Appendix F

Geotechnical Investigation

GEOTECHNICAL & INFILTRATION EVALUATION

FOR

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
ASSESSOR'S PARCEL NOS. 8468-015-010 & -024
1600 AND 1616 WEST CAMERON AVENUE
WEST COVINA, LOS ANGELES COUNTY, CALIFORNIA

PREPARED FOR

MLC HOLDINGS, INC.
5 PETERS CANYON ROAD, SUITE 310
IRVINE, CALIFORNIA 92606

PREPARED BY

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

PROJECT NO. 2409-CR **JUNE 30, 2020**

GEOTEK

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June 30, 2020
Project No. 2409-CR

MLC Holdings, Inc.
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Attention: Mr. Bret Ilich

Subject: Geotechnical & Infiltration Evaluation
Proposed Multi-Family Residential Development
Assessor's Parcel Nos. 8469-015-010 & -024
1600 and 1616 West Cameron Avenue
West Covina, Los Angeles County, California

Dear Mr. Ilich:

We are pleased to provide the results of our geotechnical and infiltration evaluation for the subject project referenced by the street address of 1600 and 1616 West Cameron Avenue in West Covina, Los Angeles County, California. This report presents the results of our evaluation and discussion of our findings.

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 should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

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TABLE OF CONTENTS

1. PURPOSE AND SCOPE OF SERVICES 1

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT 1

2.1 SITE DESCRIPTION 1
2.2 PROPOSED DEVELOPMENT 2

3. FIELD EXPLORATION & LABORATORY TESTING 2

3.1 FIELD EXPLORATION 2
3.2 LABORATORY TESTING 3

4. PERCOLATION TESTING 3

5. GEOLOGIC AND SOILS CONDITIONS 4

5.1 REGIONAL SETTING 4
5.2 EARTH MATERIALS 5
5.2.1 Existing Pavement 5
5.2.1 Alluvium 5
5.3 SURFACE WATER AND GROUNDWATER 5
5.4 FAULTING AND SEISMICITY 6
5.4.1 Seismic Design Parameters 6
5.4.2 Surface Fault Rupture 7
5.4.3 Liquefaction & Dynamic Densification 7
5.4.4 Other Seismic Hazards 8

6. CONCLUSIONS AND RECOMMENDATIONS 8

6.1 GENERAL 8
6.2 EARTHWORK CONSIDERATIONS 9
6.2.1 General 9
6.2.2 Site Clearing and Demolition 9
6.2.3 Remedial Grading 9
6.2.4 Engineered Fill 10
6.2.5 Excavation Characteristics 10
6.2.6 Trench Excavations and Backfill 10
6.2.7 Shrinkage and Subsidence 11
6.3 DESIGN RECOMMENDATIONS 11
6.3.1 Foundation Design Criteria 11
6.3.2 Miscellaneous Foundation Recommendations 14
6.3.3 Foundation Setbacks 14
6.4 RETAINING WALL DESIGN AND CONSTRUCTION 15
6.4.1 General Design Criteria 15
6.4.2 Wall Backfill and Drainage 16
6.4.3 Restrained Retaining Walls 16
6.4.4 Soil Corrosivity 17
6.4.5 Soil Sulfate Content 17
6.4.6 Import Soils 17
6.5 PRELIMINARY PAVEMENT DESIGN 18

TABLE OF CONTENTS

6.6 CONCRETE CONSTRUCTION 18
6.6.1 General *18*
6.6.2 Concrete Flatwork *19*
6.6.3 Concrete Performance *19*
6.7 POST CONSTRUCTION CONSIDERATIONS 20
6.7.1 Landscape Maintenance and Planting *20*
6.7.2 Drainage *20*
6.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS 21

7. LIMITATIONS 21

8. SELECTED REFERENCES 22

ENCLOSURES

Figure 1 – Site Location Map
Figure 2 – Boring Location Map

Appendix A – Logs of Exploratory Borings
Appendix B – Laboratory Test Results
Appendix C – Infiltration Test Results
Appendix D – Seismic Settlement Analysis
Appendix E – General Grading Guidelines

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 and geotechnical data, and general information pertinent to the site,
- Excavation of six geotechnical borings extended to depths of approximately 15 to 51 feet below grade;
- Drill two borings to a depth of about 10 and 15 feet for infiltration testing;
- Collection of bulk and undisturbed samples from the test borings;
- Performance of laboratory testing on select soil samples;
- Review and evaluation of site seismicity, and
- Compilation of this geotechnical and infiltration evaluation report which presents our findings and a general summary of pertinent geotechnical conditions relevant for site development.

The intent of this report is to aid in the evaluation of the site for future development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report will likely need to be updated based on our review of final site development plans. These should be provided to GeoTek for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The approximate 3.25-acre rectangular shaped site is located on the south side of West Cameron Avenue in West Covina, California and is referenced by the street address of 1600 and 1616 West Cameron Avenue. Toluca Avenue is located to the east of the site and the Walnut Creek channel is located about 250 feet to the southeast. The approximate location of the site is noted on the attached Figure 1, Site Location Map. Two commercial structures are located in the northeast and northwest portions of the site and the remaining areas of the site are paved with asphalt or landscaped. The age of the existing buildings is not known but is greater than 25 years. Topographically, the site is relatively level with less than about 5 feet of elevation differential sloping downward to the south. The site is surrounded by existing residential developments to the south and commercial developments to the north (north of Cameron Avenue), west and east.

2.2 PROPOSED DEVELOPMENT

It is our understanding that site development will consist of several multi-family residential structures and associated roadways and improvements. Stormwater improvements are also understood to be planned. We anticipate that the residential structures will be 3 stories in height and will be supported by a shallow post-tensioned foundation system. Although structural loading information was not available at the time of this report preparation, we anticipate maximum column and wall loads on the order of 60 kips and 4 kips per foot, respectively. Once actual structural loads are known that information should be provided to GeoTek to determine if modifications to the recommendations contained in this report are warranted.

Although grading plans have not yet been developed, we anticipate that the maximum depth of cut or fill will be less than about 5 feet, not including any remedial grading.

As site development planning progresses and plans become available, the plans should be provided to GeoTek for review and comment. Additional engineering analyses may be necessary in order to provide specific earthwork recommendations and geotechnical design parameters for actual site development.

3. FIELD EXPLORATION & LABORATORY TESTING

3.1 FIELD EXPLORATION

GeoTek performed a field exploration at this site on June 11, 2020 which consisted of excavating seven exploratory borings to depths ranging from about 21-½ to 51-½ feet. In addition, two percolation test borings about 10 and 15 feet deep were advanced within the currently proposed stormwater infiltration area. The borings were drilled with a hollow-stem auger drill rig.

The approximate locations of our site explorations are shown on the Boring Location Map, Figure 2. Logs of the borings are provided in Appendix A.

3.2 LABORATORY TESTING

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

4. PERCOLATION TESTING

Percolation testing was performed at boring locations I-1 and I-2 to assess the infiltration rate of the soils near the bottom of the proposed site basin. The testing was performed at approximate depths of 10 and 15 feet from the existing ground surface. The boring logs of the percolation borings are presented in Appendix A and the locations of the borings are shown on Figure 2.

Testing was performed in general accordance with the Los Angeles County Administrative Manual GS200.1, dated June 30, 2014, using the Boring Percolation Test Procedure. The testing consisted of drilling an eight-inch diameter test hole to a depth of about 12 inches below the desired depth and installing about two inches of gravel in the bottom of the hole. A three-inch diameter perforated PVC pipe, wrapped in filter sock, was placed in the boring excavation 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 conducted following a minimum 4-hour presoak period.

The field percolation rate, based on the stabilized rate obtained, obtained is presented below.

