Appendix F

Geotechnical Report and LADBS Approval Letter

CITY OF LOS ANGELES
CALIFORNIA

BOARD OF
BUILDING AND SAFETY COMMISSIONERS

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201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

October 8, 2020

LOG # 113792-01
SOILS/GEOLOGY FILE - 2

Belmont Village, LP
5800 Armada Drive, Suite 200
Carlsbad, CA 92008

TRACT:	Subdivision of Rancho San Jose de Buenos Ayres (MR 26-19/25) // 7803
BLOCK:	18 // 31
LOT(S):	FR 9 Arbs. 1, 4, & 5) // 4 (Arbs. 1 & 2)
LOCATION:	10822 W. Wilshire Boulevard // 10812 W. Ashton Avenue

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Addendum Report	4953-16-0251	09/14/2020	Wood

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Review Letter	113792	07/27/2020	LADBS
Geology/Soils Report	4953-16-0251	06/25/2020	Wood

The Grading Division of the Department of Building and Safety has reviewed the referenced addendum report stating that the referenced geology/soils report dated 06/25/2020 was only intended to satisfy CEQA requirements and not for design purposes. The report included a discussion of potential geologic hazards affecting the proposed development.

The subject property was investigated by the consultant in the referenced report to address a new twelve-story residential tower at the northeastern portion of the property and a two-story church administration building at the southern portion of the property. Two to three levels of subterranean parking were also proposed below the residential tower and church administration buildings. Retaining walls ranging up to 43 feet in height were proposed for the subterranean parking levels. The consultants recommend to support the proposed structures on conventional foundations bearing on native undisturbed soils.

The subject property consists of several consecutive parcels that are developed with a sanctuary building, church office building, and preschool building on the existing church grounds and a one-story residence on Ashton Avenue. The existing structures, except the sanctuary building, will be demolished for the new development. The site slopes gently to the south-southeast about 10 feet in height from Wilshire Boulevard to Ashton Avenue. Subsurface exploration performed by the consultant consisted of four hollowstem-auger borings to a maximum depth of 61½ feet and supplemented with four CPT soundings to a maximum depth of 60 feet. The earth materials at the subsurface exploration locations consist of up to 6½ feet of uncertified fill underlain by alluvium. No groundwater was encountered within the current exploration. However, seepage/groundwater was encountered at depths between 61 and 72½ feet in prior borings on the adjacent property to the east performed by legacy companies of the current consultant.

The subject site is not located within a State or City defined seismic hazard zone. The consultants conclude that the proposed development is feasible from a geotechnical standpoint and that the site is not adversely affected by significant geotechnical issues of hazards.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2020 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. This approval is limited for EIR/CEQA purposes only.

2. The design recommendations presented in the referenced report by Wood, dated 06/25/2020, are <u>not</u> approved at this time.

3. Prior to the issuance of grading/building permits, a design-level engineering geology and geotechnical engineering report shall be provided per the Department requirements and Los Angeles Building Code with appropriate design recommendations and supporting engineering analyses. (P/BC 2020-044, P/BC 2020-049, P/BC 2014-068, P/BC 2020-083, P/BC 2020-113, P/BC 2020-118, P/BC 2017-132)

EDMOND LEE Engineering Geologist Associate III	DAN L. STOICA Geotechnical Engineer I

Log No. 113792-01
213-482-0480

cc: Wood, Project Consultant
WL District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District	Log No.

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 221 N. Figueroa St., 12th Fl., Los Angeles, CA 90012 Telephone No. (213)482-0480.

B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.

C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: MR 26-19/25 117803
Block: Lots: 18/31
3. OWNER: Belmont Village, LP
Address: 5800 Armada Dr., #200
City: Carlsbad Zip: 92008
Phone (Daytime):

2. PROJECT ADDRESS:
10822 W. Wilshire Blvd. // 10812 W. Ashton Ave.
4. APPLICANT Wood.
Address:
City: Zip:
Phone (Daytime):
E-mail address: lan-anh.tran@woodplc.com

5. Report(s) Prepared by: Wood.

6. Report Date(s): 9/14/2020

7. Status of project: ☐ Proposed ☐ Under Construction ☐ Storm Damage

8. Previous site reports? ☐ YES if yes, give date(s) of report(s) and name of company who prepared report(s)

9. Previous Department actions? ☐ YES if yes, provide dates and attach a copy to expedite processing.
Dates:

10. Applicant Signature: Position:

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED		FEES
☐ Soils Engineering		No. of Lots		
☐ Geology		No. of Acres		
☒ Combined Soils Engr. & Geol.		☐ Division of Land		
☐ Supplemental		Other		
☒ Combined Supplemental	363.00	☒ Expedite		181.50
☐ Import-Export Route		☐ Response to Correction		
Cubic Yards:		☐ Expedite ONLY		
		Sub-total		544.50
		Surcharges		129.80
		TOTAL FEE		674.30

Fee Due: 674.30
Fee Verified By: ML Date: 9/16/2020
(Cashier Use Only)

Los Angeles Department of Building and Safety
Metro 4th Floor 09/16/2020 4:23:25 PM
User ID: dbarrozo
Receipt Ref Nbr: 2020260001-139
Transaction ID: 2020260001-139-1
GRADING REPORT $363.00
SYSTEMS DEV SURCH $32.67
GEN PLAN MAINT SURCH $38.12
DEV SERV CENTER SURCH $16.34
CITY PLAN SURCH $32.67
PLAN APPROVAL FEE $181.50
MISC OTHER $10.00
Amount Paid: $674.30
PCIS Number: n/a
Job Address: 10822 W Wilshire Blvd
Owners Name: Belmont Village LP
Grading Section Log Number: na

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED
☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology Date

For Soils Date

Report of Geotechnical Investigation

Proposed Belmont Village - Westwood

10822 Wilshire Boulevard and 10812 Ashton Avenue
Westwood District
Los Angeles, California
Lot 9 and Subdivision of Rancho San Jose De Buenos Ayres and Lot 4 and Tract 7803

Prepared for:

Belmont Village, L.P.
Carlsbad, California

Project 4953-16-0251

May 6, 2016
(Revised April 18, 2019)

Wood Environment & Infrastructure Solutions, Inc.
6001 Rickenbacker Road
Los Angeles, CA 90040-3031
USA
T: +1 323.889.5300
www.woodplc.com

May 6, 2016
Revised April 18, 2019
Wood Project 4953-16-0251

Mr. Stephen Brollier
EVP/Chief Development Officer
Belmont Village, L.P.
5800 Armada Drive, Suite 200
Carlsbad, California 92008

Subject: **Letter of Transmittal**
Report of Geotechnical Investigation
Proposed Belmont Village - Westwood
10822 Wilshire Boulevard and 10812 Ashton Avenue
Westwood District, Los Angeles, California
Lot 9 and Subdivision of Rancho San Jose De Buenos Ayres and
Lot 4 and Tract 7803

Dear Mr. Brollier:

We are pleased to submit the results of our geotechnical investigation for the proposed development at Westwood Presbyterian Church located at 10822 Wilshire Boulevard and the adjacent residential property at 10812 Ashton Avenue in the Westwood District of Los Angeles, California. This investigation was conducted in general accordance with our proposal dated February 19, 2016, which was authorized on March 7, 2016.

The scope of our services was planned with Mr. Brent Covey of Belmont Village, L.P. and he provided us a site plan and architectural plans of the proposed project. In addition, the structural engineer of the project, Mr. Lawrence Ho of Englekirk Structural Engineers, provided us with the design column loads of the proposed structures. This report was revised from our original report dated May 6, 2016 prepared under the name of our predecessor company Amec Foster Wheeler Environment & Infrastructure, Inc. based on comments from your land use attorney and design standard changes.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the City of Los Angeles Department of Building and Safety for their review and approval prior to obtaining a building permit.

It has been a pleasure to be of professional service to you. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,

Wood Environment & Infrastructure Solutions, Inc.

REGISTERED PROFESSIONAL ENGINEER
LARRY CHIH CHAO HONG
NO. 80989
CIVIL
STATE OF CALIFORNIA

Larry Hong
Senior 1 - Geotechnical

REGISTERED PROFESSIONAL ENGINEER
EUNG JIN JEON
3096
GEOTECHNICAL
STATE OF CALIFORNIA

Eung Jin Jeon, Ph.D.
Associate Engineer

PROFESSIONAL GEOLOGIST
ROSALIND MUNRO
NO. 1269
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Rosalind Munro
Principal Engineer Geologist

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
MARTIN BYRD HUDSON
2570
GEOTECHNICAL
EXP. 12-31-2019
STATE OF CALIFORNIA

Martin B. Hudson, Ph.D.
Principal Engineer

P:\4953 Geotech\2016-proj\160251 Belmont Village - Westwood\4.0 Project Deliverables\4.1 Reports\Final Report\4953-16-0251r02.docx\EJJ:MBH

(Electronic copies submitted)

Report of Geotechnical Investigation
Proposed Belmont Village - Westwood

10822 Wilshire Boulevard and 10812 Ashton Avenue
Westwood District
Los Angeles, California
Lot 9 and Subdivision of Rancho San Jose De Buenos Ayres and
Lot 4 and Tract 7803

Prepared For:

Belmont Village, L.P.

Carlsbad, California

Wood Environment & Infrastructure Solutions, Inc.
Los Angeles, California

May 6, 2016, Revised April 18, 2019

Project 4953-16-0251

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Appendices

Executive Summary

We have performed a geotechnical investigation for the proposed development at Westwood Presbyterian Church located at 10822 Wilshire Boulevard and the adjacent residential property at 10812 Ashton Avenue in the Westwood District of Los Angeles, California. Our subsurface explorations, engineering analyses, and foundation design recommendations are summarized below.

The proposed development includes the construction of a 12-story Belmont Village Tower (Tower) in the northeastern portion of the site and a new church office/preschool building (Educational Center) in the southern portion of the site. A plaza separating the two buildings is planned in the center of the development. The entire development will be underlain by two to three levels of subterranean parking. The basement excavation is anticipated to extend to depths ranging from 30 to 43 feet from the south side to the north side of the site.

The soil conditions beneath the site were explored by drilling four borings to depths ranging from 51 to 61½ feet below the existing grade and performing four Cone Penetration Tests (CPTs) to a depth of 60 feet below the existing grade. Fill soils, up to about 6½ feet thick, were encountered in all four of our recent exploration borings. The fill soils consist predominantly of silty sand with some fine gravels. The natural soils beneath the site consist predominantly of interbedded clay, silt, silty sand, clayey sand, and sand. Varying amounts of fine to coarse gravel were encountered in the sandy deposits.

Groundwater was not encountered in our recent borings to the maximum depth of 61½ feet below the existing grade. However, groundwater was measured between depths of 60½ to 72½ feet below grade within the 103-foot depth explored in the prior borings for the adjacent high-rise condominium building. According to the California Geological Survey (CGS), the historic-high groundwater level is about 25 feet below the existing grade.

The corrosion studies indicate that the on-site soils are severely corrosive to ferrous metals, non-aggressive to copper, and that the potential for sulfate attack on portland cement concrete is considered negligible. Measures to address corrosion potential to ferrous metals, if used in project construction, should be identified as part of final project design plans.

Based on the available geologic data, faults with the potential for surface fault rupture are not directly beneath nor trending toward the site. Therefore, the potential for surface rupture at the site due to fault plane displacement propagating to the surface is considered low. The location of the site relative to known active or potentially active faults indicates the site could be subjected to significant ground shaking, however impacts would not be significant through compliance with recommendations in this report and code requirements.

The site is located in a City of Los Angeles methane buffer zone, therefore, the potential exists for the presence of volatile gases during and after construction. Soil gas testing should be performed at the site, and based on the results, an appropriate soil gas mitigation system should be designed for the project in conformance with all City of Los Angeles regulatory requirements.

There is a potential for liquefaction in the medium dense silty sand, sand, and sandy silt layers beneath the site; we estimate that the liquefaction settlement could be on the order of ¾ inch or less beneath the planned foundation level. Through incorporation of the recommendations in this report, liquefaction-related impacts would not be significant.

The site is relatively level and the potential for slope stability hazards is considered low. The potential for other geologic hazards such as seismically-induced settlement (above the groundwater level), lateral spreading, subsidence, flooding, tsunamis, inundation, or seiches affecting the site is considered low.

The existing fill soils at grade are not considered suitable for support of the proposed structure, floor slabs on grade, pavement, or other exterior concrete walks and slabs on grade. However, the excavation for the planned basement level is anticipated to automatically remove the existing fill soils. Accordingly, the proposed structure may be supported on conventional spread/continuous footings established in the undisturbed natural soils at the planned basement level.

The excavation of the basement is anticipated to extend to depths ranging from 30 to 43 feet below existing grade, which is below the historic-high groundwater level at the site. The excavation will not extend below the current groundwater level (based on recent and prior explorations at the site or at the adjacent property), but some minor seepage should be anticipated in the excavation, and minor dewatering consisting of gravel-filled trenches installed where necessary, should be anticipated. Any such dewatering will be required to comply with existing water quality regulatory requirements. A permanent subdrain system will need to be designed for the basement or the footings and basement walls will need to be waterproofed and be designed to support hydrostatic pressure.

If the grading recommendations contained herein are implemented, floor slabs may be supported on grade at the planned basement level. However, for support of pavement or other at-grade exterior concrete walks and slabs on grade, we recommend that all existing fill soils be excavated and replaced as properly compacted fill. As an alternative to removal of the existing uncertified fill soils beneath pavement or other at-grade exterior concrete walks and slabs-on-grade, some of the fill could be left in place; however, the existing fill may be susceptible to settlement in the event of wetting or seismic ground shaking, and the magnitude of such settlement is difficult to predict and would be variable. Therefore, if the potential for some settlement and greater than normal maintenance is acceptable, only the upper 2 feet of existing fill soils need be removed and replaced as properly compacted fill beneath pavement or other at-grade exterior concrete walks and slabs on grade.

1.0 Scope

This report provides an assessment of geologic conditions as well as foundation design information and geotechnical recommendations for the proposed Belmont Village project at the Westwood Presbyterian Church located at 10822 Wilshire Boulevard and the residential property at 10812 Ashton Avenue in the Westwood District of Los Angeles, California. The location of the project site is illustrated on Figure 1, Vicinity Map. The locations of the proposed project and our recent and relevant prior exploration borings are shown on Figure 2, Plot Plan.

We previously performed geotechnical investigations for the condominium building immediately adjacent to the east side of the project site, the results of which were submitted in the following reports:

- Report of Geotechnical Investigation: Proposed High-Rise Condominium Development; Wilshire Boulevard and Malcolm Avenue, West Los Angeles, California; Prepared for Dr. Morie Hirose, our Project No. ADE-79321 (performed under the name of LeRoy Crandall and Associates, a Wood legacy company); report dated November 4, 1980.

- Report of Supplement Geotechnical Investigation: Proposed High-Rise Condominium Building Development; 10808 Wilshire Boulevard, Los Angeles, California; Prepared for Fifield Companies, our Project No. 70131-2-0276 (performed under the name of MACTEC Engineering and Consulting Inc., a Wood legacy company); report dated January 31, 2003.

We have reviewed the above two prior reports and we agree with the interpretation of geotechnical conditions provided in those reports, and accept responsibility for the use of the data contained therein.

This investigation was authorized to determine the physical characteristics of the soils at the project site, and to provide recommendations for design of new foundations and walls below grade, for floor slabs, for temporary shoring, and for grading for the project. More specifically, the scope of this investigation included the following:

- Review of the recent and prior subsurface explorations and laboratory tests, and provide a description of the soil and groundwater conditions encountered.

- Perform a limited geologic-seismic hazards evaluation.

- Provide recommendations for appropriate foundation systems together with the necessary design parameters, including frictional resistance, passive resistance, and the anticipated total and differential settlements.

- Provide seismic design parameters based on the current California Building Code (CBC).

- Provide recommendations for subgrade preparation and floor slab support.

- Provide recommendations for design of temporary shoring.

- Provide recommendations for design of walls below grade.

- Provide recommendations for grading, including site preparation, excavation and slopes, the placing of compacted fill, and quality control measures relating to earthwork.

The scope of this consultation did not include the assessment of general site environmental conditions for the presence of contaminants in the soils or groundwater of the site.

Our recommendations are based on the results of our recent and previous field explorations, laboratory tests, and appropriate engineering analyses. The results of our recent field explorations and laboratory tests, which, together with the data obtained during our previous investigations nearby the project site, form the basis of our recommendations, are presented in Appendix A. The results of our previous field explorations and laboratory tests are presented in Appendix B.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report. This report has been prepared for Belmont Village, L.P. and their design consultants to be used solely in the design of the proposed project. This report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses

2.0 Site Conditions and Project Description

The site is located at the property of the Westwood Presbyterian Church located at 10822 Wilshire Boulevard and the adjacent residential property at 10812 Ashton Avenue in the Westwood District of Los Angeles, California. Presently, the site is occupied by a sanctuary building, a 2-level church office building, and a preschool building with an outdoor play yard; the existing buildings are situated along the western property line of the site. The existing grade of the site, which consists predominantly of asphalt paving, gently slopes from north to south from Elevation 333 to 322. Other existing surface features include some minor landscaping elements in various locations throughout the site. The property at 10812 Aston Avenue is currently occupied by a 1-story single family home, with yards in front and back.

The proposed development includes the construction of a 12-story Belmont Village Tower (Tower) in the northeastern portion of the site and a new church office/preschool building (Educational Center) in the southern portion of the site. A plaza separating the two buildings is planned in the center of the development. Parking for the project will be provided in two to three levels of subterranean parking. The lowest basement floor will be established roughly at Elevation 291 feet - 10 ½ inches. The Tower will have a footprint area of 14,655 square feet. Due to the difference in ground elevation at the site, the ground surface elevation adjacent to the south side of the Tower will coincide with the finish floor elevation of Ground Level (Elevation 321 feet - 10 ½ inches) and the ground surface elevation on the north side will coincide with the finish floor elevation (Elevation 333 feet - 10½ inches) of Level 1. The footprint of the Educational Center will be 10,114 square feet. The basement excavation is anticipated to extend to depths ranging from 30 to 43 feet from the south side to the north side of the site, respectively.

It is our understanding that the church and preschool will continue to operate through the construction, therefore, the proposed buildings will be completed in two phases (Phase I and II). In Phase I, the Educational Center and a portion of the basement parking garage will be constructed; the completion of which will be followed by relocating the church office and preschool into the new building and the demolition of the existing buildings, except the Sanctuary building. Once this is completed, Phase II of the project will commence, in which the new 12-story building and the remainder portion of the underground parking will be constructed.

Based on the current design, we were provided the following structural dead-plus-live column and wall loads:

- Columns (Belmont Village Tower): 2,100 kips
- Basement Wall (Belmont Village Tower): 10 kips per foot
- Columns (Plaza): 450 kips
- Basement Wall (Plaza): 8 kips per foot
- Columns (Educational Center): 560 kips
- Basement Wall (Educational Center): 8 kips per foot.

3.0 Field Explorations and Laboratory Tests

The soil conditions beneath the site were explored by drilling four borings to depths of 51 to 61½ feet below the existing grade and performing four cone penetration tests (CPTs) to the depth of 60 feet below the existing grade. Data were also available from our prior investigation for the development of the existing condominium tower located immediately adjacent to the east of the project site. The boring locations of our recent and prior investigations as well as the CPT locations are illustrated on Figure 2. Details of the recent explorations and the logs of the borings are presented in Appendix A, the logs of the prior borings are presented in Appendix B, and the results of the recent CPTs are presented in Appendix C.

Laboratory tests were performed on selected samples obtained from the recent borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations.
- Direct Shear.
- Consolidation.
- Fines Content.
- Atterberg Limits.
- Corrosivity

All testing was performed in general accordance with applicable ASTM specifications at the time of testing. Details of the recent laboratory testing program and test results are presented in Appendix A. The prior laboratory test results are presented in Appendix B.

4.0 Limited Geologic-Seismic Hazards Evaluation

4.1 Geologic Setting

The site is located in the northern portion of the Los Angeles Basin. This basin is a major elongated northwest-trending structural depression that has been filled with sediments up to 13,000 feet thick since middle Miocene time. On a regional scale, the site is located within the boundary between the Transverse Ranges and Peninsular Ranges geomorphic provinces. The Transverse Ranges Geomorphic Province is characterized by east-west trending mountain ranges that include the Santa Monica Mountains. The southern boundary of the Transverse Ranges Geomorphic Province is marked by the Malibu Coast, Santa Monica, Hollywood, Raymond, Sierra Madre, and Cucamonga faults. The Peninsular Ranges Geomorphic Province is characterized by northwest/southeast trending alignments of mountains and hills and intervening basins, reflecting the influence of northwest trending major faults and folds controlling the general geologic structural fabric of the region. A northern boundary of the Peninsular Ranges Geomorphic Province is the Hollywood fault zone, located approximately 2.2 miles northeast of the site.

4.2 Geologic Conditions

Locally, the site is situated on an alluvial fan south of the Santa Monica Mountains (Dibblee and Ehrenspeck, 1991). The elevation of the site is approximately 320 feet (NGVD 29). The site in relation to local topographic features is shown in Figure 1, Vicinity Map.

Based on the geologic materials encountered in our recent exploratory borings, the site is locally mantled with artificial fill to a depth of approximately 6½ feet. The fill soils consist predominantly of silty sand with some fine gravels. Records are not available documenting the placement and compaction of the fill soils encountered. Deeper and/or poorer quality fill could occur between our borings and in other unexplored areas, particularly in areas where existing structures and underground utilities are present. However, it is expected the majority of the existing fill soils will be automatically removed by the planned basement excavation.

The site is underlain by late Pleistocene-age alluvial deposits generally consisting of interbedded clay, silt, silty sand, clayey sand, and sand. Varying amounts of fine to coarse gravel were encountered in the sandy deposits. The sandy soils were generally medium dense to very dense. The fine-grained soils were generally medium stiff to hard.

The site is located in the Santa Monica Subbasin of the Coastal Plain of Los Angeles Groundwater Basin (California Department of Water Resources, 2003). Groundwater was not encountered in any of our recent borings to a maximum depth of 61½ feet bgs. However, groundwater was measured between the depths of 60½ to 72½ feet below grade within the 103-foot depth explored in the prior borings for the adjacent high-rise condominium building. According the California Geological Survey (CGS), the historical high groundwater level is approximately 25 feet bgs (CDMG, 1998).

The corrosion studies indicate that the on-site soils are severely corrosive to ferrous metals, non-aggressive to copper, and that the potential for sulfate attack on portland cement concrete is considered negligible. Measures to address corrosion potential to ferrous metals, if used in project construction, should be identified as part of final project design plans.

4.3 Seismic Hazards

The closest active fault with the potential for fault surface rupture is the Santa Monica fault, located 0.6 mile south of the site (USGS/CGS, 2006, accessed 2019; Jennings and Bryant, 2010). The site is not within a currently established Alquist-Priolo Earthquake Fault Zone (A-P Zone) for surface fault rupture hazard. The closest A-P Zone has been established for the Santa Monica fault zone, which is located approximately 0.5 miles south of the site (CGS, 2018). Based on the available geologic data, the potential for surface rupture due to fault plane displacement propagating to the surface at the site is considered low.

