# TTM 82311

## LA MIRADA, CALIFORNIA

## PRELIMINARY DRAINAGE REPORT

PREPARED FOR:

WARMINGTON RESIDENTIAL

3090 Pullman Street

Costa Mesa, CA 92626

714-434-4479

Date Prepared: November 2018

Date Revised: April 2019

PREPARED BY:

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949.522.7100

Project Manager: Eric Lissner, P.E.

Project Number: 114905

ENGINEERING & CONSULTING

This report was prepared by or under the supervision of the undersigned registered civil engineer who attests to the technical information contained herein. The registered civil engineer has also judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.


4/11/2019

Eric Lissner

Date

ENGINEERING & CONSULTING

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# APPENDICES

# I. INTRODUCTION

## a. PROJECT SITE DESCRIPTION

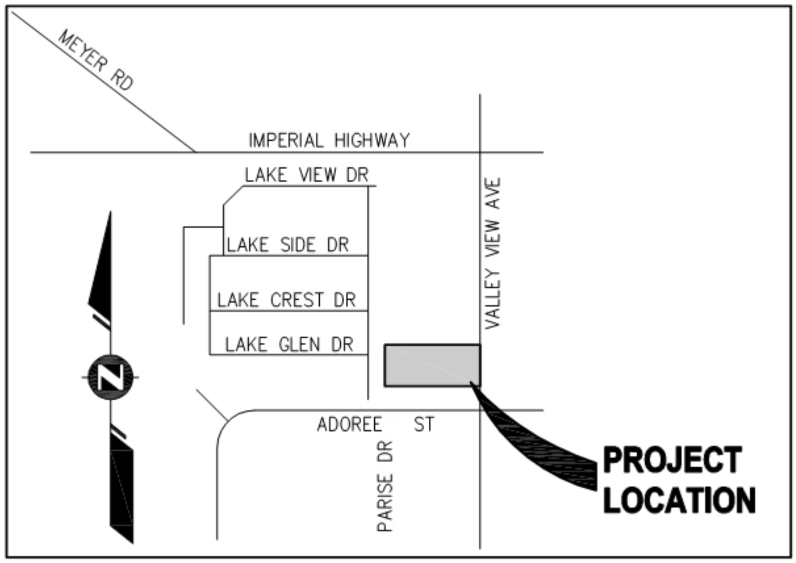
The subject site is located at 12841 Valley View Avenue in the City of La Mirada, California. Refer to vicinity map in Section 1c. The subject parcel is a rectangular lot measuring approximately 192' along the frontage of Valley View Avenue and 450' deep, consisting of 1.98 acres of land. The site is bounded by Valley View Avenue and its intersection with Adoree Street on the east, existing retail uses on the north, a mobile home park on the west, and senior housing on the south. The property is currently occupied by an abandoned thrift store building, warehouse, and surface parking lot.

The developer is proposing a residential community consisting of 39 townhomes, associated access alleys, and open space amenities. The parcel is a part of the Imperial Highway Specific Plan dated 2014, which allows for-sale residential to be developed on the parcel.

## b. PURPOSE AND SCOPE

The purpose of this preliminary study is to analyze the pre-and post-development drainage conditions in order to provide adequate drainage facilities for the proposed development project. This report also includes a discussion of Low Impact Development (LID) requirements and implemented best management practices (BMPs).

c. PROJECT LOCATION MAP




**VICINITY MAP**
N.T.S.

# II. EXISTING TOPOGRAPHIC AND HYDROLOGIC CONDITIONS

## a. EXISTING TOPOGRAPHY

The rectangular-shaped parcel is currently occupied by various commercial buildings and associated surface parking spaces. Nearly the entire site is currently improved with impervious surfaces. The site is relatively level with approximately 2% slope from north to south and 1% slope from west to east. The site has approximately 10' of elevation difference from its most extreme points.

## b. EXISTING DRAINAGE PATTERN

Generally, storm runoff from the single drainage area of the existing development leaves the property via surface flow onto Valley View Avenue. The surface low point to which existing runoff is conveyed is shown on the existing hydrology map in Appendix A. Stormwater from the project reaches Valley View Avenue and flows south, then west on Adoree Street, and then south to Parise Drive where it is intercepted by an existing storm drain system in Parise Drive.

## c. EXISTING STORM DRAIN FACILITIES

As described in section IIb, stormwater runoff from the existing development is intercepted by an existing catch basin (MTD 1835) on the west side of Parise Drive approximately 150' south of the intersection of Adoree Street & Parise Drive. MTD 1835 subsequently outlets into Milan Creek (PD 0013 and MTD 0920) before flowing into Coyote Creek North Fork, Coyote Creek proper, the San Gabriel River, and ultimately to the Pacific Ocean.

# III. HYDROLOGIC ANALYSIS

## a. STORM FREQUENCY

This report will analyze the 25-year storm for flood protection purposes in accordance with County of Los Angeles requirements.

## b. METHODOLOGY

This study was prepared using HydroCalc software (Version 1.0.3), in conformance with the Los Angeles County Hydrology Manual. Hydrology calculations are provided in Appendix C in this report.

## c. EXISTING CONDITION

As described in section II.b, the subject site consists of one drainage area. The existing hydrology analysis as performed using HydroCalc is depicted on the existing condition hydrology map in Appendix A.

*Table III.1 Existing Onsite Runoff*

| | Existing Condition 25-Year Storm | |
|---|---|---|
| | $Q_{PEAK}$ (cfs) | Area (AC) |
| Total Project | 4.5 | 1.98 |

## d. PROPOSED CONDITION

The proposed project plans to develop 39 residential townhomes on 1.98 acres. The propsoed site is consistent with the Los Angeles County Hydrology Manual land use appendix D, depicting 86% imperviousness per code 1123.

Results from the proposed hydrology analysis as performed using HydroCalc is shown on the proposed condition hydrology map in Appendix C.

# IV. PROPOSED ONSITE DRAINAGE FACILITIES

## a. UNMITIGATED PEAK STORM FLOWS

The proposed development is designed to convey surface drainage via curbs and gutters in the proposed alleyways to grate inlets located at the south ends of each of Alleys B, C, and D. The unmitigated storm flows emanating from the project are shown in Table IV.1 below. The peak runoff is consistent with the runoff generated by the site's current highly impervious developed commercial land use.

*Table IV.1 Proposed Onsite Runoff (Unmitigated)*

| | Proposed Condition 25-Year Storm | |
|---|---|---|
| | $Q_{PEAK}$ (cfs) | Area (AC) |
| Subarea A-1 | 0.9 | 0.35 |
| Subarea A-2 | 3.3 | 1.63 |
| Total Project | 4.2 | 1.98 |

## b. MITIGATED PEAK STORM FLOWS

Because the proposed condition unmitigated storm flows for the 25-year storm do not exceed the existing condition, mitigation is not required.

*Table IV.2 Runoff Summary*

| | Proposed Condition 25-Year Storm | |
|---|---|---|
| | $Q_{PEAK}$ (cfs) | Area (AC) |
| Existing Condition | 4.5 | 1.98 |
| Proposed Condition | 4.2 | 1.98 |
| Delta | -0.3 | 0 |

# V. LOW IMPACT DEVELOPMENT

This section covers the post-construction operations proposed for the development project in the City of La Mirada. It has been developed as required under State Water Resources Control Board (SWRCB) Municipal NPDES Stormwater Permit for the County of Los Angeles and the Incorporated Cities of Los Angeles County (Order No. R4-2012-0175, NPDES No. CAS004001), and in accordance with good engineering practices.

This Municipal Separate Storm Sewer (MS4) Plan will identify, at a minimum, the project performance criteria specified in the Los Angeles County MS4 Permit (Order No. R4-2012-0715), which details implementation of Best Management Practices (BMPs) that mitigate the project's Stormwater Quality Design Volume (SWQDv) defined as the runoff from the 85th percentile, 24-hour rain event, as determined from the Los Angeles County 85th Percentile Precipitation Isohyetal Map (http://dpw.lacounty.gov/wrd/hydrologygis/).

The Los Angeles County MS4 Permit requires the implementation of low impact development (LID) BMPs in addition to site design and source control measures. LID BMPs are engineered facilities that are designed to retain or biotreat runoff on the project site. All designated projects must detain the water quality volume on-site through infiltration, evapotranspiration, storm water runoff harvest and use, or a combination thereof unless it is demonstrated that it is technically infeasible to do so.

a. WATERSHED CONDITIONS

Refer to sections II.b & II.c of this report for the parcel's receiving water bodies. All channels and conveyance systems are engineered and/or hardlined to the receiving waters and therefore implementation of measures to mitigate hydrologic conditions of concern are not applicable to the site.

Coyote Creek is 303(d) listed as impaired for the following constituents: ammonia, copper, diazinon, bacteria, lead, pH, and toxicity. San Gabriel River Reach 1 is listed as impaired for bacteria and pH, while its estuary is impaired for copper, dioxin, nickel, and dissolved oxygen. Total Maximum Daily Loads (TMDLs) for the above-listed constituents are scheduled for adoption between the years 2019 and 2021. To date, a Metals TMDL has been established for the San Gabriel River since 2006.

b. SOIL AND GROUNDWATER CONDITIONS

Based upon the Geotechnical Investigation prepared by Geocon West, Inc. dated January 25, 2018, field exploration generally indicated feasible conditions for the proposed development. Groundwater was not encountered in any of the boring locations, which extended a maximum of 51 feet into the ground. A percolation test was performed at 15-20' below the existing ground surface which resulted in an observed infiltration rate of 3.47 inches per hour and was reduced using the appropriate factors to a design infiltration rate of 0.87 inches per hour. Based on design criteria in the LID Standards Manual, infiltration is considered feasible.

c. PROJECT PERFORMANCE CRITERIA

As mentioned, the proposed development project must mitigate the Stormwater Quality Design Volume (SWQDv) defined as the runoff from the 85th percentile, 24-hour storm event, as determined from the Los Angeles County 85th Percentile Precipitation Isohyetal Map, or runoff from a 0.75-inch storm, whichever is greater. Based on the isohyetal map (see Appendix B), the 85th percentile, 24-hour storm event for the project site vicinity is 0.85 inches and is utilized for determining the SWQDv for the project and sizing BMPs.

Consistent with the hierarchy in the LA County MS4 Permit, the BMPs selected shall rely on infiltration, rainfall harvest and use, and/or biofiltration, as feasible. In addition, any biofiltration features with underdrains will be designed to biofiltrate 1.5 times the portion of the SWQDv that is not retained onsite.

The following table provides the water quality volumes and flow-rates for each of the subarea under proposed conditions. BMPs selected for the project must be sized to provide the equivalent or greater treatment capacities than the listed volumes/flow-rates below. Calculations were performed utilizing the hydrologic calculator HydroCalc software developed by the Los Angeles County Department of Public Works. Detailed calculations for the proposed BMPs, performed using HydroCalc, are provided in Appendix D.

*Table V.1 Proposed Onsite SWQDv*

| | Proposed Condition 85$^{th}$ Percentile Storm | |
|---|---|---|
| | $Q_{PM}$ (cfs) | SWQDv (cu-ft) |
| Subarea A-1 | 0.12 | 844 |
| Subarea A-2 | 0.27 | 3,930 |
| Total Project | 0.39 | 4,774 |

Selection of LID features for water quality treatment is based on the pollutants of concern for the specific project site and the BMP's ability to effectively treat those pollutants, in consideration of site conditions and constraints. The LA County MS4 Permit and LID Standards Manual recognize that certain factors may limit infiltration onsite, such as presence of low-infiltrating soils, shallow groundwater, or unstable slope conditions. Similarly, storage and reuse of storm water runoff may not be suitable for sites where there is insufficient demand to reuse the collected volume of runoff (e.g., no landscape irrigation demand exists for periods longer than 1 week following a first-flush storm event). In these instances, LID biofiltration BMPs with underdrain systems may be designed and used onsite to treat any remaining runoff consistent with permit requirements.

d. INFILTRATION BMPs

As discussed in section V.b, infiltration is considered feasible for the project site. The project will maximize the amount of runoff infiltrated onsite via two drywells connected to a pipe storage system. The non-infiltrating storage consists of a six-foot storage pipe and the onsite 8" PVC storm pipes. Refer to LID calculations and exhibit in Appendix D.

*Table V.2 LID Summary*

| | Low Impact Development Summary |
|---|---|
| BMP Type | Volume (cu-ft) |
| Drywells | 598 |
| Non-infiltrating storage | 4,325 |
| Total BMP Volume | 4,923 |

The entire SWQDv is proposed to be infiltrated by the two drywells and therefore no harvest and use or biofiltration BMPs are provided.

# VI. CONCLUSION

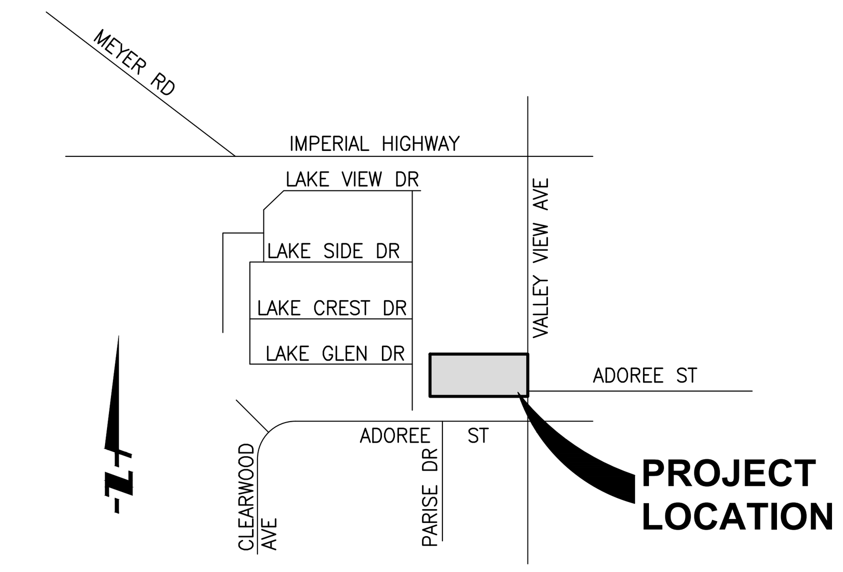
The proposed development's Hydrology and MS4 site program meets the design requirements as specified by the City of La Mirada and County of Los Angeles. The water quality BMPs are integrated into the storm drain system. Further details and narrative of the water quality facilities are provided within the project-specific MS4 permit program.

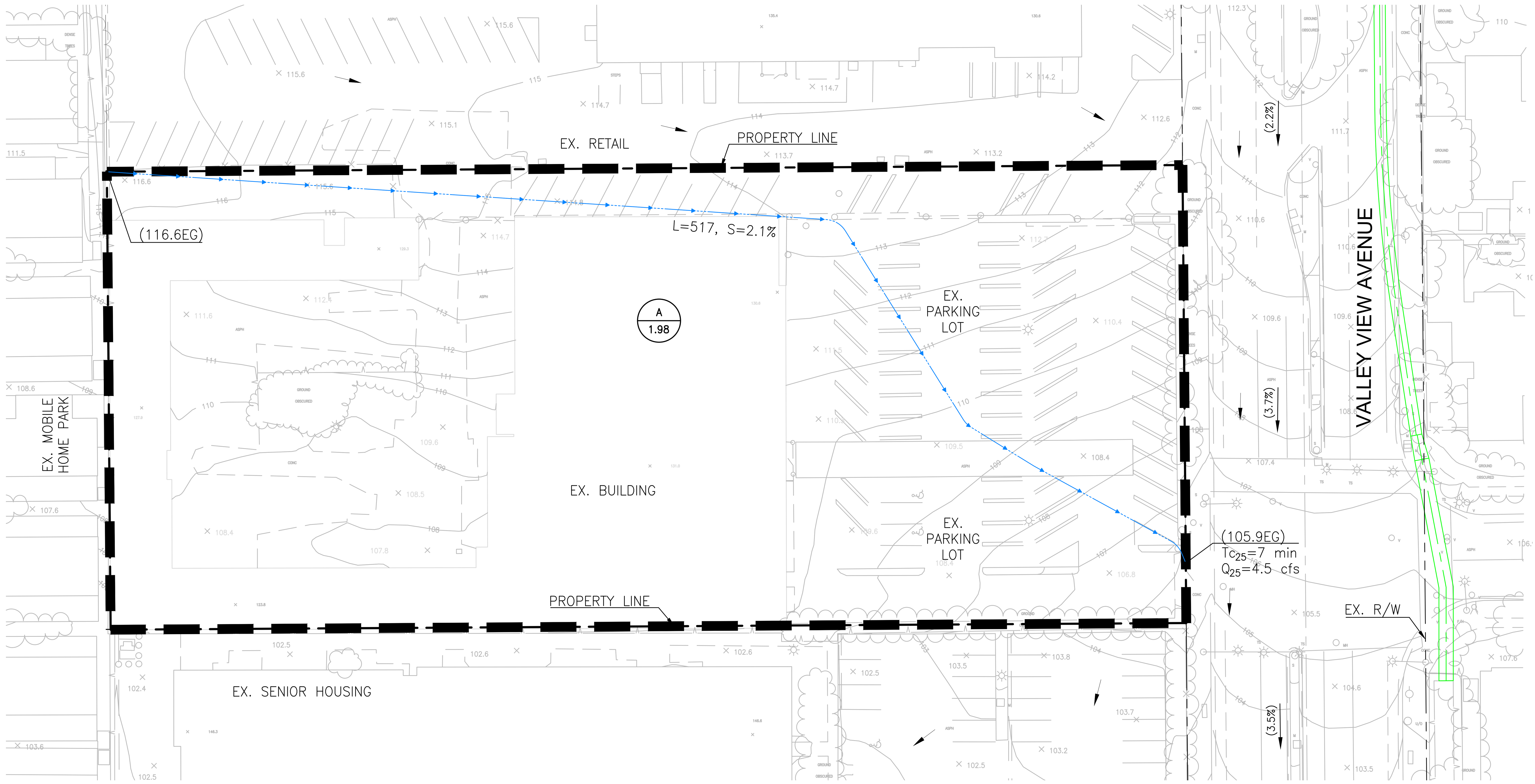
# VII. REFERENCES

1. Los Angeles County Hydrology Manual (January, 2006)
2. Los Angeles County Low Impact Development Standards Manual (February, 2014)
3. Los Angeles County Department of Public Works support files (http://ladpw.org/wrd/publication/index.cfm)
4. Los Angeles County Hydrology Map data viewer (http://dpw.lacounty.gov/wrd/hydrologygis/)

# APPENDIX A

# HYDROLOGY STUDY FOR TR 82311




## EXISTING SITE HYDROLOGIC SUMMARY

SITE ACREAGE: 1.98
SOIL TYPE: 13
25YR STORM DEPTH: 5.05IN

| | PRE-DEVELOPMENT |
|---|---|
| PERCENT IMPERVIOUSNESS | 0.9 |
| $Q_{25,\ UNMITIGATED}$ | 4.5cfs |

30' 0' 15' 30'
SCALE: 1" = 30'

## LEGEND

| | |
|---|---|
| | EXISTING PROPERTY LINE |
| | MAJOR AREA BOUNDARY |
| | DIRECTION OF SURFACE FLOW |
| (XXX.XXEG) | EXISTING GROUND |
| L=XXXX' | FLOW PATH LENGTH |
| S=XX.XX | FLOW PATH SLOPE |
| A | DRAINAGE AREA NAME |
| 0.83 | ACREAGE |
| | EXISTING STORM DRAIN |

REGISTERED PROFESSIONAL ENGINEER
ERIC LISSNER
NO. C73065
CIVIL
STATE OF CALIFORNIA

EXISTING CONDITION
HYDROLOGY MAP
TTM 82311
LA MIRADA, CA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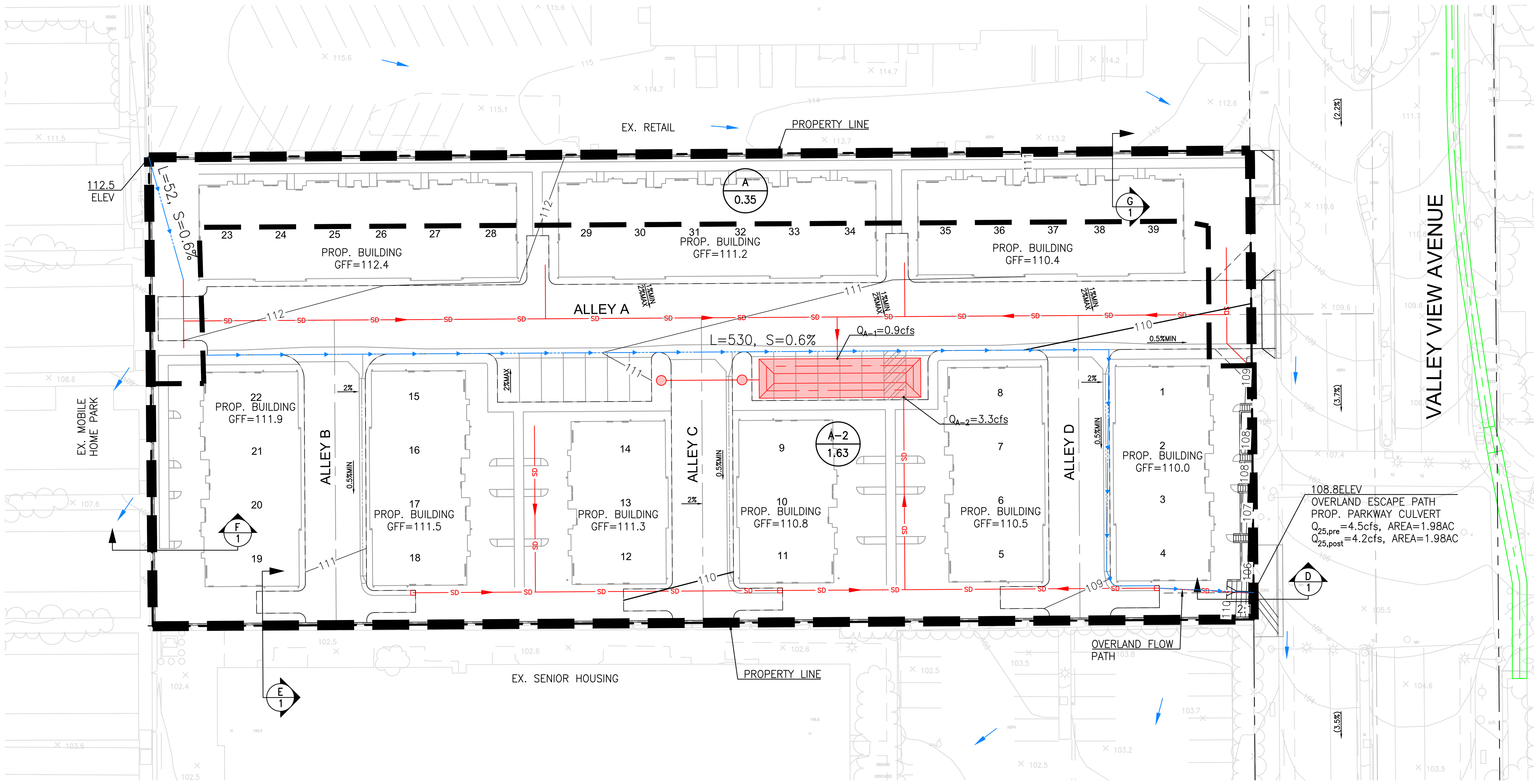
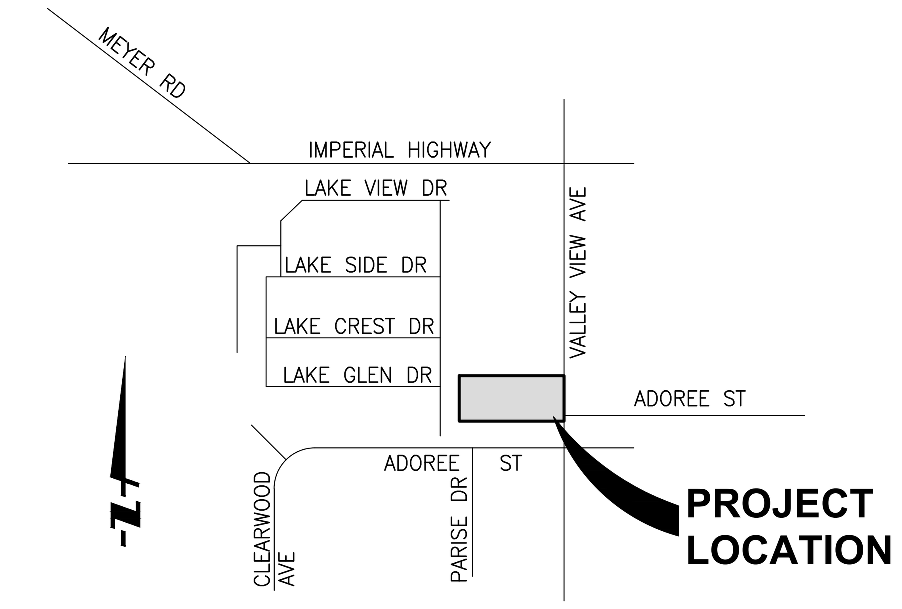
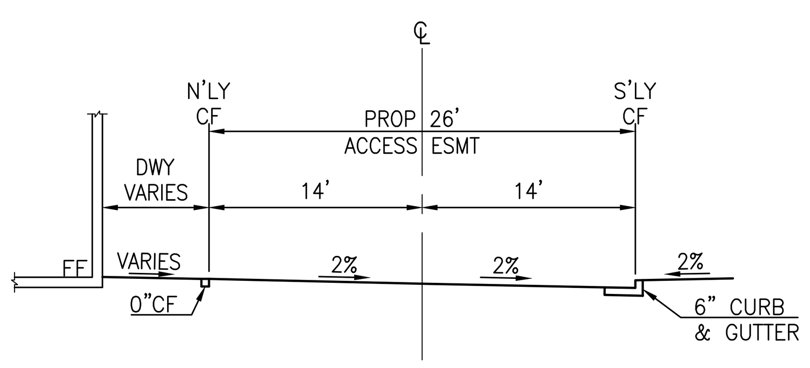
Santa Ana, California 92707
949.522.7100 | xengineeringinc.com

