APPENDIX B

GEOTECHNICAL REPORT

Supplemental Geotechnical Investigation, Proposed Residential Development, Koll Center Property Southwest Corner of Van Karman Avenue and Birch Street, Newport Beach, California

PN 19017-01
June 26, 2019

KCG KLING
Consulting
Group, Inc.

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PN 19017-01

Mr. Matthew Badran, Development Associate
The Picerne Group
5000 Birch Street
Suite 600
Newport Beach, California 92660

Subject: **Supplemental Geotechnical Investigation, Proposed Residential Development, Koll Center Property Southwest Corner of Van Karman Avenue and Birch Street, Newport Beach, California**

Dear Mr. Badran:

In accordance with your request and authorization, Kling Consulting Group, Inc. (KCG) has conducted a geotechnical investigation for the proposed project. This report presents the results of our findings from surface and subsurface exploration, geotechnical analyses, and laboratory testing of selected soil samples. Additionally, this report summarizes our conclusions and recommendations relative to the proposed development.

Based on the results of our field exploration, laboratory testing and engineering analysis, it is our opinion that the site is geotechnically feasible for the proposed development, provided the recommendations presented herein are implemented during the design, grading, and construction of the project. This report is also subject to the limitations presented in Section 6.0 of our report and the ASFE (Associated Soil and Foundation Engineers) insert included in Appendix G.

We appreciate this opportunity to be of continued service and to work with you on this project. Should you have any questions regarding this report, please do not hesitate to call.

Respectfully,

KLING CONSULTING GROUP

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TABLE OF CONTENTS

TABLE OF CONTENTS
(Continued)

Attachments:

1.0 INTRODUCTION

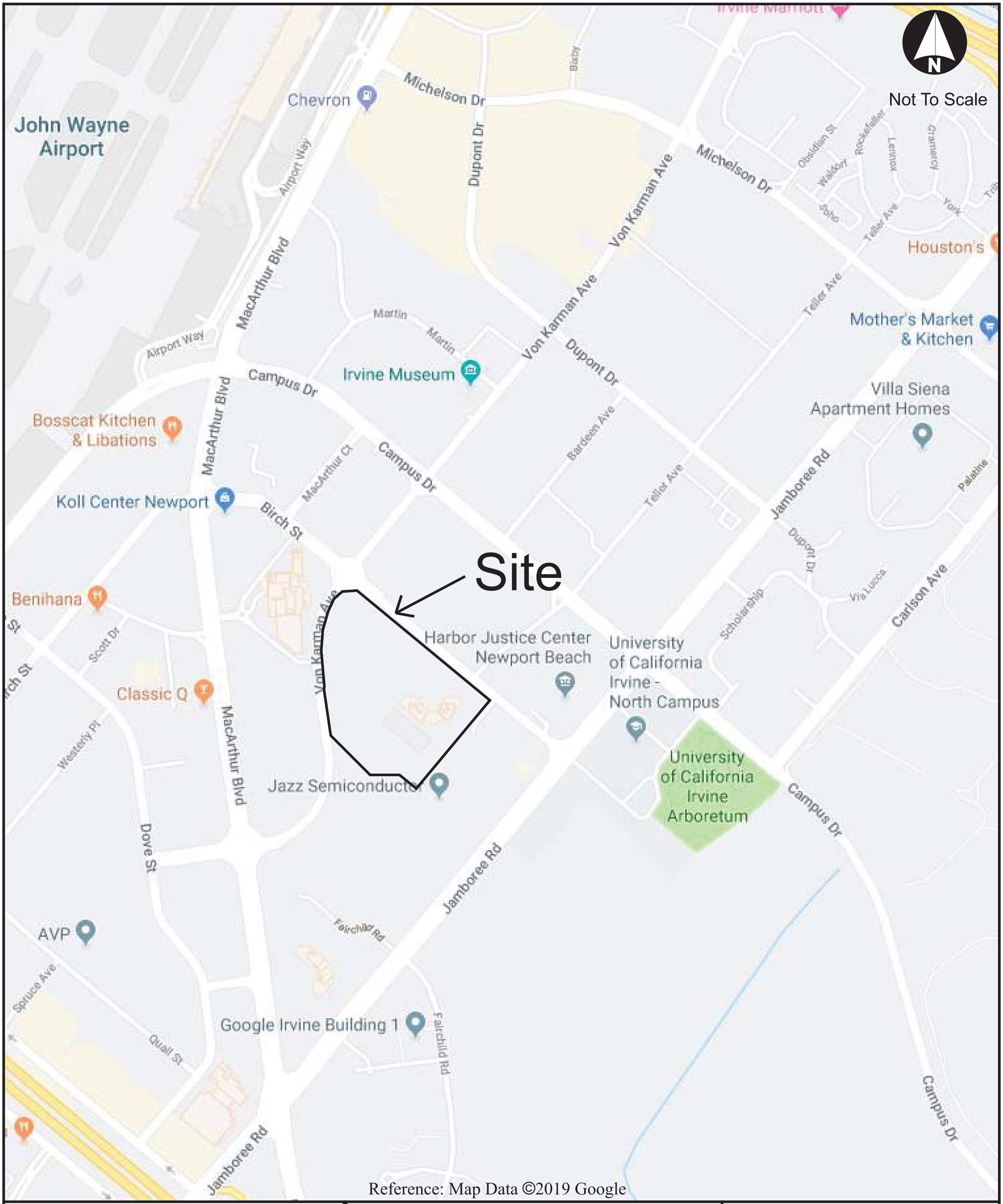
1.1 Purpose and Scope of Work

The purpose of this investigation was to further characterize and evaluate the subsurface soils in order to develop recommendations for the proposed development. The scope of work undertaken included the following tasks:

- Compilation and interpretation of available, previously documented geologic and geotechnical data for the property;
- Coordination with Underground Service Alert to mark and identify buried utilities prior to exploration;
- Field reconnaissance of the site and drilling and logging of four (4) hollow stem auger borings drilled to depths of approximately 51.5 feet below the existing ground surface within the proposed development areas. Two of the borings (B-1 and B-3) were utilized to install temporary piezometers to monitor and check for the presence of groundwater. Four (4) CPT Soundings were advanced to depths of 58 to 75 feet below the existing ground surface.
- Bulk and drive samples were obtained in the field and delivered to our laboratory for testing and evaluation;
- Laboratory testing was performed on selected soil samples. Laboratory testing included moisture/density determinations, expansion index, consolidation, direct shear;
- Engineering analysis to provide recommendations for conventional and alternative foundations to support the proposed structures including total and differential settlement, and seismic design parameters;
- Preparation of this geotechnical investigation report which presents a summary of our field exploration along with recommendations for the proposed development, seismic design parameters, general earthwork guidelines, foundations and pavements.

1.2 Site Location and Description

The subject property is located within the existing Koll Center Development. Based on a review of Google Earth® online imagery, the proposed development is situated within the existing paved parking and drive areas generally within the north-central and southern portions of the Koll Center. The property is bordered by Birch Street to the east, Von Karman to the west, and existing commercial/retail buildings and paved parking and drive areas to the north and south. **Figure 1** illustrates the geographic location of the project site.



Reference: Map Data ©2019 Google

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Site Location Map
Koll Center
Newport Beach, CA 92660

Figure:	**1**
PN:	**19017-01**
Date:	**June 2019**

1.3 Proposed Improvements

Based on our review of the information provided including a phasing plan exhibit by David Evans and Associates, Inc., dated February 21, 2017, Concept Plans by TCA Architects, dated April 26, 2019, and meetings with the design team on May 14, 2019 and May 22, 2019, it is our understanding that the proposed development will include one building for 351 apartment residences and a parking structure. The site encompasses approximately 4 acres. It is our understanding that for the residential buildings it is currently proposed to include four levels above one level of ground floor parking, with two (2) levels of subterranean for parking. It is our understanding that one level of subterranean is also being considered. The building will be podium Type V construction. The parking structure currently proposed comprises 5 levels above ground with two levels of subterranean. It is our understanding that at grade parking and one level of subterranean are also being considered for the parking structure.

Grading plans were not available; however, grading is anticipated to include cut excavations of approximately 12 feet below existing grades to achieve the proposed grades for one level of subterranean and 24 feet below existing grades to achieve the proposed grades for two levels of subterranean. No foundation plans were available. Foundations are assumed to be typical for the type of building construction proposed.

2.0 GEOLOGIC CONDITIONS

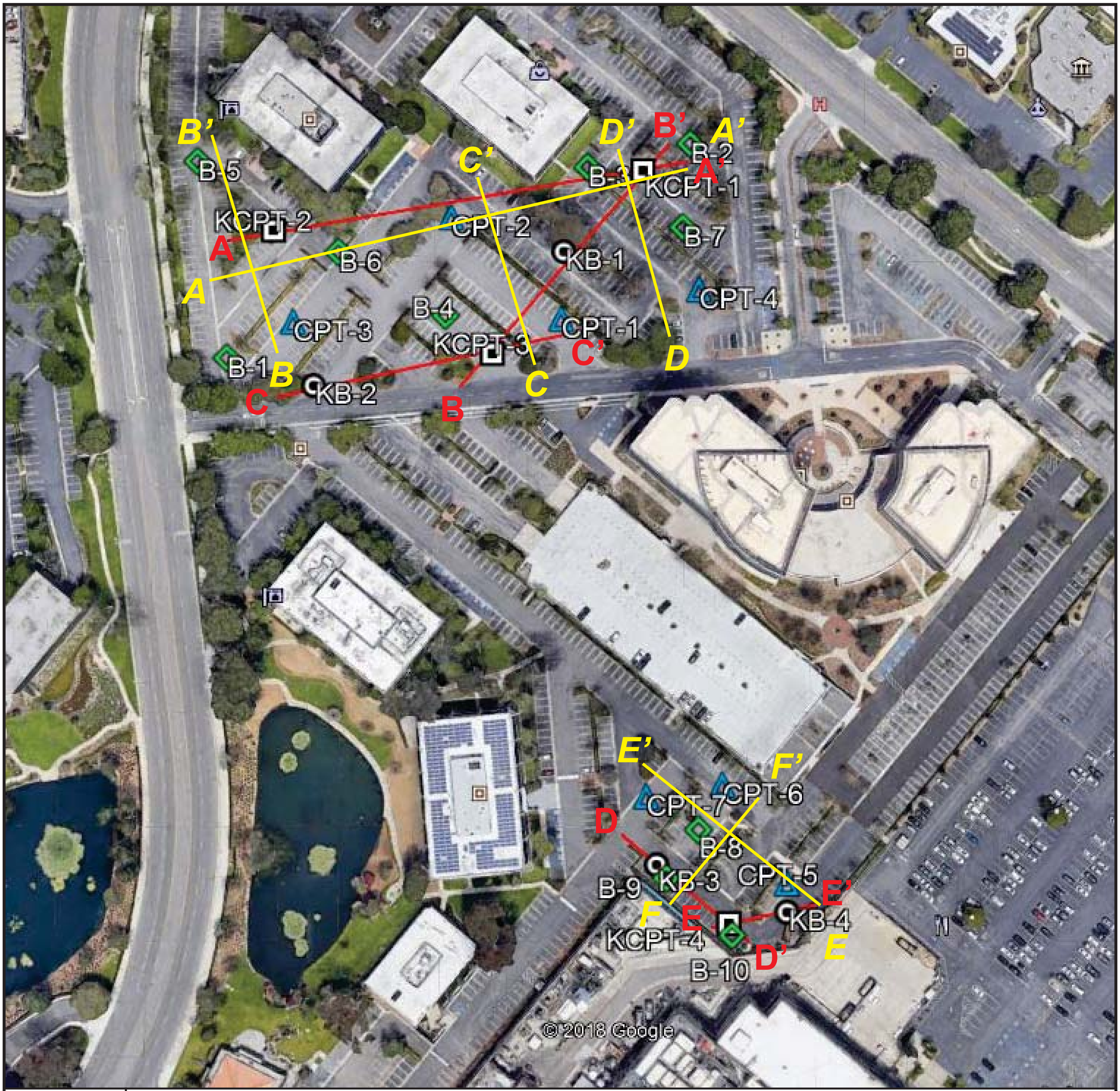
2.1 Field Exploration

On May 30 and May 31, 2019, four (4) hollow stem auger borings were drilled to depths of approximately 50 feet and Four (4) CPT Soundings were advanced to depths of 58 to 75 feet below the existing ground surface within the proposed development areas. Two of the borings (B-1 and B-3) were utilized to install temporary piezometers to monitor and check groundwater levels. Bulk and drive samples were obtained in the field from the hollow stem auger borings and delivered to our laboratory for testing and evaluation. The approximate locations of the borings are illustrated on **Figure 2**. Full descriptions of the materials encountered are presented in the boring logs in **Appendix B.**

2.2 Regional Geologic Setting and Site Specific Geology

The subject project area is situated within the southwestern portion of the Tustin Quadrangle in the city of Newport Beach, California. The sedimentary materials encountered in this portion of the quadrangle are mapped as Quaternary aged Old paralic deposits ($Qopf_a$).

Locally, the Old paralic deposit materials encountered during our subsurface exploration consisted of silty sands and sands, and sandy to silty clays which were generally medium dense to very dense and stiff and moist to wet.



N

Scale
1"= 160'

Legend

⊙ KB-1	Approximate location of KCG boring (2019)	△ CPT-1	Approximate location of EEI CPT sounding (2016)
▣ KCPT-1	Approximate location of KCG CPT sounding (2019)	A —— A'	Approximate location of KCG cross section (2019)
◇ B-1	Approximate location of EEI boring (2016)	B —— B'	Approximate location of EEI cross section (2016)

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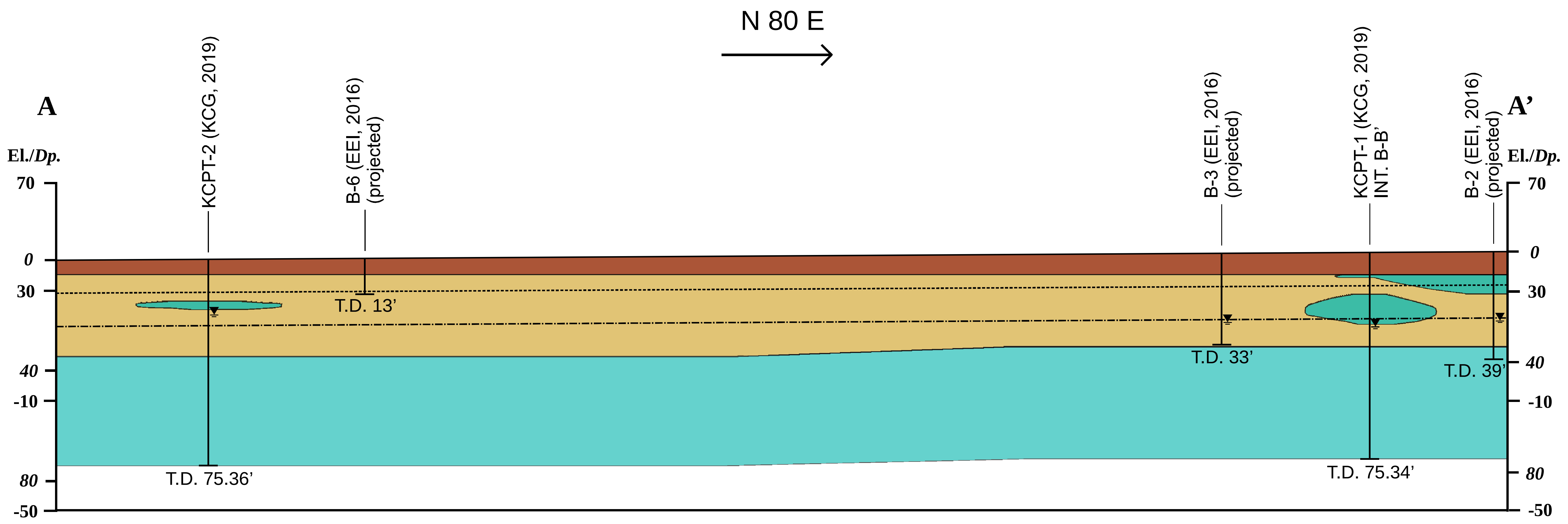
Geotechnical Map
Koll Center
@ Birch St. and Von Karman Ave.
Newport Beach, CA 92660

Figure:	**2**
PN:	**19017-01**
Date:	**June 2019**



Scale: 1” = 40’

Legend

🟫 Artificial Fill

🟦 Clay and Silty Clay

🟨 Sand and Silty Sand

▼ Perched groundwater

-------- One-level subterranean (Depth of 12 Feet)

-·-·-·-· Two-level subterranean (Depth of 24 Feet)

Reference: Kehoe Testing and Engineering, CPT-1 & CPT-2, Kling Consulting Group/Koll Center.
EEI, Supplemental Geotechnical Evaluation and Update, The Koll Center Residences, boring logs B-2, B-3 & B-6, 10/31/2016.

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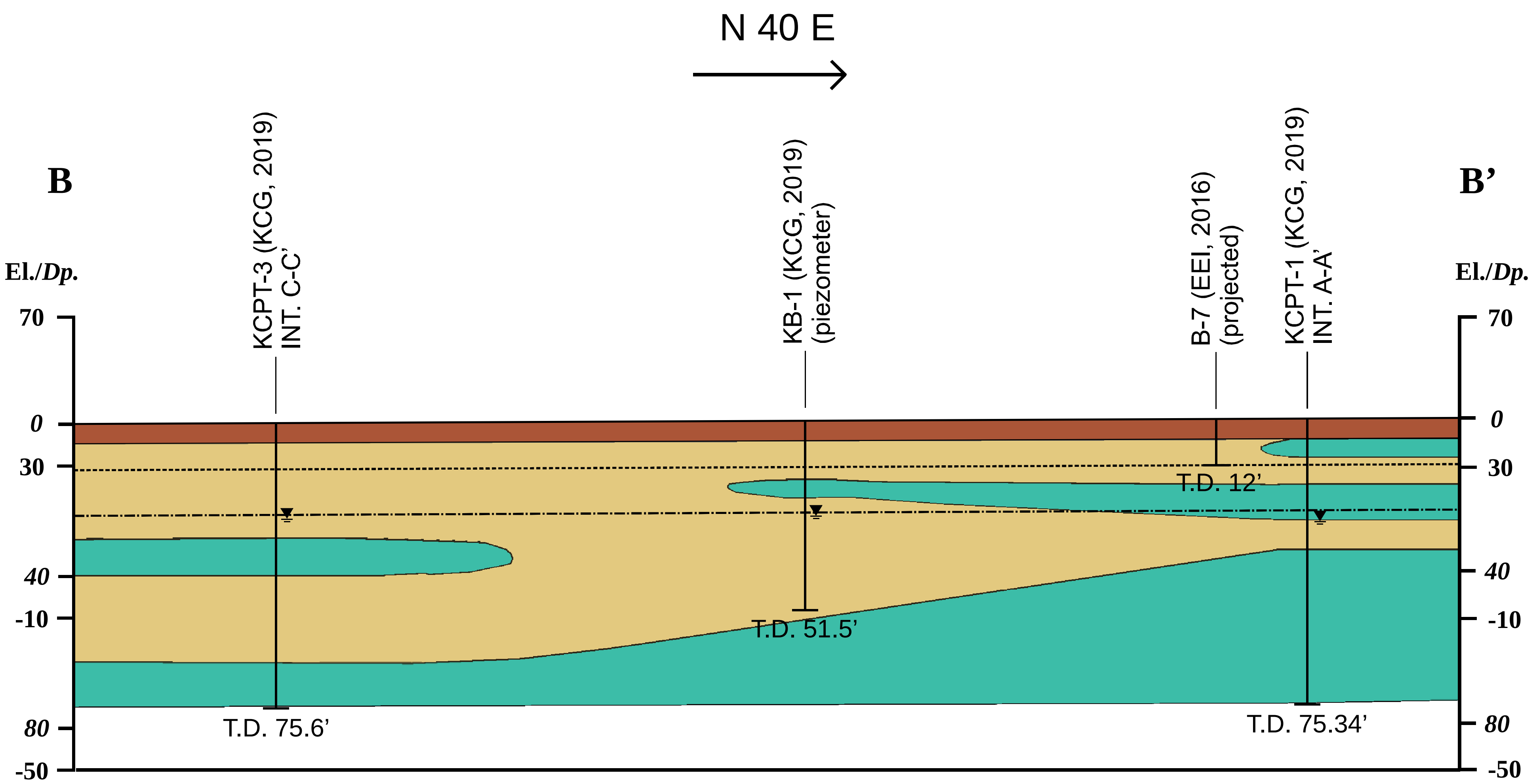
Geologic Cross Section A-A’
Koll Center
@ Birch St. and Von Karman Ave.
Newport, CA 92660

Figure:	**3**
PN:	**19017-01**
Date:	**June 2019**



Scale: 1” = 40’

Legend

🟫	Artificial Fill
🟦	Clay and Silty Clay
🟨	Sand and Silty Sand
▼	Perched groundwater
-------	One-level subterranean (Depth of 12 Feet)
-·-·-·-	Two-level subterranean (Depth of 24 Feet)

Reference: Kehoe Testing and Engineering, CPT-1 & CPT-3, Kling Consulting Group/Koll Center.
EEI, Supplemental Geotechnical Evaluation and Update, The Koll Center Residences, boring log B-7, 10/31/2016.

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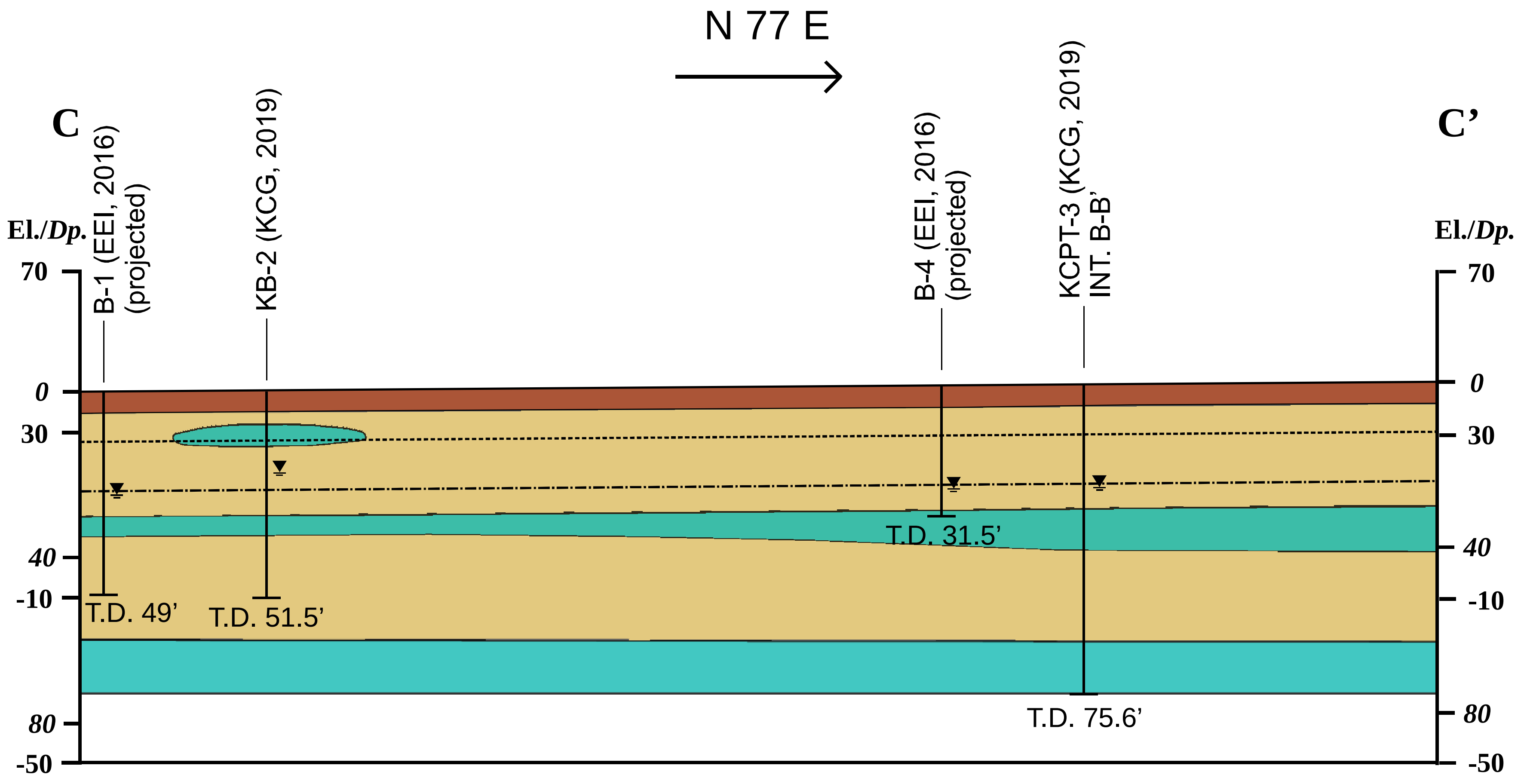
Geologic Cross Section B-B’
Koll Center
@ Birch St. and Von Karman Ave.
Newport, CA 92660

Figure:	**4**
PN:	**19017-01**
Date:	**June 2019**



Scale: 1” = 40’

Legend

🟫 Artificial Fill

🟦 Clay and Silty Clay

🟨 Sand and Silty Sand

▼ Perched groundwater

-------------------- One-level subterranean (Depth of 12 Feet)

-·-·-·-·-·-·- Two-level subterranean (Depth of 24 Feet)

Reference: Kehoe Testing and Engineering, CPT-3, Kling Consulting Group/Koll Center.
EEI, Supplemental Geotechnical Evaluation and Update, The Koll Center Residences, boring logs B-1 & B-4, 10/31/2016.

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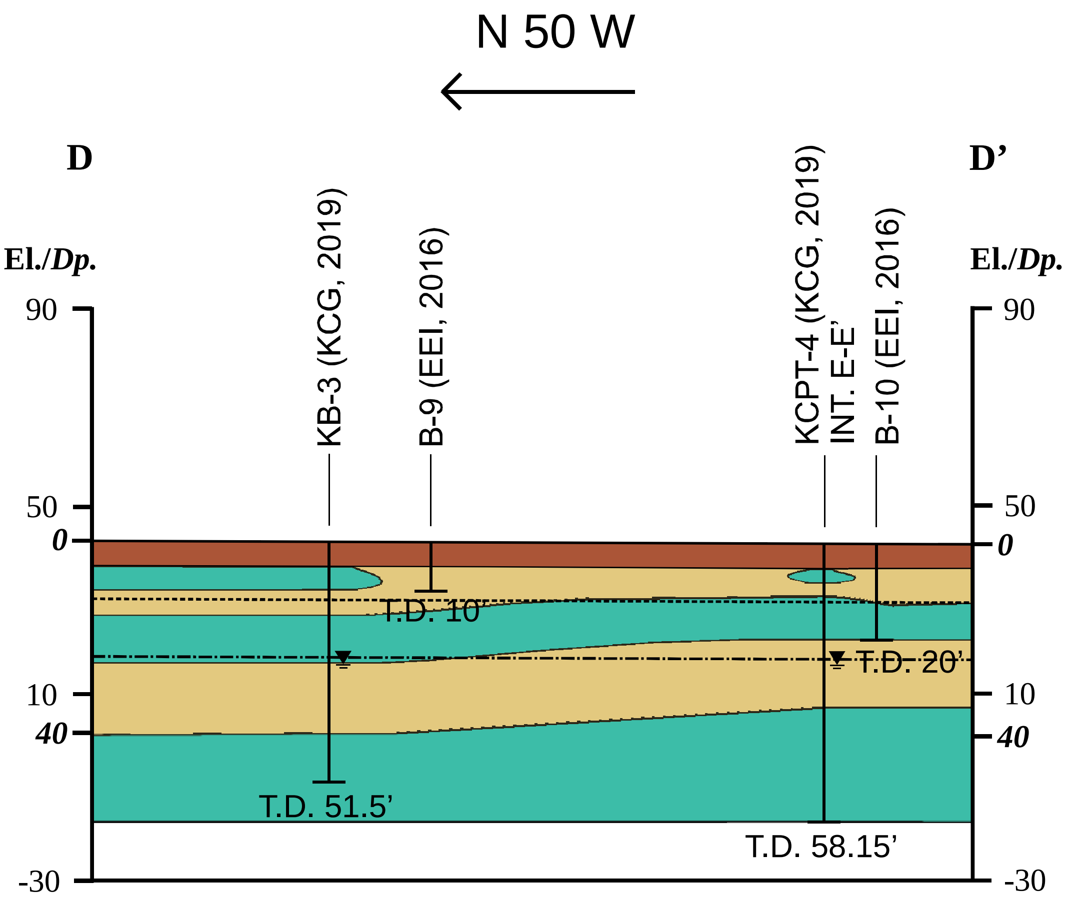
Geologic Cross Section C-C’
Koll Center
@ Birch St. and Von Karman Ave.
Newport, CA 92660

Figure: 5
PN: 19017-01
Date: June 2019



Scale: 1" = 40'

Legend

🟫	Artificial Fill
🟦	Clay and Silty Clay
🟨	Sand and Silty Sand
▼	Perched groundwater
-------------------	One-level subterranean (Depth of 12 Feet)
-·-·-·-·-·-·-	Two-level subterranean (Depth of 24 Feet)

Reference: Kehoe Testing and Engineering, CPT-4, Kling Consulting Group/Koll Center.
EEI, Supplemental Geotechnical Evaluation and Update, The Koll Center Residences, boring logs B-9 & B-10, 10/31/2016.

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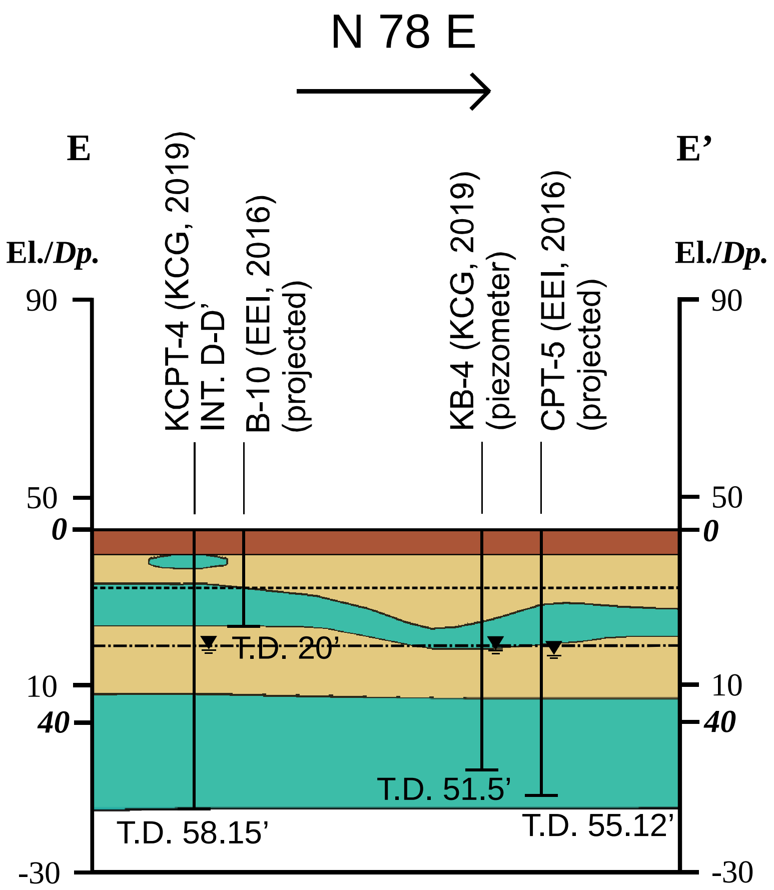
Geologic Cross Section D-D'
Koll Center
@ Birch St. and Von Karman Ave.
Newport Beach, CA 92660

Figure: **6**
PN: **19017-01**
Date: **June 2019**



Scale: 1” = 40’

Legend

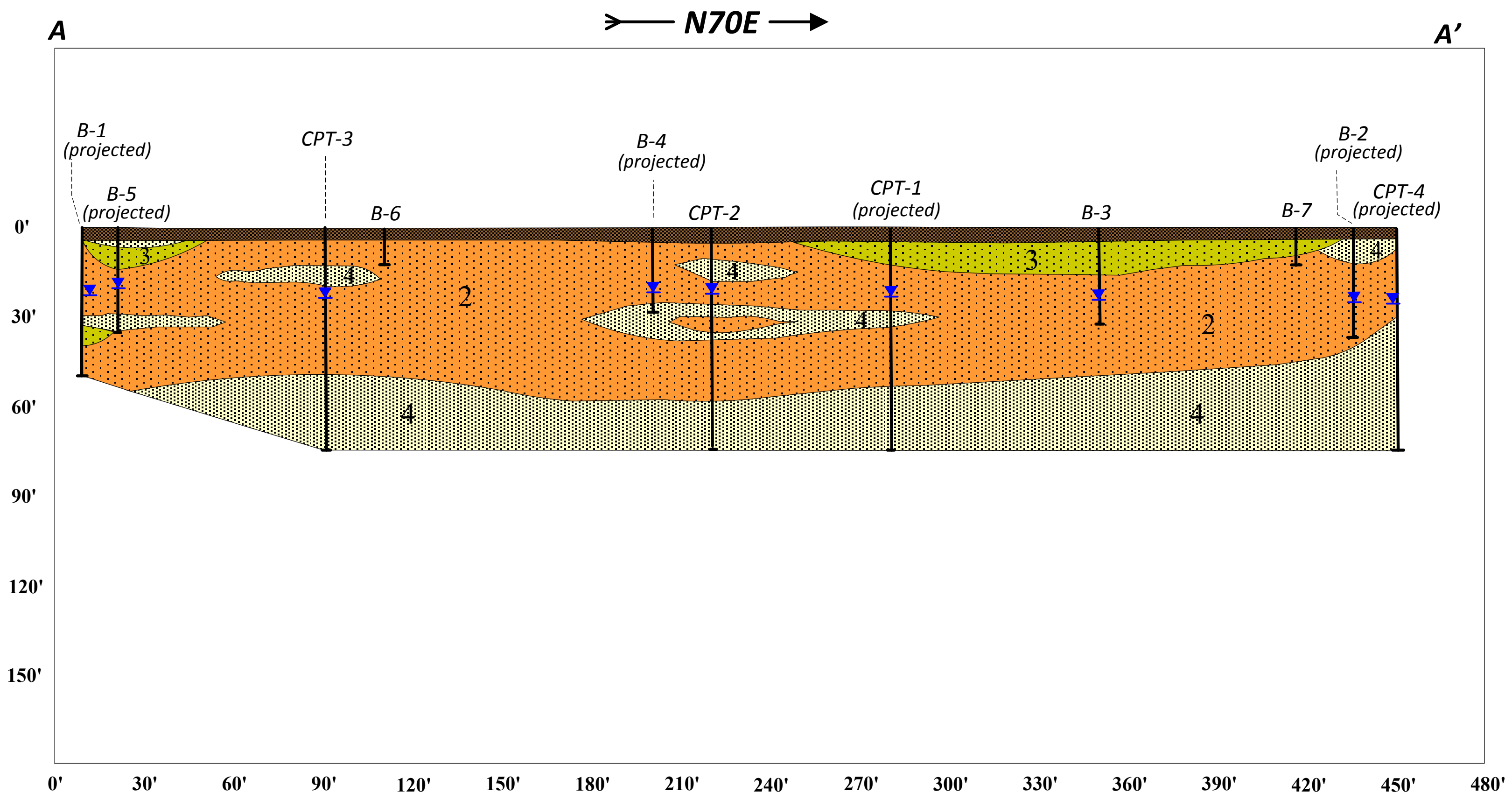
🟫	Artificial Fill
🟦	Clay and Silty Clay
🟨	Sand and Silty Sand
▼	Perched groundwater
-------------------	One-level subterranean (Depth of 12 Feet)
-·-·-·-·-·-	Two-level subterranean (Depth of 24 Feet)

Reference: Kehoe Testing and Engineering, CPT-4, Kling Consulting Group/Koll Center.
EEI, Supplemental Geotechnical Evaluation and Update, The Koll Center Residences, boring log B-10 & CPT-5, 10/31/2016.

KCG KLING Consulting Group, Inc.

