# Appendix D

# Hydrology Report

# Hydrology Report

## For

## Hesperia Industrial

6730 Santa Fe Ave E, Hesperia, CA
Project No. 23002420.00
May 30, 2023

**Prepared for:**


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**Prepared: May 30th, 2023**

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## Table of Contents

Introduction and Purpose ........ 3
Vicinity Map ........ 3
Site Coordinates ........ 4
Existing Hydrologic Conditions ........ 4
Developed Hydrologic Conditions ........ 4-5
Methodology ........ 5
Report Summary ........ 6-7
Appendix A-Hydrology Maps ........ 8
Appendix B-Intensity ........ 11
Appendix C-Calculations ........ 13
Appendix D-References ........ 21

Introduction and Purpose

This drainage report provides an analysis of the proposed hydrology characteristic for the improvements of the Hesperia Industrial warehouse. The project is located at the northwest side of Santa Fe Avenue and Jenny Street and South of the A&T & S.F. Railway in Hesperia, California. The site address is 6730 Santa Fe Avenue E. Refer to Figure 1 in the report for project location and Figure 2 for the site coordinates. The project site is approximately 6.11 acres but the northern half of the site is vacant and will not be developed. The southern half of the site is occupied for industrial warehouse use which is made up of landscaping and ac pavement. The purpose of this report is to analyze the various storm events in accordance with the San Bernadino Hydrology Manual and compare peak flow values between existing and proposed development conditions.

