Appendix C1
Geotechnical and Infiltration Evaluation

UPDATED GEOTECHNICAL AND INFILTRATION EVALUATION
FOR
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
1515 WEST 178TH STREET
CITY OF GARDENA, LOS ANGELES COUNTY, CALIFORNIA

PREPARED FOR

MELIA HOMES
8951 RESEARCH DRIVE
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PREPARED BY

GEOTEK, INC.
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PROJECT NO. 1949-CR **SEPTEMBER 10, 2018**

GEOTEK

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September 10, 2018
Project No. 1949-CR

Melia Homes
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Irvine, California 92618

Attention: Mr. Chad Brown

Subject: Updated Geotechnical and Infiltration Evaluation
Proposed Multi-Family Residential Development
1515 West 178th Street
City of Gardena, Los Angeles County, California

Dear Mr. Brown:

We are pleased to provide herein the results of our updated geotechnical and infiltration evaluation for the subject site located in the city of Gardena, Los Angeles County, California. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for earthwork, foundation design, and construction. In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development.

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

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to complete a geotechnical evaluation of the existing geotechnical conditions of the project site with respect to currently anticipated site development. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site,
- Site reconnaissance,
- Site exploration consisting of the excavation, logging, and sampling of three exploratory hollow-stem auger borings and logging and percolation testing of two hollow-stem auger borings,
- Collection of relatively undisturbed and bulk soil samples of the onsite materials,
- Laboratory testing of the soil samples obtained from the site,
- Review and evaluation of site seismicity, and
- Compilation of this geotechnical and infiltration report which presents our findings, conclusions, and recommendations for site development.

The intent of this report is to aid in the evaluation of the site for future proposed development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report may need to be updated based upon our review of the final site development plans. These plans should be provided to GeoTek, Inc. (GeoTek) for review when available.

The scope of this study does not include an assessment of environmental concerns associated with the previous and current use of the site.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The subject project site is located and addressed as 1515 West 178th Street in the city of Gardena, Los Angeles County, California. The site is a rectangular-shaped parcel consisting of approximately 5.6 acres. The property is currently occupied by a company which provides services of storage and transportation of cargo. The subject facility includes a 94,000-square foot building and associated parking lot, underground utilities, as well as hardscape and landscape improvements.

The site has a generally flat topography with a gentle fall of three to five feet to the north-northwest. Surface drainage is to the north-northwest following site topography.

The site is bounded by an Edison easement to the north, a mobile home park to the west, commercial buildings to the east, and West 178th Street with commercial buildings to the south.

The general location of the site is shown in Figure 1. The current conditions of the site are displayed on a Google Earth aerial image shown as Figure 2, Exploration Location Map.

2.2 PROPOSED DEVELOPMENT

It is our understanding that proposed development will consist of demolition of the existing site improvements, earthwork, and subsequent construction of 118 townhomes and related parking/drive areas, underground utilities, and landscape improvements. The structures are anticipated to be up to three stories in height and to utilize either shallow foundations or post-tensioned slabs. Cuts and fills are estimated to be minor (less than five feet in height). In addition, stormwater at the site may be managed via a 25-foot wide basin to be constructed near the north property line. The specific depth of the basin is unknown currently. For the preparation of this report, however, we have considered infiltration tests at two locations within the proposed basin area at approximately five feet deep.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available. Additional geotechnical field exploration, analyses and recommendations may be necessary upon review of site development plans.

3. DOCUMENT REVIEW

On December 3, 2004, Petra Geotechnical Inc., (Petra) completed a report entitled *Geotechnical Investigation, Proposed Residential Development, 115 West 178th Street, Gardena, California*. This study excavated four hollow-stem auger borings to depths ranging from 21.5 feet to 51.5 feet across the site. Petra reported the presence artificial fills in all their borings ranging between two and four feet in thickness. The fills were described as moist, medium dense to dense clayey sand containing varying amounts of gravel. Below the fills, terrace deposits were reported to exist and to be composed of moist to very moist, medium dense to hard (stiff to hard) clayey sand, sandy clay, silty sand, and silty clay. Borings B-1 and B-4 reportedly encountered groundwater at depths of about 29 and 32 feet, respectively. Petra stated that historic high groundwater level in the project area was about 15 to 20 feet below the ground surface, per the *Seismic Hazard Zone Report for the Torrance Quadrangle* (CDMG, 1998). The potential for liquefaction at the site was considered unlikely due to the high density of the sandy soils or clayey composition of the terrace deposits. The study recommended removal depths on the order of three to five feet. It also pointed out the on-site fill and native terrace deposits had a "medium" potential for expansion (EI ≈ 51), negligible sulfate content, and poor R-value characteristics (R-Value ≈ 6). Shrinkage on the order of 10 to 15 percent for the on-site fill and of 5 to 10 percent for the terrace deposits, as well as a subsidence of about 0.15 feet were estimated by Petra. The study furnished seismic design parameters for the site based on the 1997 *Uniform Building Code* and geotechnical parameters for design of conventional shallow foundations and post-tensioned slabs at the site.

On April 29, 2016, Petra completed a report entitled *Updated 2013 CBC Seismic Design Parameters; Proposed Residential Development, 1515 West 178th Street, Gardena, California*. The updated report utilized the findings of the site explorations and laboratory test results reported by Petra in 2004. The study provided updated seismic design parameters for the site based on the 2003 *California Building Code (CBC)* and updated soils parameters for design of conventional spread footings and post-tensioned slabs.

In April of 2016, Petra also finalized a *Percolation Test Summary, 1515 West 178th Street, Gardena*. This summary presented the data of two percolation tests performed within the western region of the site. Both tests were apparently conducted at a depth of 5.2 feet. After the application of the Porchet Method to the estimated percolation rates, infiltration rates of 4.09 and 8.77 inches per hour were estimated by Petra for the site soils at five feet. It should be noted that these results seem to disagree with the clayey soil profile displayed by Petra's boring logs.

Logs of the exploratory borings, laboratory test results, and infiltration test data by Petra (2004) are included in Appendix A. The locations of these explorations are shown on the Exploration Location Map presented as Figure 2.

4. FIELD EXPLORATION, LABORATORY TESTING, AND PERCOLATION TESTING

4.1 FIELD EXPLORATION

The soils underlying the site were explored on August 20, 2018 by means of excavating two exploratory borings (B-1 and B-2) within the intended building areas to depths of 26.5 feet below the existing ground surface. In addition, one exploratory boring (B-3) approximately 16.5 feet deep and two percolation test borings (P-1 and P-2) approximately six feet deep were advanced within the future basin area near the north property line. The borings were drilled with a truck-mounted hollow-stem auger drill rig.

The approximate locations of our site explorations and the borings by Petra are shown on the Exploration Location Map, Figure 2. Logs of the borings by GeoTek are provided in Appendix B.

4.2 LABORATORY TESTING

Laboratory testing was performed on selected relatively undisturbed and bulk soil samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the soil materials encountered and to evaluate the soils physical properties for use in the engineering design and analysis. Results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included in Appendix C.

4.3 PERCOLATION TESTING

Percolation testing was performed at boring locations P-1 and P-2 to assess the infiltration characteristics of the site soils underlying the proposed basin area. At the time of this investigation, the specific depth of the basin invert was unknown. For this evaluation, we assumed that the invert of the basin will be located approximately five feet below the existing ground surface. Percolation test borings were excavated to approximately one foot below the anticipated invert of the basin (i.e. six feet). The boring diameter was approximately 8 inches. Percolation testing was performed within the lower 30 to 40 inches in the borings by a representative of our firm, in general conformance with the Boring Percolation Test Procedure outlined in the *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (County of Los Angeles, 2017).

The field percolation rates are presented in the following table for each of the borings. As required, the percolation rates were corrected to account for discharge of water from both the sides and bottom of the borings. This correction was done using the Porchet Method, obtaining the infiltration rates tabulated below:

SUMMARY OF TEST RESULTS		
Boring	Field Percolation Rate (inches per hour)	Field Infiltration Rate (inches per hour)
P-1	0.20	0.01
P-2	0.20	0.01

A suitable factor of safety should be applied to the field rates to design the infiltration system. Detailed percolation/infiltration test data is included in Appendix D.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. Basically, it extends roughly 975 miles from the north and extends from the Transverse Ranges geomorphic province to the tip of Baja California, from north to south. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by older alluvial deposits (Saucedo, G.J., Greene, G.H., Kennedy, M.P., and Bezore, S.P., 2016). The closest fault to the subject site is the Newport-Inglewood Fault North Los Angeles Basin Section located approximately 3.0 miles to the east.

5.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the earth materials encountered on the site by Petra (2004) and recently by GeoTek is presented in the following sections.

5.2.1 Undocumented Artificial Fill

Undocumented artificial fill was encountered in two of our borings (Boring B-3 and P-1) to approximately three to four feet below the existing ground surface. While the rest of our borings did not note fill, fill is anticipated to be present below the existing asphalt concrete pavement and building areas. Petra (2004) also reported about two to four of fill under the site. The fill consisted of brown to reddish brown, moist, loose/soft to medium dense/stiff silty sand with gravel and clayey sand.

5.2.2 Older Alluvial Deposits

Older alluvium was encountered in our borings below the fill or below the existing asphalt concrete and extended to the maximum depth explored of about 26.5 feet. The alluvium encountered generally consisted of surficial layers of sandy lean clay and sandy silt underlain by units of silty sand and sand. The alluvium was brown to olive brown in color, moist, and stiff/dense to very stiff/very dense to the total depth explored, based on our field observations, blow counts, and in-place density determinations. The logs of the borings reported by Petra (2004) display relatively similar conditions with more predominantly clayey soils at depths.

The near surface site soils tested were found to have a "low" expansion potential when tested and classified in accordance with ASTM D 4829. Petra reported a "medium" potential for expansion for the surficial site soils.

5.3 SURFACE AND GROUNDWATER

5.3.1 Surface Water

If encountered during the earthwork construction, surface water on this site is the result of precipitation or surface run-off from surrounding sites. Overall drainage in the area is variable, and most commonly directed toward the north-northwest. Provisions for surface drainage will need to be accounted for by the project civil engineer.

5.3.2 Groundwater

Groundwater was not encountered in any of our borings drilled at the site to a maximum depth of 26.5 feet. However, Petra's deepest borings B-1 and B-4 encountered groundwater at 29 feet and 32 feet below the ground surface, respectively.

Our review of the *Historically Highest Groundwater Map* published within the *Seismic Hazard Zone Report for the Torrance Quadrangle (DMG, 1998)* did not reveal past high groundwater levels in the general area of the site. High groundwater levels on the order of ten feet below ground surface were shown on this map but for areas immediately adjacent to the existing Dominguez Channel located approximately one mile from the site to the east.

The GeoTracker database shows several groundwater monitoring wells for a property located across the street (addressed as 1500 West 178th Street) from the site, with depth to groundwater ranging from 30 to 35 feet. This information agrees with the groundwater levels of 29 to 32 feet reported by Petra (2004).

Perched groundwater or localized seepage can occur due to variations in rainfall, irrigation practices, and other factors not evident at the time of this investigation.

5.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced liquefaction or landsliding. The nearest zoned fault is the Newport-Inglewood Fault North Los Angeles Basin Section, located approximately 3.0 miles to the west.

5.4.1 Seismic Design Parameters

The site is located at approximately 33.8699 Latitude and -118.3036 Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, was determined from the USGS Website, Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude/Longitude. The results are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.603g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.593g
Site Coefficient for Site Class "D", F_a	1.0
Site Coefficient for Site Class "D", F_v	1.5
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.603g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.890g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.068g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.593g
Peak Ground Acceleration Adjusted for Site Class Effects, PGA_M	0.598g

5.5 LIQUEFACTION AND SEISMICALLY-INDUCED SETTLEMENT

The depth to groundwater in the site area is on the order of 30 feet. The logs of the deep borings reported by Petra (2004) indicate that mostly clayey soils, which are typically non-liquefiable, are present below 30 feet. The cited logs also show lesser layers with sandy soils at the referenced depths. High blow counts were recorded by Petra (2004) in these granular units; thus, they are considered to be not prone to liquefaction. Based on the above, the potential for soil liquefaction at the site is very low.

Seismically-induced settlement of the sandy soils above the groundwater table is anticipated to be on the order of 0.5 inches total and 0.25 inches differential.

5.6 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigation. Thus, the potential for landslides is considered negligible.

The potential for secondary seismic hazards such as a seiche or tsunami is considered negligible due to site elevation and distance to an open body of water.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Development of the site appears feasible from a geotechnical viewpoint. The following recommendations should be incorporated into the design and construction phases of development.

6.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Gardena, the *2016 California Building Code* (CBC), and recommendations contained in this report. Site grading plans should be reviewed by this office when they become available. Additional recommendations will likely be offered subsequent to review of these plans.

6.2.1 Site Clearing and Preparation

Site preparation should start with demolition/razing of existing site improvements and removal of deleterious materials, and vegetation. Demolition should include removal of all pavements, floor slabs, foundations, and any other below-grade construction. These materials should be properly disposed of off-site. Voids resulting from site clearing (such as removals of underground utilities, private sewage disposal systems, foundations, etc) should be replaced with engineered fill materials.

6.2.2 Removals

All existing fills and loose/soft portions of the older alluvium should be removed to expose competent alluvial materials. Competent alluvium is defined as native materials that are visually non-porous and having a relative compaction of at least 85 percent of the soil's maximum dry density as determined per ASTM D 1557. Based on our boring data and the data reported by Petra (2004), combined fill and alluvial removals of about three to five feet are anticipated to be required within the structural grading limits. As a minimum, removals should extend down and away from foundation elements at a 1:1 (h:v) projection to the recommended removal depth, or a minimum of five feet laterally.

A minimum 24 inches of engineered fill should be provided below the bottom of the proposed foundations. A representative of this firm should observe the bottom of all excavations.

A minimum of 12 inches of engineered fill should be provided below asphaltic concrete pavement and Portland cement concrete hardscape areas. The horizontal extent of removals should extend at least two feet beyond the edge.

Development plans should be reviewed by this firm when available. Depending on actual field conditions encountered during grading, locally deeper areas of removal may be recommended.

The bottom of all removals should be scarified to a minimum depth of six inches, brought to slightly above the optimum moisture content, and then recompacted to at least 90 percent of the soil's maximum dry density (ASTM D 1557). The bottoms of removals should be observed by a GeoTek representative prior to scarification.

6.2.3 Fills

The onsite soils are considered suitable for reuse as engineered fill provided they are free from vegetation, roots, and rock/concrete or hard mumps greater than six inches in maximum dimension.

Concrete generated from the demolition of existing site improvements may be incorporated into site fills provided the following guidelines are implemented: 1) concrete should be free of rebar or other deleterious materials and should be broken down to a maximum dimension of six inches; 2) concrete should not be placed within three feet of finish grade in the building pad areas or within one foot of subgrade elevations in the street/drive areas; 3) concrete should be distributed in the fill and should not be "nested" or placed in concentrated pockets.

The undercut areas should be brought to final pad elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Fill materials should be placed at or above optimum moisture content and should be compacted to a minimum relative compaction of 90 percent as determined by ASTM Test Method D 1557. Additional recommendations pertaining to fill placement are presented in Appendix E.

6.2.4 Excavation Characteristics

Excavation in the onsite soil materials is expected to be easy using heavy-duty grading equipment in good operating conditions.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the onsite materials should be stable at 1:1 (h:v) inclinations for cuts less than ten feet in height.

6.2.5 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, bulking, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage, bulking, and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 10 to 15 percent for the existing fills and of 5 to 10 percent for the upper alluvium may be considered. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction. Bulking is not considered to be a significant factor with the underlying materials within the vicinity of the anticipated construction. Subsidence on the order of up to 0.1-foot could occur.

6.2.6 Trench Excavations and Backfill

Temporary excavations within the onsite materials should be stable at 1:1 (h:v) inclinations for short durations during construction, and where cuts do not exceed ten feet in height. Temporary cuts to a maximum height of four feet can be excavated vertically, but local sloughing and/or failure could occur due to the granular nature of some of the soils at this site. Increased caution should be applied when working near or within any excavations at this site.

Trench excavations should conform to Cal-OSHA regulations. The contractor should have a competent person, per OSHA requirements, on site during construction to observe conditions and to make the appropriate recommendations.

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined per ASTM D 1557). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for road pavements should be compacted to at least 95 percent relative compaction. Much of the onsite materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than 6± inches are removed.

