Appendix E: Geotechnical and Infiltration Evaluation

THIS PAGE INTENTIONALLY LEFT BLANK

February 25, 2021

Mr. Cesar Morales, PE
CITY OF ANAHEIM PUBLIC WORKS DEPARTMENT
200 S. Anaheim Boulevard, Suite 276
Anaheim, CA 92805 GMU Project 20-286-00

Subject: Third Party Geotechnical Review, Proposed New Mult-Family Residential Development, 110 and 200 West Midway Drive, City of Anaheim

Documents Reviewed:

1. "Second Response to City of Anaheim Department of Public Works, 200 W. Midway Drive, Anaheim, OTH 2020-01299, dated December 2, 2020.

Dear Mr. Morales:

We have completed our review of the above geotechnical document. The project is **conditionally approved** from a geotechnical point of view. Our comments are provided below.

1. The geotechnical consultant shall review the precise grading plan and foundation plans for conformance with their geotechnical recommendations. Any revised or additional geotechnical recommendations should be provided in a supplemental letter, as necessary. The supplemental letter should include a discussion of whether improvements proposed at or near the property lines will need to be designed using lower soil bearing capacity of deepened footings due to partial soil recompaction. This item may be a **Condition of Approval**.

2. The recommended removal depths should be shown on the precise grading plans. This item may be a **Condition of Approval.**

3. When the final location and depths of the infiltration system are known, a design level infiltration study should be performed and submitted to the City. This item may be a **Condition of Approval**.

23241 Arroyo Vista
Rancho Santa Margarita | CA 92688
949.888.6513 | FX: 949.888.1380 | www.gmugeo.com

Should there be any questions regarding the content of this letter, please feel free to contact us.

Respectfully submitted,

Lisa L. Bates, PG, CEG 2293
Associate Engineering Geologist

David Hansen, M.Sc., PE, GE 3056
Associate Geotechnical Engineer

Electronic copy submitted

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

January 15, 2021
Project No. 2074-CR

Encore Capital Management
11766 Wilshire Boulevard, Suite 1470
Los Angeles, California 90025

Attention: Mr. David Hardy

Subject: Second Response to City of Anaheim Department of Public Works, 200 W. Midway Drive, Anaheim, OTH 2020-01299, dated December 2, 2020.

References: See Page 7

Dear Mr. Hardy:

GeoTek, Inc. (GeoTek) herein presents our responses to the City of Anaheim Department of Public Works Review Letter, dated December 2, 2020, with respect to the referenced GeoTek's *Geotechnical and Infiltration Evaluation* (2019) for the subject development, in the city of Anaheim, California. Our responses are as follow:

Comment 1:

The report submitted is greater than 1 year old and was prepared per the outdated 2016 CBC. Please provide a current letter or report discussing current site conditions and providing updated recommendations per the current 2019 California Building Code. The letter or report should include a review of the precise grading plans for the subject site with revised or additional recommendations, as necessary, including 2019 CBC seismic design parameters.

Response:

A site visit was made by an engineer from GeoTek on January 14, 2020 to observe current site conditions. The site was empty of RV vehicles and the previously existing mature trees were cut. The office building and associated structures (restrooms, pool and pool building, among others) as well as parking and hardscape areas were still present. In general, the existing site geotechnical conditions appeared to be similar to the previously described conditions in the referenced *Geotechnical and Infiltration Evaluation* report (GeoTek, 2019). As noted in our previous *Response Letter to City Comments* (GeoTek, 2020), seismic design parameters per the 2019 CBC and updated seismic settlement analysis were provided for project design. Rough/precise grading plans have not been provided for our evaluation. When such plans are available, we will review them and offer additional recommendations, if required.

Comment 2:

If the depths and locations of the future infiltration system vary significantly from the locations and depths that were tested, please provide an addendum design-level infiltration study.

Response:

We concur with the remark. When the final locations and depths of the infiltration system are known, we will assess the need for a design-level infiltration study.

Comment 3:

Soft silt and clay materials are indicated in the boring logs B-4 and B-5 at a depth of 7.5 feet; however, no consolidation tests were performed. Once building loads are known, please provide updated settlement estimate amounts in your updated letter or report and provide supporting laboratory analysis or discussion.

Response:

The referenced *Geotechnical Evaluation* (GeoTek, 2019) recommended that all existing fills and the upper loose/soft portions of the alluvium be removed to expose competent alluvium. Competent alluvium is defined as native materials that are visually relatively non-porous and having a relative compaction of at least 85 percent of the soil's maximum dry density per ASTM D 1557. In general, removals on the order of 5 feet appear to be adequate for the site. If the bottom of removals encounters soft silts/clays with an in-place relative compaction of less than 85 percent, these soils will be removed and recompacted.

If soft silts/clays remain below the depths of removals, such as the case of borings B-4 and B-5 at 7.5 feet, we have evaluated the potential static settlement of structural foundations. We have assumed a load of 5 kips/ft for strip footings and 50 kips for pad footings for the proposed three-story residential buildings. Based on an allowable soil bearing capacity of 1,800 psf, a strip footing about 2.8 feet wide and a pad footing about 5.5 feet square were used in our assessment. A footing depth of 1.0 foot was also assumed.

The settlement evaluation was performed utilizing the consolidation theory. Available correlations with the blow counts were utilized to assess the coefficient of compressibility (Cc) of the soils within the depth of influence of the footings. Also, our analyses considered that the upper 5 feet from existing grade or 3 feet below footing bottom, whichever is deeper, will be removed and replaced as engineered compacted fill, as recommended in the referenced *Geotechnical Evaluation* report (GeoTek, 2019). Our analyses indicate that the footings may be subject to a total static settlement of less than 1 inch. Differential static settlement is estimated to be less than 0.5 inch over a horizontal distance of 40 feet. It should also be noted that a soft soil layer at a depth of 7 feet below existing grade or 6 feet below the bottom of a footing (assuming a footing depth of 1 foot) is only subject to 30 percent of the footing load, decreasingly rapidly with depth.

Detailed static settlement analyses are attached to this letter.

Comment 4:

The consultant recommends an approximate 5-foot over-excavation and re-compaction for all structural areas, but only 12 inches of new fill below the proposed asphalt and concrete pavement areas. Please clarify if the structural areas include site walls and other outlying structures. Please confirm that the reduced remedial recommendations for the asphalt and concrete pavement areas will provide adequate support for the planned improvements and mitigate distress due to settlement.

Response:

Excavations of the upper 5 feet from existing grade or 3 feet below footing bottom, whichever is deeper, is recommended to be implemented within all building and wall areas.

Since pavement and flatwork improvements are light in nature, recompaction of the upper 12 inches of subgrade appear to be generally adequate. However, because of the potential for distress of the proposed improvements due to settlement of the existing undocumented fill (2-3 feet in thickness), consideration should be given to also excavate the old fill under surface improvement areas.

Comment 5:

Since proposed depths of over-excavation will range up to 5 feet in depth, please address temporary excavations along the property lines. How will the excavations be laid back to protect the adjacent properties and yet still provide proper compaction below proposed improvements? Will temporary shoring be required? If so, show the proposed locations on the precise grading plans.

Response:

To safeguard improvements on adjacent properties, we recommend that removals start 3 feet away from the property lines and go down at a 1:1 (h:v) inclination to the recommended depth of soil recompaction (5 feet). Perimeter walls and other improvements proposed at or near property lines, where partial soil recompaction may be performed, could be designed using a lower soil bearing capacity or deepened footings embedded into competent native soils, if necessary.

The above recommendations are preliminary. More refined recommendations will be provided when site grading and improvement plans are available.

Comment 6:

The pavement recommendations of the report are based on an assumed R-value; therefore, at the completion of grading, an R-value test of the actual subgrade soils within the pavement areas should be performed and the pavement recommendations revised, as necessary. This item may be a Condition of Approval.

Response:

We concur with the remark. R-value testing of the actual road subgrade will be conducted and previously recommended pavement sections will be re-evaluated.

Comment 7:

At the completion of grading, the actual expansion index of the subgrade soils below the building foundations should be confirmed by testing and the foundation recommendations revised, as necessary. If the EI is found to be greater than 20, the effective PI should also be provided. This item may be Condition of Approval.

Response:

EI of the near surface soils will be re-evaluated after the site grading is complete, and foundation parameters will be updated, if necessary.

