APPENDIX F

GEOTECHNICAL STUDY

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April 16, 2012
4953-10-1031

Mr. Jack Kurchian
President
System, LLC
9034 West Sunset Boulevard
West Hollywood, California 90069

Re: **Revised Supplemental Geotechnical Consultation**
Proposed Melrose Triangle Mixed-Use Project
Between Santa Monica Boulevard, Melrose Avenue and Almont Drive
West Hollywood, California

Dear Mr. Kurchian:

We are pleased to submit the revised results of our supplemental geotechnical consultation for the proposed Melrose Triangle Mixed-Use Project to be constructed between Santa Monica Boulevard, Melrose Avenue and Almont Drive in West Hollywood, California. We previously performed a geotechnical consultation of the current project and presented the results in a letter dated April 9, 2012. In addition, under our predecessor firm of MACTEC Engineering and Consulting, Inc., we prepared a geotechnical consultation for the project in report dated August 27, 2010 (MACTEC Project No. 4953-10-1031). This revised supplemental geotechnical consultation provides an update of our August 27, 2010 report based on modifications to the project and recent changes in the California Building Code. The recommendations in the MACTEC August 27, 2010 report remain applicable as modified by the recommendations contained in this letter.

This letter supersedes our April 9, 2012 letter.

You have provided us with updated drawings for the project dated January 10, 2012. The recent plans show minor shift in building layout as indicated in the attached Figure 1. The project plan remains essential similar with the project planned showing several buildings constructed over a single subterranean structure. The above-grade portion of the buildings shows three to five levels in height of retail, commercial and residential space. The buildings are underlain by three to four level of subterranean set approximately at Elevation 179.5 feet (or 46 feet below the existing ground surface) as shown on Figure 2.

In our opinion, the updated building configuration and information do not have any significant impact on the project as discussed in the August 27, 2010 report and as modified by the recommendations in this letter.

Seismic Coefficient

We updated the seismic site coefficients presented in our August 27, 2010 report in accordance with the 2010 California Building Code (CBC) and ASCE 7-05 Standard (ASCE, 2005) using the United States Geological Survey (USGS, 2007a) Earthquake Motion Parameters, Version 5.1.0,

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program. The site location used was Latitude 34.0812° and Longitude -118.3883° with a Site Class "D." The seismic site coefficients per the CBC are presented below:

Site Coefficient	Value
S_S (0.2 second period, Site Class B)	1.757g
S_1 (1.0 second period, Site Class B)	0.600g
F_a	1.0
F_v	1.5
$S_{MS} = F_a S_S$ (0.2 second period, Site Class D)	1.757g
$S_{M1} = F_v S_1$ (1.0 second period, Site Class D)	0.900g
S_{DS} = 2/3 x S_{MS} (0.2 second period, Site Class D)	1.171g
S_{D1} = 2/3 x S_{M1} (1.0 second period, Site Class D)	0.600g

By: LT 4/3/2012 Chkd ET 4/5/12

GENERAL LIMITATIONS

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, expressed or implied, is made as to the professional advice included in this letter.

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

Sincerely,

AMEC Environment & Infrastructure, Inc.

Lan Anh Tran
Project Engineer

Marshall Lew, PhD.
Senior Principal
Vice President

REGISTERED PROFESSIONAL ENGINEER
MARSHALL LEW
No. 522
EXP. 3-31-13
GEOTECHNICAL
STATE OF CALIFORNIA

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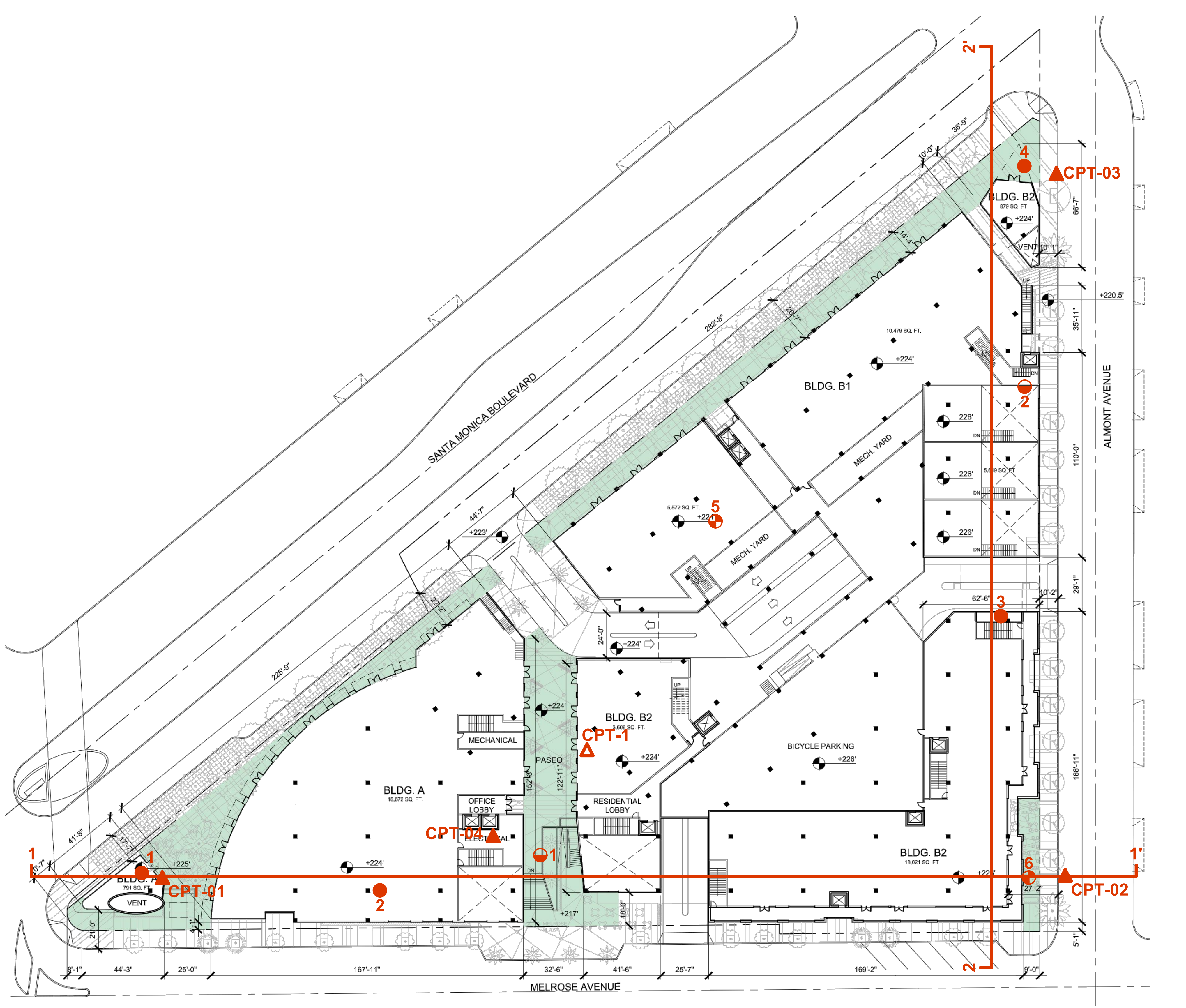
(6 copies submitted)

Attachments:	Figure 1	Plot Plan
	Figure 2	Building Sections



LEGEND:

6◕	PRIOR MACTEC INVESTIGATION (PROJECT NO. 4953-08-0811)
4●	PRIOR MACTEC INVESTIGATION (PROJECT NO. 4953-06-2101)
2◒	PRIOR MACTEC INVESTIGATION (PROJECT NO. A-85280)
	BORING LOCATION AND NUMBER
CPT-1△	PRIOR MACTEC INVESTIGATION (PROJECT NO. 4953-08-0811)
CPT-04▲	PRIOR MACTEC INVESTIGATION (PROJECT NO. 4088-08-7537)
	CONE PENETRATION TEST LOCATION AND NUMBER
2⌊⌋2'	SECTION LINE

REFERENCE:

OVERALL SITE PLAN, MELROSE TRIANGLE,
SHEET NUMBER A3.06 (DATED 01.10.2012)
BY STUDIO ONE ELEVEN.

0 30' 60' 120'
SCALE: 1" = 60'

N

amec

AMEC Environment & Infrastructure
5628 E. Slauson Avenue, Los Angeles, California 90040
Phone (323) 889-5300 Fax (323) 889-5398

PLOT PLAN

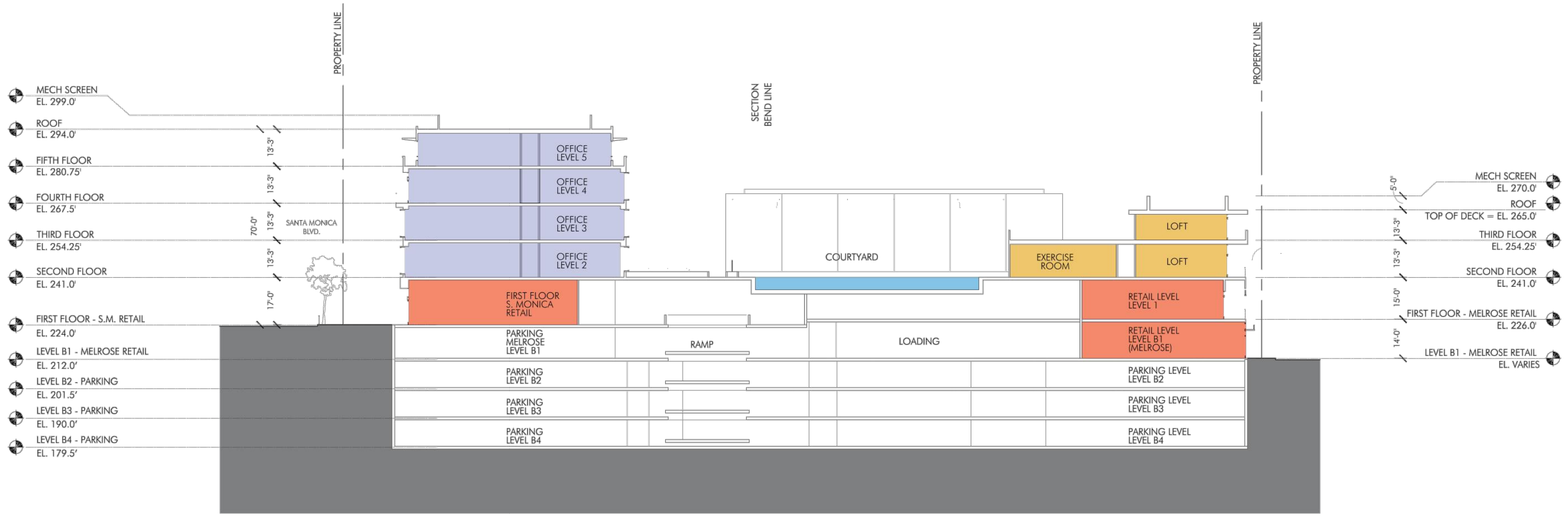
JOB:	4953-10-1031
SCALE:	1" = 60'
DRAWN:	V. Nguyen
CHKD:	L. Tran
PM:	WC
DATE:	4/9/2012

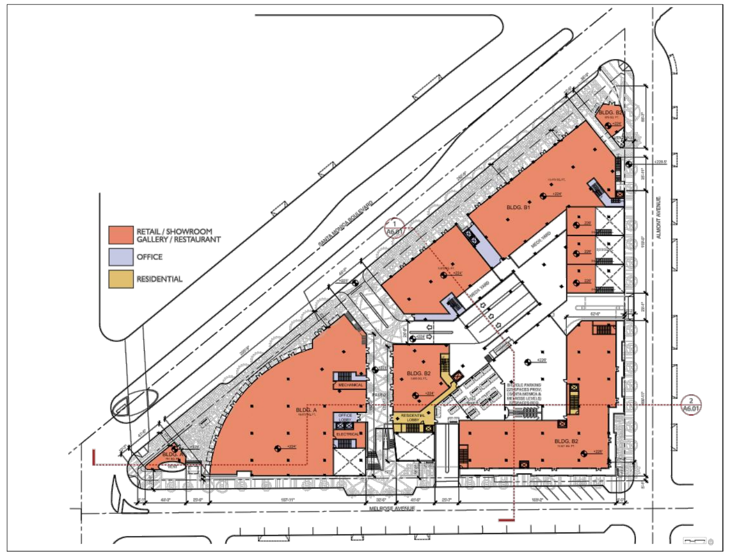
PROPOSED MELROSE TRIANGLE
MIXED-USE PROJECT
WEST HOLLYWOOD, CALIFORNIA

FIGURE NO.
1

PROJECT NO.
4953-10-1031

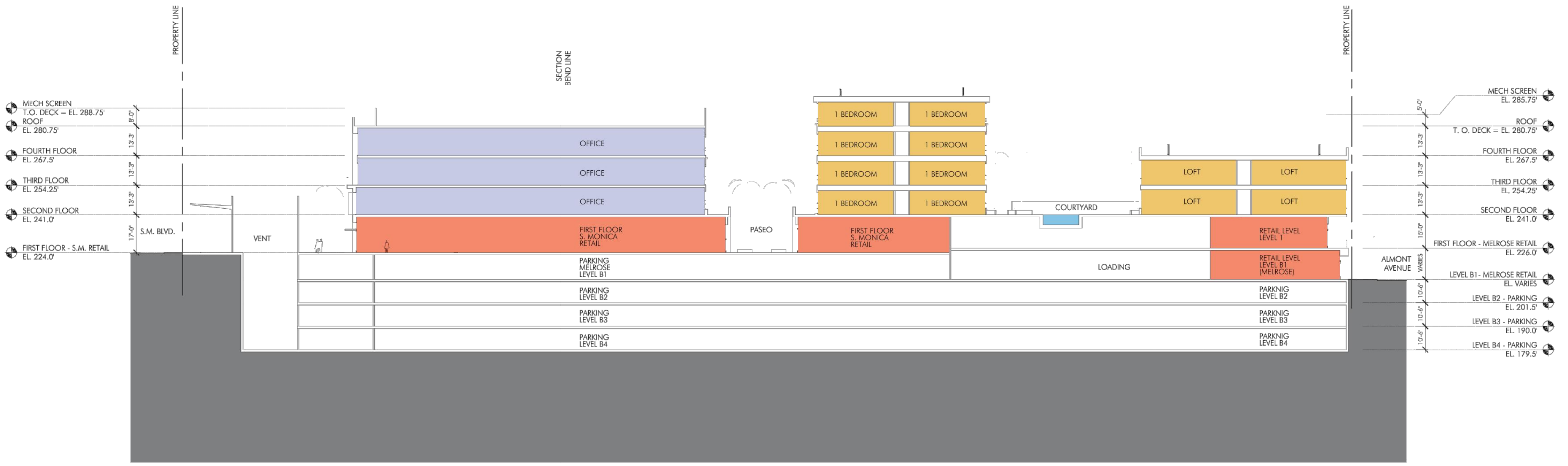
Path: P:\4953 Geotech\2010-proj\101031 Melrose Triangle\Figures\CAD\DWG\Figure-1_Plot-Plan.dwg [B-17x11]
Date: April 09, 2012 - 4:19pm By: vo.nguyen





KEY MAP
N.T.S

BUILDING SECTION 1 (NORTH/SOUTH) 1



REFERENCE:

OVERALL SITE PLAN, MELROSE TRIANGLE,
SHEET NUMBER A4.05 & A6.01 (DATED 01.10.2012)
BY STUDIO ONE ELEVEN.

BUILDING SECTION 2 (EAST/WEST) 2

0 30' 60' 120'

SCALE: 1" = 60'

amec

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BUILDING SECTIONS

JOB:	4953-10-1031
SCALE:	1" = 60'
DRAWN:	V. Nguyen
CHKD:	L. Tran
PM:	WC
DATE:	4/9/2012

PROPOSED MELROSE TRIANGLE
MIXED-USE PROJECT
WEST HOLLYWOOD, CALIFRONIA

FIGURE NO.
2

PROJECT NO.
4953-10-1031

Path: P:\4953 Geotech\2010-proj\101031 Melrose Triangle\Figures\CAD\DWG\Figure-2_Building-Section.dwg [B-17x11]
Date: April 09, 2012 - 4:23pm By: vo.nguyen

REPORT OF GEOTECHNICAL CONSULTATION PROPOSED MELROSE TRIANGLE MIXED-USE PROJECT

BETWEEN SANTA MONICA BOULEVARD,
MELROSE AVENUE AND ALMONT DRIVE
WEST HOLLYWOOD, CALIFORNIA

Prepared for:

SYSTEM, LLC

Los Angeles, California

August 27, 2010

MACTEC Project 4953-10-1031

MACTEC

MACTEC
engineering and constructing a better tomorrow

August 27, 2010

Mr. Jack Kurchian
President
System, LLC
9034 West Sunset Boulevard
West Hollywood, California 90069

Subject: **LETTER OF TRANSMITTAL**
Report of Geotechnical Consultation
Proposed Melrose Triangle Mixed-Use Project
Between Santa Monica Boulevard, Melrose Avenue and Almont Drive
West Hollywood, California
MACTEC Project 4953-10-1031

Dear Mr. Kurchian:

We are pleased to submit the results of our geotechnical consultation for the proposed Melrose Triangle Mixed-Use Project to be constructed between Santa Monica Boulevard, Melrose Avenue and Almont Drive in West Hollywood, California. This consultation was conducted in general accordance with the authorized Change Order dated July 26, 2010. We previously submitted the results of our final and original investigations for the project in reports dated July 28, 2008 (Our Project No. 4953-08-0811) and November 28, 2006 (Our Project No. 4953-06-2101), respectively. This report supersedes our July 28, 2008 and November 28, 2006 reports.

The scope of our services was planned based on discussions with you and your design team. Mr. Allen Pullman of Studio 111 Architects provided the floor plans and architectural drawings for the project. Mr. Mehran Pourzanjani of Saiful/Bouquet Structural Engineers provided us with structural details for the project.

The results of our consultation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

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

Sincerely,

MACTEC Engineering and Consulting, Inc.

Lan-Anh Tran
Project Engineer

Rosalind Munro
Principal Engineering Geologist

ROSALIND MUNRO
NO. 1269
CERTIFIED
ENGINEERING
GEOLOGIST

Marshall Lew, Ph.D.
Senior Principal Engineer
Vice President

MARSHALL LEW
No. 522
EXP. 3-31-11

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(3 copies submitted)

cc: (3) Studio 111 Architects
Attn: Mr. Allen Pullman

(1) Saiful/Bouquet Structural Engineers
Attn: Mr. Mehran Pourzanjani

REPORT OF GEOTECHNICAL CONSULTATION
PROPOSED MELROSE TRIANGLE MIXED-USE PROJECT

BETWEEN SANTA MONICA BOULEVARD,
MELROSE AVENUE AND ALMONT DRIVE
WEST HOLLYWOOD, CALIFORNIA

Prepared for:

SYSTEM, LLC

Los Angeles, California

MACTEC Engineering and Consulting

Los Angeles, California

August 27, 2010

Project 4953-10-1031

TABLE OF CONTENTS

EXECUTIVE SUMMARY

We have completed our geotechnical consultation of the proposed Melrose Triangle Mixed-Use Project in West Hollywood, California. The previous subsurface explorations, engineering analyses, and foundation design recommendations are summarized below.

Prior geotechnical investigations were performed by MACTEC and by our predecessor firm of LeRoy Crandall and Associates at the site of the proposed Melrose Triangle Mixed-Use Project. Several borings from the previous investigations are considered applicable for this current project. No new borings were drilled for this consultation.

The soil conditions at the site were previously explored by drilling a total of six borings to depths ranging from 74 to 125 feet below the ground surface (bgs) and advancing five Cone Penetration Test (CPT) soundings to depths ranging from about 50 to 120 feet bgs. Subsurface information was also available from two borings from a prior geotechnical investigation performed by our predecessor firm of LeRoy Crandall and Associates (LCA) at the site (LCA Project No. A-85280). The geotechnical recommendations in this report were developed in part using information from the previous investigations. We accept responsibility for the use and interpretation of the data presented in the previous report, and we concur with the interpretation of data presented in that report.

Fill soils, up to seven feet thick were encountered in our prior borings. The fill soils consisted of silty sand and sandy clays. Deeper fill soils may be encountered between the borings or elsewhere within the site. Any fill should be considered uncertified since records of its placement are not available; the quality of the fill would likely be variable. Nevertheless, based on the depths of excavation required for this project, all existing fill should automatically be removed beneath the building.

The natural soils primarily consisted of young alluvium deposits. The upper 60 feet of soils consist primarily of loose to very dense well-graded sand, silty sand and clayey sand and with layers of stiff to very stiff sandy lean clay, lean clay, and silty clays. The soils below a depth of 60 feet consist primarily of dense to very dense clayey sand, silty sand and well-graded sand and very stiff to hard sandy lean clay and silty clay.

Ground water was encountered between depths of 21 to 33 feet below the ground surface in our prior borings, corresponding to Elevations 192 to 196. The pore pressure in the CPT indicates ground water was at a depth of about 23 feet below the ground surface at one of the CPT locations, corresponding to Elevation 197. Our previous investigations in the area indicate ground water as shallow as 12 feet below ground surface. The California Geological Survey indicates the historic high ground-water level at the site is about 10 feet below the existing ground surface. Based on the data we have reviewed, we recommend that the ground water could be taken as Elevation 205 MSL for design purposes.

The corrosion testing indicates that the on-site soils are corrosive to ferrous metals, not aggressive to copper and would have moderate sulfate attack potential on concrete.

The basement for the proposed development will extend below the historic high ground-water level. Therefore, the basement would need to either be designed to withstand water and uplift pressure or be fully drained to prevent the buildup of hydrostatic pressure. We understand that it would be preferred to design the structure for hydrostatic pressure since drainage of the basement would require collection, pumping, disposal, and perhaps treatment of ground water. Since the structure will be designed to withstand water pressure, a mat foundation will likely be more economical than spread footings (because the slab between footings would need to be designed to withstand the uplift pressure, and because waterproofing would be more difficult with footings and a thickened slab system).

Some of the on-site clayey soils are expansive and are not suitable for use as compacted fill below slabs, walks, or behind retaining walls.

The ground-water level will be at or above the bottom level of the planned excavation. Provisions for ground-water control and/or dewatering will be necessary to allow for the proposed construction.

1.0 SCOPE

This report provides the results of our geotechnical consultation for the proposed Melrose Triangle Mixed-Use Project in West Hollywood, California. The location of the site is presented on Figure 1, Vicinity Map. The location of the proposed building and the prior exploration boring locations are presented on Figure 2.1, Plot Plan. Sections showing the height of the proposed mixed-use building and extent of basement excavations are presented on Figure 2.2. Building Sections.

This consultation was authorized to determine the static physical characteristics of the soils at the site of the proposed structure, and to provide recommendations for foundations and walls below grade, for floor slab support, for temporary shoring, and for earthwork for the development. We were to evaluate the soil and ground-water conditions at the site, including the corrosion potential of the soils, and provide the following:

- Recommendations for design of a feasible foundation system along with the necessary design parameters;
- Recommendations for design of walls below grade;
- Subgrade preparation and floor slab support; and
- Recommendations for excavations and temporary shoring;
- Grading, including site preparation, shoring, excavation and slopes, the placing of compacted fill, and quality control measures relating to earthwork.

Our recommendations are based on the results of the pertinent prior explorations, laboratory tests, and engineering analyses by us and other consultants. We have relied on subsurface data obtained from the following prior geotechnical investigation reports at the site by us and our predecessor firm of LeRoy Crandall and Associates (LCA):

- Hydrogeological Evaluation Report for the Proposed Melrose Triangle Development; Corners of Santa Monica Boulevard, Melrose Avenue and Almont Drive, West Hollywood, California, report dated January 26, 2009; (our Project No. 4088-08-7537.05).
- Report of Final Geotechnical Investigation, Proposed Melrose Triangle Mixed-Use Project, between Santa Monica Boulevard, Melrose Avenue

and Almont Drive, West Hollywood, California, Report dated July 17, 2008 (our Job No. 4953-08-0811).

- Report of Preliminary Geotechnical Investigation, Proposed Melrose Triangle Mixed-Use Project, between Santa Monica Boulevard, Melrose Avenue and Almont Drive, West Hollywood, California, Report dated November 28, 2006 (our Job No. 4953-06-2101).

- Preliminary Geotechnical Investigation, Proposed Development, Site bounded by Santa Monica Blvd, Almont Street and Melrose Avenue, West Hollywood, California, report dated September 6, 1985 (our Job No. LCA A-85280)

We have reviewed the prior report above and accept responsibility for the use and interpretation of the data presented therein. The results MACTEC field explorations and laboratory tests used in this consultation are presented in Appendix A; the results of the previous LeRoy Crandall and Associates field explorations and laboratory tests are presented in Appendix B.

The scope of this consultation includes a limited geologic-seismic study for the site. Our conclusions and recommendations are for static loading conditions only; however, this does not imply that there is a geologic or seismic hazard affecting the site. Also, the assessment of general site environmental conditions for the presence of contaminants in the soils and ground water of the site is not presented in this report.

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, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for System, LLC, and their design consultants to be used solely in the design of the proposed structure. The 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 PROJECT DESCRIPTION

System, LLC plans to construct a mixed-use development at the location shown on Figure 2.1. Based on the information provided to us, the proposed project is planned to consist of several buildings constructed over a single subterranean structure. The above-grade buildings may be three to five levels in height of retail, commercial and residential space. The buildings may be underlain by three to four levels of subterranean construction. The construction will extend to the property lines. The finished floor for the lowest of the three to four levels of basement may extend between an estimated 35 and 47 feet below the existing ground surface as illustrated on Figure 2.2, Building Sections and on Cross-Sections 1-1’ and 2-2’ presented as Figures 3.1 and 3.2. With foundations, the required excavation may be up to about 40 to 50 feet below the existing ground surface.

Mr. Mehran Pourzanjani of Saiful Bouquet Structural Engineers provided us preliminary structural information. Based on the preliminary information, the maximum column load could be on the order of 1,300 to 1,500 kips.

3.0 SITE CONDITIONS

The proposed Melrose Triangle is bounded by Santa Monica Boulevard to the northwest, Melrose Avenue to the south and Almont Street to the east. The project site is approximately 112,000 square feet in plan. The site slopes from the northwest down to the southeast. There is approximately 12 feet of difference in the ground surface from Santa Monica Boulevard to the intersection of Melrose Avenue and Almont Street. The site is currently occupied by one- to three-story parking/retail/office buildings with no basement levels. All of the on-site structures will be demolished to accommodate the new development. Various underground utilities cross the project site and many utilities would be anticipated in the streets adjacent to the site.

4.0 EXPLORATIONS AND LABORATORY TESTS

We previously performed geotechnical investigations at the site for a previous concept that was not constructed. The findings and exploration and laboratory tests were presented in reports:

- Hydrogeological Evaluation Report for the Proposed Melrose Triangle Development; Corners of Santa Monica Boulevard, Melrose Avenue and Almont Drive, West Hollywood, California, report dated January 26, 2009; (our Project No. 4088-08-7537.05).

- Report of Final Geotechnical Investigation, Proposed Melrose Triangle Mixed-Use Project, between Santa Monica Boulevard, Melrose Avenue and Almont Drive, West Hollywood, California, Report dated July 17, 2008 (our Job No. 4953-08-0811).

- Report of Preliminary Geotechnical Investigation, Proposed Melrose Triangle Mixed-Use Project, between Santa Monica Boulevard, Melrose Avenue and Almont Drive, West Hollywood, California, Report dated November 28, 2006 (our Job No. 4953-06-2101).

- Preliminary Geotechnical Investigation, Proposed Development, Site bounded by Santa Monica Blvd, Almont Street and Melrose Avenue, West Hollywood, California, report dated September 6, 1985 (our Job No. LCA A-85280)

Our prior explorations include the drilling of six borings to depths of ranging from 73 to 125 feet below the existing grade using rotary wash drilling equipment. Standard Penetration Tests (SPT) were performed in the borings using a CME auto-trip hammer to obtain the “N-value” blowcounts as indicated on the boring logs. In addition, five Cone Penetration Test (CPT) soundings were advanced to a depth ranging from about 50 to 120 feet below the ground surface. Details of the prior explorations and logs of the borings are presented in Appendix A. Data were also available from an investigation at the site performed by our predecessor firm of LeRoy Crandall and Associates (LCA). Details of the LCA explorations and logs of the borings are presented in Appendix B. The locations of the prior applicable borings are shown on Figure 2.1.

Laboratory tests were performed on selected samples obtained from the prior 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.
- Corrosivity.
- Mechanical Sieve Analysis.
- Atterberg Limits.

Details of our prior laboratory test data are presented in Appendix A.

Laboratory test results were also available from the LCA borings. All prior testing was done in general accordance with applicable ASTM specifications at the time of testing. Details of the relevant prior laboratory test data are presented in Appendix B.

5.0 SOIL AND GROUND-WATER CONDITIONS

Fill soils up to seven feet thick were encountered in the prior borings. The fill soils consisted of silty sand. Less than 1 foot of fill soils were reported in the prior borings by others on the site. Deeper fill soils may be encountered between the borings or elsewhere within the site, particularly backfill adjacent to any existing basement walls. The fill would be considered uncertified since records of its placement are not available; the quality of the fill would likely be variable. Nevertheless, based on the depths of excavation required for this project, all existing fill will automatically be removed beneath the building.

The natural soils primarily consisted of young alluvium deposits. The upper 60 feet of soils consist primarily of loose to very dense well-graded sand, silty sand and clayey sand and with layers of stiff to very stiff sandy lean clay, lean clay, and silty clays. The soils below a depth of 60 feet consist primarily of dense to very dense clayey sand, silty sand, and well-graded sand and very stiff to hard sandy lean clay and silty clay. The clay soils have a low to medium expansion potential based on Atterberg Limit testing.

