GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED TENTATIVE TRACT NO. 31225
PACIFIC LEGACY ENCORE
NORTHWEST CORNER OF WEST METZ ROAD AND NORTH A STREET
PERRIS, RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

PACIFIC COMMUNITIES BUILDER, INC.
1000 DOVE STREET, SUITE 300
NEWPORT BEACH, CALIFORNIA 92660

ATTENTION: MR. TONY ARNEST

PREPARED BY

GEOTEK, INC.
1548 NORTH MAPLE STREET
CORONA, CALIFORNIA 92880

PROJECT NO. 1889-CR **JULY 26, 2018**

GEOTEK

GeoTek, Inc.
1548 North Maple Street, Corona, California 92880
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

July 26, 2018
Project No. 1889-CR

Pacific Communities Builder, Inc.
1000 Dove Street, Suite 300
Newport Beach, California 92660

Attention: Mr. Tony Arnest

Subject: Geotechnical and Infiltration Evaluation
Proposed Tentative Tract No. 31225
Pacific Legacy Encore
Northwest Corner of West Metz Road and North A Street
Perris, Riverside County, California

We are pleased to provide the results of our geotechnical and infiltration evaluation for the proposed residential development that will be constructed on the subject site in the city of Perris. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for site preparation, foundation design, infiltration rates and construction. Based on the results of our evaluation, development of the property appears feasible from a geotechnical viewpoint provided that the recommendations presented in this report and in future reports are incorporated into design and construction.

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

Respectfully submitted,
GeoTek, Inc.

Glenn S. Fraser
GE 2381, Exp. 09/30/19
Senior Project Engineer

Edward H. LaMont
CEG 1892, Exp. 07/31/20
Principal Geologist

Noelle C. Toney
PE C 84700, Exp. 03/31/20
Project Engineer

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions for the proposed development. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- A site reconnaissance,
- Excavation of five exploratory borings for the geotechnical portion of the evaluation,
- Logging and infiltration testing of an additional three hollow stem auger borings in the vicinity of a planned storm water detention basin,
- Collection of soil samples,
- Laboratory testing of selected soil samples,
- Review and evaluation of site seismicity, and;
- Compilation of this geotechnical report which presents our preliminary 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. for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The approximately 15-acre site is located on the northwest corner of West Metz Road and North A Street in the city of Perris. The site configuration is presented on Figure 1. To assist in the preparation of this report, an undated tentative tract map prepared by SP2, Inc. was reviewed. Based on current and historical aerial maps, the area to be developed is currently vacant with no visual evidence of structural improvements. Unpaved roads are interspersed throughout the property, which is dirt-covered. The site is relatively flat and slopes downward to the northeast at an average gradient of less than two percent with a total relief of approximately 19 feet. Single-family residences are situated immediately north of the site, and single-family residences and undeveloped property are present immediately to the west.

2.2 PROPOSED DEVELOPMENT

The development will consist of the construction of 56 single-family residences. The project is currently in the conceptual design stage; however, it is anticipated that the buildings will be one- or two-story wood-frame structures incorporating concrete slab-on-grade floors. The buildings will be supported by conventional continuous and isolated footings that will impose relatively light loads on the underlying soils. A water quality mitigation lot is proposed in the extreme northeast corner of the site. It is not known if this will be a retention basin or a detention basin. The depth of the new basin is not currently known; however, it is expected to be in the range of five feet to 10 feet. Interior streets are also proposed.

Based on the site topography, maximum cuts and fills of approximately seven feet will be required for site development, and major slope and retaining wall construction is not anticipated.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

Our field exploration was conducted on May 11, 2018. For the geotechnical portion of the investigation, five test borings were excavated with a hollow-stem auger drill rig to a maximum depth of 35.1 feet. For the infiltration part of the study, three test borings were excavated to a depth ranging from five feet to 10 feet. One of the infiltration borings was also used for the geotechnical portion of the evaluation. A hollow-stem auger with an outside diameter of 8.5 inches was utilized. The inside diameter of the auger was 4.5 inches. An engineer from GeoTek, Inc. logged the exploratory borings. The boring locations are presented on Figure 2. Logs of the exploratory borings are included in Appendix A.

The exploration logs show subsurface conditions at the dates and locations indicated, and may not be representative of other locations and times. The stratification lines presented on the logs represent the approximate boundaries between soil types, and the transitions may be gradual.

Relatively undisturbed soil samples were recovered at various intervals in the geotechnical borings with a California sampler. The California sampler is a 2.9-inch outside diameter, 2.5-inch inside diameter, split barrel sampler lined with brass rings. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for testing and evaluation. In Boring 1, standard penetration tests were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. An efficiency value of 1.0 was used for the automatic trip hammer. The standard penetration test data are presented on the logs for Boring 1.

In addition, three borings were excavated in the vicinity of the proposed water quality mitigation lot to depths ranging from five feet to 10 feet. Infiltration testing was conducted in these borings in accordance with the requirements of the County of Riverside. The infiltration tests consisted of drilling an eight-inch diameter test hole to the desired depth and installing approximately two inches of gravel in the bottom of the hole. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within the boring. Water was then placed in the borings to presoak the holes and percolation testing was performed the following day.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected soil samples obtained during our field exploration. The purpose of the laboratory testing was to confirm the field classification of the soils encountered and to evaluate the physical properties of the soils for use in engineering design and analysis.

Included in our laboratory testing were moisture-density determinations on all undisturbed samples. Gradation and Atterberg limit tests were performed on selected samples and used in the seismic settlement analysis. Optimum moisture content-maximum dry density relationships were established for a typical soil type so that the relative compaction of the subsoils could be determined. Consolidation testing was performed on selected samples to evaluate the compressibility characteristics of the soils. Expansion index testing was performed on selected samples to evaluate the expansion potential of the on-site soils. Chemical testing comprised of pH, soluble sulfate, chloride and resistivity testing was conducted on selected samples. The moisture-density, Atterberg limit and gradation data are presented on the exploration logs in Appendix A. The maximum density, consolidation, expansion index and chemical test data are presented in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 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. It extends approximately 975 miles south of the Transverse Ranges geomorphic province to the tip of Baja California. 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 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 alluvium and Quaternary age older alluvium (Dibblee, T.W. and Minch, J.A., 2003). No faults are shown in the immediate site vicinity on the maps reviewed for the area.

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the soils encountered on the site is presented in the following sections. Based on our field exploration and observations, the site is generally underlain by alluvial deposits and bedrock.

4.2.1 Undocumented Fill

Undocumented fill was not encountered in our explorations. Undocumented fill may be present on areas of the site that were not explored.

4.2.2 Alluvium

Alluvial deposits consisting of loose to medium dense silty sands to clayey sands with varying amounts of gravel and stiff sandy silts were encountered in our test borings. At our boring locations the loose soils extended to a maximum depth of approximately seven feet. Expansion index testing reveals that the near-surface soils exhibit a "very low" and "low" expansion potential.

4.2.3 Older Alluvium

Quaternary-age older alluvial deposits consisting of medium dense to very dense silty sands to clayey sands with varying amounts of gravel were encountered in our test borings.

4.2.4 Bedrock

Bedrock consisting of dense to very dense granitic rock was encountered in Borings 1 through 4 at various depths. The boring numbers and depths to bedrock are presented on the following table:

BORING NUMBER	DEPTH TO BEDROCK (ft.)
1	20.5
2	17.0
3	15.0
4	17.0
5	Not encountered at the termination depth of 16.5 feet

Refusal occurred on very dense bedrock at a depth of 35.1 feet in Boring 1.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during our site visit. If encountered during earthwork construction, surface water on this site is the result of precipitation or possibly some minor surface run-off from immediately surrounding properties. Overall site area drainage is generally in a northeasterly direction, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was not encountered in our test borings. Water wells in the vicinity of the subject site have reported recent groundwater levels to be between 80 and 120 feet below the ground surface.

4.4 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 presently known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007).

4.4.1 Seismic Design Parameters

The site is located at approximately 33.7952 Latitude and -117.2360 Longitude. Site spectral accelerations (S_s and S_1), for 0.2 and 1.0 second periods for a Class "D" site, were determined from the USGS Website, Earthquake Hazards Program, U.S. Seismic Design Maps for Risk-Targeted Maximum Considered Earthquake (MCE) Ground Motion Response Accelerations for the Conterminous 48 States by Latitude/Longitude. The results are calculated in accordance with the 2010 ASCE 7 requirements and are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.500g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.600g
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}	1.500g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.900g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.000g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.600g

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

4.5 LIQUEFACTION AND SEISMICALLY-INDUCED SETTLEMENT

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

GeoTek evaluated the liquefaction potential at the site using the computer program LiquefyPro Version 5 and the results of Boring 1. An earthquake magnitude of M7.0 and an acceleration of 0.50g were used. Groundwater was not encountered in our explorations. Water wells in the vicinity of the subject site have reported recent groundwater levels to be between 80 and 120 feet below the ground surface. For a liquefaction analysis, ground water would not be a factor. Since actual liquefaction will not occur, an analysis for seismically induced settlement was performed. As recommended by the State of California Special Publication 117, our seismic settlement analysis has incorporated a safety factor of 1.3.

The liquefaction analysis is presented in Appendix C and reveals a dynamic settlement of approximately 0.1 inch. The total settlement will occur over a large area and will not affect local buried utilities. Within the building area, we would estimate the differential dynamic settlement would be about one-half the total. Since this is a relatively small value, it is our opinion that seismically-induced settlement should not be a consideration in the design of the structures.

Since liquefaction will not occur on the site, lateral spread should not be a consideration in the design of the residences.

4.6 OTHER SEISMIC HAZARDS

Based on the Riverside County Parcel Report, the site is susceptible to subsidence.

Evidence of ancient landslides or slope instability at this site was not observed during our investigation and the project site is relatively flat. Thus, the potential for landslides is considered negligible for design purposes.

