Appendix B

Geotechnical Investigation

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED REGAL CINEMAS
WESTFIELD NORTH COUNTY SHOPPING CENTER
ESCONDIDO, CALIFORNIA

Prepared for:

WESTFIELD, LLC
2049 Century Park East, 41st Floor
Century City, California 90067

Project No. 601586-008

August 7, 2015

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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Westfield, LLC
2049 Century Park East, 41st Floor
Century City, California 90067

Attention: Ms. Hannah Hieu

Subject: Preliminary Geotechnical Investigation
Proposed Regal Cinemas
Westfield North County Shopping Center
Escondido, California

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) has conducted a preliminary geotechnical investigation for the proposed Regal Cinemas project within Westfield North County Shopping Center, in Escondido, California. This report presents the results of our field investigation activities, review of previous reports, geotechnical analysis, and provides our conclusions and recommendations for the proposed improvements.

Based on the result of our preliminary geotechnical investigation, the proposed project is considered feasible from a geotechnical standpoint provided our recommendations are implemented in the design and construction of the project. If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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TABLE OF CONTENTS

Leighton

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS

Appendices

1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the proposed Regal Cinemas within the Westfield North County Shopping Center, Escondido, California (Figure 1). The purpose of our investigation was to identify and evaluate the existing geotechnical conditions present at the site and to provide conclusions and recommendations relative to the proposed development. Our scope of services included:

- Review of existing project geotechnical reports, aerial photographs, and other geologic documents and maps. References are cited in Appendix A.

- Our subsurface investigation consisted of four (4) small-diameter borings and five (5) cone penetrations test (CPT) soundings in order to obtain site-specific subsurface information for the design of the site improvements. The logs of the small-diameter borings and CPT soundings are presented in Appendix B.

- Review of previous studies and as-graded reports.

- Geotechnical evaluation of geotechnical data accumulated during our investigation including seismic, liquefaction, and settlement analysis.

- Preparation of this report presenting our findings, conclusions, and recommendations relative to the proposed development with respect to grading and foundation considerations.

1.2 Site Location and Description

The Westfield North County Shopping Center expansion project is bound by Interstate 15 to the west, Via Rancho Parkway to the south, and Beethoven Drive to the north and east (Figure 1). The existing shopping center development or mall consists of 1,270,000 square feet of enclosed retail shops and department stores with three levels. The mall is centrally located and surrounded by parking lots.

Specifically, the Regal Cinema site is located within the existing asphalt-concrete parking lot between the eastern-central mall building and Beethoven Drive. Elevations across the site range from approximately 355 feet mean sea level (msl) in the west to approximately 343 feet msl in the eastern portion of the site.

Site Coordinates:
Latitude: N33.0720°
Longitude: W117.06482°

1.3 Proposed Development

It is our understanding that the proposed regal cinema building will consist of ten auditorium rooms, lobbies, restrooms, storage/work rooms, and associated underground utilities. Proposed grades are generally anticipated to be within 1 to 3 feet of the existing site grades. We anticipate the cinema construction will consist exterior masonry bearing walls, interior metal stud wall systems and a metal roof truss system.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Subsurface Field Investigation

The purpose of our subsurface field investigation was to evaluate the physical characteristics of the onsite soils and assess the depth to competent material within the limits of the proposed improvements. The subsurface investigation also provided representative samples for laboratory testing. Prior to executing the subsurface investigation, underground Service Alert (USA) was contacted to coordinate the location and identification of nearby underground utilities.

Our subsurface investigation consisted of advancing four (4) small-diameter borings and five (5) CPT soundings to depths ranging from 5 to 55 feet below the existing ground surface (bgs). The approximate locations of the explorations are presented on our Geotechnical Map (Figure 2). Logs of the borings and CPT soundings are presented in Appendix B.

The CPT soundings were performed by Kehoe Testing and Engineering and borings were logged by a geologist from our firm. The subsurface materials were visually classified in accordance with the Unified Soil Classification System. Relatively undisturbed soil samples were obtained from the exploratory borings with a split-barrel ring sampler (2.4-inches inside diameter and 3-inches outside diameter. In addition, Standard Penetration Tests (SPT) was performed in accordance with ASTM Test Method D1586 using a 140-pound automatic-trip hammer free falling 30-inches collective soil samples were obtained for laboratory testing. Following logging and sampling, the borings were backfilled with bentonite grout in accordance with County of San Diego Department of Environmental Health (DEH) guidelines.

Laboratory testing was performed on representative samples to evaluate the expansion potential, grain size distribution and plasticity, shear strength, compressibility, maximum density, and chemical characteristics of the subsurface soils. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

2.2 Previous Geotechnical Reports

Previous geotechnical studies were performed by Leighton and Associates, Inc. (Leighton, 1979, 1982, 1983a, 1984i), for the existing Westfield North County Shopping Center. Previously performed borings include the excavation of approximately 91 small-diameter borings and 5 seismic refraction traverses. Note that the previous boring excavations used alternative number sequences to identify individual investigations (i.e., 1 through 62 for 1979 and 1982 and 101 through 129 for 1983a and 1984). The borings were located in alluvial areas, fill areas, and within the general vicinity of the structural improvements of existing North County Shopping Mall. Two recent borings for the 24 Hour Fitness were utilized for this geotechnical study.

The mall was originally sheet graded between January and September 1984, and Leighton provided continuous observation of the excavation of undocumented fill, alluvial deposits, and placement and compaction of engineered fill. Fills up to 15 feet and cuts up to 14 feet were performed during sheet grading of the site. Deeper localized fills associated with site utilities, subdrain systems, and basement backfill should be anticipated. During mass grading of the site, subsurface drains were installed in the building areas to control and lower the ground water table. In addition, during construction of reinforced box culvert a gravel blanket drain was constructed below the box culvert to serve as a subsurface drain to lower site ground water. The fore mentioned subdrains discharge into the gravel blanket of the box culvert. The anchor department stores (i.e. Sears, J.C. Penney, Nordstrom, Macy's (previously Broadway and May Co.), and Robinsons), were individually graded pads under observation and testing of Leighton. The department store grading included fine grading of the pads, retaining wall backfill, interior trench excavation and compaction, and placement of subsurface drains (see Appendix A).

As described in the previous geotechnical reports of the mall prepared by Leighton and Associates (1979 through 1985), the site has experienced several phases of documented grading. However, it should be noted that remedial grading of the underlying weathered residual soil and potentially compressible alluvial soils was not performed in the parking lot areas prior to fill placement.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Geologic Setting

The site is located in the coastal section of the Peninsular Range Province, a geomorphic province with a long and active geologic history throughout Southern California. Throughout the last 54 million years, the area known as “San Diego Embayment” has undergone several episodes of marine inundation and subsequent marine regression, resulting in the deposition of a thick sequence of marine and nonmarine sedimentary rocks on the basement rock of the Southern California batholith. The site is located adjacent to the northern boundary of the San Diego Embayment where a transition between sedimentary rocks and granitic basement rocks occur. Primarily the site is underlain by the cretaceous-aged granitic rock of the Southern California Batholith. Erosion and regional uplift created the alluvial valley and ridges of the area.

3.2 Site-Specific Geology

Based on our subsurface exploration, aerial photographic analysis, and review of pertinent geologic literature, maps and site reports, the geologic bedrock unit underlying the site consist of Cretaceous-aged Granitics. This unit is inturn overlain by surficial units consisting of artificial fill soils, alluvium and residual soil. A brief description of the geologic units encountered at the site are presented below. The approximate aerial distributions of those units are shown on the Geotechnical Map (Figure 1).

3.2.1 Artificial Fill (Map Symbol – Af)

Artificial fill was encountered in a majority of the borings performed across the site to depths ranging between 4 to 12 feet bgs. Fill soils generally consisted of dark brown to dark gray, moist, medium dense fine- to medium- grained silty sand with varying clay content. Observation and testing of the placement and compaction of these engineered fill soils was previously performed by Leighton and documented in the as-graded reports for the site (see Appendix A).

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3.2.2 Quaternary - Aged Alluvium (Map Symbol – Qal)

Alluvial deposits were encountered underlying the compacted fill soils at the site to a maximum observed depth of approximately 50 feet bgs. This is an area where an alluvial filled channel existed prior to the original site grading. The alluvium consists of brown to dark brown, loose to medium dense, wet, fine grained silty sands and sandy silts with varying clay content. The alluvial soils encountered in the CPTs were defined by the Soil Behavior Types (SBT) of predominately silts and clays; although these soil types were not as prevalently observed in the hollow-stem auger borings, the hollow stem borings were sampled at a more coarsely spaced interval than the nearly continuous interpretation by CPT method. In addition, alluvial stratigraphy by nature is discontinuous and varies by locality. It should be noted that deeper alluvial soils are likely to occur in areas that were not evaluated during this investigation.

3.2.3 Residual Soil (Mapped with – Kgr)

Residual soil was encountered in borings B-2 and B-3 beneath the alluvium and generally consisted of red-brown, medium dense to dense silty sand, medium grained sand that veneers the underlying Cretaceous granite. The residual soil is derived from the underlying granitics and varies in density as a result of the degree of in-place physical and chemical weathering the granitic rock has experienced. This type of weathering is common in crystalline rocks as, mineral concentrations, joints spacing and groundwater allow localized portions of granitic rock to weather more severely than other portions. For the purpose of this report, residual soils were mapped as Cretaceous-aged granite. Residual soils are generally considered suitable to support additional fill or structural loads.

3.2.4 Cretaceous-aged Granite (Map Symbol – Kgr)

The entire site is underlain at varying depths by bedrock material consisting of Cretaceous-aged granitic rock. As encountered during our current and previous site investigations, this unit generally excavated to gray-brown, slightly moist, dense to very dense, fine to coarse grained sand. This unit is considered to be moderately decomposed with possibly unweathered and hard rocks (core stones) within the decomposed granitic mass. Granitic rock is considered suitable to support additional or structural loads.

3.3 Ground Water

Ground water was encountered at an approximate depth of 15 to 16 feet below the existing ground surface (bgs) our investigation and is consistent in our previous geotechnical borings throughout the Westfield north county shopping center. Materials below this elevation are considered to be saturated.

3.4 Landslides

Our investigation was limited primarily to the existing flat graded existing parking lot area. No ancient landslides or other slope instability problems have been mapped on the subject site. In addition, no evidence of landsliding was encountered during our site investigation. Based on our review of geotechnical literature and our observations, landsliding is not a constraint to the currently proposed improvements.

3.5 Engineering Characteristics of On-Site Soil

Based on the results of our geotechnical investigation, the current and previous laboratory testing of representative on-site soils (Appendix C), and our professional experience on adjacent sites with similar soils, the engineering characteristics of the on-site soils are discussed below.

3.5.1 Soil Compressibility and Collapse Potential

The artificial fill soils are considered to have low compressibility and low collapse potential. The underlying alluvial materials encountered at the site generally exhibits low blow counts and localized porosity indicating an unconsolidated character, which is subject to potential settlement from heavy concentrated building loads or large amounts of additional fill.

3.5.2 Expansive Soils

Based upon our laboratory testing, review of previous geotechnical reports, borings and CPT logs, and field observations performed for the preparation of this report, the near surface fill soils (within the upper 10 to 15 feet) are expected to generally possess a low to medium expansion potential.

3.5.3 Soil Corrosivity

In general, several agencies list a set of conditions for a site to be considered a corrosive environment. The California Department of Transportation (Caltrans) considers a site to exhibit a corrosive environment if one or more of the following conditions exist: chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or pH is 5.5 or less (Caltrans, 2012).

In addition, the American Association of State Highway and Transportation Officials (AASHTO) considers the following water conditions to be considered as indicative of potential pile deterioration or a potential corrosion situation exists where: chloride concentration is greater than 500 ppm, sulfate concentration is greater than 500 ppm proximity, there is potential effluence of mine or industrial runoff, high organic content, a pH of less than 5.5, marine borers (biological), piles are exposed to wet/dry cycles. When chemical wastes are suspected, a full chemical analysis of soil and groundwater samples shall be considered (AASHTO, 2012).

A preliminary corrosive soil screening for the on-site materials was performed to evaluate their potential effect on concrete and ferrous metals. Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content.

The sample tested had a measured pH of 8.42, and a measured minimum electrical resistivity of 1177 ohm-cm. Test results also indicated that the sample had a chloride content of 86 ppm, and a soluble sulfate content of 240 ppm.

To summarize preliminary on-site soil corrosion potential, the following table provides the results of our corrosion analysis alongside conditions where Caltrans and AASHTO consider an environment to be corrosive. Please note the summary is limited to ONLY the listed conditions expressed in the table. Not all conditions considered to be constituent, part or whole, to corrosion potential were considered during our exploration and as a result our assessment of onsite corrosion potential is considered preliminary. A corrosion engineer should be consulted for further analysis and recommendations.

Table 1
Summary of Corrosion Potential

Constituent	Site Condition	Caltrans	AASHTO
Chloride Content (ppm)	86	500 or greater	500 or greater
Sulfate Content (ppm)	240	2,000 or greater	500 or greater
pH	8.42	5.5 or less	5.5 or less

Preliminary corrosion analysis indicate the site soils are generally considered not to host a corrosive environment; however, site soils in contact with buried uncoated ferrous metals is considered to pose a corrosion hazard and should be mitigated.

3.5.4 Excavation Characteristics

It is anticipated the on-site soils can be excavated with conventional heavy-duty construction equipment. Localized loose sand zones, if encountered, may require special excavation techniques (i.e. flattening of slopes) to prevent collapsing of the excavation.

3.5.5 Infiltration

We did not evaluate the site for infiltration of storm water; however, it should be noted that generally, a percolation rate less than 120 mpi is considered necessary to consider a site suitable for onsite surface infiltration of storm water. The on-site artificial fill, that consist of mixture of soils ranging from silty sands to clays with permeable and impermeable layers, can transmit and perch groundwater in unpredictable ways. In addition, the site exhibits a shallow groundwater condition. Therefore, Low

Impact Development (LID) measures may impact down gradient improvements and the use of some LID measures may not be appropriate for this project. Infiltration and Bioretention Stormwater Systems design should be reviewed by geotechnical consultant.

4.0 FAULTING AND SEISMICITY

4.1 Faulting

Our discussion of faults on the site is prefaced with a discussion of California legislation and policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault which has had surface displacement within Holocene time (about the last 11,000 years). The state geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years). This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972 and most recently revised in 2007 (Bryant and Hart, 2007). The intent of this act is to assure that unwise urban development and certain habitable structures do not occur across the traces of active faults. The subject site is not included within any Earthquake Fault Zones as created by the Alquist-Priolo Act.

San Diego County, like the rest of Southern California, is seismically active as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional fault zones such as the San Andreas, San Jacinto and Elsinore Faults Zones, as well as along less active faults such as the Rose Canyon Fault Zone.

Our review of geologic literature pertaining to the site and general vicinity indicates that there are no known major or active faults on the site (Jennings, 1994). Evidence for faulting was not encountered during our field investigation. The nearest known active fault is the Rose Canyon Fault Zone located approximately 15.5 miles (24.9 km) west of the site. Because of the lack of known active faults on the site, the potential for surface rupture at the site is considered low.

4.2 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 2 are the risk-targeted spectral acceleration parameters for the project determined in accordance with the 2013 California Building Code (CBSC, 2013) and the USGS Worldwide Seismic Design Values tool (Version 3.1.0).

Table 2 CBC Mapped Spectral Acceleration Parameters			
Site Class	D		
Site Coefficients	F_a	=	1.105
	F_v	=	1.636
Mapped MCE_R Spectral Accelerations	S_S	=	0.988g
	S_1	=	0.382g
Site Modified MCE_R Spectral Accelerations	S_{MS}	=	1.092g
	S_{M1}	=	0.625g
Design Spectral Accelerations	S_{DS}	=	0.728g
	S_{D1}	=	0.417g

Utilizing ASCE Standard 7-10, in accordance with Section 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.367g for the site. For a Site Class D, the F_{PGA} is 1.133 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.416g for the site.

4.3 Secondary Seismic Hazards

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake include shallow ground rupture, soil liquefaction and dynamic settlement, lateral spreading, seiches and tsunamis. These secondary effects of seismic shaking are discussed in the following sections.

4.3.1 Shallow Ground Rupture

No active faults are mapped crossing the site, and the site is not located within a mapped Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Shallow ground rupture due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

4.3.2 Liquefaction and Lateral Spreading

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils underlain by a near surface ground water table are most susceptible to liquefaction, while the most clayey materials are not susceptible to liquefaction. Liquefaction is characterized by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over liquefied layers. Where sloping ground conditions are present, liquefaction-induced instability can result.

For consideration in liquefaction analysis, and based on deaggregation of the Maximum Considered Earthquake event, a magnitude M6.80 is associated with the Design Earthquake Ground Motion (i.e. peak ground acceleration of 0.42g) was used. Liquefaction analysis was performed utilizing the compter program Cliq, Version 1.7.6, with a NCEER method (Robertson & Wride, 1998, 2009, and Youd, 2001).

Based on the results of geotechnical analyses, it is our opinion that the some of the alluvial deposits underlying the site are susceptible to dynamic and post-liquefaction settlements as a result of ground shaking at the design ground motions. The estimated total dynamic settlement for the existing site conditions is anticipated to range from less than 0.5 inch in the western and northern portions of the site (CPT-4 and CPT-5), and up to 3.75 inches in the centeral and eastern portions of the site (CPT-1, CPT-2 and CPT-3) based on the the applied design earthquake ground motion. Currently, the estimated differential dynamic settlement is anticipated to be on the order of 2 inches or less considering an overall evaluation of the site.

4.3.3 Tsunamis and Seiches

Based on the distance between the site and large, open bodies of water, and the elevation of the site with respect to sea level, the possibility of seiches and/or tsunamis is considered to be low.

5.0 CONCLUSIONS

Based on the results of our geotechnical study of the site, it is our opinion that the proposed improvements are feasible from a geotechnical viewpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications. The following is a summary of the significant geotechnical factors that we expect may affect development of the site.

- Our review of the geologic literature indicates there are no known major or active faults on or in the immediate vicinity of the site. In addition, evidence of active faulting was not encountered within the site during our field investigation or the prior grading operations. Because of the lack of known active faults on the site, the potential for surface rupture at the site is considered low.

- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The nearest known active fault is the Rose Canyon Fault Zone, which is located approximately 15.5 miles (24.9 km) west of the site.

- Based on our subsurface explorations and laboratory testing, the site is overlain by compacted fill soil; however, the existing near surface soils in their current condition is not suitable for support of settlement sensitive structures. Therefore, remedial grading (i.e., removal, moisture conditioning and recompaction) of the upper 2 to 3 feet of materials below existing grade or proposed grade, whichever is deeper, will be required prior to construction of the proposed improvements.

- The alluvial materials beneath the compacted fill, as encountered CPTs and borings, exhibits low blow counts and localized porosity indicating an unconsolidated character, which is subject to potential settlement from heavy concentrated building loads or large amounts of additional fill. Therefore, deep foundations, or ground improvement with conventional shallow foundations should be considered. Note that if additional fill is placed, surcharging loading and settlement monitoring should be performed, which may be 2- to 3-month period.

- Relatively thin and discontinuous layers of the alluvial materials beneath the compacted fill, as encountered CPTs, are considered susceptible to liquefaction at the design earthquake ground motion. Dynamic settlement will range from less than 0.5 to 3.75 inches, and 1.5 inches for total and differential, settlement, respectively. To mitigate adverse effects of differential settlement, deep foundations or ground improvement with conventional shallow foundations should be considered. Note that a specialty contractor with ground improvement experience and specialized equipment should be consulted for the design of the treatment area and zone(s). Also, additional CPT explorations may be warranted for a final design.

- Proposed grades are anticipated to be near or below the existing grades and deep foundation improvements are recommended. As a result, if grades are raised or alternative ground improvements are considered, revised recommendations may be warranted.

