: SCPT18-CP01
Cone: 310:T1000F10U500



Max Depth: 30.500 m / 100.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-53103_SP01.COR

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4725551m E: 596570m

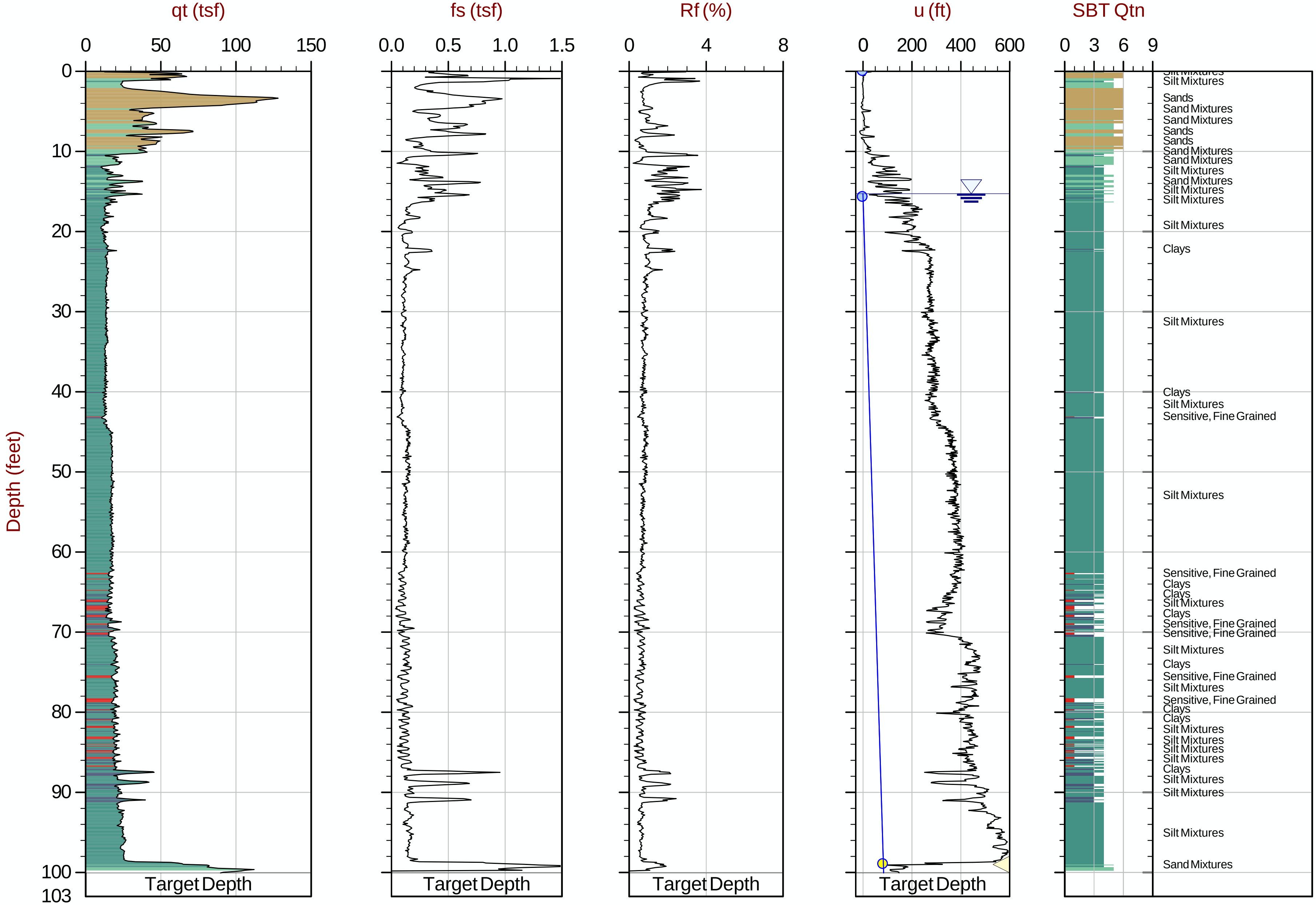
Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Dente Group

Job No: 18-53103
Date: 2018-08-31 07:29
Site: 1211 Western Avenue, Albany, NY
Sounding: SCPT18-CP02
Cone: 310:T1000F10U500



Max Depth: 30.500 m / 100.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-53103_SP02.COR