Liquefaction potential is greatest where the ground-water level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases. Groundwater was not encountered in our recent borings to the maximum depth drilled of 61½ feet. Groundwater was encountered in our prior borings between 60½ to 72½ feet below grade. The historical high groundwater level is approximately 25 feet bgs according to the CGS (CDMG, 1998). According to the City of Los Angeles and the CGS, the site is not within an area with a potential for liquefaction (City of Los Angeles, 2016; CDMG, 1999). However, we have computed the potential for liquefaction-induced settlement beneath the historic-high groundwater level in accordance with the methodology of Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). Based on the results of our analyses, we estimate that liquefaction-induced settlement could be on the order of ¾ inch or less beneath the foundation level, which is acceptable per the City of Los Angeles Department of Building and Safety (LADBS).

Seismically-induced settlement is often caused by loose to medium-dense granular soils becoming denser during ground shaking. The natural soils above the historic-high groundwater level at the site are not susceptible to seismically-induced settlement, therefore, the potential for additional seismically induced settlement (in addition to the liquefaction-induced settlement described above) is considered low.

According to the City of Los Angeles (2016) and the CGS (2018), the site is not within an area identified to have a potential for seismic slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Topographically, the site is relatively level.

The site is located 4.6 miles inland from the Pacific Ocean at an approximate elevation of 320 feet above mean sea level (NGVD 29). Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

According to the City of Los Angeles (1996) and Los Angeles County (2014), the site is not located within a potential inundation area for an earthquake-induced dam failure or seiches (oscillating waves that form in an enclosed or semi-enclosed body of water).

The site is in an area of minimal flooding potential (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2008).

The site is not located in an oil field, however, according to California Division of Oil, Gas, and Geothermal Resources (DOGGR), the site is located 0.5 mile east, and 800 feet west of the Sawtelle and Cheviot Hills Oil Fields, respectively (DOGGR, 2019). The closest known oil exploration well is located approximately 0.5 mile northwest of the site. Per DOGGR, the well is classified as "buried" (DOGGR, 2019). Since the site is adjacent to active oil fields, there is a remote possibility that undocumented abandoned wells or other undocumented wells could be encountered during excavations. Any wells encountered during construction will have to be abandoned in accordance with current DOGGR standards and regulations.

Because of the nearby oil fields, there may be a potential for methane and other volatile gases to occur beneath the site. Additionally, the City of Los Angeles maps the site within a methane buffer zone (City of Los Angeles, 2016). Therefore, testing shall be performed at the site for soil gas in accordance with the City's regulatory requirements. If testing indicates that methane is present at the site, a permanent methane gas control system may be necessary beneath the proposed buildings at the site. The City of Los Angeles has implemented extensive methane-related regulations, which the proposed project would be required to comply with.

The potential for other geologic hazards such as lateral spreading and subsidence affecting the site is considered low.

4.4 Geologic Conclusions

Based on the available geologic data, faults with the potential for surface fault rupture are not directly beneath nor trending toward the site. Therefore, the potential for surface rupture at the site due to fault plane displacement propagating to the surface is considered low.

The location of the site relative to known active or potentially active faults indicates the site could be subjected to significant ground shaking. This hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices. Through conformance with these codes, as well as the recommendations in this report, the effects of potential significant ground shaking would not be significant.

The site is located in a City of Los Angeles methane buffer zone, therefore, the potential exists for the presence of volatile gases during and after construction. Undocumented wells can also be a potential hazard. Soil gas testing shall be performed at the site in accordance with all City of Los Angeles requirements, and based on the results, an appropriate soil gas mitigation system in compliance with all applicable regulatory requirements should be designed for the project.

The site is relatively level and the potential for slope stability hazards is considered low. There is a potential for liquefaction in the medium dense silty sand, sand, and sandy silt layers beneath the site; we estimate that the liquefaction settlement could be on the order of ¾ inch or less beneath the planned foundation level. With incorporation of this report's recommendations and code compliance, liquefaction impacts would not be significant. The corrosion studies indicate that the on-site soils are severely corrosive to ferrous metals, non-aggressive to copper, and that the potential for sulfate attack on portland cement concrete is considered negligible. Measures to address corrosion potential to ferrous metals, if used in project construction, should be identified as part of final project design plans.

The potential for other geologic hazards such as seismically-induced settlement (above the groundwater level), lateral spreading, subsidence, flooding, tsunamis, inundation, or seiches affecting the site is considered low.

5.0 Recommendations

5.1 General

The existing fill soils are not considered suitable for support of the proposed structure, floor slabs on grade, pavement, or other exterior concrete walks and slabs on grade. However, the excavation for the planned basement level is anticipated to automatically remove the existing fill soils. Accordingly, the proposed structure may be supported on conventional spread/continuous footings established in the undisturbed natural soils at the planned basement level. Recommendations are provided in the following sections for design of foundations.

The excavation of the basement is anticipated to extend to depths ranging from 30 to 43 feet below existing grade, which is below the historic-high groundwater level at the site. The excavation will not extend below the current groundwater level (based on recent and prior explorations at the site or at the adjacent property), but some minor seepage should be anticipated in the excavation, and minor dewatering consisting of gravel-filled trenches installed where necessary, should be anticipated. All dewatering will occur in conformance with all applicable regulatory water quality requirements. A permanent subdrain system will need to be designed for the basement or the footings and basement walls will need to be waterproofed and be designed to support hydrostatic pressure assuming a rise of the groundwater to historic-high levels. Recommendations for temporary and permanent drainage are provided in the following sections.

If the grading recommendations contained herein are implemented, floor slabs may be supported on grade at the planned basement level. However, for support of pavement or other at-grade exterior concrete walks and slabs on grade, we recommend that all existing fill soils be excavated and replaced as properly compacted fill. As an alternative to removal of the existing uncertified fill soils beneath pavement or other at-grade exterior concrete walks and slab-on-grade, some of the fill could be left in place; however, the existing fill may be susceptible to settlement in the event of wetting of seismic ground shaking, and the magnitude of such settlement is difficult to predict and would be variable. Therefore, if the potential for some settlement and greater than normal maintenance is acceptable, only the upper 2 feet of existing fill soils need be removed and replaced as properly compacted fill beneath pavement or other at-grade exterior concrete walks and slabs on grade.

The excavation for the basement will require temporary shored walls because of insufficient space for sloped excavations. The shoring may consist of soldier piles tied-back with anchors. Recommendations for shoring and excavation are provided in the following sections.

5.2 Foundations

Bearing Value

The proposed structures may be supported on conventional spread/continuous footings underlain by the undisturbed natural soils at the planned basement level. Such footings, if carried at least 2 feet below the lowest adjacent grade or floor level may be designed to impose a net dead-plus-live load pressure of up to 6,000 pounds per square foot.

Footings for minor structures structurally separated from the proposed building, such as auxiliary retaining walls, and free-standing walls, that are located at higher elevations outside of the planned basement limits, may be designed to imposed a net dead-plus-live load pressure of 2,000 pounds per square foot, carried 1 foot below the adjacent grade. Such footings may be established in either properly compacted fill and/or undisturbed natural soils

A one-third increase may be used for wind or seismic loads.

Settlement

We estimate the static settlement of the proposed structures, with the column and wall loads provided, supported in the manner recommended, with footings designed to the bearing capacity recommended, will be on the order of ¾ inch or less. The total settlement will consist of static settlement plus liquefaction-induced settlement in the event of the design-level earthquake, and would be up to 1½ inch after the design level earthquake. Differential static settlements between adjacent columns are expected to be about ¼ inch or less. Differential settlement after the design level earthquake would be ¾ inch or less. These settlements are acceptable per LADBS guidelines.

Lateral Resistance

Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between the footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill soils may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

Foundation Observation

To verify the presence of satisfactory soils at the design elevations, the bottoms of the foundations should be observed by personnel of our firm. Foundations should be deepened as necessary to reach satisfactory supporting soils.

Inspection of the foundation excavations may also be required by the appropriate reviewing governmental agencies. The contractor should be familiar with the inspection requirements of the reviewing agencies.

Ultimate Values

The recommended bearing and lateral load design values are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Coefficient of Friction	1.5
Passive Resistance	1.5

In no event, however, shall foundation sizes be less than those required to support dead-plus-live loads when using the working stress design method.

5.3 Seismic Design Parameters

We have determined the seismic parameters in accordance with the Section 1613 of the 2016 edition of the CBC and Section 11.4 of ASCE 7-10 Standard (ASCE, 2013) using the Office of Statewide Health Planning and

Development (OSHPD), Seismic Design Maps Web Application (OSHPD, 2018). The CBC Site Class was determined to be Site Class "C" based on a review of the results of our explorations and on a review of the local soil and geologic conditions (reference Section 5.1 of this report). The mapped seismic parameters may be taken as presented in the following table:

Parameter	**Mapped Value**
S_S (0.2 second period)	2.24g
S_1 (1.0 second period)	0.82g
Site Class	D
Fa	1.0
Fv	1.5
S_{MS} = FaSS (0.2 second period)	2.24g
S_{M1} = FvS1 (1.0 second period)	1.24g
S_{DS} = 2/3 x SMS (0.2 second period)	1.49g
S_{D1} = 2/3 x SM1 (1.0 second period)	0.82g

By: EJJ 4/8/19
Chkd: GA 4/9/19

5.4 Floor Slab Support

If the subgrade is prepared as recommended in the following section on grading, floor slabs may be supported on grade at the planned basement level. If a permanent subdrain system is designed, the required filter material for the subdrain system will provide adequate support for the lower level floor slab. Any deposits loosened or overexcavated should be properly compacted to at least 90%. As an alternative to the use of a subdrain system, the lower slab can be waterproofed and designed to support the hydrostatic pressure of groundwater rising to the historic-high depth of 25 feet below the existing grade, corresponding to Elevation 307½.

If the lower slabs are to be waterproofed, the use of a capillary break may not be required. However, if a permanent subdrain system is to be used and vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4 inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	**Percent Passing**
¾"	90–100
No. 4	0–10
No. 100	0–3

A low-slump concrete should be used to minimize possible curling of the slabs. A 2-inch-thick layer of coarse sand should be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. Concrete slabs should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend our that our field representative observe the condition of the final subgrade soils immediately prior to slab on grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

5.5 Excavation and Slopes

A maximum excavation depth of about 43 feet is currently anticipated in order to establish the lowest floor elevation and construct the foundations of the proposed buildings. Due to the proximity of the excavation limits to the adjacent existing buildings and streets, it is anticipated that shoring will be required for the excavation for the basement. However, where the necessary space is available, or on the interior of the excavation, temporary unsurcharged embankments may be sloped back at 1½:1 without shoring. Adjacent to existing structures, the bottom of any unshored excavation should be restricted so as not to extend below a plane drawn at 1½:1 (horizontal to vertical) downward from the foundations of the existing structure. Where space is not available, which is more likely at the project site, shoring will be required. Data for design of shoring are presented in a following section.

Based on the findings of our explorations, the planned bottom of the majority of the basement excavation is likely to be carried in admixtures of sandy and clayey material. Clayey material may become wet and spongy when exposed to construction activities; this situation will be even more critical if the excavation is performed during the rainy season. In these areas, to provide a working base for men and equipment, a layer of 1½ inch crushed rock may be necessary over the excavated surface.

The excavations should be observed by personnel of our firm so that any necessary modifications based on variations in the soil conditions encountered can be made. All applicable safety requirements and regulations, including OSHA regulations, should be met.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within 10 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur or if possible adverse water seepage conditions are developed.

The groundwater level is not anticipated to rise to the historic-high level during the construction of the basement, therefore dewatering will only be necessary to remove seepage groundwater that may encountered at various levels. The dewatering could consist as necessary of gravel-filled trenches with a perforated pipe placed in the trench. Disposal of the seepage groundwater to a sewer or stormdrain would necessitate a National Pollutant Discharge Elimination System (NPDES) permit.

5.6 Temporary Shoring

General

Where there is not sufficient space for sloped embankments, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied back with earth anchors. Some difficulty should be anticipated in the drilling of the soldier piles and the anchors because of caving in the sandy deposits. Special techniques and measures will be necessary in some areas to permit the proper installation of the soldier piles and/or tie back anchors. As an alternative to installing steel soldier piles in drilled holes, the soldier piles may be installed by vibration or driving. If vibratory installation methods are used, an evaluation of vibration should be included along with other types of construction related impacts at the site. If there is not sufficient space to install the tie back anchors to the desired lengths on any side of the excavation, the soldier piles of the shoring system may be internally braced.

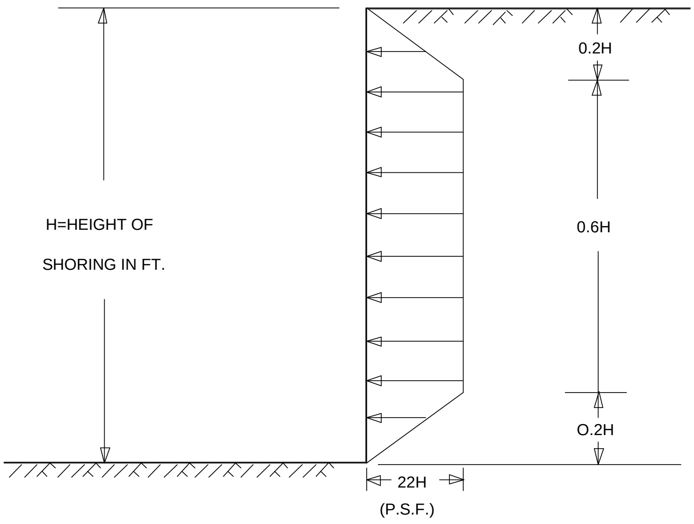
The following information on the design and installation of the shoring is as complete as possible at this time. We can furnish any additional required data as the design progresses. Also, we suggest that our firm review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

We recommend that the adjacent existing residential and commercial buildings, such as the adjacent high-rise condominium tower, surrounding the site be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in the adjacent buildings and streets should be performed and recorded and photographic records should be made.

Lateral Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination.

For the design of tied-back or braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 22H in pounds per square foot, where H is the height of the shoring in feet. Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination.



In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Furthermore, the shoring system adjacent to any existing building should also be designed to support the lateral surcharge pressures imposed by the foundations of the adjacent building. In addition, the shoring system should be designed to support the lateral surcharge pressures imposed by concrete trucks and other heavy construction equipment placed near the shoring system.

Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 600 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.3. (This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth.). In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting the downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken equal to 400 pounds per square foot.

Lagging

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. For clear spans of up to 6 feet, we recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 pounds per square foot at the mid-line between soldier piles, and 0 pounds per square foot at the soldier piles.

Anchor Design

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The anchors should extend at least 10 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in a following section. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 800 pounds per square foot. For post-grouted anchors, it is estimated that the anchors could develop an average friction up to 2,400 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

Anchor Testing

Our representative should select at least two of the initial anchors for 24-hour 200% tests, and 10 additional anchors for quick 200% tests. The purpose of the 200% tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

For post-grouted anchors where concrete is used to backfill the anchor along its entire length, the test load should be computed as that required to develop the appropriate friction along the entire bonded length of the anchor. For "200%" tests, 200% of design load should be used as the anchor head test load.
The total deflection during the 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed 0.75 inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed 0.25 inch during the 30-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

All of the production anchors should be pretested to at least 175% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 175% test should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

Internal Bracing

Raker bracing may be used to internally brace the soldier piles. If used, raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,000 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

Deflection

The deflection of a cantilevered shoring system may be estimated by the shoring engineer. The shoring should be designed to allow up to 1 inch movement at the top of shoring or less if necessary to protect adjacent structures or utilities. If greater than the estimated deflection occurs during construction, additional bracing may be necessary to minimize settlement of any adjacent structures. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. We will be pleased to discuss this further with you and the design consultants when the design of the shoring system has been finalized.

We recommend that the adjacent existing structures surrounding the site be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in the adjacent buildings and streets should be performed and recorded and photographic records made.

5.7 Walls Below Grade and Retaining Walls

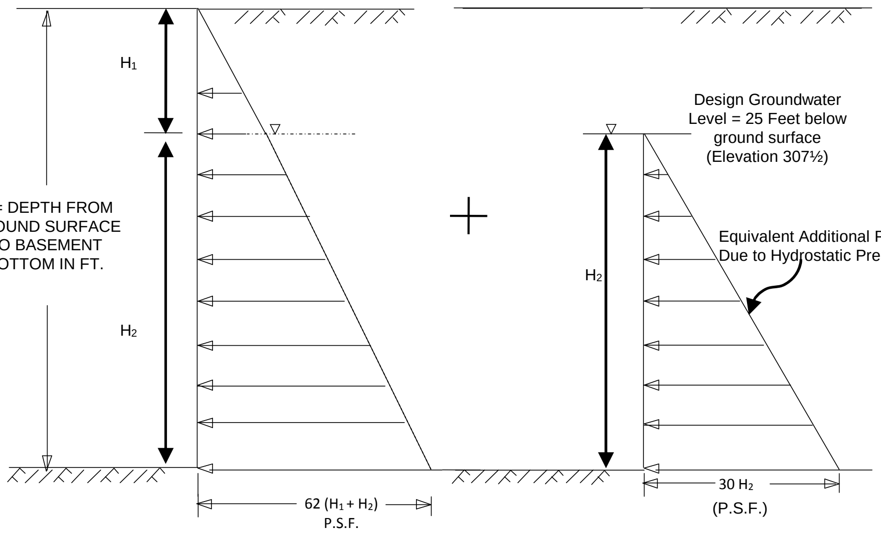
Lateral Pressures

For design of cantilevered retaining walls, where the surface of the backfill is level, it may be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot.

As required by the 2016 California Building Code, braced basement walls must be designed to resist at-rest earth pressures. Accordingly, for the case where the grade is level behind the walls, a triangular distribution of lateral earth pressure equivalent to that developed by a fluid with a density of 62 pounds per cubic foot plus any surcharge loadings occurring as a result of traffic and adjacent foundations should be used.

In addition to the recommended earth pressure, the upper 10 feet of walls adjacent to streets or vehicular traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected. Furthermore, adjacent to the existing structures, the basement walls should be designed for the appropriate lateral surcharge pressures imposed by the foundations of the existing structures.

If the permanent subdrain system is not installed beneath the basement level, the basement walls and lower slabs should be waterproofed and designed to support the hydrostatic pressure of groundwater rising to the historic-high depth of 25 feet below the existing grade, corresponding to Elevation 307½. Therefore, subterranean walls below Elevation 307½ should be designed to resist the resulting external hydrostatic pressure in addition to the lateral earth pressure and other lateral surcharge pressures discussed above. The recommended lateral pressure for this condition is shown in the following diagram.



Seismic Earth Pressures

Subterranean building walls should be designed to support an active seismic lateral pressure. The combined active static and seismic lateral earth pressure based on the design level earthquake may be taken as equivalent to that developed by a fluid with a density of 35 pounds per cubic foot. The drained active static lateral earth pressure is equivalent to a fluid with a density of 30 pounds per cubic foot. Note that in this case, the equivalent hydrostatic pressure that should be added to the static lateral earth pressure below Elevation 307½ should be taken as equivalent to that developed by a fluid having a density of 30 pounds per cubic foot.

For minor retaining walls, based on the characteristics of the upper soils, the height of the walls, and the anticipated level of ground shaking in the event of the design earthquake, we estimate that the seismic lateral earth pressure will be negligible.

Waterproofing

If the permanent subdrain system is not to be installed beneath the lower subterranean level, we recommend that walls below the depth of historical groundwater level at 25 feet below grade be waterproofed. The portions of the basement walls above this elevation should be damp-proofed or waterproofed.

Drainage

We recommend that minor retaining walls and basement walls be provided with drainage or be designed to resist hydrostatic pressure.

Unless a permanent subdrain system is installed, we recommend that the portion of the basement walls below the historic-high groundwater level be waterproofed and designed to resist the hydrostatic pressure of groundwater rising to the historic-high level, as discussed in the preceding sections.

For minor retaining walls, drainage could consist of a 4-inch-diameter perforated pipe placed with perforations down at the base of the wall. The pipe should be sloped at least 2 inches in 100 feet and surrounded by ¾-inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

5.8 Subdrain System

As previously stated, based on available official maps and data, the historical-high groundwater at the site is at a depth of about 25 feet below the existing grade. Therefore provisions must be taken to protect the building from hydrostatic pressure in case the groundwater rises to the historical level which is above the lower subterranean parking level.

There are two alternative procedures that might be followed. A permanent subdrain system could be installed beneath the lower floor of the building to maintain the water level below the lower subterranean floor slab, or the lower subterranean floor slab and the lower portions of the subterranean walls could be waterproofed and designed for the possible hydrostatic pressure. To compute the hydrostatic pressure, it may be assumed that the water level will be at a depth of 25 feet below the existing grade. The design of the lower floor slab to resist the possible hydrostatic pressure would require a thorough waterproofing installation and relatively thick floor slab. Installation of a completely watertight waterproofing system will be difficult. If such a system is desired, we suggest consulting with a contractor experienced in the installation of such system.

If it is decided to install a subdrain system, it should be realized that the permit from the State of California Regional Water Quality Control board will have to be obtained to discharge the subdrain water into the storm drain. To obtain such a permit, chemical tests will have to be performed on groundwater samples obtained at the site to verify that chemicals or pollutants within water do not exceed the allowable limits for discharging into the storm drain.

For a subdrain system, we recommend that the lower floor of the building be underlain by a layer of filter material approximately 1 foot thick. The filter material should be drained by subdrain pipes leading to sump area equipped with automatic pumping units. We suggest that the filter material meet the requirements of Class 2 Permeable Material as defined in Section 68 of the latest edition of the State of California Department of Transportation, Standard Specifications. If Class 2 material is not available, ¾-inch crushed rock separated from the adjacent soils by a filter fabric may be used. The crushed rock should have less than 5% passing a No. 200 sieve. The drain lines should consist of perforated pipe, placed with the perforation down, in trenches extending at least 1 foot below the filter material. The trenches should be backfilled with material meeting the requirements of the Class 2 Permeable Material or lined with the filter fabric and filled with ¾-inch crushed rock. The drain lines should extend around the perimeter of the building and should be spaced approximately 40 feet apart within the interior of the building. A slope of at least 2 inches per 100 feet should be used for the drain lines. Although water will not initially flow into the subdrain system, we suggest that the pumps and sumps be sized for a total inflow into the system of 300 gallons per minute. The actual inflow into the subdrain system is expected to be less.

In addition to the above drainage system, some means of draining the soils outside the exterior walls will be required. The means of accomplishing drainage outside the wall will depend primarily on the selected method of shoring and the method of constructing the exterior building walls. A drainage system behind the basement walls may be provided by strips of Miradrain 6000 (or equivalent). In our opinion, Miradrain 6000 (or equivalent), attached to the lagging and protected from the concrete placement of the walls, would provide satisfactory drainage. Continuous Miradrain may be placed at a depth starting at about 3 feet below the existing grade.

The Miradrain should be connected to weep holes at the bottom of the excavation. The weep holes should be consist of solid pipes spaced at 8 feet on centers. At the connection of the weep holes and the Miradrain, the weep holes should be embedded in 1 cubic foot of free-drainage aggregate surrounded by a filter fabric. The weep holes should drain into the subdrain system placed beneath the slab of the lower subterranean level or into a solid pipe placed beneath the edge of the lower floor slab. The solid pipe should discharge into the sump.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

We can provide additional data for the design of the subdrain system as the features of the system are developed. In addition, we suggest that the design be reviewed after the excavation has been completed. If necessary, the system could be modified as indicated by the observed conditions, including the quantity of water pumped during construction dewatering.

5.9 Grading

For support of other at-grade exterior concrete walks and slabs on grade, we recommend that all existing fill soils be excavated and replaced as properly compacted fill. If the potential for some settlement and greater than normal maintenance is acceptable, only the upper 2 feet of existing fill soils need be removed and replaced as properly compacted fill beneath pavement or other at-grade exterior concrete walks and slabs on grade.

