GeoTek, Inc.
710 E. Parkridge Avenue, Suite 105, Corona, California, 92879-1097
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

June 24, 2015
Project No. 1162-CR3

Meritage Homes
1250 Corona Pointe Court, Suite 210
Corona, California 92879

Attention: Mr. Kenneth Kim

Subject: Table of Contents
Encanto Project
Southwest Corner Commercecentre Drive and Alton Parkway
Lot 13 of Tract 13343
Lake Forest, Orange County, California

Dear Mr. Kim:

As requested by the City, we are providing a table of contents of the most pertinent documents relating to the site. They are listed in reverse chronological order.

- **Document A: GeoTek Response to City Review Checklist #2 (6/11/15)** ..... **Page 4**
- **Document B: City Review Checklist (5/26/15)** ..... **Page 9**
- **Document C: GeoTek Response to City Review Checklist (5/15/15)** ..... **Page 12**

  Note: Document C contains the reports prepared by Stoney-Miller, dated November 12, 2009 and June 9, 2104.
- **Stoney-Miller Review of Site Settlement (11/12/09)** ..... **Page 33**
- **Stoney-Miller Seismic Hazard Study (6/9/14)** ..... **Page 55**
- **Document D: City Review Checklist (11/6/14)** ..... **Page 266**
- **Document E: GeoTek Due Diligence Review (6/2/14)** ..... **Page 270**

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

Edward H. LaMont
CEG 1892, Exp. 7/31/12
Principal Geologist

Paul Hyun Jin Kim
PE 77214, Exp. 6/30/13
Project Engineer

Distribution: (1) Addressee via email

Enclosures: Documents A thru D

G:\Projects\1151 to 1200\1162CR3 Meritage Homes 25192 Commercentre Drive (Lot 13 of Tract 13343)\1162CR3 Meritage Homes Lake Forest Response to City Review 11062014.doc

Document A

GeoTek
Response to City Review Checklist #2
June 11, 2015

Encanto Project

Lot 13, Tract 13343

Lake Forest, Orange County, California

Project No. 1162-CR3

GEOTEK

June 11, 2015
Project No. 1162-CR3

Meritage Homes
1250 Corona Pointe Court, Suite 210
Corona, California 92879

Attention: Mr. Kenneth Kim

Subject: Response to City Review Checklist #2
Lot 13 of Tract 13343
Southwest Corner Commercentre Drive and Alton Parkway
Lake Forest, Orange County, California

References: *See Appendix A*

Dear Mr. Kim:

GeoTek, Inc. (GeoTek) is presenting this response to the Geotechnical Investigation Report Review Checklist, dated May 26, 2015, provided by Kling Consulting Group, Inc. on behalf of the City of Lake Forest, Department of Building and Safety. The responses are provided on an item by item basis as it pertains to the referenced soils reports in Appendix A.

Comment #1

Please clarify recommendations for site drainage. Refer to Section 1804.3 of the 2013 CBC as warranted.

Response

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times and be designed by the project civil engineer. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings. Soil areas within 10 feet of the proposed structure should slope at a minimum of 5-percent away from the building, if possible unless the area is paved. Paved areas are to be sloped at 2-percent away from the structure. Roof gutters and downspouts should discharge onto paved surfaces sloping away from the structure or into a

closed pipe system which outfalls to the street gutter pan or directly to the storm drain system. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

Comment #2

The consultant indicates that retaining walls up to 7 feet are proposed. Please provide lateral earth pressures for retaining walls due to earthquake motions in accordance with Section 1803.5.12 of the 2013 CBC.

Response

The new updated grading plan, dated June 4, 2015, now show retaining walls up to a maximum 6 feet.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

Edward H. LaMont
CEG 1892, Exp. 7/31/16
Principal Geologist

Paul Hyun Jin Kim
PE 77214, Exp. 6/30/15
Project Engineer

Distribution: (1) Addressee via email

Attachments: Grading Plan, dated June 4, 2015

G:\Projects\1151 to 1200\1162CR3 Meritage Homes 25192 Commercentre Drive (Lot 13 of Tract 13343)\1162CR3 Meritage Homes Lake Forest Response to City Review #2.doc

Bagahi Engineering, 1997, "Geotechnical Investigation of Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, CA," Job No. 82a-200-00, dated June 16.

____, 1998, "Progress Report on Monitoring of Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, California," Job No. 82a-200-00, dated October 9.

GeoTek, Inc., 2015, "Response to City Review Checklist, Lot 13, Tract 13343, Southwest Corner Commercentre Drive and Alton Parkway, Lake Forest, California," Project No: 1162-CR3, dated May 15.

Stoney-Miller Consultants, Inc., 1993, "Settlement Monitoring, Canyon 2, Lots 12-20, Tract 13343, 20858 Bake Parkway, El Toro, California," Project No: 10425-06, dated December 20.

____, 1997, "Settlement Monitoring, Canyon 2, Lots 12 through 20, Tract 13343, 20858 Bake Parkway, Pacific Commercentre, Lake Forest, California," Project No: 10425-07, dated March 7.

____, 2005, "Results of Preliminary Geotechnical Investigation, 25192 Commercentre Drive, Lake Forest, California," Project No: 12576-00, dated May 3.

____, 2006, "Site History & Future Geotechnical Work, Lot 13 of Tract 13343, 25192 Commercentre Drive, Lake Forest, California," Project No: 12655-02, dated January 24.

____, 2007, "Supplemental Geotechnical Investigation, Lot 13 of Tract 13343, 25191 Commercentre Drive, Lake Forest, California," Project No: 12655-03, dated November 12.

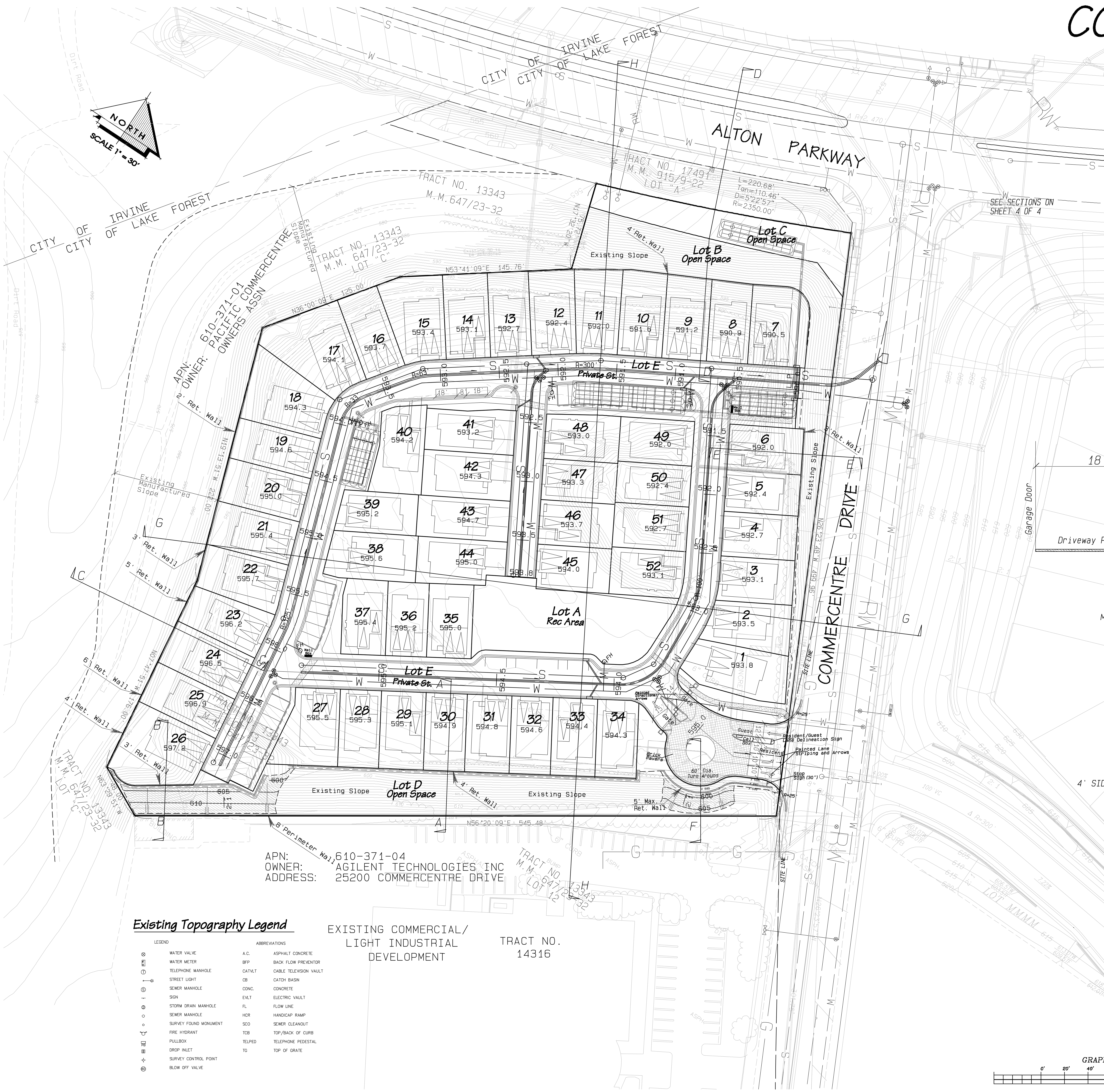
____, 2008, "Grading Earthwork Observation/Testing Report, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-10/30, dated December 24.

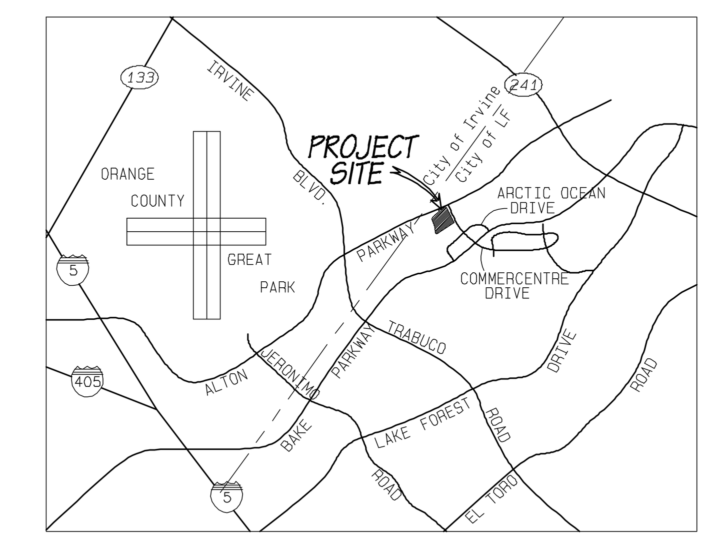
____, 2009, "Review of Site Settlement, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-50, dated November 12.

____, 2014, "Seismic Hazard Study, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-04, dated June 9, 2014.

CONCEPTUAL GRADING PLAN FOR TRACT NO. 17810
ENCANTO

IN THE CITY OF LAKE FOREST, COUNTY OF ORANGE, STATE OF CALIFORNIA





LEGAL DESCRIPTION

LOT 13 OF TRACT NO. 13343 IN THE CITY OF LAKE FOREST, COUNTY OF ORANGE, STATE OF CALIFORNIA AS SHOWN ON A MAP RECORDED IN BOOK 647, PAGES 23 THROUGH 32 INCLUSIVE OF MISCELLANEOUS MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY. ALONG WITH A VACATED PORTION OF ALTON PARKWAY R.O.W. RECORDED ON TRACT 17491.

SITE DATA

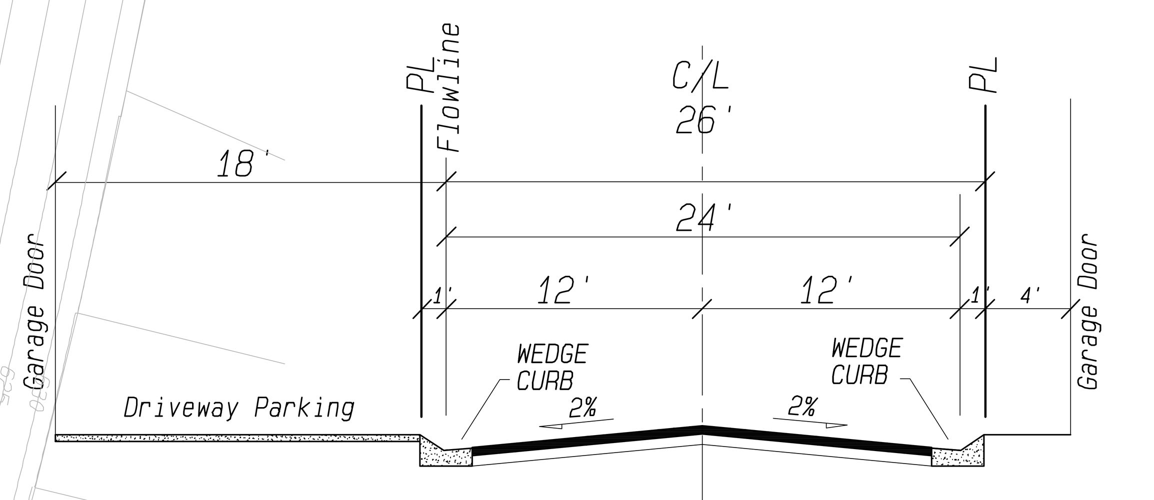
GROSS ACREAGE = 5.75 Ac.
NET ACREAGE - 5.75 Ac.
NUMBER OF UNITS = 52
GROSS DENSITY = 9.0 DU/Ac.

PRIVATE LOTS:	3.15 Ac.
STREETS AND PARKING:	1.42 Ac.
OPEN SPACE LOTS:	1.18 Ac.
	5.75 Ac.

PARKING DATA

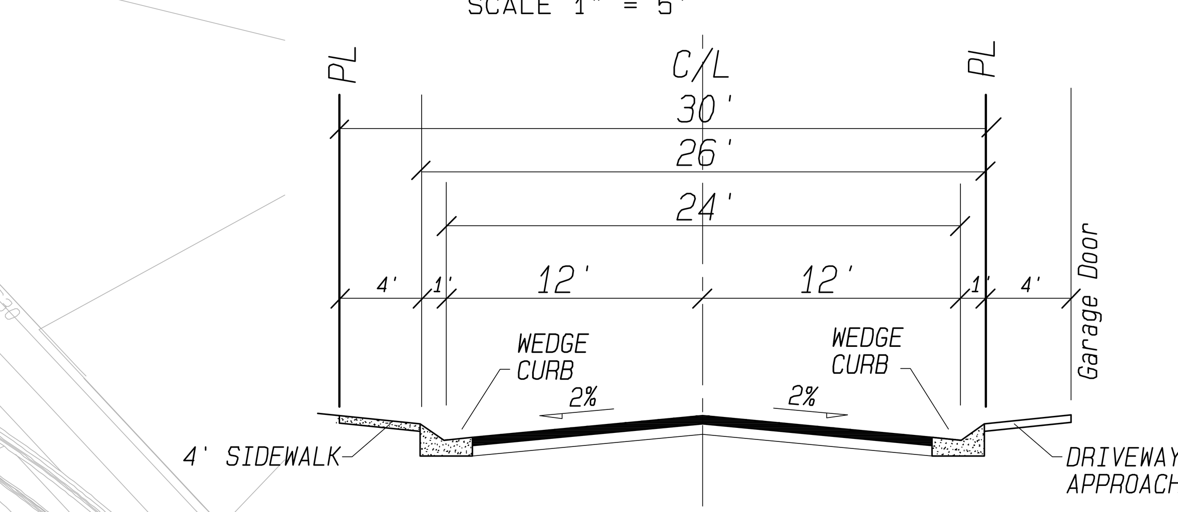
GARAGE	104 Spaces (2 Per DU)
OPEN	63 Spaces (1 Per DU + 0.2 Guest Per DU)
OTHER	6 Spaces in Double Driveways

PARKING SPACES 173 (3.2 SP/UNIT, + 6 Driveway Spaces)



TYPICAL PRIVATE STREET WITH DRIVEWAY PARKING

Minimum width shown per City Ordinance 9.168.040.C.1 and 4.
SCALE 1" = 5'



TYPICAL PRIVATE STREET WITH SIDEWALK

Minimum width shown per City Ordinance 9.168.040.C.1 and 4.
SCALE 1" = 5'

LEGEND

CATCH BASIN AND STORM DRAIN
DAYLIGHT LINE
PROPOSED FINISH CONTOUR
TENTATIVE TRACT BDY. W/ BRGS. & DIMENSIONS
DIRECTION OF GRADED SLOPE
RETAINING WALL
LOT NUMBER — 112
PAD ELEVATION — 793.8
WATER QUALITY FACILITY
FIRE HYDRANT
STREET LIGHT
STORMTECH OR EQUIVALENT WATER QUALITY CHAMBERS — MANHOLE (TYP)
DECORATIVE PAVING TREATMENT

PRELIMINARY EARTHWORK QUANTITIES (RAW)
CUT = 9,000 C.Y.
FILL = 9,000 C.Y.

EARTHWORK QUANTITIES ARE RAW CUT AND FILL, AND DO NOT INCLUDE ADJUSTMENTS FOR SHRINKAGE, BULKING OR OVEREXCAVATION. SOILS REPORT INDICATES MINIMAL SHRINKAGE OR OVEREXCAVATION.

ALL MANUFACTURED SLOPES ON BOTH FILL AND CUT SHALL BE A MAXIMUM OF 2 HORIZONTAL TO 1 VERTICAL.

GRAPHIC SCALE
0' 20' 40' 80' 160'
1 inch = 30 ft.

CONCEPTUAL GRADING PLAN FOR TRACT NO. 17810
ENCANTO

RBF J.N. 142216 JUNE 4, 2015

DIGALERT — DIAL TOLL FREE 1-800-227-2600 — AT LEAST TWO DAYS BEFORE YOU DIG — UNDERGROUND SERVICE ALERT OF SOUTHERN CALIFORNIA

ANY UNDERGROUND UTILITIES OR STRUCTURES SHOWN ON THESE PLANS WERE PLOTTED FROM CITY RECORDS. NO ADDITIONAL UTILITY OR SUBSTRUCTURE RESEARCH HAS BEEN PERFORMED. THE CONTRACTOR SHALL DETERMINE THE EXISTENCE AND THE TRUE HORIZONTAL AND VERTICAL LOCATION AND SIZE OF ANY UNDERGROUND UTILITIES OR STRUCTURES AND SHALL BE RESPONSIBLE FOR PROTECTION, ADJUSTMENT, OR DAMAGE TO ANY PUBLIC OR PRIVATE UTILITIES SHOWN OR NOT SHOWN HEREON. THE CONTRACTOR SHALL ALSO BE RESPONSIBLE FOR NOTIFYING U.S.A./DIG ALERT AT 1-800-227-2600 FOR UNDERGROUND UTILITY MARKING AT LEAST 48 HOURS PRIOR TO COMMENCING ANY CONSTRUCTION. CONSTRUCTION SHALL NOT COMMENCE PRIOR TO SAID UNDERGROUND UTILITY MARKING.

ARCHITECT / PLANNER
WILLIAM HEZMAHALCH ARCHITECTS, INC.
2850 REDHILL AVENUE, SUITE 200
SANTA ANA, CA 92705-5543
PHONE: (949) 250-0607
FAX: (949) 250-1529

SOIL ENGINEER

RBF CONSULTING — PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ■ FAX 949.472.8373 ■ www.RBF.com

REVISIONS			
Nº.	DATE	BY	DESCRIPTION
1			

PLANS PREPARED FOR:
Meritage Homes
1250 CORONA POINTE COURT, SUITE 210
CORONA, CA 92879
PHONE: (951) 547-8339
FAX: (951) 547-8301
ATTN.: KENNETH KIM

PROJECT TITLE

JOB ADDRESS
25192 COMMERCENTRE DRIVE
LAKE FOREST, CALIFORNIA 92610

LEGAL DESCRIPTION
LOT 13 OF TRACT NO. 13343, IN THE CITY OF LAKE FOREST, COUNTY OF ORANGE, STATE OF CALIFORNIA AS SHOWN ON A MAP RECORDED IN BOOK 647, PAGES 23 THROUGH 32 INCLUSIVE OF MISCELLANEOUS MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.

TRACT NO. 17810
DRAWING FILE NO.
SCALE: 1"= 30'
SHEET 3 OF 11

Document B

City of Lake Forest

Geotechnical Report Response Review Checklist

May 26, 2015

Encanto Project

Lot 13, Tract 13343

Lake Forest, Orange County, California

Project No. 1162-CR3

GEOTEK

CITY OF LAKE FOREST
GEOTECHNICAL REPORT RESPONSE REVIEW CHECKLIST

Date Received:	May 19, 2015	Date Completed:	May 26, 2015
Date of Response:	May 15, 2015		
Consultant:	Geo Tek Inc.	Their Job No.:	1162-CR3
Signed By:	Paul Hyum Jin Kim	License No.:	PE 77214 Exp 6/30/15
Signed By:	Edward H. LaMont	License No.:	CEG 1892 Exp 7/31/16
Date of Report:	June 2, 2014	Prior Review:	November 6, 2014
Signed By:	Ronald A. Reed	License No.:	GE 2524 Exp 6/30/15
Signed By:	Edward H. LaMont	License No.:	CEG 1892 Exp 7/31/16
Applicant Name:	Meritage Homes – Encanto	Legend:	N = No
Site Address:	Southwest Corner Commercentre Drive and Alton Parkway Lake Forest, California		Y = Yes NA = Not Applicable
Lot/Tract No.:	Lot 13, Tract 13343		
Proposed Project:	**Tract grading for 52 residential lots, recreation area, private driveway and associated retaining walls up to 7 feet**		

- **Recommended Actions:**

Building and Safety:

____ Report Approved ____ Conditional Approval (See Below) _X_ Additional Input Required

Items requiring response/further evaluation:

1. Please clarify recommendations for site drainage. Refer to Section 1804.3 of the 2013 CBC as warranted.

2. The consultant indicates that retaining walls of up to 7 feet are proposed. Please provide lateral earth pressures for retaining walls due to earthquake motions in accordance with Section 1803.5.12 of the 2013 CBC.

Additional Comments/Conditions of Approval (no response required):

3. Note to City Staff: Staff should confirm that the Consultants (C.E.G. and R.C.E./G.E.) have signed the final dated grading/foundation plans, thereby verifying the plans' geotechnical conformance with the Consultant's original report and associated addenda.

4. Prior to building permits being issued, the consultant shall provide a complete geotechnical foundation plan review report based on engineered plans, when available. The results of additional testing and data from the grading and laboratory testing should be included, along with detailed geologic cross sections depicting proposed structures, precise grading, temporary cuts, typical footings, recommended setbacks, property lines, and adjacent structures.

CITY OF LAKE FOREST
GEOTECHNICAL REPORT RESPONSE REVIEW CHECKLIST

Limitations:

Our review is intended to determine if the submitted report(s) comply with City of Lake Forest Codes and generally accepted geotechnical practices within the local area. The scope of our services for this third party review has been limited to a brief site visit and a review of the above referenced report and associated documents, as supplied by the City of Lake Forest. Re-analysis of reported data and/or calculations and preparation of amended construction or design recommendations are specifically not included within our scope of services. Our review should not be considered as a certification, approval or acceptance of the consultant's work, nor is it meant as an acceptance of liability for final design or construction recommendations made by the geotechnical consultant of record or the project designers or engineers.

BY: _____
James M. Lancaster, Jr., C.E.G. 1927 Expires 6/30/14
KLING CONSULTING GROUP, INC.

BY: _____
Henry F Kling, G.E. 2205, Expires 3/31/14
KLING CONSULTING GROUP, INC.

Document C

GeoTek

Response to City Review Checklist

May 15, 2015

Encanto Project

Lot 13, Tract 13343

Lake Forest, Orange County, California

Project No. 1162-CR3

GEOTEK

May 15, 2015
(Revised June 11, 2015)
Project No. 1162-CR3

Meritage Homes
1250 Corona Pointe Court, Suite 210
Corona, California 92879

Attention: Mr. Kenneth Kim

Subject: Response to City Review Checklist
Southwest Corner Commercecentre Drive and Alton Parkway
Lot 13 of Tract 13343
Lake Forest, Orange County, California

References: *See Appendix A*

Dear Mr. Kim:

GeoTek, Inc. (GeoTek) is presenting this response to Geotechnical Investigation Report Review Checklist, dated November 6, 2014, provided by Kling Consulting Group, Inc. on behalf of the City of Lake Forest, Department of Building and Safety. The responses are provided on an item by item basis as it pertains to the referenced soils reports in Appendix A.

Comment #1

Please provide a copy of the December 24, 2009 and June 2, 2014 Stoney-Miller Reports.

Response

Copies of the reports dated November 12, 2009 regarding site settlement and June 9, 2014 regarding seismic hazards are attached in Appendix B.

Comment #2

The consultant shall indicate they are now the consultant of record for the project and that they have reviewed all referenced documents. They shall provide the project address, applicant's name, and the previous consultants name. The new consultants shall address

previous conclusions and recommendations by others by either concurring or providing modification in writing. The new consultant must state that they are now the geotechnical/geologic consultant of record and accept geotechnical/geologic responsibility for the project. The consultant shall provide a finding that the proposed project is suitable from a geotechnical perspective and will not have an adverse impact on adjacent properties.

Response

GeoTek is declaring to Meritage Homes that herein GeoTek assumes responsibility as geotechnical consultant of record henceforth for the subject site located at the southwest corner of Commercentre Drive and Alton Parkway (Lot 13 of Tract 13343) located in the City of Lake Forest, Orange County, California from Stone-Miller Consultants, Bagahi Engineering and Pacific Soils.

Previous Work

The site was initially mass graded from June 1990 to July 1991, as reported by Stoney-Miller (1991). The building pad area of Lot 13 was initially graded to an elevation of roughly 590 to 592 feet mean sea level (msl), with a detention basin located toward the center of the pad, at an approximate elevation of about 580 msl. Prior to engineered fill placement, previously existing undocumented fill soil, unsaturated alluvium and otherwise loose and unconsolidated soils were removed to expose saturated alluvium or bedrock.

Subsequent to completing the removals described above, engineered fill was placed at a minimum 90 percent relative compaction up to design grades. Depths of fill across the graded pad ranged from roughly two (2) feet near the south corner of the pad to roughly 60 feet toward the north and west sides of the pad. Saturated alluvial thickness below the pad area was estimated to range from 25 to 35 feet (Stoney-Miller, 2006).

As a result of leaving saturated alluvial soils beneath the subject pad, several episodes of surcharging the site with fill soils were recommended and subsequently performed under the geotechnical observation and monitoring of both Stoney-Miller and Bagahi (see referenced reports). As part of placing the surcharge fill, the retention basin along the central portion of the property was removed and replaced with certified engineered fill to a maximum elevation of approximately 595 msl. Upon review of the most recent referenced report detailing these surcharging and settlement monitoring episodes (Stoney-Miller, 2009), the surcharging efforts for the subject site has been deemed sufficient to mitigate future ground settlements to less than 1-inch for a time period (design life) of 50 years.

The site is not located within a currently designated Earthquake Fault Zone per the Alquist-Priolo Special Studies Zone Map produced by the California Geological Survey (CGS). The nearest zoned fault is the Newport-Inglewood fault zone located approximately 11.6 miles to the west (Stoney-Miller, 2014). However, the subject property is located northeast of the San Joaquin Hills. The uplift and folding of the San Joaquin Hills is attributed to the tectonic activity of the San Joaquin Hills blind thrust fault which is considered a potentially active fault. Based on information obtained from the USGS, the subject property is located approximately 3 miles northeast of the projected surficial trace of the San Joaquin Hills blind thrust fault. It is incorporated as a seismic source in the 2008 National Seismic Hazard Maps, and the potential ground shaking effect from the San Joaquin blind thrust fault is reflected in the seismic design parameters.

Research of the California Geological Survey (CGS) seismic hazard zones map (El Toro Quadrangle) shows that the subject site is not located within an area designed as being susceptible to liquefaction but is located within a zone of potential seismically-induced slope instability. However, due to the presence of alluvial settlements coupled with groundwater encountered within 50 feet below the ground surface elevation, there is a potential for liquefaction.

The results of the liquefaction potential evaluation indicates that liquefiable layers are present, but tend to be relatively thin and discontinuous. Seismic settlements were estimated to be negligible below the building pad and a tolerable 1 to 2 inches beneath the toe of the slope along the northwest portion of the subject property (Stoney-Miller, 2014).

The fill slope along the northwest portion of the subject property was analyzed for potential seismic-induced slope instability. The results of the analysis indicates a pseudostatic factor of safety in excess of 1.1 and slope deformations of 0.4 to 1.4 inches due to a probable seismic event and 2 to 6 inches due to a maximum considered seismic event, which is acceptable per the California SP117 guidelines (Stoney-Miller, 2014).

Upon review of the referenced reports by Bagahi Engineering (1997 and 1998), Pacific Soils Engineering, Inc. (1989) and Stoney-Miller Consultants, Inc. (1990a, 1990b, 1991, 1993, 1997, 2005, 2006, 2007, 2008, 2009 and 2014), GeoTek generally concurs with the findings, conclusions and recommendation provided therein. The recommendations provided by others remain applicable unless otherwise superseded herein or in future reports by this firm.

Comment #3

The consultant shall provide a complete geotechnical grading plan review report based on an engineered grading plan, when available. The results of the subsurface investigations by others, additional laboratory testing for expansive soils and sulfate contact should be included, along with detailed geologic map and cross sections depicting proposed grading, temporary cuts, recommended setbacks, property lines, adjacent structures, settlement, foundation recommendations, retaining wall recommendations, updated seismic parameters, etc. Anticipated shoring, slot cutting, restricted use areas, geogrid, etc. should be depicted if proposed.

Response

The proposed project consists of the construction of 52 residential units with approximate pad elevations ranging from 590 msl to 597 msl. Additional improvements include a recreation area, a private driveway and retaining walls up to a maximum 6 feet high. Stormwater retention devices are also planned. A site plan, depicting the proposed structures is included hereafter in Appendix C. It should be noted, however, that the proposed site plan has been overlayed on an outdated base topo map, as the existing surcharge fills are currently being removed.

Infiltration Testing

As requested, GeoTek performed a double-ring infiltrometer test within an excavation approximately six (6) feet deep along the north portion of the property. The excavation was dug by the client in the vicinity of one of the proposed stormwater retention devices. The test was performed by a representative from our firm on May 6, 2015 in general conformance with ASTM D 3385.

The double-ring infiltrometer test resulted in an infiltration rate of 1/8 inch per hour after the infiltration rate had generally stabilized. The soil materials at the bottom of the excavation where the infiltration test was performed generally consisted of a brown silty sand. The attached Site Plan (Appendix C) shows the approximate location of the infiltration test. A copy of the double-ring infiltrometer test field data is included in Appendix D. Over the lifetime of the stormwater disposal areas, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. We recommend that an appropriate factor of safety be applied to account for soil variability and silting of the basin bottom over time.

Seismic Design Parameters

The site is located at approximately Latitude: 33.6686°N and Longitude: -117.6902°W. Site spectral accelerations (Ss and S_1), for 0.2 and 1.0 second periods for a Class "D" site, were determined from the USGS Website, Earthquake Hazards Program, U.S. Seismic Design Maps for Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for the Conterminous 48 States by Latitude/Longitude. The results are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.466g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.544g
Site Coefficient for Site Class "D", F_a	1.0
Site Coefficient for Site Class "D", F_v	1.5
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.466g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.816g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.978g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.544g
Peak Ground Acceleration Adjusted for Site Class Effects, PGA_M	0.538g

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

CONCLUSIONS AND RECOMMENDATIONS

General

The anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations provided are properly incorporated into the design and construction phases of development.

At the time of this letter, the existing surcharge fills were in the process of being removed. The last approved grade on the site is approximately at elevation 590 to 595 feet above mean sea leve (msl) as observed and approved by Stoney-Miller in 1991 and 2008. We recommend that the upper 12 inches of the last approved grade be removed and recompacted, due to

possible disturbance during surcharge fill removal and the time elapsed since the last approved grade. Site grades may then be raised with engineered fill to achieve the design elevations at the site.

Some retaining walls (maximum 6 feet high) are planned along the exterior of the subject property to provide level building pad areas. The retaining walls may be supported with conventional foundations founded in engineered fill with the appropriate setbacks.

EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Lake Forest, the 2013 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix F outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix F.

Preparation of Areas to Receive Engineered Fill

A representative of this firm should observe the bottom of all excavations. Upon approval, the exposed soils and all soils in areas to receive engineered fill should be scarified to a depth of approximately eight (8) inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557). A representative of this firm should also confirm that a minimum of 5 feet of engineered fill is present toward the southwest corner of the subject pad, where sedimentary bedrock materials have previously been identified.

Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Rocks exceeding 3 inches in greatest dimension should not be incorporated into the fill at all levels. Engineered fill should be placed in 8-inch or less lifts, moisture conditioned to at least the optimum moisture content, and compacted to a minimum relative compaction of 90% as determined by ASTM Test Method D 1557.

Excavation Characteristics

Excavation in the on-site soils is expected to be feasible utilizing heavy-duty grading equipment in good operating condition.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the on-site materials should be stable at 1:1 (horizontal:vertical) inclinations for cuts less than five (5) feet in height.

DESIGN RECOMMENDATIONS

Shallow Foundation Design Criteria

Foundation design criteria for a conventional foundation and post-tension foundation systems, are presented in this report. Based on the results of our laboratory testing, it is anticipated that the soils near subgrade will classify as having a "very low" expansion potential ($0 \leq EI \leq 20$) in accordance with ASTM D 4829. Typical design criteria for the site based upon a "very low" expansion potential are tabulated below. These are minimal recommendations and are not intended to supersede the design by the project structural engineer.

The foundation elements for the proposed structures and other improvements should be founded entirely in engineered fill soils. Foundations should be designed in accordance with the *2013 California Building Code* (CBC).

Additional expansion index and soluble sulfate testing of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions.

A summary of our conventional foundation design recommendation is presented in the following table:

GEOTECHNICAL RECOMMENDATIONS FOR SHALLOW FOUNDATION DESIGN

Design Parameter	"Very Low" Expansion Potential 0≤EI≤20
Foundation Depth or Minimum Perimeter Beam Depth (inches below the lowest adjacent grade)	Two-story – 18
Minimum Foundation Width (inches)*	Two-story – 12
Minimum Slab Thickness (inches)	4 - Actual
Sand Blanket and Moisture Retardant membrane below On-Grade Building Slabs	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**
Minimum Slab Reinforcing	6" x 6" - W1.4/W1.4 welded wire fabric placed in the middle of slab
Minimum Footing Reinforcement for Continuous Footings, Grade Beams and Retaining Wall Footings	Two No. 4 reinforcing bars, one placed near the top and one near the bottom
Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)	Minimum of 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

* Code minimums per Table 1809.7 of the 2013 CBC
** Sand should have a sand equivalent of at least 30

Post-tensioned slabs should be designed in accordance with the 2013 California Building Code (CBC) and PTI design methodology. It should be noted that expansion test results are not a controlling factor in PTI design methodology. The grain size distribution of the soils and the Atterberg Limits are the dominant soil properties considered in the PTI design methodology. Using grain size distribution, Atterberg Limits and swell test results from samples recently obtained from the near surface on-site soils, the design parameters presented in the table below were determined. The foundation design parameters presented below are derived using the computer program VOLFLO (v. 1.5) in general conformance with Design of Post-Tensioned Slabs-on-Ground, 3rd Edition (PTI, 2008) and Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI, 2012).

POST-TENSIONED SLAB DESIGN RECOMMENDATIONS

Foundation Design Parameter	Design Value
	Soil Characteristics: LL=30; PI=7; 38% passing #200 sieve, 13% passing 2μm; clay fines=34%
Edge Moisture Variation Distance, e_m - Edge Lift (swelling) - Center Lift (shrinkage)	 5.2 ft 9.0 ft
Soil Differential Movement, y_m - Edge Lift (swelling) - Center Lift (shrinkage)	 ≈0.01 in ≈-0.01 in
Ext. Perimeter Beam Embedment	One- or Two-Story – 12 inches*
Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)	Minimum 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

* Required depth of perimeter beam/stiffening rib per structural calculations may govern.
The following assumptions were used to generate e_m and y_m values: soil fabric factor = 1.0; K_0 = 0.33 (drying), 0.67 (wetting); Thornthwaite Moisture Index = -20; constant suction value = 3.8pF; post-equilibrium case assumed with wet (swelling) cycle going from 3.8 pF to 3.0 pF and drying (shrinking) cycle going from 3.8pF to 4.5pF.

The bottom of the perimeter edge beam/deepened footing should be designed to resist tension forces using either cable or conventional reinforcement, per the structural engineer.

An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of building and retaining wall footings. This value may be increased by 300 psf for each additional 12 inches of embedment depth and by 200 psf for each additional 12 inches in width to a maximum of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering short-term wind and seismic loads.

For footings designed in accordance with the recommendations presented in this report, we would anticipate a maximum settlement of less than one (1) inch and a maximum differential settlement of less than one (1) inch in a 40-foot span.

The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 2,000 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.30 may be used with dead load forces. The upper one foot of soil below the adjacent grade should not be used in calculating passive pressure. When combining passive and frictional resistance, the passive pressure component should be reduced by one-third.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2013 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2013 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a six (6) mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeability) to achieve the desired performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek recommends that a qualified person, such as the flooring contractor, structural engineer, architect and/or other experts specializing in moisture control within the building be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction.

In addition, the recommendations in this report and our services in general are not intended to address mold prevention, since we along with geotechnical consultants in general, do not practice in areas of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

Miscellaneous Foundation Recommendations

- To reduce moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.
- Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.
- Under-slab utility trenches should be compacted to at least 90 percent (ASTM D1557). Compaction should be achieved with a mechanical compaction device. If soils to be used as trench backfill have dried out, they should be thoroughly moisture conditioned prior to placement.

Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2013 CBC or City of Lake Forest requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The following recommendations are presented:

- The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least seven (7) feet and need not exceed 40 feet.
- The outside bottom edge of all footings should be set back a minimum of H/2 (where H is the slope height) from the face of any ascending slope. The setback should be at least seven (7) feet and need not exceed 15 feet.
- The bottom of all footings for new structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall footing.

RETAINING AND GARDEN WALL DESIGN AND CONSTRUCTION

General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete retaining walls to a maximum height of up to 10 feet. Additional review and recommendations should be requested for higher walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations should be embedded a minimum of 18 inches into engineered fill and should be designed in accordance with the Foundation Design section of this letter. Structural needs may govern and should be evaluated by the project structural engineer.

All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization. The seismic design parameters as discussed in this report remain applicable to all proposed earth retention structures at this site, and should be properly incorporated into the design and construction of the structures.

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations made by the designer. The backfill material placement for all earth retention structures should meet the requirement of the Engineered Fills section of this letter.

In general, cantilever earth retention structures, which are designed to yield at least 0.001H, where H is equal to the height of the earth retention structure to the base of its footing, may be designed using the active condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at top, such as typical basement walls) should be designed using the at-rest condition.

In addition to the design lateral forces due to retained earth, surcharges due to improvements, such as an adjacent building or traffic loading, should be considered in the design of the earth retention structures. Loads applied within a 1:1 (h:v) projection from the surcharge on the stem and footing of the earth retention structure should be considered in the design.

Final selection of the appropriate design parameters should be made by the designer of the earth retention structures.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf)
Level	35
2:1	60

* The design pressures assume the backfill material has an expansion index less than or equal to 20. Backfill zone includes area between the back of the wall to a plane (1:1 h:v) up from the bottom of the back of the footing to the ground surface.

Retaining Wall Backfill and Drainage

Retaining wall backfill should consist of granular, non-expansive soil with an expansion index less than or equal to 20. The wall backfill should also include a minimum one (1) foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 12 inches of the finish grade. The gravel should be separated from the earth with filter fabric (Mirafi 140N or an approved equivalent). Alternatively, a drainage composite such as Miradrain® may be used in lieu of the gravel column. The upper 12 inches should consist of compacted on-site soil. The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. The backfill materials should be placed in lifts no greater than eight (8) inches in thickness and compacted to a minimum of 90% relative compaction in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained.

Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four (4)-inch diameter

perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one (1) cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

SOIL CORROSIVITY

The soil resistivity at this site was tested in the laboratory on one sample collected during the field investigation. The results of the testing indicate that the on-site soils are considered "corrosive" to buried ferrous metal in accordance with current standards used by corrosion engineers. We recommend that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site.

SOIL SULFATE CONTENT

The sulfate content was determined in the laboratory for one soil sample. The results indicate that the water soluble sulfate result is less than 0.1 percent by weight, which is considered "not applicable" (negligible) as per Table 4.2.1 of ACI 318.

IMPORT SOILS

Import soils should have expansion characteristics similar to the on-site soils. GeoTek also recommends that the proposed import soils be tested for expansion and corrosion potential. GeoTek should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

CONCRETE FLATWORK

Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four (4) inch minimum thickness. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in industrial construction.

Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. at the site should be pre-saturated to a minimum of 100% of optimum moisture content to a depth of at least 12 inches.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Lake Forest specifications, and under the observation and testing of GeoTek and a City inspector, if necessary.

Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 0.125-inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent upon a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is also subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek suggests that control joints be placed in two orthogonal directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered "non-structural" components. We suggest that the same standards of care be applied to these features as to the structure itself.

PAVEMENT DESIGN

The recommended thicknesses presented below are considered typical and minimum for the assumed parameters. We understand that budgetary considerations sometimes warrant thinner pavement sections than those presented. However, the client, the owner, and the project principals should be aware that thinner pavement sections might result in increased maintenance costs and lower than anticipated pavement life.

In designing the proposed paved areas, the existing subgrade conditions must be considered together with the expected traffic use and loading conditions.

The conditions that will influence the pavement design can be summarized as follows:

1) Subgrade support characteristics of the subgrade. This is typically represented by a R-Value for the design of flexible pavements in this region.

2) Vehicular traffic, in terms of the number and frequency of vehicles and their range of axle loads.

3) Probable increase in vehicular use over the life of the pavement.

We recommend that the exposed subgrade be prepared in accordance with the site preparation requirements specified previously in this letter. The upper one foot of pavement subgrade should be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor (ASTM D 1557). The fill moisture content at the time of compaction should be within 1 to 3 percent above the optimum moisture content value. Undercut soil should be replaced by engineered fill.

The appropriate pavement section depends primarily upon the type of subgrade soil, shear strength, traffic load, and planned pavement life. For preliminary purposes, we have utilized traffic indices of TI=4.5 and TI=7.0. These assumed traffic indices should be verified by the project civil engineer prior to construction. Based on the results of laboratory testing, we have assumed an R-value of 30 for the near-surface soils within pavement areas. Since an evaluation of the characteristics of the actual soils at pavement subgrade can only be provided at the completion of grading, the following pavement sections should be used for planning purposes only. Final pavement designs should be evaluated after R-value tests have been performed on the actual subgrade material.

It should be noted that additional earthwork and/or ground improvement efforts may be required during grading on the actual subgrade material, in order to achieve the aforementioned design parameters and assumptions. These design thicknesses assume that a properly prepared subgrade has been achieved.

Flexible Pavement Recommendations

Assumed Traffic Index	Asphalt Concrete (AC) Thickness (in.)	Class II Aggregate Base (AB) Thickness (in.)
4.5	3	5
7.0	4	10

Asphalt Concrete (AC), Portland Cement Concrete, and Class II aggregate base should conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications and American Concrete Institute (ACI) codes. Aggregate base should be compacted to a minimum of 95 percent of the maximum dry density as determined by the modified Proctor (ASTM D 1557) prior to placement of AC. Subgrade preparation for pavement areas is included in the earthwork considerations section of this letter.

POST CONSTRUCTION CONSIDERATIONS

Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to the structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided.

Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times and be designed by the project civil engineer. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings. Soil areas within 10 feet of the proposed structure should slope at a minimum of 5-percent away from the building, if possible unless the area is paved. Paved areas are to be sloped at 2-percent away from the structure. Roof gutters and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter pan or directly to the storm drain system. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading, specifications and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to observe and document proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek representatives perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trench backfill. Also, perform field density testing of the fill materials.

- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek which can comply with the requirements of the governmental agencies having jurisdiction over the project. We recommend that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

LIMITATIONS

Our findings are based on site conditions observed and the stated sources. Thus, our comments are professional opinions that are limited to the extent of the available data. These opinions have been derived in accordance with current City/County standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

Edward H. LaMont
CEG 1892, Exp. 7/31/12
Principal Geologist

Paul Hyun Jin Kim
PE 77214, Exp. 6/30/13
Project Engineer

Distribution: (1) Addressee via email

Enclosures:

	Appendix A	References
	Appendix B	Stoney-Miller report dated November 12, 2009 and June 9, 2014
	Appendix C	Site Plan
	Appendix D	Double-ring Infiltrometer Test Results
	Appendix E	Recent Laboratory Test Data
	Appendix F	Grading Guidelines

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Bagahi Engineering, 1997, "Geotechnical Investigation of Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, CA," Job No. 82a-200-00, dated June 16.

____, 1998, "Progress Report on Monitoring of Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, California," Job No. 82a-200-00, dated October 9.

Stoney-Miller Consultants, Inc., 1993, "Settlement Monitoring, Canyon 2, Lots 12-20, Tract 13343, 20858 Bake Parkway, El Toro, California," Project No: 10425-06, dated December 20.

____, 1997, "Settlement Monitoring, Canyon 2, Lots 12 through 20, Tract 13343, 20858 Bake Parkway, Pacific Commercentre, Lake Forest, California," Project No: 10425-07, dated March 7.

____, 2005, "Results of Preliminary Geotechnical Investigation, 25192 Commercentre Drive, Lake Forest, California," Project No: 12576-00, dated May 3.

____, 2006, "Site History & Future Geotechnical Work, Lot 13 of Tract 13343, 25192 Commercentre Drive, Lake Forest, California," Project No: 12655-02, dated January 24.

____, 2007, "Supplemental Geotechnical Investigation, Lot 13 of Tract 13343, 25191 Commercentre Drive, Lake Forest, California," Project No: 12655-03, dated November 12.

____, 2008, "Grading Earthwork Observation/Testing Report, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-10/30, dated December 24.

____, 2009, "Review of Site Settlement, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-50, dated November 12.

____, 2014, "Seismic Hazard Study, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-04, dated June 9, 2014.

APPENDIX B

Stoney-Miller report dated November 12, 2009 and June 9, 2014

Response to City Review Checklist
Lot 13, Tract 13343
Lake Forest, Orange County, California
Project No. 1162-CR3

STONEY-MILLER CONSULTANTS, INC
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

November 12, 2009

BDJ Lake Forest Associates LLP
1420 Bristol Street North, Suite 100
Newport Beach, California 92660

Project No: 12655-50
Report No: 09-11123

Attention: Mr. Daniel Karcher & Mr. Larry Nelson

Subject: **Review of Site Settlement**
Lot 13 of Tract 13343
25192 Commerce Drive
Lake Forest, California

References: See Appendix A

Gentlemen:

In accordance with your request, we have reviewed the results of settlement monitoring performed at the subject site by DMC Engineering. This monitoring has focused on monitoring the settlement behavior of the subject site under the influence of an approximate 15-foot high fill surcharge. The fill surcharge was placed both to accelerate the occurrence of primary settlements and mitigate the effects of long-term secondary settlements on future site improvements. Based on the results of the monitoring and our review and evaluation of the monitoring data, it is our opinion that the fill surcharge has caused sufficient settlement at the site to mitigate future effects of alluvial settlements on commercial building structures of at least four-stories high and other typical improvements associated with a commercial office building.

PROJECT BACKGROUND

Previous Grading

Previous grading on or adjacent to the site consisted of cuts and fills, associated with mass grading, to construct a relatively-flat commercial-sized lot. This grading (see Reference 1) was performed from June 1990 to July 1991, and was performed under the observation and testing of Stoney-Miller Consultants, Inc. Grading resulted in the placement of engineered fill across the subject site to depths ranging from 2 feet within the south corner of the pad to an estimated maximum depth of about 60 feet on the north and west side of the pad. Alluvial thickness below the pad area is estimated to range from 0 to 35 feet. The pad was graded to an elevation of 590

to 592 feet above mean sea level (amsl), excepting the detention basin founded at an elevation of about 582 feet amsl (see Figure B-14).

Four settlement monuments (SM2-1, SM2-2, SM2-3 and SM2-11) were constructed onsite to monitor ground settlements after grading was complete. The monuments were constructed in the approximate locations shown on Figure B-14. Monitoring of each monument was conducted over various time intervals starting in June 1990 and ending as late as December 1996. A maximum alluvial settlement of about 12-inches was recorded at Monument SM2-3 over a time period of approximately 1,956 days (5.3 years). Based on this monitoring, it is estimated that 85 to 90 percent of the total settlements were complete by the end of 1996. The settlement survey readings are summarized in Table B-10, and plotted on Figures B-11 and B-12.

Subsequent to the completion of grading and monitoring, a fill embankment was created along the north and west periphery of the pad as a future source of fill material to infill the detention basin.

Current Grading

The more recent grading within Lot 13 of Tract 13343 consisted of bringing the subject site to proposed grades near elevation 590 to 595 feet amsl and placing an approximate 15-foot high surcharge fill at the location of the old detention basin area. The purpose of the surcharge was to mitigate the impact of alluvial settlements on improvements at the site by accelerating the occurrence of primary settlements, and the majority of the long-term secondary settlements anticipated under the fill and future footing loads. This additional grading was done from March 6, 2006 through May 30, 2008, and was performed under the observation and testing services of Stoney-Miller Consultants, Inc., and is reported in Reference 8. Fill soils were placed to an approximate elevation of 595 feet amsl on the eastern half of the basin, and 593 feet amsl on the western half. The temporary surcharge fill was then placed to an elevation of about 608 feet amsl between March 13, 2008 to April 2, 2008.

Eight settlement monuments were placed to monitor the settlement at the site. These monuments were constructed and then surveyed for the first time on March 21, 2008 or approximately 19 days after the surcharge fill was completed. Five monuments were placed within the fill surcharge area; two monuments within a cut area to the northwest and northeast of the basin; and one monument within the east parking. The location of the monuments placed within Lot 13 are shown on Figure B-13 and summarized in Table A as follows.

TABLE A, Settlement Monument Location

Settlement Monument #	Northing	Easting
SM101-TC	10553.198	9581.312
SM102-TC	10376.979	9304.094
SM103-TC	10373.757	9554.165
SM104-TC	10308.912	9611.231
SM105-TC	10278.759	9503.349
SM106-TC	10315.126	9450.201
SM107-TC	10329.407	9532.559
SM108-TC	10393.029	9717.849

Monitoring of each monument was conducted for 171 days starting March 21, 2008 and ending October 9, 2008. A maximum total settlement of about 1.4-inch was recorded at Monument SM-107-TC. A settlement monument summary is given in Figure B-1, and the survey data is plotted verses time on Figures B-2 through B-9. As of the date of this report, 570 days have elapsed since the start of settlement monitoring

CONCLUSIONS

We have reviewed the settlement monitoring data (see Appendix B) for Lot 13 of Tract 13343 to evaluate the suitability of the lot for precise grading and construction release. Based on our analyses and review of the settlement monitoring data, it is our opinion that the surcharge fill has achieved the desired settlement mitigation for Lot 13 of Tract 13343. In our opinion, the fill surcharging effects at the subject site has been sufficient to mitigate future ground settlements to less than 1-inch for a time period (design life) of 50 years. In our opinion, the fill surcharge is no longer needed. Based on the results of the monitoring and our review and evaluation of the monitoring data, it is our opinion that the fill surcharge has caused sufficient settlement at the site to mitigate future effects of alluvial settlements on commercial building structures of at least four-stories high and other typical improvements associated with a commercial office building.

Precise grading, improvement, and foundation plans should be reviewed by the Geotechnical Consultant prior to grading and construction. All future grading to prepare the site area should be performed under the observation and testing of the Geotechnical Consultant.

LIMITATIONS

Our work was performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report.

The opportunity to be of service is appreciated. If you have any questions, please call.

Respectfully submitted,

STONEY-MILLER CONSULTANTS, INC.

Russell C. Lamb, G.E. 2207
Associate Geotechnical Engineer
Registration Expires 3-31-11

MGV/RCL/rm

Attachments: Appendix A, References
Appendix B, Settlement Monitoring Data

Distribution: Addressee (4)
DMc Engineering (1)
Attention: Mr. Derek McGregor

APPENDIX A

LIST OF REFERENCES

APPENDIX A

LIST OF REFERENCES

1. Stoney-Miller Consultants, Inc., 1991, "Rough Grade Geotechnical Report, Tract 13343, a Portion of Offsite Tentative Tract 13336, a Portion of Pacific Commerce Center Project, 29858-B Bake Parkway, El Toro, California," Volumes I and II, dated October 31, 1991, Project No: 10425-13, Report No: 1-2086.

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3. Stoney-Miller Consultants, Inc., 1997, "Settlement Monitoring, Canyon 2, Lots 12 through 20, Tract 13343, 20858 Bake Parkway, El Toro, California," dated March 7, 1997, Project No: 10425-07, Report No: 7-5338.

4. Bagahi Engineering, 1997, "Geotechnical Investigation of Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive, and Alton Parkway, Lake Forest, California," dated June 16, 1997, Job No: 82a-200-00.

5. Bagahi Engineering, 1998, "Progress Report on Monitoring Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, California," dated October 9, 1998, Job No: 82a-200-00.

6. Stoney-Miller Consultants, Inc., 2005, Results of Preliminary Geotechnical Investigation, 25192 Commercentre Drive, Lake Forest, California," dated May 3, 2005, Project No: 12576-00, Report No: 05-9852.

7. Stoney-Miller Consultants, Inc., 2007, "Supplemental Geotechnical Investigation, Lot 13 of Tract 13343, 25192 Commercentre Drive, Lake Forest, California," dated December 6, 2007, Project No: 12655-03, Report No: 07-10572.

8. Stoney-Miller Consultants, Inc., 2007, "Grading Earthwork Observation/Testing Report, Lot 13 of Tract 13343, 25192 Commercentre Drive, Lake Forest, California," dated December 24, 2008, Project No: 12655-10-30, Report No: 08-10711.

PLANS

A. DMc Engineering, 2008, "Temporary Surcharge Drainage Plan for Lot 13, Tract 13343, 25192 Commercentre Drive, Lake Forest, California," Sheets 1 through 8 of 8, dated April 14, 2008, Project No: 1004.

APPENDIX B

SETTLEMENT MONITORING DATA

APPENDIX B (Report 09-11123)

SETTLEMENT MONITORING DATA

SETTLEMENT MONUMENT SUMMARY

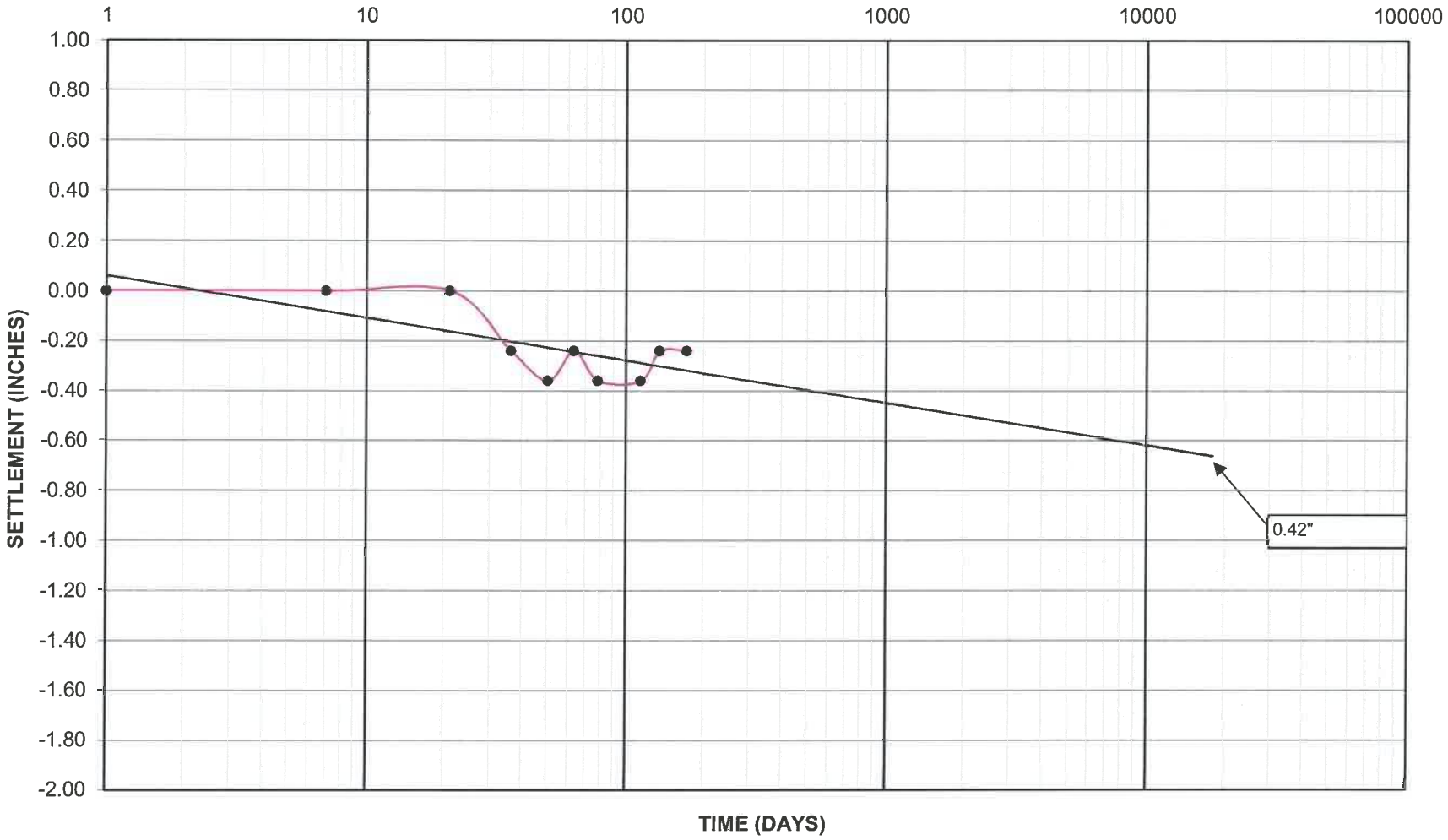
LOT 13, TRACT 13343

MONUMENT	MONITORING BEGINS	LAST READING	ELAPSED DAYS	SETTLEMENT MEASURED (INCH)	FUTURE SETTLEMENT PROJECTED TO 50 YEARS (INCH) DAY 570 TO 18250 (With Surcharge Left In-Place)	LOCATION IN SURCHARGE AREA	COMMENTS
101	4/21/2008	10/9/2008	171	0.24	0.3	OUTSIDE	AS ANTICIPATED; IN CUT AREA
102	4/21/2008	10/9/2008	171	0.36	0.3	OUTSIDE	AS ANTICIPATED; IN CUT AREA
103	4/21/2008	9/3/2008	135	0.96	1.6	**NEC**	MONUMENT DESTROYED PER 10/9/08 CIRCUIT
104	4/21/2008	10/9/2008	171	1.20	1.3	**SEC**	
105	4/21/2008	10/9/2008	171	1.08	1.3	**SWC**	
106	4/21/2008	10/9/2008	171	0.96	1.3	**NWC**	
107	4/21/2008	10/9/2008	171	1.44	1.6	**CENTER**	
108	4/21/2008	10/9/2008	171	0.36	0.3	OUTSIDE	AS ANTICIPATED; IN AREA OF NO CUT OR FILL

12655-50

FIGURE B-1

SETTLEMENT MONUMENT SM101-TC
Lot 13, Tract 13343

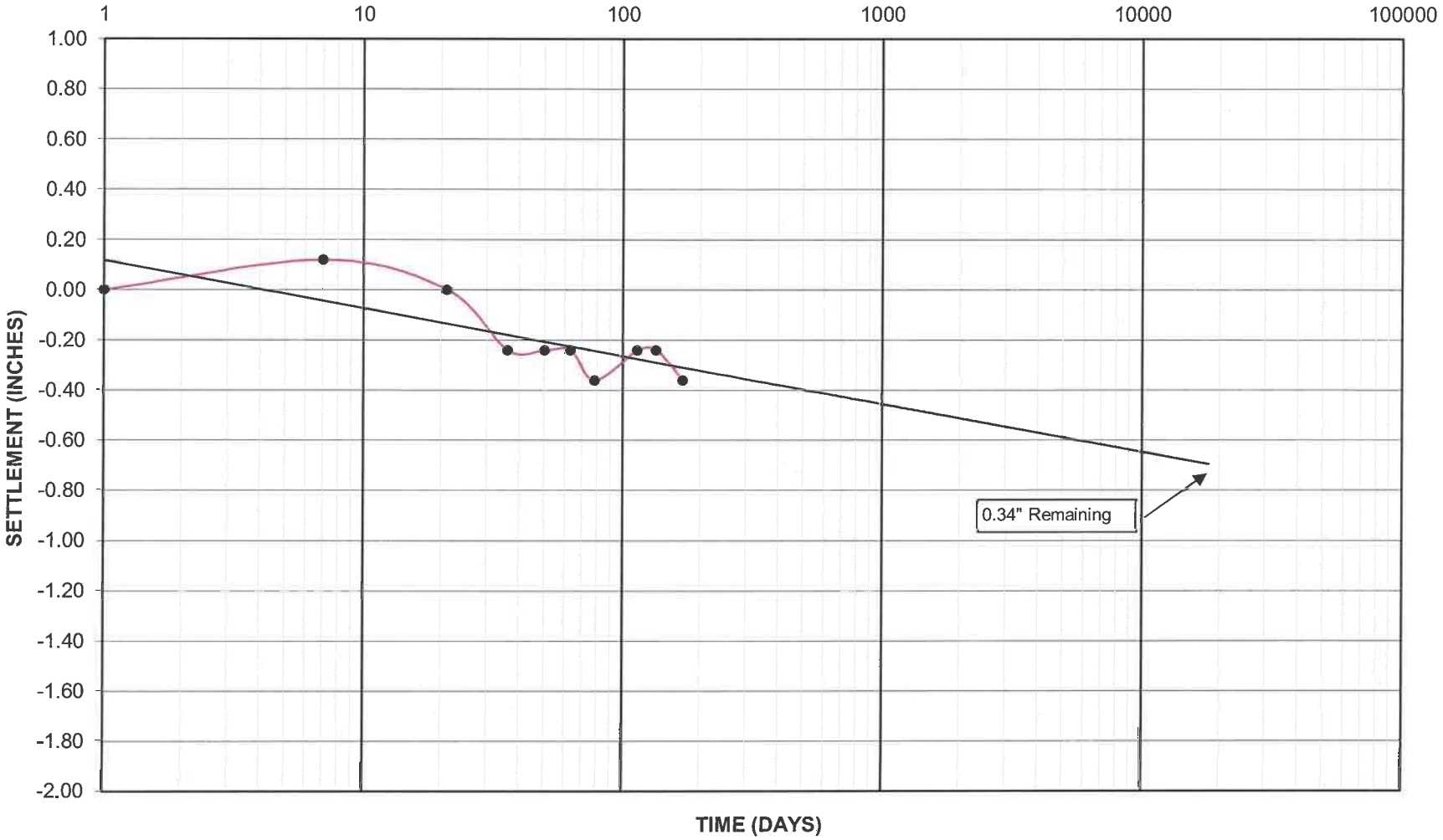


12655-50

FIGURE B-2

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM101-TC-log (2)

SETTLEMENT MONUMENT SM102-TC
Lot 13, Tract 13343



12655-50

FIGURE B-3

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM102-TC-log (2)

SETTLEMENT MONUMENT SM103-TC
Lot 13, Tract 13343

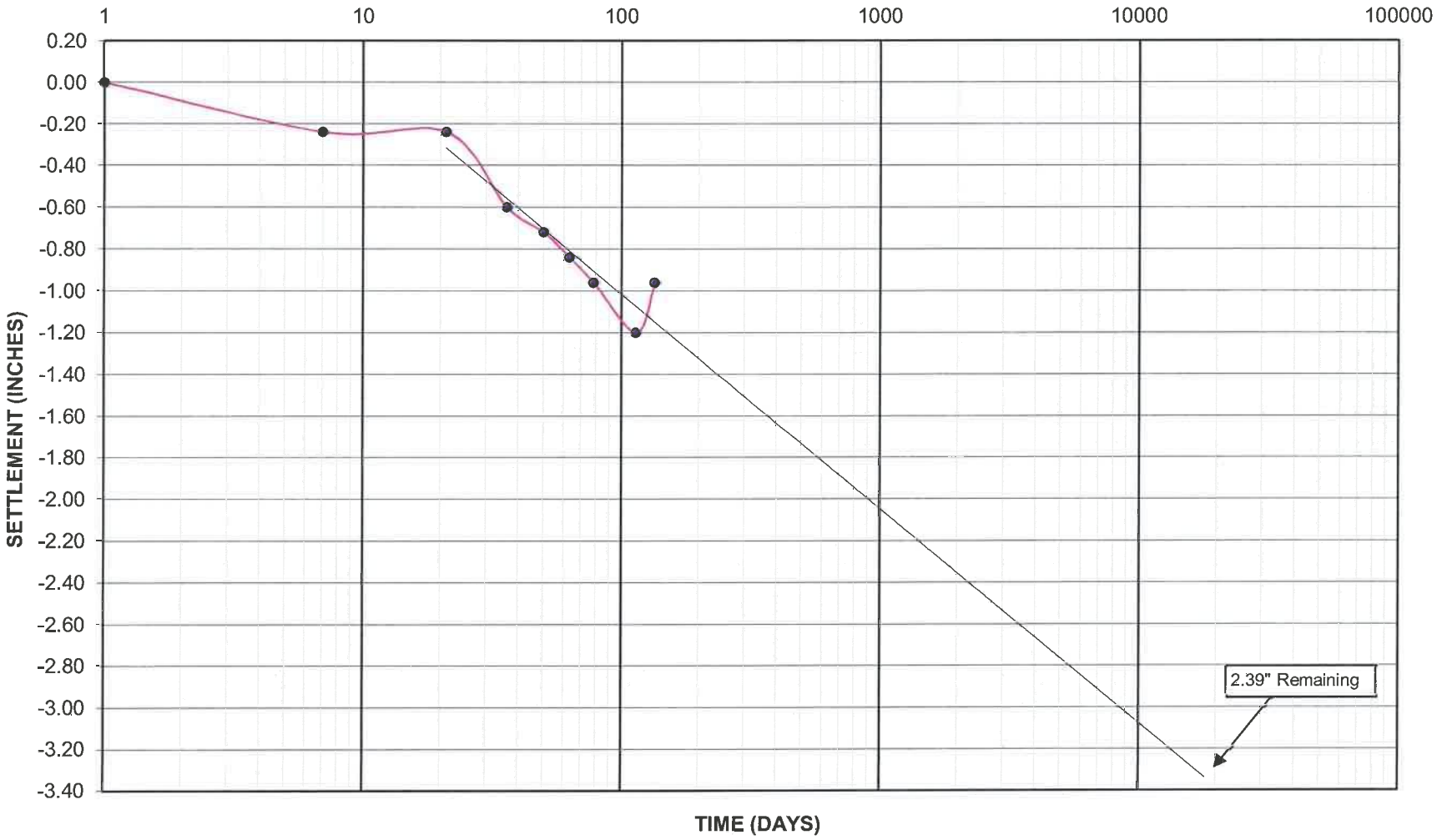
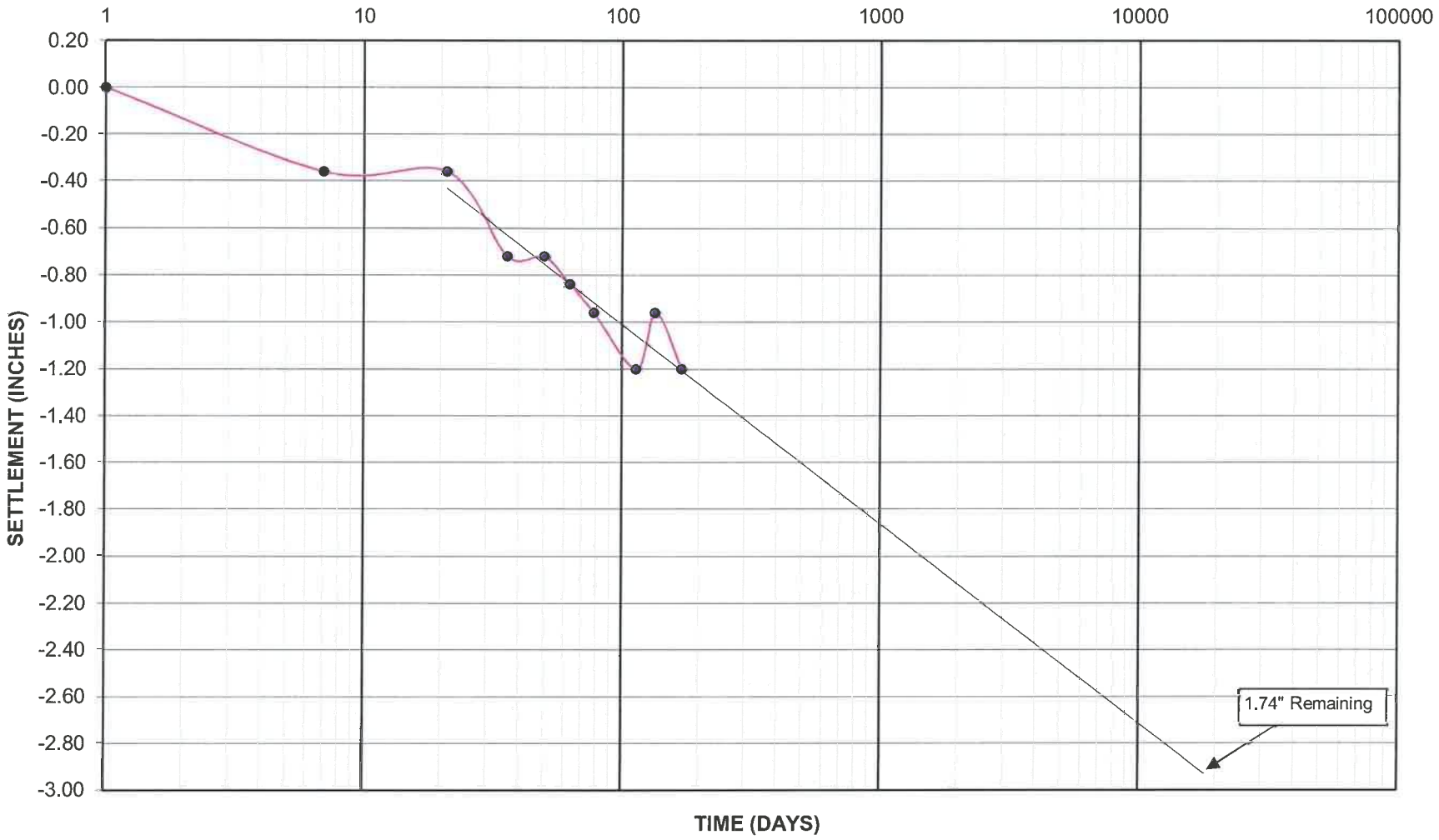


FIGURE B-4

12655-50

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM103-TC-log (3)

SETTLEMENT MONUMENT SM104-TC
Lot 13, Tract 13343

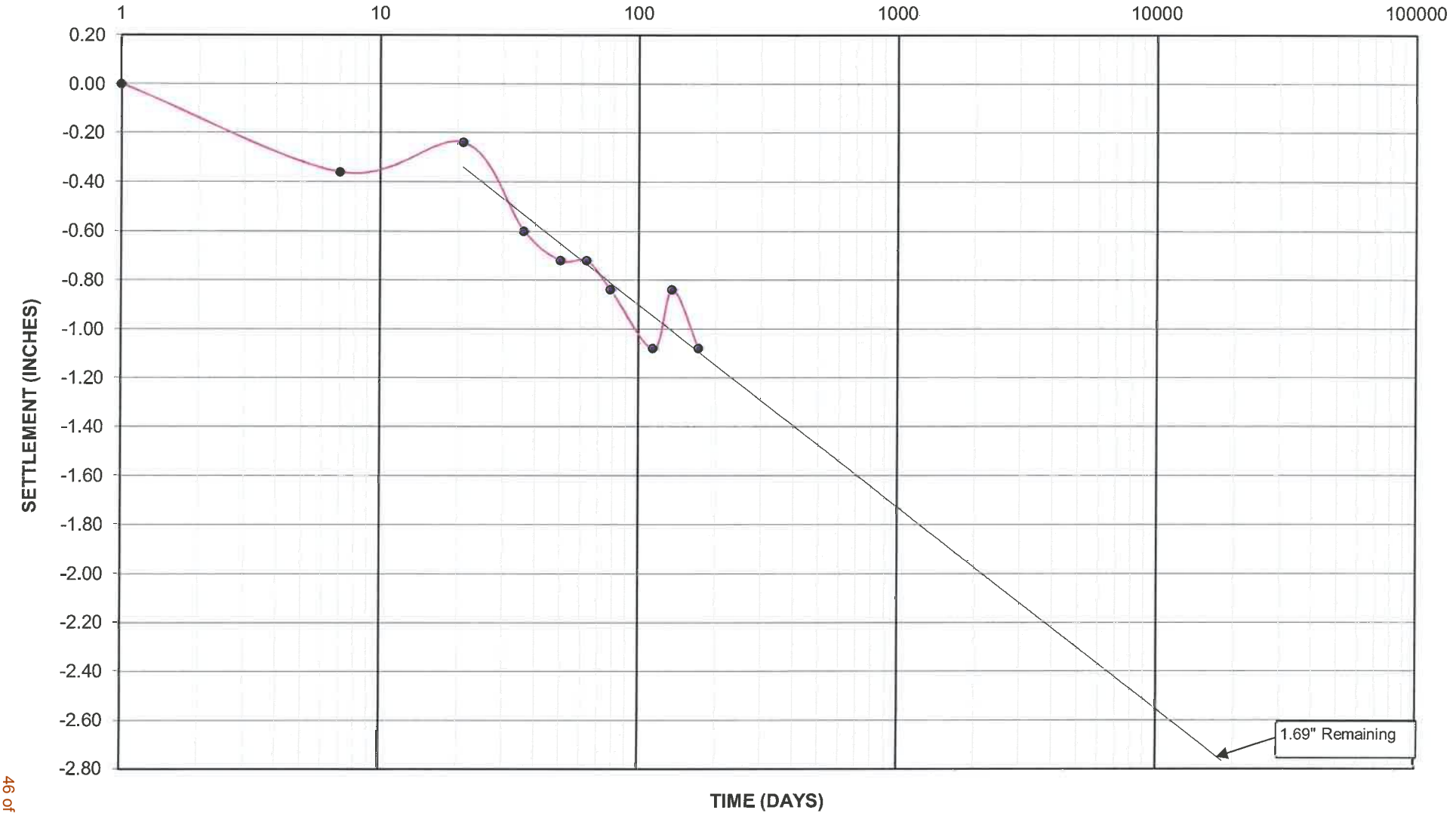


12655-50

FIGURE B-5

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM104-TC-log (3)

SETTLEMENT MONUMENT SM105-TC
Lot 13, Tract 13343



12655-50

FIGURE B-6

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM105-TC-log (3)

SETTLEMENT MONUMENT SM106-TC
Lot 13, Tract 13343

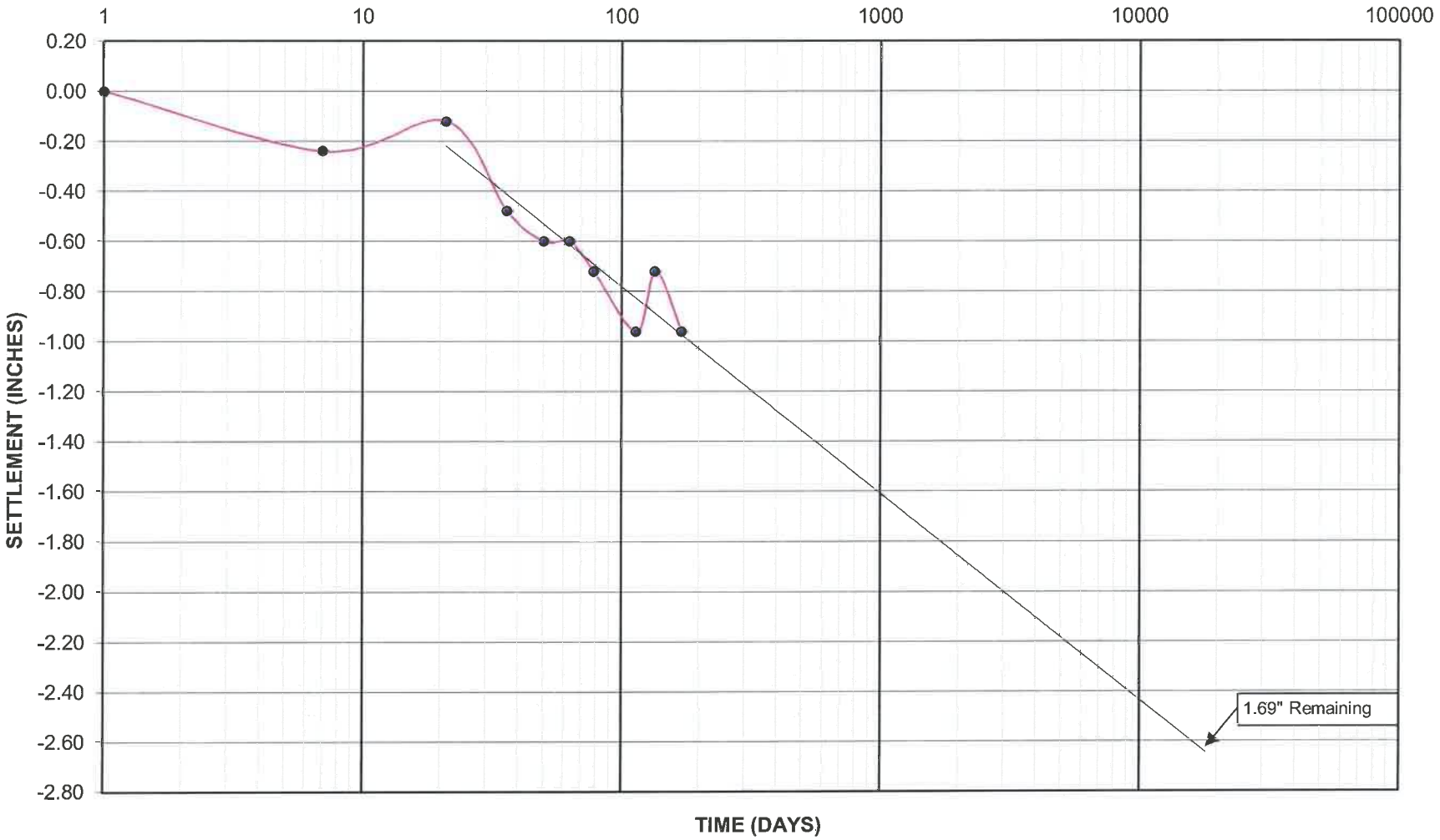


FIGURE B-7

12655-50

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM106-TC-log (3)

SETTLEMENT MONUMENT SM107-TC
Lot 13, Tract 13343

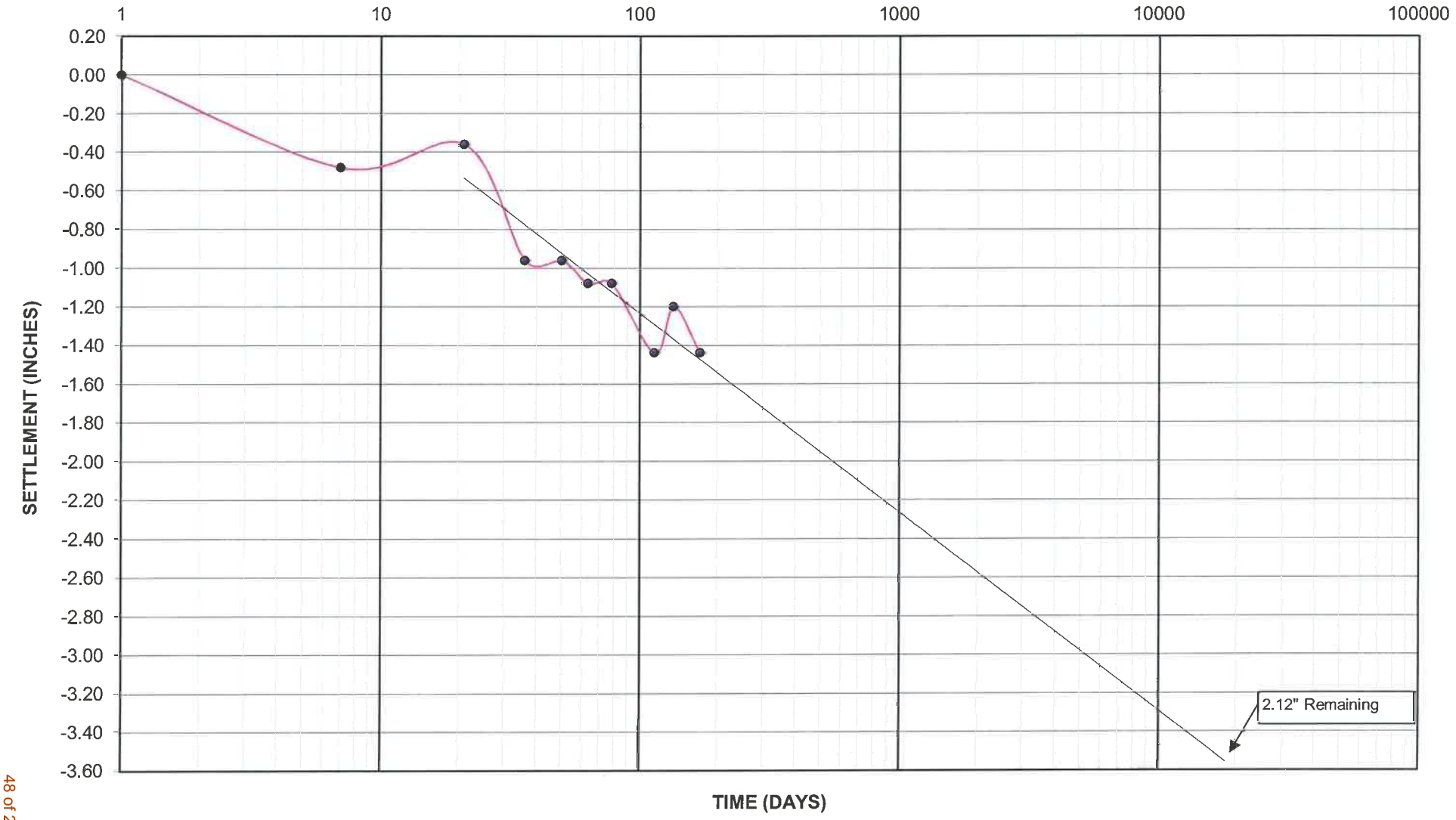
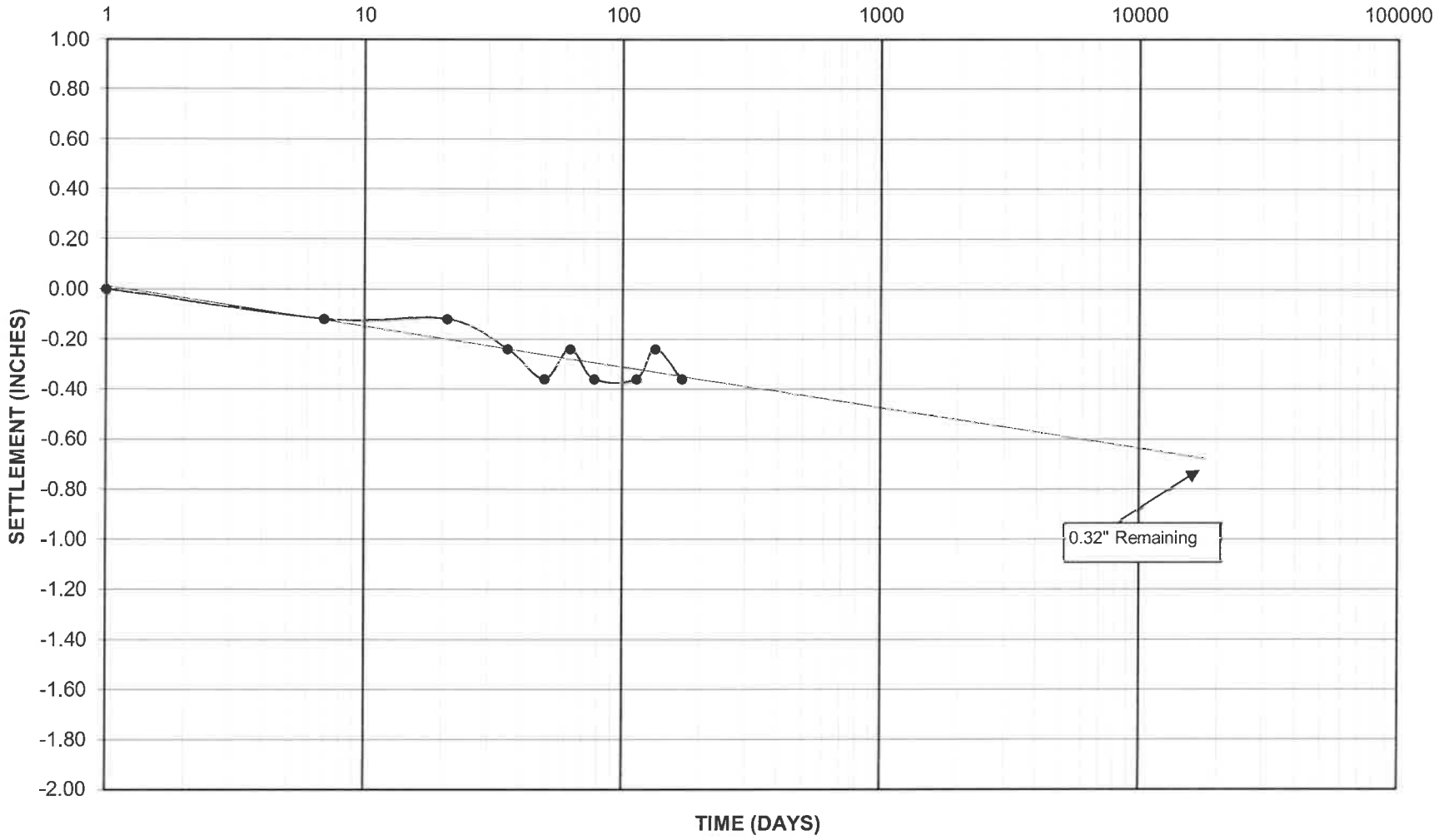


FIGURE B- 8

12655-50

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM107-TC-log (3)

SETTLEMENT MONUMENT SM108-TC
Lot 13, Tract 13343



12655-50

FIGURE B-9

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\10-9-08 Monuments Plots.xls SM108-TC-log (2)

PAST SETTLEMENT MONITORING SUMMARY
LOT 13, TRACT 13343

Monument No.	Start Reading	Grades Possibly Reached	Change In Pipe Extension	Change In Pipe Extension	End of Primary	End Reading	Reason Ended	Total Settlement (inches)	Estimated Trend at End Reading	Estimated Future Settlement As-Is (inches)	Qal Below Plate (feet)	Fill Below Plate (feet)	Fill Above Plate (feet)
SM2-1	6/22/1990 0 day	9/27/90 97 days	11/14/90 145 days Pipe cut down 5.3'			11/14/1990 145 days	PIPE BURIED 20 FEET BELOW FINAL GRADE	3.24	NOT USEFUL	Unknown	30 to 40	0	55
SM2-2	7/6/1990 14 days	9/27/90 97 days	11/14/90 145 days Pipe increased 5.2'	1/11/91 203 days Pipe increased 4.8'	5/03/91 305 days	4/9/1992 657 days	PIPE BURIED AFTER REACHING FINAL GRADE	8.92	Slowing; Settling 85% done	< 1.0	35	0	51
SM2-3	7/13/1990 21 days	10/3/90 103 days			5/03/91 305 days	10/30/1995 1956 days	PIPE DAMAGED ON 5-3-91 (315 days) BUT READINGS CONTINUED	12.3	Slowing; Settling 90% done	< 1.0	30 to 50	0	50
SM2-11	3/16/1992 633 days					12/26/1996 2379 days	BURIED UNDER FILL STOCKPILE	2.25	Constant Rate	1.5	30	60	5

FIGURE B-10

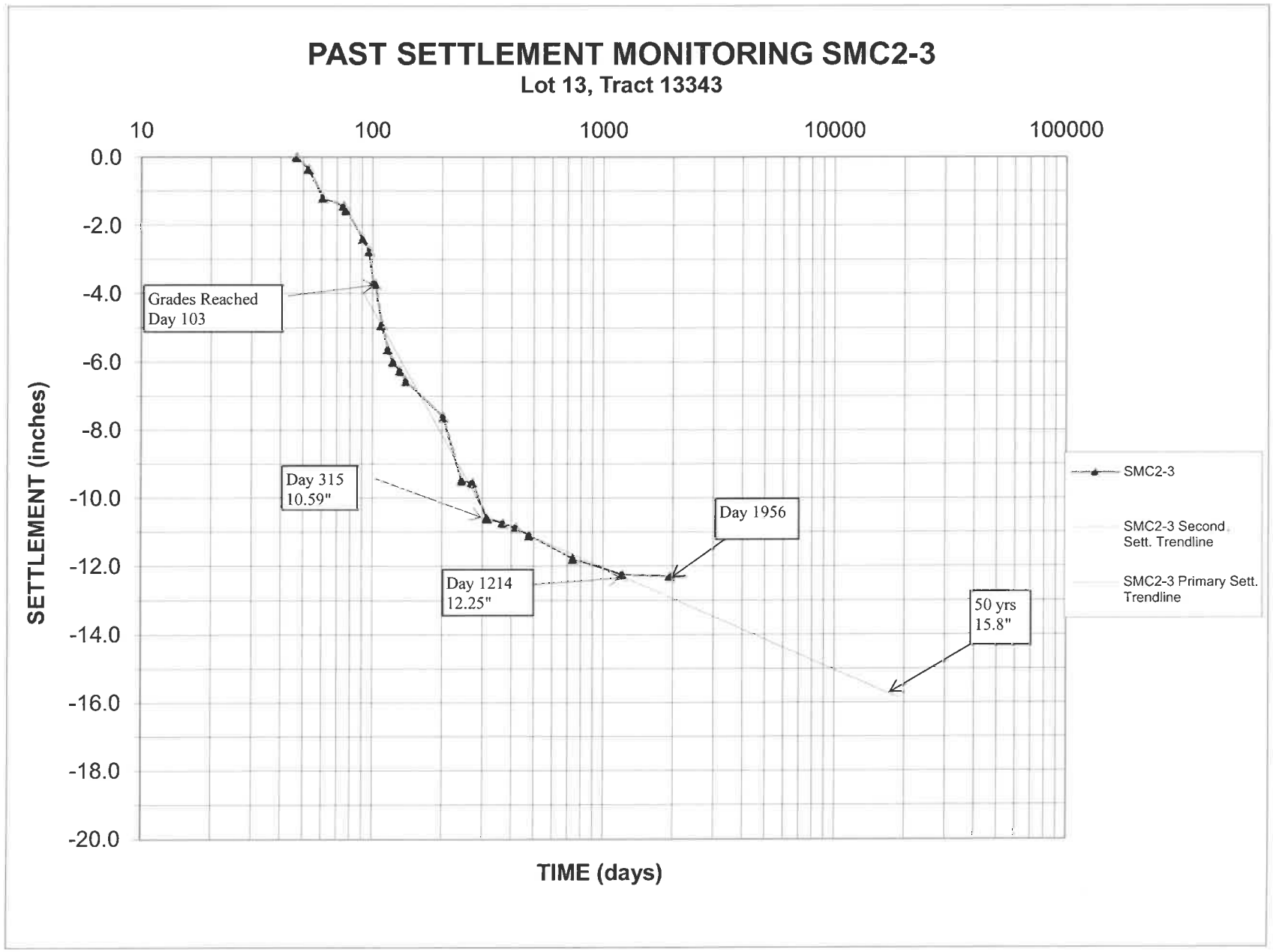


FIGURE B-11

12655-50

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\revised Settlement Monitoring Past.xls Sheet1 (5)

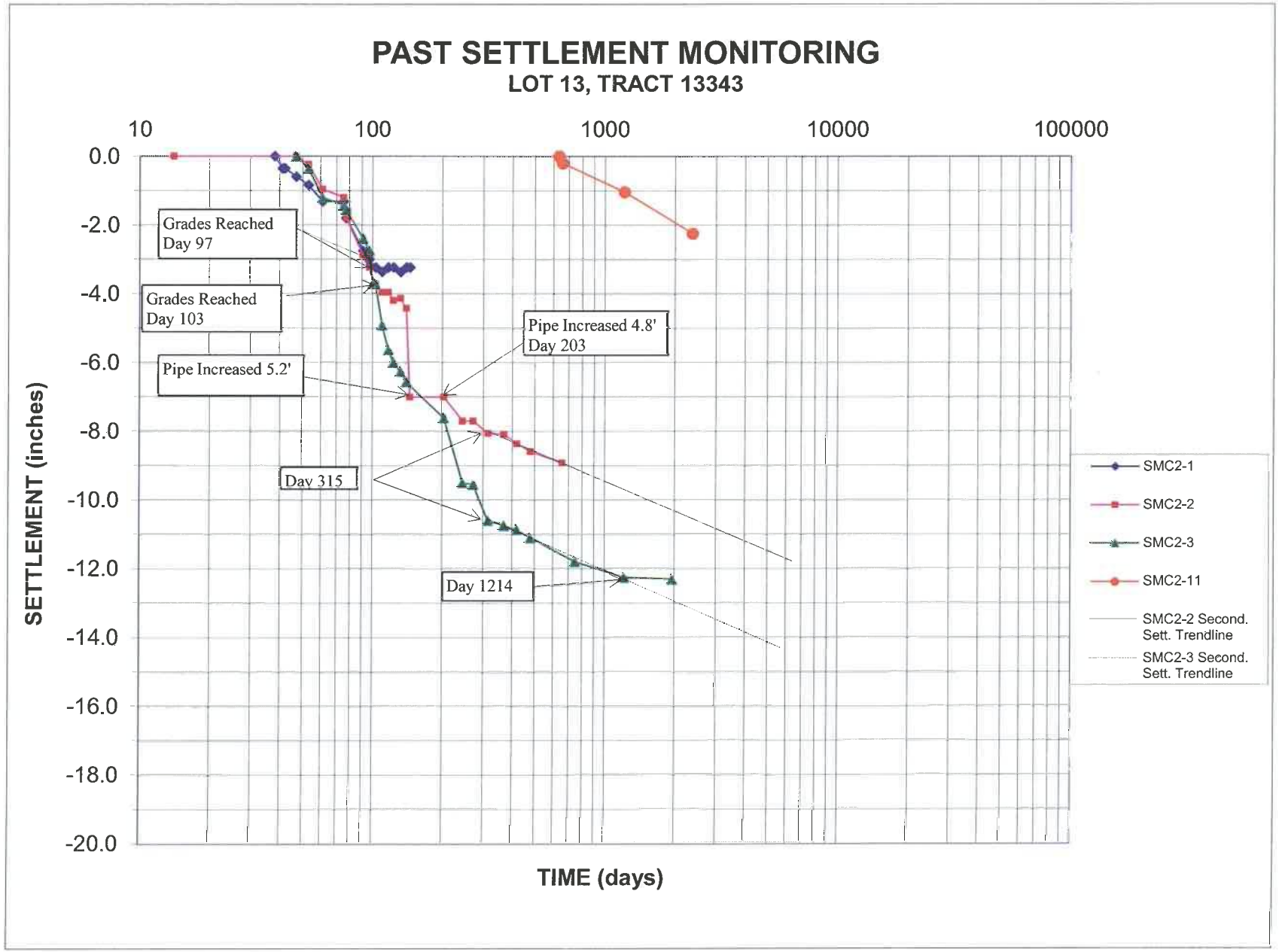
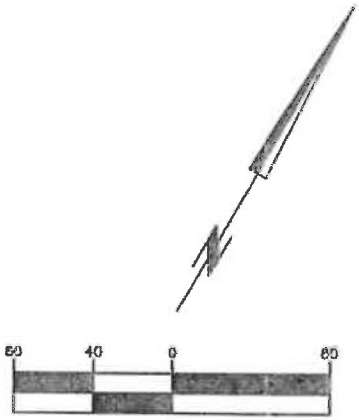


FIGURE B-12

12655-00

K:\12500-13000\12655-50\Project Coordination & Reports\Reports\Temp 09-11123\revised Settlement Monitoring Past.xls Sheet1 (6)

EXHIBIT "A



SCALE IN FEET
1 inch = 30 ft.