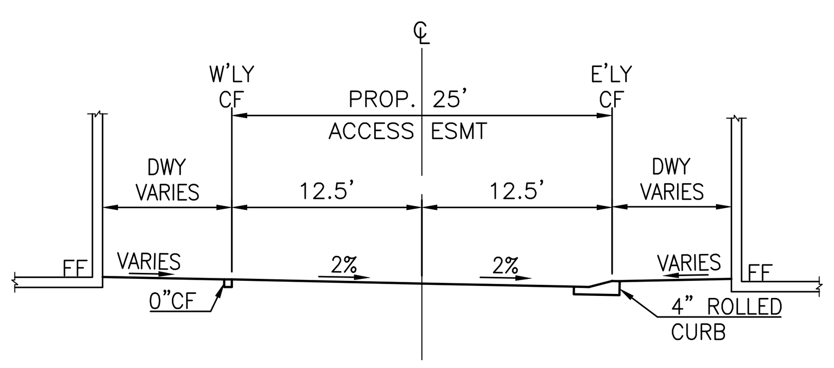
# HYDROLOGY STUDY FOR TR 82311



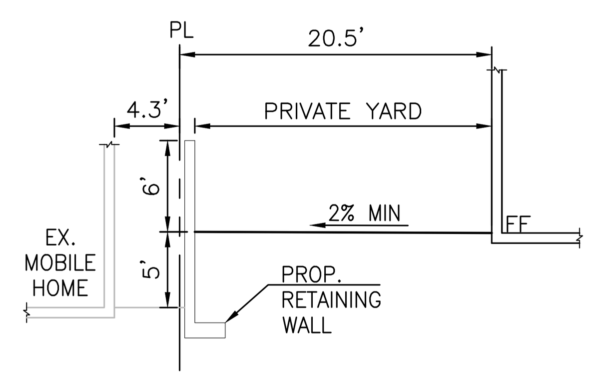



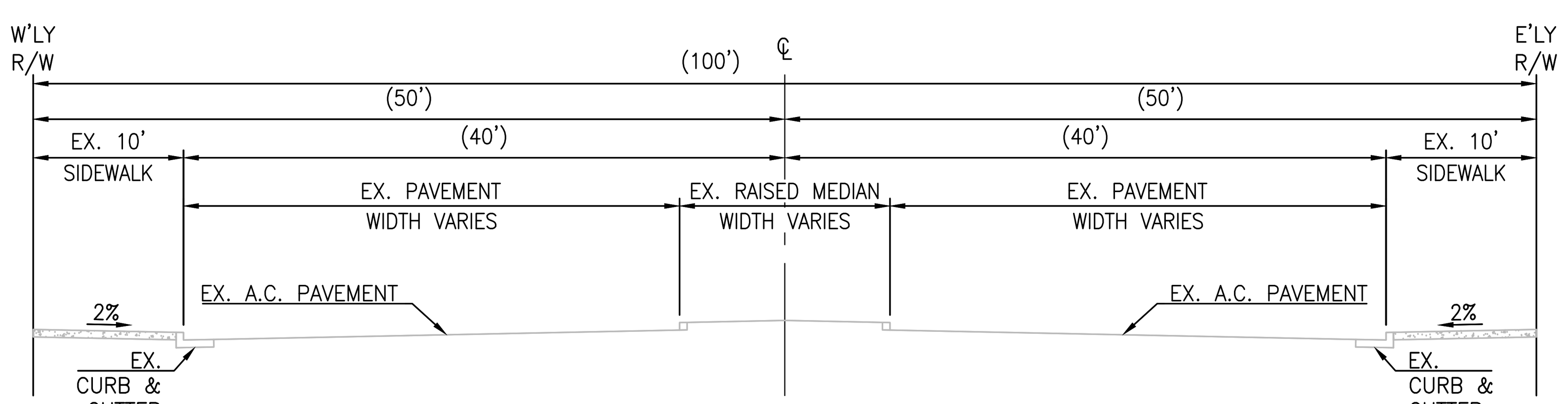
PRIVATE ALLEY A (TYP) A/1
PRIVATE DRIVEWAY AND FIRE LANE SCALE: 1"=10'



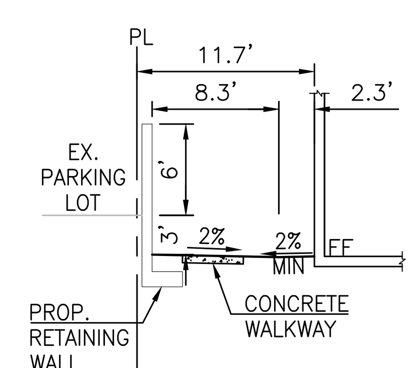
PRIVATE ALLEYS B, C & D (TYP) B/1
PRIVATE DRIVEWAY SCALE: 1"=10'



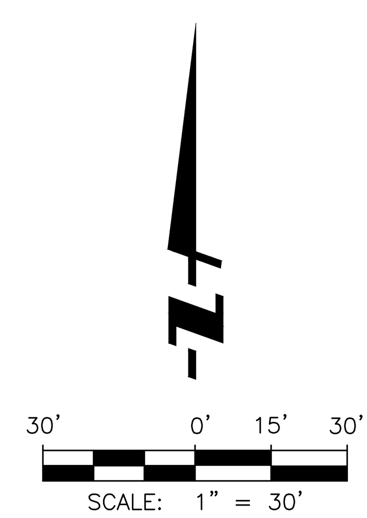
PROPERTY BOUNDARY (WEST) F/1
SCALE: 1"=10'

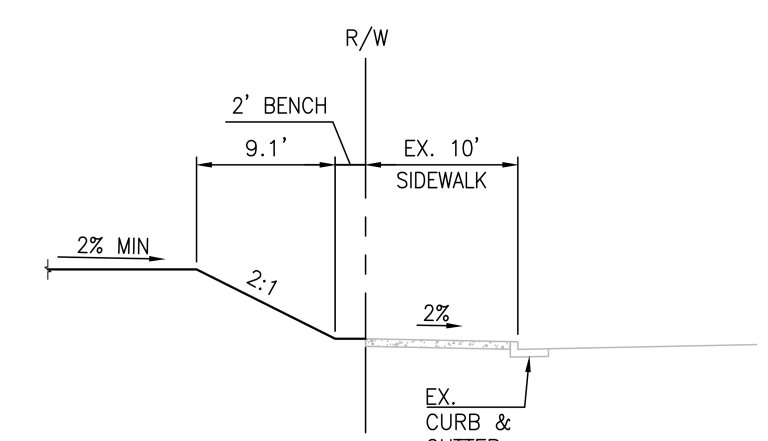


VALLEY VIEW AVENUE (TYP) C/1
SCALE: 1"=10'

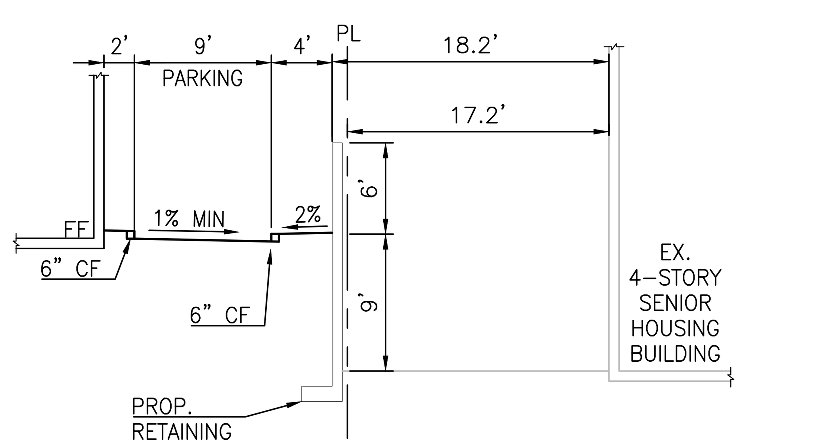


PROPERTY BOUNDARY (NORTH) G/1
SCALE: 1"=10'



OVERLAND ESCAPE ROUTE D/1
SCALE: 1"=10'
SLOPE PROTECTION TO BE PROVIDED IN FINAL ENGINEERING DESIGN



PROPERTY BOUNDARY (SOUTH) E/1
SCALE: 1"=10'

## SITE HYDROLOGIC SUMMARY

SITE ACREAGE: 1.98
SOIL TYPE: 13
25YR STORM DEPTH: 5.05IN

| | PRE-DEVELOPMENT | POST-DEVELOPMENT |
|---|---|---|
| PERCENT IMPERVIOUSNESS | 0.9 | 0.86 |
| $Q_{25,\ UNMITIGATED}$ | 4.5cfs | $Q_{A-1}$ =0.9cfs<br>$Q_{A-2}$ =3.3cfs<br>$Q_{TOTAL}$ =4.2cfs |

## NOTE

1. PROPOSED ONSITE STORM DRAIN FACILITIES AND BMPs TO BE MAINTAINED BY HOME OWNERS' ASSOCIATION
2. PROJECT SITE NOT WITHIN LOS ANGELES COUNTY ADOPTED FLOODWAY
3. PROJECT SITE NOT WITHIN FEMA FLOOD ZONE 'A'

## LEGEND

- EXISTING PROPERTY LINE
- PROPOSED DRAINAGE AREA BOUNDARY
- DIRECTION OF SURFACE FLOW
- DIRECTION OF PIPE FLOW
- MAJOR AREA NAME (A)
- ACREAGE (0.83)
- PROPOSED STREET
- PROPOSED LANDSCAPE
- PROPOSED DRYWELL
- PROPOSED WQ STORAGE
- PROPOSED STORM DRAIN (SD)
- PROPOSED CATCH BASIN
- EXISTING STORM DRAIN


PROPOSED CONDITION
HYDROLOGY MAP
TTM 82311
LA MIRADA, CA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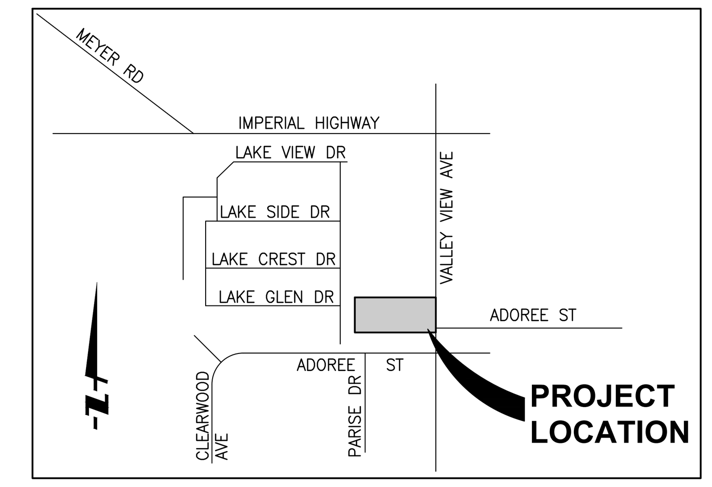
# APPENDIX B

# MAJOR LAND DIVISION
# TENTATIVE TRACT NO. 82311
# (FOR CONDOMINIUM PURPOSES)

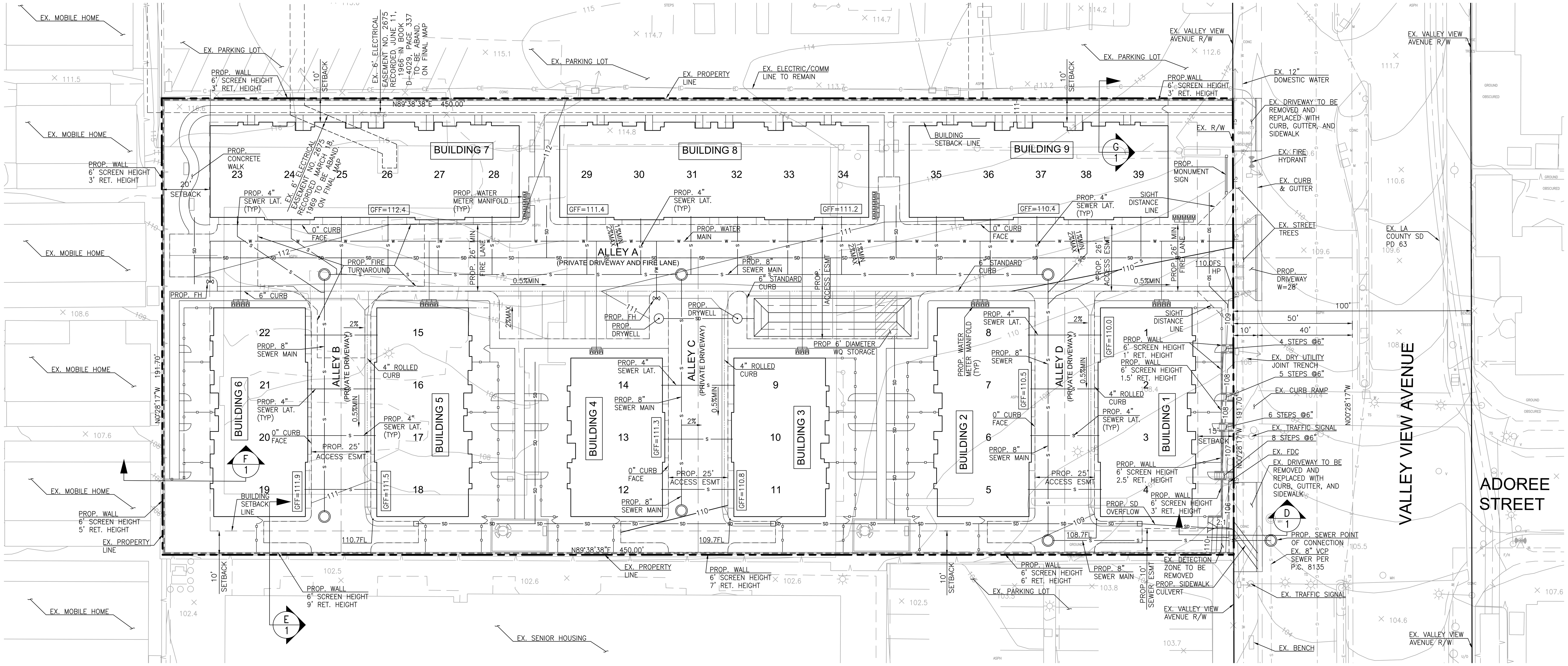
LOCATED IN THE CITY OF LA MIRADA
THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA

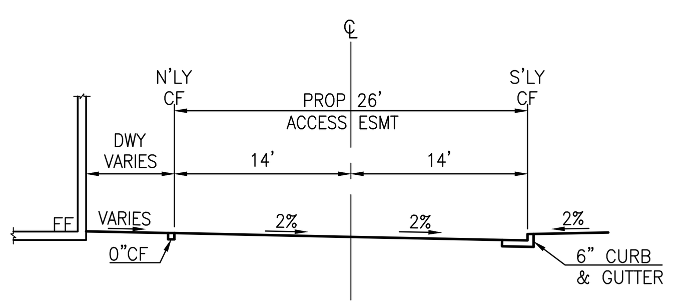
2/11/2019 - THIRD SUBMITTAL



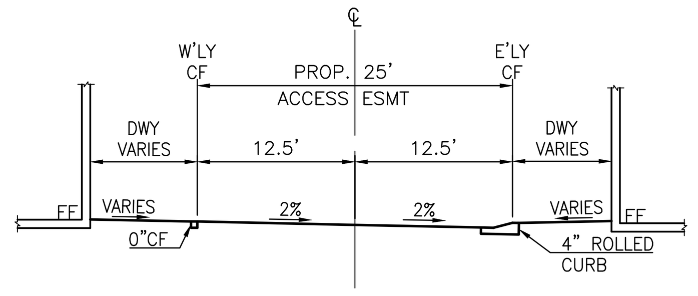

VICINITY MAP

N.T.S.

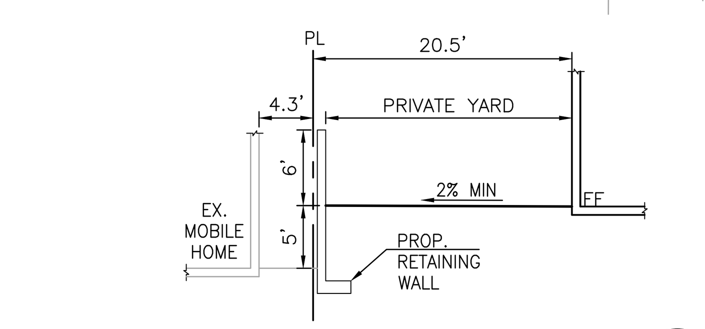


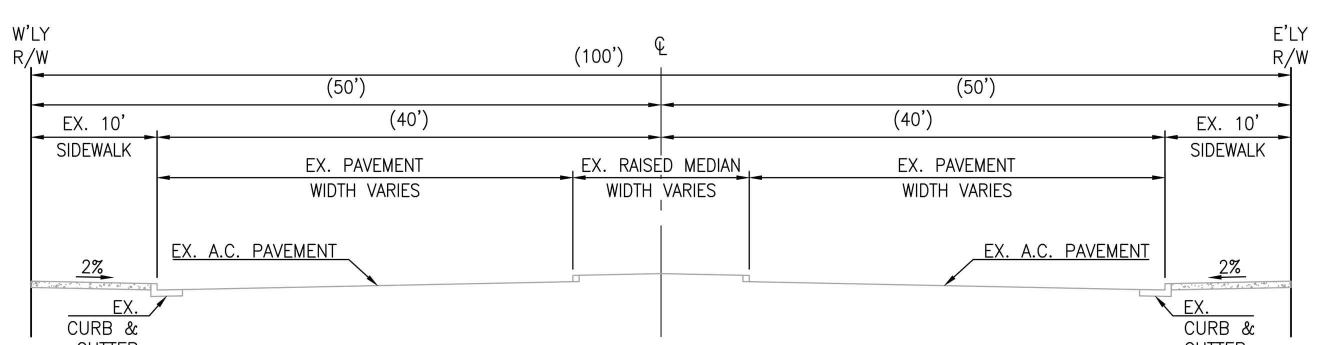

PRIVATE ALLEY A (TYP) A/1
PRIVATE DRIVEWAY AND FIRE LANE SCALE: 1"=10'



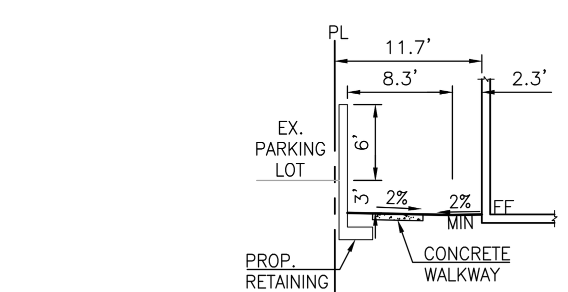

PRIVATE ALLEYS B, C & D (TYP) B/1
PRIVATE DRIVEWAY SCALE: 1"=10'



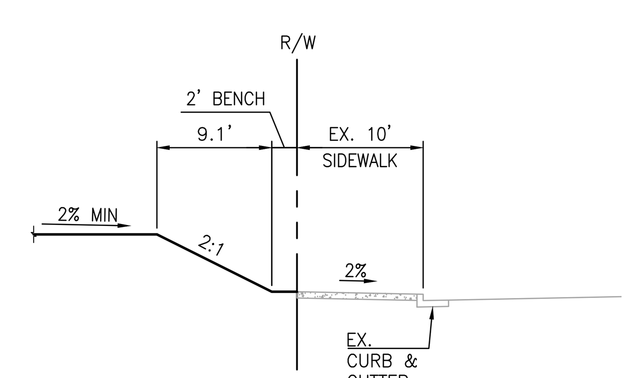

PROPERTY BOUNDARY (WEST) F/1
SCALE: 1"=10'



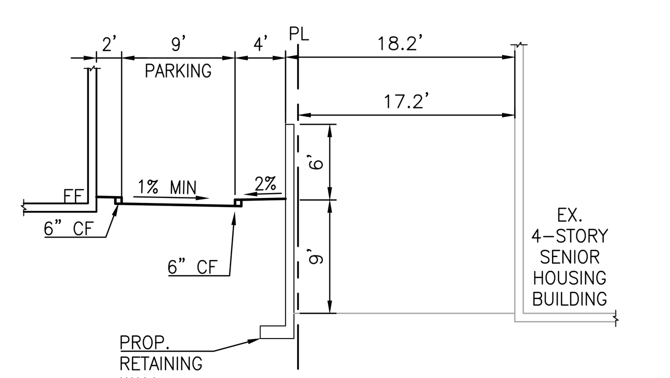

VALLEY VIEW AVENUE (TYP) C/1
SCALE: 1"=10'




PROPERTY BOUNDARY (NORTH) G/1
SCALE: 1"=10'




OVERLAND ESCAPE ROUTE D/1
SCALE: 1"=10'
SLOPE PROTECTION TO BE PROVIDED IN FINAL ENGINEERING DESIGN




PROPERTY BOUNDARY (SOUTH) E/1
SCALE: 1"=10'

## LEGEND

- EXISTING RIGHT-OF-WAY
- EXISTING PROPERTY LINE
- STREET CENTERLINE
- EXISTING EASEMENT
- PROPOSED DOMESTIC WATER
- PROPOSED SEWER
- PROPOSED SEWER MANHOLE
- PROPOSED STORM DRAIN
- 100 PROPOSED ELEVATION
- 100 EXISTING ELEVATION
- PROPOSED WATER METER MANIFOLD
- 39 PROPOSED UNIT NUMBER
- PROPOSED FIRE HYDRANT

20' 0' 10' 20'
SCALE: 1" = 20'

## ESTIMATED EARTHWORK QUANTITIES

| | CUT (CY) | FILL (CY) |
|---|---|---|
| OVEREXCAVATION | 16,000 | 16,000 |
| SHRINKAGE (7.5%) | | 1,200 |
| SUBSIDENCE | | 320 |
| RAW | 8,000 | 6,000 |
| TOTALS | 24,000 | 23,520 |
| 480 CY NET EXPORT | | |

## BUILDING AND LOT TABLE

| BUILDING # | UNIT #s | LOT # |
|---|---|---|
| 1 | 1–4 | 1 |
| 2 | 5–8 | 1 |
| 3 | 9–11 | 1 |
| 4 | 12–14 | 1 |
| 5 | 15–18 | 1 |
| 6 | 19–22 | 1 |
| 7 | 23–28 | 1 |
| 8 | 29–34 | 1 |
| 9 | 35–39 | 1 |

NOTE: PROJECT DOES NOT PROPOSE CREATION OF MULTIPLE LOTS.

## ABBREVIATIONS

| | |
|---|---|
| BLDG | BUILDING |
| BW | BACK OF WALK |
| C | CURB |
| C&G | CURB & GUTTER |
| DW | DOMESTIC WATER |
| DWY | DRIVEWAY |
| CL | CENTERLINE |
| ESMT | EASEMENT |
| EX | EXISTING |
| FG | FINISHED GRADE |
| FL | FLOWLINE |
| FS | FINISHED SURFACE |
| GFF | GARAGE FINISHED FLOOR |
| HP | HIGH POINT |
| P | PAD |
| PL | PROPERTY LINE |
| POC | POINT OF CONNECTION |
| PP | POWER POLE |
| PROP | PROPOSED |
| R/W | RIGHT OF WAY |
| RW | RECLAIMED WATER |
| S | SEWER |
| SD | STORM DRAIN |
| SWLK | SIDEWALK |
| TC | TOP OF CURB |

## BASIS OF BEARINGS

BEARING SHOWN HEREON ARE BASED UPON THE CENTERLINE OF VALLEY VIEW AVENUE AS SHOWN ON PARCEL MAP NO. 5226 P.M.B. 60/2, BEING NORTH 00°28'17" WEST (NORTH 00°04'05" EAST)

## BENCHMARK

COUNTY OF LOS ANGELES BENCHMARK NO. Y 11898
METROPOLITAN WATER DISTRICT RP TAG IN SOUTH CATCH BASIN RET 2.0 FEET WEST OF BEGINNING OF CURB RETURN (BCR) AT THE SOUTH EAST CORNER OF VALLEY VIEW AVENUE AND IMPERIAL HIGHWAY.

ELEVATION = 119.067 (NGVD88)

## LEGAL DESCRIPTION

PARCEL 3 OF PARCEL MAP NO. 5226, IN THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 60, PAGE 2, OF PARCEL MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY, MAY 23, 1962.