All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils may be used in the required fill.

Site Preparation

After the site is cleared and existing fill soils and soils disturbed due to demolition activities are excavated as recommended, the exposed soils should be carefully observed for the removal of all unsuitable deposits. Next, where fill is to be placed, the exposed soils should be scarified to a depth of 6 inches, brought to near-optimum moisture content, and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

Good drainage of surface water should be provided by adequately sloping all surfaces. Such drainage will be important to minimize infiltration of water beneath floor slabs.

Compaction

Required fill should be placed in loose lifts not more than 8 inches thick and compacted. The fill should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557 method of compaction. The moisture content of the on-site sandy soils at the time of compaction should vary no more than 2% below or above optimum moisture content.

Material for Fill

The on site soils, less any debris or organic matter, may be used in required fills. Cobbles or concrete fragments larger than 4 inches in diameter should not be used in the fill. Any required import material should consist of relatively non expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (at least 15% passing the No. 200 sieve) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by our personnel prior to being placed at the site.

5.10 Geotechnical Observation

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of our firm. This representative should perform at least the following duties:

- Observe the clearing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proofrolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.
- Observe the installation of shoring systems, including soldier beams, lagging, and anchors.
- Observe installation of drainage system behind the basement wall and subdrains.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

6.0 Basis for Recommendations

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representative at the site provides the client with an independent professional opinion regarding the geotechnically-related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

7.0 Bibliography

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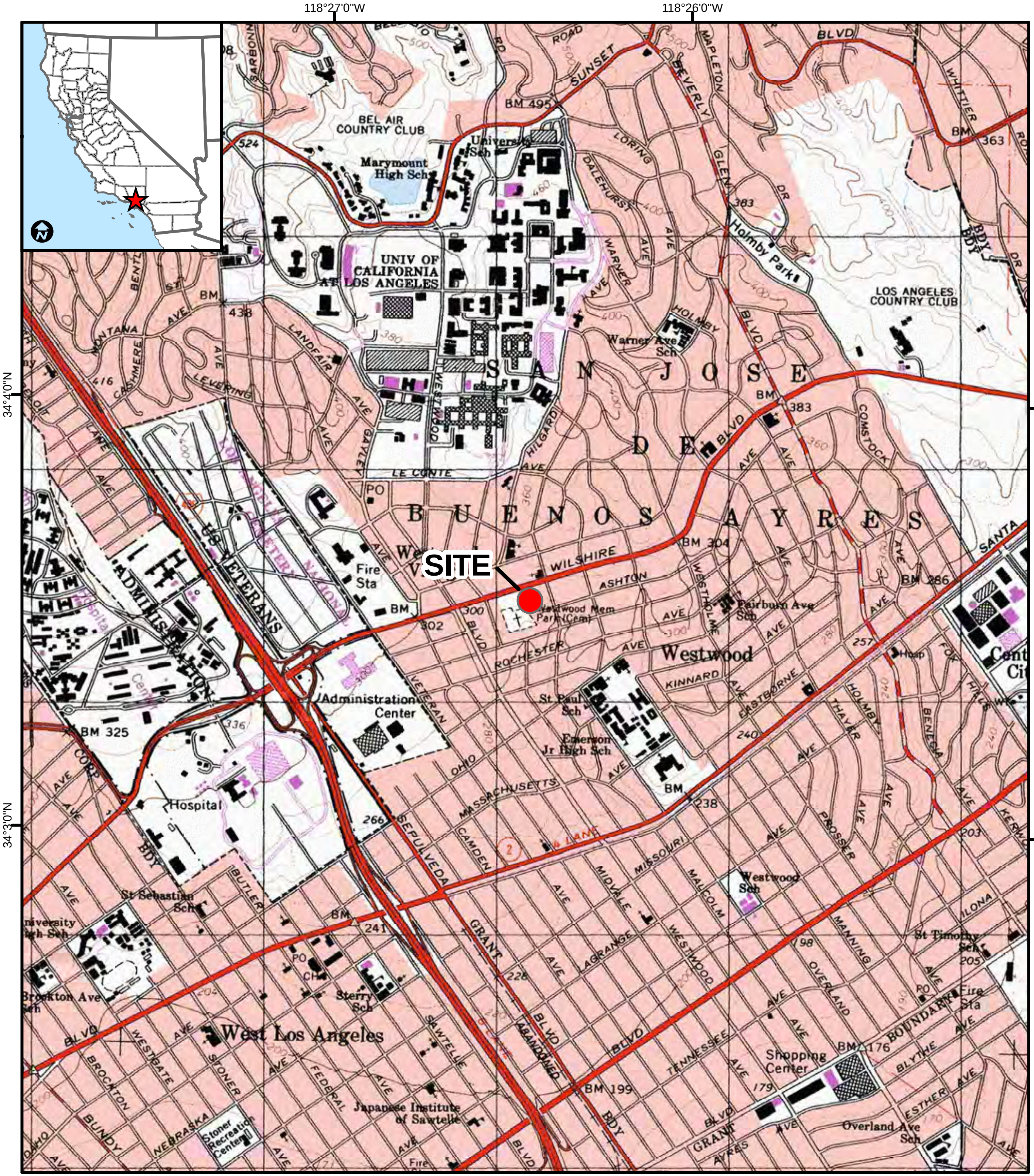
Figure 1

Site Vicinity Map



Base: USGS 7.5 minute topographic map of the Beverly Hills quadrangle

0 1,000 2,000 4,000
Feet

wood.

AMEC Foster Wheeler
Environment & Infrastructure, Inc.
6001 Rickenbacker Road
Los Angeles, California 90040
Tel: 323.889.5300
Fax: 323.721.6700

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Los Angeles, California

LAT:	34.0591
LON:	-118.4407
SCALE:	1:24,000
DRAWN:	PER
CHECK:	PJE
DATE:	04-08-16

VICINITY MAP

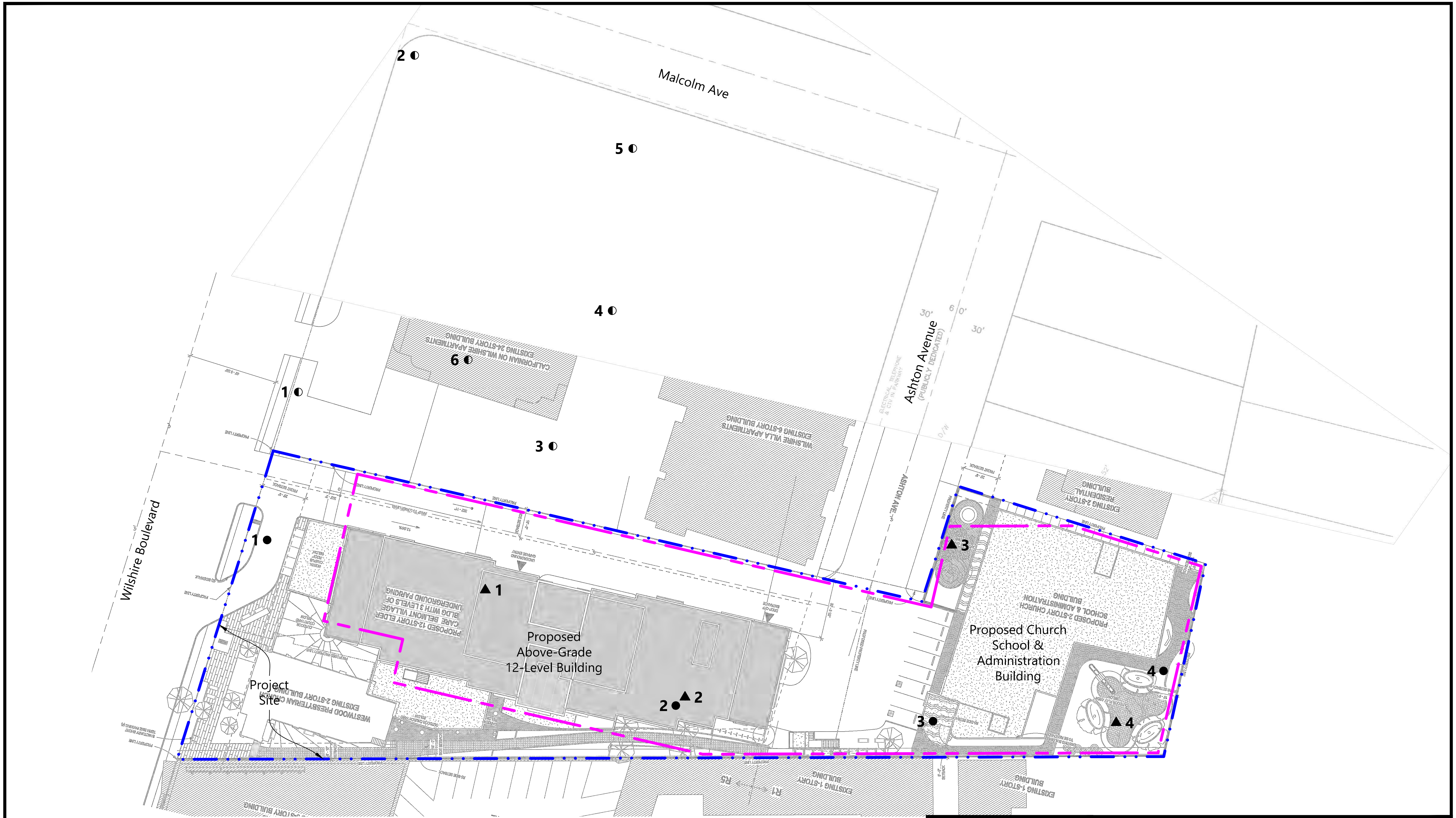
FIGURE:
1

PROJECT:
4953-16-0251

G:\4953 Geotech\2016\160251 Belmont Village - Westwood\GIS\160251 Figure 1 Site Vicinity Map.mxd

Figure 2

Plot Plan



LEGEND

- 4 ● Boring From Current Investigation
- 4 ▲ Cone Penetration Test (CPT) From Current Investigation
- 6 ◐ Boring From Prior Investigation (Our Job No. 70131-2-0276)
- Proposed Basement Limits

Reference:
-Base Map by CreSurveys, sheet 2 of 3
-Site Plan prepared by ???, sheet
ENT-101, dated November 27, 2018

0 25' 50' 100'
SCALE: 1" = 50'

wood.

Wood
Environment & Infrastructure Solutions, Inc.,
6001 Rickenbacker Rd, Los Angeles, CA 90040
Phone (323) 889-5300 Fax (323) 721-6700

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Los Angeles, California

LT,LNG:
PREPARED BY: VMN
SCALE: 1" = 50'
BY: GA
CHKD: EJJ
DATE: 04/09/2019

Plot Plan

FIGURE NO.
2

PROJECT NO.
4953-16-0251

Path: G:\4953_Geotech\2016\160251 Belmont Village - Westwood\CAD\DWG\4953-16-0251_Fig-2_Plot-Plan(19.04.09).dwg [B-17x11]
Date: April 09, 2019 - 4:30pm By: vo.nguyen

Appendix A

RECENT FIELD EXPLORATIONS AND LABORATORY TEST RESULTS

Appendix A
Recent Field Explorations and Laboratory Test Results

Recent Exploration Borings

The soil conditions beneath the site were explored by drilling four borings at the locations shown on Figure 2. The borings were drilled to depths of between 51 and 61½ feet below the existing grade using 8 inch diameter truck mounted hollow-stem auger drilling equipment.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the borings are presented on Figures A 1.1 and A-1.4; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches using a 140 pound hammer falling 30 inches is indicated on the logs. In addition to obtaining undisturbed samples, standard penetration tests (SPT) were also performed; the results of the tests are indicated on the logs. The soils are classified in the accordance with the Unified Soil Classification System described on Figure A 2.

Recent Cone Penetration Testing

To supplement the exploratory borings, four cone penetration tests (CPTs) were performed to the depth of 60 feet below the existing grade. The locations of the CPTs are presented on Figures 2. The results of the CPTs are presented in Appendix C.

Recent Laboratory Test Results

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to evaluate their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are presented to the left of the boring logs.

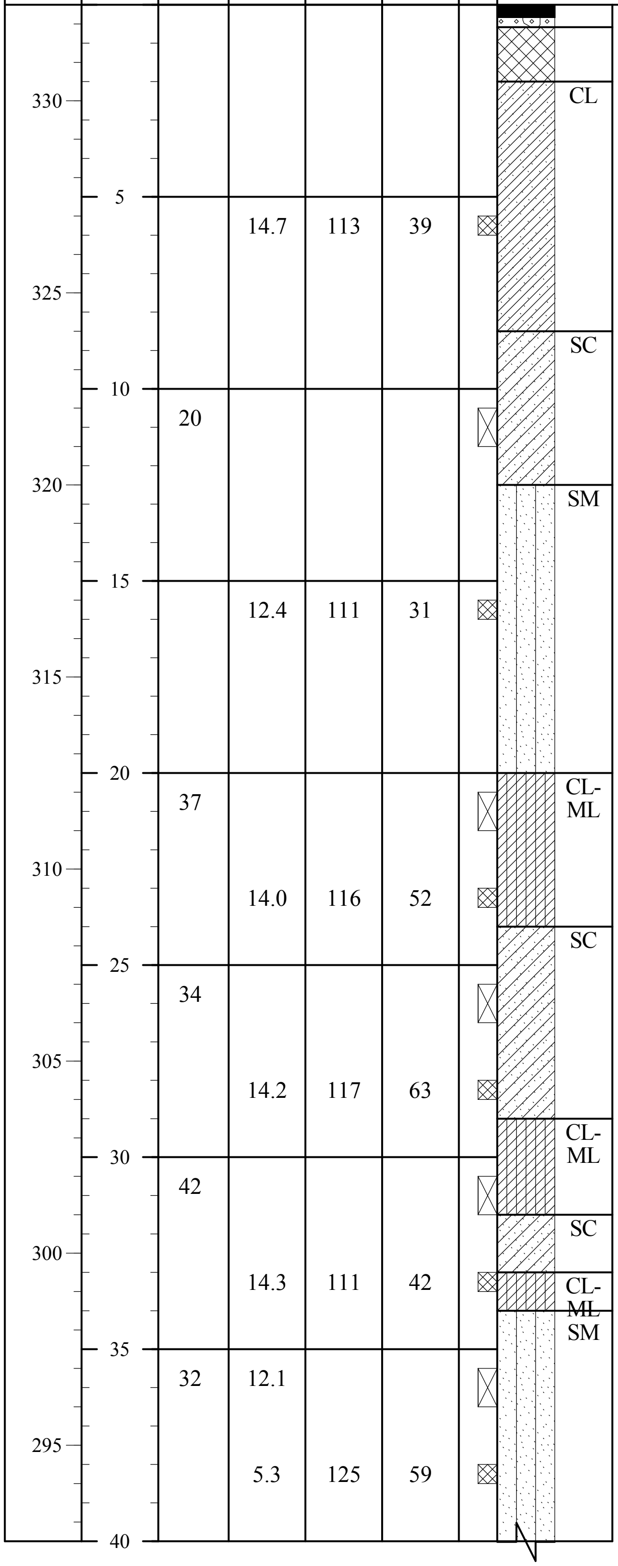
Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and at various surcharge pressures. The results of the tests are presented on Figure A 3, Direct Shear Test Data.

Confined consolidation tests were performed on three undisturbed sample to determine the compressibility of the soils. Water was added to the sample during the tests to illustrate the effect of moisture on the compressibility. The results of the test are presented on Figure A 4, Consolidation Test Data.

Soil corrosivity tests were performed on a sample of the on-site soils. The results of the tests are presented on Figure A 5.

BORING 1

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 333 **



"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
	14.7	113	39	⊠
20				☒
	12.4	111	31	⊠
37				☒
	14.0	116	52	⊠
34				☒
	14.2	117	63	⊠
42				☒
	14.3	111	42	⊠
32	12.1			☒
	5.3	125	59	⊠

4-inch thick Asphalt Concrete over 3-inch thick Sandy Gravel base course

FILL - SILTY SAND - moist, olive brown

CL — SANDY LEAN CLAY - very stiff, moist, olive brown to reddish brown, fine sand, some medium to coarse, some fine gravel (up to ¾ inch in size)

More fine sand

SC — CLAYEY SAND - medium dense, moist, olive brown, fine grained, some medium, trace fine gravel (up to ½ inch in size)

Less clayey

SM — SILTY SAND - medium dense, moist, yellow brown, fine grained, some medium, some clay

CL-ML — SILTY CLAY with SAND - hard, moist, dark olive to olive brown, fine sand

More fine sand, trace fine gravel (up to ¼ inch in size)

SC — CLAYEY SAND - dense, moist, yellow brown, fine grained, interbedded thin layers of Silty Clay

Some fine gravel (up to ½ inch in size)

CL-ML — SILTY CLAY with SAND - hard, moist, olive brown, fine sand, some fine gravel (up to ½ inch in size)
Some fine slate gravel (up to ¾ inch in size)

SC — CLAYEY SAND - moist, brown to olive brown, fine grained, trace medium, some slate gravel (up to ¾ inch in size)

CL-ML — SILTY CLAY with SAND - very stiff, moist, olive brown, fine sand, some fine to coarse gravel (up to 1½ inches in size)

SM — SILTY SAND - dense, moist, olive brown to dark yellow brown, fine grained, some medium, some clay, thin layer of fine to coarse gravel (up to ¾ inch in size) from 35¾ to 36¼ feet

Less clayey, thin layer of Sandy Gravel (up to ¼ to ½ inches in size), with some slate gravel at 37½ to 38¼ feet, less silt

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: WL
Checked By: LH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

B12SOIL_CRANDALL (NO DECIMAL) L:\70131 GEOTECH\GINTW\LIBRARY AMECFW 2018.GLB
P:\4953 GEOTECH\2016-PROJ\160251 BELMONT VILLAGE - WESTWOOD\3.2 ALL FIELD NOTES\4953-16-0251.GPJ 4/16/19

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wood.

LOG OF BORING

Project: 4953-16-0251

BORING 1 (Continued)

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 333 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								Less gravel
		50	9.2			☒		
							CL-ML	SILTY CLAY with SAND - hard, moist, olive brown, fine sand, trace to some fine gravel (up to ¼ inch in size)
290								
			10.8	108	40	▩	ML	SANDY SILT - very stiff, moist, yellow brown, fine sand
							SP-SM	POORLY GRADED SAND with SILT - very dense, moist, brown, fine grained, some medium, some fine gravel (up to ½ inch in size)
	45							
		55				☒		
285								
								Less silt
	50							
			10.9	111	33	▩	ML	SILT - very stiff, moist, yellow, trace fine sand, some clay END OF BORING AT 51 FEET
280								
	55							
275								
	60							
270								
	65							
265								
	70							
260								
	75							
255								
	80							

NOTES:

Hand augered upper 5 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with soil cuttings and patched with asphalt.

*Number of blows to drive Crandall sampler 12 inches using a 140 pound automatic hammer falling 30 inches.

** Elevations based on Survey Map by CreSurveys

Field Tech: LH
Prepared By: WL
Checked By: LH

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LOG OF BORING
Project: 4953-16-0251
Figure: A-1.1b

B12SOIL_CRANDALL (NO DECIMAL) L:\70131 GEOTECH\GINTW\LIBRARY AMECFW 2018.GLB
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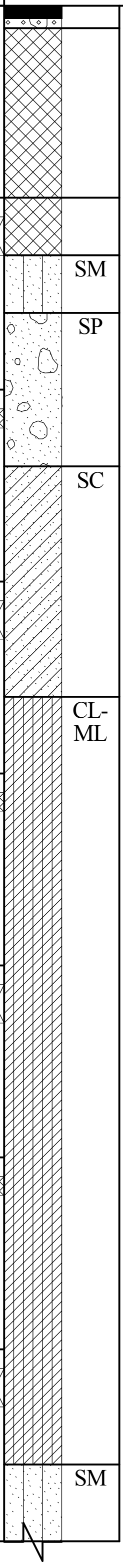
BORING 2

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 325 **



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
325							4-inch thick Asphalt Concrete over 3-inch thick Sandy Gravel base course FILL - SILTY SAND to SANDY SILT - moist, olive brown, fine grained, some fine gravel (up to ½ inch in size)
320	5	31				☒	FILL - CLAYEY SAND - moist, olive brown to dark brown, fine grained, some medium to coarse, some fine gravel (up to ½ inch in size), some asphalt fragments
							SM: SILTY SAND - moist, red brown, fine to medium grained
							SP: POORLY GRADED SAND with GRAVEL - medium dense, moist, orange brown with mottled gray, fine to coarse gravel (up to 1½ inches in size), fine to medium grained, some coarse, small clay pockets
315	10		7.9	116	45	☒	
							SC: CLAYEY SAND - medium dense, moist, olive brown with orange pink pigment and gray, fine grained, some medium
310	15	22				☒	
							CL-ML: SILTY CLAY with SAND - very stiff to hard, moist, olive brown, fine sand, some medium, trace coarse
305	20		15.4	113	26	☒	
300	25	36				☒	More clayey
295	30		13.9	115	65	☒	Less sand
290	35	28	13.6			☒	
							SM: SILTY SAND - medium dense, moist, yellow, fine grained, trace medium
	40						

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: WL
Checked By: LH

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Los Angeles, California

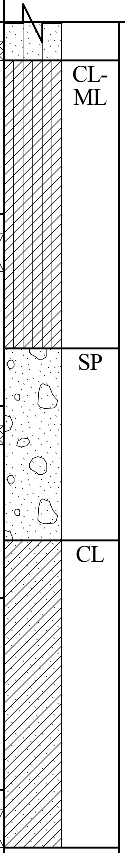
wood.

LOG OF BORING
Project: 4953-16-0251 Figure: A-1.2a

BORING 2 (Continued)

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 325 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
285			16.1	107	35	☒	CL-ML	SILTY CLAY - very stiff, moist, yellow
280	45	16	16.7			☒		More clayey
							SP	POORLY GRADED SAND with GRAVEL -very dense, moist, yellow brown with mottled gray, fine to medium grained, fine to coarse gravel (up to ¾ inch in size), alternating layers of thin Silty Sand
275	50		6.0	119	80	☒		
							CL	Less sand SANDY LEAN CLAY - hard, moist, olive brown, some fine sand
270	55							
265	60	39	21.7			☒		(LL=35, PI=22)
260	65							
255	70							
250	75							
	80							



END OF BORING AT 61½ FEET

NOTES:

Hand augered upper 5 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with soil cuttings and patched with asphalt.

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Field Tech: LH
Prepared By: WL
Checked By: LH

Proposed Belmont Village
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Los Angeles, California

wood.

