Appendix IS-2

Geotechnical Evaluation Report

CITY OF LOS ANGELES
CALIFORNIA

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EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

April 20, 2017

LOG # 97576
SOILS/GEOLOGY FILE - 2

6436 Hollywood Blvd., LLC
40 West 57th Street, 23 FL
New York, New York 10019

TRACT:	HOLLYWOOD (MR 28-59/60)
BLOCK:	14
LOTS:	1 (Arbs. 2 & 3), 2 (Arbs. 2 & 3), FR 3, FR 4, FR 5 (Arb. 1), FR 6 (Arb. 1), FR 15 (Arb. 2), FR 16 (Arb. 1)
LOCATION:	6430, 6432, 6434, 6438, 6440, & 6436 W. Hollywood Boulevard and 1624, 1626, 1628, 1634, 1638, 1640, 1642, 1644, 1646, & 1648 N. Wilcox Avenue

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Geology/Soils Report	LA-01670-01	10/07/2016	Earth Systems
Infiltration Report	LA-01670-02	10/10/2016	Earth Systems

The Grading Division of the Department of Building and Safety has reviewed the referenced reports that provide recommendations for the proposed demolition of all site structures except a 2-story structure that will be preserved and renovated, a 1-story structure, and a 17-story (15-above, 2-below grade) mixed use building with retaining walls. The earth materials at the subsurface exploration locations consist of up to 5 feet of uncertified fill underlain by alluvium. The consultants recommend to support the proposed 1-story structure on conventional foundations bearing on a blanket of properly placed fill a minimum of 3 feet thick and the proposed 17-story tower on mat-type foundations bearing on native undisturbed soils.

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

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2017 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Whenever the principal building on a site is added to, altered or repaired in excess of 50 percent of its replacement value, the entire site shall be brought up to the current Code standard. (7005.9).

   If this condition applies, a supplemental report identifying all non-conforming conditions shall be provided with recommendations to bring the entire site into conformance with the current Code standard. This shall include but not to be limited to regrading and/or retaining of steep slopes and underpinning/replacement of all existing foundations where not in conformance with current Code standards.

2. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans that clearly indicates the geologist and soils engineer have reviewed the plans prepared by the design engineer; and, that the plans include the recommendations contained in their reports (7006.1).

3. An on-site storm water infiltration system at the subject site shall not be implemented, as recommended.

4. All recommendations of the reports that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

5. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

6. A grading permit shall be obtained for all structural fill and retaining wall backfill (106.1.2).

7. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

8. If import soils are used, no footings shall be poured until the soils engineer has submitted a compaction report containing in-place shear test data and settlement data to the Grading Division of the Department; and, obtained approval (7008.2).

9. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).

10. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

11. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

12. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring, as recommended. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

13. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

14. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit (3307.3.2).

15. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

16. Unsurcharged temporary excavation may be cut vertical up to 5 feet. For excavations over 5 feet, the lower 5 feet may be cut vertically and the portion of the excavation above 5 feet shall be trimmed back at a gradient not exceeding 1.5H:1V, as recommended.

17. Shoring shall be designed for the lateral earth pressures specified in the section titled "C. Temporary Shoring" starting on page 16 of the 10/07/2016 report; all surcharge loads shall be included into the design.

18. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

19. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

20. All foundations shall derive entire support from native undisturbed soils or a blanket of properly placed fill a minimum of 3 feet thick, as recommended and approved by the soils engineer by inspection.

21. The structural designer and soils engineer shall verify and attest to the adequacy of the existing footings for underpinning by signature and license stamp, on the final plans.

22. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

23. Slabs placed on approved compacted fill shall be at least 3½ inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced a maximum of 16 inches on center each way.

24. The seismic design shall be based on a Site Class D, as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

25. Retaining walls shall be designed for the lateral earth pressures specified in the section titled "G.1 Retaining Walls" starting on page 24 of the 10/07/2016 report. All surcharge loads shall be included into the design.

26. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions as specified on section titled "G.2 Retaining Walls" of the 10/07/2017 report (1803.5.12).

27. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

28. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall

be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

29. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

30. Basement walls and floors shall be waterproofed/damp-proofed with an LA City approved "Below-grade" waterproofing/damp-proofing material with a research report number (104.2.6).

31. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

32. The structure shall be connected to the public sewer system per P/BC 2014-027.

33. All roof, pad and deck drainage shall be conducted to the street in an acceptable manner; water shall not be dispersed on to descending slopes without specific approval from the Grading Division and the consulting geologist and soils engineer (7013.10).

34. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

35. Any recommendations prepared by the geologist and/or the soils engineer for correction of geological hazards found during grading shall be submitted to the Grading Division of the Department for approval prior to use in the field (7008.2, 7008.3).

36. The geologist and soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading (7008 & 1705.6).

37. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

38. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; underpinning; protection fences; and, dust and traffic control will be scheduled (108.9.1).

39. Installation of shoring, underpinning, slot cutting excavations and/or pile installation shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6).

40. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whichever is more restrictive. Research Report #23835

41. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction

report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

42. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

CASEY LEE JENSEN
Engineering Geologist Associate II

DAN RYAN EVANGELISTA
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cc: Earth Systems, Project Consultant
LA District Office

PRELIMINARY
GEOTECHNICAL ENGINEERING REPORT
Proposed Mixed-Use Development
6430-6440 Hollywood Boulevard and
1624-1648 Wilcox Avenue
Hollywood, California
LA-01670-01

Prepared For

6436 HOLLYWOOD BLVD., LLC

October 7, 2016

Prepared By

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October 7, 2016

LA-01670-01

6436 Hollywood Blvd., LLC
40 West 57th Street, 23 FL
New York, New York 10019

Attention: Mr. David Twerdun

Subject: **Preliminary Geotechnical Engineering Report**
Proposed Mixed-Use Development
6430-6440 Hollywood Boulevard and
1624-1648 Wilcox Avenue
Hollywood, California

Presented herewith is the Preliminary Geotechnical Engineering Report prepared, as authorized, for the site of a proposed mixed-use development in the Hollywood community of the City of Los Angeles, California. The conclusions and recommendations contained in this report are based upon Earth Systems' understanding of the proposed development and on analyses of the data obtained from the field and laboratory testing programs.

The recommendations provided in this report generally pertain to criteria for site grading, shoring and foundation design. Earth Systems strives to provide analyses and recommendations in accordance with the applicable standards of care for the geotechnical engineering profession at the time the study is conducted. The submittal of this report marks the completion of the scope of geotechnical engineering services described in Earth Systems' proposal dated May 17, 2016 and authorized on June 1, 2016 for the geotechnical phase. Other services which may be required, such as grading observation and construction testing, are additional services that will be billed according to the Fee Schedule in effect at the time such services are provided. Budgets for these services, which are dependent upon design and construction schedules, can be provided when requested. Earth Systems appreciates this opportunity to provide professional geotechnical engineering services for this project. If you need clarification of the information contained in this report, or if Earth Systems can be of additional service, please contact the undersigned.

Respectfully submitted,
Earth Systems

Christopher F. Allen, E.G.
Project Engineering Geologist

Distribution:

3 – Addressee (hard copy including one unbound copy)
1 – Addressee (CD, pdf copy)

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HOLLYWOOD, CALIFORNIA

INTRODUCTION

This Preliminary Geotechnical Engineering Report has been prepared for the site of a proposed mixed-used development. The purpose of this study was to evaluate the geotechnical engineering characteristics of the on-site subsurface soils relative to the anticipated construction.

This report includes:

1. Descriptions of the field exploration and laboratory tests performed.

2. Evaluation of liquefaction potential and earthquake-induced subsidence of soils beneath the site.

3. Conclusions and recommendations relating to construction of the proposed fourteen-story mixed use facility based upon analyses of data obtained from the exploration and testing programs, and on knowledge of the general and site specific characteristics of the subsurface soils.

SITE DESCRIPTION

The approximate 1.42-acre site is at the southeast intersection of Hollywood Boulevard and Wilcox Avenue in the Hollywood community of the City of Los Angeles, California. The site is approximately one-third of a mile southwest of the Hollywood (hwy-101) freeway and south of the Cahuanga Pass (see Plates I through III). The project site is comprised of multiple parcels currently occupied by one- and two-story buildings and an asphalt covered parking lot (Site Exploration Map, Plate IV). Topographically, the property consists of relatively flat ground at an elevation of approximately 380 to 386 feet above mean sea level. Surface drainage is directed toward the southwest via sheet flow. The above-cited descriptions are intended to be illustrative, and are specifically not intended for use as a legal description of the subject property.

The following table summarizes the individual lots comprising the area of development.

Assessor Parcel	Street Address	Legal Description per LADBS
5546-007-001	6436, 6438 & 6440 W. Hollywood Blvd. 1646 & 1648 N. Wilcox Ave.	Tract: Hollywood Portion of Lots: 1 and 2 Blk: 14, Arb: 3
5546-007-002	6430, 6432 & 6434 W. Hollywood Blvd	Tract: Hollywood Portion of Lots: 1 and 2 Blk: 14, Arb: 2
5546-007-007	No Address	Tract: Hollywood Portion of Lot: 16 Blk: 14, Arb: 1
5546-007-029	1634, 1636, 1638, 1640, 1642 & 1644 N. Wilcox Ave.	Tract: Hollywood Portion of Lots: 3, 4 & 15 Blk: 14
5546-007-030	1624, 1626 & 1628 N. Wilcox Ave.	Tract: Hollywood Portion of Lots: 5 and 6 Blk: 14, Arb: 1

PROJECT DESCRIPTION

Based on information provided by members of the design team, Earth Systems understands that the existing two-story structure located on the southeast corner of Hollywood Boulevard and Wilcox Avenue will be preserved and renovated, while all other structures will be demolished. A 15-story building with two levels of subterranean parking is proposed for the lots fronting on Wilcox Avenue and a new single-story building is proposed on the second lot east of Wilcox Avenue, along Hollywood Boulevard.

Excavations for the subterranean parking garage will not exceed 40 feet in depth below existing grade. Conventional shoring and excavation techniques will be used during the construction of the subterranean garage. Sewage disposal will be provided by a public sewer system. Earth Systems has not received foundation plans as of this writing; however, column loads for the main 15-story tower are anticipated to be approximately 2,200 kips based on the type of construction and previous experience with similar type of structures. In considering loadings for a mat type foundation, an allowable "net" bearing capacity of 2,350 pounds per square foot (psf) was assumed for the loads being distributed over the full footprint of the foundation. These assumptions were used as the basis for the exploration, testing, and analyses programs, and for the recommendations contained in this report. If the anticipated foundation loads or other site conditions vary significantly from the values stated herein, the recommendations should be reconfirmed prior to completing project plans.

PURPOSE AND SCOPE OF SERVICES

The purpose of Earth Systems' services was to evaluate the project site soil conditions, and to provide preliminary geotechnical engineering conclusions and recommendations relative to the project site and the proposed construction. Earth Systems' scope of services included the following:

A. A general reconnaissance of the site and review of previous geotechnical reports for the site

B. Shallow subsurface exploration of the project site by drilling five hollow-stem auger test borings and advancing 22 cone penetration test (CPT) soundings.

C. Performing a seismic shear-wave survey at the subject site.

D. Geotechnical laboratory testing of selected soil samples obtained from the exploration program conducted for this project.

E. Geotechnical engineering analyses of the data obtained from the exploration and testing programs.

F. A summary of findings and recommendations in this written report.

Contained in this report are:

A. Discussions on local and site specific soil conditions.

B. Results of laboratory tests and field data.

C. Evaluation of the potential static and seismic induced settlements.

D. Recommendations relating to the proposed mixed-use development, including allowable foundation bearing capacity, recommendations for foundation design, estimated total and differential foundation settlements, site grading criteria, lateral earth pressures, soil expansion characteristics, and soil corrosion characteristics.

SITE HISTORY

According to the Los Angeles County Assessor, the existing on-site structures were constructed in the 1930's to 1940's. Earth Systems conducted research at the City of Los Angeles Department of Building and Safety (LADBS) archives for both the subject site lots and contiguous properties. Geotechnical reports were not available for the subject property; however, it is likely that previous structures have existed on the portions of the property currently covered with asphalt. Geotechnical reports for some of the contiguous properties were available. The following summarizes the most pertinent information from these reports as they relate to geotechnical concerns on the subject site.

1635 N. Cauenga Boulevard: Earth Systems' was able to find an available soils report for this site prepared by Pacific Soils Engineering Inc. with three subsequent addendum reports. These reports were prepared in support of a seven-story office structure with one level of subterranean parking which appears to be the building presently constructed on this site. The reports were prepared in 1981 and 1982. The consultant notes that underlying soils consist of clayey and silty sands that were likely to settle as a result of placed fill or building loads. A recommendation was made that the structure be supported on piles arranged in groups of four.

6417 Selma Avenue: This property is currently under construction but appears to be nearing completion. A soils report was obtained for this property written by GeoConcepts in 2013 with a subsequent addendum report, in support of a ten-story structure. The consultant notes that there is approximately three feet of fill on the site and that the proposed one-story of subterranean parking will bear in native soils. Native soils are described as moderately dense to dense silty sand to sand with firm to very firm strata of sandy clay. The consultant does not believe that liquefaction or dry sand settlement are a concern for this site and suggests a mat foundation be selected to transmit column loads into native soil.

6421 Selma Avenue: This property is currently under construction and is part of the 6417 Selma development. The geotechnical report for this site, written by GeoConcepts dated November 25, 2014, is in support of a six to seven level, mixed-use facility with one to three levels of subterranean parking. The report contains recommendations for a mat foundation along with cantilevered and tie-back shoring.

1622 N. Wilcox Avenue: This existing three-story hotel building borders the subject site along the south property line. Earth Systems' was able to find a soils report written in support of a remodel to part of the existing structure. The geotechnical consultant for this project did not perform geotechnical borings but utilized three test pits to depths up to four and a half feet and exposed the existing footings of the building in three locations. The building was observed to have a five foot deep basement with perimeter footings 36 to 50 inches deep and 30 inches in width.

FIELD EXPLORATION

The initial field exploration for this study was conducted in July of 2016 with additional explorations completed through August of 2016. Field exploration included drilling and sampling five (5) exploratory hollow-stem auger test borings to depths of approximately 50 to 90 feet below the existing ground surface. Additional exploration included three (3) continuous core borings to depths of approximately 50 feet and advancing 22 cone penetration test (CPT) soundings to depths of approximately 62.7 to 92.5 feet as part of a fault rupture evaluation at the site.

A seismic shear-wave survey was completed for the subject site by Terra Geosciences on August 20, 2016. The seismic shear-wave survey consisted of running two seismic lines. A copy of the report prepared by Terra Geosciences is presented in Appendix F.

The approximate locations of the exploratory test borings, CPT soundings, and seismic lines, as indicated on the attached Site Exploration Map (Plate IV), were determined by sighting and tape measuring from existing surrounding improvements. The locations of the borings, CPT soundings, and seismic lines should be considered accurate only to the degree implied by the measurement method used.

Bulk (disturbed) samples of the subsurface soils were obtained from tailings generated during drilling. These samples were secured for classification and testing purposes and represent a mixture of soils within the noted depths.

Additional soil samples ("ring samples") were secured from within the test borings using a 3-inch outside diameter ring sampler (ASTM D 3550) with a shoe similar to the drive cylinder sampler (ASTM D 2937). A 140-pound hammer falling approximately 30 inches (ASTM D 1586) drove the sampler. The hammer was operated by an automatic trip mechanism which operated at a rate of approximately 40 blows per minute. The number of blows required to drive the sampler 18 inches was recorded in six-inch increments and recorded on the boring logs. Recovered ring samples were sealed in plastic containers and transported to Earth Systems' laboratory for further classification and testing.

Further sampling and collection of disturbed soil samples was accomplished using a Standard Penetration Test (SPT) sampler in accordance with ASTM D 1586. The SPT sampler is a split barrel sampler with a 1-3/8 inch inside diameter. This sampler is also driven by a 140-pound hammer falling approximately 30 inches. The number of blows required to drive the sampler 18 inches was recorded in six-inch increments and recorded on the boring logs. Soil samples recovered by this method were sealed in plastic bags. Recovered soil samples were transported to the Earth Systems laboratory for further classification and testing.

The logs of test borings represent Earth Systems' interpretation of the field logs prepared for each test boring by Earth Systems' staff, along with their interpretation of soil conditions between samples and results of laboratory tests. The final boring logs and the log and interpretations of the CPT soundings, are included in Appendix A and Appendix B, respectively. While the noted stratification lines represent approximate boundaries between soil types, the actual transitions may be gradual.

LABORATORY TESTING

After visual and tactile classification in the field, the soil samples were brought to Earth Systems' laboratory. Soil samples were classified visually in accordance with the Unified Soil Classification System and the following tests were conducted:

A. Moisture content and dry unit weight for soil ring samples were evaluated (ASTM D 2937 and ASTM D 2216).

B. Soil grain size was evaluated using Particle Size Analysis: Mechanical Method and Hydrometer Method (ASTM D 422).

C. Atterberg Limits tests (ASTM D 4318) were conducted to obtain liquid limit (LL) and Plasticity Index (PI).

D. The relative strength characteristics of selected ring samples were estimated from the results of direct shear tests (ASTM D 3080). The specimens were placed in contact with water for at least 24 hours before testing and then sheared under normal loads ranging from approximately 1 to 4 kips per square foot (ksf). Samples were sheared to sufficient strains so that both peak and ultimate values were evaluated.

E. Consolidation tests (ASTM D 2435) were conducted on selected soil ring samples. The maximum stress during testing was 19.2 ksf. The sample were saturated at loads from 3.2 ksf to 6.4 ksf to check the hydroconsolidation potential. The samples were unloaded to 0.8 ksf to check the rebound characteristics.

F. Expansion potential was evaluated using the expansion index (EI) test (ASTM D 4829).

G. Compaction characteristics were evaluated using the "modified Proctor" (Maximum Density-Optimum Moisture) test (ASTM D 1557).

H. Soil chemistry tests consisted of pH, resistivity, conductivity, and a variety of cations and anions including soluble sulfate. Soil chemistry tests were performed by HDR-Schiff on a soil sample provided by Earth Systems.

Refer to Appendix C for the laboratory test results. Presentation of the test results provides only that information considered pertinent. References to ASTM and other test standards refer to the standard currently in effect.

GEOLOGIC SETTING

Regional Geology

The project site is located on the La Brea Plain, an area of coalescing young alluvial fans that emanate from the south flank of the Hollywood Hills, which are the eastern extension of the Santa Monica Mountains. Regionally, the project site is located at the boundary of the Peninsular Ranges and Transverse Ranges geomorphic provinces.

The Peninsular Ranges geomorphic province is characterized by elongated northwest-southeast trending geologic structures such as the nearby Newport-Inglewood fault zone. In contrast, the Transverse Ranges geomorphic province is characterized by east-west trending geologic structures such as the nearby Santa Monica fault, the Hollywood fault, and the Santa Monica Mountains. The Santa Monica and Hollywood faults are typically considered the boundary between the two geomorphic provinces in the project vicinity.

The distinctive geologic structure of the Transverse Ranges is dominated by the effects of north-south compressive deformation that has resulted in thrust faulting, strike-slip faulting and bedrock folding. These active geologic features are attributed to convergence resulting from the "Big Bend" of the San Andreas fault and the northwestern motion of the Pacific Plate, which have caused thrust fault related earthquakes such as the 1994 Northridge (Mw6.7), the 1971 San Fernando (Mw6.7), and the 1987 Whittier Narrows (Mw 6) earthquakes.

Regional Faulting

The Hollywood fault is the most significant geologic feature of the area. The projected surface trace of that fault is located as close as 700 feet north of the site. The California Geological Survey (CGS) has recently included the Hollywood fault in the Earthquake Fault (Alquist Priolo) Zoning program (CGS, 2014). This fault is considered part of the active Hollywood-Santa Monica-Raymond fault zone, a system of east trending reverse, oblique-slip, and left-lateral strike-slip faults that collectively form the southern boundary of the Transverse Ranges (Dolan, et al. 1997). Paleoseismic studies of the Hollywood and Santa Monica faults (Dolan et al., 1997, 2000a, and 2000b) suggest that these two faults have recurrence intervals of about 10,000 years, and that the Santa Monica fault last broke 1,000 to 3,000 years ago, while the Hollywood fault last ruptured 6,000 to 9,000 years ago. The Hollywood Fault Zone is considered capable of producing a Mw 6.6 earthquake if it ruptures by itself, and potentially larger if it fails with the adjacent Santa Monica or Raymond faults.

Other active or potentially active faults in the immediate vicinity of the site include the North Salt Lake fault, which forms the southern boundary of the Hollywood Basin approximately 0.8 miles south of the site, the Upper Elysian Park fault located approximately 2.1 miles east of the site, the San Vicente fault located approximately 2.7 miles south of the site and the Newport-Inglewood fault located approximately 5.2 miles southwest of the site. The site does not fall within a currently designated Earthquake Fault Rupture Hazard ("Alquist-Priolo") Zone as currently identified by CGS on the Earthquake Fault Zones Hollywood Quadrangle map dated 2014. Nor does the site fall within a liquefaction hazard zone or slope hazard zone as currently identified by CDMG on the Seismic Hazard Zones Map for the Hollywood Quadrangle dated 1999, see Plate III.

SOIL SUBSURFACE CONDITIONS

Artificial fill (af) soils were observed to mantle a majority of the site explored. The depth of fill observed ranged from approximately five feet at the location of borings B1, B3, and B5 to being negligable in boring B2. These fill soils were found to consist predominantly of loose silty sand and medium stiff silt (SM and ML soil types based upon the Unified Soil Classification System).

Alluvial deposits (Qa) were found to consist predominantly of loose to very dense silty sand, well graded sand, and clayey sand (SM, SW and SC soil types). Occasional layers of clay were also noted along with a gravel rich sand layer at depth. The logs of the test borings in Appendix A and CPT soundings in Appendix B contain more detailed descriptions of the soils encountered.

Based upon results of the Expansion Index (EI) Tests (ASTM D 4829) conducted for this investigation, the upper on-site soils are considered to have a very low expansion potential. Refer to Section H of the Recommendations section for explanations and recommendations for dealing with expansive soils.

Per ASCE 7-10 Table 20.3-1, the site should be classified as a stiff soil profile (Site Class D).

GROUNDWATER

Free groundwater was encountered at the site in the test borings at depths of approximately 90 feet below existing site grade. The Los Angeles County Department of Public Works (LACDPW) does not maintain a water monitoring well in close proximity to the site. Based on the Seismic Hazards report for the Hollywood Quadrangle (CDMG, 1998), the historic shallowest groundwater in the vicinity of the project site is over 80 feet deep. Fluctuations in groundwater levels may occur due to variations in rainfall, regional climate, and other factors.

DISCUSSION AND CONCLUSIONS

Seismic Design Parameters

The 2014 Los Angeles Building Code (2014 LABC) will be applicable to the proposed project. The building code includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were calculated by determining the jobsite coordinates, and entering them into the USGS Ground Motion Parameter Calculator generated values for Site Class B soils, then the calculated "short period" and "one second" spectral responses were input into a spreadsheet that adjusts for the actual site class of soils (Site Class D). The calculated 2013 CBC and ASCE 7-10 seismic parameters typically used for structural design are included in Appendix D and summarized in the table below.

Summary of Seismic Parameters – 2013 CBC

Site Class	(Table 20.3-1 of ASCE 7-10 with 2013 update)	D
<u>Maximum Considered Earthquake (MCE) Ground Motion</u>		
Spectral Response Acceleration, Short Period – S_s		2.506g
Spectral Response Acceleration at 1 sec. – S_1		0.937g
Site Coefficient – F_a		1.0
Site Coefficient – F_v		1.5
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}		2.506g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}		1.405g
<u>Design Earthquake Ground Motion</u>		
Short Period Spectral Response – S_{DS}		1.670g
One Second Spectral Response – S_{D1}		0.937g
Peak Ground Acceleration – PGA_M		0.976g
Reference: USGS, 2016, Latitude: 34.101N degrees Longitude: 118.331 W degrees		

Liquefaction

Liquefaction is defined as a loss of strength of saturated cohesionless soil caused by seismic shaking. Soil types most susceptible to liquefaction are loose, saturated silty to clean fine sands. The project site is not located within a defined liquefaction hazard zone as shown on the Hollywood Quadrangle (CDMG, 2014). Based on the groundwater depth encountered in the soil borings and the regional groundwater data provided by the LACDPW, groundwater beneath this site is in excess of 50 feet below the bottom of the proposed subterranean level of parking. It is generally accepted that liquefaction of soils in excess of 50 feet below a structure has a minimal impact on structures at the surface (CGS, 2008). Therefore, because of the lack of near-surface groundwater beneath the site, the potential for liquefaction-induced damage to structures at this site is considered negligible.

Seismic-Induced Settlement of Dry Sands

Dry sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. Procedures to evaluate this type of settlement were developed by Seed and Silver (1972) and later modified by Pyke, et al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998).

The blow counts obtained using the ring sampler were "converted" to an "equivalent" SPT blow count using a conversion factor of 0.63 (CGS 2008). The following correction factors were applied to both the SPT and ring sampler blow counts to arrive at "equivalent" $(N_1)_{60}$ values to be used in the analysis:

Item		Correction Factor Used
Energy Correction	C_E	1.5 – CME automatic trip hammer
Drive Rod Length	C_R	Varies with length of rod. Values used per SCEC.
Borehole Diameter	C_B	1.0 - Borehole Diameter – 4.5-inch diameter hole.
Sampler	C_S	1.2 – unlined samplers

Data from borings B2 and B4 were evaluated. Earth Systems' used the Tokimatsu and Seed procedure, as implemented by Pradel, to evaluate seismically induced settlement at this site. Based on the analysis performed with the above procedure, the seismic induced settlement of the dry sands was analyzed for an assumed groundwater level of 90 feet below existing site grade, a site peak ground acceleration equal to two-thirds of the PGA_M (0.651 g), and an earthquake magnitude of 6.48 Mw were used in this analysis. The predominant earthquake magnitude was derived from a probabilistic seismic hazard analysis run on the subject site with a return period of 475 years.

Calculations for seismically induced settlement of dry sands were analyzed for the soils between the existing ground surface and the assumed groundwater level of 90 feet, and the soils between the bottom of lower parking level and the assumed groundwater level of 90 feet below existing site grade. For the first case, calculations indicate that seismically induced settlement of dry sands could range from 1.8 to 2.7 inches. If the upper 5 feet of undocumented fill is removed and recompacted, seismically induced settlement of dry sands could range from 1.7 to 2.5 inches. Differential settlements as a result of a seismic event are anticipated to be approximately 1 inch.

Beneath the lower subterranean parking level of the main tower, calculations indicate that seismically induced settlement of dry sands could range from 1.0 to 1.4 inches. Differential settlements as a result of a seismic event are anticipated to be approximately 0.5 inch. This settlement is not significant and can be accounted for in the structural design of the tower.

Mitigation of Seismic Ground Movement Hazard

The structural foundation for the proposed high-rise building should be designed to accommodate the anticipated total and differential ground settlements and localized loss of ground support. To minimize the effects of differential ground movement on the proposed structure, Earth Systems recommends that the main 15-story tower be supported by a relatively rigid foundation system such as a structural mat slab or stiff post-tensioned slab. The proposed 1-story building on the north side of the main tower should be supported by a rigid foundation system (i.e., reinforced conventional spread footings tied together with tie-beams) underlain by a compacted engineered fill pad.

Other mitigation methods could also be considered to mitigate the seismic settlement hazard. Such methods might include ground improvement (grouting, stone columns, etc.) to reduce the susceptibility of the soil to liquefaction, or deep pile foundations that extend down to firm soils below the relative loose soils within the top 70 feet of soil profile. Such mitigation measures are likely to be more expensive than the rigid foundation system approach recommended herein. However, recommendations for such alternate measures can be provided if requested.

Site Grading

As mentioned in the Soil Conditions Section, artificial fill soils are present within the project site to depths of approximately five feet below existing grade. To provide more firm uniform bearing for proposed structures fronting Hollywood Boulevard that do not utilize existing foundations it is recommended that the upper 5 feet of near surface soils with a minimum of 3 feet below proposed foundations be removed and recompacted. At least 2 feet of soil should be removed and recompacted beneath all pavements and slabs. The bottom of all remedial excavation bottoms should expose firm native soils. Refer to Section A of the Recommendations of this report for more detailed discussions and recommendations regarding site preparation.

Temporary Excavations

As discussed in the Project Description section, temporary excavations up to 25 feet in depth are anticipated for construction of the subterranean parking levels. Excavations will have to be shored or "laid back" to appropriate temporary slope angles. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean levels will require shoring measures to provide a stable excavation.

In the deeper excavations where shoring is required, it is recommended that a tied-back and or internally braced, soldier pile and lagging shoring system be utilized. For shallow excavations requiring shoring, alternative shoring systems such as a cantilevered soldier pile design may be applicable. In addition, where the proposed excavation will be deeper than and adjacent to offsite structures and roadways, the proposed shoring should be designed to resist the surcharge imposed by vehicular traffic and the adjacent offsite structures. The depth and location of the subterranean levels on adjacent properties is not precisely known. Underpinning of adjacent structures may be required. Refer to Section C of the Recommendations section of this report for more detailed discussions and recommendations of shoring design.

Soil Compaction

Consideration should be given to the type of equipment to be used for compaction at the site. Different types of equipment are more effective with some soil types than with others. It should be understood that failure to provide the most appropriate equipment could result in inability to achieve the required degree of relative compaction, disturbance or displacement of subsequent and adjacent layers of fill, and/or the potential cost of removal of inadequately compacted fill that has been placed and subsequent delay in the grading progress.

Conventional compaction equipment (bulldozers, self-propelled or static or dynamic sheepsfoot compaction rollers, heavy rubber tired construction equipment, etc.) has a limited ability to consolidate layers of fill to the required density. Typically, loose layer thickness should not exceed 6-8 inches for heavy construction equipment and 2-4 inches for light manual equipment. Thicker layers of fill may be consolidated by utilizing specialized deep dynamic compaction; however this requires detailed geotechnical evaluation prior to being used.

Fine grained soils (clays and silts) typically should not be subjected to vibration or heavy widely distributed loads (such as smooth rollers or wide rubber tired construction equipment) during the compaction process, as this will cause an increase in the soil pore pressure resulting in 'pumping' or failure to consolidate the soil particles by expelling water and air. These soil types are best compacted by using a 'kneading' action (such as a 'sheepsfoot' compactor) or impact from a sharp blow on a small area (such as a dynamic or high speed tamping foot).

Cohesionless, non-plastic soils (sands and sand/gravel combinations) typically are best compacted using a wide, smooth, heavy static device (roller, wheels, tires etc.) or with the addition of vibration to these types of equipment. Narrow profile wheels, tires, etc., and high point load equipment typically will not perform as efficiently and will cause displacement and loosening of the adjacent soil.

Foundation Design and Settlements

The main 15-story tower building constructed over two levels of subterranean parking should be supported on a mat type foundation. Due to the relatively compressible nature of the site soils, the objective of the foundation design is to construct a compensated type foundation where the load removed during excavation of the subterranean parking garage is approximately equal to the building load applied during construction of the 15-story tower. Because a compensated type foundation is assumed for the main 15-story tower, the estimated static settlements discussed below are based on the depth of the soils removed and the average bearing pressure of the mat foundation. Earth Systems should have the opportunity to re-evaluate the anticipated static settlement once the final depth of the soils to be removed and the average bearing pressure of the mat foundation are known.

The proposed one- story structure fronting Hollywood Boulevard should be supported by a rigid foundation system (i.e., mat or "waffle" foundations, reinforced structural slabs or reinforced conventional spread footings tied together with tie-beams) underlain by a compacted engineered fill pad. Refer to Section E of the Recommendations section of this report for more detailed discussions and recommendations regarding conventional foundation design.

If the preliminary recommendations for foundation design and construction are followed, static settlement of the proposed 15-story tower should not exceed approximately two inches (2") in static settlement. This estimated settlement takes into account settlement that will occur due to reloading of the soils following excavation of the subterranean parking levels and settlement due to the building loads once the maximum past overburden pressure is exceeded. Differential settlement within a span of 100 feet can be assumed to be approximately 1 inch. Differential settlement between the new 15-story tower, the remodeled building, and the proposed one-story structure should be assumed to be equal to two inches (2"). It is recommended that foundations adjoining the new 15-story tower be designed as independent foundation systems.

The static settlement of the new, one-story structure can be assumed to be approximately one half an inch (0.5”) for spread or continuous footings founded in compacted fill as prepared in Section A of this report. Differential settlement of neighboring footings of varying loads, depths or sizes may be as high as fifty percent of the total settlement over a distance of 40 feet.

In addition to the static settlements discussed above, the seismically induced settlements during a strong seismic event will need to be considered during design of the foundations.

RECOMMENDATIONS

Based upon field exploration, laboratory testing, interpretation of the data, and past experience, the following recommendations should be incorporated into site preparation, design, and construction of the proposed mixed use facility.

A. Site Preparation

1. All vegetation, uncompacted fill, trash, pavements, abandoned underground utilities, and other debris should be removed from the proposed grading areas. Underground utilities (water, sewer, storm drain, electric, gas, cable, etc.) are anticipated within or adjacent to the proposed construction area. These utilities should be identified and relocated as required prior to performing excavations for any site grading or foundation excavations. All strippings and debris should be removed from the site in order to preclude their incorporation in site fill or remedial excavation backfill. Depressions resulting from such removals should have debris and loose soils removed and filled with suitable soils placed as recommended below.

2. Any seepage pits (i.e. cesspools), basements, underground tanks, or other similar substructures should be removed in their entirety including any liquids or sediment remaining at the bottom of the pits or tanks. Any brick, concrete or steel lining should be completely removed. The void resulting from removal of the pits or tanks should be backfilled with suitable soils placed as recommended below. This may require ramping and/or laying back side slopes to an angle to allow safe entry of personnel and equipment. Alternatively, deep shaft seepage pit excavations may be backfilled with a low-cement concrete slurry mix to within 5 feet of proposed final grade or proposed footing elevations. The final 5 feet should consist of compacted engineered fill as described below.

3. In order to minimize potential settlement problems associated with structures supported on a non-uniform thickness of compacted fill, the geotechnical engineers should be consulted for site grading recommendations relative to backfilling large and/or deep depressions resulting from removals under Item 1.

4. According to the City of Los Angeles Building Code, soils which contain less than 15% clay (< 15% finer than 0.005 mm) must be compacted to at least 95% of maximum dry density as determined by ASTM D 1557 test procedures. Any other soils may be compacted to 90% of maximum dry density. Based on the sieve and hydrometer test results, the upper soil at the site contains approximately 6 to 12% clay, therefore, the 95% compaction standard will apply. Additional sampling and testing of site soils or import soils can be conducted during grading to confirm or modify this finding.

5. To provide more firm uniform bearing conditions for the proposed at-grade structure foundations and slab-on-grade construction, Earth Systems recommends the following:

    a. Native soils and existing artificial fill should be excavated a minimum of 3 feet below the bottom of proposed footings, 5 feet below existing grade, or 5 feet below pad elevation, **whichever is deeper**. Remedial excavations should be performed to a distance of at least 5 feet laterally beyond the building perimeter, wherever possible. The base of the remedial excavation across the building pad should be a level elevation. Foundation plans and details should be checked carefully during grading to establish the actual bottom of footing elevations in the field.

    b. All existing fill within the proposed at-grade building areas (and traffic-bearing pavement areas) should be removed. The fill was observed as deep as 5 feet below the surface grade. It should be realized that deeper depths of fill material may be encountered at the time of grading.

    c. All exposed ground surfaces (subgrades) at the base of the remedial excavations should be firm, unyielding, and not excessively wet or excessively dry. If any of these conditions are not acceptable at the minimum recommended over-excavation depth, additional excavation will be required until suitable subgrade conditions are found. The density of the exposed ground may be tested and the "in-place dry density" ("IPD") of 85% relative compaction may be used as criteria for acceptable subgrade.

    d. The bottom of the remedial excavation should then be scarified (ripped) 6 inches and recompacted.

    e. The excavated soils may be reused to backfill the remedial excavations provided they are processed to remove any deleterious materials, debris, particles greater than 6 inches maximum dimension, and are properly moisture conditioned and compacted. During replacement of the excavated soils in the remedial excavations, and recompaction of the scarified soils, the soils should be moisture conditioned to near optimum moisture content and be uniformly compacted to at least 95% of maximum dry density as determined by ASTM D 1557 test procedures using mechanical compaction equipment. To aid in the compaction operation, fill should be placed in lifts not exceeding 6 inches compacted thickness**. Compaction should be verified by testing.**

f. The geotechnical consultant's representative should review the site grading prior to scarification of the bottom of the remedial excavation. Local variations in soil conditions may warrant increasing the depth of remedial excavation. Any deeper areas of loose soils should be removed and be replaced as compacted, engineered fill.

6. Soils beneath any exterior pavements and concrete flatwork, including a minimum lateral distance of at least 1 foot beyond pavement/flatwork edges, should be excavated a minimum of 2 feet below the existing grade or finished subgrade, **whichever is deeper**. All existing fill should be removed. The bottom of the remedial excavation should then be scarified (ripped) 6 inches. The scarified and excavated soils should be moisture conditioned to near optimum moisture content and be uniformly compacted to at least 95% of maximum dry density using mechanical compaction equipment. **Compaction should be verified by testing.**

7. Import soils should be equal to, or better than, the on-site soils in strength, expansion, compressibility, and soil chemistry characteristics. In general, import material should be free of organic matter and deleterious substances, have 100% passing a two inch sieve and an Expansion Index less than 20. Import soils can be evaluated prior to their use, but will not be prequalified by the geotechnical consultant. Approval of import soils will be given only after the material is on the project, either in-place, or stockpiled in adequate quantity to complete the project.

8. Suitable imported fill soils should be moisture conditioned to near optimum moisture content and be uniformly compacted to at least 90% of maximum dry density as determined by ASTM D 1557 test procedures using mechanical compaction equipment. To aid in the compaction operation, fill should be placed in lifts not exceeding six inches compacted thickness.

9. Backfill around or adjacent to confined areas (i.e. interior utility trench excavations, etc.) may be performed with a lean sand/cement slurry (aka "flowable fill" or "controlled low strength material -CLSM"). The fluidity and lift placement thickness of any such material should be controlled in order to prevent "floating" of any "submerged" structure. Certain narrow spaces such as some retaining wall back-cuts may be backfilled with gravel subject to approval by the geotechnical engineer and the City of Los Angeles. Gravel should be placed in lifts not exceeding 2 feet in thickness and vibrated or otherwise compacted to settle the gravel.

10. Roof drainage systems for the proposed structures should be designed so that runoff water is diverted away from any structure.

11. Final site grades should be designed and constructed so that all water is diverted away from all structures and not allowed to pond on or near pavement. Drainage devices should be constructed to divert drainage from the project site.

12. It is recommended that Earth Systems be retained to provide geotechnical engineering services during the grading, excavation, and foundation phases of the project. This continuity of services will allow for the geotechnical review of the design concepts and specifications relative to the recommendations of this report and will more readily allow for design changes in the event that subsurface conditions differ from those currently anticipated.

B. Excavations

1. Standard construction techniques should be sufficient for temporary site excavations. All excavations should be made in accordance with applicable regulations (including CAL/OSHA). Project safety is the responsibility of the contractor and the owner. Earth Systems will not be responsible for project safety.

2. Unshored, unsurcharged, open excavations may be cut vertically to a maximum height of no more than five feet. Excavations extending higher than five vertical feet should be sloped back above the 5-foot vertical cut to at least a one and a half horizontal to one vertical (1.5H:1V) slope or flatter. If excavations dry out, sloughing will occur. No excavation should be made within a 1:1 line projected outward from the toe of any existing footing or structure.

3. During the time excavations are open, no heavy grading equipment or other surcharge loads (i.e. excavation spoils) should be allowed within a horizontal distance from the top of any slope equal to the depth of the excavation (both distances measured from the top of the excavation slope).

4. Adequate measures should be taken to protect any structural foundations, pavements, or utilities adjacent to any excavations.

C. Temporary Shoring

1. The proposed excavation for the subterranean parking garage is expected to be approximately 25 feet deep and will be adjacent to property lines and adjacent structures. Temporary shoring will be necessary to support this excavation during construction. The shoring may consist of a soldier pile and lagging type system or similar temporary shoring system. The shoring may be cantilevered or anchored with tie-backs or internal bracing.

2. Cantilevered, shoring should be designed to resist active lateral earth pressures of 38Z pounds per square foot (psf) per foot of depth, where Z = Depth (in feet) measured below the top of the retained ground surface behind the shoring. This value is based on level ground behind the shoring.

3. **The lateral earth pressure to be resisted by shoring should be increased to allow for surcharge loads**. Surcharge pressures should be added to earth pressure for surcharges within a distance from the top of the shoring less than or equal to the shoring height. A lateral earth pressure coefficient of 0.5 (50%) of uniform vertical surcharges should be added as a horizontal shoring pressure for braced shoring or shoring with ground anchors. The resultant lateral surcharge from individual foundations or other loads behind the shoring may be estimated using commonly accepted formulas and charts based on elastic theory (e.g. NAVFAC DM7.2 Figure 11). Footing load surcharges are distributed non-uniformly with the resultant acting at an elevation typically 0.5 to 0.6H' where H' is the retained soil height below base of footing.

4. Lateral (horizontal) loads may be resisted by passive resistance of the soil against the soldier piles. An equivalent fluid weight (EFW) of 385 psf per foot of penetration in native soil may be used for lateral load design. The maximum passive pressure used for design should not exceed 4,000 psf. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

5. Axial loads on the soldier piles may be resisted by skin friction on the piles below the depth of the excavation (assumed to be approximately 25 feet below existing grade). The skin friction on the proposed soldier piles may be taken as 50 +20Z psf where "Z" is the depth below the bottom of the excavation. This calculated value should be multiplied by the perimeter surface area of the grouted soldier pile.

6. To limit sloughing, exposed soil between soldier piles should be supported by lagging and backfilled. Backfill behind lagging should consist of concrete slurry (1 to 2 sack of cement per CY) placed so that no voids remain between the lagging and the excavation face of soil. Alternatively, exposed soil can be supported through the use of reinforced gunite or other approved material designed to minimize soil movement. For design, the lateral earth pressure on lagging may be taken as 60% of the overall lateral earth pressure on the shoring system (need not include seismic pressure). All timber lagging to be left in the ground should be pressure treated in accordance with Standard Specifications for Public Works Construction, Section 204-2. Elimination of lagging from portions of the excavation may be considered depending upon the stability of the material as observed after installation of soldier piles and during excavation. Lagging may be eliminated only upon approval of the geotechnical engineer and the City of Los Angeles.

7. It is recommended that Earth Systems review the plans and specifications for the proposed shoring system and that an Earth Systems representative observe and monitor the installation of the shoring. The City of Los Angeles typically requires observation of all shoring installation procedures by a City-registered deputy grading inspector. Earth Systems can provide this service if requested.

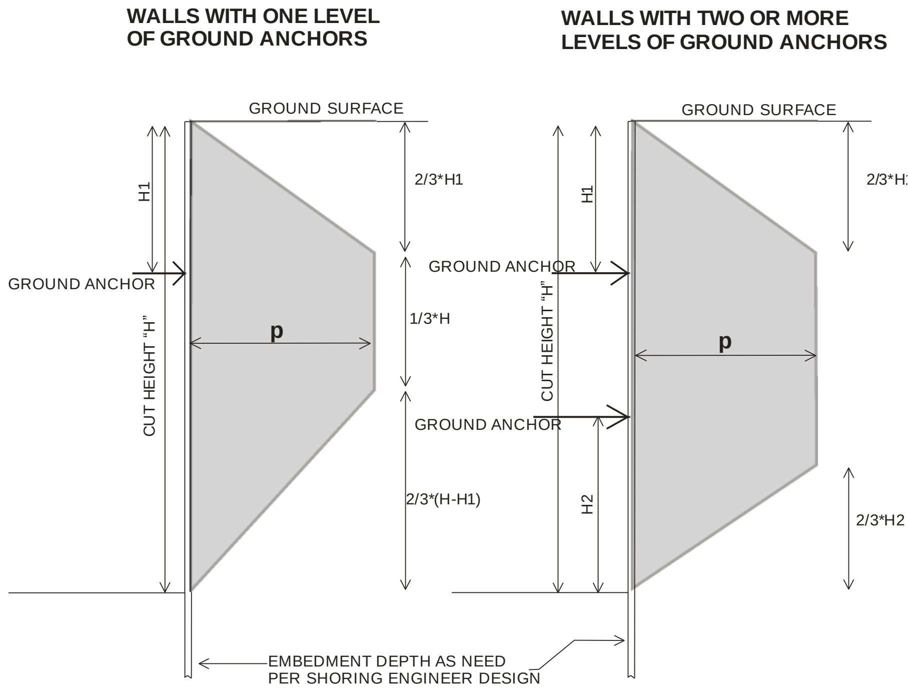
8. Vertical and lateral deflections of the shoring elements and the neighboring buildings should be verified periodically during construction by a licensed surveyor so that, if excessive deflections occur, they can be detected early and appropriate remedial measures can be taken.

9. A “baseline” condition survey (both photographic and topographic) is highly recommended prior to commencing excavation. A “baseline” survey will help identify potential areas of concern and minimize the potential for future questions regarding damage or distress due to the construction process.

10. A shoring monitoring program should be specified by the shoring/structural engineer. At a minimum, initial survey control points should be established at the top of each soldier pile prior to the start of any excavation. Soldier piles should be survey monitored at least weekly and the monitoring program should continue for a period of time to be determined by the shoring/structural engineer but continuing at least throughout the duration of all anchor installation, and grouting and all soil excavation. Monitoring points in the ground area above grouted anchors is also recommended. Real-time monitoring should take place during anchor grouting.

11. Survey and other monitoring data should be promptly submitted to the shoring/structural engineer and the geotechnical engineer for review. The shoring/structural engineer and the geotechnical engineer should be given the opportunity to evaluate any movement in excess of the specified deflections and so a remedial shoring plan can be prepared. Remedial actions may involve installation of additional soldier piles, braces, tieback anchors, or re-tensioning or post grouting of existing anchors in order to minimize movement prior to further construction.

12. A temporary tied-back (or internally braced) soldier pile shoring system retaining a level ground surface should be designed to resist the apparent earth pressure envelopes as shown in Figure 1 below. The apparent earth pressure should be distributed to the bracing, tiebacks or struts in accordance with the following figure which is based on the Federal Highway Administration Publication No. FHWA-IF-99-015. This figure is for a cut in sand and is assumed to most accurately reflect the soil conditions anticipated during excavation. The apparent earth pressures shown in this figure **do not** account for surcharge pressures.



$$p = 42H \qquad\qquad p = \frac{(27H^2)}{(H-1/3*H1-1/3*H2)}$$

Figure 1 Diagrams illustrating lateral pressure envelope for sands. Diagrams based on Federal Highway Administration's "Geotechnical Circular No. 4 Ground Anchors and Anchored Systems”.

