APPENDIX 2C

Geotechnical Report

GEOTECHNICAL FEASIBILITY STUDY

For the proposed

Lodi Energy Center Property
Lodi, San Joaquin County, CA

Project No. 6300-01-08
July 2007

Prepared for:
Ed Warner
Northern California Power Agency
477 Bret Harte Drive
Murphys, CA 95247

July 29, 2008

Project No. 6300-01-08

Ed Warner
Northern California Power Agency
477 Bret Harte Drive
Murphys, California 95247

Subject: **Geotechnical Feasibility Study**
Lodi Energy Center Property
12745 N Thornton Road
Lodi, San Joaquin County, California

Dear Mr. Warner:

Carlton Engineering, Inc. (Carlton) is pleased to present the attached report containing the results of our geotechnical feasibility study for the Lodi Energy Center project site in Lodi, San Joaquin County, California. It is our understanding that the site is ultimately proposed for development with power generation facilities with approximately 250 Megawatts capacity. The study was conducted in accordance with our Professional Services agreement, dated May 27, 2008 approving Carlton's Scope of Work found attached to the Professional Services agreement.

The accompanying report presents our findings, conclusions and recommendations developed from our geotechnical feasibility study. Contained in the report are preliminary geotechnical recommendations for earthwork, as well as descriptions of soils and geology likely to be encountered on the site, and seismic and geologic hazards. The results of subsurface exploration, which form the basis of our conclusions and recommendations, are also included in the report.

On the basis of our study, the site is feasible, from a geotechnical perspective, to receive the planned power facility, provided the recommendations included in this report are supplemented with a design-level geotechnical study. If you have any questions regarding the information contained in this report, or if we may be of further assistance, please do not hesitate to contact us.

Sincerely,

CARLTON ENGINEERING, INC.

David B. Jermstad, P.G., C.E.G.
Vice President / Geotechnical Manager

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APPENDIX A LOGS OF CPT BORINGS

This report presents the findings, conclusions, and recommendations developed from our soils and geology investigation. The study was conducted in accordance with our Professional Services agreement, dated May 27, 2008 approving Carlton's Scope of Work found attached to the Professional Services agreement. The report also contains the results of the field exploration and laboratory testing programs upon which our recommendations are based.

1.1 PROJECT DESCRIPTION

The site is located west of Interstate Highway 5 and immediately west of the City of Lodi's White Slough Water Pollution Control Facility (WPCF), within the City limits of Lodi, in San Joaquin County, California.

It is our understanding the proposed site consists of one parcel totaling approximately 2.6 acres, identified as a portion of San Joaquin County Assessor's Parcel Number 055-130-16, and a part of the property addressed as 12751 North Thornton Road, Lodi, California. The area of the site is mapped on the Terminous Topographic Quadrangle, California, United States Geological Survey 7.5-minute series topographic map and lies in Section 24, Township 3 North, Range 5 East, Mount Diablo Base and Meridian.

The proposed project is anticipated to consist of a single GE frame 7 FA, a heat recovery steam generator, a steam turbine, six-cell wet cooling tower, and ancillary equipment. The project will receive recycled water from WPCF.

1.2 PURPOSE AND SCOPE OF WORK

The purpose of the soils and geology report was to evaluate the existing geotechnical and geologic conditions within the project study area and identify potential geotechnical constraints that could impact planning, design and construction of the proposed power project. The main objectives were to characterize the subsurface materials, identify potential seismic and geologic hazards, and document our findings, conclusions and recommendations in this report. This report presents preliminary evaluations of geologic and geotechnical conditions on the site and are not intended for design. Design-level geotechnical investigations are recommended to estimate engineering parameters appropriate for design of the facility.

The scope of our land capability investigation included the following:

- Site visit to observe geological conditions;
- Review published reference documents relating to the geological conditions in the site area; and
- Preparation of this report.

Activities performed in addressing the points listed above included the following:

- Existing geologic and geotechnical reports, stereoscopic aerial photographs and topographic maps were reviewed to describe the topography, soils, mineral resources, regional seismicity, and geologic conditions of the subject property and surrounding area. Historical information was obtained and reviewed from the Environmental Data Resources Inc.

- Field geologic mapping of surface features concentrated in areas of specific concerns identified by the literature study.
- Available hydrogeologic data was used to prepare a qualitative description of the groundwater resources and springs in the project area.
- a field exploration program consisting of 4 Cone Penetrometer Tests (CPT) to depths of 50 feet within the site.

The following design information and recommendations are presented in this report:

- site plan showing the location of soil and geologic features;
- general grading and slope inclination recommendations;
- CPT test results;
- description of subsurface soil conditions found in CPT soundings;
- discussion of groundwater conditions within the depth explored;
- discussion of potential seismic hazards; and,
- discussion of potentially expansive and unsuitable soils.

2.1 REVIEW OF EXISTING GEOTECHNICAL REPORTS

A report titled *Geotechnical Investigation Proposed Combustion Turbine Project No. 1* was performed by Kleinfelder Inc for the adjacent site in February, 1993. This report identified potential hazards specific for that site and the general area. Values contained in this section should not be used for design and are referenced here for informational purposes only.

Kleinfelder, Inc. performed subsurface exploration using 8" augers using the mud-rotary technique. Samples were collected using a Modified California Sampler and Standard Penetration Sampler. Laboratory testing was performed on collected disturbed and relatively undisturbed samples.

The logs of boring indicate the surface soils encountered generally consisted of loose silty and clayey sands and soft to medium stiff sandy silts and sandy silty clays extending to approximately 2 to 3 feet below ground surface. Two borings however had loose, silty sands extending to depths of approximately 8 feet below ground surface. Below these depths, alternating layers of medium stiff to hard sandy and silty clays, sandy and clayey silts, and medium dense to very dense silty, clayey, and relatively clean sands were encountered. Groundwater was encountered in all of the borings at depths of approximately 7 feet below ground surface.

The report identifies isolated pockets of loose, near-surface, sandy soils beneath their site. Based on this information, it was their opinion that the liquefaction potential of some soils at the site is moderate to high. To mitigate the damage potential as a result of liquefaction, They recommended the site be over-excavated by 3 feet across the entire site, and 5 feet below heavily loaded structures and the material be recompacted as engineered fill. Based on these recommendations, Kleinfelder opinioned the risk of damage to structures from differential settlement would be reduced to tolerable levels.

Kleinfelder anticipated that engineered fill compacted to 90 percent of the maximum dry density would have an allowable bearing capacity of 3,000 psf. Areas that were to support heavier structures was anticipated to have an allowable bearing capacity of 4,000 psf, provided the top 5 feet beneath these structures be over-excavated and recompacted to a minimum of 95 percent of the maximum dry density prior to the placement of 3 additional feet of engineered fill also compacted to a minimum of 95 % of the maximum dry density. Based on these site preparation recommendations, Kleinfelder anticipated total settlements on the order of ½ inch and differential settlements of ½ inch for spread foundations.

2.2 FIELD EXPLORATION

Four Cone Penetrometer Test borings were sounded on June 2, 2008 within the project site as shown on the attached site plan (F2). The test boring locations are approximate based on estimated distances to prominent landmarks.

The test borings were advanced to depths of approximately 50 feet under the supervision of Ms. Moranda Kellogg of Carlton. Borings were performed using a 20 ton capacity cone with a tip area of 15 square centimeters. Data was logged every 0.05 meters. Gregg Drilling provided correlations of subsurface conditions based on data collected during the CPT sounding.

Additional subsurface testing was performed during the CPT sounding, including pore pressure dissipation tests and Shear wave velocities. The pore pressure dissipation tests were performed to evaluate the depth of groundwater, potential consolidation characteristics, and permeability at the

sampled depths. The shear wave velocities provide data that can be used for rippability, dynamic shear moduli estimations, and general subsurface soil characteristics.

Logs of the test soundings were prepared by Gregg Drilling using correlations with the CPT data to create boring logs and can be found in Appendix A along with the full report received from Gregg Drilling.

3.1 SITE CONDITIONS

The general topography of the site is characterized as being level, with berms, drainage channels and lagoons associated with the WPCF bordering the site on the north and east and a vector control facility on the south. The level site occupied by the NCPA Combustion Turbine Project #2 borders the site on the west. The site and area landforms are a product of grading and alterations to the landscape resulting from site development associated with the adjacent WPCF and the NCPA Turbine. The elevation of the site is approximately 5 feet above mean sea level.

3.2 SUBSURFACE CONDITIONS

3.2.1 Subsurface Materials

The results of our field exploration indicate that the subsurface materials generally consist of relatively flat layers of sedimentary deposits. Silty sands/sandy silts were encountered to depths of approximately 10 feet. A half foot thick layer of very dense material was encountered in CPT 3 and 4 between depths of 2 and 4 feet. A similar cemented silty sand layer was encountered by Kleinfelder during subsurface exploration adjacent to the site. The silty sands/sandy silts were underlain by a zone of clayey sand to a depth of approximately 13 feet below ground surface. A sand layer approximately 2 to 4 feet thick underlies the clayey sands to a depth of approximately 20 feet in all borings.

For CPT borings 1 and 2, the sand layer was underlain by silty clays/ clayey silts to the maximum depth investigated. In borings 3 and 4, the sand layer continued to depths of 40 feet and 30 feet respectively. This sand layer was then underlain by silty clays/clayey silts to the maximum depth explored, 50 feet.

Shear wave velocities were collected at intervals within the CPT soundings. Based on these results, the subsurface materials onsite generally have shear wave velocities of approximately 700 to 800 feet per second at a depth of approximately 10 feet below ground surface, and increasing to approximately 1500 feet per second at 50 feet below ground surface, the maximum depth explored. Waveforms for the soundings and the associated shear wave velocity calculations can be found in Appendix A of this report.

3.2.2 Groundwater

Groundwater flow is considered to be governed by topography, subsurface geologic conditions (rock units/aquifers), and geologic contacts. Aquifers below the site are characterized as a sedimentary basin system with aquifer zones controlled by sediment grain size and porosity. Depth to groundwater is anticipated to have seasonal variation.

Historical groundwater levels in the area, based on data available from DWR, generally vary from approximately 3 to 20 feet below ground surface in a two mile radius of the site over the past 30 years. Groundwater monitoring information at this site indicates the gradient direction is toward the east-southeast and static groundwater level in February 2008 was reported at about 9 feet below the ground surface in monitoring well WSM-2 located near the southeast corner of the Site.

A study conducted by Kleinfelder Engineering dated February 23, 1993 for the adjacent site reported encountering groundwater in all borings at depths of approximately 7 feet.

Based on the results of pore pressure readings (Appendix A), groundwater was encountered in all CPT soundings at a depth of approximately 10 feet below ground surface.

4.1 REGIONAL GEOLOGY

The site is located within the northern portion of the San Joaquin Valley (Great Valley Geomorphic Province of California) The Great Valley is a northwest trending structural trough, approximately 400 miles long and averaging 50 miles wide, that is filled with Cretaceous, Tertiary and Quaternary period sediments. The province is fault-bounded on the west by the Coast Ranges, and bounded on the east by the Sierra Nevada foothills with the bedrock of the Sierra Nevada block dipping gently westward beneath the valley sediments. The sedimentary formations comprising the valley fill are nearly flat-lying and are derived from erosion of the Sierra Nevada and the Coast Ranges. Gently westward dipping Tertiary volcanic flows and ash from the Sierra Nevada cover older marine and delta deposits, and in turn are covered by lake and alluvial deposits in the northern portion near the eastern edge of the valley. The western extent of the valley is filled by thousands of feet of sediment with the deepest filling in the southwestern portion being more than 30,000 feet thick.

The Califonia Division of Mines and Geology *Geologic Map of the Sacramento Quadrangle, California, 1:250,000* identifies the underlying sedimentary unit in the area of the project as being the Modesto Formation alluvium – Qm_1 (upper member). The sedimentary material is described as "arkosic alluvium."

4.2 SITE GEOLOGY

The Site is located in an area that exhibits a fairly complete stratigraphic section of Cretaceous, Tertiary, and Quaternary deposits. The soils study consisted of a compilation of available public soils data from the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Web Soil Survey (WSS) Online Soil Mapping accessed April 24, 2008. The study included a site reconnaissance and observation of exposed soil profiles. According to the USDA WSS, the project site is located within the Devries sandy loam map unit. The Devries map unit is composed of somewhat poorly drained sandy loams derived from mixed rock sources and deposited as alluvium.

The average annual temperature is approximately 60 degrees F., average annual precipitation is approximately 15 inches, and the frost-free season approximately 270 days (USDA WSS, accessed April 24, 2008).

4.3 GEOLOGIC HAZARDS

4.3.1 Expansive Soils

Expansive soils are defined as soils that undergo large volume changes (shrink or swell) due to variations in moisture content. Such volume changes may cause damaging settlement and/or heave of foundations, slabs-on-grade, pavements, etc.

Based upon the observations during the subsurface exploration, expansive soils may be present onsite. It is anticipated that damage potential due to onsite expansive soils will be low and can be mitigated using standard design and construction techniques. Geotechnical investigations should be completed to support project design and construction, evaluate areas that may be subject to expansive soils, and evaluate the expansion potential beneath the site.

4.3.2 Collapsible Soils

Collapsible soils are defined as a soil that will undergo a sudden decrease in volume when its internal structural support is lost. Internal support is a temporary strength derived from numerous sources,

including, but not limited to, cementing agents, clay-welding of soil particles, silt bonds, clay bonds, clay bridges, and capillary tension. Soils found most susceptible to collapse include loess (fine-grained, wind-deposited soils) deposits, valley alluvium deposited within a semi-arid to arid climate, and residual soil deposits. Collapsible soils are generally associated with dry regions (NAVFAC, 1982).

Based on the observations during the subsurface exploration, the collapse potential of soils beneath the site is anticipated to be low due to shallow groundwater.

4.3.3 Landslides and Slope Stability

Based on the flat topography on the site, the potential for landslides and slope stability failures on the site are anticipated to be low. No areas of active or inactive landslides on the project site were identified during our site reconnaissance or review of aerial photographs.

4.3.4 Subsidence

Subsidence can be a natural or man-made phenomena resulting from tectonic movement, consolidation, hydrocompaction, groundwater extraction, or decomposition of organic material. The USGS International Survey on Land Subsidence Database indicates subsidence in the Sacramento to Stockton area is associated with groundwater extraction.

Based on the proximity to the bay, relatively constant groundwater depth for the past 30 years, and the proposed reuse of water from the City of Lodi's White Slough Water Pollution Control Facility (WPCF) rather than development of groundwater resources, the future land subsidence due to groundwater pumping associated with this project is anticipated to be low.

4.4 FAULTING AND SHEAR ZONES

The site is not located within the Alquist-Priolo Earthquake Fault Zoning Act Special Publication. The Alquist-Priolo Earthquake Fault Zoning Act was passed in 1972 to mitigate the hazard of surface faulting to structures planned for human occupancy. No faults have been mapped as crossing the site. The nearest mapped active fault is the Greenville Fault, located approximately 30 miles to the southwest of the site.

4.5 SEISMIC CONSIDERATIONS

According to the Fault Activity Map of California and Adjacent Areas, (California Division of Mines and Geology, 1994) faults located in the proximity of the project are part of the San Andreas Fault System. The San Andreas system is active, with displacements occurring during historic time. The nearest known active fault is the Greenville Fault, located approximately 30 miles to the southwest of the project site. Strong earthquakes generated along any of the California faults may affect the project area depending on the characteristics of the earthquake and the location of the epicenter.

According to the United States Geologic Survey (USGS) Seismic Hazard Curves and Uniform Hazard Response Spectra program version 5.0.7, upper and lower level earthquake peak ground accelerations are estimated to be 0.19g and 0.09g respectively. An upper level earthquake motion is defined as being the ground motion with a 10% probability of being exceeded in 50 years, while a lower level earthquake is defined as being the ground motion with a 50% probability of being exceeded in 50 years. Peak ground accelerations associated with the maximum considered earthquake (MCE), defined in the 2007 California Building Code as a ground motion with a 2% probability of being exceeded in 50 years, is estimated to be 0.35g at the site. Earthquake magnitudes contributing to this peak ground acceleration are presented in the table below. Peak ground acceleration associated with a maximum probable earthquake (MPE), defined as the maximum earthquake that appears to be reasonably expectable to occur in a 100 year interval (Kramer, 1996), is estimated to be 0.11g.

Historically, some faults known to produce large earthquakes in the region are the Calaveras, Hayward,

San Andreas, and Greenville Faults. The California Department of Conservation Division of Mines and Geology (CDMG) Open File Report 92-1 reports that the Calaveras, Hayward, San Andreas, and Greenville, have a strike-slip style of faulting. Approximate distances, and anticipated earthquake magnitudes, associated with the Maximum Credible Earthquake are presented below (Jennings, 1994; Mualchin and Jones 1992):

Fault Name	Approximate Distance From Site (mi)	Maximum Credible Earthquake Magnitude
Calaveras	39	7.25
Hayward	48	7.25
San Andreas	69	8
Greenville	30	7.25

Seismic design should be performed in accordance to the 2007 California Building Code (ICBO, 2007). The project structural engineer should assess the need for seismic response spectra for design of the project as required by applicable codes.

4.6 SEISMIC HAZARDS

4.6.1 Liquefaction

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to experience a partial to complete loss of shear strength. Such a reduction/loss of shear strength can result in both settlement and horizontal movement (lateral spreading) of the soil mass. Analysis of the liquefaction potential for sediments beneath the site follows procedures outlined by Idriss and Boulanger.