Compaction should be achieved with a mechanical compaction device. Ponding or jetting of trench backfill is not recommended. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

6.3 DESIGN RECOMMENDATIONS

6.3.1 Foundation Design Criteria

The site soils are expected to generally have "low" (21≤EI≤50) to "medium" (51≤EI≤90) expansion potential in accordance with ASTM D 4829. The foundation elements for the proposed structures should bear entirely in engineered fill soils and should be designed in accordance with the *2016 California Building Code (CBC)*.

Presented below are post-tensioned foundation design parameters for the proposed residential dwellings at the site. These parameters are in general conformance with *Design of Post-Tensioned Slabs-on-Ground*, Third Edition *with 2008 Supplement* (PTI, 2008). These are minimal recommendations and are not intended to supersede the design by the project structural engineer.

DESIGN PARAMETERS FOR POST-TENSIONED SLABS		
Foundation Design Parameter	Design Value	
	"Low" Expansion Potential (LL≤34; PI≤19; Passing #200 Sieve ≈ 70%; Clay fines ≈ 30%)	"Medium" Expansion Potential (LL≤46; PI≤22; Passing #200 Sieve ≈ 83%; clay fines ≈ 14%)
Edge Moisture Variation Distance, e_m - Edge Lift (swelling) - Center Lift (shrinkage)	 4.8 ft 9.0 ft	 4.2 ft 8.2 ft
Soil Differential Movement, y_m - Edge Lift (swelling) - Center Lift (shrinkage)	 ≈0.48 in ≈-0.21 in	 ≈0.81 in ≈-0.34 in
Ext. Perimeter Beam Embedment	One- or Two-Story – 12 inches* Three-Story – 18 inches*	One- or Two-Story – 18 inches* Three-Story – 18 inches*
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 110% to a depth of 12 inches	Minimum 120% to a depth of 18 inches

* Required depth of perimeter beam/stiffening rib per structural calculations may govern.
The following assumptions were used to generate e_m and y_m values: Thornthwaite Moisture Index = -20; constant suction value = 3.9pF; post-equilibrium case assumed with wet (swelling) cycle going from 3.9pF to 3.0pF and drying (shrinking) cycle going from 3.9pF to 4.5pF.

Post-tensioned slabs should be designed in accordance with the *2016 CBC* and PTI design methodology.

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

A summary of our design recommendations for conventionally reinforced foundations is presented in the table below:

DESIGN PARAMETERS FOR CONVENTIONALLY REINFORCED FOUNDATIONS		
Design Parameter	"Low" Expansion Potential	"Medium" Expansion Potential
Foundation Depth or Minimum Perimeter Beam Depth for Both Interior and Exterior Footings (inches below lowest adjacent finished grade)	One- and Two-Story – 12 Three-Story - 18	One- and Two-Story – 18 Three-Story - 18
Minimum Foundation Width (Inches)*	One- and Two-Story – 12 Three-Story - 15	One- and Two-Story – 12 Three-Story - 15
Minimum Slab Thickness (inches)	4 (actual)	4 (actual)
Sand Blanket and Moisture Retardant Membrane below On-Grade Building Slabs	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**
Minimum Slab Reinforcing	No. 3 rebars 24 inches on-center, each way, placed in middle 1/3 of slab thickness	No. 3 rebars 18 inches on-center, each way, placed in middle 1/3 of slab thickness
Minimum Footing Reinforcement for Continuous Footings, Grade Beams, and Retaining Wall Footings	Two No. 4 reinforcing bars, one top and one bottom	Four No. 4 reinforcing bars, two top and two bottom
Effective Plasticity Index***	13-20	20-25
Presaturation of Subgrade Soil (Percent of Optimum/Depth in inches)	Minimum 110% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete.	Minimum 120% of the optimum moisture content to a depth of at least 18 inches prior to placing concrete.

*Code minimums per Table 1809.7 of the 2016 CBC should be complied with.
**Sand should have a sand equivalent of at least 30.
***Effective Plasticity Index should be verified at the completion of the rough grading

In general, an allowable bearing capacity of 1,500 pounds per square foot (psf) may be used for footings a minimum 12 inches deep and 12 inches wide. This value may be increased by 400 psf for each additional 12 inches in depth and 100 psf for each additional 12 inches in width to a maximum value of 3,000 psf.

The passive earth pressure may be computed as an equivalent fluid having a density of 200 psf per foot of depth, to a maximum earth pressure of 2,000 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. The upper one foot of soil below the adjacent grade should not be used in calculating passive pressure.

The above values may be increased as allowed by Code to resist short-term transient loads (e.g. seismic and wind loads).

For footings designed in accordance with the recommendations presented in this report, we would anticipate a maximum static settlement of less than one-inch and a maximum differential static settlement of less than ½-inch in a 40-foot span. Seismically-induced settlement is expected to be about 0.5 inches total and 0.25 inches differential in a 40-foot span.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these systems are provided in the 2016 *California Green Building Standards Code (CALGreen)* Section 4.505.2 and the 2016 *CBC* Section 1910.1.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as the result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. It is GeoTek's opinion that a minimum ten mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and atmospheric conditions.

Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e. thickness, composition, strength, and permeance) to achieve the desired performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

6.3.2 Miscellaneous Foundation Recommendations

- To minimize moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.
- Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

- Under-slab utility trenches should be compacted to project specifications. Compaction should be achieved with a mechanical compaction device. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

6.3.3 Foundation Set Backs

Foundations should comply with the following setbacks. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The following recommendations are presented:

- The outside bottom edge of all footings should be set back a minimum of H/2 (where H is the slope height) from the face of any ascending slope. The setback should be at least five feet and need not to exceed 15 feet. Where a retaining wall is constructed at the toe of the slope, the height of the slope should be measured from top of the wall to the top of the slope.
- The outside bottom edge of all footings should be set back a minimum of H/3 from the face of any descending slope. The setback should be at least seven feet and need not exceed 40 feet.
- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom inside edge of the wall stem.
- The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom of the nearest excavation.

6.3.4 Retaining Wall Design and Construction

6.3.4.1 General Design Criteria

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

Retaining wall foundations should be embedded into engineered fill and should be designed in accordance with *Section 6.3.1* of this report. Structural needs may govern and should be evaluated by the project structural engineer.

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

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of *Section 6.3.4.4* in this report.

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

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

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

6.3.4.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES		
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) Select Imported Backfill*	Equivalent Fluid Pressure (pcf) Select Native Backfill**
Level	36	51
2:1	55	112

*The design pressures assume the imported backfill material has an expansion index less than or equal to 20 and friction angle of at least 34 degrees. Backfill zone includes area between the back of the wall and footing to a plane (1:1 h:v) up from the bottom of the wall foundation to the ground surface.
**The design pressures assume the native backfill material has an expansion index less than or equal to 50 and friction angle of at least 25 degrees. Backfill zone includes area between the back of the wall and footing to a plane (1:1 h:v) up from the bottom of the wall foundation to the ground surface.

6.3.4.3 Restrained Retaining Walls

Retaining walls that will be restrained prior to placing and compacting backfill material or that have reentrant or male corners should be designed for an at-rest equivalent fluid pressure of 60 pcf, plus any applicable surcharge loading, for select imported backfill and level back slope condition. For select native backfill, an at-rest equivalent fluid pressure of 73 pcf should be used. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

6.3.4.4 Retaining Wall Backfill and Drainage

Retaining wall backfill should be free of deleterious and/or oversized materials and should have properties indicated in *Section 6.3.4.2*. Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one-cubic foot per linear foot of ¾ to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining wall backfill should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D 1557. The wall backfill should also include a minimum one-foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The rock should be separated from the earth with filter fabric. The upper 24 inches should consist of compacted on-site soil.

As an alternative to the drain rock and fabric, Miradrain 2000, or approved equivalent, may be used behind the retaining wall. The Miradrain 2000 should extend from the base of the wall to within two feet of the ground surface. The subdrain should be placed at the base of the wall in direct contact with the Miradrain 2000.

The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. Proper surface drainage needs to be provided and maintained.

6.3.4.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved the project geotechnical engineer or their authorized representative.

6.3.5 Pavement Design Considerations

Pavement design for proposed street improvements was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. Based on an assumed design R-value of 6 and for Traffic Indices (TIs) of 5.0 and 6.0 generally linked to roads with light vehicular traffic with occasional heavy truck traffic, the following preliminary sections were calculated:

GEOTECHNICAL RECOMMENDATION FOR MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0	4	8
6.0	4	12

Traffic Indices (TIs) used in our pavement design are considered reasonable values for the proposed residential street areas and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure. Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper 5 feet) after final grading has been completed.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. Pavement base should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum dry density (modified proctor).

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with the City of Gardena specifications, and under the observation and testing of GeoTek and a City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557). If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.

6.3.6 Soil Corrosivity

The soil resistivity was tested in the laboratory on a sample collected during our field exploration. The results of the testing (2,010 ohm-cm) indicate that the soil sample is "highly corrosive" to buried ferrous metals, based on the guidelines provided in *Corrosion Basics: An Introduction* (Roberge, 2005). Consideration should be given to consulting with a corrosion engineer.

6.3.7 Soil Sulfate Content

The sulfate content was determined in the laboratory for a representative soil sample obtained during our field exploration. The results (0.0150%) indicate that the water soluble sulfate range is less than 0.1 percent by weight which is considered "not applicable" (i.e. negligible) as per Table 4.2.1 of ACI 318. Based upon the test results, no special concrete mix design is required by Code for sulfate attack resistance. Additional testing of soils collected near finish grade should be performed subsequent to site grading.

6.3.8 Import Soils

Import soils should have an expansion index similar to the on-site soils or better. GeoTek also recommends that, as a minimum, proposed import soils be tested for soluble sulfate content. GeoTek should be notified a minimum of 72 hours of potential import sources so that appropriate sampling and laboratory testing can be performed.

6.3.9 Concrete Flatwork

6.3.9.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required due to the non-structural nature. However, the use of some reinforcement should be considered. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in residential construction.

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

Subgrade soils, classified as having "low" expansion potential, should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. at the subject site should be pre-saturated to a minimum of 110 percent of optimum moisture content to a depth of 12 inches. Subgrade soils with a "medium" expansion potential should be pre-saturated to a minimum of 120 percent of optimum moisture content to a depth of 18 inches.

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

6.3.9.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 1/8 inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent on a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is also subject to internal expansion and contraction due to external changes over time.

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

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

6.4 POST CONSTRUCTION CONSIDERATIONS

6.4.1 Landscape Maintenance and Planting

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

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

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas.

6.4.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. Pad drainage should be directed toward approved area(s) and not be blocked by other improvements.

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

6.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of onsite and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trenches.
- Perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

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

7. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of our evaluation is limited to the boundaries of the subject site. This review does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our fee estimate (P-0501418) dated May 8, 2018 and geotechnical engineering standards normally used on similar projects in this region.

8. LIMITATIONS

The materials observed on the project site appear to be representative of the area; however, soil materials vary in character between excavations or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusion and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

9. SELECTED REFERENCES

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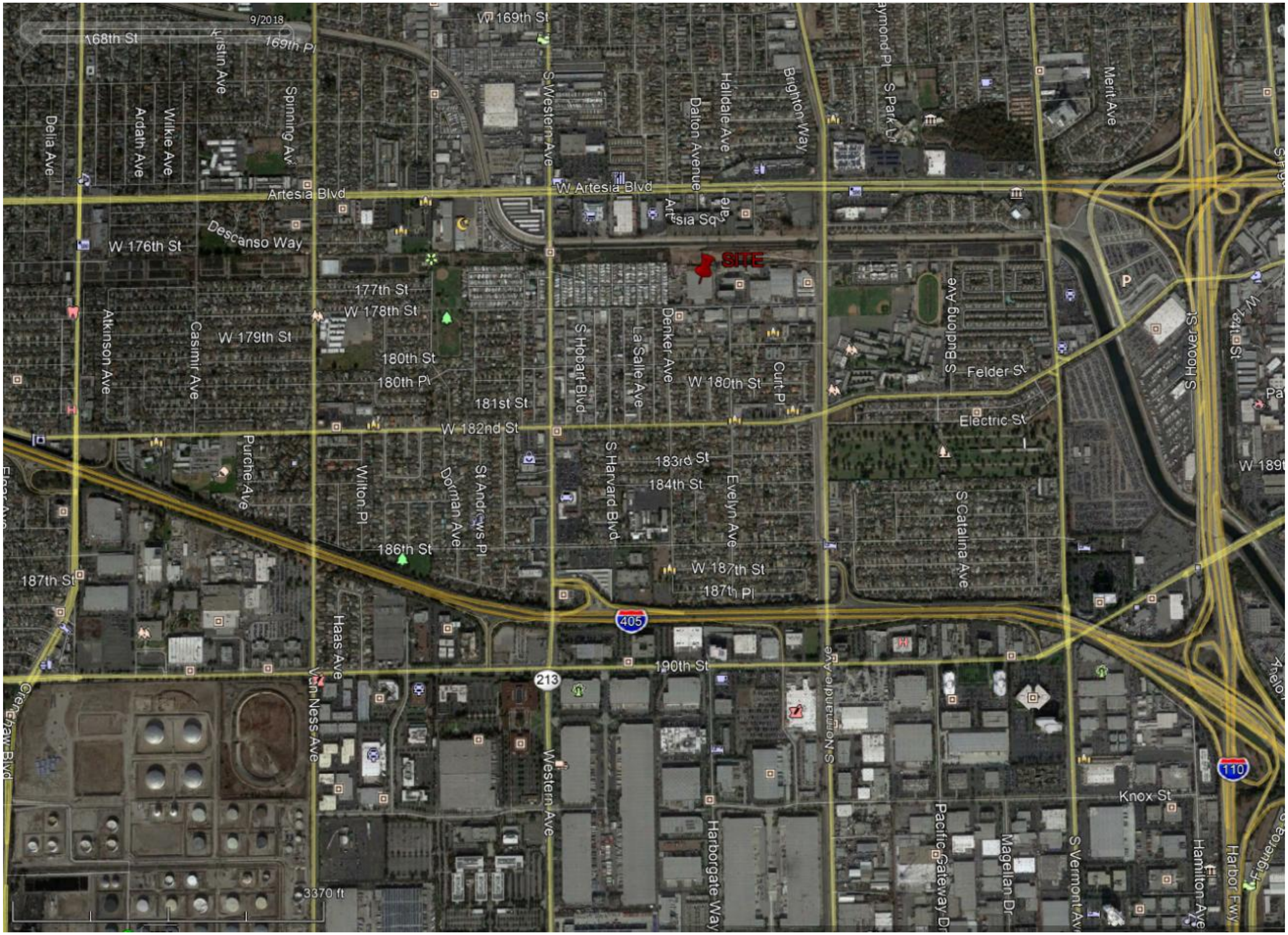