Comment 8:

With regards to the pool recommendations, it is unlikely that the pool engineer would allow any movement of the pool walls, so the use of an active earth pressure would not appear applicable. Please coordinate with the pool engineer to determine the design of the pool, the method of construction and whether the pool will be provided with a subdrain system. Based on these conditions, provide an updated later earth pressure recommendation. If a subdrain system will be used, describe how it will be outletted since the site is flat (i.e. sump pump, etc.). This item may be a Condition of Approval to obtain the pool permit.

Response:

Values for both active and at-rest earth pressures for pool design were provided in the referenced *Geotechnical and Infiltration Evaluation* report (GeoTek, 2019). Pool plans are not available at this time. Review of plans and coordination with the pool designer will be performed when possible.

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.

REGISTERED GEOLOGIST
EDWARD H. LaMONT
No. 1892
Exp.
Certified Engineering Geologist
STATE OF CALIFORNIA

LICENSED PROFESSIONAL ENGINEER
GABY M. BOGDANOFF
NO. 3133
GEOTECHNICAL
STATE OF CALIFORNIA

Edward H. LaMont
CEG 1892, Exp. 07/31/22
Principal Geologist

Gaby M. Bogdanoff
GE 3133, Exp. 06/30/22
Project Engineer

Distribution: . PDF sent to addressee via email

G:\Projects\2051 to 2100\2074CR Encore Capital Management 110 and 200 Midway Drive Anaheim\Response to City Comments\2074-CR Response to City of Anaheim Department of Public Works Comments 200 W Midway Dr.doc

Enclosures: Static Settlement Analyses

REFERENCES

American Society of Civil Engineers (ASCE), 2017, "Minimum Design Loads for Buildings and Other Structures," ASCE/SEI 7-16.

California Code of Regulations, Title 24, 2019, "California Building Code," 2 volumes.

GeoTek, Inc., 2019, "Geotechnical and Infiltration Evaluation for Proposed Multi-Family Residential Project, 110 and 200 West Midway Drive, Anaheim, Orange County, California," Project No. 2074-CR, dated February 28, 2019.

_______, 2020, "Responses to City of Anaheim Department of Public works, 200W. Midway Drive, Anaheim, OTH 2020-01299," Project No. 2074-CR, dated October 13, 2020.

Static Settlement Estimate

Wall Footing Settlement, using data from Boring B-4

q = 1800 psf
b = 2.8 ft

Ground Surface		Below Footing Base							
From (ft)	To (ft)	From (ft)	To (ft)	Thickness (ft)	σ'o (psf)	Δσ'o (psf)	σ'f (psf)	Cc	S (in)
0	5	0	4	4	287.5	1279.25	1566.7	0.008	0.27
5	7.5	4	6.5	2.5	718.8	580.04	1298.8	0.015	0.12
7.5	10	6.5	9	2.5	1006.3	375.91	1382.2	0.029	0.12
10	15	9	14	5	1437.5	232.71	1670.2	0.025	0.10
15	20	14	19	5	2012.5	146.67	2159.2	0.010	0.02
							Total Settlement =		0.62

Wall Footing Settlement, using data from Boring B-5

q = 1800 psf
b = 2.8 ft

Ground Surface		Below Footing Base							
From (ft)	To (ft)	From (ft)	To (ft)	Thickness (ft)	σ'o (psf)	Δσ'o (psf)	σ'f (psf)	Cc	S (in)
0	5	0	4	4	287.5	1279.25	1566.7	0.008	0.27
5	7.5	4	6.5	2.5	718.8	580.04	1298.8	0.007	0.05
7.5	10	6.5	9	2.5	1006.3	375.91	1382.2	0.033	0.14
10	15	9	14	5	1437.5	232.71	1670.2	0.018	0.07
15	20	14	19	5	2012.5	146.67	2159.2	0.008	0.02
							Total Settlement =		0.55

Static Settlement Estimate

Pad Footing Settlement, using data from Boring B-4

q = 1800 psf
b = 5.5 ft

Ground Surface		Below Footing Base							
From (ft)	To (ft)	From (ft)	To (ft)	Thickness (ft)	σ'o (psf)	Δσ'o (psf)	σ'f (psf)	Cc	S (in)
0	5	0	4	4	287.5	1522.08	1809.6	0.008	0.29
5	7.5	4	6.5	2.5	718.8	625.24	1344.0	0.015	0.13
7.5	10	6.5	9	2.5	1006.3	339.08	1345.3	0.029	0.11
10	15	9	14	5	1437.5	167.82	1605.3	0.025	0.07
15	20	14	19	5	2012.5	84.74	2097.2	0.010	0.01
							Total Settlement =		0.61

Pad Footing Settlement, using data from Boring B-5

q = 1800 psf
b = 5.5 ft

Ground Surface		Below Footing Base							
From (ft)	To (ft)	From (ft)	To (ft)	Thickness (ft)	σ'o (psf)	Δσ'o (psf)	σ'f (psf)	Cc	S (in)
0	5	0	4	4	287.5	1522.08	1809.6	0.008	0.29
5	7.5	4	6.5	2.5	718.8	625.24	1344.0	0.007	0.05
7.5	10	6.5	9	2.5	1006.3	339.08	1345.3	0.033	0.13
10	15	9	14	5	1437.5	167.82	1605.3	0.018	0.05
15	20	14	19	5	2012.5	84.74	2097.2	0.008	0.01
							Total Settlement =		0.54

September 3, 2020

Ms. Esperanza Rios
CITY OF ANAHEIM PUBLIC WORKS DEPARTMENT
200 S. Anaheim Boulevard, Suite 276
Anaheim, CA 92805

GMU Project 20-286-00

Subject: Third Party Geotechnical Review, Proposed New Multi-Family Residential Development, 110 and 200 West Midway Drive, City of Anaheim

Documents Reviewed:

1. *Geotechnical and Infiltration Evaluation for Proposed Multi-Family Residential Project, 110 and 200 West Midway Drive, Anaheim, Orange County, California*, prepared by Geotek, Inc., dated February 28, 2019.

2. *Preliminary Grading Plan, Tentative Tract Map No. 19112, For Condominium Purposes, 200. W. Midway Drive, City of Anaheim, County of Orange, State of California;* prepared by C&V Consulting, Inc., dated May 29, 2020.

Dear Ms. Rios:

We have completed our review of the above geotechnical documents. The status of our review is **not approved**. The comments below shall be addressed as part of the entitlement process:

1. The report submitted is greater than 1 year old and was prepared per the outdated 2016 CBC. Please provide a current letter or report discussing current site conditions and providing updated recommendations per the current 2019 California Building Code. The letter or report should include a review of the precise grading plans for the subject site with revised or additional recommendations, as necessary, including 2019 CBC seismic design parameters.

2. If the depths and locations of the future infiltration system vary significantly from the locations and depths that were tested, please provide an addendum design-level infiltration study.

3. Soft silt and clay materials are indicated in the boring logs B-4 and B-5 at a depth of 7.5 feet; however, no consolidation tests were performed. Once building loads are known, please provide updated settlement estimate amounts in your updated letter or report and provide supporting laboratory analysis or discussion.

4. The consultant recommends an approximate 5-foot over-excavation and re-compaction for all structural areas, but only 12 inches of new fill below the proposed asphalt and concrete pavement areas. Please clarify if the structural areas include site walls and other outlying structures. Please confirm that the reduced remedial recommendations for the asphalt and concrete pavement areas will provide adequate support of the planned improvements and mitigate distress due to settlement.

5. Since proposed depths of over-excavation will range up to 5 feet in depth, please address temporary excavations along the property lines. How will the excavations be laid back to protect the adjacent properties and yet still provide proper compaction below proposed improvements? Will temporary shoring be required? If so, show the proposed locations on the precise grading plans.

6. The pavement recommendations of the report are based on an assumed R-value; therefore, at the completion of grading, an R-value test of the actual subgrade soils within the pavement areas should be performed and the pavement recommendations revised, as necessary. This item may be a Condition of Approval.

7. At the completion of grading, the actual expansion index of the subgrade soils below the building foundations should be confirmed by testing and the foundation recommendations revised, as necessary. If the EI is found to be greater that 20, the effective PI should also be provided. This item may be a Condition of Approval.

