Appendix F

Preliminary Geotechnical Evaluation

REVISED PRELIMINARY Geotechnical Evaluation

FOR

PACIFIC GPA/REZONE
Northwest Corner of Linda Vista Drive & Los Posas Road
SAN MARCOS, CALIFORNIA

PREPARED FOR

The Las Posas Owner LPV, LLC, a Delaware limited liability company
2235 Encinitas Boulevard, Suite 216
Encinitas, California 92024

PREPARED BY

GEOTEK, INC.
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PROJECT NO. 3649-SD **NOVEMBER 30, 2022**

GEOTEK

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January 4, 2022
Revised November 30, 2022
Project No. 3649-SD

The Las Posas Owner LPV, LLC,
a Delaware limited liability company
2235 Encinitas Boulevard, Suite 216
Encinitas, California 92024

Attention: Mr. Greg Waite

Subject: Revised Preliminary Geotechnical Evaluation
Pacific GPA/Rezone
Northwest Corner of Linda Vista Drive & Los Posas Road
San Marcos, California

Dear Mr. Waite:

We are pleased to provide herein the results of our preliminary geotechnical evaluation for the subject property located in the city of San Marcos, California. This report presents the results of our evaluation and provides preliminary geotechnical recommendations for future earthwork, foundation design, and construction. In our opinion, the property appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the future design and construction phases of property development. The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

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

- Research and review of available geologic and geotechnical data, and general information pertinent to the property.
- Excavation of four (4) hollow stem auger borings onsite and collection of bulk and relatively undisturbed soil samples for subsequent laboratory testing.
- Two percolation test borings and infiltration analyses.
- Laboratory testing of the soil samples collected during the field investigation.
- Review and evaluation of seismicity, and
- Compilation of this geotechnical report which presents our findings of pertinent geotechnical conditions and geotechnical recommendations for future property development.

2. PROPERTY DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 Property Description

The subject property is located at the northern corner of Linda Vista Drive and Los Posas Road in the City of San Marcos, San Diego County, California (see Figure 1). The property is bounded to the southwest by Linda Vista Drive, to the southeast by Los Posas Road, to the northeast by La Mirada Drive, and to the northwest by Pacific Street. Surface conditions generally consist of rolling unimproved earthen terrain, with native grasses and vegetation. The Site is relatively flat, ranging in elevation from approximately 527 feet above mean sea level (amsl) in the southeast potion of the project area to 551 feet in the northwest corner of the project Site.

2.2 Future Development / Project Description

The 33.2-acre project site is an infill site located in the western portion of the City of San Marcos (City), at the northwest corner of S. Las Posas Road and Linda Vista Drive, comprised of Assessor's Parcel Numbers 219-222-01, 219-222-02, 219-222-03, and 219-222-04. La Mirada Drive abuts the site's northern boundary, while South Pacific Street abuts the property's western boundary. The Grand Plaza shopping center is located directly across Las Posas Road to the east. Light industrial uses are adjacent to the site's northern, southern, and western boundary, and Bradley Park is located across from the site's southwestern corner. Single- and multi-family residential uses are located to the west and south of Bradley Park.

The project consists of 449 residential units, including a mix of apartments, rowhomes, villas, and affordable flats on approximately 15.09 acres of the 33.2-acre project site. Proposed residential units would include a mix of apartments within a five-story podium building, three-story rowhomes, three-story villas, and affordable flats within a four-story building. The project includes a total of 927 parking spaces and 134,985 square feet of common open space area. 68 of the 449 total units (15% of the total) would be designated as deed-restricted affordable units (alternatively, the project reserves the option to contribute to the affordable housing fund by paying the in-lieu fee). The proposed project also includes landscaping, bio-retention areas, and circulation improvements. The remaining approximately 17.94 acres of the 33.2-acre project site would be preserved and restored as open space and habitat area. The proposed project would have a density of approximately 13.5 dwelling units per acre, including the open space and habitat area.

The project proposes a General Plan Amendment, Rezone, Specific Plan, Tentative Map, and Multi-Family Site Development Plan. The General Plan Amendment and Rezone would change the General Plan designation and Zoning from Industrial (I) to Specific Plan Area (SPA). The Specific Plan has been prepared with the intent to provide a comprehensive plan to ensure the efficient development of a new residential community. The Specific Plan serves as both a policy document and a regulatory document for the systematic implementation of the policies and goals of the General Plan. The Tentative Map presents specific lot configurations for the site. The Multi-Family Site Development Plan will configure the site for multi family dwelling units, street configuration, infrastructure, recreational open space, and private open space.

As part of the project, additional pedestrian connectivity would be provided along three of the adjacent street frontages. The project would provide a 6-foot sidewalk and Class II buffered bike lane along the project's frontage on Pacific Street; the project would provide a 12-foot urban trail (shared use path) along the project's frontage on Linda Vista Drive; and the project would also provide a 12-foot urban trail (shared use path) along the project's frontage on La Mirada Drive. In addition to the proposed sidewalk and trail connections, the project would add a bus stop and shelter with a bus turnout along South Las Posas Road adjacent to the development area and would install a 4-way traffic signal at the intersection of Linda Vista Drive and Pacific Street. Furthermore, the project would upsize approximately 1,458-feet of existing water pipe from 8-inches to 12-inches and would convert approximately 1,400-feet of existing overhead power lines to underground along La Mirada.

Structural loading information was not available at the time of this report submittal but should be provided to GeoTek once determined. For this report, we have assumed a maximum column load of 150 kips for the planned structures.