13. Performance tests should be conducted on each tie-back anchor to verify capacity. Performance testing should be performed by the shoring contractor under the supervision of the geotechnical engineer and shoring/structural engineer. The ground anchor test results must be approved by the geotechnical and shoring/structural engineers prior to excavation of soil below the tieback elevation. Specifications for anchor performance testing should be prepared by the Shoring Engineer and reviewed by the geotechnical engineer.

14. For design of tied-back shoring, it is assumed that the potential wedge of failure is determined to be a plane 60 degrees from vertical and passing through a point set back 0.2H from the face of shoring. The tie-back anchors may be installed at angles of 15 to 40 degrees below a horizontal plane. Tie-back anchors should be designed such that there is at least 15 feet of overburden soil above the grouted (bond length) portions of the anchors (see Plate V).

15. Presumptive ultimate values of load transfer for preliminary design of a small diameter, straight-shaft, gravity-grouted ground anchors for the native soils are estimated to be between 2,000 pounds to 3,500 pounds per foot of bond length. These ultimate values represent an estimate of bond length capacity and could change once performance testing is

carried out. Only the friction resistance developed beyond the assumed failure plane should be included in the tie-back design for resisting lateral loads (see Plate VI).

16. Bond lengths should extend a minimum distance of H/5 or 5 feet past the failure surface, where "H" is the excavation cut height.

17. In addition to gravity grouting of the anchors other techniques such as pressure grouting are in common use in the industry and may result in higher ultimate values of load transfer for the bond length. These methods may be used at the discretion of the contractor or shoring engineer pending there approval by the City of Los Angeles.

D. Mat Foundations

It is recommended that any building or structure constructed on this site be designed to at least the minimum standards for Seismic Zone 4 as designated by the 2014 edition of the Los Angeles Building Code (LABC) or more current version if applicable.

1. Due to the soft nature of the site soils and the potential for seismic-induced ground movement, a structural mat foundation should be used for the main 15-story tower. The mat may be conventionally reinforced or may consist of a post-tensioned slab foundation. Specific criteria for post-tensioned slab design can be provided if a post-tensioned slab foundation is selected.

2. If the subgrade soils become disturbed during excavation of the basement, some level of moisture conditioning and recompaction may be necessary to reestablish a competent and uniform surface prior to construction of the mat.

3. An allowable "net" bearing capacity of 2,350 pounds per square foot (psf), for loads distributed over the full footprint of the foundation, may be utilized for dead and sustained live loads for design of the mat foundation. This value is a "net" value that includes the compensation for soil removal assuming a 20-foot deep parking basement. An allowable "net" bearing capacity of 2,500 psf may be used for thickened edges or other concentrated load areas. These values include a safety factor of at least 3.0 may be increased by 1/3 when considering transient loads such as earthquake or wind forces.

4. A modulus of subgrade reaction ("k_p" value) of 200 pounds per square inch per inch (psi/in for standard 30-inch square bearing plate) may be used provided the foundation subgrade soils are prepared/ compacted as recommended in Section A of this report. The subgrade reaction value may be scaled for the specific mat foundation size using the relationship ks = $k_p ((B + 1)/2B)^2$ where B = the minimum width of the mat (smallest dimension) in feet.

5. The mat foundation should be designed to accommodate differential movement of up to 1 inch in a 100-foot span (1:1200 distortion ratio).

6. Resistance to lateral loading may be provided by friction acting along the mat foundation base. A coefficient of friction of 0.35 may be used for concrete foundations on native soils. This value includes a safety factor of 1.5.

7. Additional resistance to lateral loading may be provided by passive earth pressure acting against the sides of foundations or grade beams. An equivalent fluid weight (EFW) of 385 Z psf may be used for passive pressure, where Z = Depth (in feet) below the finished ground elevation. In passive pressure calculations, the upper one-foot of soil should be subtracted from the depth, Z, unless confined by pavement or slab. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

8. The excavation for the mat foundation should be cleaned of all loose or unsuitable soils and debris prior to placement of concrete. Soil generated from the foundation excavations should not be placed below the mat slab unless properly moisture conditioned and compacted.

E. Conventional Foundations

1. Conventional shallow continuous (strip) foundations or isolated pad (column) foundations may be used for new at grade construction. Shallow foundations should be supported by a minimum 3-foot thickness of compacted soils prepared as recommended in Section A. Strip footings should be stepped to maintain horizontal bottoms along sloping ground.

2. Excavations for foundations should be cleaned of all loose or unsuitable soils and debris prior to placement of concrete. Soil generated from the foundation excavations should not be placed below the floor slab unless properly moisture conditioned and compacted, and only after the area to receive fill has been properly prepared and approved.

3. Foundations for the at-grade buildings along Hollywood Boulevard should be designed as structurally independent of the foundation system of the 15-story building.

4. Continuous (wall or strip) foundations for the proposed structures founded in either a compacted fill or competent native soil may be proportioned for the following values:

   a. Design Values: An allowable "net" bearing capacity of 2,000 pounds per square foot (psf) can be utilized for dead and sustained live loads. This value includes a minimum safety factor of 3.0, and may be increased by 1/3 when transient loads (such as wind and seismic forces) are included.

   b. Continuous foundations should be embedded a minimum of 12 inches below adjacent grade and be a minimum of 18 inches in width. For interior footings, the top of floor slab may be considered the adjacent grade, however, footings should extend at least 12 inches into the recommended bearing material. Actual depth, width, and reinforcement requirements for continuous foundations depend on the Expansion Index of the bearing material (refer to Section H of Recommendations), applicable sections of the governing building code, and requirements of the structural engineer.

c. The allowable bearing capacity for continuous foundations may be increased by 350 psf for each additional 6 inches of foundation depth, and by 150 psf for each additional 6 inches of foundation width. The allowable bearing capacity should not exceed 2,500 psf to keep estimated settlements within allowable limits. Also, the edge pressure of any eccentrically loaded footing should not exceed this bearing value for either permanent or temporary loads.

d. Continuous foundations on slopes should be stepped to maintain horizontal bottoms along all portions of the foundation.

5. Isolated pad (column) foundations for the proposed structures founded in compacted fill may be proportioned for the following values:

   a. Design Values: An allowable "net" bearing capacity of 2,400 psf can be utilized for dead and sustained live loads. This value includes a minimum safety factor of 3.0, and may be increased by 1/3 when transient loads (such as wind and seismic forces) are included.

   b. Isolated pad foundations should be embedded a minimum of 12 inches below adjacent grade and be a minimum of 2 feet in width. For interior footings, the top of floor slab may be considered the adjacent grade, however, footings should extend at least 12 inches into compacted fill. Actual depth, width, and reinforcement requirements will be dependent on the Expansion Index of the bearing material (Refer to Section H of Recommendations), applicable sections of the governing building code, and requirements of the structural engineer.

   c. Isolated pad foundations should be restrained laterally in both directions by means of grade beams, structural slab, or other approved method.

   d. The allowable bearing capacity for isolated pad foundations may be increased by 400 psf for each additional 6 inches of foundation depth, and by 100 psf for each additional 6 inches of foundation width. The allowable bearing capacity should not exceed 3,000 psf for isolated pad (column) foundations to keep estimated settlements within allowable limits. Also, the edge pressure of any eccentrically loaded footing should not exceed this bearing value for either permanent or temporary loads.

6. The edge pressure of any eccentrically loaded footing should not exceed this bearing value for either permanent or temporary loads. This value includes a minimum safety factor of three, and may be increased by 1/3 when transient loads (such as wind and seismic forces) are included.

7. Resistance to lateral loading may be provided by friction acting along the foundation base. An allowable coefficient of friction of 0.35 may be used for concrete foundations bearing in site soils recompacted to at least 95% of maximum dry density as determined by ASTM D 1557 test methods], and may be used with dead loads. This value includes a safety factor of 1.5.

8. Additional resistance to lateral loading may be provided by passive earth pressure acting against the sides of foundations or grade beams. Passive pressure may be taken as 385 Z psf, where Z = Depth (in feet) below the finished ground elevation. In passive pressure calculations, the upper one-foot of soil should be subtracted from the depth, Z, unless confined by pavement or slab. The maximum passive pressure used for design should not exceed 4,000 psf. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended). Frictional and passive resistance to lateral forces may be combined without further reduction.

F. Slab-on-Grade Construction

1. Interior and exterior building concrete slab-on-grade construction should be supported by compacted soils prepared as recommended in Section A of this report.

2. A minimum of 2 inches (2”) of compacted granular material (sand or gravel) should be placed over the finished compacted subgrade prior to placing concrete. This granular material should be moisture conditioned to near optimum moisture content and uniformly compacted using mechanical compaction equipment.

3. Reinforcement of slab-on-grade construction is contingent upon the structural engineer's recommendations and the Expansion Index of the supporting soils. Since the mixing of fill soils with native soils could change the Expansion Index, additional tests should be conducted during rough grading to determine the expansion characteristics of the new subgrade soils. Structural mat slabs should be designed as outlined above under Section D of this report. It is recommended that all interior and exterior concrete slab-on-grade construction be reinforced with at least #4 bars on 16-inch centers, each way. **Reinforcement should be placed at mid-depth of the slab.** Additional reinforcement may be required once the final expansion potential of the subgrade soils is known. Actual reinforcement requirements will be dependent on the Expansion Index of the bearing soils (Refer to Section H of Recommendations), applicable sections of the governing building code, and requirements of the structural engineer.

4. A modulus of subgrade reaction (“k” value) of 200 psi/inch may be used for design of the slab-on-grade provided the subgrade soils are prepared and compacted as recommended in Section A of this report.

5. Cracks that develop in concrete slab-on-grade should be filled and sealed prior to placing floor coverings. Frequent control joints should be incorporated into the slab construction, particularly in the areas of re-entrant corners, to help control cracking.

6. Special considerations are necessary in areas where moisture-sensitive floor coverings (e.g. carpet, linoleum tile, etc.) will be applied to the top of the slab. The design of the slab and underlying capillary water barrier and vapor retarder should be designed by a qualified waterproofing expert in accordance with ACI 302.2R-06.

7. An appropriate vapor retarder should be installed in order to minimize vapor transmission from the subgrade soil to the slab ("Vapor Block" or equivalent product meeting requirements of ASTM E 1746 is recommended). The vapor retarder should be evaluated for holes and/or punctures, and the edges overlapped and taped, prior to placement of sand. Any holes or punctures observed should be properly repaired. The vapor retarder may be underlain and/or overlain by 2 inches of sand if recommended by the waterproofing expert per the criteria in ACI 302.2R-06. If sand is used, it should be lightly moistened and densified just prior to placing the concrete.

8. Relatively impervious floor coverings (i.e. vinyl, linoleum, etc.) that cover concrete slab-on-grade may block the passage of moisture vapor through the concrete slab, which could result in damage to the floor covering. It is suggested that after the concrete slab has sufficiently cured, the concrete slab surface be sealed with a commercial sealant prior to placing the floor covering. The compatibility, and recommendations for placing of the concrete sealer, mastic, and floor covering should be verified by the floor covering manufacturer prior to sealing the concrete or placing of the floor covering.

9. It is recommended that the proposed exterior perimeter slabs (sidewalks, patios, walkways, etc.) be designed to be relatively independent of foundation stems (free-floating) to help mitigate cracking due to foundation settlement and/or expansion.

10. Subgrade soils for all concrete flatwork should be moisture conditioned to near optimum moisture content to a depth of at least 18 inches within 24 hours prior to placement of concrete. Measures should be taken to maintain optimum moisture until concrete is placed. Actual depths of pre-moistening will be dependent upon the actual Expansion Index of the subgrade soils. .

<u>G. Retaining Walls</u>

1. The following static lateral earth pressures may be used in the design of any proposed retaining walls or similar structures on the subject site:

	Equivalent Fluid Weight (pcf)	
	Driving Earth Pressure*	Resisting Earth Pressure*
Active, Well-drained level backfill soil	38	385***
At-rest (restrained) wall, Level backfill soil	63**	-

* Equivalent fluid pressure (psf) per foot of soil height = equiv. fluid weight (pcf).

** For purposes of design, a wall is considered restrained if it is prevented from movement greater than 0.002H (H = height of wall in feet) at the top of the wall. According to the 2010 California Building Code foundation/retaining walls not more than 8 feet high laterally supported by flexible diaphragms (such as wood frame floors) need not be considered restrained walls.

*** Passive pressure. The upper one foot of soil should be neglected for passive pressure calculations unless confined by pavement or slab.

NOTE: The pressures recommended above were based on the assumption that retaining walls will be cast against existing shoring or that the on-site soils will be used for wall backfill and will be compacted to approximately 95% of maximum dry density. The use of select granular fill may reduce the recommended driving earth pressure. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

2. According to the 2014 LABC (1803.5.12) lateral earth pressures due to earthquakes must be considered for walls that retain at least 6 feet of earth on projects of Seismic Design Categories D, E and F. It is Earth Systems' understanding that proposed retaining walls will be 20 feet or more in height and the seismic design category for the proposed building is D. The total earth pressures on the walls will be the sum of the static active equivalent fluid pressure plus the dynamic increment of earth pressure. The static lateral earth pressures are as discussed above. The resultant of the seismic increment of earth pressure may be applied at one-third-height of the wall - triangular pressure distribution (Atik and Sitar, 2010, Lew et al, 2010). In accordance with the City of Los Angeles memorandum dated 7/16/2014 Earth Systems estimates a pseudo-static coefficient of 0.33 should be used to calculate seismic earth pressures. Using this pseudo-static coefficient and the trial wedge method Earth Systems recommends a seismic increment of earth pressure expressed as an equivalent fluid weight of 41 pcf may be used in the design of basement or retaining walls.

   These values are based on ground motions corresponding to the 2% in 50-year probability and may be used for both restrained and unrestrained retaining walls.

3. Resistance to lateral loading may be provided by friction acting along the foundation base. A coefficient of friction of 0.35 may be used in designing concrete retaining wall foundations in firm, native soils This value includes a safety factor of 1.5. Frictional and passive resistance may be combined without further reduction.

4. The lateral earth pressure to be resisted by retaining should be increased to allow for surcharge loads. The surcharge considered should include the loads from any structures or vehicle traffic within a distance approximately equal to the height of the retaining wall.

5. Backfill immediately behind any retaining structure should be a free-draining granular material. Comments on the characteristics of import soils will be given by the geotechnical consultant after the material is on the project, either in place, or stockpiled in adequate quantities to complete the project.

6. Backfill behind retaining walls should be with soils that have been properly moisture conditioned to approximately optimum moisture content and uniformly compacted to at least 95% of maximum dry density as determined by ASTM D 1557 test procedures using mechanical compaction equipment. To aid in the compaction operation, retaining wall backfill should be placed in lifts not exceeding six inches compacted thickness.

7. Compaction within the area of a 1H:1V slope from the bottom of wall excavations should be performed by hand operated compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.

8. Weepholes, backdrains, or an equivalent system of backfill drainage should be incorporated into the retaining wall design (see Plate VII for backdrain details). Waterproofing of retaining walls should be provided to help reduce the potential for efflorescent formation.

9. The final grade should be such that all water is diverted away from the retaining wall's foundation or backfill.

H. Expansive Soil

1. The Expansion Index (ASTM D 4829) of the subgrade soils should be considered when designing foundations. As stated in the Soil Conditions section, the on-site soils are considered to have a very low expansion potential. The foundation and slab-on-grade design recommendations provided in Sections E and F of this report include generally used guidelines in the Los Angeles area for foundation design for soils with the indicated degree of expansiveness.

2. The design recommendations included in this report are minimums and comply with normally accepted geotechnical engineering practices. However, actual foundation and slab-on-grade construction reinforcement should be determined by the structural engineer based upon site

specific conditions such as foundation loading and engineering characteristics of the subgrade soils.

3. If the site soils are thoroughly mixed and/or additional fill is added during site preparation, the expansion potential may change. The expansion potential of the new subgrade soils should be determined after the site preparation has been completed, and the final foundation design adjusted accordingly.

I. Utility Trenches

Standard construction techniques should be sufficient for site utility trench excavations. The surface of utility trench backfill frequently settles even when backfill is placed under optimum conditions. Structural units or pavement placed over such backfill should be designed to accommodate such movements. Jetting of utility trench backfill is not recommended.

1. Backfill of utilities within rights-of-way should be placed in strict conformance with the requirements of the governing agency. However, as a minimum it is recommended that utility trench backfill should be moisture conditioned to near optimum moisture content and be uniformly compacted to at least 95% of maximum dry density using mechanical compaction equipment. To aid in the compaction operation, utility trench backfill should be placed in lifts not exceeding six inches compacted thickness.

2. The provisions of this report relative to minimum compaction standards should govern utility trench backfill within the project boundary. In general, service lines extending inside the site should be backfilled with native soils that have been moisture conditioned and uniformly compacted to at least 95% of maximum dry density using mechanical compaction equipment. To aid in the compaction operation, utility trench backfill should be placed in lifts not exceeding six inches in compacted thickness.

3. Backfill operations should be reviewed and tested by the geotechnical engineer's representative to verify conformance with these recommendations.

J. Soil Chemical Testing

A sample of the near-surface soils was tested for pH, resistivity and conductivity, as well as a variety of cations and anions including soluble sulfates. It should be noted that the sulfate content (17 mg/Kg) is in the "S0" exposure class Table 19.3.1.1 of ACI 318-14.

Based on criteria established by the County of Los Angeles, a measurement of resistivity of 9,200 ohm-cm on the near-surface materials indicate that are "moderately corrosive" to ferrous metal (i.e. cast iron, etc.) pipes.

The test results provided in Appendix C should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. Tests should be conducted of the surface soils in the final graded pad to verify these interpretations, especially if the soils are mixed and additional fill is added during site preparation.

K. Slope Stability

Slope stability calculations were not performed because of anticipated minimal slope heights. If slope heights exceed five feet, engineering calculations should be performed to substantiate the stability of cut or fill slopes. Fill slopes should be constructed to a gradient not exceeding two horizontal to one vertical (2H:1V) and should be overfilled and trimmed back to compacted material.

L. Sub-drainage and Waterproofing

1. All retaining walls, basements, or partial-subterranean portions of the proposed structure(s) must be provided with adequate sub-drainage and back-drainage to reduce the potential for hydrostatic pressures on the structures. If adequate back-drainage and sub-drainage is not provided, the retaining walls or subterranean portions of the structure must be designed and constructed as a “water-tight” structure able to resist anticipated hydrostatic lateral pressures and uplift pressures.

2. Minimum typical details for basement retaining wall back-drainage are provided on Plates VI and VII.

3. Effective waterproofing should also be provided to reduce dampness, efflorescence, mold and other detrimental impacts of excess moisture. A qualified waterproofing specialist should be consulted for specific waterproofing recommendations and designs (Earth Systems is not responsible for waterproofing design).

CLIENT OPTIONAL SERVICES

This report was based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check conformance with the recommendations of this report. Maintaining Earth Systems as the geotechnical engineering consultant from beginning to end of this project will help provide continuity of services. The recommended services include, but are not necessarily limited to, the following:

a. Consultation as required during the final design stages of the project.
b. Review of grading and/or building plans.
c. Observation and testing during site preparation, grading, placement of engineered fill, and backfill of utility trenches.
d. Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The conclusions and recommendations submitted in this report relative to the proposed mixed use facility are based, in part, upon the data obtained from site observations during the field exploration operations, and past experience. The nature and extent of variations between the soil borings and CPT soundings may not become evident until construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report.

In the event of any change in the assumed nature or design of the proposed project as planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing. This report is issued with the understanding that it is the responsibility of 6436 Hollywood Blvd. LLC to insure that the information and recommendations contained in this report are called to the attention of the architects and engineers for the project and incorporated into the plan. It is also the responsibility of Hollywood Blvd. LLC, and its representatives, to insure that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

As the geotechnical engineers for this project, Earth Systems strives to provide its services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of Hollywood Blvd. LLC for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without the express written authorization of Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design specifications. If Earth Systems is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations.

The scope of current services for this report did not include any environmental assessment or investigation for the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around the site.

The statements contained in this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the conclusions of this report may be invalidated, wholly or partially, by changes outside of Earth Systems' control, and should therefore be reviewed after one year.

CLOSURE

Earth Systems trusts this report is sufficient at this time and meets your current needs. Earth Systems appreciates this opportunity to provide professional geotechnical engineering services for this project. If you have any questions regarding the information contained in this report, or if you require additional geotechnical engineering services, please contact the undersigned.

Respectfully submitted,
Earth Systems

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10/7/16

END OF TEXT

REFERENCES

PLATES

APPENDICES

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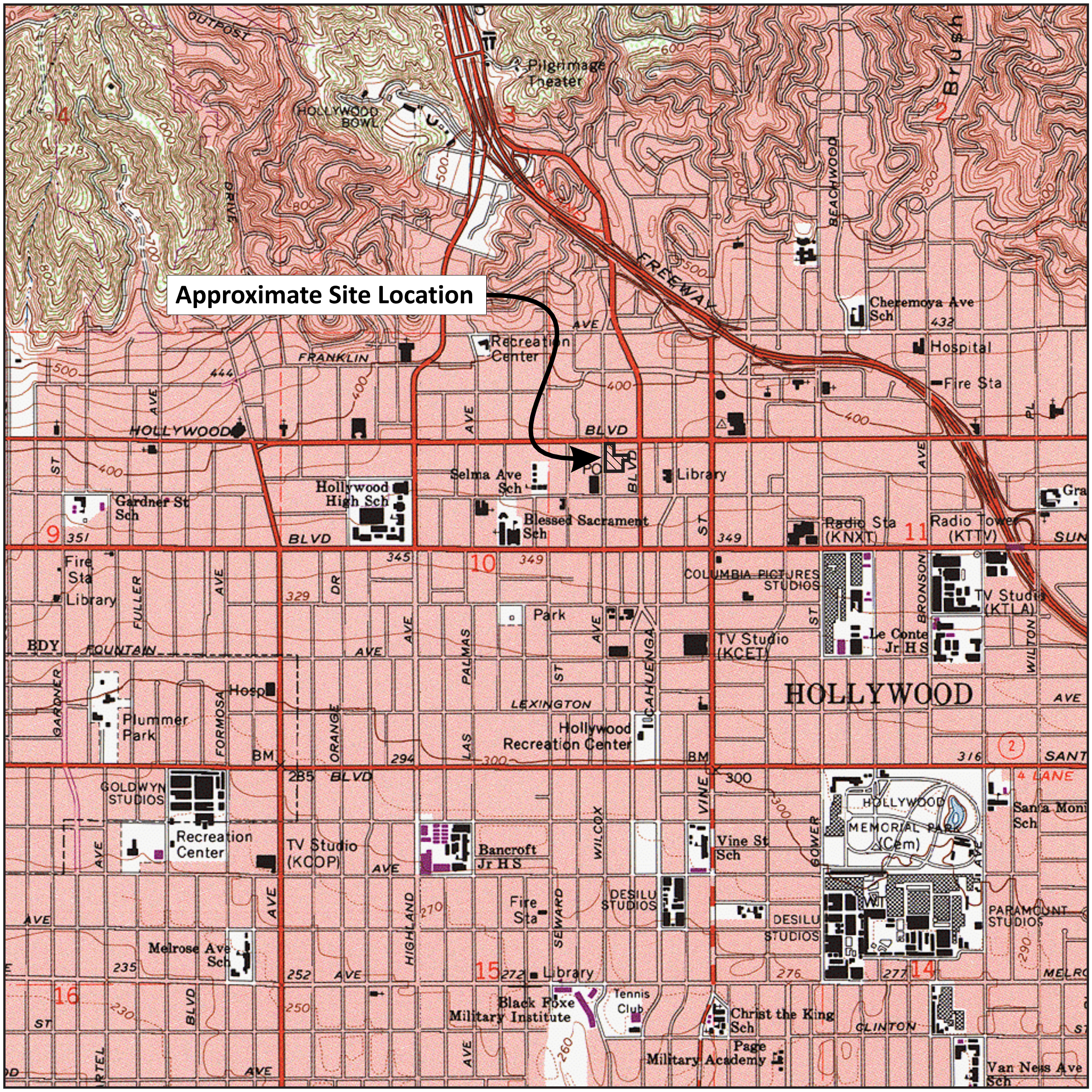
PLATES



Source: USGS, Hollywood 7.5 Minute Quadrangle, dated 1966, Photorevised 1981.

0 ft. ~ 1,000 ft. ~ 2,000 ft.

N

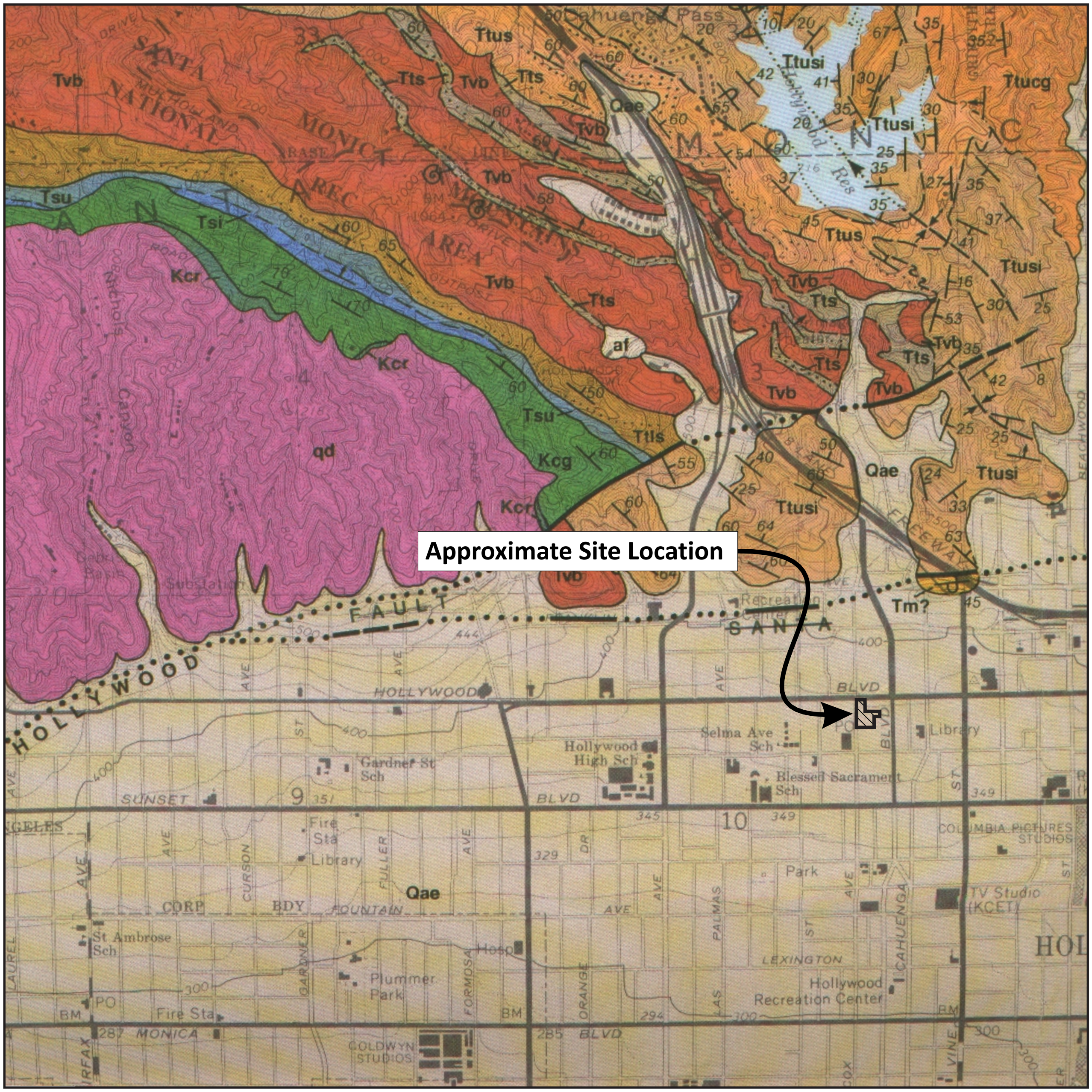
Plate I

SITE LOCATION MAP

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01



Source: Dibblee Geologic Maps, Hollywood and Burbank (south 1/2) Quadrangles, DF-30, 1991.

LEGEND

Quaternary

- af Artificial fill
- Qae Older dissected alluvium

Tertiary

- Tvb Upper Topanga Formation baslatic volcanics
- Ttls Lower Topanga Formation sandstone
- Tsu Santa Susana Formation sandstone-silty claystone

Cretaceous

- Kcg "Unnamed" strata conglomerate
- qd quartz diorite

Geologic contact

Fault

25 Bedding Attitude

N

0 ft. ~ 1,000 ft. ~ 2,000 ft.

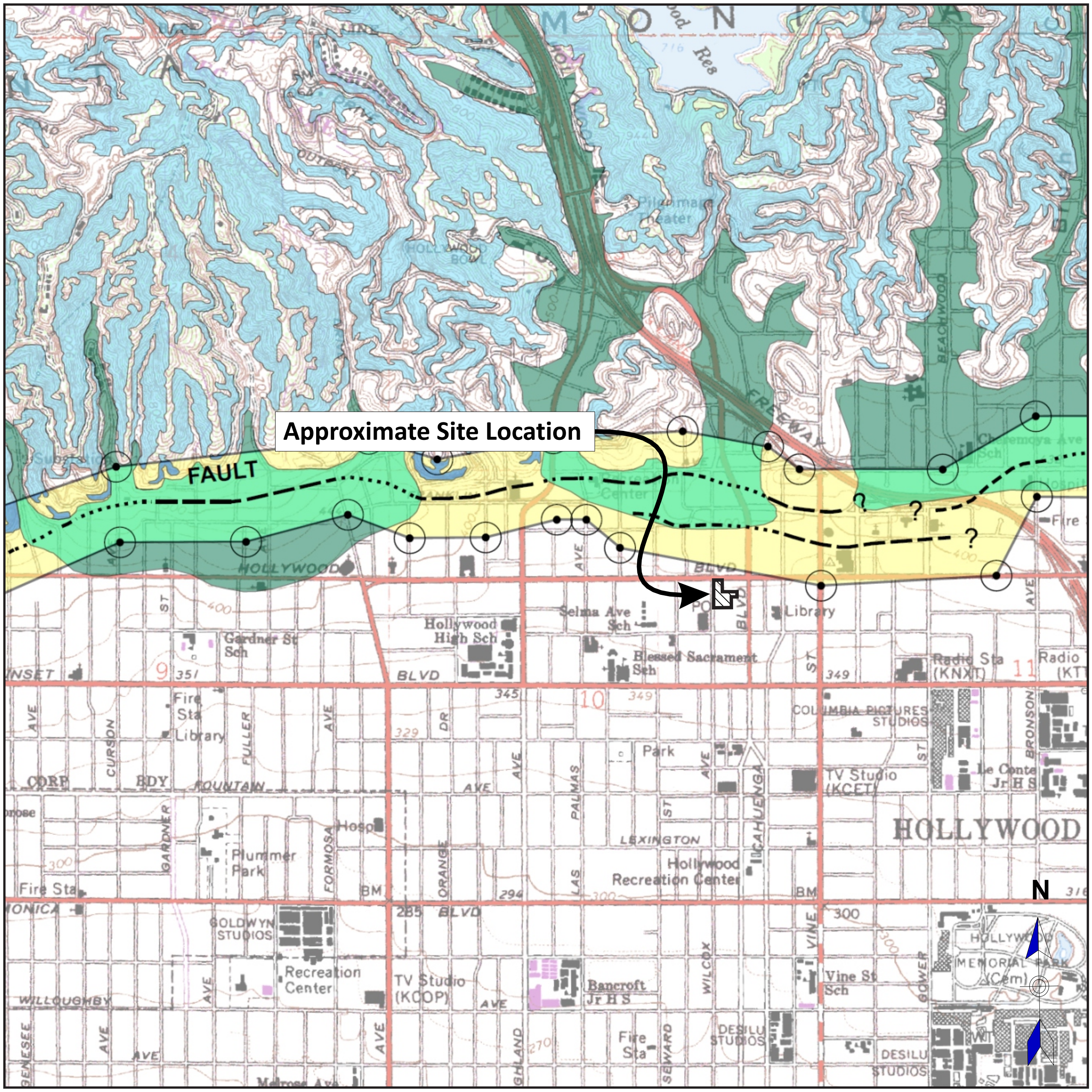
Plate II

REGIONAL GEOLOGIC MAP

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01



Source: CDMG, Hollywood Quadrangle, Seismic Hazard Zones Map, dated March 25, 1999, Earthquake Fault Zones, November 6, 2014.

MAP EXPLANATION

Liquefaction
Areas where historic occurrences of liquefaction or local geological, geotechnical and groundwater conditions include a potential for Permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) wold be required.

Earthquake induced Landslides
Areas where previous occurrences of landslide movement, or local topographic, geologic, geotechnical and subsurface water conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section (2693(c) would be required.

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments that connect encircled turning points; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

1906 C

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where infered, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

0 ft. ~ 1,000 ft. ~ 2,000 ft.

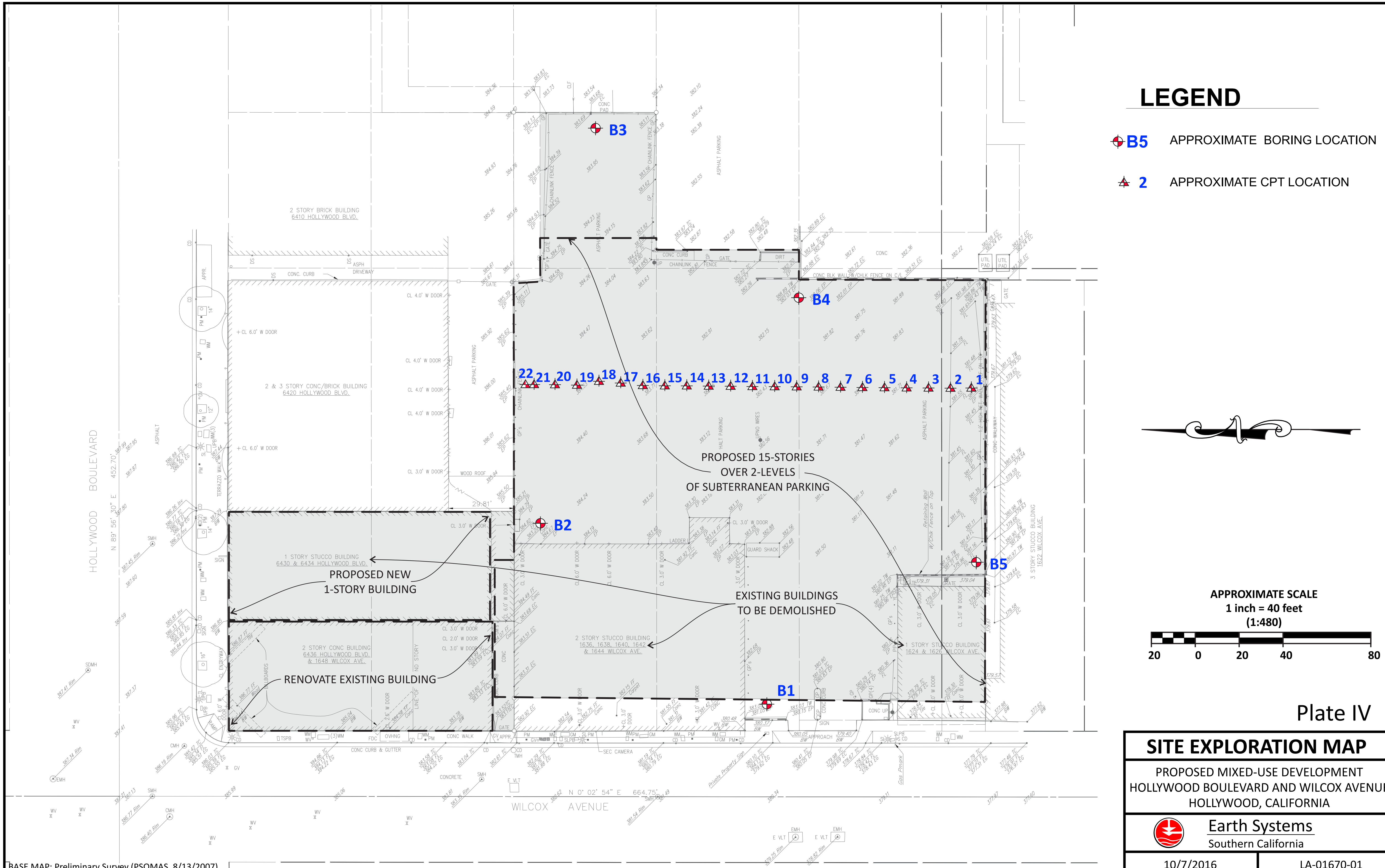
Plate III

SEISMIC HAZARD ZONES MAP

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01



BASE MAP: Preliminary Survey (PSOMAS, 8/13/2007)

Plate IV

SITE EXPLORATION MAP

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01

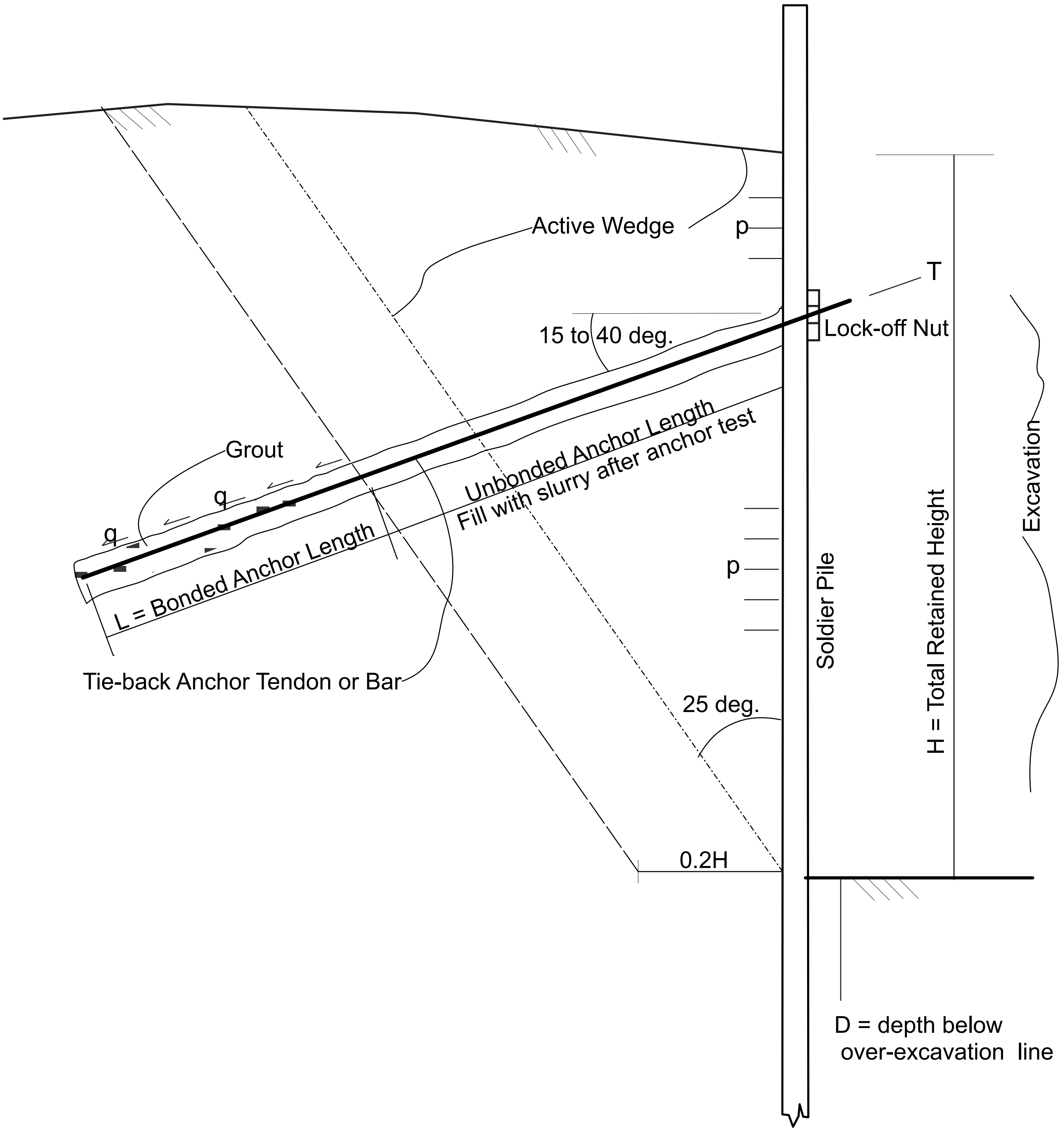


Plate V

TIE-BACK DESIGN SECTION SCHEMATIC

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01

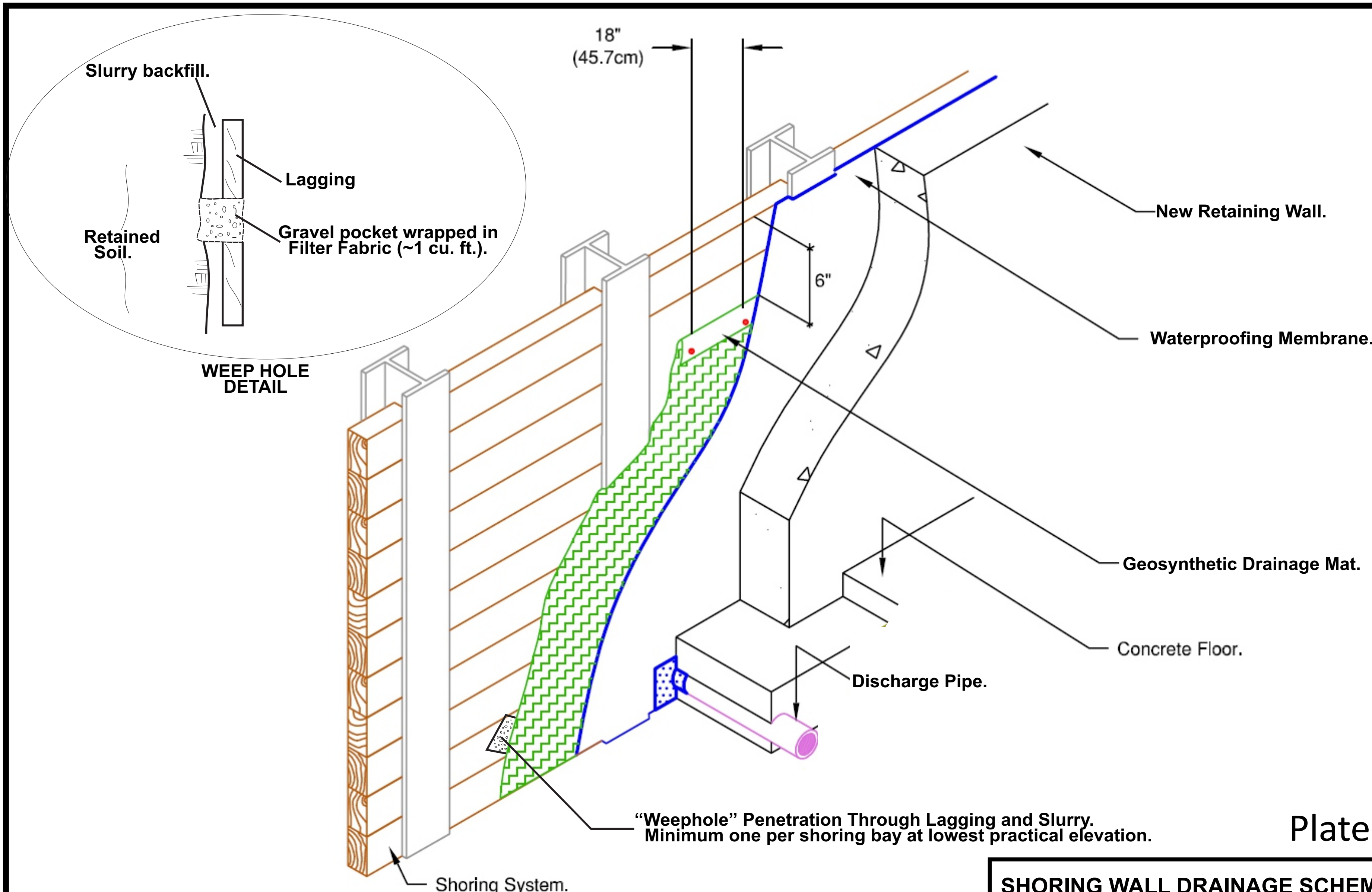


Plate VI

SHORING WALL DRAINAGE SCHEMATIC

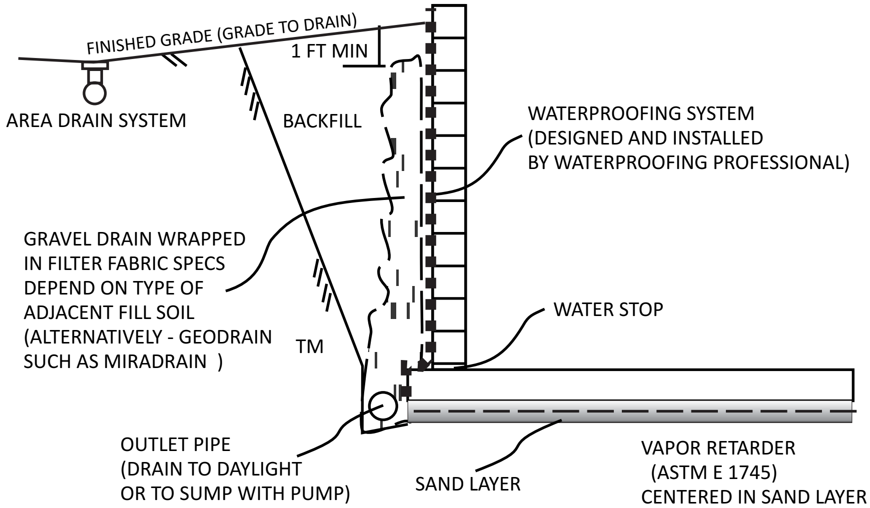
PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

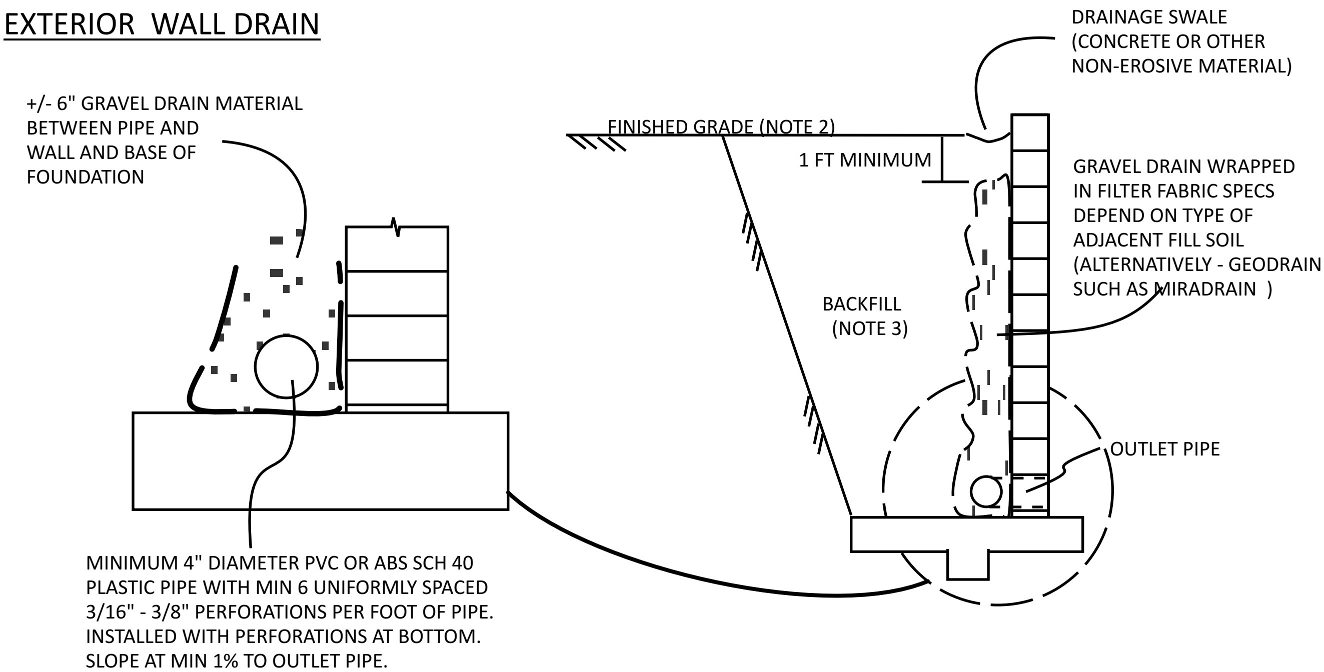
10/7/2016	LA-01670-01

Base detail obtained from Tremco, Inc.

