

Appendices

Appendix N: Geotechnical Study

Appendices

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Report of

PRELIMINARY GEOTECHNICAL INVESTIGATION

Proposed Brea Mall Development

1066 Brea Mall
Brea, California 92821

Prepared for

Simon

225 West Washington Street
Indianapolis, Indiana 46204

Prepared by

Professional Service Industries, Inc.

11980 Telegraph Road
Unit 104
Santa Fe Springs, California

February 27, 2018

PSI Project No. 00661261

February 27, 2018

Mr. Clif Poynter

Senior Civil Engineer

Simon

225 West Washington Street

Indianapolis, Indiana 46204

Subject: **Preliminary Geotechnical Investigation**
Proposed Brea Mall Development
1066 Brea Mall
Brea, California 92821
PSI Project No. 00661261

Dear Mr. Poynter;

As requested, Professional Service Industries, Inc. (PSI), an Intertek Company, is pleased to provide this report summarizing our Preliminary Geotechnical Investigation for the above referenced project in Brea, California. This report includes the results of our review of published and unpublished geologic information, subsurface exploration, laboratory testing, and our preliminary conclusions and recommendations as they pertain to the proposed development of the property.

We appreciate the opportunity to perform this Preliminary Geotechnical Investigation and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.



Douglas T. Abernathy, PE, GE
Senior Project Engineer



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Principal Consultant

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1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI) has completed a Preliminary Geotechnical Investigation for the proposed Brea Mall Development to be located in the southwest corner of the Brea Mall property in Brea, California. Our work was performed in general accordance with our proposal revised January 22, 2018 and signed February 2, 2018.

1.2 PROJECT DESCRIPTION

PSI understands that conceptually the site is proposed to be improved with several buildings ranging from 3,000 square feet (sf) to 120,000 sf along with a 5 to 6-Level parking garage holding 1,000 to 1,300 vehicles. Development of the site will also include paving, hardscape, landscaping, utilities, possibly a pool, and various other improvements. The existing Sears store is proposed to be demolished. Grading plans have not been developed but we assume grades will remain relatively level with minor adjustments of less than ± 2 feet as the site consists of an active mall and parking lots. The proposed developments may have maximum column loadings of roughly 150 to 200 kips for one-story, 200 to 250 kips for two-story, and 300 to 400 kips for three-story structures. The parking structure may induce loadings of 750 to 1,000 kips. Wall loadings are anticipated to be 3 to 15 kips/ft. Shallow foundations including slab-on-grade construction is anticipated. Slabs may support loadings of up to 150 pounds per square foot.

The preliminary geotechnical recommendations presented in this report are based on the available project information, site location, laboratory testing, and the subsurface materials encountered. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes to the project. We understand that future, more detailed plans and designs are proposed and therefore, more detailed geotechnical investigations will need to be performed for select buildings.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this preliminary geotechnical investigation was to explore the subsurface conditions and provide suitable preliminary design recommendations for the proposed developments. The investigation for this project involved drilling six geotechnical soil borings, perform percolation testing in two of the borings, laboratory testing, and geotechnical analyses. This report briefly outlines the testing procedures, presents available project information,

describes the site and subsurface conditions, and presents preliminary geotechnical recommendations for the following:

- Seismic hazards on the property;
- Site preparation and grading;
- General foundation types, depths, allowable bearing capacities, and estimates of settlement;
- Concrete floor slabs;
- Paving recommendations;
- Retaining wall recommendations;
- Percolation test results; and
- Comments regarding factors that may impact construction and performance of the proposed development.

This report is **not** intended to be used for final design purposes. Additional geotechnical investigations will be necessary based on future grading plans and foundation plans proposed for the development.

The scope of services did not include environmental assessments for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes. PSI can perform Phase I/ESA, asbestos, lead, and mold surveys of the existing site improvements and property if desired with an additional budget.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE LOCATION AND DESCRIPTION

The project site is situated at 1066 Brea Mall in Brea, California in the southwest corner of the mall property. The proposed development spans across an area of roughly 800-feet by 800-feet. The property is bounded to the west by Randolph Avenue, to the south by other commercial developments and Imperial Highway and State College Boulevard, to the east by parking areas, and to the north and east by the main mall development. At the time of our fieldwork, the site consisted of an existing two to three-story Sears store with surface parking areas, drive aisles, landscaping, hardscaping, and a roughly 15-foot high slope. The site appears to slope to the southwest with elevations (estimated from Google Earth) ranging from roughly 330 to 350 feet above mean sea level. The site is located at approximate latitude 33.9137° and longitude -117.8883°. A Site Vicinity Map showing the site location is included as Figure 1.

2.2 PREVIOUS INVESTIGATIONS

Previously Converse Consultants Orange County (Converse) performed several geotechnical investigations for the mall development as listed below:

Table 1 – Converse Consultants Reports

Year	Report
1974	Original geotechnical report for the mall development.
1974 – 1978	Grading reports for earthwork.
1975	Fill compaction reports.
1977	Fill compaction reports.
1978	Geotechnical investigation for Nordstrom.
1980	Geotechnical investigation for restaurant on perimeter access road on State College Boulevard.
1984	Geotechnical investigation for repaving perimeter road, south and east.
1985	Geotechnical investigation for hotel and parking structure on Bristol St.
1986	Geotechnical investigation for restaurant on perimeter access road on State College Boulevard.
1987	Geotechnical investigation for mall expansion to northwest and southeast.

The 1987 report describes encountering compacted fill and native terrace deposits. Soils encountered by Converse predominantly consisted of sandy clay beneath the site. Other soils were also encountered including silty sand, clayey sand, clay, silt, and sandy silt. The majority of the fill soils were encountered southwest of the Macy's store (5 to 33 feet bgs). Relatively moderate amounts of fill were encountered beneath the parking structure west of Nordstrom (0 to 13 feet bgs). Groundwater was described by Converse as being deeper than 50 feet bgs (Orange County Water District, 1976) with perched water encountered at depths ranging from 23½ to 42½ feet bgs. Existing pavements consisted of roughly 3 inches of asphalt concrete over 4 inches of aggregate base. Sulfate testing of the soils indicated 160 to 224 ppm. Expansion index testing (generally 40 to 50) indicated low to medium expansion potential. R-values ranged from 6 to 13 for sandy clay and clayey sand respectively.

2.3 REGIONAL GEOLOGY

The site is located within the La Habra 7½ minute quadrangle within the Los Angeles Basin Geomorphic Province boundary. Based on a review of the Dibblee Geologic Map of the area (2001), the site is underlain by elevated older alluvial deposits. Areas nearby (outside the proposed development area) to the southwest and southeast consist of younger alluvium that is considered potentially liquefiable.

2.4 REGIONAL SEISMICITY

The project site is located in Southern California, which has undergone a complex multiphase structural history and remains an active tectonic region with documented historic earthquakes. Generally, the seismicity within California can be attributed to faulting due to regional tectonic movement. This includes the San Andreas Fault and other sub-parallel strike-slip faults, as well as normal and thrust faulting within the State. The area of the subject site is considered seismically active. Seismic hazards within the area can be attributed to potential ground shaking resulting from earthquake events along nearby or more distant faulting.

2.5 SEISMIC HAZARD ASSESSMENT

2.5.1 Alquist-Priolo Fault Zone

The seismicity of the site was evaluated utilizing deterministic methods for active faults within the regional vicinity. According to the Alquist-Priolo Special Studies Zones Act of 1972 (revised

1994) faults have been classified as active faults which show apparent movement during the last 11,000 years (i.e., Holocene time). The site is not located within a currently designated Earthquake Fault Zone per the Alquist-Priolo Special Studies Zone Map produced by the California Geological Survey (CGS). Based on CGS mapping, the closest active fault is the Whittier Fault (Elsinore fault zone, Whittier section) located approximately 1.7 miles to the northeast from the site.

2.5.2 Liquefaction-Induced Settlement

The primary causes of damage in this general area during seismic events include ground shaking and liquefaction of the subsurface strata. Liquefaction occurs when loose granular and low plastic materials below the groundwater table are subjected to cyclic shear forces resulting from seismic events. Water pressure within the pore spaces of the soil increases with a corresponding decrease in the soil's effective stress and individual grains of soil lose contact with each other. Excess pore pressures ultimately dissipate and the soil consolidates, often causing significant total and differential settlement of the ground surface.

Liquefaction potential is greatest where the ground water level is shallow and submerged loose fine sands and silts and soft low plasticity clays occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

A review of the California Geologic Survey (CGS) map indicates that the site is not in an area designated as having a liquefaction potential. Areas of potentially liquefiable younger alluvium exist to the southwest and southeast offsite. Liquefaction analyses were performed onsite at one boring location due to high groundwater levels and questionable soil strengths. Our liquefaction analysis indicates that a zone of poorly graded sand between depths of 33 to 37 feet bgs may liquefy during the design earthquake with high groundwater levels. Seismic settlements induced from such liquefaction may result in 0.7-inch of displacement with an estimated differential settlement of less than ½ inch over a 40 foot span. Our liquefaction analysis is provided in Appendix D.

We do not believe lateral spreading or surficial disturbances will be an issue at this site due to potential liquefaction.

2.5.3 Landsliding

A review of the CGS map indicates that the site is not in an area designated as having a landslide potential. Due to the generally flat nature of the site and surrounding properties, it is our opinion that the site is not susceptible to landsliding.

2.5.4 Tsunamis and Seiches

Inundation by tsunamis (seismic or "tidal waves") are not likely to be considered a significant threat to the subject site due to the distance from the coast. The elevation of the project site is also significantly above sea level. Based on there being no bodies of water near the site, the site is also not considered subject to seiches.

2.6 SUBSURFACE CONDITIONS

2.6.1 GENERAL

The boring locations were marked in the field by a PSI representative by referencing existing landmarks based on the information provided by the client. A truck-mounted drill rig and continuous flight, hollow-stem augers were used to advance the soil borings.

During the sampling procedure, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and relatively undisturbed samples were obtained in general accordance with ASTM D3550. The SPT for soil borings is performed by driving a split-spoon sampler, with an outside diameter of 2 inches, into the undisturbed formation located at the bottom of the advanced borehole with repeated blows of a 140-pound hammer falling a vertical distance of 30 inches. The number of blows required to drive the sampler the last 12 inches of an 18-inch penetration depth is a measure of the soil consistency (N-value). If 50 blows are performed before reaching the end of a 6 inch increment this is referred to as refusal and the number of blow per inches driven is recorded, i.e. 50/4 inches. For ASTM D3550 (California Modified Sampler) the split barrel sampler possesses a 3-inch O.D. and is driven in the same manner as the SPT. The field blow counts obtained from the California Modified sampler, as indicated in the boring logs, were adjusted by 63 percent to obtain a rough correlation to SPT blow counts (SPT-N value). Samples were identified in the field, placed in sealed containers and transported to the laboratory for further classification and testing.

2.6.2 SOIL BORINGS

The subsurface conditions were explored by drilling six soil borings (B1 through B6) at the site for geotechnical exploration purposes. Two borings (B1 and B2) were drilled closer to the mall for correlations with previous borings performed by Converse Consultants in 1987. Two borings (B3 and B4) were drilled in the southwest region of the proposed development to depths of approximately 51½ feet below the ground surface (bgs) for purposes of liquefaction analysis. Percolation testing was also performed near these two borings (B3 and B4) to depths of 10 feet bgs. Two borings (B5 and B6) were drilled to depths of 31½ feet at mid-elevation portions of the site for general geotechnical analysis. Figure 2 shows the approximate boring locations. The soil types encountered at the specific boring locations are presented on the attached Boring Logs provided in Appendix A.

As indicated on the boring logs, the subsurface soils consisted of many types of soils ranging from sands to silts to clays and combinations of each. Coarse grained soils had relative densities ranging from loose to very dense while fine grained soils had consistencies ranging from soft to hard. Fill was encountered at boring locations B1 (2.3 feet bgs) and B2 (2½ feet bgs). Possible fill was encountered at boring locations B1 (2.3 to 4.7 feet bgs), B3 (1½ to 7 feet bgs), and B4 (0.8 to 5 feet bgs). Near surface loose and soft soils were encountered at boring locations B1 (1½ to 9 feet bgs) and B3 (4.7 to 7 feet bgs).

The stratification presented on the boring logs is based on a visual examination of the recovered soil samples and the interpretation of field logs by a geotechnical professional. Included on the boring logs are the standard penetration resistances (SPT N-values and California Modified sampler blows) recorded in the individual borings at standard testing intervals to the boring termination depths.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs, included in Appendix A, should be reviewed for specific information at individual boring locations. These records include soil descriptions, stratification, penetration resistance, locations of the samples and laboratory test data. The stratification shown on the boring logs represents the conditions only at the actual boring locations at the time of our exploration. Variations may occur and should be expected between boring locations. The stratification that represents the approximate boundary between subsurface materials and the actual transition may be gradual.

2.7 GROUNDWATER

Groundwater was encountered in two of our soil borings during our investigation at depths ranging from approximately 25 feet bgs (at boring location B4) to 30 feet bgs (at boring location B3). These depths are approximate as the drilling operations continued and monitoring equipment was not installed. Groundwater is not expected to impact the proposed construction. It is possible, however, that transient, saturated ground conditions at shallower depths could develop at a later time due to periods of heavy precipitation, landscape watering, leaking water lines, or other unforeseen causes. Variations in groundwater levels should be expected seasonally, annually, and from location to location.

2.8 LABORATORY TESTING

The soil samples obtained during the field exploration were transported to our laboratory and selected soil samples were tested in the laboratory to determine the material properties for evaluation. Laboratory testing on selected samples included moisture content (ASTM D2216), density (ASTM D2937), grain size distribution (ASTM D422), Atterberg Limits (ASTM D4318), expansion index (ASTM D4829), R-value (CTM 301), consolidation (ASTM D2435), direct shear (ASTM D3080), and corrosion testing [soluble sulfate (CTM 417), soluble chloride (CTM 422), pH (ASTM D4972), and minimum resistivity (CTM 643)]. Laboratory testing was performed in general accordance with ASTM and Caltrans procedures. Unless otherwise informed, the soil samples will be discarded 60 days from the issuance of this report. Laboratory test results are included in Appendix B.

2.9 PERCOLATION TESTING

A limited program of in-situ percolation testing was performed at two boring locations [B3(P1) and B4(P2)] as shown on Figure 2. Testing was performed in general accordance with the Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans for the Santa Ana Regional Water Quality Control Board (September, 2013). Results of the testing are provided below in Table 1 and are attached in an Appendix C.

Table 2 – Percolation Test Results

Test Location	Raw Infiltration Rate (in/hr)	Design Infiltration Rate (in/hr)
P1 (B3)	0.0	0.0
P2 (B4)	2.8	1.4

Measured percolation rates were reduced to determine design values that will represent long-term performance of the proposed infiltration BMPs. The recommended factor values used were obtained from the Technical Guidance Document referenced above. We have assumed that maintenance of the designed system will be performed regularly. The following assumptions were made:

Table 3 – Percolation Assumptions

Factor Category	Factor Description	Factor Value Location P1/B3	Factor Value Location P2/B4
Suitability Assessment	Soil Assessment Methods	2	2
	Predominant Soil Texture	3	1
	Site Soil Variability	4	2
	Depth to Groundwater or Impervious Layer	1	1
Design	Tributary Area Size	1	1
	Level of Pretreatment	1	1
	Redundancy	1	1
	Compaction During Constr.	1	1
Combined FS	$\sum S_{Total}$	2	1.5
Measured Rate	(in/hr)	0.0	2.8
Design Rate	(in/hr)	0.0	1.4

Use a minimum FS of 2 for design

Please note that percolation/infiltration rates are representative of the location and depth at which the tests were conducted. If infiltration is to occur at different locations or depths, different rates may exist and further testing will be necessary. Individual counties and

municipalities may have stricter guidelines and applicable correction factors for infiltration rates and, if applicable, should be utilized for design. Rates will vary depending on head applied. For calculating “time to empty”, the design infiltration rates above should be reduced by half.

The Technical Guidance Document requires that testing be performed at several locations within the proposed drainage area depending on the quantity of design flow anticipated and the size of the BMP. For slow flow areas, tests are required to be performed over several days. The limited testing performed for this investigation should be repeated several times over several days at the proposed BMP location to adequately characterize the site soils in question for infiltration purposes. Testing and design factors assumed above are subject to change and may not be representative of actual conditions anticipated for the development.

3 CONCLUSIONS AND RECOMMENDATIONS

The subject property is considered a suitable site for the proposed development from a geologic and soils engineering standpoint. We believe that development of the site will not cause adverse effects on adjacent properties. It is our opinion that the proposed facility can be constructed provided the recommendations stated in the report are followed. Test findings and statements of professional opinion do not constitute a guarantee or warranty, expressed or implied.