LOG OF BORING
Project: 4953-16-0251 Figure: A-1.2b

B12SOIL_CRANDALL (NO DECIMAL) L:\70131 GEOTECH\GINTW\LIBRARY AMECFW 2018.GLB
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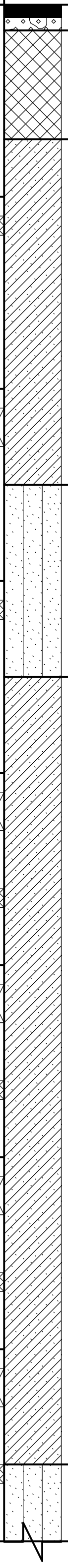
BORING 3

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 323 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								4-inch thick Asphalt Concrete over 4-inch thick Gravel base course
								FILL - SILTY SAND - moist, reddish brown, fine grained, some fine gravel
320								
							CL	SANDY LEAN CLAY - stiff to very stiff, moist, brown, fine sand, some fine gravel (up to ½ inch in size)
	5							
			14.4	115	24	▨		
315								Grades coarser
	10							Less clayey, more silt, some medium sand
		27				☒		
310							SM	SILTY SAND - medium dense, moist, yellow brown, fine grained, some fine gravel (up to ¼ inch in size), some clay
	15							
			13.4	115	24	▨		
305							CL	SANDY LEAN CLAY - very stiff, moist, brown to olive brown, fine sand, some medium
	20							
		29				☒		
								Less clayey
300			14.6	111	43	▨		Some Silty Sand nodules, yellow with mottled light gray, some fine gravel (up ¼ to ½ inch in size)
	25							
		36	13.6			☒		Becomes hard, Some medium sand, some fine gravel (up to ¼ inch in size)
								Some small Silty Sand nodules
295			14.3	118	73	▨		
	30							
		47				☒		Becomes reddish brown to brown, fine sand, trace fine gravel (up to ½ inch in size)
								Becomes brown, less sand, trace to some fine gravel (up to ¼ inch in size)
290			15.1	119	70	▨		
	35							
		31	13.7			☒		Becomes fine sand, no gravel (LL=30, PI=18)
285			14.3	113	39	▨		Less clayey, brown to olive brown, less sand
							SM	SILTY SAND - medium dense, moist, yellow brown, fine grained, trace fine gravel (up to ½ inch in size), some clay
	40							



(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: WL
Checked By: LH

Proposed Belmont Village
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Los Angeles, California

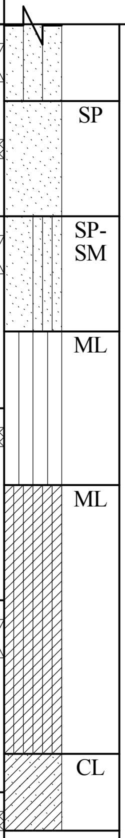
wood.

LOG OF BORING

Project: 4953-16-0251 Figure: A-1.3a

BORING 3 (Continued)

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 323 **



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
		28	12.2			☒		Less clay (32% Passing No. 200 Sieve)
280			4.9	111	75	▨	SP	POORLY GRADED SAND - dense, moist, yellow brown, fine to medium grained, some coarse, some fine to coarse gravel (up to ¾ inch in size)
	45	63				☒	SP-SM	POORLY GRADED SAND with SILT - very dense, moist, yellow, fine grained, some medium, some fine gravel (up to ½ inch in size)
275							ML	SILT - stiff, moist, yellow to light brown, some fine sand
	50		25.0	90	23	▨		
270							ML	SILT with CLAY - very stiff, moist, yellow brown to brown, some fine sand
	55	25	15.6			☒		Alternating thin beds of Clay and Silty Sand from 55½ to 56½
265							CL	SANDY LEAN CLAY with SAND - very stiff, moist, olive brown, fine sand, trace medium, trace to some fine gravel (up to ½ inch in size) (58% Passing No. 200 Sieve)
	60		16.0	111	44	▨		END OF BORING AT 61 FEET
260								
	65							
255								
	70							
250								
	75							
245								
	80							

NOTES:

Hand augered upper 5 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with soil cuttings and patched with asphalt.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: LH
Prepared By: WL
Checked By: LH

BORING 4

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 324 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								3-inch thick Asphalt Concrete over 3-inch thick Sandy Gravel base course
								FILL - SILTY SAND - moist, yellow brown, fine grained, some medium, some fine to coarse gravel (up to 1 inch in size)
320							ML	SANDY SILT - stiff, moist, olive to olive brown, fine sand, some clay, some fine gravel (up to ½ inch in size), trace roots (LL=30, PI=18)
	5		7.0	102	22	▨		
315								More fine sand, more clayey
	10	31				☒	SM/ ML	SILTY SAND to SANDY SILT - dense, moist, olive brown, fine sand, some clay, some fine to coarse gravel (up to 2/3 inch in size)
310							SM	SILTY SAND - medium dense, moist, yellow to yellow brown, fine grained, some clay, some fine to coarse gravel (up to 1 inch in size)
	15		9.2	116	30	▨		Thin layer of Clayey Sand
305								More clayey
	20	24				☒	CL	SANDY LEAN CLAY - very stiff, moist, olive brown, fine sand, trace medium, some fine gravel (up to ½ inch in size), interbedded with thin layers of Silty Clay
300							CL- ML	SILTY CLAY with SAND - moist, olvie yellow, fine sand
	25		17.6	112	46	▨		
295							ML	SANDY SILT with CLAY - hard, moist, brown, fine sand, trace fine to coarse gravel (up to ¾ inch in size)
	30	38	15.1			☒		
290							CL	SANDY LEAN CLAY - very stiff to hard, moist, olive brown, fine sand, some fine gravel (up to ½ inch in size)
	35		13.0	121	73	▨		
								Trace to some fine gravel (up to ½ inch in size)
285								
	40							

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: LH
Prepared By: WL
Checked By: LH

Proposed Belmont Village
10822 Wilshire Blvd & 10812 Ashton Ave.
Los Angeles, California

wood.

LOG OF BORING
Project: 4953-16-0251 Figure: A-1.4a

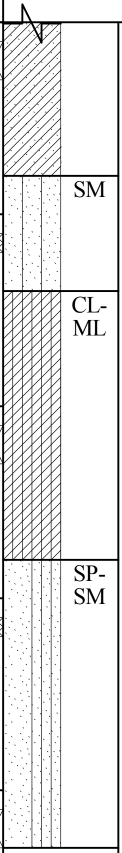
B12SOIL_CRANDALL (NO DECIMAL) L:\70131 GEOTECH\GINTW\LIBRARY AMECFW 2018.GLB
P:\4953 GEOTECH\2016-PROJ\160251 BELMONT VILLAGE - WESTWOOD\3.2 ALL FIELD NOTES\4953-16-0251.GPJ 4/16/19

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING 4 (Continued)

DATE DRILLED: March 21, 2016
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION (ft.): 324 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
		29	13.8			☒
280						
	45					
			16.8	106	37	▦
275						
	50					
		21				☒
270						
	55					
			17.8	111	49	▦
265						
	60					
		38	14.8			☒
260						
	65					
255						
	70					
250						
	75					
245						
	80					



SILTY SAND - medium dense, moist, yellow, fine grained, some alternating silt layers, some clay

More clay

SILTY CLAY with SAND to SILT with CLAY and SAND - very stiff, moist, yellow, fine sand

(65% Passing No. 200 Sieve)

POORLY GRADED SAND with SILT - dense, moist, yellow, fine grained, some medium, some fine to coarse gravel (up to ½ inch in size), some small silt pockets

Alternating thin layers of Silty Clay

More silt

Thin layer of Silty Clay, moist, pale brown to yellow, grades back to Poorly Graded Sand with Silt at 60½ feet

END OF BORING AT 61½ FEET

NOTES:

Hand augered upper 5 feet to avoid damage to utilities. Groundwater was not encountered. Boring was backfilled with soil cuttings and patched with asphalt.

Field Tech: LH
Prepared By: WL
Checked By: LH

MAJOR DIVISIONS			GROUP SYMBOLS		TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	●	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			○	GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	○	GM	Silty gravels, gravel - sand - silt mixtures.
			▨	GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	░	SW	Well graded sands, gravelly sands, little or no fines.
			░	SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	▥	SM	Silty sands, sand - silt mixtures
			▨	SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		▥	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			▨	CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			▤	OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		▥	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			▨	CH	Inorganic clays of high plasticity, fat clays
BEDROCK			░		SANDSTONE
			▤		SILTSTONE
			░		GRANITE

■	Undisturbed Sample	▮	Auger Cuttings
⊠	Split Spoon Sample	≀	Bulk Sample
▥	Rock Core	▩	Crandall Sampler
▩	Dilatometer		Modified California Sampler
▯	Packer	○	No Recovery
▽	Water Table at time of drilling	▼	Water Table after drilling

Correlation of Penetration Resistance
with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

KEY TO SYMBOLS AND DESCRIPTIONS

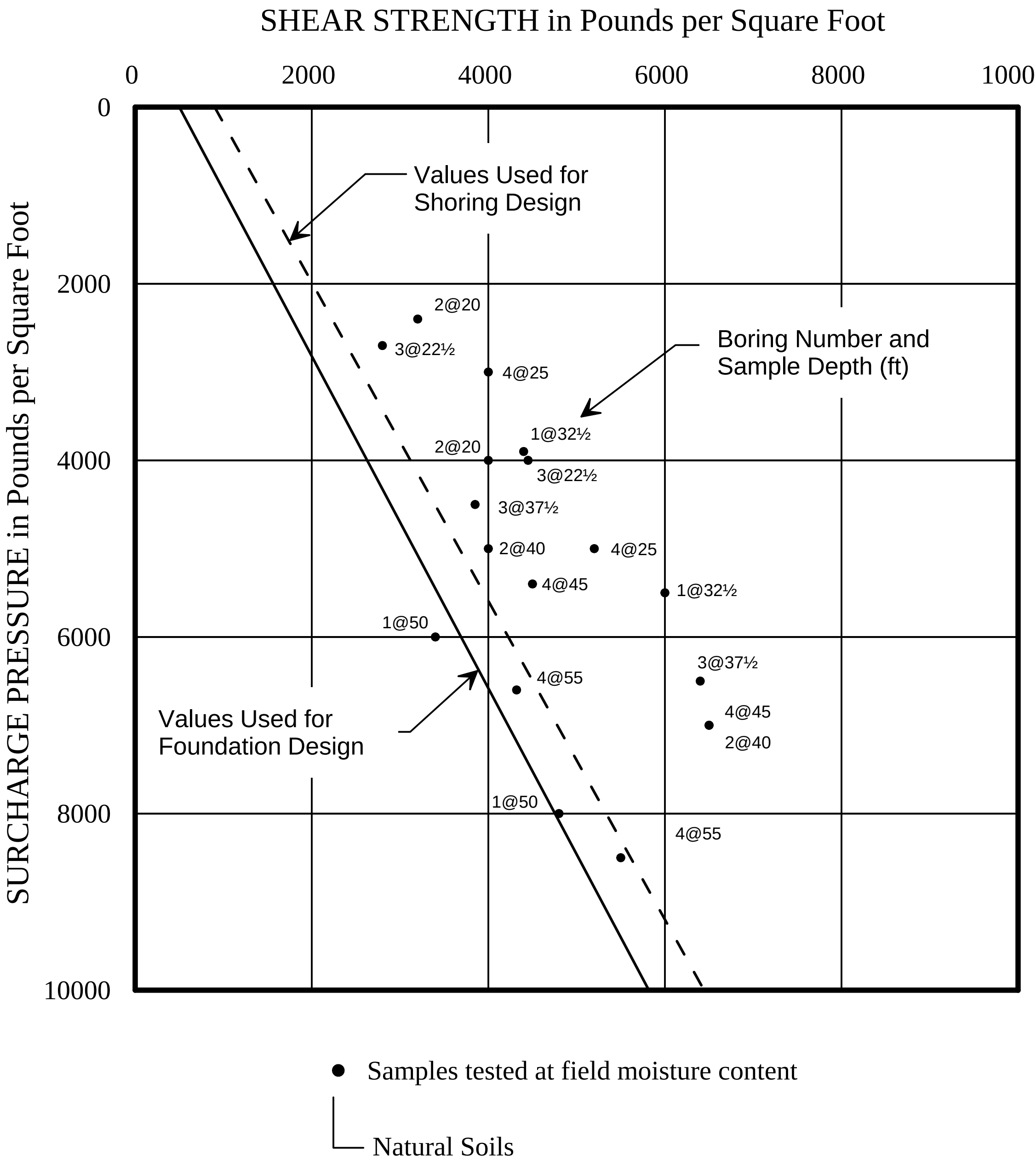
BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Figure A-2

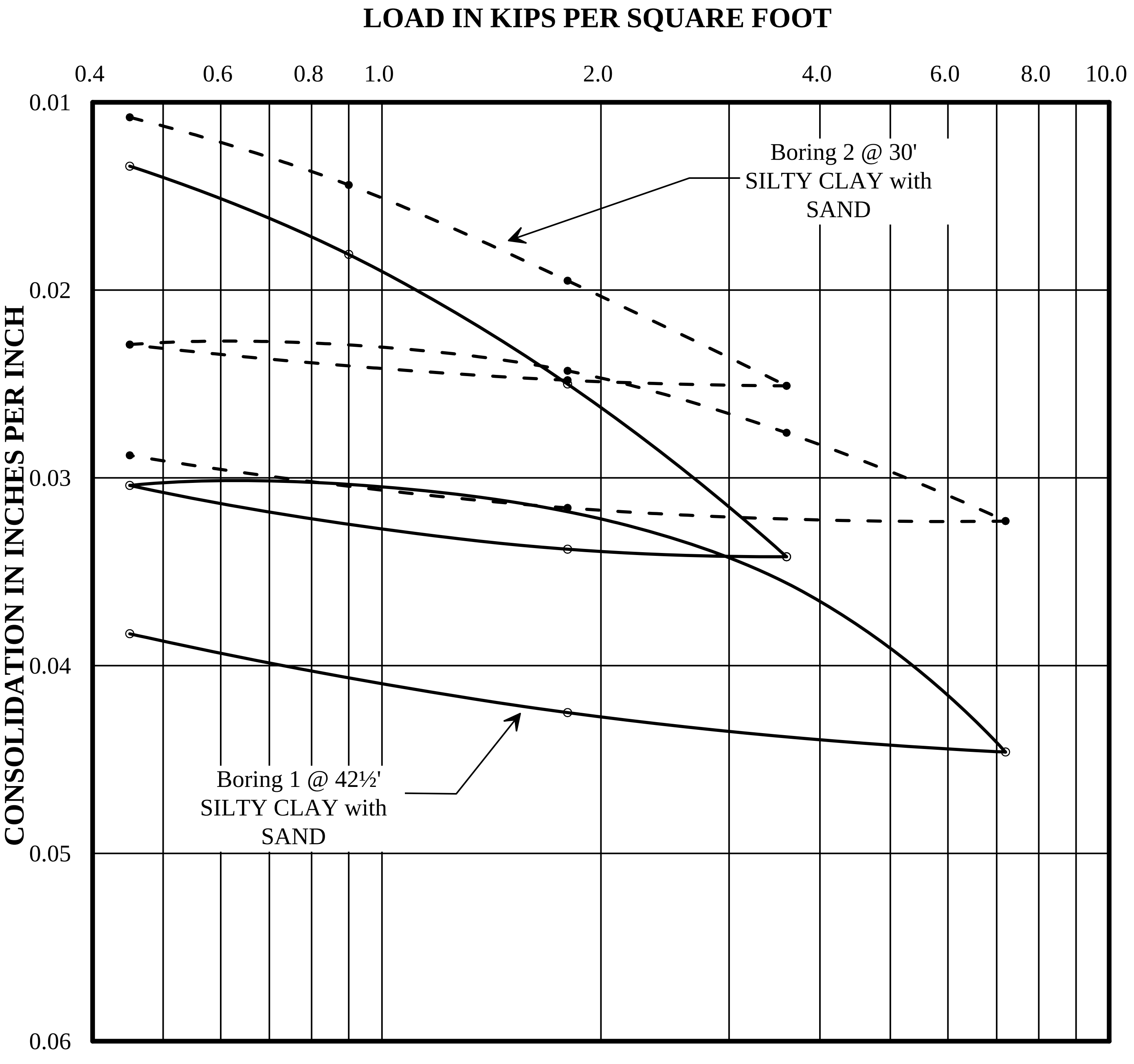


Prepared/Date: WL 4/11/2016
Checked/Date: LH 4/14/2016

Proposed Belmont Village
10822 Wilshire Boulevard and
10812 Ashton Avenue
Los Angeles, California

wood.

DIRECT SHEAR TEST DATA
Project No. 4953-16-0251
Figure A-3



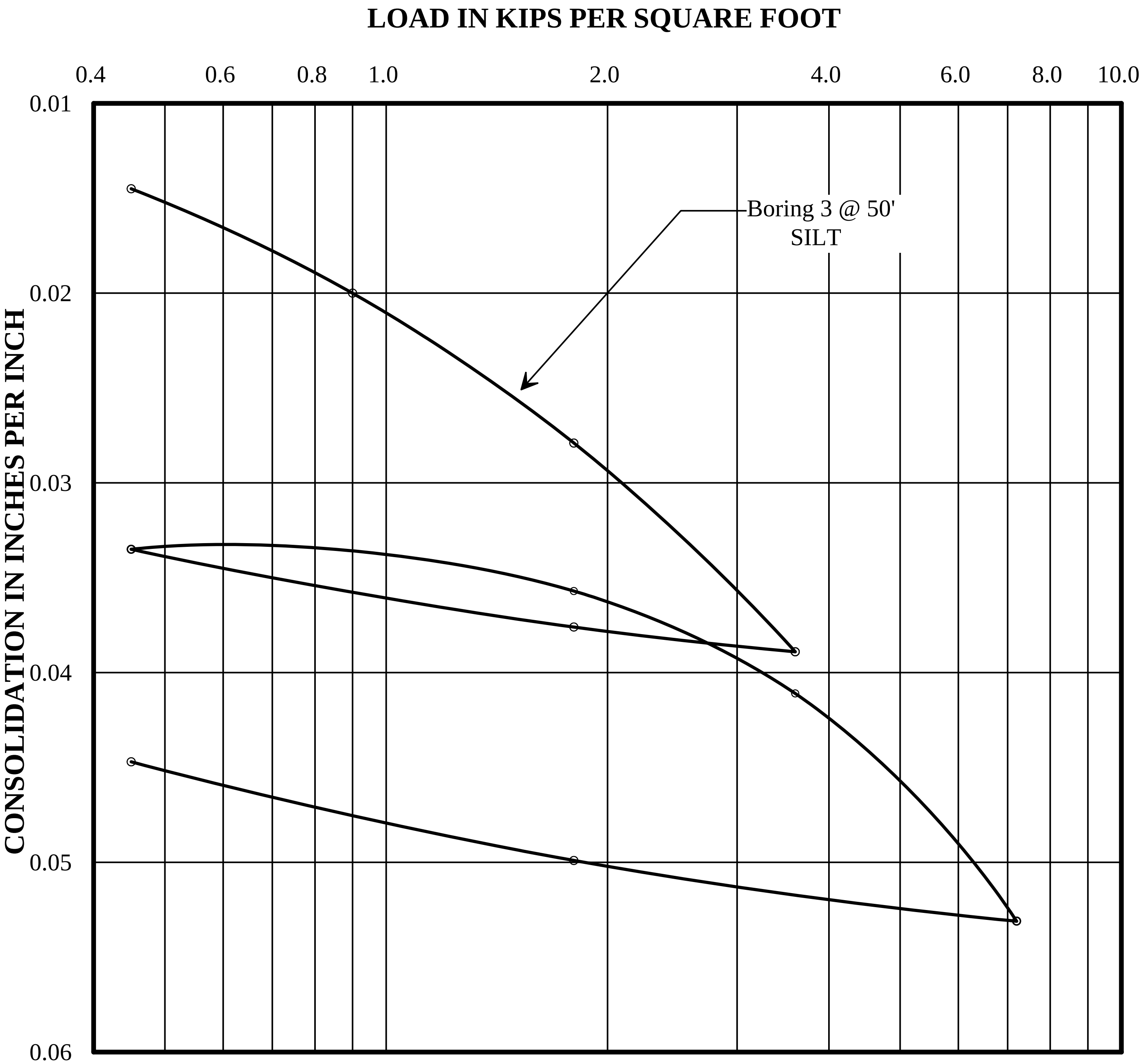
Note: Tests run at field moisture content.

Prepared/Date: WL 4/11/2016
Checked/Date: LH 4/28/2016

Proposed Belmont Village
10822 Wilshire Boulevard and
10812 Ashton Avenue
Los Angeles, California

wood.

CONSOLIDATION TEST DATA
Project 4953-16-0251
Figure A-4.1



Note: Tests run at field moisture content.

Prepared/Date: WL 4/11/2016
Checked/Date: LH 4/28/2016

Proposed Belmont Village
10822 Wilshire Boulevard and
10812 Ashton Avenue
Los Angeles, California

wood.

CONSOLIDATION TEST DATA
Project 4953-16-0251
Figure A-4.2

Table 1 - Laboratory Tests on Soil Samples

AMEC Foster Wheeler
Belmont Village-Westwood
Your #4953-16-0251, HDR Lab #16-0233LAB
29-Mar-16

Sample ID			B-1 @ 27.5' SC	B-4 @ 20' CL
Resistivity		**Units**		
as-received		ohm-cm	4,800	1,280
saturated		ohm-cm	2,680	560
pH			6.5	6.1
Electrical Conductivity		mS/cm	0.04	0.19
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	27	18
magnesium	Mg^{2+}	mg/kg	11	10
sodium	Na^{1+}	mg/kg	48	176
potassium	K^{1+}	mg/kg	6.9	6.8
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	76	58
fluoride	F^{1-}	mg/kg	7.2	5.6
chloride	Cl^{1-}	mg/kg	4.8	224
sulfate	SO_4^{2-}	mg/kg	18	48
phosphate	PO_4^{3-}	mg/kg	11	7.3
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	ND	ND
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G-187, Cations per ASTM 6919, Anions per ASTM 4327, and Alkalinity per AWWA 4110B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Figure A-5

Appendix B

PRIOR FIELD EXPLORATIONS AND LABORATORY TEST RESULTS

Appendix B
Prior Field Explorations and Laboratory Test Results

Our predecessor firms of LeRoy Crandall and Associates and MACTEC previously performed subsurface exploration and laboratory testing at the site. Boring logs are presented in Figures B-1.1 through B-1.6. The following laboratory test results are presented:

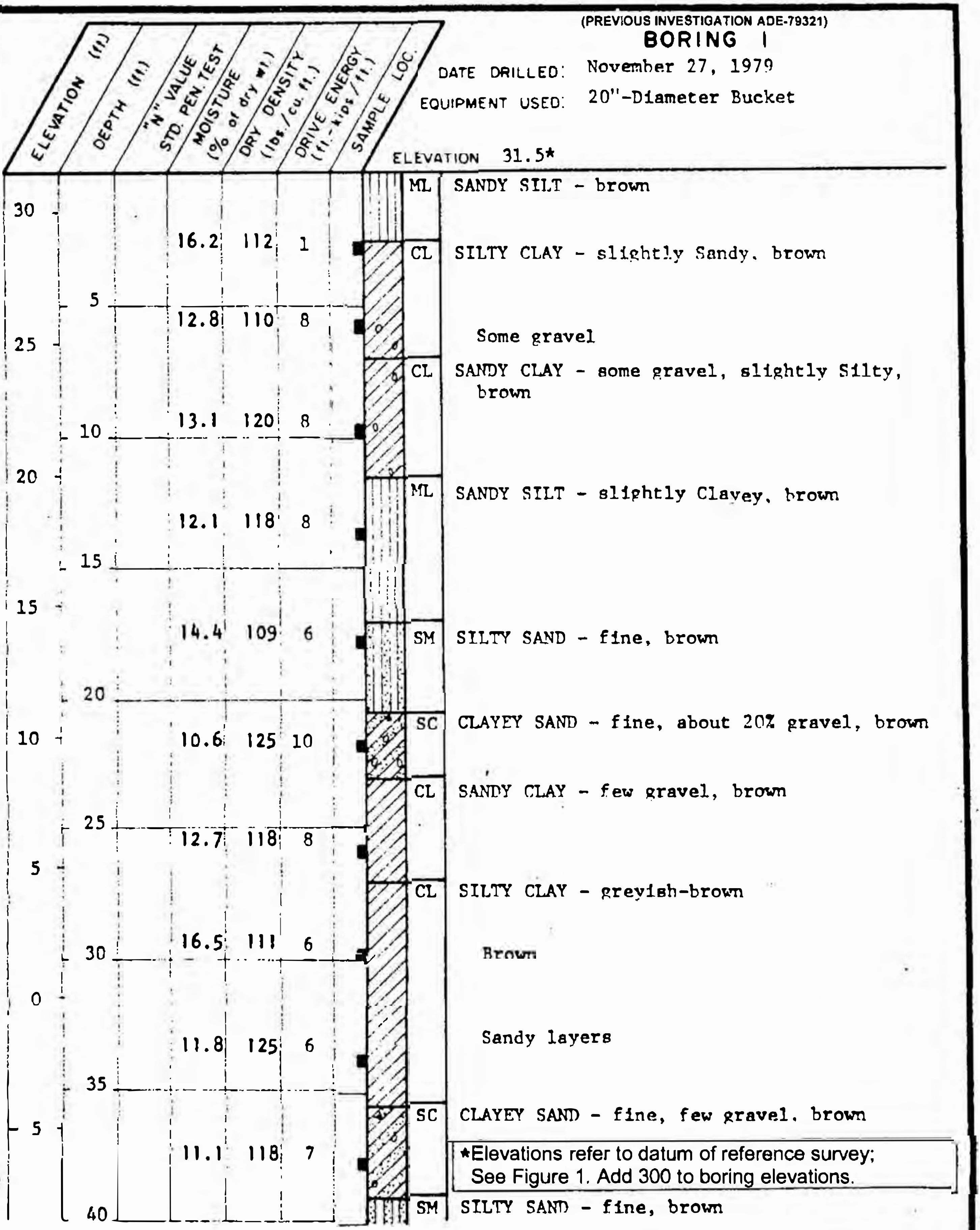
- Moisture and density: presented on the boring logs.
- Direct shear: presented in Figure B-3.
- Consolidation: presented in Figures B-4.1 through B-4.4.
- Expansion Index presented in Figure B-5.