BASEMENT WALL DRAIN



EXTERIOR WALL DRAIN



NOT TO SCALE

NOTE 1)
GRAVEL DRAIN MATERIAL SHALL CONSIST OF CLEAN PEA GRAVEL OR 3/8" GRAVEL WRAPPED IN APPROPRIATE FILTER FABRIC* OR CALIFORNIA CLASS II PERMEABLE MATERIAL

NOTE 2)
USE DRAINAGE SWALE OR GRADE TO DRAIN AWAY FROM WALL.

NOTE 3)
ENGINEERED BACKFILL COMPACTED AS RECOMMENDED IN GEOTECHNICAL REPORT. SPECIAL PROVISIONS WILL APPLY TO MODERATELY OR HIGHLY EXPANSIVE BACKFILL.

NOTE 4)
WEEP HOLES IN BASE BLOCK COURSE ARE RECOMMENDED. CARE SHOULD BE TAKEN THAT WEEPHOLES ARE NOT COVERED BY EXTERIOR GRADE OR PAVING.

Plate VII

RETAINING WALL BACKDRAIN DETAILS

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BOULEVARD AND WILCOX AVENUE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01

APPENDIX A

Unified Soil Classification System
Soil Consistency Terms
Boring Log Symbols
Logs of Test Borings

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		**GM**	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		**SW**	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNTOF FINES)		**SM**	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGAINC SILTS
HIGHLY ORGANIC SOILS				**PT**	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENT

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

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10/7/2016	LA-01670-01

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS

(Major Portion Retained on Number 200 Sieve)

Includes clean gravels and sands described as fine, medium or coarse, depending on distribution of grain sizes, and silty or clayey gravels and sands, condition is rated according to laboratory tests or estimated from resistance to sampler penetration.

Penetration Resistance* California Split Spoon (CSS) Blows/Ft		Penetration Resistance* Standard Pentrometer (SPT) Blows/Ft
0-5	Very Loose	0-4
5-15	Loose	5-10
15-40	Medium Dense	11-30
40-70	Dense	31-50
>70	Very Dense	

Fine Grained Soils

(Major Portion Passing the Number 200 Sieve)

Includes inorganic and organic silts and clays, gravelly, sandy or silty clays, and clayey silts. Consistency is rated according to laboratory tests or estimated from resistance to sampler penetration.

Penetration Resistance* California Split Spoon (CSS) Blows/Ft		Penetration Resistance* Standard Pentrometer (SPT) Blows/Ft
0-2	Very Soft	0-2
2-5	Soft	2-4
6-10	Medium Stiff	5-8
11-18	Stiff	9-15
19-36	Very Stiff	16-30
>36	Hard	

* Penetration resistance based on a 140 pound hammer falling approximately 30 inches.

SOIL CONSISTENCY TERMS

PROPOSED MIXED-USE DEVELOPMENT
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SYMBOLS COMMONLY USED ON BORING LOGS

Modified California Split Barrel Sampler

Modified California Split Barrel Sampler - No Recovery

Standard Penetration Test (SPT) Sampler

Standard Penetration Test (SPT) Sampler - No Recovery

Perched Water Level

Water Level First Encountered

Water Level After Drilling

Pocket Penetrometer (tsf)

Vane Shear (ksf)

1. The location of borings were approximately determined by pacing and/or siting from visible features. Elevations of borings are approximately determined by interpolating between plan contours. The location and elevation of the borings should be considered accurate only to the degree implied by the method used.

2. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. This data has been reviewed and interpretations made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature, and other factors at the time measurements were made.

BORING LOG SYMBOLS

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10/7/2016	LA-01670-01

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Phone: (626) 356-0955 Fax: (626) 356-0956

PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B1**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									aslphalt
									Artificial Fill (af)
			X	2, 3, 3		ML	114.2	11.0	Sandy SILT, dark brown, moist, medium stiff, fine grained sand.
5			X	3, 5, 6		SM	107.4	7.7	**Alluvium (Qa)**: Silty SAND, brown, moist, loose, fine to medium grained sand.
			X	7, 9, 9		SW	108.2	4.8	Well-graded SAND with silt, brown, moist, medium dense, fine to coarse grained sand.
10			X	3, 4, 4			114.5	10.8	Silty SAND, brown, moist, loose, fine to medium grained sand.
15			X	4, 5, 7		SM	117.3	11.8	Silty SAND, brown, moist, loose, fine to medium grained sand.
20		X		3, 4, 5					Silty SAND, brown, moist, loose, fine to medium grained sand.
25			X	4, 7, 12		ML	108.0	20.0	Sandy SILT with clay, brown, moist, very stiff, fine grained sand, some clay, slightly plastic.
30		X		3, 3, 3		SC			Clayey SAND, brown, moist, loose, fine to medium grained sand, slightly plastic, clay fines.
35			X	7, 6, 6		SM	111.4	13.7	Silty SAND, brown, moist, loose, fine to coarse grained sand, trace 1/4 inch subangular gravel.
						SC			

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DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40		■		4, 3, 6					Clayey SAND, brown, moist, loose, fine to medium grained sand, slightly plastic.
45			■	5, 11, 13		SC	115.8	15.4	Clayey SAND, brown, moist, medium dense, fine to medium sand, slightly plastic.
50		■		3, 4, 6					Clayey SAND, brown, moist, loose, fine to medium sand, slight plasticity.
55									Total depth 50 feet No free groundwater encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
60									
65									
70									
75									

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PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B2**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									6 inches of asphalt
									Alluvium (Qa):
			X	5, 6, 9		SM	113.6	4.7	Silty SAND, brown, moist, medium dense, fine grained sand, non-plastic, silt fines.
5			X	4, 7, 9		SM	107.5	4.9	Silty SAND, brown, moist, medium dense, fine grained sand, non-plastic, silt fines, few fine pores.
			X	9, 11, 15		SM	116.7	5.9	Silty SAND, brown, moist, medium dense, fine grained sand, non-plastic, silt fines, few fine pores.
10			X	8, 12, 10		SM	116.4	11.3	Silty SAND, brown, moist, medium dense, fine grained sand, trace subangular gravel about 1/4 inch in size.
15		X		40, 50 (2")		SC			Clayey SAND with silt, brown, moist, very dense, fine to medium grained sand, slightly plastic.
20			X	6, 7, 8		SM	113.8	10.4	Silty SAND, brown, moist, medium dense, fine to medium grained sand, non-plastic, silt fines.
25		X		5, 4, 3		SM			Silty SAND with clay, brown, moist, loose, fine to medium grained sand, trace subangular gravel about 1/4 inch, red gravel fragment.
30			X	5, 8, 12		SC	118.3	15.0	Clayey SAND, brown, moist, medium dense, fine grained sand, slightly plastic.
35		X		3, 7, 9		SM			Silty SAND, brown, moist, medium dense, fine to medium grained sand, some clayey sand.

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PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40			X	7, 11, 14		SM	113.6	9.7	Silty SAND, brown, moist, medium dense, fine to medium grained sand, trace gravel, few fines pores.
45		X		50 (5")		SC			Clayey SAND, brown, moist, very dense, fine to coarse grained sand, slightly plastic.
50			X	10, 16, 14		SM	120.5	7.1	Silty SAND, brown, moist, medium dense, fine to coarse grained sand, non-plastic silt fines, gray halo in sample.
55		X		3, 4, 7		SM			Silty SAND, dark brown, moist, medium dense, fine to medium grained sand, slightly plastic.
60			X	9, 21, 21		SM	118.1	12.0	Silty SAND, dark brown, moist, dense, fine to medium grained sand, grey in center of sample.
65		X		5, 4, 10		SM			Silty SAND, brown, moist, medium dense, fine to medium grained sand, non-plastic.
70			X	7, 10, 11		CL	120.6	12.6	Sandy CLAY, dark brown, moist, very stiff, fine grained sand, slightly plastic, about 40% sand.
75		X		4, 5, 7		SC			Clayey SAND, brown, moist, medium dense, fine grained sand, slightly plastic.

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PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
80			■	30, 50 (3")		SM	128.8	9.0	Silty SAND with gravel, brown, moist, very dense, fine to coarse grained sand, fine to coarse gravel, fragment in sampler suggesting cobbles, non-plastic fines.
85									
90									
95									
100									
105									
110									
115									

Total depth 80 feet

No free groundwater encountered.

Backfilled with soil cuttings.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.

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PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B3**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									2 inches of asphalt
									Artificial Fill (af):
			■	3, 2, 2		SM	100.9	15.7	Silty SAND, brown, moist, very loose, fine to medium grained sand.
5			■	3, 4, 5		SM	117.3	11.0	**Alluvium (Qa):** Silty SAND, brown, moist, loose, fine to medium grained sand.
			■	6, 7, 7		SC	110.4	7.7	Clayey SAND, brown, moist, loose, fine to medium grained sand, about 40% clay, slightly plastic, trace subangular gravel about 1/4 inch in dimension.
10		■		2, 2, 2		CL			Sandy CLAY, brown, moist, soft, some fine grained sand, slightly plastic.
15			■	5, 7, 7			108.3	19.1	Sandy CLAY, brown, moist, stiff, some fine grained sand, slightly plastic, about 40% sand.
20		■		4, 7, 6		SW			Well-graded SAND with silt, brown, moist, medium dense, fine to coarse grained sand, trace gravel.
25			■	4, 7, 9		SM	111.3	9.7	Silty SAND, brown, moist, medium dense, fine to coarse grained sand.
30		■		3, 4, 5		SC			Clayey SAND, brown, moist, loose, fine to medium grained sand, slightly plastic.
35			■	6, 8, 10			117.2	10.7	Clayey SAND, brown, moist, medium dense, fine to medium grained sand, slightly plastic, trace subangular gravel about 1/4 inch in dimension.
						SM			

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PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B3**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/18/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40		■		3, 4, 6		SM			Silty SAND, brown, moist, loose, fine to medium grained sand, non-plastic fines.
45			■	10, 11, 13		SC	111.4	12.5	Clayey SAND with gravel, brown, moist, medium dense, fine to medium grained sand, slightly plastic, some fine to coarse subrounded gravel.
50		■		5, 9, 9					Silty SAND, brown, moist, medium dense, fine to medium grained sand, trace fine gravel.
55									
60									
65									
70									
75									

Total depth 50 feet.

No free groundwater encountered.

Backfilled with soil cuttings.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.

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PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B4**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/19/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									5 inches of aslphalt
	X					SM			**Artificial Fill (af):** Silty SAND
	X		X	3, 3, 4		SM	111.6	7.4	**Alluvium (Qa):** Silty SAND, brown, moist, loose, fine to coarse grained sand.
5			X	4, 8, 11		SW	106.6	4.4	Well-graded SAND with silt, brown, moist, medium dense, medium to fine grained sand.
			X	5, 6, 9			105.9	5.5	Well-graded SAND with silt, brown, moist, medium dense, medium to fine grained sand.
10			X	5, 4, 5		SM	106.6	18.8	Silty SAND, brown, moist, loose, fine to medium grained sand.
15		X		2, 2, 3		SC			Clayey SAND, brown, moist, loose, fine grained sand, slightly plastic.
20			X	7, 3, 9		SW	110.4	9.5	Well-graded SAND with silt, brown, moist, loose, fine to medium grained sand, non-plastic, trace fine subrounded gravel.
25		X		4, 7, 12					Well-graded SAND with silt and gravel, brown, moist, medium dense, fine to coarse grained sand, fine subangular gravel.
30			X	6, 9, 12		SM	114.8	14.4	Silty SAND, brown, moist, meidum dense, fine to medium grained sand, trace fine to coarse subangular gravel.
35		X		4, 5, 5					Silty SAND, brown, moist, loose, fine to medium grained sand, trace fine to coarse subangular gravel.

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PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B4**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/19/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40			X	10, 15, 12		SM	114.1	7.3	Silty SAND, brown, moist, meidum dense, fine to medium grained sand, trace fine to coarse subangular gravel.
45		X		4, 6, 7		SC			Clayey SAND, brown, moist, medium dense, frine grained sand, slightly plastic fines.
50			X	8, 17, 26		SM	124.7	10.4	Silty SAND, brown, moist, dense, fine to medium grained sand.
55		X		3, 4, 6		CL			Sandy CLAY, brown, moist, stiff, fine grained sand, slightly plastic.
60			X	7, 22, 32		SW	122.4	8.4	Well-graded SAND with silt, brown, moist, dense, fine to coarse grained sand.
65		X		4, 8, 11		SC			Clayey SAND, brown, moist, medium dense, fine to medium grained sand, slightly plastic.
70			X	6, 12, 22			121.7	13.5	Clayey SAND, brown, moist, medium dense, fine to medium grained sand, slightly plastic, few fine pores.
75		X		11, 25, 32		SW			Well-graded SAND, brown, moist, very dense, fine to coarse grained sand, trace fine to medium gravel.
						SM			

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DRILLING DATE:	7/19/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
80			■	10, 27, 50		SM	124.8	11.9	Silty SAND with clay, brown, moist, very dense, fine to medium grained sand.
									Rig chatter, possible rock or gravel.
85		■		9, 24, 43		SW			Well-graded SAND with silt and gravel, brown, moist, very dense, fine to coarse grained sand, subangular gravel and gravel fragments indicating cobbles to boulders; top 4 inches wet.
90	▼		■	11, 50 (3")			110.7	19.8	Well-graded SAND with silt and gravel, brown, moist, very dense, fine to coarse grained sand, subangular gravel and gravel fragments indicating cobbles to boulders; top 4 inches wet.
									Total depth 90 feet
									Free groundwater encountered at 90 feet.
95									Backfilled with soil cuttings.
									Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
100									
105									
110									
115									

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PROJECT NAME:	6436 Hollywood Blvd	BORING NO:	**B5**
PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/19/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									asphalt
									Artificial Fill (af)
			X	2, 3, 3		SM	108.9	8.7	Silty SAND, black to brown, moist, loose, fine grained sand, some mottling.
5			X	4, 6, 8		SM	110.4	8.3	**Alluvium (Qa):** Silty SAND, black to brown, moist, loose, fine grained sand.
			X	5, 7, 9		SW	104.5	111.7	Well-graded SAND, brown, moist, medium dense, fine to coarse grained sand.
10		X		3, 3, 4					Well-graded SAND, brown, moist, loose, fine to coarse grained sand.
15			X	3, 5, 7		SC	111.7	15.7	Clayey SAND with silt, dark brown, moist, loose, fine grained sand, slightly plastic, few fines pores.
20		X		2, 3, 5		SW			Well-graded SAND with silt, light brown, moist, loose, fine to medium grained sand.
25			X	5, 5, 8		SC	114.4	12.7	Clayey SAND, brown, moist, loose, fine grained sand, slightly plastic.
30		X		4, 4, 4					Clayey SAND, brown, moist, loose, fine grained sand, slightly plastic, about 40% clay.
35			X	5, 9, 12		SW	115.5	5.2	Well-graded SAND with silt, brown, moist, medium dense, fine to coarse grained sand.
						SC			

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PROJECT NUMBER:	LA-01670-01	DRILL RIG:	CME 75
DRILLING DATE:	7/19/2016	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	JS

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40		■		3, 5, 7		SC			Clayey SAND, dark brown, moist, medium dense, fine grained sand, slightly plastic.
45			■	7, 11, 19		SM	120.8	10.1	Silty SAND, brown, moist, medium dense, fine grained sand, some medium sand, non-plastic fines.
50		■		3, 5, 8		SC			Clayey SAND, brown, moist, medium dense, fine to coarse sand, slightly plastic fines.
55									Total depth 50 feet. No free groundwater encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
60									
65									
70									
75									

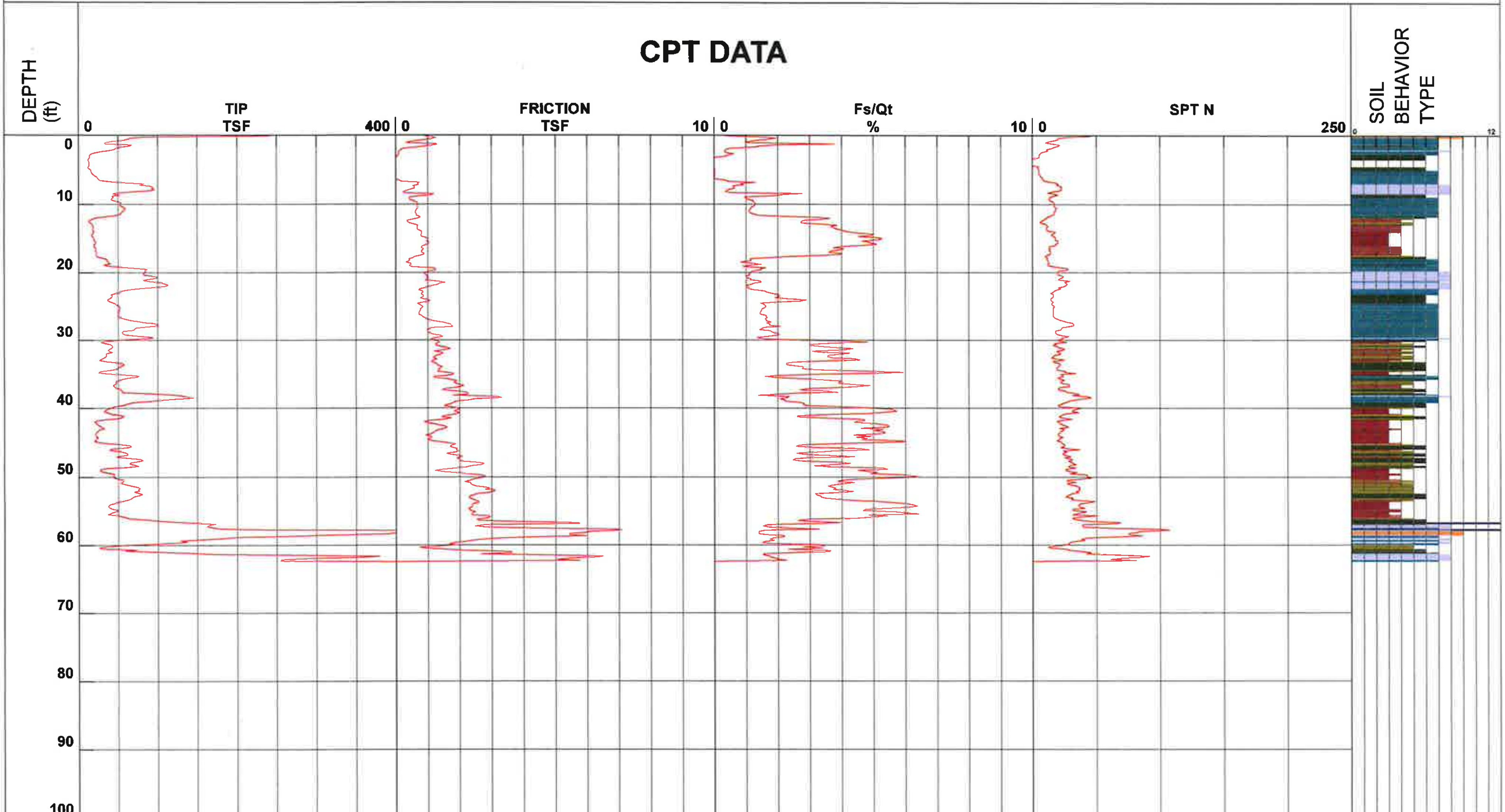
APPENDIX B

Logs of CPT Soundings

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(739).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	8/10/2016 9:23:07 AM	Maximum Depth	62.66 ft
EST GW Depth During Test	>62.66 ft				

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

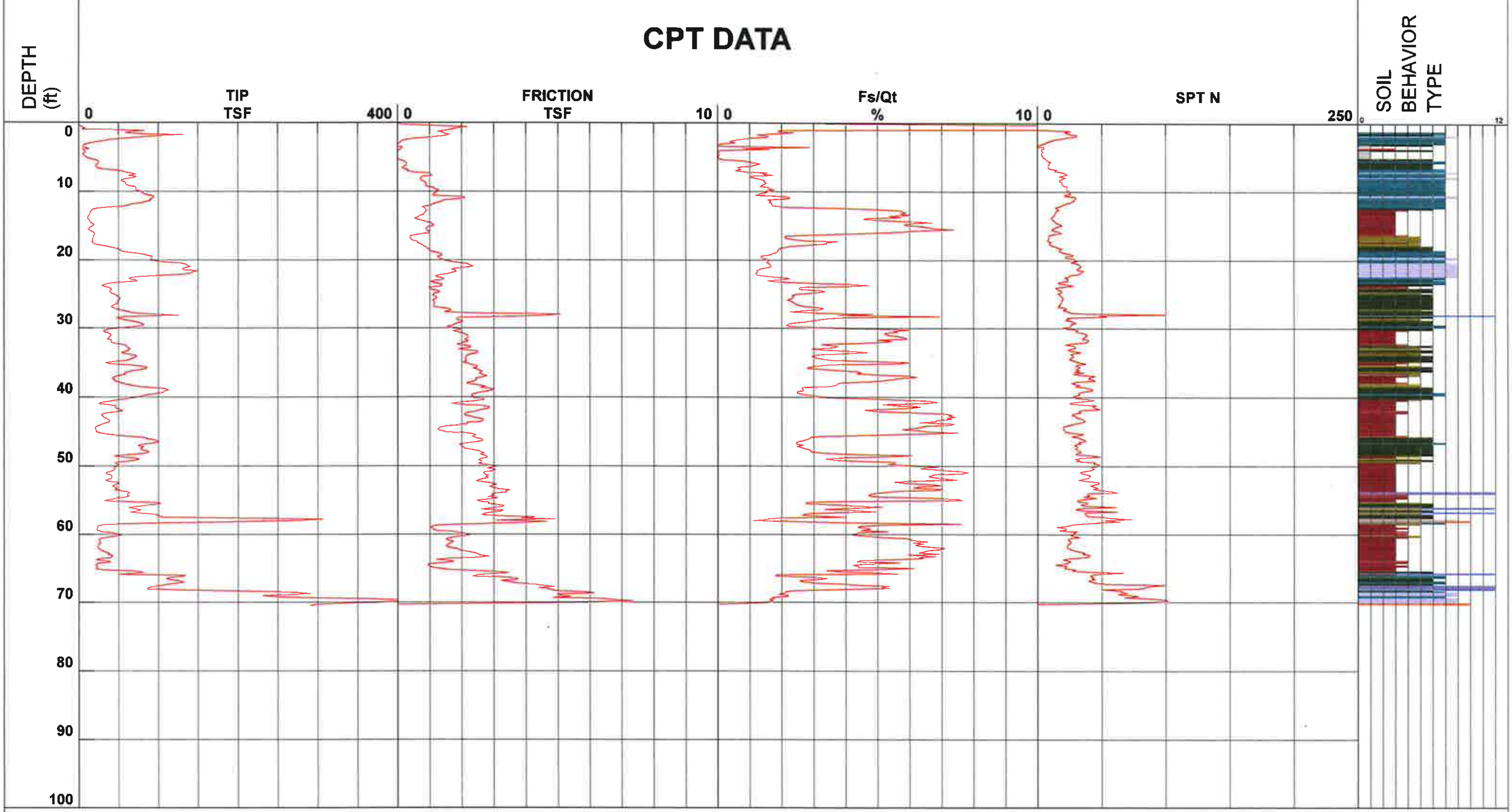
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(732).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	8/9/2016 3:34:39 PM	Maximum Depth	70.54 ft
EST GW Depth During Test		>70.54 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

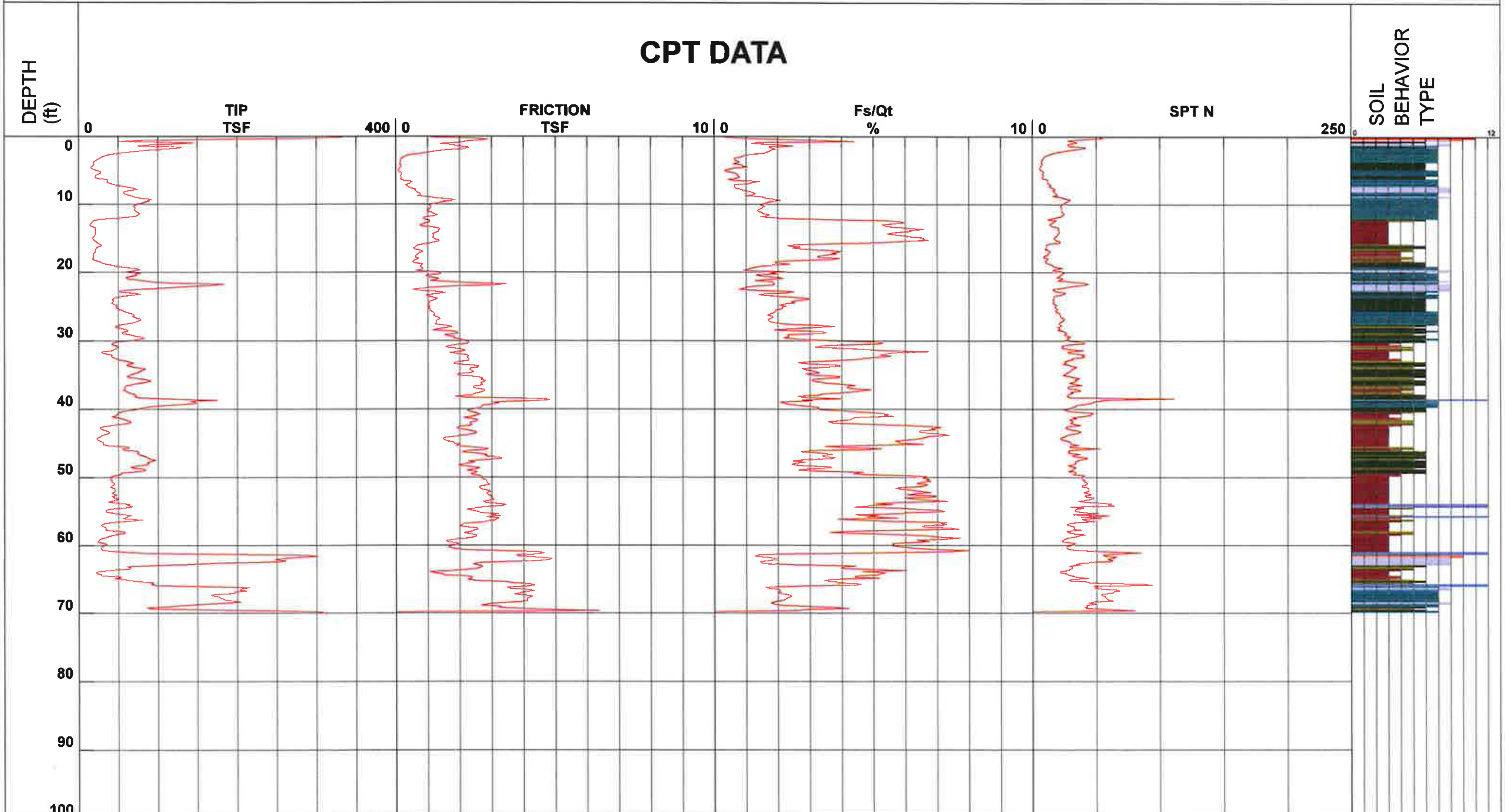
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(733).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	8/9/2016 4:41:58 PM	Maximum Depth	70.05 ft
EST GW Depth During Test		>70.05 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

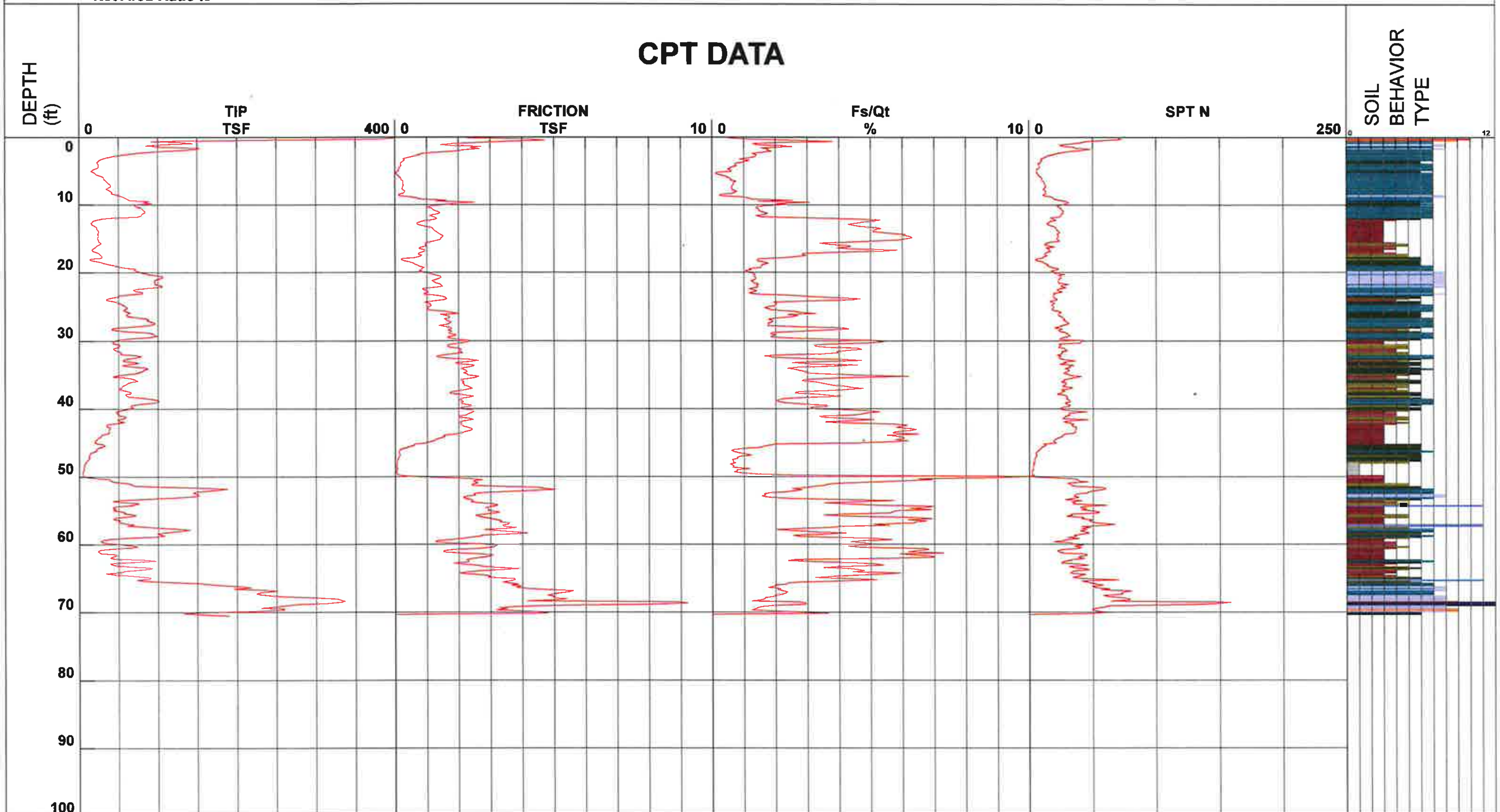
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(734).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	8/9/2016 5:26:57 PM	Maximum Depth	70.54 ft
EST GW Depth During Test		>70.54 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

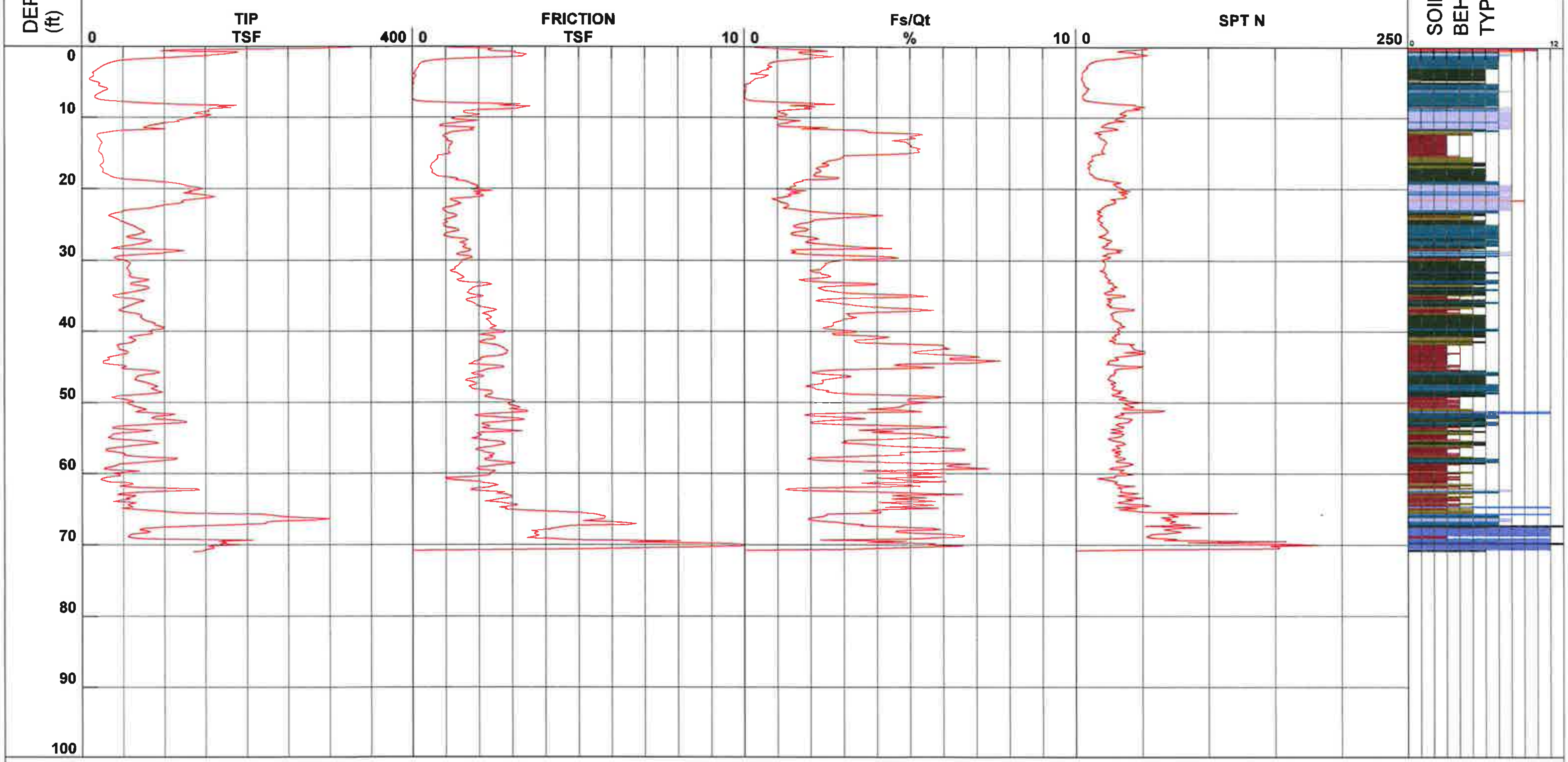
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(735).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-05	Date and Time	8/9/2016 6:17:24 PM	Maximum Depth	71.03 ft
EST GW Depth During Test	>71.03 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

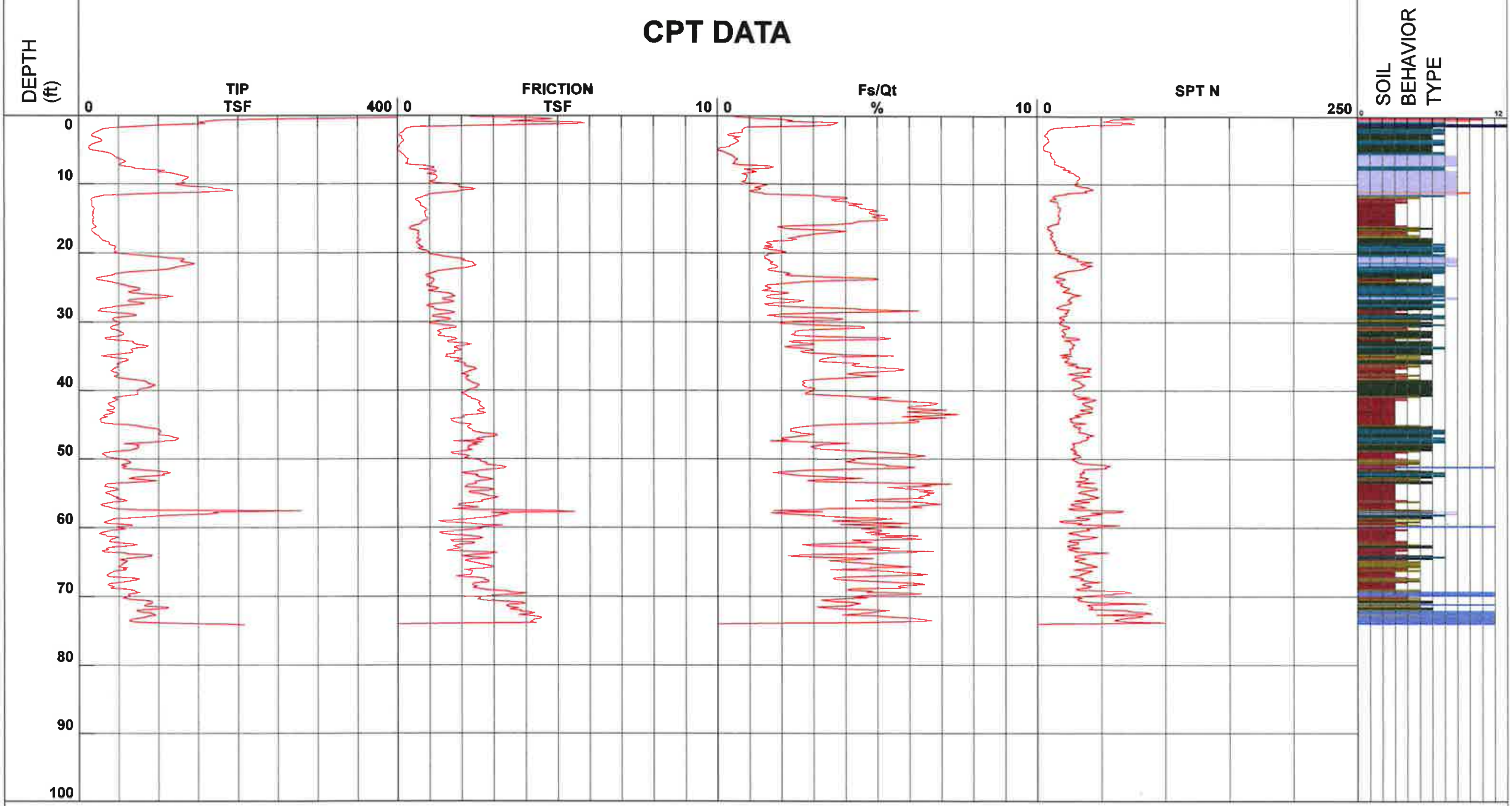
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(738).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	SCPT-06	Date and Time	8/10/2016 8:00:31 AM	Maximum Depth	74.15 ft
EST GW Depth During Test	>74.15 ft				

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

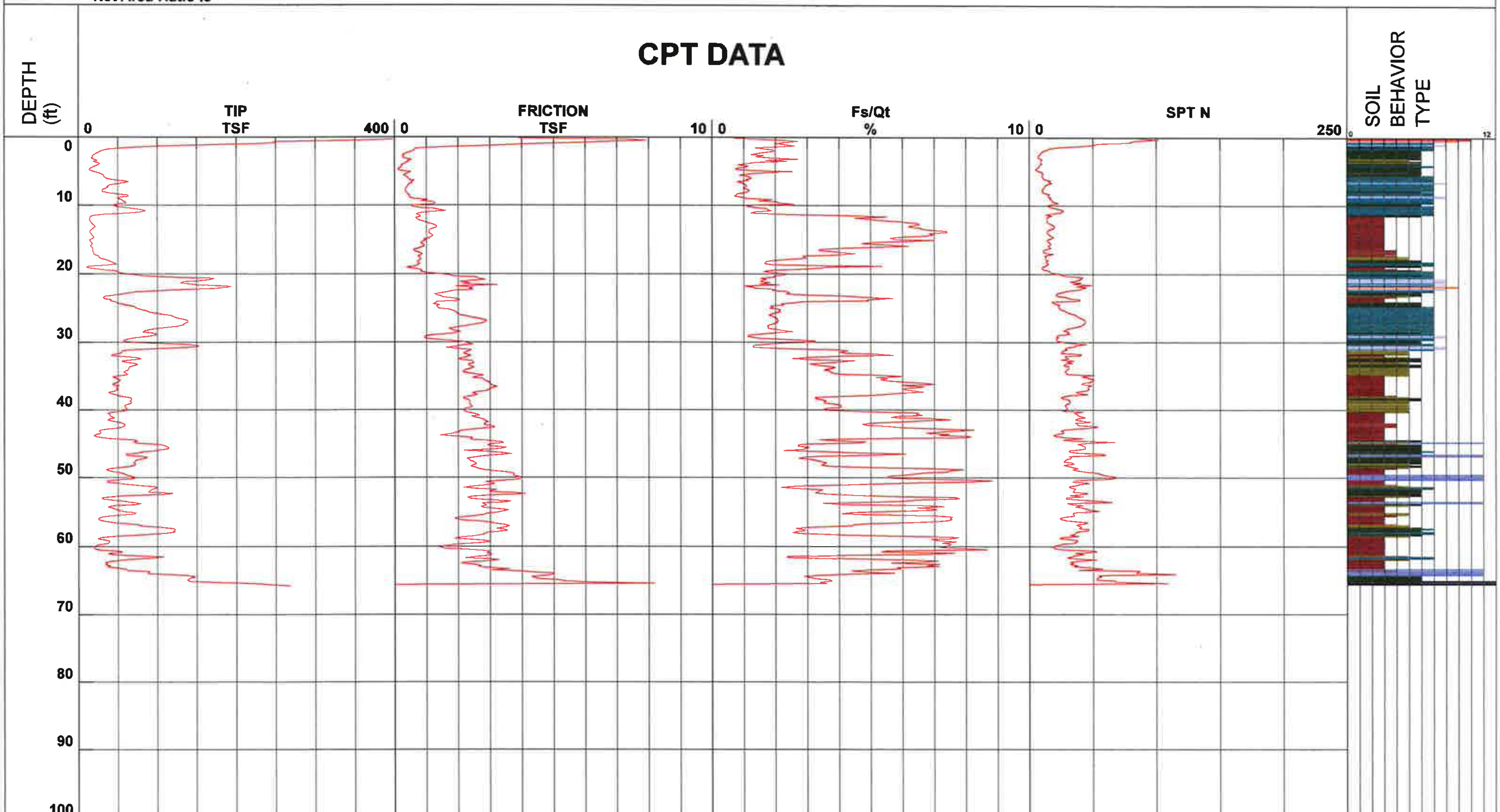
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(754).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-07	Date and Time	8/10/2016 9:32:38 PM	Maximum Depth	65.78 ft
EST GW Depth During Test		>65.78 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