Based on the relatively loose sandy soil and shallow groundwater conditions, our analyses indicate the potential for liquefaction to occur in the areas explored is considered to be moderate. Liquefiable zones are anticipated to be restricted to isolated pockets of loose, near-surface, sandy soils.

Geotechnical investigations, which will be completed to support project design and construction, should evaluate areas that may be subject to seismically induced settlement. Standard design and construction techniques can then be used to mitigate the potential for damage due to seismically induced settlement.

It is anticipated over-excavation of approximately 5 feet and recompaction as engineered fill will reduce the risk of damage to structures from differential settlement associated with liquefaction to tolerable levels. Recommendation for the extent of over-excavation and compaction characteristics of the engineered fill should be provided in the design-level geotechnical report.

4.6.2 Seismic Settlement

Seismic settlement generally occurs when relatively loose to medium dense cohesionless soils come to a more compact or dense state under earthquake shaking. The settlement can occur as a result of liquefaction or in dry soils. Due to the relatively dense state and material properties of the soil at the site, we judge that significant seismic settlement is unlikely to occur in the areas explored.

4.6.3 Ground Surface Rupture

The site is not mapped in an "Earthquake Fault Zone" (CGS, 1997). Based upon this information, the probability of ground surface rupture along a fault trace is considered to be low at the site.

4.7 BEARING CAPACITY

It is anticipated the bearing capacity for this site will be on the order of 2,500 psf. This is a preliminary value based on available information, including an assumed near surface phi value of 34 degrees and density of 110 pcf. It is anticipated that standard design and construction procedures will allow foundations to be designed within this allowable bearing capacity; However, 2,500 psf should not be used as a design value. Kleinfelder reported an allowable bearing capacity of 3,000 in their 1993 Report titled *Geotechnical Investigation Proposed Combustion Turbine Project No. 1* for the adjacent site. A more detailed bearing capacity analysis should be performed during the design level geotechnical investigations to generate design values for allowable bearing capacity.

4.8 OTHER GEOLOGIC HAZARDS AND CONCLUSIONS

The preliminary assessment for geologic hazards was based on a study of recent stereoscopic aerial photographs, geological and geotechnical literature review, and the field mapping and reconnaissance of the site. Potential geologic problems which were considered in this assessment include the potential for ground rupture, landsliding, earthquake-induced flooding, tsunamis and seiches, and collapsible and expansive soils.

Based on the available information, our recommendations and conclusions regarding the geologic and seismic hazards and site geotechnical site constraints at the project site are as follows:

- Expansive or collapsible soil conditions may be present within the building areas based on the soils observed during the site reconnaissance and the general lithology of the underlying geologic unit. Additional investigations should be performed as part of the design level geotechnical investigations to determine if the expansive and collapsible potential is significant, and recommend standard design and construction techniques to mitigate the potential for damage to structures.

- The potential for landslides or other slope failures is anticipated to be low because of the relatively flat topography.

- The potential for future land subsidence due to this project is anticipated to be low at the site since water is proposed to be recycled from the City of Lodi's White Slough Water Pollution Control Facility (WPCF), instead of using water developed from groundwater resources.

- No known faults or shear zones cross the site, and therefore the risk of ground surface rupture is anticipated to be low for the site.

- Ground shaking caused by earthquake activity of faults in the region is possible and likely to occur. Appropriate structural design criteria to mitigate the effect of such ground shaking should be applied.

- Based on available information, it is anticipated the maximum peak site acceleration for a maximum considered earthquake is 0.35g based on the proximity to nearby active faults and the results of our analysis using the United States Geologic Survey (USGS) Seismic Hazard Curves and Uniform Hazard Response Spectra program version 5.0.7.

- The potential for liquefaction beneath the site is moderate due to the shallow groundwater table and subsurface conditions encountered. Liquefiable zones are anticipated to be restricted to isolated pockets of loose, near-surface, sandy soils. Geotechnical investigations, which will be completed to support project design and construction, should evaluate areas that may be subject to seismically induced settlement. Standard design and construction techniques can then be used to mitigate the potential for damage due to seismically induced settlement.

- No large reservoirs are located in the site vicinity, however smaller reservoirs are located adjacent to the site. Based on the size, impounded water quantity, and risk due to overtopping of these nearby reservoirs, the potential of flooding and erosion due to dam failure is low. The site is not located within close proximity to a levee, therefore the potential of flooding due to levee failure is low.

- Data provided by Environmental Data Resources, Inc. indicates the site lies within a FEMA designated 100-year flood zone. However, a civil engineer should confirm the flood zone designation.

- Based on the inland location of the site, the potential for damage from tsunamis or seiches is low. Phase 3 improvements to the White Slough Water Pollution Control Facility Mitigated Negative Declaration states seiche and tsunami waves would not be a threat to the site (West Yost Associates, 2007).

- Preliminary bearing capacity evaluations indicate the bearing capacity of the onsite soils will be on the order of 2,500 pounds per square foot and 4,000 psf with an over-excavation and recompaction of 5 feet, similar to the adjacent site. This value should not be used for design purposes and should be re-evaluated and additional recommendations made as part of the design level geotechnical investigations for the site.

- All excavations and grading should conform to the 2007 California Building Code (CBC). Cuts and trenches in unconsolidated alluvial deposits may require special dewatering, sloping, and shoring considerations during design.

4.9 RECOMMENDATIONS

The primary constraint for site development is the potential for liquefaction and subsequent differential settlements. It is anticipated that zones of liquefiable soils will be limited to isolated pockets of loose, near-surface, sandy soils. A more detailed study of the liquefaction potential as well as estimation of settlements associated with liquefaction, are recommended as part of the design-level geotechnical investigations for future development at the site. Based on the results of this further investigation, recommendations to reduce the potential for damage associated with seismically induced settlements will be made. It is anticipated over-excavation of approximately 5 feet and recompaction as engineered fill will reduce the risk of damage to structures from differential settlement associated with liquefaction to tolerable levels. Recommendation for the extent of over-excavation and compaction characteristics of the engineered fill should be provided in the design-level geotechnical report. Based on shear wave velocities encountered during our subsurface investigation, it is anticipated excavations can largely be accomplished with conventional heavy equipment such as a CAT D8 dozer. Near surface cemented soils were encountered during our subsurface exploration, and by Kleinfelder for the adjacent site, that may result in somewhat slower excavation.

Additional investigations which should be performed as part of the design level geotechnical investigations include a more detailed evaluation of the expansion and collapsibility potential of subsurface materials. Based on reviewed reference documents, damage potential to proposed structures due to expansive or collapsible soils is anticipated to be low. It is anticipated the risk of damage to structures from expansive or collapsible soils will be reduced to tolerable levels by over-excavating and recompacting the top 5 feet of soil onsite, as previously discussed.

Ground shaking caused by earthquake activity of faults in the general vicinity is possible and likely to occur. Appropriate structural design to mitigate the effect of anticipated ground shaking should be conducted according to applicable standards of practice, codes, and regulations.

The preliminary assessment for geologic hazards indicates potential problems caused by ground rupture, landsliding, seismic densification, seiches, flooding due to dam failure are anticipated to be low. Slope and foundation instability due to landslides and unstable natural, cut, and fill slopes are not anticipated.

Based on the information obtained from our subsurface explorations and published reference document review, the site is feasible, from a geotechnical perspective, to receive the planned power facility, provided the recommendations included in this report are supplemented with a design-level geotechnical study providing site specific designs and recommendations for earthwork, foundations, pavement, soil conditions, and seismicity should be addressed in the design-level geotechnical study.

5.0

LIMITATIONS

This report has been prepared by Carlton Engineering, Inc. (Carlton) under the professional supervision of those senior partners and/or senior staff whose seals and signatures appear herein. The geotechnical engineering study upon which this report is based was conducted for the proposed improvements at the project site described in this report. The conclusions and recommendations contained in this report are not valid for other project sites. If the proposed project is modified or relocated, or if the subsurface conditions found during construction differ from those described in this report, Carlton should be provided the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations need revision.

The interpretations of data, findings, conclusions, recommendations and professional opinions in this report are based on the available information, site conditions and samples collected during our field exploration, and were developed in accordance with generally accepted geotechnical engineering principles and practices, and as prescribed by the client. There is no warranty, either expressed or implied. Carlton accepts no liability regarding completeness or accuracy of the information presented and/or provided to us, or any conclusions and decisions which may be made by the client or others regarding the subject site/project. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observations of construction.

This report is considered valid for the proposed project for a period of two years from the report date provided that the site conditions and development plans remain unchanged. With the passage of time, changes in the conditions of a property can occur due to natural processes or the works of man on this or adjacent properties. Legislation or the broadening of knowledge may result in changes in applicable standards. Depending on the magnitude of any changes, Carlton may require that additional studies (at additional cost) be performed and that an updated report be issued. Additional studies may disclose information which may significantly modify the findings of this report. Carlton will retain untested samples collected during our field study for a period not to exceed 90 days unless other arrangements are made with the client.

Our scope of services was limited to the proposed work described in this report, and did not address other items or areas. Our scope of services did not include environmental site assessments or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, ground water or air, on or below, or around the site. Our scope of services did not include an evaluation or investigation of the presence or absence of wetlands.

This report was prepared solely for the use of the client. The use of, or reliance upon, this report by any party, other than the client, shall be solely at the risk of such party. The client is responsible to ensure that all relevant parties to the project, including designers, contractors, subcontractors, etc., are made aware of this report in its entirety. No other entity or person shall use or rely upon this report, or any of Carlton's work products, unless expressly authorized by Carlton.

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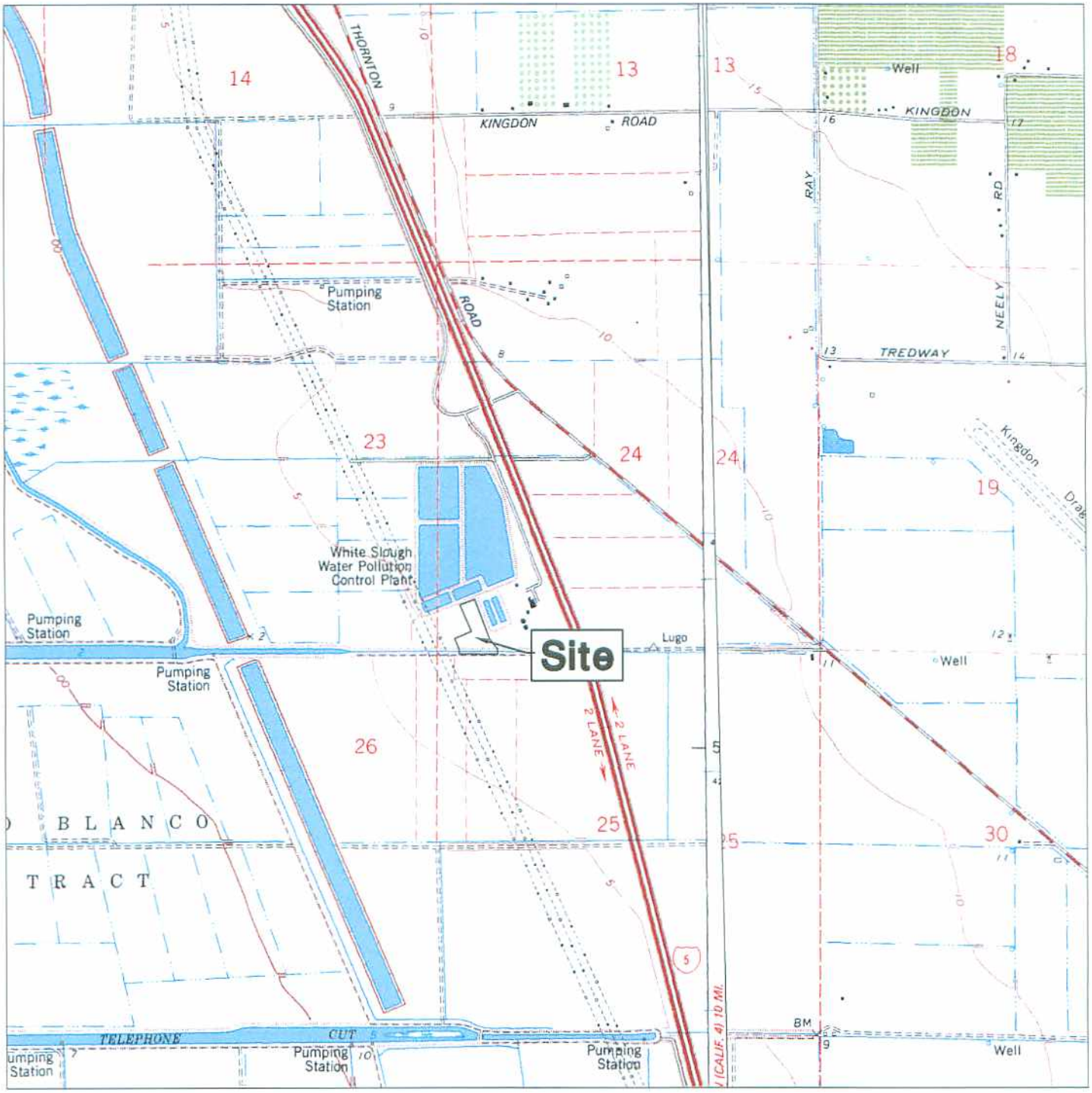
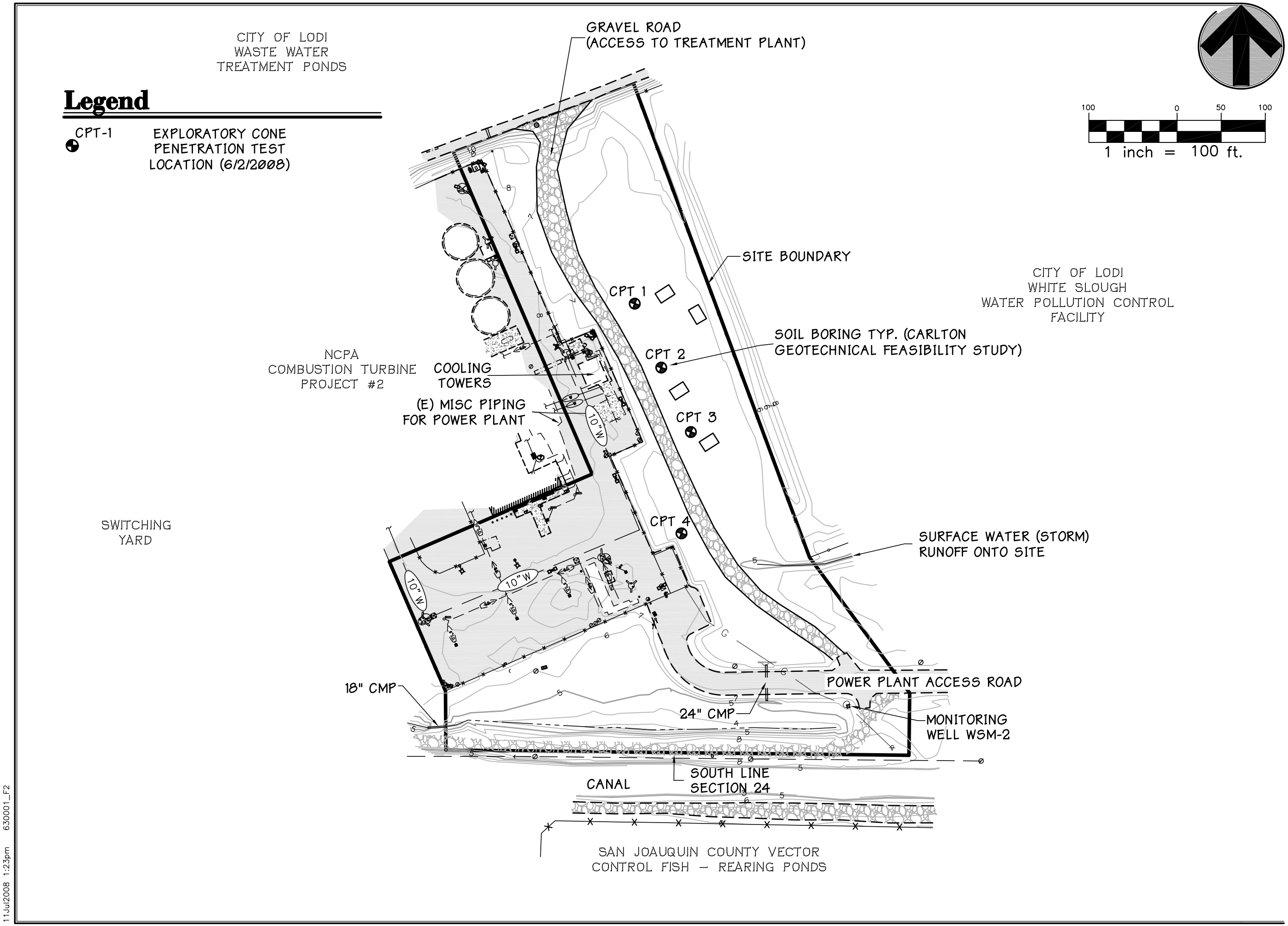


Terminous and Lodi South, CA U.S.G.S Topographic Quadrangles
T3N, R5E, Sec. 23
1:24,000

N

Project 6300-01-08	June 2008	Geotechnical Feasibility Study 12751 North Thornton Road, Lodi San Joaquin, California	FIGURE 1

Carlton Engineering, Inc.
3883 Ponderosa Road
Shingle Springs, California 95682



TEAM DRIVEN SOLUTIONS FOR THE BUILT ENVIRONMENT

CARLTON
Engineering Inc.