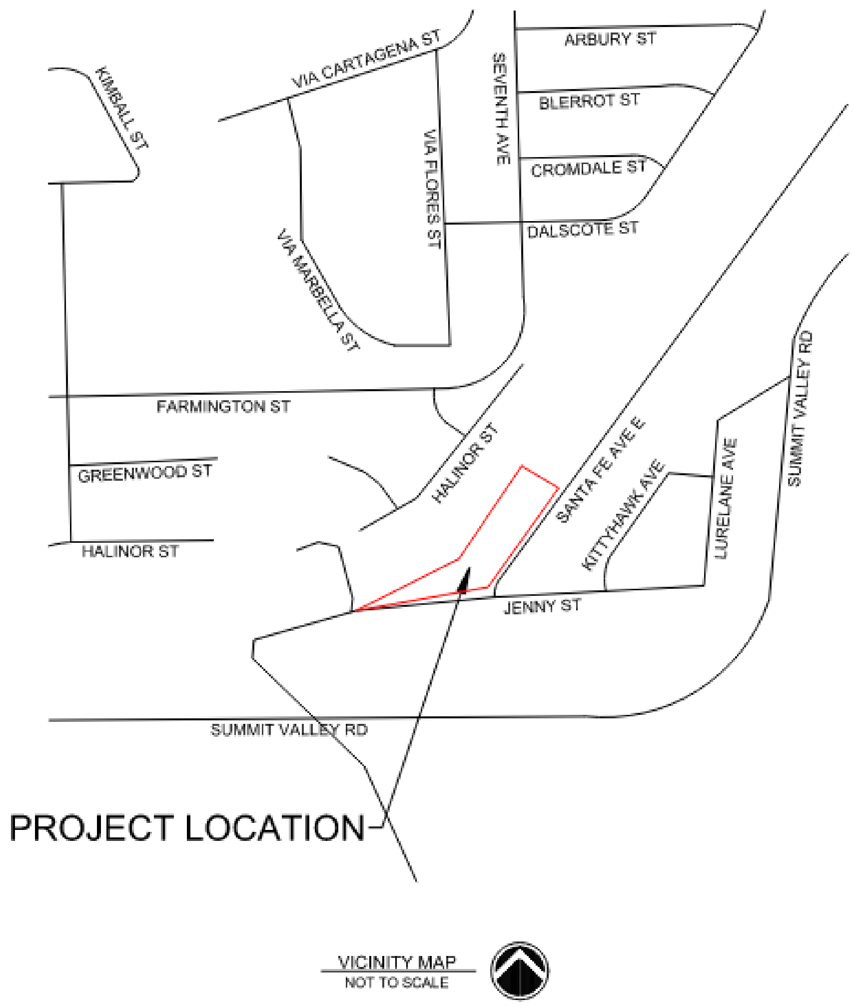



Figure 1: Vicinity Map

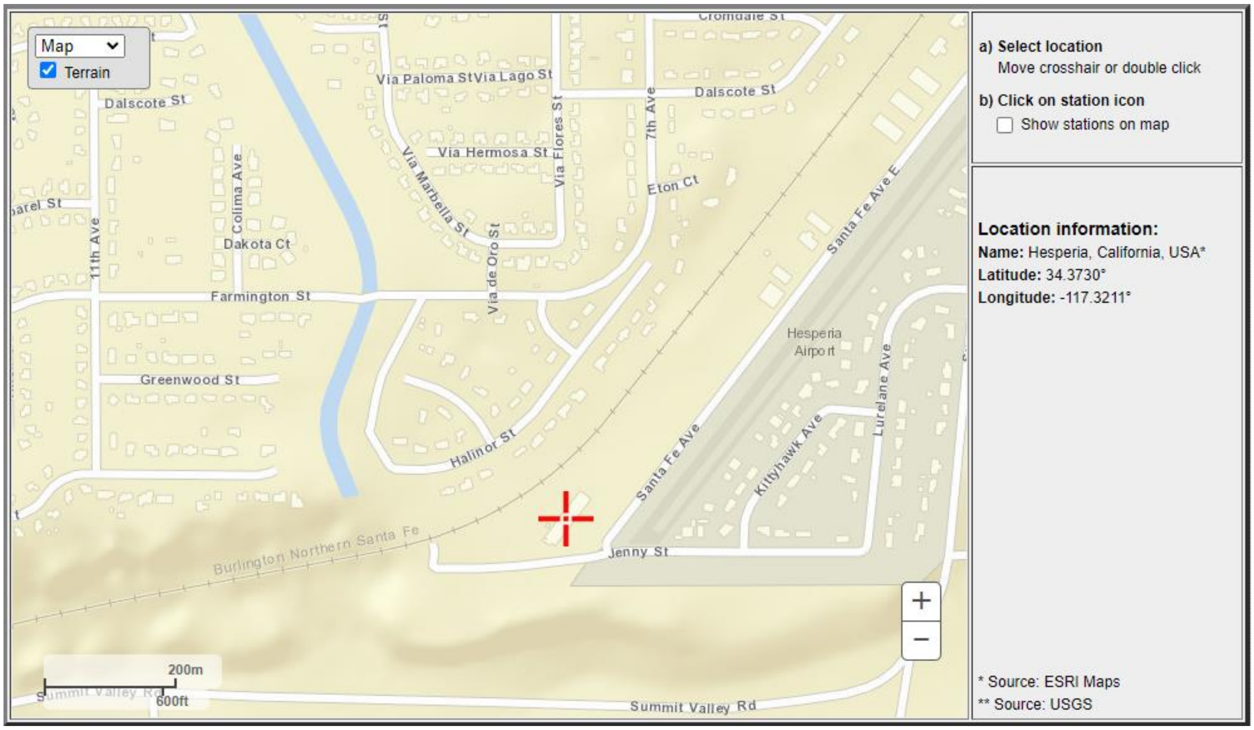



Figure 2: Site Coordinates

(Source USGS)

## Existing Hydrologic Conditions

The project site has an existing warehouse steel frame structure with a building area of 21,831 s.f. Only about 8.2% of the site including the building and pavement is developed. Most of the site consists of dirt and landscaping. A chain-linked fence surrounds the site along the property line. In Parcel 1, lot 121, the existing site generally drains west via the natural swale northwest of the existing building and sheet flows across the site southeast of the existing building. The only existing drainage feature on the east side of the site is the existing natural dirt swale. In Paracel 3, lot 120, the existing site sheet flows southeast across the entire lot. There is currently no drainage systems or water quality features in Paracel 3.

The boring done by the Salem geotechnical engineer yielded that the soils on site are predominately of silty sand with an infiltration rate of 1.12 in/hr. In addition, it was determined the highest groundwater depth per the boring results is estimated to be greater than 50 feet below the ground surface.

## Developed Hydrologic Conditions

The proposed improvements include adding two concrete loading decks for the trailers, an AC paved parking lot, and utility coordination to assist with drainage on-site. Each loading dock will have 4 depressed loading bays and will cover a square footage of 8,000-8500 s.f. The site area is relatively flat and won't require major changes in grade. The minor changes in grade will be to provide adequate drainage.

The proposed site has 3 drainage areas, DA-A, B, and C, and each of these will have its own BMP to catch and treat the stormwater runoff. Drainage area DA-A and DA-B each contain a loading zone and each loading zone will have a trench drain which will catch the runoff. The runoff at both trailer loading docks will be piped to the nearest of two proposed drywells. In the parking area there will also be a proposed 18"x18" catch basin east of the existing warehouse which will be piped to the proposed drywell in DA-A adjacent to Jenny Street.  The last drainage area DA-C, is located north of the existing building and proposed improvements, and is completely pervious but has an infiltration rate that is greater than 0.3 in/hr which means it requires a BMP as well. The BMP for drainage area C will be an infiltration basin sized to capture the runoff as it sheet flows northeast. The basin will be able to retain 470 s.f. and will be one foot deep. The drywells and infiltration basin were sized based on the LID BMP design handbook, and with regard to geotechnical data and recommendations. These BMPs will act both as retention and infiltration BMPs.

When placing new utilities on site, the trenches should be backfilled using the excavated material in loose lifts not exceeding 8 inches and compacted to 95% relative compaction. In addition, the impervious surfaces within 10 feet of the building foundation are required to be sloped at a minimum of two percent away from the building. The roof drains should fall onto splash blocks to direct the water a minimum of 5 feet away from structures or connect to the storm drain system.

## Methodology

The peak flow rate of each drainage area was calculated using the Rational Method consistent with the San Bernadino County Hydrology Manual. Peak flows were calculated for the 1-hour, 10-year, and 100-year storm events. The manual provides isohyet maps for the rainfall intensity values which were used and are provided in Appendix B. The runoff coefficient ( C ) was determined using the equations provided in the hydrology manual and the geotechnical report was used to decide which of the two equations to solve based on the design infiltration rates. See the percolation test results and locations in appendix D. Autodesk Hydra flow Express, an extension of Civil 3D, was utilized to analyze the hydrology of the site.

The site was split into three different designated areas to compare the pre- and post- conditions with respect to the volume retention needed for each area. The San Bernadino County Hydrology Manual was utilized to determine the peak runoff and volume retained per drainage area.

Report Summary

The rational method was used to calculate the peak flows for the 1-hour, 10-year and 100-year storm events. The following tables summarize the results found for each drainage area and storm event.

Total Site Area: 4.45 acres

DA A: 1.502 acres

DA B: 0.183 acres

DA C: 2.769 acres

Flowrate Equation: $Q_n = CiA$

Runoff Volume: $V_n = Qn * Tc * \left(\frac{60\ sec}{1\ \text{min}}\right)$

The total runoff of the existing development given a 100-year; 1-hour storm event is 4.69 cfs. A summary of the existing condition hydrology results is provided in Table 1 below.

Table 1: Existing Hydrology Summary

| | Area (ac) | 10-yr Q1-hr (cfs) | 100-yr Q1-hr (cfs) |
|---|---|---|---|
| DA A | 4.45 | 2.07 | 4.69 |
| Total (cfs) | 4.45 | 2.07 | 4.69 |

* Intensity values from figure D-2.

*C value from Hydrology Manual/Geotech report

The total runoff of the proposed development given a 100-year; 1-hour storm event is 6.35 cfs. A summary of the developed condition hydrology results is provided in Table 2 below.