Geologic Cross Section E-E’
Koll Center
@ Birch St. and Von Karman Ave.
Newport Beach, CA 92660

Figure:	**7**
PN:	**19017-01**
Date:	**June 2019**



LEGEND (Detailed description in Section 6.0 of this report)

1 Fill: Sands, Silts Clays and Gravels

2 Older Alluvial Fan Deposits: SAND, with trace Gravel, Silt or Clay

3 Older Alluvial Fan Deposits: SILTY-SAND and SANDY-CLAY

4 Older Alluvial Fan Deposits: CLAYS and SILTS

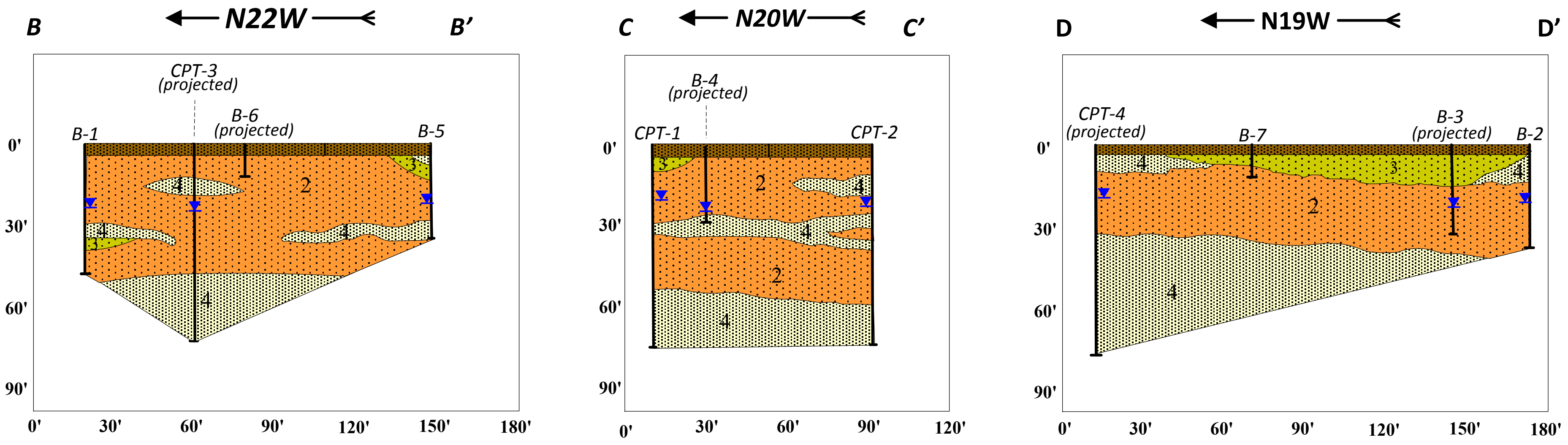
▼ Groundwater/seepage encountered

SCALE
1" = 60ft

GEOLOGIC CROSS-SECTION A-A'
Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI
Geotechnical & Environmental Solutions

FIGURE 4a



LEGEND (Detailed description in Section 6.0 of this report)

1 Fill: Sands, Silts Clays and Gravels

2 Older Alluvial Fan Deposits: SAND, with trace Gravel, Silt or Clay

3 Older Alluvial Fan Deposits: SILTY-SAND and SANDY-CLAY

4 Older Alluvial Fan Deposits: CLAYS and SILTS

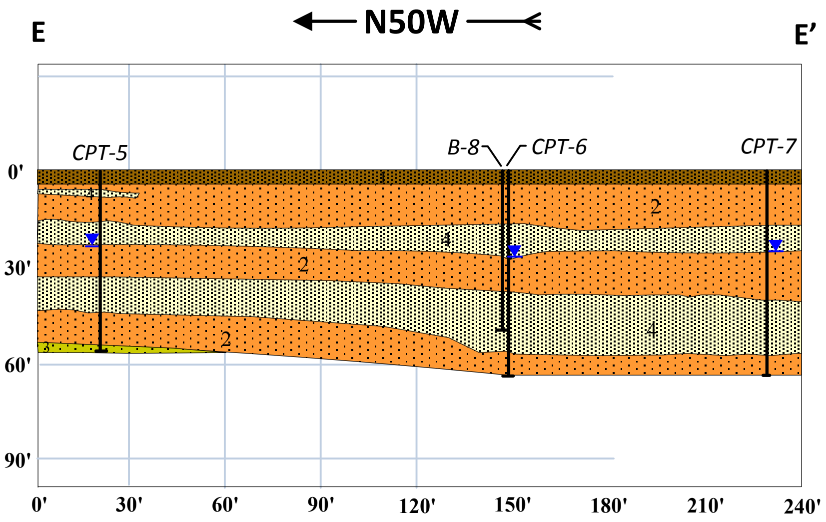
▼ Groundwater/seepage encountered

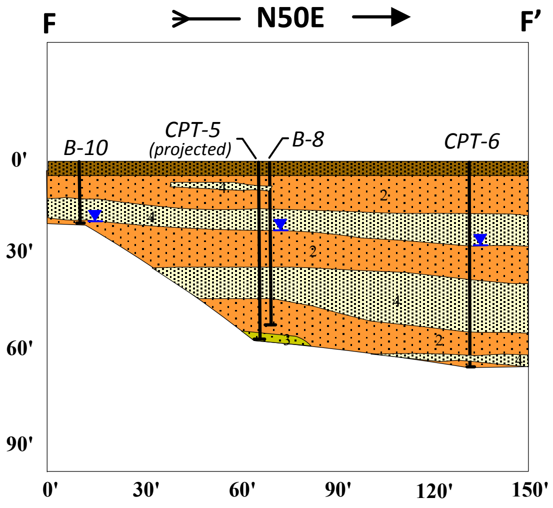
SCALE
1" = 60ft

GEOLOGIC CROSS-SECTIONS B-B'.C-C'.D-D'
Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI
Geotechnical & Environmental Solutions

FIGURE 4b





LEGEND (Detailed description in Section 6.0 of this report)

1	Fill: Sands, Silts Clays and Gravels
2	Older Alluvial Fan Deposits: SAND, with trace Gravel, Silt or Clay
3	Older Alluvial Fan Deposits: SILTY-SAND and SANDY-CLAY
4	Older Alluvial Fan Deposits: CLAYS and SILTS
▼	Groundwater/seepage encountered

SCALE
1" = 60ft

GEOLOGIC CROSS-SECTIONS E-E’,F-F’

Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI
Geotechnical & Environmental Solutions

FIGURE 4c

2.3 Groundwater

According to the California Geologic Survey (CGS) Seismic Hazard Zone Report for the Tustin 7.5-Minute Quadrangle (Reference 5), the reported and mapped historical high groundwater level is approximately 10 feet below grades in the vicinity of the site. A perched groundwater condition was encountered in our exploratory borings at depths of between 20 and 25 feet below the existing ground surface during our subsurface exploration at the time of drilling. Temporary piezometers were installed in two of our borings to monitor the presence of groundwater. Groundwater levels were measured after our drilling was completed in the two piezometers on June 6, June 12, and June 24, 2019. The groundwater level was measured at between 20.0 feet and 20.6 feet in KB-1 and 24.1 and 24.8 feet in KB-4. These perched zones are considered typical in the site vicinity where water accumulates in relatively permeable sand layers that are situated above or below relatively impermeable clay layers that act as confining layers, and does not, in our opinion, necessarily represent the static groundwater level which may be closer to 50 feet below existing grades. However, these perched zones of groundwater will potentially impact the site construction and should be considered and incorporated into the project design. We would not anticipate ground water to have a significant impact during construction for one level of subterranean founded at a depth of 12 feet below existing grades; however, groundwater is anticipated to have a significant impact for two levels of subterranean founded at a depth of 24 feet below existing grades. It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation, and other factors that may not be evident at the time of our subsurface exploration.

2.4 California Building Code Seismic Design Parameters

A geologic hazard likely to affect the project is ground-shaking as a result of movement along an active fault zone in the vicinity of the subject property. Presented below are the site seismic parameters utilizing geologic, seismic and geotechnical data gathered for the site. All structures should be designed for earthquake induced strong ground motions in accordance with the 2016 CBC procedures utilizing the following parameters:

Seismic Design Parameters

Site Class (Soil Profile)	D
Latitude	33.6659
Longitude	-117.8603
Short Period Spectral Acceleration, S_s:	1.577
1-Second Period Spectral Acceleration, S_1:	0.578
Site Coefficient, F_a:	1.00
Site Coefficient, F_v:	1.50
Maximum Considered Earthquake Spectral Response Acceleration, S_{MS}:	1.577
Maximum Considered Earthquake Spectral Response Acceleration, S_{M1}:	0.867
Design Spectral Response Acceleration, S_{DS}:	1.051
Design Spectral Response Acceleration, S_{D1}:	0.578
Site modified peak ground acceleration PGA_M	0.615
Seismic Design Category	D

2.5 Faulting and Surface Rupture

The subject property is located within an area of California known to contain a number of active and potentially active faults. The property is not located within a State of California Earthquake Fault Zone (Jennings and Bryant, 2010; Hart and Bryant, 1997). No active faults are known to cross the site. The distances of the closest major active faults from the property were generated from information provided on the USGS online resource (USGS, 2008, National Seismic Hazards Maps, Source Parameters,), with the approximate center of the site being at latitude 33.6659°N and longitude 117.8603°W. The San Joaquin Hills Fault is located approximately 1.7 miles from the site, the Newport Inglewood Fault Zone approximately 5.5 miles from the site and the Newport Inglewood (Offshore segment) located approximately 6 miles from the site. It is our opinion that the potential for surface fault rupture at the property is low.

2.6 Liquefaction Potential

Based on a review of the California Geological Survey (CGS), Seismic Hazard Zones Report Map for the Tustin Quadrangle (Reference 5), the historic groundwater is reported to be approximately 10 feet from existing grades in the vicinity of the property. Our liquefaction analysis conservatively incorporates the historic high groundwater depth of 10 feet. Our geotechnical evaluation indicates that localized and isolated sandy layers within the Old paralic deposits that underlie the site are susceptible to relatively minor amounts of liquefaction as a result of a design-level earthquake along a nearby fault and incorporating the historical high groundwater level of 10 feet below existing grades. We estimate that portions of the site could experience settlements that range from approximately less than 0.02 inches to approximately 2 inches of total seismic-induced settlement. Overall seismic induced liquefaction settlement would be reduced with removal of materials for the subterranean excavations. The portions of the site that appear to be susceptible to liquefaction and the magnitudes of seismic–induced settlement described above appear to be somewhat localized. A seismic hazard zone for the subject site area has not been established by the state of California.

2.6.1 Liquefaction Settlement Analysis

The total earthquake-induced liquefaction settlement potential was calculated using the CLiq computer program (GeoLogismiki, 2015) – 1998 NCEER analysis method, and the CPT data from KCPT-1 through KCPT-4. Our evaluation was based on the site class and adjusted peak ground acceleration of 0.615g, as presented in the **Seismic Design Parameters Table** above, and a probabilistic 2,475 year modal magnitude of 6.96. Our analysis indicates the estimated settlement due to earthquake-induced liquefaction settlement is approximately 0.02 inches to approximately 2.32 inches. We estimate that for the apartment site, liquefaction settlement would range from 0.60 to 2.32 inches; while the parking structure would have from 0.02 inches to 0.47 inches. Differential settlements are estimated to be approximately 0.3 inches for the parking structure over a distance of 50 feet, and about 1.5-inches over a distance of 50 feet for the residential apartment site. Overall seismic induced liquefaction settlement would be reduced with removal of materials for the subterranean excavations. A summary of estimated settlements based on number of subterranean levels is provided in the Table below. The materials underlying the site are overall relatively dense and stiff and the overall seismic settlement potential is anticipated to be localized and considered relatively minor. The results of our analysis are included herein in **Appendix C – Seismic Settlement Analysis.**

Summary of Estimated Earthquake Induced Liquefaction Settlement

Type of Building/Structure	Number of Subterranean Levels	Estimated Vertical Settlement (inches)	Differential Settlement Over a Distance of 50 Feet (inches)
Apartment	At Grade-No Excavation	0.6 to 2.32	1.5
	1	0.86 to 1.98	1.3
	2	0.89 to 1.69	1.3
Parking Structure	At Grade-No Excavation	0.02 to 0.47	0.3
	1	0.04	N/A
	2	0.04	N/A

2.6.2 Lateral Spreading

The potential for liquefaction induced lateral spreading is considered to be negligible. Additionally, the ground surface is generally relatively flat with no free slopes adjacent to the project.

3.0 GEOTECHNICAL ENGINEERING CONSIDERATIONS

3.1 Expansive Soil Characteristics

Based on laboratory testing performed by EEI, expansion index testing was performed on one sample of clayey materials of the upper soils which indicated a high expansion potential with an Expansion Index (EI) of 107. Overall, we anticipate that the onsite soils are variable and are considered to possess a very low to high expansion potential (Expansion Index [EI]) of 20 to 130) as defined by CBC, 2016.

3.2 Soluble Sulfate and Chloride Exposure

Laboratory test results reported in a previous geotechnical evaluation on the subject site (EEI, 2016) indicate that soils contain negligible sulfate and chloride concentrations which do not appear to pose a significant threat to concrete durability. Type II cement can be used in concrete elements that will be in contact with the upper soils at the site.

3.3 Corrosion Potential

Soil corrosivity test results reported in a previous geotechnical evaluation on the subject site (EEI, 2016) indicate that earth materials collected within the upper 20 feet of the ground surface, are highly corrosive to ferrous metals.

3.4 Earthwork Shrinkage and Subsidence

Based on our field and laboratory density tests and observations, the following estimate of shrinkage and subsidence factors of the upper fill materials to be utilized as on-site compacted fill soils are presented for design consideration.

Shrinkage Factor	-	10% to 15% Upper soils
Shrinkage Factor	-	5% to 10% subterranean levels
Subsidence Factor	-	0.10 feet

Although the above values are only approximate, they represent our best estimate of shrinkage and lost yardage which would likely occur during re-grading.

4.0 CONCLUSIONS

The following conclusions are based upon our analysis and review of geotechnical data. It is our opinion that the proposed site improvements are geotechnically feasible, provided that the recommendations of this report are followed during future site development and design.

- For conventional spread footings that are to be used to support the proposed apartment building, remedial earthwork is recommended consisting of the removal of the upper potentially compressible soils to provide a uniform fill beneath the foundations and reduce static and differential settlement. Recommendations for over excavation below proposed building foundations are discussed in Section 5.2;

- Should a reinforced mat slab be used to support the proposed apartment building and parking structure with at least one subterranean level, remedial earthwork is reduced to reprocessing of the subgrade soils. Recommendations are discussed in Section 5.2;

- In our opinion, shallow conventional or mat slab foundation systems for a five level (elevated) at grade parking structure would likely not be feasible due to the large anticipated structural loading. Therefore, a pile foundation system utilizing replacement piles such as cast-in-place continuous flight auger (CFA) piles can be considered to support the option of an at grade parking structure;

- No active faults are known to exist at the site and the risk of surface fault rupture is considered to be very low. However, the project site lies within a region of historical seismicity and will likely be subject to seismic shaking in the future;

- Our geotechnical evaluation indicates that sandy layers within the Paralic Deposits that underlie the site are susceptible to liquefaction as a result of a design-level earthquake incorporating the historical high groundwater level of 10 feet below existing grades (CGS/CDMG, 1998). The estimated settlements are in the range of 0.02 inches to approximately 2-inches, and appear to be limited to isolated and localized relatively thin zones generally between 12 and 49 feet below existing grades at the site during seismic events. We estimate that for the apartment site, liquefaction settlement would range from 0.60 to 2.3 inches; while the parking structure would range from 0.02 inches to 0.47 inches. Overall seismic induced liquefaction settlement would be reduced with removal of the upper materials for the subterranean excavations as summarized in **Section 2.6** herein. More detailed liquefaction analysis including settlement contours and CPT logs are presented in **Appendix C**. The materials underlying the site are overall relatively dense and stiff. The potential for lateral spreading to occur within the site is unlikely due to the lack of a shallow static groundwater table and the overall dense and stiff nature of the underlying on-site soils;

- It is KCG's professional opinion that liquefaction-induced ground displacements are relatively minor overall and can be mitigated through the use of a reinforced concrete structural mat foundation system for the support of the proposed apartment building and parking structure.

- Soils underlying the subject site are not considered to be susceptible to hydrocollapse;

- A perched groundwater condition was encountered in our exploratory borings at depths of between 20 and 25 feet below the existing ground surface during our subsurface exploration at the time of drilling. Although these perched zones are considered typical in the site vicinity where water accumulates in relatively permeable sand layers situated above and/or below confining clay layers, it does not, in our opinion, represent the static groundwater level which is more likely closer to 50 feet below existing grades. However, this perched zone of groundwater will potentially impact and pose a problem for the site construction for proposed two subterranean levels at or near a depth of 24 feet below existing grades. Temporary dewatering or other measures should be considered and groundwater levels incorporated into the project design for the proposed development;

- The underlying soils are variable and anticipated to possess a very low to high expansion potential.

- Based on previous laboratory soil test results (EEI, 2016), the on-site soils indicated a soluble sulfate content that is considered "Class S0" which is negligible to sulfate exposure as per the 2014 ACI Concrete Manual of Practice as indicated in Section 19, Table 3.1.1;

- Laboratory testing previously performed (EEI, 2016) on soils within the subject site has indicated the soil is likely "highly corrosive" to ferrous metals when the soil is saturated, as per Caltrans guidelines. A qualified corrosion consultant should be retained to provide more specific recommendations regarding corrosion protection.

5.0 RECOMMENDATIONS

Recommendations presented herein are preliminary and subject to revision if new information becomes available. In order to provide uniform soil support for the proposed structures, we recommend that the upper soils be removed and recompacted in those areas to receive buildings or other settlement-sensitive improvements, where not removed by planned excavations. The depth of removals will be dependent upon the type of foundation system selected and number of subterranean levels.

5.1 Earthwork Specifications

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix F, unless specifically revised or amended below. Grading should also conform to all applicable governing agency requirements. Prior to commencement of grading operations, all vegetation, organic topsoil, and man-made structures (i.e., tanks, pipes, fences, etc.) should be cleared and disposed of off-site. Any undocumented fill or backfill encountered should be removed and recompacted. All areas receiving fill should be scarified to 6 inches and/or over-excavated, moisture-conditioned to between optimum moisture and two to four percent above optimum moisture content, and re-compacted to a minimum of 90 percent relative compaction as determined by ASTM D1557. Soil material excavated from the site should be adequate for re-use as compacted fill provided it is free of trash, vegetation, and other deleterious material. All earthwork and grading operations should be performed under the observation and testing of the geotechnical consultant of record.

5.2 Remedial Earthwork and Over-Excavation

5.2.1 Proposed Residential Apartment Building

<u>Foundation Design Option: Conventional Foundation - One to two levels Subterranean</u>

In order to account for soil variability and expansion potential, reduce the potential for settlement and differential settlement, and maintain a uniform fill blanket beneath the bottom of the foundations, we recommend the subterranean level pad area be over- excavated a minimum of five (5) feet below the subterranean level finish grade elevations, or a minimum of two (2) feet below proposed foundations, *whichever is deeper*. The over-excavation should be extended laterally a minimum of five (5) feet beyond the proposed building footprint and/or foundations or equal to the depth of the over-excavation, whichever is deeper, where practical. Footings should be underlain by a minimum of two feet of engineered fill below the bottom of footings.

<u>Foundation Design Option: Mat Slab Foundation - One to two levels Subterranean</u>

For Mat slab foundation systems, the removals can be reduced to re-processing (i.e. 12-inch scarification and recompaction) and proof rolling of the subgrade soils exposed at the subterranean level.

5.2.2 Proposed Parking Structure

Foundation Design Option: Mat Slab Foundation -One to two levels Subterranean

For Mat slab foundation systems, the removals can be reduced to processing (i.e. 12-inch scarification and recompaction) and proof rolling of the subgrade soils exposed at the subterranean level.

5.2.3 Proposed Pavement and Flatwork Areas

In areas outside of proposed structural areas that would support pavement and flatwork, the exposed subgrade soils should be processed and re-compacted to a depth of 12-inches. If soils are disturbed during removal of existing improvements, the disturbed soil should be removed and replaced with compacted fill. After removals are made, exposed soils should be scarified to a depth of 6-inches, brought to near optimum moisture content, and re-compacted.

5.3 Processing of Natural Soils and Fill Placement

Processing of in-place soils exposed after clearing, grubbing and removal of unsuitable material and prior to placing fill should include the following items of work:

Scarification of the materials exposed after remedial removals should be accomplished to a depth of at least 6 inches or as dictated by actual soil conditions encountered;

The scarified soils should be brought to 2 to 4 percent above optimum moisture content by watering or drying, as required;

Compaction of the processed soils to at least 90 percent of the laboratory maximum dry density, prior to placing fill.

Fill should be placed in relatively thin (6 to 8-inch) uniform lifts; moisture conditioned to 2 to 4 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM D 1557. Actual lift thickness would depend on soil type and compaction equipment being used.

5.4 Foundation Design

All foundation criteria are considered minimum requirements that may be superseded by more stringent requirements from the architect, structural engineer, or governing agencies. Recommended geotechnical design parameters are being provided for conventional spread footings and reinforced mat slab foundation systems for the residential building; and drilled auger-cast piles for the parking structure at grade, and mat slab for the proposed parking structure with subterranean level(s).

Proposed Residential Apartment Building

Subterranean-Conventional Shallow Foundations

The following geotechnical design parameters are provided for the design of proposed conventional foundations for the proposed 4-story apartment building, with one to two levels of subterranean. The following geotechnical design parameters are provided for the design of proposed foundations for the proposed residential structure with one subterranean level, a podium level, and four-levels above. The proposed building may be supported by square pad footings utilizing a maximum allowable bearing pressure of 3000 pounds per square foot. The maximum width of the continuous footings should be no more than 8-feet with a minimum depth of 3-feet below the lowest adjacent grade (which includes the top of the slab on grade). A coefficient of friction of 0.40 may be used, along with a passive lateral resistance of 250 pounds per square foot per foot of embedment. Subgrade soil would likely require recompaction or processing for up to three feet.

If normal code requirements are used for seismic design, the allowable bearing value and coefficient of friction may be increased by 1/3 for short duration loads, such as the effect of wind or seismic forces.

If any utility lines are within a 1:1 (horizontal: vertical) projection from the bottom of a footing, they may be within the influence zone of the proposed footing load. If this condition exists, the proposed footing should be deepened so that the utility is outside the zone of influence; the utility line could also be relocated or encased with concrete slurry. These conditions should be evaluated on a case by case basis.

Subterranean -Mat Slab

A rigid mat foundation may be used for the support of the buildings at the site, provided the mat foundation is bearing within soils that are properly compacted and proof rolled in accordance with the recommendations contained herein. When properly designed and constructed, a structural mat foundation system can be expected to support high structural loads and provide relatively uniform settlement across a structure, while being able to "bridge" over local areas of dynamic and anticipated static settlement. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineers design. For designing a mat foundation, we recommend using a modulus of subgrade reaction from the table below or based on the appropriate graph of modulus versus settlement or allowable pressure graphs included within Appendix E. The table below presents a summary of modulus of subgrade reaction values versus allowable pressure for ½-inch of settlement or less.

Table of modulus of subgrade reaction (PCI) v. bearing pressure for 0.50-inch deflection

Type of Building/Structure	Number of Subterranean Levels	Net allowable Bearing Pressure (psf)	Subgrade Reaction for Total Settlement less than 0.5" (pci)
Apartment	1	2690	68
	2	4250	107
Parking Structure	1	2625	66
	2	3945	99

5.4.1 Parking Structure

At Grade- Pile Foundations

In our opinion, shallow conventional or mat slab foundation systems for a five level (elevated) at grade parking structure would likely not be feasible due to the large anticipated structural loading. Therefore, a pile foundation system utilizing replacement piles such as cast-in-place continuous flight auger (CFA) piles is presented for consideration to support the parking structure at grade. Refer to Figure A-1 in Appendix E for allowable capacities of auger-cast piles of various diameters versus depth. Capacities versus depth for shaft diameters of 24-inch to 42-inches are presented. The graphs indicate maximum capacities generally occur at depths between 50 feet and 60 feet.

Subterranean- Mat Foundation

A rigid mat foundation may be used for the support of a proposed one subterranean level parking structure at the site, provided the mat foundation is bearing within soils that are properly compacted and proof rolled in accordance with the recommendations contained herein. When properly designed and constructed, a structural mat foundation system can be expected to support high structural loads and provide relatively uniform settlement across a structure, while being able to “bridge” over local areas of dynamic and anticipated static settlement. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineers design.

For designing a mat foundation, we recommend using a modulus of subgrade reaction from the table above or from the appropriate graph presented in Appendix E.

5.5 Settlement

Static settlement of proposed foundations is not expected to exceed one (1) inch for total and one half (0.5) inch differential over 50 horizontal feet, provided the minimum remedial earthwork recommendations provided in **Section 5.2** is performed for the specific foundation system type. For preliminary design purposes, seismic induced liquefaction settlement for the apartment site is 0.6 to 2.3 inches, while the parking structure would range from 0.02 inches to 0.47 inches. Differential settlements are estimated to be approximately 0.3 inches for the parking structure over a distance of 50 feet, and about 1.5-inches over a distance of 50 feet for the residential apartment site. Overall seismic induced liquefaction settlement would be reduced with removal of materials for the subterranean excavations as summarized in **Section 2.6**. The seismic and static settlements can be additive. It may be prudent to assume a lesser horizontal distance should adjacent footings be substantially different in size.

5.6 Slab-On-Grade

These recommendations are considered to be minimum requirements for residential applications that may be superseded by more stringent requirements from the architect, structural engineer, or governing agencies.

Concrete slabs should be at least 4-inches in thickness. Actual slab thickness and reinforcement should be determined by the structural engineer based on structural loads and soil interaction. Our recommendations should be superseded by the recommendations of the structural engineer or architect.

Subgrade soils should be placed wet of the optimum moisture content and moisture should be maintained until placement of the concrete slab. Additional testing should be performed at completion of precise grading to verify our recommendations.

The slab should be underlain by a minimum two inch layer of sand; with a sand equivalent of 30 or greater. The sand layer should be underlain by a 15-mil Stego Wrap vapor retarder or equivalent product with a permeance rate of 0.012 perms and a puncture resistance of Class “A” or “B” in accordance with ASTM E 1745-97. As per the manufacturer’s recommendations all seams should overlap a minimum of 6 inches and should be sealed in accordance with the specifications provided by the vapor retarder manufacturer. All penetrations should be sealed using a combination of Stego Wrap, Stego Tape and/or Stego Mastic or approved equivalent product. The vapor retarder should be lapped downward a minimum of 12 inches where the vapor retarder encounters an interior footing or exterior thickened edge or footing. The vapor retarder should be placed on top of the sand layer if the sand is expected to become wet prior to pouring of concrete. If the sand can be kept dry prior to pouring concrete, the vapor retarder should be placed under the sand layer. The water cement ratio should be a minimum of 0.45 for all concrete within the structure that will come in contact with the on-site soil.

If moisture sensitive floor coverings are utilized, interior concrete slabs should be designed and constructed in accordance with the applicable floor covering manufacturer's specifications.

Slab subgrade soil should be pre-saturated to at least optimum moisture content to a depth of at least 12 inches below the sand layer.

Basement Slab on Grade Floors

Parking garage basement slab on grade floors should be a minimum of 4-inches in thickness and reinforced to resist shrinkage and temperature warping cracking. Actual slab thickness and steel reinforcement should be determined by the structural engineer based on environmental factors and concrete shrinkage considerations.

5.7 Permanent Subterranean Walls

We anticipate that where temporary shoring is installed, the permanent restrained retaining walls for the subterranean level will predominantly be placed directly against the temporary shoring. The design parameters provided below assume that granular non-expansive soils (Expansion Index <20 and SE≥30) are used to backfill any retaining walls. Permanent subterranean walls should be designed to resist the pressure exerted by retained soils plus any additional lateral forces due to loads placed adjacent to or near the wall. Retaining walls that are free-draining, are situated above groundwater and are to be restrained from movement at the top, such as basement walls, should be designed for an equivalent fluid weight of 60 pcf for at-rest conditions (for a level surface of retained earth). If traffic loads are planned adjacent to the walls, the walls should be designed for an additional uniform horizontal pressure of 75 and 150 psf for passenger car and truck traffic, respectively. For other surcharge loads, we recommend the walls be designed to resist a uniform horizontal pressure equal to 30 percent of the uniform surcharge load.

If backfill conditions (including the slope of the retained ground surface) differ from those assumed herein, Kling Consulting Group should be consulted to provide additional evaluation and/or recommendations as warranted. All retaining structures should be fully free draining. Building walls below grade should be waterproofed or damp-proofed, depending on the degree of moisture protection desired. The foundation system for the retaining walls should be designed in accordance with the recommendations presented in the preceding sections of this report, as appropriate. Footings should be embedded at a minimum of 18-inches below adjacent grade (excluding 6-inch landscape layer).

For resistance to lateral loads, an allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid weighing 300 pcf acting against the foundation may be used to resist lateral forces. Passive pressure in the upper 1.0-foot should be neglected unless confined by concrete slabs-on-grade or asphaltic pavement. These values may be increased by one-third for transient wind or seismic loads.

A seismic surcharge of 19 H should be applied as an equivalent fluid pressure with the resultant acting at 1/3-height above the base of the wall, where H= the retained height of the wall greater than 6 feet.

The permanent subterranean wall should be provided with an adequate backdrain system to reduce the potential for build-up of hydrostatic pressures.

Adequate drainage should be provided behind all retaining walls. The drainage system should consist of a minimum of four-inch diameter perforated PVC pipe (schedule 40 or approved equivalent) placed at the base of the retaining wall and surrounded by ¾-inch clean crushed rock wrapped in a Mirafi 140N filter fabric, or equivalent approved by the Geotechnical Engineer. The drain rock wrapped in fabric should be at least 12-inches wide and extend from the base of the wall to within two feet of the ground surface. The upper two feet of backfill should consist of compacted native soil. The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility.

For those portions of the wall not placed against shoring, the above values assume granular backfill and free-draining conditions to prevent buildup of hydrostatic pressure in the backfill. Backfill materials should meet the recommendations described in the following section of this report. Import fill materials should by approved by the soils engineer prior to placement. Wall backfill should be compacted by mechanical methods to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

5.8 Temporary Excavations

We anticipate the onsite soils can be excavated using conventional heavy duty earthmoving equipment in good condition. Shoring systems, if used, may yield during excavation causing adjacent facilities and improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor’s skill with installing the shoring system. Lateral deflections for a properly designed and constructed shoring system would likely be within ordinarily accepted limits of approximately 1-inch. A monitoring program should be established to evaluate the effects of shoring construction on other facilities.

Provided the excavations are above groundwater, temporary excavations and trench walls to a depth of four feet may be made vertically without shoring, subject to verification of safety by the contractor. Deeper excavations should be no steeper than 1.5:1 (horizontal to vertical) or braced or shored in accordance with CAL OSHA standards and guidelines. The contractor is assumed responsible for maintaining safety at the jobsite. All excavation work should be in compliance with current CAL OSHA standards. Under no circumstances should excavations be made deeper than four feet or below groundwater without shoring, bracing or laying-back, in accordance with CAL OSHA standards and guidelines. No surcharge loads should be allowed within five feet from the top of the cuts.

Existing utility lines, roadways and other easements/right-of-ways may be impacted by the temporary excavations may require shoring to obtain the full depth of the excavation.

5.9 Shoring

It is understood that a temporary or permanent shoring system may be warranted for those areas of proposed excavation for the proposed structures to achieve the subterranean level grades where space is not available for properly sloped backcuts. The shoring contractor should coordinate with the earthmoving contractor regarding sequence and requirements of installing the shoring system. The shoring contractor should also consider the potential for localized perched groundwater in the design and installation procedures of the shoring system.

We anticipate that the shoring system will be designed as a cantilever system and may consist of closely spaced steel H-Pile soldier piles and wooden lagging. Preliminary design considerations are presented in the following section for this anticipated shoring method. Please note that the method of temporary support can impact the design earth pressures. As such, Kling Consulting Group should perform a review of the shoring design and provide additional recommendations, as warranted.

Shoring systems, during excavation, may yield causing adjacent facilities and improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor’s skill with installing the shoring system. Lateral deflections for a properly designed and constructed shoring system would likely be within ordinarily accepted limits of approximately 1-inch. A monitoring program should be established to evaluate the effects of shoring construction on other facilities.

Horizontal and vertical movements of the shoring system should be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor’s responsibility. At a minimum, we recommend that the tops of the soldier beams should be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The surveyed soldier beam data points should be located at approximately 50 feet on-center. Surveying should consist of measuring movements in vertical and two perpendicular horizontal directions.

The shoring system should be designed to resist the pressure exerted by the retained soils plus any additional lateral forces due to loads applied near the top of the excavations. Cantilever shoring walls with a level backfill surface should be designed for an equivalent fluid pressure of 40 pcf. For surcharge loads due to traffic, the shoring should be designed for an additional uniform horizontal pressure of 75 psf for passenger car traffic and 150 psf for heavy truck traffic. For other surcharge loads, the wall should be designed for a uniform horizontal pressure equal to one-third the anticipated surcharge pressure. These parameters all assume a level ground surface and that temporary shoring will not be subject to hydrostatic pressures. The shoring system should be properly embedded beneath the toe of the excavation to provide adequate structural stability.

It is recommended that the design of the shoring system incorporate a passive equivalent fluid weight of 300 pcf for the shoring embedded within relatively competent old paralic deposits material. The soldier piles should be spaced no closer than 3 diameters on center. The soldier piles should be drilled and backfilled with concrete to the full depth of the passive resistance zone. The area providing the passive resistance can be assumed to have a width equal to twice the concrete pile diameter.

The recommended passive pressure for the shoring assumes a horizontal surface for the soil mass extending at least 10 feet in front of the face of the shoring, or three times the height of the surface generating passive pressure, whichever is greater. The shoring system should be embedded a sufficient depth beneath the toe of the excavation so as to provide structural stability. We recommend that a factor of safety of at least 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 2,500 psf. The assumed geotechnical conditions should be verified as necessary during shoring construction by a representative of the geotechnical consultant.

Timber lagging may be used between the soldier piles to help support the exposed soils. If lagging is to remain after construction, treated lumber should be used. Lagging should be designed for the full lateral pressure recommended above. If possible, structural walls should be cast directly against the shoring, thus eliminating the need for placing backfill within a narrow space. Voids between the soil and lagging should be properly grouted or slurried to reduce the potential for the voids to propagate to the surface.

Special provisions for wall drainage (such as the use of prefabricated composite drain) may be necessary above the groundwater table where this type of construction is used.

The performance of the proposed shoring system is highly dependent on the means and methods utilized by the contractors involved in the work and the judgment of the shoring design engineer. The shoring engineer and contractor shall be solely responsible for locating the existing improvements surrounding the site, controlling settlements of the surrounding structures and improvements within the structural and aesthetic limits. Load path and loading determination for underpinning design is the purview of the structural underpinning designer.

If the anticipated depth of excavation requires shoring that extends to depths where a cantilever shoring system is not feasible, we would be pleased to provide geotechnical recommendations for an anchored (tie-back) shoring system upon request. With deep excavations required to allow for the construction of subterranean levels that would normally require tie-back anchors, due to the proximity to the adjacent properties or structures tie-back systems may not be allowed and other options such as H-beam and lagging or rakers may be required.

5.10 Concrete Flatwork

Laboratory testing of onsite soils by others (EEE, 2016) and our experience with similar soils in the site vicinity indicate that the upper on-site soil materials present possess a very low to high expansion potential. **Appendix G** contains a table listing our hardscape recommendations for varying degrees of expansive soils. This table should be preliminarily followed for a low to high expansion potential for Expansion Index (E.I.) = 21 to 130. Additional testing should be performed during earthwork construction to confirm the as graded conditions.

5.11 Sulfate Potential

Based on the soluble sulfate test results the on-site soils possess a sulfate exposure that is considered “Class S0”. Concrete should be designed in accordance with ACI 318, Section 19 Table 3.1.1, utilizing “Class S0” sulfate exposure.

5.12 Corrosion Potential

Buried metals in contact with on-site soils should be encased, sleeved, or wrapped with a suitable dielectric material to isolate them from the on-site soils. Alternatively, plastic piping may also be used. Polyethylene sleeving should be utilized at a minimum to protect copper plumbing pipe.

For more specific recommendations regarding soil corrosivity, it is recommended that a qualified corrosion consultant be retained to provide more specific recommendations regarding corrosion protection.

5.13 Pavement Design

Pavement section design is provided below based on near surface soil conditions encountered during our investigation and assumed traffic loading.

5.13.1 Asphalt Concrete Pavement

The upper on-site subgrade soils were classified as silty and clayey sands. To allow for soil variability, we are assuming an R-Value of 10 for preliminary design purposes.

Based on an R-value of 10, the parameters below are provided for preliminary design purposes. Pavement sections were calculated for traffic indices of 4.0 and 5.5, which are commonly used for parking stalls and drive aisles subject to passenger vehicles, respectively. However, the selection of actual traffic index should be the purview of the project civil or traffic engineer.

Pavement Section Design

Location	R-Value	Traffic Index	Multiple Layered	
			Asphalt Concrete (inches)	Aggregate Base* (inches)
Parking Stall	10	4.0	3.0	6.0
Drive Aisles	10	5.5	4.0	9.0

*Aggregate base material should consist of Class 2 aggregate base materials or Crushed Miscellaneous Base (CMB).

The upper 12 inches of the subgrade soils should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM D1557). All base materials should be compacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557).

5.13.2 Portland Cement Concrete Pavement

For preliminary design of concrete pavement, it is recommended that a concrete pavement section consisting of 6-inches of concrete underlain by at least 4-inches of either Class 2 or crushed miscellaneous base be used for preliminary design. Concrete Compressive strength should be 4000 psi or greater. Aggregate base material should be compacted to a minimum of 95 percent relative compaction as per ASTM D1557. Subgrade soil should be compacted to at least 90 percent of the laboratory maximum dry density in accordance with ASTM D1557. If concrete crack control is desired, the slabs should be minimally reinforced with No. 4 rebar, placed every 24 inches on center, both ways. A 10-foot square or less grid system should be used in the construction of continuous sections of concrete pavement or as recommended by the structural engineer.

For trash enclosures, concrete pavement should consist of a minimum 8-inch thick concrete slab placed over a minimum of 6-inches of either Class 2 or crushed miscellaneous base material, compacted to 95 percent relative compaction. Concrete should have a minimum strength of 4000 psi and be reinforced with a minimum of No. 4 bars placed at 24 inches on center, in each direction, positively supported (with concrete chairs or other devices) at mid-height in the slab. Crack control joints should be placed at a 10-foot maximum spacing in each direction in the slab or as recommended by the structural engineer. Concrete mix design should incorporate the recommendations presented in the slab on grade section of this report for improved geotechnical performance.

5.14 Surface Drainage

Surface runoff from natural and graded areas should be controlled and water infiltration into the subsurface should be minimized whenever possible. Positive drainage should be maintained away from any building or structure or graded slope face and directed to suitable areas via non-erosive devises, as designed by the project civil engineer. For drainage over a soil area immediately adjacent to structures, i.e., within 10 feet horizontally or as determined during Precise Grading, a minimum of 5 percent gradient should be maintained. Pad drainage of at least 2 percent should be maintained over any soil areas if applicable. Impervious surfaces within 10 feet of a building foundation should be sloped a minimum of 2 percent away from the building. All drainage should be in accordance with Section 1804.4 of the 2016 California Building Code.

5.15 Plan Review

The geotechnical consultant should review the grading plans and comment on the anticipated effects of any major changes from the conceptual site plan used in this report. Additionally, the geotechnical consultant should review the foundation and retaining wall plans when they become available.

6.0 LIMITATIONS

Geotechnical services are provided by Kling Consulting Group, Inc. in accordance with generally accepted professional engineering and geologic practice in the area where these services are to be rendered. Client acknowledges that the present standard in the engineering and geologic and environmental profession does not include a guarantee of perfection and, except as expressly set forth in the Conditions above, no warranty, expressed or implied, is extended by KCG.

Geotechnical reports are based on the project description and proposed scope of work as described in the proposal. Our conclusions and recommendations are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions as described in the report. The results reflect our geotechnical interpretation of the limited direct evidence obtained. Our conclusions and recommendations are made contingent upon the opportunity for KCG to continue to provide geotechnical services beyond the scope in the proposal to include all geotechnical services. If parties other than KCG are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical work of the project by concurring with the recommendations in our report or by providing alternate recommendations.

It is the readers' responsibility to verify the correct interpretation and intention of the recommendations presented herein. KCG assumes no responsibility for misunderstandings or improper interpretations that result in unsatisfactory or unsafe work products. It is the reader's further responsibility to acquire copies of any supplemental reports, addenda, or responses to public agency reviews that may supersede recommendations in this report.

Kling Consulting Group, Inc. appreciates this opportunity to be of service. Should you have any questions regarding our report, please do not hesitate to call our office.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

1. American Concrete Institute, 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14).

2. American Society for Testing and Materials (ASTM), 2018, Annual Book of ASTM Standards, Volume 04.08, Construction: Soil and Rock (I), Standards D 420 - D 5876.

3. California Building Standards Commission, 2016, California Building Code, Volume 2

4. California Division of Mines and Geology (CDMG), 2000, California Department of Conservation, Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Southern Region, DMG CD 2000-003.

5. California Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the Tustin 7.5-Minute Quadrangle, Orange County, California, Seismic Hazard Zone Report 012, 1998.

6. California Department of Water Resources 2011 through 2018 Groundwater Level Data, accessed June 2019,http://wdl.water.ca.gov/waterdatalibrary/groundwater/hydrographs/

7. California Geologic Survey (CGS), Compilation of Quaternary Surficial Deposits: https://maps.conservation.ca.gov/cgs/qsd/app/, Accessed May, 2019.

8. California Geologic Survey, 1999, Earthquake Zones of Required Investigation, Seismic Hazard Zones, San Dimas Quadrangle, California, official map released March 25, 1999.

