GEOTECHNICAL REPORT

STOCKTON COURTHOUSE
STOCKTON, CALIFORNIA

Submitted to:

James Tully
NBBJ
223 Yale Avenue North
Seattle, Washington 98109

March 25, 2009
Revised April 24, 2009
Project No. 8641.000.000

.

- Expect Excellence -

Project No.
8641.000.000

March 25, 2009
Revised April 24, 2009

James Tully, Principal
NBBJ
223 Yale Avenue North
Seattle, WA 98109

Subject: Stockton Courthouse
East Weber Avenue
Stockton, California

GEOTECHNICAL REPORT

Dear Mr. Tully:

ENGEO Incorporated prepared this geotechnical report for NBBJ as outlined in our agreement dated January 27, 2009. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicates that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Paul Cottingham, CEG
Project Geologist

Jonathan Boland, GE
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
MARK M. GILBERT
No. 2191
Exp. 3/31/2010
GEOTECHNICAL
STATE OF CALIFORNIA

Mark Gilbert, GE
Principal

TABLE OF CONTENTS

Letter of Transmittal

FIGURES

1.0 INTRODUCTION

ENGEO Incorporated prepared this geotechnical report for design of the Stockton Courthouse building in Stockton, California. For our use we received the following:

1. Hunter Square Conceptual Site Diagram, delivered electronically via email by NBBJ on January 16, 2009.

2. New Stockton Courthouse, Draft EIR, Tetra Tech EM Inc., dated January 23, 2009.

3. Phase One Environmental Site Assessment, New Stockton Courthouse, Hunter Square, Stockton California, Earth Tech, dated January 31, 2008.

1.1 SCOPE OF SERVICES

ENGEO prepared this report as outlined in our agreement dated November 18, 2008. NBBJ authorized ENGEO to conduct the proposed scope of services, which included the following:

- Service Plan Development
- Subsurface Field Exploration
- Soil Laboratory Testing
- Data Analysis and Conclusions
- Report Preparation.

1.2 PROJECT LOCATION

Figure 1 displays a Site Vicinity Map. The site is located in Hunter Square, which is south of East Weber Boulevard and northwest of the intersection of Main Street and South Hunter Street in downtown Stockton, California. Hunter Square is surrounded by multi-story buildings including the existing San Joaquin County Courthouse to the east and is currently used as a parking lot with landscaped areas.

Figure 2 shows the currently proposed building footprint and our exploratory locations.

1.3 PROJECT DESCRIPTION

Based on our discussion with NBBJ and review of the information provided, we understand that site improvements will consist of the following:

1. A 10 to 13-story courthouse building with a footprint covering approximately 80 percent of the 55,000 square-foot property.

2. A partial basement with an anticipated depth of between 4 to 18 feet below grade.

3. Underground utilities

4. Paved entry and exit drives

5. Below grade retaining walls

6. Exterior concrete flatwork and landscaping areas

Details regarding site grading have yet to be determined. The precise building perimeter, interior column spacings and bay widths have not yet been developed. Additionally, structural column loads and foundation layout are still to be developed.

2.0 FINDINGS

2.1 SURFACE CONDITIONS

The majority of the site is covered with an asphalt concrete parking lot and concrete and brick flatwork. A water fountain and a shallow concrete lined pond is located in the southern portion of Hunter Square. The pond is approximately 65 feet wide, 85 feet long, and 1 to 2 feet deep. Irrigated lawn and landscape areas cover the remainder of the site.

Topography at the site is generally flat with some minor variations in the landscaped areas and a low area in the vicinity of the existing water fountain. According to USGS topographic maps, elevation of the site is approximately +15 feet (Datum: 0 feet = Mean Sea Level).

2.2 SITE HISTORY

According the project draft EIR, the site has never contained any permanent structures aside from the existing water fountain and was historically used as a town square. The draft EIR describes historic Hunter Street reclaiming an old slough in the 1800's. The slough is described as being west of the historic courthouse, likely in the eastern portion of Hunter Square. The current San Joaquin County Courthouse is now in the site of the historic courthouse, immediately to the east of Hunter Square.

Aerial photographs in the Phase One Environmental Site Assessment (ESA) date as far back as 1957. The 1957 photographs show Hunter Square as a completely paved parking lot with Hunter Street extending through the site, just west of the historic courthouse. The ESA also includes topographic maps and Sanborn maps dating back as far as 1913 and 1895 respectively; these maps show the site developed and provide no additional information regarding pre-development conditions.

2.3 SEISMIC SETTING

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site, therefore, is not anticipated.

The site does lie within a seismically active region. According to a search using the software program EQFAULT Version 3.00b (Blake, 2000), the nearest active fault is the Great Valley Fault System, which is mapped approximately 20 to 27 miles west of the site. Faults in the system are considered capable of a moment magnitude earthquake of as great as 6.7. Other active faults in the region include the Mount Diablo system approximately 27 miles away, capable of a moment magnitude of 6.7 and the Greenville Fault approximately 28 miles away capable of a moment magnitude of 6.7. See Figure 4 for regional faulting and seismicity relative to the site.

2.4 SITE GEOLOGY

The site is located in the Great Valley geomorphic province. The Great Valley is an elongate, northwest trending structural trough bound by the Coast Range on the west and the Sierra Nevada on the east. The Great Valley has been and is presently being filled with sediments primarily derived from the Sierra Nevada.

Our site reconnaissance and referenced geologic maps (Section 11) indicate that the underlying geologic formation at the site is the Quaternary Great Valley Basin Deposits. Basin deposits are generally composed of sediments deposited during flood stages of major streams in the area between natural stream levees and alluvial fans.

2.5 SUBSURFACE CONDITIONS

We visited the site on February 26, 27, and March 4, 2009 to perform our site exploration. We observed drilling of two mud rotary borings to depths of 101.5 feet and we observed six Cone Penetration Tests (CPT soundings) to depths of 50 to 75 feet. Additionally, we logged two hand auger explorations to depths of 5 and 7 feet. Exploration locations are shown on the Site Plan, Figure 2. Figure 4 displays a fence diagram with borings and CPTs as well as interpreted lithology across the site.

In general, the explorations encountered sandy or gravelly fill to depths ranging from approximately 4 to 7 feet. In hand augers HA-1 and HA-2, various metal and brick debris were encountered from 3 to 3½ feet. In HA-2, a void was encountered from 3½ to 7 feet. We could not find any documentation regarding this void, or its extents, in our review of the referenced EIR or ESA documents. Beneath the fill, we typically encountered medium stiff to hard silts and clays interlayered with fine to medium grained sand and silty sand.

Sand layers observed were typically medium dense to very dense and between 3 and 10 feet thick. However, two medium dense to loose sand layers approximately 3 to 5 feet thick were observed in boring B-1.

The historic slough discussed in Section 2.2 may be located in the eastern portion of the site. However, we found no conclusive evidence that we encountered the slough in our subsurface exploration.

Consult the Site Plan, boring logs, and CPT plots for specific soil and groundwater conditions at each location. We include our boring logs and CPT plots in Appendix A. The boring logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. Appendix A also provides additional exploratory information in the general notes to the logs.

2.6 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine their engineering properties. For this project, we performed moisture content, dry density, unconfined compression, direct shear, plasticity index, hydrometer, resistance value, and soil corrosion potential testing. Moisture contents and dry densities are recorded on the boring logs in Appendix A. All other laboratory data is included in Appendix B.

2.7 GROUNDWATER CONDITIONS

We estimated static groundwater in several of our CPT explorations based on pore pressure dissipation tests. The two deep borings were advanced with mud rotary boring methods; therefore groundwater could not be directly measured at those exploration locations. Interpreted depths to groundwater, based on dissipation tests performed during the advancement of the CPT, are tabulated below:

Table 1
Groundwater Depths

CPT Location	Approximate Depth to Groundwater (ft.)
CPT-1	25.5
CPT-2	24.0
CPT-3	22.2
CPT-4	22.5
CPT-6	24.0

We did not run dissipation tests in CPT-5. Fluctuations in the level of groundwater may occur due to variations in rainfall and other factors not evident at the time measurements were made.

Historic California Department of Water Resources online groundwater data (well number 01N06E11E002M) within the vicinity of the property shows groundwater has been as shallow as 24 to 69 feet below the ground surface (elevation -10 feet to elevation -55 feet). Other ENGEO geotechnical reports in the downtown Stockton area also show groundwater at approximately elevation -5 to -10 feet.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed courthouse provided our recommendations are incorporated into the design. We summarize our primary conclusions below.

3.1 FOUNDATION SUPPORT

Due to the anticipated high column loads, it is our opinion that the building may be supported on deep foundations deriving their support through predominantly skin friction in the underlying soil. Our subsurface explorations did not identify any notably thick granular layers for predominant end bearing support. We provide recommendations for driven concrete piles and drilled piers. Alternate deep foundations, such as augercast, tubex, or fundex piles can also be considered.

3.2 SEISMICITY AND GROUND MOTION ANALYSIS

3.2.1 Seismic Hazards

Potential seismic hazards resulting from regional moderate to large earthquakes include surface rupture, ground shaking, and liquefaction. We summarize these seismic hazards below.

3.2.1.1 Surface Rupture

The site is not located within a State of California designated Alquist-Priolo Earthquake Fault Zone and no active faults are mapped across the site. Based on this information, it is our opinion that the potential for the occurrence surface rupture is low.

3.2.1.2 Ground Shaking

Ground shaking is a potential seismic hazard for future development on the subject property. As discussed above, regional earthquakes of moderate to large magnitude may occur during the design life of the building, and these events may cause moderate to severe ground shaking at the site.

Building design should account for the potential ground shaking associated with these faults. To mitigate the ground shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum.

3.2.1.3 Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soil is subject to a temporary, but essentially total loss of shear strength because of pore pressure buildup under the reversing cyclic shear stresses associated with earthquakes. The potential for liquefaction depends on the actual depth to groundwater at the site, the density of the underlying soil, and the potential level of ground shaking.

As discussed in Section 2.5, Subsurface Conditions, the borings and CPTs encountered interbedded sands and fine-grained material to the maximum depth of our explorations. Within shallow sand layers, relatively thin zones of loose, granular materials were encountered. In addition, the CPT's encountered groundwater at depths ranging from approximately 22 to 25½ feet below grade.

Because the building is to be supported on deep foundations, and given the relatively thick capping layer of fine grained materials overlying potentially liquefiable layers, it is or opinion that risks of ground deformation due to liquefaction are low.

3.2.2 Site Specific Ground Motion Analysis

We performed a seismic hazard analyses for the project site based on the latest California Building Code (CBC, 2007) Chapter 16A. The CBC references the American Society of Civil Engineers Standard 7-05 (ASCE 7-05) Chapter 21 as the basis for developing "Site Specific Ground Motion Procedures for Seismic Design". For this analysis we used the computer program EZ-FRISK and current California fault data to model the seismic setting of the region. This procedure accounts for the following:

- Earthquake magnitude
- Rupture length
- Location of rupture
- Maximum possible earthquake magnitude
- Recurrence interval of earthquake events

We conducted probabilistic and deterministic analyses to develop the spectra shown in Figure 5. The probabilistic and deterministic spectra were derived using the Next Generation Attenuation relationships (NGA) developed by Boore-Atkinson (2007), Campbell-Bozorgnia (2008), Abrahamson and Silva (2008) and Chiou and Youngs (2008). The probabilistic analysis considered a 2 percent probability of exceedance within a 50-year period.

We calculated the deterministic response acceleration at each period by selecting the largest median spectral response acceleration for characteristic earthquakes within a 100 km (62 miles) radius and multiplying it by 1.5 (150 percent). The largest median spectral response acceleration was calculated from an earthquake on the Great Valley 7 Fault with a Moment Magnitude of 6.8 and at a distance of 33 km (20.5 miles) from the subject site. The 2 percent in 50 years probabilistic Maximum Credible Earthquake (MCE) spectrum is shown in blue and the 150 percent deterministic spectrum is shown in yellow on Figure 5. ASCE 7-05 Section 21.2.2 defines a "Deterministic Lower Limit on the MCE Response Spectrum", which is shown in brown on Figure 5. ASCE 7-05 recommends that the site-specific MCE response spectrum be taken as the lesser of the spectral response spectrum from the probabilistic analysis and the deterministic spectrum, subject to the deterministic spectrum lower limit. As a result, probabilistic spectrum is the governing MCE spectrum for the site. Figure 5 also shows the MCE Spectrum using the mapped values in magenta color.

The CBC, by reference to ASCE 7-05, states that the "Design Spectrum" be computed by taking two-thirds of the governing MCE spectrum, which in this case is the probabilistic spectrum, but cannot be less than 80 percent of the Design Response Spectrum using the mapped values. The recommended "Design Spectrum" is plotted in dark green and the Design Spectrum using the mapped values is plotted in light blue on Figure 5. Below is the table with the numeric values of the Spectra and ground motions for the short period and long period at the site.

Table 2
Spectra and Ground Motions

Coefficient	Value
Site Class	D
Design, 5% Damped, Spectral Response Acceleration from Site Specific Analysis at Short Periods, S_{DS}	0.526
Design, 5% Damped, Spectral Response Acceleration from Site Specific Analysis at a Period of 1 second, S_{D1}	0.285

3.3 EXISTING FILL

Our explorations indicate that the majority of the site is underlain by fill, which ranged from 4 to 7 feet thick at the exploration locations. It is our opinion that the existing fill is not suitable for supporting surface improvements or any structures supported by shallow foundations. If necessary for the fill to support first-floor slabs-on-grade or other lightly loaded appurtenances, then the fill will have to be removed and recompacted. We present fill removal recommendations in Section 5.

3.4 FUTURE EXPLORATION OF SUBSURFACE VOID

As discussed in the subsurface section of this report, we encountered an unexpected void between a depth of 3½ and 7 feet below grade at the location of HA-2 shown on the site plan. We were not able to define the limits of this feature during our site work, and were not able to find any reference to an underground tank, cistern, vault, or other features during our document review, which included the ESA and draft EIR for the site.

Before construction activities begin, we recommend additional exploration in the vicinity to HA-2 to determine the extent of this void and help to develop alternatives for proper removal and or backfill of the area. Depending on the nature of the feature, permits may be required through local public agencies.

To gather more data about the void, a non-destructive exploration approach would be ground penetrating radar (GPR). GPR is a geophysical method that uses radar pulses to image the subsurface. GPR can be used in a variety of media, including rock, soil, pavements and structures and can detect objects, changes in material type, and voids. Though this non-invasive method can give some idea of what lies below grade, physical exploration will likely be necessary to precisely characterize the void or other subsurface anomaly.

Another alternative to investigate the subsurface anomaly would be to excavate the area using a backhoe or excavator. Because the lateral extents of the void are not known, care should be taken to avoid driving heavy equipment on top of the void to reduce the chance for a cave in. Also, existing utilities are in close proximity to the void; we recommend the utility company be onsite during excavation or the utilities be abandoned prior to excavation. Any resulting depressions left after demolition or removal of these anomalies should be backfilled and compacted with engineered fill in accordance with Section 5.6 of the this report.

3.5 HISTORIC SLOUGH

As referenced in Section 2.2, the draft EIR describes a historic slough west of the current courthouse site, perhaps in the eastern portion of Hunter Square. A historic water channel, like a slough, may have contained very soft, fine grained, soil that could undergo settlement under the addition of increased loading from new construction.

We did not encounter evidence of the slough in any of our explorations; however evidence of the slough may be uncovered during construction. Further exploration should be considered prior to construction in an effort to locate and characterize the slough. This could be done by developing a grid pattern across the site and excavating test pits or "pot holes" using a backhoe or excavator. Depending on the location of the slough, if encountered at all, various mitigation measures could be considered including excavation of soft soil and replacement with engineered fill. The slough would have not impact on the design of the pile supported building but could cause difficulties with temporary excavations.

3.6 EXPANSIVE SOIL

We observed potentially expansive lean clay in both borings B-1 and B-2. Our laboratory testing indicates that these soils exhibit moderate shrink/swell potential with variations in moisture content. Expansive soil can cause distress to lightly-loaded foundations, floor slabs, pavements, sidewalks, and other improvements that are sensitive to soil movements. We present expansive soil mitigation recommendations in Section 5.2 of this report.

3.7 SOIL CORROSION POTENTIAL

We submitted select soil samples to an analytical lab for determination of pH, resistivity, sulfate, and chloride. The sulfate lab test results indicate the sulfate exposure may be categorized as “Moderate” in accordance with Table 19-A-4 of the California Building Code. For “Moderate” sulfate exposure, the CBC indicates that Type II Portland Cement may be used with a water/cement ratio of 0.50 for project concrete mix designs.

The samples tested had low resistivities, indicating that they are moderately to highly corrosive to buried metal.

If desired to investigate this further, we recommend a corrosion consultant be retained to determine if specific corrosion recommendations are necessary for the project. We present the analytical lab test results in Appendix B.

3.8 STATIC AND PERCHED GROUNDWATER

Since the Courthouse basement is to be approximately 4 to 18 feet deep, it does not appear that the groundwater level (22 to 25½ feet) will affect the proposed development. As discussed in Section 2.7, historic California Department of Water Resources groundwater level data within the vicinity of the property shows groundwater has been as shallow as elevation -10 feet, or approximately 25 feet below the ground surface at the site. Other ENGEO geotechnical reports in the downtown Stockton area show similar groundwater elevations of approximate elevation -5 to -10 feet.

Based on the historic groundwater data we reviewed and the currently proposed basement depth, we do not anticipate that a permanent dewatering system will be necessary beneath the basement slab. If the basement depth increases, permanent dewatering systems may be necessary.

4.0 FOUNDATION RECOMMENDATIONS

We recommend that the proposed courthouse be supported on driven concrete piles or cast-in-place, concrete drilled piers. These deep foundations will derive their vertical capacity primarily from skin friction within the stiff to hard clay layers encountered during our subsurface exploration. We developed pile and pier capacities based on an assumed pile cap bottom at approximately 10 feet below the current site grade.

Precast, prestressed concrete piles are typically 12- or 14-inch square. Drilled piers can be installed in various sizes, although common dimensions are typically 24- or 36-inch diameter. We provide vertical and lateral capacities for these typical dimensions.

4.1 VERTICAL CAPACITIES

Based on the mud rotary borings, CPT soundings and laboratory testing, we recommend the following design criteria for deep foundations.

Table 3
Allowable Vertical Downward Capacity and Pile Lengths
Driven, Precast Concrete Piles

Pile Type	Minimum Pile Length* (feet) for 100 ton Capacity	Minimum Pile Length* (feet) for 125 ton Capacity
12-inch square	60	73
14-inch square	52	64

*Based on an approximate pile top elevation of +5 feet (10 feet below current grade)

Table 4
Allowable Vertical Downward Capacity and Pier Lengths
Drilled, Cast in Place Piers

Pier Type	Minimum Pier Length* (feet) for 150 ton Capacity
24-inch diameter	63
36-inch diameter	44

*Based on an approximate pile top elevation of +5 feet (10 feet below current grade)

Increase the above downward capacities by one-third for the short-term effects of wind or seismic loading. To reduce pile group effects, space piles and piers at least 3 diameters apart, center to center. For square piles, use the least dimension for determining the effective diameter.

Structural loads and the number and configuration of pile groups are not known at this time. On a preliminary basis, we estimate that post construction pile foundation settlements will be less than 1 inch. Differential settlement between adjacent columns will be dependent on the final design of these foundation elements, although we anticipate that differential settlement will be less than about ¾ to ½ inch between columns. Once column spacings, loads and pile group configurations are determined, we should be retained to review the design and update our settlement estimates.

Allowable resistance to vertical uplift can be determined by taking two-thirds of the allowable vertical downward capacities presented in Tables 3 and 4. Increase the above uplift capacities by one-third for the short-term effects of wind or seismic loading provided piles are spaced at least 3 pile diameters or more on center. Capacity reduction for uplift on pile groups is not considered necessary.

4.2 LATERAL CAPACITIES

4.2.1 Single Pile Capacity

Lateral load resistance for pile-supported structures is developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon several factors including pile stiffness, pile embedment length, conditions of fixity at the pile cap, the physical properties of surrounding soil, and the magnitude of lateral deflections.

We used the computer program LPILE to estimate lateral pile loads for ¼- and ½-inch pile top deflections. Lateral capacities and deflection characteristics were calculated using pile stiffness (EI) of 7.35×10^9 and 1.37×10^{10} pound-inch^2 for 12- and 14-inch-square concrete piles, respectively. We also assumed a minimum 28-day compressive strength of 6,000 pounds per square inch (psi) for pile concrete and a cross sectional area defined by Santa Fe-Pomeroy, Inc for prestressed, precast concrete piles. Later characteristics for drilled piers were calculated using pier stiffness (EI) of 5.08×10^{10} and 2.57×10^{11} pound-inch^2 for 24- and 36-inch diameter concrete piers with an assumed 28-day concrete compressive strength of 3,000 psi. If pile stiffness varies by no more than 20 percent of that reported above, then load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness. For pile stiffness significantly different from the values listed above, we should be contacted to provide revised lateral pile characteristics.

Table 5
Allowable Lateral Capacities (Single 12-inch square pile)

Allowable Lateral Capacity (Kips)		
Pile Condition	¼-inch Deflection	½-inch Deflection
Free Head	8.5	12.1
Fixed Head	18.0	26.1

Table 6
Allowable Lateral Capacities (Single 14-inch square pile)

Allowable Lateral Capacity (Kips)		
Pile Condition	¼-inch Deflection	½-inch Deflection
Free Head	10.9	15.7
Fixed Head	22.9	33.3

Table 7
Allowable Lateral Capacities (Single 24-inch diameter pier)

Allowable Lateral Capacity (Kips)		
Pile Condition	**¼-inch Deflection**	**½-inch Deflection**
Free Head	19.3	27.9
Fixed Head	39.9	58.2

Table 8
Allowable Lateral Capacities (Single 36-inch diameter pier)

Allowable Lateral Capacity (Kips)		
Pile Condition	**¼-inch Deflection**	**½-inch Deflection**
Free Head	36.7	53.3
Fixed Head	75.2	111.4

The above lateral capacities represent the probable response of a <u>single</u> pile or pier under short term loading conditions and do not include a factor of safety. Suitable factors of safety should be selected based on the type of loading.

We estimated maximum bending moments and points of fixity for both driven piles and drilled piers for ¼- and ½-inch pile top deflection for both fixed and free head conditions. As referenced in the tables below, "point of fixity" is defined as a point of zero lateral deflection. We present the results in Tables 9 and 10 below:

Table 9
Load Deflection Characteristics
Precast Driven Piles

Pile Type	Deflection Characteristic	Pile Deflection Free Head		Pile Deflection Fixed Head	
		¼-inch	½-inch	¼-inch	½-inch
12-inch square	Maximum Bending Moment (in-kips)	308	504	791	1286
	*Depth to Maximum Bending Moment (feet)	5.2	5.8	0	0
	*1st Point of Fixity (feet)	9.5	10.0	12.7	13.7
	*2nd Point of Fixity (feet)	16.9	19.0	19.5	22.7
14-inch square	Maximum Bending Moment (in-kips)	440	720	1142	1856
	*Depth to Maximum Bending Moment (feet)	6.3	6.9	0	0
	*1st Point of Fixity (feet)	10.6	11.6	14.3	15.3
	*2nd Point of Fixity (feet)	19.0	21.6	24.3	27.0

*Below Top of Pile

Table 10
Load Deflection Characteristics
Cast in place Drilled Piers

Pile Type	Deflection Characteristic	Pile Deflection Free Head		Pile Deflection Fixed Head	
		¼-inch	½-inch	¼-inch	½-inch
24-inch Diameter Pier	Maximum Bending Moment (in-kips)	970	1580	2532	4122
	*Depth to Maximum Bending Moment (feet)	7.9	8.4	0	0
	*1st Point of Fixity (feet)	13.7	15.3	18.0	19.6
	*2nd Point of Fixity (feet)	29.0	32.2	33.3	33.9
36-inch Diameter Pier	Maximum Bending Moment (in-kips)	2515	4127	6610	10940
	*Depth to Maximum Bending Moment (feet)	10.7	12.1	0	0
	*1st Point of Fixity (feet)	18.4	20.2	26.1	28.3
	*2nd Point of Fixity (feet)	35.1	35.5	36.4	37.7

*Below Top of Pile

4.2.2 Group Capacity Reduction

Research has shown that the lateral capacity of a group of piles is generally less than that of a single pile for pile spacings less than 6 to 8 pile diameters. For pile groups with a minimum spacing of 3 pile diameters, we recommend reducing the single pile allowable lateral capacities by the percentages in the following table.

Table 11
Group Reduction Percentages

Number of Piles in Group	Percentage to Reduce single Pile Capacity By
2	25
4	30
9	43
16	48
25	54

Please contact us if group reduction percentages are needed for additional pile group configurations.