Ground water was encountered at depths ranging from 21 to 33 feet below the existing ground surface in the borings. The pore pressure in the CPT indicates ground water is at a depth of about 23 feet below the ground surface. The California Geological Survey indicates that the historic high ground-water level at the site is about 10 feet below the existing ground surface. Based on the data we have reviewed and the current trend limiting pumping in the area, we consider it possible that the ground water could rise in the future, and recommend design for a water table at Elevation 205 MSL.

The corrosion studies indicate that the on-site soils are moderately corrosive to corrosive to ferrous metals, not aggressive to copper and would have moderate sulfate attack on concrete.

6.0 LIMITED GEOLOGICAL SEISMIC EVALUATION

The site is located in the northern Los Angeles Basin, near the boundary of the Transverse Ranges and the Peninsular Ranges geomorphic provinces. The Santa Monica-Hollywood fault zone, located approximately 0.5 mile north-northwest of the site, is the major structural feature in the vicinity. This east-west trending fault zone is generally considered to be the boundary between the Transverse Ranges geomorphic province to the north and the Peninsular Ranges geomorphic province to the south. The Santa Monica Mountains are located less than a mile to the north and display an east-west trend that is typical of the physiographic and structural features within the Transverse Ranges province.

Artificial fill, consisting of clay and silt, was encountered to a depth up to seven feet in our prior investigation. Underlying the fill is Holocene age alluvium or alluvial fan deposits consisting of loose to very dense sand, silty sand, clayey sand and sandy clay with clay and silty clay (see Figures 3.1 and 3.2).

Ground water was encountered in our recent prior exploration borings at depths of about 21 to 33 feet below the existing ground surface. The historic high ground-water level beneath the site is at a depth of approximately 10 feet (California Division of Mines and Geology, 1998).

The Hollywood fault zone, located approximately 0.5 mile to the north-northwest, is the closest active fault to the site. Active or potentially active faults have not been mapped across or projecting toward the site and the potential for surface rupture from fault plane displacement propagating to the surface at the site is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated if designed and constructed in conformance with current building codes and engineering practices.

The site is relatively level and the absence of nearby slopes precludes slope stability hazards. The site is in a 0.2% Annual Flood Hazard Zone as defined by the Federal Insurance Administration.

Regional subsidence associated with petroleum and ground water extraction has occurred beneath and near the site in the past. However, the rate of subsidence has been reduced in recent years by fluid injection by the oil companies. Also the effect of the regional subsidence is distributed over a large area and should not create problems of differential settlement at the site.

The site is near several oil fields. Therefore, there is a slight potential for methane and other volatile gases to occur beneath the site.

According to the County of Los Angeles Seismic Safety Element (1990), the site is 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) from Lower Franklin Canyon Reservoir and Greystone Dam. These dams, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design and construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum credible earthquake (MCE) for the site.

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.

According to the California Geological Survey (formerly California Division of Mines and Geology) and the County of Los Angeles Seismic Safety Element, the site is within an area identified as having a potential for liquefaction. As stated previously, the historic high ground-water level beneath the site is at a depth of approximately 10 feet (California Division of Mines and Geology, 1998). Ground water was at a depth of approximately 21 to 33 feet below existing grade in 2006. The pore pressure in one of the CPTs indicates ground water was at a depth of about 23 feet below the ground surface, at the CPT location, corresponding to Elevation 197.

For evaluation of the liquefaction potential, we computed the peak ground acceleration (PGA) for the ground motion at the site with a 2% probability of exceedance in 50 years using the program EZ-FRISK, Version 7.26. In our calculations, we corrected the PGA to be compatible with a Magnitude 7.5 earthquake. The resulting PGA of 0.59g corresponds to the PGA for the Maximum Considered Earthquake (MCE). The obtain the PGA for use in liquefaction analyses, the MCE

PGA was multiplied by 2/3 to obtain a PGA of 0.39g for the Design Earthquake in accordance with the 2007 California Building Code (CBC).

We have evaluated the liquefaction potential of the soils underlying the site using the magnitude 7.5 compatible DE PGA, and the results of CPT and the SPTs performed in the current borings. The liquefaction potential was computed as given in the Youd and Idriss, 1997 (NCEER Technical Report 97-0022) consensus publication on liquefaction evaluation and Youd et al., 2001 summary report from 1996 NCEER and 1998 NCEER/NSF workshop on evaluation of liquefaction resistance of soils.

The results indicate the liquefaction-induced settlement resulting from the Design Earthquake is negligible for the soils beneath the foundation level, although there is potential for limited localized liquefaction to occur in the upper soils beyond the building on the order of ½ inch in the upper silty sands.

7.0 RECOMMENDATIONS

7.1 GENERAL

Excavation of up to 40 to 50 feet below the ground surface is anticipated for the planned subterranean levels. Therefore, the basement of the proposed development will extend below the historic high ground-water level. Therefore, the basement would either need to be designed to withstand hydrostatic pressure or be fully drained to prevent the buildup of hydrostatic pressure. We understand that it would be preferred to design the structure for water pressure since drainage of the basement would require collection, pumping, disposal, and perhaps treatment of ground water. If the structure is designed to withstand water pressure, a mat foundation will likely be more economical than spread footings (as otherwise the slab between isolated footings would need to be designed to withstand the uplift pressure, and because waterproofing would be more difficult to install with a footing and thickened slab system).

The natural soils at and below the planned level of excavation are generally stiff or dense, and the proposed structure may be supported on a mat foundation established in the stiff and/or dense undisturbed natural soils. Floor slabs at the lowest floor level can be supported on a layer of compacted fill over the top of the mat foundation or the top of the mat foundation can serve as the floor slab.

The upper clay soils are somewhat expansive; therefore, at-grade slabs adjacent to the building should be supported on a minimum 2-foot-thick layer of non-expansive properly compacted fill.

The ground-water level will be above the bottom level of the planned excavation. Provisions for ground-water control and/or dewatering will be necessary to allow the proposed construction. A permit from the State of California Regional Water Quality Control Board should be obtained to discharge the ground water from the site into the public storm drain for the temporary dewatering prior to and during construction of the below grade portion of the structure.

7.2 FOUNDATIONS

In this section, data are given for the following foundation design considerations:

- Bearing value for structure.
- Estimated settlement of the structure.
- Modulus of subgrade reaction.
- Lateral resistance.
- Uplift pressure.
- Ultimate values.
- Foundation construction.
- Foundation observation.

Bearing Value

For support of the proposed structure, extending 35 to 47 feet below the existing grade, a mat foundation established in the undisturbed natural soils can be designed to impose an average net dead-plus-live load soil pressure of 3,000 pounds per square foot. The mat should be sufficiently reinforced and thickened to distribute the imposed loads relatively uniformly across the mat. Localized areas of the mat may be designed to impose a net dead-plus-live load pressure of up to 5,000 pounds per square foot.

The recommended bearing values are net values and the weight of concrete in the footings and mat may be taken as 50 pounds per cubic foot; the weight of soil backfill may be neglected when determining the downward loads from the structure. A one-third increase in the bearing values may be used when considering wind or seismic loads.

Settlement

The static settlement of the proposed building supported on a mat foundation, in the manner recommended, will depend on the foundation loads imposed, but is estimated to be on the order of 1½ to 2 inches. In any event, the settlement analysis should be reviewed when final foundation load information is available. The majority of the building settlement will occur during the building construction.

Modulus of Subgrade Reaction

A modulus of subgrade reaction, k, of 200 pounds per cubic inch may be assumed for the onsite soils. For structural analyses of a mat foundation established in the natural soil, an effective vertical modulus of subgrade reaction (k) of 50 pounds per cubic inch may be used for the soils underlying the mat foundation. This value has already been adjusted to account for the size of the mat foundation; no additional reduction is necessary.

Lateral Resistance

Lateral loads may be resisted by soil friction and passive resistance of the soils. A coefficient of friction of 0.3 may be used between the mat foundation and the supporting soils. If a waterproofing barrier is placed beneath the mat, the coefficient of friction should be reduced to 0.2, and the waterproofing material should be evaluated to confirm this coefficient is obtainable. The passive resistance of undisturbed natural soils or properly compacted fill may be assumed to be 250 pounds per cubic foot. A one-third increase in the passive value may be used when considering wind or seismic loads. The passive resistance and the frictional resistance of the soils may be combined without reduction in determining the total lateral resistance.

Uplift Pressure

The base of the mat should be designed to withstand hydrostatic pressure equal to 62.4 pounds per cubic foot multiplied by the depth from the historic high ground water level (Elevation 205) to the base of the mat.

Uplift Resisting Piles

If uplift forces caused by hydrostatic pressures are greater than the dead-load building forces imposed by the mat foundation, piles may be used to resist hydrostatic uplift.

Axial Pile Capacities

The upward capacities of 12- and 14-inch-square driven concrete piles for supporting the proposed development are presented on Figure 4, Upward Driven Pile Capacities. A one-third increase may be used when considering wind or seismic loads. The capacities are only for the purpose of determination of pile length based on the uplift forces caused by hydrostatic loading and based on the strength of the soils. However, the tensile strength of the pile section itself should be checked to verify the structural capacity of the piles. A one-third increase may be used for wind or seismic loads.

Where piles in groups are required, the piles should be spaced at least 3 pile widths on centers. If the piles are so spaced, no reduction in the capacity of the piles due to group action need be considered in design.

Pile Installation

The specification of pile driving criteria for termination of pile driving will depend on the pile hammer used and the characteristics of the pile selected for construction. Once the pile type and pile driving system are selected, wave equation analysis should be performed to evaluate drivability and to develop driving criteria. The final driving criteria should be developed using wave equation analysis incorporating the results of the indicator pile program recommended below.

We recommend that five to ten indicator piles be driven at the site to verify the required pile lengths and to evaluate the efficiency of driving systems before production piles are cast or ordered. The indicator piles should be ordered 10 feet longer than the design length to allow for instrumentation and possible variations in the subsurface materials. We will provide proposed locations of indicator piles after the pile foundation plan is finalized. Dynamic measurements during the indicator pile program using a Pile Driving Analyzer (PDA) is recommended on all indicator piles to develop blowcount and refusal criteria required to develop design capacities as well as to evaluate the induced stresses on the piles and the depth of pre-drilling, if required.

All piles should be installed to the predetermined lengths to develop the necessary uplift capacities. If pre-drilling is required to maintain induced stresses on piles below acceptable levels, the auger for pre-drilling should have a cross-sectional area no larger than 80% of the cross-sectional area of the pile.

Ultimate Values

The recommended bearing and lateral load design values for the proposed building 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
Passive Pressure	1.75
Coefficient of Friction	1.25
Upward Capacity of Piles	2.0

In no event, however, should foundation sizes be less than those required for dead-plus-live loads when using the working stress (allowable) design values.

Foundation Construction

The proposed foundation excavation will extend below the ground-water level. In order to allow for construction of the mat foundation on potentially saturated or moist soils, it may be necessary to place a 4-inch thick concrete slab (or “waste” slab) at the bottom of the excavation. This would be done to allow for placement of waterproofing and construction of reinforcement without disturbance of the upper exposed soils.

Foundation Observation

To verify the presence of satisfactory soils at the design elevations, the bottom of the mat excavation 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.

7.3 SITE COEFFICIENT AND SEISMIC ZONATION

We determined the seismic site coefficients in accordance with the 2007 California Building Code (CBC) and ASCE 7-05 Standard (ASCE, 2005) using the United States Geological Survey (USGS, 2007a) Earthquake Motion Parameters, Version 5.0.9, program. The site location used was Latitude 34.0812° (North) and Longitude 118.3883° (West) with a Site Class “D.” The seismic site coefficients under the CBC code are presented below:

Site Coefficient	Value
S_S (0.2 second period, Site Class B)	1.757g
S_1 (1.0 second period, Site Class B)	0.600g
Project Site Class	S_D
F_a	1.0
F_v	1.5
$S_{MS} = F_aS_S$ (0.2 second period)	1.757g
$S_{M1} = F_vS_1$ (1.0 second period)	0.900g
$S_{DS} = 2/3 \times S_{MS}$ (0.2 second period)	1.171g
$S_{D1} = 2/3 \times S_{M1}$ (1.0 second period)	0.600g

By: LT 8/16/2010 Checked: NH 8/27/2010

7.4 RETAINING WALLS AND WALLS BELOW GRADE

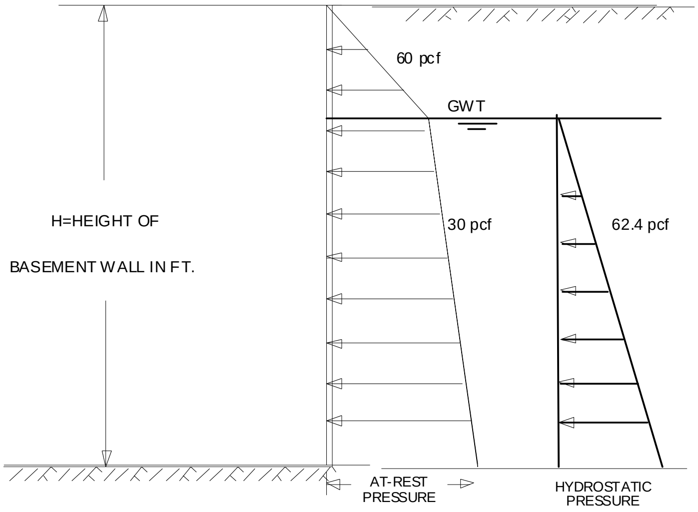
In this section, data are given for the following retaining wall considerations.

- Lateral earth pressure (for design of cantilever retaining walls and basement walls).
- Waterproofing.
- Drainage.

Lateral Earth Pressure

For design of cantilevered retaining walls, where the surface of the backfill is level, it can be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharges due to traffic, storage, and adjacent foundation loads.

For the design of the braced basement walls, lateral earth pressure plus any surcharge loadings occurring as a result of traffic, storage, and adjacent foundations should be used. The 2007 CBC requires that basement walls be designed for at-rest pressure. The design lateral load is shown on the figure below:

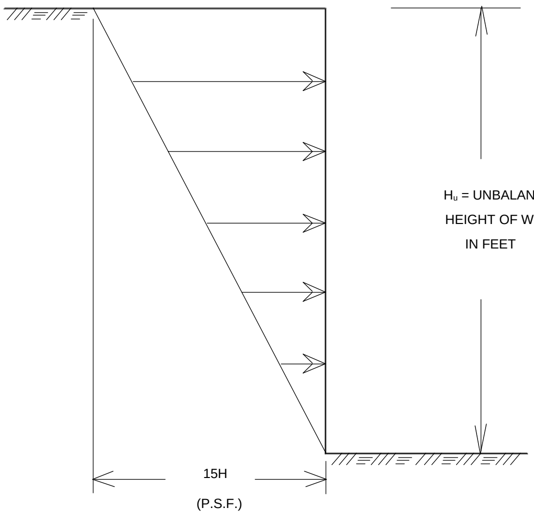


Below the ground-water table, the walls should also be designed for hydrostatic pressure.

In addition to the recommended earth pressure, the upper 10 feet of walls adjacent to streets and vehicular traffic areas should be designed to resist a uniform lateral pressure of 120 pounds per square foot, acting as a result of an assumed 350 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.

Seismic Lateral Earth Pressure

In addition to the above-mentioned lateral earth pressures, basement walls should be designed to support a seismic active pressure where there is a difference in grade from one side of the site to the other. The seismic active earth pressure should be applied to the portion of the basement walls that support the unbalanced earth. The recommended seismic active pressure distribution on the wall is illustrated in the following diagram with the maximum pressure equal to $15H_u$ pounds per square foot, where H_u is the unbalanced wall height in feet.



Because the liquefaction potential is limited to localized layers of the upper silty and clayey sands, increases in the lateral earth pressure on the basement walls are not expected to be significant. Similarly, downdrag on the basement walls are also not expected to be significant.

Waterproofing and Drainage

Walls below grade should be waterproofed to minimize the transmission of moisture through walls below grade. The design of the basement to resist water pressure will require a thorough waterproofing installation. Installation of a completely watertight waterproofing system will be difficult; therefore, we suggest consulting with a waterproofing consultant and/or contractor experienced in the installation of such a system. However nuisance water should still be assumed to possibly penetrate the waterproofing system, and a secondary system to collect water could be installed.

In addition, drainage should be provided so that the portion of the walls above Elevation 205 does not have to be designed for additional water pressure due to nuisance infiltration; as it is expected that the area around the building will be not landscaped, nuisance infiltration should be small. The means of accomplishing drainage will depend primarily on the selected method of shoring and the method of constructing the exterior building walls. Miradrain 6000 (or equivalent), attached to the lagging and protected from the concrete placement of the walls, would provide satisfactory drainage. If significant hydrocarbons are anticipated in the ground water, a hydrocarbon-resistant product such as Miradrain 8000 (or equivalent) could be used. Continuous Miradrain should be placed at a depth starting at about Elevation 208 and extend to at least Elevation 200 to allow for the collected nuisance water to be dissipated.

7.5 FLOOR SLAB SUPPORT

The undisturbed natural soil will provide adequate support for the mat foundation. At-grade concrete slabs and walls adjacent to the proposed building may also be supported on grade if the grading recommendations in the report are followed.

Based on our experience, it will likely be necessary to construct a waste slab, as discussed in Section 7.2, in order to reduce disturbance of the natural soils at the base of the excavation during construction.

For design of minor at-grade structures with floor slabs or concrete hardscape adjacent to the building, 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.

If vinyl or other moisture-sensitive floor covering is planned for slabs on grade, 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; this would not be necessary for mat foundation. 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 reduce possible curling of the slab. A 2-inch-thick layer of coarse sand can 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. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

7.6 DEWATERING/GROUND-WATER CONTROL

The mass excavation will extend below the ground-water level. Dewatering or ground-water control measures will be required. Dewatering could be accomplished by means of wells located around the perimeter of the site and supplementary wells located within the limits of the excavation. The wells could drain into sumps equipped with pumps.

Detailed dewatering and ground-water control recommendations are beyond the scope of this investigation. However, general considerations are discussed below.

The dewatering system should be designed by a competent and experienced dewatering contractor. The contractor should determine the size, spacing, and depths of the dewatering wells. In addition, the contractor should determine the locations and sizes of any necessary trenches within the excavation, and the volume of water inflow from the dewatering system.

A permit from the State of California Regional Water Quality Control Board would have to be obtained to discharge the water into a storm drain. To obtain such a permit, additional chemical tests may have to be performed on ground-water samples obtained at the site to verify that chemicals or pollutants within the water do not exceed the allowable limits for discharging into the storm drain. We anticipate that such testing could be performed by collecting water samples from the existing ground-water well or in new wells to be installed at the site.

As water is drawn from the soils during temporary dewatering of the site, the soils surrounding the site will experience additional loading which will cause some settlement of the soils beyond the footprint of the building. It is our opinion that the maximum settlement due to dewatering will be about one inch along the perimeter of the building. Settlement beyond the outline of the building is largely dependent on the geometry of the dewatered soil profile, however, it is our preliminary opinion that the maximum estimated differential settlements will be on the order of ¼-inch over 25 feet in areas directly adjacent to the site. A more detailed analysis may be provided as greater details in the dewatering geometry are analyzed.

7.7 EXCAVATION AND SLOPES

Excavations about 40 to 50 feet deep are estimated for the lower subterranean parking level of the proposed development. Where the necessary space is available, temporary unsurcharged embankments may be sloped back at 1:1 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 existing structures. Where space is not available, shoring will be required. Data for design of shoring are presented in the following section.

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. If 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 form entering the excavation and eroding the slope faces.

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, including OSHA requirements, should be met.

7.8 SHORING

General

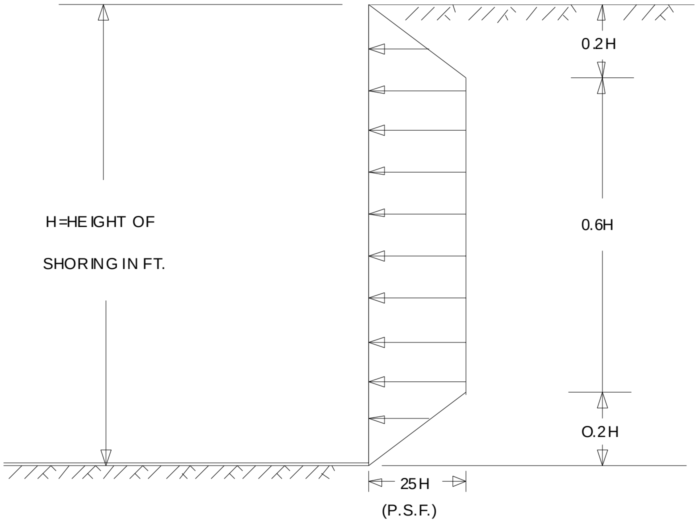
It is anticipated that temporary shoring will be required for the entire site. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied back with earth anchors. Some difficulty may be encountered in the drilling of the soldier piles and the anchors because of ground water and caving in the sandy deposits. Special techniques, such as the use of steel shell casing, drilling mud, and/or vibrating soldier piles below the excavation level, may be necessary to permit the installation of the soldier piles and/or tie-back anchors. In addition, 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 require internal bracing.

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

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 35 pounds per cubic foot.

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 25H in pounds per square foot, where H is the height of the shoring in feet. The distribution given is made assuming that the soils behind the shoring are dewatered. 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 and vehicular traffic areas should be designed to resist a uniform lateral pressure of 120 pounds per square foot, acting as a result of an assumed 350 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.

Lateral surcharge pressures imposed by cranes or concrete conveying trucks and other heavy construction equipment placed near the shoring system. We can provide estimates of these surcharge pressures will be when sufficient information is available, if authorized.

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 500 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 5,000 pounds per square foot. The passive value assumes that ground water will be close to the excavation bottom. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The soldier pile excavations may be filled with a lean slurry (1½ to 2 sack mix). However, the slurry 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 tributary area of the soldier pile may be computed using the length of the diagonal of the beam. In case the soldier piles are vibrated into position below the bottom of the drilled hole, the tributary area of the soldier piles should be limited to the width of the beam flange.

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 slurry and between the lean slurry 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 350 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 lower due to arching in the soils. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot. The pressure distribution for the lagging may be assumed to be semi-circular, where the pressure at the soldier pile is 0, and the pressure at the center is 400 pounds per square foot.

Anchor Design

Tieback 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 table below describes the minimum distance that anchors should extend beyond the potential active wedge. Anchors may require a greater length, depending on the depth of excavation to develop the desired capacities, therefore, some significant encroachment outside the property limits and into the public right-of-way should be anticipated. The city of West Hollywood may require that at least the upper row of anchors be detensioned after completion of the basement. The capacities of anchors should be determined by testing of the initial anchors as outlined in a following section. For design purposes, we estimate that drilled friction (also known as gravity-grouted) anchors will develop average friction values as presented in the table below. For post-grouted (also known as pressure-grouted) anchors, it may be estimated that the anchors will develop an average friction of three-times the friction values presented below.

Tieback Recommendations for 50-foot Excavation

Depth from ground to anchor at soldier pile (feet)	Minimum Length of Anchor beyond active wedge (ft)	Average Friction along anchor length (psf)
5-25	60	500
25-45	40	700
>45	25	900

The capacities of anchors should be determined by testing of the initial anchors as outlined in a following section. For post-grouted anchors, it may be estimated that the anchors will develop an average friction of triple the average frictions presented in the tables above. 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 needs to 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 at certain locations 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. For post-grouted anchors of 8-inch diameter or less, the anchor may be filled with concrete to the face of the shoring.

Anchor Testing

Our representative should select at least two of the initial anchors from each shored wall (for a total of six) for 24-hour 200% tests, and 5% of the remainder of the 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. If the friction assumed in the postgrouted portion, f_p, divided by the friction assumed in the non-postgrouted portion, f_n, is x:

$$f_p/f_n = x$$

then the test load can be taken as:

$$P_{test} = P_{design} * \frac{\frac{1}{x} L_u + L_a}{L_a} * M$$

where L_a=Postgrouted length of Anchor
L_u=Non-postgrouted length of Anchor
M=150% or 200%, depending on the test performed

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 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% 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 footing (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 2,500 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

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of the utilities in the adjacent streets. If it is desired to reduce the deflection of the shoring, a greater lateral earth 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 the design consultants and the contractor when the design of the shoring system has been finalized.

In addition, we recommend that the adjacent sidewalks, streets and nearby buildings be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in the nearby buildings would be prudent and recorded; photographic records should be made to document the pre-construction conditions of the nearby existing buildings.

7.9 GRADING

The existing fill soils may not have been uniformly well compacted, and we do not have record of them having been observed and tested during placement; therefore, they are not considered suitable for support of the at-grade floor slabs or hardscape adjacent to the building. The basement excavation should automatically remove the existing fill soils. However, for areas adjacent to the basement, the existing fill soils should be excavated and replaced as properly compacted fill; however, it may not be practical to excavate all of the existing fill soils. At least 2 feet of non-expansive fill should be placed below paving or slabs. All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils can be used in any required fill. This section gives recommendations for the following grading considerations:

- Site preparation.
- Compaction.
- Backfill.
- Material for fill.

Site Preparation

After the site is excavated, the exposed natural soils should be carefully observed for the removal of all unsuitable deposits. Proof-rolling or compaction of the exposed natural soils should not be performed. If unsuitable deposits are removed beneath an area of the mat, they should be replaced with a 2-sack cement slurry.

Adjacent to the proposed basement, where minor structures or hardscape is planned to be constructed, existing fill soils should be excavated and recompacted for proper support of the footings, slabs, or hardscape, if practical. Because of the expansive nature of the soils, at least the upper 2 feet of natural soil should be replaced as non-expansive properly compacted fill beneath hardscape or slabs (but not beneath footings). Where fill is 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 D1557-07 method of compaction.

Compaction

Any required fill should be placed in horizontal lifts not more than 8 inches thick and compacted to at least 90%. Relatively non-expansive soils shall be compacted at a moisture content varying no more than 2% below or above optimum moisture content. It is recommended that the moisture content of on-site clayey soils at the time of compaction be brought to between 2% and 4% over optimum moisture content.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-07 method of compaction. On-site non-expansive soils can be used in the compacted backfill. The on-site medium expansive clayey soils may be difficult to compact, and should not be used within the upper backfill or wall backfill. The on-site non-expansive soils can be used in the upper 2 feet of backfill, to provide a relatively impermeable layer when compacted to restrict the inflow of surface water into the backfill. The exterior grades should be sloped to drain away from the structure to prevent ponding of water.

Some settlement of backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

Material for Fill

The on-site soils, less debris or organic materials within any existing fill soils, may be used in the required fills. Because of their expansive characteristics, the on-site clay soils should not be used as backfill behind any walls below grade or within 2 feet of the at-grade concrete slabs and walks adjacent to the building. All required imported fill, at least the upper 2 feet of fill beneath adjacent concrete slabs and walks adjacent to the building, and wall backfill should consist of relatively non-expansive soils. Cobbles larger than 4 inches in diameter should not be used in the fill. The Expansion Index of the selected relatively non-expansive material should be less than 35. Any import material should contain sufficient fines (binder material) so as to provide a compacted fill that will be relatively impermeable and will be stable in shallow trenches. All proposed import materials should be approved by our personnel prior to being placed at the site.

7.10 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 exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade.
- 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 testing and installation of soldier piles to verify the desired diameter and depth are obtained.
- Observe the installation and testing of the temporary tie-back anchors.
- Observe the installation of and dynamic testing of driven piles to develop a pile driving criteria.
- Observe the installation of production driven piles to verify the desired capacities and lengths are achieved.

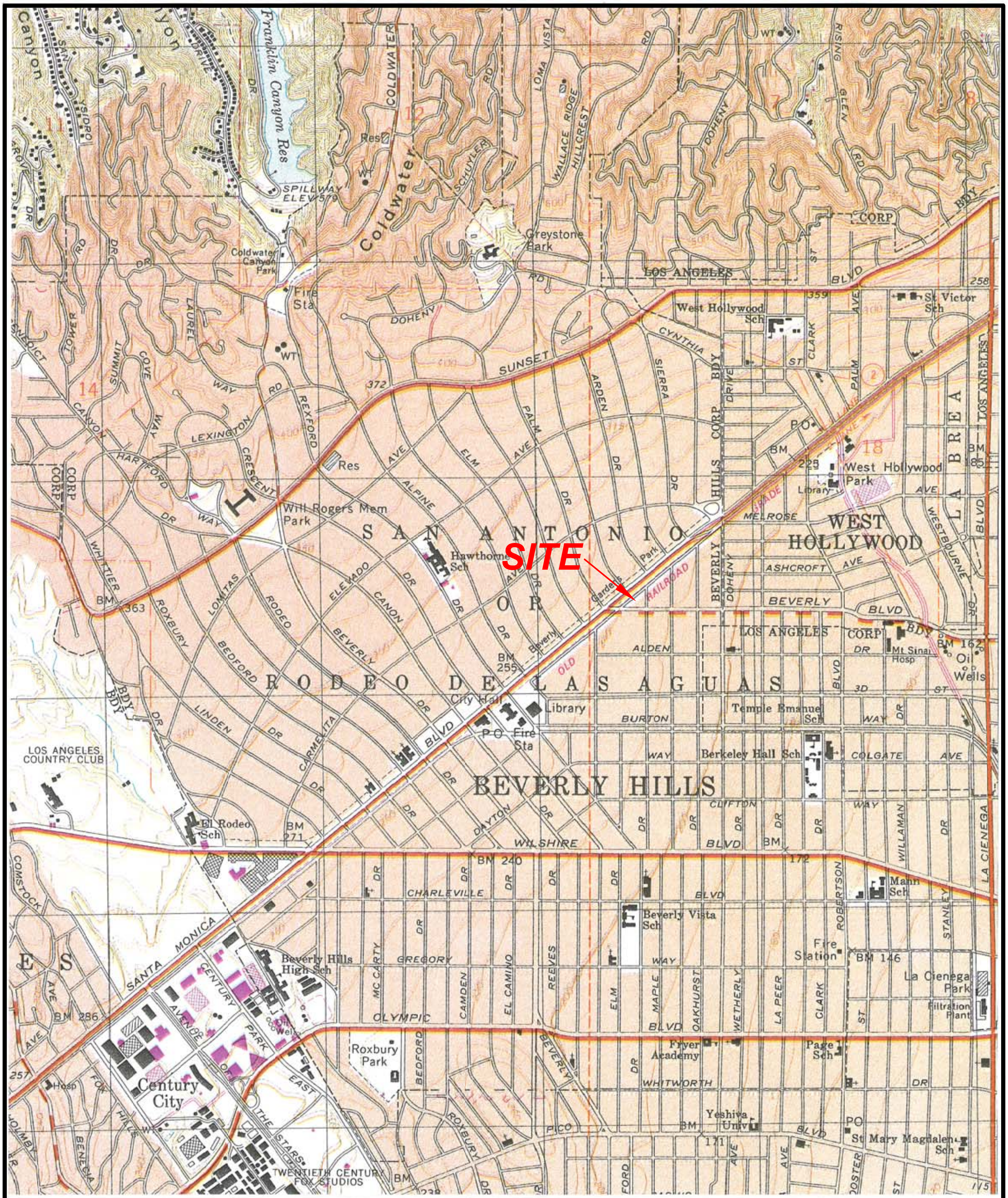
The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing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

8.0 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 current and previous 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. Several borings were deferred pending demolition of the existing buildings; a supplemental geotechnical report is to be submitted with the results of the deferred borings. Analyses will be performed to confirm the findings and recommendations of this report. It is possible that some recommendations could be modified based on the results of the supplemental explorations.