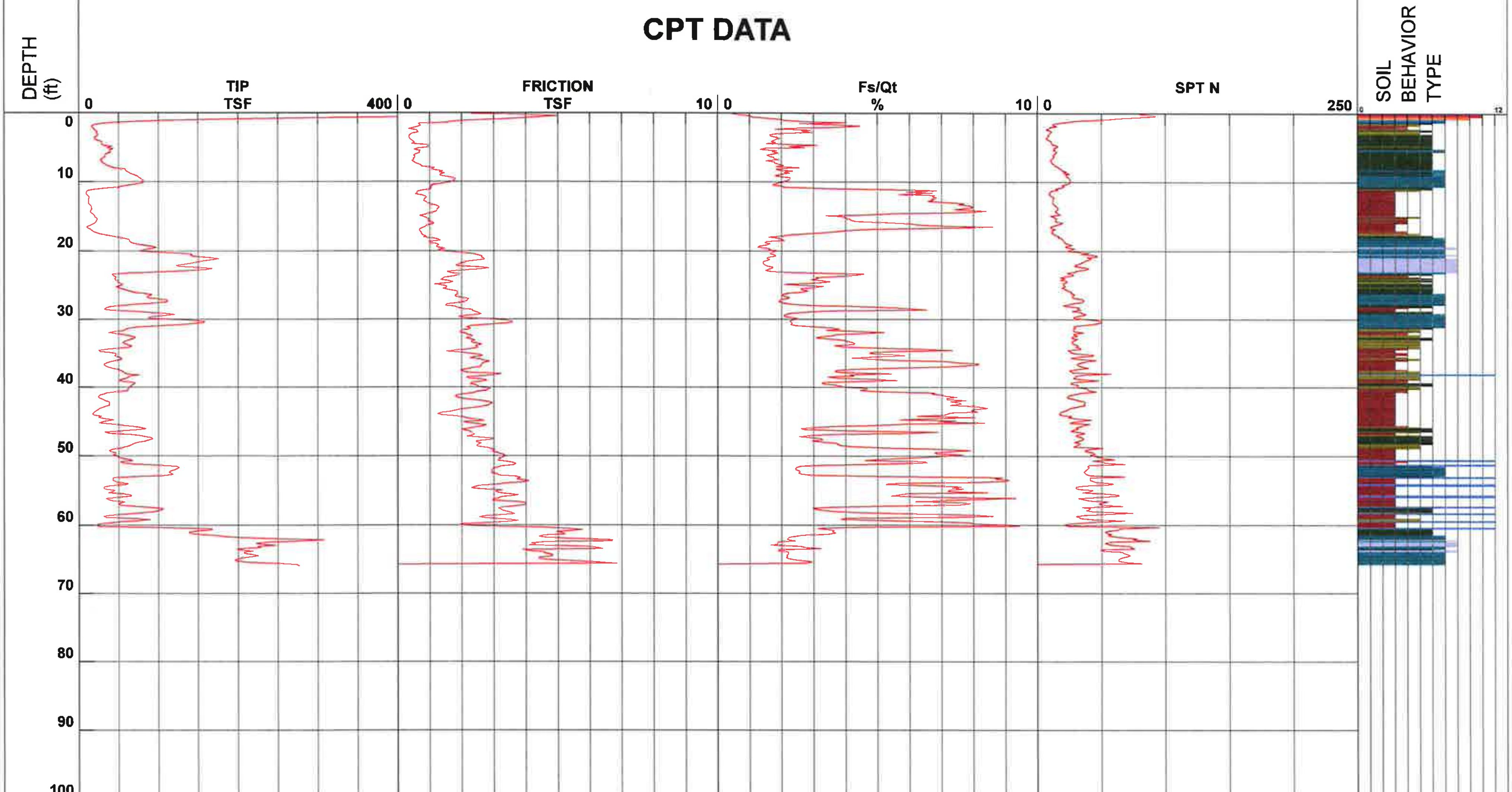
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(753).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-08	Date and Time	8/10/2016 8:38:12 PM	Maximum Depth	65.94 ft
EST GW Depth During Test		>65.94 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

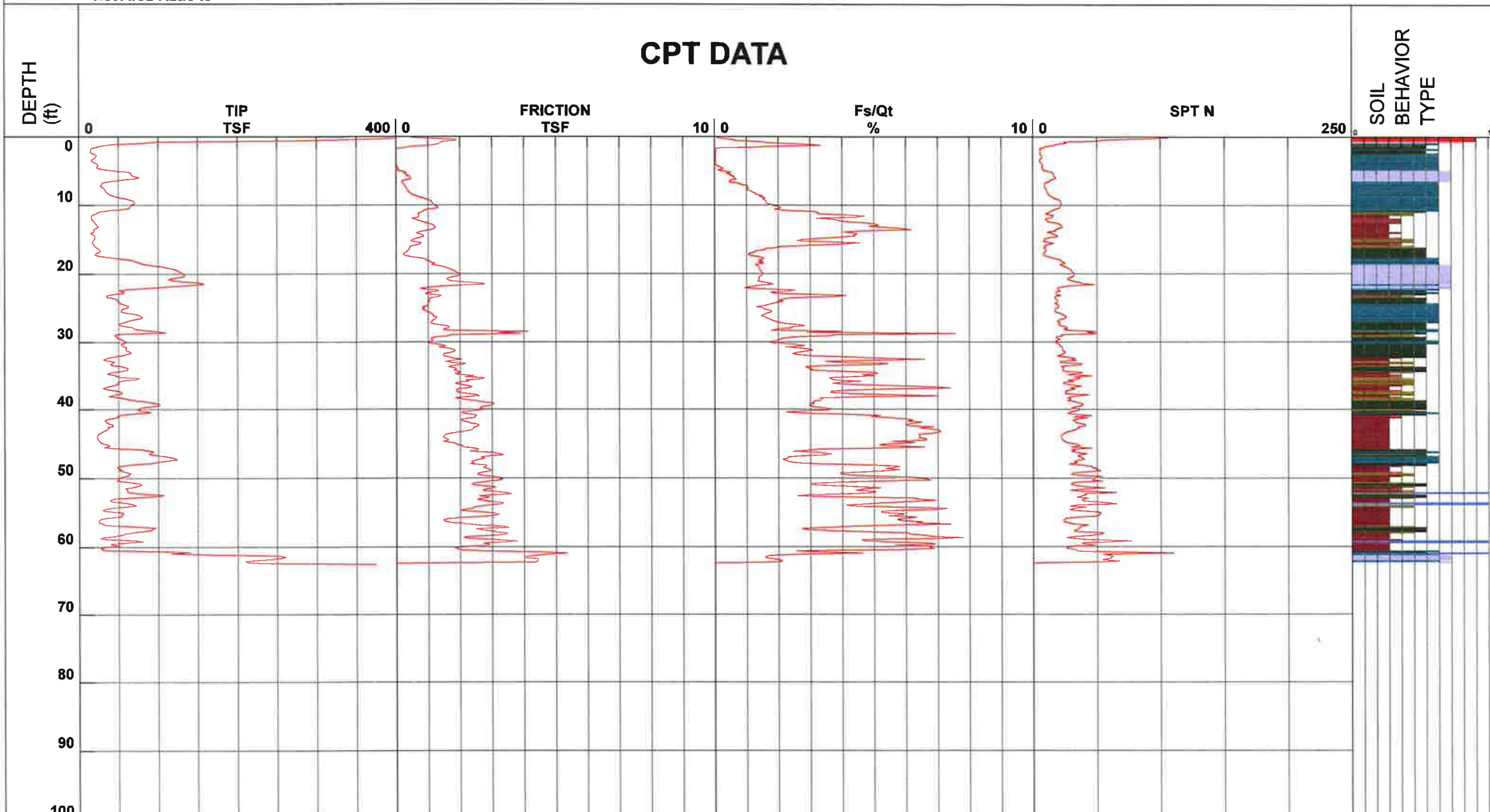
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(752).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-09	Date and Time	8/10/2016 7:44:31 PM	Maximum Depth	62.66 ft
EST GW Depth During Test		>62.66 ft			

Net Area Ratio .8



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

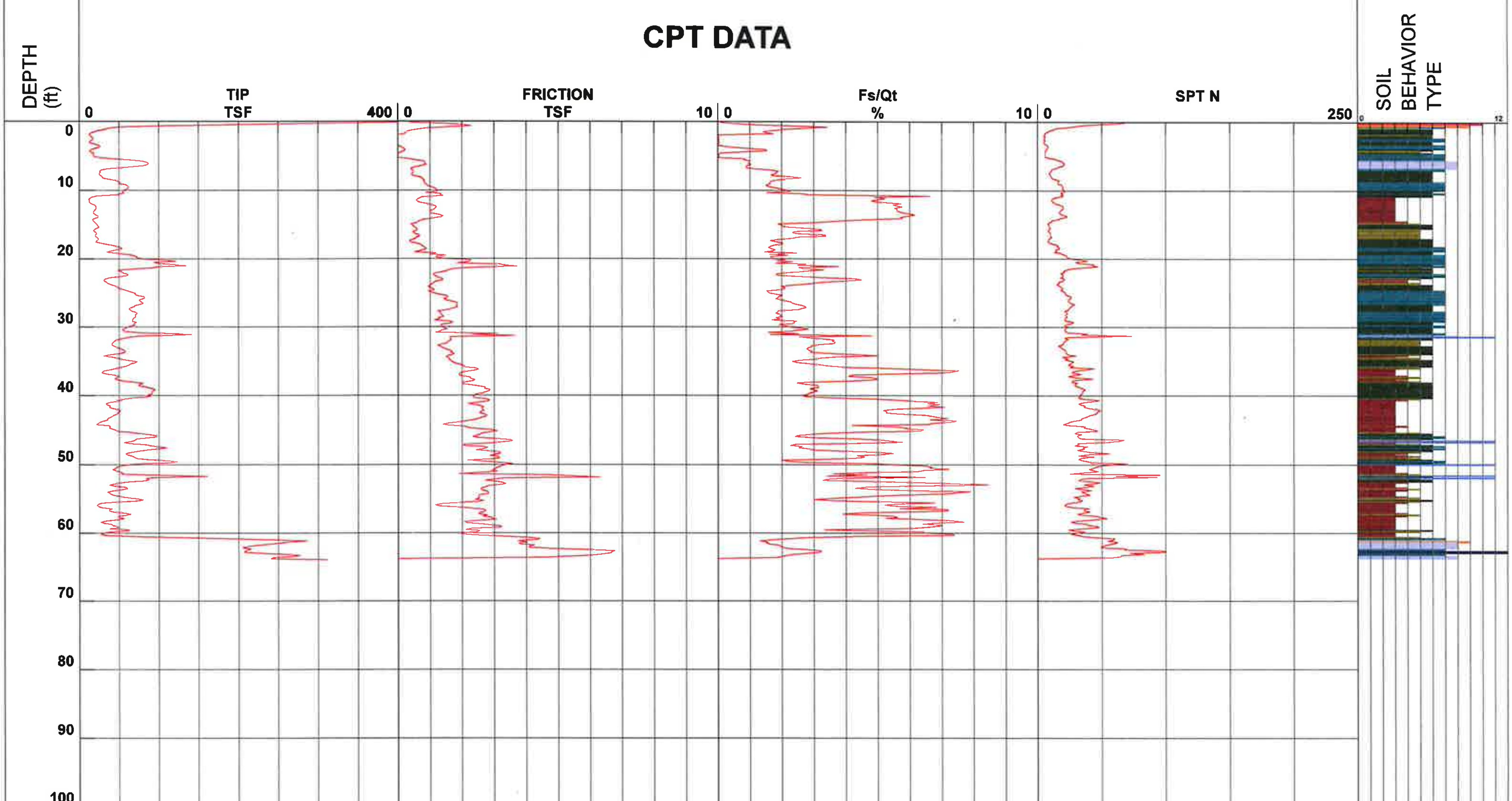
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(751).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-10	Date and Time	8/10/2016 6:52:57 PM	Maximum Depth	63.98 ft
EST GW Depth During Test	>63.98 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

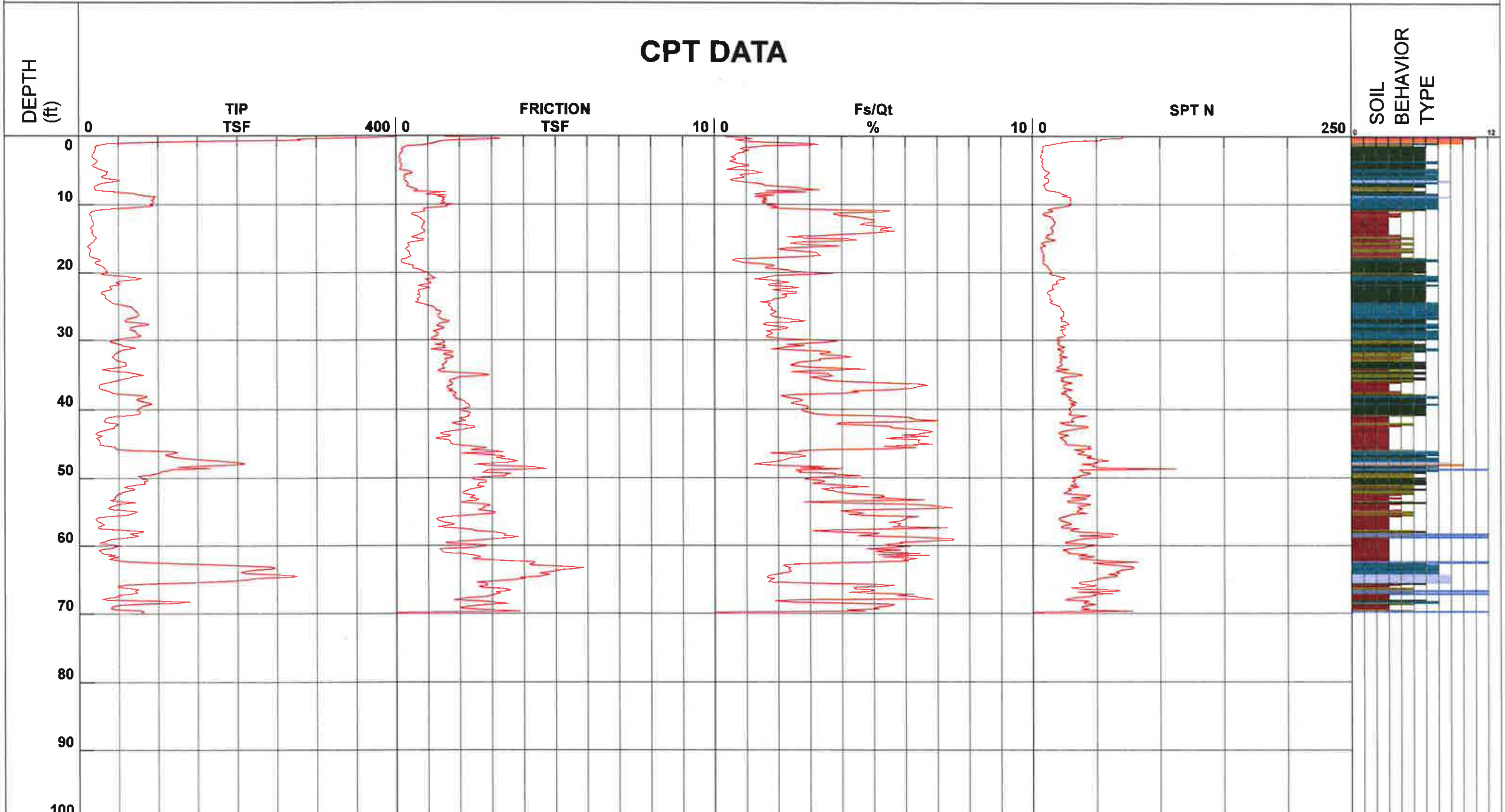
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(750).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-11	Date and Time	8/10/2016 6:09:02 PM	Maximum Depth	70.05 ft
EST GW Depth During Test	>70.05 ft				

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

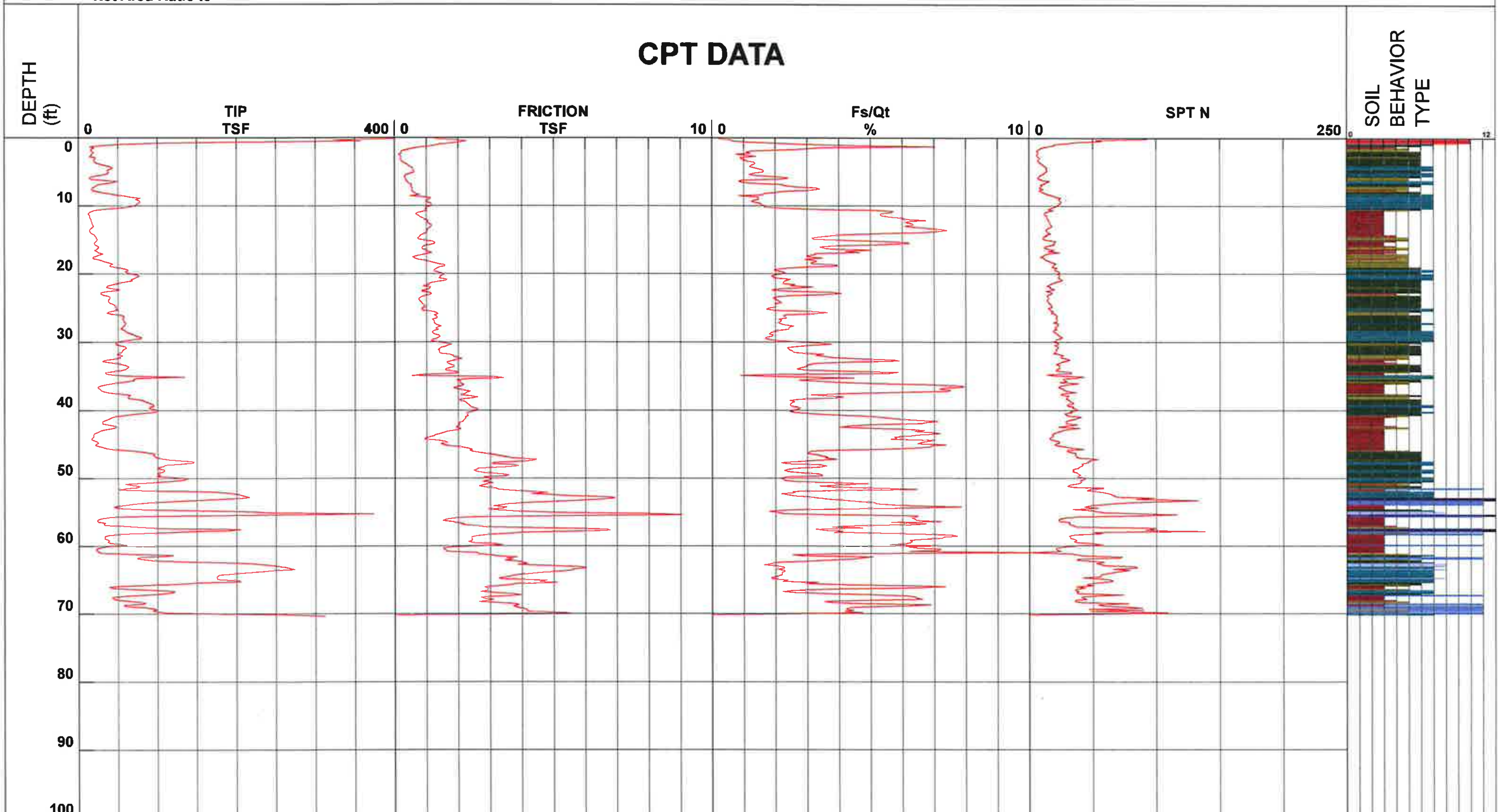
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(749).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-12	Date and Time	8/10/2016 5:24:45 PM	Maximum Depth	70.37 ft
EST GW Depth During Test	>70.37 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

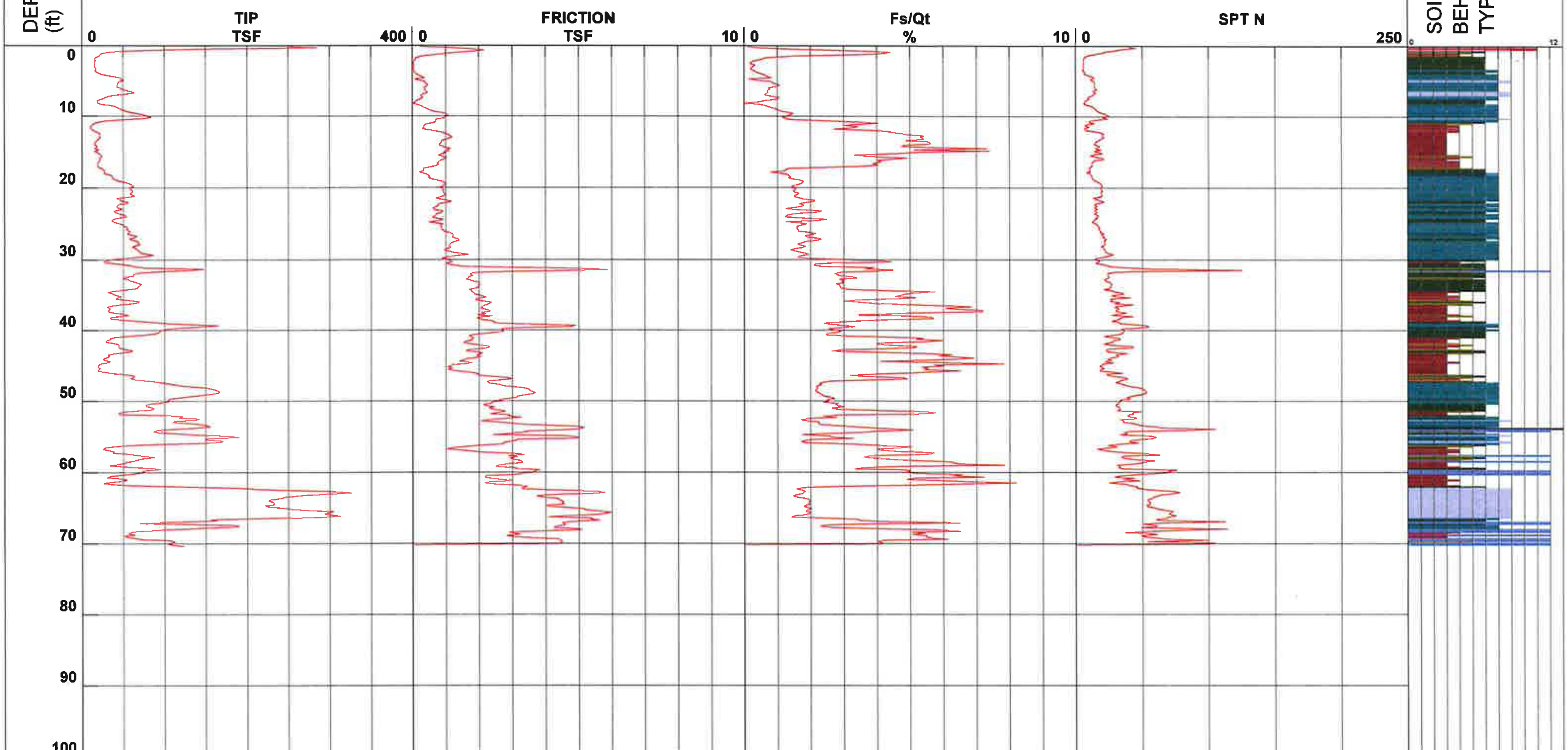
Middle Earth

Project	Hollywood and Wilcox	Operator	DG-RC
Job Number	LA-01670-01	Cone Number	DDG1281
Hole Number	CPT-13	Date and Time	8/10/2016 4:38:53 PM
EST GW Depth During Test	>70.37 ft		

Filename	SDF(748).cpt
GPS	
Maximum Depth	70.37 ft

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

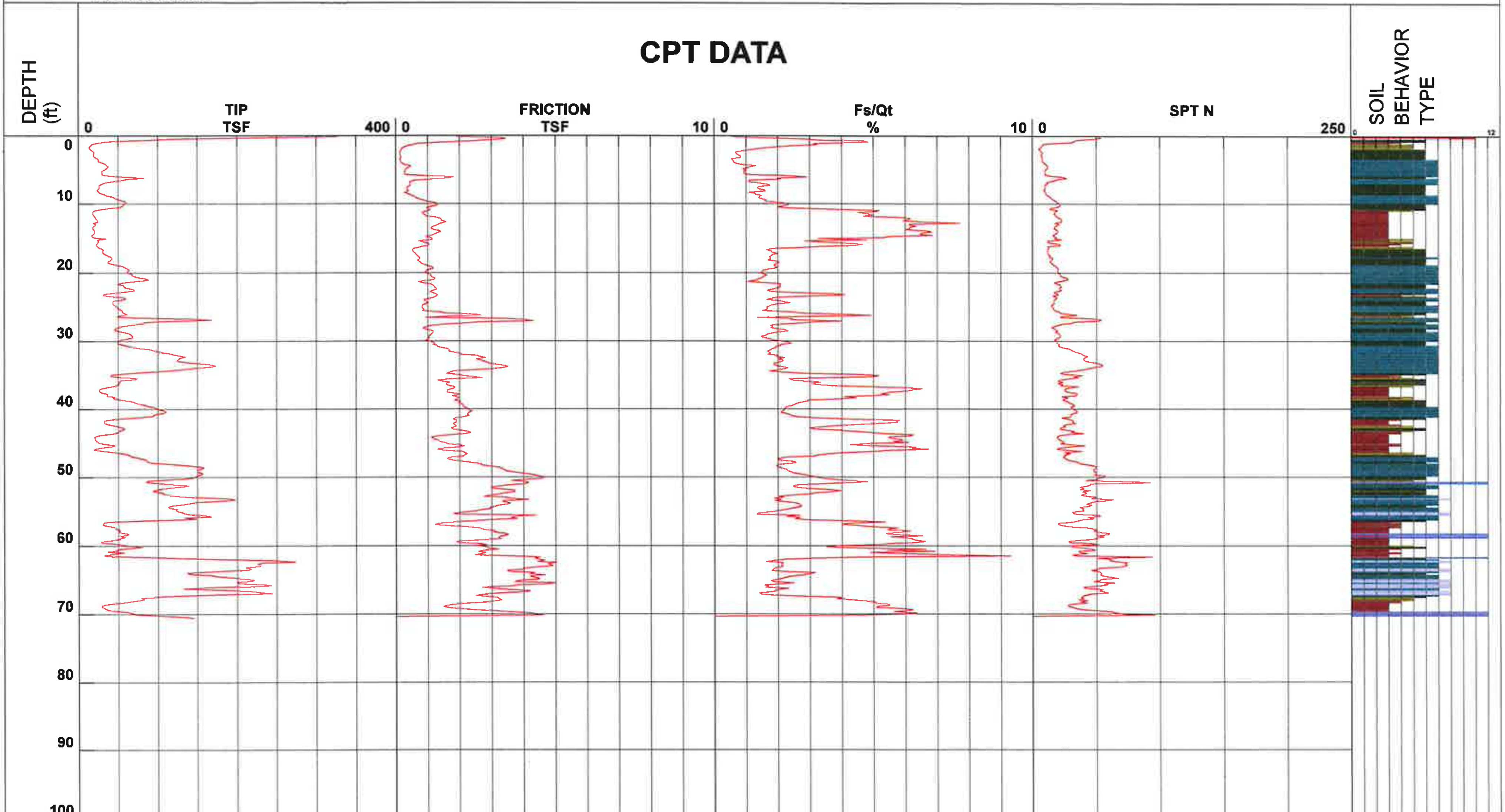
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(747).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-14	Date and Time	8/10/2016 3:56:30 PM	Maximum Depth	70.54 ft
EST GW Depth During Test		>70.54 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

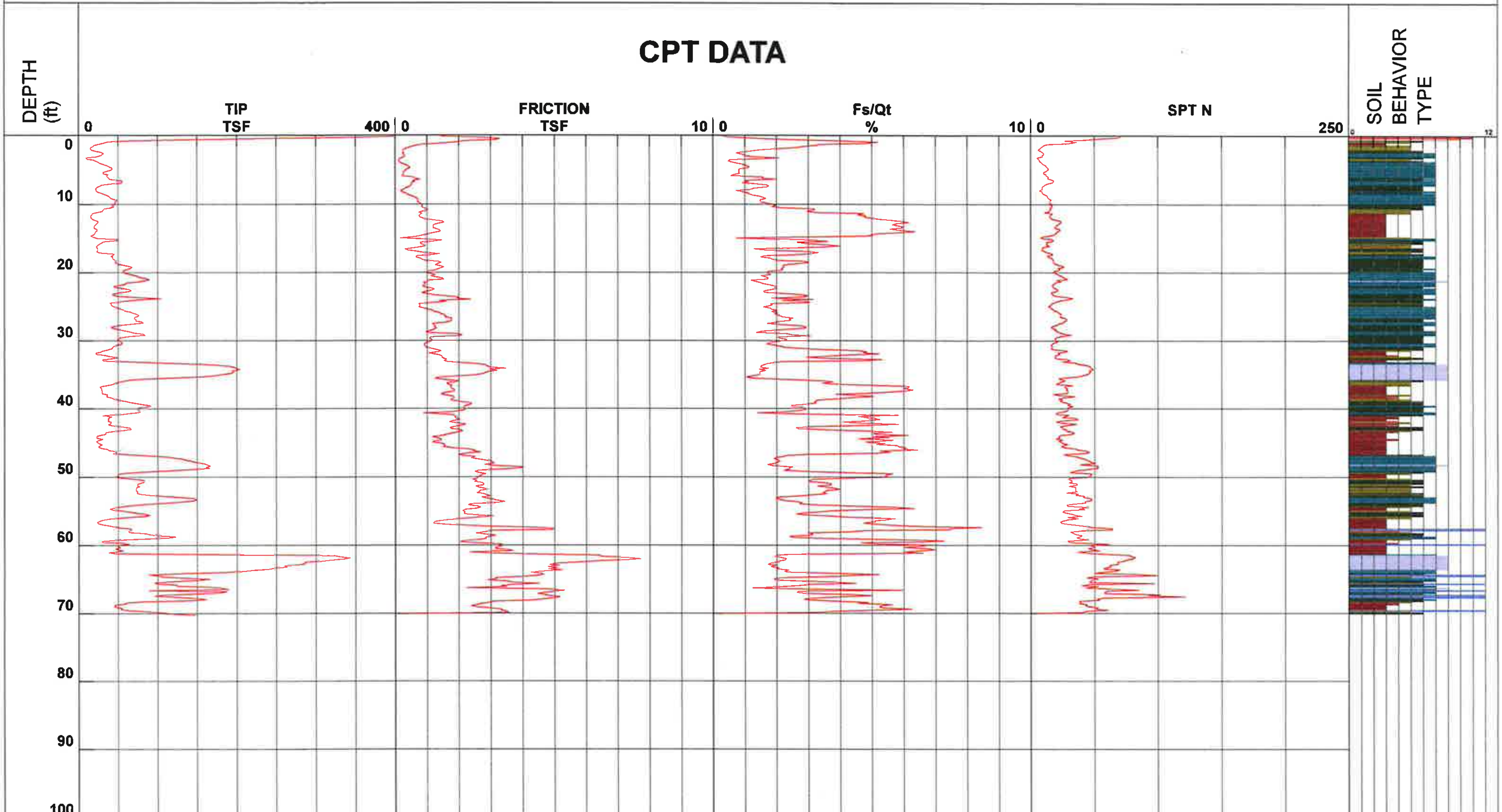
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(746).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-15	Date and Time	8/10/2016 3:11:14 PM	Maximum Depth	70.21 ft
EST GW Depth During Test		>70.21 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

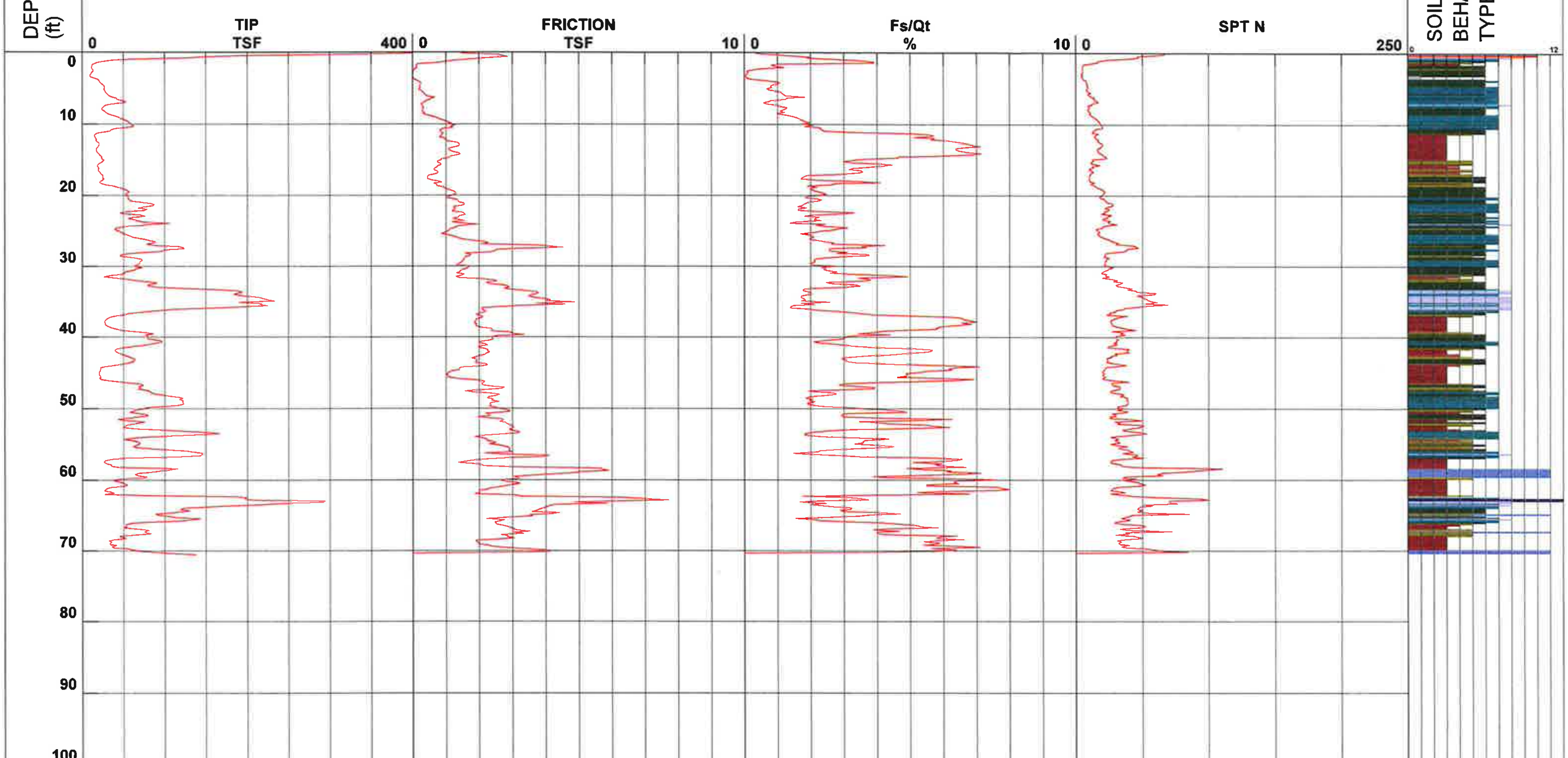
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(745).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-16	Date and Time	8/10/2016 2:29:05 PM	Maximum Depth	70.54 ft
EST GW Depth During Test		>70.54 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

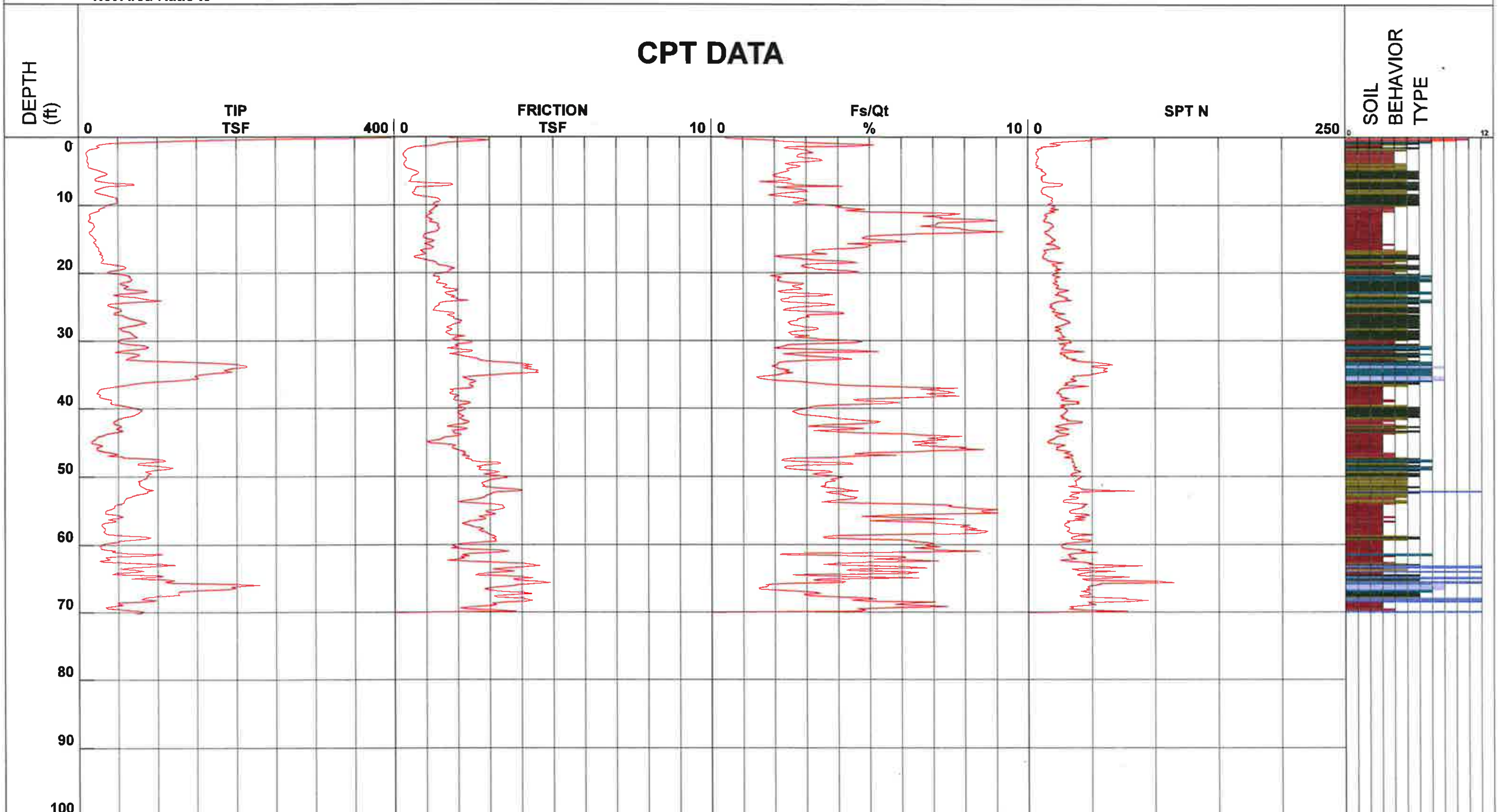
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(744).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-17	Date and Time	8/10/2016 1:45:52 PM	Maximum Depth	70.21 ft
EST GW Depth During Test		>70.21 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

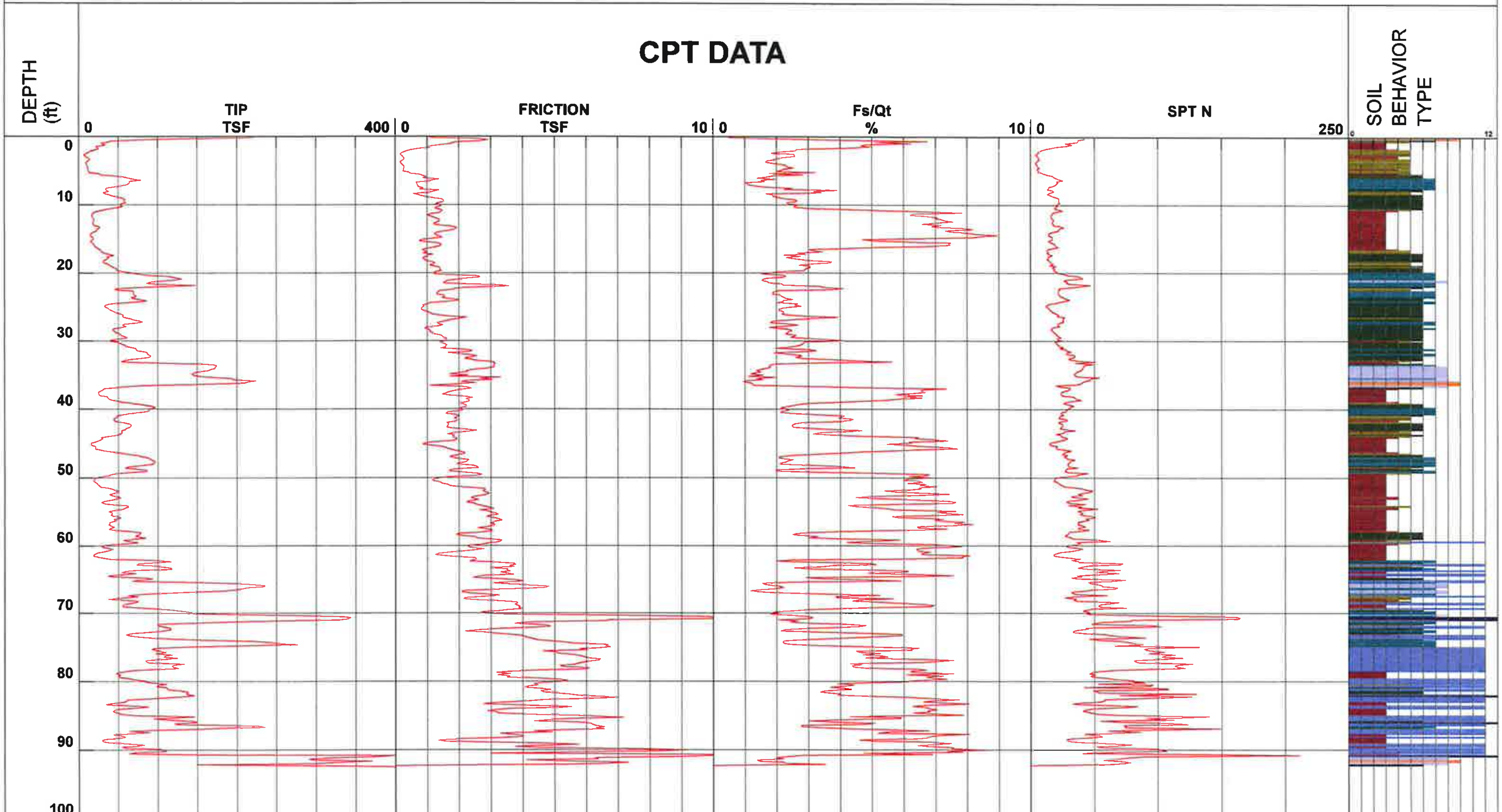
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Middle Earth

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(737).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	SCPT-18	Date and Time	8/10/2016 6:52:15 AM	Maximum Depth	92.52 ft
EST GW Depth During Test	>92.52 ft				

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

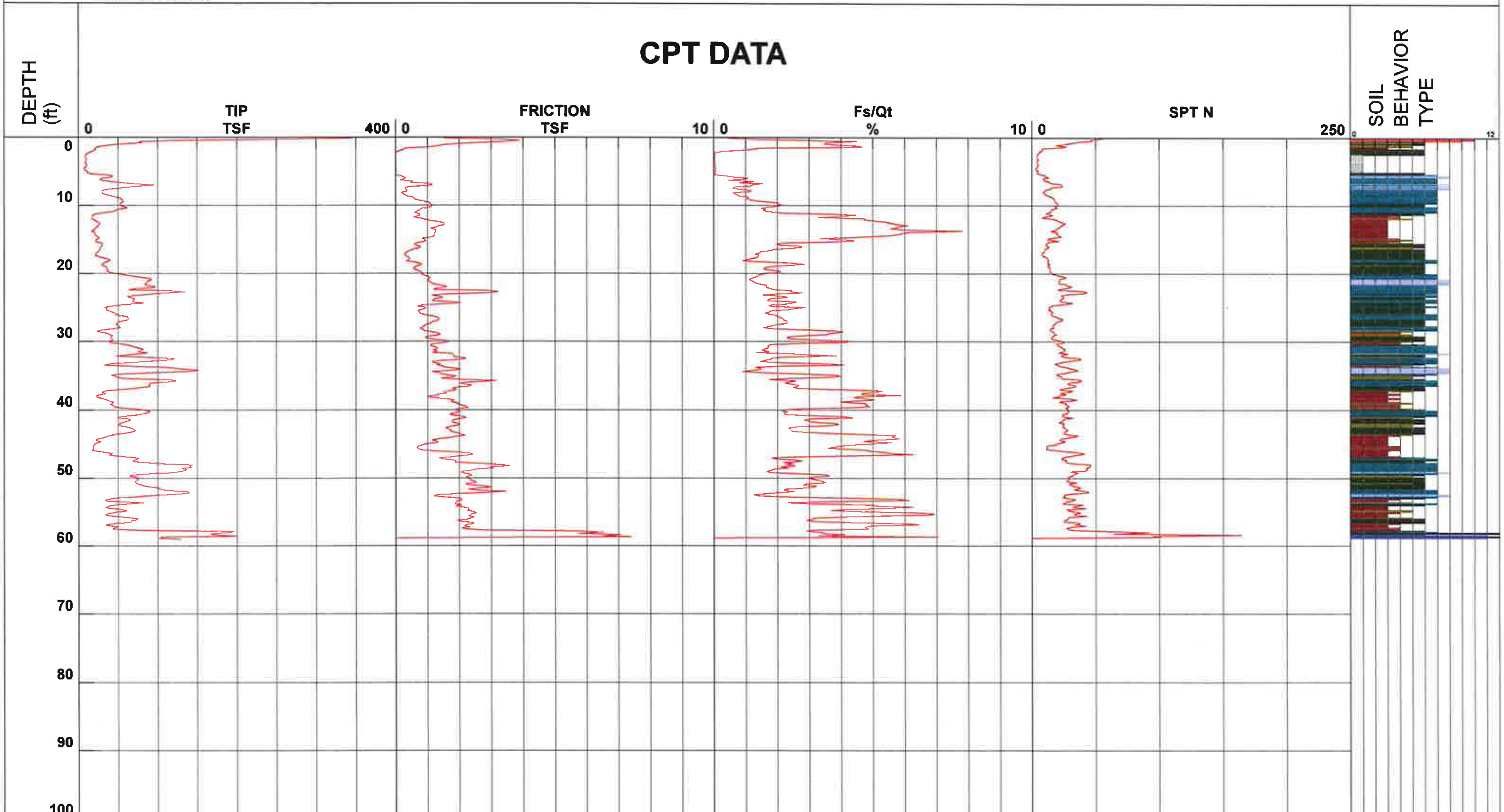
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Middle Earth

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(743).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-19	Date and Time	8/10/2016 1:06:44 PM	Maximum Depth	59.05 ft
EST GW Depth During Test	>59.05 ft				