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4725570m E: 596623m

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

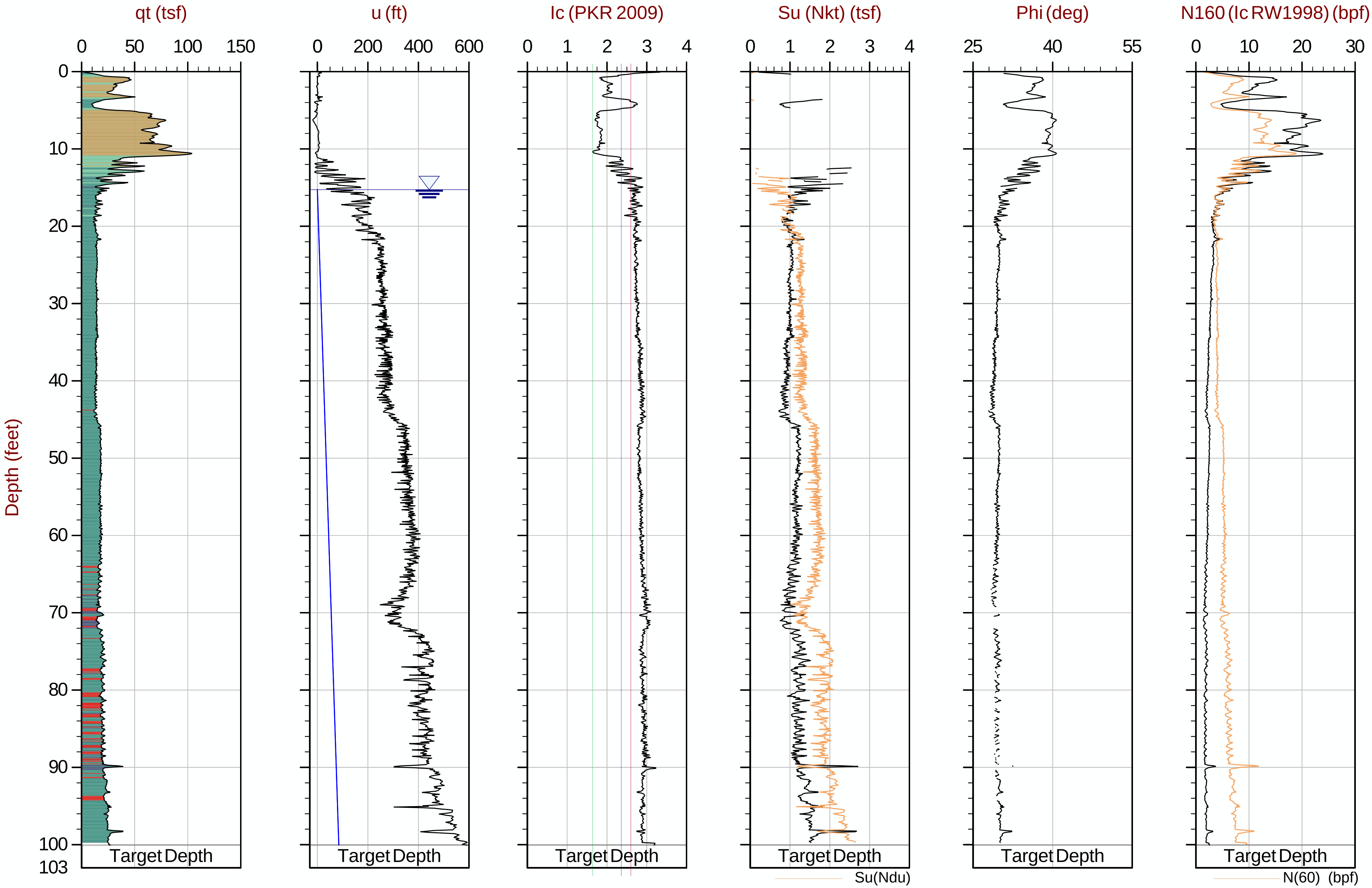
Advanced Cone Penetration Test Plots

ConeTec

ConeTec

Dente Group

Job No: 18-53103
Date: 2018-08-31 08:56
Site: 1211 Western Avenue, Albany, NY
Sounding: SCPT18-CP01
Cone: 310:T1000F10U500



Max Depth: 30.500 m / 100.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-53103_SP01.COR
Su Nkt/Ndu: 12.5 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4725551m E: 596570m