8. With regards to the pool recommendations, it is unlikely that the pool engineer would allow any movement of the pool walls, so the use of an active earth pressure would not appear applicable. Please coordinate with the pool engineer to determine the design of the pool, the method of construction, and whether the pool will be provided with a subdrain system. Based on these conditions, provide an updated lateral earth pressure recommendation. If a subdrain system will be used, describe how it will be outletted since the site is flat (i.e., sump pump, etc.). This item may be a Condition of Approval to obtain the pool permit.

Should there be any questions regarding the content of this letter, please feel free to contact us.

Respectfully submitted,

PROFESSIONAL GEOLOGIST ★ LISA L. BATES ★ No. 2293 CERTIFIED ENGINEERING GEOLOGIST ★ STATE OF CALIFORNIA

Lisa L. Bates, PG, CEG 2293
Associate Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER ★ DAVID W. HANSEN ★ GE 3056 ★ GEOTECHNICAL ★ STATE OF CALIFORNIA

David Hansen, M.Sc., PE, GE 3056
Associate Geotechnical Engineer

Electronic copy submitted

GEOTECHNICAL AND INFILTRATION EVALUATION
FOR
PROPOSED MULTI-FAMILY RESIDENTIAL PROJECT
110 AND 200 WEST MIDWAY DRIVE
ANAHEIM, ORANGE COUNTY, CALIFORNIA

PREPARED FOR

ENCORE CAPITAL MANAGEMENT
11766 WILSHIRE BOULEVARD, SUITE 1470
LOS ANGELES, CALIFORNIA 90025

PREPARED BY

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

PROJECT NO. 2074-CR **FEBRUARY 28, 2019**

GEOTEK

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

February 28, 2019
Project No. 2074-CR

Encore Capital Management
11766 Wilshire Boulevard, Suite 1470
Los Angeles, California 90025

Attention: Mr. David Hardy

Subject: Geotechnical and Infiltration Evaluation
Proposed Multi-Family Residential Project
110 and 200 West Midway Drive
Anaheim, Orange County, California

Dear Mr. Hardy:

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

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

Respectfully submitted,
GeoTek, Inc.

REGISTERED GEOLOGIST
EDWARD H. LaMONT
No. 1892
Exp.
Certified Engineering Geologist
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
GABY M. BOGDANOFF
C 66619
EXP. 06/30/20
CIVIL
STATE OF CALIFORNIA

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

Gaby Bogdanoff
C 66619, Exp. 06/30/20
Project Engineer

Distribution: (1) Addressee via email (one .pdf file)
G:\Projects\2051 to 2100\2074CR Encore Capital Management 110 and 200 Midway Drive Anaheim\Geotechnical Report\2074-CR Geotechnical and Infiltration Evaluation 110 and 200 West Midway Drive Anaheim.docx

TABLE OF CONTENTS

TABLE OF CONTENTS

1. PURPOSE AND SCOPE OF SERVICES

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

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

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

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The site is located and addressed as 110 and 200 West Midway Drive in the city of Anaheim, Orange County, California. The property is comprised of approximately 5.9 acres of land and is currently occupied by the Anaheim Resort RV Park. Various improvements such as an office building, three restroom buildings, a pool, driveways and parking areas, underground utilities, as well as flatwork and landscape improvements exist on site.

The property is generally flat with a fall of about three to four feet to the west. Surface drainage is to the west, with local variations.

The site is bounded by Midway Drive to the north with an elementary school beyond, a storage yard to the east, and a mobile home park to the west and south. The general location of the site is shown in Figure 1.

2.2 PROPOSED DEVELOPMENT

At the time of the preparation of this report, no site development plans were available. However, we understand that the site development will likely include earthwork and construction necessary for a multi-family residential project consisting of several townhomes, parking/drive areas, and landscape improvements. It is anticipated that the structures will be two to three stories, of wood-framed construction, and will utilize concrete slab-on-grade floors and shallow foundations. Cuts and fills are estimated to be less than five feet in height. In addition, we anticipate that stormwater at the site may be managed via relatively shallow infiltration systems to be located within future common areas of the development. Specific location and depth of these systems are currently unknown. During this evaluation, however, we considered infiltration tests at two site locations at depths of five feet.

3. FIELD EXPLORATION, LABORATORY TESTING, AND PERCOLATION TESTING

3.1 FIELD EXPLORATION

The soils underlying the site were explored on February 8, 2019 by means of excavating six exploratory borings (B-1 through B-6) within the anticipated building areas to depths ranging from 21.5 to 51.5 feet below the existing ground surface. In addition, two percolation test borings (P-1 and P-2) were excavated approximately five feet each within the future storm water infiltration areas within the northwestern and southwestern portions of the site. All borings were drilled with a truck-mounted hollow-stem auger drill rig.

The approximate locations of our site explorations are shown on the Site Exploration Map, Figure 2. The Site Exploration Map uses a Google Earth site image as a basemap. 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. Results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included in Appendix B.

3.3 PERCOLATION TESTING

Percolation testing was performed at boring locations P-1 and P-2 to assess the infiltration characteristics of the site soils underlying future storm water system areas. Test locations were chosen near topographic low regions of the property. Percolation test borings were excavated to approximately five feet below the existing grade, which was assumed to be the approximate invert depth of the infiltration systems. The boring diameter was approximately 8 inches. Percolation testing was performed within the lower 20 inches in the borings by a representative of our firm, in general conformance with the Boring Percolation Test Procedure outlined in the *Technical Guidance Document Appendix VII* (Orange County, 2011).

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

SUMMARY OF TEST RESULTS		
Percolation Test	Measured Percolation Rate (inches per hour)	Measured Infiltration Rate (inches per hour)
P-1	160.0	26.6
P-2	150.0	25.0

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

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The site is situated in the Peninsular Ranges province, which is one of the largest geomorphic units in western North America. Basically, it extends from the Transverse Ranges geomorphic province and the Los Angeles Basin, approximately 900 miles south 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. Three major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, Morton, D.M., and Miller, F.K., (2006) map the site to be underlain by Quaternary age younger alluvial fan deposits. Additionally, the nearest known active fault to the site is the Whittier Fault located approximately 9.5 miles to the northeast.

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the earth materials encountered on the site is presented in the following sections.

4.2.1 Undocumented Artificial Fill

Undocumented artificial fill was encountered within all of our exploratory borings and ranged from two to three feet in thickness. The fill consisted of brown, moist, silty fine to coarse grained sand.

4.2.2 Younger Alluvial Fan Deposits

Younger alluvial fan deposits were encountered in all our borings below the fills and extended to the maximum depth explored of about 51.5 feet. The alluvium observed generally consisted of silty fine to coarse grained sands with lesser layers of clayey silts and silty clays at depths. The alluvial units were olive gray to brown in color, slightly moist to moist, and had soft/loose to stiff/dense in-situ condition, based on our field observations, blow counts, and in-place density determinations.

The near surface site soils tested were found to have a "very low" expansion potential, when tested and classified in accordance with ASTM D 4829.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

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

4.3.2 Groundwater

Water was not encountered in our exploratory excavations. As noted within the *Seismic Hazard Zone Report for the Anaheim Quadrangle* (California Department of Conservation, 1997), historic high groundwater is mapped at greater than 50 feet below ground surface.

It is possible that seasonal variations (temperature, rainfall, etc.) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered in discontinuous zones within the alluvium and/or overburden. The groundwater levels presented in this report are the levels that were measured at the time of our field activities or as stated in the referenced source. It is recommended that the contractor determine the actual groundwater levels at the site at the time of the construction activities to determine the impact, if any, on the construction procedures.

4.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced liquefaction or landsliding. The nearest zoned fault is the Newport-Inglewood-Rose Canyon fault zone, north Los Angeles basin section. The Los Angeles section is located approximately 0.25 miles to the west.

4.4.1 Seismic Design Parameters

The site is located at approximately 33.812005 Latitude and -117.909200 Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, was determined from the USGS Website, Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude/Longitude. The results are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.5g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.554g
Site Coefficient for Site Class "D", F_a	1.0
Site Coefficient for Site Class "D", F_v	1.5
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.5g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.831g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.0g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.554g
Peak Ground Acceleration Adjusted for Site Class Effects, PGA_M	0.5g

4.5 LIQUEFACTION AND SEISMICALLY-INDUCED SETTLEMENT

The groundwater beneath the site is located at a depth greater than 50 feet. Therefore, the potential for soil liquefaction at the property is considered to be very low.