- The existing onsite soils appear to be suitable material for use as compacted fill provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension. Materials placed within 2 feet of the pad grade should possess an expansion index less than 90.

- Buried metal pipes and conduits in contact with on site soils have a potential for corrosion attack. A corrosion engineer should be consulted during the design and construction of improvements.

6.0 RECOMMENDATIONS

The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during earthwork operations and construction of the project, in order to confirm that our preliminary findings are representative for the site.

6.1 Earthwork

We anticipate that earthwork at the site will consist of remedial grading of the near-surface soils; grading of the building pad addition and associated improvements; utility construction; subgrade preparation in pavement areas; and foundation excavation. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those in Appendix E.

6.1.1 Site Preparation

If additional grading, such as fill placement, is planned on the site, the areas to receive structural fill, engineered structures, or hardscape should be cleared of surface and subsurface obstructions, including any existing debris and undocumented or loose fill soils, and stripped of vegetation. Removals should extend to competent documented fill soils and/or competent formational soils. Removed vegetation and debris should be properly disposed off site. Holes resulting from the removal of buried obstructions which extend below finish site grades should be replaced with suitable compacted fill material. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 12 inches, brought to above optimum moisture conditions, and recompacted to at least 90 percent relative compaction based on ASTM Test Method

D1557. If clayey soils that are more expansive (EI>90) are encountered, increased moisture and revised recommendations may be needed.

6.1.2 Excavations and Removals

Excavations of the upper onsite materials (i.e., fill and alluvium soils) may generally be accomplished with conventional heavy-duty earthwork equipment. In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back to if workers are to enter such excavations. Temporary sloping of excavations should be determined in the field by a "competent person" as defined by OSHA. For preliminary planning, sloping of excavations at 1:1 (horizontal to vertical) to a depth of 15 feet (or above ground water) may be assumed for soils and should remain stable for the period required to construct the utility, provided they are free of adverse geologic conditions or seeps. Note that excavations should not extend below a 2:1 plane extending down from existing footings unless properly designed by an engineer.

We recommend some removal and recompaction be performed across the site. In areas of improvements the grading should extend 3 feet beyond proposed additions and the minimum depth should be 2 feet below existing grades or improvement grades, whichever is deeper. The bottom of all removals should be proof rolled and uniformly compacted to 90% relative compaction prior to the placement of compacted fill soils.

Note that loose or soft fill soils associated with previous underground utilities, landscaping, and retail construction may be encountered and localized deeper removals may be required. The actual depth and extent of the required removals should be determined during grading operations by the geotechnical consultant. The removal bottom should then be scarified a minimum of 6 inches, moisture conditioned and compacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials (ASTM) Test Method D1557). Soil can then be placed to proposed finish or subgrade elevations. Expansion and sulfate testing should be performed on finish grade soils. Leighton should observe and test all fill placement during grading and observe footing excavations prior to concrete placement to confirm that the soil conditions are as anticipated.

6.1.3 Engineered Fill Placement and Compaction

The onsite existing fill soils are generally suitable for use as compacted fill provided they are free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension. All fill soils should be brought to 2 percent above optimum moisture conditions and compacted in uniform lifts to at least 90 percent relative compaction based on ASTM Test Method D1557. The upper 12 inches of subgrade and all aggregate base should be compacted to at least 95 percent beneath vehicular pavements. The optimum lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

Placement and compaction of fill should be performed in general accordance with the current City of Escondido grading ordinances, sound construction practice, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.1.4 Import Soils

Import soils, if necessary to bring the site up to the proposed grades, should be free of oversize material and debris. These soils should be granular and have an expansion index less than 50 (per ASTM Test Method D4829). Please contact this office for further evaluation of the import soils and/or borrow site prior to importation.

6.1.5 Ground Improvement

Stone columns, designed by specialty contractor with ground improvement experience, may be considered to mitigate potentially compressible soils that may settle under foundation loads and the effects of liquefaction and seismic settlement at the site. In general, we recommend that a site-specific ground improvement plan be drafted by the specialty contractor that will contain the location of stone columns including design diameter and spacing.

A field verification program using CPTs should also be used to estimate the post ground improvement settlement. CPTs should be used prior to placement of ground improvement and following ground improvement. If the total static and seismic settlement after ground improvement is determined to be greater than 1.5 inches, a mat foundation may be required. Otherwise conventional foundations may be used. Field verification procedures should be reviewed by and be acceptable to the Geotechnical Engineer. The approximate limits of ground improvement is depicted on Figure 2.

6.2 Foundation and Slab Considerations

Foundations should be designed in accordance with structural considerations and the following recommendations. The slab-on-grade recommendations assume that the soils encountered have a low to medium potential for expansion. Note that ground improvement, as discussed above, is required for the used of shallow conventional foundations.

Conventional Footings

For support of proposed building loads and site structures including retaining walls (less than 5 feet high), conventional spread and continuous footing may be used with ground improvement. Foundation may be designed for a maximum allowable bearing pressure of 3,500 pounds per square foot (psf). The footings should extend a minimum of 24 inches beneath the lowest adjacent finish grade. The allowable pressures may be increased by one-third when considering loads of short duration such as wind or seismic forces. The minimum recommended width of footings is 18 inches for continuous footings and 24 inches for square or round footings, if used.

The recommended allowable bearing capacity for conventional footings is based on a maximum allowable total and differential settlements of 1-inch and 3/4-inch, respectively. Since settlements are functions of footing size and contact bearing pressures, some differential settlement can be expected between adjacent columns, where large differential loading conditions exist. With increased footing depth to width ratios, differential settlement should be less. An additional post-liquefaction 1/2 inch over a distance of 25 feet should be allowed for in the design of the structure (i.e., an angular distortion of 1/600).

Deep Foundations

For deep foundations, we recommend the use of Cast-In-Drilled-Hole (CIDH) piles. For the analysis and development of the capacities of CIDH piles, the computer program SHAFT (Version 2012.7.3) produced by Ensoft, Inc. was used. As shown in Appendix F, the CIDH allowable capacity curves were developed for 18- to 48-inch diameter piles penetrating into dense formational material. Note that all piles need to be embedded at least 5 feet into dense formational material and observed by the geotechnical consultant. For tension or uplift capacity from the CIDH, allowable capacity curves are also shown in Appendix F.

It should be noted that the data presented on the design curves are based on the supporting capacity of the earth materials. Design considerations should also be given to the pile as a structural member. CIDH piles should be spaced at a minimum of three (3) pile diameters if group action capacity reductions are to be neglected. For piles constructed at 1.5 diameters center to center spacing, a 50 percent reduction in capacity should be taken along the affected section of the pile for each adjacent pile. Reduction values for intermediate spacing may be determined by interpolation. Anticipated settlement of ½ inch is anticipated for the proposed piles.

To resist lateral loads, if any, CIDH piles can be designed in accordance with Section 1807.3 of the 2013 CBC. For level ground conditions, we recommend lateral soil bearing pressures of 300 psf per foot of depth below the finish grade be used for determination of parameters S1 and S3, in the Non-constrained and Constrained design criteria, respectively. These values should be reduced by 50 percent to account for 2 to 1 downward sloping ground conditions, if applicable.

All pile installation should be performed under the observation of the geotechnical consultant and consistent with standard practice. Drilling equipment should be powerful enough to drill into the dense to very dense formational material to the design penetration depths. Once a pile excavation has been started, it shall be completed within 8 hours, which includes inspection, placement of the reinforcement, and placement of the concrete. Construction of piles should be sequenced such that the concrete of constructed piles are allowed to setup prior to construction of piles within 3 diameters.

Ground water should be anticipated in the pile excavations and should be considered in the development of a Contractor's Pile Installation work plan. If CIDH pile excavations are filled with water or drilling mud, concrete must be placed through a pipe extending to the bottom of the pile excavation.

Caving of friable, soft or loose soils may occur. Therefore, a starter casing should be used to protect the top of the borehole to mitigate caving conditions. In addition, the contractor should also be prepared to employ casing or other methods of advancing the drilled pile excavation (i.e., drilling mud) to mitigate caving. Use of casing should be at the contractor's discretion. If CIDH pile excavations become bell-shaped and cannot be advanced due to severe caving, the caved region may be filled with a sand/cement slurry and redrilled. Redrilling may continue when the slurry has reached suitable set and strength. In this case, it may be prudent to utilize casing or other special methods to facilitate continued drilling after the slurry has set.

6.3 Floor Slab Considerations

Slab-on-grade floors should be at least 5 inches thick and reinforced with a minimum of No. 3 rebars at 18 inches on center each way, placed at mid height in the slab. The slab should be underlain by 2-inch layer of clean sand (S.E. greater than 30). A moisture barrier (10-mil non-recycled plastic sheeting) should be placed below the sand layer if reduction of moisture vapor up through the concrete slab is desired (such as below equipment, living/office areas, etc.), which is in turn underlain by an additional 2-inches of clean sand. We recommend that the architect follow the guidance of ACI 302.2R-06 for design of the under slab moisture protection measures and development of construction specifications. We recommend control joints be provided across the slab at appropriate intervals as designed by the project architect.

Prior to placement of the vapor barrier, the upper 6-inches of subgrade soil should be moisture conditioned to a moisture content 2 percent above the laboratory optimum.

The potential for slab cracking may be further reduced by careful control of water/cement ratios. The contractor should take the appropriate precautions during the pouring of concrete in hot weather to minimize cracking of slabs. We recommend that a slip-sheet (or equivalent) be utilized above the concrete slab if crack-sensitive floor coverings are to be placed directly on the concrete slab. If heavy vehicle or equipment loading is proposed for the slabs, greater thickness and increased reinforcing may be required.

6.4 Retaining Wall Design

For design purposes, the following lateral earth pressure values in Table 3 for level or sloping backfill are recommended for retaining walls backfilled with very low to low expansion potential (Expansion Index less than 50).

Table 3
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	35	55
At-Rest	55	85
Passive	300 (maximum of 3 ksf)	150 (sloping down)

Retaining structures should be provided with a drainage system, as illustrated in Appendix E, to prevent buildup of hydrostatic pressure behind the wall. For sliding resistance, a friction coefficient of 0.35 may be used at the soil-concrete interface. The lateral passive resistance can be taken into account only if it is ensured that the soil against embedded structures will remain intact with time.

Retaining wall footings should have a minimum embedment of 18 inches below the adjacent lowest grade unless deeper footings are needed for other reasons.

If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform horizontal pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform horizontal pressure equal to 0.35q should be applied to the wall (where q is the surcharge pressure in psf). Wall backfill should be brought to optimum or above moisture content and compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend the previously discuss setback distance from the outside base of the footing to daylight.

To account for potential redistribution of forces during a seismic event, the subterranean walls should also be checked considering an additional seismic pressure distribution equal to $10H_T$ psf, where H_T equals the overall retained height in feet.

6.5 Surface Drainage and Erosion

Surface drainage should be controlled at all times. The proposed structures should have appropriate drainage systems to collect runoff. Positive surface drainage should be provided to direct surface water away from the structure toward suitable drainage facilities. In general, ponding of water should be avoided adjacent to the structure or pavements. Over-watering of the site should be avoided. Protective measures to mitigate excessive site erosion during construction should also be implemented in accordance with the latest City of Escondido grading ordinances.

6.6 Vehicular Pavements

The pavement section design below is based on the stated Traffic Index (TI) and our preliminary laboratory analysis of the site soils. We assumed an R-Value of 20 for flexible pavement design. The TI values were chosen based on our experience with similar projects. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design and City of Escondido Standard Figure No 3. The recommended flexible pavement section for this condition is given in Table 4.

Table 4 Preliminary Pavement Sections			
Traffic Description	Assumed Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base (inches)
Auto Parking	4.5	4	4
Auto Driveways	5.0	4	5
Truck Driveway	6.0	4	9

Flexible pavements should be constructed in accordance with current Caltrans Standard Specifications. Aggregate base should comply with the Caltrans Standard Specifications of Section 26. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D 1557).

For areas subject to regular truck loading (i.e., trash truck apron), we recommend a full depth of Portland Cement Concrete (P.C.C.) section of 7 inches with appropriate steel reinforcement and crack-control joints as designed by the project structural engineer. We recommend that sections be as nearly square as possible. A 3,250-psi mix that produces a 550-psi modulus of rupture should be utilized.

All pavement section materials conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications (Caltrans) and American Concrete Institute (ACI) codes. The upper 12 inches of subgrade soil and all aggregate base should be compacted to a relative compaction of at least 95 percent (based on ASTM Test Method D1557).

If pavement areas are adjacent to heavily watered landscape areas, we recommend some measure of moisture control be taken to prevent the subgrade soils from becoming saturated. It is recommended that the concrete curing separating the landscaping area from the pavement extend below the aggregate base to help seal the ends of the sections where heavy landscape watering may have access to the aggregate base. Concrete swales should be designed in roadway or parking areas subject to concentrated surface runoff.

6.7 Concrete Flatwork

Concrete sidewalks and other flatwork (including construction joints) should be designed by the project civil engineer and should have a minimum thickness of 4 inches. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 2 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement.

6.8 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as "sulfate attack." Soluble sulfate results (Appendix C) indicated negligible soluble sulfate content at the sites. We recommend that concrete in contact with earth materials be designed in accordance with Chapter 4 of ACI 318-11 (ACI, 2011).

Based on our laboratory testing, the site is not considered to exhibit a corrosive environment (Caltrans, 2012); however, there is a corrosion potential to buried uncoated metal conduits in direct contact with on site soils. Therefore, we recommend a corrosion engineer be consulted for additional evaluation of onsite soils to mitigate corrosion during design and construction.

6.9 Foundation Review

Foundation plans should be reviewed by Leighton to confirm that the recommendations in this report are incorporated in project plans.

6.10 Construction Observation

The recommendations provided in this report are based on preliminary design information, our experience during rough grading, and subsurface conditions disclosed by widely spaced excavations. The interpolated subsurface conditions should be checked in the field during construction. Construction observation of all onsite excavations and should be performed by a representative of this office so that construction is in accordance with the recommendations of this report. All footing excavations should be reviewed by this office prior to steel placement.

7.0 LIMITATIONS

The conclusions and recommendations presented in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. A final geotechnical report will be provided once final grades are known.

Project: 601586-008	Eng/Geol: WDO/MDJ
Scale: 1 " = 2,000 '	Date: August 2015

Base Map: ESRI ArcGIS Online 2015
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

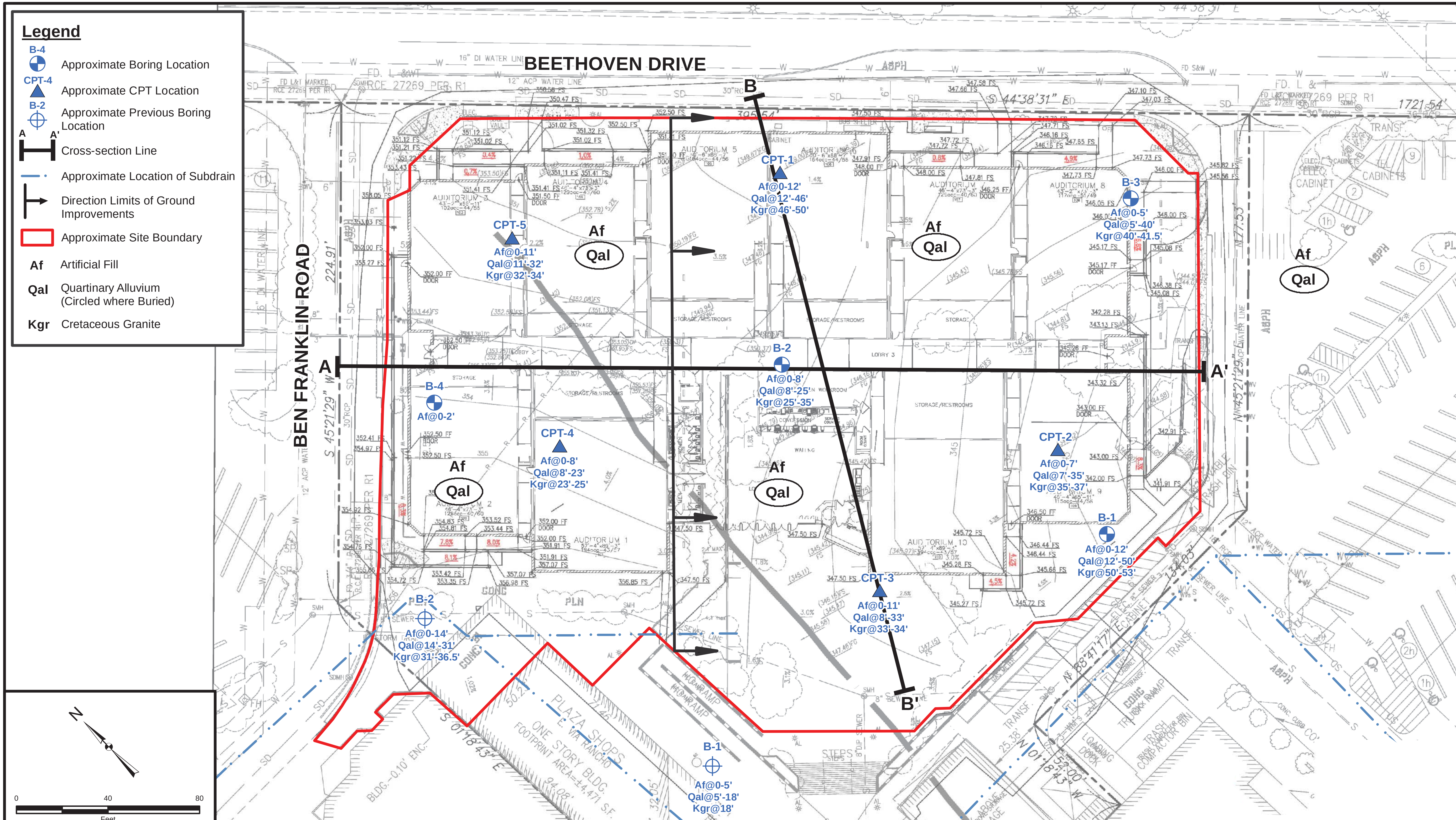
SITE LOCATION MAP

Proposed Regal Cinemas
Westfield North County Shopping Center
272 East Via Rancho Parkway
Escondido, California

Figure 1

Leighton

Map Saved as V:\Drafting\601586\008\Maps\601586_008_F01_SLM_2015_6_11.mxd on 8/6/2015 8:48:54 AM



Project: 601586-008	Eng/Geol: WDO/MDJ
Scale: 1 " = 40 '	Date: August 2015

Reference: Sheet 2 of 4, Overall Site Plan, Westfield North County - Regal Cinemas by R.A. Smith National, dated 6/3/2015.
Author: Leighton Geomatics (mmurphy)