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

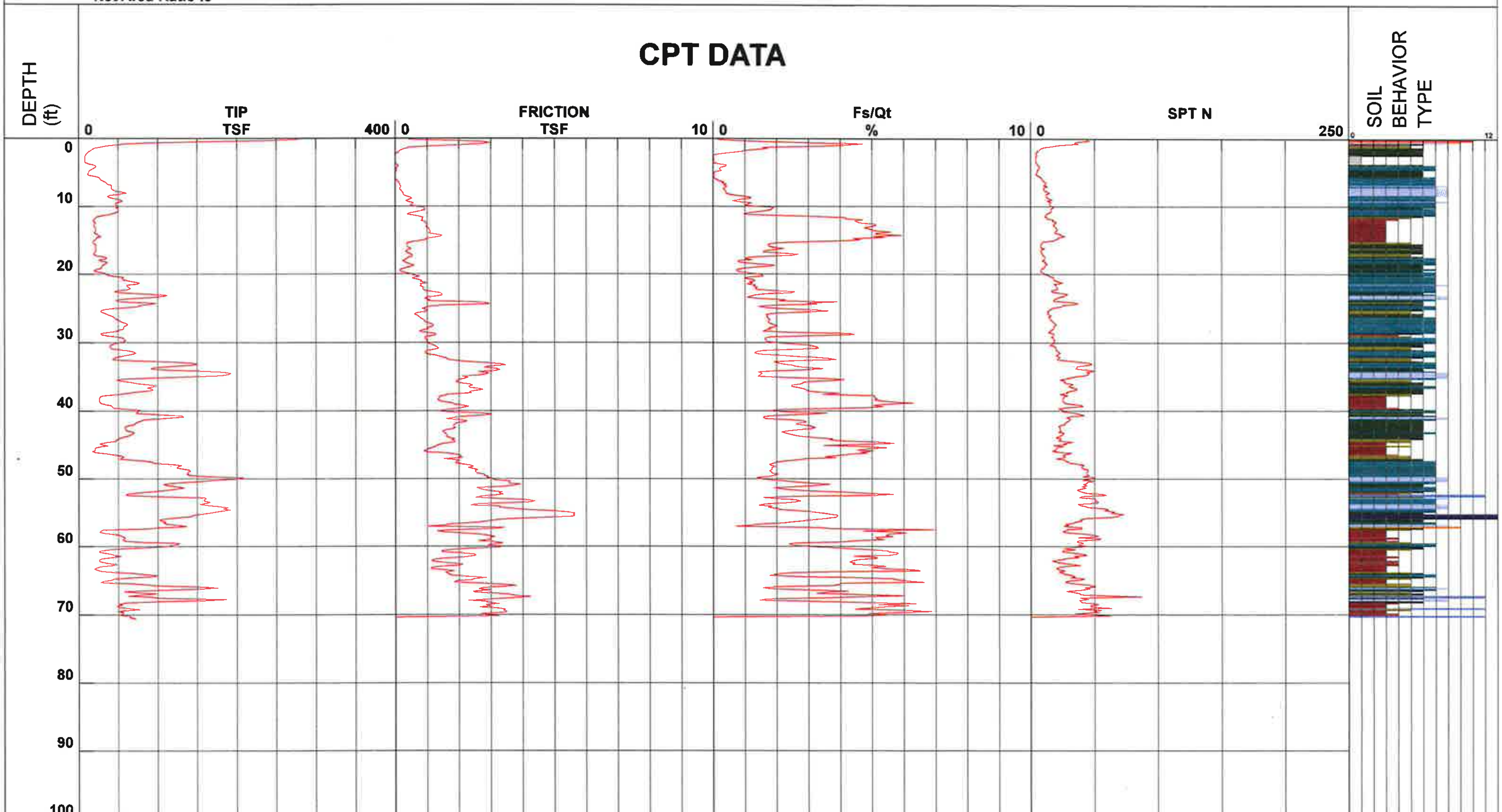
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(742).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-20	Date and Time	8/10/2016 12:14:52 PM	Maximum Depth	70.54 ft
EST GW Depth During Test	>70.54 ft				

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

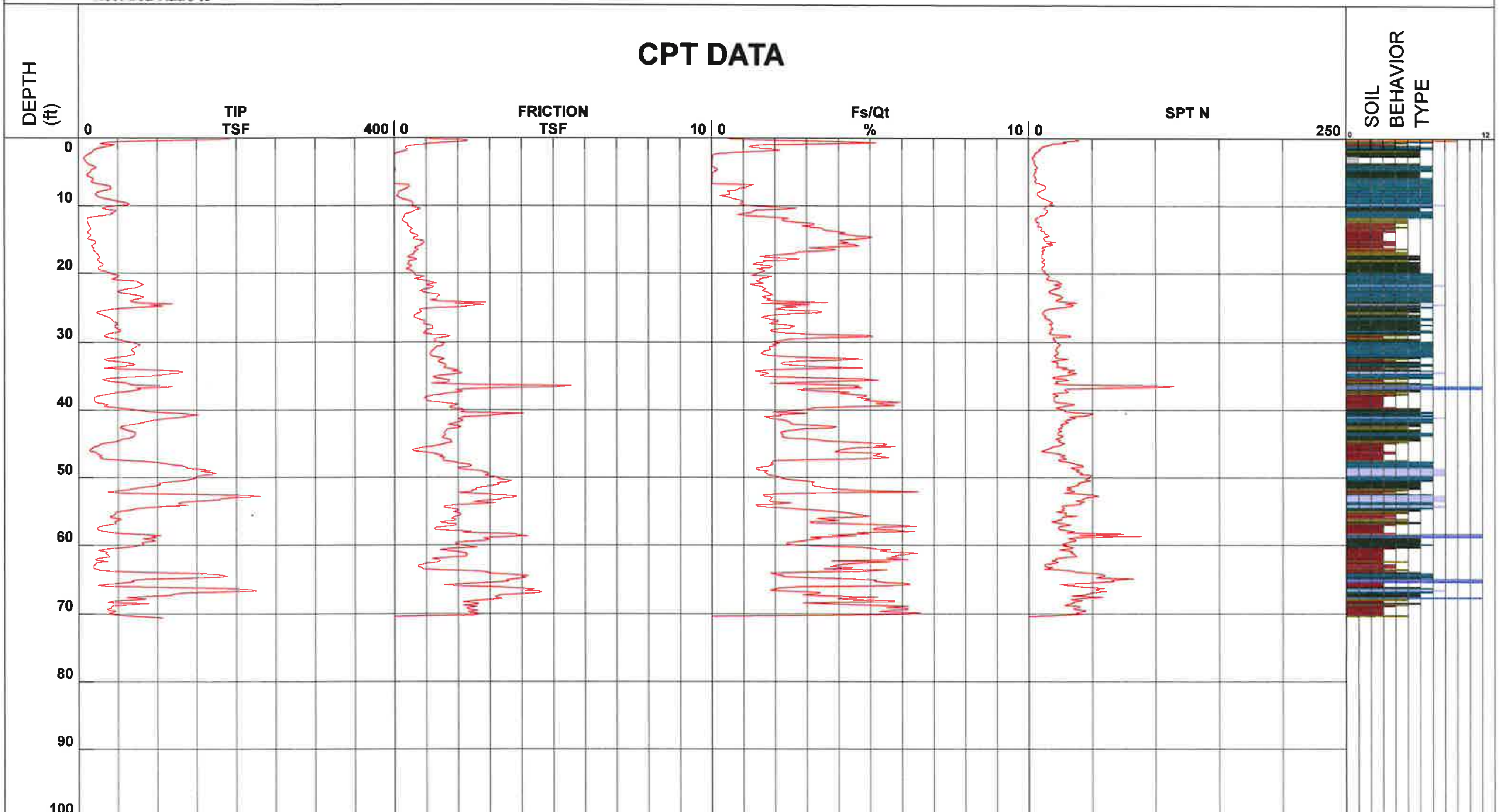
S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC
Job Number	LA-01670-01	Cone Number	DDG1281
Hole Number	CPT-21	Date and Time	8/10/2016 11:27:50 AM
EST GW Depth During Test			>70.54 ft

Filename	SDF(741).cpt
GPS	
Maximum Depth	70.54 ft

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

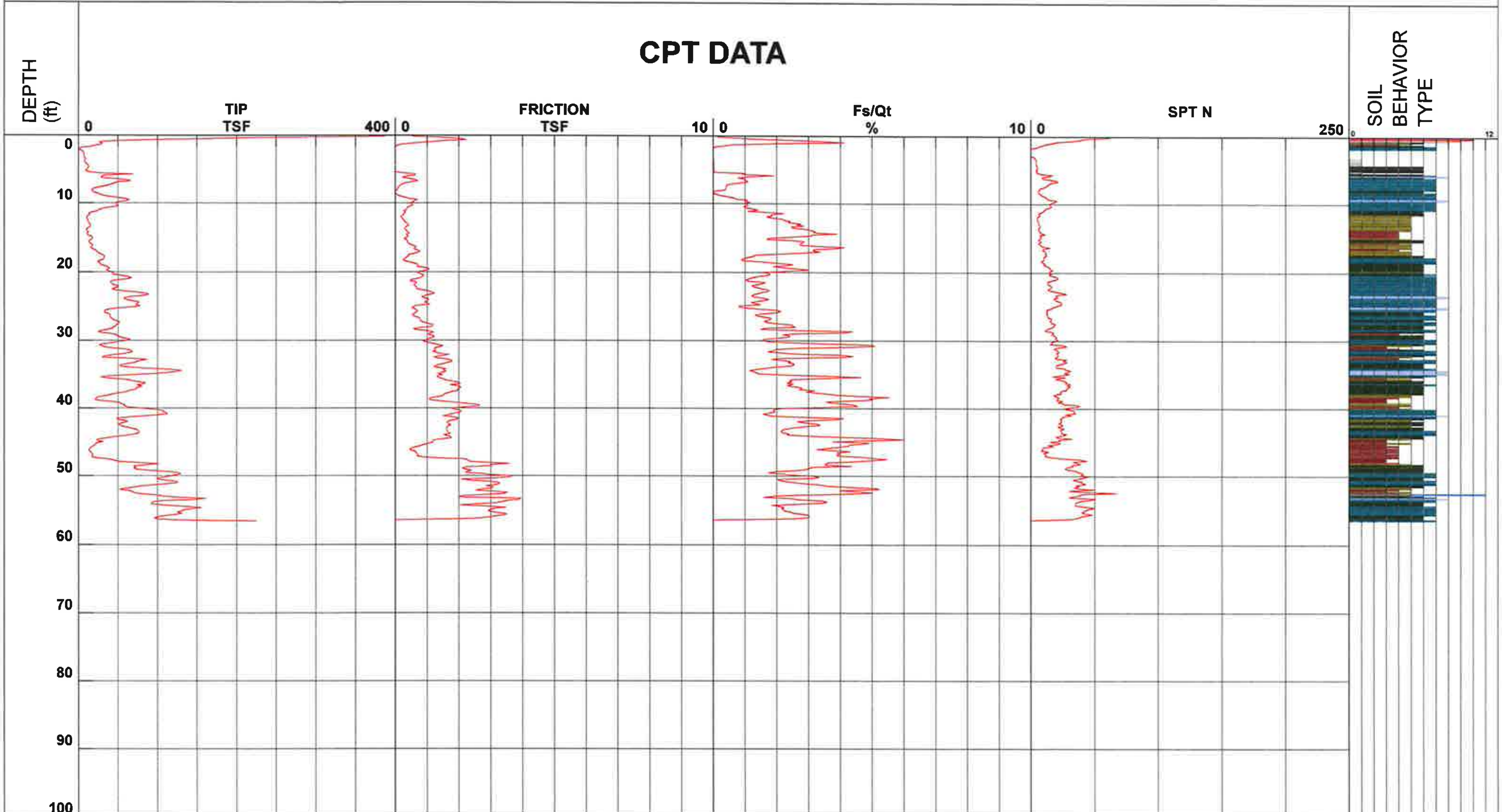
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Earth Systems

Project	Hollywood and Wilcox	Operator	DG-RC	Filename	SDF(740).cpt
Job Number	LA-01670-01	Cone Number	DDG1281	GPS	
Hole Number	CPT-22	Date and Time	8/10/2016 10:48:29 AM	Maximum Depth	56.59 ft
EST GW Depth During Test	>56.59 ft				

Net Area Ratio .8



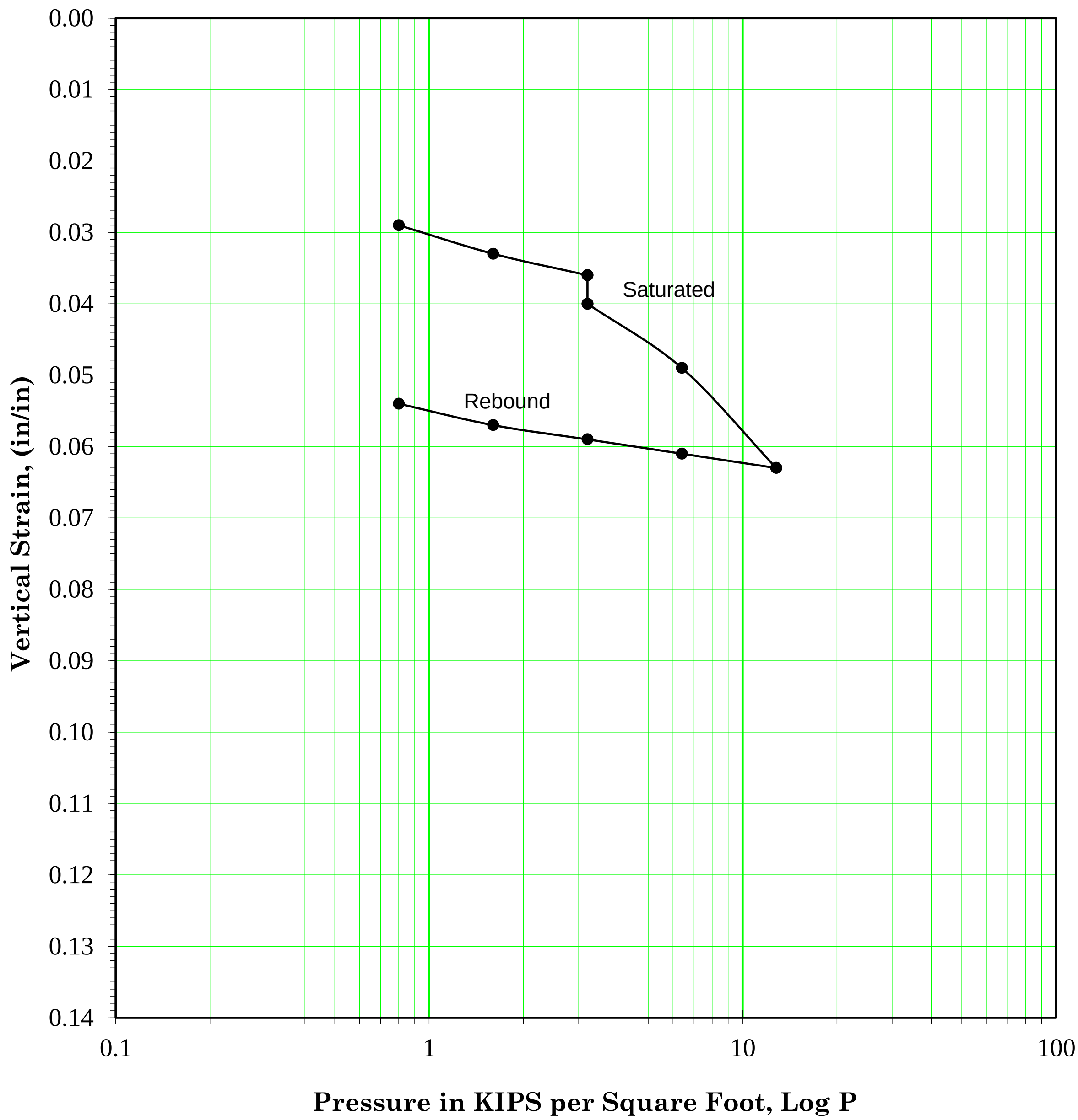
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

Laboratory Test Results



Sample Location:	B2 at 30 feet
Material:	Alluvium; Clayey SAND (SC)
Initial Dry Density:	118.3 pcf
Moisture Content:	15.0%
Hydroconsolidation:	0.4%

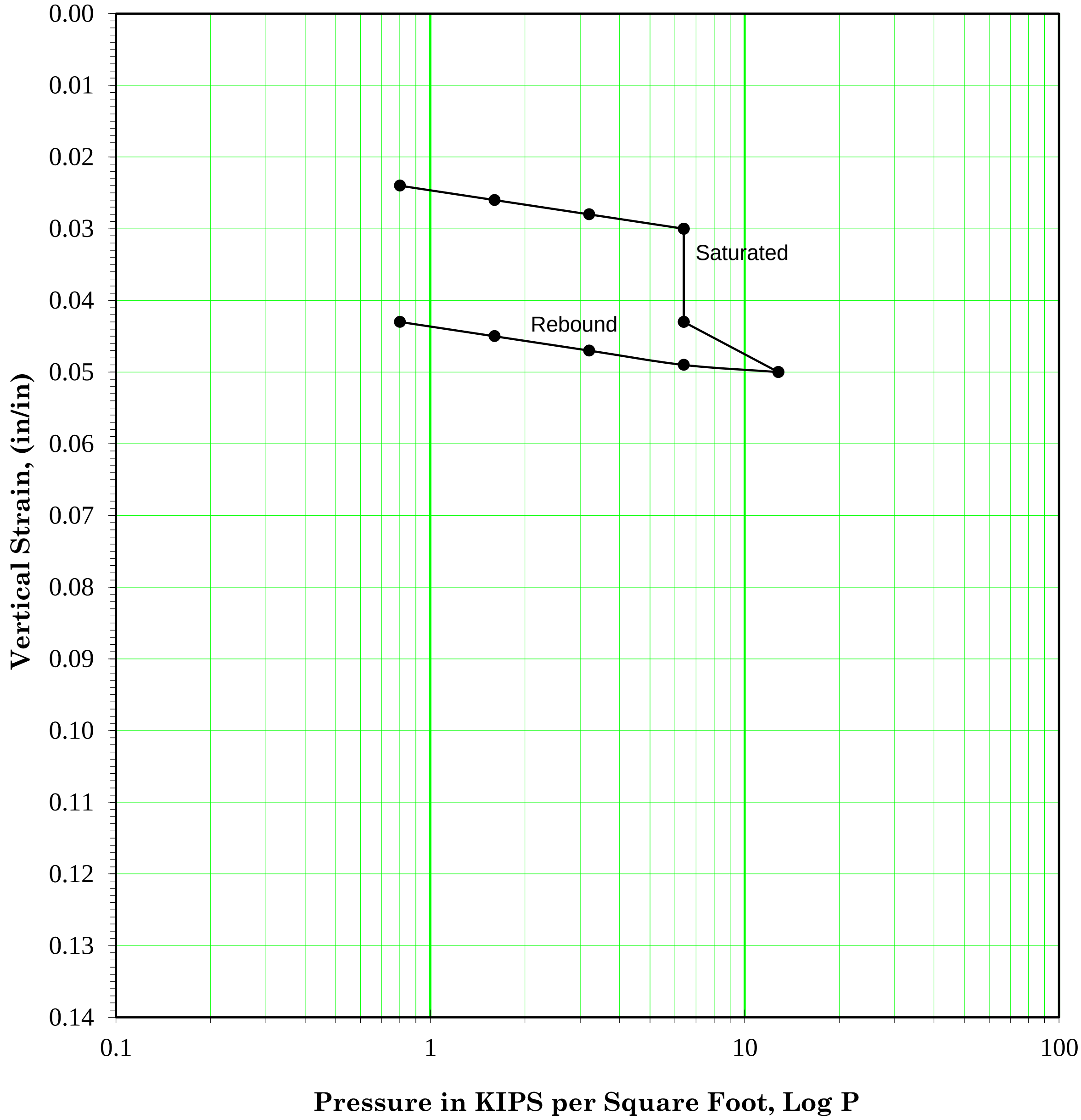
* Test Method: ASTM D-2435

CONSOLIDATION TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01



Sample Location:	B2 at 50 feet
Material:	Alluvium; Silty SAND (SM)
Initial Dry Density:	120.5 pcf
Moisture Content:	7.1%
Hydroconsolidation:	1.3%

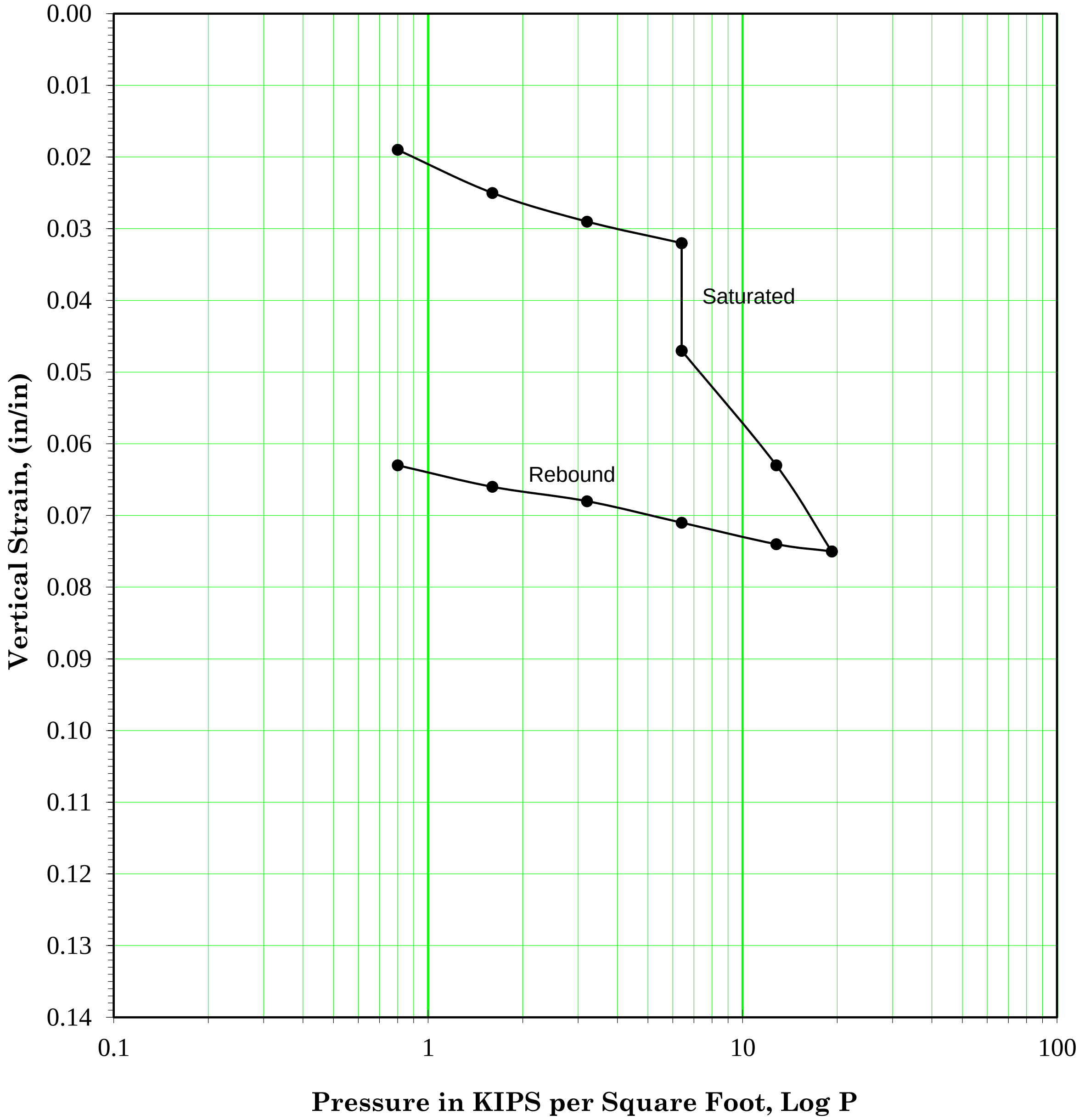
* Test Method: ASTM D-2435

CONSOLIDATION TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016 | LA-01670-01



Sample Location:	B4 at 60 feet
Material:	Alluvium; Well-graded SAND (SW)
Initial Dry Density:	122.4 pcf
Moisture Content:	8.4%
Hydroconsolidation:	1.5%

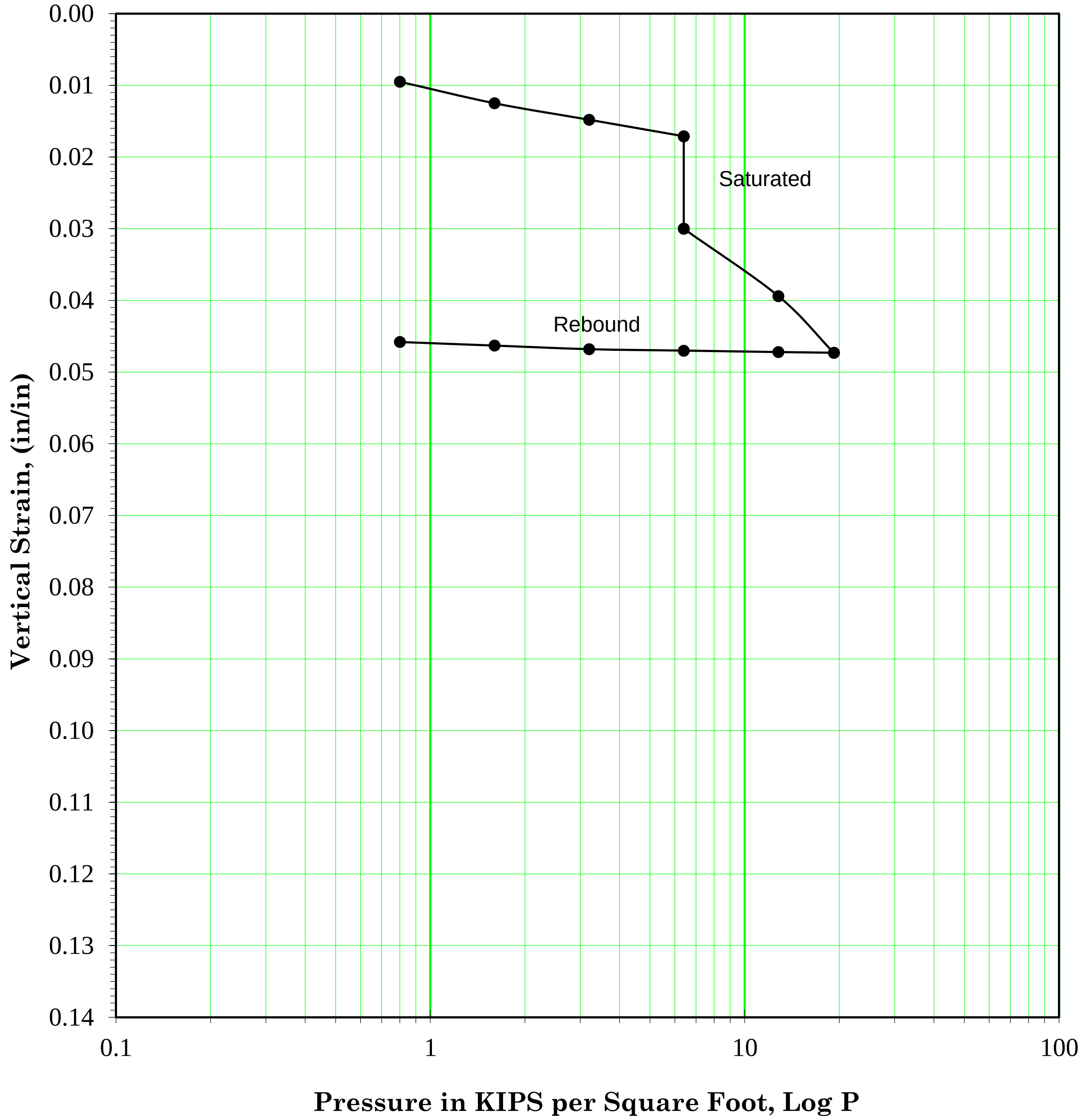
* Test Method: ASTM D-2435

CONSOLIDATION TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016 | LA-01670-01



Sample Location:	B4 at 70 feet
Material:	Alluvium; Clayey SAND (SC)
Initial Dry Density:	121.7 pcf
Moisture Content:	13.5%
Hydroconsolidation:	1.3%

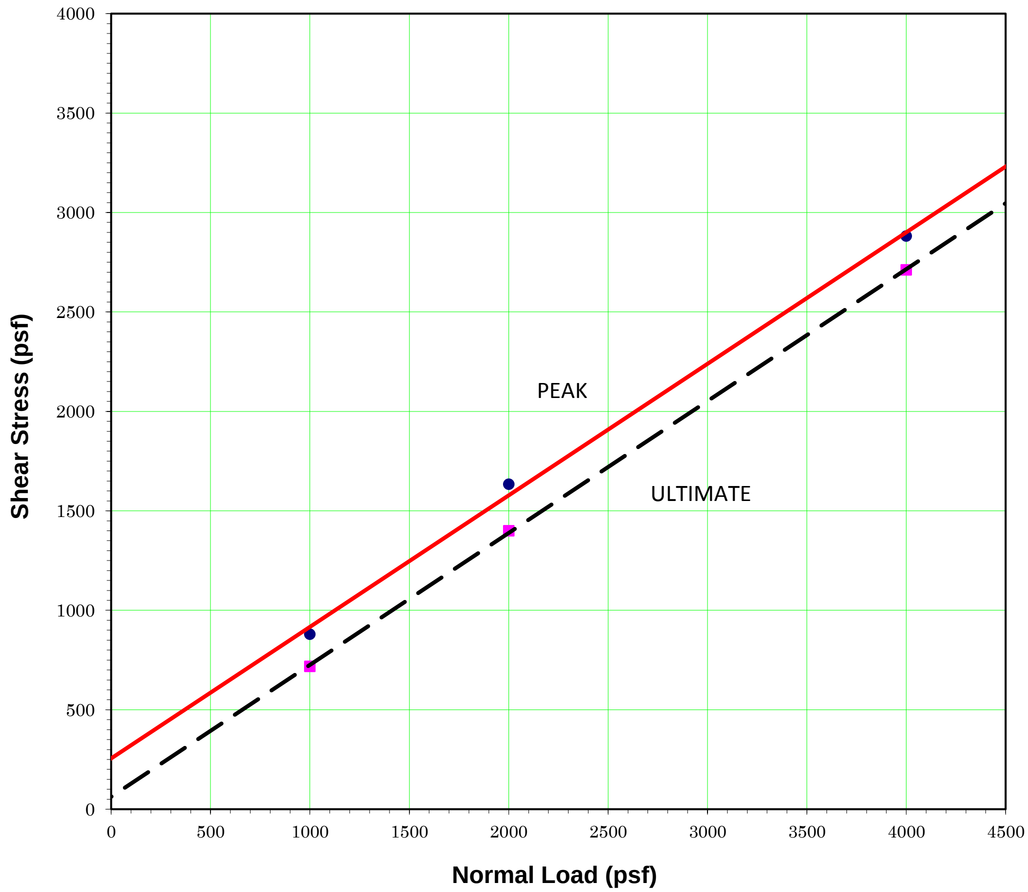
* Test Method: ASTM D-2435

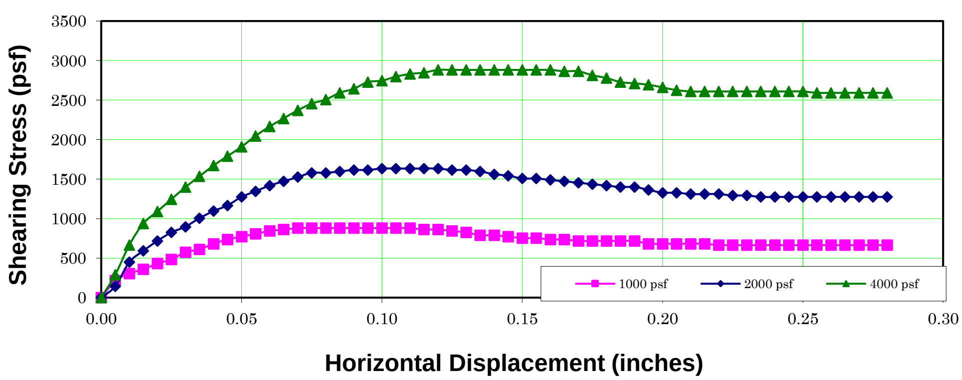
CONSOLIDATION TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016 | LA-01670-01





DIRECT SHEAR DATA*

Sample Location:	B4 at 7.5 feet	
Material:	Alluvium; Well-graded SAND (SW)	
Dry Density:	105.9 pcf	
	Initial	Final
Moisture Content:	5.5%	21.2%
Saturation:	26%	100%
	Peak	Ultimate
ϕ Angle of Friction (degrees):	33	34
c Cohesive Strength (psf):	250	60

Test Type: Peak and Ultimate Shear Rate (in/min): 0.005

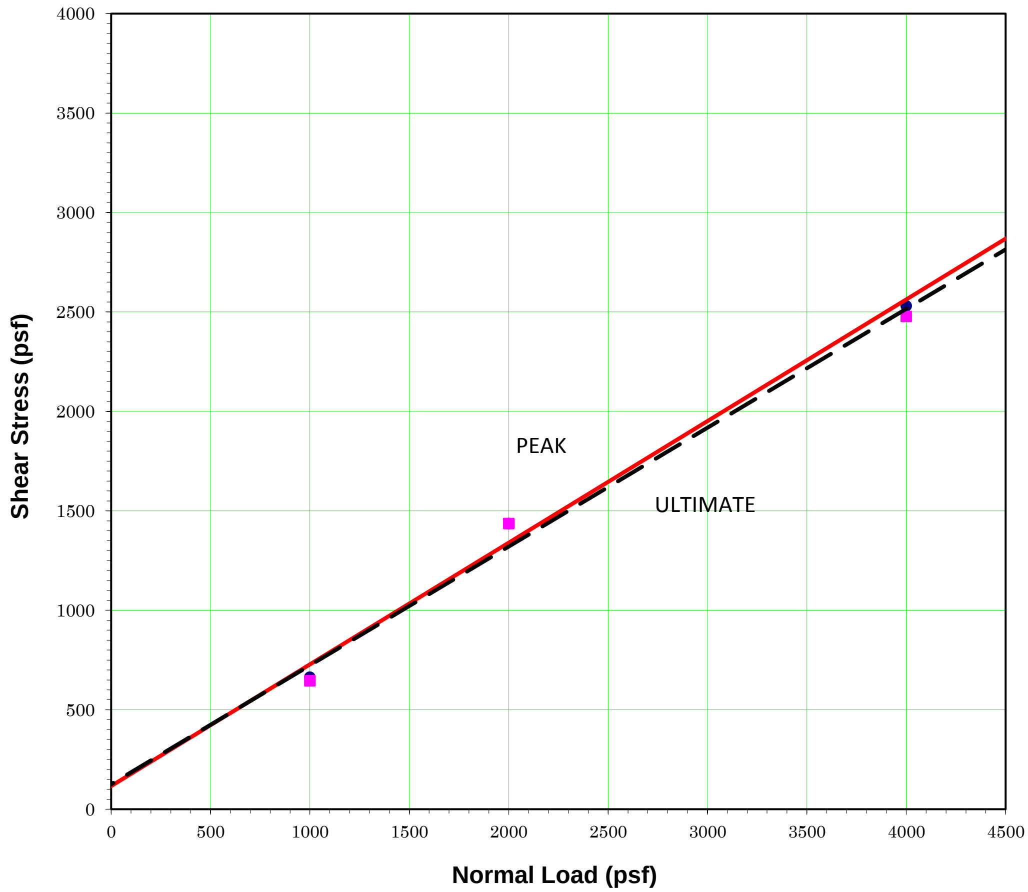
* Test Method: ASTM D-3080

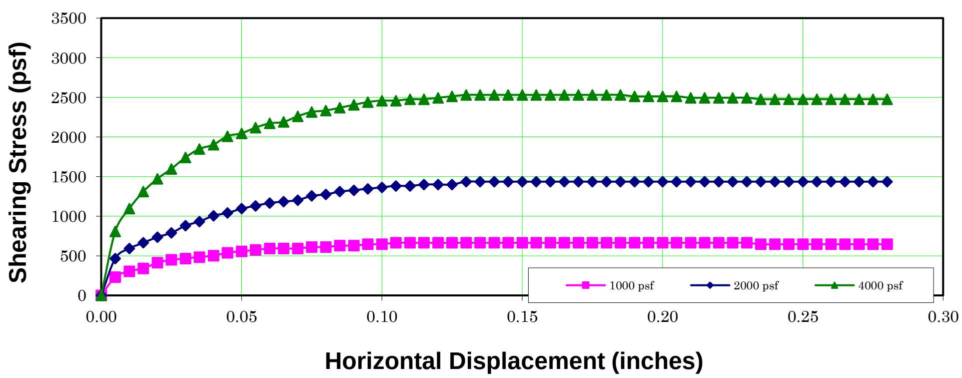
DIRECT SHEAR TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016 LA-01670-01





DIRECT SHEAR DATA*

Sample Location: B4 at 30 feet
Material: Alluvium; Silty SAND (SM)
Dry Density: 114.8 pcf

	Initial	Final
Moisture Content:	14.4%	16.6%
Saturation:	87%	100%
	Peak	Ultimate
ϕ Angle of Friction (degrees):	31	31
c Cohesive Strength (psf):	110	120

Test Type: Peak and Ultimate   Shear Rate (in/min): 0.005

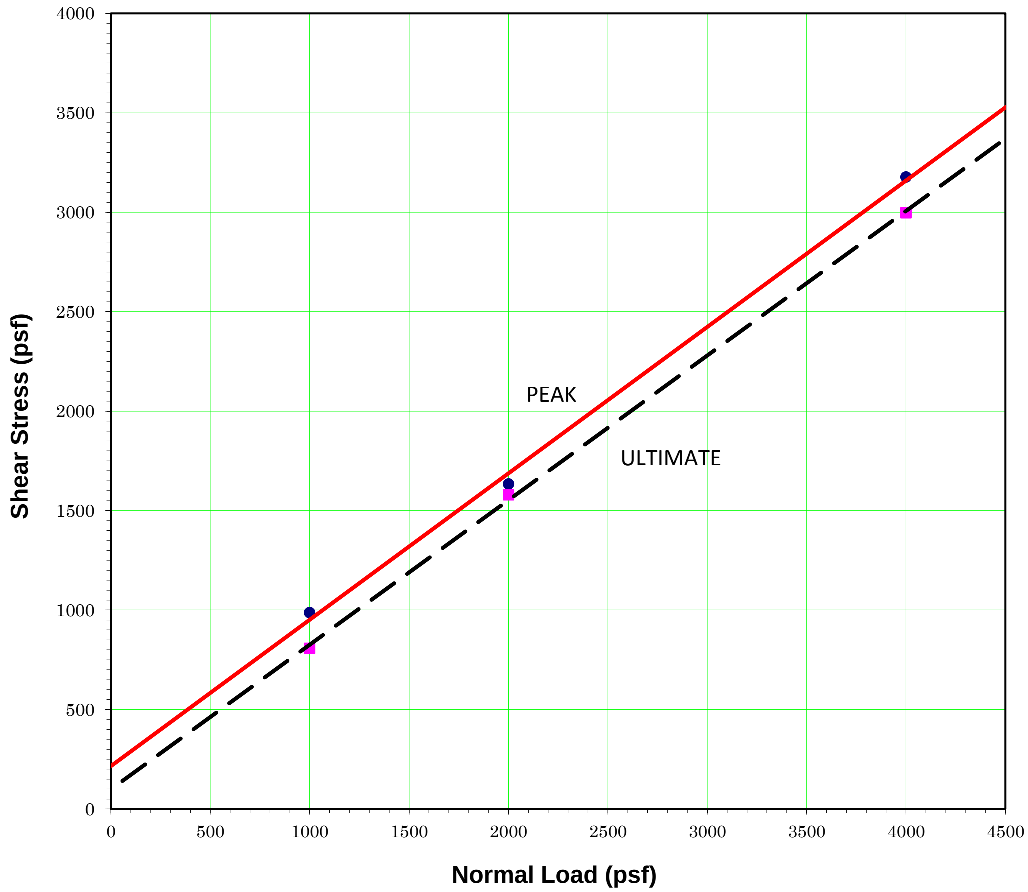
* Test Method: ASTM D-3080

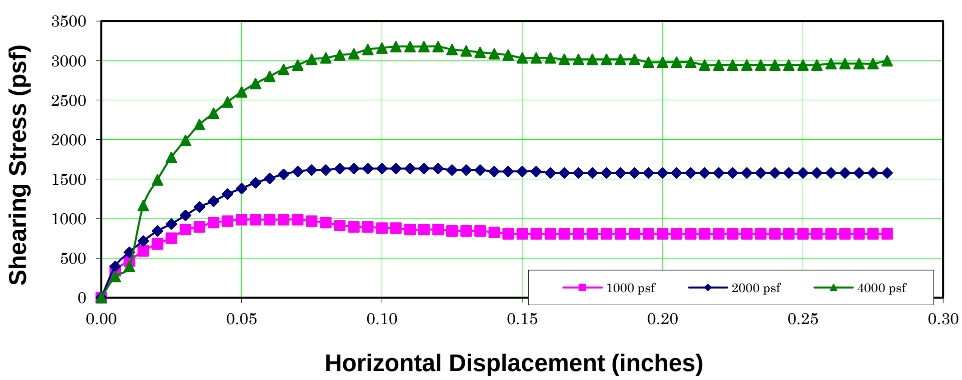
DIRECT SHEAR TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01





DIRECT SHEAR DATA*

Sample Location:	B2 at 70 feet	
Material:	Alluvium; Sandy CLAY (SC)	
Dry Density:	120.6 pcf	
	Initial	Final
Moisture Content:	13.6%	14.0%
Saturation:	97%	100%
	Peak	Ultimate
ϕ Angle of Friction (degrees):	36	36
c Cohesive Strength (psf):	210	90

Test Type: Peak and Ultimate Shear Rate (in/min): 0.005

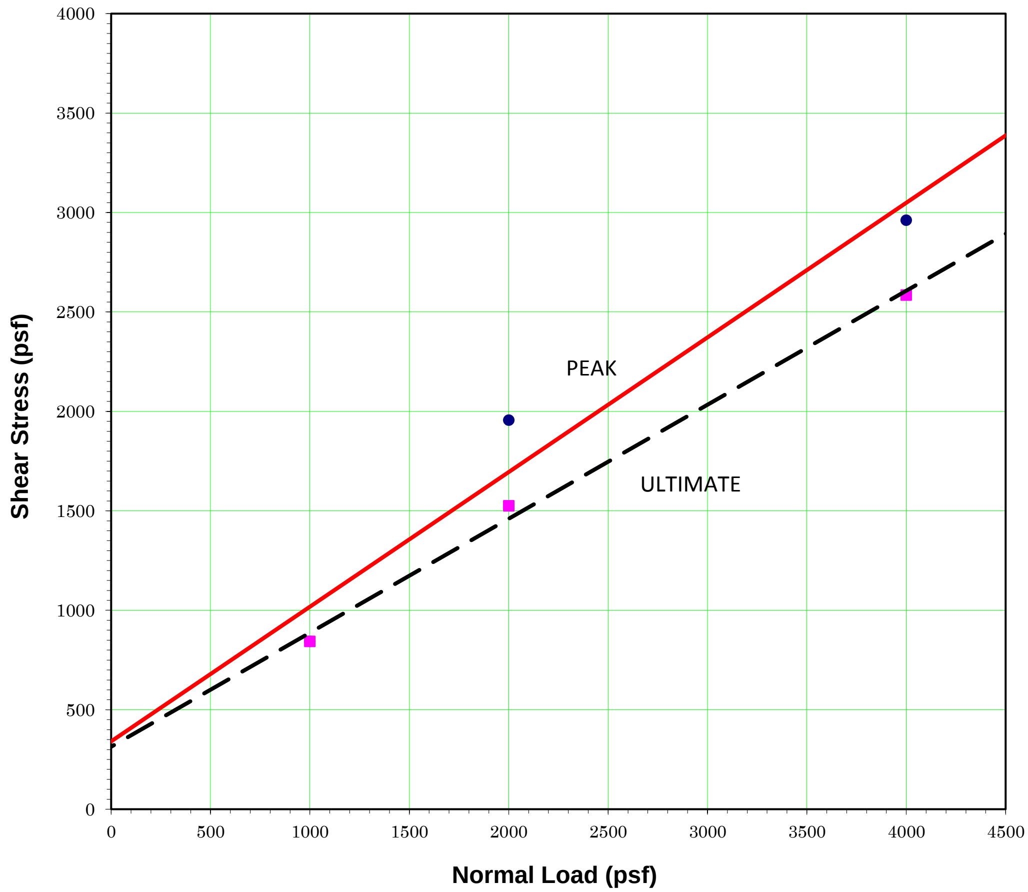
* Test Method: ASTM D-3080

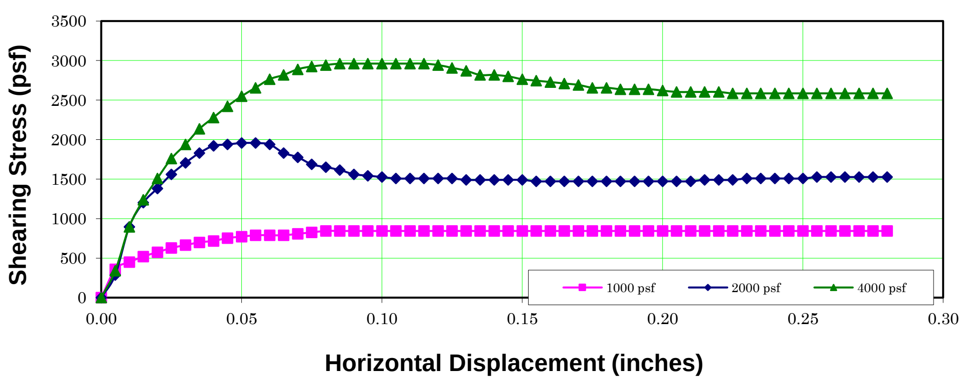
DIRECT SHEAR TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016 LA-01670-01





DIRECT SHEAR DATA*

Sample Location:	B4 at 70 feet	
Material:	Alluvium; Clayey SAND (SC)	
Dry Density:	121.7 pcf	
	Initial	Final
Moisture Content:	13.5%	13.5%
Saturation:	100%	100%
	Peak	Ultimate
ϕ Angle of Friction (degrees):	34	30
c Cohesive Strength (psf):	340	310