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.

FIGURES



REFERENCE:
U.S.G.S. 7.5-MINUTE
BEVERLY HILLS, CA
QUADRANGLE,
DATED 1995.

0 2000
Scale in Feet

N

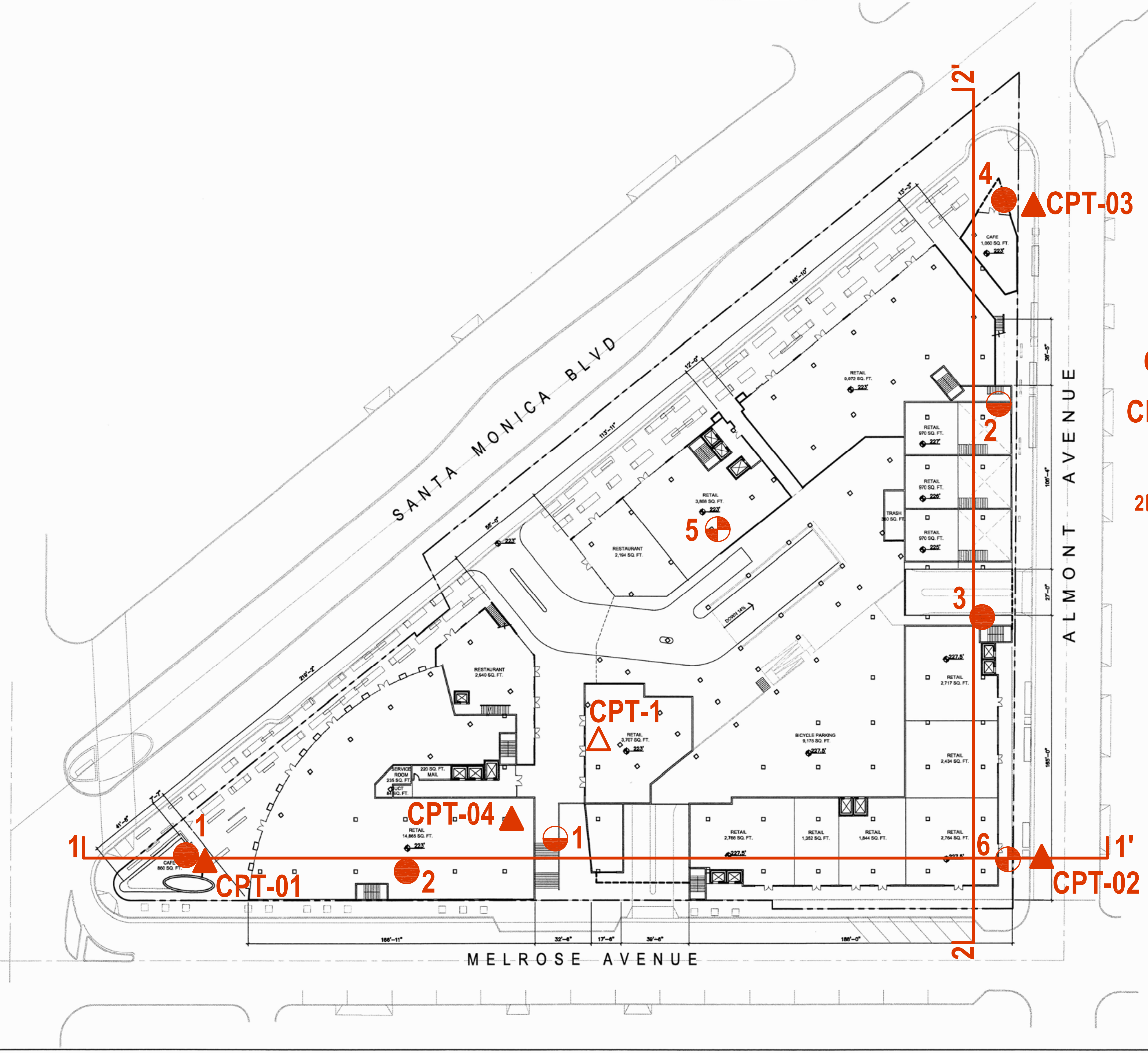
MACTEC
5628 E. SLAUSON AVENUE
LOS ANGELES, CALIFORNIA 90040
(323) 889-5300 FAX (323) 889-5398

VICINITY MAP

PROPOSED MELROSE TRIANGLE MIXED-USE PROJECT
WEST HOLLYWOOD, CALIFORNIA

FIGURE 1

PROJECT NO.	4953-10-1031	REVISION:
DATE:	97/30/2010	
SCALE:	AS NOTED	
DWG BY:	TT CHECKED BY:	LT 8/27/2010



REFERENCE:
OVERALL SITE PLAN,MELROSE TRIANGLE,
SHEET NUMBER A-0.0 (DATED 7-19-10)
BY CHARLES COMPANY.

LEGEND:

6 ⊕ PRIOR MACTEC INVESTIGATION (PROJECT NO. 4953-08-0811)

4 ● PRIOR MACTEC INVESTIGATION (PROJECT NO. 4953-06-2101)

2 ◒ PRIOR MACTEC INVESTIGATION (PROJECT NO. A-85280)

BORING LOCATION AND NUMBER

CPT-1 △ PRIOR MACTEC INVESTIGATION (PROJECT NO. 4953-08-0811)

CPT-04 ▲ PRIOR MACTEC INVESTIGATION (PROJECT NO. 4088-08-7537)

CONE PENETRATION TEST LOCATION AND NUMBER

2└──┘2' SECTION LINE

N

0 30 60 120
Scale in Feet

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FIGURE 2.1

PLOT PLAN

PROPOSED MELROSE TRIANGLE MIXED-USE PROJECT WEST HOLLYWOOD, CALIFORNIA

PROJECT NO.	4953-10-1031	REVISION:
DATE:	07/30/2010	
SCALE:	1" = 60'	
DWG BY:	TT	CHECKED BY: LT

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LOS ANGELES, CALIFORNIA 90040
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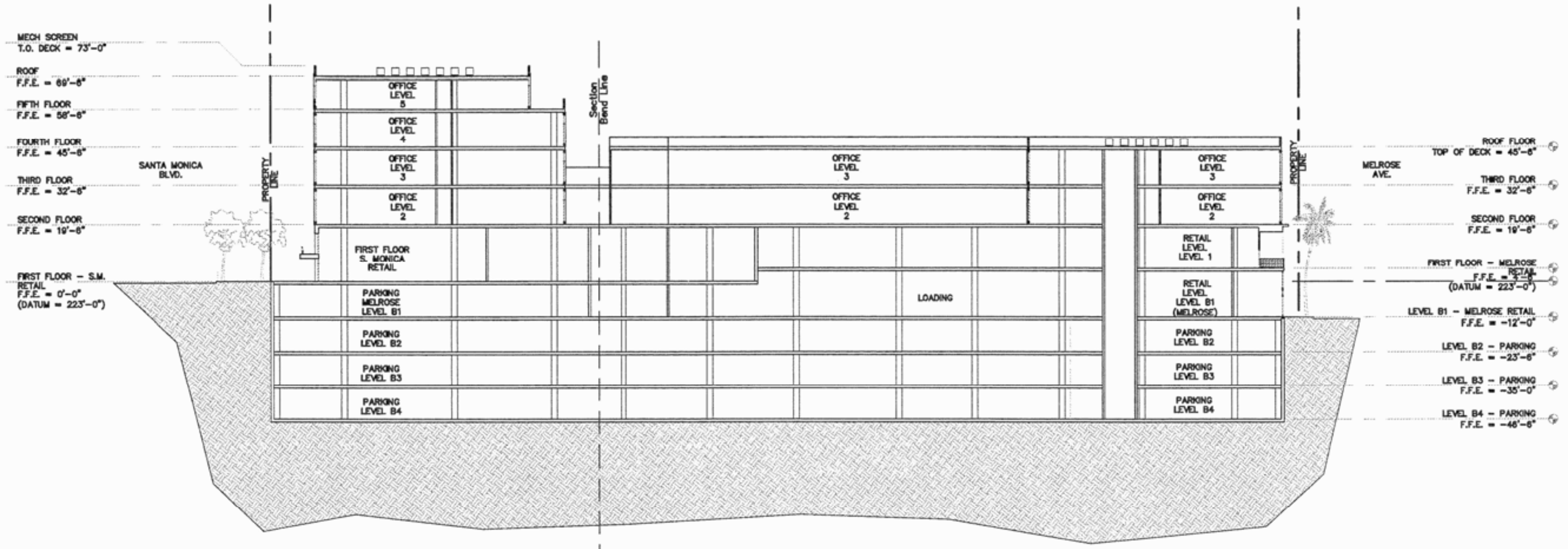
FIGURE 2.2

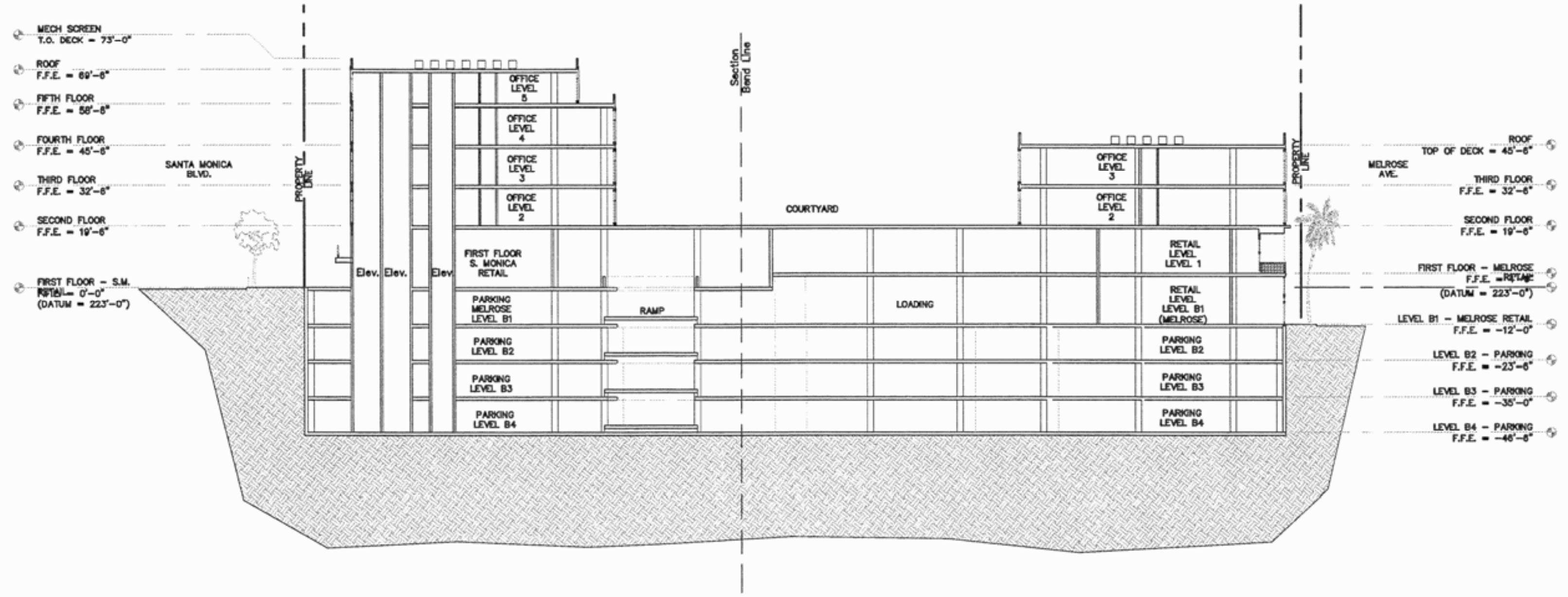
BUILDING SECTIONS

PROPOSED MELROSE TRIANGLE
MIXED-USE PROJECT
WEST HOLLYWOOD, CALIFORNIA

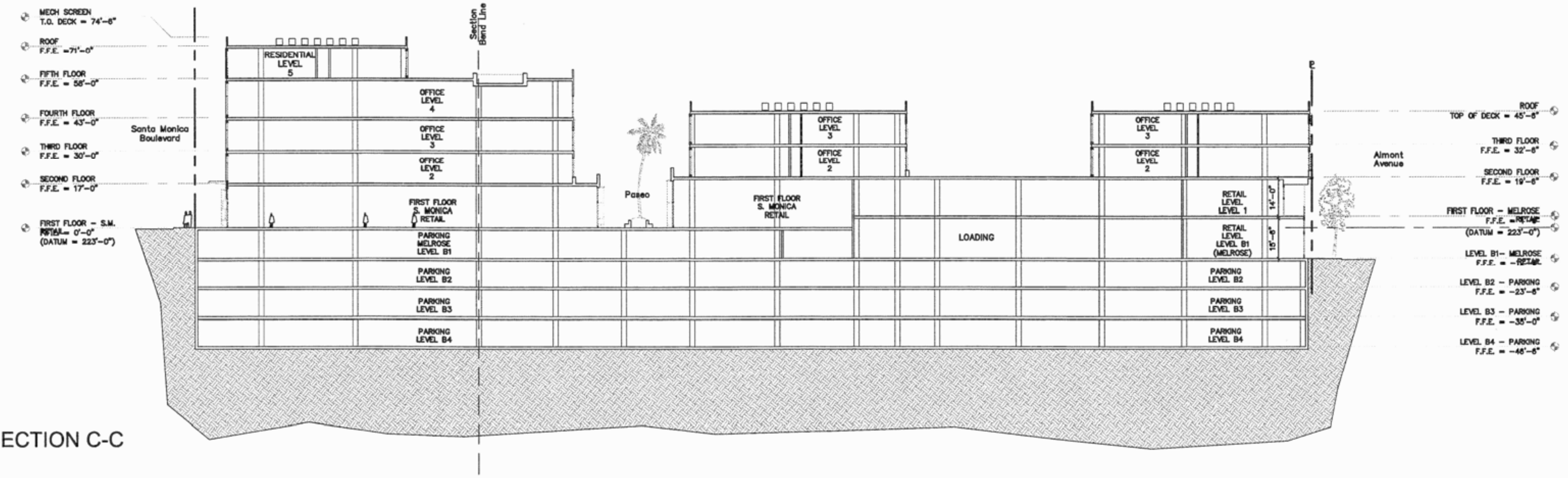
PROJECT NO.	4953-10-1031	REVISION:
DATE:	07/30/2010	
SCALE:	1" = 60'	
DWG BY:	TT	CHECKED BY: LT



SECTION A-A

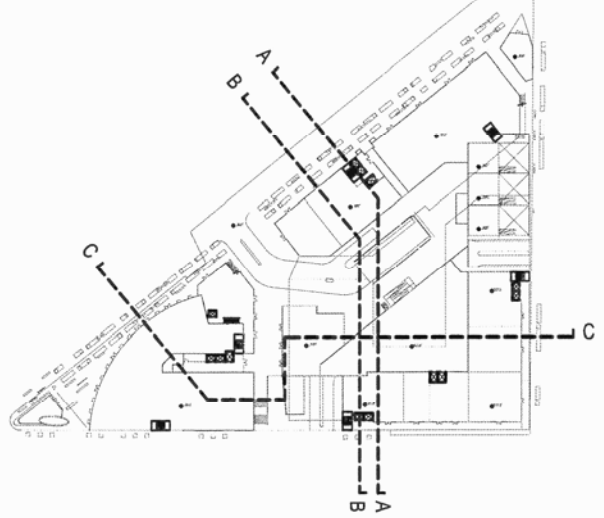


SECTION B-B

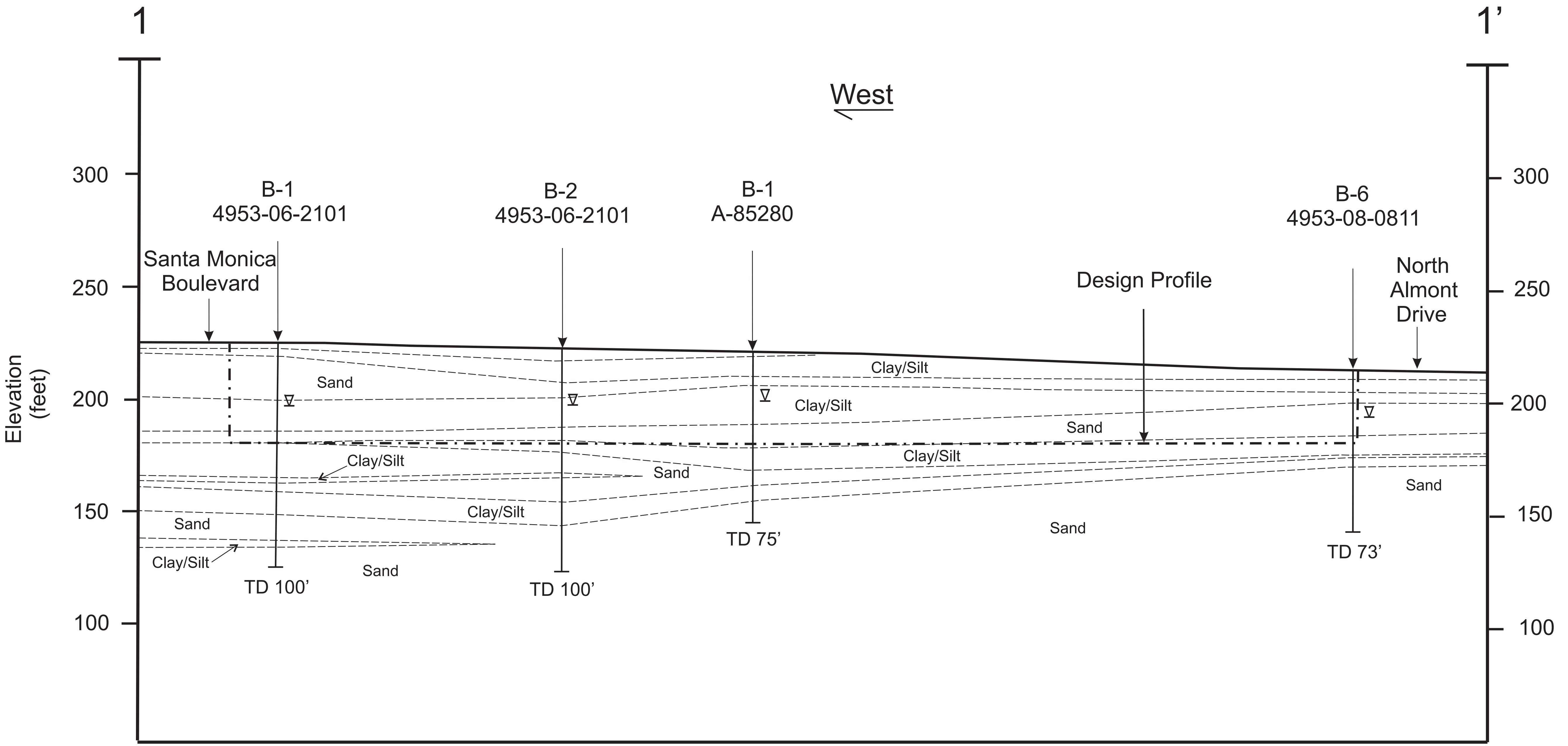


SECTION C-C

REFERENCE:
SECTION, MELROSE TRIANGLE,
SHEET NUMBER A-1.0 (DATED 7-19-10)
BY CHARLES COMPANY.



KEY PLAN



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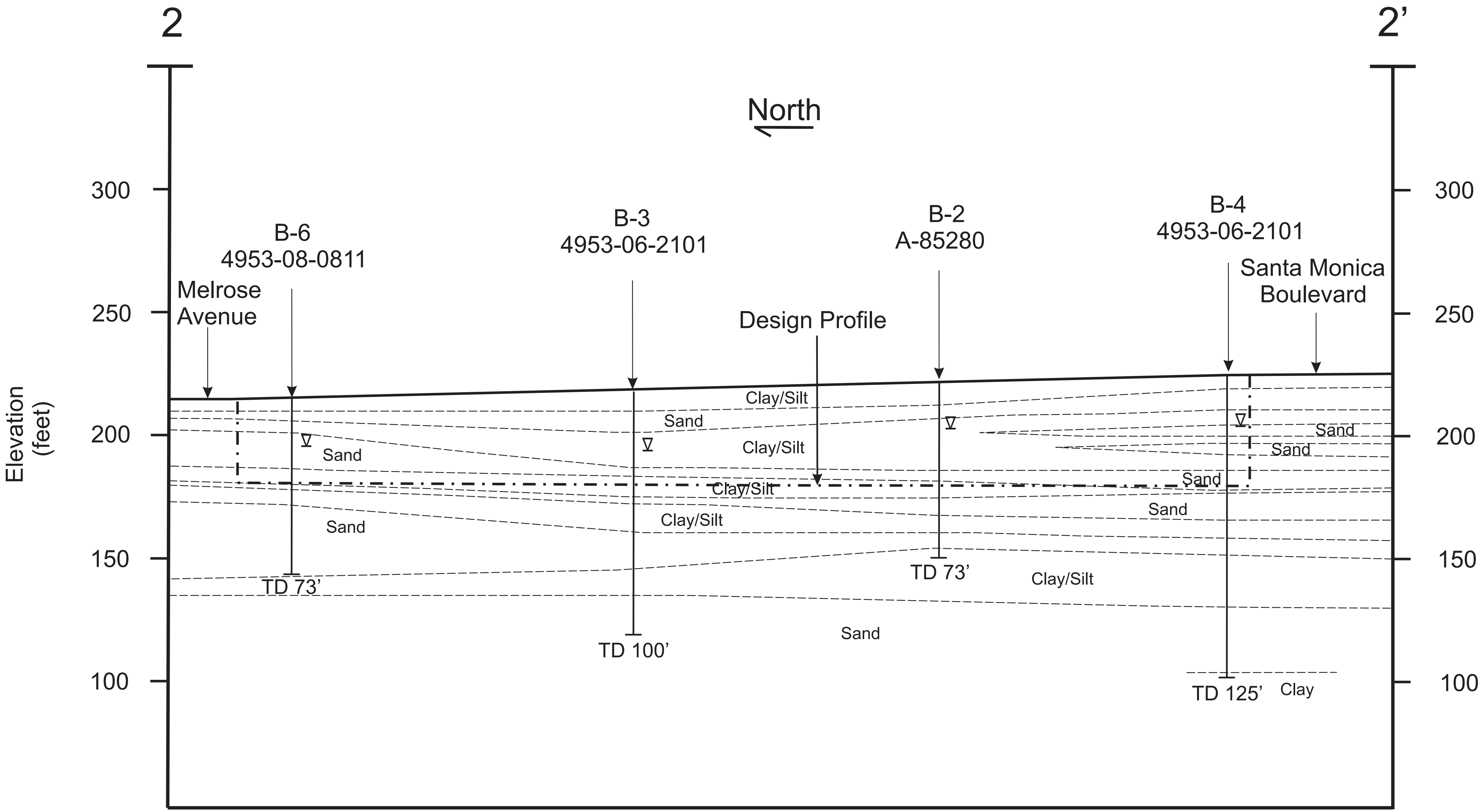
(323) 889-5300 ○ fax (323) 889-5398

Figure 3.1

Cross Section 1-1’

Melrose Triangle
West Hollywood, California

JOB NO.: 4953-10-1031	REVISIONS: RM 8/27/10
DATE: 7/8/08	
SCALE: 1” = 50’	
DRAWN BY: RM PREPARED BY: RM	
CHECKED BY: LT	



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(323) 889-5300 ○ fax (323) 889-5398

Figure 3.2

Cross Section 2-2’

Melrose Triangle
West Hollywood, California

JOB NO.: 4953-10-1031
DATE: 7/2/08
SCALE: 1” = 50’
DRAWN BY: RM PREPARED BY: RM
CHECKED BY: LT
REVISIONS: RM 8/27/10

APPENDIX A

PRIOR EXPLORATIONS AND LABORTORY TESTS BY MACTEC

APPENDIX A

PRIOR EXPLORATIONS AND LABORTORY TESTS BY MACTEC

EXPLORATIONS

The soil conditions beneath the site were explored by drilling a total of six borings to depths ranging from 74 to 125 feet below the existing grade using rotary-wash type equipment. The borings were backfilled with bentonite/cement slurry and the cuttings were stored on-site until pertinent laboratory testing was concluded and the cuttings were disposed of accordingly. In addition, five Cone Penetration Test (CPT) sounding was advanced to depths ranging from about 50 to 120 feet bgs. Subsurface information was also available from two borings from a prior investigation by our predecessor firm, LeRoy Crandall and Associates at the site. The CPT results are presented at the end of this Appendix. Pertinent prior subsurface explorations and relevant laboratory data by LeRoy Crandall and Associates are presented in Appendix B.

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 current borings are presented in Figures A-1.1a through A-1.6. The depths at which the 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 300 pound hammer falling 24 inches is indicated on the logs. The soils are classified in accordance with the Unified Soil Classification System described in Figure A-2.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine 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.

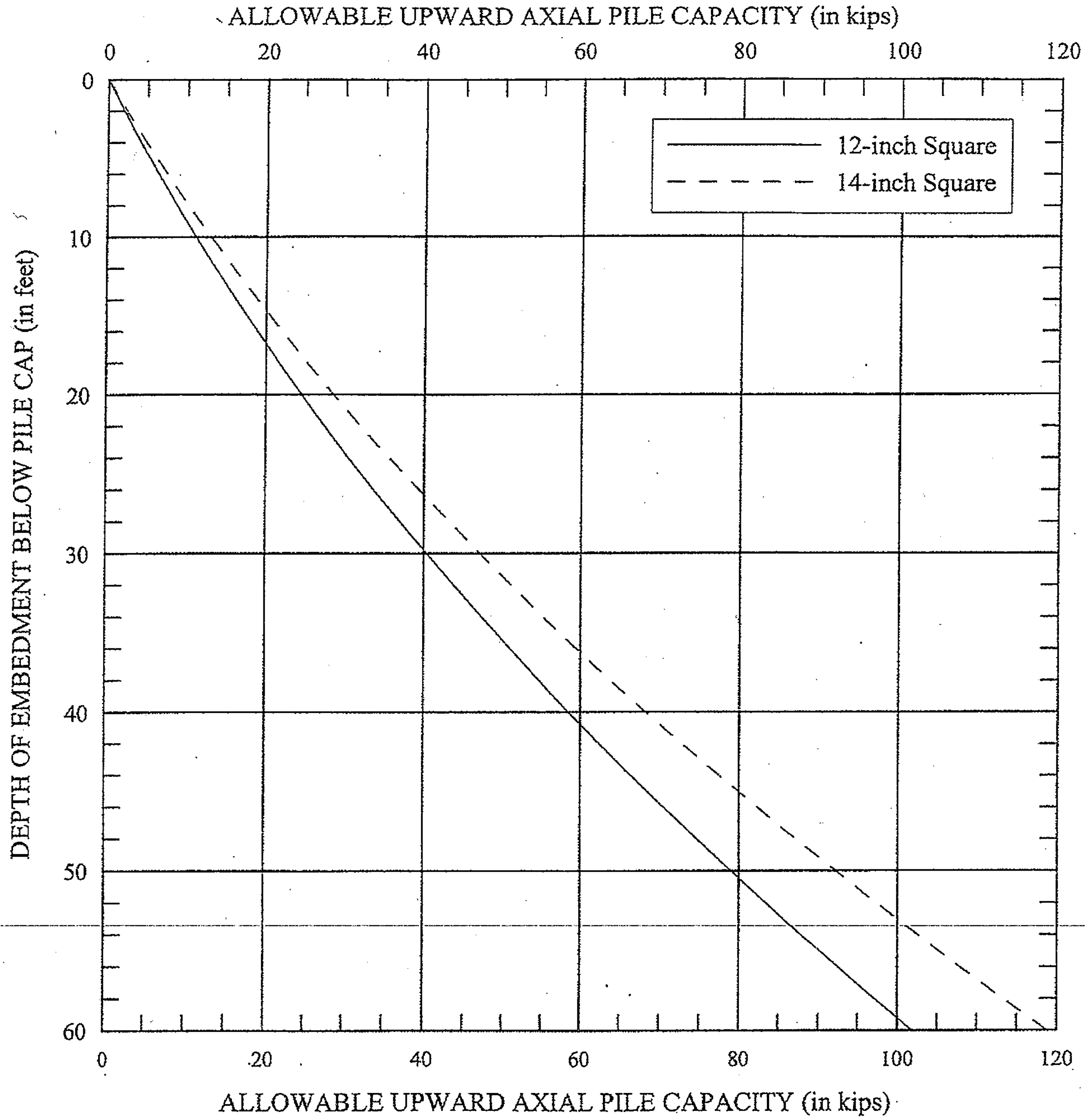
Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented in Figure A-3, Direct Shear Test Data.