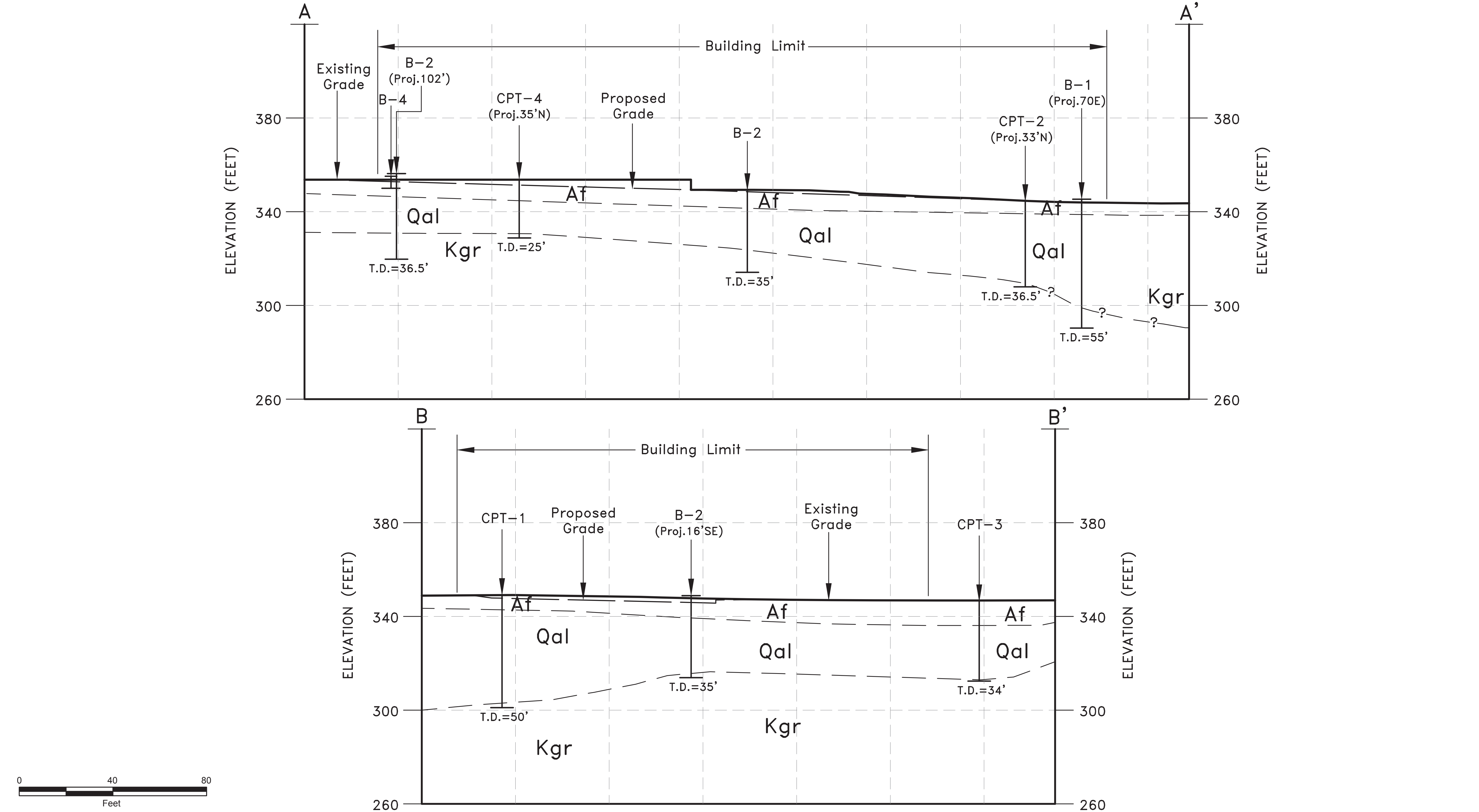
GEOTECHNICAL MAP

Proposed Regal Cinemas
Westfield North County Shopping Center
272 East Via Rancho Parkway
Escondido, California

Figure 2

Leighton

Map Saved as V:\Drafting\601586\008\Maps\601586-008_F02_GM_2015-07-31.mxd on 8/6/2015 9:26:15 AM



Project: 601586-008	Eng/Geol: WDO/MDJ	
Scale: 1"=40'	Date: August 2015	
Reference:		
Author: MAM		

CROSS-SECTIONS A-A' & B-B'

Proposed Regal Cinemas
Westfield North County Shopping Center
272 East Via Rancho Parkway
Escondido, California

Figure 3

Leighton

P:\DRAFTING\601586\008\CAD\601586-008_CROSS-SECTIONS_2015-07-31.DWG (08-07-15 1:13:24PM) Plotted by: mmurphy

APPENDIX A

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APPENDIX A

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APPENDIX B

Boring Logs
and
CPT Sounding Logs

GEOTECHNICAL BORING LOG KEY

Project No.		**Date Drilled**	
Project	KEY TO BORING LOG GRAPHICS	**Logged By**	
Drilling Co.		**Hole Diameter**	
Drilling Method		**Ground Elevation**	
Location		**Sampled By**	

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								Asphaltic concrete	
									Portland cement concrete	
								CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay	
								CH	Inorganic clay; high plasticity, fat clays	
								OL	Organic clay; medium to plasticity, organic silts	
	5							ML	Inorganic silt; clayey silt with low plasticity	
								MH	Inorganic silt; diatomaceous fine sandy or silty soils; elastic silt	
								ML-CL	Clayey silt to silty clay	
								GW	Well-graded gravel; gravel-sand mixture, little or no fines	
								GP	Poorly graded gravel; gravel-sand mixture, little or no fines	
	10							GM	Silty gravel; gravel-sand-silt mixtures	
								GC	Clayey gravel; gravel-sand-clay mixtures	
								SW	Well-graded sand; gravelly sand, little or no fines	
								SP	Poorly graded sand; gravelly sand, little or no fines	
								SM	Silty sand; poorly graded sand-silt mixtures	
	15							SC	Clayey sand; sand-clay mixtures	
									Bedrock	
									Ground water encountered at time of drilling	
				B-1					Bulk Sample	
	20			C-1					Core Sample	
				G-1					Grab Sample	
				R-1					Modified California Sampler (3" O.D., 2.5 I.D.)	
				SH-1					Shelby Tube Sampler (3" O.D.)	
				S-1					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.)	
	25			PUSH					Sampler Penetrates without Hammer Blow	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	TR	THERMAL RESISTIVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-1

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	343'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								4" ASPHALT CONCRETE over 3" AGGREGATE BASE	
							9.1	SC	ARTIFICIAL FILL (Af) @ 7": Clayey SAND with gravel, moist, dark brown, fine- to medium-grained sand with semi-plastic fines, some fine-grained gravel	EI, CR
340				B-1 3'-5'						
	5			R-1	8 13 18	123	11	SM	@ 5': Silty SAND with clay, medium dense, moist, dark brown, fine- to medium-grained sand with non-plastic fines, some semi-plastic fines, micaceous	SA
335										
	10			R-2	5 6 12	122	12		@ 10': same as above, slightly more semi-plastic fines	DS
								SM	QUATERNARY ALLUVIUM (Qal)	
330										
	15 ▽			R-3	1 2 1	105	22	SC	@ 15': Clayey SAND, very loose, wet, dark brown, micaceous, fine- to medium-grained sand with semi-plastic fines, some non-plastic fines @ 15.5': Groundwater encountered	-200, CN
325										
	20			R-4	1 4 4	98	27	SM	@ 20': Silty SAND, loose, wet, brown, fine- to medium-grained sand with non-plastic fines, micaceous	
320										
	25			R-5	3 4 6	109	17		@ 25': Silty SAND, loose, wet, brown, fine- to medium-grained sand, non-plastic fines, interbed silty fine- to medium-grained sand, trace semi-plastic fines	CN
315										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-1

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	343'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	30			R-6	4 5 8	98	27	SM	QUATERNARY ALLUVIUM (Qal) continued @ 30': Silty SAND, very stiff, wet, dark brown, non-plastic fines with fine-grained sand, micaceous	-200, CN
310										
	35			R-7	2 5 6	99	26	SM	@ 35': Silty SAND, loose, wet, black, fine-grained sand with semi-plastic fines, micaceous	
305										
	40			SPT-1	2 4 8			SM/ML	@ 40': Silty SAND to sandy SILT, medium dense to medium stiff, wet, black, non-plastic fines and fine-grained sand, micaceous	
300										
	45			R-8	3 4 9	109	20	SM/ML	@ 45': Sily SAND to sandy SILT, very stiff, wet, black, non-plastic fines with fine-grained sand, micaceous	SA
295										
	50			R-9	34/3"	115	17		CRETACEOUS GRANITE (Kgr) @ 50': Bedrock	
290										
	55			R-10	50/3"	126	11		@ 55': Bedrock **Total Drilled Depth = 55 Feet** **Total Sampled Depth = 55.25 Feet** **Groundwater encountered at 15.5 feet at time of drilling** **Backfilled with bentonite grout on 5/19/15**	
285										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-2

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	350'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
350	0								4" ASPHALT CONCRETE over 6" AGGREGATE BASE	
				B-1 2'-5'				SC	ARTIFICIAL FILL (Af) @ 10": Silty clayey SAND, brown to dark gray, moist, fine-grained sand with non-plastic fines and semi-plastic fines	RV
345	5			R-1	8 11 13	122	11	SM-SC	@ 5': Clayey silty SAND, medium dense, moist, dark brown, fine- to medium-grained sand with non-plastic fines and semi-plastic fines	
								SM-SC	QUATERNARY ALLUVIUM (Qal)	
340	10			R-2	5 5 9			SM	@ 10': Silty SAND, medium dense, dark brown, moist, fine- to medium-grained sand with non-plastic fines, micaceous	
335	15			R-3	3 5 5				@ 15': Same as above, loose	SA
330	20			R-4	3 4 4	106	22	CL	@ 20': Sandy silty CLAY, stiff, moist, dark brown, semi-plastic fines with non-plastic fines and fine-grained sand	AL, CN
325	25			SPT-1	8 21 50/5"			SM	RESIDUAL SOIL (Kgr) @ 25': Silty SAND, very dense, reddish brown to brown, moist, medium-grained sand	
									CRETACEOUS GRANITE (Kgr)	
320	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-2

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	350'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
320	30			R-5	25 50/5"	114	13		CRETACEOUS GRANITE (Kgr) @ 30': Bedrock	
315	35			R-6	50/3"				@ 35': Same as above, no recovery	
									Total Drilled Depth = 35 Feet **Total Sampled Depth = 35.25 Feet** **Groundwater encountered at 16 feet below ground surface** **Backfilled with bentonite on 5/19/15**	
310	40									
305	45									
300	50									
295	55									
290	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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GEOTECHNICAL BORING LOG B-3

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	345'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
345	0								4" ASPHALT CONCRETE over 6" AGGREGATE BASE	
								SM	ARTIFICIAL FILL (Af) @ 10": Silty SAND, moist, dark brown, fine-grained sand with non-plastic fines	
								SM	QUATERNARY OLDER ALLUVIUM (Qoa) @ 5': Silty SAND, medium dense, dark brown, moist, fine-grained sand with non-plastic fines, micaceous	
340	5			R-1	2 5 12	121	12			
335	10			R-2	3 4 5	114	14		@ 10': Same as above, loose	
330	15			R-3	1 1 1	98	25	SC	@ 15': Clayey SAND, wet, dark brown, non-plastic fines with fine-grained sand, trace medium-grained sand	CN, DS, -200
325	20			R-4	2 2 3	98	27		@ 20': Same as above, medium stiff, no medium-grained sand	
320	25			SPT-1	1 2 2				@ 25': Same as above	AL
315	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-3

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	345'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
315	30			R-5 B-1 30'-35'	6 7 9	101	24	SC-SM	@ 30': Clayey SAND, medium dense, wet, dark brown to dark gray, semi-plastic fines with non-plastic fines	DS, -200
310	35			R-6	8 9 21	90	32	CL/ML	@ 35': Sandy silty CLAY to sandy clayey SILT, hard, wet, non-plastic fines and semi-plastic fines with fine-grained sand	CN
305	40			R-7	6 6 11	122	14	SM	<u>RESIDUAL SOIL (Kgr)</u> @ 40': Silty SAND, medium dense, wet, brown to reddish brown, medium-grained sand with non-plastic fines Boring exploration terminated due to difficulty recovering sample	
300	45									
295	50									
290	55									
285	60									

Total Drilled Depth = 40 Feet
Total Sampled Depth = 41.5 Feet
Groundwater encountered at 15 feet at time of drilling
Backfilled with bentonite on 5/19/15

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-4

Project No.	601586-008	**Date Drilled**	5-19-15
Project	Westfield North County Shoppingt Center/Regal Cinemas	**Logged By**	LR
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer	**Ground Elevation**	354'
Location	See Figure 2 Boring Location Map	**Sampled By**	LR

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								4" ASPHALT CONCRETE over 8" AGGREGATE BASE	
				B-1 1'-5'				SP	ARTIFICIAL FILL (Af) @ 1': SAND, moist, light brown, fine- to medium-grained sand	
								SM	@ 2': Silty SAND, moist, dark brown, fine-grained SAND with non-plastic fines	
350										
	5								Boring exploration due to broken drill lever **Total Drilled Depth = 5 Foot** **Total Sampled Depth = 5 Feet** **No groundwater encountered at time of drilling** **Backfilled with bentonite on 5/19/15**	
345										
	10									
340										
	15									
335										
	20									
330										
	25									
325										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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

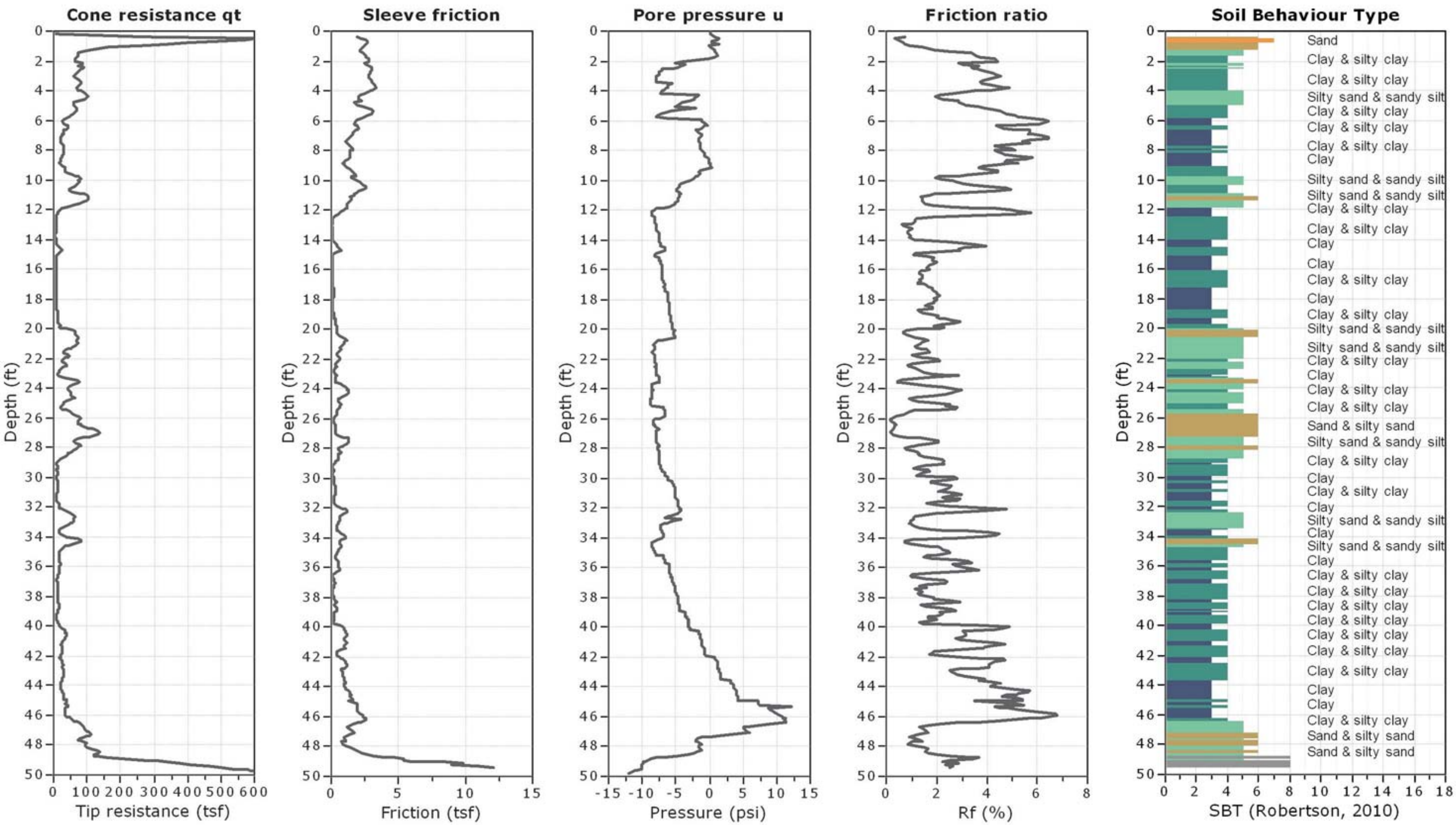
CPT: CPT-1

Total depth: 49.82 ft, Date: 5/19/2015

Cone Type: Vertek

Project: Leighton Consulting/NCM Regal

Location: 272 E. Via Rancho Parkway Escondido, CA

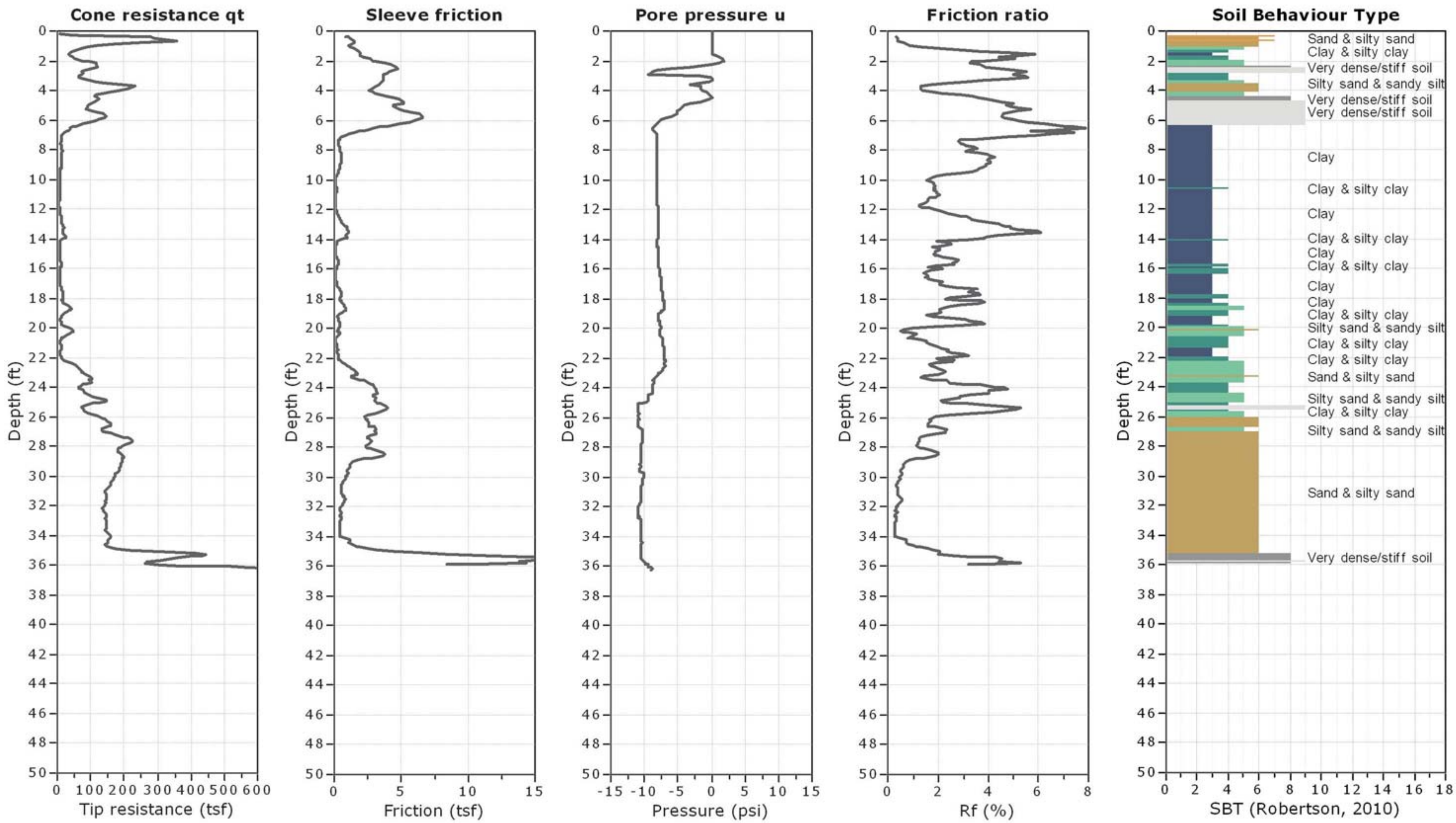


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: CPT-2

Project: Leighton Consulting/NCM Regal
Location: 272 E. Via Rancho Parkway Escondido, CA