4.3 PASSIVE RESISTANCE AGAINST PILE CAPS AND GRADE BEAMS

Lateral loads may also be resisted by passive pressure along the sides of pile caps and grade beams where poured neatly against undisturbed native soil or newly constructed engineered fill. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend an allowable passive lateral pressure of 175 pcf, given as an equivalent fluid weight, for use in design. This value includes a factor of safety of 1.5.

4.4 DRIVEN PILE INSTALLATION AND TESTING

4.4.1 Indicator Piles

We recommend that indicator piles be driven to:

- Develop production driving criteria
- More accurately estimate production pile lengths that will vary based on driving resistance and depth to various soil layers
- Determine appropriate predrill depth, if required
- Evaluate the contractors pile driving system

Both the indicator and production piles should be driven with the same pile driving system. The hammer should be capable of delivering a minimum rated driving energy of approximately 70,000 foot-pounds. We recommend that the contractor perform a wave equation analysis to confirm the compatibility and drivability of the pile driving system with the pile type and soil conditions onsite. The wave equation analysis will also help confirm that the pile driving stresses will not exceed the allowable pile stresses. We should review the wave equation results prior to mobilization of pile driving equipment to the site.

Indicator piles should be cast at least 5 feet longer than needed to confirm field pile capacities and final design lengths.

Indicator piles may be driven as production piles provided that minimum recommended tip elevations are achieved and no structural damage occurs to the pile from installation. However, optional indicator locations may be preferable to allow for evaluation of predrill depths and other factors.

Once foundation plans are finalized, including the number and layout, we should be retained to prepare an indicator pile program for the project.

4.4.2 Pile Load Tests

We recommend that the vertical allowable downward capacity be evaluated by performing a pile load test prior to production pile installation.

The load test should be performed in accordance with ASTM D1143 (Reapproved 1994) *Standard Test Method for Piles Under Static Axial Compressive Load, Standard Loading Procedure*. The Standard Loading Procedure requires loading up to 200 percent of the design load. Because testing a pile to failure can provide the best information for determining actual capacities, we recommend that additional loading be performed if the pile does not fail under 200 percent of the design load. In this case, we recommend that Section 5.1 of ASTM D1143 be performed, *Loading in Excess of Standard Test Load*, and the maximum load be increased to 300 percent of design load.

An optional uplift capacity load test may be performed in if uplift pile capacity is a significant component of the pile design. This determination should be made once more detailed structural loading conditions are available.

Test piles should be driven using the same hammer as that used for indicator and production pile driving. Prior to test pile installation, we will review the indicator pile driving logs and the wave equation analysis to select the appropriate tip elevations.

The contractor is responsible for the design, operation, and safety of the load test system. This includes supplying and installing all of the necessary components including the dial gauges and reference beams.

We should be retained to review the load test program prior to mobilization of pile test equipment to the site. We should also be retained to monitor and evaluate the entire pile load test, including test pile installation.

Load test piles should not be used as production piles. It may be feasible to use at least one of the indicator piles for the load test reaction piles.

4.4.3 Production Pile Installation

Following our analysis of the indicator pile installation and load testing, we should be retained to establish the minimum pile lengths necessary to achieve the desired pile capacities.

Production piles should be driven using the same hammer and system as the indicator and load test piles. We will use data obtained from the indicator pile program, load tests, wave equation analysis, and this geotechnical report to develop pile driving criteria for production piles.

We should be retained to observe and record the results of all production pile driving.

4.5 DRILLED PIER CONSTRUCTION

We anticipate that drilled piers will extend below the groundwater table and will therefore require placing concrete in the wet. The bottoms of drilled pier excavations should be reasonably clean and free of loose soil before reinforcing steel is installed and concrete is placed. Concrete will need to be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete at all times to avoid entrapment of water in the concrete. Concrete should have a minimum compressive strength of 3,000 psi.

Portions of the subsurface profile are indicated to include relatively clean sands that can be prone to caving. The contractor should anticipate that drilled pier excavations will require casing.

ENGEO should be on-site during drilled pier excavation to observe soil conditions encountered across the site for comparison with the soil conditions observed during our subsurface exploration. Additionally we will monitor concrete pump volumes to determine if any voids developed during excavation of the pier shaft or during casing removal.

4.6 INFLUENCE OF NEW FOUNDATIONS ON EXISTING STRUCTURES

At the time this report was prepared, details of structural loading, foundation layout, and basement footprint and depth have not been determined. The final design of these aspects of the project, as well as construction methods used, may impact existing structures adjacent to the site. Both deep excavations in close proximity to existing buildings and vibrations from pile driving can create distress to existing improvements. Depending on the proximity and depth of the basement excavation relative to existing structures, special earth retaining systems or ground improvement measures may been needed to mitigate lateral and/or vertical deflections and other distress to existing improvements. These mitigation measures could include the following:

- Underpinning existing structures
- Cast in place tangent piers
- Jet grouting
- Structural slurry wall

When properly designed and constructed, these measures can provide the necessary retention and minimize deflections to existing improvement during and following construction.

5.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal *flexing* or *pumping,* as determined by an ENGEO representative.

As used in this report, the term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define "structural areas" in Section 4 of this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 EXISTING FILL REMOVAL

The existing fill at the site is not suitable for support of improvements. If desired to construct first-floor slabs-on-grade or other lightly loaded shallow footings on the existing fill, then the fill will have to be removed and recompacted. The lateral extent and depth of fill is expected to vary. Fill depths are discussed in Sections 2.5 and 3.3. Fill may be reused as engineered fill provide it meets the recommendations in Section 5.5. Place and compact fill in accordance with Section 5.6.

If first-floor or basement slabs are designed to structurally span between pile caps, then there is no need to remove and recompact existing fill. In addition, basement excavations may fully remove any existing fill.

5.2 EXPANSIVE SOIL MITIGATION

We recommend that slabs-on-grade, pavements, and exterior flatwork be underlain by a minimum of 18-inches of non-expansive fill.

Expansive soil should not be used as backfill for below grade retaining walls.

We also provide specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a higher moisture content and controlling the amount of compaction.

5.3 GENERAL SITE CLEARING

Clear improvement areas of all surface and subsurface deleterious materials including existing foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 5.6. ENGEO should be retained to observe and test all backfilling.

See Section 3.4 regarding a subsurface void encountered in HA-2.

5.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring, during or following periods of rain, and at depths below the surface where evaporation is limited. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather;
2. Mixing with drier materials;
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by ENGEO prior to implementation.

5.5 ACCEPTABLE FILL

On-site soil material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 6 inches in maximum dimension. Also, as discussed in Section 5.2, expansive soil should not be placed in the upper 18 inches of areas with slabs-on-grade, pavements, and flatwork.

Imported fill materials should meet the above requirements and have a plasticity index less than 12. Allow ENGEO to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

5.6 FILL COMPACTION

5.6.1 Grading in Structural Areas

Non-expansive soil

Perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches;

2. Moisture condition soil to at least 1 percentage point above the optimum moisture content; **and**

3. Compact the subgrade to at least 90 percent relative compaction. Compact the upper 6-inches of finish pavement subgrade to at least 95 percent relative compaction prior to aggregate base placement.

After the subgrade soil has been compacted, place and compact acceptable fill (defined in Section 5.5) as follows:

1. Spread fill in loose lifts that do not exceed 8 inches;

2. Moisture condition lifts to slightly above the optimum moisture content; **and**

3. Compact fill to a minimum of 90 percent relative compaction; Compact the upper 6 inches of fill in pavement areas to 95 percent relative compaction prior to aggregate base placement.

Expansive Soil

Perform subgrade compaction prior to fill placement and following cutting operations.

1. Scarify to a depth of at least 8 inches;

2. Moisture condition soil to at least 3 percentage points over the optimum moisture content; **and**

3. Compact the soil to between 87 and 92 percent relative compaction.

After the subgrade has been compacted, place and compact acceptable fill (defined in Section 5.5) as follows:

1. Spread fill in loose lifts that do not exceed 8 inches;

2. Moisture condition lifts to at least 3 percentage points over the optimum moisture content; **and**

3. Compact fill to between 87 and 92 percent relative compaction.

Subgrade processing is not required where cemented soil is exposed, as determined by ENGEO's field representative.

Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

5.7 UNDERGROUND UTILITY BACKFILL

5.7.1 General

The contractor is responsible for conducting all trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe bedding materials.

5.7.2 Structural Areas

Non-expansive Soil

Place and compact trench backfill as follows:

1. Trench backfill should have a maximum particle size of 6 inches;

2. Moisture condition trench backfill to or slightly above the optimum moisture content. Moisture condition backfill outside the trench;

3. Place fill in loose lifts not exceeding 12 inches;

4. Compact fill to a minimum of 90 percent relative compaction (ASTM D1557).

Expansive Soil

Place and compact trench backfill as follows:

1. Trench backfill should have a maximum particle size of 6 inches;

2. Moisture condition trench backfill to at least 3 percentage points over the optimum moisture content. Moisture condition backfill outside the trench;

3. Place fill in loose lifts not exceeding 12 inches;

4. Compact fill to between 87 and 92 percent relative compaction (90 percent minimum relative compaction at depths of 3 feet or more below finish grades).

Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

5.8 LANDSCAPE FILL

Process, place and compact fill in accordance with Sections 5.6, except compact to at least 85 percent relative compaction (ASTM D1557).

5.9 SLOPES GRADIENTS

Construct final slope gradients to 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

5.10 BRACED EXCAVATIONS

Design excavation bracing to resist lateral earth pressure from adjoining material and from any surcharge loads such has traffic loading or loading from existing construction. The design criteria for subsurface excavation bracing are presented below.

Table 12
Recommended Lateral Earth Pressure

Earth Pressure	Equivalent Fluid Density, Drained Condition (pcf)
Active	50
At-Rest	80
Passive	300

The above lateral earth pressures assume level backfill conditions and no surcharge loading. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf).

The choice of shoring should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. Support of adjacent structures and utilities without distress is the contractor's responsibility. We recommend that ENGEO review the contractor's plan for the excavation bracing prior to construction.

5.11 SITE DRAINAGE

5.11.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we provide the following minimum recommendation for surface drainage.

1. Slope pavement areas a minimum of 1 percent towards drop inlets or other surface drainage devices.

2. Slope finished grade away from building exteriors at a minimum of 2 percent for a distance of at least 5 feet.

5.11.2 Subsurface Drainage

Although groundwater level is expected to be below the new basement, we recommend a subsurface drainage system to account for perched water and the possibility of future shallower groundwater depths. A permanent subdrain system should be designed below the basement slab.

At a minimum the subdrain system should consist of:

1. A minimum 18-inch-thick layer of washed, crushed rock below the basement slab. Crushed rock should consist of 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Place a nonwoven geotextile filter fabric such as Mirafi 140NC, or equivalent below the rock.

2. Place 4-inch-diameter perforated pipe within the rock layer at a minimum 25 foot lineal spacing. Place pipes with perforations placed down, approximately 4 inches from the bottom of the rock layer. Slope pipes at least 1 percent toward a central collector pump system.

3. Remove collected water with a suitable collector pump system.

4. Construct cleanouts for drain maintenance.

We should be retained to review the subdrainage system prior to construction.

Even with a subdrainage system, we recommend basement floors and walls be fully waterproofed to reduce vapor moisture intrusion.

6.0 SLABS-ON-GRADE

6.1 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. As discussed in Section 5.1, old fill is not suitable to support exterior flatwork. Fill should be placed in accordance with Sections 5.6 and 5.7. Additionally, exterior flatwork should be supported on a minimum of 18 inches of non-expansive material as described in Section 5.2.

Provide a minimum concrete flatwork thickness of 4 inches.

Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

6.2 BASEMENT FLOOR SLAB

6.2.1 Minimum Design Section

We anticipate the basement floor slabs will be a traditional slab-on-grade and not a "structural slab" spanning between pile caps. It is our opinion that with the construction of the subdrain system outlined in Section 5.11.2, a slab-on-grade will be adequate for the proposed courthouse. We recommend the following <u>minimum</u> design for slab-on-grade construction:

1. Provide a minimum concrete thickness of 6 inches.

2. Place minimum reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab.

The structural engineer should provide final design thickness and additional reinforcement, as necessary, for the intended structural loads.

6.2.2 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab on-grade that includes a vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-97 "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs".

2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

6.3 TRENCH BACKFILL

Backfill and compact all trenches below building slabs-on-grade and to 5 feet laterally beyond any edge in accordance with Section 5.7.

7.0 BELOW GRADE RETAINING WALLS

7.1 LATERAL EARTH PRESSURES

Design below grade retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design walls restrained from movement at the top to resist an equivalent fluid pressure of 65 pounds per cubic foot (pcf). In addition, design restrained walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Design unrestrained retaining walls with adequate drainage to resist an equivalent fluid pressure of 40 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Construct a drainage system, as recommended in Section 7.3, to reduce hydrostatic forces behind the retaining wall.

7.2 SEISMIC LATERAL EARTH PRESSURE

Based on the subsurface conditions encountered during our geotechnical exploration and the site specific peak horizontal ground acceleration of 0.21g obtained from the recommended design spectrum on Figure 5, we developed seismic design parameters for retaining walls. For calculation of seismic loading on retaining walls, apply a resultant load of 4 H^2 acting on the wall at 0.6 H from the wall base if the wall is unrestrained. If the wall is restrained from movement at the top, the resultant load would be 12 H^2 acting at 0.55 H from the wall base. In these equations, the load is in pounds per foot of wall length, and the dimension H is the height of retained earth, in feet.

7.3 RETAINING WALL DRAINAGE

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-1.025) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelope rock in a nonwoven geotextile filter fabric such as Mirafi 140NC, or equivalent.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from the wall base to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

8.0 PAVEMENT DESIGN

8.1 FLEXIBLE PAVEMENTS

We obtained a representative bulk sample of the surface soil from the westerly portion of the property, within the existing parking area, and performed an R-value test to provide data for pavement design. The results of the test are included in Appendix B and indicate an R-value of 66, which we judged to be unrealistically high for this site. Based on surface soil variability, we judged an R-value of 40 to be applicable for design. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Procedure 608 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

Table 13
Recommended Asphalt Concrete Pavement Sections

	Section	
Traffic Index	Asphalt Concrete (in.)	Class 2 Aggregate Base (in.)
5	3	5
6	3	7
7	4	7

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

Because final grading for the courthouse will likely expose various soils at the pavement subgrade elevation, we should be retained to observe and evaluate soil conditions after grading to determine the applicability of the design R-value for flexible pavement design and make any necessary modifications.

8.2 RIGID PAVEMENTS

Use concrete pavement sections to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

8.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 5.6.1. Aggregate Base should meet the requirements for ¾ -inch maximum Class 2 AB per section 26-1.02a of the latest Caltrans Standard Specifications.

9.0 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to provide construction monitoring services as outlined below:

1. Retain ENGEO to review the final grading plans prior to construction to determine whether our recommendations have been implemented, and to provide additional or modified recommendations, if necessary.

2. Retain ENGEO to perform construction monitoring to check the validity of the assumptions we made to prepare this report. Our services would include testing and observation during site clearing, grading, foundation excavation and installation, pile driving operations, underground utility construction, and pavement subgrade and aggregate base compaction.

3. If any changes occur in the nature, design or location of the proposed improvements, then retain ENGEO to review the changes and prepare a written response and validate the conclusions and recommendations in this report.

4. If 3 years or more lapse between the time this report was prepared and construction, or if conditions have changed because of natural causes or construction operations on or near the site, then retain ENGEO to review this report for applicability to the new conditions. This report is applicable only for the project and site studied.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

10.0 LIMITATIONS

This report presents geotechnical recommendations for construction of improvements discussed in Section 1.3 for the Stockton Courthouse project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied.

We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of soil and groundwater conditions across the site. Considering possible underground variability of soil, existing fill, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

The locations of our explorations are approximate and were estimated by features shown on the site plan, Figure 2.

Our services did not include soil volume change factors or flood potential.

This geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, then notify the proper regulatory officials immediately.

11.0 REFERENCES

1. Hunter Square Conceptual Site Diagram, delivered electronically via email by NBBJ on January 16, 2009.

2. New Stockton Courthouse, Draft EIR, Tetra Tech EM Inc., dated January 23, 2009.

3. Phase One Environmental Site Assessment, New Stockton Courthouse, Hunter Square, Stockton California, Earth Tech, dated January 31, 2008.

4. Geologic Map of California San Jose Sheet, Division of Mines and Geology, T. H. Rodgers, 1966.

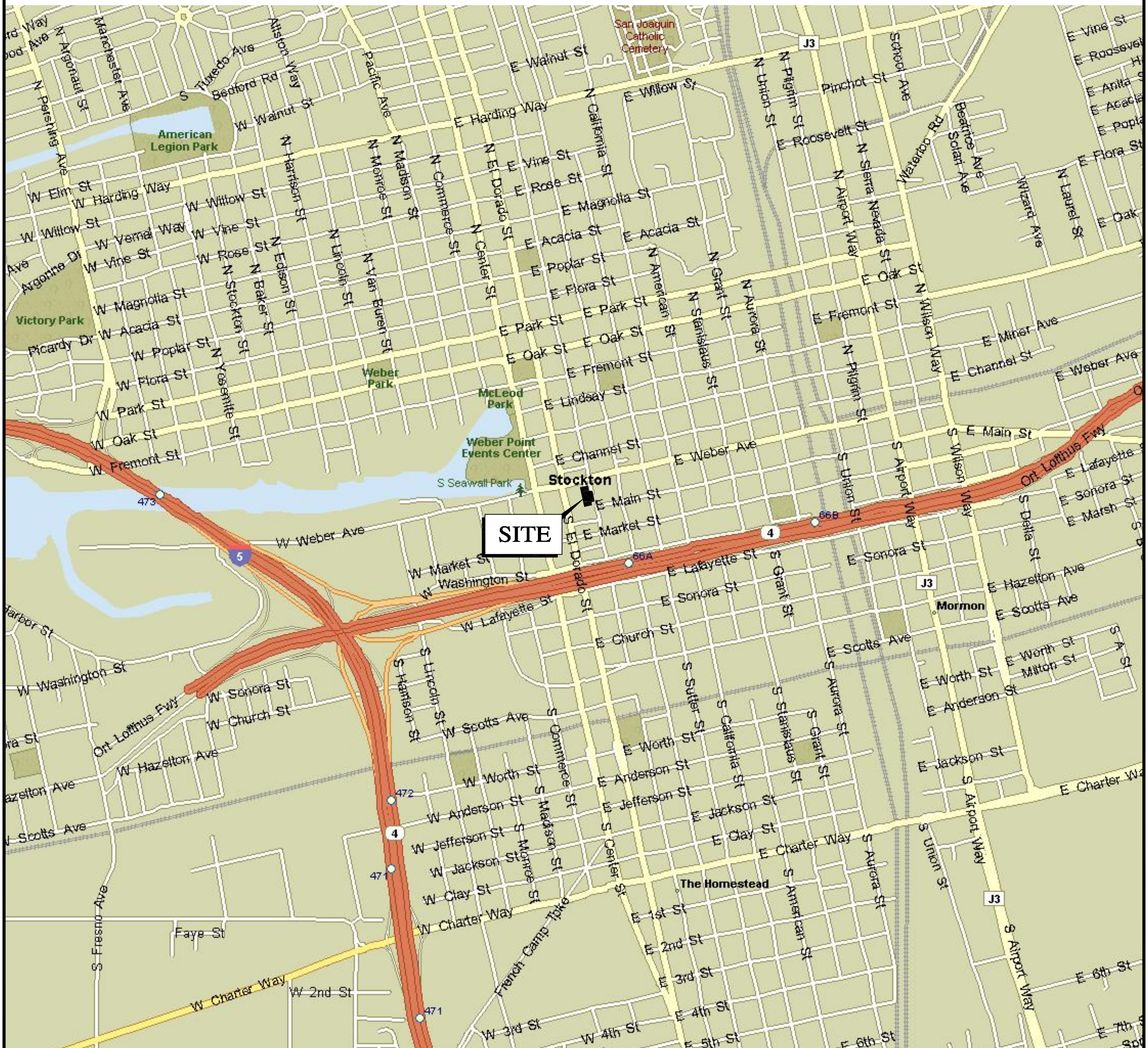
5. Preliminary Structure Contour Map of the Sacramento Valley, Showing Late Cenozoic Structural Features and Depth to Basement, Department of the Interior, United States Geologic Survey, D. S. Harwood and E. J. Helley, 1982.

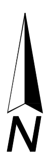
6. Geologic Map of San Joaquin County, Division of Mines, Olaf P. Jenkins, Chief, not dated.

7. Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions, Division of Mines and Geology, C. W. Jennings, 1994.

8. ENGEO Incorporated, Geotechnical Report, "Port of Stockton Primafuel Biodiesel Production Facility, Stockton California," Project No. 7896.5.001.01, dated August 30, 2007.

9. ENGEO Incorporated, Preliminary Geotechnical Report, "Stockton Waterfront Project, Stockton California," Project No. 6455.4.001.01, dated February 2, 2005.





0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: MS STREETS AND TRIPS

ENGEO Expect Excellence	VICINITY MAP STOCKTON COURTHOUSE STOCKTON, CALIFORNIA	PROJECT NO.: 8641.000.000 DATE: MARCH 2009 DRAWN BY: DLB CHECKED BY: MG	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg\8641\000\GEX\8641000000-GEX-1VicMap-0309.dwg 3-20-09 01:03:26 PM pcrockett

ORIGINAL FIGURE PRINTED IN COLOR

EXPLANATION

B-2 APPROXIMATE LOCATION OF BORING

CPT-6 APPROXIMATE LOCATION OF CONE PENETRATION TEST

HA-2 APPROXIMATE LOCATION OF HAND AUGER

0 FEET 50
0 METERS 25

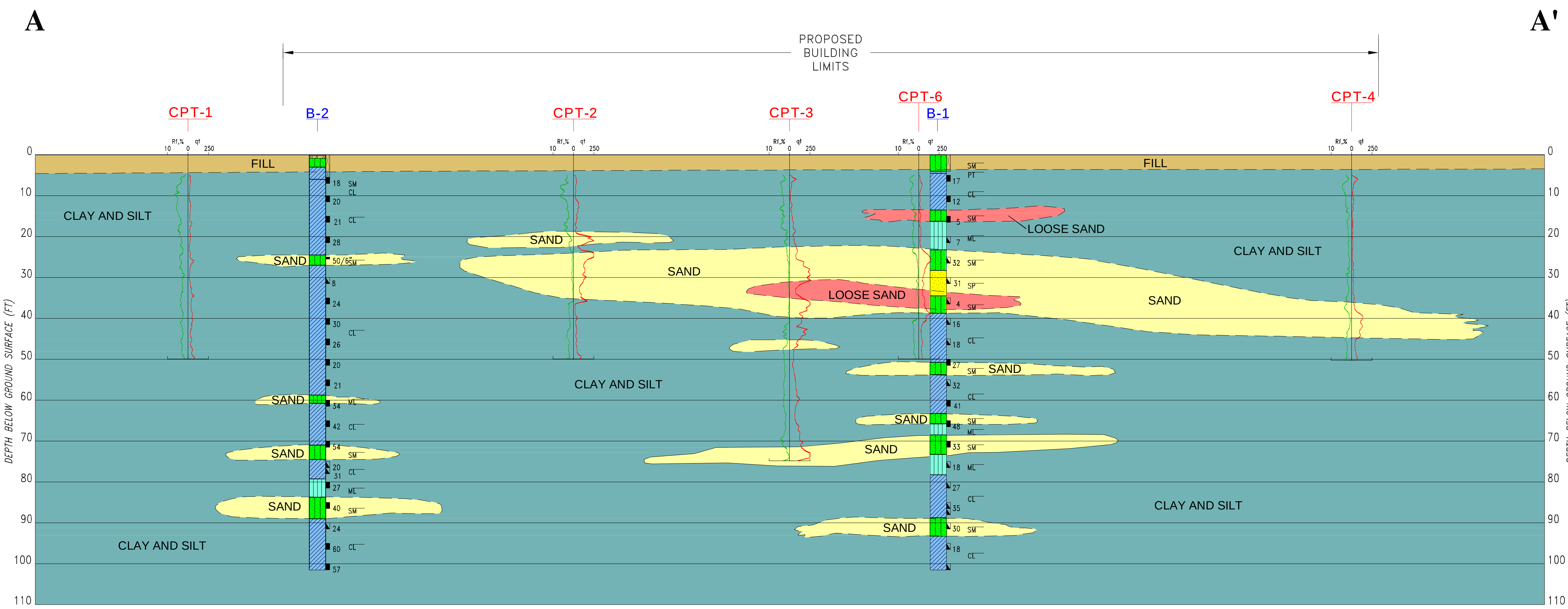
N

BASE MAP SOURCE: GOOGLE EARTH

ENGEO Expect Excellence	SITE PLAN STOCKTON COURTHOUSE STOCKTON, CALIFORNIA	PROJECT NO.: 8641.000.000		FIGURE NO. 2
		DATE: MARCH 2009		
		DRAWN BY: DLB	CHECKED BY: MG	