Test Type: Peak and Ultimate Shear Rate (in/min): 0.005

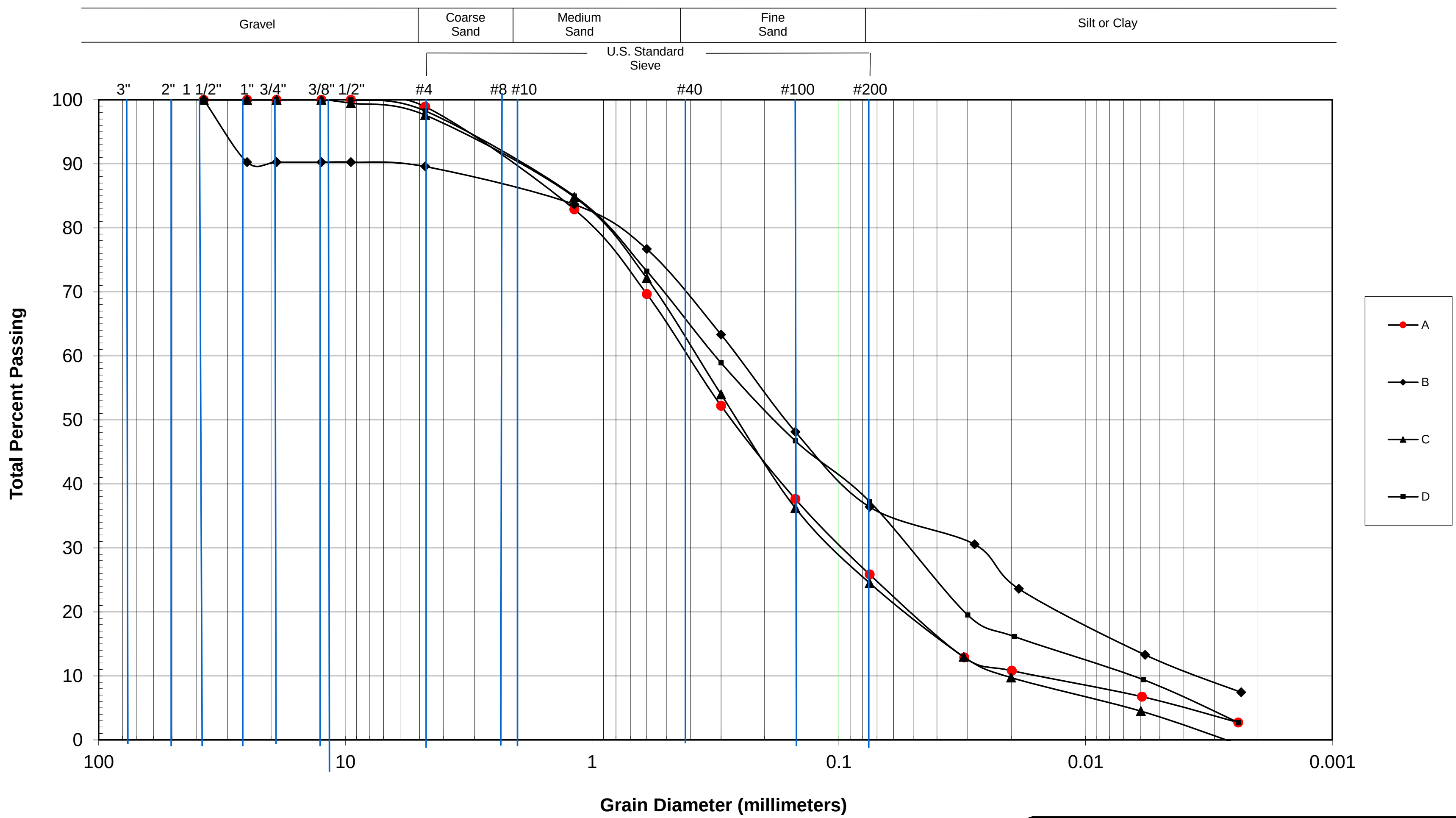
* Test Method: ASTM D-3080

DIRECT SHEAR TEST

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01



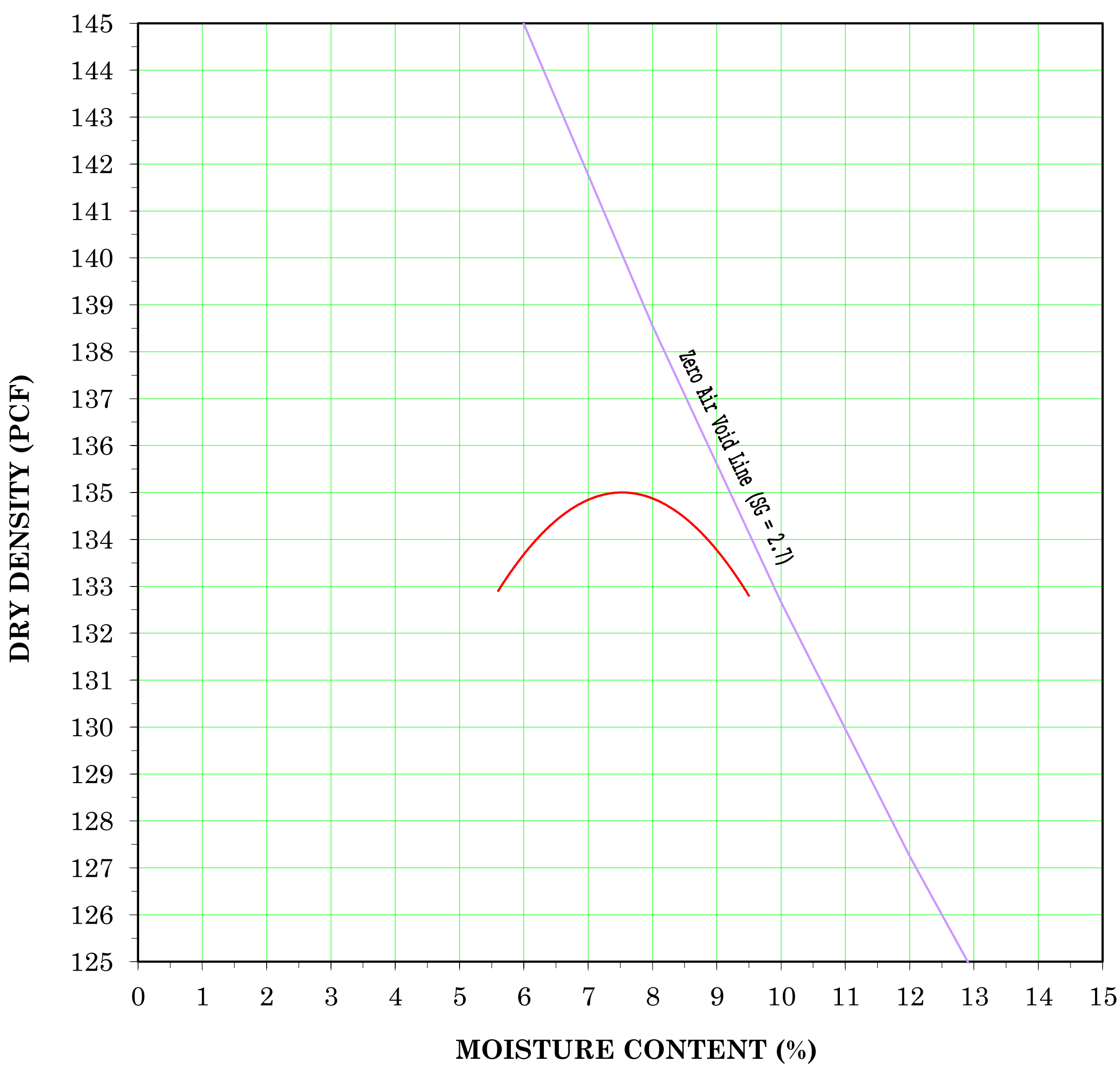
	Sample Location	Soil Type	Fines	Clay
A	B1 at 10 feet	Silty SAND some clay (SM)	26 %	6 %
B	B1 at 35 feet	Silty SAND with clay and gravel (SM)	36 %	12 %
C	B2 at 35 feet	Silty SAND (SM)	25 %	3 %
D	B2 at 55 feet	Silty SAND some clay (SM)	37 %	8 %

GRAIN SIZE ANALYSIS SUMMARY

PROPOSED MIXED-USE DEVELOPMENT
HOLLYWOOD BLVD AND WILCOX AVE
HOLLYWOOD, CALIFORNIA

Earth Systems
Southern California

10/7/2016	LA-01670-01



Maximum Density - Optimum Moisture Characteristics*

Sample Location: B4 at 0 to 5 feet

Material: Silty SAND, brown, some gravel.

Maximum Density: 135 pcf

Optimum Moisture: 7.5%

* Test Method: ASTM D-1557

MAXIMUM DENSITY - OPTIMUM MOISTURE

PROPOSED MIXED-USE DEVELOPMENT
6436 HOLLYWODD BOULEVARD
LOS ANGELES, CALIFORNIA

Earth Systems
Southern California

1/0/1900 | LA-01670-01

Table 1 - Laboratory Tests on Soil Samples

Earth Systems Southern California - Pasadena
Hollywood & Wilcox
Your #LA-01670-01, HDR Lab #16-0554LAB
27-Jul-16

Sample ID			B4 @ 0-5' af & native mix
Resistivity		**Units**	
as-received		ohm-cm	27,600
saturated		ohm-cm	9,200
pH			7.3
Electrical Conductivity		mS/cm	0.08
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	42
magnesium	Mg^{2+}	mg/kg	9.6
sodium	Na^{1+}	mg/kg	15
potassium	K^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	177
fluoride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	17
phosphate	PO_4^{3-}	mg/kg	20
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	13
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

TABLE C-1
SUMMARY OF EXPANSION INDEX* TESTING

Sample Location	Material Description	Expansion Index
B4 0 to 5 feet	Silty SAND (SM)	8

*ASTM D 4829 Test Method

TABLE C-2
SUMMARY OF ATTERBERG LIMITS** TESTS

Sample Location	Material Description	Liquid Limit	Plasticity Index
B4 at 55 feet	Sandy CLAY (CL)	31.6	12

**ASTM D 4318 Test Method

APPENDIX D

Seismic Data

USGS Design Maps Summary Report

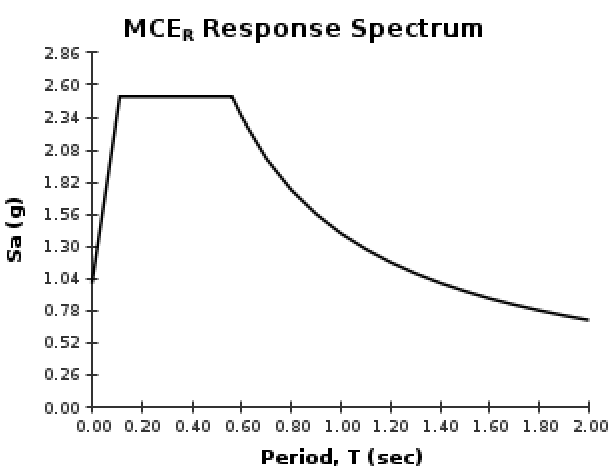
User–Specified Input

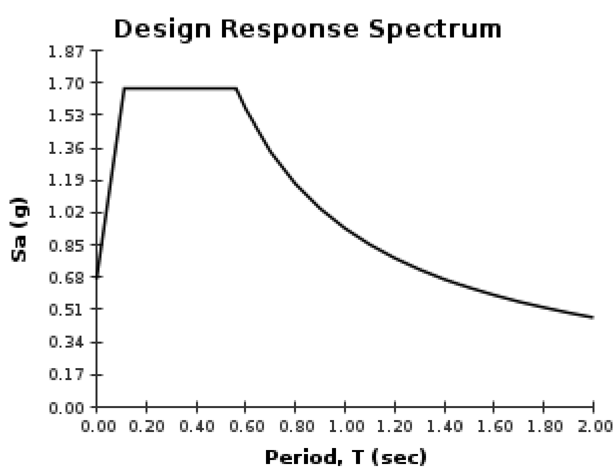
Report Title	LA-01670-01 Wed October 5, 2016 20:08:40 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.101°N, 118.331°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

$S_S =$	2.506 g	$S_{MS} =$	2.506 g	$S_{DS} =$	1.670 g
$S_1 =$	0.937 g	$S_{M1} =$	1.405 g	$S_{D1} =$	0.937 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.101°N, 118.331°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.506$ g

From Figure 22-2 [2] $S_1 = 0.937$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.506 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.937 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 2.506 = 2.506$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.500 \times 0.937 = 1.405$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

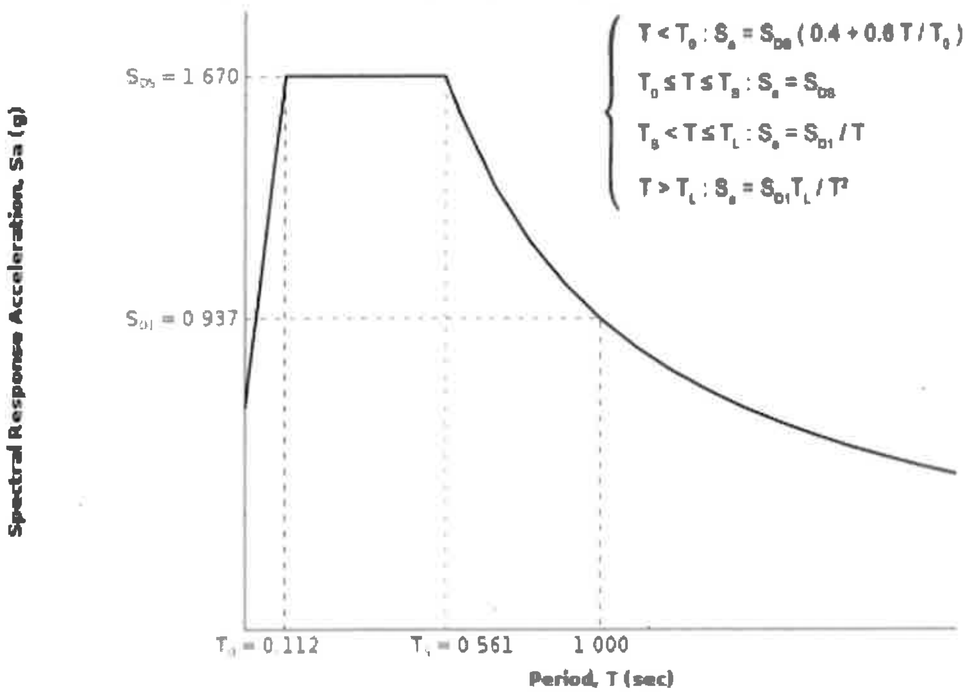
Equation (11.4-3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.506 = 1.670$ g

Equation (11.4-4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.405 = 0.937$ g

Section 11.4.5 — Design Response Spectrum

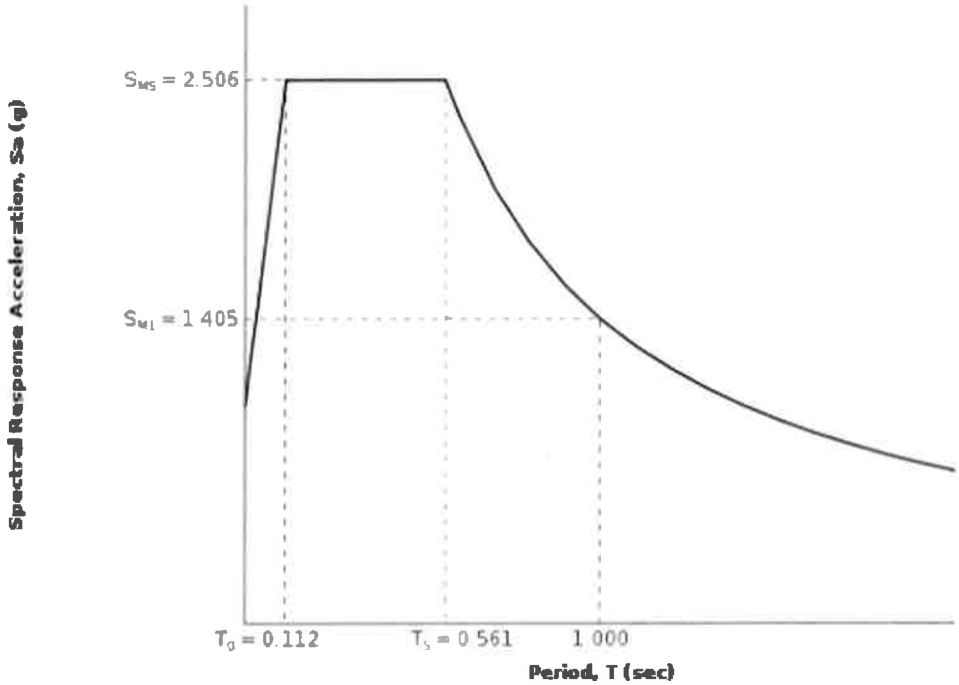
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.976

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.976 = 0.976$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.976 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.941

From Figure 22-18 [6] C_{R1} = 0.936

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.670 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.937 g, Seismic Design Category = D

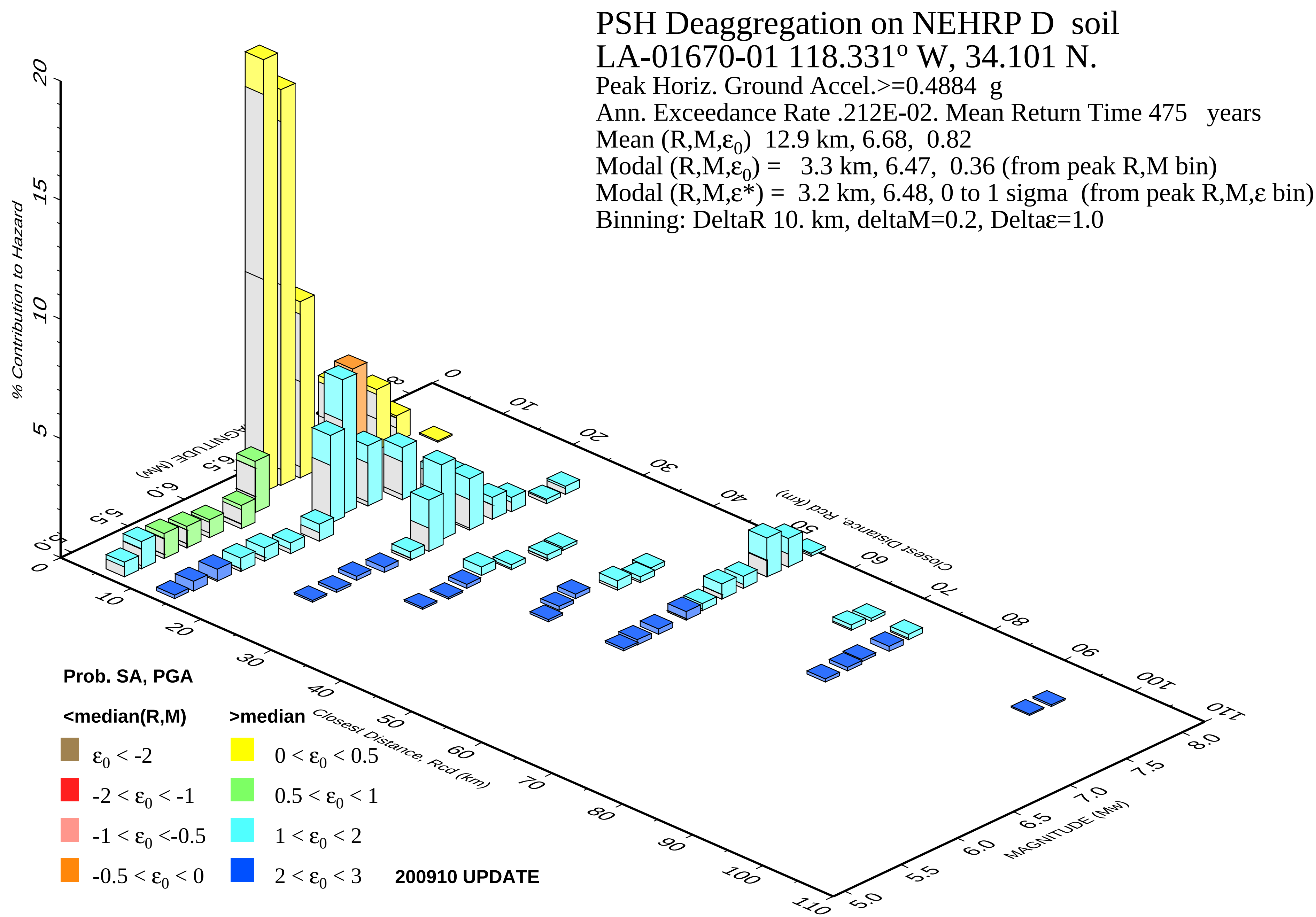
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf



GMT 2016 Sep 20 18:37:48 **Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 213. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted**

APPENDIX E

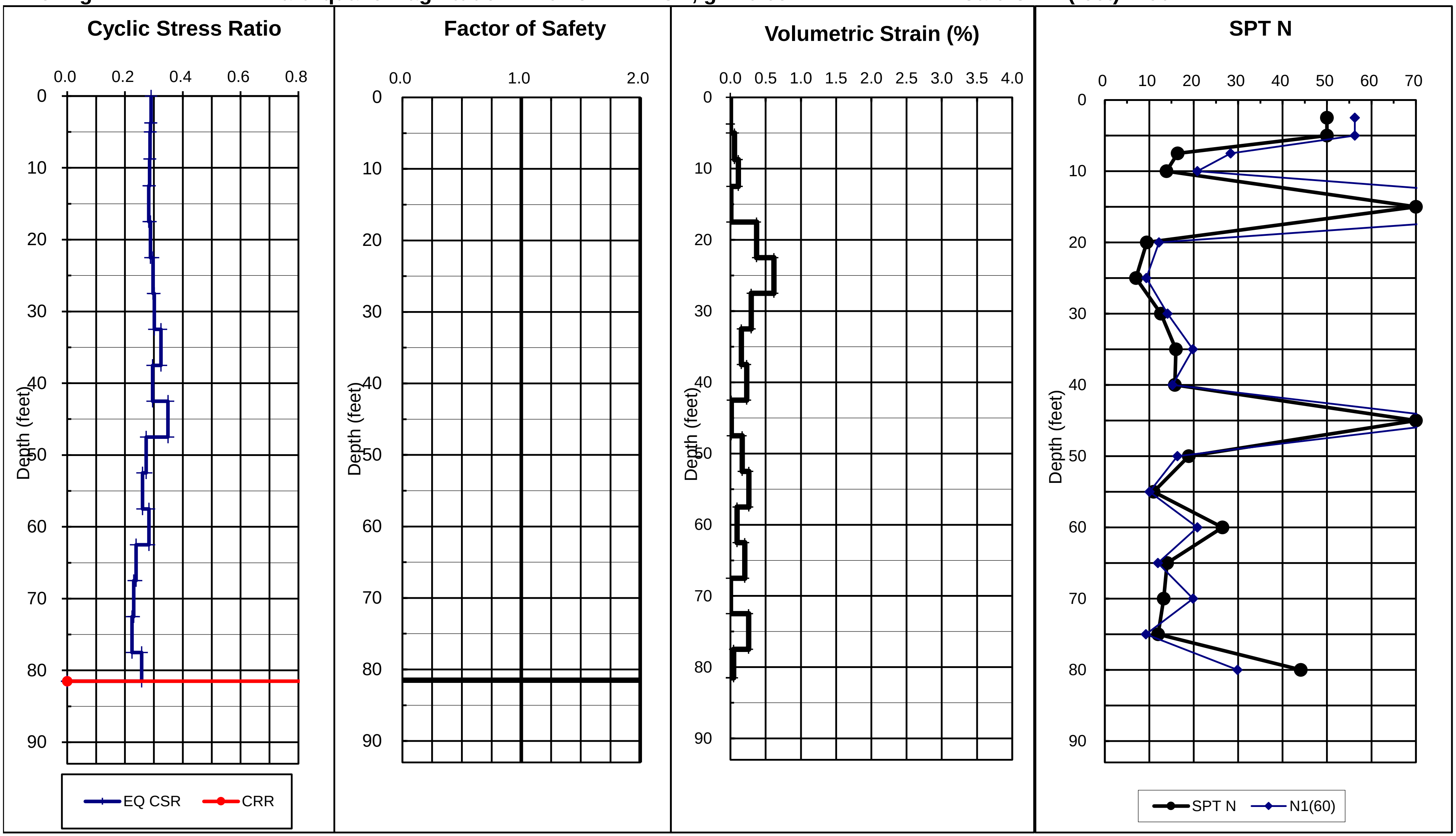
Results of Earthquake-Induced Ground Subsidence Analyses

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

6436 Hollywood Blvd. **Project No: LA-01670-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B2 **Earthquake Magnitude: 6.48** **PGA, g: 0.65** **Calc GWT (feet): 90**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 1.7 inches**

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: 6436 Hollywood Blvd.
Job No: LA-01670-01
Date: 10/7/2016
Boring: B2 Data Set: 1

Methods: Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 6.48 7.5
PGA, g: 0.651 0.45
MSF: 1.45
GWT: 100.0 feet
Calc GWT: 90.0 feet
Remediate to: 5.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.50 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: #N/A

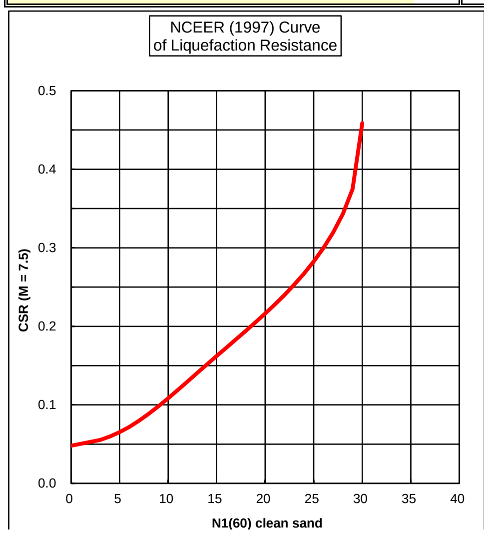
Total (ft) Liquefied Thickness: 0
Total (in.) Induced Subsidence: 1.7 upper 90ft

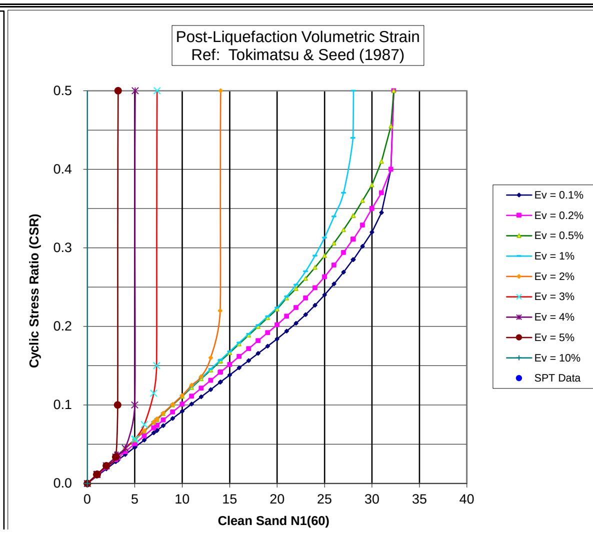
Required SF: 1.10
Minimum Calculated SF: #N/A

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 7.2

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																								
3.8	15	50		119	25	2.5	5.5	0.149	0.149	1.00	1.00	0.75	1.00	56.3				1.00	Infin.	0.290	Non-Liq.		56.3	0.00	0.00	0.100						
5.0	16	50		113	25	5.0	8.0	0.293	0.293	0.99	1.00	0.75	1.00	56.3				1.00	Infin.	0.288	Non-Liq.		56.3	0.00	0.00	0.197						
8.8	26	16	1	124	25	7.5	10.5	0.448	0.448	0.98	1.54	0.75	1.00	28.3	64	7.5	35.9	1.00	1.200	0.287	Non-Liq.	7.5	35.9	0.06	0.03	0.300	808	0.187	7.8E-04	3.9E-04	2.8E-04	0.03
12.5	22	14	1	130	25	10.0	13.0	0.606	0.606	0.98	1.32	0.76	1.00	20.8	55	6.7	27.5	1.00	0.330	0.285	Non-Liq.	6.7	27.5	0.11	0.05	0.406	860	0.251	1.1E-03	7.8E-04	5.6E-04	0.05
17.5		70	1	128	25	15.0	18.0	0.928	0.928	0.97	1.07	0.86	1.30	126.0	100	10.0	136.0	1.00	1.200	0.282	Non-Liq.	10.0	136.0	0.00	0.00	0.621	1,812	0.380	3.4E-04	3.5E-05	2.5E-05	0.00
22.5	15	9	1	126	25	20.0	23.0	1.244	1.244	0.96	0.92	0.93	1.00	12.2	42	5.7	17.9	0.97	0.193	0.288	Non-Liq.	5.7	17.9	0.37	0.22	0.834	1,067	0.504	2.3E-03	2.6E-03	1.9E-03	0.22
27.5		7	1	131	25	25.0	28.0	1.565	1.565	0.94	0.82	0.98	1.10	9.4	37	5.4	14.7	0.92	0.159	0.297	Non-Liq.	5.4	14.7	0.62	0.37	1.048	1,122	0.624	3.0E-03	4.3E-03	3.1E-03	0.37
32.5	20	13	1	136	25	30.0	33.0	1.898	1.898	0.92	0.75	1.00	1.00	14.1	45	5.9	20.0	0.89	0.216	0.301	Non-Liq.	5.9	20.0	0.29	0.18	1.272	1,369	0.739	2.1E-03	2.1E-03	1.5E-03	0.18
37.5		16	1	130	25	35.0	38.0	2.231	2.231	0.89	0.69	1.00	1.20	19.8	53	6.6	26.4	0.80	0.307	0.324	Non-Liq.	6.6	26.4	0.16	0.09	1.495	1,627	0.841	1.5E-03	1.1E-03	7.8E-04	0.09
42.5	25	16	1	125	25	40.0	43.0	2.550	2.550	0.85	0.64	1.00	1.00	15.2	47	6.0	21.3	0.84	0.231	0.295	Non-Liq.	6.0	21.3	0.23	0.14	1.708	1,618	0.918	1.7E-03	1.6E-03	1.2E-03	0.14
47.5		70	1	127	25	45.0	48.0	2.864	2.864	0.80	0.61	1.00	1.30	83.0	100	10.0	93.0	0.67	1.200	0.348	Non-Liq.	10.0	93.0	0.01	0.01	1.919	2,805	0.974	5.5E-04	8.7E-05	6.2E-05	0.01
52.5	30	19	1	129	25	50.0	53.0	3.184	3.184	0.75	0.58	1.00	1.00	16.3	48	6.2	22.5	0.80	0.247	0.273	Non-Liq.	6.2	22.5	0.17	0.10	2.133	1,843	1.014	1.3E-03	1.2E-03	8.3E-04	0.10
57.5		11	1	131	37	55.0	58.0	3.509	3.509	0.70	0.55	1.00	1.11	10.0	38	7.0	17.1	0.79	0.184	0.260	Non-Liq.	7.0	17.1	0.26	0.16	2.351	1,764	1.044	1.5E-03	1.8E-03	1.3E-03	0.16
62.5	42	26	1	132	25	60.0	63.0	3.837	3.837	0.66	0.53	1.00	1.00	20.8	55	6.7	27.5	0.68	0.331	0.283	Non-Liq.	6.7	27.5	0.09	0.06	2.571	2,164	1.070	9.5E-04	6.5E-04	4.6E-04	0.06
67.5		14	1	134	25	65.0	68.0	4.170	4.170	0.62	0.50	1.00	1.13	11.9	41	5.7	17.6	0.76	0.190	0.238	Non-Liq.	5.7	17.6	0.20	0.12	2.794	1,943	1.098	1.2E-03	1.4E-03	1.0E-03	0.12
72.5	21	13		136	50	70.0	73.0	4.508	4.508	0.59	1.00	1.00	1.00	19.8				0.75	Infin.	0.230	Non-Liq.		19.8	0.00	0.00	3.020						
77.5		12	1	138	25	75.0	78.0	4.850	4.850	0.57	0.47	1.00	1.10	9.3	36	5.4	14.6	0.74	0.158	0.224	Non-Liq.	5.4	14.6	0.26	0.16	3.250	1,970	1.165	1.2E-03	1.8E-03	1.3E-03	0.16
81.5	70	44	1	140	25	80.0	83.0	5.198	5.198	0.55	0.45	1.00	1.00	29.8	65	7.7	37.6	0.62	1.200	0.257	Non-Liq.	7.7	37.6	0.05	0.02	3.483	2,794	1.206	6.7E-04	3.1E-04	2.3E-04	0.02





$N_{1(60)} = C_N{}^*C_E{}^*C_B{}^*C_R{}^*C_S{}^*N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))

C_N = (1 atm/p'o)$^{0.5}$, max 1.7

C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners

MSF = $10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po

τ_{av} = 0.65*PGA*po*rd

G_{max} = 447*$N_{1(60)CS}{}^{(1/3)}$*$p^{0.5}$

a = 0.0389*(p/1)+0.124

b = 6400*(p/1)$^{(-0.6)}$

γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]

E_{15} = γ*($N_{1(60)CS}$/20)$^{-1.2}$

E_{nc} = (Nc/15)$^{0.45}$*E15

Nc = (MAG-4)$^{2.17}$

S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

Kσ = min of 1.0 or (p'o/1.058)$^{\text{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}}$

Dr = ($N_{1(60)}$/70)$^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

N = $N_{1(60)CS}$

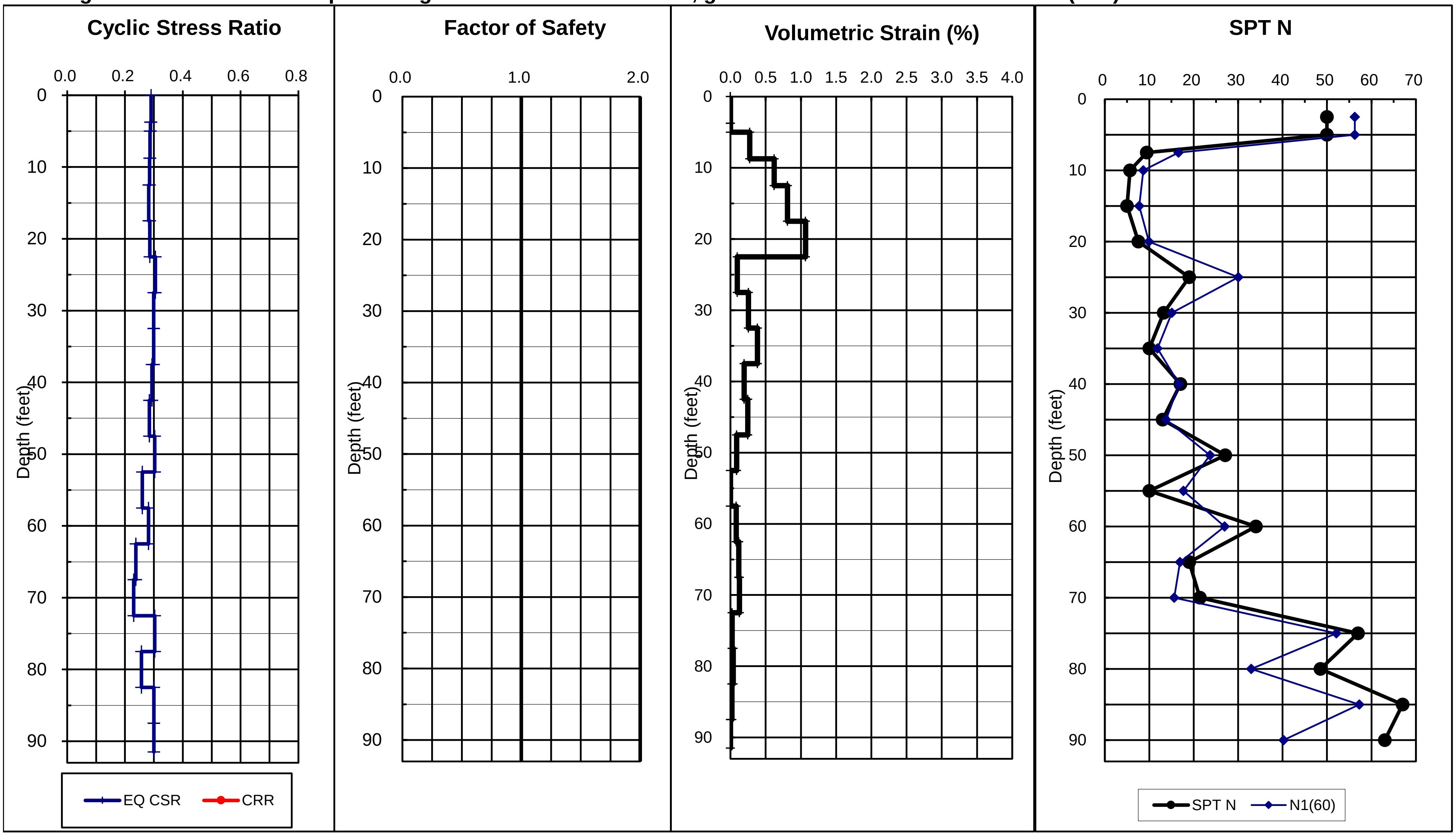
SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

6436 Hollywood Blvd. **Project No: LA-01670-01** 1996/1998 NCEER Method

Ground Compaction Remediated to 5 foot depth

Boring: B4 **Earthquake Magnitude: 6.48** **PGA, g: 0.65** **Calc GWT (feet): 90**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 2.5 inches**

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: 6436 Hollywood Blvd.
Job No: LA-01670-01
Date: 10/7/2016
Boring: B4 Data Set: 2

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 6.48 7.5
PGA, g: 0.651 0.45
MSF: 1.45
GWT: 91.5 feet
Calc GWT: 90.0 feet
Remediate to: 5.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.50 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: #N/A

Total (ft) Liquefied Thickness: 0

Total (in.) Induced Subsidence: 2.5 upper 90ft

Required SF: 1.10
Minimum Calculated SF: #N/A

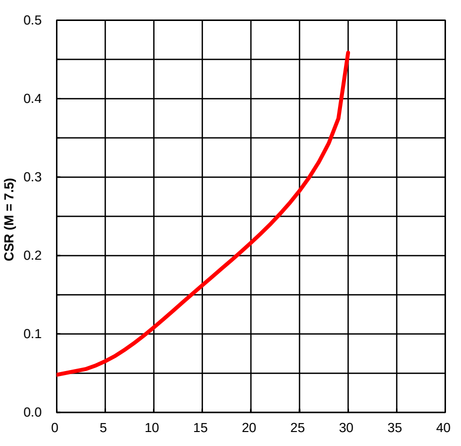
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 7.2

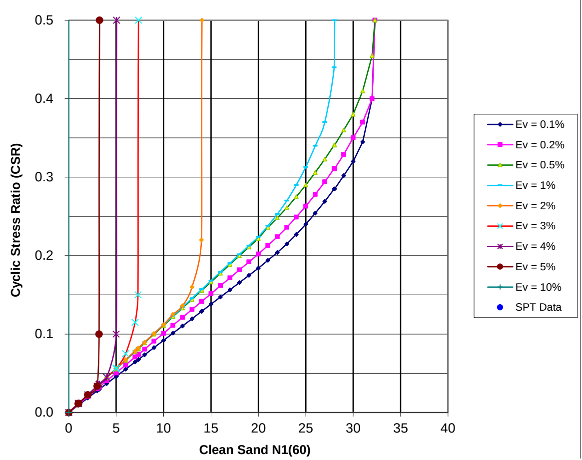
Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
								0.000																	
3.8	9	50		120	25	2.5	5.5	0.150	0.150	1.00	1.00	0.75	1.00	56.3				1.00	Infin.	0.290	**Non-Liq.**		56.3	0.00	**0.00**
5.0	19	50		111	12	5.0	8.0	0.294	0.294	0.99	1.00	0.75	1.00	56.3				1.00	Infin.	0.288	**Non-Liq.**		56.3	0.00	**0.00**
8.8	15	9	1	112	12	7.5	10.5	0.434	0.434	0.98	1.56	0.75	1.00	16.6	49	2.1	18.7	1.00	0.202	0.287	**Non-Liq.**	2.1	18.7	0.27	**0.12**
12.5	9	6	1	127	25	10.0	13.0	0.583	0.583	0.98	1.35	0.76	1.00	8.7	35	5.3	14.0	1.00	0.151	0.285	**Non-Liq.**	5.3	14.0	0.62	**0.28**
17.5		5	1	124	25	15.0	18.0	0.896	0.896	0.97	1.09	0.86	1.10	7.7	33	5.2	12.9	1.00	0.140	0.282	**Non-Liq.**	5.2	12.9	0.81	**0.49**
22.5	12	8	1	121	12	20.0	23.0	1.202	1.202	0.96	0.94	0.93	1.00	9.9	38	1.9	11.8	0.97	0.128	0.286	**Non-Liq.**	1.9	11.8	1.07	**0.64**
27.5		19	1	126	12	25.0	28.0	1.511	1.511	0.94	0.84	0.98	1.28	30.1	66	2.5	32.6	0.90	1.200	0.305	**Non-Liq.**	2.5	32.6	0.10	**0.06**
32.5	21	13	1	131	25	30.0	33.0	1.832	1.832	0.92	0.76	1.00	1.00	15.1	46	6.0	21.1	0.90	0.229	0.299	**Non-Liq.**	6.0	21.1	0.26	**0.15**
37.5		10	1	127	25	35.0	38.0	2.155	2.155	0.89	0.70	1.00	1.13	11.8	41	5.6	17.5	0.87	0.189	0.299	**Non-Liq.**	5.6	17.5	0.38	**0.23**
42.5	27	17	1	122	25	40.0	43.0	2.467	2.467	0.85	0.65	1.00	1.00	16.7	49	6.2	22.9	0.84	0.252	0.293	**Non-Liq.**	6.2	22.9	0.19	**0.12**
47.5		13	1	130	25	45.0	48.0	2.782	2.782	0.80	0.62	1.00	1.14	13.8	44	5.9	19.6	0.82	0.212	0.284	**Non-Liq.**	5.9	19.6	0.24	**0.15**
52.5	43	27	1	138	25	50.0	53.0	3.117	3.117	0.75	0.58	1.00	1.00	23.7	58	7.0	30.7	0.72	1.200	0.303	**Non-Liq.**	7.0	30.7	0.09	**0.05**
57.5		10		135	50	55.0	58.0	3.458	3.458	0.70	1.00	1.00	1.18	17.7				0.79	Infin.	0.260	**Non-Liq.**		17.7	0.00	**0.00**
62.5	54	34	1	133	12	60.0	63.0	3.793	3.793	0.66	0.53	1.00	1.00	27.0	62	2.4	29.4	0.68	0.392	0.282	**Non-Liq.**	2.4	29.4	0.08	**0.05**
67.5		19	1	135	25	65.0	68.0	4.128	4.128	0.62	0.51	1.00	1.17	16.9	49	6.2	23.2	0.76	0.255	0.238	**Non-Liq.**	6.2	23.2	0.12	**0.07**
72.5	34	21	1	138	25	70.0	73.0	4.470	4.470	0.59	0.49	1.00	1.00	15.6	47	6.1	21.7	0.75	0.237	0.230	**Non-Liq.**	6.1	21.7	0.13	**0.08**
77.5		57	1	139	12	75.0	78.0	4.816	4.816	0.57	0.47	1.00	1.30	52.1	86	3.2	55.3	0.55	1.200	0.303	**Non-Liq.**	3.2	55.3	0.02	**0.01**
82.5	77	49	1	140	25	80.0	83.0	5.164	5.164	0.55	0.45	1.00	1.00	32.9	69	8.1	41.0	0.62	1.200	0.257	**Non-Liq.**	8.1	41.0	0.04	**0.02**
87.5		67	1	136	12	85.0	88.0	5.509	5.509	0.53	0.44	1.00	1.30	57.3	90	3.4	60.6	0.52	1.200	0.300	**Non-Liq.**	3.4	60.6	0.02	**0.01**
91.5	100	63	1	133	12	90.0	93.0	5.846	5.846	0.52	0.43	1.00	1.00	40.2	76	2.8	43.0	0.50	1.200	0.300	**Non-Liq.**	2.8	43.0	0.00	**0.00**

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
3.8	0.100						
5.0	0.197						
8.8	0.291	639	0.181	1.8E-03	1.9E-03	1.4E-03	**0.12**
12.5	0.391	673	0.242	2.8E-03	4.3E-03	3.1E-03	**0.28**
17.5	0.600	813	0.367	3.3E-03	5.6E-03	4.0E-03	**0.49**
22.5	0.805	913	0.487	3.9E-03	7.4E-03	5.3E-03	**0.64**
27.5	1.012	1,436	0.602	1.2E-03	6.7E-04	4.8E-04	**0.06**
32.5	1.228	1,369	0.714	1.9E-03	1.8E-03	1.3E-03	**0.15**
37.5	1.444	1,394	0.812	2.3E-03	2.7E-03	1.9E-03	**0.23**
42.5	1.653	1,632	0.888	1.6E-03	1.4E-03	9.7E-04	**0.12**
47.5	1.864	1,646	0.946	1.7E-03	1.7E-03	1.2E-03	**0.15**
52.5	2.088	2,022	0.993	1.0E-03	6.2E-04	4.5E-04	**0.05**
57.5	2.317						
62.5	2.541	2,198	1.058	9.0E-04	5.7E-04	4.1E-04	**0.05**
67.5	2.766	2,119	1.087	9.9E-04	8.3E-04	6.0E-04	**0.07**
72.5	2.995	2,158	1.119	9.8E-04	8.9E-04	6.4E-04	**0.08**
77.5	3.227	3,059	1.157	5.5E-04	1.6E-04	1.2E-04	**0.01**
82.5	3.460	2,867	1.198	6.4E-04	2.7E-04	1.9E-04	**0.02**
87.5	3.691	3,374	1.241	5.2E-04	1.4E-04	9.8E-05	**0.01**
91.5	3.917	3,100	1.284	6.1E-04			

NCEER (1997) Curve of Liquefaction Resistance



Post-Liquefaction Volumetric Strain
Ref: Tokimatsu & Seed (1987)



$N_{1(60)} = C_N*C_E*C_B*C_R*C_S*N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
$= \min(1,\max(0.75,1.4666-2.556/(z(ft))^{0.5}))$

$C_N = (1\ atm/p'o)^{0.5}$, max 1.7

$C_S = \max(1.1,\min(1.3,1+N_{1(60)}/100))$ for SPT without liners

$MSF = 10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

$p = 0.67*po$

$\tau_{av} = 0.65*PGA*po*rd$

$G_{max} = 447*N_{1(60)CS}^{(1/3)}*p^{0.5}$

$a = 0.0389*(p/1)+0.124$

$b = 6400*(p/1)^{(-0.6)}$

$\gamma = [1+a*EXP(b*\tau_{av}/G_{max})]/[(1+a)*\tau_{av}/G_{max}]$

$E_{15} = \gamma*(N_{1(60)CS}/20)^{-1.2}$

$E_{nc} = (Nc/15)^{0.45}*E15$

$S = 2*H*E_{nc}$

$Nc = (MAG-4)^{2.17}$

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

$K\sigma$ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

$Dr = (N_{1(60)}/70)^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd
CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