3883 Ponderosa Road, Shingle Springs, CA 95682
Voice 530.677.5515 Fax 530.677.6645

NCPA
LODI ENERGY CENTER PROJECT

Geotechnical Feasibility Study
SITE MAP

Project Location: 12751 North Thornton Road Lodi, California

Property Owner: City of Lodi

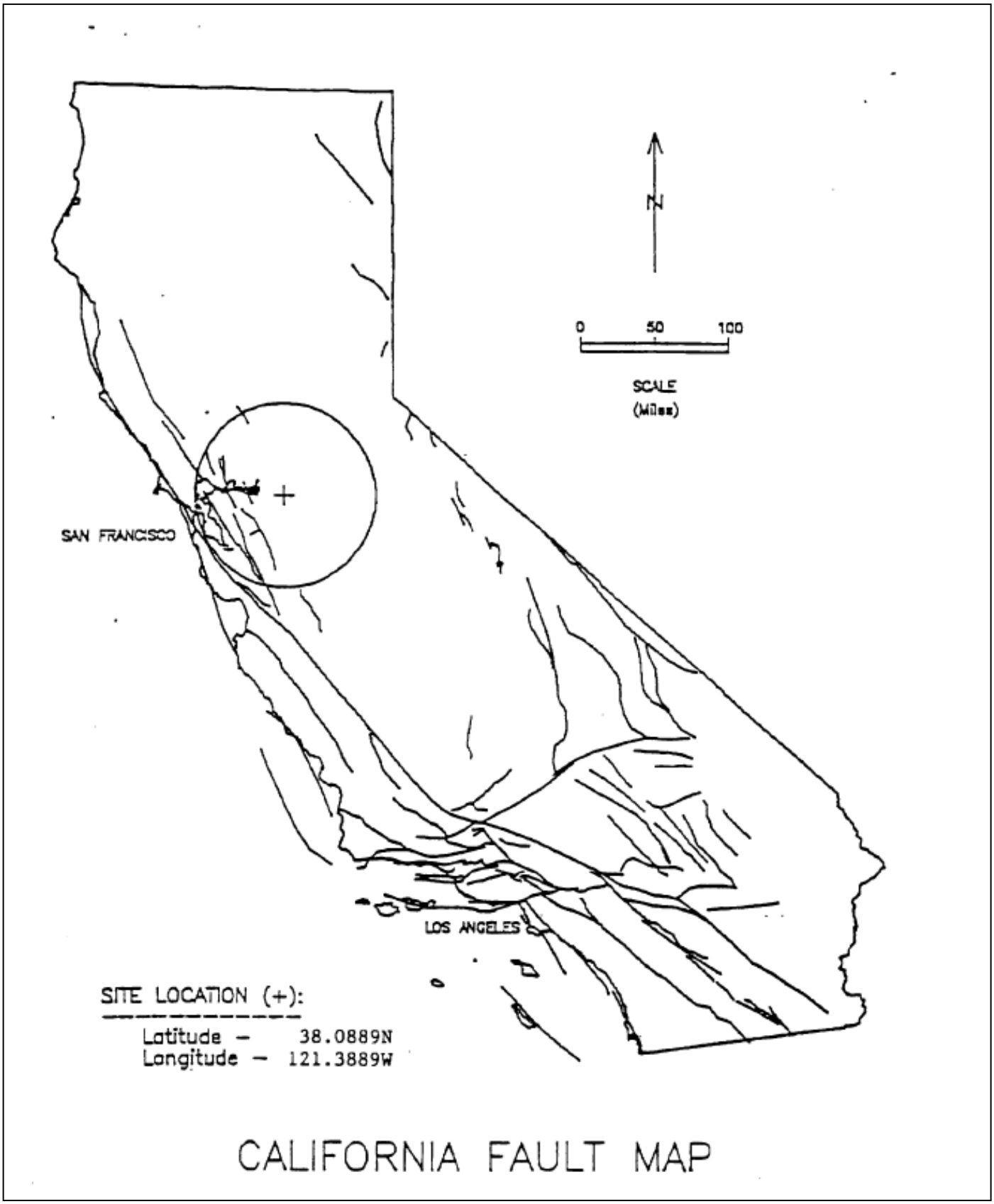
APN: 055-130-16

DESIGN	DRAWN	DATE
NLB	RED	07/10/2008
Hz SCALE		PROJECT
1"=100'		6300-01-08

CHECKED BY & DATE

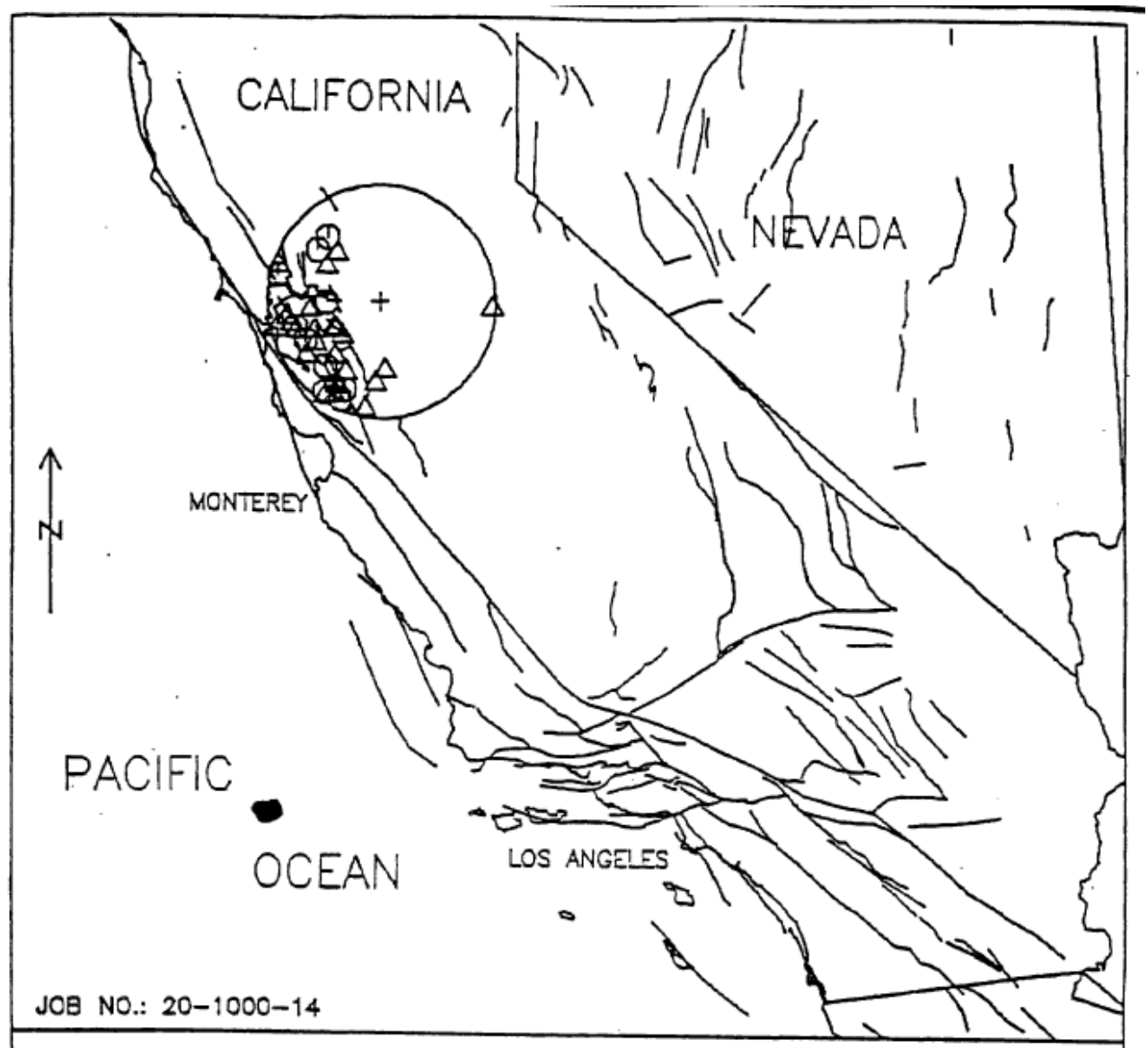
FIGURE
2

11Jul2008 1:23pm 630001_F2



Source: EQSearch, 1996

PROJECT 6300-01-08	July 2008		
CARLTON Engineering Inc.	Carlton Engineering, Inc. 3883 Ponderosa Road Shingle Springs, California 95682	Geotechnical Feasibility Report 12751 North Thornton Road, Lodi San Joaquin County, California	FIGURE 3



HISTORICAL EARTHQUAKES 1800 TO 1990

EXPLANATION

- M = 8.0 +
- M = 7.0–7.9
- M = 6.0–6.9
- M = 5.0–5.9
- M = 4.0–4.9

0 50 100

SCALE (Miles)

SITE LOCATION (+):

Latitude – 38.0889N
Longitude – 121.3889W

Source: EQSearch, 1996

PROJECT 6300-01-08	July 2008		
CARLTON Engineering Inc.	Carlton Engineering, Inc. 3883 Ponderosa Road Shingle Springs, California 95682	Geotechnical Feasibility Report 12751 North Thornton Road, Lodi San Joaquin County, California	FIGURE 4

APPENDIX A

LOGS OF CPT BORINGS

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

June 3, 2008

Carlton Engineering
Attn: Moranda Kellogg
3883 Ponderosa Rd.
Shingle Springs, California

Subject: CPT Site Investigation
NCPA Lodi Energy Center
Lodi, California
GREGG Project Number: 08-149MA

Dear Ms. Kellogg:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

950 Howe Rd • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
OTHER OFFICES: LOS ANGELES • HOUSTON • SOUTH CAROLINA
www.greggdrilling.com

Cone Penetration Test Sounding Summary

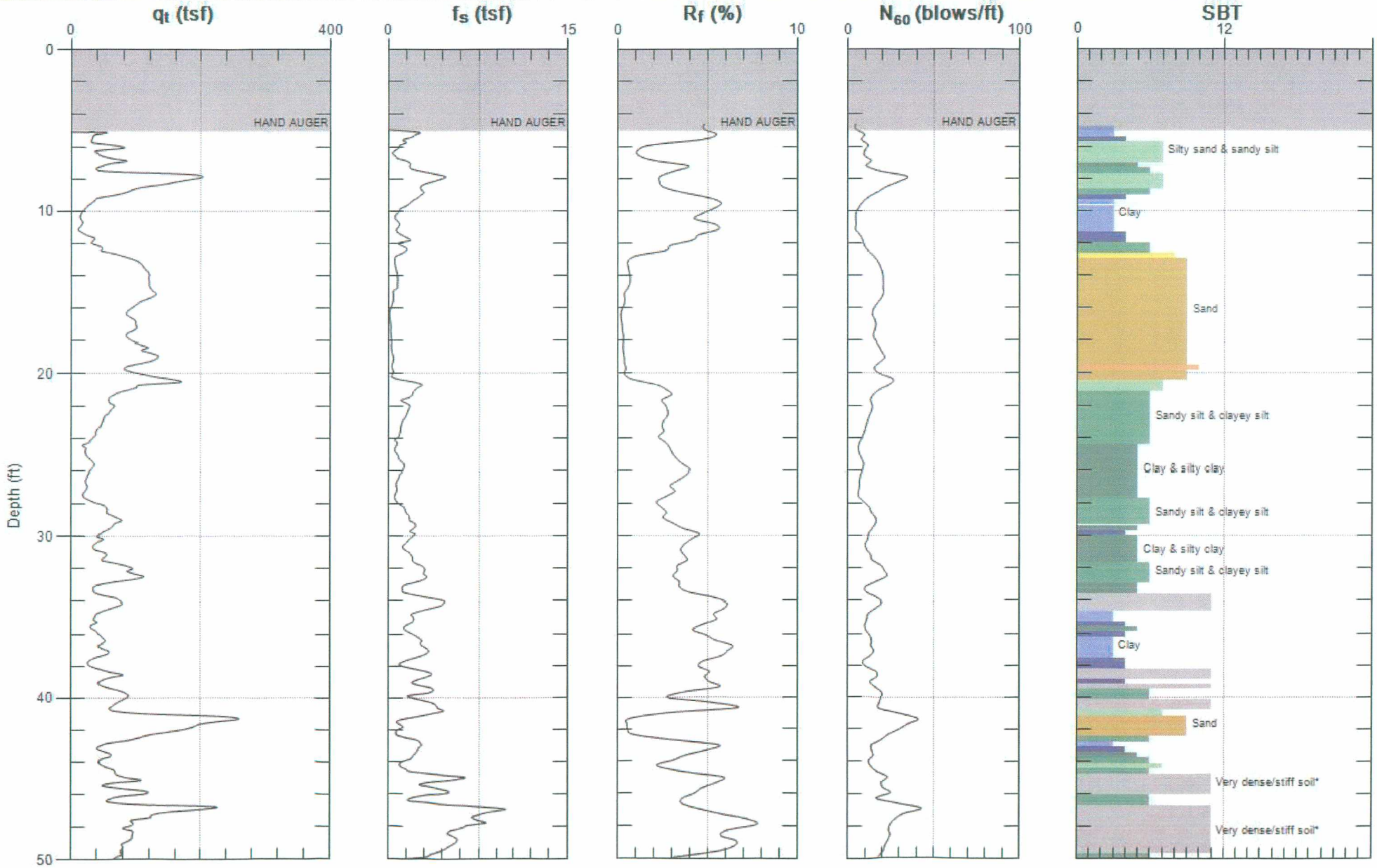
-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)
CPT-01	6/02/08	50	-	-	15.1
CPT-02	6/02/08	50	-	-	15.7
CPT-03	6/02/08	50	-	-	13.1, 17.4, 25.8, 30.7
CPT-04	6/02/08	50	-	-	16.2, 26.9

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-01

Engineer: M.KELLOGG
Date: 6/2/2008 09:01



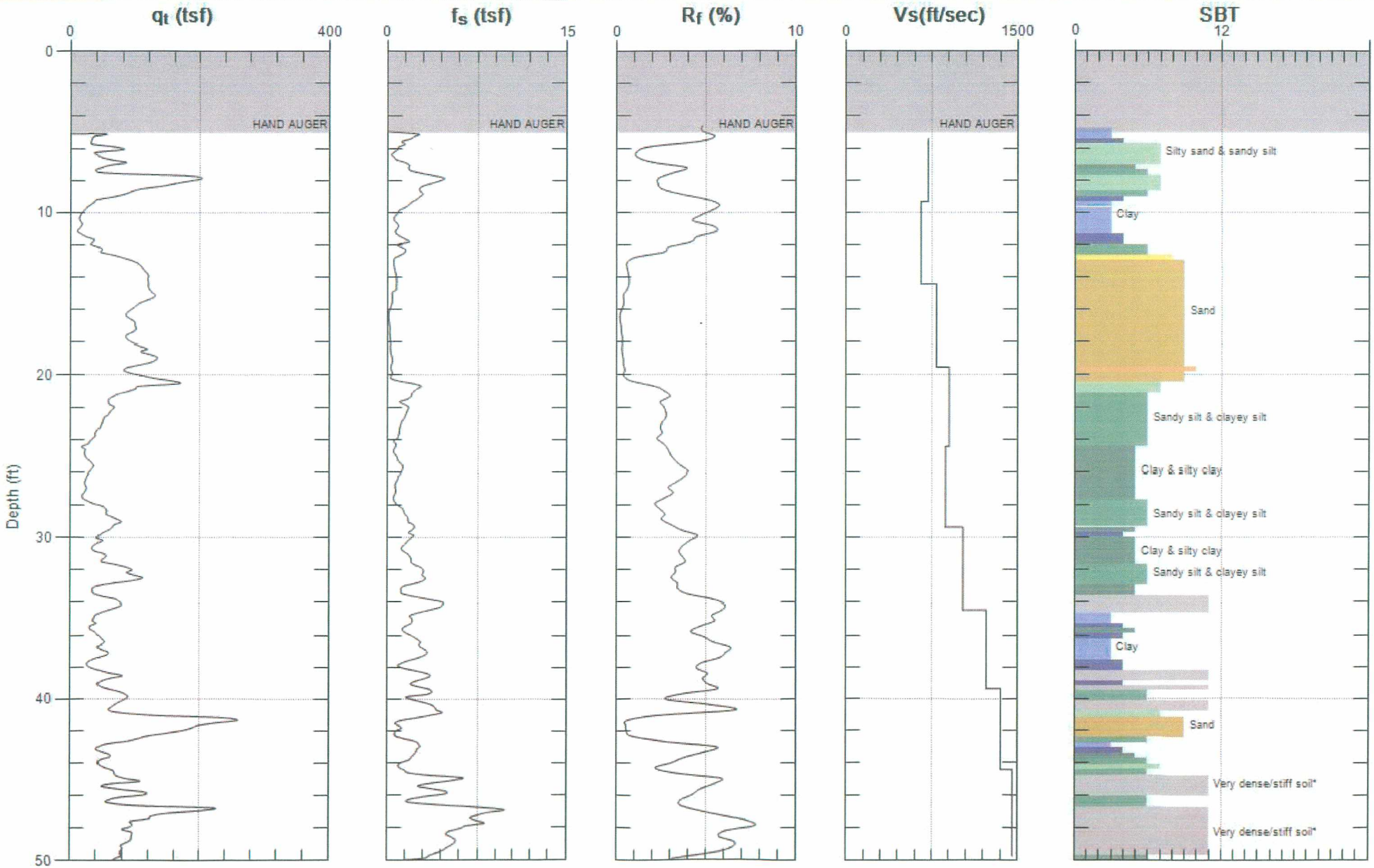
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-01