Melia Homes
1515 West 178th Street
Gardena, California

Project No. 1949-CR

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Figure 1

Site Location Map

GEOTEK

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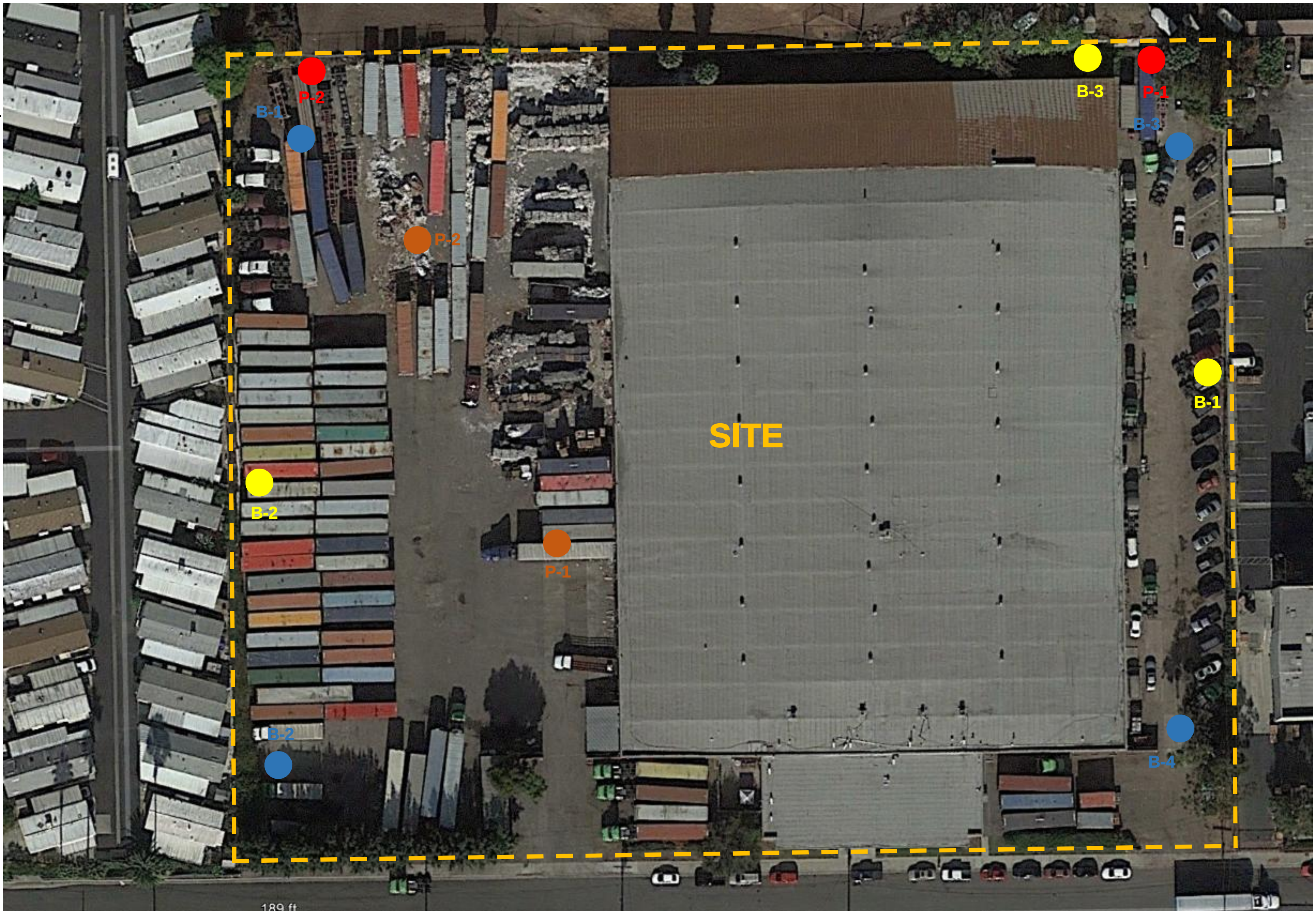

Melia Homes
1515 West 178th Street
Gardena, California

Project No. 1949-CR

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Figure 2

Exploration Location Map

GEOTEK

APPENDIX A

BORING LOGS, LABORATORY TEST RESULTS, AND INFILTRATION DATA BY PETRA (2004)

Updated Geotechnical and Infiltration Evaluation
1515 West 178th Street, Gardena, California
Project No. 1949-CR

GEOTEK

EXPLORATION LOG

Project: **Proposed 104 Townhomes**		Boring No.: **B-1**
Location: **5151 W. 178th Street, Gardena, CA**		Elevation:
Job No.: **496-04**	Client: **The Olson Company**	Date: **11/17/04**
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **EG**

Depth (Feet)	Lithology	Material Description	Water	Samples: Blows Per 6-inch	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt Pavement approximately 3 to 3.5" thick.							
		ARTIFICIAL FILL (Af) Clayey Sand (SC): Dark yellowish-brown; moist; medium dense; fine-grained sand.							R-V
		TERRACE DEPOSITS (Opu) Clayey Sand (SC): Dark yellowish-brown; moist; medium dense to dense; fine-grained sand; slightly porous upper 3 feet; abundance of clay content.		6 12 20			13.0	108.2	
5									
		Sandy Clay (CL): Yellowish-brown to dark yellowish-brown; moist; very stiff; fine-grained sand; slightly micaceous.		8 22 37			13.0	118.5	
10		Sandy Clay (CL): Dark olive brown; moist to very moist; firm to stiff; fine-grained sand; slightly micaceous.		4 7 7			17.8	111.0	CON
15		Same as 10 feet.		3 7 9			11.7	105.6	
		Silty Sand (SM): Yellowish-brown; moist; medium dense; fine-grained sand; slightly micaceous.							
20				9 14 20			18.4	108.0	
		Clayey Sand (SC): Dark olive gray; very moist; dense; fine-grained sand; slightly micaceous.							
25		Silty Clay (CL): Dark olive gray; very moist; stiff; fine-grained sand.		3 5 6					HYD AT
		Groundwater.	▼						

EXPLORATION LOG - V3 496-04.GPJ PETRA.GDT 12/1/04

PLATE A-1

EXPLORATION LOG

Project: Proposed 104 Townhomes		Boring No.: B-1
Location: 5151 W. 178th Street, Gardena, CA		Elevation:
Job No.: 496-04	Client: The Olson Company	Date: 11/17/04
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: EG

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6-inch	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		Sandy Clay to Silty Clay (CL): Dark olive gray; wet; very stiff; few to trace of fine-grained sand; slightly micaceous.		7 13 17			24.8	100.1	
35		Sandy Clay (CL): Olive brown; wet; stiff; fine-grained sand.		4 9 18					HYD AT
40		Sandy Clay (CL-CH): Olive brown; wet; very stiff; fine-grained sand.		14 29 44			26.3	98.5	
45		Same as 40 feet.		7 11 15					
50		Sandy Clay (CL): Brown; wet; hard; some fine-grained sand; some cemented clayey sand.		37 50-5"			23.3	104.4	
		Total Depth = 51.0 feet Groundwater at 29 feet Borehole backfilled with soil cuttings then patched with cold asphalt.							

EXPLORATION LOG - V3 496-04.GPJ PETRA.GDT 12/1/04

PLATE A-2

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: Proposed 104 Townhomes		Boring No.: B-2
Location: 5151 W. 178th Street, Gardena, CA		Elevation:
Job No.: 496-04	Client: The Olson Company	Date: 11/17/04
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: EG

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6-inch	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt pevement approximately 4" thick.							
		ARTIFICIAL FILL (Af) Clayey Sand (SC): Olive brown; moist; medium dense to dense; fine-grained sand; trace of gravel up to 3 feet; abundance of clay.		4 12 12			11.7	94.8	
		@ 3.5 feet: A piece of plastic.							
5		**TERRACE DEPOSITS (Qpu)** Sandy Clay (CL): Dark brown to dark olive brown; moist; stiff; fine-grained sand with some medium and coarse sand; trace of rootlets.		7 9 14			12.2	118.3	DSU
10		Sandy Clay (CL): Dark olive brown; very moist; stiff to very stiff; fine-grained sand; slightly micaceous.		11 14 18			12.7	119.1	
15		Clayey Sand (SC): Yellowish-brown; moist; very dense; fine-grained sand; micaceous.		17 23 38			15.4	111.9	
20		Clayey Sand (SC): Yellowish-brown; moist to very moist; very dense; fine-grained sand; some iron oxide staining; slightly micaceous; abundance of clay.		16 20 23			21.0	103.4	
		Total Depth = 21.5 feet No Groundwater Borehole backfilled with soil cuttings then patched with cold asphalt.							

EXPLORATION LOG - V3 496-04.GPJ PETRA.GDT 12/1/04

PLATE A-3

EXPLORATION LOG

Project: **Proposed 104 Townhomes**			Boring No.: **B-3**
Location: **5151 W. 178th Street, Gardena, CA**			Elevation:
Job No.: **496-04**	Client: **The Olson Company**		Date: **11/17/04**
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**		Logged By: **EG**

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6-inch	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt pavement approximately 4 to 4.5" thick.							
		ARTIFICIAL FILL (Af) Clayey Sand (SC): Brown; moist; medium dense to dense; fine-grained sand.							
		TERRACE DEPOSITS (Qpu) Clayey Sand (SC): Brown to light brown; moist; very dense; fine-grained sand with some medium sand; slightly micaceous.		7 40 50-5"			12.5	118.7	
5		Sandy Clay (CL): Yellowish-brown; moist; very stiff to hard; fine-grained sand; slightly micaceous.		17 37 50-4"			14.1	114.8	
10		Sandy Clay (CL): Olive yellow; moist to very moist; very stiff; fine-grained sand.		13 20 27			16.6	114.1	
15		Silty Sand with Clay (SM-SC): Yellowish-brown; moist; very dense; fine-grained sand; slightly micaceous.		15 28 33			14.2	115.6	
20		Same as 15 feet. Becomes very dense to hard.		18 30 50-5"			10.5	112.8	
		Total Depth = 21.5 feet No Groundwater Borehole backfilled with soil cuttings then patched with cold asphalt.							

PLATE A-4

Petra Geotechnical, Inc.

EXPLORATION LOG

Project: **Proposed 104 Townhomes**		Boring No.: **B-4**	
Location: **5151 W. 178th Street, Gardena, CA**		Elevation:	
Job No.: **496-04**	Client: **The Olson Company**	Date:	**11/17/04**
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **EG**	

Depth (Feet)	Material Description	Water	Blows Per 6-inch	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0	Asphalt pavement approximately 3 to 3.5" thick.					
	ARTIFICIAL FILL (Af) Clayey Sand (SC): Olive brown; moist; medium dense to dense; fine-grained sand.					MAX EXP AT SO4 pH RES CL
	TERRACE DEPOSITS (Qpu) Sandy Clay (CL): Olive brown to brown; moist; very stiff; fine-grained sand.		7 22 38	13.6	113.8	
5	Silty Sand with Clay (SM-SC): Light brown to brown; moist; very dense; fine-grained sand.		10 12 27	11.0	116.2	
10	Sandy Clay (CL): Dark brown; moist; very stiff; fine-grained sand.		12 23 36	14.8	116.5	
15	Sandy Clay (CL): Light brown; moist to very moist; stiff; few fine-grained sand; some carbonates.		10 12 15			HYD AT
20	Silty Sand with Clay (SM-SC): Brown; mois; very dense; fine-grained sand; slightly micaceous.		9 21 33			
25	Clayey Sand (SC): Brown; moist to very moist; very dense; fine-grained sand with some medium sand; slightly micaceous.		24 33 47	17.9	107.7	

PLATE A-5

EXPLORATION LOG

Project: **Proposed 104 Townhomes**		Boring No.: **B-4**	
Location: **5151 W. 178th Street, Gardena, CA**		Elevation:	
Job No.: **496-04**	Client: **The Olson Company**	Date: **11/17/04**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **EG**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per 6-inch	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Clayey Sand (SC): Olive brown; very moist to saturated; dense; fine-grained sand with some medium sand; some silt.		17 19 14					
		Groundwater.	▼						
35		Silty Sand with Clay (SM-SC): Olive brown; wet; very dense to hard; fine- to medium-grained sand with some coarse sand; trace of gravel.		19 32 50-4"			18.7	112.2	
40		Silty Sand (SM): Brown; wet; very dense to hard; fine-grained sand; some clay.		16 26 41					
45		Sandy Clay (CL): Olive brown; wet; dense; fine-grained sand.		14 17 15					HYD AT
50		Silty Clay (CL): Brown; wet; very stiff; fine-grained sand.		11 14 20					

Total Depth = 51.5 feet
Groundwater at 32 feet
Borehole backfilled with soil cuttings then patched with cold asphalt.

PLATE A-6

LABORATORY MAXIMUM DRY DENSITY[1]

Soil Type	Optimum Moisture (%)	Maximum Dry Density (pcf)
A - Clayey Sand (SC)	10.0	125

5EXPANSION INDEX TEST DATA[2]

Soil Type	Expansion Index	Expansion Potential [3]
A - Clayey Sand (SC)	51	Medium

SOLUBLE SULFATES AND CHLORIDES[4]

Soil Type	Sulfate Content (%)	Chloride Content (ppm)
A - Clayey Sand (SC)	0.0041	158

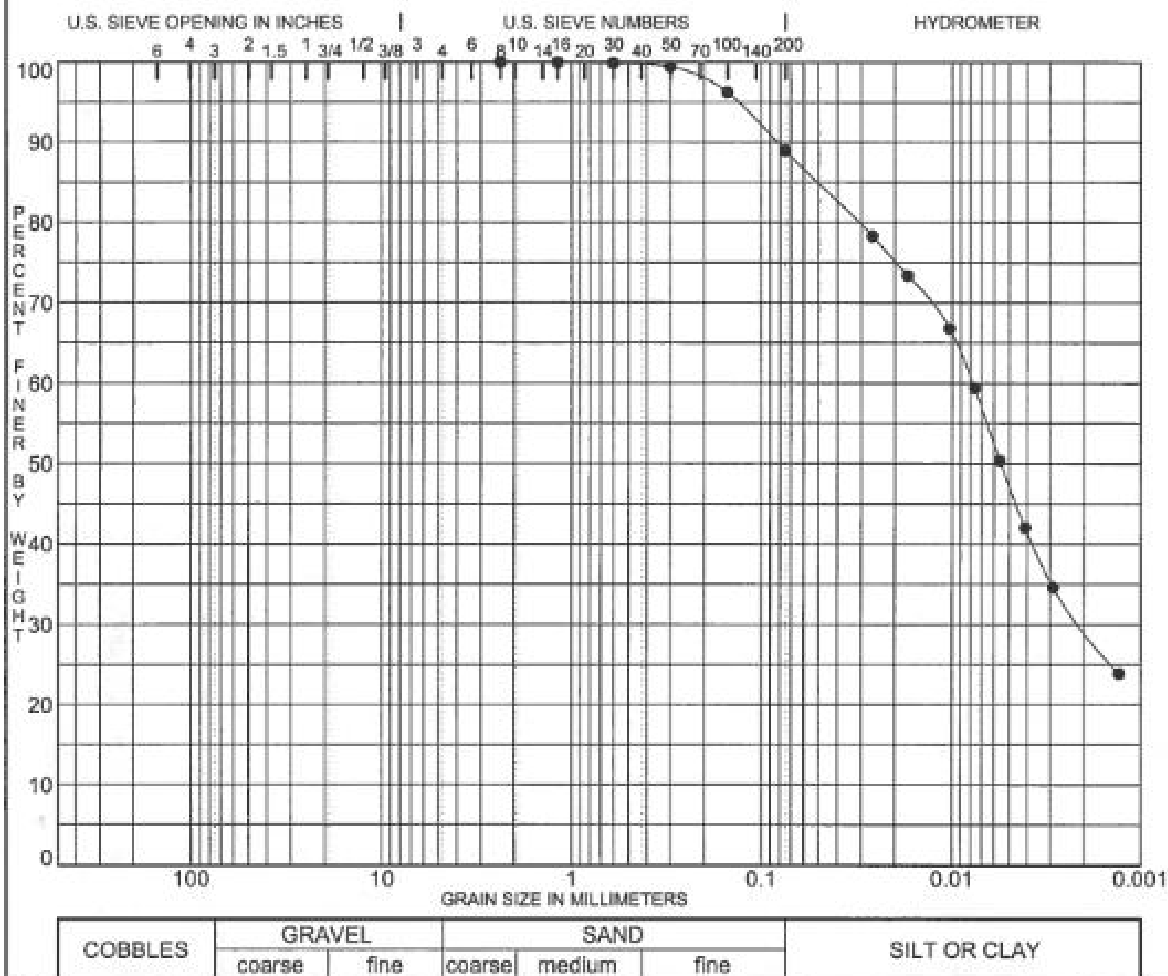
pH AND MINIMUM RESISTIVITY[5]

Soil Type	pH	Minimum Resistivity (Ohm-cn)
A - Clayey Sand (SC)	7.7	2,500

ATTERBERG LIMITS[6]

Boring Location	Liquid Limit	Plastic Limit	Plasticity Index
B-4 @ 1.5 feet	46	24	22

(1) Per ASTM Test Method D 1557-00
(2) Per Uniform Building Code Standard 18-2
(3) Per UBC Table 18-I-B, "Classification of Expansive Soils"
(4) Per California Test Method Nos. 417 and 422
(5) Per California Test Method Nos. 532 and 643
(6) Per ASTM Test Method D 4318-00