SUMMARY OF TEST RESULTS		
Boring	Depth (ft)	Measured Percolation Rate (inches per hour)
I-1	10	11.5
I-2	15	16.5

As required, a Correction Factor must be applied to the measured rate to determine the design value that will represent long-term performance of the BMP. As outlined within the LA

County Manual, the Correction Factor (Ct, also noted as Rf) is calculated using the following relationship:

$$Rf = [(2d-\Delta d)/DIA] + 1$$

Where d= initial water depth (inches)
Δd= water level drop of the final period or stabilized rate (inches)
DIA= diameter of the boring (inches)

Based on the measurements at the site, a Rf value of 5.28 and 5.0 have been calculated for I-1 and I-2, respectively.

As required, a Correction Factors for site variability (CFv) and long-term siltation (CFs) must also be considered. As noted in the LA County Manual, CFv and CFs should vary between 1 and 3. A CFv of 1 is preliminarily considered suitable and the value to be selected for CFs should be based on the level of pre-treatment and maintenance for the proposed BMPs.

Assuming CFs and CFv values of 1 and using the Correction Factor (Ct) noted above, we recommend a Total Correction Factor of 5.28 be applied to the measured rates obtained. Detailed percolation/infiltration test data is included in Appendix C.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province, just south of the Transverse Ranges province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends from the point of contact with the Transverse Ranges geomorphic province, southerly 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 mostly found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province, and the San Jacinto fault borders the province adjacent the Colorado Desert province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by alluvium consisting of gravels, sands and silts of valleys and floodplains (Dibblee, T.W. and Ehrenspeck, H.E., 1999). No active faults are shown in the immediate site vicinity on the maps reviewed for the site and site area.

5.2 EARTH MATERIALS

A brief description of the earth materials encountered during our subsurface exploration is presented in the following section. Based on the exploratory excavations and review of published geologic maps, the site is underlain by alluvium. Although not encountered, localized of undocumented fill may be present.

5.2.1 Existing Pavement

Approximately 3 to 4 inches of asphalt pavement was encountered at the ground surface at the boring locations. About 4 to 5-½ inches of aggregate base was observed beneath the pavement.

5.2.1 Alluvium

Alluvial soil was encountered within all borings beneath the existing pavement. The alluvium varied from a poorly graded sand, silty sand to a sandy silt. The sandy soils were noted to range from loose to very dense and the silt soils possessed a medium stiff to very stiff consistency. Gravel layers were also encountered. Although not encountered within any of the test borings, localized undocumented fill may be present, especially beneath existing buildings.

5.3 SURFACE WATER AND GROUNDWATER

5.3.1 Surface Water

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 northerly direction, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

5.3.2 Groundwater

Groundwater was not encountered in any of the exploratory borings which extended to a maximum depth of about 50 feet. Based on a review of the Seismic Hazard Zone Report for the Baldwin Park 7.5-Minute Quadrangle (CGS, 1998b), the historic high groundwater is estimated to be about 75 feet below grade. Based on this depth to water, groundwater-related problems are not expected during or after construction.

It is possible that seasonal variations (temperature, rainfall, etc.) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered at shallow depths following extensive rain events. If shallow perched water is encountered, we anticipate that it can be managed with conventional sump pumps.

5.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within a State of California designated *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1998b).

5.4.1 Seismic Design Parameters

The site is located at approximately 34.0688□Latitude and -117.9434□Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. Using the ASCE 7-16 option on the SEAOC/OSHPD website results in the values for S_{M1} and S_{D1} reported as "null-See Section 11.4.8" (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

For a site Class D, an exception to performing a site-specific ground motion analysis is allowed in ASCE 7-16 where S_1 exceeds 0.2 provided the value of the seismic response coefficient, Cs, is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of $T \leq 1.5Ts$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5Ts$ or Eq. 12.8-4 for $T > T_L$.

The results, based on the 2015 NEHRP and the 2019 CBC, are presented in the following table and we have assumed that the exception as allowed in ASCE 7-16 is applicable. If the exception is deemed not appropriate, a site-specific ground motion analysis will be required.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.666g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.609g
Site Coefficient for Site Class "D," F_a	1.0
Site Coefficient for Site Class "D," F_v	1.7
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.666g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	1.036g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.111g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.690g
PGA_M	0.776g
Seismic Design Category	D

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.

5.4.2 Surface Fault Rupture

The site is in a seismically active region; however, no active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007). The nearest known active fault is the Sierra Madre fault located about 5.2 miles to the north and the Whittier section of the Elsinore fault zone situated about 6.6 miles to the south. Therefore, the potential for surface rupture at the site is considered to be nil.

5.4.3 Liquefaction & Dynamic Densification

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless and some low-plastic soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding 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 most susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures. Based on a review of the Earthquake Zones of Required Investigation for the Baldwin Park Quadrangle (CGS, 1999), the site is not located within a liquefaction hazard area.

Based on the current mapping of the site and the reported depth to groundwater, it is our opinion that the potential for soil liquefaction during a seismic event is nil for the subject site.

We also evaluated the potential for dynamic densification (dry seismic settlement) resulting from seismic activity. For this analysis we used a PGA_M of 0.776 and a seismic event of 6.95. The ground acceleration and earthquake magnitudes were obtained from the USGS websites. The soil profile from Boring B-5 was also used. The results of this analysis indicate a seismic dry settlement of about ½ inch is possible. This settlement is expected to occur over a large area and differential seismic settlement of less than ¼ inch over a 40-foot span is estimated. Based on the magnitudes of estimated seismic settlements, mitigation and/or special foundation design is not considered warranted. A copy of the seismic settlement analysis is presented in Appendix D.

5.4.4 Other Seismic Hazards

The potential for secondary seismic hazards such as seiche and tsunami is considered to be remote due to site elevation and distance from an open body of water. Due to the absence of a nearby free-face and the low liquefaction hazard, the potential for lateral spreading is considered to be nil.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Development of the site appears feasible from a geotechnical viewpoint. Specific recommendations for site development provided in this report will need to be further evaluated when development plans are provided for our review. The following sections present general recommendations. More specific geotechnical recommendations for site development can be provided when more finalized site development plans are available for review.

6.2 EARTHWORK CONSIDERATIONS

6.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of West Covina, the 2019 California Building Code (CBC) and recommendations contained in this report. The General 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.

6.2.2 Site Clearing and Demolition

Site preparation should start with demolition of the existing improvements and removal of all deleterious materials and vegetation within the planned development areas of the site. Demolition of the existing buildings should include removal of all foundations, floor slabs and any below-grade construction. Existing underground utilities should also be capped off at the property lines and removed or re-routed around the new improvements. All debris and deleterious materials should be properly disposed of off-site.

6.2.3 Remedial Grading

Due to the variable consistency of the near surface soils and the expected soil disturbance resulting from demolition of the existing improvements, we recommend that the existing soils beneath the planned buildings be over-excavated to a depth of at least 3 feet below existing or finished grade and at least 1.5 feet below the bottom of the planned foundations, whichever is deeper. If existing fill soils are encountered, the over-excavations should be extended to removal of all undocumented fill. The lateral extent of this recommended over-excavation should extend at least 5 feet beyond the building perimeters and beneath all adjacent patio slabs. The soils exposed at the base of the soil over-excavations should be examined by a GeoTek representative to document that the exposed soils are suitable for support of the planned improvements. If unsuitable soils are observed, the over-excavation should be extended in depth until suitable soils, as determined by GeoTek, are encountered.

Beneath concrete flatwork and street pavements, it our opinion that the over-excavations may be limited to 12 inches below existing or finished grade, whichever is deeper, provided all existing undocumented fill is removed.

Following the recommended removals and observations by GeoTek, the exposed soils should be scarified to a depth of about 12 inches, be moisture conditioned to slightly above the soil's optimum moisture content and then be compacted to at least 90% of the soil's maximum dry density, per ASTM D 1557.

6.2.4 Engineered Fill

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Any over-sized material (greater than 3 inches in maximum dimension) should be removed from the soil prior to use as fill. The undercut areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in six-inch to eight-inch loose lifts, moisture conditioned to about two percent above the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D 1557.