9. EEI, 2016, Supplemental Geotechnical Evaluation, Proposed Mixed Use Development, The Koll Center Residences, City of Newport Beach, Orange County, California, SHO-72419.4a, dated October 31, 2016.

10. Geotracker, State of California Water Resources Control Board, https://geotracker.waterboards.ca.gov/, Accessed June, 2019.

11. Hart, E.W., and Bryant, W.A. (Hart and Bryant), 1997, Fault-Rupture Hazard Zones in California: California Department of Conservation, Division of Mines and Geology, Special Publication 42.

12. Jennings, C.W., and Bryant, W.A., (Jennings and Bryant) 2010, Fault Activity Map of California and Adjacent Areas: California Geologic Survey, Map Sheet No. 6, scale 1:750,000.

APPENDIX A

REFERENCES
(CONTINUED)

13. Kling Consulting Group, Second Party Due Diligence Geotechnical Review, Proposed Residential Development, Koll Center Property Southwest Corner of Van Karman Avenue and Birch Street, Newport Beach, California, PN 19017-00, dated May 17, 2019

14. Structural Engineers Association of California (SEAC)/Office of Statewide Health Planning and Development OSHPD:
Seismic Design Maps: https://oshpd.ca.gov/seismic maps.org, accessed June, 2019.

15. USGS, 2004, Preliminary Geologic Map of the Santa Ana 30'x60' Quadrangle, Southern California, Version 2.0, Compiled by D.M. Morton, 2004, Open File Report 99-172, SCAMP- Southern California Regional Mapping Project.

16. USGS, Earthquakes Hazard Program, National Seismic Hazards Maps, Source Parameters,https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm June, 2019.

APPENDIX B

BORING LOGS
AND CPT SOUNDINGS

LOG OF EXPLORATORY BORING

Sheet **1** of **2**

Project: **Koll Center**
Project Number: **19017-00**
Date Drilled: **5/30/19**
Logged By: **DKL**

Boring No.: **KB-1**
Driller: **Gregg Drilling**
Drill Type: **Hollow Stem**
Hammer Wt. / Drop: **140lb / 30in**
Ground Elev. [ft]: **42.0**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧓ California
☒ Bulk Sample
▼ Static Water Table

Elevation [ft]	Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
							@ **Surface - ASPHALT:** 3.0" of aggregate base material underneath the asphalt. **<u>Artificial Fill (Af):</u>** @ **0.5 feet - <u>Clayey SAND (SC):</u>** brown, fine-grained, some gravel up to 1/2", some mica, moist.			
40										
	5		⧓	8 10 12	10	111	**<u>Old Paralic deposits (Qopf):</u>** @ **5.0 feet - SAME:** very stiff.			
35										
	10		⧓	10 11 11	2	95	@ **10.0 feet - <u>Silty SAND (SM):</u>** light brown, fine-grained, micaceous, damp, medium dense.		DS	
30										
	15		⧓	5 12 18	20	101	@ **15.0 feet - <u>Sandy CLAY (CL):</u>** grey-brown, fine to coarse-grained sand, micaceous, some orange staining, damp, very stiff.		CN	
25										
	20		⧓	10 22 35	4	94	@ **20.0 feet - <u>Silty SAND (SM):</u>** olive-brown, fine to coarse-grained, micaceous, some gravel up to 1/2", some orange staining, moist, dense.			
20										
	25		⧓	4 8 10	18	107	▼ @ **25.0 feet - SAME:** wet, medium dense.		DS	
15										

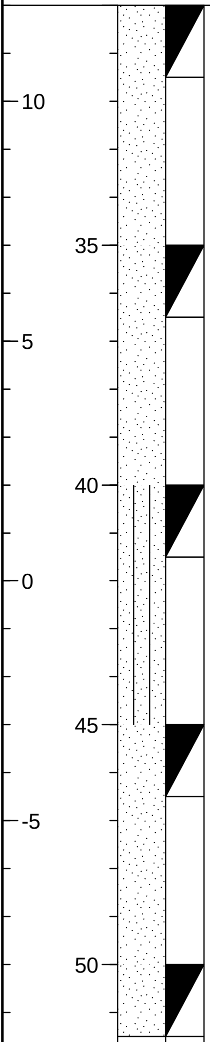
KLING Consulting Group, Inc.

LOG OF EXPLORATORY BORING

Sheet **2** of **2**

Project:	**Koll Center**	Boring No.:	**KB-1**
Project Number:	**19017-00**	Driller:	**Gregg Drilling**
Date Drilled:	**5/30/19**	Drill Type:	**Hollow Stem**
Logged By:	**DKL**	Hammer Wt. / Drop:	**140lb / 30in**
		Ground Elev. [ft]:	**42.0**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧓ California
☒ Bulk Sample
▼ Static Water Table



Elevation [ft]	Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
			◩	9 16 32			@ **30.0 feet - <u>SAND (SP):</u>** grey-brown, fine to coarse-grained, micaceous, some orange staining, wet, dense.			
	35		◩	4 8 15			@ **35.0 feet - SAME:** medium dense.			
	40		◩				@ **40.0 feet - <u>Silty fine SAND (SM):</u>** olive-brown, fine-grained, wet.			
	45		◩	16 17 24			@ **45.0 feet - <u>SAND (SP):</u>** grey-brown, fine to coarse-grained, trace gravel up to 1/4", wet, dense.			
	50		◩	13 15 19			@ **50.0 feet - <u>SAND (SP):</u>** dark grey, fine to medium-grained, wet, dense.			

Total Depth: 51.5 feet.
Perched groundwater encountered at 25.0 feet.
No caving.
Piezometer installed and secured with a well box on 5/30/2019.

KCG KLING Consulting Group, Inc.

LOG OF EXPLORATORY BORING

Sheet **1** of **2**

Project:	**Koll Center**	Boring No.:	**KB-2**
Project Number:	**19017-00**	Driller:	**Gregg Drilling**
Date Drilled:	**5/31/19**	Drill Type:	**Hollow Stem**
Logged By:	**DKL**	Hammer Wt. / Drop:	**140lb / 30in**
		Ground Elev. [ft]:	**39.0**

◩ Standard Split Spoon ■ Shelby Tube ▽ Water Level ATD

⧗ California ☒ Bulk Sample ▼ Static Water Table

Elevation [ft]	Depth [ft]	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
						@ **Surface - ASPHALT:** 3.0" of aggregate base material underneath the asphalt.			
						Artificial Fill (Af): @ **0.5 feet - Silty SAND (SM):** brown, fine to coarse-grained, trace clay, micaceous, damp.			
35									
	5	⧗	10 20 32	6	108	**Old Paralic deposits (Qopf):** @ **5.0 feet - SAME:** dense.			
30									
	10								
		⧗	7 8 19	28	93	@ **12.0 feet - Sandy CLAY (CL):** brown, some quartz, some feldspar, micaceous, wet, very stiff.		CN	
25									
	15								
20									
	20	⧗	8 15 23	19	105	▼ @ **20.0 feet - Silty SAND (SM):** grey-brown, fine to coarse-grained, micaceous, some quartz, wet, dense.			
15									
	25	⧗	8 9 15	16	109	@ **25.0 feet - Silty SAND (SM):** grey-brown, fine to coarse-grained, micaceous, wet, medium dense.		DS CN	
10									

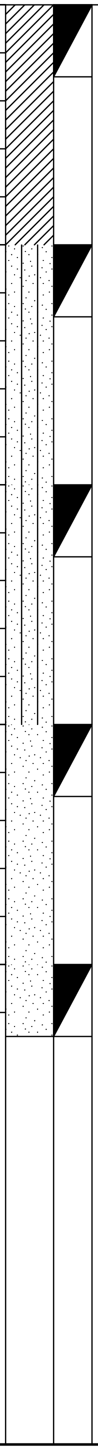
LOG OF EXPLORATORY BORING

Sheet **2** of **2**

Project: **Koll Center**
Project Number: **19017-00**
Date Drilled: **5/31/19**
Logged By: **DKL**

Boring No.: **KB-2**
Driller: **Gregg Drilling**
Drill Type: **Hollow Stem**
Hammer Wt. / Drop: **140lb / 30in**
Ground Elev. [ft]: **39.0**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧓ California
⊠ Bulk Sample
▼ Static Water Table



Elevation [ft]	Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
				2 3 4			@ **30.0 feet - Sandy CLAY (CL):** grey, fine sand, micaceous, wet, medium stiff.			
5										
	35			3 4 6			@ **35.0 feet - Silty SAND (SM):** dark grey, fine-grained, micaceous, wet, loose.			
0										
	40			4 6 11			@ **40.0 feet - SAME:** medium dense.			
-5										
	45			11 19 22			@ **45.0 feet - SAND (SP):** grey, fine to medium-grained, micaceous, some quartz, trace silt, wet, dense.			
-10										
	50			12 14 14			@ **50.0 feet - SAME:** medium dense.			
							Total depth: 51.5 feet. Perched groundwater encountered at 20.0 feet. No caving. Hole backfilled on 5/31/2019.			

KLING Consulting Group, Inc.

LOG OF EXPLORATORY BORING

Sheet **1** of **2**

Project: **Koll Center**
Project Number: **19017-00**
Date Drilled: **5/31/19**
Logged By: **DKL**

Boring No.: **KB-3**
Driller: **Gregg Drilling**
Drill Type: **Hollow Stem**
Hammer Wt. / Drop: **140lb / 30in**
Ground Elev. [ft]: **42.0**

◪ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧗ California
☒ Bulk Sample
▼ Static Water Table

Elevation [ft]	Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
		■					@ **Surface - ASPHALT:** 3.0" of aggregate base material underneath the asphalt.			
		▨					**<u>Artificial Fill (Af):</u>** @ **0.5 feet - <u>Sandy CLAY (CL):</u>** brown, fine-grained, some mica, moist.			
40										
	5	▨	⧗	4 6 10	22	98	**<u>Old Paralic deposits (Qopf):</u>** @ **5.0 feet - SAME:** stiff.			
35										
	10	░	⧗	6 14 17	5	109	@ **10.0 feet - <u>SAND (SP):</u>** brown, fine to coarse-grained, micaceous, some gravel up to 1/2", moist, medium dense.		DS	
30										
	15	▨	⧗	5 10 14	25	98	@ **15.0 feet - <u>Silty sandy CLAY (CL):</u>** grey, some orange staining, trace fine sand, moist, very stiff.		CN	
25										
	20	▨	⧗	4 8 10	30	90	@ **20.0 feet - SAME:** more staining, stiff.		CN	
20										
	25	░	⧗	9 15 28	23	99	▼ @ **25.0 feit - <u>Silty fine SAND (SM):</u>** grey, fine-grained, some orange staining, wet, dense.			
15										

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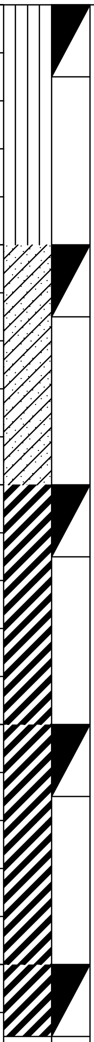
Consulting
Group, Inc.

LOG OF EXPLORATORY BORING

Sheet **2** of **2**

Project:	**Koll Center**	Boring No.:	**KB-3**
Project Number:	**19017-00**	Driller:	**Gregg Drilling**
Date Drilled:	**5/31/19**	Drill Type:	**Hollow Stem**
Logged By:	**DKL**	Hammer Wt. / Drop:	**140lb / 30in**
		Ground Elev. [ft]:	**42.0**

Legend:
- ◩ Standard Split Spoon
- ■ Shelby Tube
- ▽ Water Level ATD
- ⧗ California
- ☒ Bulk Sample
- ▼ Static Water Table



Elevation [ft]	Depth [ft]	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
	30	7 10 11			@ **30.0 feet - <u>Silty fine SAND (ML):</u>** grey, fine-grained, micaceous, wet, very stiff.			
10								
	35	4 5 6			@ **35.0 feet - <u>Clayey silty SAND (SC):</u>** grey, fine-grained, micaceous, wet, stiff.			
5								
	40	5 3 4			@ **40.0 feet - <u>CLAY (CH):</u>** grey, some shell fragments, wet, medium stiff.			
0								
	45	3 2 2			@ **45.0 feet - SAME**			
-5								
	50	2 2 3			@ **50.0 feet - SAME**			

Total depth: 51.5 feet.
Perched groundwater encountered at 25.0 feet.
No caving.
Hole backfilled on 5/31/2019.

KCG KLING Consulting Group, Inc.

LOG OF EXPLORATORY BORING

Sheet **1** of **2**

Project: **Koll Center**
Project Number: **19017-00**
Date Drilled: **5/31/19**
Logged By: **DKL**

Boring No.: **KB-4**
Driller: **Gregg Drilling**
Drill Type: **Hollow Stem**
Hammer Wt. / Drop: **140lb / 30in**
Ground Elev. [ft]: **41.0**

◩ Standard Split Spoon
■ Shelby Tube
▽ Water Level ATD
⧗ California
☒ Bulk Sample
▼ Static Water Table

Elevation [ft]	Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
40							@ **Surface - ASPHALT:** 3.0" of aggregate base material underneath the asphalt. **Artificial Fill (Af):** @ **0.5 feet - Silty SAND (SM):** brown, fine to medium-grained, micaceous, some gravel up to 1/4", trace silt, moist.			
35	5		⧗	9 12 19	9	118	**Old Paralic deposits (Qopf):** @ **5.0 feet - SAME:** medium dense.			
30	10		⧗	3 3 3	5	104	@ **10.0 feet - Silty SAND (SM):** brown, fine to coarse-grained, micaceous, some gravel up to 1/4", moist, loose.			
25	15		⧗	15 15 25	6	107	@ **15.0 feet - SAME:** some quartz, some feldspar, dense.		DS	
20	20		⧗	5 8 25	26	100	@ **20.0 feet - Silty CLAY (CL):** grey, some orange staining, moist, very stiff.		DS CN	
15	25		◩	9 15 22			▼ @ **25.0 feet - SAND (SP):** olive-brown, fine to coarse-grained, micaceous, some quartz, some feldspar, wet, dense.			

LOG OF EXPLORATORY BORING

Sheet **2** of **2**

Project: **Koll Center**
Project Number: **19017-00**
Date Drilled: **5/31/19**
Logged By: **DKL**

Boring No.: **KB-4**
Driller: **Gregg Drilling**
Drill Type: **Hollow Stem**
Hammer Wt. / Drop: **140lb / 30in**
Ground Elev. [ft]: **41.0**

◩ Standard Split Spoon
■ Shelby Tube
⧗ California
☒ Bulk Sample
▽ Water Level ATD
▼ Static Water Table

Elevation [ft]	Depth [ft]	Graphic Log	Sample Type	Blows/6"	Moisture Content [%]	Dry Density, [pcf]	SOIL DESCRIPTION and CLASSIFICATION (USCS)	Pocket Pen. [tsf]	Lab Tests	Remarks
10		░	◩	14 15 23			@ **30.0 feet - <u>SAND (SP):</u>** grey, fine to coarse-grained, some mica, trace gravel up to 1/4", wet, dense.			
5	35	▨	◩	5 4 5			@ **35.0 feet - <u>CLAY (CH):</u>** dark grey, some shell fragments, wet, stiff.			
0	40	▨	◩	2 3 4			@ **40.0 feet - SAME:** medium stiff.			
-5	45	▨	◩	2 3 3			@ **45.0 feet - SAME**			
-10	50	▧	◩	2 3 4			@ **50.0 feet - <u>Silty CLAY (CL):</u>** dark grey, micaceous, wet, medium stiff.			

Total depth: 51.5 feet.
Perched groundwater encountered at 25.0 feet.
No caving.
Piezometer installed and secured with a well box on 5/31/2019.

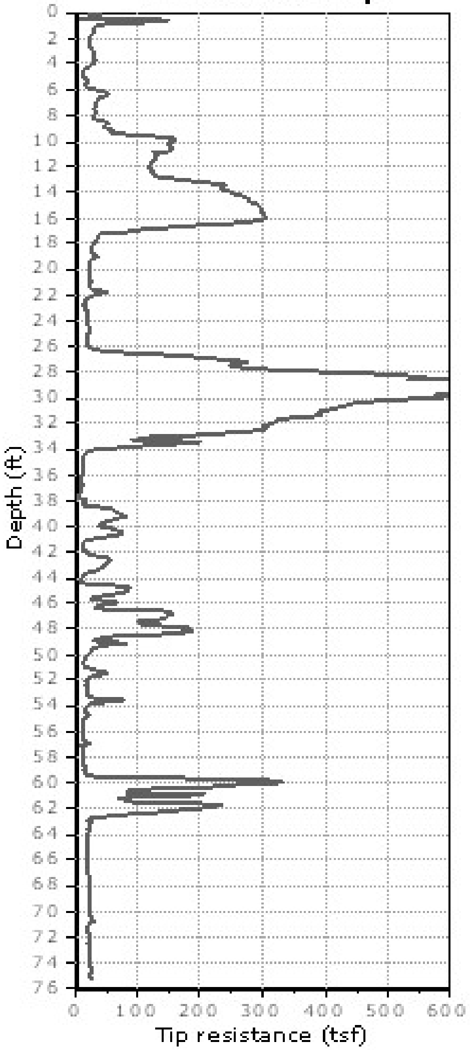
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

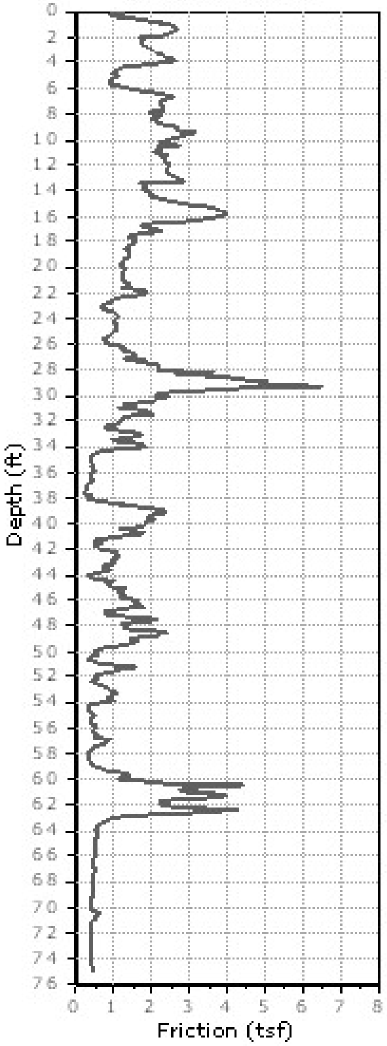
Project: **Kling Consulting Group / Koll Center**

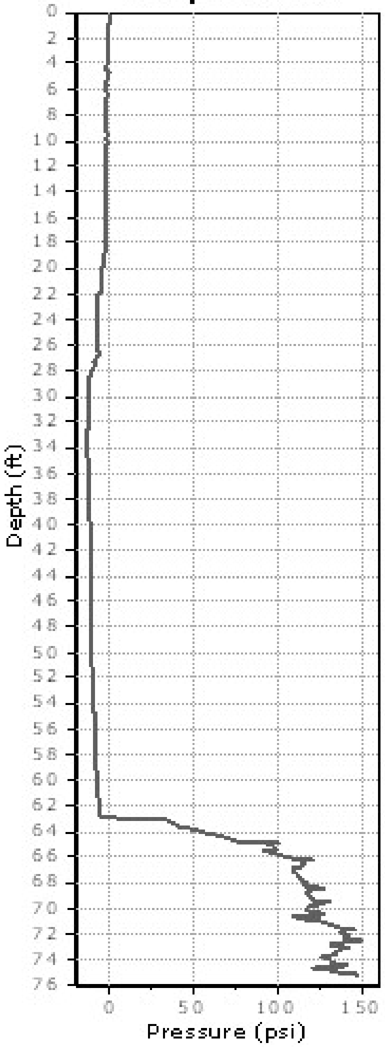
Location: **4400 Von Karman Ave, Newport Beach, CA**

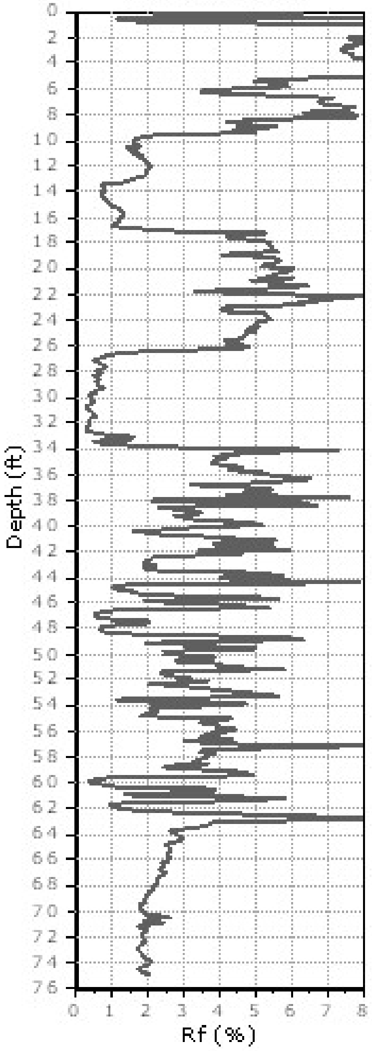
CPT-1

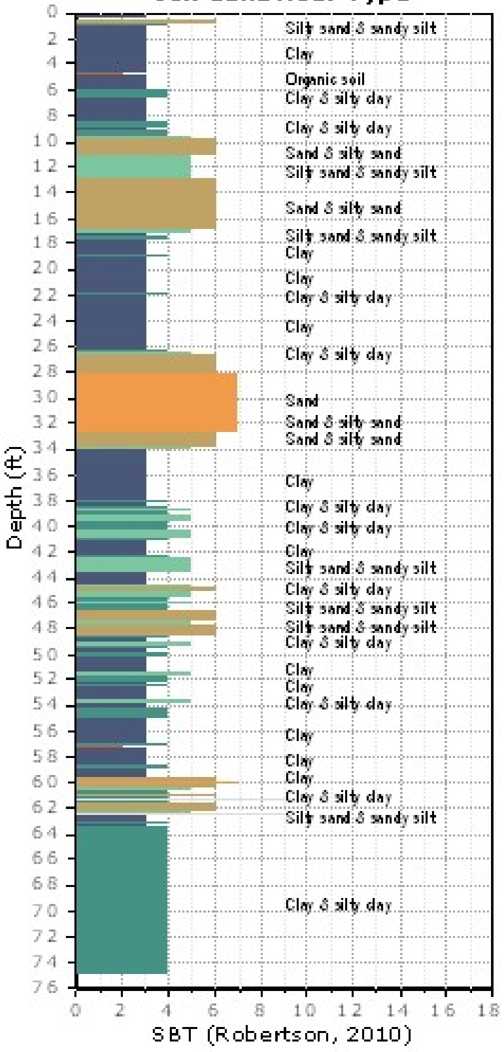
Total depth: 75.34 ft, Date: 5/30/2019


Cone resistance qt


Sleeve friction


Pore pressure u


Friction ratio


Soil Behaviour Type

Kehoe Testing and Engineering
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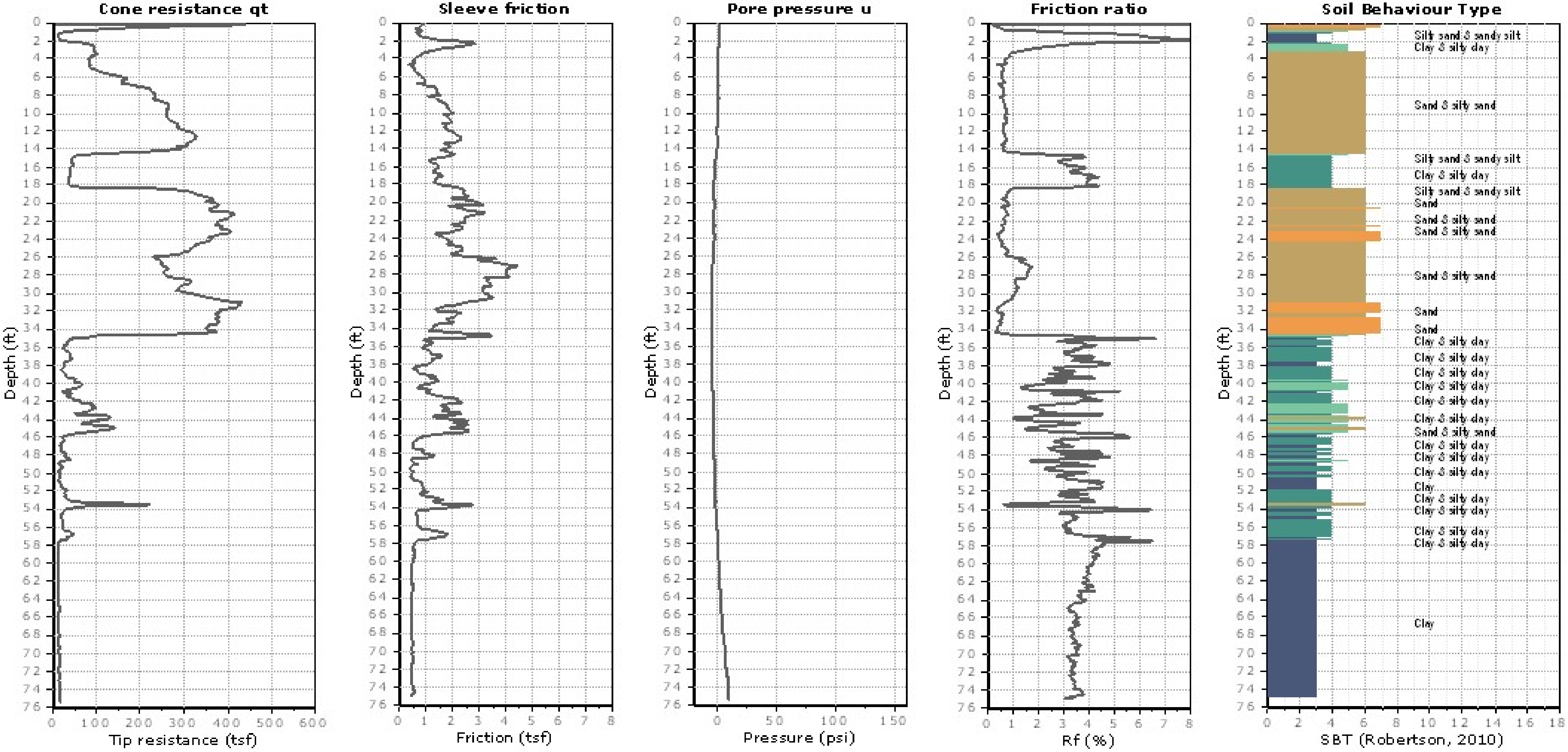
www.kehoetesting.com

Project: **Kling Consulting Group / Koll Center**

Location: **4400 Von Karman Ave, Newport Beach, CA**

CPT-2

Total depth: 75.36 ft, Date: 5/30/2019



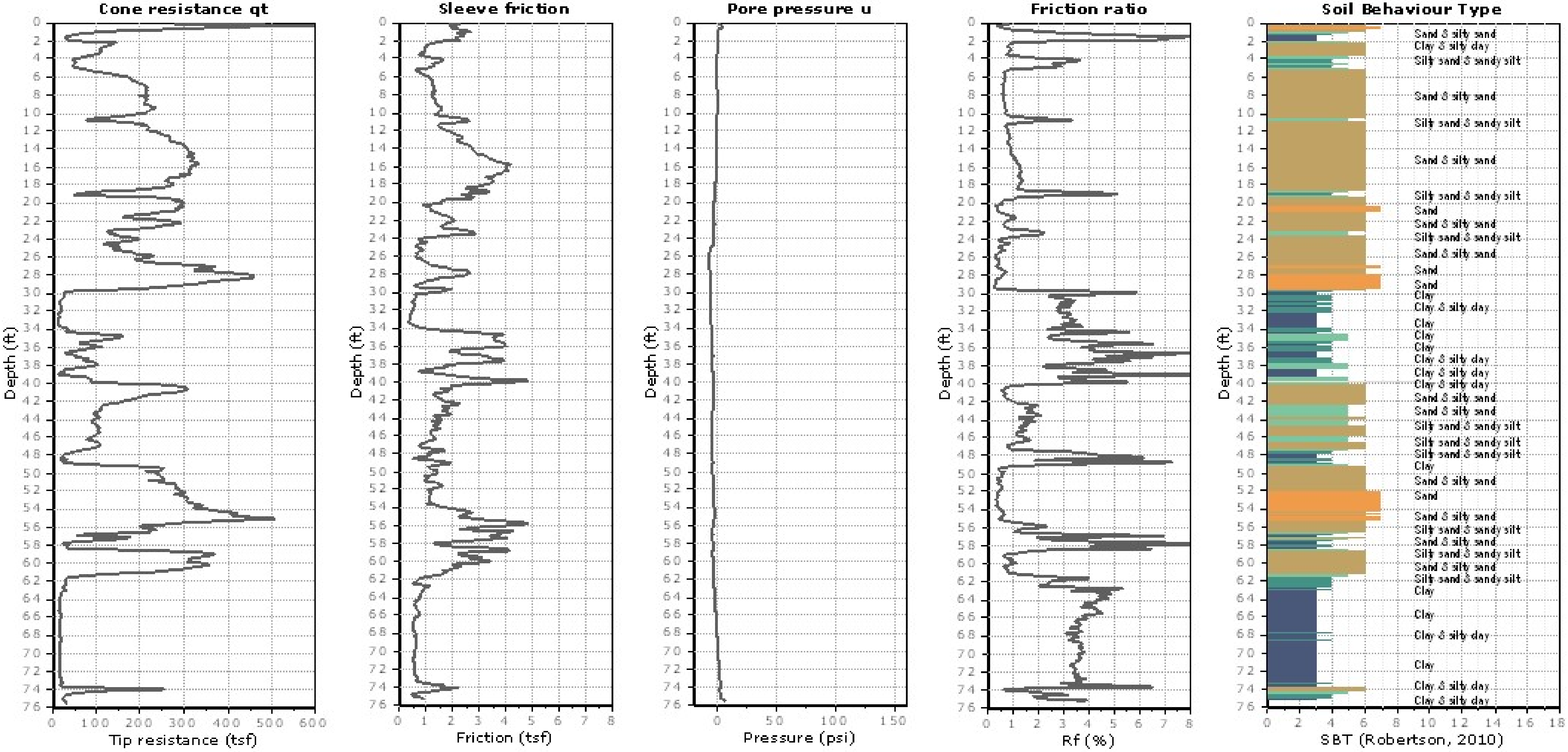
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Kling Consulting Group / Koll Center

Location: 4400 Von Karman Ave, Newport Beach, CA

CPT-3

Total depth: 75.60 ft, Date: 5/30/2019



Kehoe Testing and Engineering
714-901-7270
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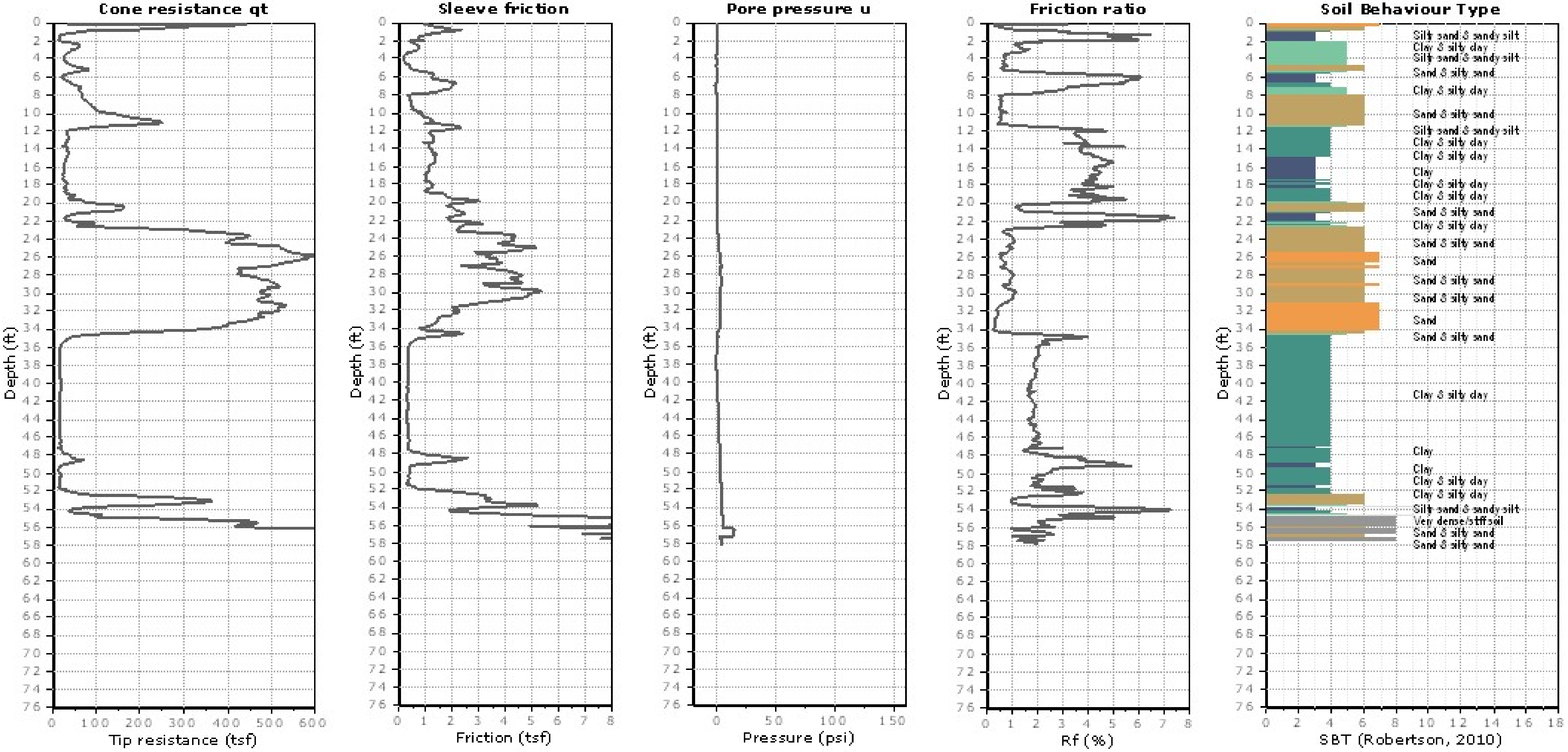
www.kehoetesting.com

Project: **Kling Consulting Group / Koll Center**
Location: **4400 Von Karman Ave, Newport Beach, CA**

CPT-4
Total depth: 58.15 ft, Date: 5/30/2019



BORING NUMBER B-1

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6"),**										
1		**FILL** SAND, reddish brown, damp, medium dense, fine to medium grained sand										
3			SP	MC	15 16 20	22		2	101			
5		**OLD PARALIC DEPOSITS** @ 5' SAND WITH SILT, yellow to reddish brown, damp, medium dense, fine grained sand		MC	13 10 12	14		7	96			
7				BULK								
8				MC	8 13 17	18		2	99			
10			SP-SM	MC	10 16 24	25		3	102			
15		@ 15' SAND, light gray to yellow brown, wet, medium dense, fine grained sand		SPT	7 11 11	25		5				
20		@ 20' becomes white to yellow brown, fine to medium grained	SP	MC	5 16 25	25		3	115			
25	▽	@ 25' becomes saturated; seepage encountered		SPT	4 8 11	21		17				

(Continued Next Page)

BORING NUMBER B-1

PAGE 2 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-1 continued...*										
29			SP									
30		@ 30' SANDY-CLAY, dark gray, moist, very stiff, trace fine grained sand, low plasticity in field		MC	4 7 12	12		22	102			
31												
32			CL									
33												
34												
35		@ 35' CLAYEY-SAND, gray, saturated, loose to medium dense, fine grained sand		SPT	4 4 6	11		23				
36												
37			SC									
38												
39												
40		@ 40' SAND WITH SILT, gray, saturated, medium dense, fine grained		MC	9 19 28	29		24	99			
41												
42												
43												
44			SP-SM									
45		@ 45' becomes dense		SPT	7 16 31	52		23				
46												
47												
48												
49		@ 49' difficult drilling										

Total Depth: 49-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-10/P-4

PAGE 1 OF 1

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 50 feet **BORING DIAMETER** 8-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	■	5-INCH A/C OVER 4-INCH BASE										
1	▨	**FILL** CLAY, dark orange-brown, moist, medium stiff	CL									
2	▨	**OLD PARALIC DEPOSITS (Qopfa)** @ 2' SANDY-CLAY, orange-brown, moist, stiff	SC									
3												
4												
5												
6	░	@ 6' SAND with SILT, light orange-brown, fine to medium-grained, moist, medium dense	SP-SM									
7												
8												
9												
10												
11												
12	▨	@ 12' CLAY, orange-brown and gray, very moist, stiff	CL									
13												
14												
15												
16												
17												
18												
19				☒ SPT	3 6 11	19						
20	░	@ 19.5' SILTY-SAND, orange-brown, fine to medium-grained, moist, medium dense	SM									

Total depth: 20-feet
No groundwater encountered
Percolation test performed
Boring backfilled on 10/10/2016

BORING NUMBER B-2

PAGE 1 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DATE STARTED 7/15/15

COMPLETED 7/15/15

GROUND ELEVATION

BORING DIAMETER 6-inch

EQUIPMENT / RIG CalPac CME B63

HAMMER EFFICIENCY (%) 67

METHOD 140 pound Auto-hammer

SPT CORRECTION 1.12

CAL CORRECTION 0.61

LOGGED BY ML

CHECKED BY

GROUNDWATER DEPTH (ft) Not Encountered

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6")**										
1		**FILL** SILTY-CLAY, dark brown to reddish brown, damp, stiff										
2												
3			CL-ML	MC	6 7 5	7		13	93			
4												
5		**OLD PARALIC DEPOSITS** @ 5' SILTY-CLAY, yellow brown, damp, very stiff, non-plastic in field		BULK MC	7 10 18	17		14	93			
6												
7												
8		@ 7.5' becomes grey brown mottled		MC	6 10 19	18		22	95			
9												
10			CL-ML	MC	4 9 24	20		18	105			
11												
12												
13												
14												
15		@ 15' SAND, light brown, damp, medium dense, fine grained		SPT	7 8 9	19		3				
16												
17				BULK								
18												
19												
20				MC	12 13 19	20		2	101			
21			SP									
22												
23												
24												
25	∇	@ 25' becomes saturated, medium dense, fine to medium grained; seepage encountered		SPT	3 6 14	22		22				
26												
27												

(Continued Next Page)

BORING NUMBER B-2

PAGE 2 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-2 continued...*										
29												
30				MC	11 16 21	23		21	97			
31			SP									
32												
33												
34												
35		@ 35' SAND WITH SILT, reddish brown, saturated, dense, fine to medium grained sand		SPT	10 16 25	46		18				
36												
37			SP-SM									
38		@ 39' difficult drilling										
39												