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.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations, and those provided by this firm at a later date are incorporated into the design and construction phases of development. Site development and grading plans should be reviewed by GeoTek, Inc. when they become available.

The on-site soils exhibit a "very low" and "low" expansion potential. The soils with a "low" expansion potential were encountered below a depth of five feet in Boring 1. Earthwork operations will combine the soils, therefore the recommendations in this report assume the presence of "very low" expansive soils. Expansion index testing should be conducted at the completion of earthwork operations to verify this assumption.

Undocumented fill soils were not encountered in our explorations. Undocumented fill may be present in areas that were not explored.

The County of Riverside indicates that the site is in a zone that is susceptible to subsidence. In order to counteract potential distress from subsidence, additional reinforcement is recommended for the building footings and on-grade floor-slabs.

5.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Perris, the County of Riverside, the 2016 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix E outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix E.

5.2.1 Site Clearing and Demolition

In areas of planned grading and improvements, the site should be cleared of vegetation and other deleterious materials. Debris should be properly disposed of off-site. Voids resulting from site clearing should be replaced with engineered fill.

5.2.2 Removals/Overexcavations

Undocumented fill should be removed below all areas to receive improvements. These areas include the new buildings, any retaining wall and screen wall footings, and pavement and hardscape areas.

5.2.3 Building Areas and Retaining Wall and Screen Wall Footings

Subsequent to removal of undocumented fill, the natural soils below and within five feet of the building envelopes and any retaining wall and screen wall footings should be removed to a depth of three feet below the natural ground surface or two feet below the bottom of the footings, whichever is greater. A representative of this firm should observe the bottom of all excavations. In areas where loose soil is present in the bottom of the excavations, the removals should continue until competent natural materials are encountered. Competent materials are defined as relatively non-porous natural soils with an in-place relative compaction of at least 85 percent. Based on the results of our test borings, compressible natural soils were present on various areas of the site to a maximum depth of seven feet but are anticipated to be between three and seven feet. Deeper deposits of loose alluvium may be present in areas that were not explored.

In cut areas, subsequent to cutting to the desired elevation, the soils should be removed to a depth of 24 inches below the bottom of the footings and floor-slabs. The purpose of this recommendation is to provide a uniform mat of engineered fill below the footings and floor-slabs.

5.2.4 Horizontal Extent of Removals

In areas where removal depths exceed five feet below the proposed building, retaining wall and screen wall footings, the horizontal limits of removals outside the perimeter of these structural elements should be equal to the depth of the soil removals below the bottom of the footings.

5.2.5 Pavement and Hardscape Areas

Undocumented fill should be removed below pavement and hardscape areas. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

5.2.6 Preparation of Excavation Bottoms

A representative of this firm should observe the bottom of all excavations. Upon approval, the exposed soils and all soils in areas to receive engineered fill should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

5.2.7 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Engineered fill should be placed in loose lifts with a thickness of eight inches or less, moisture conditioned to at least two percent above the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D-1557).

5.2.8 Excavation Characteristics

Since bedrock is relatively deep, it is anticipated that it will not be encountered during earthwork operations. This assumption should be verified during a grading plan review. Excavation in the on-site alluvial soils is expected to be feasible utilizing heavy-duty grading equipment in good operating condition. 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 on-site materials should be stable at 1½:1 (horizontal: vertical) inclinations for cuts less than five feet in height.

5.2.9 Shrinkage and Subsidence

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

Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of up to 15 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.10 foot may be anticipated for the underlying soils.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2016 CBC, are presented below. Based on laboratory test results, subsequent to earthwork operations it is anticipated that the near-surface soils may have a "very low" to "low" expansion potential.

Additional expansion index and soluble sulfate testing of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions.

A summary of our foundation design recommendations is presented in the following table:

Design Parameter	"Very Low" Expansion Potential	"Low" Expansion Potential
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	Footings supporting one floor – 12	Footings supporting one floor – 12
Minimum Foundation Width (Inches)*	Footings supporting one floor – 12	Footings supporting one floor – 12
Minimum Slab Thickness (actual)	4 inches	4 inches
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	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab	6" x 6" – W2.9/W2.9 welded wire fabric placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom of the footing	Two No. 4 reinforcing bars, one placed near the top and one near the bottom of the footing
Effective Plasticity Index***	N/A	16 – design value
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 100 percent of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum of 110 percent of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

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

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

The following criteria for design of foundations are preliminary and should be re-evaluated based on the results of additional laboratory testing of samples obtained near finish pad grade.

An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of footings 12 inches deep and 12 inches wide. This value may be increased by 300 pounds per square foot for each additional 12 inches in depth and 150 pounds per square foot for each additional 12 inches in width to a maximum value of 4,000 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

Structural foundations may be designed in accordance with the 2016 CBC, and to withstand a total settlement of 1 inch and maximum differential settlement of one-half of the total settlement over a horizontal distance of 40 feet.

The passive earth pressure may be computed as an equivalent fluid having a density of 300 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 0.30 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2016 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2013 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the vapor retarder placed on the underlying 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. Although the CBC specifies a 6 mil vapor retarder membrane, a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

A two inch layer of clean sand with a sand equivalent of at least 30 should be placed over the moisture vapor retardant membrane to promote setting of the concrete. The moisture in the sand should not exceed two percent below the optimum moisture content.

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 environmental 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 permeability) to achieve the desired performance level.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-

Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek recommends that a qualified person, such as a flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the buildings be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person should provide recommendations relative to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate.

In addition, the recommendations in this report and our services in general are not intended to address mold prevention, since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

We recommend that control joints be placed in two directions spaced approximately 24 to 36 times the thickness of the slab in inches. These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer.

5.3.2 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.

5.3.3 Foundation Set Backs

Minimum setbacks for all foundations should comply with the 2016 CBC or City of Perris requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movement and/or differential settlement. If large enough, these movements can compromise the integrity of the improvements.

- The outside top edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least five feet and need not exceed 40 feet.

- The bottom of any proposed foundations should be deepened so as to extend below a 1:1 upward projection from the bottom edge of the nearest excavation and the bottom edge of the closest footing.

5.3.4 Retaining and Garden Wall Design and Construction

5.3.4.1 General Design Criteria

Retaining wall foundations should be embedded a minimum of 18 inches into engineered fill. Retaining wall foundations should be designed in accordance with Section 5.3.1 of this report. Structural requirements may govern and should be evaluated by the project structural engineer.

All earth retention structure plans should be reviewed by this office prior to finalization.

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 wall designer. The soil used as backfill behind retaining walls should have a "very low" expansion potential and should be densified to at least 90 percent relative compaction (ASTM D-1557).

In general, cantilever earth retention structures, which are designed to yield at least 0.001H, where H is equal to the height of the structure from the top of the wall to the base of the footing may be designed using the active condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at the 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 structure or traffic loading, should be considered in the design of earth retention structures. Loads applied within a 1:1 (h:v) projection from the surcharging structure on the stem of the retaining wall should be considered in the design.

Final selection of the appropriate design parameters should be made by the project earth retention structure designer, based upon the local practices and ordinates, expected structure response, and desired level of conservatism.

5.3.4.2 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. 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, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf)
Level	43
2:1	75

* The design pressures assume the backfill materials have an expansion index less than or equal to 20. Backfill zone includes the area between the back of the wall to a plane (1:1, h:v) up from the bottom of the wall foundation to the adjacent ground surface.

5.3.4.3 Retaining Wall Backfill and Drainage

Wall backfill should include a minimum one foot wide section of ¾- to one inch clean crushed rock or approved equivalent. The rock should be placed immediately adjacent to the back of the wall and extend up from a backdrain to within approximately 12 inches of finish grade. The portion of the rock opposite the back of the wall should be covered with a layer of filter fabric comprised of Mirafi 140N or the equivalent. The upper 12 inches of backfill should consist of compacted on-site soil. Backfill placed within the active zone as defined by a 1:1 (H:V) projection from the back of the retaining wall footing up to the retained surface behind the wall should consist of very low expansive soil. The presence of other soils placed within the 1:1 projection will necessitate revision to the parameters provided and modification of wall designs.

The backfill soil should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction (ASTM D-1557). Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC, or approved equivalent) in a minimum of one cubic foot per linear foot of ¾-inch or one inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and the water should be directed to an appropriate disposal area.

Walls from two to four feet in height may be drained using localized gravel packs (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag) behind weep holes at 10 feet maximum spacing. Weep holes should be provided, or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

5.3.4.4 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 65 pcf, plus any applicable surcharge loading. 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.

5.3.4.5 Other Design Considerations

- Retaining and garden wall foundation elements should be designed in accordance with building code setback requirements. A minimum horizontal setback distance of five feet as measured from the top outside edge of the footing to an adjacent slope face is recommended.
- 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.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

5.3.5 Soil Corrosivity

Based on the chemical test results presented in Appendix A, the corrosivity test results indicate that the on-site soils are corrosive to buried ferrous metal in accordance with current standards used by corrosion engineers. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer.

5.3.6 Soil Sulfate Content

Based on the chemical test results presented in Appendix A, the sulfate test results on samples obtained from the project site indicate soluble sulfate contents of less than 0.1% by weight should be expected. Soluble sulfate contents of this level would be in the range of "not applicable" (i.e. negligible) per Table 4.2.1 of ACI 318. Based on the test results and Table 4.3.1 of ACI 318, no special concrete mix design would be necessary to resist sulfate attack.

5.3.7 Import Soils

Import soils should have a "very low" expansion potential. GeoTek, Inc. also recommends that the proposed import soils be tested for expansion and corrosivity potential. GeoTek, Inc. should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

5.3.8 Pavement Construction

Aggregate base and the upper 12 inches of subgrade should be densified to a minimum relative compaction of 95 percent in accordance with ASTM D 1557.

5.3.9 Concrete Flatwork

5.3.9.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four inch minimum thickness. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices typically utilized in 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 in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. should be pre-saturated to a minimum of 100 percent of the optimum moisture content to a depth of 12 inches.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Perris and County of Riverside specifications, and under the observation and testing of GeoTek, Inc. and a City or County inspector, if necessary.