The following preliminary geotechnical design recommendations have been developed on the basis of the previously described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, PSI should be contacted to determine if modifications to the recommendations are warranted.

Based on the results of the fieldwork, laboratory evaluation, and engineering analyses, the site can be adapted for the proposed facility. Based on the observed site conditions PSI believes that conventional spread footings are suitable to support the proposed structure. Details related to site preparation, foundation design, and construction are presented in the following sections.

3.1 EARTHQUAKE AND SEISMIC DESIGN CONSIDERATION

We have employed the 2016 California Building Code (CBC), the locally adopted version of the International Building Code, 2015 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site.

As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. PSI's deepest boring extended to 51 ½ feet bgs, but to define the Site Class for this project, we have interpreted the results of soil test borings drilled within the project site and estimated appropriate soil properties below the base of the borings to a depth of 100 feet as permitted by the code. The estimated soil properties were based upon data available in published geologic reports as well as our experience with subsurface conditions in the general site area.

Based upon our evaluation, the subsurface conditions within the site are consistent with the characteristics of a **Site Class "D"** as defined in Chapter 20.3-1 of the ASCE 7-10. The associated probabilistic ground acceleration values and site coefficients for the general site area were

obtained from the USGS geohazards web page, <http://earthquake.usgs.gov/designmaps/us/application.php>, and are presented in the table below.

Table 4 - Ground Motion Values*

Period (sec)	Mapped MCE Spectral Response Acceleration** (g)		Site Coefficients		Adjusted MCE Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	2.018	F_a	1.0	S_{Ms}	2.018	S_{Ds}	1.345
1.0	S_1	0.741	F_v	1.5	S_{M1}	1.111	S_{D1}	0.741

*2% Probability of Exceedance in 50 years

MCE = Maximum Considered Earthquake

The Site Coefficients, F_a and F_v presented in the above table were also obtained from the noted USGS webpage as a function of the site classification and mapped spectral response acceleration at the short (S_s) and 1-second (S_1) periods, but can also be interpolated from ASCE Tables 11.4-1 and 11.4-2.

Considering the use of the proposed structures and the values of S_{Ds} and S_{D1} shown above, we expect the structures to fall within **Seismic Design Category D**, as defined by ASCE Tables 11.6-1 and 11.6-2 of the code. Design of structures should comply with the requirements of the governing jurisdiction's building codes and standard practices of the Structural Engineering Association of California.

3.2 SITE PREPARATION & GRADING

The following recommendations are provided for general grading of the site. More detailed geotechnical investigations may be required depending on the proposed development.

3.2.1 SITE PREPARATION

The site should be cleared of all man-made structures, surface vegetation, utilities, and construction debris. Any previous utilities, foundations, seepage pits, or other site improvements found on site in conflict with the new construction should be removed or abandoned in place

below the proposed structures. As a minimum, it is recommended the clearing operations extend at least five feet beyond the building and a few feet beyond the development perimeters.

We recommend that at the time of initial site stripping and grading, that PSI be retained to observe proof-rolling of excavations and subgrade conditions to verify that no potentially deleterious soft soils or unforeseen conditions are present. All excavation bottoms should be firm and unyielding prior to fill being placed above.

Though PSI was not provided with project-specific grading plans, proposed cuts and fills are anticipated within the project site to establish design grades. Any new fill should be placed in accordance with the Engineered Fill section of this report.

If grading occurs in the winter rainy season, unstable subgrade conditions may be present. These conditions may require stabilizing the subgrade with by adding coarse aggregate or admixtures such as cement kiln dust. Isolated areas may be stabilized using a geogrid, such as Tensar TX160 or equal, with one foot compacted Class II aggregate base over the geogrid. Additional recommendations can be provided, as required, during construction.

Asphalt and concrete debris will likely be generated from demolition at the site during construction. These materials may require hauling offsite or may be crushed for placement onsite per geotechnical recommendations or local building authorities.

3.2.2 REMEDIAL GRADING

All undocumented fill materials, within the limits of proposed structures, should be removed and replaced as evaluated by the Geotechnical Engineer or their representative. Existing unsuitable fill soils extending to a depth of 2½ feet bgs near boring location B1 and B2 should be removed and replaced with compacted engineered fill. Areas near boring locations B1, B3, and B4 should be evaluated closely as some soils appear to possibly consist of fill materials to depths of roughly 5 to 7 feet bgs. Loose or soft soils should also be removed and replaced near boring location B1 to a depth of roughly 9 feet bgs. The soils exposed at the base of all excavations should also be scarified to a depth of at least 12 inches, be compacted as described below in Section 3.2.3.

Dependent upon the building locations and their structural loads, some over-excavation of the existing soil and replacement as a properly compacted engineered fill beneath footings may be necessary to limit static settlements to acceptable levels. The need for this over-excavation

should be determined at the time of more detailed investigations and based on specific structural loading information to be provided.

Site grades may then be raised with Engineered Fill to achieve the design elevations proposed for the site. A PSI representative should be on-site during site grading to evaluate the degree of compaction obtained by the contractor.

3.2.3 ENGINEERED FILL

Engineered Fill materials required at this site, including both native and import soil, should not contain rocks greater than 3-inches in diameter or have greater than 30 percent retained on the $\frac{3}{4}$ -inch sieve, and should not contain more than 3 percent (by weight) of organic matter or other unsuitable material. Engineered Fill soils should also have an Expansion Index (EI) that does not exceed 40. Based on our subsurface investigation, existing on-site soils are generally suitable for use as Engineered Fill provided they can be properly moisture conditioned to meet the compaction requirement. This should be confirmed by a PSI representative during grading. Import materials meeting the above requirements should be approved by the Geotechnical Engineer prior to importing to the site. Soil that are environmentally impacted should not be used as Engineered Fill onsite.

Engineered Fill should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). The moisture content of Engineered Fill should be maintained between 0 to 2 percent above the material's optimum moisture content during compaction. If the Engineered Fill is too dry, water should be uniformly applied across the affected fill area. If the Engineered Fill is too wet, it must be dried. Engineered Fill should be thoroughly mixed by disking, or other approved methods, to obtain relatively uniform moisture content throughout the lift immediately prior to compaction.

Engineered Fill should be placed in maximum lifts of 8-inches of loose material. Each lift of Engineered Fill should be tested by a PSI soils technician, working under the direction of our Project Geotechnical Engineer, prior to placement of subsequent lifts. If smaller compaction equipment such as jumping jacks or plate compactors are used thinner lifts may be required to achieve compaction.

Compaction of the backfill should be checked with a sufficient number of density tests by a representative of the Geotechnical Engineer to determine if adequate compaction is being

achieved by the contractor. The properly compacted Engineered Fill should extend horizontally outward beyond the exterior perimeter of the building and foundations a distance equal to the height of fill or 5-feet, whichever is greater, prior to significant sloping. In addition, Engineered Fill should extend horizontally outward beyond the exterior perimeter of pavements and hardscape a distance equal to the height of fill or 3-feet, whichever is greater, prior to significant sloping.

3.3 UTILITY TRENCH CONSTRUCTION

Utility trenches can be backfilled with on-site soils above the utility bedding and shading materials. Trench backfill should be moisture conditioned to between 0 to 2 percent above the optimum moisture content, compacted in 4- to 6-inch lifts to a minimum of 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). If rocks larger than 3-inches in maximum size are encountered, they should be removed from the fill material prior to placement in the utility trenches. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction, but typically require clean granular material having a sand equivalent equal to or above 30. Jetting or flooding of utility backfill is not recommended.

3.4 SOIL EXPANSION

The expansion potential of representative samples of the existing site soils is low (EI=34 & 35). Such soils must be kept at a constant moisture content to avoid shrinkage and swelling near improvements. Foundation and slab subgrades must be designed for such soils. Landscaping, irrigation, and site grading/drainage must also be designed for such soils as mentioned below. Following site grading, testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations. Depending upon the distribution of soil types and expansion characteristics, differing design recommendations may be developed to better suit the types of conditions present at the site.

3.5 CORROSION

Testing was performed to evaluate the corrosivity of the on-site soils and the potential for attack on concrete and subsurface cast iron and ductile iron utility pipes. The testing included pH, sulfate, chloride and minimum electrical resistivity. The results of the chemical analysis are as follows:

Table 3 – Corrosion Analyses

Boring Number	Sample Depth (feet)	pH	Minimum Resistivity (Ω -cm)	Soluble Sulfate (ppm)	Soluble Chlorides (ppm)
B3	1½ - 5	7.5	1100	246	145
B6	1 - 5	7.4	1050	576	230

Notes: ppm = parts per million.

Concrete mix design should follow the minimum requirements of the 2016 California Building Code and ACI 318. Laboratory testing of representative near surface soil samples indicates that the on-site soils have very low levels of sulfates such that they are characterized as having a **negligible** corrosion potential with respect to concrete based on ACI 318. Chloride levels also suggest that the site soils are **non-corrosive** to steel (iron/ferrous materials) according to Caltrans specifications. For preliminary design purposes, it is our opinion that the use of Type II cement is suitable for concrete in contact with on-site soils. Final concrete mix design should be evaluated after sulfate and chloride tests have been performed on the actual in-place subgrade material used for finished pad grades.

Based on the minimum resistivity laboratory test results, the soils are characterized as being **highly corrosive** to cast iron or ductile iron piping (National Association of Corrosion Engineers, 1984). PSI does not practice in the field of corrosion engineering. We suggest that a qualified corrosion engineer be consulted to determine if special corrosion protection is warranted. Testing for corrosivity of fill soils should be conducted after site grading.

3.6 SHALLOW FOUNDATIONS

Once proposed structural loadings and foundation plans are available, the geotechnical engineer should evaluate those plans for allowable foundation capacities which depend on the locations of the structures. On a preliminary basis, we would anticipate an allowable soil bearing pressure in the range of 3,000 to 5,000 psf, dependent upon the remedial grading performed. It is our opinion that the proposed buildings having relatively light loadings (150 to 250 kips) can be supported on conventional shallow spread footings bearing on Engineered Fill and/or stiff, or better, native soils, compacted to a firm and unyielding state. More heavily loaded foundations (>250 kips) may require deeper foundations or a ground improvement program to be more effective and to limit settlements to a tolerable level. Structures placed near boring location B3

should consider potential settlement issues. The size of the foundations, the bearing pressure and the embedment depth have a direct impact on the anticipated settlement. Once specific plans are developed and once detailed structural loading information is known, additional test borings and engineering analysis will be necessary.

3.7 SLABS ON GRADE

We anticipate that the proposed buildings may use slab-on-grade construction supported on Engineered Fill and/or suitable existing soil provided the subgrade has been prepared as described in Section 3.2 of this report. Soft or otherwise unsuitable areas observed should be addressed on a case-by-case basis by our Geotechnical Engineer.

Due to the presence of low expansive soils, we recommend that the subgrade soils beneath the building floor slab be moisture conditioned to 0 to 2 percent above the soil's optimum moisture content to a depth of at least 12 inches. This moisture conditioning is recommended to be verified by the geotechnical engineer within 48 hours of floor slab concrete placement. Floor slabs should have a nominal thickness of 5 inches and be reinforced with at least No. 3 bars placed at 18-inches centers each way. Additional steel reinforcement may be required for structural considerations.

To reduce moisture vapor transmission into interior spaces, we recommend a 10-mil vapor retarding membrane be included beneath slabs. Membrane specifications should be provided by the flooring manufacturer. Vapor retarders should be installed in accordance with ACI 302.1R and should extend to footings with no breaks (sealed between footings and around penetrations). A minimum 2 inch thick capillary break material (clean sand) should be provided above and below the vapor retarder to help protect the vapor barrier from punctures and to allow more uniform curing of the concrete slab.

This vapor retarder system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 3 pounds of water per 1000 square feet and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Exterior slabs should be isolated from the building. These slabs should be reinforced to function as independent units. Movement of these slabs should not be transmitted to the building

foundation or superstructure. We also recommend that expansive subgrade soils beneath concrete flatwork be moisture treated similar to that recommended for on-grade floor slabs.

The precautions listed below should be followed closely for construction of all slabs-on-grade. These details will not reduce the amount of movement, but are intended to reduce potential damage should some settlement of the supporting subgrade take place.

- Cracking of slabs-on-grade is normal and should be expected. Cracking can occur not only because of heaving or compression of the supporting soil, but also because of concrete curing stresses. The occurrence of concrete shrinkage cracks, and problems associated with concrete curing may be reduced and/or controlled by limiting the water/cement ratio of the concrete, proper concrete placement, finishing, and curing, and by the placement of crack control joints at frequent intervals, particularly, where re-entrant slab corners occur. The American Concrete Institute (ACI) recommends a maximum panel size (in feet) equal to approximately three times the thickness of the slab (in inches) in both directions. For example, joints are recommended at a maximum spacing of 12 feet, assuming a four-inch thick slab. We recommend also that control joints be scored three feet in from, and parallel to, the foundation walls. Using fiber reinforcement in the concrete can also help control shrinkage cracking.
- Some increase in moisture content is inevitable because of development and associated landscaping. However, extreme moisture content increases can be largely controlled by proper and responsible site drainage, building maintenance and irrigation practices. Landscaping and irrigation should be minimized immediately adjacent to buildings.

3.8 SETTLEMENT

Proposed loadings and structural foundation plans should be reviewed by the geotechnical engineer to accurately estimate settlements. Foundations (column loads <250 kips) designed and constructed in accordance with typical geotechnical recommendations are estimated to experience post-construction total settlements generally less than about 1-inch with differential settlement along a 40-foot long portion of a continuous footing, or similarly spaced column footings generally less than ½-inch. Total and differential settlements of these magnitudes are usually considered tolerable for the anticipated construction. However, the tolerance of proposed structures to predicted total and differential settlements should be confirmed by the structural engineer.

As previously noted, a seismic-induced settlement of about 0.7 inch with a corresponding differential seismic settlement of less than ½ inch over a 40 foot span is possible. Although these seismic settlements are well within levels considered tolerable (per SP 117A), we recommend that the structural engineer consider the seismic settlements in their design.

3.9 RETAINING WALLS

Standard retaining walls may be designed simply for walls up to a maximum height of six feet. Retaining walls above a height of six feet will require seismic loadings incorporated. Such walls may rely on shallow foundation recommendations to be provided.

Onsite clay soils are not considered suitable for retaining wall backfill. Clean granular soil backfill should be used from the site or imported. The backfill material should be compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557 with optimum moistures within 0 to 2 percent above optimum. Compaction may be checked by probing if needed. Flooding or jetting of the backfill should not be permitted. Granular backfill should be capped with 12-inches of relatively impervious fill (clayey soils, concrete, CDF) or pavements to seal the backfill and reduce the potential for infiltration and saturation. It should be noted that the use of heavy compaction equipment in close proximity to retaining structures could result in wall pressures exceeding design values and causing wall movement. In this regard, the contractor should take appropriate precautions during the backfill placement and use relatively light compaction equipment.

For drained conditions, a back-drainage system should be installed to drain any water that may collect behind retaining walls. The drainage system should consist of a 1 cubic foot per foot burrito drain containing free-draining crushed rock (5%<#200) placed adjacent to the wall. The gravel should be completely enclosed in an approved filter fabric (Mirafi 140N, or equivalent) to separate the gravel and backfill soil. A 4-inch diameter perforated SCH-40 PVC pipe should be placed with perforations down, behind the bottom of the retaining wall at a gradient of at least one percent. Drainage outlets should be 4-inch diameter solid SCH-40 PVC pipe, installed to drain to an appropriate outlet. Drainage should be conducted away from structural foundations. Weep holes may be used if desired. The project geotechnical engineer should approve any alternative drain system.

Waterproofing the back of the retaining walls should be considered to mitigate water staining, infiltration, and moisture migration through the wall. If a waterproofing system is used, it should be designed by a qualified waterproofing consultant.

3.10 PAVEMENT DESIGN

In designing the proposed pavement areas, the following conditions were considered:

1. Subgrade support characteristics are typically represented by an R-Value for the design of flexible pavements in this region.
2. Vehicular traffic volumes, in terms of the number and frequency of vehicles and their range of axle loads should be considered.
3. Likely changes in vehicular use over the life of the pavement should also be considered. We have assumed that the pavement areas will not experience additional traffic.
4. Pavement life cycle was considered to be 20 years.

All site preparations and grading should be performed as discussed above in Section 3.2.

Since an evaluation of the characteristics of the actual soils present at pavement subgrade can only be provided at the completion of grading, the following pavement sections should be used for planning purposes only. Final pavement designs should be evaluated after R-value tests have been performed on the actual in-place subgrade materials exposed for use during construction.

