Appendix G Geotechnical Data

Appendices

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Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

August 10, 2018

Project No. 11737.002

6509 Serrano L.P.
4040 MacArthur Boulevard, Suite 300
Newport Beach, California 92660

Attention: Mr. John Saunders

Subject: Response to Review Comments Regarding Leighton's Geotechnical Exploration Report for the Proposed Residential Development 6501-6513 East Serrano Avenue Anaheim, California

Reference: Leighton and Associates, Inc., 2017, *Geotechnical Exploration Report, Proposed Residential Development, 6501-6513 East Serrano Avenue, Anaheim, California*, Project No. 11737.001, dated October 9, 2017.

INTRODUCTION

In accordance with your request and authorization, Leighton and Associates, Inc. (Leighton) is pleased to present our response to the Preliminary Soils Report Review Comments by the City of Anaheim Department of Public Works dated May 25, 2018, regarding our referenced geotechnical exploration report (Leighton, 2017) for the subject project.

RESPONSES TO REVIEW COMMENTS

A copy of the review comments prepared by the City of Anaheim and dated May 25, 2018 is included in Appendix A. For convenience, the two (2) review comments are presented below in italics before Leighton's responses.

17781 Cowan ▪ Irvine, CA 92614-6009
949.250.1421 ▪ 949.250.1114 Fax

Comment 3.1:

As summarized within earlier correspondences prepared by the City of Anaheim, the preliminary soils report must be reviewed by the Santiago Geological Hazard Abatement District (GHAD) prior to approval by the City. Contact information for SGHAD is presented below:

Karen Holthe, CMCA, AMS
Senior Account Manager
kholthe@cardinal-online.com
Cardinal Property Management, AAMC
825 N. Park Center Dr., #101
Santa Ana, CA 92705
P (714) 779-1300 / F (714) 779-3400

Please provide a copy of the review comments and/or consent from the Santiago GHAD.

Response to Comment 3.1:

Our referenced report (Leighton, 2017) was submitted to the Santiago GHAD for review, and a copy of the Residential Grading Plan Review letter dated June 29, 2018, prepared by ENGEO (acting as the Santiago GHAD Manager) is included in Appendix B. The review letter indicates that construction of the planned residences and associated improvements, including biofiltration improvements, if constructed, does not appear that it would affect the Santiago landslide, or the ongoing mitigation efforts by the Santiago GHAD.

Comment 3.2:

Percolation testing was conducted at two locations within the site (LP-1 & LP-2). Both test locations encountered artificial fill to the total depth of the boring. Measured infiltration rates within the test borings were calculated between 0.05 and 0.06 inches per hour. Since the infiltration rates did not meet the County of Orange minimum infiltration rate (0.3 inches per hour), the consultant has concluded that infiltration beneath the site is impractical and not recommended for the proposed development.

The County of Orange, Technical Guidance Document states that infiltration testing should not be conducted in engineered or undocumented fill. While the areas tested

Leighton

were underlain by significant fill, other areas of the site are not and are reported to have sandstone bedrock located near the surface. As such, the consultant should determine if infiltration is practical within the sandstone unit encountered in various exploratory borings where present near the surface. Keep in mind that while the sandstone unit may exhibit a relatively low permeability, a dry well in the sandstone unit may result in an infiltration rate that is deemed feasible in the TGD (where infiltration rate= well flow rate/wetted area).

Response to Comment 3.2:

Sandstone and siltstone bedrock was encountered at relatively shallow depths in the borings performed at the site, primarily in the eastern and western portions of the site. The bedrock as encountered in these areas is hard and generally comprised of fine to medium grained sandstone with interbedded grey brown moderately fractured fissile siltstone. Regional geologic mapping of the site vicinity (Morton and Miller, 2006) indicates that the geologic structure of the sedimentary bedrock generally dips down to the north and northeast at inclinations on the order of approximately 15 to 30 degrees from horizontal.

Due to the subsurface conditions at the site and in its vicinity (shallow bedrock in the eastern and western portions of the site and deep canyon fill in the central portion of the site), it is our opinion that stormwater infiltration within the sandstone bedrock at the site would increase the risk of geotechnical hazards at the site and/or down gradient of the site. The risks would include the potential for adverse effects on properties down gradient caused by migration of water infiltrated into the subsurface at the site. The joints and factures in the bedrock and the interlayered and inclined (north and northeast dipping) sandstone and siltstone layers under the site provide a pathway to downslope properties where adverse effects could be caused by migrating water. The locations and lateral extents of potential water migration paths within the bedrock are very difficult and nearly impossible to estimate. Therefore, we do not recommend stormwater infiltration for the site. Consequently, additional testing to determine if infiltration is practical within the sandstone is not necessary.

Comment 3.3:

The Preliminary Soils Report shall be approved prior to filing for Planning Commission public hearing.

Response to Comment 3.3:

Acknowledged.

CLOSING

We appreciate the opportunity to be of continued service on this project. If you have any questions or if we can be of further service, please contact us at ***(866) LEIGHTON***; specifically at the phone extensions or e-mail as listed below.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

Jeffrey M. Pflueger, PG, CEG 2499
Associate Geologist
Extension 4257, jpflueger@leightongroup.com

Vincent P. Ip, PE, GE 2522
Senior Principal Engineer
Extension: 1682, vip@leightongroup.com

JMP/VPI/gv

Attachments: Appendix A – City of Anaheim Letter dated May 25, 2018
Appendix B – Santiago GHAD Residential Grading Plan Review Letter dated June 29, 2018

Distribution: (1) Addressee

Leighton

APPENDIX A

City of Anaheim Review Letter dated May 25, 2018

City of Anaheim
DEPARTMENT OF PUBLIC WORKS

May 25, 2018

To: Jeffrey M. Pflueger, P.G., C.E.G.
Vincent P. Ip, P.E., G.E.
Leighton and Associates, Inc.
17781 Cowan
Irvine, CA 92614
(949) 250-1421

RE: Review of Geotechnical Exploration Report for Proposed Residential Development
6501-6213 E. Serrano Ave.
OTH2018-01060,
First Review

Dear Mr. Pflueger:
The documents reviewed have been submitted to the City of Anaheim as a geotechnical document in support of planning approval. As such, the report was only reviewed for establishing feasibility of the proposed site development. Additional comments may be issued if submitted in support of grading or building plans during Final Engineering. Prior to approval of planning, the following items should be addressed by the consultant. The following will be required for the next plan submittal:

1. The Soils Report Comment Letter from Albus-Keefe & Associates, Inc. and a copy of this letter.
2. Two (2) copies of responses addressing all comments.
3. <u>*Preliminary Soils Report Review Comments:*</u>
   3.1 As summarized within earlier correspondences prepared by the City of Anaheim, the preliminary soils report must be reviewed by the Santiago Geological Hazard Abatement District (SGHAD) prior to approval by the City. Contact information for SGHAD is presented below:

   Karen Holthe, CMCA, AMS
   Senior Account Manager
   kholthe@cardinal-online.com
   Cardinal Property Management, AAMC
   825 N. Park Center Dr., #101
   Santa Ana, CA 92705
   P (714) 779-1300 / F (714) 779-3400

   Please provide a copy of the review comments and/or consent from the Santiago GHAD.

   3.2 Percolation testing was conducted at two locations within the site (LP-1 & LP-2). Both test locations encountered artificial fill to the total depth of the

200 S. Anaheim Blvd., Suite 276
Anaheim, CA 92805
TEL (714) 765-5176
FAX (714) 765-5225
www.anaheim.net

boring. Measured infiltration rates within the test borings were calculated between 0.05 and 0.06 inches per hour. Since the infiltration rates did not meet the County of Orange minimum infiltration rate (0.3 inches per hour), the consultant has concluded that infiltration beneath the site is impractical and not recommended for the proposed development.

The County of Orange, Technical Guidance Document states that infiltration testing should not be conducted in engineered or undocumented fill. While the areas tested were underlain by significant fill, other areas of the site are not and are reported to have sandstone bedrock located near the surface. As such, the consultant should determine if infiltration is practical within the sandstone unit encountered in various exploratory borings where present near the surface. Keep in mind that while the sandstone unit may exhibit a relatively low permeability, a dry well in the sandstone unit may result in an infiltration rate that is deemed feasible in the TGD (where infiltration rate = well flow rate/wetted area).

3.3 The Preliminary Soils Report shall be approved prior to filing for Planning Commission public hearing.

If you have any questions regarding this project, please contact me at (714) 765-4953 or at egarcia2@anaheim.net

Sincerely,

Edgar Garcia

Edgar Garcia
Associate Engineer

Attachment: Comment Letter from Albus-Keefe & Associates, Inc. dated May 9, 2018

cc;
Raul Garcia, Development Services Manager
Mike Eskander, Principal Civil Engineer
File

Albus-Keefe & Associates, Inc.
1011 N. Armando Street
Anaheim, California 92806
(714) 630-1626
(714) 630-1916 FAX

GEOTECHNICAL ENGINEERING REVIEW SHEET
CITY OF ANAHEIM

Plan Check # OTH 2018-01060

AKA Project No. 2714.00

Date: May 9, 2018

Project Name: Nohl Ranch Condos

Location: 6501 – 6513 E Serrano Avenue, Anaheim, CA

Consultant: Leighton and Associates, Inc.

Geotechnical Engineer: Vincent P. Ip, GE 2522

Engineering Geologist: Jeffrey M. Pflueger, CEG2499

Documents Reviewed:

1.) *Geotechnical Exploration Report, Proposed Residential Development, 6501-6513 East Serrano Avenue, Anaheim, California,* prepared by Leighton and Associates, Inc., dated October 9, 2017 (Project No. 11737.001).

2.) *Conceptual Grading Plan, Nohl Ranch Condos, City of Anaheim,* Prepared by Hunsaker & Associates, Inc. Sheet C-4, dated April 16, 2018.

Action:

____ Recommended Approval of Document(s) Submitted
____ Conditional Approval of Document(s) Submitted – see comments
_X_ Request Additional Data for Review – see comments

Reviewed By:

David E. Albus
Principal Engineer
G.E. 2455

REGISTERED PROFESSIONAL ENGINEER
DAVID E. ALBUS
No. 2455
Exp. 12/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

Patrick M. Keefe
Principal Engineering Geologist
CEG 2022

REGISTERED GEOLOGIST
PATRICK M. KEEFE
NO. 2022
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

AKA Project No. 2714.00 Date: May 9, 2018

COMMENTS

The documents reviewed have been submitted to the City of Anaheim as a geotechnical document in support of planning approval. As such, the report was only reviewed for establishing feasibility of the proposed site development. Additional comments may be issued if submitted in support of grading or building plans. Prior to approval of planning, the following items should be addressed by the consultant.

1) As summarized within earlier correspondences prepared by the City of Anaheim, the preliminary soils report must be reviewed by the Santiago Geological Hazard Abatement District (SGHAD) prior to approval by the City. Contact information for SGHAD is presented below:

Karen Holthe, CMCA,AMS
Senior Account Manager
kholthe@cardinal-online.com
Cardinal Property Management, AAMC
825 N. Park Center Dr., #101
Santa Ana, CA 92705
P (714) 779-1300 / F (714 779-3400

2) Percolation testing was conducted at two locations within the site (LP-1 & LP-2). Both test locations encountered artificial fill to the total depth of the boring. Measured infiltration rates within the test borings were calculated between 0.05 and 0.06 inches per hour. Since the infiltration rates did not meet the County of Orange minimum infiltration rate (0.3 inches per hour), the consultant has concluded that infiltration beneath the site is impractical and not recommended for the proposed development.

The County of Orange, Technical Guidance Document states that infiltration testing should not be conducted in engineered or undocumented fill. While the areas tested were underlain by significant fill, other areas of the site are not and are reported to have sandstone bedrock located near the surface. As such, the consultant should determine if infiltration is practical within the sandstone unit encountered in various exploratory borings where present near the surface. Keep in mind that while the sandstone unit may exhibit a relatively low permeability, a dry well in the sandstone unit may result in an infiltration rate that is deemed feasible in the TGD (where infiltration rate = well flow rate/wetted area).

APPENDIX B

Santiago GHAD Residential Grading Plan Review Letter

dated June 29, 2018

Project No.
14174.000.000

June 29, 2018

Ms. Karen Holthe
Santiago Geologic Hazard Abatement District
Cardinal Property Management
825 N. Park Center Drive, Suite 101
Santa Ana, CA 92705

Subject: 6501-6513 East Serrano Avenue
Anaheim, California

RESIDENTIAL GRADING PLAN REVIEW

References:

1. Leighton and Associates, Inc., Geotechnical Exploration Report, 6501- 6513 East Serrano Avenue, Anaheim, CA 92807; October 9, 2017, Project No. 11737.001.
2. City of Anaheim, Department of Public Works; Review of Geotechnical Exploration Report for Proposed Residential Development, 6501-6513 East Serrano Avenue, Anaheim, CA 92807; OTH2018-01060, First Review, May 25, 2018.
3. Eberhart and Stone, Plan of Control, Prepared for Proposed Santiago Geologic Hazard Abatement District, Anaheim Hills, Anaheim, California, February 22, 1999.
4. Eberhart and Stone, Santiago Landslide Area Anaheim Hills, Geologic Hazard Abatement District Benefit Area, Anaheim, California.

Dear Ms. Holthe:

ENGEO, acting as the Santiago Geologic Hazard Abatement District (GHAD) Manager, reviewed the Leighton Geotechnical Exploration Report and City of Anaheim, Department of Public Works Review of Geotechnical Exploration Report for Proposed Residential Development (References 1 and 2) for 6501-6513 East Serrano Avenue in Anaheim, California (Subject Property). The purpose of our review was to address the City of Anaheim's request that the applicant obtain written consent from the GHAD indicating that the proposed project will not significantly impact stability of the existing Santiago landslide.

As described in Reference 1, the planned residences will replace the existing commercial buildings and improvements. The residences will be two- to three-story attached multi-family residential buildings, with private drive aisles and guest parking. Onsite biofiltration is being considered for stormwater treatment and surface drainage will be directed away from the structures.

As described in the Leighton Geotechnical Exploration Report, artificial fill thickness varied beneath the Subject Property from 1 foot to greater than 76½ feet. Puente Formation bedrock was encountered in six of the eight exploratory borings underlying the artificial fill. Groundwater was not observed in the exploratory borings at the time of the Leighton exploration. Percolation testing was conducted at two of the exploratory boring locations to support design of the planned biofiltration improvements.

The Subject Property is located northwest of the Santiago GHAD as shown on the Benefit Area Site Plan (Reference 4). The planned addition is not located within the Santiago GHAD or the mapped "Limit of Surface Damage" area. As stated in the Plan of Control (Reference 3), the formation on the Santiago landslide was caused by four primary factors:

1. North-facing hillside topography.
2. Geologic structure as north-dipping strata and south-ancient faults.
3. Geologically weak materials along critical sedimentary beds and faults.
4. Rising groundwater.

Based on our review, it does not appear that construction of the planned residences and associated improvements, including biofiltration improvements, if constructed, would affect the Santiago landslide or the ongoing mitigation efforts by the Santiago GHAD. We make no representations as to the accuracy of dimensions, measurements, calculations or any portion of the design.

If you have any questions regarding the contents of this letter, please contact us.

Very truly yours,

ENGEO INCORPORATED

Haley J Trindle

Haley Trindle
ht/eh/jf

Eric Harrell

CERTIFIED ENGINEERING GEOLOGIST
ERIC HARRELL
No. 2189
STATE OF CALIFORNIA

Eric Harrell, CEG

GEOTECHNICAL EXPLORATION REPORT
PROPOSED RESIDENTIAL DEVELOPMENT
6501-6513 EAST SERRANO AVENUE
ANAHEIM, CALIFORNIA

Prepared for:

6509 Serrano L.P.

4040 MacArthur Boulevard, Suite 300
Newport Beach, California 92660

Project No. 11737.001

October 9, 2017

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

October 9, 2017

Project No. 11737.001

6509 Serrano L.P.
4040 MacArthur Boulevard, Suite 300
Newport Beach, California 92660

Attention: Mr. John Saunders

Subject: **Geotechnical Exploration Report**
Proposed Residential Development
6501-6513 East Serrano Avenue
Anaheim, California

In accordance with our proposal dated July 12, 2017, authorized by you on July 25, 2017, Leighton and Associates, Inc. (Leighton) is pleased to present this geotechnical exploration report for the proposed residential development project located at 6501-6513 East Serrano Avenue in Anaheim California.

The purpose of our study was to evaluate the geotechnical conditions at the site and to provide geotechnical recommendations for the design and construction of the project as currently proposed. The results of our exploration and recommendations are presented in this report.

17781 Cowan ▪ Irvine, CA 92614-6009
949.250.1421 ▪ 949.250.1114 Fax

We appreciate this opportunity to be of service. If you have any questions regarding this report or if we can be of further service, please call us at your convenience at **(866) *LEIGHTON***, directly at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

Jeffrey M. Pflueger, PG, CEG 2499
Associate Geologist
Ext 4257; jpflueger@leightongroup.com

Vincent P. Ip, PE, GE 2522
Senior Principal Engineer
Ext 1682; vip@leightongroup.com

JMP/VPI/JAR/lr

Distribution: (1) Addressee

Leighton

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1.0 INTRODUCTION

1.1 Site Description and Proposed Improvements

The project site is roughly 3 acres in size and is located at the northeast corner of Serrano Avenue and Nohl Ranch Road (6501, 6503, 6505, 6507, 6509, 6511 and 6513 East Serrano Avenue) in the city of Anaheim, California. The site is bordered by Serrano Avenue to the south, Nohl Ranch Road to the west, and single-family residential properties to the north and east. The site is relatively flat and is currently occupied by a commercial/retail development consisting of seven (7) one-story buildings situated in the central portion of the site surrounded by asphalt concrete (AC) paved surface parking and access drive aisles. Based on our observations, the existing improvements (i.e. pavement and buildings) generally appear to be in good condition with no obvious signs of distress. The northeast corner of the site is currently occupied by a playground area associated with a children's day care facility. The site location (latitude 33.8317°. longitude 117.7600°) and surrounding area are shown on Figure 1, *Site Location Map*. Review of the *City of Anaheim Base Map 286* (December, 2016) indicates a 5-foot wide electrical easement within the southern and western parking areas.

Based on preliminary review of historical aerial photographs and topography maps, the project site was mass graded as a part of a larger development between approximately 1966 and 1972, and the seven (7) existing structures were constructed to its current configuration by approximately 1980 (NETR, 2017). Historic topographic contours that existed within the project site boundary prior to mass grading suggest that cut and fill grading of the previously existing natural topography was required to achieve the current grade. Maximum depth of artificial fill materials below this site is greater than 75 feet in thickness in the central region of the site.

We understand the proposed residential development includes complete demolition of the existing commercial buildings and improvements at the site to allow grading and construction for a residential development consisting of several two- to three-story attached multi-family residential buildings, private drive aisles and guest parking. No subterranean level is currently planned for the buildings. It is our understanding that onsite biofiltration is being considered for best management practice for storm water treatment. Although loading information for the proposed new structures has not been provided at this time, we expect the loading will be similar to typical two- to three-story attached residential structures.

1.2 Purpose and Scope

The purpose of our geotechnical exploration was to evaluate the soil and groundwater conditions at the site through review of available data, exploratory borings and onsite percolation testing, in order to provide geotechnical recommendations for design and construction of the proposed improvements.

The scope of this geotechnical exploration included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical and geological literature pertinent to the site. References used in preparation of this report are listed in Section 6.0. In addition, we submitted a request for public records with the City of Anaheim with the intent to obtain a copy of the as-graded geotechnical report documenting the mass/rough grading of the site. City of Anaheim approved grading plans for the surrounding tract (Tract 8375) to the north and east of the project site were available; however, the as-graded geotechnical report documenting the mass/rough grading of the site was not available for our review.

- *Pre-Field Exploration Activities* – A site visit was performed by a member of our technical staff to mark the boring locations. Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to our subsurface exploration.

- *Field Exploration* – Our field exploration was performed on August 16, 2017, and consisted of six, 8-inch diameter hollow-stem auger borings (LB-1 through LB-6) each drilled to depths ranging between approximately 9.8 and 76.5 feet below existing ground surface (bgs). The approximate locations of the borings are shown on Figure 2, *Boring Location Map.*

  During drilling of the hollow-stem auger borings (LB-1 through LB-6), both bulk and drive samples were obtained from the borings for geotechnical laboratory testing. Drive samples were collected from the borings using a Modified California Ring sampler in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPT) were also performed within the

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hollow-stem auger borings in accordance with ASTM Test Method D 1586 to help in evaluating the density and consistency of the site soils. The SPT and California Ring samplers were driven for a total penetration of 18 inches, unless practical refusal was encountered, using a 140-pound automatic hammer falling freely for 30 inches. The number of blows per 6 inches of penetration was recorded on the boring logs.

The borings were logged in the field by a certified engineering geologist. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (USCS). The samples were sealed and packaged for transportation to our laboratory. After completion of drilling, the borings (LB-1 through LB-6) were backfilled to the ground surface with excess soils generated during the exploration and patched with cold-mix asphalt concrete. The boring logs are presented in Appendix A, *Field Exploration Logs*.

- *Laboratory Tests* – Laboratory tests were performed on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the onsite soil. Tests performed during this investigation include:

  - In- situ Moisture Content and Dry Density (ASTM D2216 and ASTM D2937);
  - Atterberg Limits (ASTM D 4318);
  - Gradation (ASTM D 6913);
  - Percent Passing No. 200 Sieve (ASTM D 1140);
  - Direct Shear (ASTM D 3080)
  - Consolidation (ASTM D 2435);
  - Maximum Dry Density (ASTM D 1557);
  - R-Value (California Test Method 301); and
  - Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

  Results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A. Other laboratory test results are presented in Appendix B, *Laboratory Test Results*.

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- *Percolation Testing* – During our field exploration performed on August 16, 2017, two additional 8-inch diameter hollow-stem auger borings (LP-1 and LP-2) located in the southern portion of the site in the vicinity of the proposed stormwater infiltration areas were each drilled to an approximate depth of 9 feet bgs and converted to a temporary percolation test well for subsequent percolation testing. Refer to the discussion of infiltration rate presented in Section 2.4 and the field percolation test data provided in Appendix C, *Percolation Test Results*.

- *Engineering Analysis* - The data obtained from our background review, field exploration, and laboratory testing program were evaluated and analyzed to develop geotechnical recommendations for the project as currently planned.

- *Report Preparation* - The results of the exploration are summarized in this report presenting our findings, conclusions and recommendations.

Leighton

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The project site is located within the Peninsular Ranges geomorphic province of California along the eastern margins of the Los Angeles Basin. The Los Angeles Basin is bounded to the north by the east-west trending Transverse Ranges and to the east and southeast by the northwest trending Peninsular Ranges. The Los Angeles Basin is a large structural depression formed as the San Andreas fault shifted eastward to its present location. The basin has since been filled with sediments eroded from the surrounding highlands interpreted to have a maximum thickness of over 30,000 feet (Yerkes, 1965).

The project site is located in the Santa Ana Mountains in the eastern portion of the Peralta Hills. These low-lying hills extend westward from the Santa Ana Mountains toward the Los Angeles Basin and are primarily underlain by Tertiary age (between about 2.6 to 65 million years old) mostly marine sediments deposited in the Los Angeles Basin spanning the Miocene to Pliocene Epoch (about 2.6 to 23.3 million years ago). The project site is located in an area mapped to be underlain by Miocene age Puente Formation bedrock (Soquel and La Vida Members) primarily consisting of sandstone and siltstone (Morton and Miller, 2006). The mapped geologic units in the vicinity of the project site is presented as Figure 3, *Regional Geology Map*.

2.2 Subsurface Soil Conditions

As interpreted from our subsurface explorations (hollow-stem auger borings), the site is underlain by previously placed artificial fill overlying Tertiary age sandstone and siltstone bedrock materials. The stratigraphy of the subsurface soil and bedrock materials encountered in each soil boring is presented on the boring logs (Appendix A), a general description of the earth materials as encountered are described below:

Artificial Fill

The previously placed artificial fill soil as encountered in our exploratory borings is on the order of less than a foot to over 76.5 feet thick across the site, consisting primarily of orange brown to gray brown, moist to very moist, medium dense to dense silty sand and clayey sand interlayered with medium stiff to very stiff clay, silty clay and sandy clay. Based on review of the documents provided by the City of Anaheim, the artificial fill materials encountered at the site are associated with the previous mass/rough grading of the area. No report documenting the grading activities associated with the current site development was available for review; however, based on our understanding of the City's policy, it is reasonable to assume that previous grading activities associated with the site and its vicinity were permitted and performed under the observation and testing of geotechnical consultants.

Puente Formation Bedrock

Encountered below the artificial fill in borings LB-1, LB-3, LB-4 and LB-5 at various depths was upper Miocene age marine sedimentary rocks of the Puente Formation.

The La Vida Member (Map Symbol: Tplv) is the basal stratigraphic unit of the Puente Formation encountered in boring LB-3 (Figure 2). The La Vida Member consists of orange brown to light grey brown, laminated, brittle shaley siltstone with lesser amounts of slightly well cemented sandstone. The sandstone content increases as the La Vida Member grades into the Soquel Member (Map Symbol: Tpsq) which is present below a majority of the site as encountered in borings LB-1, LB-4 and LB-5 (Figure 2). The Soquel Member consists of orange brown, massive, fine to medium grained pebbly sandstone with interbedded grey brown moderately fractured fissile siltstone. Based on blow counts and visual classification, the bedrock materials encountered were generally characterized as dense, hard and moderately oxidized.

2.3 Groundwater Conditions

Groundwater was not encountered in our borings excavated at the site to a maximum depth of approximately 76.5 feet bgs during drilling. Based on the currently proposed development scheme, groundwater is not expected to pose a constraint during and after construction.

Although groundwater is not considered a constraint for the project, seasonal fluctuations in groundwater level, localized zones of perched water including water due to nearby landscaping, and an increase in soil moisture should be anticipated during and following locally intense rainfall or stormwater runoff.

2.4 Infiltration Capacity

In-situ percolation testing was performed to evaluate the infiltration capacity of the site soils in general accordance with the Orange County *Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Programs (WQMPs)* (OCPW, 2013).

Borings LP-1 and LP-2 located in the general vicinity of the planned biofiltration treatment areas were both converted to temporary percolation test wells upon completion of drilling and sampling (Figure 2, *Exploration Location Map*). The temporary wells consisted of a 2-inch-diameter, PVC pipe with perforations from 4 to 9 feet bgs placed within each borehole. The annulus was filled with clean sand (#3 Monterey Sand) to approximately 1 foot above the perforated pipe. In general accordance with the Orange County TGD (OCPW, 2013), each percolation test well was pre-soaked prior to the testing. After the conclusion of the percolation test, the PVC pipe was removed and the test holes were backfilled with excess soil cuttings and patched with cold-mix asphalt concrete.