5.3.9.2 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 undergoes 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 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, Inc. suggests that control joints be placed in two directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

5.4 INFILTRATION TEST RESULTS

Percolation rates obtained from the infiltration testing were converted to a field infiltration rate using the Porchet Method. The field infiltration rates calculated are indicated in the following table:

SUMMARY OF FIELD INFILTRATION RATES			
Boring/Area	Depth of Test (Feet)	Material Encountered at Depth of Test	Field Infiltration Rate (Inches per Hour)
I-1	5	Sandy silt	0.97
I-2	7	Silty fine sand	1.78
I-3	10	Silty fine to medium sand with some clay	1.99

The percolation data sheets and infiltration conversion worksheets are presented in Appendix D. The recommended design field infiltration rate is 0.97 inch per hour. The civil engineer should assign a suitable safety factor to this value.

In addition, over the lifetime of the detention or retention basin, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rates to design the infiltration system.

Detailed infiltration data is included in Appendix C.

5.5 POST CONSTRUCTION CONSIDERATIONS

5.5.1 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. An abatement program to control ground-burrowing rodents should be implemented and maintained. Burrowing rodents can decrease 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 foundations. This type of landscaping should be avoided.

5.5.2 Drainage

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 adjacent to the footings and floor-slabs. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

Roof gutters should be installed that will direct the collected water at least 20 feet from the buildings.

5.6 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that specifications and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek, Inc. representatives be present during site grading and foundation construction to observe and document proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek, Inc. 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 on-site and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trench backfill. Also, perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

If requested, a construction observation and compaction report can be provided by GeoTek, Inc. 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.

6. 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 property. 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 geotechnical engineering standards normally used on similar projects in this locality.

7. LIMITATIONS

Our findings are based on site conditions observed and the stated sources. Thus, our comments are professional opinions that are limited to the extent of the available data.

GeoTek has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusions 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 of any kind is expressed or implied. Standards of care/practice are subject to change with time.

8. SELECTED REFERENCES

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Pacific Communities Builder, Inc.
Tentative Tract No. 31225
Pacific Legacy Encore Project
Perris, Riverside County, California

GeoTek Project No. 1889-CR

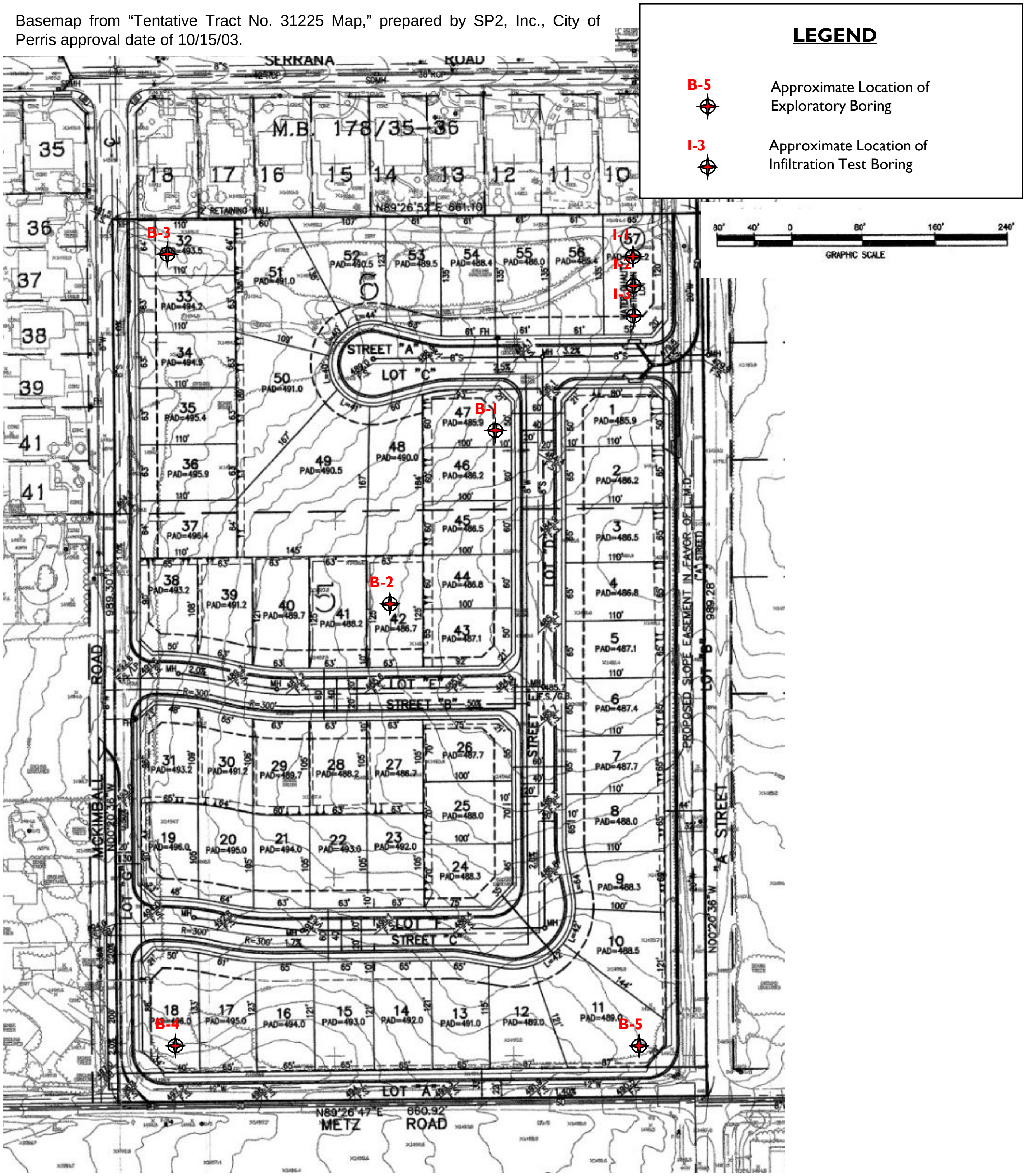
N

Figure 1

Site Location Map

GEOTEK

Basemap from "Tentative Tract No. 31225 Map," prepared by SP2, Inc., City of Perris approval date of 10/15/03.



Pacific Communities Builder, Inc.
Tentative Tract No. 31225
Pacific Legacy Encore Project
Perris, Riverside County, California

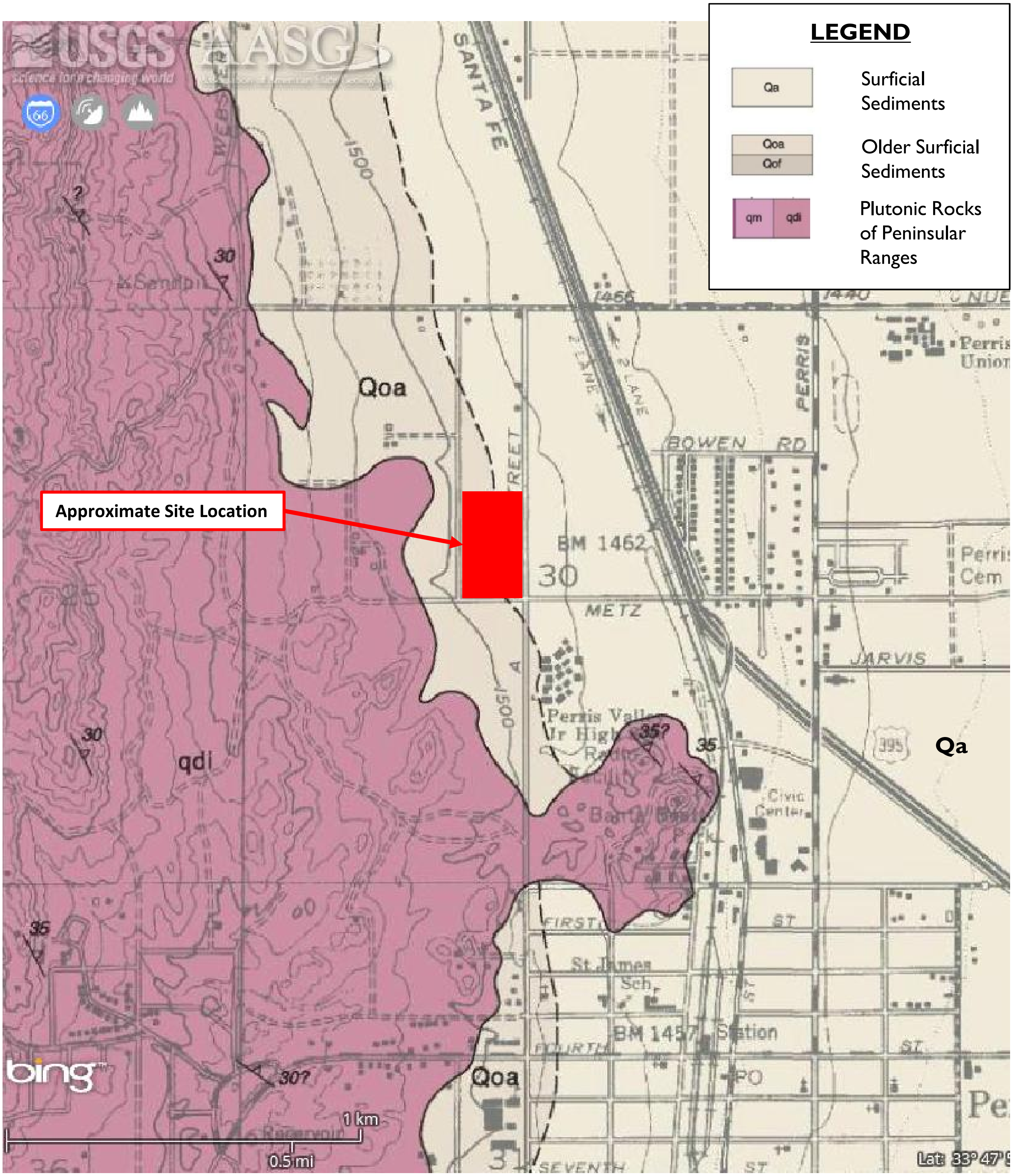
GeoTek Project No. 1889-CR

N

Figure 2

Boring Location Map

GEOTEK



Pacific Communities Builder, Inc.
Tentative Tract No. 31225
Pacific Legacy Encore Project
Perris, Riverside County, California