G:\Drafting\DRAFTING2_Dwg\8641\000\GEX\8641000000-GEX-2SitePlan-0309.dwg 3-20-09 01:00:57 PM pcrockett

ORIGINAL FIGURE PRINTED IN COLOR

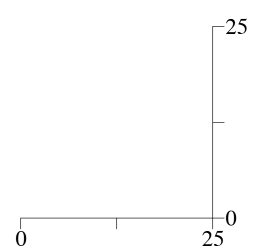


EXPLANATION

B-2 BORING

CPT-6 CONE PENETRATION TEST

NOTE:
SEE BORING AND CPT DETAILS IN APPENDIX A



SCALE IN FEET

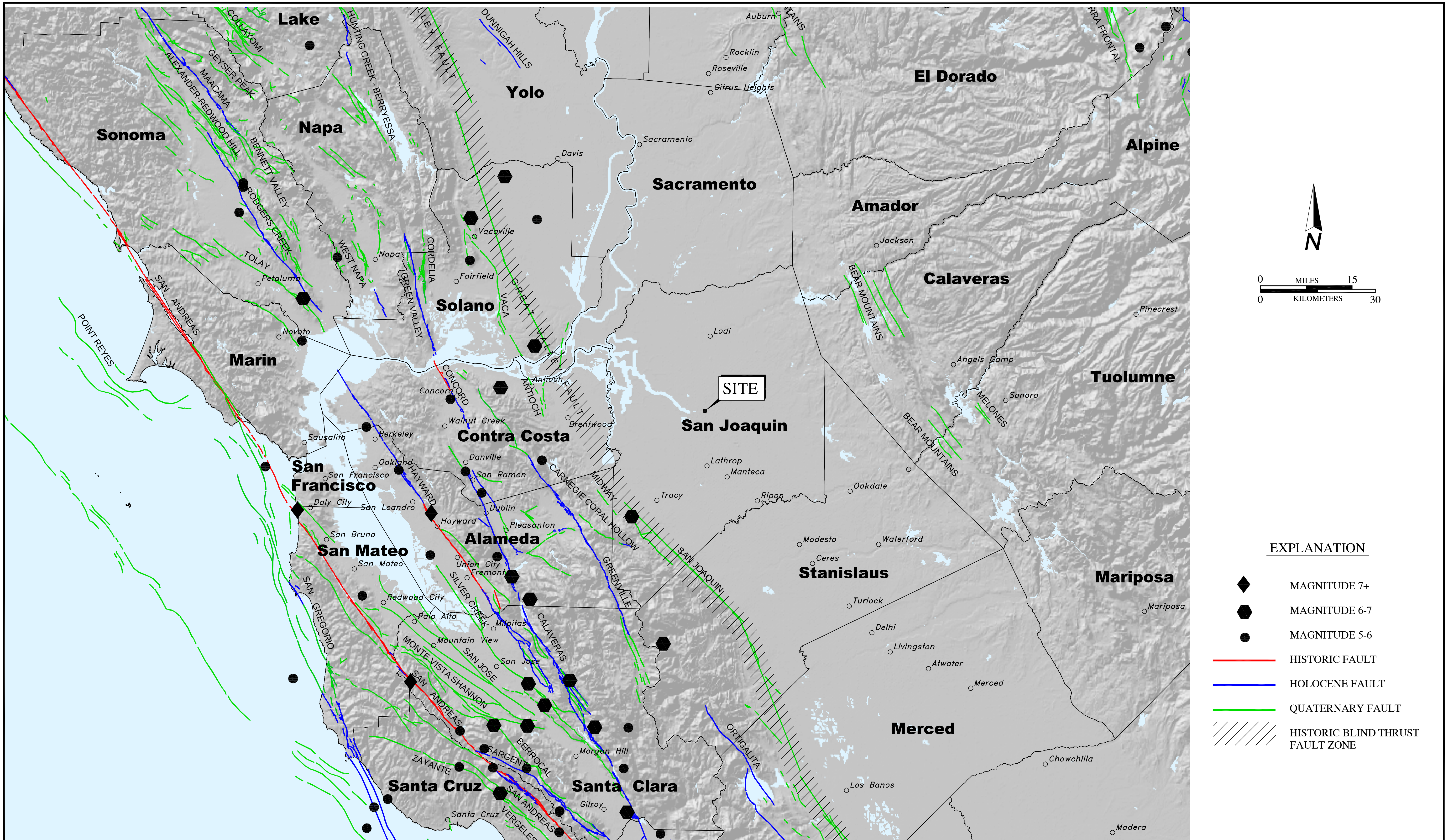
1"=25'

ENGEO
Expect Excellence

CROSS SECTION A-A'
STOCKTON COURTHOUSE
STOCKTON, CALIFORNIA

PROJECT NO.: 8641.000.000	
DATE: MARCH 2009	
DRAWN BY: DLB	CHECKED BY: MG

FIGURE NO. 3



BASE MAP SOURCE:
U.S.G.S. 1-ARC SECOND S.R.T.M. DATABASE
U.S.G.S. QUATERNARY FAULT DATABASE, MARCH, 2006
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

ENGEO — Expect Excellence

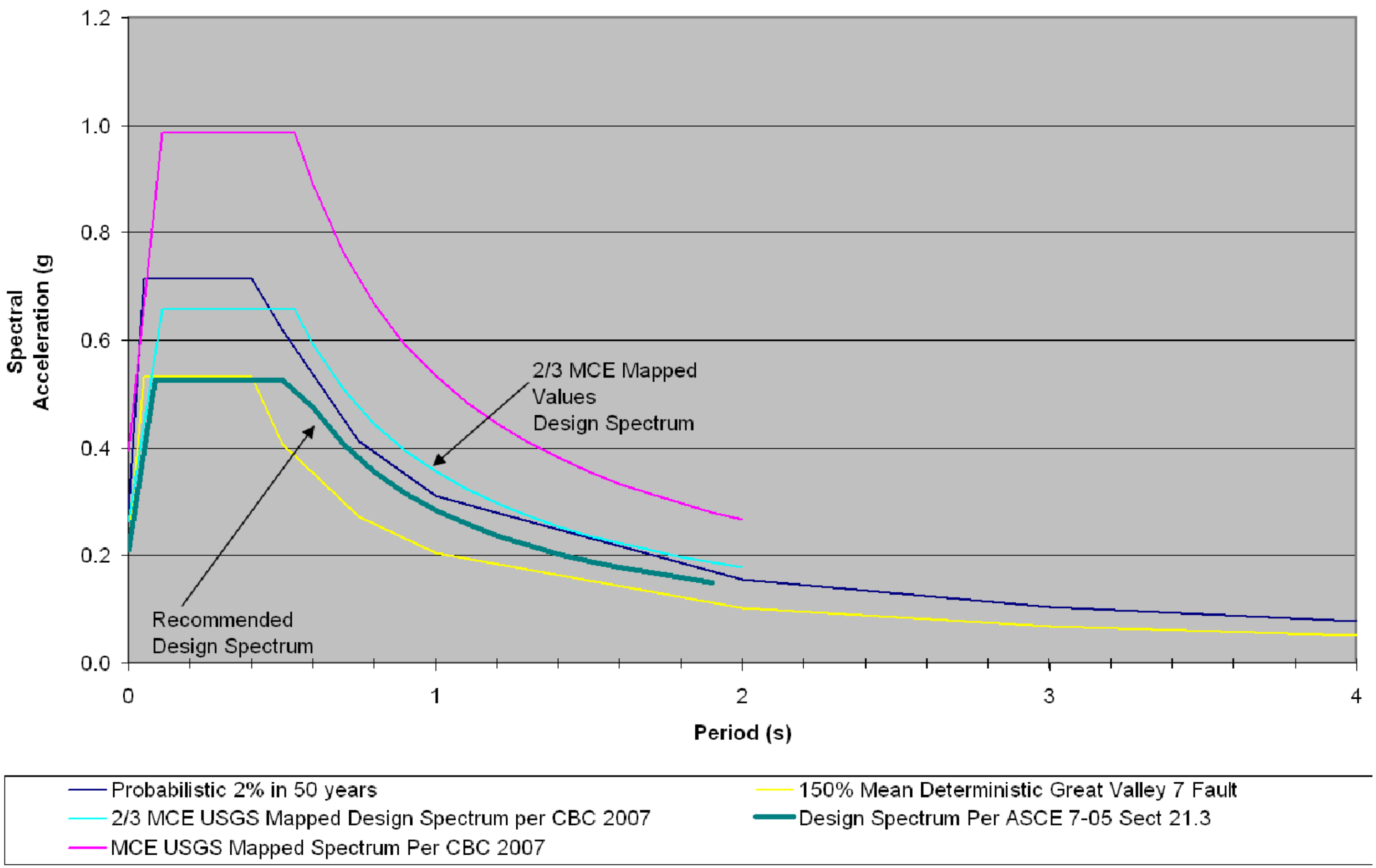
REGIONAL FAULTING AND SEISMICITY
STOCKTON COURTHOUSE
STOCKTON, CALIFORNIA

PROJECT NO.: 8641.000.000	FIGURE NO. 4
DATE: MARCH 2009	
DRAWN BY: DLB	CHECKED BY: MG

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg\8641\000\GEX\8641000000-GEX-4RegFaultAndSeisMap-0309.dwg 3-20-09 12:59:13 PM

G:\Drafting\DRAFTING2_Dwg\8641\000\GEX\8641000000-GEX-5ResponseSpectra-0309.dwg 3-20-09 12:56:44 PM pcrockett



NO SCALE

ENGEO Expect Excellence	RESPONSE SPECTRA STOCKTON COURTHOUSE STOCKTON, CALIFORNIA	PROJECT NO.: 8641.000.000 DATE: MARCH 2009 DRAWN BY: DLB CHECKED BY: MG	FIGURE NO. 5

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

Field Exploration Notes
Key to Boring Logs
Exploration Logs
CPT Soundings

APPENDIX A

FIELD EXPLORATION NOTES

We drilled 2 borings on the site for this report. An ENGEO representative supervised the drilling, and logged the subsurface conditions. A CME 75 drill rig was used to drill the borings using mud rotary methods.

The boring logs present descriptions and graphically depict the subsurface soil and groundwater conditions encountered. The maximum depth penetrated by the borings was 101.5 feet.

We obtained bulk soil samples from drill cuttings. We also retrieved soil samples at various intervals in the borings using Standard Penetration Tests (SPT) and a Modified California Sampler (3-inch O.D. split spoon sampler with thin walled liners).

The SPT and Modified California Sampler blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches.

We used a CPT rig to push the cone penetrometer to depths between approximately 50 and 75 feet at 6 locations on the site. The CPT has a 20-ton compression-type cone with a 15-square-centimeter (cm^2) base area, and a friction sleeve with a surface area of 225 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-3441. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). CPT sounding logs are presented in Appendix A.

NOTES TO THE LOGS

We determined the lines designating the interface between soil materials on the logs using visual observations. The transition between the materials may be abrupt or gradual.

The logs contain information concerning samples recovered, indications of the presence of various materials such as sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the soil conditions between samples. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs. The final logs represent our interpretation of the contents of the field logs.

KEY TO SOIL LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Inorganic silt with high plasticity
			CH - Inorganic clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Absence of moisture, dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO INCORPORATED

ENGEO
INCORPORATED

LOG OF BORING B-1

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED:	2/23/2009	LOGGED / REVIEWED BY:	P. Cottingham / JB
HOLE DEPTH:	Approx. 101½ ft.	DRILLING CONTRACTOR:	Precision Sampling
HOLE DIAMETER:	6.0 in.	DRILLING METHOD:	Mud Rotary
SURF ELEV (msl):	Approx. 15 ft.	HAMMER TYPE:	Automatic Trip Hammer

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		SILTY SAND (SM), dark brown, moist, fine- to medium-grained sand, 30% silt, [FILL]								
	1	(Hand cleared to 5 feet)								
		PEAT (PT), black, moist, silty, with wood debris, [FILL]								
5		SILTY CLAY (CL), black, very stiff, moist, medium plasticity	17	44	16	28		22.1		4.0*
	2	Unconfined Compression Test: 2,550 psf								
10	3	Grades to olive brown with rust staining	12					29.4	91.5	4.0*
	4	SILTY SAND (SM), dark yellowish brown, loose, moist, fine- to medium-grained sand, 20% silt								
15			5					26.3	96.1	1.0*
	5	SANDY SILT (ML), grayish green, medium stiff, moist, 20% fine-grained sand								
20	6	Grades to stiff, with clay	7				80			2.0* 1.0*
	7	SILTY SAND (SM), grayish green, dense, wet, fine- to medium-grained sand, 25% silt								
25										

LOG OF BORING B-1

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/23/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
SILTY SAND (SM), grayish green, dense, wet, fine- to medium-grained sand, 25% silt Grades to 10% silt	32				26	29.8		
POORLY GRADED SAND (SP), grayish green, medium dense to dense, wet, 5% fines	31				10	21.8		
SILTY SAND (SM), grayish gray, loose, wet, fine-grained sand, 20% silt	4				20	32.7		
SILTY CLAY (CL), grayish green, stiff, moist, low plasticity, with fine-grained sand	16	35	18	17		35.9		1.25*
Grades to very stiff, medium plasticity	18					34.7		3.0*

Depth in Feet: 30, 35, 40, 45, 50
Depth in Meters: 8, 9, 10, 11, 12, 13, 14, 15

LOG OF BORING B-1

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED:	2/23/2009	LOGGED / REVIEWED BY:	P. Cottingham / JB
HOLE DEPTH:	Approx. 101½ ft.	DRILLING CONTRACTOR:	Precision Sampling
HOLE DIAMETER:	6.0 in.	DRILLING METHOD:	Mud Rotary
SURF ELEV (msl):	Approx. 15 ft.	HAMMER TYPE:	Automatic Trip Hammer

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		Grades to low plasticity, medium stiff	27				32	28.5	96.1	0.75*
	16	SILTY SAND (SM), grayish green, medium dense, moist, fine-grained sand, 20% silt Grades to 45% silt								
55	17	CLAY (CL), grayish green, hard, moist, medium plasticity, weak cementation	32							4.5+*
60	18	Unconfined Compression Test: 7,224 psf	41							4.5+*
	19									
65	20	SILTY SAND (SM), grayish green, dense, wet, fine-grained sand, 25% silt								
		SANDY SILT (ML), grayish green, very stiff, moist, 40% fine-grained sand	48							4.0*
70	21	SILTY SAND (SM), grayish green, medium dense, wet, fine-grained sand, 35% silt	33				36	27.3		
	22	SILT (ML), grayish green, stiff, moist, low plasticity, trace clay, trace fine-grained sand								
75										

LOG OF BORING B-1

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/23/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
	23		SILT (ML), grayish green, stiff, moist, low plasticity, trace clay, trace fine-grained sand			18							1.25*
	24		SILTY CLAY (CL), grayish green, stiff, moist, low plasticity, 5% fine-grained sand										
80						27							
	25												
85	26					35							
			Grades to very stiff										3.75*
	27		SILTY SAND (SM), grayish green, dense, moist, fine- to medium-grained sand, 15% silt										
90						30				25	20.2		
	28		SILTY CLAY (CL), grayish green, stiff, wet, medium plasticity										
95	29					18							
	30												
100													

LOG OF BORING B-1

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/23/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SILTY CLAY (CL), grayish green, stiff, wet, medium plasticity Grades to hard, moist										4.5+*
			Boring terminated at approximately 105.5 feet. Depth to groundwater not determined due to drilling method.										

ENGEO INCORPORATED

LOG OF BORING B-2

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/27/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet: 5, 10, 15, 20, 25
Depth in Meters: 1, 2, 3, 4, 5, 6, 7

DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
1.5 inches ASPHALT CONCRETE								
8 inches AGGREGATE BASE								
SILTY SAND (SM), dark yellowish brown, moist, fine- to coarse-grained sand, with 5% fine-grained gravel, 20% silt, [FILL]								
SILTY CLAY (CL), black, medium stiff, moist, trace fine-grained sand, some reddish brown inclusions, [FILL] Bone fragment at 4 feet (Hand cleared to 5 1/2 feet) Bone fragment at 5 3/4 feet								
SILTY CLAY (CL), reddish brown, very stiff, moist, medium plasticity, 20% fine- to medium-grained sand	18					19.1	101	0.75* 3.0*
Grades to 40% fine- to medium-grained sand Grades to olive-brown, <5% fine-grained sand, rust stained rootlets, medium plasticity	20					22.4		4.0* 4.0*
Grades to hard	21					24.3		4.5+*
Grades to dark yellowish brown with rust staining, 30% fine-grained sand Grades to 45% fine- to medium-gained sand	28				72			4.5* 4.5*

LOG OF BORING B-2

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED:	2/27/2009	LOGGED / REVIEWED BY:	P. Cottingham / JB
HOLE DEPTH:	Approx. 101½ ft.	DRILLING CONTRACTOR:	Precision Sampling
HOLE DIAMETER:	6.0 in.	DRILLING METHOD:	Mud Rotary
SURF ELEV (msl):	Approx. 15 ft.	HAMMER TYPE:	Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
	8		SILTY SAND (SM), dark reddish brown, very dense, moist, 30% fine-grained gravel, 20% silt, rust staining			50/6"				21			
30	9					8					28.5		1.5* 2.5*
35	10, 11		Grades to grayish green, hard, 20% fine-grained sand, low plasticity			24							4.5+* 4.5*
40	12					30					25.3		3.5*
	13		Grades to very stiff										
45	14		Grades to hard, 10% fine-grained sand			26							4.5*
50	15												

ENGEO INCORPORATED

LOG OF BORING B-2

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/27/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

DESCRIPTION	Blow Count/Foot	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
SILTY CLAY (CL), olive brown, stiff to very stiff, moist, medium plasticity, trace fine-grained sand	20							
Grades to very stiff								2.75*
	21					27.6		3.5*
SANDY SILT (ML), grayish green, stiff, moist, 35% fine-grained sand								
SILTY CLAY (CL), grayish green, hard, moist, low plasticity, 25% fine-grained sand	34				64			4.5*
Grades to medium plasticity	42					27.6		4.5*
Grades to 35% fine sand	54				59			4.5*
SILTY SAND (SM), grayish green, very dense, moist, fine- to medium-grained sand, 30% silt								

Depth in Feet: 55, 60, 65, 70, 75

Depth in Meters: 16, 17, 18, 19, 20, 21, 22

Column headings also printed: Sample Type, Log Symbol, Water Level; Atterberg Limits spans Liquid Limit, Plastic Limit, Plasticity Index.

LOG OF BORING B-2

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/27/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
	23	SILTY CLAY (CL), grayish green, hard, moist, medium plasticity, 5% fine-grained sand	20							
			31							4.5*
	24									
80		SANDY SILT (ML), grayish green, very stiff, moist, trace carbonates, 35% fine- to medium grained sand, weak HCl reaction	27				63			2.5*
	25									
85	26	SILTY SAND (SM), grayish brown, medium dense, moist, fine- to medium-grained sand, 25% silt	40				24			
	27	SILTY CLAY (CL), grayish green, very stiff, moist, low plasticity, trace carbonates, 15% fine-grained sand								
90			24					27.3		4.5+*
	28									
95	29	Grades to hard, no carbonate	60							
	30									
100										

ENGEO INCORPORATED

LOG OF BORING B-2

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/27/2009
HOLE DEPTH: Approx. 101½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Precision Sampling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: Automatic Trip Hammer

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SILTY CLAY (CL), grayish green, very stiff, moist, low plasticity, trace carbonates, 15% fine-grained sand Grades to trace carbonate, weak cementation, HCl reaction			57							4.5+*
			Boring terminated at approximately 105.5 feet. Depth to groundwater not determined due to drilling method.										

LOG OF BORING HA-1

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED:	2/27/2009	LOGGED / REVIEWED BY:	P. Cottingham / JB
HOLE DEPTH:	Approx. 5 ft.	DRILLING CONTRACTOR:	Gregg Drilling & Testing
HOLE DIAMETER:	4.0 in.	DRILLING METHOD:	Hand Auger
SURF ELEV (msl):	Approx. 15 ft.	HAMMER TYPE:	N/A

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			POORLY GRADED SAND (SP), brown, moist, medium-grained sand, [FILL]										
	1		SILTY CLAY (CL), black with brown, moist, low plasticity, [FILL] Debris encountered (concrete, metal fragments) Medium plasticity										
5			Hand cleared to approximately 5 feet.										

LOG OF BORING HA-2

ENGEO INCORPORATED

Geotechnical Exploration
New Stockton Courthouse
Stockton, CA
8641.000.000

DATE DRILLED: 2/27/2009
HOLE DEPTH: Approx. 7 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 15 ft.

LOGGED / REVIEWED BY: P. Cottingham / JB
DRILLING CONTRACTOR: Gregg Drilling & Testing
DRILLING METHOD: Hand Auger
HAMMER TYPE: N/A

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			POORLY GRADED SAND brown, moist, medium-grained sand, [FILL]										
			GRAVELLY LEAN CLAY (CL), [FILL]										
	1		Void encountered										
5	2												
			Hand cleared to approximately 7 feet.										