The test was performed using the falling-head method which records the drop of water level inside the well over each testing period. The measured infiltration rate for the percolation tests was calculated by dividing the rate of discharge (i.e., volume of water discharged from the well during the test) by the infiltration surface area, or flow area. Detailed results of the field testing data and measured infiltration rate for the test wells are presented in Appendix C, *Percolation Test Results*. Presented in the table below is a summary of the measured infiltration rate results.

Table 1 – Measured Infiltration Rate

Boring-Percolation Test Well Designation	Approximate Depth of Test Zone Below Existing Ground Surface (feet)	Measured Infiltration Rate (inches per hour)
LP-1	5 to 9	0.06
LP-2	5 to 9	0.05

The percolation tests performed at test well locations LP-1 and LP-2 (Figure 2) yielded very low measured infiltration rates of approximately 0.06 and 0.05 inch per hour within the test zone between 5 to 9 feet bgs. These rates do not meet the minimum requirement for stormwater infiltration feasibility (0.3 inch per hour) per the Orange County (OCPW, 2013) guidelines.

Based on our current subsurface exploration, the artificial fill soils beneath the site within the zones tested generally do not provide adequate infiltration potential as indicated by the very low infiltration rates. Direct infiltration to the site soils is not recommended.

3.0 GEOLOGIC AND SEISMIC HAZARDS

Geologic and seismic hazards include surface fault rupture, seismic shaking, liquefaction, seismically-induced settlement, lateral spreading, seismically-induced landslides, flooding, seismically-induced flooding, seiches and tsunamis. The following sections discuss these hazards and their potential impact at the project site.

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008c). The closest active faults to the site are the Elsinore Fault Zone (Whittier fault), Puente Hills fault, Chino fault and the San Joaquin Hills fault, located approximately 3.6 miles, 7.7 miles, 8.2 miles and 10.5 miles from the site, respectively. The Puente Hills and San Joaquin Hills faults are both blind thrust faults that are concealed at depth, without the potential for surface fault rupture. The San Andreas fault, which is the largest active fault in California, is approximately 35 miles northeast of the site. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault Map*).

The project site is located near the eastern mapped terminus of the Peralta Hills Fault, see Figure 4, R*egional Fault Map.* The Peralta Hills Fault has long been recognized to have thrust bedrock of the La Vida Member over stream terrace deposits of probable Pleistocene age (1.8 million to 11,700 years ago). Investigations by others have suggest there is scant evidence for Holocene activity (11,700 years to present) along the Peralta Hills fault (Converse Ward Dixon, 1979). Fault investigation by Leighton and Associates Inc. (1986) did **not** encounter evidence for Holocene offsets along the Peralta Hills or secondary faults associated with the system. The California Geological Survey (CGS) based on the current zoning criteria (Bryant and Hart, 2007) has not zoned the Peralta Hills Fault.

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3.2 Strong Ground Shaking

The site is located within a seismically active region, as is Southern California in general. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. Peak Horizontal Ground Accelerations (PHGA) are generally used to evaluate the intensity of ground motion.

The code-based Maximum Considered Earthquake (MCE) corresponds to an earthquake with a probability of exceedance of 2 percent in 50 years (i.e., 2475-year return period). Using United States Geological Survey (USGS) web-based Seismic Design Maps application (USGS, 2008a), the corresponding PHGA was calculated at 0.599g. The ground motion parameters for the MCE in terms of spectra accelerations at 5 percent damping are presented in the following table:

Table 2 – 2016 CBC Based Ground Motion Parameters (Mapped Values)

Categorization/Coefficient [1]	
Site Latitude	33.831715°
Site Longitude	-117.760025°
Site Class	D
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_S	1.569g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.604g
Short Period (0.2 sec)Site Coefficient, F_a	1.0
Long Period (1 sec) Site Coefficient, F_v	1.5
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.569g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	0.906g
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.046g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.604g

(1) Source: Ground motion values were calculated using United States Geological Survey (USGS) web-based Seismic Design Maps application (USGS, 2008a)

Accordingly, the site-adjusted geometric mean Peak Ground Acceleration (PGA_m) was calculated at 0.599g (i.e., F_{PGA}=1.0). By deaggregating the PGA_m, the corresponding earthquake is an M_w 6.9 event with a distance of approximately 5.6 miles from the site (USGS, 2008b).

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The seismicity data are included in Appendix D, *Seismicity Data*. For a general view of recorded historical seismic activity see Figure 5, *Historic Seismicity Map*.

3.3 Liquefaction

As shown on the State of California Seismic Hazard Zones Map for the Orange Quadrangle (CGS, 1998), the project site is **not** located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 6, *Seismic Hazard Map*). In addition, based on our subsurface exploration, groundwater was not encountered at the project site to the maximum depth explored of 76.5 feet bgs. Based on these considerations, the potential for liquefaction occurring at the site is low.

3.4 Earthquake-Induced Settlement

Strong ground motion during earthquakes tends to rearrange looser soils particles into a more compact arrangement, especially in granular soil deposits. The cumulative effects of soil particles rearrangement during earthquake ground shaking will result in settlement of the soil column. In general, a poorly graded granular deposit is more susceptible to settlement than a fine-grained or well-graded soil. Due to the dense nature of the existing fill at the site, the potential for seismically-induced settlement is considered negligible at the site.

3.5 Earthquake-Induced Lateral Spreading

Based on the consideration that the site is not located in an area with potential for liquefaction, lateral spreading induced by soil liquefaction is not likely to occur at the site.

3.6 Earthquake-Induced Landslides

Based on the State of California Seismic Hazard Zones Map for the Orange Quadrangle (CGS, 1998), the site is **not** located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides (Figure 6, *Seismic Hazard Map*). Based on these considerations, the potential for seismically-induced landsliding is considered low. Proposed slopes, if any, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter.

It should be noted that the project site is located within the general vicinity, approximately 0.4 mile to the west of the Santiago Landslide that occurred in Anaheim Hills in 1993 as mapped by Cotton, Shires & Associates (2005). Topographic features expressive of landsliding were observed in the foothills to the south and east of the project site (Leighton, 1987). These landslides have occurred primarily within the Vaqueros Sespe Formation Sandstone and the La Vida Member of the Puente Formation. The landslides in the Vaqueros Sespe Formation likely involve highly fractured and sheared siltstone beds. Landslides in the La Vida Member are primarily located on north facing slopes and are probably bedding plain failures where local stream incision has undercut weak bedding planes. Other landslides mapped in the hills to the south and east may be failures along faults or fault derived fractures.

Based on the location of the Santiago Landslide and consideration of the geologic and topographic conditions of the project site and immediate vicinity, the potential for landsliding associated with the 1993 Santiago Landslide to occur at the site is considered low.

3.7 Earthquake-Induced Flooding

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The project site is **not** located within a flood impact zone as indicated on Figure 7, *Dam Inundation Map*. With the site located above all major water bodies in the area, the potential for seismically induced flooding to affect the site due to dam failure is negligible.

3.8 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Since no enclosed body of water is located in the vicinity of the site, the potential hazard for seiches is negligible. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location of the site and the lack of large enclosed water bodies nearby, seiche and tsunami risks are not considered hazards for the project site.

3.9 <u>Flooding Hazard</u>

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the site is **<u>not</u>** located within a flood hazard zone (Figure 8, *Flood Hazard Map*). Flooding in the vicinity of the project site is generally isolated to the main drainage channels downstream of Villa Park Dam and Walnut Canyon Reservoir. The site is located within "Zone X", or is an area determined to be outside of the 0.2 percent annual chance floodplain (FEMA, 2008).

4.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project in general accordance with 2016 CBC requirements. The following recommendations are considered minimal from a geotechnical viewpoint as there may be more restrictive requirements of the architect, structural engineer, governing agencies and the City of Anaheim.

The geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans prepared for the project.

4.1 Earthwork

We recommend all earthwork for the project be performed in accordance with the following recommendations, future grading plan review report(s), the City of Anaheim grading requirements. The General Earthwork and Grading Specifications provided in Appendix E may be used as guidelines to develop grading specifications. In case of conflict the following recommendations shall supersede those provided in Appendix E.

4.1.1 Site Preparation

After demolition, the project site should be cleared of any vegetation, trash and debris, which should be properly disposed of offsite. Efforts should be made to remove or reroute any existing utility lines that interfere the proposed construction. Any resulting cavities should be properly backfilled and compacted.

4.1.2 Site Grading

The project area is generally underlain by previously placed artificial fill overlying Tertiary age sedimentary bedrock. To provide a uniform support and reduce the potential for differential settlement, the existing artificial fill and bedrock materials should be removed and replaced with engineered fill to provide supports for the proposed building and other structural improvements. The removals should extend to a depth of at least 2 feet below the foundation bottom or 5 feet below pad grade, whichever is deeper. It should be noted that very hard sandstone bedrock materials are likely to be encountered in the eastern portion of the site and may be encountered in the western portion of the site within the zone recommended for removal and recompaction. Where feasible, overexcavation and recompaction should extend a minimum horizontal distance of 2 feet from the edges of the foundations (i.e., approximate 1:1 projection from the bottom edges of the foundations).

Leighton should verify the vertical and lateral removal and overexcavation limits during grading as local conditions may require additional removals (i.e., encountering soft or unsuitable existing fill or other deleterious materials).

Subgrade Preparation

After completion of the overexcavations and prior to fill placement, the exposed soils should be scarified to a minimum depth of 4 inches, moisture conditioned to at least 2 to 4 percentage points above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D 1557. Any soft or unsuitable earth materials encountered at the bottom of the excavations should be removed and replaced with compacted fill.

Fill Placement

The onsite soils, less any deleterious material (construction debris) or organic matter, can be reused as fills. Oversized material greater than 6 inches in maximum dimension should not be placed in the fill. It should be noted that excavation in the sandstone bedrock is likely to produce oversized materials. Any soil to be placed as fill, whether onsite soils or imported material, should be reviewed and possibly tested by Leighton.

All fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to at least 2 to 4 percentage points above optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used.

Any required import material should consist of non-corrosive and predominantly granular soils with an Expansion Index (EI) of 20 or less. The imported materials should contain sufficient fines (binder material) so as to result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being transported to the site.

Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Field and laboratory data used in our calculations included laboratory-measured maximum dry density for the general soil type encountered at the subject site, the measured in-place densities of near surface soils encountered and our experience. We preliminarily estimate the onsite artificial fill materials requiring removal and recompaction will have a shrinkage factor of approximately 5 percent (±3 percent) during grading and bedrock materials requiring removal and recompaction will have a bulking factor of approximately 5 percent (±3 percent) during grading.

The level of fill compaction, variations in the dry density of the existing soil and bedrock and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

4.2 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the Standard Specifications for Public Works Construction, ("Greenbook"), 2015 Edition. Utility trenches can be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Sand:** A uniform, sand material that has a Sand Equivalent (SE) greater-than-or-equal-to 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), 2015 Edition.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. Native and clean fill soils can be used as backfill over the pipe bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

4.3 Foundation Recommendations

Conventional shallow foundations with slab-on-grade established on engineered fill may be used to support the proposed structures. Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 4.1

Based on the blow counts recorded during drilling and results of the laboratory testing results, the existing fill materials below the depth of recommended overexcavation and recompaction are considered suitable to support new structures. The laboratory tests indicate that the existing fill soils exhibit a low potential for hydro-consolidation.

Conventional Shallow Foundations

The design recommendations for working stress design are as follows:

Table 3 – Recommendations for Conventional Shallow Foundations

	Isolated Column Foundations	Continuous Strip Foundations
Width	2 feet	1 foot
Embedment	1.0 feet	
Sustained Dead plus Live Loads		
Bearing Pressure	3,000 pounds per square foot (psf) May increased by 200 psf per foot increase in depth or width to a maximum of 4,000psf and 4,500 psf for strip and isolated column footing.	
Frictional Resistance	0.40	
Passive Resistance	280 pounds per cubic foot (pcf) Maximum 4,000 psf	
Short-term Loads (i.e., Seismic and Wind)		
Bearing pressure, friction, and passive resistance can be increased by one-third for short-term loading. The passive resistance should be reduced by one-third when combined with frictional resistance to calculate total resistance where seismically induced lateral displacement potential does not exist.		

The estimated settlement of the foundation under the recommended bearing pressure will be less than 1 inch. Because the foundation will be established in compacted fill consisting of predominately granular materials, most of the settlement will occur during construction. Furthermore, the existing fill was placed at least 45 years ago and has undergone most of the consolidation under its own weight as suggested by the consolidation test results. Therefore, we do not expect the new buildings will experience adverse effects due to long-term settlement of the fill.

Slab-on-Grade

Based on our subsurface explorations, the existing shallow fill materials at the site are predominately granular. Therefore, from a geotechnical standpoint, conventional slabs-on-grade should be at least 4 inches thick with No. 3 rebar placed at center of the slab at 18 inches on center at each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions in accordance with the current California Building Code (CBC) for a soil with low expansion potential. The recommended maximum joint spacing for the slab should not exceed 15 feet. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the Expansion Index (EI) of near-surface subgrade soils upon completion of grading.

The following parameters may be used to design the slab-on-grade:

Table 4 – Recommendations for Conventional Slabs-on-Grade

Parameters	Recommended Values
Expansion Potential	Low
Slab Thickness	4 inches (minimum)
Subgrade Reaction	200 pounds per cubic inch (pci)
Bearing Capacity	1,500 psf
Maximum joint spacing should not exceed 15 feet. The moisture of the subgrade soils should be at 120% optimum moisture to a depth of 16 inches below the slab. The subgrade soils should be evaluated by the geotechnical engineer to verify adequate moisture conditioning has been maintained prior to pouring concrete. prior to pouring concrete.	

Minor cracking of the concrete as it cures, due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. The structural engineer may consider using additional reinforcement in slabs and foundations to reduce the potential for concrete cracking.

Interior slabs-on-grade are recommended to be underlain by a synthetic sheeting to serve as a retarder to moisture vapor transmission in areas where moisture-sensitive floor covering (such as vinyl, tile, or carpet) or equipment is planned. The sheeting is recommended to be a minimum 15-mil thick Stego® Wrap installed per manufacturer's specifications. Prior to installing the synthetic sheeting, the exposed subgrade surface should be clear of all extruding rock and gravel that could damage the sheeting. The sheeting should be evaluated for the presence of punctures or tears by the installer prior to pouring concrete. Installation of the sheeting should include proper overlap and taping of seams.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

These recommended design parameters are based on responsibly maintained improvements. Such improvements include properly designed planters, if adjacent to structures. In utilizing these parameters, the structural engineer should design the foundation system to the acceptable deflection criteria determined by the architect.

We recommend that soil moisture around the immediate perimeter of the slab be maintained near optimum-moisture content (or above) during construction and up to occupancy of the structures.

Our recommendations assume a reasonable degree of owner responsibility. Property owners should be informed and educated regarding the importance of maintaining a constant level of soil moisture. Owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing expansive soils to become too dry (i.e., the soil will undergo shrinkage as it dries up, followed by swelling during the rainy season or when irrigation is resumed, resulting in potential distress to improvements and structures). Planters should not be located adjacent to foundations unless they are properly designed with drainage. Trees should also not be planted adjacent to foundations. Lawn and other landscaped areas should have proper drainage, and should not allow water to pond adjacent to structures. If the owners do not adequately maintain correct irrigation and drainage, some degree of foundation movement may occur.

4.4 Surface Drainage

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from buildings a minimum of 2 percent for earthen surfaces for a lateral distance of at least five feet and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by the use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without over watering.

4.5 Corrosion Protection Measures

For screening purposes, a representative near-surface bulk soil sample was tested for corrosivity to preliminarily evaluate corrosion potential to buried concrete (e.g., footings, retaining walls) and buries ferrous pipes. The chemical analysis test results are included in Appendix B of this report and are summarized in the table below:

Table 5 – Corrosivity Test Results

Test Parameter	Test Results	General Classification of Hazard
Water-Soluble Sulfate in Soil (ppm)	91	Negligible sulfate exposure to buried concrete
Water-Soluble Chloride in Soil (ppm)	11	Non-corrosive to buried concrete
pH	7.74	Mildly alkaline
Minimum Resistivity (saturated, ohm-cm)	2400	Corrosive to buried ferrous pipes (per Caltrans)

Based on the measured water-soluble sulfate content from the tested soil sample, concrete in contact with the soil is expected to have negligible exposure to sulfate attack per ACI 318-11. The sample tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil. Therefore, common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with CBC 2016 requirements. Type V cement should be used for concrete exposed to recycled water.

The results of the resistivity test indicate that the underlying soil is corrosive to buried ferrous metals per ASTM STP 1013. A registered corrosion engineer may be consulted to provide specific mitigation measures for protection of buried metals in direct contact with onsite soils.

4.6 Retaining Walls

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 9, *Retaining Wall Backfill and Subdrain Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall.

Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

- Active Pressure Coefficient, k_a : 0.307
- At-rest Pressure Coefficient, k_0 : 0.441
- Seismic Pressure Coefficient, k_E : 0.41 (for walls taller than 12 feet)

The passive pressure coefficient for a level ground surface is as follows:

- Passive Pressure Coefficient, $k_{p\,:}$ 3.537

The equivalent fluid pressure (EFP) can be calculated using a moist unit weight of 120 pounds per cubic foot (pcf) for the onsite granular soils. The seismic pressure should be applied as an invert triangle with the resultant at 0.6 times the height of the wall.

Recommendations for strip foundation presented in Section 4.3 may be used for designing the foundations for free-standing retaining walls.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

4.7 Concrete Flatwork

Exterior concrete slabs-on-grade should have a minimum thickness of 4 inches. Common Type II cement should be adequate for concrete flatwork not exposed to recycled water. Type V cement should be used for concrete exposed to recycled water. Concrete flatwork should be placed on previously compacted fill. If this material has been disturbed, the subgrade soil to a depth of 12 inches should be moisture conditioned to slightly above optimum moisture content and recompacted to minimum 90 percent relative compaction.

Exterior concrete driveways, ramps, curbs, gutters, sidewalks, patio slabs, and swimming pool decks, often crack. Inclusion of joints at frequent intervals and reinforcement will help control the locations of the cracks, and thus reduce *the* unsightly appearance. Construction or weakened plane joints should be spaced at intervals of 8 feet or less for driveways, ramps, sidewalks, patio slabs, pool decks, curbs and gutters. If cracking occurs, repairs may be needed to mitigate the trip hazard and/or improve the appearance.

Cracking of concrete is often not due to settlement or heave of soils, but often due to other factors such as the use of too high a water/cement ratio and/or inadequate steps being taken to prevent moisture loss during curing. These causes of concrete distress can be reduced by proper design of the concrete mix, and by proper placement and curing of the concrete.

4.8 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations, limited laboratory testing and information available at the time the report is prepared. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project.

Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- During overexcavation and removal of unsuitable soil;
- Subgrade preparation;
- Compaction of all fill materials;
- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- Pavement subgrade and base preparation;
- Placement of asphalt concrete and/or concrete; and
- When any unusual conditions are encountered.

5.0 LIMITATIONS

This report was based solely on data obtained from a limited number of geotechnical exploration, and soil samples and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton has the opportunity to observe subsurface conditions during grading and construction, to confirm that our preliminary data are representative for the site. Leighton should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. The findings, conclusion, and recommendations included in this report are considered preliminary and are subject to verification. We do not make any warranty, either expressed or implied.

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

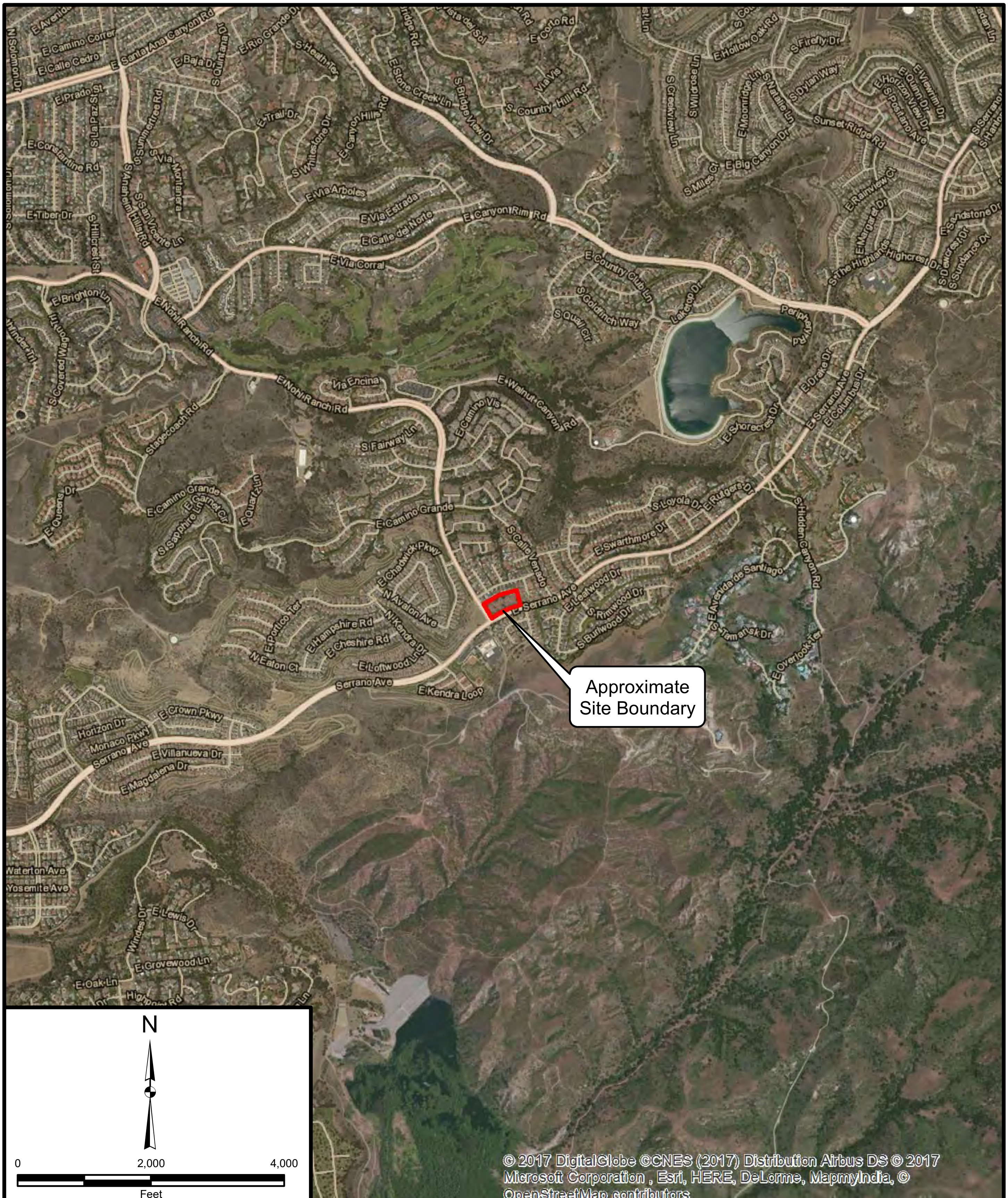
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 11737.001	Eng/Geol: VPI/JMP	SITE LOCATION MAP	Figure 1
Scale:1 " = 2,000 '	Date: October 2017	Proposed Residential Development 6501-6513 E. Serrano Avenue Anaheim, California	Leighton
Base Map: ESRI ArcGIS Online 2017 Thematic Information: Leighton Author: Leighton Geomatics (btran)			

Map Saved as V:\Drafting\11737\001\Maps\11737-001_F01_SLM_2017-10-05.mxd on 9/25/2017 2:04:27 PM

LEGEND

LB-6 T.D.76.5' Af 0-76.5' No G.W. — APPROXIMATE LOCATION OF HOLLOW-STEM AUGER BORING SHOWING TOTAL DEPTH (T.D.) AND DEPTH OF GEOLOGIC UNITS IN FEET BELOW GROUND SUFACE

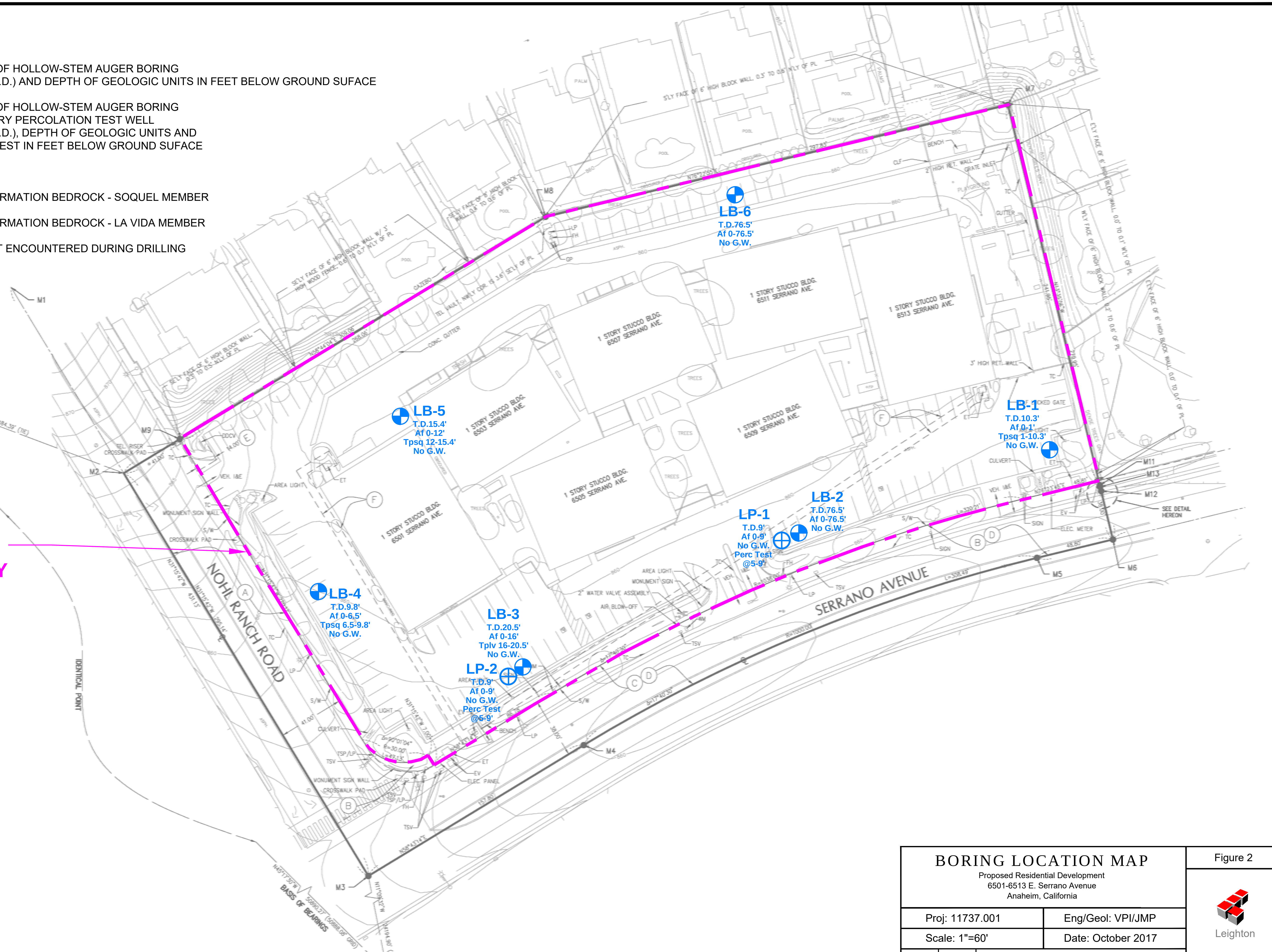
LP-2 T.D.9' Af 0-9' No G.W. Perc Test @4-9' — APPROXIMATE LOCATION OF HOLLOW-STEM AUGER BORING CONVERTED TO TEMPORARY PERCOLATION TEST WELL SHOWING TOTAL DEPTH (T.D.), DEPTH OF GEOLOGIC UNITS AND DEPTH OF PERCOLATION TEST IN FEET BELOW GROUND SUFACE