GeoTek Project No. 1889-CR

N

Figure 3

Geologic Map

GEOTEK

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Tentative Tract No. 31225
Perris, Riverside County, California
Project No. 1889-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 log of boring. 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 5 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/TRENCH LOG LEGEND

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

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/trench

(Additional denotations and symbols are provided on the log of borings/trenches)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

BORING NO.: B-1 (SHEET 1 OF 2)

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
				SM-SC	**ALLUVIAL DEPOSITS:** Silty f SAND with a trace of clay, brown, slightly moist, loose			LL = 2, PL = 0
		4 6 9			-becoming medium dense at 3 feet	7.3	124.2	
5		5 7 11		SC	Clayey f-c SAND, brown, moist, medium dense			% Passing #200 = 46.2 LL = 29, PL = 13
		13 23 38				10.6	129.5	
10		15 25 34		SM	Silty f-m SAND, brown, moist, medium dense	8.1	---	
		8 12 12						% Passing #200 = 21.4
15		10 15 22		SM	Silty f-m SAND with a trace of clay, brown, moist, medium dense	8.4	126.4	
		11 13 12						% Passing #200 = 21.1
20		50/4"	50/4"	50/4"		6.4	113.8	
		50/6"			**GRANITIC BEDROCK:** Granitic BEDROCK (quartz diorite), light brown and light gray, slightly moist, very dense			
25		50/4" 40 50/5"				5.6	114.8	
30		50/2" 50/2"			- becoming moist at 30 feet	15.4	116.8	

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:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

BORING NO.: B-1 (SHEET 2 OF 2)

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
35	SPT	50/1"						
					BORING TERMINATED AT 35.1 FEET DUE TO REFUSAL ON VERY DENSE BEDROCK			
					No groundwater encountered Boring backfilled with soil cuttings			
40								
45								
50								

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:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

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
					ALLUVIAL DEPOSITS:			
	SPT	5 7 9		SM	Silty f SAND, light brown, slightly moist, loose	3.5	110.0	
5	SPT	5 3 11				5.5	112.1	
				SM-SC	Silty f SAND with some clay, light brown, slightly moist, medium dense			
10	SPT	40 50/3"		SM	Silty f-c SAND, light brown, slightly moist, very dense	5.7	118.9	
15	SPT	50/2"			Silty f-m SAND with a trace of gravel, light gray, slightly moist, very dense (Weathered granitic bedrock)	1.3	117.7	
					GRANITIC BEDROCK: Granitic BEDROCK (quartz diorite), gray-brown, dry, very dense			
20	SPT					1.9	118.7	
					BORING TERMINATED AT 20.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
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:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

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
					OLDER ALLUVIAL DEPOSITS:			
	Ring	11 25 26		SM	Silty f-m SAND, brown, slightly moist, medium dense	3.5	---	
5	Ring	6 6 10		SM	F-m SAND with some silt, brown, dry, loose	2.1	108.5	
	Ring	40 50/3"			Silty f SAND, light brown, moist, very dense	8.2	122.5	
10	Ring	20 50/5"				8.4	128.6	
15	Ring	50/3"			**GRANITIC BEDROCK:** Granitic BEDROCK (quartz diorite), brown-orange, dry, very dense	1.5	---	
20	Ring	50/1"						
					BORING TERMINATED AT 20 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
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:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					OLDER ALLUVIAL DEPOSITS:			
	Ring	22 50/5"		SM	Silty f SAND with a trace of clay, brown, slightly moist, medium dense - becoming very dense at 1.5 feet	7.2	---	
	Ring	35 50/4"				7.6	122.6	
5				SM	F-m SAND with some silt, brown, moist, very dense			
	Ring	33 50/6"		SM	Silty f SAND, light brown, slightly moist, very dense	6.2	131.6	
10	Ring	21 31 50		SM	Silty f-m SAND, brown, slightly moist, very dense	7.2	---	
15	Ring	50/9"				3.8	---	
					GRANITIC BEDROCK: Granitic BEDROCK (quartz diorite), brown-orange, dry, very dense			
20	Ring	50/1"						
					BORING TERMINATED AT 20 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
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:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

BORING NO.: B-5

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
					ALLUVIAL DEPOSITS:			
				SC	Silty f SAND with some clay, brown, slightly moist, medium dense			
	SPT	10 13 12				3.6	117.7	
5	SPT	40 50/5"			-becoming very dense at 5 feet	5.5	118.5	
10	SPT	30 50/6"		SM	Silty f-m SAND, brown, moist, very dense	7.4	132.8	
15	SPT	13 17 23			- becoming medium dense at 15 feet	5.9	120	
					BORING TERMINATED AT 16.5 FEET			
					No groundwater encountered Boring backfilled with soil 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:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	**BORING NO.: I-1** **MATERIAL DESCRIPTION AND COMMENTS**	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					ALLUVIAL DEPOSITS:			
				ML	Sandy SILT, light brown, slightly moist, stiff			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

BORING NO.: I-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
					ALLUVIAL DEPOSITS:			
				ML	Sandy SILT, light brown, slightly moist, stiff			
5				SM	Silty f SAND, light brown, moist, loose			
					BORING TERMINATED AT 7 FEET			
					No groundwater encountered			
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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Communities Builder, Inc.	**DRILLER:**	2R Drilling	**LOGGED BY:**	DA
PROJECT NAME:	Pacific Legacy Encore	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	
PROJECT NO.:	1889-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Boring Location Map			**DATE:**	5/11/2018

BORING NO.: I-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
					ALLUVIAL DEPOSITS:			
	Large Bulk			SM	Silty f-m SAND, brown, slightly moist, loose			
	Ring	11 11 10			- becoming medium dense at 2 feet	4.5	126.3	
5	Ring	11 12 22		SM-SC	Silty f-m SAND with some clay, light brown, moist, medium dense	7.7	119.2	
	Ring	15 24 44				7.5	134.1	
10					**BORING TERMINATED AT 10 FEET**			
					No groundwater encountered			
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 B

LABORATORY TEST RESULTS

Tentative Tract No. 31225
Perris, Riverside County, California
Project No. 1889-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Atterberg Limits
Laboratory testing to determine the liquid and plastic limits was performed in general accordance with ASTM D4318. The results of the testing are included on the boring logs in Appendix A.

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 log of borings in Appendix A.

Consolidation
Consolidation testing was performed on selected samples of the site soils according to ASTM Test Method D 2435. The results of this testing is presented in Appendix B.

Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080. The rate of deformation is approximately 0.035 inch per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The results of the testing are presented in Appendix B.

Expansion Index
The expansion potential of the soils was determined by performing expansion index testing on two samples in general accordance with ASTM D 4829. The results of the testing are provided below.

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-1	1-5	Silty fine sand with a trace of clay	9	Very Low
B-1	5-10	Clayey fine to coarse sand	46	Low

In-Situ Moisture and Density
The natural water content was determined (ASTM D 2216) on samples of the materials recovered from the subsurface exploration. In addition, in-place dry density determination (ASTM D 2937) were performed on relatively undisturbed samples to measure the unity weight of the subsurface soils. Results of these tests are shown on the logs at the appropriate sample depths in Appendix A.

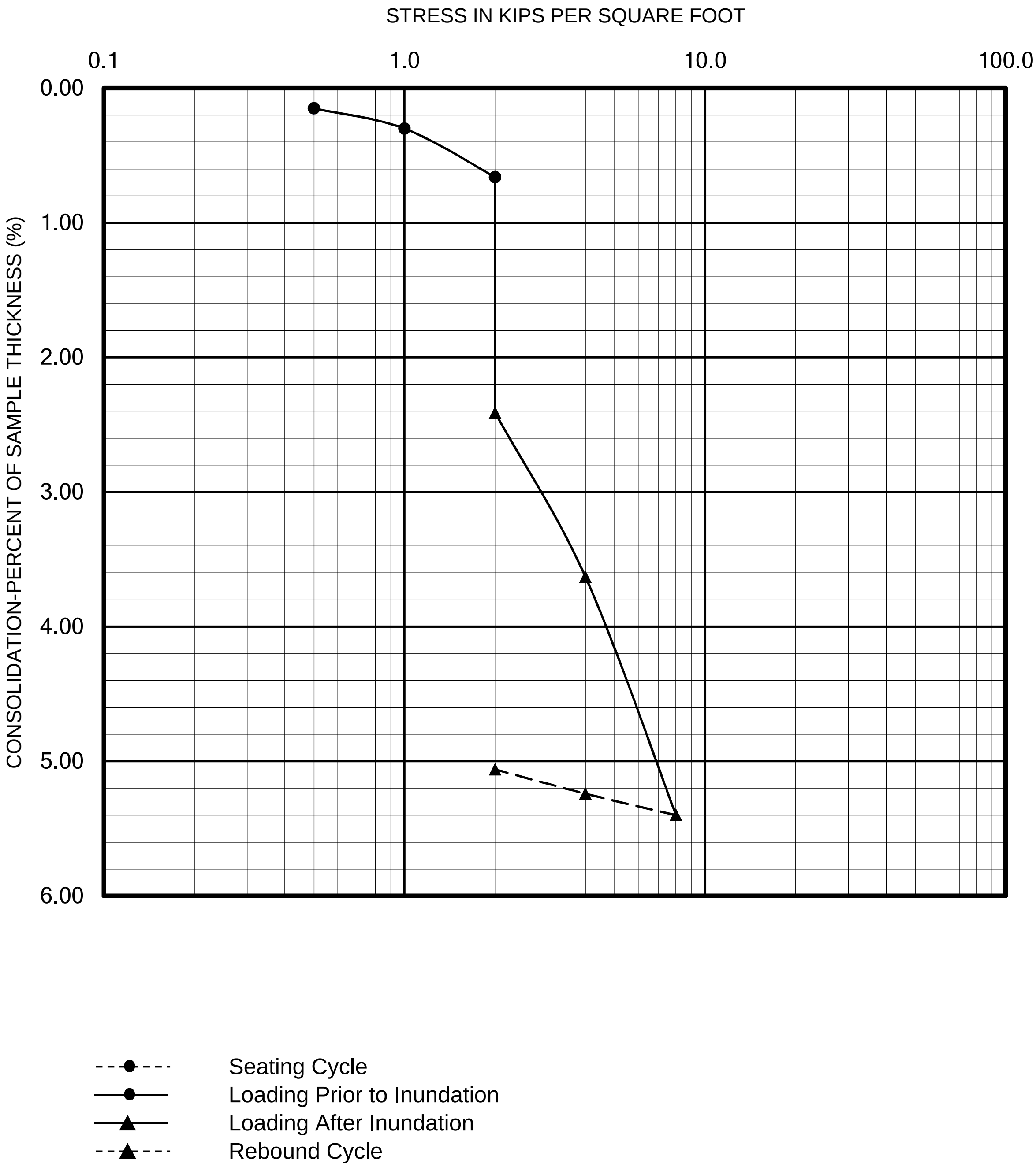
Materials Finer Than the No. 200 Sieve
A #200 sieve wash was performed on selected samples of the soils according to ASTM Test Method D 1140. The results of this testing are presented on the boring logs in Appendix A.