Engineer: M.KELLOGG
Date: 6/2/2008 09:01



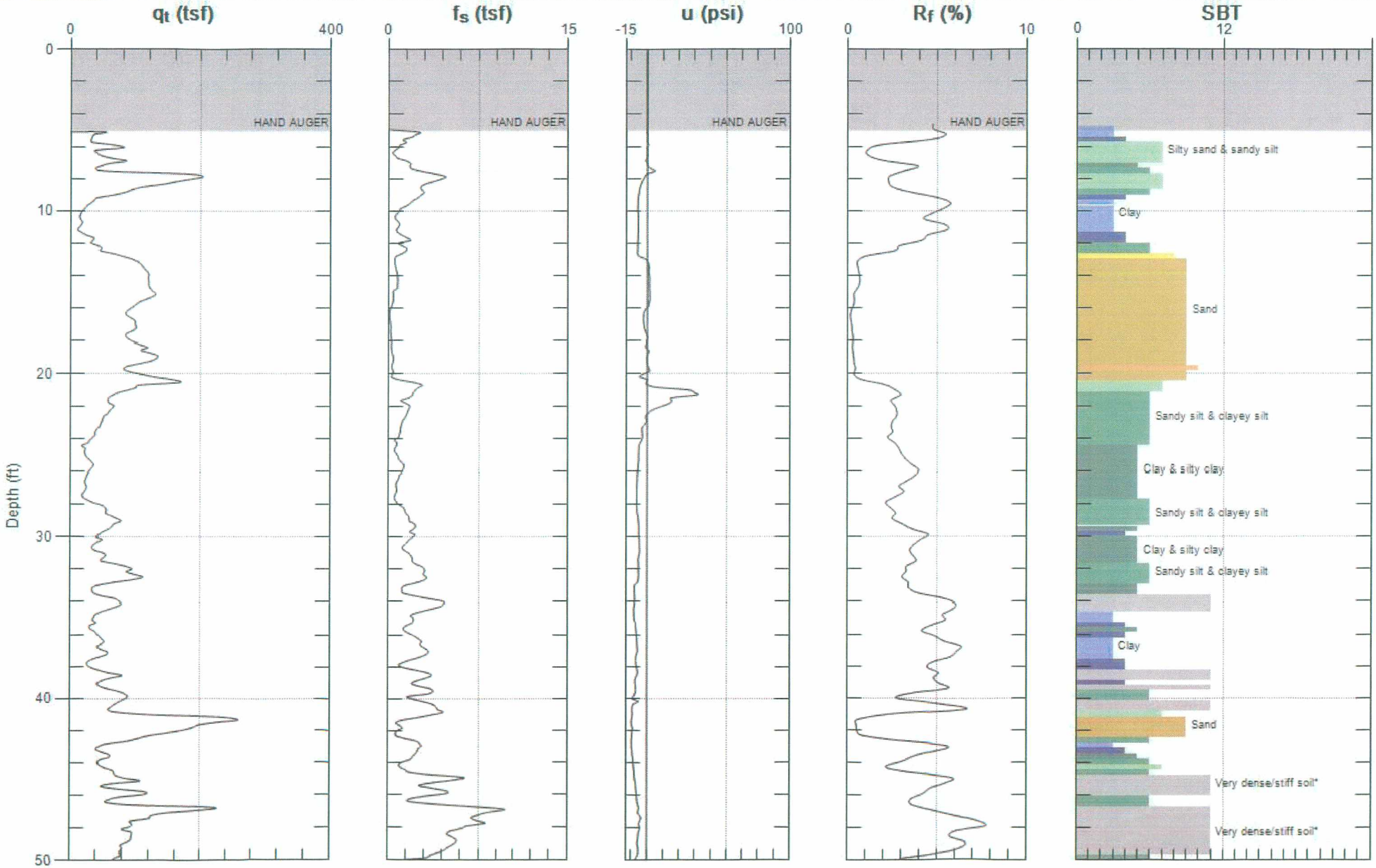
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-01

Engineer: M.KELLOGG
Date: 6/2/2008 09:01



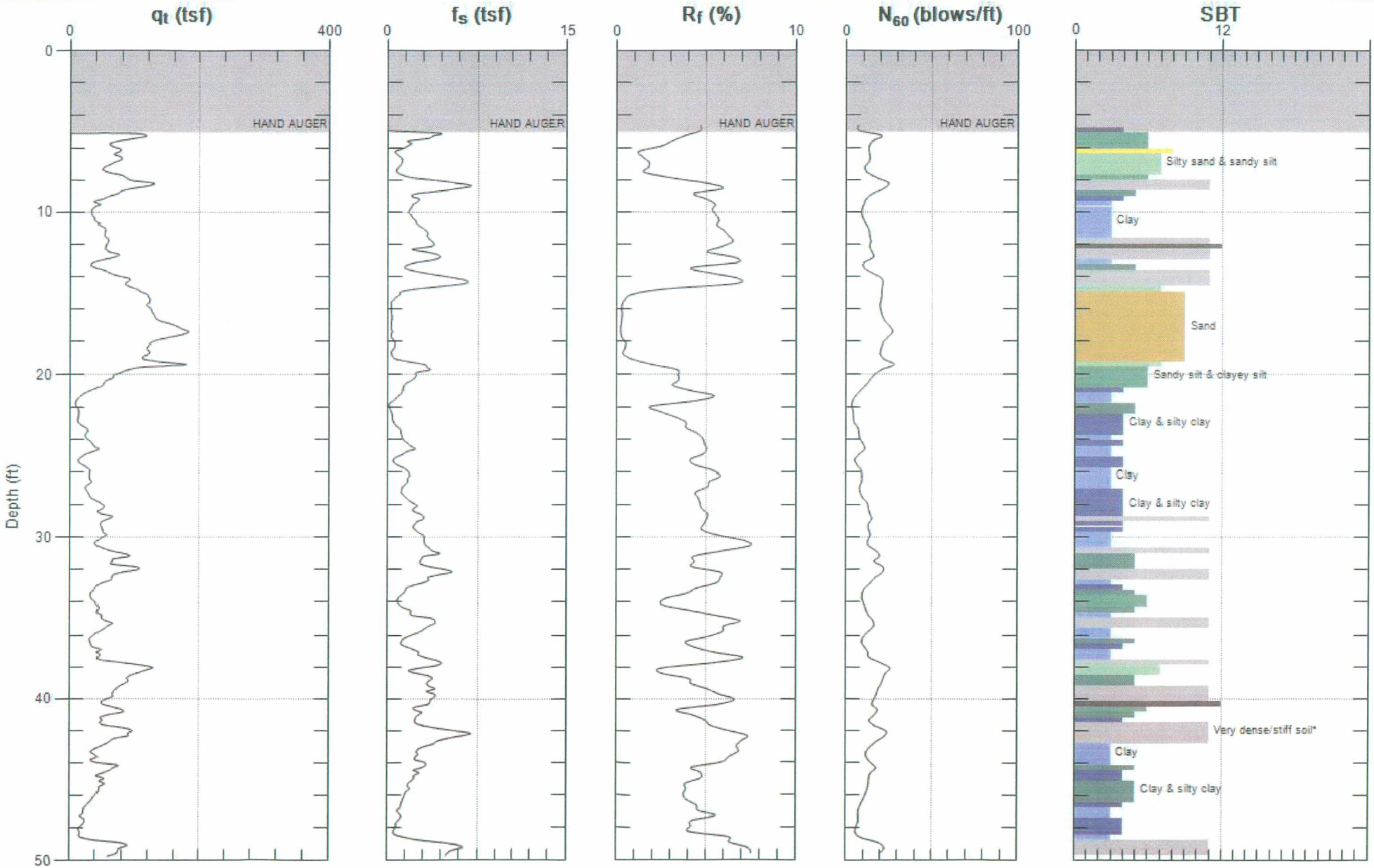
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-02

Engineer: M.KELLOGG
Date: 6/2/2008 11:35



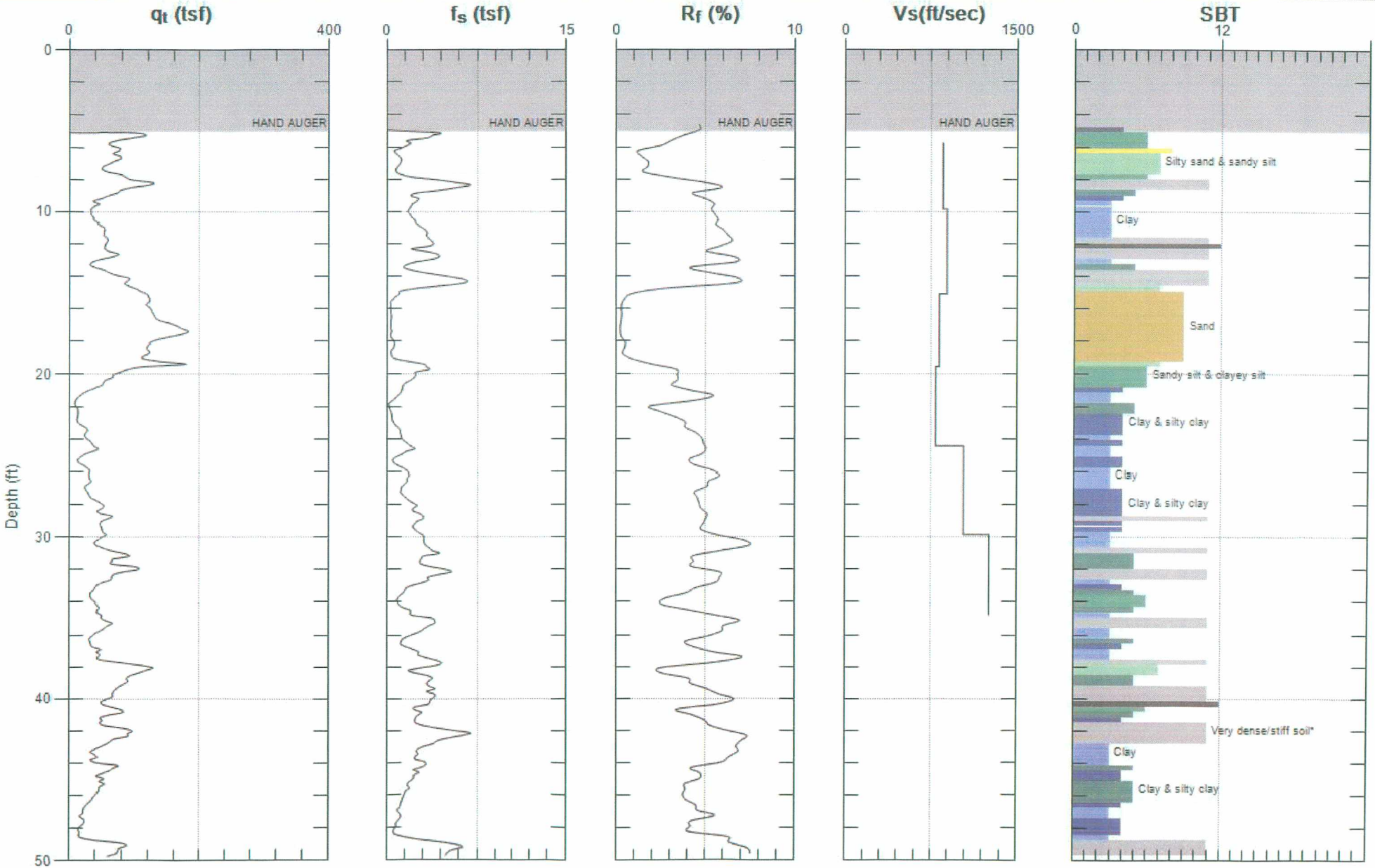
Max. Depth: 49.705 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-02

Engineer: M.KELLOGG
Date: 6/2/2008 11:35



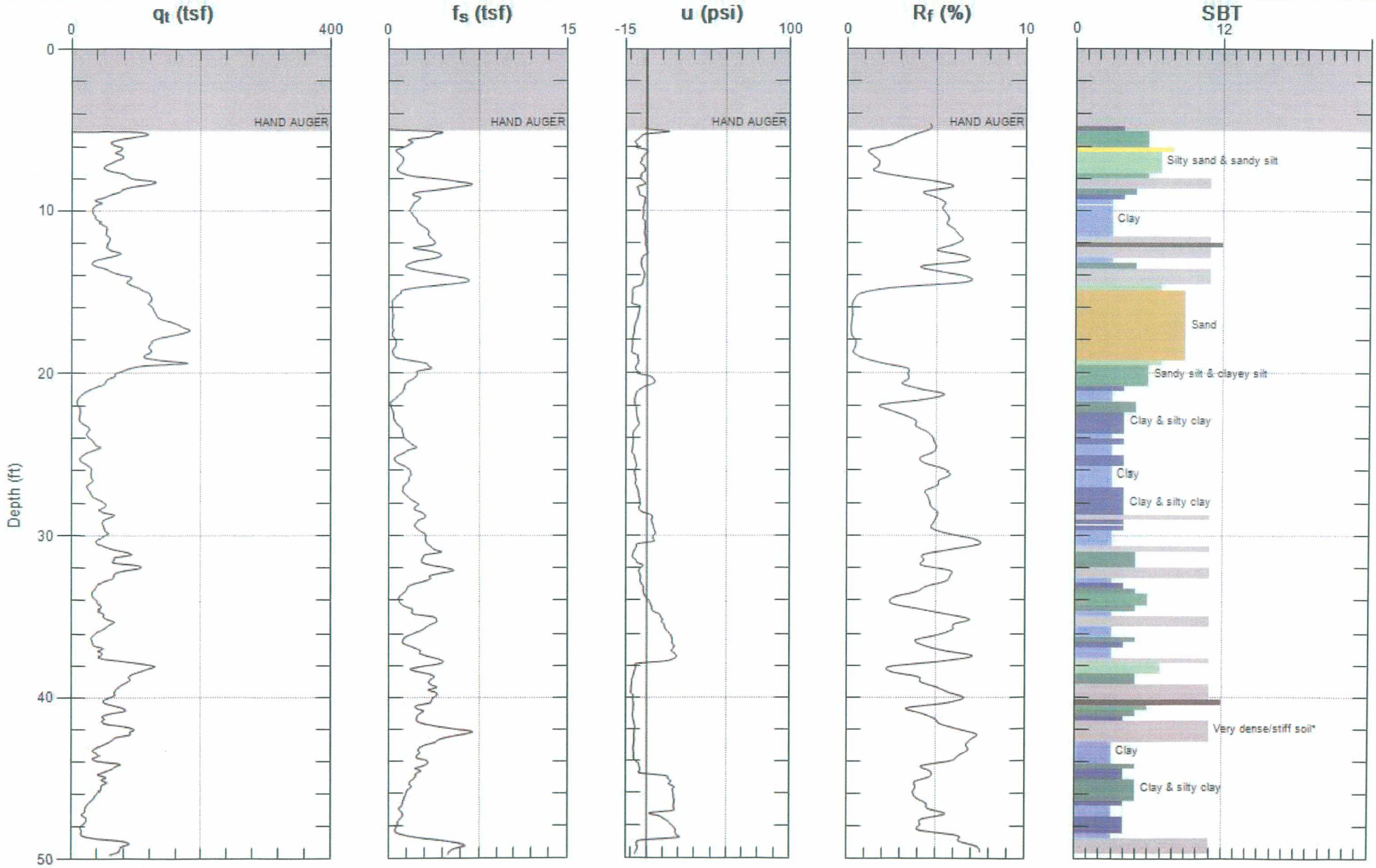
Max. Depth: 49.705 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-02

Engineer: M.KELLOGG
Date: 6/2/2008 11:35



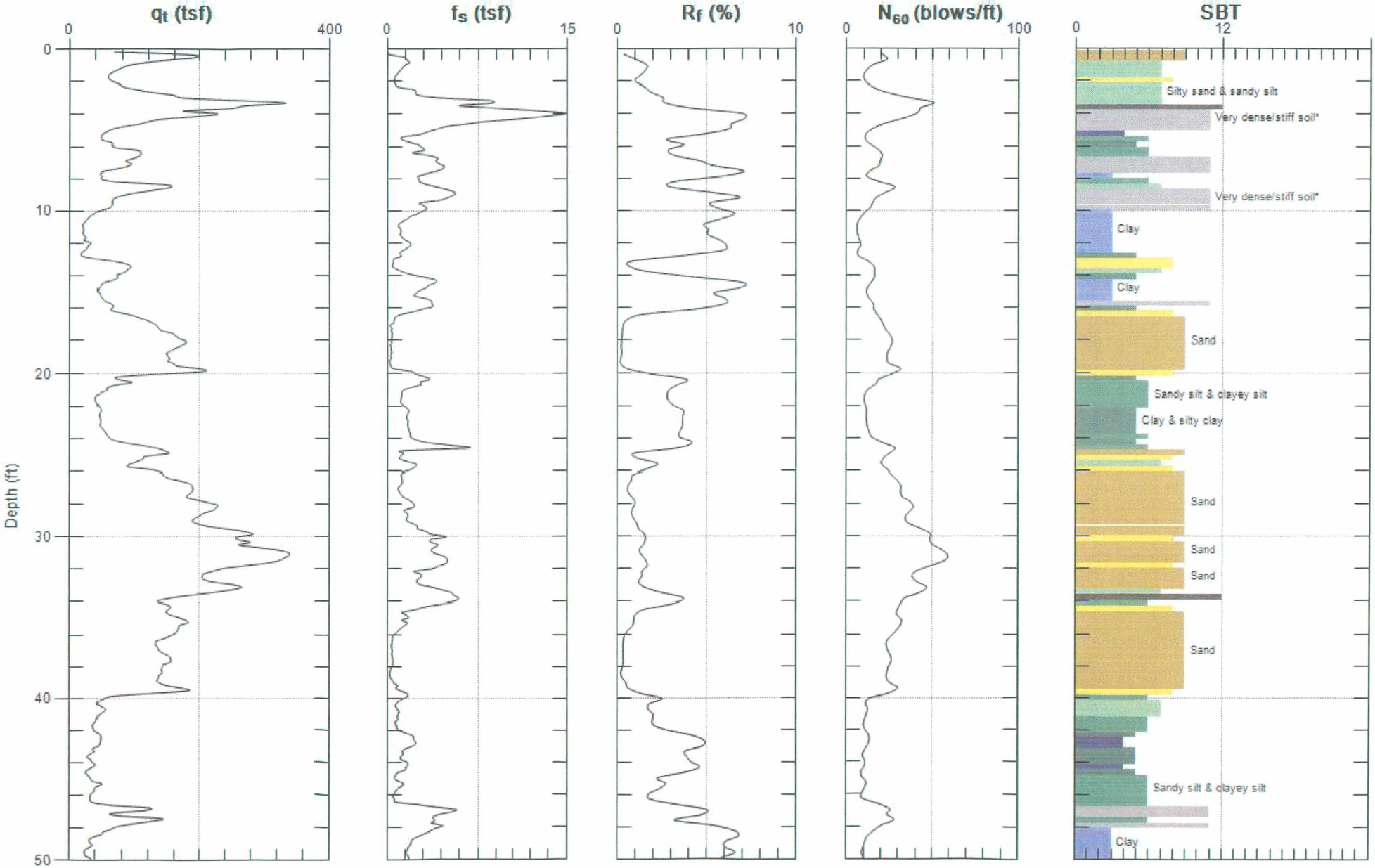
Max. Depth: 49.705 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-03