Af — ARTIFICIAL FILL

Tpsq — TERTIARY AGE PUENTE FORMATION BEDROCK - SOQUEL MEMBER

Tplv — TERTIARY AGE PUENTE FORMATION BEDROCK - LA VIDA MEMBER

GROUNDWATER (G.W.) NOT ENCOUNTERED DURING DRILLING



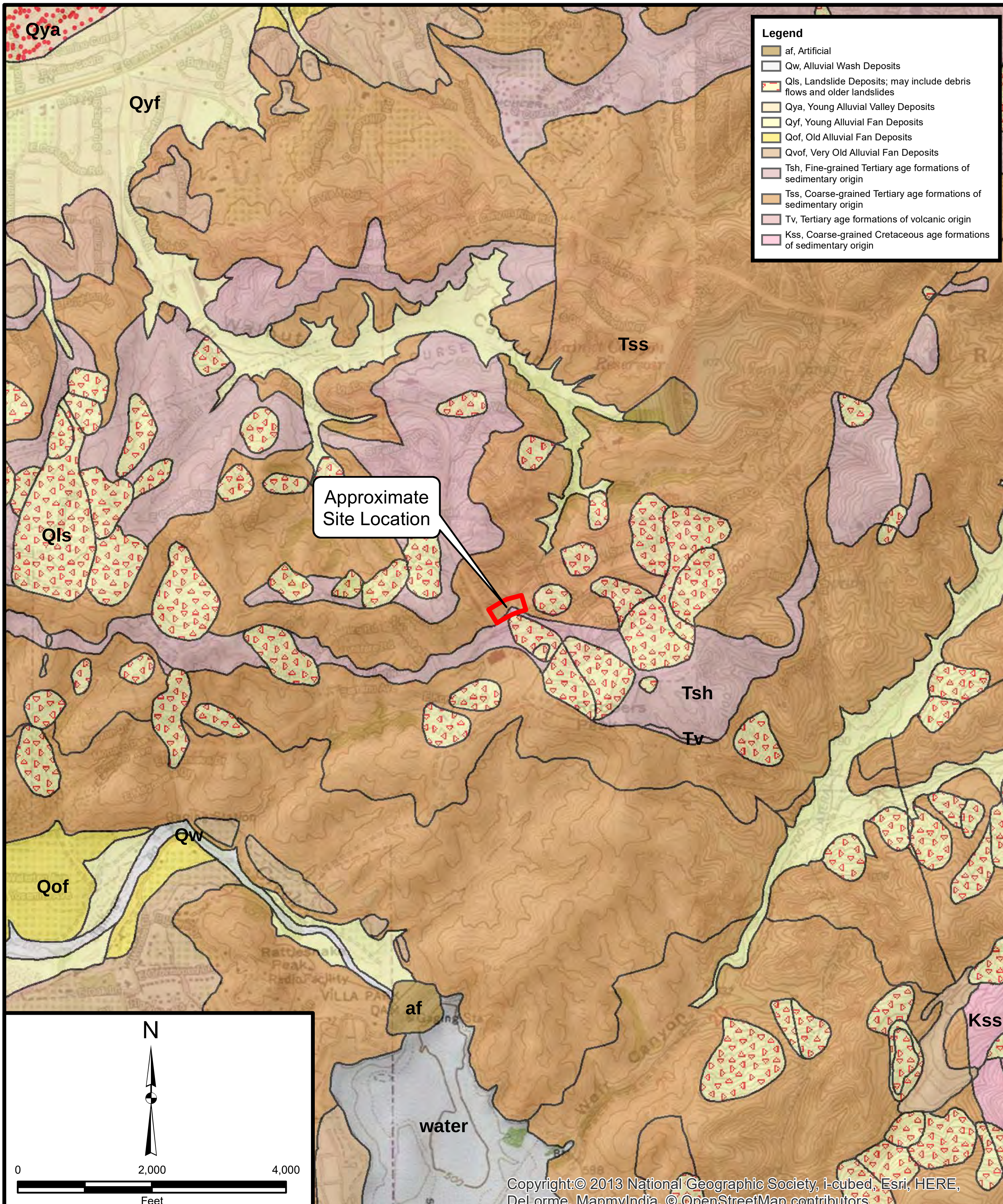
BORING LOCATION MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Proj: 11737.001	Eng/Geol: VPI/JMP
Scale: 1"=60'	Date: October 2017

Figure 2

Leighton



Project: 11737.001	Eng/Geol: VPI/JMP
Scale:1 " = 2,000 '	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, USGS
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Figure 3

Leighton

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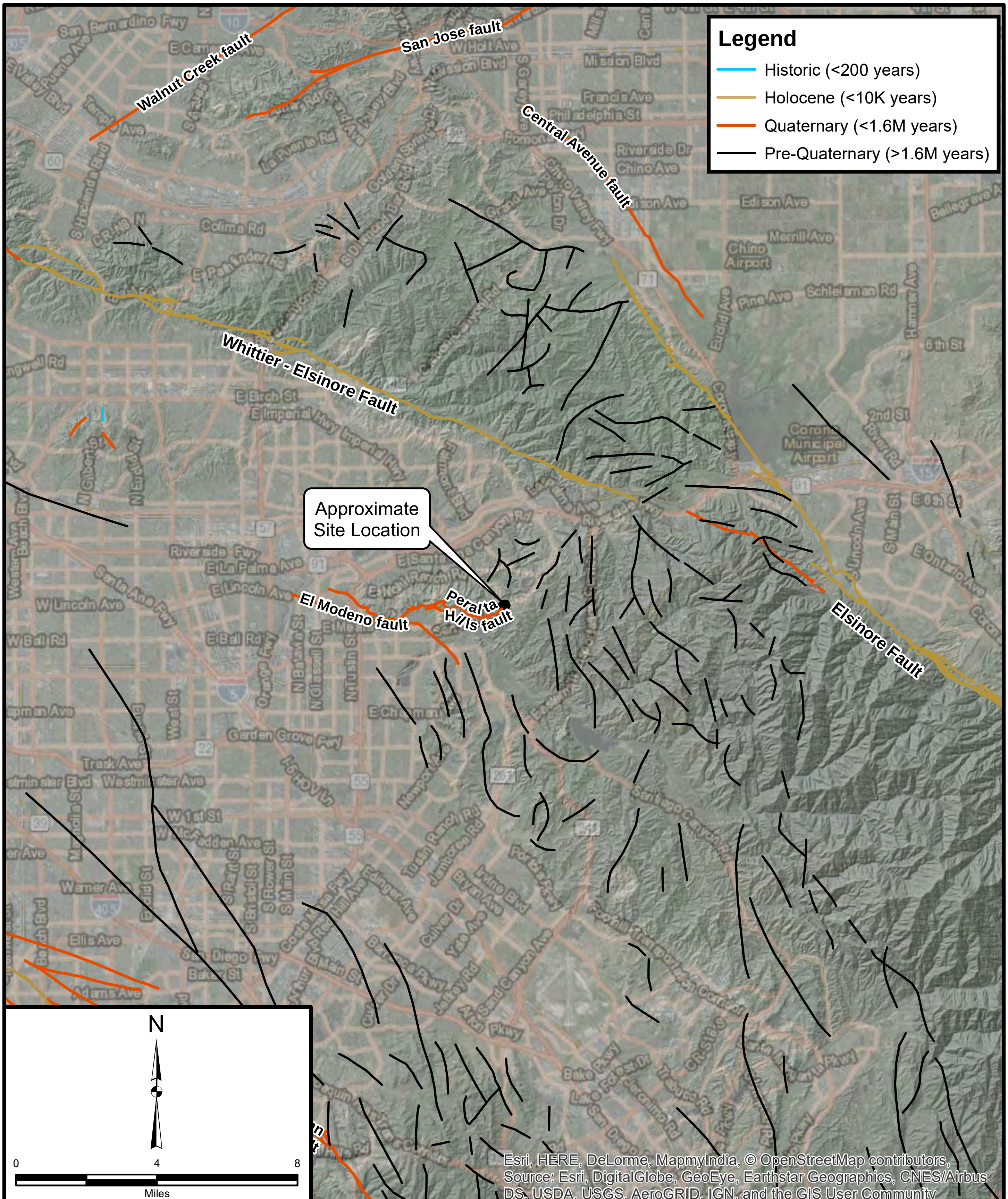

Project: 11737.001	Eng/Geol: VPI/JMP
Scale:1 " = 4 miles	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS
Author: Leighton Geomatics (btran)

REGIONAL FAULT MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Figure 4

Leighton

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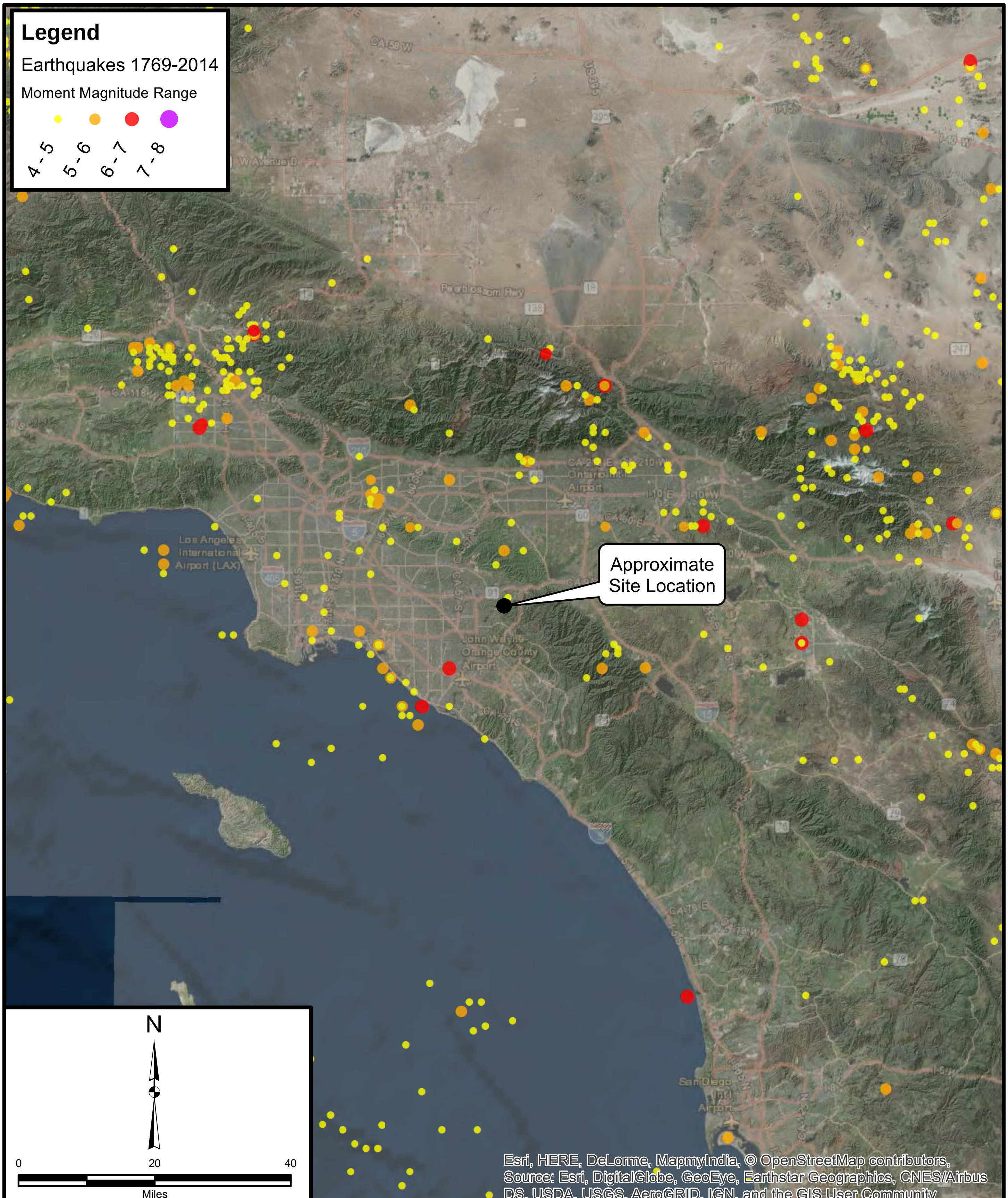

Project: 11737.001	Eng/Geol: VPI/JMP
Scale:1 " = 20 miles	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, USGS, SCEC
Author: Leighton Geomatics (btran)

HISTORIC SEISMICITY MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Figure 5

Leighton

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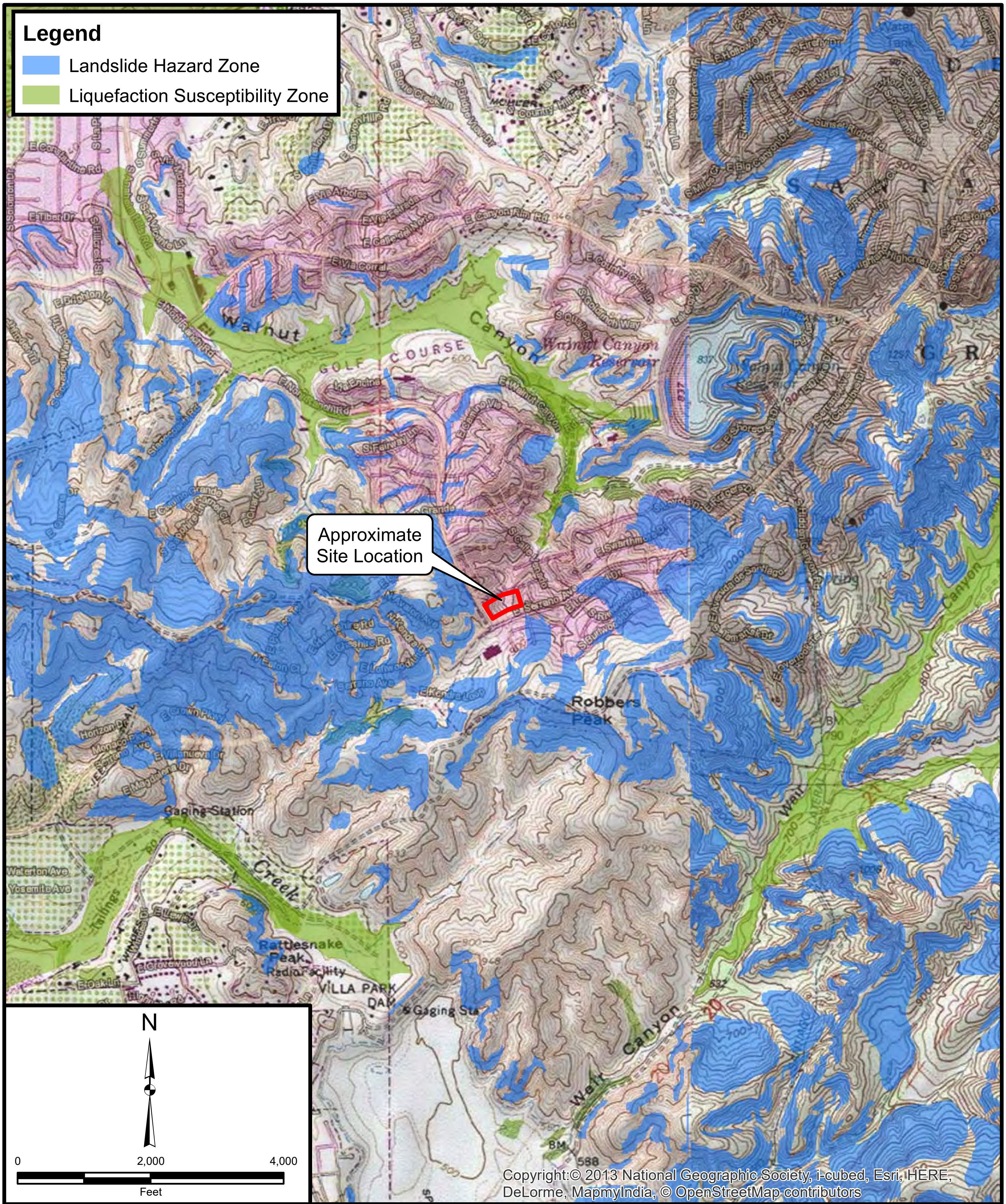


Project: 11737.001	Eng/Geol: VPI/JMP
Scale:1 " = 2,000 '	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, CGS
Author: Leighton Geomatics (btran)

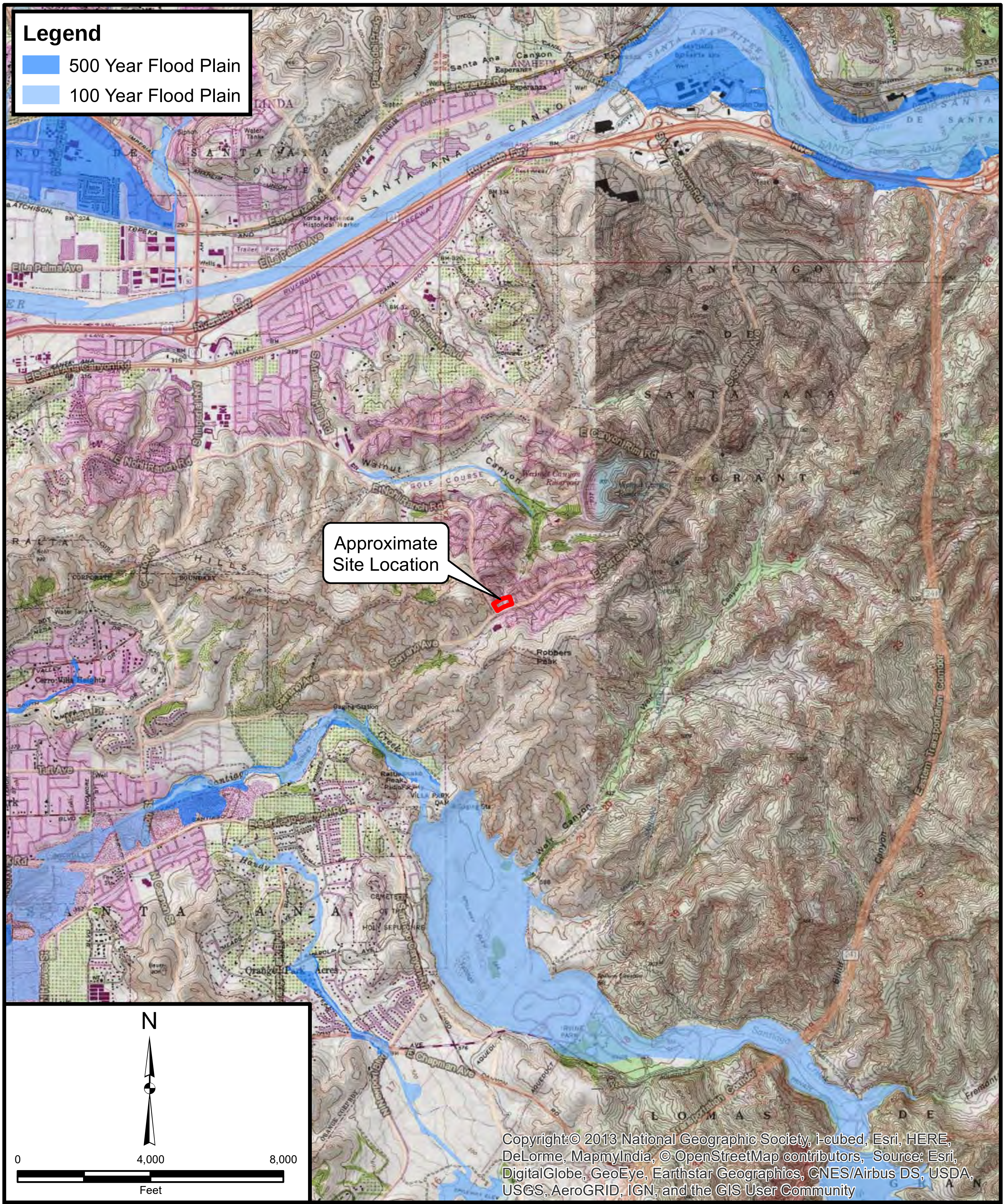
SEISMIC HAZARD MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Figure 6

Leighton

Map Saved as V:\Drafting\11737\001\Maps\11737-001_F06_SHM_2017-10-05.mxd on 9/26/2017 3:48:01 PM



Project: 11737.001	Eng/Geol: VPI/JMP
Scale:1 " = 4,000 '	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (btran)

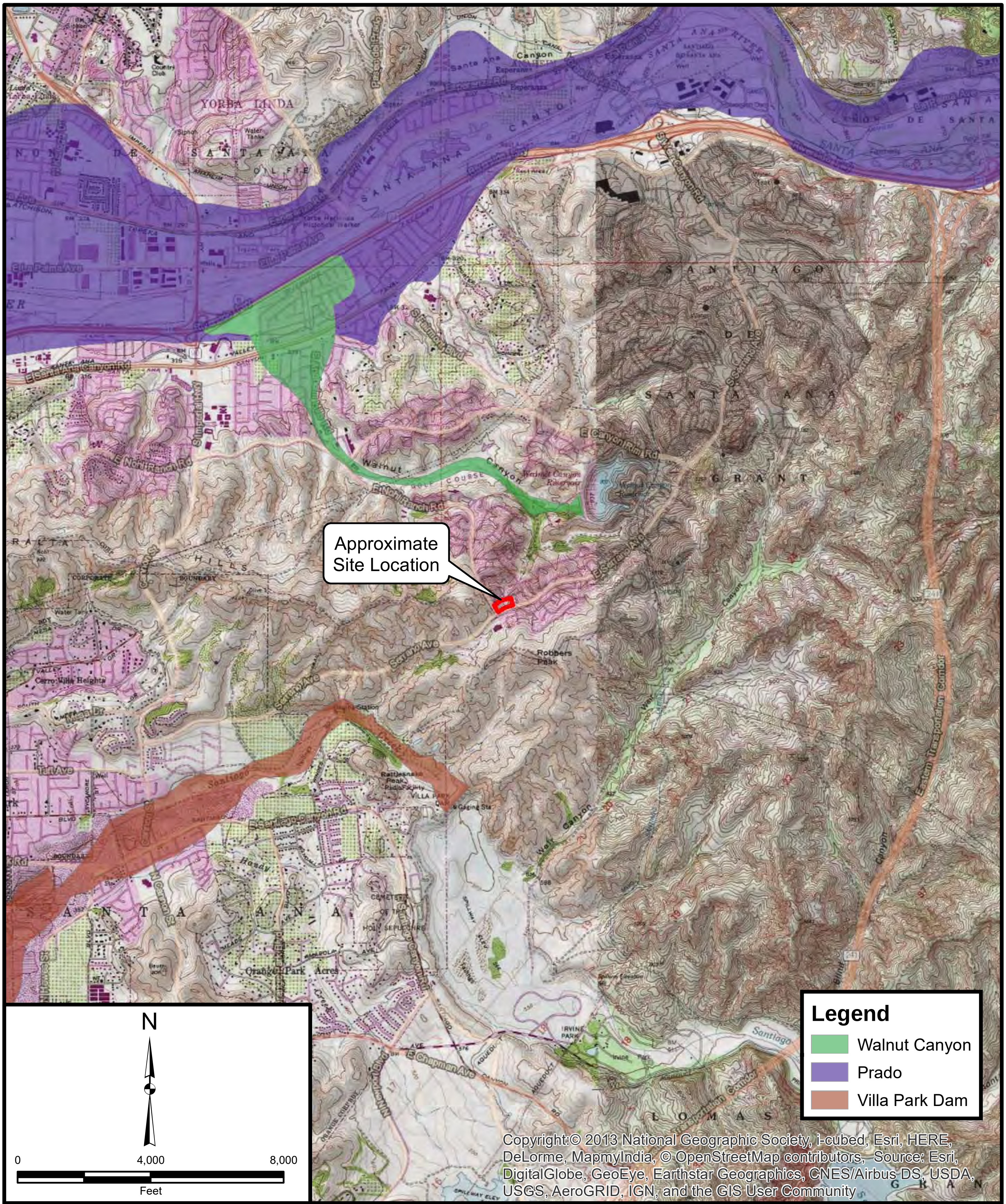
FLOOD HAZARD ZONE MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Figure 7

Leighton

Map Saved as V:\Drafting\11737\001\Maps\11737-001_F07_FHM_2017-10-05.mxd on 9/25/2017 2:23:43 PM



Project: 11737.001	Eng/Geol: VPI/JMP
Scale:1 " = 4,000 '	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (btran)