Table 2: Proposed Hydrology Summary

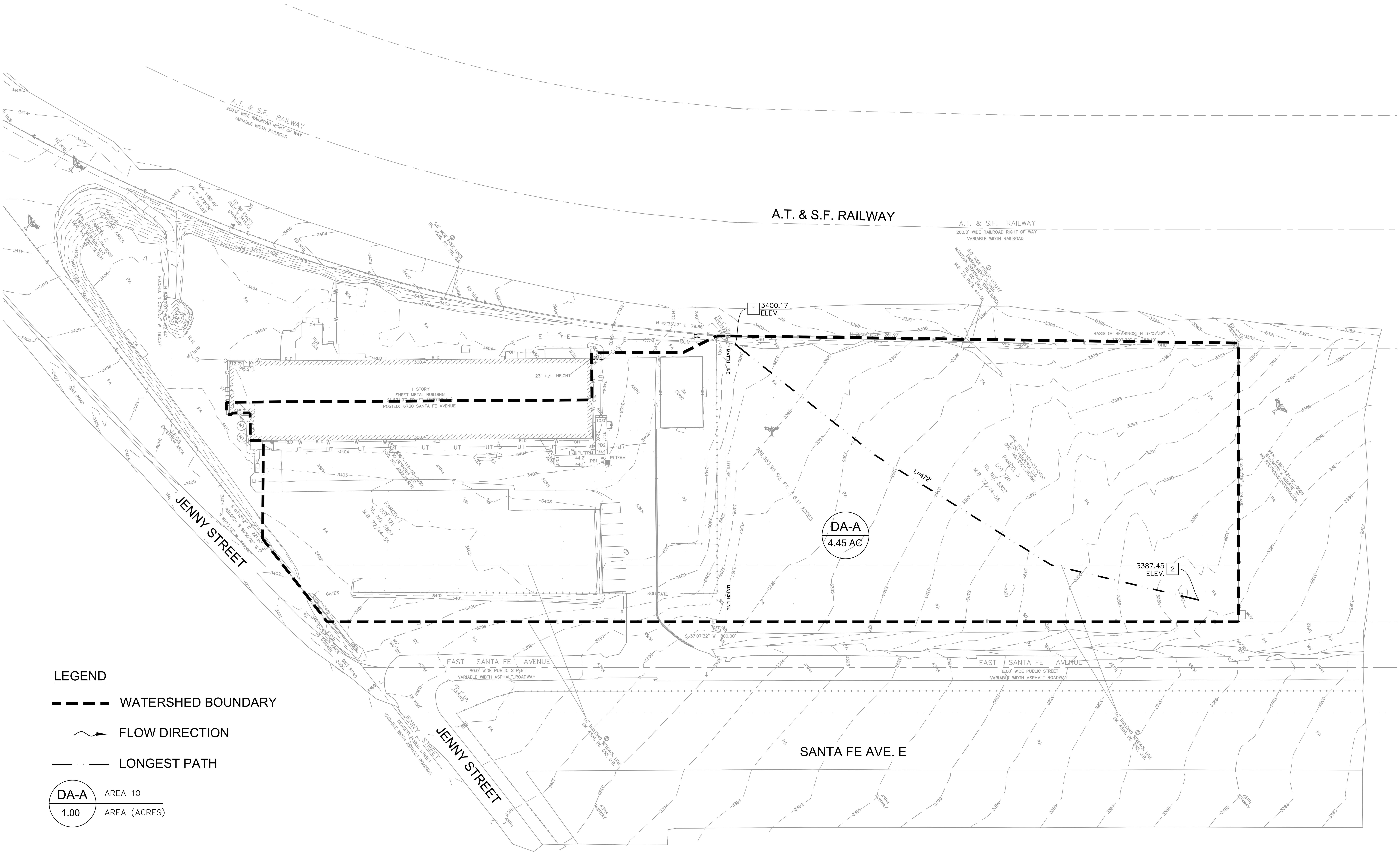
| | Area (ac) | 10-yr Q1-hr (cfs) | 100-yr Q1-hr (cfs) |
|---|---|---|---|
| DA A | 1.502 | 1.12 | 2.06 |
| DA B | 1.083 | 0.92 | 1.55 |
| DA C | 2.769 | 1.13 | 2.74 |
| Total (cfs) | 4.45 | 3.25 | 6.35 |

* Intensity values from figure D-2.

*C value from Hydrology Manual/Geotech report

A comparison of the pre- and post- development conditions show there is a 1.66 cfs increase in peak flow under the post-development conditions. Although the site will see an increase in peak runoff, the proposed BMPs are designed to capture this increase preventing offsite flow.

# Appendix A-Hydrology Maps




REVISIONS

| No. | DESCRIPTION | DATE |
| --- | --- | --- |

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# HESPERIA INDUSTRIAL
6730 SANTA FE AVENUE E HESPERIA, CA 92345

## PRE-DEVELOPMENT HYDROLOGY EXHIBIT

IMEG Project No: 23002420
File Name: Existing Hydrology Exhibit.dwg

Field Book No:####
Drawn By: AF
Checked By: RH
Date: 5/26/2023

EXH



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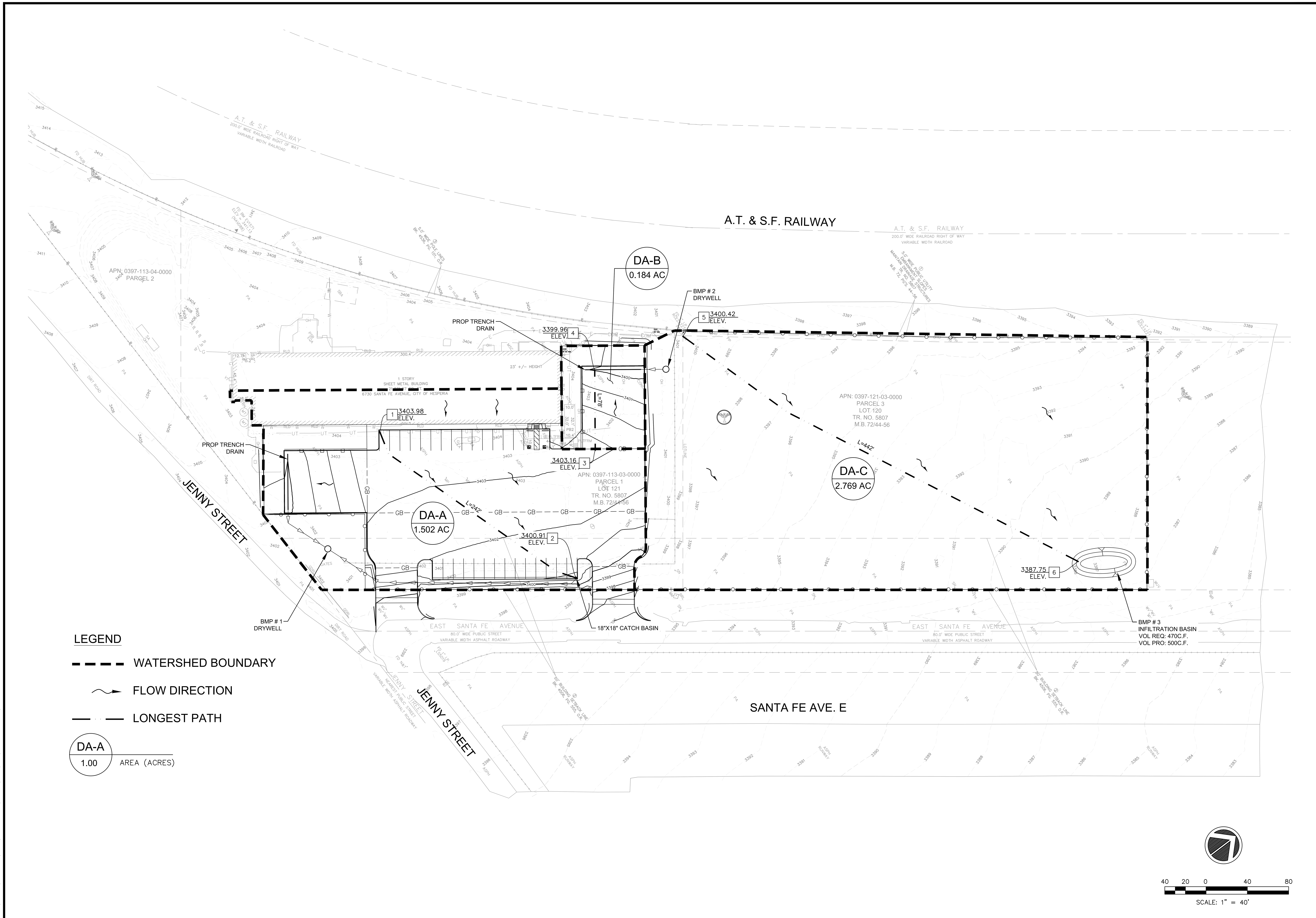