Engineer: M.KELLOGG
Date: 6/2/2008 01:11



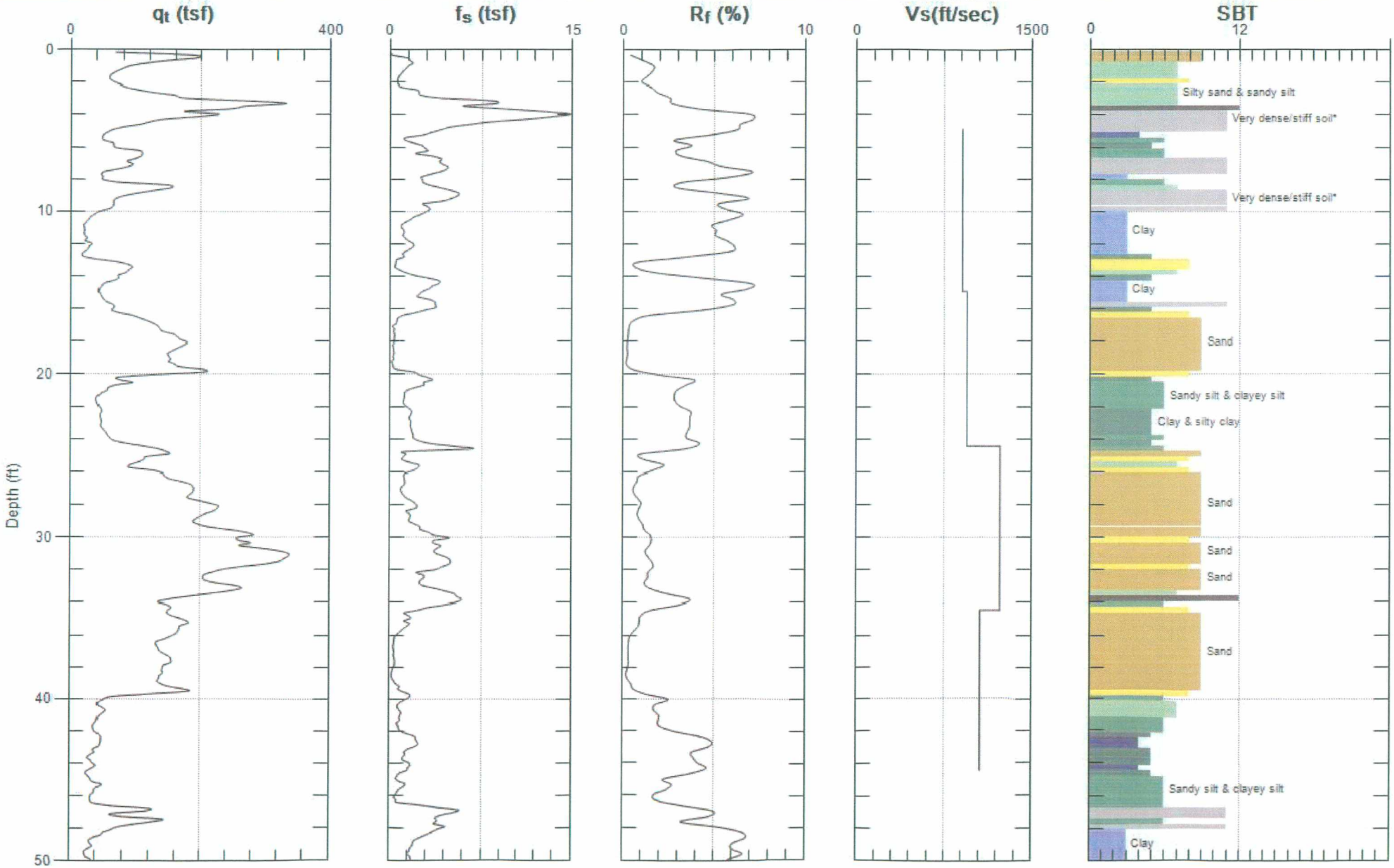
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-03

Engineer: M.KELLOGG
Date: 6/2/2008 01:11



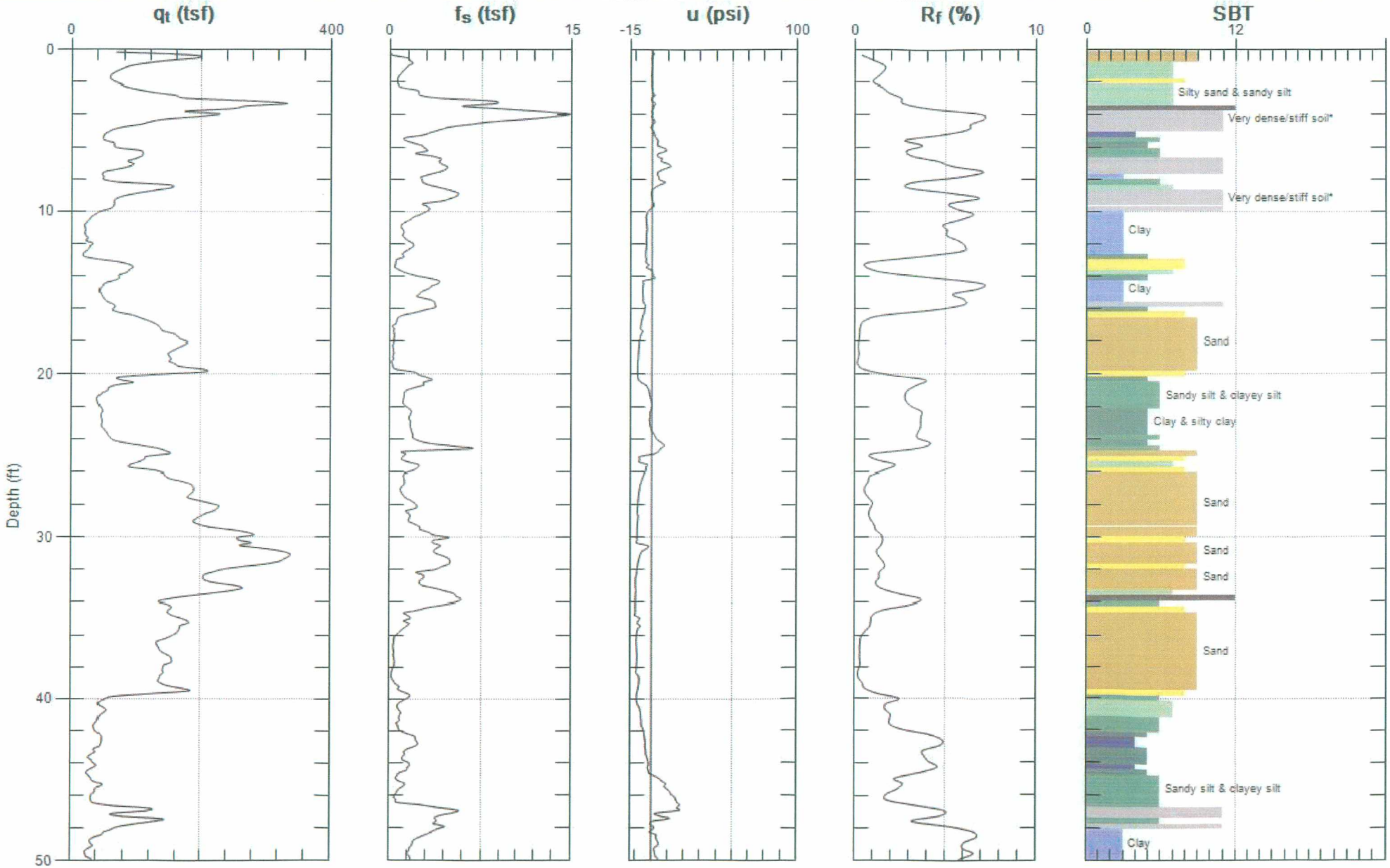
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-03

Engineer: M.KELLOGG
Date: 6/2/2008 01:11



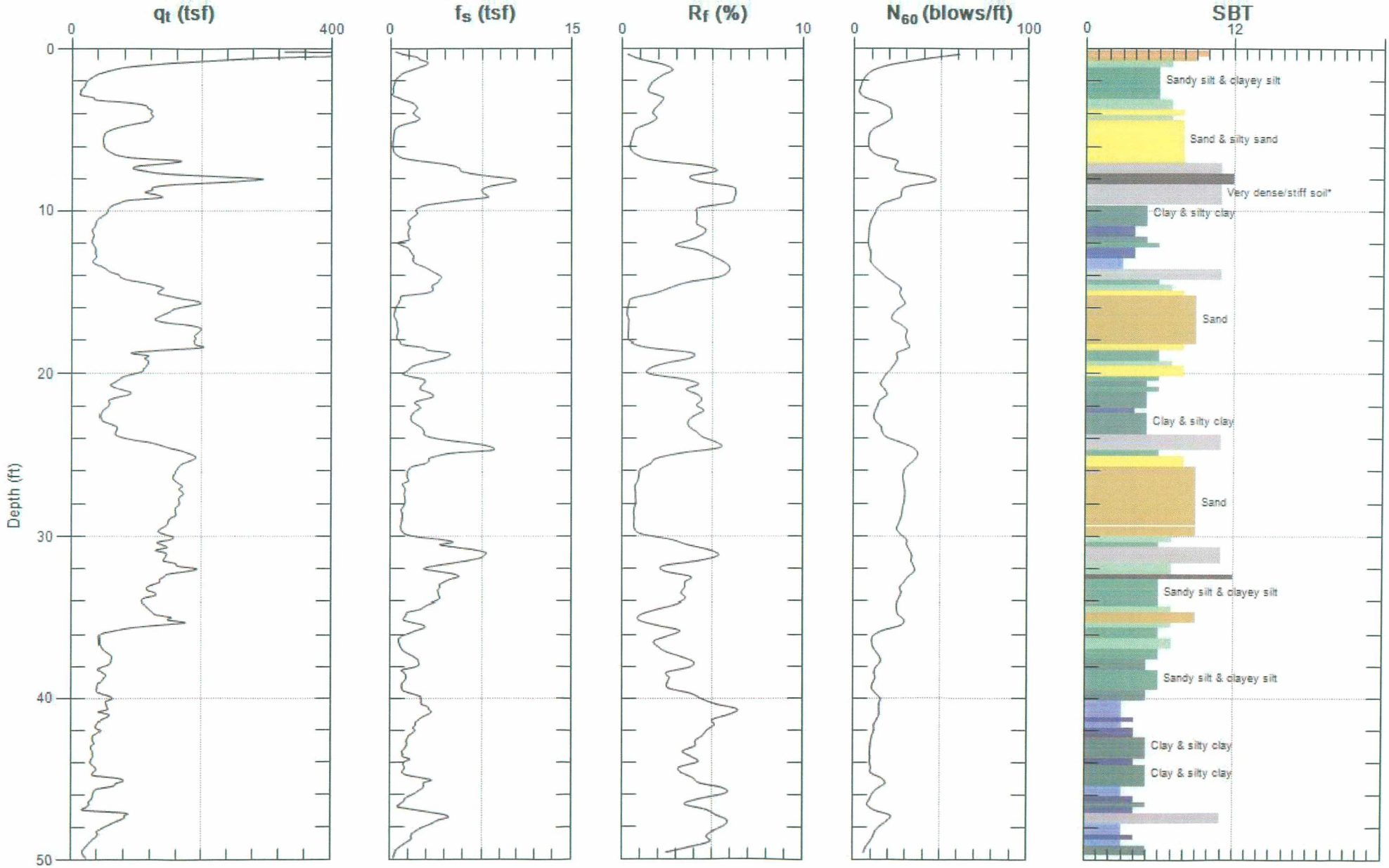
Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-04

Engineer: M.KELLOGG
Date: 6/2/2008 02:56



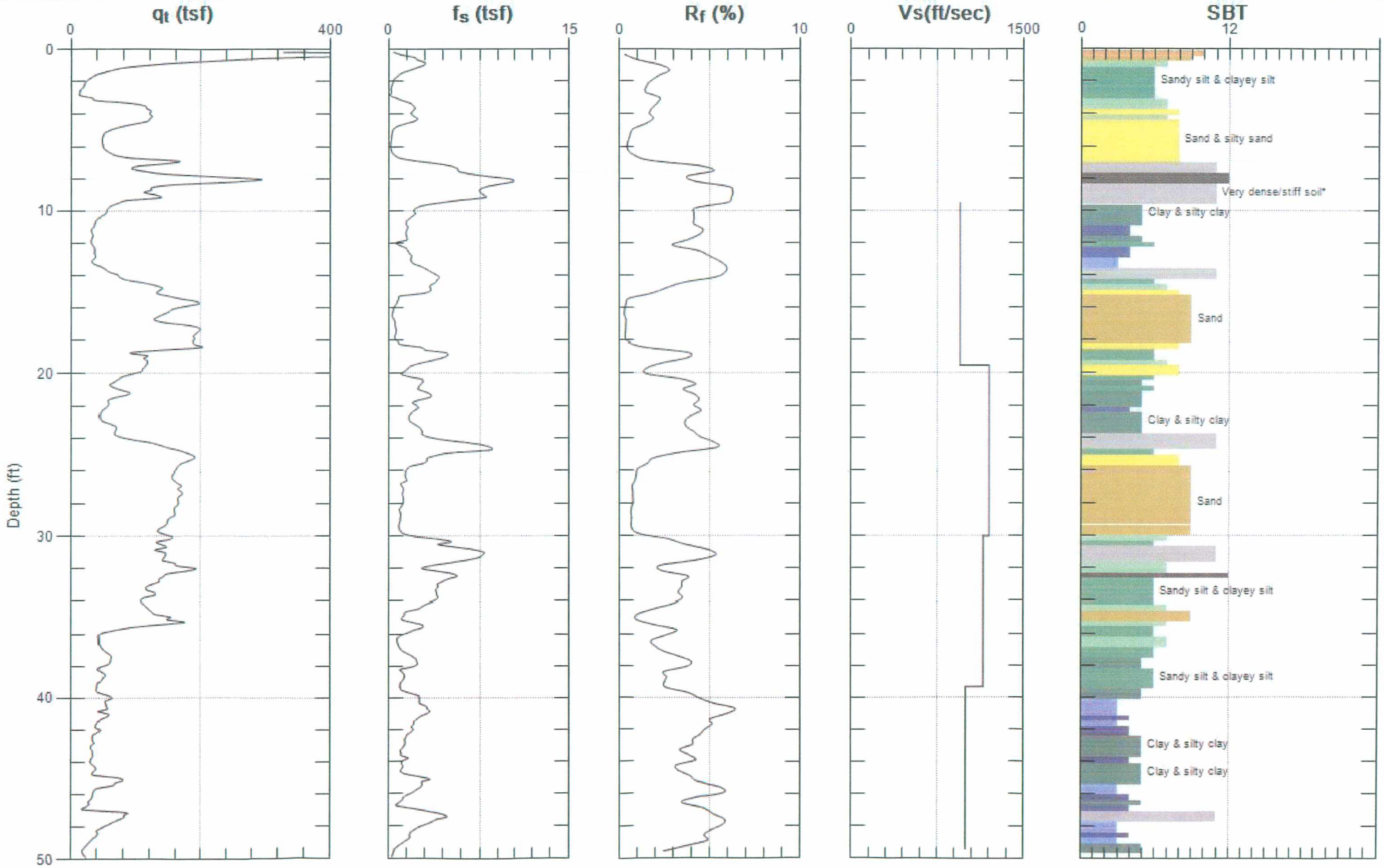
Max. Depth: 49.869 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-04