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

Based on the results of our field exploration and laboratory testing of sandy clay (worse case scenario), we recommend the pavement designs presented in the table below. The pavement design criteria are based on the soil conditions present at the site, an R-value of 6 and the Traffic Index indicated below based on the estimated traffic for the site. In more granular areas of the site, a thinner pavement section may be used if desired and designed.

Table 4 - Pavement Section Recommendations

Location	Traffic Index (TI)	Asphalt Thickness (inches)	+	Aggregate Base Thickness (inches)	Portland Cement Concrete Thickness (inches)	+	Aggregate Base Thickness (inches)
Standard Drive Aisles	5	3½	+	8½	5½	+	4
		4	+	7½			
Truck Drive Aisles	7	6	+	12	6	+	6
		7	+	10			

We recommend rigid pavements (Portland Cement Concrete, PCC) be constructed at all areas requiring heavy braking and turning such as intersections, entrances, docking bays, trash truck loading areas, etc. PCC pavement sections should incorporate appropriate steel reinforcement and crack control joints as designed by the project structural engineer. We recommend that sections be as nearly squared as possible and no more than 15-feet on a side. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. Such joints should be properly sealed. A minimum 4,000 psi mix is recommended.

Prior to placement of aggregate base, subgrade soils should be proof-rolled and scarified to a depth of 12 inches, brought to moisture contents between 0 to 2 percent above optimum, then compacted to at least 95 percent of the laboratory standard. The laboratory standard should be ASTM D1557.

Aggregate base below pavement sections should be compacted to at least 95 percent of ASTM D1557 with moisture contents within +/-2 percent of optimum. Aggregate base materials should be Caltrans Class II Base.

All materials should conform to and be placed in accordance with the latest revision of the Standard Specifications for Public Works Construction (Greenbook), the American Concrete Institute (ACI), and the Portland Cement Association (PCA).

3.11 LANDSCAPING AND IRRIGATION

Landscaping and irrigation should be positioned away from the building by at least 5 feet. Trees and shrubs should be positioned a distance away from the building equal to half of their mature height. Hardscape should closely surround the building as much as possible. Moisture conditions within the soil should be maintained at a constant level to avoid wetting or drying and associated movements. If landscaping is desired closer to the building, moisture barriers may be added to limit moisture infiltration beneath the building.

3.12 CONSTRUCTION CONSIDERATIONS

The site soils are sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. Furthermore, perched groundwater conditions can develop during periods of heavy rainfall as a result of less permeable layers impeding infiltration. In these instances, overlying subgrade soils may become unstable and require remedial measures. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

Groundwater was not encountered near the ground surface during our explorations onsite. It should be noted, however, that variations in the groundwater table may result from fluctuation in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors that may not have evident at the time of our exploration. This sometimes occurs where relatively impermeable and/or cemented materials are overlain by fill soils. We recommend that a representative of PSI be present during grading operations to evaluate areas of seepage. Drainage devices for reduction of water accumulation can be recommended if these conditions occur.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of the building and beneath the floor slabs during construction.

Final grading should be designed to provide positive drainage away from proposed structures. Soil areas within 10-feet of proposed structures should sloped at a minimum of 5 percent away from the structure, if possible. Roof leaders and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter or directly to the storm drain system.

4 REPORT LIMITATIONS

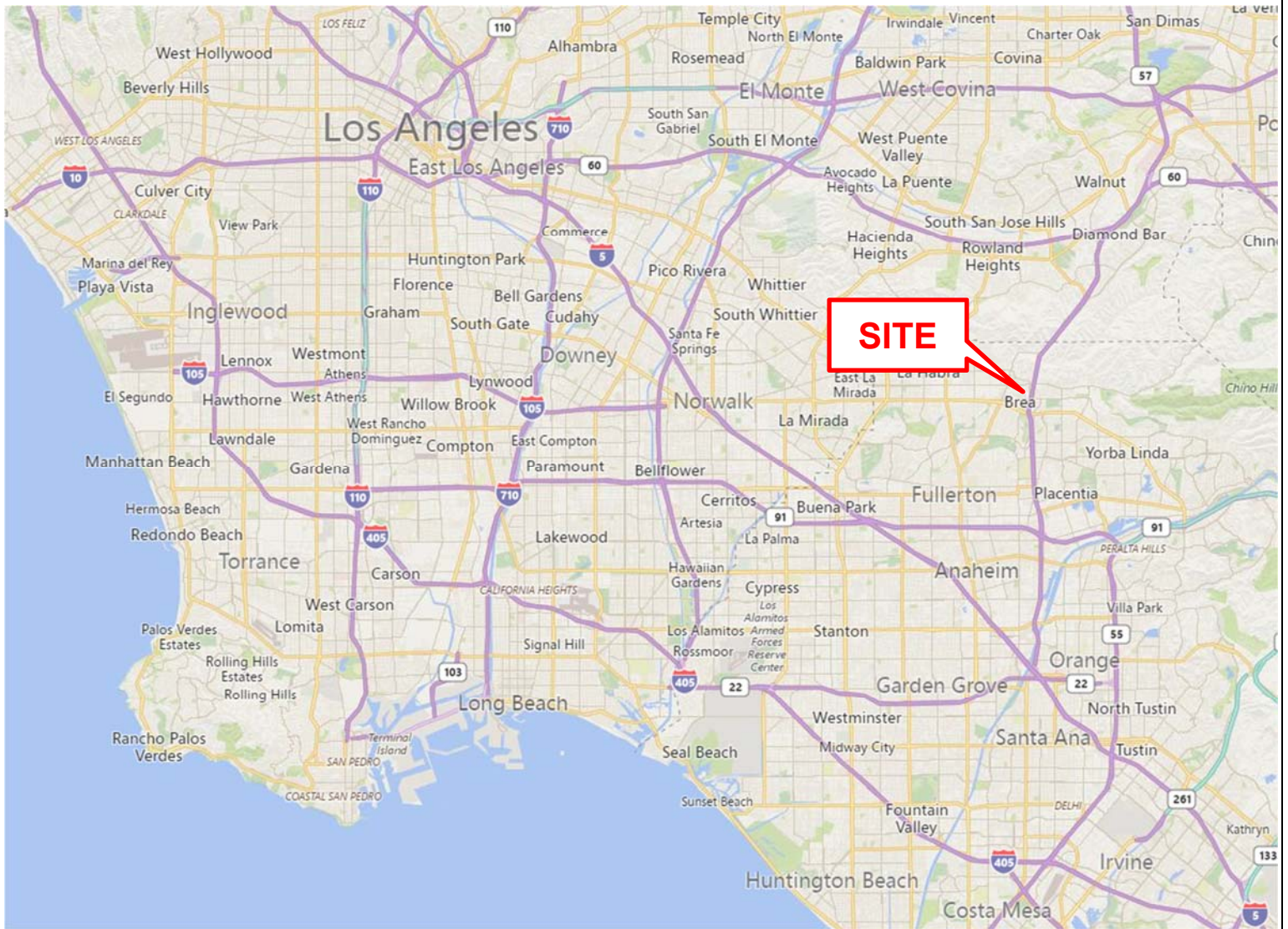
The proposed professional services have been performed, findings obtained, and preliminary recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices at the time of this report. PSI is not responsible for the conclusions, opinions, or recommendations made by others based on this data. No other warranties are implied or expressed. **Simon**, its subsidiaries and affiliates can rely upon the report under the same terms as if it was originally prepared for them.

The scope of exploration was intended to evaluate soil conditions within the influence of the proposed development. The analyses and recommendations submitted in this report are based upon the data obtained from the soil borings performed at the locations indicated. If any subsoil variations become evident during the course of this project, a re-evaluation of the recommendations contained in this report will be necessary after we have had an opportunity to observe the characteristics of the conditions encountered. The applicability of the report should also be reviewed in the event significant changes occur in the design, nature, or location of the proposed structural.

This report is considered preliminary based on limited knowledge of the proposed development. The preliminary recommendations provided herein are only intended for feasibility evaluations and conceptual designs and should not be relied upon for final design recommendations. Future design concepts and plans will require further focused geotechnical investigations to determine final design and construction recommendations for specific development structures.

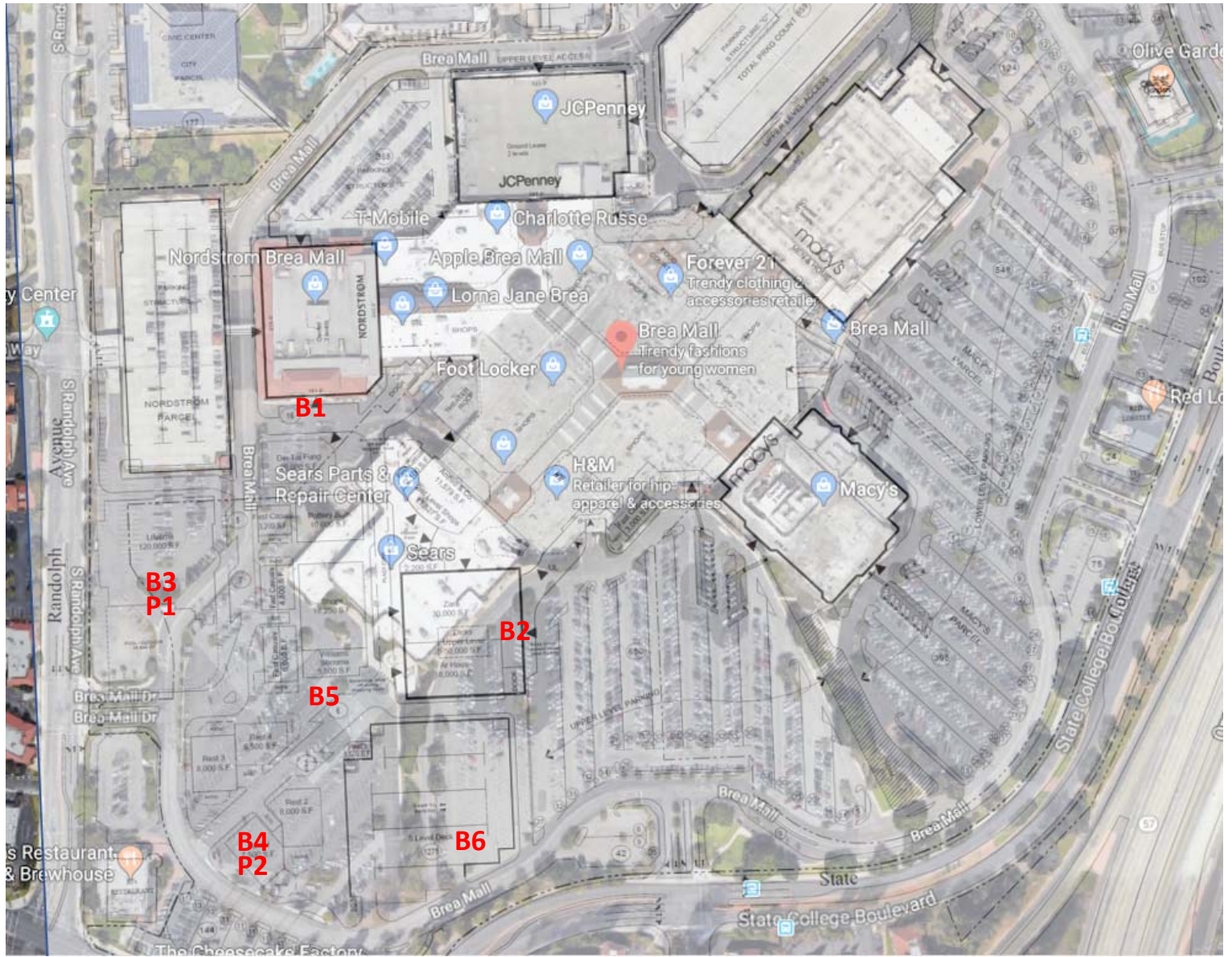


FIGURES



Reference – Bing Maps

	DATE: 2/27/2018	Proposed Brea Mall Development 1066 Brea Mall Brea, California 92821	PSI PROJECT NUMBER: 00661261	
PSI, INC. 11980 TELEGRAPH ROAD, UNIT 104 SANTA FE SPRINGS, CALIFORNIA	PREPARED BY: DTA	SITE VICINITY MAP	FIGURE 1	



Legend

- B6** Approximate Boring Location
- P2** Approximate Percolation Location

Reference – Google Maps

	DATE: 2/27/2018	Proposed Brea Mall Development 1066 Brea Mall Brea, California 92821	PSI PROJECT NUMBER: 00661261	
PSI, INC. 11980 TELEGRAPH ROAD, UNIT 104 SANTA FE SPRINGS, CALIFORNIA	PREPARED BY: DTA	BORING LOCATION MAP	FIGURE 2	



APPENDIX A

BORING LOGS



GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.	☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
HSA: Hollow Stem Auger - typically 3 1/4" or 4 1/4" I.D. openings, except where noted.	■ ST: Shelby Tube - 3" O.D., except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry	▮ RC: Rock Core
R.C.: Diamond Bit Core Sampler	↓ TC: Texas Cone
H.A.: Hand Auger	☞ BS: Bulk Sample
P.A.: Power Auger - Handheld motorized auger	☒ PM: Pressuremeter
	CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N ₆₀ : A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q _u : Unconfined compressive strength, TSF
Q _p : Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL), %
DD: Dry unit weight, pcf
▼, ▽, ▾ Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS ANGULARITY OF COARSE-GRAINED PARTICLES

Relative Density	N - Blows/foot	Description	Criteria
Very Loose	0 - 4	Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Loose	4 - 10	Subangular:	Particles are similar to angular description, but have rounded edges
Medium Dense	10 - 30	Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Dense	30 - 50	Rounded:	Particles have smoothly curved sides and no edges
Very Dense	50 - 80		
Extremely Dense	80+		

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (3/4 in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to 3/4 in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%



GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

<u>Q_u - TSF</u>	<u>N - Blows/foot</u>	<u>Consistency</u>
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

<u>Description</u>	<u>Criteria</u>
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

<u>Descriptive Term</u>	<u>% Dry Weight</u>
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

<u>Description</u>	<u>Criteria</u>	<u>Description</u>	<u>Criteria</u>
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick	Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick	Lensed:	Inclusion of small pockets of different soils
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing	Layer:	Inclusion greater than 3 inches thick (75 mm)
Slickensided:	Fracture planes appear polished or glossy, sometimes striated	Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
		Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

<u>Q_u - TSF</u>	<u>Consistency</u>
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

<u>Description</u>	<u>Criteria</u>
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

<u>Voids</u>	<u>Void Diameter</u>
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

<u>(Typically Sedimentary Rock)</u>	
<u>Component</u>	<u>Size Range</u>
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

<u>Rock Mass Description</u>	<u>RQD Value</u>
Excellent	90 - 100
Good	75 - 90
Fair	50 - 75
Poor	25 - 50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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



DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 11.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9153° LONGITUDE: -117.8883° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR		BORING B1 Water BORING LOCATION:	
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Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @	Additional Remarks
	0					5-inches Asphalt 13-inches Mix of Base with Silty Sand				X Moisture PL LL	
						FILL - Silty SAND , orange-brown, with pieces of brick, moist.	SM				
						Silty SAND , orange-brown, with gravel, little cohesion, loose, moist. [Possible Fill]	SM	3-3-3 N=4	10		DD = 112 pcf
	5					Silty CLAY , brown, soft, moist.	CL	2-1-2 N=3			
						Silty SAND , light brown, with trace gravel and cohesion, loose, moist.	SM	2-3-3 N=4	12		DD = 110 pcf
	10					SAND , light brown, dense, moist.	SP	6-12-18 N=30			
Boring Terminated at 11-1/2 feet below ground surface. All CalMod blowcounts multiplied by a factor of 0.63 to represent SPT N-Value. No groundwater encountered. Soil cuttings used as backfill, with asphalt cold patch.											

	Professional Service Industries, Inc. 11980 Telegraph Rd, Unit 104 Santa Fe Springs, CA 90670 Telephone: (714) 484-8600	PROJECT NO.: 00661261 PROJECT: Simon ~ Proposed Brea Mall Development LOCATION: 100 Brea Mall Brea, CA
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DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 11.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9143° LONGITUDE: -117.8871° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR		BORING B2 Water	
BORING LOCATION:					

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @	Additional Remarks
						6-inches Asphalt					
						6-inches Aggregate Base					
						FILL - Sandy SILT , light brown, with trace clay, moist.	ML				
						Clayey SILT , brown, with roots, moist.	ML				
						Silty CLAY , orange-brown, very stiff, moist.	CL	10-14-23 N=23	14	X Moisture PL LL 0 25 50	DD = 115 pcf
	5					Sandy CLAY , light orange-brown, hard, moist.	CL	8-15-27 N=42			
						Clayey SAND , light orange-brown, very dense to dense, moist.	SC	19-42-50/4" N=58	9	X Qu * Qp 0 2.0 4.0	>> DD = 121 pcf
	10							14-14-23 N=37			
Boring Terminated at 11-1/2 feet below ground surface. All CalMod blowcounts multiplied by a factor of 0.63 to represent SPT N-Value. No groundwater encountered. Soil cuttings used as backfill, with asphalt cold patch.											