## ASSESSOR PARCEL NUMBER

8044–031–018

## FEMA FLOOD ZONE

THE SITE IS LOCATED IN FLOOD ZONE X, AREA DETERMINED TO BE OUTSIDE THE 0.2% ANNUAL CHANCE FLOODPLAIN, PER FIRM 06307C1842F

## EXISTING SITE INFORMATION

| | |
|---|---|
| CITY: | LA MIRADA |
| EXISTING AND PROPOSED ZONING: | SPECIFIC PLAN |
| SPECIFIC PLAN: | IMPERIAL HIGHWAY SPECIFIC PLAN |
| PLANNING AREA: | PA–1B |

## PA-1B DEVELOPMENT STANDARDS

| | |
|---|---|
| BUILDING COVERAGE: | 50% |
| FRONT YARD SETBACK: | 15' |
| SIDE YARD SETBACK: | 10' |
| REAR YARD SETBACK: | 20' |
| MAX HEIGHT LIMIT: | 50' |
| MAX DENSITY: | 45 DU/AC |
| REQUIRED OPEN SPACE: | 400SF/UNIT (EXCLUSIVE OF FRONT SETBACK & 10' MIN DIM) |
| REQUIRED PARKING: | 2.33 SP/UNIT |

## SITE ADDRESS

12841 VALLEY VIEW AVE.
LA MIRADA, CA

## OWNER

SHEKINAH CHURCH INTERNATIONAL
12841 VALLEY VIEW AVENUE
LA MIRADA, CA

## DEVELOPER

WARMINGTON RESIDENTIAL
3090 PULLMAN STREET
COSTA MESA, CA, 92626
TEL: 714–434–4479

CONTACT: WILLIAM INGHRAM

## CIVIL ENGINEER

X ENGINEERING & CONSULTING, INC.
6 HUTTON CENTRE DR., SUITE 650,
SANTA ANA, CA 92707
TEL: 949.522.7100

CONTACT: ERIC LISSNER, PE

## GEOTECHNICAL ENGINEER

GEOCON WEST, INC.
3303 N. SAN FERNANDO BLVD., SUITE 100
BURBANK, CALIFORNIA, 91504
TEL: 818.841.8388

CONTACT: JELISA ADAMS, GE 3092

## SHEET INDEX



## BOUNDARY MONUMENT NOTE

BOUNDARY MONUMENTS TO BE SET AFTER RECORDATION OF FINAL MAP.

## EXISTING STRUCTURE DISPOSITION

1. ALL ONSITE STRUCTURES ARE TO BE DEMOLISHED EXCEPT AS NOTED PRIOR TO FINAL MAP APPROVAL.
2. ALL OFFSITE STRUCTURES ARE TO REMAIN EXCEPT AS NOTED.

PREPARED BY:

X ENGINEERING & CONSULTING, INC.
6 Hutton Centre Drive, Suite 650
Santa Ana, California 92707
949.522.7100 | xengineeringinc.com

REGISTERED PROFESSIONAL ENGINEER ERIC LISSNER NO. C84264 CIVIL STATE OF CALIFORNIA

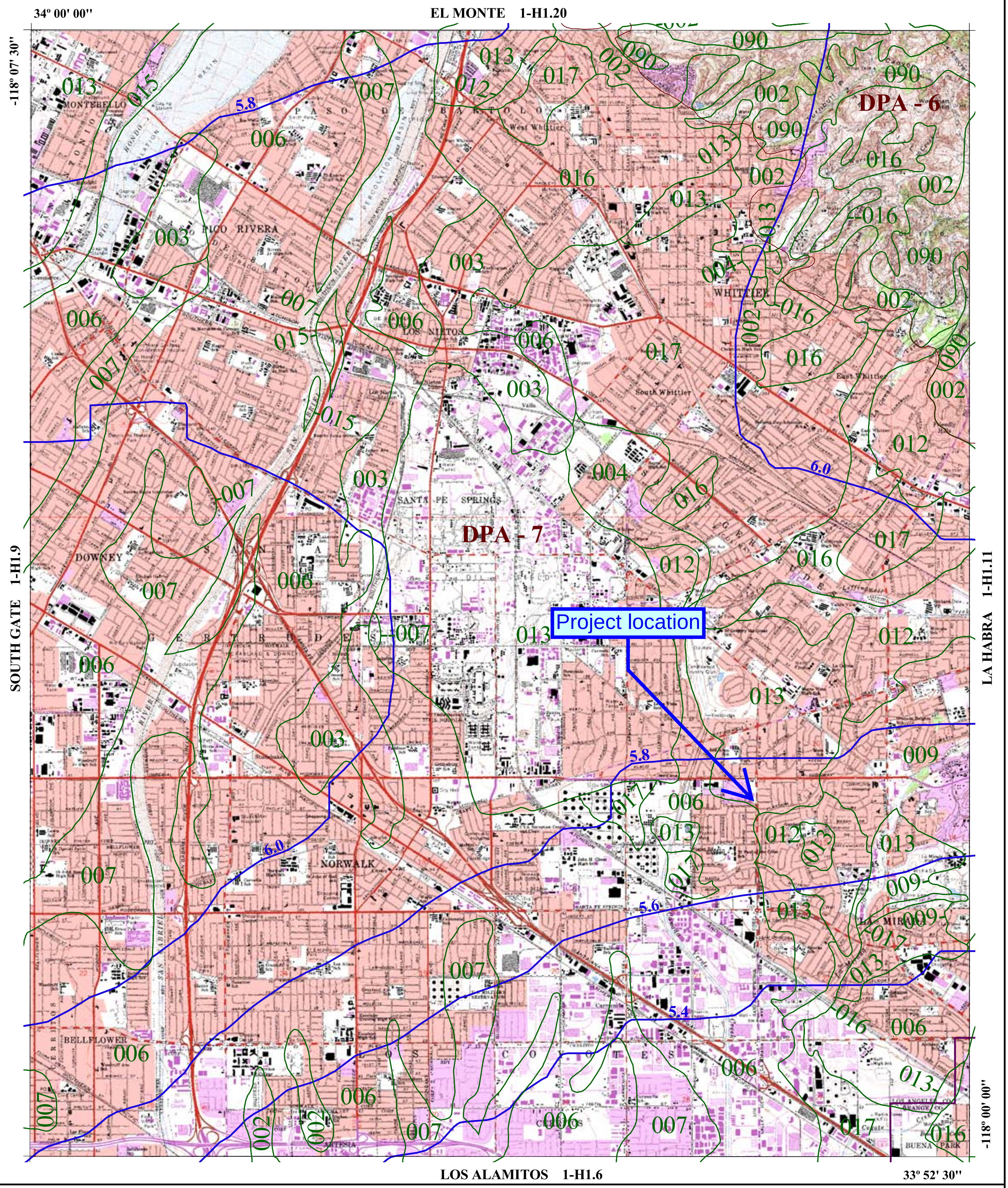
DESIGNED BY: E.L.
DRAWN BY: N.R.S. / NS
CHECKED BY: E.L.

| △ | BY | REVISIONS | DATE | APP'D |
|---|---|---|---|---|
| | | | | |

SHEET 1 OF 1

CITY OF LA MIRADA
COUNTY OF LOS ANGELES

TITLE SHEET
TENTATIVE TRACT MAP 82311
12841 VALLEY VIEW AVE.
LA MIRADA, CA




016 SOIL CLASSIFICATION AREA

7.2 INCHES OF RAINFALL

DPA - 6 DEBRIS POTENTIAL AREA

1 0 1 2 Miles

25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878

10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714

# W H I T T I E R

# 50-YEAR 24-HOUR ISOHYET

1-H1.10

LOS ANGELES COUNTY PUBLIC WORKS Public Service That Works

# GEOTECHNICAL INVESTIGATION

## PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
## 12841 VALLEY VIEW AVENUE
## LA MIRADA, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

**WARMINGTON RESIDENTIAL**
**COSTA MESA, CALIFORNIA**

**PROJECT NO. A9708-88-01**

**JANUARY 25, 2018**

Project No. A9708-88-01
January 25, 2018

Mr. William Inghram
Warmington Residential
3090 Pullman Street
Costa Mesa, California 92626

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
12841 VALLEY VIEW AVENUE, LA MIRADA, CALIFORNIA

Dear Mr. Inghram:

In accordance with your authorization of our proposal dated December 12, 2017, we have performed a geotechnical investigation for the proposed multi-family residential development located at 12841 Valley View Avenue in the City of La Mirada, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

**GEOCON WEST, INC.**

REGISTERED PROFESSIONAL ENGINEER PETRINA ZEN C87489 CIVIL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER JELISA THOMAS ADAMS GE3092 GEOTECHNICAL STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST SUSAN FRANZEN KIRKGARD No. 1754 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

| | | |
|---|---|---|
| Petrina Zen | Jelisa Thomas Adams | Susan F. Kirkgard |
| PE 87489 | GE 3092 | CEG 1754 |

(EMAIL) Addressee

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**TABLE OF CONTENTS (Continued)**

# GEOTECHNICAL INVESTIGATION

## 1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential development located at 12841 Valley View Avenue in the City of La Mirada, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on December 20, 2017, by excavating four 8-inch diameter borings to depths between approximately 20 and 51 feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

## 2. SITE AND PROJECT DESCRIPTION

The subject site is located at 12841 Valley View Avenue in the City of La Mirada, California. The site is a rectangular-shaped parcel and is currently occupied by a former warehouse facility and associated parking lot. The site is bounded by a commercial development consisting of one- and two-story structures and paved surface parking and drive lanes to the north, a four-story multi-family residential structure and associated paved surface parking lots to the south, Valley View Avenue to the east, and single-family residential structures to the west. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of shrubs and trees, which are located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a 42-unit multi-family residential development. Preliminary project plans indicate that the development will consist of six structures, anticipated to be two- or three-stories, and constructed at or near present grade (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 300 kips, and wall loads will be up to 3 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

## 3. GEOLOGIC SETTING

The site is located in the eastern portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Whittier Fault located 4.7 miles to the northeast.

## 4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Pleistocene age alluvial deposits consisting of sand, silt and clay (Dibblee, 2001, California Geological Survey [CGS], 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

### 4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 5 feet below existing ground surface. The artificial fill generally consists of brown to dark brown or yellowish brown sandy silt. The artificial fill is characterized as slightly moist and firm. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

## 4.2 Alluvium

Alluvium was encountered beneath the fill. The alluvium generally consists of light brown to dark brown or yellowish brown to reddish brown interbedded clay, sandy clay, sandy silt, silty sand, sand with silt and poorly graded sand. The alluvial soils are primarily fine- to medium-grained, slightly moist, and soft to hard or loose to very dense.

# 5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Whittier Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 10 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in our borings, drilled to a maximum depth of 51 feet below the existing ground surface. Based on the lack of groundwater in our borings, and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.17).

# 6. GEOLOGIC HAZARDS

## 6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007; CGS, 2018b). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018a) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Whittier Fault located approximately 4.7 miles to the northeast (Ziony and Jones, 1989). Other nearby active faults include the Newport-Inglewood Fault Zone, the Hollywood Fault, the Duarte Fault, the Sierra Madre Fault, the Palos Verdes Fault Zone, the Chino Fault, and the Elsinore Fault located approximately 10.5 miles southwest, 15.5 miles north, 16.5 miles north-northeast, 17 miles north-northeast, 18 miles southwest, 19 miles east-northeast, and 23 miles east of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 38 miles northeast of the site.

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987, $M_w$ 5.9 Whittier Narrows earthquake and the January 17, 1994, $M_w$ 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

## 6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

**LIST OF HISTORIC EARTHQUAKES**

| Earthquake (Oldest to Youngest) | Date of Earthquake | Magnitude | Distance to Epicenter (Miles) | Direction to Epicenter |
|---|---|---|---|---|
| San Jacinto-Hemet area | April 21, 1918 | 6.8 | 60 | E |
| Near Redlands | July 23, 1923 | 6.3 | 45 | E |
| Long Beach | March 10, 1933 | 6.4 | 21 | SSE |
| Tehachapi | July 21, 1952 | 7.5 | 94 | NW |
| San Fernando | February 9, 1971 | 6.6 | 40 | NW |
| Whittier Narrows | October 1, 1987 | 5.9 | 10 | NNW |
| Sierra Madre | June 28, 1991 | 5.8 | 24 | N |
| Landers | June 28, 1992 | 7.3 | 93 | ENE |
| Big Bear | June 28, 1992 | 6.4 | 72 | ENE |
| Northridge | January 17, 1994 | 6.7 | 36 | NW |
| Hector Mine | October 16, 1999 | 7.1 | 111 | ENE |

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

## 6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake ($MCE_R$).

**2016 CBC SEISMIC DESIGN PARAMETERS**

| Parameter | Value | 2016 CBC Reference |
|---|---|---|
| Site Class | D | Section 1613.3.2 |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (short), $S_S$ | 1.961g | Figure 1613.3.1(1) |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), $S_1$ | 0.702g | Figure 1613.3.1(2) |
| Site Coefficient, $F_A$ | 1.0 | Table 1613.3.3(1) |
| Site Coefficient, $F_V$ | 1.5 | Table 1613.3.3(2) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration (short), $S_{MS}$ | 1.961g | Section 1613.3.3 (Eqn 16-37) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration – (1 sec), $S_{M1}$ | 1.054g | Section 1613.3.3 (Eqn 16-38) |
| 5% Damped Design Spectral Response Acceleration (short), $S_{DS}$ | 1.307g | Section 1613.3.4 (Eqn 16-39) |
| 5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$ | 0.702g | Section 1613.3.4 (Eqn 16-40) |

The table below presents the mapped maximum considered geometric mean ($MCE_G$) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

**ASCE 7-10 PEAK GROUND ACCELERATION**

| Parameter | Value | ASCE 7-10 Reference |
|---|---|---|
| Mapped $MCE_G$ Peak Ground Acceleration, PGA | 0.757g | Figure 22-7 |
| Site Coefficient, $F_{PGA}$ | 1.0 | Table 11.8-1 |
| Site Class Modified $MCE_G$ Peak Ground Acceleration, $PGA_M$ | 0.757g | Section 11.8.3 (Eqn 11.8-1) |

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.65 magnitude event occurring at a hypocentral distance of 7.16 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.65 magnitude occurring at a hypocentral distance of 14.87 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

## 6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Whittier Quadrangle (CDMG, 1999) and the City of La Mirada General Plan (City of La Mirada, 2003) indicate that the southwestern portion of the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 12 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits and saturated moisture content of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B6.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 10 feet below the ground surface, a magnitude 6.65 earthquake, and a peak horizontal acceleration of 0.505g (⅔$PGA_M$). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level could be susceptible to approximately 2.5 inches of total settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

It is our understanding that the intent of the Building Code is to maintain "Life Safety" during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structures for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structures.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 10 feet below the ground surface, a magnitude 6.65 earthquake, and a peak horizontal acceleration of 0.757g ($PGA_M$). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level could be susceptible to approximately 3.1 inches of total settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 7 and 8).

### 6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the southwest. The City of La Mirada (2003) and the County of Los Angeles (Leighton, 1990) indicate the site is not within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

## 6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Los Angeles County Safety Element (Leighton, 1990), the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

## 6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2018: LACDPW, 2018b). Also, the City of La Mirada (2003) indicates the site is not within a 100-year or 500-year flood zone.

## 6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website (DOGGR, 2018), the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

## 6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

# 7. CONCLUSIONS AND RECOMMENDATIONS

## 7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 Up to 5 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures which occupy the site will likely disturb the upper few feet of soil. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.3 The enclosed liquefaction and seismically-induced settlement analyses indicate that the site soils could be susceptible to approximately 2½ inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔$PGA_M$). Differential settlement at the foundation level is anticipated to be less than 1¼ inches over a distance of 30 feet. The foundation design recommendations presented herein are intended to mitigate the effects of settlement on proposed improvements.

7.1.4 The foundation system for the proposed structures must be able to provide sufficient support for the structures and minimize the effects of differential settlement resulting from a liquefaction event. Based on these considerations, it is recommended that the proposed structures be supported on a reinforced concrete mat foundation or a post tensioned foundation system deriving support on a blanket of newly placed engineered fill.

7.1.5 It is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 It should be noted that implementation of the recommendations presented herein is not intended to completely prevent damage to the structures during the occurrence of strong ground shaking as a result of nearby earthquakes. It is intended that the structures be designed in such a way that the amount of damage incurred as a result of strong ground shaking be minimized.

7.1.7 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential movements.

7.1.8 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.9 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.1.10 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils generally found at or below a depth of 30 inches below existing ground surface, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.11 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.12 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.16).

7.1.13 Once the design and foundation loading configuration for the proposed development proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.14 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

## 7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.15).

7.2.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a "low" expansive potential (EI = 35); and are classified as "expansive" based on the 2016 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

## 7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B9) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B9) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

## 7.4 Grading

7.4.1 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration is suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

7.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

7.4.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer.

All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.4 As a minimum, it is recommended that the upper 5 feet of existing earth materials within the proposed building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

7.4.7. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.4.8 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils generally found at or below a depth of 30 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.4.10 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential movements. Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.11 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B9). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.4.12 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

## 7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 5 and 10 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent.

7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

## 7.6 Mat Foundation Recommendations

7.6.1 Subsequent to the recommended grading, a reinforced concrete mat foundation may be utilized for support of the proposed structures. The reinforced concrete mat foundation should derive support in the newly placed engineered fill and be underlain by at least 3 feet of newly placed engineered fill.

7.6.2 The recommended maximum allowable bearing value for the design of a reinforced concrete mat foundation is 3,250 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.6.3 It is recommended that a modulus of subgrade reaction of 100 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in newly placed engineered fill. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: $K_R$ = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

7.6.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

7.6.5 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slab and new placed engineered fill without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.6.6 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.6.7 This office should be provided a copy of the final construction plans so that the recommendations presented herein could be properly reviewed and revised if necessary.

## 7.7 Post Tensioned Foundation Recommendations

7.7.1 A post-tensioned concrete slab and foundation system may also be used for support of the proposed structures. The post-tensioned system should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2016 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential settlement. The post-tensioned design should incorporate the geotechnical parameters presented in the following table, which are based on the guidelines presented in the PTI, Third Edition design manual.

**POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS**

| Post-Tensioning Institute (PTI) Third Edition Design Parameters | Value |
|---|---|
| Thornthwaite Index | -20 |
| Equilibrium Suction | 3.9 |
| Edge Lift Moisture Variation Distance, $e_M$ (feet) | 5.3 |
| Edge Lift, $y_M$ (inches) | 0.61 |
| Center Lift Moisture Variation Distance, $e_M$ (feet) | 9.0 |
| Center Lift, $y_M$ (inches) | 0.3 |

7.7.2 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

7.7.3 If the structural engineer proposes a post-tensioned foundation design method other than PTI DC 10.5:

- The post-tensioned foundation system design parameters above are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches. The embedment depths should be measured from the lowest adjacent pad grade.

7.7.4 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures

7.7.5 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless specifically designed by the structural engineer.

7.7.6 Foundations may be designed for an allowable soil bearing pressure of 3,250 psf (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

7.7.7 Consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

7.7.8 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.

7.7.9 Foundation excavations should be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.) prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are consistent with those expected and have been extended to appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

7.7.10 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

7.7.11 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

## 7.8 Foundation Settlement

7.8.1 The enclosed liquefaction and seismically-induced settlement analyses indicate that the site soils could be susceptible to approximately 2.5 inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔$PGA_M$). The differential settlement at the foundation level is anticipated to be less than 1.25 inches over a distance of 30 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

7.8.2 The maximum expected static settlement for a structure supported on a mat foundation system or post-tensioned foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 3,250 psf is estimated to be less than ¾ inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet. Based on seismic considerations, the proposed structures supported on a mat foundation system should be designed for a combined static and seismically-induced differential settlement of 1¾ inches over a distance of 20 feet.

7.8.3 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

## 7.9 Miscellaneous Foundations

7.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils generally found at or below a depth of 30 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

7.9.2 If the soils exposed in the excavation bottom are loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

## 7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils may be computed as an equivalent fluid having a density of 230 pounds per cubic foot (pcf) with a maximum earth pressure of 2,300 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

## 7.11 Exterior Concrete Slabs-on-Grade

7.11.1 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.11.2 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

7.11.3 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

## 7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

**PRELIMINARY PAVEMENT DESIGN SECTIONS**

| Location | Estimated Traffic Index (TI) | Asphalt Concrete (inches) | Class 2 Aggregate Base (inches) |
|---|---|---|---|
| Automobile Parking and Driveways | 4.0 | 3.0 | 4.0 |
| Trash Truck & Fire Lanes | 7.0 | 4.0 | 12.0 |

7.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

## 7.13 Retaining Walls Design

7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.13.2 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

7.13.3 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

7.13.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.13.5 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

7.13.6 Retaining wall foundations may be supported on conventional foundations deriving support in newly placed engineered fill.

7.13.7 Continuous footings may be designed for an allowable bearing capacity of 1,500 psf, and should be a minimum of 12 inches in width and 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.13.8 Isolated spread foundations may be designed for an allowable bearing capacity of 2,000 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.13.9 The soil bearing pressure above may be increased by 200 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 2,500 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

7.13.10 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.13.11 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.13.12 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

## 7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 9). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 10).

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

## 7.15 Temporary Excavations

7.15.1 Excavations up to 5 feet in height may be required during grading and construction operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.15.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 10 feet. A uniform slope does not have a vertical portion.

7.15.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special excavation measures can be provided under separate cover, as necessary.

7.15.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

## 7.16 Stormwater Infiltration

7.16.1 During the December 20, 2017, site exploration, boring B4 was utilized to perform percolation testing. The boring was advanced to the depth listed in the table below. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with gravel. The boring was then filled with water to pre-saturate the soils. On December 21, 2017, the casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figure 11.

| Boring | Soil Type | Infiltration Depth (ft) | Measured Percolation Rate (in / hour) | Design Infiltration Rate (in / hour) |
|---|---|---|---|---|
| B4 | Silty Sand (SM) | 15-20 | 3.47 | 0.87 |

7.16.2 Based on the test method utilized (Boring Percolation Test), the reduction factor $RF_t$ may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor $RF_v$ be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor $RF_s$ may be taken as 1.0. Additional reduction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines.

7.16.3 The results of the percolation testing indicate that the soils at depths in the above table are conductive to infiltration. It is our opinion that the soil zone encountered at the depth and location as listed in the table above are suitable for infiltration of stormwater.

7.16.4 It is our further opinion that infiltration of stormwater and will not induce excessive hydro-consolidation (see Figures B3 through B5), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.16.5 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.16.6 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.

7.16.7 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.16.8 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

## 7.17 Surface Drainage

7.17.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.17.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.17.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.17.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

### 7.18 Plan Review

7.18.1 Grading and foundation plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

## LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

## LIST OF REFERENCES


Bryant, W.A. and Hart, E.W., 2007, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps*, California Division of Mines and Geology Special Publication 42, interim revision.

California Department of Water Resources, 2018, *Groundwater Level Data by Township, Range, and Section,* Web Site Address: http://www.water.ca.gov/waterdatalibrary/groundwater/hydrographs/index_trs.cfm.

California Department of Water Resources, 1961, *Planned Utilization of Groundwater Basins of the Coastal Plain of Los Angeles County*, Bulletin 104, Appendix A.

California Division of Mines and Geology, 1999; *State of California Seismic Hazard Zones, Whittier Quadrangle,* Official Map, Released: March 25, 1999.

California Division of Mines and Geology, 1998, *Seismic Hazard Evaluation of the Whittier 7.5-Minute Quadrangle, Los Angeles and Orange Counties, California,* Open File Report 98-28.

California Division of Oil, Gas and Geothermal Resources, 2018, Division of Oil, Gas, and Geothermal Resources Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close. Accessed January, 17, 2018.

California Geological Survey, 2018a, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.

California Geological Survey, 2018b, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California,* Special Publication 42, Revised 2018.

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

Dibblee, T. W., Jr., 2001, *Geologic Map of the Whittier and La Habra Quadrangles, (Western Puente Hills), Los Angeles and Orange Counties, California,* Dibblee Foundation Map # DF-74.

FEMA, 2018, Online Flood Hazard Maps, *Flood Insurance Rate Map, Los Angeles County, California and Unincorporated Areas, Map Number 06037C1842F, Date Accessed: January 17, 2018,* http://www.esri.com/hazards/index.html.

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California,* California Geological Survey Geologic Data Map No. 6.

La Mirada, City of, 2003, *General Plan,* Adopted March 25, 2003.

Leighton and Associates, Inc., 1990, *Technical Appendix to the Safety Element of the Los Angeles County General Plan, Hazard Reduction in Los Angeles County*.

## LIST OF REFERENCES (Continued)

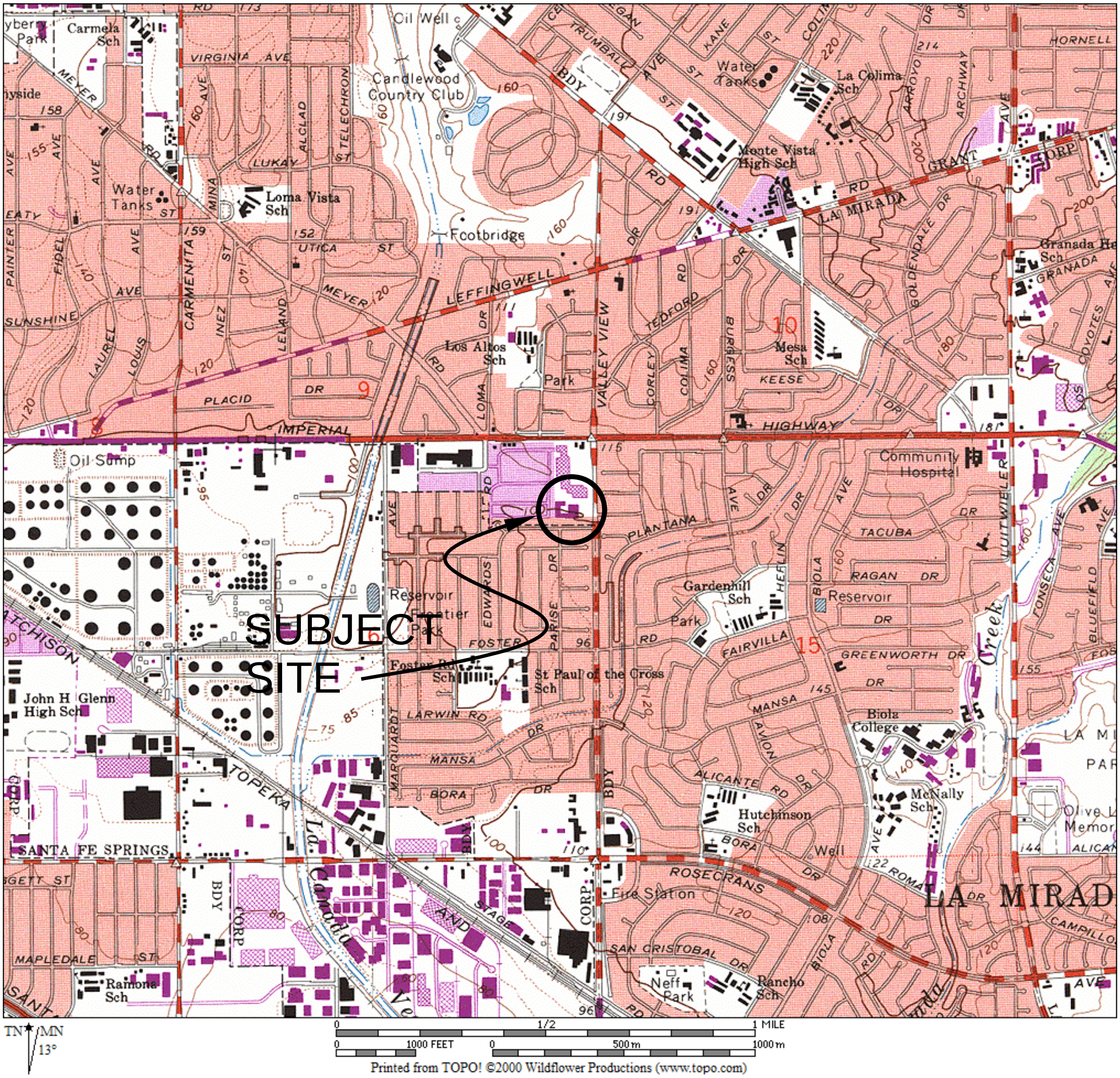
Los Angeles County Department of Public Works, 2018a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

Los Angeles County Department of Public Works, 2018b, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

Toppozada, T., Branum, D., Petersen, M, Hallstrom, C., and Reichle, M., 2000, *Epicenters and Areas Damaged by M> 5 California Earthquakes, 1800 – 1999,* California Geological Survey, Map Sheet 49.