LEGEND

SM-108 ◆ RECENT SETTLEMENT MO

BY

DMC ENGINEERING

CIVIL ■ SURVEYING ■ PLANNING ■ CONSTRUCTION

Tel: (949) 753-9393 ♦ Fax: (949) 753-9322
18 Technology Drive, Suite 100, Irvine, California 92618
E-Mail: dmc@dmceng.com ♦ www.dmceng.com

PREPARED FOR

BDJ LAKE FOREST ASSOC. LP.
1420 BRISTOL STREET N. SUITE 100
NEWPORT BEACH, CA 92660

RECE

SM2-1
SM2-11
SM2-2
SM2-3
COMMERCE CENTER DRIVE
EX
APP
OLD
0

STONEY-MILLER CONSULTANTS, INC
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

June 9, 2014

BDJ Lake Forest Associates LLP
1420 Bristol Street North, Suite 100
Newport Beach, California 92660

Project No: 12655-04
Report No: 14-13775

Attention: Mr. Larry Nelson

Subject: **Supplemental Geotechnical Evaluation**
Seismic Hazard Study
Lot 13 of Tract 13343
25192 Commerce Drive
Lake Forest, California

References: See Appendix A

Dear Mr. Nelson:

In response to your request, we have performed a supplemental geotechnical evaluation of the subject property to assess the seismic hazard potential for development. The property address is 25192 Commercentre Drive, Lake Forest, California. The property is also known as Lot 13 of Tract 13343. The approximate location of the property is shown on the Site Location Map, Figure 1. Our evaluation is specifically to address a potential seismic hazard mapped in a State of California Seismic Hazard Zone Report for the El Toro Quadrangle. Our evaluation is directed to the evaluation and quantification of property seismicity, and the potential for liquefaction, settlement, slope instability, and slope deformation during an earthquake.

We understand that potential buyers of the property may develop the property with residential housing or commercial buildings. Supporting improvements include the access streets, parking areas, recreation building and amenities, and landscaped areas. Site grading is not known at this time but is assumed to be within a few feet of original rough grades. Buildings are anticipated to be up to three stories high and supported on shallow foundations or mat slab systems.

SITE HISTORY

The site was rough-graded in the early 1990's as part of the Pacific Commercentre development (References 1 through 3), with additional grading through 2008 to achieve the current site conditions. The rough-graded condition includes variable depths of engineered fill soil below the building pad supported by bedrock and alluvial soil, with a maximum 45-foot high slope graded at a 2:1 (horizontal:vertical) slope ratio.

Rough grading on or adjacent to the site consisted of cuts and fills to construct a relatively-flat commercial-sized lot. This grading (Reference 1) was performed from June 1990 to July 1991, and was performed under the observation and testing of Stoney-Miller Consultants, Inc. Grading resulted in the placement of engineered fill across the subject site to depths ranging from 2 feet within the south corner of the pad to an estimated maximum depth of about 60 feet on the north and west side of the pad. Alluvial thickness below the pad area is estimated to range from 0 to 35 feet. The pad was graded to an elevation of 590 to 592 feet above mean sea level (amsl), excepting the detention basin founded at an elevation of about 582 feet amsl.

More recent site grading consisted of bringing the subject site to proposed rough grades near elevation 590 to 595 feet amsl and placing an approximate 15-foot high surcharge fill at the location of the old detention basin area (References 4 through 8). The purpose of the surcharge was to mitigate the impact of alluvial settlements on improvements at the site by accelerating the occurrence of primary settlements, and the majority of the long-term secondary settlements anticipated under the fill and future footing loads. This additional grading was done from March 6, 2006, through May 30, 2008, and was performed under the observation and testing services of Stoney-Miller Consultants, Inc., and is reported in Reference 8. Fill soils were placed to an approximate elevation of 595 feet amsl on the eastern half of the basin, and 593 feet amsl on the western half. The temporary surcharge fill was then placed to an elevation of about 608 feet amsl between March 13, 2008 to April 2, 2008.

In January 2001, the State of California released a seismic hazard zones map for the El Toro quadrangle which includes the subject property. A portion of this map is shown in Figure 2, Seismic Hazard Map, and includes the approximate boundaries of the subject site. A state-mandated zone of required investigation for possible earthquake-induced landsliding is shown at the western boundary of the lot. The site and entire El Toro quadrangle is not located in an Alquist-Prioli Fault zone. The Alquist-Priolo Earthquake Fault Zoning was enacted to prevent the construction of buildings used for human occupancy on the surface trace of active faults.

GEOLOGIC SETTING

The topography of the area generally consists of the gently west-sloping Tustin Plain that merges to the east with the foothills of the Santa Ana Mountains that are characterized by southwest-trending canyons, washes, and ridges. The area lies within the San Diego Creek Watershed, a complex system of predominantly southwest-draining canyons and washes that ultimately reach Newport Bay. These include Borrego Canyon which is the main canyon drainage adjacent the site.

The alluvium within the Borrego Canyon drainage near the subject site is underlain at depth by bedrock of the Vaqueros-Sespe formation (undifferentiated) of late Eocene- to early Miocene-age, the Puente Formation of middle- to late Miocene-age and the Oso member of the Capistrano

formation of late Miocene- to early Pliocene-age. The Vaqueros-Sespe and Puente bedrock consists of interbedded sandstone and siltstone, whereas the Oso member bedrock consists of fine sandstone.

Groundwater was detected at an approximate depth of 30 feet below grade within both recent hollow-stem auger borings and CPT soundings using pore pressure dissipation testing. The historical high groundwater for this site is estimated to be at 10 feet below grade (CGS, 2006, Reference I.), which is consistent with groundwater levels observed during rough grading in the early 1990's. Groundwater levels may seasonally fluctuate or increase as a result of development but should not be a constraint to site development.

SITE INVESTIGATION

General

The subject site investigation performed herein was specifically to address the seismic hazard potential for earthquake-induced landsliding along a portion of a slope existing at the western corner of the building pad shown on the Geotechnical Map, Figure 3. The ground conditions in this area are discussed in References 1 through 8. The rough grading report (Reference 1) provides field density testing of the engineered fill and base of removal elevations for the alluvium and bedrock areas. This information is provided in Appendix B, Project Background, Figures B.1 and B.2. Included in previous evaluations are previous exploratory holes including four Cone Penetration Tests (CPT) in 2005 and two borings by Bagahi Engineering in 1997. Logs of these exploratory holes are also included in Appendix B.

Field Exploration

Subsurface conditions near the toe of the western slope were explored on April 23, 2014 and May 6, 2014. The approximate locations of these explorations are shown in Figure 3, Geotechnical Map. GeoTek Inc. (GT) placed two hollow-stem auger borings to a maximum depth of 51.5 feet. Both disturbed and relatively undisturbed samples were collected for laboratory testing. Groundwater was encountered at 29.8 to 30.8 feet below grade. Draft copies of GT Borings B-1 and B-2 are included in Appendix C, Field Exploration. Stoney-Miller Consultants, Inc. (SMC) placed two CPT soundings to a maximum depth of 50 feet adjacent the same toe of slope area. Logs of these CPT soundings are included in Appendix C. Pore pressure dissipation tests were performed in CPT-1 and CPT-2 with groundwater estimated at a depth of 33 feet below grade.

Laboratory Testing

Laboratory testing was performed by GeoTek, Inc. on samples retrieved from their borings. Laboratory test results for grain-size analyses and Atterberg limits testing is summarized on the draft boring logs in Appendix C.

SEISMIC CONSIDERATIONS

Published Studies

One of the principles of seismic analyses and prediction is the premise that earthquakes are more likely to occur on geologically younger faults, and less likely to occur on older faults. For many years, studies have described faults with Holocene movement (within the last 11,000 years) as "active", and faults with documented Pleistocene movement (within the last 1.6 million years) and with undetermined Holocene movement as "potentially active". Informally, many studies have described faults documented to have no Holocene movement as "inactive". Recent geologic and seismic publications are attempting to clarify the nomenclature describing faults to more accurately represent the potential affects from earthquakes.

Reports by the California Division of Mines and Geology indicate faults with documented Holocene or Historic (within the last 200 years) movement should be considered "active". However, "potentially active" faults are more appropriately characterized in terms of the last period of documented movement. The Fault Activity Map of California (Jennings, C.W.; 2010) defines three categories for onshore "potentially active" faults. The categories are associated with the time of the last displacement evidenced on a given fault and are summarized in Table 1.

Table 1
Definitions of Fault Activity in California

Activity	Category	Recency of Movement
Active	Historic	Within the last 200 years
	Holocene	Within the last 11,000 years
Potentially Active	Late Quaternary	Within the last 700,000 years
	Quaternary	Within or possibly within the last 1.6 million years
	Pre-Quaternary	Before the last 1.6 million years

It is important to note these categories embrace all Pre-Holocene faults as "potentially active" and provide no methodology to designate a given fault as "inactive". Although the likelihood of an earthquake or movement to occur on a given fault significantly decreases with inactivity over geologic time, the potential for such events to occur on any fault cannot be completely eliminated within the current level of understanding.

Local and Regional Faults

The closest published active fault to the site is the San Joaquin Hills Fault, approximately 3.9 miles from the site (Blake, T.F., 2000, CDMG/2004, References J and K). Other active faults in the vicinity of the site include the offshore extension of the Newport-Inglewood Fault Zone, approximately 11.6 miles west, the Palos Verdes Fault, approximately 25.2 miles to the northwest, the Coronado Bank Fault, approximately 29.6 miles to the southwest, and the San Andreas Fault, approximately 43.5 miles to the northeast.

The San Joaquin Hills Fault is not indicated in some reports as being a causative seismic feature, (Jennings, C.W.; 2010, Reference L). However, the California Geological Survey includes the structure which underlies the San Joaquin Hills from Dana Point to Newport Beach, ramping upward toward the Irvine area, in data files as a significant earthquake source with a potential for strong events (M6.6). With the fault's location approximately 3.9 miles to the northwest, it is calculated as the most significant seismic source to affect this site. Given the present level of understanding of this structure it is, in our opinion, appropriate to include this portion of the fault as a causative seismic feature.

Ground Motion Analyses

The potential ground motions from earthquakes that could impact the sites were analyzed through probabilistic methods. The probabilistic method considers the regional seismic history and the slip rates of faults within a 100-mile radius of the subject site. Utilizing attenuation relationships (Bozorgnia, et al.; 1999, unconstrained/pleist. Soil, Reference M)), one can estimate the ground motion history of the site and attempt to predict the probability of future accelerations within a given period of time. The study indicates the maximum site acceleration from 1800 to 2014 was approximately 0.13g and occurred during a magnitude 6.3 Long Beach Earthquake 16.3 miles from the site on March 11, 1933.

It is noted that the estimation of peak ground accelerations presented above is provided for the interest of the Client and is required by local (City or County) review agencies. The values derived are not directly utilized in structural design of residential or commercial structures. Seismic parameters for use by the structural engineer in accordance with 2013 California Building Code (CBC) in design of the proposed structure(s) are presented in Appendix D, Seismicity Background.

Site Classification for Seismic Design

In order to determine the seismic design parameters provided in this report, the upper 100 feet of soil underlying the subject site has been classified in accordance with Section 1613.5.5 of the 2013 CBC and Section 20.3.1 of ASCE 7-10. Given the soils underlying the site are liquefiable and classify as Site Class F per the 2013 CBC, a site-response analyses would be required.

However proposed new structures are anticipated to have a fundamental period of vibration less than 0.5 seconds (to be verified by the structural engineer). As such, Section 20.3.1 in ASCE 7-10 provides an exception that indicates such liquefiable sites may be classified in accordance with Table 20.3-1 without performing site-response analyses.

Given this exception, the results of our field testing indicate the average shear wave velocity within the upper 50 feet of the alluvium varies from 603 to 1138 feet per second (see Figure D.1, Appendix D). The shear wave velocity of the engineered fill soil and bedrock are estimated at 1,000 and 2,000 feet per second, respectively. The site shear wave velocity, Vs(100'), of 986 to 1,077 feet per second (300 to 328 m/sec) is determined in Figure D.2, Appendix D using the procedure from CBC 1613.5.5. The site is then considered to have a "Stiff Soil" profile and a Site Class of D.

Secondary Seismic Hazards

Review of the Seismic Hazards Zone Map of the El Toro Quadrangle (CGS, 2001, Reference N) indicates that the site is not within a "zone of required investigation" for liquefaction, but is in a "zone of required investigation" for landsliding. Based on the results of our subsurface investigation and calculations, there is a potential for liquefaction to occur at this site. Our evaluation of potential liquefaction and landsliding are discussed below.

Other secondary seismic hazards can include deep rupture, shallow ground cracking, and ground settlement. Without onsite faulting, the potential for deep fault rupture is not present. The potential for shallow ground cracking to occur during an earthquake is a possibility at any site, but does not pose a significant hazard to site development. For a design seismic event, we estimate settlement of sands located above the water table to be less than 0.5-inch.

Appraisal of Liquefaction Potential

Liquefaction is a process by which water-saturated sandy sediments lose strength and fail during strong earthquake shaking. Loss of bearing strength from liquefaction can result in deformations which allow buildings to settle and tilt. Factors known to influence liquefaction include soil type, grain size, relative density, depth to groundwater, and intensity and duration of seismic ground shaking.

Although the Seismic Hazard Zones Map for the El Toro Quadrangle (CGS, 2001, Reference N) does not identify the site and adjoining area as within a zone of required investigation for liquefaction, it does identify it as a zone for landsliding study. Since the site and western slope are comprised of predominantly sandy fill soil and alluvium materials, the site is considered grossly stable for static conditions. The potential landslide hazard would then be due to weakening of the soil during a seismic event.

Seismic Settlements

Liquefaction analyses were made for the CPT-01 to CPT-04 (2005), GT Borings B-1 and B-2, and CPT-1 and CPT-2 (2014). CPTs were evaluated using computer program CLiq version 1.7.5.27 and boring blowcount data was analyzed using the computer program LiquefyPro by CivilSoft. Analyses were generally made using a maximum considered earthquake of magnitude M7.0 with a peak ground acceleration of 0.6g. The analyses are all included in Appendix E, Liquefaction Analyses.

All evaluations of the CPTs and borings show that some of the alluvium may liquefy during the maximum considered earthquake. However, the liquefiable layers tend to be relatively thin and discontinuous. The CLiq program (CPT analyses) provides estimates of ground settlement that are modified using the method of Cetin, et al. (2009) which linearly weights the ground strains with depth from 1.0 at the surface to 0.0 at a depth of 50 feet.

The liquefaction results for CPT-01through CPT-04 are provided on Figures E1 through E.36, Appendix E. In summary, the effects of liquefaction below the building pad area are nil. This is due to the Cetin et al. procedure with the alluvium depths more than 50 feet below grade.

The liquefaction results for CPT-1 and CPT-2 near the toe of slope area are given on Figures E.37 through E.53, Appendix E. In summary, the effects of liquefaction at the toe of slope area are minor with respect to ground settlement of 0.8 to 1.7 inches. A screening level evaluation of lateral spreading provides an estimate of 12 to 24 inches of deformation. This screening-level of deformation requires that seismic slope stability and deformations be checked using estimated residual strengths of the alluvium after liquefaction has occurred. This evaluation is made in the next section to show that seismic slope deformations are limited and acceptable for buildings according to current code requirements.

The liquefaction results for Borings B-1 and B-2 are given on Figures E.54 through E.61, Appendix E. In summary, the effects of liquefaction at the toe of slope area are moderate with respect to ground settlement of 3.7 to 8.3 inches. If the procedure of Cetin, et al. was used here, the settlement range would be halved to approximately 2 to 4 inches. This should not be a concern for the slope area.

In conclusion, our evaluations indicate insignificant seismic settlements beneath the building pad area and tolerable settlements of 1 to 2 inches beneath the toe of slope area.

Potential Slope Deformations

For certain conditions, ground liquefaction from ground shaking can cause gently sloping terrain to fail laterally, particularly where an open slope face is present. Based on our lateral spreading screening analyses with the CLiq program, lateral spreading deformations need to be reviewed and analyzed more closely to assess whether they are a concern given the subsurface and seismic conditions at the property. Since soil strengths may be reduced during or after seismic events, soil strengths are considered next followed by seismic slope stability and deformations.

Seismic Soil Strengths

The evaluation of the dynamic strength of the alluvium is made using the work of Robertson (2010) and Boulanger and Idriss (2006). Robertson discusses the evaluation of flow liquefaction and liquefied strength using the Cone Penetration Test, and Boulanger and Idriss discuss the liquefaction susceptibility of silt and clay materials. A full discussion of the soil strength evaluation is provided in Appendix F; a summary is provided below.

Robertson (2010) discusses conditions where the soil may contract or strain-soften under earthquake loading, and the annotated soil behavior type (SBT) chart may be used as a tool to screen the site soils for contractive behavior. All CPT soundings were evaluated using the SBT chart and the results of this evaluation are shown in Figure F.1. The CPT data in Appendix E is used to approximate the distribution of the CPT data which is in each zone. Materials with strain-hardening behavior represent 83 to 95 percent of the soil materials explored onsite. Materials with potentially detrimental strain-softening response represent an estimated 5 to 17 percent of the soil materials explored onsite. Soil materials with strain-softening type behavior are further defined using a similar SBT chart (Figure F.16) which shows the mean of flow liquefaction materials from case studies as having a $Q_{tn,cs} < 50$. The CPT data in Appendix E is used to determine the soil layers where this occurs; these soil depth increments are shown on the far right-hand column of Figure 1. The data from Geotek borings B-1 and B-2 may be evaluated in a similar manner using a normalized clean sand blow count value of 15 or less as indicative of contractive behavior. Five samples from the borings are determined as possibly susceptible to flow liquefaction. The depth intervals where potential flow liquefaction materials were detected onsite are shown in cross section on Figure F.4. The contractive soil material locations under the building pad are sparse and not continuous, whereas those at the slope toe are more frequent but still discontinuous. Approximately 10 to 40 percent of the materials from 34 to 50 feet below grade at the toe of slope may be subject to strain-softening behavior.

The evaluation of materials for sand- or clay-like behavior uses the work of Boulanger and Idriss (see Figures F.19 and F.20). A summary of this evaluation is shown in Figures F.5 through F.8. With the majority of the plasticity indices (PI) of 7 or more, we generally concur with the identification of soil types from the GeoTek borings. Soil materials with the $PI \geq 7$ would

potentially be subject to "cycling softening", instead of contractive liquefaction behavior of sand materials during a seismic event.

The dynamic strength of the alluvium is then made using the results of the CLiq and CPeT-IT programs by Geologismiki Geotechnical. Estimates of seismic strength for sand-like materials are provided on the far right-hand side of Figures E.6, E.15, E.24, E.33, E.42, and E.50. These estimates only require that the effective overburden pressure be known to determine the residual strength as an effective cohesion in pounds/square foot. Estimates of seismic strength for clay-like materials are made using the undrained strength of the soil materials.

The data from these analyses were placed in a spreadsheet for calculation, and the results for the buried alluvium are summarized on Figure F.9. The alluvial strengths are based on evaluation of CPT-1 and CPT-2 (2014) data. Figure F.9 shows a plot of estimated shear strengths for static and seismic loading conditions, and the shear strength value selected for seismic slope stability evaluation. Seismic soil strengths were calculated using results from the CLiq program, but limited by an upper-bound undrained strength ratio to 0.45 as recommended by Robertson (2010). The seismic soil strengths were then chosen in consideration of both the seismic residual and static shear strengths. An average or more conservative static strength was selected for the soil materials above 34 feet at the toe of slope since these soils did not exhibit strain-softening tendencies. Since 10 to 40 percent of the material below 34 feet showed some strain-softening potential, the seismic shear strength was selected as somewhat less than the average of the static and seismic strengths. Seismic soil strength of alluvium materials below a depth of 50 feet below grade was modeled as a cohesionless soil with a friction angle of 25 degrees which is consistent with the recommended by Robertson (2010).

Slope Stability and Deformation

With the seismic strength for the alluvium defined, improved evaluations are made for the lateral slope deformation potential. A full discussion of the evaluation of slope stability and deformation is provided in Appendix G; a summary is provided below.

Seismic slope stability analyses were made for Cross Sections A-A shown in Figures 3 and 4. Analyses are made for both an arcuate- and block-shaped surface. The analyses results are provided in Appendix G. The results indicate an adequate seismic factor of safety of 1.1 or more for a pseudostatic horizontal acceleration of 0.15 g. These results are provided on Figures G.1 through G.11 for the arcuate-shaped surface and on Figures G.30 through G.37 for the block-shaped surface. These results are sufficient to satisfy current code requirements.

Additional seismic slope stability analyses are made for the arcuate and block surface of Cross Section A-A' to estimate the yield acceleration (horizontal acceleration for a factor of safety of 1.0). The analyses results are provided on Figures G.12 through G.25 for the arcuate surface and on Figures G.38 through G.46 for the block surface. The results indicate yield accelerations from 0.19 to 0.23 in order to achieve a factor of safety of 1.0.

Slope deformations estimates are made for the arcuate and block surface on Figures G.28-29 and G.47-48. These deformation estimates are made using the method of Rathje and Antonakos (2011). For the seismicity estimates provided in Appendix D, the slope deformation is estimated for a maximum considered seismic event and probable seismic event (probability of exceedance of 2 percent and 10 percent in 50 years, respectively). Slope deformation estimates for the maximum considered event range from 2 to 6 inches. Slope deformation estimates for the probable seismic event range from 0.4 to 1.4 inches. These are within the acceptable range for buildable areas per SP 117 guidelines. Effects of slope deformation are estimated to manifest at distances of 50 feet or less from the top of slope.

In conclusion, our evaluations indicate that factors of safety for pseudostatic analyses are 1.1 or more in compliance with current code requirements and the generally accepted standards of practice. Slope deformations are estimated at 1.5 inch or less for a probable seismic event which is acceptable per the California SP117 guidelines. Slope deformations are estimated at 2 to 6 inches for the maximum considered seismic event.

CONCLUSIONS

Based on our investigation and evaluation, it is concluded that there is a very low to low seismic hazard due to liquefaction and slope instability present on the subject lot. Although liquefaction of the underlying sandier materials may occur under the maximum considered earthquake, settlements and lateral slope movements should be within the acceptable range for the site slope and planned residential or commercial improvements. Precise grading, improvement plans, and foundation plans should be reviewed by the Geotechnical Consultant prior to grading and construction. All future grading to prepare the site area should be performed under the observation and testing of the Geotechnical Consultant.

LIMITATIONS

Our work was performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report. The opportunity to be of service is appreciated. If you have any questions, please call.

Respectfully submitted,

STONEY-MILLER CONSULTANTS, INC.

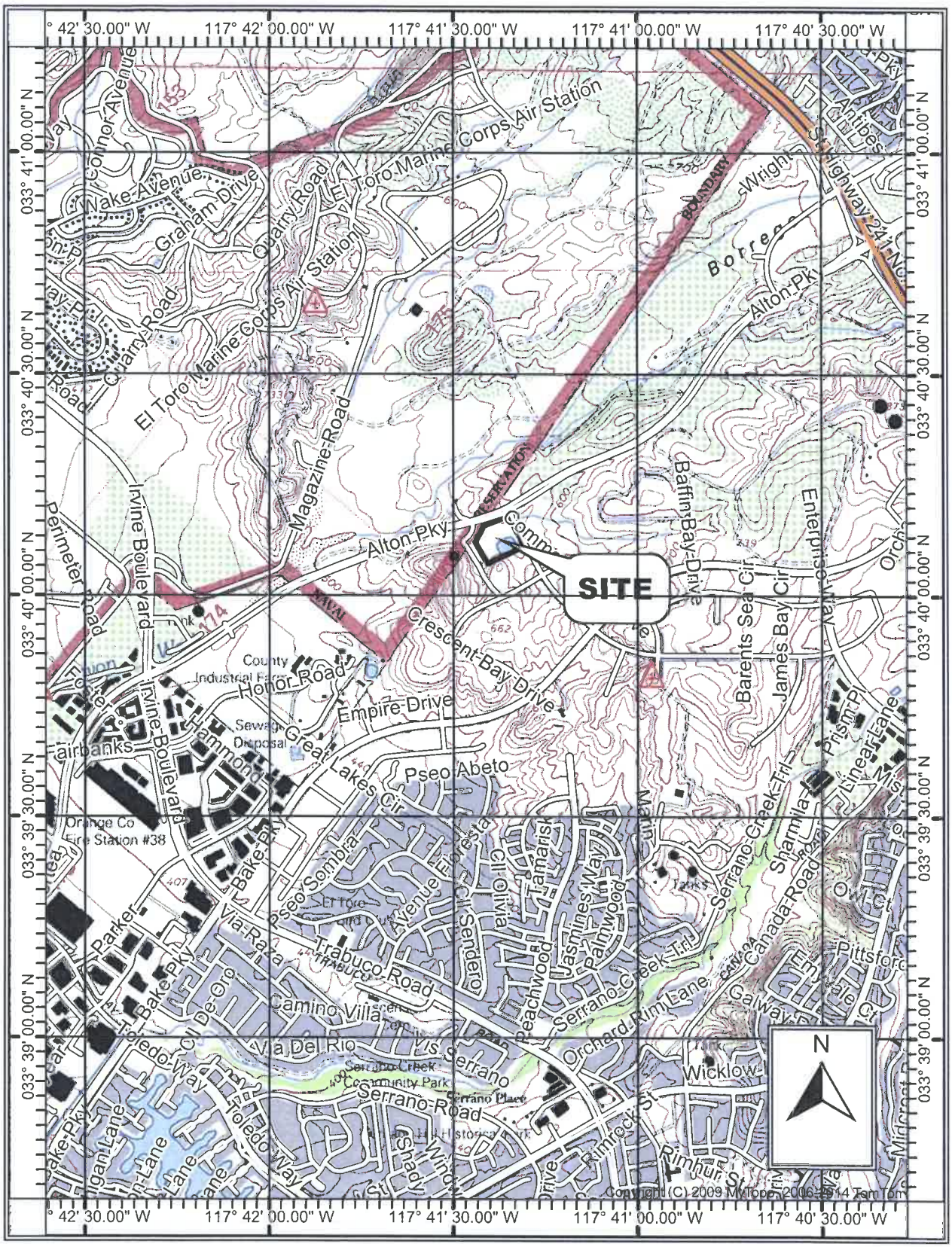
Russell C. Lamb, G.E. 2207
Chief Geotechnical Engineer
Registration Expires 3-31-15

Kevin A. Trigg, E.G. 1619
Chief Engineering Geologist
Registration Expires 12-31-14

RCL/KAT/dd

Attachments:
Figure 1, Site Location Map
Figure 2, Seismic Hazard Map
Figure 3, Geotechnical Map
Figure 4, Geotechnical Cross Section
Appendix A, References
Appendix B, Project Background
Appendix C, Field Exploration
Appendix D, Seismicity Background
Appendix E, Liquefaction Analyses
Appendix F, Dynamic Strength Evaluation
Appendix G, Seismic Slope Stability and Deformation

Distribution: (5) Addressee



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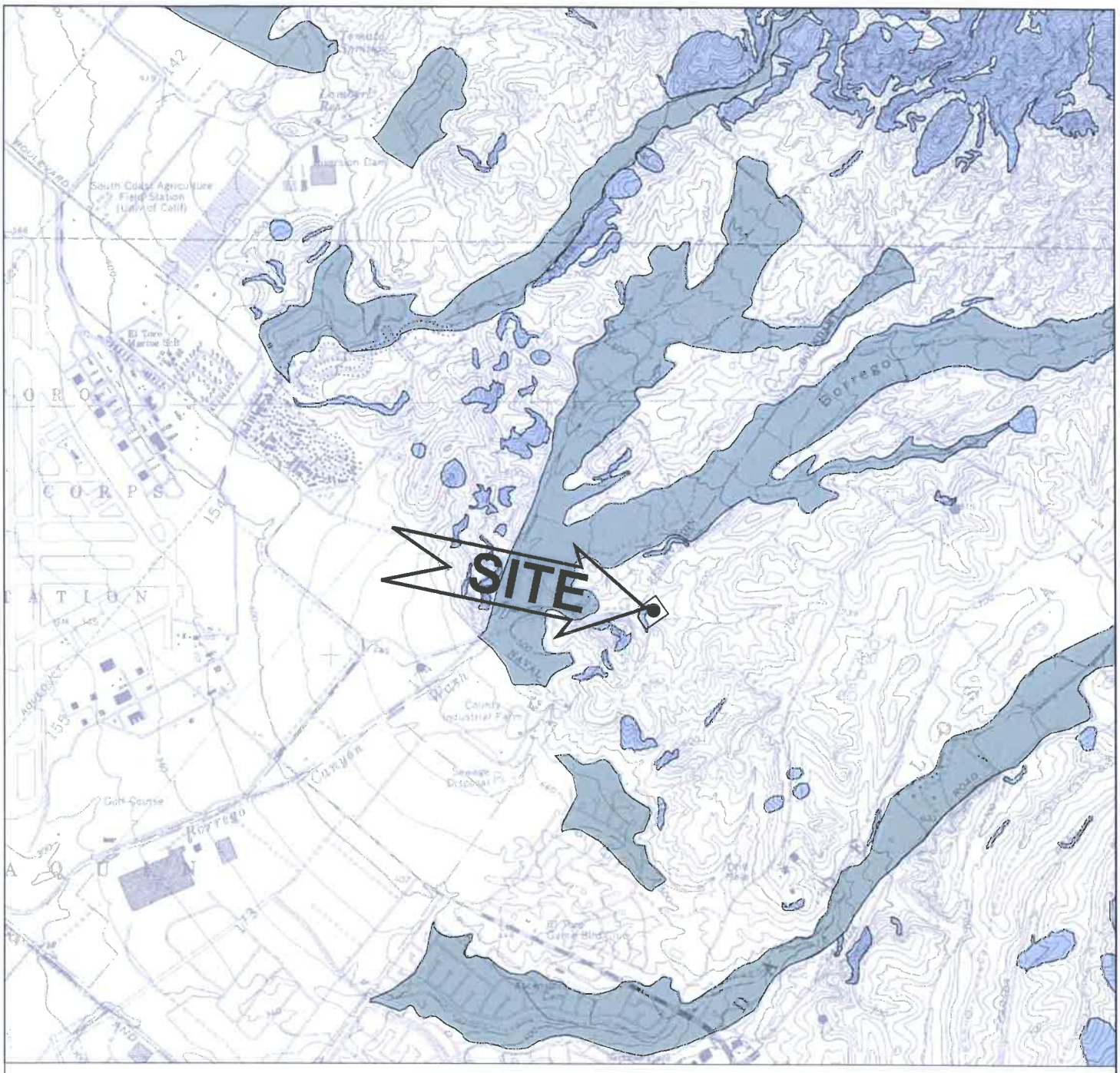
FEET

SMC

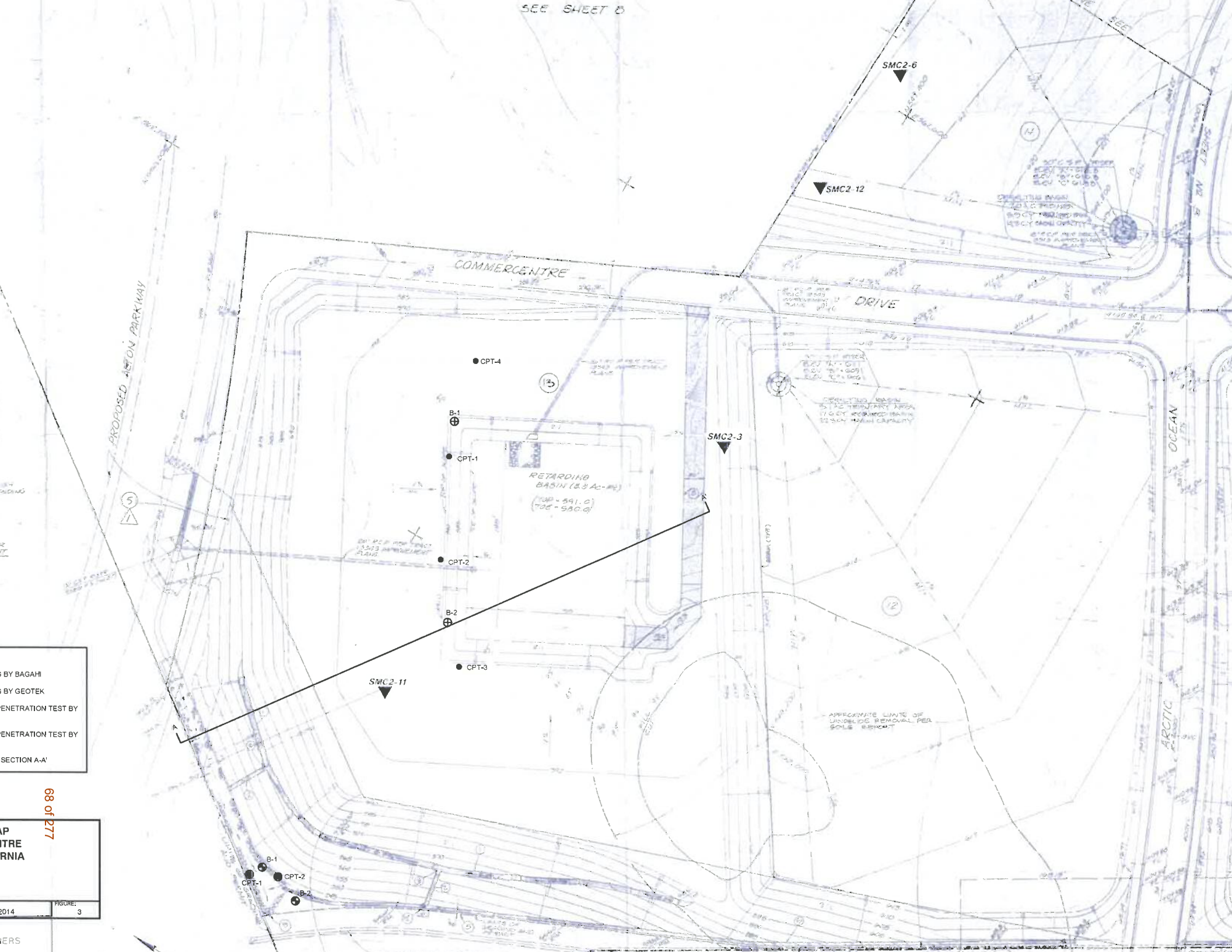
SITE LOCATION MAP

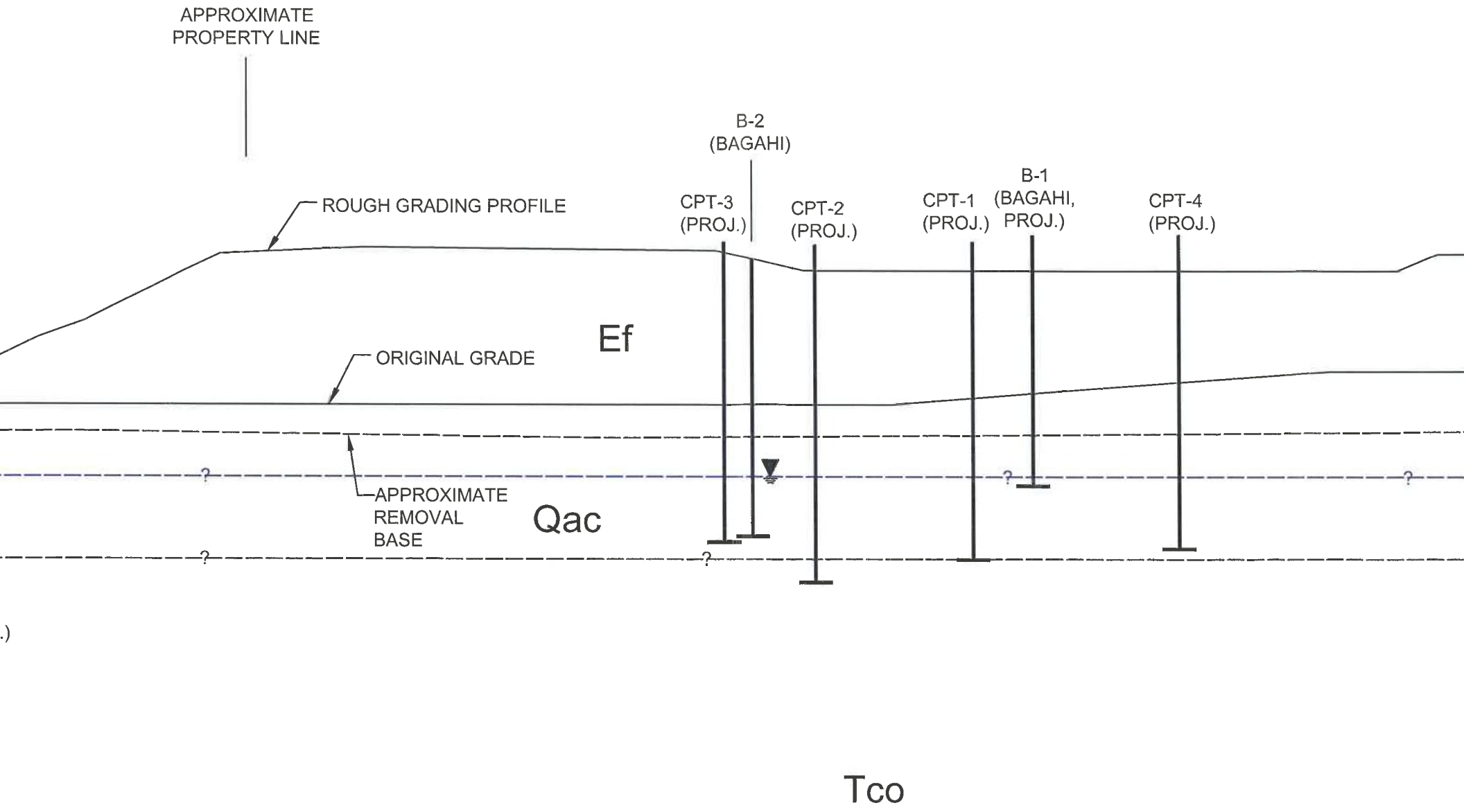
PROJECT NO. 60119-01 DATE: MAY 2014 FIGURE NO. 1



SEISMIC HAZARD MAP

JOB NO.:	DATE:	FIGURE NO.:
12655-00	MAY 2014	2


SEE SHEET 8
SMC2-6
SMC2-12
COMMERCE CENTRE
DRIVE
PROPOSED AEON PARKWAY
OCEAN
ARCTIC
CPT-4
B-1
CPT-1
SMC2-3
RETARDING
CPT-2
B-2
CPT-3
SMC2-11
APPROXIMATE LIMIT OF
B-1
CPT-2
CPT-1
B-2
BY BAGAHI
BY GEOTEK
PENETRATION TEST BY
PENETRATION TEST BY
SECTION A-A'
AP
ITRE
RNIA
FIGURE:
3
2014
ERS


APPROXIMATE
PROPERTY LINE
B-2
(BAGAHI)
ROUGH GRADING PROFILE
CPT-3
(PROJ.)
CPT-2
(PROJ.)
CPT-1
(PROJ.)
B-1
(BAGAHI,
PROJ.)
CPT-4
(PROJ.)
Ef
ORIGINAL GRADE
APPROXIMATE
REMOVAL
BASE
Qac
Tco

APPENDIX A

LIST OF REFERENCES

APPENDIX A

LIST OF REFERENCES

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2. Stoney-Miller Consultants, Inc., 1993, "Settlement Monitoring, Canyon 2, Lots 12 through 20, Tract 13343, 20858 Bake Parkway, El Toro, California," dated December 10, 1993, Revised December 20, 1993, Project No: 10425-06, Report No. 3-3301.

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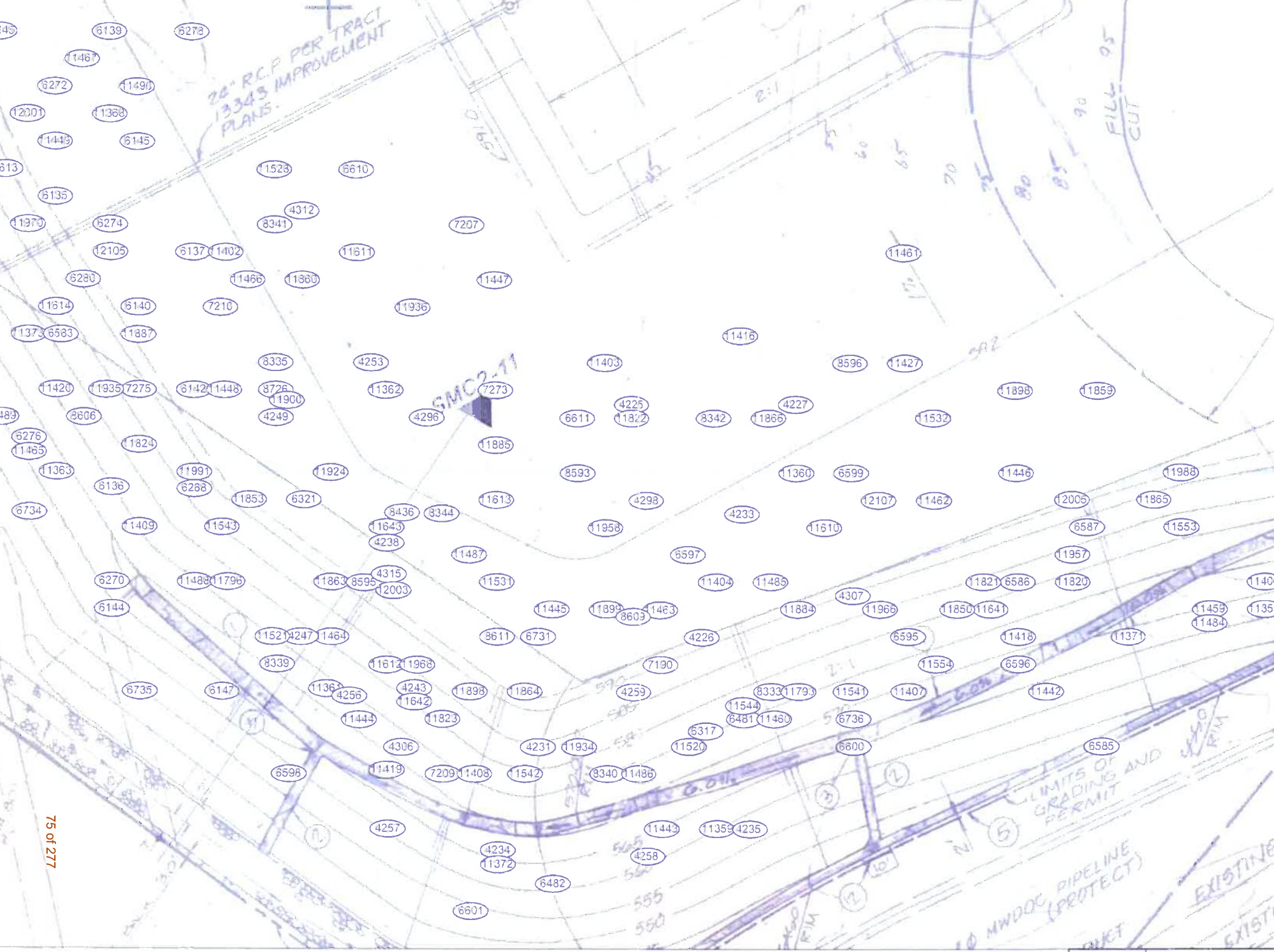
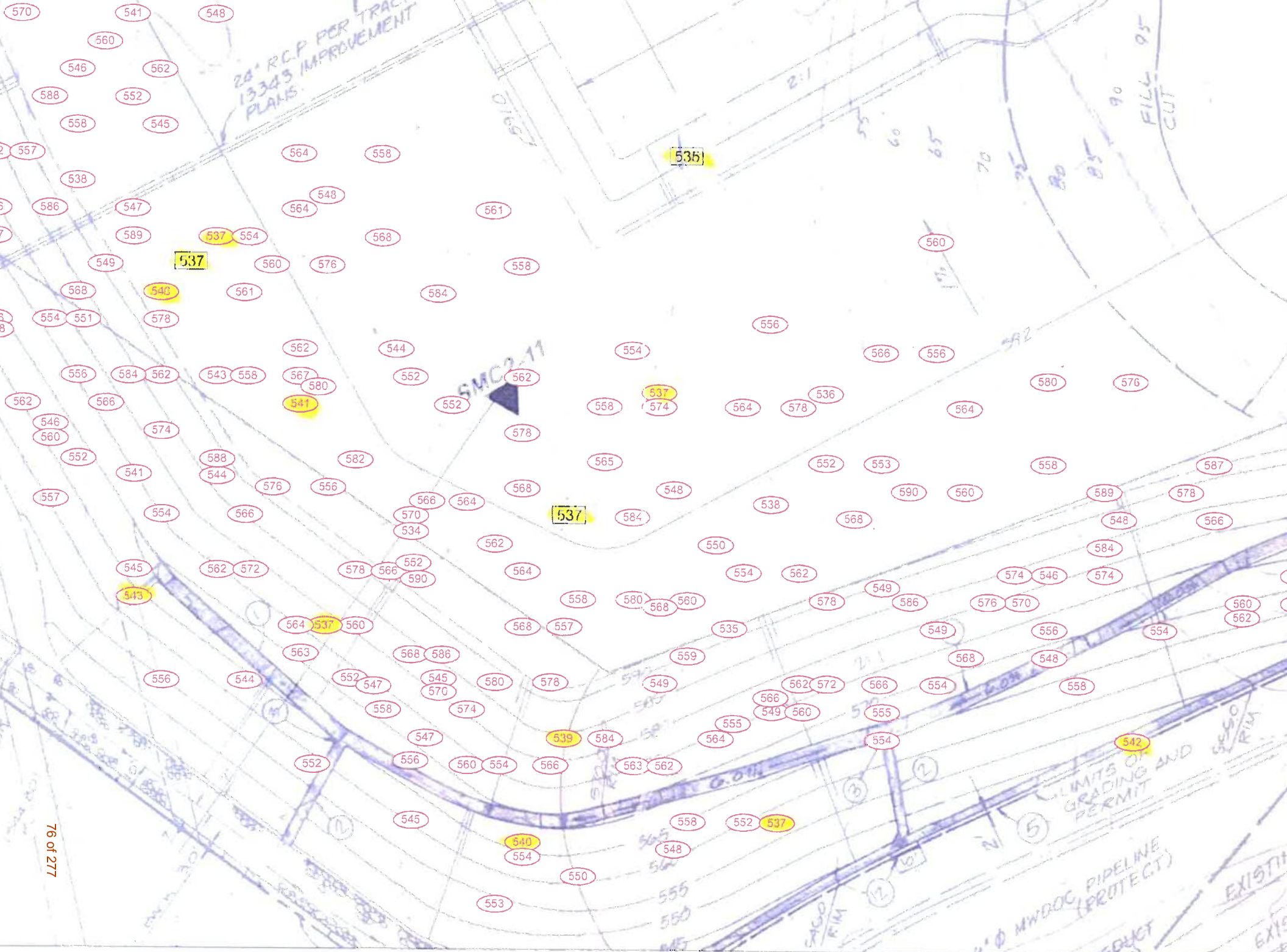
APPENDIX B

PROJECT BACKGROUND



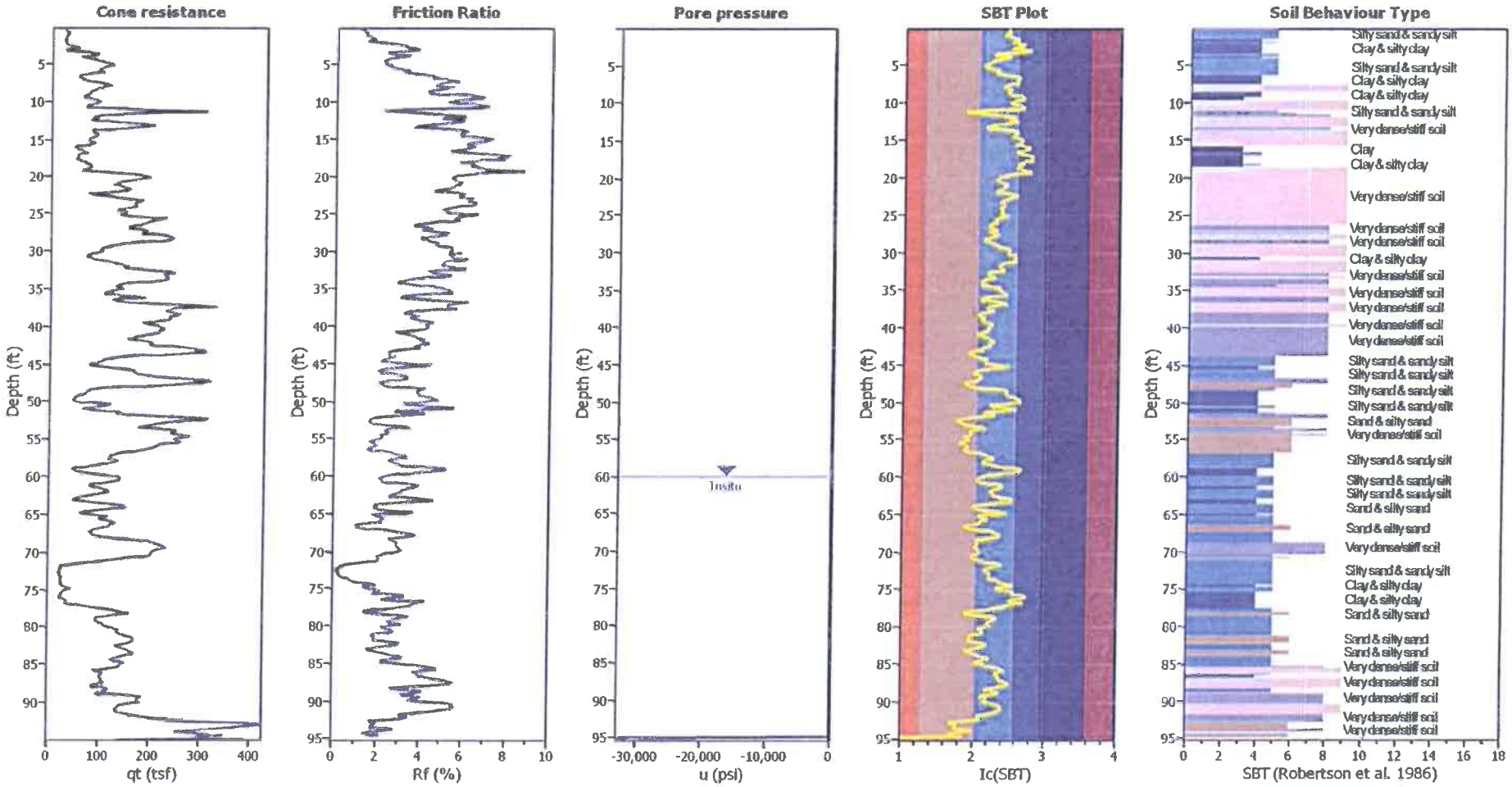
DENSITY TES
TRA



537 APPROXIMATE ELEVATION OF FIELD DENSITY TEST

DENSITY TEST

CPT basic interpretation plots



Input parameters and analysis data

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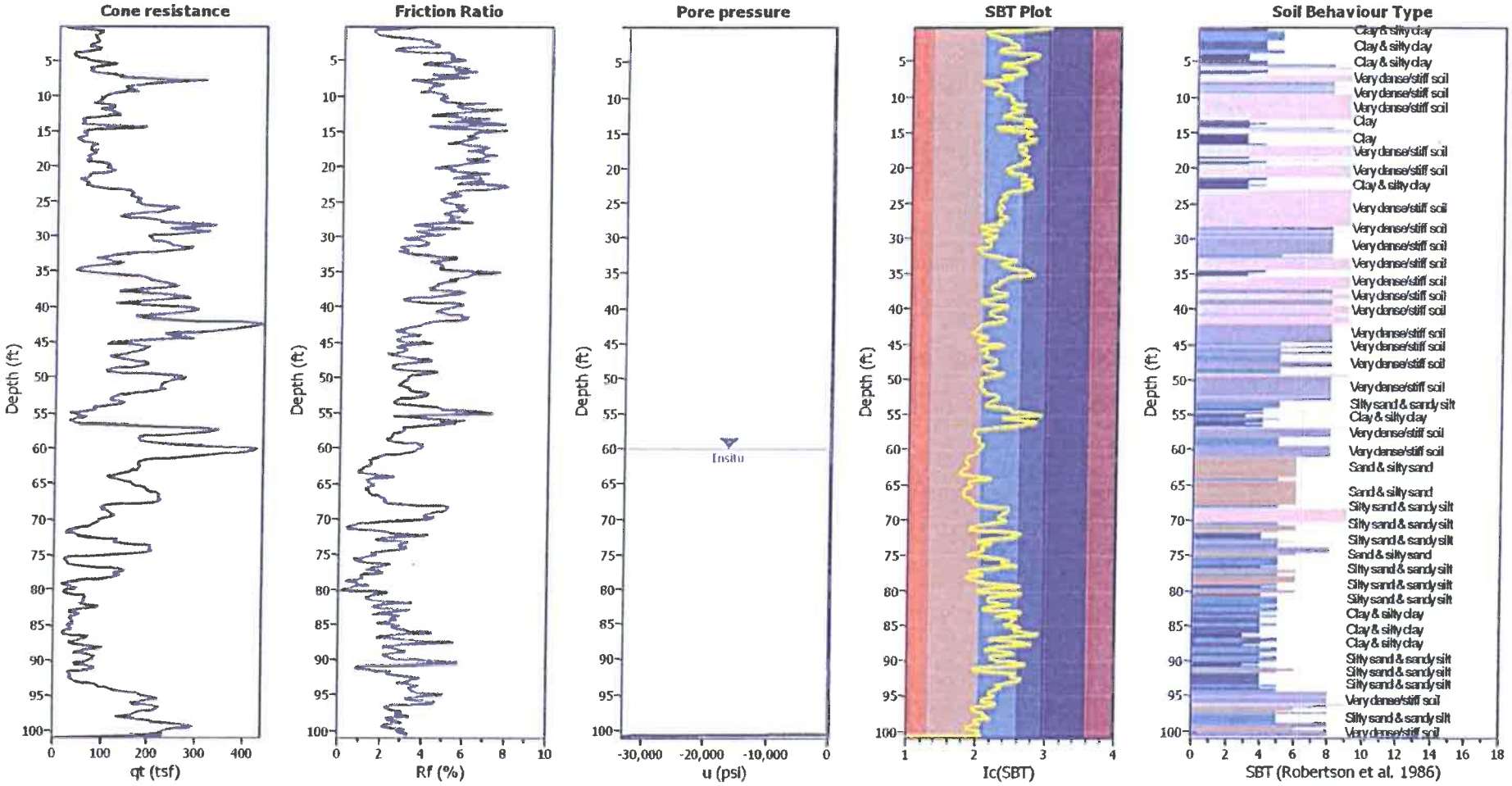
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots



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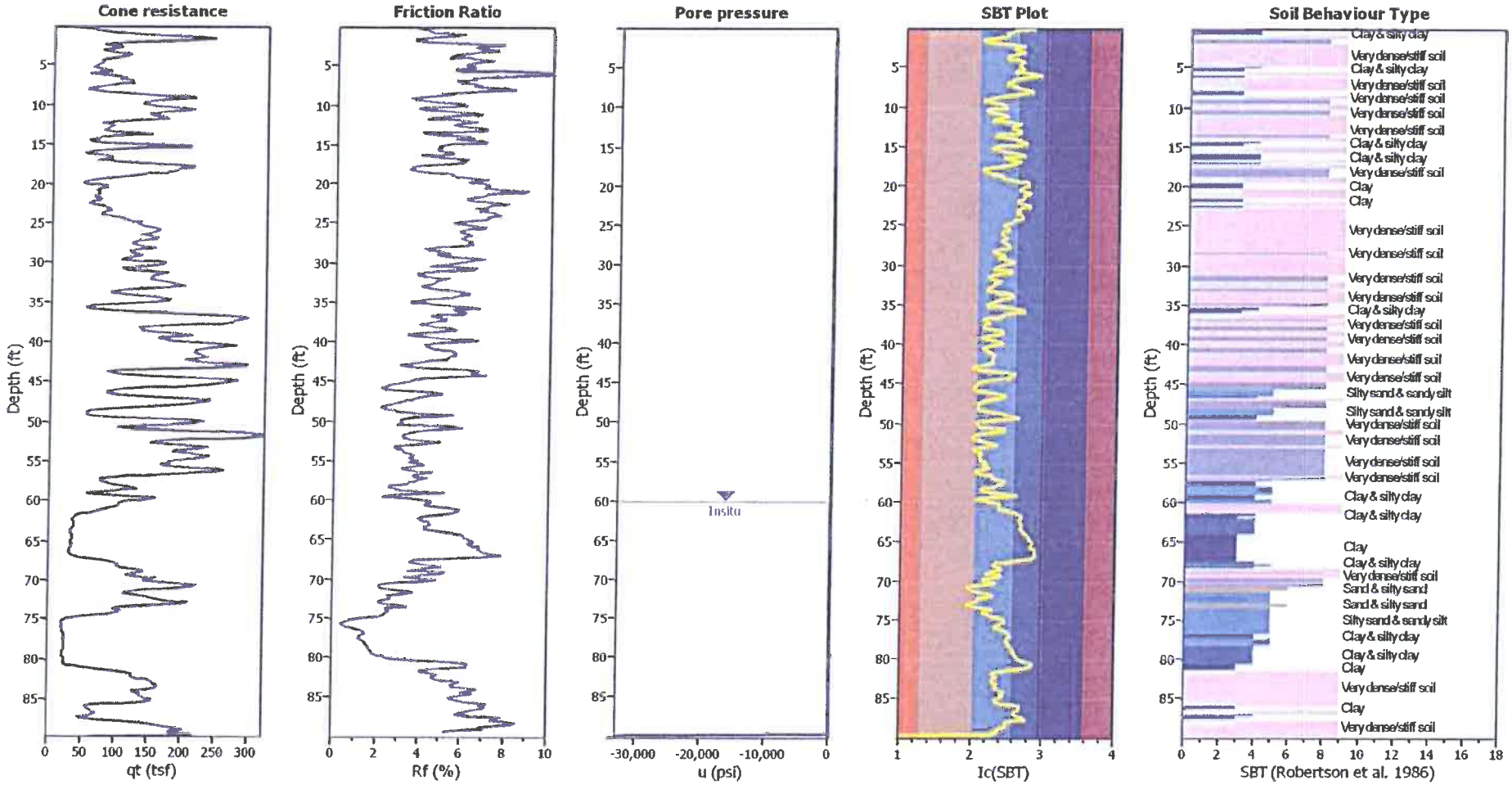
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots



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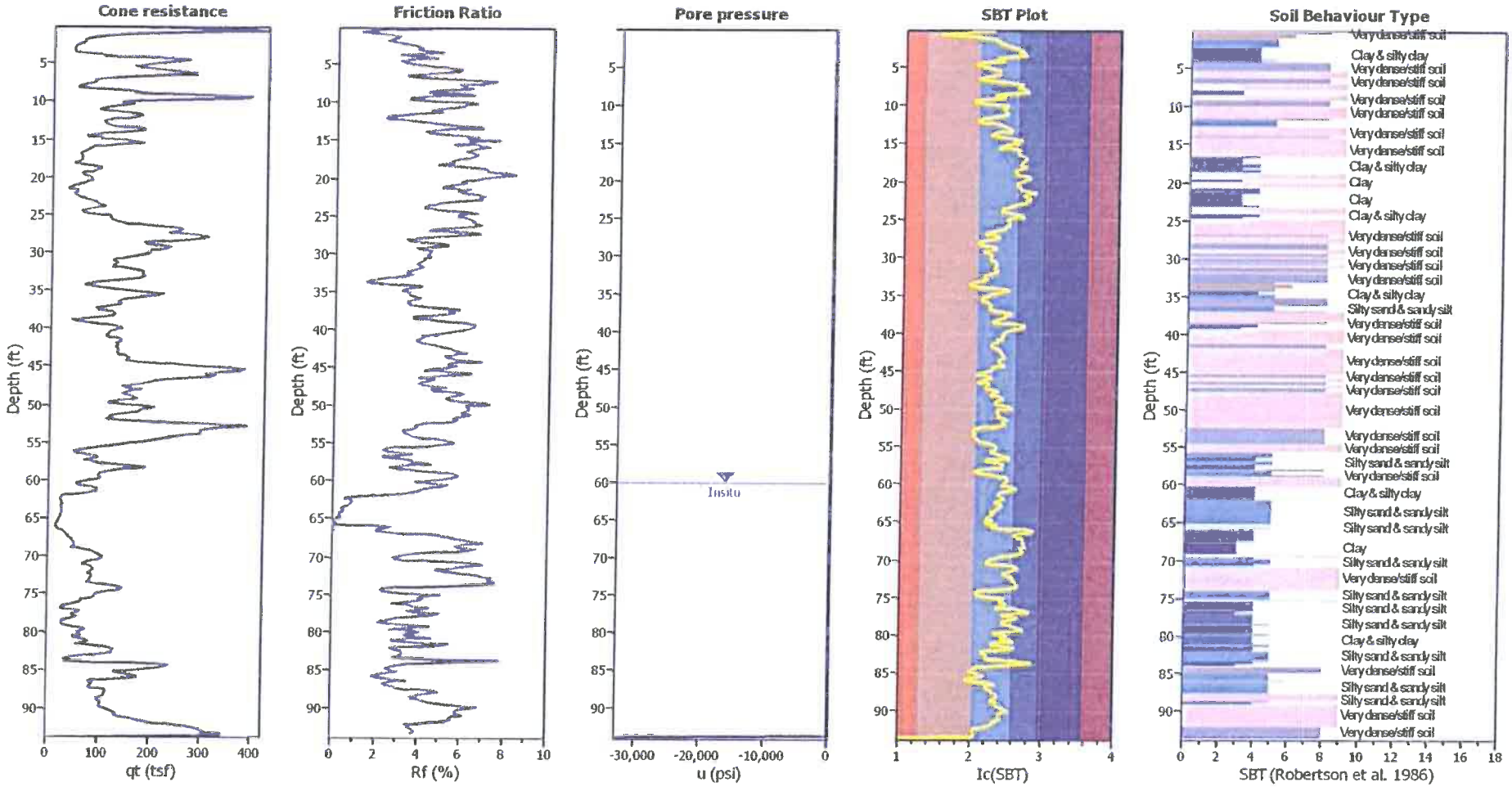
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots



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Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

DATE OBSERVED: 4-18-97

LOG OF BORING NO. B-1

LOCATION: See Plot Plan

LOGGED BY: BW METHOD OF DRILLING: Bucket Auger GROUND EL.: 591±

DRILLING CONTRACTOR: Ledesma HAMMER: See Below lbs. FALLING: 12 inches

Depth (ft.)	Soil Type	Sample Type		Blows/ Foot	Field Moist. (%)	Field Density (pcf)	Description
							This log is part of the report prepared by Bagahi Engineering and should be read together with the report for complete interpretation. The summary applies only at the location of this boring and at the time of drilling. The data contained in this log is a simplification of actual conditions encountered.
2-		B			11.4		FILL 0-3' Tan to yellowish tan silty SAND
4-			R	2	10.5	113.4	3-7' Tan to yellowish tan silty SAND intermixed with blocky greyish green siltstone fragments, moderate moisture and density
6-							
8-							7-8' Yellowish brown fine silty SAND
10-			R	3	19.0	106.5	
12-							8-18' Yellow to dark green (mottled) sandy SILT, moist
14-			R	2	19.1	98.0	
16-							
18-							18-22' Dark green clayey SILT with dark grey green siltstone fragments, moist, moderate to well densities
20-			R	5	21.1	106.9	
22-							22-23.5' Dark brown silty SAND with occasional organics, small rootlets, moist
24-			R	5	11.4	114.5	
26-							23.5-33' Light brown to yellowish tan silty SAND, moist
28-							33-34.5' Greyish green and brown mottled silty SAND
30-			R	4	6.5	119.2	
32-							34.5-45' Dark brown silty SAND with very occasional greyish green siltstone fragments, moist, dense
34-			R	5	10.0	124.0	
36-							
38-							
40-			R	4	10.3	120.8	
42-							
44-							

JOB NO.: 82a-200-00 Lot 11 FIGURE: B-4

DATE OBSERVED: 4-18-97

LOG OF BORING NO. B-1

LOCATION: See Plot Plan

LOGGED BY: BW

METHOD OF DRILLING: Bucket Auger

GROUND EL.: 591±

DRILLING CONTRACTOR: Ledesma

HAMMER: See Below lbs.

FALLING: 12 inches

Depth (ft.)	Soil Type	Sample Type		Blows/ Foot	Field Moist. (%)	Field Density (pcf)	Description
							This log is part of the report prepared by Bagahi Engineering and should be read together with the report for complete interpretation. The summary applies only at the location of this boring and at the time of drilling. The data contained in this log is a simplification of actual conditions encountered.
		B			10.6		
46-			R	9	10.7	125.7	45-55' Dark brown silty SAND to sandy SILT with occasional organic debris (ie-rootlets), exudes strong petrochemical odor @ 45-48'
48-							
50-			R	11	8.8	119.6	
52-							
54-							
			R	6	18.2	110.7	
56-							55' ALLUVIUM
58-							55-60' Tan to medium brown silty SAND, porous, moist
60-			R	2	25.6	99.9	60-62' Medium to dark brown sandy clayey SILT, porous, moist
62-							62-69' Dark brown to dark grey clayey SAND and sandy SILT, organic odor, very moist
64-							
			R	2	19.1	97.2	
66-							69-75' Dark grey to black sandy, clayey SILT, strong organic odor in some horizons (containing visible organic debris), very moist, free moisture on rings @ 70'
68-							
70-			R	2	24.1	101.9	@ 75' Ring sample saturated, dark grey silty SAND, barely stayed in sampler, attempted to continue drilling with smaller (18") bucket, no further progress due to caving and material running out of bottom of buckets.
72-							
74-							
			R	13	20.5	109.0	
76-							
78-							
80-							T.D. 75' Caving 72-75' Water 72' after drilling
82-							Kelly Weights see Boring B-2
84-							
86-							
88-							

JOB NO.: 82a-200-00 Lot 11

FIGURE: B-5

SHEET 2 OF 2

LOGS\log-82a

BAGAHI ENGINEERING

DATE OBSERVED: 4-18-97

LOG OF BORING NO. B-2

LOCATION: See Plot Plan

LOGGED BY: BW

METHOD OF DRILLING: Bucket Auger

GROUND EL.: 591±

DRILLING CONTRACTOR: Ledesma

HAMMER: See Below lbs.

FALLING: 12 inches

Depth (ft.)	Soil Type	Sample Type		Blows/ Foot	Field Moist. (%)	Field Density (pcf)	Description
							This log is part of the report prepared by Bagahi Engineering and should be read together with the report for complete interpretation. The summary applies only at the location of this boring and at the time of drilling. The data contained in this log is a simplification of actual conditions encountered.
2-		B			9.5		FILL 0-1.5' Tan silty SAND, very dry, medium dense
4-							1.5-27' Tan/orange/olive brown mottled sandy SILT to silty SAND with occasional greyish green siltstone fragments, moist, moderately dense to dense
6-			R	2	17.2	105.6	
8-							
10-			R	3	14.6	115.5	
12-							14' Orange silty SAND layer approx. 6" thick
14-							
16-			R	2	33.5	91.7	
18-							
20-			R	2	22.2	103.4	
22-							27-32' Dark grey and brown silty SAND, moist, dense
24-							
26-			R	6	15.4	117.3	
28-							
30-			R	2	13.2	119.7	32-42.5' Yellow/tan/dark brown mottled medium grained silty SAND, slightly moist to moist, moderately dense
32-							
34-							
36-			R	4	8.1	106.3	
38-							
40-			R	8	11.5	125.7	
42-							
44-							

JOB NO.: 82a-200-00 Lot 11

FIGURE: B-6

LOGS\log-82a

BAGAHI ENGINEERING

DATE OBSERVED: 4-18-97

LOG OF BORING NO. B-2

LOCATION: See Plot Plan

LOGGED BY: BW

METHOD OF DRILLING: Bucket Auger

GROUND EL.: 591±

DRILLING CONTRACTOR: Ledesma

HAMMER: See Below lbs.

