

APPENDIX C: GEOTECHNICAL EVALUATION

Byer Geotechnical, Inc.,
Geotechnical Engineering Exploration and Fault Rupture
Hazard Evaluation...5600-5616 West Franklin Avenue and
1857 - 1859 North Garfield Place, Hollywood, California,
November 30, 2020.

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BYER GEOTECHNICAL, INC.

November 30, 2020
BG 23176

I & L Investment and Management, Inc.
9201 Wilshire Boulevard, Suite 202
Beverly Hills, California 90210

Attention: Mr. Ilan Gorodezki

Subject

Transmittal of Geotechnical Engineering Exploration and Fault Rupture Hazard Evaluation
Proposed Five-Story Residential Building over Subterranean Parking Level
Lot 23, Grider and Hamilton's Garfield Place Tract
5600 - 5616 West Franklin Avenue and 1857 - 1859 North Garfield Place
Hollywood, California

Dear Mr. Gorodezki:

Byer Geotechnical has completed our report dated November 30, 2020, which describes the geotechnical engineering and fault rupture hazard conditions with respect to the proposed project. The reviewing agency for this document is the City of Los Angeles, Department of Building and Safety (LADBS). The reviewing agency requires two unbound copies, one with a wet signature, a CD (PDF format), an application form, and a filing fee. Four copies of the report and the CD are enclosed.

It is our understanding that you or your representative will file the report and CD with the LADBS. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
Senior Project Engineer



BYER GEOTECHNICAL, INC.

**GEOTECHNICAL ENGINEERING EXPLORATION AND
FAULT RUPTURE HAZARD EVALUATION
PROPOSED FIVE-STORY RESIDENTIAL BUILDING OVER
SUBTERRANEAN PARKING LEVEL
LOT 23, GRIDER AND HAMILTON'S GARFIELD PLACE TRACT
5600 - 5616 WEST FRANKLIN AVENUE AND 1857 - 1859 NORTH GARFIELD PLACE
HOLLYWOOD, CALIFORNIA
FOR I & L INVESTMENTS AND MANAGEMENT, INC.
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 23176
NOVEMBER 30, 2020**

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INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geotechnical engineering exploration and fault rupture hazard evaluation performed on the subject site. The purpose of this study is to evaluate the nature, distribution, engineering properties, and geologic hazards of the earth materials underlying the site with respect to construction of the proposed project. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from consultation with Mr. Ilan Gorodezki and the preliminary plans prepared by West Pacifica Design - Construction, Inc., dated February 11, 2019. Final plans have not been prepared and await the conclusions and recommendations of this report. The project consists of construction of a five-story residential building over one subterranean parking level. The ground floor of the proposed building will consist of living units, building amenities, and parking spaces. The upper four levels will consist of wood-frame residential units. The footprint of the subterranean parking level is planned to occupy the majority of the site, as shown on the enclosed Geologic Map. Retaining walls up to 10 feet high are planned to support excavations for the subterranean parking level. An access ramp for the subterranean level is planned in the northwest portion of the proposed building via Franklin Avenue. The existing one-story mechanic shop, two-story apartment building, and associated improvements are to be removed from the site.

RESEARCH

Research of agency records was conducted to locate previous geotechnical reports for the subject site and vicinity. No reports were located.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Mr. Ilan Gorodezki. The preliminary plans prepared by West Pacifica Design - Construction, Inc., dated February 11, 2019, were a guide to our work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the enclosed Geologic Map and cross sections. The scope of this exploration did not include an assessment of general site environmental

conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on January 16 and 17, and February 18, 2020, with the aid of a hollow-stem-auger drill rig and an electronic piezocone penetrometer (CPT). It included drilling four borings (B1 through B4) and advancing nine CPT soundings (CPT1 through CPT9) to approximate depths of 31½ to 100 feet below existing grade. Boring B4 was advanced using five feet long continuous core sample barrels. The CPT soundings and Boring B4 were placed roughly in a straight line within the western portion of the site, extending from the north to the south property lines. The CPT soundings and Boring B4 were spaced approximately 6 to 13 feet. Samples of the earth materials were obtained from Borings 1-3 and delivered to our soils engineering laboratory for testing and analysis. The borings tailings and samples were visually logged by the project soils engineer. The continuous core from Boring 4 was logged by a certified engineering geologist. Following drilling, logging, and sampling, the borings were backfilled, mechanically tamped, and patched with asphalt. The CPT soundings were backfilled with bentonite and capped with asphalt.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, review of agency files, preparation of cross sections, preparation of the Geologic Map, geologic and engineering analysis, and preparation of this report. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results. Appendix II and Appendix III contain the CPT results and interpretations.

The proposed project and the locations of the borings and CPTs are shown on the enclosed Geologic Map. Subsurface distribution of the earth materials and the proposed project are shown on Sections A, B and C. Section A shows the closely spaced CPT soundings and continuous core Boring 4.

SITE DESCRIPTION

The subject property consists of a partially-graded 180 feet by 100 feet lot on the south flank of the Santa Monica Mountains, in the Hollywood section of the city of Los Angeles, California (34.1051° N Latitude, 118.3112° W Longitude). As depicted on the enclosed Aerial Vicinity Map, the property is bounded by Franklin Avenue on the north, Garfield Place on the east, a vacant lot on the south (Lot 22), and a four to five-story at-grade apartment building with a partial subterranean parking level to the rear on the west. The property is located approximately one-half of a mile northeast of the Hollywood (101) Freeway. A one-story mechanic shop and a two-story apartment building currently occupy the east and west portions of the site, respectively. Asphalt and concrete-paved driveways and parking areas occupy the remaining portions of the site. The surrounding area has been developed with single- and multi-family residential buildings. Immaculate Heart College is present across Franklin Avenue, north of the site.

Past grading on the site has consisted of placing minor amounts of fill to create the level pads for the existing buildings and associated improvements. Vegetation on the site consists of a front lawn and a few trees around the existing apartment building. Surface drainage is by sheetflow runoff down the contours of the land to the south.

GROUNDWATER

Groundwater was encountered in Boring B4 and all the CPT soundings at the approximate depths and elevations shown on the following table:

Depths and Elevations of Observed Groundwater			
Exploration	Ground Surface Elevation (feet above MSL)	Depth to Groundwater (feet)	Water Table Elevation (feet above MSL)
B4	417.0	82.0	335.0
CPT1	419.5	80.0	339.5
CPT2	419.0	80.0	339.0
CPT3	418.5	80.0	338.5
CPT4	418.5	81.0	337.5
CPT5	418.5	83.5	335.0
CPT6	418.0	82.0	336.0
CPT7	417.5	80.0	337.5
CPT8	417.5	81.0	336.5
CPT9	417.0	82.0	335.0

In *Seismic Hazard Zone Report 026*, the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was greater than 80 feet below ground surface (CGS, 1998), as shown on the enclosed Historic-High Groundwater Map.

Seasonal fluctuations in groundwater levels occur due to variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

METHANE ZONES

The City of Los Angeles Ordinance No. 175790 established methane mitigation requirements and includes construction standards to control methane intrusion into buildings. The subject property is not mapped within either a Methane Zone or Methane Buffer Zone.

EARTH MATERIALS

Fill (Afu)

Fill, associated with previous site grading, underlies the subject site to a maximum observed depth of 2 feet in Boring 2. Greater depths of fill may occur locally. The fill consists of clayey sand that is medium brown, moist, and contains trace amounts of asphalt debris. The existing fill is not suitable for support of any type of structure. Based on the current configuration of the proposed building, any fill is expected to be removed during the excavation for the subterranean parking level.

Recent Alluvium (Qa)

Natural recent alluvium blankets the subject site. The alluvium is 10 to 15 feet thick across the site. The alluvium consists of silty sand and clayey sand that are light olive brown and dark grayish-brown, moist, and slightly dense to slightly firm.

Older Alluvium (Qae/Qoa)

Older alluvium deposits underlie the subject property below 10 to 15 feet, and were encountered in the borings and CPT soundings. The older alluvium generally consists of debris flow deposits comprised of sandy clay that is brown, dark yellowish-brown, and dark reddish-brown, moist to saturated, and generally very stiff to hard. Minor stream flow deposits comprised of mostly sand were also observed.

GENERAL SEISMIC CONSIDERATIONS

Regional Tectonic Setting

The subject property is located in the Hollywood District of the city of Los Angeles, which is an active seismic region within the southern portion of the Transverse Ranges physiographic province. Moderate to strong earthquakes can occur on numerous local faults. The Transverse Ranges province is dominated by east-west trending features like the Santa Monica Mountains, to the north. Faulting in the Transverse Ranges province is dominated by thrusts with a left-lateral component.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset are considered "potentially active." Faults that have historically produced earthquakes (last 200 years) are "Historically Active". Faults that show evidence of movement within the past 11,000 years (Holocene Epoch) are considered to be "active faults." Alquist-Priolo Special Studies Fault Zones (AP zones) have been designated by the California Geological Survey (CGS) around many faults that have been indicated to be active. The Hollywood Quadrangle, which contains the subject property, contains an Alquist-Priolo zone around the Hollywood Fault encompassing the subject property.

Known active faults in the region include the San Andreas, Santa Monica, and Hollywood Faults. Fifty active faults were found within a 100-kilometer-radius search area from the site using EZ-FRISK V7.65 computer program. The results of seismic-source analysis are listed in Appendix IV. General locations of the regional active faults, with respect to the subject site, are shown on the enclosed Regional Fault Map (Appendix V).

The closest mapped "active" fault is the Santa Monica Fault, a Type B fault that is located 0.4 kilometer (0.3 miles) north of the site. The Santa Monica Fault is capable of producing a maximum moment magnitude of 7.4 and an average slip rate of 1.0 ± 0.5 millimeters per year (Cao et al., 2003). The San Andreas Fault, a Type A fault, is located 52.6 kilometers (32.7 miles) northeast of

the site. General locations of regional active faults with respect to the subject site are shown on the enclosed Regional Fault Map (Appendix V).

Based on the Hollywood Quadrangle Earthquake Fault Zone, the site is mapped within a 1,000 to 2,800-foot-wide zone designated around the Hollywood Fault, as shown on the enclosed Seismic Hazard Zones Map (see Appendix V). Based on the Navigate LA website (<http://navigatela.lacity.org/NavigateLA/>), the site is located approximately 125 feet south of the mapped trace of the Hollywood Fault, as shown on the enclosed Local Fault Map (Appendix V). The *Geologic Compilation of Quaternary Surficial Deposits in Southern California* (Bedrosian, 2010) shows the Hollywood Fault south of the subject site (see Regional Geologic Map #2 - Appendix V). Regional Geologic Map #1 (Dibblee) shows the Hollywood Fault north of the subject property. Regional Geologic Map #3 (Hoots) does not show a fault in the vicinity of the subject property.

REGIONAL TECTONIC SETTING

Plate 1 - Regional Tectonic Map

The enclosed Regional Tectonic Map (Plate 1 - Appendix V) shows the general tectonic setting of the Hollywood Fault in Southern California, with surface faults based on the CGS California Fault Activity Map of Bryant and Jennings (2010) and blind thrusts in greater Los Angeles based on the Southern California Earthquake Center (SCEC) Community Fault Model (CFM) as implemented by Marshall et al. (2009). Major regional tectonic trends in the vicinity of the Hollywood Fault are emphasized. Also included are boundaries between domains that vary in the direction of maximum horizontal stress (S_{Hmax}) or in the predominant fault style (thrust, strike-slip, or normal), after Yang and Hauksson (2013), and the distribution of slip on the Northridge blind thrust plane, after Wald et al. (1996). Plate 1 illustrates that the Hollywood Fault is a segment of the Transverse Ranges South Boundary (TRSB) fault system, located between the north terminations of the Newport-Inglewood Zone (NIZ) and the Elysian Park Thrust (EPT). The Los Angeles Basin central trough

and the Puente Hills Thrust system also terminate against the Hollywood Fault. This implies that the Hollywood Fault system must accommodate part of the deformation that, within the Los Angeles Basin, is accommodated across the northwest-striking structures. Compressional and extensional structures have formed side-by-side along the Hollywood Fault, which is likely the direct result of this interaction. Convergence is indicated by the uplift of the eastern Santa Monica Mountains watersheds, and extension is indicated by the formation of the Hollywood Basin and by observations of normal displacement on individual strands of the Hollywood Fault (Dolan et al., 1997, 2000; and Lindvall et al., 2001).

Plate 2 - Regional Structure Map - Ballona Watershed

Plate 2 in Appendix V shows the regional topography along the range front of the Santa Monica Mountains, from the Los Angeles river to Santa Monica, based on the 1920s series USGS Topographic maps. The location of Plate 2 is indicated on Plate 1. The location of the project site is labeled "p." The designated active portion of the Hollywood Fault is about nine miles long, from the Los Angeles River to Benedict Canyon, and runs through densely-developed portions of the cities of Los Angeles (Hollywood area), West Hollywood, and Beverly Hills. Plate 2 includes geologic structures that are relevant to the structural setting of the Hollywood Fault, including fault branches mapped on the recently issued Earthquake Fault Zones (CGS, 2014); fault scarps; an interpreted fault branch, HF-1, or south branch, which traces the gravity contrast between high-density basement rocks and low-density alluvium, based on the inversion of gravity data, after Hildenbrand et al. (2001); and a mapped surface fault branch, HF-4, which juxtaposes granitic rock of the Feliz pluton against Tertiary sedimentary rocks just west of the Los Angeles river (shown after Dibblee, 1992). North of the Hollywood Fault, the structure of the Santa Monica Mountains is complex and the deformation history is poorly understood. Fault scarps associated with the Hollywood Fault, the Santa Monica Fault, and the Macarthur Park escarpment are after Dolan and Sieh (1992) and Dolan et al. (1997; 2000a). Plate 2 shows granitic plutons (named partly after Neuerburg, 1953), and major faults and fold axes, after Dibblee (1992) and Meigs and Oskin (2002). The name "Whitley Terrace Fault" is used for an unnamed northwest-striking fault that intersects the Hollywood Fault near La

Brea Avenue, just northwest of the project site. The Whitley Terrace Fault has a right-lateral component of displacement in plan view, based on offset Tertiary rocks (not shown) and offset of the Griffith Park Syncline (Dibblee, 1992; Meigs and Oskin, 2002). The Whitley Terrace Fault has a strong geomorphic expression, suggesting it may have been recently active and forms part of the Hollywood Fault. Plate 2 also shows a major step or bend in the Hollywood Fault alignment, which we refer to as the "Hollywood Fault bend," and which is expressed along HF-1, along the range front, as indicated by the south boundary of the Nichols pluton, and along the mapped fault scarps. Immediately south of the Hollywood Fault is the Hollywood Basin, a subsiding basin filled with alluvial deposits. Plate 2 shows structure contours, after Hummon et al. (1994), on the base of Quaternary marine gravels that form the base of the alluvial deposits. To the south, the Hollywood Basin is bound by the North Salt Lake Fault (Wright, 1991; Tsutsumi et al., 2001). Plate 2 also shows Salt Lake Fault branch SLF-1, based on gravity data, after Hildenbrand et al. (2001). South of the Hollywood Basin, Plate 2 shows the Wilshire Arch, the San Vicente Fault, the East Beverly Hills-La Cienega Monocline, the Las Cienegas Fault, and the Central Trough (Tsutsumi et al., 2001; Wright, 1991). In the Elysian Hills and further east, Plate 2 shows fold axes of the Elysian Park Anticlinorium system, after Oskin et al. (2000), and the Elysian Park Fault, after Bullard and Lettis (1993). Along the NIZ, Plate 2 shows selected buried and surface structures, compiled from various sources. Surface faults in and around the Baldwin Hills are shown after Castle (1960), including the Inglewood Fault, the Potrero Fault, the Baldwin Fault, and the West Baldwin Fault. The "Tieje Fault" is a fault that was reported, but not mapped, by Tieje (1926) from observations during construction of the North Outfall Sewer. The Tieje Fault is a buried east-side-down fault that offsets a boulder bed, likely a Los Angeles river deposit, on the order of 30 feet. The location of the Tieje Fault shown on Plate 2, based on the description by Tieje (1926) and additional data, aligns with an east-side-down fault mapped by Castle (1960). Ongoing activity on this fault is implied by its location at the edge of a large cienega (Hall, 1888) that coincides with an area that appears to have subsided relative to the projected elevation of the Los Angeles fan surface (see Plate 3). The buried Inglewood Anticline (shown after Wright, 1987) is a double-plunging anticline that apparently formed as a south-vergent fault-propagation fold during development of the ENE-dipping Sentous blind thrust (not shown, see Wright, 1991; Elliott et al., 2009; Halliburton, 2011). The Inglewood

Anticline is offset right-laterally by the ongoing uplift of the Baldwin Hills, implying that the Sentous Thrust system remains active. The Baldwin Graben has formed between the Inglewood Fault and a parallel fault to the east, the West Inglewood fault, which has an east-side-down offset, and that we postulate may connect with a near-vertical fault at elevation ~7,000 in a well in the Cheviot Hills oil field (Tsutsumi et al., 2001, Fig 4g). Our interpretation differs from that of Tsutsumi et al. (2001), who connects this fault with the West Beverly Hills Lineament (WBHL). Instead, we postulate the existence of a second graben, located west of the Baldwin Hills, which connects across Ballona Gap with the "Cheviot Graben," which contains the greater Benedict Canyon drainage (see Plate 3). The existence of this graben in the Baldwin Hills is indicated by the West Baldwin Fault, which is one of several unnamed local west-dipping normal faults mapped by Castle (1960). We further postulate that the Ballona Creek/Los Angeles river exploited this graben and eroded the west portion of the Baldwin Hills dome, which appears truncated over the west portion of the underlying Inglewood Anticline. This interpretation, though speculative, is consistent with the existence of numerous parallel faults that form a horst between the Inglewood Fault and the West Baldwin Fault (Wright, 1987), and with the en-echelon arrangement of structures along the Newport-Inglewood Zone.

Plate 3 - Regional Drainage Map - Ballona Watershed

Plate 3 in Appendix V shows the regional drainage system within the Ballona watershed. Selected 25- and 100-foot-elevation contours are shown locally to emphasize aspects of the topography, including alluvial fans. The main streams within the Ballona watershed include Ballona Creek, which flows through Ballona Gap (see Plate 2); Centinela Creek, fed from artesian springs at Centinela Springs, where the artesian groundwater basin spilled over the Newport-Inglewood Zone at "Potrero Gap" (see Plate 2); the greater Benedict Canyon drainage, flowing along the WBHL (see Plate 2); the greater Sepulveda drainage, which collects most of the drainage from the area east of the WBHL; selected drainages from the Elysian Hills (see also Plate 4); and the Los Angeles River. Plate 3 also shows aspects of the subsurface drainage system, including marsh areas and artesian springs after Hall (1888), and the limits of the artesian belt, modified after Mendenhall (1905). Plate

3 illustrates the absence of a surface drainage system south of the Hollywood Fault, which implies that drainage occurred through the subsurface. West of the WBHL, drainage south of the range front is confined to wide alluviated canyons that are incised into uplifted older alluvial and marine terraces. Surface drainage west of the WBHL occurred through channels that are incised up to 15 feet into the flat alluvial canyon floors. Alluvial fans that formed along these drainages include the Sepulveda and Marcasel fans (fan names are informal and for reference only) along the greater Sepulveda drainage channel and the Cattaraugus fan along the greater Benedict drainage channel. The Glendale fan is also shown for reference. The Los Angeles River currently flows toward Long Beach, but the Ballona watershed divide runs across the center of the Los Angeles fan, indicating that the Los Angeles river flowed through Ballona Gap when the west portion of the fan was aggrading. During these periods, drainage and sediment supplied by the Los Angeles River likely dominated the downstream part of the Ballona drainage system.

Plate 4 - Regional Digital Elevation Model

Plate 4 in Appendix V shows the regional topography based on the USGS digital elevation model (DEM). Man-made features that differ from the 1920s topography shown on Plate 2 include: channelization of the main drainages; cuts and fills associated with freeways (labeled); and some graded residential areas, including one north of Los Feliz Boulevard and east of Western Avenue. Plate 4 illustrates the general lack of a surface drainage system south of the Hollywood Fault, in contrast to the areas west of the WBHL and south of the Wilshire Arch (see Plate 2); the anomalous topography of the Nichols pluton, with a radial, rather than linear drainage pattern; the steep alluvial slopes south of the Nichols pluton and west of Laurel Canyon; the Hollywood Fault main bend; and the abrupt termination of the Santa Monica Mountains at the Los Angeles River, illustrated by the subdued relief north of HF-1 and east of the Los Angeles River (see Plate 2).

Plate 5 - Micro Seismicity Map

Plate 5 in Appendix V shows the epicenters of micro seismic events between 1981 and 2011, plotted using the relocated catalog of Yang et al. (2012) and the fault mechanisms catalog of Hauksson and Shearer (2012). Fault mechanisms are plotted as beachballs (see Cronin, 2004), and are color-coded for fault style, with thrust fault mechanisms (rake from -45 to -135 degrees) orange, normal fault mechanisms (rake from 45 to 135 degrees) blue, and strike-slip mechanisms in yellow (rake is defined as positive when hanging wall moves upward). The beachballs are plotted using a lower hemisphere stereographic projection, and consist of two conjugate possible slip planes, with the extensional quadrant shaded. Seismic data alone do not discriminate between the true and auxiliary slip plane. Plate 5 illustrates that there are numerous small extensional micro seismic events below the Elysian Hills and the North Shelf, generally associated with north-striking faults. A cluster of micro seismic events below Beverly Hills below the WBHL is consistent with right-lateral strike slip along the WBHL, and includes events on several small north-striking extensional faults. Below the Hollywood Basin, a very shallow event with epicenter near Yucca Street and Vine Street is consistent with the location and orientation of the Hollywood Fault south branch (HF-1) and indicates left-lateral strike-slip. A much deeper event with an oblique normal mechanism has an epicenter near the trace of the North Salt Lake Fault and just south of the Hollywood Basin, associated with east-striking faults.

Since the Northridge Earthquake in 1994, micro seismic activity has increased along the north portion of the NIZ, including the WBHL and, more recently (2014), further north below the Santa Monica Mountains. This activity is consistent with a right-lateral movement along the NIZ being driven by a NW-striking zone of ductile deformation below the NIZ that extends north below the Santa Monica Mountains, and with complex mechanical coupling across the base of the seismogenic zone (Legg et al., 2004; Nicholson et al., 1994).

Plate 6 - Structure Map, Hollywood Fault Area

Plate 6 in Appendix V shows a detail of Plate 2. It includes locations where the branches of the Hollywood Fault have been encountered during previous investigations. The location of the Metro Redline Tunnel is shown after Stirbys et al. (1999). The location of red-tagged buildings as a result of the 1994 Northridge Earthquake, which correlate with the location of the eastern portion of the Hollywood Basin, are shown after Hildenbrand et al. (2001). Plate 6 illustrates that the project site is located where the south branch of the Hollywood Fault (HF-1) converges with the recently-active branches mapped by the CGS (2014).

Plate 7 - Alluvial Fan Map

Plate 7 in Appendix V shows the alluvial fans and associated watersheds along the Hollywood Fault. The fans are modeled by fitting a cone, characterized by an apex and a constant slope, by minimizing the difference in elevation between the actual topography and the cone model, using a least-squares method. The goodness of fit is indicated by the standard deviation of the elevation difference, and by the distribution of the elevation difference. The latter, which we term *differential topography*, was determined for each fan as a contour map. The limits of alluvial fans shown on Plate 7 were determined for a differential elevation of plus five feet. The apex locations are indicated by black dots. Fans are named for nearby streets. The subject property is located between two fans: the Van Ness fan (Brush Canyon Drainage) and the Hobart fan, (Griffith Canyon Drainage).

Section A Transect

Since the site is located within an AP Zone designated around the Hollywood Fault, a subsurface exploration program of closely spaced cone penetrometer soundings (CPT) was performed at the site to determine whether variations in the stratigraphy and/or groundwater depth indicate a fault crossing the site and proposed project. A total of 9, closely spaced CPT's were performed by Kehoe Testing and Engineering (KTE) on January 16 and 17, 2020, each to a depth of about 100 feet. A continuous

core boring B4 was drilled at the south end of the line of CPT soundings to a depth of 84.5 feet. The locations of the CPTs and B4 are plotted on the Geologic Map. Section A shows the CPT soundings, the core boring, and the interpreted stratigraphy underlying the site.

A “Summary of Cone Penetrometer Test Data”, including the graphic CPT interpretations by Kehoe Testing & Engineering (KTE) are enclosed. The CPT data presented in this report are based on a separate analysis using the electronic data files by KTE, which provide depth, tip stress, sleeve stress, and pore pressure at 0.05-meter intervals. The KTE data were resampled at an 0.1-foot spacing.

Section A shows the CPT data in the form of *CPT Profiles*, which consist of the tip stress curve to the right, the sleeve stress curve to the left, and a colored area between. The color coding used on the CPT Profiles is based on the coordinates on a normalized *Soil Behavior Type plot* (SBT-plot), with the logarithm of normalized tip stress (Log Q) on the vertical axis and the logarithm of normalized friction ratio (Log R) on the horizontal axis, following the methodology of Robertson (2009). Plate 1 A shows the SBT-plot after Robertson (1990), with nine SBT classes (see Plate 1 C). As discussed by Robertson (2016), the SBT classes were intended to correlate CPT data with the USCS classification, which is based on the texture or grain size distribution as determined by a sieve analysis, and which ignores inter-grain relationships (microstructure). The SBT-plot on Plate 1 A contains a *zone of normal consolidation*, which runs from top left to bottom right. Soils that formed by normal deposition and consolidation, also called *ideal soils* (Robertson, 2016) tend to plot along this zone. Soils that have developed a *microstructure* due to secondary processes such as aging, cementation, and over-consolidation, tend to plot upward and to the right of the zone of normal consolidation. Plate 1 D illustrates the color-coding used in the CPT Profiles, with a color that becomes darker with increasing stiffness (moving up and to the right) within each SBT field. The CPT Profiles plotted on Section A are plotted separately on the enclosed CPT Graphs, which are also provided in electronic form (pdf format). Since colors in the CPT Graphs are generally poorly reproduced on paper, CPT Graphs are best viewed in electronic form (pdf). Preparation of the CPT Profile is described in more detail on the enclosed CPT Graphs.

SUBSURFACE CONDITIONS

General Site Stratigraphy

The surficial geologic units encountered were divided by age and distinct stratigraphic packages and include: artificial fill (af), young alluvial deposits (Qa/Qal) of Holocene/Late Pleistocene age, and older alluvial deposits (Qae/Qoa) of Late Pleistocene age. A generalized stratigraphic column of the sediments encountered in our explorations is summarized in Table 1.

Table 1 - General Site Stratigraphy			
Epoch	Time Scale (Age)	Geologic Symbol	Stratigraphic Unit
Holocene	Historic	af	<u>Artificial Fill</u> Silty Sand, locally containing varying amounts of rock fragments
	Present to 11,000 years	Qa/Qal	<u>Recent Alluvium</u> Primarily sand with minor sandy clay
Pleistocene	11,000 to >100,000 years	Qae/Qoa	<u>Older Alluvium</u> Primarily horizontally layered debris flows consisting of clays and sands

The alluvial section encountered along the exploration transect consists of a thin cover (10 to 12 feet) of young alluvium deposited unconformably over a series of stacked, truncated, and buried older alluvial fan deposits. The lateral variability that was observed is a result of localized erosion, scouring, infilling, and stacking of these materials in an active alluvial environment. Surfaces that were stable long enough form a robust soil, which is then buried by a new deposit, or scoured out (truncated) and possibly in-filled with younger material. The amount of erosion that occurred at each truncated soil is unknown.

Relative Age Estimates

A key for determining whether active faults cross the project area is identifying the depth to the Pleistocene aged older alluvium. The CPT interpretations consistently indicate that at about 10 to 12 feet the soil behavior plots above/to the right of the normally-consolidated behavior zone. The CPT interpretation consists of three parts: a CPT profile (left), an SBTn-Plot (normalized Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were resampled from the original data (approximately 0.16 foot spacing) using a 1-foot moving average. For each data point, the normalized Soil Behavior Type coordinates $\{\text{Log Fr}, \text{Log } q_{cn}\}$ are plotted on the SBTn-Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. Based on the CPT interpretations, the depth to Pleistocene aged older alluvium along transect A is 10 to 12 feet.

Marker Beds

The CPT profiles and core boring 4 have been plotted on Section A and were used to estimate continuous layering. Two continuous horizontal layers are indicated by the CPT profiles within the recent alluvium (Markers #1 and #2). The horizontal contact between Recent Alluvium and Older Alluvium is plotted as a bold dashed line and is labeled Marker #3. The location of the contact was

determined at the depths where the fs and qt values increased significantly. The generally horizontal continuity of Markers #1 and #2, and the recent alluvium/older alluvium contact across the transect indicates an absence of faulting at the site within the last 11,000 years.

Within the Older Alluvium deposits, numerous horizontal layers have been marked by fine dashed lines. Some of these contacts extend between only a few of the CPT profiles. The brown and orange colors indicate sandy layers which tend to have less lateral continuity as they are interpreted to be channel deposits of locally concentrated higher velocity flows. The blue and light blue layers are more clayey which tend to extend laterally. A deeper sandy layer is shown as Marker 4 extending from CPT 2 to B4, indicating no faulting with the last 30,000 to 100,000 years.

Another indicator is the groundwater table shown on Section A. The elevations shown are based on the groundwater levels measured in B4 after 30 to 40 minutes to allow stabilization. The elevations shown at the CPT soundings were taken immediately after withdrawing the probe and were subject to some variation due to limited caving of upper layers. The groundwater level is relatively constant across the site, which also indicates a lack of active faulting across the site.

SEISMIC HAZARD CONCLUSIONS AND RECOMMENDATIONS

Based on the results, Byer Geotechnical concludes that no active faults (last 11,000 years) or fault-related features were encountered. Since the exploration did not extend 50 feet north and south of the site, a mat foundation is recommended for the building.

It is our opinion that no other restrictions on future development at the site are necessary with respect to the hazard of surface fault rupture. However, a future earthquake originating on the Hollywood Fault Zone could produce very strong ground motions at the site that should be taken into consideration in the project design.

SEISMIC DESIGN COEFFICIENTS

The following table lists the applicable seismic coefficients for the project based on the current City of Los Angeles Building Code:

SEISMIC COEFFICIENTS (2020 City of Los Angeles Building Code - Based on ASCE Standard 7-16)		
Latitude = 34.1051° N Longitude = 118.3112° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3.3, ASCE Standard 7-16	Alluvium and Older Alluvium - D	
Mapped Spectral Accelerations from Figures 22-1 and 22-2 and USGS	$S_s = 2.114 \text{ (g)}$	$S_1 = 0.759 \text{ (g)}$
Site Coefficients from Tables 11.4-1 and 11.4-2 and USGS	$F_A = 1.0$	$F_v = 1.7 \text{ (g)}$
Maximum Considered Spectral Response Accelerations from Equations 11.4-1 and 11.4-2	$S_{MS} = 2.114 \text{ (g)}$	$S_{M1} = 1.290 \text{ (g)}$
Design Spectral Response Accelerations from Equations 11.4-3 and 11.4-4	$S_{DS} = 1.409 \text{ (g)}$	$S_{D1} = 0.860 \text{ (g)}$
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	$PGA_M = 0.998 \text{ (g)}$	

Reference: U.S. Geological Survey, **Geologic Hazards Science Center**, **U. S. Seismic Design Maps**, <http://earthquake.usgs.gov/designmaps/us/application.php>

The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is greater than 0.75g. Therefore, the project is considered to be in Seismic Design Category E.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Seismic Hazard Deaggregation Analysis

Probabilistic seismic hazard deaggregation analysis was performed on the subject site. Seismic parameters were determined using currently-available earthquake and fault information utilizing data from the United States Geological Survey (USGS) National Seismic Hazard Mapping Project (USGS, 2008). An average shear-wave velocity (V_{s30}) of 259 meters-per-second (Site Class D) was used in the analysis. Hazard deaggregation indicates a predominant modal earthquake magnitude of 6.9 (M_w) at a modal distance of 5.3 kilometers. The Peak Horizontal Ground Acceleration (PHGA) with a 10-percent probability of exceedance in 50 years is estimated to be 0.53g on the subject site. These ground motions could occur at the site during the life of the project. Results of the analysis are graphically presented in the enclosed "Seismic Hazard Deaggregation Chart" (Appendix IV).

Based on a Site Class D, the MCE_G peak ground acceleration adjusted for Site Class effects, PGA_M , is 0.998g. The pseudo-static seismic coefficient (k_h) was derived according to the LADBS memorandum dated July 16, 2014. The horizontal pseudo-static seismic coefficient (k_h) was selected as one-third of the PGA_M (0.33g) and was used in the seismic calculations for the cantilever and restrained retaining walls. These ground motions could occur at the site during the life of the project.

Site-Specific Ground Motion Analysis

Site-specific ground motion analysis was performed in accordance with Chapter 21 of the American Society of Civil Engineers (ASCE) Standard 7-16. The probabilistic and deterministic seismic response spectra, based on maximum rotated component of spectral response at five-percent damping, are enclosed. The analysis is also based on a probability of exceedance of two percent in 50 years (2,475-return period). A computerized program, EZ-FRISK V7.65, was used to generate the seismic response spectra. An averaging of three Next Generation Attenuation relations (Chiou-Youngs 2007 NGA USGS 2008 MRC; Boore-Atkinson 2008 NGA USGS 2008 MRC; and Campbell-Bozorgnia 2008 NGA USGS 2008 MRC) was incorporated in both the probabilistic and

deterministic analyses to estimate ground motions at the subject site. The deterministic response spectrum was generated using the 84th percentile of the maximum rotated component of spectral response at five-percent damping. A shear-wave velocity (V_{s30}) of 259 meters-per-second (Site Class D) was used in the analysis.

The design response spectrum was generated by multiplying the lesser of the deterministic and probabilistic response spectra by two-thirds, according to Sections 21.2.3 and 21.3 of ASCE Standard 7-16. The deterministic lower-limit response spectrum was determined according to Section 21.2.2 of the ASCE Standard 7-16. Spectral response accelerations for selected periods are shown in the following table:

Spectral Response Accelerations (g)*									
Seismic Response Spectra	Fundamental Period (seconds)								
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Probabilistic MCE_R	2.0535	2.1289	2.1346	2.1045	1.9858	1.8921	1.7877	1.6448	1.5305
Probabilistic (ASCE 7-16)	1.4093	1.4093	1.4093	1.4093	1.4093	1.4093	1.4093	1.4056	1.2650
Deterministic MCE_R (84 th Percentile)	1.3980	1.5720	1.6780	1.7620	1.7350	1.7100	1.6430	1.5460	1.4650
Deterministic Lower Limit on MCE_R Response Spectrum	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000
Site Specific MCE_R	1.5000	1.5720	1.6780	1.7620	1.7350	1.7100	1.6430	1.5460	1.5000
80% Design Response Spectrum	1.1270	1.1270	1.1270	1.1270	1.1270	1.1270	1.1270	1.1240	1.0120
Site-Specific Design Response Spectrum	1.1270	1.1270	1.1270	1.1750	1.1570	1.1400	1.1270	1.1240	1.0120

* Reference: *American Society of Civil Engineers (ASCE), Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standard 7-16, 2016.*

The data included in the table above are plotted and presented in the enclosed Site-Specific Seismic Response Spectra figure (see Appendix IV). Detailed calculations for fundamental periods up to eight seconds are also included in the "Site-Specific Ground Motion Analysis" table (see Appendix IV).

As shown on the enclosed Site-Specific Seismic Response Spectra figure, the site-specific design response spectrum is equal or greater than or equal to 80 percent of the probabilistic response

spectrum. According to Section 21.3 of ASCE Standard 7-16, the design response spectrum shall not be less than 80 percent of the probabilistic response spectrum.

Based on Section 21.4 of the ASCE Standard 7-16, the design earthquake spectral response acceleration parameters at short period, S_{DS} , and at one-second period, S_{D1} , derived from the site-specific ground motion analysis, are shown in the following table:

SITE-SPECIFIC SPECTRAL RESPONSE ACCELERATION PARAMETERS (Based on ASCE Standard 7-16 - Chapter 21)		
Latitude = 34.1051° N Longitude = 118.3112° W	Short Period (0.2s)	One-Second Period
Maximum Considered Spectral Response Accelerations Chapter 21 - Section 21.4	$S_{MS} = 1.691 \text{ (g)}$	$S_{M1} = 1.586 \text{ (g)}$
Design Spectral Response Accelerations Chapter 21 - Section 21.4	$S_{DS} = 1.127 \text{ (g)}$	$S_{D1} = 1.057 \text{ (g)}$

Liquefaction

The CGS has not mapped the site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required, as shown on the enclosed Seismic Hazard Zones Map.

Current and historic high groundwater levels are not present onsite. Therefore, the earth materials underlying the subject site are not considered subject to liquefaction.

Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water, such as lakes and reservoirs, in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. The site is not located near any lake or reservoir. In

addition, the site is at an average elevation of 420 feet above mean sea level and is located approximately 12½ miles from the shoreline. Therefore, the risk to the project from seiches or tsunamis is considered nil.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, three hollow-stem auger borings, one continuous-core boring, nine CPT soundings, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that development of the proposed project is feasible from a geotechnical engineering standpoint, provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

Based on our research and the results of the field exploration, it is concluded that there is no evidence for the presence of an active fault crossing the subject site.

The recommended bearing material is future compacted fill. A mat foundation should be used to support the proposed building since the fault study did not extend 50 feet north and south of the subject property. Soils to be exposed at finished grade are expected to exhibit a low expansion potential.

Slightly dense silty and clayey sand soil is anticipated at the bottom of the subterranean level excavation. Remedial grading will likely be necessary to prepare a firm subgrade for the mat foundation. This can be achieved by removing and recompacting the remaining recent alluvium below the bottom of the mat foundation, which is estimated to range from two to three feet.

Geotechnical issues affecting the project include temporary excavations up to 15 feet in height, including an estimate of the mat foundation embedment depth and depth of removal and recompaction. Temporary shoring, consisting of soldier piles and continuous lagging is recommended to facilitate the construction of the subterranean retaining walls and to support offsite improvements. Recommendations for temporary shoring are included in the "Temporary Excavations" section of this report.

Based on the findings of the field exploration, groundwater is not expected in the shoring-pile basement and foundation excavations.

SITE PREPARATION - REMOVALS

Surficial materials consisting of loose alluvium are present on the site. Remedial grading is recommended to improve site conditions. The remaining alluvium below the bottom of the mat foundation should be removed and replaced as certified compacted fill. The following general grading specifications may be used in preparation of the grading plan and job specifications. Byer Geotechnical would appreciate the opportunity of reviewing the plans to ensure that these recommendations are included. The grading contractor should be provided with a copy of this report.

- A. The area to receive compacted fill should be prepared by removing all vegetation, demolition debris, and remaining two to three feet of alluvium below the mat foundation. The exposed excavated area should be observed by the soils engineer/geologist prior to placing compacted fill. Removal depths can be found in the "Site Preparation - Removals" section above. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum dry density.
- B. The proposed building site shall be excavated to a minimum depth of two to three feet below the bottom of the mat foundation to expose a firm bottom consisting of older alluvium. The excavated areas shall be observed by the geologist prior to placing compacted fill.

- C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts, moistened as required, and compacted in six-inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.
- D. The moisture content of the fill should be near the optimum moisture content. When the moisture content of the fill is too wet or dry of optimum, the fill shall be moisture conditioned and mixed until the proper moisture is attained.
- E. The fill shall be compacted to at least 90 percent of the maximum laboratory dry density for the material used. The maximum dry density shall be determined by ASTM D 1557-12 or equivalent.
- F. Field observation and testing shall be performed by the soils engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until 90 percent relative compaction is obtained. A minimum of one compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

FOUNDATION DESIGN

Mat Foundation

A mat foundation is recommended to support the proposed building, provided it is founded in firm alluvium or future compacted fill. The minimum thickness of the mat should be 12 inches. The structural engineer may require a greater thickness. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Mat (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Future Compacted Fill	12	3,000	0.33	220	3,000

For bearing calculations, the weight of the concrete may be neglected. The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

The bottom of the mat foundation should be free from loose material and construction debris, and should be approved by the geotechnical engineer prior to placing forms, steel, or concrete.

Modulus of Subgrade Reaction

The allowable modulus of subgrade reaction, k_f , is 300 kips-per-cubic-foot for a 12-inch by 12-inch footing. The modulus should be reduced for larger footings, such as the proposed mat. For rectangular footings of dimensions B (width) x L (length), the following formula may be used (Bowles, 1996):

$$k_s = k_f * (m + 0.5) / (1.5 * m)$$

where k_s = Modulus of subgrade reaction for a full-size mat foundation, and
 $m = L / B$.
 L = Length of Mat Foundation (feet)
 B = Width of Mat Foundation (feet)

Foundation Settlement

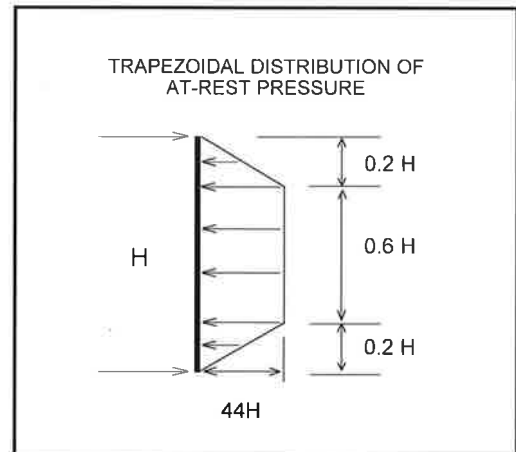
Settlement of the mat foundation system is expected to occur on initial application of loading. A total static settlement of 0.5 inch to one inch may be anticipated. Differential static settlement should not exceed 0.5 inch within a horizontal distance of 30 feet.

RETAINING WALLS

General Design

Cantilever retaining walls up to 10 feet high, with a level backslope and uniform vehicular surcharge of 300 pounds, may be designed for an equivalent fluid pressure of 43 pounds-per-cubic-foot (see Calculation Sheet #1a). Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of $\frac{3}{4}$ -inch crushed gravel.

Subterranean retaining walls, which will be restrained, should be designed for an at-rest lateral earth pressure of $44H$, where H is the height of the wall. The diagram illustrates the trapezoidal distribution of earth pressure. The design earth pressures assume that the walls are free draining. Surcharge loads from vehicular traffic and adjacent buildings should be added to the at-rest pressure for restrained retaining walls. Surcharge loads may be calculated using NAVFAC DM-7.02 Design Manual, LADBS Information Bulletin P/BC 2020-083, or an equivalent method.



Seismic analysis of the proposed cantilever retaining walls indicates an additional 297 pounds of loading due to seismic forces is required, since the calculated seismic thrust is more than the design active thrust for a retained height of 10 feet (see Calculation Sheet #1Sa). An additional 1,673 pounds of loading due to seismic forces is required on the west restrained subterranean retaining wall adjacent to the existing four-story apartment building (see Calculation Sheet #3Sa). The seismic load should be applied at a height of $0.3H$ measured from the base of the wall. Seismic analysis also indicates that no additional loading due to seismic forces is required on the north, east, and west restrained retaining walls since the calculated seismic thrust is less than the design at-rest thrust for a retained height of 10 feet (see Calculation Sheets #2Sa).

Subterranean retaining walls should be provided with a subdrain covered with a minimum of 12 inches of $\frac{3}{4}$ -inch crushed gravel. An alternative subdrain system consisting of Miradrain and gravel pockets connected to a solid pipe outlet may be used behind the subterranean retaining walls. The gravel pockets should be placed at the bottom of the retaining wall, midway between the shoring bays. A sump pump will be required for basement subdrains. The gravel pockets should be excavated to penetrate the slurry backfill behind the lagging to ensure contact with the earth materials behind the lagging.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557-12, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with $\frac{3}{4}$ -inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted-fill blanket to the surface. Restrained walls should not be backfilled until the restraining system is in place.

Foundation Design

Retaining walls may be supported on the mat foundation.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls can deflect up to one percent of their height in response to loading. This deflection is normal and results in lateral movement and settlement of the backfill toward the wall. The zone of influence is within a 1:1 plane from the bottom of the wall. Hard surfaces or footings placed on the retaining wall backfill should be designed to avoid the effects

of differential settlement from this movement. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

TEMPORARY EXCAVATIONS

Temporary excavations will be required to construct the subterranean retaining walls of the proposed building, to prepare a compacted fill blanket underneath the mat foundation, and to support offsite improvements. The excavations are expected to be up to 15 feet in height, including an estimate of the mat foundation embedment depth and depth of removal and recompaction, and will expose alluvium. The alluvium is capable of maintaining vertical excavations up to five feet (see Calculation Sheet #4). Where vertical excavations in the alluvium exceed five feet in height, the upper portion should be trimmed to 1:1 (45 degrees).

Vertical excavations removing support from adjacent footings or adjacent to property lines will require the use of temporary shoring such as soldier piles. Design values can be found in the following "Soldier Piles" section.

The geologist should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

Soldier Piles

Drilled, cast-in-place concrete soldier piles may be utilized as temporary shoring to support temporary excavations to construct the subterranean retaining walls of the proposed building, to prepare a compacted fill blanket underneath the mat foundation, and to support offsite

improvements. The piles should be a minimum of 18 inches in diameter and a minimum of eight feet into the alluvium and older alluvium below the excavation. Piles may be assumed fixed at three feet into the alluvium and older alluvium below the excavation. The piles may be designed for a skin friction of 400 pounds-per-square-foot for that portion of pile in contact with the alluvium and older alluvium below the excavation. Piles should be spaced a maximum of eight feet on center. Shoring spacing may be increased up to 10 feet on center in local areas such as ramp approaches and corners of shoring. The piles may be designed for the lateral pressures shown in the following table:

Shoring Height (feet)	Type of Surcharge	Maximum Surcharge (pounds)	Active Equivalent Fluid Pressure (pcf)	Trapezoidal Pressure	Reference
15	Vehicle	300 Uniform Load	30	19H	Calculation Sheet #5a
15	Building	4,000 Line Load	33	21H	Calculation Sheet #6a

If rakers are incorporated in the temporary shoring system, the soldier piles should be designed for the trapezoidal pressures shown on the table above, where H is the shored height.

The equivalent fluid pressure should be multiplied by the pile spacing. The piles may be included in the permanent retaining wall. Where a combination of sloped embankment and shoring is used, the pressure will be greater and must be determined for each combination.

Groundwater is not anticipated in the soldier pile excavations.

Lateral Design

The friction value is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance

to lateral loading may be provided by passive earth pressure within the alluvium and older alluvium below the excavation.

Passive earth pressure may be computed as an equivalent fluid having a density of 220 pounds-per-cubic-foot. The maximum allowable earth pressure is 4,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½-pile diameters on center may be considered isolated.

Rakers

Rakers may be used to internally brace the soldier piles. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 4,000 pounds-per-square-foot may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

Lagging

Continuous lagging is anticipated between the soldier piles. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. Lagging should be designed for the recommended earth pressure, but may be limited to a maximum value of 400 pounds-per-square-foot. The space behind lagging should be backfilled with cement slurry.

Lagging should be placed behind the front flange of the shoring steel I-beams. In some cases, the shoring is designed with the lagging behind the rear flange of the shoring steel I-beams. This is to maximize the interior area and position the walls as near the property lines as possible. During the installation of lagging behind the rear flange, the shoring is not supporting the excavation while the lagging is placed and backfilled. This can cause damage to adjacent offsite improvements, such as

buildings, site walls, sidewalks, etc. If lagging is to be placed behind the rear flange of the I-beams, the lagging should be installed in slot cuts (ABC method), where lagging is installed and slurry-backfilled in the "A" slots before the "B" and "C" slots are excavated for lagging. Also, the maximum vertical height exposed should be no more than five feet.