As planning progresses and additional or revised plans become available, they should be provided to GeoTek for review and comment. If plans vary significantly, additional geotechnical field exploration, laboratory testing and engineering analyses may be necessary to provide specific earthwork recommendations and geotechnical design parameters for actual development plans.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 Field Exploration

Our field study conducted on September 29, 2020 consisted of a reconnaissance, excavation of geotechnical hollow stem auger borings B-3 through B-6 to depths of about 19-½ feet below grade and 2 percolation test borings P-1 and P-2 to depths of about 3 feet below grade. The borings were drilled with a truck mounted rubber tire CME-75 drilling rig and included collection of bulk and relatively undisturbed driven soil samples for subsequent laboratory testing. Proposed borings B-1 and B-2 were not performed due to biological constraints and omitted from this report. Several variations of proposed boring locations were presented to California Fish and Wildlife (CFW), however, CFW only authorized the locations presented on Figure 2. A representative from our firm visually logged the borings and collected soil samples for laboratory analysis. Approximate locations of exploration locations are presented on the Boring Location Map, Figure 2. A description of material encountered in the borings is included in Appendix A.

3.2 Percolation and Infiltration Testing

Borings P-1 and P-2 were advanced to approximate depths of 3 feet below existing grade with an 8-inch diameter and converted to a dry well for percolation testing approximately 5-10 feet away from boring location B-6. Following completion of the boring excavations, percolation testing was performed by a representative from our firm in general conformance with the city of San

Marcos BMP Design Manual. The boreholes were presoaked over-night and the testing was performed the following day. Percolation testing was performed by adding potable water to the borings, recording the initial depth to water and allowing the water to percolate for 30 minutes and the depth to water was measured. Water was generally added to each boring following each reading increment. In general, the percolation testing was performed for approximately 6 hours to allow rates to stabilize. Results of the final percolation increment were used to calculate an infiltration rate in inches per hour via the Porchet method.

For design of shallow infiltration basins, converting percolation rates to infiltration rates via the Porchet method is generally acceptable and appropriate, as this method factors out the sidewall component of the percolation results and represents the bottom conditions of a shallow basin (infiltration). Therefore, the percolation data for borings P-1 and P-2 were converted via the Porchet method. This method is consistent with the guidelines referenced in the City of San Marcos BMP Design Manual. Results of our infiltration analysis without a factor of safety are presented in the follow table for each of the test areas.

Location	Depth (inches)	Infiltration Rate (inches per hour)*
P-1	35.75	0.07
P-2	34.75	0.07

* Rate was converted to an infiltration rate via the Porchet method

Copies of infiltration conversion sheets are included in Appendix A.

The material exposed along the boring sidewalls and at the bottom of P-1 and P-2 were native soils. The tests performed and reported are indicative of native soils. At the time of investigation, groundwater was not encountered in the vicinity.

Over the lifetime of the storm water disposal areas, the percolation rates may be affected by silt build up and biological activities, as well as local variations in soil conditions. An appropriate factor of safety used to compute the design percolation rate should be considered at the discretion of the design engineer and acceptance of the plan reviewer.

3.2 Laboratory Testing

Laboratory testing was performed on bulk and relatively undisturbed soil samples collected during the field explorations. The purpose of the laboratory testing was to evaluate their physical and chemical properties for use in engineering design and analysis. Results of the laboratory testing program, along with a brief description and relevant information regarding testing procedures, are included in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 Regional Setting

The subject property is located in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. Basically, it extends roughly 975 miles from the north and northeasterly adjacent 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 zones trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province. The Newport-Inglewood-Rose Canyon Fault zone meanders the southwest margin of the province, but can be more appropriately defined by the Pacific Ocean. No faults are shown in the immediate vicinity on the map reviewed for the area.

4.2 EARTH MATERIALS

A brief description of the earth materials encountered during our subsurface exploration is presented in the following sections. Based on our field observations and review of published geologic maps the subject property is locally underlain by recent alluvium and Santiago Formation bedrock.

4.2.1 Undocumented Fill (Not Mapped)

Undocumented fill soils were observed as sporadic end-dump piles from illegal dumping. These soils are not considered suitable for support of structural improvements but may be re-used as engineered fill if properly processed and placed.

4.2.2 Alluvium (Map Symbol Qal)

The most recent regional geologic map reviewed showed the geology (Kennedy, 2007) for the eastern portion of the area along Los Posas Road to be alluvial deposits, however, based on our evaluation, alluvium appears to be limited to a smaller extent along a natural drainage swale along Los Posas Road. As encountered in boring B-6, the alluvium generally consisted of silty fine sands.

4.2.3 Tertiary Santiago Formation (Map Symbol Tsa)

The most recently dated regional geologic map showing the overall geology (Kennedy, 2007), indicates Santiago Formation sedimentary bedrock at the surface on the western majority of the property; however, based on our evaluation the Santiago Formation appears to be near the surface across most of the property. As encountered in the borings, Santiago Formation was observed as a dark brown to black clay over sandstone.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during our visit. If encountered during earthwork construction, surface water is likely the result of precipitation. Overall, area drainage is in a southeast direction. Provisions for surface drainage will need to be accounted for by the future project civil engineer.

4.3.2 Groundwater

A static groundwater table was not encountered during drilling operations. Excavations B-5 and B-6 appears to have encountered perched water at a depth of approximately 19 and 15 feet (respectively) and cuttings in B-4 at a depth of 17 feet suggest a perch groundwater. Based on the anticipated depth of removals, groundwater is not anticipated to be a factor in future development. Localized perched groundwater may be present, but is also not anticipated to be a factor in future development with the exception that seasonal water levels are likely to impact storm water management.

4.4 EARTHQUAKE HAZARDS

4.4.1 Surface Fault Rupture

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The property is in a seismically active region. No active or potentially active fault is known to exist at this property nor is the property situated within an *"Alquist-Priolo"* Earthquake Fault Zone or a Special Studies Zone (Bryant and Hart, 2007). No faults transecting the property were identified on the readily available geologic maps reviewed. The nearest known active fault is the Newport Inglewood-Rose Canyon fault located about 11 miles to the southwest of the property.

4.4.2 Liquefaction/Seismic 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, consolidation and 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.