Professional Service Industries, Inc.
 11980 Telegraph Rd, Unit 104
 Santa Fe Springs, CA 90670
 Telephone: (714) 484-8600

PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA

DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 51.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9144° LONGITUDE: -117.8892° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR		BORING B3 Water ▽ While Drilling 41.2 feet ▽ Upon Completion 29.2 feet BORING LOCATION:	
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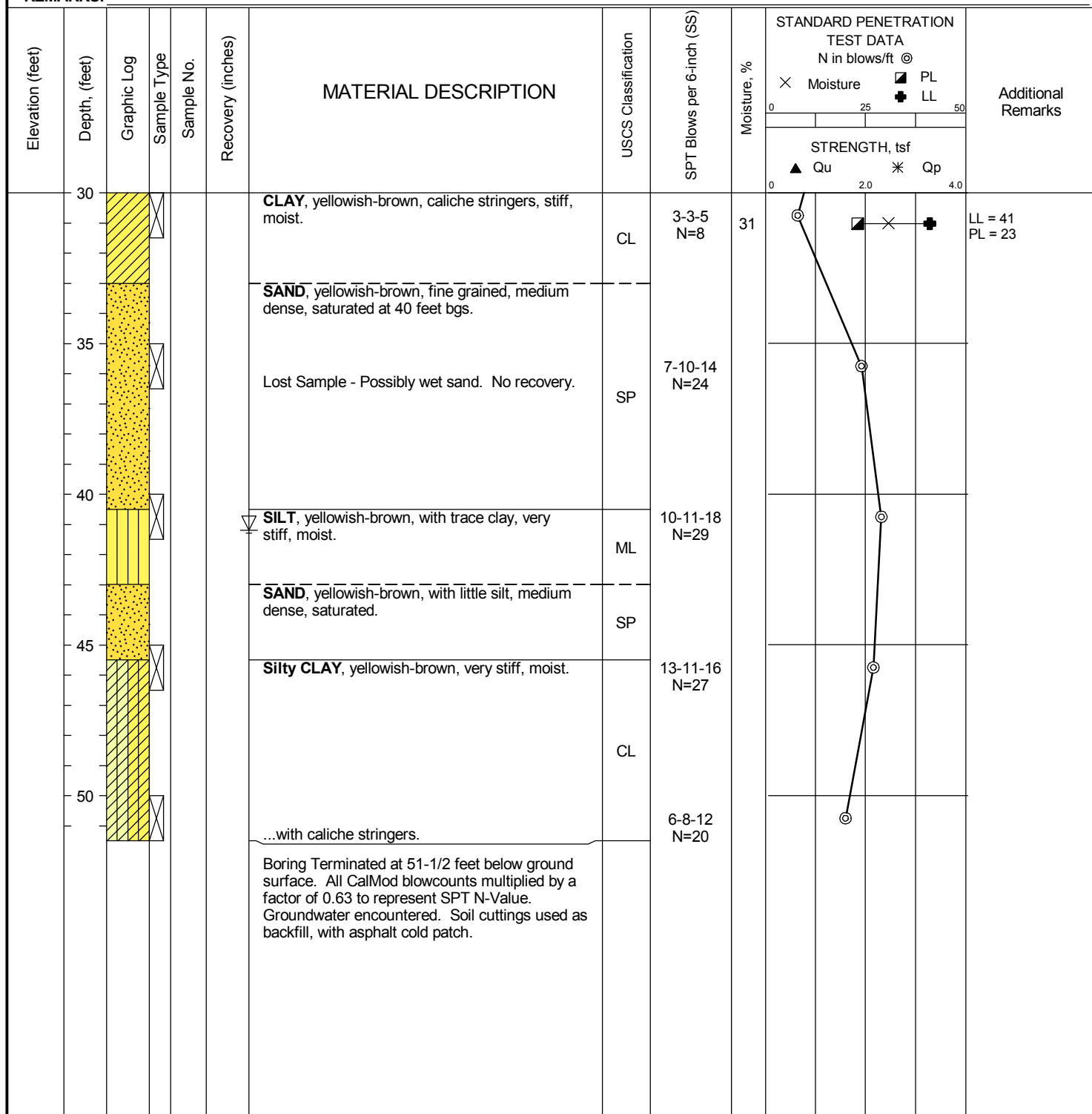
Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @	STRENGTH, tsf	Additional Remarks
										X Moisture ▣ PL ▣ LL 0 25 50	▲ Qu * Qp 0 2.0 4.0	
0						12-inches Asphalt						
						6-inches Aggregate Base						
						Sandy CLAY , orange and gray and brown, with silt and gravel, very stiff, moist. [Possible Fill]	CL	13-15-17 N=20	7			R-Value, Corrosion Series LL = 41 PL = 18 DD = 123 pcf
5						Silty SAND , yellow-brown, with clay, loose, moist. [Possible Fill]	SM	3-3-6 N=9				
						Silty CLAY , brown, with sand, stiff to very stiff, moist.		6-6-10 N=10	16			DD = 111 pcf
10						Brownish-gray at tip of sampler.		5-6-8 N=14	25			LL = 40 PL = 17
15						Gray at tip of sampler.	CL	7-8-14 N=14	18			DD = 109 pcf
20						Becomes light brown.		4-5-8 N=13				
25						Becomes yellow-brown, with caliche stringers.		6-8-11 N=19				
30						CLAY , yellowish-brown, caliche stringers, stiff, moist.	CL					

Continued Next Page

Professional Service Industries, Inc.
 11980 Telegraph Rd, Unit 104
 Santa Fe Springs, CA 90670
 Telephone: (714) 484-8600

PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA

DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 51.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9144° LONGITUDE: -117.8892° STATION: N/A OFFSET: N/A REMARKS:	DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR	BORING B3
		Water ▽ While Drilling 41.2 feet ▽ Upon Completion 29.2 feet ▽
BORING LOCATION:		



Professional Service Industries, Inc.
 11980 Telegraph Rd, Unit 104
 Santa Fe Springs, CA 90670
 Telephone: (714) 484-8600

PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA

DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 51.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9132° LONGITUDE: -117.8888° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR		BORING B4 Water ▽ While Drilling ▽ Upon Completion ▽ 24.8 feet 30.3 feet BORING LOCATION:	
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Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @		Additional Remarks
										X Moisture ▣ PL ▣ LL STRENGTH, tsf ▲ Qu * Qp		
0						3-1/2-inches Asphalt						
						6-inches Aggregate Base						
						Poorly Graded SAND, tan, fine grained, loose, slightly moist. [Possible Fill]	SP					
	5					Poorly Graded SAND, tan, fine grained, with occasional gravel, dense to very dense, moist.	SP	19-26-50 N=48	4	X		Sieve
	10						SP	12-15-20 N=35				DD = 107 pcf
	15					Sandy SILT, reddish-brown, with clay lens, hard, moist.	ML	24-42-50/4" N=58	4	X		>> DD = 110 pcf Sieve
	20					Silty CLAY, orange-brown, very stiff, moist.	CL	18-28-30 N=58				>> ④
	25					Silty SAND, orange-brown, medium dense, saturated.	SM	9-11-16 N=27				
	30							8-10-13 N=23				
Continued Next Page												

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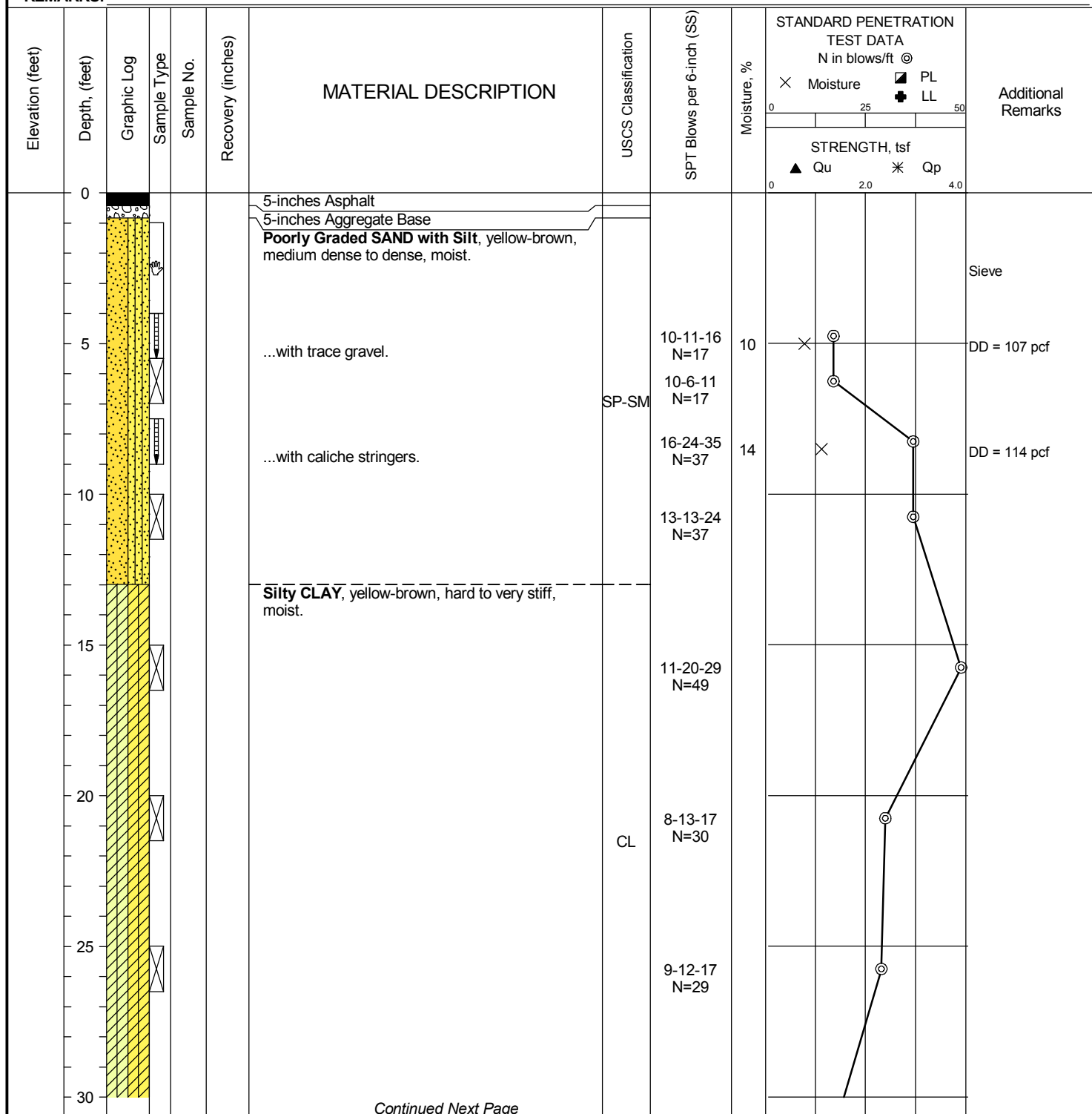
PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA

DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 51.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9132° LONGITUDE: -117.8888° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR		BORING B4 Water ∇ While Drilling ▼ Upon Completion ∇ 24.8 feet 30.3 feet BORING LOCATION:	
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Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @	Additional Remarks
30					▼	Silty CLAY , yellow-brown, very stiff, moist.	CL	6-9-13 N=22	19	X Moisture ▲ Qu □ PL + LL	LL = 36 PL = 17
35						Poorly Graded SAND with Silt , yellow-brown, medium dense to very dense, with occasional saturated sand layers, wet.	SP-SM	3-5-9 N=14			Sieve
40								14-30-39 N=69			>>⊙
45						Silty CLAY , yellow-brown, hard to very stiff, moist.	CL	7-15-17 N=32			
50								5-7-12 N=19			
Boring Terminated at 51-1/2 feet below ground surface. All CalMod blowcounts multiplied by a factor of 0.63 to represent SPT N-Value. Groundwater encountered. Soil cuttings used as backfill, with asphalt cold patch.											

	Professional Service Industries, Inc. 11980 Telegraph Rd, Unit 104 Santa Fe Springs, CA 90670 Telephone: (714) 484-8600	PROJECT NO.: 00661261 PROJECT: Simon ~ Proposed Brea Mall Development LOCATION: 100 Brea Mall Brea, CA
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DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 31.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9139° LONGITUDE: -117.8882° STATION: N/A OFFSET: N/A REMARKS:	DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR	BORING B5
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Continued Next Page



Professional Service Industries, Inc.
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 Santa Fe Springs, CA 90670
 Telephone: (714) 484-8600

PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA

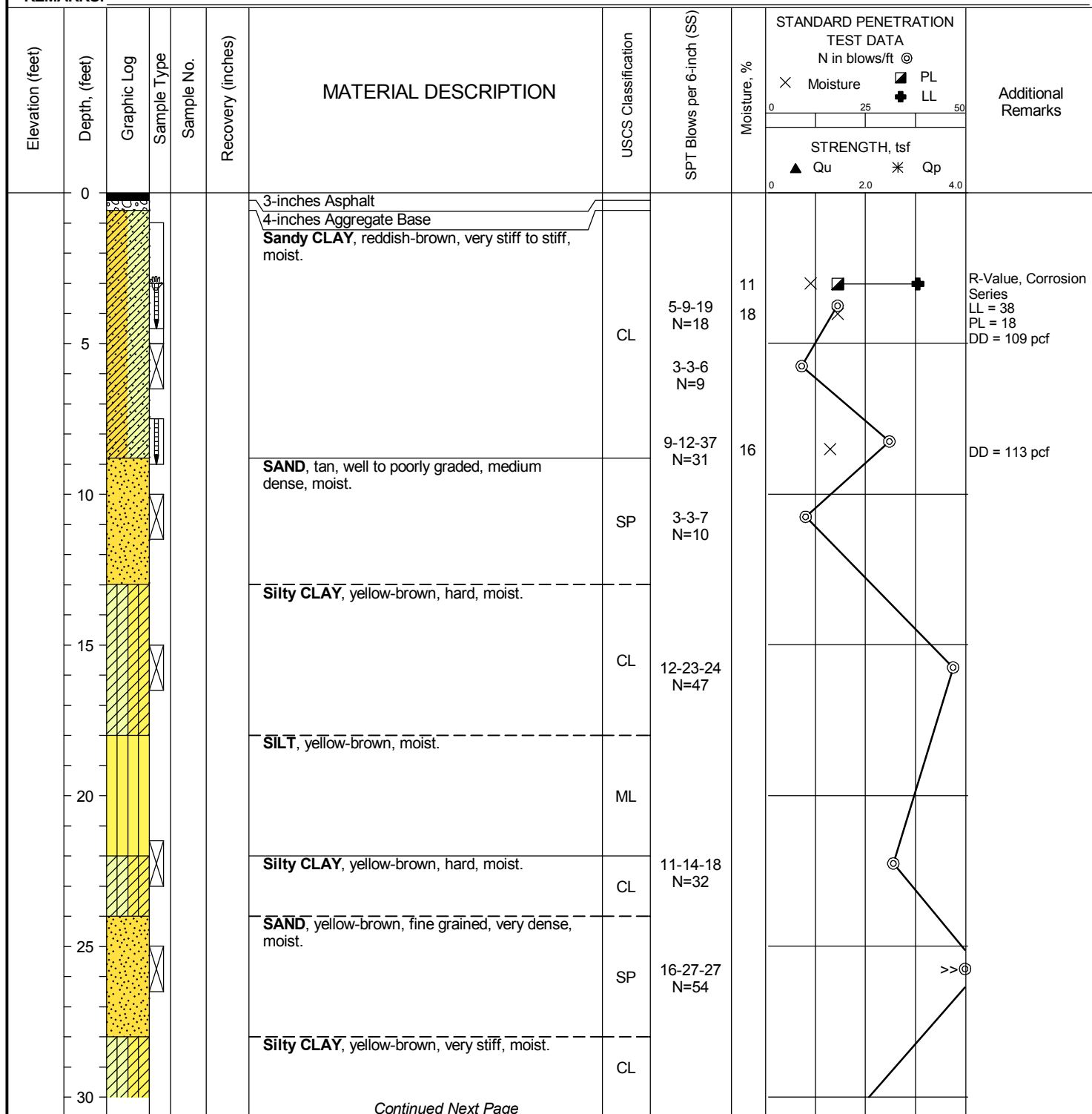
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				BORING LOCATION:									
Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @ × Moisture ▣ PL 0 25 50 ▲ Qu ✱ Qp 0 2.0 4.0			Additional Remarks
	30					Silty CLAY , yellow-brown, hard to very stiff, moist. Boring Terminated at 31-1/2 feet below ground surface. All CalMod blowcounts multiplied by a factor of 0.63 to represent SPT N-Value. Groundwater not encountered. Soil cuttings used as backfill, with asphalt cold patch.	CL	6-8-10 N=18					