U.S. Geological Survey, 1972, *Whittier 7.5-Minute Quadrangle Topographic Map*.

Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.




REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, WHITTIER, CA QUADRANGLE

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

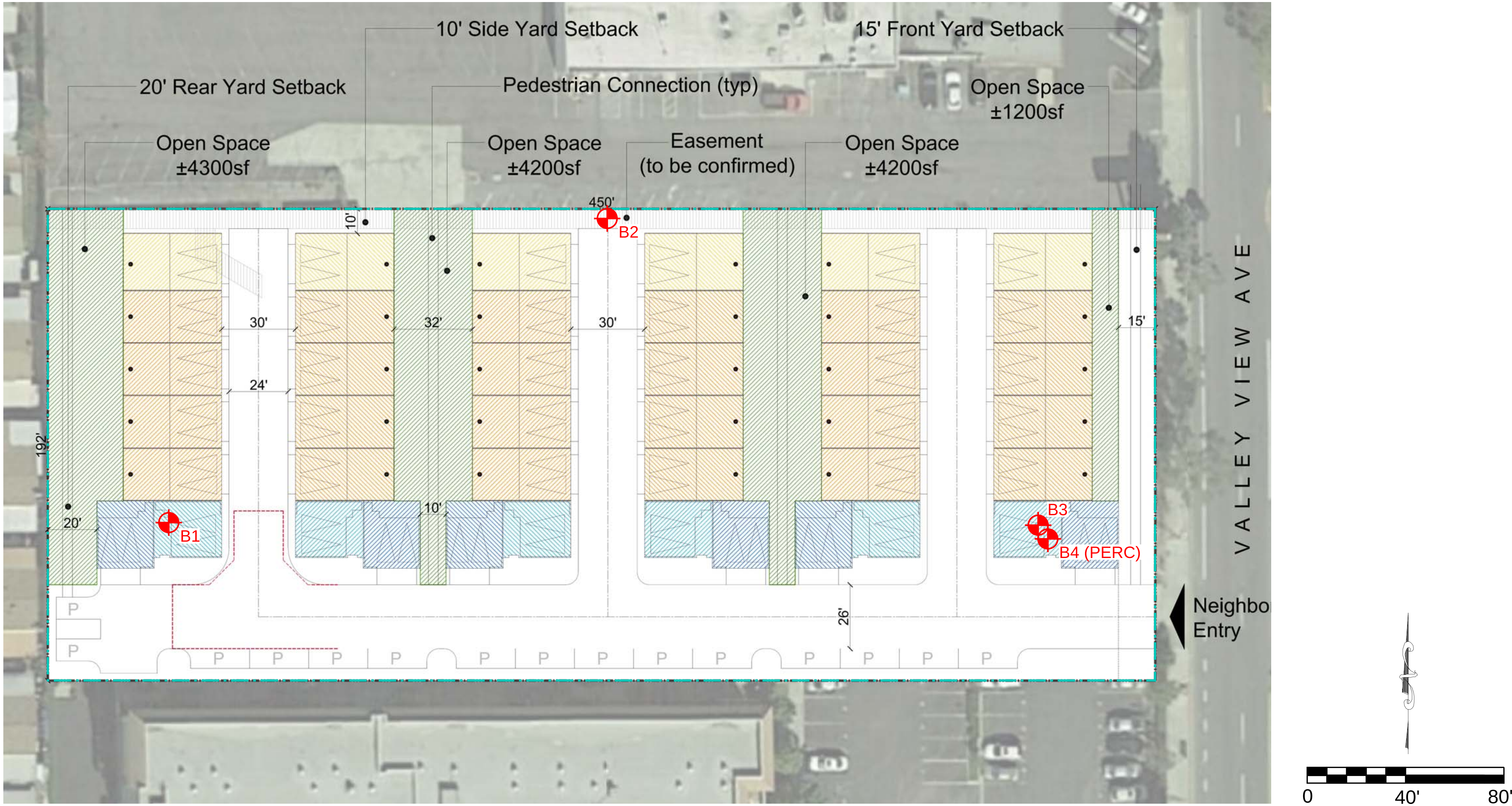
| DRAFTED BY: RA | CHECKED BY: SFK |
|---|---|

# VICINITY MAP

## RESIDENTIAL DEVELOPMENT

WARMINGTON RESIDENTIAL CALIFORNIA, INC.
12841 VALLEY VIEW AVENUE
LA MIRADA, CALIFORNIA

| JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. 1 |
|---|---|---|




# LEGEND

B4 Approximate Location of Boring

Approximate Location of Property Line

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DRAFTED BY: PZ | CHECKED BY: JTA

## SITE PLAN

RESIDENTIAL DEVELOPMENT

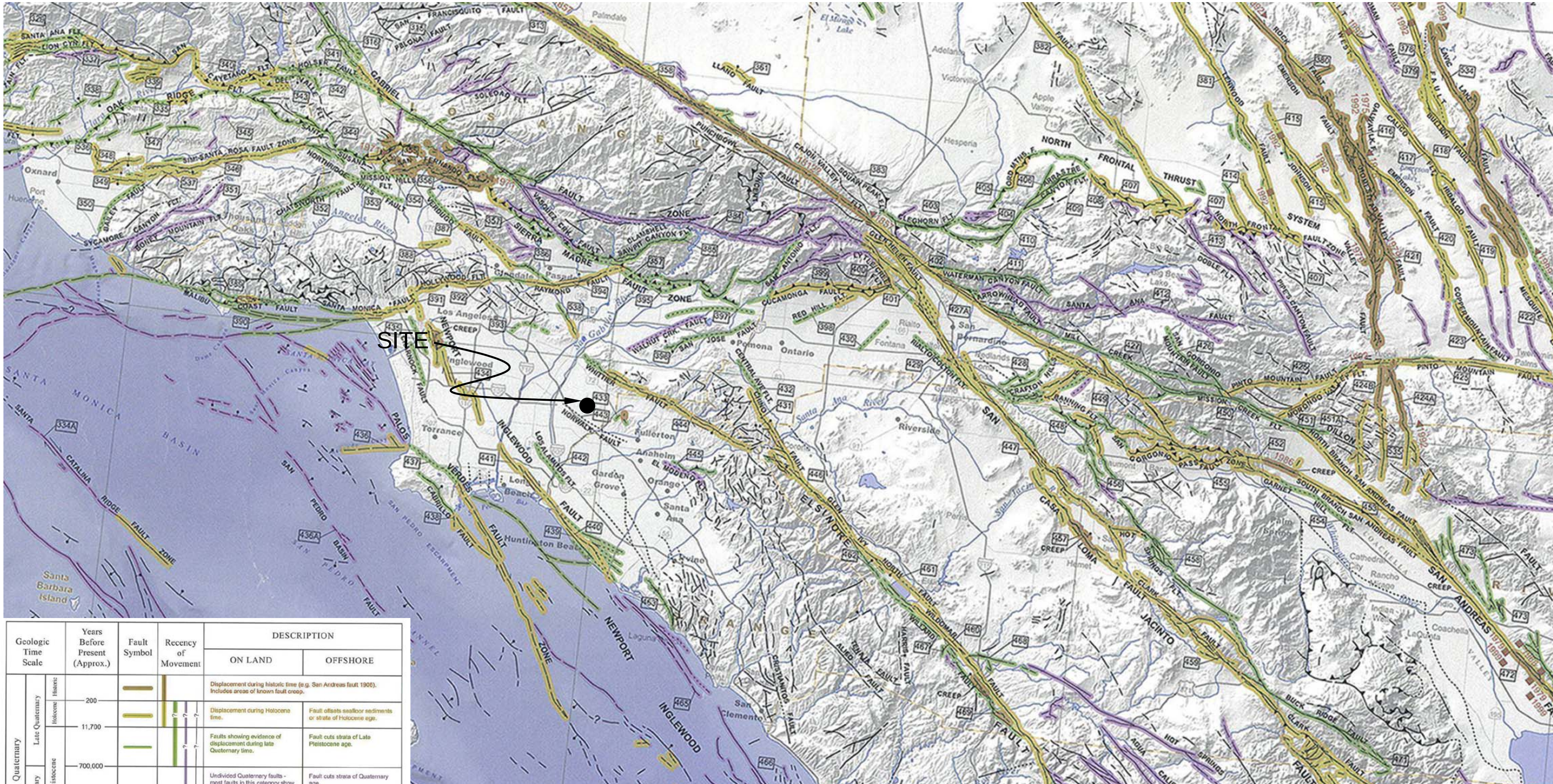
WARMINGTON RESIDENTIAL CALIFORNIA, INC.

12841 VALLEY VIEW AVENUE

LA MIRADA, CALIFORNIA

JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. 2

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.




| Geologic Time Scale | | | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION: ON LAND | DESCRIPTION: OFFSHORE |
|---|---|---|---|---|---|---|---|
| Quaternary | Late Quaternary | Historic | | | | Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep. | |
| | | | 200 | | | | |
| | | Holocene | | | ? ? ? | Displacement during Holocene time. | Fault offsets seafloor sediments or strata of Holocene age. |
| | | | 11,700 | | | | |
| | | Pleistocene | | | ? ? | Faults showing evidence of displacement during late Quaternary time. | Fault cuts strata of Late Pleistocene age. |
| | | | 700,000 | | | | |
| | Early Quaternary | | | | ? | Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. | Fault cuts strata of Quaternary age. |
| | | | 1,600,000* | | | | |
| Pre-Quaternary | | | | | | Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive. | Fault cuts strata of Pliocene or older age. |
| | | | 4.5 billion (Age of Earth) | | | | |

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

N

0 12 24 Miles

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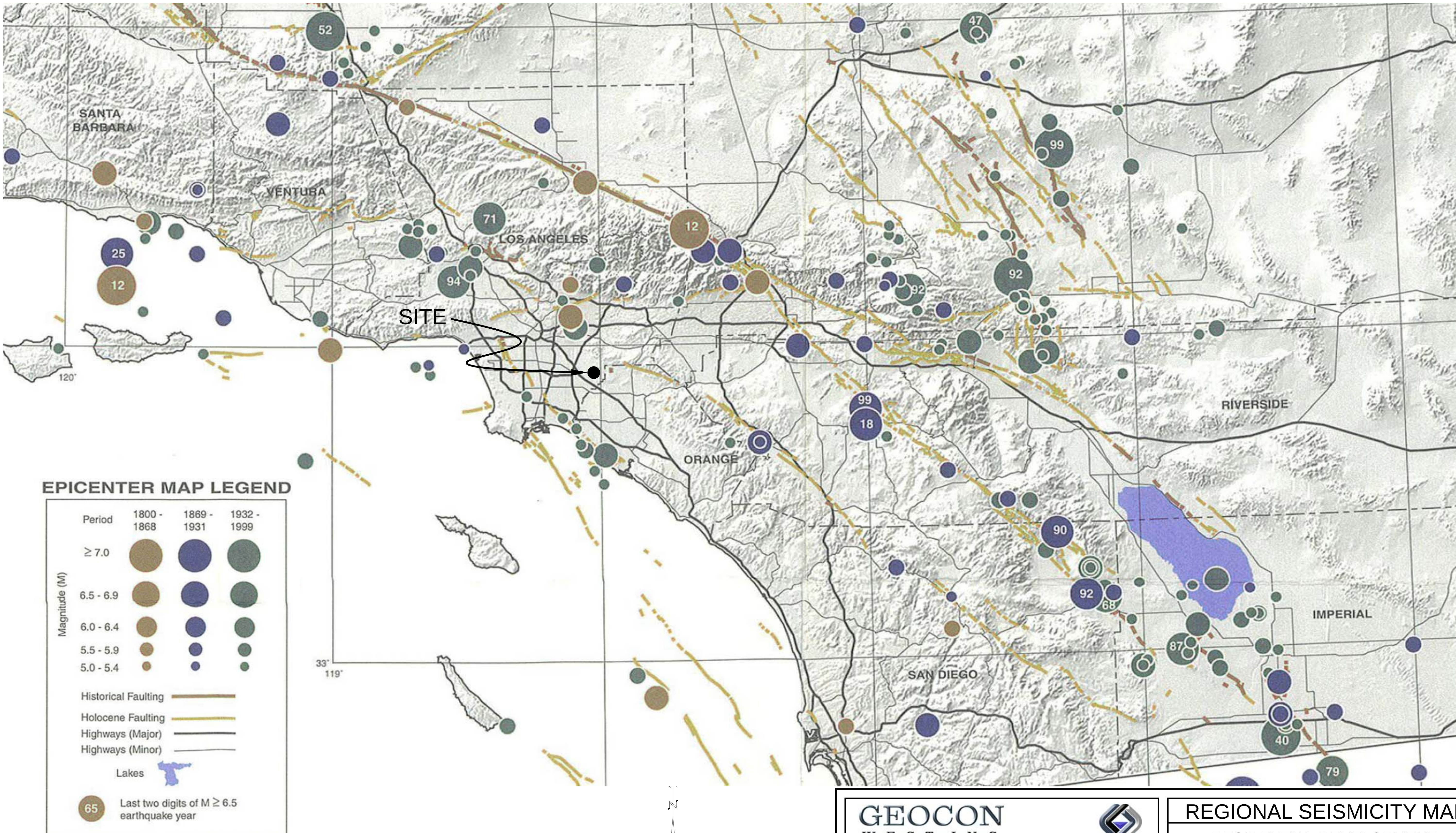
# REGIONAL FAULT MAP

RESIDENTIAL DEVELOPMENT
WARMINGTON RESIDENTIAL CALIFORNIA, INC.
12841 VALLEY VIEW AVENUE
LA MIRADA, CALIFORNIA

JAN 2018 | PROJECT NO. A9708-88-01 | FIG. 3




Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

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# REGIONAL SEISMICITY MAP

RESIDENTIAL DEVELOPMENT
WARMINGTON RESIDENTIAL CALIFORNIA, INC.
12841 VALLEY VIEW AVENUE
LA MIRADA, CALIFORNIA

| | | |
|---|---|---|
| JAN 2018 | PROJECT NO. A9708-88-01 | FIG. 4 |

GEOCON

Client : Warmington La Mirada
File No. : A9708-88-01
Boring : 1

# EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
# DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

| | |
|---|---|
| Earthquake Magnitude: | 6.65 |
| Peak Horiz. Acceleration $PGA_M$ (g): | 0.757 |
| 2/3 $PGA_M$ (g): | 0.505 |
| Calculated Mag.Wtg.Factor: | 0.739 |
| Historic High Groundwater: | 10.0 |
| Groundwater Depth During Exploration: | 50.0 |

ENERGY & ROD CORRECTIONS:

| | |
|---|---|
| Energy Correction (CE) for N60: | 1.25 |
| Rod Len.Corr.(CR)(0-no or 1-yes): | 1.0 |
| Bore Dia. Corr. (CB): | 1.15 |
| Sampler Corr. (CS): | 1.20 |
| Use Ksigma (0 or 1): | 1.0 |

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

| Depth to Base (ft) | Total Unit Wt. (pcf) | Water (0 or 1) | FIELD SPT (N) | Depth of SPT (ft) | Liq.Sus. (0 or 1) | -200 (%) | Est. Dr (%) | CN Factor | Corrected (N1)60 | Eff. Unit Wt. (psf) | Resist. CRR | rd Factor | Induced CSR | Liquefac. Safe.Fact. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.998 | 0.242 | -- |
| 2.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.993 | 0.241 | -- |
| 3.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.989 | 0.240 | -- |
| 4.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.984 | 0.239 | -- |
| 5.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 1.925 | 14.9 | 125.2 | 0.163 | 0.979 | 0.237 | -- |
| 6.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.740 | 20.5 | 127.7 | 0.224 | 0.975 | 0.236 | -- |
| 7.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.598 | 19.4 | 127.7 | 0.211 | 0.970 | 0.235 | -- |
| 8.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.486 | 18.5 | 127.7 | 0.201 | 0.966 | 0.234 | -- |
| 9.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.395 | 17.8 | 127.7 | 0.194 | 0.961 | 0.233 | -- |
| 10.0 | 127.7 | 1 | 12.0 | 10.0 | 1 | 51 | 68 | 1.319 | 27.5 | 65.3 | 0.335 | 0.957 | 0.238 | 1.41 |
| 11.0 | 127.7 | 1 | 12.0 | 10.0 | 1 | 51 | 68 | 1.254 | 26.5 | 65.3 | 0.312 | 0.952 | 0.248 | 1.26 |
| 12.0 | 127.7 | 1 | 12.0 | 10.0 | 1 | 51 | 68 | 1.197 | 25.6 | 65.3 | 0.296 | 0.947 | 0.257 | 1.15 |
| 13.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.147 | 12.8 | 70.1 | ~ | 0.943 | 0.265 | ~ |
| 14.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.102 | 12.3 | 70.1 | ~ | 0.938 | 0.272 | ~ |
| 15.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.062 | 11.8 | 70.1 | ~ | 0.934 | 0.278 | ~ |
| 16.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.026 | 11.4 | 70.1 | ~ | 0.929 | 0.283 | ~ |
| 17.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 0.993 | 11.1 | 70.1 | ~ | 0.925 | 0.288 | ~ |
| 18.0 | 131.8 | 1 | 27.0 | 17.5 | 1 | 60 | 94 | 0.964 | 45.4 | 69.4 | Infin. | 0.920 | 0.292 | Non-Liq. |
| 19.0 | 131.8 | 1 | 27.0 | 17.5 | 1 | 60 | 94 | 0.937 | 44.3 | 69.4 | Infin. | 0.915 | 0.295 | Non-Liq. |
| 20.0 | 131.8 | 1 | 27.0 | 17.5 | 1 | 60 | 94 | 0.912 | 43.3 | 69.4 | Infin. | 0.911 | 0.299 | Non-Liq. |
| 21.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.889 | 26.2 | 66.9 | 0.307 | 0.906 | 0.302 | 1.02 |
| 22.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.868 | 25.6 | 66.9 | 0.295 | 0.902 | 0.304 | 0.97 |
| 23.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.848 | 25.0 | 66.9 | 0.286 | 0.897 | 0.306 | 0.93 |
| 24.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.830 | 24.5 | 66.9 | 0.277 | 0.893 | 0.308 | 0.90 |
| 25.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.813 | 24.0 | 66.9 | 0.269 | 0.888 | 0.310 | 0.87 |
| 26.0 | 129.3 | 1 | 7.0 | 25.0 | 1 | 7 | 45 | 0.797 | 9.7 | 66.9 | 0.105 | 0.883 | 0.312 | 0.34 |
| 27.0 | 106.0 | 1 | 7.0 | 25.0 | 1 | 7 | 45 | 0.783 | 9.5 | 43.6 | 0.104 | 0.879 | 0.314 | 0.33 |
| 28.0 | 106.0 | 1 | 7.0 | 25.0 | 1 | 7 | 45 | 0.771 | 9.4 | 43.6 | 0.102 | 0.874 | 0.316 | 0.32 |
| 29.0 | 106.0 | 1 | 22.0 | 30.0 | 1 | 7 | 77 | 0.760 | 29.3 | 43.6 | 0.394 | 0.870 | 0.318 | 1.24 |
| 30.0 | 106.0 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.749 | 28.6 | 43.6 | 0.362 | 0.865 | 0.319 | 1.13 |
| 31.0 | 106.0 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.738 | 28.2 | 43.6 | 0.349 | 0.861 | 0.321 | 1.09 |
| 32.0 | 109.8 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.728 | 27.8 | 47.4 | 0.338 | 0.856 | 0.322 | 1.05 |
| 33.0 | 109.8 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.718 | 27.5 | 47.4 | 0.328 | 0.851 | 0.324 | 1.01 |
| 34.0 | 109.8 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.709 | 27.1 | 47.4 | 0.320 | 0.847 | 0.325 | 0.98 |
| 35.0 | 109.8 | 1 | 13.0 | 35.0 | 1 | 5 | 58 | 0.699 | 15.7 | 47.4 | 0.165 | 0.842 | 0.326 | 0.51 |
| 36.0 | 109.8 | 1 | 13.0 | 35.0 | 1 | 5 | 58 | 0.691 | 15.5 | 47.4 | 0.163 | 0.838 | 0.326 | 0.50 |
| 37.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.682 | 58.8 | 45.1 | Infin. | 0.833 | 0.327 | Non-Liq. |
| 38.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.674 | 58.1 | 45.1 | Infin. | 0.829 | 0.328 | Non-Liq. |
| 39.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.666 | 57.5 | 45.1 | Infin. | 0.824 | 0.328 | Non-Liq. |
| 40.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.659 | 56.8 | 45.1 | Infin. | 0.819 | 0.329 | Non-Liq. |
| 41.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.652 | 56.2 | 45.1 | Infin. | 0.815 | 0.329 | Non-Liq. |
| 42.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.645 | 55.6 | 45.1 | Infin. | 0.810 | 0.329 | Non-Liq. |
| 43.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.637 | 37.4 | 72.6 | Infin. | 0.806 | 0.329 | Non-Liq. |
| 44.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.629 | 36.9 | 72.6 | Infin. | 0.801 | 0.328 | Non-Liq. |
| 45.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.621 | 36.4 | 72.6 | Infin. | 0.797 | 0.327 | Non-Liq. |
| 46.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.613 | 36.0 | 72.6 | Infin. | 0.792 | 0.326 | Non-Liq. |
| 47.0 | 125.4 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.606 | 35.6 | 63.0 | Infin. | 0.787 | 0.325 | Non-Liq. |
| 48.0 | 125.4 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.600 | 35.2 | 63.0 | Infin. | 0.783 | 0.324 | Non-Liq. |
| 49.0 | 125.4 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.593 | 34.8 | 63.0 | Infin. | 0.778 | 0.323 | Non-Liq. |
| 50.0 | 125.4 | 1 | 28.0 | 50.0 | 1 | 34 | 78 | 0.589 | 35.3 | 63.0 | Infin. | 0.774 | 0.322 | Non-Liq. |
| 51.0 | 125.4 | 1 | 28.0 | 50.0 | 1 | 34 | 78 | 0.586 | 35.1 | 63.0 | Infin. | 0.769 | 0.321 | Non-Liq. |