6.2.5 Excavation Characteristics

Excavations in the on-site alluvial materials and engineered fill materials should be readily accomplished with heavy-duty earthmoving or excavating equipment in good operating condition. Some localized cobbles and/or gravel layers may be encountered.

6.2.6 Trench Excavations and Backfill

Temporary trench excavations within the on-site materials should be stable at 1:1 inclination for short durations during construction and where cuts do not exceed 10 feet in height. We anticipate that temporary cuts to a maximum height of four feet can be excavated vertically.

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

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

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

6.2.7 Shrinkage and Subsidence

For planning purposes, a shrinkage factor of about 5 to 10 percent may be considered for materials that may need to be removed and replaced. A subsidence loss of about 0.1 foot should also be anticipated.

Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork construction.

6.3 DESIGN RECOMMENDATIONS

6.3.1 Foundation Design Criteria

Following site grading, the soils are expected to have a "very low" ($0 \leq EI \leq 20$) to "low" ($21 < EI \leq 50$) expansion potential in accordance with ASTM D 4829. It is our understanding that a post-tensioned slab will be utilized for the project.

The CBC indicates the Post Tensioning Institute (PTI) design methodology is intended for expansive soils conditions, which do not apply for the very low expansion condition. Therefore, the preliminary post tension design recommendations presented in this report are based on "low" expansive soil conditions. Final post tension foundation design should be determined on a lot based on Expansion Index and plasticity testing of the as-graded soil conditions.

Presented below are preliminary post-tension foundation design parameters for proposed residences derived in general conformance with *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* (PTI, 2012). Post-tensioned slabs should be designed in accordance with the *2019 California Building Code* (CBC) and PTI design methodology.

Foundation Design Parameter	
Edge Moisture Variation Distance, e_m - Edge Lift (swelling) - Center Lift (shrinkage)	 3.3 ft 5.6 ft
Soil Differential Movement, y_m - Edge Lift (swelling) - Center Lift (shrinkage)	 ≈0.55 in ≈-0.15 in
Ext. Perimeter Beam Embedment	One- or Two-Story – 12 inches*
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 110% to a depth of 12 inches

* Required depth of perimeter beam/stiffening rib per structural calculations may govern.
The following assumptions were used to generate e_m and y_m values: LL=33 PL=19; and PI=14. Thornthwaite Moisture Index = -25; constant suction value = 3.6pF; post-equilibrium case assumed with wet (swelling) cycle going from 4.0pF to 3.0pF and drying (shrinking) cycle going from 4.0pF to 4.5pF.

An allowable bearing capacity of 2,500 pounds per square foot (psf) may be used for design of building and retaining wall footings. This allowable soil bearing capacity is based on a minimum foundation depth and width of 12 inches. This value may be increased by 500 psf for each additional 12 inches of embedment depth and by 250 psf for each additional 12 inches in width to a maximum of 3,500 psf. The allowable bearing capacity may be increased by one-third when considering short-term wind and seismic loads.

The bottom of the perimeter edge beam/deepened footing for post tension systems should be deepened a minimum of 12 inches and designed to resist tension forces using either cable or conventional reinforcement, per the structural engineer.

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. We estimate static settlement of foundations designed as recommended in this report to be less than 1 inch total and ½ inch differential over a 40 foot span.

The passive earth pressure may be computed as an equivalent fluid having a density of 230 psf per foot of depth, to a maximum earth pressure of 2,000 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. 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 2019 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2019 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 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 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, it is GeoTek's opinion that 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 thick 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 limited 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 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.

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.

6.3.2 Miscellaneous Foundation Recommendations

6.3.2.1 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.

6.3.2.2 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.

6.3.3 Foundation Setbacks

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

6.4 RETAINING WALL DESIGN AND CONSTRUCTION

6.4.1 General Design Criteria

Recommendations presented herein may apply to typical masonry or concrete vertical retaining walls to a maximum height of six feet. Additional review and recommendations should be requested for higher walls.

Retaining wall foundations embedded a minimum of 12 inches into engineered fill should be designed using an allowable bearing capacity of 2,500 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 230 psf per foot of depth, to a maximum earth pressure of 2,500 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	42
2:1	65

*Backfill should consist of imported sand other approved materials with an expansion index less than or equal to 20.

The above equivalent fluid weights do not include superimposed loading conditions such as expansive soils, 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 "D", an incremental seismic equivalent fluid pressure of 24.3 pcf is recommended, where required. This pressure can be assumed to be a conventional triangular distribution.

6.4.2 Wall Backfill and Drainage

Wall backfill should include a minimum one-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 the wall and extend up from the backdrain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted on-site materials. The 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 eight inches in thickness and compacted to a minimum of 90 percent relative compaction 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 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 be directed (via a solid outlet pipe) to an appropriate disposal area.

Walls from two to four feet in height may be drained using localized gravel packs behind weep holes at 8 feet maximum spacing (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag). 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.

6.4.3 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 62 pcf (very low expansive 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.

6.4.3.1 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 bottom outside edge of the footing to a sloped 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 by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

6.4.4 Soil Corrosivity

The soil resistivity at this site was tested in the laboratory on one sample collected by our firm. The results of the testing indicate that the soil sample was considered *"highly corrosive"* (1,809 ohm-cm) to buried ferrous metals in accordance with current standards commonly used by corrosion engineers. Consideration should be given to consulting with a corrosion engineer. The laboratory test results are provided in Appendix B.

6.4.5 Soil Sulfate Content

The sulfate content was determined in the laboratory for one representative soil sample collected by our firm. The results indicate that the water-soluble sulfate for the tested sample was less than 0.1 percent by weight, which is considered "not applicable" (i.e. negligible) as per Table 4.2.1 of ACI 318. Based upon the test result, no special concrete mix design is required for sulfate attack resistance. The laboratory test result is provided in Appendix B.

6.4.6 Import Soils

Import soils should have expansion characteristics similar to the on-site soils. GeoTek also recommends that the proposed import soils be tested for expansion and sulfate potential. GeoTek should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

6.5 PRELIMINARY PAVEMENT DESIGN

A preliminary pavement section has been developed based on assumed traffic loading and our estimate of the pavement subgrade soils following completion of site grading. Given the preliminary nature of the pavement sections presented below, final pavement design should be based on R-value testing of the as-graded soils and the known or assigned Traffic Indexes for the site roadways. Based on the near-surface soil types encountered in our test borings, we estimate that an as-graded R-value of 20 is appropriate for this preliminary design. For this preliminary design, we have assumed a Traffic Index of 5.5. Based on the above discussion, the following preliminary pavement design is presented.

Street	Assumed Traffic Index	Asphaltic Concrete/Aggregate Base (inches)
Interior Streets	5.5	3/9

The final pavement sections are subject to the review and approval by the local jurisdictional agency. Performance of the pavement sections will ultimately be based largely on construction methods, traffic loading and subgrade performance. All aggregate base and the upper 12 inches of subgrade should be compacted to at least 95% of the material's maximum dry density, per ASTM D-1557.

All pavement installation, including preparation and compaction of subgrade and base material and placement and rolling of asphaltic concrete, should be done in accordance with the City of West Covina specifications, and under the observation and testing of GeoTek and a City or County inspector where required.

The aggregate base should consist of crushed rock with an R-Value and gradation in accordance with Crushed Aggregate Base (Section 200-2 of the "Greenbook"). Minimum compaction requirements should be 95 percent for both subgrade and aggregate base (ASTM D 1557). Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

6.6 CONCRETE CONSTRUCTION

6.6.1 General

Concrete construction should follow the 2019 CBC and ACI guidelines regarding design, mix placement and curing of the concrete. If desired, we could provide quality control testing of the concrete during construction.

6.6.2 Concrete Flatwork

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in industrial 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 120% of the optimum moisture content to a depth of 18 inches.