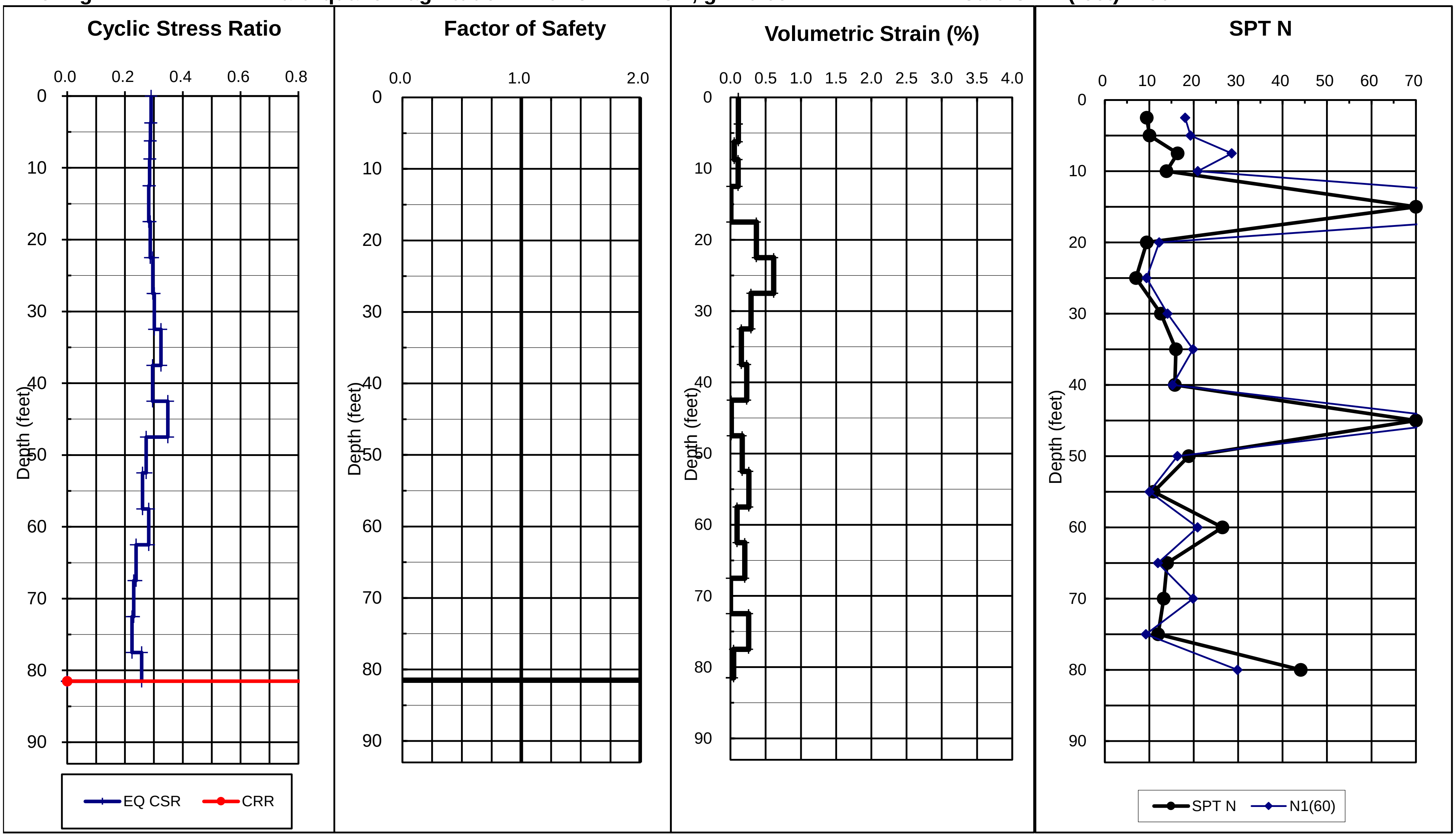
$N = N_{1(60)CS}$

$SF = CRR_{7.5,1atm}/CSR*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

6436 Hollywood Blvd. Project No: LA-01670-01 1996/1998 NCEER Method

Boring: B2 Earthquake Magnitude: 6.48 PGA, g: 0.65 Calc GWT (feet): 90



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 1.8 inches

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: 6436 Hollywood Blvd.
Job No: LA-01670-01
Date: 10/7/2016
Boring: B2 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 6.48 7.5
PGA, g: 0.651 0.45
MSF: 1.45
GWT: 100.0 feet
Calc GWT: 90.0 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.50 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: #N/A
Required SF: 1.10
Minimum Calculated SF: #N/A

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
0	1.8
	upper 90ft

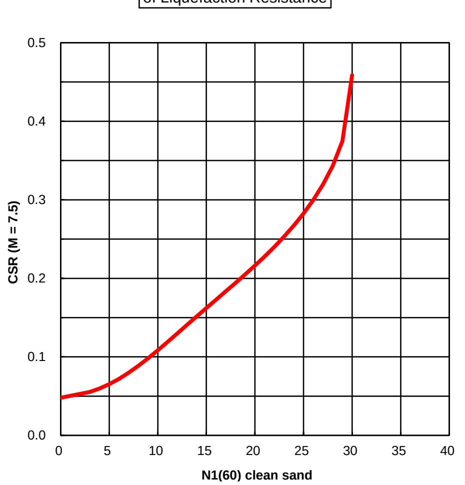
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 7.2

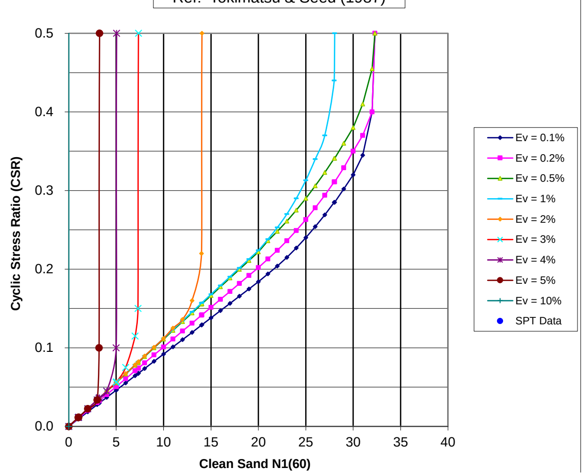
Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)
								0.000																	
3.8	15	9	1	119	25	2.5	5.5	0.149	0.149	1.00	1.70	0.75	1.00	18.1	51	6.4	24.4	1.00	0.274	0.290	Non-Liq.	6.4	24.4	0.11	0.05
6.3	16	10	1	113	25	5.0	8.0	0.293	0.293	0.99	1.70	0.75	1.00	19.3	52	6.5	25.8	1.00	0.296	0.288	Non-Liq.	6.5	25.8	0.11	0.03
8.8	26	16	1	124	25	7.5	10.5	0.441	0.441	0.98	1.55	0.75	1.00	28.5	64	7.6	36.1	1.00	1.200	0.287	Non-Liq.	7.6	36.1	0.05	0.02
12.5	22	14	1	130	25	10.0	13.0	0.599	0.599	0.98	1.33	0.76	1.00	20.9	55	6.7	27.6	1.00	0.333	0.285	Non-Liq.	6.7	27.6	0.11	0.05
17.5		70	1	128	25	15.0	18.0	0.921	0.921	0.97	1.07	0.86	1.30	126.4	100	10.0	136.4	1.00	1.200	0.282	Non-Liq.	10.0	136.4	0.00	0.00
22.5	15	9	1	126	25	20.0	23.0	1.237	1.237	0.96	0.92	0.93	1.00	12.2	42	5.7	17.9	0.97	0.193	0.287	Non-Liq.	5.7	17.9	0.37	0.22
27.5		7	1	131	25	25.0	28.0	1.558	1.558	0.94	0.82	0.98	1.10	9.4	37	5.4	14.7	0.93	0.159	0.296	Non-Liq.	5.4	14.7	0.61	0.37
32.5	20	13	1	136	25	30.0	33.0	1.891	1.891	0.92	0.75	1.00	1.00	14.1	45	5.9	20.0	0.89	0.217	0.301	Non-Liq.	5.9	20.0	0.29	0.18
37.5		16	1	130	25	35.0	38.0	2.224	2.224	0.89	0.69	1.00	1.20	19.8	53	6.6	26.4	0.80	0.308	0.324	Non-Liq.	6.6	26.4	0.15	0.09
42.5	25	16	1	125	25	40.0	43.0	2.543	2.543	0.85	0.65	1.00	1.00	15.2	47	6.0	21.3	0.84	0.231	0.295	Non-Liq.	6.0	21.3	0.23	0.14
47.5		70	1	127	25	45.0	48.0	2.857	2.857	0.80	0.61	1.00	1.30	83.1	100	10.0	93.1	0.67	1.200	0.348	Non-Liq.	10.0	93.1	0.01	0.01
52.5	30	19	1	129	25	50.0	53.0	3.177	3.177	0.75	0.58	1.00	1.00	16.4	48	6.2	22.5	0.80	0.247	0.273	Non-Liq.	6.2	22.5	0.17	0.10
57.5		11	1	131	37	55.0	58.0	3.502	3.502	0.70	0.55	1.00	1.11	10.1	38	7.0	17.1	0.79	0.184	0.260	Non-Liq.	7.0	17.1	0.26	0.16
62.5	42	26	1	132	25	60.0	63.0	3.831	3.831	0.66	0.53	1.00	1.00	20.9	55	6.7	27.5	0.68	0.332	0.282	Non-Liq.	6.7	27.5	0.09	0.06
67.5		14	1	134	25	65.0	68.0	4.164	4.164	0.62	0.50	1.00	1.13	11.9	41	5.7	17.6	0.76	0.190	0.238	Non-Liq.	5.7	17.6	0.20	0.12
72.5	21	13		136	50	70.0	73.0	4.501	4.501	0.59	1.00	1.00	1.00	19.8				0.75	Infin.	0.230	Non-Liq.		19.8	0.00	0.00
77.5		12	1	138	25	75.0	78.0	4.843	4.843	0.57	0.47	1.00	1.10	9.3	36	5.4	14.6	0.74	0.158	0.224	Non-Liq.	5.4	14.6	0.26	0.16
81.5	70	44	1	140	25	80.0	83.0	5.191	5.191	0.55	0.45	1.00	1.00	29.9	65	7.7	37.6	0.62	1.200	0.257	Non-Liq.	7.7	37.6	0.05	0.02

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
3.8	0.100	409	0.063	1.0E-03	7.9E-04	5.6E-04	0.05
6.3	0.197	586	0.123	1.1E-03	7.7E-04	5.6E-04	0.03
8.8	0.296	803	0.184	7.7E-04	3.8E-04	2.7E-04	0.02
12.5	0.402	856	0.248	1.1E-03	7.7E-04	5.5E-04	0.05
17.5	0.617	1,807	0.377	3.4E-04	3.4E-05	2.5E-05	0.00
22.5	0.829	1,065	0.501	2.3E-03	2.6E-03	1.8E-03	0.22
27.5	1.044	1,120	0.621	3.0E-03	4.3E-03	3.1E-03	0.37
32.5	1.267	1,367	0.737	2.0E-03	2.0E-03	1.5E-03	0.18
37.5	1.490	1,625	0.838	1.5E-03	1.1E-03	7.7E-04	0.09
42.5	1.704	1,617	0.916	1.7E-03	1.6E-03	1.2E-03	0.14
47.5	1.914	2,803	0.972	5.5E-04	8.6E-05	6.2E-05	0.01
52.5	2.129	1,842	1.012	1.3E-03	1.2E-03	8.3E-04	0.10
57.5	2.346	1,763	1.042	1.5E-03	1.8E-03	1.3E-03	0.16
62.5	2.567	2,163	1.069	9.5E-04	6.5E-04	4.6E-04	0.06
67.5	2.790	1,942	1.096	1.2E-03	1.4E-03	1.0E-03	0.12
72.5	3.016						
77.5	3.245	1,969	1.163	1.2E-03	1.8E-03	1.3E-03	0.16
81.5	3.478	2,793	1.204	6.7E-04	3.1E-04	2.3E-04	0.02

NCEER (1997) Curve of Liquefaction Resistance



Post-Liquefaction Volumetric Strain
Ref: Tokimatsu & Seed (1987)



$N_{1(60)} = C_N*C_E*C_B*C_R*C_S*N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))

C_N = (1 atm/p'o)$^{0.5}$, max 1.7

C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners

MSF = $10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po

τ_{av} = 0.65*PGA*po*rd

G_{max} = 447*$N_{1(60)CS}^{(1/3)}$*$p^{0.5}$

a = 0.0389*(p/1)+0.124

b = 6400*(p/1)$^{(-0.6)}$

γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]

E_{15} = γ*($N_{1(60)CS}$/20)$^{-1.2}$

E_{nc} = (Nc/15)$^{0.45}$*E15

Nc = (MAG-4)$^{2.17}$

S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

Kσ = min of 1.0 or (p'o/1.058)$^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

Dr = ($N_{1(60)}$/70)$^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

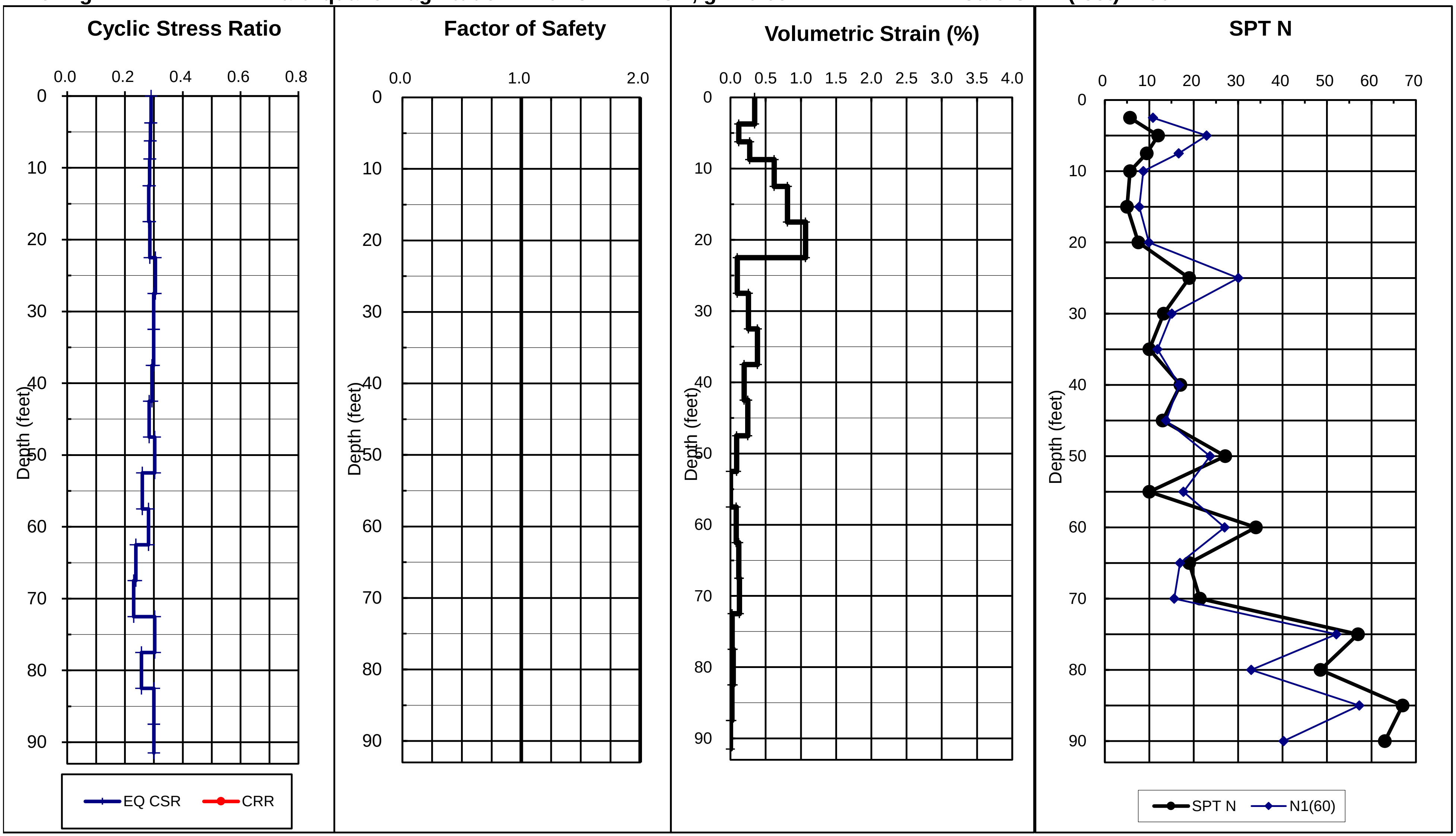
N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

6436 Hollywood Blvd. **Project No: LA-01670-01** 1996/1998 NCEER Method

Boring: B4 **Earthquake Magnitude: 6.48** **PGA, g: 0.65** **Calc GWT (feet): 90**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 2.7 inches**

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: 6436 Hollywood Blvd.
Job No: LA-01670-01
Date: 10/7/2016
Boring: B4 Data Set: 2

Methods: Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 6.48 7.5
PGA, g: 0.651 0.45
MSF: 1.45
GWT: 91.5 feet
Calc GWT: 90.0 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.50 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: #N/A
Required SF: 1.10
Minimum Calculated SF: #N/A

Total (ft) Liquefied Thickness
0

Total (in.) Induced Subsidence
2.7

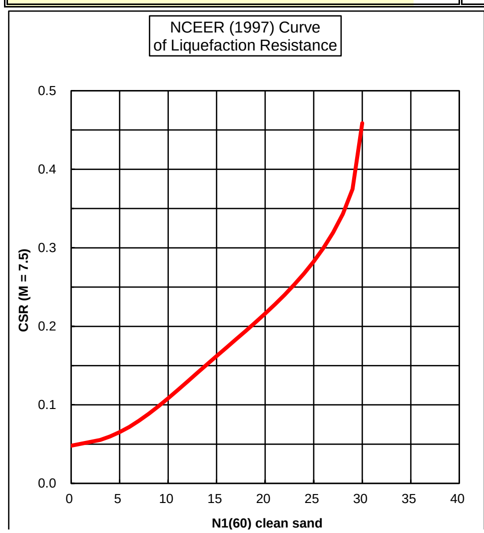
upper 90ft

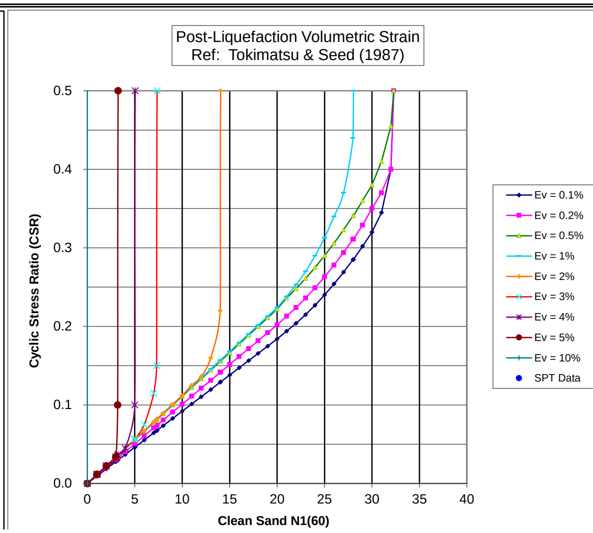
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 7.2

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)
								0.000																	
3.8	9	6	1	120	25	2.5	5.5	0.150	0.150	1.00	1.70	0.75	1.00	10.8	39	5.5	16.4	1.00	0.177	0.290	Non-Liq.	5.5	16.4	0.34	0.15
6.3	19	12	1	111	12	5.0	8.0	0.294	0.294	0.99	1.70	0.75	1.00	22.9	57	2.3	25.2	1.00	0.285	0.288	Non-Liq.	2.3	25.2	0.12	0.04
8.8	15	9	1	112	12	7.5	10.5	0.434	0.434	0.98	1.56	0.75	1.00	16.6	49	2.1	18.7	1.00	0.202	0.287	Non-Liq.	2.1	18.7	0.27	0.08
12.5	9	6	1	127	25	10.0	13.0	0.583	0.583	0.98	1.35	0.76	1.00	8.7	35	5.3	14.0	1.00	0.151	0.285	Non-Liq.	5.3	14.0	0.62	0.28
17.5		5	1	124	25	15.0	18.0	0.896	0.896	0.97	1.09	0.86	1.10	7.7	33	5.2	12.9	1.00	0.140	0.282	Non-Liq.	5.2	12.9	0.81	0.49
22.5	12	8	1	121	12	20.0	23.0	1.202	1.202	0.96	0.94	0.93	1.00	9.9	38	1.9	11.8	0.97	0.128	0.286	Non-Liq.	1.9	11.8	1.07	0.64
27.5		19	1	126	12	25.0	28.0	1.510	1.510	0.94	0.84	0.98	1.28	30.1	66	2.5	32.6	0.90	1.200	0.305	Non-Liq.	2.5	32.6	0.10	0.06
32.5	21	13	1	131	25	30.0	33.0	1.832	1.832	0.92	0.76	1.00	1.00	15.1	46	6.0	21.1	0.90	0.229	0.299	Non-Liq.	6.0	21.1	0.26	0.15
37.5		10	1	127	25	35.0	38.0	2.155	2.155	0.89	0.70	1.00	1.13	11.8	41	5.6	17.5	0.87	0.189	0.299	Non-Liq.	5.6	17.5	0.38	0.23
42.5	27	17	1	122	25	40.0	43.0	2.466	2.466	0.85	0.65	1.00	1.00	16.7	49	6.2	22.9	0.84	0.252	0.293	Non-Liq.	6.2	22.9	0.19	0.12
47.5		13	1	130	25	45.0	48.0	2.782	2.782	0.80	0.62	1.00	1.14	13.8	44	5.9	19.6	0.82	0.212	0.284	Non-Liq.	5.9	19.6	0.24	0.15
52.5	43	27	1	138	25	50.0	53.0	3.117	3.117	0.75	0.58	1.00	1.00	23.7	58	7.0	30.7	0.72	1.200	0.303	Non-Liq.	7.0	30.7	0.09	0.05
57.5		10		135	50	55.0	58.0	3.458	3.458	0.70	1.00	1.00	1.18	17.7				0.79	Infin.	0.260	Non-Liq.		17.7	0.00	0.00
62.5	54	34	1	133	12	60.0	63.0	3.792	3.792	0.66	0.53	1.00	1.00	27.0	62	2.4	29.4	0.68	0.392	0.282	Non-Liq.	2.4	29.4	0.08	0.05
67.5		19	1	135	25	65.0	68.0	4.127	4.127	0.62	0.51	1.00	1.17	16.9	49	6.2	23.2	0.76	0.255	0.238	Non-Liq.	6.2	23.2	0.12	0.07
72.5	34	21	1	138	25	70.0	73.0	4.469	4.469	0.59	0.49	1.00	1.00	15.6	47	6.1	21.7	0.75	0.237	0.230	Non-Liq.	6.1	21.7	0.13	0.08
77.5		57	1	139	12	75.0	78.0	4.816	4.816	0.57	0.47	1.00	1.30	52.1	86	3.2	55.3	0.55	1.200	0.303	Non-Liq.	3.2	55.3	0.02	0.01
82.5	77	49	1	140	25	80.0	83.0	5.164	5.164	0.55	0.45	1.00	1.00	32.9	69	8.1	41.0	0.62	1.200	0.257	Non-Liq.	8.1	41.0	0.04	0.02
87.5		67	1	136	12	85.0	88.0	5.509	5.509	0.53	0.44	1.00	1.30	57.3	90	3.4	60.6	0.52	1.200	0.300	Non-Liq.	3.4	60.6	0.02	0.01
91.5	100	63	1	133	12	90.0	93.0	5.845	5.845	0.52	0.43	1.00	1.00	40.2	76	2.8	43.0	0.50	1.200	0.300	Non-Liq.	2.8	43.0	0.00	0.00

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
3.8	0.100	360	0.063	1.9E-03	2.4E-03	1.7E-03	0.15
6.3	0.197	582	0.123	1.1E-03	8.2E-04	5.9E-04	0.04
8.8	0.291	639	0.181	1.7E-03	1.9E-03	1.4E-03	0.08
12.5	0.390	673	0.241	2.8E-03	4.3E-03	3.1E-03	0.28
17.5	0.600	813	0.367	3.3E-03	5.6E-03	4.0E-03	0.49
22.5	0.805	913	0.487	3.9E-03	7.4E-03	5.3E-03	0.64
27.5	1.012	1,436	0.602	1.2E-03	6.7E-04	4.8E-04	0.06
32.5	1.228	1,369	0.714	1.9E-03	1.8E-03	1.3E-03	0.15
37.5	1.444	1,394	0.812	2.3E-03	2.7E-03	1.9E-03	0.23
42.5	1.653	1,632	0.888	1.6E-03	1.4E-03	9.7E-04	0.12
47.5	1.864	1,646	0.946	1.7E-03	1.7E-03	1.2E-03	0.15
52.5	2.088	2,022	0.993	1.0E-03	6.2E-04	4.5E-04	0.05
57.5	2.317						
62.5	2.541	2,198	1.058	9.0E-04	5.7E-04	4.1E-04	0.05
67.5	2.765	2,119	1.087	9.9E-04	8.3E-04	6.0E-04	0.07
72.5	2.995	2,158	1.119	9.8E-04	8.9E-04	6.4E-04	0.08
77.5	3.227	3,059	1.157	5.5E-04	1.6E-04	1.2E-04	0.01
82.5	3.460	2,867	1.198	6.4E-04	2.7E-04	1.9E-04	0.02
87.5	3.691	3,373	1.241	5.2E-04	1.4E-04	9.8E-05	0.01
91.5	3.916	3,100	1.284	6.1E-04			





$N_{1(60)} = C_N{*}C_E{*}C_B{*}C_R{*}C_S{*}N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))

C_N = (1 atm/p'o)$^{0.5}$, max 1.7

C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners

MSF = $10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po

τ_{av} = 0.65*PGA*po*rd

G_{max} = 447*$N_{1(60)CS}^{(1/3)}$*$p^{0.5}$

a = 0.0389*(p/1)+0.124

b = 6400*(p/1)$^{(-0.6)}$

γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]

E_{15} = γ*($N_{1(60)CS}$/20)$^{-1.2}$

E_{nc} = (Nc/15)$^{0.45}$*E15

Nc = (MAG-4)$^{2.17}$

S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$

Kσ = min of 1.0 or (p'o/1.058)$^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

Dr = ($N_{1(60)}$/70)$^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

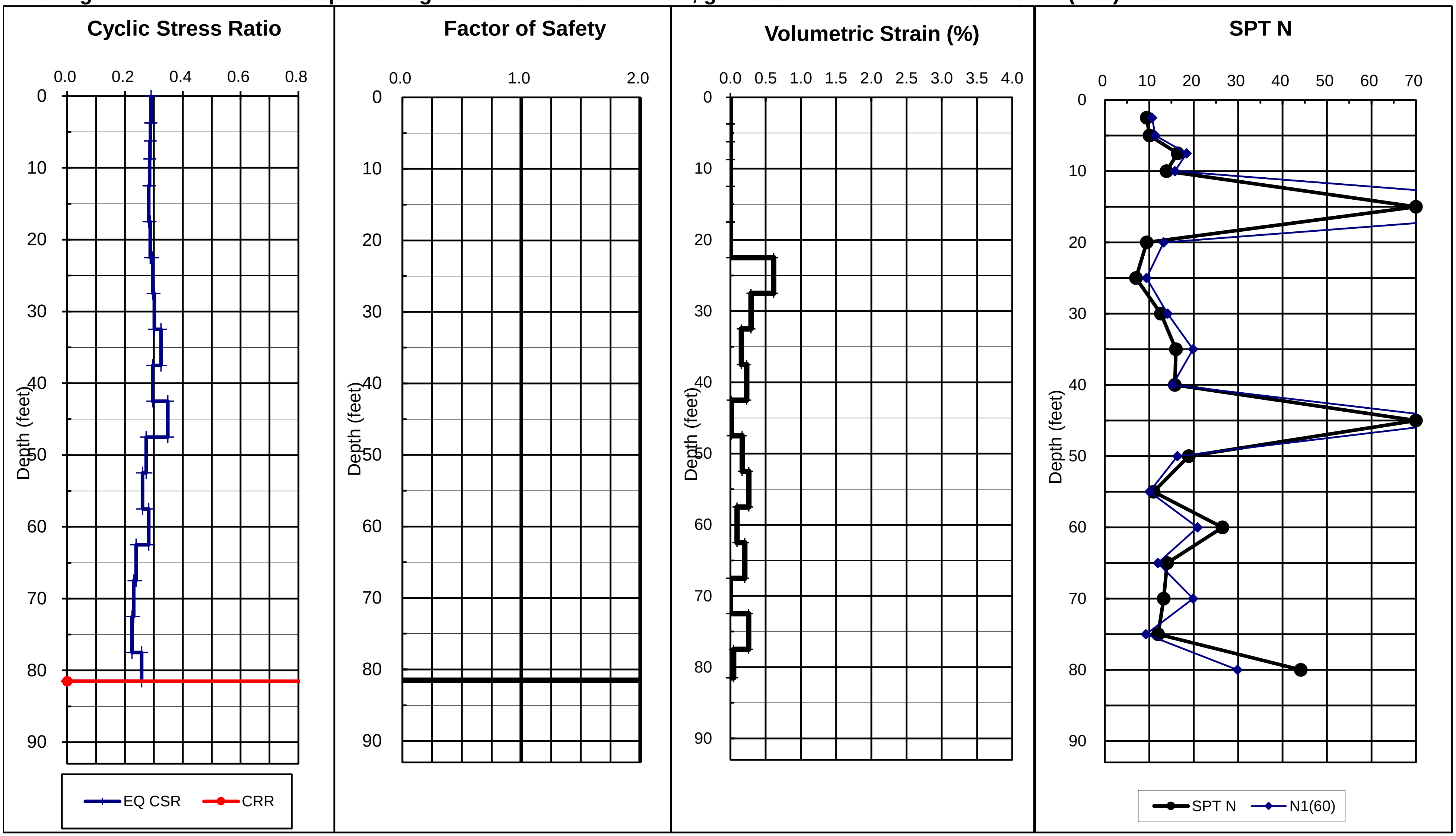
N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

6436 Hollywood Blvd. **Project No: LA-01670-01** 1996/1998 NCEER Method

Boring: B2 **Earthquake Magnitude: 6.48** **PGA, g: 0.65** **Calc GWT (feet): 90**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 1.4 inches

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: 6436 Hollywood Blvd.
Job No: LA-01670-01
Date: 10/7/2016
Boring: B2 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 6.48 7.5
PGA, g: 0.651 0.45
MSF: 1.45
GWT: 100.0 feet
Calc GWT: 90.0 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.50 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

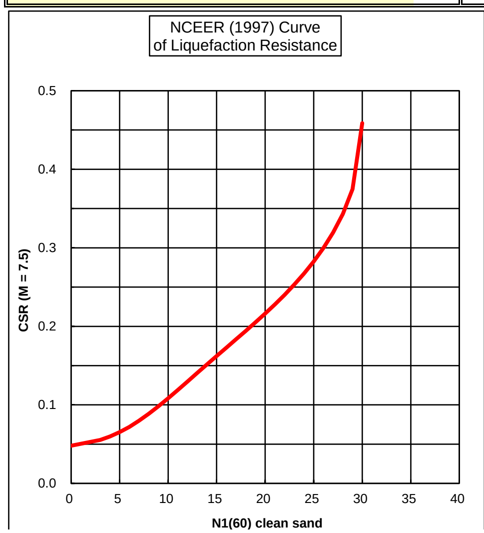
Threshold Acceler., g: #N/A
Required SF: 1.10
Minimum Calculated SF: #N/A

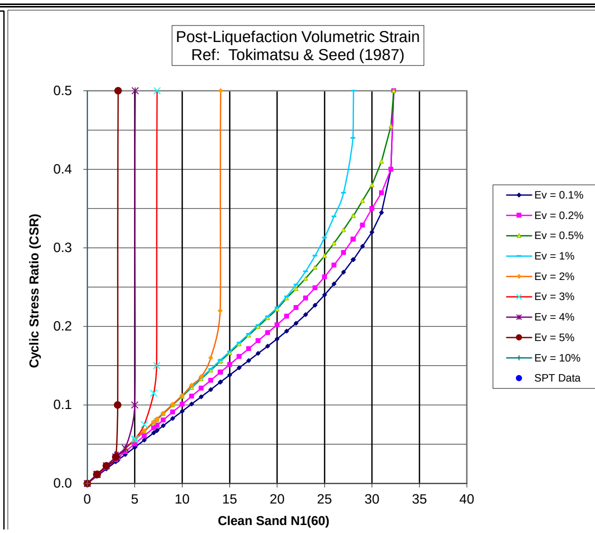
Total (ft) Liquefied Thickness	**Total (in.) Induced Subsidence**
0	1.4
	upper 90ft

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 7.2

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
								0.000																								
3.8	15	9		119	25	2.5	5.5	0.149	0.149	1.00	1.00	0.75	1.00	10.6				1.00	Infin.	0.290	Non-Liq.		10.6	0.00	0.00	0.100						
6.3	16	10		113	25	5.0	8.0	0.293	0.293	0.99	1.00	0.75	1.00	11.3				1.00	Infin.	0.288	Non-Liq.		11.3	0.00	0.00	0.197						
8.8	26	16		124	25	7.5	10.5	0.441	0.441	0.98	1.00	0.75	1.00	18.4				1.00	Infin.	0.287	Non-Liq.		18.4	0.00	0.00	0.296						
12.5	22	14		130	25	10.0	13.0	0.599	0.599	0.98	1.00	0.76	1.00	15.8				1.00	Infin.	0.285	Non-Liq.		15.8	0.00	0.00	0.402						
17.5		70		128	25	15.0	18.0	0.921	0.921	0.97	1.00	0.86	1.30	117.9				1.00	Infin.	0.282	Non-Liq.		117.9	0.00	0.00	0.617						
22.5	15	9		126	25	20.0	23.0	1.237	1.237	0.96	1.00	0.93	1.00	13.2				0.97	Infin.	0.287	Non-Liq.		13.2	0.00	0.00	0.829						
27.5		7	1	131	25	25.0	28.0	1.558	1.558	0.94	0.82	0.98	1.10	9.4	37	5.4	14.7	0.93	0.159	0.296	Non-Liq.	5.4	14.7	0.61	0.37	1.044	1,120	0.621	3.0E-03	4.3E-03	3.1E-03	0.37
32.5	20	13	1	136	25	30.0	33.0	1.891	1.891	0.92	0.75	1.00	1.00	14.1	45	5.9	20.0	0.89	0.217	0.301	Non-Liq.	5.9	20.0	0.29	0.18	1.267	1,367	0.737	2.0E-03	2.0E-03	1.5E-03	0.18
37.5		16	1	130	25	35.0	38.0	2.224	2.224	0.89	0.69	1.00	1.20	19.8	53	6.6	26.4	0.80	0.308	0.324	Non-Liq.	6.6	26.4	0.15	0.09	1.490	1,625	0.838	1.5E-03	1.1E-03	7.7E-04	0.09
42.5	25	16	1	125	25	40.0	43.0	2.543	2.543	0.85	0.65	1.00	1.00	15.2	47	6.0	21.3	0.84	0.231	0.295	Non-Liq.	6.0	21.3	0.23	0.14	1.704	1,617	0.916	1.7E-03	1.6E-03	1.2E-03	0.14
47.5		70	1	127	25	45.0	48.0	2.857	2.857	0.80	0.61	1.00	1.30	83.1	100	10.0	93.1	0.67	1.200	0.348	Non-Liq.	10.0	93.1	0.01	0.01	1.914	2,803	0.972	5.5E-04	8.6E-05	6.2E-05	0.01
52.5	30	19	1	129	25	50.0	53.0	3.177	3.177	0.75	0.58	1.00	1.00	16.4	48	6.2	22.5	0.80	0.247	0.273	Non-Liq.	6.2	22.5	0.17	0.10	2.129	1,842	1.012	1.3E-03	1.2E-03	8.3E-04	0.10
57.5		11	1	131	37	55.0	58.0	3.502	3.502	0.70	0.55	1.00	1.11	10.1	38	7.0	17.1	0.79	0.184	0.260	Non-Liq.	7.0	17.1	0.26	0.16	2.346	1,763	1.042	1.5E-03	1.8E-03	1.3E-03	0.16
62.5	42	26	1	132	25	60.0	63.0	3.831	3.831	0.66	0.53	1.00	1.00	20.9	55	6.7	27.5	0.68	0.332	0.282	Non-Liq.	6.7	27.5	0.09	0.06	2.567	2,163	1.069	9.5E-04	6.5E-04	4.6E-04	0.06
67.5		14	1	134	25	65.0	68.0	4.164	4.164	0.62	0.50	1.00	1.13	11.9	41	5.7	17.6	0.76	0.190	0.238	Non-Liq.	5.7	17.6	0.20	0.12	2.790	1,942	1.096	1.2E-03	1.4E-03	1.0E-03	0.12
72.5	21	13		136	50	70.0	73.0	4.501	4.501	0.59	1.00	1.00	1.00	19.8				0.75	Infin.	0.230	Non-Liq.		19.8	0.00	0.00	3.016						
77.5		12	1	138	25	75.0	78.0	4.843	4.843	0.57	0.47	1.00	1.10	9.3	36	5.4	14.6	0.74	0.158	0.224	Non-Liq.	5.4	14.6	0.26	0.16	3.245	1,969	1.163	1.2E-03	1.8E-03	1.3E-03	0.16
81.5	70	44	1	140	25	80.0	83.0	5.191	5.191	0.55	0.45	1.00	1.00	29.9	65	7.7	37.6	0.62	1.200	0.257	Non-Liq.	7.7	37.6	0.05	0.02	3.478	2,793	1.204	6.7E-04	3.1E-04	2.3E-04	0.02





$N_{1(60)} = C_N{*}C_E{*}C_B{*}C_R{*}C_S{*}N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= $\min(1,\max(0.75,1.4666-2.556/(z(ft))^{0.5}))$

$C_N = (1\ atm/p'o)^{0.5}$, max 1.7

$C_S = \max(1.1,\min(1.3,1+N_{1(60)}/100))$ for SPT without liners

$MSF = 10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

$p = 0.67{*}po$

$\tau_{av} = 0.65{*}PGA{*}po{*}rd$

$G_{max} = 447{*}N_{1(60)CS}{}^{(1/3)}{*}p^{0.5}$

$a = 0.0389{*}(p/1)+0.124$

$b = 6400{*}(p/1)^{(-0.6)}$

$\gamma = [1+a{*}EXP(b{*}\tau_{av}/G_{max})]/[(1+a){*}\tau_{av}/G_{max}]$

$E_{15} = \gamma{*}(N_{1(60)CS}/20)^{-1.2}$

$E_{nc} = (Nc/15)^{0.45}{*}E15$

$Nc = (MAG-4)^{2.17}$

$S = 2{*}H{*}E_{nc}$

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

$K\sigma$ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

$Dr = (N_{1(60)}/70)^{0.5}$

$CSReq = 0.65{*}PGA{*}(po/p'o){*}rd$

$CSR^* = CSReq/MSF/K\sigma$

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

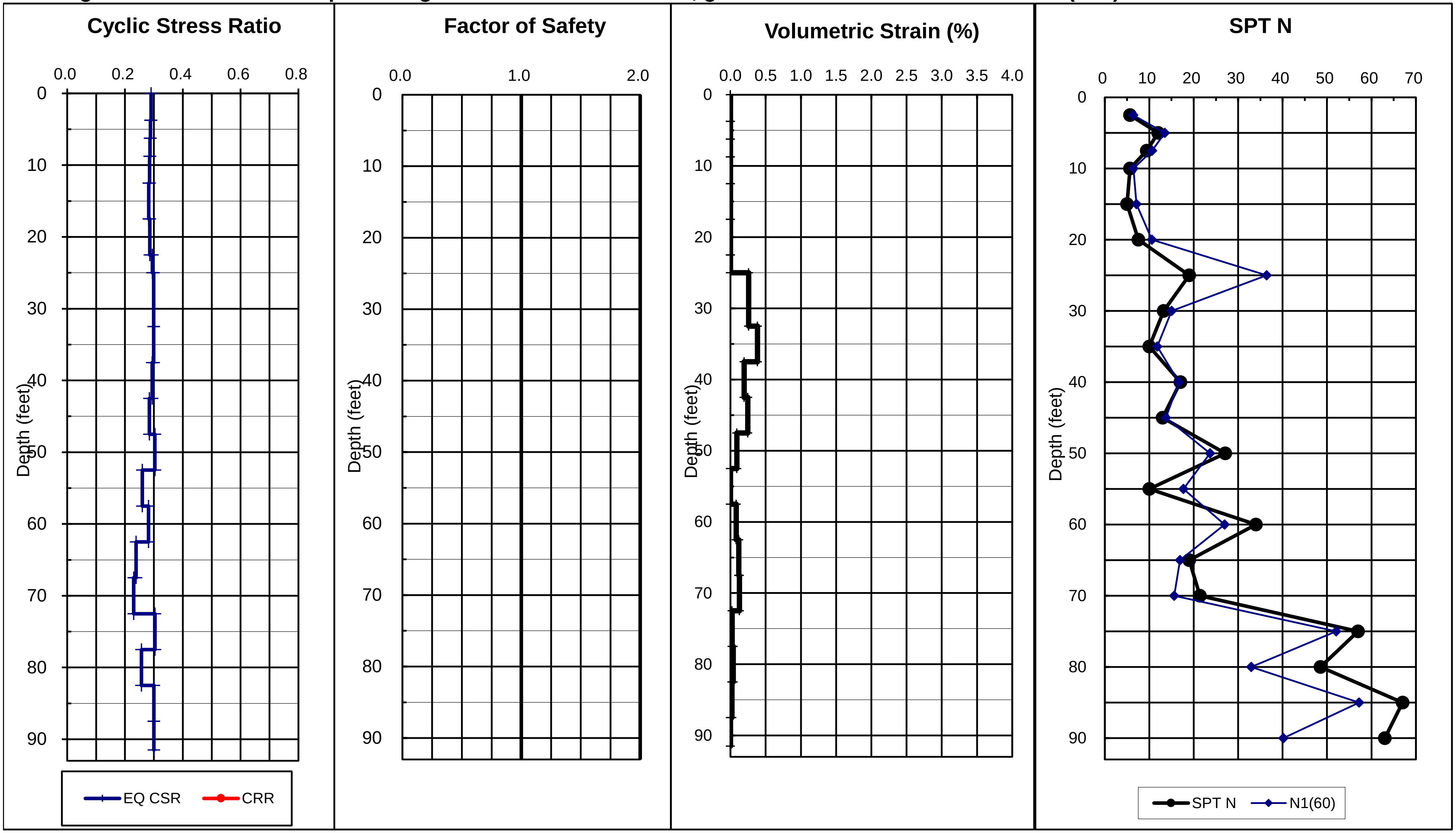
$N = N_{1(60)CS}$

$SF = CRR_{7.5,1atm}/CSR^*$

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

6436 Hollywood Blvd. **Project No: LA-01670-01** 1996/1998 NCEER Method

Boring: B4 **Earthquake Magnitude: 6.48** **PGA, g: 0.65** **Calc GWT (feet): 90**



Total Thickness of Liquefiable Layers: 0.0 feet **Estimated Total Ground Subsidence: 1.0 inches**

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Project: 6436 Hollywood Blvd.
Job No: LA-01670-01
Date: 10/7/2016
Boring: B4 Data Set: 2

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 6.48 7.5
PGA, g: 0.651 0.45
MSF: 1.45
GWT: 91.5 feet
Calc GWT: 90.0 feet
Remediate to: 0.0 **feet**

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.50 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: **#N/A**

Total (ft) Liquefied Thickness: **0**

Required SF: 1.10
Minimum Calculated SF: **#N/A**

Total (in.) Induced Subsidence: **1.0**
upper 90ft

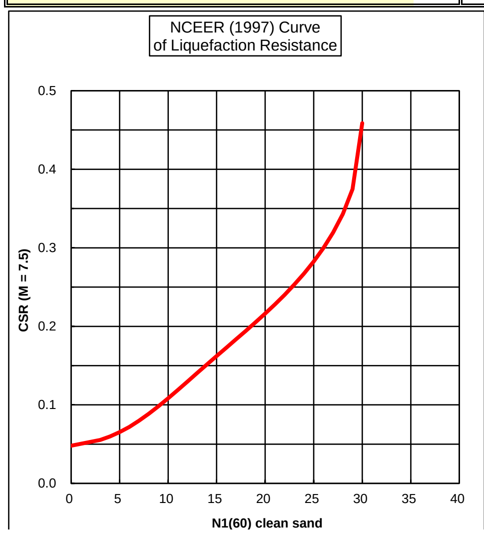
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