Deflection

Some deflection of the shored embankment should be anticipated. Where shoring is planned adjacent to existing structures, it is recommended that lateral deflection not exceed one-half of an inch. For shoring not surcharged by a structure, the allowable deflection is deferred to the structural engineer. If greater deflection occurs during construction, additional bracing or anchors may be necessary to minimize deflection. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

EXTERIOR CONCRETE DECKS

Decking should be cast over firm alluvium or approved compacted fill and reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

CEMENT TYPE AND CORROSION PROTECTION

A representative sample of the near-surface soil was obtained during field exploration for laboratory testing. Corrosion test results are included in Appendix I. The results indicate that concrete structures in contact with the soils onsite will have negligible exposure to water-soluble sulfates in

the soil. According to Table 4.3.1 of Section 4.3 of the ACI 318 Building Code, Type II cement may be used for concrete construction.

The results of the laboratory testing also indicate that the near-surface soil onsite is considered corrosive to ferrous metals. Special mitigation measures for corrosion protection of steel and other metallic elements in contact with the soil may be required. The corrosion information presented in Appendix I of this report should be provided to the underground utility subcontractor.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Low-Impact Development (LID) Requirements

Typically, infiltration systems are utilized in areas underlain by pervious granular earth materials that have high percolation characteristics. In addition, infiltration systems are normally planned at least 10 feet from adjacent property lines or public right-of-way, and 10 feet from a 1:1 plane projected from the bottom of adjacent structural foundations. A thick layer of clay underlies the subject site underneath the subterranean level of the proposed building. This clay layer is very stiff to hard and impermeable, and is considered unsuitable for water infiltration. Therefore, onsite infiltration is not recommended.

As an alternative, a biofiltration system, a capture-and-reuse system, or equivalent, may be installed on the site in accordance with the City of Los Angeles Best Management Practices (City of Los

Angeles, 2011). A planter box may be used to capture and treat storm-water runoff through different soil layers before discharging water to the street storm drain. The planter box should be an impermeable rigid structure that is equipped with an underdrain to prevent water infiltration to the underlying subsurface earth materials. Planter boxes may be situated aboveground and placed adjacent to buildings. Planter boxes should be designed as freestanding and for an inward equivalent fluid pressure of 43 pounds-per-cubic-foot. This fluid pressure includes possible vehicular surcharge. Byer Geotechnical, Inc., should be provided with the final plans to verify the location of the planter boxes.

Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground and perched water may result if irrigation water is excessively applied. Irrigation systems should be adjusted to provide the minimum water needed. Adjustments should be made for changes in climate and rainfall.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage, and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾-inch crushed gravel to help the collection of water. Landscape areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. The engineer/geologist should observe bottoms for fill, compaction of fill, temporary and soldier pile excavations, lagging and slurry backfill, and subdrains. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request. The geologist will prepare reports summarizing pile excavations.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

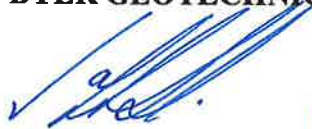
This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.


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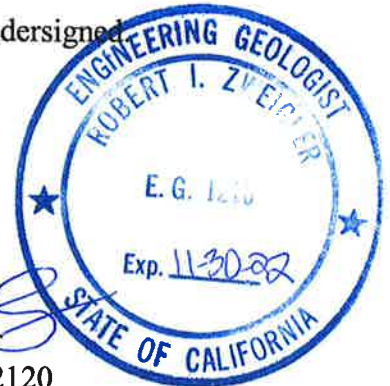
Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.


Raffi S. Babayan
P. E. 72168




Robert I. Zweigler
E. G. 1210/G. E. 2120



RSB:RIZ:cl

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ENCLOSURES AND DISTRIBUTION

- Enc: List of References (6 Pages)
- Appendix I - Laboratory Testing and Log of Borings
 - Laboratory Testing (2 Pages)
 - Shear Test Diagrams (3 Pages)
 - Consolidation Curves (5 Pages)
 - Log of Borings B1 - B3 (8 Pages)
 - Log of Continuous Core Boring 4 (6 Pages)
 - Appendix II - Electronic Piezocone Penetrometer (CPT) Test Results by Kehoe Testing & Engineering
 - Interpretation of Electronic Piezocone (CPT) Data (19 Pages)
 - Appendix III - Graphic CPT Charts
 - Graphic CPT Charts (18 Pages)
 - Appendix IV - Calculations
 - Seismic Sources (2 Pages)
 - Seismic Hazard Deaggregation Chart
 - Site-Specific Ground Motion Analysis (2 Pages)
 - Retaining Wall Calculation Sheets #1 - #3 (10 Pages)
 - Temporary Excavation Height Calculation Sheet #4
 - Soldier Pile Calculation Sheets #5 and #6 (4 Pages)
 - Appendix V - Figures
 - Aerial Vicinity Map
 - Regional Topographic Map
 - Historic Topographic Map
 - Regional Geologic Map #1
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 - Regional Fault Map
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 - Seismic Hazard Zones Map
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 - Plate 1 - Regional Tectonic Map
 - Plate 2 - Regional Structure Map
 - Plate 3 - Regional Drainage Map, Ballona Watershed
 - Plate 4 - Regional DEM (Digital Elevation Model)
 - Plate 5 - Micro Seismicity Map
 - Plate 6 - Structure Map, Hollywood Fault Area
 - Plate 7 - Alluvial Fan Map
 - Geologic Map
 - Sections B and C (2 Sheets)

ENCLOSURES AND DISTRIBUTION (Continued)

In Pocket: Section A

xc: (4) Addressee (E-mail and Mail)

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Software

EZ-FRISK 7.65, Risk Engineering, Inc.

November 30, 2020
BG 23176

APPENDIX I

Laboratory Testing and Log of Borings

LABORATORY TESTING

Undisturbed and bulk samples of the alluvium and older alluvium were obtained from Borings B1, B2, and B3 and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring-lined, barrel sampler conforming to ASTM D 3550-01 with successive drops of the sampler. Experience has shown that sampling causes some disturbance of the sample. However, the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-10. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-10. The results are shown on the enclosed Log of Borings.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill were determined using the procedures outlined in ASTM D 1557-12, a five-layer standard. Remolded samples were prepared at 90 percent of the maximum dry density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Earth Material	USCS + Color Soil Type	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
B2	0 - 10	Fill / Alluvium	Silty Clayey Sand Brown	110.0	17.0	37 - Low

Expansion Test

To find the expansiveness of the soil, a swell test was performed using the procedures outlined in ASTM D 4829-11. Based upon the testing, the near-surface soil is expected to exhibit a low expansion potential.

LABORATORY TESTING (Continued)

Shear Tests

Shear tests were performed on samples of the alluvium and future compacted fill using the procedures outlined in ASTM D 3080-11 and a strain controlled, direct-shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inch per minute. The samples were tested in both *in-situ* and artificially saturated conditions. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the enclosed Shear Test Diagrams.

Consolidation

Consolidation tests were performed on *in situ* samples of the older alluvium using the procedures outlined in ASTM D 2435-11. Results are graphed on the enclosed Consolidation Curves.

Corrosion

A representative bulk sample of the near-surface soil was transported to Environmental Geotechnology Laboratory for chemical testing. The testing was performed in accordance with Caltrans Standards 643 (pH), 422 (Chloride Content), 417 (Sulfate Content), and 532 (Resistivity). The results of the testing are reported in the following table:

CHEMICAL TEST RESULTS TABLE

Sample	Depth (Feet)	pH	Chloride (PPM)	Sulfate (%)	Resistivity (Ohm-cm)
B2	0 - 10	7.34	190	0.023	1,100

The chloride and sulfate contents of the soil are negligible and not a factor in corrosion. The pH is near neutral and not a factor. The resistivity indicates that the soil is considered corrosive to ferrous metals.



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SHEAR TEST DIAGRAM #1

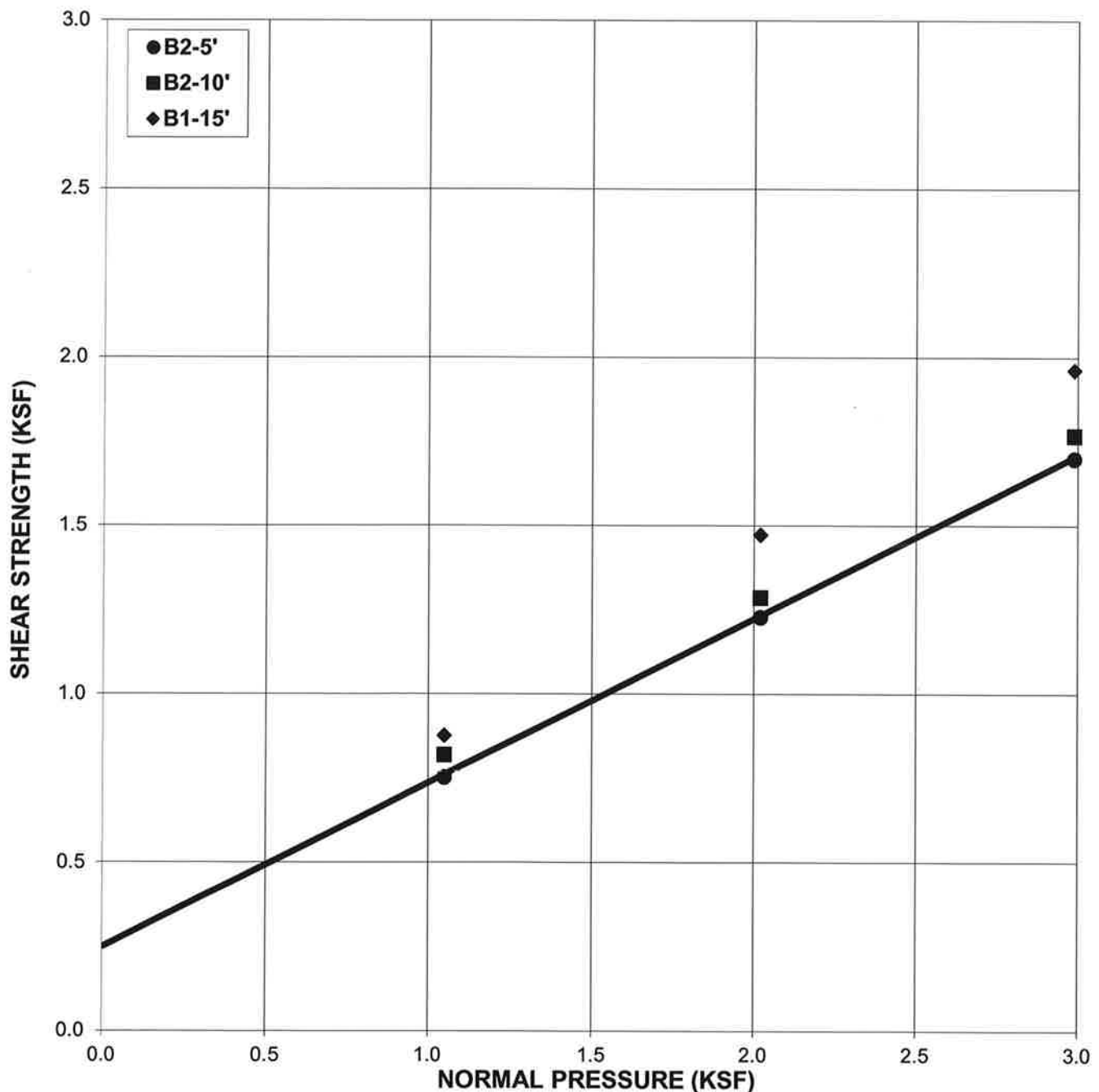
BG: **23176** ENGINEER: **RSB**
CLIENT: **I&L Investment and Management, Inc.**

EARTH MATERIAL: **Alluvium (Saturated)**

Phi Angle = **26.0 degrees**
Cohesion = **250 psf**

Average Moisture Content **29.1%**
Average Dry Density (pcf) **92.9**
Average Saturation **99%**

DIRECT SHEAR TEST - ASTM D-3080 (ULTIMATE VALUES)





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SHEAR TEST DIAGRAM #2

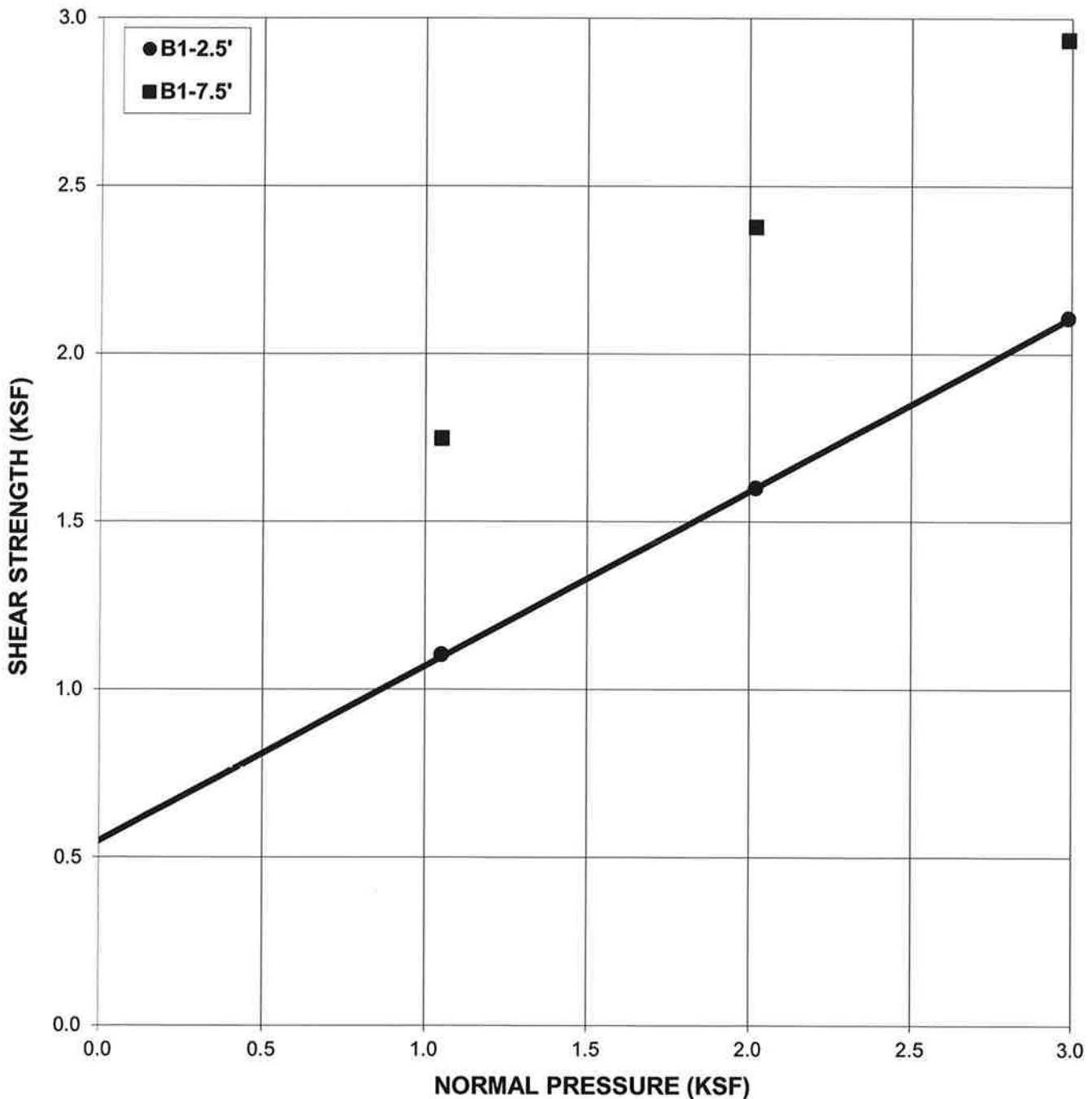
BG: 23176 ENGINEER: RSB
CLIENT: I&L Investment and Management, Inc.

EARTH MATERIAL: Alluvium (In-Situ)

Phi Angle = 27.5 degrees
Cohesion = 550 psf

Average Moisture Content 11.9%
Average Dry Density (pcf) 102.0
Average Saturation 51%

DIRECT SHEAR TEST - ASTM D-3080 (ULTIMATE VALUES)





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SHEAR TEST DIAGRAM #3

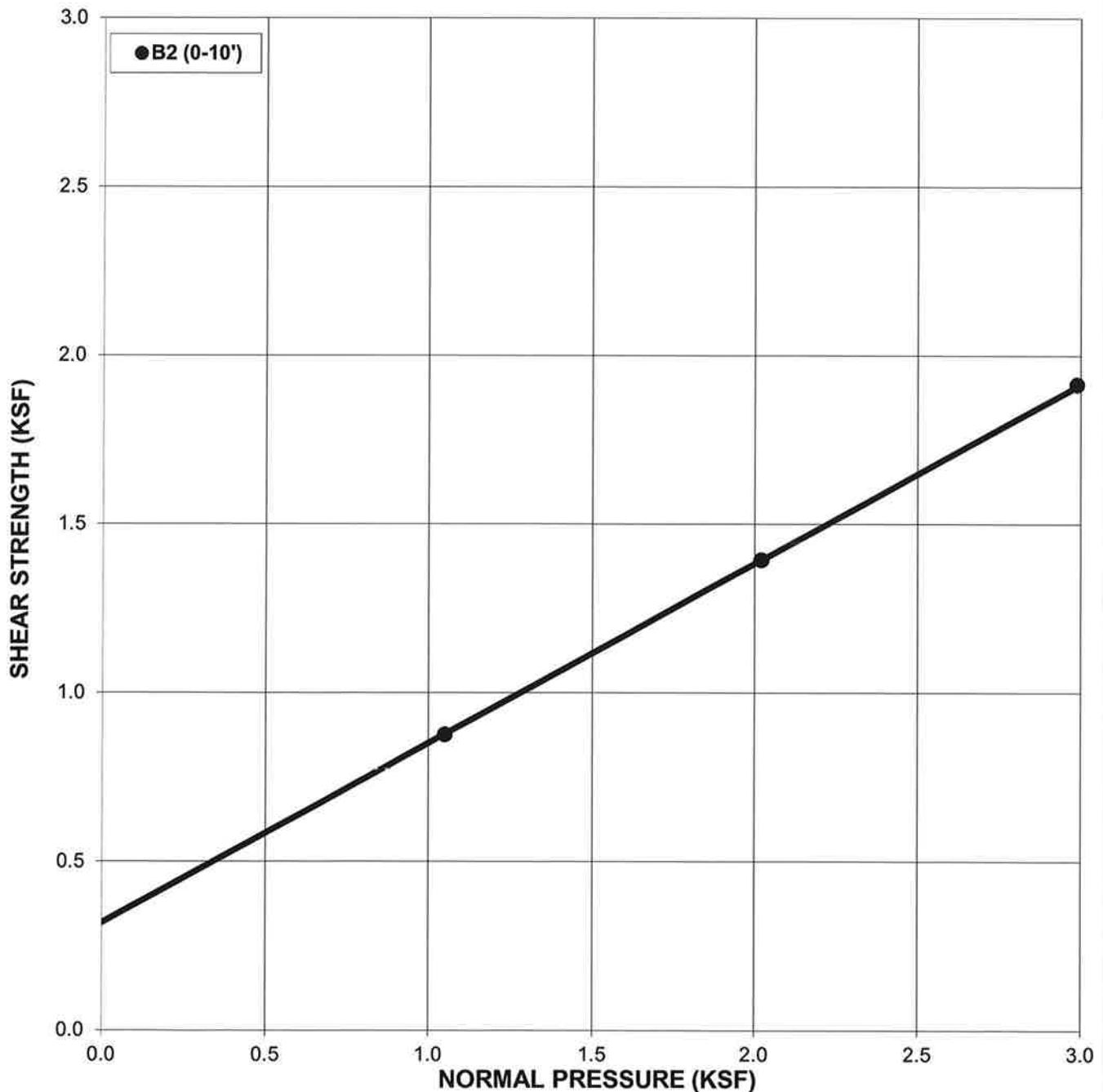
BG: **23176** ENGINEER: **RSB**
CLIENT: **I&L Investment and Management, Inc.**

EARTH MATERIAL: **Future Compacted Fill**
(Remolded at 90%)

Phi Angle = **28.0 degrees**
Cohesion = **320 psf**

Moisture Content **25.1%**
Dry Density (pcf) **99.0**
Saturation **98%**

DIRECT SHEAR TEST - ASTM D-3080 (ULTIMATE VALUES)





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CONSOLIDATION CURVE #1

BG: **23176**

ENGINEER: **RSB**

CLIENT: **I&L Investment and Management, Inc.**

Earth Material: Older Alluvium

Sample Location: B1-15'

Dry Weight (pcf): 93.9

Initial Moisture: 24.6%

Initial Saturation: 85.7%

Water Added at (psf) 1237

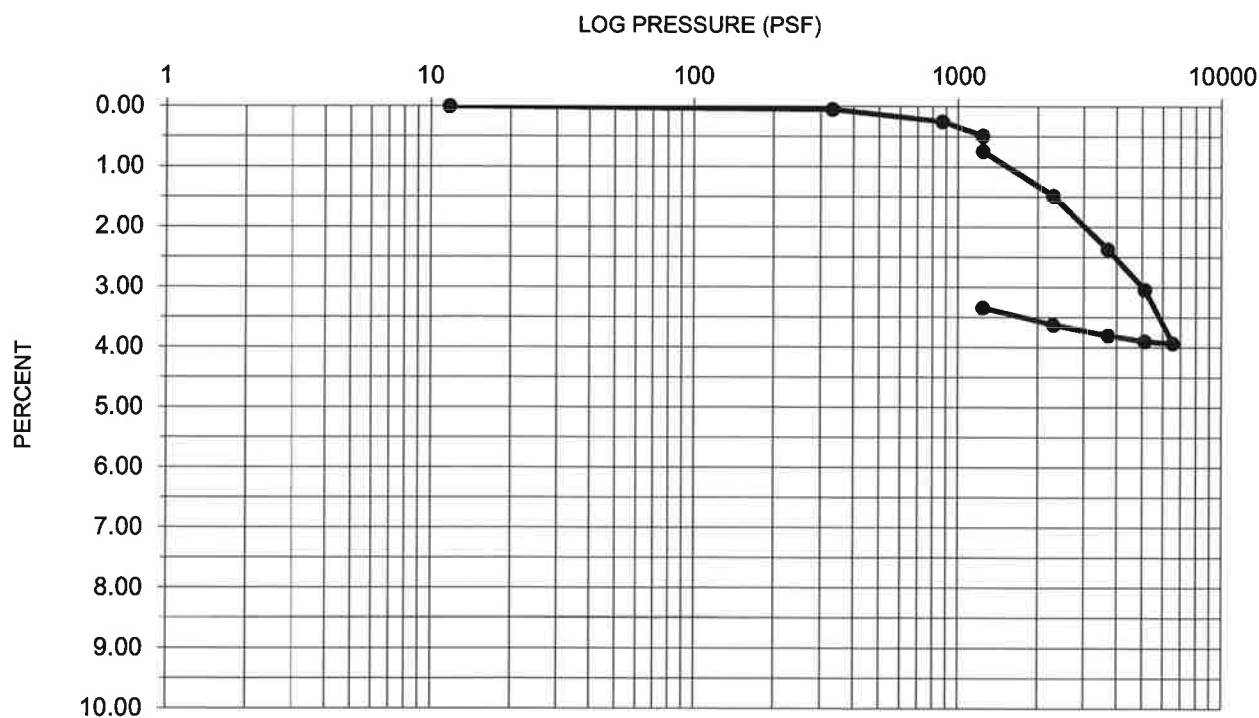
Specific Gravity: 2.65

Initial Void Ratio: 0.76

Compression Index (Cc): 0.147

Recompression Index (Cr): 0.019

CONSOLIDATION DIAGRAM (ASTM D 2435-11)





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CONSOLIDATION CURVE #2

BG: **23176**

ENGINEER: **RSB**

CLIENT: **I&L Investment and Management, Inc.**

Earth Material: Older Alluvium

Sample Location: B2-20'

Dry Weight (pcf): 103.9

Initial Moisture: 20.4%

Initial Saturation: 91.5%

Water Added at (psf) 1237

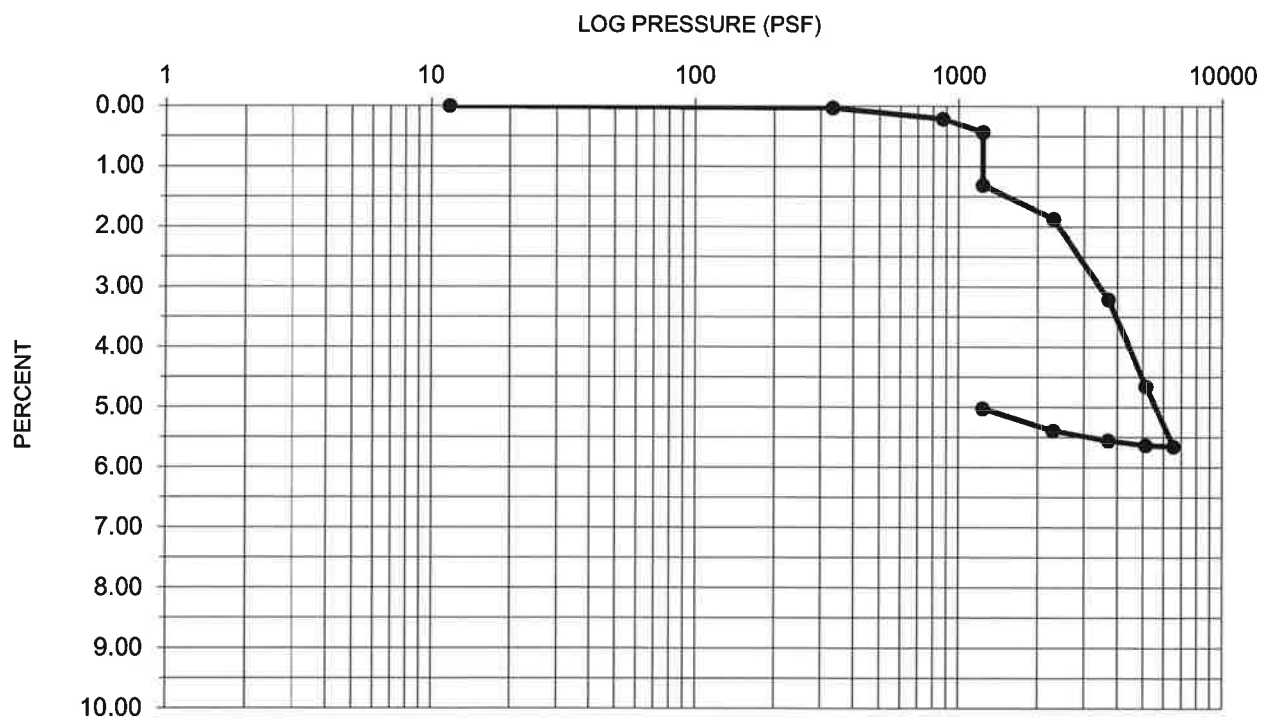
Specific Gravity: 2.65

Initial Void Ratio: 0.59

Compression Index (Cc): 0.162

Recompression Index (Cr): 0.022

CONSOLIDATION DIAGRAM (ASTM D 2435-11)





**BYER
GEOTECHNICAL
INC.**

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tel 818.549.9959 fax 818.543.3747

CONSOLIDATION CURVE #3

BG: **23176**

ENGINEER: **RSB**

CLIENT: **I&L Investment and Management, Inc.**

Earth Material: Older Alluvium

Sample Location: B1-25'

Dry Weight (pcf): 103.6

Initial Moisture: 22.5%

Initial Saturation: 100.0%

Water Added at (psf) 1237

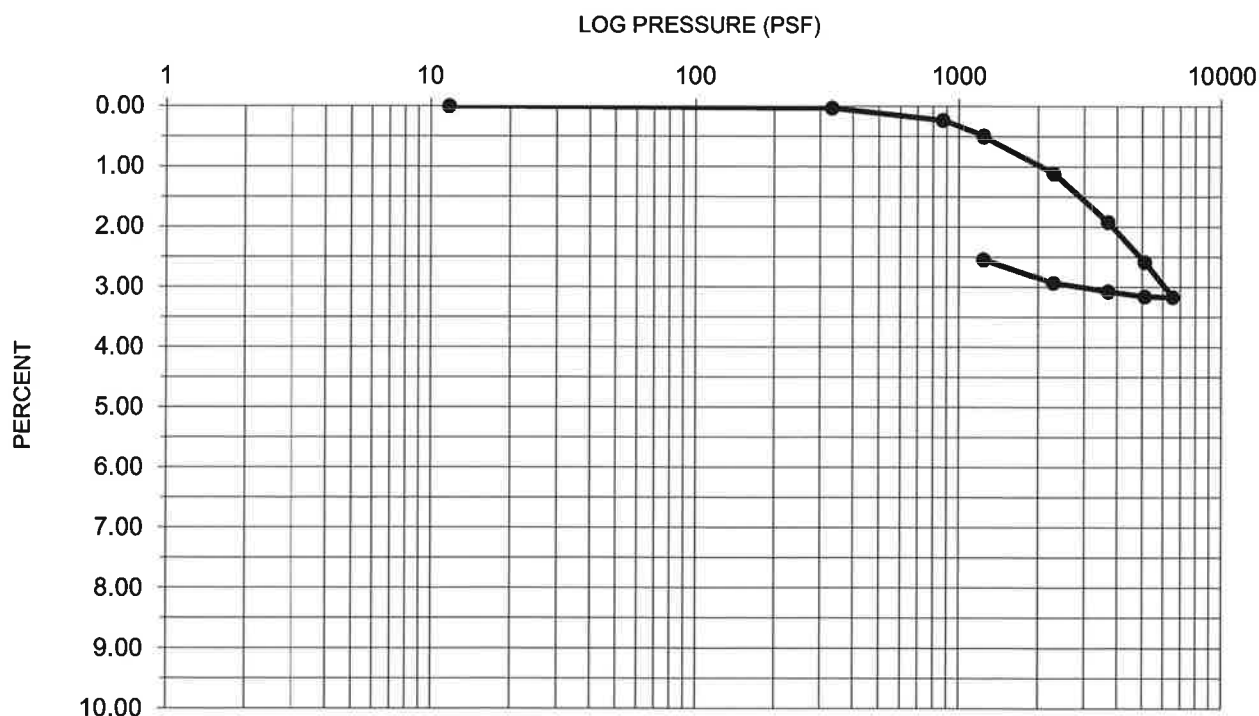
Specific Gravity: 2.65

Initial Void Ratio: 0.60

Compression Index (Cc): 0.087

Recompression Index (Cr): 0.023

CONSOLIDATION DIAGRAM (ASTM D 2435-11)





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CONSOLIDATION CURVE #4

BG: **23176**

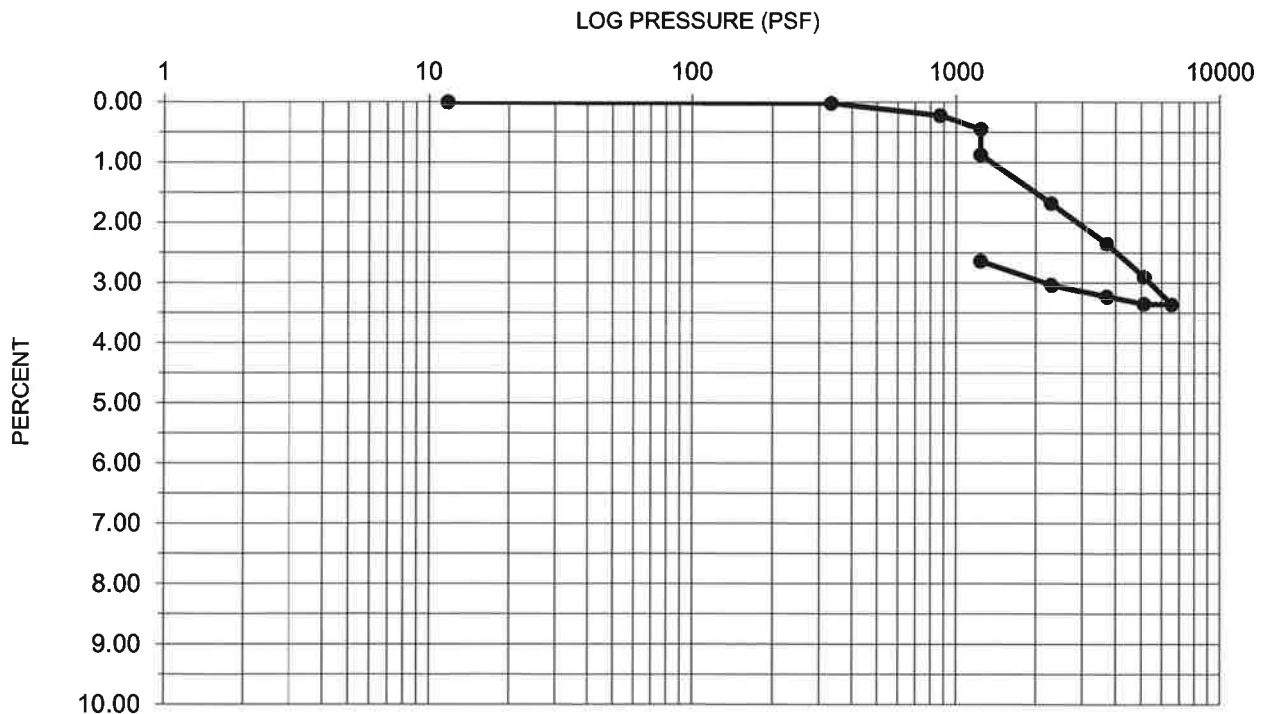
ENGINEER: **RSB**

CLIENT: **I&L Investment and Management, Inc.**

Earth Material: Older Alluvium
Sample Location: B3-30'
Dry Weight (pcf): 117.7
Initial Moisture: 15.3%
Initial Saturation: 100.0%
Water Added at (psf) 1237

Specific Gravity: 2.65
Initial Void Ratio: 0.41
Compression Index (Cc): 0.061
Recompression Index (Cr): 0.021

CONSOLIDATION DIAGRAM (ASTM D 2435-11)





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CONSOLIDATION CURVE #5

BG: **23176**

ENGINEER: **RSB**

CLIENT: **I&L Investment and Management, Inc.**

Earth Material: Older Alluvium

Sample Location: B2-40'

Dry Weight (pcf): 111.7

Initial Moisture: 18.1%

Initial Saturation: 100.0%

Water Added at (psf) 1237

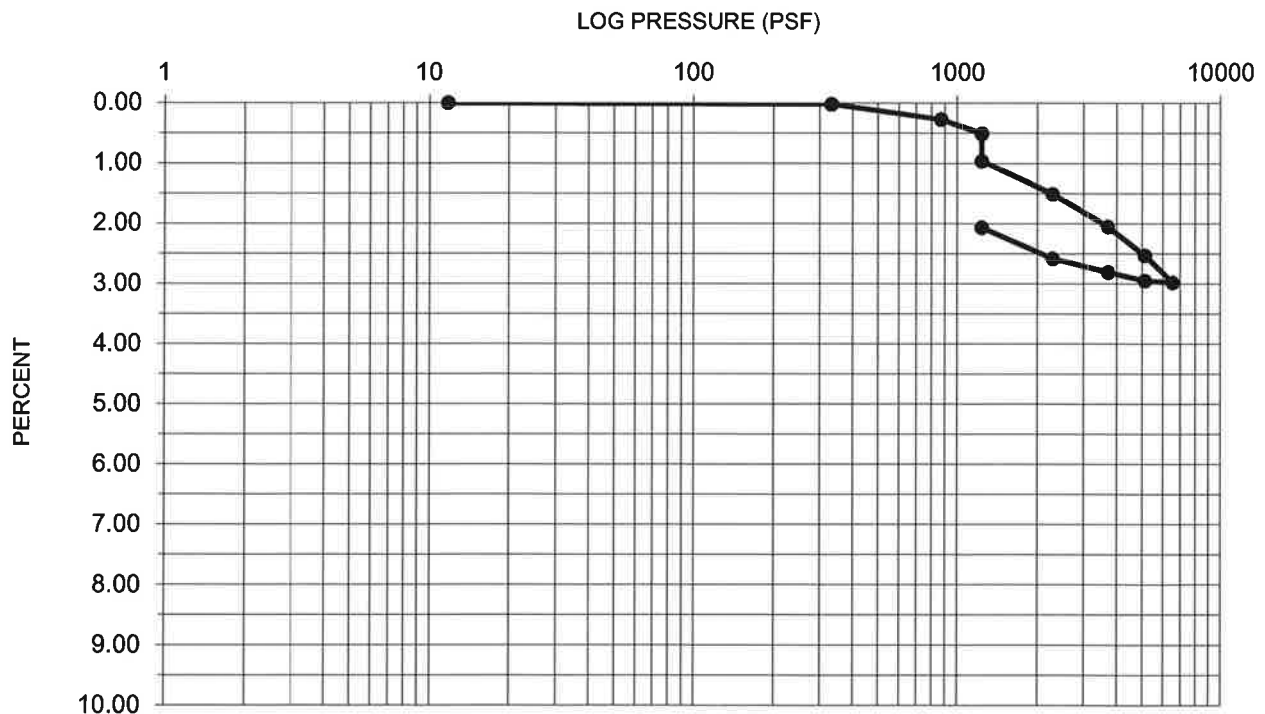
Specific Gravity: 2.65

Initial Void Ratio: 0.48

Compression Index (Cc): 0.062

Recompression Index (Cr): 0.028

CONSOLIDATION DIAGRAM (ASTM D 2435-11)





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LOG OF BORING B1

BG No. 23176

PAGE 1 OF 2

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB

CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches

ELEV. TOP OF HOLE 419 ft

BORING LOG BY RSB - GINT STD US BYER GDT - 11/30/20 - 5:24 - P:\23000 - 23999\23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD\23176 BORING LOGS GPJ

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 2.5" asphalt over 3" base.								
		(SM) <u>ALLUVIUM (Qa)</u> : 0.45' - 2.5': Silty SAND, brown, moist, fine sand, some medium sand.		SM						
415	5	(SC) 2.5': Clayey SAND, brown, moist, loose, fine sand, some medium to coarse sand.		SC	R1	4 4 4	13.6	91.6	44.7	Direct Shear
		(SM) 5': Silty SAND, brown, moist, loose, fine sand, some medium sand, trace coarse sand.		SM	S1	2 3 3	13.2			
410	10	(SM) 7.5': Silty SAND, brown, moist, loose, fine to medium sand, some coarse sand.		SM	R2	5 7 6	10.2	112.4	57.1	Direct Shear
		(SM) 10': Silty SAND, brown, moist, loose, fine to medium sand, some coarse sand.		SM	S2	2 3 3	14.9			
405	15	(CL) <u>OLDER ALLUVIUM (Qae)</u> : 15': Sandy CLAY, brown, moist, stiff to very stiff, fine sand, some medium sand.		CL	R3	5 9 14	24.6	94	85.7	Direct Shear, Consolidation
400	20	(CL) 20': Sandy CLAY, dark brown, moist, very stiff, fine sand, moderately tough.		CL	S3	4 8 15	23.8			
395	25									

Ring Sample

Standard Penetration
Test



BYER GEOTECHNICAL, INC.

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LOG OF BORING B1

BG No. 23176

PAGE 2 OF 2

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB



CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches


ELEV. TOP OF HOLE 419 ft

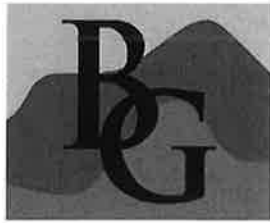
ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25									
		(CL) 25': Sandy CLAY, dark brown, moist, very stiff, fine sand, some medium sand, moderately tough.		CL	R4	4 15 27	22.5	103.6	100	Consolidation
390										
	30									
		(CL) 30': Sandy CLAY, dark brown, moist, very stiff, fine sand, some medium sand, moderately tough.		CL	S4	4 6 12	22.8			

End at 31.5 Feet; No Groundwater; No Fill.

BORING LOG BYER BY RSB - GINT STD US BYER GDT - 11/30/20 15:24 - P:\23000 - 23999\23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD\23176 BORING LOGS.GPJ

Ring Sample

 Standard Penetration
Test



BYER GEOTECHNICAL, INC.

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LOG OF BORING B2

BG No. 23176

PAGE 1 OF 3

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB

CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches

ELEV. TOP OF HOLE 420 ft

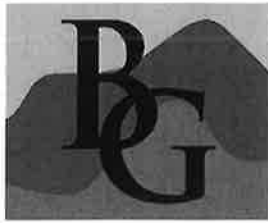
ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
420	0	Surface: 4" asphalt over 1.5" base.								
		(SC) FILL (Afu): 0.45' - 2': Clayey SAND, medium brown, moist, fine to medium sand, trace small asphalt debris.		SC						
		(SM) ALLUVIUM (Qa): (SM) 2.5': Silty SAND, brown, moist, loose, fine sand, some medium sand.		SM						
				SM						
					S1	2 3 3	17.1			
415	5	(SM) 5': Silty SAND, brown, moist, medium dense, fine to medium sand, trace coarse sand.		SM	Bag1	7 11 11	16.1	89.9	50.6	Max, EI, Corrosion Suite Direct Shear
		(SM) 7.5': Silty SAND, brown, moist, loose, fine to medium sand.		SM						
				SM						
					S2	2 2 3	14.8			
410	10	(SM) 10': Silty SAND, brown, moist, loose, fine to medium sand, trace coarse sand.		SM						
				SM						
					R2	3 5 8	9.7	94.7	34.5	Direct Shear
405	15	(CL) OLDER ALLUVIUM (Qae): 15': Sandy CLAY, brown, moist, medium stiff, fine sand, trace medium sand.		CL						
				CL						
					S3	2 2 5	25.2			
400	20	(CL) 20': Sandy CLAY, dark brown, moist, very stiff, fine sand, some medium sand, moderately tough.		CL						
				CL						
					R3	6 17 28	20.4	103.9	91.6	Consolidation
395	25									

BORING LOG BY RSB - GINT STD US BYER GDT - 11/30/20 15:24 - P:\23000 - 23999\23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD\23176 BORING LOGS GPJ

Bulk Sample

Standard Penetration Test

Ring Sample



BYER GEOTECHNICAL, INC.

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LOG OF BORING B2

BG No. 23176

PAGE 2 OF 3

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB

CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches

ELEV. TOP OF HOLE 420 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
395	25	(CL) 25': Sandy CLAY, dark brown, moist, very stiff, fine sand, some medium sand, moderately tough.		CL	S4	5 9 15	14.7			
390	30	(CL) 30': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, trace coarse sand, tough.		CL	R4	20 33 50/4"	14.4	118.5	96.6	
385	35	(CL) 35': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, trace coarse sand, tough.		CL	R5	9 25 30	18.3	111.3	100	
380	40	(CL) 40': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, trace coarse sand, tough.		CL	R6	23 47 50/5"	18.1	111.7	100	Consolidation
375	45	(CL) 45': Sandy CLAY, dark yellowish-brown, moist, very stiff, fine to medium sand.		CL	R7	6 11 27	13.5	114.3	80.1	
370	50									

BORING LOG BYER BY RSB - GINT STD US BYER GDT - 11/30/20 15:24 - P:\23000 - 23999\23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD\23176 BORING LOGS GPJ

Bulk Sample

Standard Penetration Test

Ring Sample



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LOG OF BORING B2

BG No. **23176**

PAGE **3** OF **3**

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB


CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches


ELEV. TOP OF HOLE 420 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
370	50	(CL) 50': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, trace coarse sand, moderately tough.		CL	R8	16 43 50/4"	16.1	115.9	100	
365	55	(CL) 55': Sandy CLAY, dark reddish-brown, moist, very stiff to hard, fine sand.		CL	R9	5 8 40	15.9	116.3	100	
360	60	(CL) 60': Sandy CLAY, dark reddish-brown, moist, hard, fine to medium sand, trace coarse sand, moderately tough.		CL	R10	11 26 41	11.7	121.1	84.9	

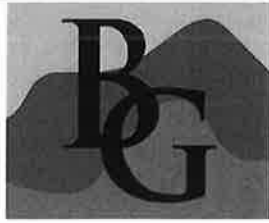
End at 61.5 Feet; No Groundwater; Fill to 2 Feet.

BORING LOG BYER BY RSB - GINT STD US BYER GDT - 11/30/20 15:24 - P:\23000 - 23999\23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD\23176 BORING LOGS.GPJ

 Bulk Sample

 Standard Penetration Test

 Ring Sample



BYER GEOTECHNICAL, INC.

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LOG OF BORING B3

BG No. 23176

PAGE 1 OF 3

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB

CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches

ELEV. TOP OF HOLE 417 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 3" asphalt, no base.		SM						
415		(SM) ALLUVIUM (Qa): 0.25' - 2.5': Silty SAND, brown, moist, fine to medium sand.		SM						
		(SM) 2.5': Silty SAND, brown, moist, loose, fine to medium sand.		SM	S1	2 3 3	13.4			
5		(SC) 5': Clayey SAND, dark brown, moist, medium dense, fine sand, some medium sand.		SC	R1	5 7 11	10.9	84.2	30	
410		(SC) 7.5': Clayey SAND, brown, moist, loose, fine sand, some medium sand.		SC	S2	2 3 3	14.7			
10		(SC) 10': Clayey SAND, brown, moist, loose, fine to medium sand.		SC	R2	5 6 7	13.3	94.1	46.5	
405										
15		(CL) OLDER ALLUVIUM (Qae): 15': Sandy CLAY, brown, moist, medium stiff, fine sand, trace medium sand.		CL	S3	2 2 3	14.9			
400										
20		(CL) 20': Sandy CLAY, dark yellowish-brown, moist, very stiff, fine sand, trace medium sand, moderately tough.		CL	R3	3 9 28	17.9	112.1	100	
395										
25										

BORING LOG BYER BY RSB - GINT STD US BYER GDT - 11/30/20 15:24 - P\230000 - 239999\23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD\23176 BORING LOGS.GPJ

Standard Penetration Test

Ring Sample



BYER GEOTECHNICAL, INC.

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LOG OF BORING B3

BG No. 23176

PAGE 2 OF 3

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB

CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches

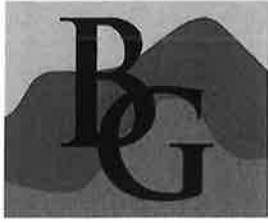
ELEV. TOP OF HOLE 417 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
390	25	(CL) 25': Sandy CLAY, dark yellowish-brown, moist, very stiff, fine sand, trace medium sand.		CL	S4	4 7 11	17.1			
385	30	(CL) 30': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, tough.		CL	R4	10 27 41	15.3	117.7	100	Consolidation
380	35	(CL) 35': Sandy CLAY, dark yellowish-brown, moist, very stiff to hard, fine to medium sand, moderately tough.		CL	S5	5 9 21	15.8			
375	40	(CL) 40': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, moderately tough.		CL	R5	9 29 50/5"	14.8	118.7	100	
370	45	(CL) 45': Sandy CLAY, dark yellowish-brown, moist, hard, fine to medium sand, moderately tough.		CL	R6	11 26 47	15.2	118	100	
50	50									

BORING LOG BY RSB - CINT STD US BYER GDT - 11/30/20 15:24 - P:23000 - 23999/23176 I&L INVESTMENT - FRANKLIN AVE - HOLLYWOOD 23176 BORING LOGS GPJ

Standard Penetration Test

Ring Sample



BYER GEOTECHNICAL, INC.