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

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

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

6.7 POST CONSTRUCTION CONSIDERATIONS

6.7.1 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. 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 foundations. This type of landscaping should be avoided. Planters within 10 feet of the buildings should be above ground and underlain by a concrete slab. Waterproofing of the foundation and/or subdrains may be warranted and advisable. We could discuss these issues, if desired, when plans are made available.

6.7.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground 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.

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

6.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site 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 site 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 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 when necessary.
- Observe the fill for uniformity during placement including utility trenches.
- Test the fill for field density and relative compaction.
- Test the near-surface soils to verify proper moisture content.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

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

7. LIMITATIONS

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. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our proposal (Proposal No. 0600220) dated June 3, 2020 and geotechnical engineering standards normally used on similar projects in this region.

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

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our 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.

8. SELECTED REFERENCES

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MLC Holdings, Inc.
1600 and 1616 West Cameron Avenue
APNs 8468-015-010 and -024
West Covina, Los Angeles County, California

Project No. 2409-CR

NORTH

Figure 1

Site Location Map

GEOTEK

MLC Holdings, Inc.
Proposed Multi-Family Residential Development
1600 and 1616 West Cameron Avenue
West Covina, Los Angeles County, California

GeoTek Project No. 2409-CR

Figure 2

Boring Location Map

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Residential Development

West Covina, Los Angeles County, California

Project No. 2409-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 LOG LEGEND

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

SOILS

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

GEOLOGIC

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

(Additional denotations and symbols are provided on the log of boring)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

BORING NO.: B- 1

Depth (ft)	Sample Type (SAMPLES)	Blows/ 6 in (SAMPLES)	Sample Number (SAMPLES)	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%) (Laboratory Testing)	Dry Density (pcf) (Laboratory Testing)	Others (Laboratory Testing)
					AC = 4" **AB = 5"**			
	Large Bulk				**Alluvium:**			EI, MD, SH, SR EI=39
	Ring	9 12 14	R1	ML	Sandy SILT, dark brown, moist, very stiff, roothairs, trace f gravel	15.5	116.9	LL=33, PL=19, PI=14
5	Ring	3 7 10	R2		Same, stiff	10.3	112.3	Collapse PP = 3.5
	Ring	4 6 7	R3		Same	8.0	116.0	
10	Ring	6 11 14	R4	SM-SP	Slightly silty f SAND, orangish brown, slightly moist, medium dense, pinhole pores locally			SA % Passing #200 = 6.2
15	Ring	3 6 11	R5	SP	F-m SAND, tan, yellowish brown, dry, medium dense, very friable			
20	Ring	12 20 25	R6		Same, dense, trace f gravel			
					BORING TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with soil cuttings pp= pocket penetrometer test (tsf)			
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:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

BORING NO.: B- 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
					AC = 3" **AB = 4"**			
	Large Bulk				**Alluvium:**			
	Ring	3 7 9	R1	ML	Clayey SILT, dark brown, moist, stiff, slightly micaceous	10.0	115.6	
5	Ring	6 7 8	R2		Same	5.5	107.4	PP = 4.25
	Ring	3 4 7	R3	SP	F SAND with SILT, orangish brown, moist, loose	5.5		
10	Ring	7 9 14	R4	SP	F-m SAND, gray, tan, dry to slightly moist, medium dense			
15	Ring	12 16 18	R5		Same			
20	Ring	11 15 20	R6	SW	Gravelly f-c SAND, tan, dry, medium dense, very friable			
					BORING TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with soil cuttings pp= pocket penetrometer test (tsf)			
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:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

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
					AC = 3" **AB = 5"**			
		6 6 10	R1	ML	**Alluvium:** Sandy SILT, dark brown, moist, stiff, trace clay	15.7	113.9	PP = 4.0
5		2 3 4	R2		Same, medium stiff	17.4	107.7	PP = 4.25
		4 7 10	R3	SM-ML	Silty f SAND/Sandy SILT, orangish brown, slightly moist, medium dense/stiff	7.3	114.3	Collapse
10		9 18 20	R4	SP	F SAND, tan, gray, dry, medium dense, very friable			
15		9 21 50/6	R5		Same, very dense, trace f-c gravel and cobbles			
20		10 18 20	R6	SP	F-m SAND, gray, dry to slightly moist, medium dense			
25					**BORING TERMINATED AT 21.5 FEET** No groundwater encountered Boring backfilled with soil cuttings pp= pocket penetrometer test (tsf)			
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:		**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:		**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	
PROJECT NO.:	-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	/ /

BORING NO.: B- 4

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
					AC = 3" **AB = 4"**			
		4 5 5	R1	ML	**Alluvium:** Clayey SILT, dark brown, moist, medium stiff, trace f-c gravel	17.6	106.2	PP = 3.25
5		3 3 4	R2	ML	Sandy SILT, dark brown, moist, medium stiff, pinhole pores locally	13.9	107.5	
		5 6 8	R3		Same, stiff	12.6	113.3	
10		4 6 7	R4		Same, slightly moist			
15		6 10 17	R5	SP	F SAND, tan, gray, dry, medium dense, friable			
20		11 17 23			No recovery			
					BORING TERMINATED AT 21.5 FEET			
					N groundwater encountered Boring backfilled with soil cuttings pp=pocket penetrometer test (tsf)			
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:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

BORING NO.: B- 5
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
					AC = 3.5" **AB = 5.5"**			
	Large Bulk				**Alluvium**			
	Ring	5 6 6	R1	ML	Sandy SILT, dark brown, moist, stiff, trace roothairs	16.6	108.1	
5	Ring	4 4 7	R2		Same, pinhole pores locally	12.7	112.6	
	Ring	5 6 14	R3	SM-ML	Silty F SAND/Sandy SILT, dark orangish brown, slightly moist to moist, medium dense/stiff, trace pinhole pores	7.9	118.2	
10	Ring	4 8 7	R4		Same			
15	Ring	4 9 11	R5	SP	F SAND, grayish brown, tan, slightly moist, medium dense, friable			
20	Ring	19 32 50/5.5			No recovery, very dense			
25	SPT	10 18 22	S1	SM	Silty f SAND, brown, dry to slightly moist, dense, friable			SA % Passing #200 = 31.9
30	SPT	5 15 30	S2	SP	F-m SAND with GRAVEL, grayish brown, slightly moist, dense, subangular to subrounded gravel			

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:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

BORING NO.: B-5 Sheet 2 of 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
					Drilling slowed due to gravels and cobbles			
35	SPT	10 18 25	S3	SP	F-m SAND, tan, slightly moist, dense, friable			
40	SPT	40 50/6	S4	SW-SP	Slightly silty f-c SAND with GRAVEL, grayish brown, tan, dry to slightly moist, very dense, friable			SA % Passing #200 = 6.6
45	SPT	20 28 34	S5		Same			
50	SPT	20 25 20	S6	SP	F-m SAND, grayish brown, slightly moist, dense, friable, trace f-c gravel			

BORING TERMINATED AT 51.5 FEET

No groundwater encountered
Boring backfilled with soil cuttings

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:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

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
					AC = 3" **AB = 4"**			
					Alluvium			
	Ring	3 4 5	R1	ML	Clayey SILT, dark brown, moist, stiff, trace f gravel and carbon	19.7	104.9	PP = 2.75
5	Ring	6 6 7	R2		Same	14.1	102.3	Collapse
	Ring	3 4 5	R3	ML	Sandy SILT, dark orangish brown, moist, stiff, pinhole pores locally	10.7	113.9	PP = 2.75
10	Ring	4 5 6	R4		Same			
15	Ring	6 7 8	R5	SP	F SAND, grayish brown, tan, slightly moist, medium dense, friable			
					Drilling slowed due to gravels and cobbles			
20	Ring	19 28 48		SP	Gravelly f-m SAND, gray, slightly moist, very dense, very friable			
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings pp= pocket penetrometer test (tsf)			
25								
30								

Laboratory Testing: Water Content, Dry Density, Others

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:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

BORING NO.: I-1

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
					AC = 3" **AB = 4"**			
					Alluvium:			
				ML	Clayey SILT, dark brown, slightly moist, stiff			
5					Moist			
				SP	F SAND with SILT, orangish brown, moist, medium dense			
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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	MLC Holdings, Inc.	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	1600 and 1616 W. Cameron Ave.	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Evan
PROJECT NO.:	2409-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	6/11/2020

BORING NO.: I-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
					AC = 3" **AB = 4"**			
				ML	**Alluvium:** Clayey SILT, dark brown, slightly moist, stiff			
5								
				SP	Sandy SILT, orangish brown, moist, stiff			
10								
				SP	F SAND, tan, yellowish brown, dry to slightly moist, medium dense			
15					**BORING TERMINATED AT 15 FEET** No groundwater encountered			
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

Residential Development
West Covina, Los Angeles County, California
Project No. 2409-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 shown on the logs of exploratory borings 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 logs of exploratory borings in Appendix A.