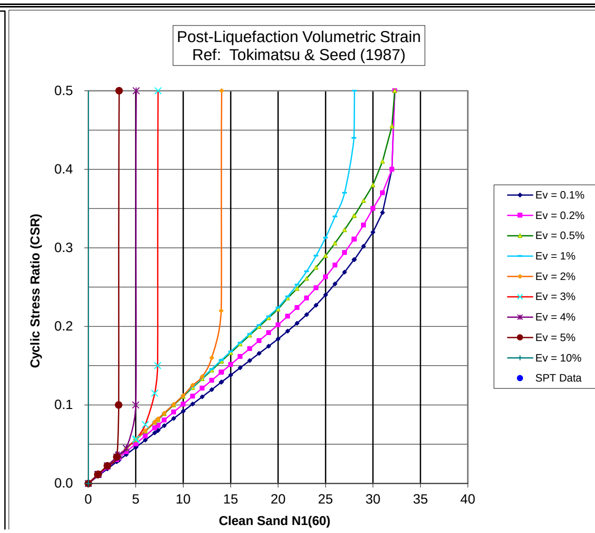
Nc = 7.2

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$
								0.000						
3.8	9	**6**		120	25	2.5	5.5	0.150	0.150	1.00	1.00	0.75	1.00	6.4
6.3	19	**12**		111	12	5.0	8.0	0.294	0.294	0.99	1.00	0.75	1.00	13.5
8.8	15	**9**		112	12	7.5	10.5	0.434	0.434	0.98	1.00	0.75	1.00	10.6
12.5	9	**6**		127	25	10.0	13.0	0.583	0.583	0.98	1.00	0.76	1.00	6.4
17.5		**5**		124	25	15.0	18.0	0.896	0.896	0.97	1.00	0.86	1.10	7.1
22.5	12	**8**		121	12	20.0	23.0	1.202	1.202	0.96	1.00	0.93	1.00	10.6
25.0		**19**		126	12	25.0	28.0	1.510	1.510	0.94	1.00	0.98	1.30	36.4
32.5	21	**13**	1	131	25	30.0	33.0	1.839	1.839	0.92	0.76	1.00	1.00	15.1
37.5		**10**	1	127	25	35.0	38.0	2.161	2.161	0.89	0.70	1.00	1.13	11.8
42.5	27	**17**	1	122	25	40.0	43.0	2.473	2.473	0.85	0.65	1.00	1.00	16.7
47.5		**13**	1	130	25	45.0	48.0	2.789	2.789	0.80	0.62	1.00	1.14	13.7
52.5	43	**27**	1	138	25	50.0	53.0	3.123	3.123	0.75	0.58	1.00	1.00	23.7
57.5		**10**		135	50	55.0	58.0	3.464	3.464	0.70	1.00	1.00	1.18	17.7
62.5	54	**34**	1	133	12	60.0	63.0	3.799	3.799	0.66	0.53	1.00	1.00	26.9
67.5		**19**	1	135	25	65.0	68.0	4.134	4.134	0.62	0.51	1.00	1.17	16.9
72.5	34	**21**	1	138	25	70.0	73.0	4.476	4.476	0.59	0.49	1.00	1.00	15.6
77.5		**57**	1	139	12	75.0	78.0	4.822	4.822	0.57	0.47	1.00	1.30	52.1
82.5	77	**49**	1	140	25	80.0	83.0	5.170	5.170	0.55	0.45	1.00	1.00	32.9
87.5		**67**	1	136	12	85.0	88.0	5.516	5.516	0.53	0.44	1.00	1.30	57.2
91.5	100	**63**	1	133	12	90.0	93.0	5.852	5.852	0.52	0.43	1.00	1.00	40.2

Base Depth (feet)	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**
3.8				1.00	Infin.	0.290	**Non-Liq.**		6.4	0.00	**0.00**
6.3				1.00	Infin.	0.288	**Non-Liq.**		13.5	0.00	**0.00**
8.8				1.00	Infin.	0.287	**Non-Liq.**		10.6	0.00	**0.00**
12.5				1.00	Infin.	0.285	**Non-Liq.**		6.4	0.00	**0.00**
17.5				1.00	Infin.	0.282	**Non-Liq.**		7.1	0.00	**0.00**
22.5				0.97	Infin.	0.286	**Non-Liq.**		10.6	0.00	**0.00**
25.0				0.93	Infin.	0.294	**Non-Liq.**		36.4	0.00	**0.00**
32.5	46	6.0	21.1	0.90	0.229	0.299	**Non-Liq.**	6.0	21.1	0.26	**0.23**
37.5	41	5.6	17.5	0.87	0.188	0.299	**Non-Liq.**	5.6	17.5	0.38	**0.23**
42.5	49	6.2	22.9	0.84	0.252	0.294	**Non-Liq.**	6.2	22.9	0.19	**0.12**
47.5	44	5.9	19.6	0.82	0.212	0.284	**Non-Liq.**	5.9	19.6	0.25	**0.15**
52.5	58	7.0	30.7	0.72	1.200	0.303	**Non-Liq.**	7.0	30.7	0.09	**0.05**
57.5				0.79	Infin.	0.260	**Non-Liq.**		17.7	0.00	**0.00**
62.5	62	2.4	29.3	0.68	0.390	0.282	**Non-Liq.**	2.4	29.3	0.08	**0.05**
67.5	49	6.2	23.1	0.76	0.255	0.238	**Non-Liq.**	6.2	23.1	0.12	**0.07**
72.5	47	6.1	21.7	0.75	0.236	0.230	**Non-Liq.**	6.1	21.7	0.13	**0.08**
77.5	86	3.2	55.3	0.55	1.200	0.303	**Non-Liq.**	3.2	55.3	0.02	**0.01**
82.5	69	8.1	41.0	0.62	1.200	0.257	**Non-Liq.**	8.1	41.0	0.04	**0.02**
87.5	90	3.4	60.6	0.52	1.200	0.300	**Non-Liq.**	3.4	60.6	0.02	**0.01**
91.5	76	2.8	43.0	0.50	1.200	0.300	**Non-Liq.**	2.8	43.0	0.00	**0.00**

Base Depth (feet)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
3.8	0.100						
6.3	0.197						
8.8	0.291						
12.5	0.390						
17.5	0.600						
22.5	0.805						
25.0	1.012						
32.5	1.232	1,370	0.716	1.9E-03	1.8E-03	1.3E-03	**0.23**
37.5	1.448	1,396	0.815	2.3E-03	2.7E-03	1.9E-03	**0.23**
42.5	1.657	1,634	0.891	1.6E-03	1.4E-03	9.7E-04	**0.12**
47.5	1.868	1,648	0.948	1.7E-03	1.7E-03	1.2E-03	**0.15**
52.5	2.092	2,024	0.995	1.0E-03	6.2E-04	4.5E-04	**0.05**
57.5	2.321						
62.5	2.545	2,199	1.060	9.0E-04	5.7E-04	4.1E-04	**0.05**
67.5	2.770	2,120	1.088	9.9E-04	8.3E-04	6.0E-04	**0.07**
72.5	2.999	2,159	1.121	9.8E-04	8.9E-04	6.4E-04	**0.08**
77.5	3.231	3,060	1.158	5.5E-04	1.6E-04	1.2E-04	**0.01**
82.5	3.464	2,869	1.199	6.4E-04	2.7E-04	1.9E-04	**0.02**
87.5	3.695	3,375	1.243	5.2E-04	1.4E-04	9.8E-05	**0.01**
91.5	3.921	3,101	1.286	6.1E-04			





$N_{1(60)} = C_N*C_E*C_B*C_R*C_S*N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
$= \min(1,\max(0.75,1.4666-2.556/(z(ft))^{0.5}))$

$C_N = (1\ atm/p'o)^{0.5}$, max 1.7

$C_S = \max(1.1,\min(1.3,1+N_{1(60)}/100))$ for SPT without liners

$MSF = 10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

$p = 0.67*po$

$\tau_{av} = 0.65*PGA*po*rd$

$G_{max} = 447*N_{1(60)CS}^{(1/3)}*p^{0.5}$

$a = 0.0389*(p/1)+0.124$

$b = 6400*(p/1)^{(-0.6)}$

$\gamma = [1+a*EXP(b*\tau_{av}/G_{max})]/[(1+a)*\tau_{av}/G_{max}]$

$E_{15} = \gamma*(N_{1(60)CS}/20)^{-1.2}$

$E_{nc} = (Nc/15)^{0.45}*E15$

$Nc = (MAG-4)^{2.17}$

$S = 2*H*E_{nc}$

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

$K\sigma$ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

$Dr = (N_{1(60)}/70)^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

$N = N_{1(60)CS}$

$SF = CRR_{7.5,1atm}/CSR^*$

APPENDIX F

Seismic Shear-Wave Survey

TERRA GEOSCIENCES

SEISMIC SHEAR-WAVE SURVEY

HOLLYWOOD AND WILCOX PROJECT

6436 HOLLYWOOD BOULEVARD

CITY OF LOS ANGLES, CALIFORNIA

Project No. 162901-1

August 29, 2016

Prepared for:

Earth Systems Southern California
2122 E. Walnut Street
Suite 200
Pasadena, CA 91107

Earth Systems Southern California
2122 E. Walnut Street, Suite 200
Pasadena, CA 91107

Attention: Mr. Christopher Allen

Regarding: Seismic Shear-Wave Survey
Hollywood and Wilcox Project
6436 Hollywood Boulevard
City of Los Angeles, California
ESSC Project No. LA-011670-03

INTRODUCTION

As requested, this firm has performed a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods for the above-referenced site. The purpose of this survey was to assess the one-dimensional average shear-wave velocity profile beneath the subject survey area to a minimum depth of 100 feet. Geologic mapping of the surficial earth materials by Yerkes (1997) indicates that the local survey area is underlain by undifferentiated Holocene age alluvium generally comprised of unconsolidated and uncemented gravel, sand, silt, and clay.

The location of the seismic traverses have been approximated on a captured Google™ Earth image (Google™ Earth, 2014) and appears as the Seismic Line Location Map, Plate 1. Additionally, site photographs of the survey lines have been included on Plate 2 for reference purposes. As authorized by you, the following services were performed during this study:

- ***Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included two traverses for shear-wave velocity analysis purposes.***
- ***Preparation of this report, presenting the results of our findings with respect to the shear-wave velocities of the subsurface earth materials.***

Accompanying Map, Illustrations, and Appendices

Plate 1 - Seismic Line Location Map
Plate 2 - Site Photographs
Appendix A - Seismic Line SW-1
Appendix B - Seismic Line SW-2
Appendix C - References

SUMMARY OF SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

Two survey traverses (Seismic Lines SW-1 and SW-2) were performed along the locations as selected by you and have been approximated on the Seismic Line Location Map (see Plate 1), each being 184 feet in length. For data collection, the field survey was performed using a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to insure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM survey lines used a series of twenty-four 4.5-Hz geophones that were spaced at regular 8-foot intervals. For the MASW survey, the ground vibrations were recorded using a one-second record length at a sampling rate of 0.5-milliseconds. Two separate seismic records were obtained with the shot point located at a distance of 30 feet off the end of each survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Three hammer impacts were stacked for each shot point to increase the signal to noise ratio to enhance the data and extend the penetration depth.

The MAM survey did not require the introduction of any artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 30 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2009). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave (V_s) model based on the observed data. These initial models were then inverted in order to converge on the best fit of the initial models and the observed data, creating the final V_s curves as presented within Appendices A and B.

The data acquisition went very smoothly and the quality was considered to be very good. Analysis of the final seismic models for both survey lines revealed that the average shear-wave velocity ("weighted average") increases with depth with no velocity reversals being encountered.

As tabulated below, the average shear-wave velocities within the upper 100 feet of the subject survey area range from **680.4** feet/second (Seismic Line SW-1) to **713.9** feet per second (Seismic Line SW-2), which are displayed on the shear-wave models within Appendices A and B for visual and reference purposes.

The "weighted average" velocity was computed from a formula that is used by the ASCE (2007; ASCE 7-05, 20.4.1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V_s100). This formula is as follows:

$$\mathbf{V100' = 100/[(T1/V1) + (T2/V2) + ...+ (TN/VN)]}$$

Where t1, t2, t3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The shear-wave models display these calculated layer depth boundaries and associated seismic velocities (feet/second) to the maximum data limits obtained which reached just over 200 feet for both survey lines. It should be noted that only the upper 100 feet of each survey line was used for the shear-wave velocity determination (V_s100) as discussed above.

The associated Active and Passive Dispersion Curve along with the resultant Combined Dispersion Curve for each seismic line are also included within Appendices A and B, which present the results and quality of the data used for analysis.

SEISMIC LINE SW-1

DEPTH RANGE (feet)	SHEAR-WAVE VELOCITY(feet/second)
0 – 25.0	517
25.0 – 66.7	610
66.7 – 125.0	1,096
125.0 – 200.0 (max depth limit)	1,440
Average Shear-Wave Velocity (0-100 feet): 680.4 ft/sec.	

SEISMIC LINE SW-2

DEPTH RANGE (feet)	SHEAR-WAVE VELOCITY(feet/second)
0 – 24.4	510
24.4 – 65.0	660
65.0 – 121.9	1,137
121.9 – 183.0 (max depth limit)	1,351
Average Shear-Wave Velocity (0-100 feet): 713.9 ft/sec.	

CLOSURE

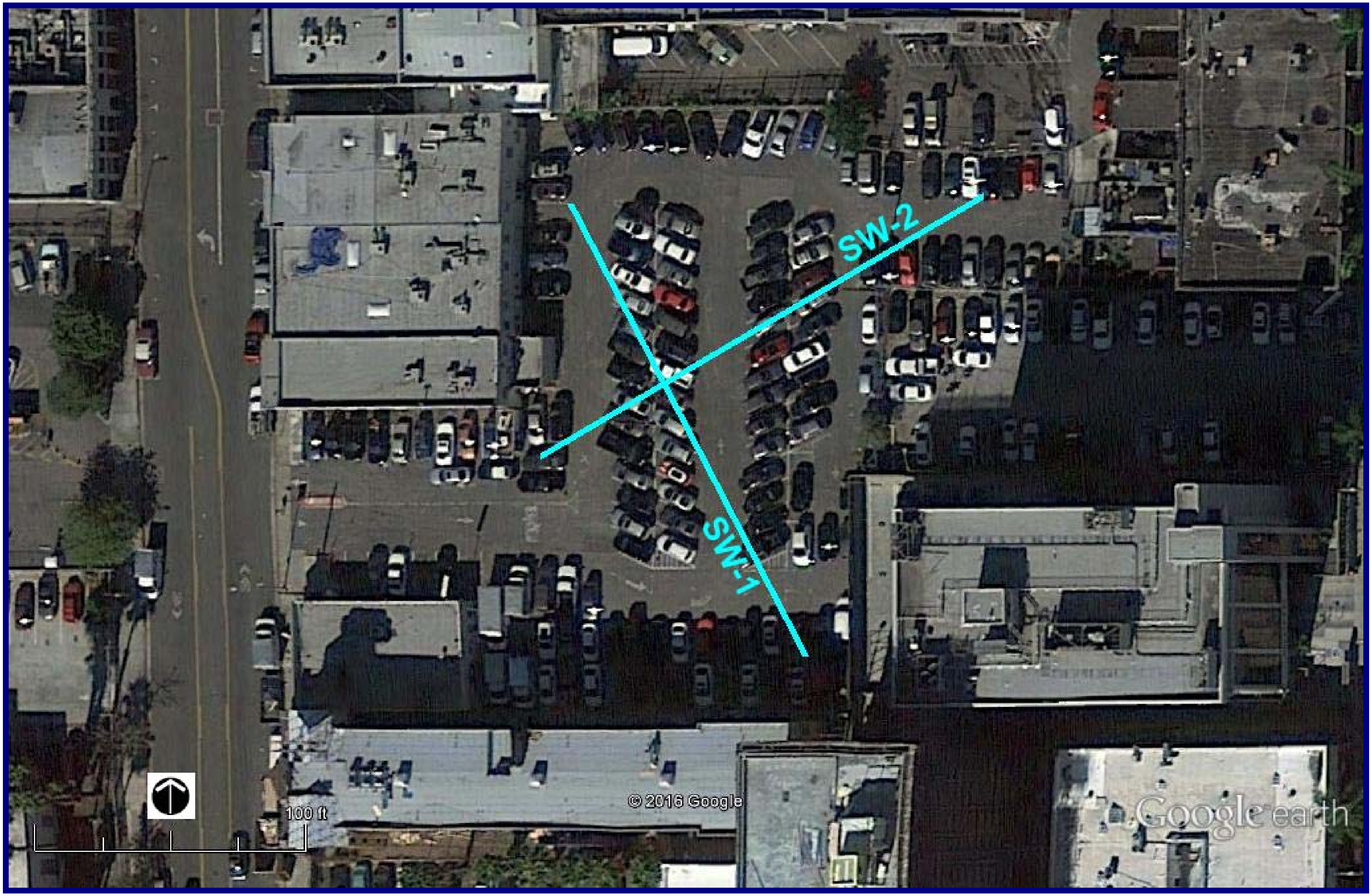
The field survey was performed by the undersigned on August 27, 2016, using "state of the art" geophysical equipment and techniques along the selected portion of the subject study area as directed by you. It is important to note that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and in the interpretation and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied. If the client does not understand the limitations of this geophysical survey, additional input should be sought from the consultant.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

SEISMIC LINE LOCATION MAP



PROJECT NO. 162901-1

PLATE 1

SITE PHOTOGRAPHS

View looking southeast along Seismic Line SW-1

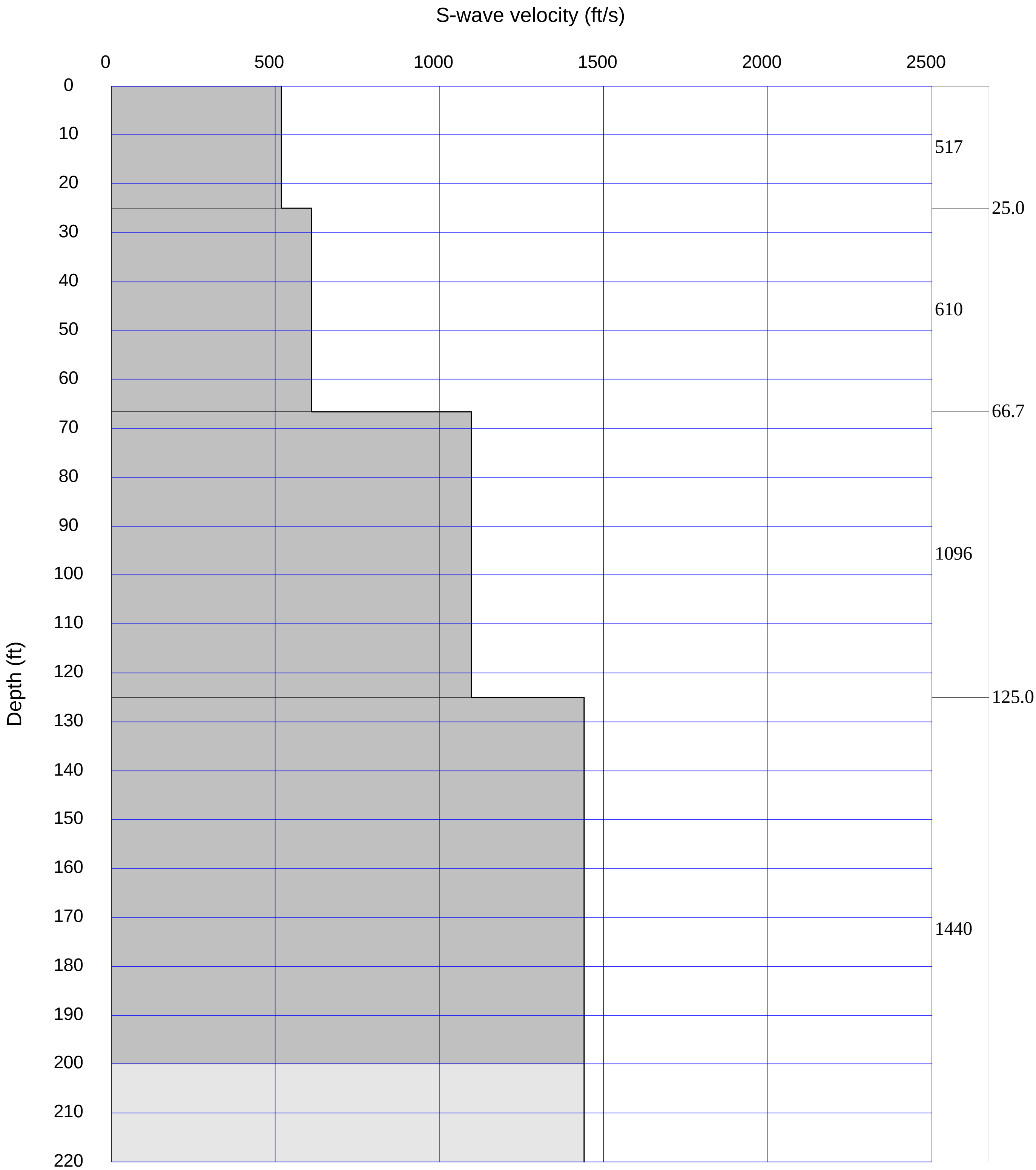
View looking southwest along Seismic Line SW-2

APPENDIX A

SEISMIC LINE SW-1

SEISMIC LINE SW-1

SHEAR-WAVE MODEL

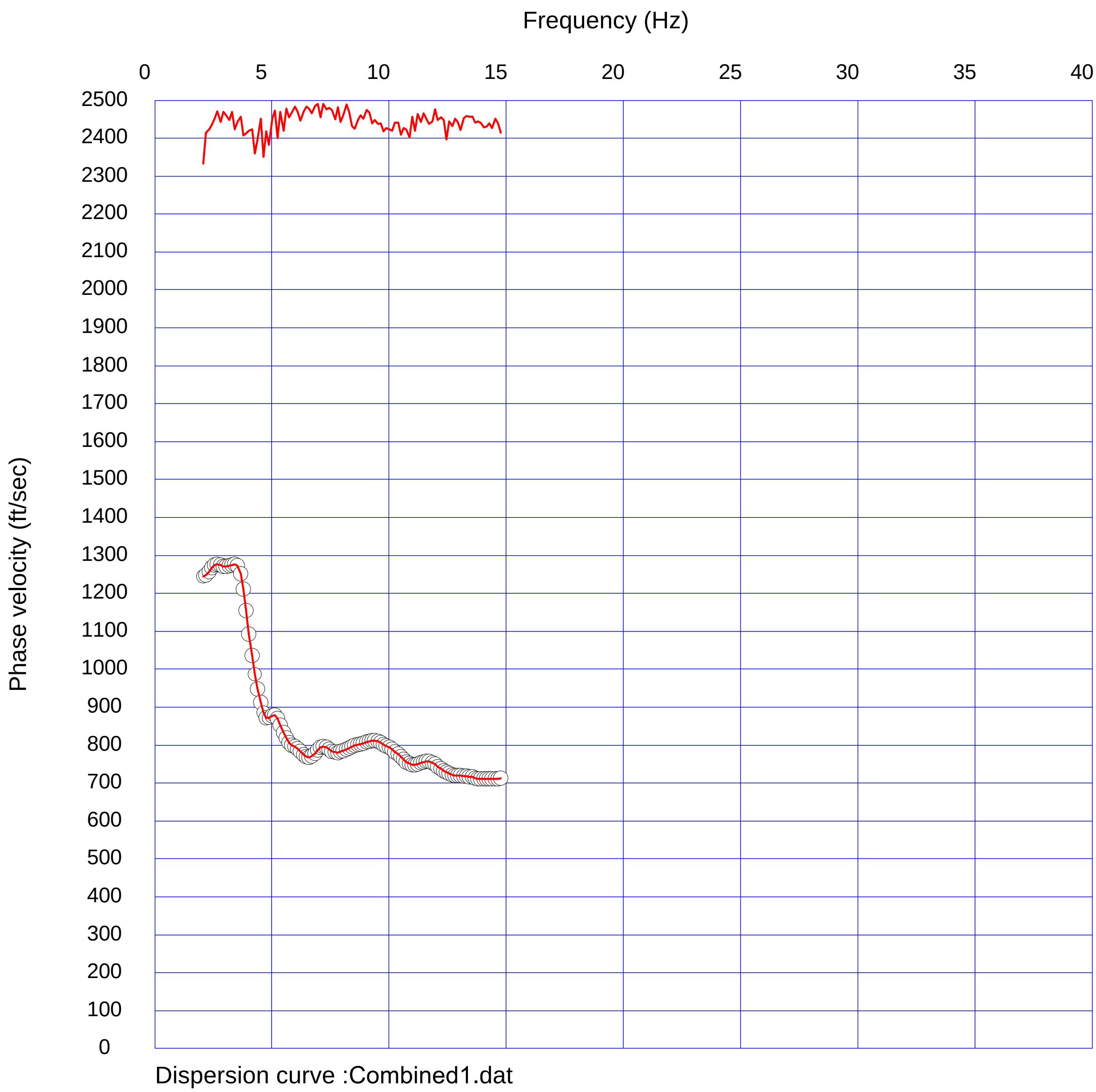


S-wave velocity model (inverted): Final1.rst

Average Vs 100ft = 680.4 ft/sec

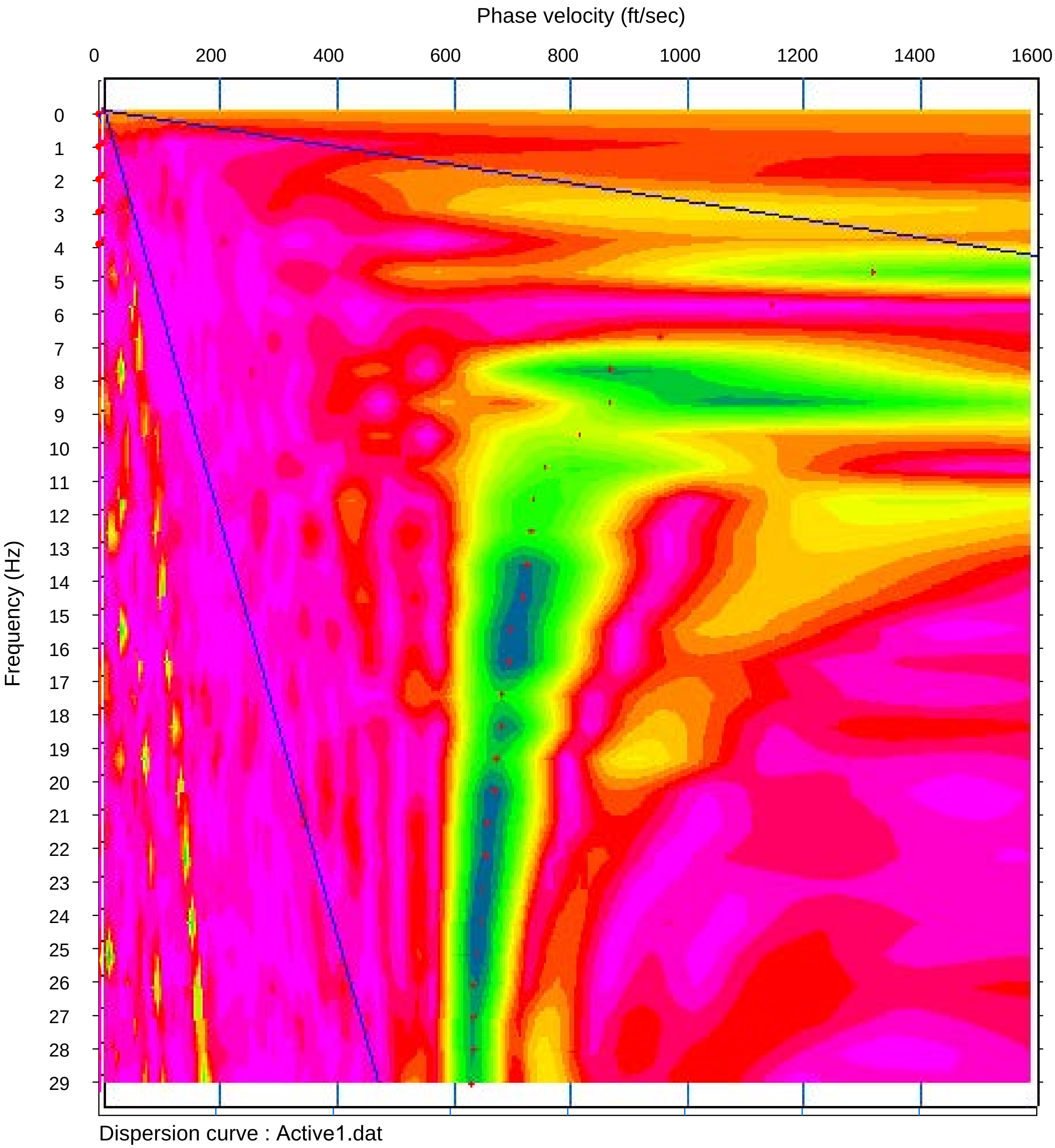
SEISMIC LINE SW-1

SHEAR-WAVE MODEL



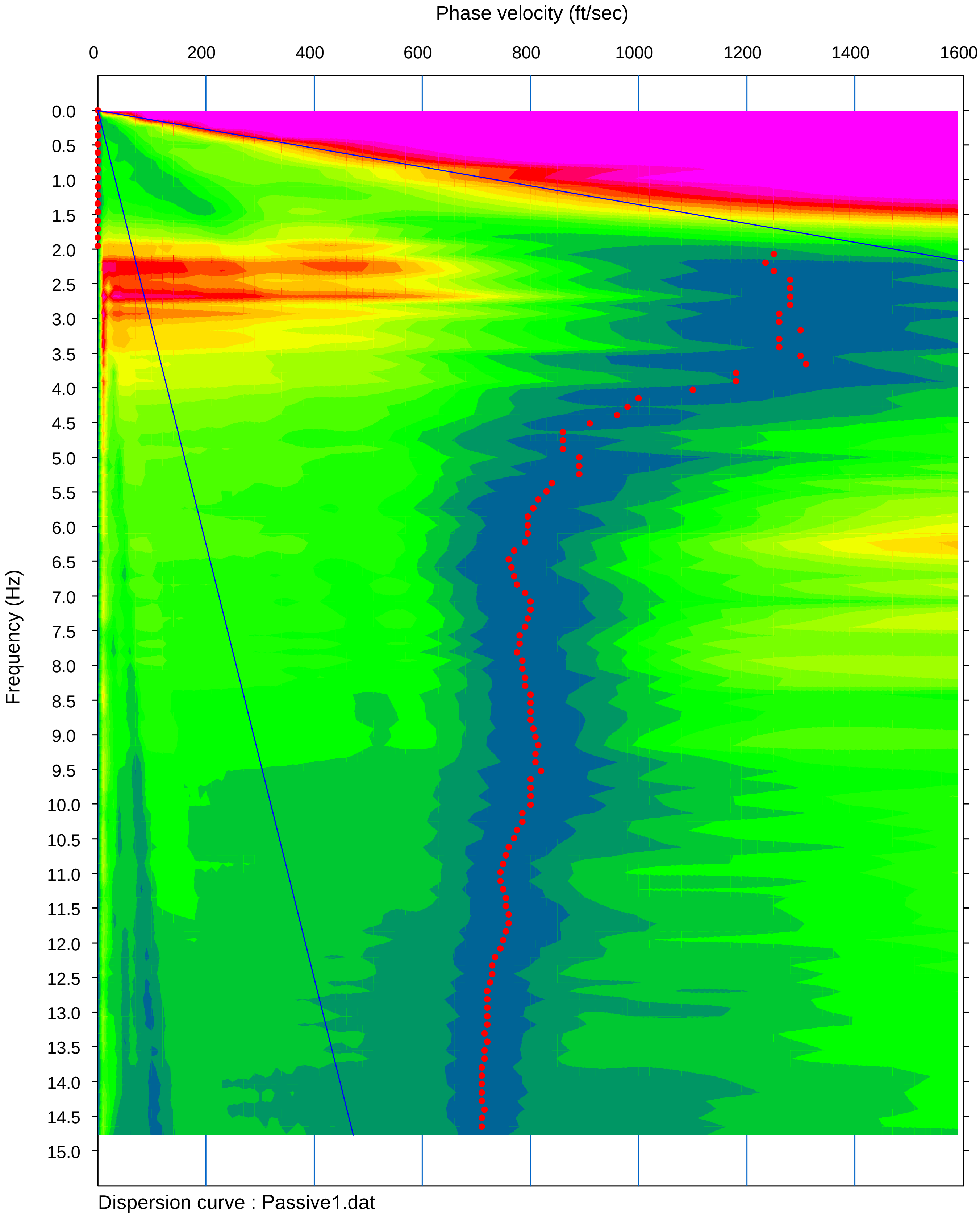
COMBINED DISPERSION CURVE

SEISMIC LINE SW-1



ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



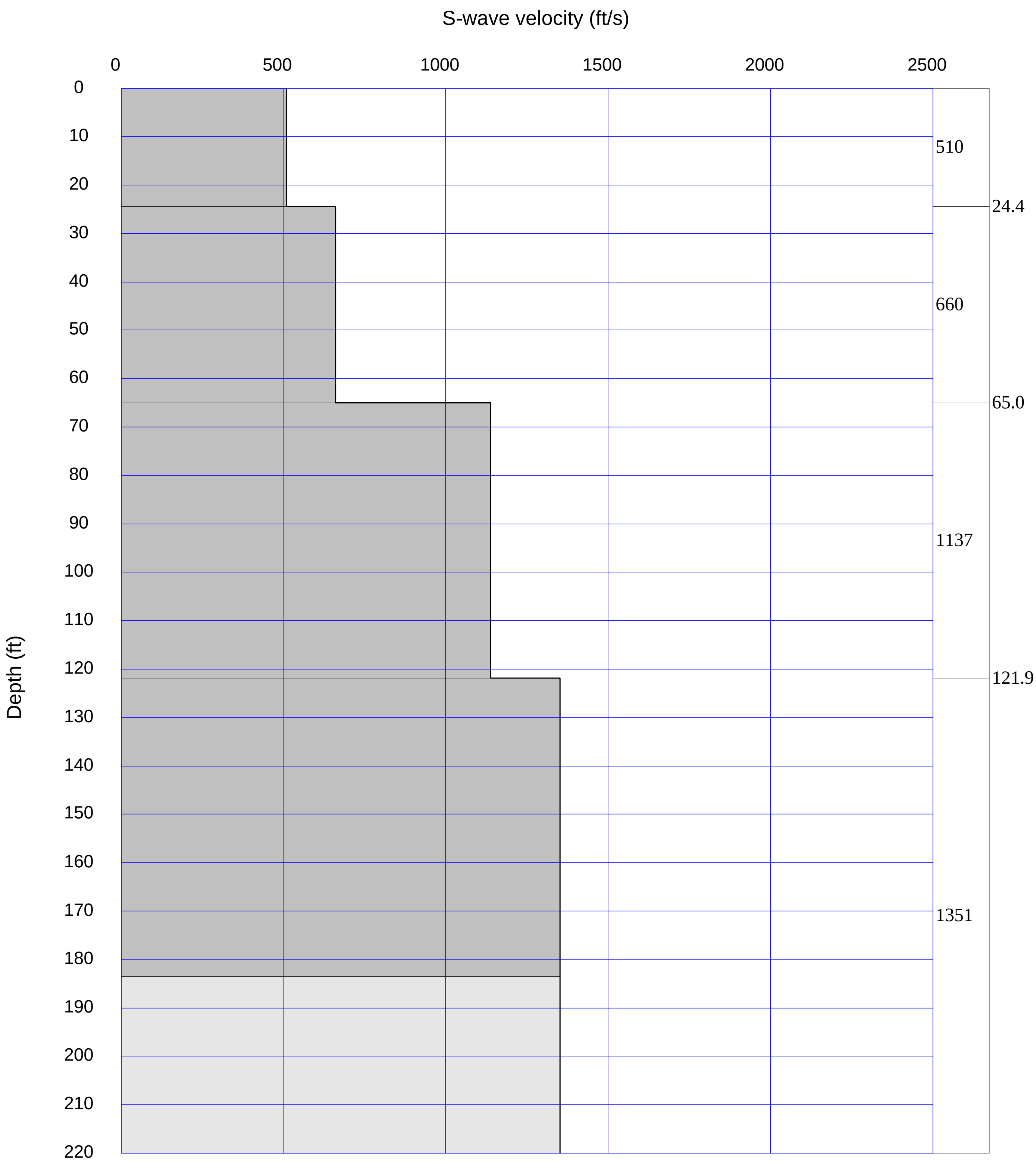
PASSIVE DISPERSION CURVE

APPENDIX B

SEISMIC LINE SW-2

SEISMIC LINE SW-2

SHEAR-WAVE MODEL

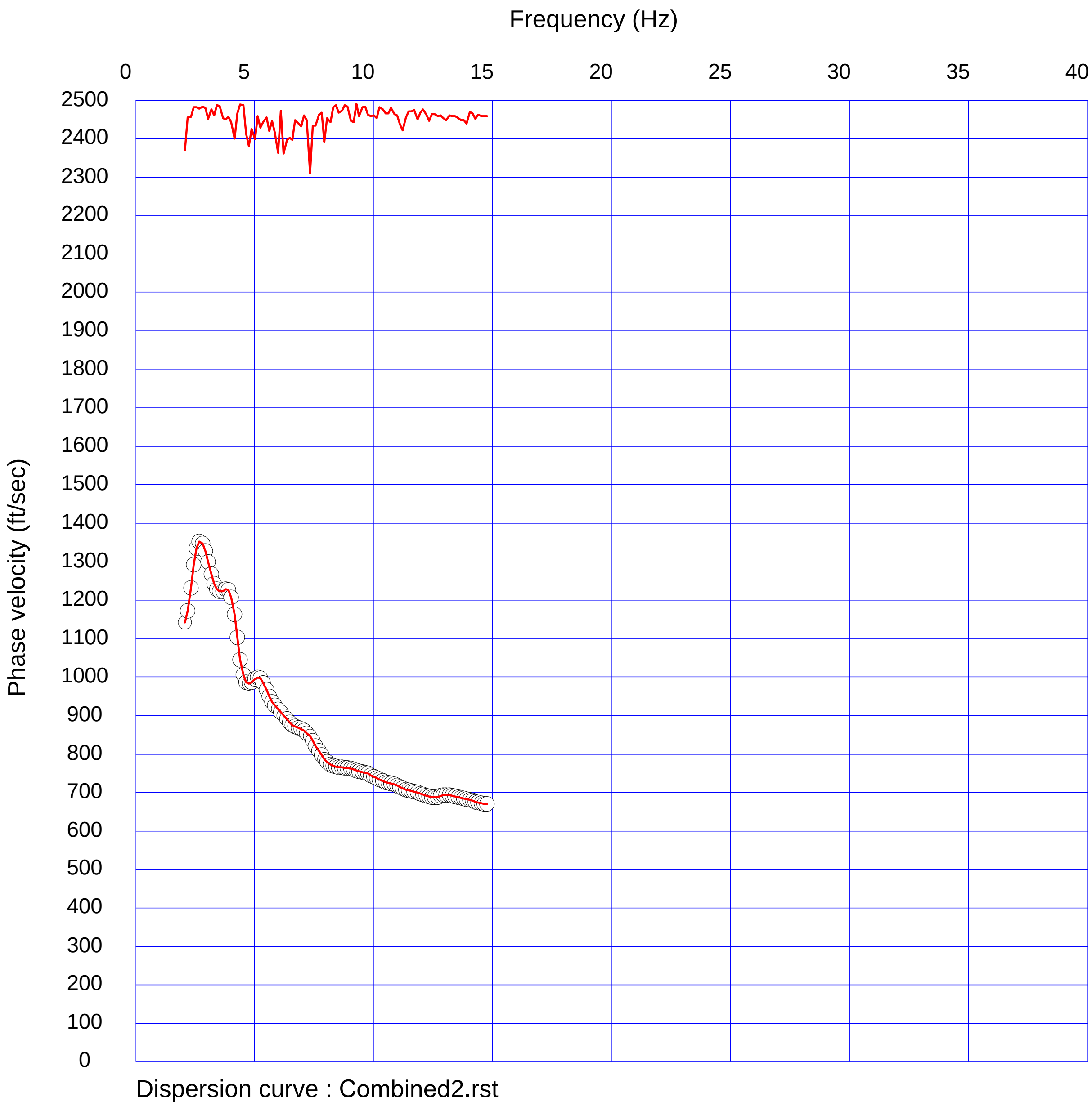


S-wave velocity model (inverted): Final2.rst

Average Vs 100ft = 713.9 ft/sec

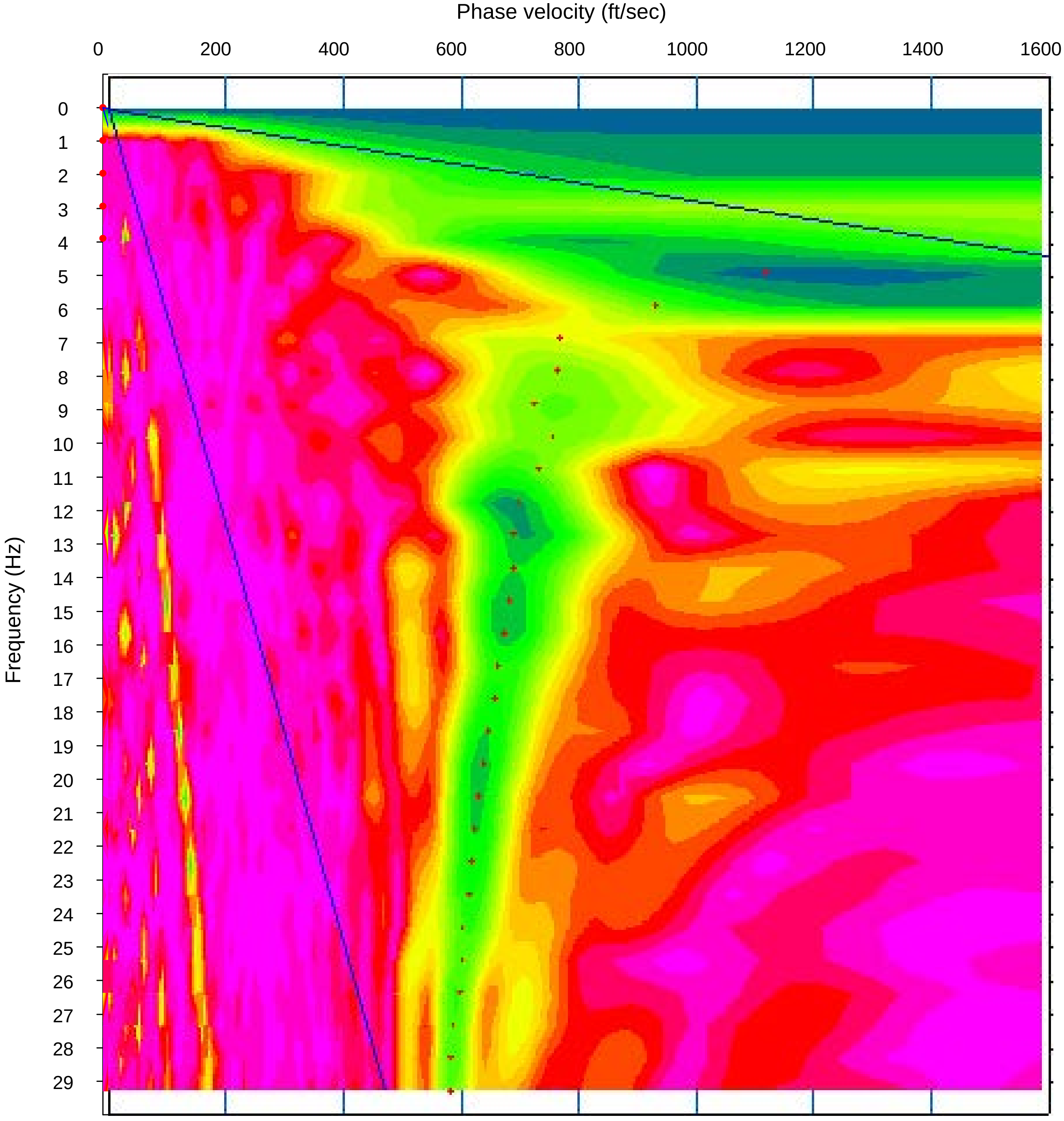
SEISMIC LINE SW-2

SHEAR-WAVE MODEL



COMBINED DISPERSION CURVE

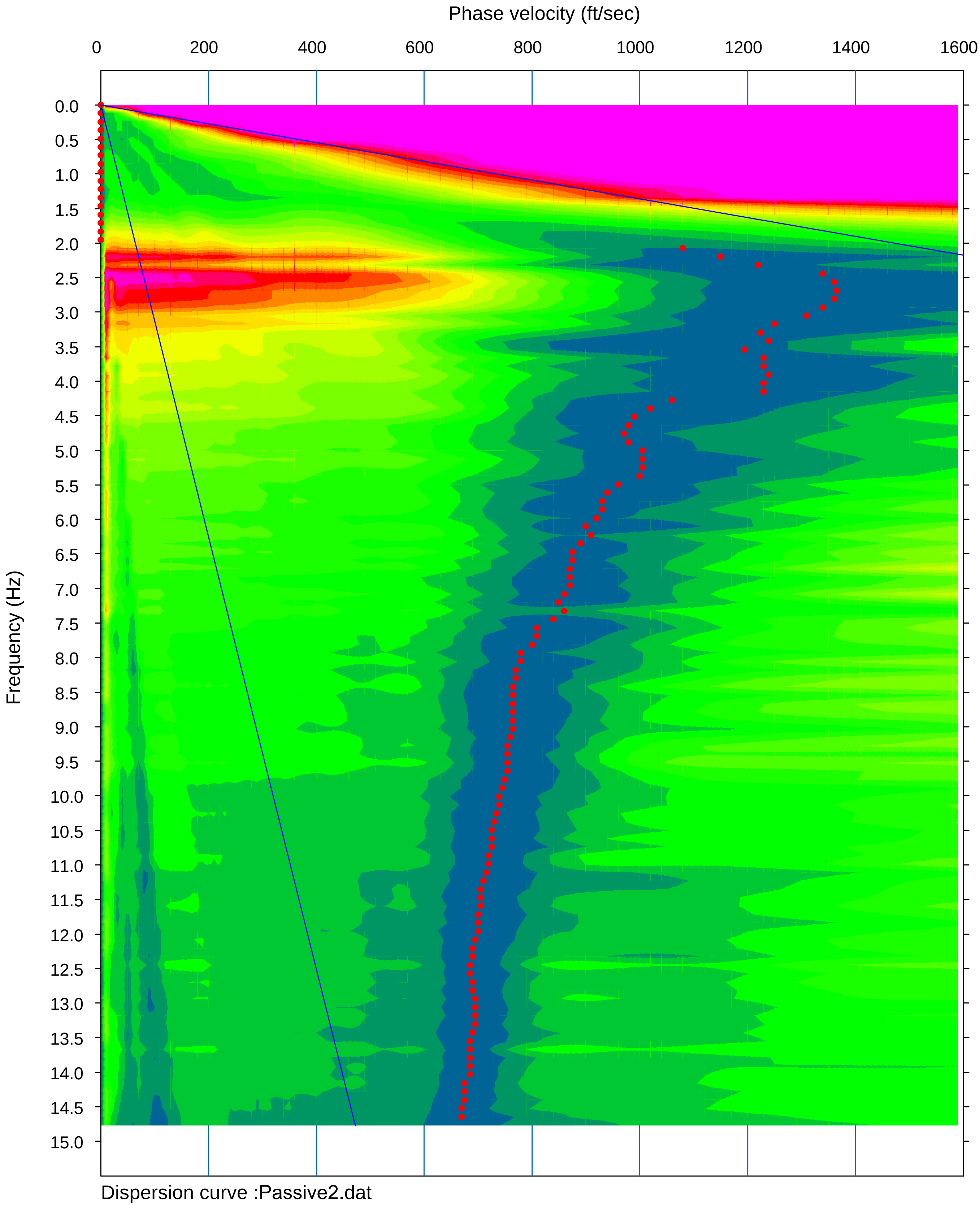
SEISMIC LINE SW-2



Dispersion curve Active2.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-2



PASSIVE DISPERSION CURVE

APPENDIX C

REFERENCES

REFERENCES

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