DAM INUNDATION MAP

Proposed Residential Development
6501-6513 E. Serrano Avenue
Anaheim, California

Figure 8

Leighton

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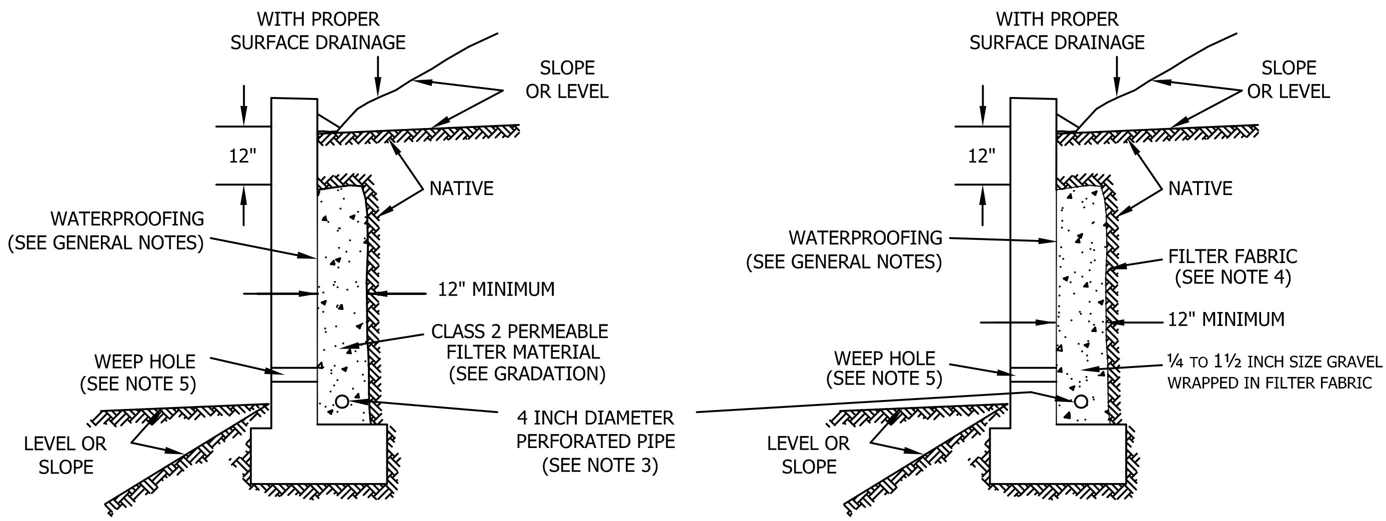
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton
Figure 9

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APPENDIX A

FIELD EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	858'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB-1					@surface: 5-inches asphalt concrete over 5-inches aggregate base	DS,CR, MD,RV, SA
									Bedrock: Puente Formation - Soquel Member (Tpsq): @0.8': SANDSTONE, gray brown to orange brown, moist, dense, fine to medium sand	
855										
	5			R1	33 50/4"				@5': very dense	
850										
	10			R2	50/4"				@10': limited recovery in sampler shoe only	
									Total Depth of Boring: 10.3 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and patched with cold-mix asphalt	
845										
	15									
840										
	20									
835										
	25									
830										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-2

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	859'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			BB-1				SM	@surface: 5.5-inches asphalt concrete over 3-inches aggregate base **Artificial Fill (Af):** @0.7': Silty SAND, orange brown, moist, tight, fine to medium sand, material derived from local bedrock	SA
855	5			R1	10 11 12	104	21	CL-SC	@5': Sandy CLAY to Clayey SAND, orange brown, moist, stiff/medium dense, some siltstone clasts	
				S1	10 7 10			SM	@7.5': Silty SAND with clay, orange brown, slightly moist to moist, medium dense, fine to medium sand	
850	10			R2	11 14 31	117	13		@10': Silty SAND with clay, orange brown, moist, medium dense, fine to medium sand, some siltstone/sandstone clasts	-200
845	15			R3	16 28 50/5"				@15': Silty SAND, orange brown, moist, very dense, fine to medium sand, some clasts of sandstone	
840	20			R4	10 24 43	119	13		@20': Silty SAND with clay, orange brown, moist, dense, fine to medium sand, some siltstone clasts	-200
835	25			R5	10 18 28			SM-ML	@25': Silty SAND to Sandy SILT, orange brown to gray brown, medium dense/very stiff, fine to medium sand, some sandstone clasts	
830	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
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- SG SPECIFIC GRAVITY
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* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-2

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	859'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

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Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S2	2 3 7			SM	@30': Silty SAND with clay, orange brown to gray brown, moist, stiff, some siltstone/sandstone clasts	-200
825										
	35			R6	10 18 29	109	16	SM-SC	@35': Silty SAND to Clayey SAND, orange brown, moist, medium dense, fine to medium sand, some siltstone/sandstone clasts	
820										
	40			S3	7 7 11				@40': same as above	
815										
	45			R7	15 40 45			SC	@45': Clayey SAND with gravel, orange brown, very moist, hard/very dense, fine to medium sand	-200, CN
810										
	50			S4	4 5 7			CL	@50': Sandy Lean CLAY with silt, orange brown to gray brown, moist to very moist, medium stiff, some siltstone/sandstone clasts	AL
805										
	55			R8	7 20 30	99	24	SC	@55': Clayey SAND to Sandy CLAY, orange brown to gray brown, moist to very moist, dense/very stiff, fine to medium sand, some siltstone/sandstone clasts	
800										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
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- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-2

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
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Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	60			S5	6 7 9			SC	@60': Clayey SAND, orange brown, moist to very moist, medium dense, some siltstone/sandstone clasts	-200
795										
	65			R9	7 16 37	96	28	SM	@65': Silty SAND, gray to bluish green, moist to very moist, very dense, fine to medium sand, abundant siltstone/sandstone clasts	
790										
	70			S6	4 9 12			CL	@70': Sandy Lean CLAY, gray to orange brown, moist to very moist, very stiff, with abundant siltstone/sandstone clasts	-200
785										
	75			R10	8 24 34	97	27		@75': Silty CLAY, gray brown to bluish green, moist to very moist, hard, with siltstone clasts	
									Total Depth of Boring: 76.5 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and patched with cold-mix asphalt	
780										
	80									
775										
	85									
770										
	90									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-3

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	859'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BB1				SM	@surface: 3.5-inches asphalt concrete over 3-inches aggregate base **Artificial Fill (Af):** @0.6': Silty SAND with clay, orange brown to gray brown, moist, tight, fine to medium sand	SA
855	5			R1	5 21 45			CL-SC	@5': Sandy CLAY to Clayey SAND, orange brown to brown, moist, hard/dense, fine to medium sand	
				S1	8 9 10			SM	@7.5': Silty SAND, orange brown to gray, moist, medium dense, fine to medium sand, some siltstone/sandstone clasts	
850	10			R2	16 21 27				@10': Silty SAND, orange brown to blue gray, moist, dense, fine to medium sand, some siltstone/sandstone clasts	
845	15			R3	8 35 50/2"			CL	@15': Sandy CLAY, orange brown, moist, stiff, fine sand, some siltstone/sandstone clasts **Bedrock: Puente Formation - La Vida Member (Tplv):** @16': SILTSTONE, gray to orange brown, slightly moist, hard	
840	20			R4	50/5"				@20': SILTSTONE, orange brown to gray, slightly moist, hard, oxidized Total Depth of Boring: 20.5 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and patched with cold-mix asphalt	
835	25									
830	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-4

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	860'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
860	0								@surface: 3.5-inches asphalt concrete over 3-inches aggregate base	
									Artificial Fill (Af): @0.6': Silty SAND, orange brown, slightly moist, tight, fine to medium sand, few gravels	
855	5			R1	11 15 20			SM	@5': Silty SAND, orange brown to gray brown, slightly moist, medium dense, fine to medium sand, some sandstone clasts	SA
									Bedrock: Puente Formation - Soquel Member (Tpsq): @6.5': harder drilling, approximate bedrock contact assumed	
				R2	50/5"				@8': no recovery, hard	
				S1	37 50/4"				@9': SANDSTONE, light yellow brown, slightly moist, hard, fine to medium sand	
850	10								Total Depth of Boring: 9.8 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and patched with cold-mix asphalt	
845	15									
840	20									
835	25									
830	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-5

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	861'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BB1				SM	@surface: 3.5-inches asphalt concrete over 3-inches aggregate base **Artificial Fill (Af):** @0.6': Silty SAND, medium brown to orange brown, moist, tight, fine to medium sand, few gravels	SA
860										
855	5			R1	11 13 17				@5': Silty SAND, medium brown to orange brown, moist, medium dense, fine to medium sand, some fine siltstone/sandstone clasts	
850	10			R2	17 40 41				@10': dense	
									Bedrock: Puente Formation - Soquel Member (Tpsq): @12': harder drilling, approximate bedrock contact assumed	
845	15			R3	50/5"				@15': SANDSTONE, light gray with orange oxidation, moist, hard, fine to medium sand	
									Total Depth of Boring: 15.4 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and patched with cold-mix asphalt	
840	20									
835	25									
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-6

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	860'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
860	0			BB1				SM	@surface: 2.5-inches asphalt concrete over 3.5-inches aggregate base **Artificial Fill (Af):** @0.6': Silty SAND, medium orange brown, moist, tight, fine to medium sand	
855	5			R1	14 27 50/5"	119	10		@5': Silty SAND, orange brown, moist, very dense, fine to medium sand, some sandstone clasts	
850	10			R2	10 21 48				@10': some large siltstone clasts	
845	15			R3	9 15 21	95	23	SC-CL	@15': Clayey SAND to Sandy CLAY, orange brown to gray, moist, medium dense/very stiff, some siltstone clasts	
840	20			R4	8 31 50/5"			SM	@20': Silty SAND, orange brown, moist, very dense, fine to medium sand, some sandstone clasts	
835	25			R5	20 50/6"	116	12			
830	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-6

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	860'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
830	30			R6	22 42 50/5"			SM-SC	@30': Silty SAND to Clayey SAND, orange brown, moist, very dense, fine to medium sand, some fine siltstone/sandstone clasts	
825	35			R7	15 28 49	117	14		@35': dense, abundant siltstone	
820	40			R8	38 38 42				@40': dense, abundant siltstone/sandstone clasts	
815	45			R9	7 21 50/4"	116	14		@45': orange brown to blue gray, dense, abundant siltstone/sandstone clasts	
810	50			R10	50/4"				@50': no recovery, possible cobble at head of auger	
805	55			R11	25 50/6"				@55': no recovery, possible cobble at head of auger	
800	60									

SAMPLE TYPES:
- **B** BULK SAMPLE
- **C** CORE SAMPLE
- **G** GRAB SAMPLE
- **R** RING SAMPLE
- **S** SPLIT SPOON SAMPLE
- **T** TUBE SAMPLE

TYPE OF TESTS:
- **-200** % FINES PASSING
- **AL** ATTERBERG LIMITS
- **CN** CONSOLIDATION
- **CO** COLLAPSE
- **CR** CORROSION
- **CU** UNDRAINED TRIAXIAL
- **DS** DIRECT SHEAR
- **EI** EXPANSION INDEX
- **H** HYDROMETER
- **MD** MAXIMUM DENSITY
- **PP** POCKET PENETROMETER
- **RV** R VALUE
- **SA** SIEVE ANALYSIS
- **SE** SAND EQUIVALENT
- **SG** SPECIFIC GRAVITY
- **UC** UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-6

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	860'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
800	60			S1	39 38 42				@60': limited recovery in sampler shoe limited to mechanically broken cobble fragments	
795	65			S2	6 8 18			SM	@65': Silty SAND, orange brown to gray brown, moist, medium dense, fine to medium sand, some siltstone/sandstone clasts	
790	70			R12	12 28 50/4"	102	22	CL	@70': Silty CLAY, blue gray, moist, hard, large siltstone clasts	
785	75			R13	15 19 20			SM	@75': Silty SAND, blue gray, very moist to wet, medium dense, fine to medium sand, some sandstone clasts	CN, AL
									Total Depth of Boring: 76.5 feet bgs No groundwater encountered during drilling Boring backfilled with soil cuttings and patched with cold-mix asphalt	
780	80									
775	85									
770	90									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LP-1

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	859'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	@surface: 5.5-inches asphalt concrete over 3-inches aggregate base **Artificial Fill (Af):** @0.7': Silty SAND, orange brown, moist, tight, fine to medium sand, material derived from local bedrock	
855	5							CL-SC	@5': Sandy CLAY to Clayey SAND, orange brown, moist, some siltstone fragments	
								SM	@7.5': Silty SAND with clay, orange brown, slightly moist to moist, fine to medium sand	
850	10								No sampling performed, lithology inferred from adjacent boring LB-2 Total Depth of Boring: 9 feet bgs No groundwater encountered during drilling Temporary percolation well installed: 2-inch solid PVC @ 0-4 feet bgs 2-inch slotted PVC (0.020") @ 4-9 feet bgs #3 Monterey Sand @ 3-9 feet bgs Well casing removed upon completion of testing and boring backfilled with soil cuttings and patched with cold-mix asphalt	
845	15									
840	20									
835	25									
830	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LP-2

Project No.	11737.001	**Date Drilled**	8-16-17
Project	Serrano - Nohl Ranch Condos	**Logged By**	JMP
Drilling Co.	Martini Drilling Corp.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	859'
Location	6501-6513 Serrano Avenue, Anaheim, CA	**Sampled By**	JMP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	@surface: 3.5-inches asphalt concrete over 3-inches aggregate base **Artificial Fill (Af):** @0.6': Silty SAND with clay, orange brown to gray brown, moist, tight, fine to medium sand	
855	5							CL-SC	@5': Sandy CLAY to Clayey SAND, orange brown to brown, moist, fine to medium sand	
850								SM	@7.5': Silty SAND, orange brown to gray, moist, fine to medium sand, some siltstone/sandstone clasts	
	10								No sampling performed, lithology inferred from adjacent boring LB-3 Total Depth of Boring: 9 feet bgs No groundwater encountered during drilling Temporary percolation well installed: 2-inch solid PVC @ 0-4 feet bgs 2-inch slotted PVC (0.020") @ 4-9 feet bgs #3 Monterey Sand @ 3-9 feet bgs Well casing removed upon completion of testing and boring backfilled with soil cuttings and patched with cold-mix asphalt	
845	15									
840	20									
835	25									
830	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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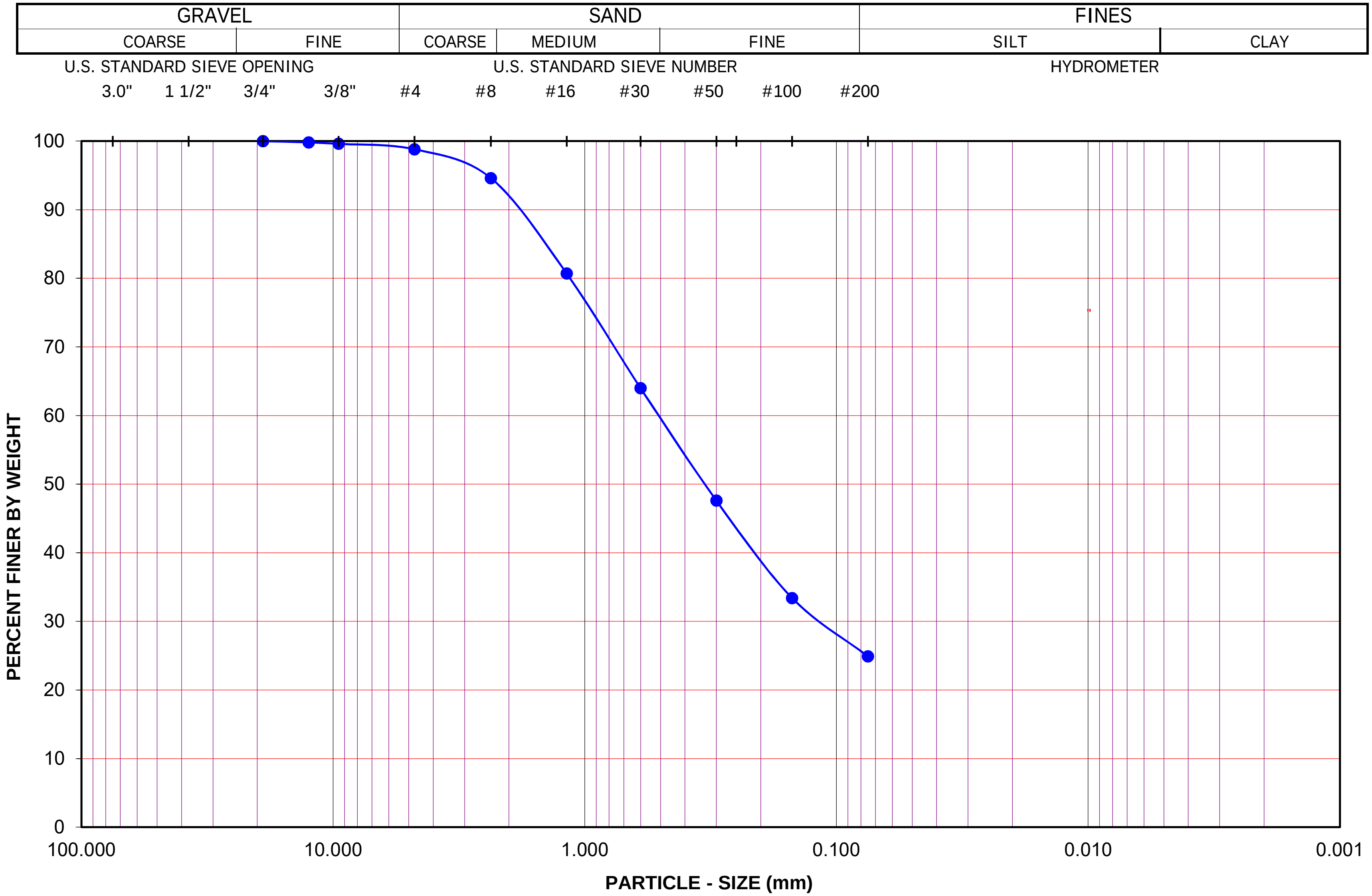
APPENDIX B

LABORATORY TEST RESULTS



Project Name: Serrano

Project No.: 11737.001

Boring No.: LB-1 Sample No.: BB1

Depth (feet): 0-5 Soil Type : SC

Soil Identification: Olive brown silty, clayey sand (SC-SM)

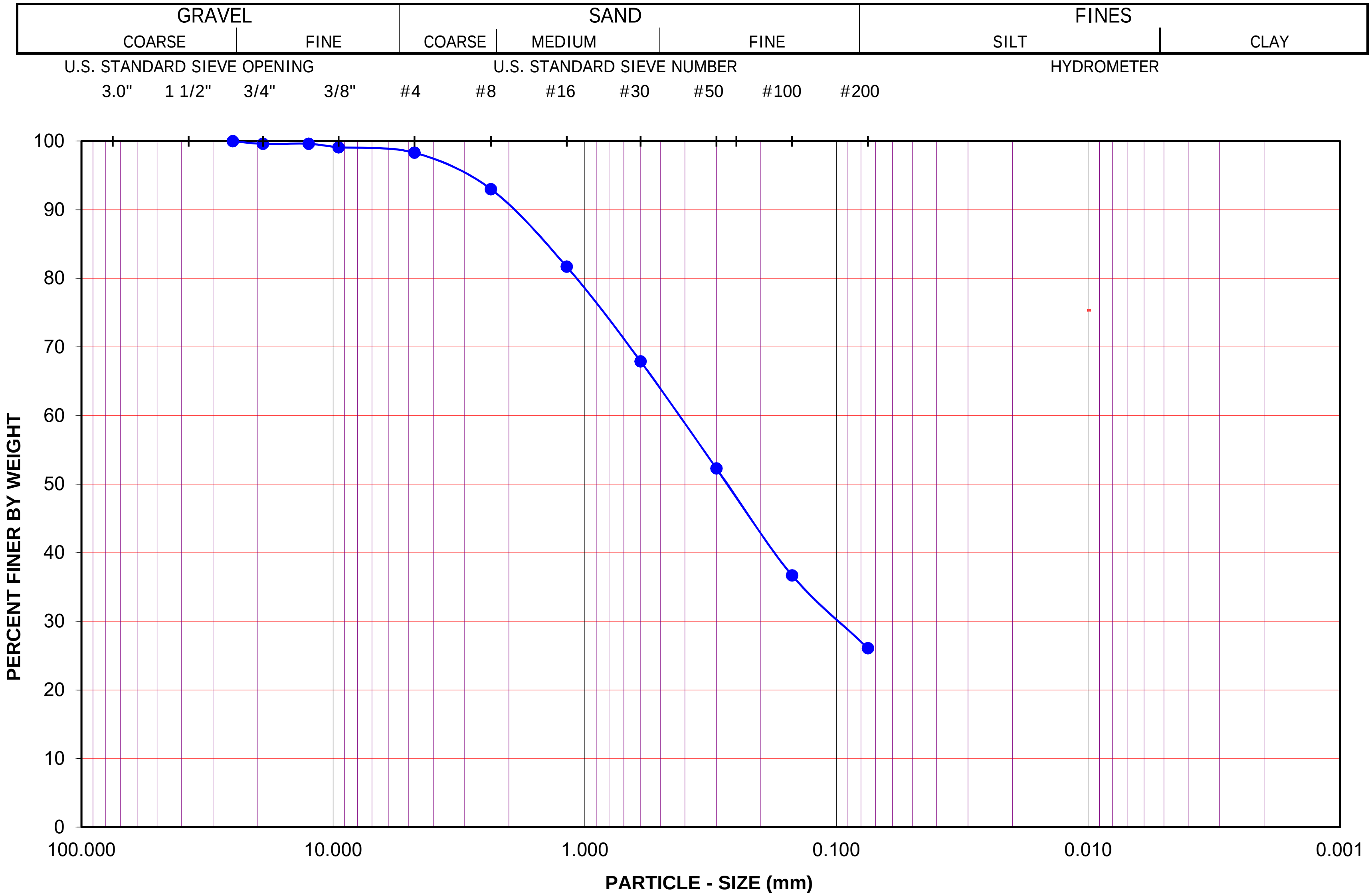
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 74 : 25

Sep-17

SA LB-1, BB1 @ 0-5



Project Name: Serrano

Project No.: 11737.001

Boring No.: LB-2 Sample No.: BB1

Depth (feet): 0-5 Soil Type : SM

Soil Identification: Yellowish brown silty sand (SM)

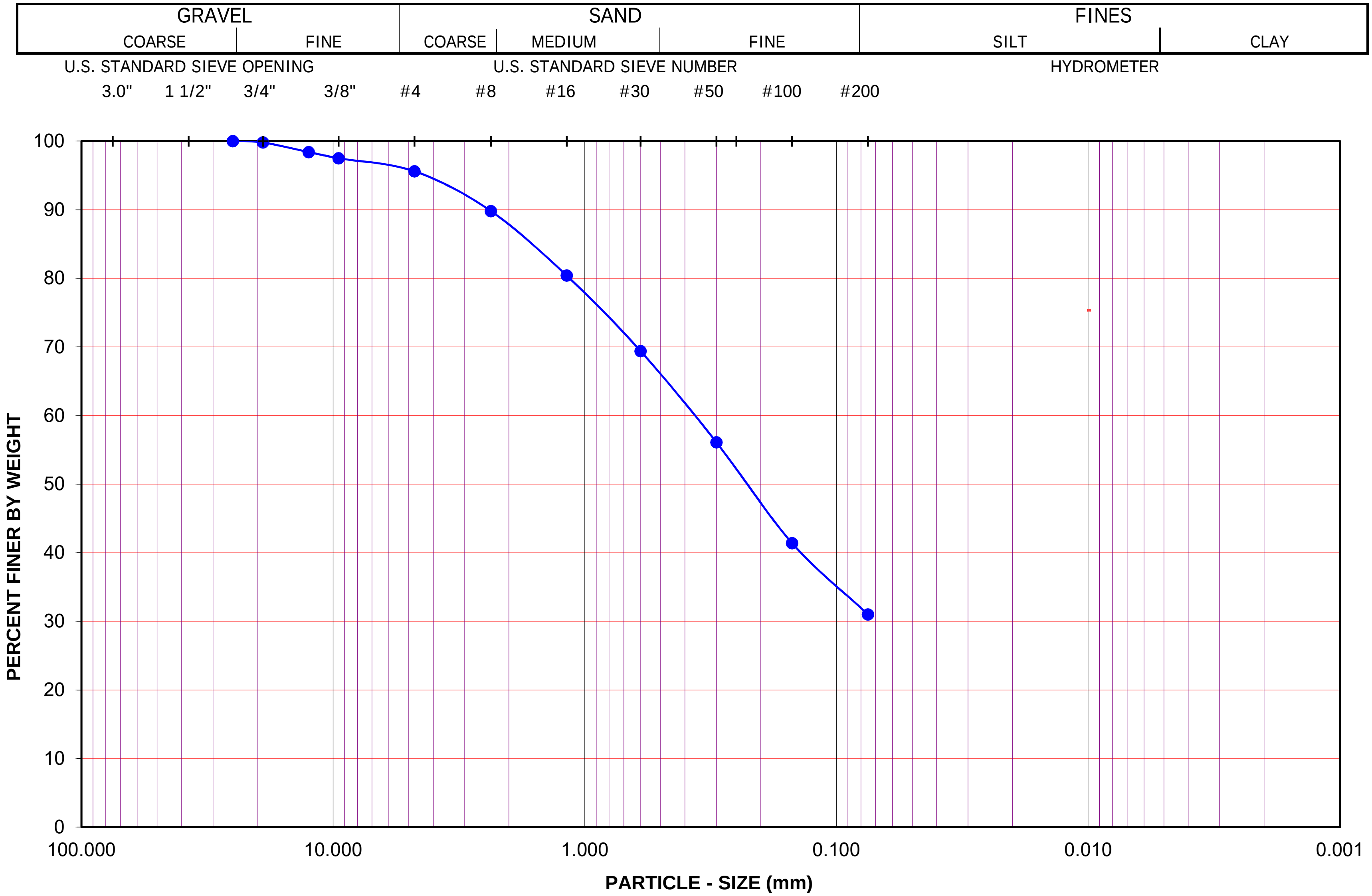
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 2 : 72 : 26

Sep-17

SA LB-2, BB1 @ 0-5



Project Name: Serrano

Project No.: 11737.001

Boring No.: LB-3 Sample No.: BB1

Depth (feet): 0-5 Soil Type : SM

Soil Identification: Yellowish brown silty sand (SM)