Seismically-induced settlement at the property is anticipated to be less than 0.5 inches total and 0.25 inches differential in a horizontal distance of 40 feet.

4.6 OTHER SEISMIC HAZARDS

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

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

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

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

5.2 EARTHWORK CONSIDERATIONS

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

5.2.1 Site Clearing and Preparation

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

5.2.2 Removals

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

A minimum 36 inches of engineered fill should be provided below the bottom of the proposed foundations. A representative of this firm should observe the bottom of all excavations.

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

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

The bottom of all removals should be scarified to a minimum depth of 12 inches, brought to slightly above the optimum moisture content, and then recompacted to at least 90 percent of the soil's maximum dry density (ASTM D 1557). The bottoms of removals should be observed by a GeoTek representative prior to scarification.

5.2.3 Fills

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

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

5.2.4 Excavation Characteristics

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

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

5.2.5 Shrinkage and Subsidence

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

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

5.2.6 Trench Excavations and Backfill

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

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

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

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

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Our laboratory test results showed that the site soils have "very low" ($0 \le EI \le 20$) expansion potential in accordance with ASTM D 4829. However, this should be verified by additional testing after the site is rough graded.

The foundation elements for the proposed structures should bear entirely in engineered fill soils and should be designed in accordance with the *2016 California Building Code (CBC)*.

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

DESIGN PARAMETERS FOR CONVENTIONALLY-REINFORCED SPREAD FOUNDATIONS	
Design Parameter	"Very Low" Expansion Potential
Foundation Depth or Minimum Perimeter Beam Depth for Both Interior and Exterior Footings (inches below lowest adjacent finished grade)	One- and Two-Story – 12 Three-Story – 15
Minimum Foundation Width (Inches)*	One- and Two-Story – 12 Three-Story – 18
Minimum Slab Thickness (inches)	4 (actual)
Sand Blanket and Moisture Retardant Membrane below On-Grade Building Slabs	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric or No. 3 rebars at 24 inches on center placed in middle of slab
Minimum Footing Reinforcement for Continuous Footings, Grade Beams, and Retaining Wall Footings	Two No. 4 reinforcing bars, one top and one bottom
Effective Plasticity Index***	NA
Presaturation of Subgrade Soil (Percent of Optimum/Depth in inches)	Minimum 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete.

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

In general, an allowable bearing capacity of 1,800 pounds per square foot (psf) may be used for footings a minimum 12 inches deep and 12 inches wide. This value may be increased by 400 psf for each additional 12 inches in depth and 100 psf for each additional 12 inches in width to a maximum value of 3,000 psf.

The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.40 may be used with dead load forces. The upper one foot of soil below the adjacent grade should not be used in calculating passive pressure.

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

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

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

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

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

5.3.2 Miscellaneous Foundation Recommendations

- To minimize moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete, or concrete slurry where they intercept the perimeter footing or thickened slab edge.
- Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.
- Under-slab utility trenches should be compacted to project specifications. Compaction should be achieved with a mechanical compaction device. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

5.3.3 Foundation Set Backs

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

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

5.3.4 Retaining Wall Design and Construction

5.3.4.1 General Design Criteria

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

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

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

Earthwork considerations, site clearing, and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of *Section 5.3.4.4* in this report.

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

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

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

5.3.4.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) Onsite Materials*
Level	40
2:1	62

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

5.3.4.3 Restrained Retaining Walls

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

5.3.4.4 Retaining Wall Backfill and Drainage

Retaining wall backfill should be free of deleterious and/or oversized materials and should have properties indicated in *Section 5.3.4.2*. Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one-cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

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

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

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

5.3.4.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.

- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved the project geotechnical engineer or their authorized representative.

5.3.4.6 Pool Construction

The proposed swimming pool should derive support entirely from engineered fill. A minimum 12 inches of fill compacted to at least 90 percent of the soil's maximum dry density per ASTM D 1557 should be provided below the pool shell.

The pool walls should be designed for at-rest soil conditions using an equivalent fluid pressure of 60 pcf for at-rest conditions. Pool walls surcharged by adjacent structures should be designed for additional pressures. Alternatively, the pool walls may be designed as freestanding walls using the active soil state conditions provided that some lateral movement of the pool walls would be acceptable. If the active state is to be used, an equivalent fluid pressure of 40 pcf is considered suitable. These recommended pressures are based on drained conditions and using granular native soils (EI<20) as wall backfill. If a drain system adjacent/beneath the pool is not provided, the pool walls should then be designed for an equivalent fluid pressure of 97 pcf for the at-rest condition and 85 pcf for the active condition.

As noted above, the use of the lower (drained condition) at-rest or active soil pressures will require a subdrain system beneath/adjacent to the pool. A typical subdrain system includes a series of four-inch diameter perforated drain pipes encapsulated with at least one cubic foot of free-draining material per linear foot of pipe. The free-draining material should be encapsulated within a geotextile to prevent migration of fines into the drainage medium. The drain pipes should be routed to an acceptable discharge location, as determined by the civil engineer/pool designed. If desired, GeoTek can review the subdrain system once designed to determine if additional measures are warranted.

Pool decking supported on grade should be separated from the pool bond beam by a full-depth, mastic construction joint. If it is desired to extend the pool deck over the bond beam, consideration should be given to designing the deck as a structural slab supported by the pool shell. This will reduce the possibility of deck cracking occurring along the outer edge of the bond beam. We also recommend that the pool decking subgrade be "pre-saturated" prior to concrete placement. The subgrade soils should be moisture conditioned to at least 100 percent of the soil's optimum moisture content to a depth of 12 inches, prior to concrete placement. Testing by the geotechnical engineer is recommended to confirm that the soils have been adequately moisture treated.

Pool decking may consist of 5-inch thick concrete and the use of reinforcement is suggested. A minimum of #3 bars spaced 24 inches or 6" x 6" – W1.4/W1.4 welded wire mesh placed on center may be used. Control joints should be placed in two directions and located a distance apart approximately equal to 24 to 36 times the slab thickness. The project structural engineer should provide final design recommendations.

5.3.5 Pavement Design Considerations

Pavement design for proposed on-site parking and drive areas was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. Based on an assumed design R-value of 40 and for Traffic Indices (TIs) of 5.0 and 6.0 generally associated with this type of projects, the following preliminary sections were calculated:

GEOTECHNICAL RECOMMENDATION FOR MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0	4*	4*
6.0	4*	5

*Minimum thickness required for private street sections per Standard Detail No. 162 (City of Anaheim, 2004)

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

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

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

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

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

5.3.6 Soil Corrosivity

The soil resistivity was tested in the laboratory on two samples collected during our field exploration. The results of the testing (16,080 and 19,430 ohm-cm) indicate that the soil samples are "mildly corrosive" to buried ferrous metals, based on the guidelines provided in *Corrosion Basics: An Introduction* (Roberge, 2005). Results of chloride analyses (12 and 15 ppm) suggest that the on-site soils have negligible chloride concentrations. Consideration should be given to consulting with a corrosion engineer.

5.3.7 Soil Sulfate Content

The sulfate content was determined in the laboratory for representative soil samples obtained during our field exploration. The results (0.0027 and 0.0036 percent) indicate that the water-soluble sulfate range is less than 0.1 percent by weight which is considered "not applicable" (i.e. negligible) as per Table 4.2.1 of ACI 318. Based upon the test results, no special concrete mix design is required by Code for sulfate attack resistance. Additional testing of soils collected near finish grade should be performed subsequent to site grading.

5.3.8 Import Soils

Import soils should have an expansion index of less than 20 ("very low"). GeoTek also recommends that, as a minimum, proposed import soils be tested for soluble sulfate content. GeoTek should be notified a minimum of 72 hours of potential import sources so that appropriate sampling and laboratory testing can be performed.

5.3.9 Concrete Flatwork

5.3.9.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks, and driveways should be designed using a four-inch minimum thickness. 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 educational construction.

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

Soils below exterior flatwork should pre-moistened prior to placing concrete. Subgrade soils classified as having "very low" expansion potential should be pre-saturated to a minimum of 100 percent of optimum moisture content to a depth of 12 inches.