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PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA

DATE STARTED: 2/1/18 DATE COMPLETED: 2/1/18 COMPLETION DEPTH: 31.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 33.9133° LONGITUDE: -117.8873° STATION: N/A OFFSET: N/A REMARKS:	DRILL COMPANY: 2R Drilling DRILLER: George LOGGED BY: DTA DRILL RIG: CME-75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS-SS+CalMod HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: RR	BORING B6
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Continued Next Page



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PROJECT NO.: 00661261
PROJECT: Simon ~ Proposed Brea Mall Development
LOCATION: 100 Brea Mall
 Brea, CA



APPENDIX B

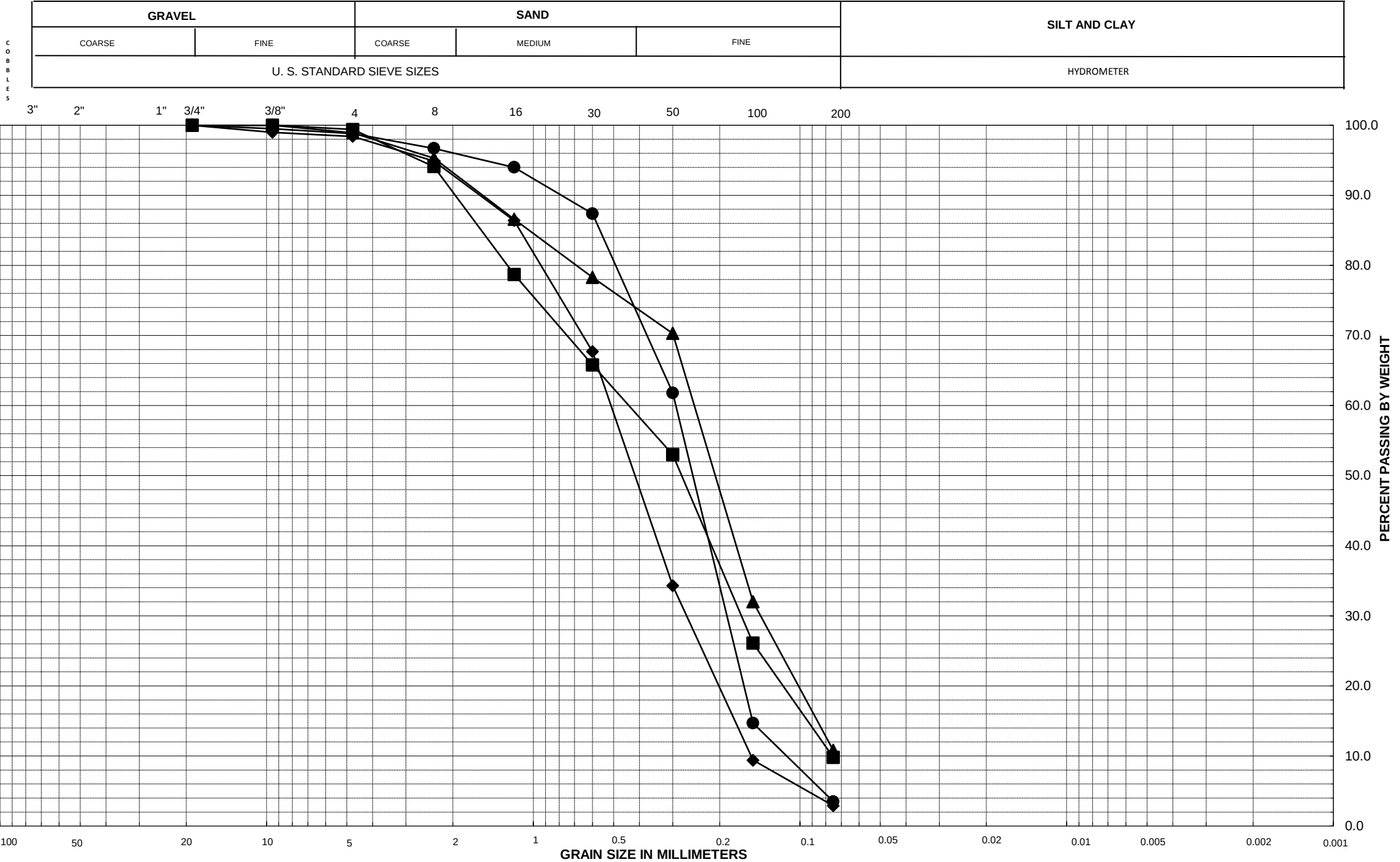
LABORATORY TEST RESULTS



LABORATORY TEST RESULTS

Boring	Depth (ft.)	Test	Result
B3	1½ - 5	Soluble Sulfate (ppm) Soluble Chloride (ppm) Minimum Resistivity (Ω-cm) pH R-Value Liquid Limit (%) Plastic Limit (%) Plastic Index (%) Expansion Index	246 145 1100 7.5 11 41 18 23 35
B3	8½	Consolidation	See Plot
B3	11	Liquid Limit (%) Plastic Limit (%) Plastic Index (%)	40 17 24
B3	31	Liquid Limit (%) Plastic Limit (%) Plastic Index (%)	41 23 17
B4	2½ - 5½	Sieve Analysis	See Plot
B4	11	Sieve Analysis	See Plot
B4	31	Liquid Limit (%) Plastic Limit (%) Plastic Index (%)	36 17 19
B4	36	Sieve Analysis	See Plot
B5	1 - 4	Sieve Analysis	See Plot
B6	1 - 5	Soluble Sulfate (ppm) Soluble Chloride (ppm) Minimum Resistivity (Ω-cm) pH R-Value Liquid Limit (%) Plastic Limit (%) Plastic Index (%) Direct Shear Expansion Index	576 230 1050 7.4 6 38 18 21 See Plot 34

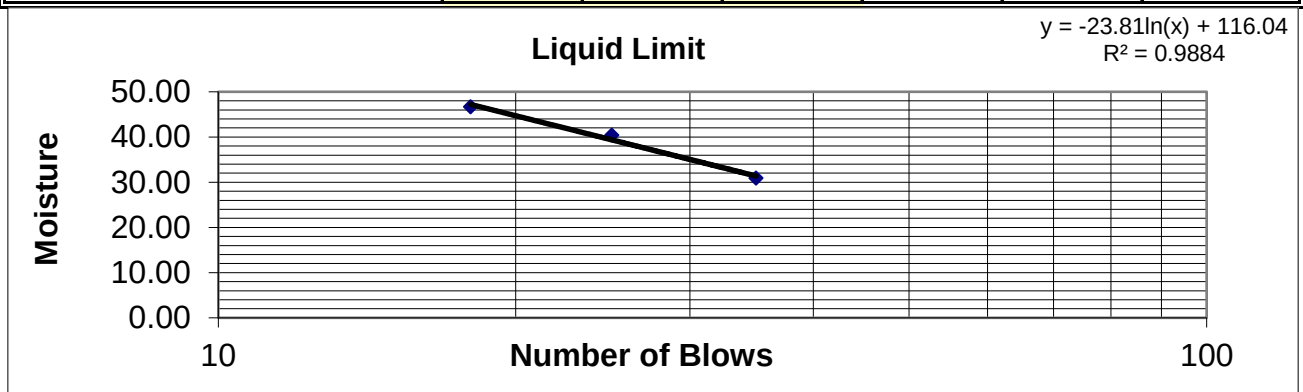
UNIFIED SOIL CLASSIFICATION



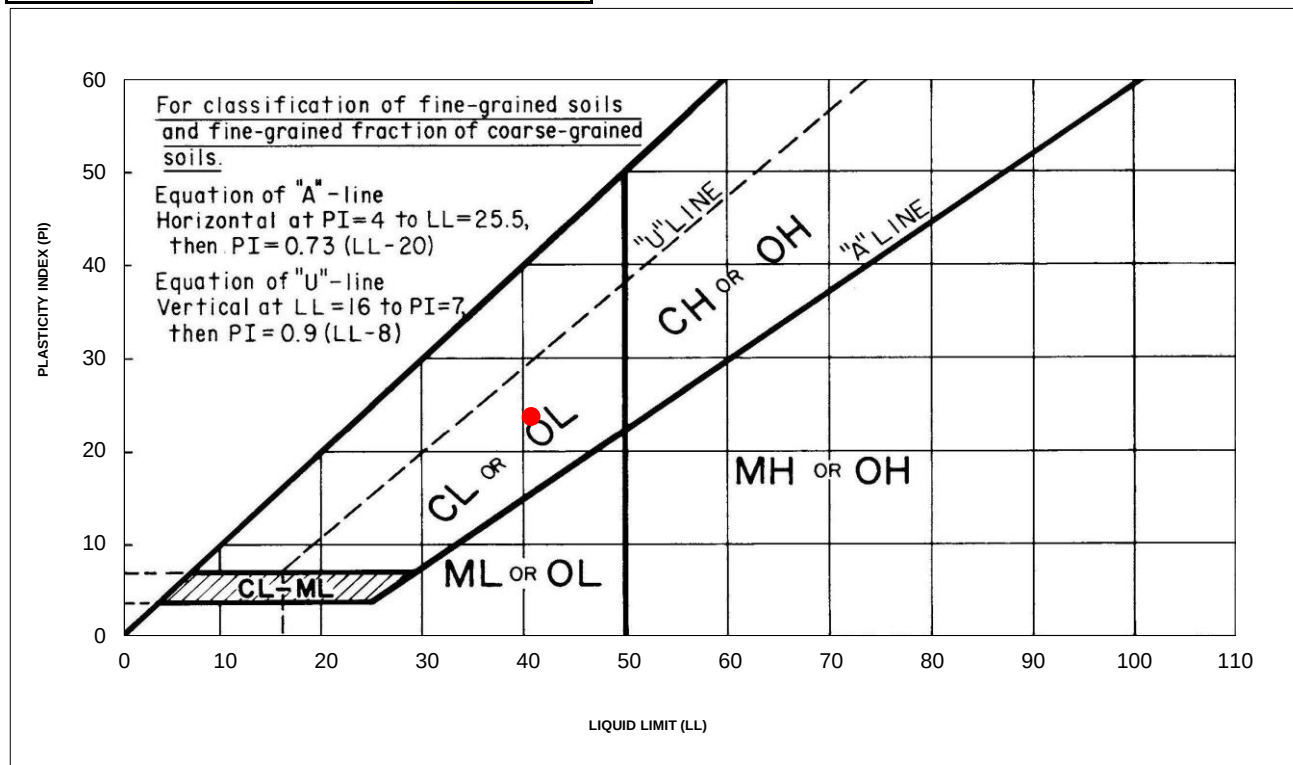
Exploration No.	Sample No.	Depth (ft)	SYMBOL	W _n (%)	LL	PI	% Clay	Description and Classification	D ₆₀	D ₃₀	D ₁₀	C _u	C _c
B4		2.5'-5.5'	●					Poorly Graded Sand (SP)	0.29	0.19	0.11	2.6	1.1
B4		11'	◆					Poorly Graded Sand (SP)	0.51	0.26	0.15	3.4	0.9
B4		36'	■					Poorly Graded Sand with Silt (SP-SM)	0.43	0.15	0.073	5.9	0.7
B5		1'-4'	▲					Poorly Graded Sand with Silt (SP-SM)	0.24	0.14	0.073	3.3	1.1
			□										
PROJECT NAME: Brea Mall PROJECT NUMBER: 00661261								PARTICLE-SIZE DISTRIBUTION CURVES ASTM D422	intertek psi				

Project Name	Simon ~ Brea Mall	Project Number	00661261
Laboratory Number	5296	Date Tested	2/7/2018
Boring & Sample No.	B3 @ 11 ft.		
Sample Description	Silty Clay		

	LIQUID LIMIT			PLASTIC LIMIT	
WEIGHT OF CAN + WET SOIL	46.70	42.10	44.00	19.5	19.7
WEIGHT OF CAN + DRY SOIL	40.40	35.60	36.20	17.9	18.2
WEIGHT OF CAN	20.00	19.50	19.50	8.60	8.8
WEIGHT OF WATER	6.30	6.50	7.80	1.6	1.5
WEIGHT OF DRY SOIL	20.40	16.10	16.70	9.3	9.4
MOISTURE CONTENT	30.88	40.37	46.71	17.2	16.0
NUMBER OF BLOWS	35	25	18		

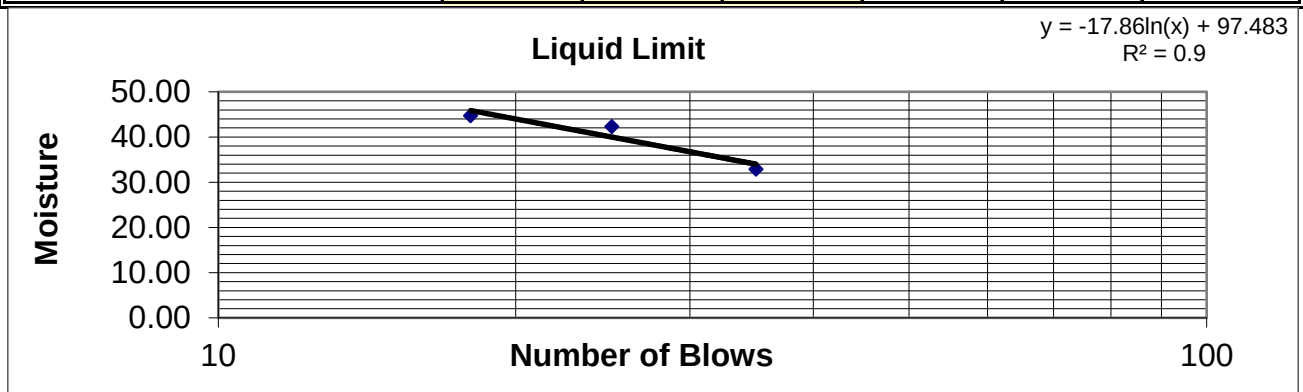


TEST RESULTS	
LIQUID LIMIT	40
PLASTIC LIMIT	17
PLASTICITY INDEX	24
Classification	CL

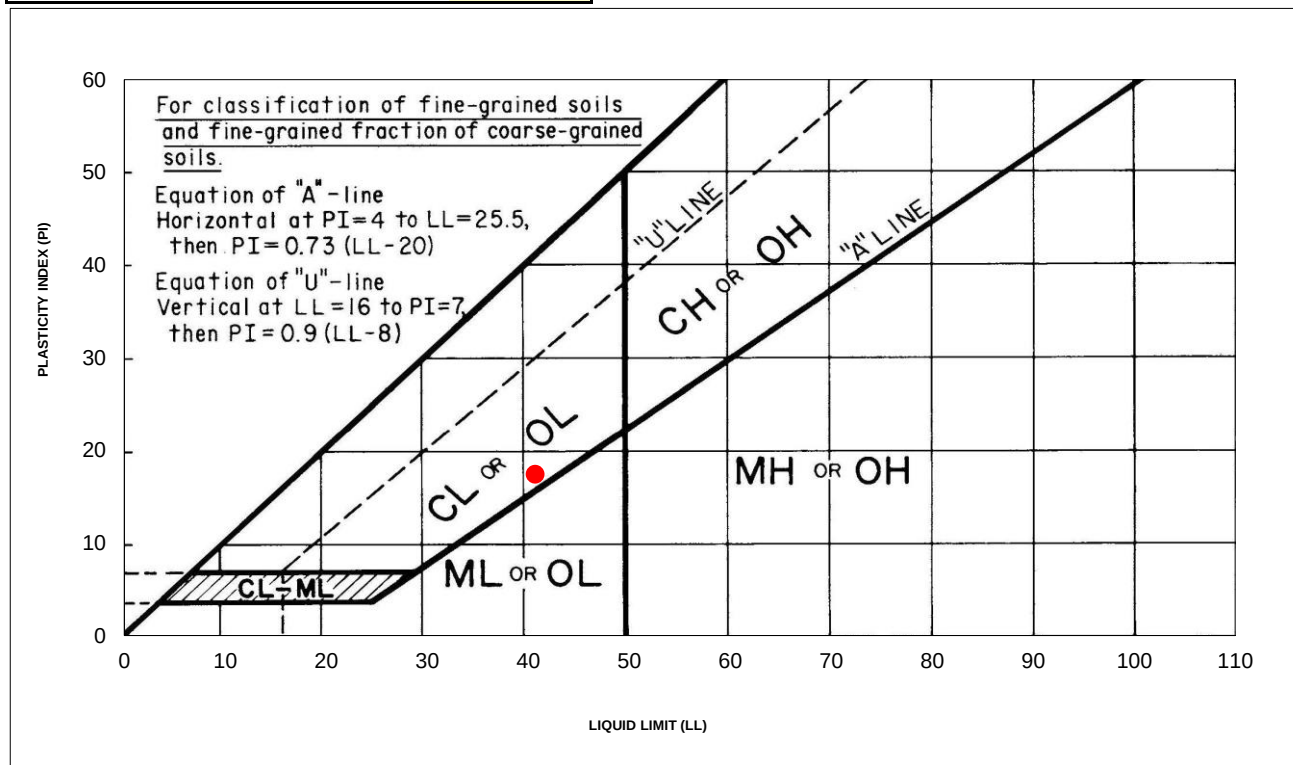


Project Name	Simon ~ Brea Mall	Project Number	00661261
Laboratory Number	5296	Date Tested	2/7/2018
Boring & Sample No.	B3 @ 31 ft.		
Sample Description	Silty Clay		