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT.* It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

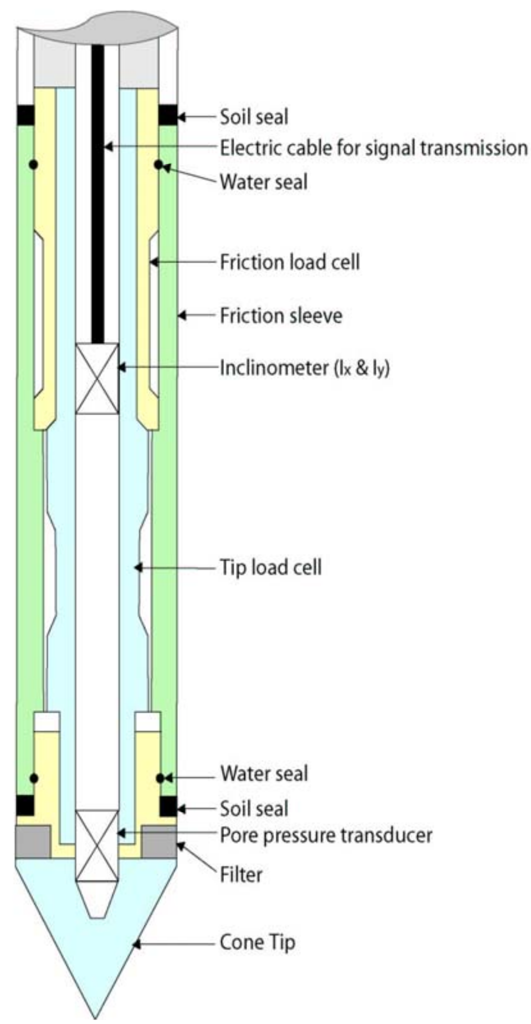


Figure CPT

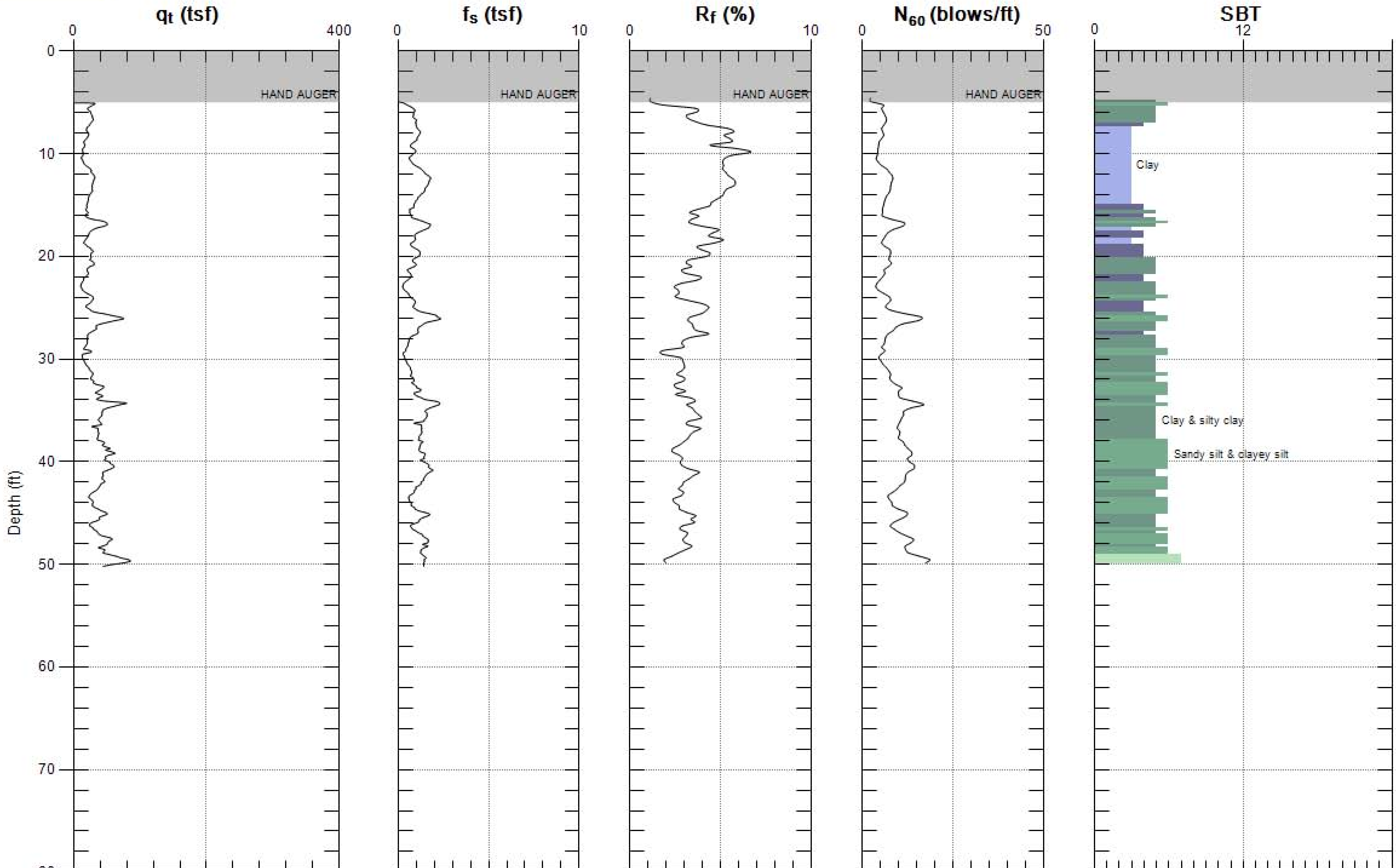
When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

GREGG

ENGEO

Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-1 Date: 2/27/2009 06:59



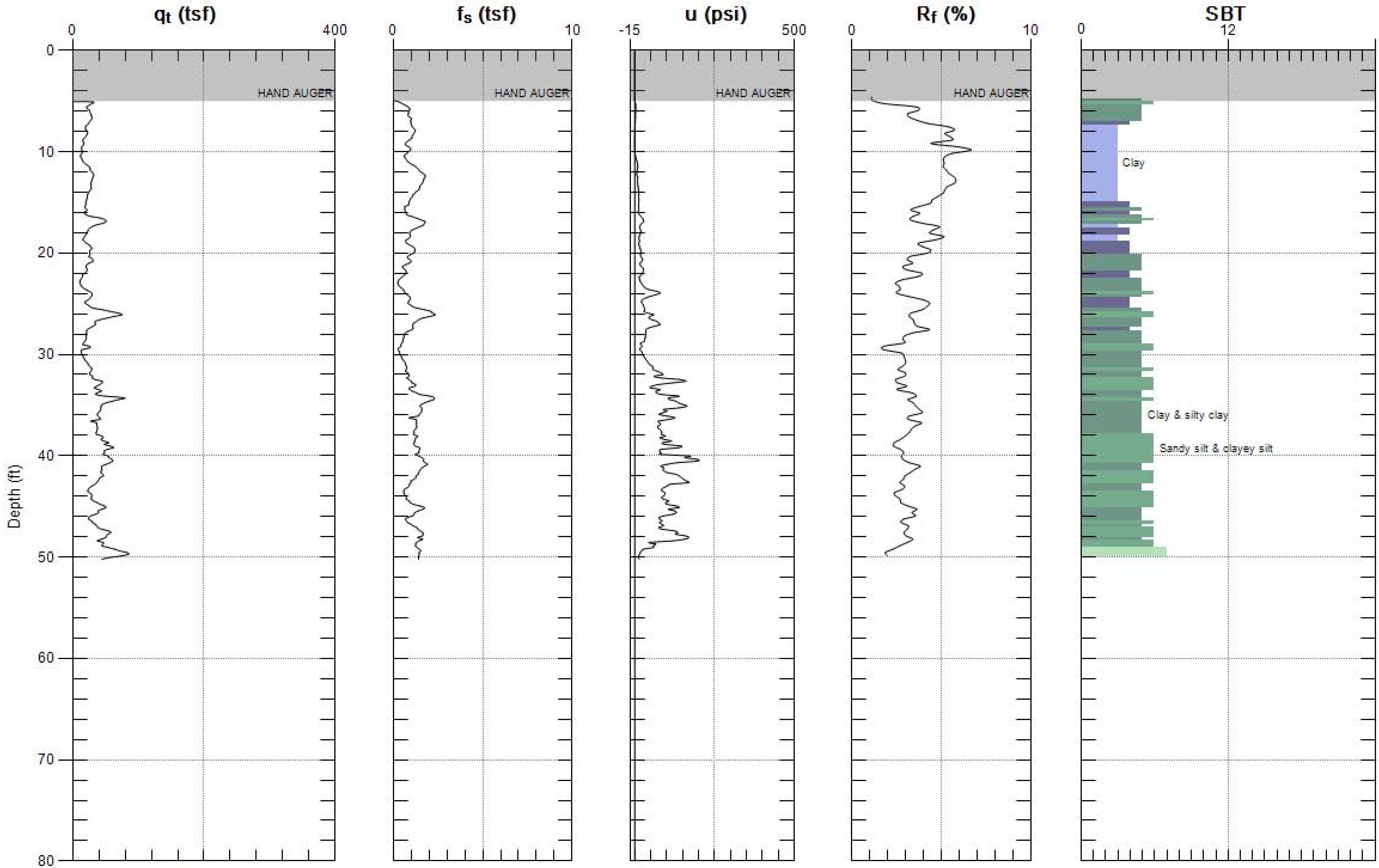
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-1 Date: 2/27/2009 06:59



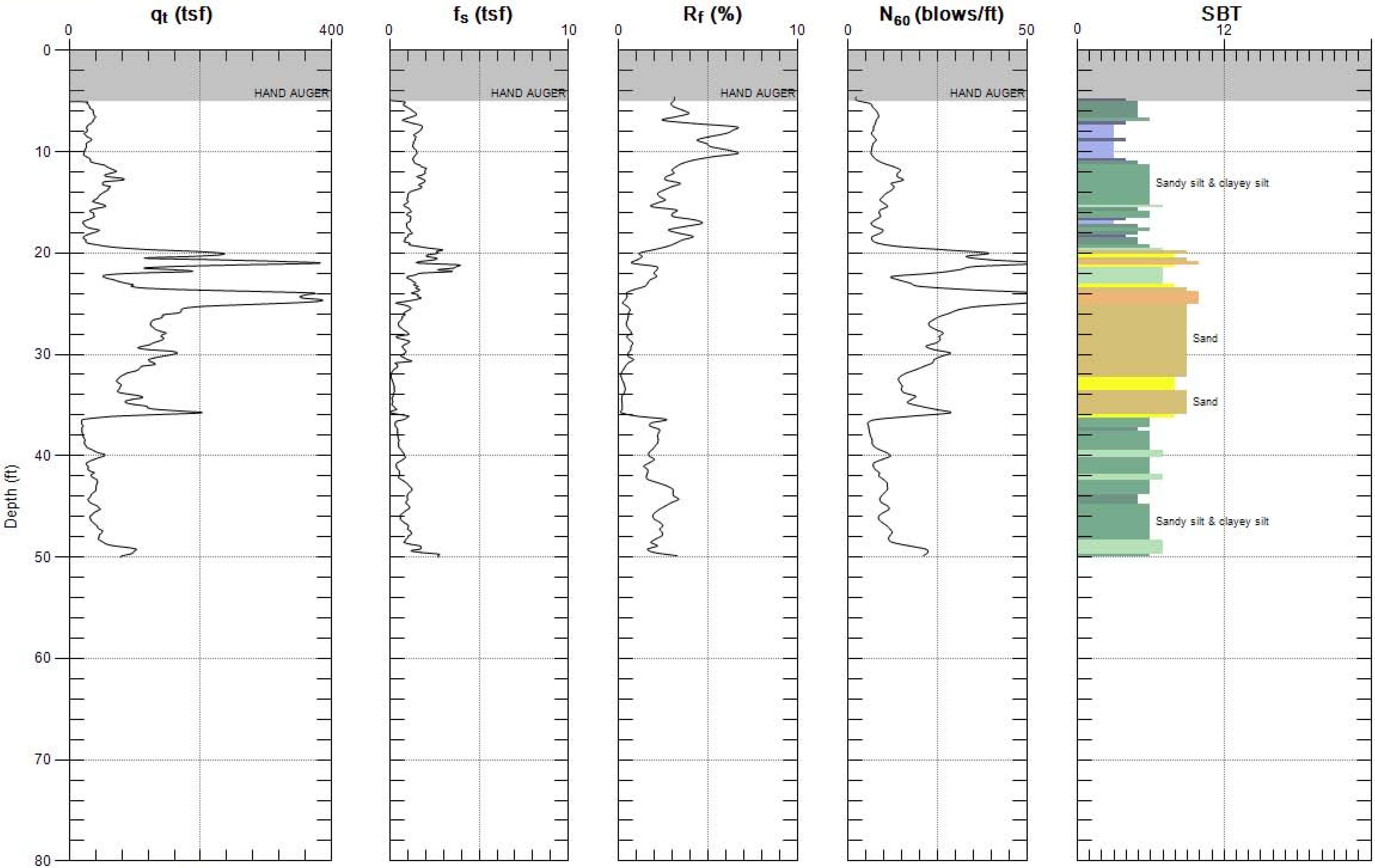
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-2 Date: 2/27/2009 08:36



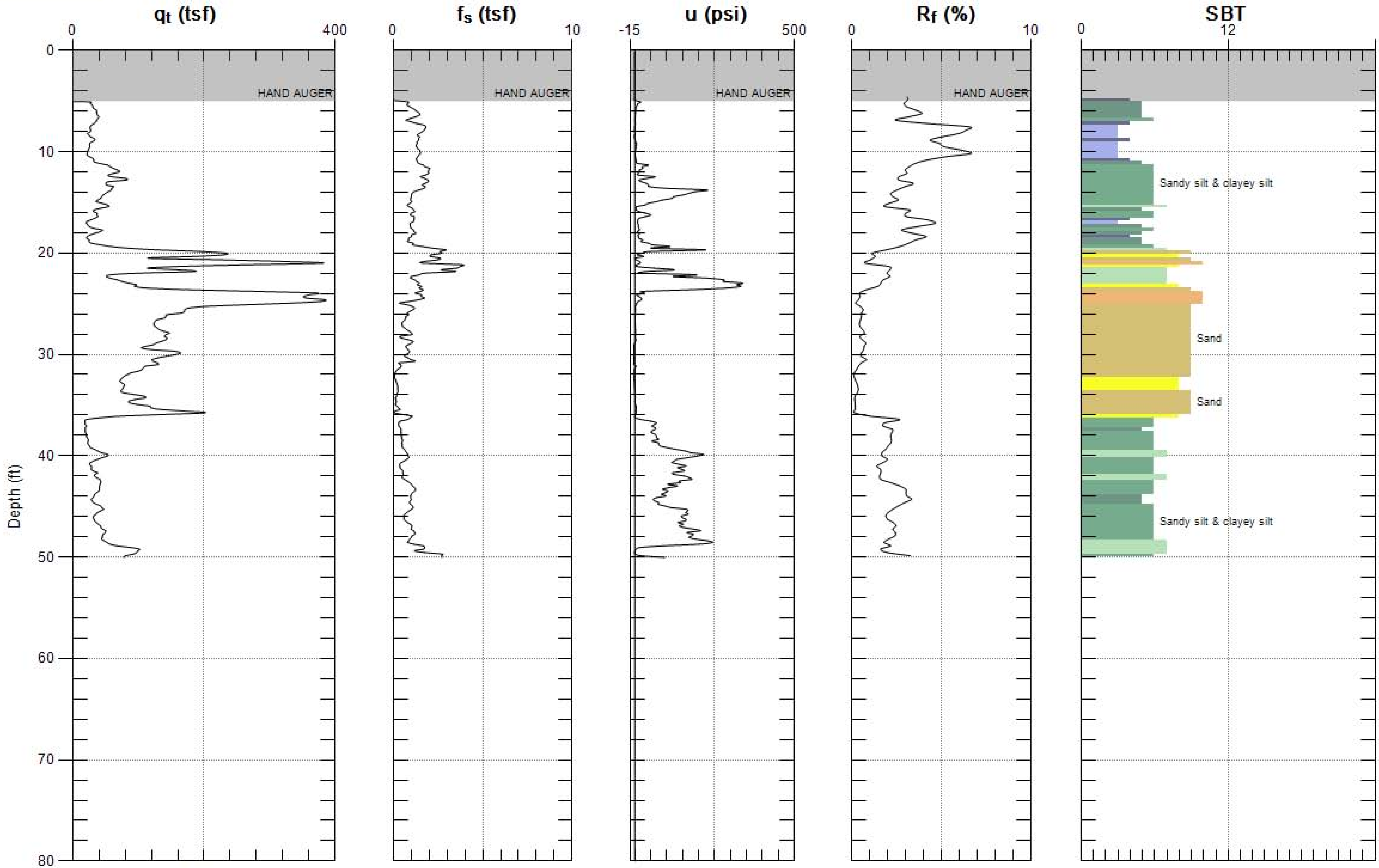
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-2 Date: 2/27/2009 08:36



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

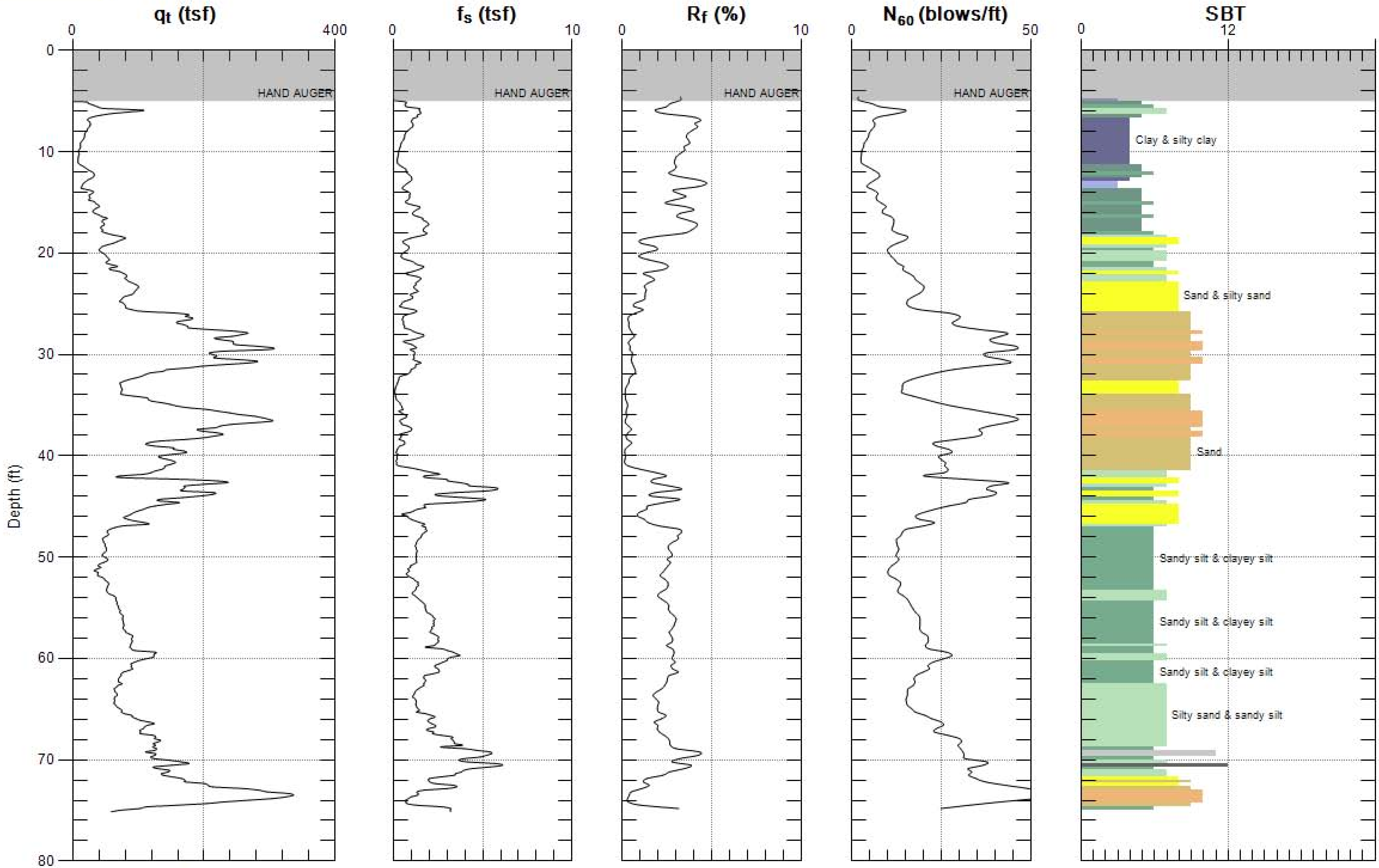
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: STOCKTON COURTHOUSE **Engineer: P.COTTINGHAM**

Sounding: CPT-3 **Date: 2/27/2009 12:01**



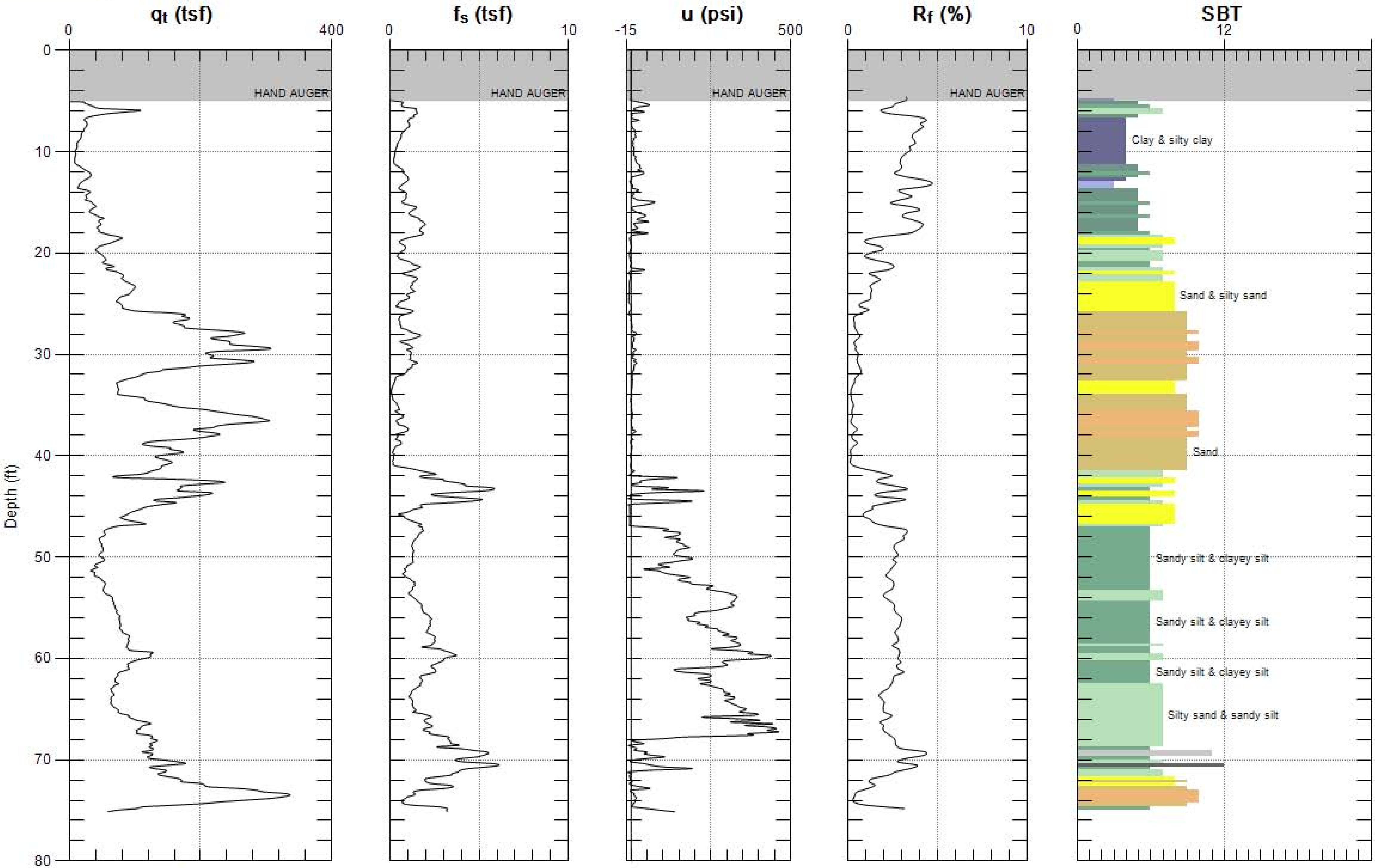
Max. Depth: 75.131 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

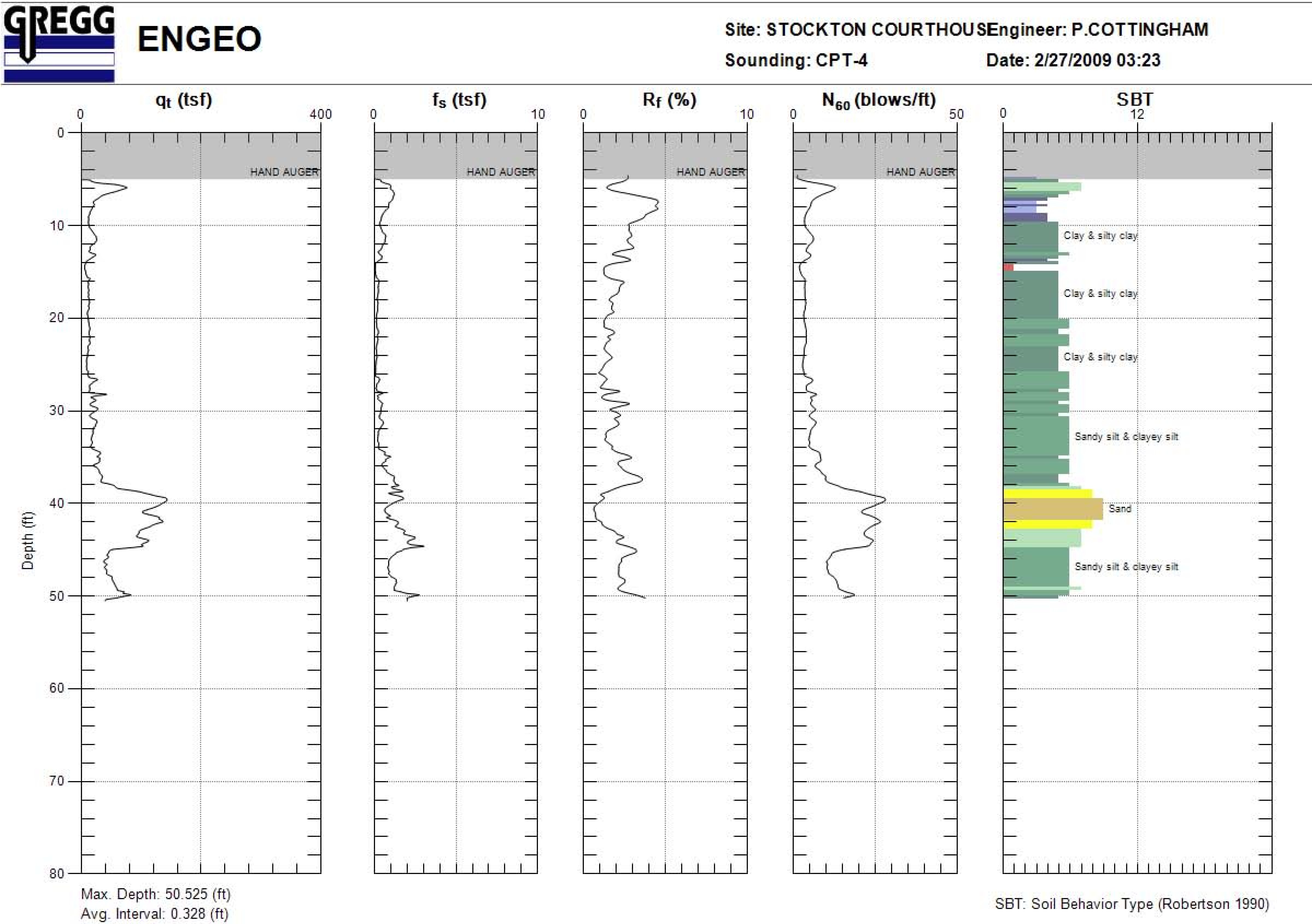
Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-3 Date: 2/27/2009 12:01