Total Depth: 39-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-3

PAGE 1 OF 2

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	■	**ASPHALT (5") / BASE (6")**										
1	▩	**FILL** CLAY, dark reddish brown, damp, very stiff, medium plasticity in field	CL									
3	▩			MC	6 8 13	13		15	113			
5	░	**OLD PARALIC DEPOSITS** @ 5' SILTY-SAND, yellow brown, damp, loose, fine grained sand	SM	MC	5 6 10	10		8	101			
8	░	@ 7.5' becomes light gray to yellow brown mottled, medium dense		MC	5 9 12	13		5	100			
10	░			MC	8 13 18	19		5	91			
15	░	@ 15' SAND, light gray to yellow brown, damp, medium dense, fine to medium grained	SP	SPT	6 9 12	23		2				
20	░			MC	11 19 23	26		2	92			
25	░ ∇	@ 25' becomes reddish brown, saturated, dense; seepage encountered		SPT	10 14 16	34		18				
27	░			BULK								

(Continued Next Page)

BORING NUMBER B-3

PAGE 2 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-3 continued...*										
29			SP									
30		@ 30' SAND, light brown, saturated, very dense, fine to coarse grained; seepage encountered		MC	26 40 50/5"			9	120			
31			SW									
32		@ 33' difficult drilling										
33												

Total Depth: 33-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-4

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15
COMPLETED 7/15/15
GROUND ELEVATION
BORING DIAMETER 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12
CAL CORRECTION 0.61
LOGGED BY ML
CHECKED BY
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6")**										
1		**FILL** CLAYEY-SAND, reddish brown, damp, medium dense, fine to medium grained										
2												
3			SC	MC	7 9 11	12		8	114			
4												
5		**OLD PARALIC DEPOSITS** @ 5' SAND, light reddish brown, moist, medium dense, fine to medium grained		MC	7 11 17	17		4	105			
6												
7												
8				MC	10 14 21	21		4	106			
9												
10				MC	10 18 21	24		3	96			
11												
12												
13												
14												
15		@ 15' becomes light brown		SPT	8 10 16	29		4				
16			SP									
17												
18												
19												
20				MC	13 19 29	29		17	104			
21												
22												
23												
24												
25		▽ @ 25' becomes dense, saturated; seepage encountered		SPT	8 17 29	51		24				
26												
27												

(Continued Next Page)

BORING NUMBER B-4

PAGE 2 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28 29	⁘	*Boring B-4 continued...*	SP									
30 31	▨	@ 30' SILTY-CLAY, brown to reddish brown, moist, hard, medium plasticity in field	CL-ML	⧗ MC	16 17 30	29		33	83			

Total Depth: 31.5-feet
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-5

PAGE 1 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DATE STARTED 7/16/15 **COMPLETED** 7/16/15

GROUND ELEVATION **BORING DIAMETER** 6-inch

EQUIPMENT / RIG CalPac CME B63

HAMMER EFFICIENCY (%) 67

METHOD 140 pound Auto-hammer

SPT CORRECTION 1.12 **CAL CORRECTION** 0.61

LOGGED BY ML **CHECKED BY**

GROUNDWATER DEPTH (ft) Not Encountered

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (8") / BASE (4")**										
1		**FILL** CLAYEY-SILT, gray and brown mixed, moist, medium stiff, trace sand										
3			CL-ML	MC	3 3 3	4		10	99			
5		**OLD PARALIC DEPOSITS** @ 5' SANDY-CLAY, reddish brown, moist, stiff	CL	MC	3 8 11	12		18	101			
7.5		@ 7.5' SILTY-SAND, light reddish brown, moist, medium dense		MC	7 14 26	25		5	113			
10			SM	MC	12 16 28	27		5	111			
15		@ 15' SAND, light gray, moist, dense, fine grained		MC	10 28 46	45		3	103			
20		∇ @ 20' becomes saturated, medium dense, some thin clay layers; seepage encountered	SP	MC	10 14 16	18		17	98			
25				MC	11 22 30	32		16	108			

(Continued Next Page)

BORING NUMBER B-5

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-5 continued...*										
29			SP									
30		@ 30' CLAY, gray-brown, moist, stiff		MC	7 7 10	10		20	103			
31												
32												
33			CL									
34												
35		@ 35' increase in silt content, becomes very stiff		MC	7 10 18	17		23	104			
36												

Total Depth: 36.5-feet
Seepage encountered at 20'

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-6 / P-1

PAGE 1 OF 1

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DATE STARTED 7/16/15 **COMPLETED** 7/16/15

GROUND ELEVATION

BORING DIAMETER 6-inch

EQUIPMENT / RIG CalPac CME B63

HAMMER EFFICIENCY (%) 67

METHOD 140 pound Auto-hammer

SPT CORRECTION 1.12 **CAL CORRECTION** 0.61

LOGGED BY ML **CHECKED BY**

GROUNDWATER DEPTH (ft) Not Encountered

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	■	**ASPHALT (6") / BASE (5")**										
1 2 3 4	▩	**FILL** SILTY-SAND, reddish brown, moist, medium dense	SM									
5 6 7 8 9	░	**OLD PARALIC DEPOSITS** @ 5' SAND, reddish brown, wet, medium dense, fine to medium grained	SP	MC	8 14 16	18		10	112			
10 11 12	░	@ 10' SAND, dark reddish brown, wet, medium dense, fine to coarse grained	SW	MC	9 15 20	21		3	136			
13												

Total Depth: 13-feet
No Groundwater Encountered
Percolation Test Performed

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-7 / P-2

PAGE 1 OF 1

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/16/15
COMPLETED 7/16/15
GROUND ELEVATION
BORING DIAMETER 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12
CAL CORRECTION 0.61
LOGGED BY ML
CHECKED BY
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	■	**ASPHALT (6") / BASE (5")**										
1	▦	**FILL** SILTY-SAND, reddish brown, moist, medium dense	SM									
2												
3												
4												
5	▨	**OLD PARALIC DEPOSITS** @ 5' CLAYEY-SAND, reddish brown, moist, medium dense, fine grained	SC	MC	5 9 13	14		4	112			
6												
7												
8												
9												
10	░	@ 10' SAND, yellow brown, damp, medium dense, fine grained	SP	MC	6 15 18	20		3	106			
11												
12												

Total Depth: 12-feet
No Groundwater Encountered
Percolation Test Performed

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-8

PAGE 1 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DATE STARTED 10/10/16 **COMPLETED** 10/10/16

GROUND ELEVATION 51 feet **BORING DIAMETER** 6-inch

EQUIPMENT / RIG CalPac CME B61

HAMMER EFFICIENCY (%) 67

METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer

SPT CORRECTION 1.12 **CAL CORRECTION** 0.61

LOGGED BY BM **CHECKED BY**

GROUNDWATER DEPTH (ft) Not Encountered

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 7-INCH BASE										
1		**FILL** CLAY, dark brown, moist, soft	CL									
2		**OLD PARALIC DEPOSITS (Qopfa)** @ 2.5' CLAYEY-SAND, orange-brown, fine to medium-grained, moist, loose										
3				BULK								
4												
5			SC	MC	5 9 12	13						
6												
7		@ 7.5' SAND, light orange-brown, fine to medium-grained, damp, medium dense		MC	8 14 21	21						
8			SP									
9												
10		@ 10' SAND, light orange, well-sorted grains with some gravel, damp, medium dense		MC	12 18 26	27						
11												
12			SW									
13												
14												
15		@ 15' CLAY, orange-brown and gray, moist, stiff		MC	4 7 14	13						
16												
17												
18												
19												
20			CL	MC	5 8 12	12						
21												
22												
23												
24												
25		@ 25' SAND, light orange, fine to medium-grained, moist, medium dense		MC	16 28 40	42						
26	∇	@ 26' Perched groundwater encountered	SP									
27												

(Continued Next Page)

BORING NUMBER B-8

PAGE 2 OF 2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28 29		@ 25' SAND, light orange, fine to medium-grained, moist, medium dense*(continued)*	SP									
30 31 32 33 34		@ 30' SAND, light gray, well-sorted grains, wet, medium dense	SW	MC	19 33 42	46						
35 36 37 38 39		@ 35' SAND, dark gray, fine-grained, wet, medium dense	SP	MC	11 37 50	53						
40 41 42 43 44		@ 40' CLAY, dark gray, high plasticity, saturated, medium stiff	CH	MC	2 4 6	6						
45 46 47 48 49		@ 45' Becomes soft		SPT	1 2 2	4						
50 51				SPT	1 2 2	4						

Total depth: 51.5-feet
Perched groundwater encountered at 26-feet
Boring backfilled on 10/10/2016

BORING NUMBER B-9/P-3

PAGE 1 OF 1

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DATE STARTED 10/10/16 **COMPLETED** 10/10/16

GROUND ELEVATION 51 feet **BORING DIAMETER** 8-inch

EQUIPMENT / RIG CalPac CME B61

HAMMER EFFICIENCY (%) 67

METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer

SPT CORRECTION 1.12 **CAL CORRECTION** 0.61

LOGGED BY BM **CHECKED BY**

GROUNDWATER DEPTH (ft) Not Encountered

NOTES

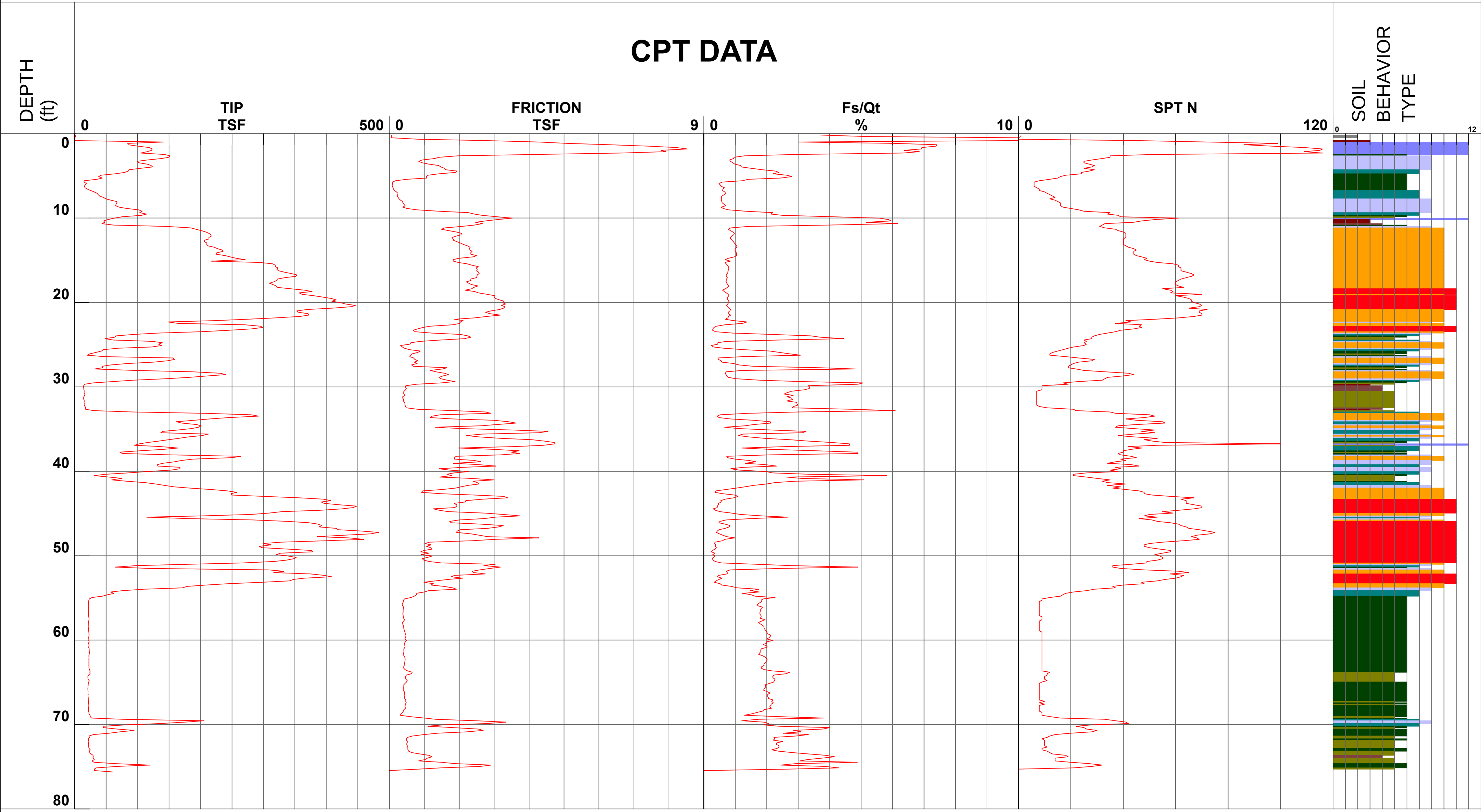
DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	■	5-INCH A/C OVER 4-INCH BASE										
1	▨	**FILL** CLAY, dark orange-brown, moist, medium stiff	CL									
2 3 4	▨	**OLD PARALIC DEPOSITS (Qopfa)** @ 2' SANDY-CLAY, orange-brown, moist, stiff	SC									
5 6 7 8	▒	@ 5' SAND with SILT, light orange-brown, fine to medium-grained, moist, medium dense	SP-SM									
9	▒			☒ SPT	7 11 14	28						
10												

Total depth: 10-feet
No groundwater encountered
Percolation test performed
Boring backfilled on 10/10/2016

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(685).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	7/21/2015 8:28:08 AM	Maximum Depth	75.62 ft
EST GW Depth During Test		23.00 ft			

Net Area Ratio .8



■ 1 - sensitive fine grained

■ 2 - organic material

■ 3 - clay

■ 4 - silty clay to clay

■ 5 - clayey silt to silty clay

■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt

■ 8 - sand to silty sand

■ 9 - sand

■ 10 - gravelly sand to sand

■ 11 - very stiff fine grained (*)

■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

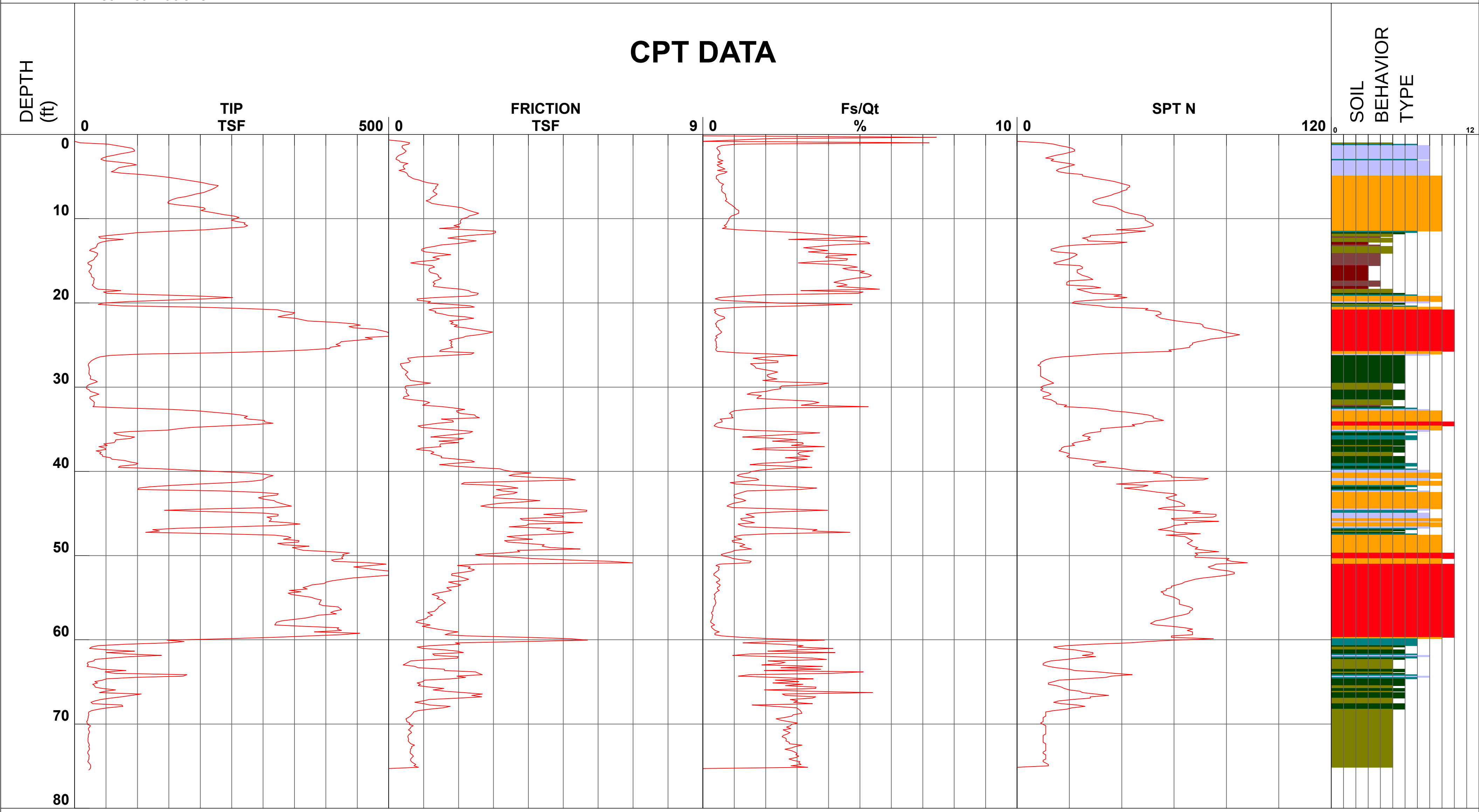
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(686).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	7/21/2015 9:44:18 AM	Maximum Depth	75.46 ft
EST GW Depth During Test		23.00 ft			

Net Area Ratio .8

CPT DATA



⬜ 1 - sensitive fine grained
⬛ 2 - organic material
🟥 3 - clay
🟫 4 - silty clay to clay
🟩 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt
🟦 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand
🟥 10 - gravelly sand to sand
🟦 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

EEI

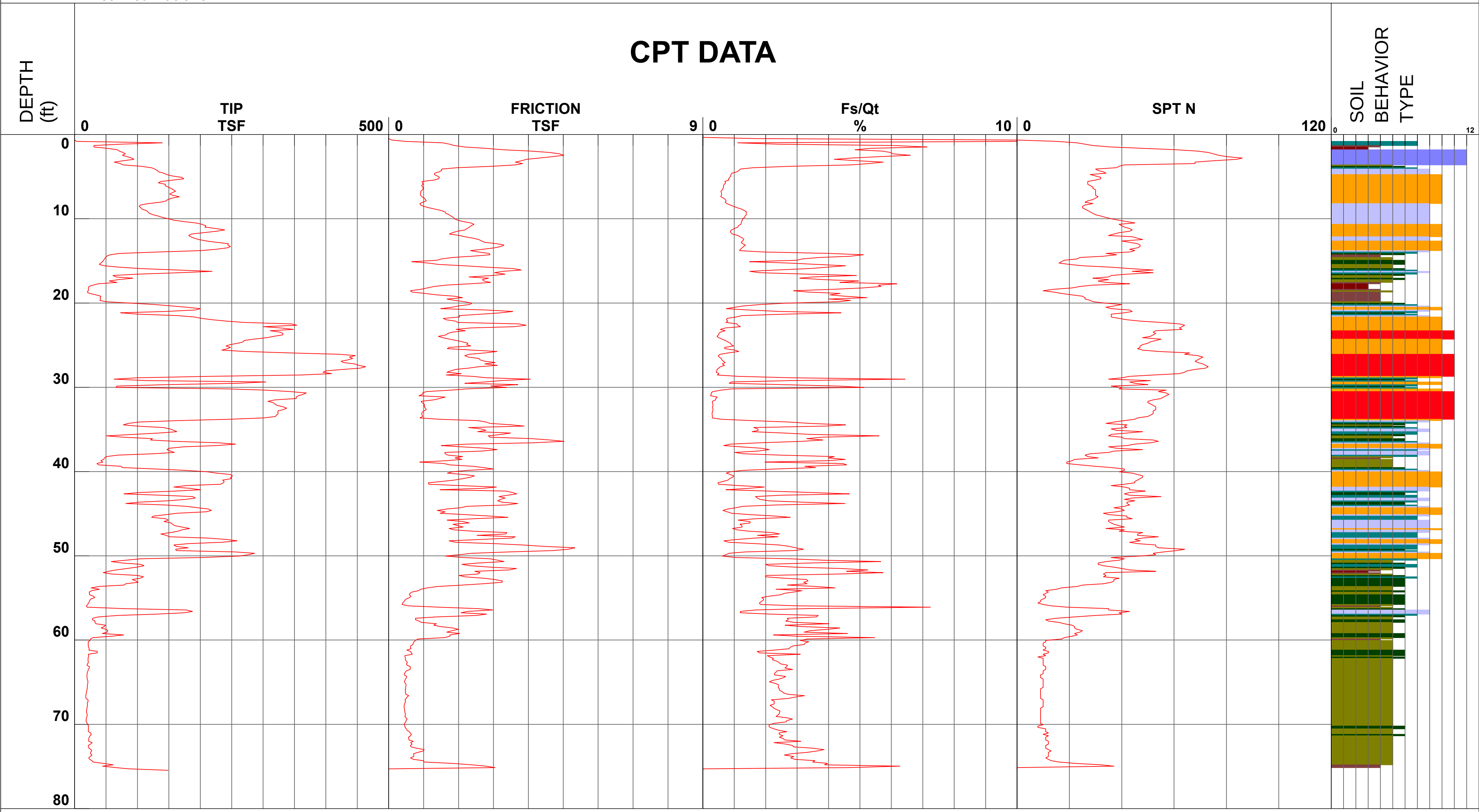
Project The Koll Center Residences
Job Number SHO-72189.4a
Hole Number CPT-03
EST GW Depth During Test 23.00 ft

Operator RC-BH
Cone Number DDG1281
Date and Time 7/21/2015 10:50:47 AM

Filename SDF(687).cpt
GPS
Maximum Depth 75.46 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

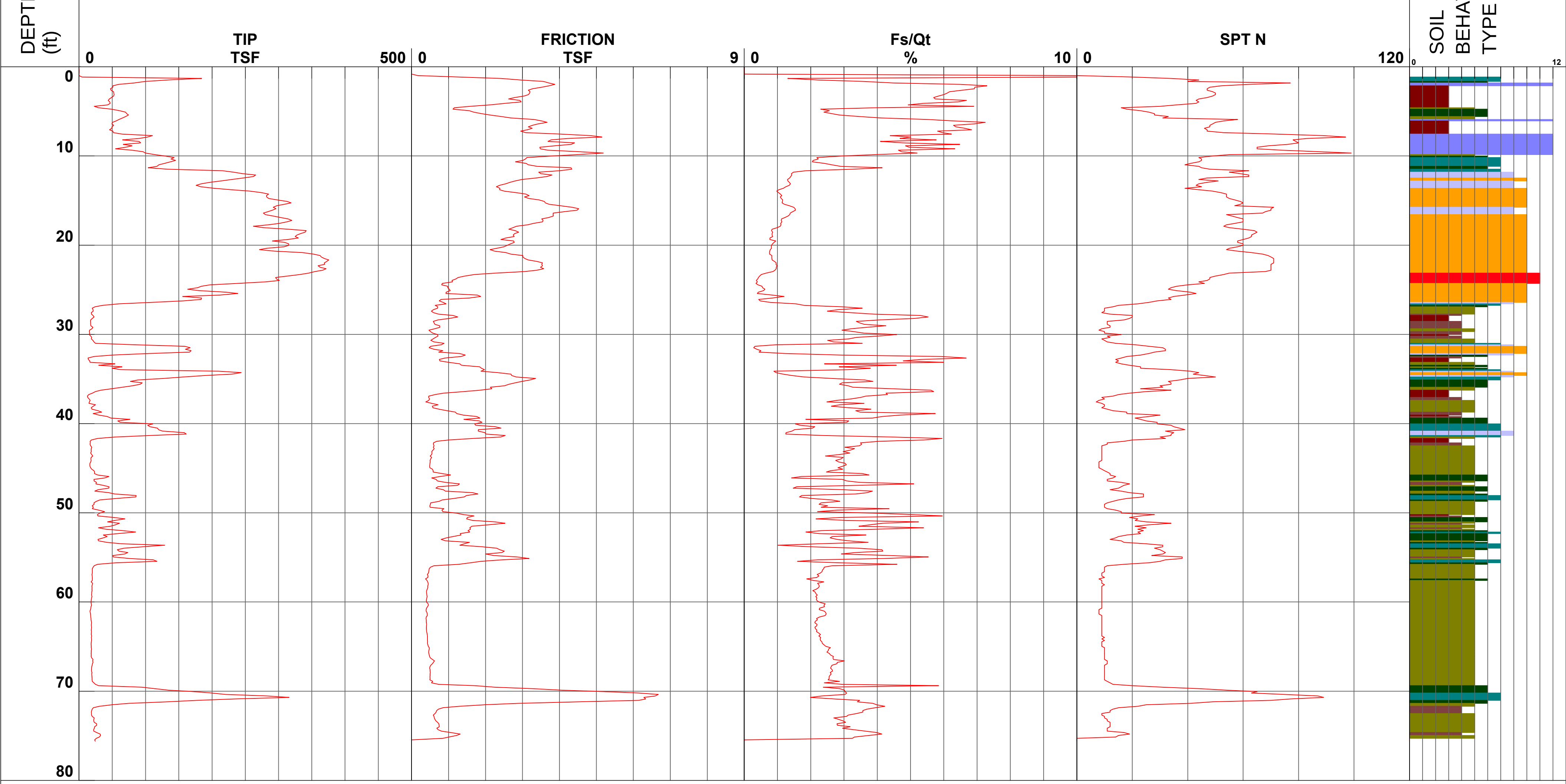
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project: The Koll Center Residences
Job Number: SHO-72189.4a
Hole Number: CPT-04
EST GW Depth During Test: 23.00 ft
Operator: RC-BH
Cone Number: DDG1281
Date and Time: 7/21/2015 11:49:48 AM
Filename: SDF(688).cpt
GPS:
Maximum Depth: 75.62 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

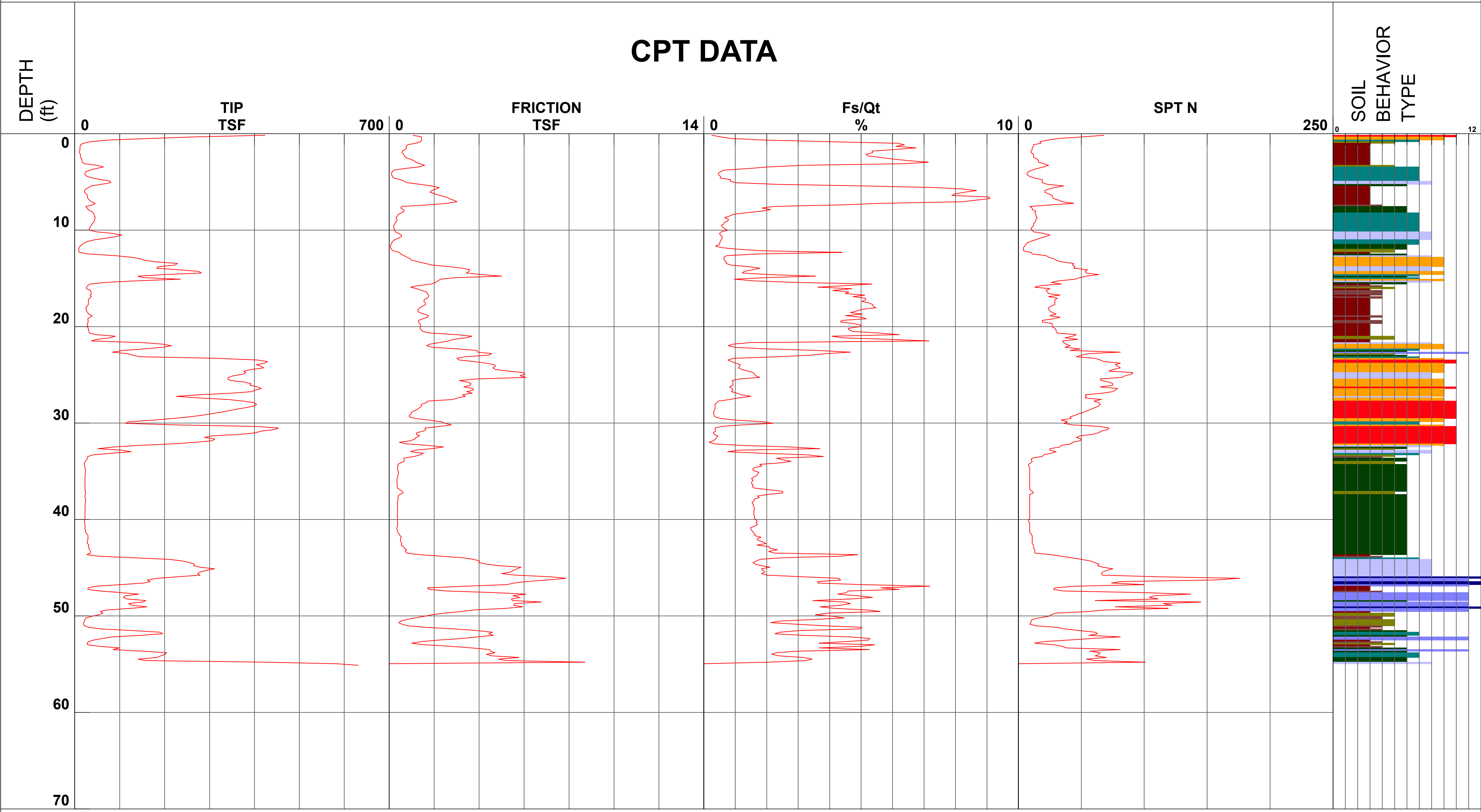
S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth
Geo Testing Inc.

EEI

Project The Koll Center Residences
Job Number SHO-72189.4a
Hole Number CPT-05
EST GW Depth During Test 26.00 ft
Operator DG-RC
Cone Number DDG1281
Date and Time 10/18/2016 9:56:17 AM
Filename SDF(195).cpt
GPS
Maximum Depth 55.12 ft

Net Area Ratio .8



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

Cone Size 10cm squared

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

EEI

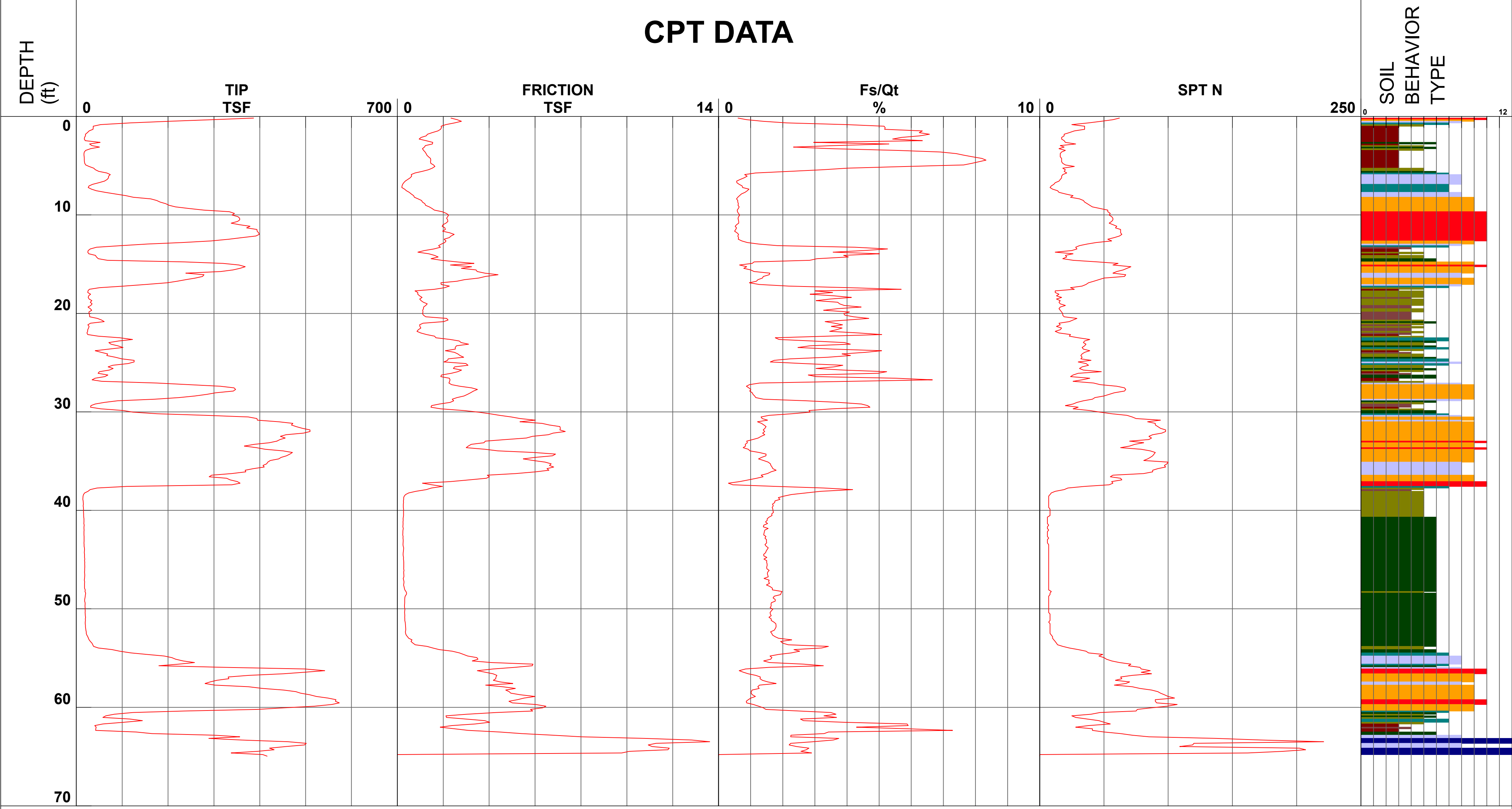
Project The Koll Center Residences
Job Number SHO-72189.4a
Hole Number CPT-06
EST GW Depth During Test 26.00 ft

Operator DG-RC
Cone Number DDG1281
Date and Time 10/18/2016 10:49:31 AM

Filename SDF(196).cpt
GPS
Maximum Depth 64.96 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

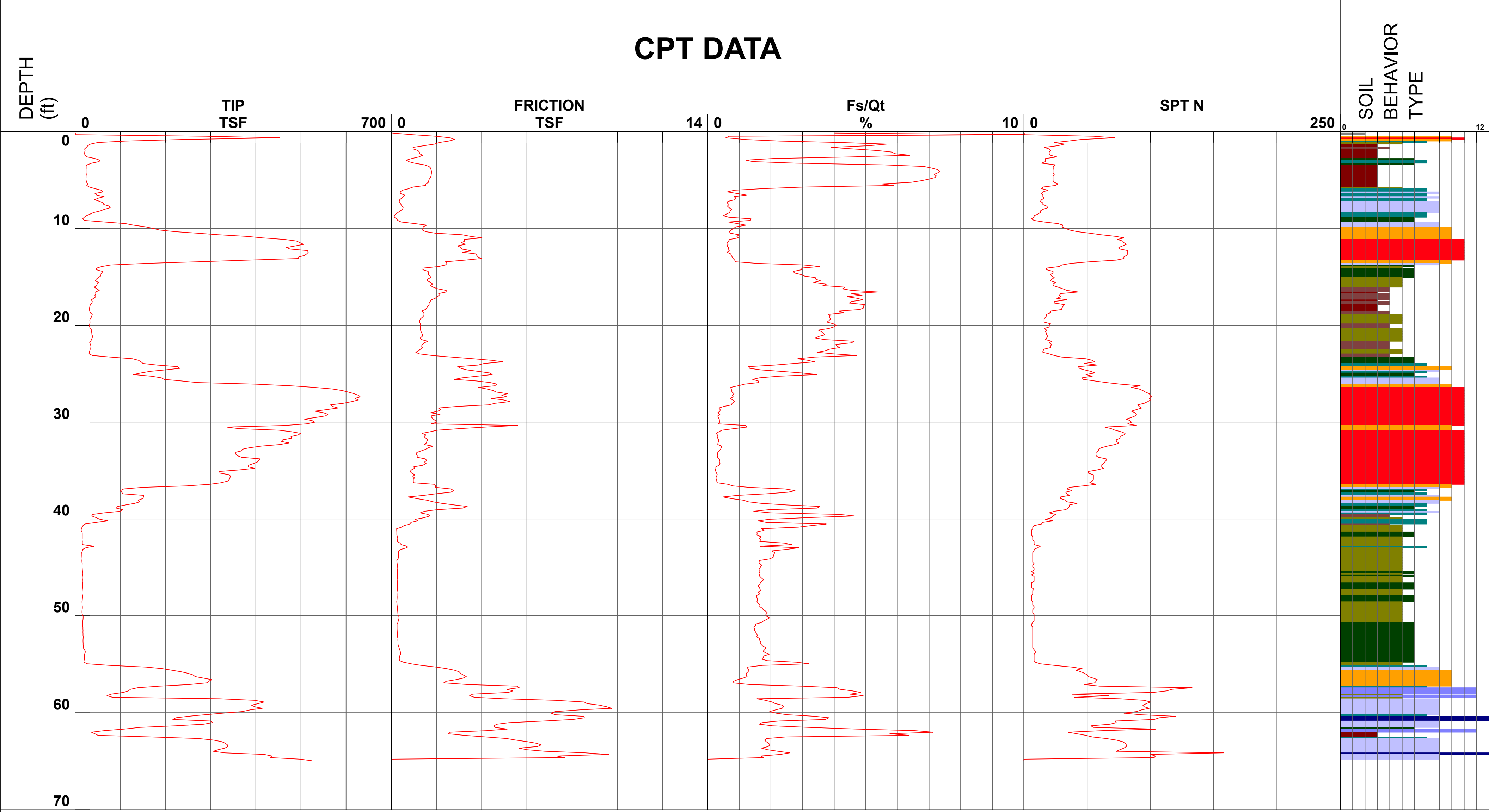
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Middle Earth Geo Testing Inc.