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

5.3.9.2 Concrete Performance

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

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

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

5.4 POST CONSTRUCTION CONSIDERATIONS

5.4.1 Landscape Maintenance and Planting

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

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

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

5.4.2 Drainage

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

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

5.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

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

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

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

6. INTENT

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

The scope of our evaluation is limited to the boundaries of the subject site. This review does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our fee estimate (P-0103419) dated January 28, 2019 and geotechnical engineering standards normally used on similar projects in this region.

7. LIMITATIONS

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

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

8. SELECTED REFERENCES

ASCE, 2006, "Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05".

ASTM, 2011, "Soil and Rock: American Society for Testing and Materials", vol. 4.08 and 4.09.

Bryant, W.A., and Hart, E.W., 2007, "Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps", California Geological Survey: Special Publication 42.

California Code of Regulations, Title 24, 2016 "California Building Code", 3 volumes.

California Department of Conservation, 1997, "Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles, Orange County, California", Seismic Hazard Zone Report 03.

_____, 2008, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," Special Publication 117A.

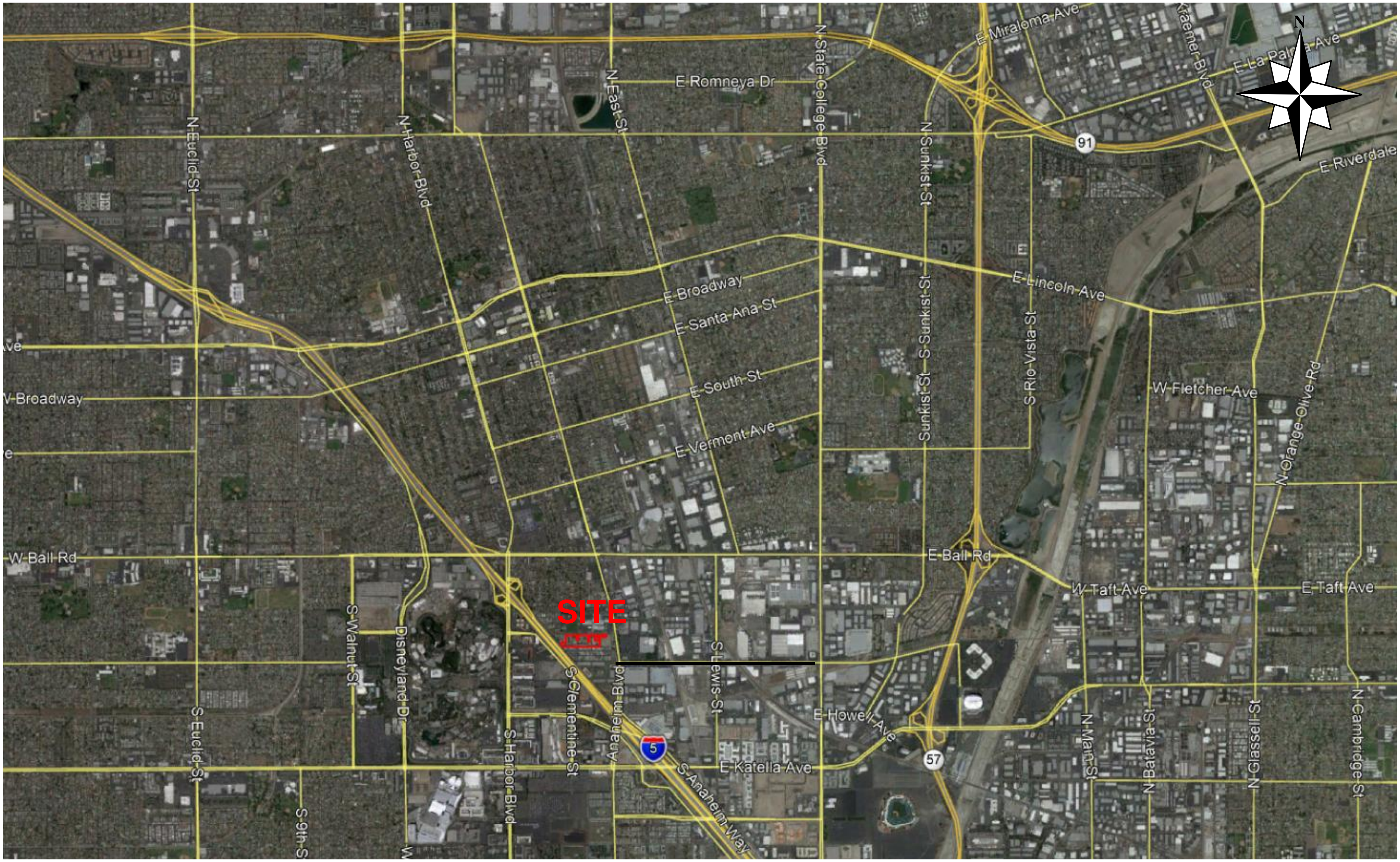
City of Anaheim Department of Public Works, (2004), "Private Street Sections", Standard Detail No. 162, dated January 13

Department of Conservation, 1997, "Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles, Orange County, California", Seismic Hazard Zone Report 03.

Orange County, 2011, "Technical Guidance Document Appendix VII".

Roberge, P.R., 2005, "Corrosion Basics: An Introduction", 2nd Edition.

Seismic Design Values for Buildings (http://earthquake.usgs.gov/research/hazmaps/design).



Encore Capital Management
110 and 200 West Midway Drive
Anaheim, California
Project No. 2074-CR

Scale: As Shown

0 1 mi

Figure 1

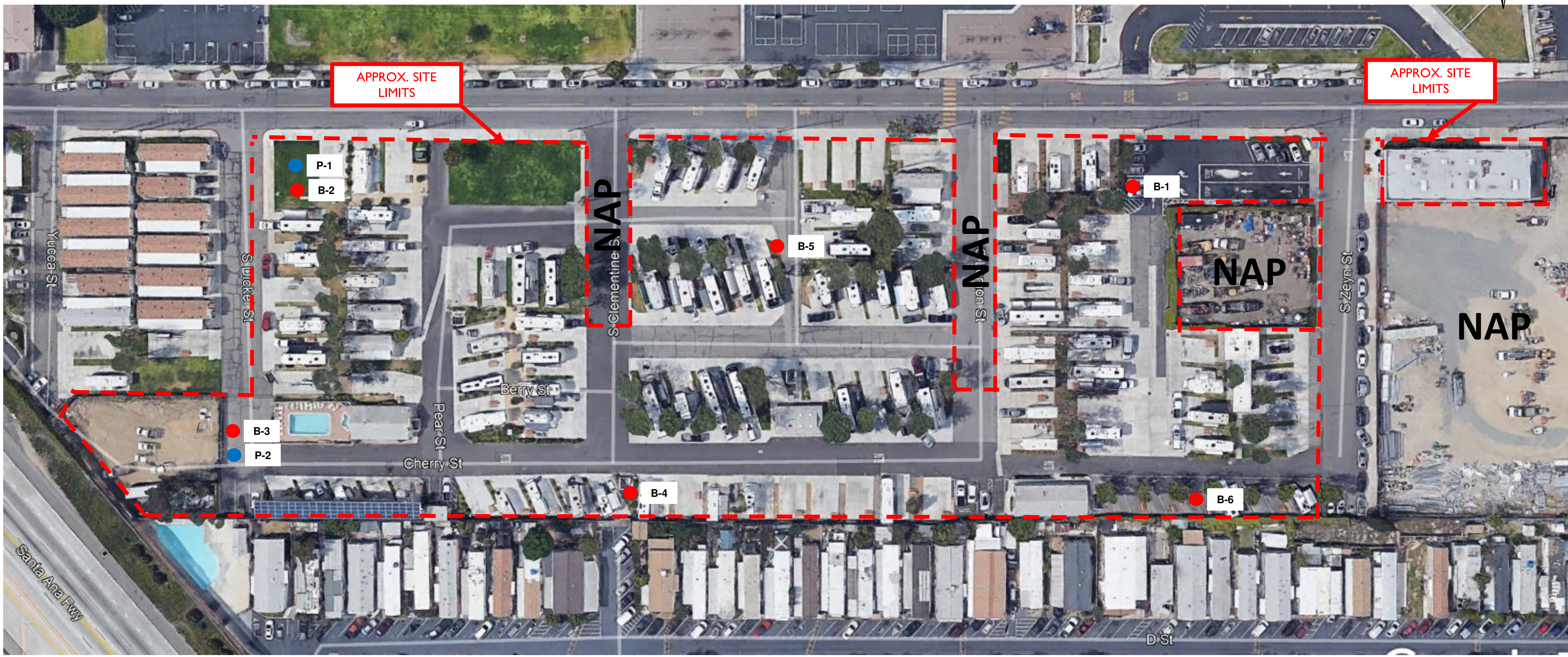
Site Location Map

GEOTEK

LEGEND

- B-6 Approximate Location of Hollow-Stem Auger Boring
- P-2 Approximate Location of Percolation Boring