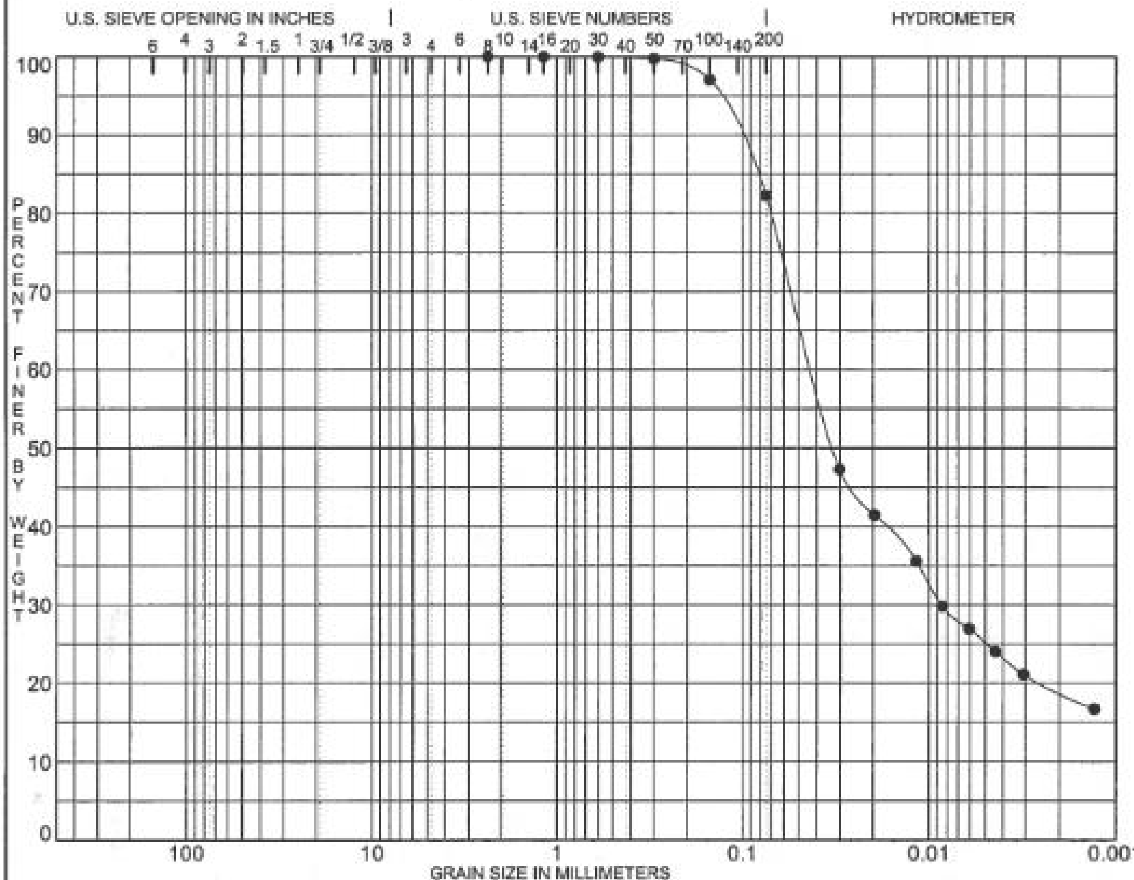


Specimen Identification			Classification	MC%	LL	PL	PI	Cc	Cu
●	B-1	25.0	Silty Clay (CL)		48	26	22		

Specimen Identification			D100	D60	D30	D50	%Gravel	%Sand	%Silt	%Clay
●	B-1	25.0	2.36	0.01	0.002	0.0055	0.0	11.0	41.7	47.3

GRAIN SIZE - V1 496-04.GPJ PETRA.GDT 12/1/04

J.N. 496-04	GRAIN SIZE ANALYSIS	December, 2004
PETRA GEOTECHNICAL, INC.		PLATE B-2



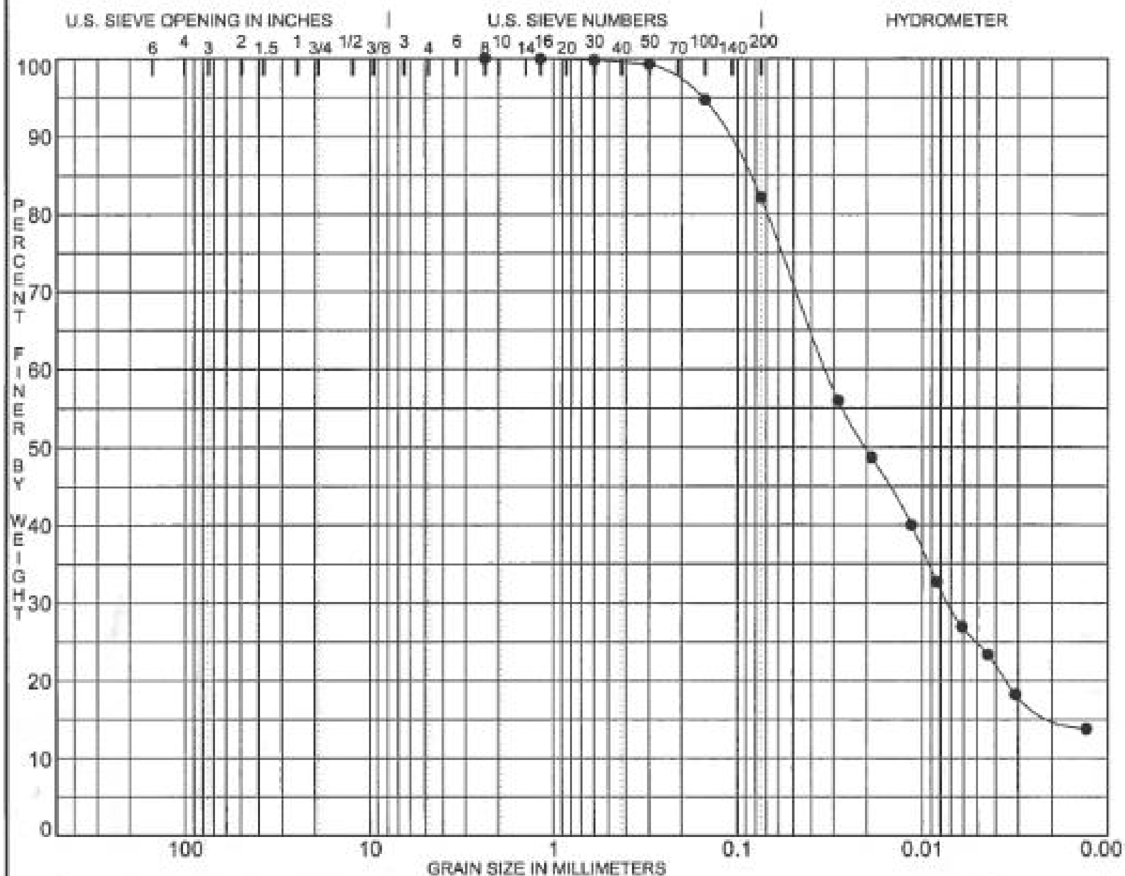
COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-1	35.0	Sandy Clay (CL)		42	22	20		

	Specimen Identification		D100	D60	D30	D50	%Gravel	%Sand	%Silt	%Clay
●	B-1	35.0	2.36	0.04	0.009	0.0324	0.0	17.7	57.2	25.1

GRAIN SIZE - V1 496-04.GPJ PETRA.GDT 12/1/04

J.N. 496-04	GRAIN SIZE ANALYSIS	December, 2004
PETRA GEOTECHNICAL, INC.		PLATE B-3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-4	15.0	Sandy Clay (CL)		41	22	19		

	Specimen Identification		D100	D60	D30	D50	%Gravel	%Sand	%Silt	%Clay
●	B-4	15.0	2.36	0.03	0.007	0.0203	0.0	17.8	57.5	24.7

GRAIN SIZE - V1 496-04.GPJ PETRA.GDT 12/1/04

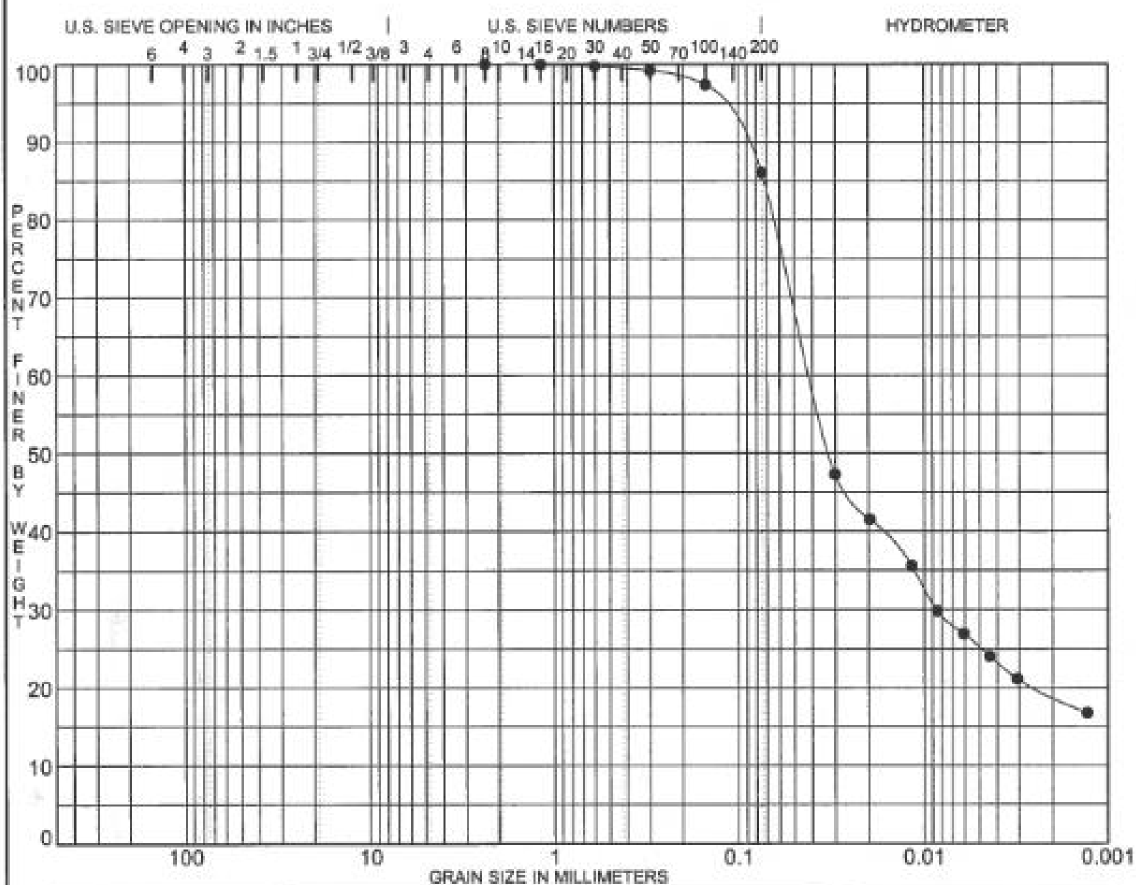
J.N. 496-04

PETRA GEOTECHNICAL, INC.

GRAIN SIZE ANALYSIS

December, 2004

PLATE B-4



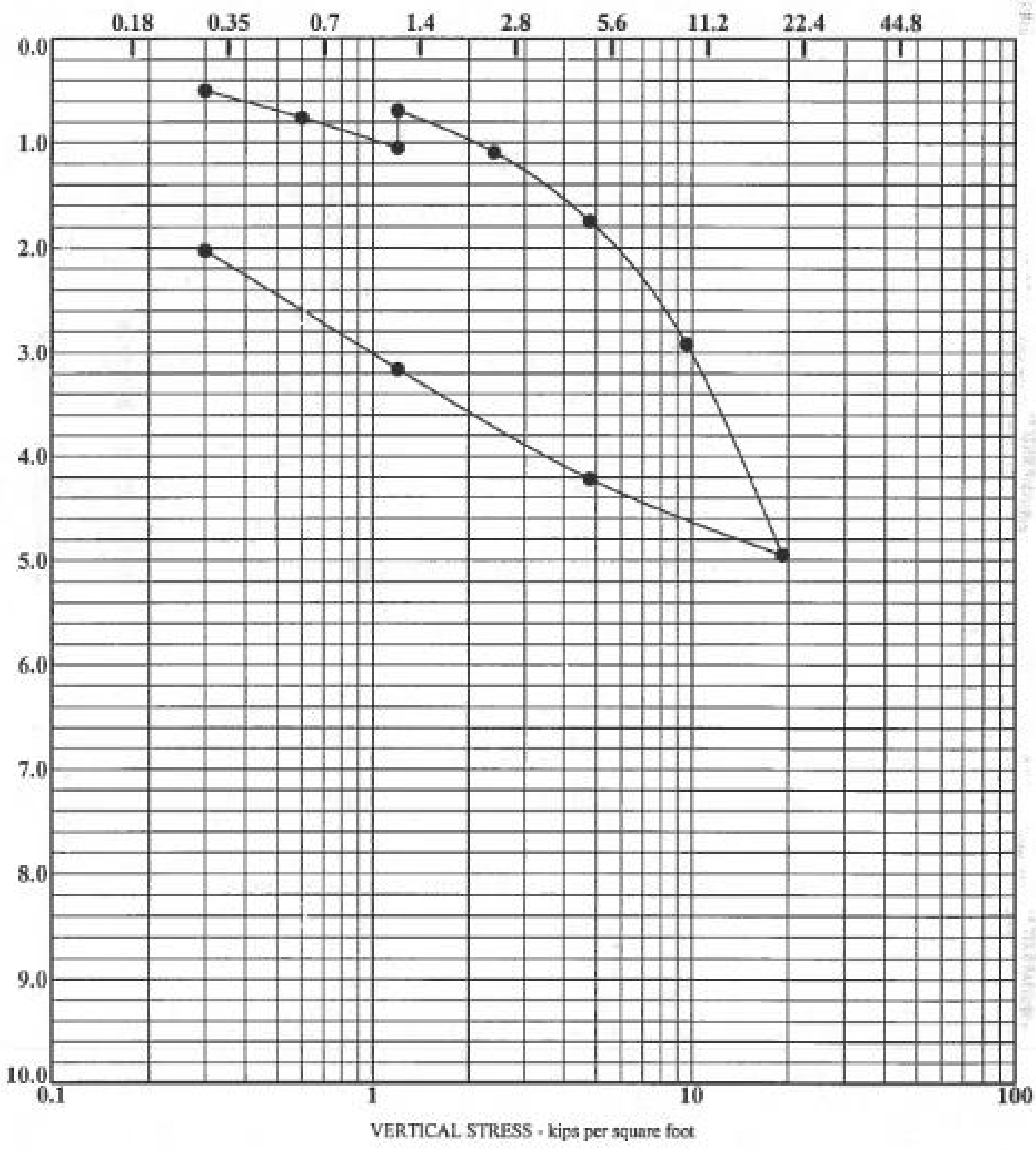
COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	MC%	LL	PL	PI	Cc	Cu
●	B-4	45.0	Sandy Clay (CL)		43	23	20		

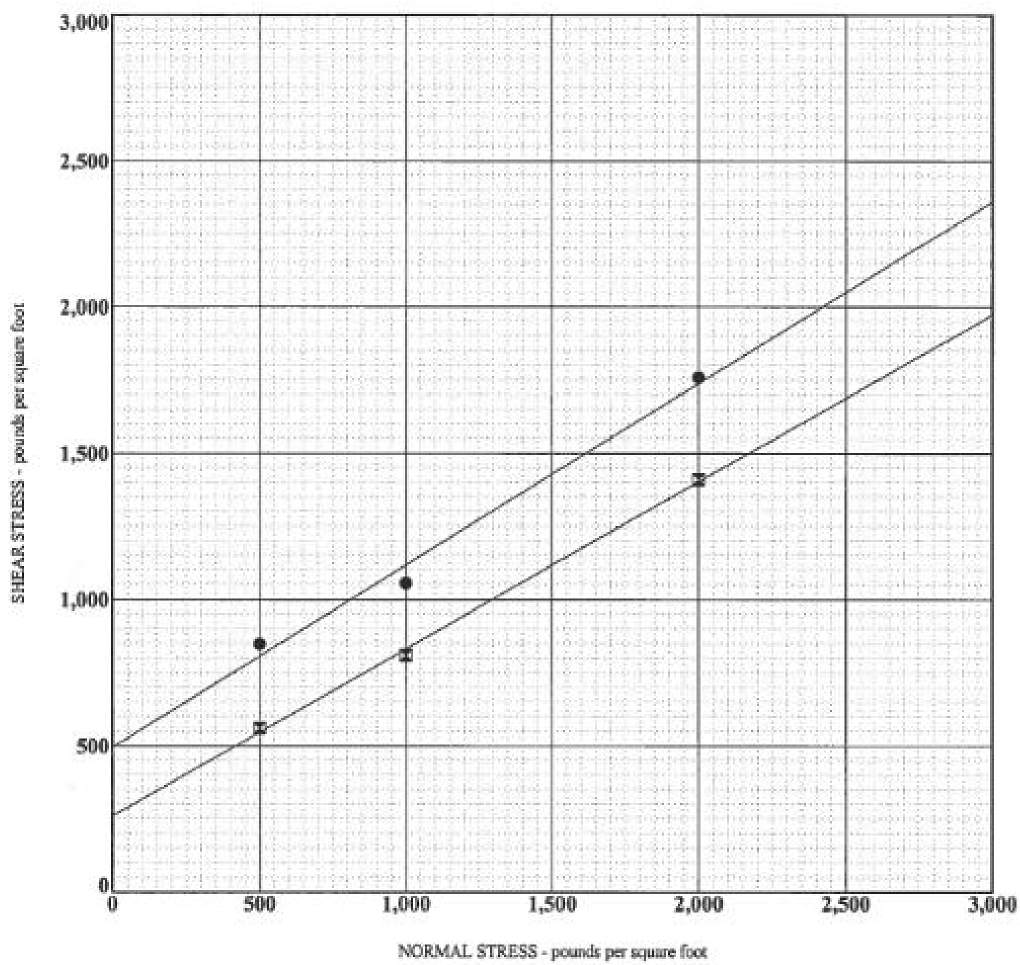
Specimen Identification			D100	D60	D30	D50	%Gravel	%Sand	%Silt	%Clay
●	B-4	45.0	2.36	0.04	0.009	0.0322	0.0	14.0	60.9	25.1