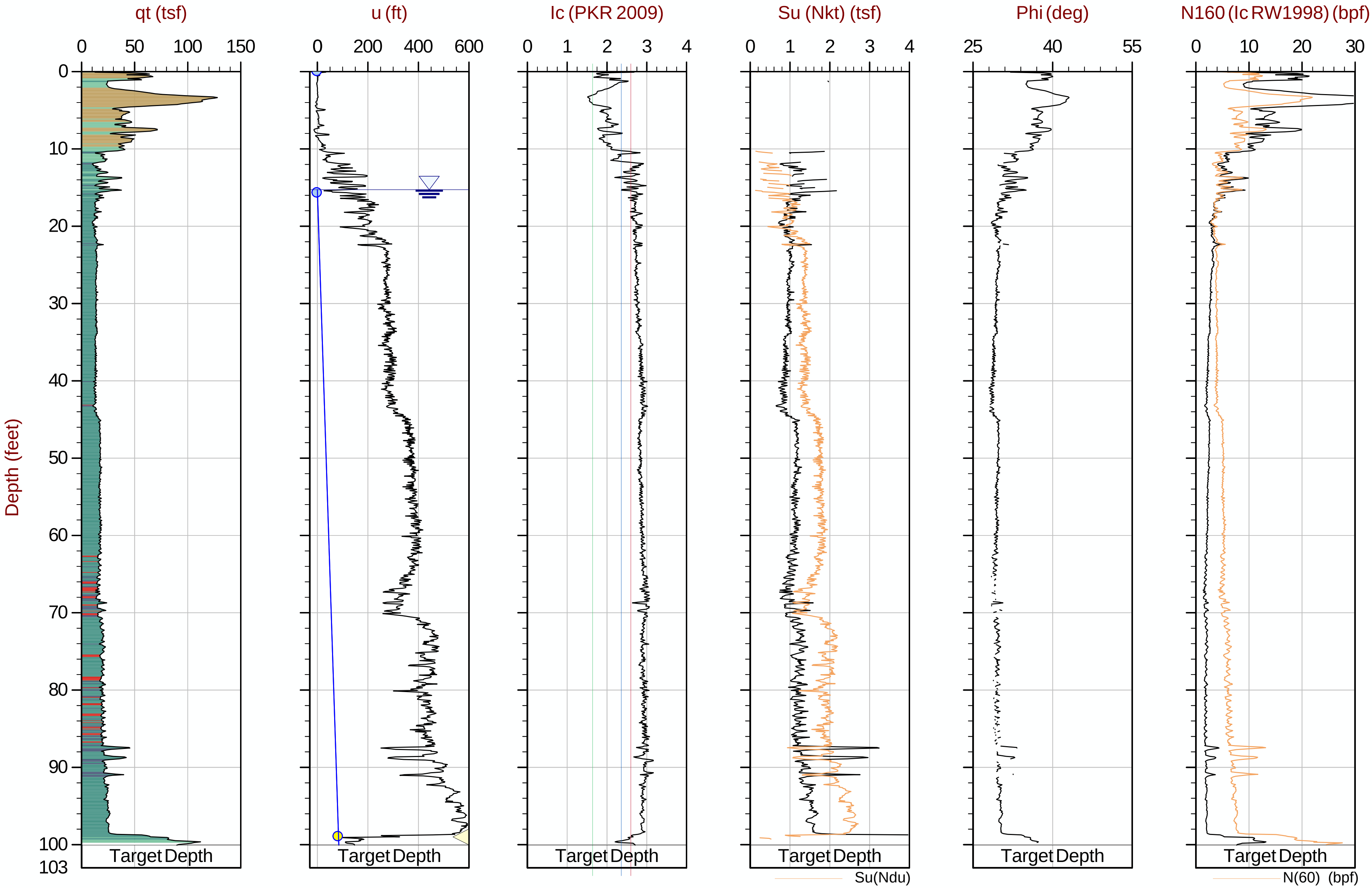
Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Dente Group

Job No: 18-53103
Date: 2018-08-31 07:29
Site: 1211 Western Avenue, Albany, NY
Sounding: SCPT18-CP02
Cone: 310:T1000F10U500



Max Depth: 30.500 m / 100.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-53103_SP02.COR
Su Nkt/Ndu: 12.5 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4725570m E: 596623m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

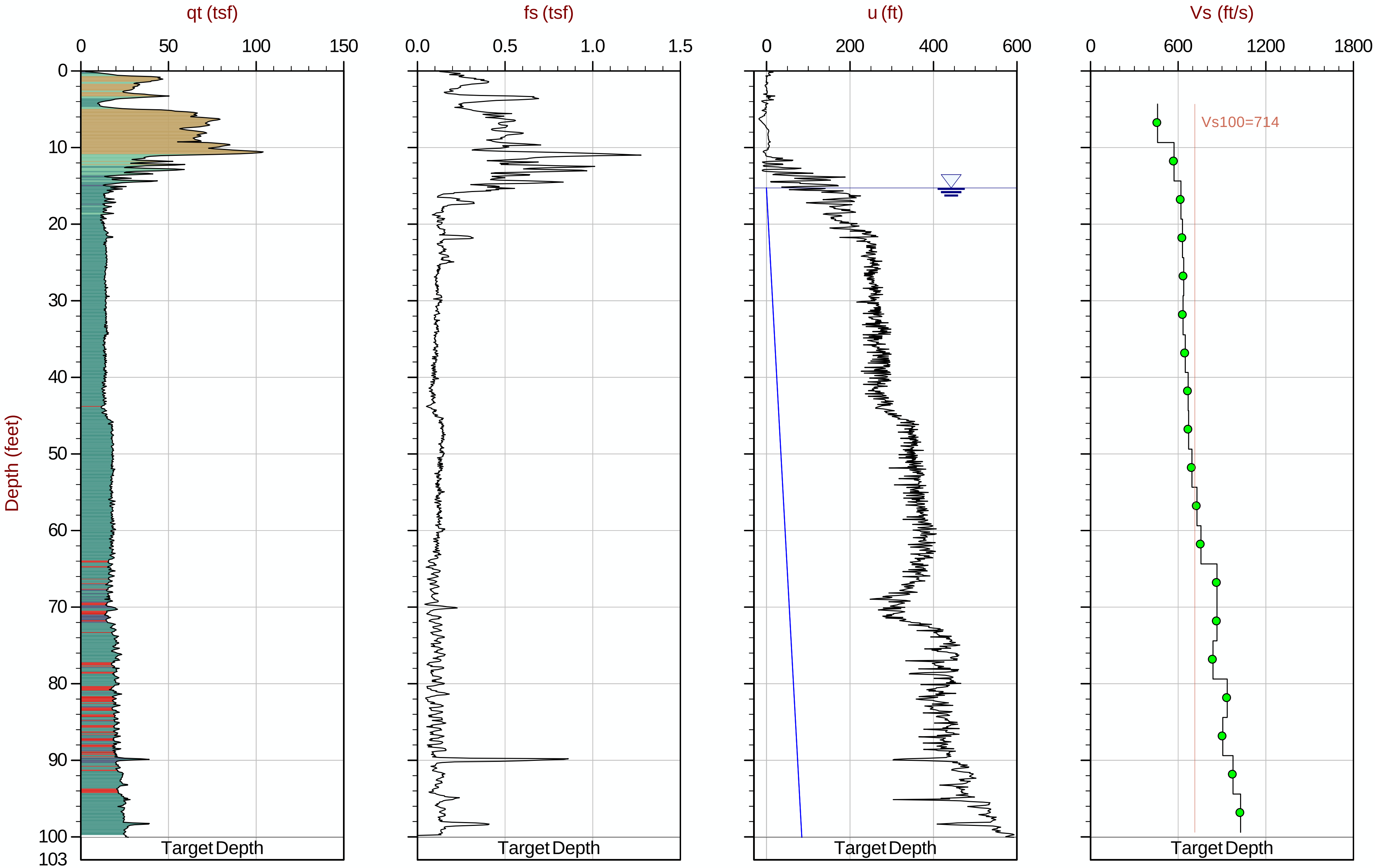
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Plots