N



Encore Capital Management

110 and 200 West Midway Drive
Anaheim, Orange County, California

GeoTek Project No. 2074-CR

Scale: As Shown

0 110'

Figure 2

Site Exploration Map

GEOTEK

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Geotechnical and Infiltration Evaluation
110 and 200 West Midway Drive, Anaheim, California
Project No. 2074-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

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

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

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

B – BORING LOG LEGEND

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

SOILS

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

GEOLOGIC

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

BORING NO.: B-1

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					UNDOCUMENTED FILL			
				SM	Silty f-c SAND, brown, moist, some rootlets			
					YOUNGER ALLUVIAL FAN			
				SM	Silty f-c SAND, brown, moist, medium dense			
5	Ring	13 15 12		SP	F-c SAND, light gray, moist, medium dense, trace fine gravel	4.4	114.0	
	Ring	8 9 14			Same as above			
10	Ring	9 10 11			Same as above	4.9	109.0	
15	Ring	9 13		SM	Silty f-c SAND, brown, moist, medium dense	3.7	112.6	
		18		SP	F-m SAND, light gray, slightly moist, medium dense			
20	Ring	8 11 21			F-c SAND, light gray, dry, medium dense			
25	Ring	7 13 17		ML	Clayey SILT to SILT, brown, moist, stiff	11.0	112.0	
30	Ring	11 22 32		SM/ML	Silty f-c SAND to sandy SILT, olive brown, moist, dense/stiff			
					BORING TERMINATED AT 31.5 FEET No groundwater encountered Boring backfilled with 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:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

BORING NO.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
	Large Bulk				**UNDOCUMENTED FILL**			EI, SR
				SM	Silty f-c SAND, brown, moist, some rootlets			
					YOUNGER ALLUVIAL FAN			
				SM/SP	Silty f-c SAND to f-c SAND, light gray, moist, medium dense			
5	Ring	7 8 11		SP	F-c SAND, light gray, moist, loose to medium dense, trace fine gravel	7.8	115.5	
	Ring	5 10 13			Same as above, becomes medium dense	10.8	108.9	
10	Ring	7 11 15		SM/SP	Silty f-c SAND to f-c SAND, light gray, moist, medium dense	8.1	112.3	
15 #	SPT	5 6 7		ML	F sandy SILT, brown, moist to very moist, soft/loose			
20	Ring	5 10 13		ML/CL	Clayey silty to silty CLAY, brown, very moist, stiff, trace fine grained sand	17.2	111.8	
25	SPT	5 7 8		ML	Clayey f sandy SILT, olive brown, moist, soft			
30	Ring	10 21 32		SM	Silty f-c SAND, light gray, moist, dense	7.7	112.9	
					BORING TERMINATED AT 31.5 FEET No groundwater encountered Boring backfilled with 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:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					UNDOCUMENTED FILL			
				SM	Silty f-c SAND, light brown, slightly moist			
					YOUNGER ALLUVIAL FAN			
					Silty f-c SAND, light grayish brown, slightly moist, trace rootlets			
5	Ring	4 5 8		SP	F-c SAND, light gray, moist, loose, trace fine gravel	8.3		
	Ring	5 7 10			Same as above, no recovery			
10	Ring	7 8 12			F-c SAND, light gray, dry to slightly moist, medium dense, some fine gravel	1.8	108.9	
15 #	Ring	10 13 23		SM	Silty f-m SAND, light gray, dry to slightly moist, medium dense			
20	Ring	14 18 25			Same as above	2.4	117.3	
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					UNDOCUMENTED FILL			
				SM	Silty f-m SAND, dark brown, moist, some rootlets			
					YOUNGER ALLUVIAL FAN			
				SM	Silty f-m SAND, grayish brown, moist, trace rootlets			
5	Ring	6 8 10		SM/SP	Silty f-c SAND to f-c SAND, gray to grayish brown, moist, medium dense	7.9	98.1	
	Ring	3 5 7		ML	F sandy SILT to SILT, brown, very moist, soft, trace rootlets	22.9	99.5	
10	Ring	3 6 9			Clayey SILT to SILT, olive brown, very moist, stiff	32.8	99.4	
15 #	Ring	8 17 23		SM/SP	Silty f-c SAND to f-c SAND, light gray, dry to slightly moist, medium dense			
20	Ring	11 13 22			Same as above			
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
	Large Bulk				**UNDOCUMENTED FILL**			MD, EI, SH, SR
				SM/ML	Silty f-m SAND to sandy SILT, brown, moist, some rootlets			
					YOUNGER ALLUVIAL FAN			
				SM	Silty f-c SAND, brownish gray, moist			
5	Ring	23 13 10		SP	F-c SAND, brownish gray, moist, medium dense	5.4	114.3	
	Ring	3 3 5		CL/ML	Silty CLAY to clayey SILT, brown, very moist, soft to medium, trace rootlets	29.0	101.0	
10	Ring	4 7 8		SM	Silty f SAND, brown, very moist, loose/soft	19.5	114.9	
15 #	Ring	7 12 15		SP	F-c SAND, light gray, moist, medium dense	7.0	117.6	
20	Ring	12 14 17			Same as above			
25	Ring	6 10 15		ML	F sandy clayey SILT, brown, very moist, stiff	18.0	115.3	
30	Ring	28 30 22		ML	F sandy SILT, grayish brown, moist, dense			

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:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

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
35	SPT	11 19 21		SP	F-c SAND, light gray, slightly moist, medium dense	3.9	114.4	
40	SPT	14 24		ML	Clayey SILT, olive brown, moist, stiff			
		50/5"		SM	Silty f-m SAND, whitish gray, slightly moist, very dense			
45	SPT	18 37 50		SP	F-c SAND, whitish gray, slightly moist, dense	4.7	116.5	
#								
50	SPT	15 23 30			Same as above			

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:	Encore Capital Management	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	DRW
PROJECT NAME:	110 and 200 Midway Drive	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2074-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME 75
LOCATION:	See Site Exploration Map			**DATE:**	2/8/2019

BORING NO.: B-6

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					UNDOCUMENTED FILL			
				SM/ML	Silty f-m SAND to sandy SILT, brown, moist, some rootlets			
					YOUNGER ALLUVIAL FAN			
				SM	Silty f-c SAND, brownish gray, moist			
5	Ring	7 8 10		SP	F-c SAND, grayish brown, moist, medium dense	4.7	115.6	
	Ring	4 7 8			Same as above, no recovery	4.2	114.0	
10	Ring	4 7 14			F-c SAND, light gray, slightly moist, medium dense, some fine gravel	3.1	119.0	
15 #	Ring	4 7 8		ML	F sandy SILT to SILT, olive brown, moist, stiff			
20	Ring	5 15 20		SM/SP	Silty f-c SAND to f-c SAND, light gray, slightly moist, medium dense	3.0	112.9	
					BORING TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

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

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

APPENDIX B

LABORATORY TEST RESULTS

Geotechnical and Infiltration Evaluation
110 and 200 West Midway Drive, Anaheim, California
Project No. 2074-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