Engineer: M.KELLOGG
Date: 6/2/2008 02:56



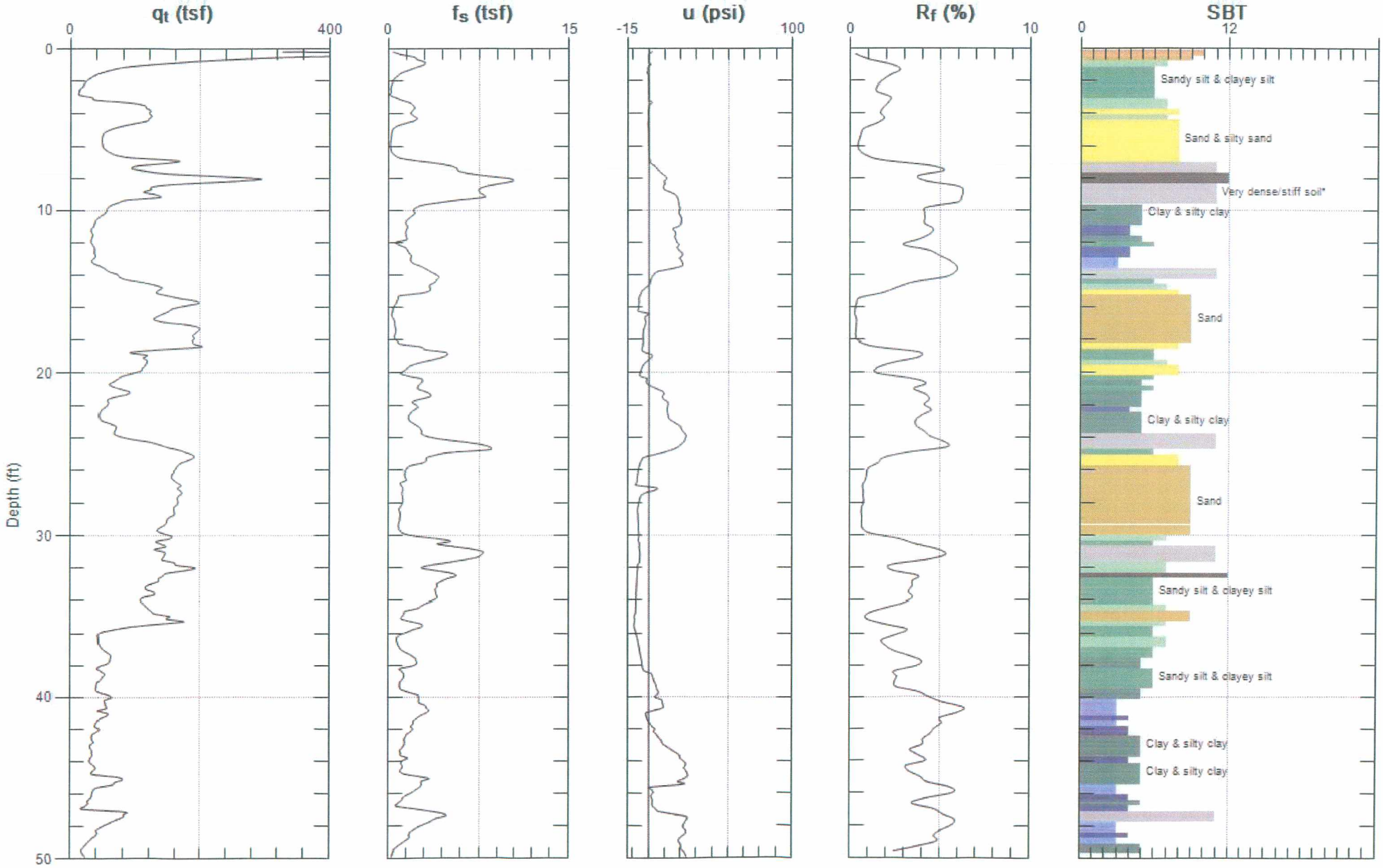
Max. Depth: 49.869 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CARLTON ENGINEERING

Site: NCPA LODI
Sounding: CPT-04

Engineer: M.KELLOGG
Date: 6/2/2008 02:56



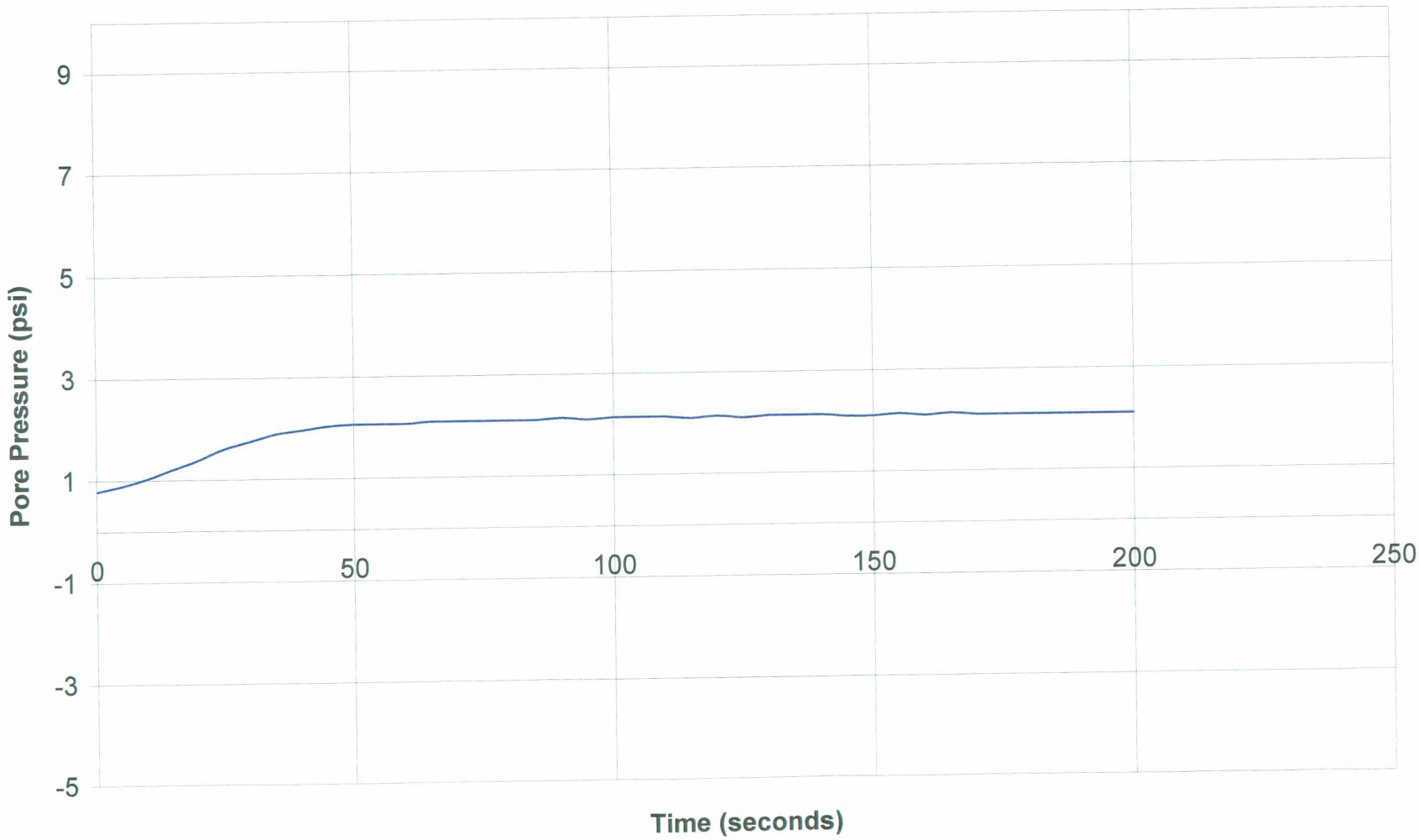
Max. Depth: 49.869 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

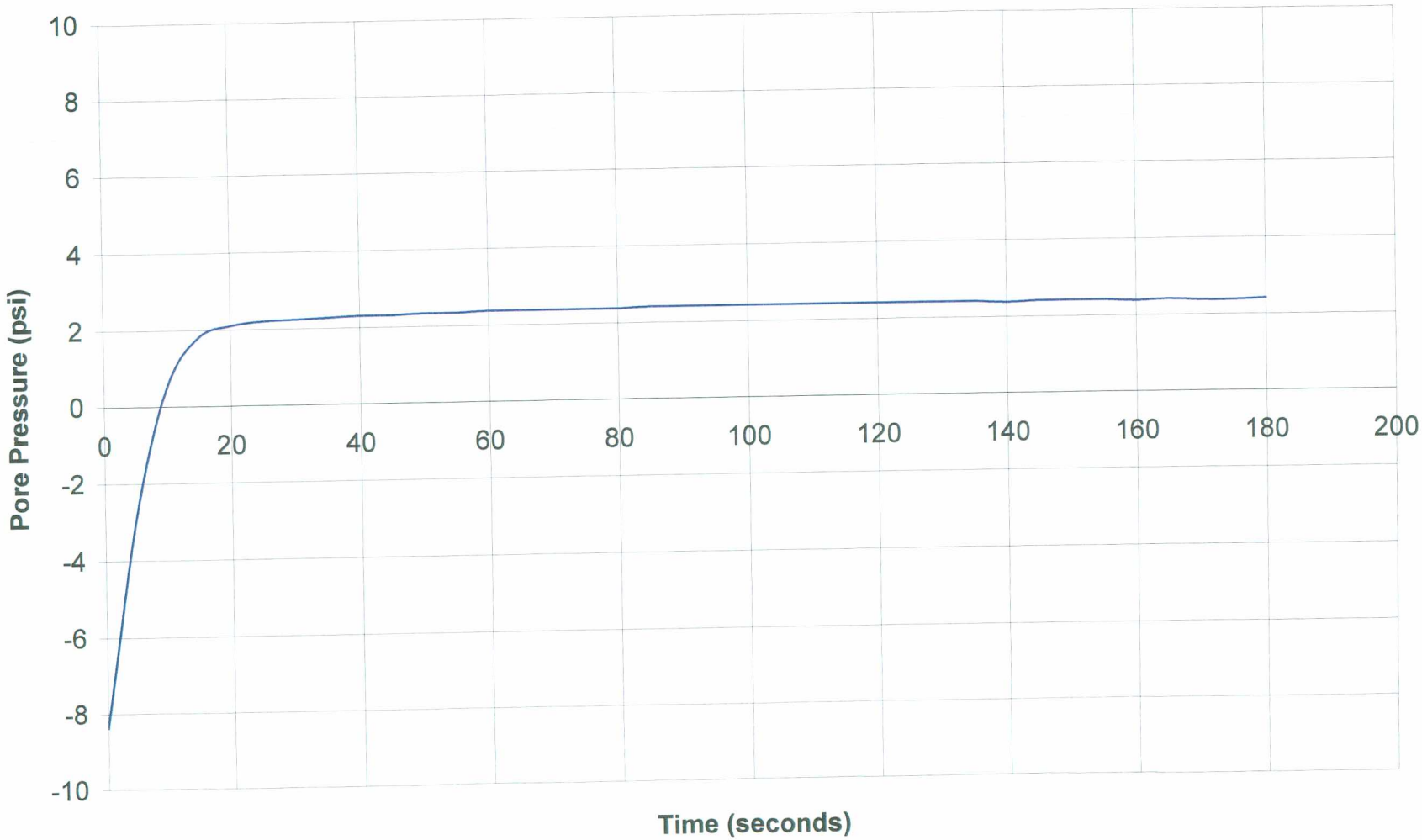
Sounding: CPT-01
Depth: 15.092
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

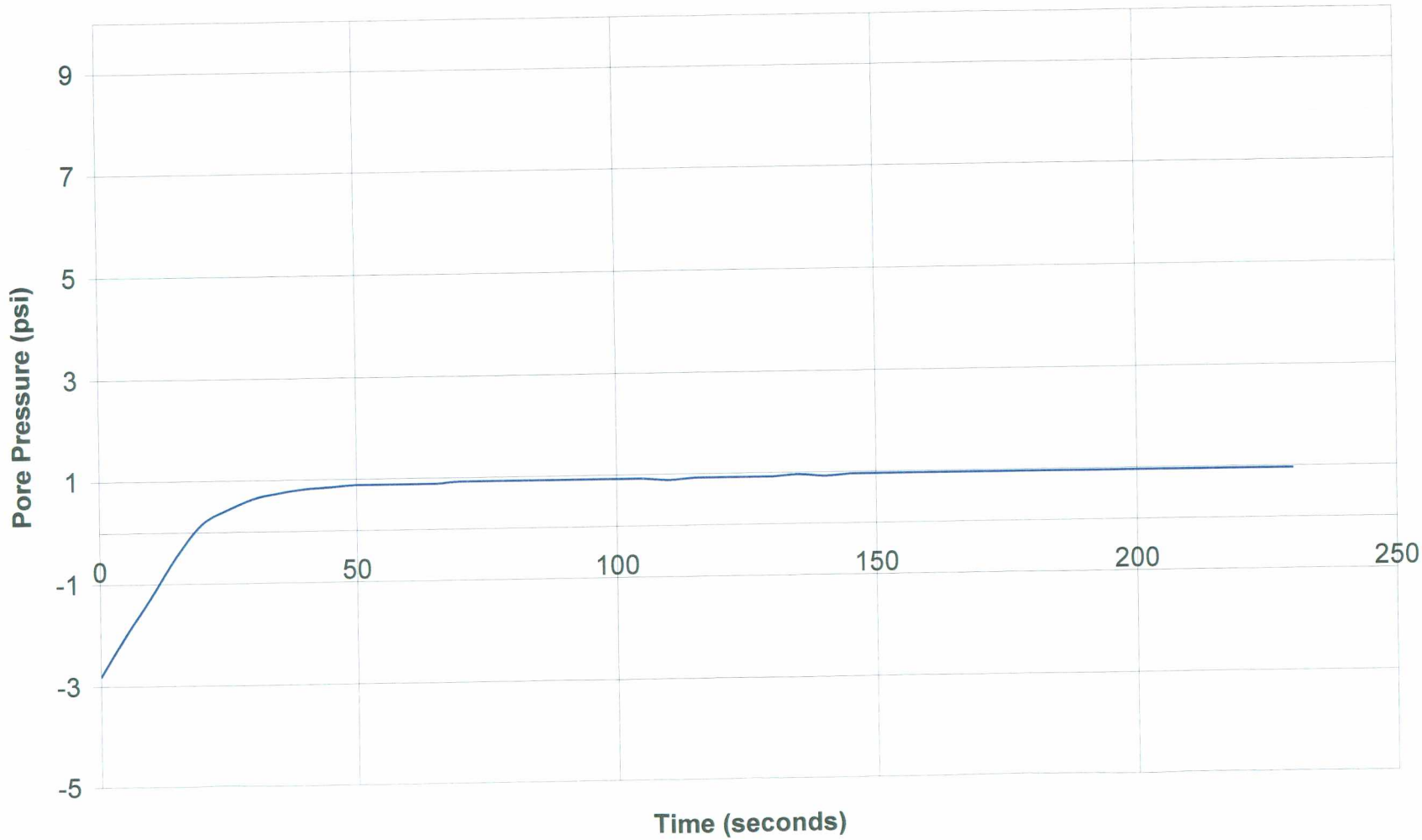
Sounding: CPT-02
Depth: 15.748
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

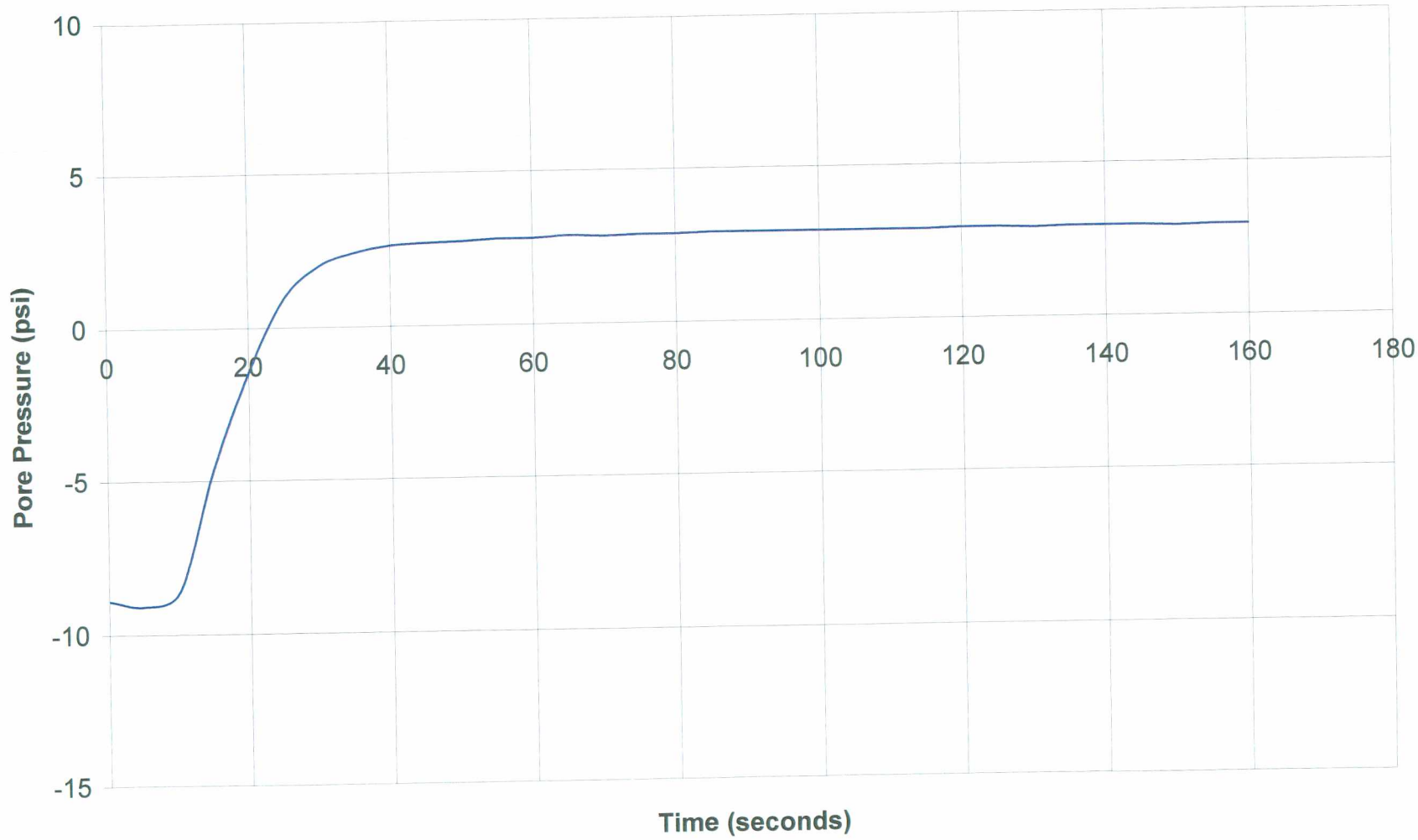
Sounding: CPT-03
Depth: 13.123
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