GEOCON

Client : Warmington La Mirada
File No. : A9708-88-01
Boring : 1

# LIQUEFACTION SETTLEMENT ANALYSIS
# DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

| | |
|---|---|
| Earthquake Magnitude: | 6.65 |
| PGAM (g): | 0.757 |
| 2/3 PGAM (g): | 0.50 |
| Calculated Mag.Wtg.Factor: | 0.739 |
| Historic High Groundwater: | 10.0 |
| Groundwater @ Exploration: | 50.0 |

| DEPTH TO BASE | BLOW COUNT N | WET DENSITY (PCF) | TOTAL STRESS O (TSF) | EFFECT STRESS O' (TSF) | REL. DEN. Dr (%) | ADJUST BLOWS (N1)60 | Tav/σ'o | LIQUEFACTION SAFETY FACTOR | Volumetric Strain [e15} (%) | EQ. SETTLE. Pe (in.) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 6 | 125.18 | 0.031 | 0.031 | 53 | 16 | 0.328 | -- | 0.00 | 0.00 |
| 2.0 | 6 | 125.18 | 0.094 | 0.094 | 53 | 16 | 0.328 | -- | 0.00 | 0.00 |
| 3.0 | 6 | 125.18 | 0.156 | 0.156 | 53 | 16 | 0.328 | -- | 0.00 | 0.00 |
| 4.0 | 6 | 125.18 | 0.219 | 0.219 | 53 | 16 | 0.328 | -- | 0.00 | 0.00 |
| 5.0 | 6 | 125.18 | 0.282 | 0.282 | 53 | 15 | 0.328 | -- | 0.00 | 0.00 |
| 6.0 | 6 | 127.738 | 0.345 | 0.345 | 53 | 21 | 0.328 | -- | 0.00 | 0.00 |
| 7.0 | 6 | 127.738 | 0.409 | 0.409 | 53 | 19 | 0.328 | -- | 0.00 | 0.00 |
| 8.0 | 6 | 127.738 | 0.473 | 0.473 | 53 | 19 | 0.328 | -- | 0.00 | 0.00 |
| 9.0 | 6 | 127.738 | 0.536 | 0.536 | 53 | 18 | 0.328 | -- | 0.00 | 0.00 |
| 10.0 | 12 | 127.738 | 0.600 | 0.585 | 68 | 27 | 0.337 | 1.41 | 0.00 | 0.00 |
| 11.0 | 12 | 127.738 | 0.664 | 0.617 | 68 | 26 | 0.353 | 1.26 | 0.00 | 0.00 |
| 12.0 | 12 | 127.738 | 0.728 | 0.650 | 68 | 26 | 0.368 | 1.15 | 0.00 | 0.00 |
| 13.0 | 8 | 132.4605 | 0.793 | 0.684 | | 13 | 0.381 | ~ | 0.00 | 0.00 |
| 14.0 | 8 | 132.4605 | 0.859 | 0.719 | | 12 | 0.392 | ~ | 0.00 | 0.00 |
| 15.0 | 8 | 132.4605 | 0.926 | 0.754 | | 12 | 0.403 | ~ | 0.00 | 0.00 |
| 16.0 | 8 | 132.4605 | 0.992 | 0.789 | | 11 | 0.413 | ~ | 0.00 | 0.00 |
| 17.0 | 8 | 132.4605 | 1.058 | 0.824 | | 11 | 0.421 | ~ | 0.00 | 0.00 |
| 18.0 | 27 | 131.7895 | 1.124 | 0.859 | 94 | 45 | 0.430 | Non-Liq. | 0.00 | 0.00 |
| 19.0 | 27 | 131.7895 | 1.190 | 0.894 | 94 | 44 | 0.437 | Non-Liq. | 0.00 | 0.00 |
| 20.0 | 27 | 131.7895 | 1.256 | 0.928 | 94 | 43 | 0.444 | Non-Liq. | 0.00 | 0.00 |
| 21.0 | 19 | 129.2522 | 1.321 | 0.962 | 77 | 26 | 0.451 | 1.02 | 1.10 | 0.13 |
| 22.0 | 19 | 129.2522 | 1.386 | 0.996 | 77 | 26 | 0.457 | 0.97 | 1.10 | 0.13 |
| 23.0 | 19 | 129.2522 | 1.450 | 1.029 | 77 | 25 | 0.463 | 0.93 | 1.10 | 0.13 |
| 24.0 | 19 | 129.2522 | 1.515 | 1.063 | 77 | 24 | 0.468 | 0.90 | 1.30 | 0.16 |
| 25.0 | 19 | 129.2522 | 1.580 | 1.096 | 77 | 24 | 0.473 | 0.87 | 1.30 | 0.16 |
| 26.0 | 7 | 129.2522 | 1.644 | 1.130 | 45 | 10 | 0.478 | 0.34 | 2.70 | 0.32 |
| 27.0 | 7 | 106.0324 | 1.703 | 1.157 | 45 | 9 | 0.483 | 0.33 | 2.70 | 0.32 |
| 28.0 | 7 | 106.0324 | 1.756 | 1.179 | 45 | 9 | 0.489 | 0.32 | 2.70 | 0.32 |
| 29.0 | 22 | 106.0324 | 1.809 | 1.201 | 77 | 29 | 0.494 | 1.24 | 0.00 | 0.00 |
| 30.0 | 22 | 106.0324 | 1.862 | 1.223 | 77 | 29 | 0.500 | 1.13 | 0.00 | 0.00 |
| 31.0 | 22 | 106.0324 | 1.915 | 1.244 | 77 | 28 | 0.505 | 1.09 | 0.75 | 0.09 |
| 32.0 | 22 | 109.8303 | 1.969 | 1.267 | 77 | 28 | 0.510 | 1.05 | 0.75 | 0.09 |
| 33.0 | 22 | 109.8303 | 2.024 | 1.291 | 77 | 27 | 0.515 | 1.01 | 1.10 | 0.13 |
| 34.0 | 22 | 109.8303 | 2.079 | 1.315 | 77 | 27 | 0.519 | 0.98 | 1.10 | 0.13 |
| 35.0 | 13 | 109.8303 | 2.134 | 1.338 | 58 | 16 | 0.523 | 0.51 | 1.70 | 0.20 |
| 36.0 | 13 | 109.8303 | 2.189 | 1.362 | 58 | 15 | 0.527 | 0.50 | 1.70 | 0.20 |
| 37.0 | 50 | 107.536 | 2.243 | 1.385 | 111 | 59 | 0.531 | Non-Liq. | 0.00 | 0.00 |
| 38.0 | 50 | 107.536 | 2.297 | 1.408 | 111 | 58 | 0.536 | Non-Liq. | 0.00 | 0.00 |
| 39.0 | 50 | 107.536 | 2.351 | 1.430 | 111 | 57 | 0.539 | Non-Liq. | 0.00 | 0.00 |
| 40.0 | 50 | 107.536 | 2.404 | 1.453 | 111 | 57 | 0.543 | Non-Liq. | 0.00 | 0.00 |
| 41.0 | 50 | 107.536 | 2.458 | 1.475 | 111 | 56 | 0.547 | Non-Liq. | 0.00 | 0.00 |
| 42.0 | 50 | 107.536 | 2.512 | 1.498 | 111 | 56 | 0.550 | Non-Liq. | 0.00 | 0.00 |
| 43.0 | 34 | 135.0228 | 2.573 | 1.527 | 89 | 37 | 0.553 | Non-Liq. | 0.00 | 0.00 |
| 44.0 | 34 | 135.0228 | 2.640 | 1.564 | 89 | 37 | 0.554 | Non-Liq. | 0.00 | 0.00 |
| 45.0 | 34 | 135.0228 | 2.708 | 1.600 | 89 | 36 | 0.555 | Non-Liq. | 0.00 | 0.00 |
| 46.0 | 34 | 135.0228 | 2.775 | 1.636 | 89 | 36 | 0.557 | Non-Liq. | 0.00 | 0.00 |
| 47.0 | 34 | 125.4391 | 2.840 | 1.670 | 89 | 36 | 0.558 | Non-Liq. | 0.00 | 0.00 |
| 48.0 | 34 | 125.4391 | 2.903 | 1.702 | 89 | 35 | 0.560 | Non-Liq. | 0.00 | 0.00 |
| 49.0 | 34 | 125.4391 | 2.966 | 1.733 | 89 | 35 | 0.562 | Non-Liq. | 0.00 | 0.00 |
| 50.0 | 28 | 125.4391 | 3.028 | 1.765 | 78 | 35 | 0.563 | Non-Liq. | 0.00 | 0.00 |
| 51.0 | 28 | 125.4391 | 3.091 | 1.796 | 78 | 35 | 0.565 | Non-Liq. | 0.00 | 0.00 |

TOTAL SETTLEMENT = 2.5 INCHES

Figure 6

GEOCON

Client : Warmington La Mirada
File No. : A9708-88-01
Boring : 1

# EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
# MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

| | |
|---|---|
| Earthquake Magnitude: | 6.65 |
| Peak Horiz. Acceleration $PGA_M$ (g): | 0.757 |
| Calculated Mag.Wtg.Factor: | 0.739 |
| Historic High Groundwater: | 10.0 |
| Groundwater Depth During Exploration: | 50.0 |

ENERGY & ROD CORRECTIONS:

| | |
|---|---|
| Energy Correction (CE) for N60: | 1.25 |
| Rod Len.Corr.(CR)(0-no or 1-yes): | 1.0 |
| Bore Dia. Corr. (CB): | 1.15 |
| Sampler Corr. (CS): | 1.20 |
| Use Ksigma (0 or 1): | 1.0 |

LIQUEFACTION CALCULATIONS:

| | |
|---|---|
| Unit Wt. Water (pcf): | 62.4 |

| Depth to Base (ft) | Total Unit Wt. (pcf) | Water (0 or 1) | FIELD SPT (N) | Depth of SPT (ft) | Liq.Sus. (0 or 1) | -200 (%) | Est. Dr (%) | CN Factor | Corrected (N1)60 | Eff. Unit Wt. (psf) | Resist. CRR | rd Factor | Induced CSR | Liquefac. Safe.Fact. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.998 | 0.363 | -- |
| 2.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.993 | 0.361 | -- |
| 3.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.989 | 0.359 | -- |
| 4.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 2.000 | 15.5 | 125.2 | 0.169 | 0.984 | 0.358 | -- |
| 5.0 | 125.2 | 0 | 6.0 | 5.0 | 1 | | 53 | 1.925 | 14.9 | 125.2 | 0.163 | 0.979 | 0.356 | -- |
| 6.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.740 | 20.5 | 127.7 | 0.224 | 0.975 | 0.354 | -- |
| 7.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.598 | 19.4 | 127.7 | 0.211 | 0.970 | 0.353 | -- |
| 8.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.486 | 18.5 | 127.7 | 0.201 | 0.966 | 0.351 | -- |
| 9.0 | 127.7 | 0 | 6.0 | 5.0 | 1 | 61 | 53 | 1.395 | 17.8 | 127.7 | 0.194 | 0.961 | 0.349 | -- |
| 10.0 | 127.7 | 1 | 12.0 | 10.0 | 1 | 51 | 68 | 1.319 | 27.5 | 65.3 | 0.335 | 0.957 | 0.357 | 0.94 |
| 11.0 | 127.7 | 1 | 12.0 | 10.0 | 1 | 51 | 68 | 1.254 | 26.5 | 65.3 | 0.312 | 0.952 | 0.372 | 0.84 |
| 12.0 | 127.7 | 1 | 12.0 | 10.0 | 1 | 51 | 68 | 1.197 | 25.6 | 65.3 | 0.296 | 0.947 | 0.386 | 0.77 |
| 13.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.147 | 12.8 | 70.1 | ~ | 0.943 | 0.397 | ~ |
| 14.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.102 | 12.3 | 70.1 | ~ | 0.938 | 0.408 | ~ |
| 15.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.062 | 11.8 | 70.1 | ~ | 0.934 | 0.417 | ~ |
| 16.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 1.026 | 11.4 | 70.1 | ~ | 0.929 | 0.424 | ~ |
| 17.0 | 132.5 | 1 | 8.0 | 15.0 | 0 | | | 0.993 | 11.1 | 70.1 | ~ | 0.925 | 0.431 | ~ |
| 18.0 | 131.8 | 1 | 27.0 | 17.5 | 1 | 60 | 94 | 0.964 | 45.4 | 69.4 | Infin. | 0.920 | 0.438 | Non-Liq. |
| 19.0 | 131.8 | 1 | 27.0 | 17.5 | 1 | 60 | 94 | 0.937 | 44.3 | 69.4 | Infin. | 0.915 | 0.443 | Non-Liq. |
| 20.0 | 131.8 | 1 | 27.0 | 17.5 | 1 | 60 | 94 | 0.912 | 43.3 | 69.4 | Infin. | 0.911 | 0.448 | Non-Liq. |
| 21.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.889 | 26.2 | 66.9 | 0.307 | 0.906 | 0.452 | 0.68 |
| 22.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.868 | 25.6 | 66.9 | 0.295 | 0.902 | 0.456 | 0.65 |
| 23.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.848 | 25.0 | 66.9 | 0.286 | 0.897 | 0.459 | 0.62 |
| 24.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.830 | 24.5 | 66.9 | 0.277 | 0.893 | 0.462 | 0.60 |
| 25.0 | 129.3 | 1 | 19.0 | 20.0 | 1 | 6 | 77 | 0.813 | 24.0 | 66.9 | 0.269 | 0.888 | 0.465 | 0.58 |
| 26.0 | 129.3 | 1 | 7.0 | 25.0 | 1 | 7 | 45 | 0.797 | 9.7 | 66.9 | 0.105 | 0.883 | 0.467 | 0.23 |
| 27.0 | 106.0 | 1 | 7.0 | 25.0 | 1 | 7 | 45 | 0.783 | 9.5 | 43.6 | 0.104 | 0.879 | 0.470 | 0.22 |
| 28.0 | 106.0 | 1 | 7.0 | 25.0 | 1 | 7 | 45 | 0.771 | 9.4 | 43.6 | 0.102 | 0.874 | 0.473 | 0.22 |
| 29.0 | 106.0 | 1 | 22.0 | 30.0 | 1 | 7 | 77 | 0.760 | 29.3 | 43.6 | 0.394 | 0.870 | 0.476 | 0.83 |
| 30.0 | 106.0 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.749 | 28.6 | 43.6 | 0.362 | 0.865 | 0.479 | 0.76 |
| 31.0 | 106.0 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.738 | 28.2 | 43.6 | 0.349 | 0.861 | 0.481 | 0.73 |
| 32.0 | 109.8 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.728 | 27.8 | 47.4 | 0.338 | 0.856 | 0.483 | 0.70 |
| 33.0 | 109.8 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.718 | 27.5 | 47.4 | 0.328 | 0.851 | 0.485 | 0.68 |
| 34.0 | 109.8 | 1 | 22.0 | 30.0 | 1 | 6 | 77 | 0.709 | 27.1 | 47.4 | 0.320 | 0.847 | 0.487 | 0.66 |
| 35.0 | 109.8 | 1 | 13.0 | 35.0 | 1 | 5 | 58 | 0.699 | 15.7 | 47.4 | 0.165 | 0.842 | 0.488 | 0.34 |
| 36.0 | 109.8 | 1 | 13.0 | 35.0 | 1 | 5 | 58 | 0.691 | 15.5 | 47.4 | 0.163 | 0.838 | 0.489 | 0.33 |
| 37.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.682 | 58.8 | 45.1 | Infin. | 0.833 | 0.490 | Non-Liq. |
| 38.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.674 | 58.1 | 45.1 | Infin. | 0.829 | 0.491 | Non-Liq. |
| 39.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.666 | 57.5 | 45.1 | Infin. | 0.824 | 0.492 | Non-Liq. |
| 40.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.659 | 56.8 | 45.1 | Infin. | 0.819 | 0.493 | Non-Liq. |
| 41.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.652 | 56.2 | 45.1 | Infin. | 0.815 | 0.493 | Non-Liq. |
| 42.0 | 107.5 | 1 | 50.0 | 40.0 | 1 | 5 | 111 | 0.645 | 55.6 | 45.1 | Infin. | 0.810 | 0.494 | Non-Liq. |
| 43.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.637 | 37.4 | 72.6 | Infin. | 0.806 | 0.493 | Non-Liq. |
| 44.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.629 | 36.9 | 72.6 | Infin. | 0.801 | 0.492 | Non-Liq. |
| 45.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.621 | 36.4 | 72.6 | Infin. | 0.797 | 0.490 | Non-Liq. |
| 46.0 | 135.0 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.613 | 36.0 | 72.6 | Infin. | 0.792 | 0.488 | Non-Liq. |
| 47.0 | 125.4 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.606 | 35.6 | 63.0 | Infin. | 0.787 | 0.487 | Non-Liq. |
| 48.0 | 125.4 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.600 | 35.2 | 63.0 | Infin. | 0.783 | 0.485 | Non-Liq. |
| 49.0 | 125.4 | 1 | 34.0 | 45.0 | 1 | | 89 | 0.593 | 34.8 | 63.0 | Infin. | 0.778 | 0.484 | Non-Liq. |
| 50.0 | 125.4 | 1 | 28.0 | 50.0 | 1 | 34 | 78 | 0.589 | 35.3 | 63.0 | Infin. | 0.774 | 0.482 | Non-Liq. |
| 51.0 | 125.4 | 1 | 28.0 | 50.0 | 1 | 34 | 78 | 0.586 | 35.1 | 63.0 | Infin. | 0.769 | 0.481 | Non-Liq. |

Client : Warmington La Mirada
File No. : A9708-88-01
Boring : 1

# LIQUEFACTION SETTLEMENT ANALYSIS
# MAXIMUM CONSIDERED EARTHQUAKE

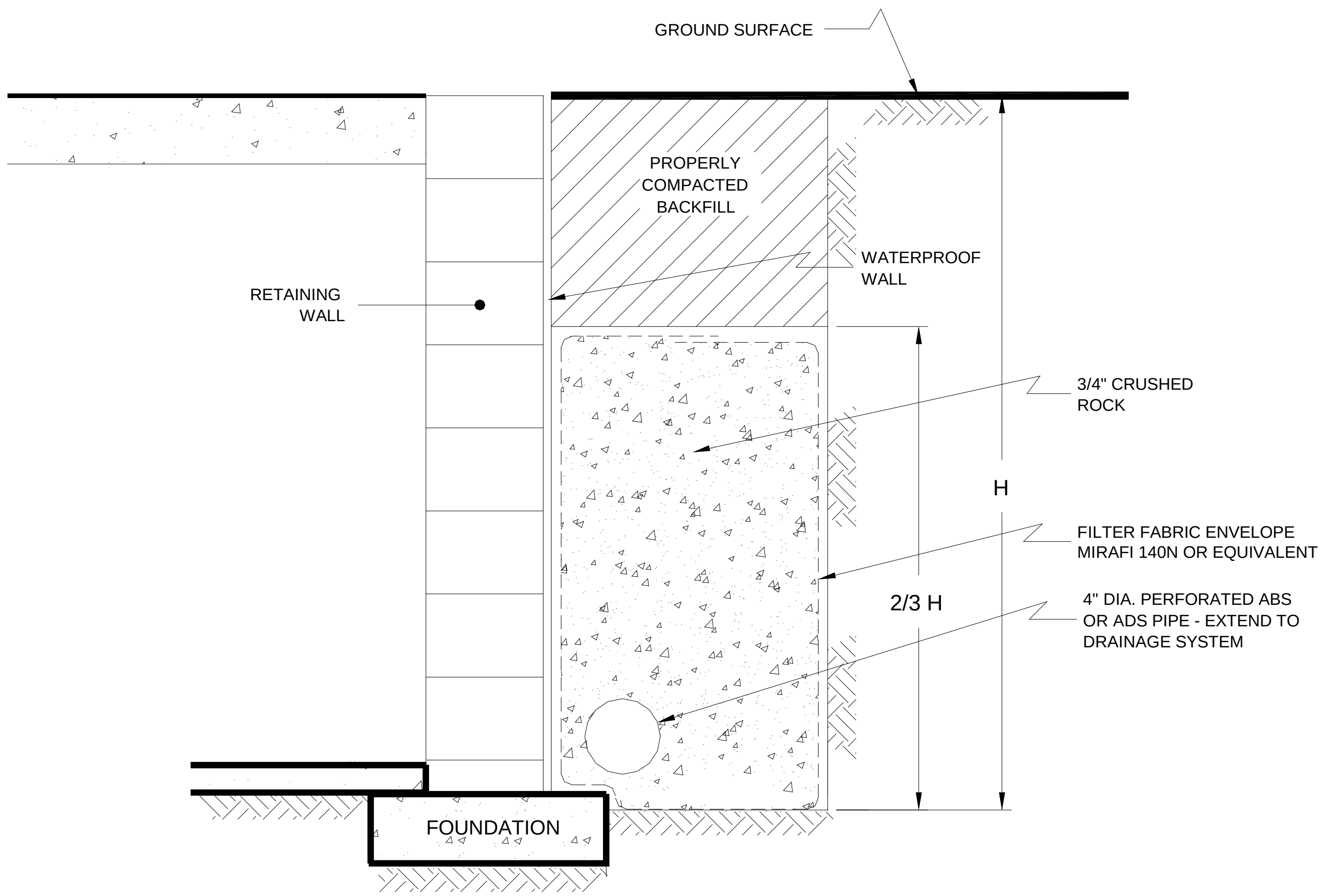
(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

| | |
|---|---|
| Earthquake Magnitude: | 6.65 |
| $PGA_M$ (g): | 0.757 |
| Calculated Mag.Wtg.Factor: | 0.739 |
| Historic High Groundwater: | 10.0 |
| Groundwater @ Exploration: | 50.0 |

| DEPTH TO BASE | BLOW COUNT N | WET DENSITY (PCF) | TOTAL STRESS O (TSF) | EFFECT STRESS O' (TSF) | REL. DEN. Dr (%) | ADJUST BLOWS (N1)60 | Tav/σ'o | LIQUEFACTION SAFETY FACTOR | Volumetric Strain [$e_{15}$} (%) | EQ. SETTLE. Pe (in.) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 6 | 125.18 | 0.031 | 0.031 | 53 | 16 | 0.492 | -- | 0.00 | 0.00 |
| 2.0 | 6 | 125.18 | 0.094 | 0.094 | 53 | 16 | 0.492 | -- | 0.00 | 0.00 |
| 3.0 | 6 | 125.18 | 0.156 | 0.156 | 53 | 16 | 0.492 | -- | 0.00 | 0.00 |
| 4.0 | 6 | 125.18 | 0.219 | 0.219 | 53 | 16 | 0.492 | -- | 0.00 | 0.00 |
| 5.0 | 6 | 125.18 | 0.282 | 0.282 | 53 | 15 | 0.492 | -- | 0.00 | 0.00 |
| 6.0 | 6 | 127.738 | 0.345 | 0.345 | 53 | 21 | 0.492 | -- | 0.00 | 0.00 |
| 7.0 | 6 | 127.738 | 0.409 | 0.409 | 53 | 19 | 0.492 | -- | 0.00 | 0.00 |
| 8.0 | 6 | 127.738 | 0.473 | 0.473 | 53 | 19 | 0.492 | -- | 0.00 | 0.00 |
| 9.0 | 6 | 127.738 | 0.536 | 0.536 | 53 | 18 | 0.492 | -- | 0.00 | 0.00 |
| 10.0 | 12 | 127.738 | 0.600 | 0.585 | 68 | 27 | 0.505 | 0.94 | 1.10 | 0.13 |
| 11.0 | 12 | 127.738 | 0.664 | 0.617 | 68 | 26 | 0.529 | 0.84 | 1.10 | 0.13 |
| 12.0 | 12 | 127.738 | 0.728 | 0.650 | 68 | 26 | 0.551 | 0.77 | 1.10 | 0.13 |
| 13.0 | 8 | 132.4605 | 0.793 | 0.684 | | 13 | 0.571 | ~ | 0.00 | 0.00 |
| 14.0 | 8 | 132.4605 | 0.859 | 0.719 | | 12 | 0.588 | ~ | 0.00 | 0.00 |
| 15.0 | 8 | 132.4605 | 0.926 | 0.754 | | 12 | 0.604 | ~ | 0.00 | 0.00 |
| 16.0 | 8 | 132.4605 | 0.992 | 0.789 | | 11 | 0.619 | ~ | 0.00 | 0.00 |
| 17.0 | 8 | 132.4605 | 1.058 | 0.824 | | 11 | 0.632 | ~ | 0.00 | 0.00 |
| 18.0 | 27 | 131.7895 | 1.124 | 0.859 | 94 | 45 | 0.644 | Non-Liq. | 0.00 | 0.00 |
| 19.0 | 27 | 131.7895 | 1.190 | 0.894 | 94 | 44 | 0.655 | Non-Liq. | 0.00 | 0.00 |
| 20.0 | 27 | 131.7895 | 1.256 | 0.928 | 94 | 43 | 0.666 | Non-Liq. | 0.00 | 0.00 |
| 21.0 | 19 | 129.2522 | 1.321 | 0.962 | 77 | 26 | 0.675 | 0.68 | 1.10 | 0.13 |
| 22.0 | 19 | 129.2522 | 1.386 | 0.996 | 77 | 26 | 0.685 | 0.65 | 1.10 | 0.13 |
| 23.0 | 19 | 129.2522 | 1.450 | 1.029 | 77 | 25 | 0.693 | 0.62 | 1.10 | 0.13 |
| 24.0 | 19 | 129.2522 | 1.515 | 1.063 | 77 | 24 | 0.702 | 0.60 | 1.30 | 0.16 |
| 25.0 | 19 | 129.2522 | 1.580 | 1.096 | 77 | 24 | 0.709 | 0.58 | 1.30 | 0.16 |
| 26.0 | 7 | 129.2522 | 1.644 | 1.130 | 45 | 10 | 0.716 | 0.23 | 2.70 | 0.32 |
| 27.0 | 7 | 106.0324 | 1.703 | 1.157 | 45 | 9 | 0.724 | 0.22 | 2.70 | 0.32 |
| 28.0 | 7 | 106.0324 | 1.756 | 1.179 | 45 | 9 | 0.733 | 0.22 | 2.70 | 0.32 |
| 29.0 | 22 | 106.0324 | 1.809 | 1.201 | 77 | 29 | 0.741 | 0.83 | 0.75 | 0.09 |
| 30.0 | 22 | 106.0324 | 1.862 | 1.223 | 77 | 29 | 0.749 | 0.76 | 0.75 | 0.09 |
| 31.0 | 22 | 106.0324 | 1.915 | 1.244 | 77 | 28 | 0.757 | 0.73 | 0.75 | 0.09 |
| 32.0 | 22 | 109.8303 | 1.969 | 1.267 | 77 | 28 | 0.765 | 0.70 | 0.75 | 0.09 |
| 33.0 | 22 | 109.8303 | 2.024 | 1.291 | 77 | 27 | 0.772 | 0.68 | 1.10 | 0.13 |
| 34.0 | 22 | 109.8303 | 2.079 | 1.315 | 77 | 27 | 0.778 | 0.66 | 1.10 | 0.13 |
| 35.0 | 13 | 109.8303 | 2.134 | 1.338 | 58 | 16 | 0.785 | 0.34 | 1.70 | 0.20 |
| 36.0 | 13 | 109.8303 | 2.189 | 1.362 | 58 | 15 | 0.791 | 0.33 | 1.70 | 0.20 |
| 37.0 | 50 | 107.536 | 2.243 | 1.385 | 111 | 59 | 0.797 | Non-Liq. | 0.00 | 0.00 |
| 38.0 | 50 | 107.536 | 2.297 | 1.408 | 111 | 58 | 0.803 | Non-Liq. | 0.00 | 0.00 |
| 39.0 | 50 | 107.536 | 2.351 | 1.430 | 111 | 57 | 0.809 | Non-Liq. | 0.00 | 0.00 |
| 40.0 | 50 | 107.536 | 2.404 | 1.453 | 111 | 57 | 0.814 | Non-Liq. | 0.00 | 0.00 |
| 41.0 | 50 | 107.536 | 2.458 | 1.475 | 111 | 56 | 0.820 | Non-Liq. | 0.00 | 0.00 |
| 42.0 | 50 | 107.536 | 2.512 | 1.498 | 111 | 56 | 0.825 | Non-Liq. | 0.00 | 0.00 |
| 43.0 | 34 | 135.0228 | 2.573 | 1.527 | 89 | 37 | 0.829 | Non-Liq. | 0.00 | 0.00 |
| 44.0 | 34 | 135.0228 | 2.640 | 1.564 | 89 | 37 | 0.831 | Non-Liq. | 0.00 | 0.00 |
| 45.0 | 34 | 135.0228 | 2.708 | 1.600 | 89 | 36 | 0.833 | Non-Liq. | 0.00 | 0.00 |
| 46.0 | 34 | 135.0228 | 2.775 | 1.636 | 89 | 36 | 0.834 | Non-Liq. | 0.00 | 0.00 |
| 47.0 | 34 | 125.4391 | 2.840 | 1.670 | 89 | 36 | 0.837 | Non-Liq. | 0.00 | 0.00 |
| 48.0 | 34 | 125.4391 | 2.903 | 1.702 | 89 | 35 | 0.839 | Non-Liq. | 0.00 | 0.00 |
| 49.0 | 34 | 125.4391 | 2.966 | 1.733 | 89 | 35 | 0.842 | Non-Liq. | 0.00 | 0.00 |
| 50.0 | 28 | 125.4391 | 3.028 | 1.765 | 78 | 35 | 0.844 | Non-Liq. | 0.00 | 0.00 |
| 51.0 | 28 | 125.4391 | 3.091 | 1.796 | 78 | 35 | 0.847 | Non-Liq. | 0.00 | 0.00 |