Collapse
Collapse testing was performed on selected samples of the site soils according to ASTM Test Method D 4546. The results of this testing are 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
Expansion Index testing was performed on two representative soil samples. 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-1	1-5	Sandy Silt	39	Low

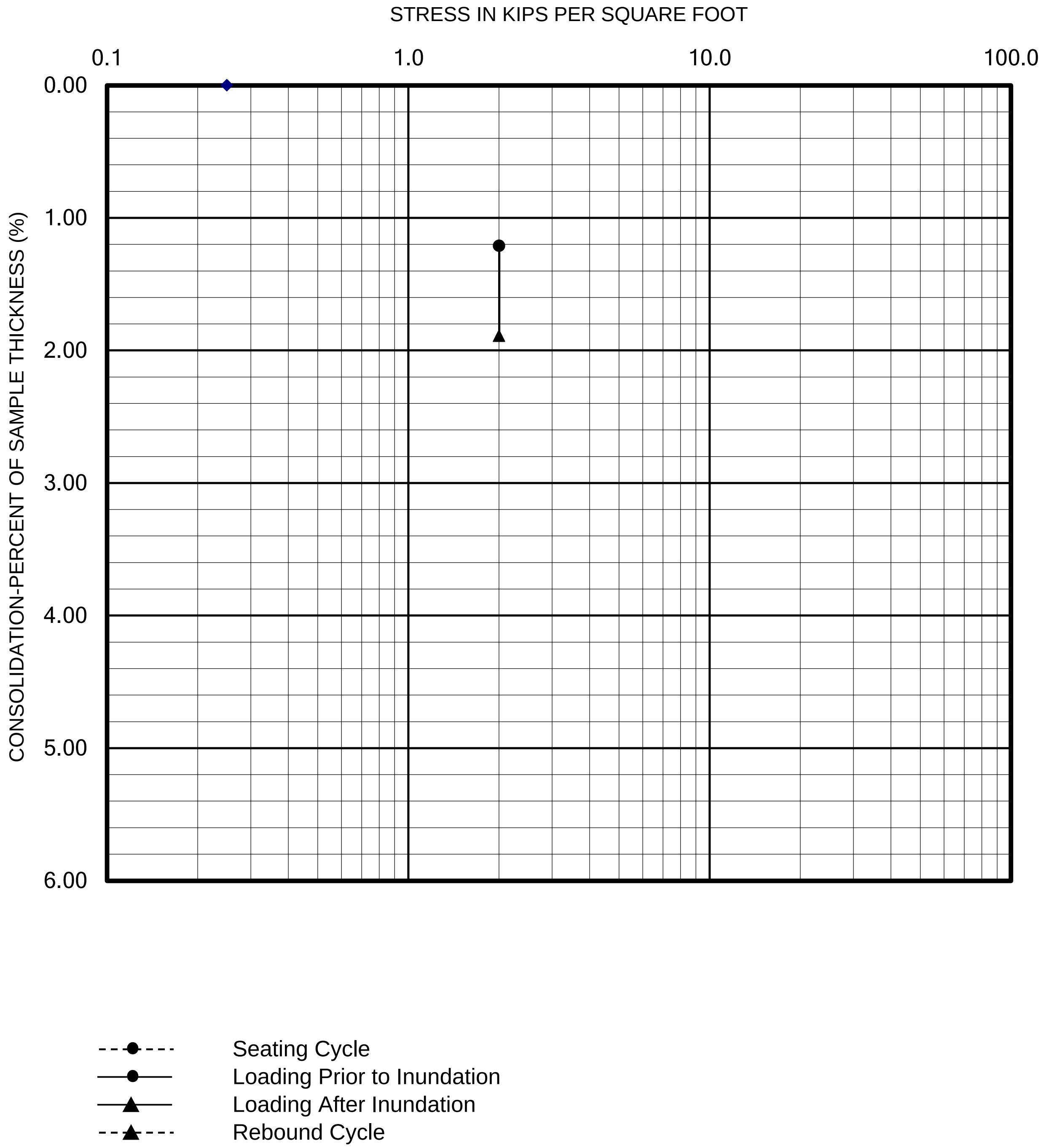
Moisture-Density Relationship
Laboratory testing was performed on a representative site sample collected during the recent subsurface exploration. The laboratory maximum dry density and optimum moisture content for the sample tested was determined in general accordance with test method ASTM Test Procedure D 1557. The results are included in Appendix B.

Percent of Soil Finer than No. 200 Sieve
Tests to determine the percent of soil finer than No. 200 sieve were performed on selected samples obtained from the property. The tests were conducted in general accordance with ASTM D1140. The test results are shown on the logs of borings in Appendix A.

Sulfate Content, Resistivity and Chloride Content

Testing to determine the water-soluble sulfate content, resistivity testing and the chloride content was performed by others. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	pH ASTM G51	Chloride ASTM D4327 (ppm)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-1	1-5	8.05	4.8	0.0233	1,809