Total depth: 36.25 ft, Date: 5/19/2015
Cone Type: Vertek



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

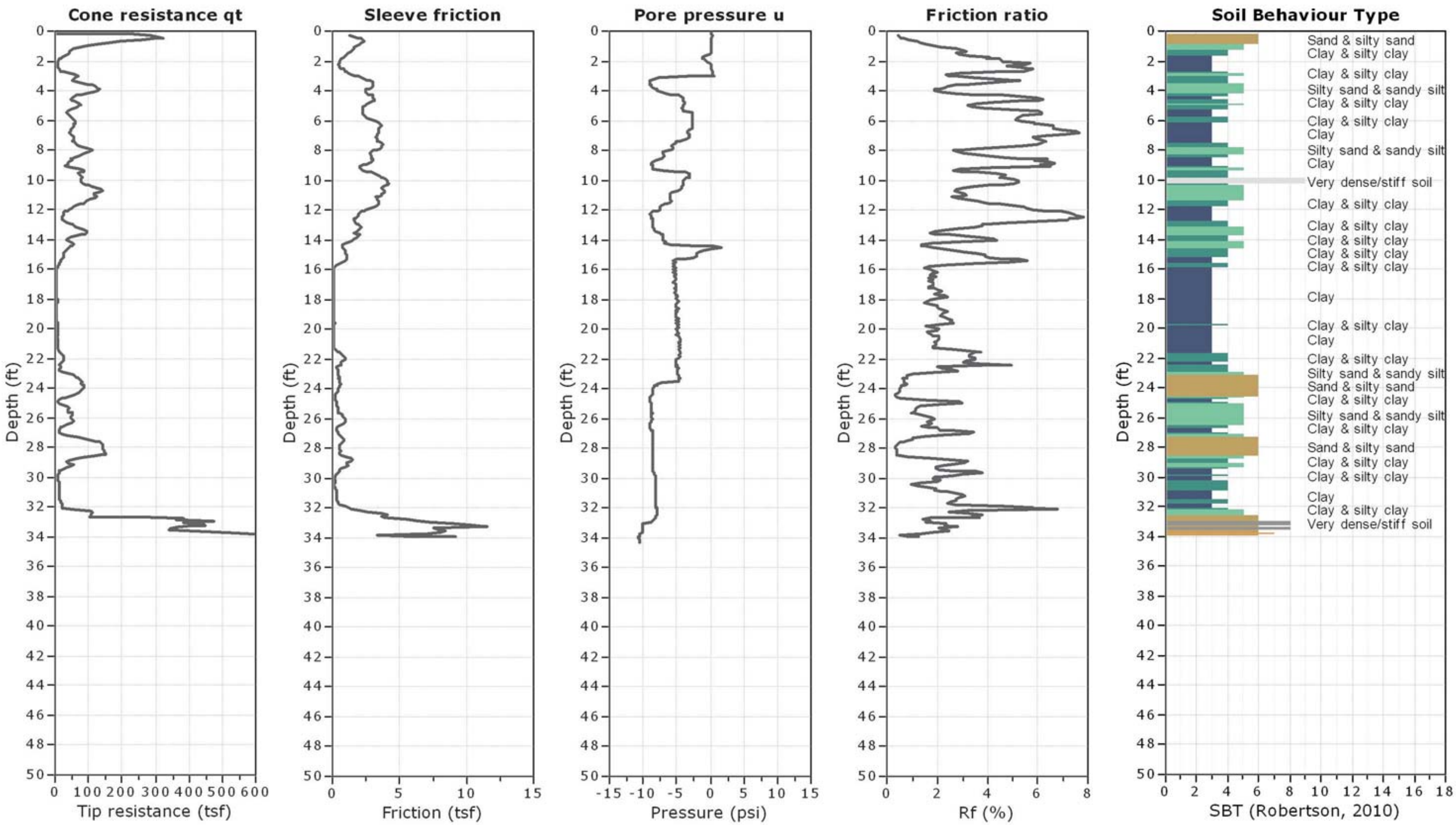
CPT: CPT-3

Project: Leighton Consulting/NCM Regal

Location: 272 E. Via Rancho Parkway Escondido, CA

Total depth: 34.32 ft, Date: 5/19/2015

Cone Type: Vertek



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

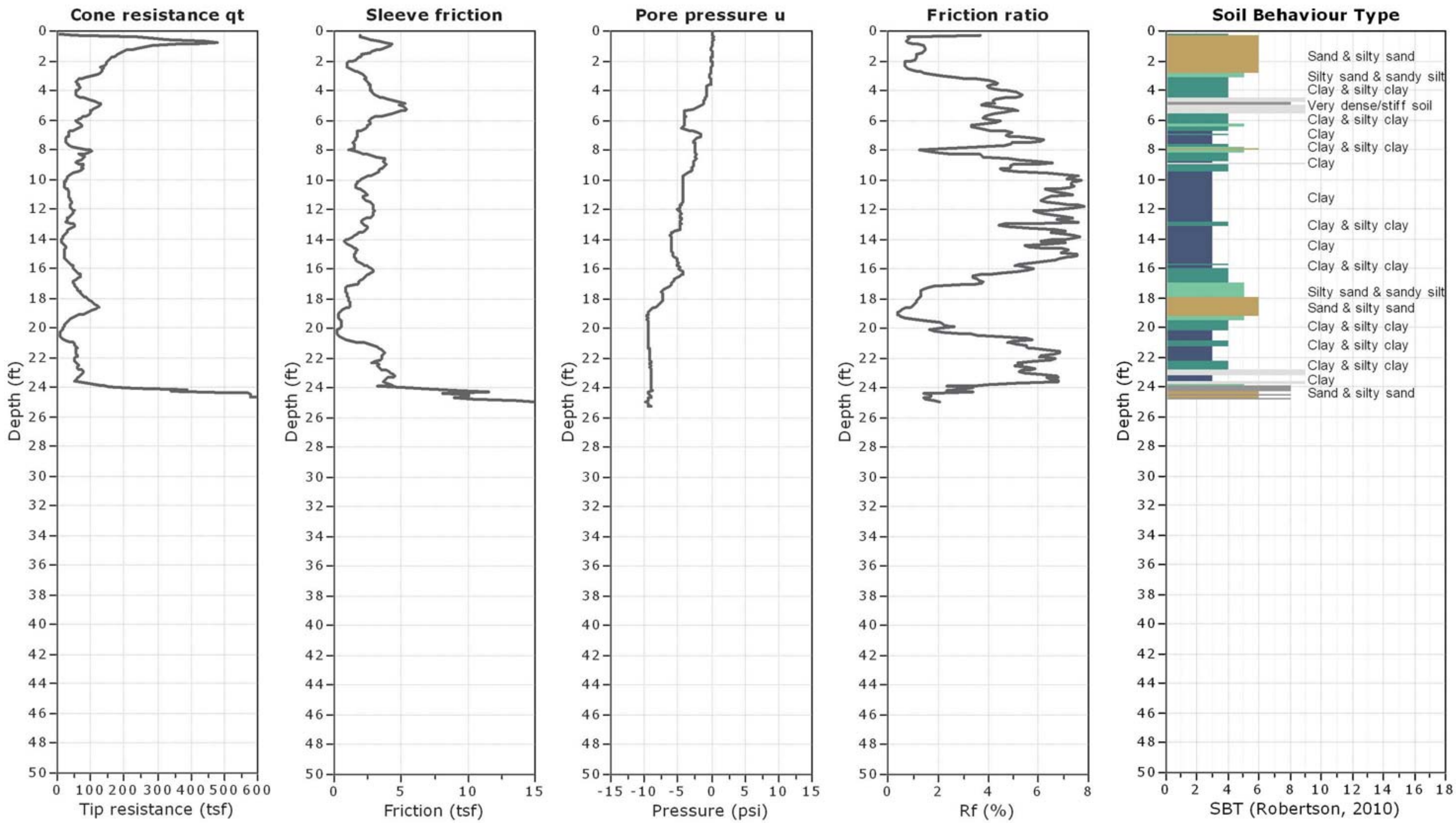
CPT: CPT-4

Total depth: 25.27 ft, Date: 5/19/2015

Cone Type: Vertek

Project: Leighton Consulting/NCM Regal

Location: 272 E. Via Rancho Parkway Escondido, CA

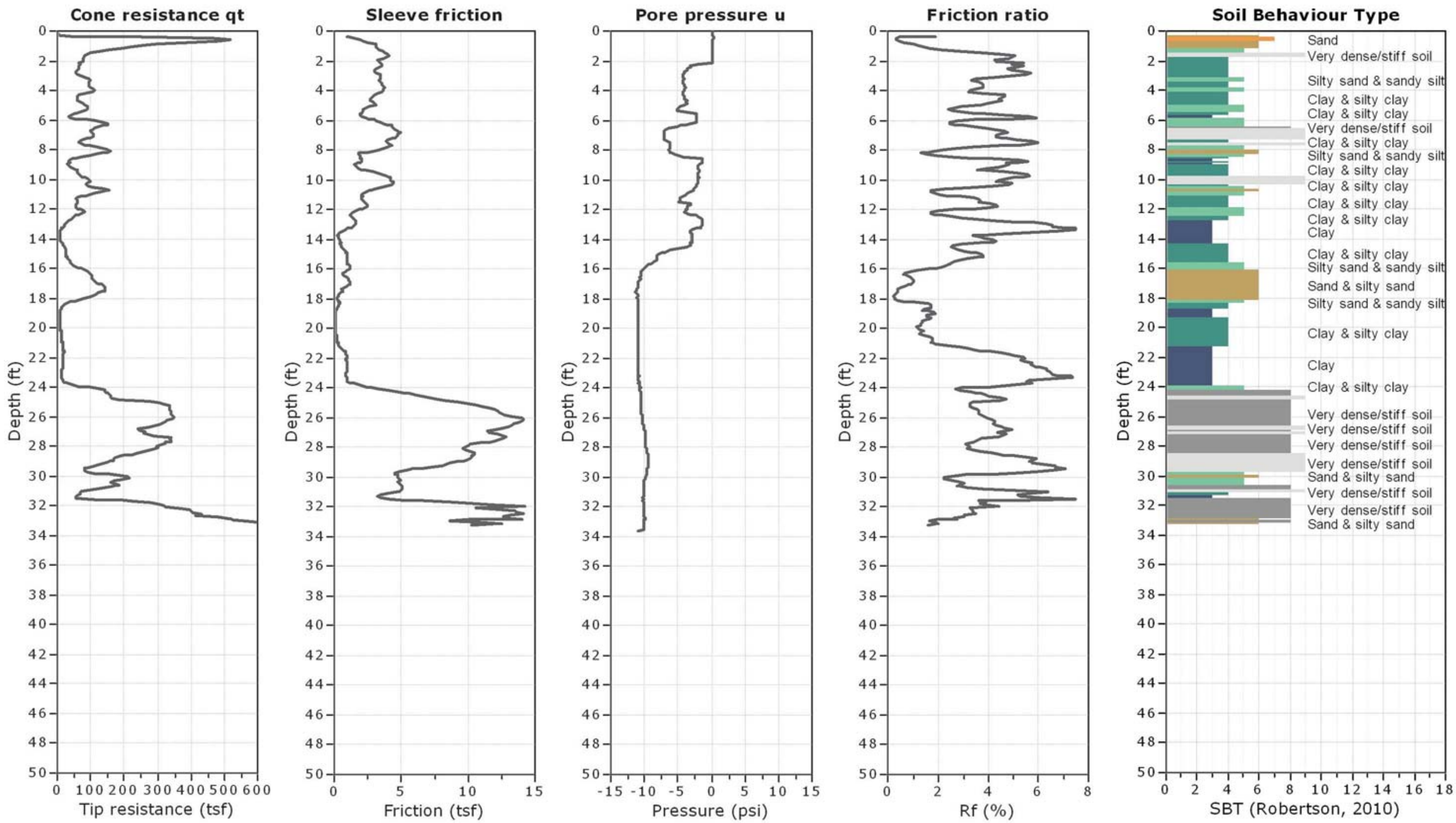


Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: CPT-5

Project: Leighton Consulting/NCM Regal
Location: 272 E. Via Rancho Parkway Escondido, CA

Total depth: 33.61 ft, Date: 5/19/2015
Cone Type: Vertek



GEOTECHNICAL BORING LOG B-1

Project No.	601586-002	**Date Drilled**	6-4-10
Project	Westfield North County Parking Area	**Logged By**	MDJ
Drilling Co.	Baja Explorations	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	357'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								0-6": Concrete 6"-10": Decomposed Granite	
355				B-1				SC	ARTIFICIAL FILL (Afu) @ 6"-10": Clayey SAND: Dark brown, moist, loose	
	5			R-1	10	107	13	SC	QUATERNARY ALLUVIUM (Qal) @ 5': Clayey SAND: Dark brown, moist, loose; possible clean alluvial derived fill, slightly micaceous	
350									@ 8': Clayey medium SAND: Dark brown, moist, loose; micaceous	
	10			R-2	12	109	14			
345										
	15			R-3	32	118	12	SM	@ 14': Denser drilling @ 14': Silty SAND with CLAY: Dark brown, moist, medium dense; micaceous residual soil (Kgr derived)	
340				R-4	39	119	10	SM	RESIDUAL SOIL (Kgr) @ 18': Silty medium SAND: Brown to red-brown, damp, medium dense	
	20									
335								SM	CRETACEOUS GRANITE @ 23': Excavates to silty medium to coarse SAND: Orange-brown, damp, very dense	
	25			R-5	50/3"					
330									Total Depth = 26.5 Feet No ground water encountered at time of drilling Backfilled with bentonite grout on 6/4/10	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG B-2

Project No.	601586-002	**Date Drilled**	6-8-10
Project	Westfield North County Parking Area	**Logged By**	MDJ
Drilling Co.	Baja Explorations	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	357'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SC	0-6": Concrete	
355				B-1 1'-4'					ARTIFICIAL FILL (Afu) @ 6": Clayey SAND: Brown, moist, medium dense	
	5			R-1	23	112	14		@ 5': Clayey SAND: Brown, moist, medium dense	
350										
	10			R-2	37	120	15	SM-SC	@ 9.5': Clayey SAND to silty SAND: Dark brown, moist, medium dense; slightly porous, possible Qal	
345										
	15							SM	QUATERNARY ALLUVIUM (Qal)	
				R-3	25	118	12		@ 16': Silty fine to medium SAND with CLAY: Red-brown to dark brown, moist, medium dense; slightly micaceous	
340										
	20			R-4	13	113	15		@ 20': Silty fine to medium SAND with clay: Brown with dark brown areas, moist, loose	
335										
	25			R-5	8	112	15		@ 25': Silty fine SAND: Brown, moist to wet, loose	
330										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
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- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG B-2

Project No.	601586-002	**Date Drilled**	6-8-10
Project	Westfield North County Parking Area	**Logged By**	MDJ
Drilling Co.	Baja Explorations	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	357'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-6	32	115	16	SM	QUATERNARY ALLUVIUM (Qal) continued	
325									RESIDUAL SOIL (Kgr) @ 31': Silty medium SAND: Orange-brown, damp, medium dense to dense; decomposed granite in tip	
	35			R-7	50/6"	110	18		@ 35': Silty medium SAND: Orange-brown, damp, medium dense to dense; decomposed granite in tip	
320									Total Depth = 36.5 Feet Ground water encountered at 30 feet at time of drilling Backfilled with bentonite grout on 6/8/10	
	40									
315										
	45									
310										
	50									
305										
	55									
300										
	60									

SOIL DESCRIPTION: This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

APPENDIX C

Laboratory Testing

APPENDIX C

Laboratory Testing Procedures and Test Results

Atterberg Limits: The Atterberg Limits were determined in accordance with ASTM Test Method D 4318 for engineering classification of the fine-grained materials. Test results are presented on the attached figures.

Chloride Content: Chloride content was tested in accordance with DOT California Test No 422. The results are presented below:

Sample Location	Chloride Content, ppm
B-1 @ 3 to 5 feet	86

Consolidation Tests: Consolidation tests were performed on selected, relatively undisturbed ring samples in accordance with Modified ASTM Test Method D2435. Samples were placed in a consolidometer and loads were applied in geometric progression. The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. The consolidation pressure curves are presented on the attached figures.

Direct Shear Test: Direct shear tests (ASTM D 3080) were performed on selected samples which was soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box and reloading of the sample, the pore pressures set up in the sample (due to the transfer) were allowed to dissipate for a period of approximately 1-hour prior to application of shearing force. The samples were tested under various normal loads utilizing a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of 0.0025 inches per minute. After a shear strain of 0.2 inches, the motor was stopped and the sample was allowed to "relax" for approximately 15 minutes. The stress drop during the relaxation period was recorded. It is anticipated that, in a majority of samples tested, the 15 minutes relaxing of the samples is sufficient to allow dissipation of pore pressures that may have set up in the samples due to shearing. The drained peak strength was estimated by deducting the shear force reduction during the relaxation period from the peak shear values. The shear values at the end of shearing are considered to be ultimate values and

APPENIX C (Continued)

are presented on the attached figure. The samples were either remolded to 90% relative compaction, undisturbed, or the samples were tested in a torsional shear machine to evaluate the remolded clay seam properties. The test results are presented in attached figures.

Expansion Index Tests: The expansion potential of selected materials was evaluated by the Expansion Index Text, ASTM Test Method 4829. Specimens are molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

Sample Location	Description	Expansion Index	Expansion Potential
B-1 @ 10 to 15 feet	Gray Brown Clayey SAND	36	Low

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with DOT California Test Method 643. The results are presented in the table below:

Sample Location	pH	Minimum Resistivity (ohms-cm)
B-1 @ 3 to 5 feet	8.42	1177

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed on relatively undisturbed samples obtained from the test borings and in general accordance to ASTM D 2937. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from "undisturbed" or disturbed samples.

Particle Size Analysis: Particle size analysis was performed in general conformance with methods according to ASTM D 1140 and D 6913. Plots and tables of the sieve results are provided on figures in this appendix.

APPENIX C (Continued)

R-Value Test: R-Value testing was performed in general accordance with DOT California Test Method 301. The results are presented on attached figure.