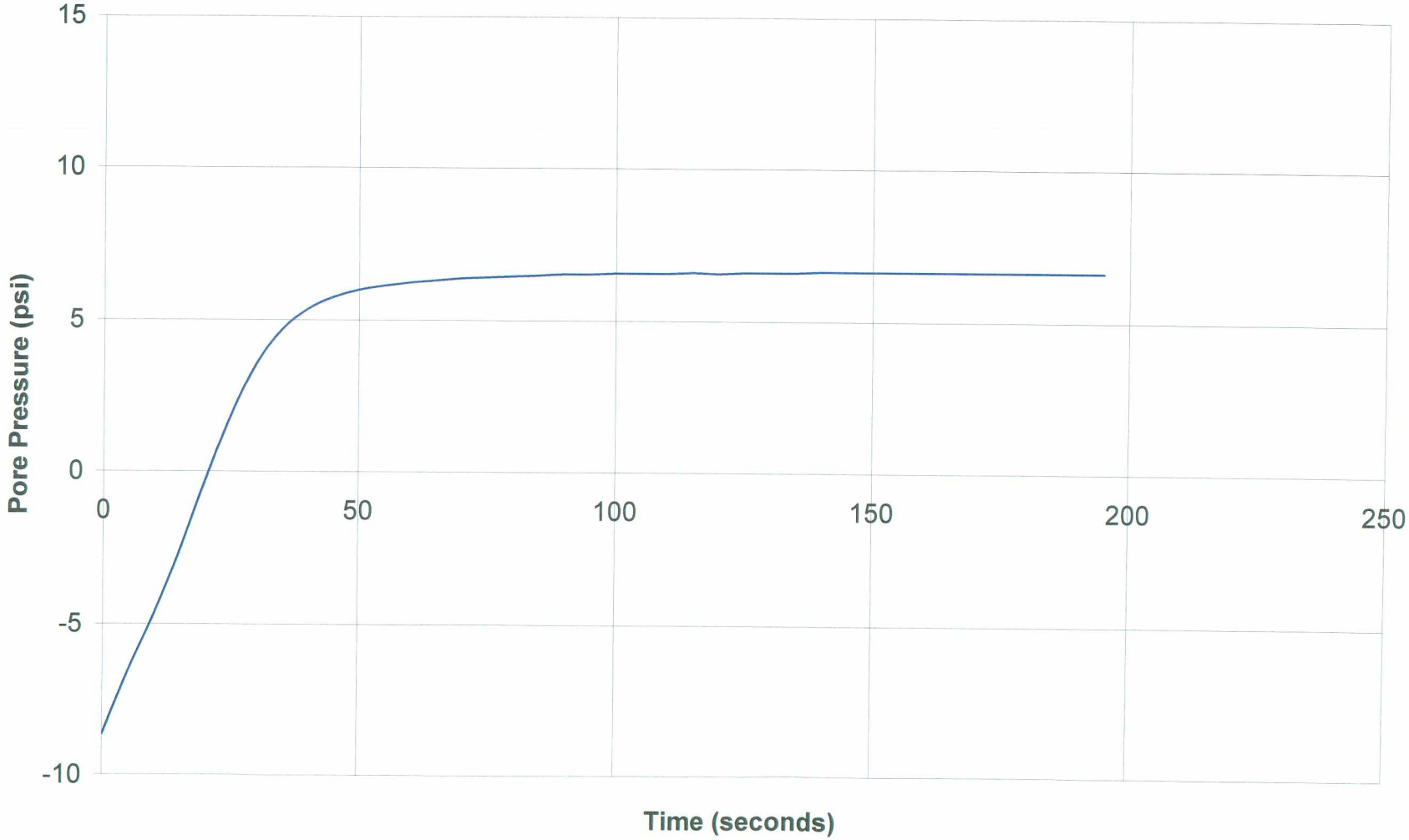
Sounding: CPT-03
Depth: 17.388
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

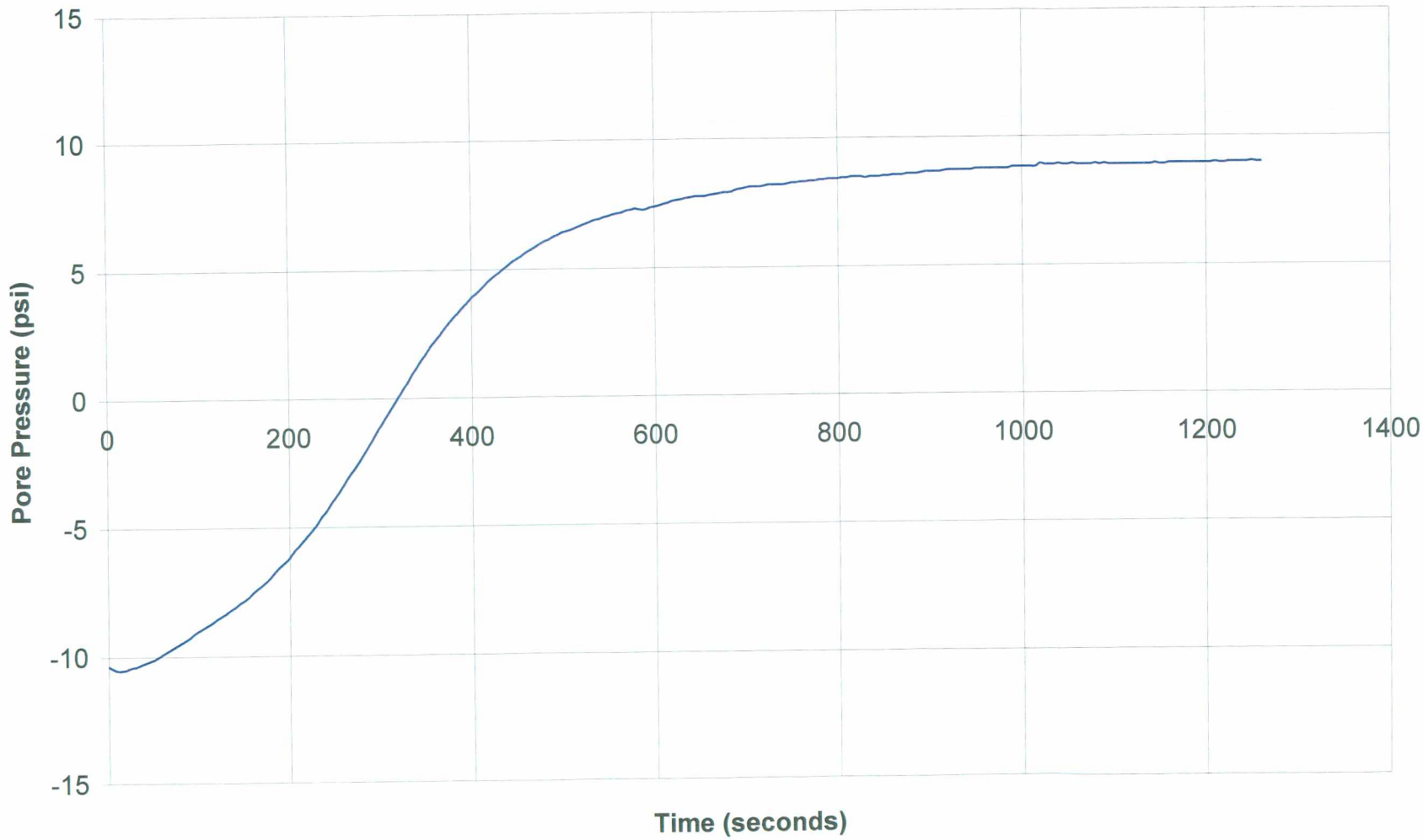
Sounding: CPT-03
Depth: 25.755
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

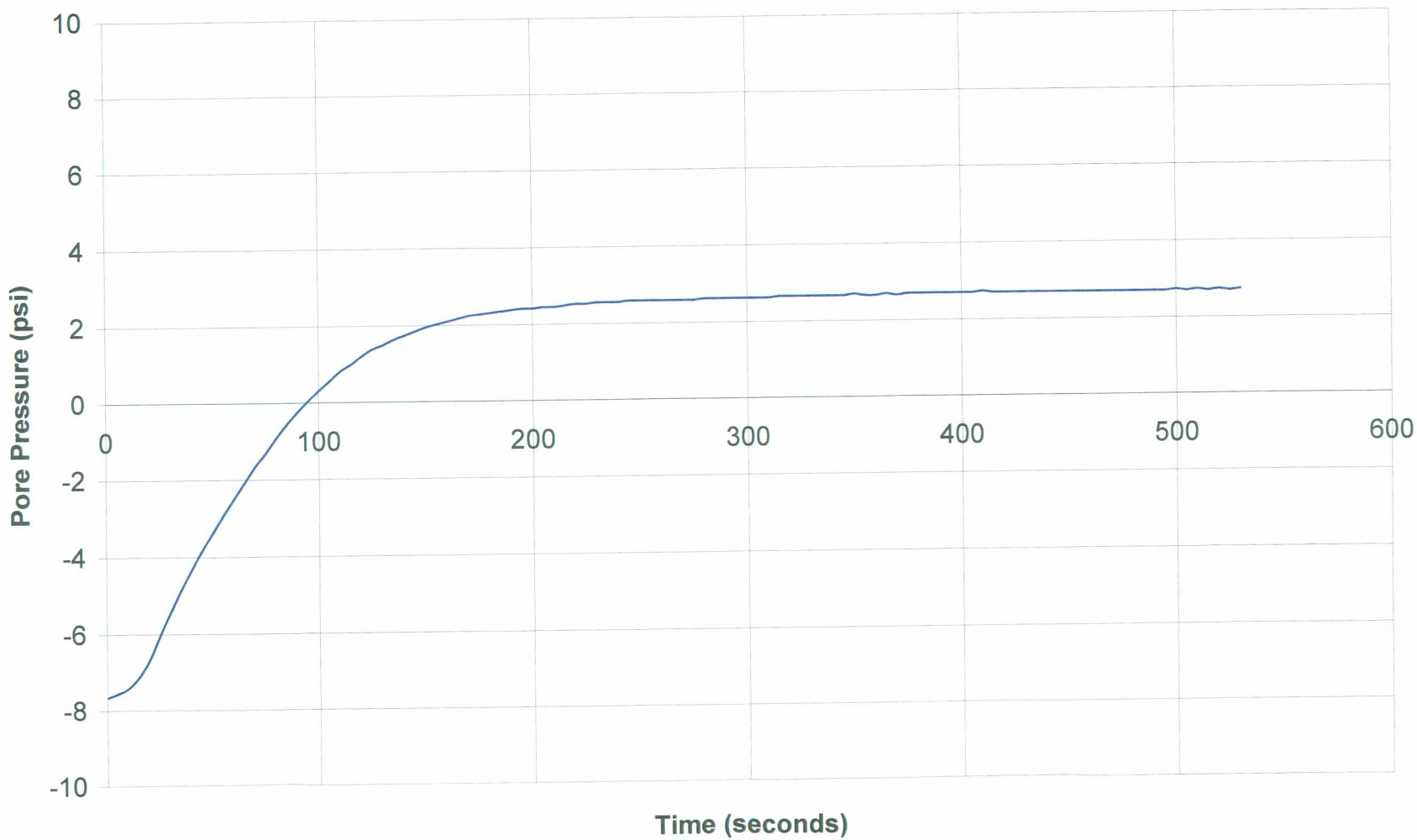
Sounding: CPT-03
Depth: 30.676
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

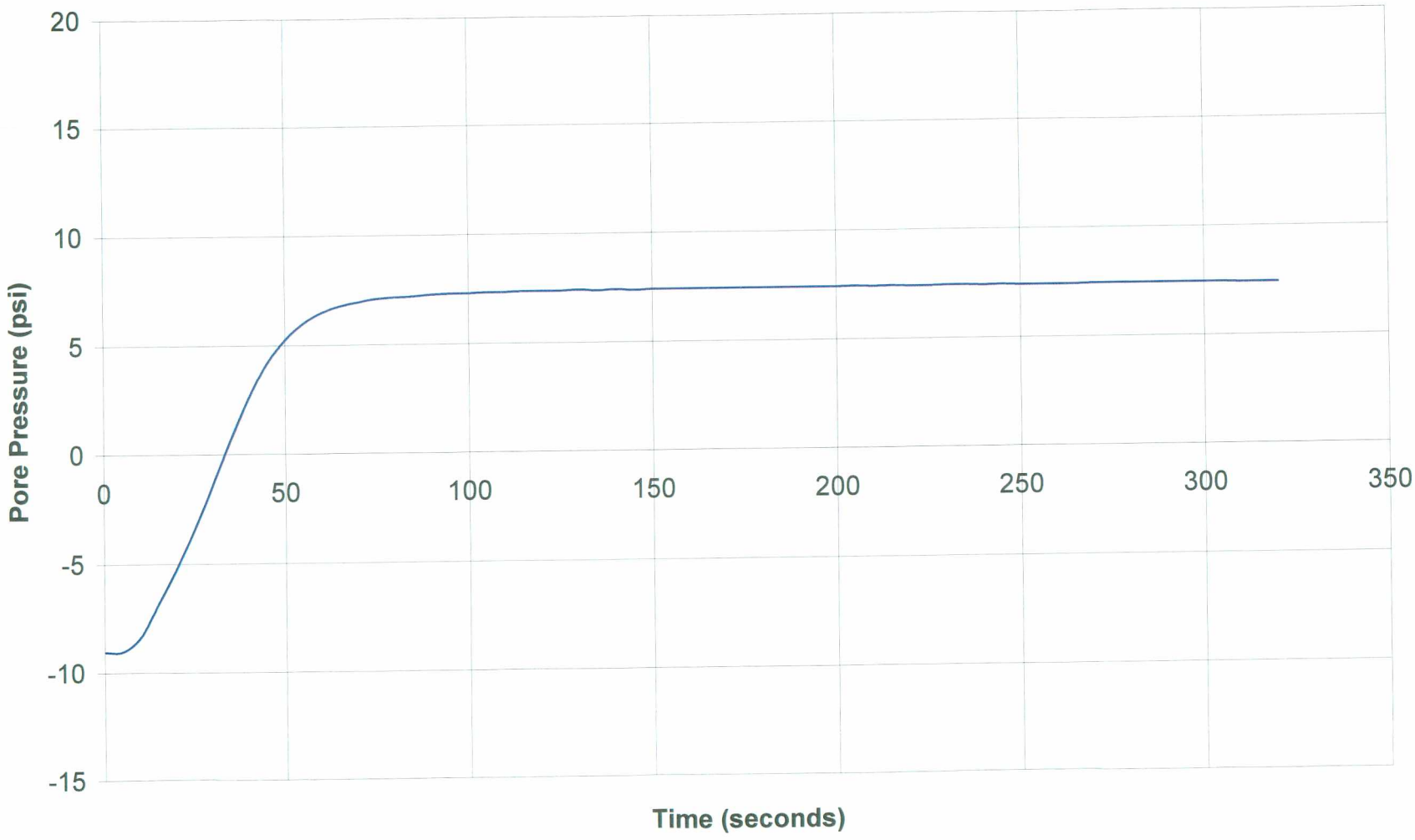
Sounding: CPT-04
Depth: 16.24
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG DRILLING & TESTING