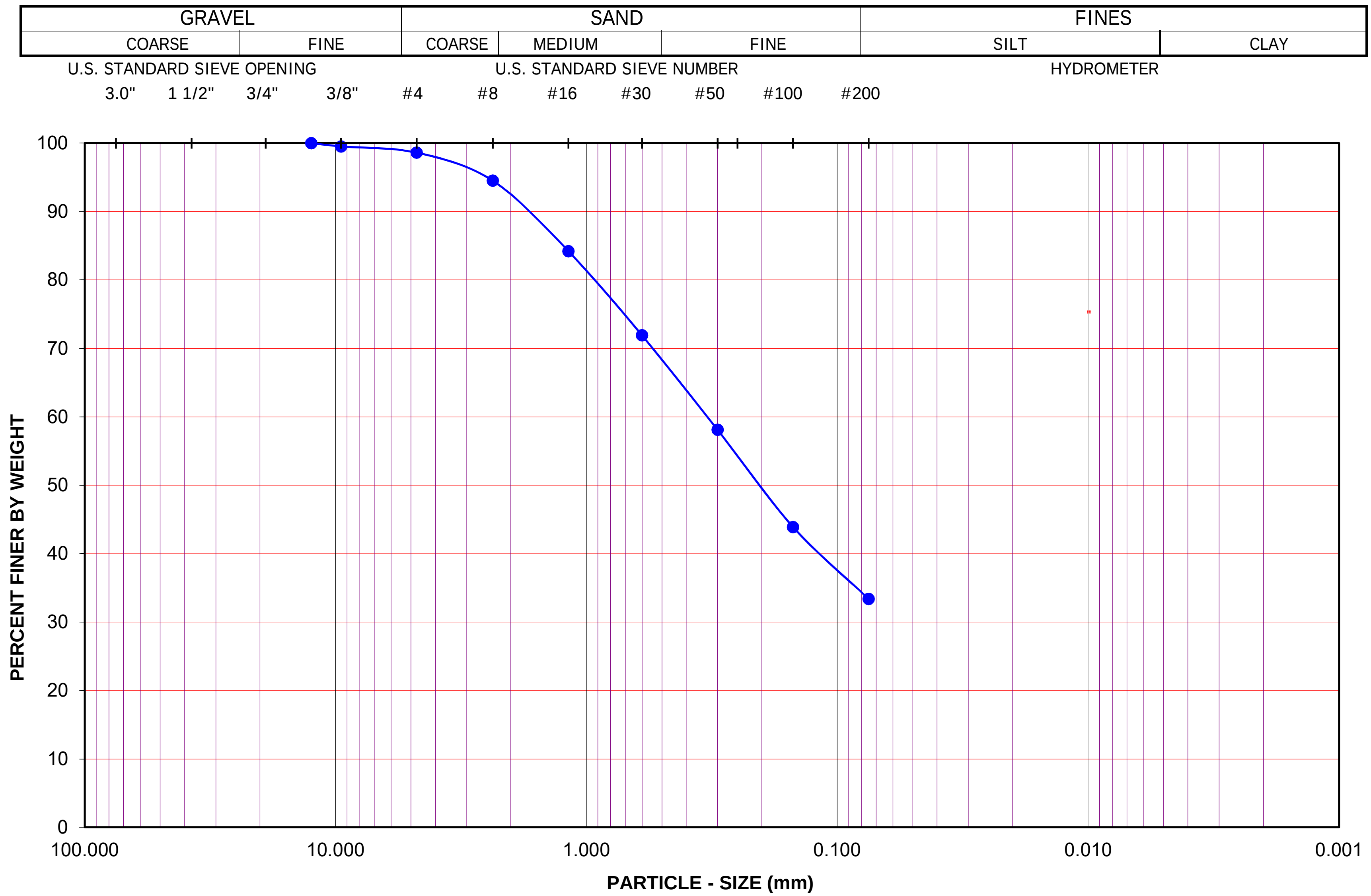
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 4 : 65 : 31

Sep-17

SA LB-3, BB1 @ 0-5



Project Name: Serrano

Project No.: 11737.001

Boring No.: LB-4 Sample No.: R1

Depth (feet): 5.0 Soil Type : SM

Soil Identification: Yellowish brown silty sand (SM)

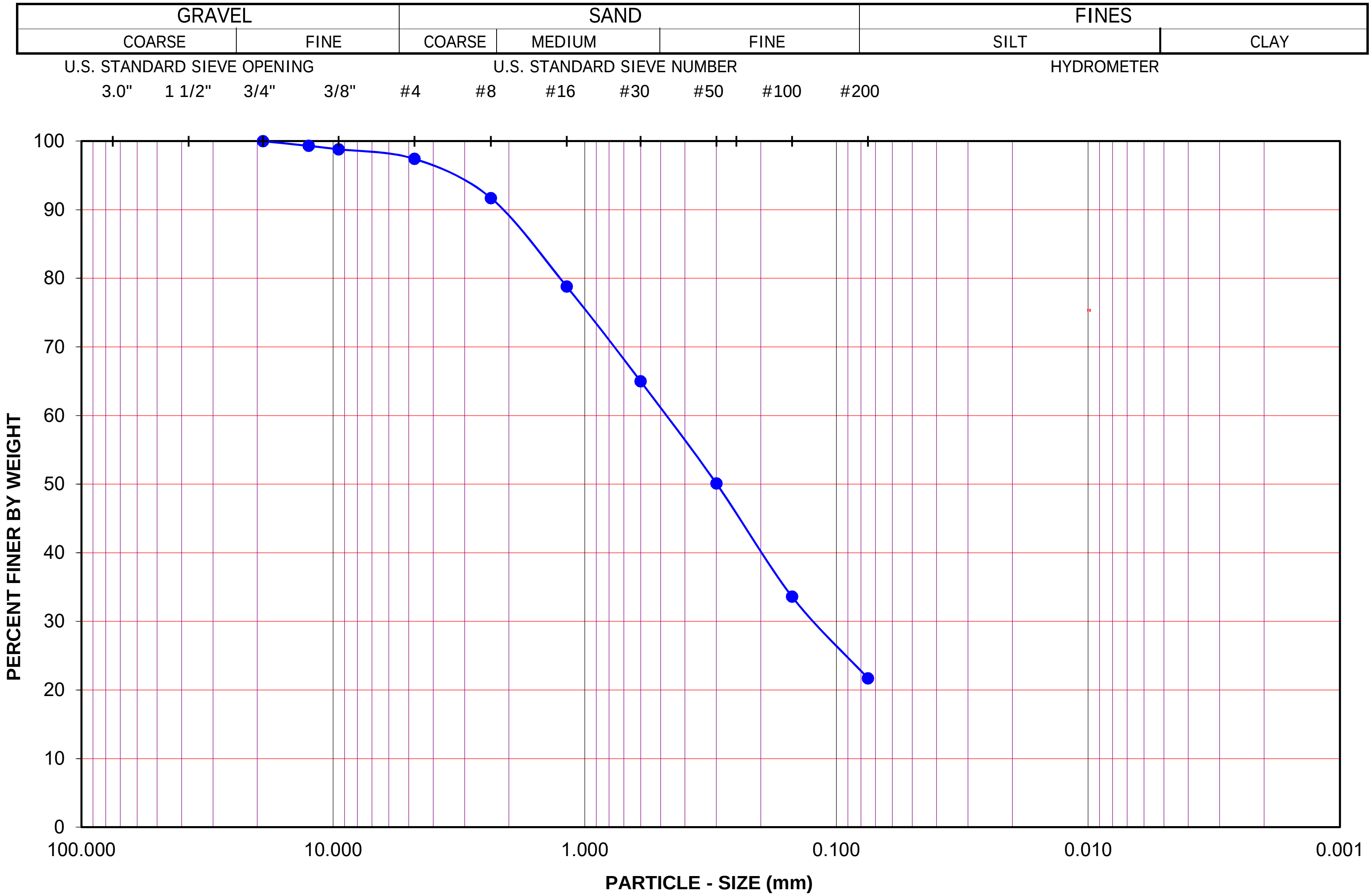
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 1 : 66 : 33

Sep-17

SA LB-4, R1 @ 5



Project Name: Serrano

Project No.: 11737.001

Boring No.: LB-5 Sample No.: BB1

Depth (feet): 0-5 Soil Type : SM

Soil Identification: Yellowish brown silty sand (SM)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 3 : 75 : 22

Sep-17

SA LB-5, BB1 @ 0-5

Boring No.	LB-2	LB-2	LB-2	LB-2	LB-2	LB-2		
Sample No.	R2	R4	S2	R7	S5	S6		
Depth (ft.)	10.0	20.0	30.0	45.0	60.0	70.0		
Sample Type	Ring	Ring	SPT	Ring	SPT	SPT		
Soil Identification	Brown silty sand (SM)	Brown silty sand (SM)	Brown silty sand (SM)	Olive yellow clayey sand with gravel (SC)g (one 2.5" gravel, 227.8 g)	Grayish brown clayey sand (SC)	Grayish brown sandy lean clay s(CL)		
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0		
Dry Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0		
Weight of Container (g)	1.0	1.0	1.0	1.0	1.0	1.0		
Moisture Content (%)	0.0	0.0	0.0	0.0	0.0	0.0		
Sample Dry Weight Determination								
Weight of Sample + Container (g)	863.4	811.6	1087.7	997.6	1014.2	955.6		
Weight of Container (g)	108.5	107.8	201.4	96.0	215.1	206.4		
Weight of Dry Sample (g)	754.9	703.8	886.3	901.6	799.1	749.2		
Container No.:	929	57	XP	IP-2	PHD	D-7		
After Wash								
Method (A or B)	A	A	A	A	A	A		
Dry Weight of Sample + Cont. (g)	713.6	652.2	741.2	835.4	747.1	566.0		
Weight of Container (g)	108.5	107.8	201.4	96.0	215.1	206.4		
Dry Weight of Sample (g)	605.1	544.4	539.8	739.4	532.0	359.6		
% Passing No. 200 Sieve	**19.8**	**22.6**	**39.1**	**18.0**	**33.4**	**52.0**		
% Retained No. 200 Sieve	80.2	77.4	60.9	82.0	66.6	48.0		

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name:	Serrano
Project No.:	11737.001
Client Name:	6509 Serrano LP
Tested By:	S. Felter
Date:	08/24/17

Boring No.	LB-6	LB-6	LB-6	LB-6	LB-6	LB-6		
Sample No.	R1	R3	R5	R7	R9	R12		
Depth (ft.)	5.0	15.0	25.0	35.0	45.0	70.0		
Sample Type	Ring	Ring	Ring	Ring	Ring	Ring		
Soil Identification	Grayish brown silty sand (SM)	Grayish brown silty sand (SM)	Grayish brown silty sand (SM)	Grayish brown silty sand (SM)	Grayish brown silty, clayey sand (SC-SM)	Grayish brown silty, clayey sand (SC-SM)		
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0		
Dry Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0	0.0		
Weight of Container (g)	1.0	1.0	1.0	1.0	1.0	1.0		
Moisture Content (%)	0.0	0.0	0.0	0.0	0.0	0.0		
Sample Dry Weight Determination								
Weight of Sample + Container (g)	850.5	625.8	824.3	838.0	1011.9	757.6		
Weight of Container (g)	106.7	108.4	107.5	109.0	300.3	108.0		
Weight of Dry Sample (g)	743.8	517.4	716.8	729.0	711.6	649.6		
Container No.:	912	934	A-15	927	IMC-1	R-2		
After Wash								
Method (A or B)	A	A	A	A	A	A		
Dry Weight of Sample + Cont. (g)	714.2	452.7	673.9	662.1	856.5	550.9		
Weight of Container (g)	106.7	108.4	107.5	109.0	300.3	108.0		
Dry Weight of Sample (g)	607.5	344.3	566.4	553.1	556.2	442.9		
% Passing No. 200 Sieve	**18.3**	**33.5**	**21.0**	**24.1**	**21.8**	**31.8**		
% Retained No. 200 Sieve	81.7	66.5	79.0	75.9	78.2	68.2		

Leighton

PERCENT PASSING No. 200 SIEVE ASTM D 1140

Project Name:	Serrano
Project No.:	11737.001
Client Name:	6509 Serrano LP
Tested By:	S. Felter
Date:	08/24/17

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	Serrano	Tested By:	R. Manning	Date:	09/01/17
Project No. :	11737.001	Input By:	G. Bathala	Date:	09/13/17
Boring No.:	LB-2	Checked By:	J. Ward		
Sample No.:	S4	Depth (ft.)	50.0		
Soil Identification:	Yellowish brown sandy lean clay s(CL)				

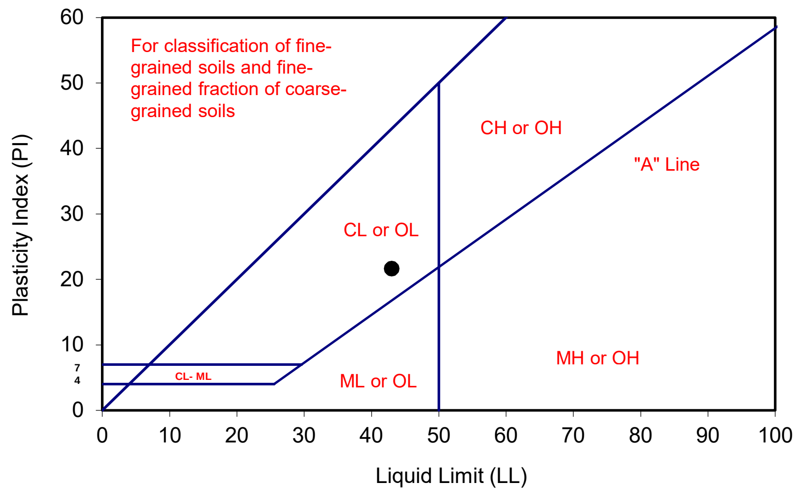
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			35	27	22	
Wet Wt. of Soil + Cont. (g)	20.33	20.49	24.85	23.17	22.42	
Dry Wt. of Soil + Cont. (g)	19.15	19.27	21.61	20.33	19.72	
Wt. of Container (g)	13.64	13.55	13.61	13.63	13.60	
Moisture Content (%) [Wn]	21.42	21.33	40.50	42.39	44.12	

Liquid Limit	**43**
Plastic Limit	**21**
Plasticity Index	**22**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 16.79

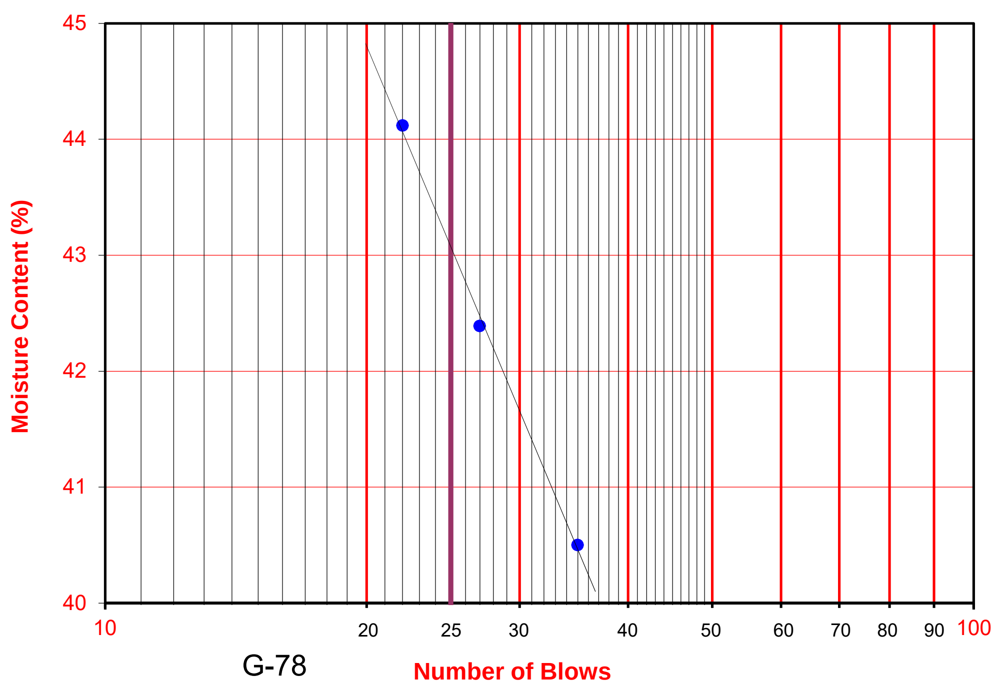
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	Serrano	Tested By:	R. Manning	Date:	08/30/17
Project No. :	11737.001	Input By:	G. Bathala	Date:	09/13/17
Boring No.:	LB-6	Checked By:	J. Ward		
Sample No.:	R13	Depth (ft.)	75.0		
Soil Identification:	Olive yellow silty sand (SM)				

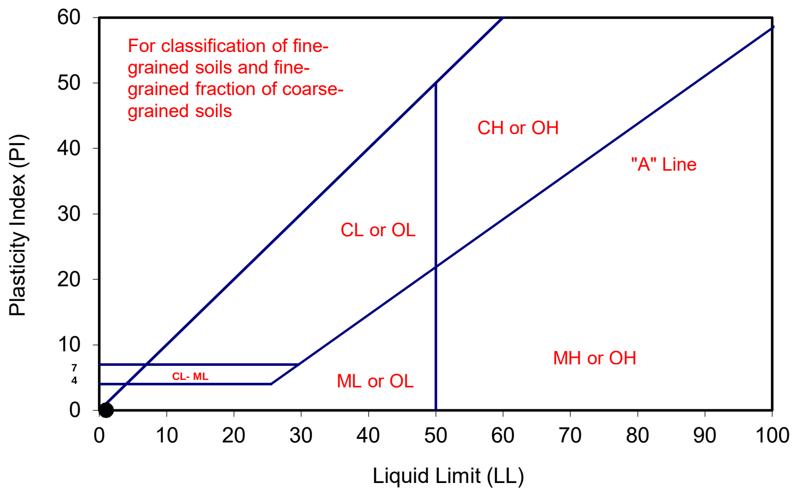
TEST NO.	PLASTIC LIMIT 1	PLASTIC LIMIT 2	LIQUID LIMIT 1	LIQUID LIMIT 2	LIQUID LIMIT 3	LIQUID LIMIT 4
Number of Blows [N]			13			
Wet Wt. of Soil + Cont. (g)	**Cannot be rolled:**		26.29	**Cannot get more than 13 blows:**		
Dry Wt. of Soil + Cont. (g)	**NonPlastic**		23.84	**NonPlastic**		
Wt. of Container (g)			13.67			
Moisture Content (%) [Wn]			24.09			

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**NP**

PI at "A" - Line = 0.73(LL-20) =

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



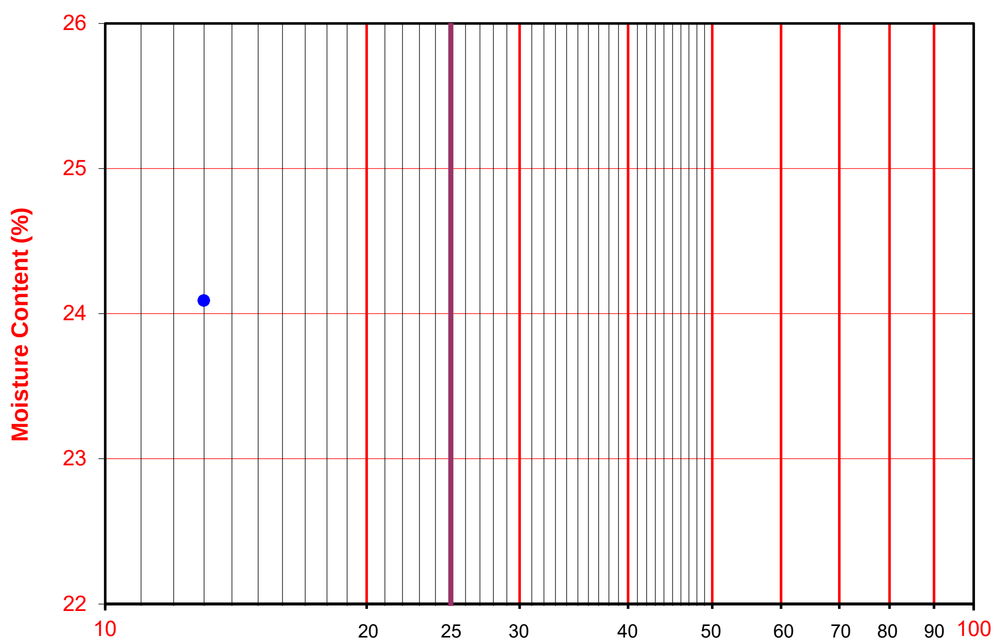
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

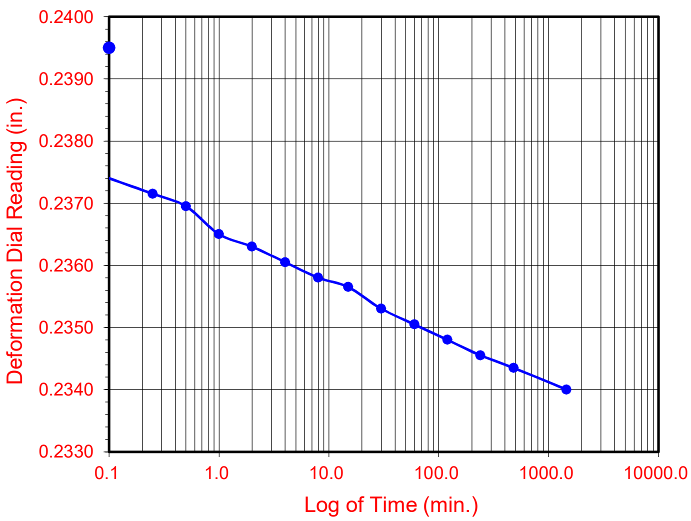
X Dry Preparation
Multipoint - Dry

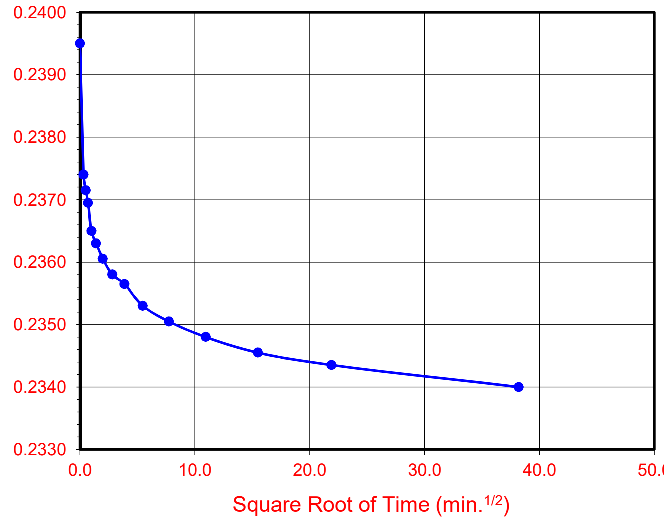
X Procedure A
Multipoint Test

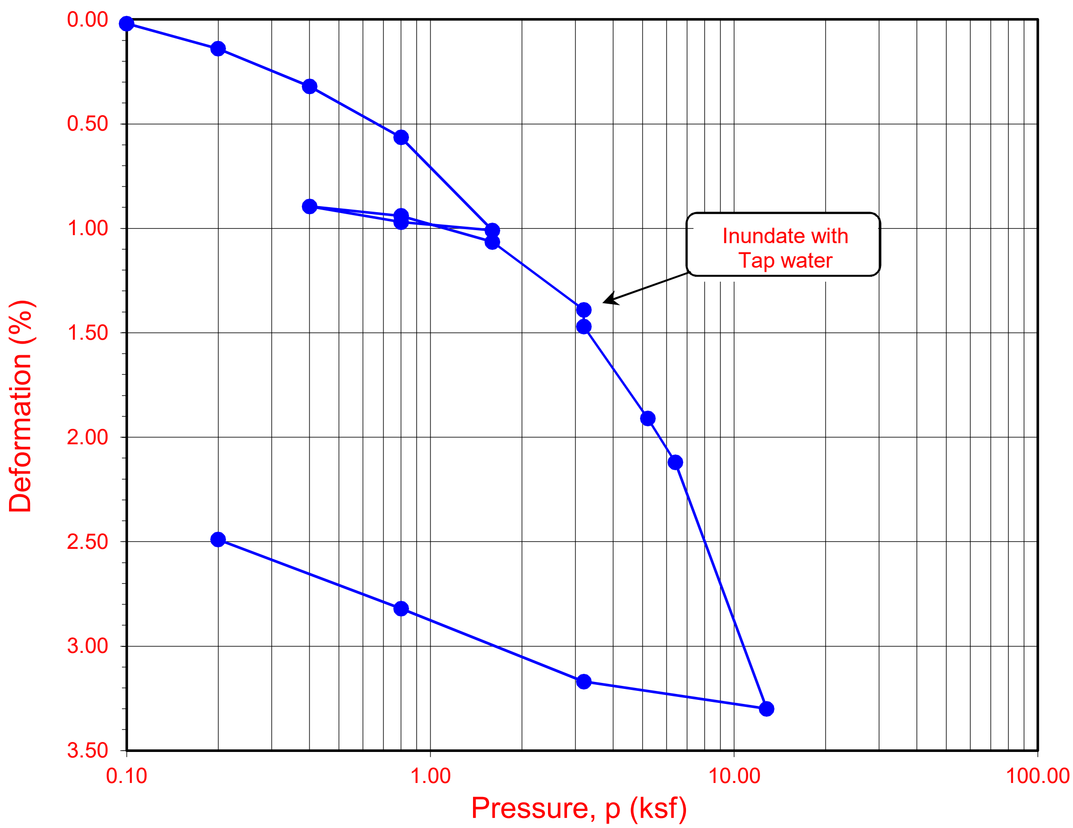
☐ Procedure B
One-point Test



Time Readings @ 5.2 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**R7**	**45.0**	**13.3**	**12.7**	**116.4**	**120.3**	**0.448**	**0.412**	**80**	**86**

Soil Identification: Olive yellow clayey sand with gravel (SC)g

Leighton

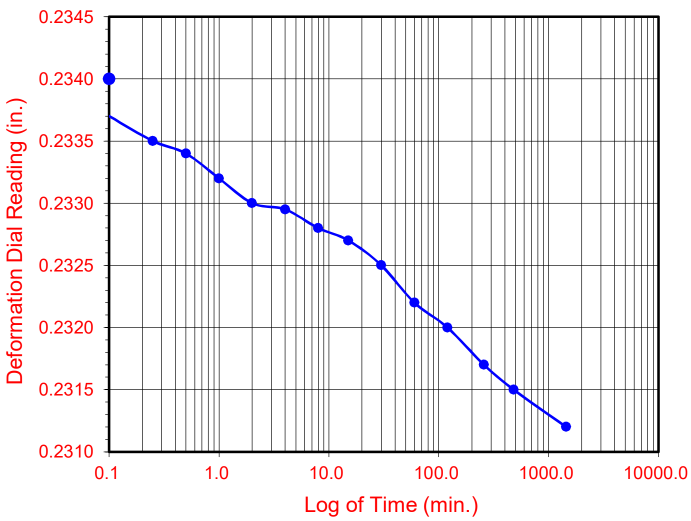
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