Confined consolidation tests were performed on four undisturbed samples to determine the compressibility of the soils. The results of the tests are presented in Figures A-4.1 and A-4.3, Consolidation Test Data.

To determine the particle size distribution of the soils and to aid in classifying the soils, mechanical analyses were performed on three samples. The results of the mechanical analyses are presented on Figures A-5.1 and 5.2, Particle Size Distribution.

In addition to the full mechanical analyses, tests to determine the percentage of fines (material passing through a -200 sieve) in selected samples were performed. The results of these tests are presented on the boring logs.

Soil Corrosivity tests were performed on samples of the on-site soils. The results of the tests are presented in Figure A-6.



NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 3 pile diameters on centers.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

Prepared/Date: JR 10/24/06
Checked/Date: ML 10/24/06

Melrose Triangle
West Hollywood, California

MACTEC

ALLOWABLE UPWARD
AXIAL PILE CAPACITY

Project No. 4953-08-0811

Figure 4

BORING 1

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 222.5**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								3" Thick Asphalt Concrete
							SM	SILTY SAND - slightly moist, brown
220							CL	SANDY LEAN CLAY - slightly moist, brown
			14.3	119	8	▩		
	5							
							SC	CLAYEY SAND - slightly moist, brown
215			12.8	110	8	▩		
	10							
			11.7	116	9	▩		
210		6				☒		31.9% Passing No. 200 sieve
	15							Becomes more gravelly, few cobbles
			13.3	118	9	▩		
205								
		17				☒		
	20							
							SM	SILTY SAND - moist, brown
200			11.7	116	27	▩		
								Becomes sandier, few gravel
							CL	SANDY LEAN CLAY - moist, brown, few gravel
	25	25				☒		
195			19.0	110	15	▩		
	30							
		16				☒		LL=28, PI=13
190								
			20.5	110	15	▩		
	35							
								Becomes less gravelly
185		12				☒		
	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: CMC
Prepared By: AO
Checked By: LT

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING

Project: 4953-06-2101
Figure: A-1.1a

BORING 1 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 222.5**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
			13.2	117	35	▩	SW	WELL-GRADED SAND - moist to very moist, brown, some gravel Becomes more gravelly
180								
		18				☒		
							CL-ML	SILTY CLAY - moist, brown
	45						SM	SILTY SAND - brown, saturated, few gravel
			13.4	118	43	▩		
							SW	WELL-GRADED SAND - wet, brown, some gravel
175								
		87				☒		
	50							
170								brownish gray
			13.3	118	50	▩		
	55							
165								
		50				☒		
	60						CL	SANDY LEAN CLAY - moist, brown, few fine gravel
160							SM	SILTY SAND - moist, brown, few gravel
			11.6	124	33	▩		
	65							
							CL	SANDY LEAN CLAY - moist, brown, few gravel
155								
		45				☒		
	70							
150								Layers of sand and gravel
			13.7	117	43	▩		
	75							
							SC	CLAYEY SAND - moist, brown
145								
		61				☒		Becomes gravelly
	80							

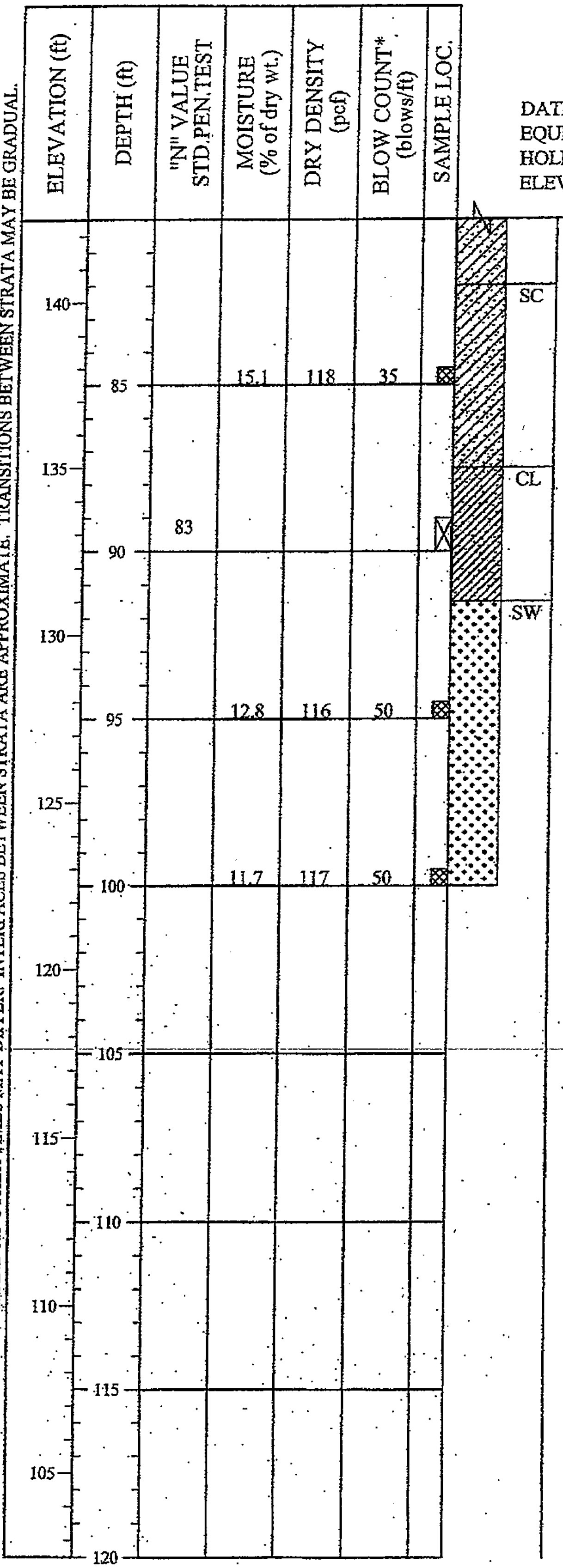
THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER.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

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: CMC
Prepared By: AO
Checked By: Ø

BORING 1 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 222.5**



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
	85		15.1	118	35	⊠
	90	83				☒
	95		12.8	116	50	⊠
	100		11.7	117	50	⊠

SC CLAYEY SAND - moist, brown, few gravel

CL SANDY LEAN CLAY - moist, brown, some gravel

SW WELL-GRADED SAND - wet, brown

END BORING AT 100 FEET

NOTES: Mud used during drilling process. Bailed hole to 50 feet +/- clean water. Water level at 31.5 feet at completion. Some caving at 41 feet. Water at 28.5 feet after 15 mins and 25 mins. Stable water possibly under hydrostatic pressure.Some slight seepage at (24'+/-, 21'-24'). First noticeable water at 39.5 feet. Boring grouted with a cement-bentonite mixture.

CMU Auto-trip hammer used in obtaining the "N-VALUE" Standard Penetration Test (SPT) blowcounts.

* Number of blows required to drive the Crandall sampler 12 inches using a 300 pound hammer falling 24 inches.

**Elevations are approximate and surveyed in prior to drilling.

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: CMC
Prepared By: AO
Checked By: CT

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING
Project: 4953-06-2101 Figure: A-1.1c

BORING 2

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 217.8**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
215						
	5		13.5	118	14	▨
210						
	10		14.4	114	4	▨
205		14				☒
	15		9.6	114	8	▨
200		6				☒
	20				15	○
195		36				☒
	25					
190			20.8	106	15	▨
	30	9				☒
185			17.6	112	8	▨
	35	30				☒
180						
	40		10.8	122	31	▨

SM	3" Thick Asphalt Concrete FILL - SILTY SAND - moist, brown, few gravel and cobbles
CL-ML	SILTY CLAY - moist, brown, trace gravel
CL	SANDY LEAN CLAY - moist, dark brown, some gravel
SC	CLAYEY SAND - with fine gravel, moist, brown
SW	WELL-GRADED SAND - wet, light brown, some gravel No Recovery Becomes less gravelly
CL-ML	SILTY CLAY - moist, brown, trace gravel
ML	SANDY SILT - moist, brown, fine to medium grained sand
SW	WELL-GRADED SAND - saturated, brown, some gravel Becomes less gravelly

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: CMC
Prepared By: AO
Checked By: LT

(CONTINUED ON FOLLOWING FIGURE)

BORING 2 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 217.8**

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.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							CL-ML	SILTY CLAY - moist, brown
175		12				☒		
	45						SM	SILTY SAND - with fine gravel, moist, brown Layer of clean sand
170								
	50		15.8	114	36	☒	SW	WELL-GRADED SAND - moist to wet, brown, trace gravel
165								
		37				☒		
	55						CL-ML	SILTY CLAY - fine to medium grained, brown, few sand
							SM	SILTY SAND - moist, fine to medium grained, some gravel
160								
	60		19.9	108	18	☒		
155		76				☒	ML	SANDY SILT - slightly moist, brown, fine to medium sand
	65						SM	SILTY SAND - with some fine gravel, very moist, brown
							CL-ML	SILTY CLAY - moist, brown, some sand
150								
	70		13.2	119	35	☒		
								Layers of well-graded sand and gravel
145								
		41				☒		Becomes sandy
	75							
140								Layer of well-graded sand with gravel
							SM	SILTY SAND - moist, brown, some gravel
	80		14.4	117	30	☒		

Field Tech: CMC
Prepared By: AO
Checked By: ℒ

(CONTINUED ON FOLLOWING FIGURE)

Melrose Triangle
West Hollywood, California

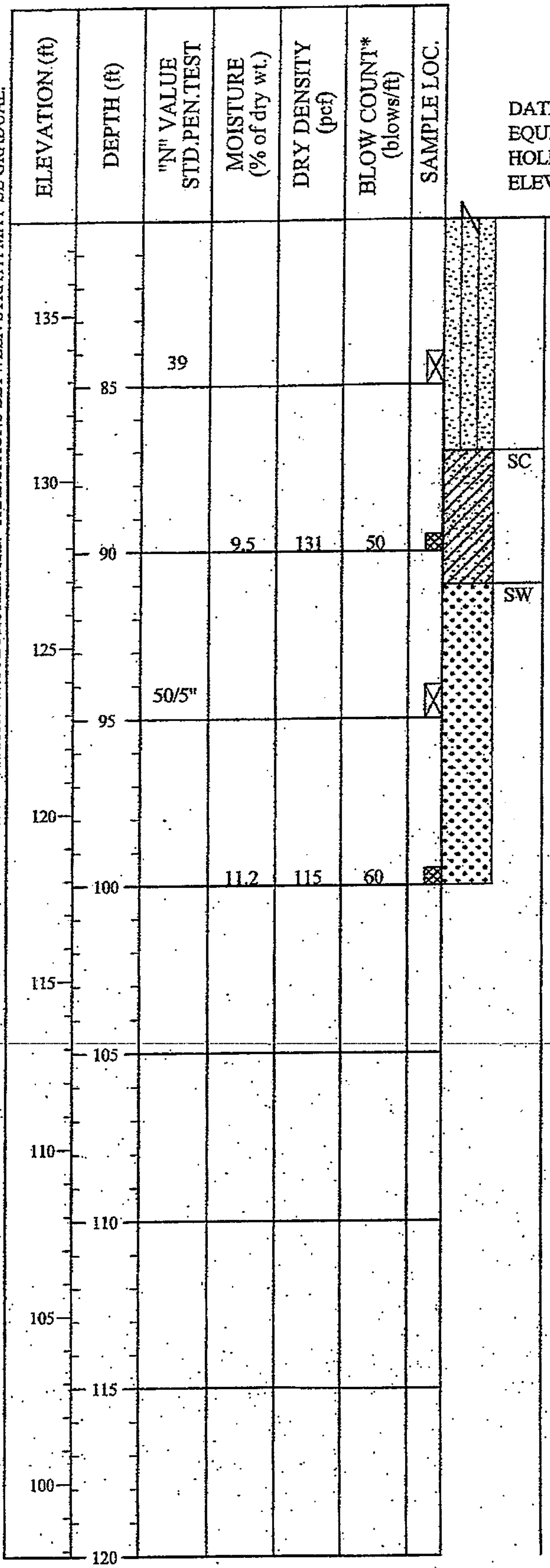
MACTEC

LOG OF BORING
Project: 4953-06-2101 Figure: A-1.2b

BORING 2 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 217.8**

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.



Thin layers of sand and gravel

CLAYEY SAND - moist, brown, trace gravel

WELL-GRADED SAND - saturated, brown, some gravel

END BORING AT 100 FEET

NOTES: Mud used during drilling process. Bailed hole to 35 feet +/- clean water. Water level remaining at 27 feet during bailing. Left hole open overnight. Water at 24 feet after 15 hrs. Some caving at 41 feet. Very coarse gravel at 19 1/2 to 21 1/2 feet, lost circulation. Boring grouted with a cement-bentonite mixture.

CMU Auto-trip hammer used in obtaining the "N-VALUE" Standard Penetration Test (SPT) blowcounts.

Field Tech: CMC
Prepared By: AO
Checked By: LJ

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING
Project: 4953-06-2101 Figure: A-1.2c

B12SOIL CRANDALL(DECIMAL ELE) 62101.GPJ LAW CRAN.GDT 7/15/08

BORING 3

DATE DRILLED: October 5, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 222.1**

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.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								4" Thick Asphalt Concrete, 4" Thick Base Course, 4" thick Concrete slab
							CL	SANDY LEAN CLAY - moist, fine to coarse grained, dark brown
220								
	5		18.3	103	11	▨		
215			16.3	112	15	▨		
	10						SM	SILTY SAND - moist, brown
			11.3	107	8	▨		
							CL	SANDY LEAN CLAY - moist, brown, fine to medium sand
210							SW-SM	WELL-GRADED SAND with SILT - moist, brown
		13				☒		
	15							
205			9.8	110	8	▨		
							ML	SANDY SILT - very moist, dark brown
		8				☒		
	20							
200							SM	SILTY SAND - moist, brown, some gravel
			13.3	118	15	▣		
							CL-ML	SILTY CLAY - saturated to moist, brown, some sand
	25	6				☒		
195								
			19.6	113	16	▨		
	30							
		20				☒		Becomes stiffer, less saturated
190								
							SW	WELL-GRADED SAND - saturated, brown, some gravel
	35		15.6	113	23	▨		Disturbed sample Some clay in bit
							CL	SANDY LEAN CLAY - very moist, brown, few fine gravel
185		7				☒		LL=30, PI=14
	40							

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: AR
Prepared By: AO
Checked By: LJ

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING

Project: 4953-06-2101 Figure: A-1.3a

BORING 3 (Continued)

DATE DRILLED: October 5, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 222.1**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
			17.2	112	15	⊠		
180		9				☒		
	45						SP	POORLY GRADED SAND - very moist, brown
175			21.1	106	23	⊠		Siltier with some clay
							CL-ML	SILTY CLAY - saturated, brown, some sand and gravel
		19				☒		
	50							
170								
	55				13	⊠		
165								
							SW	WELL-GRADED SAND - moist, brown, fine to coarse, some gravel
		54				☒		
	60							
							CL	SANDY LEAN CLAY - moist, brown
							SW	WELL-GRADED SAND - brown, some clay
160								
							SC	CLAYEY SAND - brown to dark brown, moist, some gravel
	65		9.6	129	39	⊠		
155								
		41				☒		
	70							
150								
							CL	SANDY LEAN CLAY - very moist, black
	75		13.6	122	38	⊠		
145								
		35				☒		
	80							

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: AR
Prepared By: AO
Checked By: LI

(CONTINUED ON FOLLOWING FIGURE)

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING

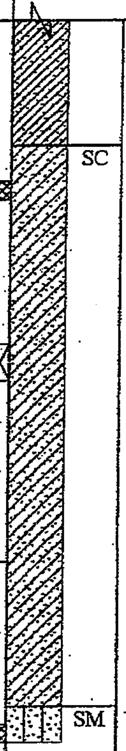
Project: 4953-06-2101 Figure: A-1.3b

BORING 3 (Continued)

DATE DRILLED: October 5, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 222.1**

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.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
140						
	85		13.9	117	48	☒
135						
	90	28				⊠
130						
	95					
125						
	100		11.8	124	65	☒
120						
	105					
115						
	110					
110						
	115					
105						
	120					



LL=31, PI=15

CLAYEY SAND - very moist, brown, some gravel

43% Passing No. 200 sieve

SILTY SAND - moist, brown, some gravel

END BORING AT 100 FEET

NOTES: Mud used during drilling process. Well within 5 feet, water at 26.6 feet. Boring grouted with a cement-bentonite mixture.

CMU Auto-trip hammer used in obtaining the "N-VALUE" Standard Penetration Test (SPT) blowcounts.

Field Tech: AR
Prepared By: AO
Checked By: LJ

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING
Project: 4953-06-2101 Figure: A-1.3c

BORING 4

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 213.7**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								5" Thick Concrete Slab
							CL	SANDY LEAN CLAY- slightly moist, dark brown
								few gravel
210								
	5							
			13.8	116	9	▩		
							SM	SILTY SAND - moist, brown, few gravel
205			11.7	109	5	▩		
	10							
		8				☒		
200							CL	SANDY LEAN CLAY - moist, brown, few gravel
			16.1	109	6	▩		
	15							
								moderately soft
		5				☒		
195								
							ML	SANDY SILT - very moist, brown
	20							
			16.5	113	5	▩		
								Possible seepage at 21.5 to 25 feet
		5				☒		
190								56.5% Passing No. 200 sieve
	25						CL	SANDY LEAN CLAY - brown, moist
			17.7	111	16	▩		
							SW	WELL-GRADED SAND - with some gravel, moist, brown
185								
		32				☒		
	30							
								from 30 feet to 34 feet - siltier layers
			12.2	120	15	▩		
							ML	SANDY SILT - very moist to wet, brown, firm, some gravel
180								
	35	64				☒		
175								
	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.

B12SOIL CRANDALL(DECIMAL ELE) 62101.GPJ LAW CRAN.GDT 7/15/08

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: CMC
Prepared By: AO
Checked By:

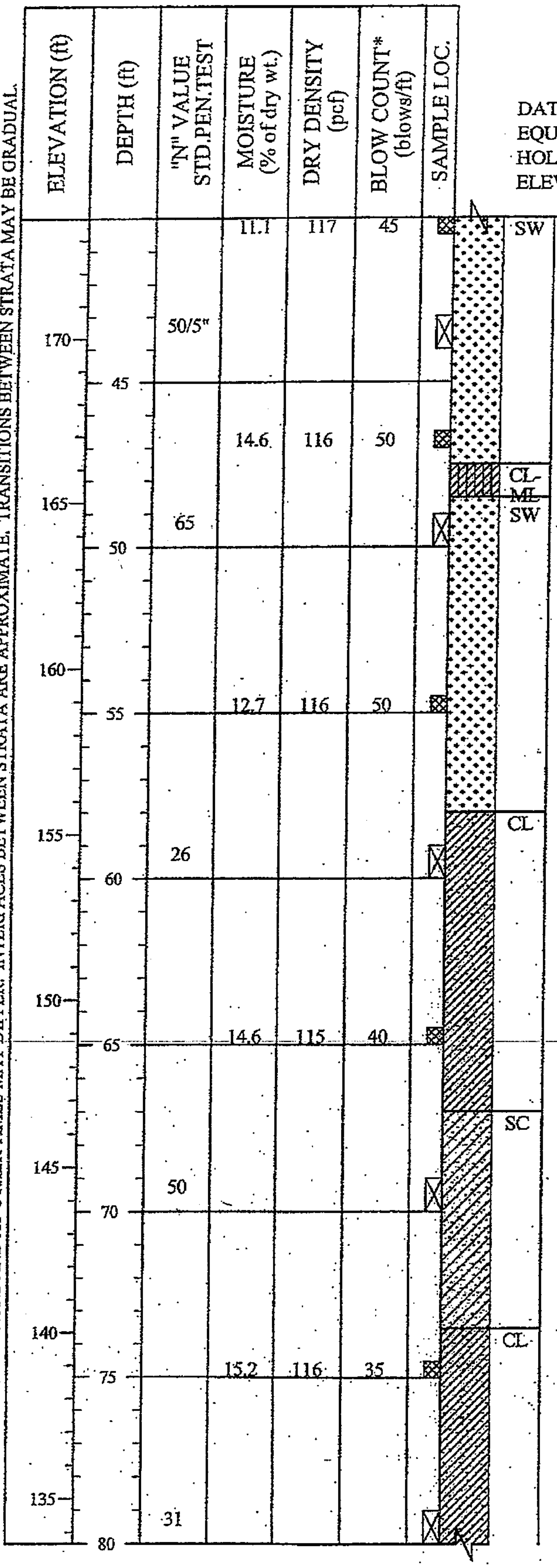
Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING
Project: 4953-06-2101
Figure: A-1.4a

BORING 4 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 213.7**



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
			11.1	117	45	▩	SW	WELL-GRADED SAND - very moist, brown
		50/5"				☒		more gravel and cobbles large cobbles
			14.6	116	50	▩		
							CL-ML	SILTY CLAY - saturated, brown
		65				☒	SW	WELL-GRADED SAND - moist, brown, some gravel
								Some larger gravel and small cobbles
			12.7	116	50	▩		
		26				☒	CL	SANDY LEAN CLAY - moist, brown, some gravel
			14.6	115	40	▩		
		50				☒	SC	CLAYEY SAND - slightly moist, brown
							CL	SANDY LEAN CLAY - moist, black
			15.2	116	35	▩		
		31				☒		

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: CMC
Prepared By: AO
Checked By: U

Melrose Triangle
West Hollywood, California

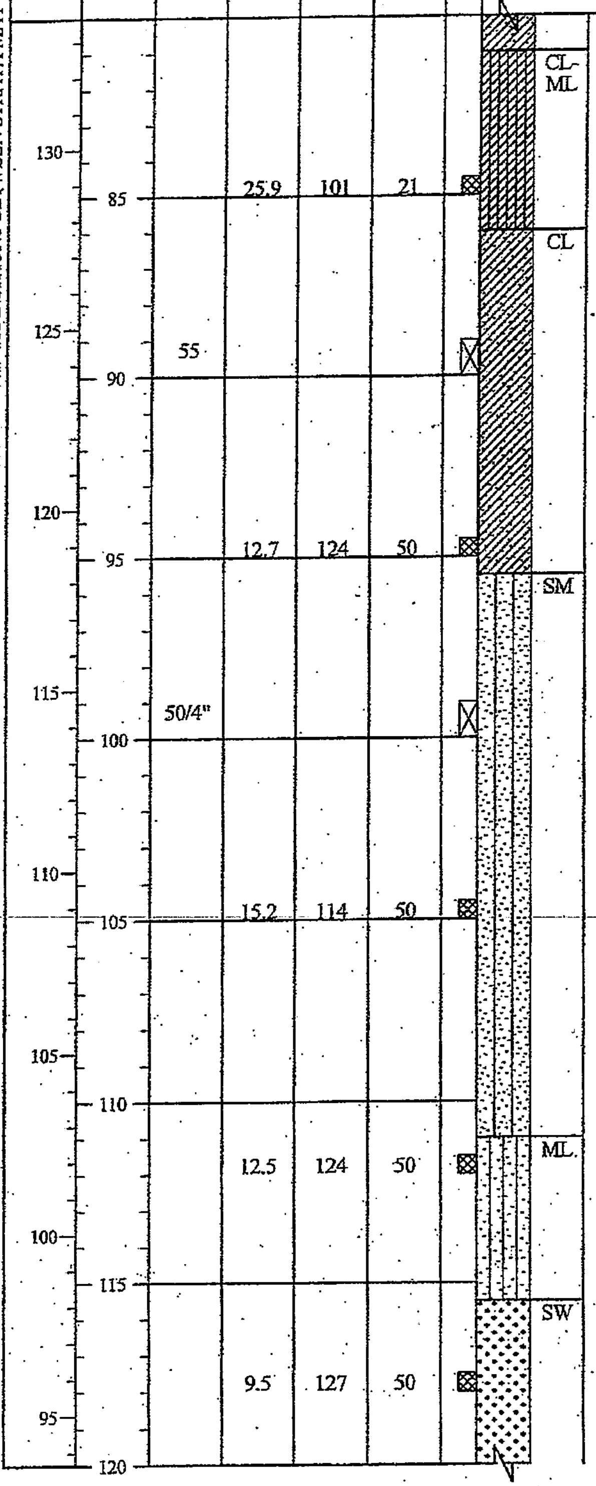
MACTEC

LOG OF BORING

Project: 4953-06-2101 Figure: A-1.4b

BORING 4 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 213.7**



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
			25.9	101	21	☒
		55				☒
			12.7	124	50	☒
		50/4"				☒
			15.2	114	50	☒
			12.5	124	50	☒
			9.5	127	50	☒

LL=36, PI=19

CL-ML SILTY CLAY - gray, moist, brown

70.3% Passing No. 200 sieve

CL SANDY LEAN CLAY - moist, brown, some gravel

SM SILTY SAND - slightly moist, brown, few gravel

ML SANDY SILT - brown, moist, some gravel

few small layers of clay

SW WELL-GRADED SAND - moist, brown, some gravel

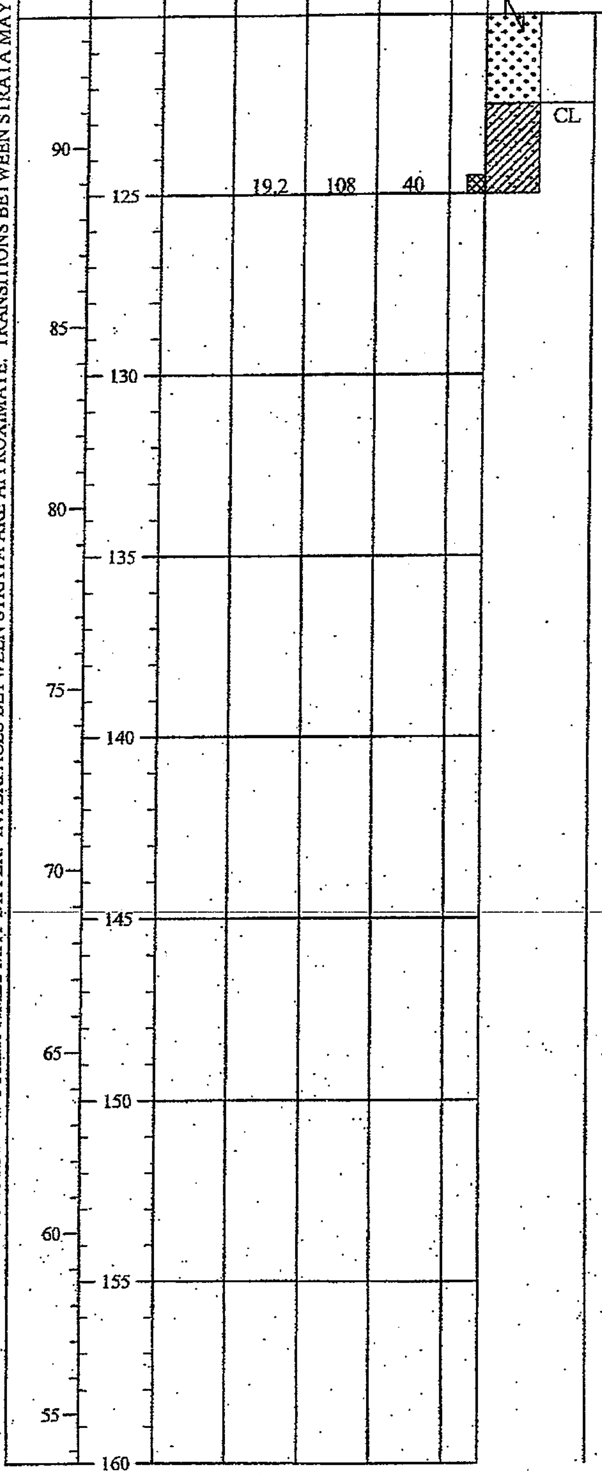
(CONTINUED ON FOLLOWING FIGURE)

Field Tech: CMC
Prepared By: AO
Checked By: LT

BORING 4 (Continued)

DATE DRILLED: October 3, 2006
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 213.7**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
	125		19.2	108	40	⊠



SANDY LEAN CLAY - slightly moist, brown

END OF BORING AT 125 FEET

Mud used during drilling process. Bailed hole to 46 feet. Water level remaining at 21.8 feet during bailing. Seepage at 28 feet. Some caving to 48 feet. Boring grouted with a cement-bentonite mixture.

CMU Auto-trip hammer used in obtaining the "N-VALUE" Standard Penetration Test (SPT) blowcounts.