CONETEC

Dente Group

Job No: 18-53103
Date: 2018-08-31 08:56
Site: 1211 Western Avenue, Albany, NY
Sounding: SCPT18-CP01
Cone: 310:T1000F10U500



Max Depth: 30.500 m / 100.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-53103_SP01.COR

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4725551m E: 596570m

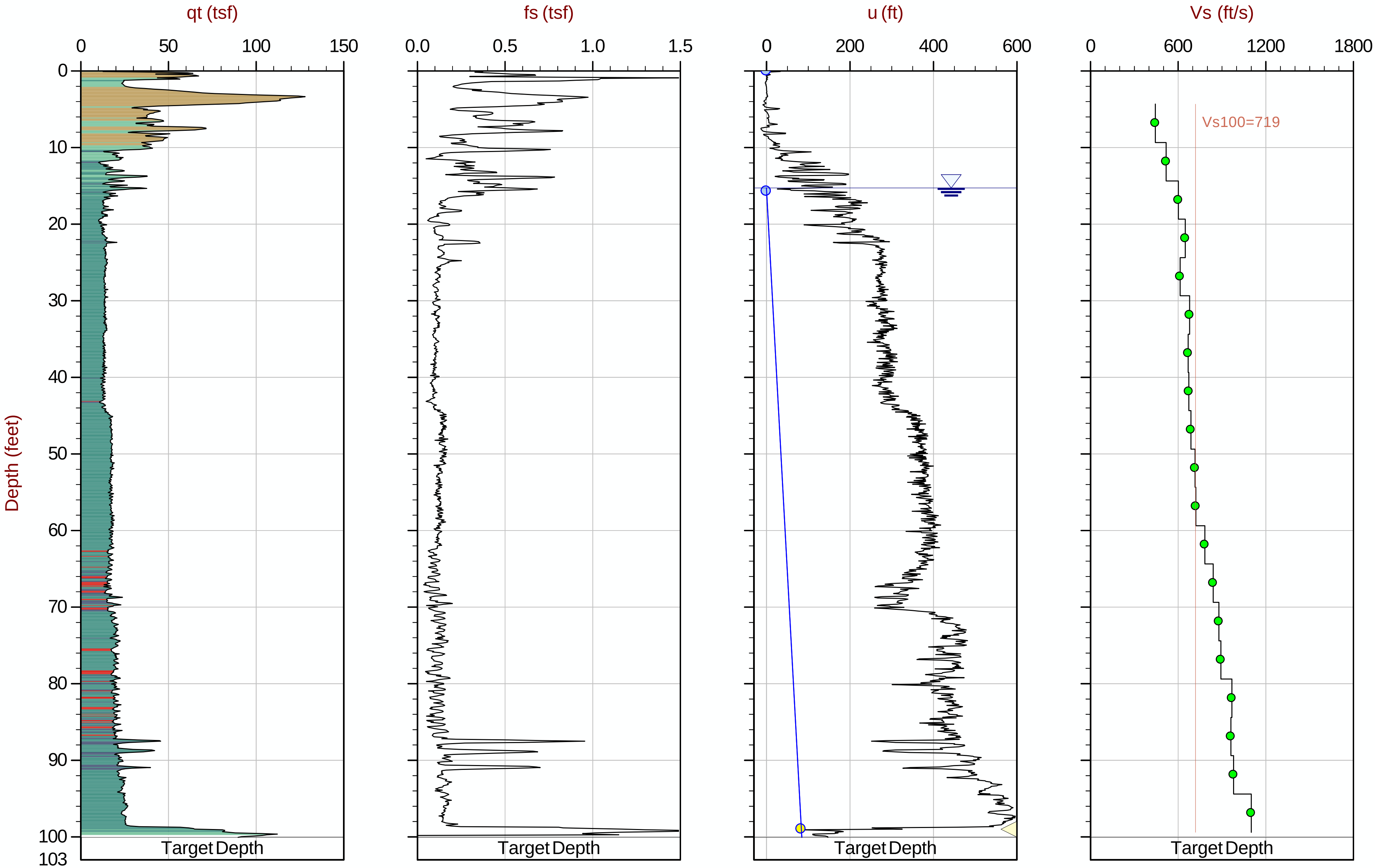
Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Dente Group