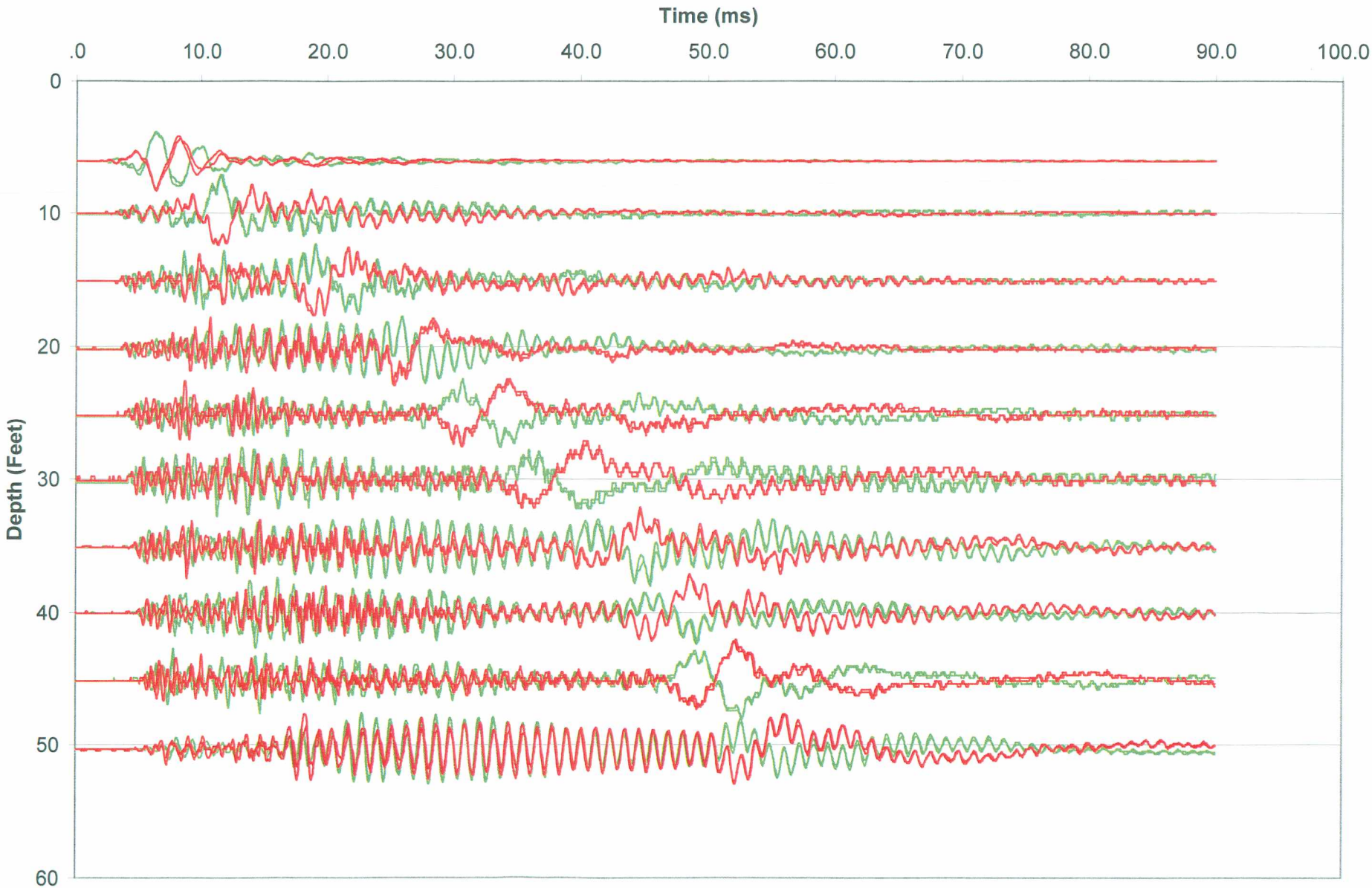
Pore Pressure Dissipation Test

Sounding: CPT-04
Depth: 26.903
Site: NCPA LODI
Engineer: M.KELLOGG



GREGG

Waveforms for Sounding CPT-01



Shear Wave Velocity Calculations

NCPA LODI

CPT-01

Geophone Offset: 0.66 Feet

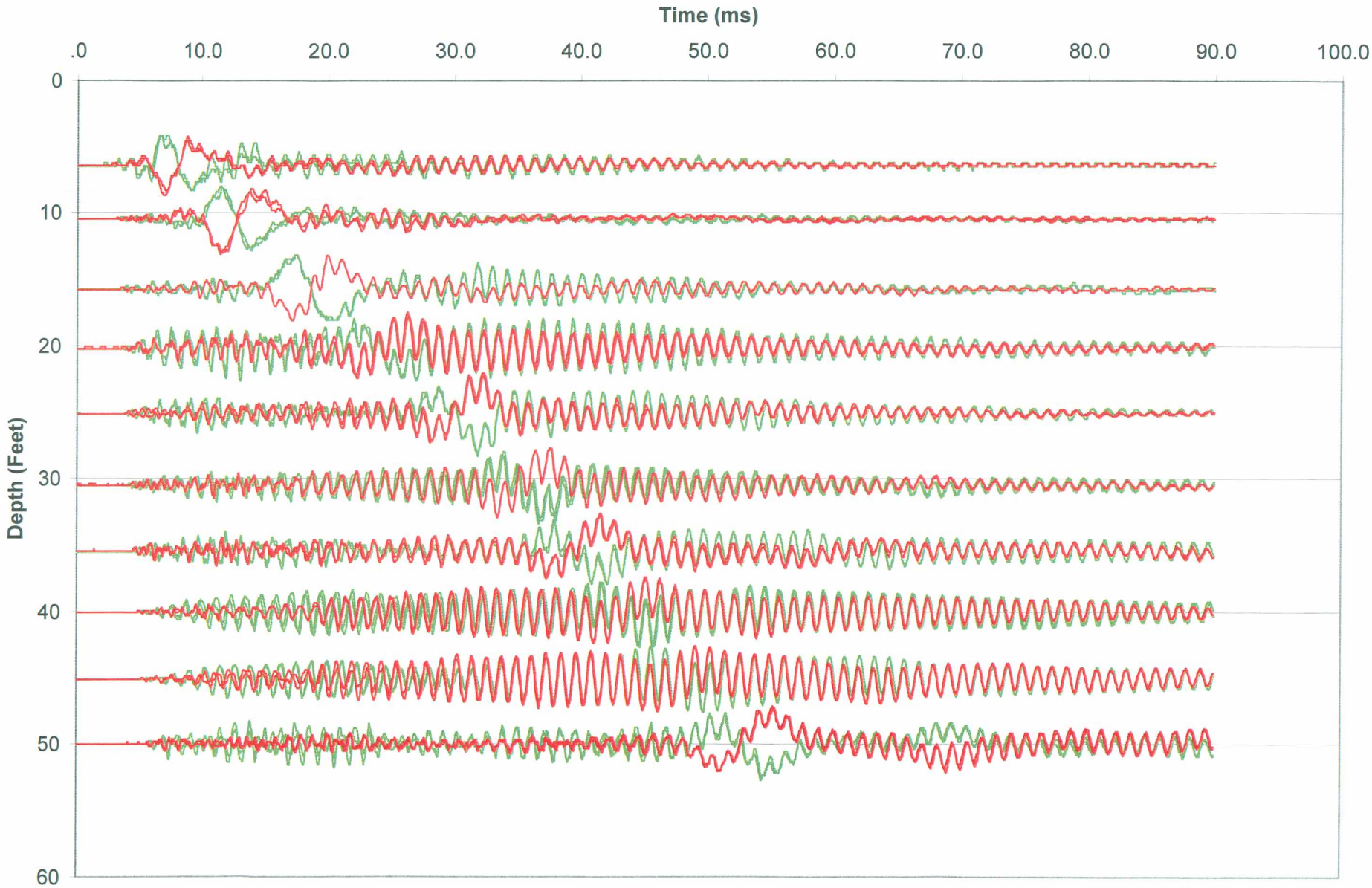
Source Offset: 1.67 Feet

6/2/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
6.07	5.41	5.66	5.66	7.3000			
10.01	9.35	9.49	3.83	12.6500	5.3500	716.5	7.38
15.09	14.43	14.53	5.03	20.3500	7.7000	653.7	11.89
20.18	19.52	19.59	5.06	26.7500	6.4000	790.7	16.97
25.10	24.44	24.50	4.91	32.2000	5.4500	900.4	21.98
30.02	29.36	29.41	4.91	37.8500	5.6500	869.3	26.90
35.10	34.44	34.49	5.08	42.8000	4.9500	1025.9	31.90
40.03	39.37	39.40	4.92	46.8000	4.0000	1229.0	36.91
45.11	44.45	44.48	5.08	50.5500	3.7500	1355.0	41.91
50.36	49.70	49.73	5.25	54.1500	3.6000	1457.2	47.08

GREGG

Waveforms for Sounding CPT-02



Shear Wave Velocity Calculations

NCPA LODI

CPT-02

Geophone Offset: 0.66 Feet

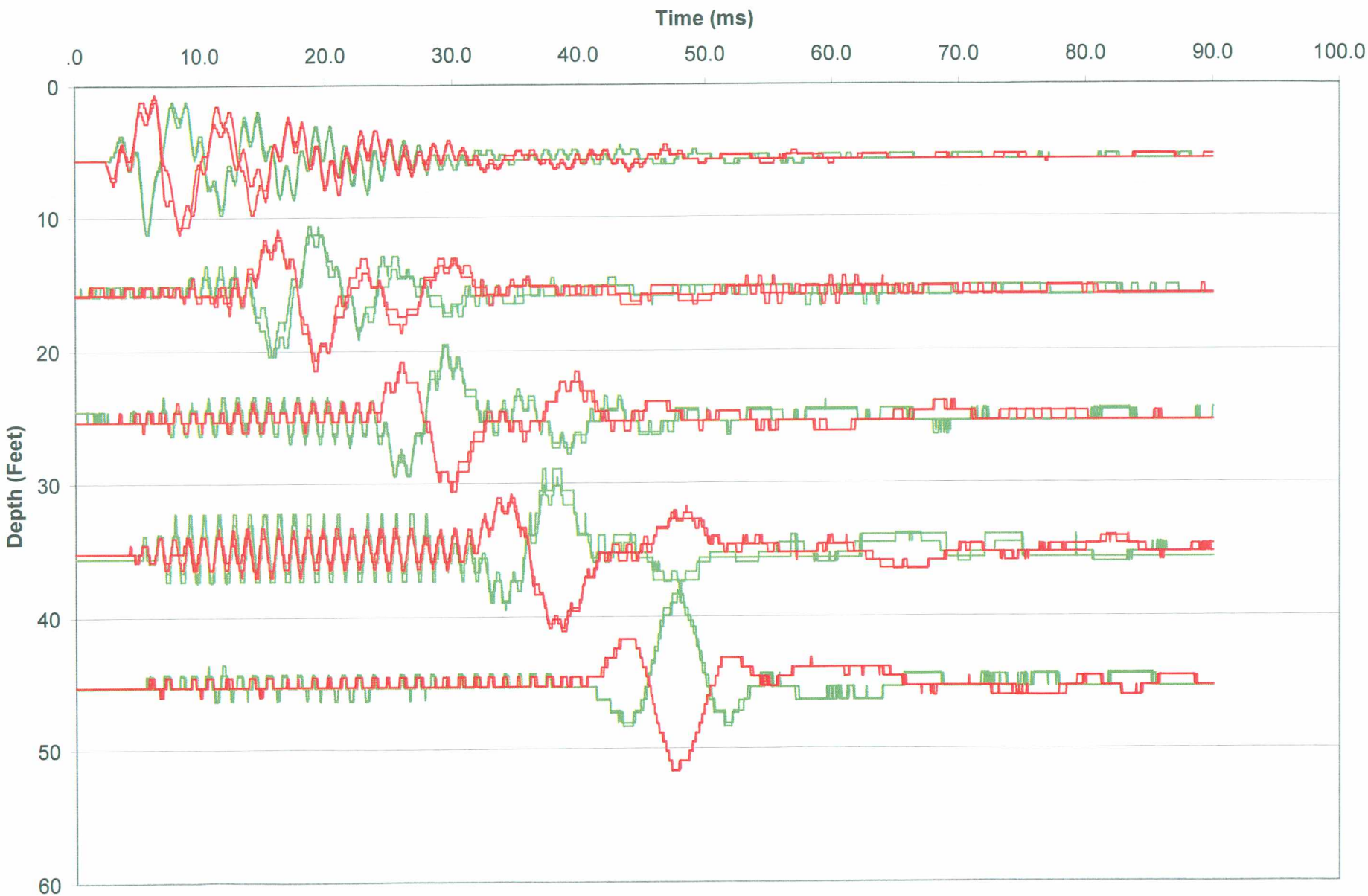
Source Offset: 1.67 Feet

6/2/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
6.40	5.74	5.98	5.98	8.1000			
10.50	9.84	9.98	4.00	12.8000	4.7000	851.8	7.79
15.75	15.09	15.18	5.20	18.6500	5.8500	889.0	12.46
20.18	19.52	19.59	4.41	24.0000	5.3500	824.0	17.30
25.10	24.44	24.50	4.91	30.2000	6.2000	791.4	21.98
30.51	29.85	29.90	5.40	35.4000	5.2000	1039.0	27.15
35.43	34.77	34.81	4.91	39.3000	3.9000	1260.2	32.31
40.03	39.37	39.40	4.59				37.07
45.11	44.45	44.48	5.08				41.91
50.03	49.37	49.40	4.92	52.8000			46.91

GREGG

Waveforms for Sounding CPT-03



Shear Wave Velocity Calculations

NCPA LODI

CPT-03

Geophone Offset: 0.66 Feet

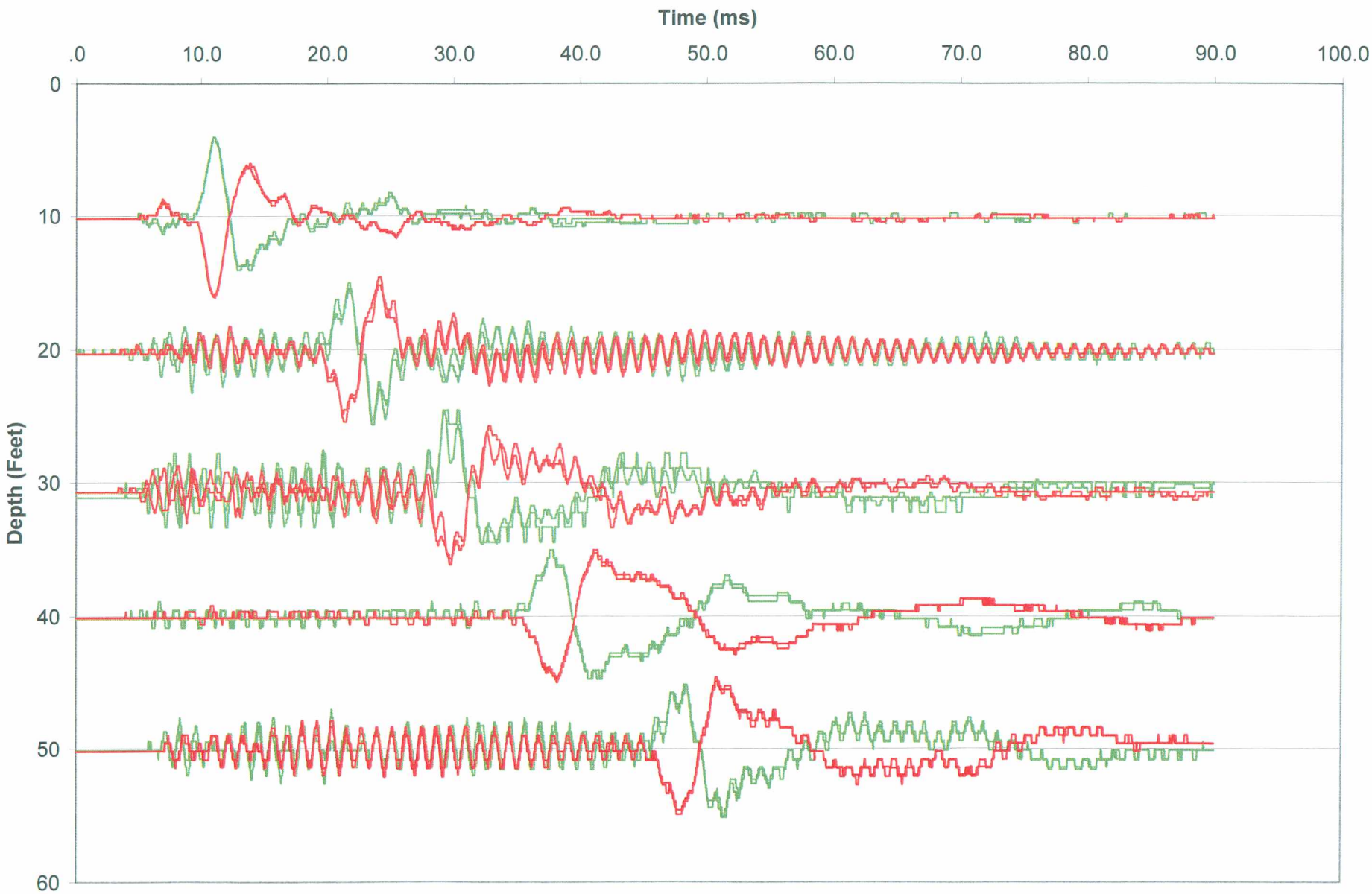
Source Offset: 1.67 Feet

6/2/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
5.58	4.92	5.19	5.19	6.9500			
15.58	14.92	15.02	9.82	17.8000	10.8500	905.4	9.92
25.10	24.44	24.50	9.48	27.8000	10.0000	947.8	19.68
35.10	34.44	34.49	9.99	35.9000	8.1000	1233.3	29.44
45.11	44.45	44.48	10.00	45.3000	9.4000	1063.6	39.45

GREGG

Waveforms for Sounding CPT-04



Shear Wave Velocity Calculations

NCPA LODI

CPT-04

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

6/2/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.17	9.51	9.66	9.66	12.2500			
20.18	19.52	19.59	9.93	22.7000	10.4500	950.5	14.51
30.68	30.02	30.06	10.47	31.4000	8.7000	1203.9	24.77
40.03	39.37	39.40	9.34	39.5000	8.1000	1153.0	34.69
50.03	49.37	49.40	10.00	49.5000	10.0000	999.9	44.37