TOTAL SETTLEMENT = 3.1 INCHES




NO SCALE

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BLVD. - SUITE J - IRVINE, CA 92620

PHONE (949) 491-6570

# RETAINING WALL DRAIN DETAIL

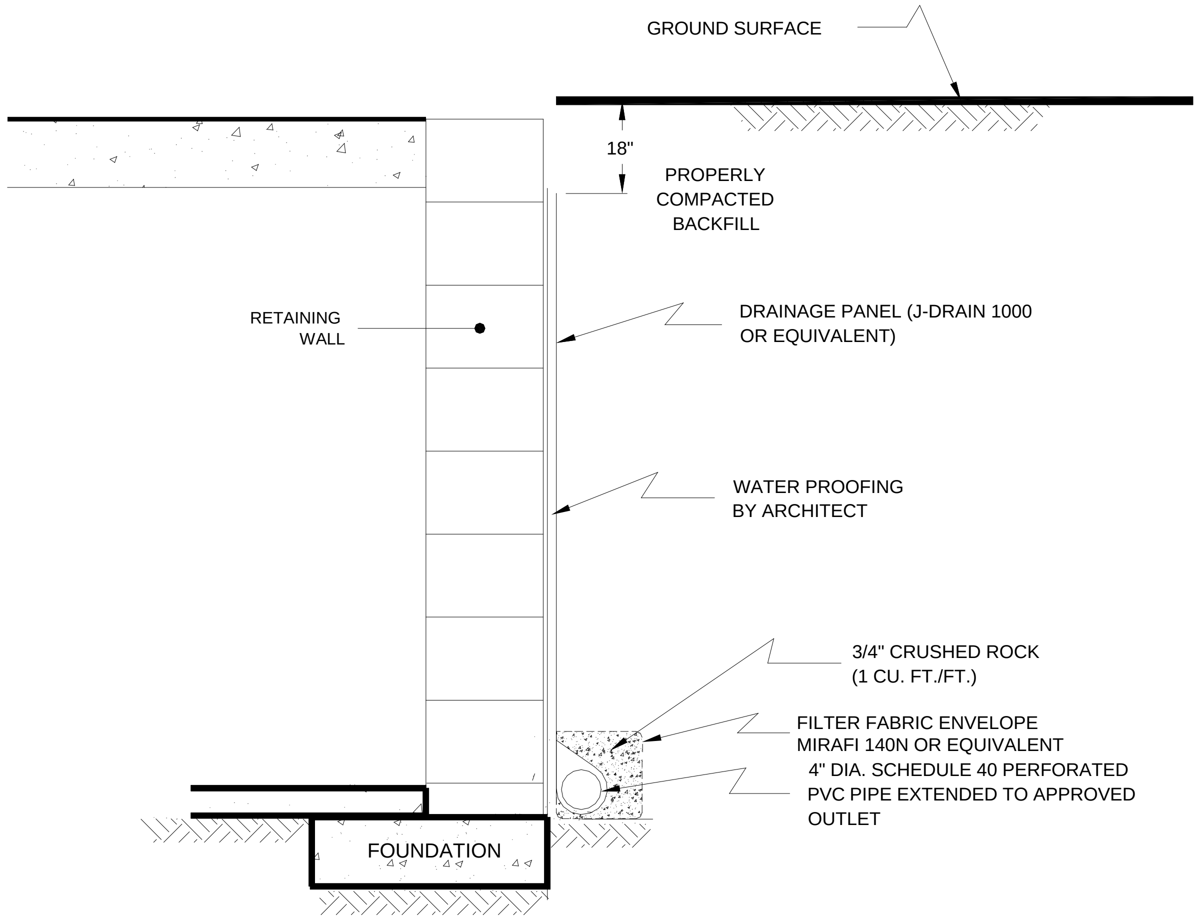
## RESIDENTIAL DEVELOPMENT

WARMINGTON RESIDENTIAL CALIFORNIA, INC.

12841 VALLEY VIEW AVENUE

LA MIRADA, CALIFORNIA

| | | | | |
|---|---|---|---|---|
| DRAFTED BY: PZ | CHECKED BY: JTA | JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. 9 |




NO SCALE

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BLVD. - SUITE J - IRVINE, CA 92620
PHONE (949) 491-6570

# RETAINING WALL DRAIN DETAIL

RESIDENTIAL DEVELOPMENT
WARMINGTON RESIDENTIAL CALIFORNIA, INC.
12841 VALLEY VIEW AVENUE
LA MIRADA, CALIFORNIA

| | | | | |
|---|---|---|---|---|
| DRAFTED BY: PZ | CHECKED BY: JTA | JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. 10 |

## BORING PERCOLATION TEST FIELD LOG

| | | | |
|---|---|---|---|
| **Date:** | 12/21/2017 | **Boring/Test Number:** | B4 |
| **Project Number:** | A9708-88-01 | **Diameter of Boring:** | 8 inches |
| **Project Location:** | Warmington La Mirada | **Diameter of Casing:** | 2 inches |
| **Earth Description:** | SM | **Depth of Boring:** | 20 feet |
| **Tested By:** | PZ | **Depth to Invert of BMP:** | 15 feet |
| **Liquid Description:** | Clear Clean Tap Water | **Depth to Water Table:** | 50 feet |
| **Measurement Method:** | Sounder | **Depth to Initial Water Depth ($d_1$):** | 180 inches |
| **Start Time for Pre-Soak:** | 7:45 AM | **Water Remaining in Boring (Y/N):** | N |
| **Start Time for Standard:** | 8:45 AM | **Standard Time Interval Between Readings:** | 30 min |

| Reading Number | Time Start (hh:mm) | Time End (hh:mm) | Elapsed Time Δtime (min) | Water Drop During Standard Time Interval, Δd (in) | Soil Description Notes Comments |
|---|---|---|---|---|---|
| 1 | 8:50 AM | 9:20 AM | 30 | 58.2 | |
| 2 | 9:22 AM | 9:52 AM | 30 | 57.6 | |
| 3 | 9:56 AM | 10:26 AM | 30 | 54.6 | |
| 4 | 10:30 AM | 11:00 AM | 30 | 54.0 | |
| 5 | 11:05 AM | 11:35 AM | 30 | 54.0 | |
| 6 | 11:40 AM | 12:10 PM | 30 | 53.4 | Stabilized Readings |
| 7 | 12:16 PM | 12:46 PM | 30 | 54.0 | Achieved with Readings |
| 8 | 12:52 PM | 1:22 PM | 30 | 54.0 | 6, 7, and 8 |

## MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches

Test Section Height, h: 60.0 inches

$$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$$

A = **1558** $in^2$

$$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$$

$$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$$

| | | | | | |
|---|---|---|---|---|---|
| Reading 6 | V = **2684** $in^3$ | Percolation Rate = | **3.45** | inches/hour |
| Reading 7 | V = **2714** $in^3$ | Percolation Rate = | **3.48** | inches/hour |
| Reading 8 | V = **2714** $in^3$ | Percolation Rate = | **3.48** | inches/hour |

Measured Percolation Rate = **3.47** inches/hour

**Reduction Factors**

Boring Percolation Test, $RF_t$ = 2

Site Variability, $RF_v$ = 1

Long Term Siltation, $RF_s$ = 1

$$Total\ Reduction\ Factor, RF = RF_t + RF_v + RF_s$$

Total Reduction Factor = 4

**Design Infiltration Rate**

$$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ / RF$$

Design Infiltration Rate = **0.87** inches/hour

**FIGURE 11**

# APPENDIX A

## APPENDIX A

## FIELD INVESTIGATION

The site was explored on December 20, 2017, by excavating four 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths between approximately 20 and 51 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^3/_8$-inch diameter brass sampler rings to facilitate soil removal and testing. Standard Penetration Testing was performed in boring B1 and bulk samples were also obtained..

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A4. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are shown on Figure 2.

PROJECT NO. A9708-88-01

## BORING 1

ELEV. (MSL.) -- DATE COMPLETED 12/20/17

EQUIPMENT HOLLOW STEM AUGER BY: PZ

| DEPTH IN FEET | SAMPLE NO. | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT*) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|
| 0 | BULK 0-5' | | **ARTIFICIAL FILL** Sandy Silt, firm, slightly moist, dark brown, fine-grained. | | | |
| 2 | B1@2.5' | CL | **ALLUVIUM** Clay, soft, slightly moist, dark brown. - dark gray | 10 | 110.0 | 13.8 |
| 4 | | | | | | |
| 6 | B1@5' | CL-ML | Sandy Silty Clay, soft, slightly moist, dark brown, fine-grained. | 6 | -- | 13.8 |
| 8 | B1@7.5' | | - firm, increase in sand content | 18 | 115.6 | 10.5 |
| 10 | B1@10' | ML | Sandy Silt, firm, slightly moist, brown, fine-grained. | 12 | -- | 11.9 |
| 12 | B1@12.5' | CL | Clay with Sand, stiff, slightly moist, brown, fine-grained. | 23 | 116.5 | 13.7 |
| 14 | | | | | | |
| 16 | B1@15' | | - soft, increase in sand content, trace medium-grained | 8 | -- | 14.6 |
| 18 | B1@17.5' | CL | Sandy Clay, hard, slightly moist, yellowish brown, fine-grained. | 49 | 114.5 | 15.1 |
| 20 | B1@20' | SP-SM | Sand with Silt, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained. | 19 | -- | 4.3 |
| 22 | B1@22.5' | | - fine- to medium-grained | 46 | 119.9 | 7.8 |
| 24 | | | | | | |
| 26 | B1@25' | | - loose | 7 | -- | 9.0 |
| 28 | B1@27.5' | | - fine-grained | 18 | 100.6 | 5.4 |
| 30 | B1@30' | | - medium dense | 22 | -- | 5.5 |
| 32 | B1@32.5' | | - dense, light brown | 72 | 104.9 | 4.7 |
| 34 | | | | | | |

A9708-88-01 BORING LOG.GPJ

**Figure A1,**
**Log of Boring 1, Page 1 of 2**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9708-88-01

# BORING 1

ELEV. (MSL.) -- DATE COMPLETED 12/20/17

EQUIPMENT HOLLOW STEM AUGER BY: PZ

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT*) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 36 | B1@35' | | | SP-SM | - medium dense, decrease in silt content | 13 | -- | 3.9 |
| 38 | B1@37.5' | | | | - very dense | 50 (6") | 104.0 | 3.4 |
| 40 | B1@40' | | | | | 50 | -- | 2.9 |
| 42 | B1@42.5' | | | ML | Sandy Silt, hard, slightly moist, reddish brown, fine-grained. | 51 | 116.6 | 15.8 |
| 44, 46 | B1@45' | | | | - increase in sand content | 34 | -- | 16.6 |
| 48 | B1@47.5' | | | | - yellowish brown | 50 (5") | 111.7 | 12.3 |
| 50 | B1@50' | | | SM | Silty Sand, medium dense, slightly moist, yellowish brown, fine-grained. | 28 | -- | 8.4 |

Total depth of Boring: 51 feet.
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with cement bentonite grout.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9708-88-01 BORING LOG.GPJ

**Figure A1,**
**Log of Boring 1, Page 2 of 2**

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. A9708-88-01

# BORING 2

ELEV. (MSL.) -- DATE COMPLETED 12/20/17

EQUIPMENT HOLLOW STEM AUGER BY: PZ

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT*) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ARTIFICIAL FILL**<br>Sandy Silt, firm, slightly moist, brown to yellowish brown, fine-grained. | | | |
| 2 | B2@2' | | | ML | **ALLUVIUM**<br>Sandy Silt, firm, slightly moist, reddish brown, fine-grained. | 14 | 118.8 | 11.0 |
| 5 | B2@5' | | | | - stiff, trace medium-grained | 31 | 128.3 | 12.0 |
| 7 | B2@7' | | | | | 32 | 113.3 | 14.0 |
| 9 | | | | | - firm, yellowish brown, increase in sand content | | | |
| 10 | B2@10' | | | | | 20 | 117.9 | 11.1 |
| 12 | B2@12' | | | SM | Silty Sand, loose, slightly moist, yellowish brown, fine- to medium-grained. | 11 | 107.5 | 6.1 |
| 15 | B2@15' | | | | - decrease in silt content | 16 | 103.7 | 5.8 |
| 19.5 | | | | ML | Sandy Silt, stiff, slightly moist, yellowish brown, fine-grained. | | | |
| 20 | B2@20' | | | | | 32 | 99.7 | 23.7 |
| 25 | B2@25' | | | | - hard<br>- increase in sand content | 50 (3") | 100.9 | 21.8 |
| 26 | | | | SP-SM | Sand with Silt, very dense, slightly moist, light brown, fine-grained. | | | |
| 30 | B2@30' | | | | | 50 (6") | 100.7 | 5.5 |

Total depth of Boring: 30.5 feet.
Fill to 2 feet. No groundwater encountered.
Backfilled with soil cuttings and tamped. Asphalt patched.
*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9708-88-01 BORING LOG.GPJ

**Figure A2,**
**Log of Boring 2, Page 1 of 1**

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE



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## BORING 3

ELEV. (MSL.) -- DATE COMPLETED 12/20/17

EQUIPMENT HOLLOW STEM AUGER BY: PZ

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT*) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ARTIFICIAL FILL**<br>Sandy Silt, firm, slightly moist, brown to dark brown, fine-grained, trace brick fragements. | | | |
| 2 | B3@2' | | | | | 17 | 120.6 | 10.2 |
| 4 | | | | | | | | |
| 5 | B3@5' | | | ML | **ALLUVIUM**<br>Sandy Silt, firm, slightly moist, reddish brown, fine-grained, trace medium-grained. | 12 | 115.0 | 6.8 |
| 6 | | | | | | | | |
| 7 | B3@7' | | | | | 23 | 124.1 | 7.9 |
| 8 | | | | | | | | |
| 10 | B3@10' | | | | | 17 | 117.1 | 7.3 |
| 12 | B3@12' | | | | | 16 | 111.5 | 7.5 |
| 14 | | | | SM | Silty Sand, medium dense, slightly moist, reddish brown, fine- to medium-grained. | | | |
| 15 | B3@15' | | | | | 28 | 110.3 | 5.1 |
| 16 | | | | | - yellowish brown, decrease in silt content | | | |
| 17 | B3@17' | | | | | 25 | 109.3 | 6.7 |
| 18 | | | | | | | | |
| 19 | | | | SP | Sand, poorly graded, dense, slightly moist, light brown, fine-grained. | | | |
| 20 | B3@20' | | | | | 61 | 105.9 | 5.4 |
| 22 | | | | | | | | |
| 24 | | | | ML | Sandy Silt, hard, slightly moist, light brown, fine-grained. | | | |
| 25 | B3@25' | | | | | 53 | 107.0 | 19.4 |
| 26 | | | | | | | | |
| 28 | | | | | | | | |
| 29 | | | | SM | Silty Sand, dense, slightly moist, yellowish brown, fine- to medium-grained. | | | |
| 30 | B3@30' | | | | | 67 | 112.0 | 6.6 |

Total depth of Boring: 30.5 feet.
Fill to 5 feet. No groundwater encountered.
Backfilled with soil cuttings and tamped. Asphalt patched.
*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9708-88-01 BORING LOG.GPJ

**Figure A3,**
**Log of Boring 3, Page 1 of 1**

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. A9708-88-01

# BORING 4

ELEV. (MSL.) -- DATE COMPLETED 12/20/17

EQUIPMENT HOLLOW STEM AUGER BY: PZ

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT*) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0–5 | | | | | **ARTIFICIAL FILL** Sandy Silt, firm, slightly moist, brown to dark brown, fine-grained. | | | |
| 5–14 | | | | ML | **ALLUVIUM** Sandy Silt, firm, slightly moist, reddish brown, fine-grained, trace medium-grained. | | | |
| 14–19 | | | | SM | Silty Sand, medium dense, slightly moist, reddish brown, fine- to medium-grained. | | | |
| 19–20 | | | | SP | Sand, poorly graded, dense, slightly moist, light brown, fine-grained. | | | |

Total depth of Boring: 20 feet.
Fill to 5 feet.
No groundwater encountered.
Percolation testing performed.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

**Figure A4,**
**Log of Boring 4, Page 1 of 1**

A9708-88-01 BORING LOG.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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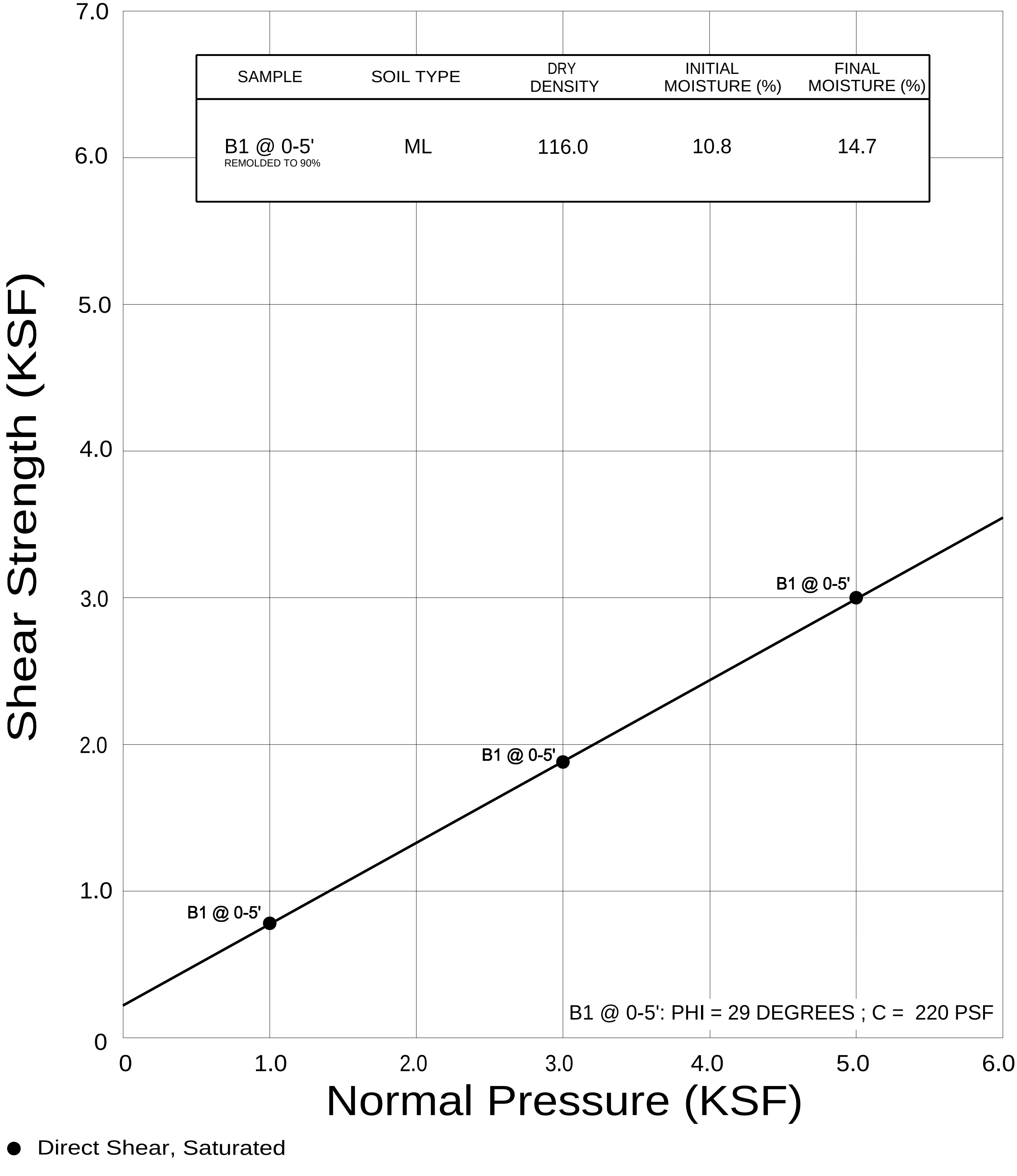
# APPENDIX B

## APPENDIX B

## LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain-size, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B9. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.




| SAMPLE | SOIL TYPE | DRY DENSITY | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B1 @ 0-5' REMOLDED TO 90% | ML | 116.0 | 10.8 | 14.7 |

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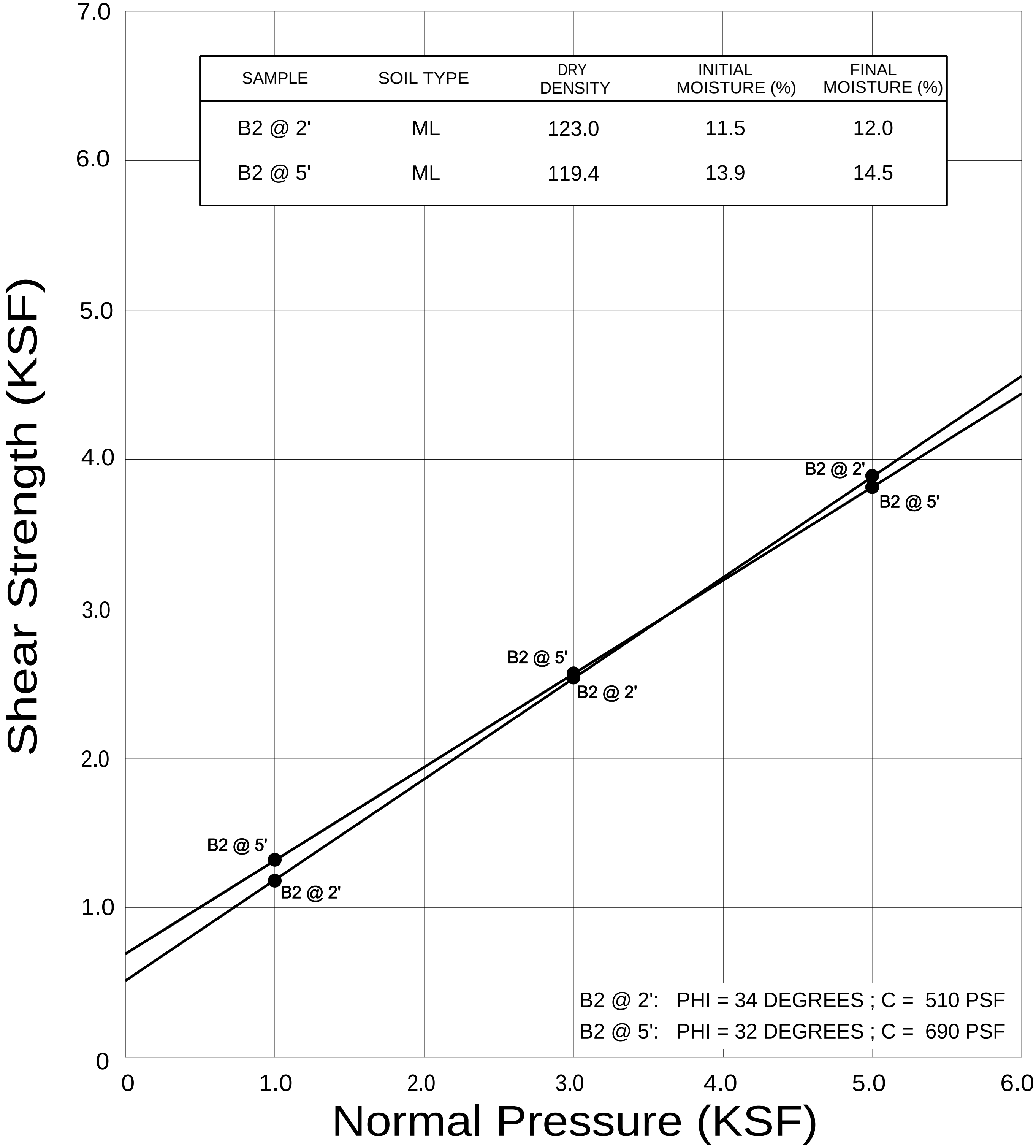
DRAFTED BY: JS | CHECKED BY: JTA