The liquefaction potential and seismic settlement potential is considered negligible provided remedial grading recommendations presented herein are completed and due to the general dense to very dense nature of underlying shallow bedrock, as well as planned fill placement.

4.4.3 Other Seismic Hazards

Due to the relatively flat nature of the property, the potential for landslides and rockfall is considered negligible. The potential for secondary seismic hazards such as seiche and tsunami is remote due to property elevation and distance from an open body of water.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Future development of the property appears feasible from a geotechnical viewpoint provided that the following recommendations are incorporated in the design and construction phases of the development. The following sections present general recommendations for currently anticipated future development. Due to the preliminary nature of this report, supplemental geotechnical evaluations of the property are anticipated at future dates once more detailed development plans are available. Those supplemental geotechnical recommendations will supersede the preliminary recommendations provided in this report.

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the city of San Marcos, the 2019 (or current) California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix C outline general procedures and do not anticipate all specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix C.

5.2.2 Property Clearing and Preparation

Property preparation should start with removal of deleterious materials and vegetation. These materials should be disposed of properly off property. Any existing underground improvements, utilities and trench backfill should also be removed or be further evaluated as part of future development operations.

5.2.3 Remedial Grading

Prior to placement of fill materials and in all structural areas the upper variable, potentially compressible materials should be removed. Removals should include all existing fill, colluvium/weathered bedrock materials and we anticipate that the removals will extend approximately 5 feet below existing grade and should also extend to at least 2 feet below the bottom of proposed footings. The lateral extent of removals beyond the outside edge of all settlement sensitive structures/foundations should be equivalent to that vertically removed or five feet, whichever is greater. Depending on actual field conditions encountered during grading, locally deeper and/or shallower areas of removal may be necessary.

In pavement areas, removals should extend at least 2 feet below existing grade or one foot below finished subgrade whichever is lowest.

The bottom of all removals should be scarified to a minimum depth of six (6) inches, brought to slightly above optimum moisture content, and then compacted to at least 90% of the soil's maximum dry density, per ASTM D1557 prior to fill placement. The remedial excavation bottoms should be observed by a GeoTek representative prior to scarification. The resultant voids from remedial grading/overexcavation should be filled with materials placed in general accordance with Section 5.2.4 Engineered Fill of this report.

5.2.4 Engineered Fill

Onsite materials are generally considered suitable for reuse as engineered fill provided they are free from vegetation, roots, debris, and rock/concrete or hard lumps greater than six (6) inches in maximum dimension. The earthwork contractor should have the proposed excavated materials to be used as engineered fill at this property approved by the soils engineer prior to placement.

Engineered fill materials should be moisture conditioned to at or above optimum moisture content and compacted in horizontal lifts not exceeding 8 inch in loose thickness to a minimum relative compaction of 90% as determined in accordance with laboratory test procedure ASTM D 1557.

If fill is being placed on slopes steeper than 5:1 (h:v), the fill should be properly benched into the existing slopes and a sufficient size keyway shall be constructed in accordance with grading guidelines presented in Appendix C.

5.2.5 Excavation Characteristics

Excavations can generally be accomplished with heavy-duty earthmoving or excavating equipment in good operating condition. Excavations in sedimentary bedrock may require special excavation equipment and/or techniques.

5.2.6 Shrinkage and Bulking

Several factors will impact earthwork balancing, including bedrock bulking, undocumented fill and colluvium shrinkage, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage and bulking are largely dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage/bulking factor ranging of plus or minus 5 percent may be considered for surficial undocumented fill materials and alluvium. A bulking factor of 5 to 10 percent may be considered for the upper 3 feet of Santiago Formation bedrock requiring removal and re-compaction. Subsidence should not be a factor, if removals are completed as recommended.

5.2.7 Trench Excavations and Backfill

Temporary excavations within should be stable at 1:1 inclinations for short durations during construction, and where cuts do not exceed 10 feet in height. Temporary cuts to a maximum height of 4 feet can be excavated vertically.

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

Utility trench backfill should be compacted to at least 90% relative compaction of the maximum dry density as determined per ASTM D 1557. Under-slab trenches should also be compacted to specifications.

Onsite materials may not be suitable for use as bedding material, but should be suitable as backfill provided particles larger than 6± inches are removed.

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

5.3 DESIGN RECOMMENDATIONS

5.3.1 Stormwater Infiltration

Many factors control infiltration of surface waters into the subsurface, such as consistency of native soils and bedrock, geologic structure, fill consistency, material density differences, and existing groundwater conditions. In consideration of the shallow bedrock and anticipated shallow groundwater, infiltration of stormwater into the subsurface is not recommended from a geotechnical perspective. Stormwater quality control basins should be constructed with an impermeable liner along the sides and bottom.

5.3.2 Foundation Design Criteria

Preliminary foundation design criteria, in general conformance with the 2019 CBC, are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer. The preliminary recommendations presented below are based on an assumed maximum column load of 150 kips for the planned buildings. Once actual structural loads and grading concepts are known, supplemental recommendations may be warranted which may require additional test borings and laboratory testing.

Based on our visual classification of materials encountered and as verified by laboratory testing, soils near subgrade are anticipate to exhibit a “medium” (51 ≤ EI ≤ 90) potential for expansion per ASTM D4829. Materials with “low” (21 ≤ EI ≤ 50) expansiveness may also be encountered at depth and could be placed as engineered fill soils depending on grading logistics. Additional laboratory testing should be performed at the time of supplemental geotechnical evaluations and upon completion of grading activities to verify the expansion potential and plasticity index of the subgrade soils.

The following criteria for design of foundations are preliminary. Additional laboratory testing of the samples obtained during grading should be performed and final recommendations should be based on as-graded soil conditions.