FALLING: 12 inches

Depth (ft.)	Soil Type	Sample Type	Blows/ Foot	Field Moist. (%)	Field Density (pcf)	Description
						This log is part of the report prepared by Bagahi Engineering and should be read together with the report for complete interpretation. The summary applies only at the location of this boring and at the time of drilling. The data contained in this log is a simplification of actual conditions encountered.
46-		R	4	10.8	121.8	42.5-54' Dark brown silty SAND, moist, dense w/occasional organics
48-						
50-		R	7	6.3	114.8	54-59' Layer of organics, sticks, rootlets
52-						@ 59' Alluvium
54-		R	3	11.4	107.6	59-80' Medium brown silty SAND, slightly porous, moist to wet
56-						
58-						@ 65' Free water noted at bottom of hole, sample disturbed
60-		R	2	15.2	107.6	
62-						
64-						78-80' Black to dark grey silty SAND and clayey SAND, highly odorous (organics)
66-		R	2	24.4	–	
		R	2	20.6	115.9	
68-		R	3	21.6	107.3	
70-		R	5			Water at 63' after 10 minutes Drilling stopped @ 83' due to caving and material running through buckets (as in B-1)
72-						
74-		R	20			T.D. 83' Caving from 67-83'
76-						Kelly Weights: 0-24': 3548
78-						24-47': 2577 47-73': 1648
80-		R	12			73-95': 810
82-						
84-						
86-						
88-						

JOB NO.: 82a-200-00 Lot 11

FIGURE: B-7

SHEET 2 OF 2

BAGAHI ENGINEERING

LOGS\log-82a

APPENDIX C

FIELD EXPLORATION

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	DRILLER:	2K Drilling	LOGGED BY:	JMP
PROJECT NAME:	25192 Commercentter Drive	DRILL METHOD:	8" Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	1162-CR3	HAMMER:	Auto 140#/30"	RIG TYPE:	CME 75 (track rig)
LOCATION:	See Exploration Location Map			DATE:	4/23/2014

BORING NO.: B-1

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
			B1		**ARTIFICIAL FILL**			
		5 4 9	S1	SM/CL	@ 2.5': Silty fine SAND to silty CLAY interlayered, light brown to brown, slightly moist, medium dense/stiff.			
5		24 33 30	R1	SP/SM	@ 5': SAND to silty fine SAND, light brown, slightly moist, dense.	3.4	113.7	
		5 5 4	S2	SM	**ALLUVIUM** @ 7.5': Silty fine SAND, brown, moist, loose.			
10		14 24 39	R2	SM	@ 10': same as above, dense.	7.3	120.1	25.5% passing #200
		6 9 9	S3	SM	@ 12.5': same as above, medium dense			26.1% passing #200
15		15 16 19	R3	SM	@ 15': same as above, medium dense, trace clay.	12.4	121.3	31.9% passing #200
		5 7 6	S4	SM	@ 17.5': same as above, medium dense, trace clay.			24.9% passing #200
20		9 12 12	R4	SM	@ 20': same as above, medium dense, trace clay.	13.0	118.1	36.9% passing #200
		4 6 6	S5	SM-SC	@ 22.5': Silty clayey fine SAND, brown, moist, medium dense, one piece of gravel.	13.4		LL=30, PL=13, PI=17 46.2% passing #200
25		10 16 20	R5	SM-SC	@ 25': same as above.	15.2	117.3	31.4% passing #200
		4 5 8	S6	SM-SC/ CL	@ 27.5': Silty clayey fine SAND to CLAY interlayered, brown, moist to very moist, medium dense/stiff.			36.3% passing #200
30 ▽		5 7 9	R6	CL	@ 30': Fine sandy CLAY, brown, very moist, stiff, groundwater encountered at 30.8'.	24.1	104.5	LL=29, PL=15, PI=14

DRAFT

LEGEND

Sample type: Ring, SPT, Small Bulk, Large Bulk, No Recovery, Water Table

Lab testing: AL = Accerberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC = Consolidation; MD = Maximum Density

C.1

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	DRILLER:	2R Drilling	LOGGED BY:	JMP
PROJECT NAME:	25192 Commercenter Drive	DRILL METHOD:	8" Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	1162-CR3	HAMMER:	Auto 140#/30"	RIG TYPE:	CME 75 (truck rig)
LOCATION:	See Exploration Location Map			DATE:	4/23/2014

BORING NO.: B-1 (continued)

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					continued:			
		4 5 11	S7	SM	@ 32.5': Silty fine SAND, brownish gray, wet, medium dense.			30.3% passing #200
35		7 9 11	R7	SM	@ 35': same as above.	18.4	110.9	
		2 2 3	S8	SM	@ 37.5': Silty fine SAND, brownish gray, wet, loose.			19.9% passing #200
40		7 4 3	R8		@ 40': no recovery.			
		push 1 1	S9	CL	@ 42.5': Fine sandy CLAY, dark grayish brown, wet, very soft.	25.5		LL=28, PL=15, PI=13
45		push 4 4	R9	CL	@ 45': same as above, medium stiff.	26.0	101.0	
		1 2 2	S10	CL	@ 47.5': Silty fine to medium SAND, dark grayish brown, wet, loose.	23.7		33.9% passing #200
50		8 24 27	R10	SM	@ 50': Silty fine SAND, dark bluish gray, wet, dense.	17.3	114.5	
					BORING TERMINATED AT 51.5'			
					Groundwater encountered at 30.8' Boring backfilled with soil cuttings.			
55								
60								

DRAFT

LEGEND

Sample type: --Ring --SPT --Small Bulk --Large Bulk --No Recovery --Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC = Consolidation; MD = Maximum Density

C. 2

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	DRILLER:	2R Drilling	LOGGED BY:	JMP
PROJECT NAME:	25192 Commercenter Drive	DRILL METHOD:	8" Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	1162-CRJ	HAMMER:	Auto 140#/30"	RIG TYPE:	CME 75 (track rig)
LOCATION:	See Exploration Location Map			DATE:	4/23/2014

BORING NO.: B-2

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Laboratory Testing: Others
					ARTIFICIAL FILL			
		22 30 25	R1	SM/ML	@ 2.5': Silty fine SAND to fine sandy SILT interlayered, light brown to brown, slightly moist, dense/hard.			
5		7 9 15	S1	SM/ML	@ 5': same as above, medium dense/very stiff.			
		18 18 25	R2	SM	@ 7.5': Silty fine SAND, light brown, slightly moist, medium dense.	3.4	103.6	
10		6 8 8	S2	SM	**ALLUVIUM** @ 10': Silty fine SAND, light to medium brown, slightly moist, medium dense, uniform.			9.5% passing #200
		7 9 15	R3	SM	@ 12.5': same as above.	4.0	97.3	
15		4 5 5	S3	SM	@ 15': same as above, loose to medium dense.			18.7% passing #200
		8 16 23	R4	SM/ML	@ 17.5': Silty fine SAND to sandy clayey SILT, brown, moist, medium dense/very stiff.	7.7	116.5	LL=21, PL=18, PI=3 27.5% passing #200
20		6 9 10	S4	ML/CL	@ 20': Silty sandy CLAY to sandy clayey SILT, brown to reddish brown, moist, very stiff.	11.2		LL=24, PL=17, PI=7
		11 22 30	R5	ML/CL	@ 22.5': same as above, hard.	14.5	119.3	
25		4 6 7	S5	CL	@ 25': Silty sandy CLAY, brown, moist, stiff.	15.2		LL=30, PL=14, PI=16
		12 21 19	R6	SM/SC	@ 27.5': Silty clayey fine SAND, medium brown, very moist, medium dense.	14.1	115.4	22.2% passing #200
30		3 4 4	S6	SM	@ 30': Silty f-m SAND, brownish gray, wet, loose, groundwater encountered at 29.8'			27.3% passing #200

DRAFT

LEGEND

Sample type: Ring, SPT, Small Bulk, Large Bulk, No Recovery, Water Table

Lab testing: AL = Attarberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC = Consolidation; MD = Maximum Density

C. 3

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	DRILLER:	2R Drilling	LOGGED BY:	JMP
PROJECT NAME:	25192 Commercenter Drive	DRILL METHOD:	8" Hollow Stem	OPERATOR:	Jeff
PROJECT NO.:	1162-CR3	HAMMER:	Auto 140#/30"	RIG TYPE:	CME 75 (truck rig)
LOCATION:	See Exploration Location Map			DATE:	4/23/2014

BORING NO.: B-2 (continued)

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					continued:			
		4 4 6	R7	CL	@ 32.5': Sandy CLAY, brown, wet, medium stiff.	21.0	105.1	LL=31, PL=15, PI=16
35		push 2 1	S7	CL	@ 35': same as above, soft.	18.6		
		5 7 10	R8	CL	@ 37.5': same as above, stiff.	19.4	111	
40		2 3 3	S8	CL/SC	@ 40': Sandy CLAY to silty clayey fine SAND, brown to reddish brown, wet, medium stiff/loose, some gravels interlayered.	20.3		LL=33, PL=14, PI=19 41.0% passing #200
		20 42 50/4"	R9		BEDROCK-SESPE AND VAQUEROS FORMATIONS, undifferentiated @ 42.5': Silty fine SANDSTONE, reddish brown with green mottling, slightly moist to moist, hard.	17.9	113.8	
45		10 16 18	S9		@ 45': same as above, dense.			
		44 50/3"	R10		@ 47.5': SILTSTONE, reddish brown, slightly moist, hard.	14.3	123.4	
50					**BORING TERMINATED AT 48.3'** Groundwater encountered at 29.8'. Boring backfilled with soil cuttings.			
55								
60								

DRAFT

LEGEND

Sample type: Ring, SPT, Small Bulk, Large Bulk, No Recovery, Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC = Consolidation; MD = Maximum Density

C. 4

GREGG

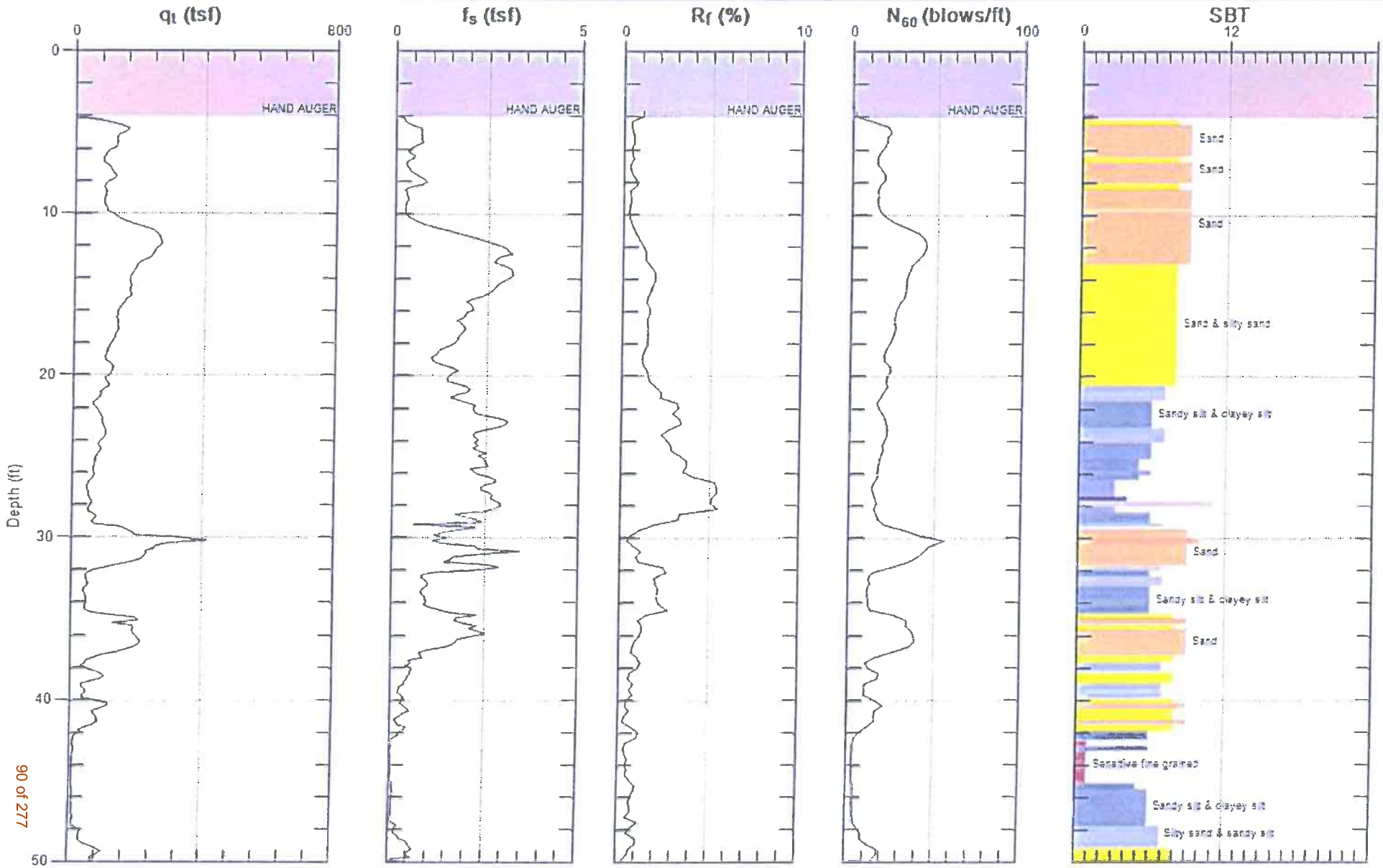
STONEY MILLER

Site: BDJ

Sounding: CPT-1

Engineer: Z.WANG

Date: 5/6/2014 09:10



Max. Depth: 50.033 (ft)

Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

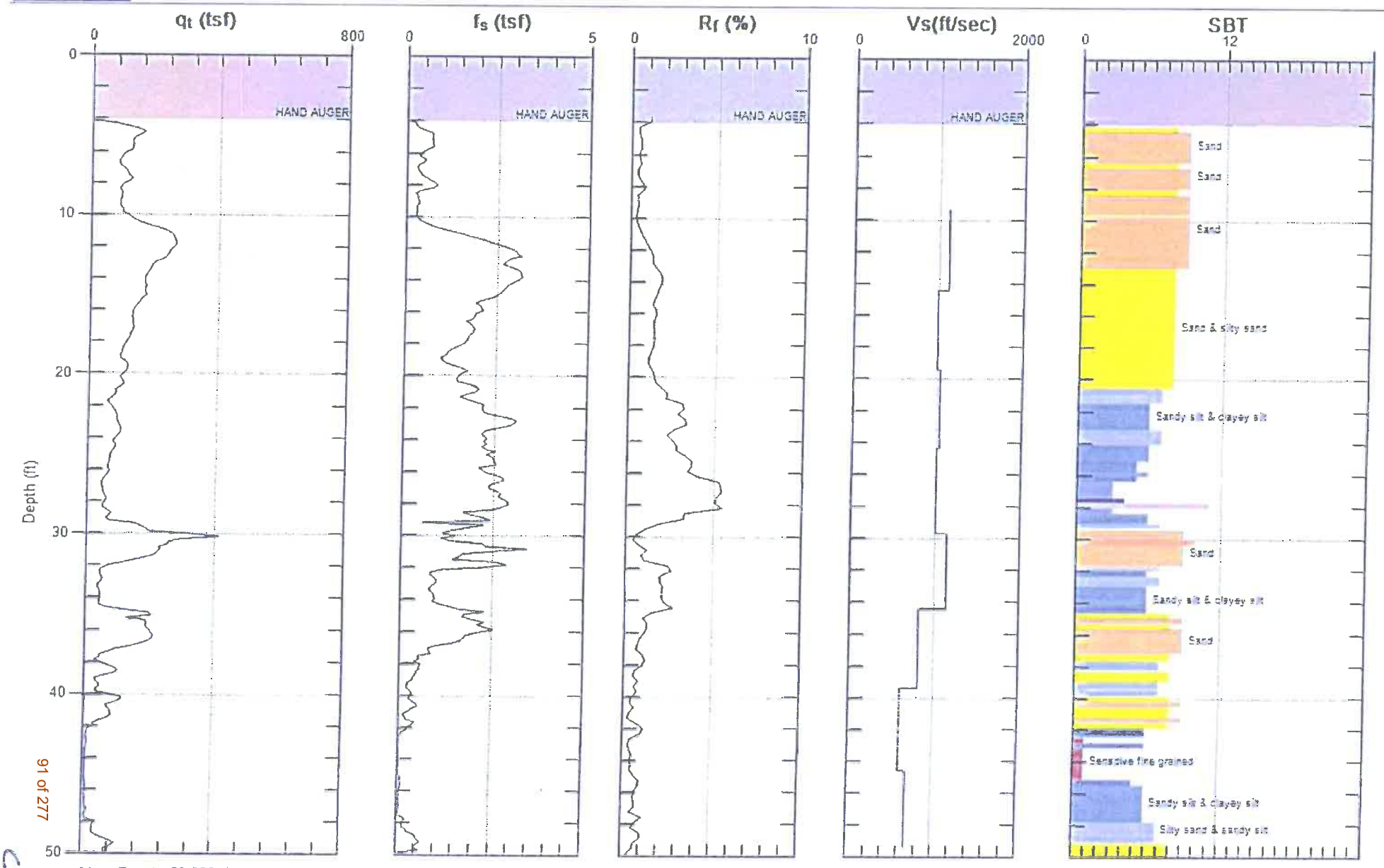
STONEY MILLER

Site: BDJ

Sounding: CPT-1

Engineer: Z.WANG

Date: 5/6/2014 09:10



Max. Depth: 50.033 (ft)

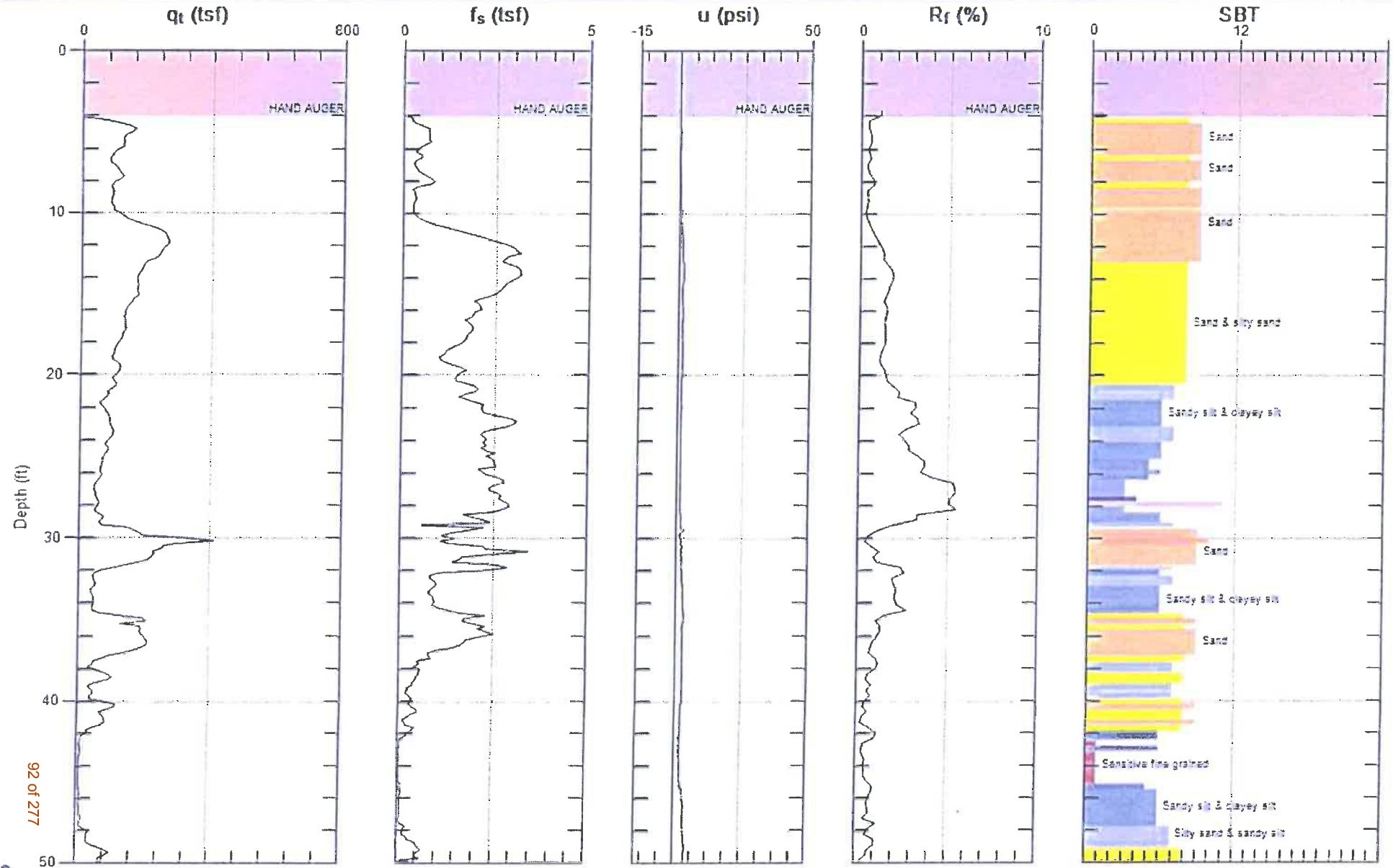
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

STONEY MILLER

Site: BDJ
Sounding: CPT-1

Engineer: Z.WANG
Date: 5/6/2014 09:10



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

SBT: Soil Behavior Type (Robertson 1990)

GREGG

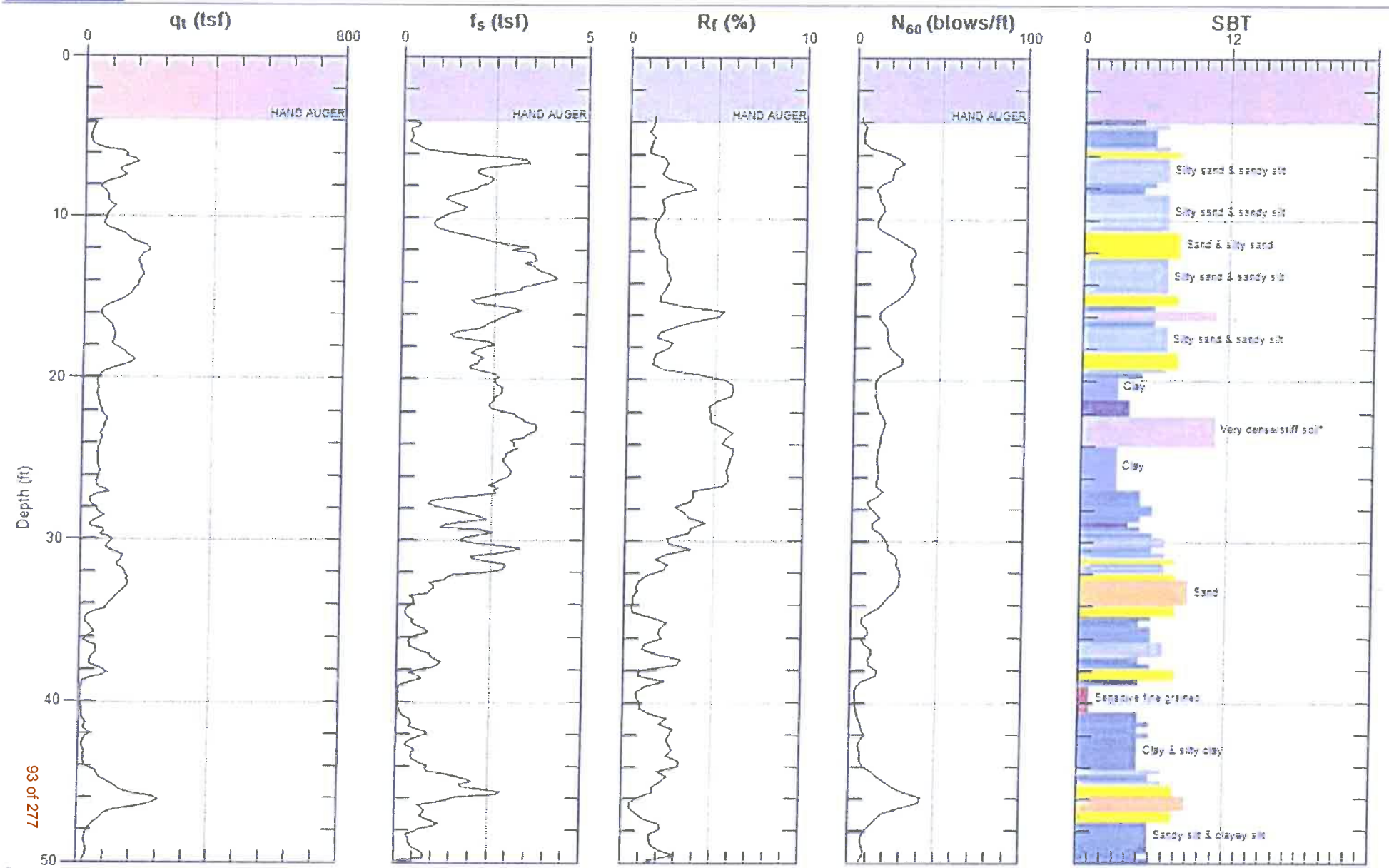
STONEY MILLER

Site: BDJ

Sounding: CPT-2

Engineer: Z.WANG

Date: 5/6/2014 11:19



Max. Depth: 50.033 (ft)

Avg. Interval: 0.326 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

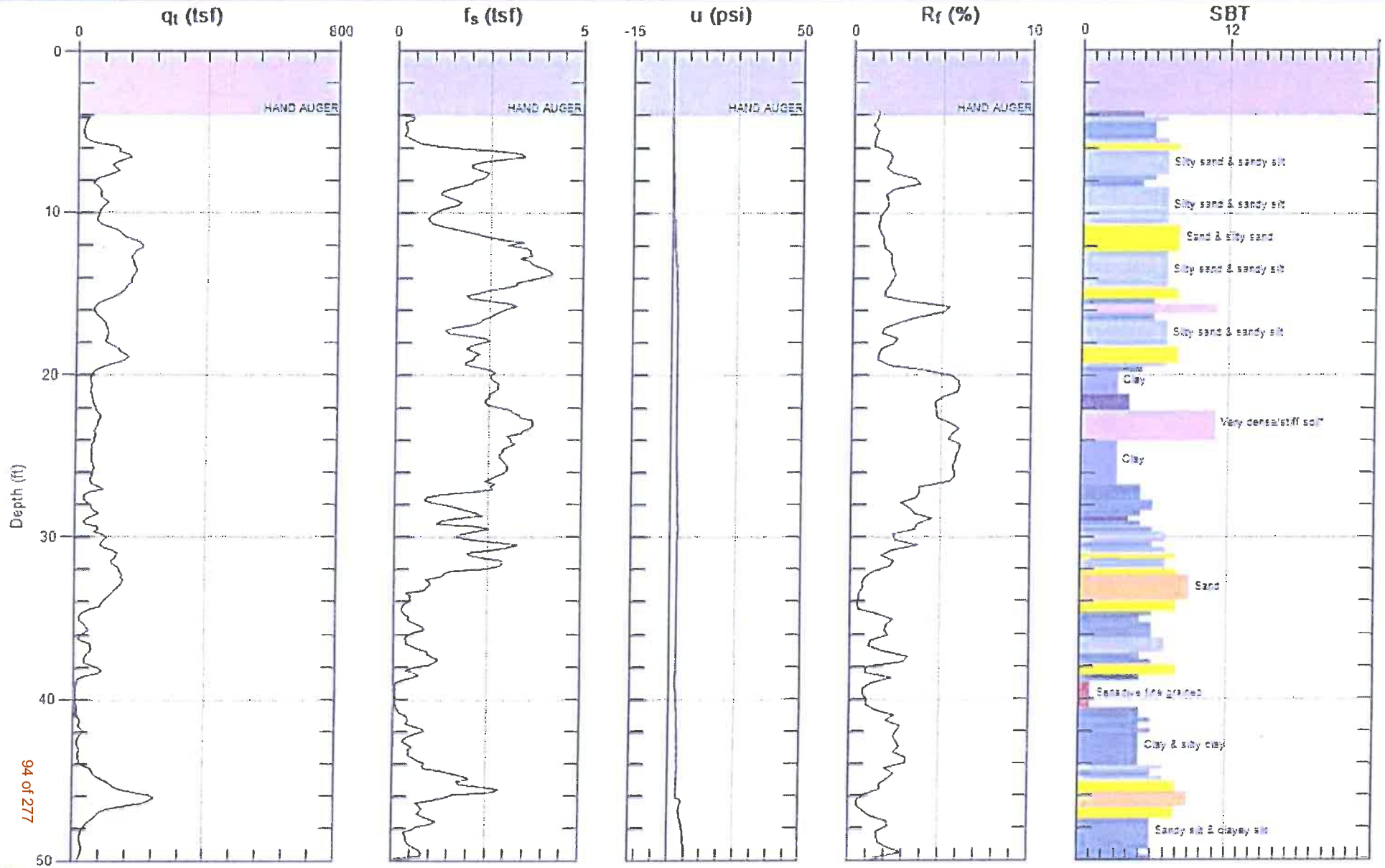
STONEY MILLER

Site: BDJ

Sounding: CPT-2

Engineer: Z.WANG

Date: 5/6/2014 11:19



Max. Depth: 50.033 (ft)

Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Shear Wave Velocity Calculations

BDJ
CPT-1

Geophone Offset: 0.66 Feet
Source Offset: 8.17 Feet

05/06/14

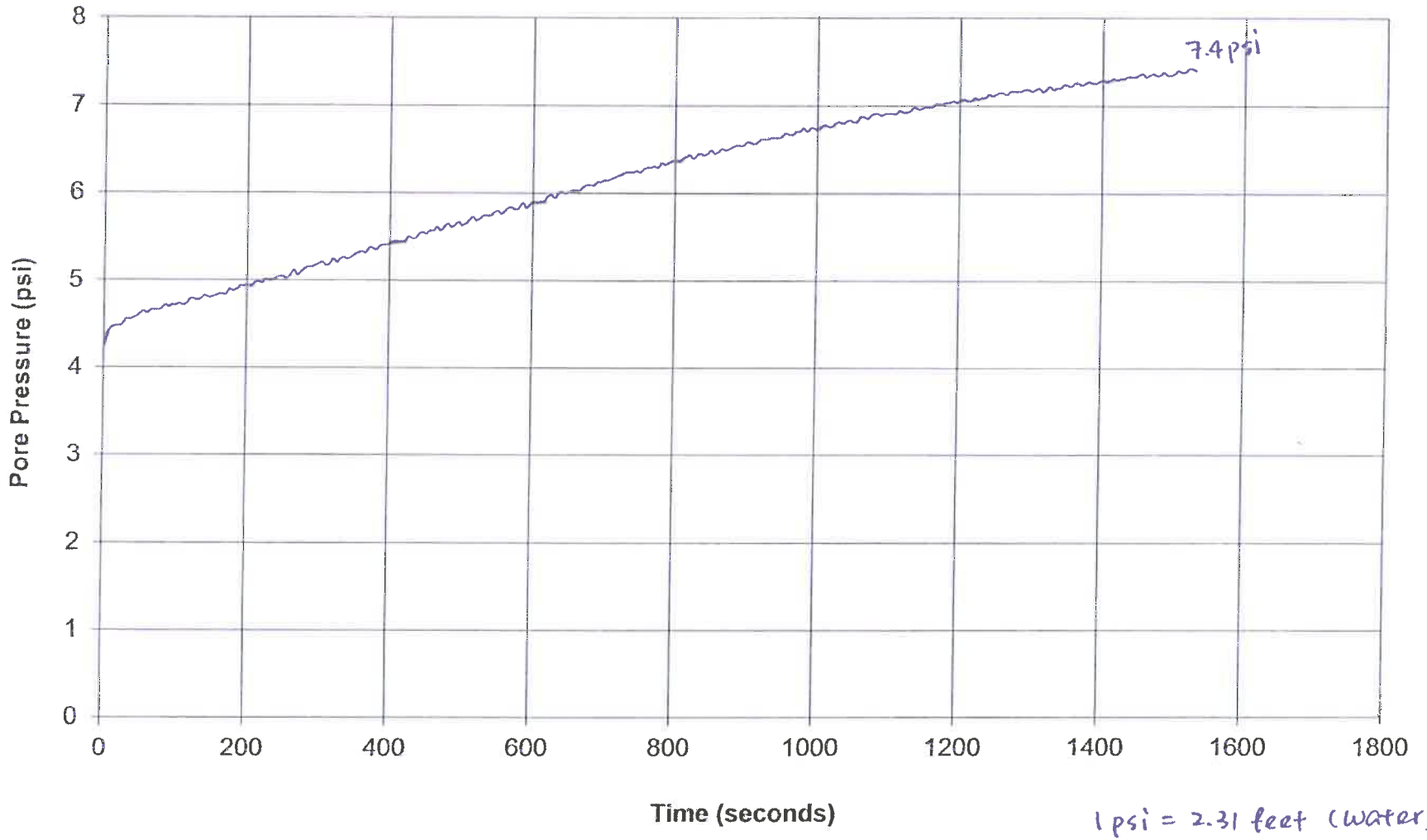
Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	12.41	12.41	16.9500			
15.09	14.43	16.58	4.17	20.7000	3.7500	1112.1	11.89
20.01	19.35	21.01	4.42	25.2000	4.5000	983.0	16.89
25.10	24.44	25.77	4.76	29.8500	4.6500	1023.9	21.90
30.35	29.69	30.79	5.02	34.9000	5.0500	994.8	27.06
35.10	34.44	35.40	4.61	38.9500	4.0500	1138.1	32.07
40.03	39.37	40.20	4.80	44.8500	5.9000	814.3	36.91
45.28	44.62	45.36	5.15	53.4000	8.5500	602.6	41.99
50.03	49.37	50.04	4.69	60.2000	6.8000	689.2	46.99

C.10

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-1
Depth: 50.0326575
Site: BDJ
Engineer: Z.WANG



1 psi = 2.31 feet (water)

Hwater = 7.4 x 2.31 = 17 ft

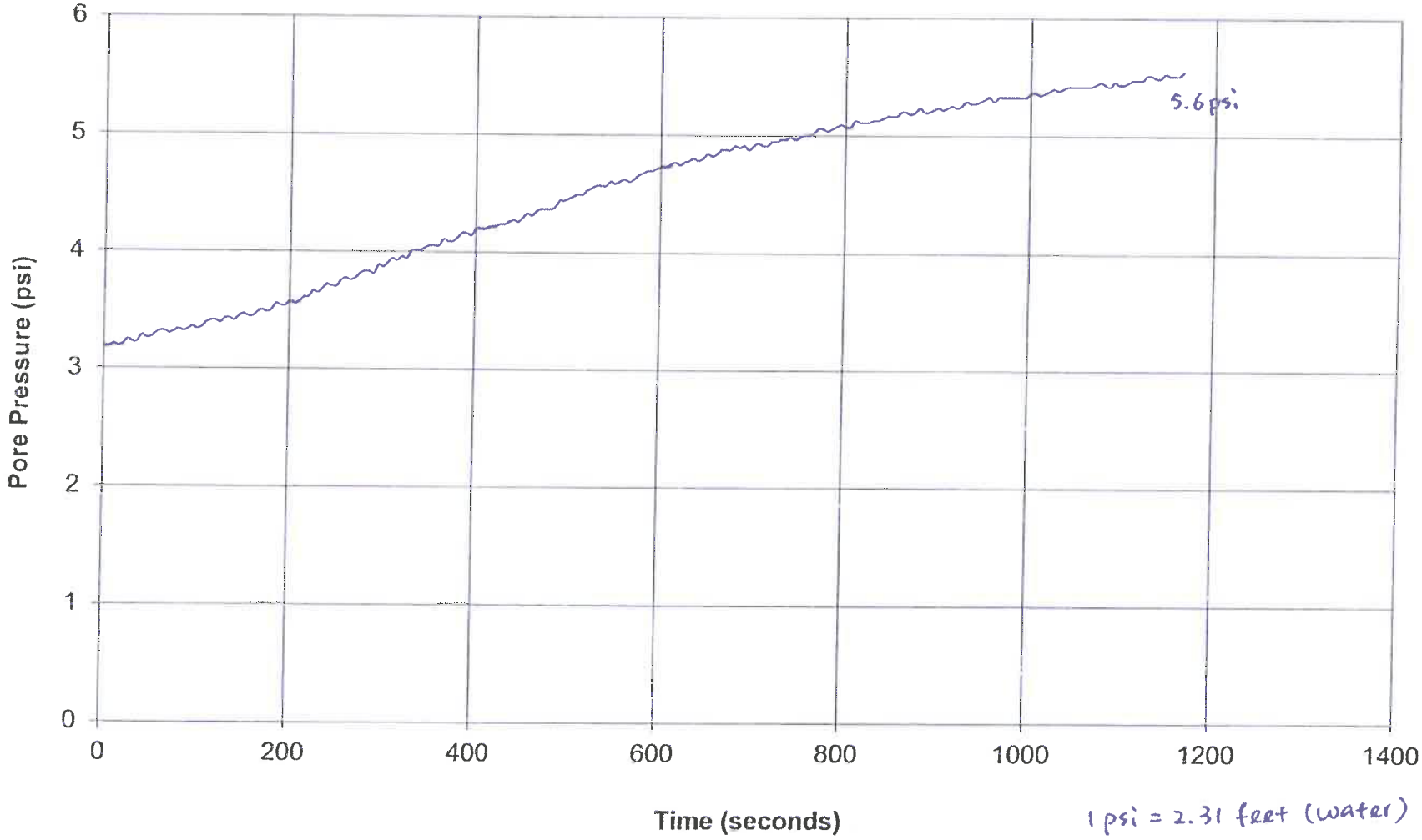
Dwater = Dcone - Hwater
= 50 - 17 = 33 ft

C.1)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-2
Depth: 46.0956615
Site: BDJ
Engineer: Z.WANG



1 psi = 2.31 feet (water)

Hwater = 2.31 × 5.6 = 12.9 ft

Dwater = 46 − 13 = 33 ft

C.12

APPENDIX D

SEISMICITY BACKGROUND

APPENDIX D

SEISMICITY BACKGROUND

The shear wave velocity of the upper 50 feet of alluvium at the toe of slope ranged from 603 to 1138 feet per second (Figure D.1). The shear wave velocity of the engineered fill soil and bedrock are estimated at 1,000 and 2,000 feet per second, respectively. The site shear wave velocity, Vs(100'), of 986 to 1,077 feet per second (300 to 328 m/sec) is determined in Figure D.2 using the procedure from CBC 1613.5.5.

The target site response spectra was developed using https://geohazards.usgs.gov/deaggint/2008/, the USGS 2008 Interactive Deaggregation website. The spectra was developed using a site shear wave velocity of 300 meters per second, a return period of 2,475 years, and a site location of 33.645 latitude and -117.685 longitude. The results of these analyses are provided in Figures D.3 through D.5 which are output from the website. The site shear wave velocity of 300 meters/second is listed on the lower footer on Figures D.3 through D.5.

The peak ground velocity and mean ground motion period may be used to aid in estimation of slope deformation during earthquake ground motions. Using an earthquake magnitude of 7.0 and peak ground acceleration of 0.6g, Figure D.6 provides an empirical peak ground velocity of 31.56 inches per second (80.15 cm/sec). Using an earthquake magnitude of 7.0 and fault rupture distance of 3.8 km, Figure D.7 provides an empirical mean ground motion period of 0.467 seconds.

As an assist for use of ASCE 7-10 and the CBC, the USGS website was used to develop design data for use by the geotechnical engineer and structural engineer in seismic analyses. This data is included herein as Figures D.8 through D.13.

Shear Wave Velocity Calculations

BDJ
CPT-1

Geophone Offset: 0.66 Feet
Source Offset: 8.17 Feet

05/06/14

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	12.41	12.41	16.9500			
15.09	14.43	16.58	4.17	20.7000	3.7500	1112.1	11.89
20.01	19.35	21.01	4.42	25.2000	4.5000	983.0	16.89
25.10	24.44	25.77	4.76	29.8500	4.6500	1023.9	21.90
30.35	29.69	30.79	5.02	34.9000	5.0500	994.8	27.06
35.10	34.44	35.40	4.61	38.9500	4.0500	1138.1	32.07
40.03	39.37	40.20	4.80	44.8500	5.9000	814.3	36.91
45.28	44.62	45.36	5.15	53.4000	8.5500	602.6	41.99
50.03	49.37	50.04	4.69	60.2000	6.8000	689.2	46.99

D.1

CBC SECTION 1613.5.5: SITE CLASSIFICATION USING SHEARWAVE VELOCITY PROCEDURE

TOE OF SLOPE

EARTH MATERIAL	TEST DEPTH (FEET)	Vi SHEAR WAVE VELOCITY (FT/SEC)	INCREMENT BEGIN (FEET)	END (FEET)	di (FEET)	di/Vi
FILL	0-10	1000	0	10	10	0.0100
QAL	10-30	1000	10	30	20	0.0200
QAL	30-60	700	30	60	30	0.0429
BEDROCK	60-100	2000	60	100	40	0.0200
				SUM =	100	0.0929
				Vi(100) =	**1077**	**FT/SEC**
					328	**M/SEC**

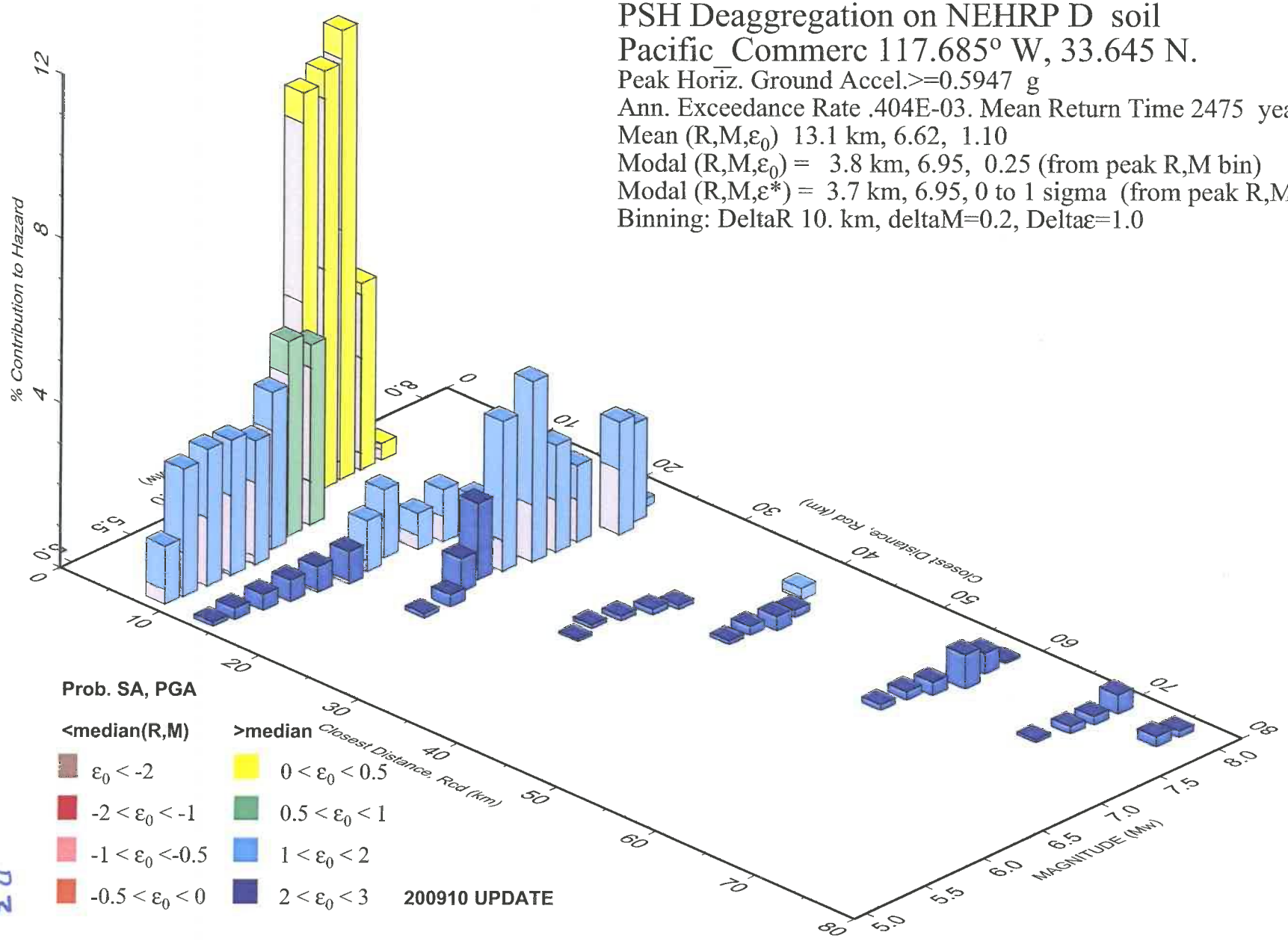
TOP OF SLOPE

EARTH MATERIAL	TEST DEPTH (FEET)	Vi SHEAR WAVE VELOCITY (FT/SEC)	INCREMENT BEGIN (FEET)	END (FEET)	di (FEET)	di/Vi
FILL	0-55	1000	0	55	55	0.0550
QAL	55-75	1000	55	75	20	0.0200
QAL	75-90	700	75	90	15	0.0214
BEDROCK	90-100	2000	90	100	10	0.0050
				SUM =	100	0.1014
				Vi(100) =	**986**	**FT/SEC**
					300	**M/SEC**

D.2.

PSH Deaggregation on NEHRP D soil
Pacific_Commerc 117.685° W, 33.645 N.

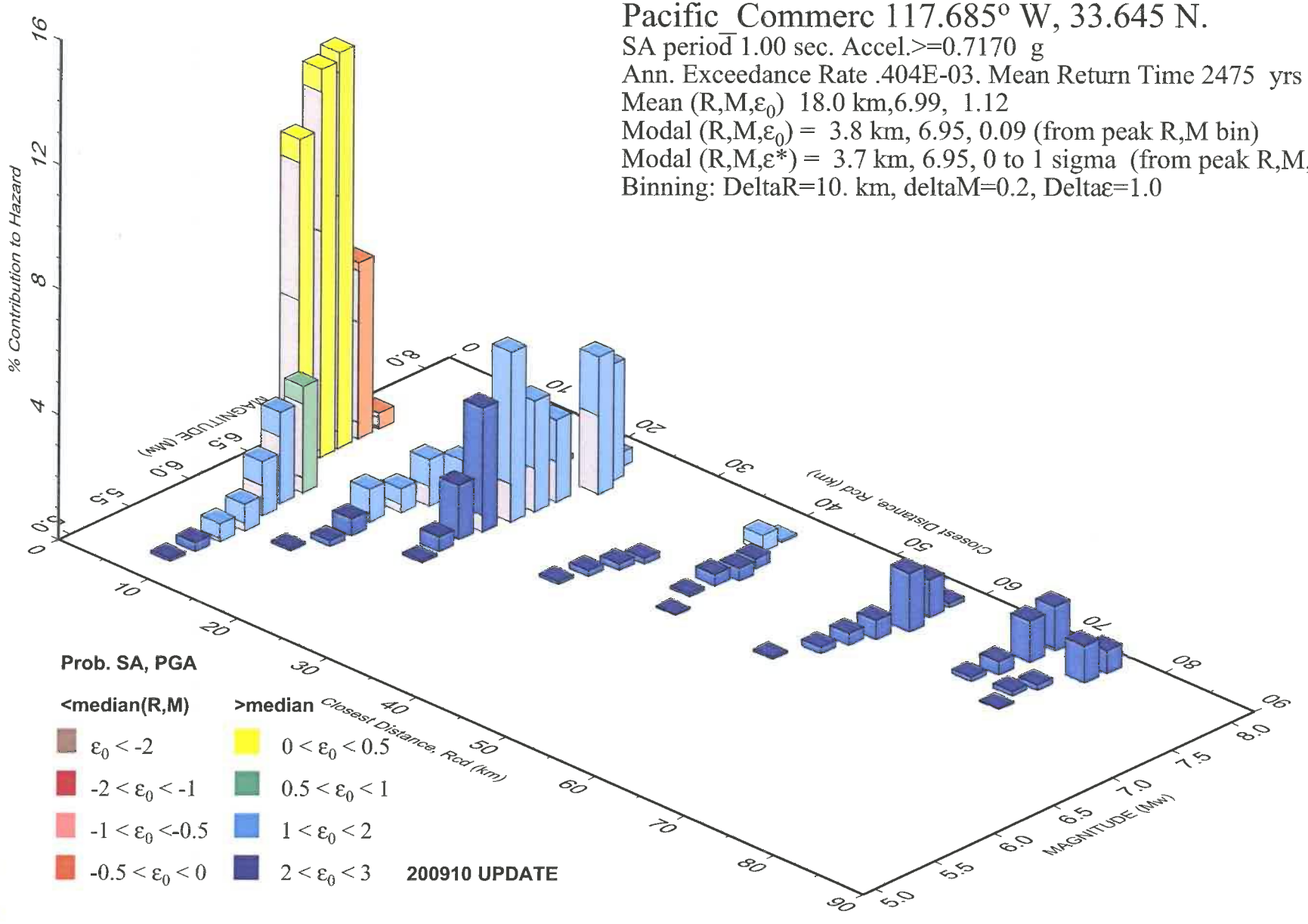
Peak Horiz. Ground Accel.>=0.5947 g
Ann. Exceedance Rate .404E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 13.1 km, 6.62, 1.10
Modal (R,M,ε_0) = 3.8 km, 6.95, 0.25 (from peak R,M bin)
Modal (R,M,ε^*) = 3.7 km, 6.95, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2014 May 15 23:12:10 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 300. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
Pacific_Commerc 117.685° W, 33.645 N.

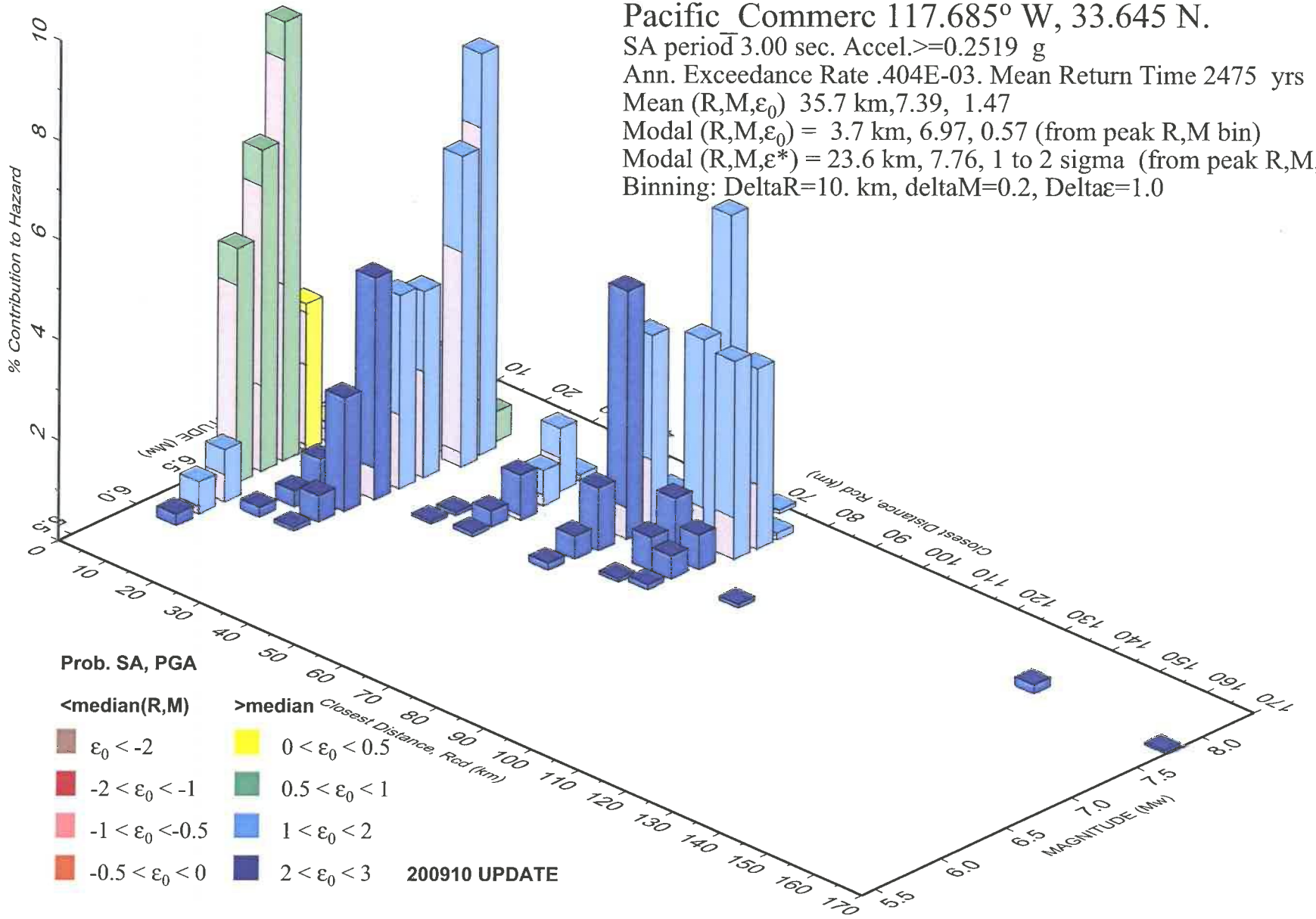
SA period 1.00 sec. Accel.>=0.7170 g
Ann. Exceedance Rate .404E-03. Mean Return Time 2475 yrs
Mean (R,M,ε_0) 18.0 km,6.99, 1.12
Modal (R,M,ε_0) = 3.8 km, 6.95, 0.09 (from peak R,M bin)
Modal (R,M,ε^*) = 3.7 km, 6.95, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2014 May 15 23:13:42 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 300. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
Pacific_Commerc 117.685° W, 33.645 N.

SA period 3.00 sec. Accel.>=0.2519 g
Ann. Exceedance Rate .404E-03. Mean Return Time 2475 yrs
Mean (R,M,ε_0) 35.7 km,7.39, 1.47
Modal (R,M,ε_0) = 3.7 km, 6.97, 0.57 (from peak R,M bin)
Modal (R,M,ε^*) = 23.6 km, 7.76, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2014 May 15 23:14:23 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 300. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

Empirical Estimate of Earthquake Peak Ground Velocity

Reference: "Selection of Ground Motion Time Series and Limits on Scaling", Soil Dynamics and Earthquake Engineering, Vol.26, pp.477-482

Input Parameters	
Moment Magnitude (M_w)	7.00
Peak Ground Acceleration (g)	0.60
Spectral Acceleration at 1 sec Period (g)	0.72
Spectral Acceleration at 3 sec Period (g)	0.25
Number of Standard Deviations ε	0.00

Result	
PGV (cm/sec)	80.15
PGV (in/sec)	31.56

Empirical Estimate of Mean Period, Average Spectral Period, and Predominant Period

Reference: "Empirical Relationships for Frequency Content Parameters of Earthquake Ground Motions", Earthquake Spectra, Vol.20, pp.119-144

Input Parameters		
Moment Magnitude (M_w)	7.00	
Closest Distance to Fault Rupture (km)	3.80	
Site Class	D	
Site Class Indicator Sc	0.00	0 OR 1
Site Class Indicator S_D	0.00	0 OR 1
Fault Forward Directivity FD	0.00	
Number of Standard Deviations ε	0.00	

FD=1 for sites with M ≥ 6.0, R ≤ 20.0 km, azimuth angle ≤ 30 deg, and rupture length ratio ≥ 0.5.

FD=0 otherwise.

Tm		Tave		To	
c11	-1.00	c21	-0.89	c31	-1.78
c12	0.18	c22	0.29	c32	0.30
c13	0.0038	c23	0.0030	c33	0.0045
c14	0.078	c24	0.070	c34	0.150
c15	0.27	c25	0.25	c35	0.33
c16	0.40	c26	0.37	c36	0.24
σ1b	0.42	σ2b	0.38	σ3b	0.38
σ1c	0.38	σ2c	0.36	σ3c	0.33
σ1d	0.31	σ2d	0.29	σ3d	0.31
τ1	0.17	τ2	0.13	τ3	0.22

Mean Period	SD =	0.000	lnTm=	-0.761	Tm=	0.467	sec
Average Spectral Period	SD =	0.000	lnTave=	-0.516	Tave=	0.597	sec
Smoothed Spectral Predominant Period	SD =	0.000	lnTo=	-1.388	To=	0.250	sec

D.1

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.645°N, 117.685°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 = 1.447 g
From Figure 22-2 [2]	S_1 = 0.540 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	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 = 1.447 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	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 = 0.540 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_a S_S = 1.000 \times 1.447 = 1.447$ g

Equation (11.4–2): $S_{M1} = F_v S_1 = 1.500 \times 0.540 = 0.810$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

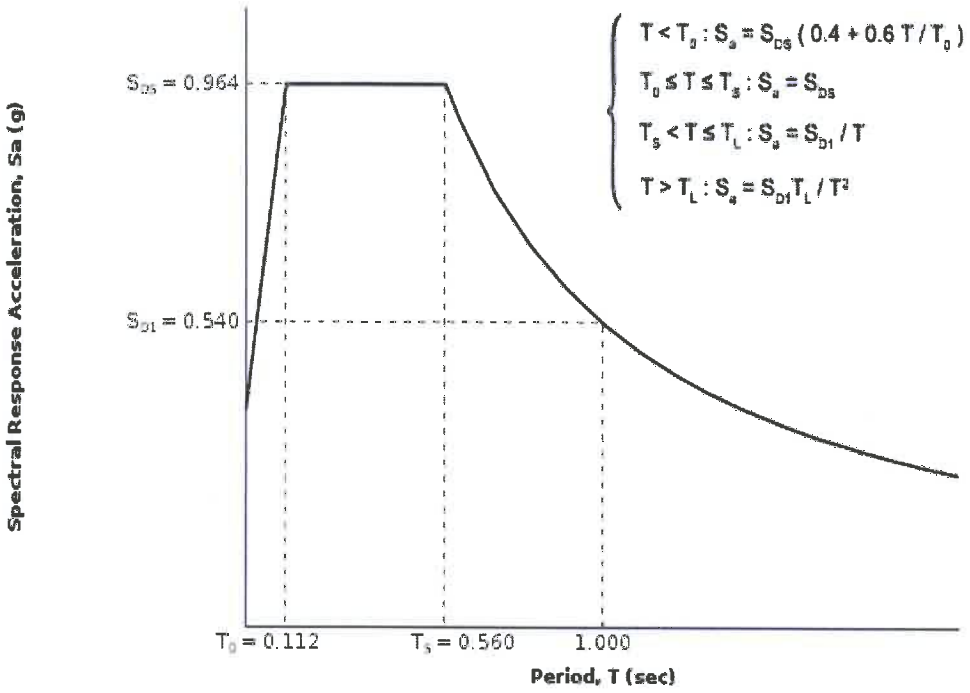
Equation (11.4–3): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 1.447 = 0.964$ g

Equation (11.4–4): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.810 = 0.540$ g

Section 11.4.5 — Design Response Spectrum

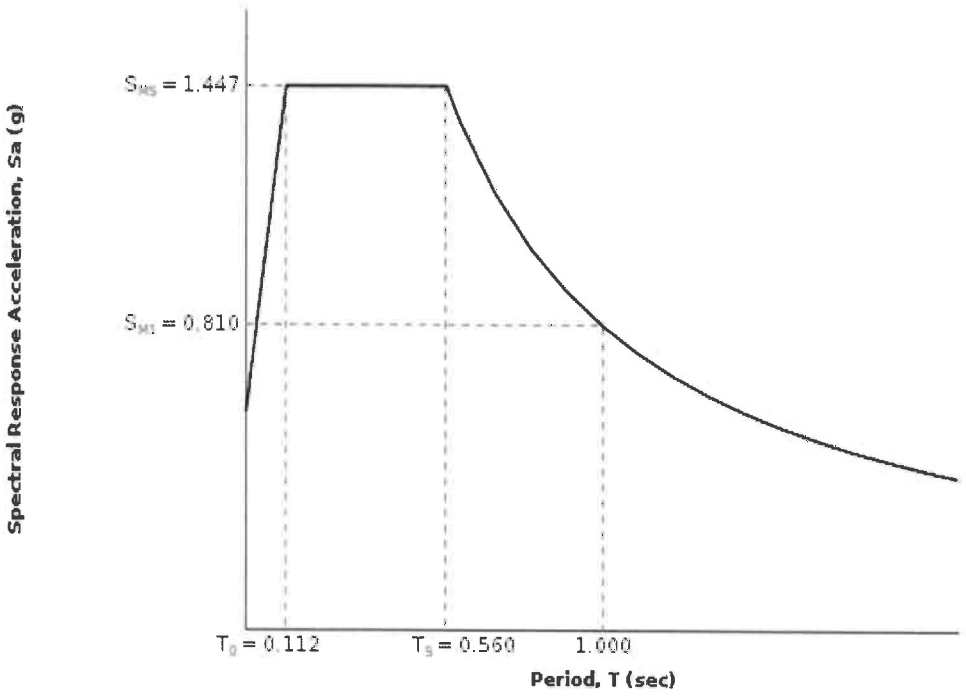
From Figure 22-12 [3] $T_L = 8$ seconds

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 = 0.538

Equation (11.8–1): $PGA_M = F_{PGA} PGA = 1.000 \times 0.538 = 0.538$ 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 = 0.538 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} = 1.012

From Figure 22-18 [6] C_{R1} = 1.045

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
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.964 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
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.540 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" = D

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

D.13

APPENDIX E

LIQUEFACTION ANALYSES

APPENDIX E

LIQUEFACTION ANALYSES

Liquefaction analyses were made for the CPT-01 to CPT-04 (2005), GT Borings B-1 and B-2, and CPT-1 and CPT-2 (2014). CPTs were evaluated using computer program CLiq version 1.7.5.27 and boring data was analyzed using the computer program LiquefyPro by CivilSoft. Analyses were made conservatively using a maximum considered earthquake with a magnitude M7.0 earthquake and a peak ground acceleration of 0.6g. The analyses are all included in Appendix E.

All evaluations of the CPTs and borings show that some of the alluvium may liquefy during the maximum considered earthquake. However, the liquefiable layers tend to be relatively thin and discontinuous. The CLiq program provides estimates of ground settlement that are modified using the method of Cetin, et al. (2009) which linearly weights the ground strains with depth from 1.0 at the surface to 0.0 at a depth of 50 feet.

The liquefaction results for CPT-01through CPT-04 are provided on Figures E1 through E.36. In summary, the effects of liquefaction below the building pad area are nil. This is due to the Cetin et al. procedure with the alluvium depths more than 50 feet below grade.

The liquefaction results for CPT-1 and CPT-2 are given on Figures E.37 through E.53. In summary, the effects of liquefaction at the toe of slope area are minor with respect to ground settlement of 0.8 to 1.7 inches with lateral spreading conservatively estimated at 12 to 24 inches. Seismic slope stability will need to be estimated using estimated residual strengths of the alluvium after liquefaction has occurred. Then slope deformation potential can be estimated using available empirical procedures.

The liquefaction results for Borings B-1 and B-2 are given on Figures E.54 through E.61. In summary, the effects of liquefaction at the toe of slope area are minor with respect to ground settlement of 3.7 to 8.3 inches. If the procedure of Cetin, et al was used here, the settlement range would be halved to approximately 2 to 4 inches. This should not be a concern for the slope area.

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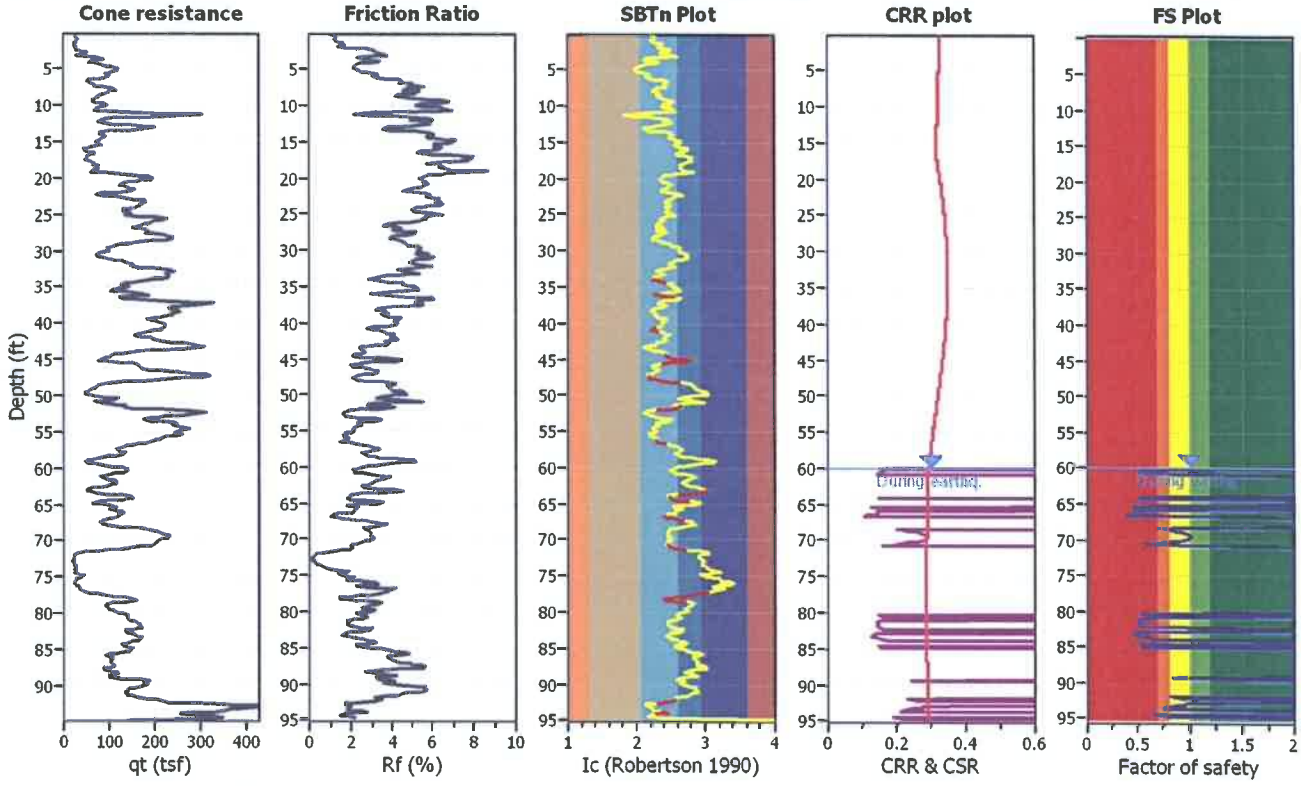
LIQUEFACTION ANALYSIS REPORT

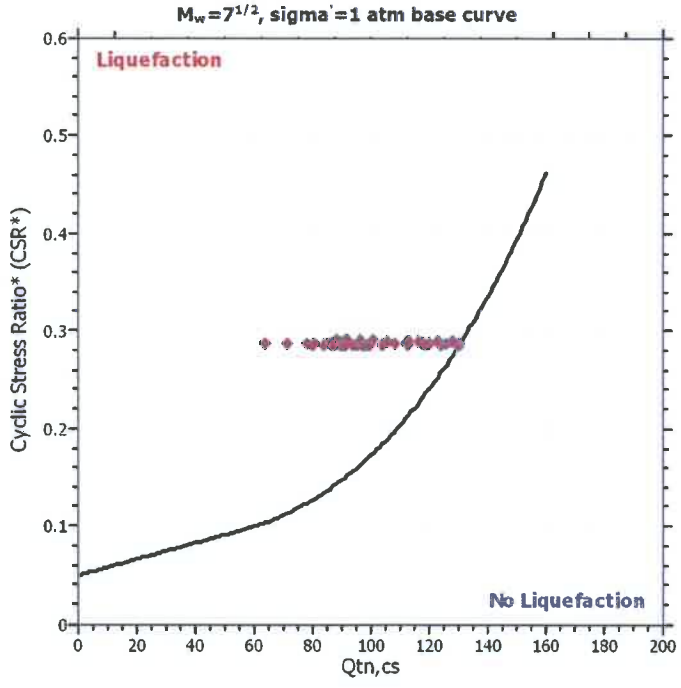
Project title : Pacific Commercenter Lot 13 **Location : Lake Forest, CA**

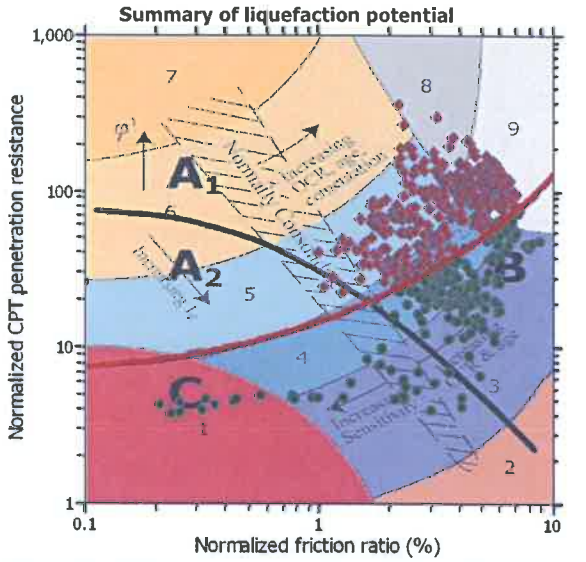
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	60.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

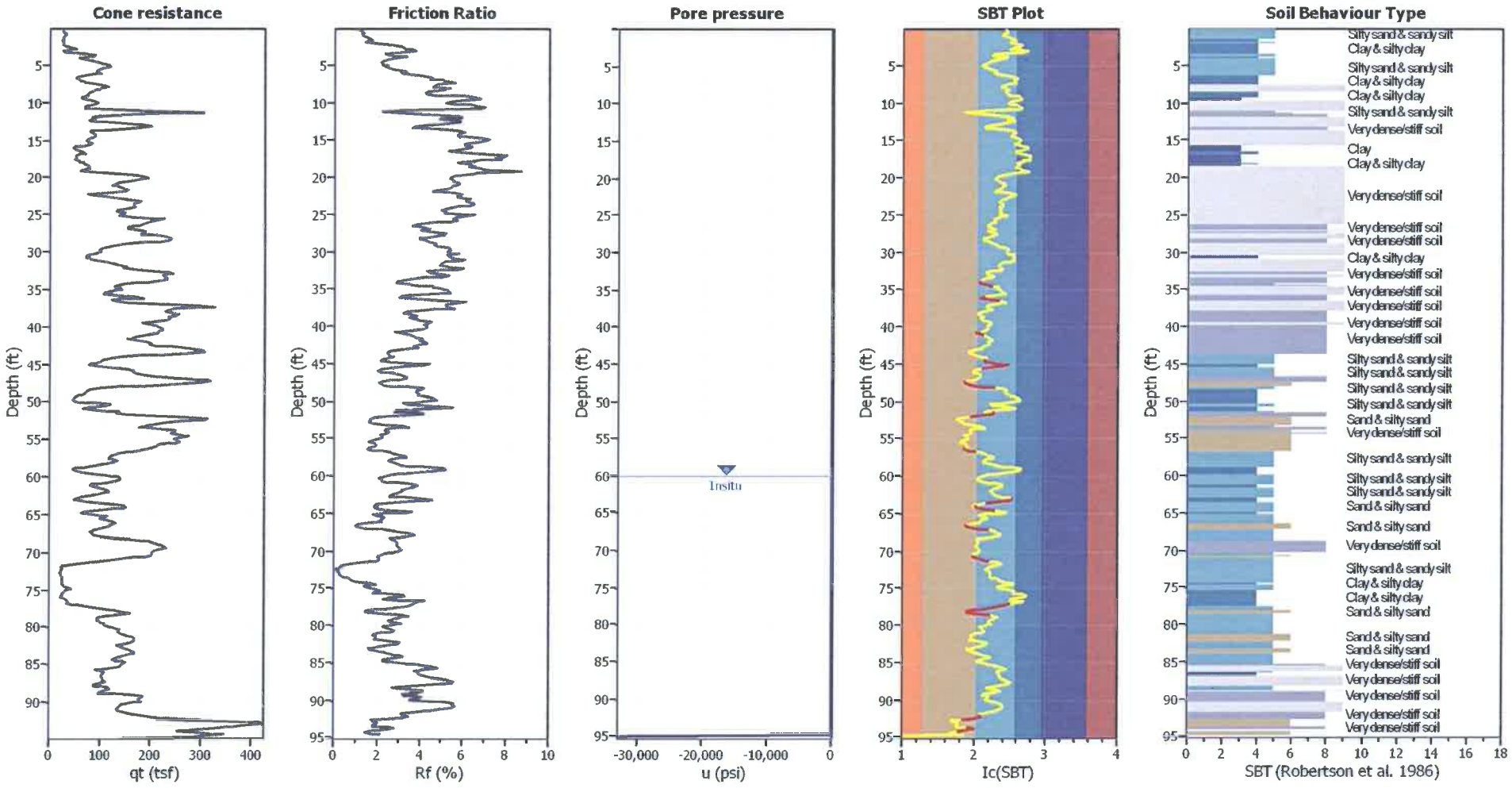






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

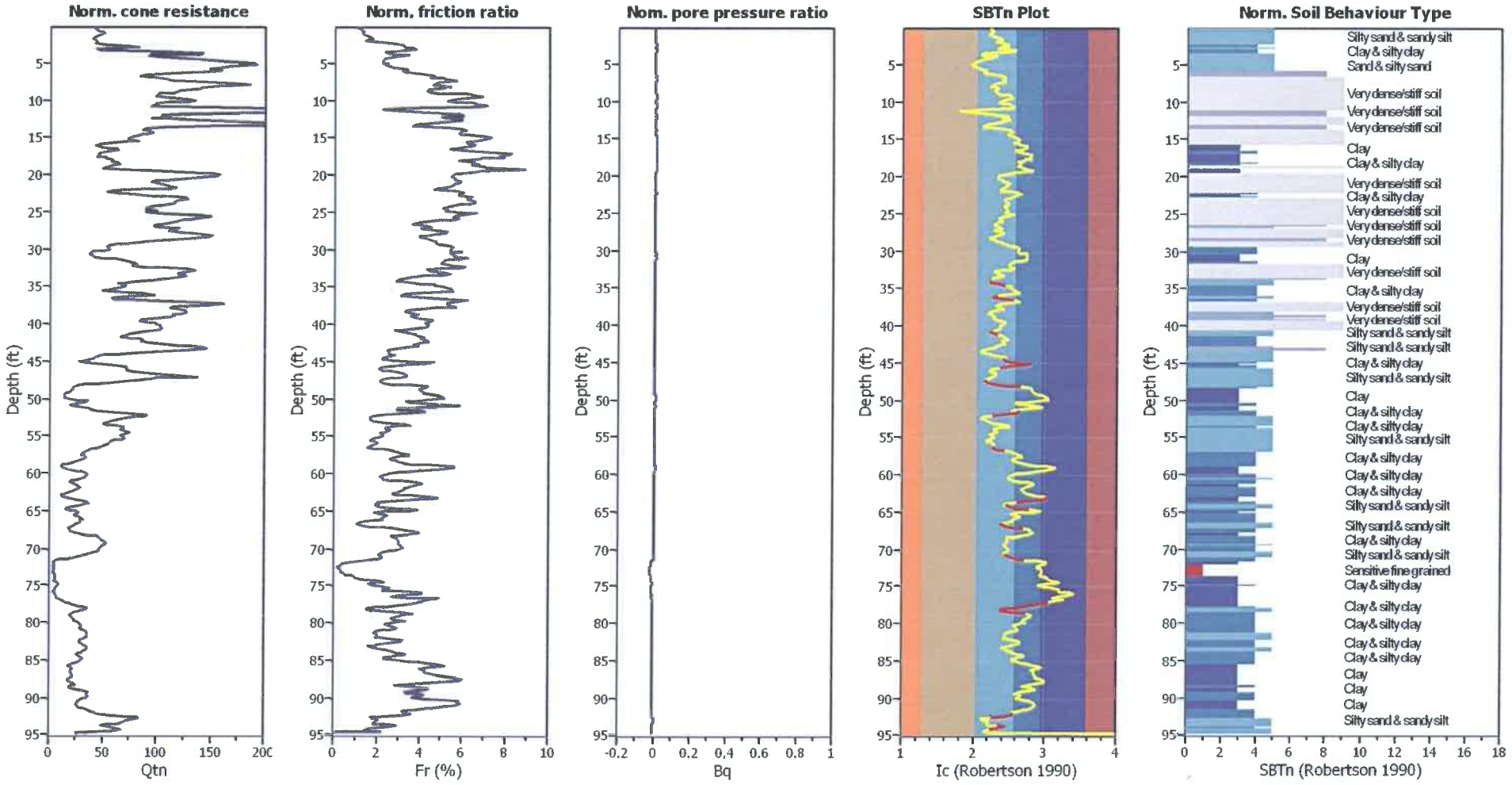
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

E.2

CPT basic interpretation plots (normalized)



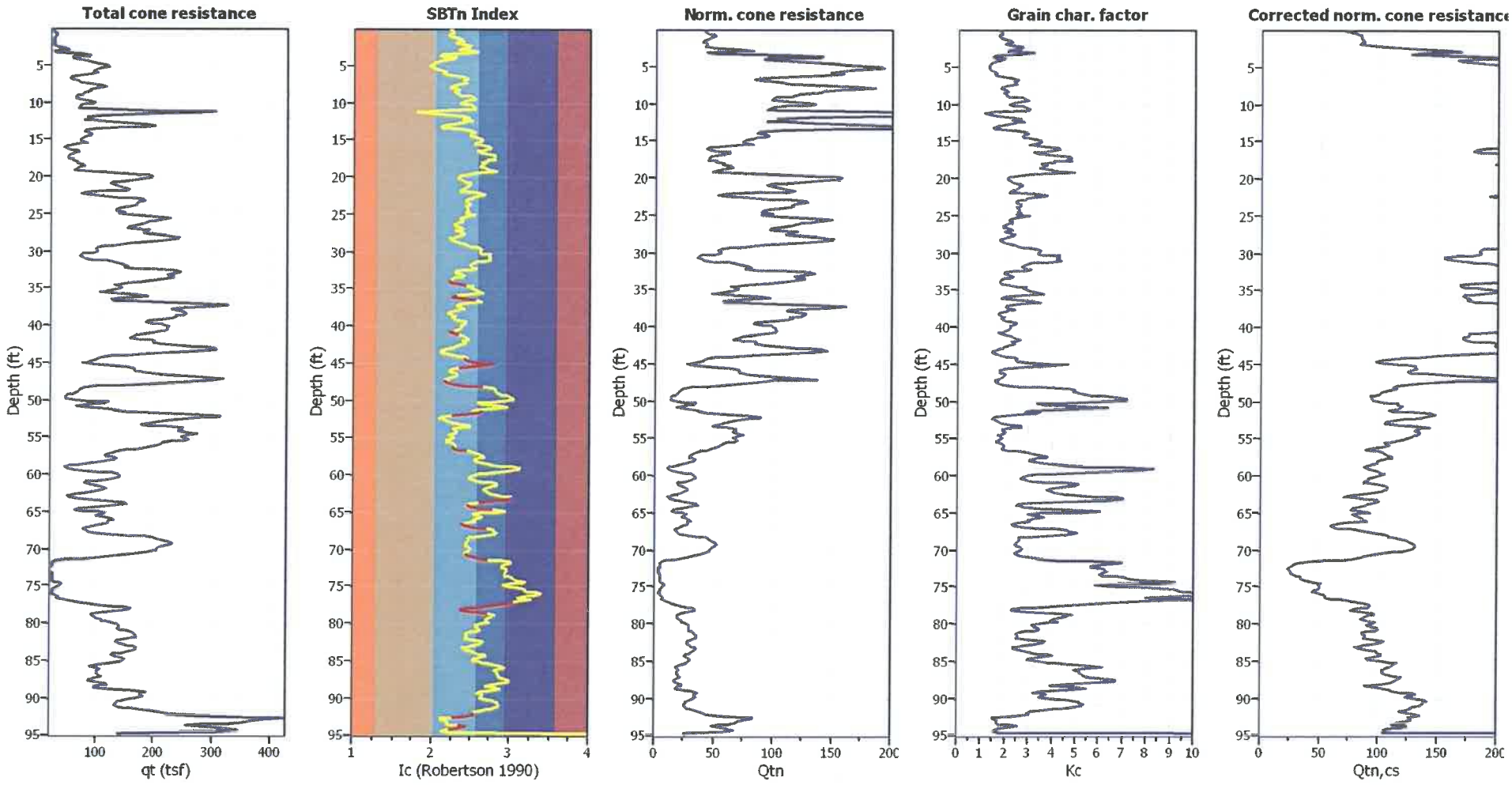
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)

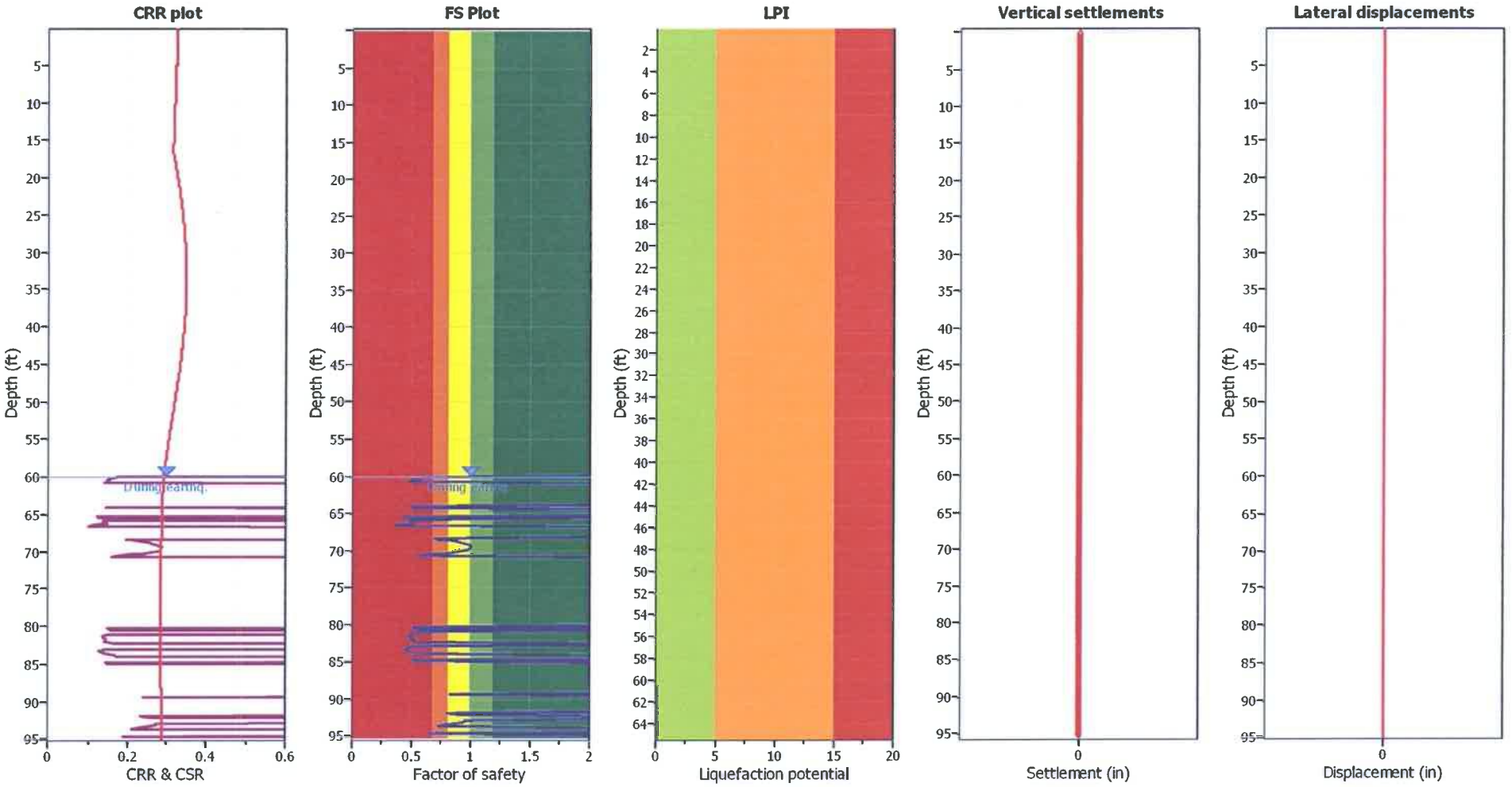


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

E.4

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

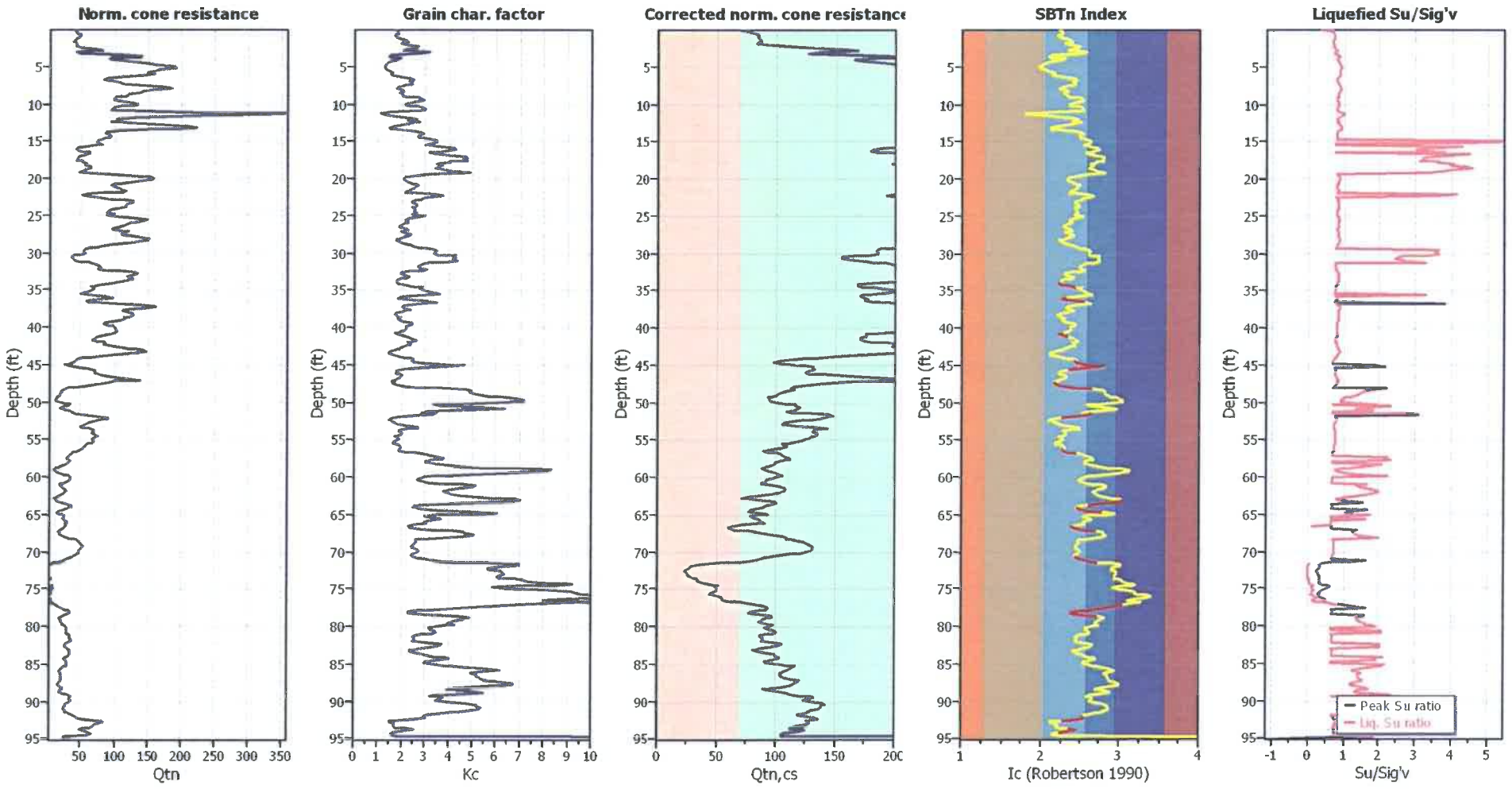
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

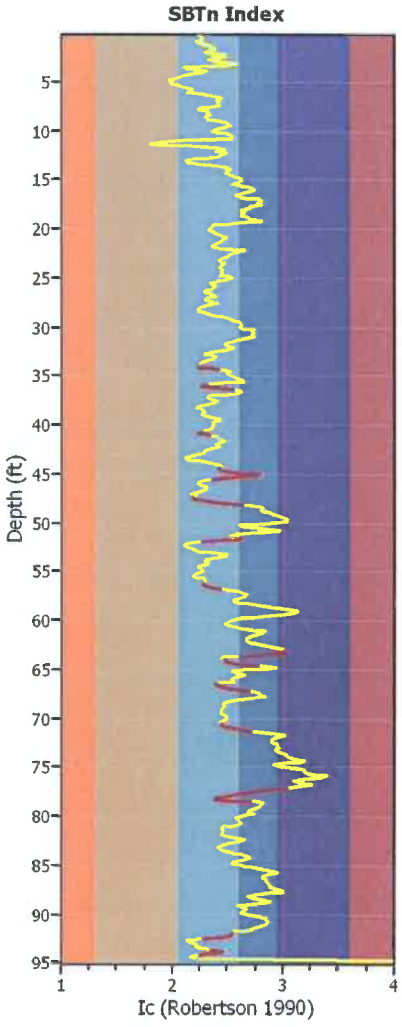
E.1

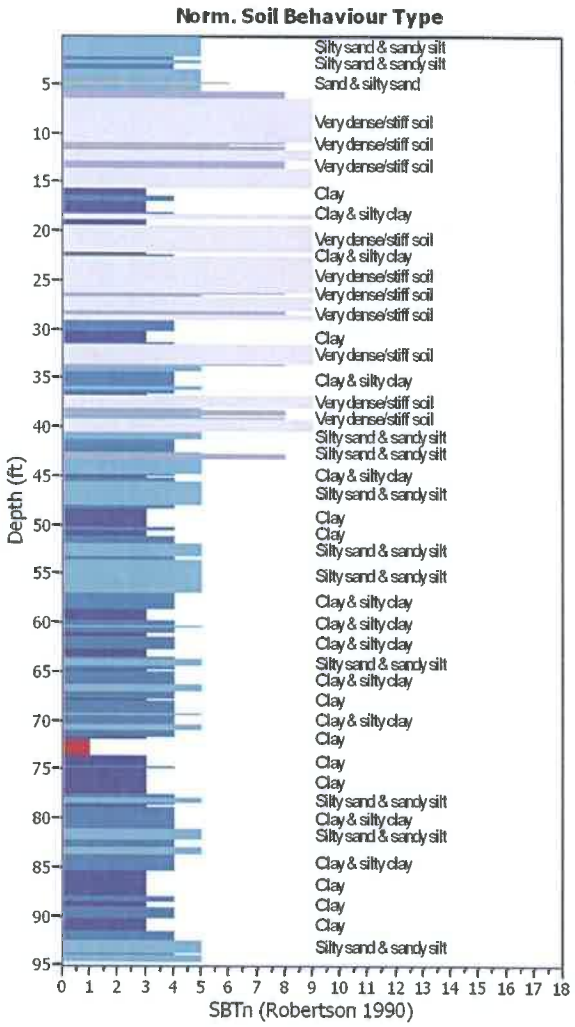
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





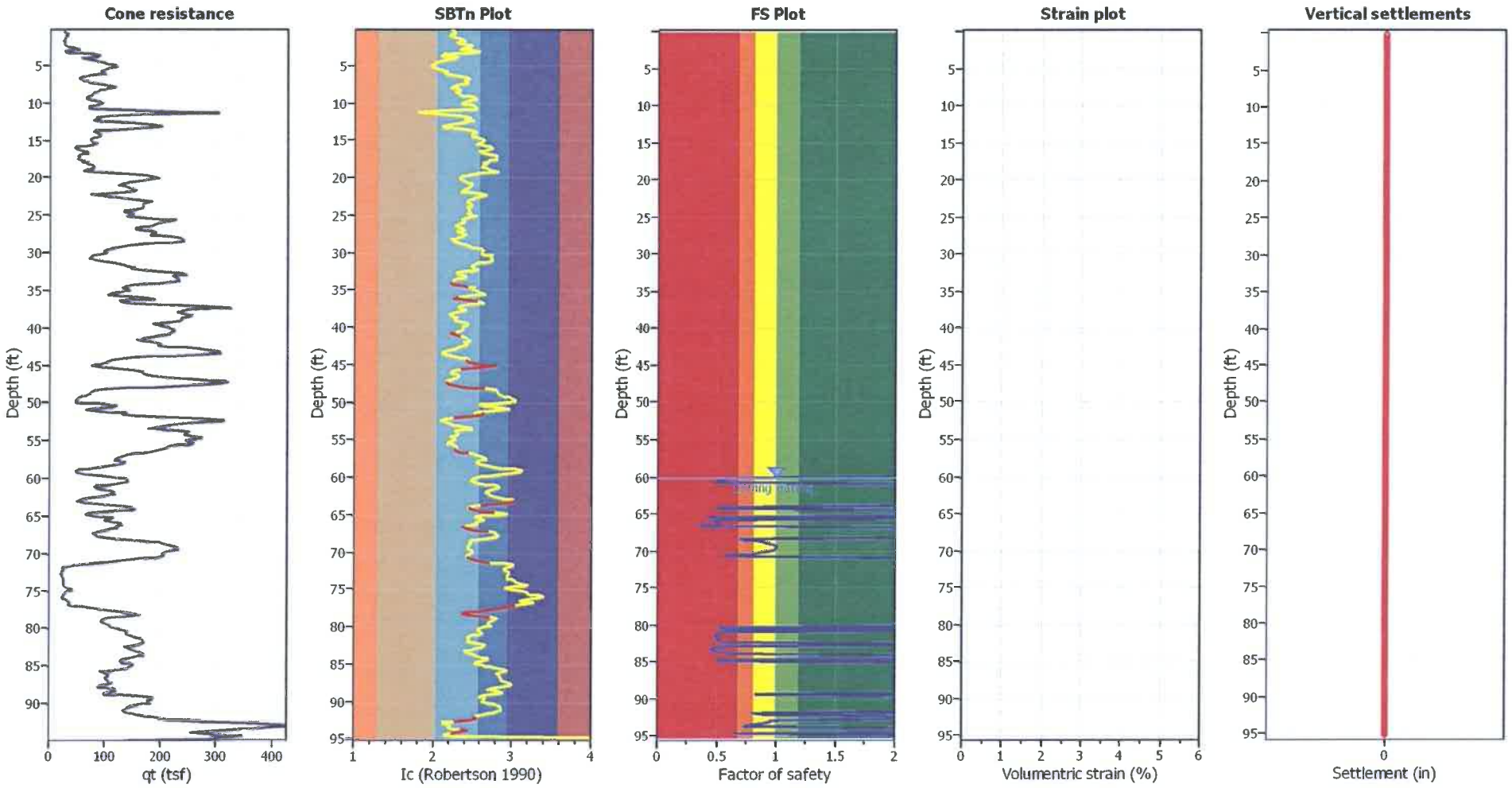
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	580
Total points excluded:	71
Exclusion percentage:	12.24%
Number of layers detected:	16