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LOG OF BORING B3

BG No. 23176

PAGE 3 OF 3

CLIENT I&L Investment and Management, Inc.

REPORT DATE 11/30/20

DRILL DATE 1/17/20

PROJECT LOCATION 5600-5616 W. Franklin Ave., Hollywood, CA

LOGGED BY RSB


CONTRACTOR Martini Drilling

DRILLING METHOD Hollow-Stem Auger

HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches

ELEV. TOP OF HOLE 417 ft

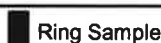
ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50	(CL) 50': Sandy CLAY, dark reddish-brown, moist, hard, fine to medium sand, moderately tough.		CL	R7	14 30 45	16.9	113.6	98.2	

End at 51.5 Feet; No Groundwater; No Fill.

BORING LOG BYER BY RSB - GINT STD US BYER GDT - 11/30/20 - 5:24 - P:\23000 - 23999\23176 I&L INVESTMENT_FRANKLIN AVE_HOLLYWOOD\23176 BORING LOGS GPJ



Standard Penetration
Test



Ring Sample

Soil Description - Section A, Continuous Core Boring 4 Top Elevation 418		
Depth (feet)	Horizon	Description
0.0-3.5	NR	No Recovery
3.5-4.0	B	RECENT ALLUVIUM: Light olive-brown (2.5Y 5/3d), Sandy Loam, massive structure, fine to medium grained, granular, no gravel, non-sticky, non-plastic, very firm, hard, no clay films observed, gradational lower boundary, few pores.
4.0-4.1	C	Marker Bed #2 Contains 50% rounded granitic gravel in Sandy Loam matrix.
4.1-5.0	2B	Very dark grayish brown (10YR 3/2d) Sandy Clay, very fine and coarse grained, massive subangular blocky structure, <10% gravel, firm, slightly hard, non sticky, non plastic, no clay films observed, undetermined lower boundary
5.0-6.3	NR	No Recovery
6.3-7.8	2B	Very dark grayish brown (10YR 3/2d), Sandy Clay, massive subangular blocky structure, very fine to medium and coarse grained, no gravel, firm, slightly hard, non-sticky, non-plastic, no clay films observed, clear transition to:
7.8-8.3	2C	Olive (2.5Y 4/3d), Sand, friable, non sticky, non plastic, single grain granular structure, medium to coarse grained, no gravel, no clay, loose, films bridging grains, clear transition to:
8.3-8.7	2C	2" rounded gravel (>75%) in Dark grayish brown (10YR4/2d) Sandy Clay matrix, friable, slightly hard
8.7-10.0	3B	OLDER ALLUVIUM: Dark grayish brown (10YR4/2d), Sandy Clay, massive structure, very and fine to coarse grained, no gravel, slightly sticky, slightly plastic, friable, slightly hard, very few clay films, pores, few decaying roots,
10.0-12.4	3C	OLDER ALLUVIUM: Marker #3 Olive (2.5Y 5/3d), Loam, massive structure, very fine to medium grained, no gravel, hard, non-sticky, non-plastic, very firm, very hard, no clay films observed, few pores, gradational boundary to:
12.4-13.9	3C	Olive (2.5Y 4/3d), Silty Loam, massive structure, very fine to fine grained, no gravel, non sticky, non plastic, firm, hard, no clay films bridging grains, many pores, gradational boundary to:

Soil Description - Section A, Continuous Core Boring 4 Top Elevation 418		
Depth (feet)	Horizon	Section A; B-4 Description
13.9-15.0	3C	Olive (2.5Y 4/4d), Silty Clay Loam, massive structure, very fine to medium grained, no gravel, non-sticky, non-plastic, firm, slightly hard, pores, no clay films observed
15.0-16.5	NR	No Recovery
16.5-18.1	3C	Olive (2.5Y 4/4d), Loam, massive subangular blocky structure, very friable, very fine to medium grained, no gravel, non sticky, non plastic, very few clay films bridging grains
18.1-19.8	4B	VERY OLD ALLUVIUM: Very dark grayish brown (10YR 3/2d), Clay Loam, massive subangular blocky structure, no gravel, hard, slightly sticky, slightly plastic, extremely firm, extremely hard, few clay films bridging grains
19.8 -24.8	4C	Brown (10YR 4/3d), Clay Loam, weak columnar structure, very fine to fine grained, no gravel, slightly sticky, slightly plastic, clay films observed, some carbonates, few pores, clear boundary to:
24.8-25.8	4C	Dark yellowish-brown (10YR 5/4d), Clay Loam, moderate columnar structure, very fine to fine and coarse grained, <10% gravel, slightly sticky, slightly plastic, clay films observed, extremely firm, extremely hard, clay films observed
25.8-27.8	4C	Dark yellowish-brown (10YR 5/4d), Clay Loam, weak columnar structure, no gravel, slightly sticky, slightly plastic, firm, hard, clay films observed, black staining
27.8-29.0	5B	Brown (10YR 4/3d), Sandy Clay Loam, weak columnar structure, very fine to fine and coarse grained, no gravel, slightly sticky, slightly plastic, firm, slightly hard, few clay films bridging grains
29.0-30.0	5C	Brown (10YR 4/3d), Loam, weak columnar structure, very fine to fine grained, no gravel, slightly sticky, slightly plastic, extremely firm, extremely hard, clay films bridging grains

Soil Description - Section A, Continuous Core Boring 4 Top Elevation 418		
Depth (feet)	Horizon	Section A; B-4 Description
30.0-34.0	5C	Yellowish-brown (10YR 5/4d), Clay Loam, medium subangular blocky structure, very fine to fine and very coarse grained, no gravel, slightly sticky, slightly plastic, very firm, hard, clay films observed, black staining
34.0-35.0	5C	Yellowish-brown (10YR 5/4d), Sandy Loam, medium columnar structure, very fine to fine and coarse grained, no gravel, slightly sticky, slightly plastic, extremely firm, extremely hard, clay films observed, gradational lower boundary to:
35.0-36.0	5C	Yellowish-brown (10YR 5/4d), Clay Loam, very fine to fine and coarse grained, no gravel, slightly sticky, slightly plastic, very firm, hard, clay films bridging grains, black staining
36.0-37.5	5C	Mottled dark yellowish-brown (10YR 4/6d) with abundant black staining, Loam, medium columnar structure, very fine to fine and coarse grained, no gravel, slightly sticky, slightly plastic, firm, hard, clay films,
37.5-38.3	5C	Brown (7.5YR 4/4d), Loam, medium subangular blocky structure, very fine to medium grained, no gravel, slightly sticky, slightly plastic, firm, slightly hard, clay films
38.3-38.8	5C	Brown (7.5YR 4/4d), Sand, granular to horizontally layered, medium to coarse grained, <10% gravel, non sticky, non plastic, firm, slightly hard, abrupt contact with:
38.8-42.8	5C	Brown (7.5YR 4/4d), Sandy Loam, subangular blocky structure, fine to coarse grained, <10% gravel, non-sticky to slightly sticky, non-plastic, very few clay films on gravel, undetermined lower boundary.
42.8-43.4	5C	Yellowish-brown (10YR 5/4d), Sandy Loam, subangular blocky structure, no gravel, fine to coarse grained, non sticky, non plastic, firm, hard, clay films

Soil Description - Section A, Continuous Core Boring 4 Top Elevation 418		
Depth (feet)	Horizon	Section A; B-4 Description
43.4-44.6	5C	Dark Yellowish Brown (10YR 4/4d), , Sandy Loam, subangular blocky structure, <10% gravel, fine to coarse grained, non sticky, non plastic, firm, slightly hard, clay films
44.6-48.4	5C	Dark Yellowish Brown (10YR 4/4d), Sandy Clay Loam, fine to medium grained, no gravel, non sticky, non plastic, firm, slightly hard, clay films
48.4-50.0	6B	Brown (7.5YR 4/3d), Sandy Clay Loam, moderate columnar structure, very fine to fine and coarse grained, no gravel, slightly sticky to sticky, slightly plastic, very firm, hard, clay films on ped faces, gradational lower contact to:
50.0-51.0	6C	Brown (7.5YR 5/3d), Clay Loam, moderate subangular blocky structure, very fine to fine grained, no gravel, slightly sticky, slightly plastic to plastic, friable, slightly hard, common distinct clay films on ped faces, gradational lower boundary to:
51.0-52.0	6C	Brown (7.5YR 4/4d), Sandy Clay Loam, moderate subangular blocky structure, very fine to fine grained, no gravel, non sticky, non plastic, friable, slightly hard, few clay films
52.0-53.0	6C	Brown (7.5YR 5/3d), Sandy Clay Loam, moderate subangular blocky structure, very fine to fine grained, no gravel, non sticky, non plastic, friable, slightly hard, few clay films
53.0-53.5	6C	Brown (7.5YR 5/3d), Clay Loam, moderate angular blocky structure, very fine to medium grained, <10% gravel, slightly sticky, plastic, few clay films on ped faces, 2" Gravel at base of unit
53.5-54.3	5C	Brown (7.5YR 5/3d), Sandy Clay Loam, weak subangular blocky structure, no gravel, slightly sticky, slightly plastic, few clay films
54.3-55.0	5C	Marker Bed #4Gray (10YR 6/1d), Loam, fine to coarse grained, no gravel, non-sticky to slightly sticky, non-plastic to slightly plastic, extremely firm, very hard, few clay films on gravel, clear lower boundary to:

Soil Description - Section A, Continuous Core Boring 4 Top Elevation 418		
Depth (feet)	Horizon	Section A; B-4 Description
55.0-66.0	7B	Brown (10YR 5/3d), Clay Loam, moderate columnar structure, very fine to fine and coarse grained, no gravel, slightly sticky to sticky, slightly plastic, very firm, hard, clay films on ped faces,
66.0-68.0	7C	Yellowish-brown (10YR 5/4d), Sandy Clay Loam, subangular blocky structure, fine to coarse grained, no gravel, non sticky, non plastic, few clay films
68.0-68.6	7C	Dark yellowish-brown (10YR 4/4d), Clay Loam, subangular blocky structure, very fine to medium grained, no gravel, slightly sticky, slightly plastic, clay films
68.6-70.0	8B	Dark grayish brown (10YR4/2d), Clay Loam, subangular blocky structure, very fine to medium grained, no gravel, slightly sticky, slightly plastic, extremely firm, extremely hard, clay films
70.0-73.0	8C	Brown (10YR5/3d), Sandy Clay Loam, subangular blocky structure, fine to coarse grained, no gravel, sticky, slightly plastic, firm, slightly hard, clay films
73.0-75.0	9B	Yellowish Brown(10YR5/4d), Clay Loam, subangular blocky structure, very fine to fine grained, no gravel, slightly sticky, slightly plastic, very firm, very hard, clay films
75.0-78.0	9C	Yellowish Brown (10YR5/4d), Sandy Clay Loam, subangular blocky structure, fine to coarse grained, no gravel, sticky, plastic, firm, slightly hard, clay films
78.0-80.0	10B	Brown (7.5YR5/3d), Clay Loam, subangular blocky structure, very fine to fine grained, no gravel, slightly sticky, slightly plastic, firm, hard, clay films
80.0-82.5	10B	Dark Yellowish Brown (10YR4/4d), Clay Loam, subangular blocky structure, very fine to fine grained, no gravel, sticky, plastic, extremely firm, extremely hard, clay films

Soil Description - Section A, Continuous Core Boring 4 Top Elevation 418		
Depth (feet)	Horizon	Section A; B-4 Description
82.5-84.0	10B	Brown (7.5YR4/2d), Clay, subangular blocky structure, very fine to fine grained, no gravel, sticky, plastic, extremely firm, extremely hard, clay films
84.0-84.5	10B	Brown (10YR4/3d), Clay, subangular blocky structure, very fine to fine grained, no gravel, sticky, plastic, very firm, very hard, clay films

End at 84.5 feet. Perched groundwater filled boring to 82 feet, after removing augers.

November 30, 2020
BG 23176

APPENDIX II

Electronic Piezocone Penetrometer (CPT) Test Results
by Kehoe Testing & Engineering

SUMMARY OF CONE PENETRATION TEST DATA

Project:

**5600 W. Franklin Avenue
W. Hollywood, CA
January 16-17, 2020**

Prepared for:

**Mr. Raffi Babayan
Byer Geotechnical, Inc.
1461 E. Chevy Chase Drive, Ste 200
Glendale, CA 91206-4090
Office (818) 549-9959 / Fax (818) 543-3747**

Prepared by:



KEHOE TESTING & ENGINEERING

5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

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- 1. INTRODUCTION**
- 2. SUMMARY OF FIELD WORK**
- 3. FIELD EQUIPMENT & PROCEDURES**
- 4. CONE PENETRATION TEST DATA & INTERPRETATION**

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Pore Pressure Dissipation Graphs
- CPT Data Files (sent via email)

SUMMARY OF CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 5600 W. Franklin Avenue in W. Hollywood, California. The work was performed by Kehoe Testing & Engineering (KTE) on January 16-17, 2020. The scope of work was performed as directed by Byer Geotechnical, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at 11 locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	100	
CPT-2	100	
CPT-3	2	Refusal
CPT-3A	100	
CPT-4	100	
CPT-5	100	
CPT-6	3	Refusal
CPT-6A	100	
CPT-7	100	
CPT-8	100	
CPT-9	100	

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm² cone and recorded the following parameters at approximately 2.5 cm depth intervals:

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

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of

baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test...", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (q_c), sleeve friction (f_s), and penetration pore pressure (u). The friction ratio (R_f), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on q_c , f_s and u . In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

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

Sincerely,

KEHOE TESTING & ENGINEERING



Steven P. Kehoe
President

APPENDIX



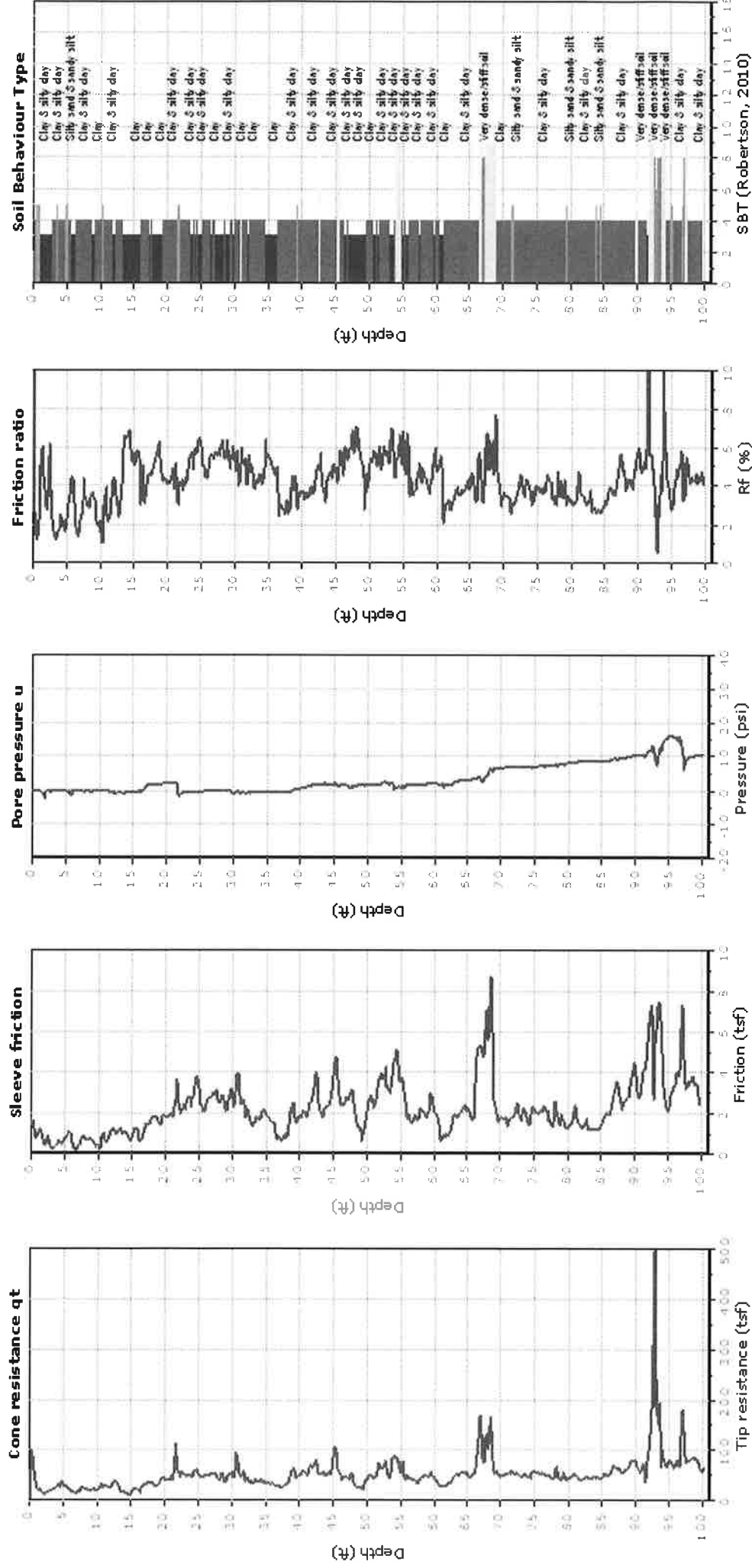
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

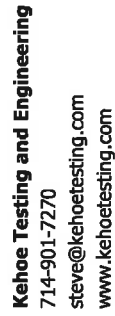
Project: Byer Geotechnical

Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-1

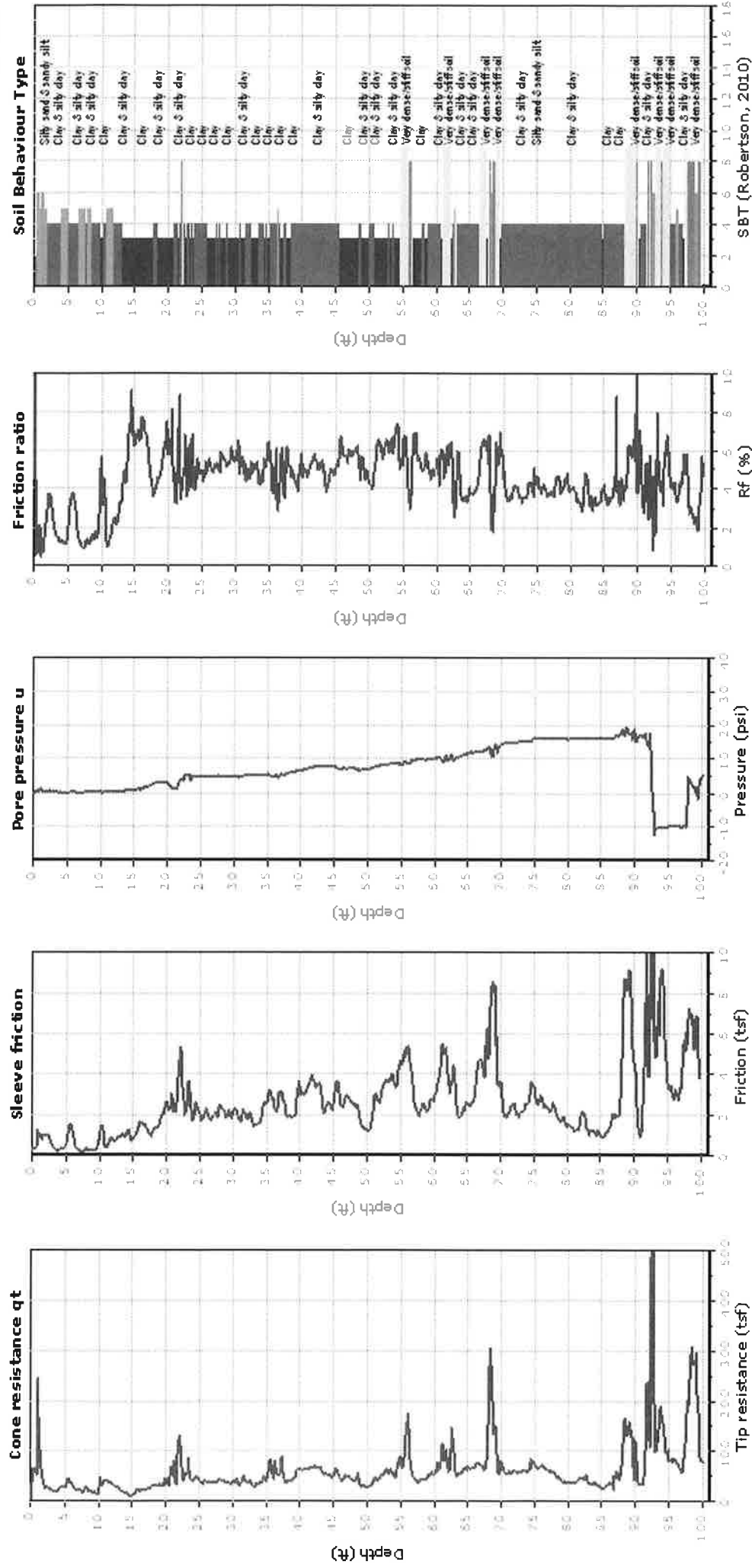
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Location: 5600 W. Franklin Ave, W. Hollywood, CA

Total depth: 100.21 ft, Date: 1/16/2020





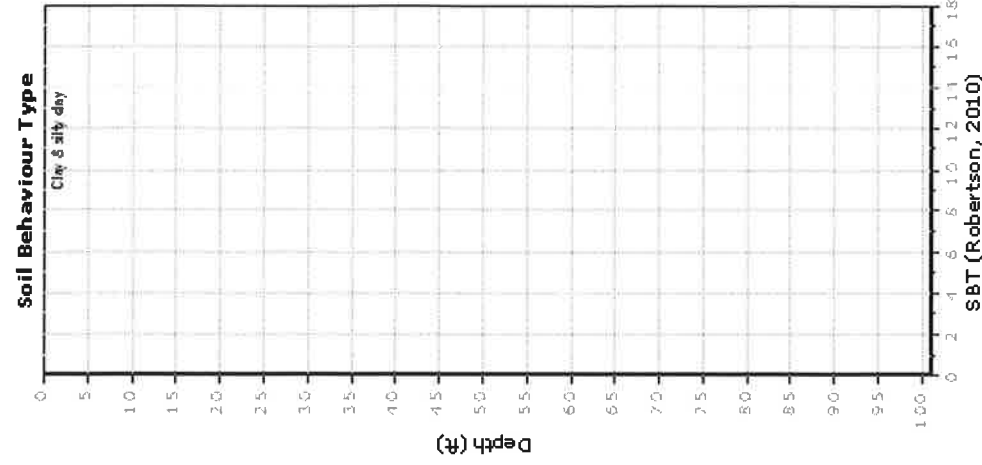
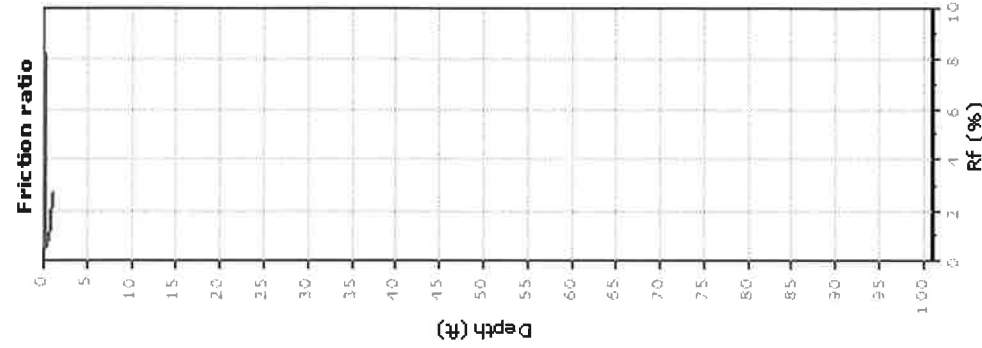
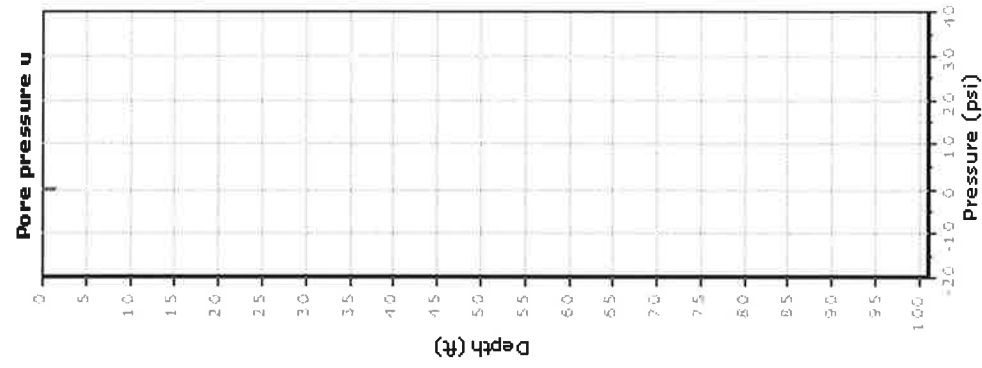
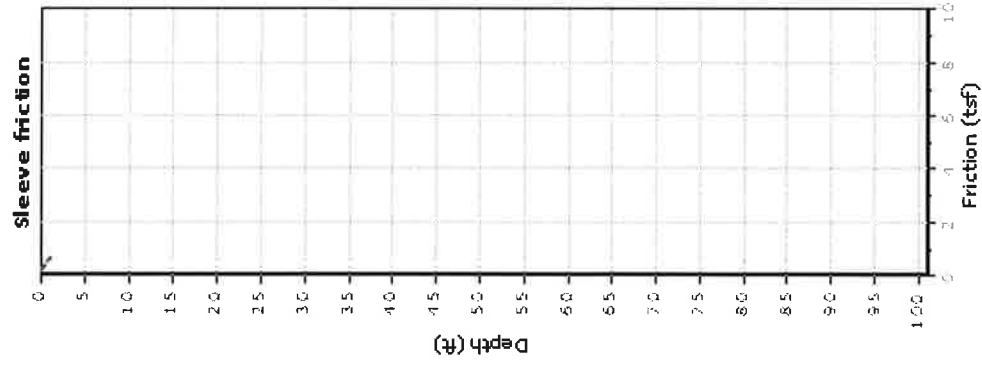
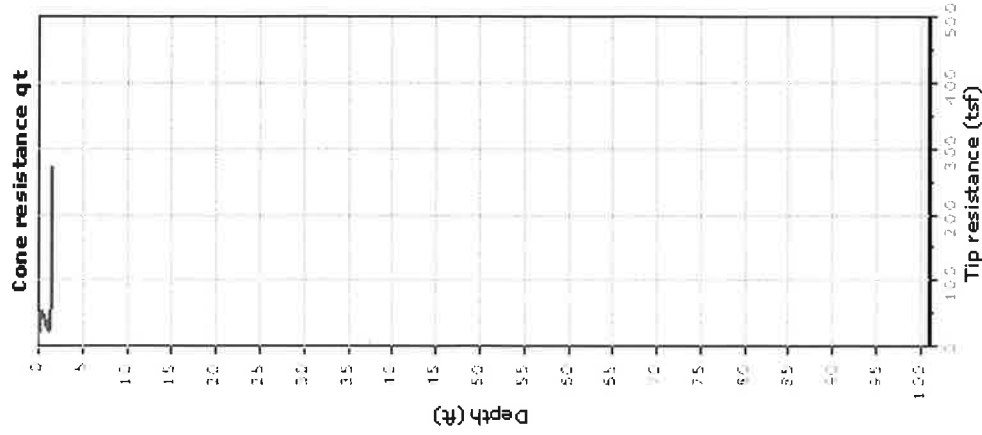
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Byer Geotechnical

Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-3

Total depth: 1.53 ft, Date: 1/16/2020

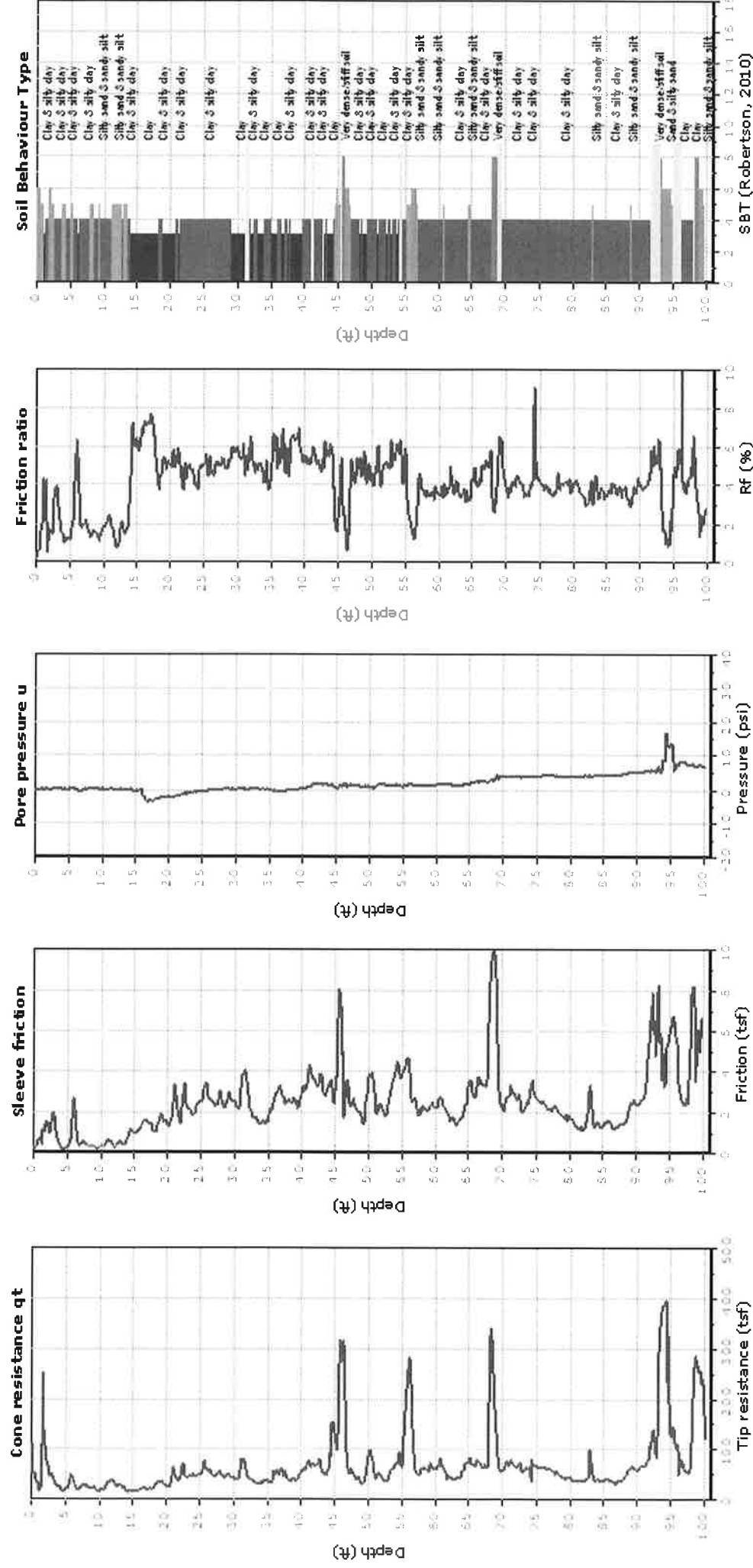




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Project: Byer Geotechnical
Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-3A
Total depth: 100.20 ft, Date: 1/16/2020





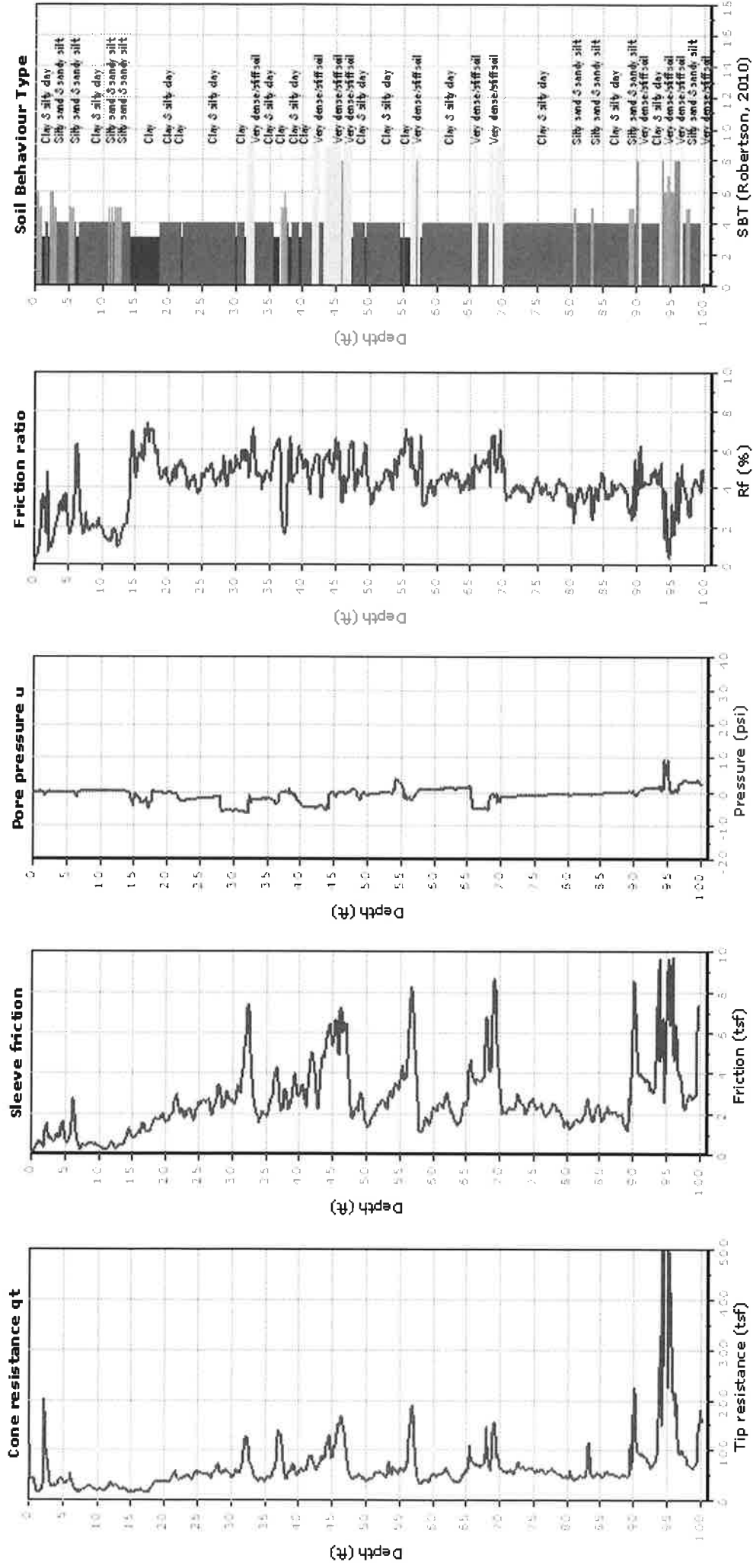
Kehoe Testing and Engineering
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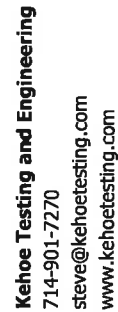
Project: Byer Geotechnical

Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-4

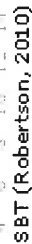
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Location: 5600 W. Franklin Ave, W. Hollywood, CA

Total depth: 100.35 ft. Date: 1/16/2020





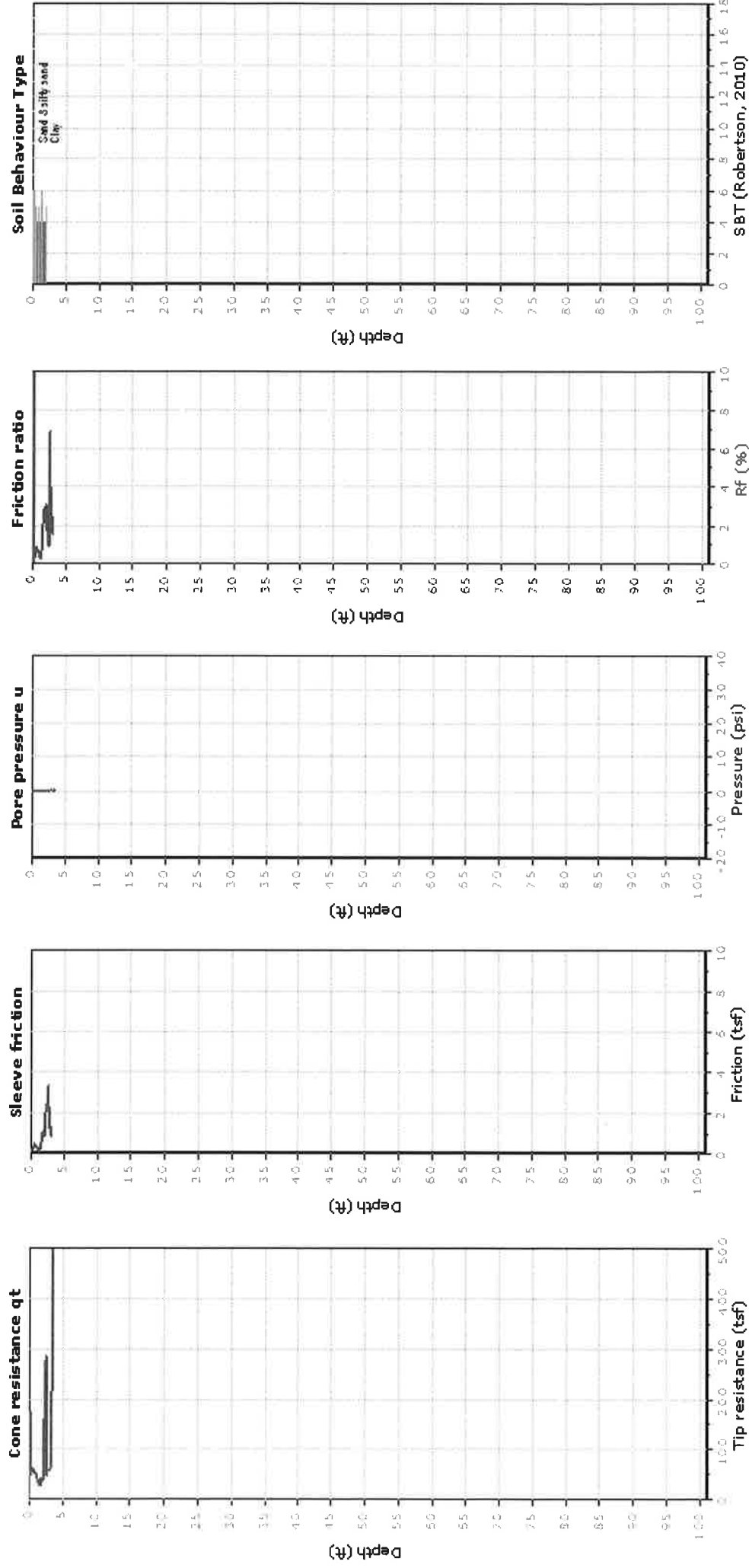
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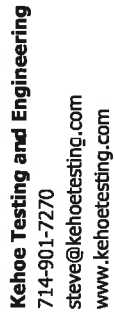
Project: Byer Geotechnical

Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-6

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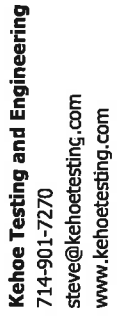




Location: 5600 W. Franklin Ave, W. Hollywood, CA

Total depth: 100.20 ft, Date: 1/17/2020





Location: 5600 W. Franklin Ave, W. Hollywood, CA

Total depth: 100.34 ft, Date: 1/17/2020





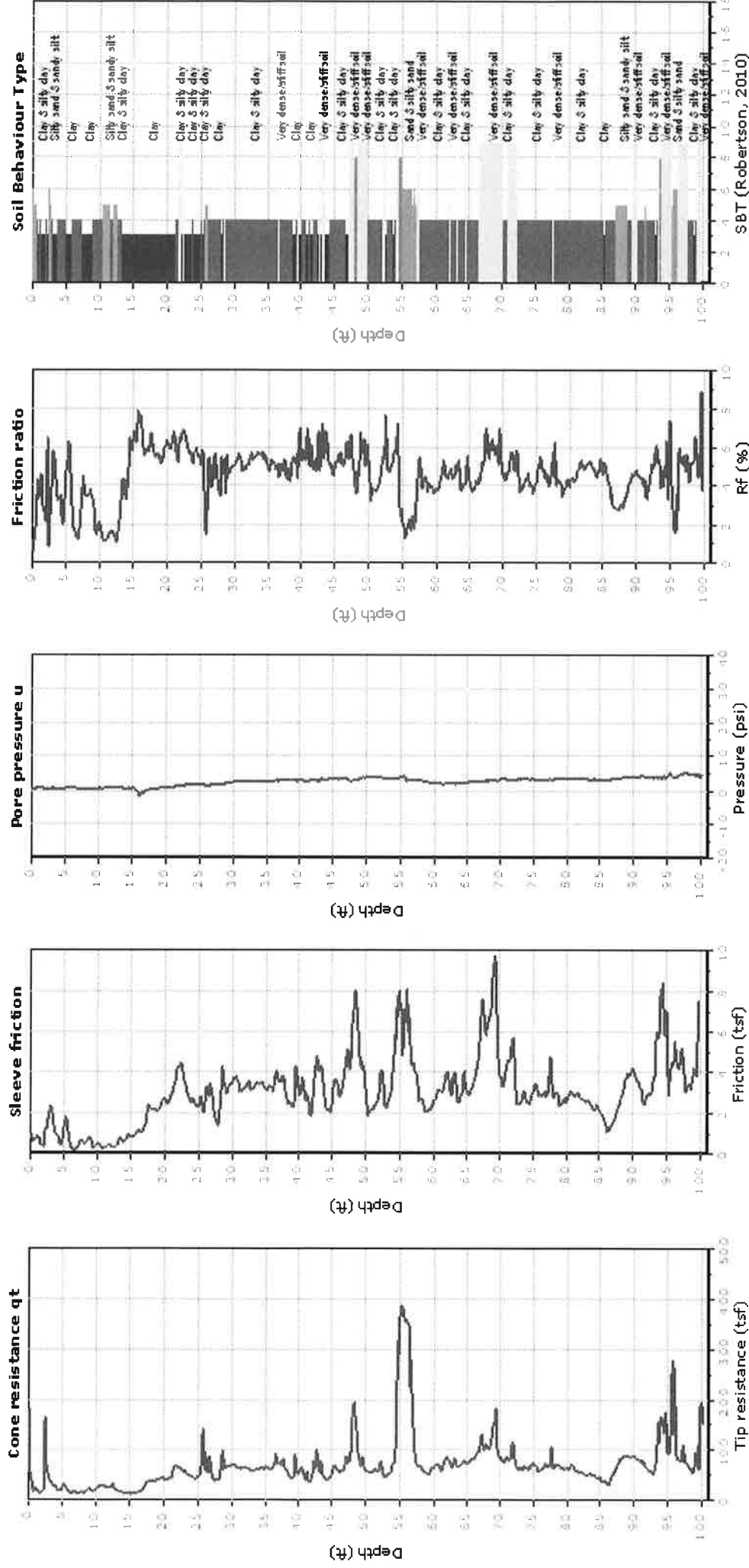
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714-901-7270
steve@kehoetesting.com
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Project: Byer Geotechnical

Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-8

Total depth: 100.20 ft, Date: 1/17/2020





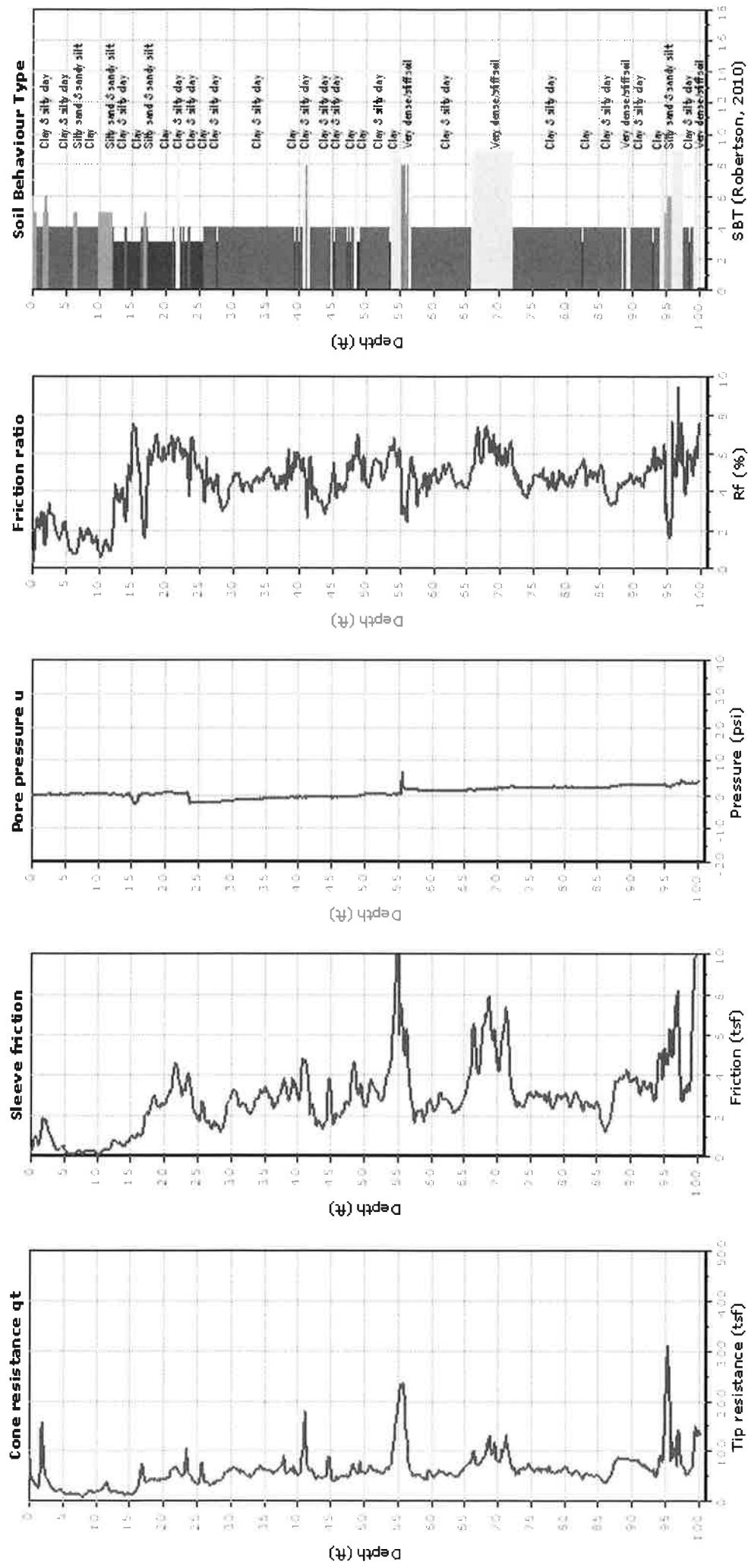
Kehoe Testing and Engineering
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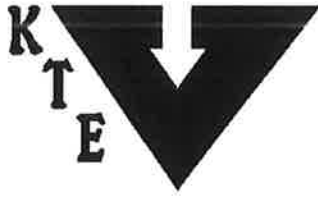
Project: Byer Geotechnical