Soluble Sulfates: The soluble sulfate of a selected sample was determined by standard geochemical methods. The test results are presented in the table below:

Sample Location	Sulfate Content (%)
B-1 @ 3 to 5 feet	0.024

ATTERBERG LIMITS
ASTM D 4318

Leighton

Project Name:	NCM / REGAL	Tested By:	BCC	Date:	6/18/2015
Project No. :	601586.008	Input By:	BCC	Date:	6/25/2015
Boring No.:	B-2	Checked By:	BCC	Date:	6/26/15
Sample No.:	R-4	Depth (ft.)	20.0		
Sample Description:	CL: BROWN LEAN CLAY				

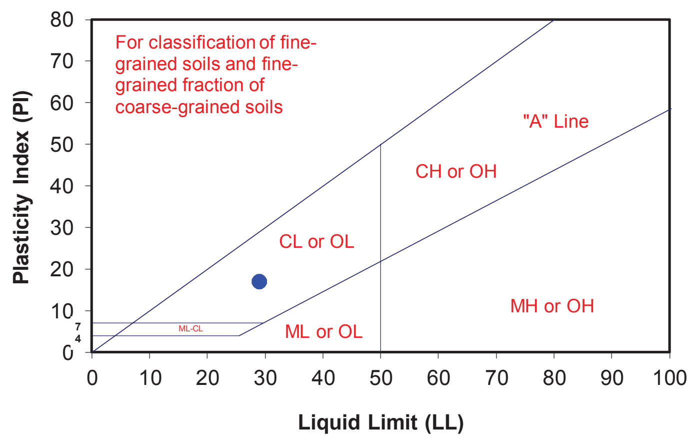
	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows			25	20	15	
Wet Wt. of Soil + Cont. (g)	6.98	7.21	11.99	8.99	9.99	
Dry Wt. of Soil + Cont. (g)	6.38	6.60	9.62	7.27	8.01	
Wt. of Container (g)	1.30	1.30	1.33	1.34	1.33	
Moisture Content (%)	11.8	11.5	28.6	29.0	29.6	

Liquid Limit	29
Plastic Limit	12
Plasticity Index	17
Classification	CL

PI at "A" - Line = 0.73(LL-20) = 6.57

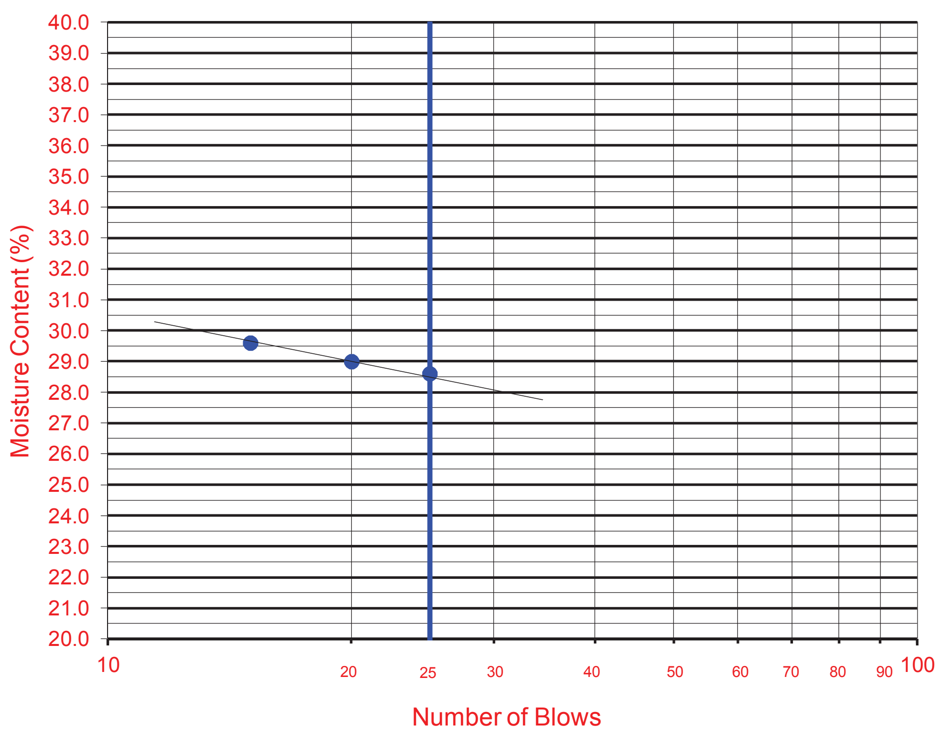
One - Point Liquid Limit Calculation

LL =Wn(N/25) $^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Rev. 03-14

ATTERBERG LIMITS
ASTM D 4318

Leighton

Project Name:	NCM / REGAL	Tested By:	BCC	Date:	6/18/2015
Project No. :	601586.008	Input By:	BCC	Date:	6/25/2015
Boring No.:	B-3	Checked By:	BCC	Date:	6/26/15
Sample No.:	SPT-1	Depth (ft.)	25.0		

Sample Description: CL: BROWN LEAN CLAY

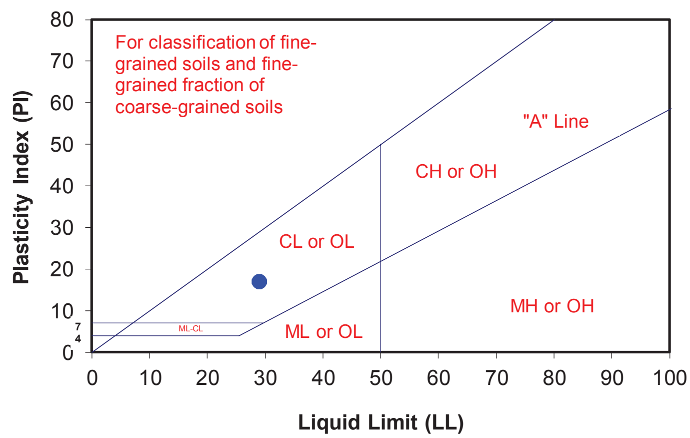
	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows			29	20	15	
Wet Wt. of Soil + Cont. (g)	7.55	7.47	8.59	9.05	7.92	
Dry Wt. of Soil + Cont. (g)	6.87	6.84	6.97	7.29	6.40	
Wt. of Container (g)	1.30	1.30	1.35	1.35	1.34	
Moisture Content (%)	12.2	11.4	28.8	29.6	30.0	

Liquid Limit	**29**
Plastic Limit	**12**
Plasticity Index	**17**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) = 6.57

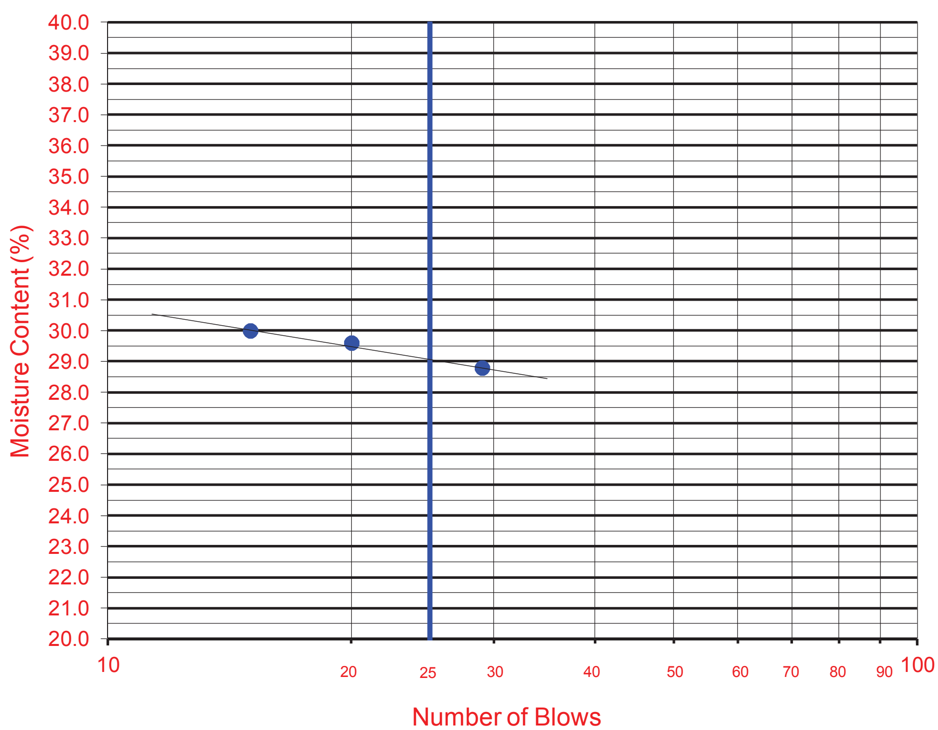
One - Point Liquid Limit Calculation

LL =Wn(N/25) $^{0.121}$



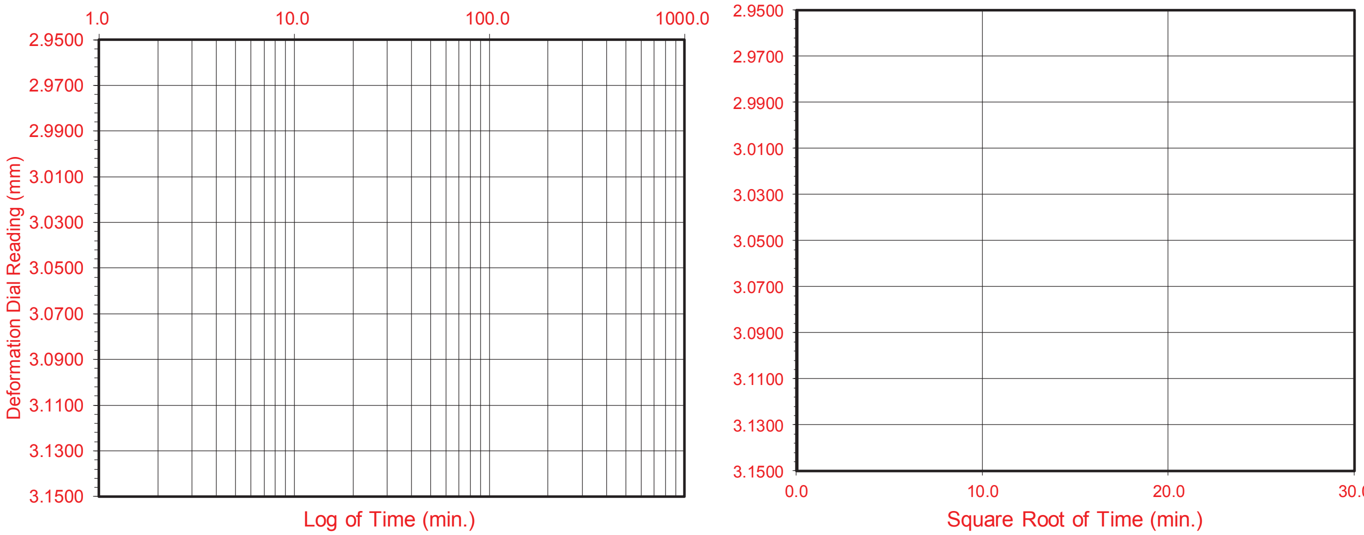
PROCEDURES USED

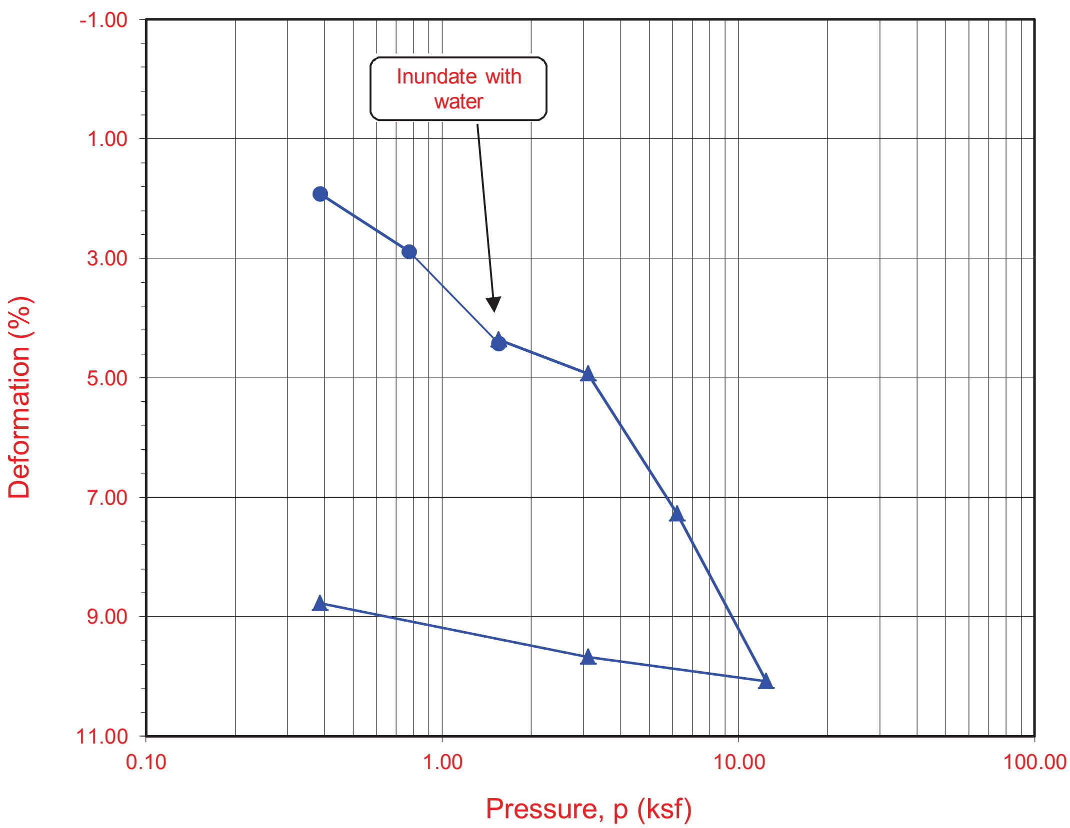
- [ ] Wet Preparation
  Multipoint - Wet
- [x] Dry Preparation
  Multipoint - Dry
- [x] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Rev. 03-14

Time Readings @ 0 kPa





Boring No.	Sample No.:	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-1	**R-3**	**15**	**21.8**	**15.9**	**104.5**	**114.5**	**0.614**	**0.472**	**96**	**91**

Sample Description:

SC: BROWN CLAYEY SAND

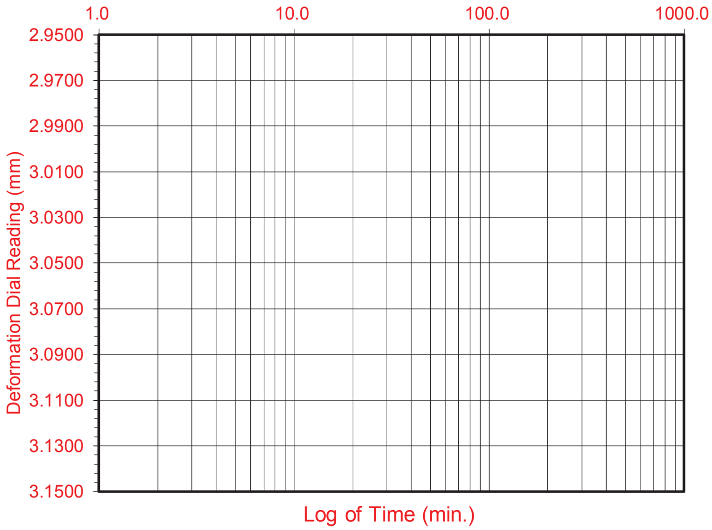
Leighton

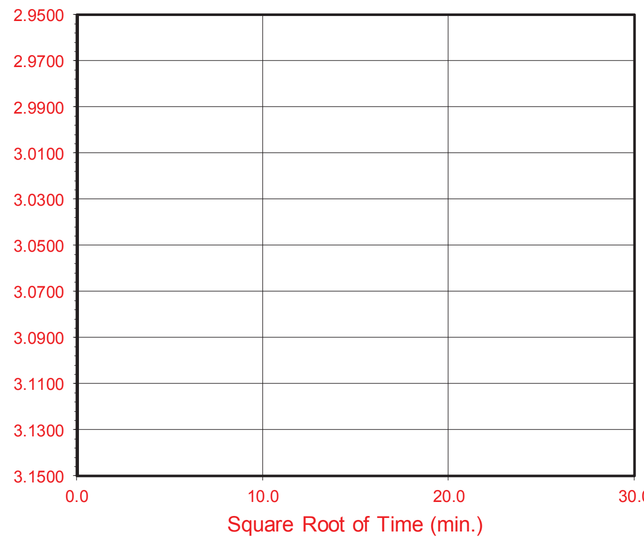
Project No.:	601586.008
Project Name:	NCM / REGAL

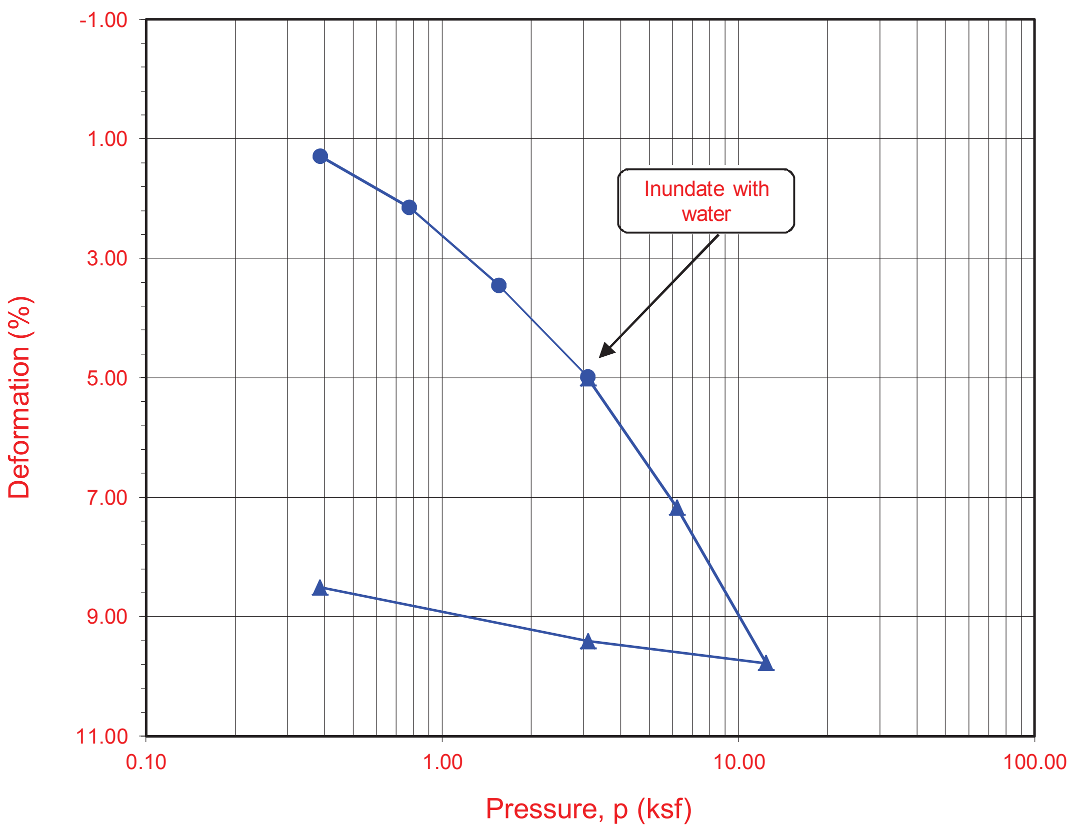
ONE - DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Rev. 10-14

Time Readings @ 0 kPa







Boring No.	Sample No.:	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-1	**R-5**	**25**	**20.0**	**15.2**	**109.2**	**119.3**	**0.544**	**0.412**	**99**	**100**

Sample Description:

SM: BROWN SILTY SAND

Leighton

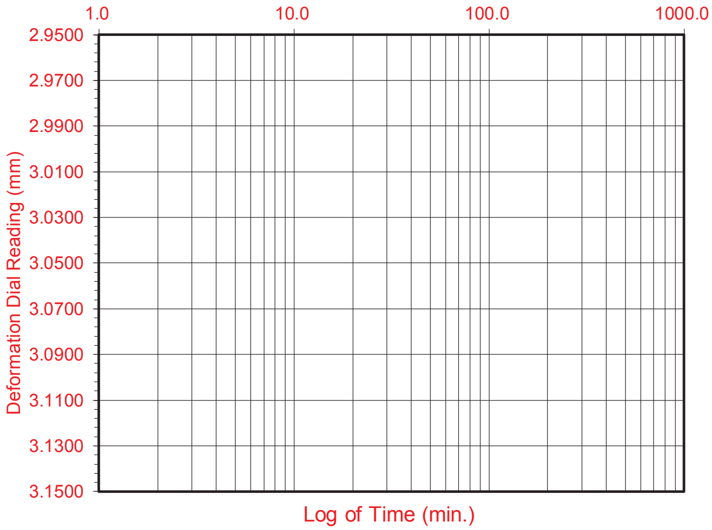
Project No.:	601586.008
Project Name:	NCM / REGAL