	LIQUID LIMIT			PLASTIC LIMIT	
WEIGHT OF CAN + WET SOIL	38.20	40.50	44.10	18.5	18
WEIGHT OF CAN + DRY SOIL	33.60	34.20	36.50	16.7	16.2
WEIGHT OF CAN	19.60	19.30	19.50	8.80	8.6
WEIGHT OF WATER	4.60	6.30	7.60	1.8	1.8
WEIGHT OF DRY SOIL	14.00	14.90	17.00	7.9	7.6
MOISTURE CONTENT	32.86	42.28	44.71	22.8	23.7
NUMBER OF BLOWS	35	25	18		

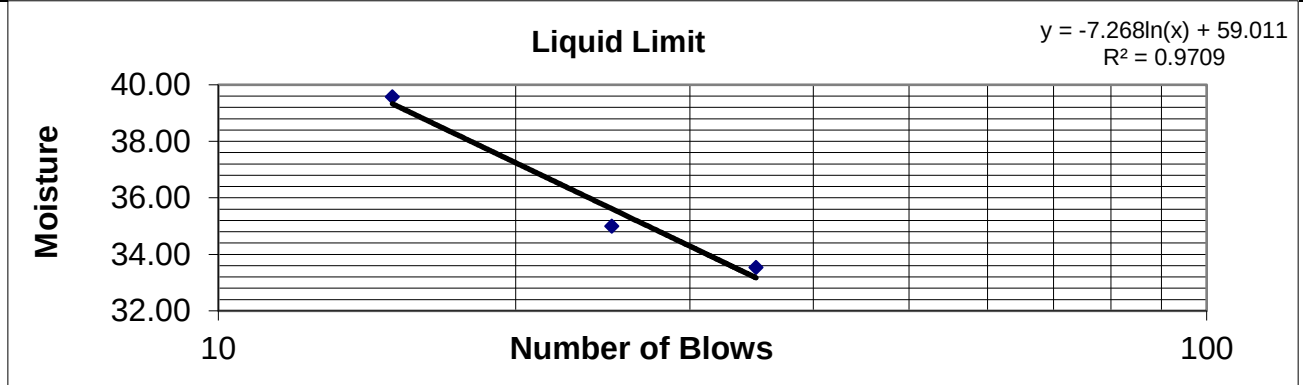


TEST RESULTS	
LIQUID LIMIT	41
PLASTIC LIMIT	23
PLASTICITY INDEX	17
Classification	CL

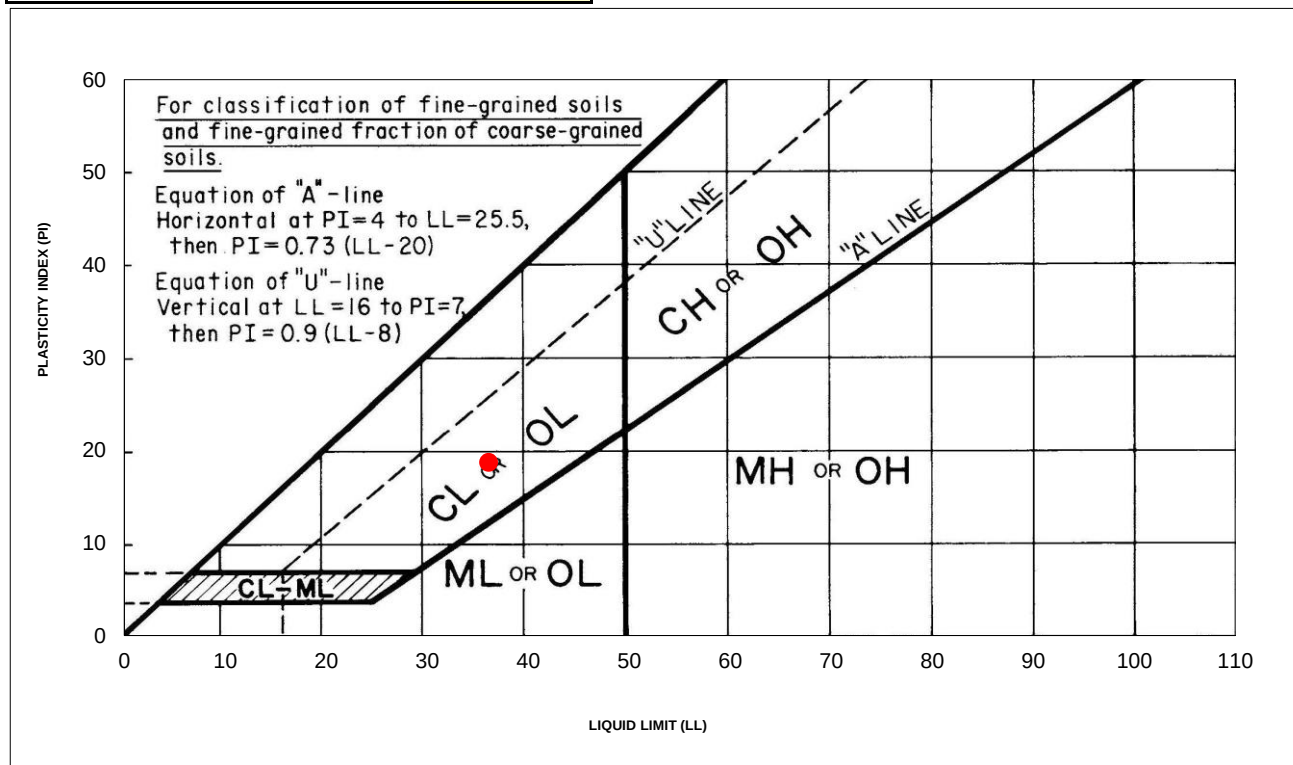


Project Name	Simon ~ Brea Mall	Project Number	00661261
Laboratory Number	5296	Date Tested	2/7/2018
Boring & Sample No.	B4 @ 31 ft.		
Sample Description	Silty Clay		

	LIQUID LIMIT			PLASTIC LIMIT	
WEIGHT OF CAN + WET SOIL	41.10	43.70	45.90	19.5	18.7
WEIGHT OF CAN + DRY SOIL	35.70	37.40	38.50	17.9	17.2
WEIGHT OF CAN	19.60	19.40	19.80	8.60	8.6
WEIGHT OF WATER	5.40	6.30	7.40	1.6	1.5
WEIGHT OF DRY SOIL	16.10	18.00	18.70	9.3	8.6
MOISTURE CONTENT	33.54	35.00	39.57	17.2	17.4
NUMBER OF BLOWS	35	25	15		

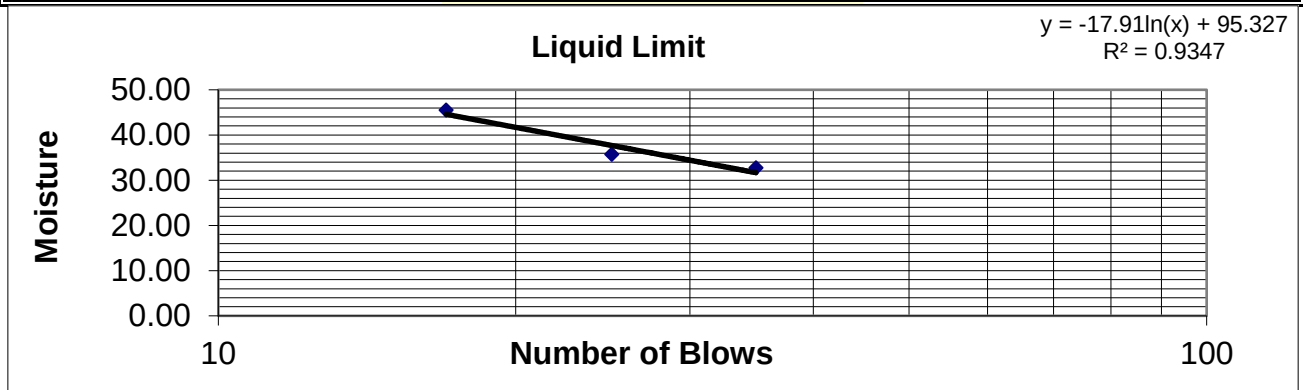


TEST RESULTS	
LIQUID LIMIT	36
PLASTIC LIMIT	17
PLASTICITY INDEX	19
Classification	CL

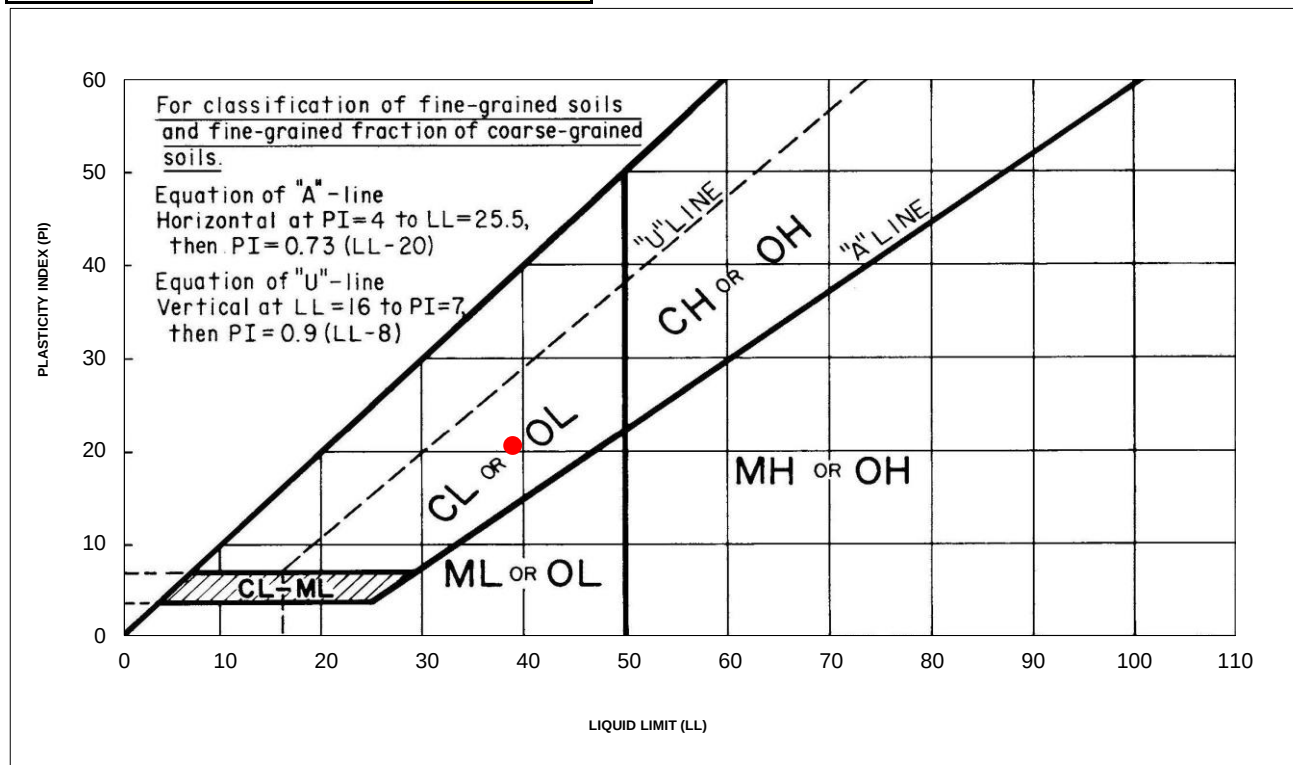


Project Name	Simon ~ Brea Mall	Project Number	00661261
Laboratory Number	5296	Date Tested	2/7/2018
Boring & Sample No.	B6 @ 1-5 ft.		
Sample Description	Silty Clay		

	LIQUID LIMIT			PLASTIC LIMIT	
WEIGHT OF CAN + WET SOIL	40.90	42.40	42.20	20.2	20.7
WEIGHT OF CAN + DRY SOIL	35.60	36.40	35.10	18.5	18.8
WEIGHT OF CAN	19.40	19.60	19.50	8.60	8.6
WEIGHT OF WATER	5.30	6.00	7.10	1.7	1.9
WEIGHT OF DRY SOIL	16.20	16.80	15.60	9.9	10.2
MOISTURE CONTENT	32.72	35.71	45.51	17.2	18.6
NUMBER OF BLOWS	35	25	17		

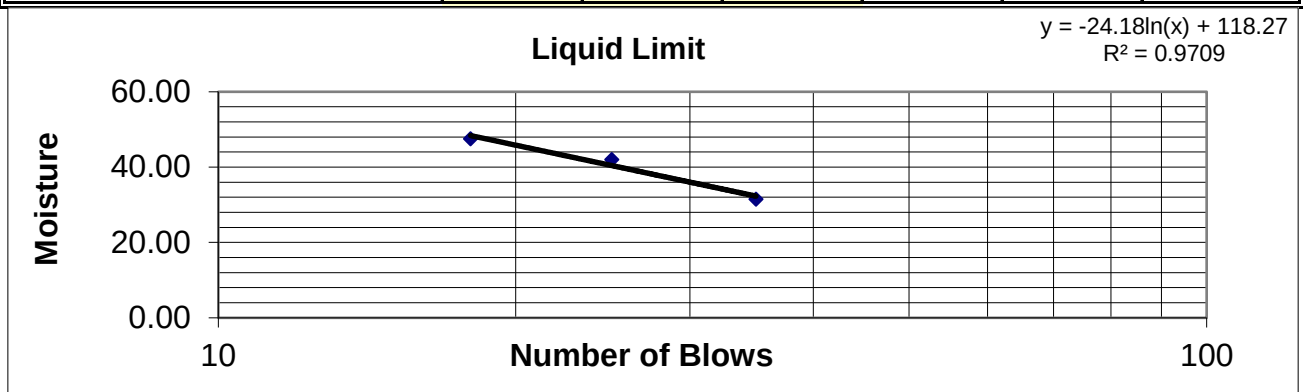


TEST RESULTS	
LIQUID LIMIT	38
PLASTIC LIMIT	18
PLASTICITY INDEX	21
Classification	CL