Moisture-Density Relationship
Laboratory testing was performed on two samples obtained during the subsurface exploration. The laboratory maximum dry density and optimum moisture content was determined in general accordance with ASTM D 1557. The results of the testing are provided below.

Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	1-5	Silty fine sand with a trace of clay	134.0	8.0
B-1	5-10	Clayey F to C sand	133.5	8.5

Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content was performed by others in general accordance with ASTM D516. Resistivity testing was completed by others in general accordance with ASTM G187. Testing to determine the chloride content was performed by others in general accordance with ASTM D512B. The results of the testing are provided below.

Boring No.	Depth (ft.)	pH CT-643	Chloride CT-422 (ppm)	Sulfate CT-417 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-1	1-5	7.23	66	0.0150	12,060
B-1	5-10	8.96	168	0.0210	3,618



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

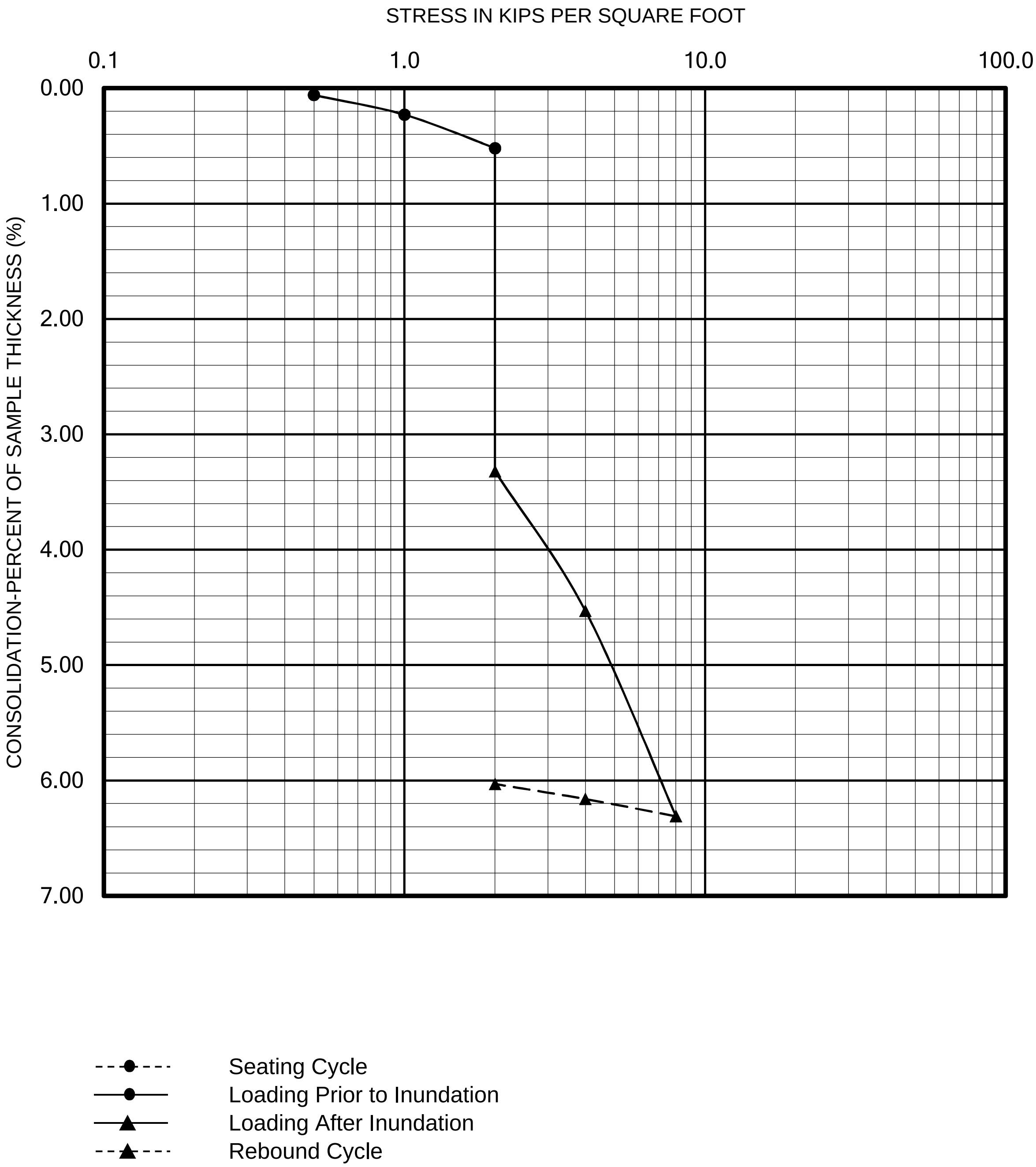
GEOTEK