(PREVIOUS INVESTIGATION ADE-79321)

BORING 1

DATE DRILLED: November 27, 1979
EQUIPMENT USED: 20"-Diameter Bucket

ELEVATION 31.5*



MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)
16.2	112	1
12.8	110	8
13.1	120	8
12.1	118	8
14.4	109	6
10.6	125	10
12.7	118	8
16.5	111	6
11.8	125	6
11.1	118	7

ML SANDY SILT - brown

CL SILTY CLAY - slightly Sandy, brown

Some gravel

CL SANDY CLAY - some gravel, slightly Silty, brown

ML SANDY SILT - slightly Clayey, brown

SM SILTY SAND - fine, brown

SC CLAYEY SAND - fine, about 20% gravel, brown

CL SANDY CLAY - few gravel, brown

CL SILTY CLAY - greyish-brown

Brown

Sandy layers

SC CLAYEY SAND - fine, few gravel, brown

*Elevations refer to datum of reference survey; See Figure 1. Add 300 to boring elevations.

SM SILTY SAND - fine, brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LeROY CRANDALL AND ASSOCIATES

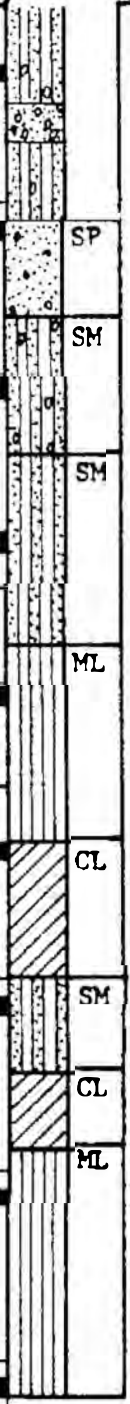
FIGURE B-1.1

BORING 1 (CONTINUED)

DATE DRILLED: November 27. 1979

EQUIPMENT USED: 20"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.
-10			18.4	105	9	■
	45					
			5.4	118	12	■
-15						
			4.7	119	8	■
	50					
-20						
			12.4	111	12	■
	55					
-25						
			20.2	103	5	■
	60					
-30			22.9	103	5	■
	65		13.2	116	10	■
-35						
	70					
			9.5	118	7	■
-40						
	75					
			11.7	122	10	■
-45						
	80					



Few lenses of Silt

Some gravel

Layer of Sand, well graded. about 30% gravel

SP SAND - fine, about 20% gravel, light brown

SM SILTY SAND - well graded, about 20% gravel, light brown

SM SILTY SAND - fine, brown

Lenses of Silt

ML SANDY SILT - brown

Some lenses of Clay

CL SILTY CLAY - brown

SM SILTY SAND - fine. brown

CL SILTY CLAY - brown

ML SANDY SILT - lenses of Clay, brown

NOTE: Water not encountered. No caving.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.1b

(PREVIOUS INVESTIGATION ADE-79321)

BORING 2

DATE DRILLED: November 30, 1979

EQUIPMENT USED: 20"-Diameter Bucket

ELEVATION 32.6

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							ML	FILL - SILT - dark brown
							ML	SANDY SILT - dark brown
30			9.9	114	5	■		
							CL	SILTY CLAY - few gravel, brown, mottled with grey and red
	5							
			10.5	121	10	■		
							CL	SANDY CLAY - few gravel, brown
25								
	10		13.1	118	5	■		
							CL	SILTY CLAY - brown
20								
			19.1	105	5	■		
	15							
							SM	SILTY SAND - fine, few gravel, brown
15			8.1	100	6	■		
							SW	SAND - well graded, about 20% gravel, light brown
	20							
							ML	SANDY SILT - brown
10			19.5	108	3	■		
							ML	CLAYEY SILT - brown
	25							
			13.5	120	13	■		
5								
							CL	SILTY CLAY - brown
	30		13.8	119	9	■		
0								Layer of Clayey Silt, brown
			17.5	110	5	■		
	35							
							ML	CLAYEY SILT - brown
-5			17.4	110	6	■		
	40							

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB DATE 12-11-79 DR. W.P. CHKD

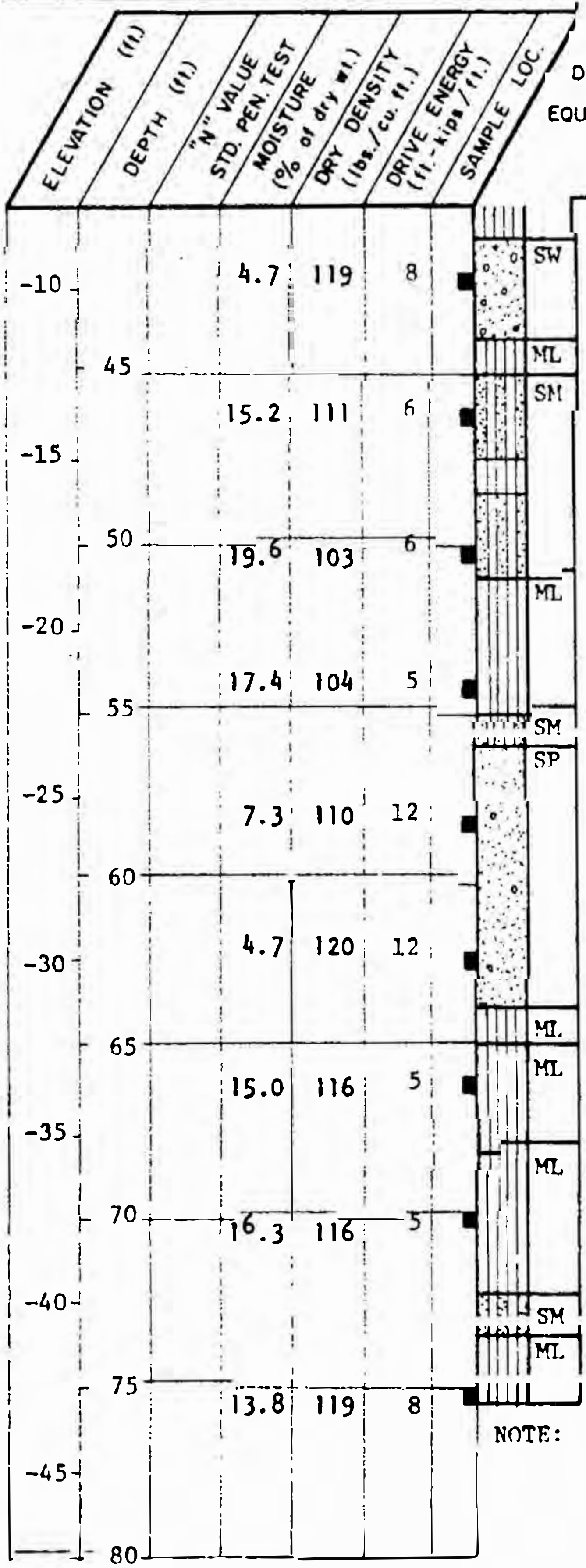
LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.2a

BORING 2 (CONTINUED)

DATE DRILLED: November 30, 1979

EQUIPMENT USED: 20"-Diameter Bucket



MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
4.7	119	8
15.2	111	6
19.6	103	6
17.4	104	5
7.3	110	12
4.7	120	12
15.0	116	5
16.3	116	5
13.8	119	8

Sandy, few gravel

SW SAND - well graded, about 20% gravel, light brown and grey

ML SANDY SILT - light brown

SM SILTY SAND - very fine, light brown to brown

Layer of Clayey Silt, brown

ML SANDY SILT - brown

SM SILTY SAND - fine, light brown

SP SAND - fine, light brown
About 10% gravel
Lenses of Silt

ML CLAYEY SILT - brown

ML SANDY SILT - slightly Clayey, brown

ML CLAYEY SILT - some Sand, brown

SM SILTY SAND - fine, brown

ML SANDY SILT - brown

NOTE: Water seepage encountered at 72½'. Water level measured at a depth of 72½' 15 minutes after completion of drilling. Patchy caving from 58' to 64' (to 2½' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

(PREVIOUS INVESTIGATION ADE-79321)

BORING 3

DATE DRILLED: November 29, 1979

EQUIPMENT USED: 20"-Diameter Bucket

ELEVATION 28.8

ELEVATION (ft.)	DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)	SAMPLE LOC.		
							SM	2" ASPHALTIC PAVING FILL - SILTY SAND - fine, patches of Silt, mottled light brown
			11.8	108	1	■		
25							ML	SANDY SILT - dark brown
	5		15.8	112	1	■	CL	SILTY CLAY - brown
20			13.7	120	11	■		Slightly Sandy
	10							
							SM	SILTY SAND - fine, brown
			10.5	123	11	■		Well graded, about 10% gravel
15	15							
			6.4	115	10	■	SW	SAND - well graded, about 10% gravel, light brown
10	20							
			6.6	113	10	■		
							CL	SILTY CLAY - slightly Sandy, brown, mottled with grey and red
5	25		12.3	122	16	■		
							ML	CLAYEY SILT - slightly Sandy, brown
0			8.7	125	7	■		Layer of Silty Sand, fine, brown
	30							
			14.0	119	8	■		
-5							CL	SILTY CLAY - few gravel, greyish-brown
	35							

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Form 123 JOB A-79321 DATE 12-11-79 DR. [illegible] DE. [illegible] W.P. [illegible] CHKD [illegible]

LOG OF BORING

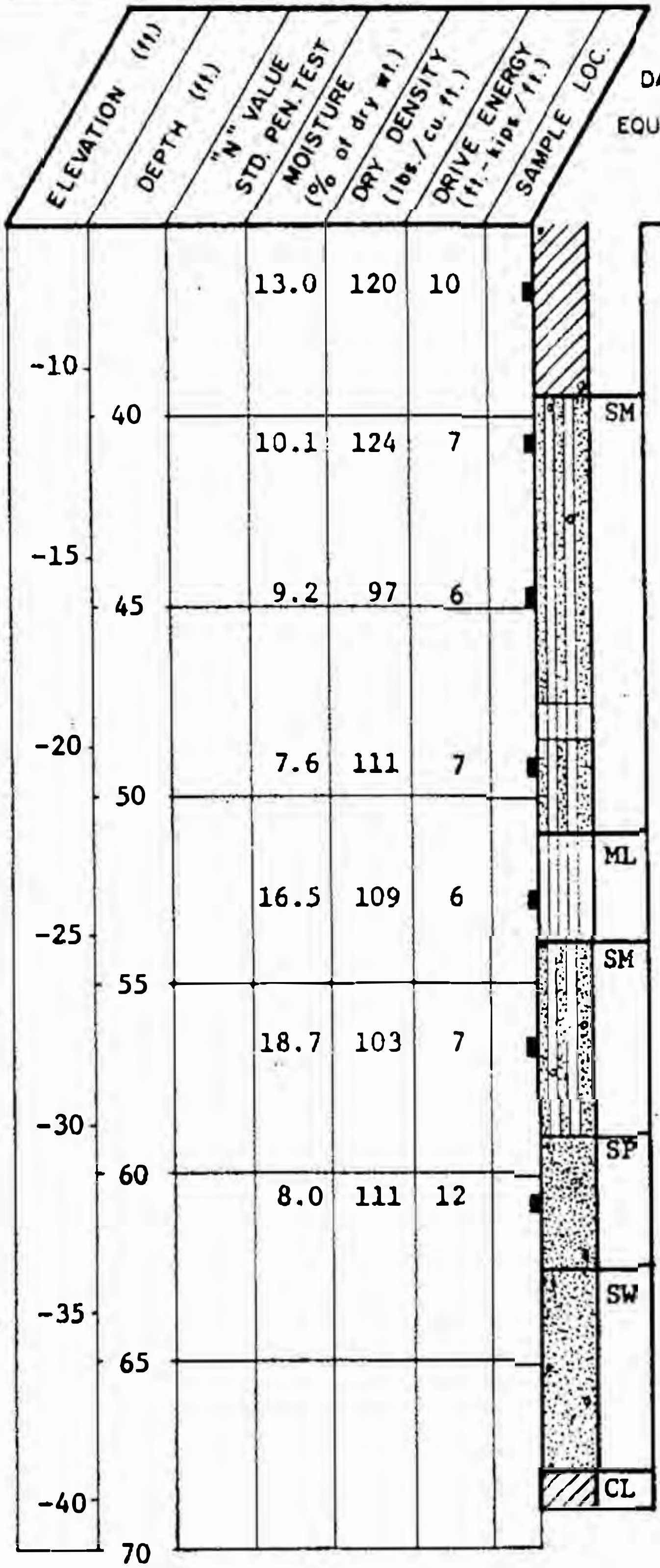
LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.3a

BORING 3 (CONTINUED)

DATE DRILLED: November 29, 1979

EQUIPMENT USED 20"-Diameter Bucket



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	DRIVE ENERGY (ft.-kips/ft.)
	13.0	120	10
	10.1	124	7
	9.2	97	6
	7.6	111	7
	16.5	109	6
	18.7	103	7
	8.0	111	12

SM SILTY SAND - fine, about 20% gravel, light brown

Light brown
Few or no gravel

Layer of Clayey Silt, brown

ML SANDY SILT - lenses of Clayey Silt, light brown

SM SILTY SAND - very fine, light brown

Few gravel

SP SAND - fine, about 20% gravel, light brown

X

SW SAND - well graded, about 30% gravel, light brown

About 40% gravel

CL SILTY CLAY - brown
(BORING TERMINATED DUE TO CAVING)

NOTE: Water encountered at 63'. Water level measured at a depth of 63' 15 minutes after completion of drilling. Heavy caving below 59' (to 3½' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

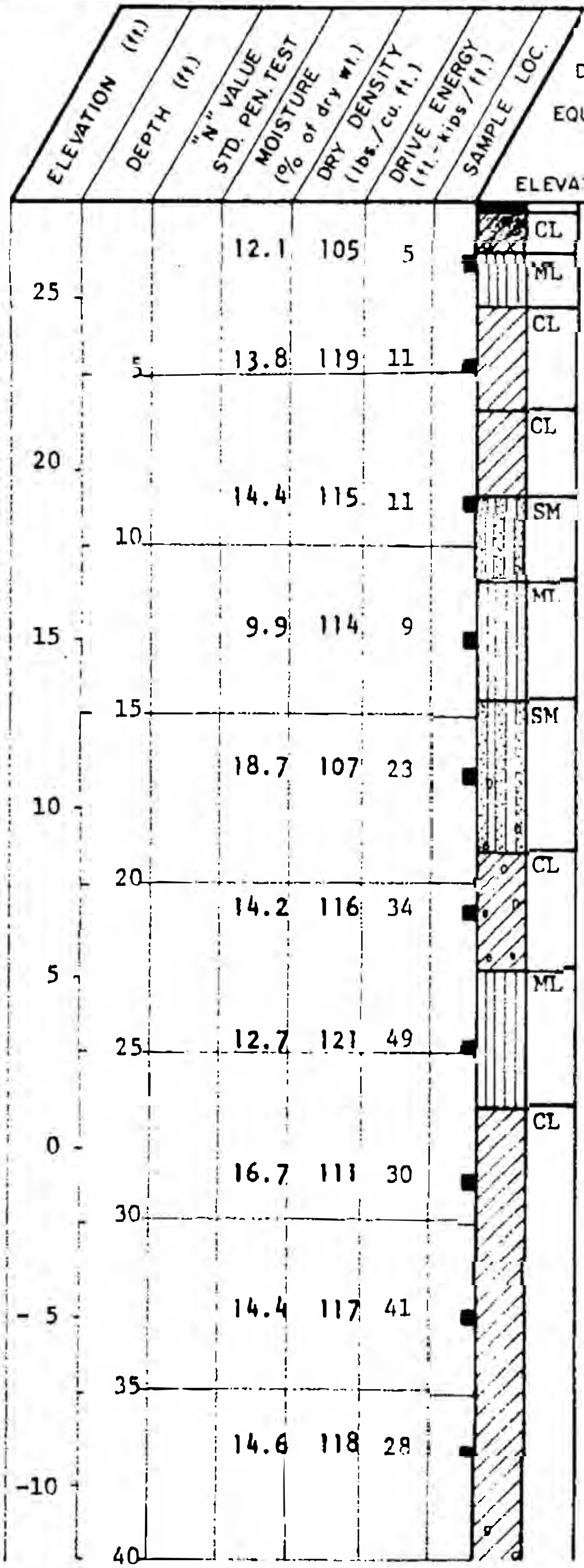
(PREVIOUS INVESTIGATION ADE-79321)

BORING 4

DATE DRILLED: December 3, 1979

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 27.8



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
	12.1	105	5
	13.8	119	11
	14.4	115	11
	9.9	114	9
	18.7	107	23
	14.2	116	34
	12.7	121	49
	16.7	111	30
	14.4	117	41
	14.6	118	28

4" CONCRETE SLAB

CL FILL - CLAY - few gravel, some debris, brown

ML SANDY SILT - dark brown

CL SILTY CLAY - brown

CL SANDY CLAY - brown

SM SILTY SAND - fine, brown

ML CLAYEY SILT - brown

SM SILTY SAND - fine, brown

Few gravel

CL SANDY CLAY - about 10% gravel, brown

ML CLAYEY SILT - slightly Sandy, brown

CL SILTY CLAY - slightly Sandy, brown with red and grey

Brown

Few gravel

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

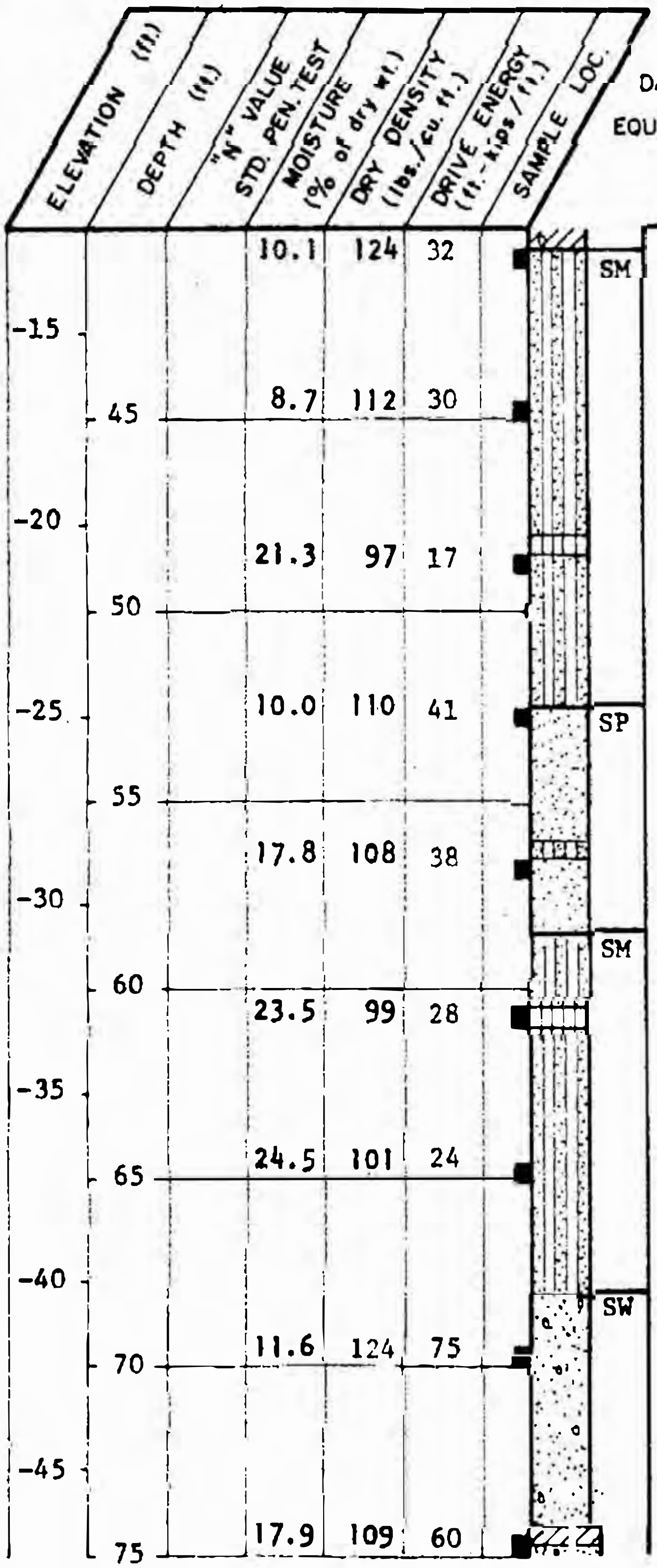
LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.4a

BORING 4 (CONTINUED)

DATE DRILLED: December 3, 1979

EQUIPMENT USED: 5"-Diameter Rotary Wash



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
	10.1	124	32
	8.7	112	30
	21.3	97	17
	10.0	110	41
	17.8	108	38
	23.5	99	28
	24.5	101	24
	11.6	124	75
	17.9	109	60

SM SILTY SAND - fine, brown

Layer of Sandy Silt, brown

SP SAND - fine, light brown

Few gravel
Layer of Silty Sand

SM SILTY SAND - fine, light brown

Lense of Silt

SW SAND - well graded, about 20% gravel, layers of fine Sand, brown with light grey

Layer of Sandy Clay, brown

(CONTINUED ON FOLLOWING PLATE)