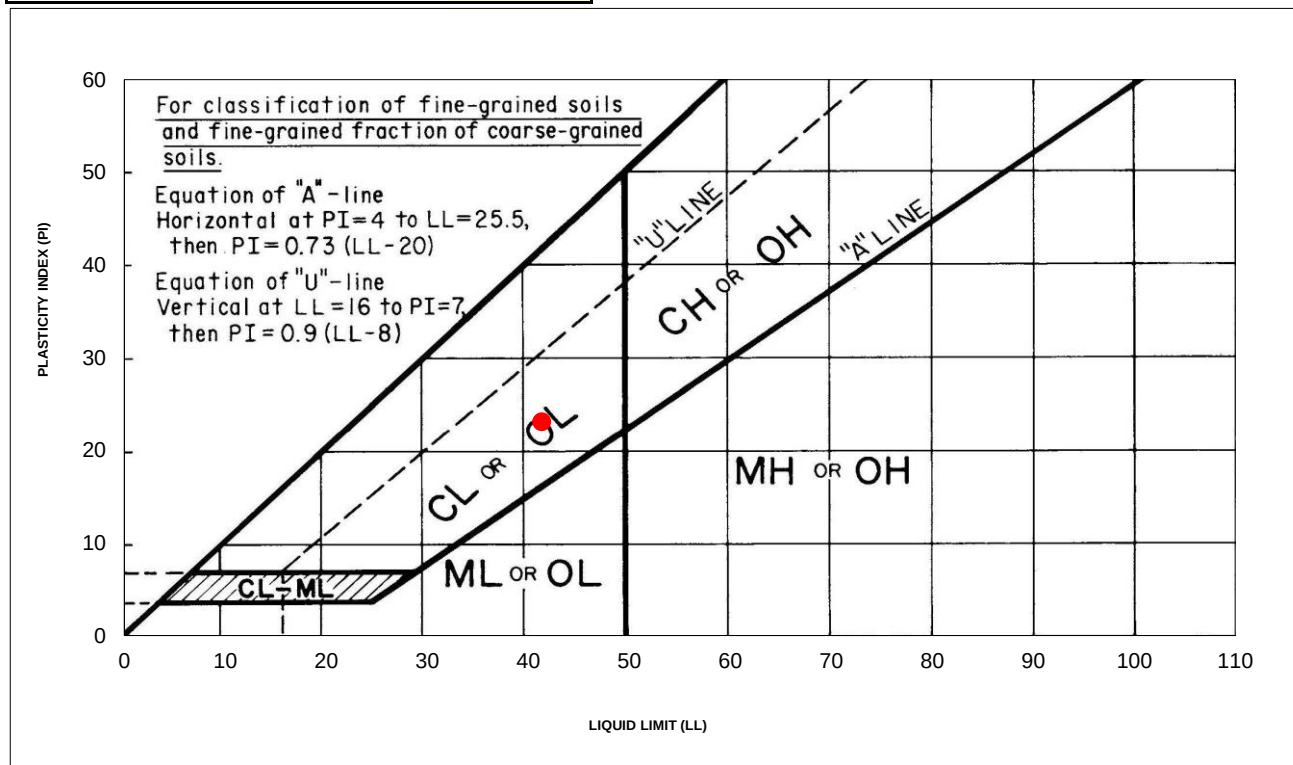


Project Name	Simon ~ Brea Mall	Project Number	00661261
Laboratory Number	5296	Date Tested	2/9/2018
Boring & Sample No.	B3 @ 1.5-5 ft.		
Sample Description	Silty Clay		

	LIQUID LIMIT			PLASTIC LIMIT	
WEIGHT OF CAN + WET SOIL	46.30	41.80	43.70	19.7	19.8
WEIGHT OF CAN + DRY SOIL	40.00	35.20	35.90	17.8	18.3
WEIGHT OF CAN	20.00	19.50	19.50	8.60	8.8
WEIGHT OF WATER	6.30	6.60	7.80	1.9	1.5
WEIGHT OF DRY SOIL	20.00	15.70	16.40	9.2	9.5
MOISTURE CONTENT	31.50	42.04	47.56	20.7	15.8
NUMBER OF BLOWS	35	25	18		

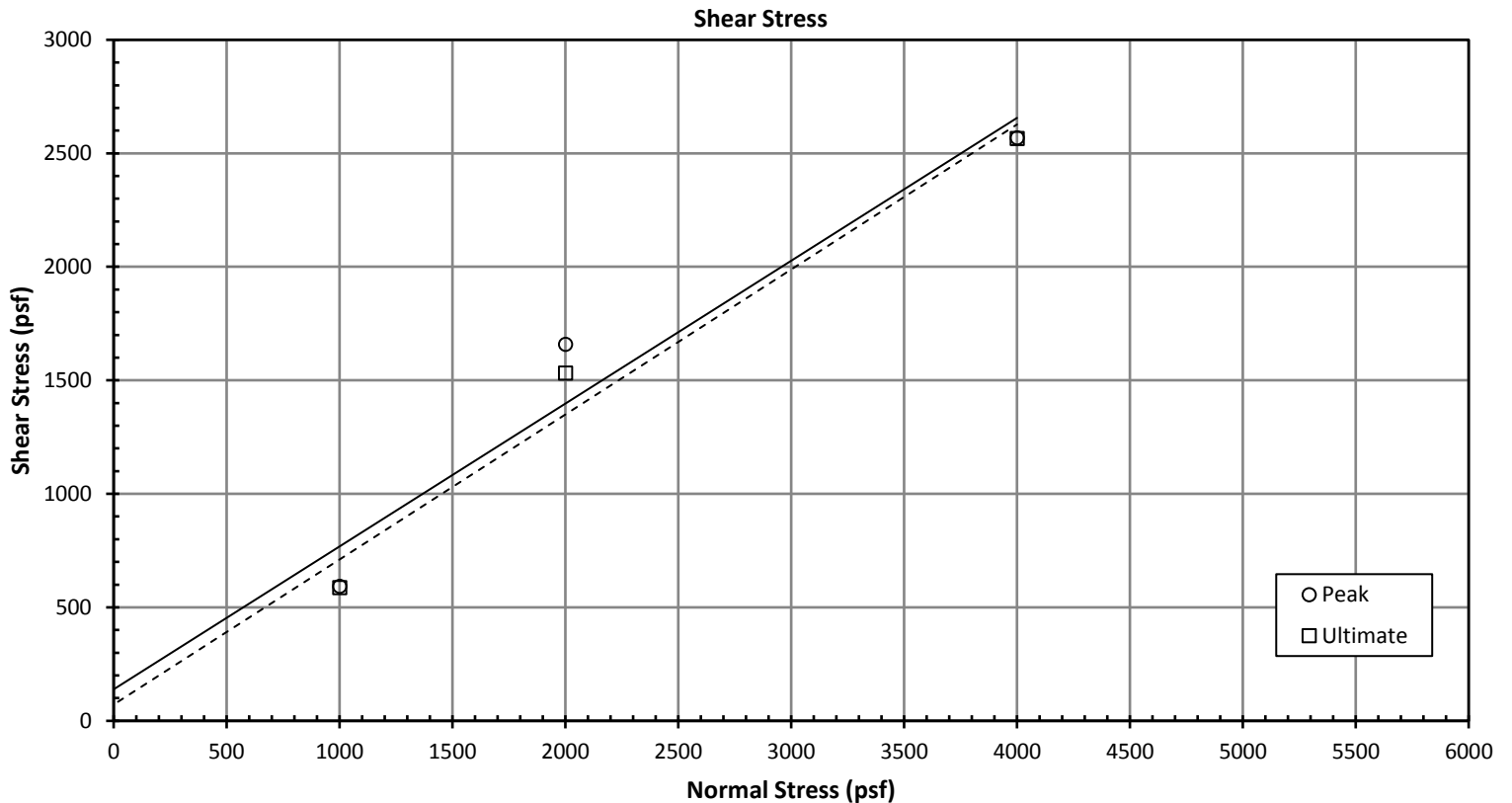
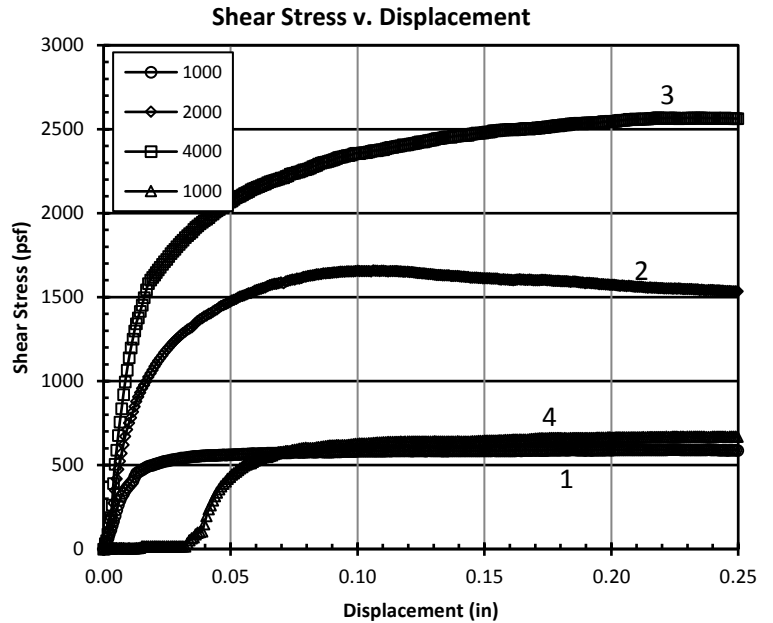
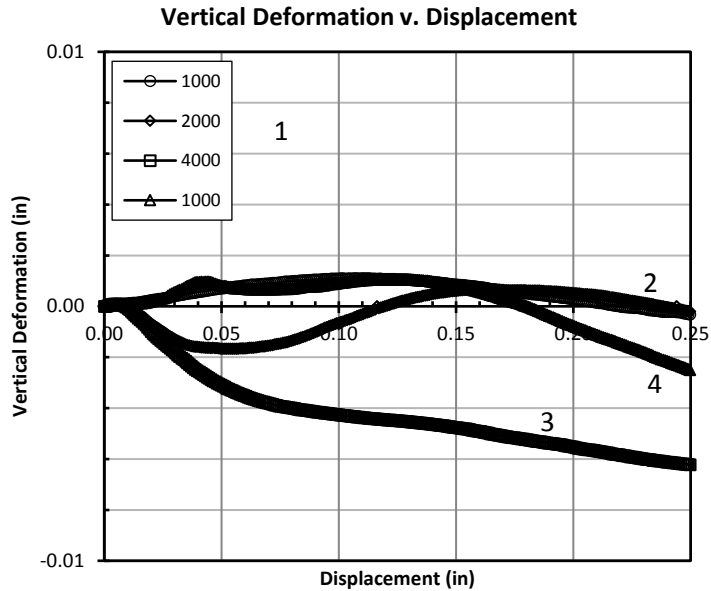


TEST RESULTS	
LIQUID LIMIT	41
PLASTIC LIMIT	18
PLASTICITY INDEX	23
Classification	CL



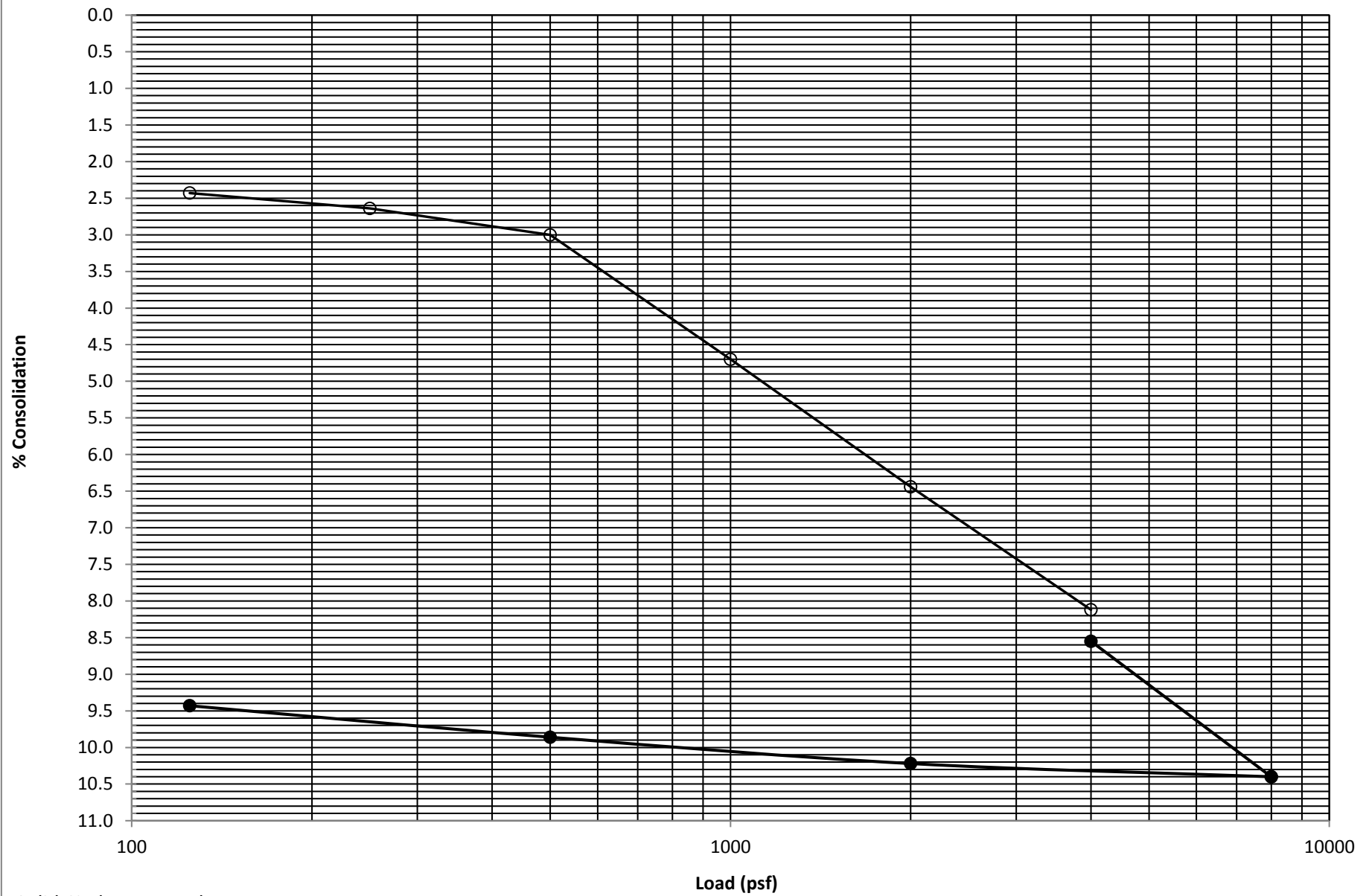
Samples Tested	1	2	3	4
Boring ID	B6	B6	B6	B6
Depth (ft.)	1-5	1-5	1-5	1-5
Initial Moisture Content (%)	9.98	9.98	9.98	9.98
Final Moisture Content (%)	21.75	20.91	19.67	17.62
Normal Stress (psf)	1000	1000	2000	4000
Maximum Shear Stress (psf)	594	670	1659	2569
Ultimate Shear Stress (psf)	588	670	1533	2566
ASTM D3080	Soil Type	Silty Clay	Silty Clay	Silty Clay

	Peak	Ultim.
Friction, ϕ (Deg)	33	33
Cohesion (psf)	81	48
Sample Type:	Remolded	
Method:	Drained	
Consolidation:	Yes	
Saturation:	Yes	
Strain Rate (in/min):	0.0032	



B3
8.5 ft

Consolidation



Solid Circle = Wetted

'R' VALUE CA 301

Client: PSI

Date: 2/14/18

By: LD

Client's Job No.: # 00661261-5296

Sample No.: B-6 @ 1' - 5'

GLA Reference: 2008-026

Soil Type: Brown, Sandy Clay

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	70	55	50	
Initial Moisture Content	%	9.1	9.1	9.1	
Water Added	ml	80	110	130	
Moisture at Compaction	%	16.4	19.1	20.9	
Sample & Mold Weight	gms	3180	3165	3109	
Mold Weight	gms	2101	2110	2104	
Net Sample Weight	gms	1079	1055	1005	
Sample Height	in.	2.49	2.56	2.48	
Dry Density	pcf	112.8	104.8	101.5	
Pressure	lbs	6395	4070	2480	
Exudation Pressure	psi	509	324	197	
Expansion Dial	x 0.0001	75	38	21	
Expansion Pressure	psf	325	165	91	
Ph at 1000lbs	psi	40	60	68	
Ph at 2000lbs	psi	110	140	154	
Displacement	turns	3.34	3.82	4.21	
R' Value		25	9	2	
Corrected 'R' Value		25	9	2	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	8
By Expansion Pressure :	6
TI =	5

SAMPLE NO.:	B-6 @ 1' - 5'			B-3 @ 1.5' - 5'								
Depth:	Sandy Clay			Sandy Clay								
DIRECT SHEAR TEST (type)												
Initial Moisture Content %												
Dry Density (pcf)												
Normal Stress (psf)												
Peak Shear Stress (psf)												
Ultimate Shear Stress (psf)												
Cohesion (psf)												
Internal Friction Angle (degrees)												
EXPANSION TEST UBC STD 18-2												
Initial Dry Density (pcf)												
Initial Moisture Content %												
Final Moisture Content %												
Pressure (psf)												
Expansion Index	Swell	%										
CORROSIVITY TEST												
Resistivity (CTM643) (ohm-cm)	1050			1100								
pH (CTM643)	7.4			7.5								
CHEMICAL TESTS												
Soluble Sulfate (CTM 417) (ppm)	576			246								
Chloride Content (CTM 422) (ppm)	230			145								
Wash #200 Sieve (ASTM-1140) %												
Sand Equivalent (ASTM D2419)												

'R' VALUE CA 301

Client: PSI

Date: 2/14/18

By: LD

Client's Job No.: # 00661261-1

Sample No.: B-3 @ 1.5' - 5'

GLA Reference: 2008-026

Soil Type: Brown, Sandy Clay

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	120	60	50	
Initial Moisture Content	%	7.1	7.1	7.1	
Water Added	ml	80	120	140	
Moisture at Compaction	%	14.2	17.8	19.6	
Sample & Mold Weight	gms	3144	3173	3148	
Mold Weight	gms	2103	2098	2096	
Net Sample Weight	gms	1041	1075	1052	
Sample Height	in.	2.35	2.49	2.501	
Dry Density	pcf	117.5	111.0	106.6	
Pressure	lbs	9250	4100	2680	
Exudation Pressure	psi	736	326	213	
Expansion Dial	x 0.0001	93	20	4	
Expansion Pressure	psf	403	87	17	
Ph at 1000lbs	psi	26	56	66	
Ph at 2000lbs	psi	75	131	151	
Displacement	turns	2.89	3.82	4.09	
R' Value		50	13	4	
Corrected 'R' Value		50	13	4	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	11
By Expansion Pressure :	N/A
TI =	5



APPENDIX C
PERCOLATION TEST RESULTS

Boring Percolation Test Datasheet

Client: **Simon**

Project: **Brea Mall**

Address: **100 Brea Mall**

Brea, CA

Test Date: **2/2/2018**

Work Order No.: **00661261**

Technician: **Lloyd Guss + Manny R.**

Weather: **Sunny, Clear**

Test Hole Number/Location: **B3**

Depth of Total Test Hole bottom below land surface (inches): **9.5** ft.