Location: 5600 W. Franklin Ave, W. Hollywood, CA

CPT-9

Total depth: 100.21 ft, Date: 1/17/2020



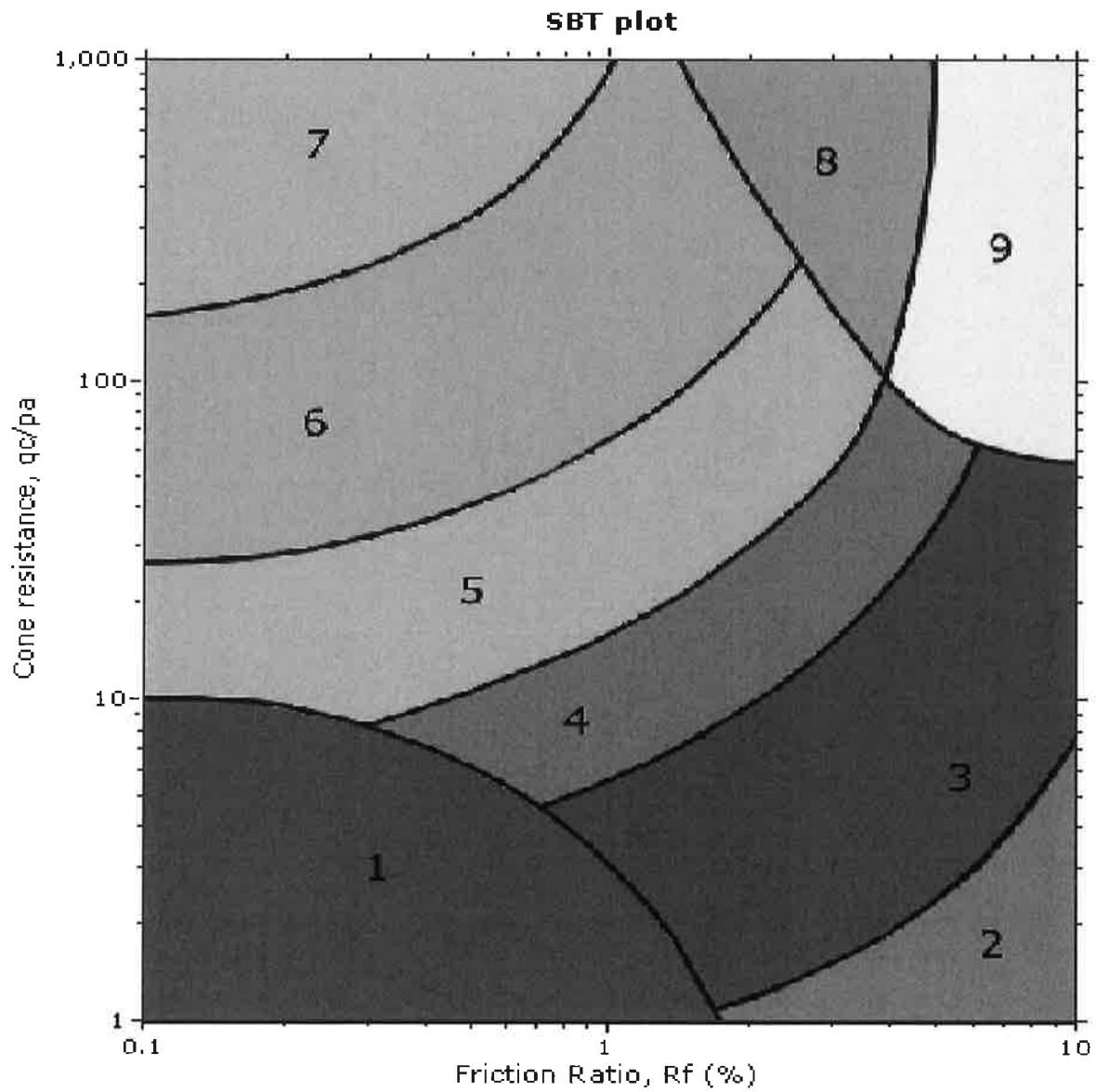


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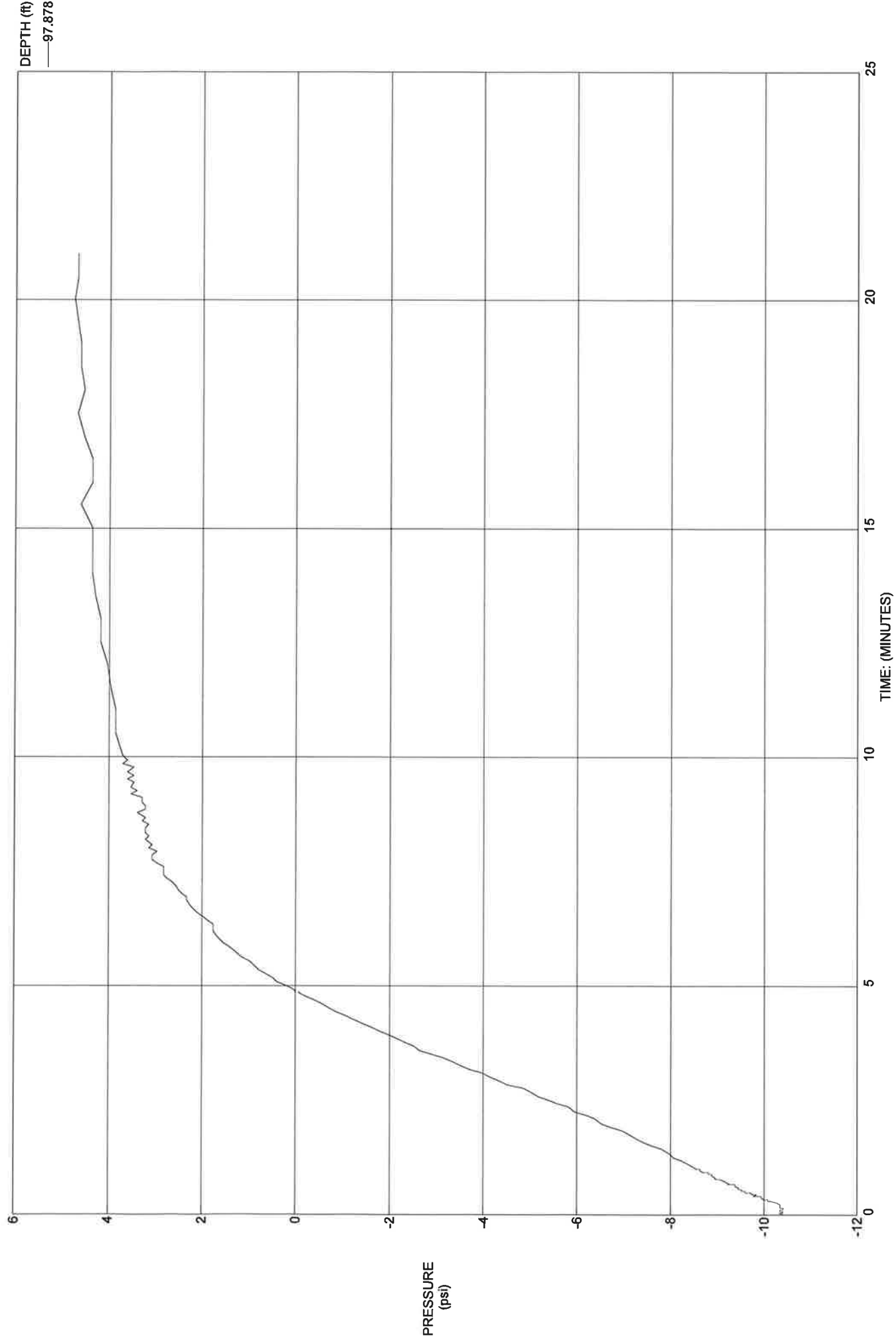
www.kehoetesting.com



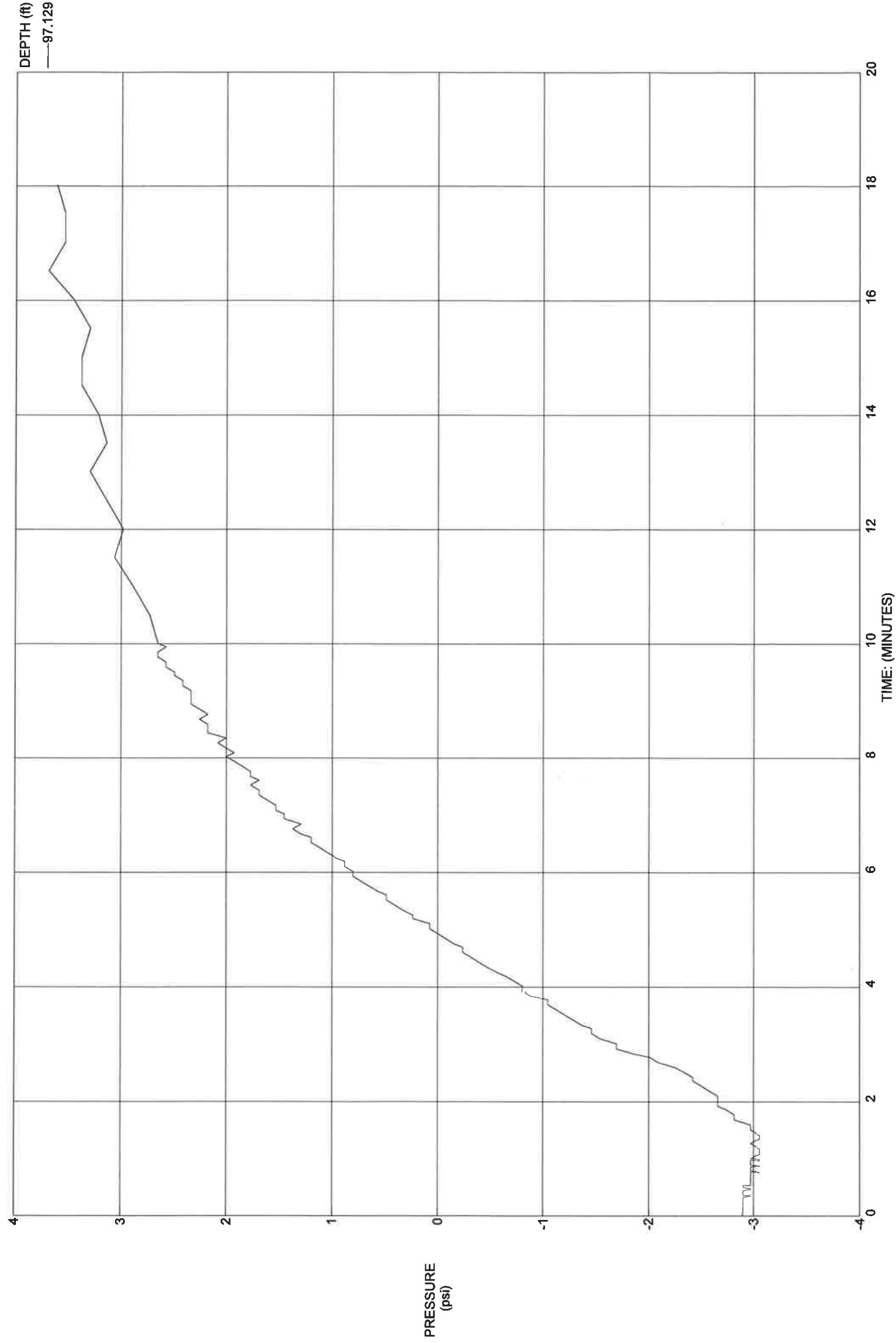
SBT legend

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

TEST ID: CPT-2



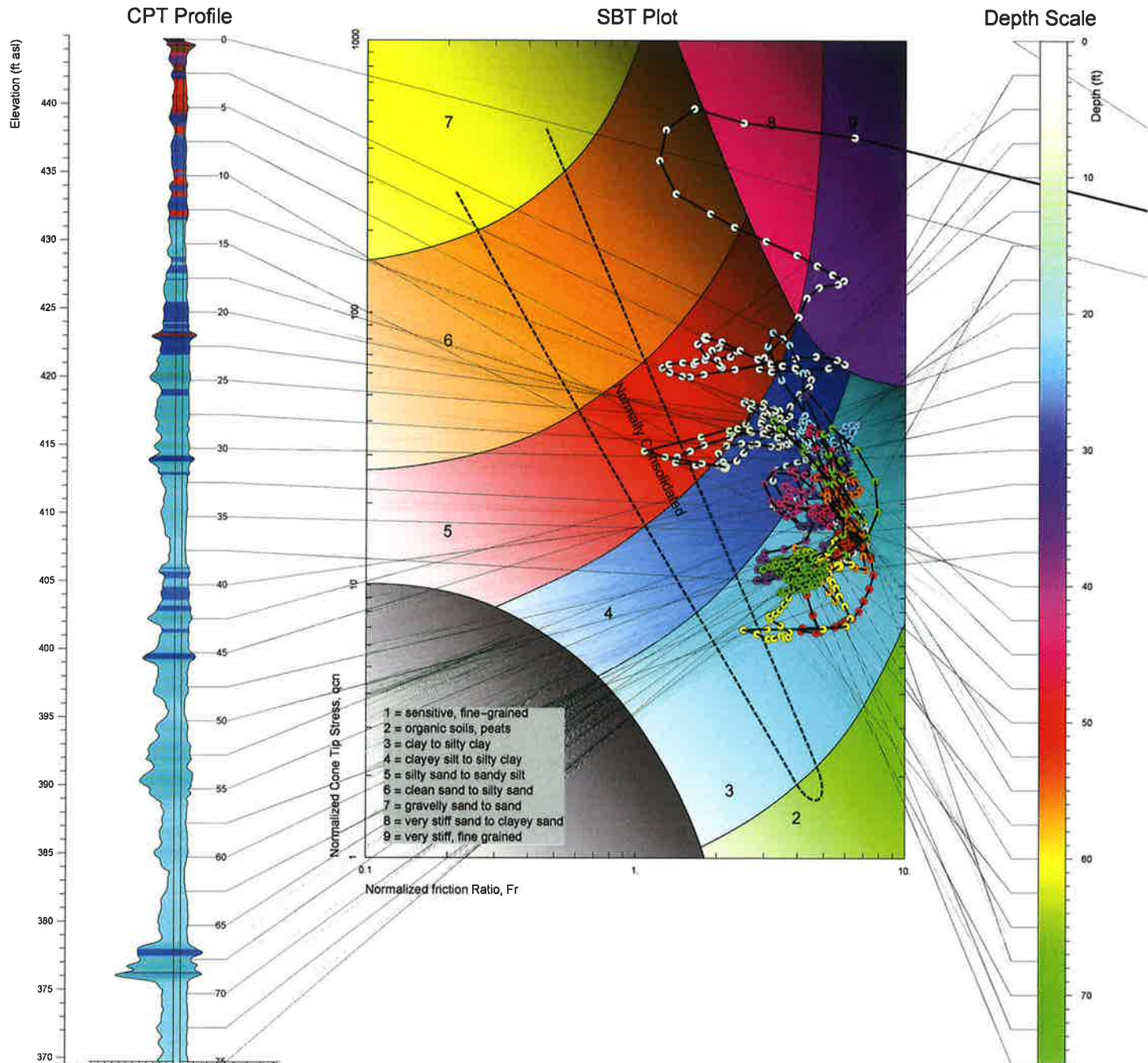
TEST ID: CPT-7



November 30, 2020
BG 23176

APPENDIX III

Graphic CPT Charts



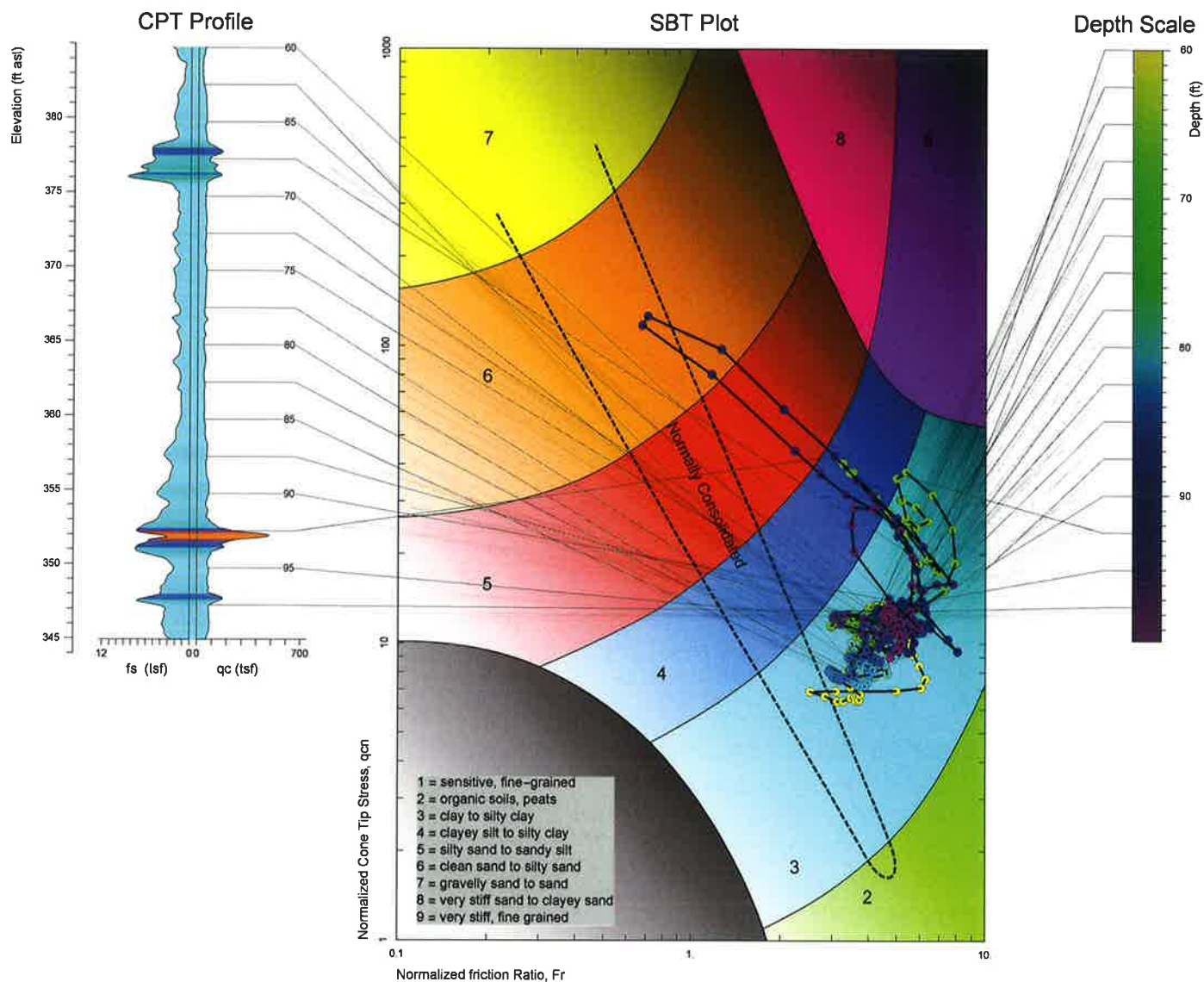
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log Fr$, $\log q_{cn}$) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-01

analyzed on: Mon 27 Jan 2020



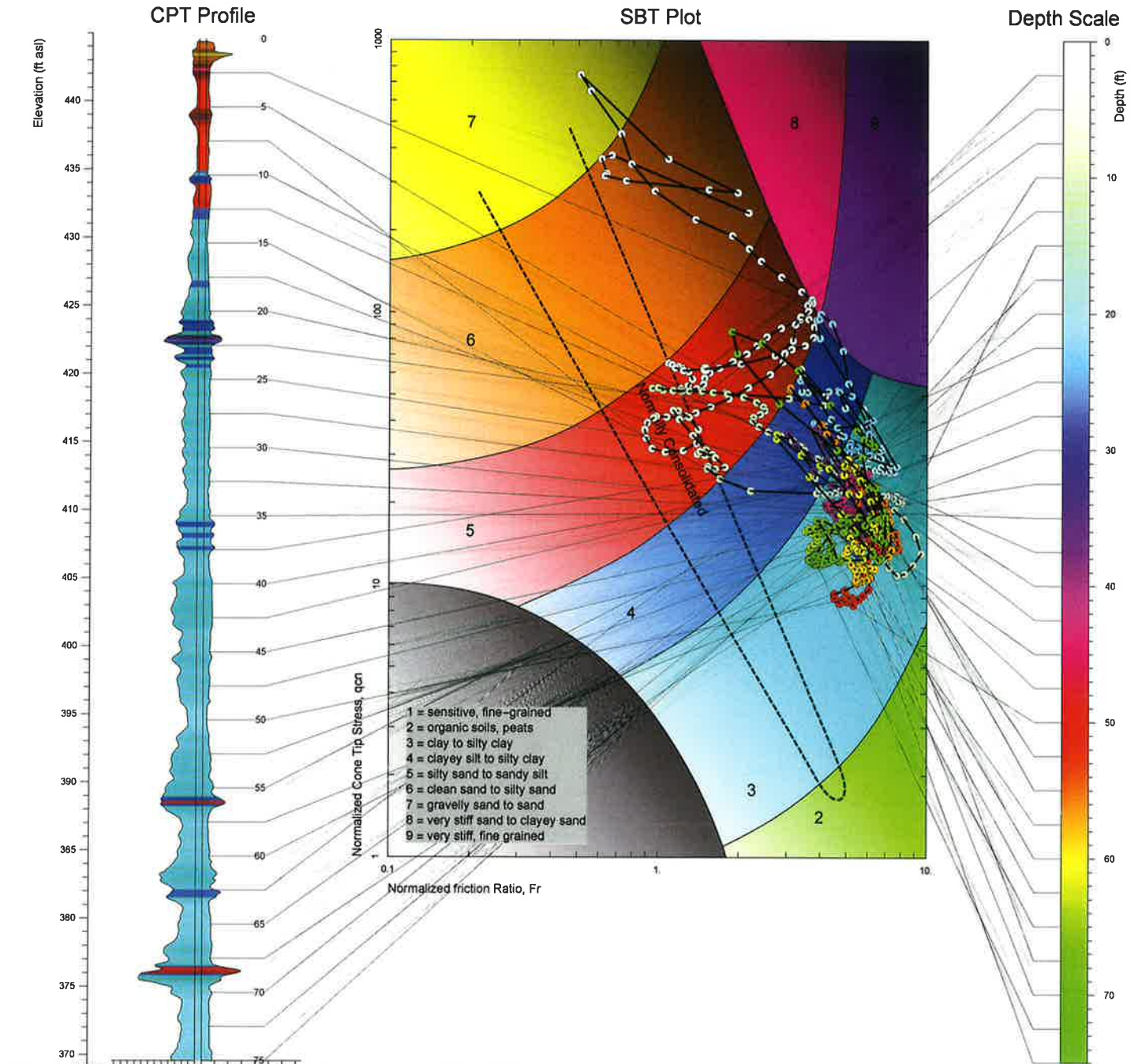
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-01

analyzed on: Mon 27 Jan 2020

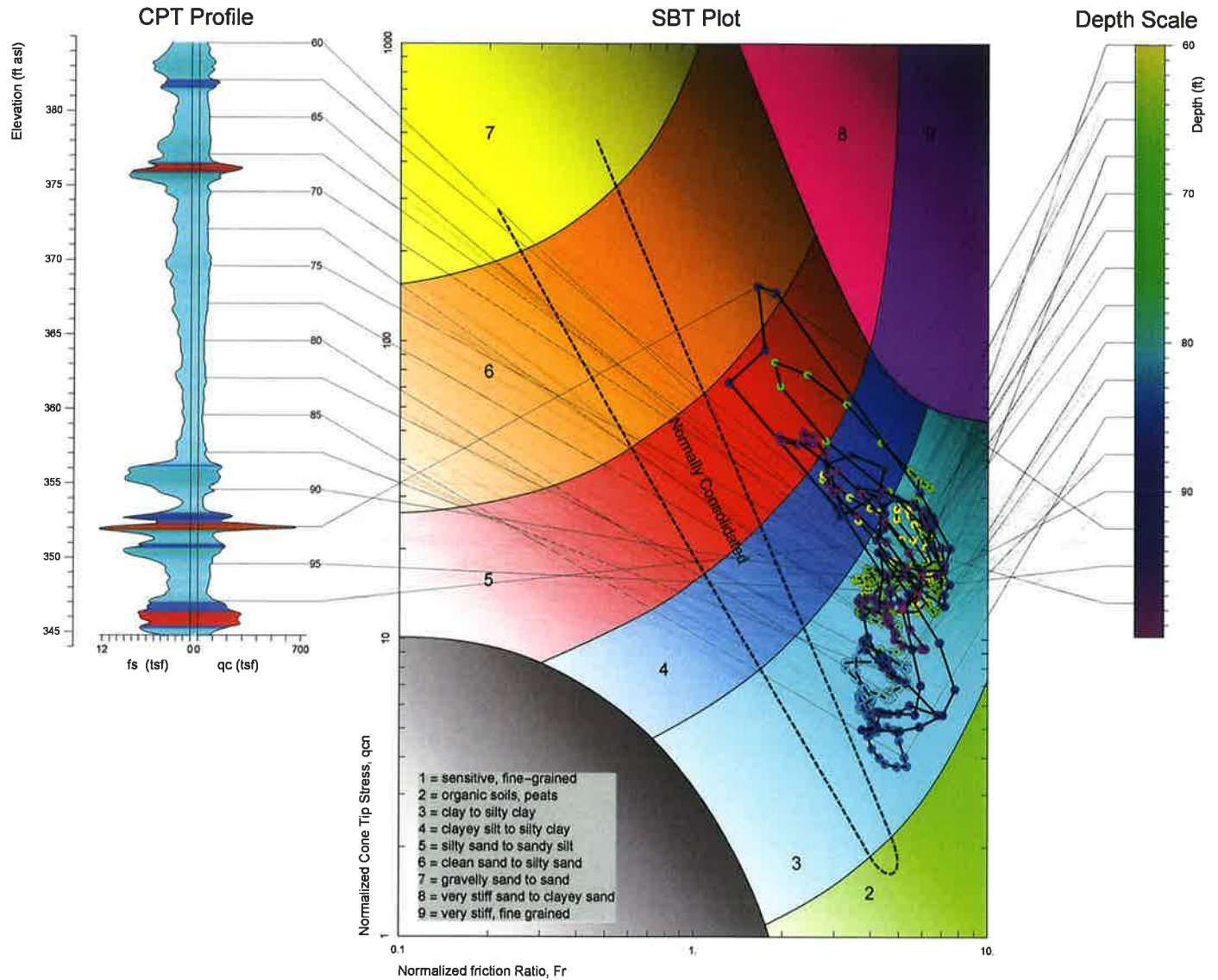


BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-02

analyzed on: Mon 27 Jan 2020



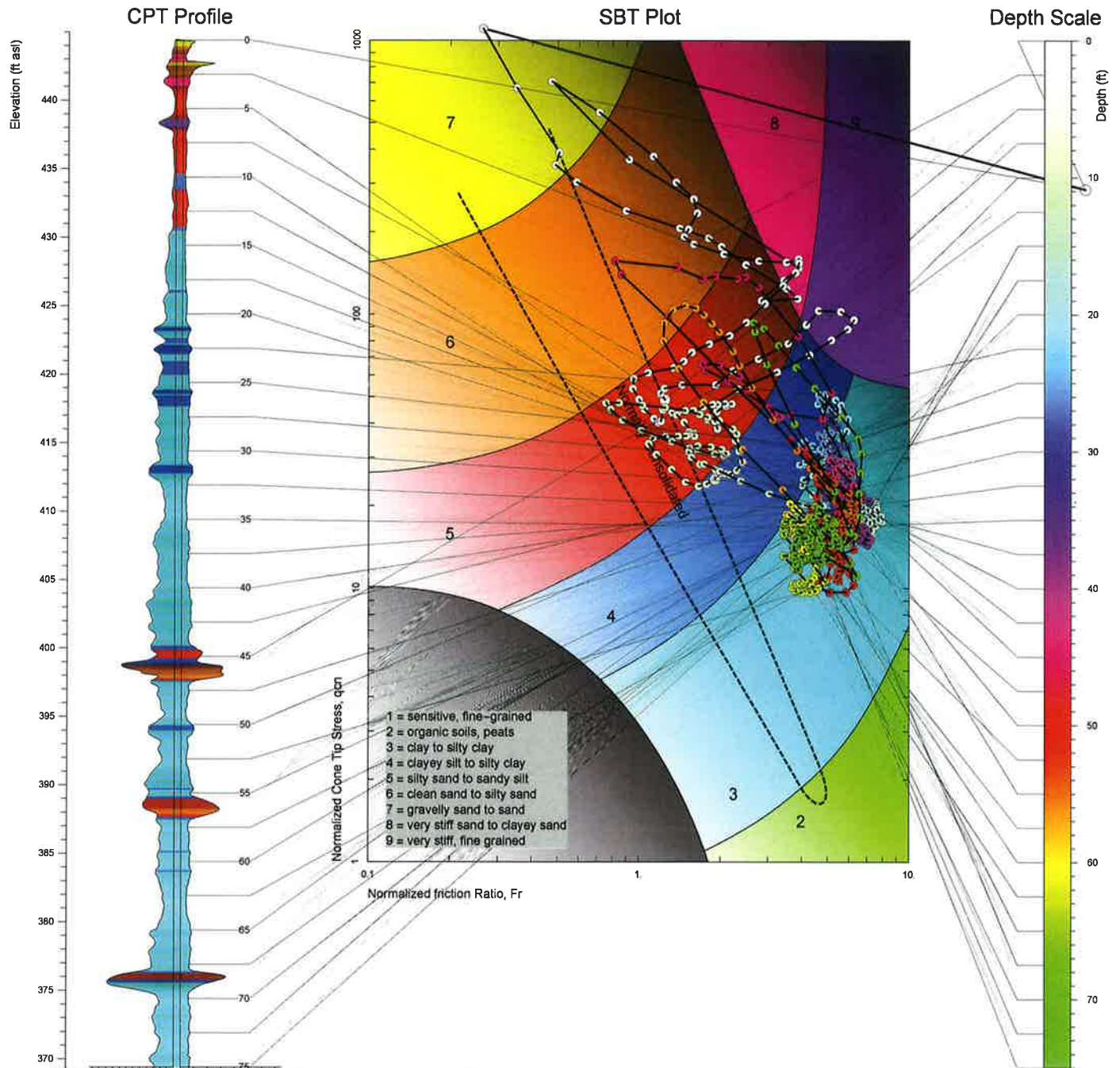
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-02

analyzed on: Mon 27 Jan 2020



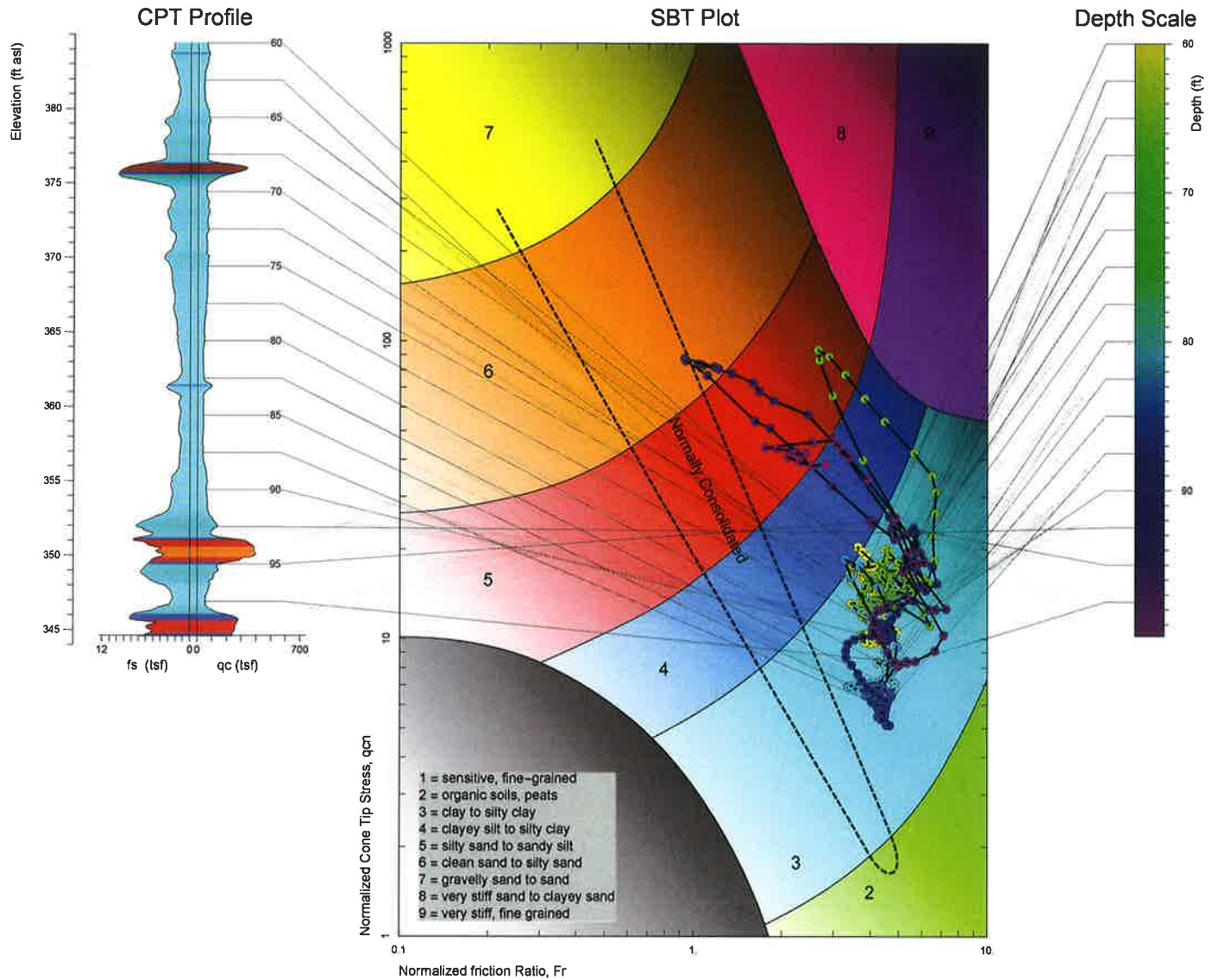
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve friction (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log F_r$, $\log q_{cn}$) are plotted on the SBT Plot, where F_r is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing F_r and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-03

analyzed on: Mon 27 Jan 2020



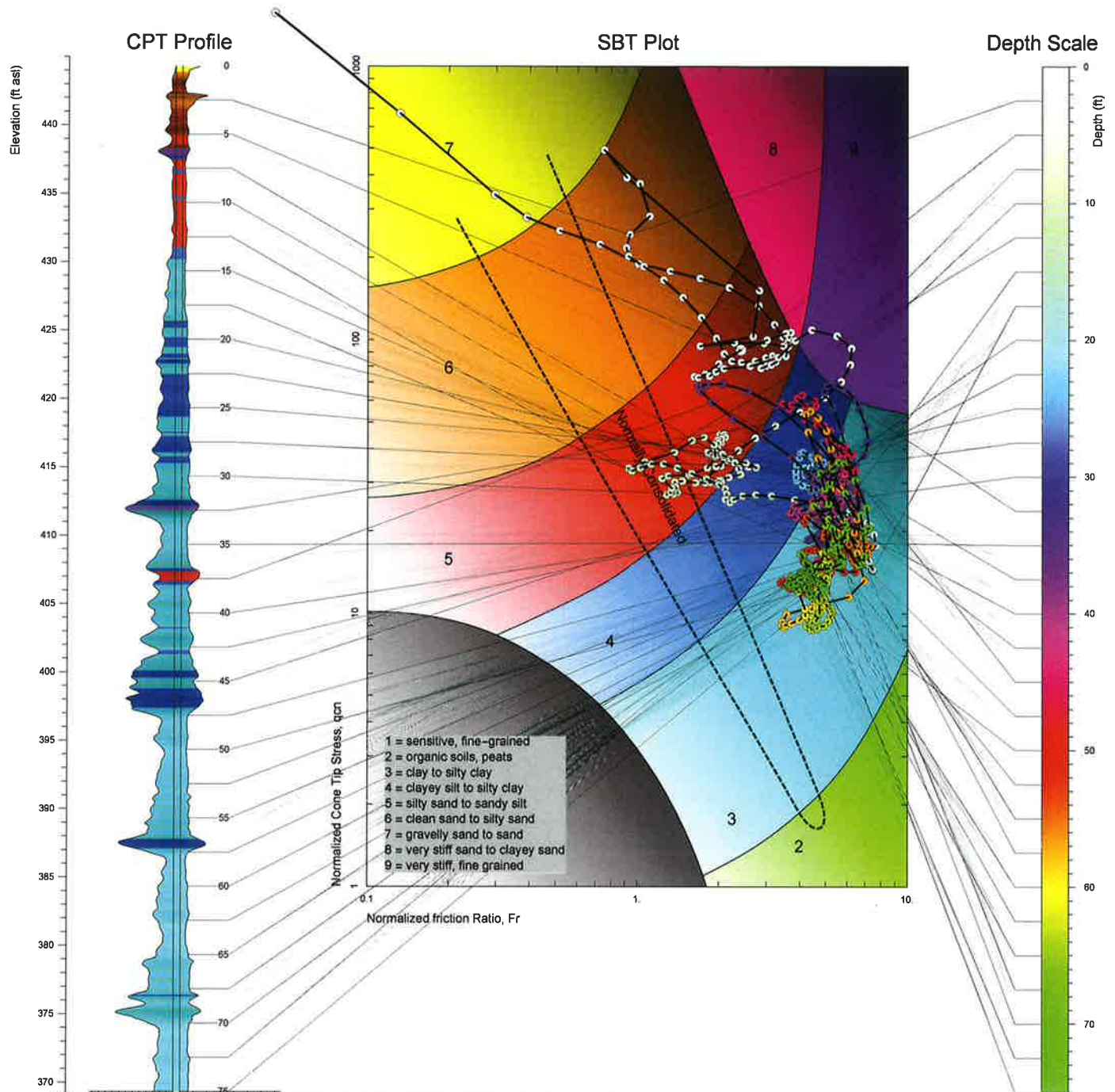
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-03

analyzed on: Mon 27 Jan 2020



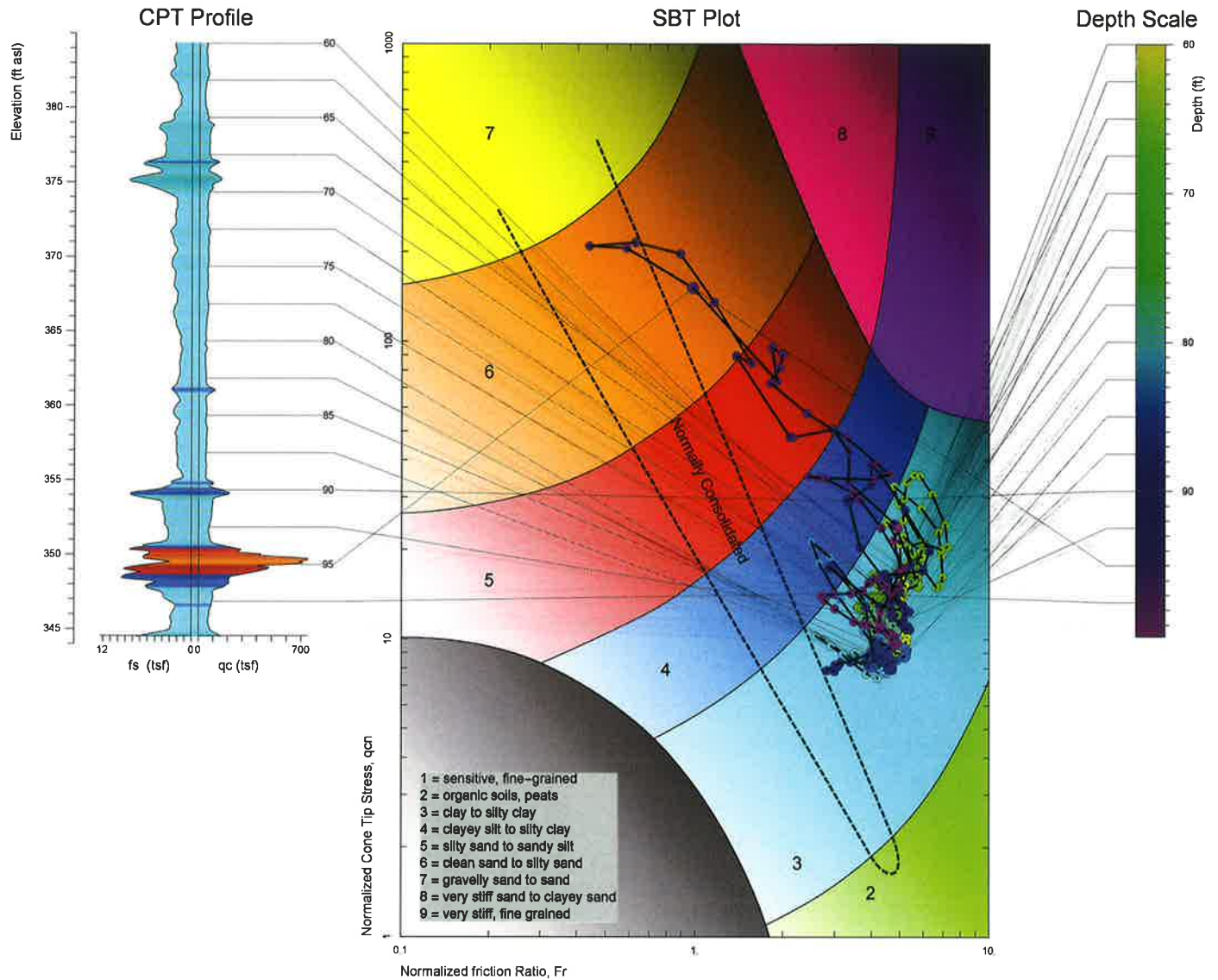
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log Fr$, $\log q_{cn}$) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-04

analyzed on: Mon 27 Jan 2020



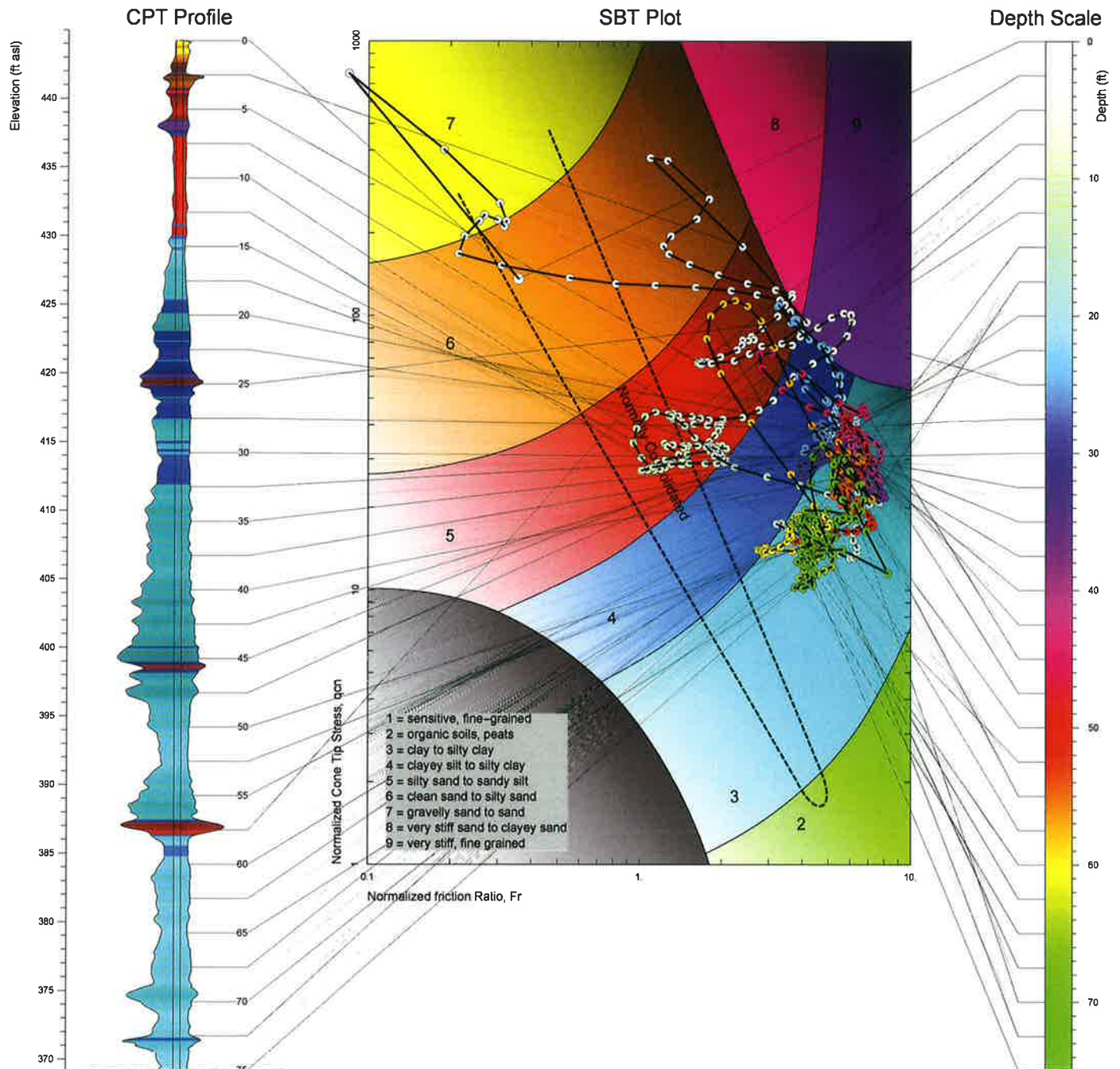
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-04