Project The Koll Center Residences
Job Number SHO-72189.4a
Hole Number CPT-07
EST GW Depth During Test 26.00 ft
Operator DG-RC
Cone Number DDG1281
Date and Time 10/18/2016 12:13:26 PM
Filename SDF(197).cpt
GPS
Maximum Depth 64.96 ft

Net Area Ratio .8



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

Cone Size 10cm squared

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

APPENDIX C

SEISMIC SETTLEMENT ANALYSIS

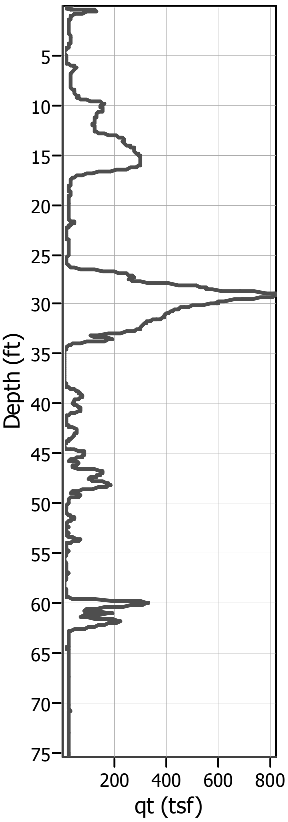
LIQUEFACTION ANALYSIS REPORT

Project title : **Location :**

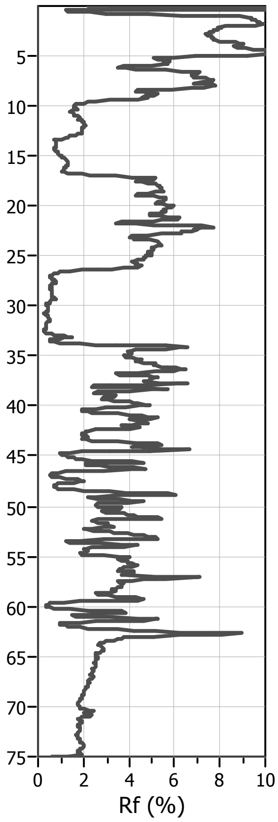
CPT file : CPT-01

Input parameters and analysis data

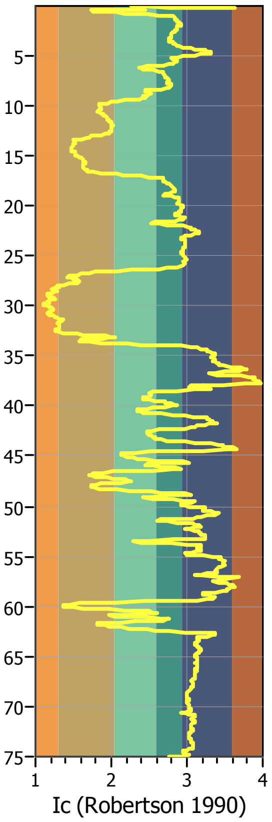
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.96	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



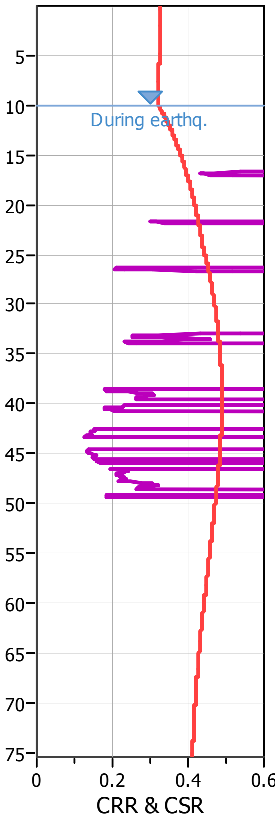
Cone resistance



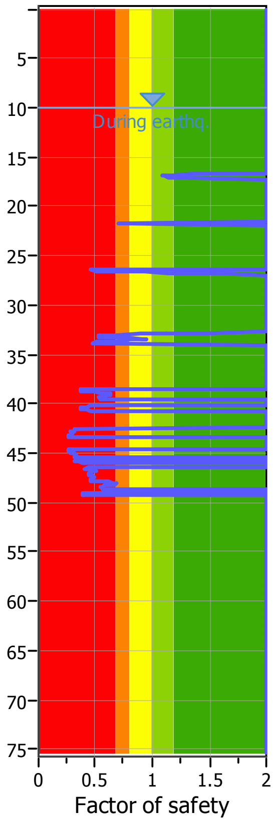
Friction Ratio



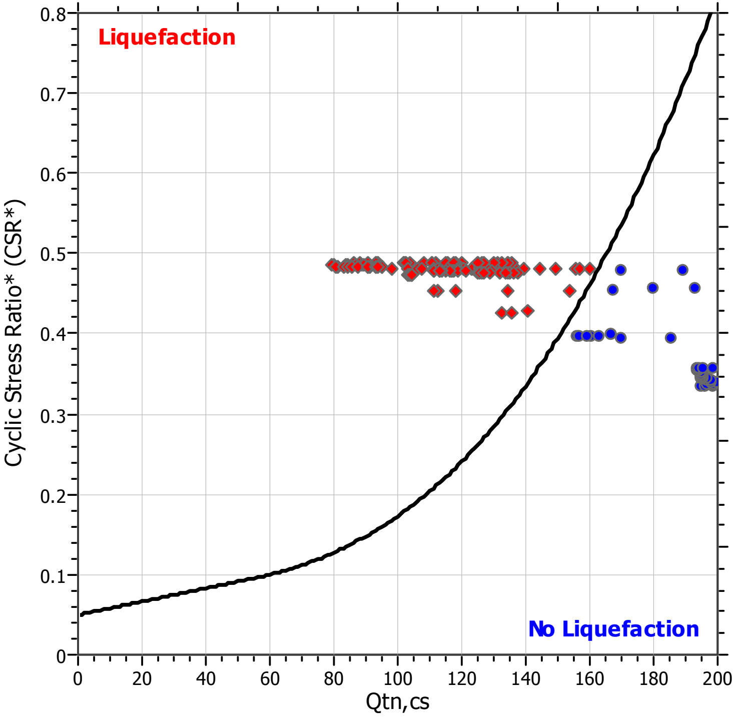
SBTn Plot



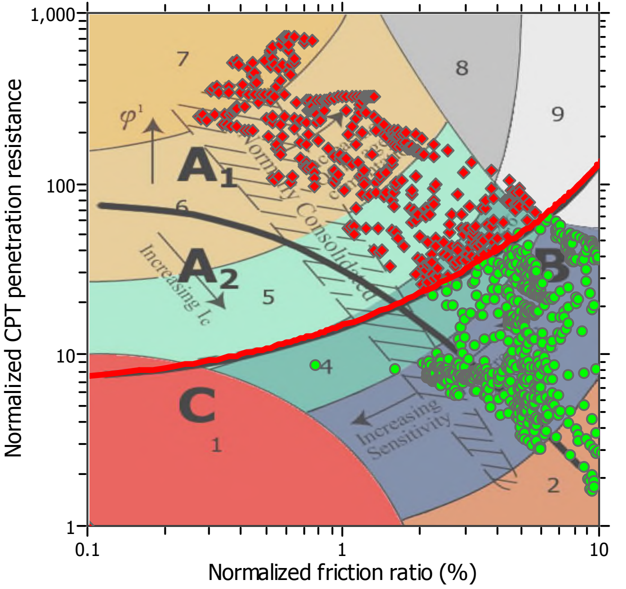
CRR plot



FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential

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

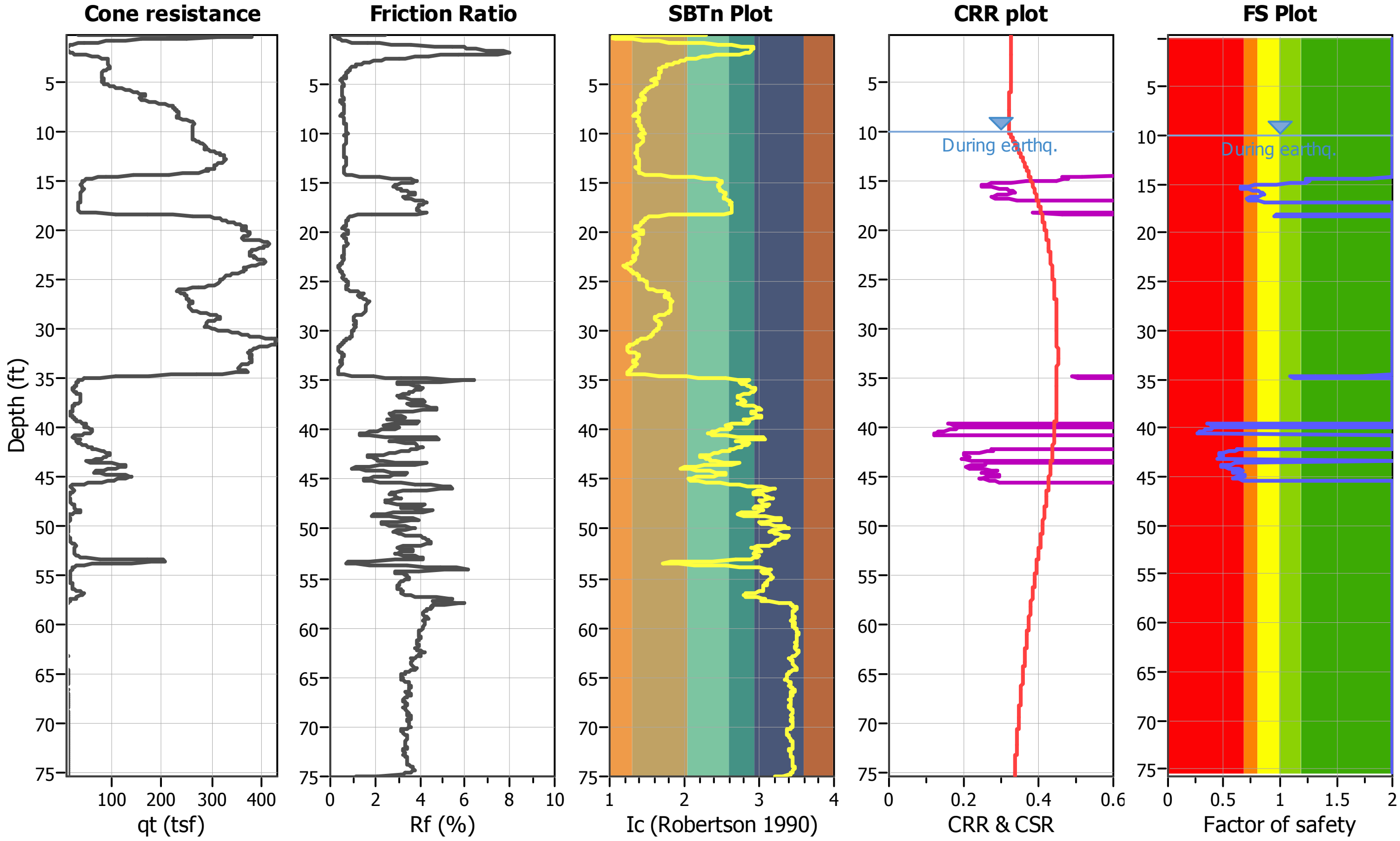
LIQUEFACTION ANALYSIS REPORT

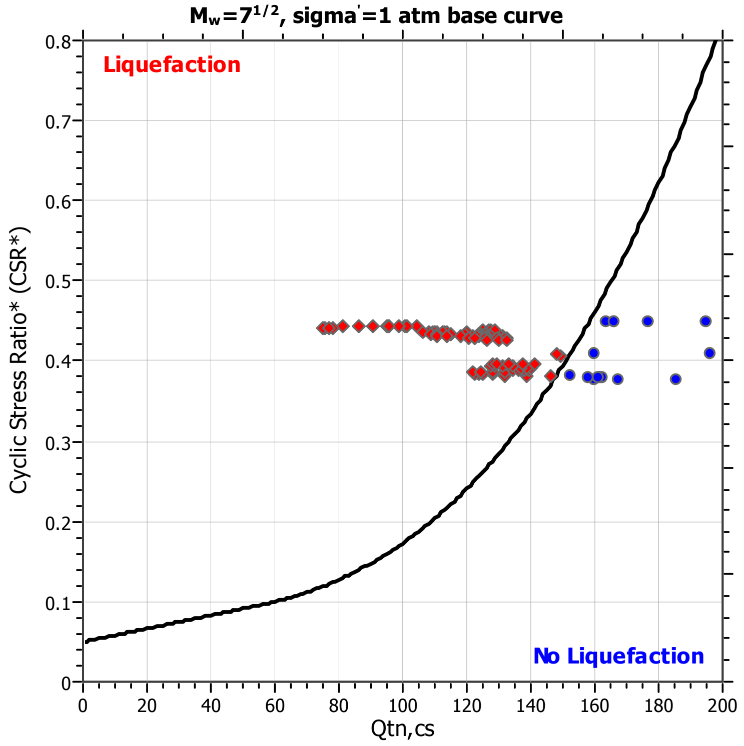
Project title : **Location :**

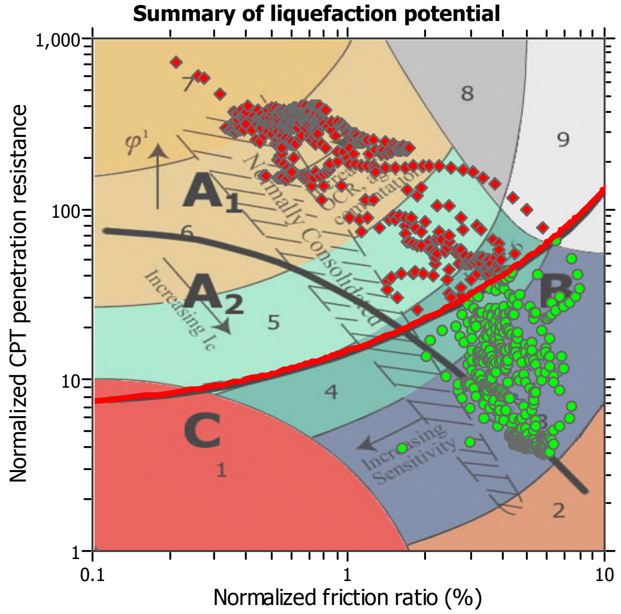
CPT file : CPT-02

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:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.96	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	No		


Cone resistance **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**


$M_w=7^{1/2}$, sigma'=1 atm base curve


Summary of liquefaction potential

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

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www.klingconsultinggroup.com

LIQUEFACTION ANALYSIS REPORT

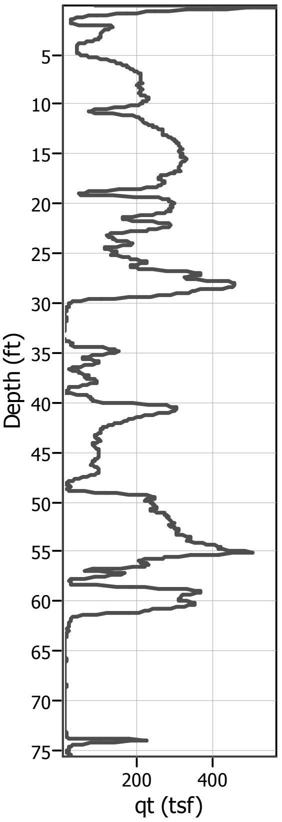
Project title : **Location :**

CPT file : CPT-03

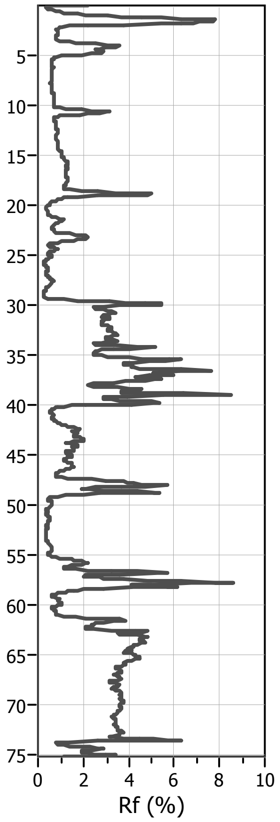
Input parameters and analysis data

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:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.96	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	No		

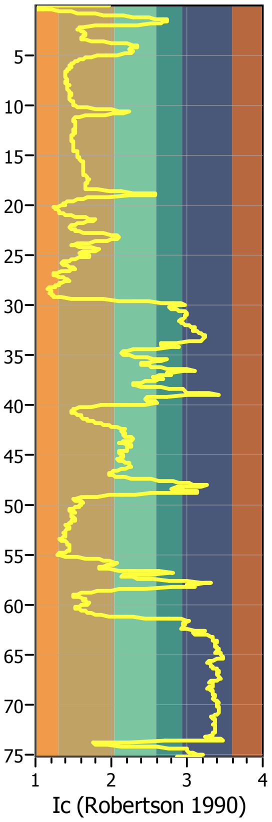
Cone resistance



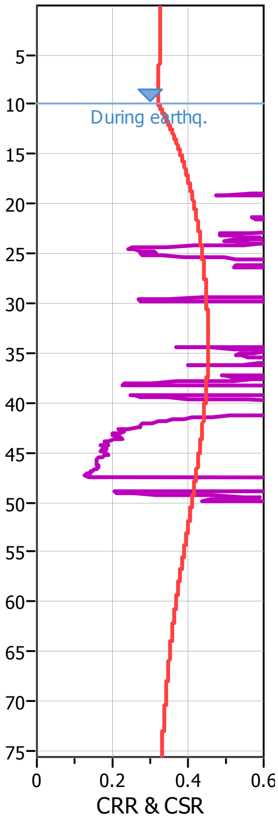
Friction Ratio



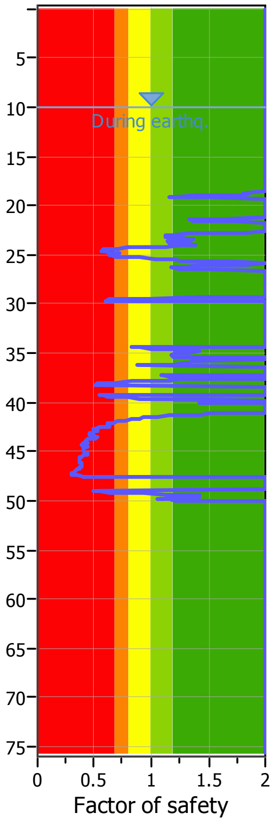
SBTn Plot



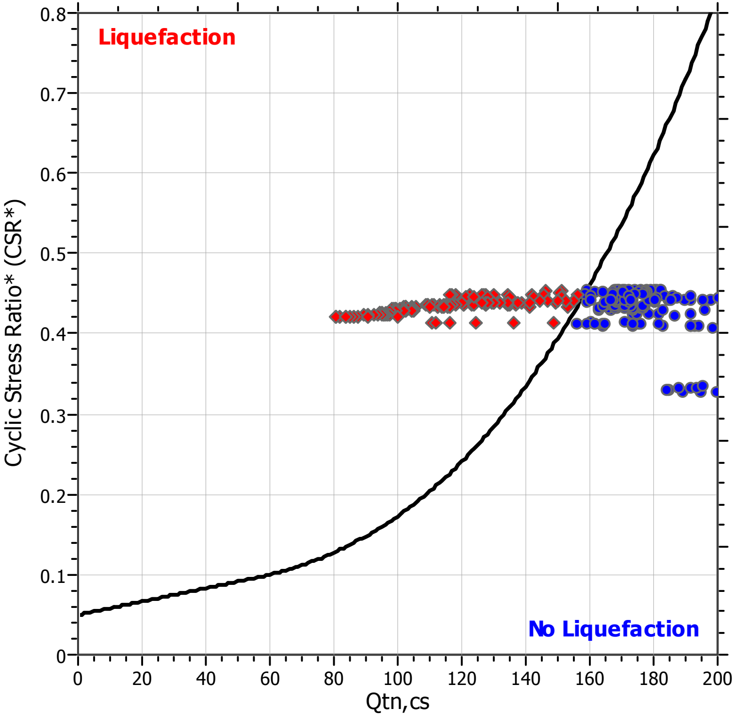
CRR plot



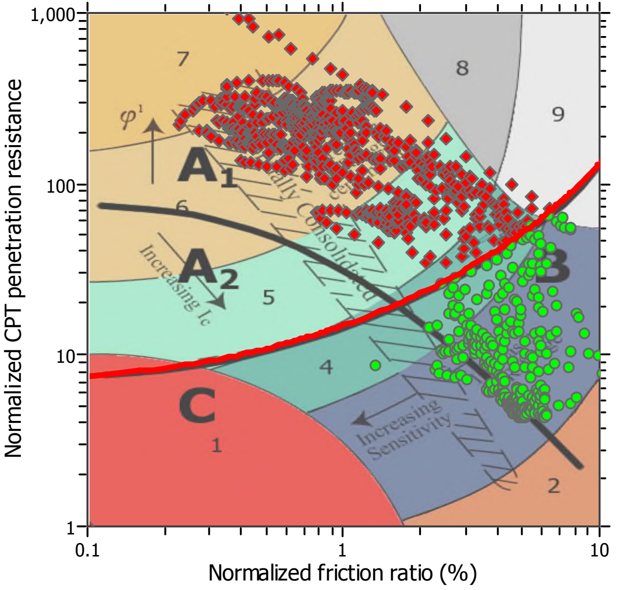
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



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

LIQUEFACTION ANALYSIS REPORT

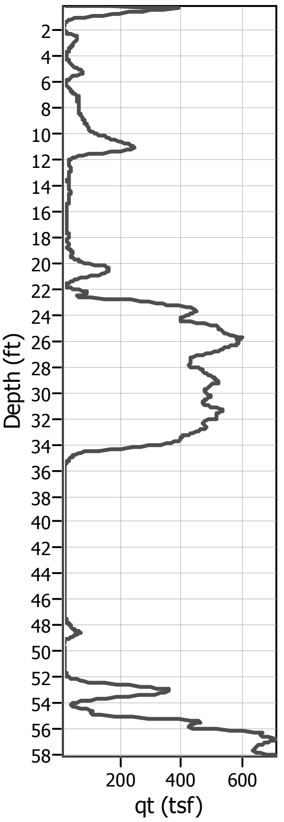
Project title : **Location :**

CPT file : CPT-04

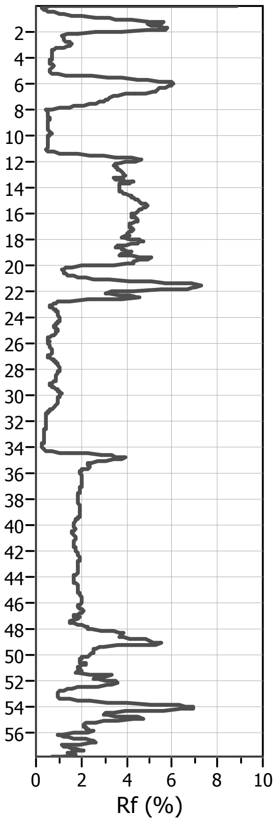
Input parameters and analysis data

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:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.96	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	No		

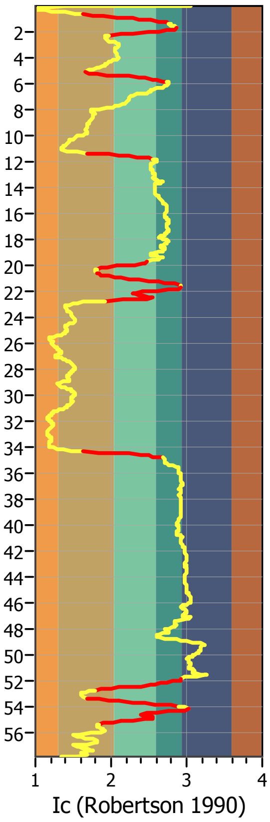
Cone resistance



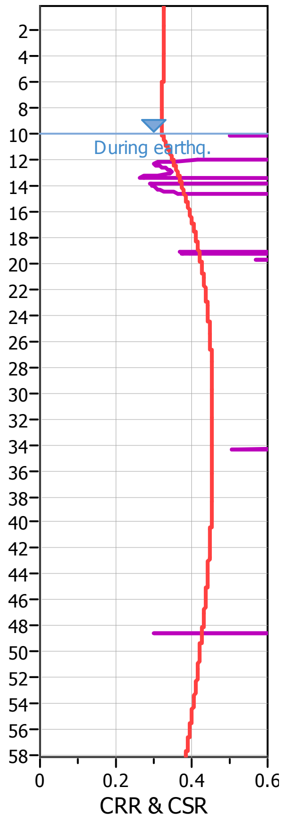
Friction Ratio



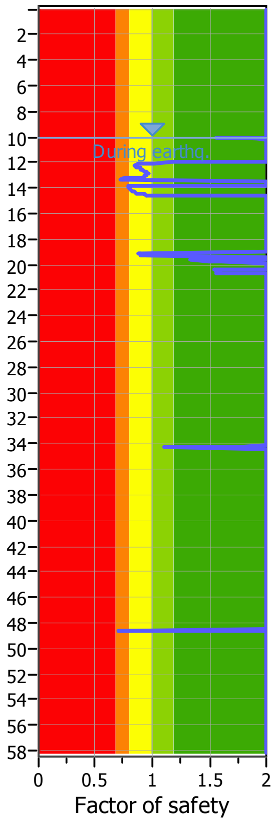
SBTn Plot



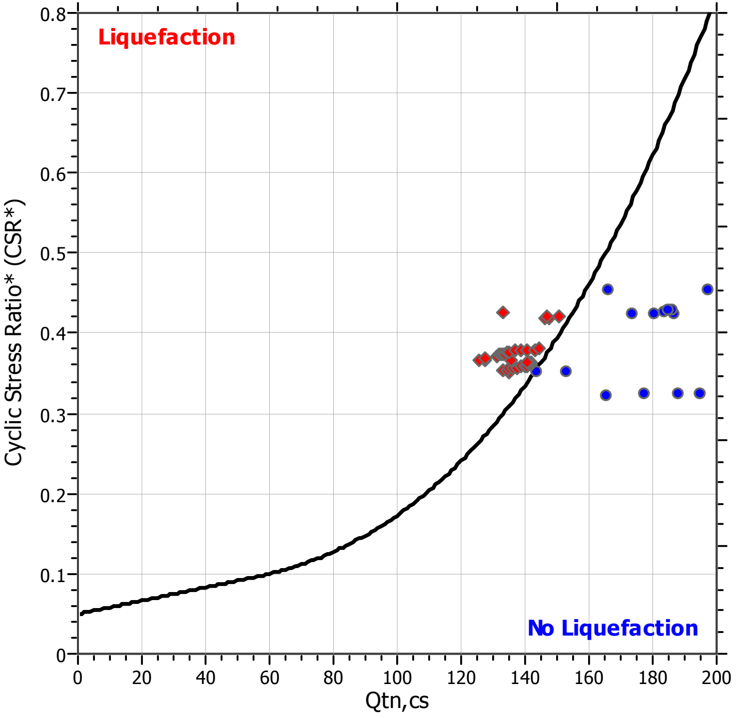
CRR plot



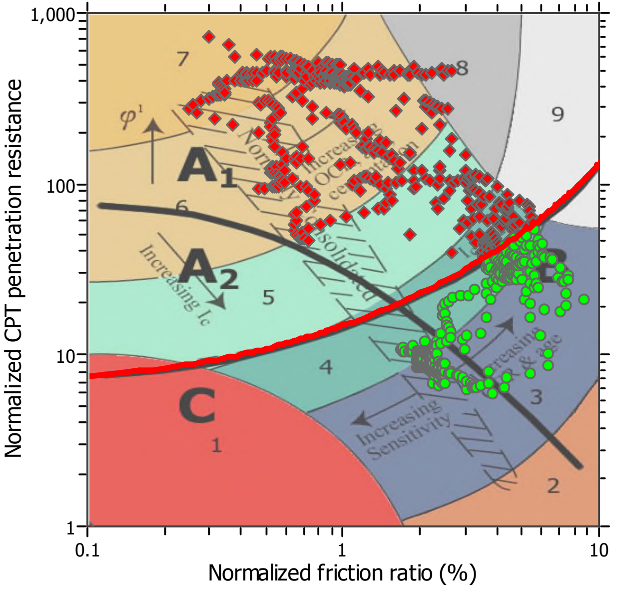
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

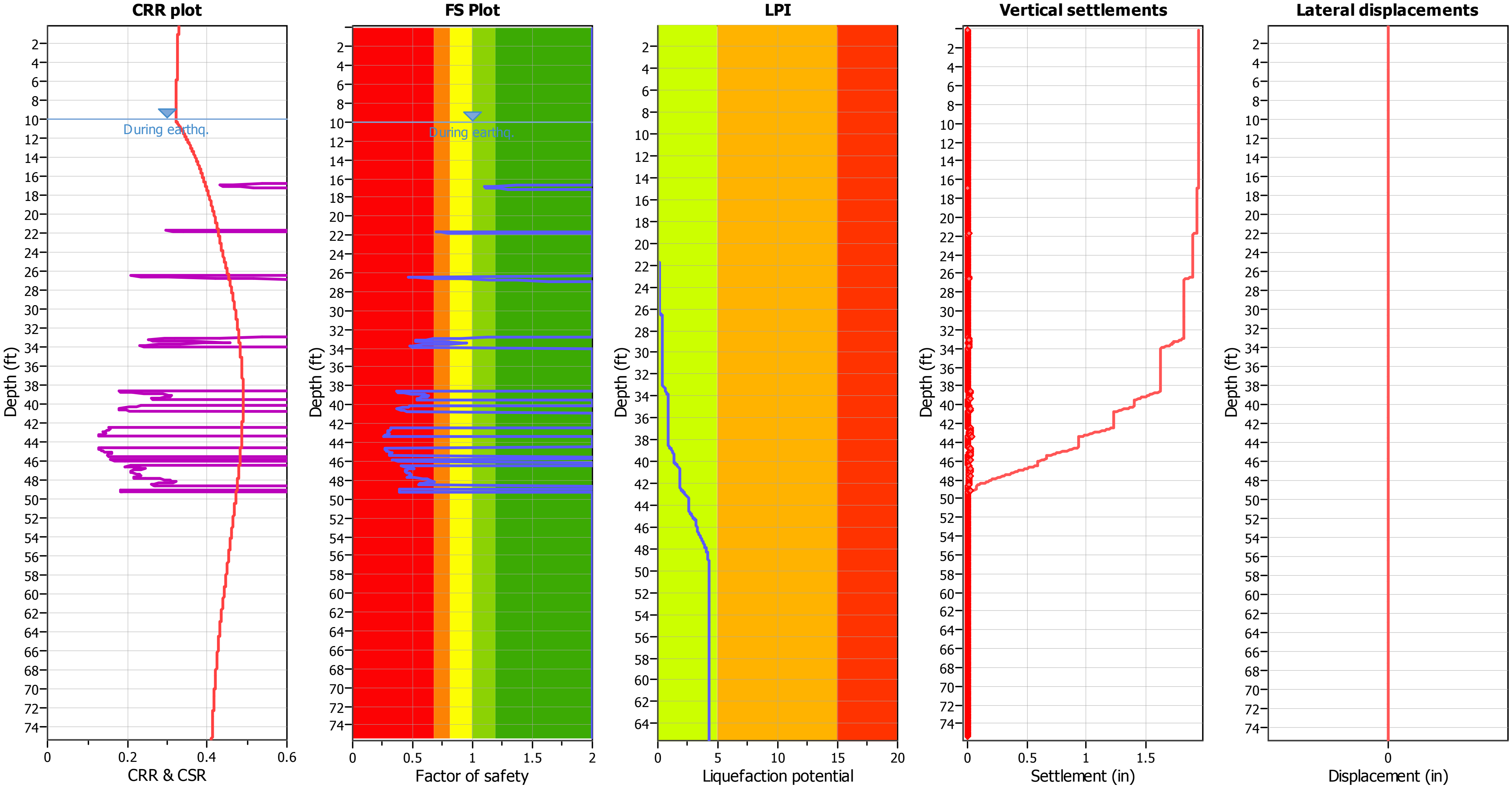


Summary of liquefaction potential



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

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:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.96	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

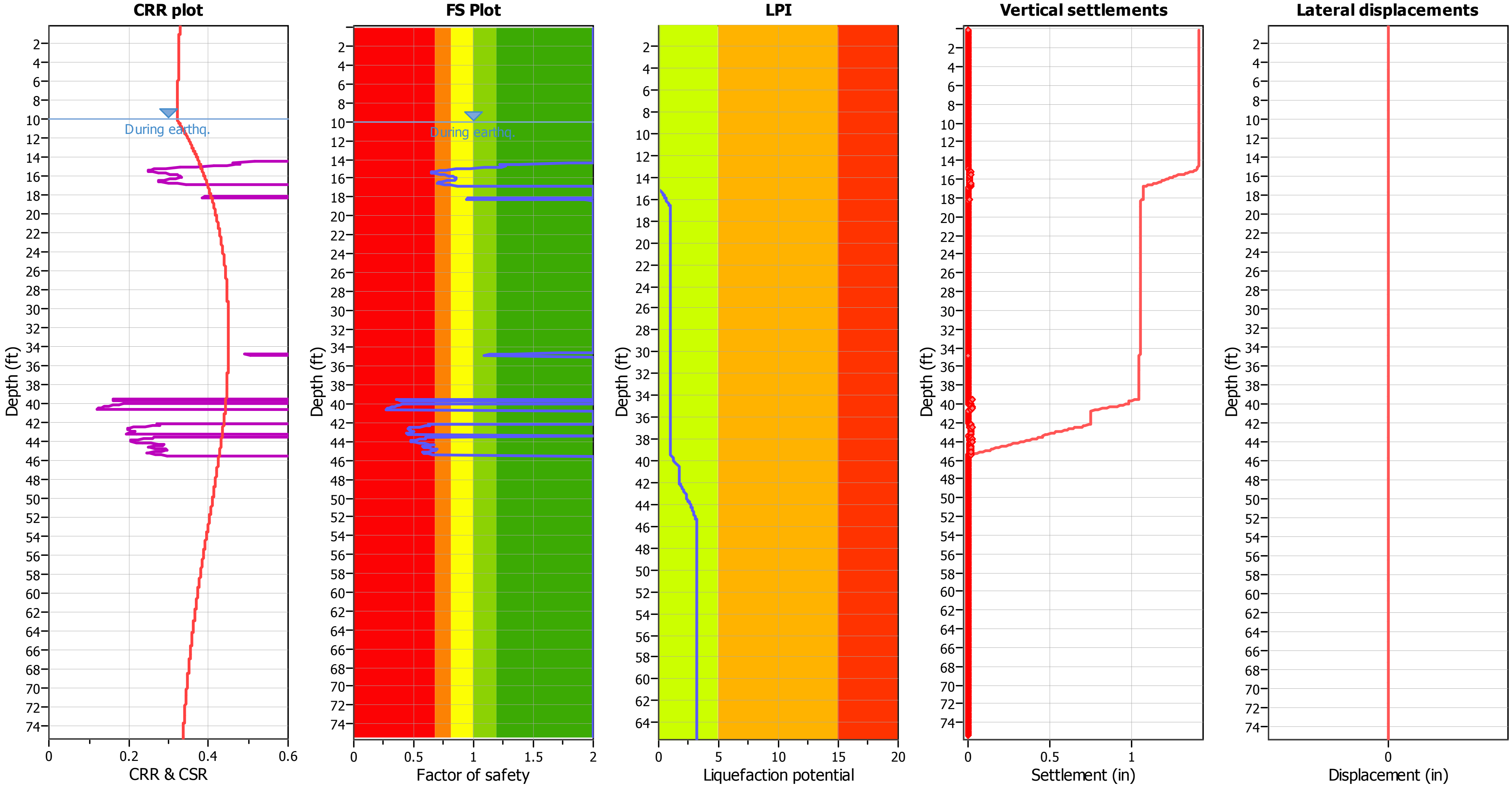
F.S. color scheme

- 🟥 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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.96
Peak ground acceleration: 0.61
Depth to water table (insitu): 10.00 ft

Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: No
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 50.00 ft

F.S. color scheme

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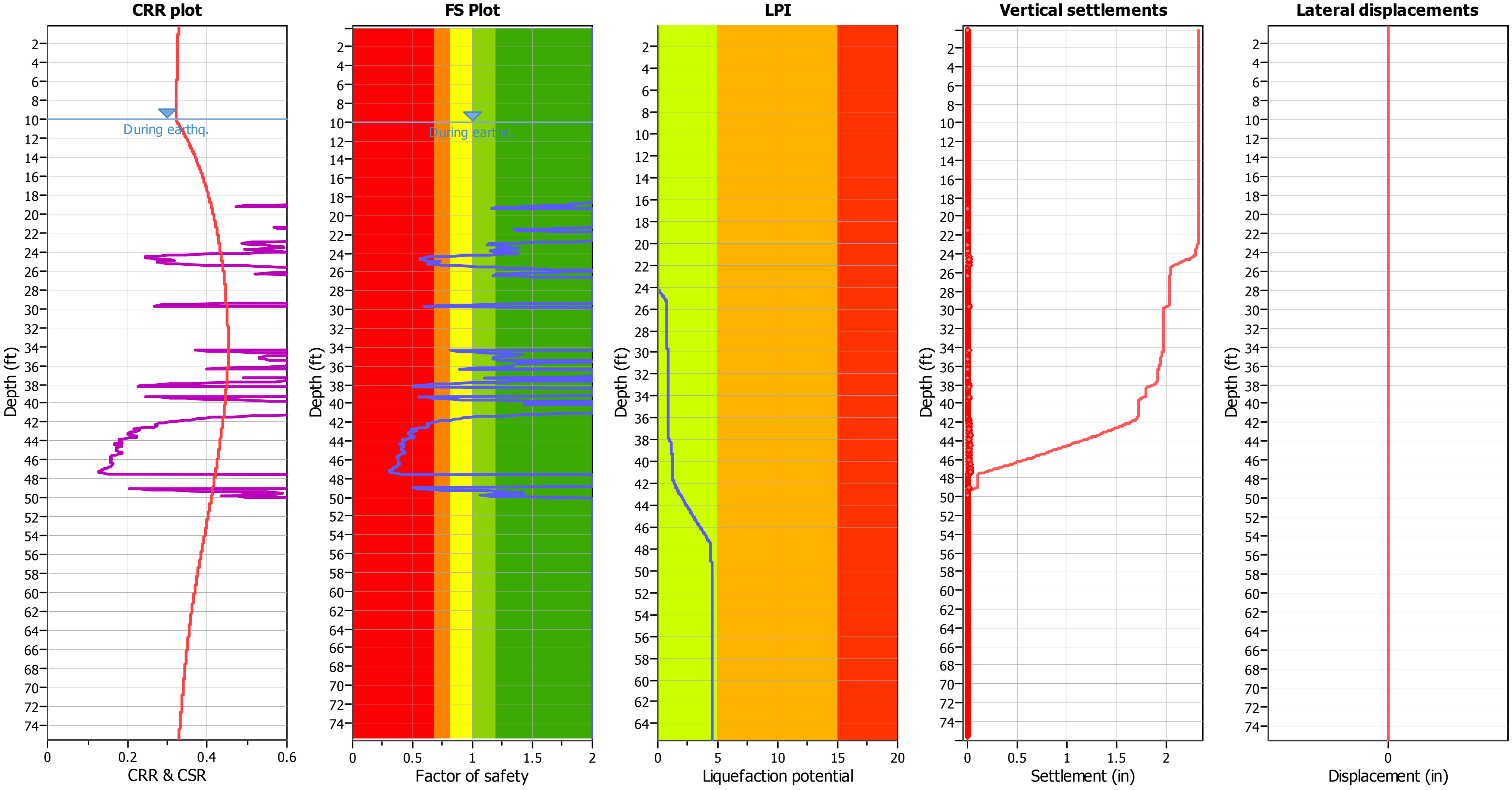
Liquefaction analysis overall plots