Job No: 18-53103
Date: 2018-08-31 07:29
Site: 1211 Western Avenue, Albany, NY
Sounding: SCPT18-CP02
Cone: 310:T1000F10U500



Max Depth: 30.500 m / 100.06 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-53103_SP02.COR

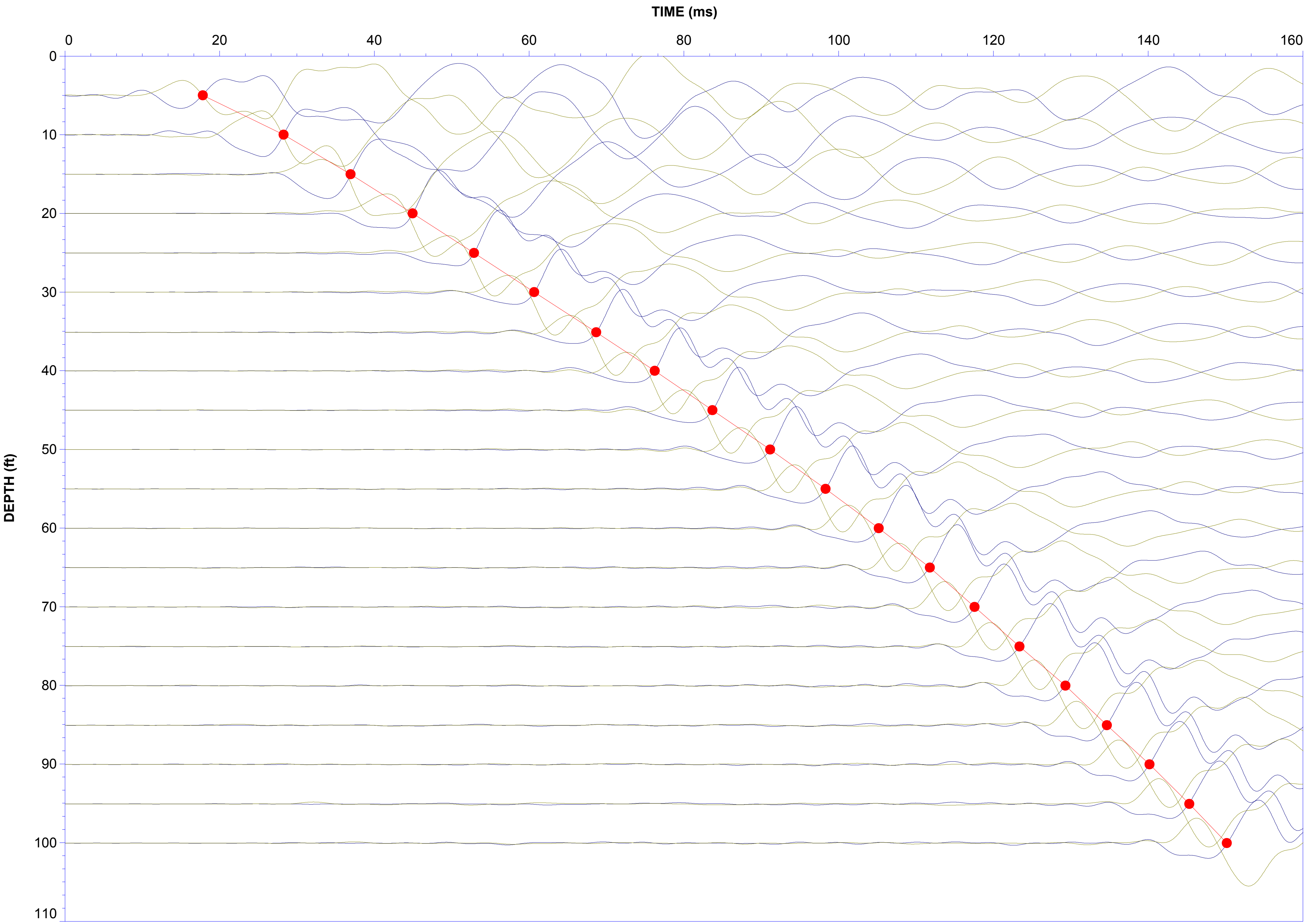
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 4725570m E: 596623m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