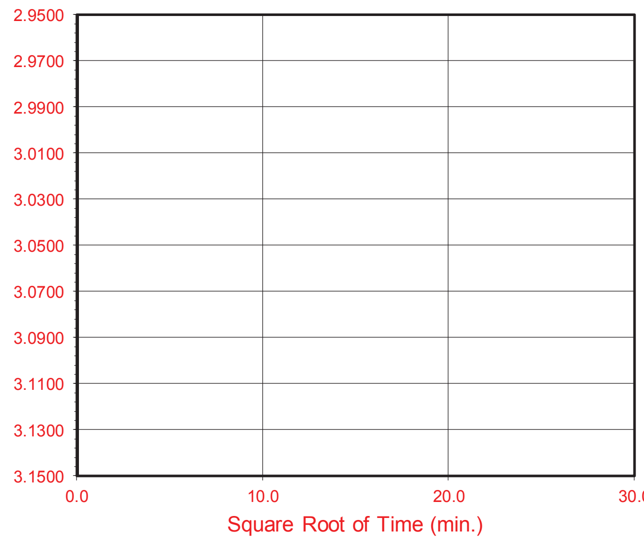
ONE - DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

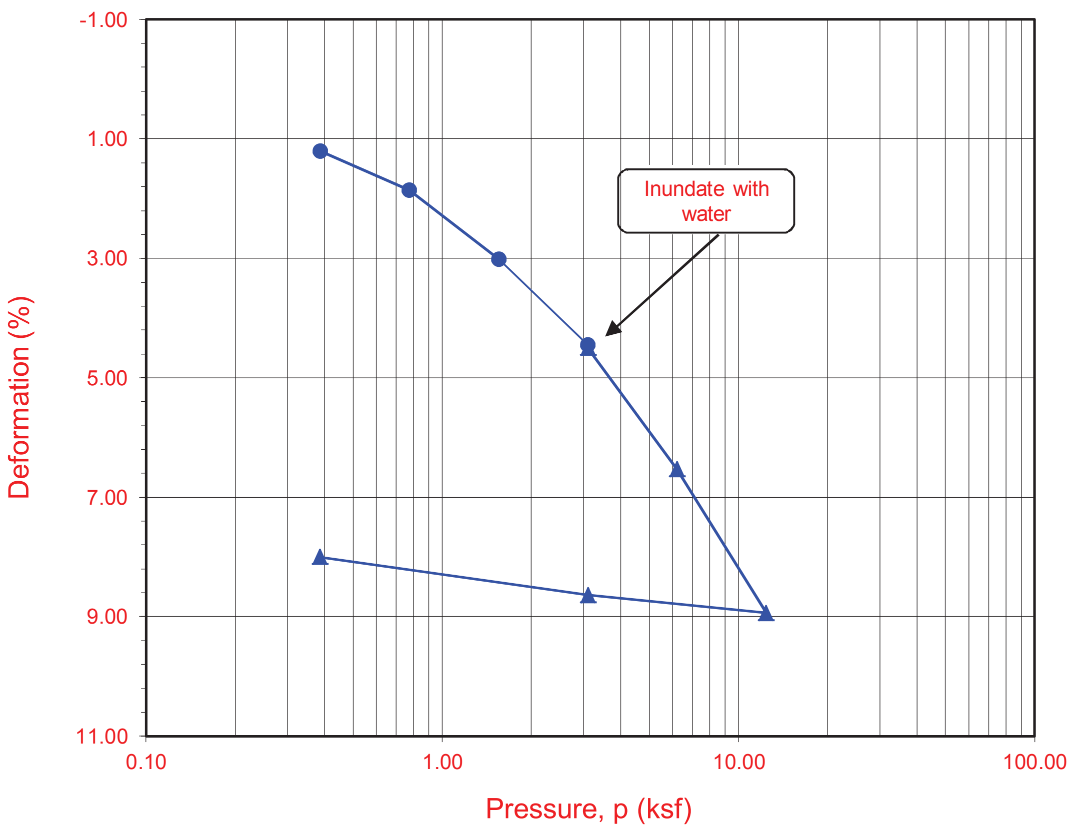
ASTM D 2435

Rev. 10-14

Time Readings @ 0 kPa







Boring No.	Sample No.:	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-1	**R-6**	**30**	**19.9**	**15.2**	**109.8**	**119.3**	**0.536**	**0.413**	**100**	**100**

Sample Description:

SM: BROWN SILTY SAND

Leighton

Project No.: 601586.008

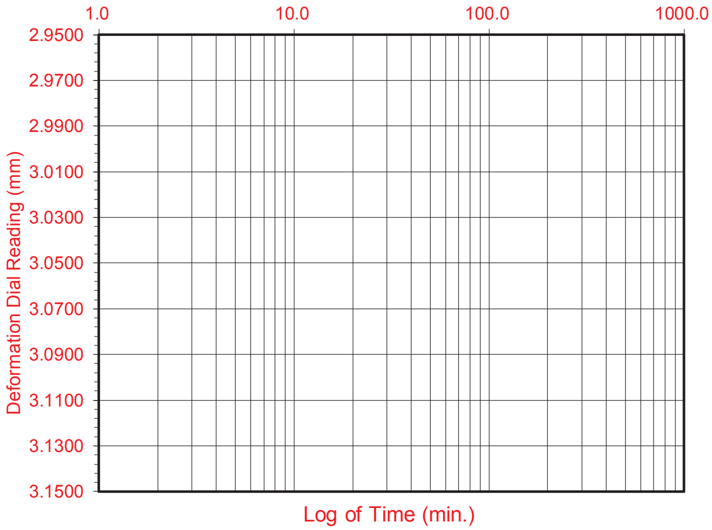
Project Name: NCM / REGAL

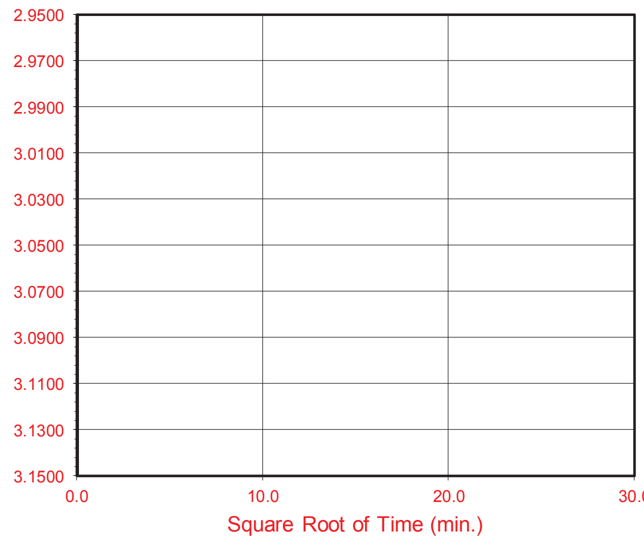
ONE - DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

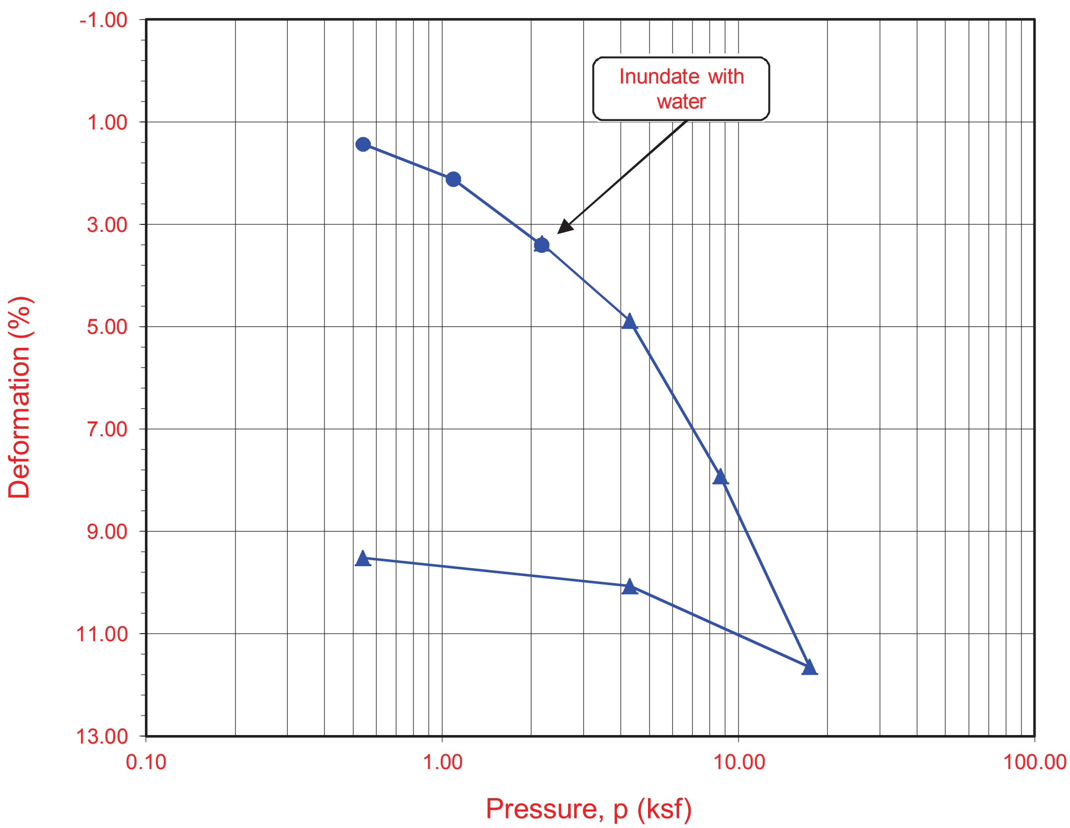
ASTM D 2435

Rev. 10-14

Time Readings @ 0 kPa







Boring No.	Sample No.:	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-2	**R-4**	**20**	**21.6**	**16.5**	**105.5**	**116.5**	**0.598**	**0.446**	**97**	**100**

Sample Description:

CL: BROWN LEAN CLAY

Leighton

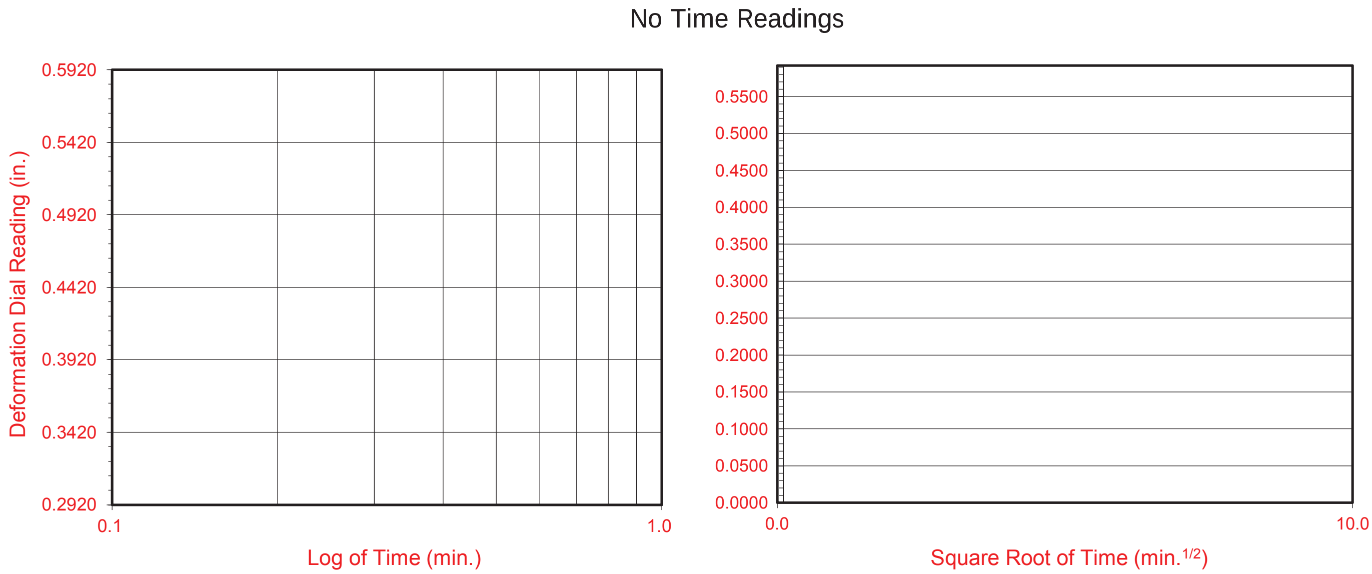
Project No.: 601586.008

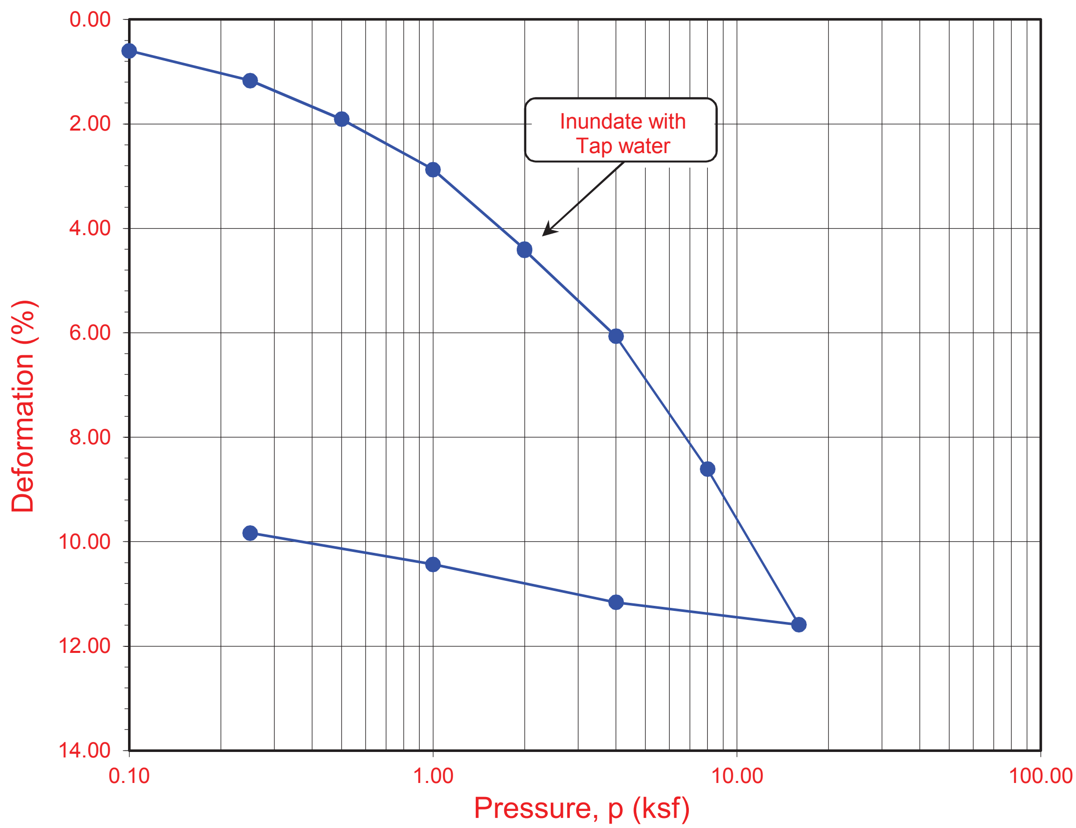
Project Name: NCM / REGAL

ONE - DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Rev. 10-14





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-3	**R-3**	**15.0**	**25.1**	**20.8**	**98.4**	**106.9**	**0.713**	**0.544**	**95**	**98**

Soil Identification: SC: DARK GRAYISH BROWN CLAYEY SAND

Leighton

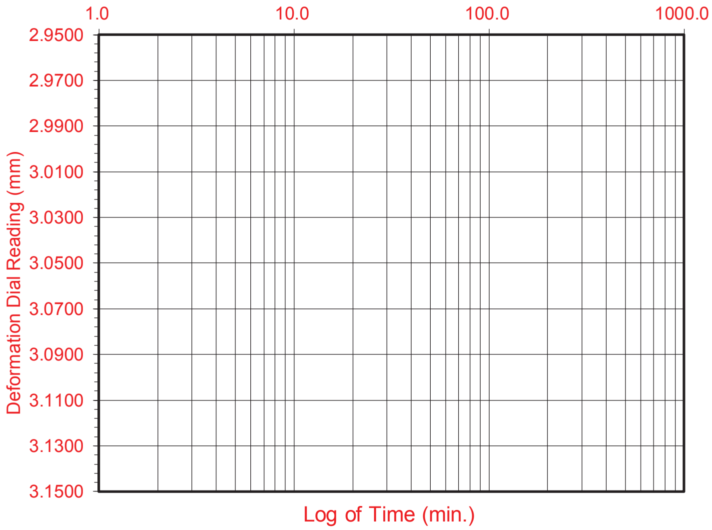
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

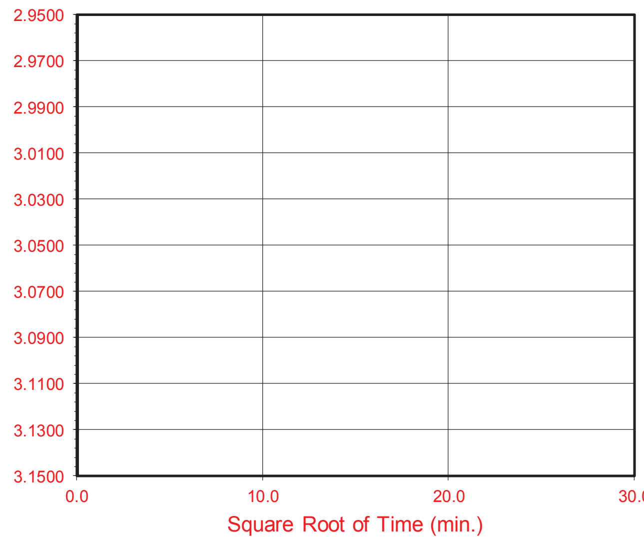
Project No.: 601586-008

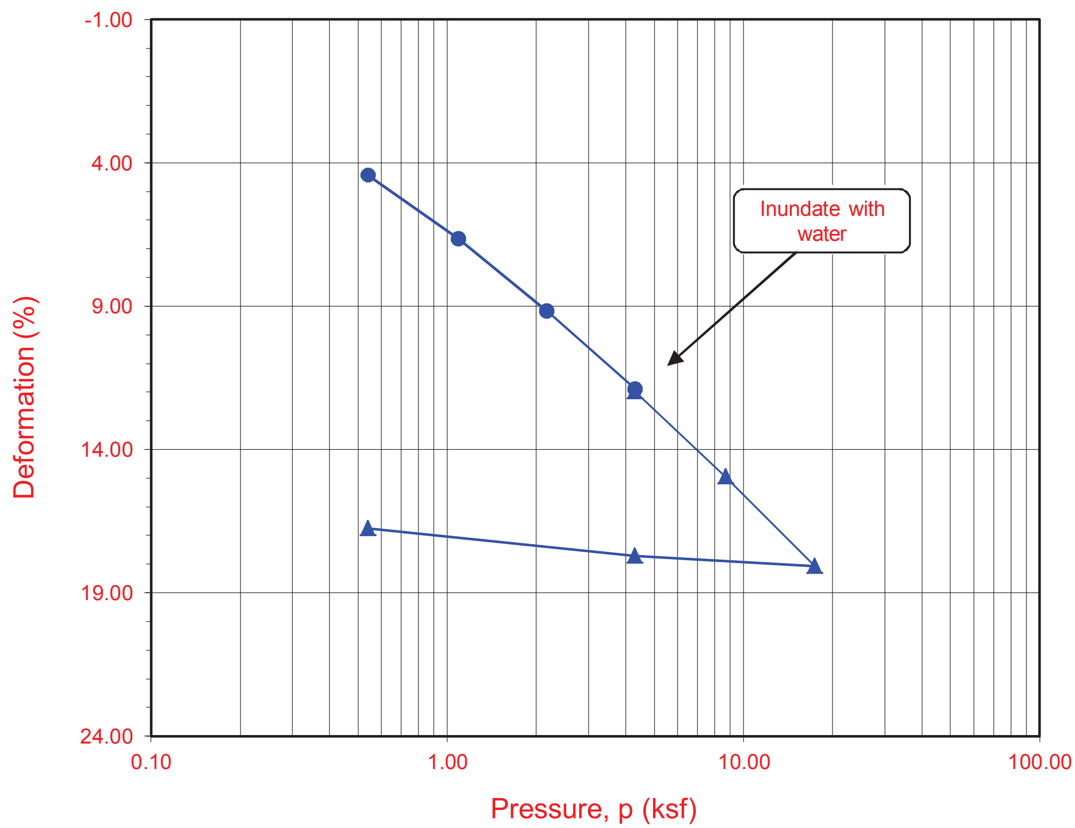
NCM / REGAL

06-15

Time Readings @ 0 kPa







Boring No.	Sample No.:	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-3	**R-6**	**20**	**25.1**	**14.7**	**100.5**	**120.8**	**0.602**	**0.396**	**100**	**100**