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:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.96	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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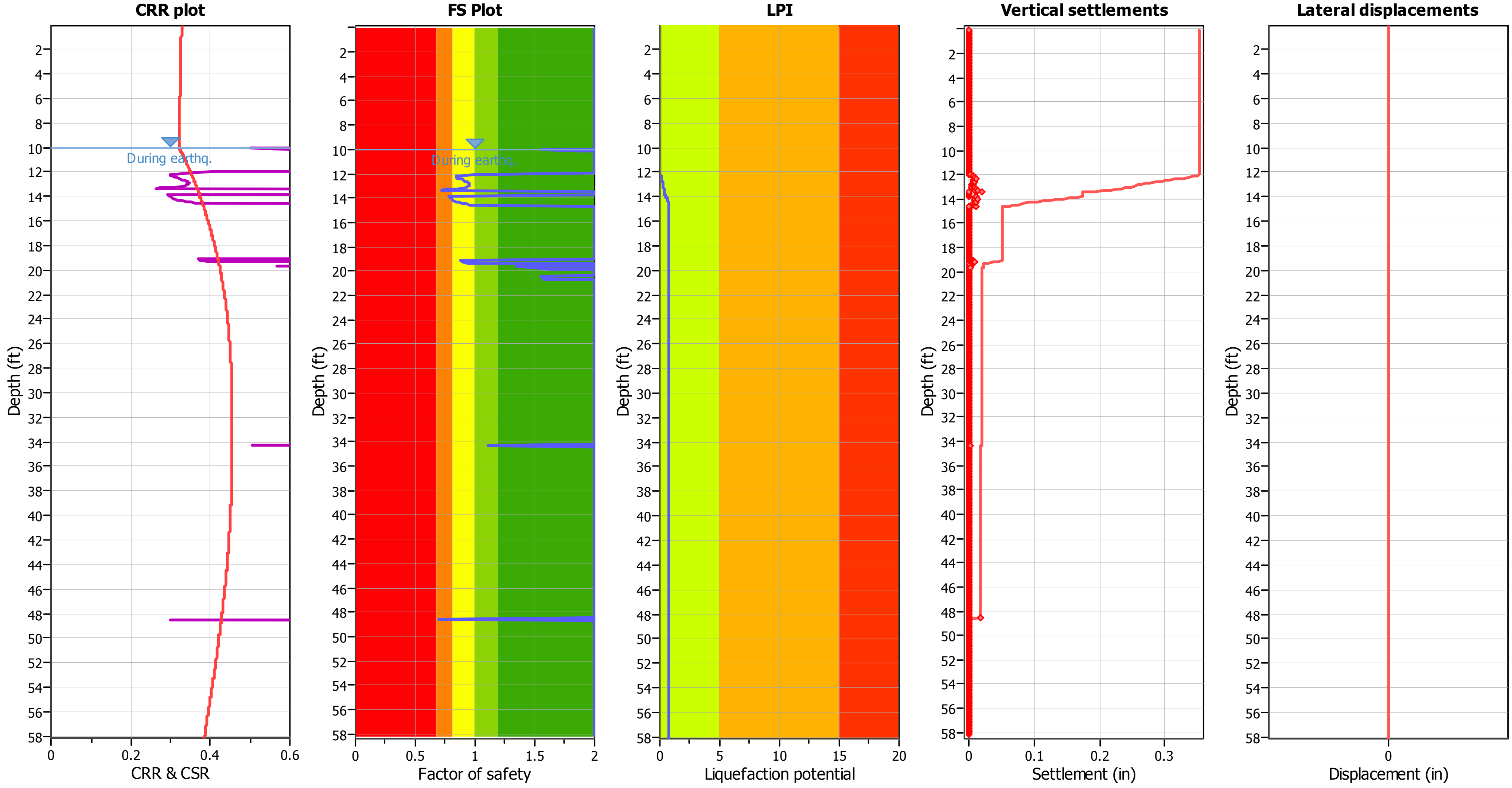
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.96
Peak ground acceleration: 0.61
Depth to water table (insitu): 10.00 ft

Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: No
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 50.00 ft

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
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LPI color scheme

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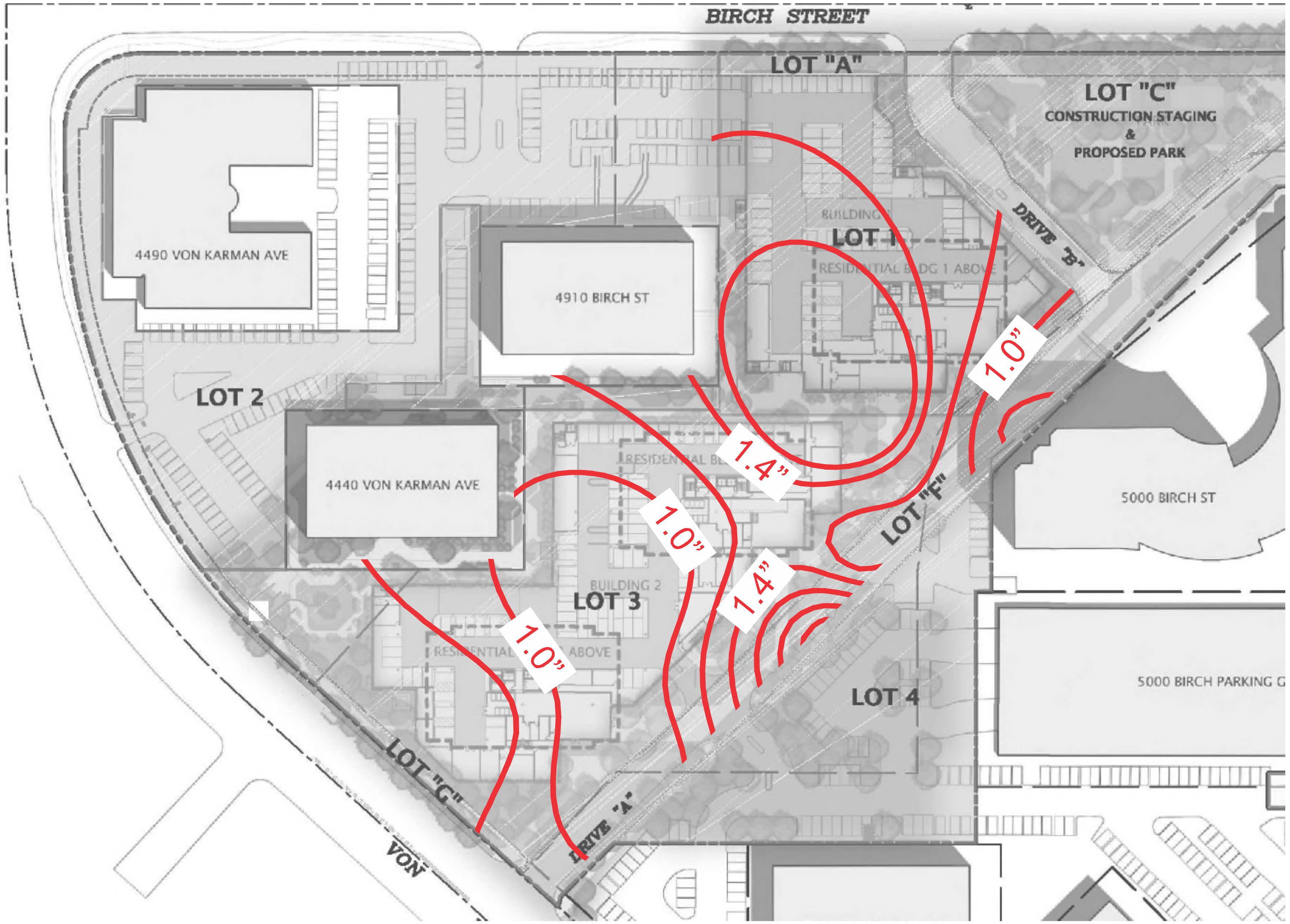

0 120 240

Scale: 1” = 120’

N

Legend

2.0” Approximate Liquefaction Induced Settlement in 0.2 Inches

Reference: David Evans and Associates, Koll Center Residences Newport Phasing, Newport CA, Dated: 2/21/2017

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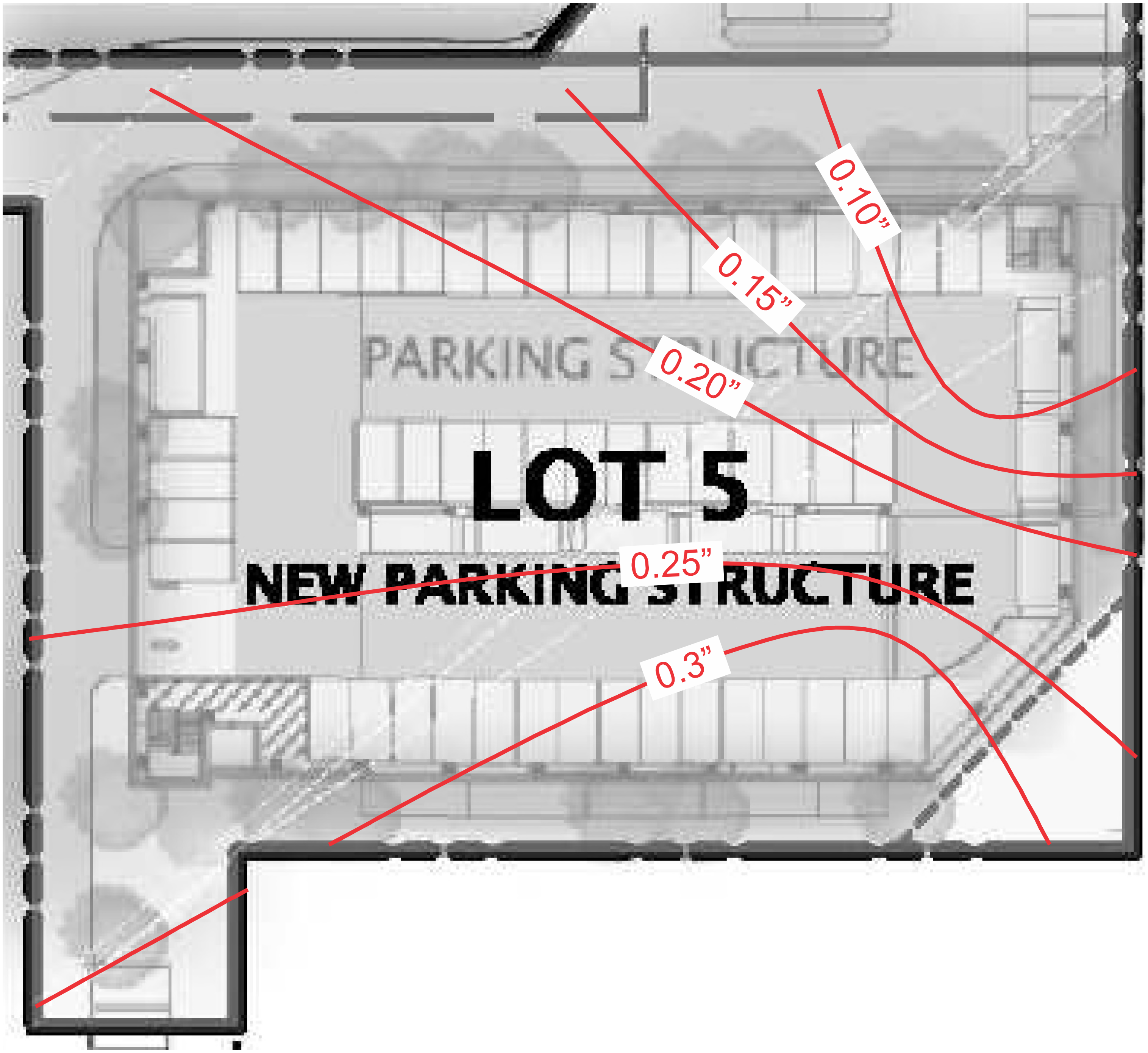
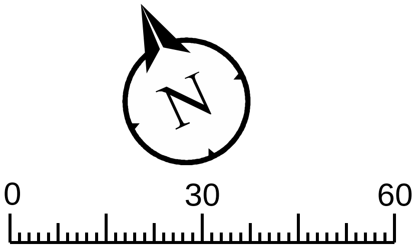
Seismic Induce Settlement Contours
Koll Center Apartment Building
Newport, California

Figure:	B-1
PN:	19017-01
Date:	June, 2019



Legend

0.3” Approximate Liquefaction Induced Settlement in Inches



Scale: 1” = 30’

Reference: David Evans and Associates, Koll Center Residences Newport Phasing, Newport CA, Dated: 2/21/2017

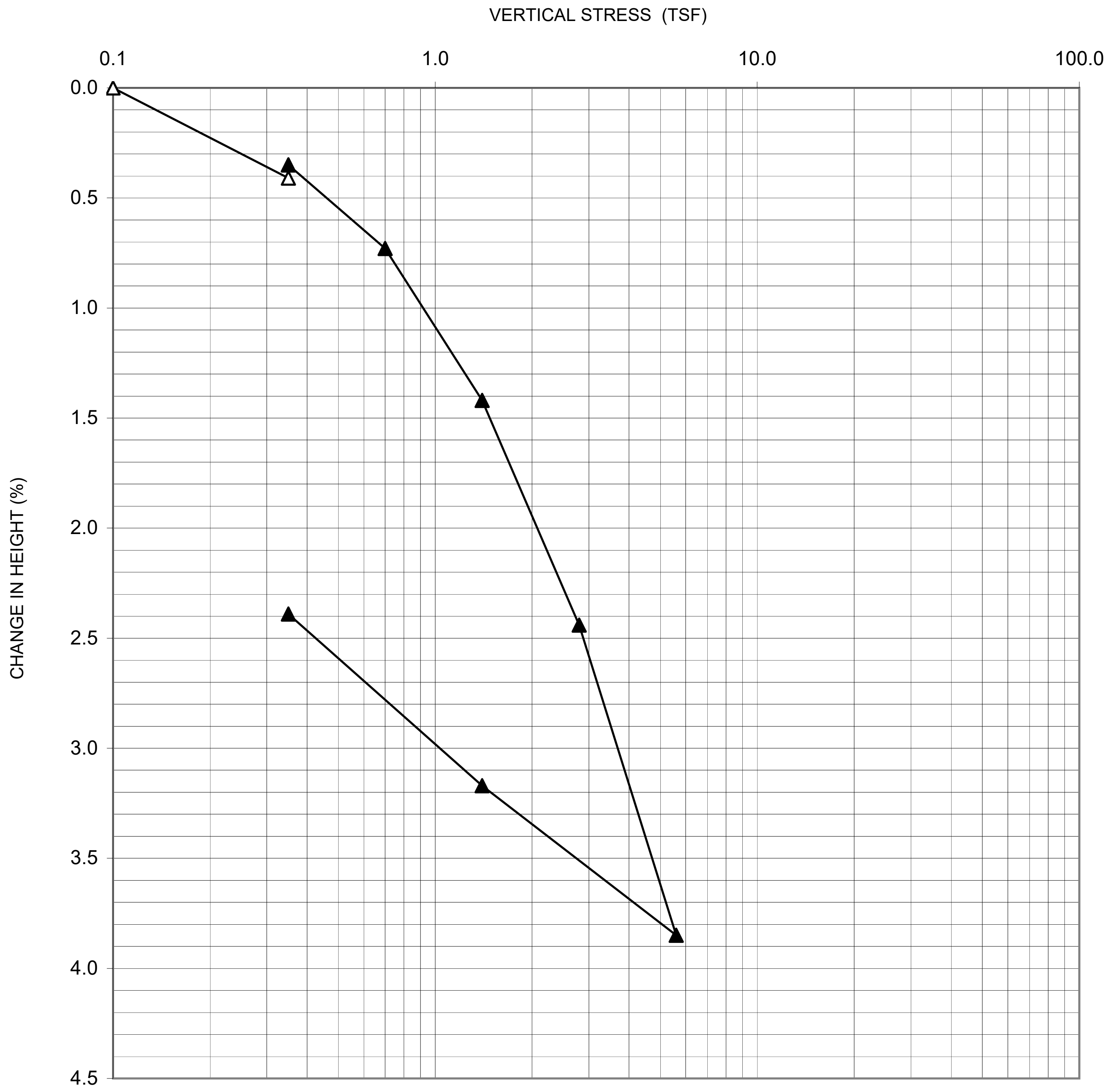
KCG KLING Consulting Group, Inc.

Seismic Induce Settlement Contours
Koll Center Parking Structure
Newport, California

Figure:	B-2
PN:	19017-01
Date:	June, 2019

APPENDIX D

LABORATORY TEST RESULTS



PROJECT NO.: 19017-01

SOIL DESCRIPTIONS: OLIVE BROWN SANDY CLAY (CL)

BORING NO./LOCATION : KB - 1

DEPTH / ELEV. : 15'

LIQUID LIMIT : -

SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	22.1	98.6	85.0	0.697
FINAL	0.9761	27.3	100.9	111.5	0.657

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Irvine, Ca. 92614
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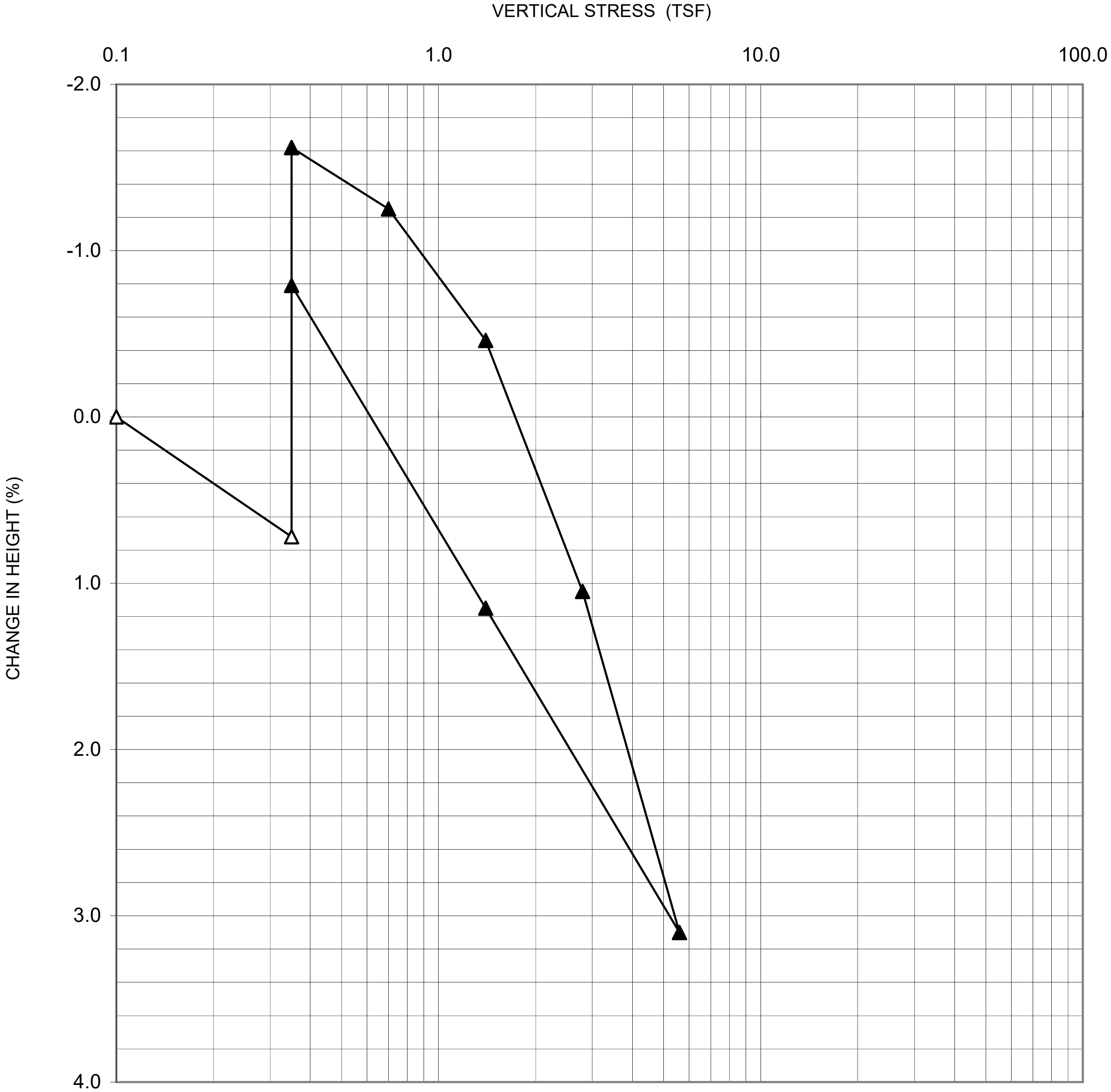
CONSOLIDATION TEST CURVE



PROJECT NO.: 19017-01

SOIL DESCRIPTIONS: BROWN SANDY CLAY (CL)

BORING NO./LOCATION : KB - 2

DEPTH / ELEV. : 12'

LIQUID LIMIT : -

SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	29.2	96.6	106.7	0.732
FINAL	1.0079	32.4	95.8	116.3	0.746

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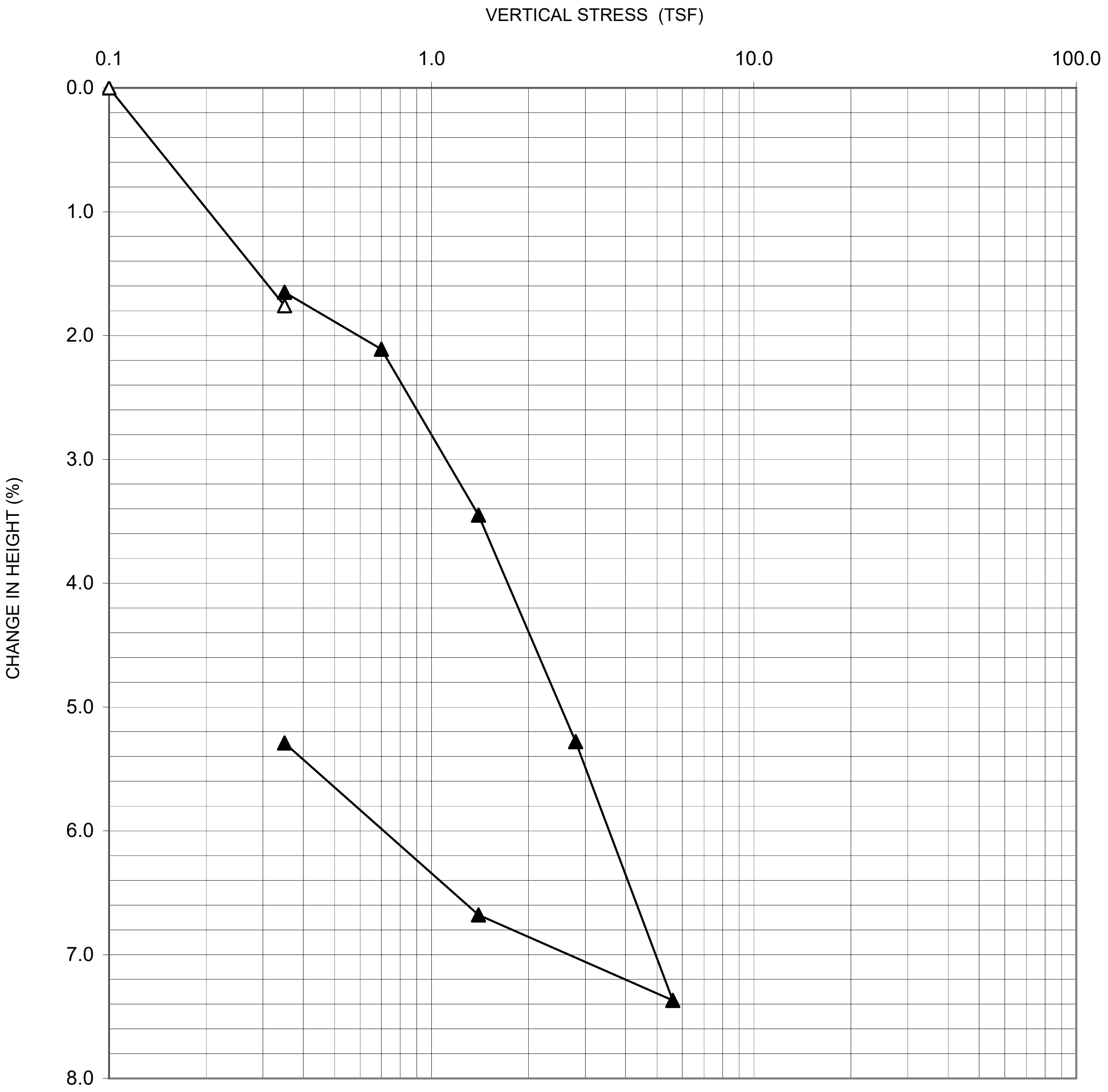
Tel: (949)797-6241 Fax: (949)797-6260

CONSOLIDATION TEST CURVE



PROJECT NO.: 19017-01

SOIL DESCRIPTIONS: BROWN SILTY SAND W/ TRACE OF CLAY (SM)

BORING NO./LOCATION : KB - 2

DEPTH / ELEV. : 25'

LIQUID LIMIT : -

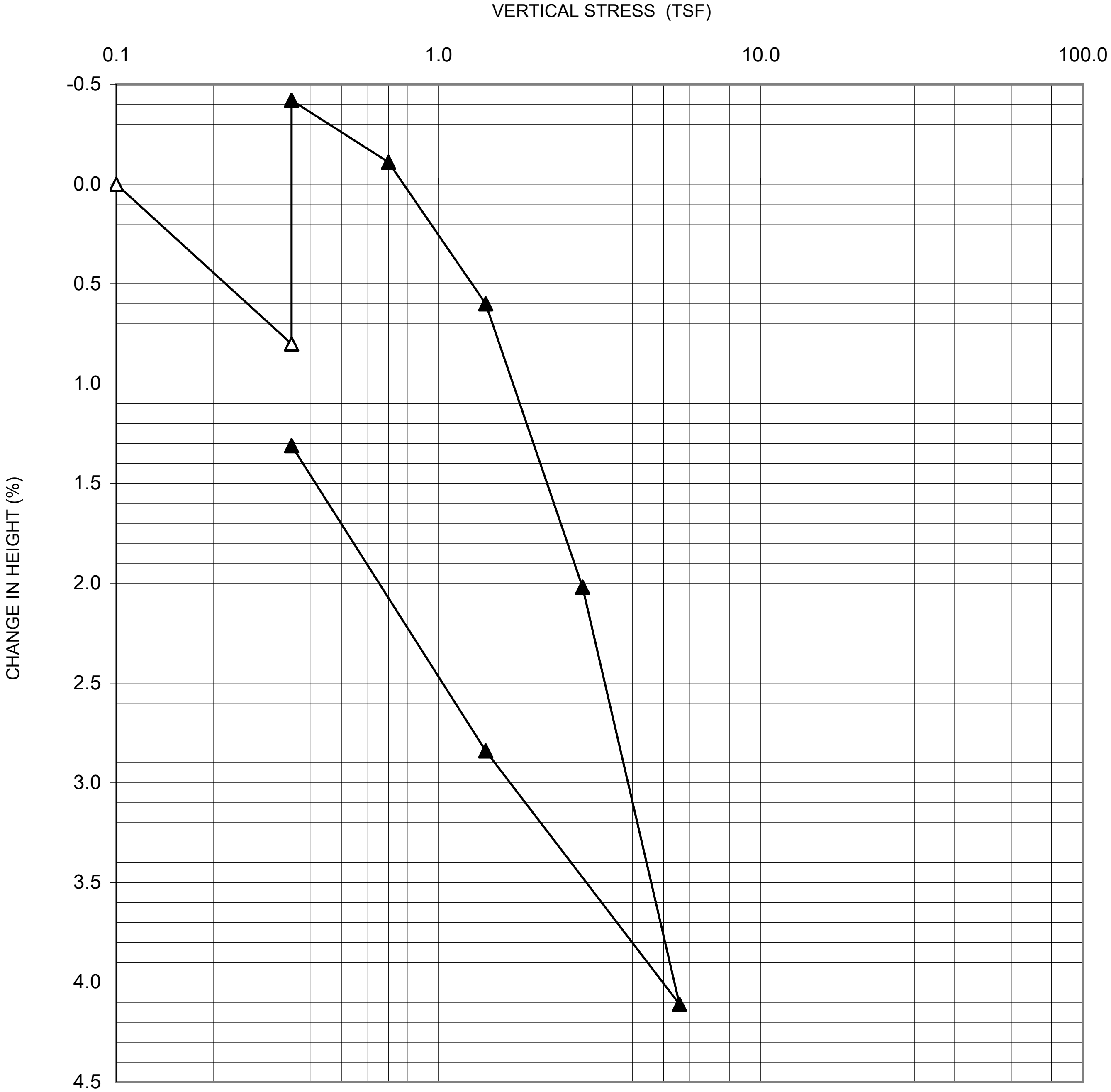
SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	24.2	104.4	107.6	0.602
FINAL	0.9471	22.5	110.2	116.6	0.518

CONSOLIDATION TEST CURVE



PROJECT NO.: 19017-01

SOIL DESCRIPTIONS: BROWN SANDY CLAY (CL)

BORING NO./LOCATION : KB - 3

DEPTH / ELEV. : 15'

LIQUID LIMIT : -

SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	25.7	99.7	101.8	0.678
FINAL	0.9869	32.9	101.0	134.4	0.656

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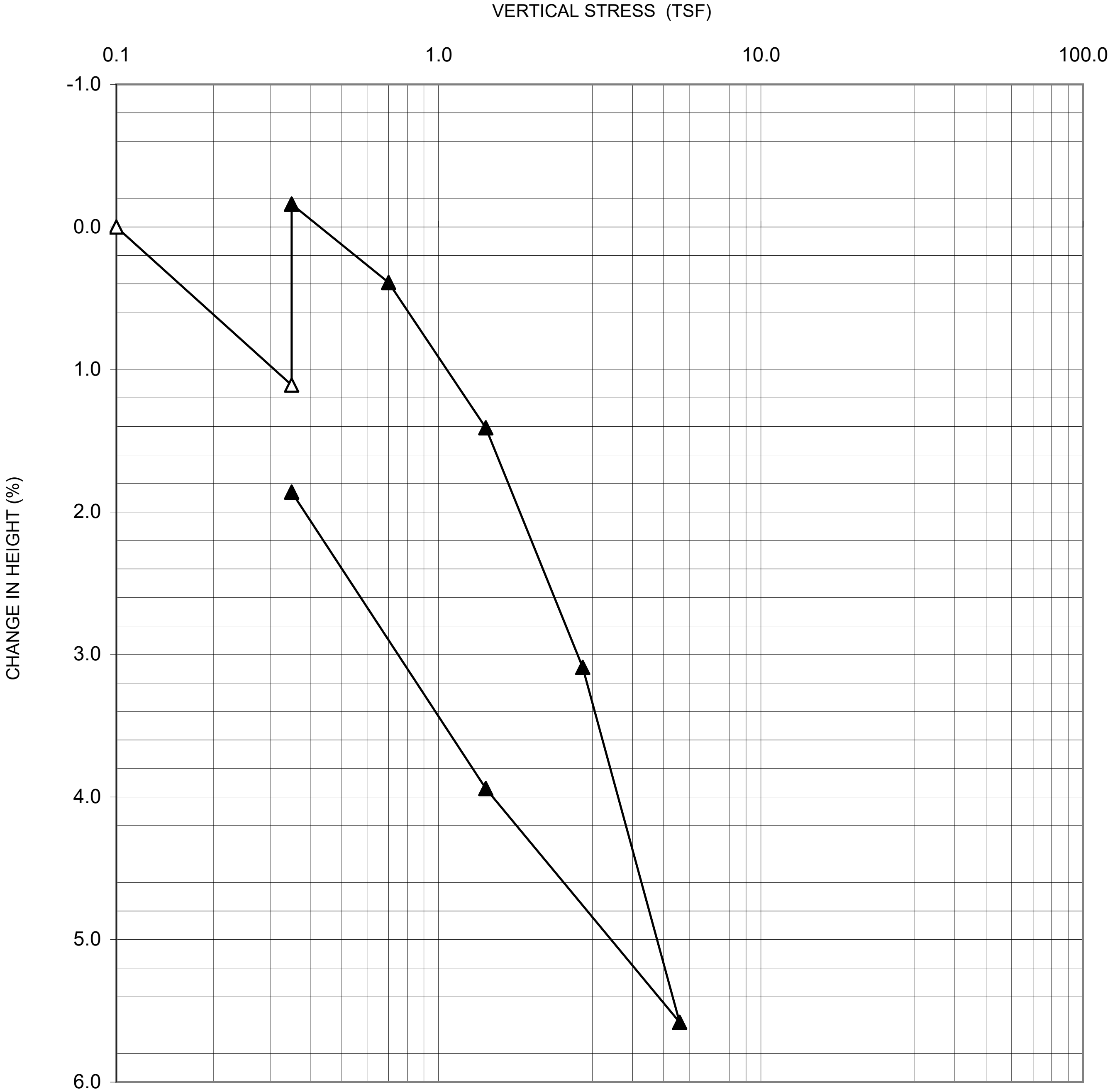
Tel: (949)797-6241 Fax: (949)797-6260

CONSOLIDATION TEST CURVE



PROJECT NO.: 19017-01

SOIL DESCRIPTIONS: BROWN SANDY CLAY (CL)

BORING NO./LOCATION : KB - 3

DEPTH / ELEV. : 20'

LIQUID LIMIT : -

SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	31.6	91.4	102.2	0.830
FINAL	0.9814	36.3	93.1	122.0	0.796

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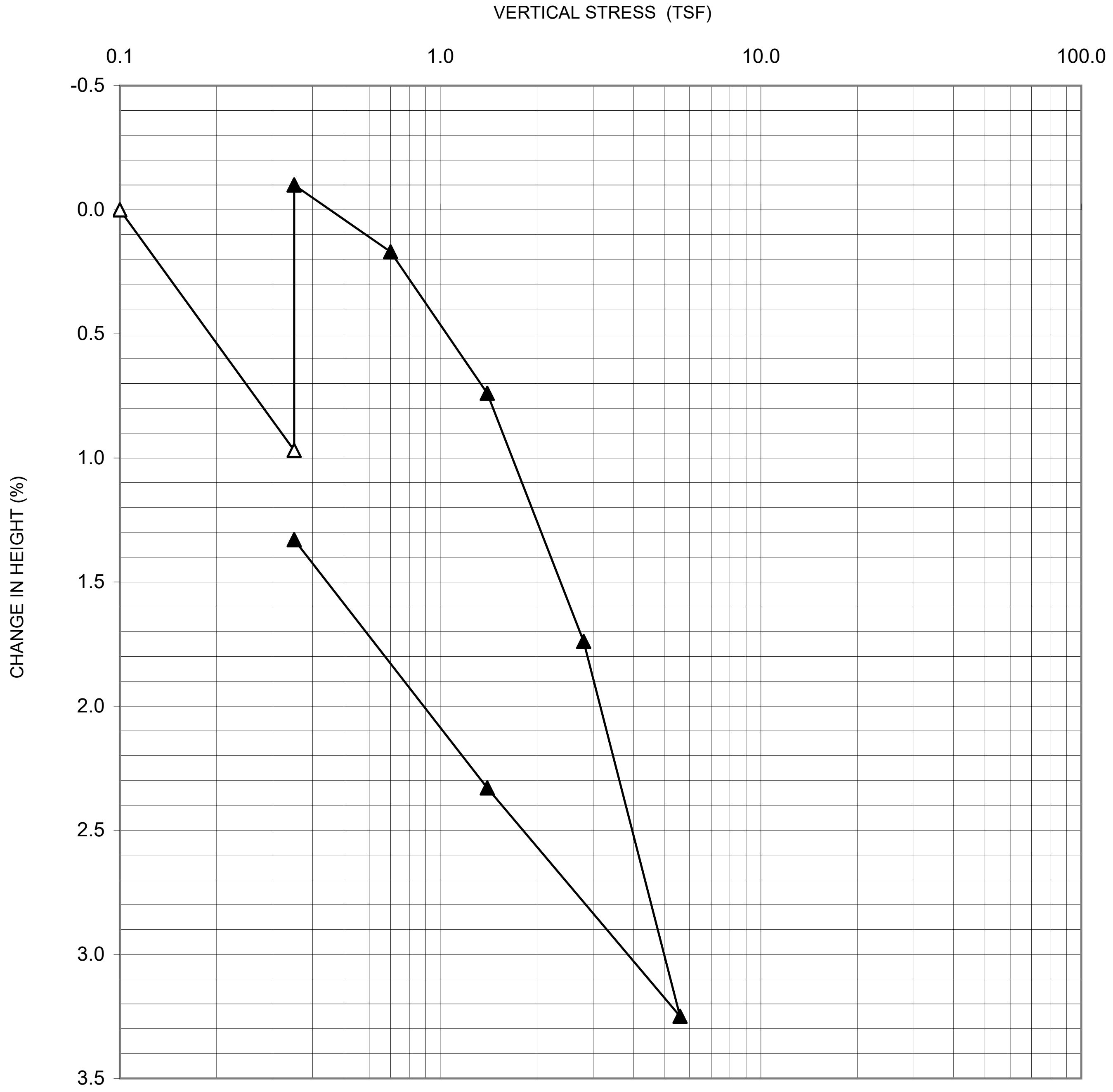
CONSOLIDATION TEST CURVE



PROJECT NO.: 19017-01

SOIL DESCRIPTIONS: OLIVE SANDY CLAY (CL)

BORING NO./LOCATION : KB - 4

DEPTH / ELEV. : 20'

LIQUID LIMIT : -

SPECIFIC GRAVITY : 2.68 (Assumed)

PLASTIC LIMIT: -

REMARKS :

	SPECIMEN HEIGHT (INCHES)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SATURATION (%)	VOID RATIO
INITIAL	1.0000	22.0	106.2	102.7	0.575
FINAL	0.9867	24.6	107.6	118.7	0.555

KCG KLING Consulting Group, Inc.

18008 Sky Park Circle, Suite 250
Irvine, Ca. 92614
Tel: (949)797-6241 Fax: (949)797-6260

CONSOLIDATION TEST CURVE

Project Name : KOLL CENTER

Project No. : 19017-01

Boring / Sample No : KB - 1

Depth : 10' (ft.)

Tested By : RB

Date: 1-Jun-19

Sample Descriptions / Classification : BROWN SILTY FINE SAND (SM)

Applied Normal Load	(ksf)	1.0		2.0		4.0	
Shear Stress,(Peak)	(ksf)	0.780		1.152		2.628	
Shear Stress,(Ultimate)	(ksf)	0.636		1.128		2.304	
Density and Saturation		Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring	(gms)	155.88	181.64	163.28	186.19	156.23	182.65
Dry Weight of Soil + Ring	(gms)		153.87		161.21		154.26
Weight of Water	(gms)	-	46.54	-	47.28	-	45.81
Weight of Ring	(gms)	-	42.24	-	46.06	-	44.97
Weight of Dry Soil	(gms)	-	111.63	-	115.15	-	109.29
Moisture Content	(%)	1.8	41.7	1.8	41.1	1.8	41.9
Wet Density	(pcf)	94.9	116.4	97.9	117.0	92.9	114.9
Dry Density	(pcf)	-	82.1	-	82.9	-	81.0
Specific Gravity,G_s	(Assumed)	2.68					
Thickness of Specimen,	(in.)	1.00					
Degree of Saturation,	(%)	4.7	107.8	4.7	108.2	4.5	105.5
Void Ratio		-	1.036	-	1.017	-	1.065

Lateral Displacement, d_h 0.36 (in.)

Displacement Rate, d_r 0.05 (in./min.)

Elapsed Time of Test, t_e 7.20 (min.)