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

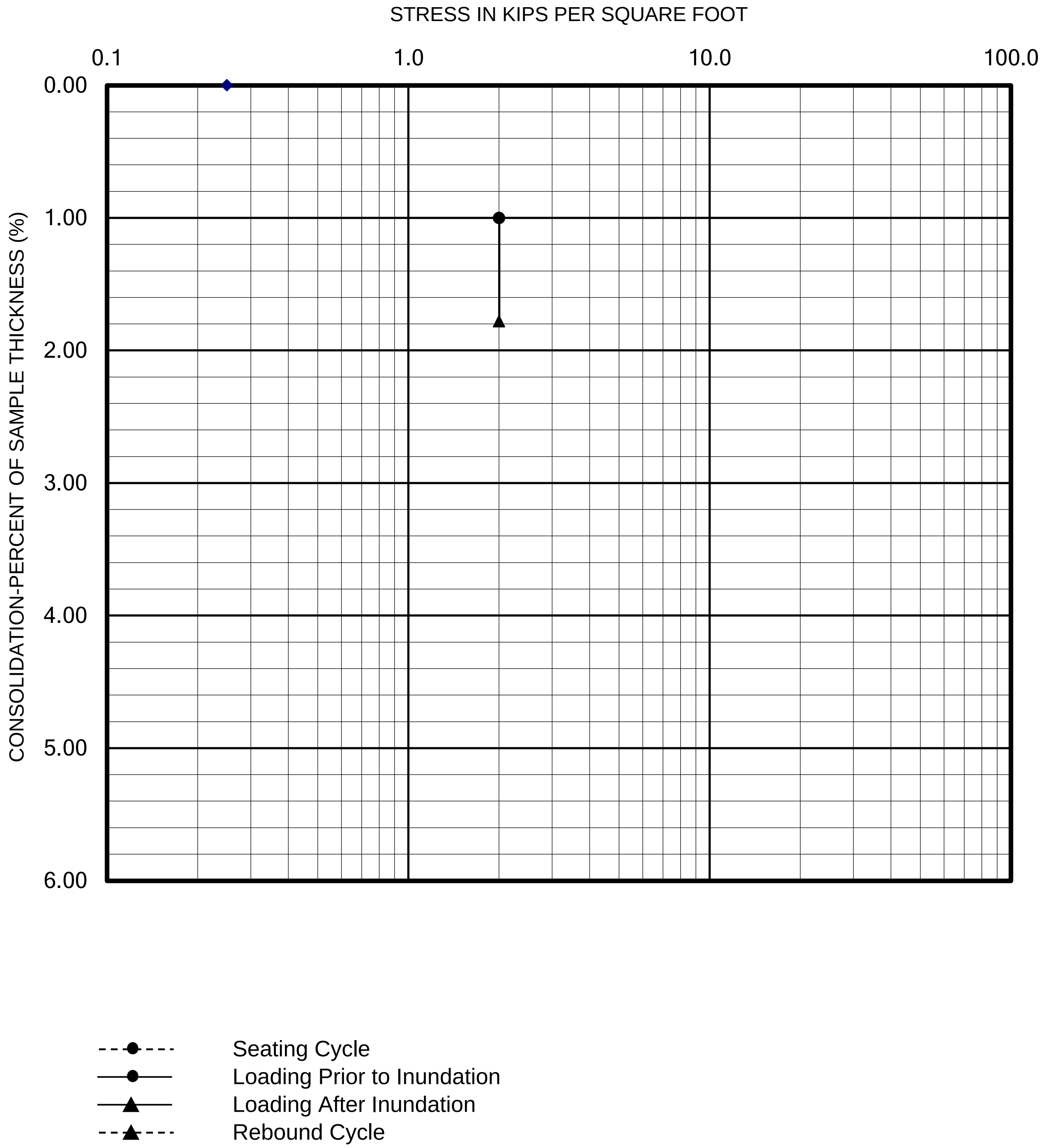
GEOTEK

COLLAPSE REPORT

Sample: B-1 @ 5'

West Cameron Avenue
West Covina, California

CHECKED BY: RRR	Lab: DI
PROJECT NO.: 2409-CR	Date: 06/2020



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

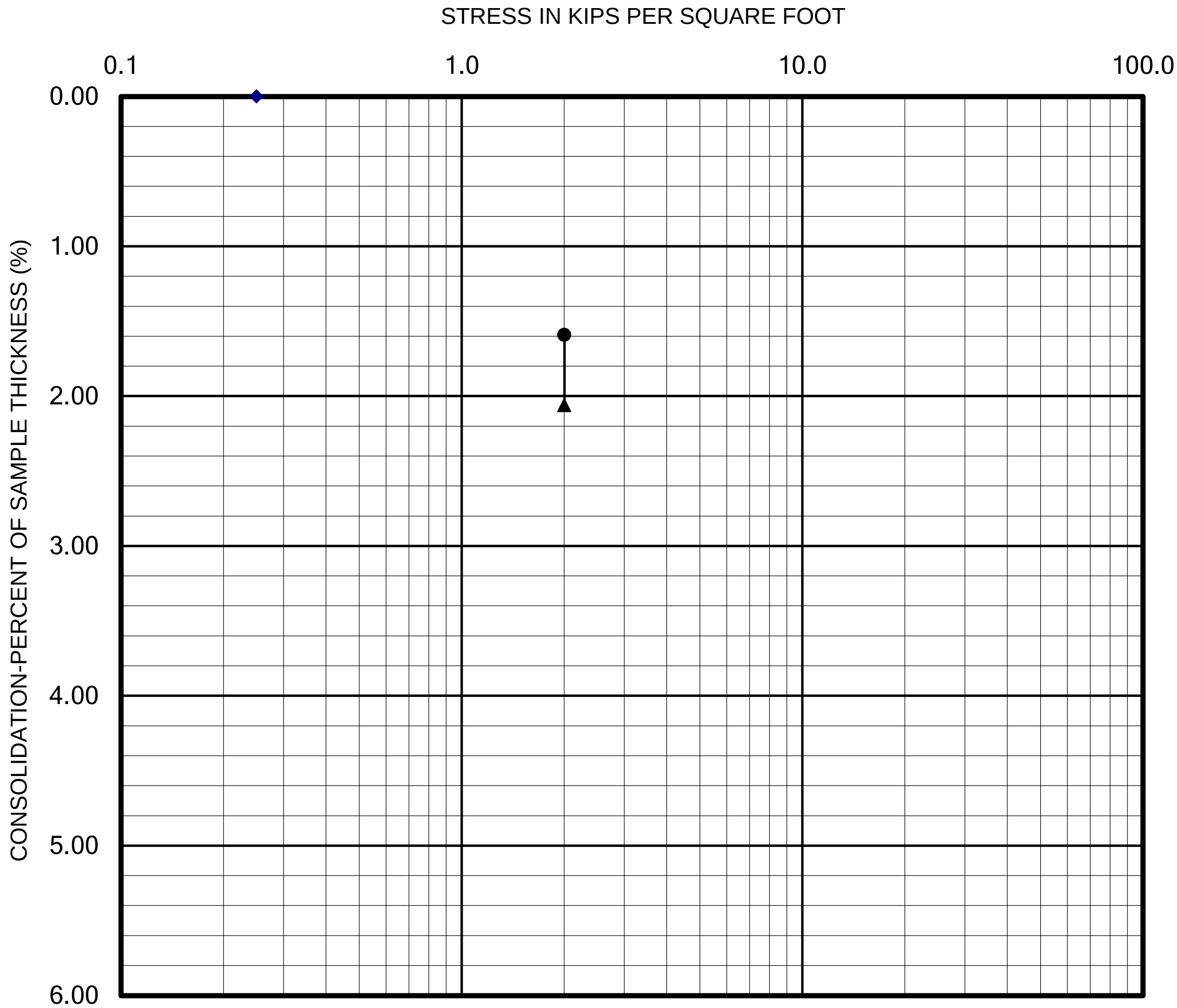
GEOTEK

CHECKED BY: RRR	Lab: DI
PROJECT NO.: 2409-CR	Date: 06/2020

COLLAPSE REPORT

Sample: B-3 @ 7'