analyzed on: Mon 27 Jan 2020



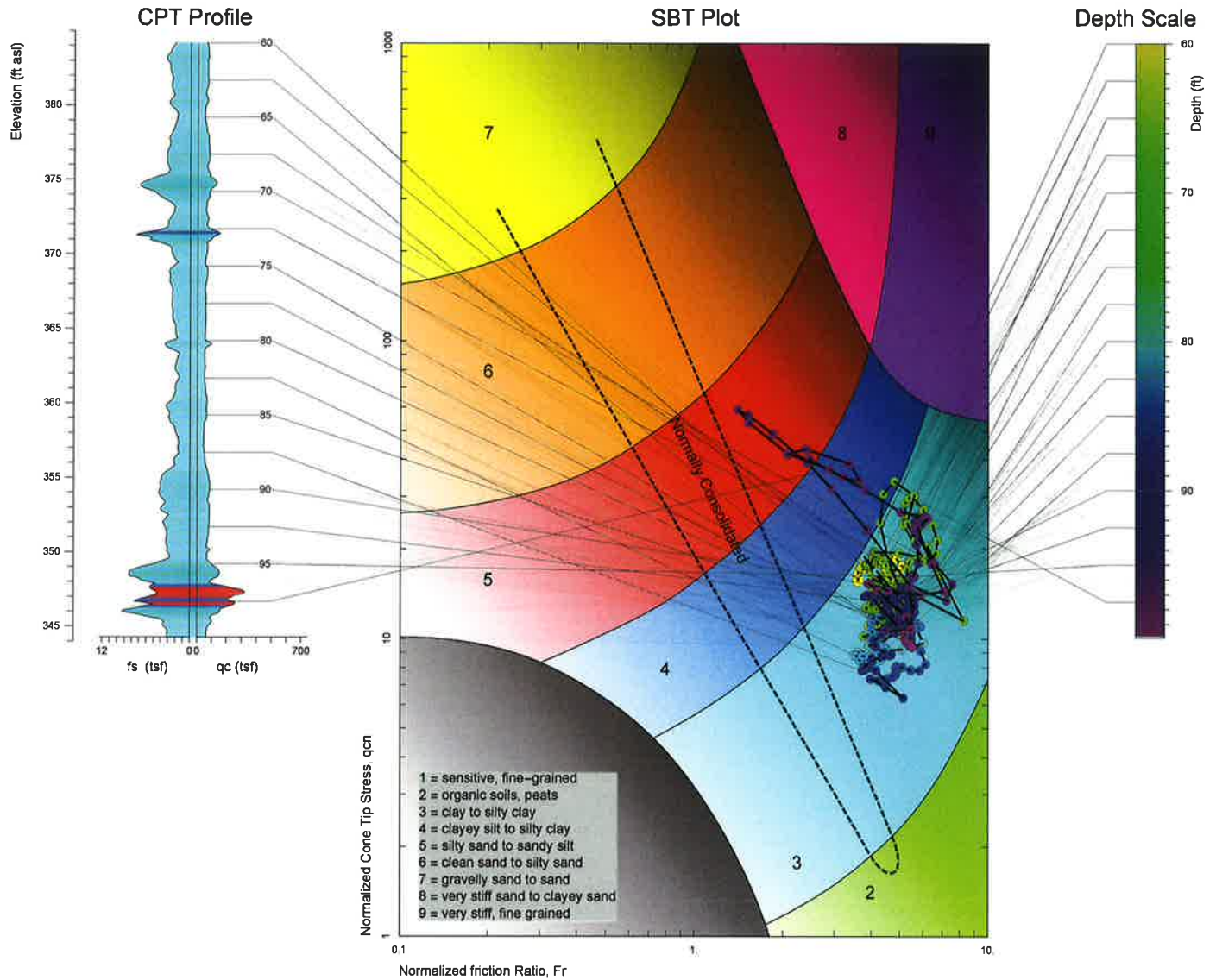
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve friction (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log Fr$, $\log q_{cn}$) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-05

analyzed on: Mon 27 Jan 2020



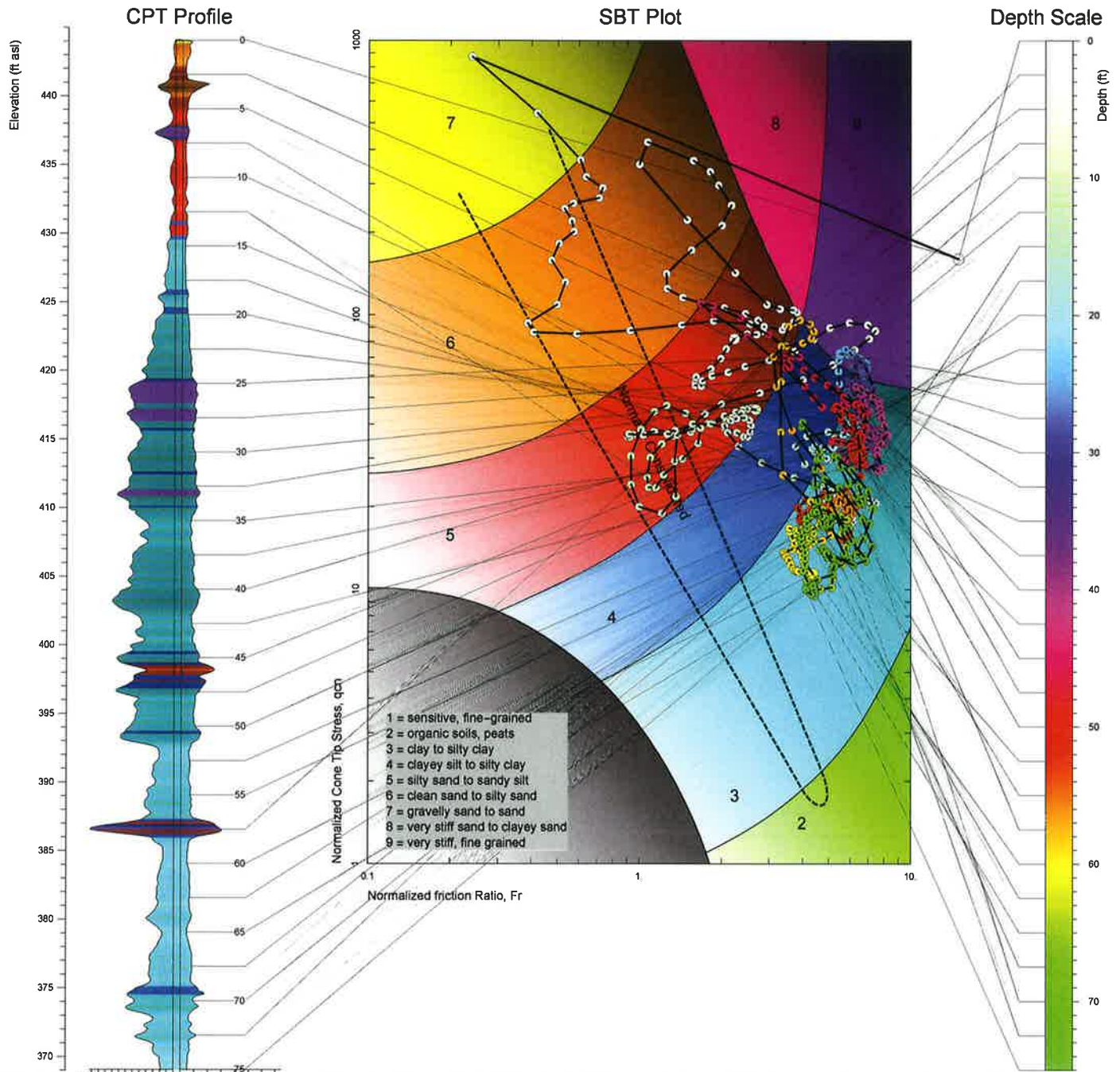
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-05

analyzed on: Mon 27 Jan 2020



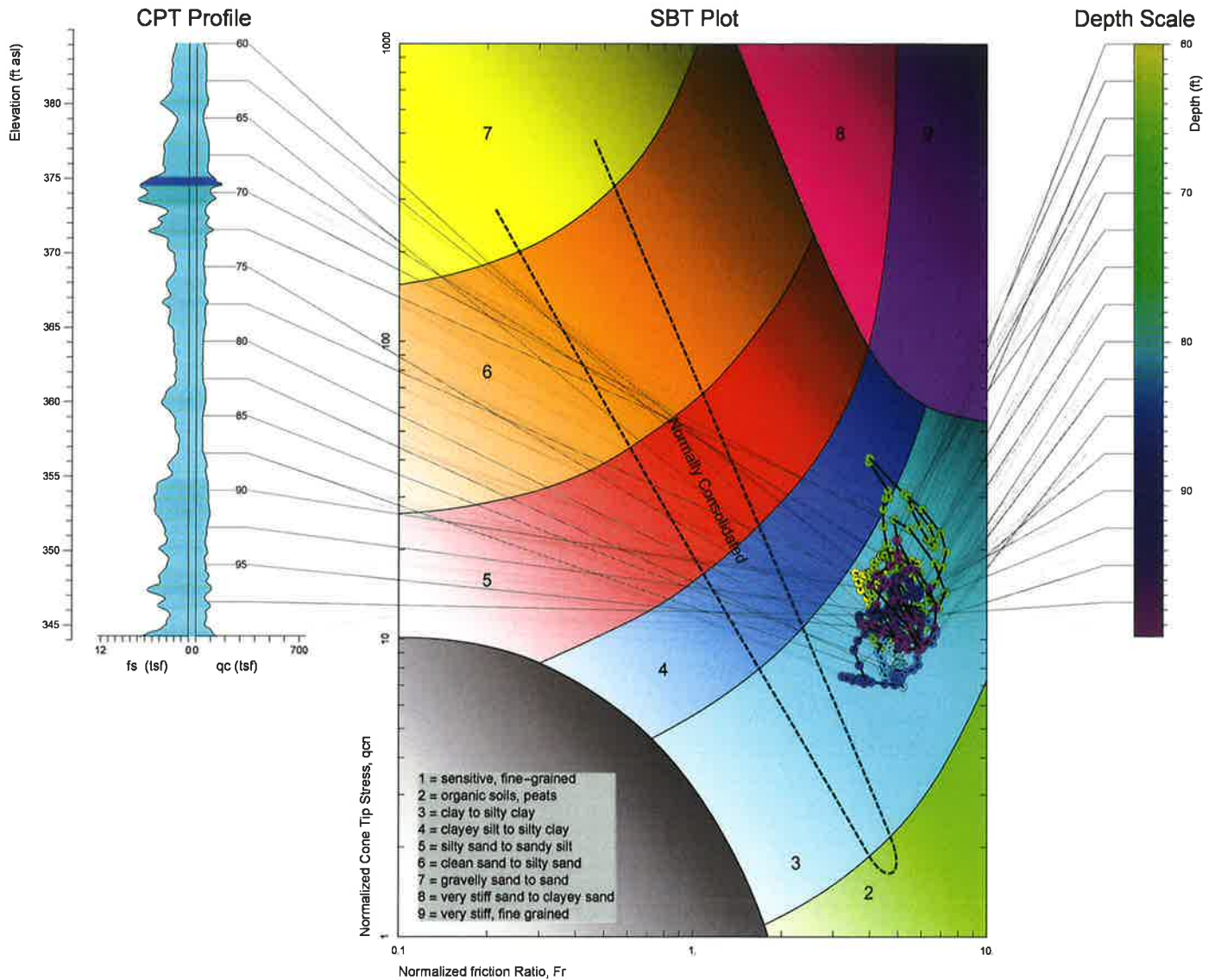
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve friction (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log Fr$, $\log q_{cn}$) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-06

analyzed on: Mon 27 Jan 2020



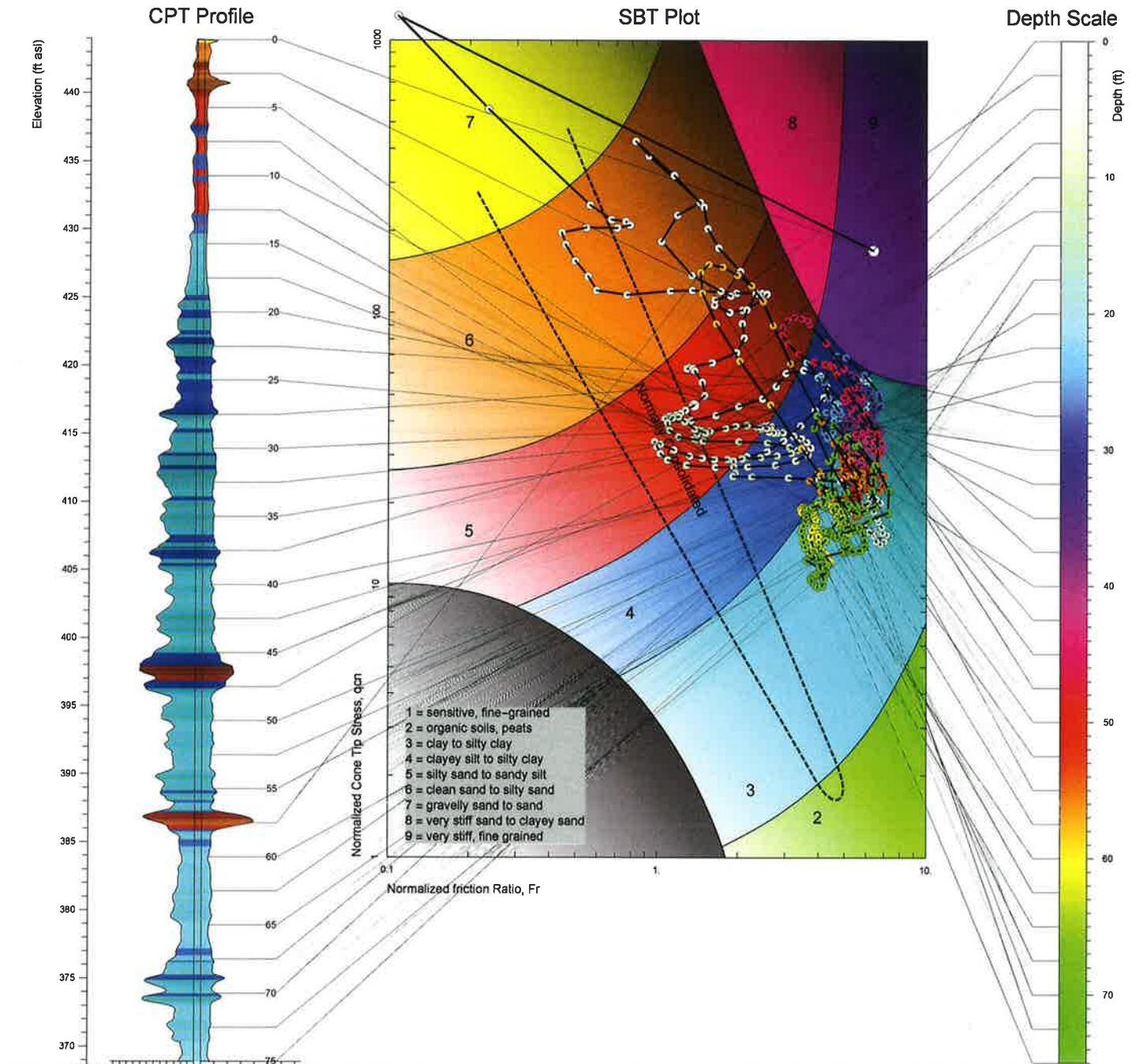
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-06

analyzed on: Mon 27 Jan 2020



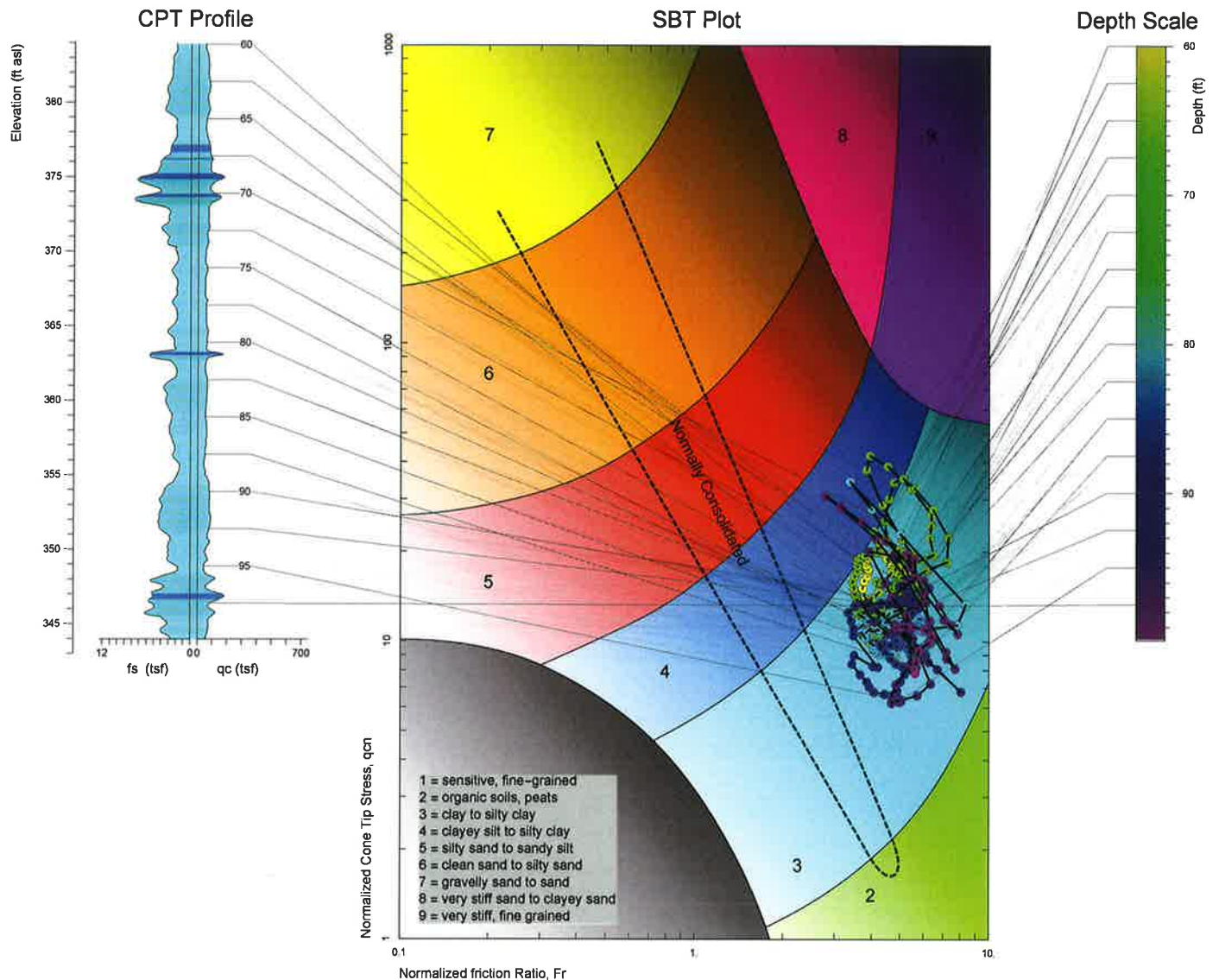
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve friction (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log F_r$, $\log q_{cn}$) are plotted on the SBT Plot, where F_r is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing F_r and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-07

analyzed on: Mon 27 Jan 2020



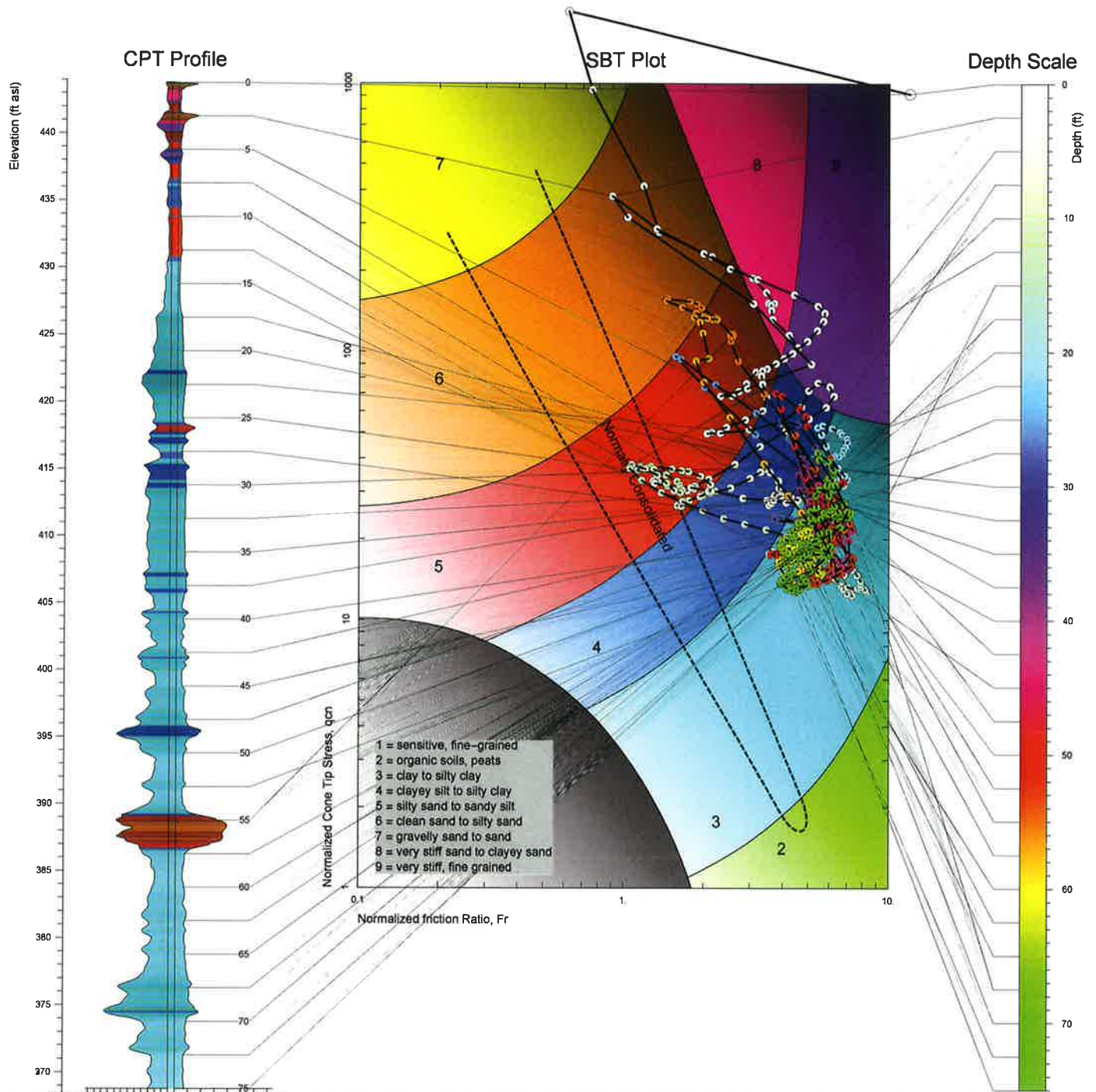
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-07

analyzed on: Mon 27 Jan 2020



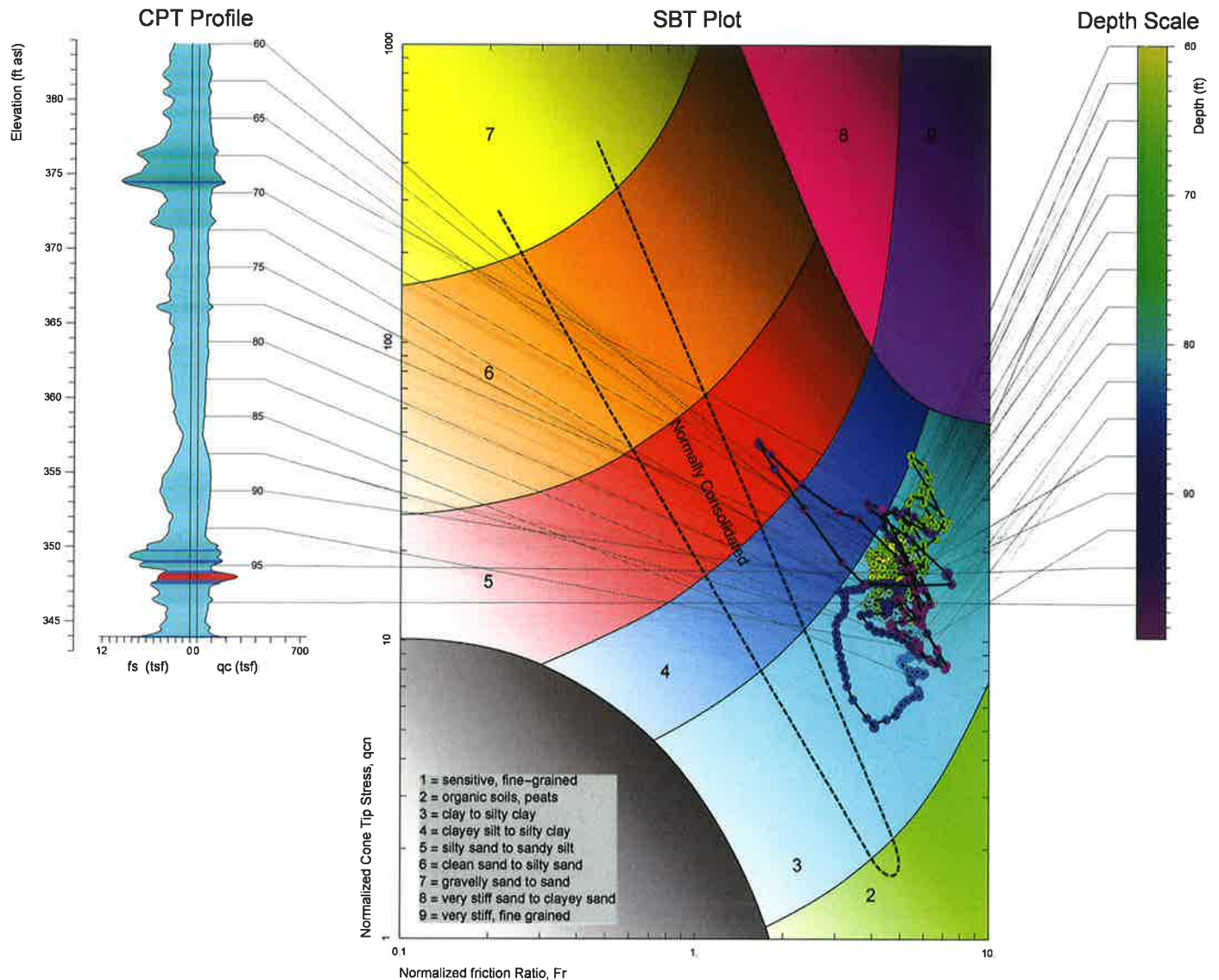
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve friction (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log Fr$, $\log q_{cn}$) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-08

analyzed on: Mon 27 Jan 2020



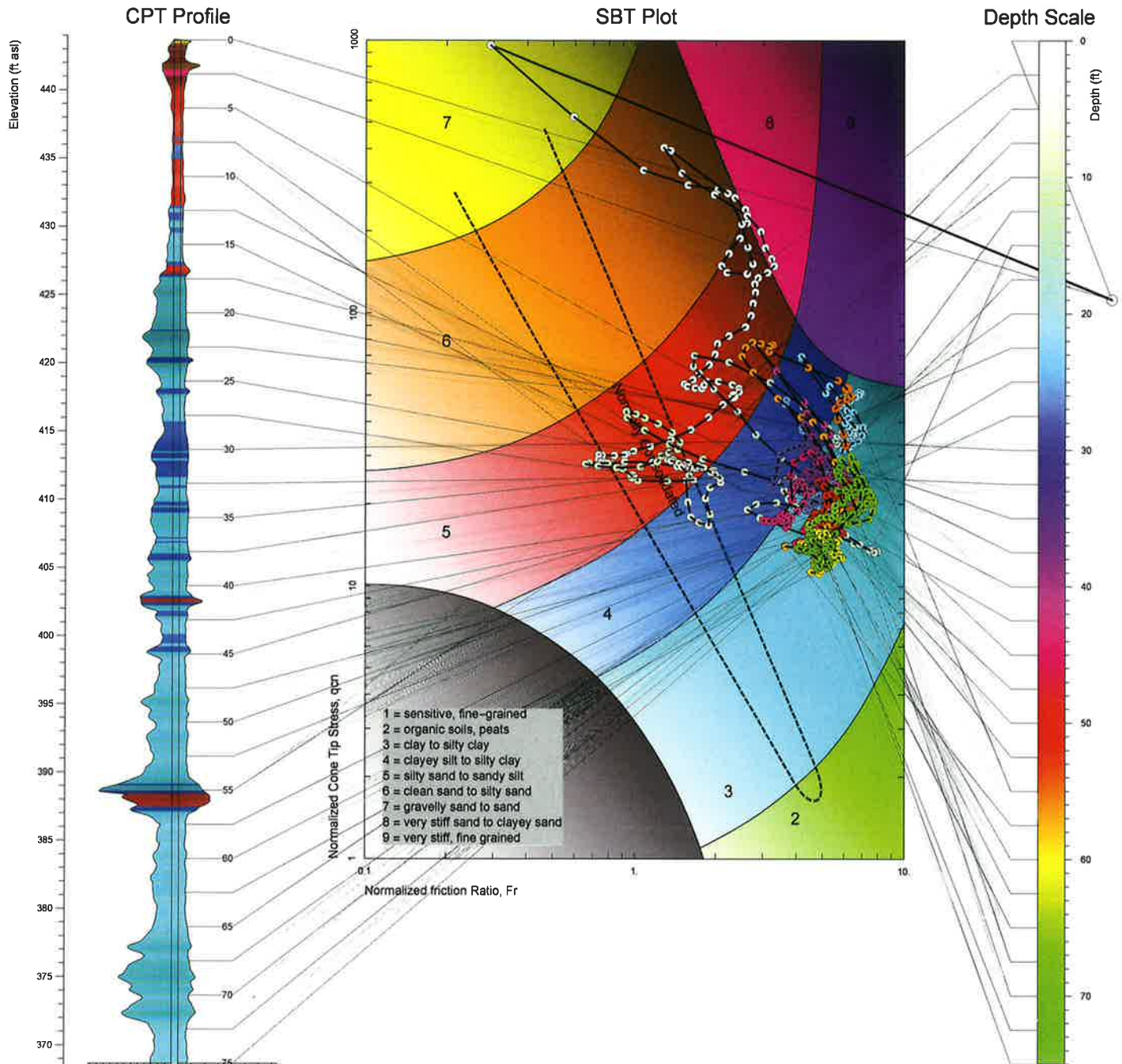
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-08

analyzed on: Mon 27 Jan 2020



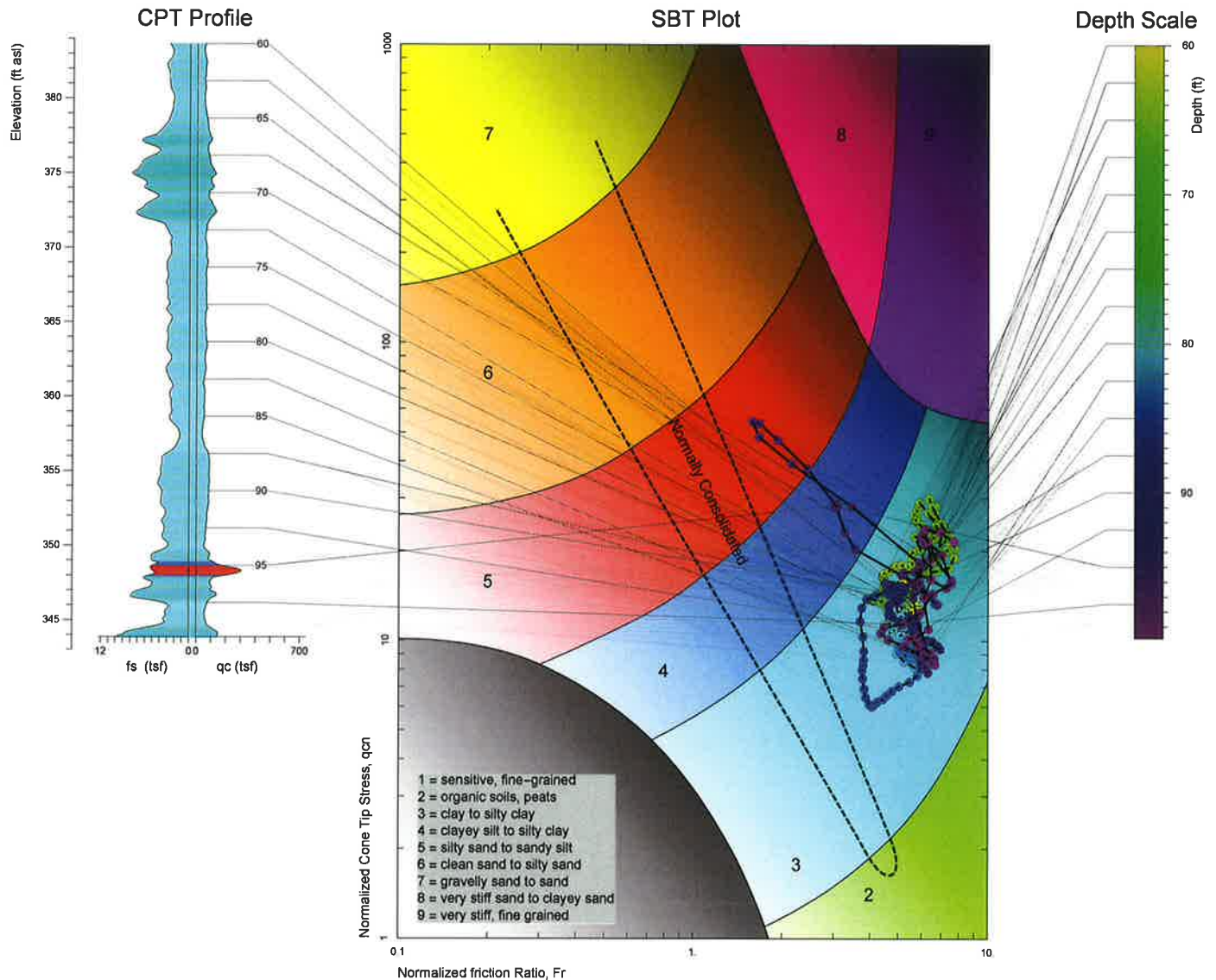
Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (f_s) to the left and a curve for the non-normalized cone tip stress (q_c) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates ($\log Fr$, $\log q_{cn}$) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and q_{cn} is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the f_s and q_c curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and q_{cn} , corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The f_s - and q_c -axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-09

analyzed on: Mon 27 Jan 2020



Notes: The CPT interpretation consists of three parts: a CPT profile (left), an SBT Plot (Soil Behavior Type, center), and a Depth Scale (right). The CPT Profile consists of 'raw' data, with a curve for the non-normalized sleeve stress (fs) to the left and a curve for the non-normalized cone tip stress (qc) to the right, plotted as a function of depth. The data points have an 0.1-foot spacing, and were re-sampled from the original data (approx. 0.16 foot spacing). For each data point, the normalized Soil Behaviour Type coordinates (Log Fr, Log qcn) are plotted on the SBT Plot, where Fr is the normalized friction ratio, and qcn is the normalized cone tip stress, calculated after Robertson (2009). In the CPT profile, each 0.1-foot layer between the fs and qc curves is color-coded based on the SBT type (numbered 1 to 9), determined by the field in which the data point plots, with field boundaries after Robertson (1990). For each SBT type, the color in the SBT plot becomes darker with increasing Fr and qcn, corresponding to increasing age, overconsolidation ratio (OCR) and/or cementation. The fs- and qc-axes in the CPT profile are separated to show the color-coding for weak soil materials. Data points in the SBT Plot are color-coded for depth, corresponding to the Depth Scale. Colors in the Depth Scale cycle through the spectrum, repeating every 60 feet, with colors becoming darker with depth. Depth colors and SBT colors are unrelated. For each data point, the corresponding locations in the CPT Profile, the SBT Plot and on the Depth Scale, are connected by a tie line, with thicker tie lines shown at 5-foot intervals.

BG-23176 I&L Investment

analyzed by HH

CPT No: CPT-09

analyzed on: Mon 27 Jan 2020

November 30, 2020
BG 23176

APPENDIX IV

Calculations

SEISMIC SOURCES
EZ-FRISK V7.65



DETERMINISTIC CALCULATION
OF PEAK GROUND ACCELERATION BASED ON DIGITIZED FAULT DATA

BG: 23176 ANALYSIS DATE: 5/18/2020
CLIENT: I&L Investment and Management, Inc. ENGINEER: RSB
PROJECT DESCRIPTION: Proposed 5-Story Building over One Subterranean Level

SITE COORDINATES: LATITUDE: 34.1051
LONGITUDE: -118.3112

SEARCH RADIUS: 100 km

ATTENUATION RELATIONS: CHIOU-YOUNGS (2007) NGA USGS 2008 MRC
BOORE-ATKINSON (2008) NGA USGS 2008 MRC
CAMPBELL-BOZORGNIA (2008) NGA USGS 2008 MRC

SEISMIC SOURCE SUMMARY
DETERMINISTIC SITE PARAMETERS

FAULT NAME	APPROXIMATE DISTANCE		MAXIMUM EARTHQUAKE MAGNITUDE	PEAK GROUND ACCELERATION
	(km)	(mi)	(Mw)	(g)
Santa Monica	0.4	0.3	7.4	0.766
Hollywood	0.7	0.4	6.7	0.647
Elysian Park (Upper)	3.4	2.1	6.7	0.638
Puente Hills (LA)	6.1	3.8	7.0	0.628
Puente Hills	7.3	4.5	7.1	0.556
Raymond	8.3	5.2	6.8	0.393
Verdugo	9.1	5.7	6.9	0.385
Newport-Inglewood	10.0	6.2	7.5	0.405
Sierra Madre	16.2	10.1	7.2	0.307
Sierra Madre Connected	16.2	10.1	7.3	0.315
Puente Hills (Santa Fe Springs)	18.6	11.6	6.7	0.327
Sierra Madre (San Fernando)	18.9	11.7	6.7	0.241
Malibu Coast	21.3	13.2	7.0	0.248
Northridge	22.5	14.0	6.9	0.310
San Gabriel	23.8	14.8	7.3	0.245
Anacapa-Dume	24.1	15.0	7.2	0.264

FAULT NAME	APPROXIMATE DISTANCE		MAXIMUM EATHQUAKE MAGNITUDE	PEAK GROUND ACCELERATION
	(km)	(mi)	(Mw)	(g)
Palos Verdes	27.1	16.9	7.3	0.226
Palos Verdes Connected	27.1	16.9	7.7	0.258
Elsinore	27.2	16.9	7.9	0.271
Clamshell-Sawpit	29.2	18.1	6.7	0.180
Santa Susana, alt 1	29.6	18.4	6.9	0.189
Puente Hills (Coyote Hills)	31.7	19.7	6.9	0.206
Holser, alt 1	39.0	24.2	6.8	0.156
San Jose	39.8	24.7	6.7	0.140
Simi-Santa Rosa	41.7	25.9	6.9	0.145
Oak Ridge Connected	46.9	29.2	7.4	0.179
Chino	47.5	29.5	6.8	0.122
Oak Ridge (Onshore)	48.6	30.2	7.2	0.167
Cucamonga	52.0	32.3	6.7	0.111
Southern San Andreas	52.6	32.7	8.2	0.209
San Cayetano	55.4	34.4	7.2	0.133
San Joaquin Hills	57.5	35.7	7.1	0.136
Imp Extensional Gridded, Char, Normal	47.0	29.2	7.0	0.121
Imp Extensional Gridded, Char, Strike Slip	47.0	29.2	7.0	0.146
Imp Extensional Gridded, GR, Normal	47.1	29.3	7.0	0.121
Imp Extensional Gridded, GR, Strike Slip	47.1	29.3	7.0	0.233
San Jacinto	71.8	44.6	7.9	0.145
Santa Ynez (East)	73.1	45.4	7.2	0.103
Santa Ynez Connected	73.4	45.6	7.4	0.113
Ventura-Pitas Point	80.8	50.2	7.0	0.092
Pitas Point Connected	80.8	50.2	7.3	0.109
Cleghorn	81.2	50.5	6.8	0.073
Oak Ridge (Offshore)	85.2	53.0	7.0	0.080
Mission Ridge-Arroyo Parida-Santa Ana	86.3	53.7	6.9	0.074
Channel Islands Thrust	88.3	54.9	7.3	0.106
Santa Cruz Island	88.5	55.0	7.2	0.083
Red Mountain	94.9	59.0	7.4	0.089
Garlock	95.1	59.1	7.7	0.105
Pleito	96.9	60.2	7.1	0.073
North Frontal (West)	98.7	61.3	7.2	0.076

50 Faults found within a 100 km Search Radius.

Closest Fault to the Site: Santa Monica

Distance = 0.42 km (0.26mi)

Largest Peak Ground Acceleration: 0.766 g

The San Andreas Fault is Located Aproximately 52.6 km (32.7 mi) from the Site.



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SEISMIC HAZARD DEAGGREGATION CHART (Probability of Exceedance: 10% in 50 years)

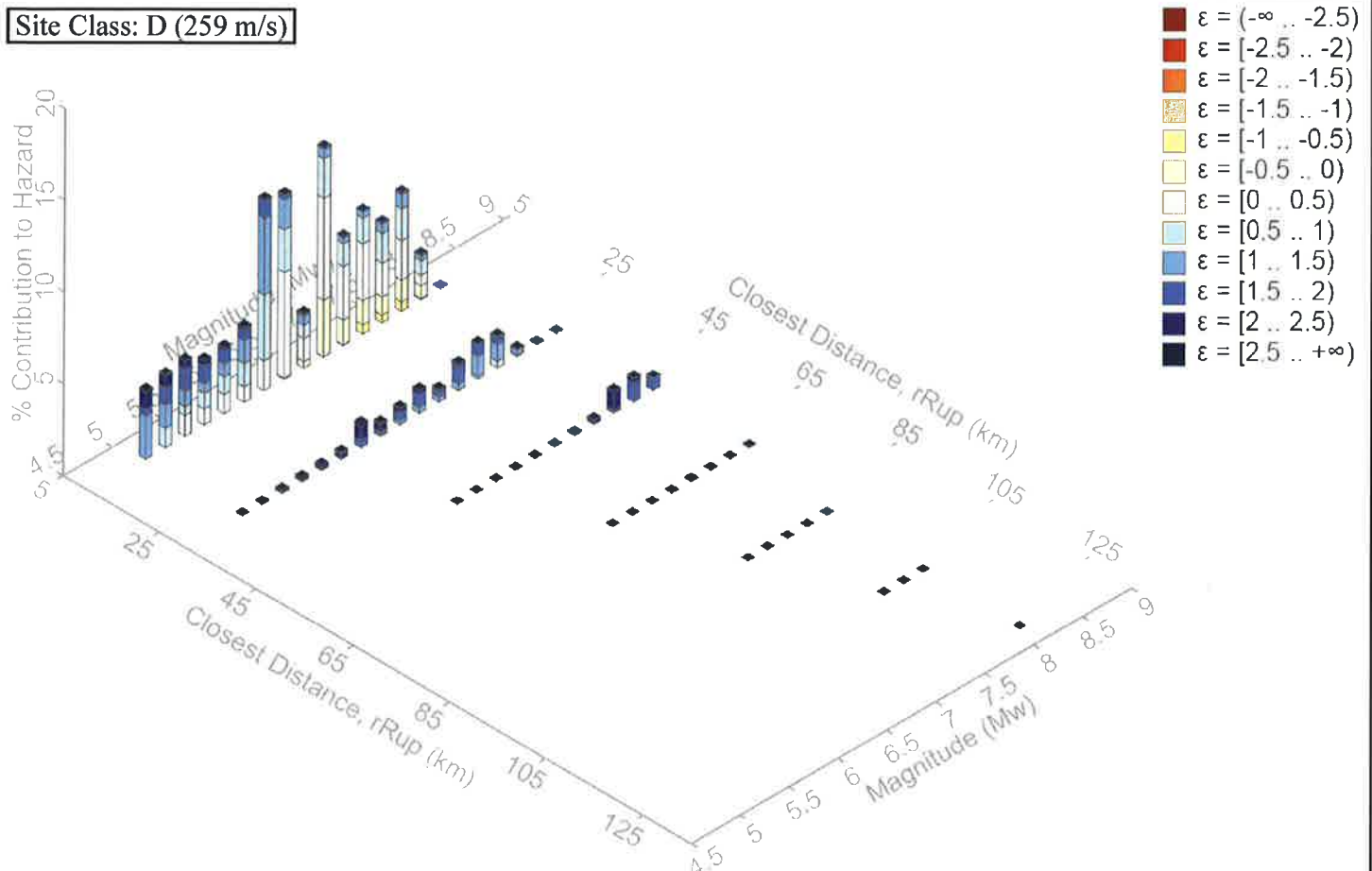
BG: 23176

ENGINEER: RSB

CLIENT: I&L INVESTMENT AND
MANAGEMENT, INC.

REFERENCE: USGS, 2020, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Continuous U.S. 2014 (update) (v4.2.0) Edition, <https://earthquake.usgs.gov/hazards/interactive/index.php>.