DESIGN PARAMETES FOR CONVENTIONALL REINFORCED SHALLOW FOUNDATIONS		
Design Parameter	"Low" Expansion Potential (21≤EI≤50)	"Medium" Expansion Potential (51≤EI≤90)
Foundation Embedment Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent finished grade)	One- and Two Story – 12 Three-Story – 18 Four and Five-Story – 24	One- and Two Story – 18 Three -Story- 24 Four and Five-Story – 30
Minimum Foundation Width (Inches)*	One- and Two Story – 12 Three-Story- 15 Four and Five-Story – 18	One- and Two Story – 12 Three-Story – 18 Four and Five-Story – 24
Minimum Slab Thickness (actual)	4 inches	4 inches
Minimum Slab Reinforcing	No. 3 rebar 24" on-center, each way, placed in the middle one-third of the slab thickness	No. 3 rebar 18" on-center, each way, placed in the middle one-third of the slab thickness
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one top and one bottom	Four No. 4 reinforcing bars, two top and two bottom
Effective Plasticity Index**	<15	16< PI<30
Presaturation of Subgrade Soil (percent of optimum moisture content)	Minimum 110% to a depth of 12 inches	Minimum 120% to a depth of 18 inches

*Code minimums per Table 1809.7 of the 2019 CBC should be complied with.

**Effective Plasticity Index should be verified at the completion of the rough grading.

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

The following recommendations should be implemented into the design:

- Preliminarily, an allowable bearing capacity of 2,500 pounds per square foot (psf) may be considered for design of continuous and perimeter footings that meet the depth and width requirements in the table above. This value may be increased by 400 psf for each additional 12 inches in depth and 100 psf for each additional 12 inches in width to a maximum value of 3,000 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads). It may be possible to utilize a higher allowable soil bearing pressure for foundations directly supported by bedrock. The determination of an allowable soil bearing pressure on bedrock should be determined once foundation loads and elevations are known.

- Based on our experience in the area, structural foundations may be designed in accordance with 2019 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 30 feet. Seismically induced settlement is considered to be minimal.

- The passive earth pressure may preliminarily be computed as an equivalent fluid having a density of 220 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

- A grade beam, a minimum of 12 inches wide and 12 inches deep, should be utilized across large entrances, however, the base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

- We recommend that control joints be placed in two directions spaced the numeric equivalent roughly 24 times the thickness of the slab in inches (e.g. a 4 inch slab would have control joints at 96 inch [8 feet] centers). These joints are a widely accepted means to control cracks and should be reviewed by the structural engineer.

5.3.3 Underslab Moisture Membrane

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 2019 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2019 CBC Section 1907.1

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 atop 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 that 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, it is GeoTek's opinion that a minimum 10 mil membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and 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 does not practice in the field of moisture vapor transmission evaluation/migration, since that practice is not a geotechnical discipline. Therefore, we recommend that a qualified person, such as the flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the building be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person (or persons) 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.

5.3.4 Miscellaneous Foundation Recommendations

- To reduce 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.

- Spoils 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.

5.3.5 Foundation Set Backs

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom 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 7 feet and need not exceed 40 feet.

- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall stem. This applies to the existing retaining walls along the perimeter, if they are to remain.

- The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 projection upward from the bottom of the nearest excavation.

5.3.6 Seismic Design Parameters

The property is located at approximately 33.1394 Latitude and -117.1961 Longitude. Spectral accelerations (Ss and S1), for 0.2 and 1.0 second periods for a risk targeted two (2) percent probability of exceedance in 50 years (MCER) were determined using the web interface provided by SEAOC/OSHPD (https://seismicmaps.org) to access the USGS Seismic Design Parameters. We have selected a Site Class "C" based on the apparent density of the old alluvial deposits. The results, based on ASCE 7-16 and the 2019 CBC, are presented in the following table.

SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	0.897g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.331g
Coefficient for Site Class "C", F_a	1.2
Coefficient for Site Class "C", F_v	1.5
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 0.2 Second, S_{MS}	1.077g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 1.0 Second, S_{M1}	0.496g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.718g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.331g
Modified Peak Ground Acceleration (PGA_M)	0.465g
Seismic Design Category	D

5.3.7 Soil Sulfate Content

Nearby sulfate content test results indicate water soluble sulfate is less than 0.1 percent by weight, which is considered "S0" as per Table 19.3.1.1 of ACI 318-14, as such no special recommendations for concrete are included herein.

5.3.8 Preliminary Pavement Design

Traffic indices have not been provided to our firm during this stage of planning. In addition, remedial graded has not been completed to evaluate specific street subgrade conditions. Therefore, we have referenced the minimum structural sections based on the City of San Marcos' Urban Street Design for residential streets (San Marcos, 2020) and are presented below.

PRELIMINARY ASPHALT PAVEMENT STRUCTURAL SECTION		
Design Criteria+	Asphaltic Concrete (AC) Thickness (inches)	Aggregate Base (AB) Thickness (inches)
Residential	3.0	6.0

As noted in the Urban Street Design document, actual structural pavement design is to be determined by the geotechnical engineer's testing (R-value) of the subgrade. Thus, the actual R-value of subgrade soils can only be determined at the completion of grading for street subgrade and the above values are subject to change based laboratory testing of the as-graded soils near subgrade elevations. The final pavement design may be thicker than the City of San Marcos minimum thickness presented above.

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

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

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Preliminary plans are not yet available, if retaining walls are added at a later date, the recommendations presented herein may apply to typical masonry or concrete vertical retaining walls to a maximum height of 10 feet. Additional review and recommendations should be requested for higher walls.

Retaining wall foundations embedded a minimum of 18 inches into engineered fill or dense formational materials should be designed using an allowable bearing capacity of 2500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads). The passive earth pressure may be computed as an equivalent fluid having a density of 220 psf per foot of depth, to a maximum earth pressure of 3,000 psf. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in the table below for specific slope gradients of retained materials.

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Select Backfill*
Level	35
2:1	55

*Select backfill should consist of native or imported sand other approved materials with an SE>30 and an EI≤20 and should be provided throughout the active zone.