Max. Depth: 75.131 (ft)
Avg. Interval: 0.328 (ft)

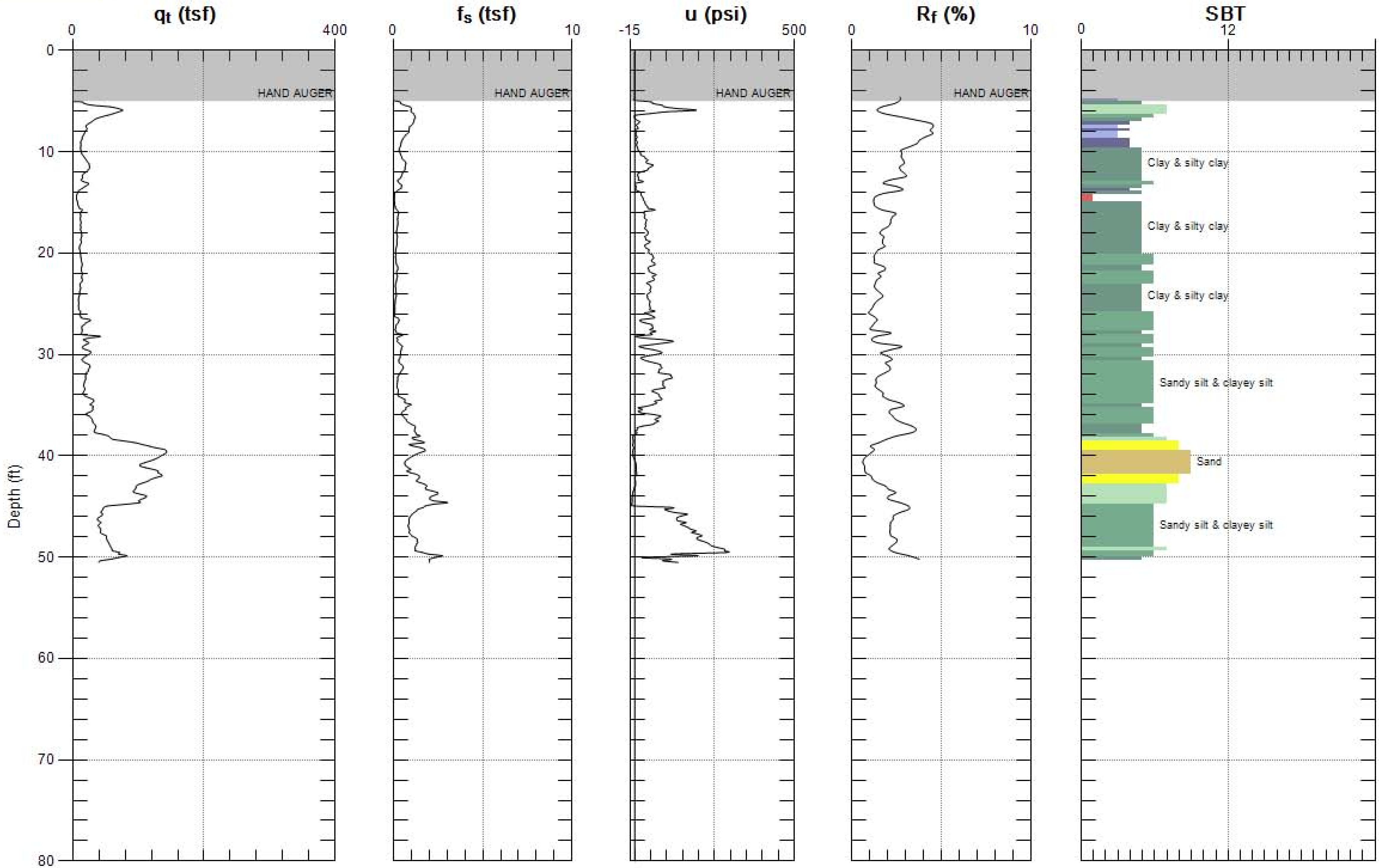
SBT: Soil Behavior Type (Robertson 1990)


GREGG
ENGEO
Site: STOCKTON COURTHOUSE
Engineer: P.COTTINGHAM
Sounding: CPT-4
Date: 2/27/2009 03:23
qt (tsf)
fs (tsf)
Rf (%)
N60 (blows/ft)
SBT
HAND AUGER
Clay & silty clay
Clay & silty clay
Clay & silty clay
Sandy silt & clayey silt
Sand
Sandy silt & clayey silt
Depth (ft)
Max. Depth: 50.525 (ft)
Avg. Interval: 0.328 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: STOCKTON COURTHOUSE **Engineer: P.COTTINGHAM**

Sounding: CPT-4 **Date: 2/27/2009 03:23**



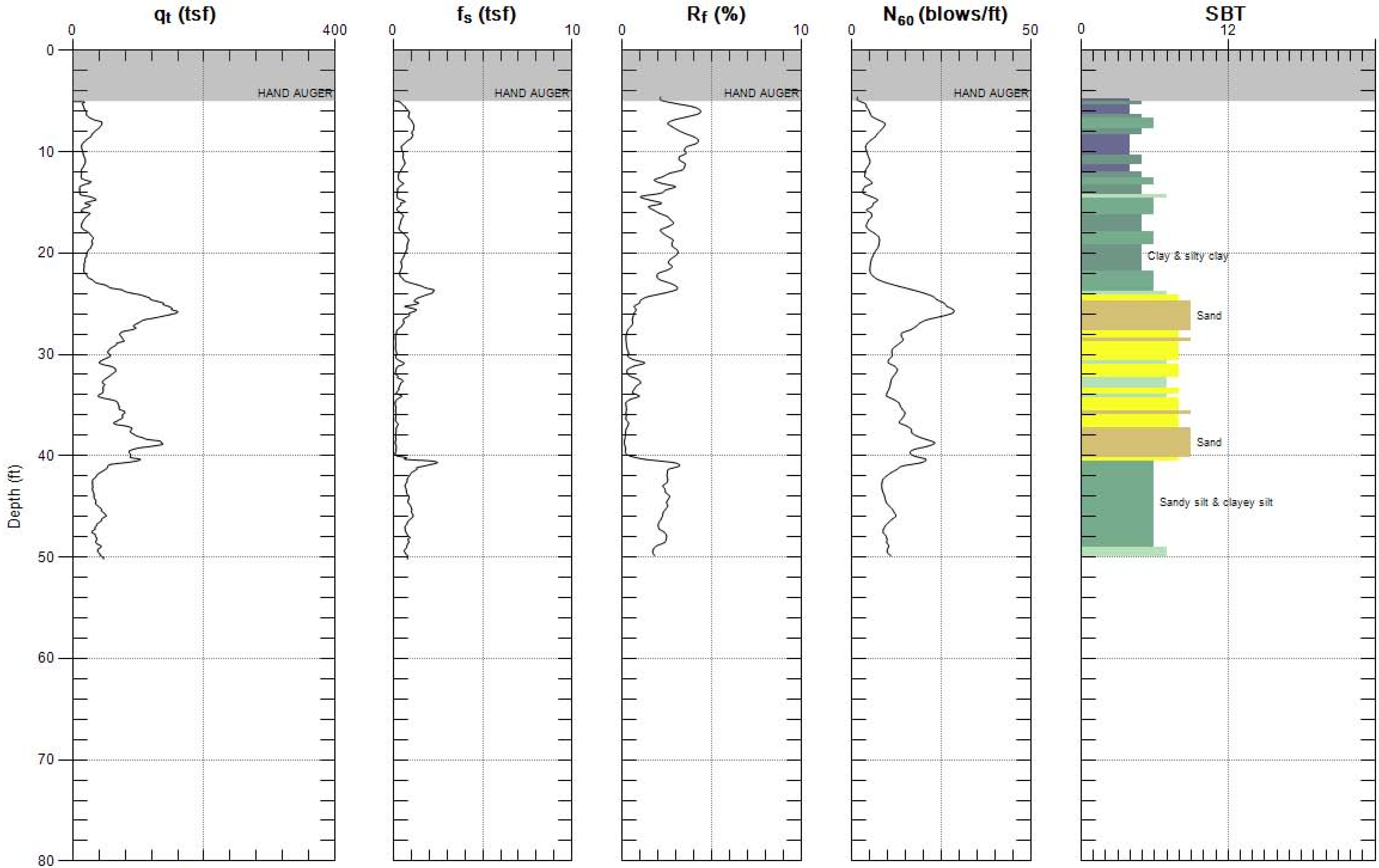
Max. Depth: 50.525 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **ENGEO**

Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-6 Date: 2/27/2009 10:46



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

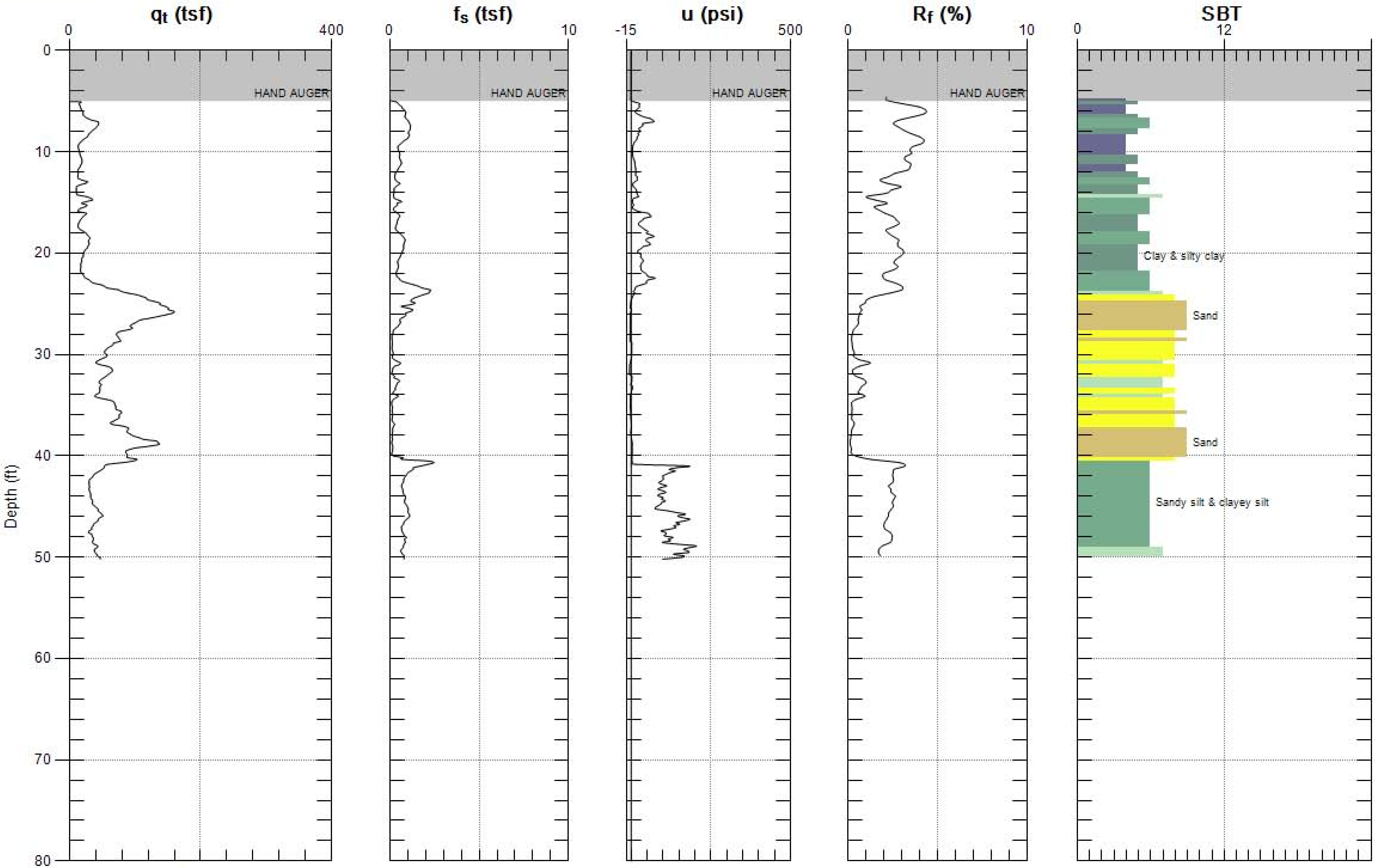
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: STOCKTON COURTHOUSE Engineer: P.COTTINGHAM

Sounding: CPT-6 Date: 2/27/2009 10:46



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

SBT: Soil Behavior Type (Robertson 1990)

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPDT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded by a computer system.

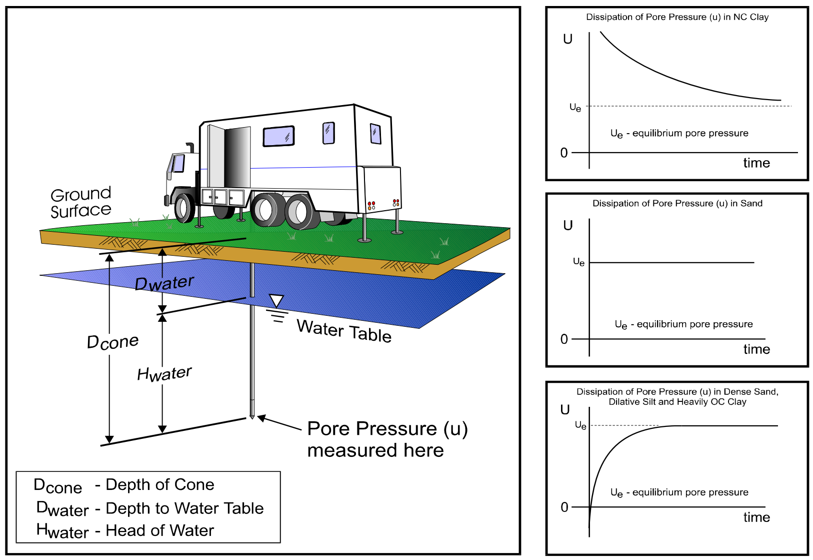
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992.

A summary of the pore pressure dissipation tests is summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

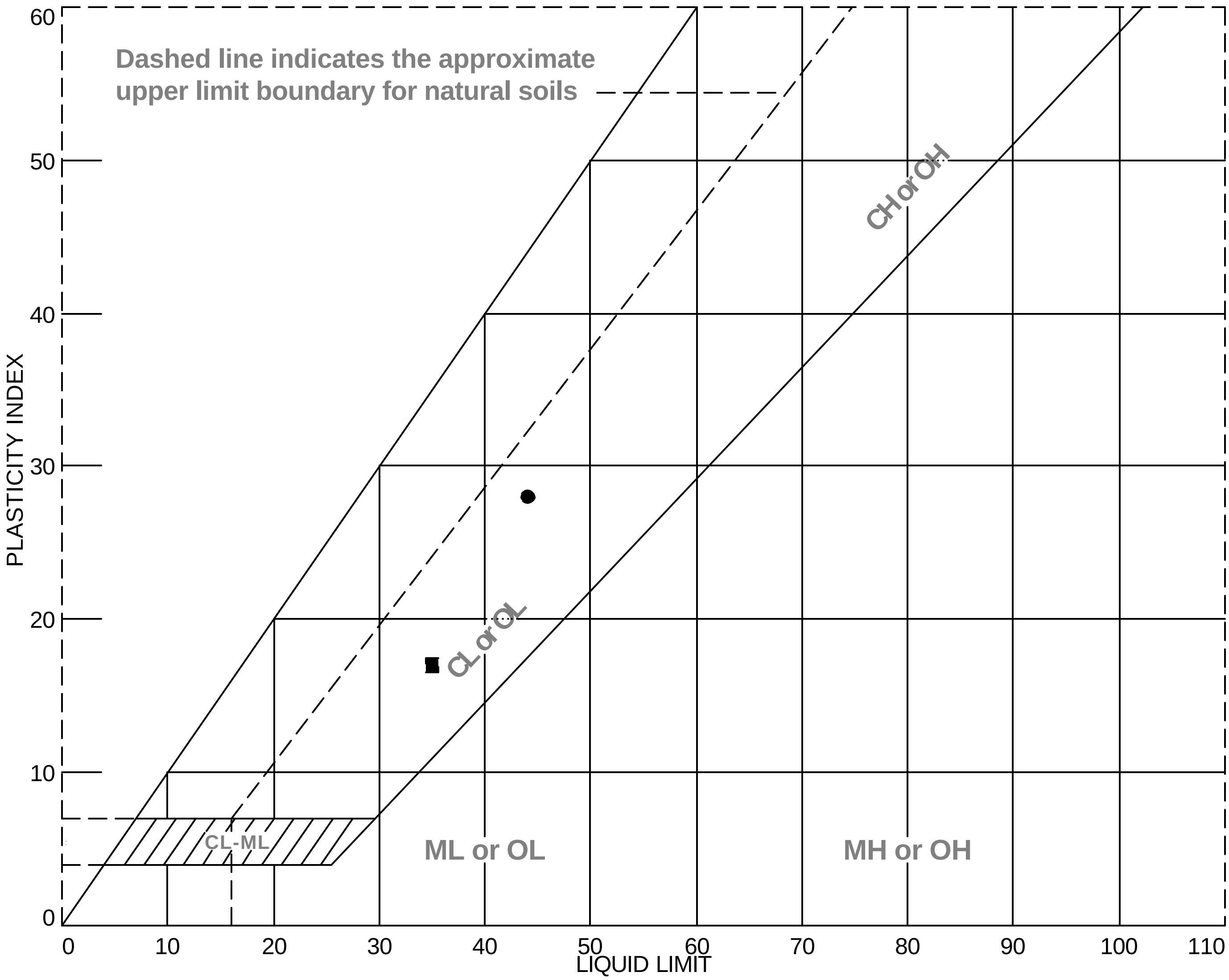
Figure PPDT

APPENDIX B
LABORATORY TEST DATA

Liquid and Plastic Limits Test Report
Unconfined Compression Test Report
Particle Size Distribution Report
R-Value Test Report
Direst Sheer Test Report
Analytical Results of Soil Corrosion

APPENDIX B

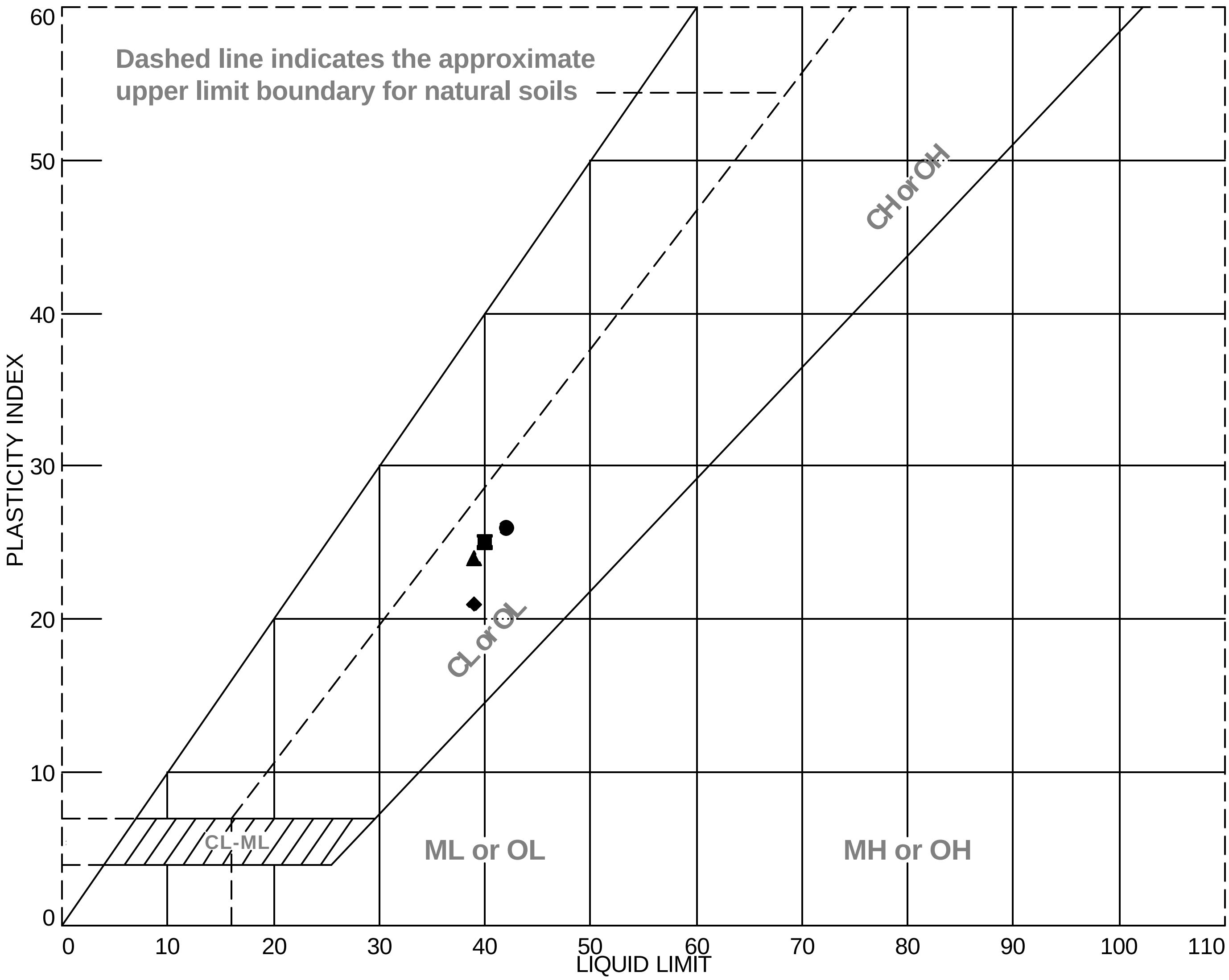
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	GEX	B1@5.5	5 1/2 feet	22.1	16	44	28	CL
■	GEX	B1@40	40 feet	35.9	18	35	17	CL

ENGEO, Inc. **Rocklin, CA**	**Client:** NBBJ **Project:** New Stockton Courthouse **Project No.:** 8641.000.000 **Figure**

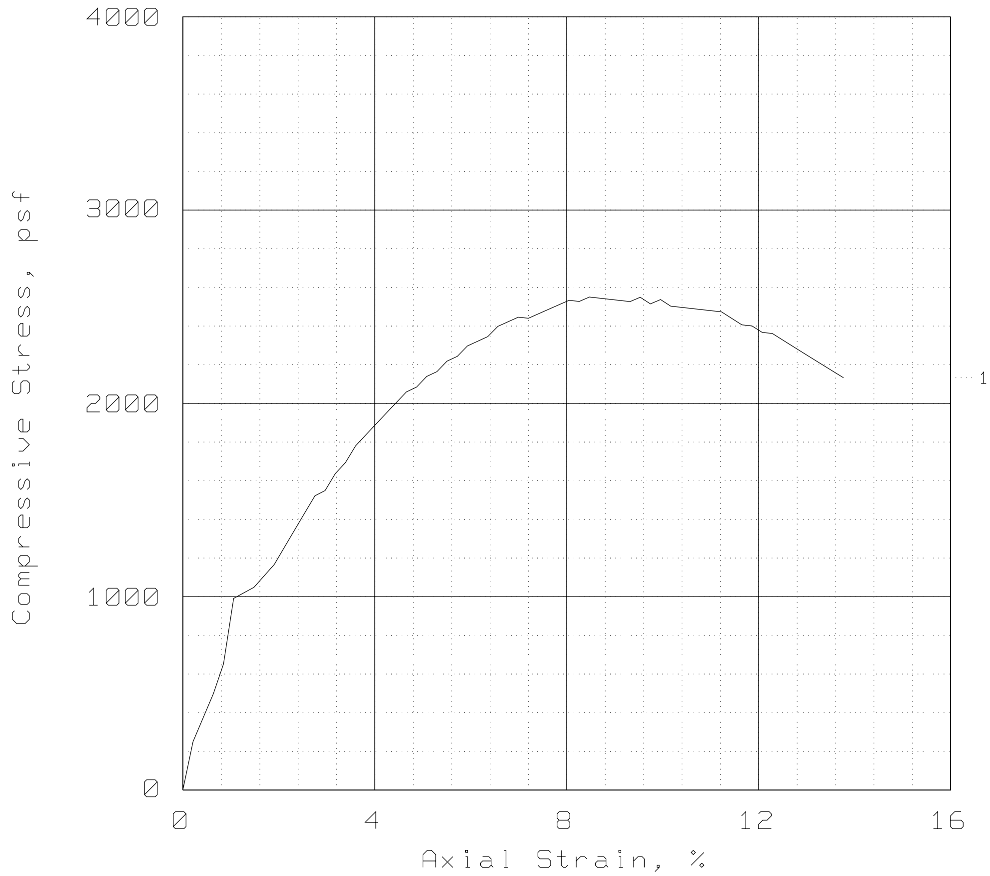
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	GEX	B2@6	6 feet		16	42	26	CL
■	GEX	B2@15.5	15 1/2 feet	24.3	15	40	25	CL
▲	GEX	B2@30	30 feet	28.5	15	39	24	CL
◆	GEX	B2@65.5	65 1/2 feet	27.6	18	39	21	CL