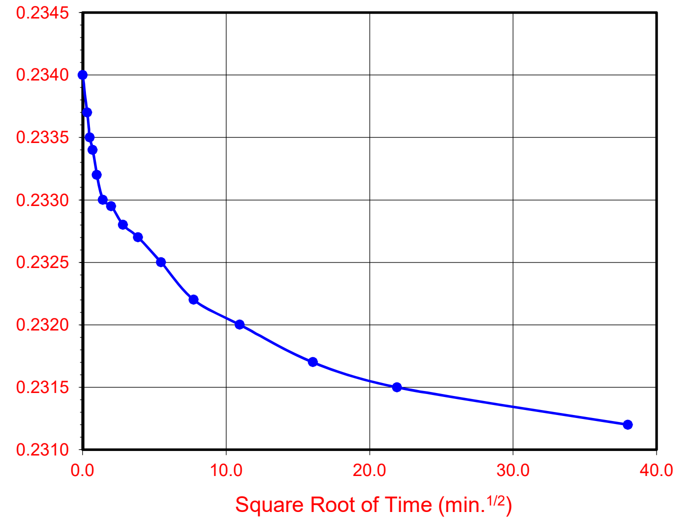
Project No.: 11737.001

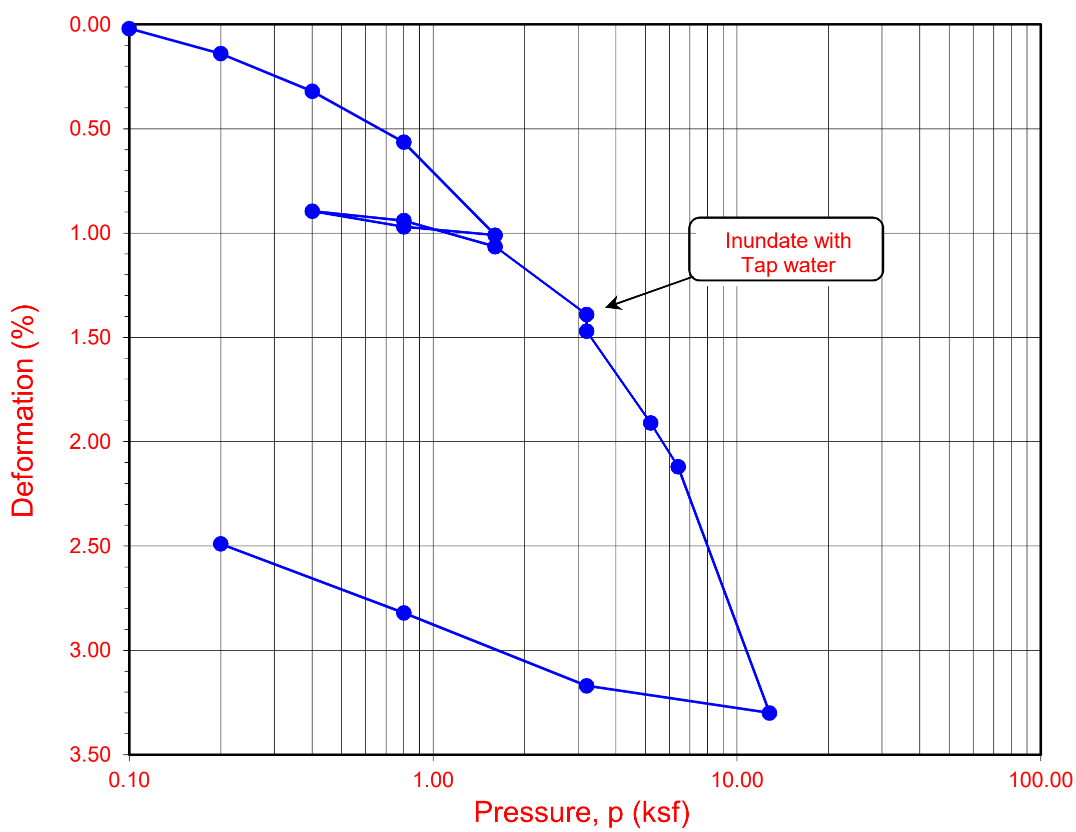
Serrano

09-17

Time Readings @ 6.4 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-2	**R7**	**45.0**	**13.3**	**12.7**	**116.4**	**120.3**	**0.448**	**0.412**	**80**	**86**

Soil Identification: Olive yellow clayey sand with gravel (SC)g

Leighton

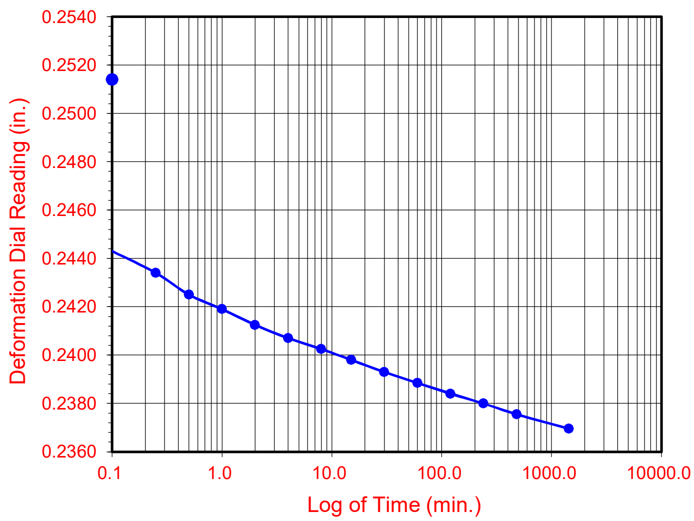
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

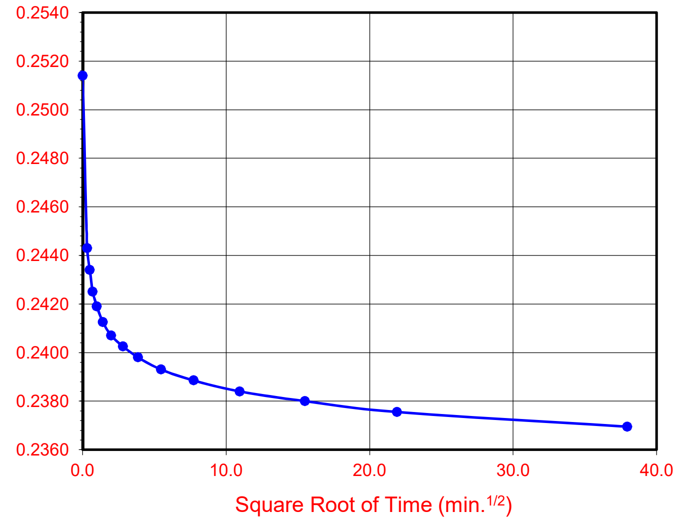
Project No.: 11737.001

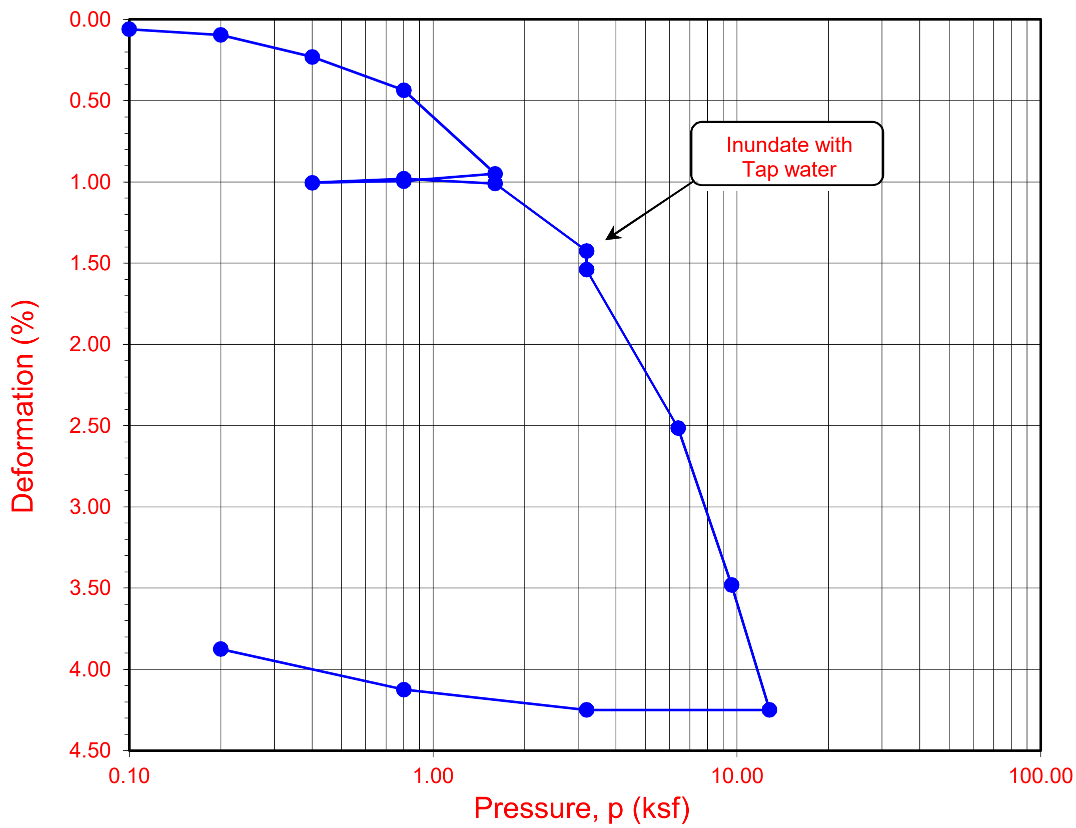
Serrano

09-17

Time Readings @ 6.4 ksf





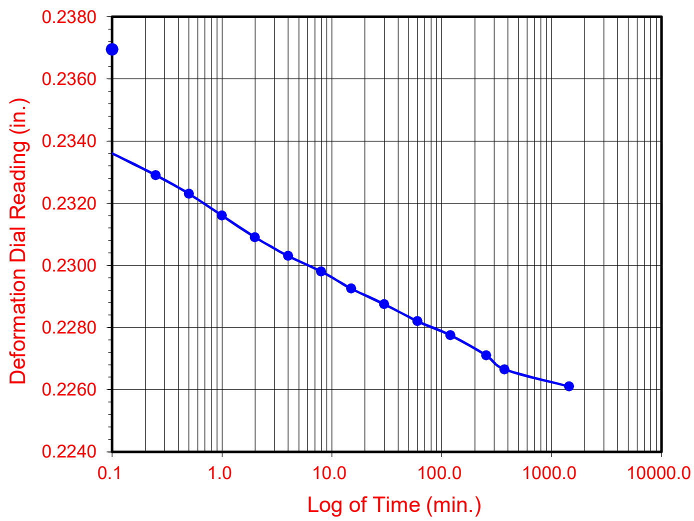


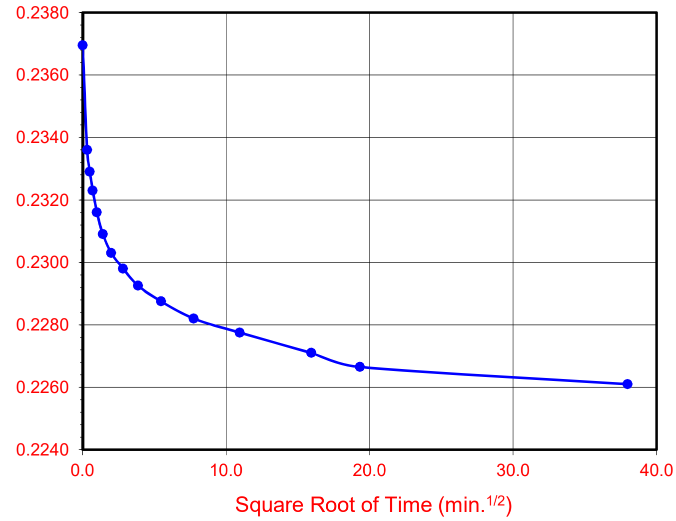
Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-6	**R13**	**75.0**	**8.8**	**18.2**	**114.9**	**109.5**	**0.468**	**0.411**	**51**	**91**

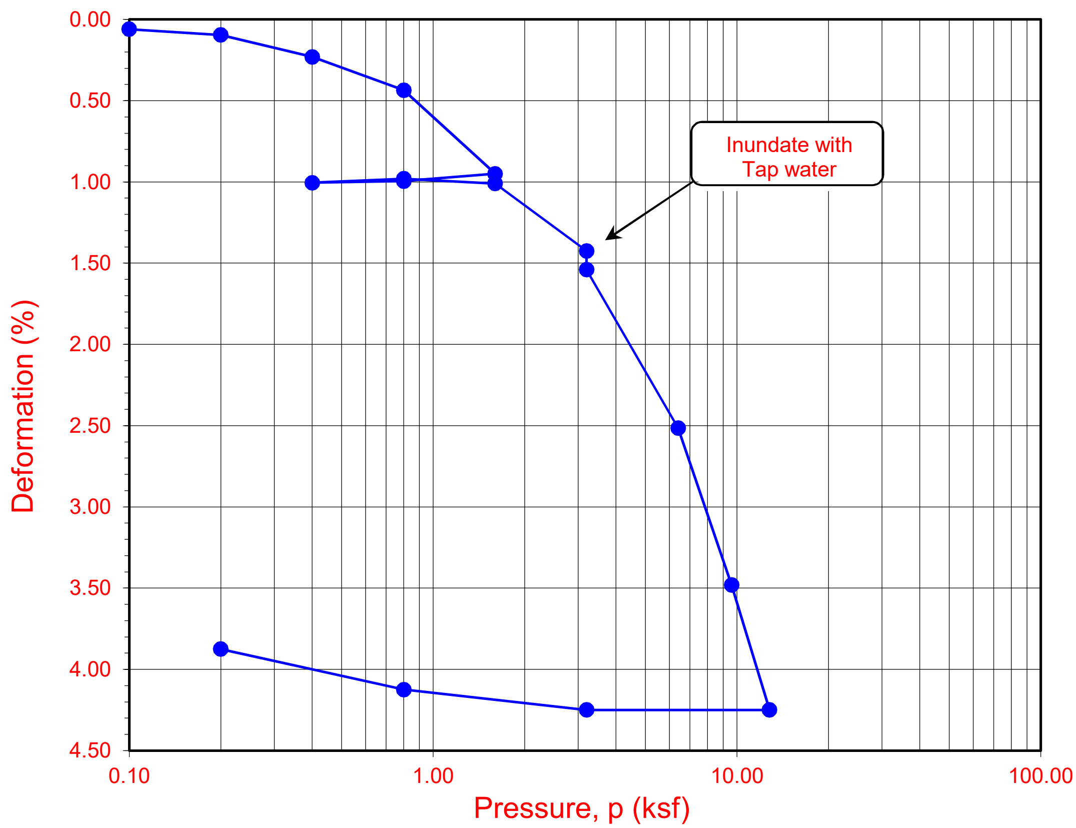
Soil Identification: Olive yellow silty sand (SM)

Leighton	**ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS** **ASTM D 2435**	Project No.: 11737.001 Serrano 09-17

Time Readings @ 9.6 ksf







Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-6	**R13**	**75.0**	**8.8**	**18.2**	**114.9**	**109.5**	**0.468**	**0.411**	**51**	**91**

Soil Identification: Olive yellow silty sand (SM)

Leighton	**ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS** **ASTM D 2435**	Project No.: 11737.001 Serrano 09-17

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Leighton

Project Name:	Serrano	Tested By:	G. Bathala
Date:	08/28/17		
Project No.:	11737.001	Checked By:	J. Ward
Date:	09/21/17		
Boring No.:	LB-1	Sample Type:	Ring
Sample No.:	R1	Depth (ft.)	5.0
Sample Description:	Olive gray silty, clayey sand (SC-SM)		

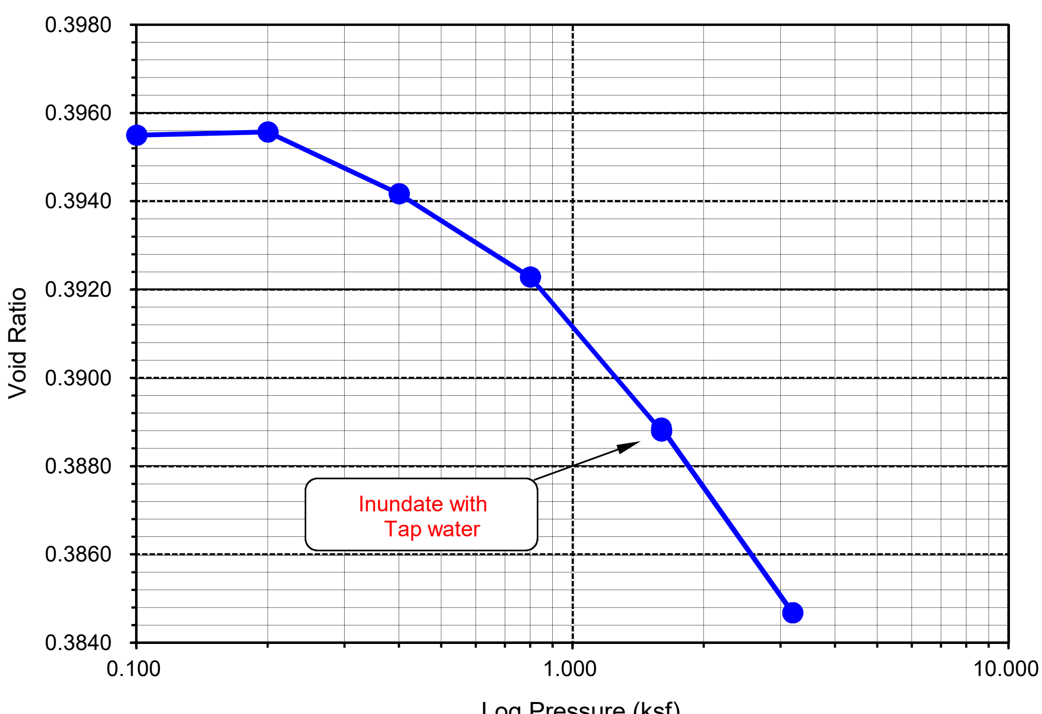
Initial Dry Density (pcf):	120.8
Initial Moisture (%):	11.10
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2950
Diameter(in):	2.370

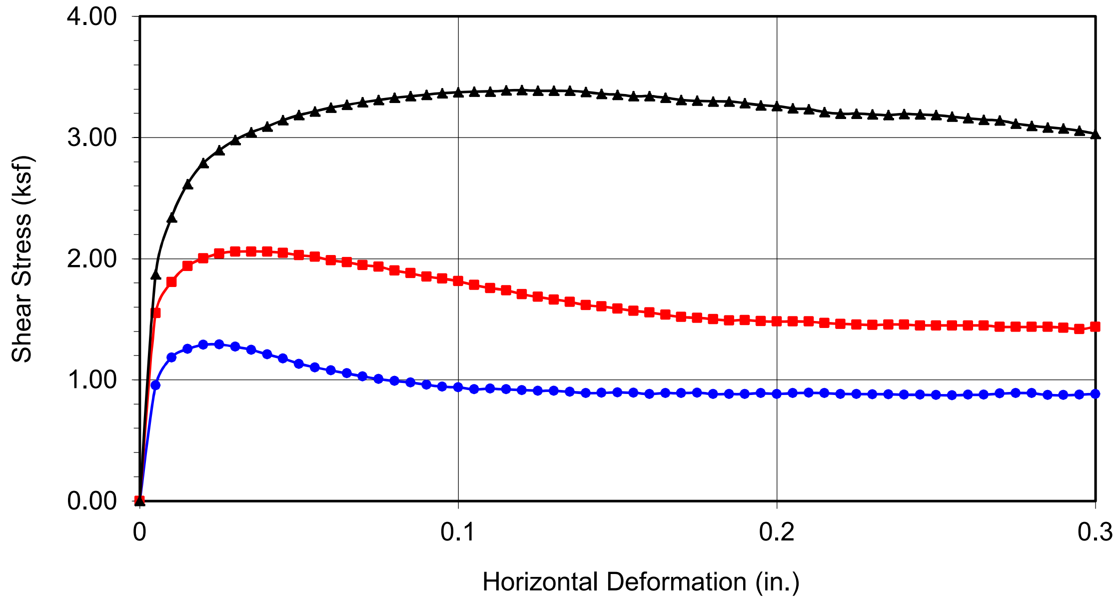
Final Dry Density (pcf):	121.7
Final Moisture (%) :	12.3
Initial Void ratio:	0.3958
Specific Gravity(assumed):	2.70
Initial Saturation (%)	75.7

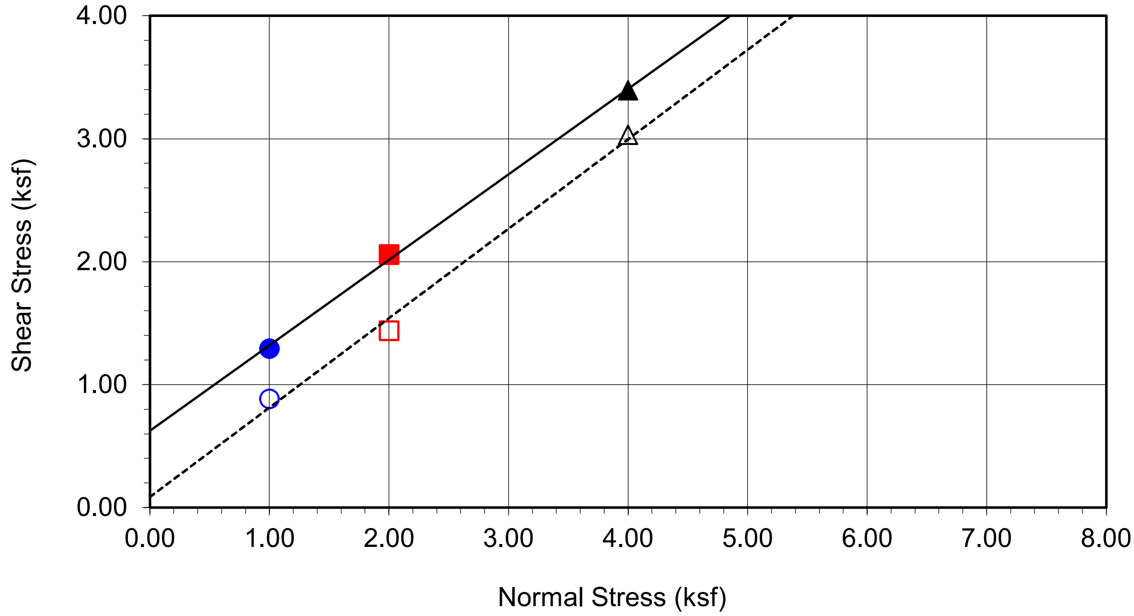
Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.2947	0.9998	0.00	-0.02	0.3955	-0.02
0.20	0.2941	0.9991	0.07	-0.09	0.3956	-0.02
0.40	0.2917	0.9967	0.21	-0.33	0.3942	-0.12
0.80	0.2896	0.9947	0.28	-0.53	0.3923	-0.25
1.60	0.2858	0.9909	0.41	-0.91	0.3888	-0.50
H2O	0.2859	0.9909	0.41	-0.91	0.3889	-0.50
3.20	0.2817	0.9867	0.53	-1.33	0.3847	-0.80

Percent Swell (+) / Settlement (-) After Inundation = 0.01

Void Ratio - Log Pressure Curve







Boring No.	**LB-1**
Sample No.	**BB1**
Depth (ft)	**0-5**
Sample Type: 90% Remold	
Soil Identification: Olive brown silty, clayey sand (SC-SM)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	626	35
Ultimate	86	36

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.292	■ 2.059	▲ 3.392
Shear Stress @ End of Test (ksf)	○ 0.883	□ 1.437	△ 3.031
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.70	8.70	8.70
Dry Density (pcf)	117.9	117.9	117.8
Saturation (%)	54.6	54.6	54.6
Soil Height Before Shearing (in.)	0.9935	0.9916	0.9820
Final Moisture Content (%)	14.1	13.7	13.4

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 11737.001

Serrano

08-17

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Serrano	Tested By:	O. Figueroa	Date:	08/25/17
Project No.:	11737.001	Input By:	J. Ward	Date:	08/28/17
Boring No.:	LB-1	Depth (ft.):	0-5		
Sample No.:	BB1				
Soil Identification:	Olive brown silty, clayey sand (SC-SM)				

Preparation Method: [X] Moist [ ] Dry [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3901	4002	3958			
Weight of Mold (g)	1857	1857	1857			
Net Weight of Soil (g)	2044	2145	2101			
Wet Weight of Soil + Cont. (g)	340.2	416.4	445.1			
Dry Weight of Soil + Cont. (g)	323.0	386.8	404.6			
Weight of Container (g)	39.2	39.6	39.5			
Moisture Content (%)	6.06	8.53	11.09			
Wet Density (pcf)	135.3	142.0	139.1			
Dry Density (pcf)	127.6	130.9	125.2			

Maximum Dry Density (pcf) **131.0** **Optimum Moisture Content (%)** **8.5**

PROCEDURE USED

[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

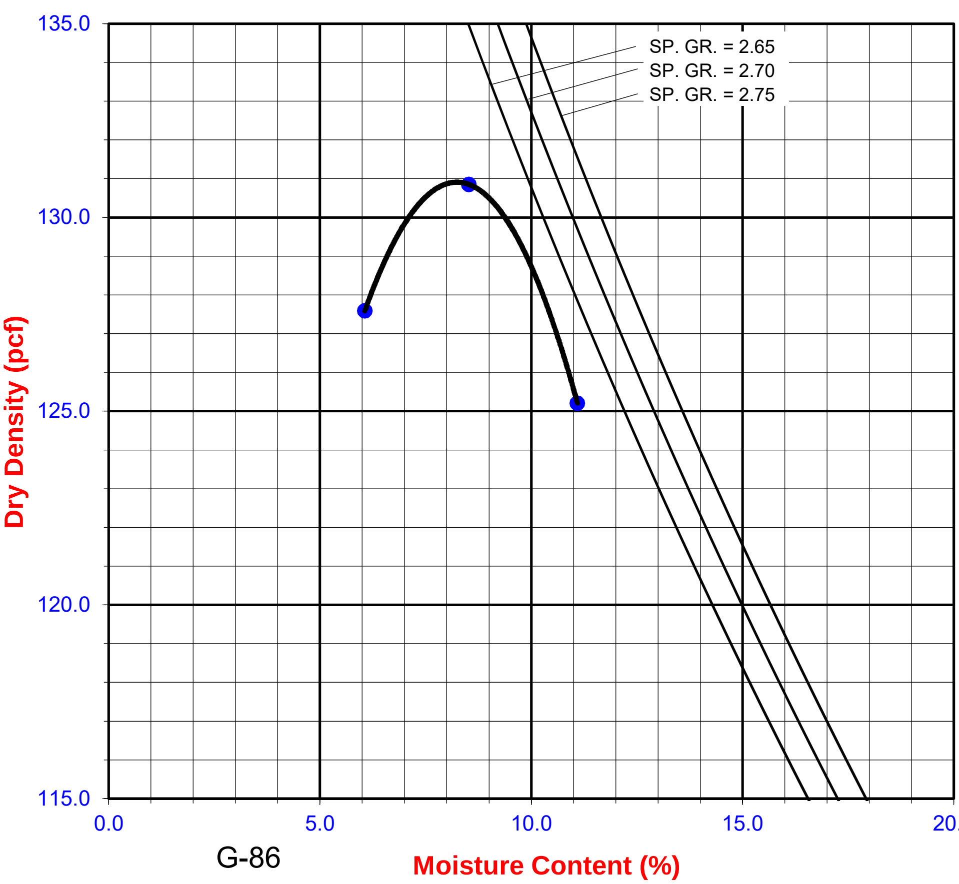
[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
1:74:25
GR:SA:FI