Site Class: D (259 m/s)



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr⁻¹
PGA ground motion: 0.53049793 g

Recovered targets

Return period: 504.07469 yrs
Exceedance rate: 0.001983833 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.11 %

Mode (largest m-r bin)

m: 6.9
r: 5.27 km
ε: 0.21 σ
Contribution: 11.38 %

Mode (largest m-r-ε bin)

m: 6.53
r: 4.44 km
ε: 0.29 σ
Contribution: 5.76 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Site-Specific Ground Motion Analysis (Based on ASCE 7-16 Standard)



BG: 23176	Client: I&L Investment and Management, Inc.	Analysis Date: 5/18/20
Project Description: Proposed 5-Story Building over Subterranean Level	Engineer: RSB	

Ss (0.2s) =	2.114	Latitude:	34.1051	Periods (seconds):		80% of Sections. 11.4.3 & 11.4.4 of ASCE 7-16	RESULTS Design Values ASCE 7-16 (Section 21.4)	
S1 (1s) =	0.759	Longitude:	-118.3112	T _o =	0.180			
Fa =	1.00	Site Class:	D	T _s =	0.898			
Fv =	2.50			T _L =	8			
SMs =	2.114		Fig. 22-18A	S _{MS} =	1.586	<	1.691	1.691
SM1 =	1.898	C _{RS} :	0.894	S _{M1} =	1.586	>	1.518	1.586
SDs =	1.409		Fig. 22-19A	S _{DS} =	1.057	<	1.127	1.127
SD1 =	1.265	C _{R1} :	0.895	S _{D1} =	1.057	>	1.012	1.057

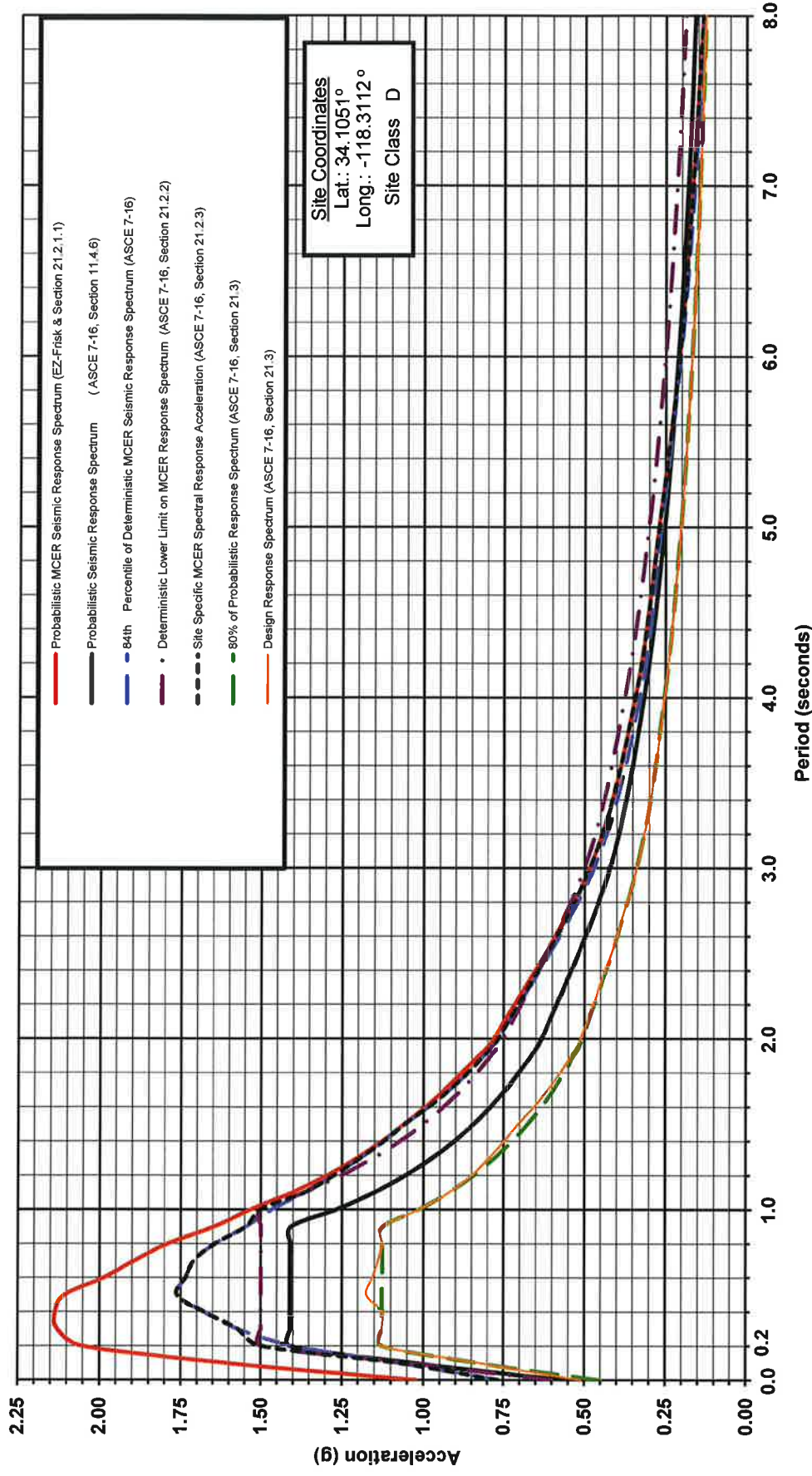
Fundamental Period	Risk Coefficient C _R (Method 1, Section 21.2.1.1, ASCE 7-16)	Probabilistic MCE _R Seismic Response Spectrum (EZ-Frisk & Section 21.2.1.1)	Probabilistic Seismic Response Spectrum (ASCE 7-16, Section 11.4.6)	84 th Percentile of Deterministic MCE _R Seismic Response Spectrum (ASCE 7-16)	Deterministic Lower Limit on MCE _R Response Spectrum (ASCE 7-16, Section 21.2.2)	Site Specific MCE _R Spectral Response Acceleration (ASCE 7-16, Section 21.2.3)	80% of Probabilistic Response Spectrum (ASCE 7-16, Section 21.3)	Design Response Spectrum (ASCE 7-16, Section 21.3)
T (sec)		Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
0.0	0.894	1.0227	0.5637	0.7663	0.600	0.766	0.451	0.511
0.1	0.894	1.6083	1.0335	1.0760	1.050	1.076	0.827	0.827
0.2	0.894	2.0535	1.4093	1.3980	1.500	1.500	1.127	1.127
0.3	0.894	2.1289	1.4093	1.5720	1.500	1.572	1.127	1.127
0.4	0.894	2.1346	1.4093	1.6780	1.500	1.678	1.127	1.127
0.5	0.894	2.1045	1.4093	1.7620	1.500	1.762	1.127	1.175
0.6	0.895	1.9858	1.4093	1.7350	1.500	1.735	1.127	1.157
0.7	0.895	1.8921	1.4093	1.7100	1.500	1.710	1.127	1.140
0.8	0.895	1.7877	1.4093	1.6430	1.500	1.643	1.127	1.127
0.9	0.895	1.6448	1.4056	1.5460	1.500	1.546	1.124	1.124
1.0	0.895	1.5305	1.2650	1.4650	1.500	1.500	1.012	1.012
1.1	0.895	1.4007	1.1500	1.3630	1.364	1.364	0.920	0.920
1.2	0.895	1.2933	1.0542	1.2740	1.250	1.274	0.843	0.849
1.3	0.895	1.2038	0.9731	1.1940	1.154	1.194	0.778	0.796
1.4	0.895	1.1268	0.9036	1.1220	1.071	1.122	0.723	0.748
1.5	0.895	1.0588	0.8433	1.0570	1.000	1.057	0.675	0.705
1.6	0.895	0.9926	0.7906	0.9838	0.938	0.984	0.633	0.656
1.7	0.895	0.9344	0.7441	0.9192	0.882	0.919	0.595	0.613
1.8	0.895	0.8818	0.7028	0.8621	0.833	0.862	0.562	0.575
1.9	0.895	0.8274	0.6658	0.8117	0.789	0.812	0.533	0.541
2.0	0.895	0.7804	0.6325	0.7667	0.750	0.767	0.506	0.511
3.0	0.895	0.4838	0.4217	0.4706	0.500	0.484	0.337	0.337
4.0	0.895	0.3425	0.3163	0.3282	0.375	0.343	0.253	0.253
5.0	0.895	0.2692	0.2530	0.2641	0.300	0.269	0.202	0.202
6.0	0.895	0.2067	0.2108	0.2029	0.250	0.207	0.169	0.169
7.0	0.895	0.1667	0.1807	0.1608	0.214	0.167	0.145	0.145
8.0	0.895	0.1345	0.1581	0.1304	0.188	0.135	0.127	0.127

* The Probabilistic and Deterministic Seismic Response Spectra are Based on the Maximum Rotated Component (MRC) of Ground Motion.

References:

- American Society of Civil Engineers (ASCE), 2016, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Standard ASCE/SEI 7-16, Chapter 21.
- Division of the State Architect (DSA), 2009, *Use of the Next Generation Attenuation (NGA) Relations*, State of California, Department of General Services, DSA Bulletin 09-01, Effective March 1, 2009.

SEISMIC RESPONSE SPECTRA



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SITE-SPECIFIC SEISMIC RESPONSE SPECTRA

Proposed 5-Story Building over Subterranean Level

BG: 23176

Client: I&L Investment and Management, Inc.

Engineer: RSB

Analysis Date: May 18, 2020



BYER GEOTECHNICAL INC.

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RETAINING WALL CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #1a
Cantilevered Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #1
Cohesion, Coh 250.0 psf
Phi Angle, ϕ 26.0 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device RETAINING WALL
Type CANTILEVERED
Retained Height, H 10 feet
Wall Friction Angle, δ 0 degrees
External Surcharge see below
General Backslope Condition* level
Loading STATIC

Calculation Safety Factor, FS 1.5

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid 773 trials
Trial Wedges Analyzed, Secondary Search Window 324 trials
Critical Failure Angle, α 50.8 degrees
Area of Critical Wedge 40.1 square feet
Length of Critical Failure Plane, L 11.2 feet
Depth of Critical Tension Crack 1.3 feet
Horizontal Upslope Distance to Critical Tension Crack 7.1 feet
Effective Backslope on Critical Wedge, β_{eff} 0.0 degrees
Factored Phi Angle on Slip Plane, ϕ' 18.0 degrees
Factored Cohesion on Critical Slip Plane, C' 166.7 psf
Weight of Critical Wedge, W 5,014 pounds
External Surcharge on Critical Wedge, V 1,380 pounds
Static Gravitational Driving Force, W' 6,394 pounds
Mobilized Cohesive Force, $C'L$ 1,872 pounds
Mobilized Frictional Force, R 5,880 pounds
Calculated Unbalanced Force, P 1,999 pounds
Calculated Horizontal Unbalanced Force, P_h 1,999 pounds
Calculated Equivalent Fluid Pressure 40.0 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP 43.0 pcf

Design Horizontal Force 2,150 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist, elev)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	10		
(0,10)	(0,10)			
(3,10)	(3,10)			
(13,10)	(13,10)			
(15,10)	(15,10)			
(18,10)	(18,10)			
(20,10)	(20,10)			

Uniform Load: 300 psf

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED RETAINING WALL, WITH A RETAINED HEIGHT OF UP TO 10 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 43 POUNDS PER CUBIC FOOT.

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.



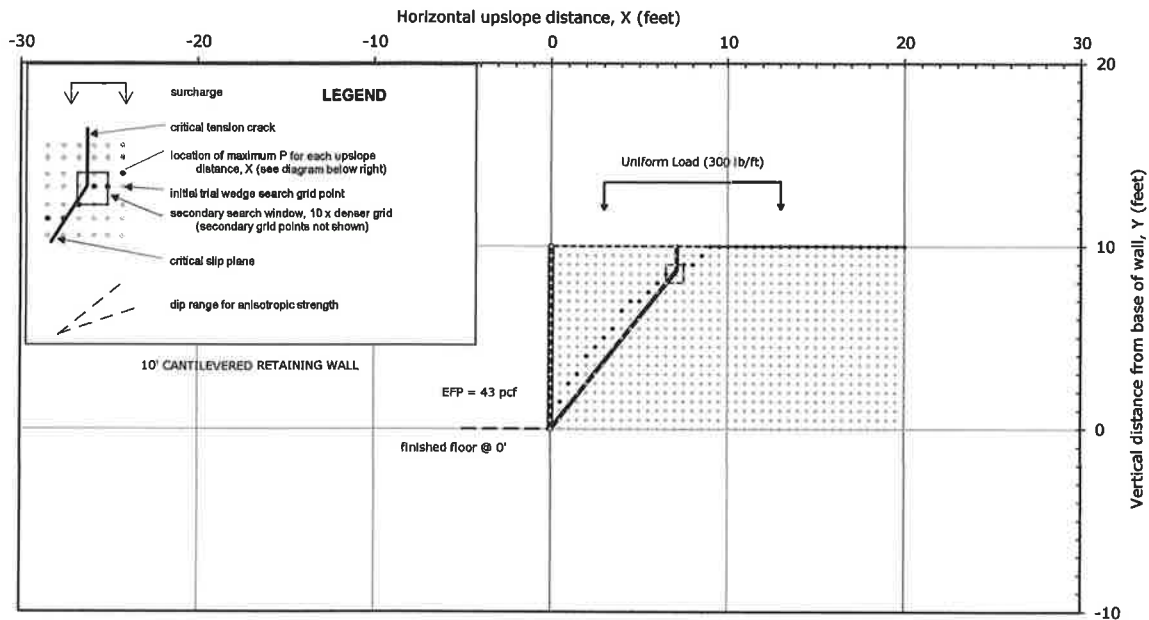
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RETAINING WALL CALCULATION

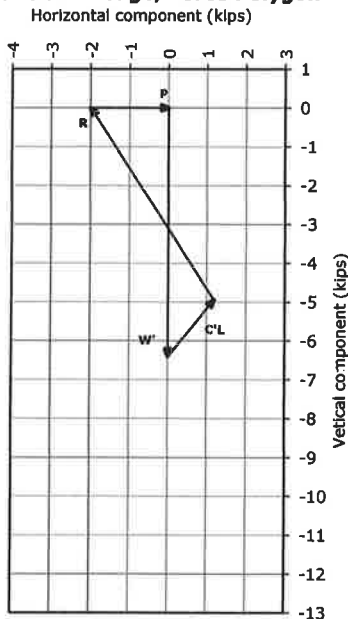
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #1b
Cantilevered Retaining Wall, basement

Cross Section and Critical Active Wedge



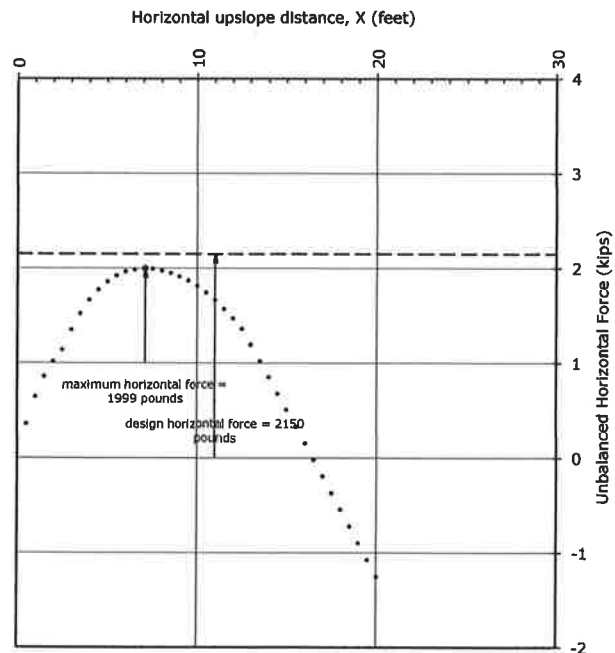
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.



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RETAINING WALL CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #1Sa
Cantilevered Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOB-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1982), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #1
Cohesion, Coh 250.0 psf
Phi Angle, ϕ 26.0 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device RETAINING WALL
Type CANTILEVERED

Retained Height, H 10 feet
Wall Friction Angle, δ 0 degrees

External Surcharge see below

General Backslope Condition* level

Loading SEISMIC
PGA_M 1.00 g

Pseudostatic Coefficients:

horizontal, K_h *** 0.33 g

vertical, K_v **** 0.00 g

Calculation Safety Factor, FS 1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** $K_v > 0$ indicates downward acceleration and upward inertial force

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist, elev)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	10		
(0,10)	(0,10)			
(3,10)	(3,10)			Uniform Load: 300 psf
(13,10)	(13,10)			
(15,10)	(15,10)			
(18,10)	(18,10)			
(20,10)	(20,10)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case	
Critical Failure Angle, α	50.8 degrees
Area of Critical Wedge	40.1 square feet
Length of Critical Failure Plane, L	11.2 feet
Depth of Critical Tension Crack	1.3 feet
Horizontal Upslope Distance to Critical Tension Crack	7.1 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, ϕ'	26.0 degrees
Factored Cohesion on Critical Slip Plane, C'	250.0 psf
Weight of Critical Wedge, W	5,014 pounds
External Surcharge on Critical Wedge, V	1,380 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, W_d	6,739 pounds
Mobilized Cohesive Force, $C'L$	2,807 pounds
Mobilized Frictional Force, R	4,647 pounds
Calculated Unbalanced Force, P	2,447 pounds
Calculated Horizontal Unbalanced Force, P_h	2,447 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	2,447 pounds
Recommended Static Horizontal Force from sheet 1a	2,150 pounds
Calculated Seismic Force ***	297 pounds

*** the seismic force should be applied at 0.6H, where H is the retained height

CONCLUSIONS

THE CALCULATED SEISMIC FORCE ON THE WALL IS THE DIFFERENCE BETWEEN THE PSEUDO-STATIC AND STATIC FORCE, AND IS 297 POUNDS. THE WALL SHOULD BE DESIGNED FOR THIS FORCE IN ADDITION TO THE RECOMMENDED DESIGN PARAMETERS ON SHEET 1A. THE SEISMIC FORCE MAY BE APPLIED AT 0.6H ABOVE THE BASE, WHERE H IS THE RETAINED HEIGHT.



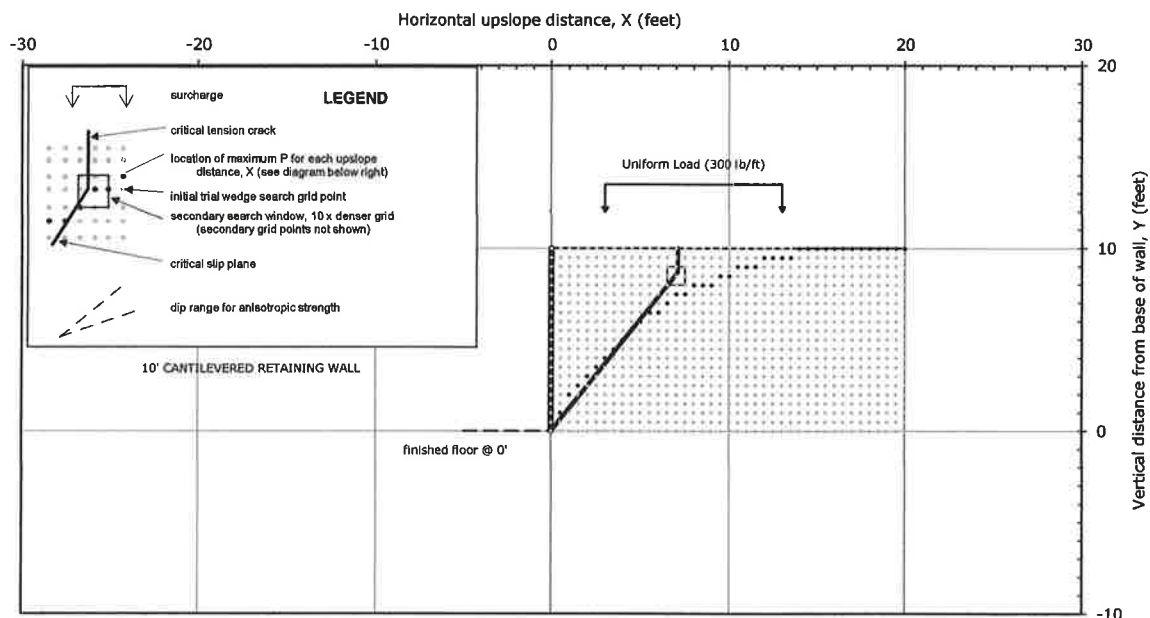
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RETAINING WALL CALCULATION

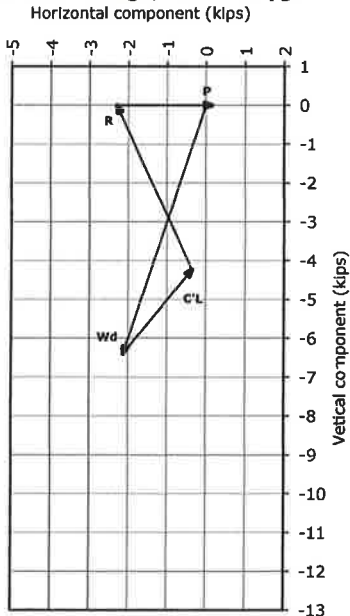
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #1Sb
Cantilevered Retaining Wall, basement

Cross Section and Critical Active Wedge



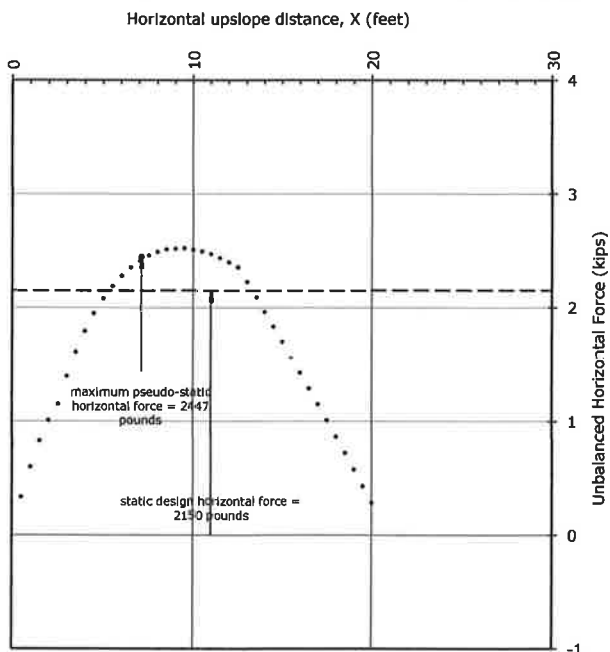
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, P_h , is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, W_d ; the mobilized cohesive force, $C'L$; the mobilized frictional force, R ; and the unbalanced pressure, P , for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, P_h (kips)



The maximum calculated horizontal unbalanced pressure, P_h , is plotted for each upslope distance, X. The location of the maximum P_h for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.



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RETAINING WALL CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #2a
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #1
Cohesion, Coh 250.0 psf
Phi Angle, ϕ 26.0 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device RETAINING WALL
Type RESTRAINED

Retained Height, H 10 feet
Wall Friction Angle, δ 0 degrees

External Surcharge NO

General Backslope Condition* level
Loading STATIC

Calculation Safety Factor, FS 1.5

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid 773 trials
Trial Wedges Analyzed, Secondary Search Window 324 trials
Critical Failure Angle, α 54.1 degrees
Area of Critical Wedge 31.4 square feet
Length of Critical Failure Plane, L 7.8 feet
Depth of Critical Tension Crack 3.7 feet
Horizontal Upslope Distance to Critical Tension Crack 4.6 feet
Effective Backslope on Critical Wedge, β_{eff} 0.0 degrees
Factored Phi Angle on Slip Plane, ϕ' 18.0 degrees
Factored Cohesion on Critical Slip Plane, C' 166.7 psf
Weight of Critical Wedge, W 3,924 pounds
External Surcharge on Critical Wedge, V 0 pounds
Static Gravitational Driving Force, W' 3,924 pounds
Mobilized Cohesive Force, $C'L$ 1,307 pounds
Mobilized Frictional Force, R 3,546 pounds
Calculated Unbalanced Force, P 1,321 pounds
Calculated Horizontal Unbalanced Force, P_h 1,321 pounds

Calculated Trapezoidal Design Pressure * 16.5 H psf
Calculated At-Rest Equivalent Fluid Pressure ** 70.2 pcf
Calculated At-Rest Trapezoidal Earth Pressure * 43.9 H psf

RECOMMENDED DESIGN PARAMETERS

Trapezoidal Design Pressure, TDP* 44 H psf
Design Horizontal Force 3,520 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist, elev)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	10		
(0,10)	(0,10)			
(3,10)	(3,10)			
(13,10)	(13,10)			
(15,10)	(15,10)			
(18,10)	(18,10)			
(20,10)	(20,10)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

* H is restrained height, see report for diagram of trapezoidal pressure distribution
** at-rest equivalent fluid pressure is calculated as: $\gamma (1 - \sin(\phi))$

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED RESTRAINED RETAINING WALL, WITH A RETAINED HEIGHT OF UP TO 10 FEET, MAY BE DESIGNED FOR A TRAPEZOIDAL DESIGN PRESSURE (TDP) OF 44 H POUNDS PER SQUARE FOOT, WHERE H IS THE RETAINED HEIGHT. SEE REPORT FOR DIAGRAM OF TRAPEZOIDAL PRESSURE DISTRIBUTION.

THE STATIC DESIGN IS GOVERNED BY THE AT-REST CONDITION.



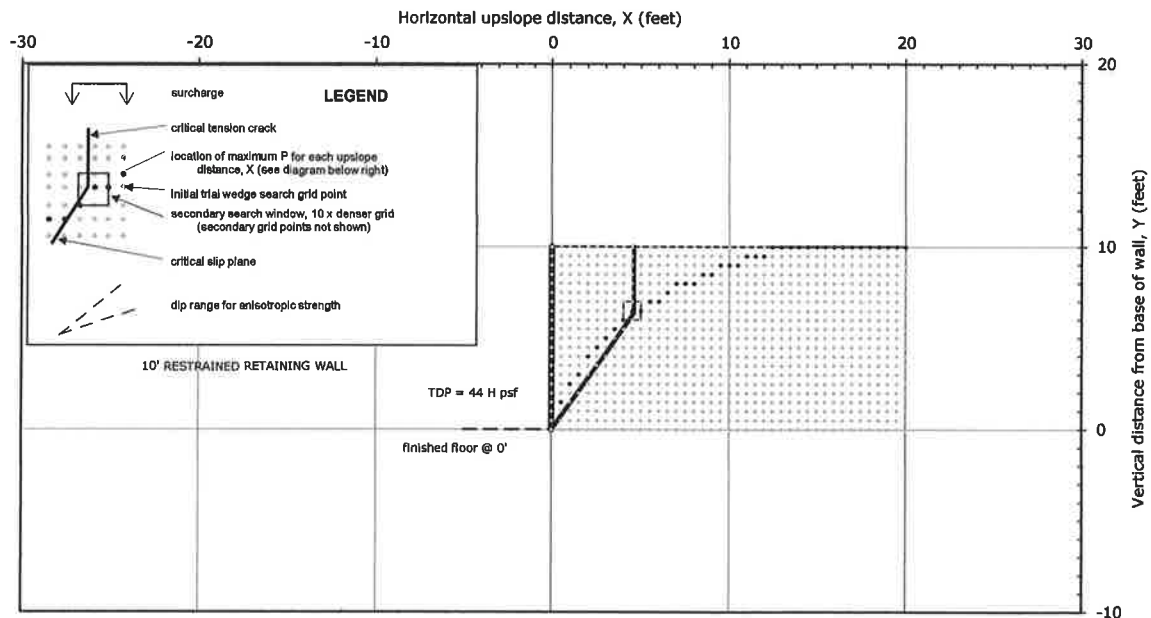
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RETAINING WALL CALCULATION

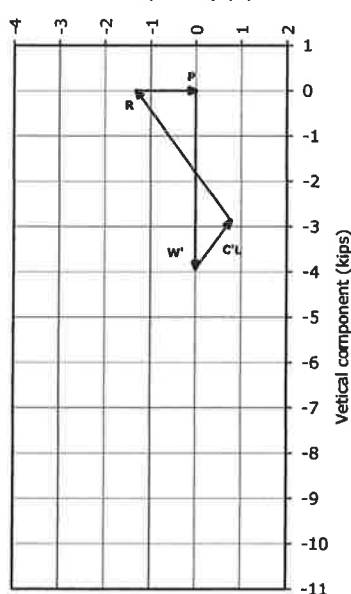
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #2b
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



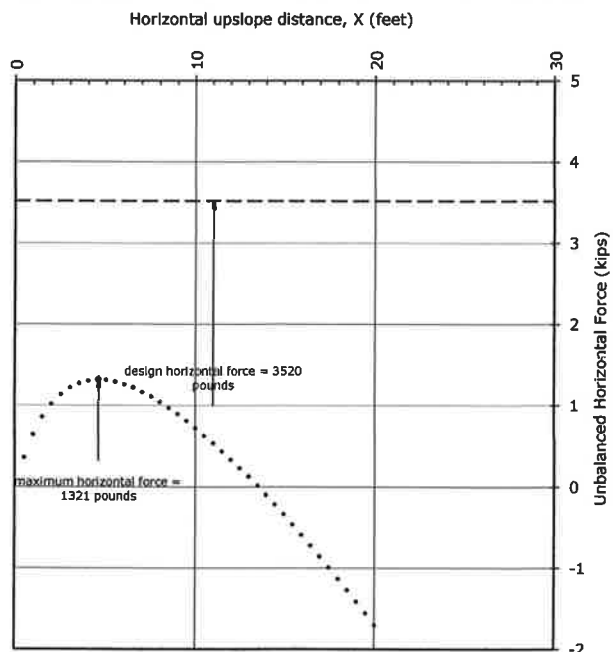
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.



BYER GEOTECHNICAL INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #2Sa
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBÉ-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-82-11 (1992), P. 78 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #1
Cohesion, Coh 250.0 psf
Phi Angle, ϕ 26.0 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device RETAINING WALL
Type RETRAINED

Retained Height, H 10 feet
Wall Friction Angle, δ 0 degrees

External Surcharge see below

General Backslope Condition* level
Loading SEISMIC
PGA_M 1.00 g

Pseudostatic Coefficients:

horizontal, K_h *** 0.33 g

vertical, K_v **** 0.00 g

Calculation Safety Factor, FS 1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** $K_v > 0$ indicates downward acceleration and upward inertial force

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist, elev)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	10		
(0,10)	(0,10)			
(3,10)	(3,10)			Uniform Load: 300 psf
(13,10)	(13,10)			
(15,10)	(15,10)			
(18,10)	(18,10)			
(20,10)	(20,10)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case
Critical Failure Angle, α 50.8 degrees
Area of Critical Wedge 40.1 square feet
Length of Critical Failure Plane, L 11.2 feet
Depth of Critical Tension Crack 1.3 feet
Horizontal Upslope Distance to Critical Tension Crack 7.1 feet
Effective Backslope on Critical Wedge, β_{eff} 0.0 degrees
Factored Phi Angle on Slip Plane, ϕ' 26.0 degrees
Factored Cohesion on Critical Slip Plane, C' 250.0 psf
Weight of Critical Wedge, W 5,014 pounds
External Surcharge on Critical Wedge, V 1,380 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, W_d 6,739 pounds
Mobilized Cohesive Force, C'_L 2,807 pounds
Mobilized Frictional Force, R 4,647 pounds
Calculated Unbalanced Force, P 2,447 pounds
Calculated Horizontal Unbalanced Force, P_h 2,447 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force 2,447 pounds
Recommended Static Horizontal Force from sheet 2a 3,520 pounds

CONCLUSIONS

THE CALCULATED STATIC FORCE EXCEEDS THE CALCULATED PSEUDO-STATIC FORCE. THEREFORE, THE RECOMMENDED DESIGN PARAMETERS ON SHEET 2A ARE SUFFICIENT.



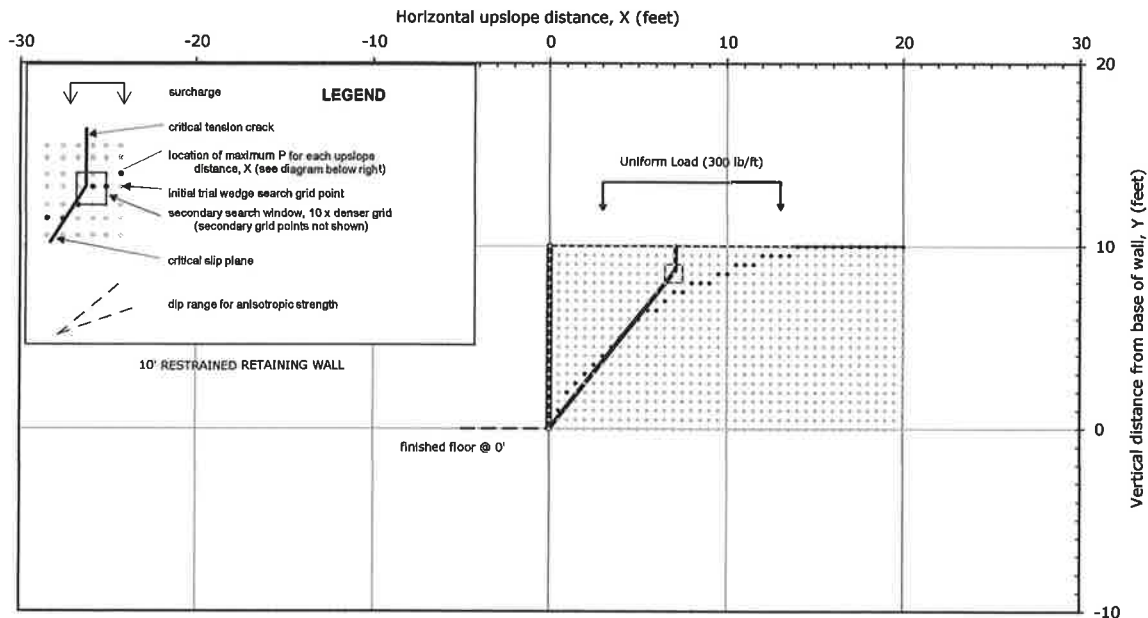
**BYER
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INC.**

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

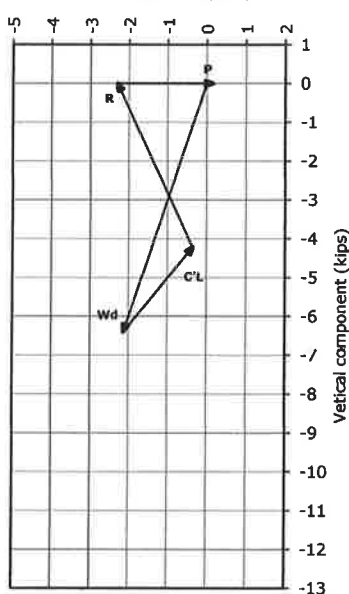
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #2Sb
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



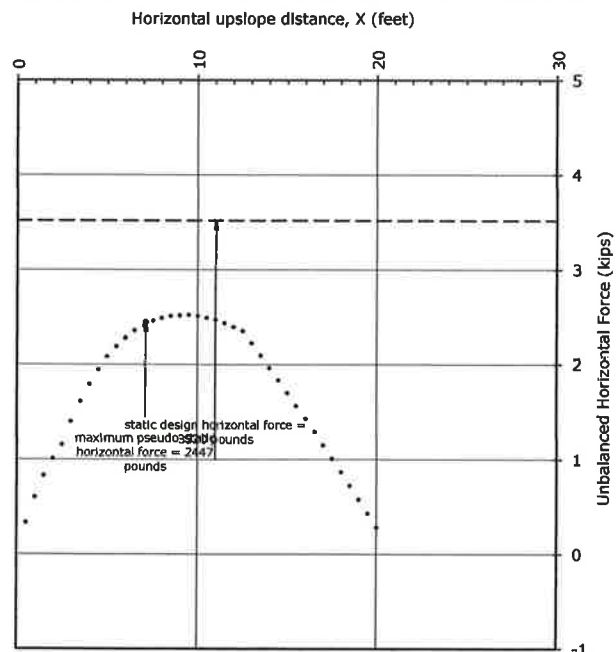
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.



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tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #3Sa
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBÉ-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #1
Cohesion, Coh 250.0 psf
Phi Angle, ϕ 26.0 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device RETAINING WALL

Type RETRAINED

Retained Height, H 10 feet

Wall Friction Angle, δ 0 degrees

External Surcharge see below

General Backslope Condition* level

Loading SEISMIC
PGA_M 1.00 g

Pseudostatic Coefficients:

horizontal, K_h *** 0.33 g

vertical, K_v **** 0.00 g

Calculation Safety Factor, FS 1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** $K_v > 0$ indicates downward acceleration and upward inertial force

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist., elev.)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	10		
(0,10)	(0,10)			
(3,10)	(3,10)			Line Load: 4000 psf
(4,10)	(4,10)			
(15,10)	(15,10)			
(18,10)	(18,10)			
(20,10)	(20,10)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case	
Critical Failure Angle, α	68.5 degrees
Area of Critical Wedge	19.4 square feet
Length of Critical Failure Plane, L	9.6 feet
Depth of Critical Tension Crack	1.1 feet
Horizontal Upslope Distance to Critical Tension Crack	3.5 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, ϕ'	26.0 degrees
Factored Cohesion on Critical Slip Plane, C'	250.0 psf
Weight of Critical Wedge, W	2,428 pounds
External Surcharge on Critical Wedge, V	4,000 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, Wd	6,774 pounds
Mobilized Cohesive Force, C'L	2,391 pounds
Mobilized Frictional Force, R	5,704 pounds
Calculated Unbalanced Force, P	5,193 pounds
Calculated Horizontal Unbalanced Force, P _h	5,193 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	5,193 pounds
Recommended Static Horizontal Force from sheet 3a	3,520 pounds
Calculated Seismic Force ***	1,673 pounds

*** the seismic force should be applied at 0.6H, where H is the retained height

CONCLUSIONS

THE CALCULATED SEISMIC FORCE ON THE WALL IS THE DIFFERENCE BETWEEN THE PSEUDO-STATIC AND STATIC FORCE, AND IS 1673 POUNDS. THE WALL SHOULD BE DESIGNED FOR THIS FORCE IN ADDITION TO THE RECOMMENDED DESIGN PARAMETERS ON SHEET 3A. THE SEISMIC FORCE MAY BE APPLIED AT 0.6H ABOVE THE BASE, WHERE H IS THE RETAINED HEIGHT.



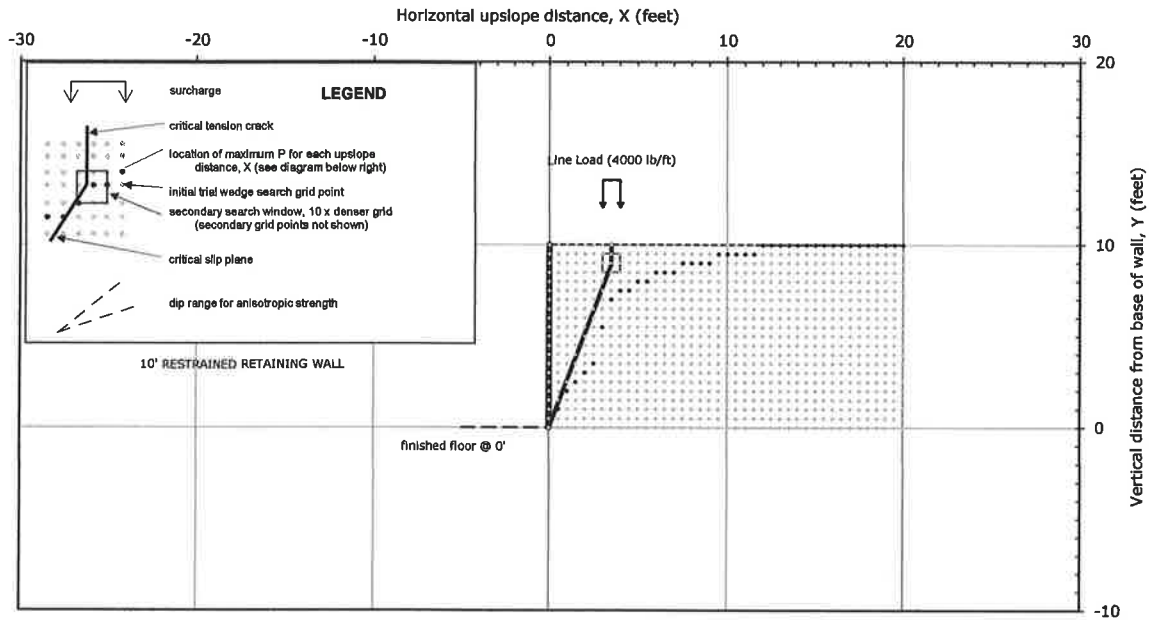
**BYER
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tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

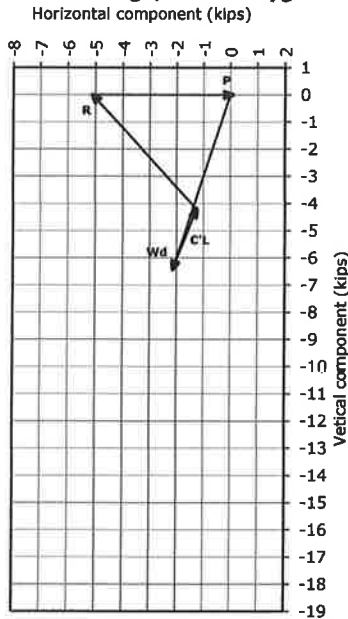
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #3Sb
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



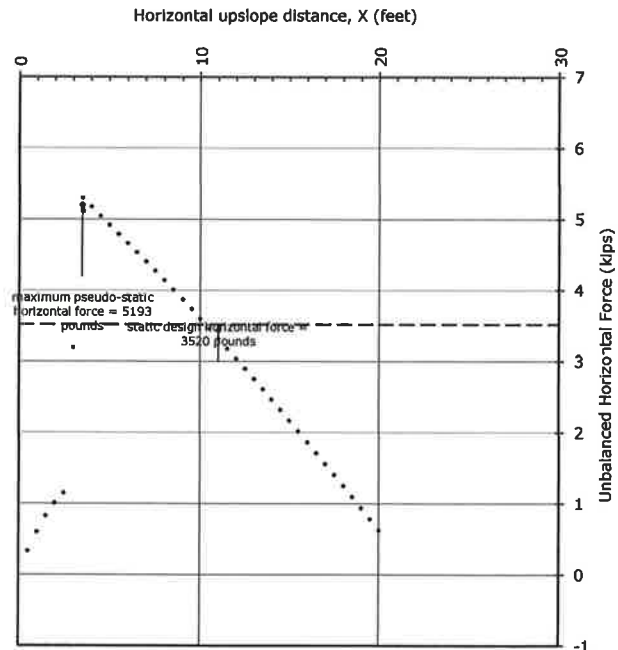
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.



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TEMPORARY EXCAVATION HEIGHT

BG: **23176** ENGINEER: **RSB**
CLIENT: **I & L Investment and Management, Inc.**

CALCULATION SHEET # **4**

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium (In-Situ)	WALL HEIGHT:	5 feet
SHEAR DIAGRAM:	2	BACKSLOPE ANGLE:	45 degrees
COHESION:	550 psf	SURCHARGE:	0 pounds
PHI ANGLE:	27.5 degrees	SURCHARGE TYPE:	u Uniform
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	1 feet
CD (C/FS):	440.0 psf	FINAL TENSION CRACK:	15 feet
PHID = $\text{ATAN}(\text{TAN}(\text{PHI})/\text{FS})$ =	22.6 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	41 degrees
AREA OF TRIAL FAILURE WEDGE	5.1 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	633.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	765 trials
LENGTH OF FAILURE PLANE	1.3 feet
DEPTH OF TENSION CRACK	5.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	1.0 feet
CALCULATED HORIZONTAL THRUST	-356.7 pounds
CALCULATED EQUIVALENT FLUID PRESSURE	-28.5 pcf
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	5.0 feet

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY VERTICAL
EXCAVATIONS UP TO FIVE FEET HIGH WITH 1:1 TRIM HAVE A
NEGATIVE THRUST AND ARE TEMPORARILY STABLE.



BYER GEOTECHNICAL INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #5a
Cantilevered Shoring Pile, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED SHORING PILE. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #2
Cohesion, Coh 550.0 psf
Phi Angle, ϕ 27.5 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device SHORING PILE
Type CANTILEVERED
Retained Height, H 15 feet
Wall Friction Angle, δ 0 degrees
External Surcharge see below
General Backslope Condition* level
Loading STATIC

Calculation Safety Factor, FS 1.25

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid 1531 trials
Trial Wedges Analyzed, Secondary Search Window 324 trials
Critical Failure Angle, α 54.3 degrees
Area of Critical Wedge 56.7 square feet
Length of Critical Failure Plane, L 8.4 feet
Depth of Critical Tension Crack 8.2 feet
Horizontal Upslope Distance to Critical Tension Crack 4.9 feet
Effective Backslope on Critical Wedge, β_{eff} 0.0 degrees
Factored Phi Angle on Slip Plane, ϕ' 22.6 degrees
Factored Cohesion on Critical Slip Plane, C' 440.0 psf
Weight of Critical Wedge, W 7,089 pounds
External Surcharge on Critical Wedge, V 717 pounds
Static Gravitational Driving Force, W' 7,806 pounds
Mobilized Cohesive Force, $C'L$ 3,685 pounds
Mobilized Frictional Force, R 5,656 pounds
Calculated Unbalanced Force, P 819 pounds
Calculated Horizontal Unbalanced Force, P_h 819 pounds
Calculated Equivalent Fluid Pressure 7.3 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP 30.0 pcf

Design Horizontal Force 3,375 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist, elev)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	15		
(0,15)	(0,15)			
(3,15)	(3,15)			
(13,15)	(13,15)			
(15,15)	(15,15)			
(18,15)	(18,15)			
(28,15)	(28,15)			

Uniform Load: 300 psf

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED SHORING PILE, WITH A RETAINED HEIGHT OF UP TO 15 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 30 POUNDS PER CUBIC FOOT. FOR PILES, THE PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.



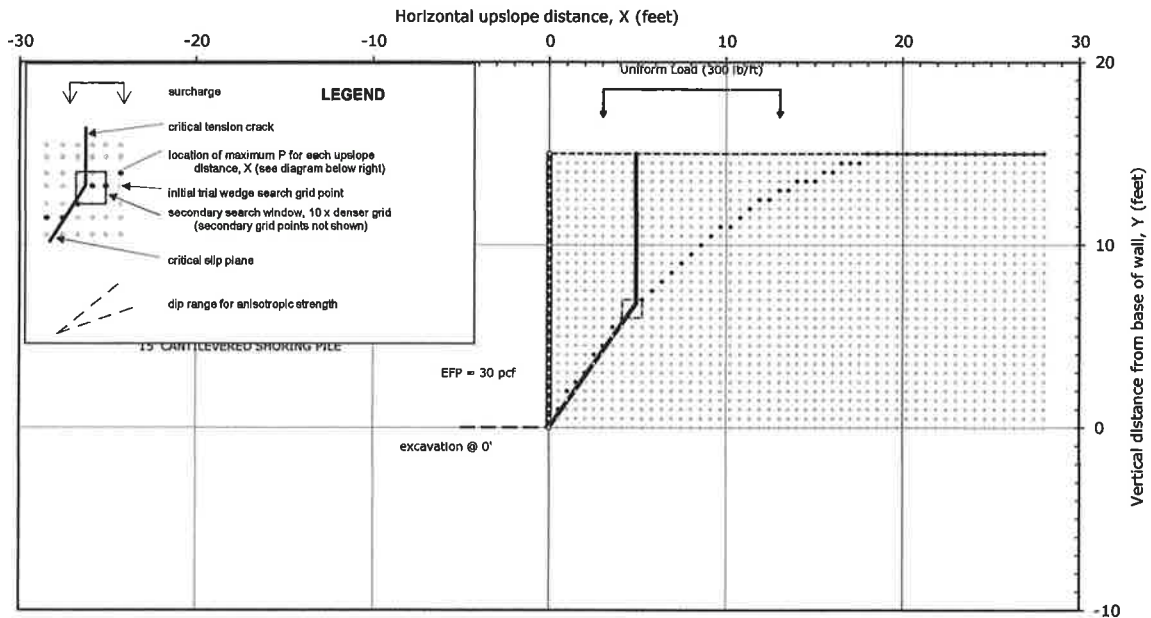
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tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

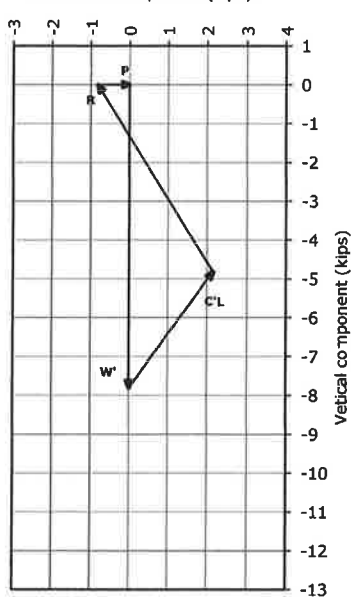
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #5b
Cantilevered Shoring Pile, basement

Cross Section and Critical Active Wedge



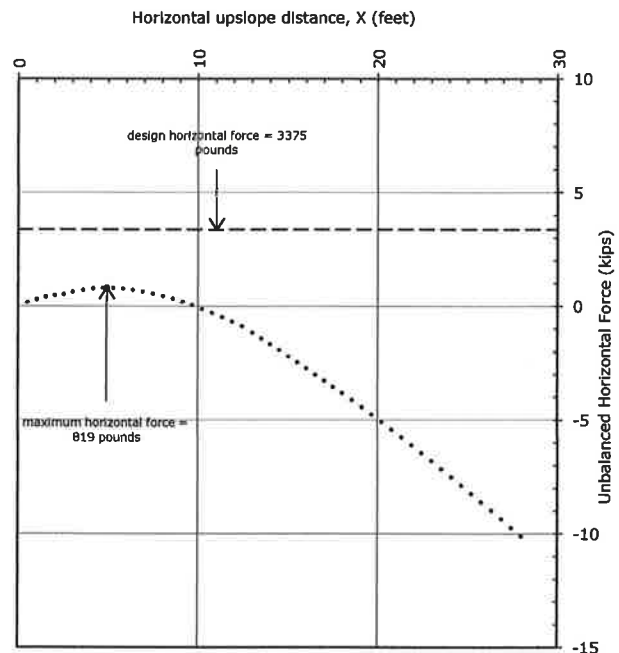
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, P_h , is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, P_h (kips)



The maximum calculated horizontal unbalanced pressure, P_h , is plotted for each upslope distance, X. The location of the maximum P_h for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.