J.N. 496-04	GRAIN SIZE ANALYSIS	December, 2004
PETRA GEOTECHNICAL, INC.		PLATE B-5

SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-1 @ 10.0	Sandy Clay (CL)	111.0	17.8	93	1.20



J.N. 496-04	CONSOLIDATION TEST RESULTS	November, 2004
PETRA GEOTECHNICAL, INC.		PLATE B-6



SAMPLE LOCATION	DESCRIPTION	FRICTION ANGLE (°)	COHESION (PSF)
● B-2 @ 5.0	Clayey Sand (SC) - Peak	32	500
⧗ B-2 @ 5.0	Clayey Sand (SC) - Ultimate	30	260

NOTES:

Undisturbed Test Samples
All Samples Were Inundated Prior to Shearing

J.N. 496-04	DIRECT SHEAR TEST DATA	December, 2004
PETRA GEOTECHNICAL, INC.	UNDISTURBED TEST SAMPLES	PLATE B-7

DIRECT SHEAR 496-04.GPJ PETRA.GDT 12/1/04

PROJECT NO. :	94170-00/PETRA GEO.#496-04	DATE :	22-Nov-04
PROJECT NAME :	OLSON	TESTED BY:	RB
SAMPLE NO. / LOCATION :	B-1 @ 1' - 5'	SAMPLED BY:	
SAMPLE DESCRIPTIONS / CLASSIFICATION :	DK. BROWN SANDY CLAY (CL/CH)		

TRIAL NO.		1		2		3		4	
MOLD NUMBER		11		7		AC1			
WATER ADDED	(ML)	95		100		105			
COMPACTOR PRESSURE	(PSI)	60		45		40			
GROSS WEIGHT	(GMS)	3223		3200		3201			
TARE WEIGHT	(GMS)	2133		2121		2120			
SAMPLE WET WEIGHT	(GMS)	1090		1079		1081			
EXUDATION PRESSURE	(PSI)	343		302		227			
SAMPLE HEIGHT	(IN.)	2.51		2.49		2.5			
EXPANSION	(IN.x10^{-4})	11		3		0			
STABILITY @ 160 PSI (2000 LBS) / @ 80 PSI (1000 LBS)		145	71	147	74	148	74		
TURNS DISPLACEMENT		3.53		3.58		3.57			
R-VALUE UNCORRECTED		7		6		5			
R-VALUE CORRECTED		7		6		5			
MOISTURE CONTENT	(%)	18.9		19.2		19.5			
DRY DENSITY	(PCF)	110.7		110.2		109.6			
ASSUMED TRAFFIC INDEX		4.0		4.0		4.0			
G.E. BY STABILITY		0.95		0.96		0.97			
G.E. BY EXPANSION		0.37		0.10		0.00			
R-VALUE @ EQUILIBRIUM	(BY EXUDATION)	6							
Gf		1.25							

REMARKS :

CHECKED BY: GARY GOLDMAN

DATE : 11/23/04

REGISTERED PROFESSIONAL ENGINEER
GARY B. GOLDMAN
No. 2587
Exp. 12-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

THE DATA ABOVE IS BASED UPON PROCESSING AND TESTING OF SAMPLES "AS RECEIVED" FROM THE FIELD
TEST PROCEEDURES IN GENERAL CONFORMANCE TO LATEST REVISIONS OF CA TEST METHOD 301.

ZEISER KLING CONSULTANTS, INC.
1221 E. Dyer Road, Suite 105; Santa Ana, CA 92705
Tel: (714) 755-1355; Fax: (714) 755-1366

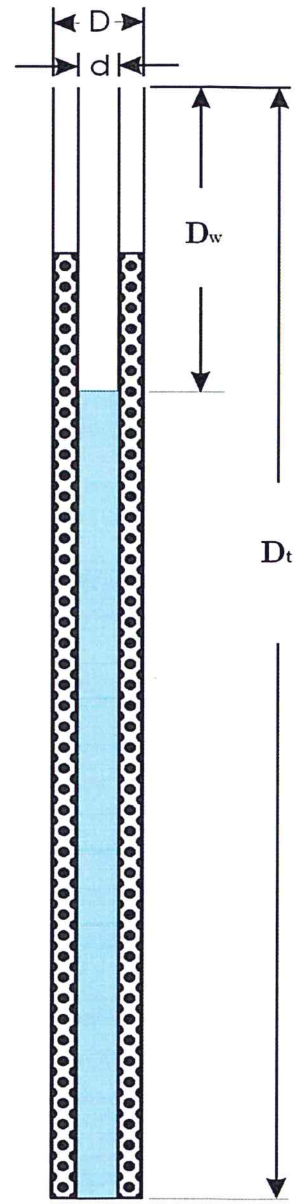
R - VALUE DATA

PLATE B-8

Test Number: P-1

Total Depth of Boring, D_t (ft): 5.2
Diameter of Hole, D (in): 7
Diameter of Pipe, d (in): 3
Agg. Correction (% Voids): 42

Time Interval (min)	Depth to Water Surface D_w (ft)		Change in Head (in)	Perc. Rate (min/in)	Perc. Rate (gal/day/ft^2)
	1st Reading	2nd Reading			
10	1.25	4.17	47.00	0.21	122.93
10	0.92	3.83	35.00	0.29	81.22
10	0.92	3.75	34.00	0.29	77.85
10	0.88	3.63	33.00	0.30	73.61
10	0.83	3.50	32.00	0.31	69.36
10	0.75	3.46	32.50	0.31	69.14



Percolation Rate: 0.31 Minutes/Inch
69.14 gal/day/ft^2

Infiltration Rate: 8.77 Inches/Hour
(Porchet Method)

Reference: RCFCWCD, Design Handbook for LIDBMP, dated September, 2011 **or**
SARWQCB, Technical Guidance Document Appendix VII, dated March 22, 2011

PETRA GEOSCIENCES, INC.
3190 Airport Loop Drive, Suite J-1
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

PERCOLATION TEST SUMMARY

1515 West 178th Street
Gardena, California

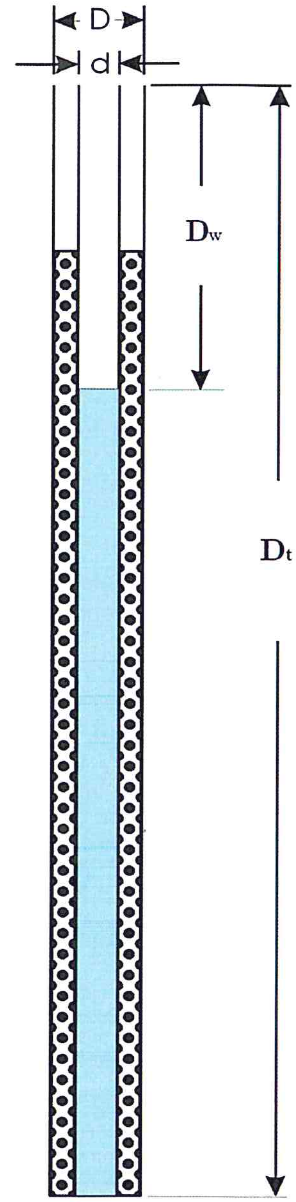
PETRA GEOSCIENCES INC.

DATE: April, 2016
J.N.: 16-149

Figure 1

Test Number: P-2

Total Depth of Boring, D_t (ft): 5.2
Diameter of Hole, D (in): 7
Diameter of Pipe, d (in): 3
Agg. Correction (% Voids): 42



Time Interval (min)	Depth to Water Surface D_w (ft)		Change in Head (in)	Perc. Rate (min/in)	Perc. Rate (gal/day/ft^2)
	1st Reading	2nd Reading			
10	1.00	2.67	20.00	0.50	39.27
10	1.00	2.58	19.00	0.53	36.84
11	0.92	2.50	19.00	0.58	32.75
10	1.00	2.50	18.00	0.56	34.51
10	0.96	2.46	17.50	0.57	33.18
10	0.88	2.33	17.50	0.57	32.25

Percolation Rate: 0.57 Minutes/Inch
32.25 gal/day/ft^2

Infiltration Rate: 4.09 Inches/Hour
(Porchet Method)

Reference: RCFCWCD, Design Handbook for LIDBMP, dated September, 2011 or SARWQCB, Technical Guidance Document Appendix VII, dated March 22, 2011

PETRA GEOSCIENCES, INC.
3190 Airport Loop Drive, Suite J-1
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

PERCOLATION TEST SUMMARY

1515 West 178th Street
Gardena, California

PETRA GEOSCIENCES INC.

DATE: April, 2016
J.N.: 16-149

Figure 2

APPENDIX B

BORING LOGS BY GEOTEK

Updated Geotechnical and Infiltration Evaluation
1515 West 178th Street, Gardena, California
Project No. 1949-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)
The ring sampler is driven into the ground in accordance with ASTM Test Method D 3550. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the logs of borings. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

Bulk Samples (Large)
These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)
These are plastic bag samples which are normally airtight and contain less than five pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – BORING LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip
C: Contact line	
………..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
━━━	Thick solid line denotes end of boring

(Additional denotations and symbols are provided on the logs of borings)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	1515 W 178th Street	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Adrian
PROJECT NO.:	1949-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	SIMCO
LOCATION:	See Boring Location Map			**DATE:**	8/20/2018

BORING NO.: B-1

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					3" Asphaltic Concrete			
					Older Alluvium (Qal)			
	Large Bulk			CL	Sandy CLAY, red-brown, slightly moist			EI, AL, SA
5	SPT	35 28 32		ML	Clayey SILT with some f sand, brown, slightly moist, stiff	11.5	126.0	
	SPT	11 18 23			Becomes red-brown, moist, stiff	14.1	117.1	
10	SPT	13 28 38		CL	Silty and sandy CLAY, olive brown, moist, very stiff	15.0	122.1	
15 #	SPT	13 26 20		SM	Silty f SAND with a trace clay, brown, moist, dense	12.3	119.4	
20	SPT	11 21 27		SM	Silty f-m SAND, brown, moist, dense			
25	SPT	10 15 20		SP	F-m SAND, brown, moist, dense	14.1	113.7	
					Boring Terminated at 26.5 feet Groundwater not encountered Boring backfilled with cuttings and capped with cold-patch asphalt			
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	1515 W 178th Street	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Adrian
PROJECT NO.:	1949-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	SIMCO
LOCATION:	See Boring Location Map			**DATE:**	8/20/2018

BORING NO.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					3" Asphaltic Concrete over 1" Agreggate Base			
					Older Alluvium (Qal)			
	Large Bulk			CL	Sandy CLAY, olive brown, moist, stiff			MD, EI, SA, AL, DS, SR
5	Ring	7 11 13			Same as above	13.1	120.6	
	Ring	9 18 31		CL	Silty CLAY with some f sand and caliche, light red-brown, moist, very stiff	12.5	126.1	
10	Ring	15 28 30			Same as above with a trace of caliche	12.6	124.8	
	Ring	9 18 22		ML	F sandy SILT, olive moderate brown, moist, very stiff	14.3	118.9	
15 #	Ring	8 17 23		SM	Silty f SAND, light reddish brown, moist, dense			
20	Ring	10 23 27			Becomes moderate olive browin, moist, dense	12.4	118.3	
25	Ring	8 15 22		SM	Silty f SAND with few clay, moderate olive, moist, dense			
					Boring Terminated at 26.5 feet Groundwater not encountered Boring backfilled with cuttings and capped with cold-patch asphalt			
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	1515 W 178th Street	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Adrian
PROJECT NO.:	1949-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	SIMCO
LOCATION:	See Boring Location Map			**DATE:**	8/20/2018

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Artificial Fill			
				SM	Silty f SAND with gravel, moderate brown, slightly moist			
					Older Alluvium (Qal)			
5	Ring	28 50/6		ML	F sandy SILT, light reddish brown, slightly moist, hard			
10	Ring	18 26 32		SM/ML	Silty f SAND to f sandy SILT, light brown, slightly moist, dense to very stiff, trace rootlets	9.8	119.0	HC
15 #	Ring	13 25 36		SM/ML	Silty f SAND to f sandy SILT, light brown, slightly moist, dense to very stiff, trace rootlets	11.8	125.9	HC
					Boring Terminated at 16.5 feet Groundwater not encountered Boring backfilled with cuttings			
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	1515 W 178th Street	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Adrian
PROJECT NO.:	1949-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	SIMCO
LOCATION:	See Boring Location Map			**DATE:**	8/20/2018

BORING NO.: P-1

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
				SM	**Artificial Fill** Silty f SAND with gravel, moderate brown, slightly moist			
5				ML	**Older Alluvium (Qal)** F sandy SILT, light reddish brown, slightly moist, hard			
10					**Boring Terminated at 6.0 feet**			
15 #								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Melia Homes	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	1515 W 178th Street	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Adrian
PROJECT NO.:	1949-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	SIMCO
LOCATION:	See Boring Location Map			**DATE:**	8/20/2018

BORING NO.: P-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					3" Asphaltic Concrete over 4" Agreggate Base			
				ML/CL	**Older Alluvium (Qal)** Clayey SILT with some f sand, moderate brown, very moist			
5	Large Bulk				F sandy SILT, light reddish brown, very moist			
					Boring Terminated at 6.0 feet			
10								
15	#							
20								
25								
30								

LEGEND

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

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

APPENDIX C

LABORATORY TEST RESULTS BY GEOTEK

Updated Geotechnical and Infiltration Evaluation
1515 West 178th Street, Gardena, California
Project No. 1949-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance to the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the logs of exploratory borings in Appendix B.

In Situ Moisture Content and Unit Weight
The field moisture content was measured in the laboratory on selected samples collected during the field investigation. The field moisture content is determined as a percentage of the dry unit weight. The dry density was measured in the laboratory on selected ring samples. The results are shown on the logs of exploratory borings in Appendix B.

Moisture-Density Relationship
Laboratory testing was performed on a sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with test method ASTM Test Procedure D 1557. The results are presented herein.

Direct Shear
Direct shear testing was performed on remolded samples of the surficial soils according to ASTM Test Method D 3080. The results of these tests are presented herein.

Consolidation/Collapse
Consolidation/collapse tests were conducted in accordance with ASTM D2435. The results of these tests are presented herein.

Expansion Index
The expansion potential of the soils was determined by performing expansion index tests on two representative soil samples from the site in general accordance with ASTM D 4829. The results of these tests are presented herein.

Atterberg Limits
Atterberg limits testing were performed on two clayey samples collected from the site. The tests were performed in general accordance with ASTM D 4318. The test results are presented herein.

Sieve/Hydrometer
Sieve/hydrometer testing was performed on two clayey samples collected from the site. The tests were performed in general accordance with ASTM D 6913 and D 7928. The test results are presented herein.