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|---|---|---|
| | | |

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## POST-DEVELOPMENT HYDROLOGY EXHIBIT

IMEG Project No: 23002420
File Name: Proposed Hydrology Exhibit.dwg

Field Book No: ####
Drawn By: AF
Checked By: RH
Date: 5/30/2023

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Sheet 2 of 2

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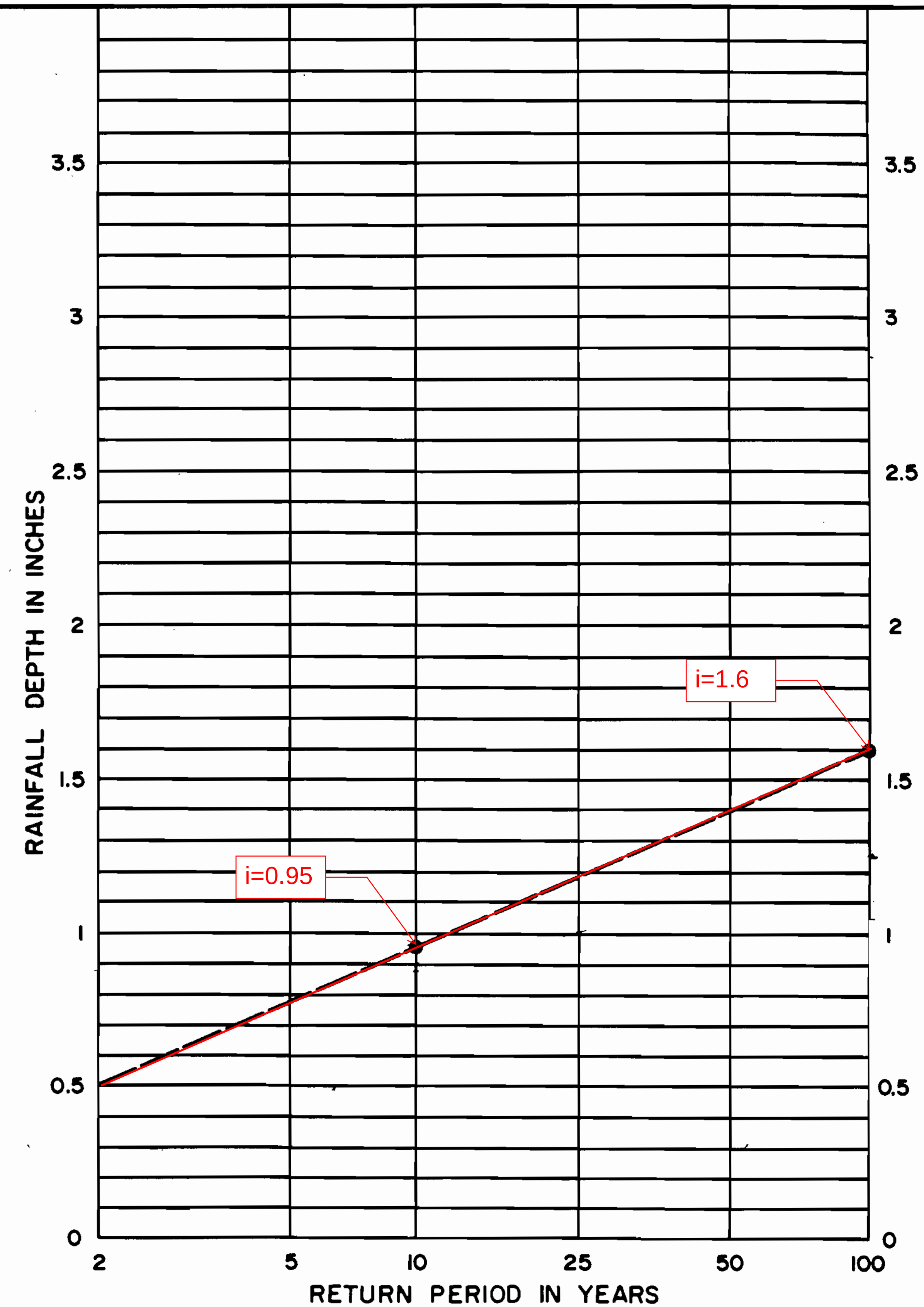
# Appendix B- Intensity




NOTE:

1. FOR INTERMEDIATE RETURN PERIODS PLOT 10-YEAR AND 100-YEAR ONE HOUR VALUES FROM MAPS, THEN CONNECT POINTS AND READ VALUE FOR DESIRED RETURN PERIOD. FOR EXAMPLE GIVEN 10-YEAR ONE HOUR = 0.95" AND 100-YEAR CNE HOUR = 1.60", 25-YEAR ONE HOUR = 1.18".