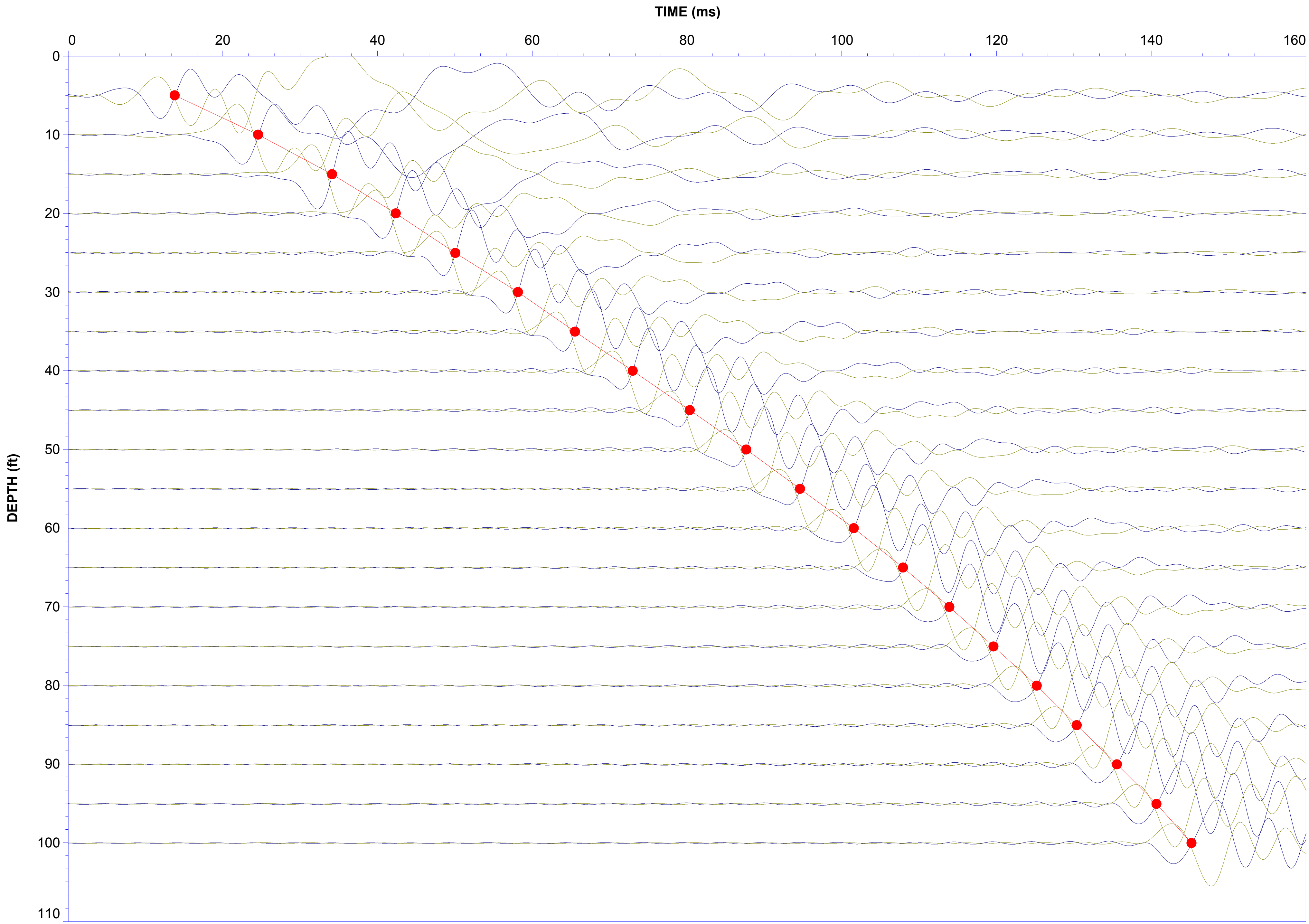
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Wave Traces

Job No: 18-53103 | Client: Dente Group | Project Title: 1211 Western Avenue, Albany, NY | Hole: SCPT18-CP01 | Filter: 0-200hz | Date: 8/31/2018



Job No: 18-53103	Client: Dente Group	Project Title: 1211 Western Avenue, Albany, NY	Hole: SCPT18-CP02	Filter: 0-200hz	Date: 8/31/2018



Seismic Cone Penetration Test Tabular Results (Vs)

Job No: 18-53103
Client: Dente Group
Project: 1211 Western Avenue, Albany, NY
Sounding ID: SCPT18-CP01
Date: 31-Aug-2018

Seismic Source: Beam
Source Offset (ft): 1.97
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
4.99	4.33	4.76			
10.01	9.35	9.56	4.80	10.46	459
15.03	14.37	14.50	4.95	8.66	572
20.01	19.36	19.46	4.95	8.01	619
25.03	24.38	24.46	5.00	7.93	630
30.02	29.36	29.43	4.97	7.79	638
35.10	34.45	34.50	5.08	8.01	634
40.03	39.37	39.42	4.91	7.57	649
45.01	44.36	44.40	4.98	7.45	669
50.03	49.38	49.42	5.02	7.46	672
55.02	54.36	54.40	4.98	7.17	695
60.04	59.38	59.42	5.02	6.88	729
65.03	64.37	64.40	4.98	6.59	756
70.05	69.39	69.42	5.02	5.79	866
75.07	74.41	74.44	5.02	5.79	866
80.05	79.40	79.42	4.99	5.94	839
85.07	84.42	84.44	5.02	5.36	936
90.06	89.40	89.42	4.99	5.50	906
95.08	94.42	94.44	5.02	5.14	976
100.07	99.41	99.43	4.99	4.85	1028