CONSOLIDATION REPORT

Sample:
B-2 @ 5

App. B

CHECKED BY: NCT	Lab: DI
PROJECT NO.: 1889-CR	Date: 5/29/2018



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

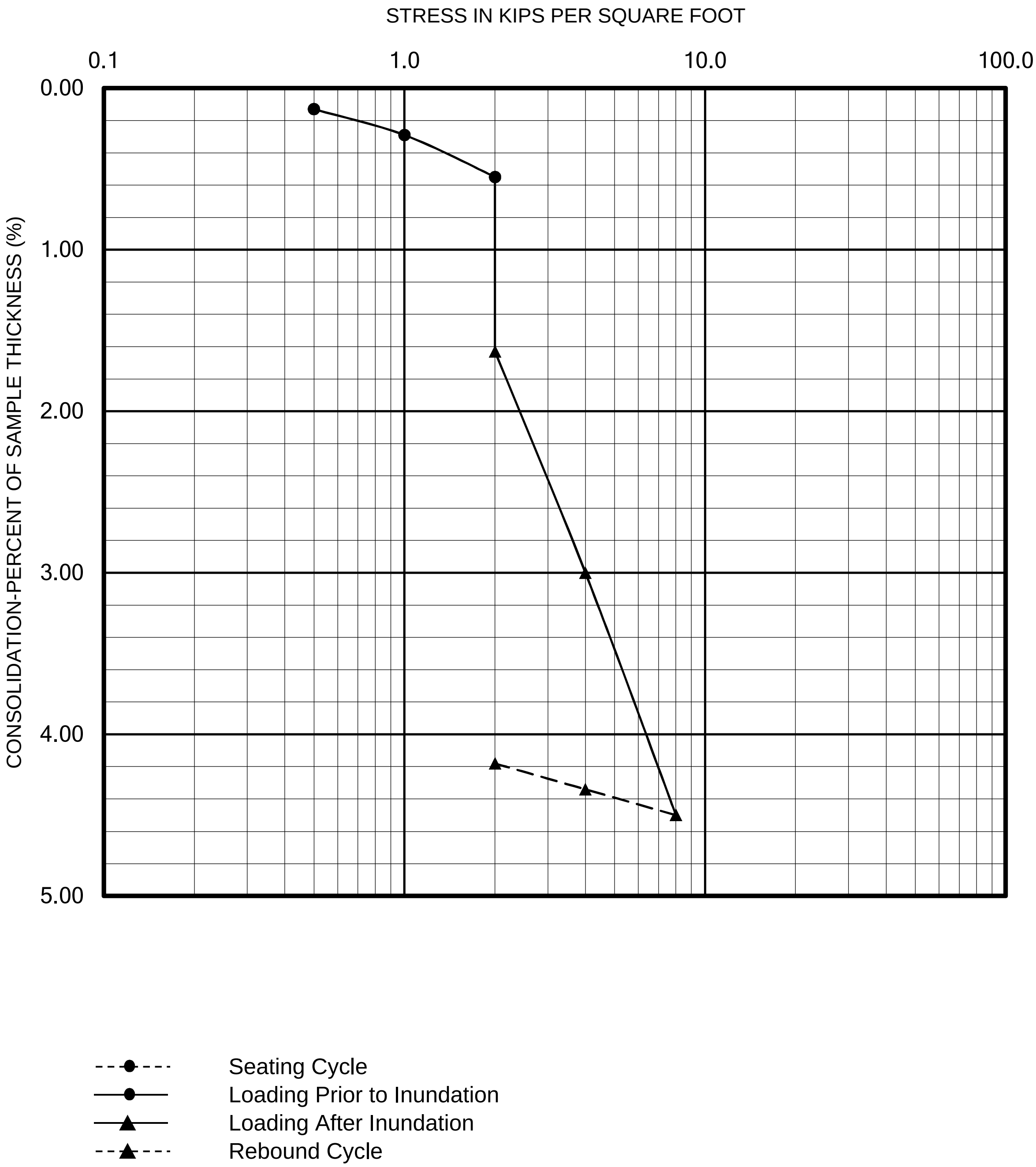
GEOTEK

CHECKED BY: NCT	Lab: DI
PROJECT NO.: 1889-CR	Date: 5/29/2018

CONSOLIDATION REPORT

Sample:
B-3 @ 5

App. B



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

GEOTEK

CHECKED BY: NCT	Lab: DI
PROJECT NO.: 1889-CR	Date: 5/29/2018

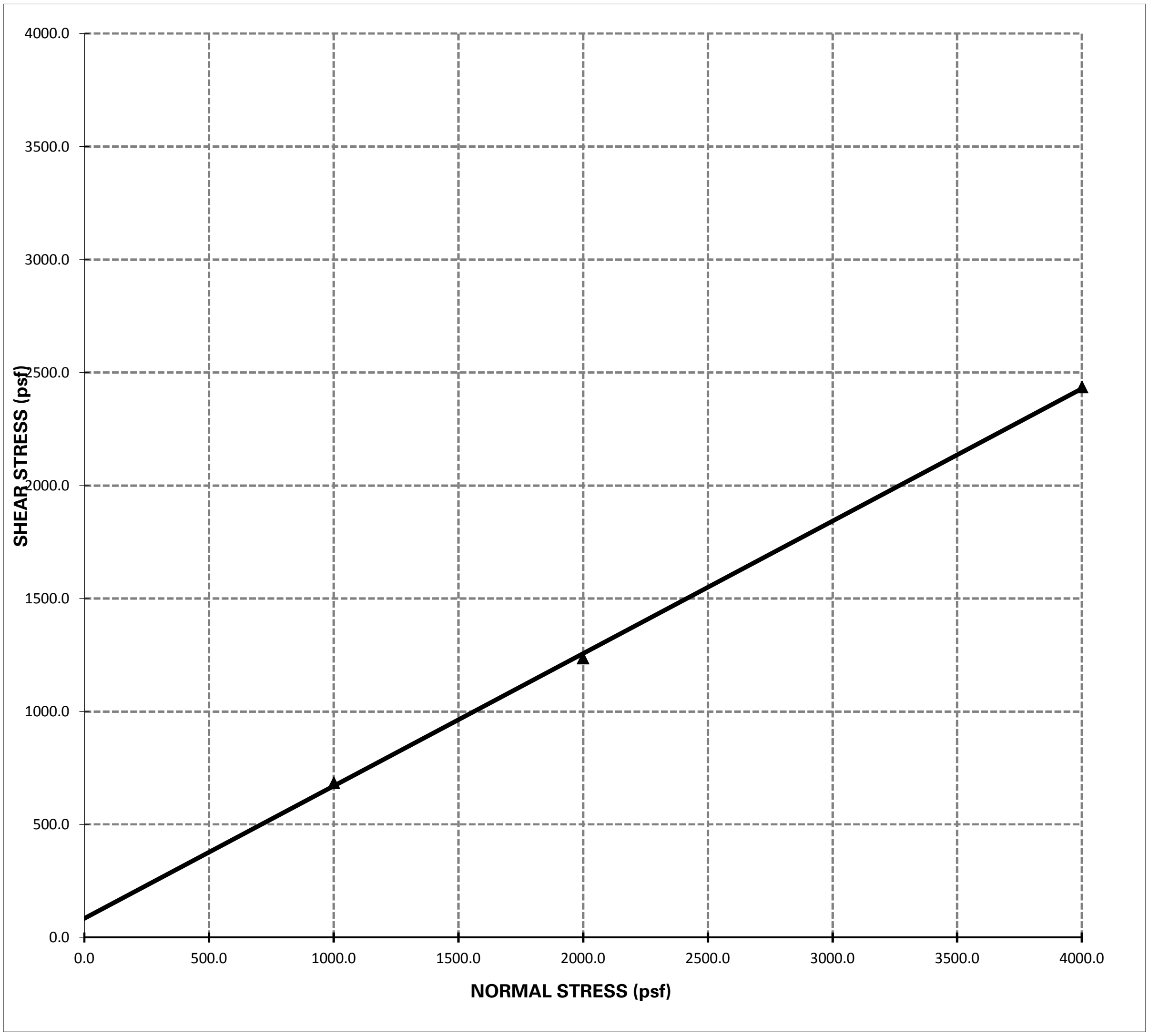
CONSOLIDATION REPORT

Sample:
I-3 @ 6

App. B

DIRECT SHEAR TEST

Project Name:	TR 31225, Perris	***Sample Location:***	B-1 @ 1 - 5
Project Number:	1889-CR	***Date Tested:***	6/6/2018



Shear Strength: Φ = **30.4 °** , **C = 84.00 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.

APPENDIX C

SEISMIC SETTLEMENT ANALYSIS

Tentative Tract No. 31225
Perris, Riverside County, California
Project No. 1889-CR

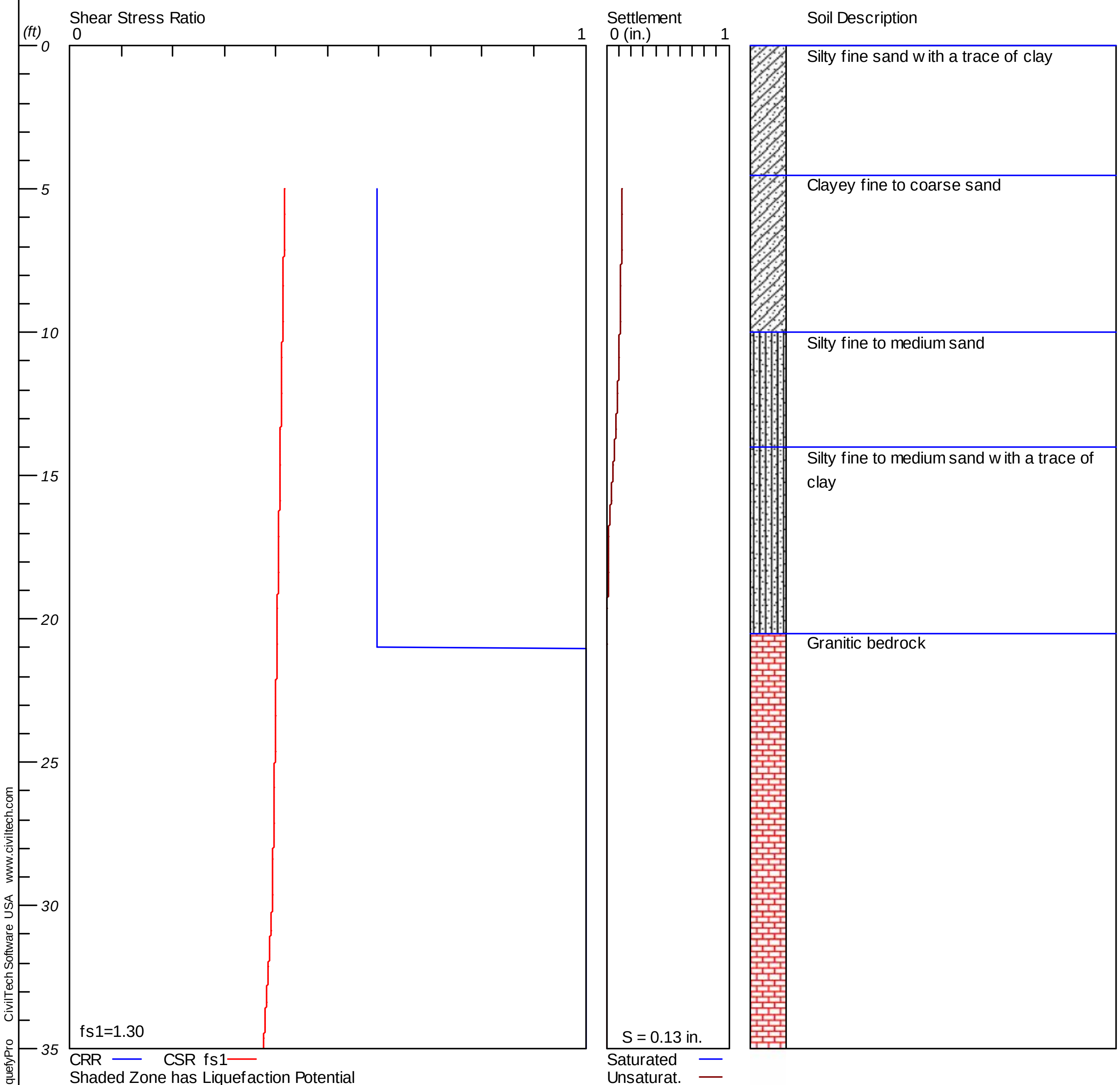
GEOTEK

LIQUEFACTION ANALYSIS

Pacific Legacy Encore

Hole No.=B-1 Water Depth=80 ft

Magnitude=7.0
Acceleration=0.50g



GeoTek, Inc.