REFERENCE: NOAA ATLAS 2, VOLUME XI-CAL., 1973

**SAN BERNARDINO COUNTY**
HYDROLOGY MANUAL

RAINFALL DEPTH VERSUS RETURN PERIOD FOR PARTIAL DURATION SERIES



FIGURE D-2

# Appendix C- Calculations

Calculations

Flowrate Equation: $Q_n = C\text{i}A$

Runoff Volume:$V_n = Qn * Tc * \left(\frac{60\ sec}{1\ \text{min}}\right)$

Runoff Coefficient: $c = \begin{cases} 0.90(a_i + \frac{(I-Fp)ap}{I}, for\ I\ greater\ than\ Fpi \\ 0.90ai, for\ I\ less\ than\ or\ equal\ to\ Fp \end{cases}$

Rainfall intensity- per Figure D-2 in Hydrology Manual

## Pre-Development Condition

$DA - A = 4.45\ acres$

$I_{10yr,1hour} = 0.95\ in/hr$

$I_{10yr,1hour} = 1.6\ in/hr$

$$C = \begin{cases} 0.90(a_i + \dfrac{(I-Fp)ap}{I}, for\ I\ greater\ than\ Fpi \\ 0.90ai, for\ I\ less\ than\ or\ equal\ to\ Fp \end{cases}$$

Ai (ratio of impervious): 0.14779

Ap (ratio of pervious): 0.8522

I 10yr (intensity): 0.95 in/hr

I 100yr (intensity): 1.6 in/hr

Fp(per P-1 in Geotech report): 0.5 in/hr

$$C_{10yr} = 0.90(0.14779 + \frac{(0.95 - 0.5)0.8522}{0.95} = 0.49$$

$$C_{100yr} = 0.90(0.14779 + \frac{(1.6 - 0.5)0.8522}{1.6} = 0.66$$

Peak Flows:

DA-A

$$Q_{10yr,1hour} = CiA = (0.49) * \left(0.95 \frac{in}{hr}\right) * (4.45) = 2.07\ cfs$$

$$Q_{100\ yr,1\ hour} = CiA = (0.66) * \left(1.6 \frac{in}{hr}\right) * (4.45) = 4.69\ cfs$$

Time of Concentration: Tc= 13 min

See plate D-3 for Time of Concentration results.

Runoff Volume:

Runoff Volume:$V_n = Qn * Tc * \left(\frac{60\ sec}{1\ \text{min}}\right)$

$$V_{10year,1hour} = 2.07 * 13 * \left(\frac{60\ sec}{1\ \text{min}}\right) =$$

$$V_{100year,1hour} = 4.69 * 13 * \left(\frac{60\ sec}{1\ \text{min}}\right) =$$

## **Post-Development Condition**

$DA - A = 1.502acres$

$DA - B = 1.083\ acres$

$DA - C = 2.769\ acres$

DA-A

$$C = \begin{cases} 0.90(a_i + \frac{(I - Fp)ap}{I}, for\ I\ greater\ than\ Fpi \\ 0.90ai, for\ I\ less\ than\ or\ equal\ to\ Fp \end{cases}$$

Ai (ratio of impervious): 0.881

Ap (ratio of pervious): 0.119

I (intensity): 0.95 in/hr

I 100yr (intensity): 1.6 in/hr

Fp(per P-2 in Geotech report): 1.03 in/hr

$$C_{10yr} = 0.90(0.881) = 0.79$$

$$C_{100yr} = 0.90(0.881 + \frac{(1.6 - 0.5)0.119}{1.6} = 0.86$$

Peak Flows:

$$Q_{10yr,1hour} = CiA = (0.79) * \left(0.95 \frac{in}{hr}\right) * (1.502) = 1.12\ cfs$$

$$Q_{100\ yr,1hour} = CiA = (0.86) * \left(1.6 \frac{in}{hr}\right) * (1.502) = 2.06\ cfs$$

Time of Concentration: Tc= 6.5 min

See plate D-3 for Time of Concentration results.

Runoff Volume:

Runoff Volume:$V_n = Qn * Tc * \left(\frac{60\ sec}{1\ \text{min}}\right)$

$$V_{10year,1hour} = 1.19 * 6.5 * \left(\frac{60\ sec}{1\ \text{min}}\right) = 464.1\ cf$$

$$V_{100year,1hour} = 2.06 * 6.5 * \left(\frac{60\ sec}{1\ \text{min}}\right) = 803.4\ cf$$

DA-B

$$C = \begin{cases} 0.90(a_i + \dfrac{(I - Fp)ap}{I}, for\ I\ greater\ than\ Fpi \\ 0.90ai, for\ I\ less\ than\ or\ equal\ to\ Fp \end{cases}$$

Ai (ratio of impervious): 1.0

Ap (ratio of pervious): 0

I (intensity): 0.95 in/hr

I 100yr (intensity): 1.6 in/hr

Fp(per P-2 in Geotech report): 1.03 in/hr

$C_{10yr} = 0.90(1) = 0.9$

$$C_{100yr} = 0.90(1 + \frac{(1.6 - 0.5)0}{1.6} = 0.9$$

Peak Flows:

$$Q_{10yr,1hour} = CiA = (0.9) * \left(0.95\frac{in}{hr}\right) * (1.083) = 0.92\ cfs$$

$$Q_{100\ yr,1\ hour} = CiA = (0.9) * \left(1.6\frac{in}{hr}\right) * (1.083) = 1.55\ cfs$$

Time of Concentration: Tc= 4 min

See plate D-3 for Time of Concentration results.