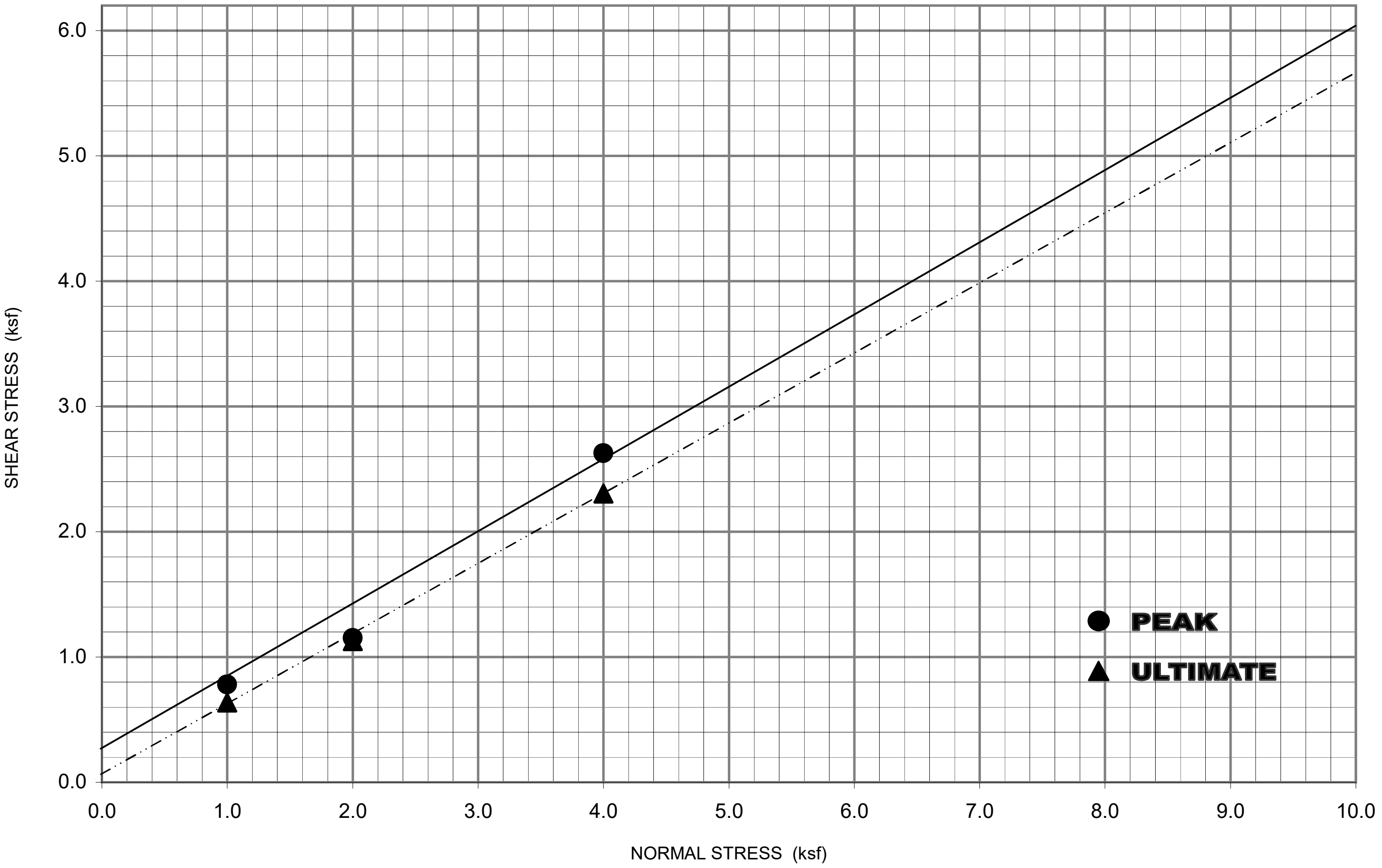
Specimen : Undisturbed : X

Remolded : -

Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	250	50
Friction Angle, ϕ	30	29

Remarks :



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DIRECT SHEAR TEST

(ASTM D3080)

Project Name : KOLL CENTER
Project No. : 19017-01
Boring / Sample No : KB - 1
Depth : 25' (ft.)
Tested By : RB
Date: 2-Jun-19
Sample Descriptions / Classification : OLIVE BROWN SILTY FINE SAND (SM)

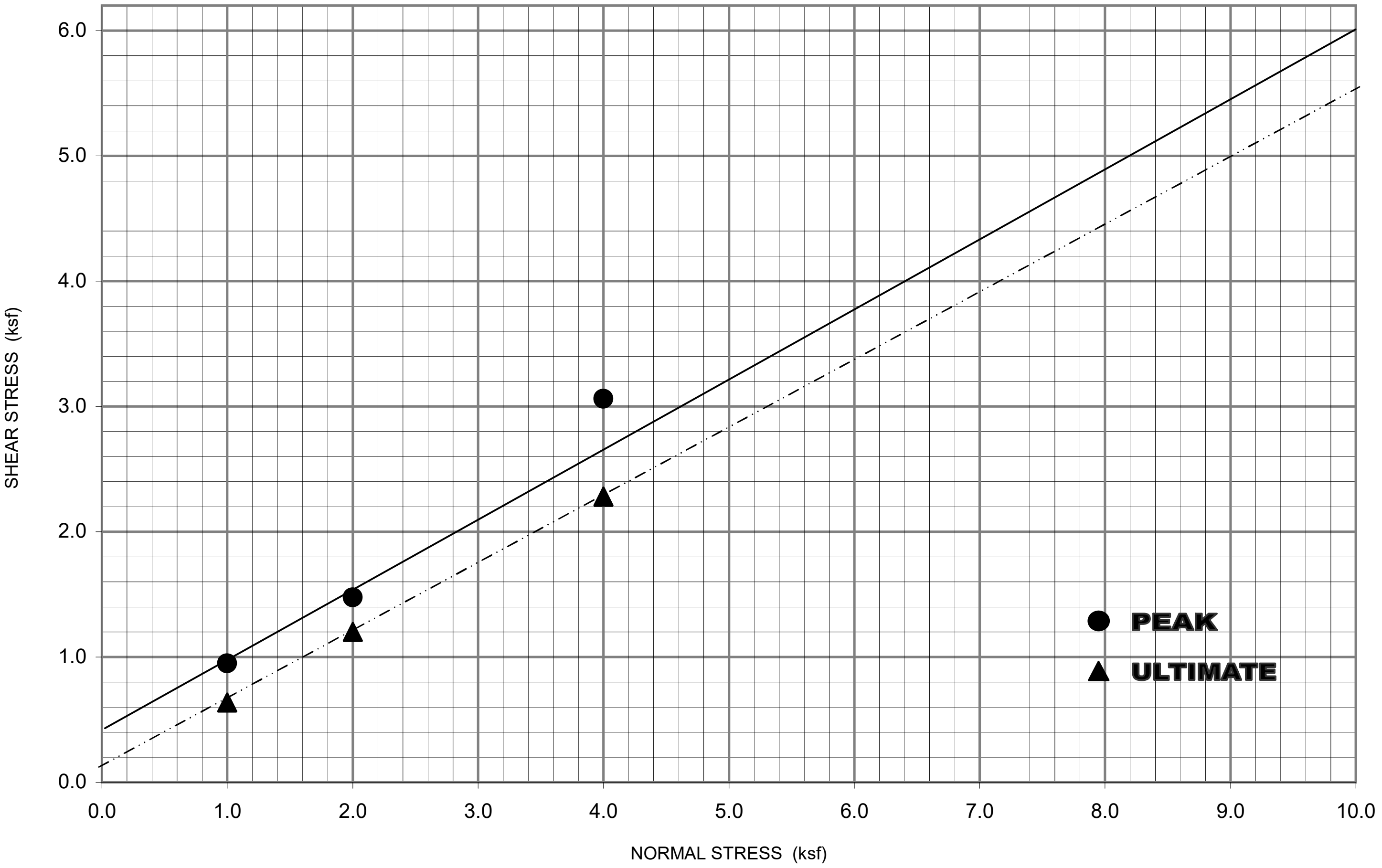
Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,(Peak) (ksf)	0.948		1.476		3.060	
Shear Stress,(Ultimate) (ksf)	0.636		1.200		2.280	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring (gms)	200.13	198.37	199.77	198.58	192.61	191.43
Dry Weight of Soil + Ring (gms)		175.98		175.63		168.99
Weight of Water (gms)	-	46.54	-	47.28	-	45.81
Weight of Ring (gms)	-	44.70	-	44.43	-	40.64
Weight of Dry Soil (gms)	-	131.28	-	131.20	-	128.35
Moisture Content (%)	18.4	35.5	18.4	36.0	18.4	35.7
Wet Density (pcf)	129.8	128.3	129.7	128.7	126.9	125.9
Dry Density (pcf)	-	94.7	-	94.6	-	92.8
Specific Gravity,G_s (Assumed)	2.68					
Thickness of Specimen, (in.)	1.00					
Degree of Saturation, (%)	64.4	124.0	64.2	125.7	61.4	119.1
Void Ratio	-	0.766	-	0.768	-	0.803

Lateral Displacement, d_h 0.36 (in.)
Displacement Rate, d_r 0.05 (in./min.)
Elapsed Time of Test, t_e 7.20 (min.)

Specimen : Undisturbed : X
Remolded : -
Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	400	100
Friction Angle, ϕ	29	28

Remarks :



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DIRECT SHEAR TEST

(ASTM D3080)

Project Name : KOLL CENTER

Project No. : 19017-01

Boring / Sample No : KB - 2

Depth : 25' (ft.)

Tested By : RB

Date: 7-Jun-19

Sample Descriptions / Classification : BROWN SILTY SAND W/ TRACE OF CLAY (SM)

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,(Peak) (ksf)	0.924		1.416		2.544	
Shear Stress,(Ultimate) (ksf)	0.672		1.140		2.280	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring (gms)	199.68	199.35	204.54	204.47	193.21	193.37
Dry Weight of Soil + Ring (gms)		178.86		182.95		172.93
Weight of Water (gms)	-	46.54	-	47.28	-	45.81
Weight of Ring (gms)	-	45.39	-	44.58	-	42.94
Weight of Dry Soil (gms)	-	133.47	-	138.37	-	129.99
Moisture Content (%)	15.6	34.9	15.6	34.2	15.6	35.2
Wet Density (pcf)	128.8	128.5	133.6	133.5	125.5	125.6
Dry Density (pcf)	-	95.3	-	99.5	-	92.8
Specific Gravity,G_s (Assumed)	2.68					
Thickness of Specimen, (in.)	1.00					
Degree of Saturation, (%)	55.4	123.7	61.4	134.4	52.2	117.9
Void Ratio	-	0.755	-	0.681	-	0.801

Lateral Displacement, d_h 0.36 (in.)

Displacement Rate, d_r 0.05 (in./min.)

Elapsed Time of Test, t_e 7.20 (min.)

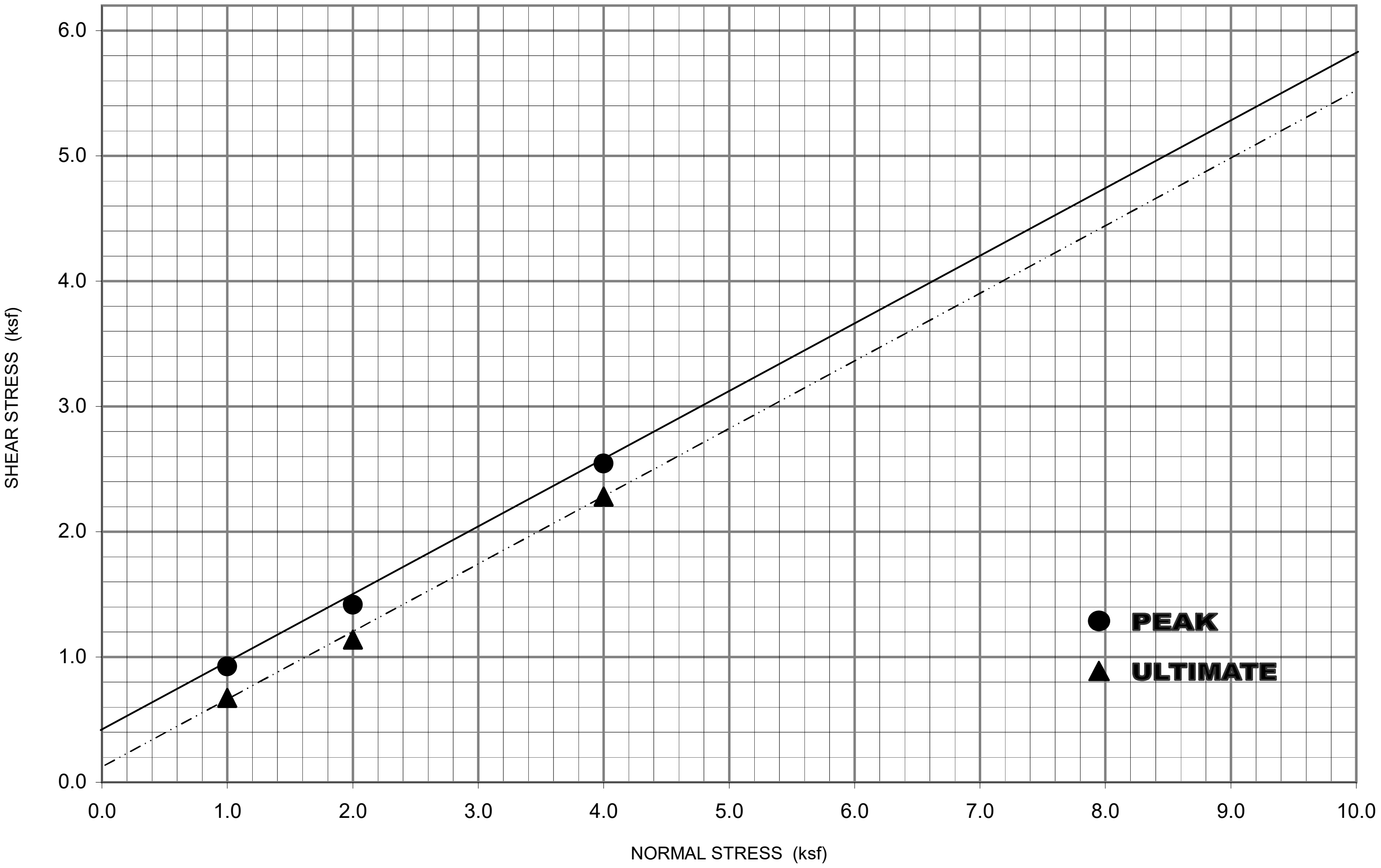
Specimen : Undisturbed : X

Remolded : -

Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	200	100
Friction Angle, ϕ	29	28

Remarks :



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DIRECT SHEAR TEST

(ASTM D3080)

Project Name : KOLL CENTER

Project No. : 19017-01

Boring / Sample No : KB - 3

Depth : 10' (ft.)

Tested By : RB

Date: 7-Jun-19

Sample Descriptions / Classification : BROWN SILTY SAND (SP/SM)

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,(Peak) (ksf)	0.936		1.572		2.880	
Shear Stress,(Ultimate) (ksf)	0.672		1.296		2.580	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring (gms)	181.11	195.92	180.45	195.57	177.85	195.76
Dry Weight of Soil + Ring (gms)		174.23		173.62		171.04
Weight of Water (gms)	-	46.54	-	47.28	-	45.81
Weight of Ring (gms)	-	44.36	-	44.7	-	42.62
Weight of Dry Soil (gms)	-	129.87	-	128.92	-	128.42
Moisture Content (%)	5.3	35.8	5.3	36.7	5.3	35.7
Wet Density (pcf)	114.2	126.5	113.4	125.9	112.9	127.8
Dry Density (pcf)	-	93.1	-	92.1	-	94.2
Specific Gravity,G_s (Assumed)	2.68					
Thickness of Specimen, (in.)	1.00					
Degree of Saturation, (%)	17.9	120.7	17.4	120.6	18.3	123.3
Void Ratio	-	0.796	-	0.815	-	0.775

Lateral Displacement, d_h 0.36 (in.)

Displacement Rate, d_r 0.05 (in./min.)

Elapsed Time of Test, t_e 7.20 (min.)

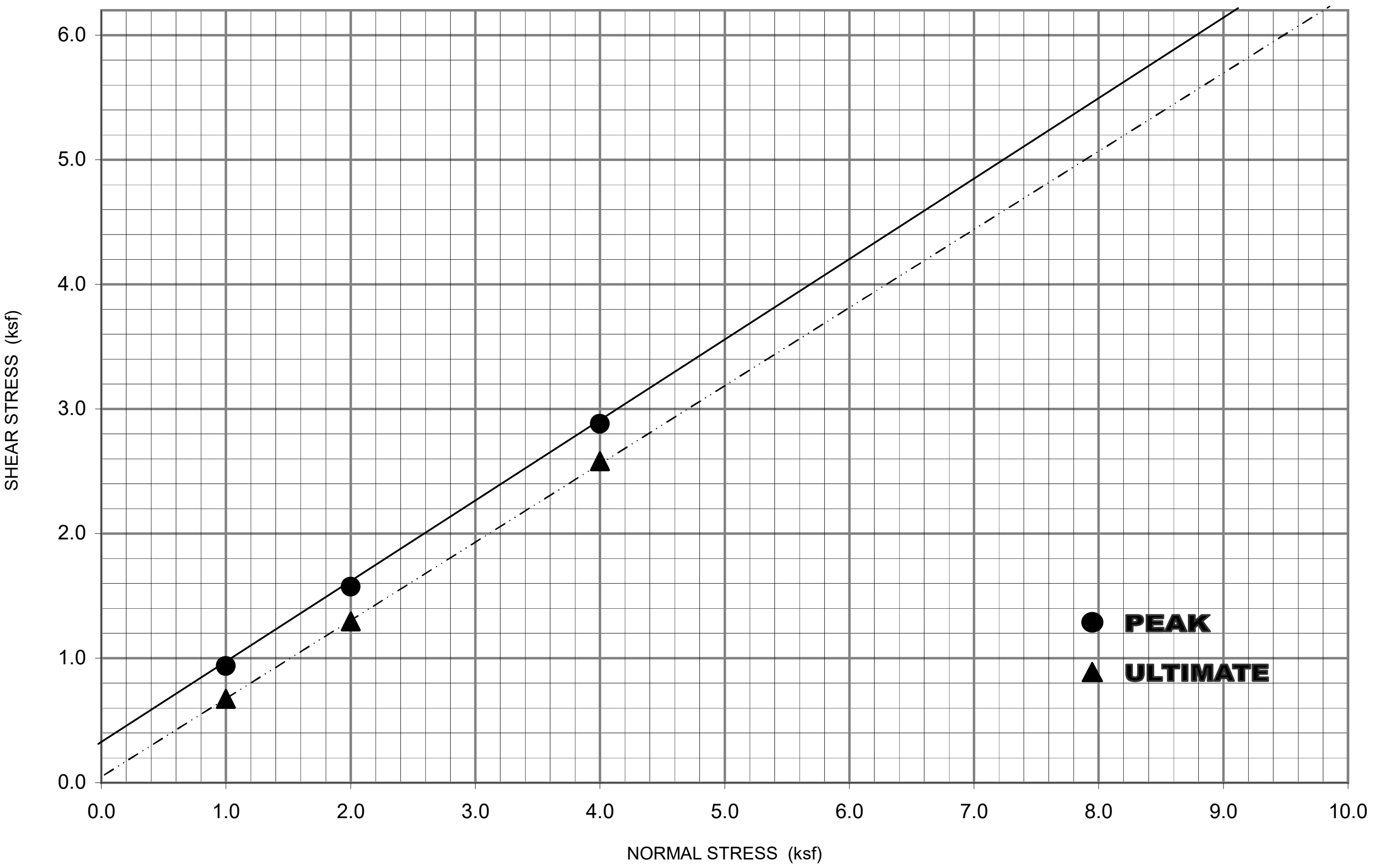
Specimen : Undisturbed : X

Remolded : -

Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	300	0
Friction Angle, ϕ	33	32

Remarks :



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DIRECT SHEAR TEST
(ASTM D3080)

Project Name : KOLL CENTER

Project No. : 19017-01

Boring / Sample No : KB - 4

Depth : 15' (ft.)

Tested By : RB

Date: 5-Jun-19

Sample Descriptions / Classification : REDDISH BROWN SILTY SAND (SM)

Applied Normal Load (ksf)	1.0		2.0		4.0	
Shear Stress,(Peak) (ksf)	0.888		1.476		2.664	
Shear Stress,(Ultimate) (ksf)	0.660		1.176		2.364	
Density and Saturation	Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring (gms)	178.15	195.61	183.99	196.63	182.45	197.33
Dry Weight of Soil + Ring (gms)		169.98		175.63		174.17
Weight of Water (gms)	-	46.54	-	47.28	-	45.81
Weight of Ring (gms)	-	42.26	-	45.04	-	44.74
Weight of Dry Soil (gms)	-	127.72	-	130.59	-	129.43
Moisture Content (%)	6.4	36.4	6.4	36.2	6.4	35.4
Wet Density (pcf)	113.5	128.0	116.0	126.5	115.0	127.4
Dry Density (pcf)	-	93.8	-	92.9	-	94.1
Specific Gravity,G_s (Assumed)	2.68					
Thickness of Specimen, (in.)	1.00					
Degree of Saturation, (%)	21.9	124.8	21.4	121.2	22.1	122.0
Void Ratio	-	0.783	-	0.800	-	0.778

Lateral Displacement, d_h 0.36 (in.)

Displacement Rate, d_r 0.05 (in./min.)

Elapsed Time of Test, t_e 7.20 (min.)

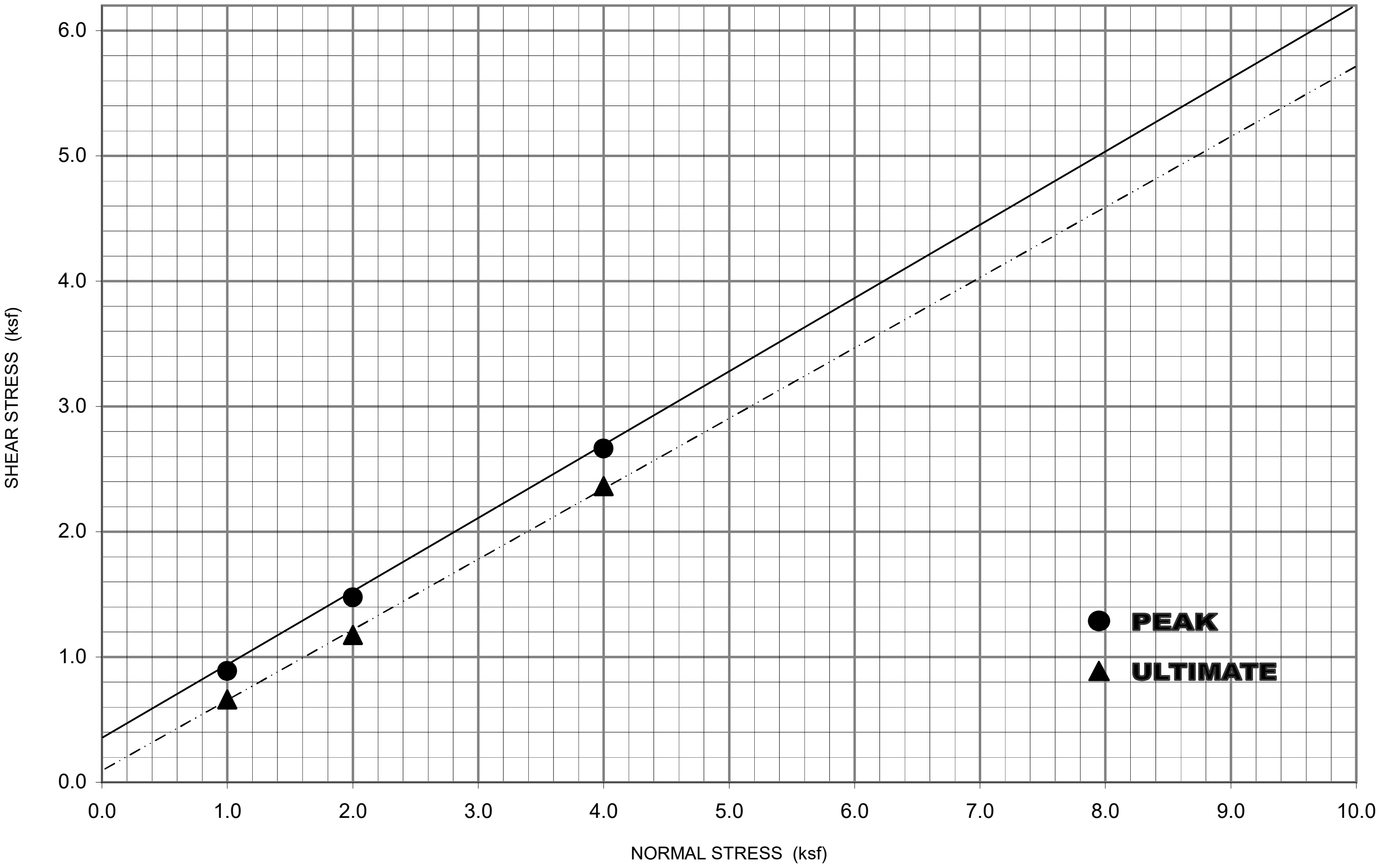
Specimen : Undisturbed : X

Remolded : -

Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	320	70
Friction Angle, ϕ	30	29

Remarks :



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DIRECT SHEAR TEST

(ASTM D3080)

Project Name : KOLL CENTER
Project No. : 19017-01
Boring / Sample No : KB - 4
Depth : 20' (ft.)
Tested By : RB
Date: 6-Jun-19
Sample Descriptions / Classification : OLIVE BROWN SANDY CLAY (CL)

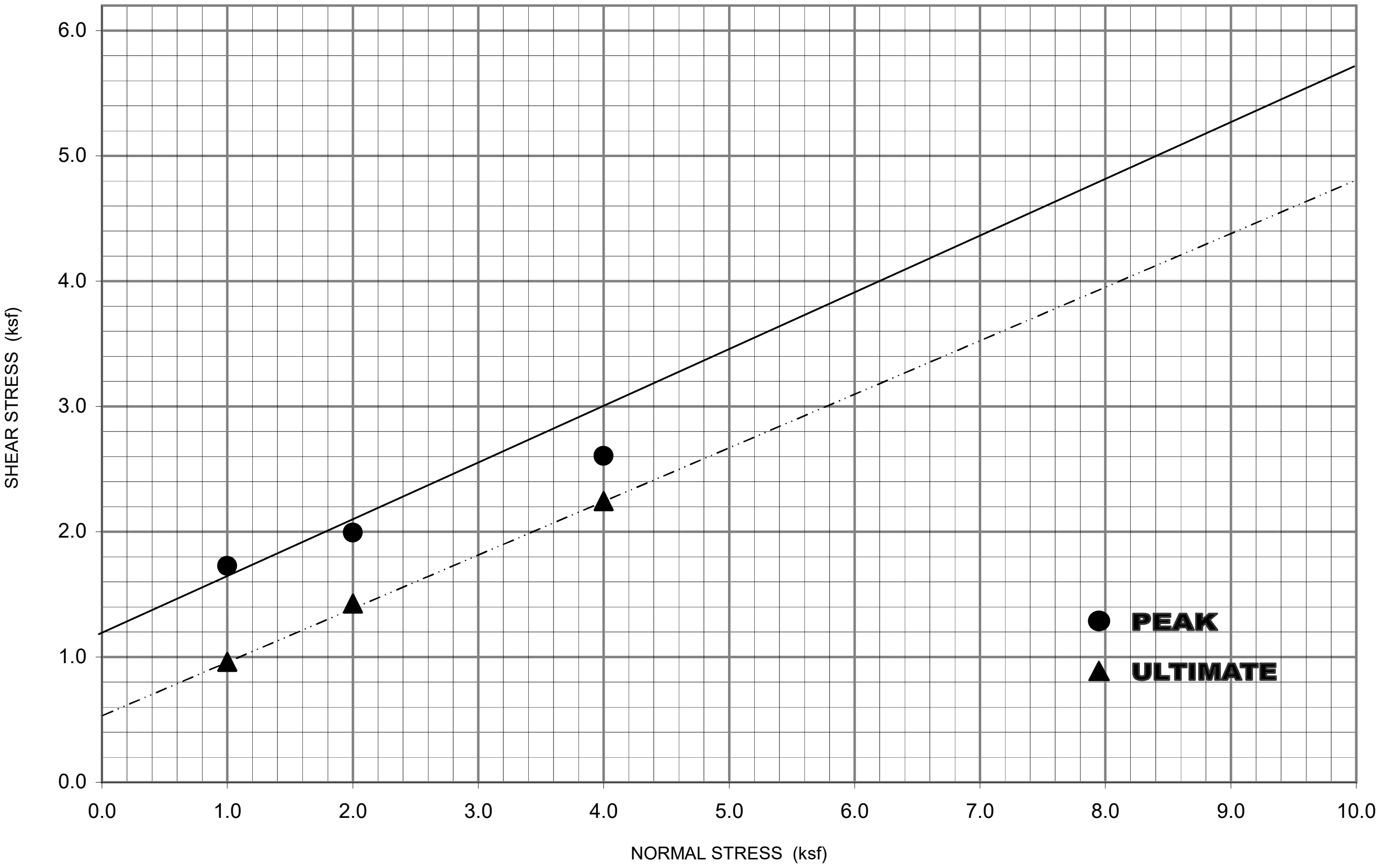
Applied Normal Load	(ksf)	1.0		2.0		4.0	
Shear Stress,(Peak)	(ksf)	1.728		1.992		2.604	
Shear Stress,(Ultimate)	(ksf)	0.960		1.428		2.244	
Density and Saturation		Initial	Final	Initial	Final	Initial	Final
Wet Weight of Soil + Ring	(gms)	194.3	194.98	194.6	195.56	199.20	200.82
Dry Weight of Soil + Ring	(gms)		163.20		163.48		167.50
Weight of Water	(gms)	-	46.54	-	47.28	-	45.81
Weight of Ring	(gms)	-	43.10	-	43.34	-	45.13
Weight of Dry Soil	(gms)	-	120.10	-	120.14	-	122.37
Moisture Content	(%)	25.9	38.8	25.9	39.4	25.9	37.4
Wet Density	(pcf)	126.3	126.8	126.3	127.1	128.6	130.0
Dry Density	(pcf)	-	91.4	-	91.2	-	94.6
Specific Gravity,G_s	(Assumed)	2.68					
Thickness of Specimen,	(in.)	1.00					
Degree of Saturation,	(%)	83.6	125.1	83.2	126.4	90.3	130.5
Void Ratio		-	0.830	-	0.834	-	0.769

Lateral Displacement, d_h 0.36 (in.)
Displacement Rate, d_r 0.05 (in./min.)
Elapsed Time of Test, t_e 7.20 (min.)

Specimen : Undisturbed : X
Remolded : -
Reconstituted : -

	PEAK	ULTIMATE
Cohesion,c (psf)	1150	500
Friction Angle, ϕ	24	23

Remarks :



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DIRECT SHEAR TEST

(ASTM D3080)

APPENDIX D

LABORATORY TEST PROCEDURES

VISUAL CLASSIFICATION OF SOILS

As a part of the routine laboratory soil testing, the soil samples are visually classified in accordance with the Unified Soil Classification System by experienced laboratory technicians. If necessary, in order to verify the visual classification, selected samples are classified utilizing the results of Standard Classification tests performed in accordance with ASTM D2487.

MOISTURE CONTENT AND DRY DENSITY DETERMINATION

Moisture content and dry density determinations were performed on relatively undisturbed samples obtained during our field exploration. The field moisture content is obtained by methods described in ASTM D2216. The in-situ dry unit weight was computed using the net weight and volume of the relatively undisturbed samples. The results of these tests are presented on the borings logs in Appendix B.

DIRECT SHEAR TESTS

Direct shear tests were performed in general accordance with ASTM D3080 on selected remolded and/or undisturbed samples that were pre-soaked for a minimum of 24 hours. The samples were then tested under various normal loads; a different specimen being used for each normal load. The samples were sheared in a motor driven, strain-controlled direct shear testing apparatus at a strain rate of 0.05 in. per minute. The results of this test are presented in the Laboratory Summary and graphically as an attachment in this Appendix.

CONSOLIDATION TESTS

Consolidation tests were performed in general accordance with ASTM D2435 on selected, relatively undisturbed, ring samples recovered from the exploratory excavations. Samples are placed in a consolidometer where increasing load increments are applied in geometric progression. The soil specimen is placed between porous stones that allow water to infiltrate and to flow of the soil sample. During the loading stages prior to the addition of water, the soil sample is sealed in order to prevent evaporation of soil water. The load increment where water was added is indicated on the consolidation pressure curves. The percent consolidation for each load cycle is recorded as the ratio of the amount of vertical compression to the original 1-inch height. The results of these tests are presented graphically as an attachment in this Appendix.

APPENDIX D

LABORATORY TEST SUMMARY

Direct Shear

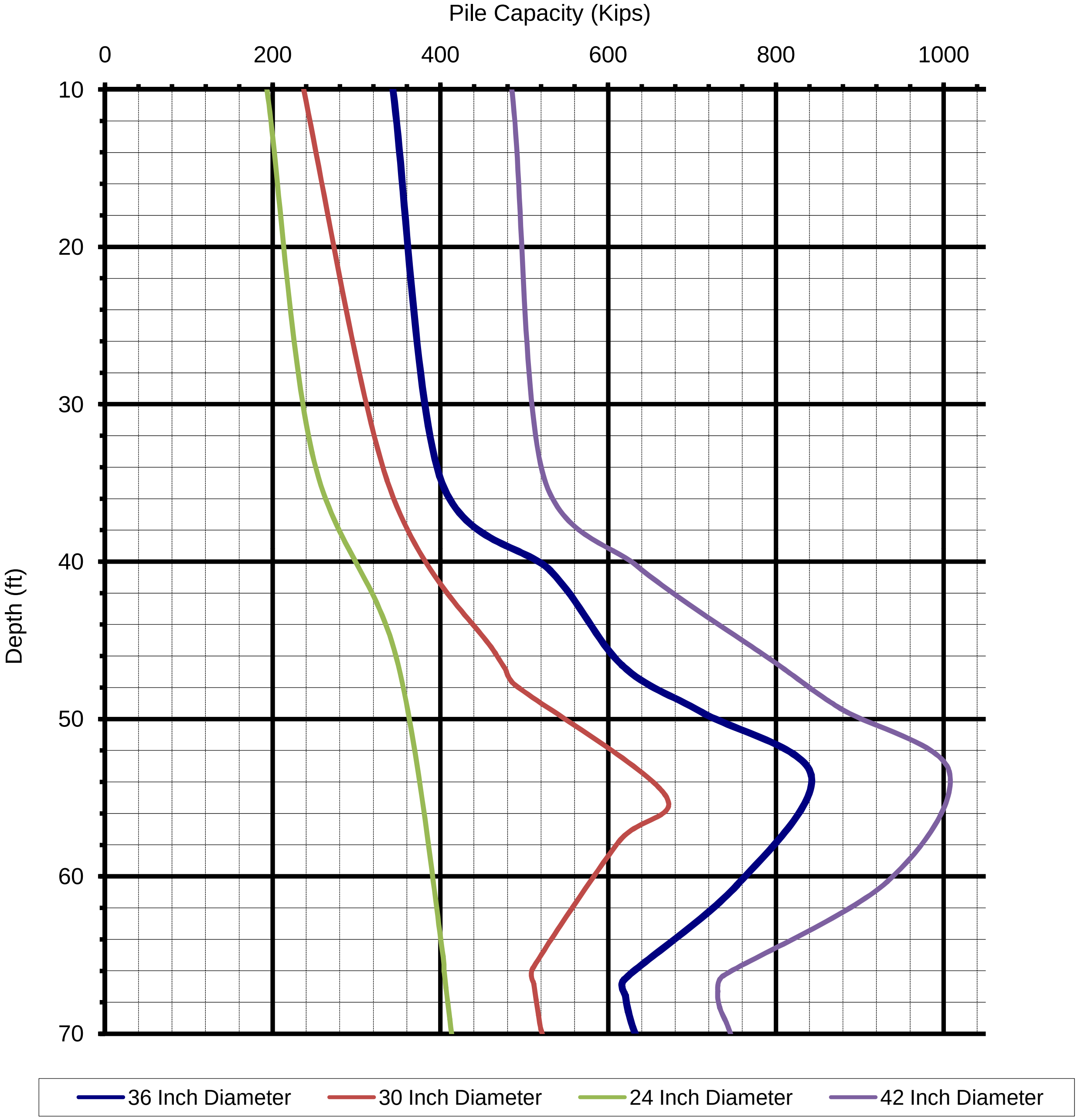
Location	Soil Description	Cohesion	Friction angle
KB-1 @ 10’	Brown Silty Sand (SM)	50 psf	29 degrees
KB-1@ 25’	Brown Silty Sand (SM)	100 psf	28 degrees
KB-2 @ 25’	Brown Silty Sand (SM)	100 psf	28 degrees
KB-3 @ 10’	Brown Sand (SP)	0 psf	32 degrees
KB-4 @ 15’	Brown Silty Sand (SM)	70 psf	29 degrees
KB-4 @ 20’	Brown Sandy Clay (CL)	500 psf	23 degrees

* Test also plotted graphically following the tables.