Sample Description:

ML: BROWN SILT

Project No.: 601586.008

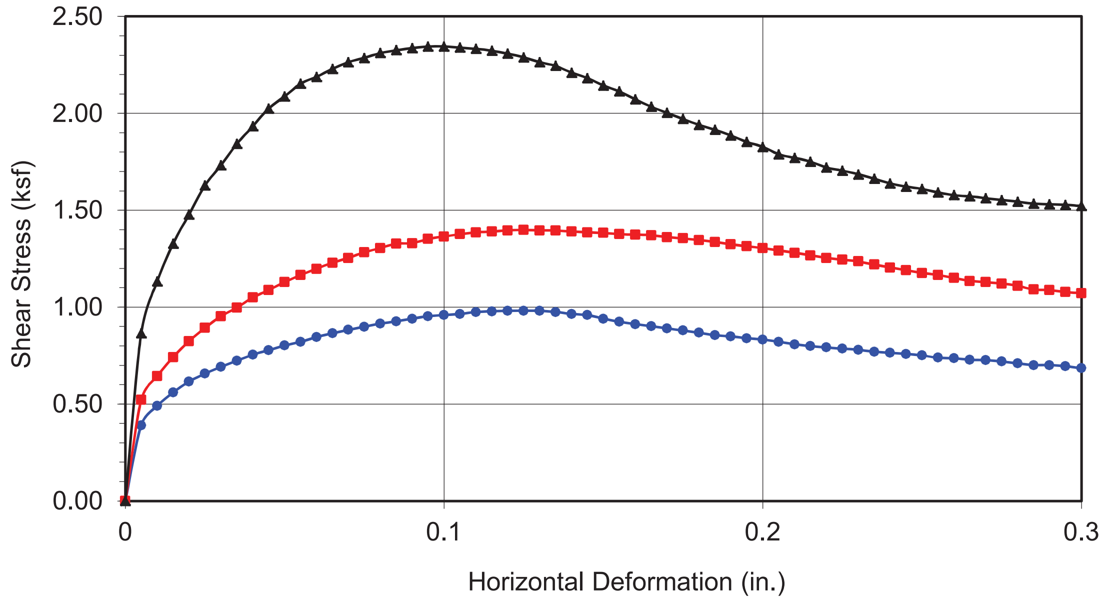
Project Name: NCM / REGAL

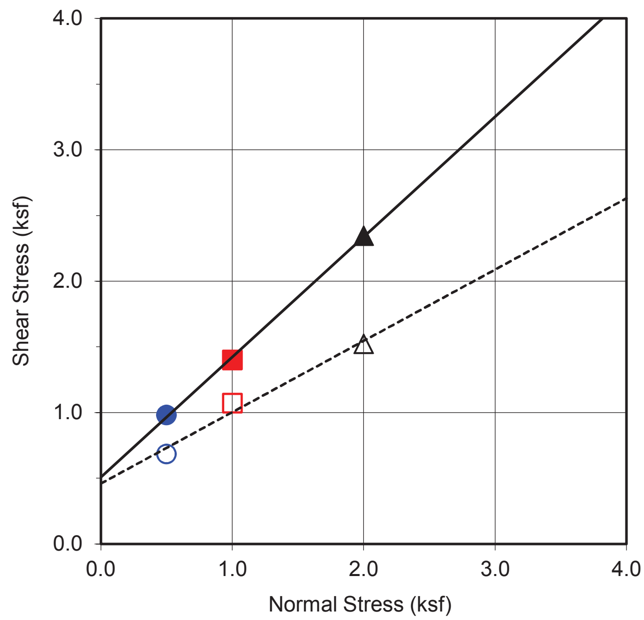
Leighton

ONE - DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Rev. 10-14





Boring No.	**B-1**
Sample No.	**R-2**
Depth (ft)	**10**
Sample Type:	RING
Soil Identification: SC: DARK BROWN CLAYEY SAND	

Strength Parameters

	C (psf)	ϕ (°)
Peak	508.0	42.4
Ultimate	460.0	28.5

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.981	■ 1.399	▲ 2.345
Shear Stress @ End of Test (ksf)	○ 0.685	□ 1.072	△ 1.522
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	12.00	12.00	12.00
Dry Density (pcf)	119.5	120.4	123.0
Saturation (%)	78.9	81.1	87.6
Soil Height Before Shearing (in.)	0.9960	0.9916	0.9894
Final Moisture Content (%)	16.6	15.7	13.6

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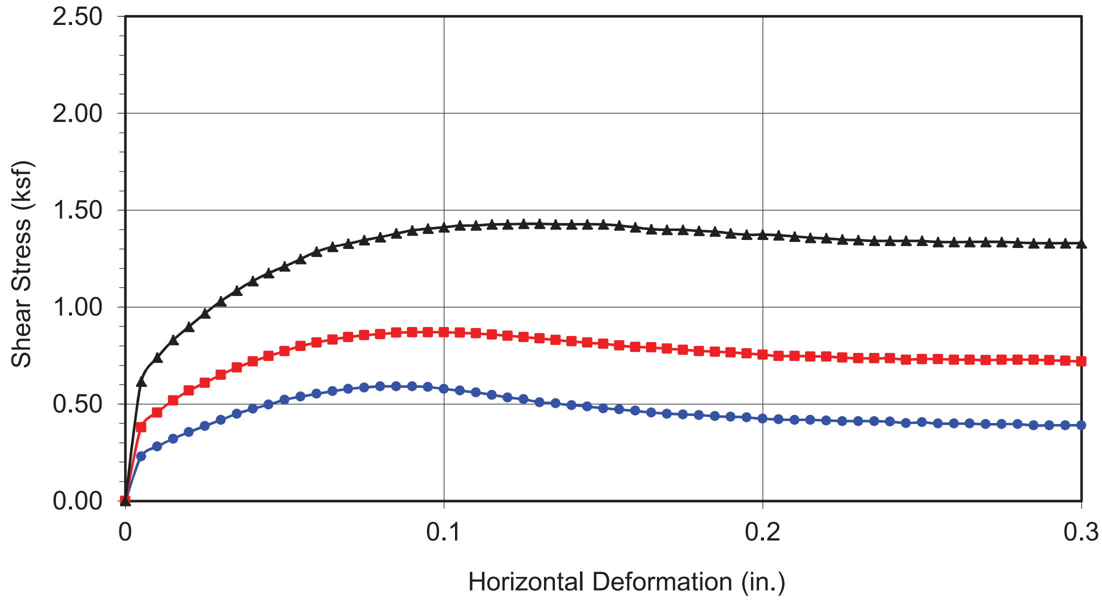
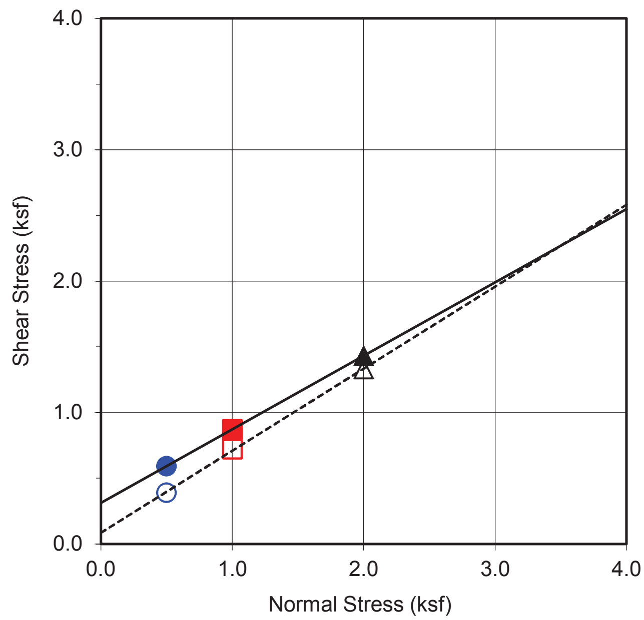
DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 601586-008

NCM / REGAL

06-15





Boring No.	**B-3**
Sample No.	**R-3**
Depth (ft)	**15**
Sample Type:	RING
Soil Identification: SC: DARK GRAYISH BROWN CLAYEY SAND	

Strength Parameters

	C (psf)	ϕ (°)
Peak	311.5	29.2
Ultimate	85.0	32.0

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.591	■ 0.871	▲ 1.430
Shear Stress @ End of Test (ksf)	○ 0.390	□ 0.720	△ 1.330
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	25.14	25.14	25.14
Dry Density (pcf)	98.6	98.5	99.2
Saturation (%)	95.6	95.4	97.0
Soil Height Before Shearing (in.)	0.9877	0.9797	0.9608
Final Moisture Content (%)	24.6	23.9	23.2

Leighton

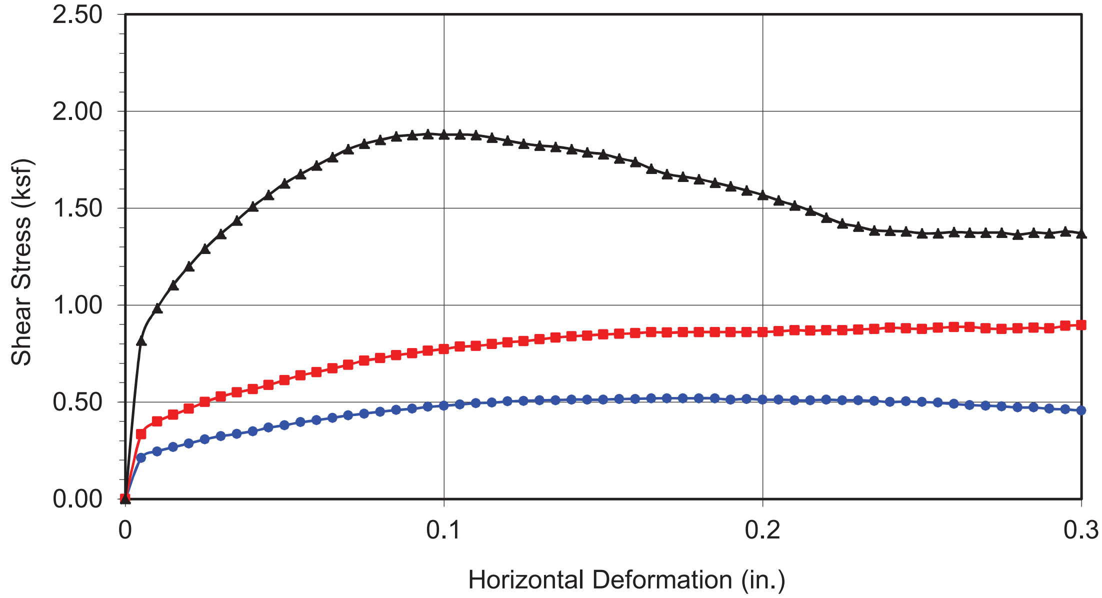
DIRECT SHEAR TEST RESULTS

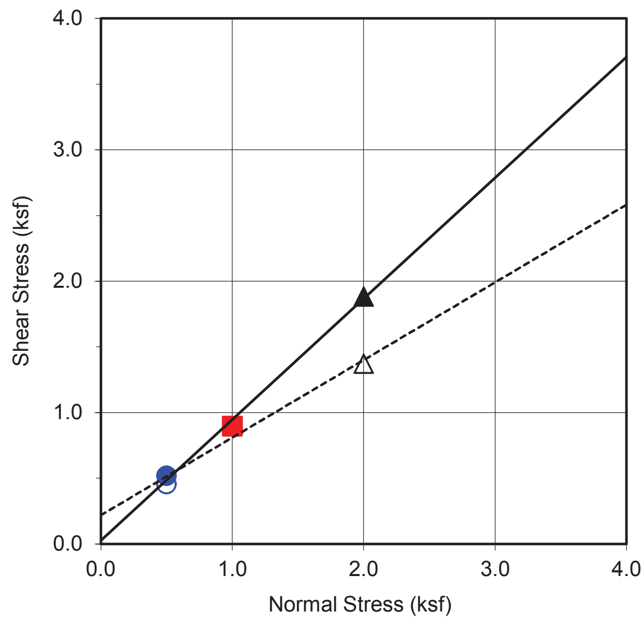
Consolidated Undrained

Project No.: 601586-008

NCM / REGAL

06-15





Boring No.	**B-3**
Sample No.	**R-5**
Depth (ft)	**30**
Sample Type:	RING

Soil Identification:
SC-SM: OLIVE GRAY SILTY, CLAYEY SAND

Strength Parameters

	C (psf)	ϕ (°)
Peak	25.5	42.6
Ultimate	218.5	30.6

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.519	■ 0.896	▲ 1.883
Shear Stress @ End of Test (ksf)	○ 0.456	□ 0.896	△ 1.371
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	23.92	23.92	23.92
Dry Density (pcf)	99.6	99.9	104.5
Saturation (%)	93.2	94.0	105.3
Soil Height Before Shearing (in.)	0.9683	0.9562	0.9767
Final Moisture Content (%)	26.0	22.5	19.4

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DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 601586-008

NCM / REGAL

06-15

Leighton

R-VALUE TEST RESULTS

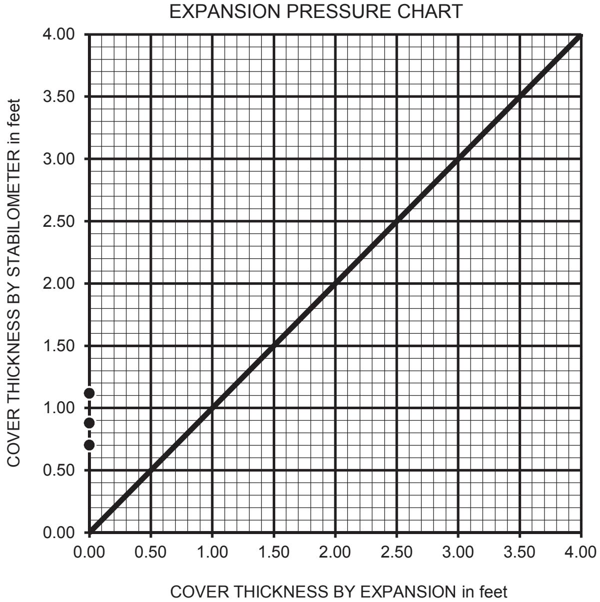
DOT CA Test 301

PROJECT NAME:	NCM / REGAL	PROJECT NUMBER:	601586.008
BORING NUMBER:	B-2	DEPTH (FT.):	2-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	SF
SAMPLE DESCRIPTION:	SC-SM: BROWN SILTY, CLAYEY SAND	DATE COMPLETED:	6/4/2015

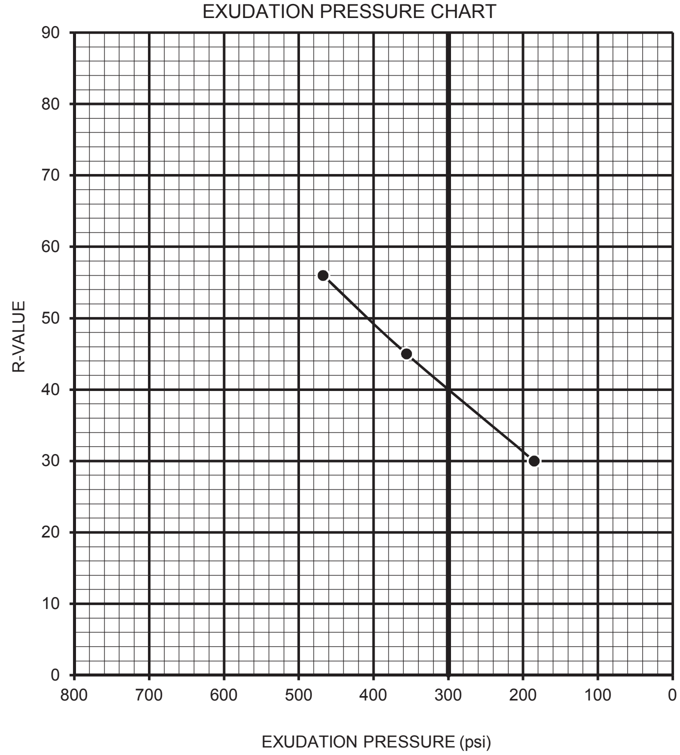
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.6	12.0	13.0
HEIGHT OF SAMPLE, Inches	2.48	2.47	2.47
DRY DENSITY, pcf	107.6	126.2	125.5
COMPACTOR PRESSURE, psi	175	125	50
EXUDATION PRESSURE, psi	467	356	185
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	59	71	100
TURNS DISPLACEMENT	3.40	3.85	3.58
R-VALUE UNCORRECTED	56	45	30
R-VALUE CORRECTED	56	45	30

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.70	0.88	1.12
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	40
EQUILIBRIUM R-VALUE:	40

Boring No.	B-1	B-1						
Sample No.	R-3	R-6						
Depth (ft.)	15.0	30.0						
Sample Type	RING	RING						
Visual Soil Classification	SC	SM						
Moisture Correction								
Wet Weight of Soil + Container (g)	100.1	100.2						
Dry Weight of Soil + Container (g)	79.8	78.6						
Weight of Container (g)	0.0	0.0						
Moisture Content (%)	25.4	27.5						
Container No.:	**	**						
Sample Dry Weight Determination								
Weight of Sample + Container (g)	100.1	100.2						
Weight of Container (g)	0.0	0.0						
Weight of Dry Sample (g)	79.8	78.6						
Container No.:	**	**						
After Wash								
Dry Weight of Sample + Container (g)	43.5	47.1						
Weight of Container (g)	0.0	0.0						
Dry Weight of Sample (g)	43.5	47.1						
% Passing No. 200 Sieve	**45**	**40**						
% Retained No. 200 Sieve	55	60						