# DIRECT SHEAR TEST RESULTS

## RESIDENTIAL DEVELOPMENT

WARMINGTON RESIDENTIAL CALIFORNIA, INC.
12841 VALLEY VIEW AVENUE
LA MIRADA, CALIFORNIA

JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B1




| SAMPLE | SOIL TYPE | DRY DENSITY | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B2 @ 2' | ML | 123.0 | 11.5 | 12.0 |
| B2 @ 5' | ML | 119.4 | 13.9 | 14.5 |

● Direct Shear, Saturated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# DIRECT SHEAR TEST RESULTS

## RESIDENTIAL DEVELOPMENT

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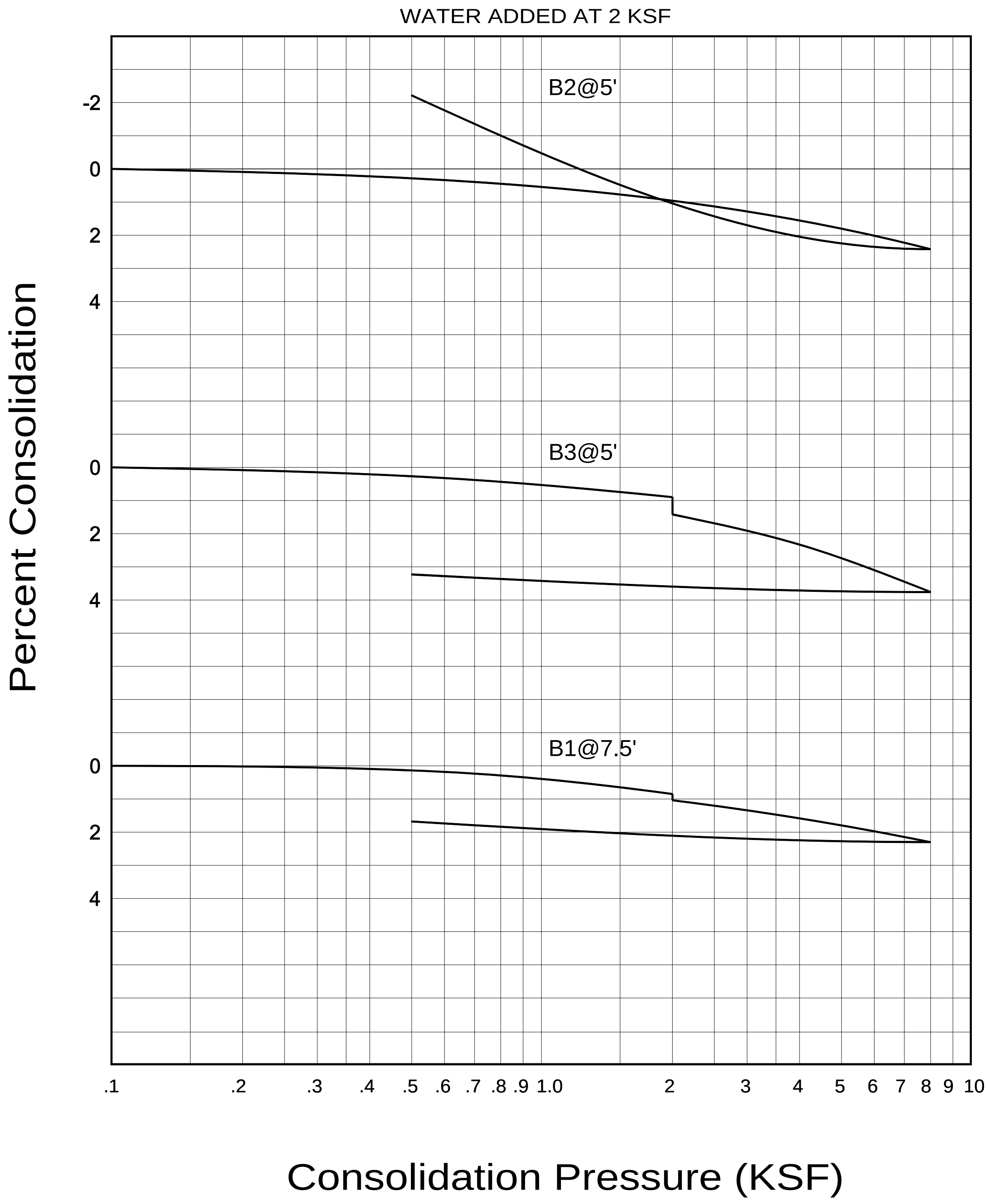
LA MIRADA, CALIFORNIA

JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B2





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# CONSOLIDATION TEST RESULTS

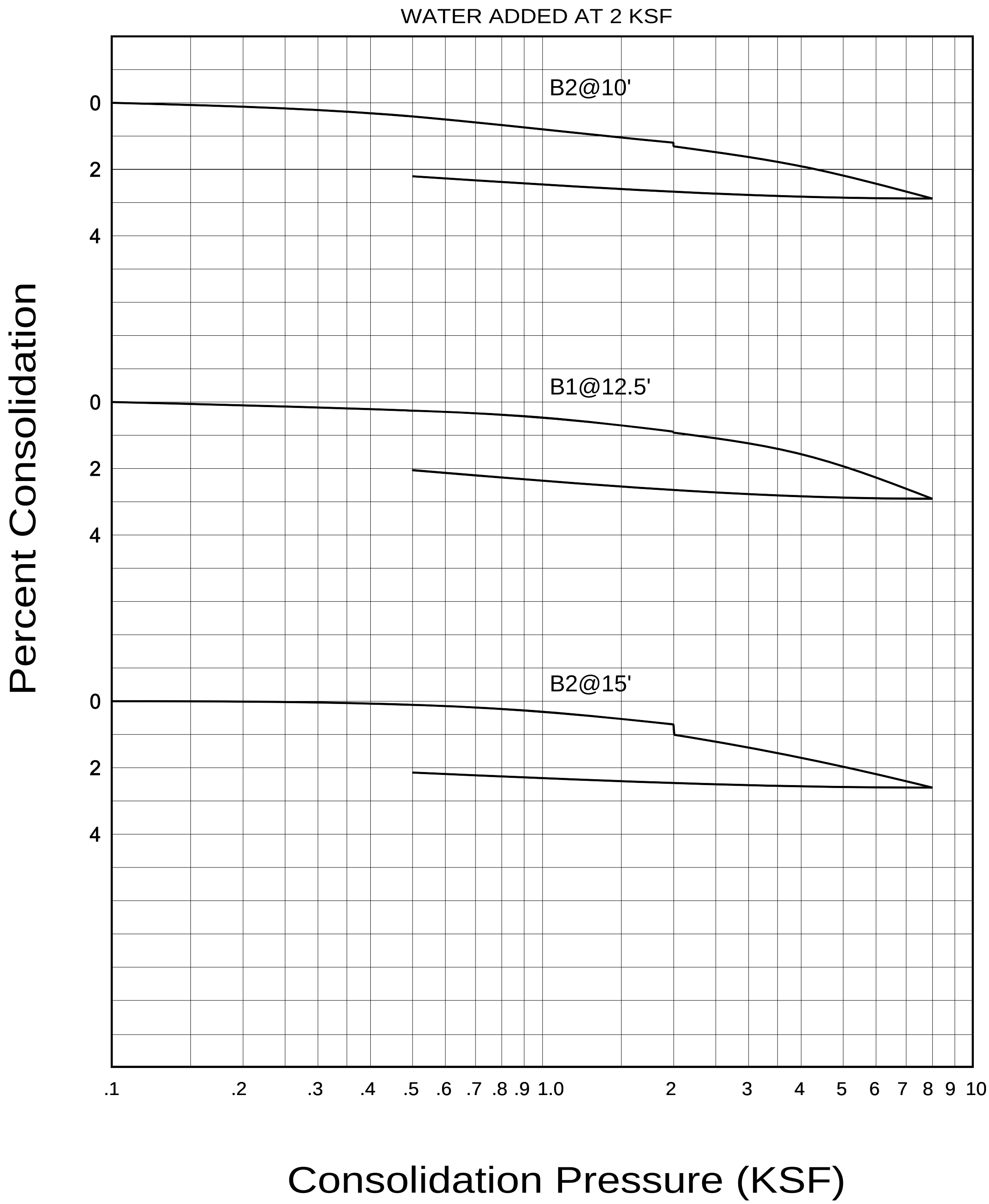
## RESIDENTIAL DEVELOPMENT

WARMINGTON RESIDENTIAL CALIFORNIA, INC.

12841 VALLEY VIEW AVENUE

LA MIRADA, CALIFORNIA

JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B3

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| | |
|---|---|
| DRAFTED BY: JS | CHECKED BY: JTA |

## CONSOLIDATION TEST RESULTS

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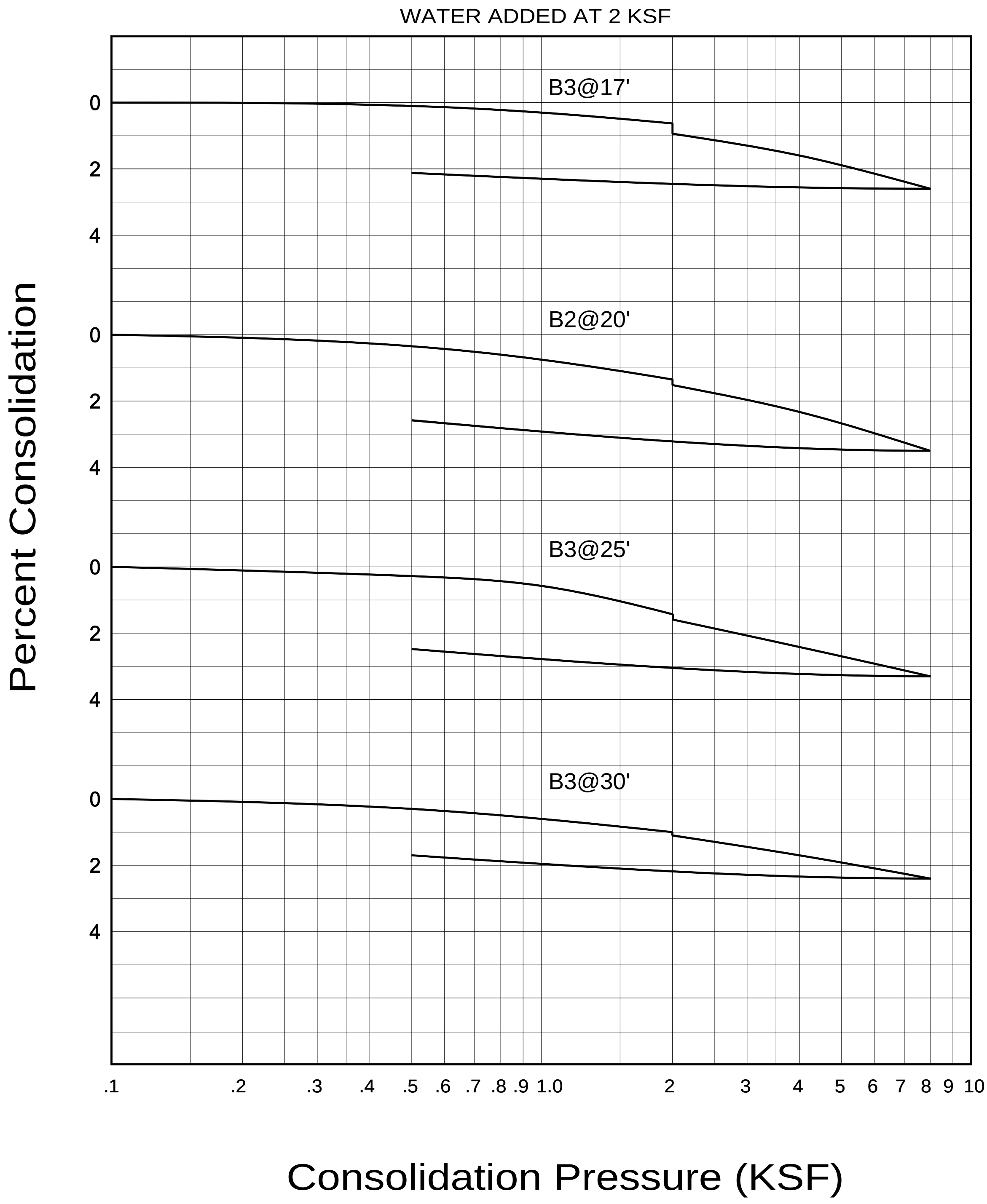
LA MIRADA, CALIFORNIA

| | | |
|---|---|---|
| JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B4 |





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|---|---|

# CONSOLIDATION TEST RESULTS

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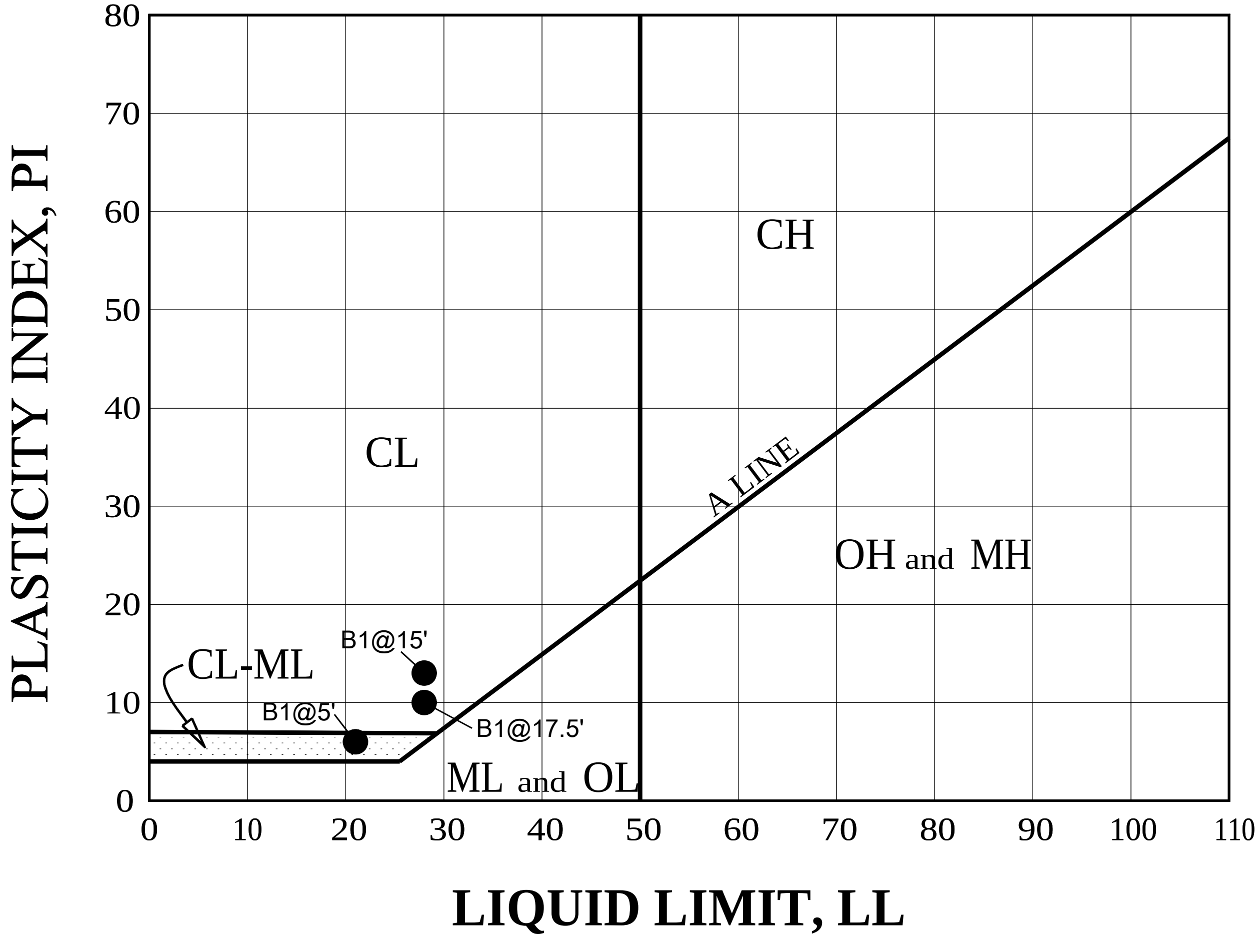
| JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B5 |
|---|---|---|




| BORING NUMBER | DEPTH (FEET) | LL | PL | PI | MOISTURE CONTENT AT SATURATION | SOIL BEHAVIOR |
|---|---|---|---|---|---|---|
| **B1** | **5** | **21** | **15** | **6** | **- -** | **CL-ML** |
| **B1** | **15** | **28** | **15** | **13** | **12.0** | **CL** |
| **B1** | **17.5** | **28** | **18** | **10** | **- -** | **CL** |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# ATTERBERG LIMITS

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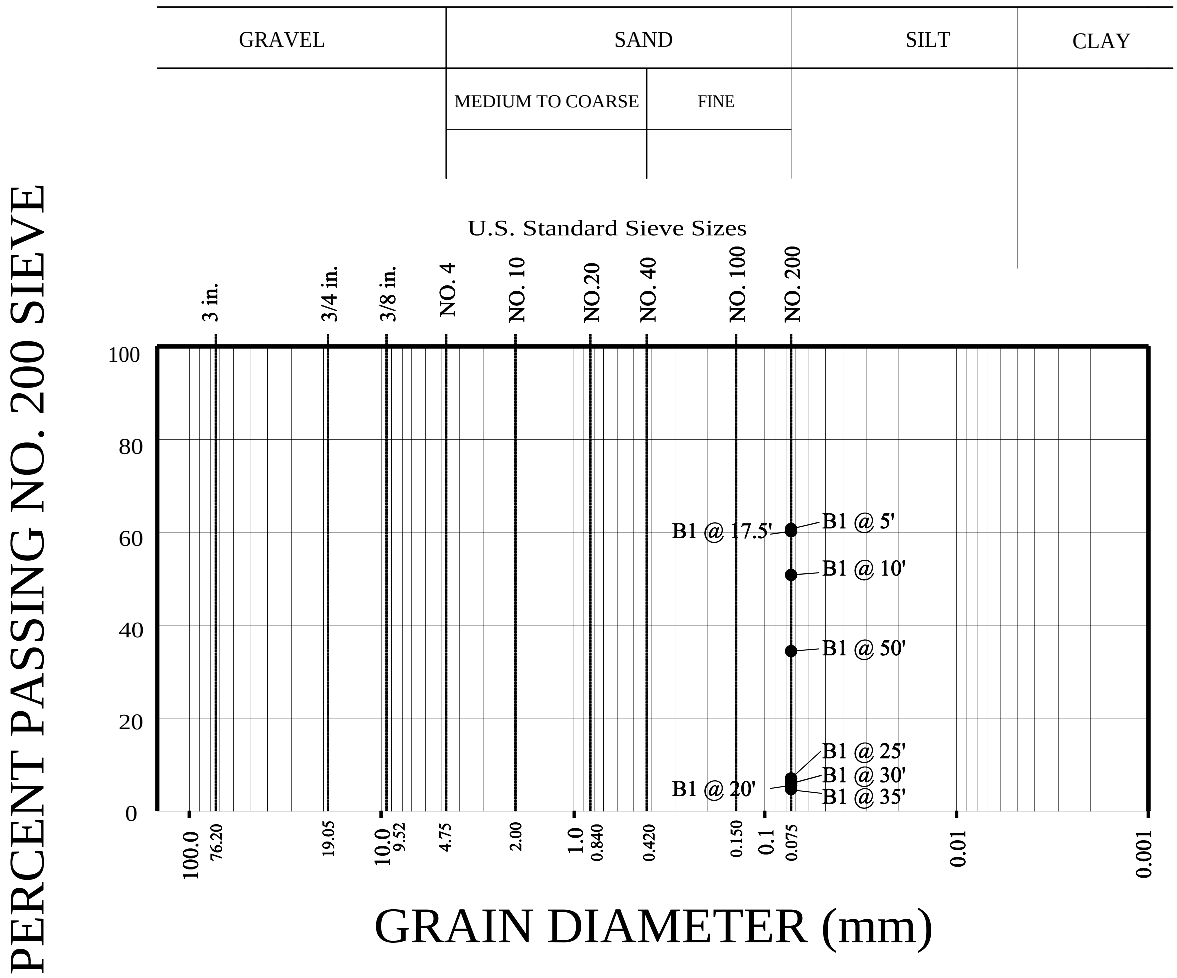
| DRAFTED BY: JS | CHECKED BY: JTA | JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B6 |
|---|---|---|---|---|




| SAMPLE | PERCENT PASSING NO. 200 SIEVE |
|---|---|
| B1 @ 5' | 60.7 |
| B1 @ 10' | 50.8 |
| B1 @ 17.5' | 60.2 |
| B1 @ 20' | 5.5 |
| B1 @ 25' | 7.0 |
| B1 @ 30' | 5.9 |
| B1 @ 35' | 4.7 |
| B1 @ 50' | 34.4 |

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# GRAIN SIZE ANALYSIS

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|---|---|---|---|---|

## SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
## ASTM D 4829-11

| Sample No. | Moisture Content (%) | | Dry Density (pcf) | Expansion Index | *UBC Classification | **CBC Classification |
|---|---|---|---|---|---|---|
| | Before | After | | | | |
| B1 @ 0-5' | 8.7 | 18.6 | 115.7 | 35 | Low | Expansive |

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2016 California Building Code, Section 1803.5.3

## SUMMARY OF LABORATORY MAXIMUM DENSITY AND
## AND OPTIMUM MOISTURE CONTENT TEST RESULTS
## ASTM D 1557-12

| Sample No. | Soil Description | Maximum Dry Density (pcf) | Optimum Moisture (%) |
|---|---|---|---|
| B1 @ 0-5' | Dark Brown Clayey Silt | 130.0 | 9.7 |

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LABORATORY TEST RESULTS

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LA MIRADA, CALIFORNIA

| DRAFTED BY: JS | CHECKED BY: JTA | JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B8 |
|---|---|---|---|---|

## SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

| Sample No. | pH | Resistivity (ohm centimeters) |
|---|---|---|
| B1 @ 0-5' | 8.6 | 1167 (Corrosive) |

## SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

| Sample No. | Chloride Ion Content (%) |
|---|---|
| B1 @ 0-5' | 0.036 |

## SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

| Sample No. | Water Soluble Sulfate (% $SO_4$) | Sulfate Exposure* |
|---|---|---|
| B1 @ 0-5' | 0.001 | Negligible |

* Reference: 2016 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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CORROSIVITY TEST RESULTS

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| DRAFTED BY: JS | CHECKED BY: JTA | JAN. 2018 | PROJECT NO. A9708-88-01 | FIG. B9 |
|---|---|---|---|---|

# APPENDIX C

# Peak Flow Hydrologic Analysis

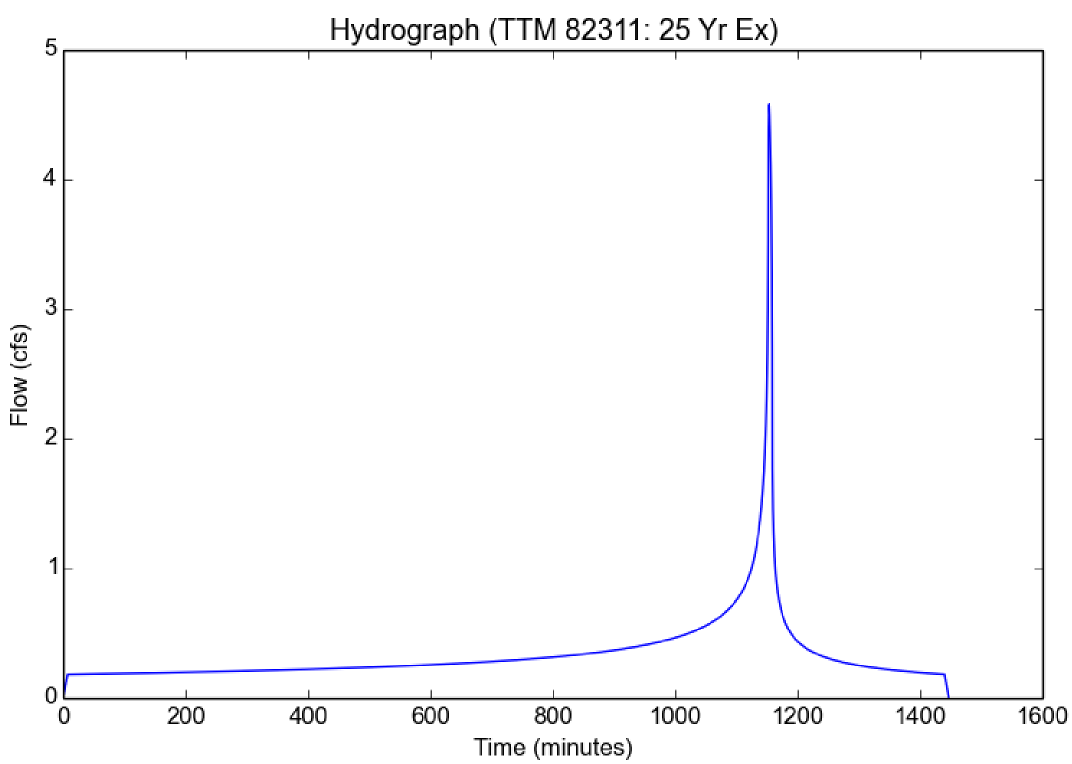
File location: J:/114905_LA_Mirada_42/5.2 Reports/Entitlement/Hydrology - Preliminary/Calcs/Hydrocalc - Hyd/_PDF/TTM 82311 - 25 Yr Ex.pdf
Version: HydroCalc 1.0.2

## Input Parameters

| | |
|---|---|
| Project Name | TTM 82311 |
| Subarea ID | 25 Yr Ex |
| Area (ac) | 1.98 |
| Flow Path Length (ft) | 517.0 |
| Flow Path Slope (vft/hft) | 0.021 |
| 50-yr Rainfall Depth (in) | 5.75 |
| Percent Impervious | 0.9 |
| Soil Type | 13 |
| Design Storm Frequency | 25-yr |
| Fire Factor | 0 |
| LID | False |