Job No: 18-53103
Client: Dente Group
Project: 1211 Western Avenue, Albany, NY
Sounding ID: SCPT18-CP02
Date: 31-Aug-2018

Seismic Source: Beam
Source Offset (ft): 1.97
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
4.99	4.33	4.76			
10.01	9.35	9.56	4.80	10.80	444
15.03	14.37	14.50	4.95	9.55	518
20.01	19.36	19.46	4.95	8.23	602
25.03	24.38	24.46	5.00	7.70	649
30.02	29.36	29.43	4.97	8.10	614
35.04	34.38	34.44	5.01	7.38	679
40.03	39.37	39.42	4.98	7.44	669
45.01	44.36	44.40	4.98	7.40	673
50.03	49.38	49.42	5.02	7.29	688
55.02	54.36	54.40	4.98	6.96	716
60.04	59.38	59.42	5.02	6.96	721
65.03	64.37	64.40	4.98	6.37	783
70.05	69.39	69.42	5.02	5.97	840
75.07	74.41	74.44	5.02	5.71	879
80.05	79.40	79.42	4.99	5.58	893
85.07	84.42	84.44	5.02	5.19	968
90.06	89.40	89.42	4.99	5.19	961
95.08	94.42	94.44	5.02	5.12	980
100.07	99.41	99.43	4.99	4.53	1101

Pore Pressure Dissipation Summary and

Pore Pressure Dissipation Plots

ConeTec

Job No: 18-53103
Client: Dente Group
Project: 1211 Western Avenue, Albany, NY
Start Date: 31-Aug-2018
End Date: 31-Aug-2018

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)	Estimated Phreatic Surface (ft)	t_{50} [a] (s)	Assumed Rigidity Index (I_r)	c_h [b] (cm^2/min)
SCPT18-CP02	18-53103_SP02.PPD	10	600	99.00	83.27	15.73		28.62	100	16.35
Totals	1 dissipations		10.0 min							

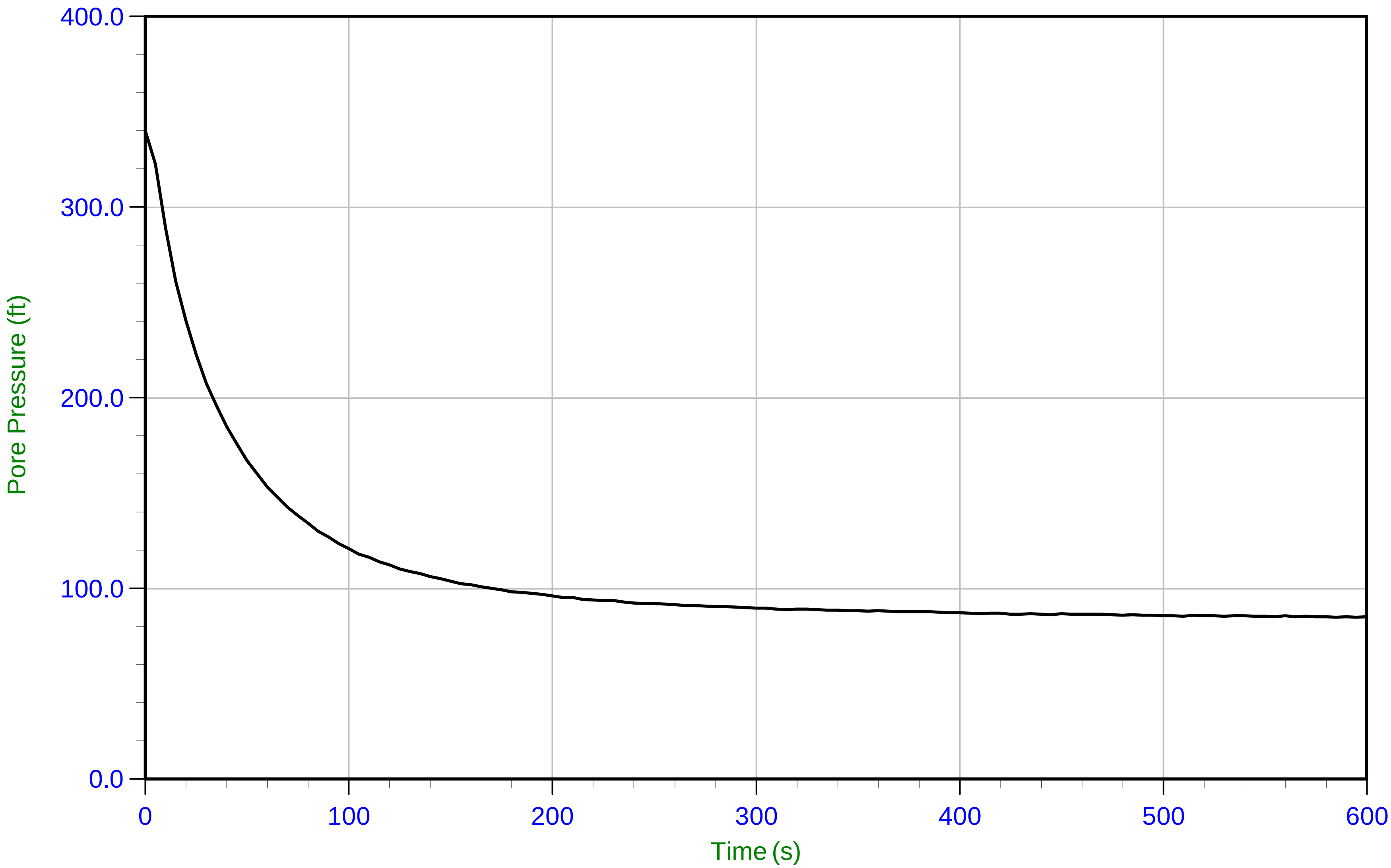
a. Time is relative to where umax occurred
b. Houlsby and Teh, 1991