Estimation of post-earthquake settlements

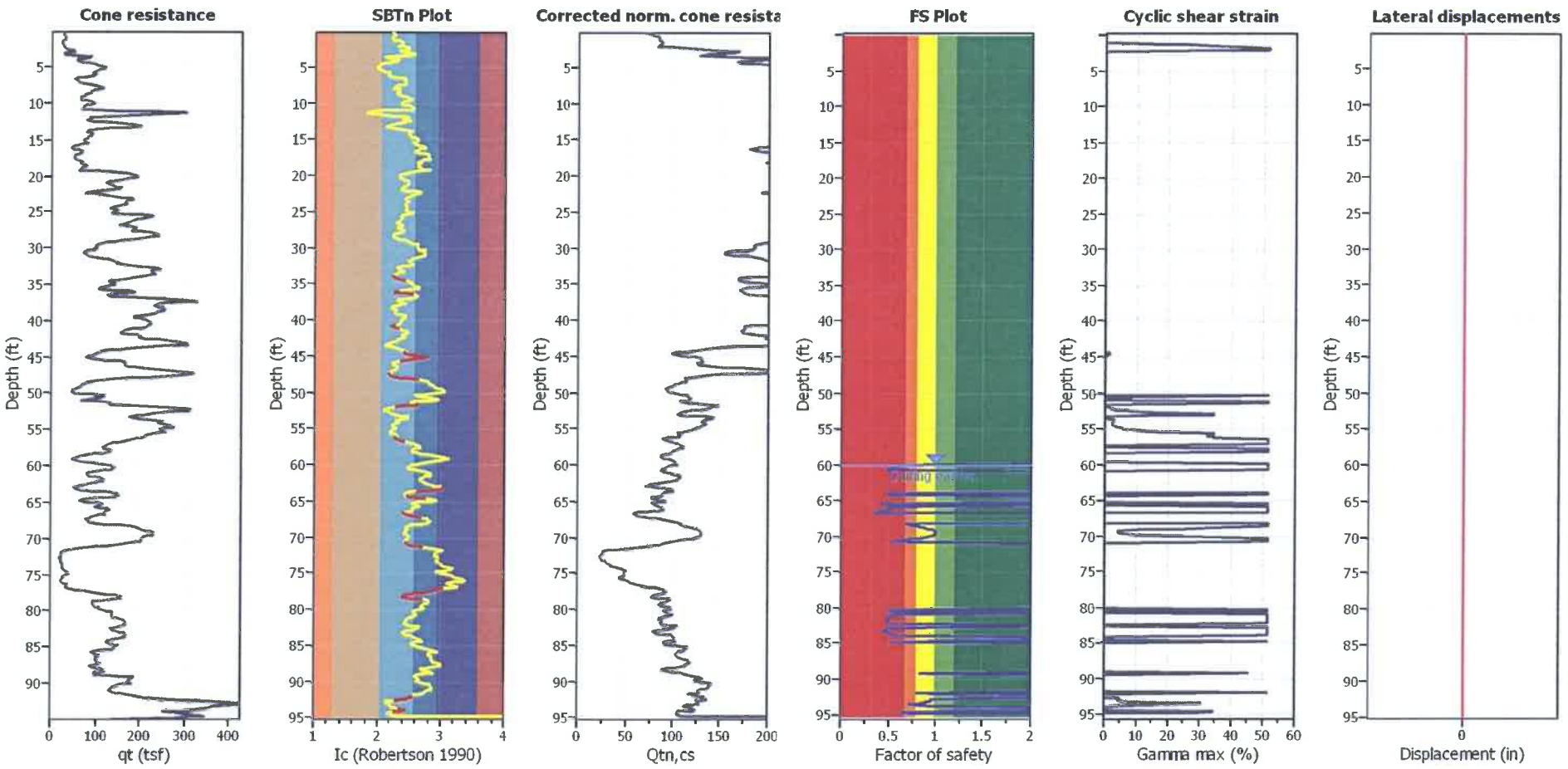


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
Ic:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

123837747

E.9

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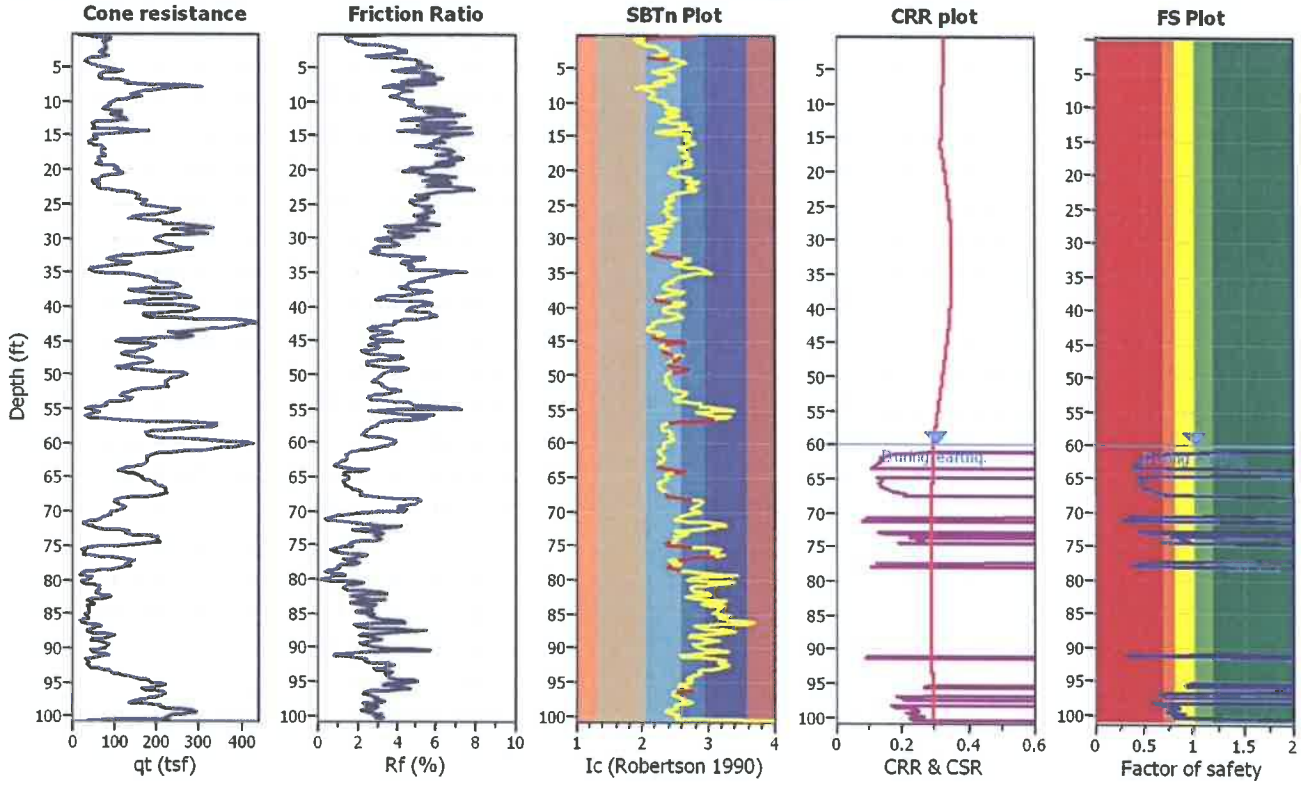
LIQUEFACTION ANALYSIS REPORT

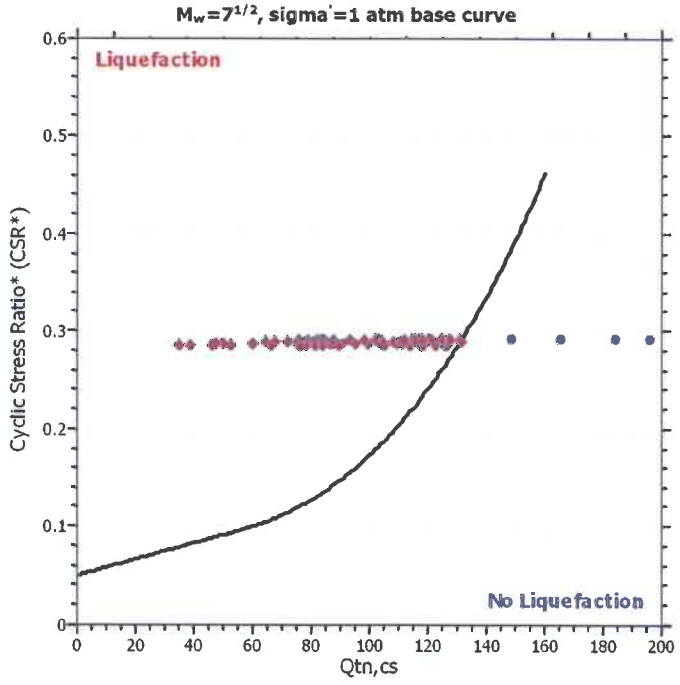
Project title : Pacific Commercenter Lot 13 **Location : Lake Forest, CA**

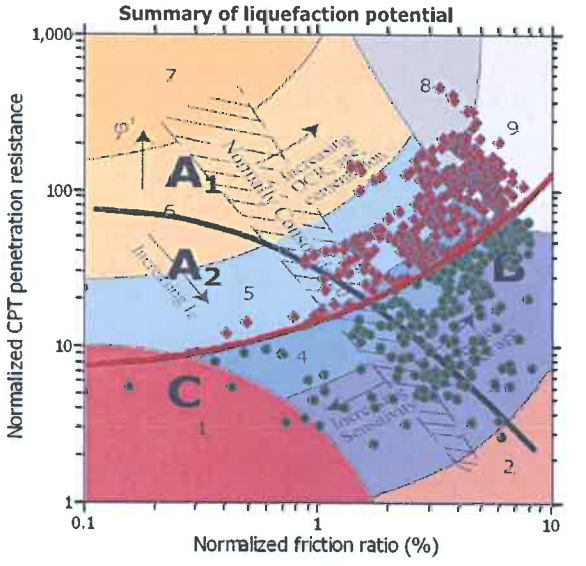
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	60.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

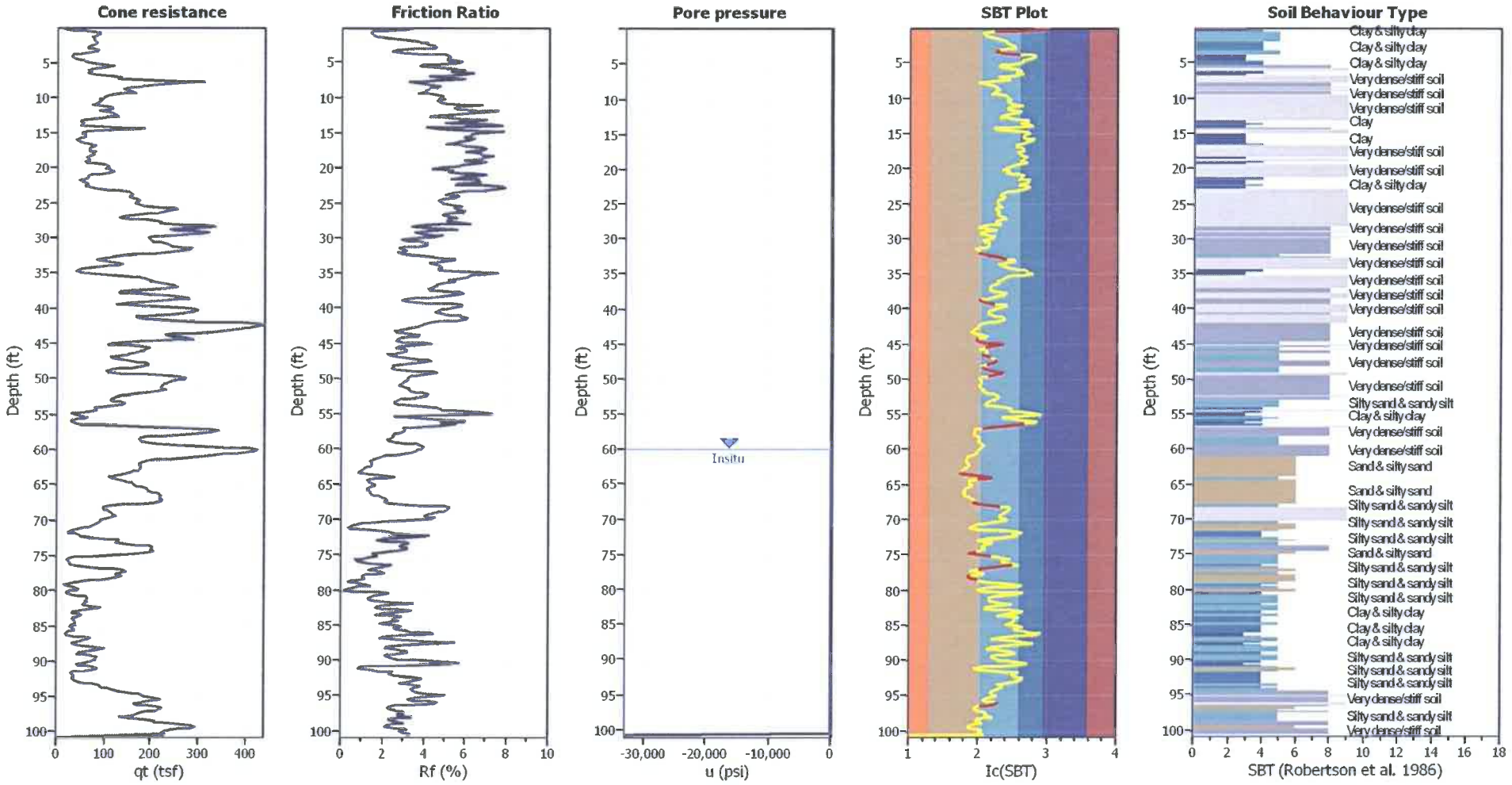






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



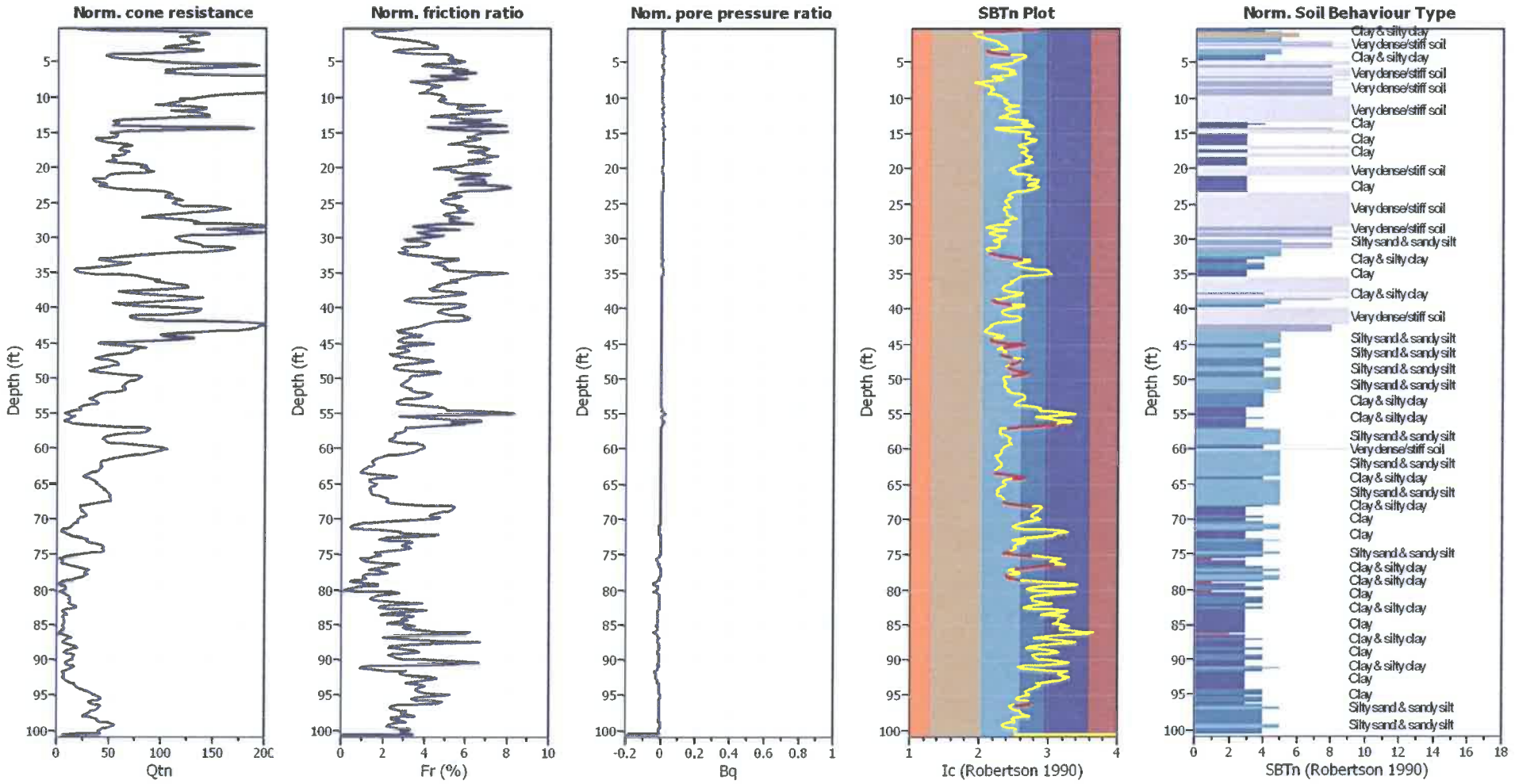
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)

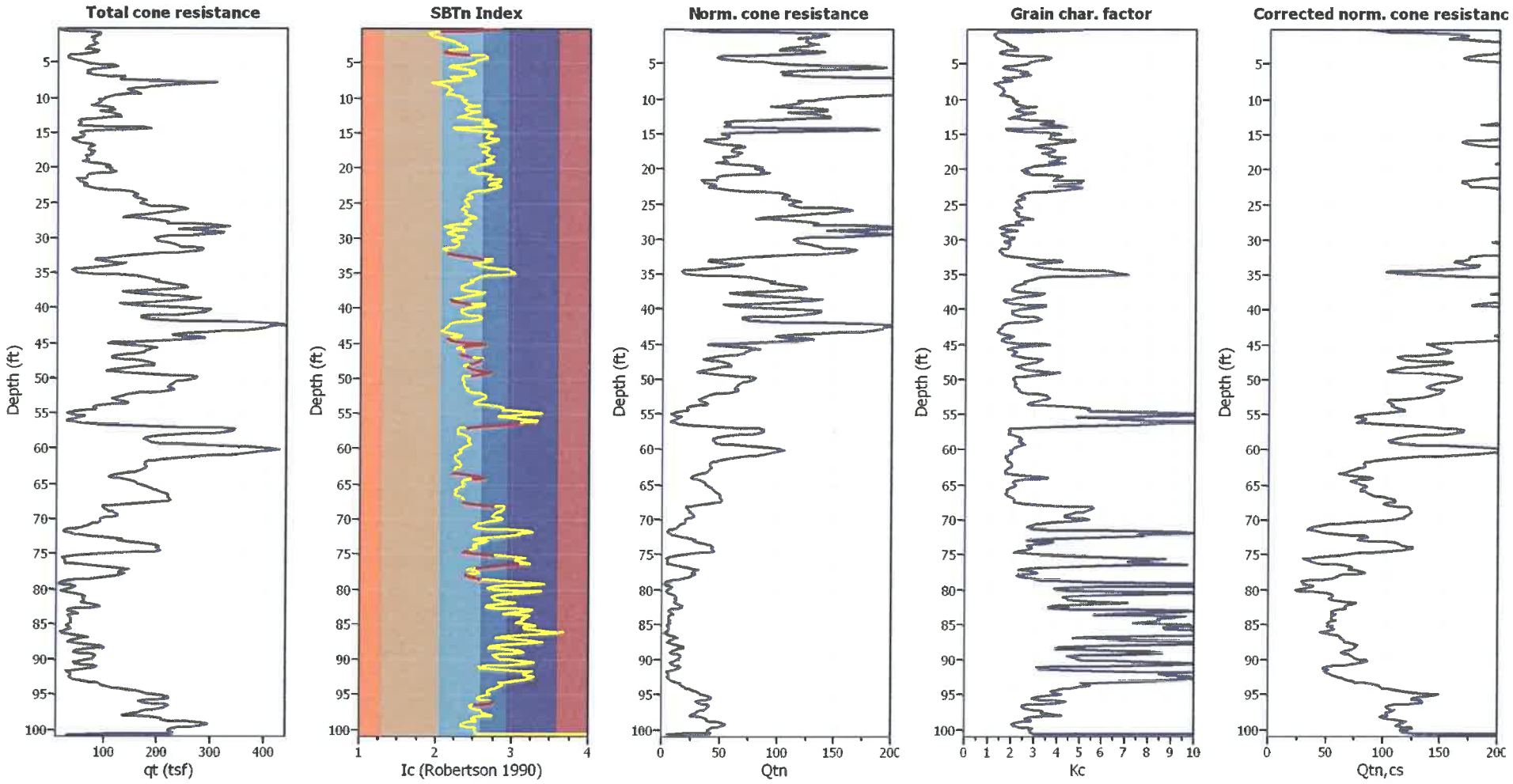


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

E.12

Liquefaction analysis overall plots (intermediate results)

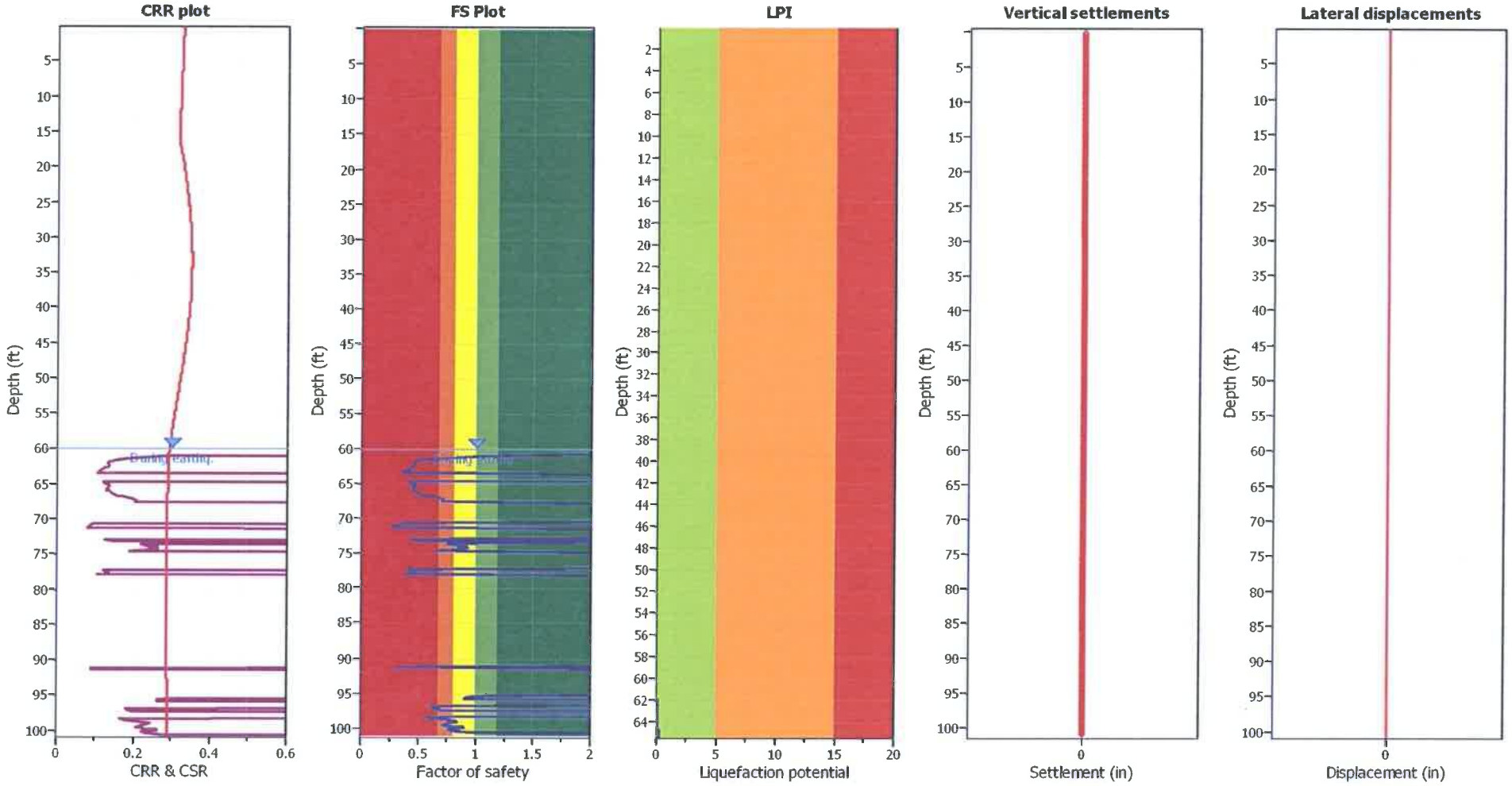


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (Insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

E.13

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

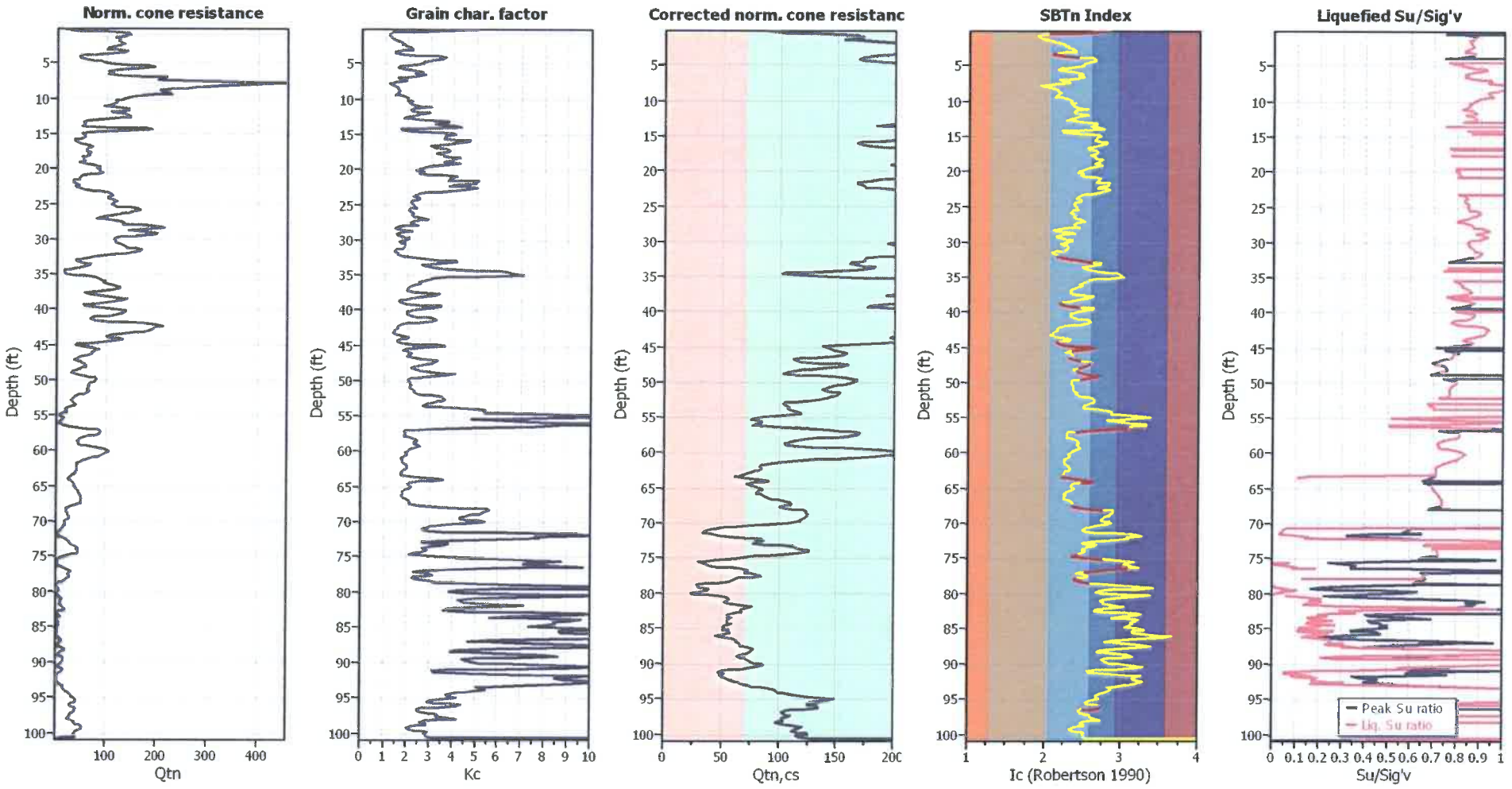
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

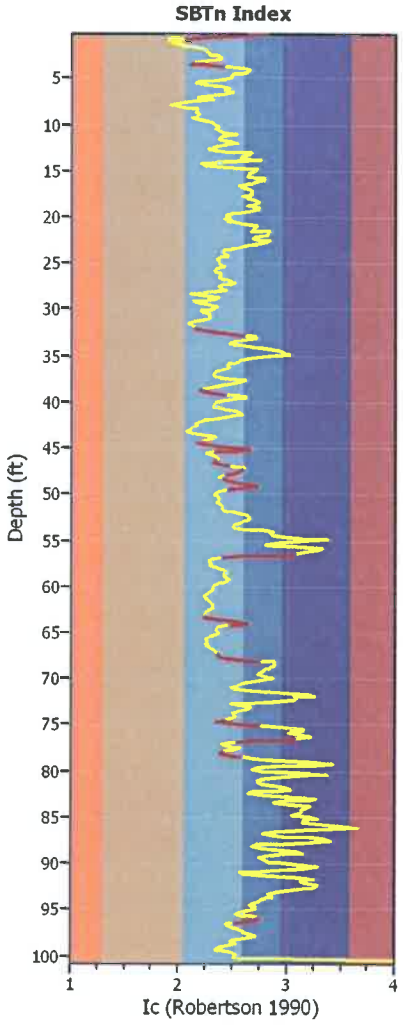
E.15

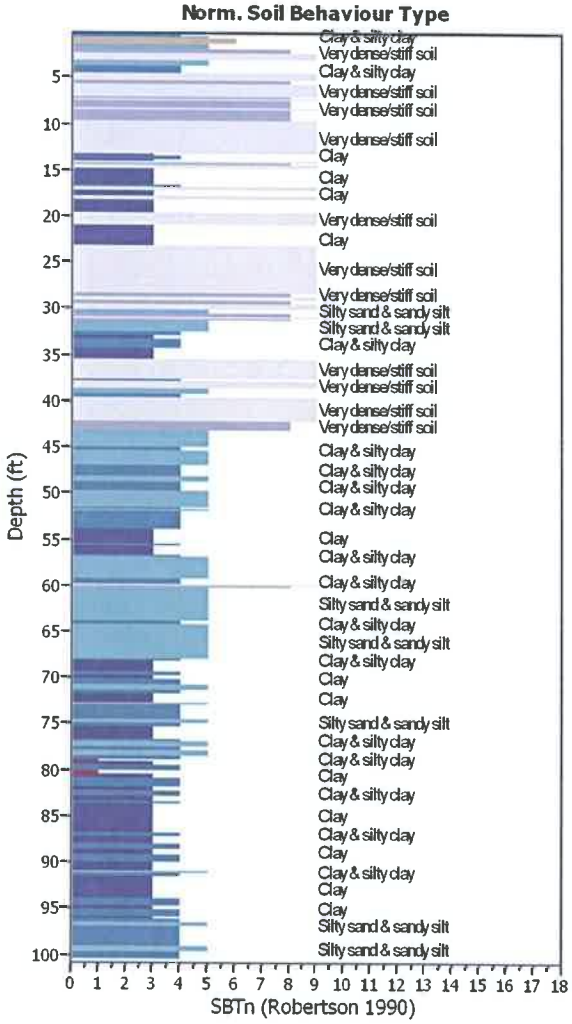
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





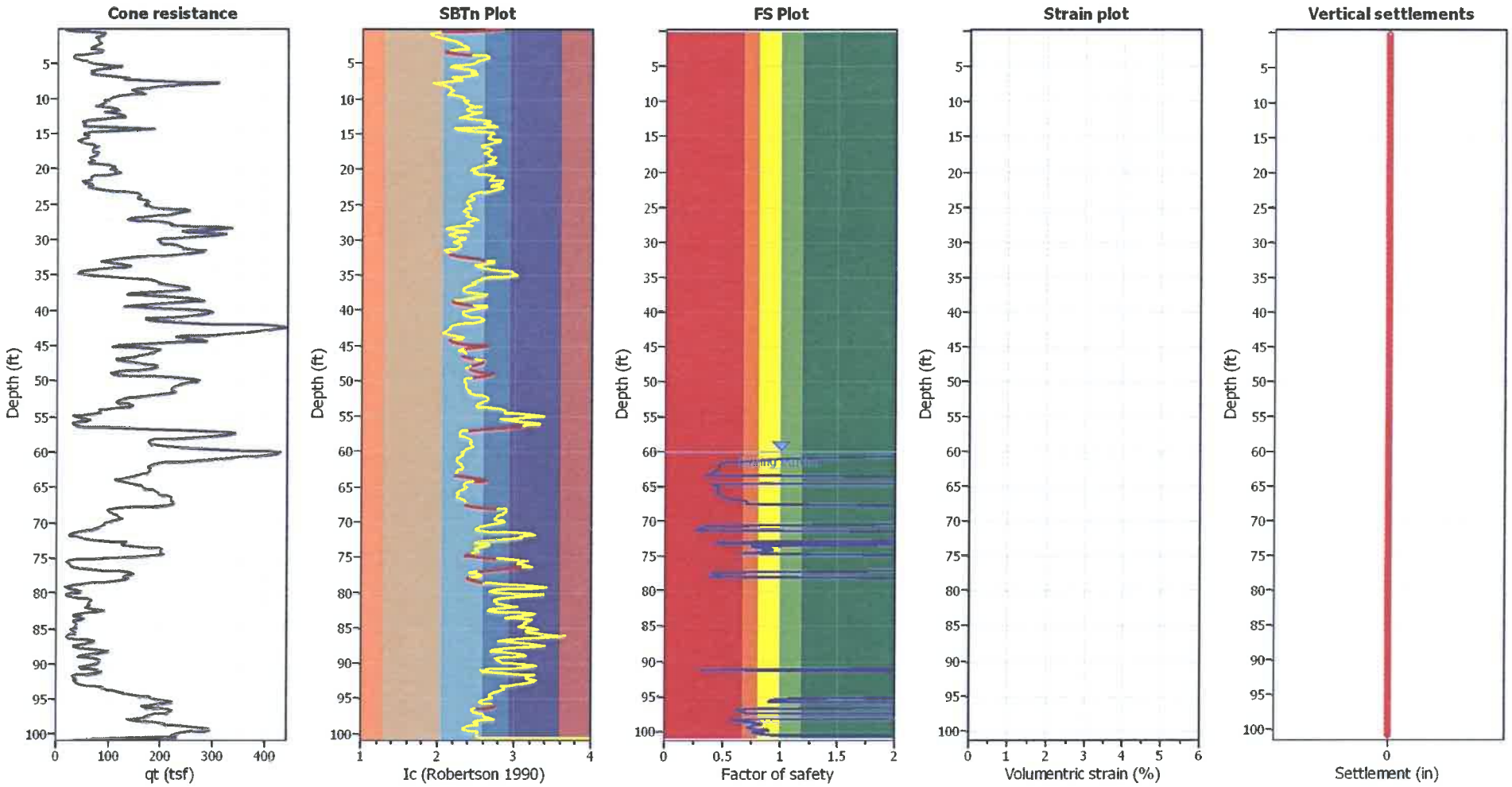
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	615
Total points excluded:	80
Exclusion percentage:	13.01%
Number of layers detected:	18

Estimation of post-earthquake settlements



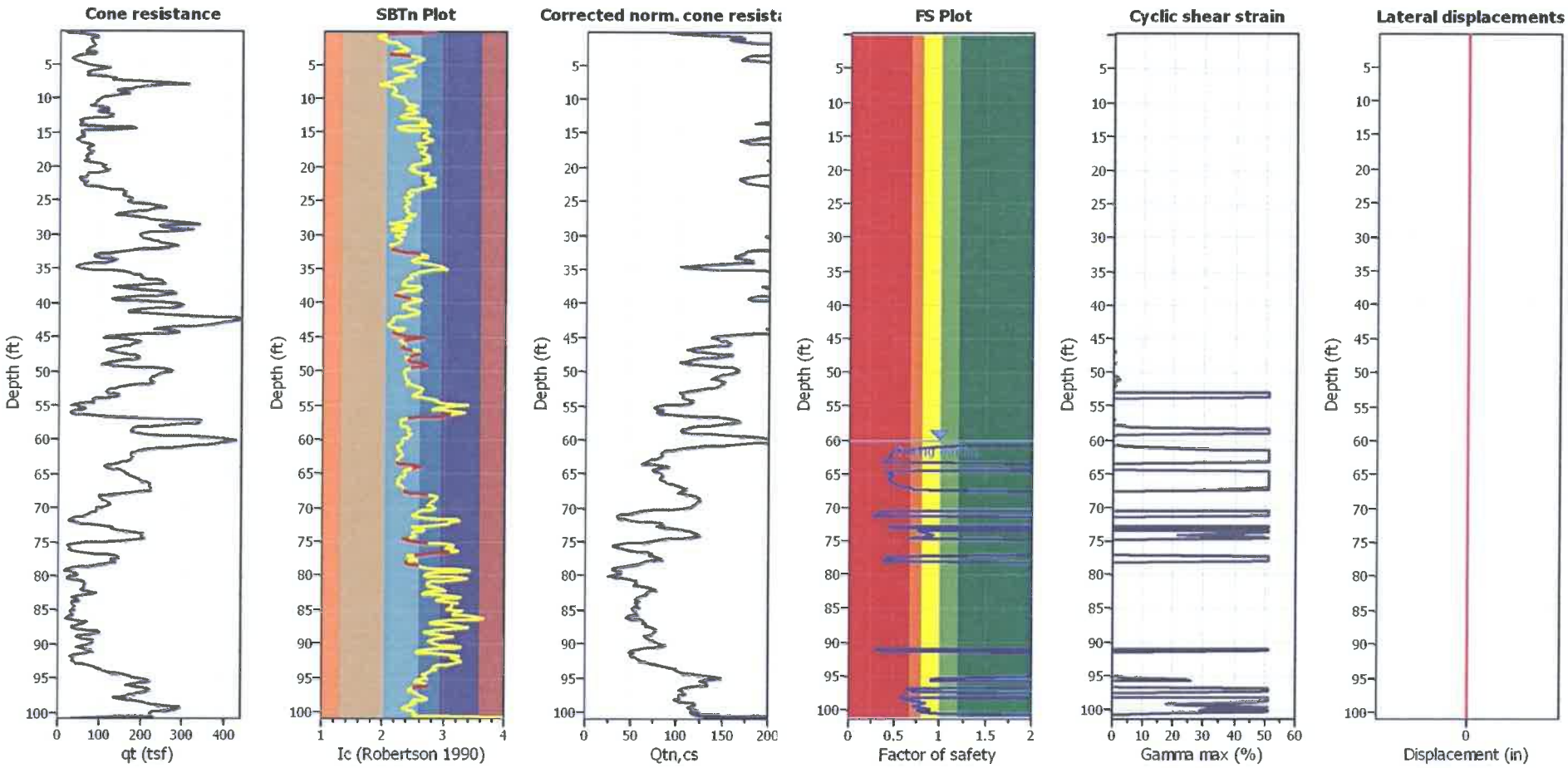
Abbreviations

q_c:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

E.17

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$(Q_{tn,cs})$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

E.16

STONEY-MILLER CONSULTANTS, INC.
14 Hughes, Suite B-101
Irvine, CA 92618

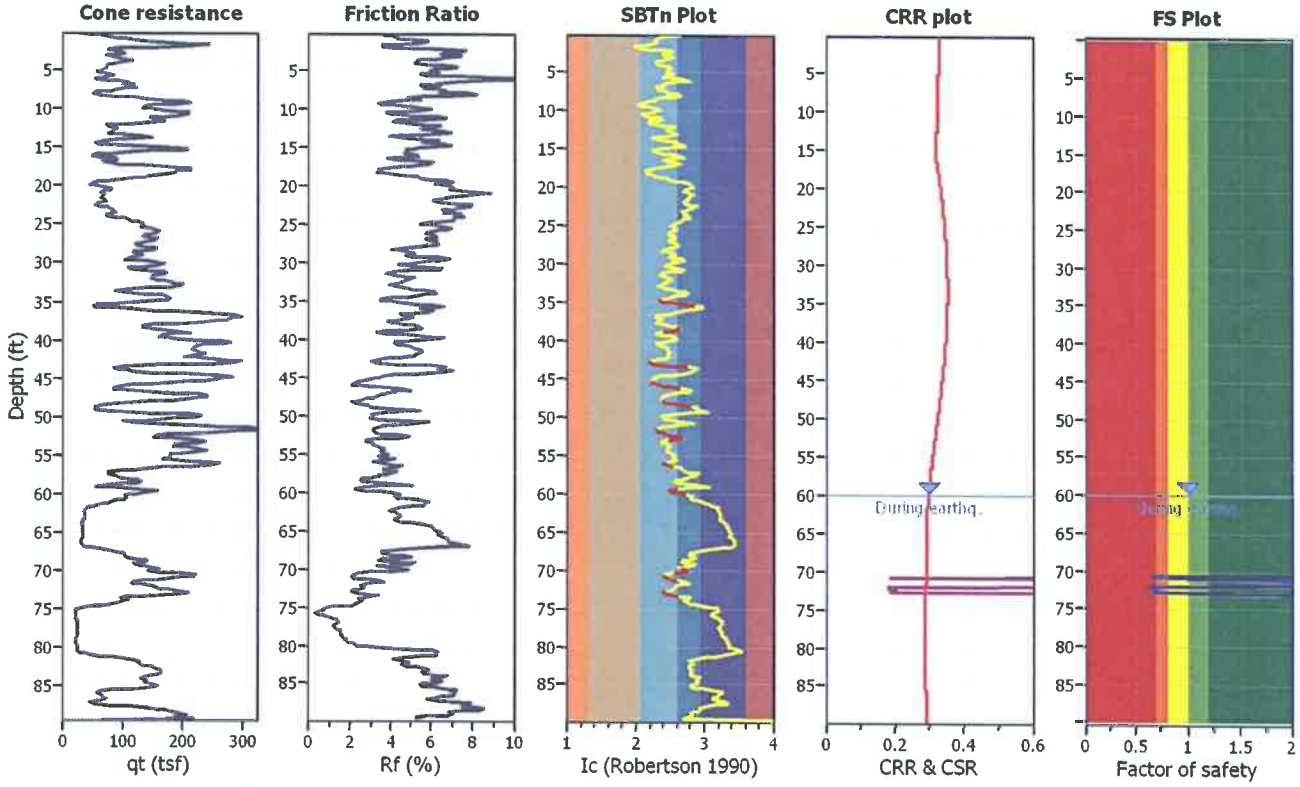
LIQUEFACTION ANALYSIS REPORT

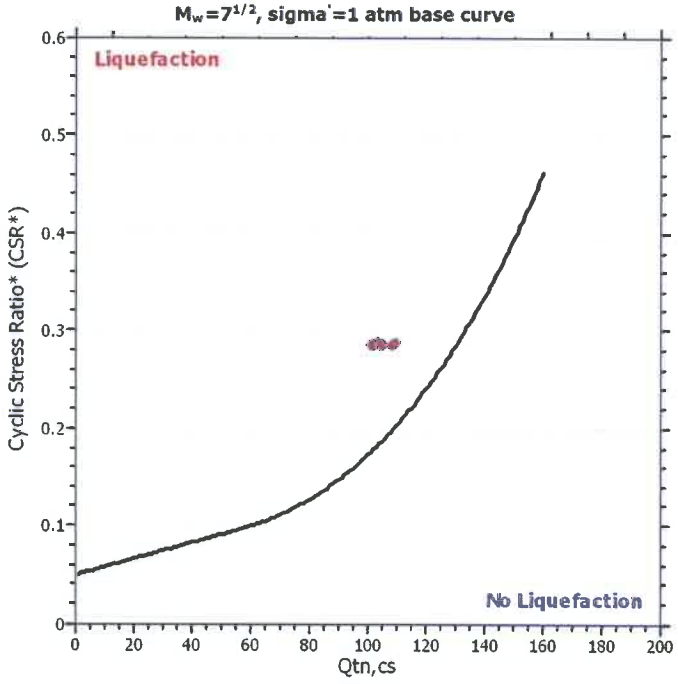
Project title : Pacific Commercenter Lot 13 **Location : Lake Forest, CA**

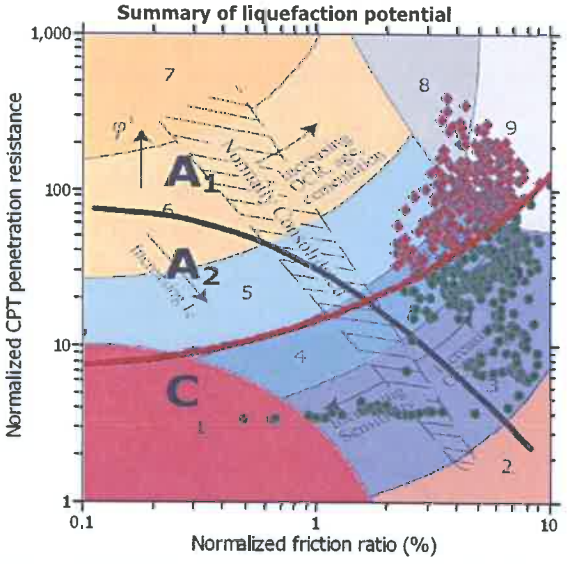
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	60.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

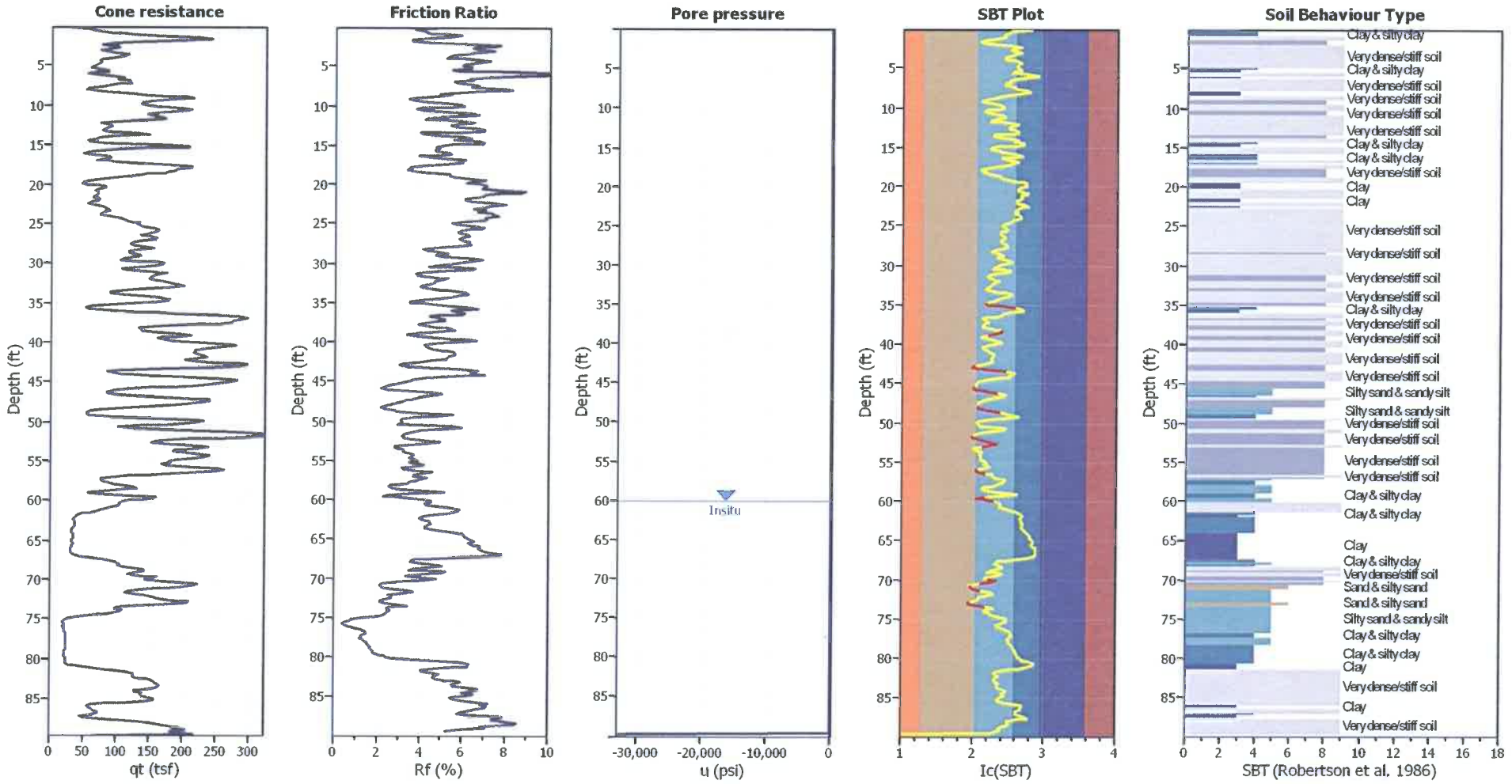






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

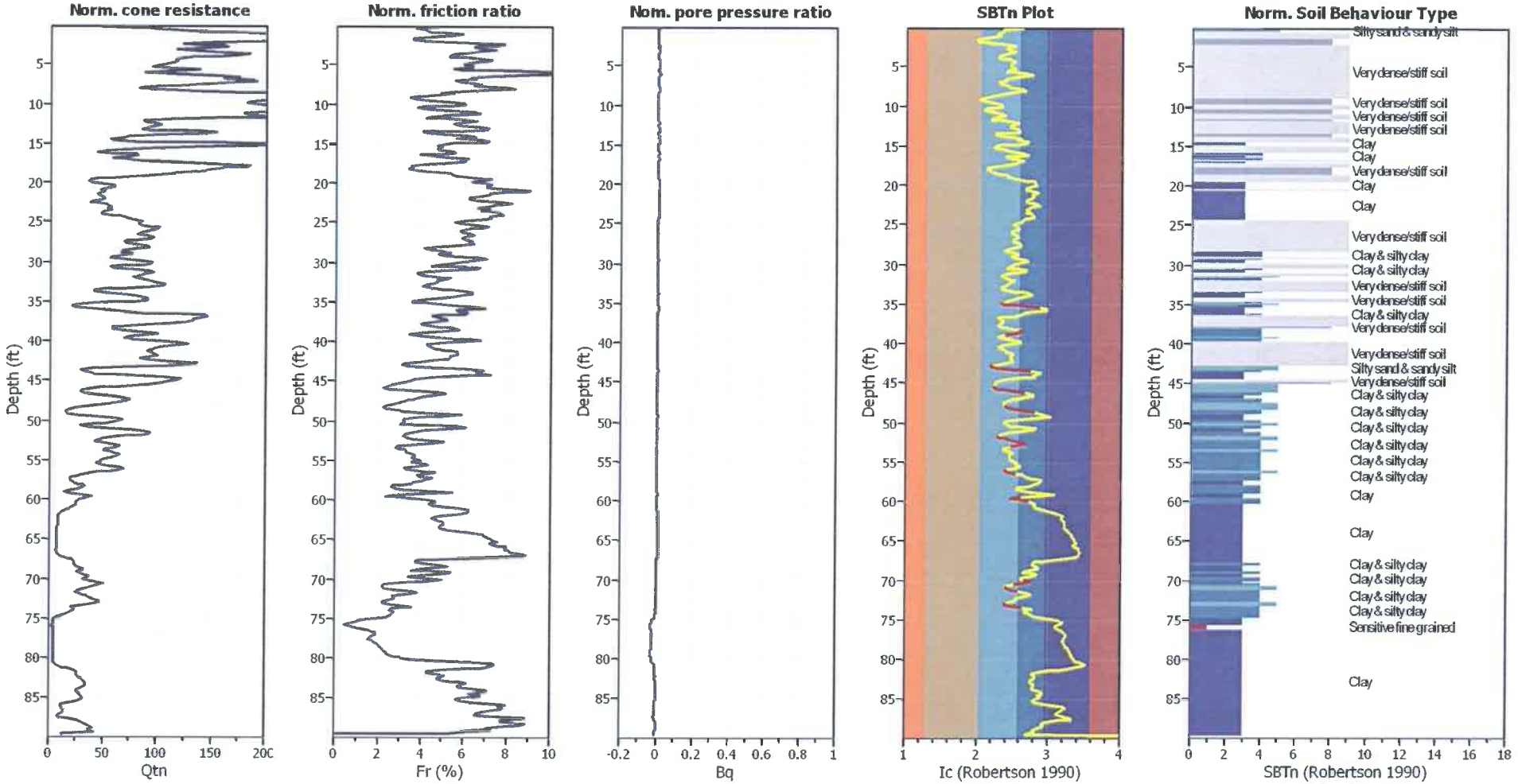
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

E.2b

CPT basic interpretation plots (normalized)



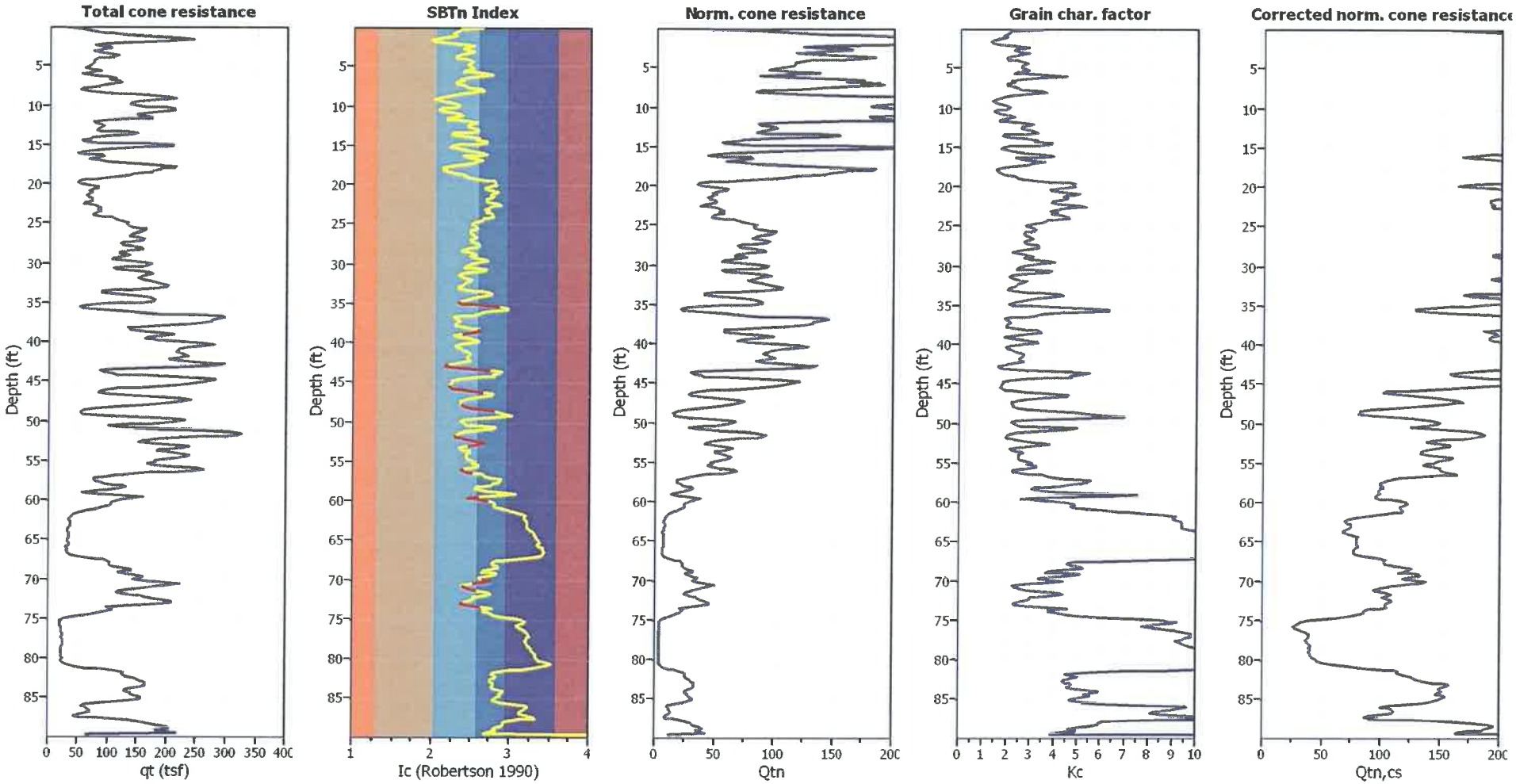
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)

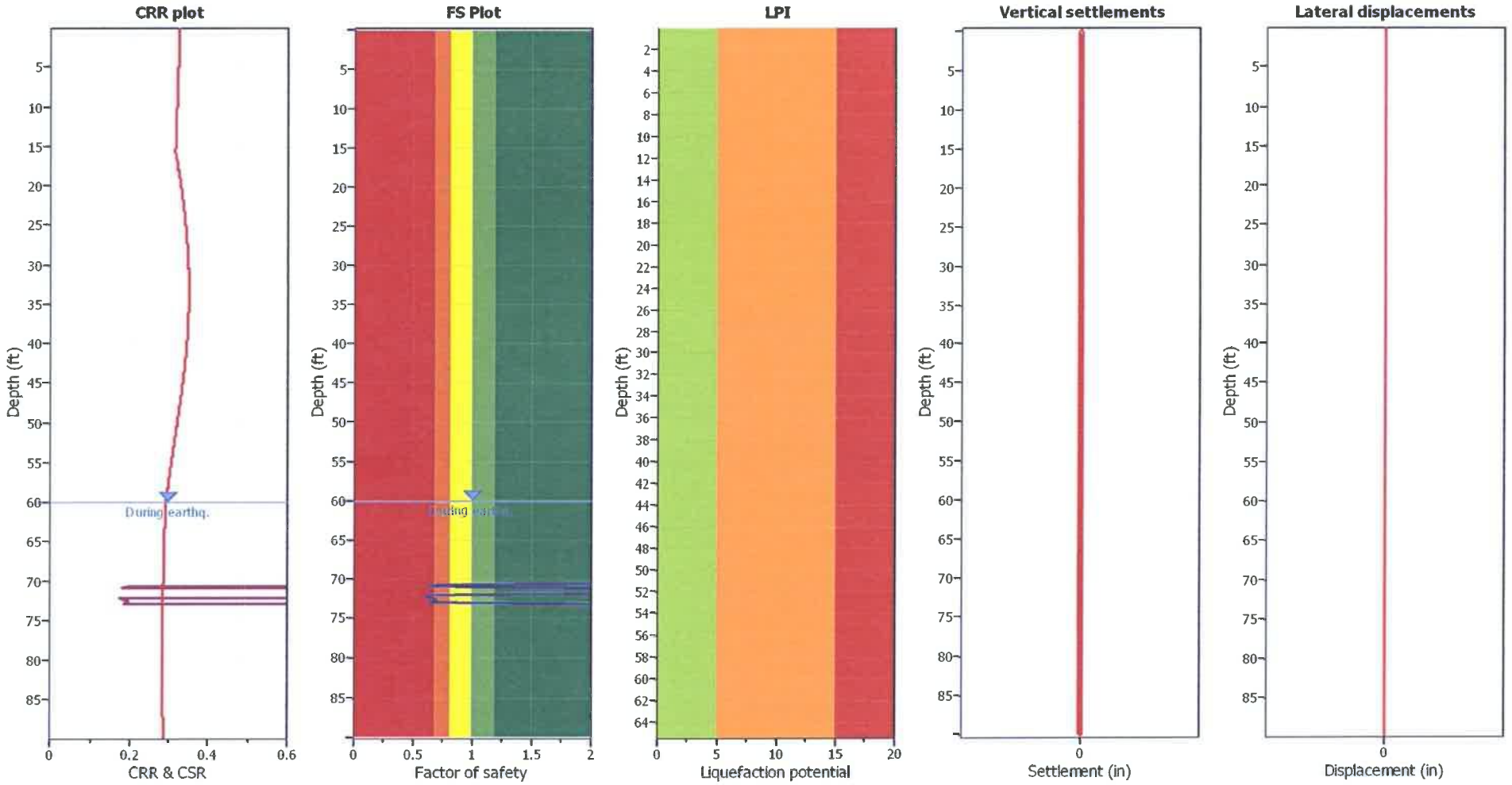


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

E.22

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

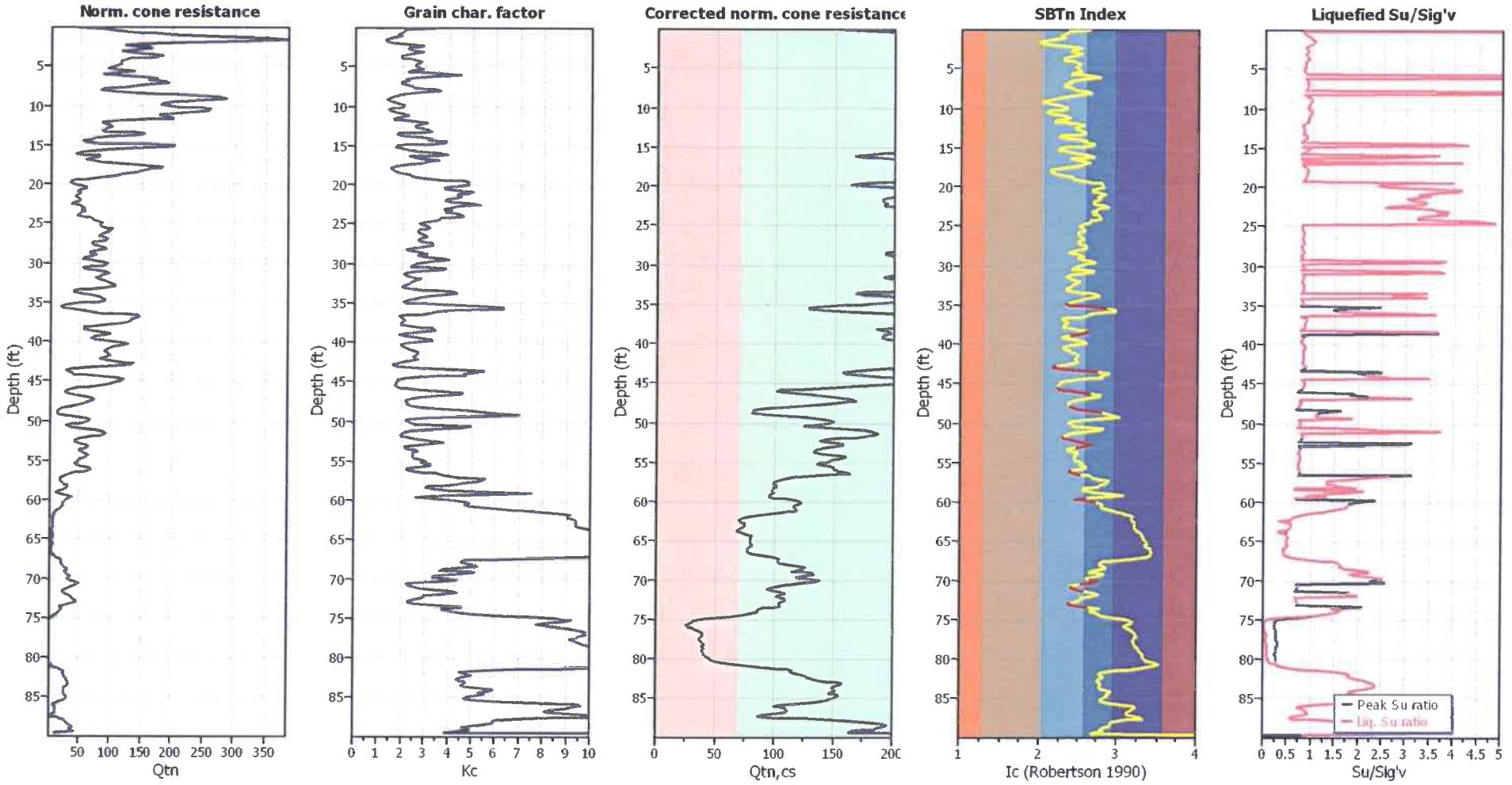
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

E.23

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

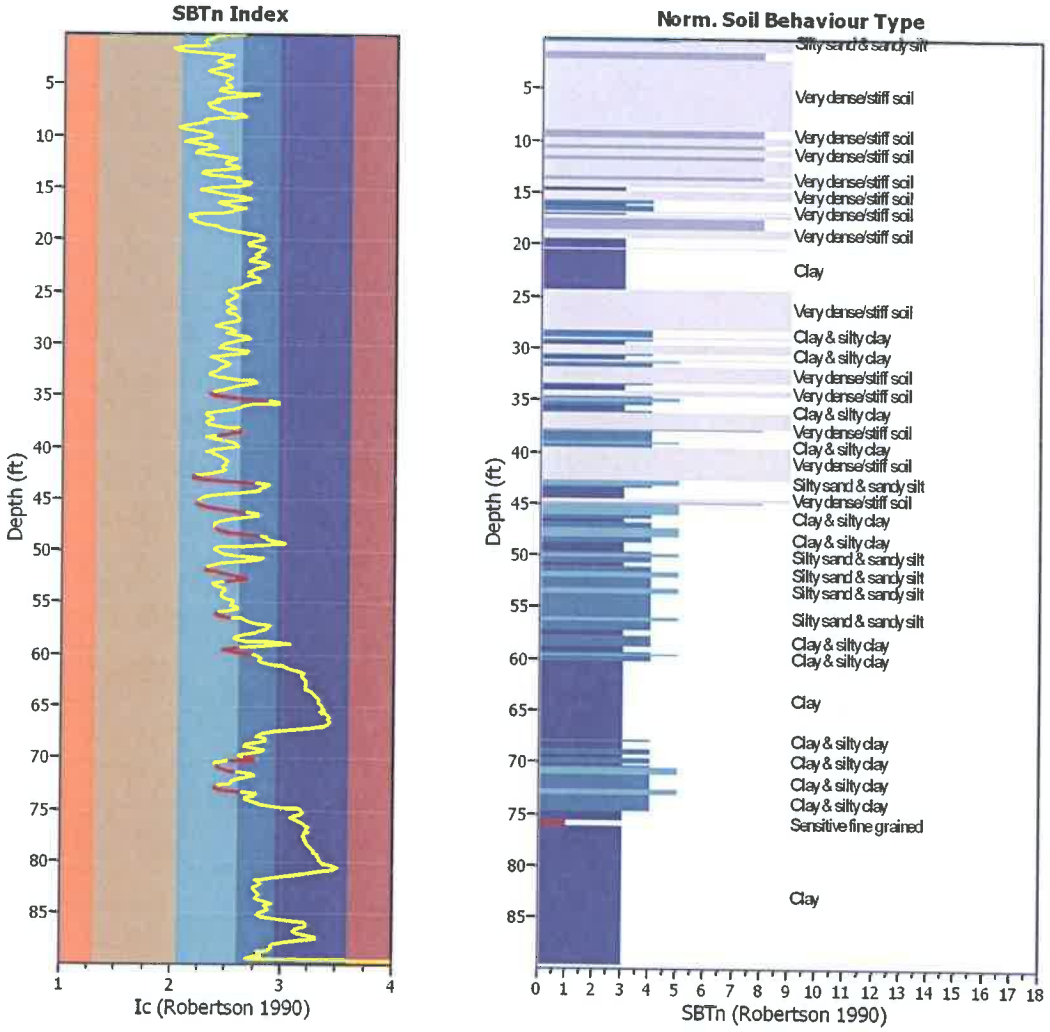
E.24

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



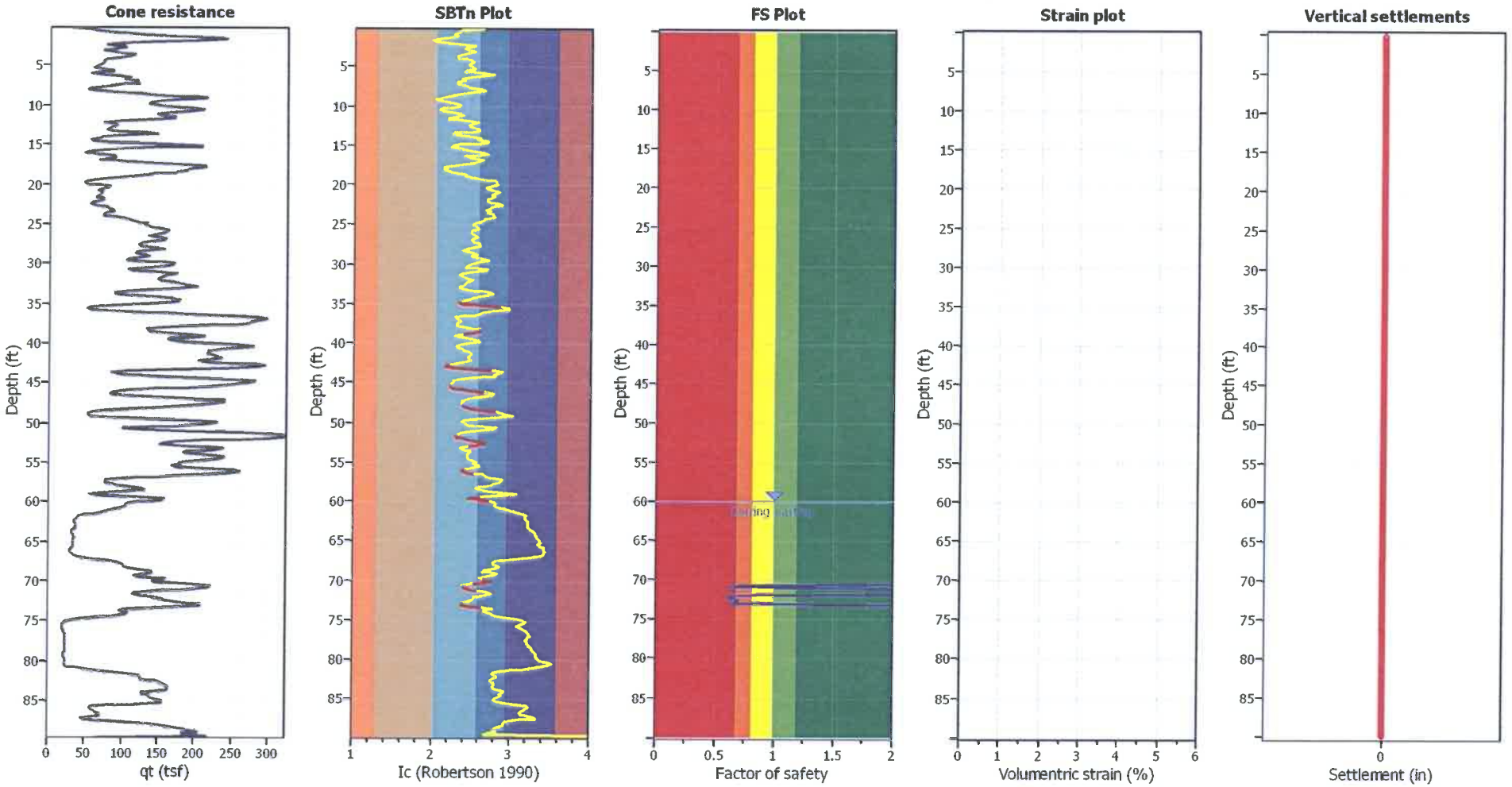
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	548
Total points excluded:	56
Exclusion percentage:	10.22%
Number of layers detected:	12

Estimation of post-earthquake settlements

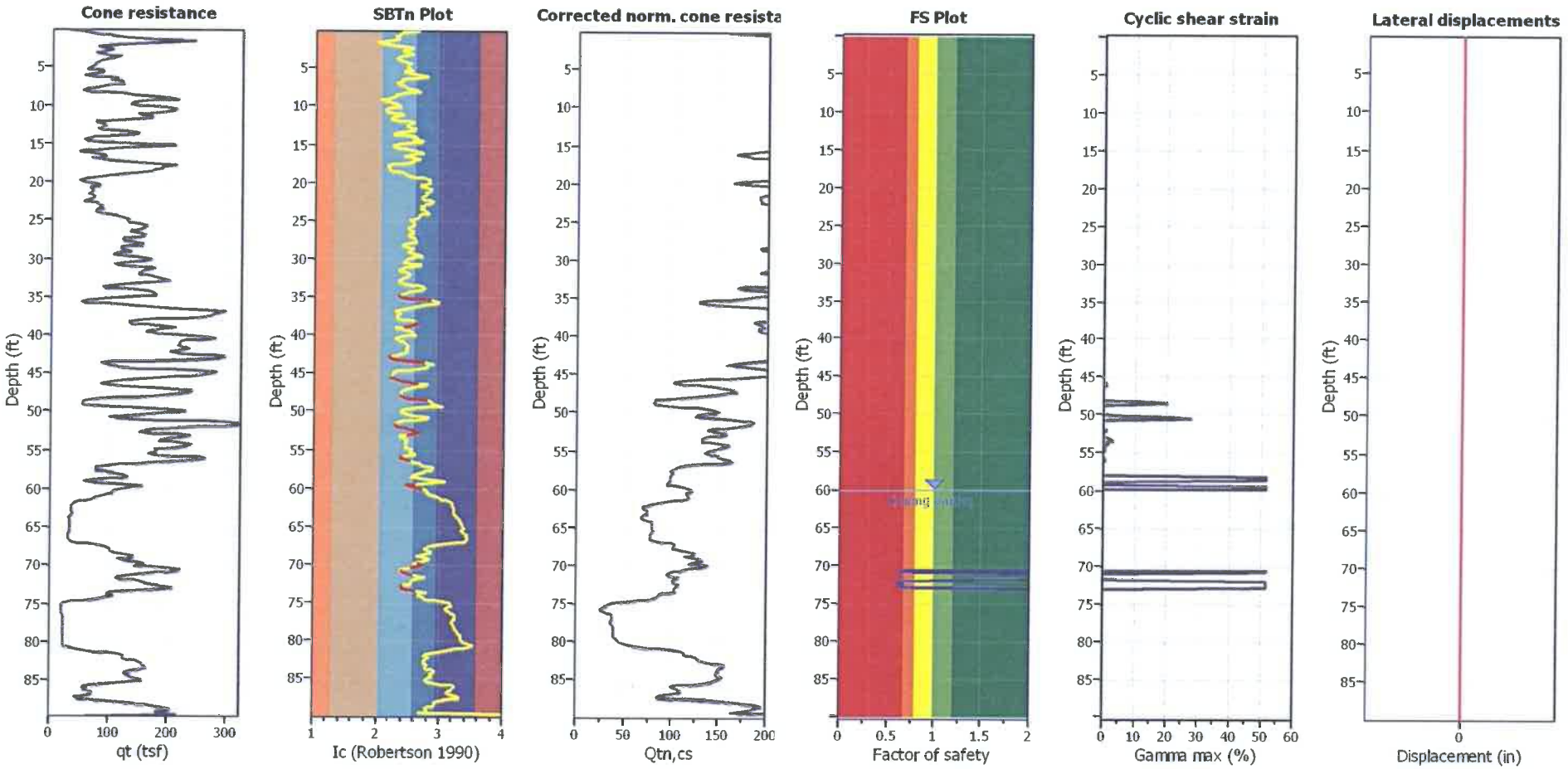


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)



Abbreviations

Q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

E.27

STONEY-MILLER CONSULTANTS, INC.
14 Hughes, Suite B-101
Irvine, CA 92618

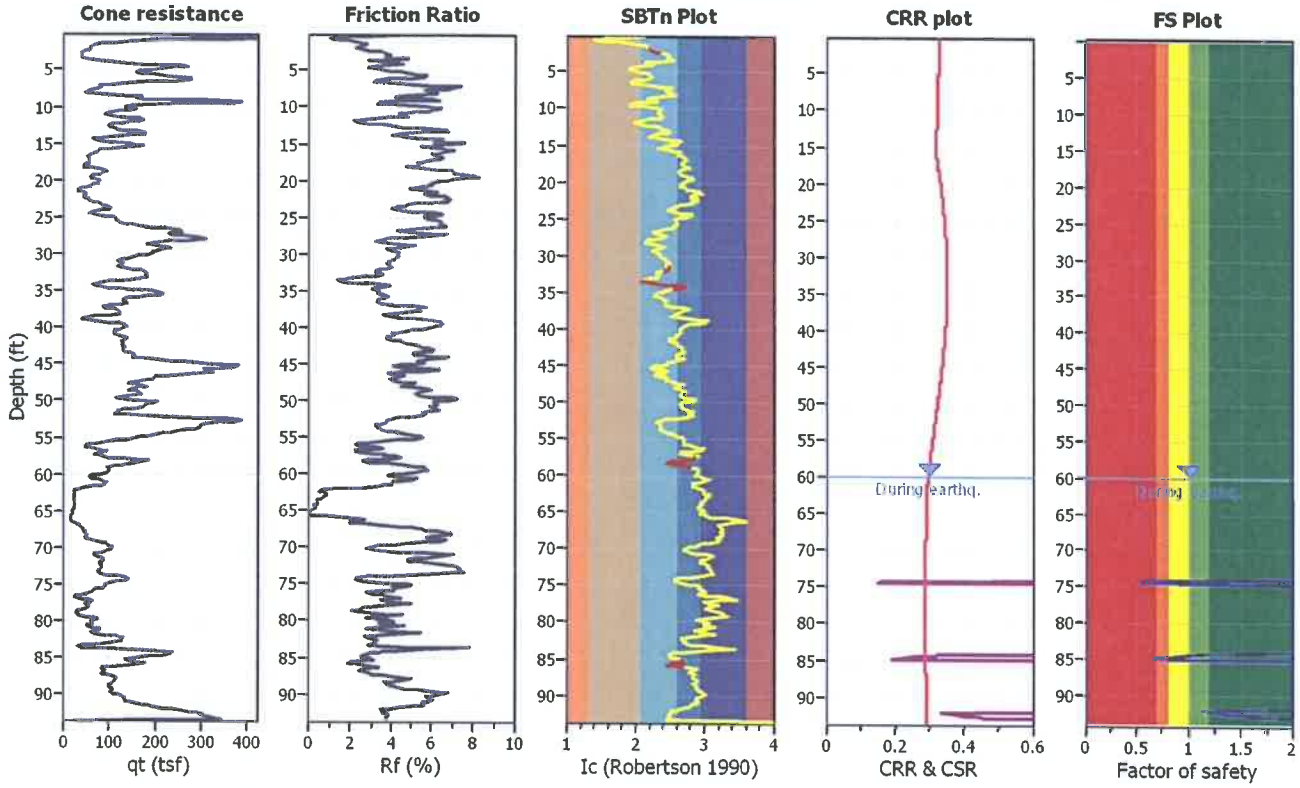
LIQUEFACTION ANALYSIS REPORT

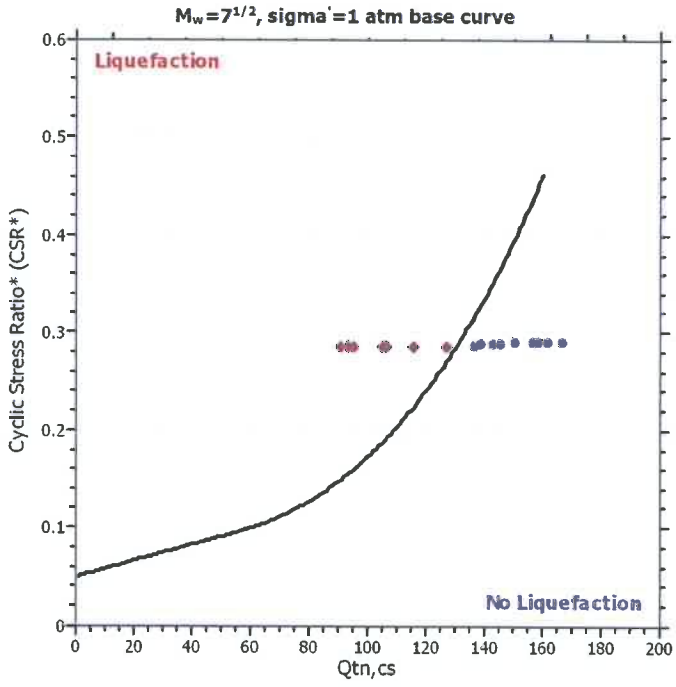
Project title : Pacific Commercenter Lot 13 **Location : Lake Forest, CA**

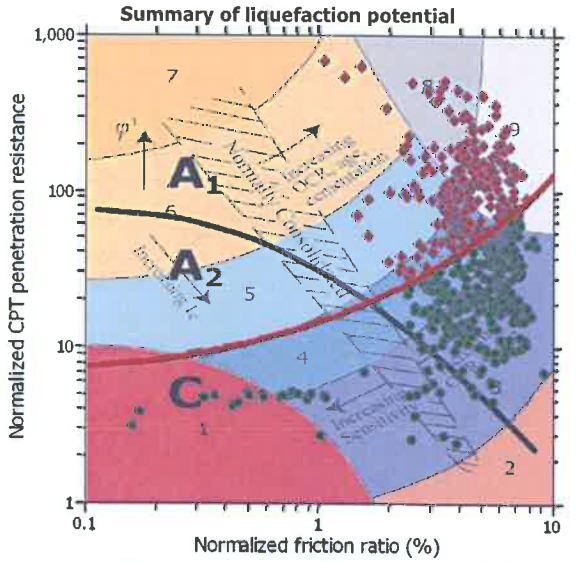
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	60.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	60.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