ENGEO, Inc. Rocklin, CA	**Client:** NBBJ **Project:** New Stockton Courthouse **Project No.:** 8641.000.000 **Figure**

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	2550			
Undrained shear strength, psf	1275			
Failure strain, %	8.5			
Strain rate, %/min	2.18			
Water content, %	24.9			
Wet density, pcf	113.5			
Dry density, pcf	90.9			
Saturation, %	80.3			
Void ratio	0.8202			
Specimen diameter, in	2.42			
Specimen height, in	4.72			
Height/diameter ratio	1.95			

Description: Black silty clay (CL)

GS= 2.65 Type: In situ

Project No.: 8641.000.000

Date: 2/23/09

Remarks:

Assumed Specific Gravity

Fig. No.: ______

Client: NBBJ

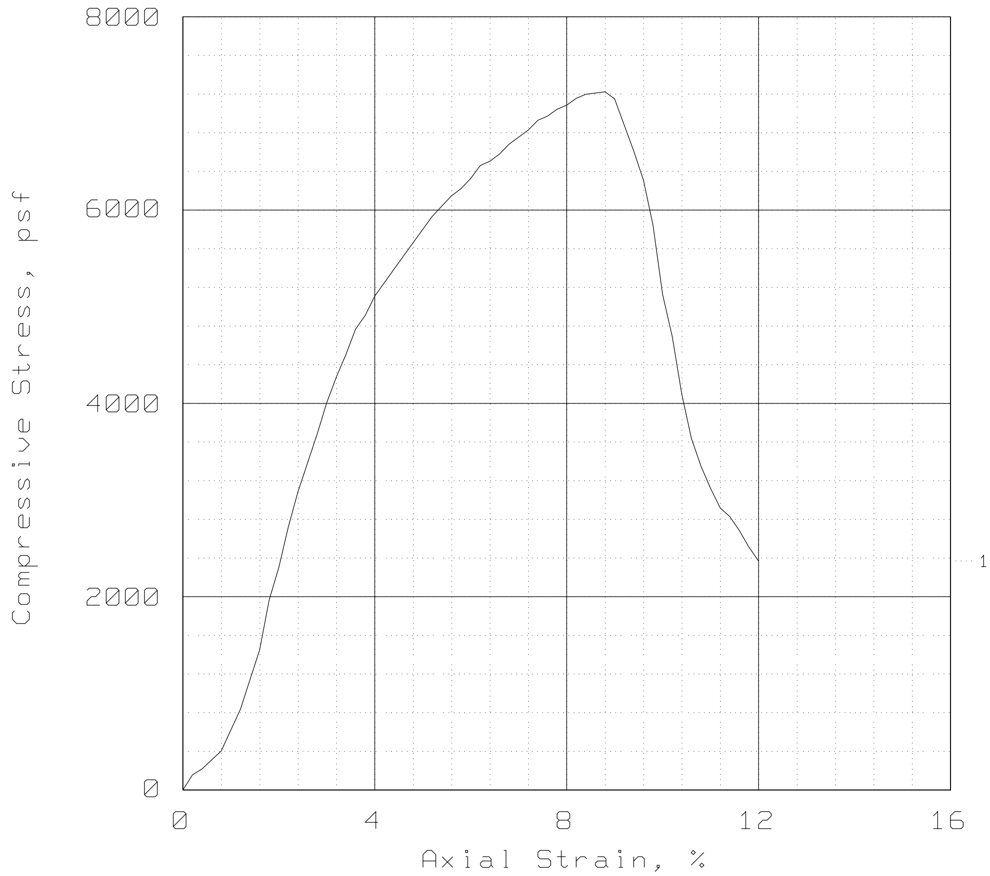
Project: New Stockton Courthouse

Location: B1@6.0

UNCONFINED COMPRESSION TEST

ENGEO, INCORPORATED

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	7224			
Undrained shear strength, psf	3612			
Failure strain, %	8.8			
Strain rate, %/min	1.71			
Water content, %	27.2			
Wet density, pcf	123.6			
Dry density, pcf	97.2			
Saturation, %	99.9			
Void ratio	0.7342			
Specimen diameter, in	2.42			
Specimen height, in	5.00			
Height/diameter ratio	2.07			

Description: Grayish green silty sand (SM)

GS= 2.7 Type: In situ

Project No.: 8641.000.000

Date: 2/23/09

Remarks:

Assumed Specific Gravity

Fig. No.: ______

Client: NBBJ

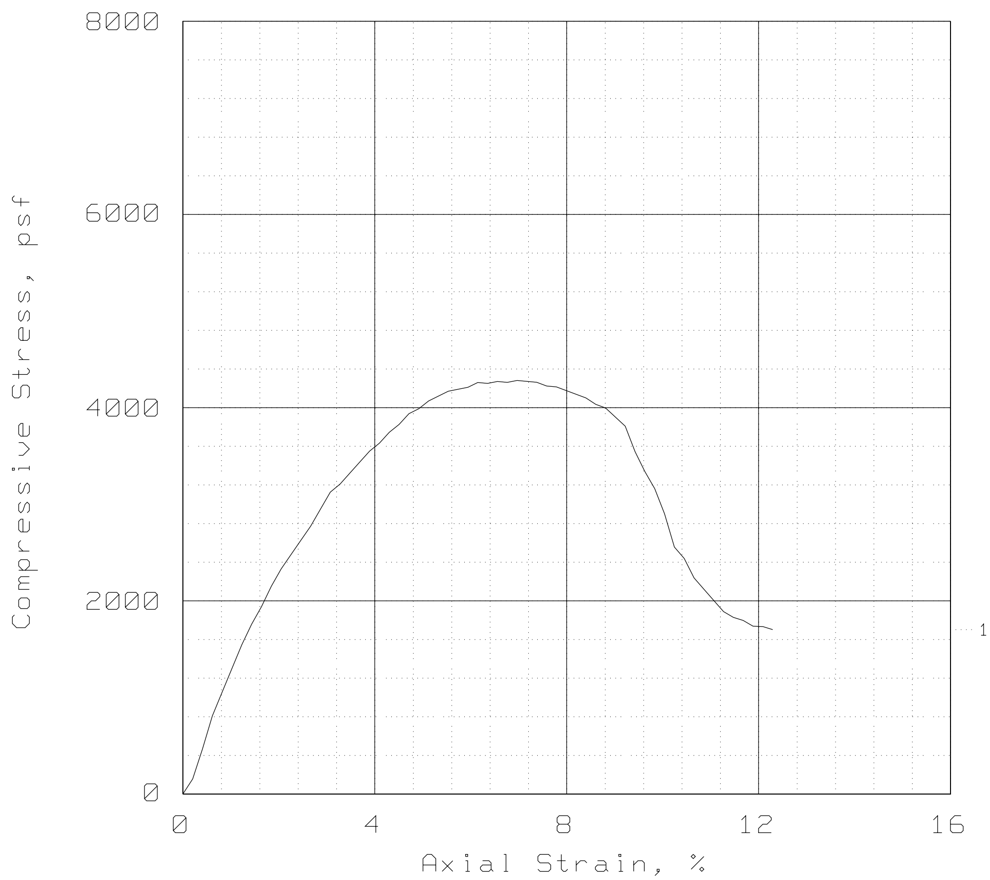
Project: New Stockton Courthouse

Location: B1@60.5

UNCONFINED COMPRESSION TEST

ENGEO, INCORPORATED

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	4282			
Undrained shear strength, psf	2141			
Failure strain, %	7.0			
Strain rate, %/min	1.68			
Water content, %	20.1			
Wet density, pcf	133.8			
Dry density, pcf	111.4			
Saturation, %	98.7			
Void ratio	0.5690			
Specimen diameter, in	2.42			
Specimen height, in	4.88			
Height/diameter ratio	2.02			

Description: Grayish green silty clay (CL)

GS= 2.8 | Type: In situ

Project No.: 8641.000.000

Date: 3/1/09

Remarks:

Assumed Specific Gravity

Fig. No.: ______

Client: NBBJ

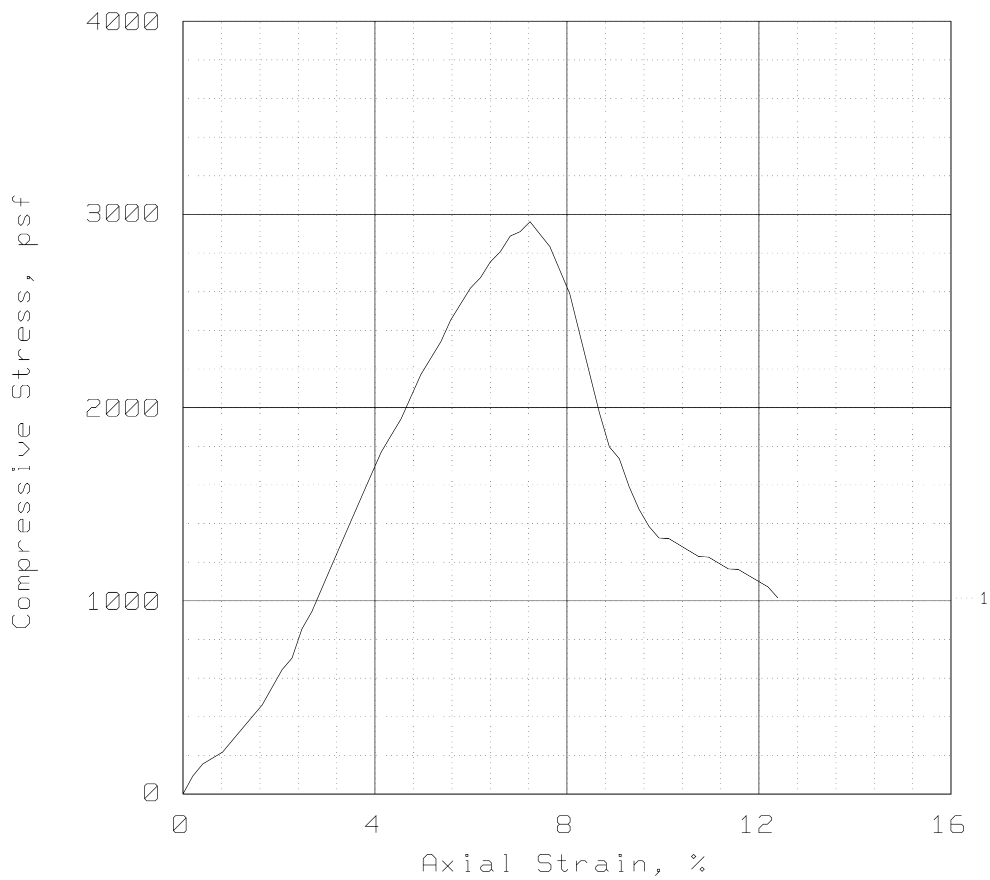
Project: New Stockton Courthouse

Location: B2@36

UNCONFINED COMPRESSION TEST

ENGEO, INCORPORATED

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	2962			
Undrained shear strength, psf	1481			
Failure strain, %	7.2			
Strain rate, %/min	1.70			
Water content, %	35.1			
Wet density, pcf	117.6			
Dry density, pcf	87.1			
Saturation, %	97.4			
Void ratio	1.0076			
Specimen diameter, in	2.42			
Specimen height, in	4.84			
Height/diameter ratio	2.00			

Description: Grayish green silty clay (CL)

GS= 2.8 Type: In situ

Project No.: 8641.000.000

Date: 3/1/09

Remarks:

Assumed Specific Gravity

Fig. No.: ______

Client: NBBJ

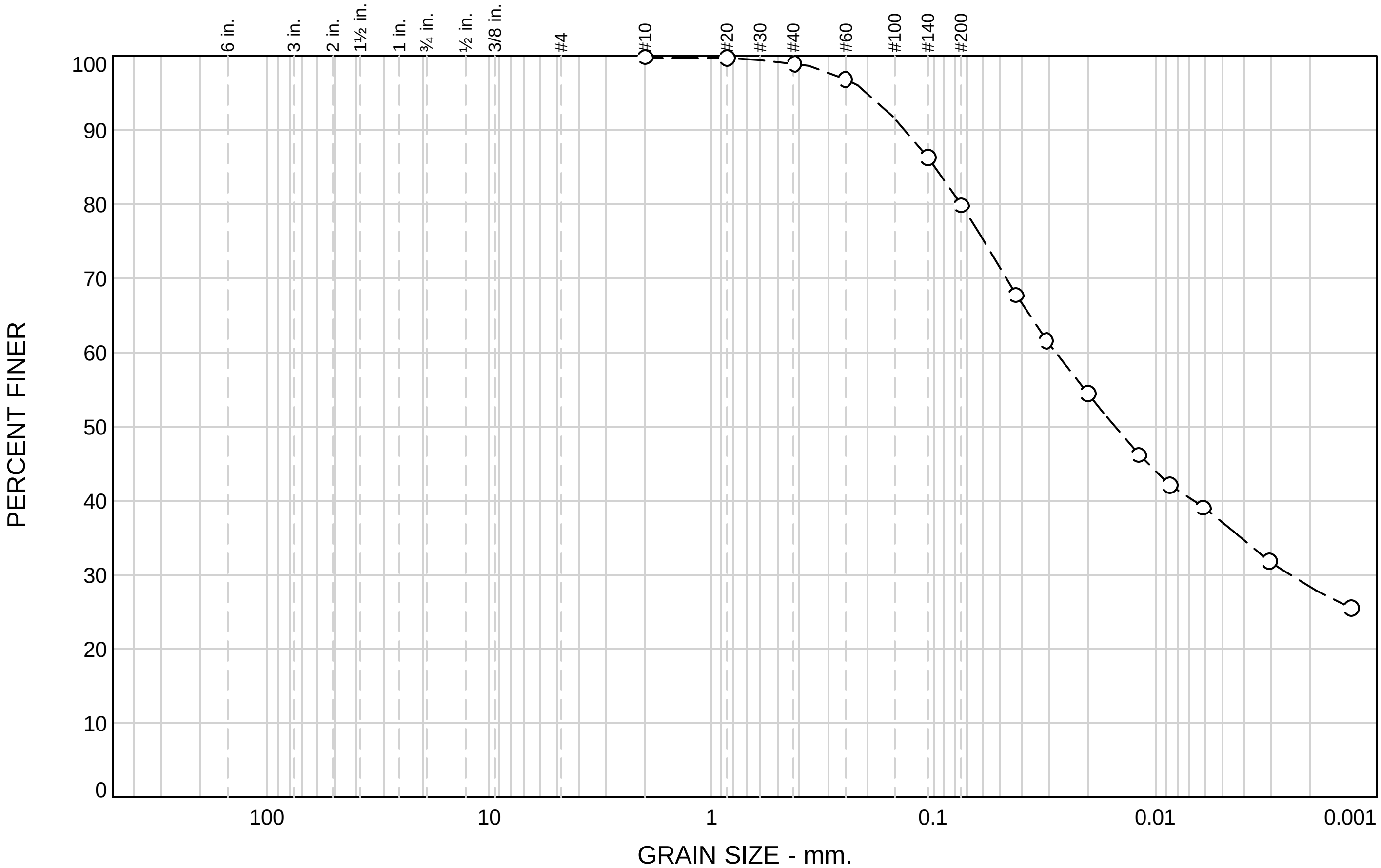
Project: New Stockton Courthouse

Location: B2@46

UNCONFINED COMPRESSION TEST

ENGEO, INCORPORATED

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.8	19.2	51.6	28.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.8		
#40	99.2		
#60	97.0		
#140	86.5		
#200	80.0		
0.0429 mm.	67.9		
0.0311 mm.	61.7		
0.0202 mm.	54.5		
0.0120 mm.	46.3		
0.0086 mm.	42.2		
0.0061 mm.	39.2		
0.0031 mm.	32.0		
0.0013 mm.	25.6		

* (no specification provided)

Soil Description

Grayish green clayey silt with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.0973 D_{60}= 0.0281 D_{50}= 0.0153
D_{30}= 0.0025 D_{15}= D_{10}=
C_u= C_c=

Classification

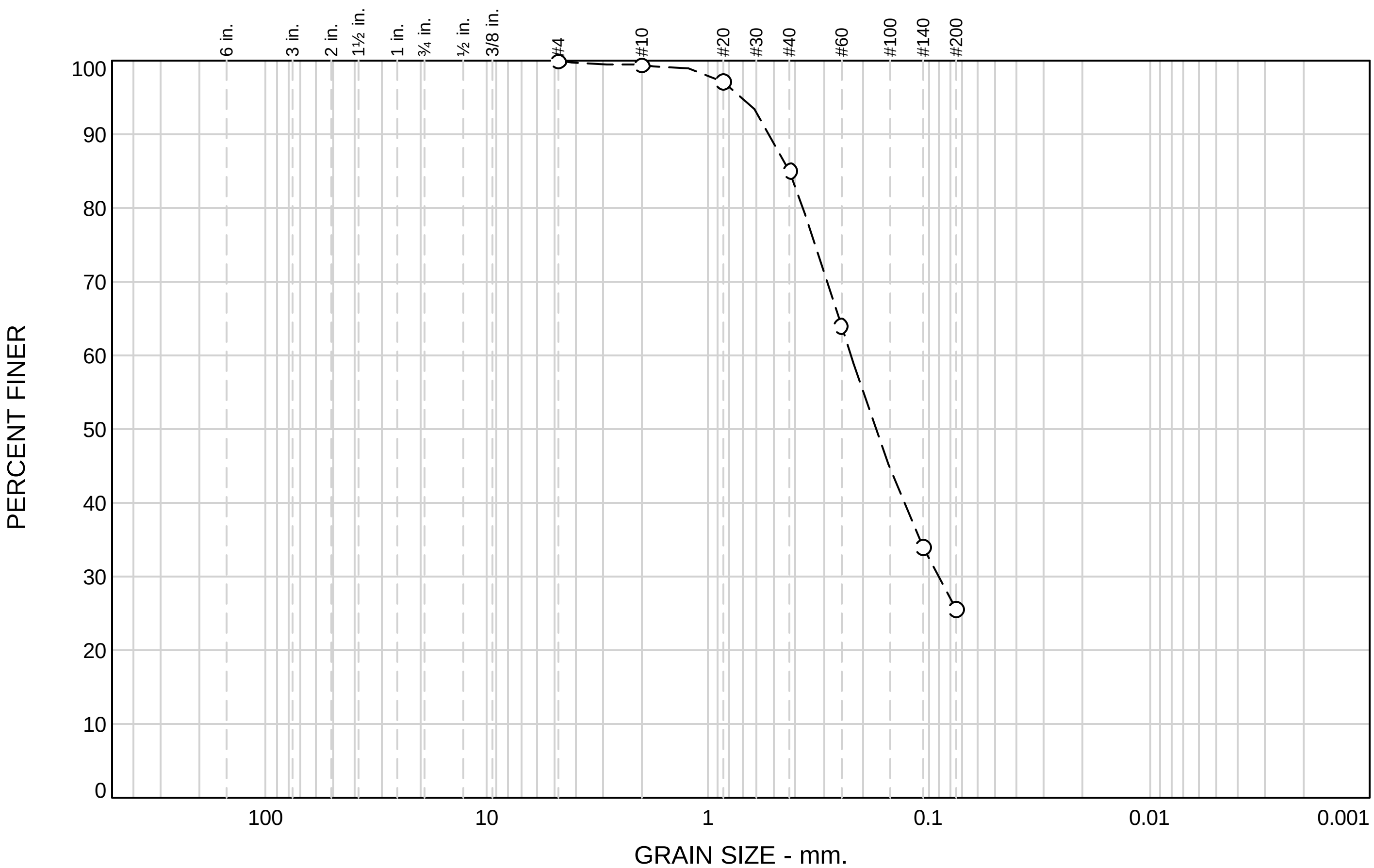
USCS= ML AASHTO=

Remarks

Sample No.: B1@20 **Source of Sample:** GEX **Date:** 2/23/09
Location: **Elev./Depth:** 20 feet

ENGEO, Inc. Rocklin, CA	**Client:** NBBJ **Project:** New Stockton Courthouse **Project No:** 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.5	14.4	59.5	25.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.5		
#20	97.3		
#40	85.1		
#60	64.1		
#140	34.0		
#200	25.6		

* (no specification provided)

Soil Description

Grayish brown silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.4230 D_{60}= 0.2263 D_{50}= 0.1748
D_{30}= 0.0904 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

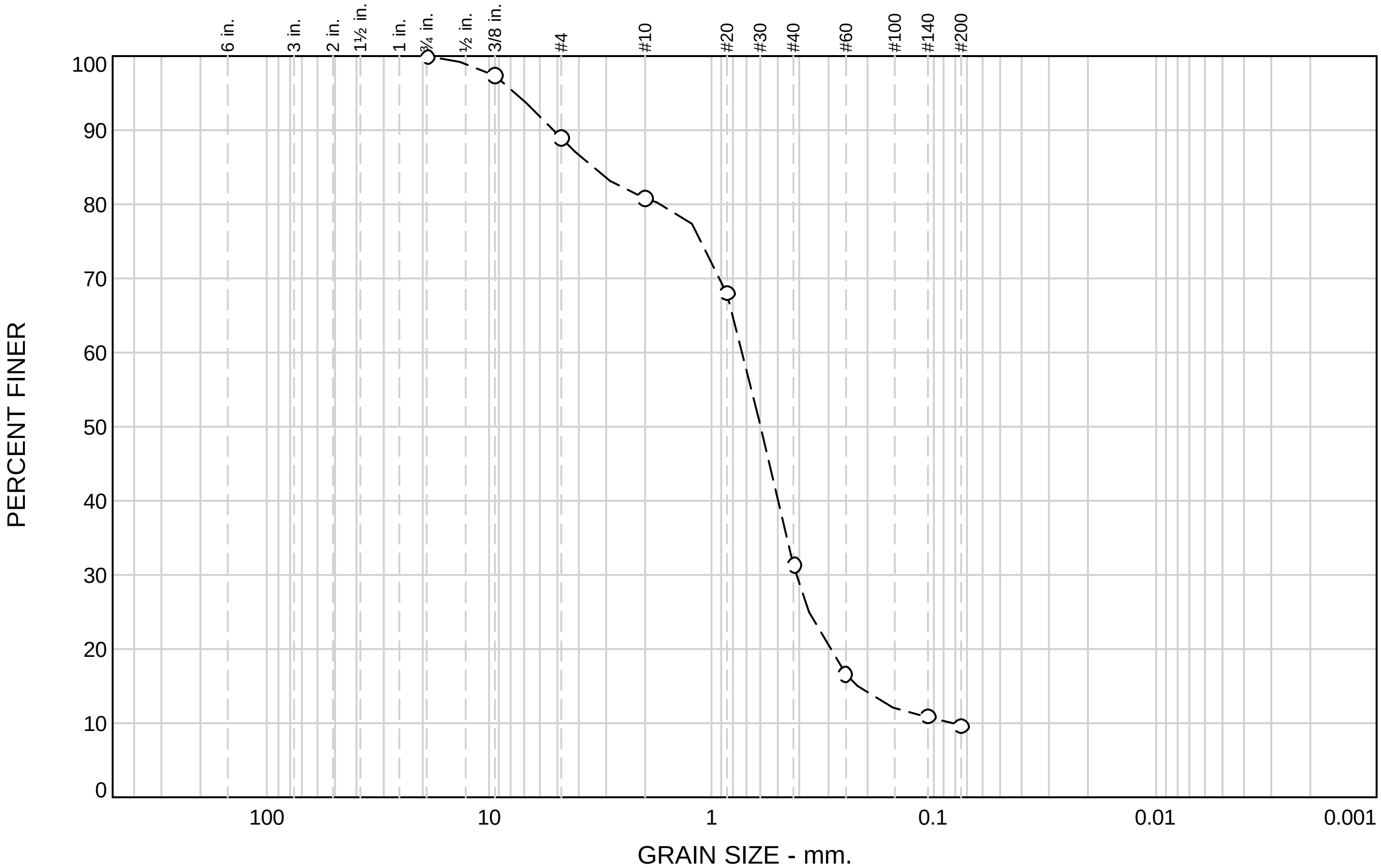
Sample No.: B1@25 **Source of Sample:** GEX **Date:** 2/23/09
Location: **Elev./Depth:** 25 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.0	8.2	49.3	21.8	9.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.375	97.5		
#4	89.0		
#10	80.8		
#20	68.1		
#40	31.5		
#60	16.8		
#140	11.0		
#200	9.7		