APPENDIX E

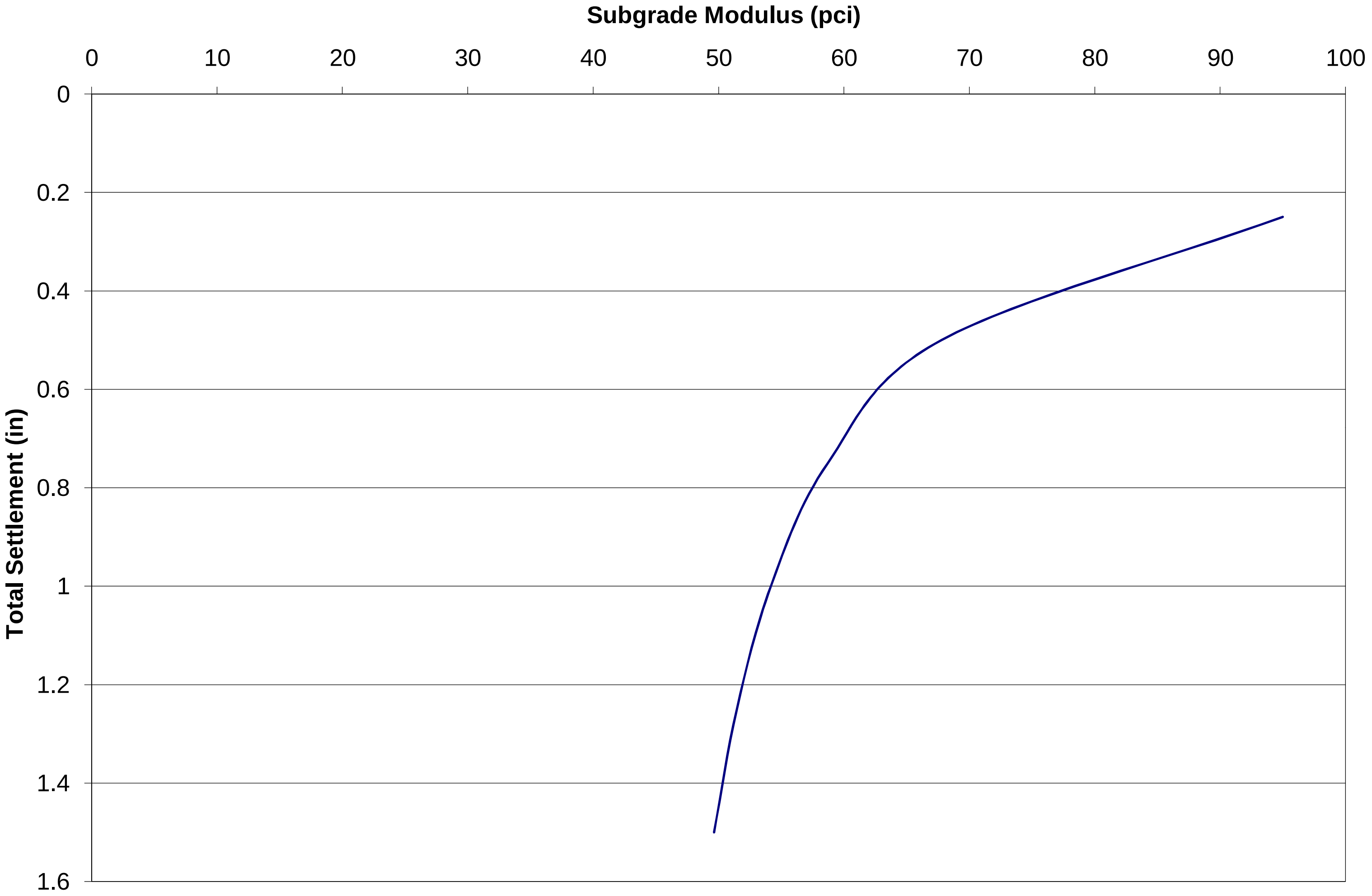
FOUNDATION DESIGN CRITERIA

Parking Structure Drilled Pile Downward Capacity

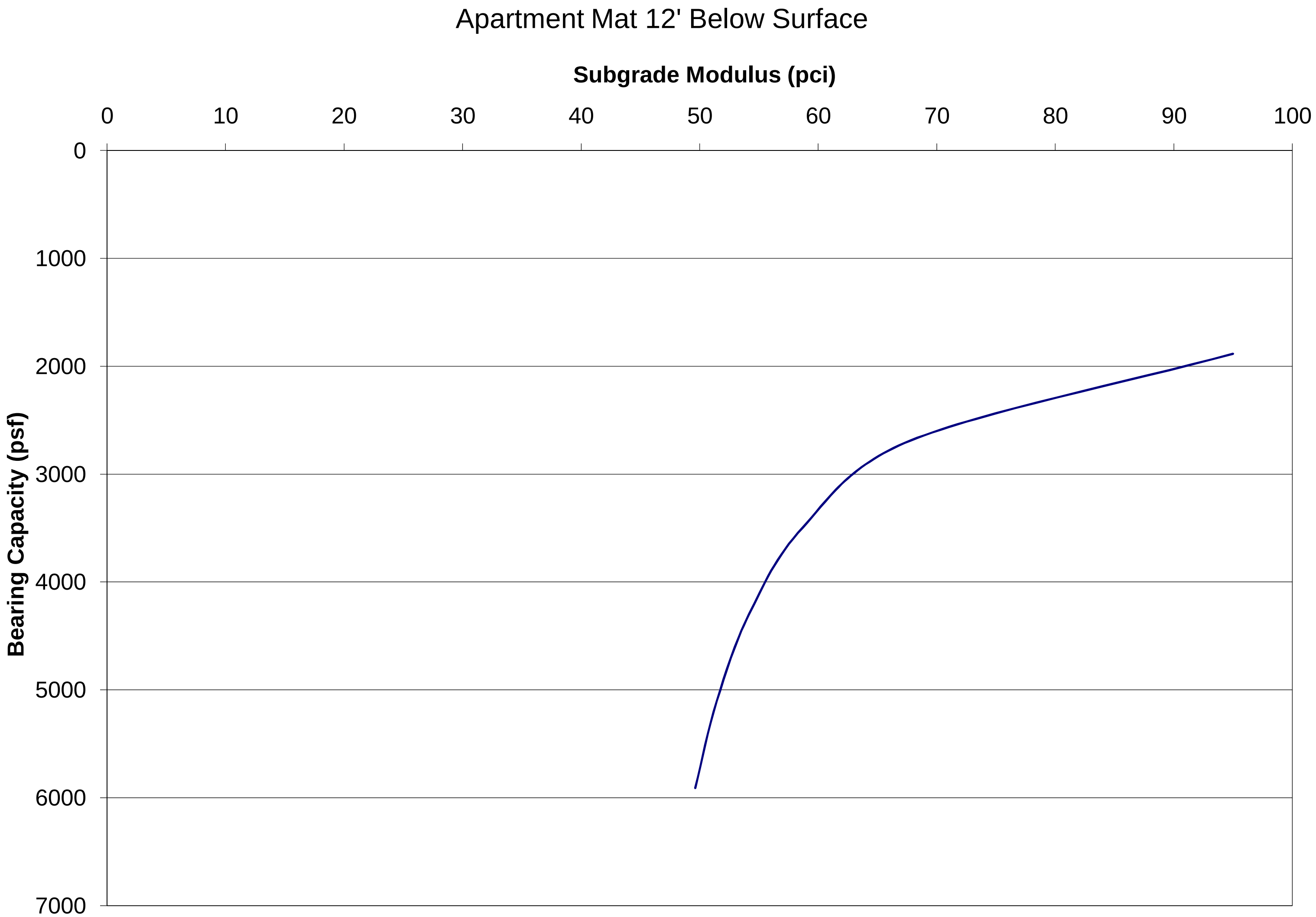


PN:19017-01
Koll Center Parking Structure
Figure A-1

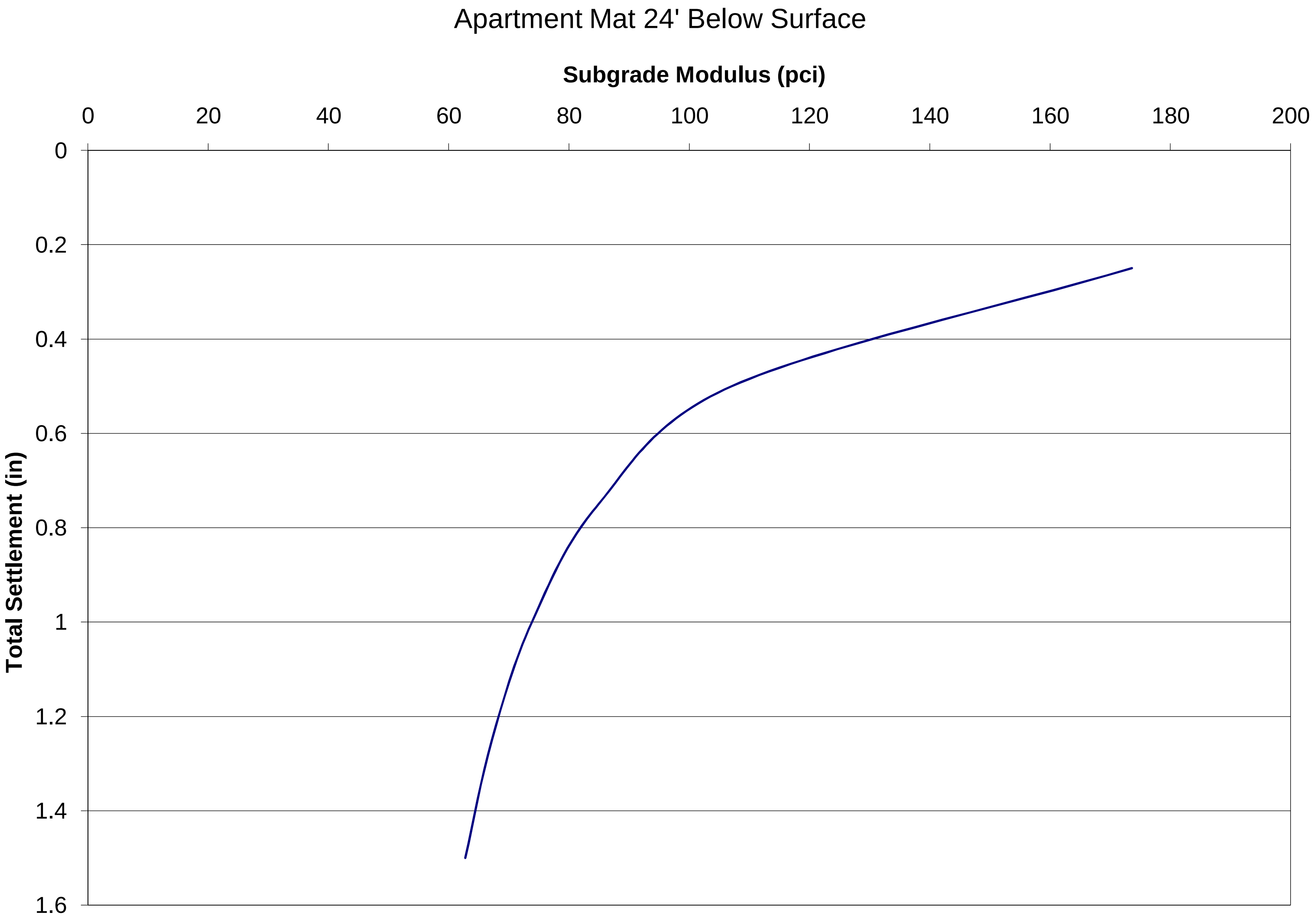
Apartment Mat 12' Below Surface



PN: 19017-01
Koll Center
Figure A-2

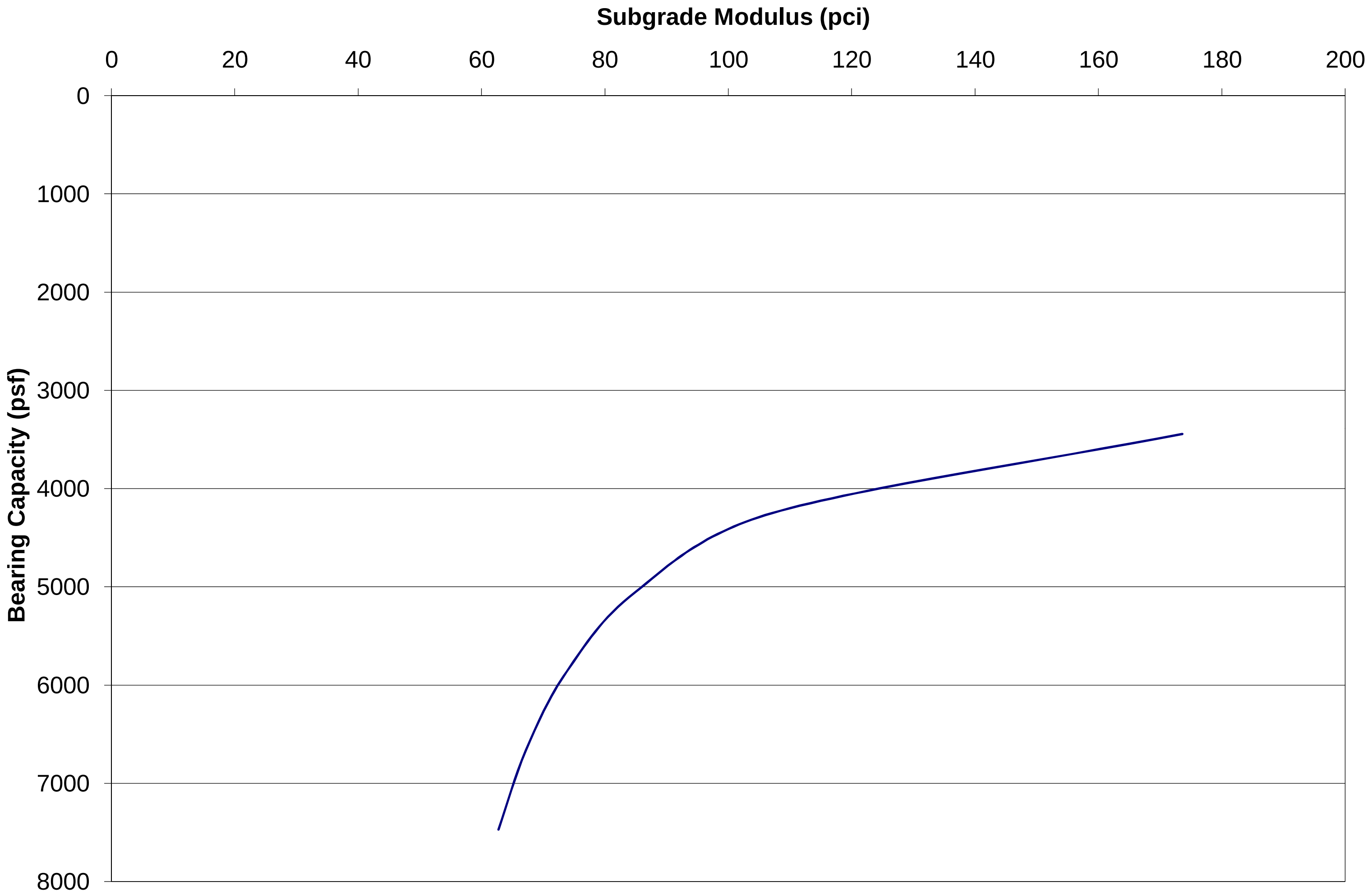


PN: 19017-01
Koll Center
Figure A-3



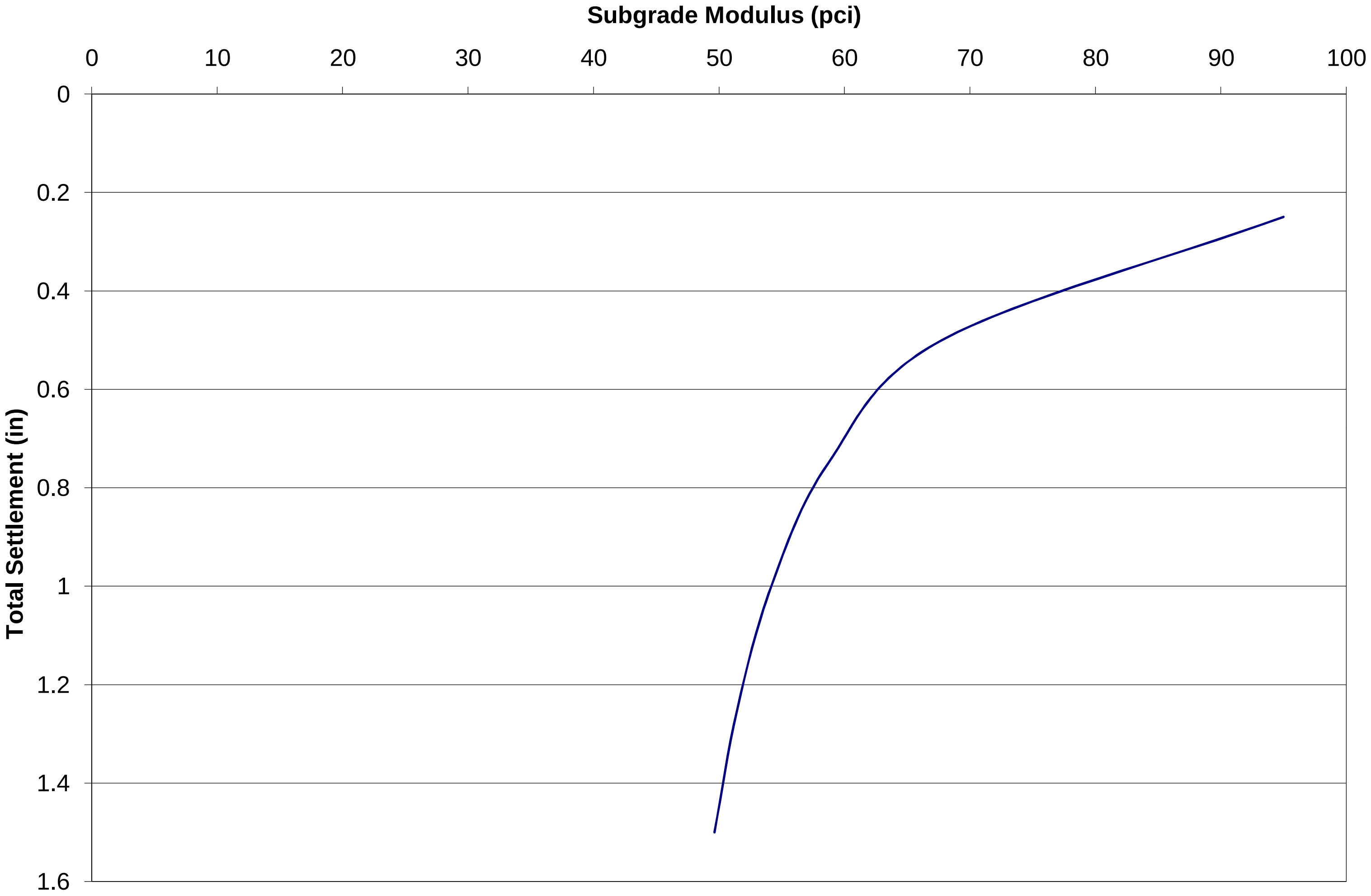
PN: 19017-01
Koll Center
Figure A-4

Apartment Mat 24' Below Surface

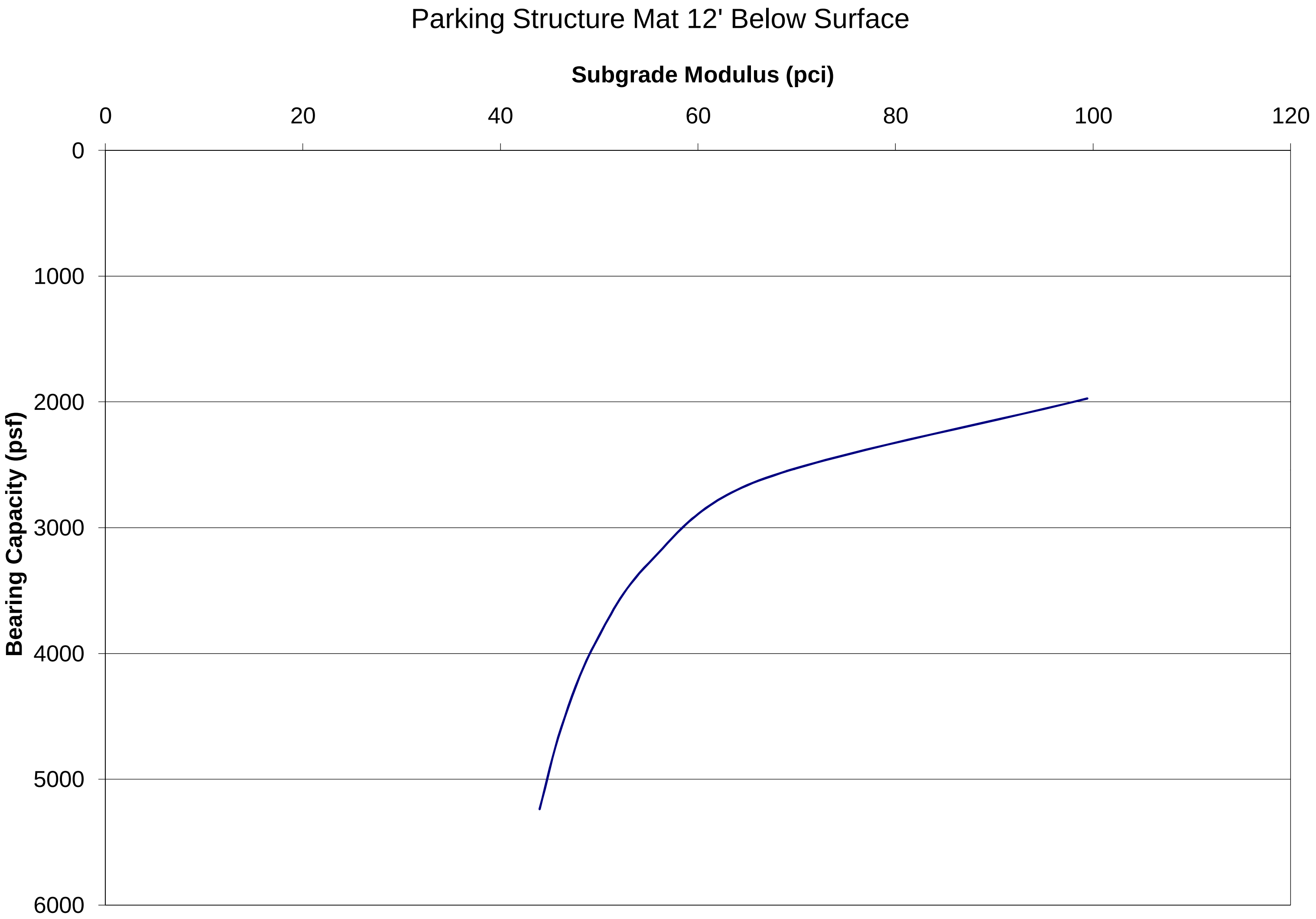


PN: 19017-01
Koll Center
Figure A-5

Parking Structure Mat 12' Below Surface

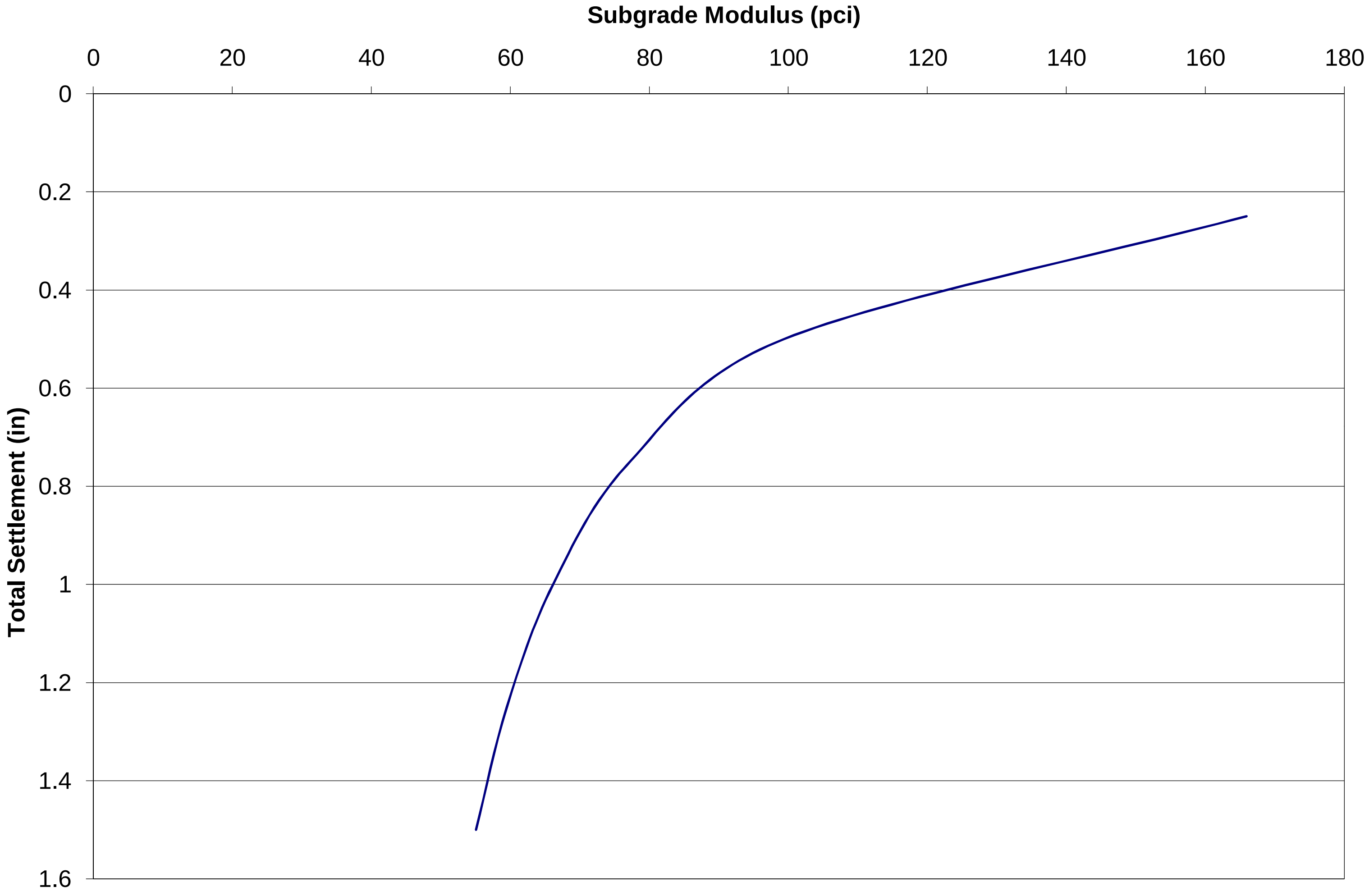


PN: 19017-01
Koll Center
Figure A-6



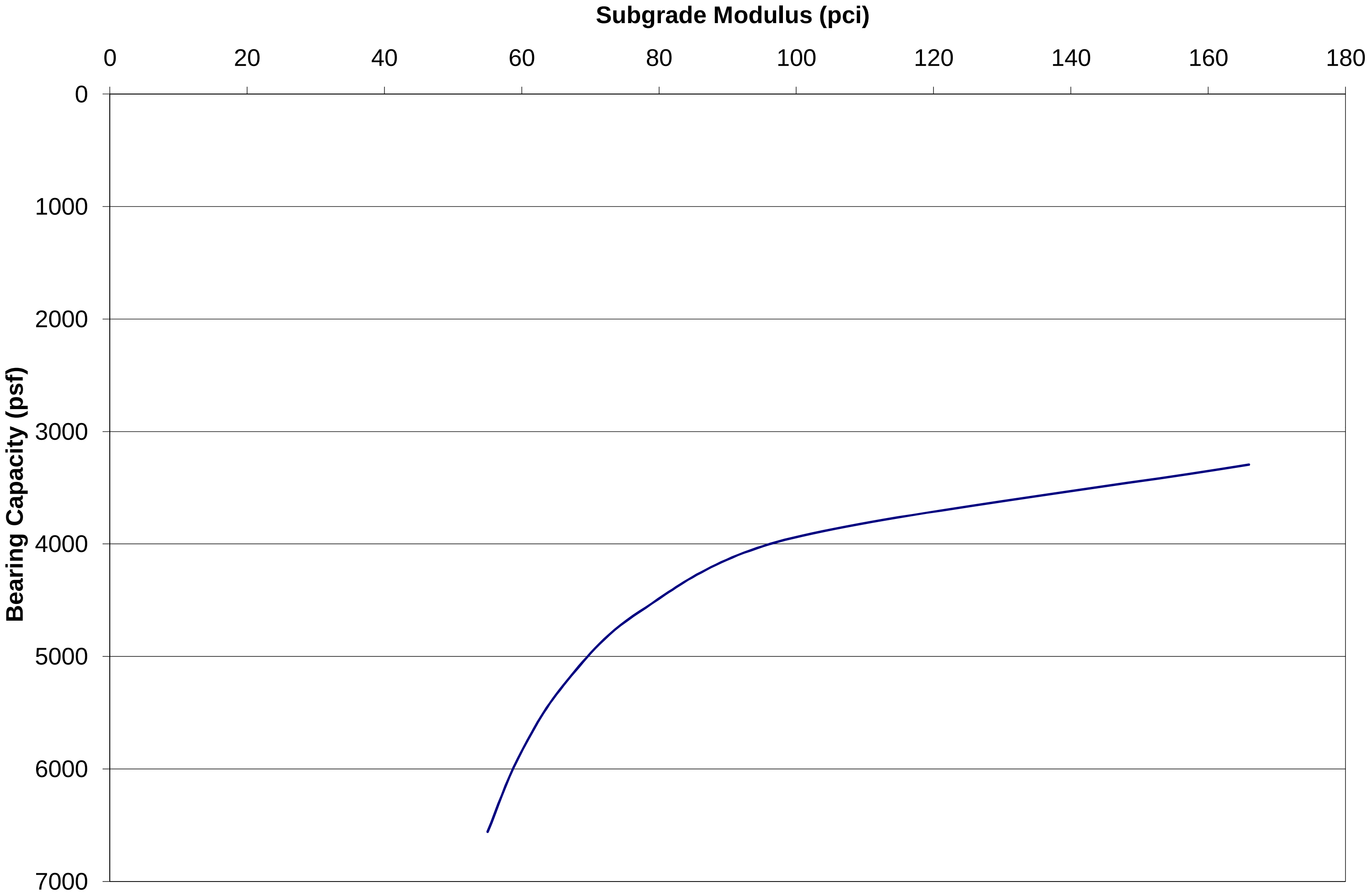
PN: 19017-01
Koll Center
Figure A-7

Parking Structure Mat 24' Below Surface



PN: 19017-01
Koll Center
Figure A-8

Parking Structure Mat 24' Below Surface



PN: 19017-01
Koll Center
Figure A-9

APPENDIX F

GENERAL EARTHWORK AND GRADING GUIDELINES

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 GENERAL INTENT

These specifications present general procedures and requirements for grading and earthwork as shown on the project grading plans, including preparation of areas to be filled, placement of fill, installation of subsurface drainage, and excavations. The recommendations contained in the geotechnical report(s) are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict. Evaluations performed by the geotechnical consultant during the course of grading may result in new specifications or recommendations in addition to those contained in the geotechnical report(s).

2.0 EARTHWORK OBSERVATION AND TESTING

Prior to the commencement of grading, a qualified geotechnical consultant (soils engineer and engineering geologist, and their representatives) shall be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the geotechnical consultant provide adequate testing and observation so that he may determine that the work was accomplished as specified. If conditions exposed during grading differ significantly from those interpreted during the preliminary design investigation, the geotechnical consultant shall inform the client, recommend appropriate changes in the geotechnical design to account for the observed conditions, and notify City or County grading authorities, as necessary. It shall be the responsibility of the contractor to assist the geotechnical consultant and keep him apprised of work schedules and changes so that he may schedule his personnel accordingly.

The Project Geotechnical Consultant shall observe processing, moisture conditioning, and compaction of fill and subgrade materials. Testing of compacted fill in representative locations shall be performed by the Project Geotechnical Consultant’s field representative. Daily reports and test results shall be provided to the client representative on a regular and frequent basis. Maximum dry density tests used to determine the degree of compaction and optimum moisture content shall be performed in accordance with the American Society for Testing and Materials test method ASTM D1557.

It shall be the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the work in accordance with the geotechnical report(s) applicable grading codes and project grading plans. If, in the opinion of the geotechnical consultant, unsatisfactory conditions, such as questionable soil, poor moisture condition, inadequate compaction, adverse weather, etc., are resulting in the quality of work less than required in these specifications, the geotechnical consultant will be empowered to reject the work and recommend that construction be stopped until the conditions are rectified.

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
(Continued)

3.0 PREPARATION OF AREA TO BE FILLED

3.1 Clearing and Grubbing

All brush, vegetation, trash, debris and other deleterious material shall be removed from fill areas and disposed of off site. Vegetation cleared from the site shall not be placed within engineered compacted fill areas.

3.2 Processing

The existing ground which is determined to be satisfactory for support of fill shall be scarified to a minimum depth of six (6) inches. Existing ground which is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until the soils are broken down and free of large clay lumps or clods and until the working surface is reasonably uniform and free of uneven features which would inhibit uniform compaction.

3.3 Overexcavation

Soft, dry, spongy, highly fractured or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, shall be overexcavated to firm ground, and verified by the project geotechnical consultant.

3.4 Moisture Conditioning

Overexcavated and processed soils shall be watered, dried-back, blended, and/or mixed as required to attain a uniform moisture content near optimum.

3.5 Recompaction

Overexcavated and processed soils which have been properly mixed and moisture-conditioned shall be recompacted to a minimum relative compaction of 90 percent, ASTM D1557.

3.6 Evaluation of Areas to Receive Fill

All areas to receive fill, including processed areas, removal areas and toe-of-fill benches shall be observed, tested, and/or mapped by the geotechnical consultant prior to fill placement. A written evaluation of the area to be filled shall be obtained by the Contractor prior to placement of fill.

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
(Continued)

4.0 FILL MATERIAL

4.1 General

Material to be placed as fill shall be free of roots, grasses, branches, wood or other organic matter and other deleterious materials, and shall be tested by the geotechnical consultant prior to use as fill. Soils of poor gradation, expansion, or strength characteristics shall be placed in areas designated by the geotechnical consultant or shall be mixed with other soils to serve as satisfactory fill material.

4.2 Oversize Material

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fills, unless the location, materials, and disposal methods are specifically recommended by the geotechnical consultant. Oversized disposal operations shall be such that nesting of oversize material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet vertically of finish grade or construction, unless specifically recommended by the geotechnical consultant.

4.3 Import

If importing of fill material is required for grading, the import material shall meet the requirements of Section 4.1. Samples of import soils shall be provided for testing a minimum of 48 hours before the import materials are brought on site.

5.0 FILL PLACEMENT AND COMPACTION

5.1 Fill Lifts

Fill material shall be placed in prepared areas in near-horizontal layers not exceeding 8 inches in loose thickness. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to attain uniformity of material and moisture in each layer.

APPENDIX F

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
(Continued)

5.2 Fill Moisture

Fill layers at a moisture content less than optimum shall be watered and mixed, and wet fill layers shall be aerated by scarification or shall be blended with drier material. Moisture-conditioning and mixing of fill layers shall continue until the fill material is at a uniformly processed at a minimum of 125 percent of the optimum moisture content.

5.3 Fill Compaction

After each layer has been evenly spread, moisture-conditioned, mixed, and shall be uniformly compacted to not less than 90 percent of the maximum dry density at a minimum of 125 percent of the optimum moisture content. Compaction equipment shall be adequately sized and shall be either specifically designed for soil compaction or of proven reliability, to efficiently achieve the specified degree of compaction.

5.4 Compaction Testing

Field tests to check the fill moisture and degree of compaction will be performed by the geotechnical consultant. The location and frequency of tests shall be at the geotechnical consultant's discretion. In general, the tests will be taken at an interval not exceeding 2 feet in vertical elevation and/or 1,000 cubic yards of fill placed.

6.0 EXCAVATION

Excavation and cut slopes will be geologically mapped and examined during grading. Sufficient time shall be allowed by the contractor to permit geologic mapping of excavation bottoms and cut slopes. If directed by the geotechnical consultant, further excavation or overexcavation and refilling of cut areas shall be performed, and/or remedial grading of cut slopes. All fill-over-cut slopes are to be graded, unless otherwise stated, shall be constructed as a fill slope with the use of minimum width stabilization fills, as necessary.

APPENDIX G

HARDSCAPE RECOMMENDATIONS

KCG KLING Consulting Group, Inc.

HARDSCAPE RECOMMENDATIONS FOR EXPANSIVE SOILS
(Residential)

Description	Minimum Concrete Thickness (Inches)	Subgrade Pre-Soaking Depth	Reinforcement (1), (3), (5)	Cutoff Barrier or Edge Thickness (Inches)	Joint (2), (5) Spacing (Max)	Base
Common Sidewalks - Isolated			N.R.		4-5 Feet	N.R.
EI<21	4 (Nominal)	Optimum to 12"		N.R.		
EI 21-50	4 (Nominal)	120% of Optimum to 12"		N.R.		
EI 51-90	4 (Nominal)	120% of Optimum to 18"		18		
EI 91-130	4 (Nominal)	130% of Optimum to 18"		18		
EI>130	4 (Nominal)	140% of Optimum to 18" (or 5% over optimum, whichever is greater)		24		
Common Sidewalks - Not Isolated (N.I.)				For exposed edges utilize recommendations for isolated condition	4-5 Feet	N.R.
EI<21	4 (Nominal)	Optimum to 12"	N.R.			
EI 21-50	4 (Nominal)	120% of Optimum to 12"	N.R.			
EI 51-90	4 (Nominal)	120% of Optimum to 18"	#3 @ 18" OC,EW			
EI 91-130	4 (Nominal)	130% of Optimum to 24"	#3 @ 12" OC,EW			
EI>130	4 (Full)	140% of Optimum to 24" (or 5% over optimum, whichever is greater)	#4 @ 12" OC,EW			
City/County Standard Sidewalks	4 (Nominal) or C.S.	Same as Common Sidewalk Not Isolated or C.S.	Same as Common Sidewalk Not Isolated or C.S.	Same as Common Sidewalk No Isolated or C.S.	4-5 Feet	N.R.
Private Driveways (1 Unit)	4 (Full)	Same as Common Sidewalk Not Isolated	6x6 - W2.9xW2.9 Mesh	Same as Common Sidewalk Isolated	10 Feet	N.R.
Shared Driveways (2 Units)	5 (Full)	Same as Common Sidewalk Not Isolated	6x6 - W2.9xW2.9 Mesh	Same as Common Sidewalk Isolated	10 Feet	N.R.
Courts or Enhanced Concrete (where higher degree of crack control is desired)						Sand Leveling course, if desired
EI<21	5 (Full)	Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 21-50	5 (Full)	120% of Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 51-90	5 (Full)	120% of Optimum to 18"	#3 @ 18" OC,EW	18	10 Feet	
EI 91-130	6 (Full)	130% of Optimum to 24"	#3 @ 12" OC,EW	24	10 Feet	
EI>130	6 (Full)	140% of Optimum to 24" (or 5% over optimum, whichever is greater)	#4 @ 12" OC,EW	24	8 Feet	
Concrete Pavement (4)	6.0	N.R.	Same as for Courts	Same as for Courts	10 Feet	6" aggregate base
Patios and Entryways						N.R.
E<21	4 (Nominal)	Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 21-50	4 (Nominal)	120% of Optimum to 12"	6x6 - W1.4xW1.4 Mesh	N.R.	10 Feet	
EI 51-90	4 (Nominal)	120% of Optimum to 18"	6x6 - W2.9xW2.9 Mesh or #3 @ 24" OC,EW	12	10 Feet	
EI 91-130	4 (Nominal)	130% of Optimum to 18"	#3 @ 18" OC,EW	18	10 Feet	
EI>130	4 (Full)	130% of Optimum to 24" (or 5% over optimum, whichever is greater)	#3 @ 12" OC,EW	24	5 Feet	

Other Considerations:

Square concrete panels when possible

Maintain positive drainage for concrete flatwork
NR = Not required; C.S. = City/County Standard;
OC = On Center
EW = Each Way
NI = Not Isolated

Notes:

(1) Reinforcement to extend into cutoff barrier in thickened edge

(2) Joint at curves or angle points
(3) Reinforcement may be superseded by the structural engineer
(4) Actual design should be based on soil "k" value and proposed traffic loads. Reinforcement and joint spacing requirements should be determined by the structural engineer.
(5) These recommendations are not intended to mitigate against cracking caused by shrinkage and temperature warping. Mitigation of these aspects should be provided by the structural engineer.

General Notes:

The recommendations herein should be considered as general guidelines and should be implemented if a "moderate" degree of crack control is desired. Should a higher degree of risk management be desired, these recommendations could be revised upon request.

APPENDIX H

ASFE INSERT

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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