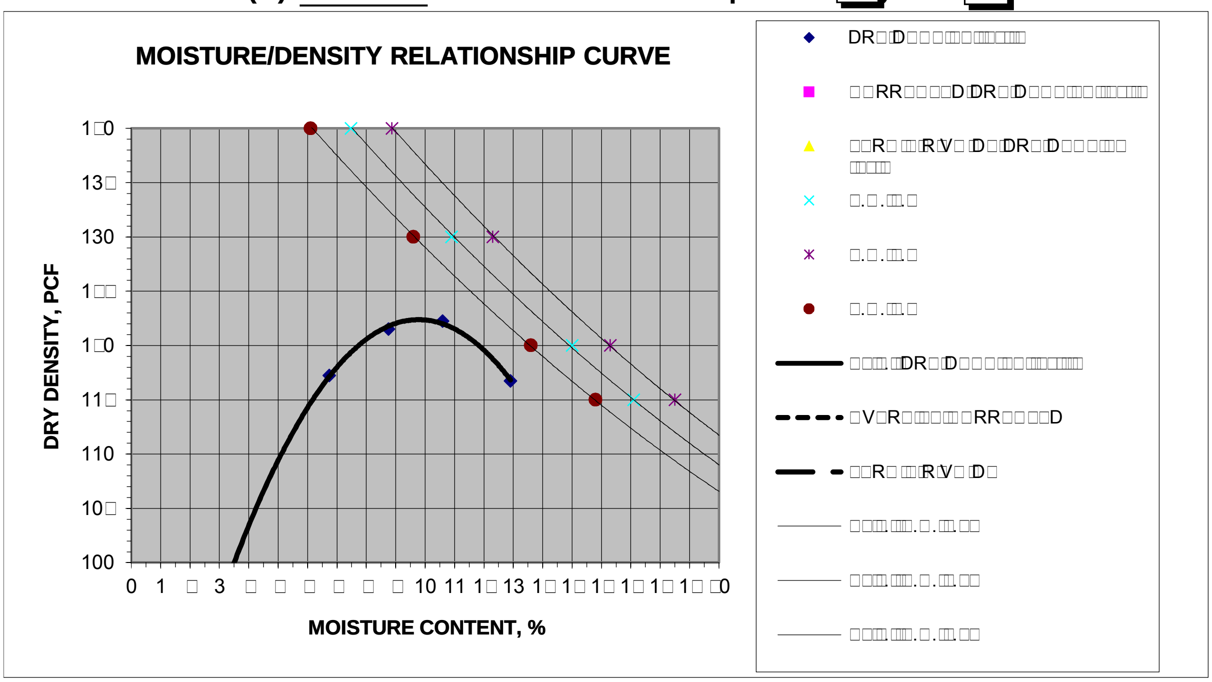
Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content was performed by others in general accordance with California Test No. 417. Resistivity testing was completed by others in general accordance with California Test No. 643. Testing to determine the chloride content was performed by others in general accordance with California Test No. 422. The results are included herein.

GEOTEK

MOISTURE/DENSITY RELATIONSHIP

Client:	M[illegible]	**Job No.:**	1[illegible]R
Project:	1[illegible]	**Lab No.:**	[illegible]
Location:	[illegible]		
Material Type:	D[illegible]d		
Material Supplier:			
Material Source:			
Sample Location:	[illegible]		
Sampled By:	D[illegible]	**Date Sampled:**	[illegible]
Received By:	D[illegible]	**Date Received:**	[illegible]
Tested By:	[illegible]	**Date Tested:**	[illegible]
Reviewed By:		**Date Reviewed:**	

Test Procedure:	[illegible]M1[illegible]	**Method:**	[illegible]
Oversized Material (%):	[illegible].0	**Correction Required:**	☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	1[illegible]	**@ Optimum Moisture, %**	10.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	[illegible]
	[illegible]
	[illegible]

Atterberg Limits:

	[illegible]
	[illegible]
	[illegible]

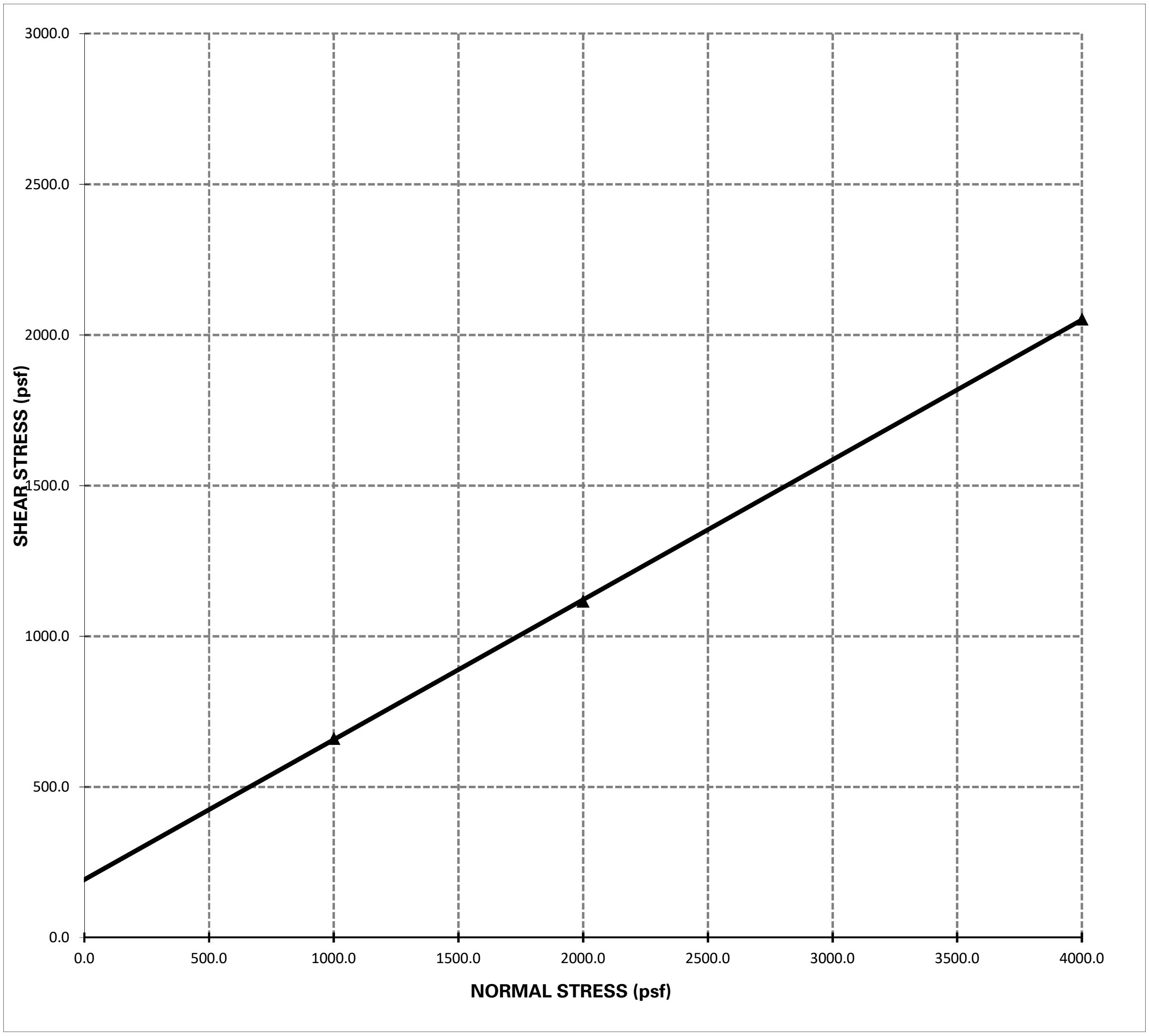
Classification:

[illegible]

[illegible]

DIRECT SHEAR TEST

Project Name:	1515 W. 178TH St., Gardena	***Sample Location:***	B-2 @ 1 - 5 ft
Project Number:	1949-CR	***Date Tested:***	9/4/2018

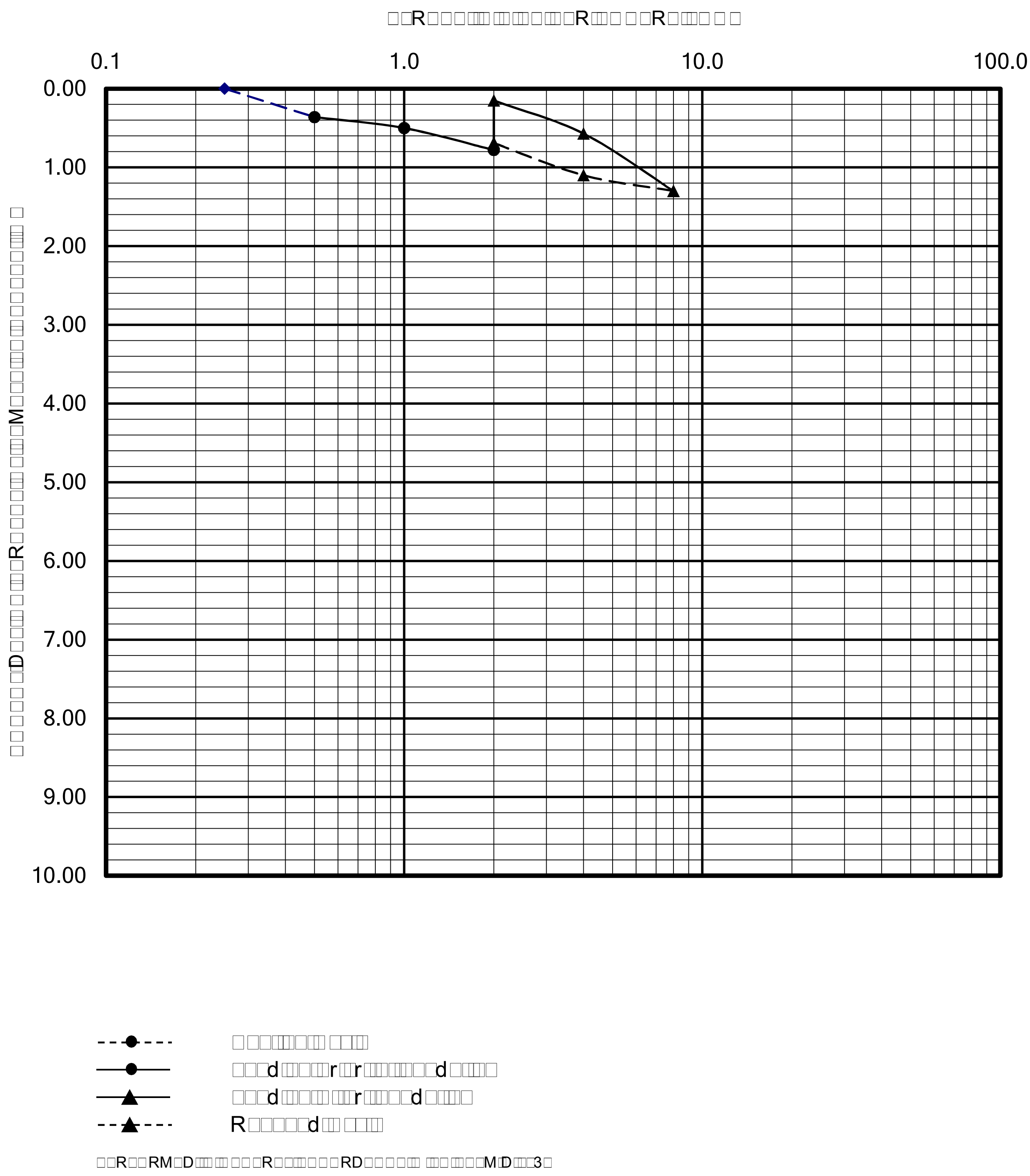


Shear Strength: Φ = **24.9** $^{\circ}$, **C = 192.00 psf**

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

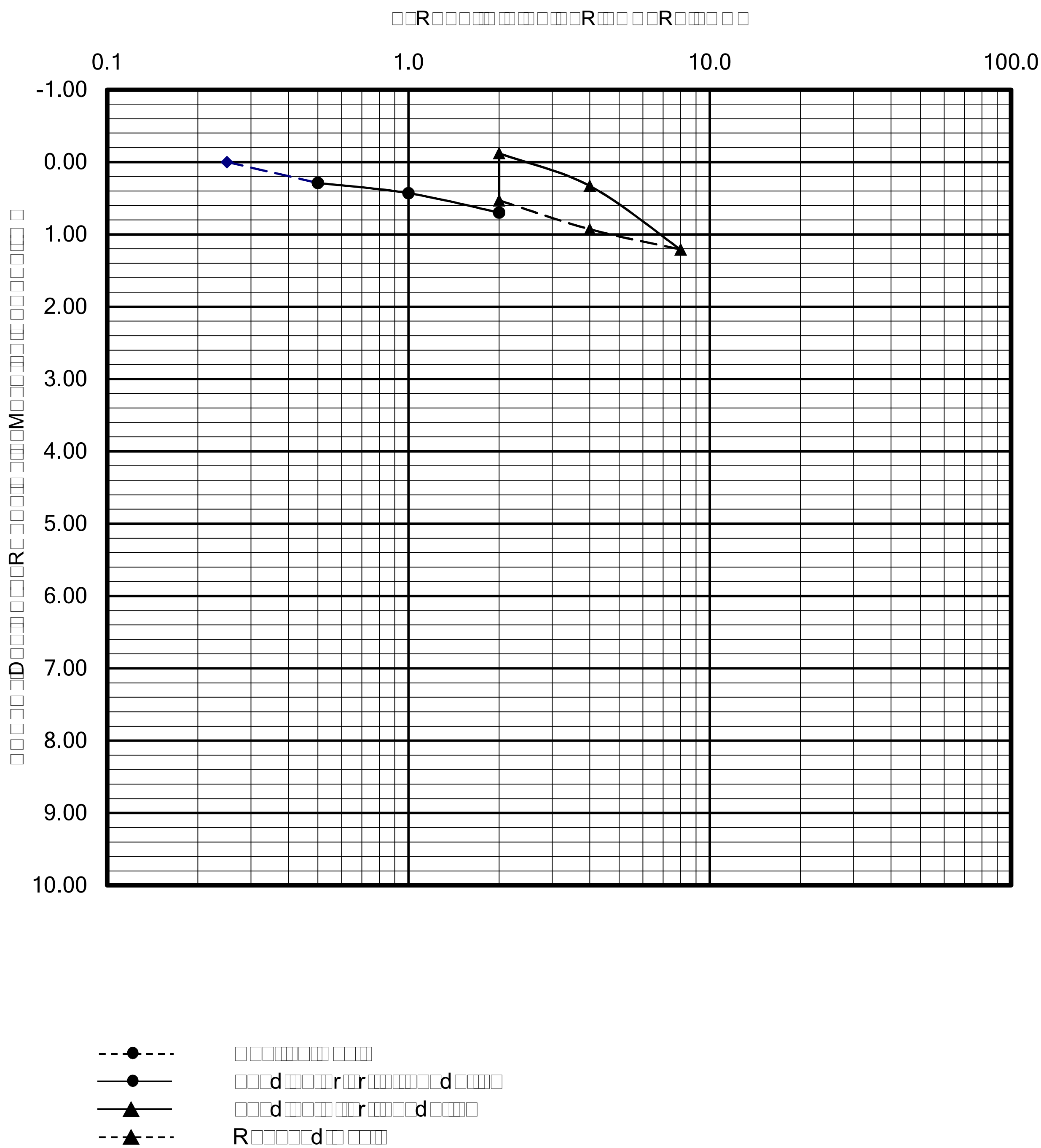


GEOTEK

CONSOLIDATION REPORT

Sample:

B-3 @ 10 ft



GEOTEK

CONSOLIDATION REPORT

Sample:

B-3 @ 15 ft

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	[illegible]
Project Number:	[illegible]
Project Location:	[illegible]
Tested/ Checked By:	[illegible]
Date Tested:	[illegible]
Sample Source:	[illegible]
Sample Description:	

[illegible]

DENSITY DETERMINATION

A	[illegible]	[illegible]
B	[illegible]	[illegible]
C	[illegible]	**396.2**
D	[illegible]	**119.5**
E	[illegible]	**108.6**

SATURATION DETERMINATION

F	[illegible]	10.0
G	[illegible]	**2.70**
H	[illegible] °C, (pcf)	**62.4**
I	[illegible]	**49.0**

READINGS

[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

FINAL MOISTURE

[illegible]	[illegible]
[illegible]	**19.8**

EXPANSION INDEX = 33

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	M[illegible]	***Tested/ Checked By:***	D[illegible]D[illegible] [illegible] [illegible]r[illegible]
Project Number:	1[illegible]R	***Date Tested:***	[illegible]01[illegible]
Project Location:	1[illegible]1[illegible].[illegible]1[illegible]rd[illegible]	***Sample Source:***	[illegible]1[illegible]
		Sample Description:	

R[illegible] ______ R[illegible]D[illegible] [illegible].01" R[illegible].1"

DENSITY DETERMINATION

A	[illegible]d[illegible]r[illegible]	[illegible].[illegible]
B	[illegible]r[illegible]	3[illegible].[illegible]
C	[illegible]	**386.7**
D	[illegible]D[illegible]3[illegible]0.301[illegible]	**116.6**
E	Dr[illegible]D[illegible]3[illegible]D[illegible]1.[illegible]	**106.0**