Runoff Volume:

Runoff Volume:$V_n = Qn * Tc * \left(\frac{60\ sec}{1\ \text{min}}\right)$

$$V_{10year,1hour} = 0.92 * 4 * \left(\frac{60\ sec}{1\ \text{min}}\right) = 220.8\ cf$$

$$V_{100year,1hour} = 1.55 * 4 * \left(\frac{60\ sec}{1\ \text{min}}\right) = 372\ cf$$

DA-C

$$C = \begin{cases} 0.90(a_i + \frac{(I - Fp)ap}{I}, for\ I\ greater\ than\ Fpi \\ 0.90ai, for\ I\ less\ than\ or\ equal\ to\ Fp \end{cases}$$

Ai (ratio of impervious): 0

Ap (ratio of pervious): 1

I (intensity): 0.95 in/hr

I 100yr (intensity): 1.6 in/hr

Fp(per P-1 in Geotech report): 0.5 in/hr

$$C_{10yr} = 0.90(0 + \frac{(0.95 - 0.5)1}{0.95} = 0.43$$

$$C_{100yr} = 0.90(0 + \frac{(1.6 - 0.5)1}{1.6} = 0.62$$

Peak Flows:

$$Q_{10yr,1hour} = CiA = (0.43) * \left(0.95 \frac{in}{hr}\right) * (2.769) = 1.13\ \ cfs$$

$$Q_{100\ yr,1\ hour} = CiA = (0.62) * \left(1.6 \frac{in}{hr}\right) * (2.769) = 2.74\ cfs$$

Time of Concentration: Tc= 12.5 min

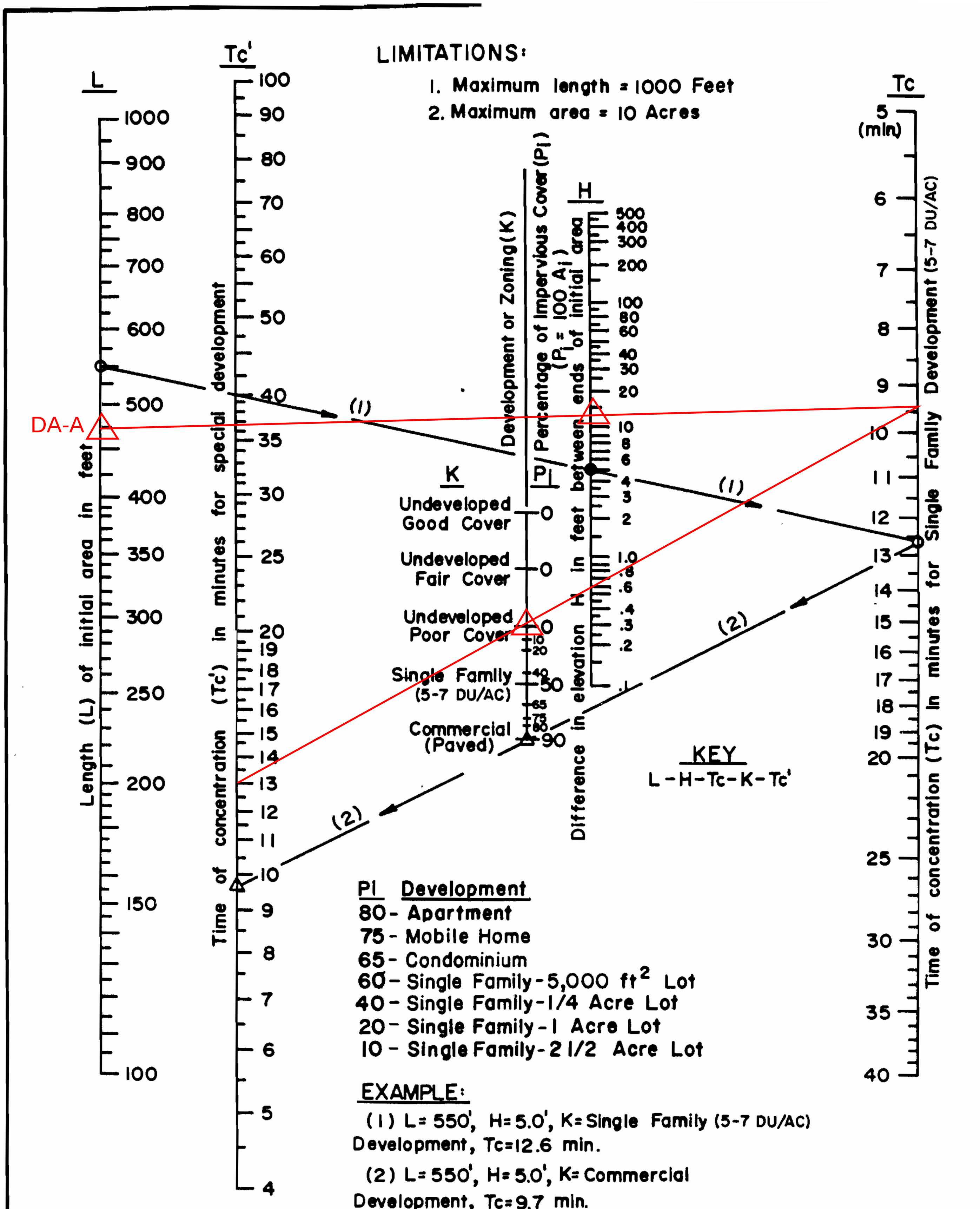
See plate D-3 for Time of Concentration results.

Runoff Volume:

Runoff Volume: $V_n = Qn * Tc * \left(\frac{60\ sec}{1\ \text{min}}\right)$

$$V_{10year,1hour} = 1.13 * 12.5 * \left(\frac{60\ sec}{1\ \text{min}}\right) = 847.5\ cf$$

$$V_{100year,1hour} = 2.74 * 12.5 * \left(\frac{60\ sec}{1\ \text{min}}\right) = 2{,}055\ cf$$