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(DECIMAL ELE) 62101.GPJ LAW CRAN.GDT 7/15/08

Field Tech: CMC
Prepared By: AO
Checked By: LT

Melrose Triangle
West Hollywood, California

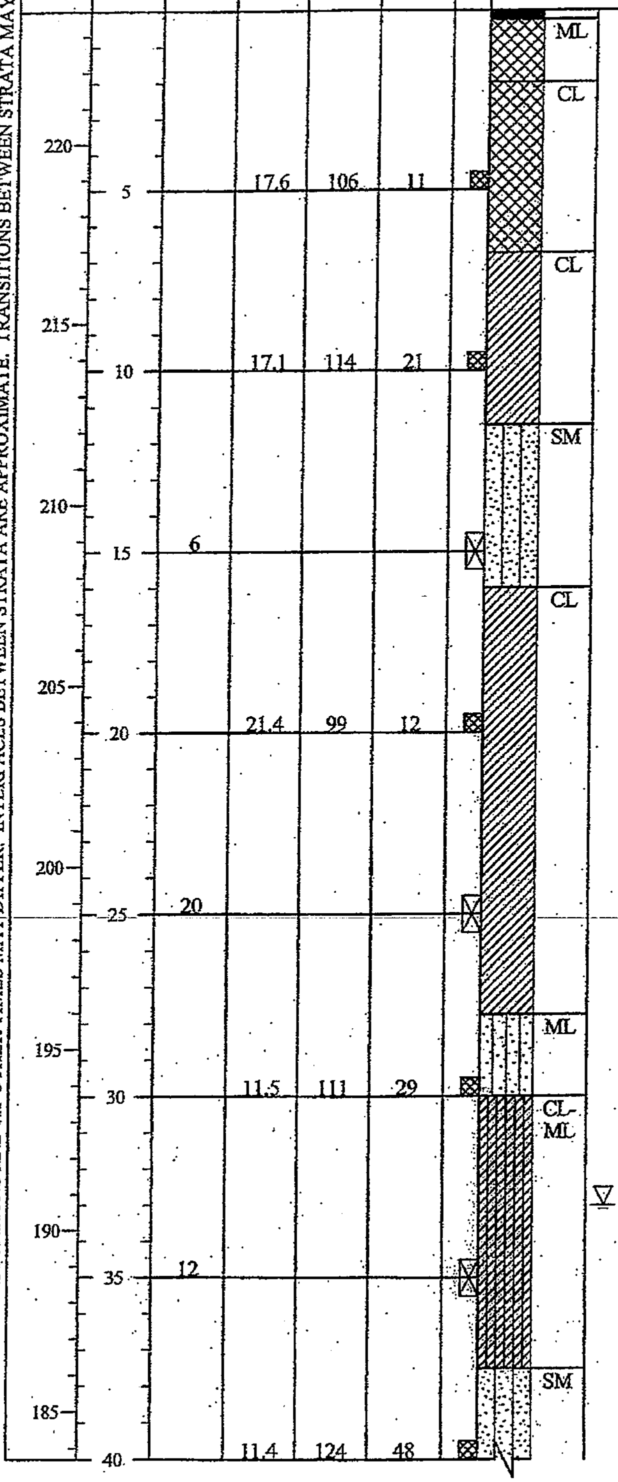
MACTEC

LOG OF BORING
Project: 4953-06-2101 Figure: A-1.4d

BORING 5

DATE DRILLED: May 19-20, 2008
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 223.7**

DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
5		17.6	106	11	⊠
10		17.1	114	21	⊠
15	6				☒
20		21.4	99	12	⊠
25	20				☒
30		11.5	111	29	⊠
35	12				☒
40		11.4	124	48	⊠



	3" Thick Asphalt Concrete
ML	FILL - SILT and CLAY - moist, brown, some sand and gravel
CL	FILL - SANDY CLAY - moist, brown, some gravel
CL	LEAN CLAY - moist, light brown, fine to medium sand
	LL = 44, PI = 23
SM	SILTY SAND - moist, light brown, fine to medium
	37.8% Passing No. 200 Sieve
CL	LEAN CLAY - moist, light brown, fine to coarse sand
	58.9% Passing No. 200 Sieve, LL = 40, PI = 17
	54.3% Passing No. 200 Sieve
ML	SANDY SILT - moist, light brown, fine to coarse sand
CL-ML	SILTY CLAY - moist, light brown
	wet, LL = 34, PI = 16
SM	SILTY SAND - wet, light brown, fine to coarse, some gravel
	24.6% Passing No. 200 Sieve

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: GMC
Prepared By: LT
Checked By: JT

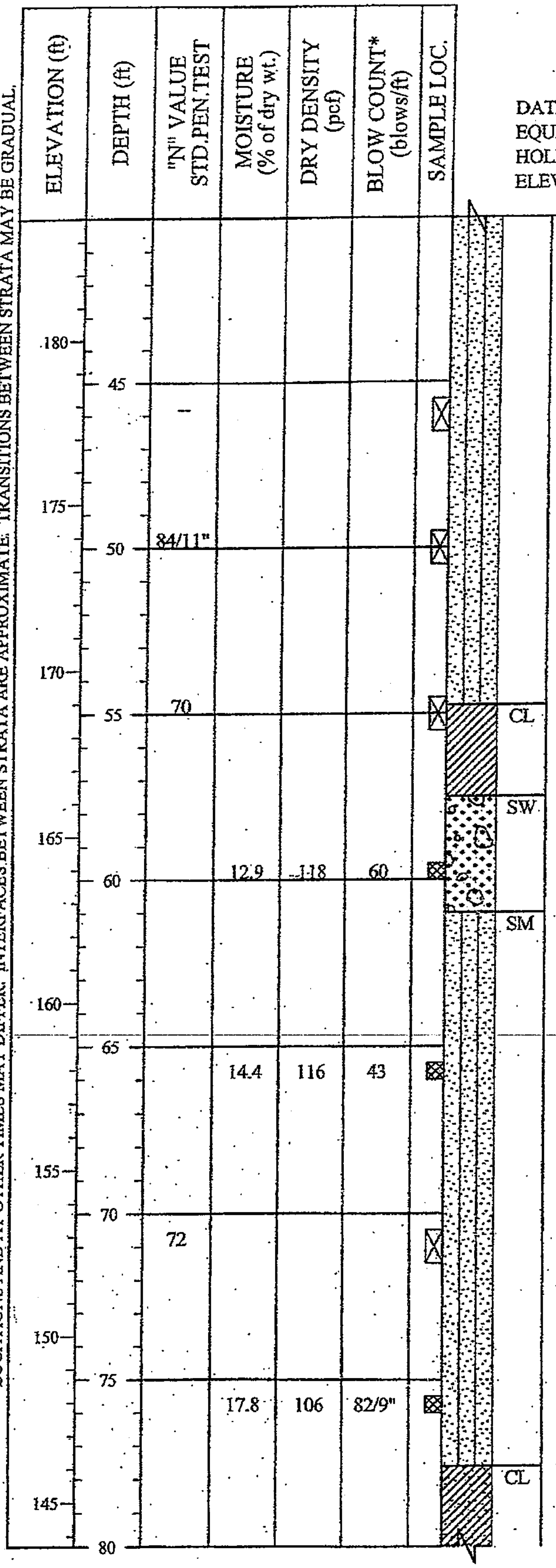
Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING
Project: 4953-08-0811
Figure: A-1.5a

BORING 5 (Continued)

DATE DRILLED: May 19-20, 2008
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 223.7**



Layer of cobbles to 47 feet

SILTY CLAY - wet, light brown, some fine sand

WELL GRADED SAND with Gravel - wet, light brown, fine to coarse

Lenses of Silt

SILTY SAND - wet, light brown, fine to coarse, some slate gravel

Layer of large gravel

More gravel to 73 feet, 17.7% Passing No. 200 Sieve

SANDY CLAY - wet, gray, fine to medium sand

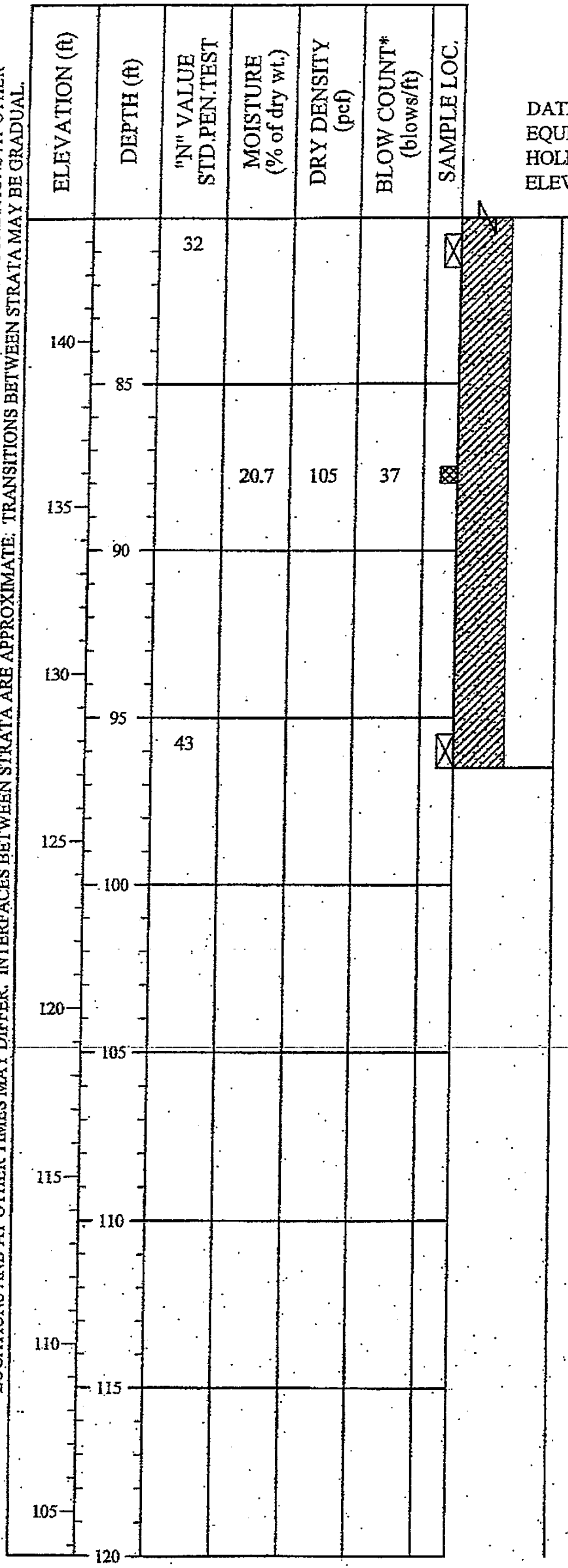
(CONTINUED ON FOLLOWING FIGURE)

Field Tech: GMC
Prepared By: LT
Checked By: JT

BORING 5 (Continued)

DATE DRILLED: May 19-20, 2008
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 223.7**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.
		32				☒
			20.7	105	37	▩
		43				☒



END BORING AT 96.5 FEET

NOTES: Mud used during drilling process. Bailed hole to 36 feet. Water level at 33 feet at completion of drilling. Some caving occur in boring. Boring grouted with a cement-bentonite mixture.

CMU Auto-trip hammer used in obtaining the "N-VALUE" Standard Penetration Test (SPT) blowcounts.

* Number of blows required to drive the Crandall sampler 12 inches using a 300 pound hammer falling 24 inches.

**Elevations are approximate and surveyed based on referenced datum.

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(DECIMAL ELE) 80811.GPJ LAW_CRAN.GDT 7/15/08

Field Tech: GMC
Prepared By: LT
Checked By: JT

BORING 6

DATE DRILLED: May 21, 2008
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 209.8**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
	0							4" Concrete Slab
							ML	FILL - CLAYEY SILT - moist, dark brown, fine to medium sand, some slate fragment
							ML	CLAYEY SILT - moist, brown, fine sand
							SM	SILTY SAND - moist, brown, fine to coarse, some clay
205	5		17.2	–	9	▩		
							CL	LEAN CLAY - moist, brown, fine to medium sand
200	10	7				☒		Thin layer of silty sand, fine to coarse, some rootlets
							SM	SILTY SAND - moist, brown and some dark gray, fine to coarse, some clay, some small slate fragments
195	15		–	–	16	○		
190	20	11				☒	∇	
185	25		–	–	31	○		
							ML	CLAYEY SILT - moist, brown and dark gray, fine to coarse sand
180	30	20				☒		
							SW	WELL GRADED SAND - moist, brown, fine to coarse, some gravel
175	35		–	–	32	○		
							CL	SANDY LEAN CLAY - moist, brown, fine to coarse
170	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.

B12SOIL CRANDALL(DECIMAL ELE) 80811.GPJ LAW CRAN.GDT 7/15/08

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: AR
Prepared By: LT
Checked By: JT

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING
Project: 4953-08-0811 Figure: A-1.6a

BORING 6 (Continued)

DATE DRILLED: May 21, 2008
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 5
ELEVATION: 209.8**

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.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
		17				☒		LL = 32, PI = 12
							SM	SILTY SAND - moist, brownish gray, very fine to medium
165	45	36				☒		
160	50	43				☒		16.6% Passing No. 200 Sieve
155	55	42				☒		
150	60							More gravel
		51				☒		
145	65							
140	70							
								END BORING AT 73 FEET DUE TO DIFFICULT DRILLING
135	75							
130	80							

NOTES: Mud used during drilling process. Bailed hole to 22 feet. Ground water measured at 21 feet after completion of drilling. Boring grouted with a cement-bentonite mixture.

Field Tech: AR
Prepared By: LT
Checked By: JI

B12SOIL CRANDALL(DECIMAL ELE) 8081I.GPJ LAW CRAN.GDT 7/15/08

Melrose Triangle
West Hollywood, California

MACTEC

LOG OF BORING

Project: 4953-08-0811 Figure: A-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
☒	Standard Penetration Test	≋	Bulk Sample
▯	Rock Core	▩	Crandall Sampler
▩	Dilatometer	^	Pressure Meter
□	Packer	○	No Recovery
▽	Water Table at time of drilling	▼	Water Table after drilling

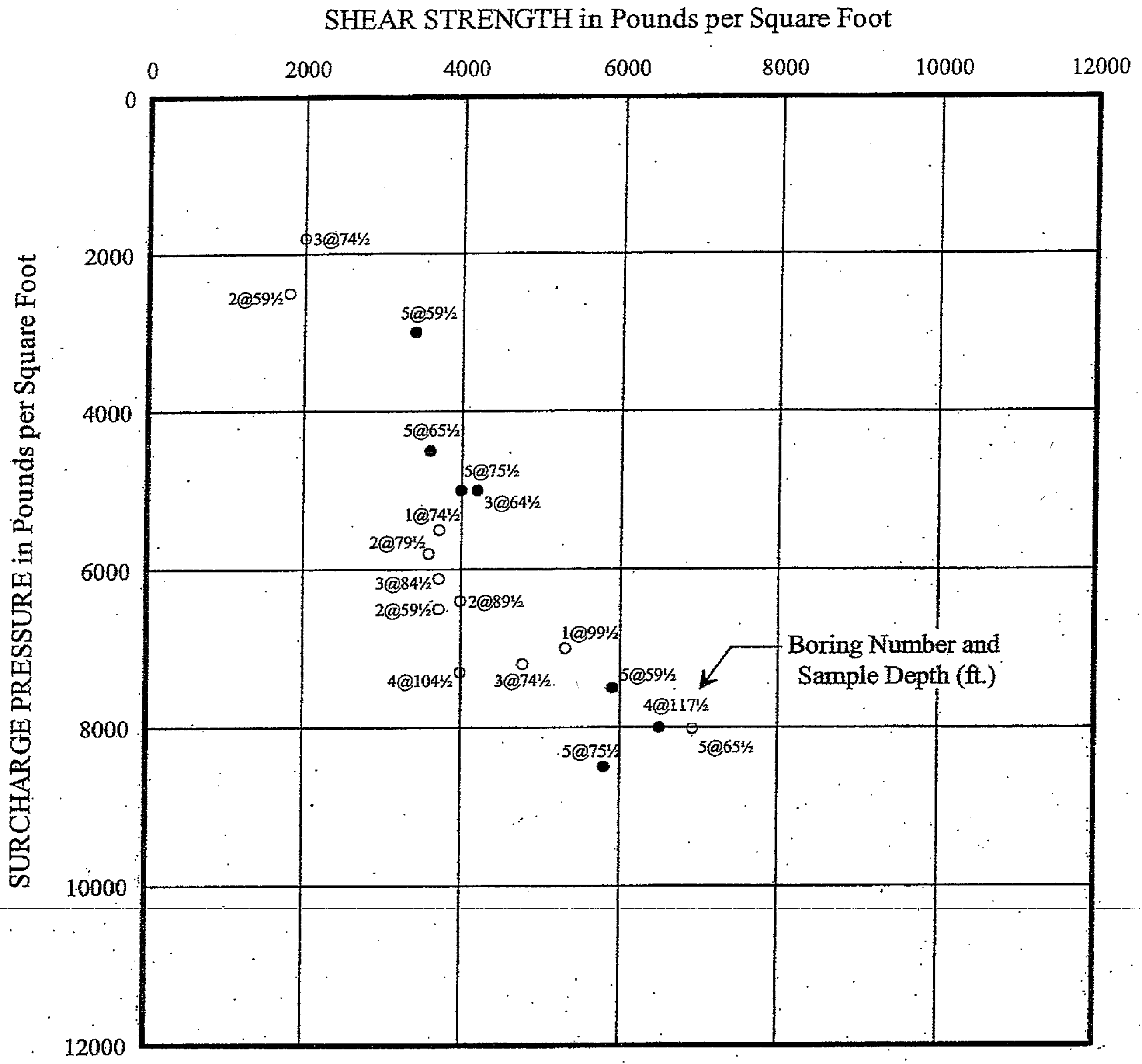
Correlation of Penetration Resistance with Relative Density and Consistency

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

KEY TO SYMBOLS AND DESCRIPTIONS

MACTEC

Figure A-2



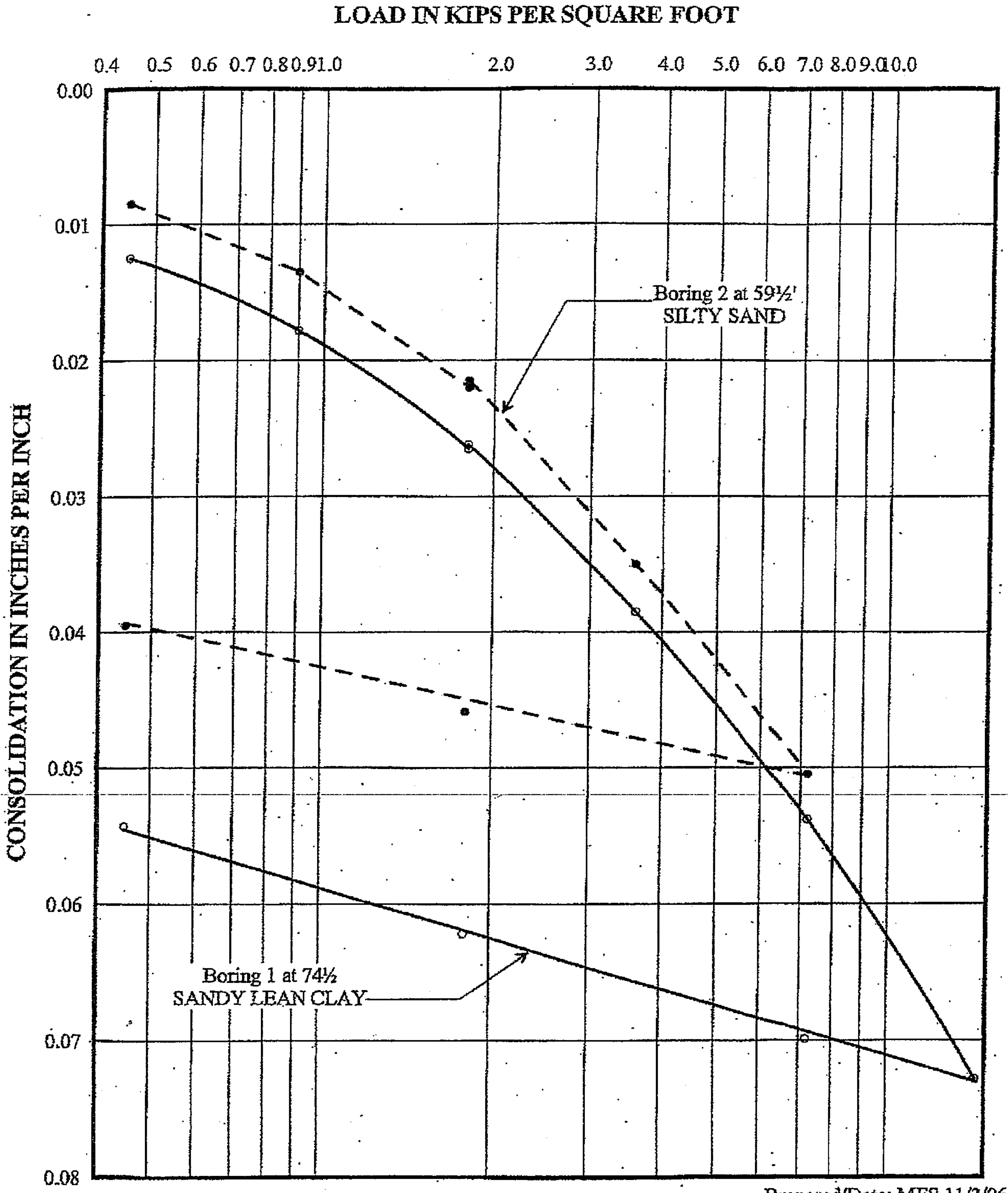
KEY:
- ● Samples tested at field condition.
- ○ Samples tested at a moisture content near saturation.

Prepared/Date: LT 6/27/08
Checked/Date: JY

Melrose Triangle
West Hollywood, California

MACTEC

DIRECT SHEAR TEST DATA
Project No. 4953-08-0811
Figure A-3



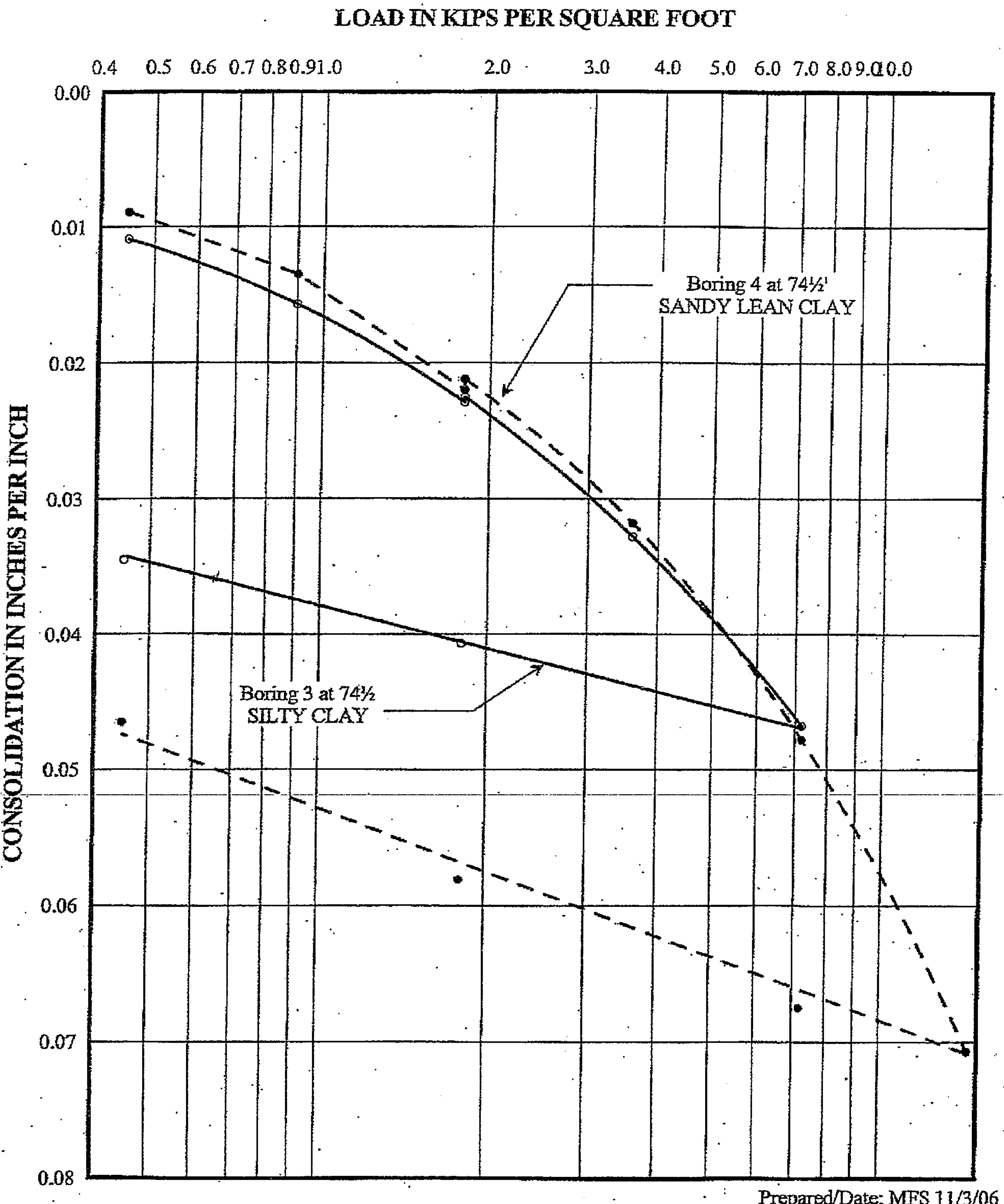
NOTE: Water added to samples after consolidation under a load of 1.8 kips per square feet

Prepared/Date: MFS 11/3/06
Checked/Date: JR

Melrose Triangle
West Hollywood, California

MACTEC

CONSOLIDATION TEST DATA
Project No. 4953-06-2101
Figure A-4.1



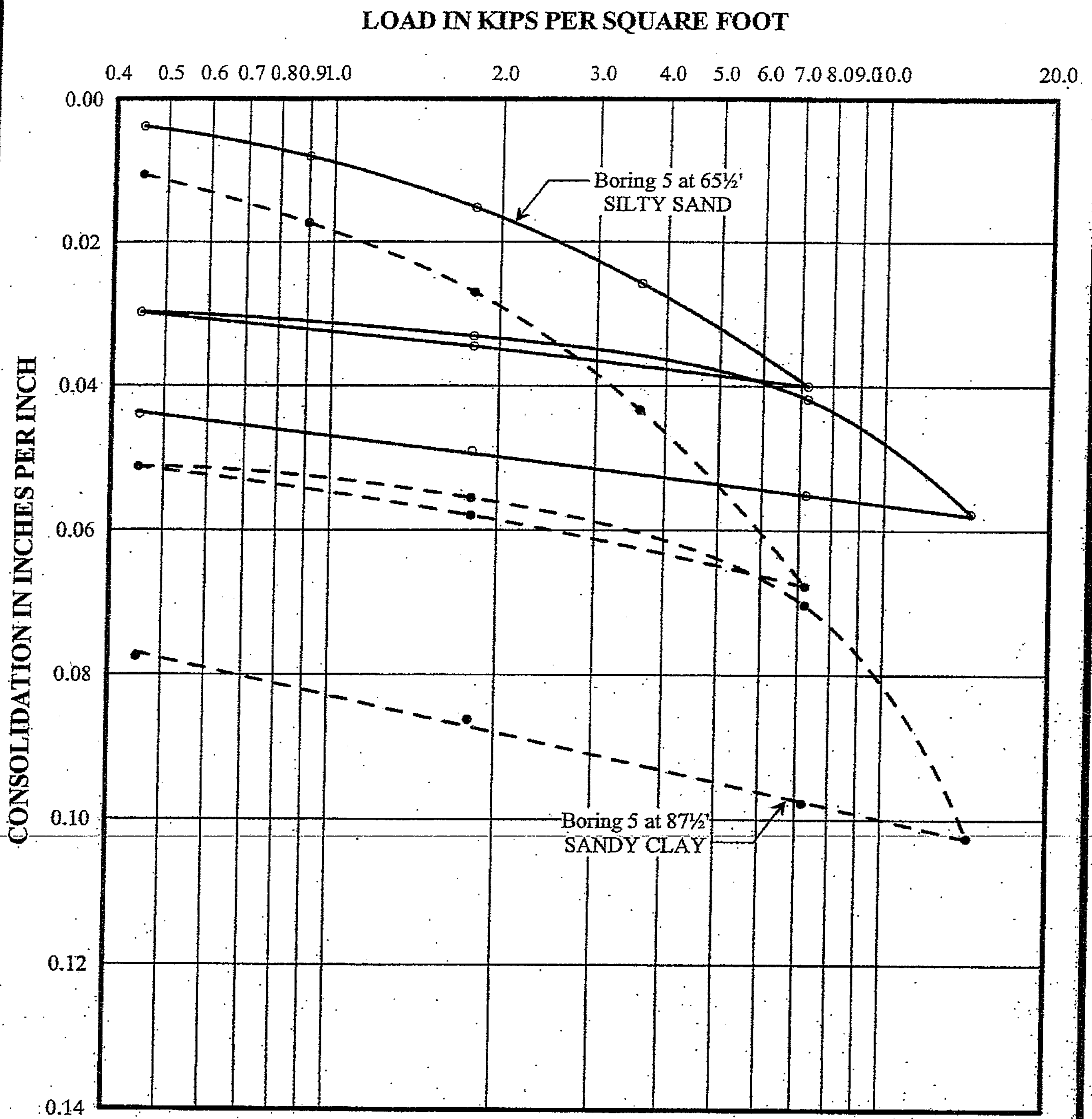
NOTE: Water added to samples after consolidation under a load of 1.8 kips per square feet

Prepared/Date: MFS 11/3/06
Checked/Date: JR

Melrose Triangle
West Hollywood, California

MACTEC

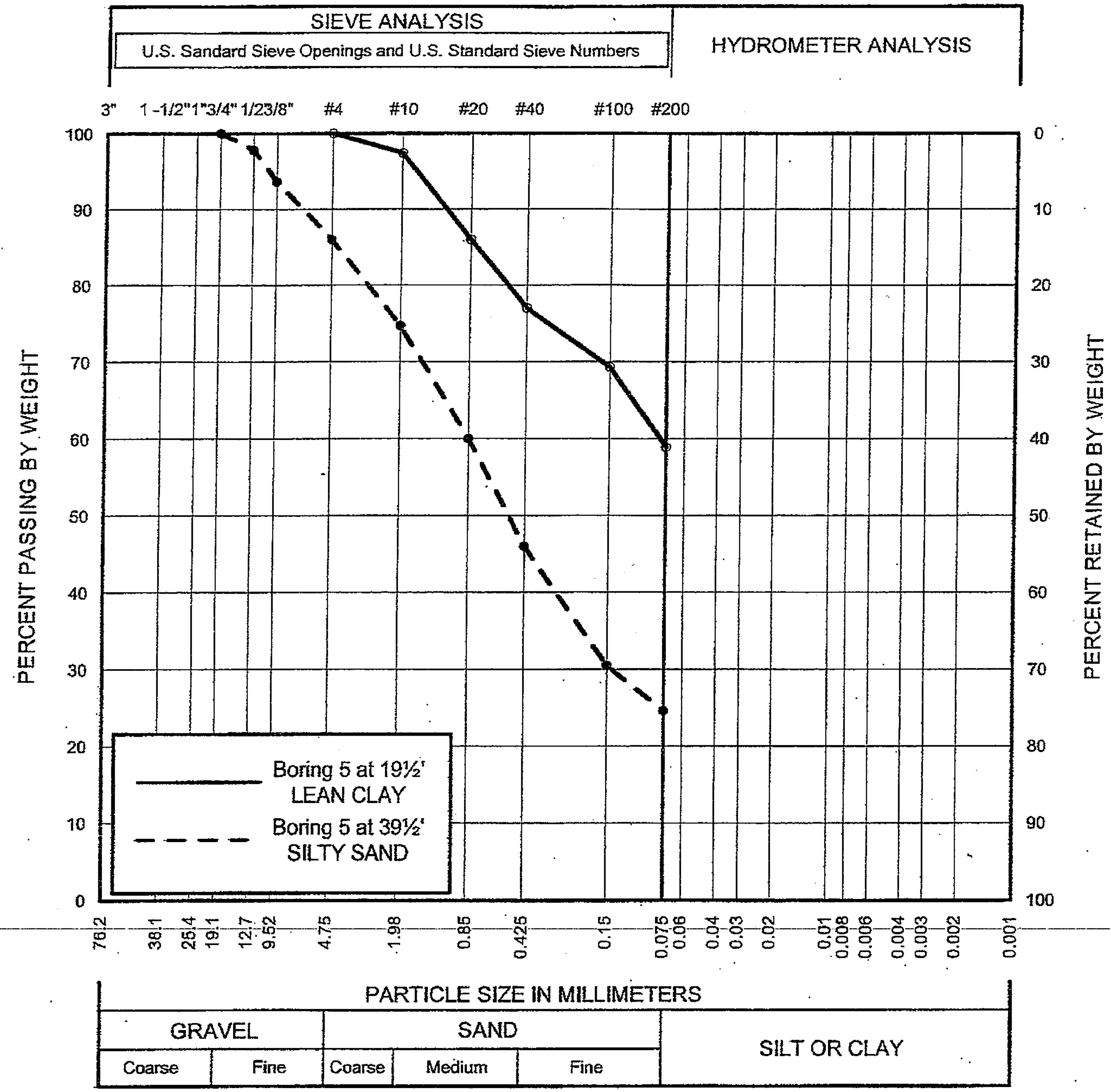
CONSOLIDATION TEST DATA
Project No. 4953-06-2101
Figure A-4.2



NOTE: Samples tested at field moisture content.