The above equivalent fluid weights do not include other superimposed loading conditions such as expansive soil, vehicular traffic, structures, seismic conditions or adverse geologic conditions.

Additional lateral forces can be induced on retaining walls during an earthquake. For level backfill and a Site Class "C", a supplemental earthquake-induced equivalent fluid pressure of 14.4 pcf should be considered, where required. This force can be assumed to act as a typical fluid pressure, resulting in a triangular pressure distribution. The 2019 CBC only requires the additional earthquake induced lateral force be considered on retaining walls in excess of six (6) feet in height; however, the additional force may be applied in design of lesser walls at the discretion of the wall designer.

5.4.2 Restrained Retaining Walls

Any retaining wall that will be restrained prior to placing backfill or walls that have male or reentrant corners should be designed for at-rest soil conditions using an equivalent fluid pressure of 60 pcf (select backfill), plus any applicable surcharge loading. For areas having male or reentrant corners, the restrained wall design should extend a minimum distance equal to twice the height of the wall laterally from the corner, or as otherwise determined by the structural engineer.

5.4.3 Wall Backfill and Drainage

Wall backfill should include a minimum one (1) foot wide section of ¾ to 1-inch clean crushed rock (or approved equivalent). The rock should be placed immediately adjacent to the back of wall and extend up from the backdrain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. If the walls are designed using the "select" backfill design parameters, then the "select" materials shall be 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. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs.

The backfill materials should be placed in lifts no greater than 8-inches in thickness and compacted to a minimum of 90% of the maximum dry density as determined in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of walls should be performed where moisture migration through the wall 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 (1) cubic foot per lineal foot of 3/8 to one (1) inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and be directed (via a solid outlet pipe) to an appropriate disposal area.

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

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

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be warranted and advisable. We could discuss these issues, if desired, when plans are made available.

5.5.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive 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. Property drainage should conform to Section 1804.4 of the 2019 CBC. Roof gutters and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter pan or directly to the storm drain system. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

5.6 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that future grading, specifications, retaining wall/shoring plans and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. Additional recommendations may be necessary based on these reviews. We also recommend that GeoTek representatives be present during future grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek's representative perform at least the following duties:

- Observe 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 when necessary.
- Observe the fill for uniformity during placement including utility trenches.
- Observe and test the fill for field density and relative compaction.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

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

6. LIMITATIONS

The scope of our evaluation is limited to the area explored that is shown on the Boring Location Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of proposed construction as indicated to us by the client. The scope is based on our understanding of the future development and the client's needs, our proposal (Proposal No. P-0300620-SD) dated March 25, 2020 and geotechnical engineering standards normally used on similar property in this region.

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

Since our recommendations are based on the property 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 is expressed or implied. Standards of practice are subject to change with time.

7. SELECTED REFERENCES

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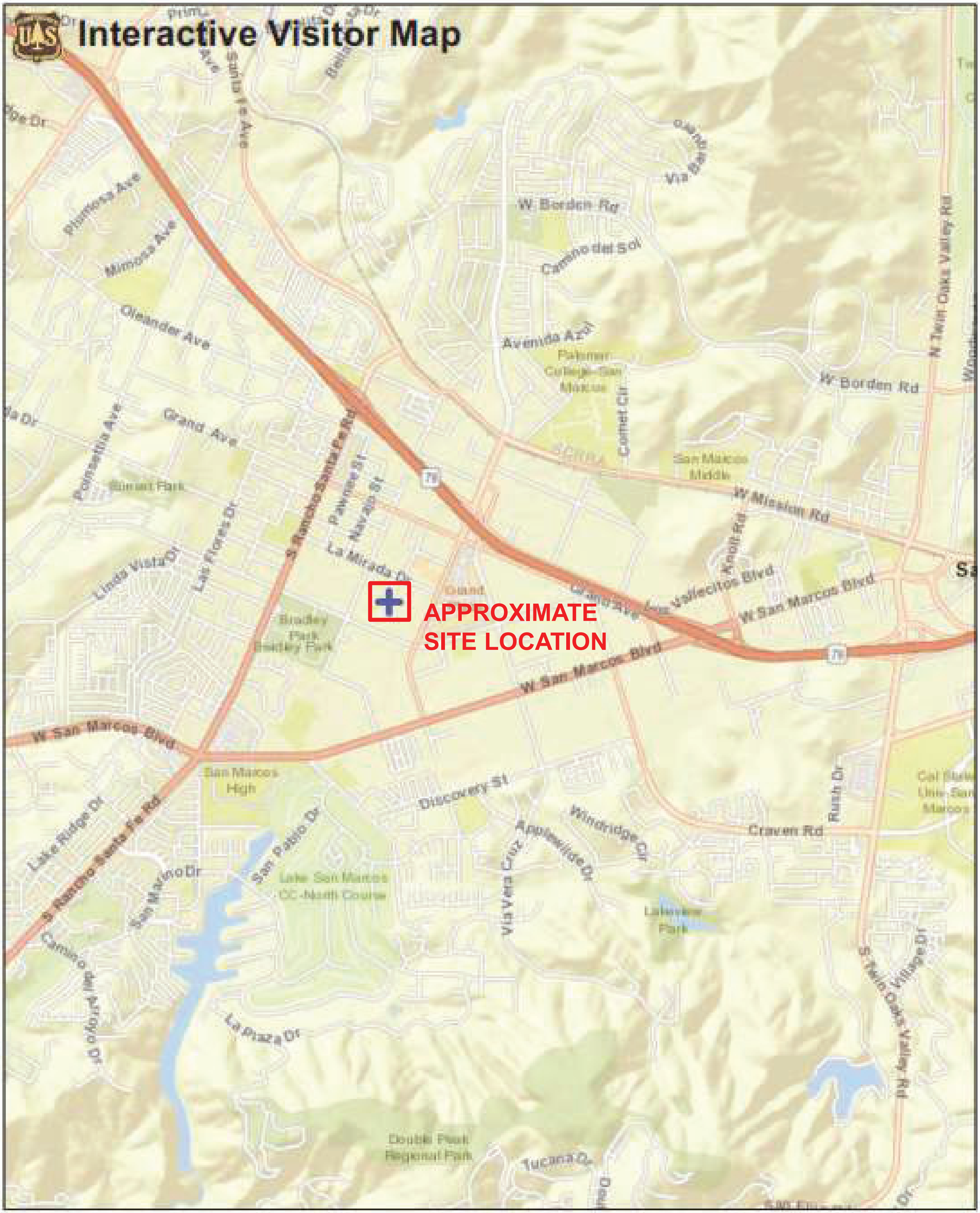