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

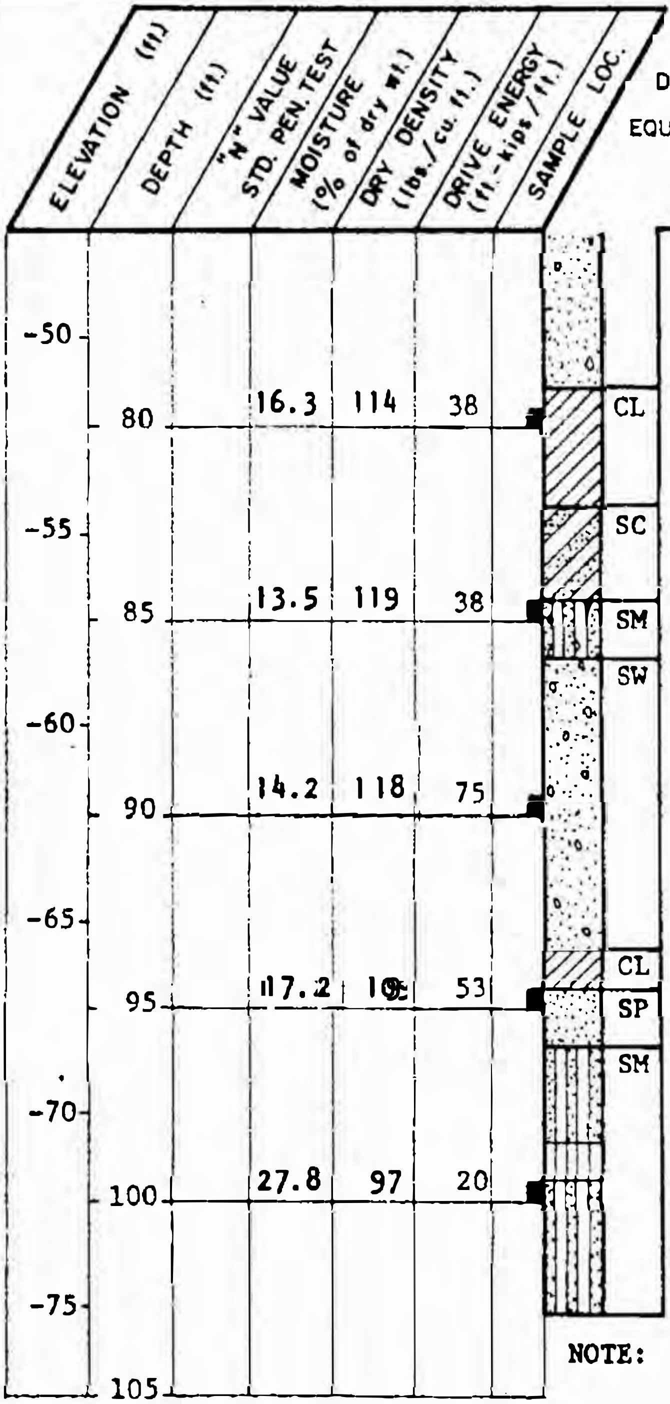
FIGURE B-1.4b

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 4 (CONTINUED)

DATE DRILLED: December 3, 1979

EQUIPMENT USED: 5"-Diameter Rotary Wash



DEPTH (ft.)	"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
80		16.3	114	38
85		13.5	119	38
90		14.2	118	75
95		17.2	109	53
100		27.8	97	20

CL	SANDY CLAY - slightly Silty, brown
SC	CLAYEY SAND - fine, brown
SM	SILTY SAND - fine, about 20% gravel, brown
SW	SAND - well graded, about 30% gravel, light grey and brown
CL	SANDY CLAY - slightly Silty, brown
SP	SAND - fine, brown
SM	SILTY SAND - fine, brown and grey
	Layer of Sandy Silt, brown and grey

(4)

NOTE: Drilling mud used in drilling process. 2" PVC installed for downhole seismic survey; lower 10' perforated and wrapped with screen; annular space around outside of pipe back-filled with gravel. Water level measured at a depth of 61' on 12/11/79.

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.4c

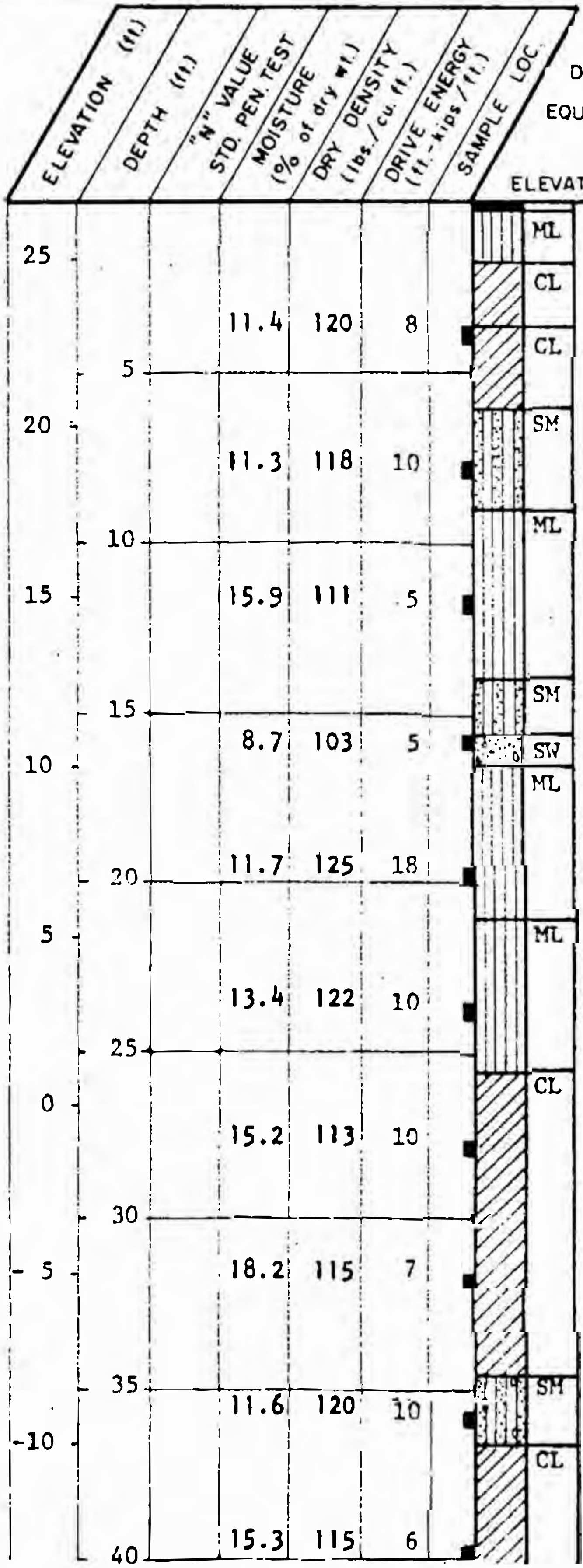
(PREVIOUS INVESTIGATION ADE-79321)

BORING 5

DATE DRILLED: November 28, 1979

EQUIPMENT USED: 20"-Diameter Bucket

ELEVATION 26.7



MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
11.4	120	8
11.3	118	10
15.9	111	5
8.7	103	5
11.7	125	18
13.4	122	10
15.2	113	10
18.2	115	7
11.6	120	10
15.3	115	6

3" CONCRETE PAVING

ML SANDY SILT - dark brown

CL SILTY CLAY - slightly Sandy, brown

CL SANDY CLAY - some Silt, few gravel, brown

SM SILTY SAND - fine, slightly Clayey, few gravel, brown

ML SANDY SILT - slightly Clayey, brown

SM SILTY SAND - fine, brown

SW SAND - well graded, about 20% gravel, light brown

ML SANDY SILT - greyish-brown

About 30% gravel

ML CLAYEY SILT - some Sand, brown

CL SILTY CLAY - few gravel, brown and greyish-brown

SM SILTY SAND - fine, lenses of Silt, few gravel, brown

CL SILTY CLAY - brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

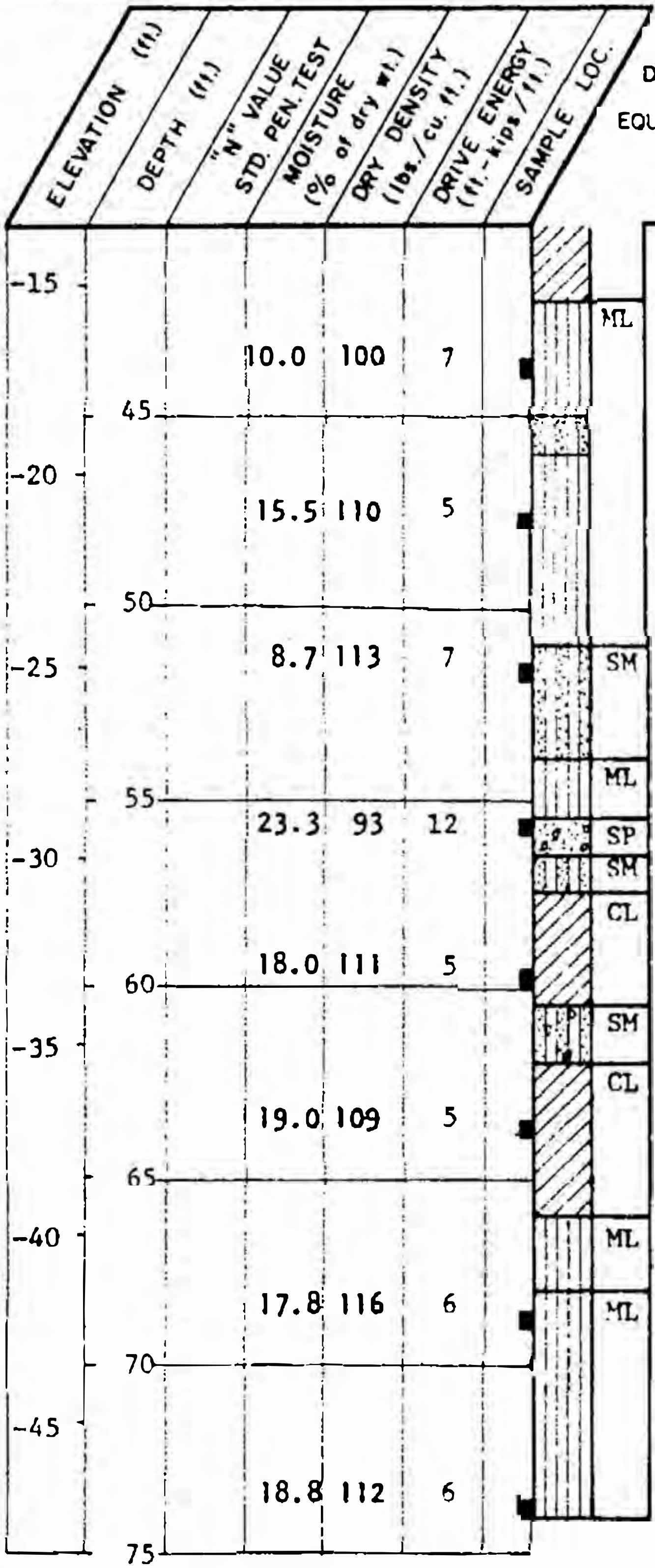
LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.5a

BORING 5 (CONTINUED)

DATE DRILLED: November 28, 1979

EQUIPMENT USED: 20"-Diameter Bucket



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
	10.0	100	7
	15.5	110	5
	8.7	113	7
	23.3	93	12
	18.0	111	5
	19.0	109	5
	17.8	116	6
	18.8	112	6

Few gravel

ML SANDY SILT - light brown

Layer of Silty Sand, fine, light brown

Lenses of Clay

Brown

X

SM SILTY SAND - fine, light brown

ML SANDY SILT - brown

SP SAND - fine, about 20% gravel, light brown

SM SILTY SAND - fine, brown

CL SILTY CLAY - brown and greyish-brown

Some streaks of Sand

SM SILTY SAND - fine, few gravel, freyish-brown

CL SILTY CLAY - brown

ML SANDY SILT - brown

ML CLAYEY SILT - slightly Sandy, brown

NOTE: Water seepage encountered at 60½'. Water level at 71' 40 minutes after completion of drilling. Sloughing from 60½' to 62' (to 3' in diameter).

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

LOG OF BORING

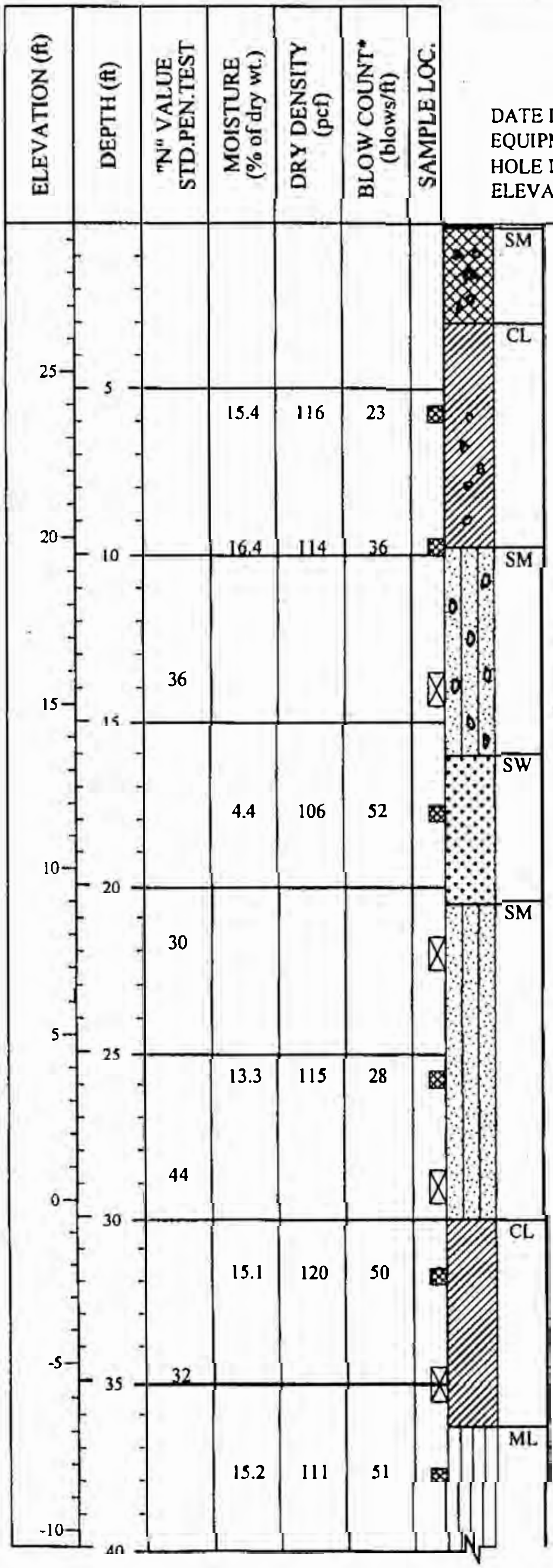
LeROY CRANDALL AND ASSOCIATES

FIGURE B-1.5b

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING 6**

DATE DRILLED: January 10, 2003
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 29.5



"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
					SM	1½" Asphaltic Paving FILL - SILTY SAND - fine to coarse, some Gravel and pieces of Brick, brown
					CL	SILTY CLAY - light brown
	15.4	116	23	▩		Some Sand and Gravel
	16.4	114	36	▩	SM	SILTY SAND - fine to medium, about 10% Gravel, light brown
36				☒		
					SW	SAND - well graded, light brownish grey
	4.4	106	52	▩		
					SM	SILTY SAND - fine, light brown
30				☒		
	13.3	115	28	▩		Lenses of Clay and Silt
44				☒		Brown
					CL	SILTY CLAY - dark brown
	15.1	120	50	▩		
32				☒		
					ML	SANDY SILT - lenses of Silty Sand, light brown
	15.2	111	51	▩		

* Number of blows required to drive the Crandall sampler 12 inches using a 140 pound hammer falling 30 inches.

** BORING 6 WAS DRILLED RECENTLY FOR CURRENT INVESTIGATION.

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: AR
Prepared By: MM
Checked By: MS

Proposed High-Rise Condominium
Wilshire Blvd. and Malcolm Ave.
Los Angeles, California

MACTEC

LOG OF BORING
Project: 70131-2-0276
Figure: B-1.6a

BORING 6** (Continued)

DATE DRILLED: January 10, 2003
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 29.5

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
		28				☒	SM	SILTY SAND - fine to medium, light brown
			12.3	104	28	▨		
-15	45							
		32				☒		
-20	50		21.0	96	31	▨		
								Lenses of Sandy Silt
		40				☒		
-25	55							
							SP	SAND - fine, light brown
			7.1	98	46	▨		
-30	60						SM	SILTY SAND - fine, lenses of Sandy Silt, light brown
		49				☒		
-35	65							
			15.0	117	55	▨	SP	SAND - fine to medium, about 15% Gravel, lenses of Silty Sand, light brown
-40	70	43				☒		
			-	-	62	○		Sample not recovered
-45	75							
							CL	SANDY CLAY - light brown
		34				☒		
								Layer of Silty Sand
-50	80		19.5	110	28	▨		END OF BORING AT 80 FEET

NOTE: To obtain future water level measurement and sampling, installed a 2-inch diameter PVC pipe to 80'. Pipe perforated below 50 feet. Backfilled with sand to within 50' of ground surface and grouted with cement above 45'. A bentonite plug placed between 45' and 50' of ground surface. Water level measured in the monitoring well at a depth of 65', 10 minutes later.

Field Tech: AR
Prepared By: MM
Checked By:

B12SOIL CRANDALL 20276.GPJ LAW CRAN.GDT 1/30/03

Proposed High-Rise Condominium
Wilshire Blvd. and Malcolm Ave.
Los Angeles, California

MACTEC

LOG OF BORING
Project: 70131-2-0276 Figure: B-1.6b

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

Undisturbed Sample	Auger Cuttings
Split Spoon Sample	Bulk Sample
Rock Core	Crandall Sampler
Dilatometer	Pressure Meter
Packer	○ No Recovery
▽ Water Table at time of drilling	▼ Water Table after 24 hours

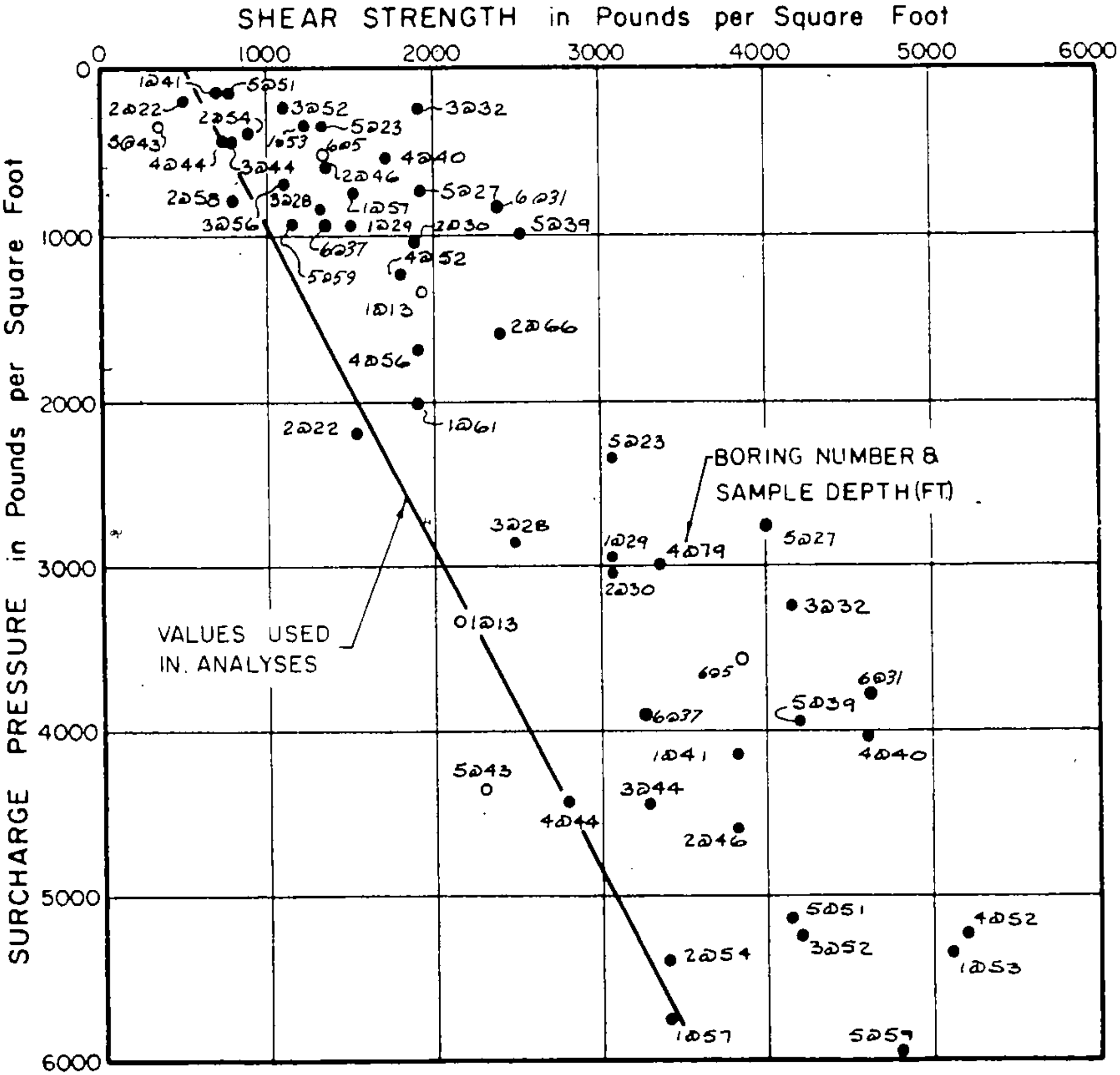
Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 20	Firm	5 - 8	Firm
21 - 30	Very Firm	9 - 15	Stiff
31 - 50	Dense	16 - 30	Very Stiff
Over 50	Very Dense	Over 31	Hard

KEY TO SYMBOLS AND DESCRIPTIONS

MACTEC

FIGURE B-2



KEY:

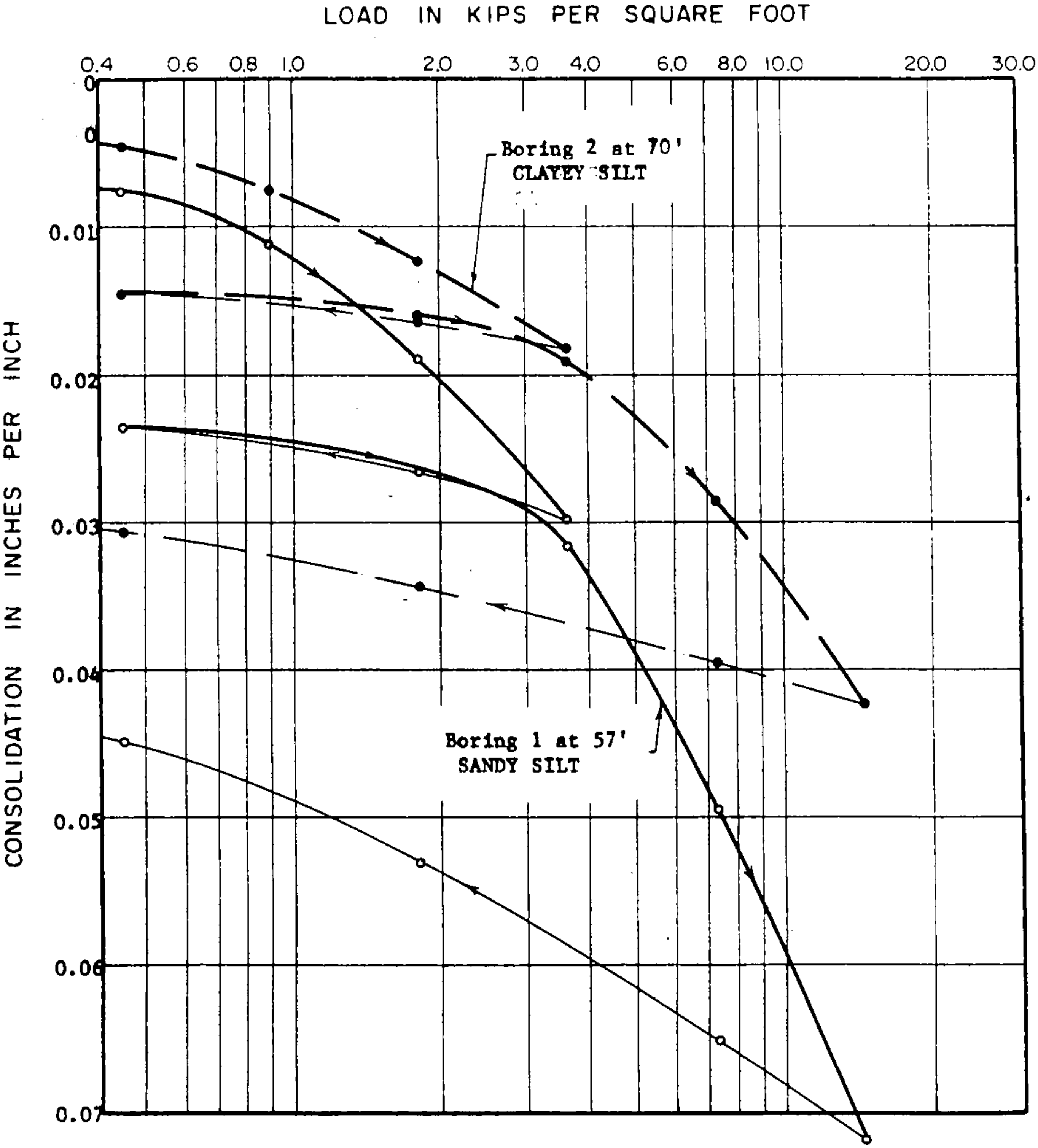
- ● Tests at field moisture content
- ○ Tests at increased moisture content

NOTE:
Boring 6 was drilled recently.

DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE B-3

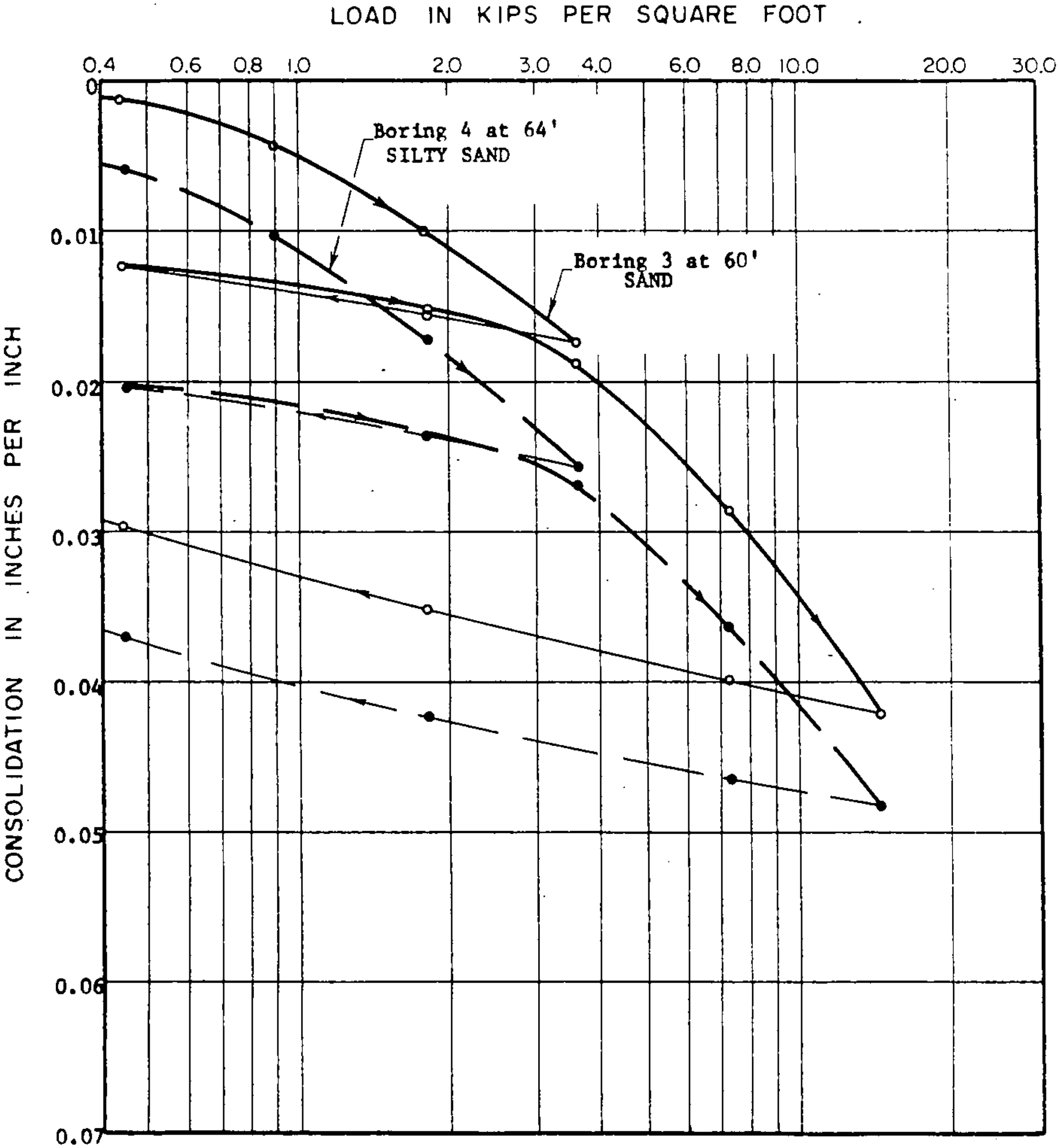


NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE B-4.1

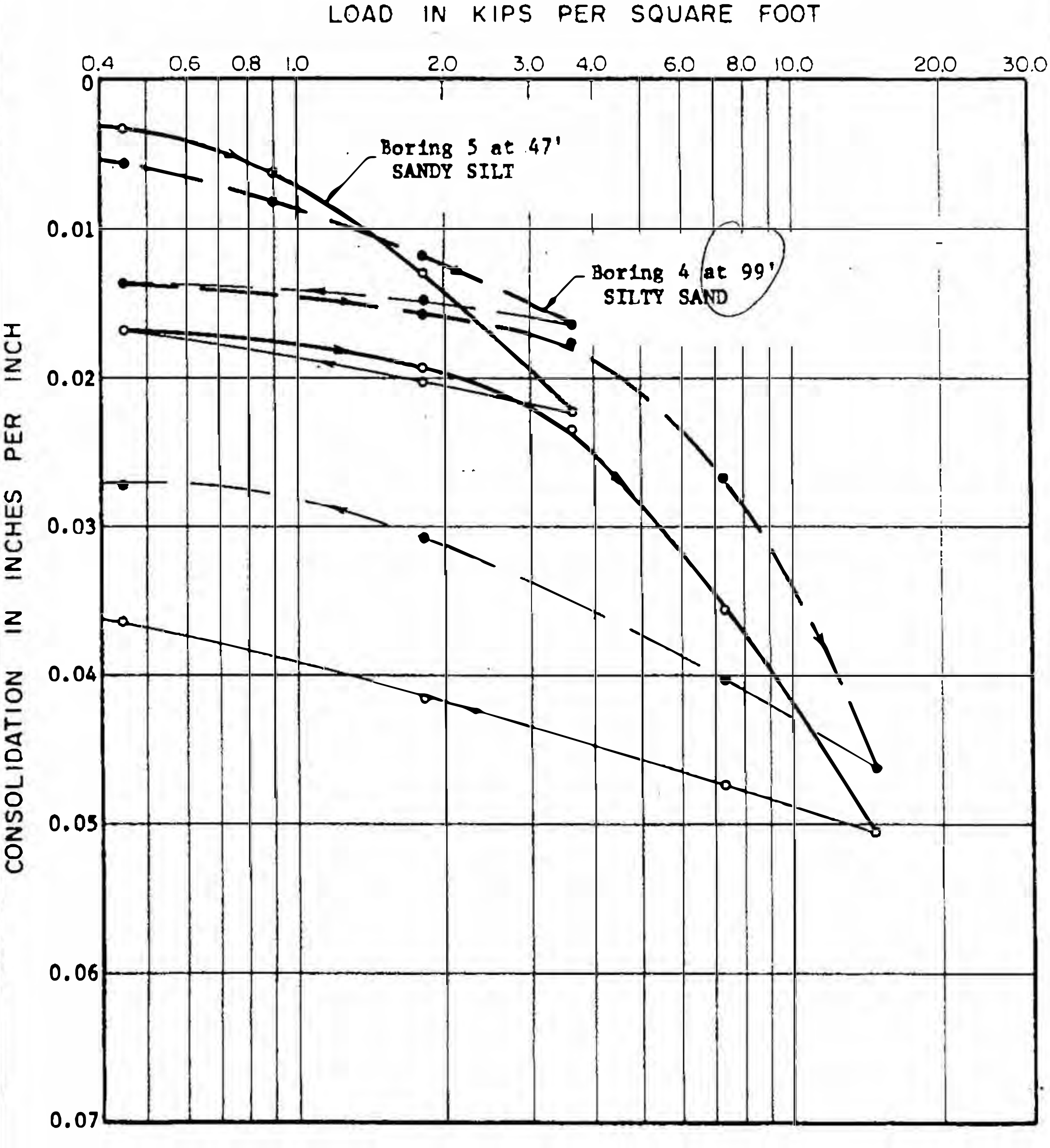


NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE B-4.2



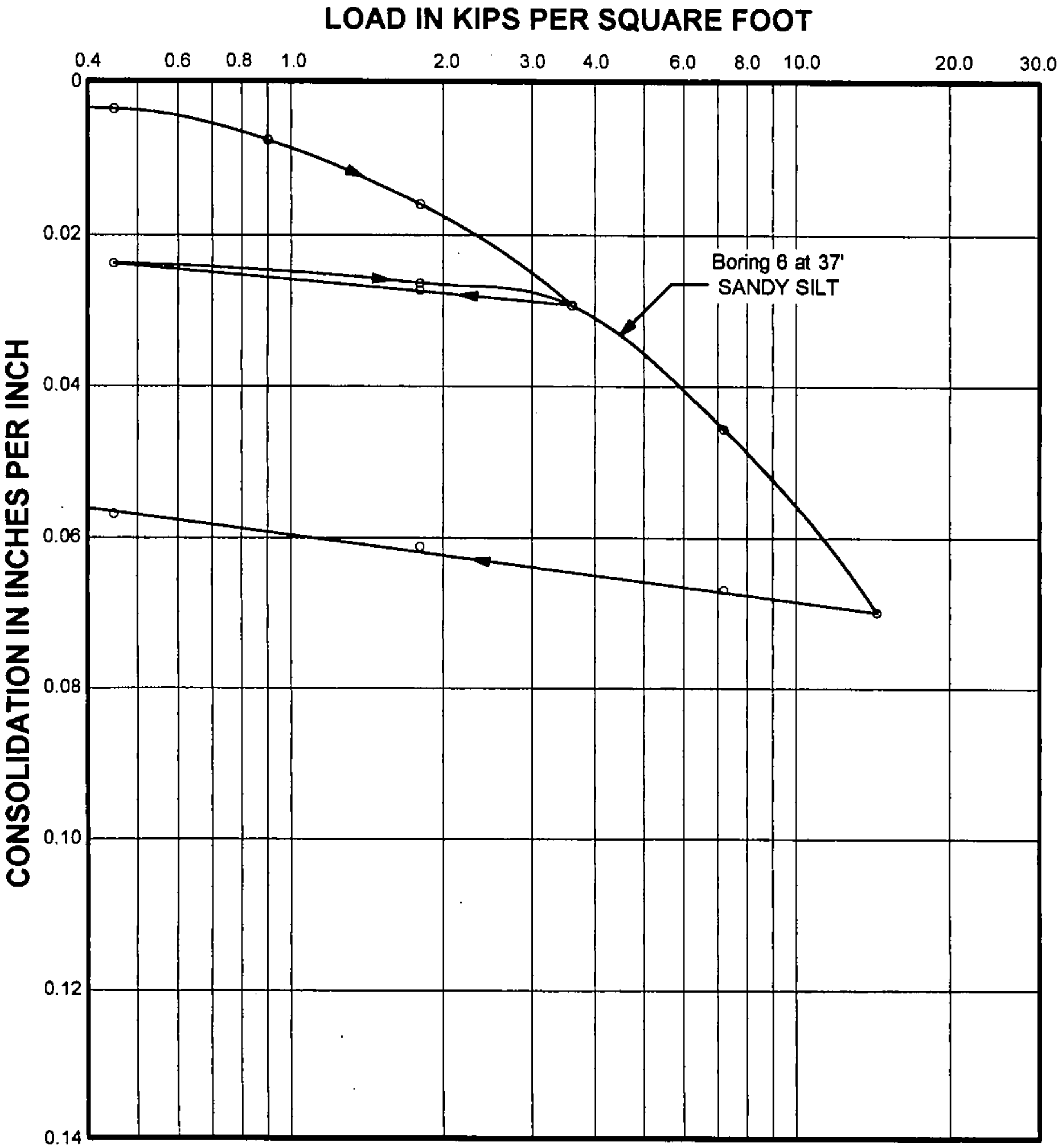
NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE B-4.3

JOB 701312-0276 DATE 1/23/03 DR MM OE MS CHKD MP



NOTE: Sample tested at field moisture content.

CONSOLIDATION TEST DATA

MACTEC

FIGURE B - 4.4

BORING NUMBER AND SAMPLE DEPTH:	6 at 3' to 8'
SOIL TYPE:	SILTY CLAY
CONFINING PRESSURE: (lbs./sq.ft.)	144
INITIAL MOISTURE CONTENT: (% of dry wt.)	10.1
FINAL MOISTURE CONTENT: (% of dry wt.)	26.5
DRY DENSITY: (lbs./cu.ft.)	109
EXPANSION INDEX:	74

TEST METHOD: ASTM Designation D4829-88

EXPANSION INDEX TEST DATA

MACTEC

JOB 70131-2-0276 DATE 1/20/03 DR MM OE MS CHKD

FIGURE B - 5

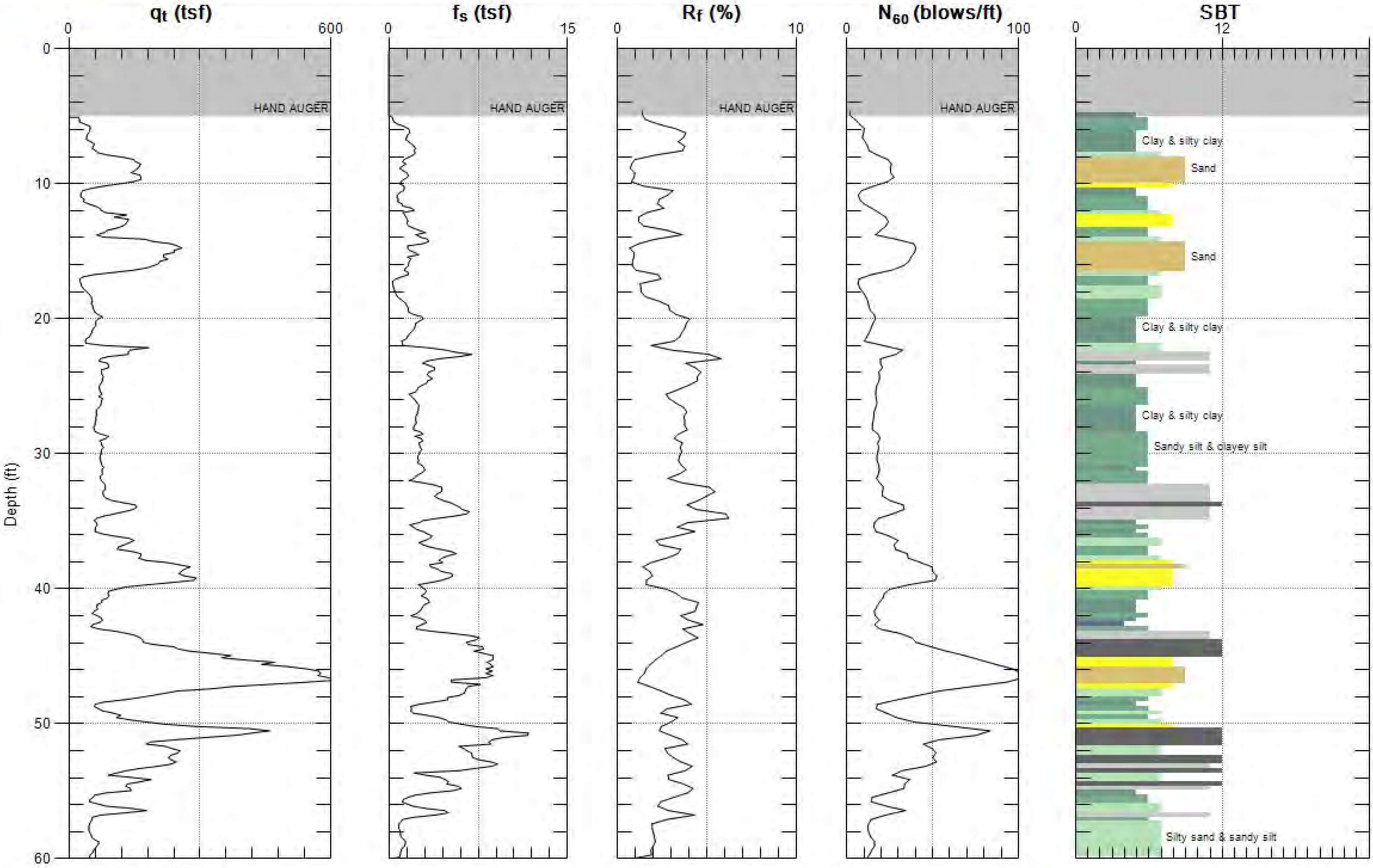
Appendix C

CONE PENETRATION TEST (CPT) RESULTS

GREGG **AMEC FW**

Site: PRESBYTERIAN CHURCH **Engineer: L.HONG**

Sounding: CPT-1 **Date: 3/22/2016 08:23**



Max. Depth: 60.203 (ft)
Avg. Interval: 0.328 (ft)

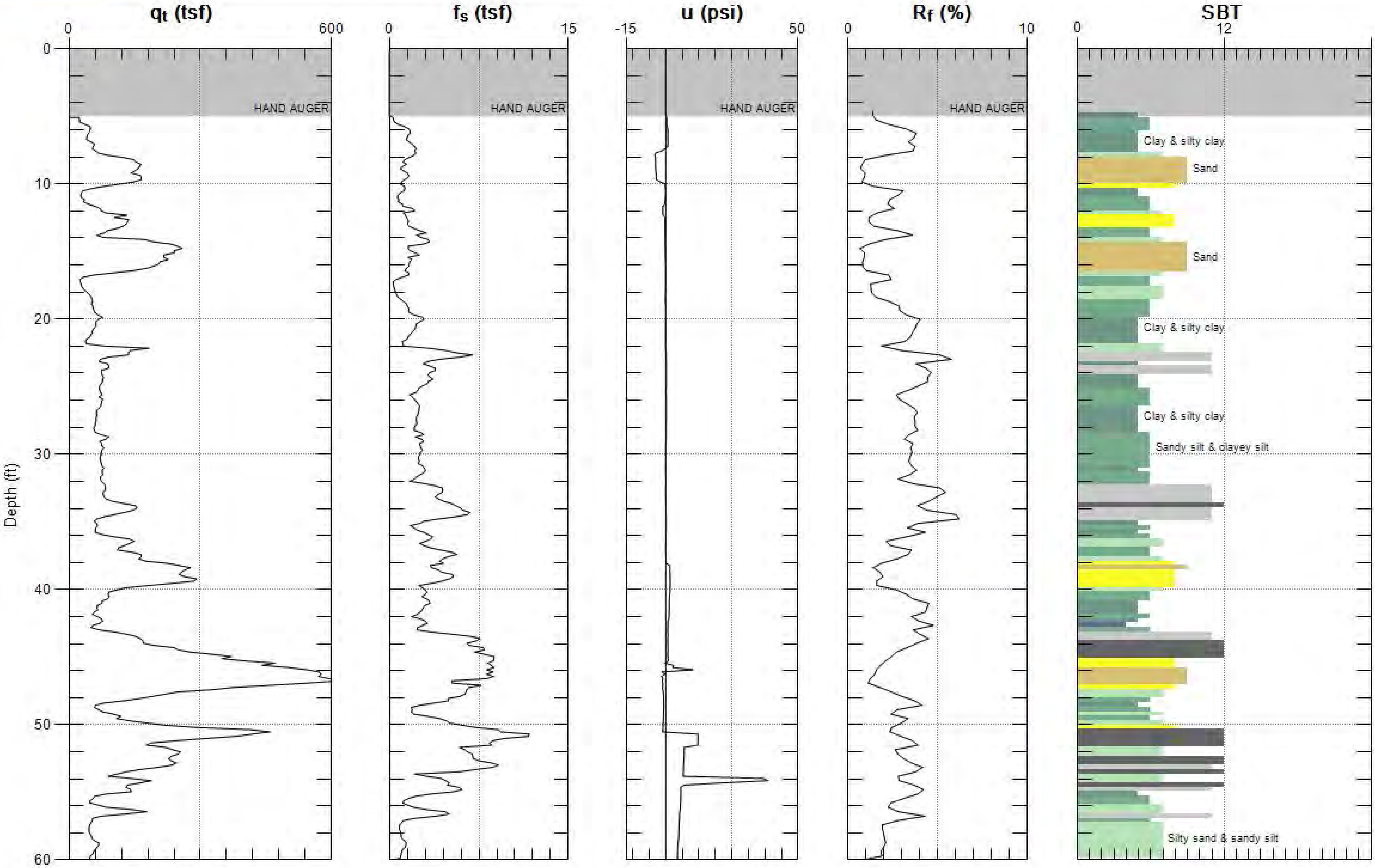
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FW

Site: PRESBYTERIAN CHURCH **Engineer: L.HONG**

Sounding: CPT-1 **Date: 3/22/2016 08:23**



Max. Depth: 60.203 (ft)
Avg. Interval: 0.328 (ft)