Prepared/Date: LT 3/10/2008
Checked/Date: VT

Melrose Triangle West Hollywood, California	MACTEC	CONSOLIDATION TEST DATA Project No. 4953-08-0811 Figure A-4.3

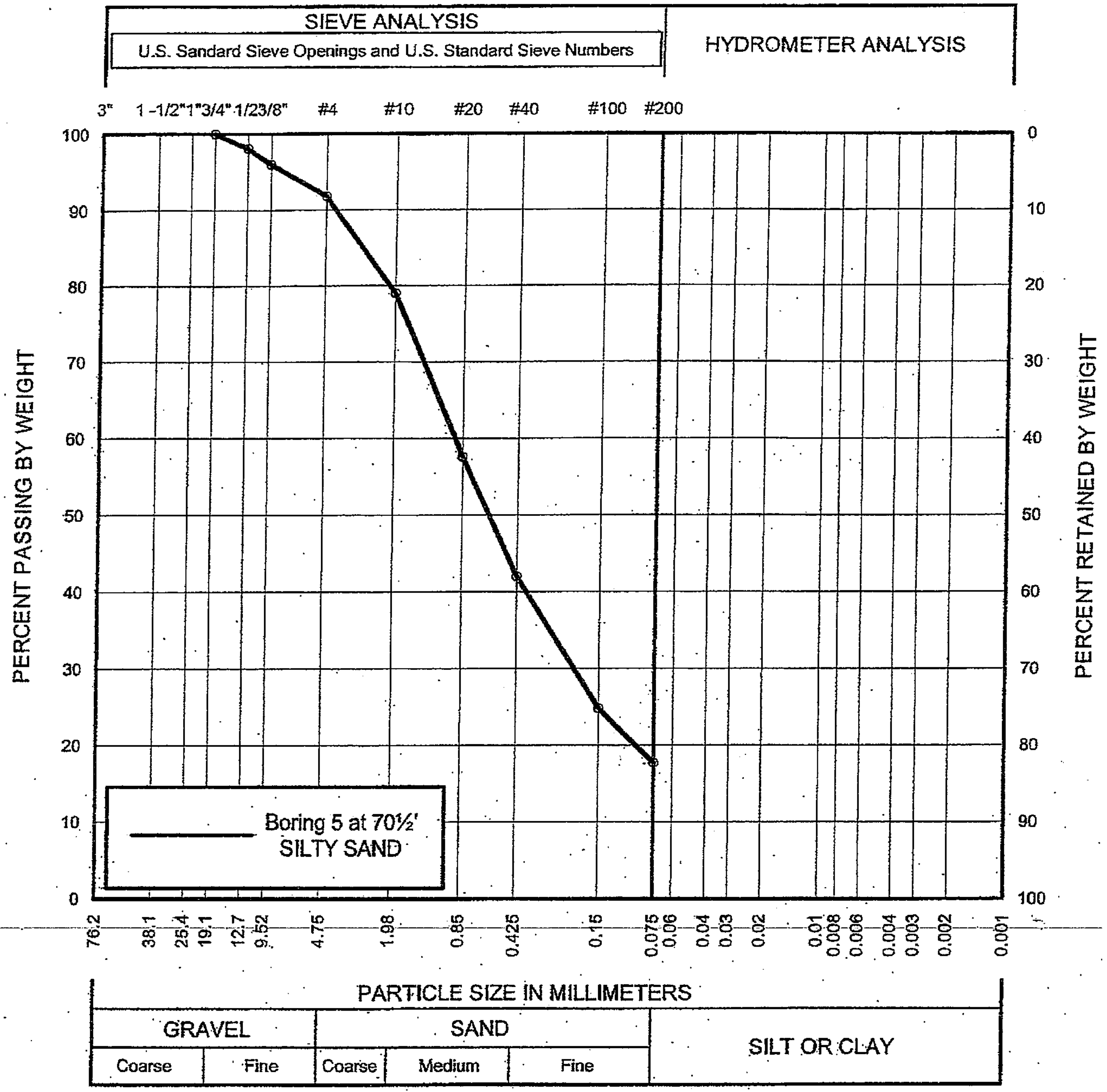


Prepared/Date: LT 6/26/08
Checked/Date: JT

Melrose Triangle
West Hollywood, California

MACTEC

PARTICLE SIZE DISTRIBUTION
Project No. 4953-08-0811
Figure A-5.1



Prepared/Date: LT 6/26/08
Checked/Date:

Melrose Triangle
West Hollywood, California

MACTEC

PARTICLE SIZE DISTRIBUTION
Project No. 4953-08-0811
Figure A-5.2

2008-proj\80811\Test docs\sieve.grf

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Table 1 - Laboratory Tests on Soil Samples

MACTEC
Melrose
Your #4953-06-2101, SA #06-1812LAB
20-Oct-06

Sample ID			B-1 @ 4.5 Fill	B-2 @ 9.5 Fill	B-3 @ 7.5 Fill
Resistivity		**Units**			
as-received		ohm-cm	68,000	18,800	13,200
saturated		ohm-cm	1,440	1,120	1,000
pH			7.6	7.7	7.2
Electrical Conductivity		mS/cm	0.21	0.36	0.18
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	142	267	69
magnesium	Mg^{2+}	mg/kg	27	33	17
sodium	Na^{1+}	mg/kg	59	120	123
potassium	K^{1+}	mg/kg	32	36	28
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	278	403	275
flouride	F^{1-}	mg/kg	1.8	3.9	1.8
chloride	Cl^{1-}	mg/kg	18	23	7.2
sulfate	SO_4^{2-}	mg/kg	163	450	127
phosphate	PO_4^{3-}	mg/kg	1.0	4.2	1.0
Other Tests					
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	1.2	ND	ND
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis 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-6

SUMMARY
OF
CONE PENETRATION TEST DATA

Project:

Melrose Avenue & N. Almont Drive
W. Hollywood, CA
May 19, 2008

Prepared for:

Mr. Mark Murphy
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Prepared by:

KEHOE TESTING & ENGINEERING
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Office (714) 901-7270 / Fax (714) 901-7289

TABLE OF CONTENTS

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at Melrose Avenue & N. Almont Drive in W. Hollywood, California. The work was performed by Kehoe Testing & Engineering (KTE) on May 19, 2008. The scope of work was performed as directed by MACTEC Engineering & Consulting, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at one location to determine the soil lithology. Groundwater measurements were taken in the open CPT hole approximately ten minutes after completing the CPT sounding. The following **TABLE 2.1** summarizes the CPT soundings performed:

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	49	Refusal, hole open to 8 ft (dry)
CPT-2		
CPT-3		

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cordless cone system manufactured by Geotech Equipment. **FIGURE 3.1** provides a schematic drawing of this system. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a direct push rig anchored with an eight-inch diameter ground anchor. This rig has a pushing capacity of approximately 15 tons. The cones used during the program recorded the following parameters at 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

The above parameters were recorded and viewed in real time using a portable computer and stored on a diskette for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly. The electronic cones manufactured by Geotech Equipment have downhole analog to digital conversion and temperature compensation to provide accurate load measurements. **FIGURE 3.2** provides the specifications of the cone penetrometer used for this project.

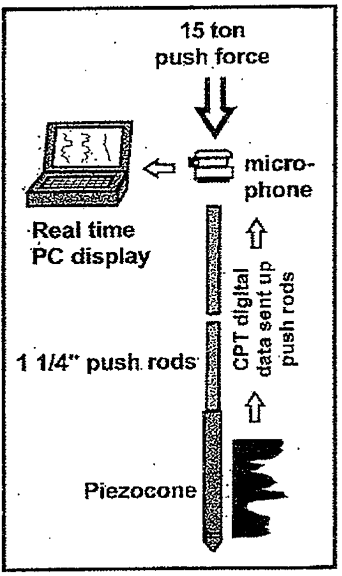


FIGURE 3.1 – CPT System

CONE SERIAL #:	3283
CAPACITIES	
Cone Resistance Capacity:	100 Mpa
Friction Sleeve Capacity:	1.00 Mpa
Pore Pressure Capacity:	2.50 Mpa
DIMENSIONS	
Tip Projected Area:	10 cm^2
Friction Sleeve Area:	150 cm^2
Pore Pressure Filter Thickness:	5 mm
Area Ratio:	0.85
PORE PRESSURE FILTER	
Material:	Sintered Brass
Saturation Material:	Glycerin

FIGURE 3.2 – Specifications of Cone Penetrometer

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the CPT Classification Chart (provided in the Appendix) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Output from the interpretation program CPTINT provides averaged CPT data over one-foot intervals. The CPTINT output includes Soil Classification Zones (uses "non normalized" chart), SPT N Values, Undrained Shear Strength (Su) and Friction Angle (phi). A summary of the equations used for the tabulated parameters is provided in the CPTINT Correlation Table in the Appendix.

The interpretation of soils encountered on this project was carried out using correlations developed by Robertson et al, 1988. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgment and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

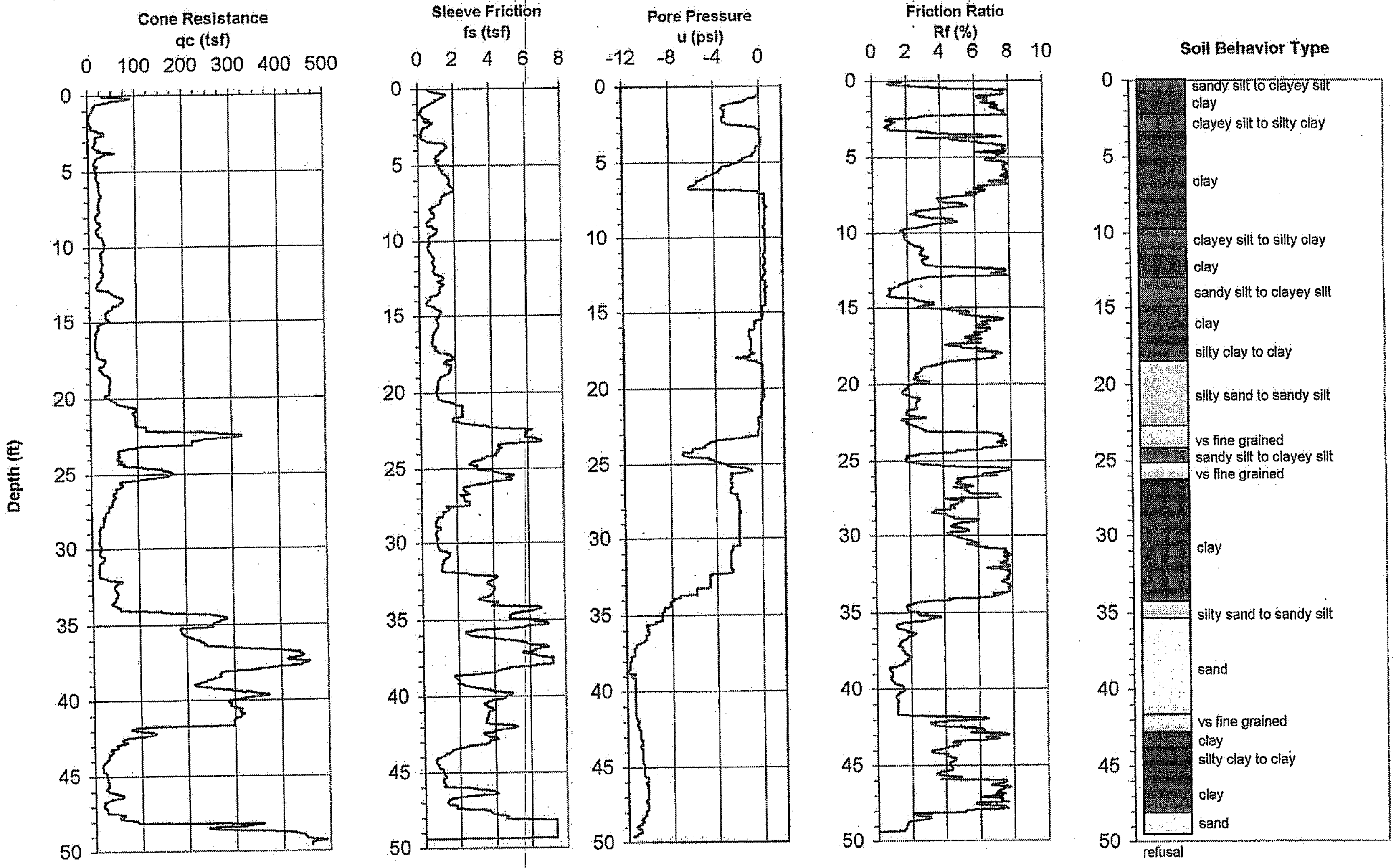
05/21/08-kr-74-8654

APPENDIX

KTE
KEHOE TESTING & ENGINEERING

Project : Melrose Ave & N. Almont Dr., W. Hollywood, CA **Client :** MACTEC

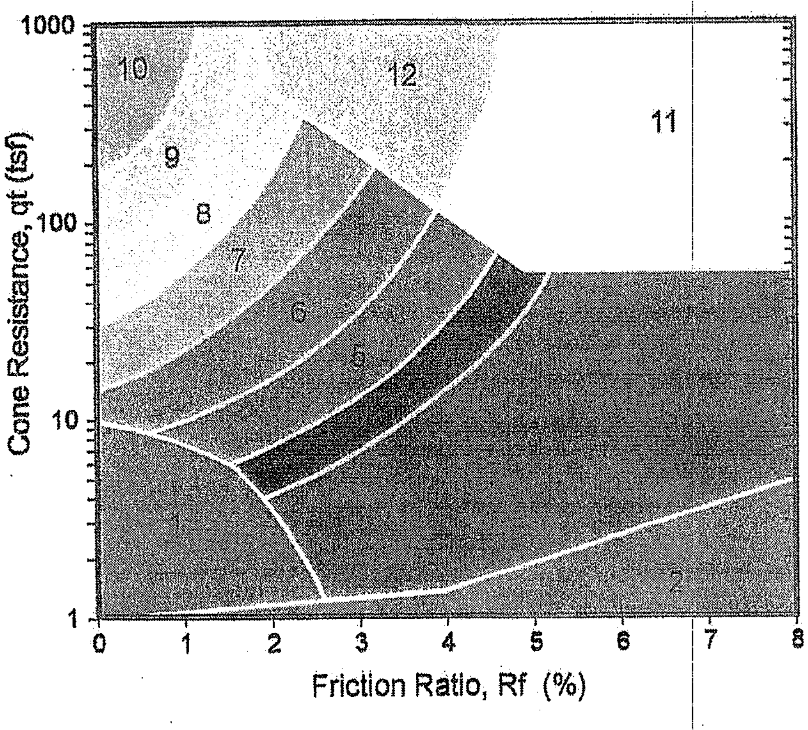
Location : CPT-1 (PROJECT No. 4953-08-0811) **CPT Date :** 5/19/08



KEHOE TESTING & ENGINEERING

CPT Classification Chart

(after Robertson and Campanella, 1988)



Zone		q_t / N	Soil Behavior Type	UCSCS
1	▨	2	sensitive fine grained	OL-OH
2	▨	1	organic material	Pt-OH
3	▨	1	clay	CH
4	▨	1.5	silty clay to clay	CL-CH
5	▨	2	clayey silt to silty clay	ML-CL
6	▨	2.5	sandy silt to clayey silt	MH-ML
7	▨	3	silty sand to sandy silt	SM-ML
8	▨	4	sand to silty sand	SP-SM
9	▨	5	sand	SP
10	▨	6	gravelly sand to sand	SW-SP
11		1	very stiff fine grained *	CL-MH
12	▨	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

```
PUT FILE: c:\temp\CPT-1.CSV   |---------------------------------------------------------
 Depth      Qc(avg)     Fs(avg)    Rf         Rf Zone     Spt N      Spt N1     Su
 (feet)     (TSF)       (TSF)      (%)        (zone #)    (blow/ft)  (blow/ft)  (TSF)
-----------------------------------------------------------------------------------------
  0.500      36.756      1.087      2.958      6          14         21           9E9
  1.500       4.256      0.290      6.861      3           4          6         0.275
  2.500      18.717      0.408      2.182      5           9         14         1.236
  3.500      22.281      0.860      3.858      4          14         21         1.471
  4.500      15.754      1.129      7.173      3          15         23         1.031
  5.500      18.970      1.439      7.606      3          18         27         1.239
  6.500      23.786      1.774      7.476      3          23         35         1.555
  7.500      20.804      1.087      5.224      3          20         30         1.356
  8.500      19.104      0.673      3.520      4          12         18         1.239
  9.500      27.125      0.790      2.912      5          13         20         1.770
 10.500      27.950      0.580      2.076      6          11         15           9E9
 11.500      27.933      0.814      2.912      5          13         17         1.815
 12.500      23.471      1.155      4.920      3          22         27         1.513
 13.500      61.803      0.935      1.512      7          20         23           9E9
 14.500      39.805      0.840      2.111      6          15         16           9E9
 15.500      18.726      1.011      5.397      3          18         19         1.184
 16.500      12.144      0.755      6.223      3          12         12         0.741
 17.500      23.464      1.349      5.753      3          22         21         1.491
 18.500      34.324      1.572      4.581      4          22         20         2.212
 19.500      38.831      1.010      2.601      6          15         13           9E9
 20.500      71.443      1.409      1.972      7          23         19           9E9
 21.500      98.258      2.209      2.248      7          31         25           9E9
 22.500     232.648      5.103      2.193      7          74         57           9E9
 23.500      87.608      5.150      5.881     11          84         64           9E9
 24.500     102.666      3.378      3.293      6          39         29           9E9
 25.500     102.212      4.335      4.242     11          98         72           9E9
 26.500      49.279      2.546      5.170      3          47         34         3.173
 27.500      35.343      2.054      5.816      3          34         24         2.240
 28.500      22.556      0.951      4.219      4          14         10         1.384
 29.500      17.835      0.871      4.890      3          17         12         1.065
 30.500      20.671      1.212      5.870      3          20         14         1.249
 31.500      16.578      1.268      7.665      3          16         11         0.972
 32.500      50.215      3.802      7.578      3          48         32         3.209
 33.500      50.680      3.812      7.533      3          48         31         3.235
 34.500     233.038      5.521      2.370      7          74         47           9E9
 35.500     208.570      4.754      2.280      7          67         42           9E9
 36.500     316.723      5.453      1.722      8          76         47           9E9
 37.500     415.712      6.673      1.606      9          80         49           9E9
 38.500     253.956      3.085      1.215      9          49         30           9E9
 39.500     306.948      3.338      1.088      9          59         35           9E9
 40.500     302.967      3.843      1.269      9          58         34           9E9
 41.499     246.275      3.783      1.537      8          59         35           9E9
 42.499      85.001      3.972      4.681     11          81         47           9E9
 43.499      35.719      1.941      5.455      3          34         19         2.192
 44.499      19.899      0.807      4.081      4          13          7         1.133
 45.499      26.740      1.095      4.116      4          17         10         1.585
 46.499      35.239      2.547      7.253      3          34         19         2.148
 47.499      46.191      3.035      6.588      3          44         24         2.874
 48.499     315.924      7.064      2.237      8          76         41           9E9
 49.499     464.566      4.435      0.955      9          89         48           9E9
```

CPTCP.TBL - CPTINT Correlation and Parameters Table File
-------------with NOTES & References at end-------------

Program: CPTINT - CPT Cone Interpretation Program
Version: 5.2
Table File by: Dr. R. G. (DICK) Campanella, P.Eng.
Rev. Dated: April 3, 2002

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Depth average see NOTE #1	Depth averaged over specified range (see menu)		All	All
Parameter Averaging	Averaged over range specified for depth. If no values exist, your choice is zero's or no value		All	All
Qc, Tip Stress	measured tip force/area	#6,#8	All	All
Qt corrtd for U2 see NOTE #2	Qt = Qc + (1 - a) x U2 and a = tip area ratio Defaults to U2 if given or uses U1 or U3 times Const.	#6,#8	All	All
[Note: Input value from input file is used if defined, not calculated]				
Q (Qt Normalized)	$Q = \frac{Qt - sv}{sv'}$	#9 & 13	All	All
Fs	measured sleeve force/area	#6,#8	All	All
Rf Friction Ratio (if Rf>8, Rf=8)	$Rf = \frac{Fs}{Qt} \times 100\%$	#6,#8	All	All
F (Rf Normalized)	$F = \frac{Fs}{(Qt - sv)} \times 100\%$	#9 & 13	All	All
Gamma Total Unit Weight (Soil + Water) see NOTE #3	Based on Rf or Bq Classif. Zone Zone # Gamma = kN/m^3 1 Qt<4bar 15.70 1 Qt=4bar 17.30 2 Rf<5% 13.36 2 Rf=5% 11.80 2 Bq Zone 12.58 3 Qt<10bar 18.86 3 Qt=10bar 19.65 4, 5 & 6 Qt<20bar 18.86 4, 5 & 6 Qt=20bar 19.65 7 18.86 8 & 9 19.65 10 20.44 11 & 12 21.22		All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
U Penetration Pore Pressure see NOTE #4	U1,measured on Face of tip U2,measured Behind Tip at shoulder (std location) U3,measured Behind Friction Sleeve		All	All
Water Table	Depth below ground surface to where pore pressure = 0 Make negative if water level is above ground		All	All
Uo Hydrostatic Pore Pressure see NOTE #4	Uo = water depth,Hw x unit weight water, Gamma or Uo=Hw=depth-depth to water table if depth<water table,Uo = 0		All	All
dU Excess Pore Pressure	dU = U2 - Uo Defaults to U2 if given or uses U1 or U3 x const.		All	All
DPPR (Differential Pore Pressure Ratio)	$DPPR = \frac{dU}{Qt} = \frac{U - Uo}{Qt}$ Defaults to U2 if given or uses U1 or U3 x const.	#6,#8	All	All
Bq	$Bq = \frac{dU}{Qt - sv}$	# 4 # 8 # 13	All	All
OS (Overburden Stress)	OS = sv = S (Gamma x Depth)		All	All
EOS (Effective Overburden Stress)	EOS = sv' = OS - Uo = sv - Uo		All	All
Rf Zone Soil Behavior Type see NOTE #5	Classification chart for Qc and Rf Zone # = Soil Behavior Type 1=sensitive fine grained 2=organic material 3=clay 4=silty clay 5=clayey silt 6=sandy silt 7=silty sand 8=fine sand 9=sand 10=gravelly sand 11=very stiff fine grained ¥ 12=sand to clayey sand ¥ ¥ overconsolidated or cemented	#6 #8, Fig4.3	All	1<Qt<1000bar 0<Rf<8%

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Bq Zone Soil Behavior Type	Classification chart for Qc and Bq (same zone #'s as Rf above)	#8 Fig 4.3	All	0<Qt<1000bar -0.1<Bq<1.4
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After R&C(1983) see NOTE #6	Qt/N ratio per zone Zone # 1, Qt/N 2; Zone # 7, Qt/N 3 Zone # 2, Qt/N 1; Zone # 8, Qt/N 4 Zone # 3, Qt/N 1; Zone # 9, Qt/N 5 Zone # 4, Qt/N 1.5; Zone # 10, Qt/N 6 Zone # 5, Qt/N 2; Zone # 11, Qt/N 1 Zone # 6, Qt/N 2.5; Zone # 12, Qt/N 2	# 7 # 8 Fig 4.2	All	All
Spt N1(60) Normalized for Overburden str	Spt N1(60) = Cn x Spt N(60) where Cn = (sv')^(-0.77)	# 8	All	0.5<Cn<1.5
Dr Relative Density see NOTE #7	Specific Sands: $Dr = \frac{100}{C2} * \ln\left[\frac{Qc}{C0\ sv'^{C1}}\right]$ where: All are NC & UNAGED	# 8		
Compressibility	Sand; C0; C1; C2			
moderate	Ticino; 17.37; .558; 2.58	# 1	/ Sand-- \	7 to 10 0<Qt<500bar 0<sv'<5bar
high	Schmertmann; 15.32; .520; 2.75	# 1		
all	ALL SANDS: NC, OC, ALL TESTS $Dr=C3 + C4\log_{10}\left[\frac{\left[\frac{Qc}{C1}\right]}{\left[\frac{sv'+C2}{C0}\right]}\right]$ where: C0 0.100; C1 0.0981; C2 0.5; C3 -98; C4 66	# 5	Sand	7 to 10 (6 possible)
Phi Friction Angle	Methods: 1) Robertson & Campanella 2) Durgunoglu & Mitchell 3) Janbu beta = +15 degree 4) Janbu beta = 0 degree 5) Janbu beta = -15 degree	#6, #8 # 2 #6, #8 #6, #8 #6, #8	/ Sand-- \	7 to 10 & 6 0<Qt<500bar 0<sv'<4bar 29<phi<49

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Gmax Maximum Shear Modulus at very small strains	Clay: Gmax = alpha x Qt Sand: Digitized figure of Qc vs Gmax with interpolation between sv'curves,R&C method	# 8 Fig4.18 # 6 # 8 Fig4.13	Clay Sand	1 to 6 (6 possible) 7 to 10 .25<sv'<8bar
CSR(Qc), t/s LEVEL ground Liquefaction SAND Resistance see NOTE #8	Seed's CSR vs N1(60) graph for specified equake Magnitude.Can include silty sand corr. for Zone 7. N1(60) from CPT correlations.	# 11 # 12	Sand	7 to 10 (6 possible)
CSR(Eq), t/s Cyclic Stress Ratio applied by design quake	$CSR(Eq) = 0.65 \frac{Amax}{g} \frac{sv}{svo'} rd$ Amax=max surface acceleratn including Amplification	# 12 # 3	Sand	7 to 10 (6 possible)
[Note: Input value from input file is used if defined, & not calculated]				
rd Reduction Factor to find CSR(Eq)	Digitized graph to use for depth vs rd: 1) Seed's mean 2) Fraser Delta	# 12 # 3	Sand	(6 possible) 7 to 10 0<depth<30m
FL,Safety Factor against Liquefaction	FL = CSR(Qc)/CSR(Eq)	# 3	Sand	7 to 10 (6 possible)
Qcr Critical Bearng required to resist Liquefctn	Qcr backcalculated from CSR(Eq) for a specified FL. Qcr is only for the given GWT,EOS,OS,Amax/g & Eq.Mag	# 12	Sand	7 to 10 (6 possible)
Su, Undrained Shear Strength of CLAY METHODS:	Nk: $Su = \frac{Qc - st}{Nk}$	# 8	Clay	1 to 6
	Nke: $Su = \frac{Qt - U2}{Nke}$		Clay	1 to 6
	Nkt: $Su = \frac{Qt - sv}{Nkt}$		Clay	1 to 6
	Nc: $Su = \frac{Qt}{Nc}$		Clay	1 to 6
see NOTE #9	NdU: $Su = \frac{dU2}{NdU}$ (dU1 or dU3)		Clay	1 to 6

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Su/EOS	$Su/EOS = \frac{Su}{sv'}$	# 8	Clay	1 to 6
Ko (NC) Normally Consolidated	$(Ko)NC = 1 - Sin(f)$ see NOTE #10	# 8	Sand	7 to 10 (6 possible)
Ko (OC) Over Consolidated	$(Ko)OC = (Ko)NC \times OCR^{0.42}$	# 8	Sand	7 to 10 (6 possible)
E25 Youngs Modulus	E25 = alpha x Qt where user input alpha	# 8 4.11&12	Sand	(6) 7 to 10 0<Qt<500bar
M Constrained Modulus	CLAY: M = alpha x Qt where user input alpha	# 8 Tabl4.3	Clay	1 to 6
	SAND: Methods: Qt: M = alpha x Qt		Sand	7 to 10 (6 possible)
	Baldi: $\frac{M}{Qt} = C0 \times pa \left(\frac{sv'}{pa}\right)^{C1} \times OCR^{C2} \times \exp(C3\ Dr)$	# 8 Fig4.10	Sand	7 to 10
OCR (Clay) Over-Consolidation Ratio see NOTE #11	$OCR = \left[\frac{\frac{Su}{svo'}}{\left(\frac{Su}{svo'}\right)_{NC}}\right]^{1.25}$	# 6 # 8 Fig4.19	Clay	1 to 6
Ic Material Index After J&D(1993) see NOTE #18	$Ic = \left[\left(3-\log_{10}(Q(1-Bq))\right)^{2} + \left(1.5+1.3\log_{10} F\right)^{2}\right]^{0.5}$	# 13 # 17	All	All
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After J&D(1993) see NOTE #16	Qc/N = 8.5(1-(Ic/4.75)) where Qc in bars	# 13	All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
State Parameter State, (e-units) Current Void Void Ratio minus Critical Void Ratio	$\text{State} = \dfrac{\ln\left[\dfrac{3M + 8.5M/F}{Q(1-Bq)}\right]}{11.9 - 1.33F}$ $M = \dfrac{6 \text{ Sin fcv}}{3 - \text{Sin fcv}}$ fcv = const. vol. Phi angle	# 14	All	All
Fines Content FC(%) Percent less than #200 Sieve After Davies,99	FC(%) = 42.4179(Ic) - 54.8574 FC(%) = 0% if Ic < 1.2933 FC(%) = 100% if Ic > 3.6508	# 15	All	All
OCR (Clay) Overcons. Ratio by Pore Press. U1 & U2 or U1 & U3 see NOTE #17	OCR = 0.5 + 1.50(PPD) PPD = (U1 - U2)/Uo or PPD = (U1 - U3)/Uo and default 0.5 & 1.5 are settable	# 16	Clay	1 to 6

1. Depth averaging may be in 0.5, 1, 2.5 or 5 ft. intervals or 0.1, 0.25, 0.5 or 1.0 m intervals, or no depth averaging if zero is selected. The average is the mean value of the readings in the interval. The depth value is the mid-depth of the averaged interval. It is convenient to start at half the depth averaging interval. For example, if you want "even" depths and the depth averaging is set at 0.50 m then start at 0.25 to get values of depth of 0.5, 1.0, 1.5, etc.