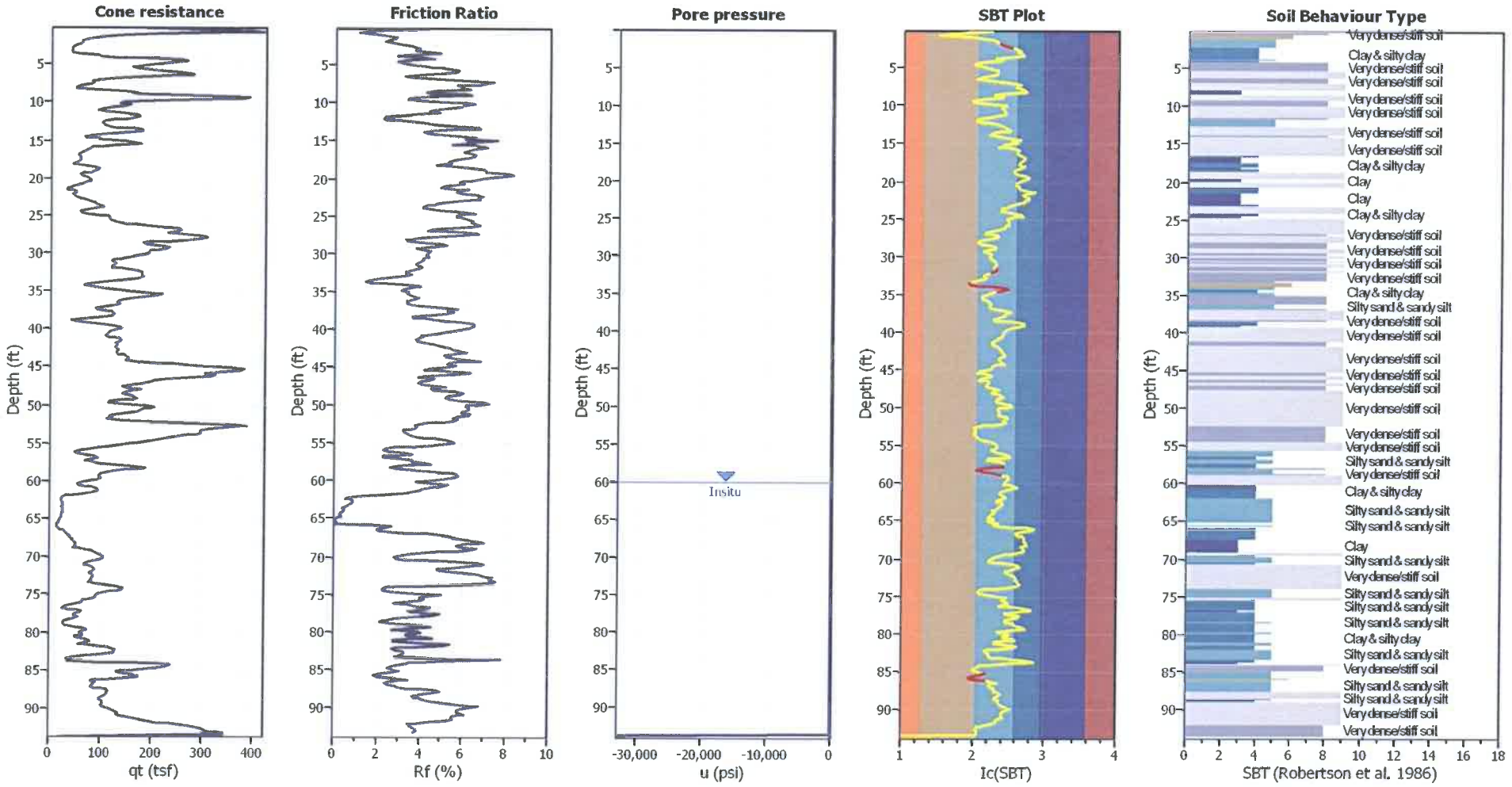






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



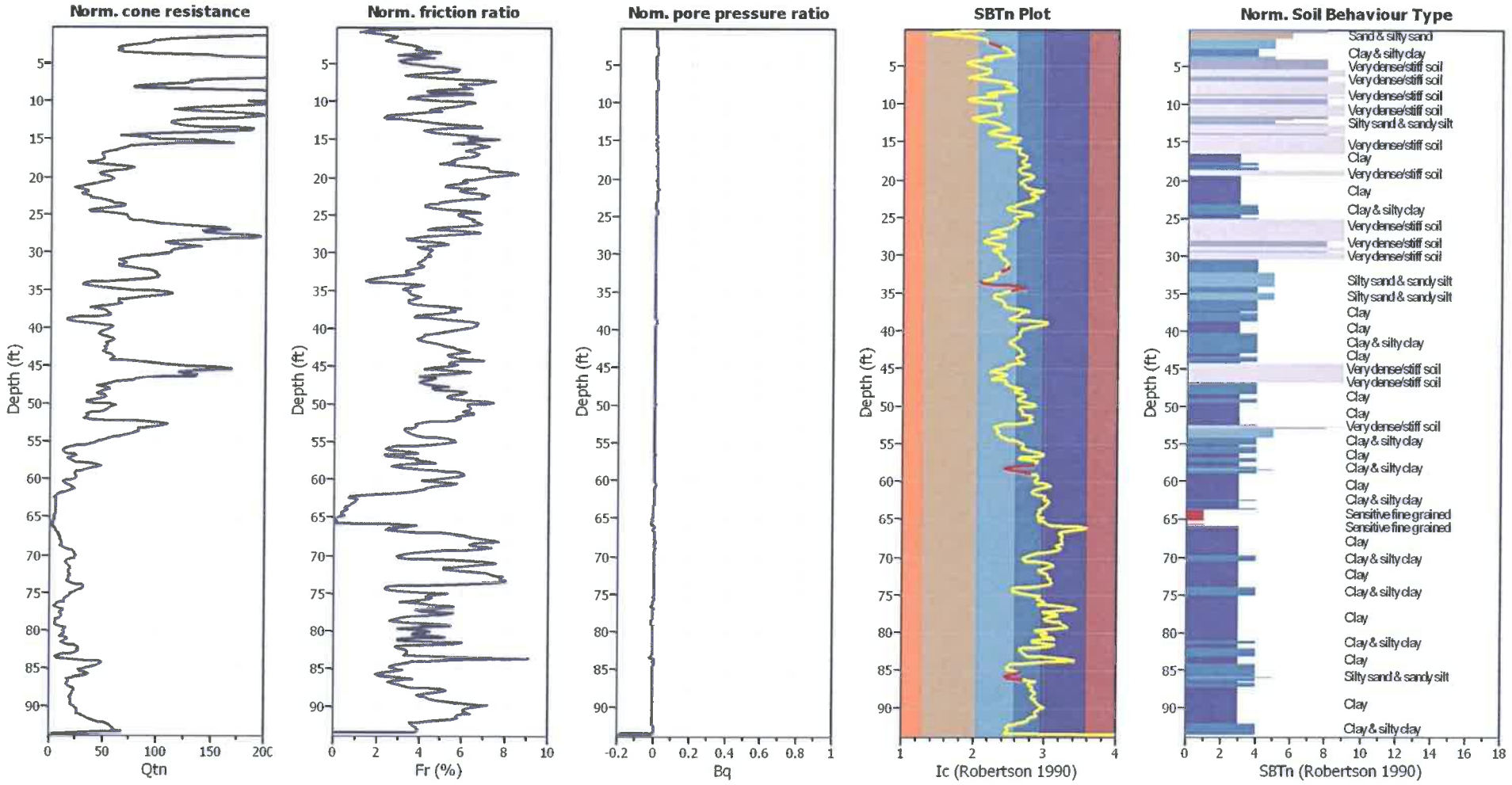
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



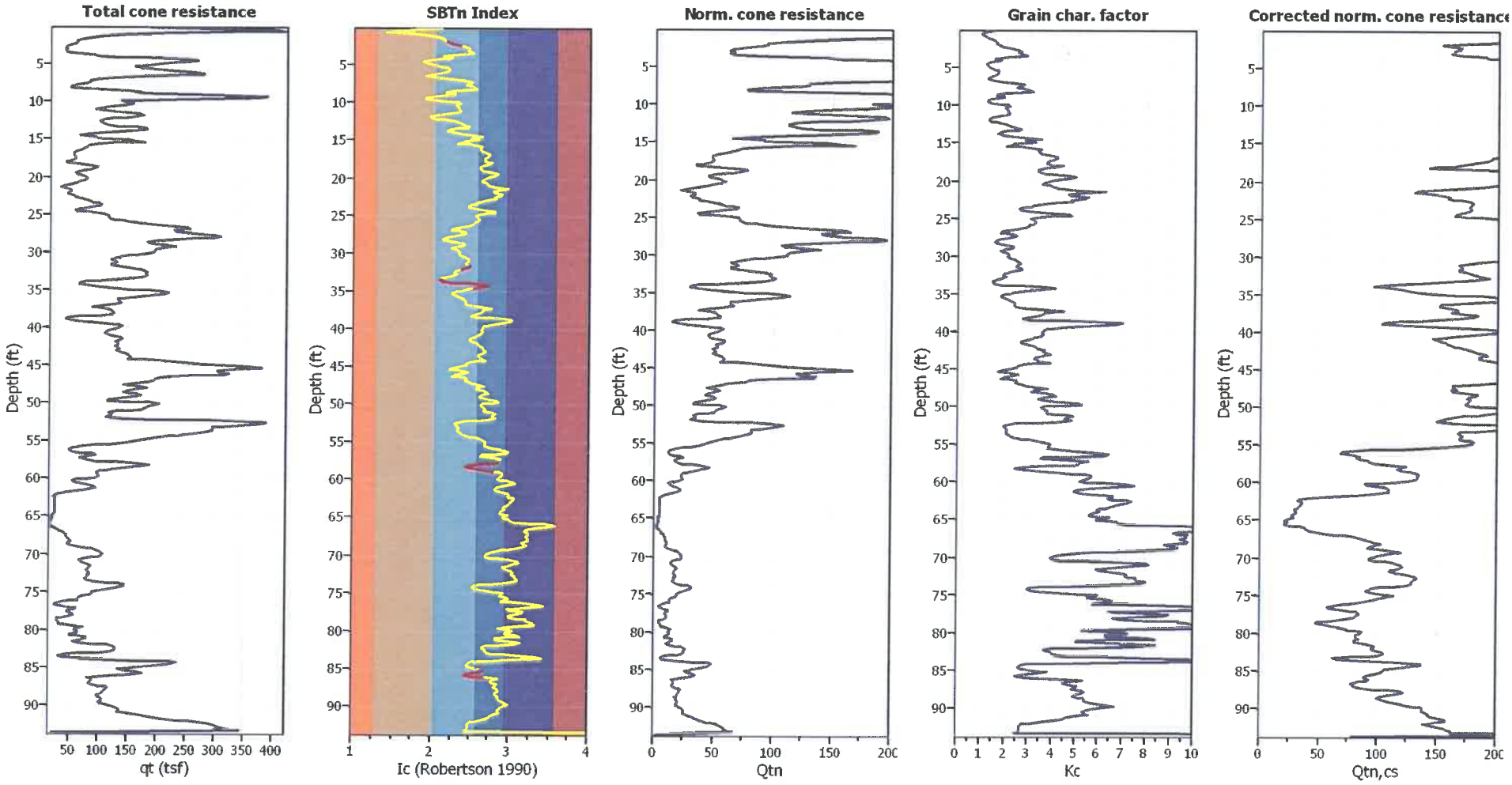
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

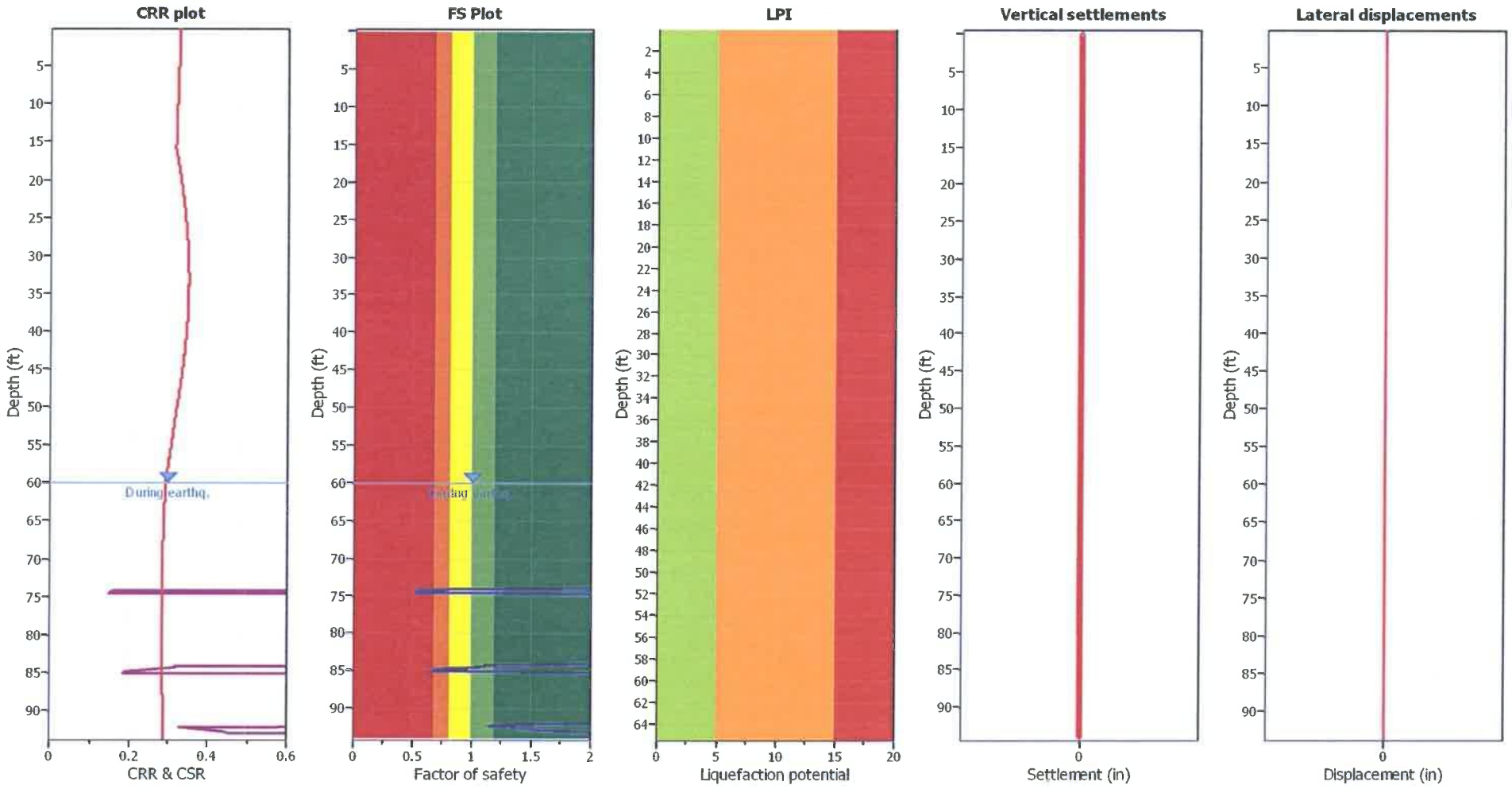
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

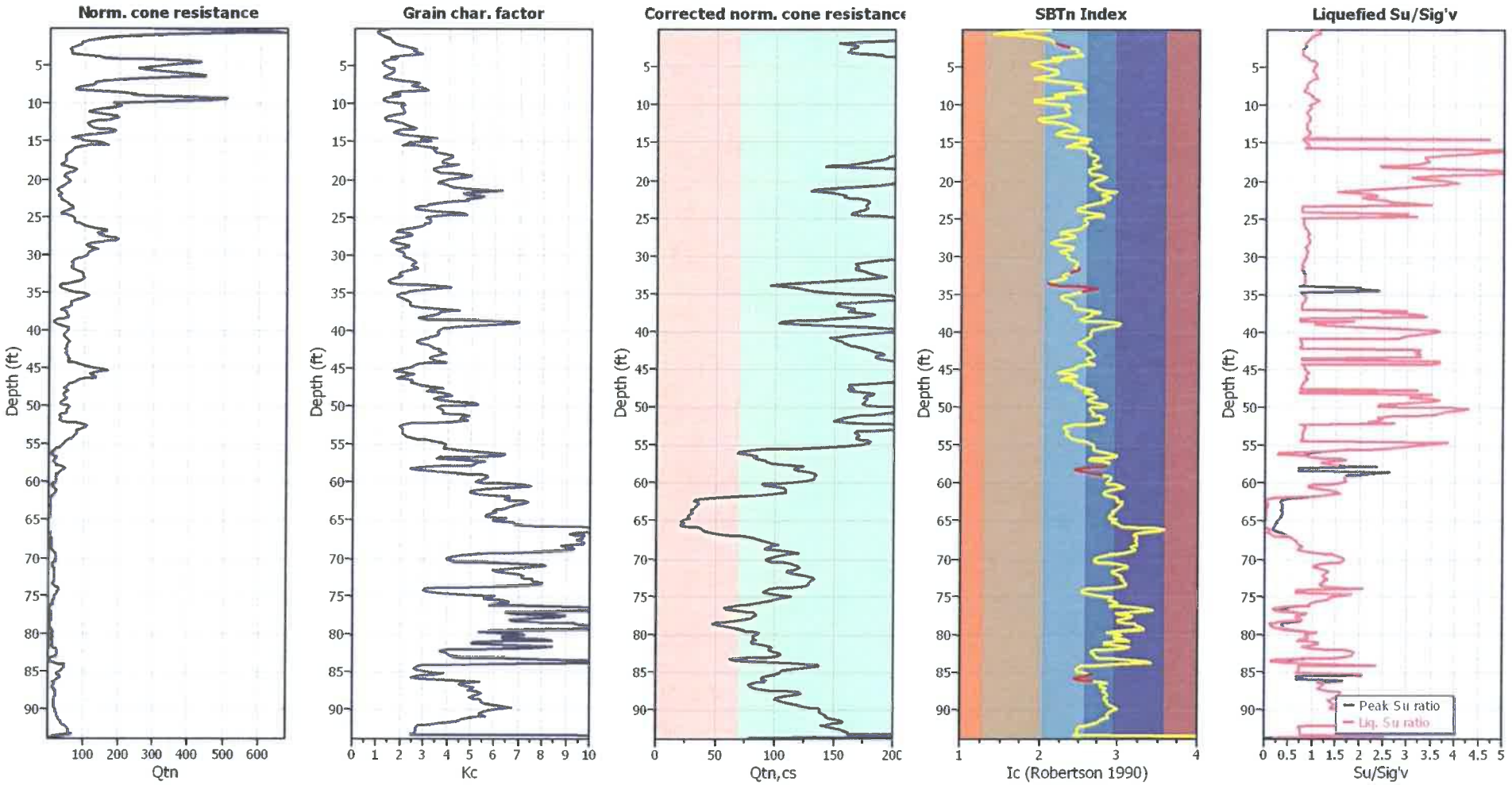
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

E.32

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	60.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	60.00 ft	Fill height:	N/A	Limit depth:	N/A

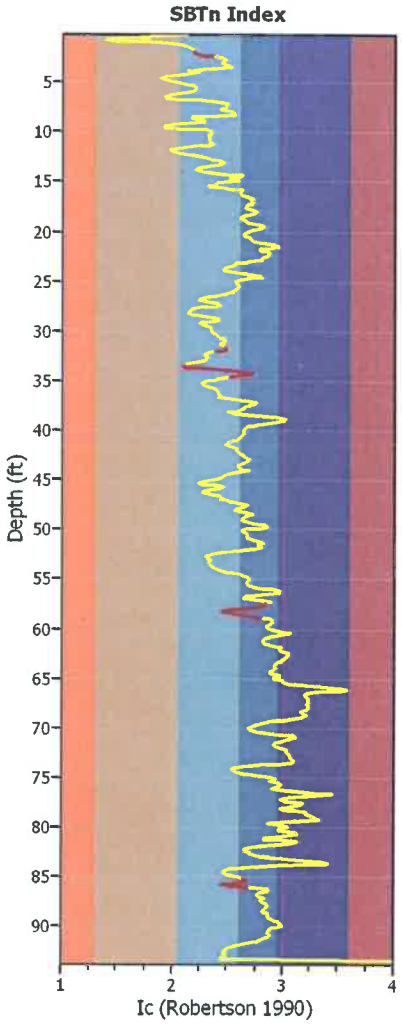
E.33

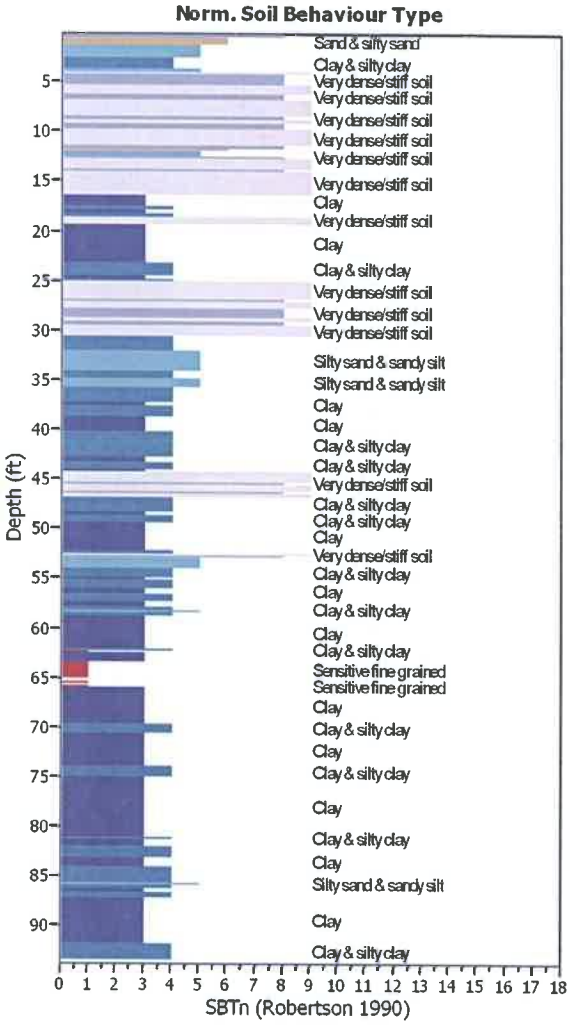
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





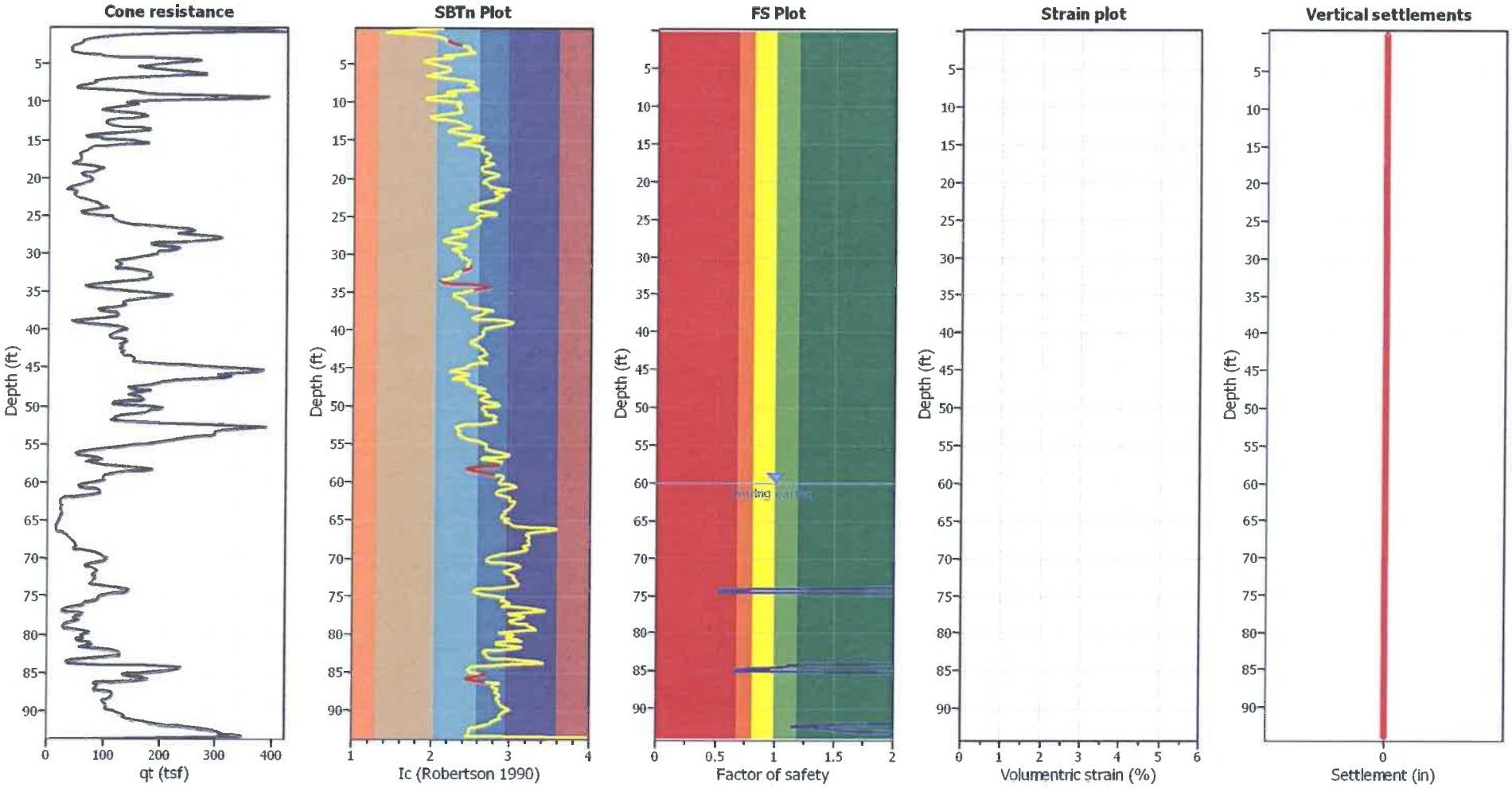
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	572
Total points excluded:	35
Exclusion percentage:	6.12%
Number of layers detected:	8

Estimation of post-earthquake settlements

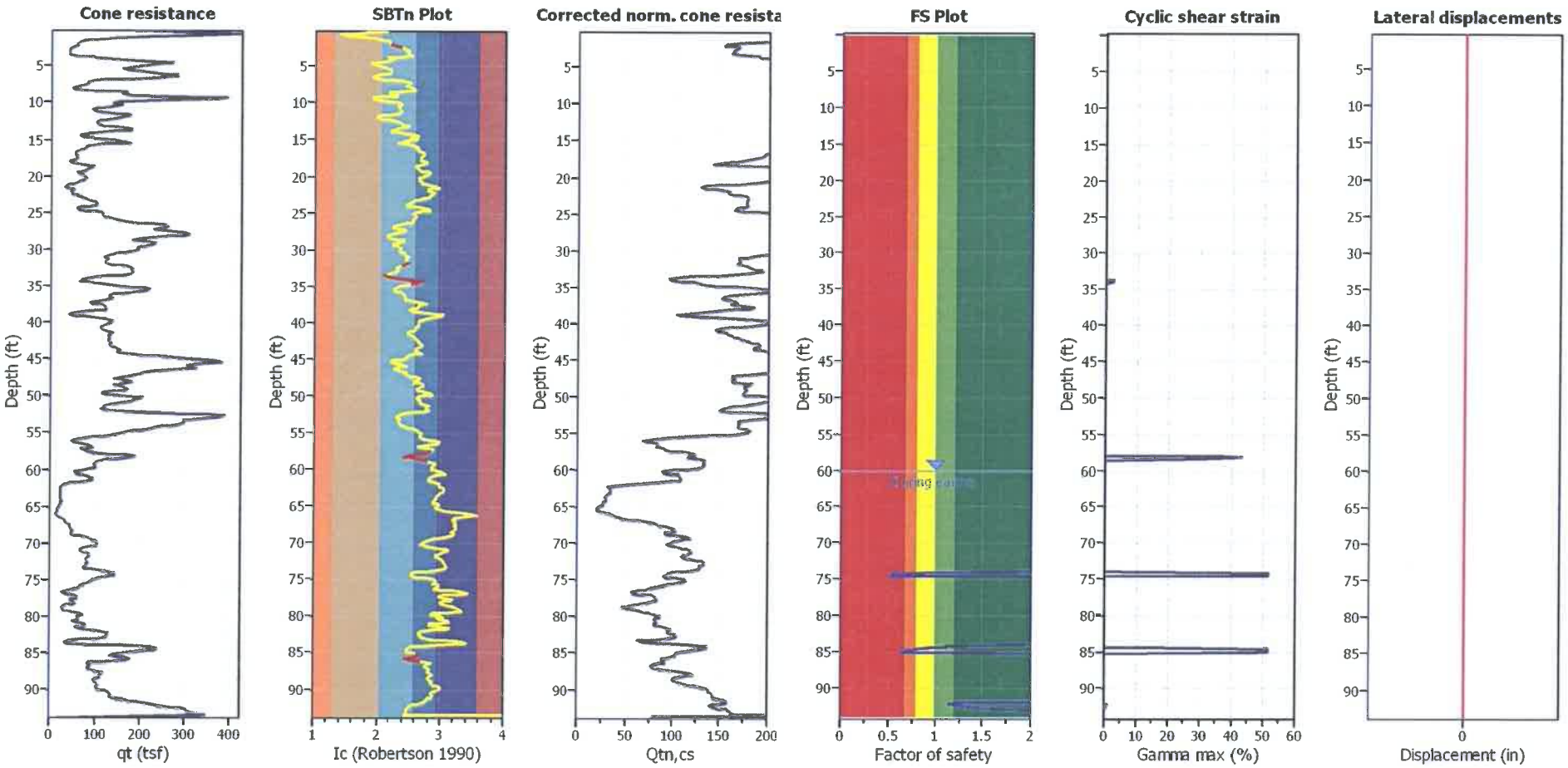


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

E.3.b

STONEY-MILLER CONSULTANTS, INC.
14 Hughes, Suite B-101
Irvine, CA 92618

LIQUEFACTION ANALYSIS REPORT

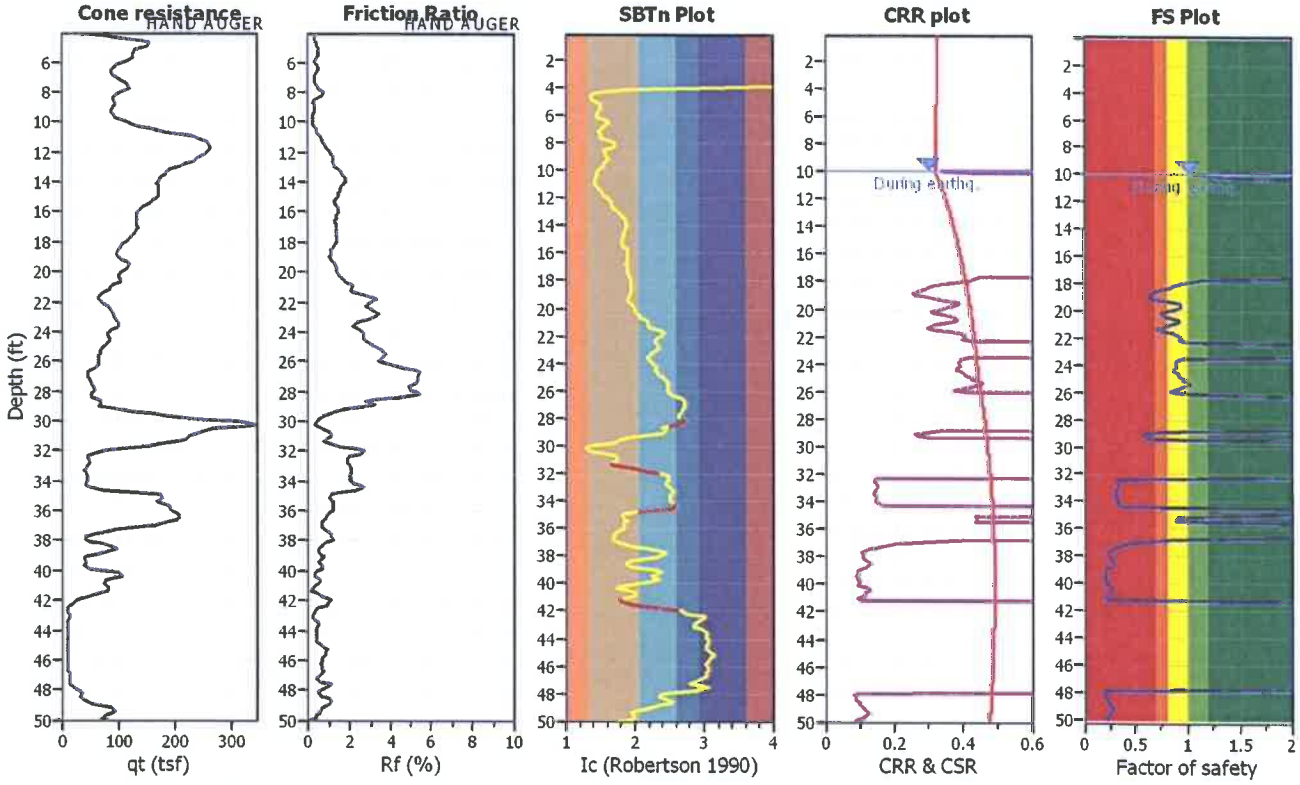
Project title : Pacific Commercenter Lot 13

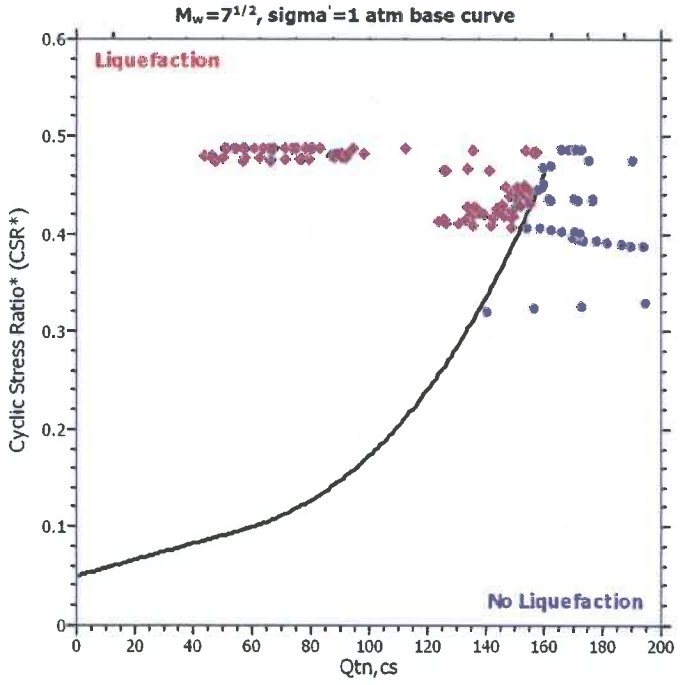
Location : Alton Parkway + Pacific Commercentre Drive

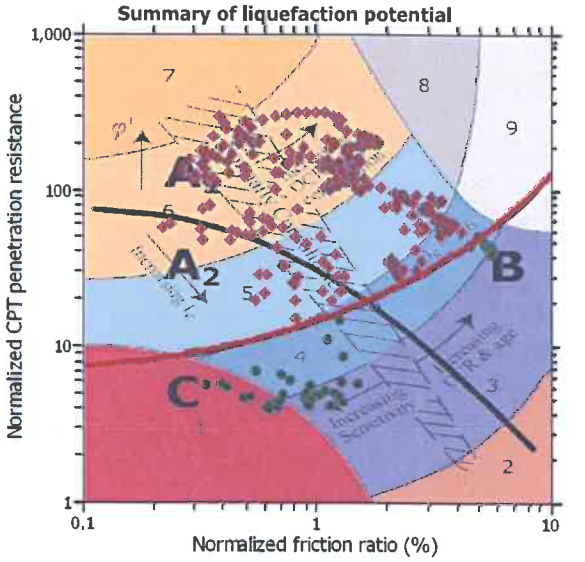
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

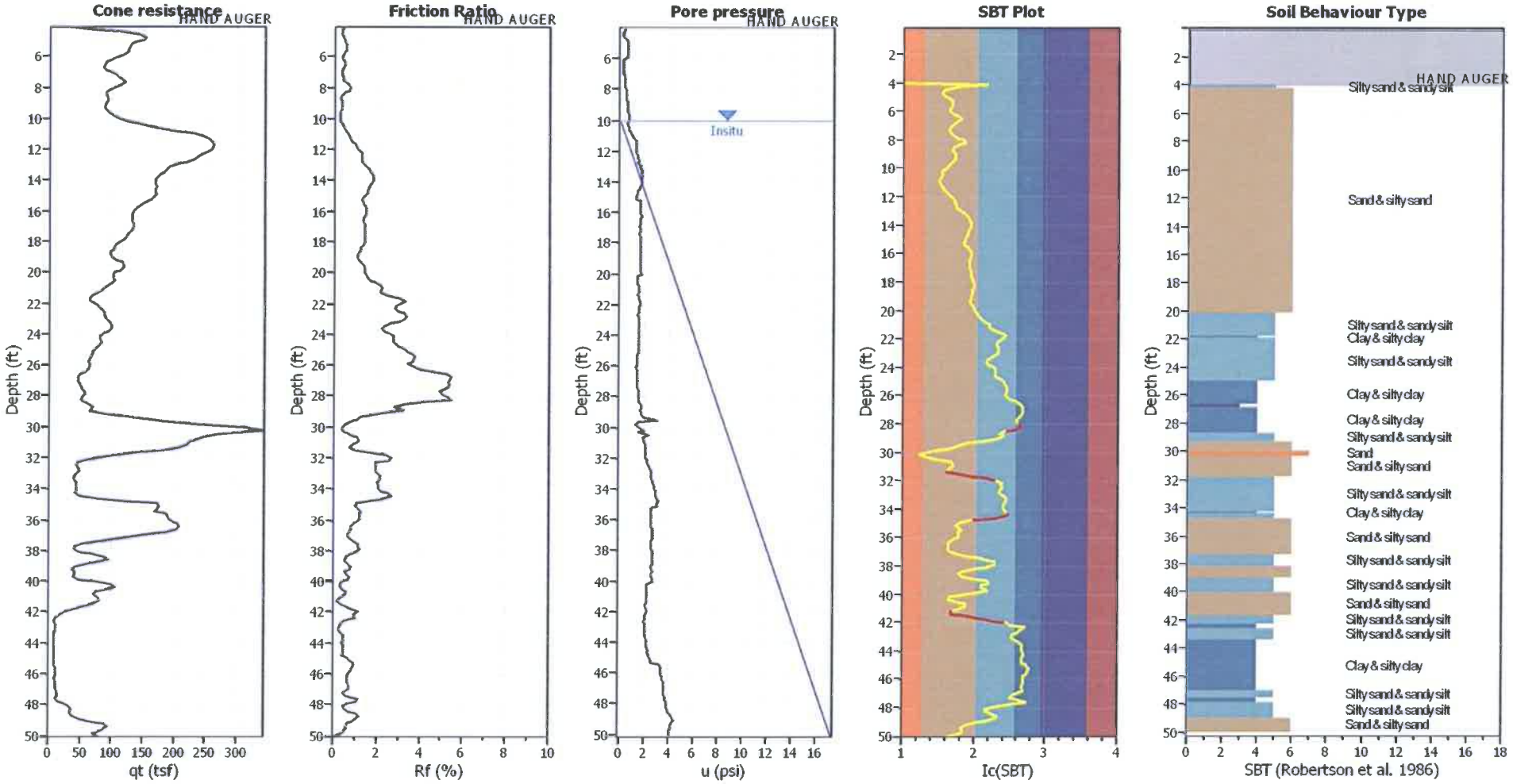






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



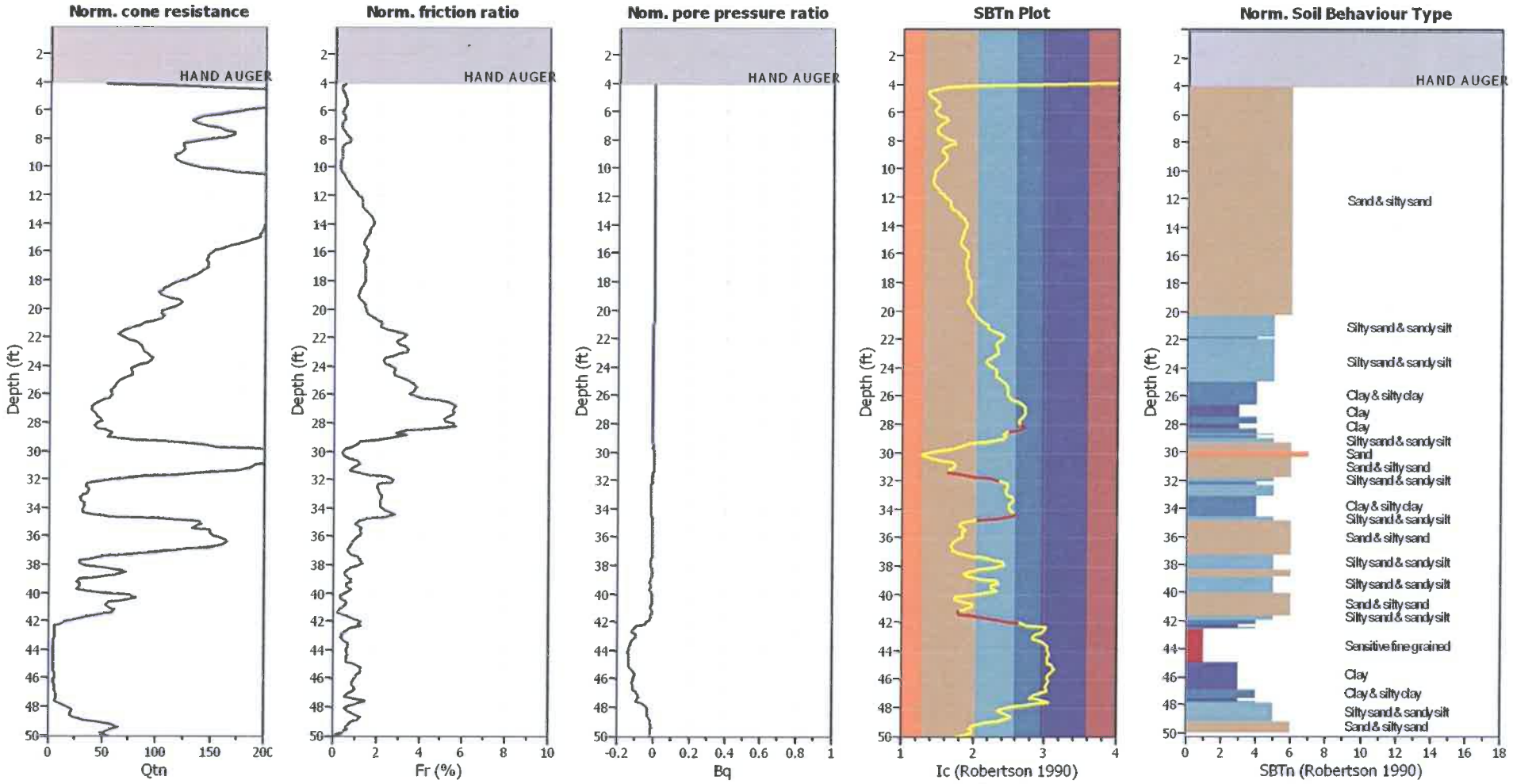
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



Input parameters and analysis data

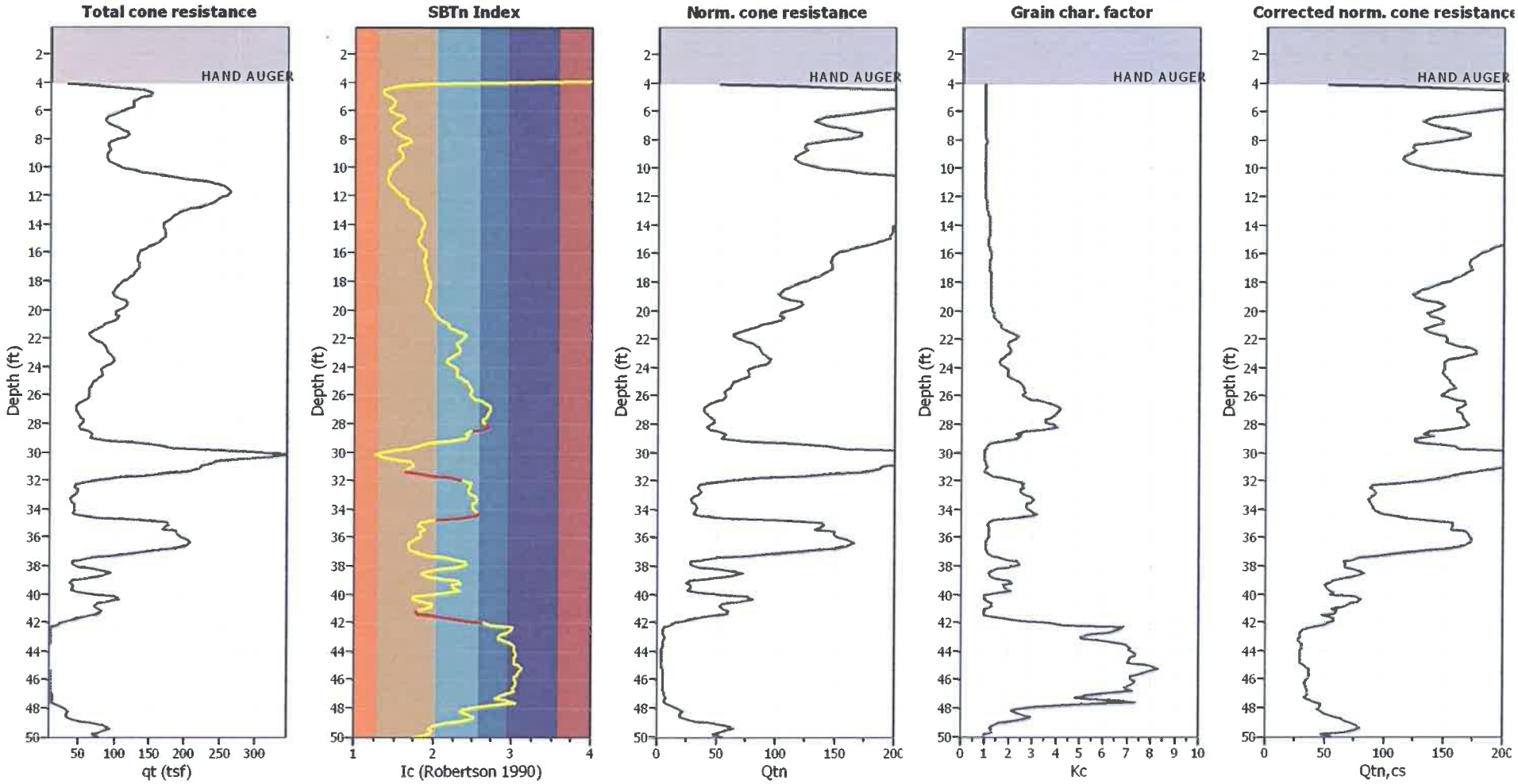
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

E.29

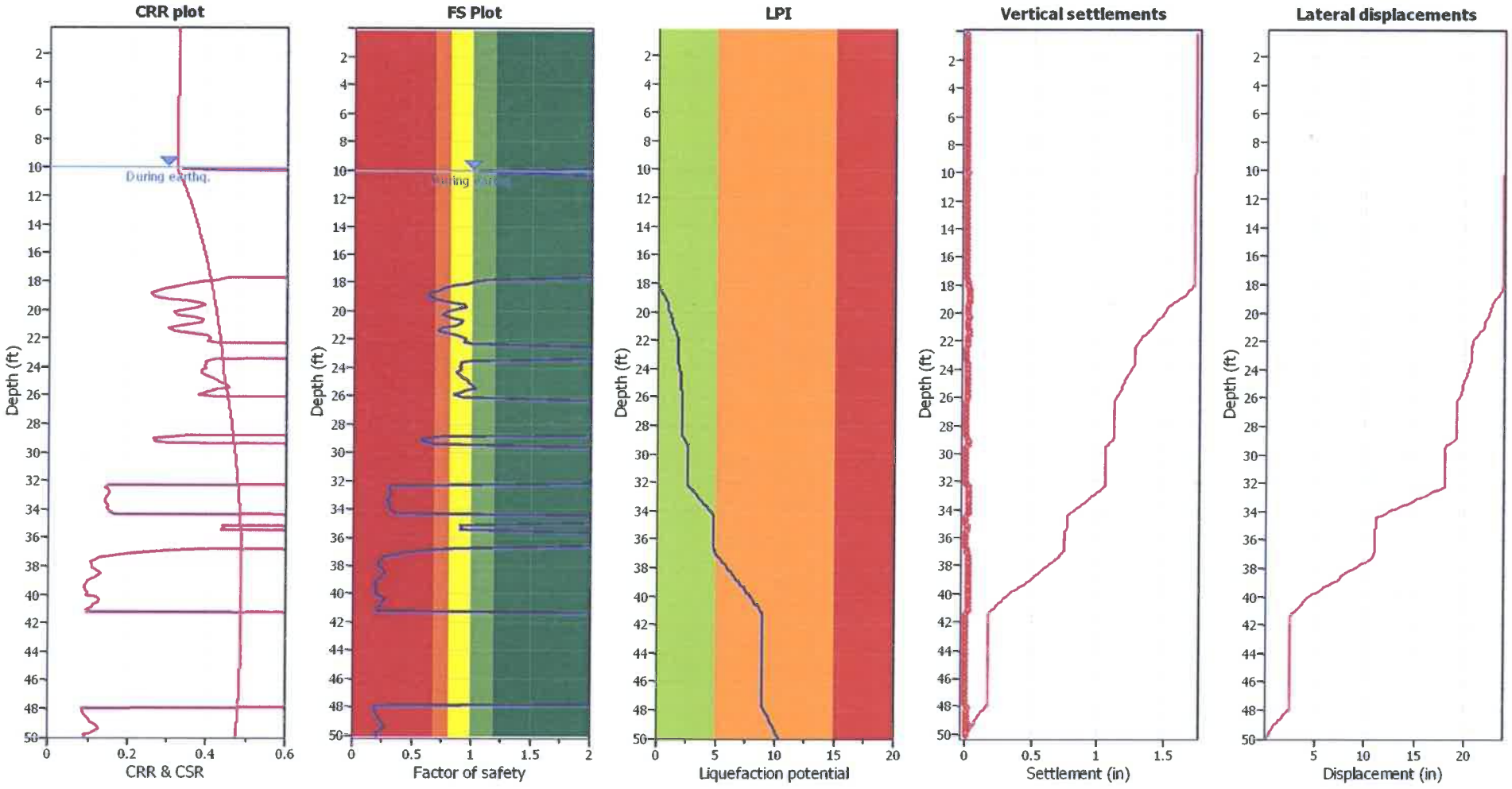
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

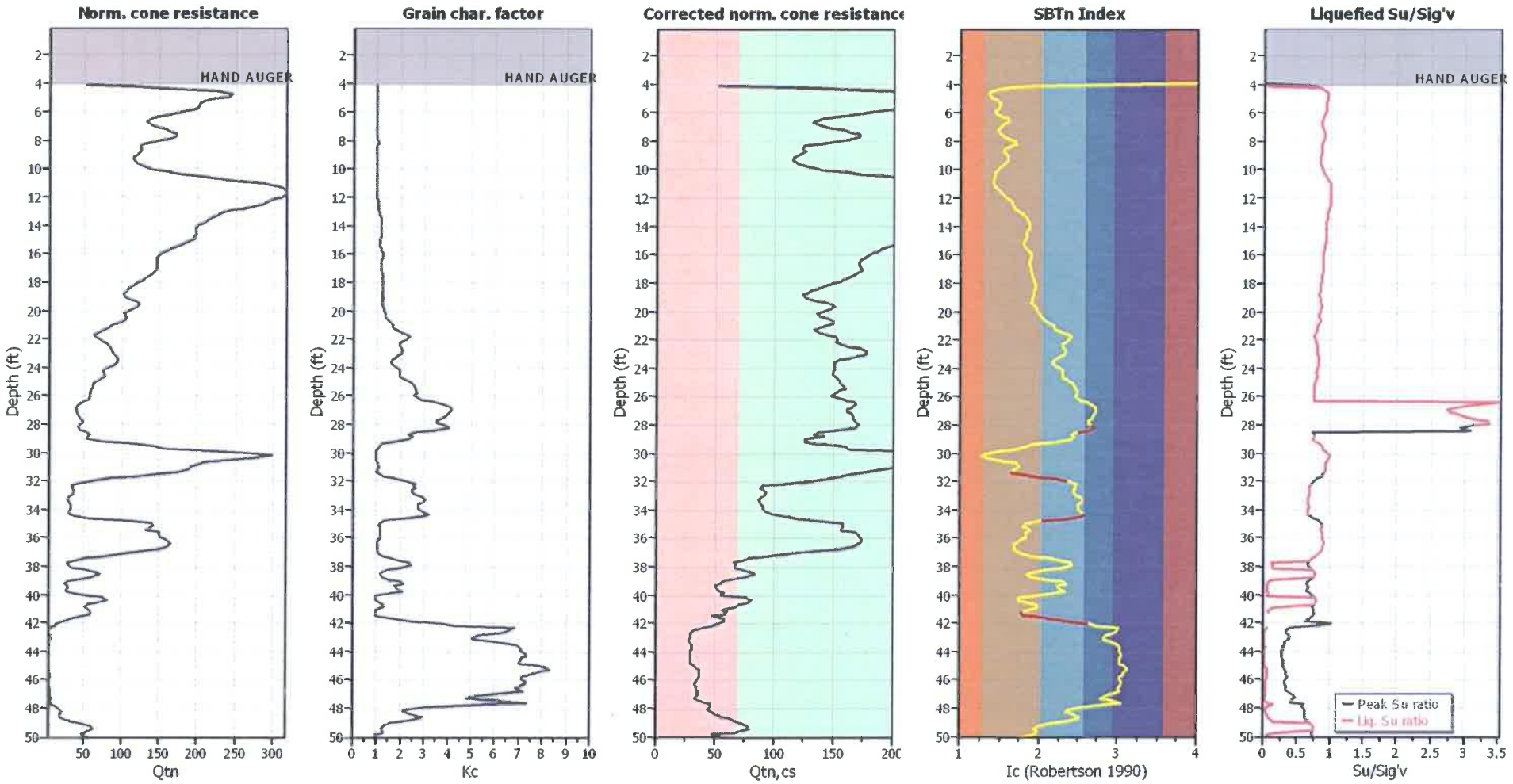
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

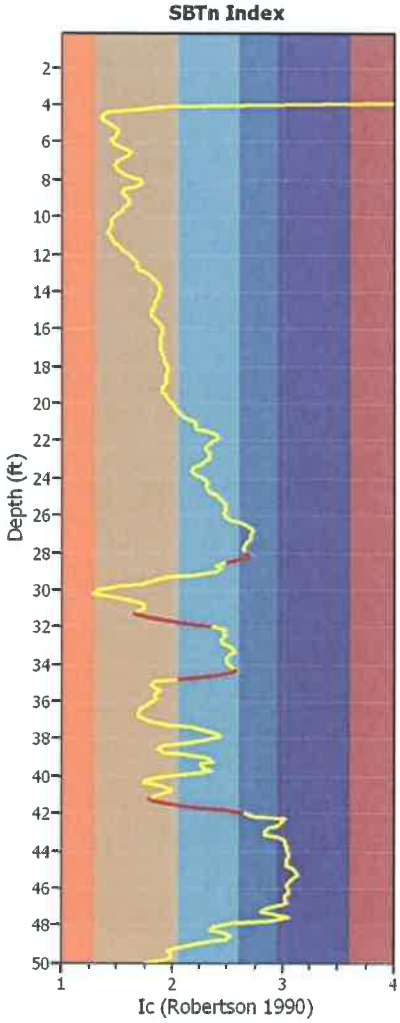
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

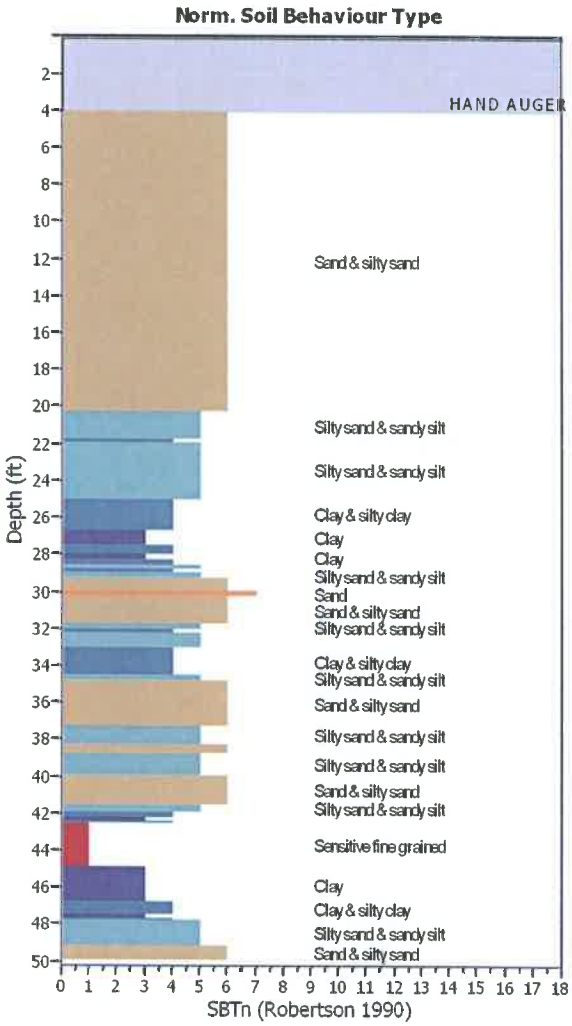
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





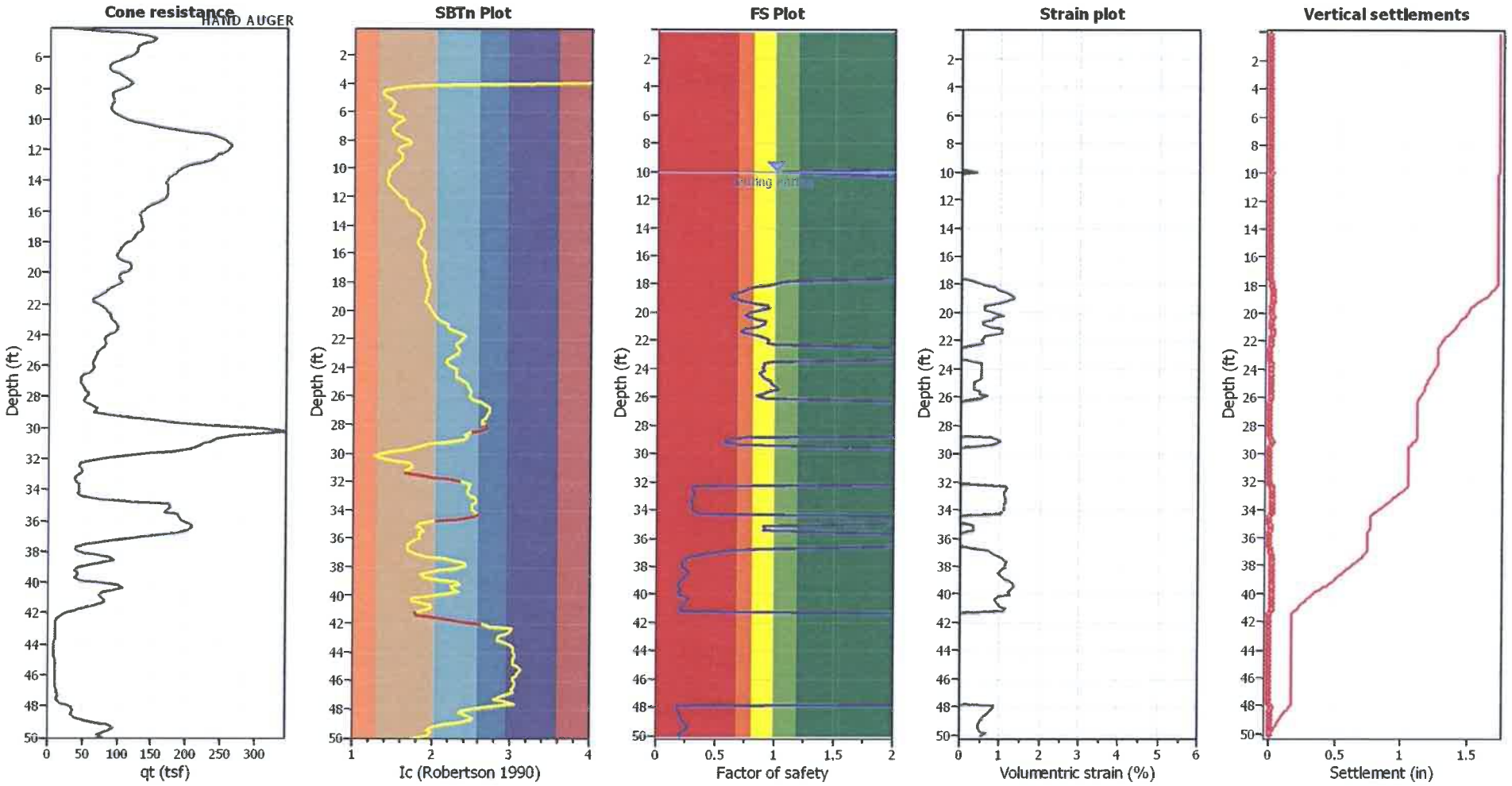
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	305
Total points excluded:	19
Exclusion percentage:	6.23%
Number of layers detected:	4

Estimation of post-earthquake settlements

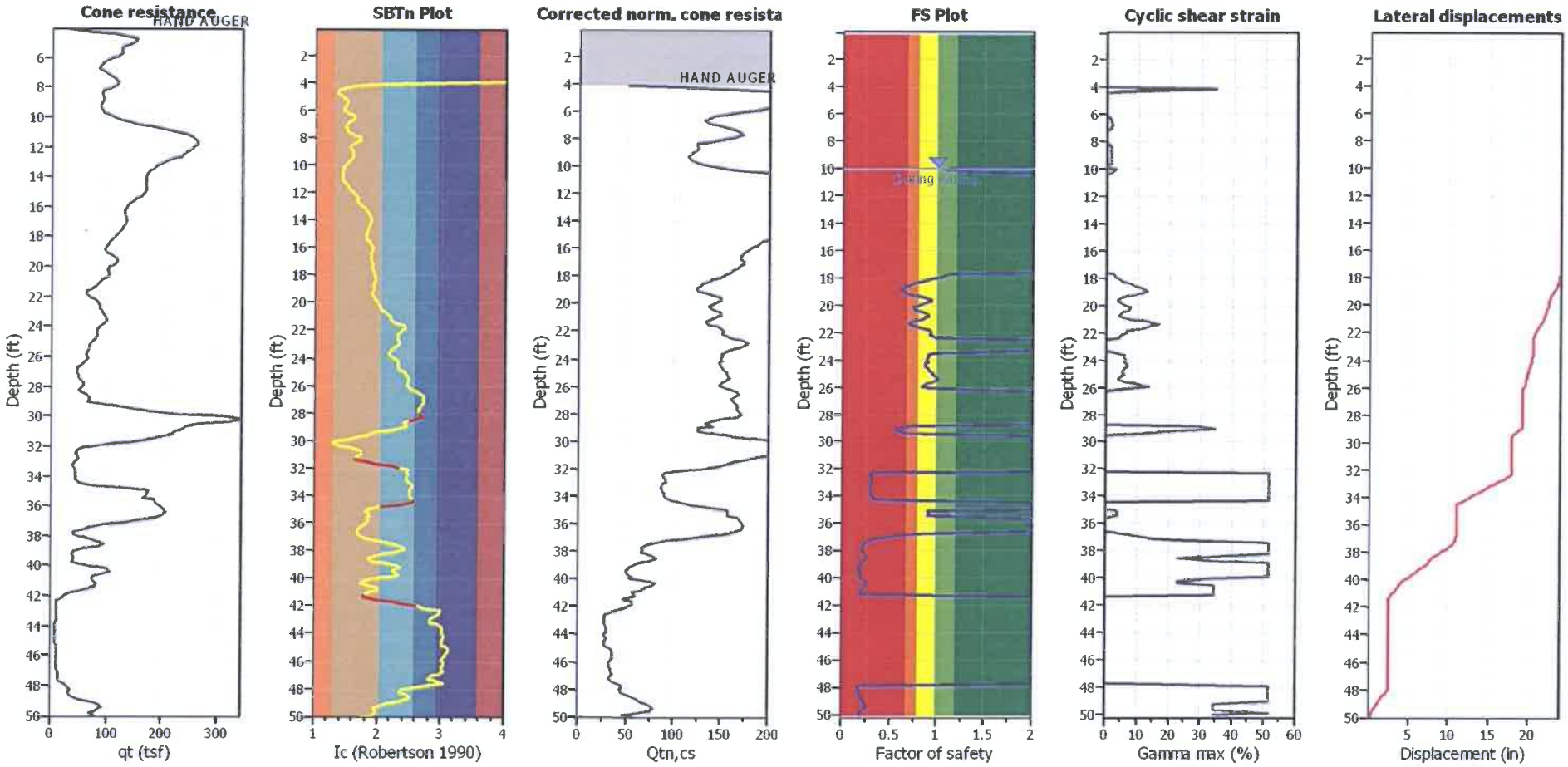


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$(Q_{tn,cs})$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

STONEY-MILLER CONSULTANTS, INC.
14 Hughes, Suite B-101
Irvine, CA 92618

LIQUEFACTION ANALYSIS REPORT

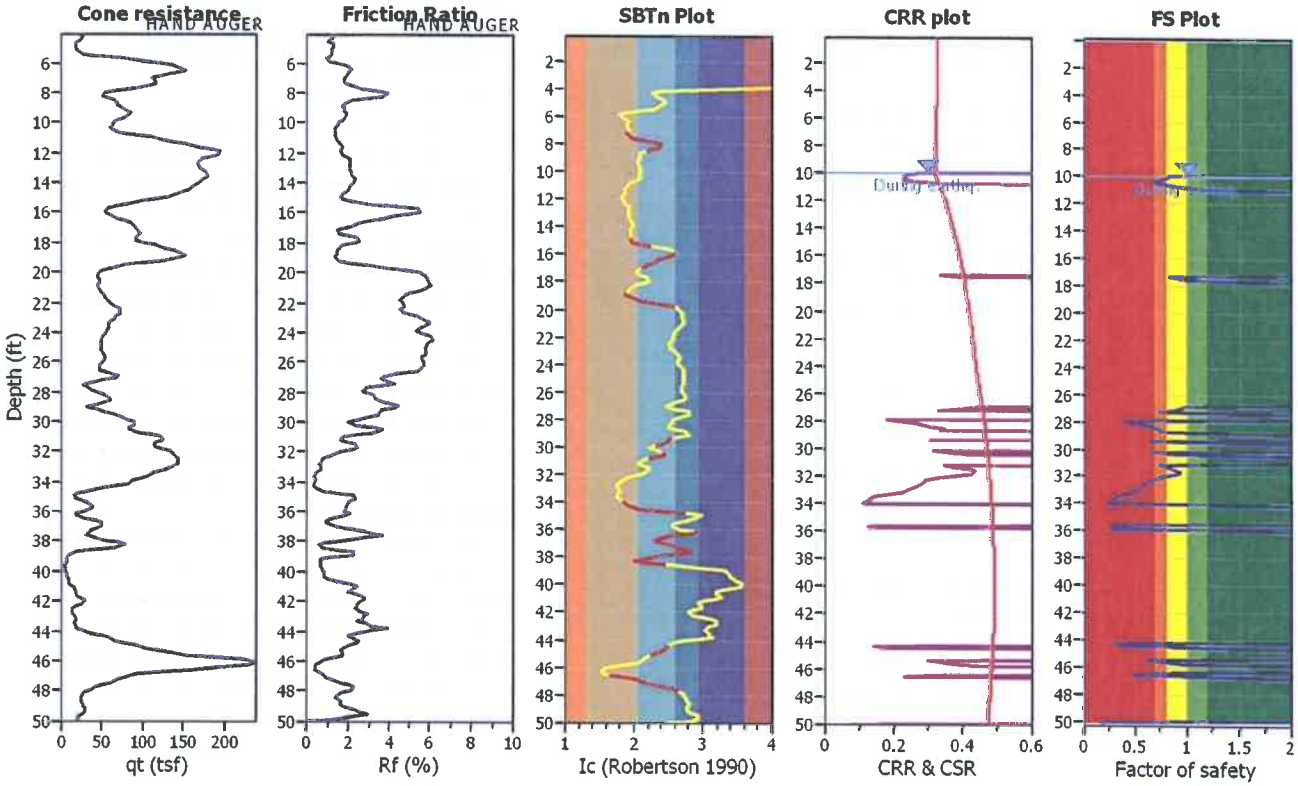
Project title : Pacific Commercenter Lot 13

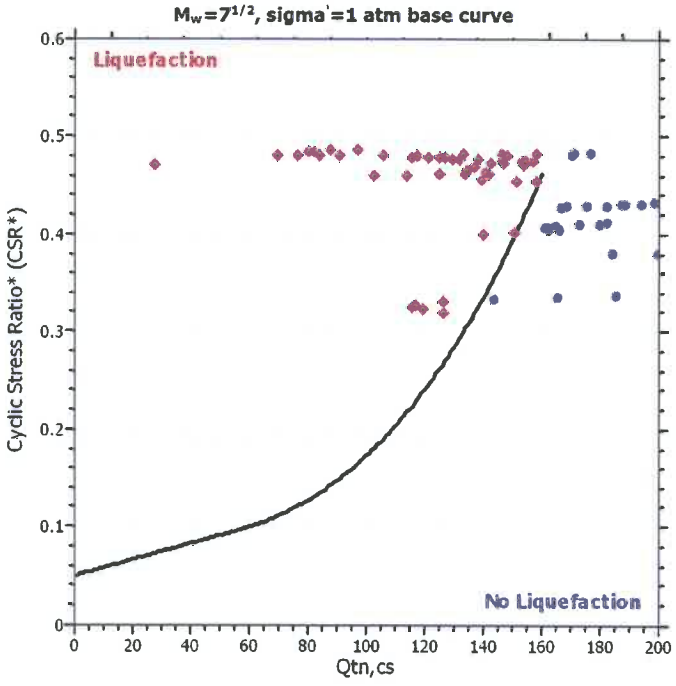
Location : Alton Parkway + Pacific Commercentre Drive

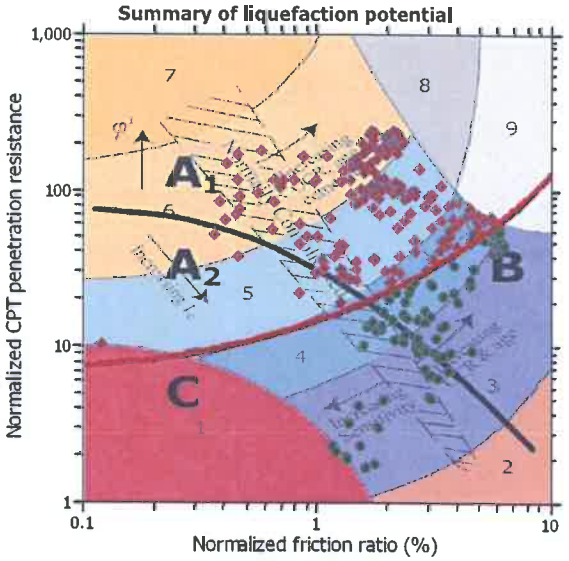
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

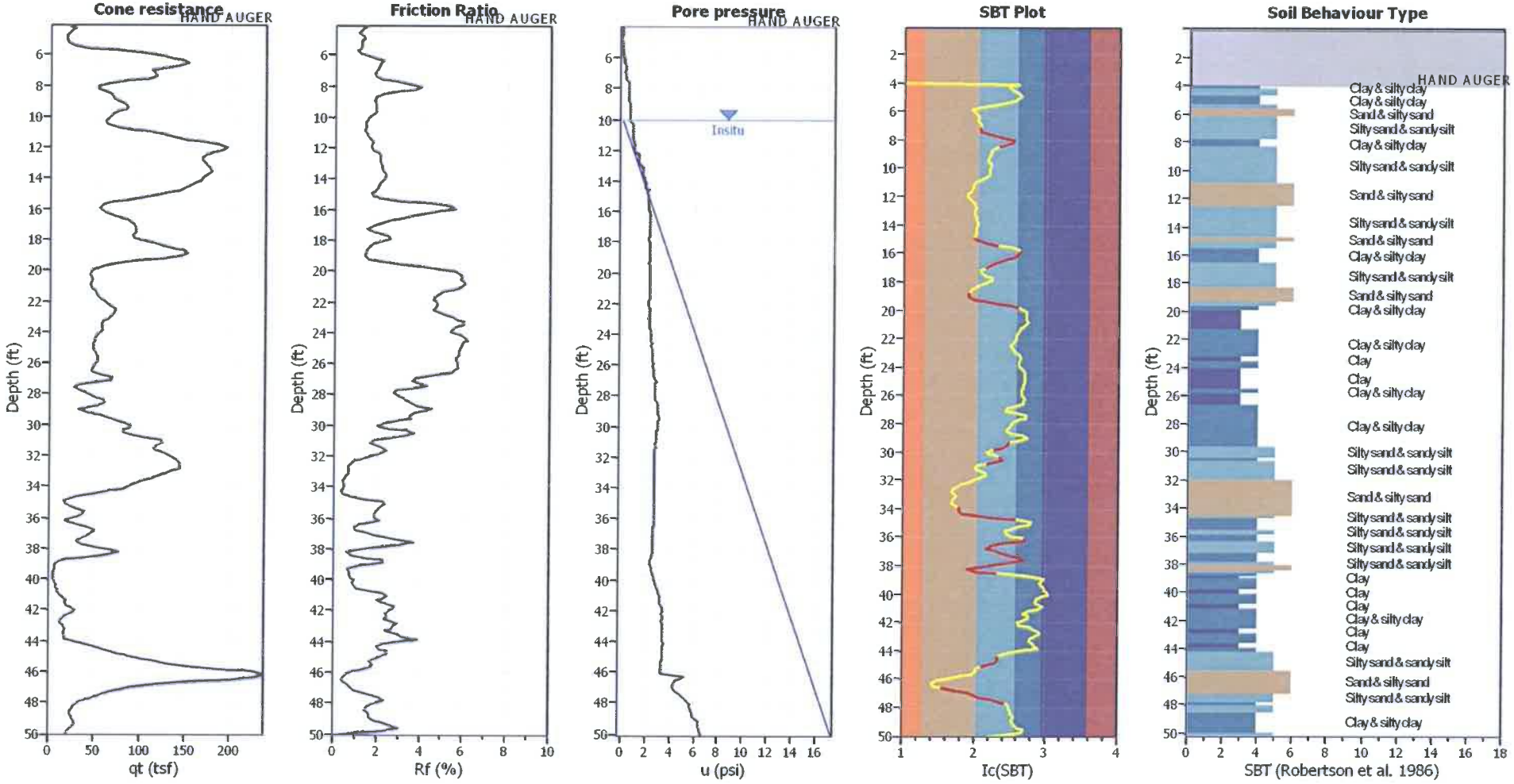






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

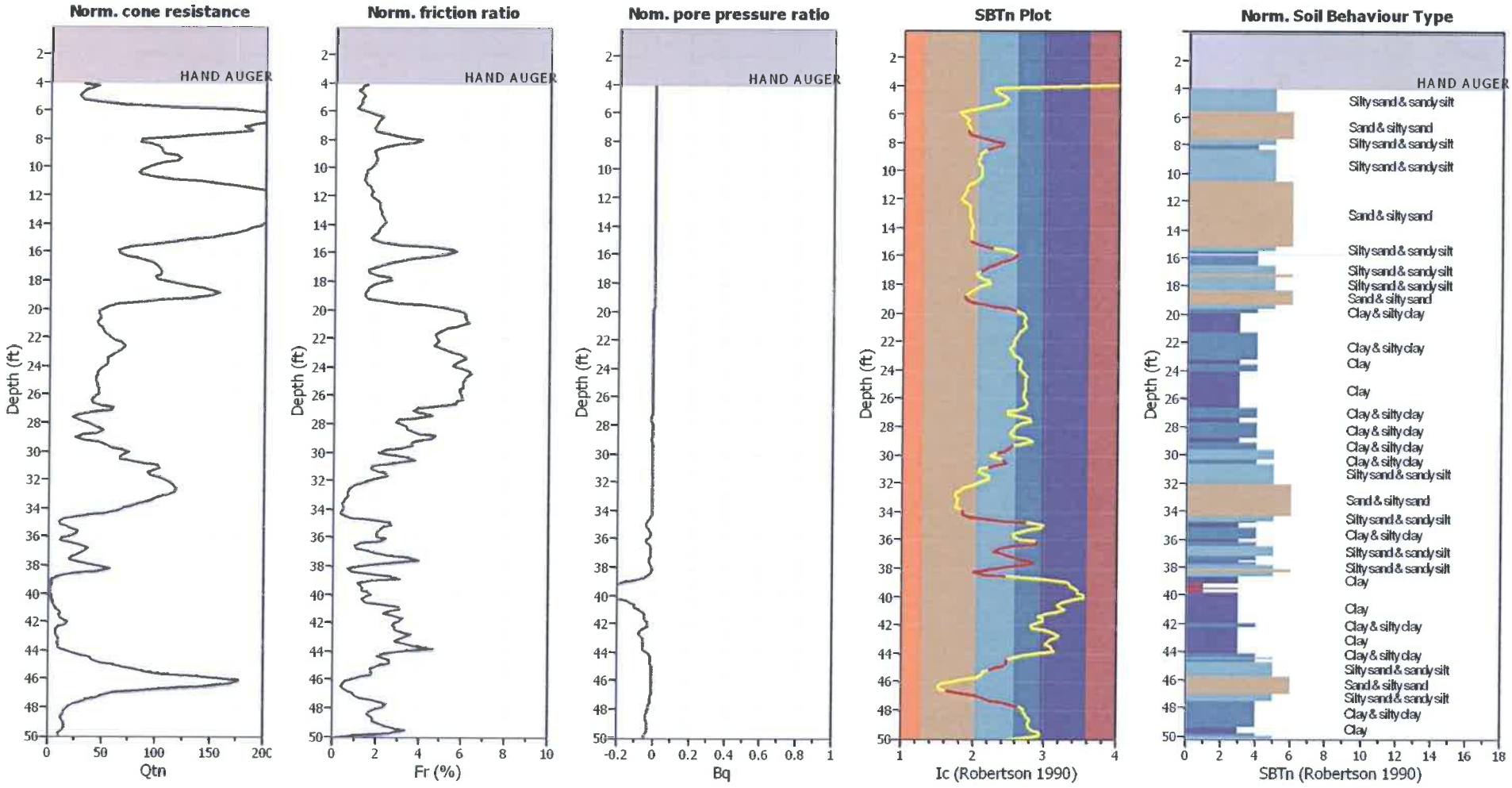
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

E.4b

CPT basic interpretation plots (normalized)



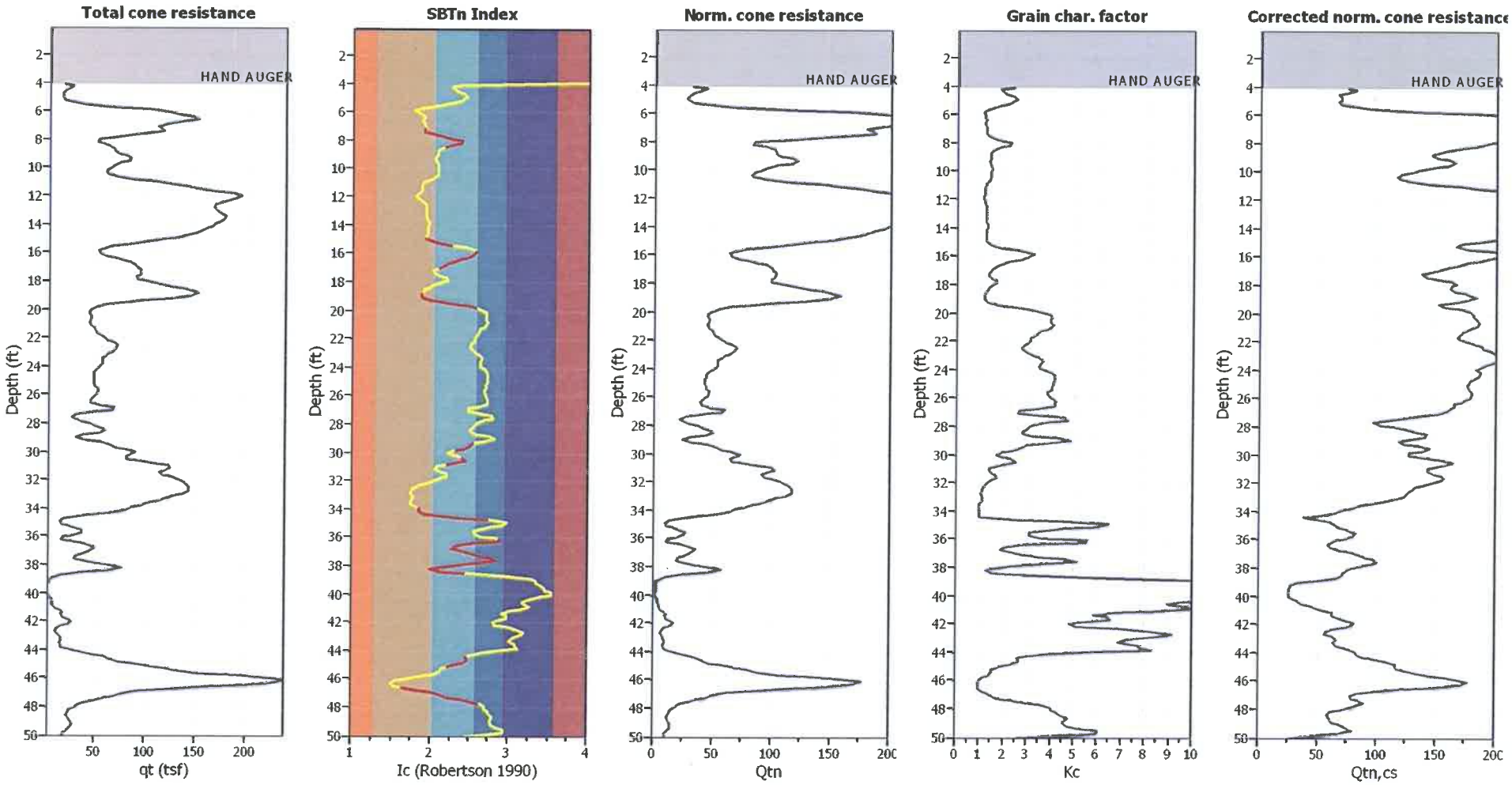
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)

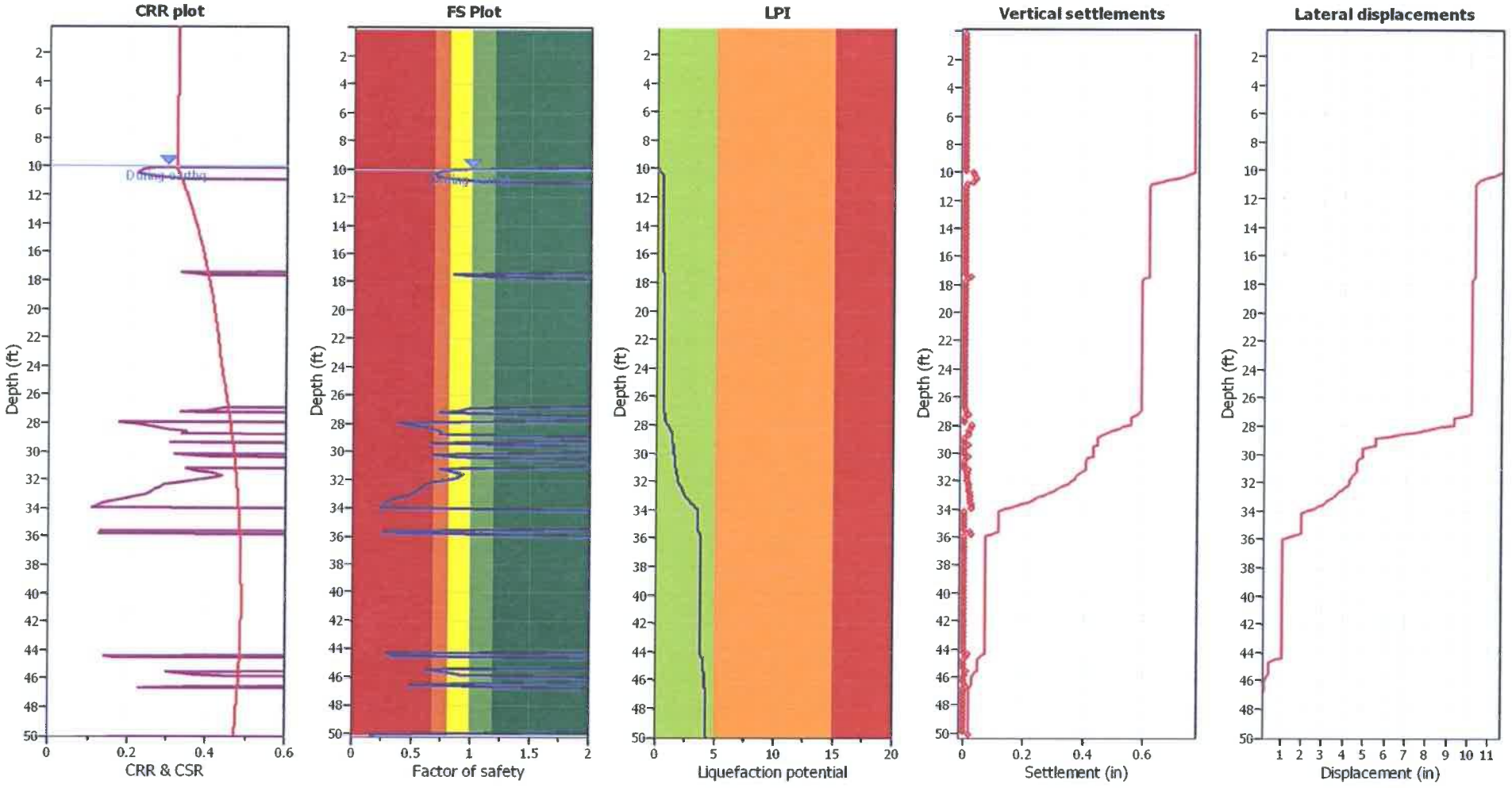


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

E.46

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

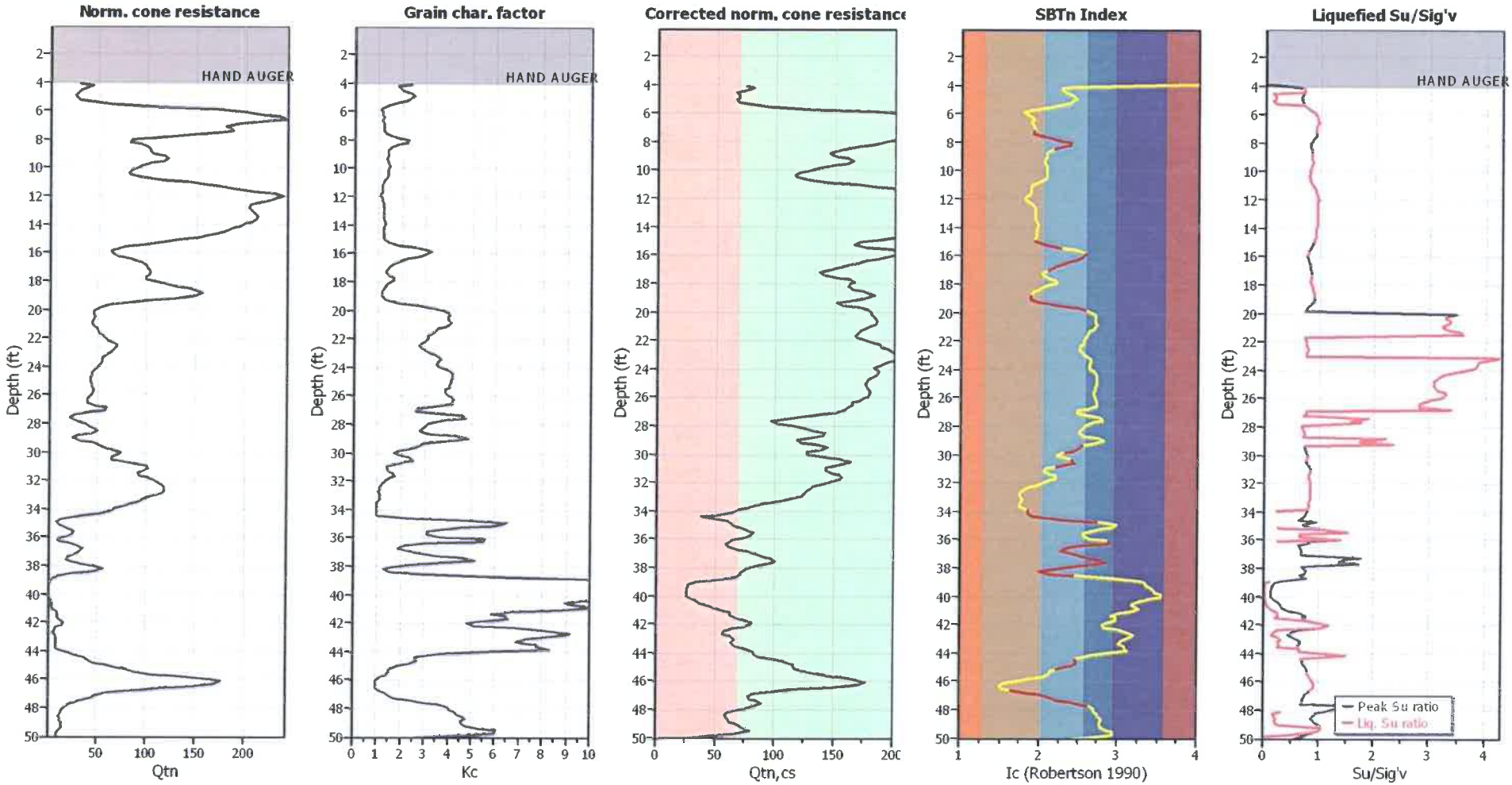
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