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

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

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

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

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

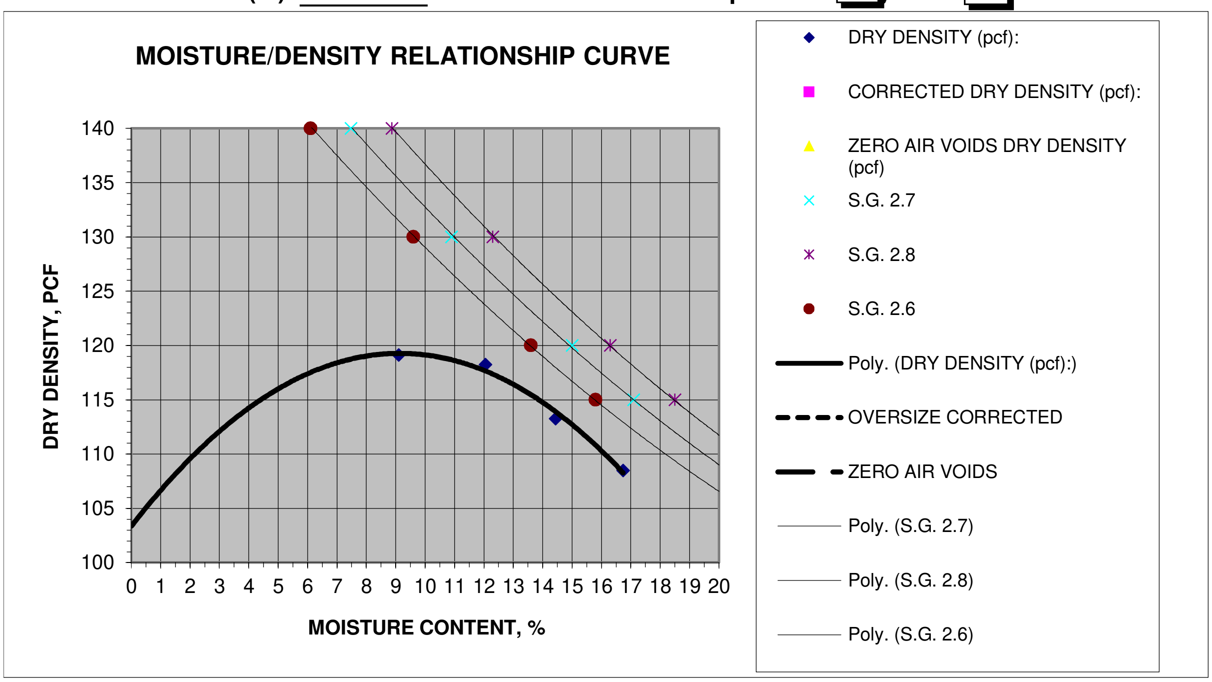
Sulfate Content, Resistivity, and Chloride Content
Testing to determine the water-soluble sulfate content was performed by others in general accordance with California Test No. 417. Resistivity testing was completed by others in general accordance with California Test No. 643. Testing to determine the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

MOISTURE/DENSITY RELATIONSHIP

Client: Encore Capital Management
Job No.: 2074-CR
Project: 200 & 110 Midway Dr.
Lab No.: Corona
Location: Anaheim
Material Type: Gray Brown Fine Sand
Material Supplier:
Material Source:
Sample Location: B-5 @ 0 - 5 ft

Sampled By: DRW
Date Sampled: 8-Feb-19
Received By: DLI
Date Received: 11-Feb-19
Tested By: DA
Date Tested: 18-Feb-19
Reviewed By:
Date Reviewed:

Test Procedure: ASTM 1557
Method: A
Oversized Material (%): 0.0
Correction Required: yes x no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	120.0	**@ Optimum Moisture, %**	10.5
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:

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Encore Capital Management	***Tested/ Checked By:***	DI Lab No Corona
Project Number:	2074-CR	***Date Tested:***	2/18/2019
Project Location:	110 & 200 Midway Dr., Anaheim	***Sample Source:***	B-2 @ 0 - 5 ft
		Sample Description:	

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	755.2
B	Weight of ring (gm)	363.0
C	Net weight of sample (gm)	**392.2**
D	Wet Density, lb / ft3 (C*0.3016)	**118.3**
E	Dry Density, lb / ft3 (D/1.F)	**107.3**

SATURATION DETERMINATION

F	Moisture Content, %	10.2
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**48.3**

READINGS

DATE	TIME	READING	
2/18/2019	4:22	0.2890	Initial
	4:32	0.2890	10 min/Dry
2/19/2019	4:35	0.2890	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
783.0	**17.3**

EXPANSION INDEX = 0

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Encore Capital Management	***Tested/ Checked By:***	DI Lab No Corona
Project Number:	2074-CR	***Date Tested:***	2/18/2019
Project Location:	110 & 200 Midway Dr., Anaheim	***Sample Source:***	B-5 @ 0 - 5 ft
		Sample Description:	

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	752.6
B	Weight of ring (gm)	363.3
C	Net weight of sample (gm)	**389.3**
D	Wet Density, lb / ft3 (C*0.3016)	**117.4**
E	Dry Density, lb / ft3 (D/1.F)	**106.3**

SATURATION DETERMINATION

F	Moisture Content, %	10.5
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**48.4**

READINGS

DATE	TIME	READING	
2/18/2019	7:45	0.2460	Initial
	7:55	0.2460	10 min/Dry
2/19/2019	8:00	0.2460	Final

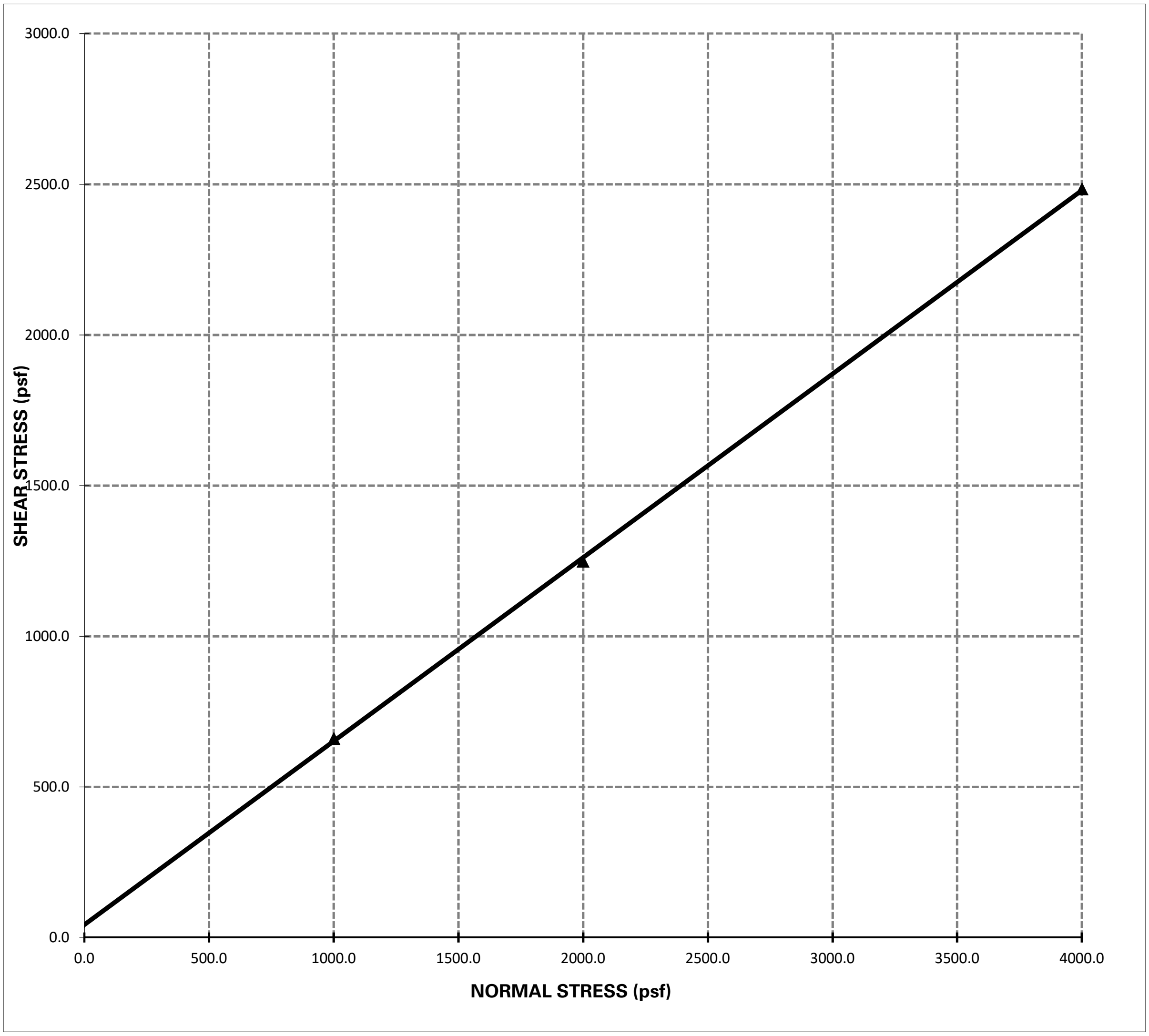
FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
780.8	**17.7**

EXPANSION INDEX = 0

DIRECT SHEAR TEST

Project Name:	110 & 200 Midway Dr., Anaheim	***Sample Location:***	B-5 @ 0 - 5 ft
Project Number:	2074-CR	***Date Tested:***	2/20/2019



Shear Strength: Φ = **31.4 °**, **C = 42.00 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.