Atterberg Limits:

LL,PL,PI



R-VALUE TEST RESULTS

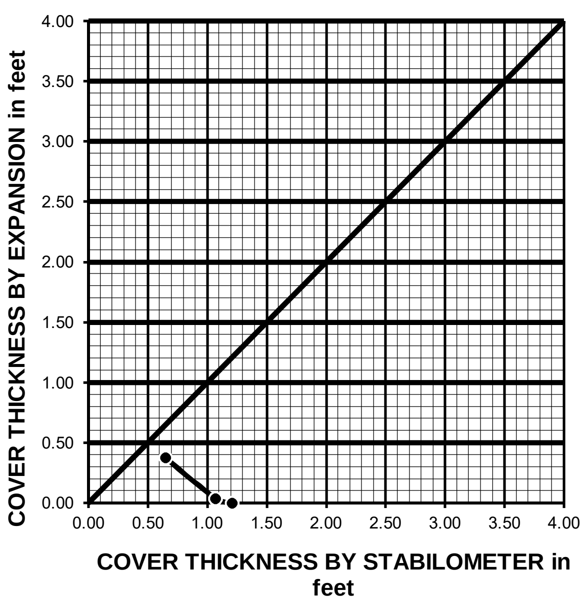
ASTM D 2844

Project Name:	Serrano	Date:	09/07/17
Project Number:	11737.001	Technician:	F. Mina
Boring Number:	LB-1	Depth (ft.):	0-5
Sample Number:	B-1	Sample Location:	N/A
Sample Description:	Olive brown silty, clayey sand (SC-SM)		

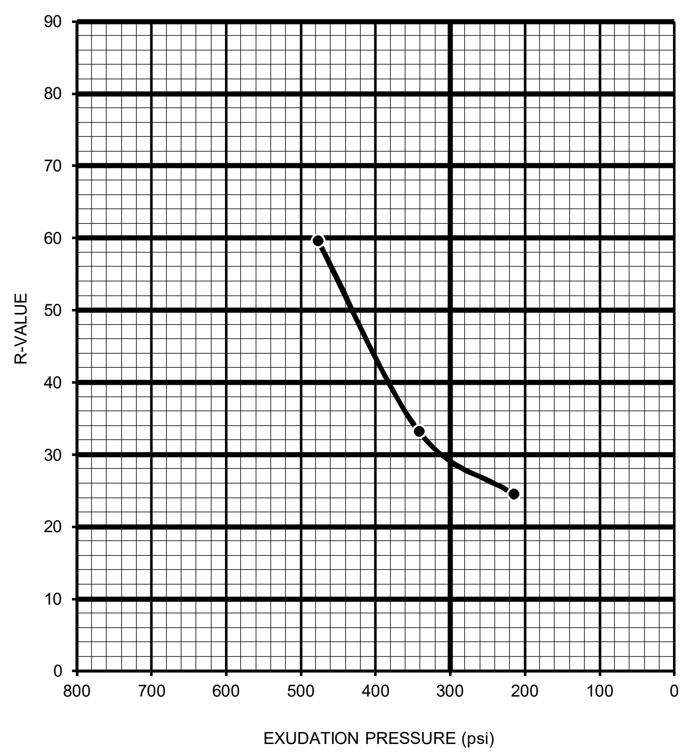
TEST SPECIMEN	A	B	C
MOISTURE AT COMPACTION %	10.4	11.5	12.6
HEIGHT OF SAMPLE, Inches	2.46	2.48	2.53
DRY DENSITY, pcf	124.0	124.3	120.8
COMPACTOR AIR PRESSURE, psi	200	150	125
EXUDATION PRESSURE, psi	477	342	215
EXPANSION, Inches x 10exp-4	10	1	0
STABILITY Ph 2,000 lbs (160 psi)	45	85	100
TURNS DISPLACEMENT	4.33	4.44	4.62
R-VALUE UNCORRECTED	60	33	25
R-VALUE CORRECTED	60	33	25

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.65	1.07	1.21
EXPANSION PRESSURE THICKNESS, ft.	0.38	0.04	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	69
R-VALUE BY EXUDATION:	29
EQUILIBRIUM R-VALUE:	29

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Serrano | Tested By : G. Berdy | Date: 08/24/17

Project No. : 11737.001 | Data Input By: G. Bathala | Date: 09/15/17

Boring No.	LB-1			
Sample No.	BB1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SC-SM)			
Wet Weight of Soil + Container (g)	208.31			
Dry Weight of Soil + Container (g)	195.72			
Weight of Container (g)	58.70			
Moisture Content (%)	9.19			
Weight of Soaked Soil (g)	100.54			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	92			
Crucible No.	26			
Furnace Temperature (°C)	860			
Time In / Time Out	9:00/9:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	20.9369			
Wt. of Crucible (g)	20.9349			
Wt. of Residue (g) (A)	0.0020			
PPM of Sulfate (A) x 41150	82.30			
PPM of Sulfate, Dry Weight Basis	**91**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.3			
PPM of Chloride (C -0.2) * 100 * 30 / B	10			
PPM of Chloride, Dry Wt. Basis	**11**			

pH TEST, DOT California Test 643

pH Value	**7.74**			
Temperature °C	20.5			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	Serrano	Tested By :	G. Berdy	Date: 08/28/17
Project No. :	11737.001	Data Input By:	G. Bathala	Date: 09/15/17
Boring No.:	LB-1	Depth (ft.) :	0-5	
Sample No. :	BB1			

Soil Identification:* Olive brown (SC-SM)

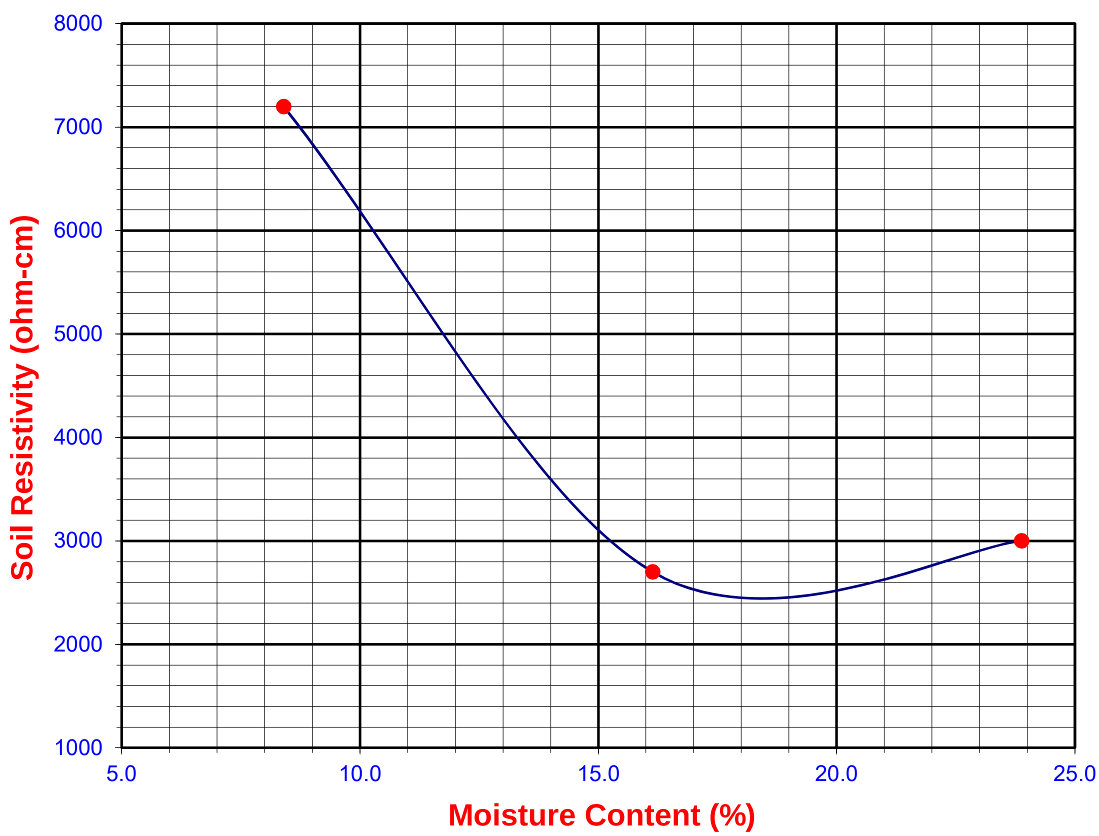
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	8.40	7200	7200
2	20	16.14	2700	2700
3	30	23.88	3000	3000
4				
5				

Moisture Content (%) (MCi)	0.66
Wet Wt. of Soil + Cont. (g)	90.64
Dry Wt. of Soil + Cont. (g)	90.47
Wt. of Container (g)	64.68
Container No.	
Initial Soil Wt. (g) (Wt)	130.03
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
2400	**18.4**	**91**	**11**	**7.74**	**20.5**



APPENDIX C

PERCOLATION TEST RESULTS

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Boring Percolation Test Data Sheet

Project Number:	11737.001	**Test Hole Number:**	LP-1
Project Name:	Serrano	**Date Excavated:**	8/16/2017
Earth Description:	Artificial Fill	**Date Tested:**	8/17/2017
Liquid Description:	Tap water	**Depth of boring (ft):**	9
Tested By:	JMP	**Diameter of boring (in):**	8
Time Interval Standard		**Diameter of casing (in):**	2
Start Time for Pre-Soak:	8/16/2017 9:00AM	**Length of slotted of casing (ft):**	5
Start Time for Standard:	8/17/2017 7:47AM	**Depth to Initial Water Depth (ft):**	4
Standard Time Interval		**Porosity of Annulus Material, *n* :**	0.35
Between Readings, mins:	30	**Bentonite Plug at Bottom:**	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
1	7:47	30	4.40	55.2	3.4	8.93	0.09
	8:17		4.68	51.8			
2	8:17	30	4.68	51.8	2.4	12.50	0.07
	8:47		4.88	49.4			
3	8:47	30	4.88	49.4	2.0	14.71	0.06
	9:17		5.05	47.4			
4	9:17	30	4.95	48.6	1.9	15.63	0.06
	9:47		5.11	46.7			
5	9:47	30	4.95	48.6	2.5	11.90	0.08
	10:17		5.16	46.1			
6	10:17	30	4.96	48.5	2.4	12.50	0.08
	10:47		5.16	46.1			
7	10:47	30	4.97	48.4	2.2	13.89	0.07
	11:17		5.15	46.2			
8	11:17	30	4.95	48.6	2.3	13.16	0.07
	11:47		5.14	46.3			
9	11:47	30	4.97	48.4	2.0	14.71	0.06
	12:17		5.14	46.3			
10	12:17	30	4.96	48.5	2.0	14.71	0.06
	12:47		5.13	46.4			
11	12:47	30	4.97	48.4	1.9	15.63	0.06
	13:17		5.13	46.4			
12	13:17	30	4.96	48.5	1.9	15.63	0.06
	13:47		5.12	46.6			

Infiltration Rate (I) = Flow Volume/Flow Area/Δt

Infiltration Rate, I (Last Reading) = $\boxed{\mathbf{0.06}}$ **in./hr.**

Boring Percolation Test Data Sheet

Project Number:	11737.001	**Test Hole Number:**	LP-2
Project Name:	Serrano	**Date Excavated:**	8/16/2017
Earth Description:	Artificial Fill	**Date Tested:**	8/17/2017
Liquid Description:	Tap water	**Depth of boring (ft):**	9
Tested By:	JMP	**Diameter of boring (in):**	8
Time Interval Standard		**Diameter of casing (in):**	2
Start Time for Pre-Soak:	8/16/2017 9:00AM	**Length of slotted of casing (ft):**	5
Start Time for Standard:	8/17/2017 8:01AM	**Depth to Initial Water Depth (ft):**	4
Standard Time Interval		**Porosity of Annulus Material, *n* :**	0.35
Between Readings, mins:	30	**Bentonite Plug at Bottom:**	No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Percolation Rate (min./in.)	Infiltration Rate (in./hr.)
1	8:01	30	4.25	57.0	1.4	20.83	0.04
	8:31		4.37	55.6			
2	8:31	30	4.37	55.6	1.3	22.73	0.04
	9:01		4.48	54.2			
3	9:01	30	4.48	54.2	1.2	25.00	0.03
	9:31		4.58	53.0			
4	9:31	30	4.58	53.0	1.2	25.00	0.03
	10:01		4.68	51.8			
5	10:01	30	4.68	51.8	1.1	27.78	0.03
	10:31		4.77	50.8			
6	10:31	30	4.77	50.8	1.2	25.00	0.04
	11:01		4.87	49.6			
7	11:01	30	4.87	49.6	1.1	27.78	0.03
	11:31		4.96	48.5			
8	11:31	30	4.96	48.5	1.2	25.00	0.04
	12:01		5.06	47.3			
9	12:01	30	4.99	48.1	-10.6	-2.84	-0.30
	12:31		4.11	58.7			
10	12:31	30	4.97	48.4	1.6	19.23	0.05
	13:01		5.10	46.8			
11	13:01	30	4.96	48.5	1.6	19.23	0.05
	13:31		5.09	46.9			
12	13:31	30	4.98	48.2	1.4	20.83	0.05
	14:01		5.10	46.8			

Infiltration Rate (I) = Flow Volume/Flow Area/Δt

Infiltration Rate, I (Last Reading) = **0.05** **in./hr.**

APPENDIX D

SEISMICITY DATA

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USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.83172°N, 117.76003°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.569 g
From Figure 22-2 [2]	S_1 = 0.604 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_S$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content w ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.569 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.604 g, F_v = 1.500

Equation (11.4–1):	$S_{MS} = F_aS_S = 1.000 \times 1.569 = 1.569$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 1.500 \times 0.604 = 0.906$ g

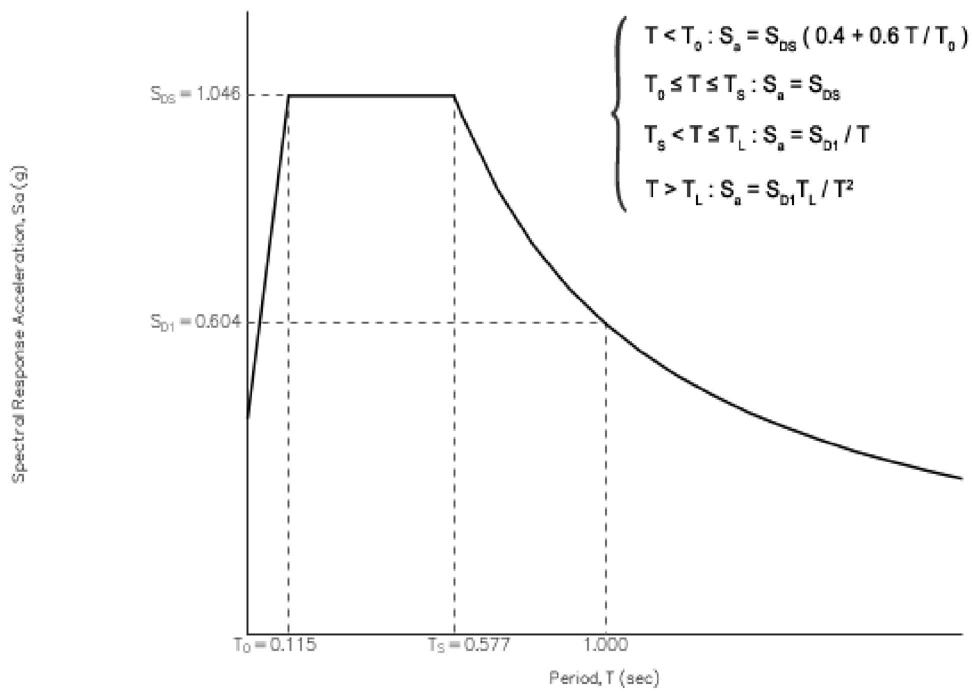
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.569 = 1.046$ g
Equation (11.4–4):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.906 = 0.604$ g

Section 11.4.5 — Design Response Spectrum

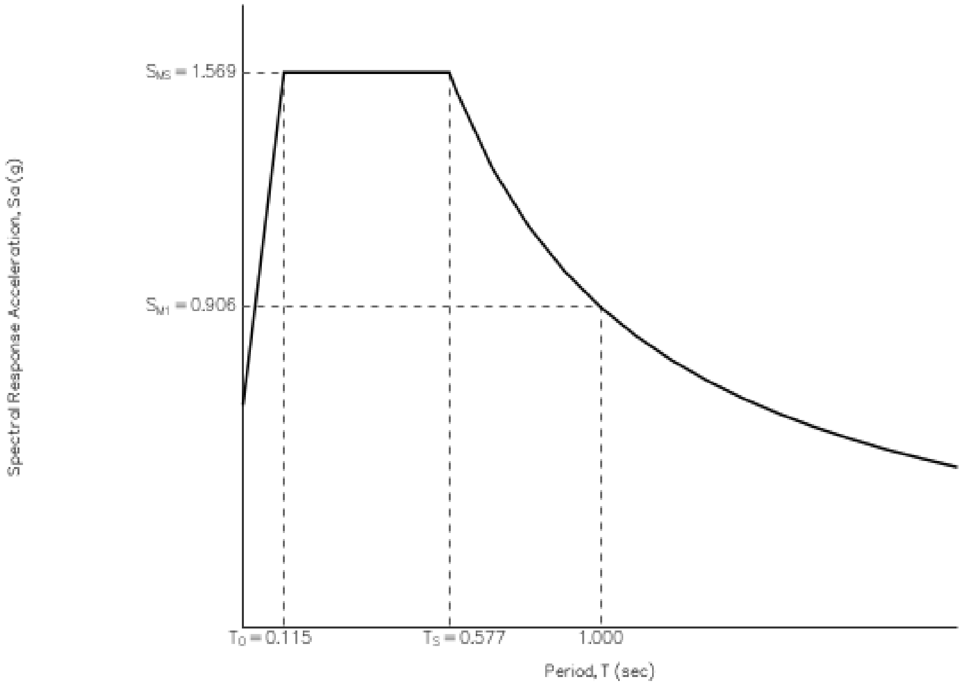
From Figure 22-12 [3]	T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.599

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.599 = 0.599$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.599 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.003

From Figure 22-18 [6] C_{R1} = 1.020

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.046 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.604 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is E for buildings in Risk Categories I, II, and III, and F for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. Figure 22-1: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. Figure 22-17: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. Figure 22-18: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX E

GENERAL EARTHWORK AND GRADING RECOMMENDATIONS

APPENDIX E
LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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Standard Details

1.0 GENERAL

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

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The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 PREPARATION OF AREAS TO BE FILLED

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 FILL MATERIAL

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

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4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 SUBDRAIN INSTALLATION

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 TRENCH BACKFILLS

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

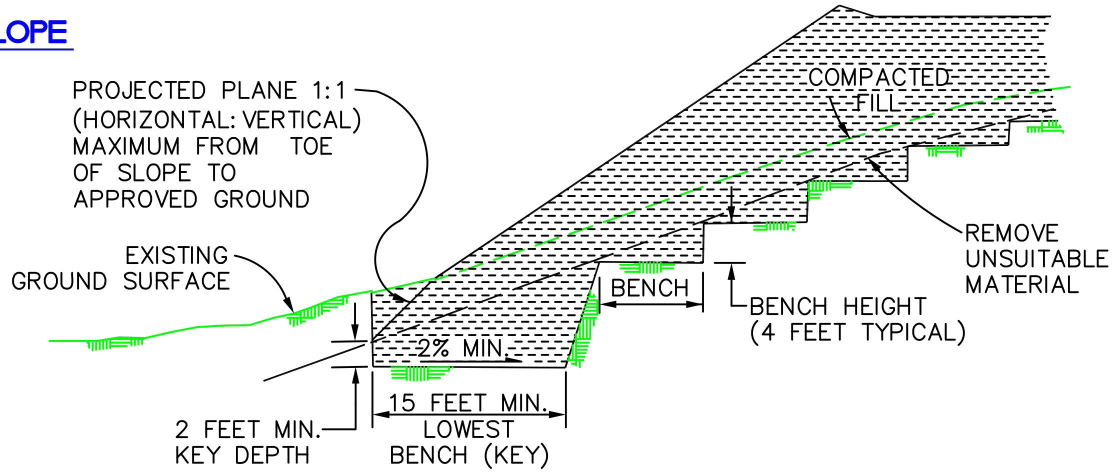
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

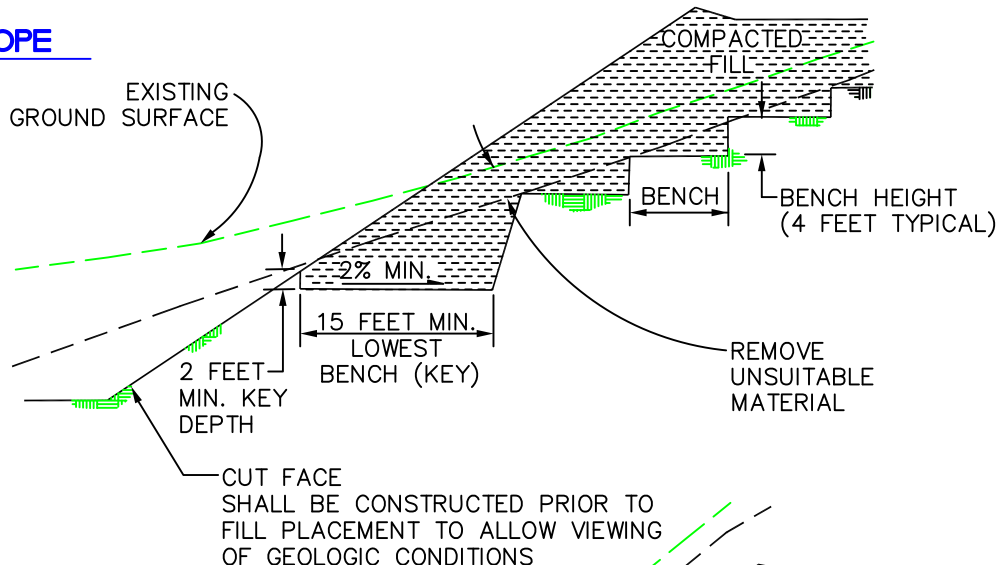
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