2. Basic input CPTU data columns are for Depth, Qc, Fs, U1, U2, U3, INC and TEMP may be selected. In addition the following parameters may also be specified as an INPUT data column: Qt, Gamma, Uo, Spt N, Rf Zone, Bq Zone and CSR(EQ). These values will be used where required to obtain other interpreted parameters. If they are not specified the program will estimate them when they are required. For example, you can create an OUTPUT data file of any of the above parameters and then edit some or all of the values to suite your measurements or your desires to specify their values. You can do that with "Gamma" values to input your measurements of unit weight, or with "Uo" if you want to input values of pore water pressure other than hydrostatic, or with any of the other input parameters. You would use your edited file of adjusted data as your new INPUT data file. Thus, you can specify these parameters if you want to override the Program's values.

You can also use the designated value of "9E9" to denote an unknown value.

You can use the "OTHER" designation to input other data that exists on your input file and identify its units. This allows you to output it, without operating on it, if you choose.

It is best NOT to use depth averaging when using input data that is not continuous at regular depth intervals. Always use DEPTH AVERAGING with extreme caution since the program averages ALL INPUT parameters over the interval chosen irregardless of soil type. Careful use of start and end depth choises can make depth averaging very effective.

3. Since there is no data in the file within the initial depth interval, a default Gamma (unit weight) must be specified from the surface to the starting depth. This is done in the "Param" Menu in units of kN/m^3 (1kN/m^3=6.36pcf). Also, you can specify the values of Gamma to be used by the program as in NOTE #2 above.

4. If pore pressures are not measured by the cone then the program will take Qc as being equal to Qt for all interpretations requiring Qt. Also, Uo may be specified in the input file as a column of Uo vs depth values, if the water pressures are not hydrostatic. See NOTE #2 for more info on customizing input data.

5. You can choose to use either the Rf classif. Zone or the Bq classif. Zone to divide soil into Undrained Parameters (Zones 1 to 6) and Drained Parameters (Zones 7 to 10) in the "Param" Menu. (However, in order to use the Bq Zone you must have Pore Pressure, U2, data.) Also, you may choose to switch Zone 6 to a Drained Zone from its Undrained Zone status. This is done if you feel that the soil identified as Zone 6 (sandy silt) is really coaser (using other sources of information) and/or you want it analyzed as a Drained rather than Undrained soil. Finally, the soil behavior names in each zone were shortened in version 5.0 for simplicity. For example, Zone 6 was named "sandy silt to clayey silt" but was shortened to "sandy silt".

6. Spt N is the same as Spt N(60) for 60% transferred energy. This value is calculated from the Qt/N ratios given for each Soil Zone (you can specify either Rf or Bq Zone) and these values are used in the Level Ground Liquefaction analysis. Values of Spt N may be specified in the Input File, if indepedently measured values are to be used. We suggest that you not use depth averaging if you only have selected Spt N values at a few depths. You may use "9E9" for missing data.

7. If Dr values are negative then soil is very loose or likely more of an undrained soil like a silty sand rather than a drained soil for which the Dr correlations were developed. Use Dr interpretations very cautiously since they also assume the soil is free draining, uncemented, unaged and has the same compressibility of grains as the soil used for the correlations in chamber calibration tests.

8. The simplified sand liquefaction analysis for level ground according to Seed et al requires Spt N1(60) and earthquake magnitude to obtain the cyclic stress ratio to cause liquefaction, CSR(Qc). The design maximum ground acceleration, the depth-reduction factor, Rd, and overburden total and effective stresses are required to calculate the cyclic stress ratio applied by the design earthquake, CSR(EQ). The program estimates the N1(60) values from the cone stresses, the operator identifies the earthquake magnitude and Seed et al chart is used to get CSR(Qc). The program also calculates CSR(EQ) from the user specified maximum ground acceleration including any amplification factors, the calculated overburden stresses and either Seed's mean or the Fraser Delta Rd factor. The Fraser Delta is used only when amplification factors of the order of 2 or more are used. See Reference Nos. 3, 6, 11 and 12 for more information. The user can INPUT specific values for Spt N, CSR(EQ), Soil Zones, Gamma's, etc. in order to customize the analysis for the existing data base of information. It is recommended that you do not use depth averaging when using specific input data but make calculations at specific depths where external input data exists. The calculated value of Qcr is the minimum value of cone bearing stress required at a given depth such that the factor of safety against liquefaction, or the ratio FL = CSR(Qc)/CSR(EQ) have the specified value for a given earthquake magnitude, max. ground acceleration, depth reduction factor, and calculated overburden stresses. This value of Qcr is useful to identify the required minimum level of soil improvement for a given design condition.

9. The NdU method to calculate undrained shear strength has been extended to allow the user to choose either dU1, or dU2 or dU3 provided such pore pressure measurements exist.

10. The Overconsolidation Ratio, OCR, for the sand must be estimated by the user in the "Param" menu if you want to estimate Ko in the sand layers. For the typical normally consolidated sand, OCR = 1.0.

11. It is currently only possible to estimate the OCR for a clay, which makes use of the correlations obtained from extensive laboratory tests.

12. An improved calculation and print routine was added to version 5.0 which uses swap routines to reduce memory requirements, but slows down the calculations.

13. The classification charts for Rf has been extended at all boundaries such that values of Rf>8 and values of Qc<1.00 are possible. The Bq classification chart which requires dU2 and can now accept values of Bq>1.2 and Qt<1. Unfortunately, this feature does not work.

14. Version 5.1ppd added several enhancements to the program. You may input an average vertical flow gradient, which is applied over the entire profile depth to be analysed so adjust the depth of interest accordingly. Zero gives hydrostatic and no flow, a negative gradient is upward flow which increases pore pressure and reduces vertical effective stress. A positive gradient gives downward flow.

15. A State Parameter or current void ratio minus critical void ratio is calculated according to the paper by Ref. 14, Plewes, Davies and Jefferies, 1994.

16. An alternate method to estimate SPT from CPT is provided according to Ref. 13, Jefferies and Davies, 1993 in ASTM.

17. An alternate method to estimate OCR in clays is provided which uses the measured pore pressure difference, ppd, so both U1 and U2 or U1 and U3 must be measured at the same time. (see Ref. 16)

18. Version 5.2 added the value Ic (Material Index) according to Jefferies & Davies, 1993, 1991 (Ref. 13 & 17) which combines all Normalized parameters Q, F and Bq.
(Note: QtN was changed to Q and RfN to F.)

18A. In Version 5.2, if at any depth the value of Bq>1 (in very sensitive saturated soil)then Bq is made equal to 0.99. Also, if Rf>8 it is made 7.99. These changes have a negligable effect on the results.

19. FC(%) or percent of dry weight less than #200 sieve (.074mm) was also added according to Davies, 1999 Ref.#15)

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2) Durgunoglu, H.T. and Mitchell, J.K., 1975, "Static Penetration Resistance of Soils: I-Analysis", Proceedings of the ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Raleigh, NC

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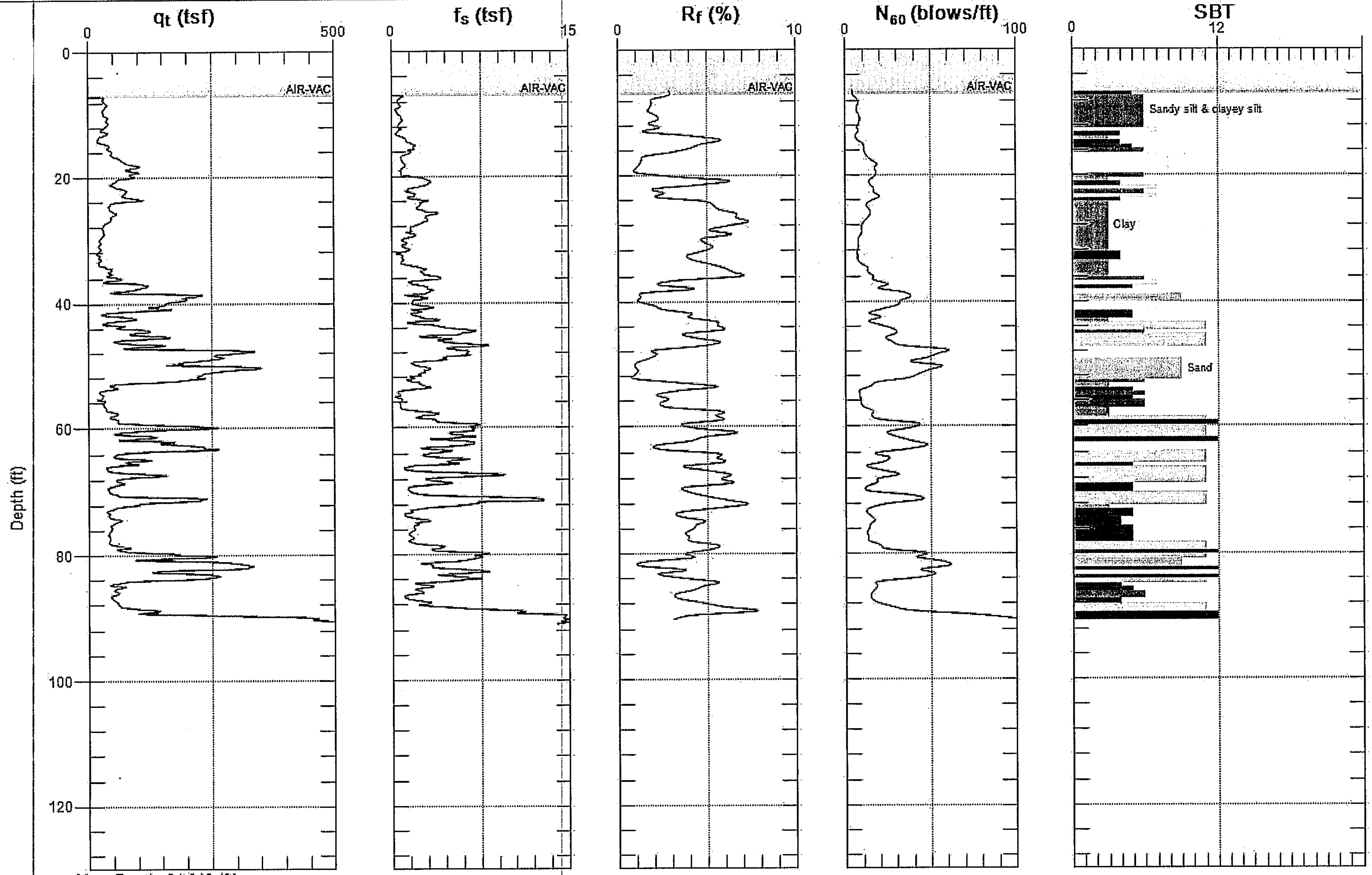
GREGG **MACTEC**

PRIOR MACTEC INVESTIGATION
(PROJECT No. 4088-08-7537)

Site: MELROSE TRIANGLE
Sounding: CPT-01

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 08:05



Max. Depth: 91.043 (ft)
Avg. Interval: 0.656 (ft)

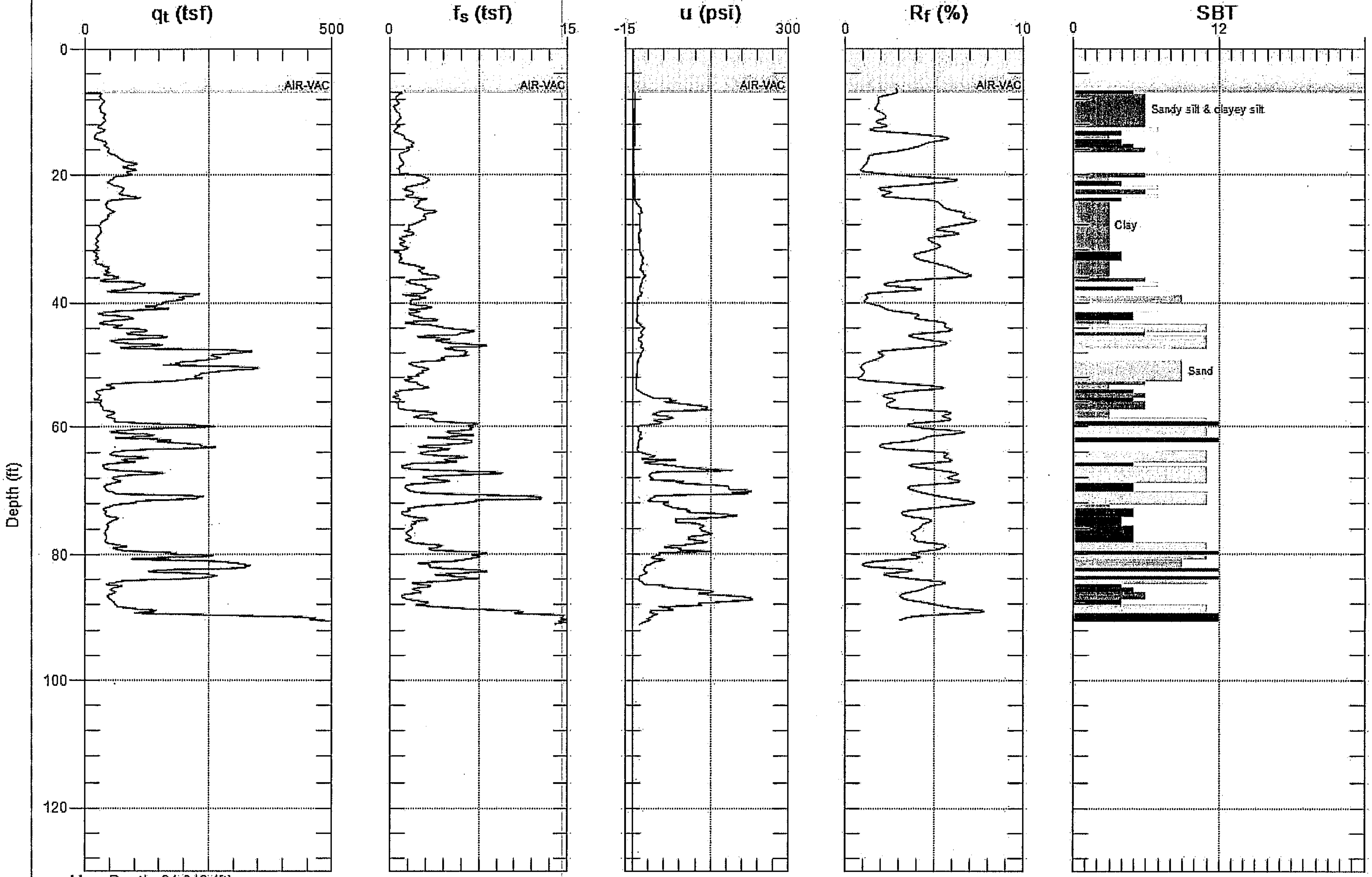
SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

PRIOR MACTEC INVESTIGATION
PROJECT NO. 4088-08-7537

Site: MELROSE TRIANGLE
Sounding: CPT-01

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 08:05



Max. Depth: 91.043 (ft)
Avg. Interval: 0.656 (ft)

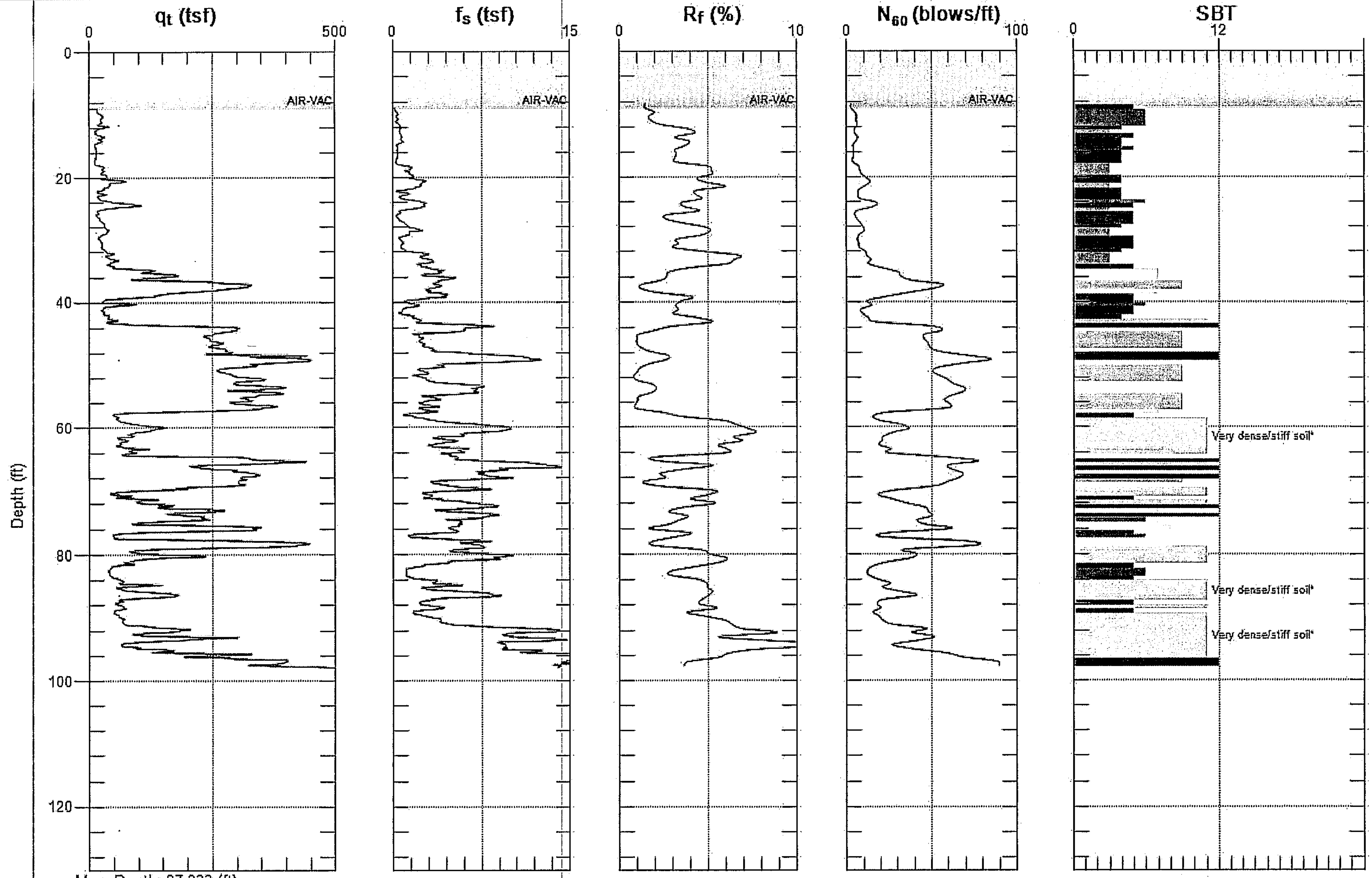
SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

PRIOR MACTEC INVESTIGATION
PROJECT No 4088-08·7538

Site: MELROSE TRIANGLE
Sounding: CPT-02

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 11:18



Max. Depth: 97.933 (ft)
Avg. Interval: 0.656 (ft)

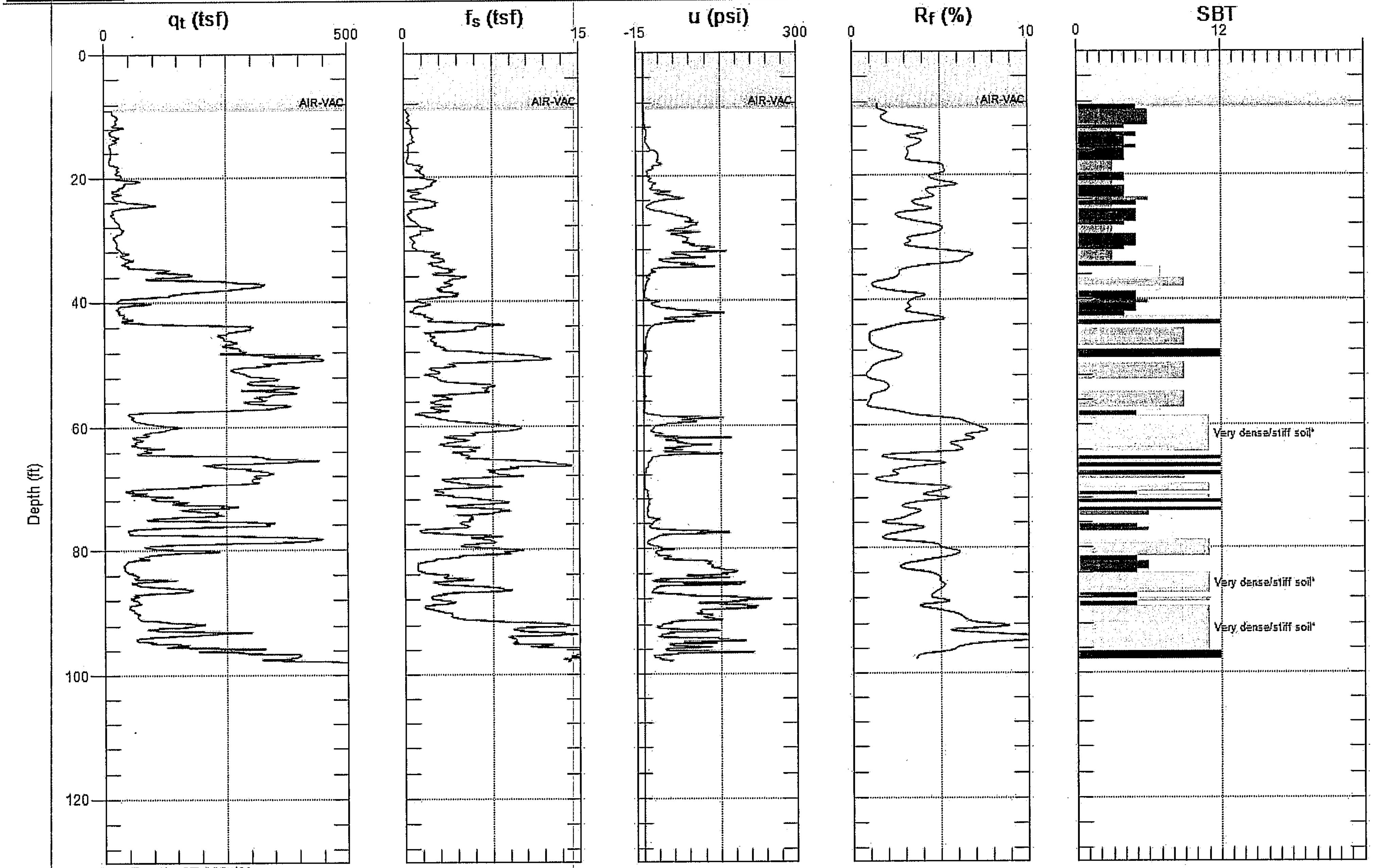
SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

PRIOR MACTEC INVESIGATION
PROJECT NO 4088-08-7537

Site: MELROSE TRIANGLE
Sounding: CPT-02

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 11:18



Max. Depth: 97.933 (ft)
Avg. Interval: 0.656 (ft)

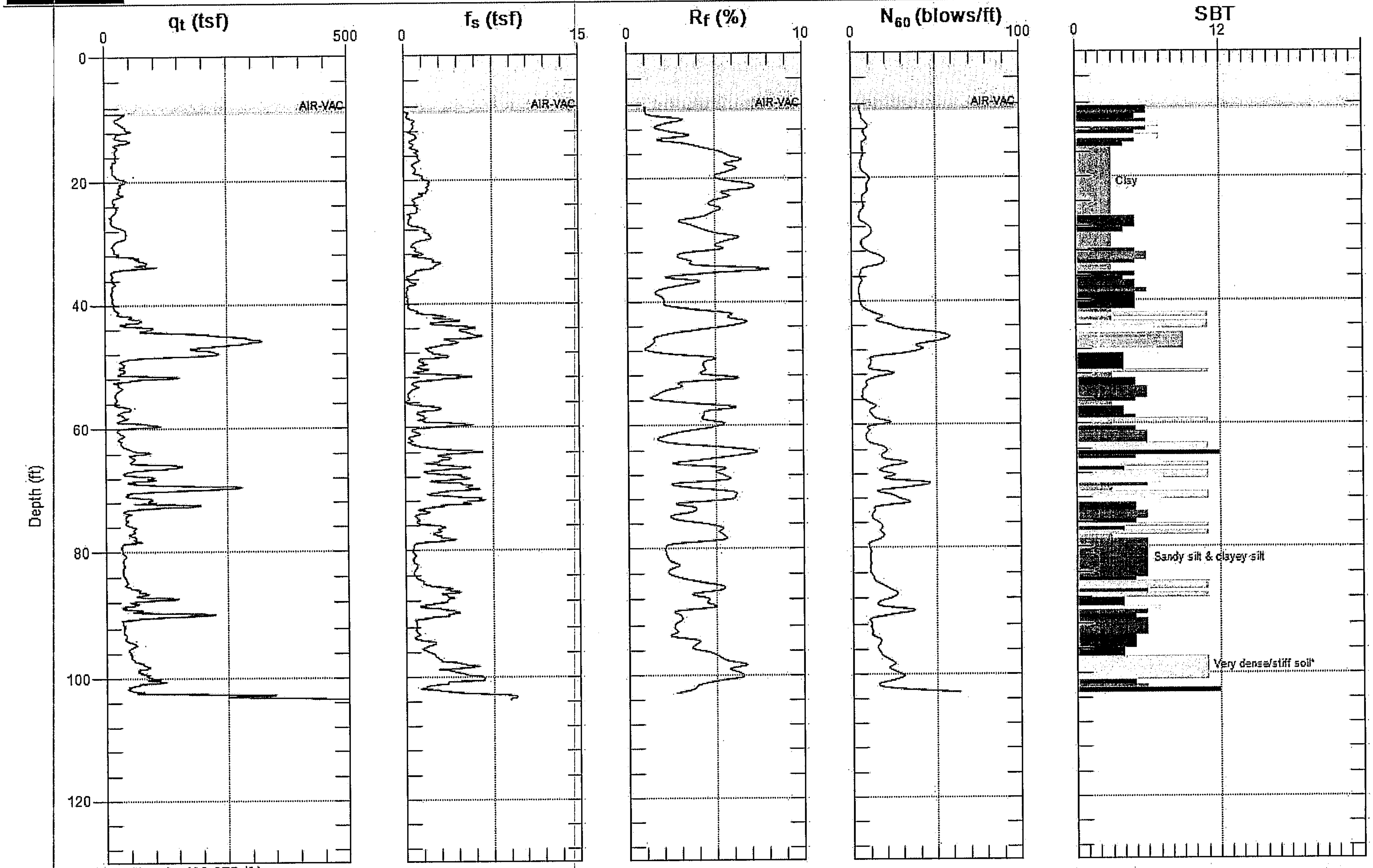
SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

PRIOR MACTEC INVESTIGATION
PROJECT No. 4088-08-2537

Site: MELROSE TRIANGLE
Sounding: CPT-03

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 10:25



Max. Depth: 103.675 (ft)
Avg. Interval: 0.656 (ft)