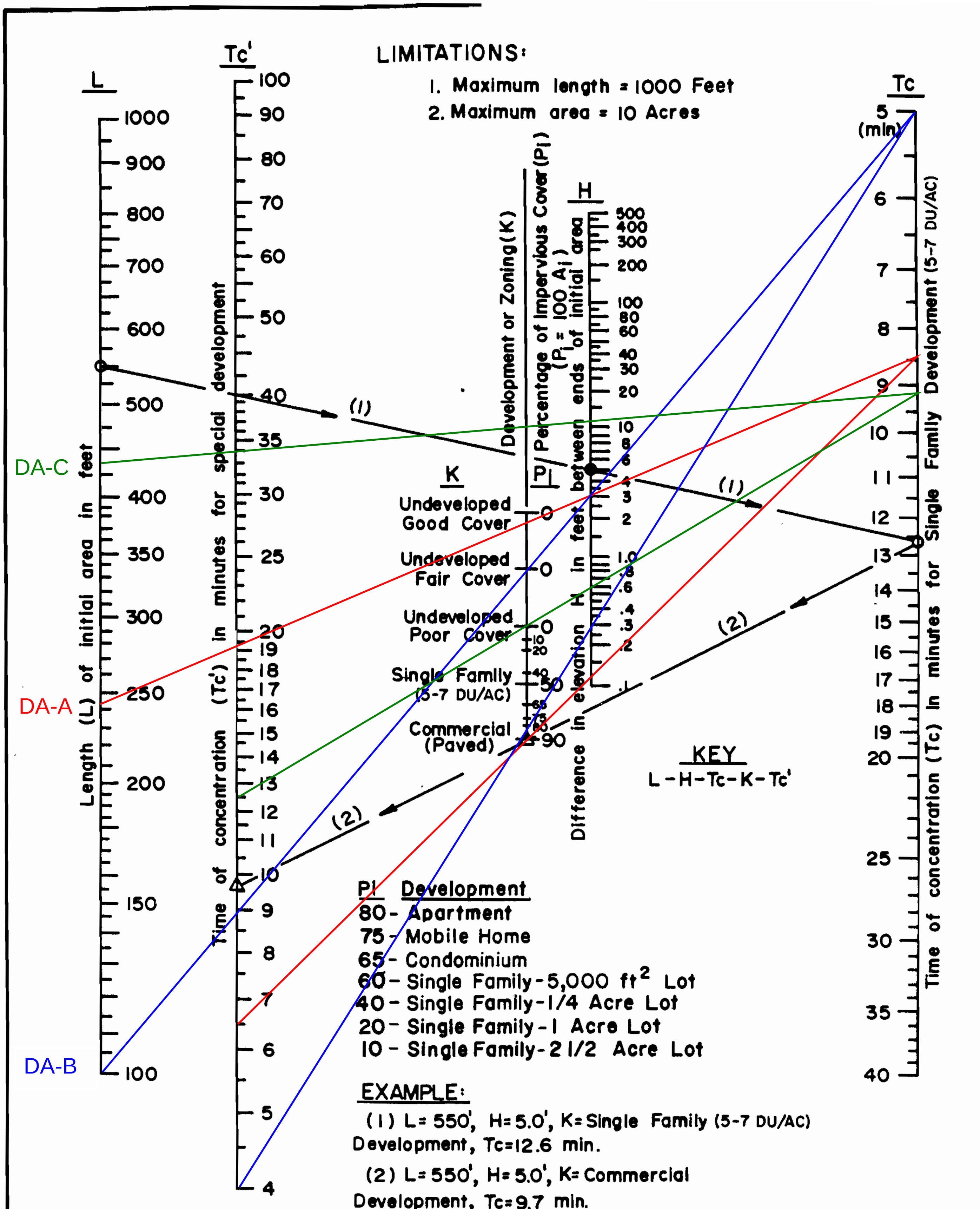



**SAN BERNARDINO COUNTY**

**HYDROLOGY MANUAL**

**TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA**

Figure D-1




**SAN BERNARDINO COUNTY**

HYDROLOGY MANUAL

TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA

Figure D-1

# Appendix D-References

SECTION D

# RATIONAL METHOD

## D.1. RATIONAL METHOD EQUATION

The rational method was originally developed to estimate peak discharges from small (less then one square mile) urban and developed areas and its use should normally be limited to those conditions. The rational method equation relates rainfall intensity, a runoff coefficient, and drainage area size to the peak runoff from the drainage area. This relationship is expressed by the equation:

$$Q = CIA \tag{D.1}$$

where

| | | |
|---|---|---|
| Q | = | the peak discharge in cubic feet per second (cfs) |
| C | = | a runoff coefficient representing the ratio of runoff depth to rainfall depth (dimensionless) |
| I | = | the time-averaged rainfall intensity for a storm duration equal to the time of concentration (inches/hr) |
| A | = | drainage area (acres) |

The values of the runoff coefficient (C) and the rainfall intensity (I) are based on a study of drainage area characteristics such as type and condition of the runoff surfaces and the time of concentration. These factors and the limitations of the rational method equation are discussed in the following sections. Drainage area (A) may be determined by planimetering a suitable topographic map of the project area.

Data required for the computation of peak discharge by the rational method are: (i) rainfall intensity (I) for a storm of specified duration and selected design frequency; (ii) drainage area characteristics of size (A), shape, slope; and (iii) a runoff coefficient (C).

## D.2. LIMITATIONS OF THE RATIONAL METHOD

The validity of the relationship expressed by the rational method equation holds true only if certain assumptions are reasonably correct and limitations of the method are observed. Two basic assumptions are that (i) the frequency of a storm runoff is the same as the frequency of the rainfall producing this runoff; i.e., a 25-year recurrence interval rainfall will provide a 25-year recurrence interval storm runoff, and (ii) that the peak runoff occurs when all parts of the drainage area are contributing to the runoff.

The rational method equation is only applicable where the rainfall intensity (I) can be assumed to be uniformly distributed over the drainage area at a uniform rate throughout the duration of the storm. This condition is generally assumed to reasonably apply to small areas of less than 640 acres. Beyond this limit, the rainfall distribution may vary considerably from the point values given in rainfall isohyetal maps and the rational method equation may be inappropriate.

The selection of the runoff coefficient (C) is another major limitation for the rational method equation. For small urban and developed areas the runoff coefficient can be reasonably well estimated from field and aerial photo studies. For larger areas where the determination of the runoff coefficient is to be based on vegetation type, cover density, the infiltration capacity of the ground surface, and the slope of the drainage area, an estimate of the runoff coefficient may be subject to a much greater error due to the variability of the drainage area characteristics. Rainfall losses due to evaporation, transpiration, depression and channel storage are inadequately evaluated, and may appreciably affect the estimate of the watershed peak rate of runoff, especially in natural cover and desert catchment areas. The effects of depth-area-duration (or depth-area) factors are not accounted for in the simple intensity-duration curve used for rational method studies. For large drainage areas, the absence of depth-area adjustments can result in significant differences in the estimate of the average depth of catchment point rainfalls.