Liquefaction Analysis Summary

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****

LIQUEFACTION ANALYSIS SUMMARY

****
****

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Input File Name: G:\Projects\1851 to 1900\1889CR Pacific Communities Builder Tract No. 31225 Perris\Liquefaction\Liquefaction Analysis.liq
Title: Pacific Legacy Encore
Subtitle:

Surface Elev.=
Hole No.=B-1
Depth of Hole= 35.00 ft
Water Table during Earthquake= 80.00 ft
Water Table during In-Situ Testing= 80.00 ft
Max. Acceleration= 0.5 g
Earthquake Magnitude= 7.00

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=35.00 ft
Water Table during Earthquake= 80.00 ft
Water Table during In-Situ Testing= 80.00 ft
Max. Acceleration=0.5 g
Earthquake Magnitude=7.00
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

Liquefaction Analysis Summary

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
5.00	18.00	143.00	46.20
11.50	24.00	145.00	21.40
16.50	25.00	137.00	21.10
21.00	100.00	121.00	NoLiq
26.00	120.00	121.00	NoLiq
31.00	300.00	135.00	NoLiq
35.00	600.00	135.00	NoLiq

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.13 in.
Total Settlement of Saturated and Unsaturated Sands=0.13 in.
Differential Settlement=0.064 to 0.084 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
5.00	0.60	0.42	5.00	0.00	0.13	0.13
6.00	0.60	0.42	5.00	0.00	0.12	0.12
7.00	0.60	0.42	5.00	0.00	0.12	0.12
8.00	0.60	0.41	5.00	0.00	0.11	0.11
9.00	0.60	0.41	5.00	0.00	0.11	0.11
10.00	0.60	0.41	5.00	0.00	0.10	0.10
11.00	0.60	0.41	5.00	0.00	0.10	0.10
12.00	0.60	0.41	5.00	0.00	0.09	0.09
13.00	0.60	0.41	5.00	0.00	0.08	0.08
14.00	0.60	0.41	5.00	0.00	0.06	0.06
15.00	0.60	0.41	5.00	0.00	0.05	0.05
16.00	0.60	0.41	5.00	0.00	0.03	0.03
17.00	0.60	0.41	5.00	0.00	0.02	0.02
18.00	0.60	0.40	5.00	0.00	0.01	0.01
19.00	0.60	0.40	5.00	0.00	0.01	0.01
20.00	0.60	0.40	5.00	0.00	0.00	0.00
21.00	0.60	0.40	5.00	0.00	0.00	0.00
22.00	2.00	0.40	5.00	0.00	0.00	0.00
23.00	2.00	0.40	5.00	0.00	0.00	0.00
24.00	2.00	0.40	5.00	0.00	0.00	0.00
25.00	2.00	0.40	5.00	0.00	0.00	0.00
26.00	2.00	0.40	5.00	0.00	0.00	0.00
27.00	2.00	0.40	5.00	0.00	0.00	0.00
28.00	2.00	0.39	5.00	0.00	0.00	0.00
29.00	2.00	0.39	5.00	0.00	0.00	0.00

30.00	2.00	0.39	5.00	0.00	0.00	0.00
31.00	2.00	0.39	5.00	0.00	0.00	0.00
32.00	2.00	0.39	5.00	0.00	0.00	0.00
33.00	2.00	0.38	5.00	0.00	0.00	0.00
34.00	2.00	0.38	5.00	0.00	0.00	0.00
35.00	2.00	0.38	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

---

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

APPENDIX D

INFILTRATION DATA

Tentative Tract No. 31225
Perris, Riverside County, California
Project No. 1889-CR

GEOTEK

GeoTek, Inc.

PERCOLATION TESTING

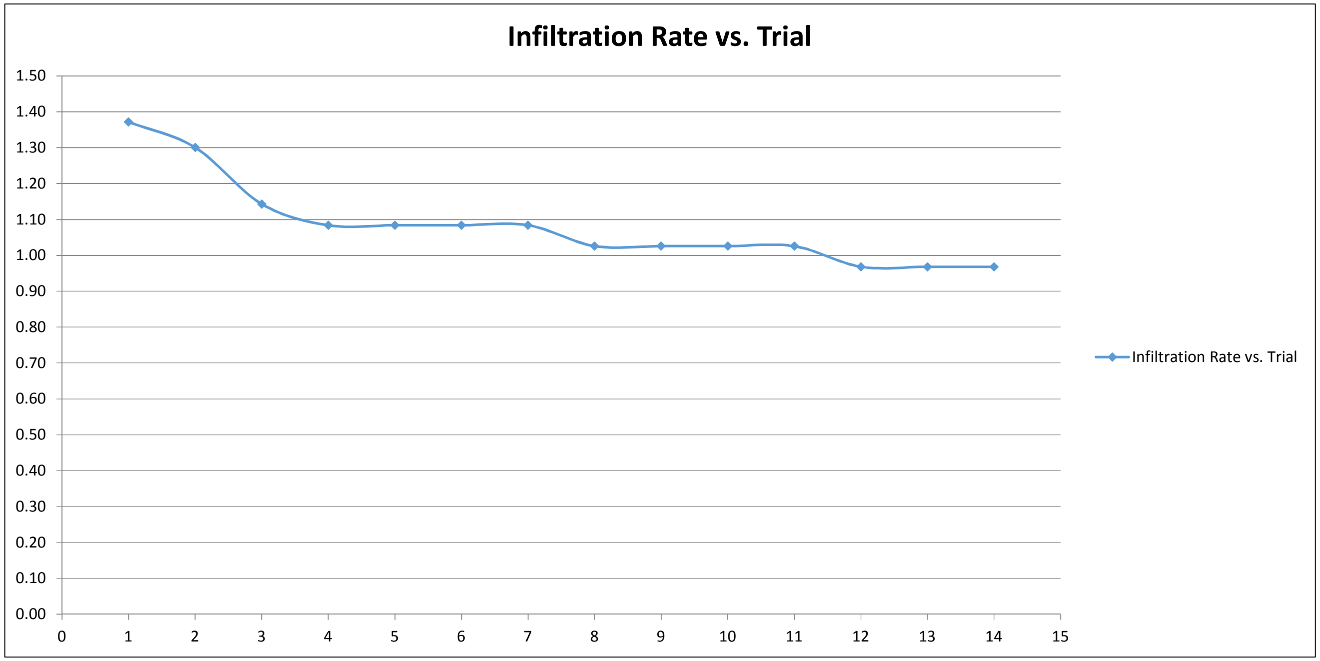
Shallow Percolation Test (<10 ft) | **Test No. I-1**

1889-CR

Depth of Hole (D_T) in.	60
Boring Radius, in.	4

Trial No.	Time Interval (ΔT) Min.	Initial Depth (D_0) in.	Final Depth (D_f) in.	Change In Level (ΔD) in.	Perc Rate (min/in)	Infiltration Rate (in/hr)
Non-Sandy Soil Trial 1	25	40.00	45.50	5.50	0.22	1.37
Non-Sandy Soil Trial 2	25	40.00	45.25	5.25	0.21	1.30
1	30	40.00	45.50	5.50	1.10	1.14
2	30	40.00	45.25	5.25	1.05	1.08
3	30	40.00	45.25	5.25	1.05	1.08
4	30	40.00	45.25	5.25	1.05	1.08
5	30	40.00	45.25	5.25	1.05	1.08
6	30	40.00	45.00	5.00	1.00	1.03
7	30	40.00	45.00	5.00	1.00	1.03
8	30	40.00	45.00	5.00	1.00	1.03
9	30	40.00	45.00	5.00	1.00	1.03
10	30	40.00	44.75	4.75	0.95	0.97
11	30	40.00	44.75	4.75	0.95	0.97
12	30	40.00	44.75	4.75	0.95	0.97

Initial Height (H_0)	Final Height (H_f)	Height Change (ΔH)	Height Average (H_{avg})
20	14.5	5.5	17.25
20	14.75	5.25	17.375
20	14.5	5.5	17.25
20	14.75	5.25	17.375
20	14.75	5.25	17.375
20	14.75	5.25	17.375
20	14.75	5.25	17.375
20	15	5	17.5
20	15	5	17.5
20	15	5	17.5
20	15	5	17.5
20	15.25	4.75	17.625
20	15.25	4.75	17.625
20	15.25	4.75	17.625



GeoTek, Inc.
PERCOLATION TESTING

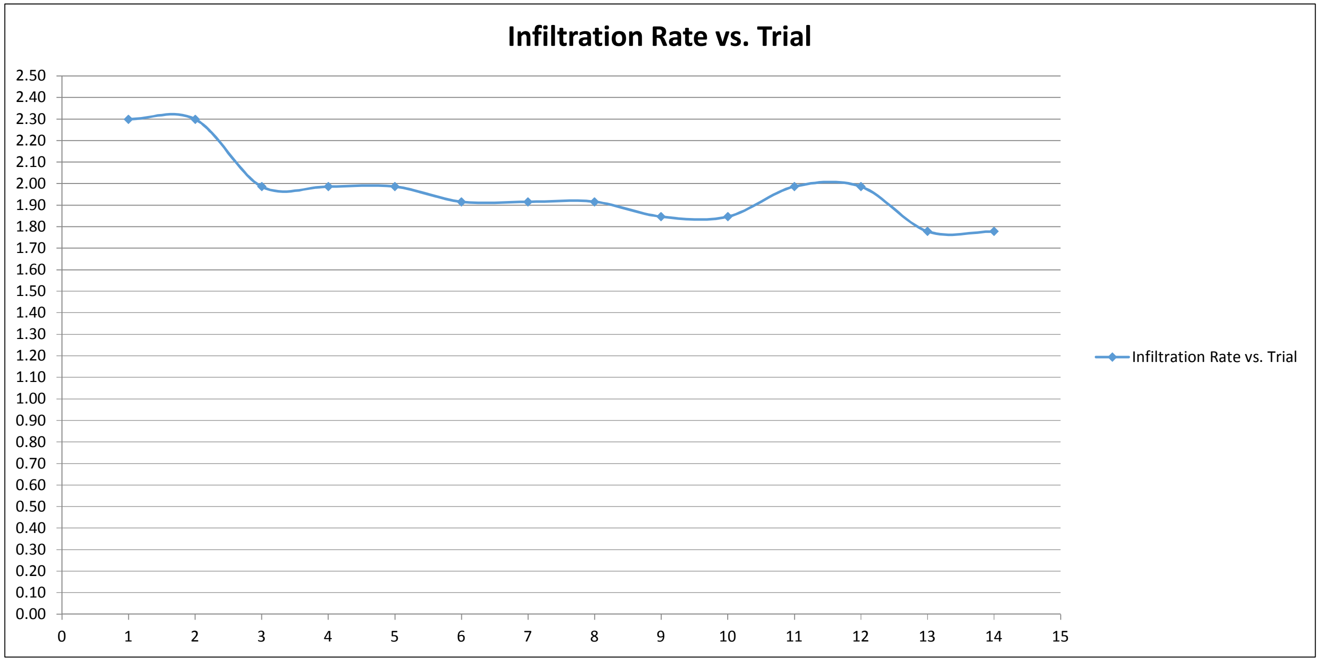
Shallow Percolation Test (<10 ft)