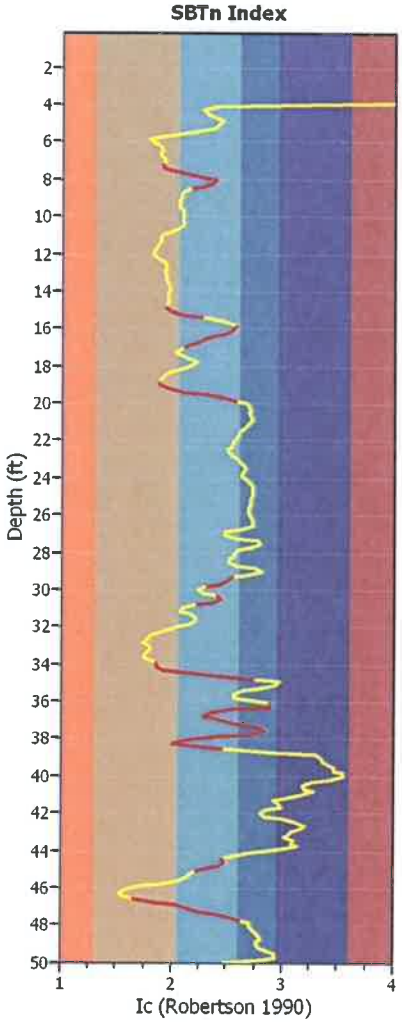
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

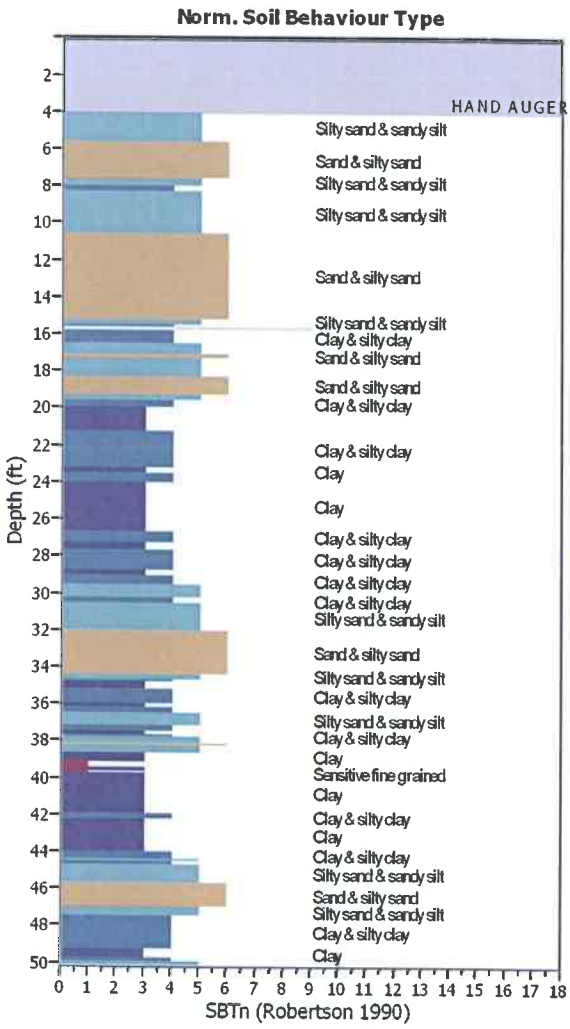
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





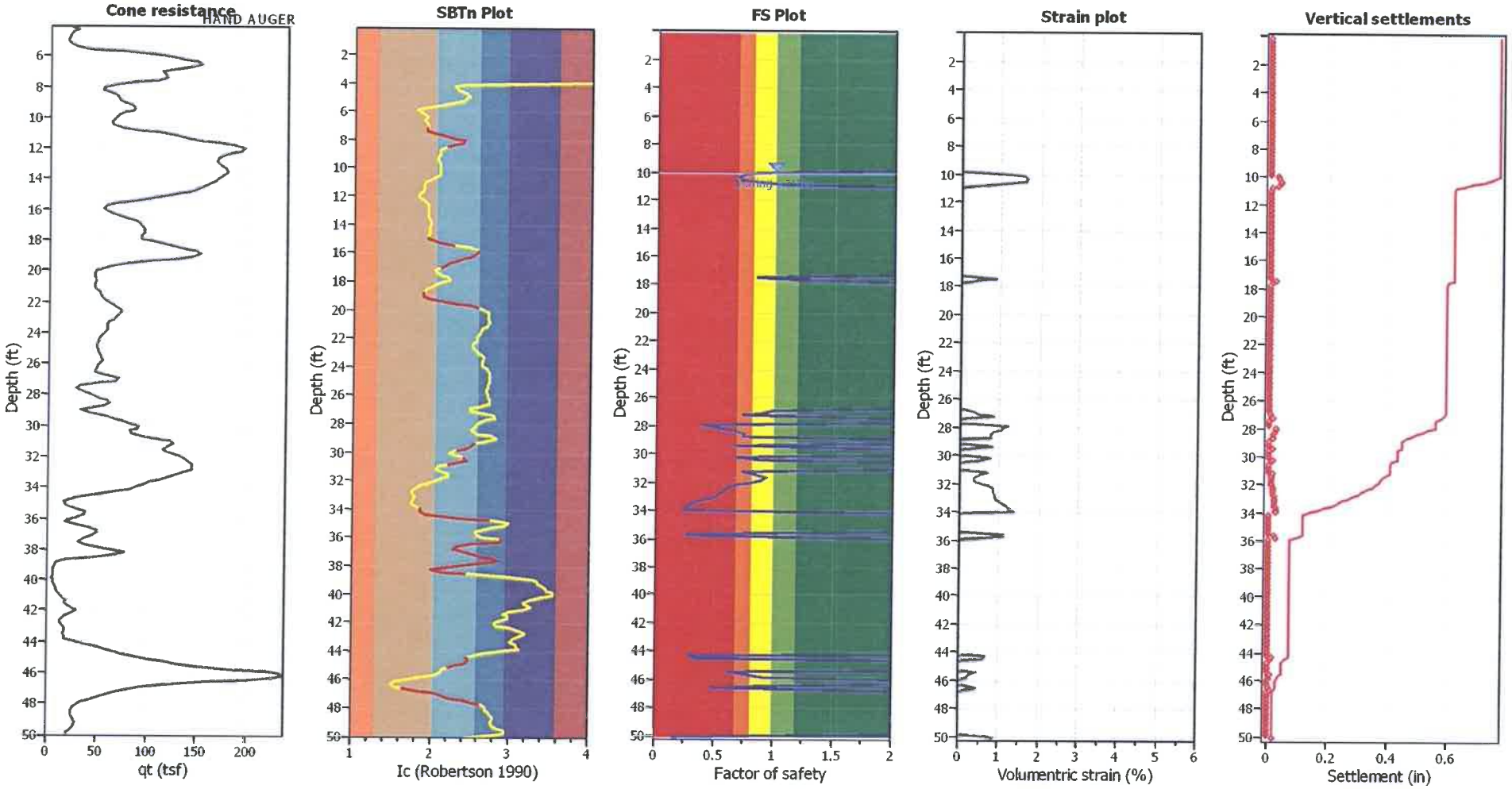
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	305
Total points excluded:	74
Exclusion percentage:	24.26%
Number of layers detected:	14

Estimation of post-earthquake settlements

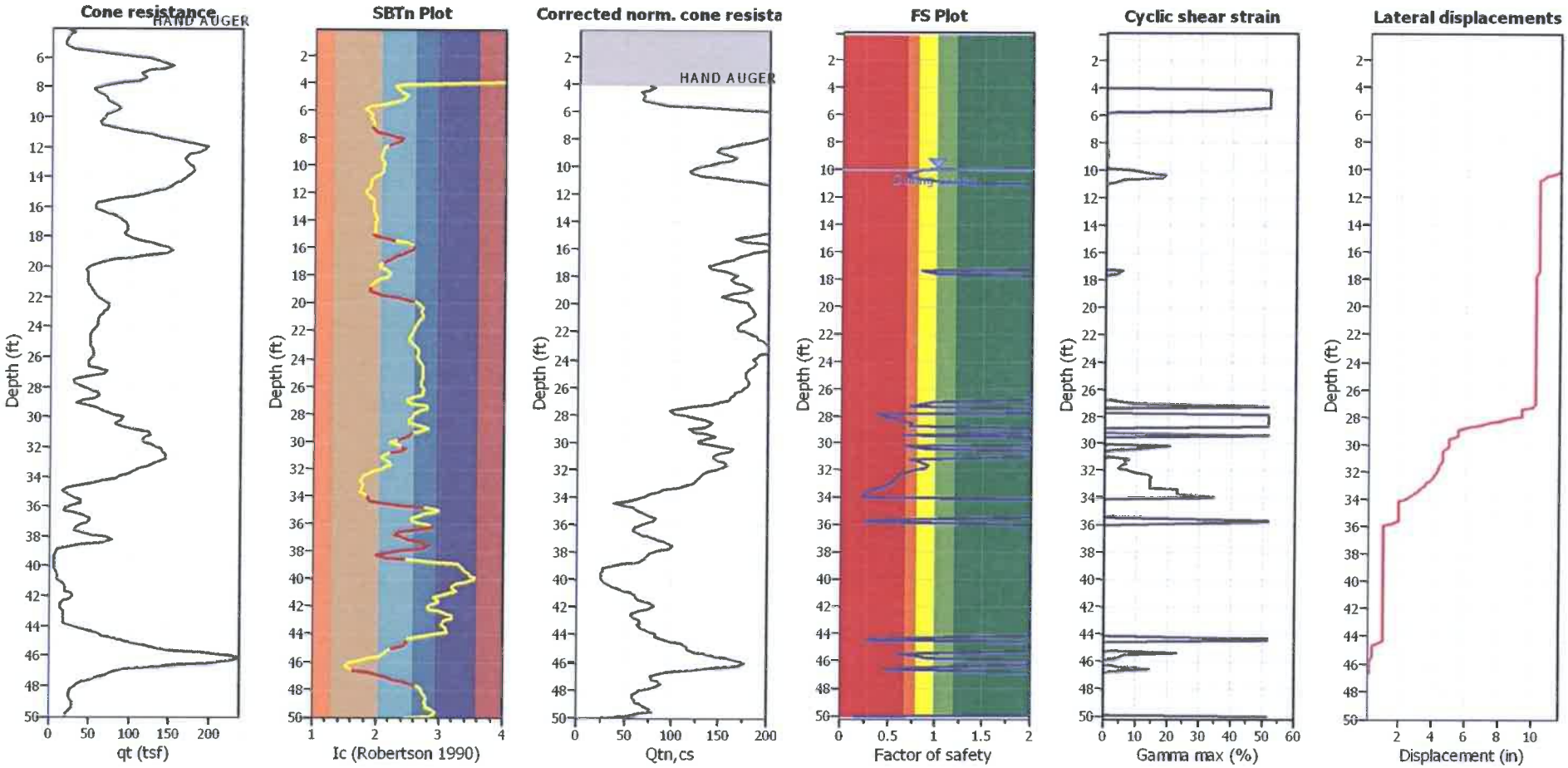


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
F_s:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 1.00 %)



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

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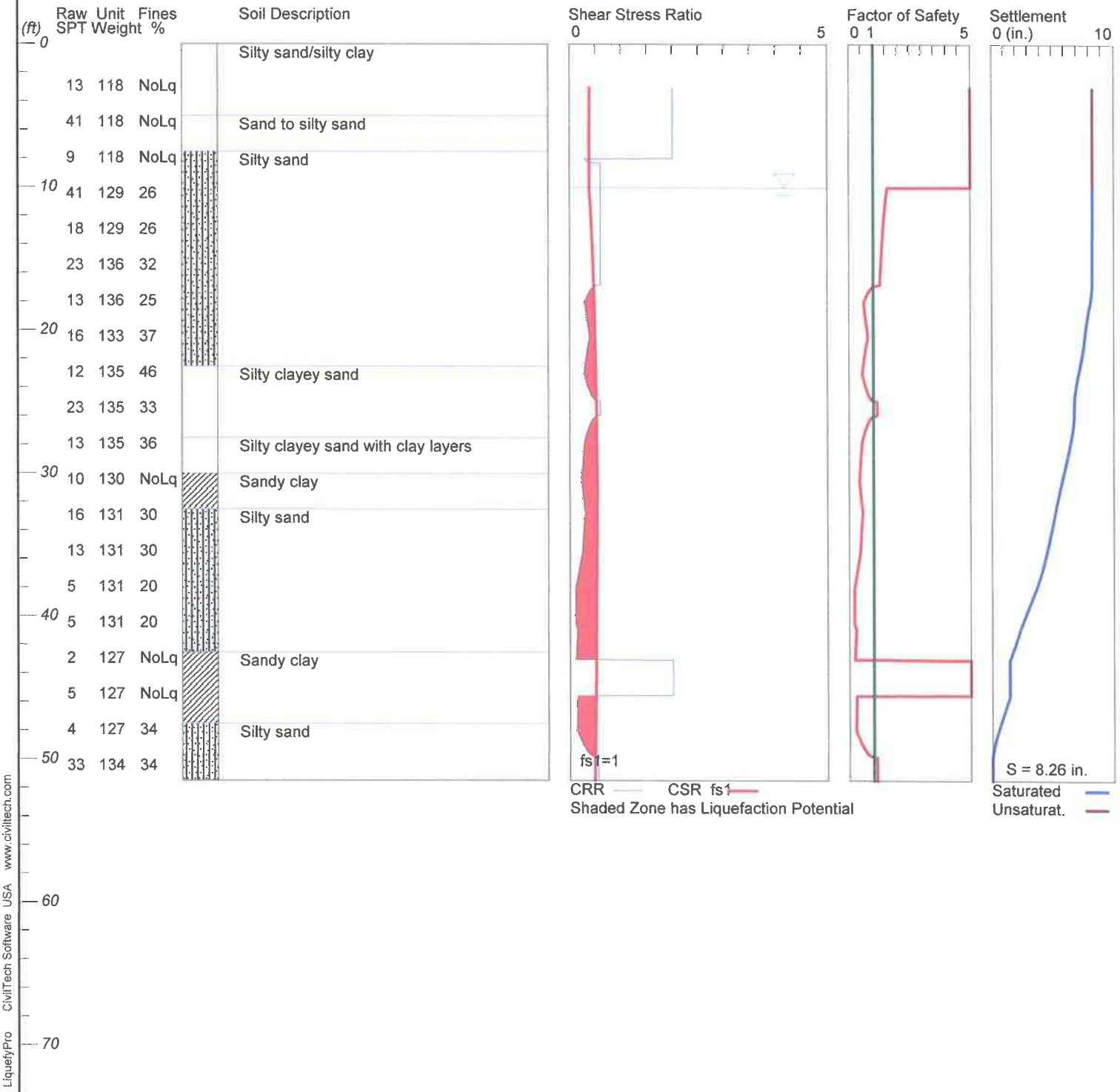
E.53

LIQUEFACTION ANALYSIS

Pacific Center, Lot 13, Geotek Logs

Hole No.=B-1 Water Depth=10 ft

Magnitude=7
Acceleration=.595g



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Input File Name: C:\Users\gina\Desktop\Pac lot13-b1-2.liq
Title: Pacific Center, Lot 13, Geotek Logs
Subtitle: Geotek B-1

Surface Elev.=
Hole No.=B-1
Depth of Hole= 51.50 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 30.80 ft
Max. Acceleration= 0.6 g
Earthquake Magnitude= 7.00

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=51.50 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 30.80 ft
Max. Acceleration=0.6 g
Earthquake Magnitude=7.00
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=1)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
3.00	13.00	118.00	NoLiq
5.50	41.00	118.00	NoLiq
8.00	9.00	118.00	NoLiq
10.50	41.00	129.00	26.00
13.00	18.00	129.00	26.00
15.50	23.00	136.00	32.00
18.00	13.00	136.00	25.00
20.50	16.00	133.00	37.00
23.00	12.00	135.00	46.00
25.50	23.00	135.00	33.00
28.00	13.00	135.00	36.00
30.50	10.00	130.00	NoLiq
33.00	16.00	131.00	30.00
35.50	13.00	131.00	30.00
38.00	5.00	131.00	20.00
40.50	5.00	131.00	20.00
43.00	2.00	127.00	NoLiq
45.50	5.00	127.00	NoLiq
48.00	4.00	127.00	34.00

50.50 33.00 134.00 34.00

Output Results:

Settlement of Saturated Sands=8.25 in.
Settlement of Unsaturated Sands=0.01 in.
Total Settlement of Saturated and Unsaturated Sands=8.26 in.
Differential Settlement=4.131 to 5.453 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
3.00	2.00	0.38	5.00	8.25	0.01	8.26
4.00	2.00	0.38	5.00	8.25	0.01	8.26
5.00	2.00	0.38	5.00	8.25	0.01	8.26
6.00	2.00	0.38	5.00	8.25	0.01	8.26
7.00	2.00	0.38	5.00	8.25	0.01	8.26
8.00	0.28	0.38	5.00	8.25	0.01	8.26
9.00	0.60	0.38	5.00	8.25	0.00	8.26
10.00	0.60	0.38	1.58	8.25	0.00	8.25
11.00	0.60	0.40	1.51	8.25	0.00	8.25
12.00	0.60	0.41	1.45	8.25	0.00	8.25
13.00	0.60	0.43	1.40	8.25	0.00	8.25
14.00	0.60	0.44	1.36	8.24	0.00	8.24
15.00	0.60	0.45	1.33	8.24	0.00	8.24
16.00	0.60	0.46	1.30	8.24	0.00	8.24
17.00	0.43	0.47	0.93*	8.22	0.00	8.22
18.00	0.29	0.48	0.60*	8.06	0.00	8.06
19.00	0.32	0.48	0.67*	7.84	0.00	7.84
20.00	0.37	0.49	0.75*	7.67	0.00	7.67
21.00	0.35	0.50	0.71*	7.53	0.00	7.53
22.00	0.31	0.50	0.61*	7.34	0.00	7.34
23.00	0.27	0.51	0.54*	7.11	0.00	7.11
24.00	0.35	0.51	0.69*	6.89	0.00	6.89
25.00	0.60	0.52	1.16	6.78	0.00	6.78
26.00	0.50	0.52	0.97*	6.76	0.00	6.76
27.00	0.34	0.52	0.65*	6.63	0.00	6.63
28.00	0.28	0.53	0.53*	6.41	0.00	6.41
29.00	0.25	0.53	0.48*	6.16	0.00	6.16
30.00	0.23	0.53	0.44*	5.89	0.00	5.89
31.00	0.24	0.53	0.45*	5.62	0.00	5.62
32.00	0.27	0.53	0.51*	5.36	0.00	5.36
33.00	0.28	0.53	0.54*	5.12	0.00	5.12
34.00	0.26	0.53	0.50*	4.88	0.00	4.88
35.00	0.24	0.53	0.47*	4.63	0.00	4.63
36.00	0.21	0.52	0.40*	4.35	0.00	4.35
37.00	0.16	0.52	0.30*	4.02	0.00	4.02
38.00	0.11	0.52	0.21*	3.62	0.00	3.62
39.00	0.11	0.52	0.20*	3.16	0.00	3.16
40.00	0.11	0.51	0.20*	2.70	0.00	2.70
41.00	0.14	0.51	0.27*	2.26	0.00	2.26
42.00	0.12	0.51	0.25*	1.86	0.00	1.86
43.00	0.11	0.51	0.22*	1.43	0.00	1.43
44.00	2.00	0.50	5.00	1.43	0.00	1.43
45.00	2.00	0.50	5.00	1.43	0.00	1.43
46.00	0.14	0.50	0.28*	1.26	0.00	1.26
47.00	0.14	0.49	0.27*	0.87	0.00	0.87
48.00	0.13	0.49	0.26*	0.48	0.00	0.48
49.00	0.25	0.49	0.52*	0.15	0.00	0.15
50.00	0.55	0.48	1.13	0.00	0.00	0.00
51.00	0.55	0.48	1.14	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils

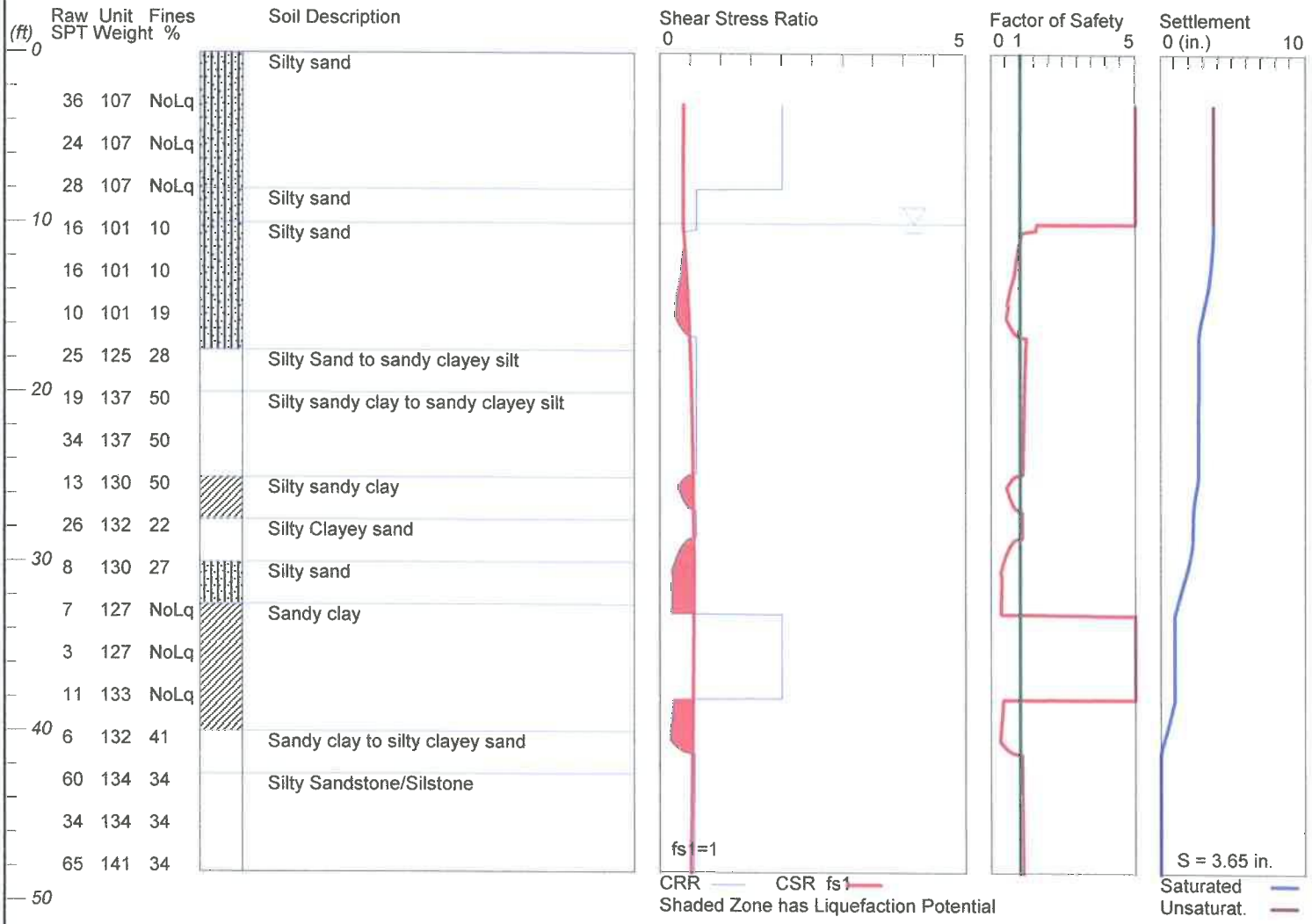
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

Pacific Center, Lot 13, Geotek Logs

Hole No.=B-2 Water Depth=10 ft

Magnitude=7
Acceleration=.595g



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Input File Name: C:\Users\gina\Desktop\Pac lot13-b2-2.liq
Title: Pacific Center, Lot 13, Geotek Logs
Subtitle: Geotek B-2

Surface Elev.=
Hole No.=B-2
Depth of Hole= 48.30 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 29.80 ft
Max. Acceleration= 0.6 g
Earthquake Magnitude= 7.00

Input Data:
Surface Elev.=
Hole No.=B-2
Depth of Hole=48.30 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 29.80 ft
Max. Acceleration=0.6 g
Earthquake Magnitude=7.00
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=1)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
3.00	36.00	107.00	NoLiq
5.50	24.00	107.00	NoLiq
8.00	28.00	107.00	NoLiq
10.50	16.00	101.00	10.00
13.00	16.00	101.00	10.00
15.50	10.00	101.00	19.00
18.00	25.00	125.00	28.00
20.50	19.00	137.00	50.00
23.00	34.00	137.00	50.00
25.50	13.00	130.00	50.00
28.00	26.00	132.00	22.00
30.50	8.00	130.00	27.00
33.00	7.00	127.00	NoLiq
35.50	3.00	127.00	NoLiq
38.00	11.00	133.00	NoLiq
40.50	6.00	132.00	41.00
43.00	60.00	134.00	34.00
45.50	34.00	134.00	34.00
48.00	65.00	141.00	34.00

Output Results:

Settlement of Saturated Sands=3.65 in.
Settlement of Unsaturated Sands=0.01 in.
Total Settlement of Saturated and Unsaturated Sands=3.65 in.
Differential Settlement=1.827 to 2.411 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
3.00	2.00	0.38	5.00	3.65	0.01	3.65
4.00	2.00	0.38	5.00	3.65	0.01	3.65
5.00	2.00	0.38	5.00	3.65	0.01	3.65
6.00	2.00	0.38	5.00	3.65	0.01	3.65
7.00	2.00	0.38	5.00	3.65	0.01	3.65
8.00	0.60	0.38	5.00	3.65	0.01	3.65
9.00	0.60	0.38	5.00	3.65	0.00	3.65
10.00	0.60	0.38	1.58	3.65	0.00	3.65
11.00	0.40	0.40	0.99*	3.61	0.00	3.61
12.00	0.37	0.42	0.88*	3.53	0.00	3.53
13.00	0.35	0.43	0.80*	3.41	0.00	3.41
14.00	0.29	0.45	0.64*	3.22	0.00	3.22
15.00	0.27	0.47	0.57*	2.97	0.00	2.97
16.00	0.31	0.48	0.66*	2.72	0.00	2.72
17.00	0.60	0.49	1.21	2.62	0.00	2.62
18.00	0.60	0.50	1.19	2.62	0.00	2.62
19.00	0.60	0.51	1.17	2.62	0.00	2.62
20.00	0.60	0.52	1.15	2.62	0.00	2.62
21.00	0.60	0.52	1.14	2.61	0.00	2.61
22.00	0.60	0.53	1.13	2.61	0.00	2.61
23.00	0.60	0.53	1.12	2.61	0.00	2.61
24.00	0.60	0.54	1.11	2.61	0.00	2.61
25.00	0.39	0.54	0.72*	2.58	0.00	2.58
26.00	0.34	0.55	0.62*	2.38	0.00	2.38
27.00	0.60	0.55	1.08	2.25	0.00	2.25
28.00	0.60	0.55	1.08	2.22	0.00	2.22
29.00	0.36	0.56	0.64*	2.16	0.00	2.16
30.00	0.23	0.56	0.41*	1.92	0.00	1.92
31.00	0.20	0.56	0.36*	1.61	0.00	1.61
32.00	0.20	0.56	0.35*	1.29	0.00	1.29
33.00	0.19	0.56	0.34*	0.96	0.00	0.96
34.00	2.00	0.55	5.00	0.96	0.00	0.96
35.00	2.00	0.55	5.00	0.96	0.00	0.96
36.00	2.00	0.55	5.00	0.96	0.00	0.96
37.00	2.00	0.55	5.00	0.96	0.00	0.96
38.00	2.00	0.54	5.00	0.96	0.00	0.96
39.00	0.21	0.54	0.38*	0.68	0.00	0.68
40.00	0.18	0.54	0.33*	0.36	0.00	0.36
41.00	0.31	0.54	0.58*	0.05	0.00	0.05
42.00	0.57	0.53	1.07	0.00	0.00	0.00
43.00	0.57	0.53	1.08	0.00	0.00	0.00
44.00	0.57	0.52	1.08	0.00	0.00	0.00
45.00	0.57	0.52	1.09	0.00	0.00	0.00
46.00	0.56	0.52	1.09	0.00	0.00	0.00
47.00	0.56	0.51	1.10	0.00	0.00	0.00
48.00	0.56	0.51	1.10	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils
CSRsf Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S. Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat Settlement from saturated sands

S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

APPENDIX F

DYNAMIC STRENGTH EVALUATION

APPENDIX F

DYNAMIC STRENGTH EVALUATION

The evaluation of the dynamic strength of the alluvium is made using the work of Robertson (2010) and Boulanger and Idriss (2006). Robertson discusses the evaluation of flow liquefaction and liquefied strength using the Cone Penetration Test, and Boulanger and Idriss discuss the liquefaction susceptibility of silt and clay materials.

Robertson discusses conditions where the soil may contract or strain-soften under earthquake loading (see attached Figures F.14 through F.18). These figures provide the basis for screening the site soils for contractive behavior. The first screen tool is given on Figure F.18 which shows the portions of the normalized soil behavior type (SBT) chart. Zones A2 and C are considered subject to strain-softening type behavior, whereas zones A1 and B present dilative (strain-hardening) behavior. All CPT soundings were evaluated using the SBT chart and the results of this evaluation are shown in Figure F.1. The CPT data in Appendix E is used to approximate the distribution of the CPT data which is in each zone. Materials with strain-hardening behavior represent 83 to 95 percent of the soil materials explored onsite. Materials with potentially detrimental strain-softening response represent an estimated 5 to 17 percent of the soil materials explored onsite. Soil materials with strain-softening type behavior are further defined using Figure F.16 which shows the mean of flow liquefaction materials from case studies as having a $Qtn,cs < 50$. The CPT data in Appendix E is used to determine the soil layers where this occurs; these soil depth increments are shown on the far right-hand column of Figure 1. The data from Geotek borings B-1 and B-2 may be evaluated in a similar manner using a $(N1)_{60\text{-}cs}$ value below 15 as indicative of contractive behavior. Figures F.2 and F.3 show 5 samples from the borings which may be subject to flow liquefaction. The depth intervals where potential flow liquefaction materials were detected onsite are shown in cross section on Figure F.4. The contractive soil material locations under the building pad are sparse and not continuous, whereas those at the slope toe are more frequent but still discontinuous. Approximately 10 to 40 percent of the materials from 34 to 50 feet below grade at the toe of slope may be subject to strain-softening behavior.

The evaluation of materials for sand- or clay-like behavior using the Boulanger and Idriss work (see Figures F.19 and F.20). The summary is shown in Figures F.5 through F.8. With the majority of the plasticity indices (PI) of 7 or more, we generally concur with the identification of soil types from the GeoTek borings. Soil materials with the $PI \geq 7$ would potentially be subject to “cycling softening”, instead of contractive liquefaction behavior of sand materials during a seismic event.

The dynamic strength of the alluvium is made using the results of the CLiq and CPeT-IT programs by Geologismiki Geotechnical. Estimates of seismic strength for sand-like materials are provided on the far right-hand side of Figures E.6, E.15, E.24, E.33, E.42, and E.50. These estimates only require that the effective overburden pressure be known to determine the residual

strength as an effective cohesion in pounds/square foot. Estimates of seismic strength for clay-like materials are determined using the undrained strength of the soil materials.

The digital data from these analyses were placed in a spreadsheet for calculation. Calculations for the static and seismic strength calculations for the buried alluvium beneath the pad area are provided in Figures F.9 through F.13. These strengths are based on evaluation of CPT-1 and CPT-2 (2014) data. Figure F.9 shows a plot of estimated shear strengths for static and seismic loading conditions, and the shear strength value selected for seismic slope stability evaluation. Seismic soil strengths were calculated using results from the CLiq program (see Figures E.42 through E.50), but limited by limiting the undrained strength ratio to 0.45 as recommended by Robertson (2010); refer to Figure F.17 for this recommendation. The seismic soil strengths were then chosen in consideration of both the seismic residual and static shear strengths. An average or more conservative static strength was selected for the soil materials above 34 feet at the toe of slope since these soils did not exhibit strain-softening tendencies. Since 10 to 40 percent of the material below 34 feet showed some strain-softening potential, the seismic shear strength was selected as somewhat less than the average of the static and seismic strengths.

The seismic soil strength of alluvium materials below a depth of 50 feet below grade was modeled as a cohesionless soil with a friction angle of 25 degrees which is consistent with the recommended by Robertson (2010); see Figure F.17.

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CPT-ID	TOTAL CPT POINTS	POINTS IN SBT ZONE A1/B	A2	C	DEPTH W/ $Q_{tn,cs}$ < 50 (NOTE 1)
CPT-01 (2005)	580 (E7)	546 (94%)	3 (E1) (<1%)	31 (E1) (5%)	72-75' (E4)
CPT-02 (2005)	615 (E16)	555 (90%)	10 (E10) (2%)	50 (E10) (8.1%)	70-71' 75-76' 78-80' (E13)
CPT-03 (2005)	548 (E25)	523 (95%)	0 (E19) (0%)	25 (E19) (5%)	74-80' (E22)
CPT-04 (2005)	572 (E34)	540 (94%)	0 (E28)	32 (E-28) (6%)	62-67' (E-31)
CPT-1 (2014)	305 (E43)	252 (83%)	23 (E37) (8%)	30 (E37) (9%)	42-46' (E40)
CPT-2 (2014)	305 (E51)	268 (88%)	7 (E46) (2%)	30 (E46) (10%)	39-41' (E48)

Ref. Robertson 2010 Evaluation of Flow Liquefaction and Liquified Strength Using Cone Penetration Test, ASCE JGGE June 2010

Note: (1) USE $Q_{tn,cs}$ < 50 since this line is closer to mean of case histories

Stoney-Miller Consultants, Inc.
www.stoneymiller.com

Geofirm
www.geofirm.com

REVIEW CPT "CONTRACTIVE" BEHAVIOR

PREPARED BY:	DATE:	PROJECT NO.:
RLL	6-1-14	12655-04
REVIEWED BY:	DATE:	FIGURE NO.:
		E.1

Project No. 1162-CR3
Meritage Homes
Boring B-1
Input parameters:
Peak ground accel (g)= 0.539
Factor of Safety= 1.3
Earthquake Magnitude, M= 6.7
Water table depth during drilling (m)= 9.4
Water table depth for analysis (m)= 3.8
Average γ above water table (kN/m^3)= 18.9
Average γ below water table (kN/m^3)= 16.5
Borehole diameter (mm)= 102
Requires correction of sampler liners (YES/NO): YES
Rod lengths assumed equal to the depth plus 1.5m (for above ground extension).

SPT sample number	Depth (ft)	Depth (m)	Measured N	Soil Type (USCS)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)	Engergy ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa) during drilling	σ_{vc}' (kPa) for liq. anal.	C_N (based on drilling gw level)	$(N_1)_{60}$	ΔN for fines content	$(N_1)_{60\text{-}cs}$	Stress reduct. coeff., r_d	CSR	MSF for sand	K s
		0.0																					
1	3	0.8	13	0	Unsaturated	0	78	1.3	1	0.75	1.27	16.2	14.4	14.4	14.4	1.70	27.5	0.0	27.5	1.00	0.350	1.23	
2	5	1.5	43	0	Unsaturated	0	78	1.3	1	0.80	1.30	58.1	28.7	28.7	28.7	1.39	80.9	0.0	80.9	0.99	0.347	1.23	
3	8	2.3	9	0	Unsaturated	0	78	1.3	1	0.80	1.16	10.9	43.1	43.1	43.1	1.50	16.3	0.0	16.3	0.98	0.343	1.23	
4	10	3.0	41	0	Unsaturated	26	78	1.3	1	0.85	1.30	58.9	57.5	57.5	57.5	1.16	68.3	5.1	73.5	0.97	0.340	1.23	
5	13	3.8	18	0	0	26	78	1.3	1	0.85	1.29	25.7	71.8	71.8	71.8	1.13	29.1	5.1	34.3	0.96	0.335	1.23	
6	15	4.6	23	0	0	32	78	1.3	1	0.95	1.30	36.9	86.2	86.2	78.7	1.05	38.8	5.4	44.2	0.95	0.363	1.23	
7	18	5.3	16	0	0	25	78	1.3	1	0.95	1.25	24.6	100.5	100.5	85.6	1.00	24.7	5.1	29.8	0.93	0.384	1.23	
8	20	6.1	16	0	0	37	78	1.3	1	0.95	1.23	24.3	114.9	114.9	92.5	0.95	23.0	5.5	28.6	0.92	0.400	1.23	
9	23	6.9	12	0	0	46	78	1.3	1	0.95	1.15	17.1	129.3	129.3	99.4	0.89	15.1	5.6	20.7	0.90	0.412	1.23	
10	25	7.6	23	0	0	33	78	1.3	1	0.95	1.30	36.9	143.6	143.6	106.3	0.89	32.7	5.5	38.2	0.89	0.422	1.23	
11	28	8.4	13	0	0	36	78	1.3	1	0.95	1.15	18.4	158.0	158.0	113.2	0.80	14.8	5.5	20.3	0.87	0.428	1.23	
12	30	9.1	10	0	clay	0	78	1.3	1	1.00	1.30	16.9	172.4	172.4	120.0	0.87	n.a.	n.a.	n.a.	0.86	0.433	1.23	
13	33	9.9	16	0	0	30	78	1.3	1	1.00	1.19	24.8	185.7	181.2	125.9	0.77	19.1	5.4	24.4	0.84	0.437	1.23	
14	35	10.7	13	0	0	30	78	1.3	1	1.00	1.14	19.3	198.2	186.3	130.9	0.74	14.3	5.4	19.6	0.83	0.440	1.23	
15	38	11.4	5	0	0	20	78	1.3	1	1.00	1.10	7.2	210.8	191.4	136.0	0.67	4.8	4.5	9.3	0.81	0.442	1.23	
16	40	12.2	5	0	0	20	78	1.3	1	1.00	1.10	7.2	223.4	196.5	141.1	0.66	4.7	4.5	9.2	0.80	0.443	1.23	
17	43	13.0	2	0	clay	0	78	1.3	1	1.00	1.30	3.4	235.9	201.5	146.2	0.83	n.a.	n.a.	n.a.	0.78	0.443	1.23	
18	45	13.7	5	0	clay	0	78	1.3	1	1.00	1.30	8.5	248.5	206.6	151.3	0.83	n.a.	n.a.	n.a.	0.77	0.442	1.23	
19	48	14.5	2	0	0	34	78	1.3	1	1.00	1.10	2.9	261.1	211.7	156.4	0.80	1.7	5.5	7.2	0.75	0.440	1.23	
20	50	15.2	33	0	0	0	78	1.3	1	1.00	1.30	55.8	273.6	216.8	161.5	0.82	45.5	0.0	45.5	0.74	0.438	1.23	
21	50																						

EXAMPLE

F.2

Project No. 1162-CR3

Meritage Homes

Boring B-2

Input parameters:

Peak ground accel (g)= 0.539

Factor of Safety= 1.3

Earthquake Magnitude, M= 6.7

Water table depth during drilling (m)= 9.1

Water table depth for analysis (m)= 3.8

Average γ above water table (kN/m³)= 18.9

Average γ below water table (kN/m³)= 16.5

Borehole diameter (mm)= 102

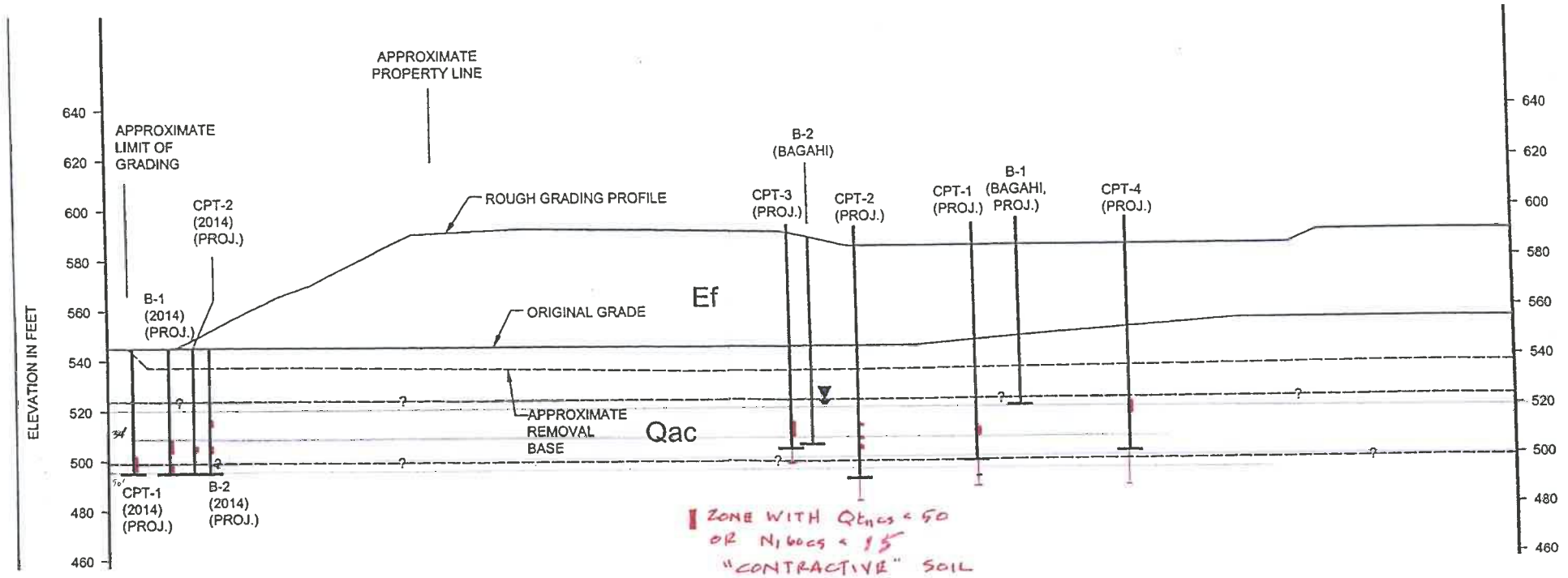
Requires correction of sampler liners (YES/NO): YES

Rod lengths assumed equal to the depth plus 1.5m (for above ground extension).

SPT sample number	Depth (ft)	Depth (m)	Measured N	Soil Type (USCS)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)	Engergy ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vo} (kPa)	σ_{vc}' (kPa) during drilling	σ_{vc}' (kPa) for liq. anal.	C_N (based on drilling gw level)	$(N_1)_{60}$	ΔN for fines content	$(N_1)_{60\text{-}cs}$	Stress reduct. coeff., r_d	CSR	MSF for sand	K s
		0.0																					
1	3	0.8	36	0	Unsaturated	0	78	1.3	1	0.75	1.30	45.6	14.4	14.4	14.4	1.67	76.2	0.0	76.2	1.00	0.350	1.23	
2	5	1.5	24	0	Unsaturated	0	78	1.3	1	0.80	1.30	32.4	28.7	28.7	28.7	1.40	45.4	0.0	45.4	0.99	0.347	1.23	
3	8	2.3	28	0	Unsaturated	0	78	1.3	1	0.80	1.30	37.9	43.1	43.1	43.1	1.25	47.4	0.0	47.4	0.98	0.343	1.23	
4	10	3.0	14	0	Unsaturated	10	78	1.3	1	0.85	1.24	19.2	57.5	57.5	57.5	1.26	24.2	1.1	25.3	0.97	0.340	1.23	
5	13	3.8	16	0	0	10	78	1.3	1	0.85	1.25	22.2	71.8	71.8	71.8	1.14	25.4	1.1	26.5	0.96	0.335	1.23	
6	15	4.6	10	0	0	19	78	1.3	1	0.95	1.15	14.2	86.2	86.2	78.7	1.08	15.4	4.3	19.7	0.95	0.363	1.23	
7	18	5.3	25	0	0	28	78	1.3	1	0.95	1.30	40.1	100.5	100.5	85.6	1.00	40.2	5.3	45.5	0.93	0.384	1.23	
8	20	6.1	19	0	0	50	78	1.3	1	0.95	1.29	30.2	114.9	114.9	92.5	0.95	28.8	5.6	34.4	0.92	0.400	1.23	
9	23	6.9	34	0	0	50	78	1.3	1	0.95	1.30	54.6	129.3	129.3	99.4	0.94	51.2	5.6	56.8	0.90	0.412	1.23	
10	25	7.6	13	0	0	50	78	1.3	1	0.95	1.16	18.6	143.6	143.6	106.3	0.84	15.7	5.6	21.3	0.89	0.422	1.23	
11	28	8.4	26	0	0	22	78	1.3	1	0.95	1.30	41.7	158.0	158.0	113.2	0.87	36.1	4.8	40.9	0.88	0.428	1.23	
12	30	9.1	7	0	0	27	78	1.3	1	1.00	1.10	10.0	172.4	172.4	120.0	0.74	7.4	5.2	12.6	0.86	0.433	1.23	
13	33	9.9	7	0	clay	0	78	1.3	1	1.00	1.30	11.8	184.9	177.5	125.1	0.86	n.a.	n.a.	n.a.	0.84	0.439	1.23	
14	35	10.7	3	0	clay	0	78	1.3	1	1.00	1.30	5.1	197.5	182.6	130.2	0.86	n.a.	n.a.	n.a.	0.83	0.441	1.23	
15	38	11.4	11	0	clay	0	78	1.3	1	1.00	1.30	18.6	210.1	187.6	135.3	0.85	n.a.	n.a.	n.a.	0.81	0.443	1.23	
16	40	12.2	6	0	0	41	78	1.3	1	1.00	1.10	8.6	222.6	192.7	140.4	0.68	5.8	5.6	11.4	0.80	0.444	1.23	
17	43	13.0	60	0	0	0	78	1.3	1	1.00	1.30	101.4	235.2	197.8	145.5	0.84	85.0	0.0	85.0	0.78	0.443	1.23	
18	45	13.7	34	0	0	0	78	1.3	1	1.00	1.30	57.5	247.8	202.9	150.6	0.83	47.8	0.0	47.8	0.77	0.443	1.23	
19	48	14.5	65	0	0	0	78	1.3	1	1.00	1.30	109.9	260.3	208.0	155.7	0.83	90.8	0.0	90.8	0.75	0.441	1.23	
20	48	14.5	51	0	0	0	78	1.3	1	1.00	1.30	86.2	260.9	208.2	155.9	0.83	71.2	0.0	71.2	0.75	0.441	1.23	
21	0																						

2 AE

F.3


APPROXIMATE PROPERTY LINE
APPROXIMATE LIMIT OF GRADING
CPT-2 (2014) (PROJ.)
B-1 (2014) (PROJ.)
ROUGH GRADING PROFILE
ORIGINAL GRADE
Ef
APPROXIMATE REMOVAL BASE
Qac
CPT-1 (2014) (PROJ.)
B-2 (2014) (PROJ.)
B-2 (BAGAHI)
CPT-3 (PROJ.)
CPT-2 (PROJ.)
CPT-1 (PROJ.)
B-1 (BAGAHI, PROJ.)
CPT-4 (PROJ.)
ELEVATION IN FEET
640
620
600
580
560
540
520
500
480
460
ZONE WITH Qtncs < 50 OR N1(60)cs < 15 "CONTRACTIVE" SOIL

F.4

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JMP
PROJECT NAME:	25192 Commercenter Drive	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Jeff
PROJECT NO.:	1162-CR3	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75 (track rig)
LOCATION:	See Exploration Location Map			**DATE:**	4/23/2014

BORING NO.: B-1

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
			B1		**ARTIFICIAL FILL**			
		5 4 9	S1	SM/CL	@ 2.5': Silty fine SAND to silty CLAY interlayered, light brown to brown, slightly moist, medium dense/stiff.			
5		24 33 30	R1	SP/SM	@ 5': SAND to silty fine SAND, light brown, slightly moist, dense.	3.4	113.7	
		5 5 4	S2	SM	**ALLUVIUM** @ 7.5': Silty fine SAND, brown, moist, loose.			
10		14 24 39	R2	SM	@ 10': same as above, dense.	7.3	120.1	25.5% passing #200
		6 9 9	S3	SM	@ 12.5': same as above, medium dense.			26.1% passing #200
15		15 16 19	R3	SM	@ 15': same as above, medium dense, trace clay.	12.4	121.3	31.9% passing #200
		5 7 6	S4	SM	@ 17.5': same as above, medium dense, trace clay.			24.9% passing #200
20		9 12 12	R4	SM	@ 20': same as above, medium dense, trace clay.	13.0	118.1	36.9% passing #200
		4 6 6	S5	SM-SC	@ 22.5': Silty clayey fine SAND, brown, moist, medium dense, one piece of gravel.	13.4		LL=30, PL=13, PI=17 46.2% passing #200
25		10 16 20	R5	SM-SC	@ 25': same as above.	15.2	117.3	33.4% passing #200
		4 5 8	S6	SM-SC/CL	@ 27.5': Silty clayey f-m SAND to CLAY interlayered, brown, moist to very moist, medium dense/stiff.			36.3% passing #200
30		5 7 9	R6	CL	@ 30': Fine sandy CLAY, brown, very moist, stiff, groundwater encountered at 30.8'.	24.1	104.5	LL=29, PL=15, PI=14

PI>7 clay

clay

DRAFT

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

Ref: Boulanger & Idriss Liquefaction Susceptibility Criteria for Silts and Clays ASCE JGGE Nov. 2006

F.5

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JMP
PROJECT NAME:	25192 Commercenter Drive	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Jeff
PROJECT NO.:	1162-CR3	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75 (track rig)
LOCATION:	See Exploration Location Map			**DATE:**	4/23/2014

BORING NO.: B-1 (continued)

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Laboratory Testing: Others
					continued:			
		4 5 11	S7	SM	@ 32.5': Silty fine SAND, brownish gray, wet, medium dense.			30.3% passing #200
35		7 9 11	R7	SM	@ 35': same as above.	18.4	110.9	
		2 2 3	S8	SM	@ 37.5': Silty fine SAND, brownish gray, wet, loose. $(N_1)_{60\text{-}CS} = 9.3$			19.9% passing #200
40		7 4 3	R8		@ 40': no recovery. $(N_1)_{60\text{-}CS} = 9.2$			
		push 1 1	S9	CL	@ 42.5': Fine sandy CLAY, dark grayish brown, wet, very soft.	25.5		LL=28, PL=15, (PI=13) clay
45		push 4 4	R9	CL	@ 45': same as above, medium stiff.	26.0	101.0	
		1 2 2	S10	CL	@ 47.5': Silty fine to medium SAND, dark grayish brown, wet, loose. $(N_1)_{60\text{-}CS} = 7.2$	23.7		33.9% passing #200
50		8 24 27	R10	SM	@ 50': Silty fine SAND, dark bluish gray, wet, dense.	17.3	114.5	
					BORING TERMINATED AT 51.5' Groundwater encountered at 30.8'. Boring backfilled with soil cuttings.			
55								
60								

DRAFT

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JMP
PROJECT NAME:	25192 Commercenter Drive	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Jeff
PROJECT NO.:	1162-CR3	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75 (track rig)
LOCATION:	See Exploration Location Map			**DATE:**	4/23/2014

BORING NO.: B-2

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Laboratory Testing: Others	
					ARTIFICIAL FILL				
		22 30 25	R1	SM/ML	@ 2.5': Silty fine SAND to fine sandy SILT interlayered, light brown to brown, slightly moist, dense/hard.				
5		7 9 15	S1	SM/ML	@ 5': same as above, medium dense/very stiff.				
		18 18 25	R2	SM	@ 7.5': Silty fine SAND, light brown, slightly moist, medium dense.	3.4	103.6		
10		6 8 8	S2	SM	**ALLUVIUM** @ 10': Silty fine SAND, light to medium brown, slightly moist, medium dense, uniform.			9.5% passing #200	
		7 9 15	R3	SM	@ 12.5': same as above.	4.0	97.3		
15		4 5 5	S3	SM	@ 15': same as above, loose to medium dense.			18.7% passing #200	
		8 16 23	R4	SM/ML	@ 17.5': Silty fine SAND to sandy clayey SILT, brown, moist, medium dense/very stiff.	7.7	116.5	LL=20, PL=18, PI=3 27.5% passing #200	sand
20		6 9 10	S4	ML/CL	@ 20': Silty sandy CLAY to sandy clayey SILT, brown to reddish brown, moist, very stiff.	11.2		LL=24, PL=17, PI=7	clay
		11 22 30	R5	ML/CL	@ 22.5': same as above, hard.	14.5	119.3		
25		4 6 7	S5	CL	@ 25': Silty sandy CLAY, brown, moist, stiff.	15.2		LL=30, PL=14, PI=16	clay
		12 21 19	R6	SM/SC	@ 27.5': Silty clayey fine SAND, medium brown, very moist, medium dense.	14.1	115.4	22.2% passing #200	
30		3 4 4	S6	SM	@ 30': Silty f-m SAND, brownish gray, wet, loose, groundwater encountered at 29.8'. (H1)60-c8 = 12.6			27.3% passing #200	

DRAFT

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

F.7

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Meritage Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	JMP
PROJECT NAME:	25192 Commercenter Drive	**DRILL METHOD:**	8" Hollow Stem	**OPERATOR:**	Jeff
PROJECT NO.:	1162-CR3	**HAMMER:**	Auto 140#/30"	**RIG TYPE:**	CME 75 (track rig)
LOCATION:	See Exploration Location Map			**DATE:**	4/23/2014

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	BORING NO.: B-2 (continued) / MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Laboratory Testing: Others
					continued:			
		4 4 6	R7	CL	@ 32.5': Sandy CLAY, brown, wet, medium stiff.	21.0	105.1	LL=34, PL=15, PI=19 clay
35		push 2 1	S7	CL	@ 35': same as above, soft.	18.6		
		5 7 10	R8	CL	@ 37.5': same as above, stiff.	19.4	111	
40		2 3 3	S8	CL/SC	@ 40': Sandy CLAY to silty clayey fine SAND, brown to reddish brown, wet, medium stiff/loose, some gravels interlayered. (N1)60-cs = 11.4	20.3		LL=33, PL=14, PI=19 41.0% passing #200 clay
		20 42 50/4"	R9		**BEDROCK-SESPE AND VAQUEROS FORMATIONS, undifferentiated** @ 42.5': Silty fine SANDSTONE, reddish brown with blue green mottling, slightly moist to moist, hard.	17.9	113.8	
45		10 16 18	S9		@ 45': same as above, dense.			
		44 50/3"	R10		@ 47.5': SILTSTONE, reddish brown, slightly moist, hard.	14.3	123.4	
50					**BORING TERMINATED AT 48.3'** Groundwater encountered at 29.8'. Boring backfilled with soil cuttings.			
55								
60								

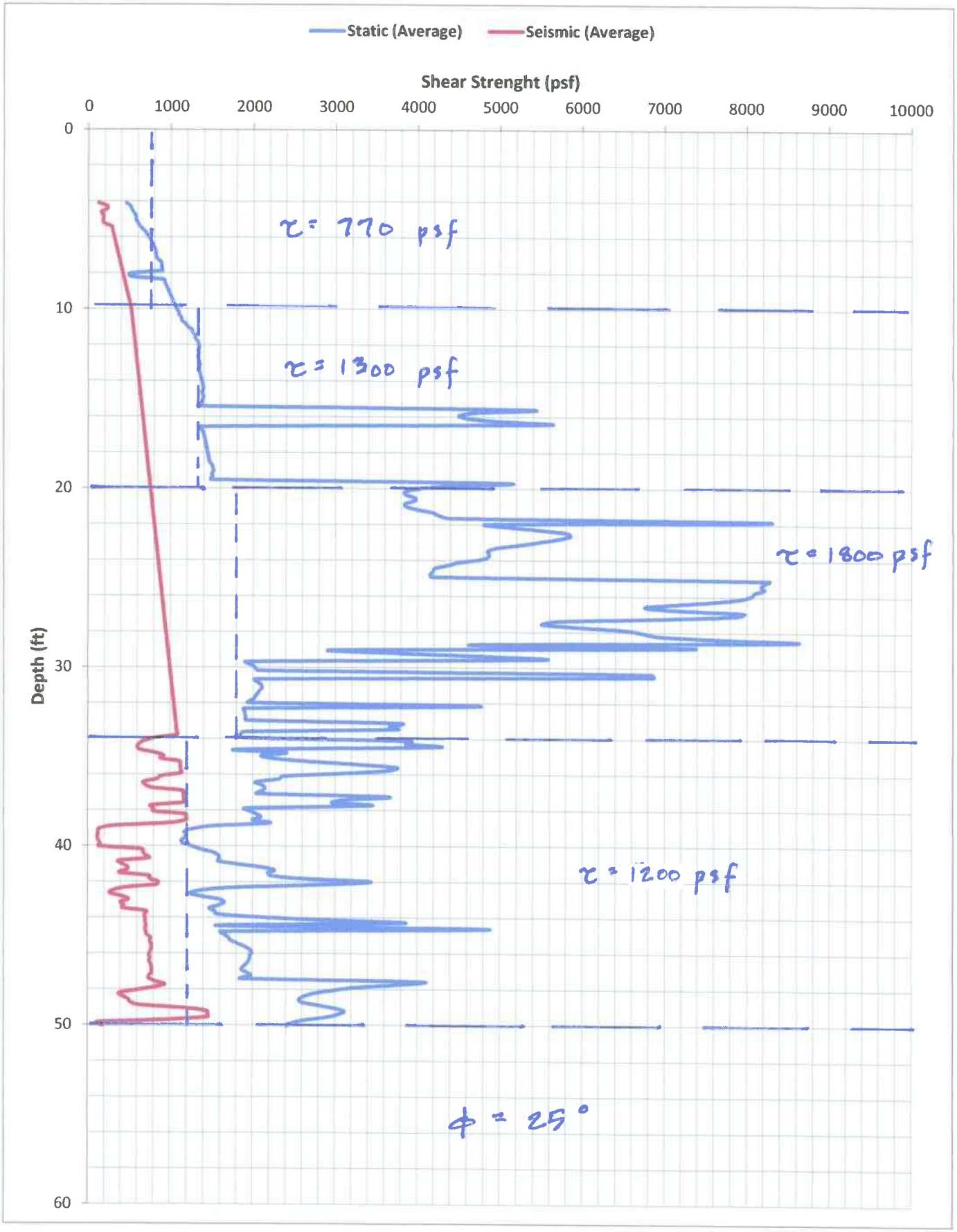
DRAFT

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resistivity Test; SH = Shear Test; HC = Consolidation; MD = Maximum Density

2014 CPT
TOE OF SLOPE



CPT-1					CPT-2					Seismic Residual Average (psf)	Static Strength Average (psf)	
Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)	Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)			
0.16	HAND AUGER		NA	NA	0.16	HAND AUGER		NA	NA	0.00	0.00	
0.33	HAND AUGER		NA	NA	0.33	HAND AUGER		NA	NA	0.00	0.00	
0.49	HAND AUGER		NA	NA	0.49	HAND AUGER		NA	NA	0.00	0.00	
0.66	HAND AUGER		NA	NA	0.66	HAND AUGER		NA	NA	0.00	0.00	
0.82	HAND AUGER		NA	NA	0.82	HAND AUGER		NA	NA	0.00	0.00	
0.98	HAND AUGER		NA	NA	0.98	HAND AUGER		NA	NA	0.00	0.00	
1.15	HAND AUGER		NA	NA	1.15	HAND AUGER		NA	NA	0.00	0.00	
1.31	HAND AUGER		NA	NA	1.31	HAND AUGER		NA	NA	0.00	0.00	
1.48	HAND AUGER		NA	NA	1.48	HAND AUGER		NA	NA	0.00	0.00	
1.64	HAND AUGER		NA	NA	1.64	HAND AUGER		NA	NA	0.00	0.00	
1.80	HAND AUGER		NA	NA	1.80	HAND AUGER		NA	NA	0.00	0.00	
1.97	HAND AUGER		NA	NA	1.97	HAND AUGER		NA	NA	0.00	0.00	
2.13	HAND AUGER		NA	NA	2.13	HAND AUGER		NA	NA	0.00	0.00	
2.30	HAND AUGER		NA	NA	2.30	HAND AUGER		NA	NA	0.00	0.00	
2.46	HAND AUGER		NA	NA	2.46	HAND AUGER		NA	NA	0.00	0.00	
2.63	HAND AUGER		NA	NA	2.63	HAND AUGER		NA	NA	0.00	0.00	
2.79	HAND AUGER		NA	NA	2.79	HAND AUGER		NA	NA	0.00	0.00	
2.95	HAND AUGER		NA	NA	2.95	HAND AUGER		NA	NA	0.00	0.00	
3.12	HAND AUGER		NA	NA	3.12	HAND AUGER		NA	NA	0.00	0.00	
3.28	HAND AUGER		NA	NA	3.28	HAND AUGER		NA	NA	0.00	0.00	
3.45	HAND AUGER		NA	NA	3.45	HAND AUGER		NA	NA	0.00	0.00	
3.61	HAND AUGER		NA	NA	3.61	HAND AUGER		NA	NA	0.00	0.00	
3.77	HAND AUGER		NA	NA	3.77	HAND AUGER		NA	NA	0.00	0.00	
3.94	HAND AUGER		NA	NA	3.94	HAND AUGER		NA	NA	0.00	0.00	
4.10	0.07	0.07	31	468	4.10	0.68	0.45	212	443	121.50	455.56	
4.27	0.85	0.45	221	526	4.27	0.72	0.45	221	483	220.71	504.78	
4.43	0.91	0.45	229	559	4.43	0.70	0.45	229	484	229.20	521.21	
4.59	0.95	0.45	238	593	4.59	0.13	0.13	69	484	153.38	538.15	
4.76	0.96	0.45	246	628	4.76	0.19	0.19	101	497	173.70	562.87	
4.92	0.96	0.45	255	646	4.92	0.19	0.19	105	496	179.86	571.38	
5.09	0.95	0.45	263	645	5.09	0.14	0.14	80	509	171.44	577.39	
5.25	0.93	0.45	272	663	5.25	0.19	0.19	117	539	194.36	600.73	
5.41	0.93	0.45	280	680	5.41	0.71	0.45	280	569	280.12	624.78	
5.58	0.93	0.45	289	698	5.58	0.77	0.45	289	619	288.61	658.72	
5.74	0.93	0.45	297	716	5.74	0.85	0.45	297	674	297.10	694.79	
5.91	0.92	0.45	306	712	5.91	0.90	0.45	306	712	305.64	711.55	
6.07	0.91	0.45	314	729	6.07	0.93	0.45	314	751	314.12	739.73	
6.23	0.89	0.45	323	723	6.23	0.94	0.45	323	792	322.61	757.76	
6.40	0.88	0.45	331	740	6.40	0.95	0.45	331	811	331.10	775.06	
6.56	0.87	0.45	340	756	6.56	0.96	0.45	340	829	339.58	792.36	
6.73	0.87	0.45	348	772	6.73	0.95	0.45	348	847	348.07	809.67	
6.89	0.87	0.45	357	789	6.89	0.92	0.45	357	839	356.56	813.83	
7.05	0.88	0.45	365	805	7.05	0.91	0.45	365	830	365.04	817.80	
7.22	0.89	0.45	374	822	7.22	0.92	0.45	374	847	373.53	834.48	
7.38	0.90	0.45	382	864	7.38	0.92	0.45	382	892	382.02	878.01	
7.55	0.90	0.45	391	881	7.55	0.91	0.45	391	881	390.51	881.36	
7.71	0.91	0.45	399	898	7.71	0.87	0.45	399	871	398.99	884.55	
7.87	0.90	0.45	407	915	7.87	0.84	0.45	407	860	407.48	887.57	
8.04	0.88	0.45	416	904	8.04	0.80	0.45	416	100	415.97	501.77	
8.20	0.87	0.45	424	920	8.20	0.80	0.45	424	100	424.45	509.97	
8.37	0.86	0.45	433	936	8.37	0.82	0.45	433	879	432.94	907.71	
8.53	0.86	0.45	441	923	8.53	0.83	0.45	441	923	441.43	923.11	
8.69	0.86	0.45	450	939	8.69	0.83	0.45	450	939	449.91	938.94	
8.86	0.86	0.45	458	955	8.86	0.83	0.45	458	955	458.40	954.77	
9.02	0.85	0.45	467	971	9.02	0.84	0.45	467	971	466.89	970.59	
9.19	0.85	0.45	475	986	9.19	0.85	0.45	475	986	475.38	986.42	
9.35	0.85	0.45	484	1002	9.35	0.85	0.45	484	1002	483.86	1002.24	
9.51	0.85	0.45	492	1018	9.51	0.85	0.45	492	1018	492.35	1018.07	
9.68	0.85	0.45	501	1034	9.68	0.83	0.45	501	1034	500.84	1033.89	AVE 0-10
9.84	0.86	0.45	509	1084	9.84	0.82	0.45	509	1017	509.38	1050.31	769.22
10.01	0.88	0.45	518	1100	10.01	0.81	0.45	518	1032	517.67	1065.77	
10.17	0.89	0.45	522	1144	10.17	0.80	0.45	522	1039	521.55	1091.05	
10.34	0.91	0.45	525	1151	10.34	0.80	0.45	525	1046	525.43	1098.43	
10.50	0.93	0.45	529	1197	10.50	0.80	0.45	529	1053	529.31	1124.69	
10.66	0.95	0.45	533	1205	10.66	0.82	0.45	533	1059	533.19	1132.20	
10.83	0.97	0.45	537	1253	10.83	0.85	0.45	537	1101	537.08	1177.01	
10.99	0.98	0.45	541	1261	10.99	0.88	0.45	541	1145	540.96	1202.94	
11.16	1.00	0.45	545	1311	11.16	0.89	0.45	545	1190	544.84	1250.46	
11.32	1.00	0.45	549	1319	11.32	0.91	0.45	549	1198	548.72	1258.66	
11.48	1.00	0.45	553	1328	11.48	0.92	0.45	553	1245	552.60	1286.57	
11.65	1.00	0.45	556	1337	11.65	0.94	0.45	556	1253	556.48	1294.91	
11.81	1.00	0.45	560	1345	11.81	0.95	0.45	560	1303	560.37	1323.90	
11.98	1.00	0.45	564	1354	11.98	0.96	0.45	564	1311	564.25	1332.37	
12.14	1.00	0.45	568	1363	12.14	0.95	0.45	568	1319	568.13	1340.85	
12.30	0.99	0.45	572	1371	12.30	0.95	0.45	572	1285	572.01	1328.25	
12.47	0.99	0.45	576	1380	12.47	0.94	0.45	576	1293	575.89	1336.58	
12.63	0.98	0.45	580	1344	12.63	0.93	0.45	580	1301	579.78	1322.81	
12.80	0.97	0.45	584	1353	12.80	0.93	0.45	584	1309	583.66	1331.00	

CPT-1					CPT-2					Seismic Residual Average (psf)	Static Strength Average (psf)	
Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)	Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)			
12.96	0.96	0.45	588	1361	12.96	0.93	0.45	588	1318	587.54	1339.19	
13.12	0.95	0.45	591	1369	13.12	0.94	0.45	591	1326	591.42	1347.38	
13.29	0.95	0.45	595	1334	13.29	0.94	0.45	595	1334	595.30	1333.62	
13.45	0.94	0.45	599	1342	13.45	0.94	0.45	599	1342	599.19	1341.66	
13.62	0.94	0.45	603	1350	13.62	0.94	0.45	603	1350	603.07	1349.71	
13.78	0.93	0.45	607	1358	13.78	0.93	0.45	607	1358	606.97	1357.80	
13.94	0.93	0.45	611	1366	13.94	0.93	0.45	611	1366	610.85	1365.85	
14.11	0.93	0.45	615	1374	14.11	0.92	0.45	615	1374	614.74	1373.89	
14.27	0.93	0.45	619	1382	14.27	0.92	0.45	619	1382	618.62	1381.94	
14.44	0.93	0.45	623	1390	14.44	0.91	0.45	623	1390	622.50	1389.98	
14.60	0.93	0.45	626	1398	14.60	0.91	0.45	626	1353	626.38	1375.68	
14.76	0.93	0.45	630	1406	14.76	0.90	0.45	630	1361	630.26	1383.58	
14.93	0.93	0.45	634	1414	14.93	0.89	0.45	634	1369	634.15	1391.49	
15.09	0.92	0.45	638	1422	15.09	0.87	0.45	638	1333	638.03	1377.33	
15.26	0.92	0.45	642	1430	15.26	0.85	0.45	642	1297	641.91	1363.57	
15.42	0.91	0.45	646	1392	15.42	0.82	0.45	646	1262	645.79	1327.14	
15.58	0.90	0.45	650	1400	15.58	0.79	0.45	650	9333	649.67	5366.21	
15.75	0.90	0.45	654	1408	15.75	0.77	0.45	654	8012	653.56	4709.64	
15.91	0.89	0.45	657	1415	15.91	0.77	0.45	657	7577	657.44	4495.99	
16.08	0.88	0.45	661	1423	16.08	0.77	0.45	661	7783	661.32	4603.30	
16.24	0.88	0.45	665	1385	16.24	0.78	0.45	665	8599	665.20	4991.97	
16.40	0.88	0.45	669	1392	16.40	0.79	0.45	669	9765	669.08	5578.56	
16.57	0.88	0.45	673	1400	16.57	0.81	0.45	673	1311	672.96	1355.51	
16.73	0.88	0.45	677	1407	16.73	0.82	0.45	677	1318	676.85	1362.75	
16.90	0.88	0.45	681	1415	16.90	0.83	0.45	681	1369	680.73	1392.16	
17.06	0.88	0.45	685	1422	17.06	0.83	0.45	685	1377	684.61	1399.53	
17.22	0.88	0.45	688	1430	17.22	0.83	0.45	688	1384	688.49	1406.90	
17.39	0.87	0.45	692	1437	17.39	0.83	0.45	692	1391	692.37	1414.27	
17.55	0.87	0.45	696	1445	17.55	0.83	0.45	696	1398	696.26	1421.64	
17.72	0.87	0.45	700	1453	17.72	0.83	0.45	700	1406	700.16	1429.05	
17.88	0.86	0.45	704	1460	17.88	0.83	0.45	704	1413	704.04	1436.42	
18.05	0.85	0.45	708	1468	18.05	0.84	0.45	708	1420	707.93	1443.79	
18.21	0.85	0.45	712	1427	18.21	0.85	0.45	712	1475	711.81	1451.16	
18.37	0.84	0.45	716	1435	18.37	0.87	0.45	716	1483	715.69	1458.53	
18.54	0.84	0.45	720	1442	18.54	0.88	0.45	720	1490	719.57	1465.89	
18.70	0.83	0.45	723	1449	18.70	0.89	0.45	723	1548	723.45	1498.28	
18.87	0.83	0.45	727	1456	18.87	0.89	0.45	727	1555	727.33	1505.78	
19.03	0.83	0.45	731	1463	19.03	0.88	0.45	731	1563	731.22	1513.28	
19.19	0.84	0.45	735	1471	19.19	0.86	0.45	735	1520	735.10	1495.37	
19.36	0.85	0.45	739	1528	19.36	0.83	0.45	739	1478	738.98	1502.74	
19.52	0.86	0.45	743	1535	19.52	0.80	0.45	743	1437	742.86	1485.91	AVES 10-20
19.69	0.85	0.45	747	1543	19.69	0.77	0.45	747	8616	746.74	5079.04	1850.40
19.85	0.85	0.45	751	1500	19.85	0.74	0.45	751	7239	750.63	4369.24	AVEQ 10-20
20.01	0.84	0.45	755	1507	20.01	3.47	0.45	755	6480	754.51	3993.51	636.08
20.18	0.83	0.45	758	1514	20.18	3.29	0.45	758	6171	758.39	3842.59	
20.34	0.83	0.45	762	1521	20.34	3.28	0.45	762	6197	762.27	3859.42	
20.51	0.84	0.45	766	1529	20.51	3.37	0.45	766	6395	766.15	3961.81	
20.67	0.83	0.45	770	1536	20.67	3.35	0.45	770	6388	770.04	3962.06	
20.83	0.82	0.45	774	1493	20.83	3.23	0.45	774	6199	773.92	3845.63	
21.00	0.81	0.45	778	1500	21.00	3.22	0.45	778	6218	777.80	3859.04	
21.16	0.80	0.45	782	1507	21.16	3.33	0.45	782	6465	781.68	3985.83	
21.33	0.79	0.45	786	1514	21.33	3.50	0.45	786	6838	785.56	4175.58	
21.49	0.78	0.45	789	1471	21.49	3.59	0.45	789	7035	789.47	4252.83	
21.65	0.77	0.45	793	1477	21.65	0.74	0.45	793	7329	793.35	4403.36	
21.82	0.76	0.45	797	8896	21.82	0.74	0.45	797	7712	797.23	8304.02	
21.98	0.77	0.45	801	1491	21.98	0.75	0.45	801	8177	801.11	4833.72	
22.15	0.79	0.45	805	1498	22.15	0.76	0.45	805	8741	805.00	5119.53	
22.31	0.80	0.45	809	1556	22.31	0.77	0.45	809	9450	808.88	5502.58	
22.47	0.80	0.45	813	1563	22.47	0.78	0.45	813	10065	812.76	5813.72	
22.64	0.80	0.45	817	1570	22.64	0.78	0.45	817	10134	816.64	5851.67	
22.80	0.81	0.45	821	1577	22.80	0.77	0.45	821	9841	820.52	5708.61	
22.97	0.81	0.45	824	1584	22.97	0.77	0.45	824	9379	824.41	5481.30	
23.13	0.81	0.45	828	1591	23.13	4.30	0.45	828	8903	828.29	5246.72	
23.29	0.82	0.45	832	1598	23.29	4.01	0.45	832	8341	832.17	4969.00	
23.46	0.82	0.45	836	1604	23.46	3.87	0.45	836	8096	836.05	4850.47	
23.62	0.82	0.45	840	1611	23.62	3.86	0.45	840	8110	839.93	4860.55	
23.79	0.81	0.45	844	1618	23.79	3.83	0.45	844	8102	843.81	4860.05	
23.95	0.80	0.45	848	1625	23.95	3.72	0.45	848	7904	847.70	4764.73	
24.11	0.79	0.45	852	1579	24.11	3.48	0.45	852	7427	851.58	4502.52	
24.28	0.79	0.45	855	1585	24.28	3.33	0.45	855	7140	855.46	4362.62	
24.44	0.79	0.45	859	1592	24.44	3.16	0.45	859	6826	859.34	4208.80	
24.61	0.79	0.45	863	1599	24.61	3.13	0.45	863	6780	863.22	4189.59	
24.77	0.78	0.45	867	1605	24.77	3.07	0.45	867	6701	867.11	4153.15	
24.93	0.77	0.45	871	1612	24.93	3.07	0.45	871	6731	870.99	4171.72	
25.10	0.77	0.45	875	9788	25.10	3.06	0.45	875	6744	874.87	8265.99	
25.26	0.76	0.45	879	9527	25.26	3.10	0.45	879	6865	878.75	8196.22	
25.43	0.76	0.45	883	9358	25.43	3.15	0.45	883	6992	882.66	8174.90	
25.59	0.75	0.45	887	9088	25.59	3.28	0.45	887	7332	886.54	8209.96	

F.11

CPT-1					CPT-2					Seismic Residual Average (psf)	Static Strength Average (psf)	
Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)	Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)			
25.76	0.75	0.45	890	8814	25.76	3.29	0.45	890	7371	890.42	8092.52	
25.92	0.75	0.45	894	8908	25.92	3.20	0.45	894	7212	894.30	8059.90	
26.08	0.75	0.45	898	8976	26.08	3.05	0.45	898	6911	898.18	7943.75	
26.25	0.75	0.45	902	8739	26.25	2.89	0.45	902	6572	902.07	7655.22	
26.41	3.55	0.45	906	7921	26.41	2.81	0.45	906	6415	905.95	7167.78	
26.58	3.14	0.45	910	7041	26.58	2.82	0.45	910	6472	909.83	6756.58	
26.74	2.85	0.45	914	6427	26.74	3.39	0.45	914	7826	913.71	7126.55	
26.90	2.75	0.45	918	6231	26.90	0.76	0.45	918	9692	917.59	7961.41	
27.07	2.80	0.45	921	6371	27.07	0.75	0.45	921	9337	921.48	7853.96	
27.23	2.89	0.45	925	6606	27.23	0.72	0.45	925	7231	925.36	6918.24	
27.40	3.00	0.45	929	6896	27.40	1.88	0.45	929	4427	929.24	5661.52	
27.56	3.17	0.45	933	7319	27.56	1.56	0.45	933	3687	933.12	5502.96	
27.72	3.35	0.45	937	7761	27.72	1.76	0.45	937	4159	937.00	5959.77	
27.89	3.38	0.45	941	7872	27.89	0.67	0.45	941	5188	940.89	6529.99	
28.05	3.16	0.45	945	7387	28.05	0.69	0.45	945	6103	944.77	6744.74	
28.22	2.94	0.45	949	6901	28.22	0.71	0.45	949	6981	948.65	6941.25	
28.38	3.09	0.45	953	7302	28.38	0.73	0.45	953	8111	952.53	7706.57	
28.54	0.74	0.45	956	8576	28.54	0.74	0.45	956	8492	956.41	8534.21	
28.71	0.75	0.45	960	1708	28.71	0.72	0.45	960	7553	960.29	4630.29	
28.87	0.75	0.45	964	9364	28.87	2.20	0.45	964	5372	964.18	7368.08	
29.04	0.74	0.45	968	1721	29.04	1.74	0.45	968	4257	968.06	2988.85	
29.20	0.79	0.45	972	1787	29.20	2.32	0.45	972	5706	971.94	3746.85	
29.36	0.84	0.45	976	1920	29.36	0.72	0.45	976	7778	975.85	4848.70	
29.53	0.88	0.45	980	1993	29.53	0.74	0.45	980	9008	979.73	5500.49	
29.69	0.89	0.45	984	2068	29.69	0.75	0.45	984	1747	983.61	1907.61	
29.86	0.93	0.45	987	2146	29.86	0.77	0.45	987	1814	987.49	1980.40	
30.02	0.98	0.45	991	2227	30.02	0.79	0.45	991	1821	991.37	2024.34	
30.18	0.99	0.45	995	2312	30.18	0.78	0.45	995	1828	995.26	2069.82	
30.35	0.98	0.45	999	2320	30.35	0.77	0.45	999	11359	999.14	6839.65	
30.51	0.95	0.45	1003	2252	30.51	0.77	0.45	1003	11474	1003.02	6863.33	
30.68	0.94	0.45	1007	2187	30.68	0.79	0.45	1007	1848	1006.90	2017.36	
30.84	0.93	0.45	1011	2195	30.84	0.81	0.45	1011	1919	1010.78	2056.76	
31.00	0.92	0.45	1015	2203	31.00	0.83	0.45	1015	1992	1014.66	2097.33	
31.17	0.92	0.45	1019	2211	31.17	0.83	0.45	1019	1999	1018.55	2104.97	
31.33	0.91	0.45	1022	2146	31.33	0.82	0.45	1022	2006	1022.43	2076.13	
31.50	0.89	0.45	1026	2154	31.50	0.82	0.45	1026	1947	1026.31	2050.20	
31.66	0.85	0.45	1030	2090	31.66	0.82	0.45	1030	1954	1030.19	2021.96	
31.82	0.81	0.45	1034	1961	31.82	0.83	0.45	1034	2028	1034.07	1994.52	
31.99	0.76	0.45	1038	1838	31.99	0.84	0.45	1038	2035	1037.96	1936.78	
32.15	0.71	0.45	1042	7495	32.15	0.84	0.45	1042	2043	1041.84	4768.66	
32.32	0.69	0.45	1046	1727	32.32	0.85	0.45	1046	2050	1045.72	1888.54	
32.48	0.69	0.45	1050	1733	32.48	0.85	0.45	1050	2057	1049.60	1895.18	
32.64	0.69	0.45	1053	1739	32.64	0.85	0.45	1053	2064	1053.48	1901.82	
32.81	0.70	0.45	1057	1745	32.81	0.85	0.45	1057	2072	1057.37	1908.46	
32.97	0.69	0.45	1061	1751	32.97	0.84	0.45	1061	2079	1061.25	1915.09	
33.14	0.67	0.45	1065	5537	33.14	0.83	0.45	1065	2086	1065.13	3811.74	
33.30	0.66	0.45	1069	5224	33.30	0.82	0.45	1069	2093	1069.03	3658.90	
33.47	0.67	0.45	1073	5487	33.47	0.81	0.45	1073	2031	1072.92	3758.87	AVES 20-34
33.63	0.68	0.45	1077	1714	33.63	0.80	0.45	1077	2038	1076.80	1875.87	4745.01
33.79	0.68	0.45	1081	1720	33.79	0.79	0.45	1081	1976	1080.68	1848.05	AVEQ 20-34
33.96	0.68	0.45	1085	1726	33.96	0.23	0.23	562	1983	823.71	1854.33	916.49
34.12	0.68	0.45	1088	5861	34.12	0.12	0.12	285	1990	686.97	3925.37	
34.29	0.67	0.45	1092	5789	34.29	0.07	0.07	158	1929	625.19	3859.29	
34.45	0.69	0.45	1096	6711	34.45	0.04	0.04	103	1806	599.55	4258.43	
34.61	0.75	0.45	1100	1942	34.61	0.06	0.06	157	1688	628.36	1814.87	
34.78	0.83	0.45	1104	2159	34.78	0.21	0.21	514	2648	808.86	2403.35	
34.94	0.87	0.45	1108	2240	34.94	0.30	0.30	732	1957	920.16	2098.62	
35.11	0.88	0.45	1112	2248	35.11	0.27	0.27	657	2270	884.46	2258.80	
35.27	0.87	0.45	1116	2255	35.27	1.11	0.45	1116	3119	1115.62	2686.89	
35.43	0.87	0.45	1119	2263	35.43	1.52	0.45	1119	4291	1119.50	3276.78	
35.60	0.88	0.45	1123	2348	35.60	0.66	0.45	1123	5160	1123.38	3753.67	
35.76	0.89	0.45	1127	2356	35.76	0.65	0.45	1127	4979	1127.26	3667.35	
35.93	0.89	0.45	1131	2363	35.93	1.38	0.45	1131	3934	1131.14	3148.64	
36.09	0.89	0.45	1135	2371	36.09	0.25	0.25	639	2381	887.25	2375.98	
36.25	0.90	0.45	1139	2379	36.25	0.22	0.22	546	2248	842.57	2313.48	
36.42	0.90	0.45	1143	2387	36.42	0.08	0.08	211	1687	676.83	2036.74	
36.58	0.90	0.45	1147	2394	36.58	0.10	0.10	247	1819	696.75	2106.57	
36.75	0.88	0.45	1151	2402	36.75	0.16	0.16	414	1890	782.46	2146.21	
36.91	0.86	0.45	1154	2330	36.91	0.69	0.45	1154	1896	1154.44	2113.20	
37.07	0.83	0.45	1158	2260	37.07	0.67	0.45	1158	1836	1158.32	2048.04	
37.24	0.79	0.45	1162	2118	37.24	1.76	0.45	1162	5152	1162.22	3634.82	
37.40	0.74	0.45	1166	1983	37.40	1.47	0.45	1166	4320	1166.11	3151.42	
37.57	0.70	0.45	1170	1921	37.57	1.36	0.45	1170	4008	1169.99	2964.46	
37.73	0.13	0.13	349	1860	37.73	1.69	0.45	1174	4991	761.64	3425.29	
37.89	0.16	0.16	422	1865	37.89	0.69	0.45	1178	1933	799.71	1898.97	
38.06	0.16	0.16	408	1939	38.06	0.74	0.45	1182	2008	794.89	1973.22	
38.22	0.74	0.45	1186	2014	38.22	0.75	0.45	1186	2085	1185.51	2049.64	
38.39	0.77	0.45	1189	2165	38.39	0.72	0.45	1189	2020	1189.40	2092.67	

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CPT-1					CPT-2					Seismic Residual Average (psf)	Static Strength Average (psf)	
Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)	Depth (ft)	Su ratio Liq.		Seismic Residual Strength (psf)	Static Shear Strength (psf)			
38.55	0.78	0.45	1193	2172	38.55	0.66	0.45	1193	1822	1193.28	1996.91	
38.71	0.76	0.45	1197	2105	38.71	0.29	0.29	765	2298	981.32	2201.32	
38.88	0.11	0.11	306	2039	38.88	0.11	0.11	295	991	300.32	1515.30	
39.04	0.08	0.08	207	1906	39.04	0.03	0.03	71	647	139.16	1276.34	
39.21	0.07	0.07	180	1844	39.21	0.02	0.02	66	503	123.15	1173.93	
39.37	0.06	0.06	162	1918	39.37	0.03	0.03	80	468	121.13	1192.65	
39.53	0.06	0.06	169	1924	39.53	0.02	0.02	63	425	115.80	1174.34	
39.70	0.07	0.07	186	1861	39.70	0.02	0.02	66	413	126.28	1137.26	
39.86	0.08	0.08	214	1935	39.86	0.03	0.03	75	371	144.69	1153.01	
40.03	0.07	0.07	191	2157	40.03	0.03	0.03	81	383	135.73	1269.79	
40.19	0.79	0.45	1232	2239	40.19	0.03	0.03	94	500	662.86	1369.39	
40.35	0.80	0.45	1236	2324	40.35	0.04	0.04	108	703	672.08	1513.37	
40.52	0.77	0.45	1240	2253	40.52	0.06	0.06	161	899	700.26	1575.79	
40.68	0.75	0.45	1244	2183	40.68	0.09	0.09	240	997	742.04	1589.89	
40.85	0.15	0.15	420	2114	40.85	0.12	0.12	327	1044	373.47	1578.94	
41.01	0.08	0.08	213	2196	41.01	0.23	0.23	628	1425	420.71	1810.62	
41.18	0.09	0.09	256	2202	41.18	0.25	0.25	706	1992	481.07	2097.03	
41.34	0.08	0.08	231	2209	41.34	0.26	0.26	719	2317	475.18	2262.70	
41.50	0.06	0.06	155	2139	41.50	0.22	0.22	620	2225	387.67	2182.28	
41.67	0.08	0.08	228	1999	41.67	0.79	0.45	1267	2467	747.30	2232.88	
41.83	0.08	0.08	229	1934	41.83	1.08	0.45	1271	3361	749.84	2647.31	
42.00	0.15	0.15	437	3192	42.00	1.18	0.45	1275	3687	855.91	3439.44	
42.16	0.14	0.14	401	2394	42.16	1.03	0.45	1279	3230	840.03	2811.97	
42.32	0.06	0.06	175	1333	42.32	0.26	0.26	746	2164	460.50	1748.23	
42.49	0.02	0.02	69	1095	42.49	0.19	0.19	538	1612	303.16	1353.56	
42.65	0.02	0.02	52	1076	42.65	0.17	0.17	484	1400	267.98	1238.10	
42.82	0.03	0.03	90	1210	42.82	0.26	0.26	738	1554	414.48	1381.91	
42.98	0.02	0.02	47	1268	42.98	0.33	0.33	941	1926	493.97	1597.38	
43.14	0.02	0.02	49	1237	43.14	0.26	0.26	756	2070	402.28	1653.02	
43.31	0.02	0.02	64	1061	43.31	0.28	0.28	807	2107	435.26	1583.95	
43.47	0.03	0.03	84	1007	43.47	0.26	0.26	760	1938	421.81	1472.48	
43.64	0.04	0.04	109	998	43.64	0.65	0.45	1314	2080	711.50	1538.72	
43.80	0.02	0.02	71	966	43.80	0.67	0.45	1318	2145	694.35	1555.24	
43.96	0.02	0.02	68	935	43.96	0.97	0.45	1321	3138	694.52	2036.68	
44.13	0.03	0.03	77	906	44.13	1.51	0.45	1325	4862	701.08	2884.12	
44.29	0.02	0.02	73	930	44.29	0.68	0.45	1329	6723	701.24	3826.38	
44.46	0.03	0.03	92	941	44.46	0.70	0.45	1333	2174	712.41	1557.37	
44.62	0.02	0.02	59	955	44.62	0.71	0.45	1337	8807	697.87	4881.18	
44.78	0.03	0.03	77	955	44.78	0.73	0.45	1341	2265	709.06	1609.91	
44.95	0.03	0.03	88	966	44.95	0.75	0.45	1345	2352	716.59	1658.75	
45.11	0.06	0.06	177	970	45.11	0.78	0.45	1349	2441	762.86	1705.57	
45.28	0.05	0.05	163	1022	45.28	0.79	0.45	1352	2448	757.50	1735.05	
45.44	0.06	0.06	179	1091	45.44	0.81	0.45	1356	2541	767.85	1816.08	
45.60	0.06	0.06	185	1161	45.60	0.83	0.45	1360	2636	772.66	1898.63	
45.77	0.06	0.06	176	1187	45.77	0.87	0.45	1364	2735	770.06	1961.24	
45.93	0.05	0.05	143	1140	45.93	0.90	0.45	1368	2837	755.41	1988.39	
46.10	0.04	0.04	121	1088	46.10	0.91	0.45	1372	2845	746.29	1966.39	
46.26	0.05	0.05	139	1072	46.26	0.91	0.45	1376	2853	757.59	1962.33	
46.42	0.04	0.04	114	1136	46.42	0.88	0.45	1380	2765	746.58	1950.35	
46.59	0.05	0.05	145	1165	46.59	0.85	0.45	1384	2680	764.42	1922.28	
46.75	0.05	0.05	159	1162	46.75	0.80	0.45	1387	2597	772.96	1879.36	
46.92	0.05	0.05	164	1305	46.92	0.76	0.45	1391	2430	777.78	1867.53	
47.08	0.05	0.05	159	1525	47.08	0.74	0.45	1395	2353	777.01	1938.95	
47.24	0.03	0.03	99	1592	47.24	0.72	0.45	1399	2359	749.11	1975.22	
47.41	0.03	0.03	86	1423	47.41	0.70	0.45	1403	2283	744.54	1853.08	
47.57	0.08	0.08	249	1352	47.57	0.67	0.45	1407	6792	828.04	4071.67	
47.74	0.15	0.15	461	2023	47.74	1.60	0.45	1411	5519	936.03	3770.80	
47.90	0.05	0.05	166	1915	47.90	1.32	0.45	1415	4571	790.10	3243.00	
48.06	0.05	0.05	155	2070	48.06	0.32	0.32	1015	3875	584.89	2972.26	
48.23	0.05	0.05	168	2075	48.23	0.19	0.19	597	3407	382.17	2740.94	
48.39	0.06	0.06	188	2080	48.39	0.21	0.21	655	3168	421.86	2624.26	
48.56	0.08	0.08	252	2086	48.56	0.23	0.23	730	3037	491.09	2561.40	
48.72	0.10	0.10	326	2091	48.72	0.22	0.22	703	3091	514.63	2591.31	
48.89	0.14	0.14	453	2255	48.89	0.25	0.25	784	3321	618.55	2788.15	
49.05	0.72	0.45	1442	2428	49.05	0.29	0.29	919	3541	1180.27	2984.59	
49.21	0.75	0.45	1446	2521	49.21	1.05	0.45	1446	3701	1445.67	3110.77	
49.38	0.77	0.45	1450	2527	49.38	1.01	0.45	1450	3575	1449.55	3051.06	
49.54	0.76	0.45	1453	2534	49.54	0.93	0.45	1453	3298	1453.44	2915.73	AVES 34-50
49.71	0.14	0.14	459	2453	49.71	0.45	0.45	1441	2835	950.29	2643.82	2282.03
49.87	0.06	0.06	179	2459	49.87	0.01	0.01	37	2496	107.63	2477.82	AVEQ 34-50
50.03	0.08	0.08	249	NA	50.03	0.03	0.03	109	2409	179.10	2408.94	714.10
									Avg.	655.67	2525.03	

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Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test

P. K. Robertson[1]

Abstract: Flow liquefaction is a major design issue for large soil structures such as mine tailings impoundments and earth dams. If a soil is strain softening in undrained shear and, hence, susceptible to flow liquefaction, an estimate of the resulting liquefied shear strength is required for stability analyses. Many procedures have been published for estimating the residual or liquefied shear strength of cohesionless soils. This paper presents cone penetration test-based relationships to evaluate the susceptibility to strength loss and liquefied shear strength for a wide range of soils. Case-history analyses by a number of investigators are reviewed and used with some additional case histories. Extrapolations beyond the case-history data are guided by laboratory studies and theory.