CONETEC

Dente Group

Job No: 18-53103
Date: 31-Aug-2018 07:29:29
Site: 1211 Western Avenue, Albany, NY

Sounding: SCPT18-CP02
Cone: AD310 Area=10 cm²



Trace Summary:

Filename: 18-53103_SP02.PPD
Depth: 30.175 m / 98.998 ft
Duration: 600.0 s

U Min: 85.0 ft
U Max: 340.3 ft

WT: 4.793 m / 15.725 ft
Ueq: 83.3 ft
U(50): 211.76 ft

T(50): 28.6 s
Ir: 100
Ch: 16.3 cm²/min

APPENDIX E

INFILTRATION TEST RESULTS

Proposed Apartment Building
Albany, New York

A Terracon COMPANY

INFILTRATION TEST RESULTS

PROJECT: Proposed Apartment Building	**PROJECT NO.** JB185131
PROJECT LOCATION: 1211 Western Ave. Albany, NY	**TEST DATE:** 9/06/18
WEATHER:	**TESTER:** S. Loiselle

Test Location	Test Depth (feet)	Trial No.	Water Drop (inches)	Elapsed Time (hours)	Infiltration Rate (inches/hour)
I-1	4.5	1	0.0	1	0.0
		2	0.0	1	0.0
		3	0.0	1	0.0
		4	0.0	1	0.0
		Infiltration Rate for Trial No. 4 = 0.0 inches per hour Average Infiltration Rate for Trials No. 1-4 = 0.0 inches per hour NOTE: 23.5" of presoak water was in pipe before refilling to 24" depth for testing.			
I-2	4.0	1	0.0	1	0.0
		2	0.75	1	0.75
		3	0.75	1	0.75
		4	0.75	1	0.75
		Infiltration Rate for Trial No. 4 = 0.75 inches per hour Average Infiltration Rate for Trials No. 1-4 = 0.6 inches per hour NOTE: 14" of presoak water was in pipe before refilling to 24" depth for testing.			

Notes:

(1) Testing was conducted in general accord with the "Infiltration Testing Requirements" contained in Appendix D of the New York State Storm Water Management Design Manual.

(2) Test pipes were installed in boreholes made adjacent to test borings I-1 and I-2.

<u>SOIL CLASSIFICATION AT TEST DEPTH</u>

Test Location I-1: **FILL:** Brown Fine SAND and SILT, Wet, Loose

Test Location I-2: **FILL:** Dark Gray SILT, Little Fine Sand, Moist, Loose

A Terracon COMPANY

INFILTRATION TEST RESULTS

PROJECT: Proposed Apartment Building	**PROJECT NO.** JB185131
PROJECT LOCATION: 1211 Western Ave. Albany, NY	**TEST DATE:** 9/06/18
WEATHER:	**TESTER:** S. Loiselle

Test Location	Test Depth (feet)	Trial No.	Water Drop (inches)	Elapsed Time (hours)	Infiltration Rate (inches/hour)
I-3	2.5	1	0.0	1	0.0
		2	0.0	1	0.0
		3	0.0	1	0.0
		4	0.0	1	0.0
		Infiltration Rate for Trial No. 4 = 0.0 inches per hour Average Infiltration Rate for Trials No. 1-4 = 0.0 inches per hour NOTE: 24" of presoak water was in pipe at start of testing.			
I-4	3.0	1	1	1	1
		2	1	1	1
		3	1	1	1
		4	1	1	1
		Infiltration Rate for Trial No. 4 = 1 inch per hour Average Infiltration Rate for Trials No. 1-4 = 1 inch per hour NOTE: 8" of presoak water was in pipe before refilling to 24" depth for testing.			

Notes:

(1) Testing was conducted in general accord with the "Infiltration Testing Requirements" contained in Appendix D of the New York State Storm Water Management Design Manual.

(2) Test pipes were installed in boreholes made adjacent to test borings I-3 and I-4.

SOIL CLASSIFICATION AT TEST DEPTH

Test Location I-3: Light Brown Mottled SILT, Little Fine Sand, Moist to Wet, Loose

Test Location I-4: Light Grayish Brown Fine SAND, Some Silt, Moist, Loose