Test hole cross-section (check a box and indicate size):

☒ Diameter: **8** inches

☐ Square: _____ inches

The land surface at the top of the test hole is (check one):

☐ Undisturbed Native Soil

☐ Cut Surface

☐ Fill Surface

☒ Other (describe): **Paved Parking Lot**

Soil Data from Test Hole:

Depth (ft)	Soil Texture
0 - 4.75	Sandy Clay
4.75 - 7	Silty Sand
7 - 28	Silty Clay

Perforated PVC Pipe Diameter = 4"

Solid end cap on pipe.

Pipe Length = 10.25' (10'3")

Stick up above asphalt = 18"

Depth of water during testing =

35.16 inches

Average Head during testing =

35.1 inches

Test Hole Presoaking

Presoak >12in. = If drains away in 30 min. (x2), Test @10min. If not, Test @30min.

Run #	Start Time (H:M)	Start Depth to WL (ft) bgs	Start Time	End Depth to WL (ft)	End Time	Measured Water Drop (ft)	Measured Water Drop (in)	Elapsed Time (min)
1	9:05	8.38		8.38	9:30	0.0	0.00	0:25
2	9:30	8.38		8.38	10:00	0.0	0.00	0:30

Test Hole Percolation:

Fill to Presoak Level or below

Porchet Method

Run #	Start Time (H:M)	Start Depth to WL (ft) bgs	End Time (H:M)	End Depth to WL (ft)	Measured Water Drop (ft)	Measured Water Drop (in.)	WL % Change (<10%?)	Time Interval (min)	Infiltration Rate (in/hr)
1	10:00	8.38	10:35	8.38	0.00	0.0		35	0.0
2	10:35	8.38	11:05	8.38	0.00	0.0	0.0	30	0.0
3	11:05	8.38	11:35	8.38	0.00	0.0	0.0	30	0.0
4	11:35	8.38	12:11	8.39	0.01	0.1	0.3	36	0.0
5	12:17	8.39	12:51	8.42	0.03	0.4	0.7	34	0.0
6	12:56	8.00	13:26	8.00	0.00	0.0	1.0	30	0.0
7	13:26	8.00	13:56	8.00	0.00	0.0	0.0	30	0.0
8	13:56	8.00	14:26	8.02	0.02	0.2	0.7	30	0.0
9	14:26	8.02	14:56	8.04	0.02	0.2	0.0	30	0.0
10	14:56	8.04	15:26	8.07	0.03	0.4	0.3	30	0.0
11	15:26	8.07	15:56	8.08	0.01	0.1	0.7	30	0.0

Final Stabilized Infiltration Rate: **0.0** in/hour

Boring Percolation Test Datasheet

Client: **Simon**

Project: **Brea Mall**

Address: **100 Brea Mall**

Brea, CA

Test Date: **2/2/2018**

Work Order No.: **00661261**

Technician: **Lloyd Guss + Manny R.**

Weather: **Sunny, Clear**

Test Hole Number/Location: **B4**

Depth of Total Test Hole bottom below land surface (inches): **9.7** ft.

Test hole cross-section (check a box and indicate size):

☒ Diameter: **8** inches

☐ Square: _____ inches

The land surface at the top of the test hole is (check one):

☐ Undisturbed Native Soil

☐ Cut Surface

☐ Fill Surface

☒ Other (describe): **Paved Parking Lot**

Soil Data from Test Hole:

Depth (ft)	Soil Texture
0 - 5	Sand
5 - 12	Sand
12 -18	Sandy Silt

Perforated PVC Pipe Diameter = 4"

Solid end cap on pipe.

Pipe Length = 9.9' (9'11")

Stick up above asphalt = 0"

Depth of water during testing =

22.44 inches

Average Head during testing =

19.92 inches

Test Hole Presoaking

Presoak >12in. = If drains away in 30 min. (x2), Test @10min. If not, Test @30min.

Run #	Start Time (H:M)	Start Depth to WL (ft) bgs	Start Time	End Depth to WL (ft)	End Time	Measured Water Drop (ft)	Measured Water Drop (in)	Elapsed Time (min)
1	10:10	9.6		9.7	10:35	0.1	1.20	0:25
2	10:31	8.03		9.7	10:56	1.7	20.04	0:25

Test Hole Percolation:

Fill to Presoak Level or below

Porchet Method

Run #	Start Time (H:M)	Start Depth to WL (ft) bgs	End Time (H:M)	End Depth to WL (ft)	Measured Water Drop (ft)	Measured Water Drop (in.)	WL % Change (<10%?)	Time Interval (min)	Infiltration Rate (in/hr)
1	10:58	8.04	11:23	8.90	0.86	10.3		25	2.3
2	11:27	8.03	11:37	8.52	0.49	5.9	19.8	10	3.2
3	11:40	8.05	11:50	8.50	0.45	5.4	2.1	10	3.0
4	11:55	8.05	12:05	8.50	0.45	5.4	0.0	10	3.0
5	12:07	8.03	12:17	8.55	0.52	6.2	3.7	10	3.4
6	12:20	8.03	12:30	8.51	0.48	5.8	2.1	10	3.2
7	12:37	8.03	12:47	8.45	0.42	5.0	3.2	10	2.8

Final Stabilized Infiltration Rate: **2.8** in/hour



APPENDIX D

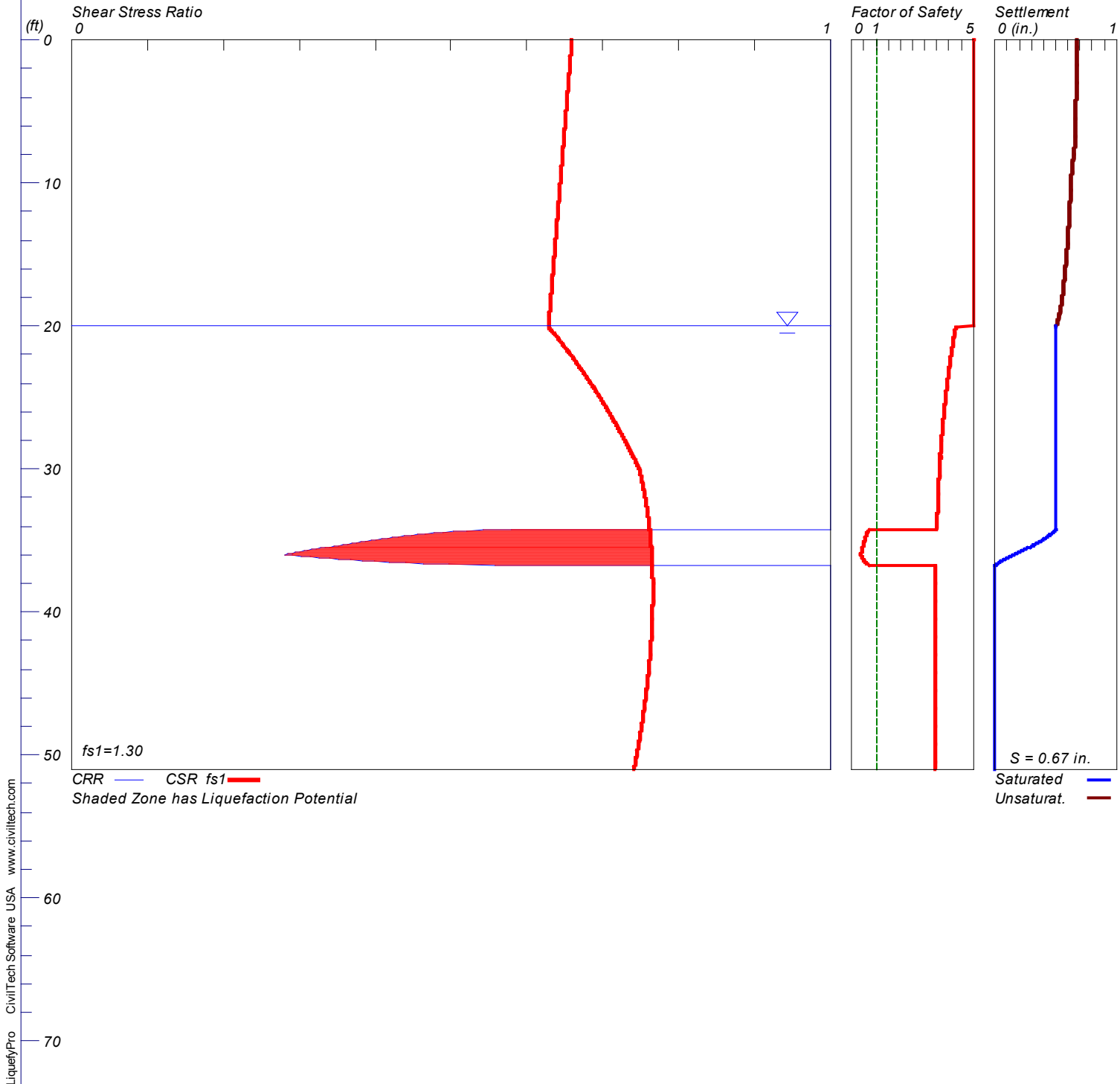
LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

Simon ~ Brea Mall

Hole No.=B4 Water Depth=20 ft

Magnitude=6.69
Acceleration=0.78g



LIQUEFACTION ANALYSIS SUMMARY
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Input File Name: P:\0066\GEO\2018\1261 Simon BreaMall GEO\Analysis\00661261 Liquef B4.liq
Title: Simon ~ Brea Mall
Subtitle: SW Development

Surface Elev.=
Hole No.=B4
Depth of Hole= 51.00 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 25.00 ft
Max. Acceleration= 0.78 g
Earthquake Magnitude= 6.69

Input Data:

Surface Elev.=
Hole No.=B4
Depth of Hole=51.00 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 25.00 ft
Max. Acceleration=0.78 g
Earthquake Magnitude=6.69

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine*
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, $C_e = 1.25$
7. Borehole Diameter, $C_b = 1.15$
8. Sampling Method, $C_s = 1.2$
9. User request factor of safety (apply to CSR) , User= 1.3
Plot one CSR curve ($f_{s1} = \text{User}$)
10. Use Curve Smoothing: Yes*
- * Recommended Options

In-Situ Test Data:

Depth ft	SPT pcf	gamma %	Fines
-------------	------------	------------	-------

0.00	48.00	111.00	5.00
6.00	48.00	111.00	5.00
8.00	35.00	114.00	5.00

11.00	58.00	114.00	5.00
16.00	58.00	115.00	30.00
21.00	27.00	115.00	60.00
26.00	23.00	120.00	30.00
31.00	22.00	115.00	60.00
36.00	14.00	120.00	10.00
41.00	69.00	120.00	10.00
46.00	32.00	120.00	60.00
51.00	19.00	120.00	60.00

Output Results:

Settlement of Saturated Sands=0.50 in.
Settlement of Unsaturated Sands=0.17 in.
Total Settlement of Saturated and Unsaturated Sands=0.67 in.
Differential Settlement=0.336 to 0.444 in.

Depth ft	CRRm	CSRsf in.	F.S. in.	S_sat. in.	S_dry	S_all
-------------	------	--------------	-------------	---------------	-------	-------

0.00	2.68	0.66	5.00	0.50	0.17	0.67
1.00	2.68	0.66	5.00	0.50	0.17	0.67
2.00	2.68	0.66	5.00	0.50	0.17	0.67
3.00	2.68	0.65	5.00	0.50	0.17	0.67
4.00	2.68	0.65	5.00	0.50	0.16	0.67
5.00	2.68	0.65	5.00	0.50	0.16	0.66
6.00	2.68	0.65	5.00	0.50	0.16	0.66
7.00	2.68	0.65	5.00	0.50	0.15	0.66
8.00	2.68	0.65	5.00	0.50	0.15	0.65
9.00	2.68	0.65	5.00	0.50	0.13	0.63
10.00	2.68	0.64	5.00	0.50	0.12	0.62
11.00	2.68	0.64	5.00	0.50	0.12	0.62
12.00	2.68	0.64	5.00	0.50	0.11	0.61
13.00	2.68	0.64	5.00	0.50	0.10	0.61
14.00	2.68	0.64	5.00	0.50	0.10	0.60
15.00	2.68	0.64	5.00	0.50	0.09	0.59
16.00	2.68	0.63	5.00	0.50	0.08	0.58
17.00	2.68	0.63	5.00	0.50	0.06	0.57
18.00	2.68	0.63	5.00	0.50	0.05	0.55
19.00	2.68	0.63	5.00	0.50	0.03	0.53
20.00	2.68	0.63	5.00	0.50	0.00	0.50
21.00	2.68	0.64	4.17	0.50	0.00	0.50
22.00	2.68	0.66	4.08	0.50	0.00	0.50
23.00	2.68	0.67	3.99	0.50	0.00	0.50
24.00	2.68	0.68	3.92	0.50	0.00	0.50
25.00	2.68	0.70	3.85	0.50	0.00	0.50
26.00	2.68	0.71	3.79	0.50	0.00	0.50
27.00	2.68	0.72	3.73	0.50	0.00	0.50
28.00	2.68	0.73	3.68	0.50	0.00	0.50
29.00	2.68	0.74	3.63	0.50	0.00	0.50
30.00	2.69	0.75	3.59	0.50	0.00	0.50
31.00	2.68	0.75	3.56	0.50	0.00	0.50
32.00	2.67	0.76	3.54	0.50	0.00	0.50
33.00	2.67	0.76	3.51	0.50	0.00	0.50

34.00	2.66	0.76	3.49	0.50	0.00	0.50
35.00	0.39	0.76	0.52*	0.38	0.00	0.38
36.00	0.28	0.76	0.37*	0.15	0.00	0.15
37.00	2.64	0.77	3.44	0.00	0.00	0.00
38.00	2.63	0.77	3.43	0.00	0.00	0.00
39.00	2.62	0.77	3.42	0.00	0.00	0.00
40.00	2.61	0.77	3.41	0.00	0.00	0.00
41.00	2.61	0.76	3.41	0.00	0.00	0.00
42.00	2.60	0.76	3.40	0.00	0.00	0.00
43.00	2.59	0.76	3.40	0.00	0.00	0.00
44.00	2.58	0.76	3.40	0.00	0.00	0.00
45.00	2.58	0.76	3.40	0.00	0.00	0.00
46.00	2.57	0.76	3.40	0.00	0.00	0.00
47.00	2.56	0.75	3.40	0.00	0.00	0.00
48.00	2.55	0.75	3.40	0.00	0.00	0.00
49.00	2.55	0.75	3.41	0.00	0.00	0.00
50.00	2.54	0.74	3.41	0.00	0.00	0.00
51.00	2.53	0.74	3.42	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone

(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units Depth = ft, Stress or Pressure = atm (tsf), Unit Weight = pcf, Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft²)

CRRm Cyclic resistance ratio from soils

CSRsf Cyclic stress ratio induced by a given earthquake (with user request factor of safety)

F.S. Factor of Safety against liquefaction, F.S.=CRRm/CSRsf

S_sat Settlement from saturated sands

S_dry Settlement from Unsaturated Sands

S_all Total Settlement from Saturated and Unsaturated Sands

NoLiq No-Liquefy Soils