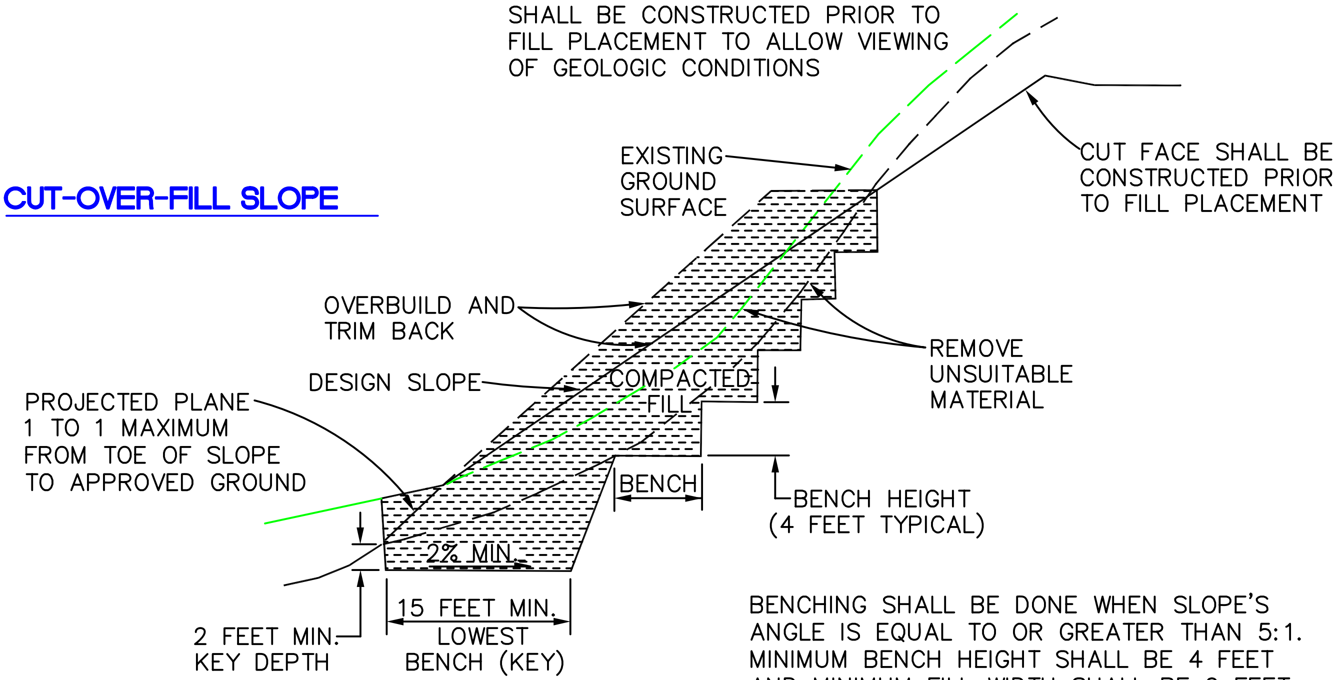
FILL SLOPE



FILL-OVER-CUT SLOPE

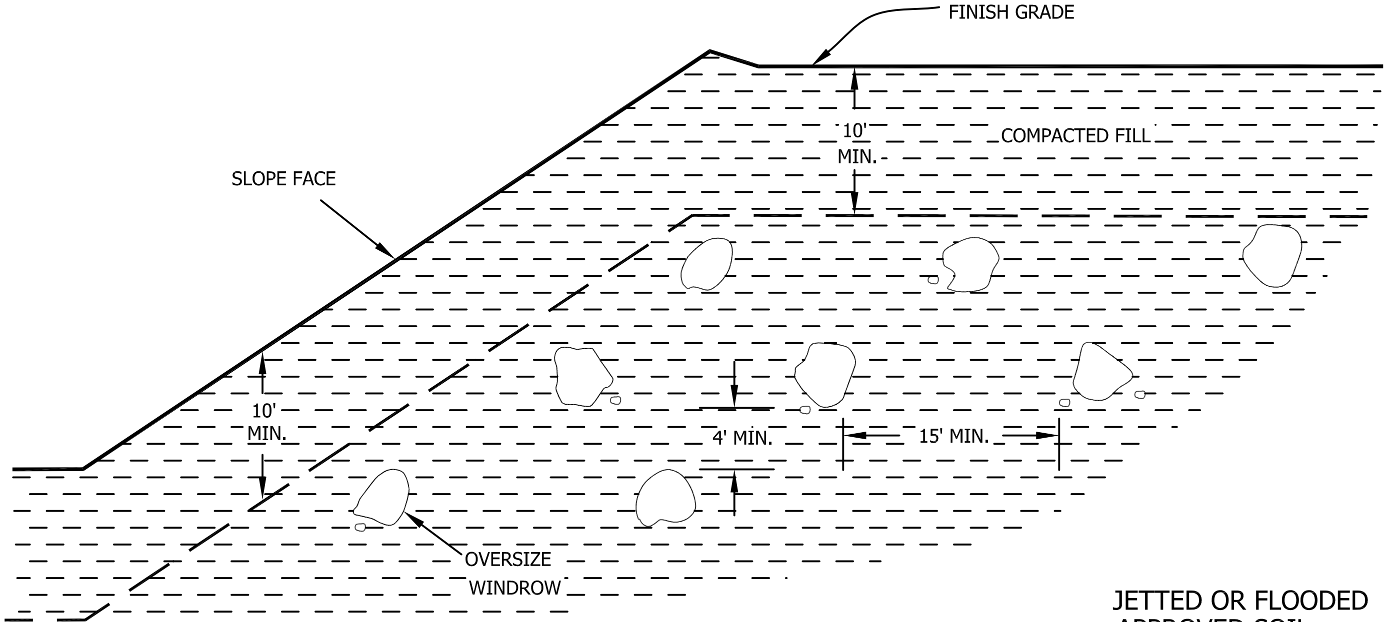


CUT-OVER-FILL SLOPE

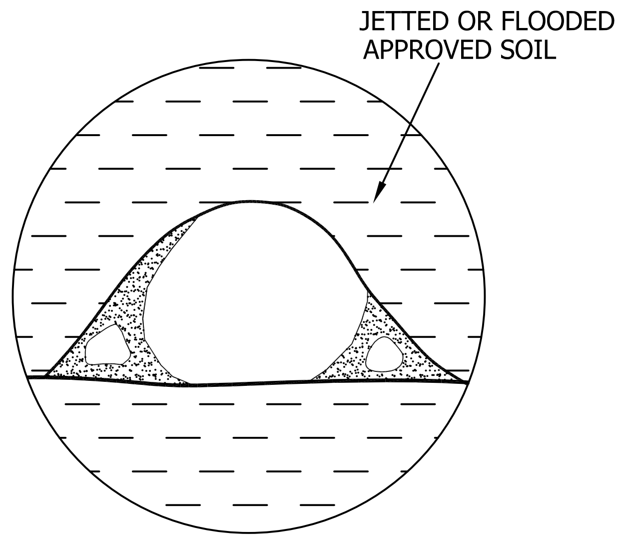


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A G-110	Leighton

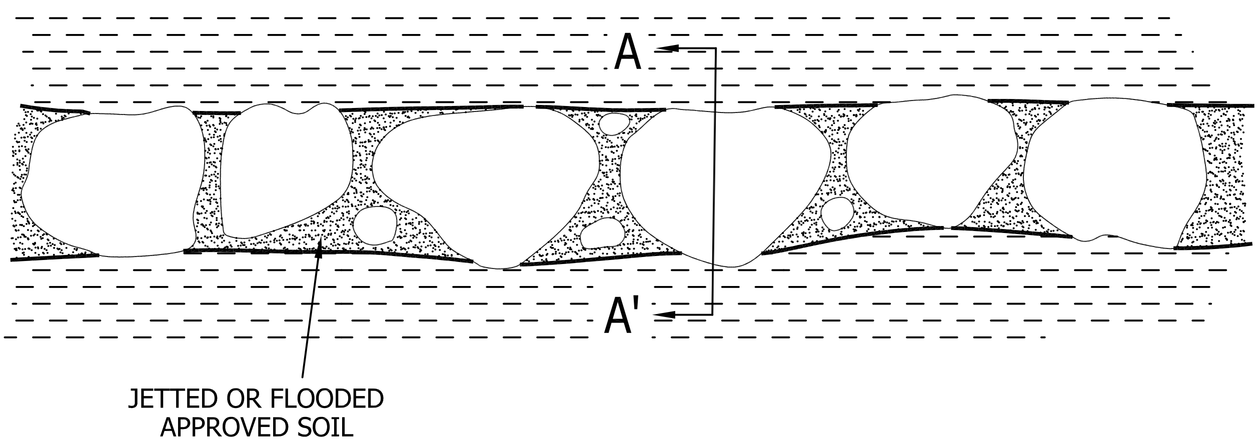


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



SECTION A-A'

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B G-111	Leighton

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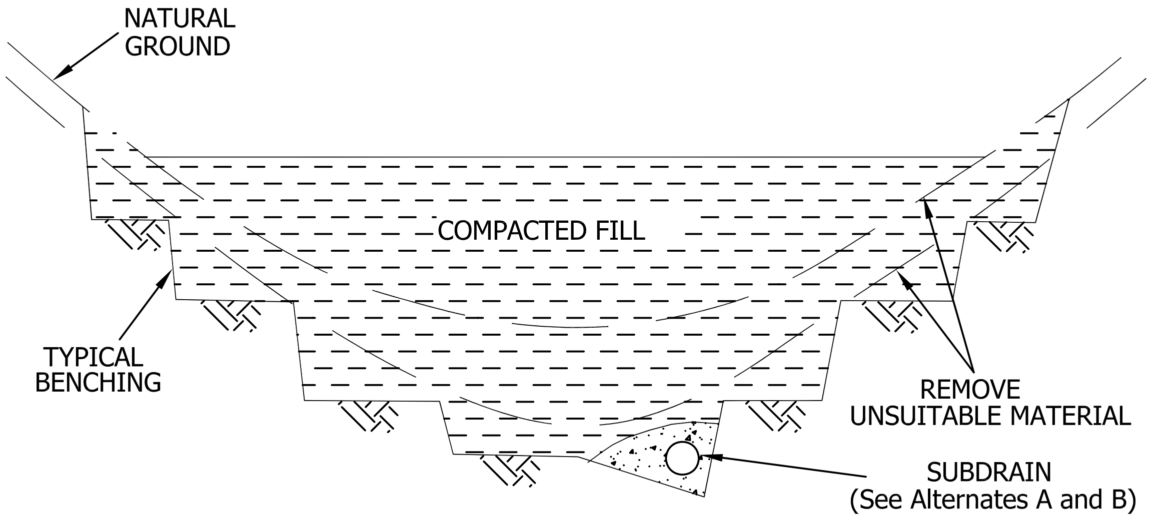
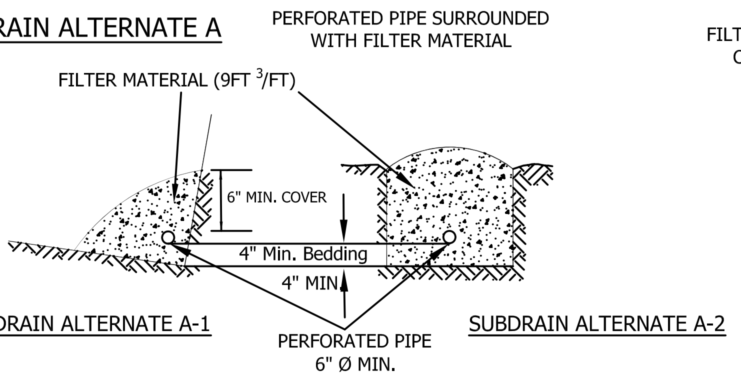


SUBDRAIN ALTERNATE A

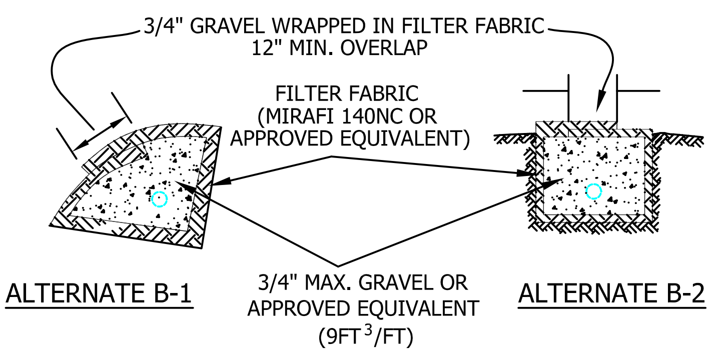


FILTER MATERIAL

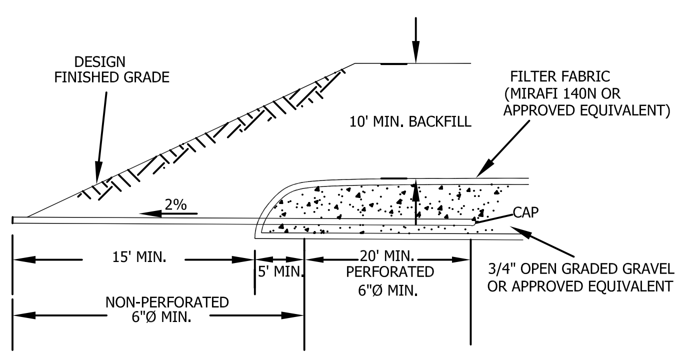
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



DETAIL OF CANYON SUBDRAIN TERMINAL



○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C G-112	Leighton

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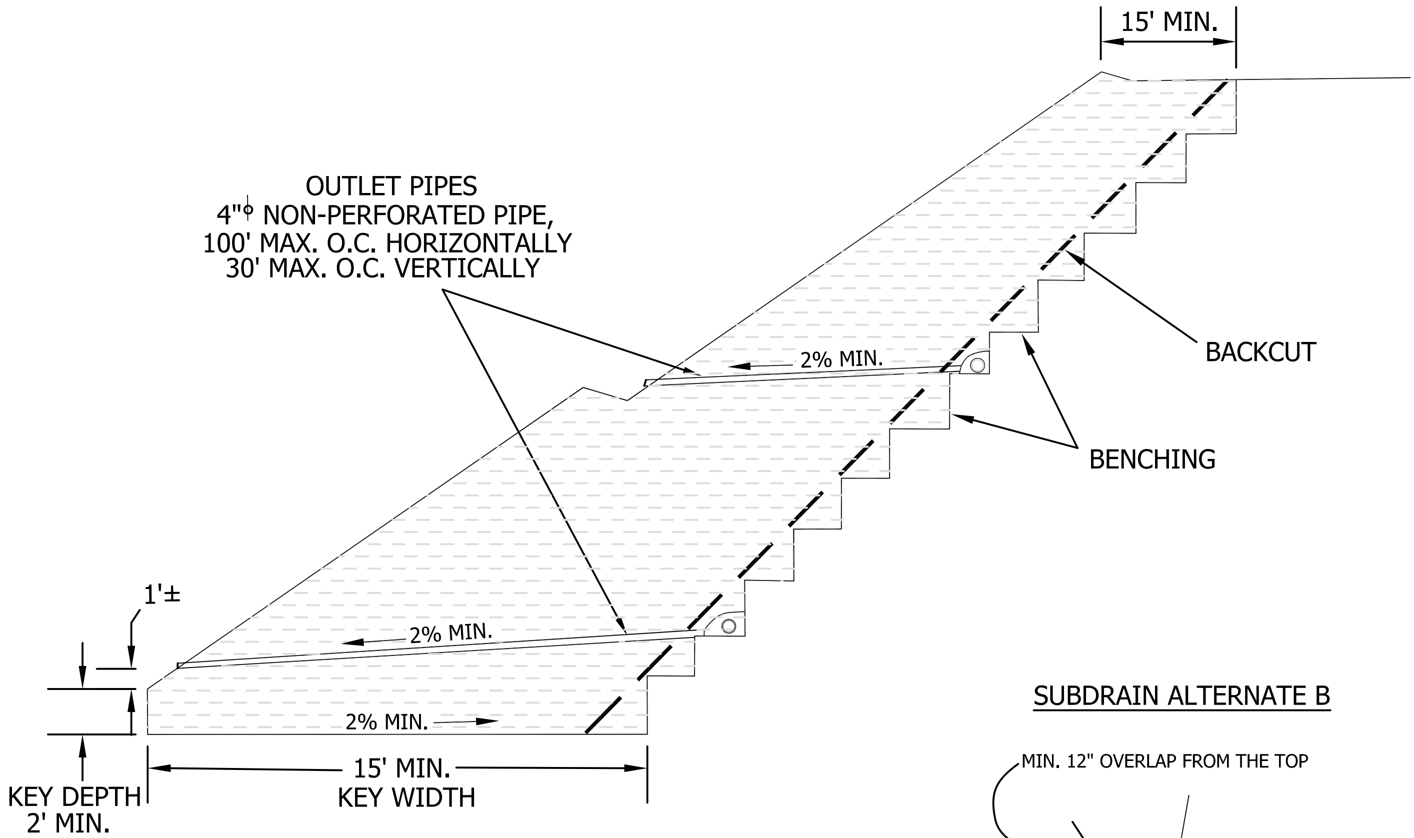
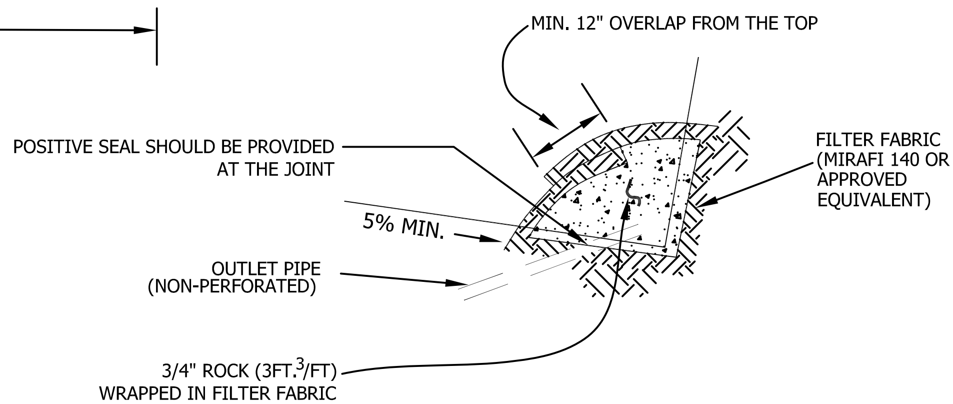
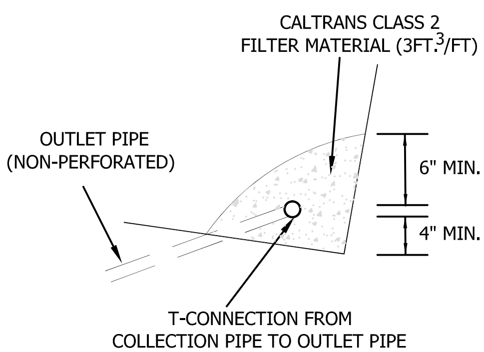


SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D G-113	Leighton

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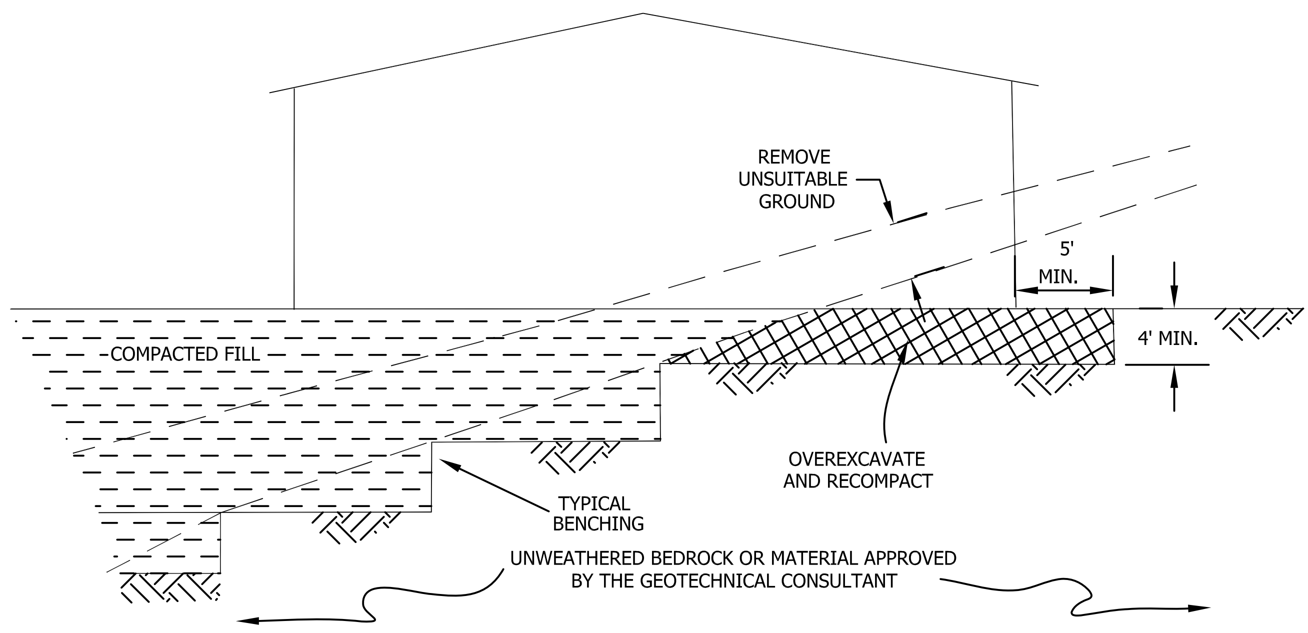
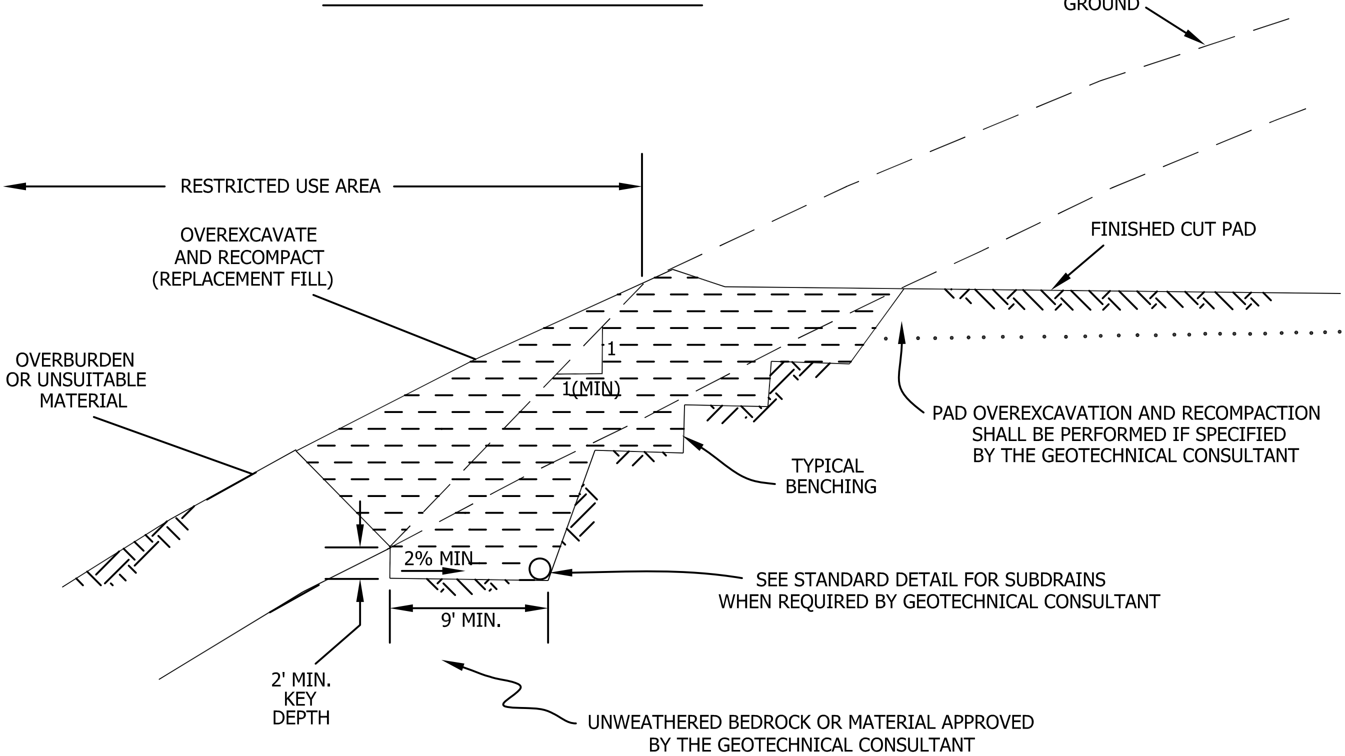
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E G-114	Leighton

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Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

November 8, 2018

Project No. 11737.002

6509 Serrano L.P.
4040 MacArthur Boulevard, Suite 300
Newport Beach, California 92660

Attention: Mr. John Saunders

Subject: Addendum No. 1 to Geotechnical Exploration Report
Proposed Residential Development
6501-6513 East Serrano Avenue
Anaheim, California

References: Leighton and Associates, Inc., 2017, *Geotechnical Exploration Report, Proposed Residential Development, 6501-6513 East Serrano Avenue, Anaheim, California*, Project No. 11737.001, dated October 9, 2017.

Leighton and Associates, Inc., 2018, Response to Review Comments Regarding Leighton's *Geotechnical Exploration Report for the Proposed Residential Development, 6501-6513 East Serrano Avenue, Anaheim, California*, Project No. 11737.002, dated August 10, 2018.

In accordance with your request and authorization, Leighton and Associates, Inc. (Leighton) is pleased to present this addendum to our referenced geotechnical exploration report (Leighton, 2017) for the subject project. Based on review of the revised Site Plan for the project prepared by Hunsaker & Associates Irvine, Inc. and plot dated October 1, 2018, the proposed development plan changed slightly from what previously proposed at the time of our report (Leighton, 2017). The changes include reducing the number of dwellings from 60 to 58 dwellings, with slight changes to the configurations of the proposed buildings in the northeastern portion of the site. Based on review of the currently proposed site plan, the recommendations presented in our referenced report (Leighton, 2017) remain applicable.

17781 Cowan ▪ Irvine, CA 92614-6009
949.250.1421 ▪ 949.250.1114 Fax

We appreciate the opportunity to be of continued service on this project. If you have any questions or if we can be of further service, please contact us at ***(866) LEIGHTON***; specifically at the phone extensions or e-mail as listed below.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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Vincent P. Ip, PE, GE 2522
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JMP/VPI/lr

Distribution: (1) Addressee

Leighton

Project No.
14174.000.000

June 29, 2018

Ms. Karen Holthe
Santiago Geologic Hazard Abatement District
Cardinal Property Management
825 N. Park Center Drive, Suite 101
Santa Ana, CA 92705

Subject: 6501-6513 East Serrano Avenue
Anaheim, California

RESIDENTIAL GRADING PLAN REVIEW

References:

1. Leighton and Associates, Inc., Geotechnical Exploration Report, 6501- 6513 East Serrano Avenue, Anaheim, CA 92807; October 9, 2017, Project No. 11737.001.
2. City of Anaheim, Department of Public Works; Review of Geotechnical Exploration Report for Proposed Residential Development, 6501-6513 East Serrano Avenue, Anaheim, CA 92807; OTH2018-01060, First Review, May 25, 2018.
3. Eberhart and Stone, Plan of Control, Prepared for Proposed Santiago Geologic Hazard Abatement District, Anaheim Hills, Anaheim, California, February 22, 1999.
4. Eberhart and Stone, Santiago Landslide Area Anaheim Hills, Geologic Hazard Abatement District Benefit Area, Anaheim, California.

Dear Ms. Holthe:

ENGEO, acting as the Santiago Geologic Hazard Abatement District (GHAD) Manager, reviewed the Leighton Geotechnical Exploration Report and City of Anaheim, Department of Public Works Review of Geotechnical Exploration Report for Proposed Residential Development (References 1 and 2) for 6501-6513 East Serrano Avenue in Anaheim, California (Subject Property). The purpose of our review was to address the City of Anaheim's request that the applicant obtain written consent from the GHAD indicating that the proposed project will not significantly impact stability of the existing Santiago landslide.

As described in Reference 1, the planned residences will replace the existing commercial buildings and improvements. The residences will be two- to three-story attached multi-family residential buildings, with private drive aisles and guest parking. Onsite biofiltration is being considered for stormwater treatment and surface drainage will be directed away from the structures.

As described in the Leighton Geotechnical Exploration Report, artificial fill thickness varied beneath the Subject Property from 1 foot to greater than 76½ feet. Puente Formation bedrock was encountered in six of the eight exploratory borings underlying the artificial fill. Groundwater was not observed in the exploratory borings at the time of the Leighton exploration. Percolation testing was conducted at two of the exploratory boring locations to support design of the planned biofiltration improvements.

The Subject Property is located northwest of the Santiago GHAD as shown on the Benefit Area Site Plan (Reference 4). The planned addition is not located within the Santiago GHAD or the mapped “Limit of Surface Damage” area. As stated in the Plan of Control (Reference 3), the formation on the Santiago landslide was caused by four primary factors:

1. North-facing hillside topography.
2. Geologic structure as north-dipping strata and south-ancient faults.
3. Geologically weak materials along critical sedimentary beds and faults.
4. Rising groundwater.

Based on our review, it does not appear that construction of the planned residences and associated improvements, including biofiltration improvements, if constructed, would affect the Santiago landslide or the ongoing mitigation efforts by the Santiago GHAD. We make no representations as to the accuracy of dimensions, measurements, calculations or any portion of the design.

If you have any questions regarding the contents of this letter, please contact us.

Very truly yours,

ENGEO INCORPORATED

Haley Trindle
ht/eh/jf

CERTIFIED ENGINEERING GEOLOGIST
ERIC HARRELL
No. 2189
STATE OF CALIFORNIA

Eric Harrell, CEG