* (no specification provided)

Soil Description

Grayish green well graded sand with silt and some gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.3926 D_{60}= 0.7154 D_{50}= 0.5987
D_{30}= 0.4107 D_{15}= 0.2183 D_{10}= 0.0802
C_u= 8.92 C_c= 2.94

Classification

USCS= SW-SM AASHTO=

Remarks

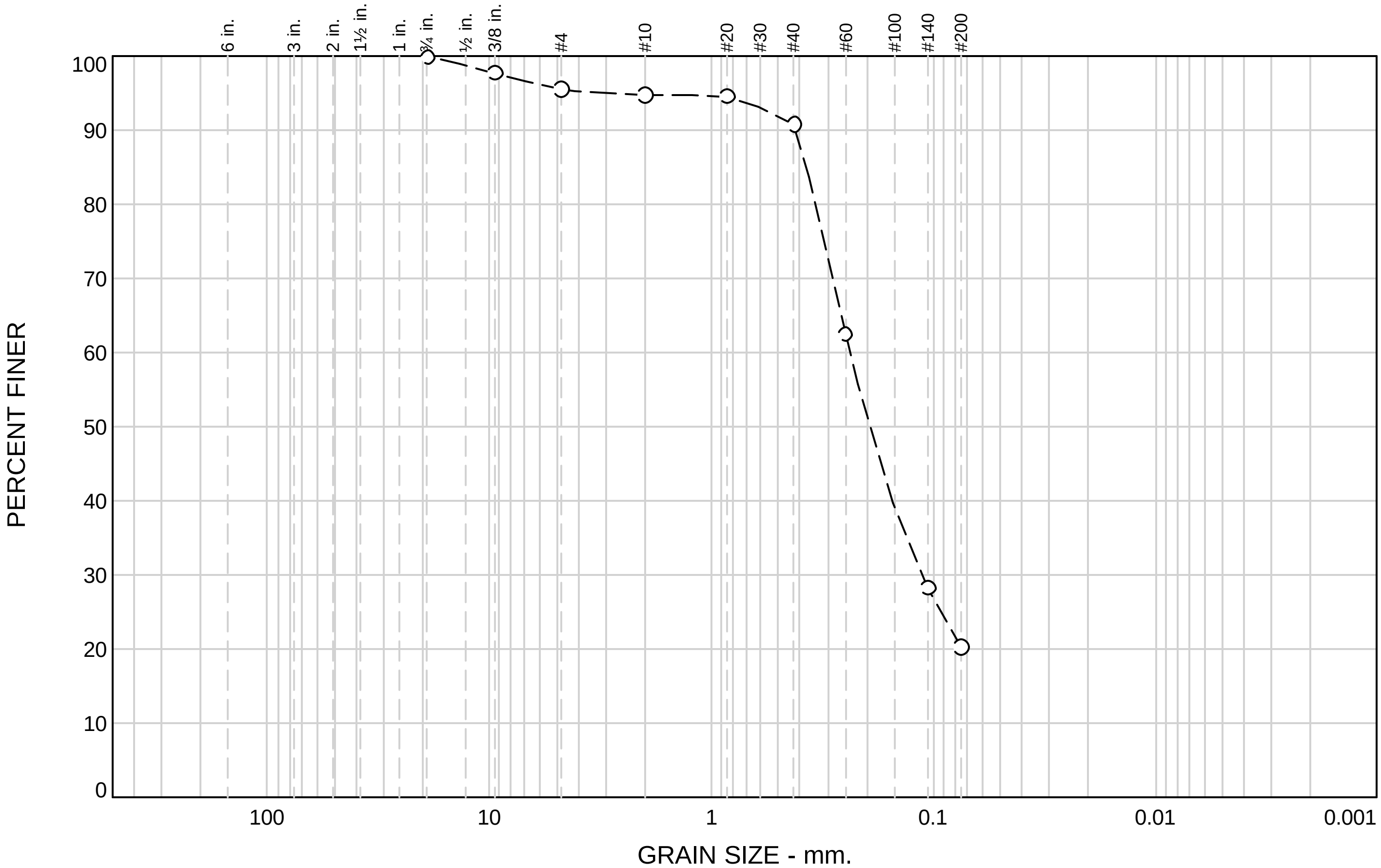
Sample No.: B1@30 **Source of Sample:** GEX **Date:**
Location: **Elev./Depth:** 30 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.4	0.7	4.1	70.5	20.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.375	97.9		
#4	95.6		
#10	94.9		
#20	94.7		
#40	90.8		
#60	62.6		
#140	28.4		
#200	20.3		

* (no specification provided)

Soil Description

Grayish green silty fine sand

Atterberg Limits

PL= LL= SM PI=

Coefficients

D_{85}= 0.3699 D_{60}= 0.2386 D_{50}= 0.1955
D_{30}= 0.1125 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

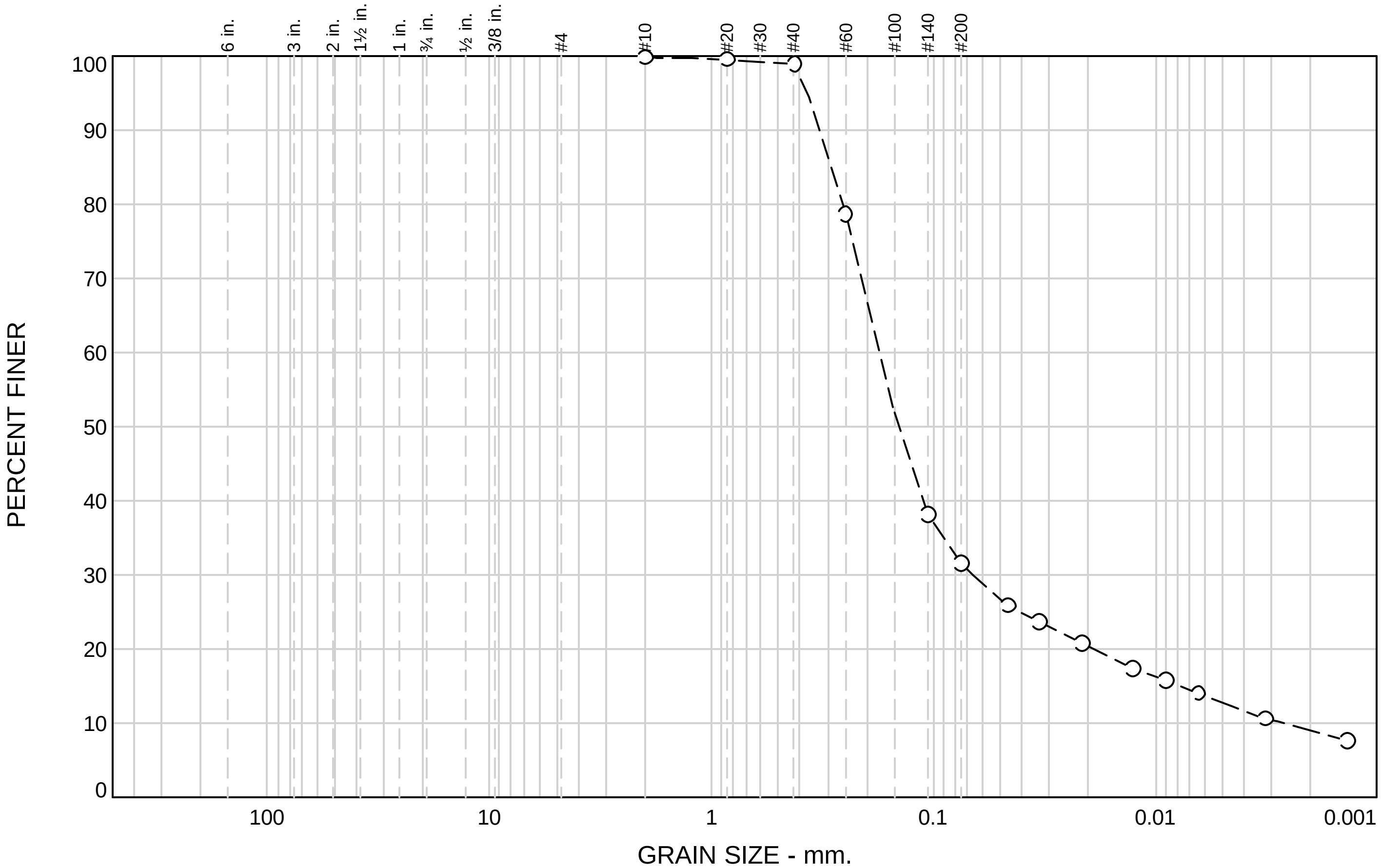
Sample No.: B1@35 **Source of Sample:** GEX **Date:** 2/23/09
Location: **Elev./Depth:** 35 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.9	67.3	22.8	9.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.7		
#40	99.1		
#60	78.7		
#140	38.2		
#200	31.8		
0.0464 mm.	26.0		
0.0333 mm.	23.7		
0.0214 mm.	20.9		
0.0126 mm.	17.5		
0.0090 mm.	15.8		
0.0064 mm.	14.1		
0.0032 mm.	10.7		
0.0014 mm.	7.8		

* (no specification provided)

Soil Description

Grayish green silty fine sand with some clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.2840 D_{60}= 0.1758 D_{50}= 0.1440
D_{30}= 0.0659 D_{15}= 0.0076 D_{10}= 0.0027
C_u= 66.21 C_c= 9.29

Classification

USCS= SM AASHTO=

Remarks

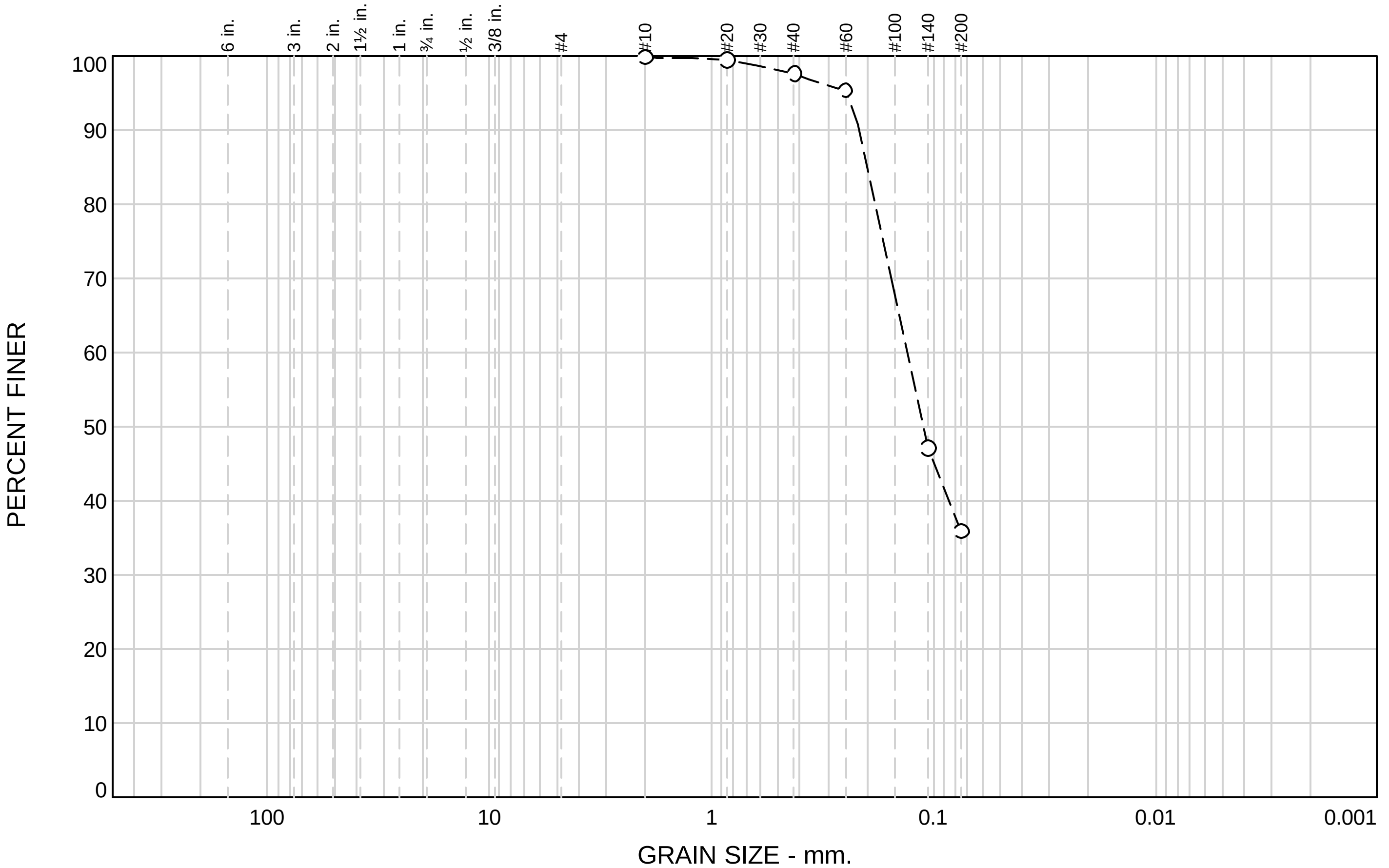
Sample No.: B1@51 **Source of Sample:** GEX **Date:** 2/23/09
Location: **Elev./Depth:** 51 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	2.3	61.7	36.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.7		
#40	97.7		
#60	95.5		
#140	47.2		
#200	36.0		

* (no specification provided)

Soil Description

Grayish green silty fine sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.1969 D_{60}= 0.1333 D_{50}= 0.1123
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

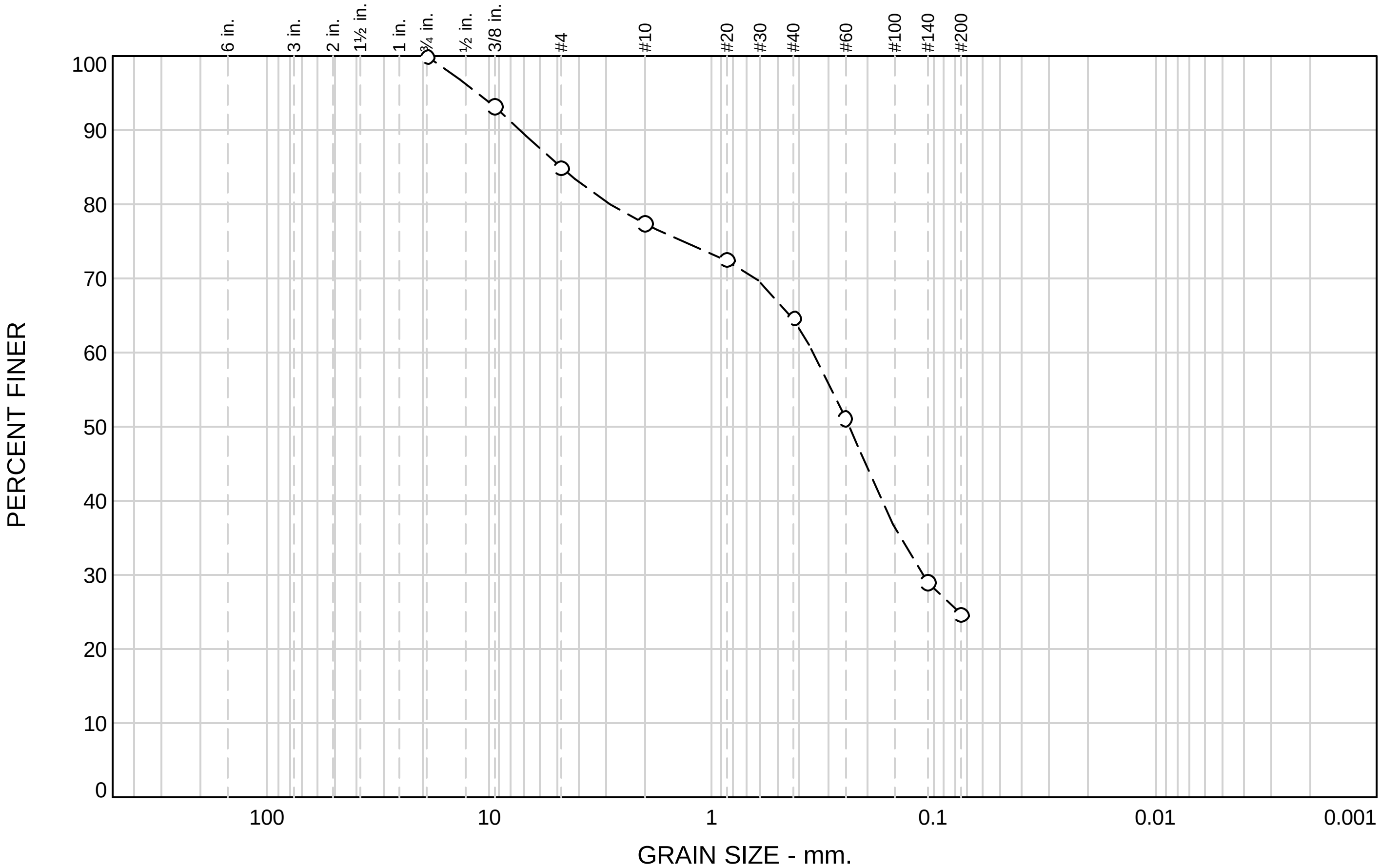
Sample No.: B1@70.5 **Source of Sample:** GEX **Date:** 2/23/09
Location: **Elev./Depth:** 70 1/2 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	15.0	7.5	12.8	40.0	24.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.375	93.2		
#4	85.0		
#10	77.5		
#20	72.6		
#40	64.7		
#60	51.1		
#140	29.0		
#200	24.7		

* (no specification provided)

Soil Description

Grayish green silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 4.7354 D_{60}= 0.3448 D_{50}= 0.2410
D_{30}= 0.1123 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

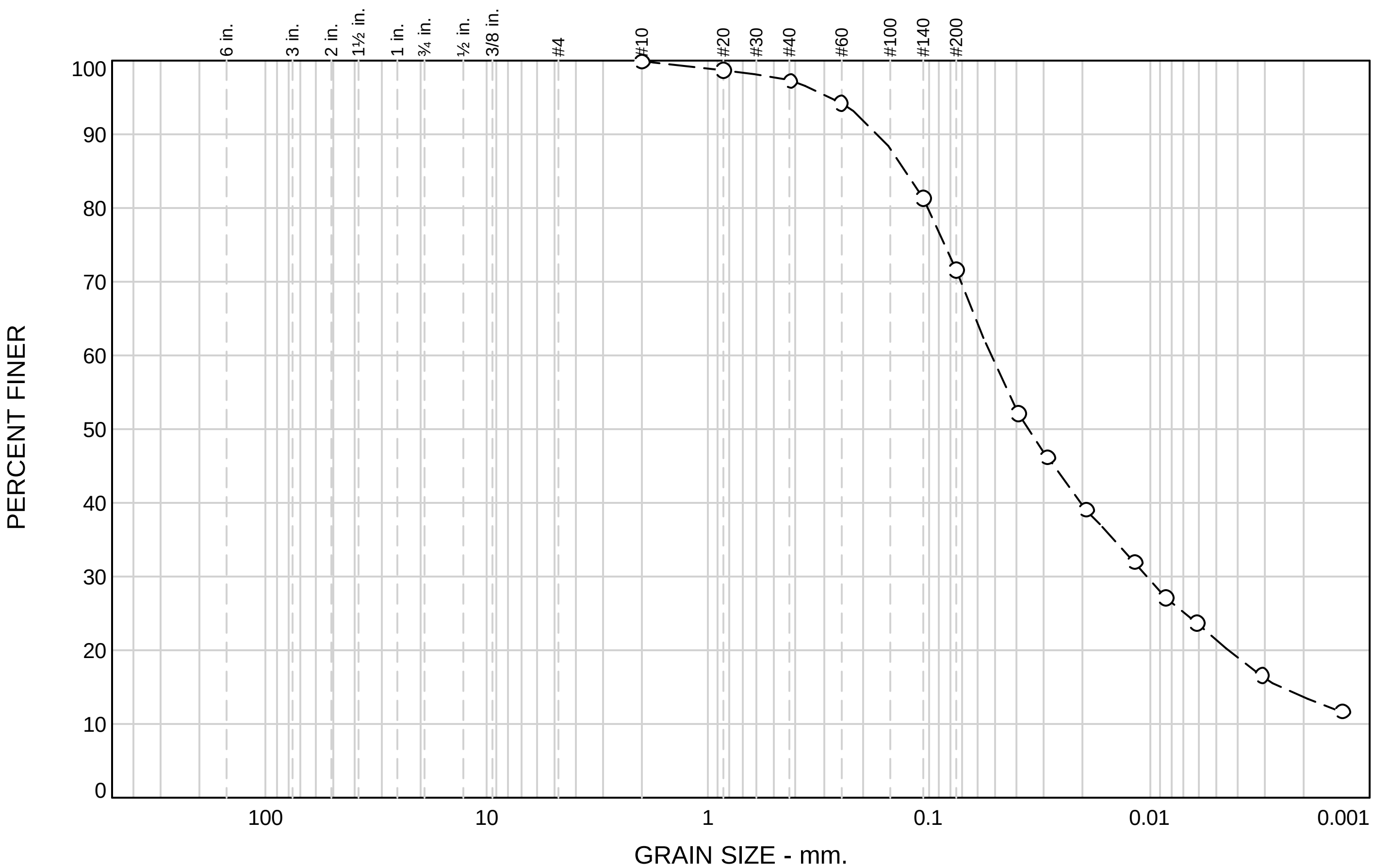
Sample No.: B1@90 **Source of Sample:** GEX **Date:** 2/23/09
Location: **Elev./Depth:** 90 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	2.6	25.7	58.0	13.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	98.9		
#40	97.4		
#60	94.3		
#140	81.4		
#200	71.7		
0.0391 mm.	52.3		
0.0290 mm.	46.3		
0.0193 mm.	39.2		
0.0117 mm.	32.0		
0.0085 mm.	27.3		
0.0061 mm.	23.7		
0.0031 mm.	16.7		
0.0013 mm.	11.8		

* (no specification provided)

Soil Description

Dark yellowish brown sandy silt with clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.1250 D_{60}= 0.0518 D_{50}= 0.0352
D_{30}= 0.0102 D_{15}= 0.0025 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

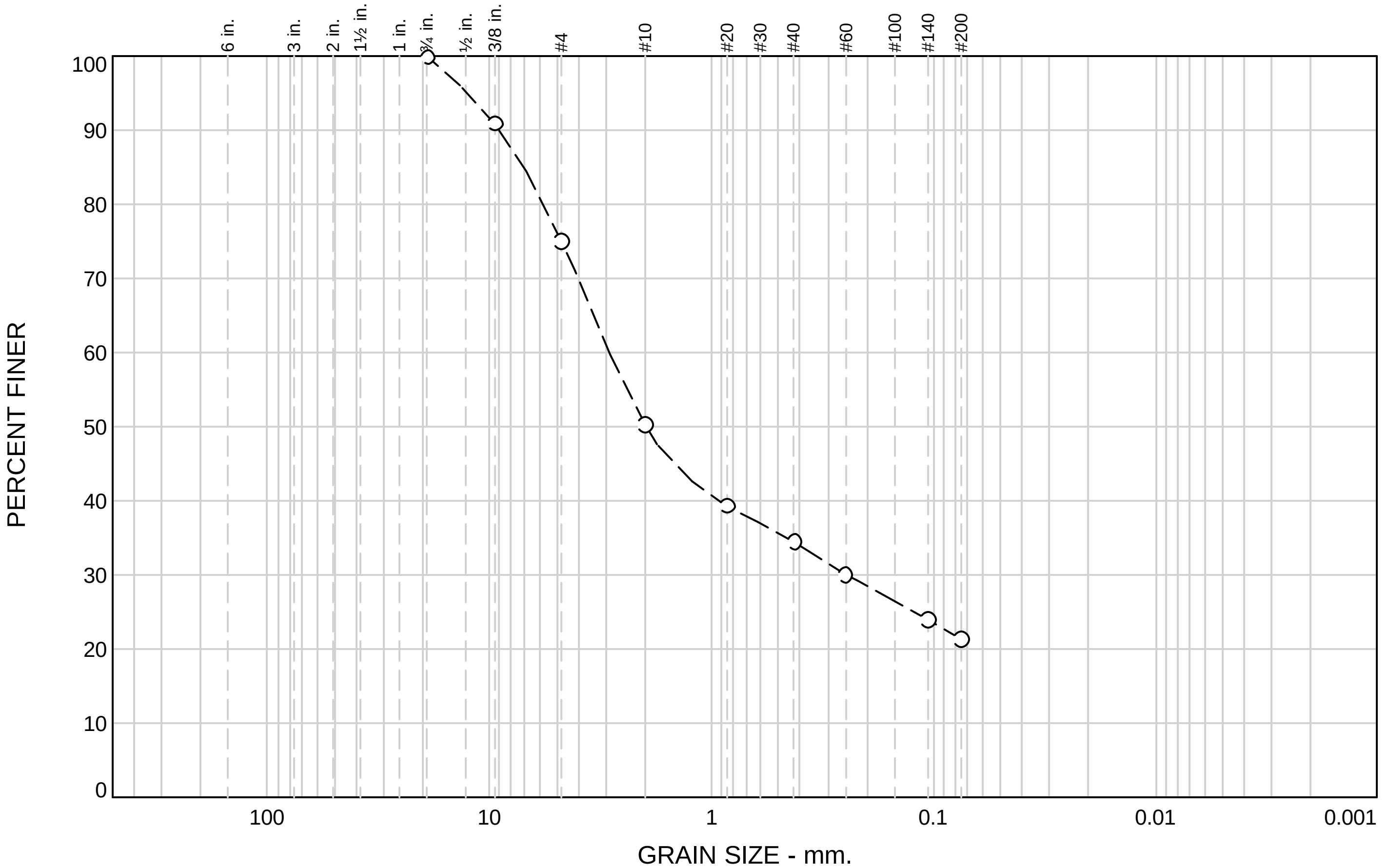
Sample No.: B2@21 **Source of Sample:** GEX **Date:**
Location: **Elev./Depth:** 21 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	24.9	24.8	15.8	13.1	21.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.375	91.0		
#4	75.1		
#10	50.3		
#20	39.5		
#40	34.5		
#60	30.2		
#140	24.1		
#200	21.4		