SATURATION DETERMINATION

F	M[illegible]r[illegible]	10.0
G	[illegible]r[illegible]v[illegible]d	**2.56**
H	[illegible]r[illegible]0 °C, (pcf)	**62.4**
I	[illegible]r[illegible]	**50.5**

READINGS

D[illegible]	[illegible]M[illegible]	R[illegible]D[illegible]	
[illegible]01[illegible]	1[illegible]10	0.11[illegible]0	[illegible]
	1[illegible]0	0.11[illegible]0	10[illegible]Dr[illegible]
[illegible]01[illegible]	1[illegible]30	0.1[illegible]0	[illegible]

FINAL MOISTURE

[illegible] [illegible]r[illegible]	[illegible]M[illegible]r[illegible]
[illegible].[illegible]	**20.4**

EXPANSION INDEX = 38

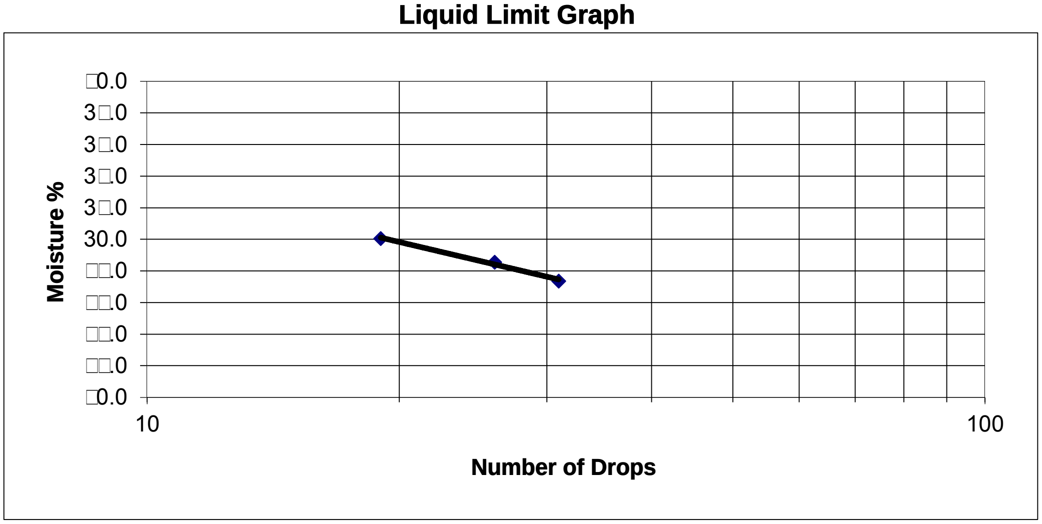
GEOTEK

ATTERBERG LIMITS DATA

□□d□□□□□□□□□□□		□□□□□□.	1□□□□□R
□□□ □□□□□□□□r		□□□□□	M□□□□□□□□□
□□□ □□□□□□□	□□□	□r□□□□	1□1□□□.□1□□□□□□.□□□rd□□□
□□□□□□	□□1□□ □1□□□□□		
□□□□d□□□	D□		

	Plastic Limit			Liquid Limit		
Number of Blows				31	□□	1□
Determination	1	□	3	□	□	□
Dish						
Wt. of Dish + Wet Soil	13.3□	13.□□		□0.□3	□0.□□	□0.30
Wt. of Dish + Dry Soil	1□.3□	1□.□□		1□.1□	1□.1□	1□.00
Wt. of Moisture	1.00	1.0□		3.0□	3.1□	3.30
Wt. of Dish	□.0□	□.10		□.0□	□.0□	□.0□
Wt. of Dry Soil	□.□□	□.□□		11.11	11.03	10.□□
Moisture Content %	1□.0	1□.3		□□.□	□□.□	30.1

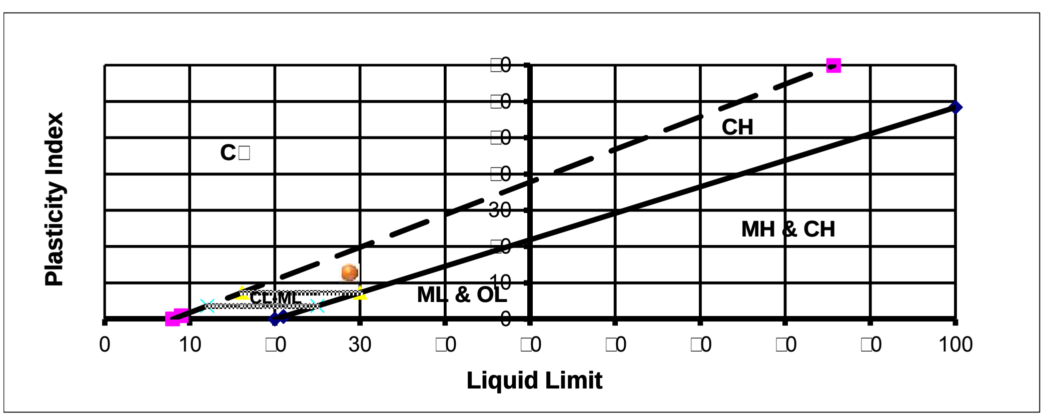
Liquid Limit Graph



□□□□d□□□ □
□□

□□□□□□□□ □
1□

□□□□□□□□□□d□□
13



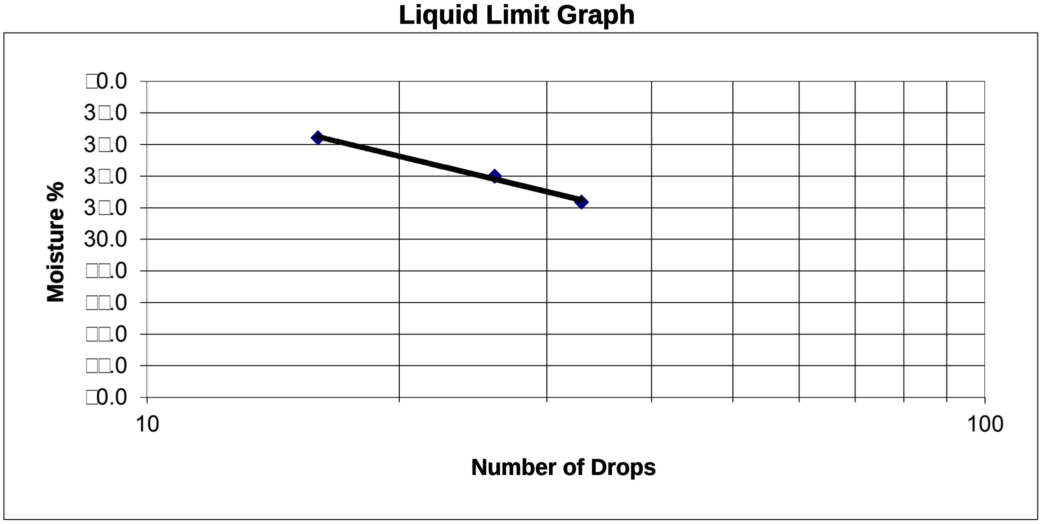
GEOTEK

ATTERBERG LIMITS DATA

□□d□ □□□□□□□□		□□□□□.	1□□□□R
□□□ □□□□□□ □□r		□□□□□	M□□□□□ □□ □□
□□□ □□□□□□	□□□	□r□□□□	1□1 □□□ .□1□□□□ □□.□□□ □rd□□□
□□□□□□□	□□□□ □1□□□□□		
□□□□d□□□	D□		

	Plastic Limit			Liquid Limit		
Number of Blows				33	□□	1□
Determination	1	□	3	□	□	□
Dish						
Wt. of Dish + Wet Soil	13.□□	13.3□		□0.□□	□0.□□	□0.1□
Wt. of Dish + Dry Soil	1□.□□	1□.□1		1□.□□	1□.□□	1□.□1
Wt. of Moisture	1.00	0.□□		3.□□	3.□0	3.□□
Wt. of Dish	□.0□	□.0□		□.11	□.0□	□.0□
Wt. of Dry Soil	□.□□	□.3□		10.□□	10.□□	10.3□
Moisture Content %	1□.□	1□.3		3□.□	3□.0	3□.□

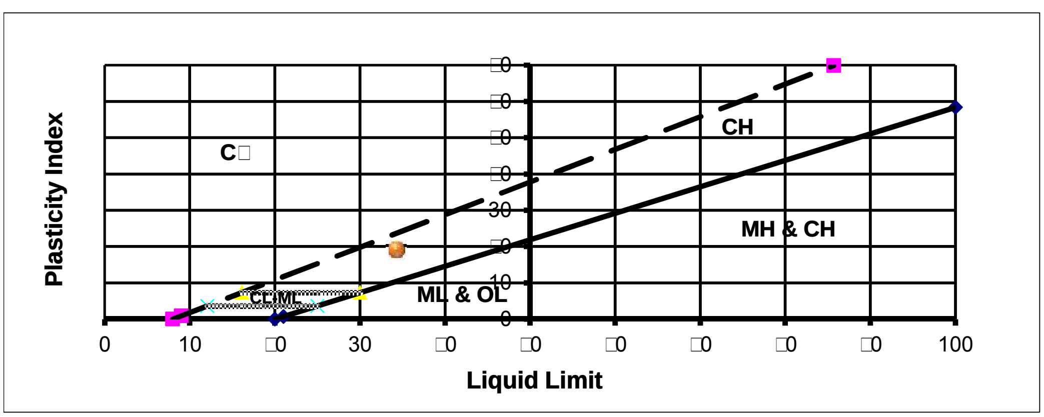
Liquid Limit Graph



□□□□d□□□ □
3□

□□□□□□□□ □
1□

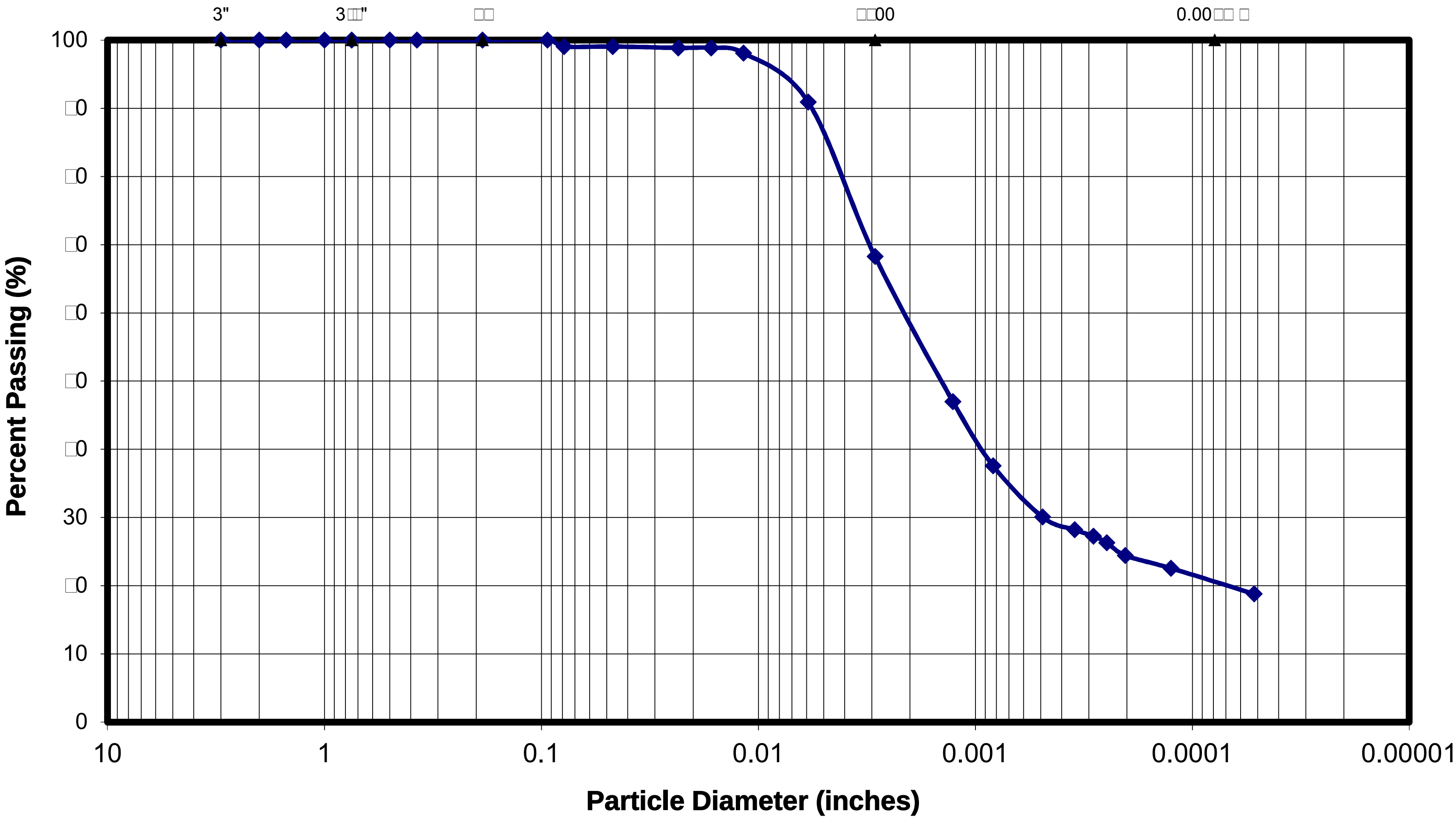
□□□□□□□□□d□□
1□



GEOTEK

Project No. 1□□□□□R
Client: M□□□□□ □□ □□
Project Name: 1□□□ .1□□□□ □□.□□□ □rd□
Date: □31□01□
Sample Desc: □□1□□ □1□□□□□□
GeoTek Lab No: 0

Sieve Analysis w/Hydrometer ASTM D422

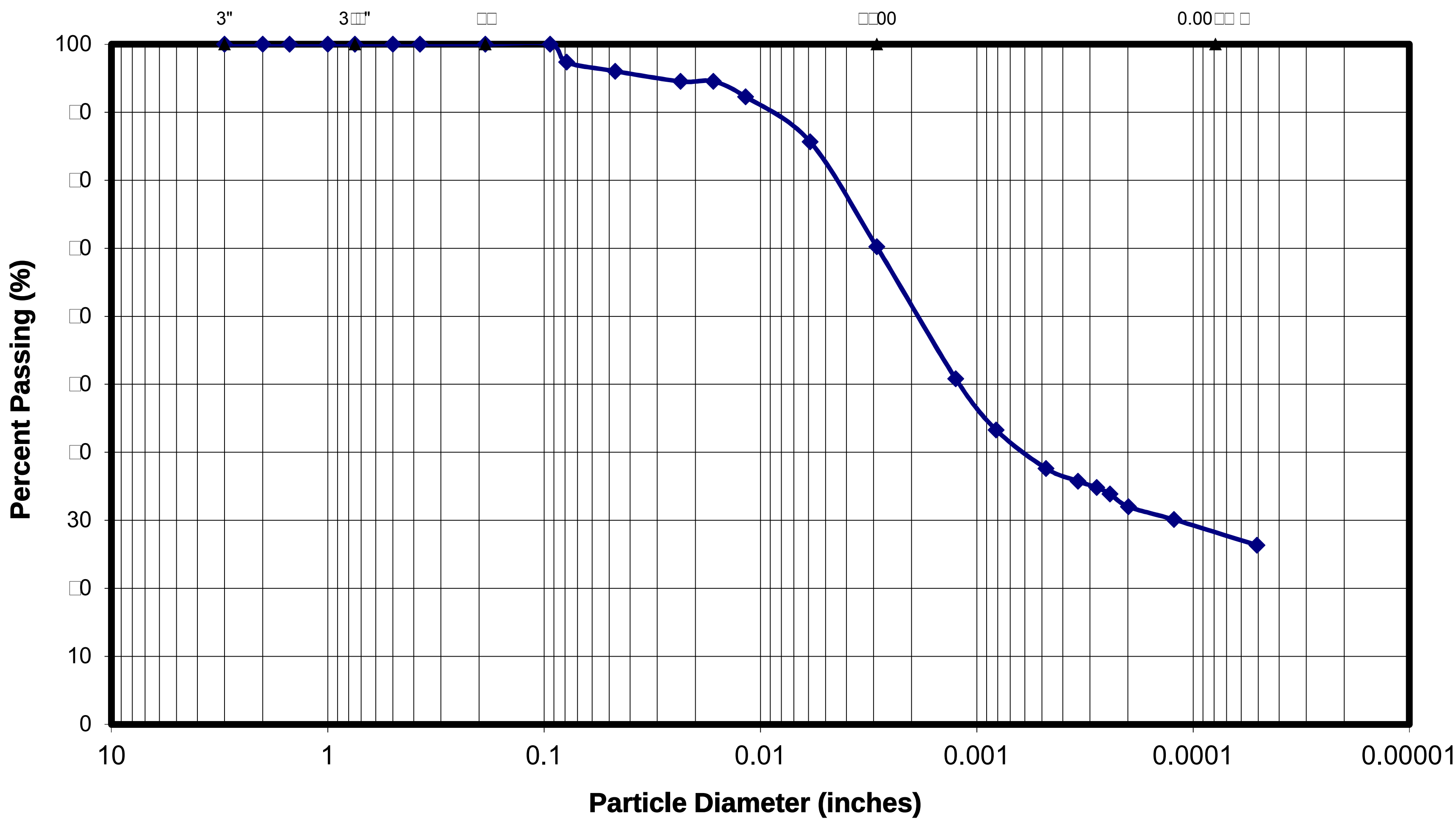


Reviewed By: ____________________ **Date:** ____________________

GEOTEK

Project No. 1□□□□□R
Client: M□□□□□ □□ □□
Project Name: 1□□□ .1□□□□ □□.□□□ □rd□
Date: □31 □01□
Sample Desc: □□□□□ □1 □□□□□□
GeoTek Lab No: 0

Sieve Analysis w/Hydrometer ASTM D422



Reviewed By: ____________________ **Date:** ____________________

Soil Analysis Lab Results

Client: Geotek Inc
Job Name: 1515 W. 178th St., Gardena
Client Job Number: 1949-CR
Project X Job Number: S180830B
September 5, 2018

	Method	ASTM G187		ASTM D516		ASTM D512B		SM 4500-NO3-E	SM 4500-NH3-C	SM 4500-S2-D	ASTM G200	ASTM G51
Bore# / Description	**Depth**	**Resistivity** As Rec'd	Minimum	**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
B-2	0-5.0	18,760	2,010	150	0.0150	42	0.0042	30	0.5	0.03	197	7.77

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract

Please call if you have any questions.