West Cameron Avenue
West Covina, California



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

CHECKED BY: RRR	Lab: DI
PROJECT NO.: 2409-CR	Date: 06/2020

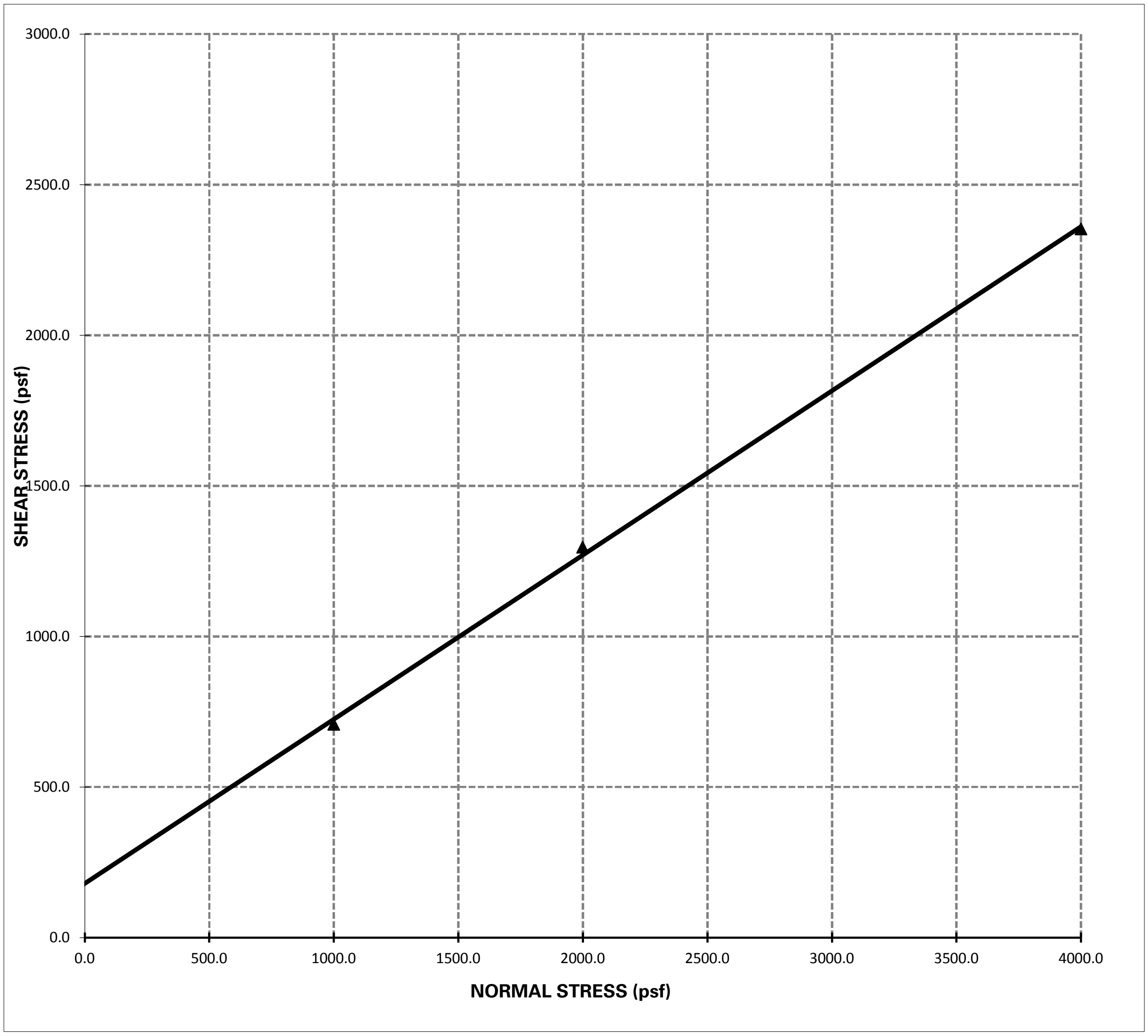
COLLAPSE REPORT

Sample: B-6 @ 5'

West Cameron Avenue
West Covina, California

DIRECT SHEAR TEST

Project Name:	1600 & 1616 West Cameron Ave.	***Sample Location:***	B-1 @ 1 - 5
Project Number:	2409-CR	***Date Tested:***	6/29/2020



Shear Strength: $\Phi =$ **28.6 °** , **C = 180.00 psf**

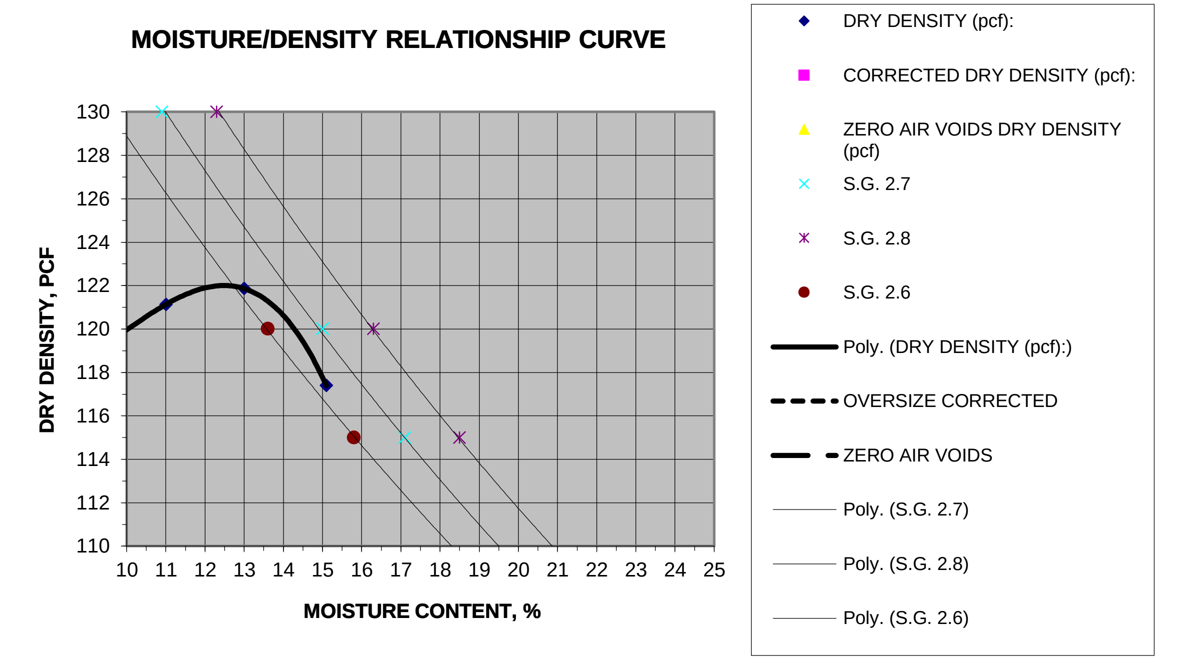
Notes:
1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.035 in/min.

MOISTURE/DENSITY RELATIONSHIP

Client:	MLC Holdings	**Job No.:**	2409-CR
Project:	1600 & 1616 West Cameron Ave.	**Lab No.:**	Corona
Location:	West Covina		
Material Type:	Brown Clayey F - M Sand		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-1 @ 1 - 5		
	-		
Sampled By:	KM	**Date Sampled:**	6/12/2020
Received By:	DLI	**Date Received:**	6/15/2020
Tested By:	DLI	**Date Tested:**	6/23/2020
Reviewed By:	-	**Date Reviewed:**	-

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 8.8 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	122.5	**@ Optimum Moisture, %**	12.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification: __________

AASHTO Soils Classification: __________

Results Only Soil Testing for 1600 & 1616 W. Cameron, West Covina

June 29, 2020

Prepared for:
Anna Scott
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92880
ascott@geotekusa.com

Project X Job#: S200624E
Client Job or PO#: 2409-CR

Respectfully Submitted,

Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
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REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: 1600 & 1616 W. Cameron, West Covina
Client Job Number: 2409-CR
Project X Job Number: S200624E
June 29, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity**		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
						As Rec'd	**Minimum**												
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ohm-cm)**	**(Ohm-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B-1	1.0-5.0	233.3	0.0233	4.8	0.0005	10,050	1,809	8.05	193	0.15	0.9	0.6	ND	52.5	2.6	9.9	52.6	3.1	0.6

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

APPENDIX C

INFILTRATION TEST RESULTS

Residential Development
West Covina, Los Angeles County, California
Project No. 2409-CR

GEOTEK

PERCOLATION DATA SHEET

Project: 1600-1601 W. CAMERON AVENUE Job No.: 2409-CR

Test Hole No.: I-1 Tested By: DVG Date: 6/13/2020

Depth of Hole As Drilled: 120" Before Test: 120" After Test: 120"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	722		120	20			PRESOAK 30 MIN
	752	30			11	9	WATER REMAINS IN BORING
	754		120	20			PRESOAK 30 MIN
	824	30			11	9	WATER REMAINS IN BORING
	824		120	20			4 HR PRESOAK
	1224	240					
	1224		120	20			DETERMINE TIME INTERVAL
	1254	30			12	8	30 MIN
	1256		120	20			
	126	30			12 3/4	7 1/4	1ST 30 MIN.
	128		120	20			
	158	30			13 1/4	6 3/4	2ND 30 MIN.
	200		120	20			
	230	30			13 3/4	6 1/4	3RD 30 MIN.
	232		120	20			
	302	30			13 3/4	6 1/4	4TH 30 MIN.
	304		120	20			
	334	30			14	6	5TH 30 MIN.
	336		120	20			
	406	30			14	6	6TH 30 MIN.