Interactive Visitor Map from US Forestry Service, 2021

NOT TO SCALE

GEOTEK
1384 Poinsettia Avenue, Suite A
Vista, California 92081-8505

N

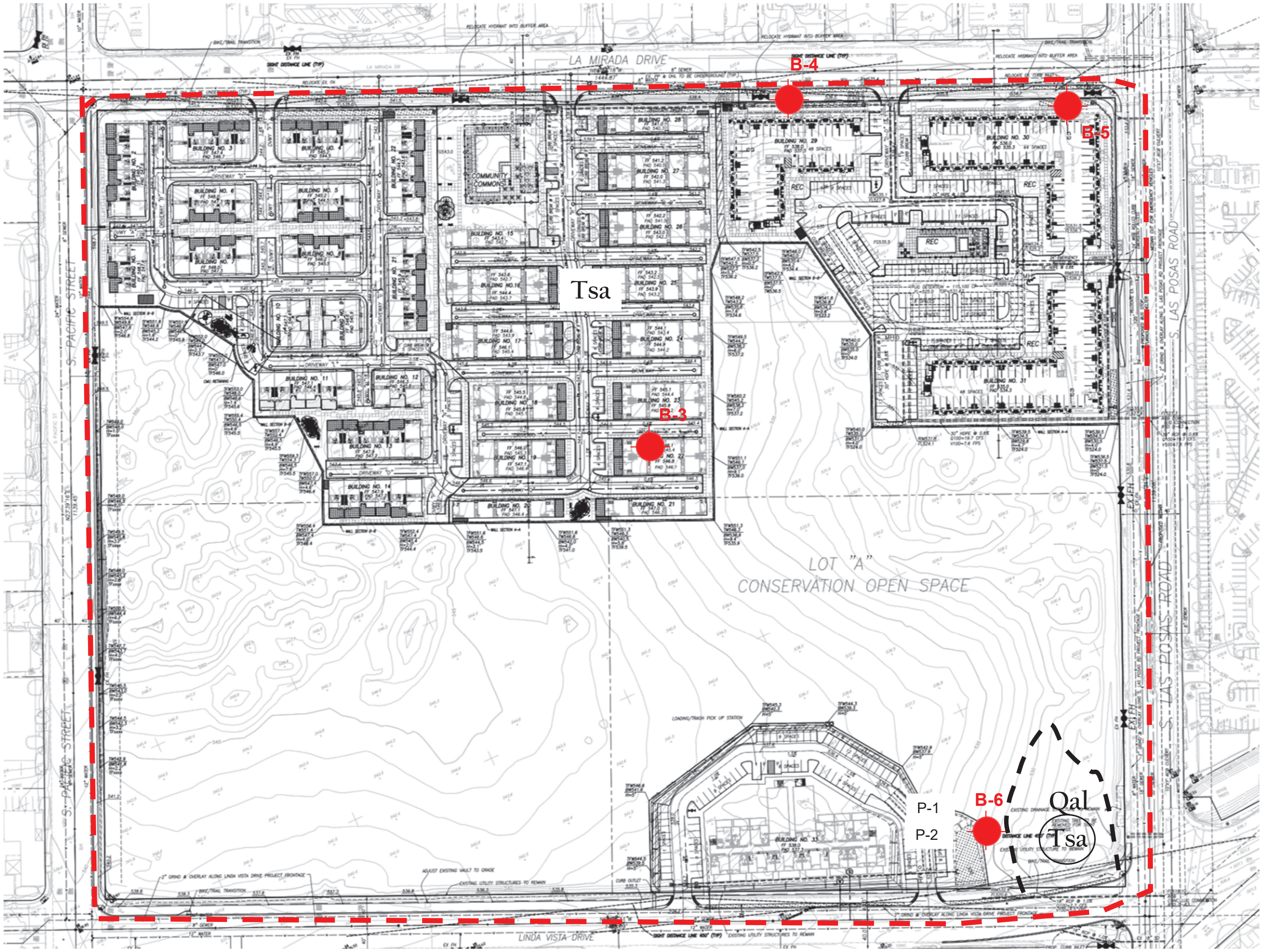
Site Location Map

Pacific GPA/Rezone Project
NW Corner of Linda Vista & Las Posas Rd.
San Marcos, California

PN: 3649-SD

November 2022

Figure 1



LEGEND

B-6 APPROXIMATE LOCATION OF BORING TEST LOCATIONS

LIMITS OF PROPERTY

Qal ALLUVIUM

Tsa SANTIAGO FORMATION, CIRCLED WHERE BURIED

The Los Posas Owner LPV, LLC
Pacific GPA/Rezone
NW Corner of Linda Vista Dr. & Las Posas
San Marcos, California

PN: 3649-SD November 2022

Figure 2

Boring Location Plan

GEOTEK
1384 Poinsettia Avenue, Suite A
Vista, California 92081

APPENDIX A

EXPLORATORY BORING LOGS

AND

INFILTRATION WORKSHEETS

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 –EXCAVATION LOG LEGEND

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

(Additional denotations and symbols are provided on the logs)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	The Las Posas Owner LPV, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Pacific San Marcos	**DRILL METHOD:**	CME-75 HAS	**OPERATOR:**	
PROJECT NO.:	3649-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	See Boring Locaiton Map	**ELEVATION:**		**DATE:**	9/29/2020