## Output Results

| | |
|---|---|
| Modeled (25-yr) Rainfall Depth (in) | 5.0485 |
| Peak Intensity (in/hr) | 2.5715 |
| Undeveloped Runoff Coefficient (Cu) | 0.896 |
| Developed Runoff Coefficient (Cd) | 0.8996 |
| Time of Concentration (min) | 7.0 |
| Clear Peak Flow Rate (cfs) | 4.5804 |
| Burned Peak Flow Rate (cfs) | 4.5804 |
| 24-Hr Clear Runoff Volume (ac-ft) | 0.6832 |
| 24-Hr Clear Runoff Volume (cu-ft) | 29760.9956 |



# Peak Flow Hydrologic Analysis

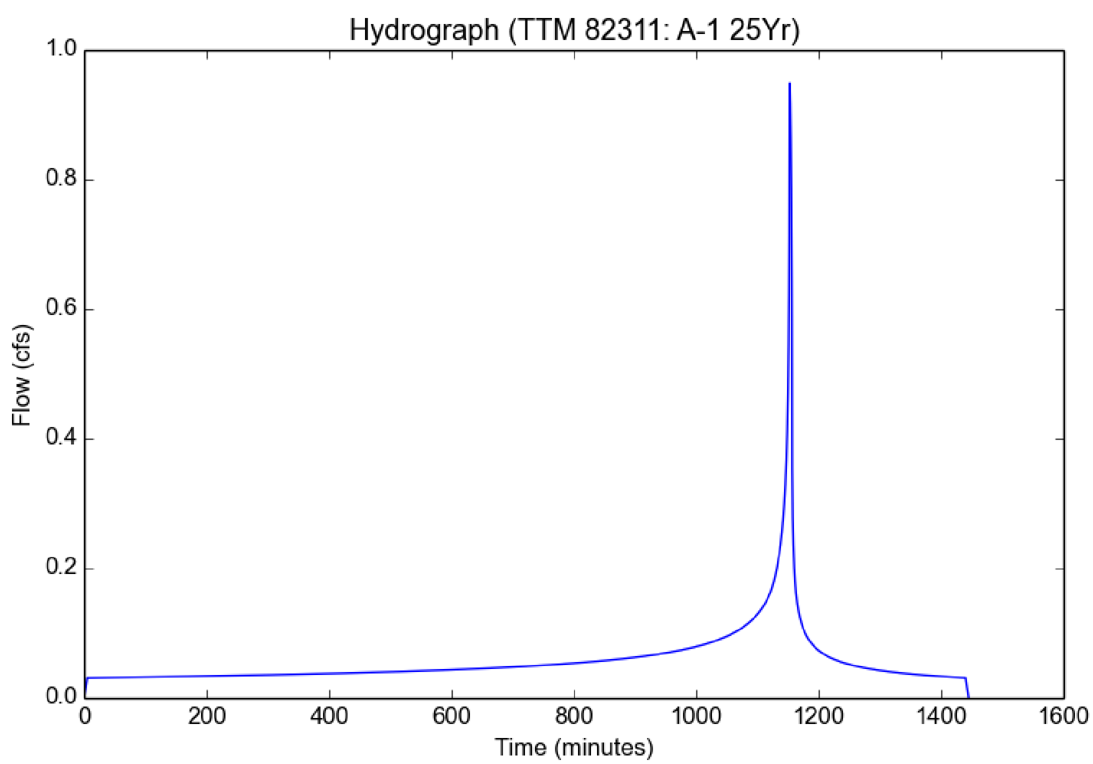
File location: C:/Users/Hydro/Desktop/TTM 82311 La Mirada/TTM 82311 - A-1 25Yr.pdf
Version: HydroCalc 1.0.3

## Input Parameters

| | |
|---|---|
| Project Name | TTM 82311 |
| Subarea ID | A-1 25Yr |
| Area (ac) | 0.35 |
| Flow Path Length (ft) | 52.0 |
| Flow Path Slope (vft/hft) | 0.006 |
| 50-yr Rainfall Depth (in) | 5.75 |
| Percent Impervious | 0.86 |
| Soil Type | 13 |
| Design Storm Frequency | 25-yr |
| Fire Factor | 0 |
| LID | False |

## Output Results

| | |
|---|---|
| Modeled (25-yr) Rainfall Depth (in) | 5.0485 |
| Peak Intensity (in/hr) | 3.0121 |
| Undeveloped Runoff Coefficient (Cu) | 0.9 |
| Developed Runoff Coefficient (Cd) | 0.9 |
| Time of Concentration (min) | 5.0 |
| Clear Peak Flow Rate (cfs) | 0.9488 |
| Burned Peak Flow Rate (cfs) | 0.9488 |
| 24-Hr Clear Runoff Volume (ac-ft) | 0.1165 |
| 24-Hr Clear Runoff Volume (cu-ft) | 5074.4478 |



# Peak Flow Hydrologic Analysis

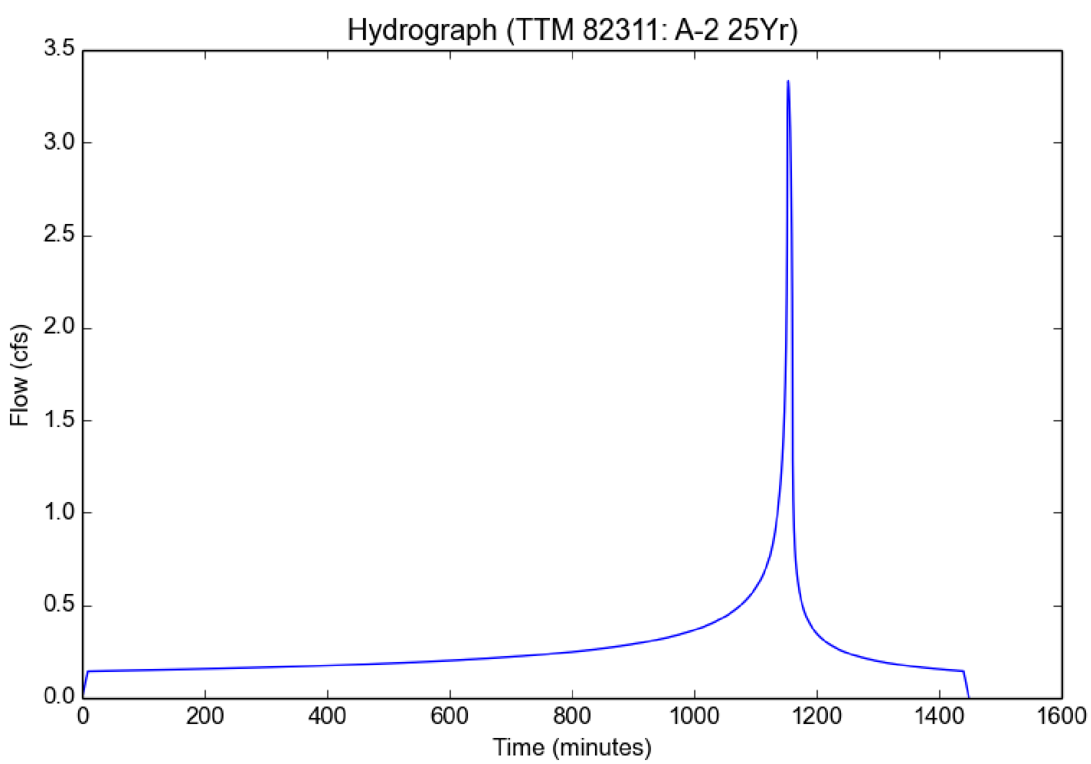
File location: C:/Users/Hydro/Desktop/TTM 82311 La Mirada/TTM 82311 - A-2 25Yr.pdf
Version: HydroCalc 1.0.3

## Input Parameters

| | |
|---|---|
| Project Name | TTM 82311 |
| Subarea ID | A-2 25Yr |
| Area (ac) | 1.63 |
| Flow Path Length (ft) | 530.0 |
| Flow Path Slope (vft/hft) | 0.006 |
| 50-yr Rainfall Depth (in) | 5.75 |
| Percent Impervious | 0.86 |
| Soil Type | 13 |
| Design Storm Frequency | 25-yr |
| Fire Factor | 0 |
| LID | False |

## Output Results

| | |
|---|---|
| Modeled (25-yr) Rainfall Depth (in) | 5.0485 |
| Peak Intensity (in/hr) | 2.285 |
| Undeveloped Runoff Coefficient (Cu) | 0.8627 |
| Developed Runoff Coefficient (Cd) | 0.8948 |
| Time of Concentration (min) | 9.0 |
| Clear Peak Flow Rate (cfs) | 3.3326 |
| Burned Peak Flow Rate (cfs) | 3.3326 |
| 24-Hr Clear Runoff Volume (ac-ft) | 0.5426 |
| 24-Hr Clear Runoff Volume (cu-ft) | 23634.1107 |



APPENDIX D

# Peak Flow Hydrologic Analysis

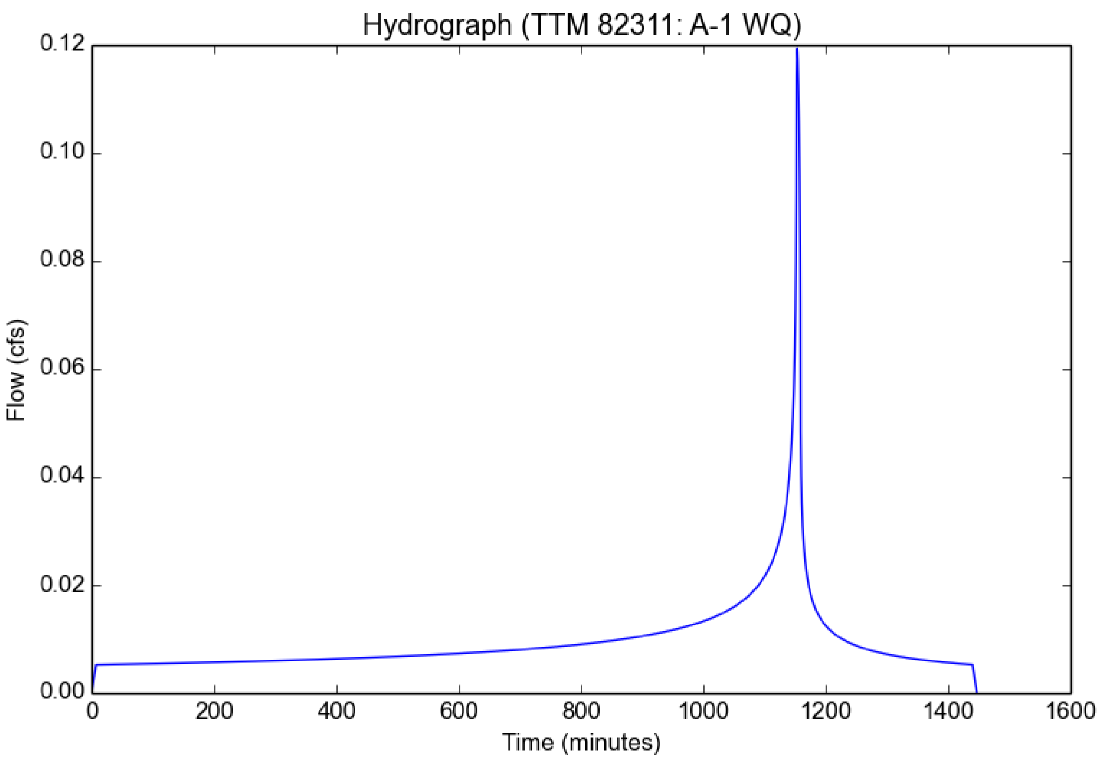
File location: C:/Users/Hydro/Desktop/TTM 82311 La Mirada/TTM 82311 - A-1 WQ.pdf
Version: HydroCalc 1.0.3

## Input Parameters

| | |
|---|---|
| Project Name | TTM 82311 |
| Subarea ID | A-1 WQ |
| Area (ac) | 0.35 |
| Flow Path Length (ft) | 52.0 |
| Flow Path Slope (vft/hft) | 0.006 |
| 85th Percentile Rainfall Depth (in) | 0.85 |
| Percent Impervious | 0.86 |
| Soil Type | 13 |
| Design Storm Frequency | 85th percentile storm |
| Fire Factor | 0 |
| LID | True |

## Output Results

| | |
|---|---|
| Modeled (85th percentile storm) Rainfall Depth (in) | 0.85 |
| Peak Intensity (in/hr) | 0.433 |
| Undeveloped Runoff Coefficient (Cu) | 0.1 |
| Developed Runoff Coefficient (Cd) | 0.788 |
| Time of Concentration (min) | 7.0 |
| Clear Peak Flow Rate (cfs) | 0.1194 |
| Burned Peak Flow Rate (cfs) | 0.1194 |
| 24-Hr Clear Runoff Volume (ac-ft) | 0.0194 |
| 24-Hr Clear Runoff Volume (cu-ft) | 843.9485 |



# Peak Flow Hydrologic Analysis

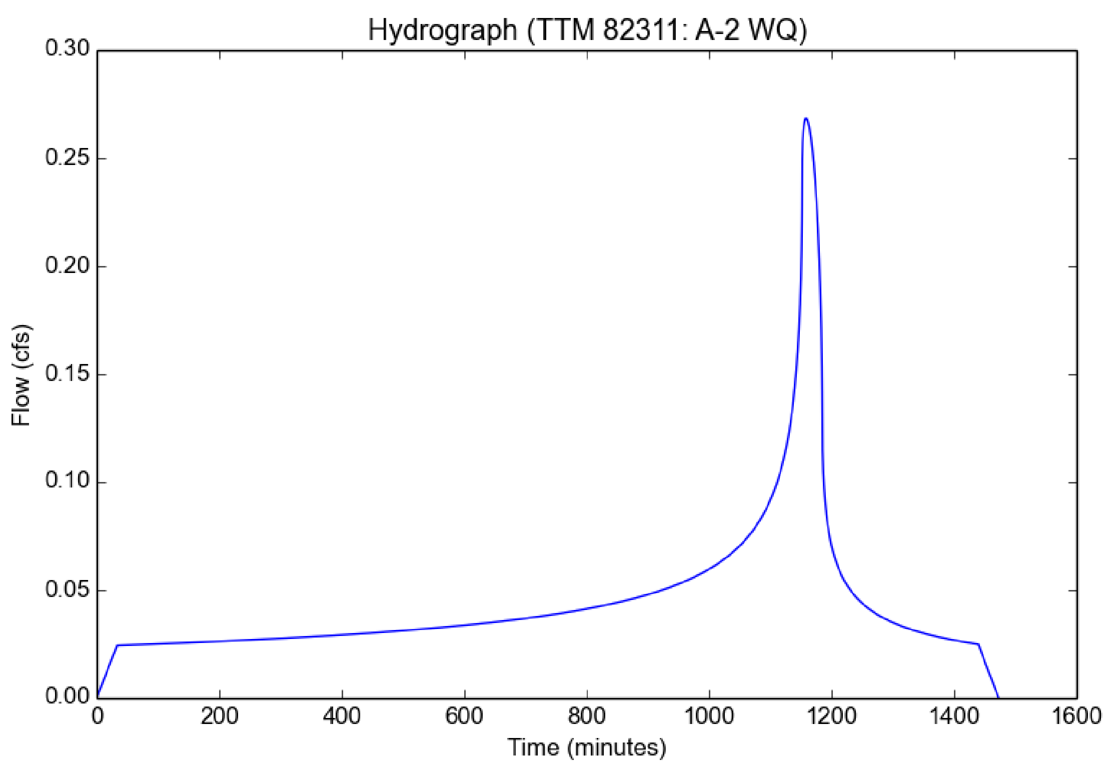
File location: C:/Users/Hydro/Desktop/TTM 82311 La Mirada/TTM 82311 - A-2 WQ.pdf
Version: HydroCalc 1.0.3

## Input Parameters

| | |
|---|---|
| Project Name | TTM 82311 |
| Subarea ID | A-2 WQ |
| Area (ac) | 1.63 |
| Flow Path Length (ft) | 530.0 |
| Flow Path Slope (vft/hft) | 0.006 |
| 85th Percentile Rainfall Depth (in) | 0.85 |
| Percent Impervious | 0.86 |
| Soil Type | 13 |
| Design Storm Frequency | 85th percentile storm |
| Fire Factor | 0 |
| LID | True |

## Output Results

| | |
|---|---|
| Modeled (85th percentile storm) Rainfall Depth (in) | 0.85 |
| Peak Intensity (in/hr) | 0.2089 |
| Undeveloped Runoff Coefficient (Cu) | 0.1 |
| Developed Runoff Coefficient (Cd) | 0.788 |
| Time of Concentration (min) | 33.0 |
| Clear Peak Flow Rate (cfs) | 0.2683 |
| Burned Peak Flow Rate (cfs) | 0.2683 |
| 24-Hr Clear Runoff Volume (ac-ft) | 0.0902 |
| 24-Hr Clear Runoff Volume (cu-ft) | 3930.4411 |

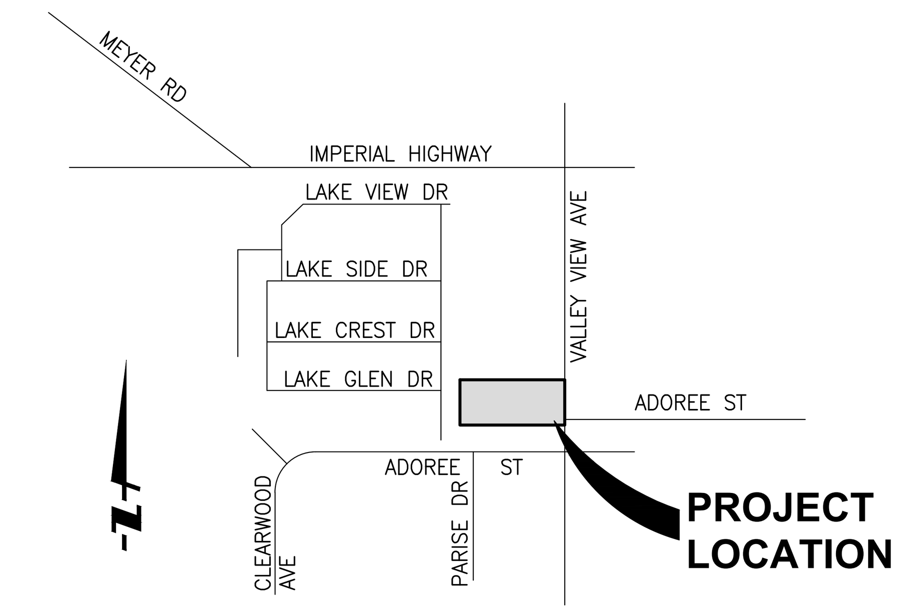


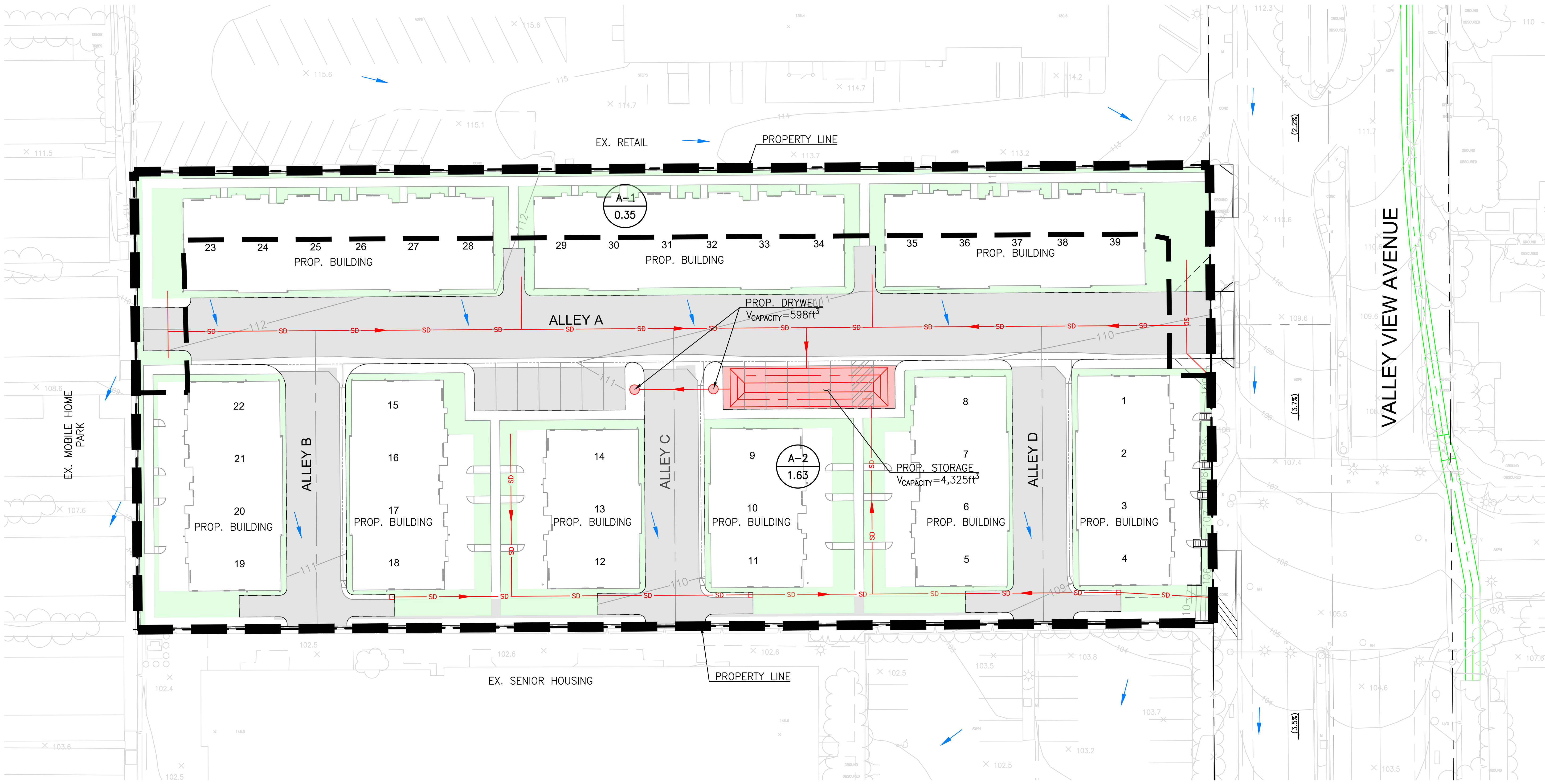
| Subarea Designation | Trib Area (Ac) | DCV (cuft) | Q (cfs) | BMP Type | Field Infiltration Rate per soils report (in/hr) | Total correction factor | Design Infiltration Rate per soils report (in/hr) | # of Drywells | Drywell diameter (ft) | Drywell infiltrating depth (ft) | Drywell non-infiltrating depth (ft) | Drawdown time (hr) | Provided infiltration footprint (ft^2) | Provided Drywell Volume (cu-ft) | Provided non-infiltrating volume (ft^3) | Provided volume (cuft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Total site | 1.98 | 4,774 | | Drywells | | | 0.87 | 2 | 4 | 27 | 13 | 94 | 704 | 598 | 4,325 | 4923 |

| Storage Pipe Volume | | |
|---|---|---|
| Diameter= | 6 | ft |
| Length= | 140 | ft |
| Volume= | 3958 | cu-ft |

| PVC Pipe Volume | | |
|---|---|---|
| Diameter= | 8 | in |
| Length= | 1051 | ft |
| Volume= | 367 | cu-ft |

# LID STUDY FOR TR 82311




## LOW IMPACT DEVELOPMENT SUMMARY

DESIGNATED PROJECT
SITE TOTAL ACREAGE: 1.98AC
SOIL TYPE: 13
85-PERCENTILE STORM DEPTH: 0.85in
DESIGN INFILTRATION RATE: 0.87in/hr

| | PRE-DEVELOPMENT | POST-DEVELOPMENT |
|---|---|---|
| PERCENT IMPERVIOUSNESS | 0.9 | 0.86 |
| DESIGN CAPTURE VOLUME (DCV) | - | A-1: 844$ft^3$<br>A-2: 3930 $ft^3$<br>TOTAL: 4774 $ft^3$ |
| BMP TYPE | - | DRYWELLS<br>DETENTION BASIN |
| PROVIDED BMP VOLUME | - | DRYWELL: 598 $ft^3$<br>STORAGE: 4325 $ft^3$<br>TOTAL: 4923 $ft^3$ |

## NOTE

1. PROPOSED ONSITE STORM DRAIN FACILITIES AND BMPs TO BE MAINTAINED BY HOME OWNERS' ASSOCIATION.
2. REFER TO PRELIMINARY DRAINAGE REPORT SECTION V. FOR BMP CAPACITY CALCULATIONS.
3. PROJECT SITE IS NOT WITHIN LOS ANGELES COUNTY ADOPTED FLOODWAY.
4. PROJECT SITE IS NOT WITHIN FEMA FLOOD ZONE "A".

## LEGEND

- EXISTING PROPERTY LINE
- PROPOSED DRAINAGE AREA BOUNDARY
- DIRECTION OF SURFACE FLOW
- DIRECTION OF PIPE FLOW
- A — MAJOR AREA NAME
- 0.83 — ACREAGE
- PROPOSED STREET
- PROPOSED LANDSCAPE
- PROPOSED DRYWELL
- PROPOSED WQ STORAGE
- SD — PROPOSED STORM DRAIN
- PROPOSED CATCH BASIN
- EXISTING STORM DRAIN

30' 0' 15' 30'
SCALE: 1" = 30'

REGISTERED PROFESSIONAL ENGINEER
ERIC LISSNER
NO. C73065
CIVIL
STATE OF CALIFORNIA

## PROPOSED LID EXHIBIT
TTM 82311
LA MIRADA, CA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