* (no specification provided)

Soil Description

Dark reddish brown silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 7.0112 D_{60}= 2.8872 D_{50}= 1.9730
D_{30}= 0.2444 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

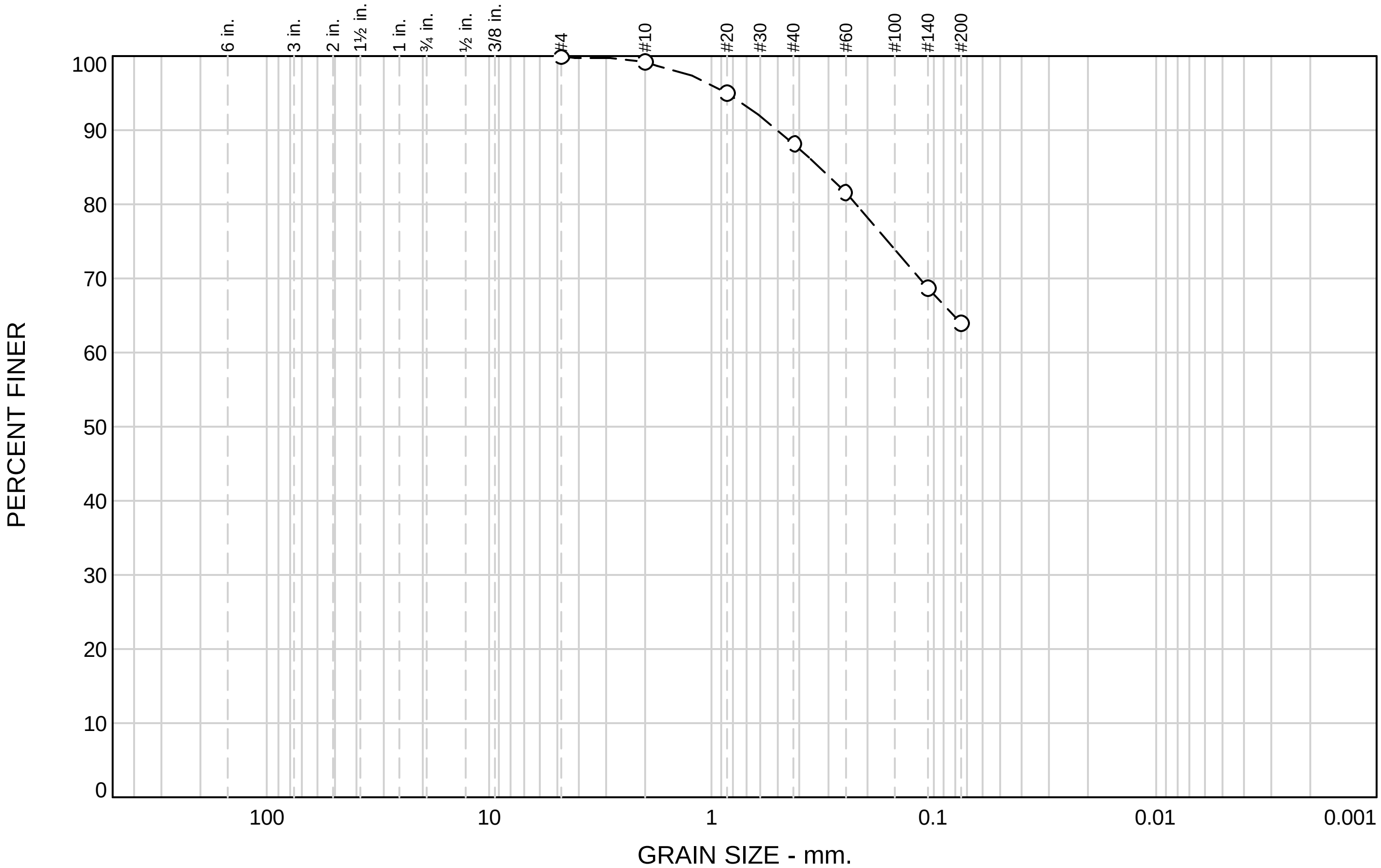
Sample No.: B2@25 **Source of Sample:** GEX **Date:** 3/1/09
Location: **Elev./Depth:** 25 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.8	10.9	24.1	64.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.2		
#20	95.2		
#40	88.3		
#60	81.8		
#140	68.8		
#200	64.2		

* (no specification provided)

Soil Description

Grayish green sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.3207 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

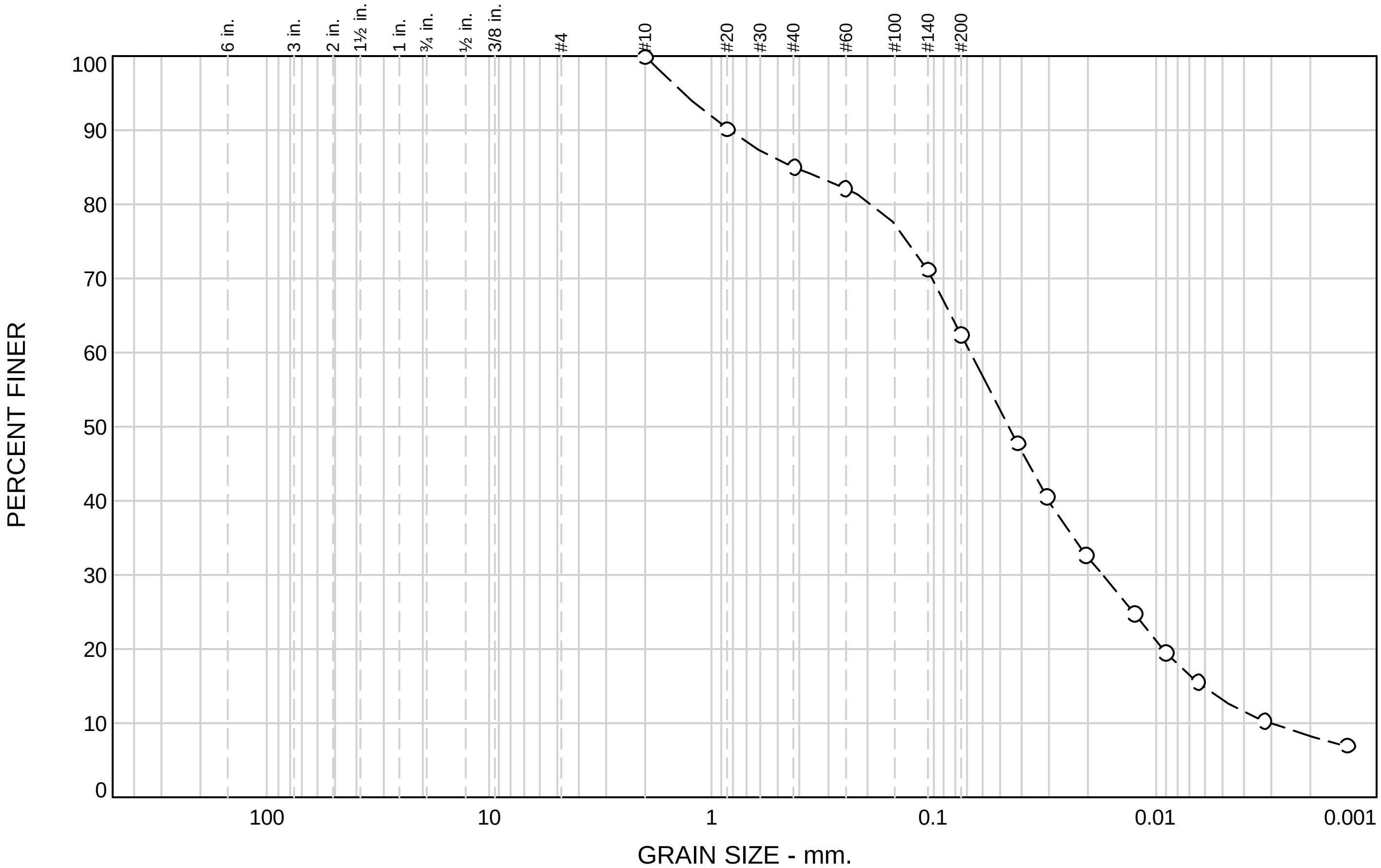
Sample No.: B2@60.5 **Source of Sample:** GEX **Date:** 3/1/09
Location: **Elev./Depth:** 60 1/2 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	14.8	22.7	54.2	8.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	90.3		
#40	85.2		
#60	82.3		
#140	71.3		
#200	62.5		
0.0418 mm.	47.9		
0.0310 mm.	40.6		
0.0205 mm.	32.8		
0.0123 mm.	24.9		
0.0089 mm.	19.7		
0.0064 mm.	15.7		
0.0032 mm.	10.4		
0.0014 mm.	7.1		

* (no specification provided)

Soil Description

Grayish green sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.4122 D_{60}= 0.0682 D_{50}= 0.0456
D_{30}= 0.0172 D_{15}= 0.0060 D_{10}= 0.0030
C_u= 22.95 C_c= 1.45

Classification

USCS= ML AASHTO=

Remarks

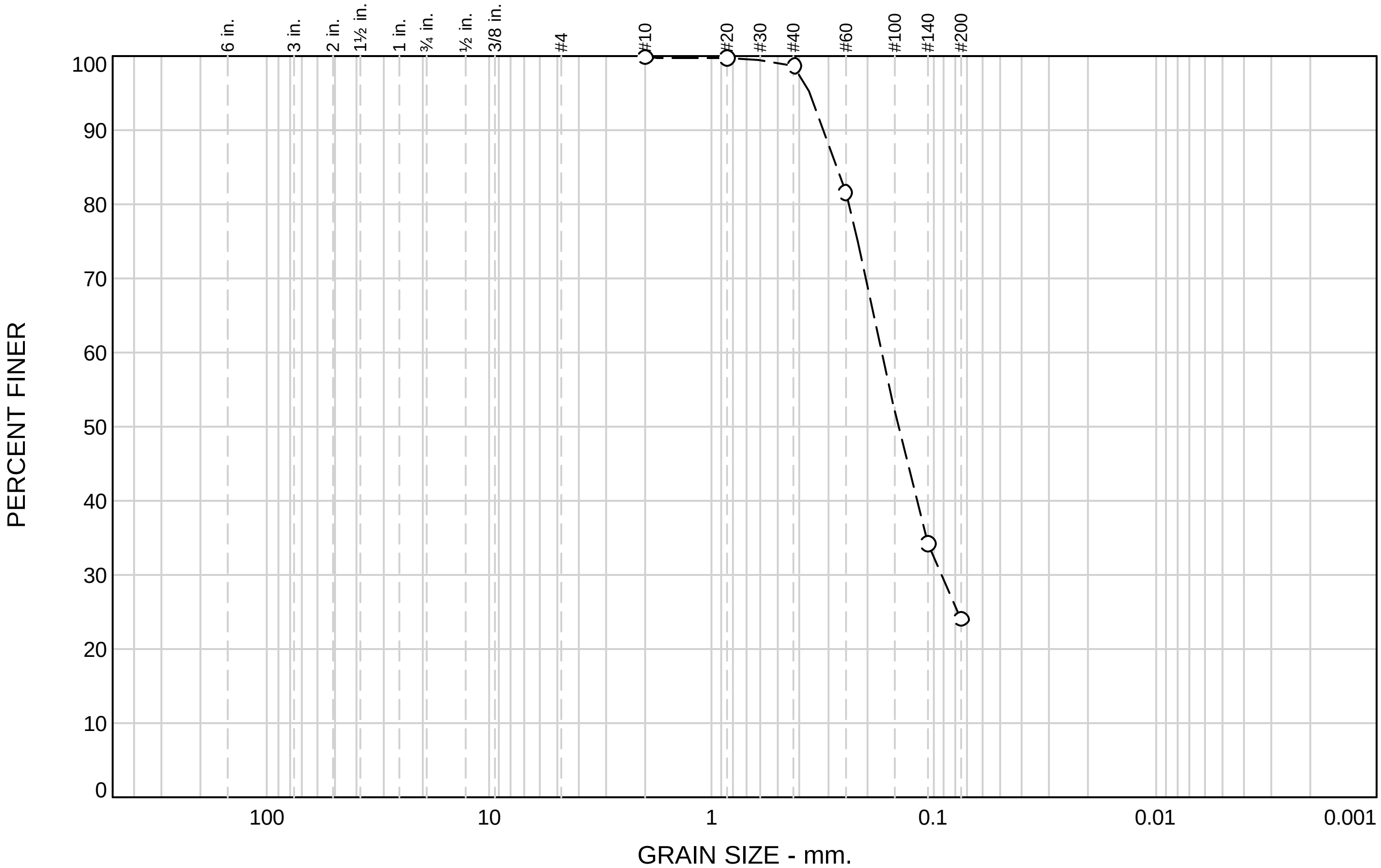
Sample No.: B2@81 **Source of Sample:** GEX **Date:**
Location: **Elev./Depth:** 81 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.2	74.6	24.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.9		
#40	98.8		
#60	81.6		
#140	34.4		
#200	24.2		

* (no specification provided)

Soil Description

Grayish brown silty fine sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.2685 D_{60}= 0.1711 D_{50}= 0.1447
D_{30}= 0.0933 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B2@85.5 **Source of Sample:** GEX **Date:** 3/1/09
Location: **Elev./Depth:** 85 1/2 feet

ENGEO, Inc.

Rocklin, CA

Client: NBBJ
Project: New Stockton Courthouse
Project No: 8641.000.000 **Figure**

ENGEO INCORPORATED

R-VALUE TEST DATA
CAL-301

PROJECT NAME: New Stockton Courthouse

REPORT DATE: 3/9/09

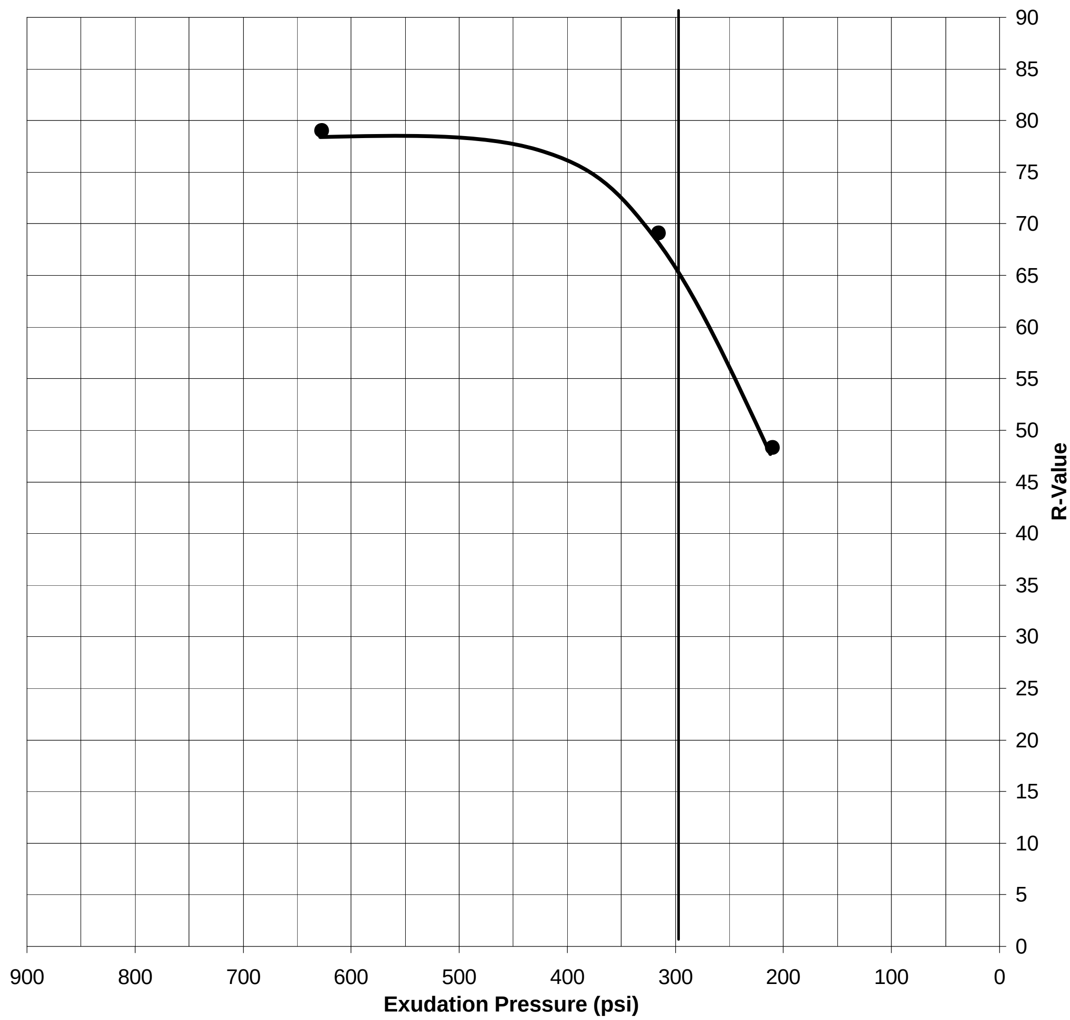
PROJECT NO. 8641.000.000

TESTED BY: SN

SAMPLE LOCATION: B2@0-3

SAMPLE DATE: 3/2/09

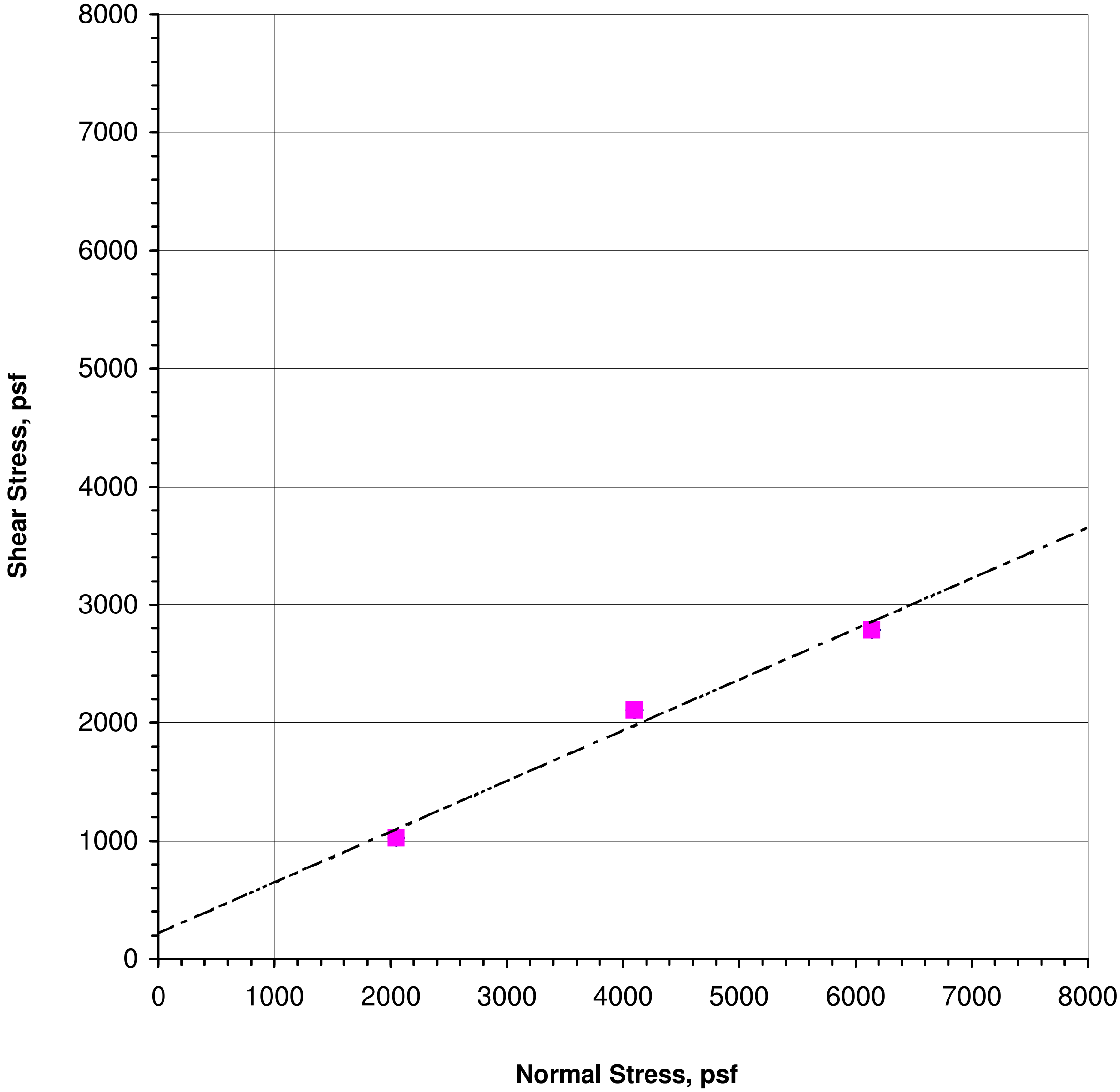
SAMPLE DESCRIPTION: Dark reddish brown silty sand with gravel (SM)



Specimen	A	B	C
Exudation Pressure, p.s.i.	630	318	212
Expansion dial (.0001")	7	6	0
Expansion Pressure, p.s.f.	30	26	0
Resistance Value, "R"	78	68	48
% Moisture at Test	8.8	9.2	9.7
Dry Density at Test, p.c.f.	132.0	131.2	129.5
"R" Value at 300 p.s.i., Exudation Pressure	66		

ENGEO Incorporated, 2213 Plaza Drive, Rocklin CA 95765 (916) 786-8883

Direct Shear Test
ASTM Test Method D3080



	FRICTION ANGLE	COHESION
Peak	23 degrees	212 psf
Softened	23 degrees	212 psf

Dry Density:	95.5 pcf
Moisture Content:	27.8 %
USCS Classification:	CL
Shear Type:	CD
Shear Rate:	Slow

ENGEO INCORPORATED	**NEW STOCKTON COURT HOUSE** **Stockton, California**	**Job No.:**	**8641.000.000**	**Figure No.**
		Sample Number:	**B1@11'**	
		Date:	**3/9/2009**	

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 03/06/2009
Date Submitted 03/03/2009

To: Paul Cottingham
Engeo Inc.
2213 Plaza Dr.
Rocklin, CA 95765

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 8641.000.000 Site ID : B1@16.0.
Thank you for your business.

* For future reference to this analysis please use SUN # 55183-110889.

EVALUATION FOR SOIL CORROSION

Soil pH	6.88		
Minimum Resistivity	2.79	ohm-cm (x1000)	
Chloride	12.3 ppm	00.00123	%
Sulfate	21.5 ppm	00.00215	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 03/06/2009
Date Submitted 03/03/2009

To: Paul Cottingham
Engeo Inc.
2213 Plaza Dr.
Rocklin, CA 95765

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 8641.000.000 Site ID : B2@10.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 55183-110890.

EVALUATION FOR SOIL CORROSION

Soil pH	7.84	
Minimum Resistivity	0.88 ohm-cm (x1000)	
Chloride	12.4 ppm	00.00124 %
Sulfate	161.0 ppm	00.01610 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422