PERCOLATION DATA SHEET

Project: 1600-1601 W. CAMERON AVENUE Job No.: 2409-CR

Test Hole No.: I-1 Tested By: DVG Date: 6/13/2020

Depth of Hole As Drilled: 120" Before Test: 120" After Test: 120"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	408		120	20			
	438	30			14	6	7TH 30 MIN.
	440		120	20			
	510	30			14 1/4	5 3/4	8TH 30 MIN.

Project: 1600-1616 W CAMERON AVENUE Job No.: 2409-CR

Test Hole No.: I-2 Tested By: DVG Date: 6/13/2020

Depth of Hole As Drilled: 180 Before Test: 180" After Test: 180"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	715		180	20			PRESOAK 30 MIN
	745	30			8	12	WATER REMAINS IN BORING
	747		180	20			PRESOAK 30 MIN
	817	30			8	12	WATER REMAINS IN BORING
	817		180	20			4HR PRESOAK
	1217	240					
	1217		180	20			DETERMINE TIME INTERVAL
	1247	30			9	11	
	1249		180	20			
	119	30			9	11	1ST 30 MIN.
	121		180	20			
	151	30			9 1/4	10 3/4	2ND 30 MIN.
	153		180	20			
	223	30			10 1/4	9 3/4	3RD 30 MIN.
	225		180	20			
	255	30			11	9	4TH 30 MIN.
	257		180	20			
	327	30			11 1/2	8 1/2	5TH 30 MIN.
	329		180	20			
	359	30			11 1/2	8 1/2	6TH 30 MIN.

PERCOLATION DATA SHEET

Project: 1600-1616 W. CAMERON AVENUE Job No.: 2409-CR

Test Hole No.: I-2 Tested By: DVG Date: 6/13/2020

Depth of Hole As Drilled: 180" Before Test: 180" After Test: 180"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
	401		180	20			
	431	30			11 3/4	8 1/4	7TH 30 MIN.
	433		180	20			
	503	30			11 3/4	8 1/4	8TH 30 MIN

APPENDIX D

SEISMIC SETTLEMENT ANALYSIS

Residential Development

West Covina, Los Angeles County, California

Project No. 2409-CR

GEOTEK

****

LIQUEFACTION ANALYSIS SUMMARY

****

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Input File Name: UNTITLED
Title: West Covina
Subtitle: 2409-CR

Surface Elev.=
Hole No.=B-5
Depth of Hole= 30.00 ft
Water Table during Earthquake= 75.00 ft
Water Table during In-Situ Testing= 75.00 ft
Max. Acceleration= 0.78 g
Earthquake Magnitude= 6.95

Input Data:

Surface Elev.=
Hole No.=B-5
Depth of Hole=30.00 ft
Water Table during Earthquake= 75.00 ft
Water Table during In-Situ Testing= 75.00 ft
Max. Acceleration=0.78 g
Earthquake Magnitude=6.95
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= 1.15
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	7.00	120.00	60.00
7.00	13.00	125.00	45.00
10.00	10.00	125.00	45.00
14.50	13.00	125.00	5.00
18.00	53.00	130.00	5.00
23.00	40.00	130.00	32.00
29.50	45.00	130.00	5.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.50 in.
Total Settlement of Saturated and Unsaturated Sands=0.50 in.
Differential Settlement=0.248 to 0.328 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	0.32	0.50	5.00	0.00	0.50	0.50
1.00	0.36	0.50	5.00	0.00	0.49	0.49
2.00	0.42	0.50	5.00	0.00	0.49	0.49
3.00	0.61	0.50	5.00	0.00	0.49	0.49
4.00	0.61	0.50	5.00	0.00	0.48	0.48
5.00	0.61	0.50	5.00	0.00	0.47	0.47
6.00	0.61	0.50	5.00	0.00	0.47	0.47
7.00	0.61	0.50	5.00	0.00	0.46	0.46
8.00	0.61	0.49	5.00	0.00	0.44	0.44
9.00	0.61	0.49	5.00	0.00	0.40	0.40
10.00	0.42	0.49	5.00	0.00	0.37	0.37
11.00	0.43	0.49	5.00	0.00	0.34	0.34
12.00	0.39	0.49	5.00	0.00	0.32	0.32
13.00	0.34	0.49	5.00	0.00	0.28	0.28
14.00	0.29	0.49	5.00	0.00	0.21	0.21
15.00	0.61	0.49	5.00	0.00	0.13	0.13
16.00	0.61	0.49	5.00	0.00	0.11	0.11
17.00	0.61	0.48	5.00	0.00	0.10	0.10
18.00	0.61	0.48	5.00	0.00	0.09	0.09
19.00	0.61	0.48	5.00	0.00	0.09	0.09
20.00	0.61	0.48	5.00	0.00	0.08	0.08
21.00	0.61	0.48	5.00	0.00	0.07	0.07
22.00	0.61	0.48	5.00	0.00	0.07	0.07
23.00	0.61	0.48	5.00	0.00	0.06	0.06
24.00	0.61	0.48	5.00	0.00	0.05	0.05
25.00	0.61	0.47	5.00	0.00	0.05	0.05

26.00	0.61	0.47	5.00	0.00	0.04	0.04
27.00	0.61	0.47	5.00	0.00	0.03	0.03
28.00	0.60	0.47	5.00	0.00	0.02	0.02
29.00	0.60	0.47	5.00	0.00	0.01	0.01
30.00	0.60	0.47	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 E

GENERAL GRADING GUIDELINES

Residential Development
West Covina, Los Angeles County, California
Project No. 2409-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 Uniform 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.

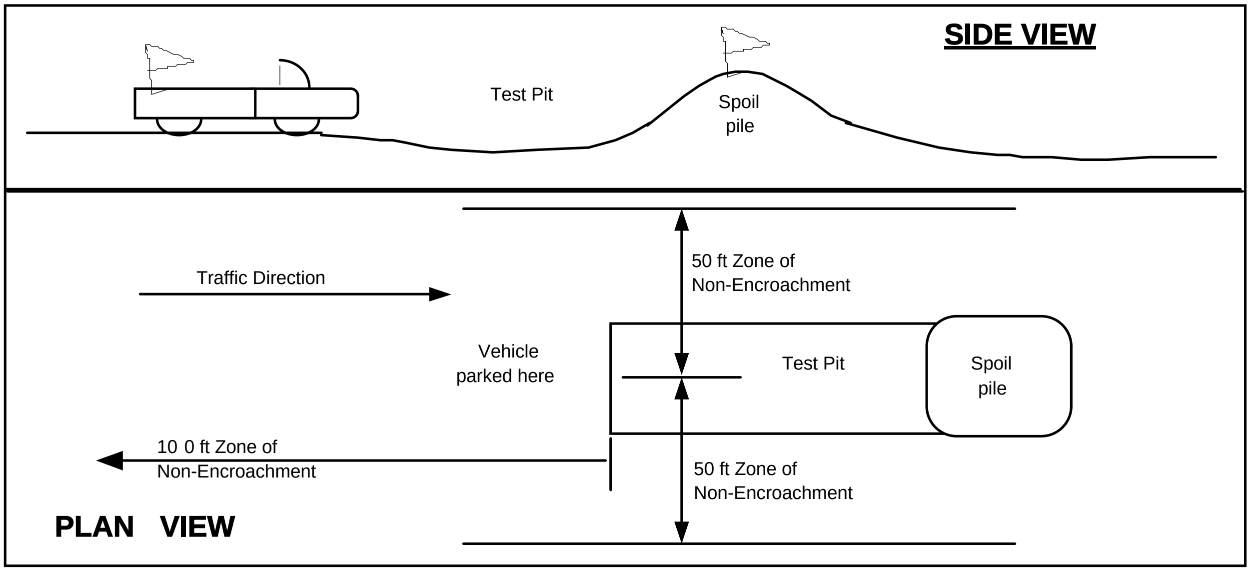
Test Pits Location, Orientation and Clearance

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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