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
	Large Bulk		BB-1		**Dirt Road** Weathered Santiago/Colluvium **CLAY, dark brown-black, dry, trace sand**			
					Tertiary Santiago Formation Clayey medium SANDSTONE, pale yellow, moist, very dense, transition in sample			
5	Ring	13 22 74	R-1			9.4	121.9	
10	Large Bulk	10 21 21	S-1		Clayey medium SANDSTONE, pale yellow, moist, very dense			
15	Ring	11 50/6"	R-2		Clayey medium SANDSTONE, pale yellow, moist, very dense	11.4	124.3	
	Large Bulk	12 20 24	S-2		Clayey fine SANDSTONE, orange brown and play yellow, moist, very dense, light olive gray claystone in shoe, moist, hard, bedded oxidation			
20					**HOLE TERMINATED AT 19.5 FEET** No groundwater encountered 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 | CO = Consolidation test | MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	The Las Posas Owner LPV, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Pacific San Marcos	**DRILL METHOD:**	8" HSA 3.75" ID	**OPERATOR:**	
PROJECT NO.:	3649-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	See Boring Locaiton Map	**ELEVATION:**		**DATE:**	9/29/2020

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
0					**Vegatation over**			
	Large Bulk		BB-1		**Weathered Santiago/Colluvium** Silty fine SAND, light brown, dry, loose			
5	Ring	15 14 18	R-1		**Tertiary Santiago Formation** Clayed medium SANDSTONE, pale yellow white, very mosit, medium dense, mottled orange oxidation	8.9	124.2	
10	Large Bulk	12 21 34	S-1		Clayey medium SANDSTONE, light gray, very moist, very dense, ~15% fine			
15	Ring	13 50/6"	R-2		Clayey medium SAND, light gray, very moist, very dense, ~15% fine	12.5	119.1	
					Cuttings are very wet, dark orange brown, 30-40% fine			
	Large Bulk	12 26 33			Clayey fine SANDSTONE, mottled orange, brown and pale white, very moist, very dense			
20					**HOLE TERMINATED AT 19.5 FEET** Some groundwater encountered at 17 feet 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	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	The Las Posas Owner LPV, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	BRM
PROJECT NAME:	Pacific San Marcos	**DRILL METHOD:**	8" HSA 3.75" ID	**OPERATOR:**	
PROJECT NO.:	3649-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME 75
LOCATION:	See Boring Locaiton Map	**ELEVATION:**		**DATE:**	9/29/2020

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Dirt Road Patch, Slight Vegatation			
					Tertiary Santiago Formation			
	Large Bulk		BB-1		Clayey SAND			
5	Ring	12 27 54	R-1		Clayey SANDSTONE, tan-yellow, moist	7.8	129.4	
10	Large Bulk		S-1		Silty fine SANDSTONE, oxidation layers tan, damp			
15	Ring	10 17 25	R-2		Silty fine SANDSTONE, orange brown, moist, very dense CLAYSTONE, dark brown, moist, very stiff, fne cut sandy sandstone, dark brown, moist	11.6	109.6	
	Ring	19 27 41	S-2	Water Table	Silty medium-coarse SANDSTONE, light gray, wet, very dense			
20					**HOLE TERMINATED AT 19.5 FEET** Groundwater encountered at 18 feet 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	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	The Las Posas Owner LPV, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Pacific San Marcos	**DRILL METHOD:**	8" HSA 3.75" ID	**OPERATOR:**	
PROJECT NO.:	3649-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	See Boring Locaiton Map	**ELEVATION:**		**DATE:**	9/29/2020

BORING NO.: B-6

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Grassland Over Alluvium (Qal) Silty fine SAND, light brown, dry, loose			
					Tertiary Santiago Formation			
5	Ring	16 26 40	R-1		Silty coarse SANDSTONE, interbedded light gray and orange brown, moist, very dense, ~15% fines	8	120.6	
10	Large Bulk	4 7 12	S-2		CLAYSTONE, medium gray, fractured, well healed with orange oxidized fine sand			
15	Ring	11 27 28	R-2		Medium-coarse SANDSTONE, richly oxidized orange brown, wet, very dense, laminiated silty and clayey sandstone, ~20% fine, moist	10.7	128.8	
	Large Bulk	7 14 29	S-2		CLAYSTONE, dark gray, moist, hard Slity fine SANDSTONE, dark gray, moist, 40% fine, decomposed organic matter			
20					**HOLE TERMINATED AT 19.5 FEET** Perched groundwater encountered at 15 feet 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 | CO = Consolidation test | MD = Maximum Density

PERCOLATION DATA SHEET

Project: Pacific San Marcos **Job No.:** 3649-SD.

Test Hole No.: PB-1 **Tested By:** MRB, **Date:** 9/30/20.

Depth of Hole As Drilled: 35.75" **Before Test:** ______ **After Test:** ______.