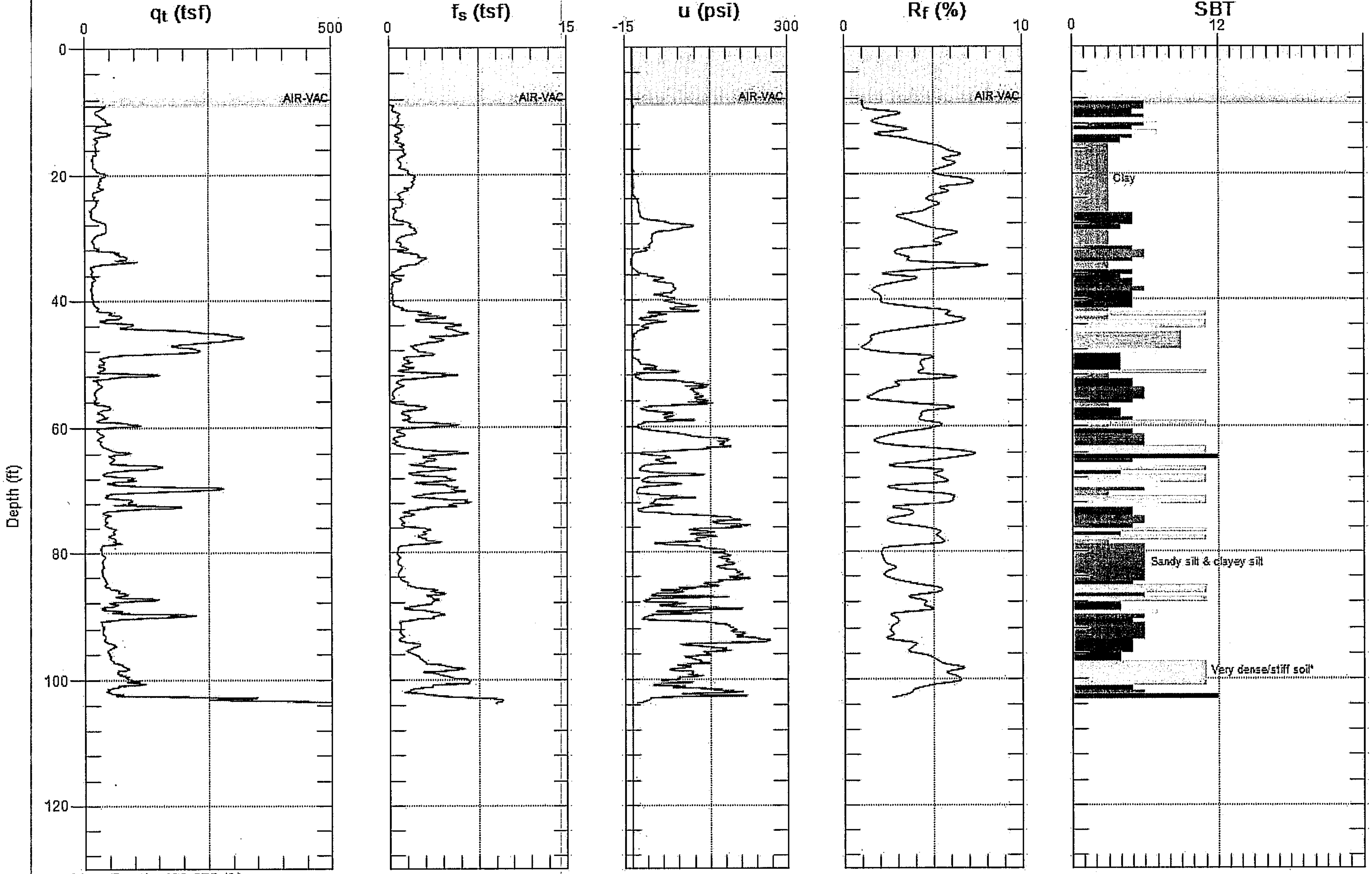
SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

PRIOR MACTEC INVESTIGATION
PROJECT NO. 4088-08-7537

Site: MELROSE TRIANGLE
Sounding: CPT-03

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 10:25



Max. Depth: 103.675 (ft)
Avg. Interval: 0.656 (ft)

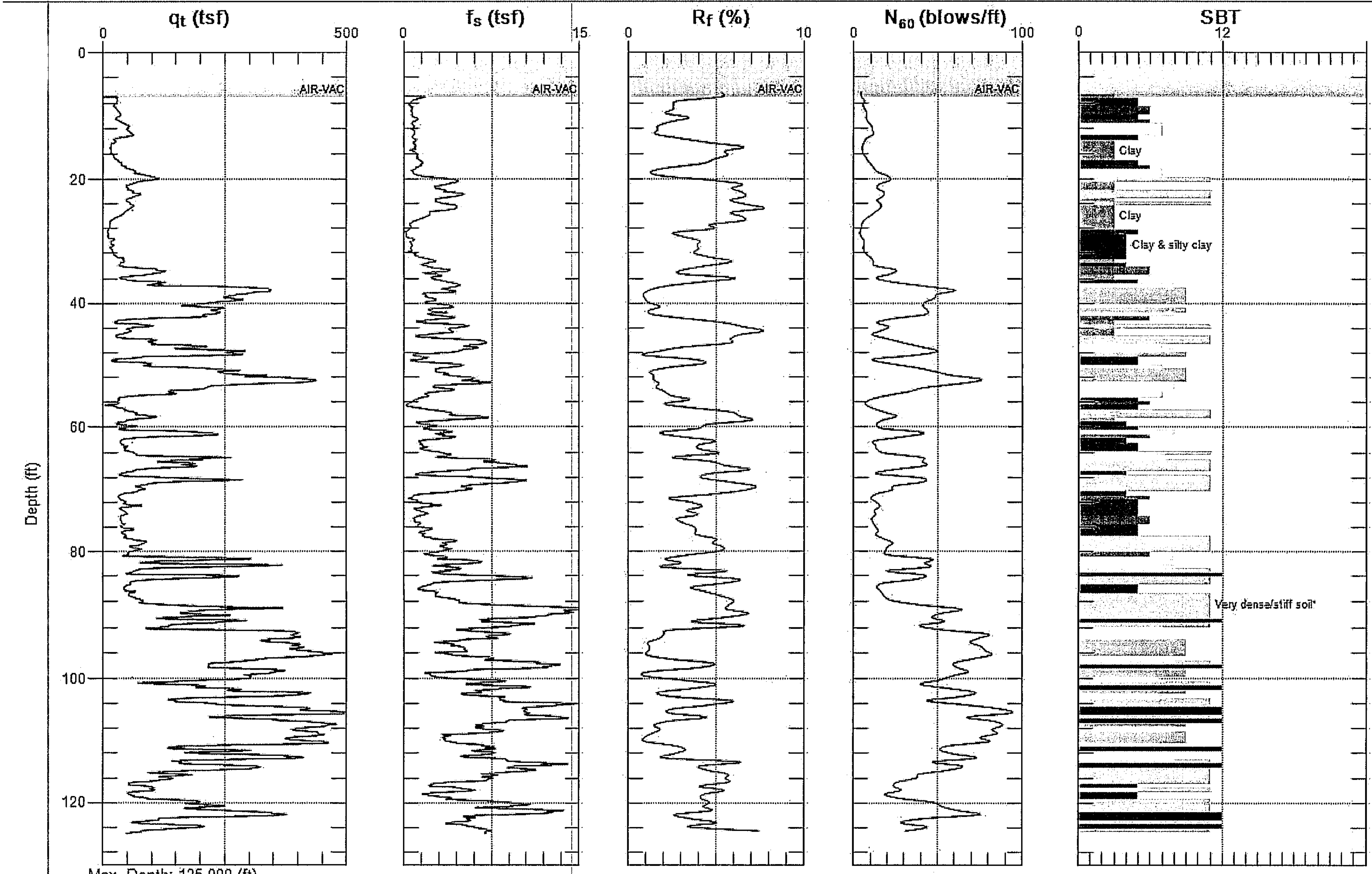
SBT: Soil Behavior Type (Robertson 1990)

MACTEC

PRIOR MACTEC INVESTIGATION
PROJECT No. 4088-08-7537

Site: MELROSE TRIANGLE
Sounding: CPT-04

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 09:08



Max. Depth: 125.000 (ft)
Avg. Interval: 0.656 (ft)

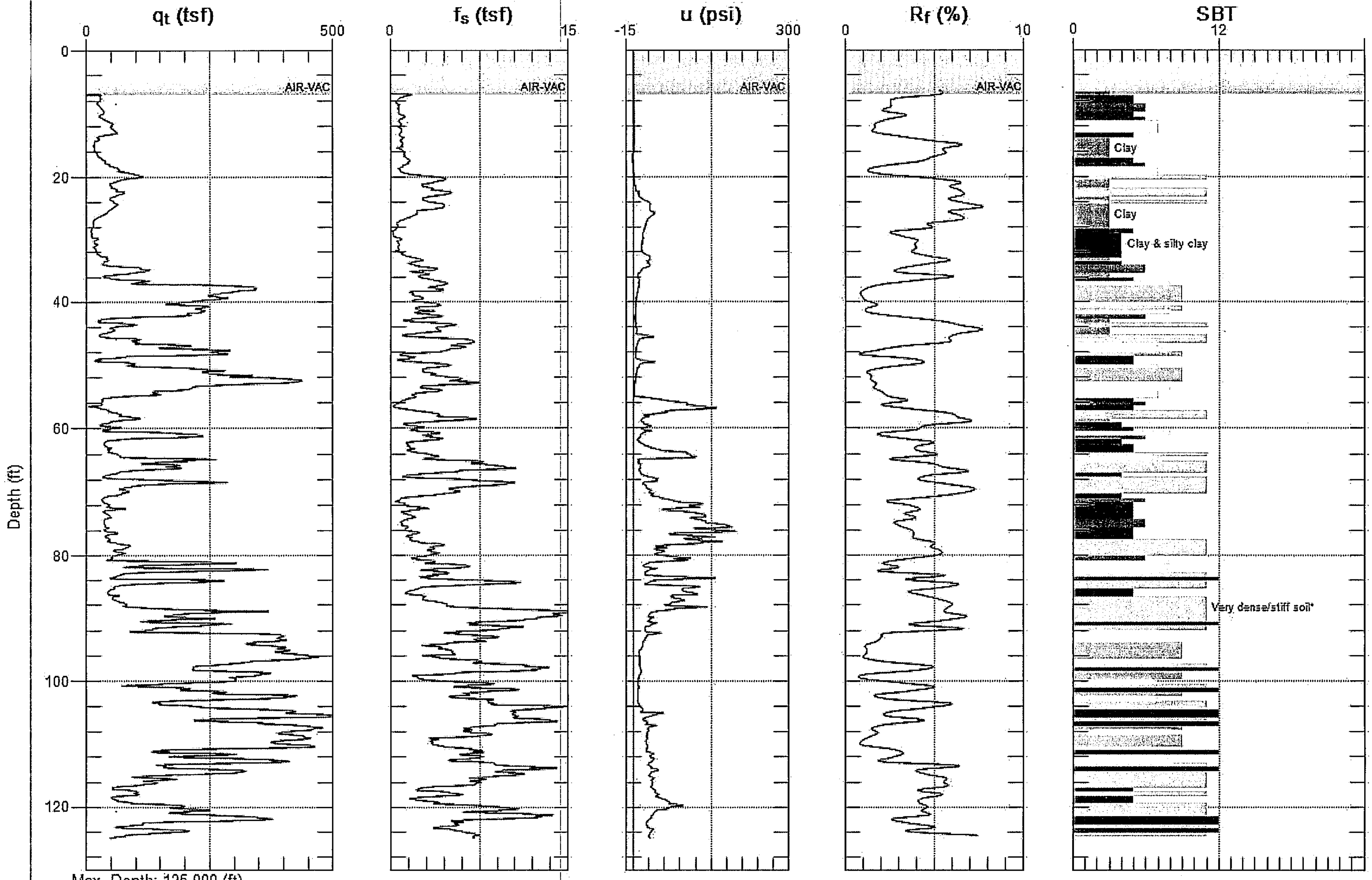
SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Prior MACTEC Investigation
Project No. 4088-08-7537

Site: MELROSE TRIANGLE
Sounding: CPT-04

Engineer: W.CHAMBERLAIN
Date: 8/19/2008 09:08



Max. Depth: 125.000 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX B

PRIOR EXPLORATIONS AND LABORATORY TESTS BY LeROY CRANDALL AND ASSOCIATES (1985)

APPENDIX B

PRIOR EXPLORATIONS AND LABORATORY TESTS BY LeROY CRANDALL AND ASSOCIATES (1985)

Subsurface information was available from two of the prior borings from a previous investigation performed at the site (Proposed Development, site bounded by Santa Monica Boulevard, Melrose Avenue, and Almont Street; Project No. A-85280). The borings were drilled with a 5-inch-diameter rotary-wash type drilling equipment to a depth of 75 feet below the ground surface. The logs of the borings and applicable laboratory tests results are presented herein.

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.

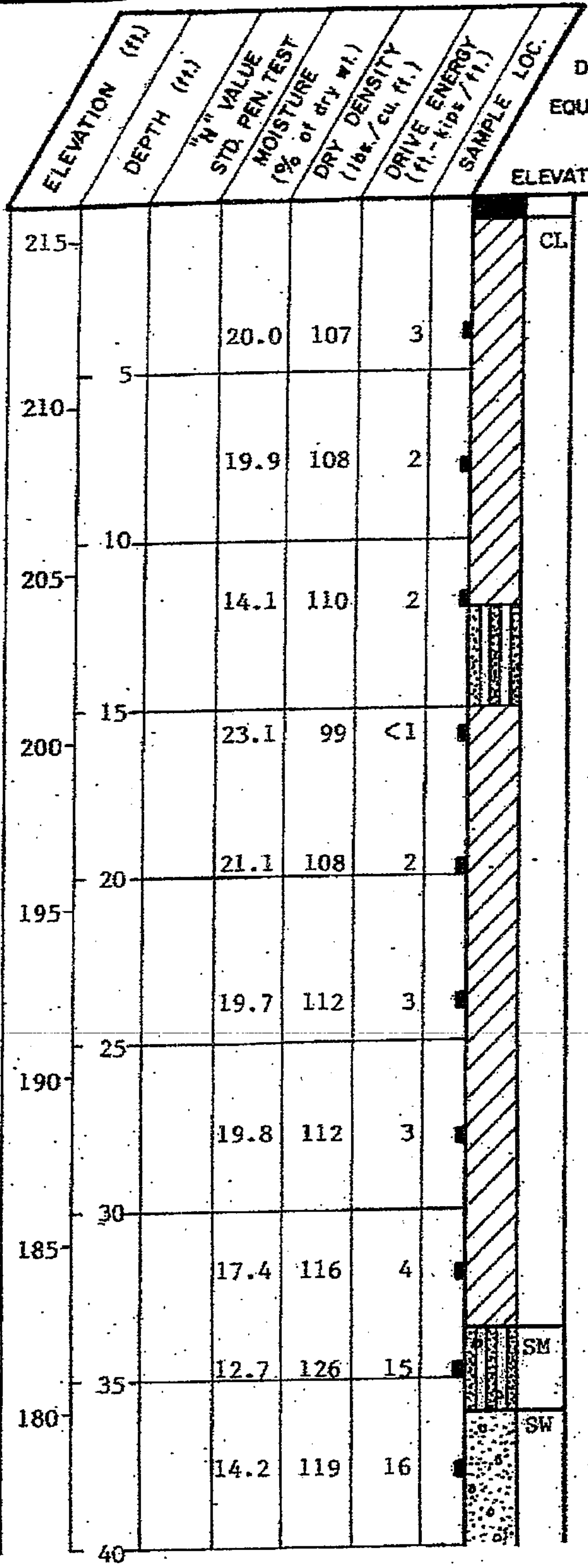
Form 123 JOB A-85280 DATE 8-13-85 DR. SANDY ID MD W.P. dmk CHKD

BORING 1

DATE DRILLED: August 8, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 215.9*



"N" VALUE STD. PEN. TEST	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	DRIVE ENERGY (ft.-kips/ft.)
	20.0	107	3
	19.9	108	2
	14.1	110	2
	23.1	99	<1
	21.1	108	2
	19.7	112	3
	19.8	112	3
	17.4	116	4
	12.7	126	15
	14.2	119	16

6" Asphaltic Paving

SANDY CLAY - brown

Layer of Silty Sand, brown

Brownish-grey

*Elevations refer to datum of reference boundary survey; see Plate 1 for location and elevation of bench mark.

SILTY SAND - well graded, some Clay, few gravel, brown

SAND - well graded, some gravel, grey

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

FIGURE B-1

BORING 1 (CONTINUED)

DATE DRILLED: August 8, 1985
EQUIPMENT USED: 5"-Diameter Rotary Wash

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.		
			14.4	117	13	■		
175							CL	SANDY CLAY - brown
			26.3	102	6	■		
	45							
170								
			22.7	108	5	■		
	50							
165							ML	CLAYEY SILT - brown
			24.8	102	9	■		
	55						ML	SANDY SILT - brown
160								
							CL	SANDY CLAY - brown
			17.1	117	6	■		
	60							
155								
							SM	SILTY SAND - fine, some Clay, gravel, brown
			11.6	127	19	■		
	65							
150								
			13.5	126	21	■		
	70							
145								
	75		11.5	127	18	■		

NOTES:

1) Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 19' 7 days after removal of mud.

2) Bucket boring drilled to 21½' adjacent to Boring 1 on 8/10/85. Water seepage encountered at 19½'. Water level measured at 20½' at completion of drilling and at 19' 5 minutes later.

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 124 JOB A-85280 DATE 8-13-85 DR. SANDY C.E. MD W.P. lmh CHKD

LOG OF BORING

LeROY CRANDALL AND ASSOCIATES

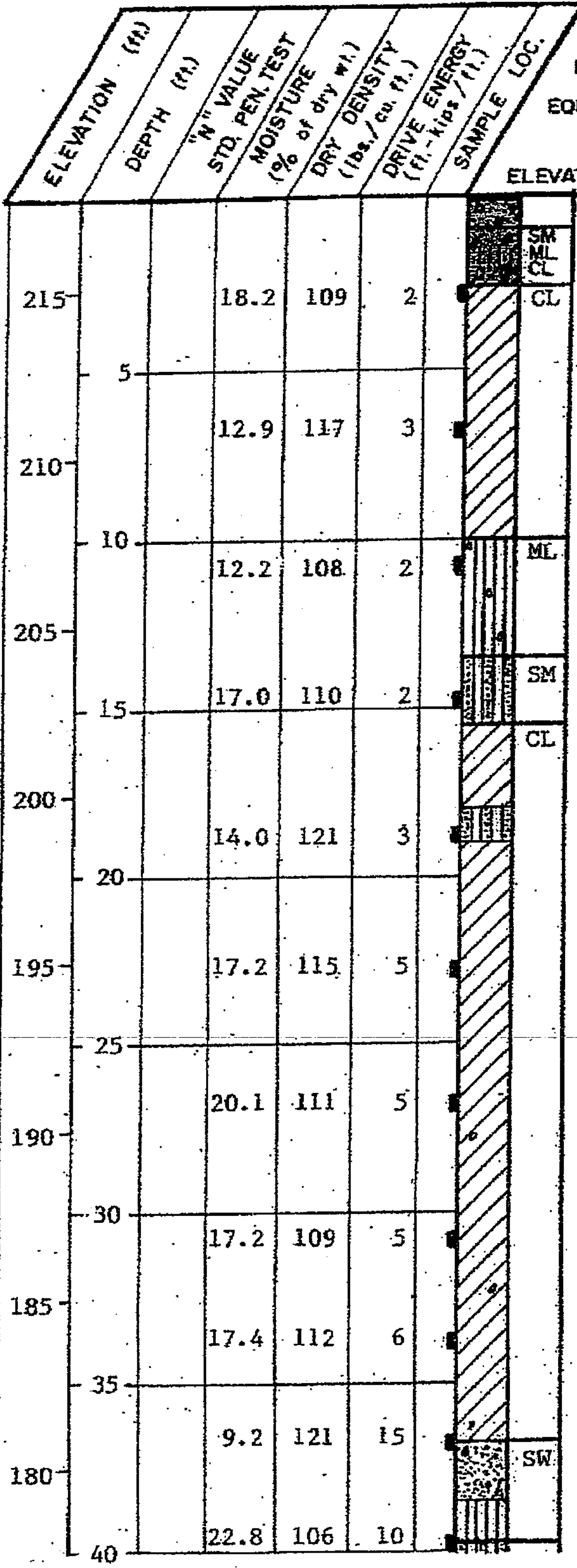
FIGURE B-2

BORING 2

DATE DRILLED: August 9, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash

ELEVATION 217.6



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.
			18.2	109	2	
			12.9	117	3	
			12.2	108	2	
			17.0	110	2	
			14.0	121	3	
			17.2	115	5	
			20.1	111	5	
			17.2	109	5	
			17.4	112	6	
			9.2	121	15	
			22.8	106	10	

1" Asphaltic Paving Over 9" Concrete Slab

SM ML CL — FILL - SAND, SILT and CLAY - pieces of wood, brick and concrete, brown

CL — SANDY CLAY - dark brown

Reddish-brown

ML — SANDY SILT - some Clay, gravel, brown

SM — SILTY SAND - fine, some Clay, brown

CL — SANDY CLAY - brown

Layer of Silty Sand

Few gravel

SW — SAND - well graded, some gravel, grey

Layer of Sandy Silt

(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 A-85280 DATE 8-13-85 DR. SANDY D.E. ID W.P. MD dmh CHKD

LeROY CRANDALL AND ASSOCIATES

FIGURE B-3

BORING 2 (CONTINUED)

DATE DRILLED: August 9, 1985

EQUIPMENT USED: 5"-Diameter Rotary Wash

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.
175			13.3	119	11	■
	45		14.2	122	9	■
170			9.4	123	16	■
	50					
165			11.6	127	27	■
	55					
160			17.9	105	11	■
	60					
155			14.7	124	15	■
	65					
150			20.4	106	11	■
	70					
145			17.7	116	8	■
	75					

SM	SILTY SAND - fine, brown
CL	SANDY CLAY - some gravel, brown
SW	SAND - well graded, some gravel, grey
CL	SANDY CLAY - brown
ML	SANDY SILT - some gravel, greyish-brown
SM	SILTY SAND - fine, brown
CL	SANDY CLAY - grey

NOTES:

1) Drilling mud used in drilling process. Mud removed at completion of drilling. Water level measured at 20' 6 days after removal of mud.

2) Bucket boring drilled to 21½' adjacent to Boring 2 on 8/10/85. Water seepage encountered at 20½'. Traces of water at bottom of boring at completion of drilling. Water level measured at 20½' 5 minutes after completion of drilling.

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-4

Form 124 JOB A-85280 DATE 8-13-85 DR. Sandy MD W.P. dmh CHKD

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 gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. 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 amt. 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 or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays 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.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 ¾ in. 3 in. (12 in.)

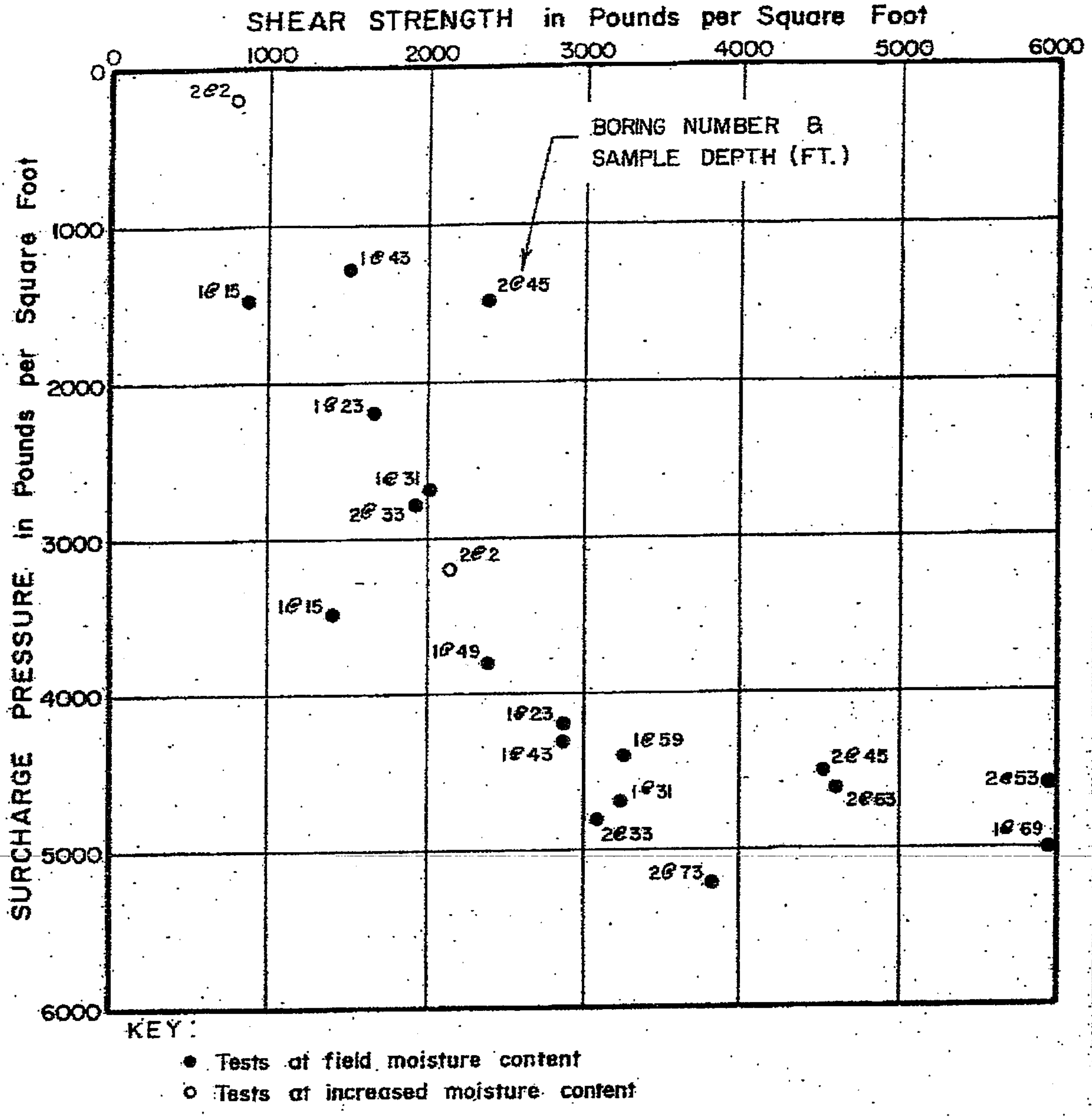
U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

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

LEROY CRANDALL & ASSOCIATES

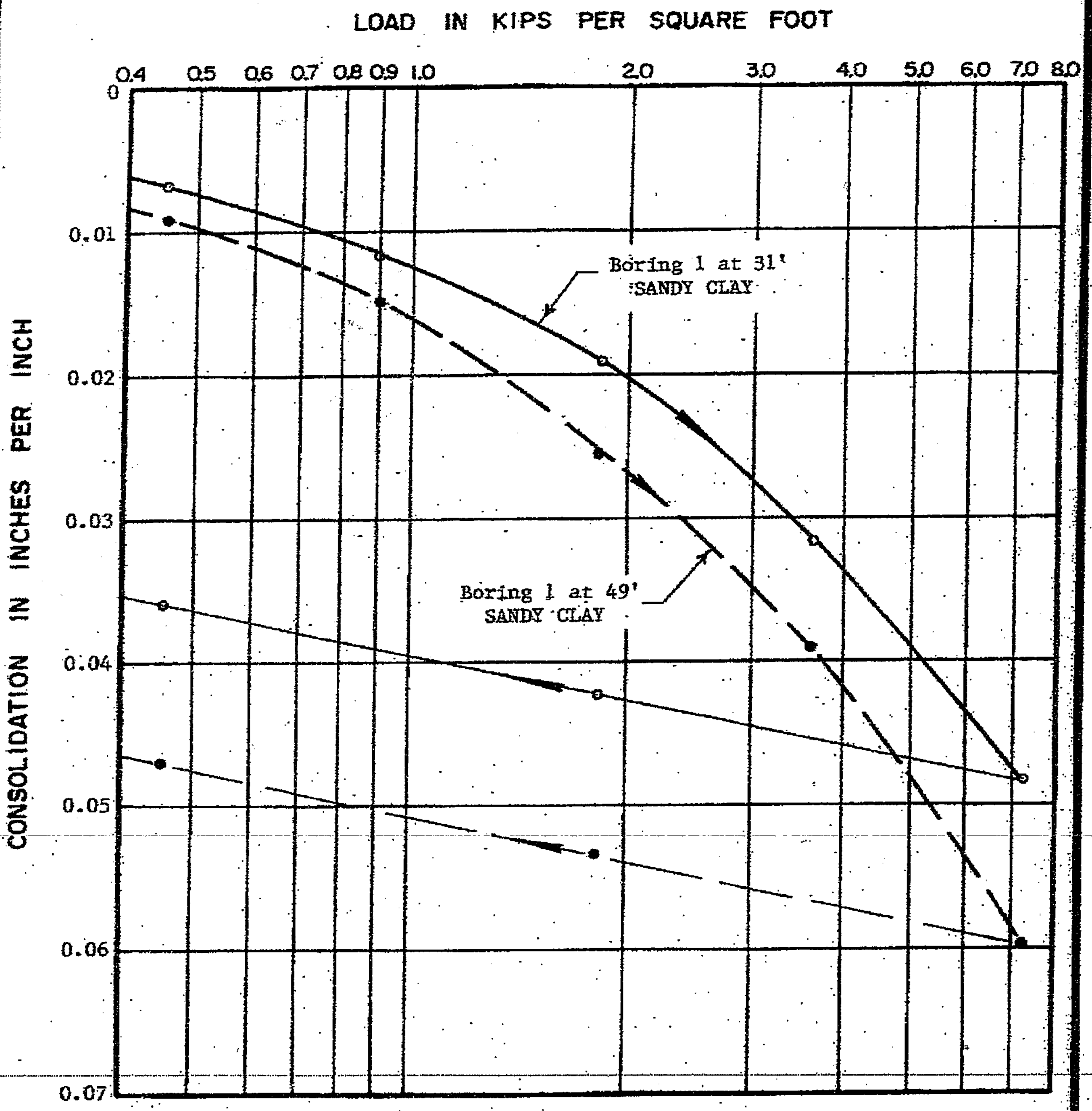
FIGURE B-5



DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

FIGURE B-6



NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL AND ASSOCIATES

FIGURE B-7

FORM 116 JOB A-85230 DATE 8/23/85 DR. C. MD W.P. mh CHKD