DOI: 10.1061/(ASCE)GT.1943-5606.0000286

CE Database subject headings: Liquefaction; Cone penetration tests; Soil structures; Case studies; History.

Author keywords: Liquefaction; Strength loss; Cone penetration test; Case histories.

Introduction

Soil liquefaction is a major concern for structures constructed with or on saturated sandy soils. The phenomenon of soil liquefaction has been recognized for many years. Terzaghi and Peck (1967) referred to "spontaneous liquefaction" to describe the sudden loss of strength of very loose sands that caused flow slides due to a slight disturbance. Since 1964, much work has been carried out to explain and understand soil liquefaction. This progress has been described in a series of state-of-the-art papers such as those by Yoshimi et al. (1977), Seed (1979), Finn (1981), Ishihara (1993), Robertson and Fear (1995), Youd et al. (2001), and Robertson (2009c). Much of the work in the past three decades has been on liquefaction induced during earthquake loading (i.e., cyclic liquefaction).

Robertson and Wride (1998) distinguished between liquefaction due to cyclic loading, where the effective overburden stress can reach zero during cyclic loading with a resulting loss of soil stiffness (cyclic liquefaction/softening), and liquefaction due to strain softening with a resulting loss of shear strength (flow liquefaction) and presented a simplified flowchart to aid in the evaluation. Flow liquefaction is also referred to as static liquefaction (e.g., Jefferies and Been 2006). However, since the phenomenon can be triggered by either static or cyclic loading, the term flow liquefaction is used throughout this paper. Flow liquefaction can occur in any saturated or near saturated metastable soil such as very loose cohesionless sands and silts as well as very sensitive clays. For failure of a soil structure, such as a slope or embankment, a sufficient volume of material must strain soften. The resulting failure can be a slide or a flow depending on the material characteristics of the soils and ground geometry. The resulting movements are due to internal, gravity-induced stresses and can occur after the triggering mechanism occurs.

Flow liquefaction is a major design issue for large soil structures such as mine tailings impoundments and earth dams. A liquefaction analysis of sloping ground (i.e., ground subject to a static driving shear stress) is a challenge for geotechnical engineers. Many procedures have been published for estimating the residual or liquefied shear strength of cohesionless soils. Some procedures require a laboratory testing of field samples obtained by ground freezing techniques (e.g., Robertson et al. 2000) or samples obtained by high-quality tube samples coupled with procedures for "correcting" the shear strength for disturbance during sampling and testing (e.g., Castro 1975). However, procedures based on case histories remain the most popular (e.g., Seed 1987; Davis et al. 1988; Seed and Harder 1990; Stark and Mesri 1992; Ishihara 1993; Konrad and Watts 1995; Yoshimine et al. 1999; Olson and Stark 2002; Idriss and Boulanger 2007). Olson and Stark (2002, 2003) suggested a detailed procedure, based on an extensive database of case histories, consisting of three tasks: (1) evaluate susceptibility to strength loss; (2) evaluate liquefied shear strength, $s_{u(\mathrm{liq})}$, and postliquefaction stability; and (3) evaluate if liquefaction will be triggered. Olson and Stark (2002) defined the liquefied shear strength, $s_{u(\mathrm{liq})}$, as the shear strength mobilized at large deformation by a saturated contractive soil following the triggering of a strain-softening response. Others have used the term undrained residual shear strength (e.g., Seed and Harder 1990) or undrained steady-state shear strength (Poulos et al. 1985). The term liquefied shear strength will be used in this paper to be consistent with the more recent Olson and Stark (2002) terminology. The Olson and Stark (2002) procedure uses normalized penetration resistance with no correction for soil type.

Idriss and Boulanger (2007) suggested a correlation to evaluate the liquefied shear strength using an equivalent clean sand normalized penetration resistance. To obtain equivalent clean sand penetration resistance values, Idriss and Boulanger (2007) used the original standard penetration test (SPT)-based correction factors suggested by Seed (1987) and Seed and Harder (1990),

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Note. This manuscript was submitted on June 2, 2009; approved on November 17, 2009; published online on November 19, 2009. Discussion period open until November 1, 2010; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 136, No. 6, June 1, 2010. ©ASCE, ISSN 1090-0241/2010/6-842–853/$25.00.

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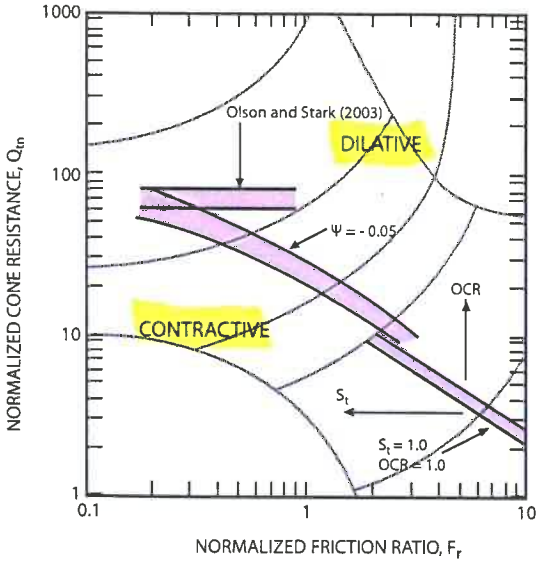

Fig. 3. Approximate boundary between dilative and contractive soil response using normalized CPT parameters

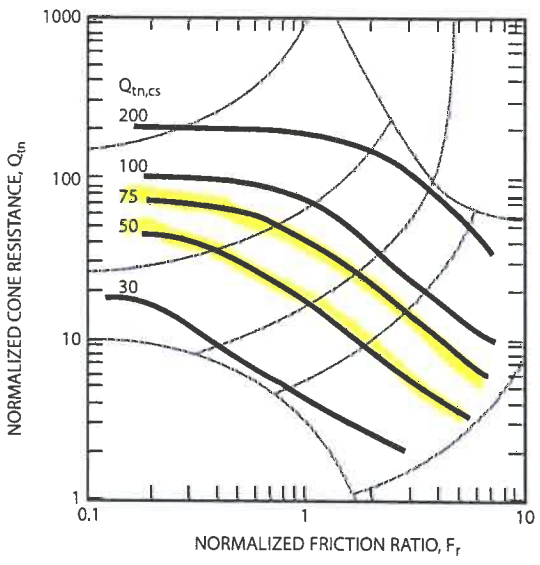

Fig. 4. Contours of equivalent clean sand normalized cone resistance, $Q_{tn,cs}$, based on corrections suggested by Robertson and Wride (1998)

cepts to the CPT results (Been et al. 1986) and to soil liquefaction (Jefferies and Been 2006). The state parameter (ψ) is defined as the difference between the in situ void ratio, e_o, and the void ratio at critical state, e_{cs}, at the same mean effective stress, p' (Been and Jefferies 1985). Jefferies and Been (2006) provided a detailed description of the evaluation of the soil state using the CPT and show that the inverse problem of evaluating the state from the CPT response is complex and depends on several soil parameters. For high risk projects, a detailed interpretation of the CPT results using laboratory results and numerical modeling can be appropriate (e.g., Shuttle and Cunning 2007), although soil variability can complicate the interpretation procedure.

For low risk projects and in the initial screening for high risk projects, it is often appropriate to estimate the state of the soil directly from the CPT results. Plewes et al. (1992) developed a screening method to estimate the state of the soil using the normalized CPT results. This was subsequently updated and modified by Jefferies and Been (2006). In a general sense, soils with a state denser than the critical state ($\psi < 0$) will be dilative and will strain harden in undrained shear, whereas soils with a state looser than the critical state ($\psi > 0$) will be contractive and will strain soften in undrained shear. Jefferies and Been (2006) and Shuttle and Cunning (2007) suggested that when a soil has a state parameter $\psi > -0.05$, strain softening and strength loss in undrained shear can be expected. Hence, defining a region on the CPT SBT chart that represents a state parameter of about -0.05 is helpful as a screening technique to identify the susceptibility for flow liquefaction.

Based on the work of Plewes et al. (1992), Jefferies and Been (2006), and Shuttle and Cunning (2007), combined with the test results from frozen samples (Robertson et al. 2000), it is possible to identify a zone on the normalized SBT, based on Q_{tn} and F_r, that represents the approximate boundary between the dilative and contractive soil response as shown in Fig. 3. Also identified in Fig. 3 is the region that defines normally consolidated clays with a sensitivity of 1 or more based on the work of Robertson (2009b). Robertson (2009b) showed that as the soil sensitivity increases in fine-grained soils, the measured CPT normalized friction ratio (F_r) decreases as indicated in Fig. 3. As shown by Plewes et al. (1992) and Jefferies and Been (2006) and illustrated in Fig. 3, there is a clear trend for the normalized cone resistance to decrease as I_c increases for the same soil state. Fig. 3 indicates that the region in the lower left portion of the SBT chart defines soils that are likely susceptible to contractive behavior and strength loss in undrained shear. Included in Fig. 3 is the approximate boundary between the contractive and dilative soil response suggested by Olson and Stark (2003) based on the normalized penetration resistance. The Olson and Stark (2003) criteria vary slightly with effective overburden stress and a range is presented. It is clear from Fig. 3 that the criterion suggested by Olson and Stark (2003) applies only to clean sands where typically $F_r < 1\%$.

Robertson and Wride (1998) suggested a correction factor to correct the normalized cone resistance in silty sands to an equivalent clean sand value ($Q_{tn,cs}$) using the following:

$$Q_{tn,cs} = K_c Q_{tn} \tag{6}$$

where K_c=a correction factor that is a function of grain characteristics (combined influence of FC, mineralogy, and plasticity) of the soil that can be estimated using I_c as follows:

$$K_c = 1.0 \text{ if } I_c \leq 1.64 \tag{7}$$

$$K_c = 5.581 I_c^3 - 0.403 I_c^4 - 21.63 I_c^2 + 33.75 I_c - 17.88 \text{ if } I_c > 1.64 \tag{8}$$

Fig. 4 shows the contours of the equivalent clean sand cone resistance, $Q_{tn,cs}$, on the CPT SBT chart. The contours of $Q_{tn,cs}$ follow a trend similar to the dilative-contractive boundary defined in Fig. 3 and that a value of $Q_{tn,cs}$ between 50 and 70 likely represents the boundary between the contractive and dilative state for a wide range of soils. Robertson (2010) indicated that the contours of $Q_{tn,cs}$ are essentially contours of the state parameter (ψ). Hence, soils with a constant value of $Q_{tn,cs}$ have essentially a similar state parameter and hence a similar response to loading. Although the concept of an equivalent clean sand penetration resistance stems from the early work of Seed (1979), based on a "FC correction," Jefferies and Been (2006) showed that a correc-

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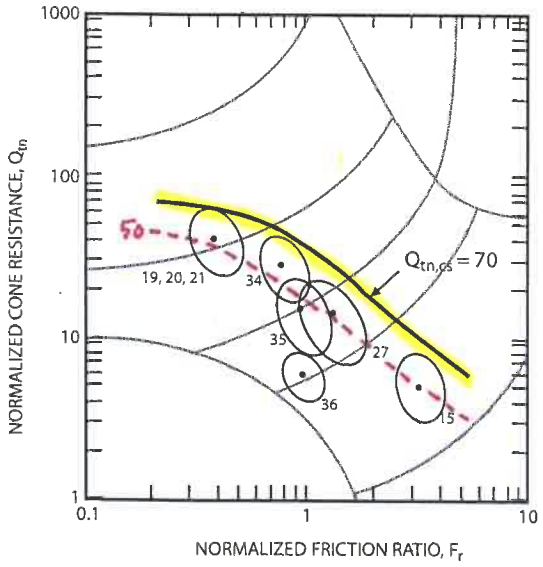

Fig. 5. Range (mean and standard deviation) of normalized CPT data from Class A flow liquefaction failure case histories (numbers represent case histories from Table 1)

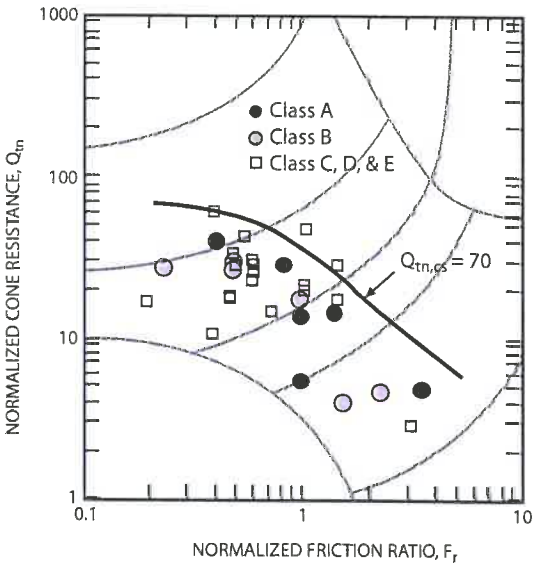

Fig. 6. Mean values of normalized CPT data from flow liquefaction failure case histories

tion based on the slope of the critical state line (λ) is essentially similar but conceptually more correct. Jefferies and Been (2006) also showed that λ was a function of the CPT SBT index (I_c). Hence, the equivalent clean sand cone resistance approach using a correction based on I_c, as suggested by Robertson and Wride (1998), is supported, in a general sense, by the theoretical approach described by Jefferies and Been (2006) based on critical state soil mechanics.

Fig. 5 presents the CPT data from the six Class A case histories in terms of measured normalized cone results on the normalized CPT-based SBT chart. The CPT data shown in Fig. 5 represent approximately the mean ± 1 standard deviation values within the zone that was considered to have experienced strength loss and involved in the slide. The data within ± 1 standard deviation represent about 68% of the data (i.e., approximately 16% is smaller and 16% larger). The measured CPT results from each case history represent a region on the normalized SBT chart that illustrate the variation in CPT values within each deposit that experienced strength loss. The mean value would represent approximately the center of each region and the 20-percentile value would be approximately the lower right portion of each region of data for a case history. Also included in Fig. 5 is a contour that represents a clean sand equivalent penetration resistance, $Q_{tn,cs} = 70$. The contour of $Q_{tn,cs} \leq 70$ captures all the CPT results defined by the mean+1 standard deviation (i.e., about 70% of the results within the deposit) for the Class A case histories.

Several flow liquefaction failures have also been documented in the quick clays found in Norway and eastern Canada (e.g., Rissa landslide of 1978). These "quick clays" have normalized CPT values in the region of $Q_{tn} < 10$ and $F_r < 2\%$ (Lunne et al. 1997) similar to the sensitive silty clay found at the Canadian Mine case history (No. 36, Table 1). Hence, the flow liquefaction failures in quick clay also plot within the region defined by the contour $Q_{tn,cs} \leq 70$.

Fig. 6 presents the mean values for all the case histories, but the six cases where full CPT measurements were available (Class A) are represented with solid symbols; Class B by the large shaded symbols; and the less reliable estimated values (Classes C, D, and E) are represented with small open symbols. The contour of $Q_{tn,cs} = 70$ captures the case-history records reasonably well and captures the change in the CPT response as soils vary from clean sands to soft sensitive clays and silts. Any small variation in the estimated values of F_r and I_c for the Classes C, D, and E case histories using the Q_{tn} values selected by Olson (2001) would have little influence on the overall observation. Soils with a $Q_{tn,cs} > 70$ are likely dilative and strain hardening in undrained shear consistent with the state parameter approach described by Jefferies and Been (2006). Hence, the criteria of $Q_{tn,cs} \leq 70$ can be used as a conservative screening method to evaluate if a soil is susceptible to strain softening in undrained shear and, hence, susceptible to flow liquefaction.

Evaluation of Liquefied Shear Strength

Estimating liquefied shear strength values ($s_{u(liq)}$) from failure case histories uses stability calculations that require many simplifying assumptions and idealizations. Most approaches use limit equilibrium methods based on either prefailure or postfailure geometry and may or may not include inertial effects. Many case histories involved retrogressive sliding that is rarely accounted for in the back-analyses. Hence, the evaluation of the liquefied shear strength based on case histories is often approximate at best. Many researchers (e.g., Seed 1987; Seed and Harder 1990; Wride et al. 1999; Olson and Stark 2002; Jefferies and Been 2006) have estimated the shear strength of liquefied soils based on case histories. Jefferies and Been (2006) showed that there is a theoretical link between the state parameter and the liquefied undrained shear strength ratio ($s_{u(liq)}/\sigma'_{vo}$). Since clean sand equivalent normalized penetration resistance ($Q_{tn,cs}$) is essentially equivalent to the state parameter, values of $Q_{tn,cs}$ are compared to $s_{u(liq)}/\sigma'_{vo}$ for the case histories, with emphasis on the Class A case histories. The range of published estimated $s_{u(liq)}/\sigma'_{vo}$ values for the case histories is presented in Table 2 using the same numbering system as shown in Table 1. Olson and Stark (2002) and Olson (2001) described the sources of uncertainty in the analyses and their relative importance. Some case histories have a wide range of estimated

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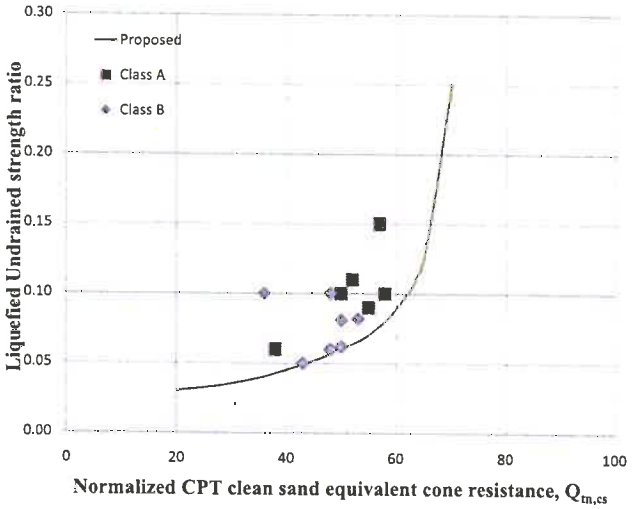

Fig. 7. Liquefied shear strength ratio and normalized CPT clean sand equivalent penetration resistance from Classes A and B flow liquefaction failure case histories

drained shear strength will typically exceed the drained shear strength, although cavitation can limit the full value of the undrained strength. The strength ratio in terms of drained shear strength (in simple shear) can be represented as

$$\tau/\sigma'_{vo} = \tan\varphi' \quad (9)$$

The drained strength ratio in triaxial compression can be larger than defined by Eq. (9). Hence, for soils that are strain hardening in undrained shear, i.e., where $Q_{tn,cs} > 70$, a conservative low estimate of the undrained strength ratio is about 0.4–0.5.

Fig. 7 shows the best estimate values for the liquefied strength ratios and mean CPT clean sand equivalent penetration resistance values for Classes A and B case histories. The more reliable CPT results from the case histories that are Class A are shown in large solid symbols and Class B by the large shaded symbols. Classes C, D, and E case histories are not included in Fig. 7 because of the lack of reliability and uncertainty. However, their assigned values are included in Table 2 for completeness. Based on the observation that no case history, with reliable measured CPT results, had a mean clean sand equivalent normalized penetration resistance, $Q_{tn,cs} > 70$ and the observation that most soils are strain hardening (i.e., not susceptible to strength loss) in undrained shear when $Q_{tn,cs} > 70$ with $\tau/\sigma'_{vo} \sim 0.4$, a proposed lower bound relationship is included in Fig. 7. Based on the above observations, the proposed relationship trends toward $s_{u(liq)}/\sigma'_{vo} = 0.4$ at $Q_{tn,cs} = 70$.

The proposed single relationship can be represented by the following approximate equation: when $Q_{tn,cs} \leq 70$

$$s_{u(liq)}/\sigma'_{vo} = [0.02199 - 0.0003124 Q_{tn,cs}]/[1 - 0.02676 Q_{tn,cs} + 0.0001783 (Q_{tn,cs})^2] \quad (10)$$

where $0.03 \leq s_{u(liq)}/\sigma'_{vo} \leq \tan\varphi'$.

Unlike Seed and Harder (1990) and Olson and Stark (2002) a single relationship is recommended to conservatively capture the more reliable case-history data combined with the observation that most soils are strain hardening when $Q_{tn,cs} > 70$ and therefore not susceptible to flow liquefaction. Table 2 presents the possible range of values for $s_{u(liq)}/\sigma'_{vo}$ and the proposed relationship captures almost all the range for the more reliable Classes A and B case histories. The proposed relationship avoids the possibility of extrapolating beyond the available case-history data, since the proposed relationship trends toward the drained shear strength ratio represented by $\tan\varphi'$. A conservative, essentially lower bound, relationship was selected to capture much of the variability in the back-calculated liquefied shear strength values from the case histories and to recognize the need for caution in soils where $Q_{tn,cs} < 70$.

The proposed relationship can be conservatively low in sensitive clays, where the remolded shear strength ratio $s_{u(r)}/\sigma'_{vo}$ can be defined by

$$s_{u(r)}/\sigma'_{vo} = f_s/\sigma'_{vo} = (F_r Q_{tn})/100 \quad (11)$$

Eq. (11) is based on the observation that the remolded shear strength for most clay soils is approximately equal to the CPT sleeve friction, f_s (Robertson 2009b).

When the proposed relationship is applied to all CPT results within a sounding, it is recommended that the average value of liquefied shear strength within a noninterlayered soil deposit that is considered to be susceptible to strength loss should be applied for stability analyses, since the relationship was based on average CPT values within noninterlayered deposits from case histories. Hence, the variability within one soil type is captured, as illustrated by the range of CPT results for each case history shown in Fig. 5. To avoid excessive conservatism at low confining stress a lower bound value of $s_{u(liq)} = 1$ kPa should be assumed.

Idriss and Boulanger (2007) suggested an alternate more conservative relationship and included a relationship for conditions where the effects of void redistribution could be significant, although little evidence was presented to support this and little guidance was provided to evaluate when this might occur. The Idriss and Boulanger (2007) relationship is inconsistent with the observation that no case histories exist where $Q_{tn,cs} > 70$. The LSFD case history may provide some guidance, since it is possible that the more dense sand layers were not susceptible to strain softening (i.e., flow liquefaction) initially but may have experienced some strain softening due to either void redistribution and/or particle intermixing after the earthquake. The representative mean CPT values in the weaker sand layers for the LSFD case history were $Q_{tn} = 35$, $F_r = 2.3$, and $Q_{tn,cs} = 90$. Many of the sand layers were more dense with even higher values of Q_{tn} and $Q_{tn,cs}$. Given the initial dense state of these sand layers it is likely that strength loss was confined primarily to the weaker sandy silt layers. The CPT results presented in Fig. 2 show that much of the zone where strength loss was considered to have taken place was composed primarily of the weaker sand silt. Although the denser sand layers may not have experienced strength loss, the resulting mixing of the soils as deformations became large may have resulted in the slightly higher back-calculated average strength ratio. In cases where less permeable layers may inhibit pore pressure dissipation and where void redistribution could occur, a more conservative estimate may be appropriate, and for high risk projects, more detailed field and analytical studies should be used.

Summary

An update of the CPT-based criteria to evaluate the susceptibility to flow liquefaction and the liquefied strength ratio based on case histories has been presented. The criteria of $Q_{tn,cs} \leq 70$ can be used as a conservative screening method to evaluate if a

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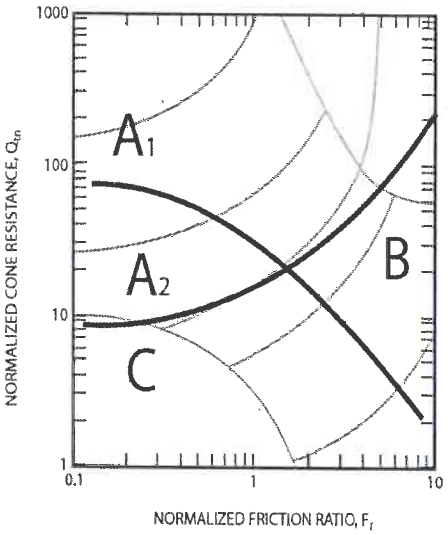

Fig. 8. CPT SBT chart for liquefaction and cyclic softening potential: *cohesionless soils* (A_1 and A_2)—evaluate potential behavior using CPT-based case-history liquefaction correlations. A_1: cyclic liquefaction possible depending on level and duration of cyclic loading. A_2: cyclic liquefaction and flow liquefaction possible depending on loading and ground geometry. *Cohesive soils* (B and C)—evaluate potential behavior based on in situ or laboratory test measurements or estimates of monotonic and cyclic undrained shear strengths. B: cyclic softening possible depending on level and duration of cyclic loading. C: cyclic softening and flow liquefaction possible depending on soil sensitivity, loading, and ground geometry.

soil is susceptible to strain softening in undrained shear and, hence, susceptible to flow liquefaction. The recommended correction factors to derive the clean sand equivalent penetration resistance are supported by the general concepts in critical state soil mechanics.

The case histories indicate that very young, very loose, nonplastic or low-plastic soils tend to be more susceptible to significant and rapid strength loss than older, denser, and/or more plastic soils. Ladd et al. (1977) and Leroueil and Hight (2003) showed that soils with high plasticity tend to be more ductile than soils of low plasticity. Low plastic, highly sensitive, fine-grained soils that have a shear strain at peak undrained shear strength less than about 2% and a rapid loss of strength are more likely to experience rapid strain softening and flow liquefaction than more plastic soils.

Robertson (2009c) presented a summary of the CPT SBT Q_{tn}-F_r chart to identify zones of potential liquefaction and/or cyclic softening as shown in Fig. 8 that can be used as a guide for the choice of engineering procedures to be used in evaluating potential liquefaction and strength loss in different types of soils. The boundary between Zones A_1 and A_2 and Zones B and C can be approximated by the CPT SBT I_c=2.60. The boundary between Zones A_1 and B and Zones A_2 and C can be defined by $Q_{tn,cs}$=70. Zones A_1 and A_2 correspond to cohesionless, predominately sandlike soils for which it is appropriate to use existing CPT case-history based cyclic liquefaction correlations (e.g., Youd et al. 2001; Robertson and Wride 1998). The soils in Zones A_1 and A_2 are susceptible to cyclic liquefaction, while the looser soils in Zone A_2 are more susceptible to strength loss and flow liquefaction. Zones B and C correspond to fine-grained, predominately claylike soils for which it is more appropriate to use procedures similar to, or modified from, those used to evaluate the undrained shear strength of clays (e.g., field vane tests, CPT, and shear strength tests on high-quality thin-walled tube samples). The soils in Zones B and C are susceptible to cyclic softening (e.g., accumulation of strains if the peak seismic/cyclic stresses are sufficiently large), but the softer soils in Zone C can be more sensitive and susceptible to potential strength loss and possible flow liquefaction. For moderate to high risk projects, an undisturbed sampling of soils in Zones B and C is recommended to determine the soil response, since soils in these zones are generally more suitable for conventional high-quality sampling and laboratory testing. High-quality samples are required, since the peak undrained shear strength and the strain to peak strength are sensitive to soil disturbance (Leroueil and Hight 2003). For low risk projects, disturbed samples should be obtained for soils in Zones B and C to estimate if the soils will respond either more sandlike or claylike (Idriss and Boulanger 2004), and evaluate sensitivity (Leroueil et al. 1983), based on Atterberg limits and water content.

A lower bound relationship between the liquefied shear strength and clean sand equivalent normalized penetration resistance is proposed that avoids the need to extrapolate beyond the case-history database. In cases where less permeable layers may inhibit pore pressure dissipation and where void redistribution could occur, a more conservative estimate may be appropriate, and for high risk projects, more detailed field and analytical studies should be carried out. When the proposed relationship is applied to all CPT results within a noninterlayered soil deposit, it is recommended that the average value of liquefied shear strength should be applied for stability analyses, since the relationship was based on average CPT values within noninterlayered deposits from the case histories. To avoid excessive conservatism at low confining stress, a lower bound value of $s_{u(\mathrm{liq})}$=1 kPa should be assumed.

The case histories have shown that when significant strength loss occurs in critical sections of a soil structure, failures are often rapid, occur with little warning, and the resulting deformations are often very large. Experience has also shown that the trigger events can be very small. In general, it is often prudent to assume that triggering will occur if the soils are susceptible to strength loss. Hence, the design for high risk soil structures should be carried out with caution. In general the emphasis in design is primarily on the evaluation of susceptibility and the resulting liquefied shear strength.

The CPT is a useful in situ test that can provide continuous estimates of the potential for flow liquefaction. However, the CPT-based approach is a simplified method that should be used appropriately depending on the risk of the project. For low risk projects, the CPT-based method is appropriate when combined with selective samples to confirm the soil type as well as conservative estimates of soil response. For moderate risk projects, the CPT-based method should be combined with appropriate additional in situ testing, as well as selected undisturbed sampling and laboratory testing, to confirm soil response. A thin-walled tube sampling is generally applicable to fine-grained soils in Zones B and C. For high risk projects, the CPT-based method should be used as an initial screening to identify the extent and nature of potential problems followed by additional in situ testing and appropriate laboratory testing on high-quality samples. An advanced numerical modeling is appropriate for high risk projects where initial screening indicates a need.

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Liquefaction Susceptibility Criteria for Silts and Clays

Ross W. Boulanger, M.ASCE[1]; and I. M. Idriss, M.ASCE[2]

Abstract: New liquefaction susceptibility criteria for saturated silts and clays are presented that are based on the mechanics of their stress-strain behavior and which provide improved guidance for selecting engineering procedures for estimating potential strains and strength loss during seismic loading. Monotonic and cyclic undrained loading test data for silts and clays show that they transition, over a fairly narrow range of plasticity indices (PI), from soils that behave more fundamentally like sands (sand-like behavior) to soils that behave more fundamentally like clays (clay-like behavior), with the distinction having a direct correspondence to the type of engineering procedures that are best suited to evaluating their seismic behavior. It is recommended that the term liquefaction be reserved for describing the development of significant strains or strength loss in fine-grained soils exhibiting sand-like behavior, whereas the term cyclic softening failure be used to describe similar phenomena in fine-grained soils exhibiting clay-like behavior. For practical purposes, clay-like behavior can be expected for fine-grained soils that have PI $\geq$ 7, although a slightly lower transition point for soils with a CL-ML classification (perhaps PI $\geq$ 5 or 6) would be equally consistent with the available data. Issues related to the practical application of these criteria are discussed.

DOI: 10.1061/(ASCE)1090-0241(2006)132:11(1413)

CE Database subject headings: Silts; Clays; Fine-grained soils; Liquefaction; Earthquakes; Cyclic strength; Soil strength.

Introduction

Recent experiences with ground failure in low-plasticity silts and clays during strong earthquakes (e.g., Chu et al. 2004; Bray et al. 2004a; Martin et al. 2004) have highlighted the fact that earthquake loading can trigger the development of significant strains and strength loss in a broad range of saturated soils, from sand to clay, even though earthquake-induced ground failure is observed less frequently in clays than in sands. These experiences have demonstrated the need for an improved fundamental understanding of the seismic behavior of fine-grained soils over a broad range of plasticity characteristics and for related guidance on the engineering procedures that are most appropriate for evaluating their seismic behavior. The procedures that are best used to estimate potential strains and strength loss during earthquake loading are different for sand from those for clay, in the same way that the procedures for estimating their static shear strength and stiffness properties are different. For low-plasticity silts and clays, there is insufficient guidance on the engineering procedures that are most appropriate for estimating potential strains and strength loss during earthquake loading. Consequently, it is common in practice to assume that soils that classify as "liquefiable" by current liquefaction susceptibility criteria should be evaluated using procedures developed primarily for sands, even though current liquefaction susceptibility criteria were not necessarily developed for that purpose.

This paper presents new liquefaction susceptibility criteria for saturated silts and clays subject to earthquake loading. Existing liquefaction susceptibility criteria and their limitations are discussed first. Monotonic and cyclic undrained loading behavior for saturated soils are reviewed, from which it is shown that fine-grained soils transition from behavior that is more fundamentally like sands (sand-like behavior) to behavior that is more fundamentally like clays (clay-like behavior) over a fairly narrow range of Atterberg limits. In describing the observed behaviors, the term liquefaction is used for describing the development of significant strains or strength loss in fine-grained soils exhibiting sand-like behavior, while the term cyclic softening is used to describe similar phenomena in fine-grained soils exhibiting clay-like behavior. Criteria are recommended for distinguishing between fine-grained soils that may exhibit sand-like behavior (liquefaction) versus clay-like behavior (cyclic softening), with corresponding recommendations for engineering procedures that are best suited for assessing their respective behaviors. Additional details on the development of these liquefaction susceptibility criteria are given in Boulanger and Idriss (2004, 2005), along with engineering procedures for evaluating cyclic softening potential in clay-like soils and the application of these criteria and procedures to case histories involving a range of earthquake-induced ground deformations at sites underlain by silt and clay strata.

Current Liquefaction Susceptibility Criteria

The Chinese criteria have been widely used since the early 1980s as a means for evaluating the liquefaction susceptibility of silts and clays. These criteria were based on observations of "liquefaction" in fine-grained soils at various sites in China during strong earthquakes, as reported by Wang (1979). Atterberg limits for those soils that were reported to have liquefied plot as CL,

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Note. Discussion open until April 1, 2007. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on September 2, 2005; approved on May 20, 2006. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 132, No. 11, November 1, 2006. ©ASCE, ISSN 1090-0241/2006/11-1413–1426/$25.00.

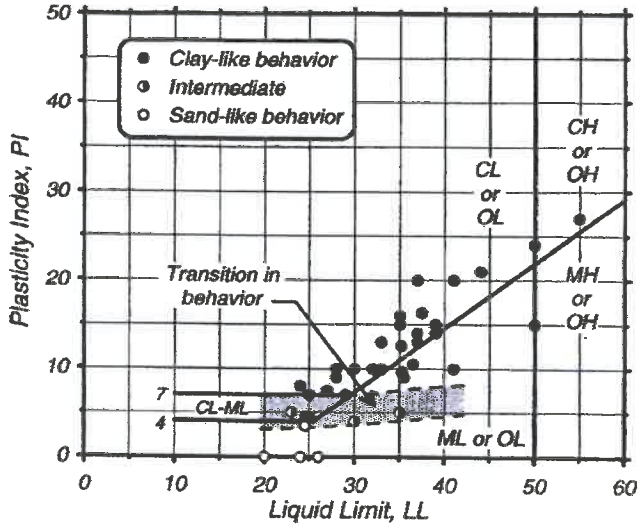

Fig. 14. Atterberg limits chart showing representative values for each soil that exhibited clay-like, sand-like, or intermediate behavior

Summary and Recommendations

The first step in evaluating the potential for ground failure in silts and clays during earthquakes is to determine the appropriate framework and engineering procedures to be used in the evaluation. For fine-grained soils that behave more fundamentally like clays, the cyclic and monotonic undrained shear strengths are closely related and show relatively unique stress-history normalized behaviors. Cyclic strengths can then be evaluated based on information from in situ testing, laboratory testing, and empirical correlations that are similar to, or build upon, established procedures for evaluating the monotonic undrained shear strengths of such deposits (Boulanger and Idriss 2004). For fine-grained soils that behave more fundamentally like sands, the cyclic strengths may be more appropriately estimated within the framework of existing SPT- and CPT-based liquefaction correlations.

For practical purposes, fine-grained soils can confidently be expected to exhibit clay-like behavior if they have PI≥7. This criterion provides a slightly conservative interpretation of the likely transition interval (Figs. 14 and 15), and includes all CL soils by definition. If a soil plots as CL-ML, the PI criterion may be reduced to PI≥5 and still be consistent with the available data. Fine-grained soils that do not meet the above criteria should be considered as likely exhibiting sand-like behavior (i.e., liquefiable), unless shown otherwise through detailed in situ and laboratory testing. Fine-grained soils with PI values of 3–6 may exhibit intermediate behavior, such that detailed in situ and laboratory testing may still provide benefits relative to reliance on SPT- and CPT-based liquefaction correlations alone. Thus, the proposed criteria provide a reasonable screening guide, after which the potential benefits of more detailed in situ and laboratory testing programs can be assessed relative to the site heterogeneity, level of seismic hazard, and other project specific conditions.

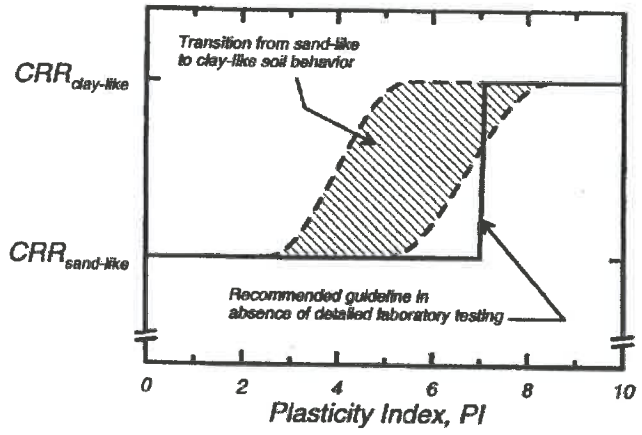

Fig. 15. Schematic illustration of the transition from sand-like to clay-like behavior for fine-grained soils with increasing PI, and the recommended guideline for practice

The use of the Chinese criteria should be discontinued. Similar empirical index-test based criteria that do not adequately consider the differences between sand-like and clay-like behavior may envelope soil conditions where certain types of ground failure have been observed in past earthquakes, but generally do not provide adequate distinctions with regard to the engineering procedures that are appropriate for predicting potential ground failure in future earthquakes.

Procedures for evaluating the potential for cyclic softening of clay-like fine-grained soils are presented in Boulanger and Idriss (2004), along with an evaluation of those procedures against three case histories involving areas of failure and nonfailure in fine-grained soils. The recommended liquefaction susceptibility criteria and the cyclic softening analysis procedures were shown to provide an improved means for distinguishing between the conditions that did and did not lead to ground deformations at these sites. It is hoped that the recommended liquefaction susceptibility criteria and cyclic softening analysis procedures will provide a framework for future developments and refinements in this area, as additional case histories and experimental data are compiled and analyzed.

Acknowledgments

The experimental work on blended silt mixtures performed by Steve Romero was supported by the National Science Foundation under Award No. CMS-95-9502530. Dr. Yoshi Moriwaki provided data on tailing slimes from his personal files. Dr. Lelio Mejia provided cyclic laboratory test data on the CWOC and Chino sites, as well as other testing data from his personal files. Professors C. C. Ladd and D. J. DeGroot provided data on silts and clays from their personal records. The writers are also grateful for the many discussions and valuable comments provided by Professors J. K. Mitchell, J. P. Stewart, and D. J. DeGroot and Drs. L. Mejia, Y. Moriwaki, and F. Makdisi.

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APPENDIX G

SEISMIC SLOPE STABILITY ANALYSES

APPENDIX G

SEISMIC SLOPE STABILITY ANALYSES

Seismic Analysis Procedure

1. Analyze surfaces using a pseudostatic horizontal acceleration of 0.15 g.
2. Perform seismic screening using Rathje and Antonokos (2011). Determine displacement (u) with 50 percent probability of occurrence.
   a. If u $\leq$ 15 cm and potential slip surface does not impact a building, then okay.
   b. If u $\leq$ 5 cm and potential slip surface does impact a building, then okay.

Per special Publication 117- Guidelines for Landslide Hazards, this criteria strictly applies to an earthquake probability of 10 percent probability of exceedance in 50 years, whereas the 2013 CBC now requires evaluation for an earthquake with a 2 percent probability of exceedance in 50 years. The slope deformation criteria for the higher risk probability would be somewhat more than the values provide in 2.a and 2.b.

Analyses

Seismic slope stability analyses were made for Cross Sections A-A shown in Figures 3 and 4. Analyses are made for both an arcuate- and block-shaped surface. The analyses results are provided on Figures G.1 through G.48.

The results indicate an adequate seismic factor of safety of 1.1 or more for a pseudostatic horizontal acceleration of 0.15 g. These results are provided on Figures G.1 through G.11 for the arcuate-shaped surface and on Figures G.30 through G.37 for the block-shaped surface. These results are sufficient to satisfy current code requirements.

Additional seismic slope stability analyses are made for the arcuate and block surface of Cross Section A-A' to estimate the yield acceleration (horizontal acceleration for a factor of safety of 1.0). The analyses results are provided on Figures G.12 through G.25 for the arcuate surface and on Figures G.38 through G.46 for the block surface. The results indicate a yield acceleration from 0.19 to 0.23 in order to achieve a factor of safety of 1.0.

Slope deformations estimates are made for the arcuate and block surface on Figures G.28-29 and G47-48. These deformation estimates are made using the method of Rathje and Antonakos (2011). For the seismicity estimates provided in Appendix D, the slope deformation is estimated for an earthquake probability of exceedance of 2 percent and 10 percent in 50 years. Slope deformation estimates (Figures G.28 and G.47) for the 2 percent risk range from 5.4 to 15.6 cm (2.1 to 6.1 inches). Slope deformation estimates (Figures G.29 and G.48) for the 10 percent risk range from 1.0 to 3.4 cm (0.4 to 1.4 inches). These are within the acceptable range for buildable areas per SP 117 guidelines.

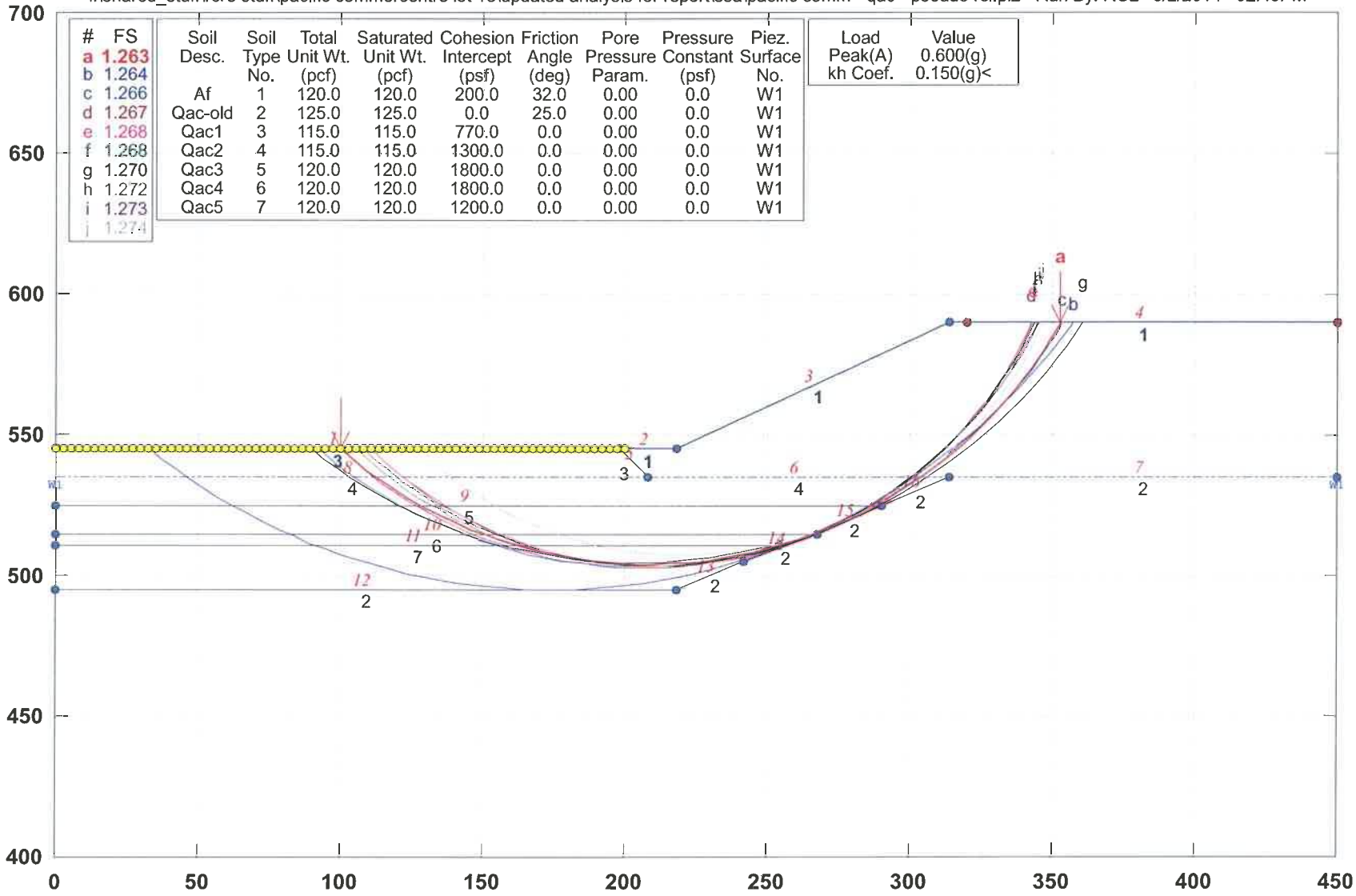
Pacific Commercenter Lot 13 Qac - Seismic - Pseudostatic

I:\shared_stuff\rcl's stuff\pacific commercentre lot 13\updated analysis for report\ssa\pacific comm - qac - pseudo rcl.pl2 Run By: RCL 6/2/2014 02:45PM

#	FS
a	1.263
b	1.264
c	1.266
d	1.267
e	1.268
f	1.268
g	1.270
h	1.272
i	1.273
j	1.274

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Af	1	120.0	120.0	200.0	32.0	0.00	0.0	W1
Qac-old	2	125.0	125.0	0.0	25.0	0.00	0.0	W1
Qac1	3	115.0	115.0	770.0	0.0	0.00	0.0	W1
Qac2	4	115.0	115.0	1300.0	0.0	0.00	0.0	W1
Qac3	5	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac4	6	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac5	7	120.0	120.0	1200.0	0.0	0.00	0.0	W1

Load	Value
Peak(A)	0.600(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.263

Safety Factors Are Calculated By The Modified Bishop Method

G.1

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

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****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 6/2/2014
Time of Run: 02:45PM
Run By: RCL
Input Data Filename: L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDATED analysis for report\SSA\pacific comm - qac - pseudo rcl.in
Output Filename: L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDATED analysis for report\SSA\pacific comm - qac - pseudo rcl.OUT
Unit System: English
Plotted Output Filename: L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDATED analysis for report\SSA\pacific comm - qac - pseudo rcl.PLT
PROBLEM DESCRIPTION: Pacific Commercenter Lot 13
Qac - Seismic - Pseudostatic

BOUNDARY COORDINATES
4 Top Boundaries
16 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	545.00	198.00	545.00	3
2	198.00	545.00	218.00	545.00	1
3	218.00	545.00	314.00	590.00	1
4	314.00	590.00	450.00	590.00	1
5	198.00	545.00	208.00	535.00	3
6	208.00	535.00	314.00	535.00	4
7	314.00	535.00	450.00	535.00	2
8	0.00	535.00	208.00	535.00	4
9	0.00	525.00	290.00	525.00	5
10	0.00	515.00	268.00	515.00	6
11	0.00	511.00	254.00	511.00	7
12	0.00	495.00	218.00	495.00	2
13	218.00	495.00	242.00	505.00	2
14	242.00	505.00	268.00	515.00	2
15	268.00	515.00	290.00	525.00	2
16	290.00	525.00	314.00	535.00	2

User Specified Y-Origin = 400.00(ft)
User Specified X-Plus Value = 50.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	200.0	32.0	0.00	0.0	1
2	125.0	125.0	0.0	25.0	0.00	0.0	1
3	115.0	115.0	770.0	0.0	0.00	0.0	1
4	115.0	115.0	1300.0	0.0	0.00	0.0	1
5	120.0	120.0	1800.0	0.0	0.00	0.0	1
6	120.0	120.0	1800.0	0.0	0.00	0.0	1
7	120.0	120.0	1200.0	0.0	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	535.00

G.2

```
    2        450.00      535.00
Specified Peak Ground Acceleration Coefficient (A) =   0.600(g)
Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
Specified Seismic Pore-Pressure Factor =   0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
7500 Trial Surfaces Have Been Generated.
 100 Surface(s) Initiate(s) From Each Of    75 Points Equally Spaced
Along The Ground Surface Between  X =   0.00(ft)
                             and  X = 200.00(ft)
Each Surface Terminates Between   X = 320.00(ft)
                             and  X = 450.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =      0.00(ft)
 6.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By The Modified Bishop Method * *
      Total Number of Trial Surfaces Attempted =     0
      Number of Trial Surfaces With Valid FS =    0
      Statistical Data On All Valid FS Values:
         FS Max =   0.000   FS Min = 500.000   FS Ave =  NaN
         Standard Deviation =    0.000   Coefficient of Variation =  NaN     %
      Failure Surface Specified By 51 Coordinate Points
        Point      X-Surf      Y-Surf
         No.        (ft)        (ft)
          1       100.000     545.000
          2       104.585     541.129
          3       109.307     537.429
          4       114.162     533.903
          5       119.142     530.556
          6       124.241     527.394
          7       129.452     524.420
          8       134.768     521.638
          9       140.182     519.051
         10       145.687     516.665
         11       151.275     514.480
         12       156.939     512.501
         13       162.672     510.730
         14       168.466     509.170
         15       174.312     507.822
         16       180.204     506.688
         17       186.133     505.770
         18       192.092     505.069
         19       198.073     504.586
         20       204.067     504.322
         21       210.067     504.276
         22       216.064     504.450
         23       222.052     504.842
         24       228.020     505.453
         25       233.963     506.281
         26       239.871     507.325
         27       245.738     508.584
         28       251.554     510.057
         29       257.313     511.741
         30       263.007     513.634
         31       268.627     515.733
         32       274.168     518.036
         33       279.620     520.540
         34       284.978     523.241
         35       290.233     526.136
         36       295.380     529.221
         37       300.410     532.491
         38       305.317     535.943
         39       310.096     539.572
```


G.3

40	314.738	543.373
41	319.239	547.341
42	323.592	551.470
43	327.792	555.755
44	331.832	560.191
45	335.708	564.771
46	339.414	569.490
47	342.945	574.341
48	346.297	579.317
49	349.465	584.413
50	352.445	589.620
51	352.644	590.000

Circle Center At X = 208.312 ; Y = 668.638 ; and Radius = 164.371

Factor of Safety

*** 1.263 ***

Individual data on the 0 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)

Failure Surface Specified By 63 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	32.432	545.000
2	37.151	541.293
3	41.966	537.714
4	46.875	534.265
5	51.875	530.948
6	56.961	527.765
7	62.131	524.719
8	67.380	521.813
9	72.705	519.047
10	78.101	516.425
11	83.566	513.947
12	89.095	511.617
13	94.684	509.434
14	100.329	507.402
15	106.027	505.521
16	111.772	503.793
17	117.562	502.219
18	123.392	500.799
19	129.257	499.536
20	135.155	498.430
21	141.079	497.482
22	147.027	496.692
23	152.994	496.061
24	158.975	495.590
25	164.967	495.279
26	170.965	495.127
27	176.965	495.136
28	182.963	495.305
29	188.954	495.634
30	194.934	496.122
31	200.899	496.771
32	206.844	497.578
33	212.766	498.543
34	218.660	499.666
35	224.522	500.946
36	230.347	502.382
37	236.133	503.973
38	241.873	505.718
39	247.565	507.616
40	253.205	509.665
41	258.787	511.863
42	264.309	514.210
43	269.767	516.703
44	275.156	519.341

45	280.472	522.122
46	285.713	525.044
47	290.874	528.105
48	295.951	531.302
49	300.941	534.634
50	305.840	538.097
51	310.645	541.691
52	315.352	545.411
53	319.959	549.255
54	324.461	553.222
55	328.856	557.306
56	333.140	561.507
57	337.311	565.821
58	341.365	570.244
59	345.299	574.774
60	349.111	579.407
61	352.798	584.141
62	356.358	588.971
63	357.074	590.000

Circle Center At X = 173.639 ; Y = 719.893 ; and Radius = 224.782

Factor of Safety

*** 1.264 ***

Failure Surface Specified By 52 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.892	545.000
2	96.508	541.167
3	101.255	537.498
4	106.127	533.996
5	111.119	530.666
6	116.223	527.513
7	121.434	524.539
8	126.746	521.748
9	132.151	519.145
10	137.645	516.731
11	143.219	514.511
12	148.867	512.486
13	154.582	510.659
14	160.357	509.033
15	166.186	507.608
16	172.060	506.388
17	177.974	505.374
18	183.919	504.566
19	189.889	503.966
20	195.876	503.575
21	201.874	503.392
22	207.874	503.419
23	213.869	503.655
24	219.853	504.099
25	225.817	504.752
26	231.755	505.613
27	237.659	506.680
28	243.523	507.952
29	249.338	509.428
30	255.099	511.106
31	260.798	512.983
32	266.428	515.058
33	271.982	517.328
34	277.453	519.790
35	282.835	522.442
36	288.122	525.279
37	293.307	528.299
38	298.383	531.498
39	303.344	534.872
40	308.185	538.417
41	312.899	542.129
42	317.481	546.002

43	321.926	550.033
44	326.227	554.217
45	330.379	558.548
46	334.378	563.021
47	338.219	567.630
48	341.896	572.371
49	345.407	577.237
50	348.745	582.223
51	351.908	587.322
52	353.442	590.000

Circle Center At X = 204.098 ; Y = 675.424 ; and Radius = 172.049

Factor of Safety

*** 1.266 ***

Failure Surface Specified By 48 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	110.811	545.000
2	115.153	540.860
3	119.666	536.906
4	124.341	533.145
5	129.170	529.584
6	134.145	526.229
7	139.256	523.086
8	144.495	520.161
9	149.852	517.459
10	155.318	514.985
11	160.883	512.742
12	166.538	510.736
13	172.272	508.969
14	178.075	507.445
15	183.937	506.166
16	189.848	505.135
17	195.796	504.353
18	201.773	503.822
19	207.766	503.543
20	213.766	503.516
21	219.762	503.741
22	225.743	504.218
23	231.699	504.947
24	237.618	505.925
25	243.492	507.151
26	249.308	508.623
27	255.058	510.338
28	260.730	512.294
29	266.316	514.486
30	271.804	516.911
31	277.185	519.565
32	282.450	522.443
33	287.589	525.539
34	292.593	528.849
35	297.454	532.367
36	302.163	536.085
37	306.711	539.999
38	311.091	544.100
39	315.294	548.381
40	319.313	552.836
41	323.142	557.456
42	326.773	562.232
43	330.200	567.157
44	333.417	572.222
45	336.418	577.418
46	339.198	582.735
47	341.752	588.164
48	342.523	590.000

Circle Center At X = 211.397 ; Y = 646.127 ; and Radius = 142.633

Factor of Safety

*** 1.267 ***

Failure Surface Specified By 48 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	110.811	545.000
2	115.172	540.879
3	119.701	536.944
4	124.392	533.202
5	129.234	529.660
6	134.221	526.323
7	139.342	523.197
8	144.590	520.289
9	149.955	517.602
10	155.428	515.142
11	160.998	512.913
12	166.657	510.919
13	172.395	509.164
14	178.201	507.650
15	184.065	506.380
16	189.977	505.357
17	195.926	504.581
18	201.903	504.055
19	207.897	503.780
20	213.897	503.755
21	219.893	503.982
22	225.874	504.459
23	231.829	505.185
24	237.750	506.161
25	243.624	507.382
26	249.442	508.849
27	255.194	510.557
28	260.869	512.505
29	266.457	514.688
30	271.950	517.103
31	277.337	519.746
32	282.608	522.611
33	287.755	525.695
34	292.769	528.991
35	297.640	532.494
36	302.361	536.197
37	306.922	540.095
38	311.317	544.180
39	315.537	548.445
40	319.575	552.883
41	323.423	557.486
42	327.076	562.246
43	330.527	567.155
44	333.769	572.203
45	336.797	577.383
46	339.606	582.685
47	342.191	588.099
48	343.002	590.000

Circle Center At X = 211.473 ; Y = 647.136 ; and Radius = 143.404

Factor of Safety

*** 1.268 ***

Failure Surface Specified By 48 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	108.108	545.000
2	112.489	540.900
3	117.035	536.984
4	121.737	533.257
5	126.588	529.727
6	131.581	526.399
7	136.705	523.278
8	141.954	520.370
9	147.317	517.680
10	152.786	515.213

11	158.352	512.971
12	164.005	510.961
13	169.735	509.184
14	175.534	507.643
15	181.391	506.342
16	187.297	505.282
17	193.241	504.466
18	199.214	503.894
19	205.205	503.567
20	211.205	503.487
21	217.202	503.653
22	223.188	504.065
23	229.152	504.723
24	235.084	505.624
25	240.974	506.769
26	246.812	508.154
27	252.588	509.777
28	258.293	511.636
29	263.916	513.728
30	269.449	516.048
31	274.882	518.594
32	280.207	521.361
33	285.413	524.343
34	290.492	527.537
35	295.436	530.937
36	300.236	534.536
37	304.885	538.330
38	309.374	542.311
39	313.696	546.473
40	317.843	550.809
41	321.809	555.311
42	325.587	559.972
43	329.170	564.785
44	332.553	569.740
45	335.730	574.830
46	338.695	580.046
47	341.443	585.380
48	343.589	590.000

Circle Center At X = 210.151 ; Y = 649.643 ; and Radius = 146.161

Factor of Safety

*** 1.268 ***

Failure Surface Specified By 53 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	89.189	545.000
2	93.936	541.330
3	98.799	537.816
4	103.774	534.462
5	108.855	531.271
6	114.037	528.246
7	119.315	525.392
8	124.682	522.710
9	130.133	520.203
10	135.663	517.875
11	141.266	515.728
12	146.935	513.763
13	152.665	511.984
14	158.450	510.391
15	164.283	508.987
16	170.159	507.773
17	176.071	506.750
18	182.014	505.920
19	187.980	505.283
20	193.963	504.840
21	199.958	504.591
22	205.958	504.537
23	211.956	504.678

24	217.947	505.013
25	223.923	505.543
26	229.880	506.266
27	235.809	507.182
28	241.706	508.290
29	247.564	509.589
30	253.376	511.077
31	259.137	512.753
32	264.841	514.615
33	270.481	516.662
34	276.052	518.890
35	281.548	521.298
36	286.963	523.883
37	292.291	526.642
38	297.526	529.573
39	302.664	532.672
40	307.698	535.936
41	312.624	539.362
42	317.436	542.946
43	322.129	546.684
44	326.699	550.572
45	331.140	554.607
46	335.447	558.784
47	339.617	563.098
48	343.645	567.545
49	347.526	572.121
50	351.256	576.820
51	354.833	581.638
52	358.250	586.569
53	360.467	590.000

Circle Center At X = 204.622 ; Y = 689.400 ; and Radius = 184.867

Factor of Safety

*** 1.270 ***

Failure Surface Specified By 49 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	105.405	545.000
2	109.831	540.948
3	114.415	537.077
4	119.150	533.392
5	124.028	529.899
6	129.043	526.605
7	134.185	523.513
8	139.447	520.630
9	144.820	517.960
10	150.296	515.507
11	155.865	513.275
12	161.519	511.267
13	167.249	509.487
14	173.046	507.938
15	178.900	506.622
16	184.802	505.542
17	190.742	504.698
18	196.711	504.093
19	202.700	503.727
20	208.699	503.600
21	214.698	503.714
22	220.687	504.068
23	226.658	504.661
24	232.600	505.493
25	238.504	506.562
26	244.361	507.866
27	250.160	509.403
28	255.894	511.171
29	261.552	513.167
30	267.126	515.388
31	272.607	517.830

32	277.985	520.489
33	283.253	523.362
34	288.401	526.443
35	293.423	529.727
36	298.309	533.210
37	303.051	536.885
38	307.643	540.747
39	312.076	544.790
40	316.345	549.007
41	320.441	553.391
42	324.358	557.936
43	328.091	562.634
44	331.632	567.477
45	334.977	572.458
46	338.120	577.569
47	341.056	582.802
48	343.780	588.147
49	344.633	590.000

Circle Center At X = 208.851 ; Y = 653.541 ; and Radius = 149.940

Factor of Safety

*** 1.272 ***

Failure Surface Specified By 50 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.000	545.000
2	104.450	540.975
3	109.052	537.125
4	113.800	533.457
5	118.686	529.975
6	123.704	526.685
7	128.845	523.592
8	134.102	520.700
9	139.467	518.014
10	144.932	515.538
11	150.490	513.275
12	156.130	511.230
13	161.846	509.404
14	167.628	507.802
15	173.467	506.424
16	179.356	505.273
17	185.285	504.352
18	191.245	503.661
19	197.227	503.201
20	203.223	502.974
21	209.223	502.978
22	215.218	503.215
23	221.200	503.685
24	227.159	504.385
25	233.086	505.316
26	238.973	506.476
27	244.811	507.863
28	250.590	509.474
29	256.303	511.309
30	261.940	513.364
31	267.493	515.635
32	272.955	518.120
33	278.316	520.814
34	283.568	523.714
35	288.705	526.815
36	293.717	530.113
37	298.598	533.603
38	303.340	537.279
39	307.936	541.136
40	312.379	545.168
41	316.663	549.369
42	320.781	553.733
43	324.726	558.253

44	328.494	562.923
45	332.078	567.735
46	335.473	572.682
47	338.673	577.757
48	341.675	582.952
49	344.473	588.260
50	345.306	590.000

Circle Center At X = 206.091 ; Y = 657.786 ; and Radius = 154.841

Factor of Safety

*** 1.273 ***

Failure Surface Specified By 45 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	127.027	545.000
2	131.462	540.959
3	136.070	537.116
4	140.843	533.480
5	145.771	530.057
6	150.845	526.855
7	156.054	523.878
8	161.390	521.134
9	166.841	518.627
10	172.397	516.362
11	178.047	514.343
12	183.781	512.575
13	189.586	511.061
14	195.453	509.803
15	201.370	508.805
16	207.324	508.068
17	213.305	507.593
18	219.302	507.382
19	225.301	507.435
20	231.293	507.751
21	237.265	508.331
22	243.206	509.173
23	249.103	510.275
24	254.947	511.635
25	260.725	513.252
26	266.427	515.120
27	272.041	517.238
28	277.556	519.600
29	282.962	522.203
30	288.249	525.041
31	293.405	528.109
32	298.422	531.400
33	303.289	534.909
34	307.997	538.628
35	312.537	542.551
36	316.900	546.670
37	321.078	550.977
38	325.062	555.463
39	328.845	560.120
40	332.420	564.939
41	335.779	569.910
42	338.916	575.024
43	341.826	580.272
44	344.502	585.642
45	346.439	590.000

Circle Center At X = 221.102 ; Y = 643.779 ; and Radius = 136.409

Factor of Safety

*** 1.274 ***

**** END OF GSTABL7 OUTPUT ****

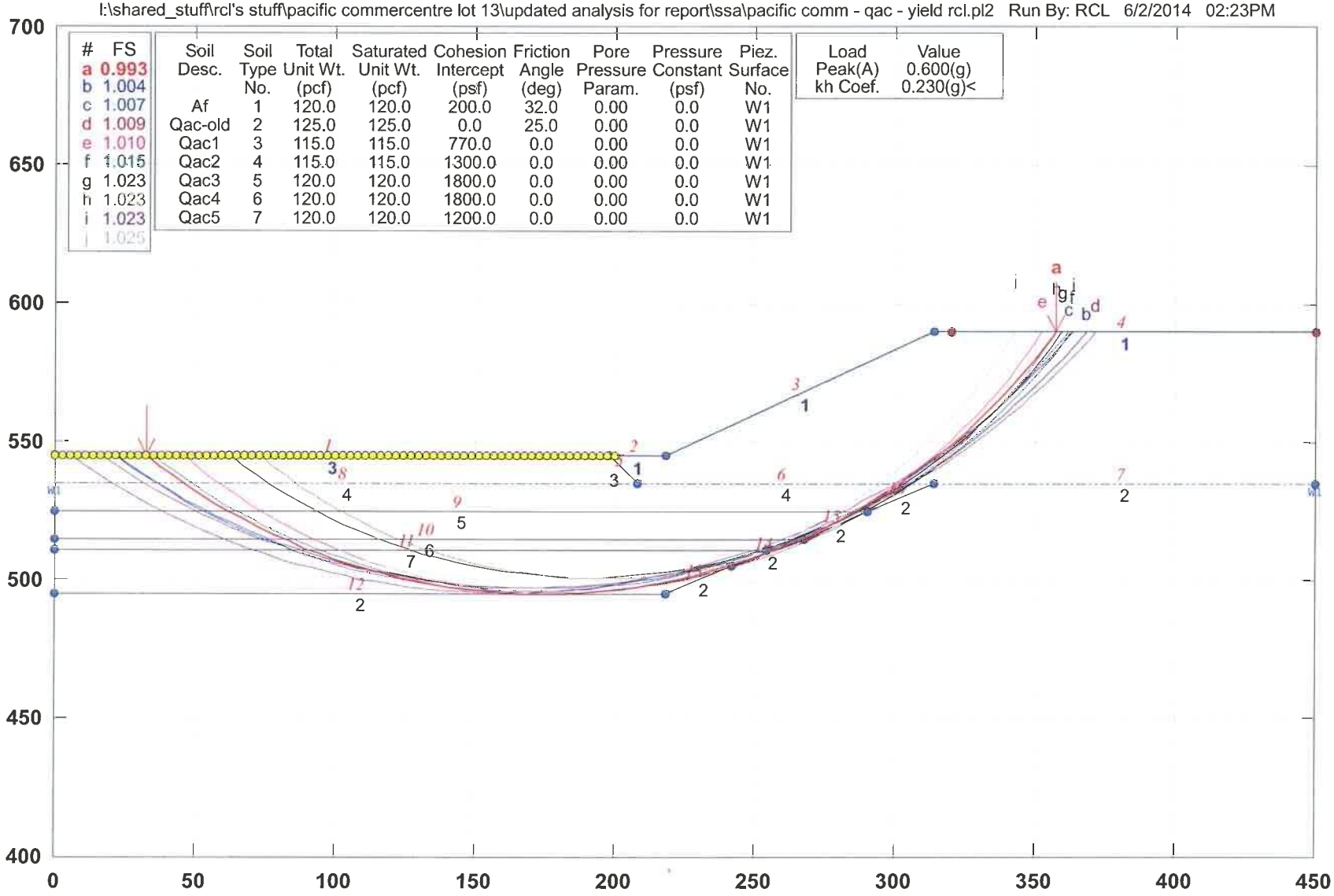
Pacific Commercenter Lot 13 Qac - Seismic -Yield Accel

I:\shared_stuff\rcl's stuff\pacific commercentre lot 13\updated analysis for report\ssa\pacific comm - qac - yield rcl.pl2 Run By: RCL 6/2/2014 02:23PM

#	FS
a	0.993
b	1.004
c	1.007
d	1.009
e	1.010
f	1.015
g	1.023
h	1.023
i	1.023
j	1.025

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Af	1	120.0	120.0	200.0	32.0	0.00	0.0	W1
Qac-old	2	125.0	125.0	0.0	25.0	0.00	0.0	W1
Qac1	3	115.0	115.0	770.0	0.0	0.00	0.0	W1
Qac2	4	115.0	115.0	1300.0	0.0	0.00	0.0	W1
Qac3	5	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac4	6	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac5	7	120.0	120.0	1200.0	0.0	0.00	0.0	W1

Load	Value
Peak(A)	0.600(g)
kh Coef.	0.230(g)<



GSTABL7 v.2 FSmin=0.993

Safety Factors Are Calculated By The Modified Bishop Method

G.12

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                                  *** GSTABL7 ***
                        ** GSTABL7 by Garry H. Gregory, P.E. **
          ** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **
                     (All Rights Reserved-Unauthorized Use Prohibited)
    ************************************************************************
                          SLOPE STABILITY ANALYSIS SYSTEM
              Modified Bishop, Simplified Janbu, or GLE Method of Slices.
              (Includes Spencer & Morgenstern-Price Type Analysis)
              Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
              Nonlinear Undrained Shear Strength, Curved Phi Envelope,
              Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
              Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
    ************************************************************************
    Analysis Run Date:        6/2/2014
    Time of Run:              02:23PM
    Run By:                   RCL
    Input Data Filename:      L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDA
TED analysis for report\SSA\pacific comm - qac - yield rcl.in
    Output Filename:          L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDA
TED analysis for report\SSA\pacific comm - qac - yield rcl.OUT
    Unit System:              English
    Plotted Output Filename:  L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDA
TED analysis for report\SSA\pacific comm - qac - yield rcl.PLT
    PROBLEM DESCRIPTION:  Pacific Commercenter Lot 13
                          Qac - Seismic -Yield Accel
    BOUNDARY COORDINATES
```


G.13

4 Top Boundaries
16 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	545.00	198.00	545.00	3
2	198.00	545.00	218.00	545.00	1
3	218.00	545.00	314.00	590.00	1
4	314.00	590.00	450.00	590.00	1
5	198.00	545.00	208.00	535.00	3
6	208.00	535.00	314.00	535.00	4
7	314.00	535.00	450.00	535.00	2
8	0.00	535.00	208.00	535.00	4
9	0.00	525.00	290.00	525.00	5
10	0.00	515.00	268.00	515.00	6
11	0.00	511.00	254.00	511.00	7
12	0.00	495.00	218.00	495.00	2
13	218.00	495.00	242.00	505.00	2
14	242.00	505.00	268.00	515.00	2
15	268.00	515.00	290.00	525.00	2
16	290.00	525.00	314.00	535.00	2

User Specified Y-Origin = 400.00(ft)
User Specified X-Plus Value = 50.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	200.0	32.0	0.00	0.0	1
2	125.0	125.0	0.0	25.0	0.00	0.0	1
3	115.0	115.0	770.0	0.0	0.00	0.0	1
4	115.0	115.0	1300.0	0.0	0.00	0.0	1
5	120.0	120.0	1800.0	0.0	0.00	0.0	1
6	120.0	120.0	1800.0	0.0	0.00	0.0	1
7	120.0	120.0	1200.0	0.0	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	535.00
2	450.00	535.00

Specified Peak Ground Acceleration Coefficient (A) = 0.600(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.230(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
7500 Trial Surfaces Have Been Generated.
100 Surface(s) Initiate(s) From Each Of 75 Points Equally Spaced
Along The Ground Surface Between X = 0.00(ft)
and X = 200.00(ft)
Each Surface Terminates Between X = 320.00(ft)
and X = 450.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
6.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Attempted = 0
Number of Trial Surfaces With Valid FS = 0
Statistical Data On All Valid FS Values:
FS Max = 0.000 FS Min = 500.000 FS Ave = NaN
Standard Deviation = 0.000 Coefficient of Variation = NaN

Failure Surface Specified By 63 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	32.432	545.000
2	37.151	541.293
3	41.966	537.714
4	46.875	534.265
5	51.875	530.948
6	56.961	527.765
7	62.131	524.719
8	67.380	521.813
9	72.705	519.047
10	78.101	516.425
11	83.566	513.947
12	89.095	511.617
13	94.684	509.434
14	100.329	507.402
15	106.027	505.521
16	111.772	503.793
17	117.562	502.219
18	123.392	500.799
19	129.257	499.536
20	135.155	498.430
21	141.079	497.482
22	147.027	496.692
23	152.994	496.061
24	158.975	495.590
25	164.967	495.279
26	170.965	495.127
27	176.965	495.136
28	182.963	495.305
29	188.954	495.634
30	194.934	496.122
31	200.899	496.771
32	206.844	497.578
33	212.766	498.543
34	218.660	499.666
35	224.522	500.946
36	230.347	502.382
37	236.133	503.973
38	241.873	505.718
39	247.565	507.616
40	253.205	509.665
41	258.787	511.863
42	264.309	514.210
43	269.767	516.703
44	275.156	519.341
45	280.472	522.122
46	285.713	525.044
47	290.874	528.105
48	295.951	531.302
49	300.941	534.634
50	305.840	538.097
51	310.645	541.691
52	315.352	545.411
53	319.959	549.255
54	324.461	553.222
55	328.856	557.306
56	333.140	561.507
57	337.311	565.821
58	341.365	570.244
59	345.299	574.774
60	349.111	579.407
61	352.798	584.141
62	356.358	588.971
63	357.074	590.000

Circle Center At X = 173.639 ; Y = 719.893 ; and Radius = 224.782

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Factor of Safety
*** 0.993 ***
Individual data on the 0 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)

Failure Surface Specified By 66 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	21.622	545.000
2	26.536	541.558
3	31.531	538.233
4	36.601	535.026
5	41.746	531.938
6	46.962	528.973
7	52.246	526.130
8	57.595	523.412
9	63.007	520.821
10	68.477	518.357
11	74.005	516.022
12	79.585	513.818
13	85.216	511.745
14	90.893	509.805
15	96.615	507.999
16	102.377	506.328
17	108.178	504.792
18	114.012	503.393
19	119.878	502.131
20	125.772	501.007
21	131.690	500.022
22	137.630	499.176
23	143.589	498.469
24	149.562	497.903
25	155.547	497.477
26	161.540	497.191
27	167.538	497.047
28	173.538	497.043
29	179.537	497.180
30	185.530	497.457
31	191.516	497.876
32	197.490	498.434
33	203.449	499.133
34	209.390	499.971
35	215.310	500.949
36	221.205	502.065
37	227.072	503.319
38	232.909	504.711
39	238.711	506.239
40	244.476	507.903
41	250.200	509.701
42	255.880	511.634
43	261.513	513.699
44	267.096	515.896
45	272.627	518.224
46	278.101	520.681
47	283.516	523.265
48	288.868	525.976
49	294.156	528.811
50	299.376	531.770
51	304.524	534.851
52	309.599	538.052
53	314.598	541.370
54	319.517	544.806
55	324.354	548.355
56	329.107	552.018
57	333.772	555.790

58	338.348	559.672
59	342.831	563.659
60	347.219	567.751
61	351.511	571.945
62	355.702	576.238
63	359.792	580.628
64	363.777	585.113
65	367.656	589.691
66	367.906	590.000

Circle Center At X = 170.703 ; Y = 752.655 ; and Radius = 255.628

Factor of Safety

*** 1.004 ***

Failure Surface Specified By 65 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	21.622	545.000
2	26.466	541.460
3	31.395	538.039
4	36.407	534.740
5	41.498	531.565
6	46.665	528.515
7	51.905	525.593
8	57.215	522.800
9	62.592	520.137
10	68.033	517.607
11	73.533	515.210
12	79.091	512.949
13	84.702	510.825
14	90.364	508.838
15	96.072	506.991
16	101.824	505.284
17	107.616	503.718
18	113.445	502.294
19	119.307	501.013
20	125.198	499.876
21	131.115	498.884
22	137.055	498.036
23	143.014	497.334
24	148.988	496.778
25	154.974	496.369
26	160.968	496.106
27	166.967	495.990
28	172.967	496.020
29	178.965	496.198
30	184.956	496.522
31	190.937	496.992
32	196.906	497.609
33	202.857	498.372
34	208.788	499.280
35	214.695	500.333
36	220.574	501.530
37	226.423	502.870
38	232.236	504.353
39	238.012	505.978
40	243.746	507.744
41	249.436	509.650
42	255.077	511.694
43	260.666	513.875
44	266.200	516.193
45	271.676	518.645
46	277.091	521.231
47	282.440	523.948
48	287.721	526.796
49	292.931	529.772
50	298.067	532.874
51	303.125	536.101
52	308.103	539.451

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53	312.998	542.921
54	317.806	546.511
55	322.524	550.217
56	327.151	554.037
57	331.683	557.969
58	336.117	562.011
59	340.451	566.160
60	344.682	570.414
61	348.808	574.771
62	352.826	579.227
63	356.734	583.780
64	360.529	588.427
65	361.751	590.000

Circle Center At X = 168.717 ; Y = 741.203 ; and Radius = 245.219

Factor of Safety

*** 1.007 ***

Failure Surface Specified By 67 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	16.216	545.000
2	21.187	541.640
3	26.232	538.393
4	31.349	535.259
5	36.534	532.240
6	41.786	529.339
7	47.101	526.555
8	52.478	523.892
9	57.912	521.349
10	63.403	518.929
11	68.946	516.632
12	74.539	514.460
13	80.179	512.414
14	85.864	510.494
15	91.590	508.702
16	97.355	507.039
17	103.155	505.506
18	108.989	504.102
19	114.852	502.830
20	120.743	501.689
21	126.658	500.681
22	132.593	499.805
23	138.547	499.062
24	144.516	498.453
25	150.497	497.977
26	156.488	497.636
27	162.484	497.428
28	168.484	497.355
29	174.483	497.417
30	180.480	497.613
31	186.471	497.943
32	192.453	498.407
33	198.423	499.004
34	204.378	499.736
35	210.316	500.600
36	216.232	501.597
37	222.125	502.727
38	227.991	503.988
39	233.827	505.380
40	239.631	506.902
41	245.399	508.554
42	251.129	510.335
43	256.817	512.244
44	262.461	514.279
45	268.059	516.440
46	273.606	518.726
47	279.101	521.136
48	284.540	523.668

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49	289.922	526.322
50	295.242	529.095
51	300.500	531.986
52	305.691	534.995
53	310.813	538.119
54	315.865	541.357
55	320.842	544.707
56	325.744	548.168
57	330.566	551.738
58	335.308	555.414
59	339.966	559.196
60	344.538	563.082
61	349.022	567.068
62	353.415	571.155
63	357.716	575.338
64	361.922	579.617
65	366.032	583.989
66	370.042	588.451
67	371.372	590.000

Circle Center At X = 168.741 ; Y = 765.317 ; and Radius = 267.962

Factor of Safety

*** 1.009 ***

Failure Surface Specified By 60 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	45.946	545.000
2	50.549	541.152
3	55.264	537.440
4	60.085	533.868
5	65.008	530.439
6	70.030	527.156
7	75.145	524.020
8	80.351	521.036
9	85.641	518.206
10	91.012	515.532
11	96.459	513.016
12	101.978	510.660
13	107.563	508.468
14	113.210	506.440
15	118.913	504.578
16	124.669	502.883
17	130.472	501.358
18	136.317	500.004
19	142.200	498.821
20	148.114	497.811
21	154.055	496.975
22	160.019	496.313
23	165.999	495.826
24	171.991	495.514
25	177.989	495.378
26	183.989	495.418
27	189.985	495.633
28	195.973	496.023
29	201.946	496.589
30	207.900	497.330
31	213.830	498.244
32	219.730	499.332
33	225.597	500.592
34	231.423	502.023
35	237.206	503.624
36	242.939	505.394
37	248.618	507.331
38	254.237	509.433
39	259.793	511.699
40	265.280	514.127
41	270.693	516.714
42	276.029	519.459

43	281.281	522.359
44	286.447	525.411
45	291.521	528.613
46	296.499	531.963
47	301.377	535.457
48	306.150	539.092
49	310.815	542.865
50	315.368	546.773
51	319.804	550.813
52	324.120	554.981
53	328.312	559.274
54	332.376	563.688
55	336.310	568.218
56	340.109	572.862
57	343.771	577.615
58	347.292	582.474
59	350.669	587.433
60	352.309	590.000

Circle Center At X = 179.631 ; Y = 700.233 ; and Radius = 204.863

Factor of Safety

*** 1.010 ***

Failure Surface Specified By 63 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	35.135	545.000
2	39.882	541.330
3	44.723	537.786
4	49.656	534.370
5	54.677	531.085
6	59.782	527.932
7	64.968	524.915
8	70.231	522.034
9	75.568	519.293
10	80.976	516.693
11	86.449	514.235
12	91.985	511.922
13	97.580	509.755
14	103.230	507.735
15	108.931	505.865
16	114.679	504.144
17	120.470	502.575
18	126.301	501.158
19	132.166	499.895
20	138.063	498.786
21	143.987	497.832
22	149.933	497.034
23	155.899	496.392
24	161.879	495.907
25	167.870	495.579
26	173.868	495.408
27	179.868	495.395
28	185.866	495.539
29	191.858	495.840
30	197.841	496.299
31	203.809	496.914
32	209.759	497.685
33	215.687	498.613
34	221.589	499.695
35	227.460	500.932
36	233.297	502.323
37	239.095	503.866
38	244.850	505.561
39	250.560	507.406
40	256.219	509.401
41	261.823	511.543
42	267.370	513.831
43	272.854	516.264

44	278.273	518.840
45	283.622	521.558
46	288.898	524.415
47	294.098	527.409
48	299.217	530.538
49	304.252	533.801
50	309.200	537.195
51	314.057	540.718
52	318.821	544.366
53	323.486	548.139
54	328.051	552.032
55	332.513	556.044
56	336.868	560.171
57	341.113	564.412
58	345.245	568.762
59	349.262	573.219
60	353.160	577.780
61	356.937	582.442
62	360.591	587.201
63	362.626	590.000

Circle Center At X = 177.379 ; Y = 724.078 ; and Radius = 228.697

Factor of Safety

*** 1.015 ***

Failure Surface Specified By 58 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	62.162	545.000
2	66.931	541.359
3	71.804	537.858
4	76.776	534.500
5	81.845	531.289
6	87.004	528.226
7	92.251	525.315
8	97.580	522.558
9	102.987	519.957
10	108.467	517.515
11	114.016	515.233
12	119.629	513.113
13	125.302	511.158
14	131.029	509.369
15	136.805	507.747
16	142.627	506.294
17	148.488	505.010
18	154.384	503.898
19	160.310	502.959
20	166.261	502.191
21	172.231	501.598
22	178.216	501.178
23	184.211	500.933
24	190.211	500.863
25	196.210	500.967
26	202.204	501.245
27	208.187	501.698
28	214.154	502.325
29	220.100	503.125
30	226.021	504.098
31	231.910	505.243
32	237.764	506.559
33	243.578	508.044
34	249.345	509.698
35	255.062	511.519
36	260.724	513.506
37	266.325	515.657
38	271.861	517.970
39	277.328	520.443
40	282.720	523.074
41	288.034	525.861

42	293.264	528.801
43	298.406	531.892
44	303.456	535.132
45	308.410	538.517
46	313.263	542.045
47	318.012	545.713
48	322.652	549.517
49	327.179	553.455
50	331.589	557.523
51	335.880	561.717
52	340.046	566.035
53	344.085	570.472
54	347.993	575.024
55	351.767	579.689
56	355.404	584.461
57	358.900	589.337
58	359.347	590.000

Circle Center At X = 189.632 ; Y = 706.993 ; and Radius = 206.132

Factor of Safety

*** 1.023 ***

Failure Surface Specified By 56 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	72.973	545.000
2	77.636	541.224
3	82.414	537.596
4	87.305	534.119
5	92.302	530.798
6	97.400	527.635
7	102.596	524.634
8	107.883	521.798
9	113.257	519.129
10	118.712	516.630
11	124.242	514.303
12	129.843	512.151
13	135.508	510.176
14	141.233	508.380
15	147.012	506.764
16	152.838	505.330
17	158.706	504.080
18	164.611	503.015
19	170.546	502.136
20	176.506	501.443
21	182.485	500.938
22	188.476	500.621
23	194.475	500.491
24	200.475	500.551
25	206.469	500.798
26	212.454	501.234
27	218.421	501.857
28	224.366	502.667
29	230.283	503.664
30	236.165	504.845
31	242.008	506.211
32	247.805	507.759
33	253.550	509.489
34	259.238	511.398
35	264.864	513.485
36	270.421	515.747
37	275.904	518.182
38	281.309	520.789
39	286.629	523.563
40	291.859	526.504
41	296.994	529.607
42	302.029	532.870
43	306.960	536.289
44	311.780	539.861

45	316.487	543.583
46	321.074	547.450
47	325.537	551.460
48	329.872	555.608
49	334.076	559.890
50	338.142	564.302
51	342.068	568.839
52	345.850	573.497
53	349.484	578.271
54	352.966	583.158
55	356.293	588.151
56	357.443	590.000

Circle Center At X = 195.578 ; Y = 691.591 ; and Radius = 191.104

Factor of Safety

*** 1.023 ***

Failure Surface Specified By 68 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	5.405	545.000
2	10.328	541.570
3	15.327	538.252
4	20.400	535.048
5	25.544	531.958
6	30.756	528.986
7	36.033	526.132
8	41.374	523.398
9	46.775	520.784
10	52.234	518.294
11	57.747	515.927
12	63.312	513.684
13	68.927	511.568
14	74.588	509.579
15	80.292	507.719
16	86.037	505.987
17	91.819	504.386
18	97.636	502.915
19	103.484	501.575
20	109.362	500.368
21	115.265	499.294
22	121.191	498.353
23	127.136	497.547
24	133.098	496.874
25	139.074	496.336
26	145.061	495.933
27	151.055	495.665
28	157.053	495.532
29	163.053	495.535
30	169.051	495.673
31	175.045	495.946
32	181.031	496.354
33	187.007	496.897
34	192.968	497.575
35	198.913	498.387
36	204.838	499.333
37	210.740	500.412
38	216.617	501.624
39	222.464	502.968
40	228.280	504.444
41	234.061	506.051
42	239.804	507.787
43	245.506	509.653
44	251.165	511.646
45	256.778	513.767
46	262.342	516.014
47	267.853	518.386
48	273.309	520.881
49	278.708	523.499

G.23

50	284.046	526.238
51	289.322	529.097
52	294.531	532.074
53	299.672	535.167
54	304.742	538.376
55	309.738	541.698
56	314.658	545.133
57	319.500	548.677
58	324.260	552.329
59	328.936	556.088
60	333.527	559.951
61	338.029	563.917
62	342.441	567.984
63	346.760	572.149
64	350.984	576.410
65	355.111	580.766
66	359.138	585.213
67	363.064	589.750
68	363.271	590.000

Circle Center At X = 159.937 ; Y = 761.551 ; and Radius = 266.034

Factor of Safety

*** 1.023 ***

Failure Surface Specified By 58 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	48.649	545.000
2	53.157	541.040
3	57.786	537.224
4	62.533	533.554
5	67.392	530.034
6	72.359	526.669
7	77.429	523.460
8	82.597	520.411
9	87.858	517.526
10	93.206	514.806
11	98.637	512.255
12	104.145	509.876
13	109.724	507.669
14	115.370	505.639
15	121.077	503.786
16	126.839	502.112
17	132.650	500.620
18	138.505	499.310
19	144.399	498.184
20	150.324	497.242
21	156.277	496.487
22	162.250	495.918
23	168.238	495.537
24	174.234	495.342
25	180.234	495.336
26	186.232	495.517
27	192.220	495.886
28	198.195	496.442
29	204.148	497.184
30	210.076	498.113
31	215.972	499.226
32	221.830	500.524
33	227.645	502.004
34	233.410	503.665
35	239.121	505.505
36	244.771	507.524
37	250.355	509.718
38	255.868	512.086
39	261.305	514.625
40	266.659	517.333
41	271.926	520.207
42	277.100	523.245