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tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

BG 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #6a
Cantilevered Shoring Pile, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED SHORING PILE. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1086, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material Alluvium
Shear Diagram #2
Cohesion, Coh 550.0 psf
Phi Angle, ϕ 27.5 degrees
Density, γ 125.0 pcf

Anisotropic Strength Function NO

Restraining Device SHORING PILE
Type CANTILEVERED
Retained Height, H 15 feet
Wall Friction Angle, δ 0 degrees
External Surcharge see below
General Backslope Condition* level
Loading STATIC

Calculation Safety Factor, FS 1.25

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid 1531 trials
Trial Wedges Analyzed, Secondary Search Window 324 trials
Critical Failure Angle, α 63.8 degrees
Area of Critical Wedge 40.1 square feet
Length of Critical Failure Plane, L 7.9 feet
Depth of Critical Tension Crack 7.9 feet
Horizontal Upslope Distance to Critical Tension Crack 3.5 feet
Effective Backslope on Critical Wedge, β_{eff} 0.0 degrees
Factored Phi Angle on Slip Plane, ϕ' 22.6 degrees
Factored Cohesion on Critical Slip Plane, C' 440.0 psf
Weight of Critical Wedge, W 5,009 pounds
External Surcharge on Critical Wedge, V 4,000 pounds
Static Gravitational Driving Force, W' 9,009 pounds
Mobilized Cohesive Force, $C'L$ 3,483 pounds
Mobilized Frictional Force, R 7,816 pounds
Calculated Unbalanced Force, P 3,603 pounds
Calculated Horizontal Unbalanced Force, P_h 3,603 pounds
Calculated Equivalent Fluid Pressure 32.0 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP 33.0 pcf

Design Horizontal Force 3,713 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist., elev.)	(X, Y)	H (ft)	β (deg)	surcharge
(0,0)	(0,0)	15		
(0,15)	(0,15)			
(3,15)	(3,15)			Line Load: 4000 psf
(4,15)	(4,15)			
(15,15)	(15,15)			
(18,15)	(18,15)			
(28,15)	(28,15)			

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED SHORING PILE, WITH A RETAINED HEIGHT OF UP TO 15 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 33 POUNDS PER CUBIC FOOT. FOR PILES, THE PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β , and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.



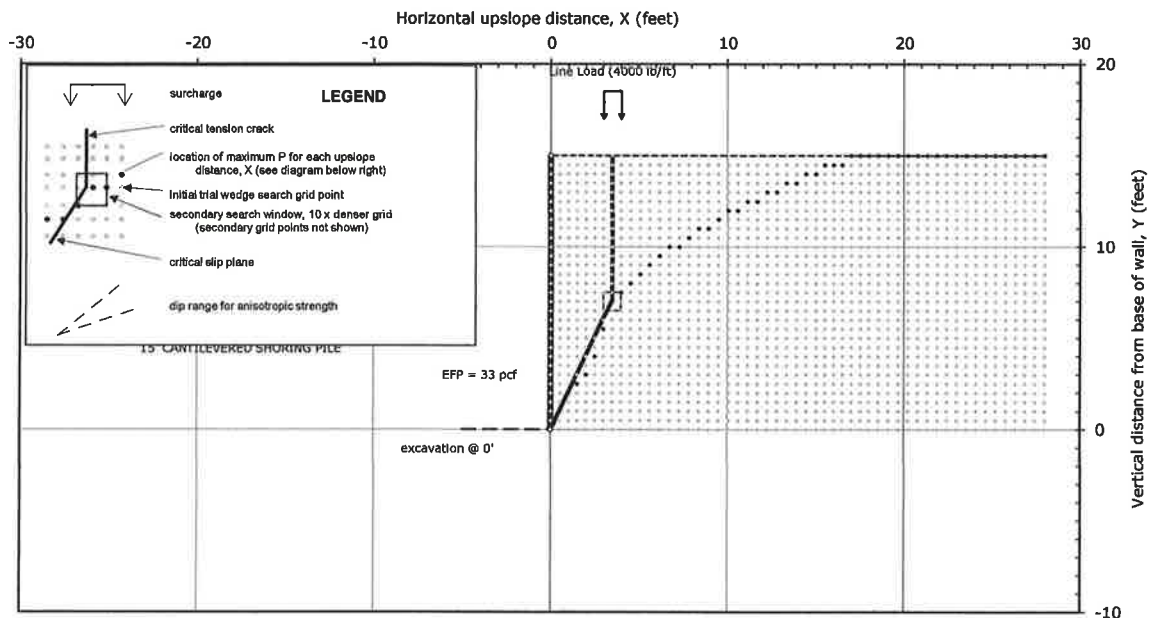
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SHORING PILE CALCULATION

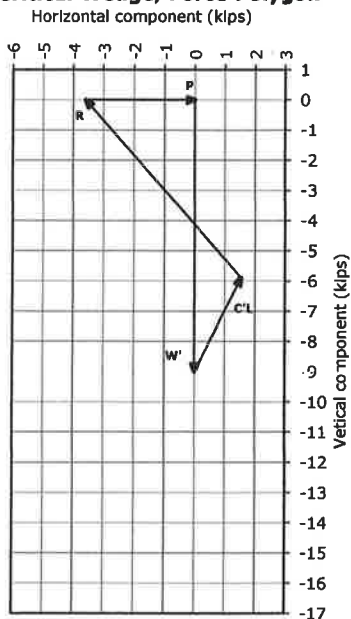
BG: 23176 CLIENT: I & L Investment and Management, Inc.
CONSULTANT: RSB
SHEET: #6b
Cantilevered Shoring Pile, basement

Cross Section and Critical Active Wedge



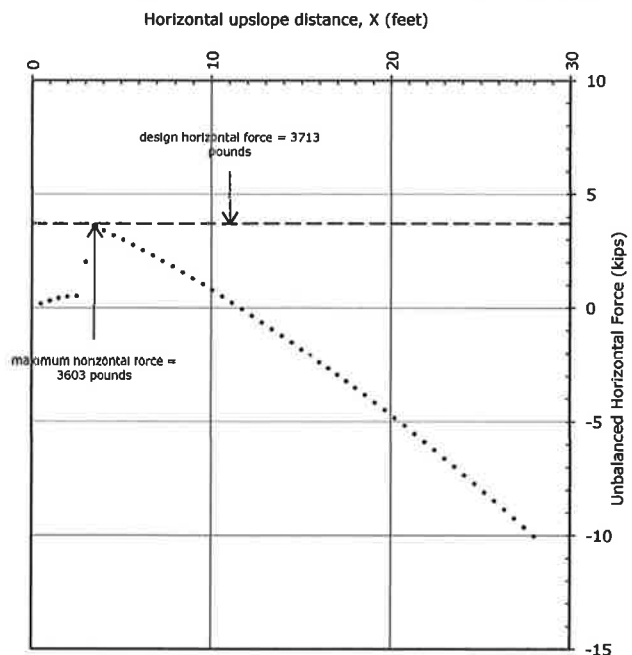
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, P_h , is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, P_h (kips)



The maximum calculated horizontal unbalanced pressure, P_h , is plotted for each upslope distance, X. The location of the maximum P_h for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

APPENDIX V

Figures



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AERIAL VICINITY MAP

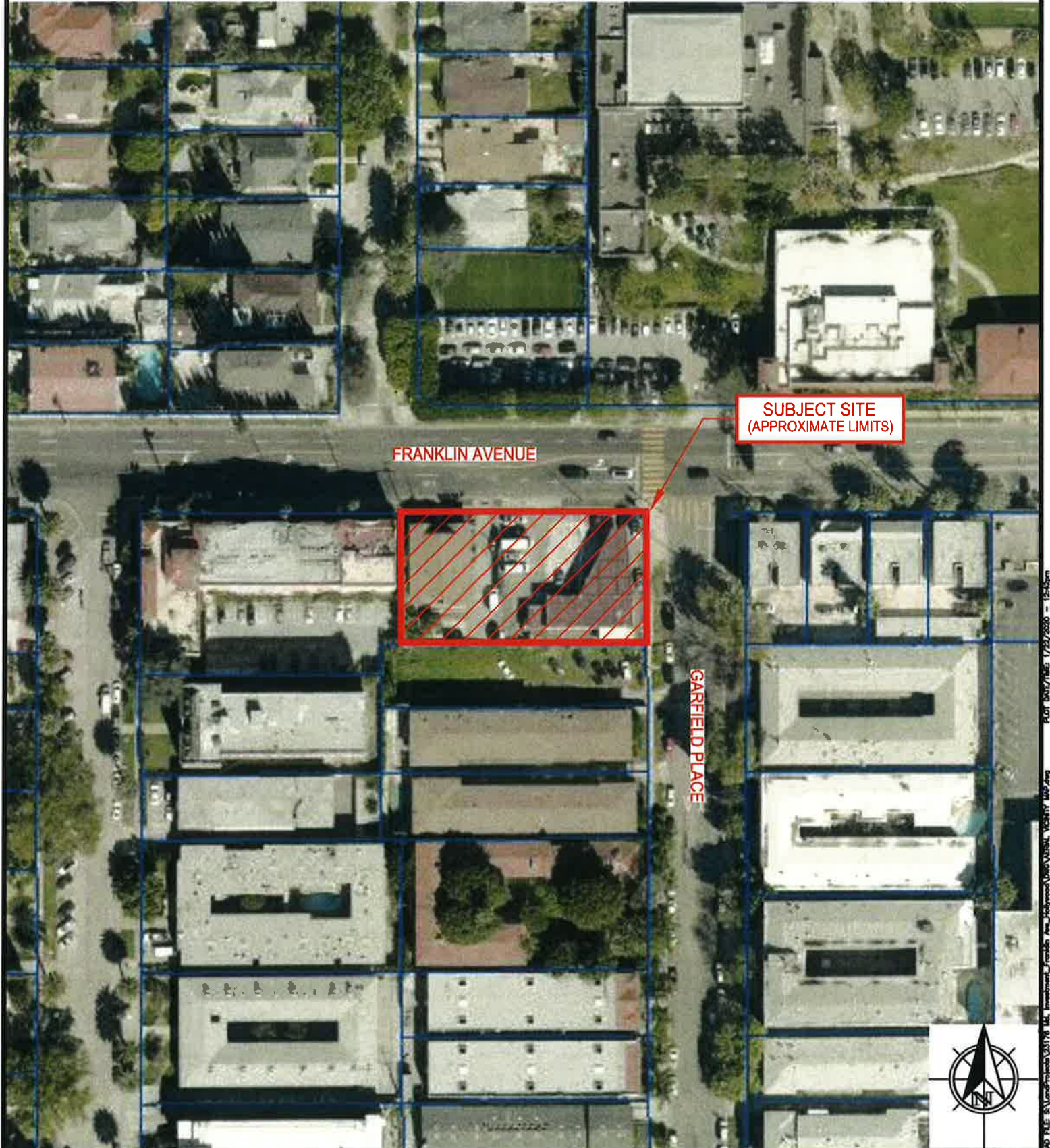
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 100'

REFERENCE: LOS ANGELES COUNTY DEPARTMENT OF REGIONAL PLANNING, GIS-NET, 2013, http://gis.planning.lacounty.gov/GIS-NET_Public/Viewer.html





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REGIONAL TOPOGRAPHIC MAP

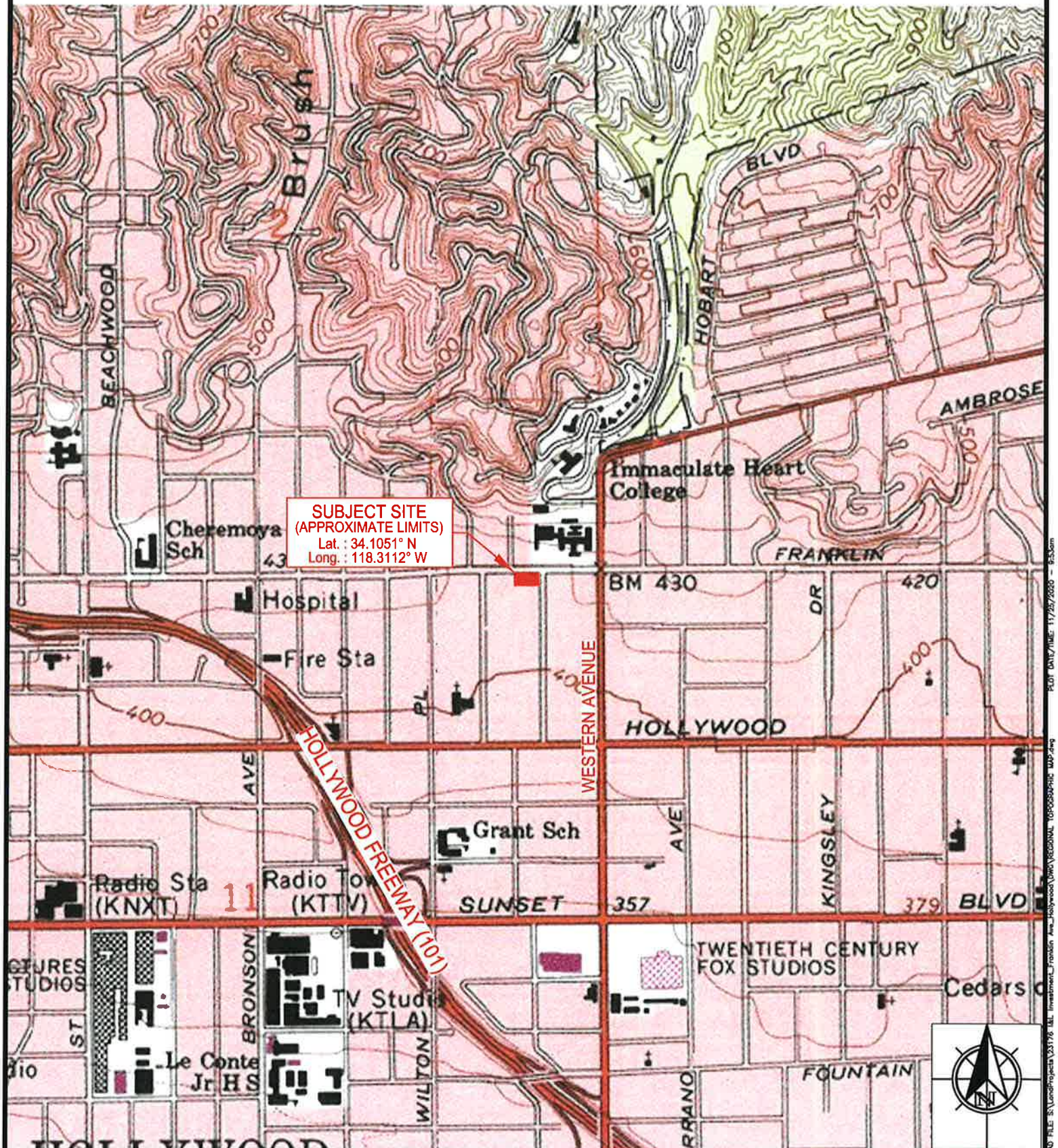
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, HOLLYWOOD 7.5-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1981.





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HISTORIC TOPOGRAPHIC MAP

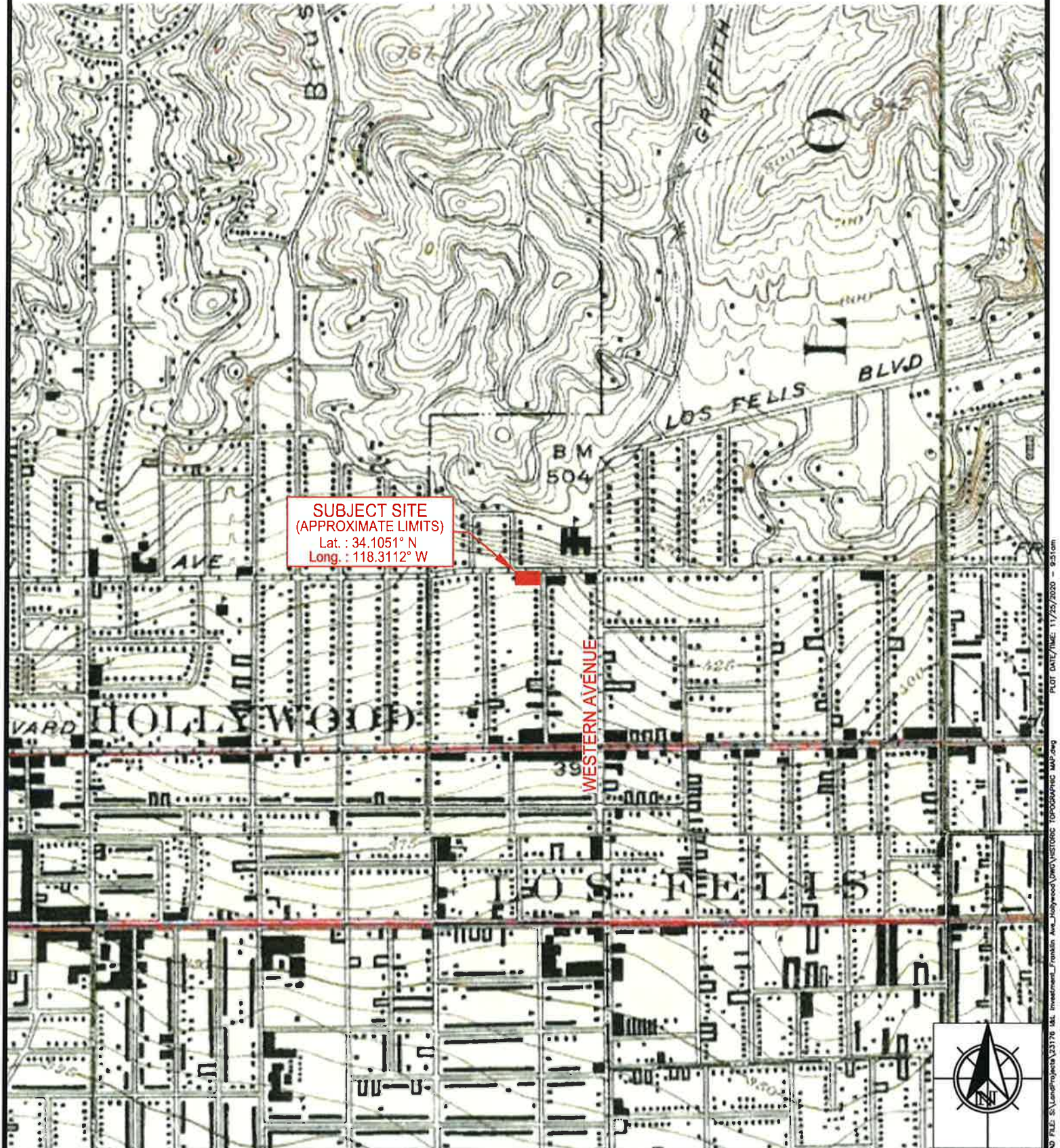
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, HOLLYWOOD 6-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1926.





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REGIONAL GEOLOGIC MAP #1

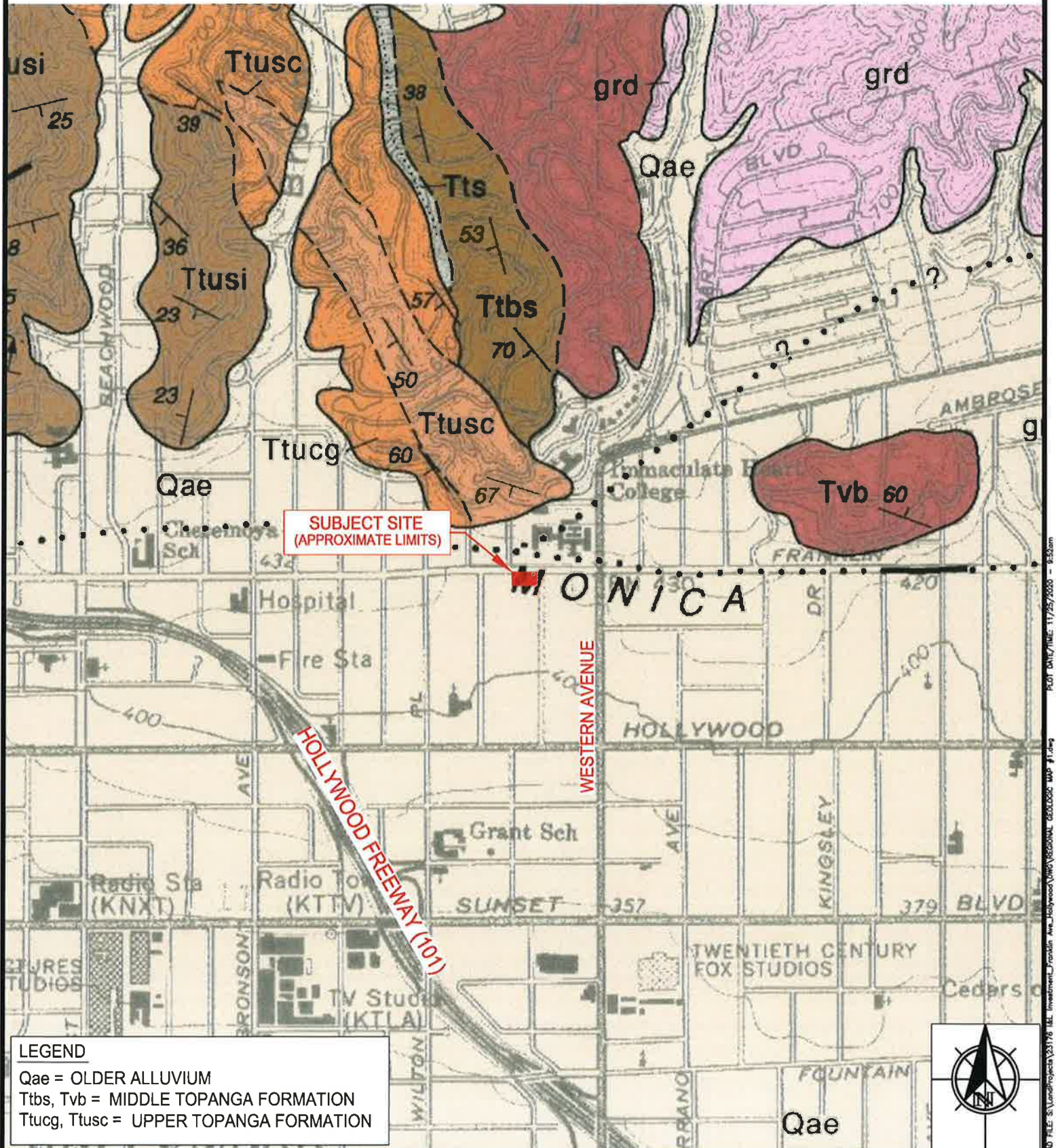
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: DIBBLEE, T.W. (1991), GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES, LOS ANGELES, CALIFORNIA
DIBBLEE GEOLOGICAL FOUNDATION, MAP DF-30.





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REGIONAL GEOLOGIC MAP #2

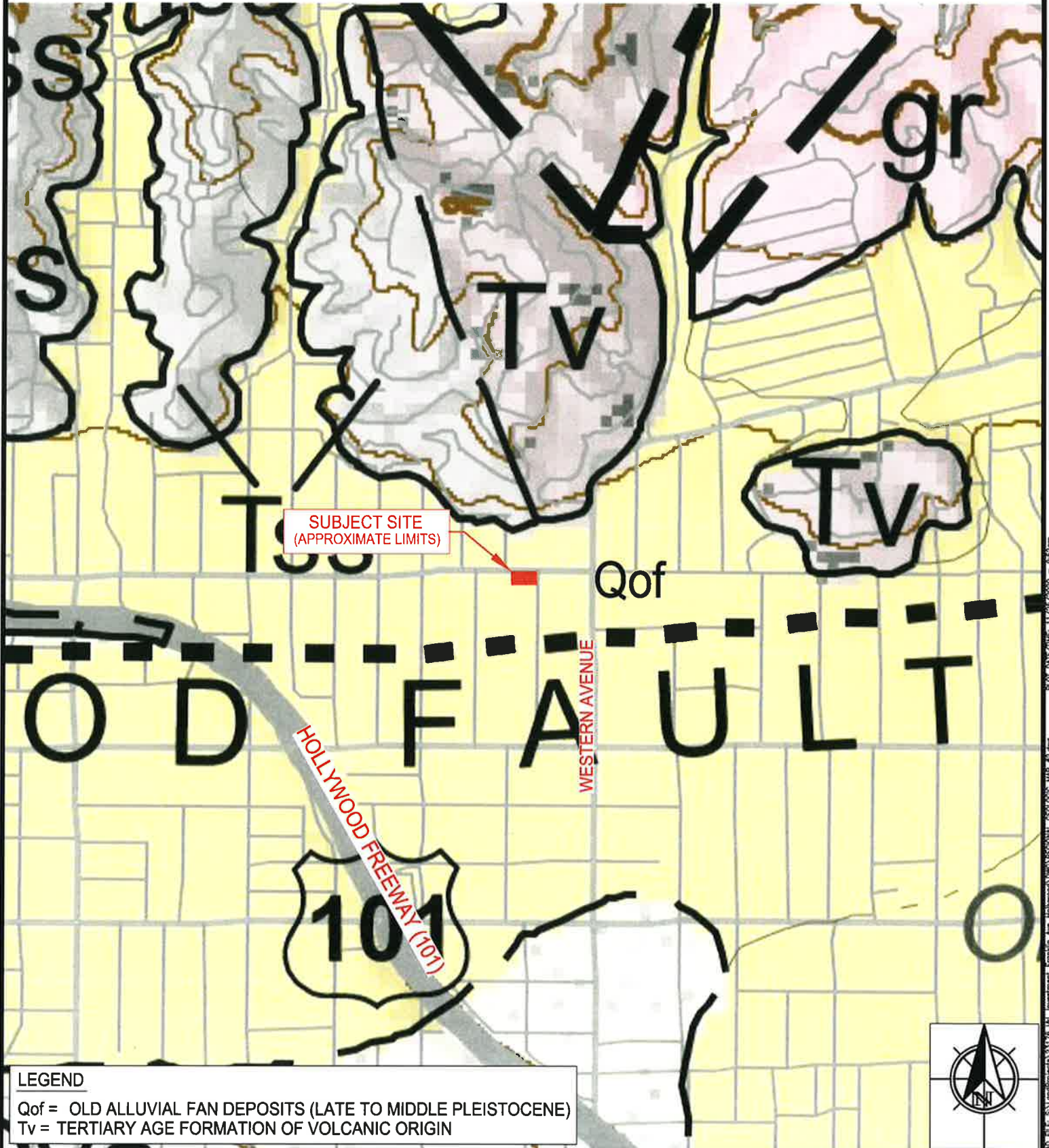
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

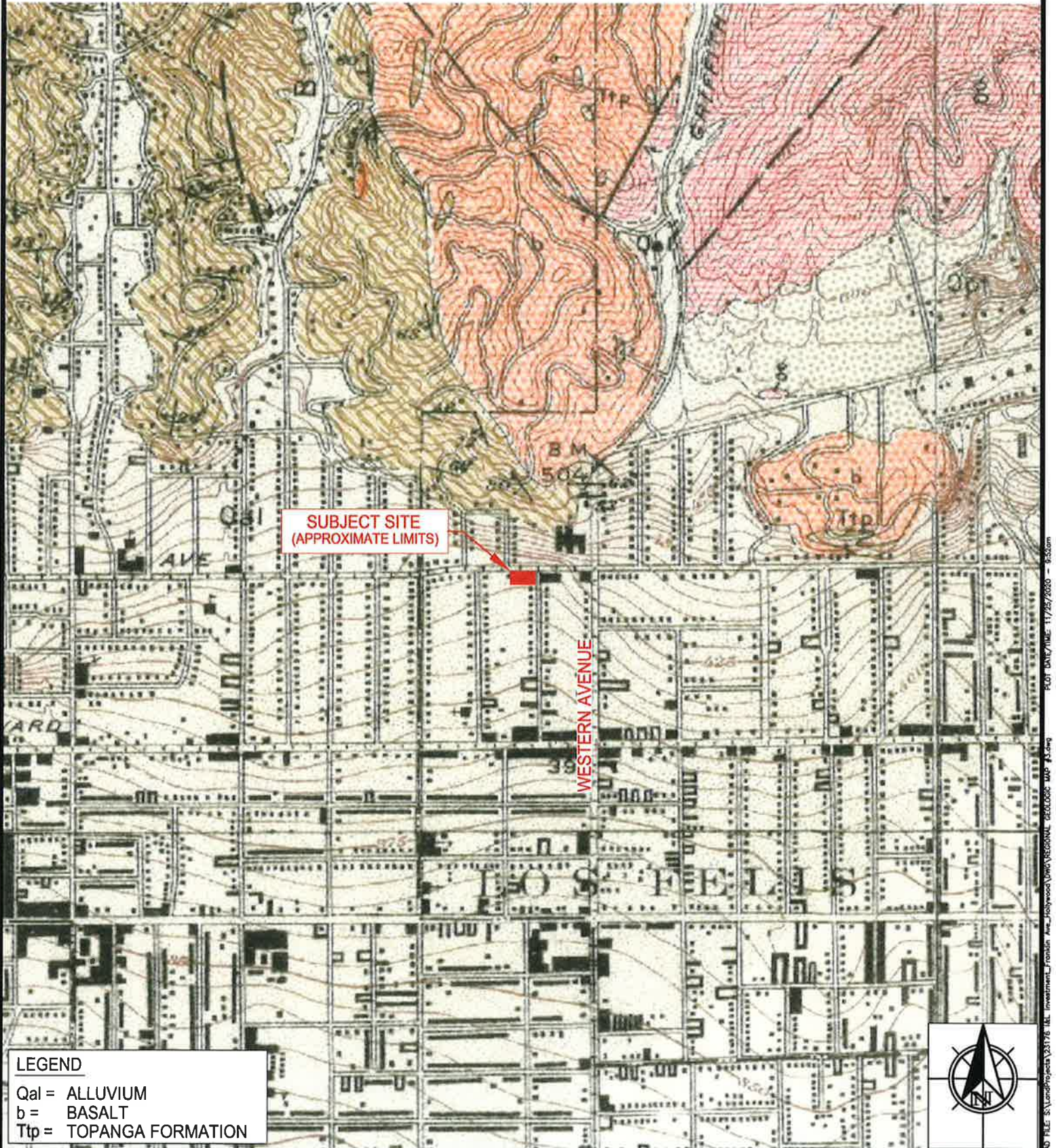
DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: BEDROSSIAN, T.L., AND ROFFERS, P.D. (2010) GEOLOGIC COMPILATION OF QUATERNARY SURFICIAL DEPOSITS IN SOUTHERN CALIFORNIA, LOS ANGELES 30' X 60' QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, SPECIAL REPORT 217, PLATE 9, JULY 2010.



REFERENCE: GEOLOGIC MAP OF THE EASTERN PART OF THE SANTA MONICA MOUNTAINS AND ADJACENT AREAS, LOS ANGELES COUNTY, CALIFORNIA., GEOLOGY BY H.W. HOOTS, BASED FROM SURVEY MADE IN 1923-1925, PROFESSIONAL PAPER 165, PLATE 16.





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REGIONAL GEOLOGIC MAP #4

BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: AS NOTED

REFERENCE: CALIFORNIA GEOLOGICAL SURVEY FAULT EVALUATION REPORT FER 253, SUPPLEMENT NO. 1; THE HOLLYWOOD FAULT IN THE HOLLYWOOD 7.5' QUADRANGLE LOS ANGELES COUNTY, CALIFORNIA, BY JANIS L. HERNANDEZ ENGINEERING GEOLOGIST, DATED NOVEMBER 05, 2014. PAGE 22, FIGURE 14.

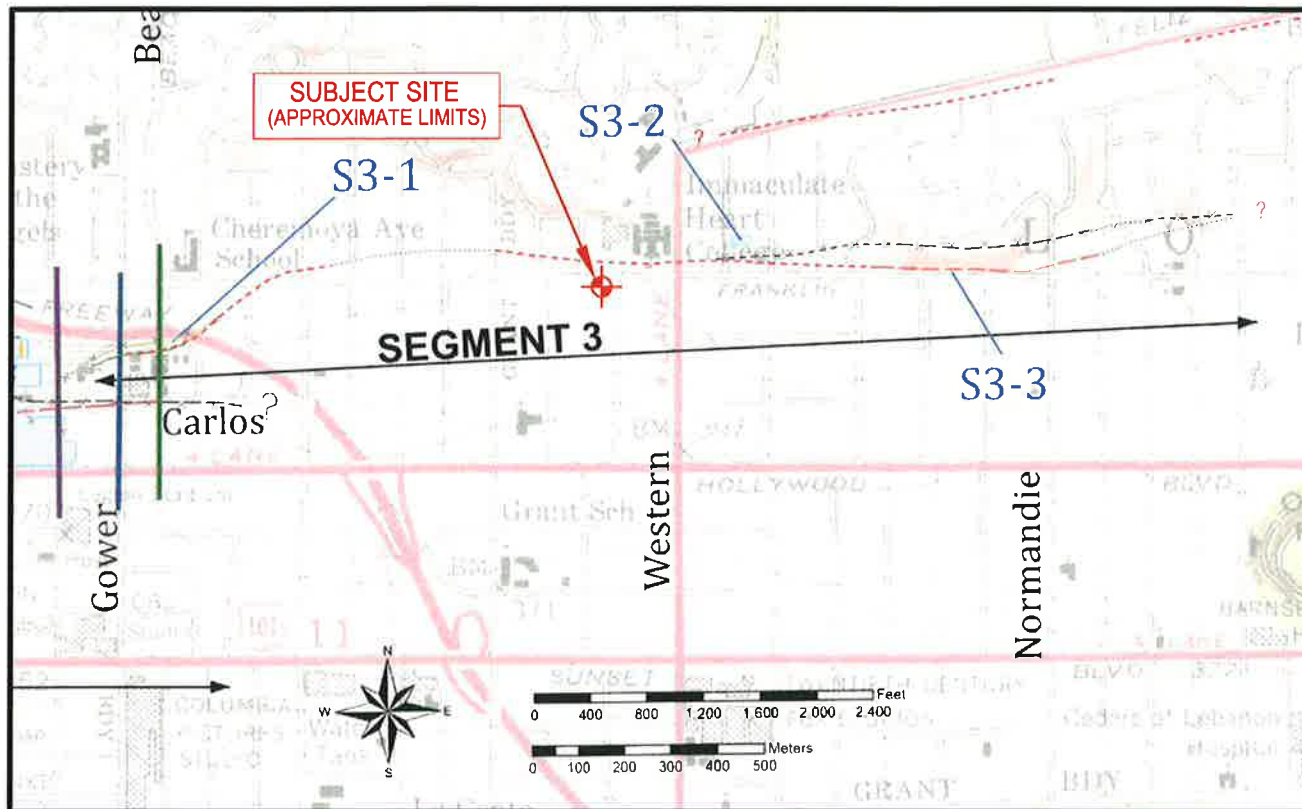


Figure 14 – Localities within Segment 3. North-south lines near Gower are elevation survey transect locations by LCI. At locality S3-1, the red fault trace indicates the revised fault trace location based on breaks in slope identified within the LCI surveys. Purple cross hatch on the black fault trace indicates location of fault trace as reported in the FER. Locality S3-2 is the Bay City Geology study site. Locality S3-3 indicates the modified fault trace (in red) based on revised interpretation of the scarp utilizing the 1926 topographic base map as described in the text.

Bay City Geology, Inc. (2014) performed an investigation for a site located within the Preliminary EFZ at locality S3-2 (Figure 14). Three large diameter borings were drilled, revealing a thin layer of older alluvium, underlain by weathered Topanga Formation siltstone and sandstone, grading to less weathered bedrock at about 21 feet bgs. Bedding planes in Topanga measured in the borings had northwest strikes, with dips to the southwest. Bay City stated that similar geologic units were encountered in all of the borings at similar depths, where they indicated this was positive evidence for continuous, unfaulted geologic units across the site. From these data, they inferred the Hollywood fault is located south of the site.

The data presented in the Bay City Geology report is consistent with the interpretation presented in the initial FER that the Hollywood Fault lies south of this site as shown on the preliminary Alquist-Priolo Earthquake Fault Zone map.

At locality S3-3 (Figure 14), CGS revised the location of the Hollywood Fault trace along Franklin Avenue east of Western, based on a review of the 1926 topographic map covering this area. Our initial interpretation in this area utilized a later version of the topographic map, where topographic details were more subtle. Our reinterpretation of the fault trace follows closely with the trace mapped by Dolan et al, 1997.



C:\Users\jgarcia\Documents\Projects\23176 I&L Investment\Regional Map #4.mxd 6/25/2020 10:46am



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REGIONAL FAULT MAP

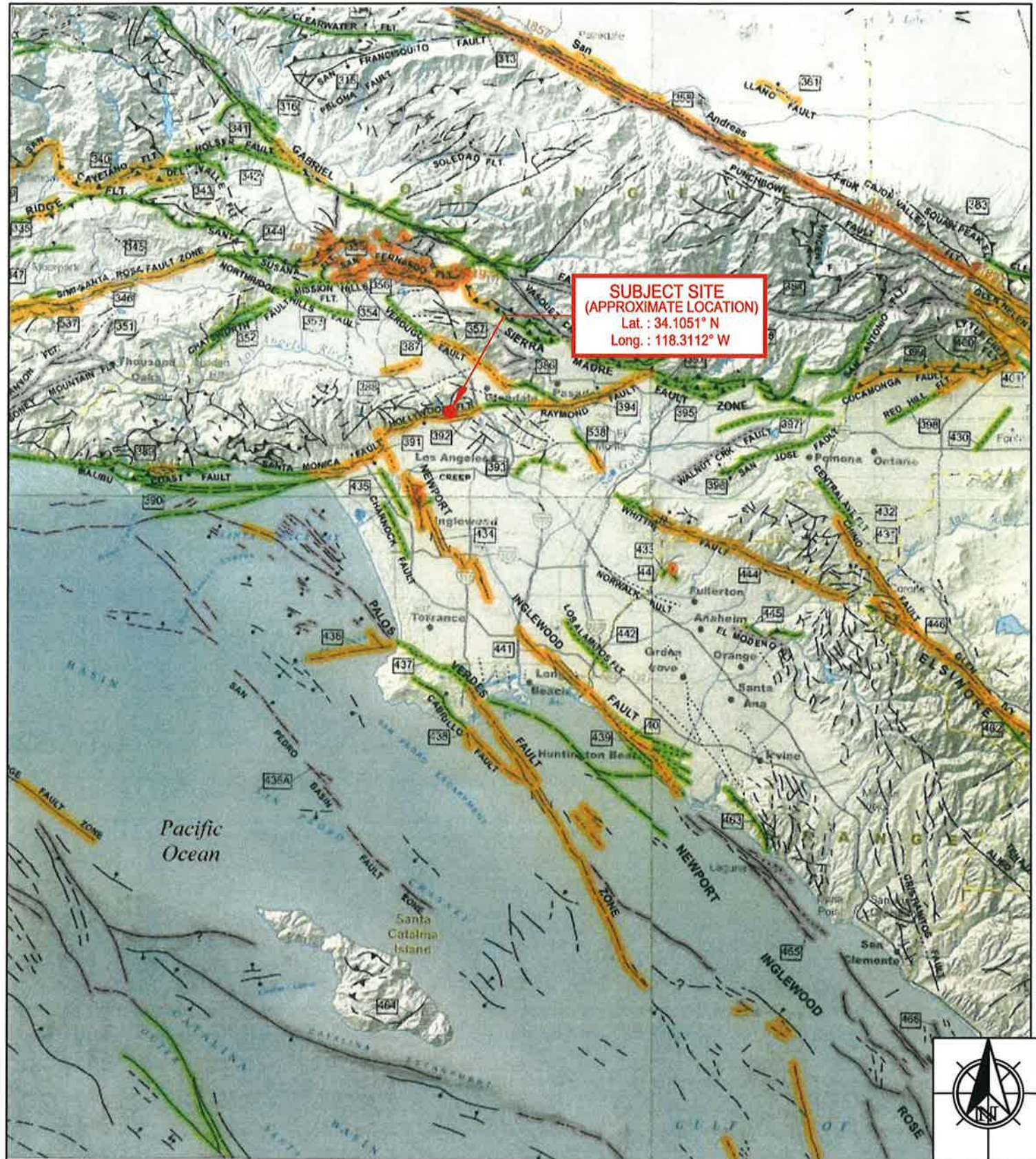
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 12 MILES

REFERENCE: JENNINGS, C.W., AND BRYANT, W.A., 2010, FAULT ACTIVITY MAP OF CALIFORNIA GEOLOGICAL SURVEY, 150th ANNIVERSARY, MAP No 6.



LOCAL FAULT MAP

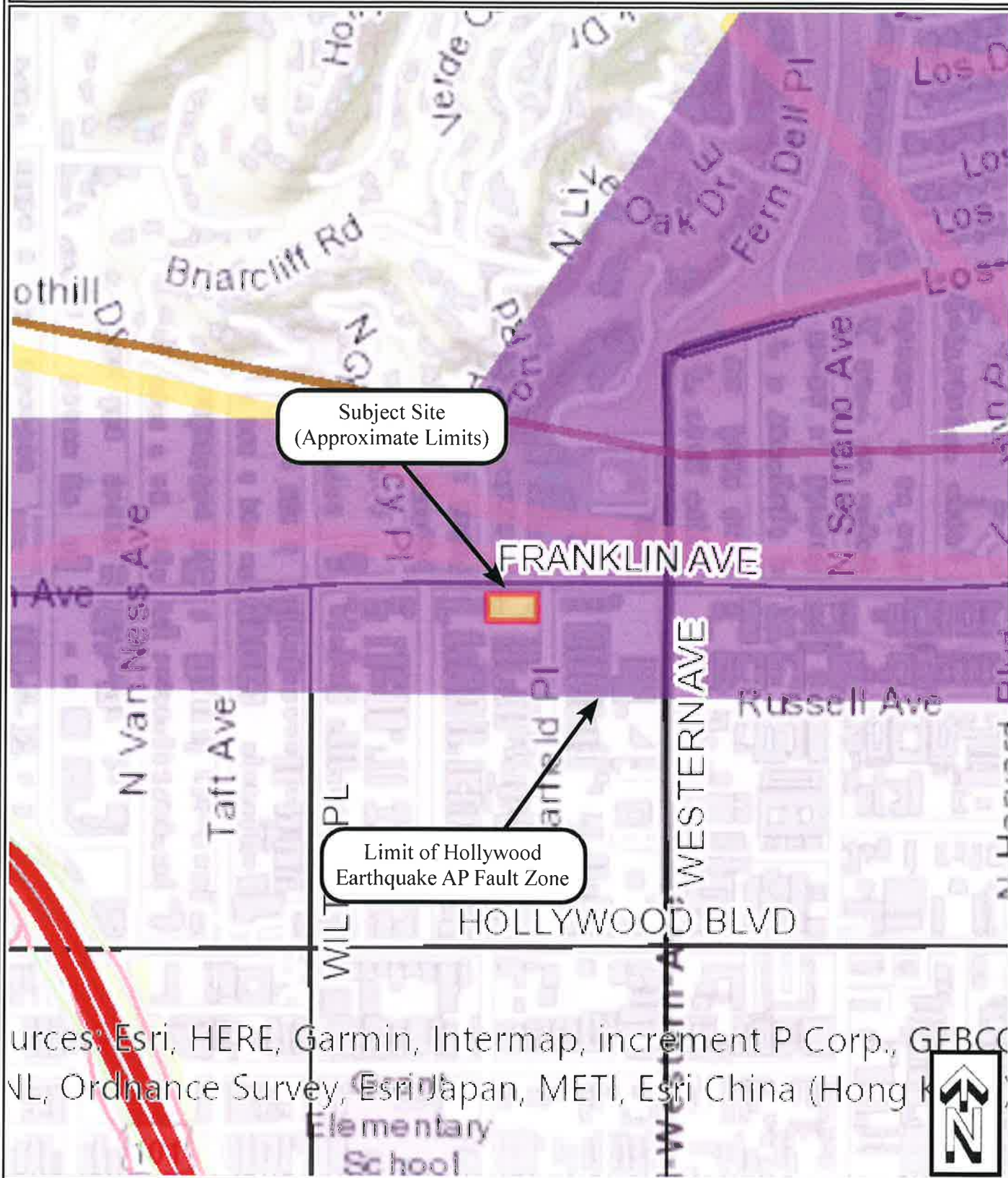
BG: 23176

CLIENT: I&L INVESTMENT
AND MANAGEMENT, INC.

ENGINEER: RSB

SCALE: 1" = 500'

REFERENCE: <http://navigatela.lacity.org/NavigateLA/>





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SEISMIC HAZARD ZONES MAP

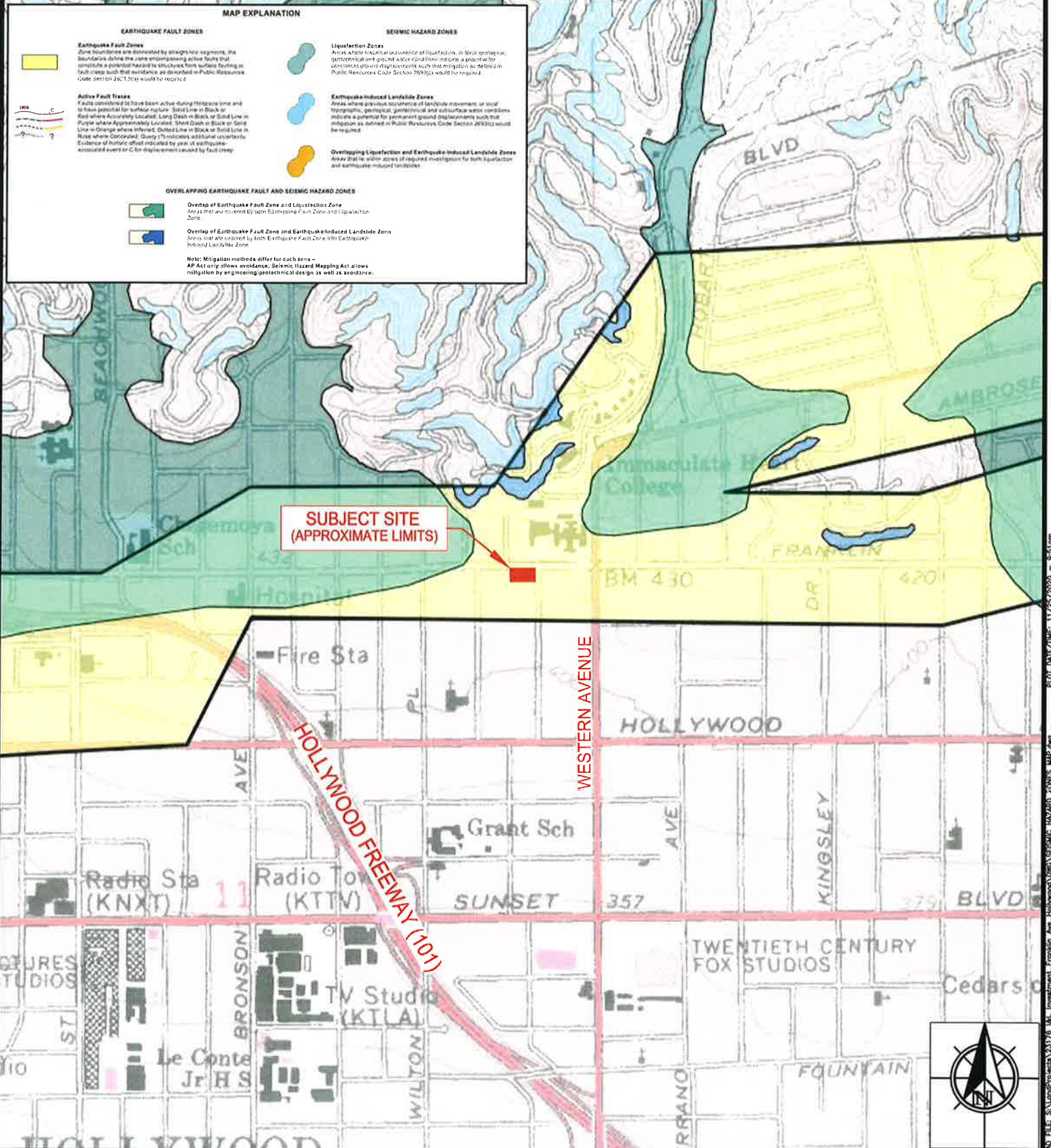
BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION BEVERLY HILLS QUADRANGLE; EARTHQUAKE FAULT ZONES, DATED NOVEMBER 06, 2014 AND SEISMIC HAZARD ZONES, DATED MARCH 25, 1999.