Soil Analysis Lab Results

Client: Geotek Inc
Job Name: 110 & 200 Midway Dr., Anaheim
Client Job Number: 2074-CR
Project X Job Number: S190219D
February 20, 2019

	Method	ASTM G187		ASTM D516		ASTM D512B		SM 4500-NO3-E	SM 4500-NH3-C	SM 4500-S2-D	ASTM G200	ASTM G51
Bore# / Description	Depth	Resistivity As Rec'd	Resistivity Minimum	Sulfates		Chlorides		Nitrate	Ammonia	Sulfide	Redox	pH
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
B2	0.0-5.0	737,000	19,430	27	0.0027	15	0.0015	9	3.8	0.33	136	9.27
B5	0.0-5.0	22,110	16,080	36	0.0036	12	0.0012	36	4.5	0.24	138	7.89

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

Please call if you have any questions.

Prepared by,

Ernesto Padilla, BSME
Field Engineer
Prepared by,

Nathan Jacob
Lab Technician

Respectfully Submitted,

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

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

APPENDIX C

PERCOLATION TEST DATA

Geotechnical and Infiltration Evaluation
110 and 200 West Midway Drive, Anaheim, California
Project No. 2074-CR

GEOTEK

Client:	**Encore Capital Management**
Project:	**110 and 200 West Midway Drive**
Project No:	**2074-CR**
Date:	**2/11/2019**

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

Percolation Rate (Porchet Method)

Time Interval, Δt =	7.5	min
Final Depth to Water, D_F =	60.00	in
Test Hole Radius, r =	4	in
Initial Depth to Water, D_O =	40.00	in
Total Test Hole Depth, D_T =	60.00	in

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

$H_O = D_T - D_O$ =	20	in
$H_F = D_T - D_F$ =	0	in
$\Delta H = \Delta D = H_O - H_F$ =	20	in
$H_{avg} = (H_O+H_F)/2$ =	10	in

I_t = 26.67 **Inches per Hour**

GEOTEK

Client:	**Encore Capital Management**
Project:	**110 and 200 West Midway Drive**
Project No:	**2074-CR**
Date:	**2/11/2019**

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

Percolation Rate (Porchet Method)

Time Interval, Δt =	8	min
Final Depth to Water, D_F =	60.00	in
Test Hole Radius, r =	4	in
Initial Depth to Water, D_O =	40.00	in
Total Test Hole Depth, D_T =	60.00	in

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

$H_O = D_T - D_O$ =	20	in
$H_F = D_T - D_F$ =	0	in
$\Delta H = \Delta D = H_O - H_F$ =	20	in
$Havg = (H_O+H_F)/2$ =	10	in

I_t = 25.00 **Inches per Hour**

GEOTEK

PERCOLATION DATA SHEET

Project: 110 and 200 Midway Drive, Anaheim **Job No.:** 2074-CR

Test Hole No.: Boring No. P-1 **Tested by:** DRW **Date:** February 9,2019

Depth of Hole As Drilled: 60 Inches **Before Test:** 60 Inches **After Test:** 60 Inches

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
1	12:35 PM	25	60	40	60	20	20 inches of water dissipated before the 25 minute reading
2	1:00 PM	25	60	40	60	20	
3	1:27 PM	5	60	40	60	20	Recorded the time it took 20 inches of water to dissipate
4	1:32 PM	5	60	40	60	20	
5	1:37 PM	5	60	40	60	20	
6	1:43 PM	6	60	40	60	20	
7	1:50 PM	6.5	60	40	60	20	
8	1:57 PM	6.5	60	40	60	20	
9	2:05 PM	7	60	40	60	20	
10	2:13 PM	7.5	60	40	60	20	
11	2:20 PM	7.5	60	40	60	20	
12	2:28 PM	7.5	60	40	60	20	

PERCOLATION DATA SHEET

Project: 110 and 200 Midway Drive, Anaheim **Job No.:** 2074-CR

Test Hole No.: Boring No. P-2 **Tested by:** DRW **Date:** February 9,2019

Depth of Hole As Drilled: 60 Inches **Before Test:** 60 Inches **After Test:** 60 Inches

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
1	10:18 AM	25	60	40	60	20	20 inches of water dissipated before the 25 minute reading
2	10:43 AM	25	60	40	60	20	
3	11:18 AM	5	60	40	60	20	Recorded the time it took 20 inches of water to dissipate
4	11:25 AM	5	60	40	60	20	
5	11:32 AM	6	60	40	60	20	
6	11:40 AM	6.5	60	40	60	20	
7	11:47 AM	6.5	60	40	60	20	
8	11:55 AM	7	60	40	60	20	
9	12:05 PM	7	60	40	60	20	
10	12:15 PM	8	60	40	60	20	
11	12:25 PM	8	60	40	60	20	
12	12:35 PM	8	60	40	60	20	

APPENDIX D

GENERAL EARTHWORK AND GRADING GUIDELINES

Geotechnical and Infiltration Evaluation
110 and 200 West Midway Drive, Anaheim, California
Project No. 2074-CR

GEOTEK

GENERAL GRADING GUIDELINES

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

General

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

Preconstruction Meeting

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

Grading Observation and Testing

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

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

Site Clearing

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

Treatment of Existing Ground

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

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

Fill Placement

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

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

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

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

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

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

JOB SAFETY

General

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

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

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

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

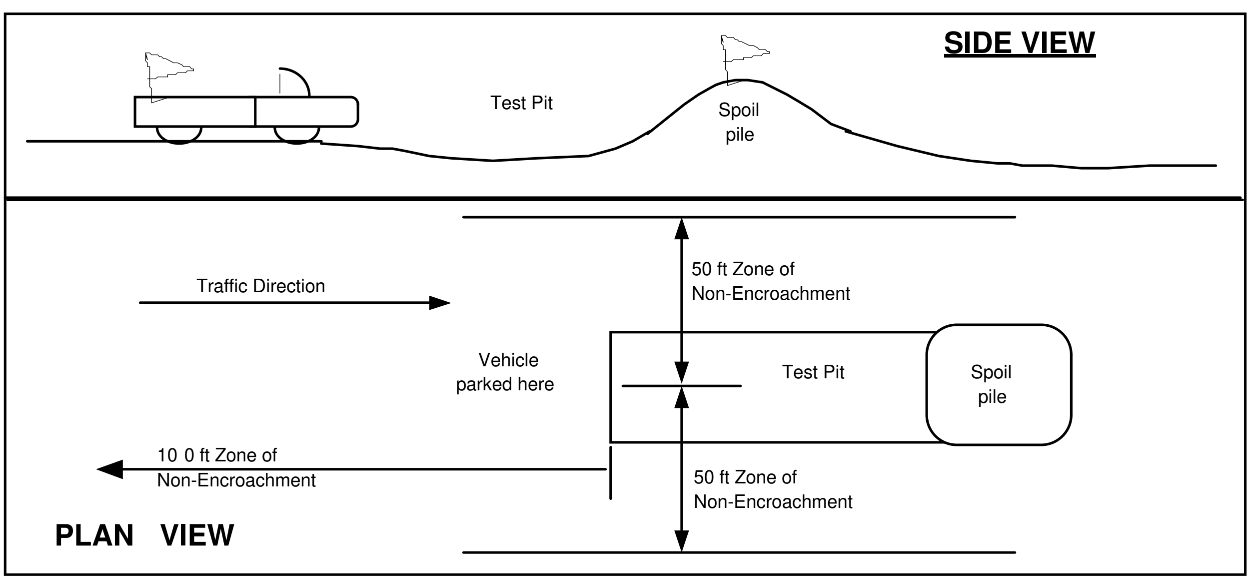
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

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

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

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

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.