```
    43        282.177      526.443
    44        287.151      529.798
    45        292.018      533.307
    46        296.772      536.966
    47        301.410      540.773
    48        305.927      544.723
    49        310.318      548.812
    50        314.578      553.037
    51        318.705      557.392
    52        322.693      561.875
    53        326.539      566.480
    54        330.239      571.204
    55        333.789      576.040
    56        337.187      580.986
    57        340.428      586.035
    58        342.801      590.000
Circle Center At X =   177.434 ; Y =   687.062 ; and Radius =   191.748
      Factor of Safety
     ***    1.025   ***
         **** END OF GSTABL7 OUTPUT ****
```

G.25

(G.26 & G.27 Deleted)

Sliding displacement models from Rathje and Antonakos (2011)

Prediction of dynamic response and sliding displacement of a sliding mass during earthquake shaking

Cross Section: A-A'

File: 2% in 50 year hazard

Inputs	
PGA (g)	0.54
PGV (cm/s)	80.15
k_y (g)	0.23
T_m (s)	0.467
T_s (s)	0.18
M	7
T_s/T_m	0.385439

Dynamic Response			
	median	-1σ	+1σ
kmax (g)	0.428	0.333	0.550
k-velmax (cm/s)	86.70	67.52	111.32

Displacement Given Median Dynamic Response					
Displacement Model			Sliding Displacements, D (cm)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	1.69	0.867	5.40	2.27	12.84
[kmax, k-velmax]	2.08	0.553	7.98	4.59	13.87
			Sliding Displacements, D (in)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	1.69	0.867	2.12	0.89	5.06
[kmax, k-velmax]	2.08	0.553	3.14	1.81	5.46

PGA	Peak Ground Acceleration (g)
PGV	Peak Ground Velocity (cm/s)
k_y	Critical or Yield Acceleration (g)
T_m	Mean Period of an Earthquake Motion (s)
T_s	Site Period or the Natural Period of a Sliding Mass (s). For rigid sliding set T_s = 0 s
M	Earthquake Magnitude
kmax	maximum seismic coefficient (g)
k-velmax	the maximum velocity of the seismic coefficient-time history (cm/s)
D	Predicted Sliding Displacment (cm)

Reference: Rathje, E.M. and Antonakos, G. 2011. "A Unified Model for Predicting Earthquake-Induced Sliding Displacements of Rigid and Flexible Slopes," Engineering Geology, 122, 51-60, doi:10.1016/j.enggeo.2010.12.004.

Sliding displacement models from Rathje and Antonakos (2011)

Prediction of dynamic response and sliding displacement of a sliding mass during earthquake shaking

Cross Section: A-A'

File: ≃ 10% in 50 year hazard

Inputs	
PGA (g)	0.35
PGV (cm/s)	41.44
k_y (g)	0.23
T_m (s)	0.501
T_s (s)	0.18
M	7
T_s/T_m	0.359281

Dynamic Response			
	median	-1σ	+1σ
kmax (g)	0.331	0.258	0.425
k-velmax (cm/s)	45.29	35.27	58.15

Displacement Given Median Dynamic Response					
Displacement Model			Sliding Displacements, D (cm)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	0.23	0.918	1.26	0.50	3.15
[kmax, k-velmax]	-0.03	0.598	0.97	0.53	1.76
			Sliding Displacements, D (in)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	0.23	0.918	0.50	0.20	1.24
[kmax, k-velmax]	-0.03	0.598	0.38	0.21	0.69

$\pm\frac{1}{2}$" okay

PGA	Peak Ground Acceleration (g)
PGV	Peak Ground Velocity (cm/s)
k_y	Critical or Yield Acceleration (g)
T_m	Mean Period of an Earthquake Motion (s)
T_s	Site Period or the Natural Period of a Sliding Mass (s). For rigid sliding set T_s = 0 s
M	Earthquake Magnitude
kmax	maximum seismic coefficient (g)
k-velmax	the maximum velocity of the seismic coefficient-time history (cm/s)
D	Predicted Sliding Displacment (cm)

Reference: Rathje, E.M. and Antonakos, G. 2011. "A Unified Model for Predicting Earthquake-Induced Sliding Displacements of Rigid and Flexible Slopes," Engineering Geology, 122, 51-60, doi:10.1016/j.enggeo.2010.12.004.

5.29

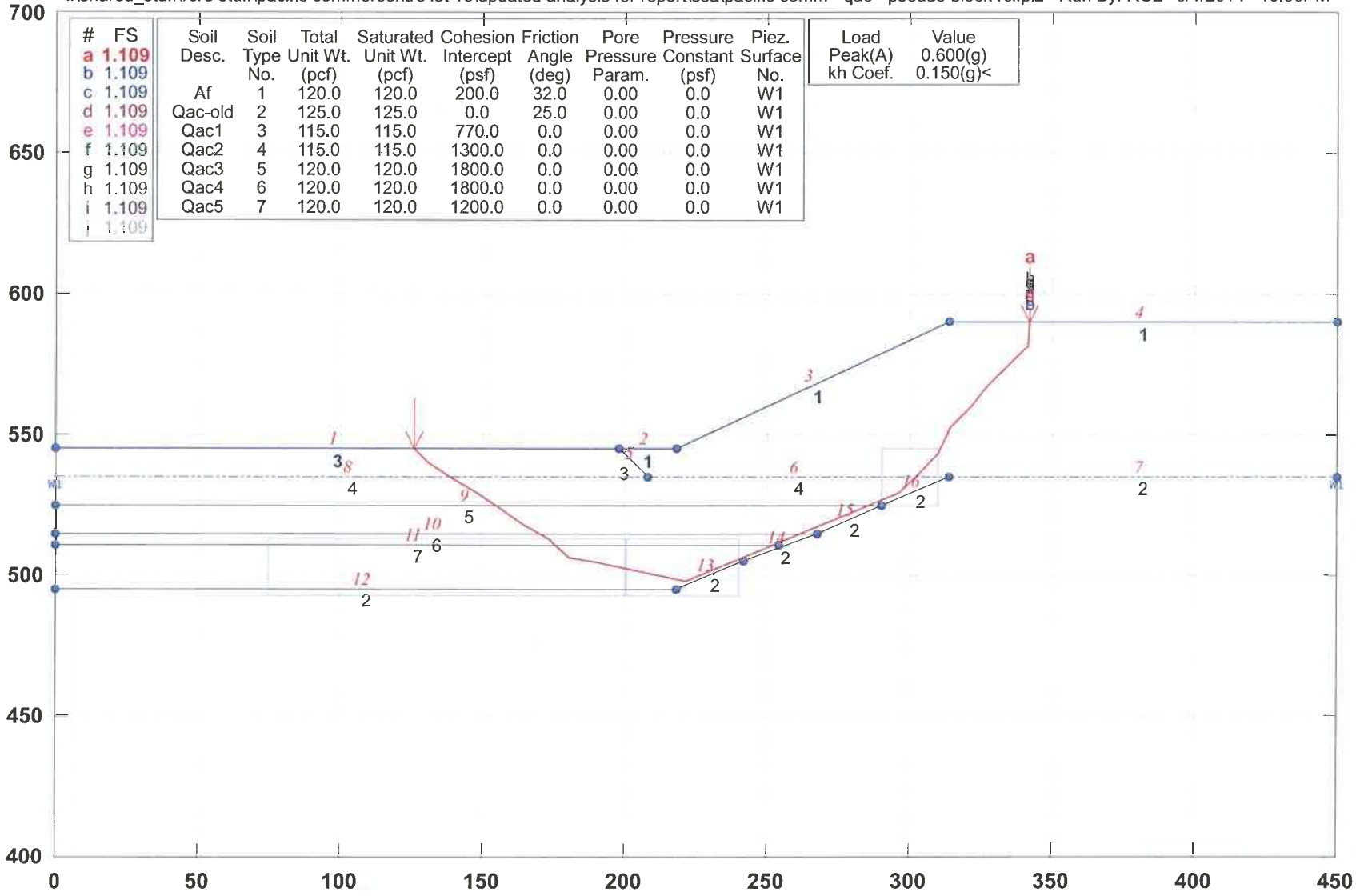
Pacific Commercenter Lot 13 Qac - Seismic -Pseudostatic -Block

I:\shared_stuff\rcl's stuff\pacific commercentre lot 13\updated analysis for report\ssa\pacific comm - qac - pseudo block rcl.pl2 Run By: RCL 6/1/2014 10:56PM

#	FS
a	1.109
b	1.109
c	1.109
d	1.109
e	1.109
f	1.109
g	1.109
h	1.109
i	1.109
j	1.109

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Af	1	120.0	120.0	200.0	32.0	0.00	0.0	W1
Qac-old	2	125.0	125.0	0.0	25.0	0.00	0.0	W1
Qac1	3	115.0	115.0	770.0	0.0	0.00	0.0	W1
Qac2	4	115.0	115.0	1300.0	0.0	0.00	0.0	W1
Qac3	5	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac4	6	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac5	7	120.0	120.0	1200.0	0.0	0.00	0.0	W1

Load	Value
Peak(A)	0.600(g)
kh Coef.	0.150(g)<



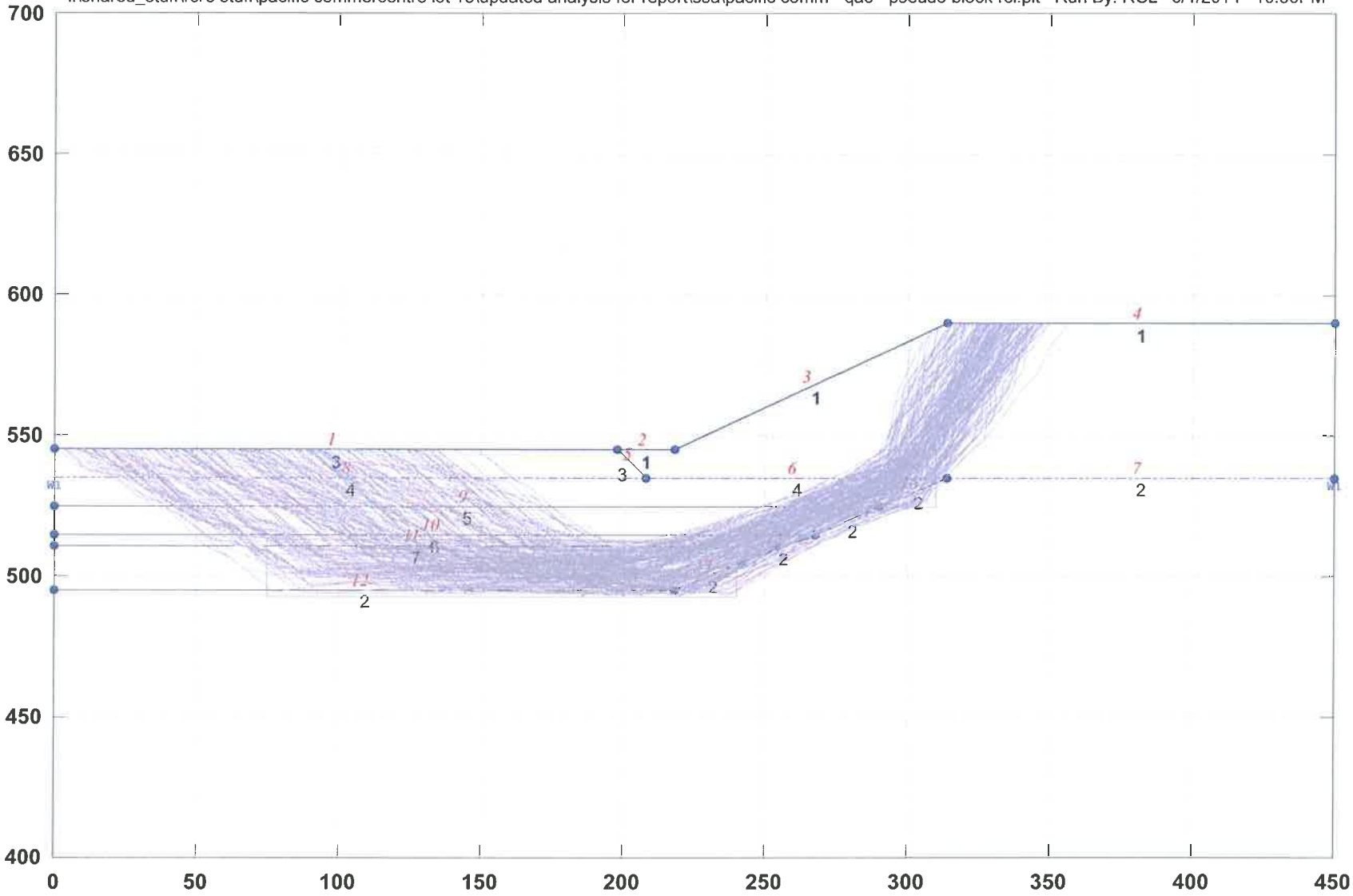
GSTABL7 v.2 FSmin=1.109

Safety Factors Are Calculated By The Simplified Janbu Method

G.3o

Pacific Commercenter Lot 13 Qac - Seismic -Pseudostatic -Block

I:\shared_stuff\rcl's stuff\pacific commercentre lot 13\updated analysis for report\ssa\pacific comm - qac - pseudo block rcl.plt Run By: RCL 6/1/2014 10:56PM



G-31

*** GSTABL7 ***

** GSTABL7 by Garry H. Gregory, P.E. **

** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 6/1/2014
Time of Run: 10:56PM
Run By: RCL
Input Data Filename: L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDATED analysis for report\SSA\pacific comm - qac - pseudo block rcl.in
Output Filename: L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDATED analysis for report\SSA\pacific comm - qac - pseudo block rcl.OUT
Unit System: English
Plotted Output Filename: L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDATED analysis for report\SSA\pacific comm - qac - pseudo block rcl.PLT
PROBLEM DESCRIPTION: Pacific Commercenter Lot 13
Qac - Seismic -Pseudostatic -Block

BOUNDARY COORDINATES

4 Top Boundaries
16 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	545.00	198.00	545.00	3
2	198.00	545.00	218.00	545.00	1
3	218.00	545.00	314.00	590.00	1
4	314.00	590.00	450.00	590.00	1
5	198.00	545.00	208.00	535.00	3
6	208.00	535.00	314.00	535.00	4
7	314.00	535.00	450.00	535.00	2
8	0.00	535.00	208.00	535.00	4
9	0.00	525.00	290.00	525.00	5
10	0.00	515.00	268.00	515.00	6
11	0.00	511.00	254.00	511.00	7
12	0.00	495.00	218.00	495.00	2
13	218.00	495.00	242.00	505.00	2
14	242.00	505.00	268.00	515.00	2
15	268.00	515.00	290.00	525.00	2
16	290.00	525.00	314.00	535.00	2

User Specified Y-Origin = 400.00(ft)
User Specified X-Plus Value = 50.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	200.0	32.0	0.00	0.0	1
2	125.0	125.0	0.0	25.0	0.00	0.0	1
3	115.0	115.0	770.0	0.0	0.00	0.0	1
4	115.0	115.0	1300.0	0.0	0.00	0.0	1
5	120.0	120.0	1800.0	0.0	0.00	0.0	1
6	120.0	120.0	1800.0	0.0	0.00	0.0	1
7	120.0	120.0	1200.0	0.0	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	535.00

 G.32

2 450.00 535.00

Specified Peak Ground Acceleration Coefficient (A) = 0.600(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
4000 Trial Surfaces Have Been Generated.
3 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	75.00	503.00	200.00	503.00	20.00
2	201.00	503.00	240.00	503.00	20.00
3	290.00	535.00	310.00	535.00	20.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Simplified Janbu Method * *
Total Number of Trial Surfaces Attempted = 4000
Number of Trial Surfaces With Valid FS = 4000
Statistical Data On All Valid FS Values:
FS Max = 1.796 FS Min = 1.109 FS Ave = 1.390
Standard Deviation = 0.139 Coefficient of Variation = 9.99 %
Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Individual data on the 30 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Surcharge Force Ver (lbs)	Surcharge Load (lbs)
1	5.2	1423.9	0.0	0.0	0.	0.	213.6	0.0	0.0
2	7.2	6078.6	0.0	0.0	0.	0.	911.8	0.0	0.0
3	0.9	1071.8	0.0	23.0	0.	0.	160.8	0.0	0.0
4	8.6	13050.3	0.0	2016.8	0.	0.	1957.5	0.0	0.0
5	6.1	12497.1	0.0	3638.7	0.	0.	1874.6	0.0	0.0
6	2.2	5150.1	0.0	1756.9	0.	0.	772.5	0.0	0.0
7	8.4	23493.6	0.0	8877.5	0.	0.	3524.0	0.0	0.0
8	5.2	17386.4	0.0	6984.7	0.	0.	2608.0	0.0	0.0
9	3.4	12347.2	0.0	5164.8	0.	0.	1852.1	0.0	0.0
10	2.1	8028.8	0.0	4162.9	0.	0.	1204.3	0.0	0.0
11	5.1	21766.2	0.0	11725.7	0.	0.	3264.9	0.0	0.0
12	9.9	46082.6	0.0	18584.9	0.	0.	6912.4	0.0	0.0

13	7.8	38136.7	0.0	15651.3	0.	0.	5720.5	0.0	0.0
14	10.0	51060.0	0.0	21107.6	0.	0.	7659.0	0.0	0.0
15	10.0	53699.5	0.0	22374.6	0.	0.	8054.9	0.0	0.0
16	3.6	20293.5	0.0	8369.9	0.	0.	3044.0	0.0	0.0
17	31.1	182806.5	0.0	63920.8	0.	0.	27421.0	0.0	0.0
18	9.7	58062.7	0.0	14348.3	0.	0.	8709.4	0.0	0.0
19	24.1	147824.7	0.0	24457.3	0.	0.	22173.7	0.0	0.0
20	9.9	61742.4	0.0	5310.2	0.	0.	9261.4	0.0	0.0
21	5.9	35915.7	0.0	1534.5	0.	0.	5387.4	0.0	0.0
22	1.2	6953.4	0.0	0.0	0.	0.	1043.0	0.0	0.0
23	6.6	37076.7	0.0	0.0	0.	0.	5561.5	0.0	0.0
24	3.9	19505.8	0.0	0.0	0.	0.	2925.9	0.0	0.0
25	0.3	1212.1	0.0	0.0	0.	0.	181.8	0.0	0.0
26	7.0	28100.1	0.0	0.0	0.	0.	4215.0	0.0	0.0
27	6.3	19729.2	0.0	0.0	0.	0.	2959.4	0.0	0.0
28	7.1	15877.1	0.0	0.0	0.	0.	2381.6	0.0	0.0
29	7.1	9901.4	0.0	0.0	0.	0.	1485.2	0.0	0.0
30	0.4	205.6	0.0	0.0	0.	0.	30.8	0.0	0.0

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

G.34

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795

G.35

18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
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8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
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17	334.563	574.795
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Factor of Safety
*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
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13	310.050	543.678
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*** 1.109 ***

Failure Surface Specified By 19 Coordinate Points

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17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety

*** 1.109 ***

**** END OF GSTABL7 OUTPUT ****

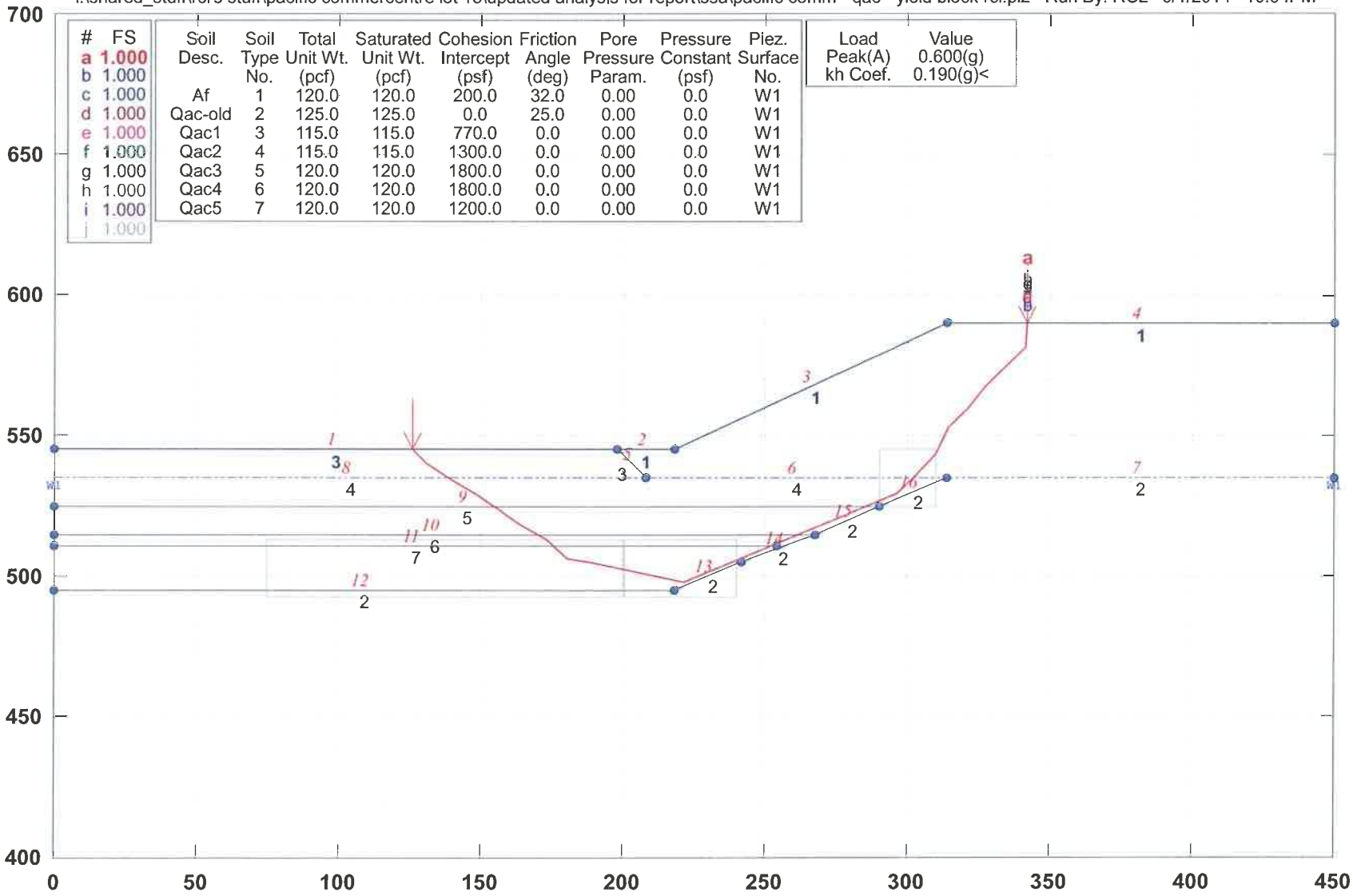
Pacific Commercenter Lot 13 Qac - Seismic -Yield -Block

I:\shared_stuff\rcl's stuff\pacific commercentre lot 13\updated analysis for report\ssa\pacific comm - qac - yield block rcl.pl2 Run By: RCL 6/1/2014 10:54PM

#	FS
a	1.000
b	1.000
c	1.000
d	1.000
e	1.000
f	1.000
g	1.000
h	1.000
i	1.000
j	1.000

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Af	1	120.0	120.0	200.0	32.0	0.00	0.0	W1
Qac-old	2	125.0	125.0	0.0	25.0	0.00	0.0	W1
Qac1	3	115.0	115.0	770.0	0.0	0.00	0.0	W1
Qac2	4	115.0	115.0	1300.0	0.0	0.00	0.0	W1
Qac3	5	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac4	6	120.0	120.0	1800.0	0.0	0.00	0.0	W1
Qac5	7	120.0	120.0	1200.0	0.0	0.00	0.0	W1

Load	Value
Peak(A)	0.600(g)
kh Coef.	0.190(g)<



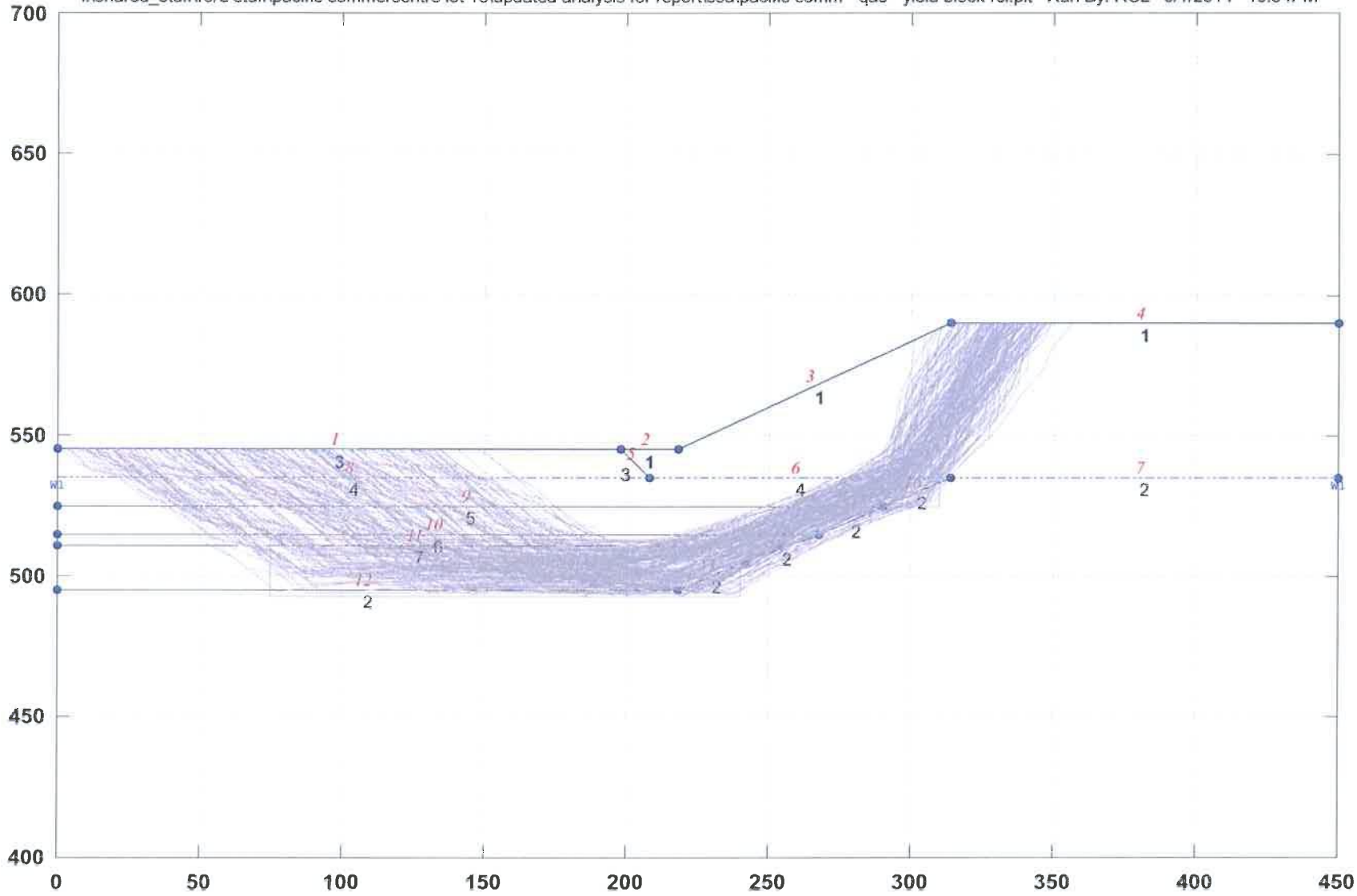
GSTABL7 v.2 FSmin=1.000

Safety Factors Are Calculated By The Simplified Janbu Method

G.28

Pacific Commercenter Lot 13 Qac - Seismic -Yield -Block

I:\shared_stuff\rcl's stuff\pacific commercentre lot 13\updated analysis for report\ssa\pacific comm - qac - yield block rcl.plt Run By: RCL 6/1/2014 10:54PM



G.39

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                                  *** GSTABL7 ***
                        ** GSTABL7 by Garry H. Gregory, P.E. **
          ** Original Version 1.0, January 1996; Current Version 2.005, Sept. 2006 **
                        (All Rights Reserved-Unauthorized Use Prohibited)
    ************************************************************************
                          SLOPE STABILITY ANALYSIS SYSTEM
               Modified Bishop, Simplified Janbu, or GLE Method of Slices.
               (Includes Spencer & Morgenstern-Price Type Analysis)
               Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
               Nonlinear Undrained Shear Strength, Curved Phi Envelope,
               Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
               Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
    ************************************************************************
    Analysis Run Date:        6/1/2014
    Time of Run:              10:54PM
    Run By:                   RCL
    Input Data Filename:      L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDA
TED analysis for report\SSA\pacific comm - qac - yield block rcl.in
    Output Filename:          L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDA
TED analysis for report\SSA\pacific comm - qac - yield block rcl.OUT
    Unit System:              English
    Plotted Output Filename:  L:\Shared_Stuff\RCL's Stuff\Pacific Commercentre Lot 13\UPDA
TED analysis for report\SSA\pacific comm - qac - yield block rcl.PLT
    PROBLEM DESCRIPTION:  Pacific Commercenter Lot 13
                          Qac - Seismic -Yield -Block
    BOUNDARY COORDINATES
```

4 Top Boundaries
16 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	545.00	198.00	545.00	3
2	198.00	545.00	218.00	545.00	1
3	218.00	545.00	314.00	590.00	1
4	314.00	590.00	450.00	590.00	1
5	198.00	545.00	208.00	535.00	3
6	208.00	535.00	314.00	535.00	4
7	314.00	535.00	450.00	535.00	2
8	0.00	535.00	208.00	535.00	4
9	0.00	525.00	290.00	525.00	5
10	0.00	515.00	268.00	515.00	6
11	0.00	511.00	254.00	511.00	7
12	0.00	495.00	218.00	495.00	2
13	218.00	495.00	242.00	505.00	2
14	242.00	505.00	268.00	515.00	2
15	268.00	515.00	290.00	525.00	2
16	290.00	525.00	314.00	535.00	2

User Specified Y-Origin = 400.00(ft)
User Specified X-Plus Value = 50.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	200.0	32.0	0.00	0.0	1
2	125.0	125.0	0.0	25.0	0.00	0.0	1
3	115.0	115.0	770.0	0.0	0.00	0.0	1
4	115.0	115.0	1300.0	0.0	0.00	0.0	1
5	120.0	120.0	1800.0	0.0	0.00	0.0	1
6	120.0	120.0	1800.0	0.0	0.00	0.0	1
7	120.0	120.0	1200.0	0.0	0.00	0.0	1

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	535.00
2	450.00	535.00

Specified Peak Ground Acceleration Coefficient (A) = 0.600(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.190(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
4000 Trial Surfaces Have Been Generated.
3 Boxes Specified For Generation Of Central Block Base Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	75.00	503.00	200.00	503.00	20.00
2	201.00	503.00	240.00	503.00	20.00
3	290.00	535.00	310.00	535.00	20.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Simplified Janbu Method * *
Total Number of Trial Surfaces Attempted = 4000
Number of Trial Surfaces With Valid FS = 4000
Statistical Data On All Valid FS Values:
FS Max = 1.605 FS Min = 1.000 FS Ave = 1.231

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Standard Deviation = 0.120 Coefficient of Variation = 9.73 %

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety

*** 1.000 ***

Individual data on the 30 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Surcharge Load Ver (lbs)	Surcharge Load (lbs)
1	5.2	1423.9	0.0	0.0	0.	0.	270.5	0.0	0.0
2	7.2	6078.6	0.0	0.0	0.	0.	1154.9	0.0	0.0
3	0.9	1071.8	0.0	23.0	0.	0.	203.6	0.0	0.0
4	8.6	13050.3	0.0	2016.8	0.	0.	2479.6	0.0	0.0
5	6.1	12497.1	0.0	3638.7	0.	0.	2374.4	0.0	0.0
6	2.2	5150.1	0.0	1756.9	0.	0.	978.5	0.0	0.0
7	8.4	23493.6	0.0	8877.5	0.	0.	4463.8	0.0	0.0
8	5.2	17386.4	0.0	6984.7	0.	0.	3303.4	0.0	0.0
9	3.4	12347.2	0.0	5164.8	0.	0.	2346.0	0.0	0.0
10	2.1	8028.8	0.0	4162.9	0.	0.	1525.5	0.0	0.0
11	5.1	21766.2	0.0	11725.7	0.	0.	4135.6	0.0	0.0
12	9.9	46082.6	0.0	18584.9	0.	0.	8755.7	0.0	0.0
13	7.8	38136.7	0.0	15651.3	0.	0.	7246.0	0.0	0.0
14	10.0	51060.0	0.0	21107.6	0.	0.	9701.4	0.0	0.0
15	10.0	53699.5	0.0	22374.6	0.	0.	10202.9	0.0	0.0
16	3.6	20293.5	0.0	8369.9	0.	0.	3855.8	0.0	0.0
17	31.1	182806.5	0.0	63920.8	0.	0.	34733.2	0.0	0.0
18	9.7	58062.7	0.0	14348.3	0.	0.	11031.9	0.0	0.0
19	24.1	147824.7	0.0	24457.3	0.	0.	28086.7	0.0	0.0
20	9.9	61742.4	0.0	5310.2	0.	0.	11731.1	0.0	0.0
21	5.9	35915.7	0.0	1534.5	0.	0.	6824.0	0.0	0.0
22	1.2	6953.4	0.0	0.0	0.	0.	1321.2	0.0	0.0
23	6.6	37076.7	0.0	0.0	0.	0.	7044.6	0.0	0.0
24	3.9	19505.8	0.0	0.0	0.	0.	3706.1	0.0	0.0
25	0.3	1212.1	0.0	0.0	0.	0.	230.3	0.0	0.0
26	7.0	28100.1	0.0	0.0	0.	0.	5339.0	0.0	0.0
27	6.3	19729.2	0.0	0.0	0.	0.	3748.5	0.0	0.0
28	7.1	15877.1	0.0	0.0	0.	0.	3016.6	0.0	0.0
29	7.1	9901.4	0.0	0.0	0.	0.	1881.3	0.0	0.0
30	0.4	205.6	0.0	0.0	0.	0.	39.1	0.0	0.0

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197

6.42

5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.000 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
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6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
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14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.000 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.000 ***

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No.	(ft)	(ft)
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10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.000 ***

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Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
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9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867

G.44

19	342.055	590.000

Factor of Safety
*** 1.000 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.000 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
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11	296.376	529.095
12	303.429	536.184
13	310.050	543.678
14	314.269	552.745
15	321.225	559.929
16	327.505	567.711
17	334.563	574.795
18	341.634	581.867
19	342.055	590.000

Factor of Safety
*** 1.000 ***

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	126.034	545.000
2	131.245	540.248
3	139.313	534.339
4	147.889	529.197
5	156.115	523.510
6	164.484	518.036
7	173.136	513.023
8	180.306	506.052
9	190.166	504.381
10	221.602	498.121
11	296.376	529.095
12	303.429	536.184
13	310.050	543.678

```
14          314.269       552.745
15          321.225       559.929
16          327.505       567.711
17          334.563       574.795
18          341.634       581.867
19          342.055       590.000
    Factor of Safety
***      1.000    ***
     **** END OF GSTABL7 OUTPUT ****
```

Sliding displacement models from Rathje and Antonakos (2011)

Prediction of dynamic response and sliding displacement of a sliding mass during earthquake shaking

Cross Section: A-A'

File: 2% in 50 year hazard

Inputs	
PGA (g)	0.54
PGV (cm/s)	80.15
k_y (g)	0.19
T_m (s)	0.467
T_s (s)	0.18
M	7
T_s/T_m	0.385439

Dynamic Response			
	median	-1σ	+1σ
kmax (g)	0.428	0.333	0.550
k-velmax (cm/s)	86.70	67.52	111.32

Displacement Given Median Dynamic Response					
Displacement Model			Sliding Displacements, D (cm)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	2.36	0.837	10.58	4.58	24.42
[kmax, k-velmax]	2.75	0.526	15.57	9.20	26.36
			Sliding Displacements, D (in)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	2.36	0.837	4.16	1.80	9.62
[kmax, k-velmax]	2.75	0.526	6.13	3.62	10.38

PGA	Peak Ground Acceleration (g)
PGV	Peak Ground Velocity (cm/s)
k_y	Critical or Yield Acceleration (g)
T_m	Mean Period of an Earthquake Motion (s)
T_s	Site Period or the Natural Period of a Sliding Mass (s). For rigid sliding set T_s = 0 s
M	Earthquake Magnitude
kmax	maximum seismic coefficient (g)
k-velmax	the maximum velocity of the seismic coefficient-time history (cm/s)
D	Predicted Sliding Displacment (cm)

Reference: Rathje, E.M. and Antonakos, G. 2011. "A Unified Model for Predicting Earthquake-Induced Sliding Displacements of Rigid and Flexible Slopes," Engineering Geology, 122, 51-60, doi:10.1016/j.enggeo.2010.12.004.

2.47

Sliding displacement models from Rathje and Antonakos (2011)

Prediction of dynamic response and sliding displacement of a sliding mass during earthquake shaking

Cross Section: A-A'

File: ~10% in 50 year hazard

Inputs	
PGA (g)	0.35
PGV (cm/s)	41.44
k_y (g)	0.19
T_m (s)	0.501
T_s (s)	0.18
M	7
T_s/T_m	0.359281

Dynamic Response			
	median	-1σ	+1σ
kmax (g)	0.331	0.258	0.425
k-velmax (cm/s)	45.29	35.27	58.15

Displacement Given Median Dynamic Response					
Displacement Model			Sliding Displacements, D (cm)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	1.23	0.879	3.42	1.42	8.25
[kmax, k-velmax]	0.97	0.563	2.63	1.50	4.62
			Sliding Displacements, D (in)		
	μ_{lnD}	σ_{lnD}	median	-1σ	+1σ
[kmax, M]	1.23	0.879	1.35	0.56	3.25
[kmax, k-velmax]	0.97	0.563	1.04	0.59	1.82

~1" okay

PGA	Peak Ground Acceleration (g)
PGV	Peak Ground Velocity (cm/s)
k_y	Critical or Yield Acceleration (g)
T_m	Mean Period of an Earthquake Motion (s)
T_s	Site Period or the Natural Period of a Sliding Mass (s). For rigid sliding set T_s = 0 s
M	Earthquake Magnitude
kmax	maximum seismic coefficient (g)
k-velmax	the maximum velocity of the seismic coefficient-time history (cm/s)
D	Predicted Sliding Displacment (cm)

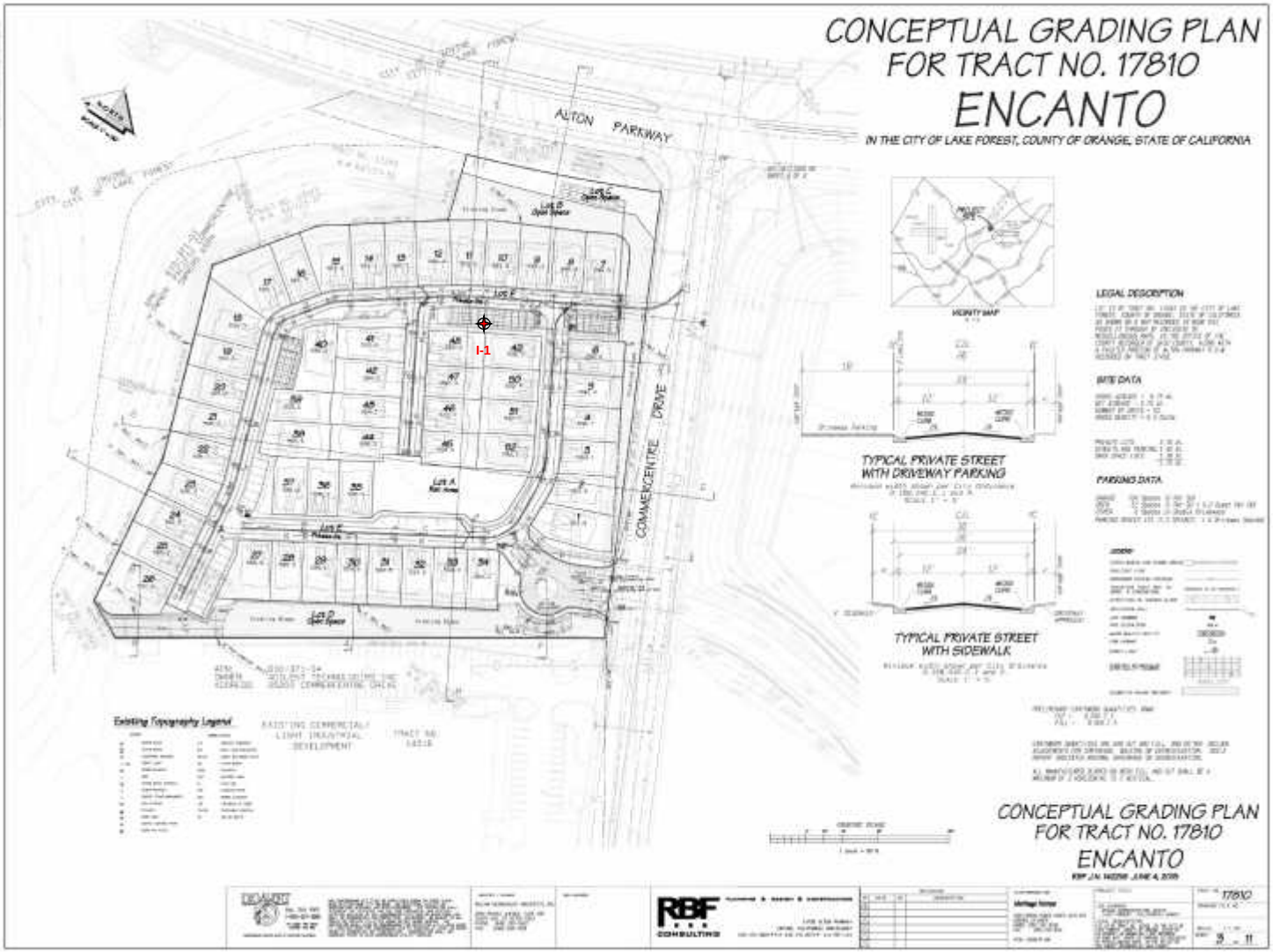
Reference: Rathje, E.M. and Antonakos, G. 2011. "A Unified Model for Predicting Earthquake-Induced Sliding Displacements of Rigid and Flexible Slopes," Engineering Geology, 122, 51-60, doi:10.1016/j.enggeo.2010.12.004.

52.48

APPENDIX C

Site Plan

Response to City Review Checklist
Lot 13, Tract 13343
Lake Forest, Orange County, California
Project No. 1162-CR3



LEGEND

I-1 — Approximate Location of Double-ring Infiltrometer Test

Meritage Homes
Southwest Corner of Commercentre and
Alton Parkway
Lot 13 of Tract 13343
Lake Forest, Orange County, California

GeoTek Project No. 1162-CR3

GEOTEK

Figure 1

Site Plan

APPENDIX D

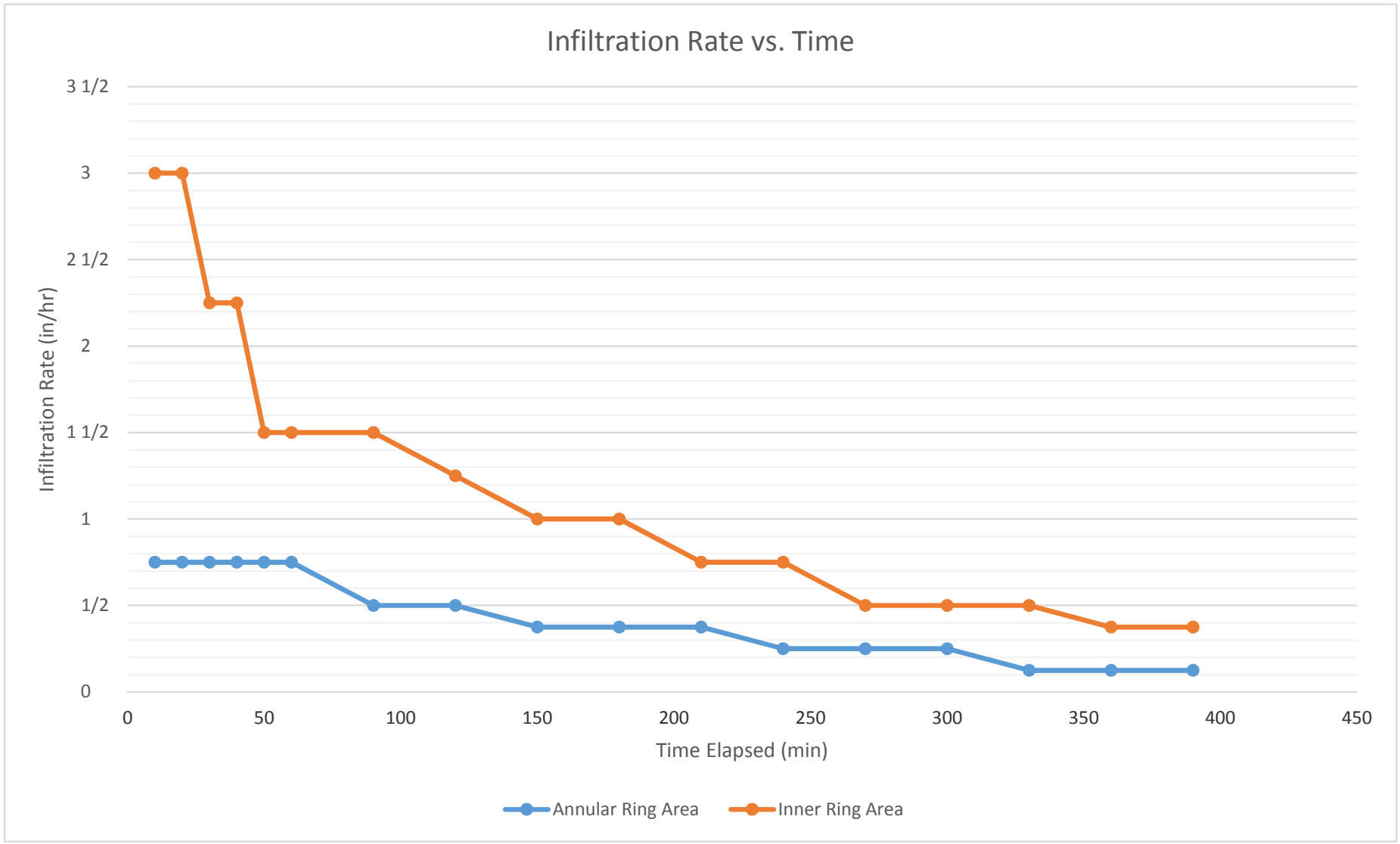
Double-ring Infiltrometer Test Results

Response to City Review Checklist
Lot 13, Tract 13343
Lake Forest, Orange County, California
Project No. 1162-CR3

DOUBLE RING INFILTROMETER TEST DATA

Project Name:	1162-CR3	**Liquid Used:**	Tap Water
Trench No.:	I-1	**Equipment:**	Double-Ring Infiltrometer
Tested by:	DVG	**Liquid Level Mainted by Using:**	Tape Measure
Date of Testing:	5/6/2015	**Penetration of Rings Into Soil:**	Inner Ring - 4 in.; Outer Ring - 6 in.
Inner Ring Diameter (in.):	12	**Outer Ring Diameter (in):**	24
Water Table Depth (ft.):	>50	**USCS Classification:**	SM

				FLOW READINGS					INFILTRATION RATE		
TRIAL NO.	**START/ END**	**TIME**	**INCREMENTAL TIME (min)**	**INNER RING (in)**	**RING FLOW (water added in ml)**	**ANNULAR READING (in)**	**SPACE FLOW (water added in ml)**	**LIQUID TEMP. (F)**	**INNER (in/hr)**	**ANNULAR (in/hr)**	**REMARKS**
1	S	9:05:00 AM		10		12					
	E	9:15:00 AM	10	9 7/8	232	11 1/2	2780	72	3/4	3	
2	S	9:17:00 AM		10		12					
	E	9:27:00 AM	10	9 7/8	232	11 1/2	2780	72	3/4	3	
3	S	9:29:00 AM		10		12					
	E	9:39:00 AM	10	9 7/8	232	11 5/8	2085	72	3/4	2 1/4	
4	S	9:41:00 AM		10		12					
	E	9:51:00 AM	10	9 7/8	232	11 5/8	2085	72	3/4	2 1/4	
5	S	9:53:00 AM		10		12					
	E	10:03:00 AM	10	9 7/8	232	11 3/4	1390	72	3/4	1 1/2	
6	S	10:05:00 AM		10		12					
	E	10:15:00 AM	10	9 7/8	232	11 3/4	1390	72	3/4	1 1/2	
7	S	10:17:00 AM		10		12					
	E	10:47:00 AM	30	9 3/4	463	11 1/4	4170	72	1/2	1 1/2	
8	S	10:49:00 AM		10		12					
	E	11:19:00 AM	30	9 3/4	463	11 3/8	3475	72	1/2	1 1/4	
9	S	11:21:00 AM		10		12					
	E	11:51:00 AM	30	9 13/16	348	11 1/2	2780	72	3/8	1	
10	S	11:53:00 AM		10		12					
	E	12:23:00 PM	30	9 13/16	348	11 1/2	2780	72	3/8	1	
11	S	12:25:00 PM		10		12					
	E	12:55:00 PM	30	9 13/16	348	11 5/8	2085	72	3/8	3/4	
12	S	12:57:00 PM		10		12					
	E	1:27:00 PM	30	9 7/8	232	11 5/8	2085	72	1/4	3/4	
13	S	1:29:00 PM		10		12					
	E	1:59:00 PM	30	9 7/8	232	11 3/4	1390	72	1/4	1/2	
14	S	2:01:00 PM		10		12					
	E	2:31:00 PM	30	9 7/8	232	11 3/4	1390	72	1/4	1/2	
15	S	2:33:00 PM		10		12					
	E	3:03:00 PM	30	9 15/16	116	11 3/4	1390	72	1/8	1/2	
16	S	3:05:00 PM		10		12					
	E	3:35:00 PM	30	9 15/16	116	11 13/16	1043	72	1/8	3/8	
17	S	3:37:00 PM		10		12					
	E	4:07:00 PM	30	9 15/16	116	11 13/16	1043	72	1/8	3/8	


Infiltration Rate vs. Time
Infiltration Rate (in/hr)
3 1/2
3
2 1/2
2
1 1/2
1
1/2
0
0
50
100
150
200
250
300
350
400
450
Time Elapsed (min)
Annular Ring Area
Inner Ring Area

APPENDIX E

Recent Laboratory Test Data

Response to City Review Checklist
Lot 13, Tract 13343
Lake Forest, Orange County, California
Project No. 1162-CR3

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Meritage	***Tested/ Checked By:***	DI Lab No Corona
Project Number:	1162-CR3	***Date Tested:***	5/7/2015
Project Location:	Commercentre & Alton Parkway	***Sample Source:***	NW Corner Subgrade
		Sample Description:	Silty Sand

Ring #:_____ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	754.4
B	Weight of ring (gm)	366.1
C	Net weight of sample (gm)	**388.3**
D	Wet Density, lb / ft3 (C*0.3016)	**117.1**
E	Dry Density, lb / ft3 (D/1.F)	**105.0**

SATURATION DETERMINATION

F	Moisture Content, %	11.5
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20°C, (pcf)	**62.3**
I	% Saturation	**51.6**

READINGS			
DATE	TIME	READING	
5/7/2015	5:37	0.2380	Initial
	5:47	0.2380	10 min/Dry
	9:37	0.2490	
5/8/2015	5:37	0.2490	Final

FINAL MOISTURE	
Final Weight of wet sample & tare	% Moisture
781.5	**18.5**

EXPANSION INDEX = 11

Cal Land Engineering, Inc.
dba Quartech Consultants
Geotechnical, Environmental, and Civil Engineering

GeoTek, Inc.
710 East Parkridge Avenue, Suite 105
Corona, California 92879

Client: Meritage
W.O.: 1162-CR3
Project: Commer Center, Lake Forrest

Date: May 12, 2015
QCI Project No.: 15-167-005e
Summarized by: KA

Corrosivity Test Results

Sample ID	Sample Depth (Feet)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 (% By Weight)	Resistivity CT-532 (643) (ohm-cm)
Bulk	N/A	7.19	92	0.0440	930

576 East Lambert Road, Brea, California 92821; Tel: 714-671-1050; Fax: 714-671-1090

APPENDIX F

Grading Guidelines

Response to City Review Checklist
Lot 13, Tract 13343
Lake Forest, Orange County, California
Project No. 1162-CR3

GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the California Building Code, CBC (2013) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.

5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.
6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed unless otherwise specifically indicated in the text of this report.

2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
   a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
   b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
   a) They are not placed in concentrated pockets;
   b) There is a sufficient percentage of fine-grained material to surround the rocks;
   c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable

methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.

2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

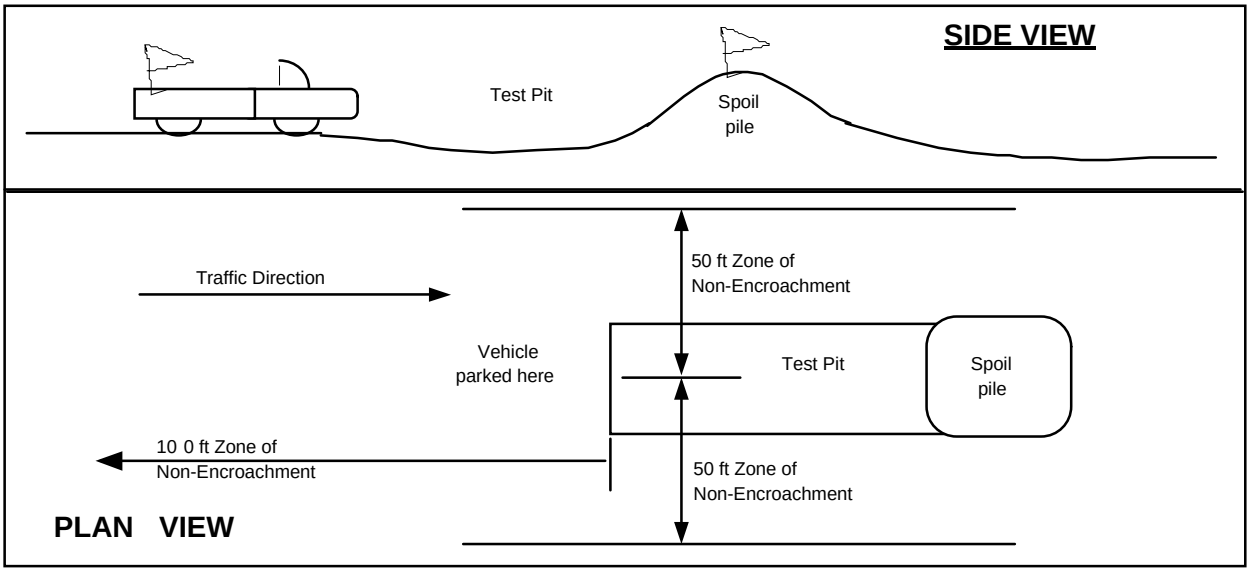
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractors representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project

manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

Document D

City of Lake Forest

Geotechnical Investigation Report Review Checklist

November 6, 2014

Encanto Project

Lot 13, Tract 13343

Lake Forest, Orange County, California

Project No. 1162-CR3

GEOTEK

CITY OF LAKE FOREST
GEOTECHNICAL INVESTIGATION REPORT REVIEW CHECKLIST

Date Authorized:	October 28, 2014	Date Completed:	November 6, 2014
Date of Report:	June 2, 2014		
Consultant:	Geo Tek Inc.	Their Job No.:	1162-CR3
Signed By:	Ronald A. Reed	License No.:	GE 2524 Exp 6/30/15
Signed By:	Edward H. LaMont	License No.:	CEG 1892 Exp 7/31/16
Applicant Name:	Meritage Homes – Encanto	Legend:	N = No
Site Address:	Southwest Corner Commercentre Drive and Alton Parkway		Y = Yes
	Lake Forest, California		NA = Not Applicable
Lot/Tract No.:	Lot 13, Tract 13343		
Proposed Project:			

- **Project Information /Background:**

Y/N Review of existing City files
Y/N Reference to site(s) by street address
Y/N Grading or remedial grading proposed
Y/N Reference to grading/foundation plans by date
Y/N Subsurface investigation

- **Geologic Hazards**:

Hazard	Discussion of Hazard	Mitigation Required	Recommendations for Mitigation
Adverse geologic structure	Y/N/NA	Y/N/NA	Y/N/NA
Collapsible soils	Y/N/NA	Y/N/NA	Y/N/NA
Debris/mud flow	Y/N/NA	Y/N/NA	Y/N/NA
Differential settlement	Y/N/NA	Y/N/NA	Y/N/NA
Erosion/drainage	Y/N/NA	Y/N/NA	Y/N/NA
Expansive soils	Y/N/NA	Y/N/NA	Y/N/NA
Faulting	Y/N/NA	Y/N/NA	Y/N/NA
Fractured bedrock	Y/N/NA	Y/N/NA	Y/N/NA
Groundwater	Y/N/NA	Y/N/NA	Y/N/NA
Landslide	Y/N/NA	Y/N/NA	Y/N/NA
Liquefaction	Y/N/NA	Y/N/NA	Y/N/NA
Settlement	Y/N/NA	Y/N/NA	Y/N/NA
Soil/rock creep	Y/N/NA	Y/N/NA	Y/N/NA
Sulfate rich soils	Y/N/NA	Y/N/NA	Y/N/NA

- **Supporting Analysis/Data:**

Y/N/NA Slope stability calculations
Y/N/NA Shear strength values
Y/N/NA Other laboratory data
Y/N/NA Seismic coefficients/parameters
Y/N/NA Boring/trench logs
Y/N/NA Liquefaction study
Y/N/NA Calculations supporting recommendations
Y/N/NA Reference list

- **Recommendations For:**

Y/N/NA Foundations
Y/N/NA Retaining walls
Y/N/NA Foundation setbacks
Y/N/NA Slabs
Y/N/NA Flatwork
Y/N/NA Grading
Y/N/NA Pools/spas
Y/N/NA Slope/bluff setbacks
Y/N/NA Temporary excavations

- **Geologic Map/Cross-Sections: None provided**

Y/N Accurate topographic base extending sufficiently offsite
Y/N Surficial drainage
Y/N Existing structures
Y/N Boring/trenches plotted
Y/N Geologic contacts/data illustrated
Y/N Consistency with adjoining data/maps
Y/N Cross-Sections sufficient in number, location and detail
Y/N Proposed topography
Y/N Slope gradients
Y/N Proposed structures
Y/N Legend, scale, north arrow
Y/N Location of cross-section(s) shown
Y/N Illustrate setbacks, if any
Y/N Proposed excavations

- **Report Closure:**

CITY OF LAKE FOREST
GEOTECHNICAL INVESTIGATION REPORT REVIEW CHECKLIST

- **Report Closure:**

Y/N Statement as to the adequacy of the site for the intended use.
Y/N Statement that proposed development will not adversely impact adjoining sites.
Y/N Signature of C.E.G.
Y/N Signature of R.C.E. or G.E.

None to City Staff: The submitted report is a Due Diligence report for the subject site and as such does not meet the requirements of a geotechnical investigation. This report relies on previously completed geotechnical investigations, grading reports for previously placed artificial fill materials and reports addressing settlement and surcharge.

- **Recommended Actions:**

Building and Safety:

____ Report Approved ____ Conditional Approval (See Below) _X_ Additional Input Required

Items requiring response/further evaluation:

1. Please provide a copy of the December 24, 2009 and June 2, 2014 Stoney-Miller reports.

2. The consultant shall indicate they are now the consultant of record for the project and that they have reviewed all referenced documents. They shall provide the project address, applicant's name, and the previous consultant's name. The New consultants shall address previous conclusions and recommendations by others by either concurring or providing modifications in writing. The new consultant must state that they are now the geotechnical/geologic consultant of record and accept geotechnical/geologic responsibility for the project. The consultant shall provide a finding that the proposed project is suitable from a geotechnical perspective and will not have an impact on adjacent properties.

3. The consultant shall provide a complete geotechnical grading plan review report based on an engineered grading plan, when available. The results of subsurface investigation by others, additional laboratory testing for expansive soils and sulfate content should be included, along with detailed geologic map and cross sections depicting proposed grading, temporary cuts, recommended setbacks, property lines, adjacent structures, settlement, foundation recommendations, retaining wall recommendations, grading recommendations, updated seismic parameters, etc. Anticipated shoring, slot cutting, restricted use areas, geogrid, etc. should be depicted if proposed.

Additional Comments/Conditions of Approval (no response required):

4. Note to City Staff: Staff should confirm that the Consultants (C.E.G. and R.C.E./G.E.) have signed the final dated grading/foundation plans, thereby verifying the plans' geotechnical conformance with the Consultant's original report and associated addenda.

5. Prior to building permits being issued, the consultant shall provide a complete geotechnical foundation plan review report based on engineered plans, when available. The results of additional testing and data from the grading and laboratory testing should be included, along with detailed geologic cross sections depicting proposed structures, precise grading, temporary cuts, typical footings, recommended setbacks, property lines, and adjacent structures.

CITY OF LAKE FOREST
GEOTECHNICAL INVESTIGATION REPORT REVIEW CHECKLIST

Limitations:

Our review is intended to determine if the submitted report(s) comply with City of Lake Forest Codes and generally accepted geotechnical practices within the local area. The scope of our services for this third party review has been limited to a brief site visit and a review of the above referenced report and associated documents, as supplied by the City of Lake Forest. Re-analysis of reported data and/or calculations and preparation of amended construction or design recommendations are specifically not included within our scope of services. Our review should not be considered as a certification, approval or acceptance of the consultant's work, nor is it meant as an acceptance of liability for final design or construction recommendations made by the geotechnical consultant of record or the project designers or engineers.

BY: _____
James M. Lancaster, Jr., C.E.G. 1927 Expires 6/30/14
KLING CONSULTING GROUP, INC.

BY: _____
Henry F Kling, G.E. 2205, Expires 3/31/14
KLING CONSULTING GROUP, INC.

Document E

GeoTek

Due Diligence Geotechnical Review

June 2, 2014

Encanto Project

Lot 13, Tract 13343

Lake Forest, Orange County, California

Project No. 1162-CR3

GEOTEK

GeoTek, Inc.
710 E. Parkridge Avenue, Suite 105, Corona, California 92879-1097
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

June 2, 2014
Project No. 1162-CR3

Meritage Homes
1250 Corona Pointe Court, Suite 210
Corona, California 92879

Attention: Mr. Peter Vanek

Subject: Due Diligence Geotechnical Review
Proposed Residential Development
Lot 13 of Tract 13343
Lake Forest, Orange County, California

References: Page 6

Dear Mr. Vanek:

In accordance with your request, GeoTek, Inc. (GeoTek) has prepared this Due Diligence Geotechnical Review letter for the subject property. Our general review and review comments are based on a site reconnaissance by representatives of GeoTek completed on March 19, 2014, a limited geotechnical evaluation completed by this firm (GeoTek, 2014), review of existing geotechnical reports for the site provided by you (see References) and our experience in the vicinity and with this type of project development.

❖ Site Description

Lot 13 of Tract 13343 is located southwest of the intersection of Alton Parkway and Commercentre Drive in the City of Lake Forest, Orange County, California. The site consists of roughly 4.5 acres of land, which has previously been graded. An ascending slope is located along the southeast side of the site, adjacent Lot 12 of Tract 13343. An existing industrial/commercial building is located on the adjacent developed property. Descending slopes are located along the northeast, northwest and southwest sides of the subject site (see Figure 1).

❖ Geologic/Geotechnical Conditions

Based on recent site observations and review of the referenced geotechnical reports, the project site is underlain by engineered fill materials which is underlain by both saturated alluvial materials and sedimentary bedrock identified as Vaqueros-Sespe Formation sandstone (Stoney-Miller, 1991). The subject property is not located within a State of California designated Earthquake Fault Zone (*Alquist-Priolo*) or a Special Studies Zone for liquefaction potential, but is partially in an area for landslide potential (see Figure 2).

Initial rough grading of the site was under the observation and testing services of Stoney-Miller Consultants, Inc. Consultants (Stoney-Miller, 1991), with subsequent geotechnical site work reported on by both Stoney-Miller and Bagahi Engineering, Inc. (Bagahi).

The site was initially mass graded from June 1990 to July 1991, as reported by Stoney-Miller (1991). The building pad area of Lot 13 was initially graded to an elevation of roughly 590 to 592 feet msl, with a detention basin located toward the center of the pad, as shown on the USGS map for the area (depicted on Figure 1) and which was graded to an elevation of about 580 msl. Prior to engineered fill placement, previously existing undocumented fill soil, unsaturated alluvium and otherwise loose and unconsolidated soils were removed to expose saturated alluvium or bedrock.

Subsequent to completing the removals described above, engineered fill was placed up to design grades using a 90 percent minimum relative compaction standard. Depths of fill across the graded pad ranged from roughly two (2) feet near the south corner of the pad to roughly 60 feet toward the north and west sides of the pad. Saturated alluvial thickness below the pad area was estimated to range from 25 to 35 feet (Stoney-Miller, 2006), to as much as 60 feet (Stoney-Miller, 1991) feet where it was left in place.

As a result of leaving saturated alluvial soils beneath the subject pad, several episodes of surcharging the site with fill soils were recommended and subsequently performed under the geotechnical observation and monitoring of both Stoney-Miller and Bagahi (see referenced reports). Upon review of the most recent referenced report detailing these surcharging and settlement monitoring episodes (Stoney-Miller, 2009), the surcharging efforts for the subject site has been deemed sufficient to mitigate future ground settlements to less than 1-inch for a time period (design life) of 50 years.

The geotechnical report of rough grading by Stoney-Miller (1991) also provides as-built conditions with respect to geotechnical aspects for the project site.

In addition to the above and as a result of the western site slope area being partially supported by alluvium and designated as within a Seismic Hazard Zone for landslide potential, Stoney-Miller completed a recent slope stability analyses of this area based on their knowledge of the site and a supplemental evaluation (2014). According to Stoney-Miller's analyses, there is a potential for some lateral displacement of the existing site fill slope located toward the western edge of the property. The potential horizontal displacement appears to be on the order of 15 centimeters or less within roughly 50 feet as measured horizontally from the top of slope along a portion of the slope area described above (Stoney-Miller, 2014). Potential vertical settlement in the toe area of the slope as a result of liquefaction was estimated to be on the order of 0.8 to 1.7 inches (Stoney-Miller, 2014).

❖ Existing Site Observations

Minor amounts of vegetation currently exist on site slopes as well as undeveloped lots.

Rodent burrows were observed throughout the site within the graded slopes and pad areas.

Minor to moderate erosion is visible across site, including on slopes associated with the existing surcharge fills and the finish pad.

Several large stockpile/surcharge fills are present on the site. These fills locally have over-steepened slopes, and are reported to have been placed at a minimum relative compaction of 85%. Significant erosion was also locally noted on these surcharge fill areas.

❖ Conclusions and Recommendations

The information provided in the referenced reports appears adequate from a due diligence review perspective.

Items of note that should be addressed for the subject project include the following:

- Existing trash, construction debris, and other deleterious materials present onsite will need to be removed and wasted offsite prior to lot reconditioning.
- The existing surcharge fills will need to be removed, and pad grade returned to design grade elevation(s) or lower, prior to site development. The resulting pad will need to be surficially moisture conditioned and re-compacted prior to development.
- As indicated in the text of this review, the site is partially situated in an area (Seismic Hazard Potential Zone) designated by the State of California as requiring a landslide potential evaluation. A recent geotechnical evaluation addressing this issue has been completed by Stoney-Miller (2014). As such, possible adverse effect(s) associated with the downward and outward movement resultant from the potential seismically triggered lateral slope movement on proposed structural improvements will need to be addressed prior to site development in this area.
- No Expansion Index (EI) testing results for the subject lot was provided. Expansion testing of soils near finish grade elevations for the subject lot will need to be performed in order to provide foundation design criteria.
- Sulfate and corrosion tests results are not provided for the subject lot. Testing of soils from near finish grade elevations at the subject site will need to be performed in order to provide recommendations for site construction.
- Additional expansion, sulfate and corrosion potential testing should be performed. Additional recommendations should may be presented by a corrosion engineer.
- Foundation and seismic design criteria for the site should be updated to current industry code guidelines. Conventional slab-on-grade and/or post-tensioned foundation recommendations will likely be considered feasible for support of the anticipated residential structures. Building foundations will likely be required to be designed to mitigate the possible additional lateral displacement discussed in the report by Stoney-Miller (2014) in the area near the western site slope. Site development plans will need to be reviewed prior to providing site specific geotechnical foundation design criteria.

LIMITATIONS

Our findings are based on site conditions observed and the stated sources. Thus, our comments are professional opinions that are limited to the extent of the available data. These opinions have been derived in accordance with current City/County standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

Respectfully submitted,
GeoTek, Inc.

Edward H. LaMont
CEG 1892, Exp. 7/31/14
Principal Geologist

Ronald A. Reed
GE 2524, Exp 6/30/15
Senior Project Engineer

Enclosures: Figure 1 – Site Location Map
Figure 2 – Seismic Hazard Map

Distribution: (1) Addressee via email

G:\Projects\1151 to 1200\1162CR3 Meritage Homes 25192 Commercentre Drive (Lot 13 of Tract 13343)\Geotechnical Due Diligence\1162CR3 Meritage Homes Due Diligence Lake Forest.doc

REFERENCES

Bagahi Engineering, 1997, "Geotechnical Investigation of Settleemnt, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, CA," Job No. 82a-200-00, dated June 16.

_____, 1998, "Progress Report on Monitoring of Settlement, Tract 13343, Lot 11 (Lot 13), SWC Commercentre Drive and Alton Parkway, Lake Forest, California," Job No. 82a-200-00, dated October 9.

GeoTek, Inc., 2014, "Draft Limited Geotechnical Evaluation, Alluvial Soils Adjacent to Lot 13 of Tract 13343, Lake Forest, Orange County, California," Project No. 1162-CR3, dated May 12.

Stoney-Miller Consultants, Inc., 1993, "Settlement Monitoring, Canyon 2, Lots 12-20, Tract 13343, 20858 Bake Parkway, El Toro, California," Project No: 10425-06, dated December 20.

_____, 1997, "Settlement Monitoring,Canyon 2, Lots 12 through 20, Tract 13343, 20858 Bake Parkway, Pacific Commercentre, Lake Forest, California," Project No: 10425-07, dated March 7.

_____, 2005, "Results of Prelminary Geotechnical Investigation, 25192 Commercentre Drive, Lake Forest, California," Project No: 12576-00, dated May 3.

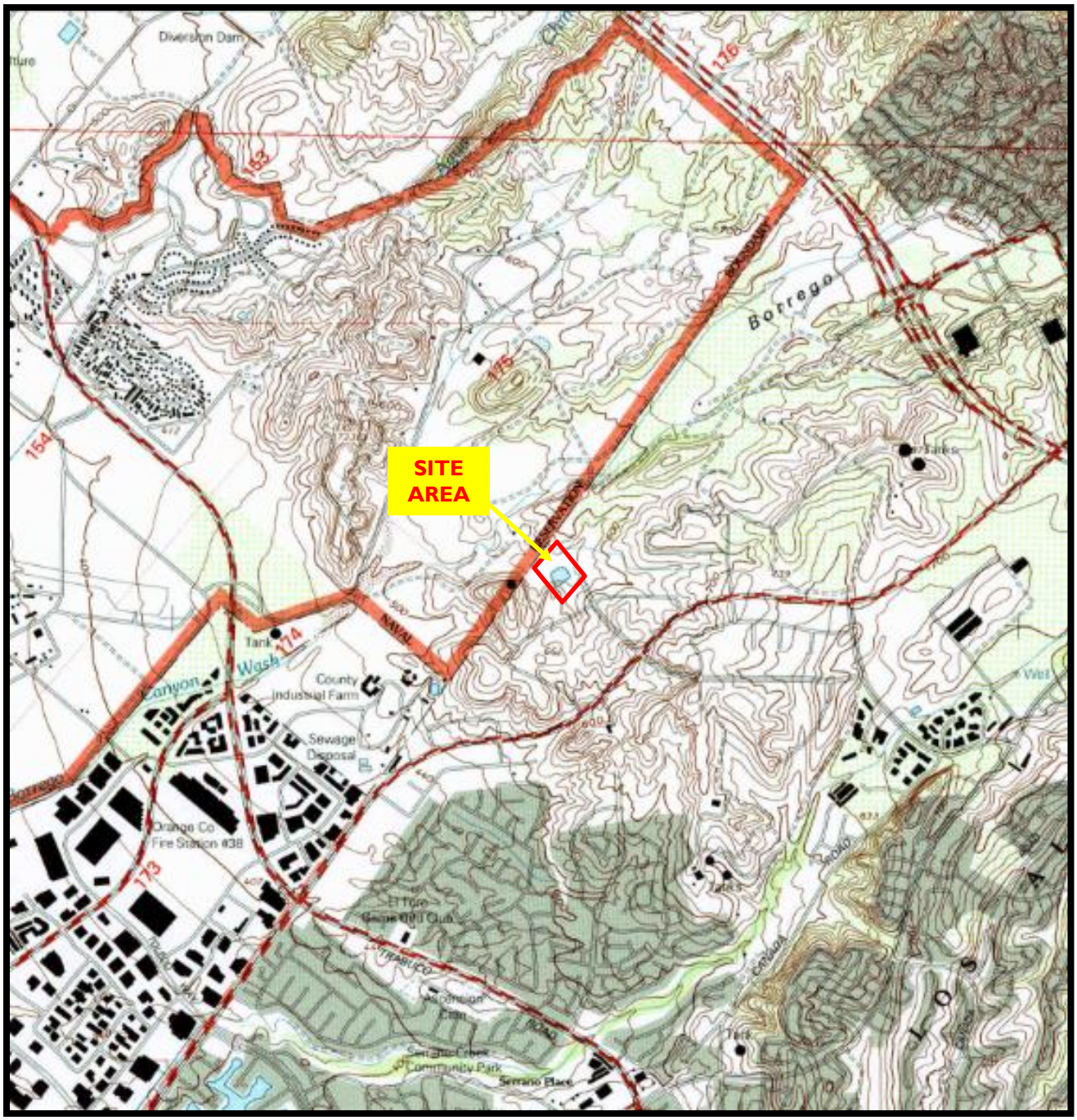
_____, 2006, "Site History & Future Geotechnical Work, Lot 13 of Tract 13343, 25192 Commercentre Drive, Lake Forest, California," Project No: 12655-02, dated January 24.

_____, 2007, "Supplemental Geotechnical Investigation, Lot 13 of Tract 13343, 25191 Commercentre Drive, Lake Forest, California," Project No: 12655-03, dated November 12.

_____, 2008, "Grading Earthwork Observation/Testing Report, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-10/30, dated December 24.

_____, 2009, "Review of Site Settlement, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-50, dated November 12.

_____, 2014, "Supplemental Geotechnical Evaluation, Seismic Hazard Study, Lot 13 of Tract 13343, 25192 Commerce Drive, Lake Forest, California," Project No: 12655-04, Report No: 14-13775, dated June 2.



Meritage Homes
25192 Commercentre Drive
Lot 13 of Tract 13343
City of Lake Forest
County of Orange, California

GeoTek Project No. 1162-CR3

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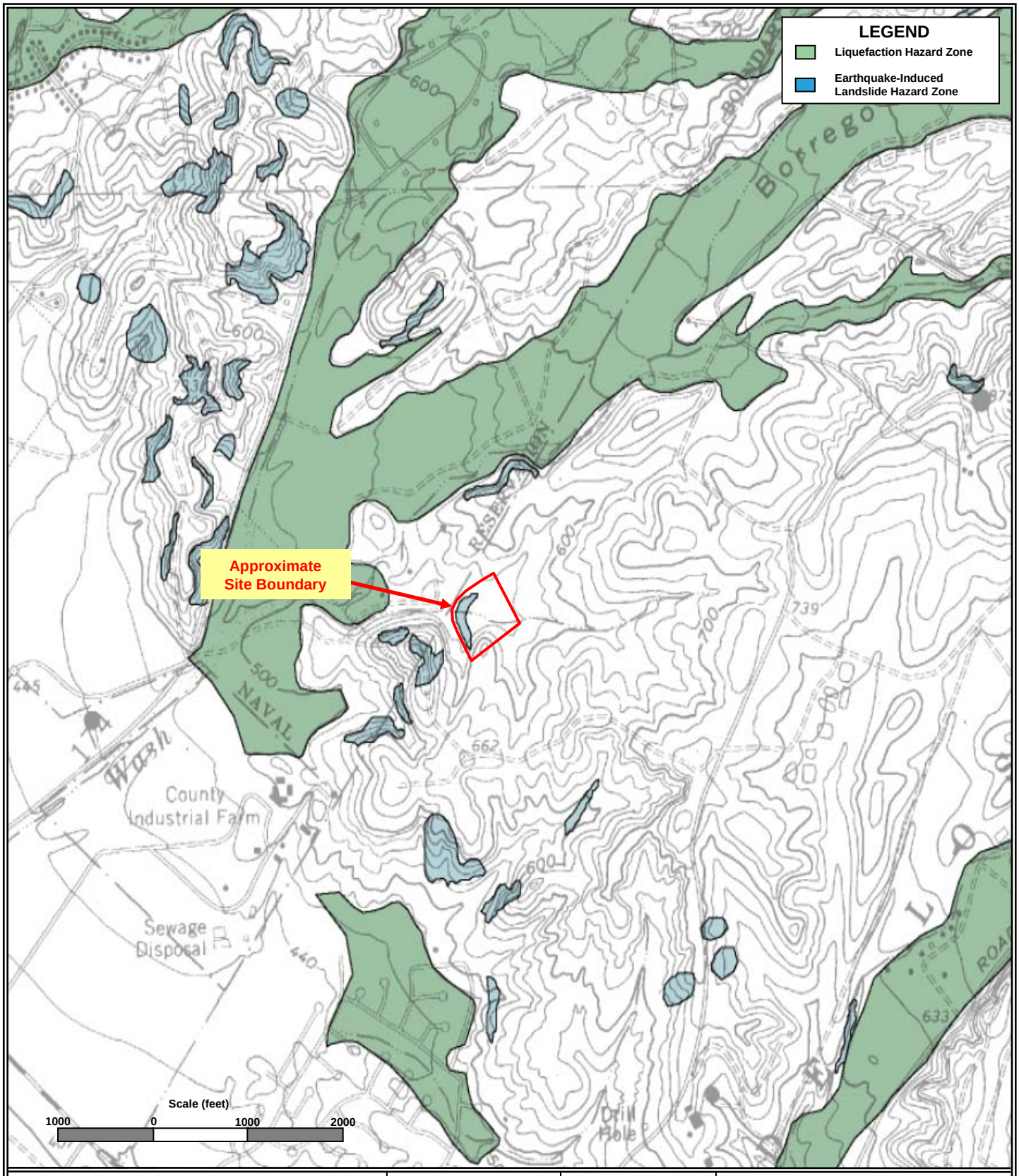
Figure 1

Site Location Map

GEOTEK



Meritage Homes
25192 Commercenter Drive
Lot 13 of Tract 13343
City of Lake Forest
Orange County, California

GeoTek Project No. 1162-CR3

Figure 2

Seismic Hazard Map

GEOTEK