Test No. I-2
1889-CR

Depth of Hole (D_T) in.	84
Boring Radius, in.	4

Trial No.	Time Interval (ΔT) Min.	Initial Depth (D_0) in.	Final Depth (D_f) in.	Change In Level (ΔD) in.	Perc Rate (min/in)	Infiltration Rate (in/hr)
Non-Sandy Soil Trial 1	25	64.00	72.50	8.50	0.34	2.30
Non-Sandy Soil Trial 2	25	64.00	72.50	8.50	0.34	2.30
1	30	64.00	72.75	8.75	1.75	1.99
2	30	64.00	72.75	8.75	1.75	1.99
3	30	64.00	72.75	8.75	1.75	1.99
4	30	64.00	72.50	8.50	1.70	1.92
5	30	64.00	72.50	8.50	1.70	1.92
6	30	64.00	72.50	8.50	1.70	1.92
7	30	64.00	72.25	8.25	1.65	1.85
8	30	64.00	72.25	8.25	1.65	1.85
9	30	64.00	72.75	8.75	1.75	1.99
10	30	64.00	72.75	8.75	1.75	1.99
11	30	64.00	72.00	8.00	1.60	1.78
12	30	64.00	72.00	8.00	1.60	1.78

Initial Height (H_0)	Final Height (H_f)	Height Change (ΔH)	Height Average (H_{avg})
20	11.5	8.5	15.75
20	11.5	8.5	15.75
20	11.25	8.75	15.625
20	11.25	8.75	15.625
20	11.25	8.75	15.625
20	11.5	8.5	15.75
20	11.5	8.5	15.75
20	11.5	8.5	15.75
20	11.75	8.25	15.875
20	11.75	8.25	15.875
20	11.25	8.75	15.625
20	11.25	8.75	15.625
20	12.00	8	16
20	12.00	8	16



GeoTek, Inc.
PERCOLATION TESTING

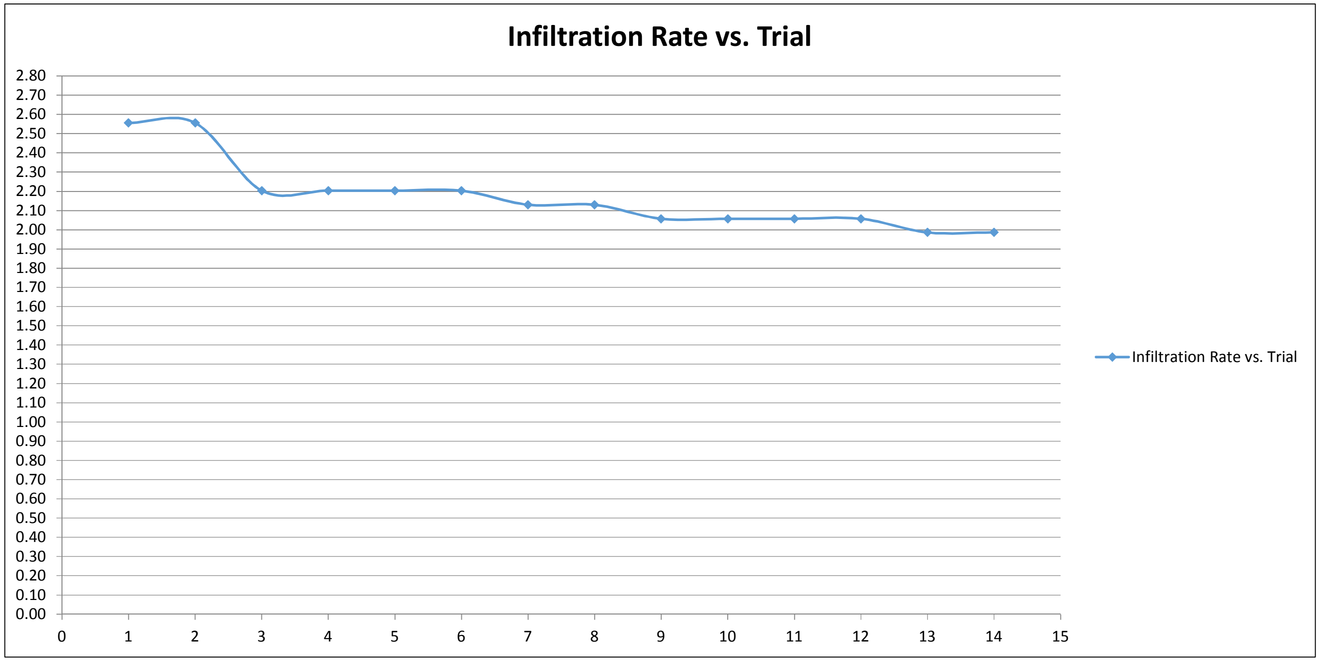
Shallow Percolation Test (<10 ft) — **Test No. I-3**

1889-CR

Depth of Hole (D_T) in.	120
Boring Radius, in.	4

Trial No.	Time Interval (ΔT) Min.	Initial Depth (D_0) in.	Final Depth (D_f) in.	Change In Level (ΔD) in.	Perc Rate (min/in)	Infiltration Rate (in/hr)
Non-Sandy Soil Trial 1	25	100.00	109.25	9.25	0.37	2.56
Non-Sandy Soil Trial 2	25	100.00	109.25	9.25	0.37	2.56
1	30	100.00	109.50	9.50	1.90	2.20
2	30	100.00	109.50	9.50	1.90	2.20
3	30	100.00	109.50	9.50	1.90	2.20
4	30	100.00	109.50	9.50	1.90	2.20
5	30	100.00	109.25	9.25	1.85	2.13
6	30	100.00	109.25	9.25	1.85	2.13
7	30	100.00	109.00	9.00	1.80	2.06
8	30	100.00	109.00	9.00	1.80	2.06
9	30	100.00	109.00	9.00	1.80	2.06
10	30	100.00	109.00	9.00	1.80	2.06
11	30	100.00	108.75	8.75	1.75	1.99
12	30	100.00	108.75	8.75	1.75	1.99

Initial Height (H_0)	Final Height (H_f)	Height Change (ΔH)	Height Average (H_{avg})
20	10.75	9.25	15.375
20	10.75	9.25	15.375
20	10.5	9.5	15.25
20	10.5	9.5	15.25
20	10.5	9.5	15.25
20	10.5	9.5	15.25
20	10.75	9.25	15.375
20	10.75	9.25	15.375
20	11	9	15.5
20	11	9	15.5
20	11	9	15.5
20	11	9	15.5
20	11.25	8.75	15.625
20	11.25	8.75	15.625



APPENDIX E

GENERAL GRADING GUIDELINES

Tentative Tract No. 31225
Perris, Riverside County, California
Project No. 1889-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.

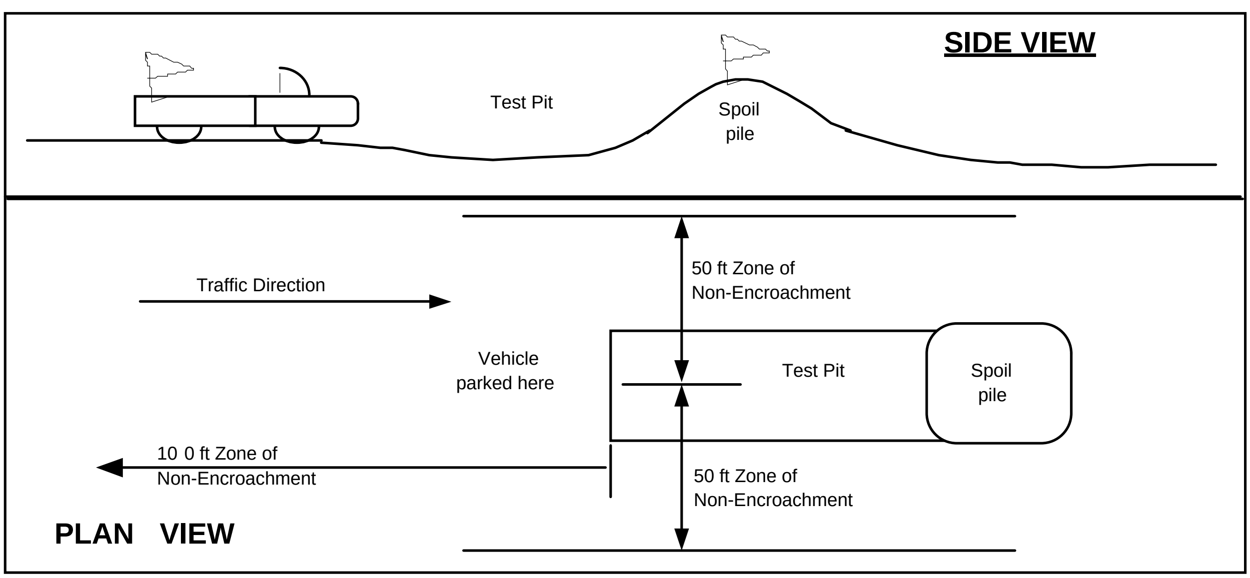
Test Pits Location, Orientation and Clearance

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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