Prepared by,

Nathan Jacob
Lab Technician

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

APPENDIX D

INFILTRATION TEST DATA BY GEOTEK

Updated Geotechnical and Infiltration Evaluation
1515 West 178th Street, Gardena, California
Project No. 1949-CR

GEOTEK

Client:	**Melia Homes**
Project:	**1515 West 178th Street, Gardena, CA**
Project No:	**1949-CR**
Date:	**8/21/2018**

Boring No. **P-1/I-1**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30	min.
Final Depth to Water, D_F =	0.1	in.
Test Hole Radius, r =	4	in.
Initial Depth to Water, D_O =	0	in.
Total Test Hole Depth, D_T =	29.625	in.

Equation - $$I_t = \frac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$$

$H_O = D_T - D_O$ =	29.625	in.
$H_F = D_T - D_F$ =	29.525	in.
$\Delta H = \Delta D = H_O - H_F$ =	0.1	in.
Havg = $(H_O+H_F)/2$ =	29.575	in.

I_t = 0.01 **Inches per Hour**

GEOTEK

Client:	**Melia Homes**
Project:	**1515 West 178th Street, Gardena, CA**
Project No:	**1949-CR**
Date:	**8/21/2018**

Boring No. **P-2/I-2**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30	min.
Final Depth to Water, D_F =	0.1	in.
Test Hole Radius, r =	4	in.
Initial Depth to Water, D_O =	0	in.
Total Test Hole Depth, D_T =	42	in.

Equation - $$I_t = \frac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$$

$H_O = D_T - D_O$ =	42	in.
$H_F = D_T - D_F$ =	41.9	in.
$\Delta H = \Delta D = H_O - H_F$ =	0.1	in.
$H_{avg} = (H_O+H_F)/2$ =	41.95	in.

I_t = 0.01 **Inches per Hour**

GEOTEK

Boring/Excavation Percolation Testing Field Log

Date □□1□01□

Project Location 1□1□□ .□1□□□□□.□□ □rd□□□□□□

Boring/Test Number □□□□1

Earth Description

Diameter of Boring □.0" **Diameter of Casing** 3.0"

Tested by D□

Depth of Boring □.0□

Liquid Description □□□□□□□□□ □□r

Depth to Invert of BMP □.0□

Measurement Method □□□□d□r

Depth to Water Table 30.0□

Depth to Initial Water Depth (d_1) □□.□□□"

Time Interval Standard

Start Time for Pre-Soak □00□□

Water Remaining In Boring (Y/N) □□□

Start Time for Standard □30□□

Standard Time Interval Between Readings 30□□ □

Reading Number	Time Start/End (hh:mm)	Elapsed Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/Comments
1	□30□□ □00□□	30	0.1	0.2	
□	□00□□ □30□□	30	0.1	0.2	
3	□30□□ □00□□	30	0.1	0.2	
□	□00□□ □30□□	30	0.1	0.2	
□	□30□□ 10□00□□	30	0.1	0.2	
□	10□00□□ 10□30□□	30	0.1	0.2	
□	10□30□□ 11□00□□	30	0.1	0.2	
□	11□00□□ 11□30□□	30	0.1	0.2	

Boring/Excavation Percolation Testing Field Log

Date [illegible]1[illegible]01[illegible]

Project Location 1[illegible]1[illegible] .[illegible]1[illegible].[illegible] [illegible]rd[illegible]
Earth Description
Tested by D[illegible]
Liquid Description [illegible]r
Measurement Method [illegible]d[illegible]r

Boring/Test Number [illegible]
Diameter of Boring [illegible].0" **Diameter of Casing** 3.0"
Depth of Boring [illegible].0[illegible]
Depth to Invert of BMP [illegible].0[illegible]
Depth to Water Table 30.0[illegible]
Depth to Initial Water Depth (d_1) [illegible]"

Time Interval Standard
Start Time for Pre-Soak [illegible]3[illegible]
Start Time for Standard [illegible]3[illegible]

Water Remaining In Boring (Y/N) [illegible]
Standard Time Interval Between Readings 30[illegible]

Reading Number	Time Start/End (hh:mm)	Elapsed Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/Comments
1	[illegible]3[illegible] / [illegible]3[illegible]	30	0.1	0.2	
[illegible]	[illegible]3[illegible] / [illegible]3[illegible]	30	0.1	0.2	
3	[illegible]3[illegible] / [illegible]3[illegible]	30	0.1	0.2	
[illegible]	[illegible]3[illegible] / [illegible]3[illegible]	30	0.1	0.2	
[illegible]	[illegible]3[illegible] / [illegible]3[illegible]	30	0.1	0.2	
[illegible]	[illegible]3[illegible] / 10[illegible]3[illegible]	30	0.1	0.2	
[illegible]	10[illegible]3[illegible] / 10[illegible]3[illegible]	30	0.1	0.2	
[illegible]	10[illegible]3[illegible] / 11[illegible]3[illegible]	30	0.1	0.2	

APPENDIX E

GENERAL EARTHWORK AND GRADING GUIDELINES

Updated Geotechnical and Infiltration Evaluation
1515 West 178th Street, Gardena, California
Project No. 1949-CR

GEOTEK

GENERAL GRADING GUIDELINES

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

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the California Building Code, CBC (2016) and the guidelines presented below.

Preconstruction Meeting

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

Grading Observation and Testing

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

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

Site Clearing

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

Treatment of Existing Ground

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

2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
   a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
   b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
   a) They are not placed in concentrated pockets;
   b) There is a sufficient percentage of fine-grained material to surround the rocks;
   c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable

methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.

2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

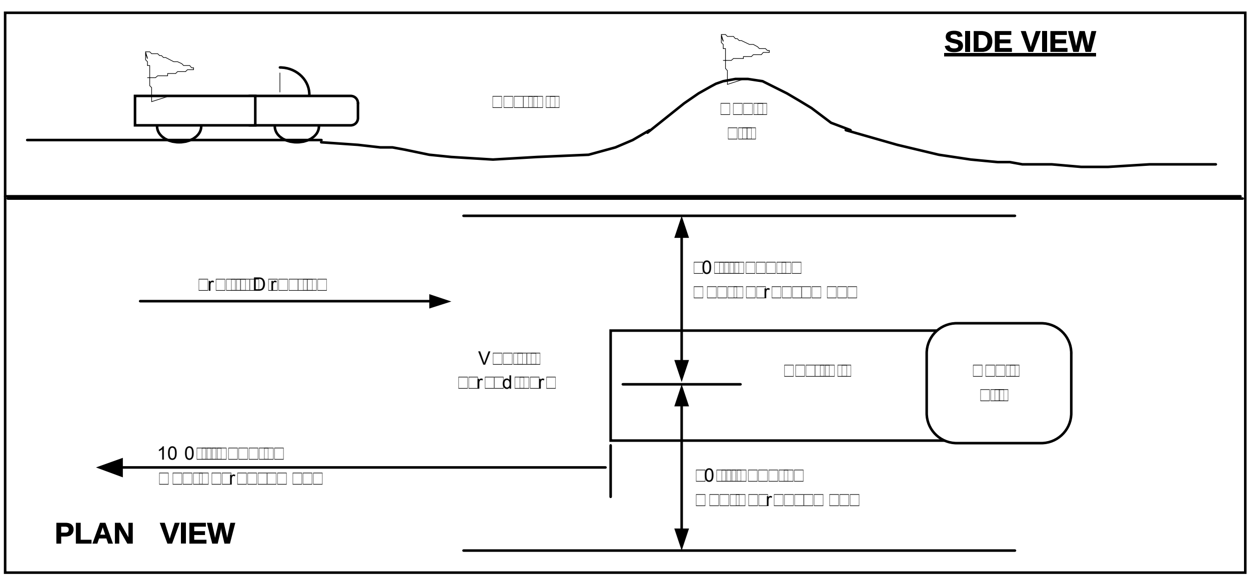
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractors representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor’s representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

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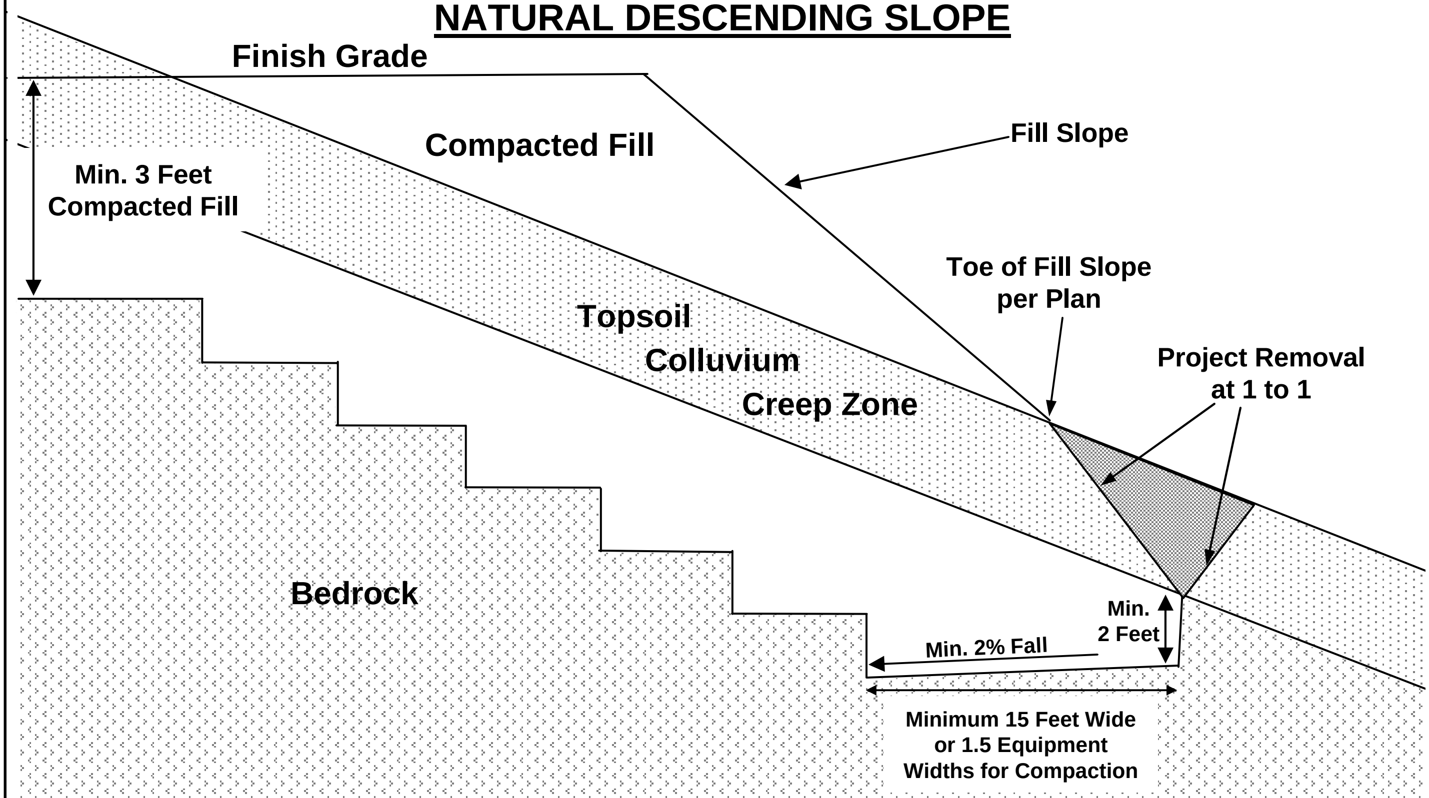
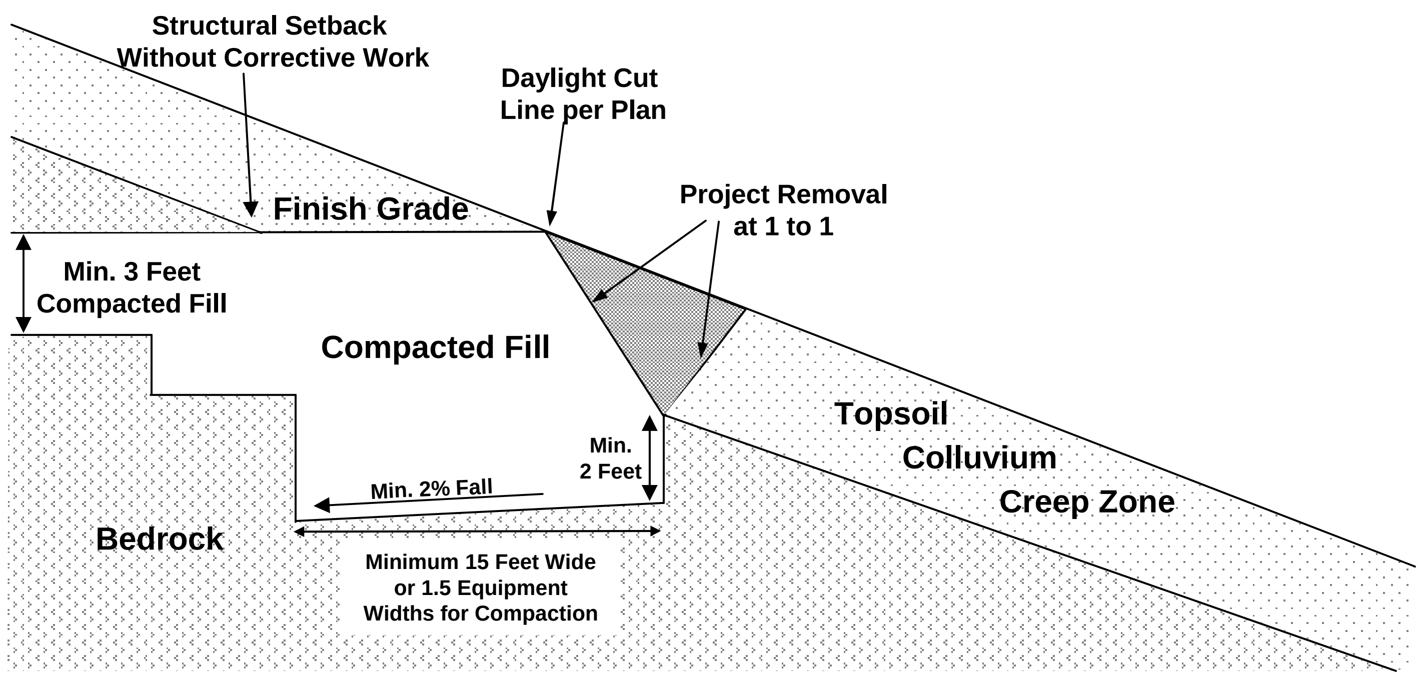
TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



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TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE E-1