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HISTORIC-HIGH GROUNDWATER MAP

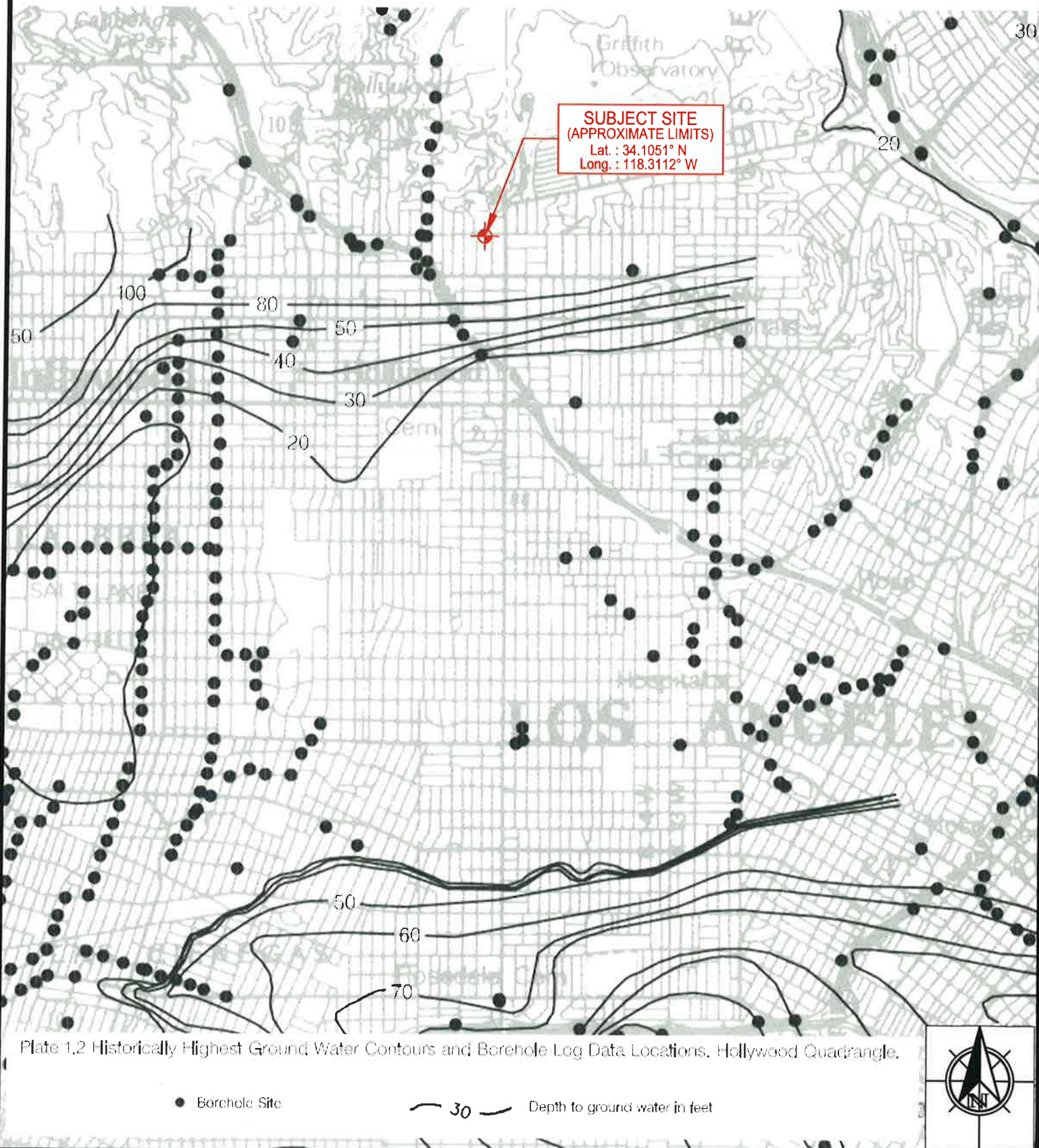
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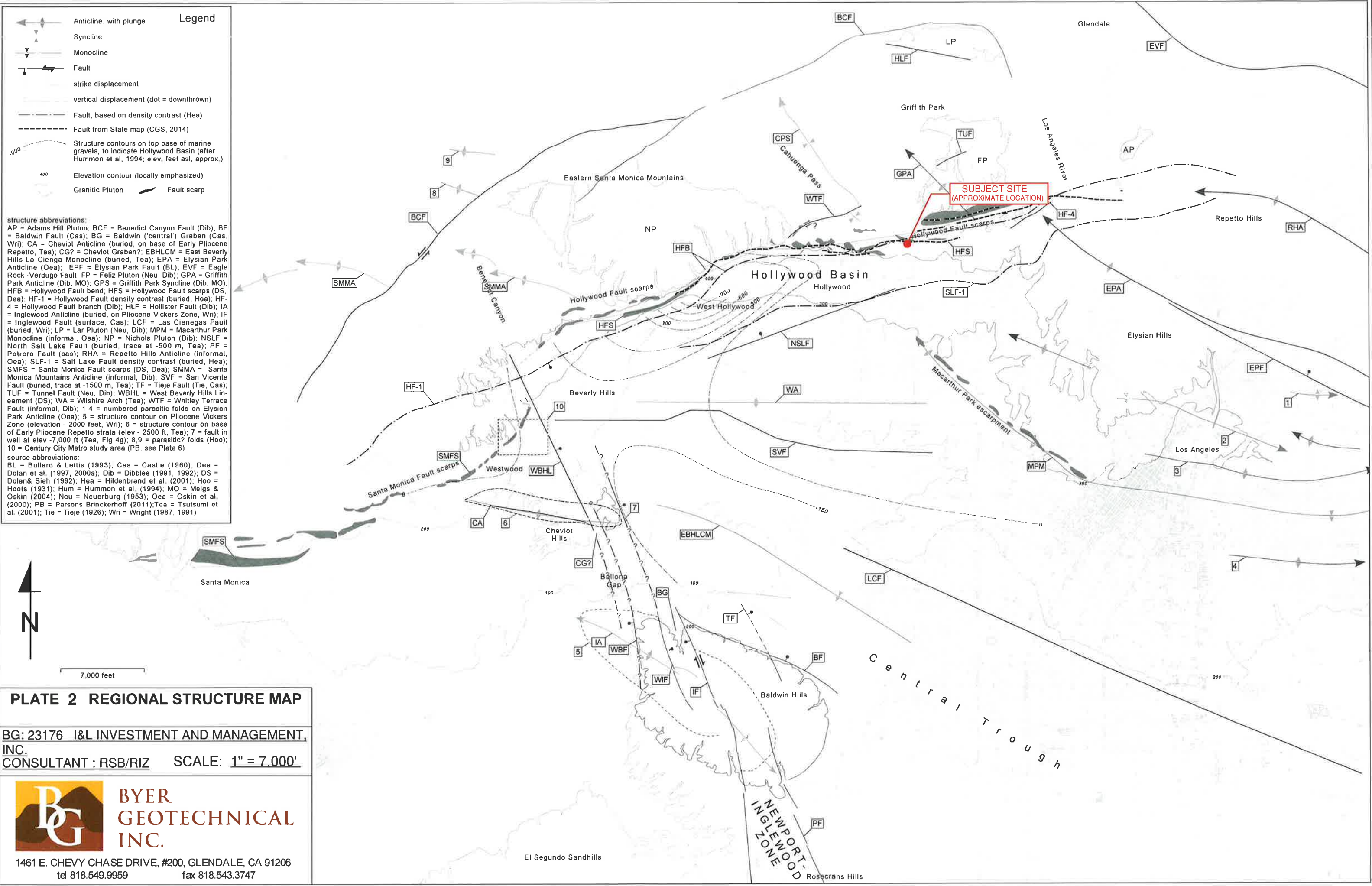
CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 4000'

REFERENCE: CGS, 1998, Seismic Hazard Zone Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.






structure abbreviations:
AP = Adams Hill Pluton; BCF = Benedict Canyon Fault (Dib); BF = Baldwin Fault (Cas); BG = Baldwin ('central') Graben (Cas, Wri); CA = Cheviot Anticline (buried, on base of Early Pliocene Repetto, Tea); CG? = Cheviot Graben?; EBHLCM = East Beverly Hills-La Cienega Monocline (buried, Tea); EPA = Elysian Park Anticline (Oea); EPF = Elysian Park Fault (BL); EVF = Eagle Rock-Verdugo Fault; FP = Feliz Pluton (Neu, Dib); GPA = Griffith Park Anticline (Dib, MO); GPS = Griffith Park Syncline (Dib, MO); HFB = Hollywood Fault bend; HFS = Hollywood Fault scarps (DS, Dea); HF-1 = Hollywood Fault density contrast (buried, Hea); HF-4 = Hollywood Fault branch (Dib); HLF = Hollister Fault (Dib); IA = Inglewood Anticline (buried, on Pliocene Vickers Zone, Wri); IF = Inglewood Fault (surface, Cas); LCF = Las Cienegas Fault (buried, Wri); LP = Lar Pluton (Neu, Dib); MPM = Macarthur Park Monocline (informal, Oea); NP = Nichols Pluton (Dib); NSLF = North Salt Lake Fault (buried, trace at -500 m, Tea); PF = Potrero Fault (cas); RHA = Repetto Hills Anticline (informal, Oea); SLF-1 = Salt Lake Fault density contrast (buried, Hea); SMFS = Santa Monica Fault scarps (DS, Dea); SMMA = Santa Monica Mountains Anticline (informal, Dib); SVF = San Vicente Fault (buried, trace at -1500 m, Tea); TF = Tije Fault (Tie, Cas); TUF = Tunnel Fault (Neu, Dib); WBHL = West Beverly Hills Lineament (DS); WA = Wilshire Arch (Tea); WTF = Whitley Terrace Fault (informal, Dib); 1-4 = numbered parasitic folds on Elysian Park Anticline (Oea); 5 = structure contour on Pliocene Vickers Zone (elevation - 2000 feet, Wri); 6 = structure contour on base of Early Pliocene Repetto strata (elev - 2500 ft, Tea); 7 = fault in well at elev -7,000 ft (Tea, Fig 4g); 8,9 = parasitic? folds (Hoo); 10 = Century City Metro study area (PB, see Plate 6)

source abbreviations:
BL = Bullard & Lettis (1993); Cas = Castle (1960); Dea = Dolan et al. (1997, 2000a); Dib = Dibblee (1991, 1992); DS = Dolan & Sieh (1992); Hea = Hildenbrand et al. (2001); Hoo = Hoots (1931); Hum = Hummon et al. (1994); MO = Meigs & Oskin (2004); Neu = Neuberger (1953); Oea = Oskin et al. (2000); PB = Parsons Brinckerhoff (2011); Tea = Tsutsumi et al. (2001); Tie = Tije (1926); Wri = Wright (1987, 1991)

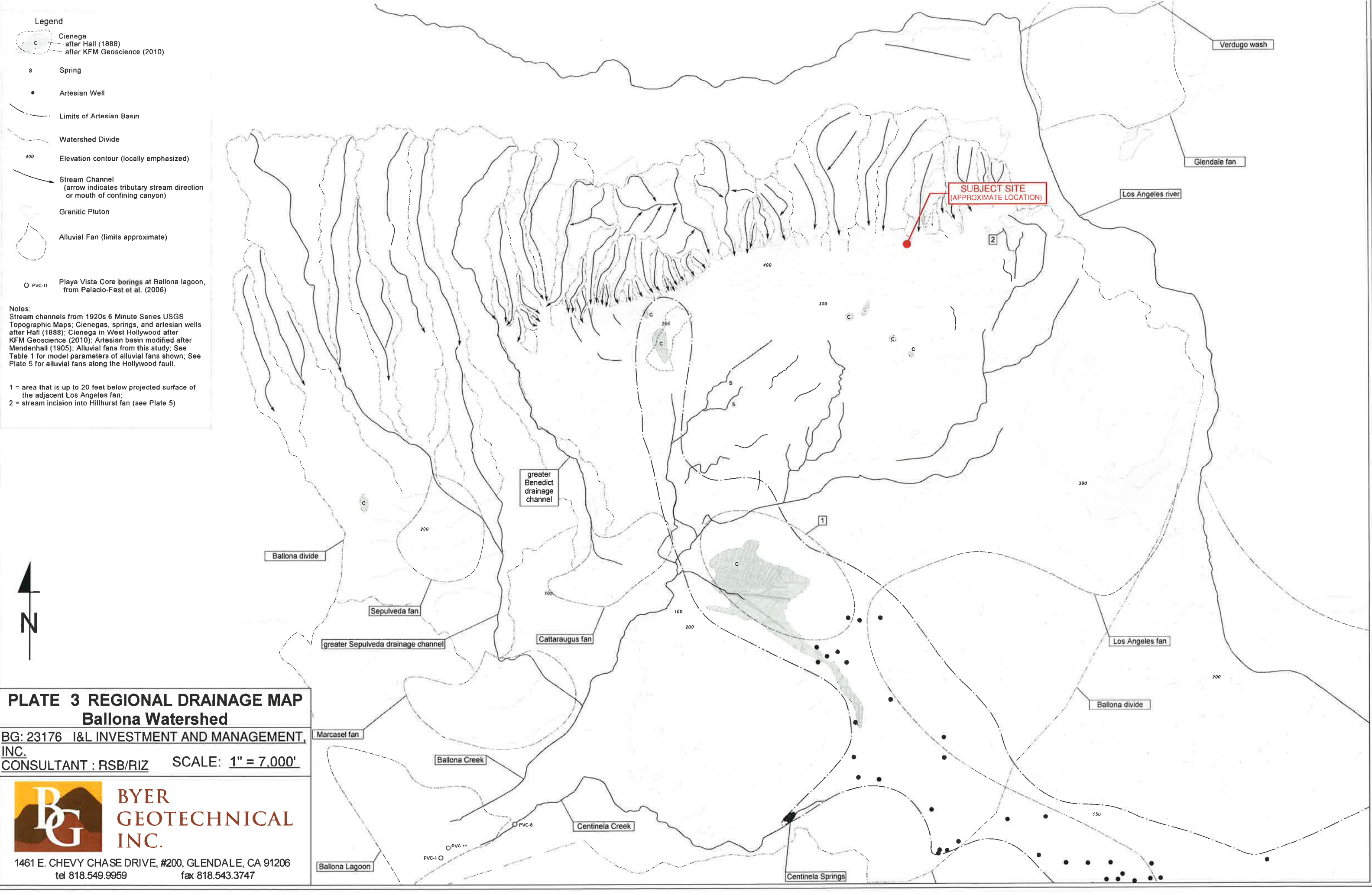
PLATE 2 REGIONAL STRUCTURE MAP

BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.
CONSULTANT : RSB/RIZ SCALE: 1" = 7,000'



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Legend

- c Cienega
after Hall (1888)
after KFM Geoscience (2010)
- s Spring
- Artesian Well
- Limits of Artesian Basin
- Watershed Divide
- 400 Elevation contour (locally emphasized)
- Stream Channel
(arrow indicates tributary stream direction
or mouth of confining canyon)
- Granitic Pluton
- Alluvial Fan (limits approximate)
- PVC-11 Playa Vista Core borings at Ballona lagoon,
from Palacio-Fest et al. (2006)

Notes:
Stream channels from 1920s 6 Minute Series USGS
Topographic Maps; Cienegas, springs, and artesian wells
after Hall (1888); Cienega in West Hollywood after
KFM Geoscience (2010); Artesian basin modified after
Mendenhall (1905); Alluvial fans from this study; See
Table 1 for model parameters of alluvial fans shown; See
Plate 5 for alluvial fans along the Hollywood fault.

1 = area that is up to 20 feet below projected surface of
the adjacent Los Angeles fan;
2 = stream incision into Hillhurst fan (see Plate 5)

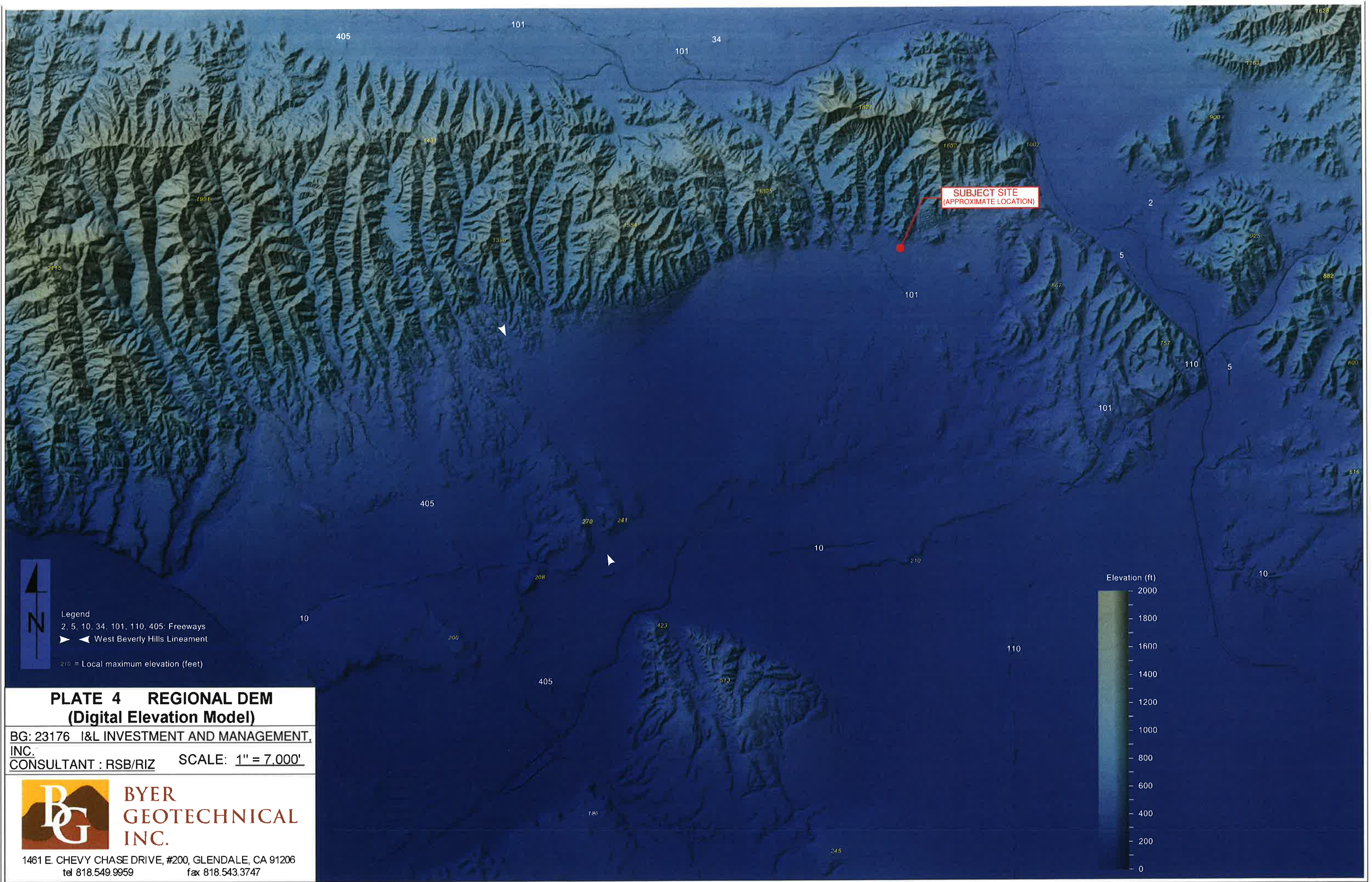
PLATE 3 REGIONAL DRAINAGE MAP
Ballona Watershed

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CONSULTANT : RSB/RIZ SCALE: 1" = 7,000'



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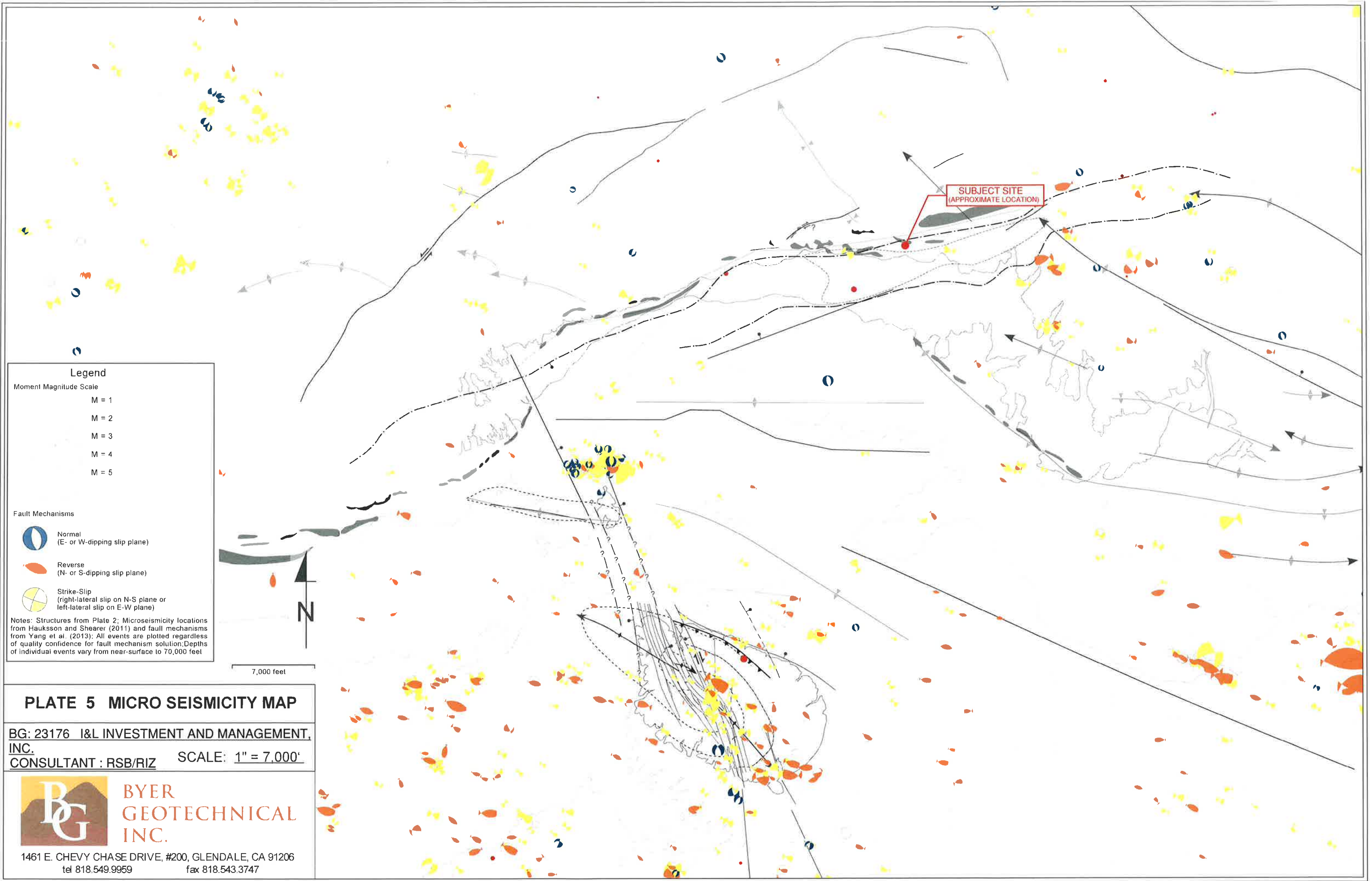


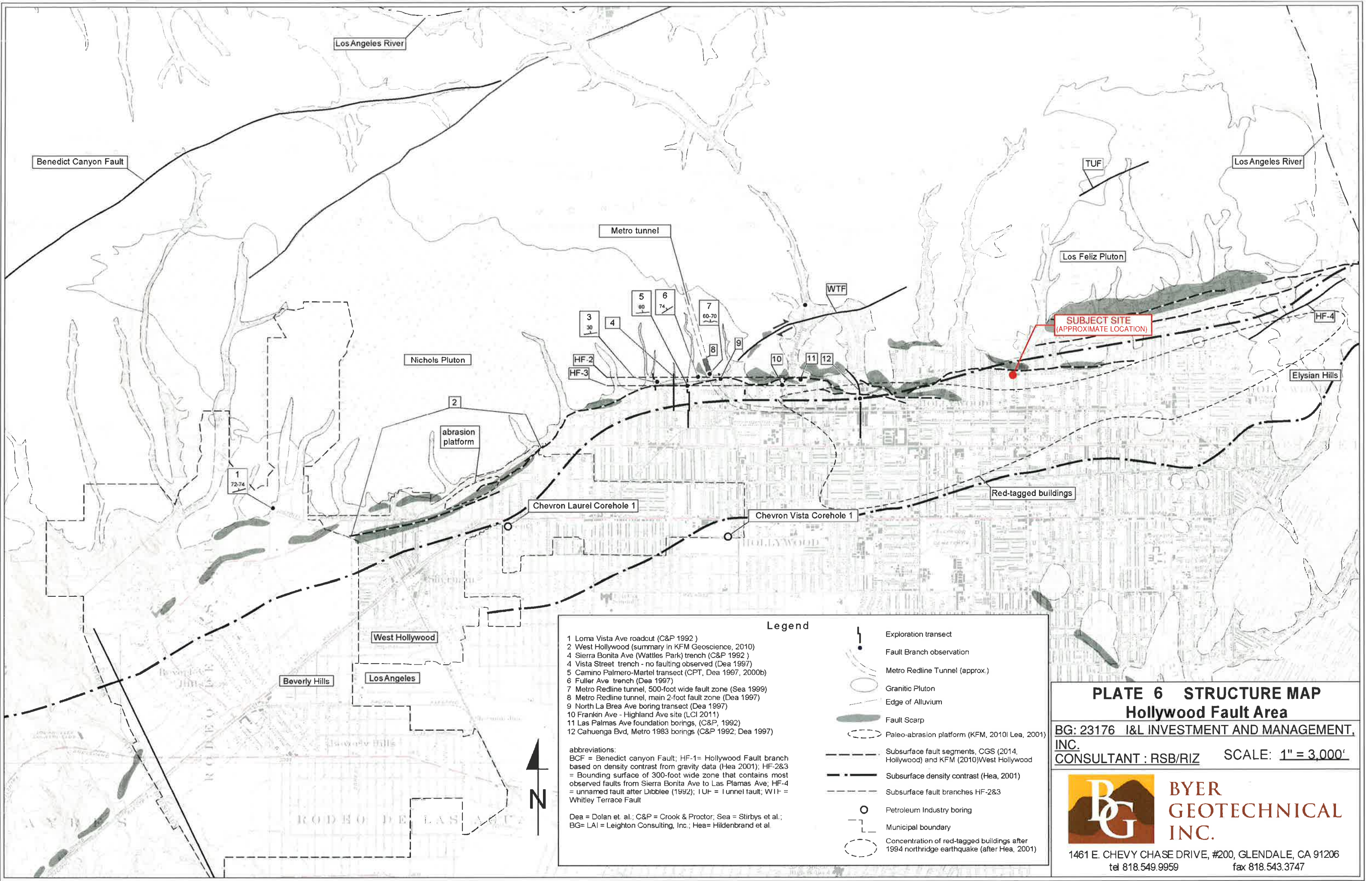
PLATE 5 MICRO SEISMICITY MAP

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Legend

1 Loma Vista Ave roadcut (C&P 1992)
2 West Hollywood (summary in KFM Geoscience, 2010)
4 Sierra Bonita Ave (Wattles Park) trench (C&P 1992)
4 Vista Street trench - no faulting observed (Dea 1997)
5 Camino Palmero-Martel transect (CPT, Dea 1997, 2000b)
6 Fuller Ave trench (Dea 1997)
7 Metro Redline tunnel, 500-foot wide fault zone (Sea 1999)
8 Metro Redline tunnel, main 2-foot fault zone (Dea 1997)
9 North La Brea Ave boring transect (Dea 1997)
10 Franklin Ave - Highland Ave site (LCI 2011)
11 Las Palmas Ave foundation borings, (C&P, 1992)
12 Cahuenga Blvd, Metro 1983 borings (C&P 1992; Dea 1997)

abbreviations:
BCF = Benedict canyon Fault; HF-1= Hollywood Fault branch based on density contrast from gravity data (Hea 2001); HF-2&3 = Bounding surface of 300-foot wide zone that contains most observed faults from Sierra Bonita Ave to Las Palmas Ave; HF-4 = unnamed fault after Dibblee (1992); IUF = Tunnel fault; WTF = Whitley Terrace Fault

Dea = Dolan et. al.; C&P = Crook & Proctor; Sea = Stirbys et al.; BG= LAI = Leighton Consulting, Inc.; Hea= Hildenbrand et al.

Exploration transect

Fault Branch observation

Metro Redline Tunnel (approx.)

Granitic Pluton

Edge of Alluvium

Fault Scarp

Paleo-abrasion platform (KFM, 2010| Lea, 2001)

Subsurface fault segments, CGS (2014, Hollywood) and KFM (2010)|West Hollywood

Subsurface density contrast (Hea, 2001)

Subsurface fault branches HF-2&3

Petroleum Industry boring

Municipal boundary

Concentration of red-tagged buildings after 1994 northridge earthquake (after Hea, 2001)

PLATE 6 STRUCTURE MAP
Hollywood Fault Area

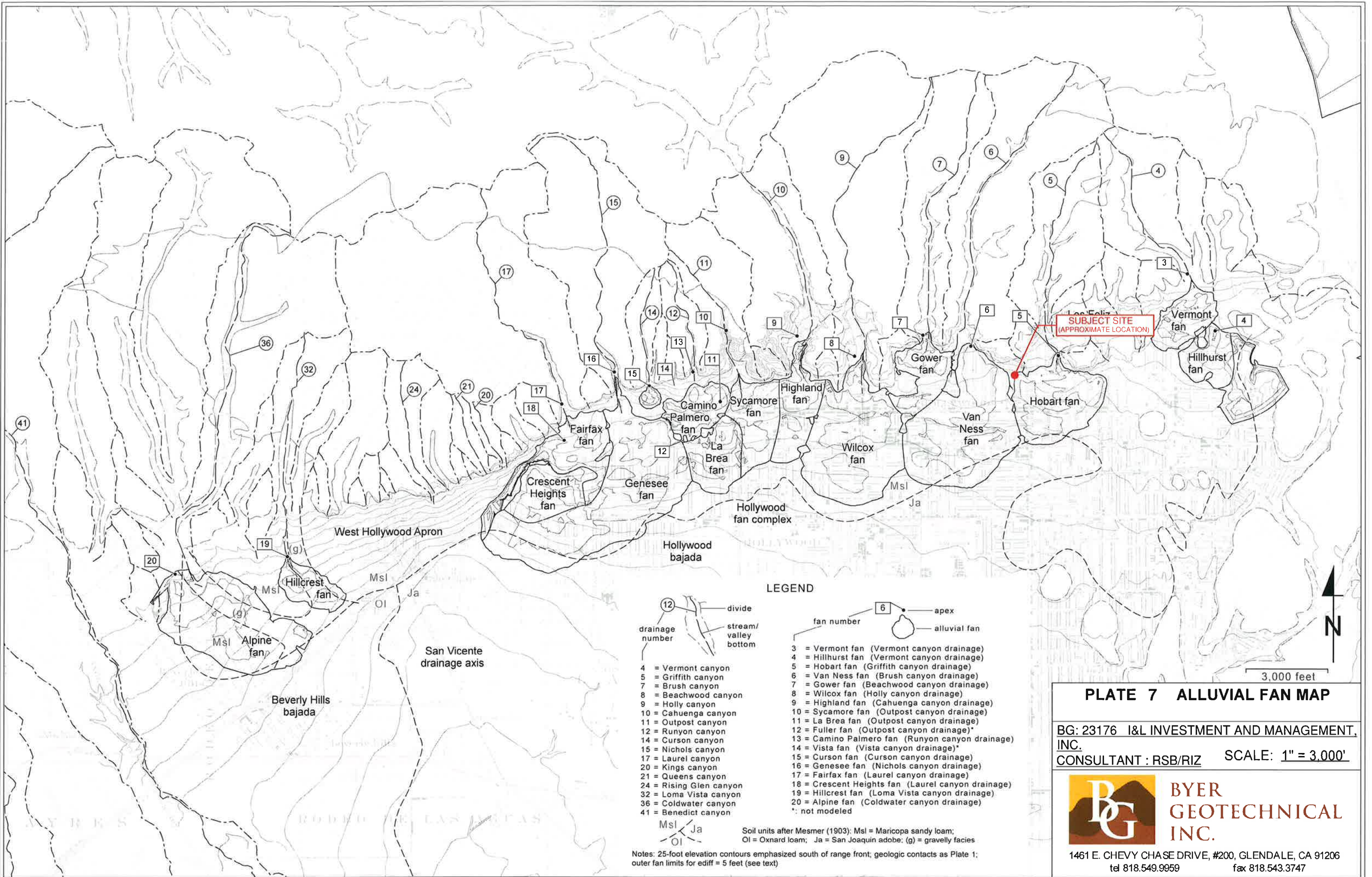
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CONSULTANT : RSB/RIZ

SCALE: 1" = 3,000'

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LEGEND

- drainage number

12 = divide

stream/valley bottom
- fan number

apex

alluvial fan
- 4 = Vermont canyon

5 = Griffith canyon

7 = Brush canyon

8 = Beachwood canyon

9 = Holly canyon

10 = Cahuenga canyon

11 = Outpost canyon

12 = Runyon canyon

14 = Curson canyon

15 = Nichols canyon

17 = Laurel canyon

20 = Kings canyon

21 = Queens canyon

24 = Rising Glen canyon

32 = Loma Vista canyon

36 = Coldwater canyon

41 = Benedict canyon
- 3 = Vermont fan (Vermont canyon drainage)

4 = Hillhurst fan (Vermont canyon drainage)

5 = Hobart fan (Griffith canyon drainage)

6 = Van Ness fan (Brush canyon drainage)

7 = Gower fan (Beachwood canyon drainage)

8 = Wilcox fan (Holly canyon drainage)

9 = Highland fan (Cahuenga canyon drainage)

10 = Sycamore fan (Outpost canyon drainage)

11 = La Brea fan (Outpost canyon drainage)

12 = Fuller fan (Outpost canyon drainage)*

13 = Camino Palmero fan (Runyon canyon drainage)

14 = Vista fan (Vista canyon drainage)*

15 = Curson fan (Curson canyon drainage)

16 = Genesee fan (Nichols canyon drainage)

17 = Fairfax fan (Laurel canyon drainage)

18 = Crescent Heights fan (Laurel canyon drainage)

19 = Hillcrest fan (Loma Vista canyon drainage)

20 = Alpine fan (Coldwater canyon drainage)

*: not modeled

Soil units after Mesmer (1903): Msl = Maricopa sandy loam; OI = Oxnard loam; Ja = San Joaquin adobe; (g) = gravelly facies

Notes: 25-foot elevation contours emphasized south of range front; geologic contacts as Plate 1; outer fan limits for ediff = 5 feet (see text)

PLATE 7 ALLUVIAL FAN MAP

BG: 23176 I&L INVESTMENT AND MANAGEMENT, INC.

CONSULTANT : RSB/RIZ

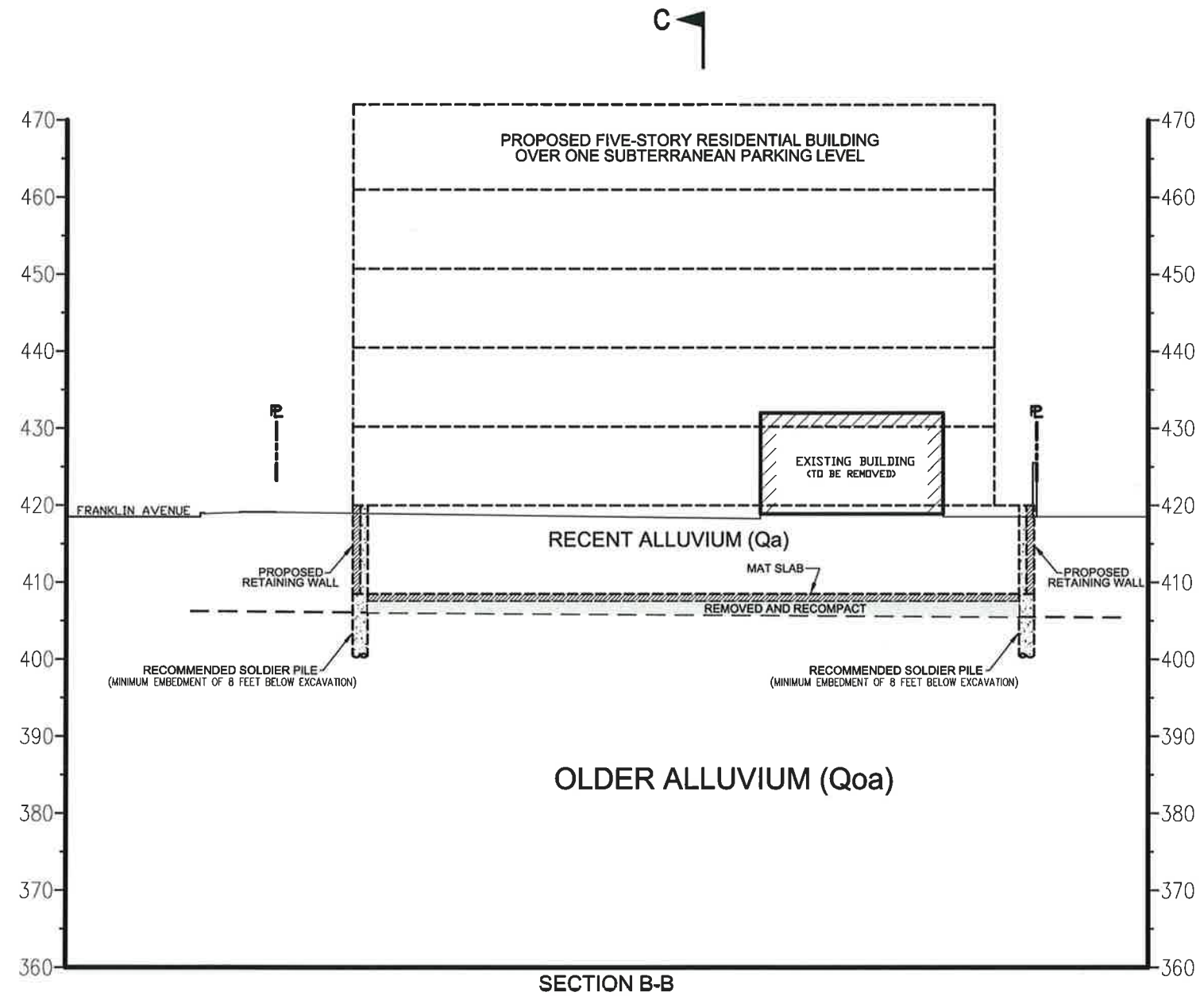
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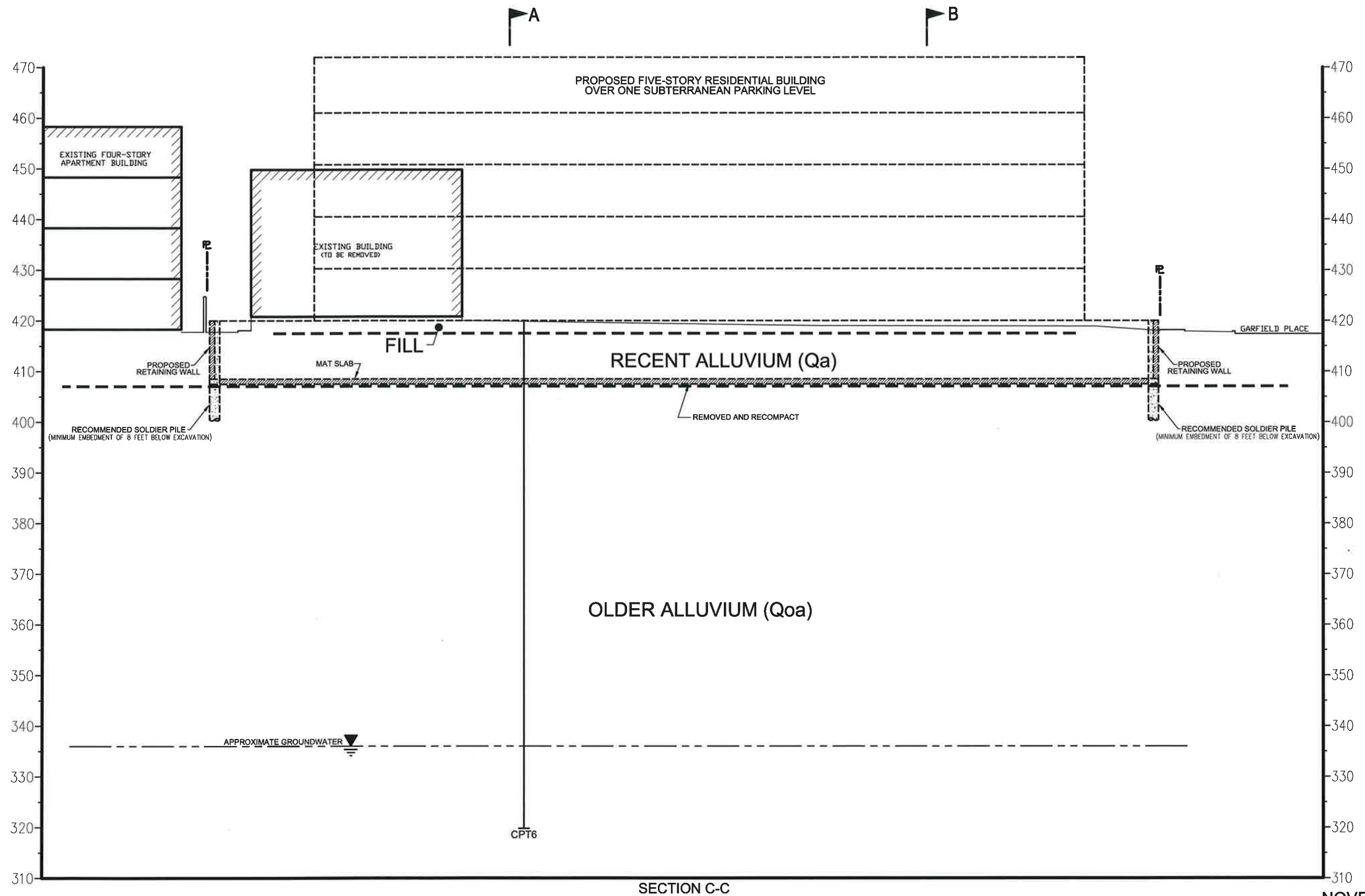
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SECTION B	
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DRAWN BY : AS	



SECTION C-C

NOVEMBER 30, 2020

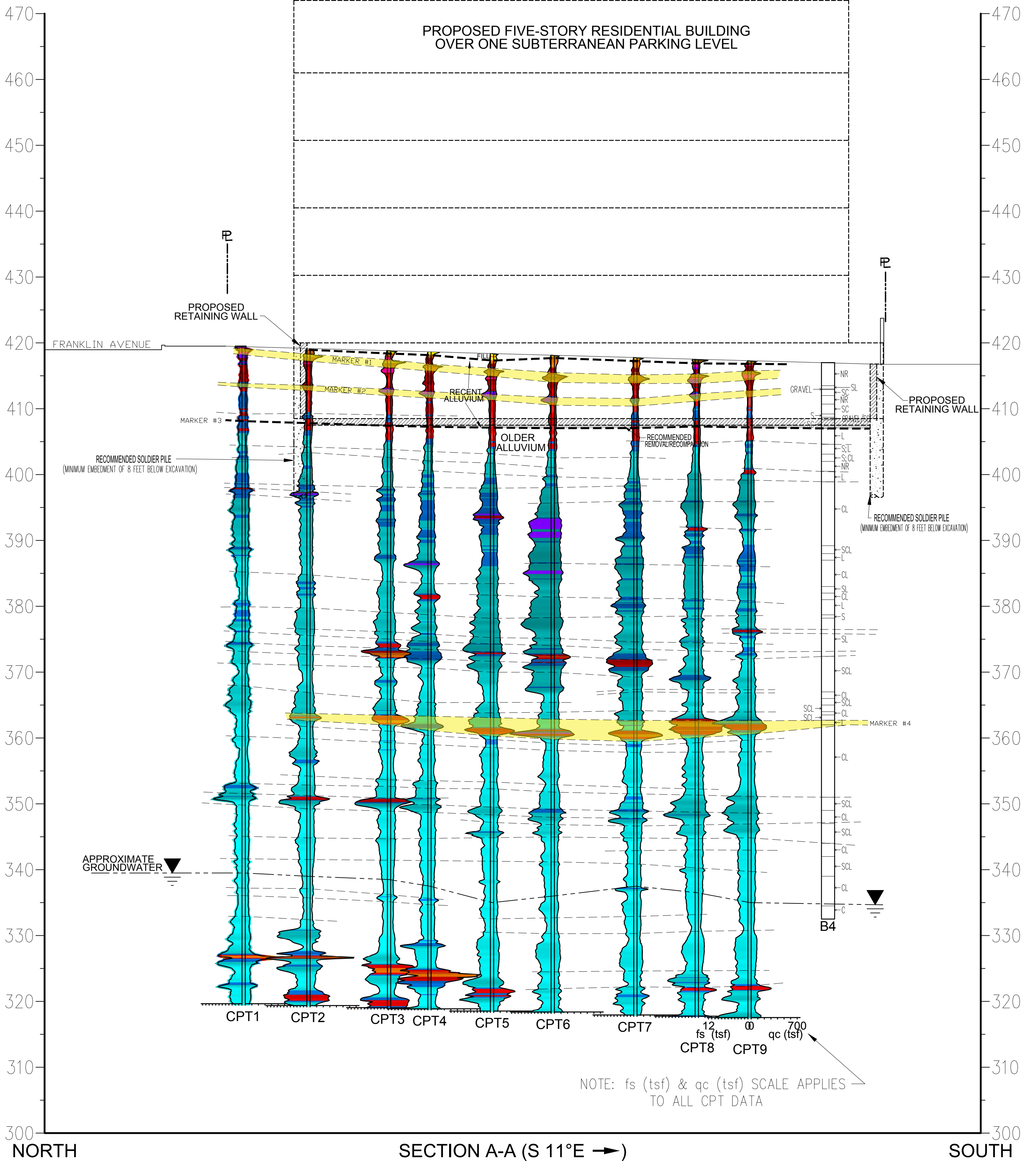
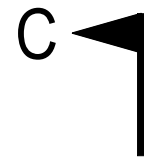


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SECTION C		
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CONSULTANT : RIZ		SCALE: 1" = 20'
DRAWN BY : AS		

CAD: F:\F:\S\1\andP\projects\23176 IM Investment Franklin Ave Hollywood\DWG\GFOUMP 01212020.dwg
 PILOT DATE/TIME: 11/30/2020 3:05pm



NOTE: fs (tsf) & qc (tsf) SCALE APPLIES TO ALL CPT DATA

NOVEMBER 30, 2020



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SECTION A

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CONSULTANT: RIZ

DRAWN BY: AS

SCALE: 1" = 10'