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
1	8:56	30	35.75	10.5	11.75	1.25	0.03	
2	9:26	30	35.75	9.75	10.5	0.75	0.02	
3	9:56	30	35.75	8.75	9.5	0.75	0.02	
4	10:26	30	35.75	8.75	9.5	0.75	0.02	
5	10:56	30	35.75	8.25	9	0.75	0.02	
6	11:26	30	35.75	8.25	9	0.75	0.02	
7	11:56	30	35.75	7.5	8.25	0.75	0.02	
8	12:26	30	35.75	7.5	8.25	0.75	0.02	
9	12:56	30	35.75	6.5	7.25	0.75	0.02	
10	13:26	30	35.75	5.75	6.75	1	0.03	
11	13:56	30	35.75	6	7	1	0.03	
12	14:26	30	35.75	6.75	7.25	0.5	0.01	
13								
14								
15								
16								

Client:	**The Las Posas Owner LPV, LLC**
Project:	**Pacific**
Project No:	**3649-SD**
Date:	**9/30/2020**

Boring No. **P-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	7.25	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	6.75	inches
Total Test Hole Depth, D_T =	35.75	inches

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

$H_O = D_T - D_O$ =	29.00	inches
$H_F = D_T - D_F$ =	28.50	inches
$\Delta H = \Delta D = H_O - H_F$ =	0.50	inches
$H_{avg} = (H_O+H_F)/2$ =	28.75	inches

I_t = 0.07 **Inches per Hour**

GEOTEK

PERCOLATION DATA SHEET

Project: Pacific San Marcos **Job No.:** 3649-SD.

Test Hole No.: PB-2 **Tested By:** MRB, **Date:** 9/30/20.

Depth of Hole As Drilled: 34.75 **Before Test:** **After Test:** .

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	∆ In Water Level (Inches)	Rate (minutes per inch)	Comments
1	8:59	30	34.75	10.25	10.9	0.65	0.02	
2	9:29	30	34.75	9	9.25	0.25	0.01	
3	9:59	30	34.75	8.5	8.75	0.25	0.01	
4	10:29	30	34.75	7.75	8.5	0.75	0.02	
5	10:59	30	34.75	7	7.75	0.75	0.02	
6	11:29	30	34.75	7	7.75	0.75	0.02	
7	11:59	30	34.75	6.5	7.25	0.75	0.02	
8	12:29	30	34.75	6.5	7.25	0.75	0.02	
9	12:59	30	34.75	6.5	7	0.5	0.01	
10	13:29	30	34.75	6.75	7.25	0.5	0.01	
11	13:59	30	34.75	6.5	7	0.5	0.01	
12	14:29	30	34.75	6.75	7.25	0.5	0.01	
13								
14								

Client:	**The Las Posas Owner LPV, LLC**
Project:	**Pacific**
Project No:	**3649-SD**
Date:	**9/30/2020**

Boring No. **P-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30	minutes
Final Depth to Water, D_F =	7.25	inches
Test Hole Radius, r =	4.00	inches
Initial Depth to Water, D_O =	6.75	inches
Total Test Hole Depth, D_T =	34.75	inches

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

$H_O = D_T - D_O$ =	28.00	inches
$H_F = D_T - D_F$ =	27.50	inches
$\Delta H = \Delta D = H_O - H_F$ =	0.50	inches
$H_{avg} = (H_O+H_F)/2$ =	27.75	inches

I_t = 0.07 **Inches per Hour**

GEOTEK

APPENDIX B

RESULTS OF LABORATORY TESTING

SUMMARY OF LABORATORY TESTING

Identification and Classification

Soils were identified visually in general accordance to the standard practice for description and identification of soils (ASTM D2488). The soil identifications and classifications are shown on the logs of exploratory borings in Appendix A.

Expansion Index

Expansion Index testing was performed on one soil sample. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below.

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-3	0-5	Dark Brown Black Clay	81	Medium

Moisture-Density Relationship

Laboratory testing was performed on one sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with test method ASTM Test Procedure D1557. The results of the testing are provided below.

Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-3	0-5	Dark Brown Black Clay	114.5	15.0

APPENDIX C

GENERAL EARTHWORK GRADING GUIDELINES

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 (2019) 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.

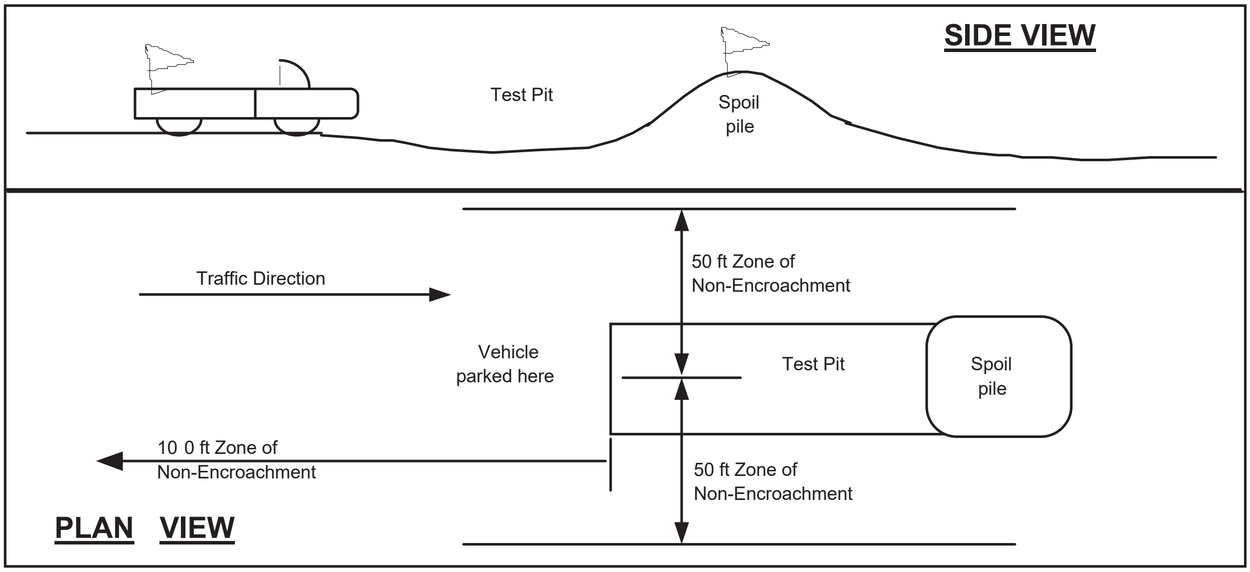
Test Pits Location, Orientation and Clearance

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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