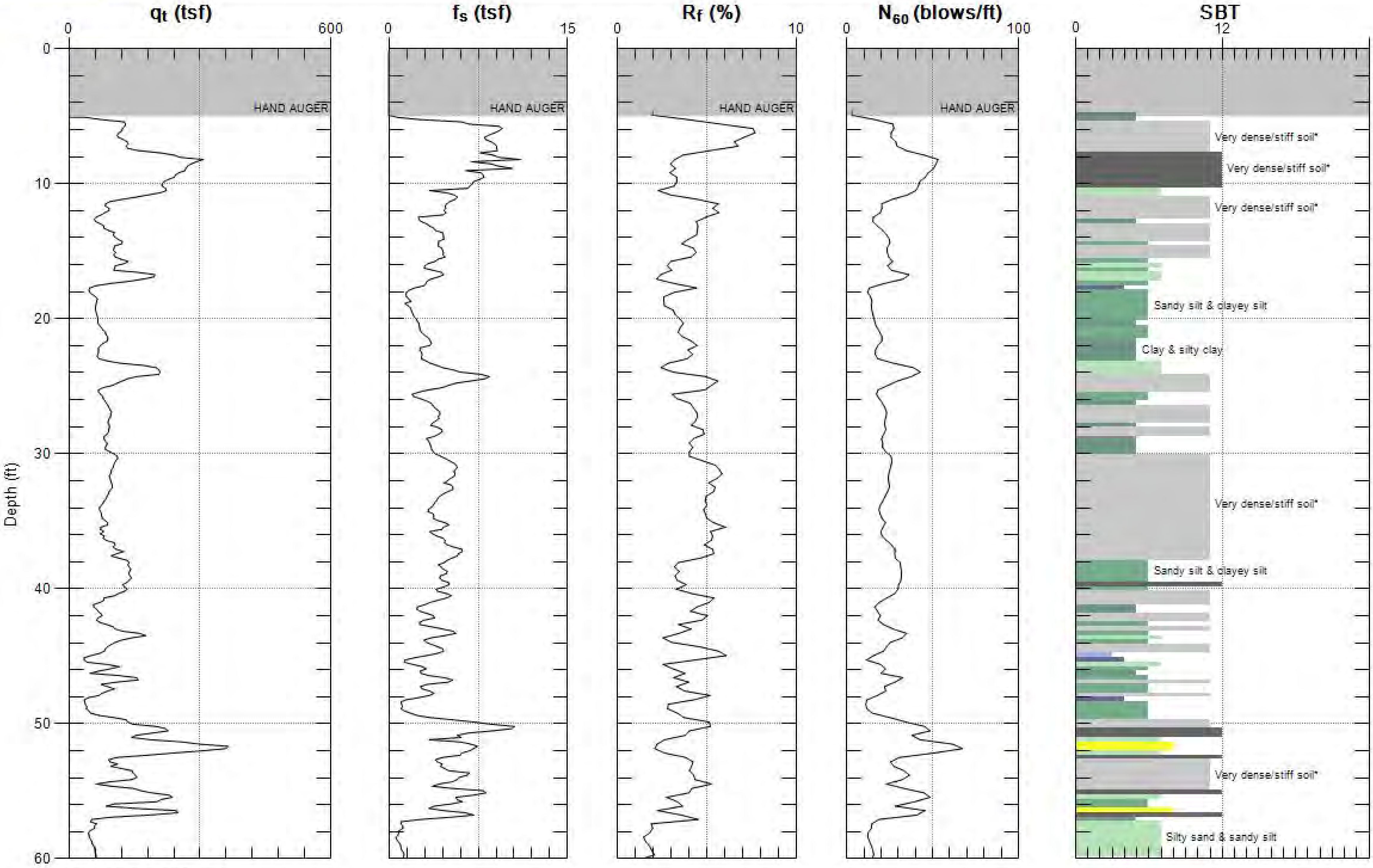
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FW

Site: PRESBYTERIAN CHURCH **Engineer: L.HONG**

Sounding: CPT-2 **Date: 3/22/2016 09:32**



Max. Depth: 60.367 (ft)
Avg. Interval: 0.328 (ft)

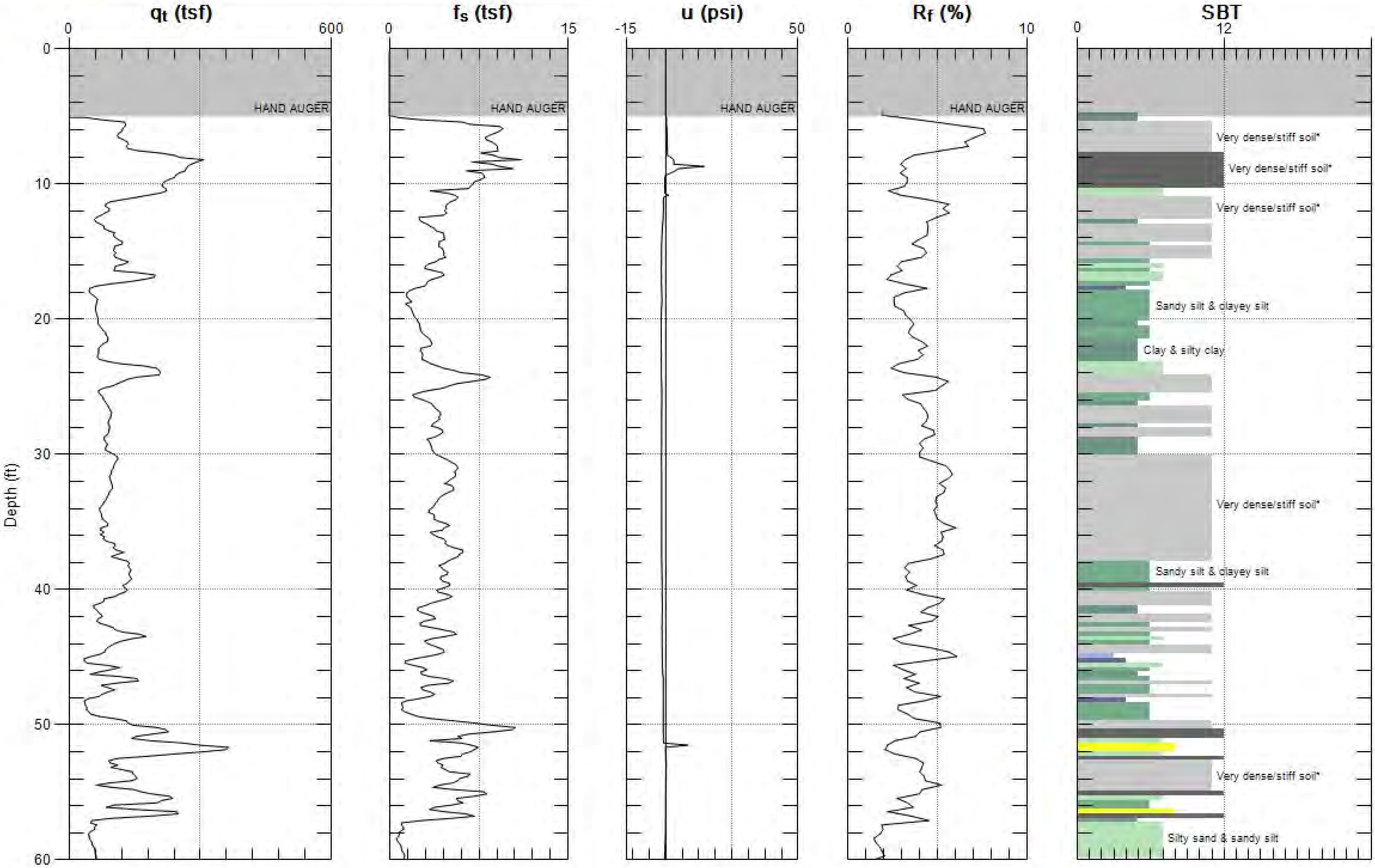
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FW

Site: PRESBYTERIAN CHURCH **Engineer:** L.HONG

Sounding: CPT-2 **Date:** 3/22/2016 09:32



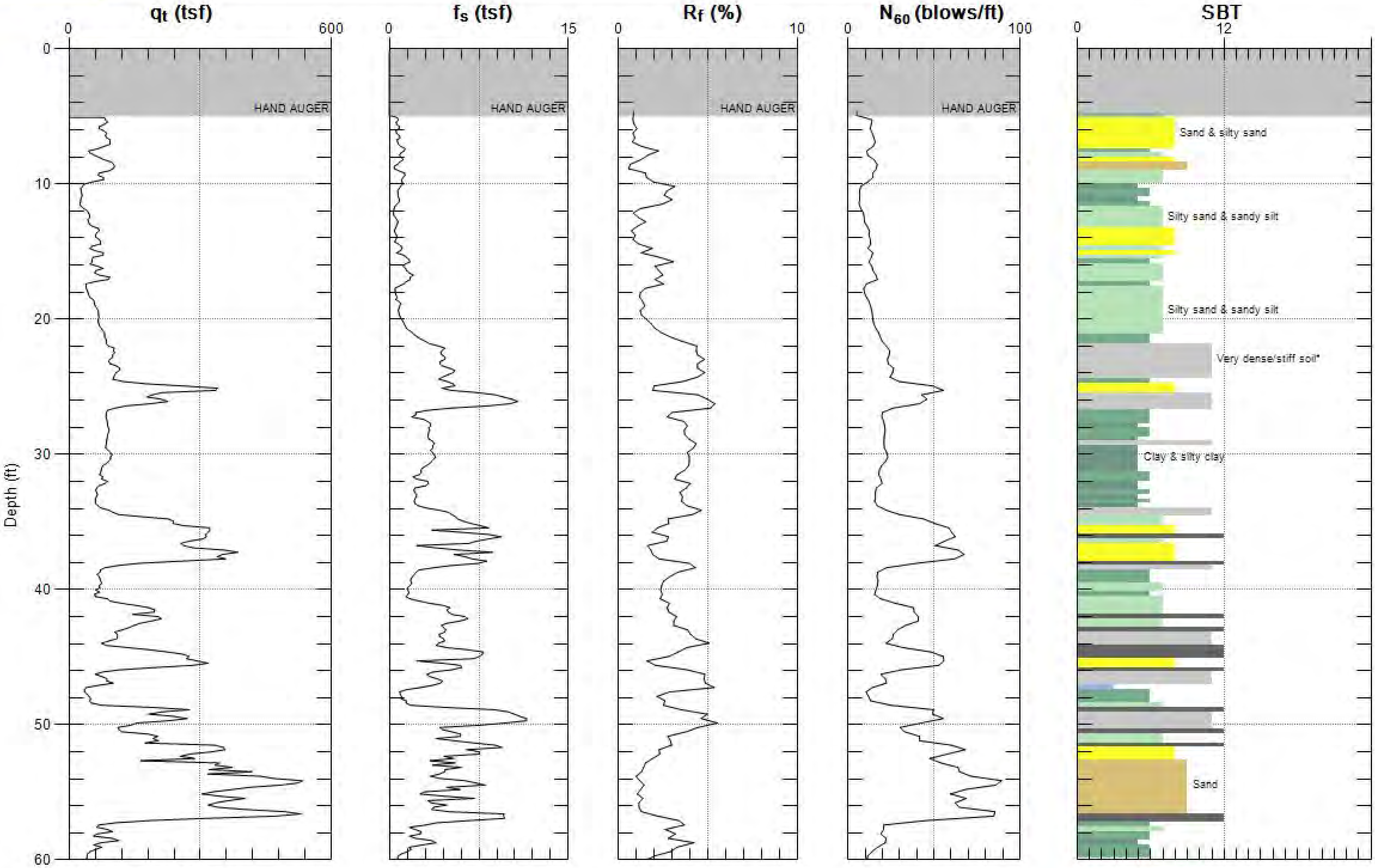
Max. Depth: 60.367 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

AMEC FW

Site: PRESBYTERIAN CHURCH **Engineer: L.HONG**

Sounding: CPT-3 **Date: 3/22/2016 11:40**



Max. Depth: 60.531 (ft)
Avg. Interval: 0.328 (ft)

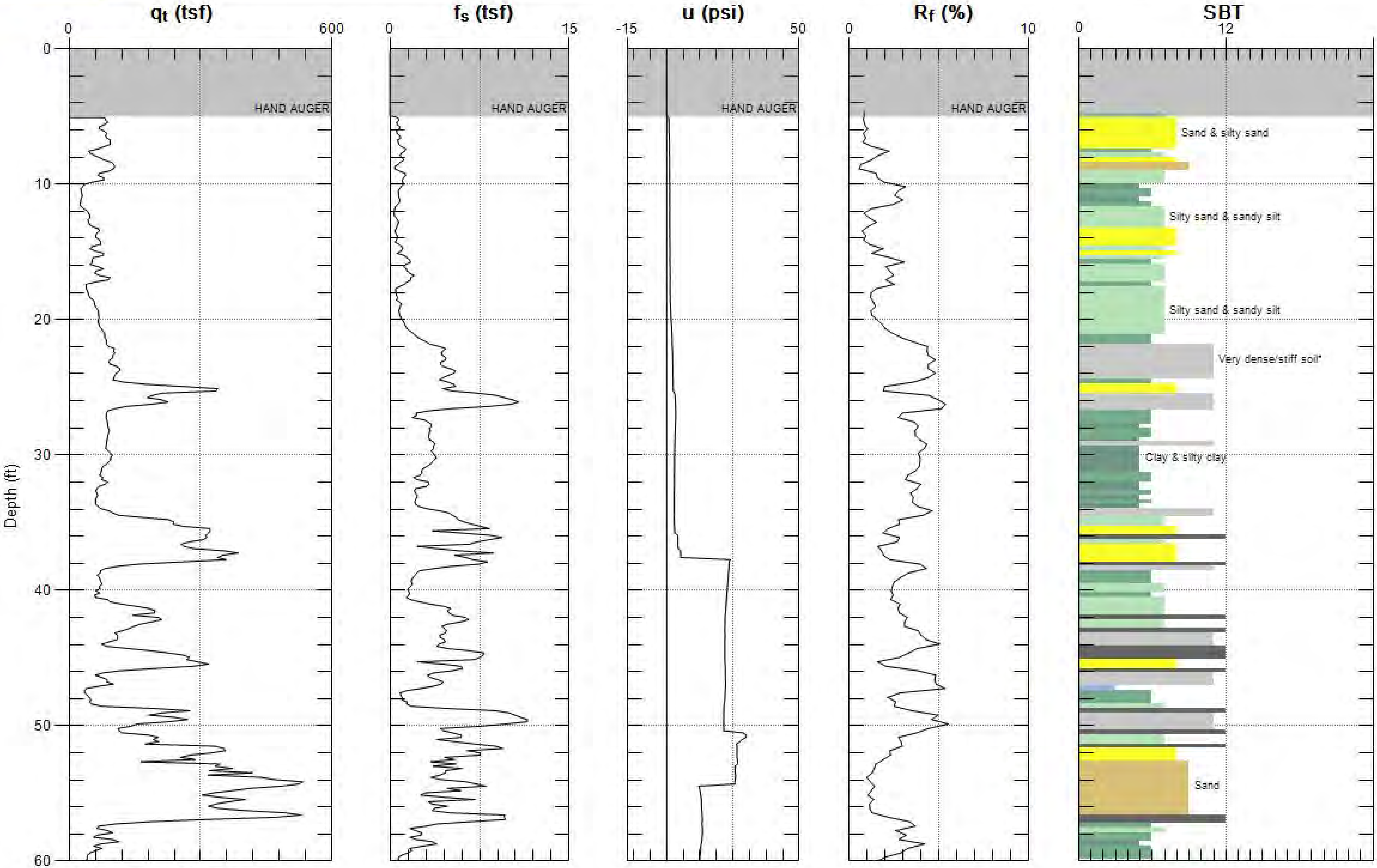
SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FW

Site: PRESBYTERIAN CHURCH **Engineer: L.HONG**

Sounding: CPT-3 **Date: 3/22/2016 11:40**



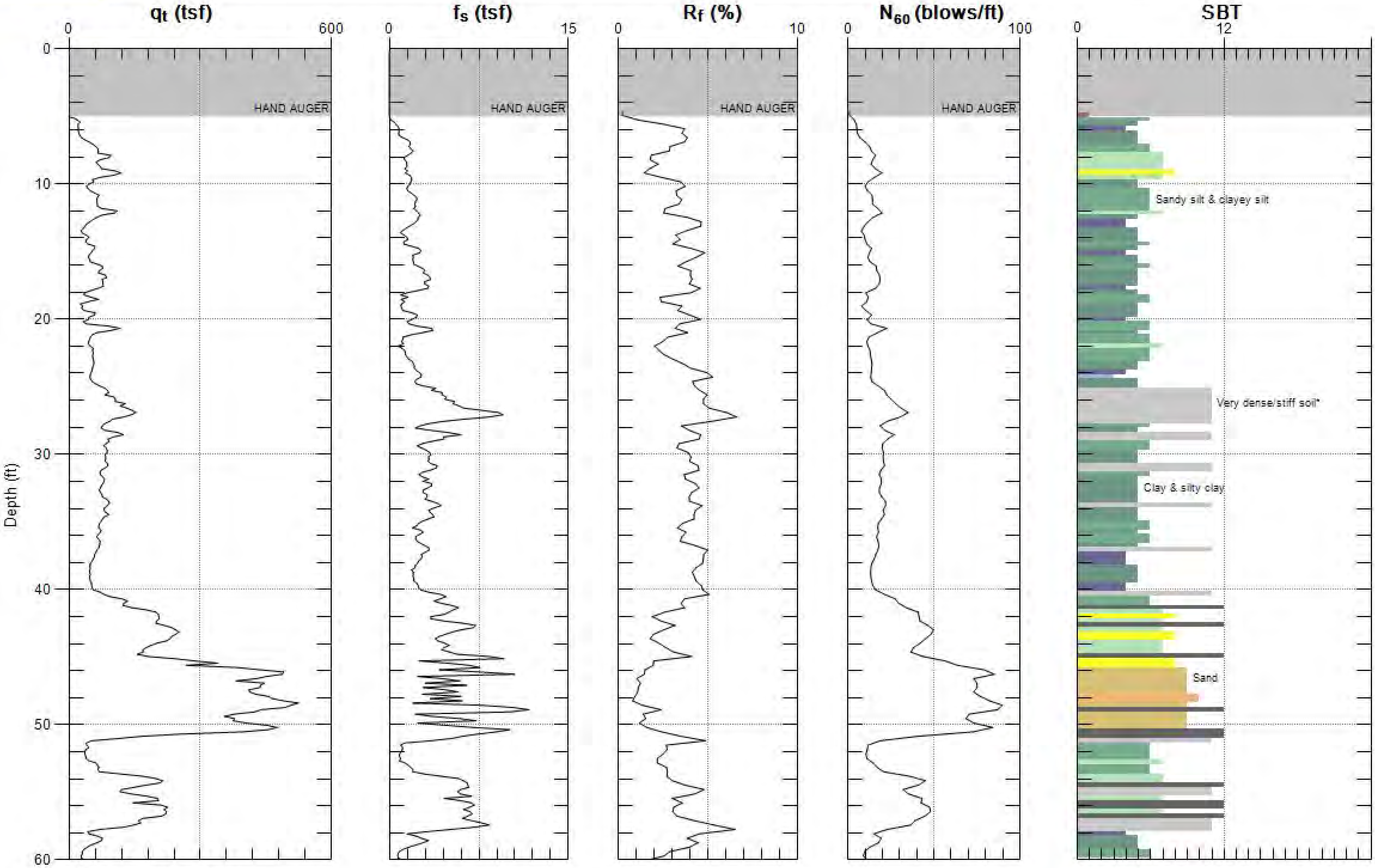
Max. Depth: 60.531 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

AMEC FW

Site: PRESBYTERIAN CHURCH
Engineer: L.HONG
Sounding: CPT-4
Date: 3/22/2016 10:29



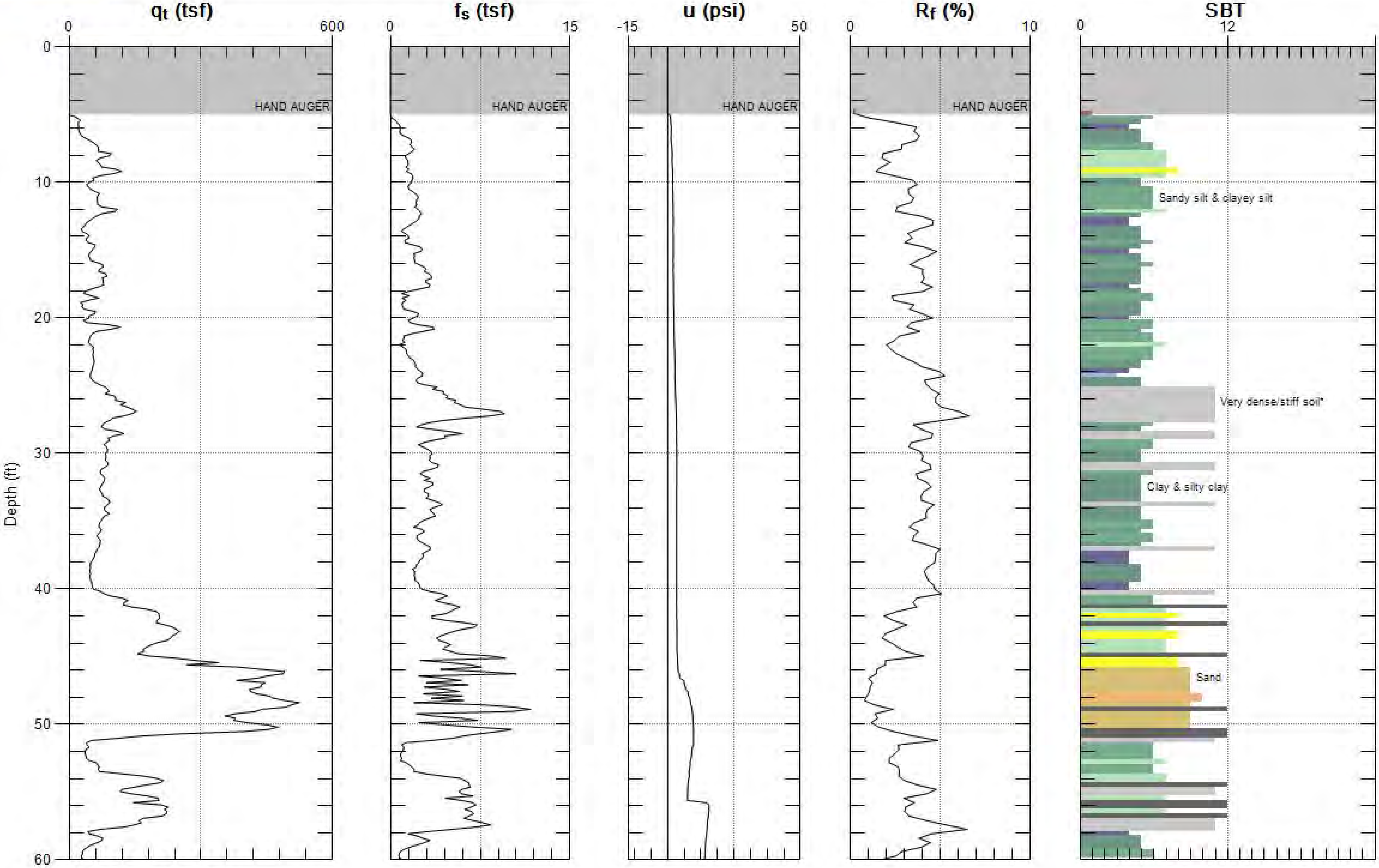
Max. Depth: 60.367 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **AMEC FW**

Site: PRESBYTERIAN CHURCH **Engineer: L.HONG**

Sounding: CPT-4 **Date: 3/22/2016 10:29**



Max. Depth: 60.367 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)