PERCENT PASSING No. 200 SIEVE
ASTM D 1140

Leighton

Project Name: NCM / REGAL
Project No.: 601586.008

Tested By: BCC/SN Date: 6/18/2015

Rev. 10-04

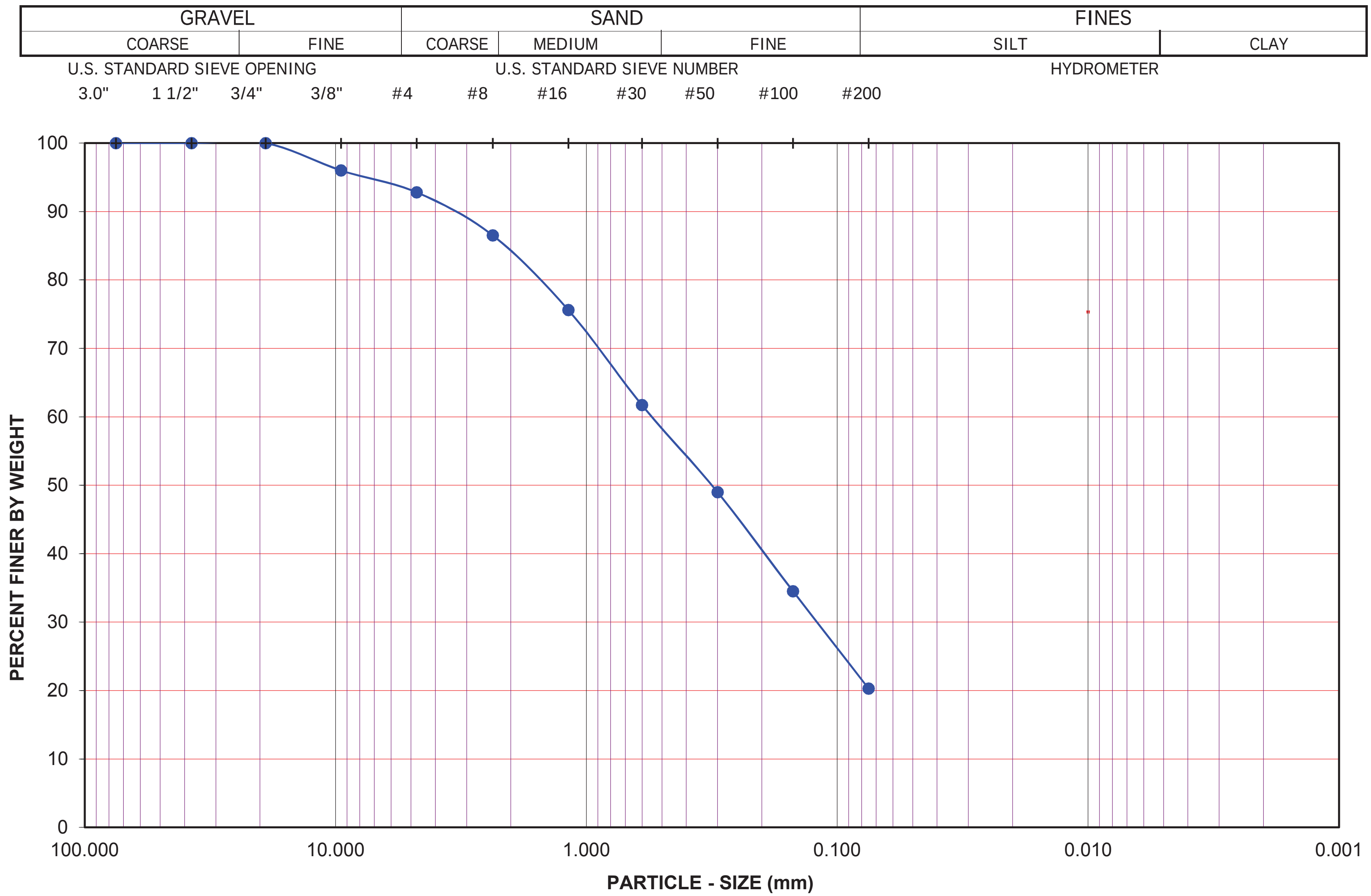
200 WASH B-1,R3~B-1,R6

Boring No.	B-3	B-3							
Sample No.	R-3	R-5							
Depth (ft.)	15.0	30.0							
Sample Type	RING	RING							
Soil Identification	SC: DARK GRAYISH BROWN CLAYEY SAND	SC-SM: OLIVE GRAY SILTY, CLAYEY SAND							
Moisture Correction									
Wet Weight of Soil + Container (g)	0.0	0.0							
Dry Weight of Soil + Container (g)	0.0	0.0							
Weight of Container (g)	1.0	1.0							
Moisture Content (%)	0.0	0.0							
Sample Dry Weight Determination									
Weight of Sample + Container (g)	722.6	592.1							
Weight of Container (g)	247.2	204.4							
Weight of Dry Sample (g)	475.4	387.7							
Container No.:									
After Wash									
Method (A or B)	B	B							
Dry Weight of Sample + Cont. (g)	501.5	474.8							
Weight of Container (g)	247.2	204.4							
Dry Weight of Sample (g)	254.3	270.4							
% Passing No. 200 Sieve	**46.5**	**30.3**							
% Retained No. 200 Sieve	53.5	69.7							

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: NCM / REGAL
Project No.: 601586-008
Client Name: WESTFIELD CORPORATION
Tested By: GB Date: 06/08/15



Project Name: MISSION GORGE ROAD IMPROVEMENTS

Project No.: 11055.001

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

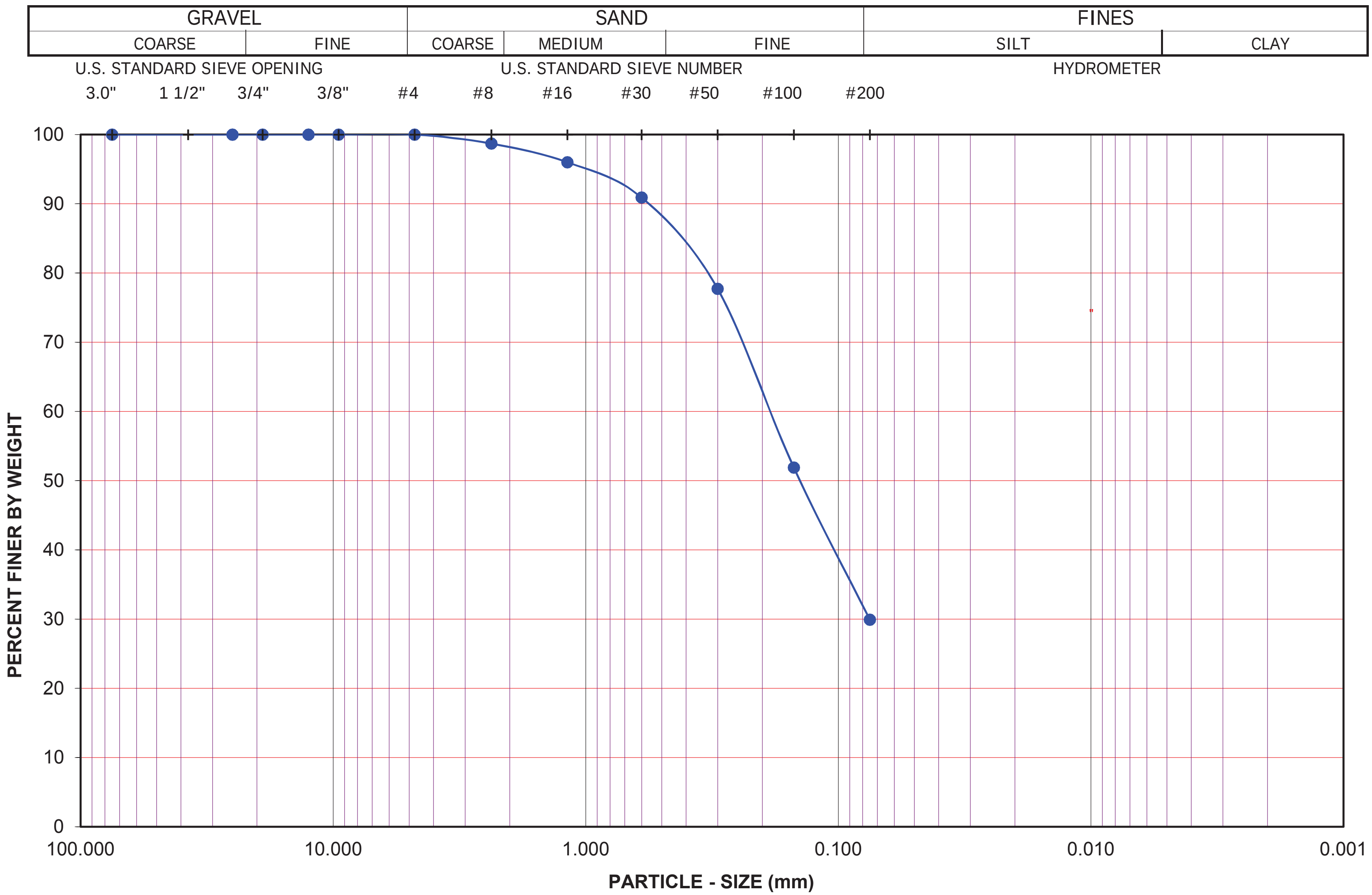
Exploration No.: B-1 Sample No.: B-2

Depth (feet): 2.0-5.0 Soil Type : SM

Soil Identification: SM: BROWN SILTY SAND WITH FEW GRAVEL

GR:SA:FI : (%) 7 : 73 : 20

Jul-15



Project Name: NCM / REGAL

Project No.: 601586.008

Exploration No.: B-1 Sample No.: R-8

Depth (feet): 45.0 Soil Type : SM

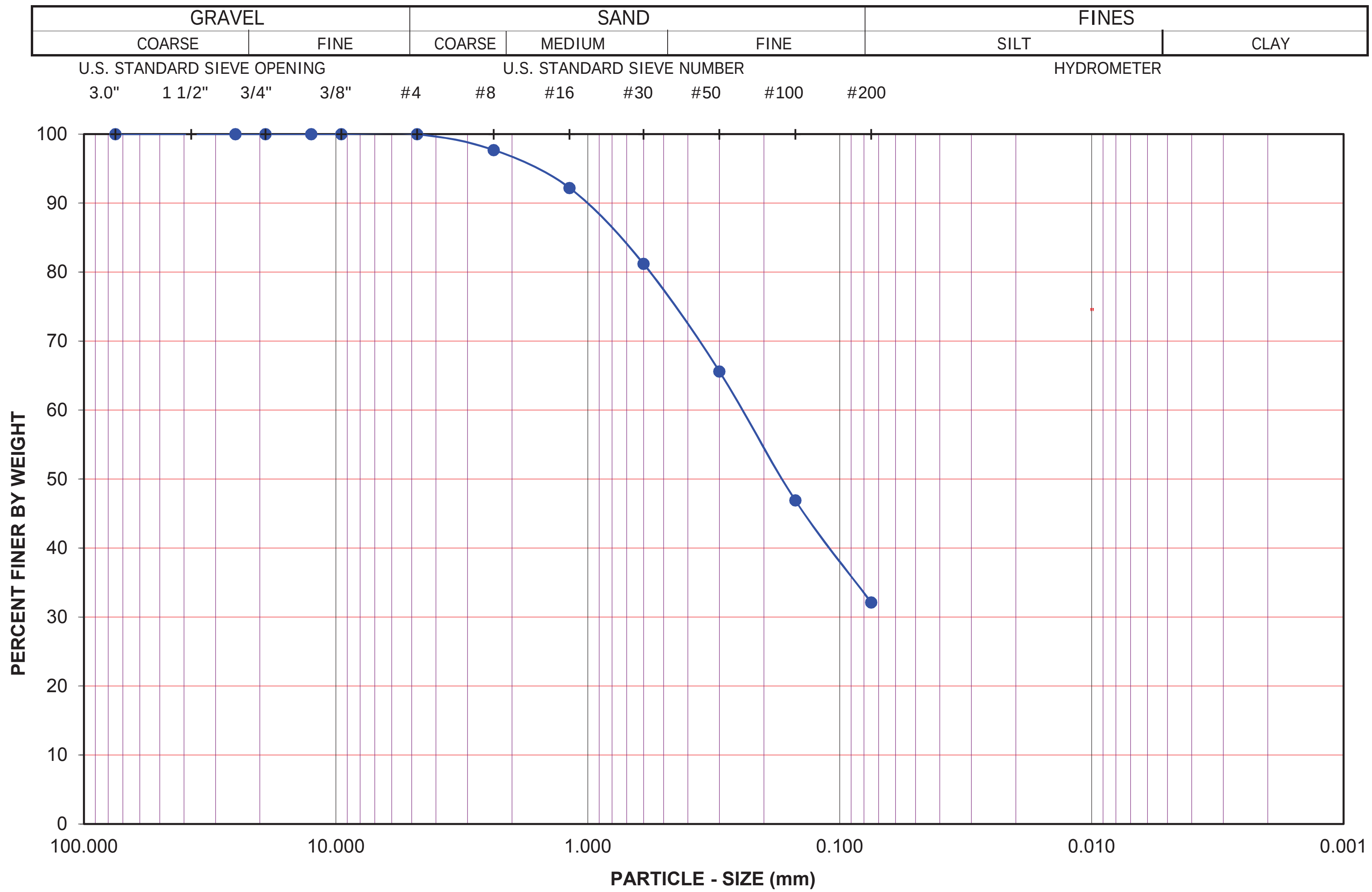
Soil Identification: SM: BROWN SILTY SAND

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 70 : 30

Jun-15



Project Name: NCM / REGAL

Project No.: 601586.008

Exploration No.: B-2

Sample No.: R-3

Depth (feet): 15.0

Soil Type : SM

Soil Identification: SM: GRAYISH-BROWN SILTY SAND

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 68 : 32

Jun-15

APPENDIX D

Liquefaction Analysis

Leighton
San Diego, CA

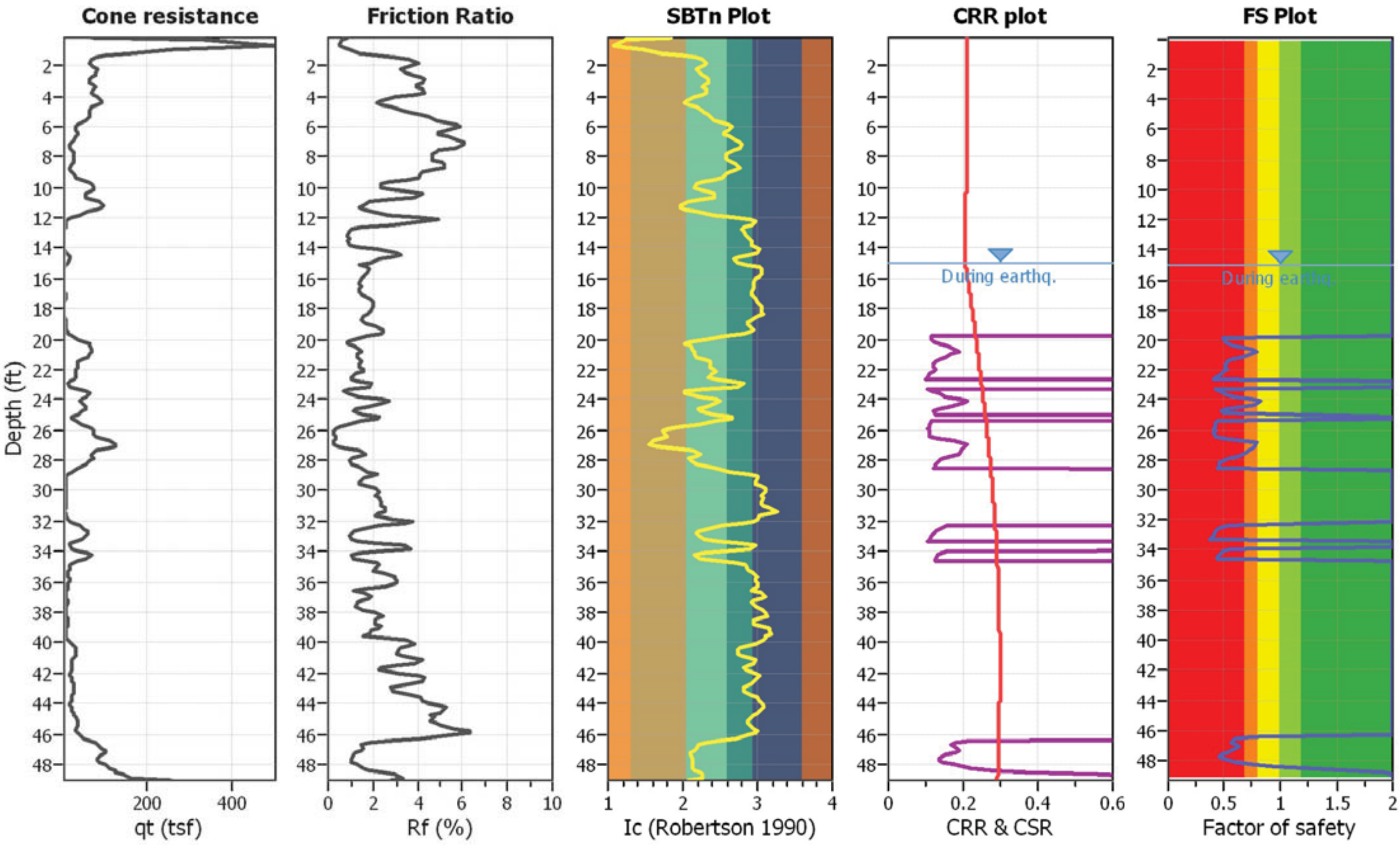
LIQUEFACTION ANALYSIS REPORT

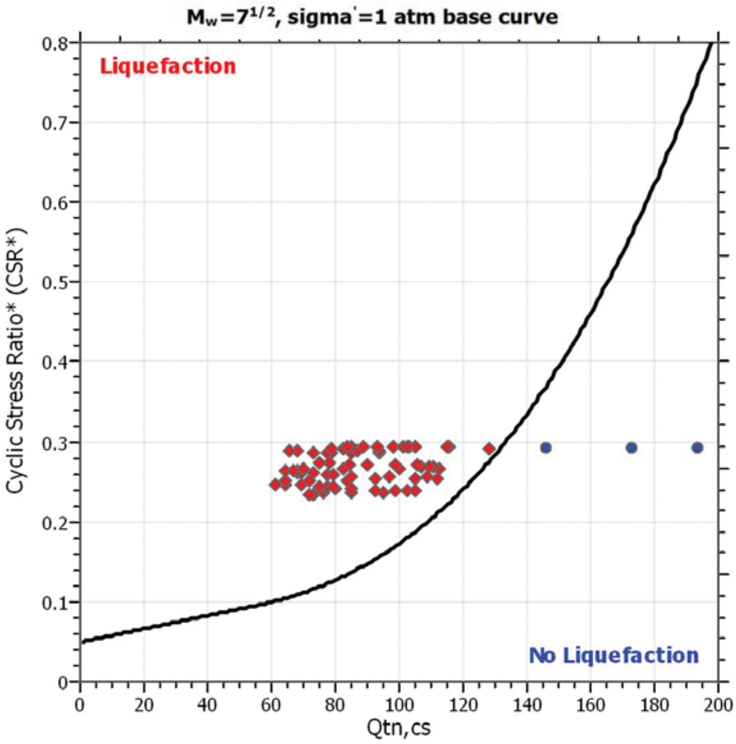
Project title : Westfield NCM Regal **Location : Escondido**

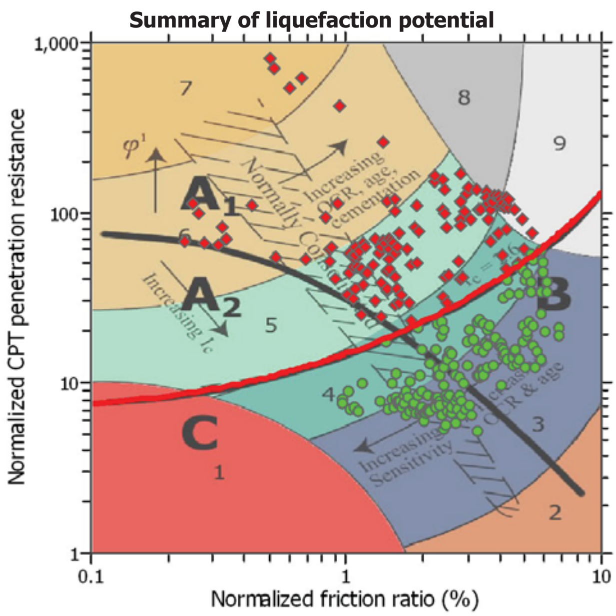
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.42	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







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