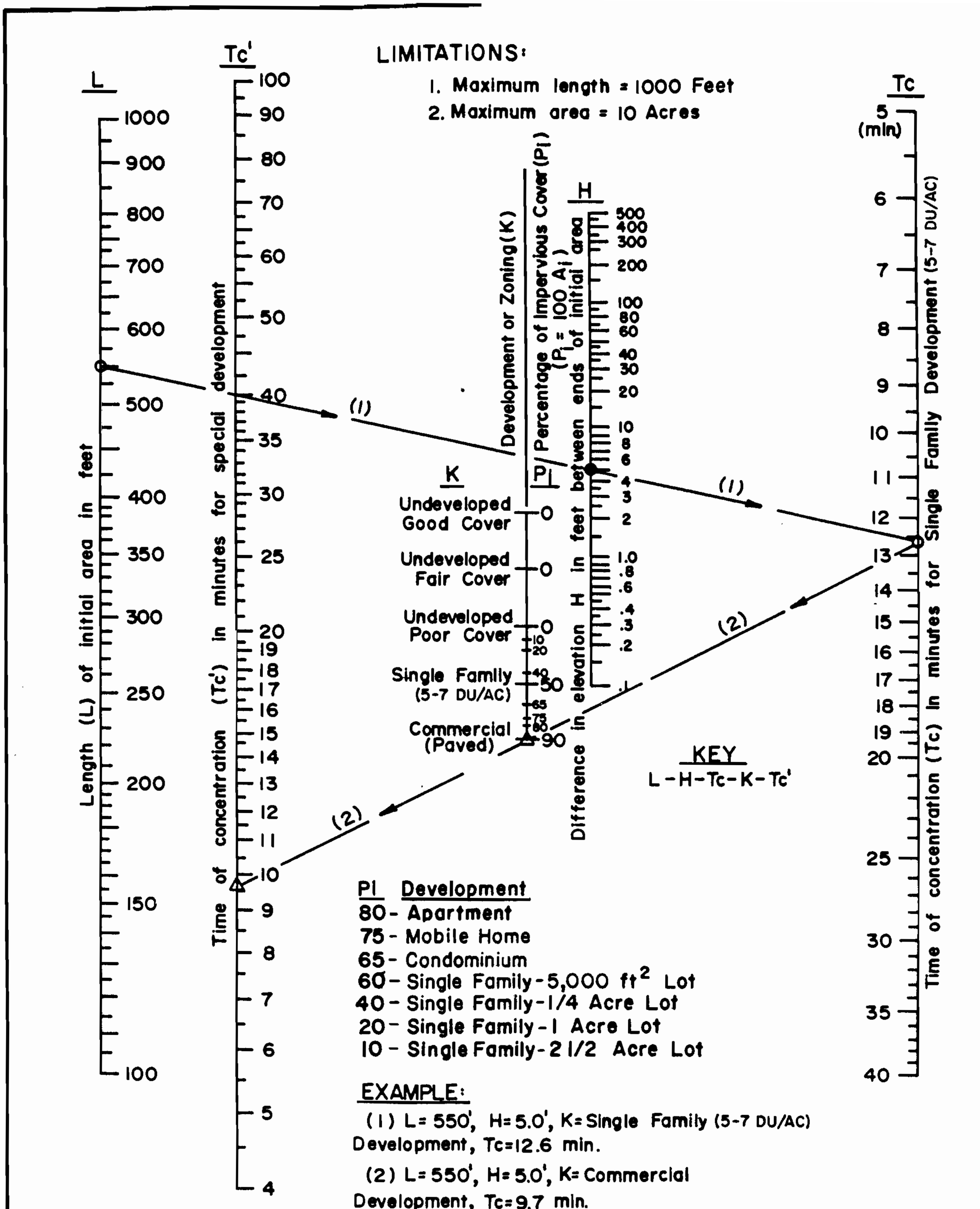
The above limitations indicate that an estimate of the peak rate of runoff becomes less reliable as the drainage area becomes larger and the rational method equation should generally not be used for drainage areas larger than 640 acres.

## D.3. CRITICAL DURATION (TIME OF CONCENTRATION)

The critical duration of the storm rainfall required in the rational method equation is based on the time of concentration of the drainage area.

The time of concentration (Tc) is defined as the interval of time (in minutes) required for the flow at a given point to become a maximum under a uniform rainfall intensity. Often this occurs when all parts of the drainage area are contributing to the flow. Generally, the time of concentration is the interval of time from the beginning of rainfall for water from the hydraulically most remote portion of the drainage area to reach the point of concentration; e.g., the inlet of the drainage structure. The time of concentration is a function of many variables including the length of the flow path from the most remote point of an area to the concentration point, the slope and other characteristics of natural and improved channels in the area, the loss rate characteristics of the soil, and the extent and type of development.

For rational method studies based on this manual, the time of concentration for an initial subarea may be estimated from the nomograph of Figure D-1. The time of concentration for the next downstream subarea is computed by adding to the initial Tc, the time required for the computed peak flow to travel to the next concentration point. Time of concentration is computed for each subsequent subarea by computing the runoff peak flow rate travel time between subareas and adding to the cumulative sum.




**SAN BERNARDINO COUNTY**

**HYDROLOGY MANUAL**

**TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA**

Figure D-1

When the flow is concentrated in curb and gutters, drainage channels or conduits, the flow velocity may be estimated by the well-known Manning's equation

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{D.2}$$

where

| | | |
|---|---|---|
| V | = | mean velocity (fps) |
| n | = | Manning coefficient of roughness |
| R | = | hydraulic radius (feet) |
| S | = | energy slope which equals the conduit invert slope for uniform flow |

The travel time will then be the flow distance divided by the velocity of flow.

Computations of travel time through subareas which continually add runoff to the peak flow (e.g., streetflow) should be based on the average peak flow through the subarea. This average peak flow is generally a simple average of the peak flow rates estimated at the upstream and downstream points of the subarea.

The initial subarea Tc estimation often is the most significant factor leading to the Tc computation of a watershed. Small development studies typically utilize only initial subarea estimations due to the small subarea sizes. Larger study areas generally show high sensitivity to the initial subarea Tc. Consequently, judgment is needed when developing initial subarea Tc estimates. The nomograph of Figure D-1 is based on the Kirpich formula and relates an initial subarea Tc to subarea slope and development type. It is assumed in the nomograph that overland flow effects dominate the travel time hydraulics.

It is noted that the Tc computation procedure is based upon the summation of an initial subarea time of concentration with the several travel times estimated by normal depth flow-velocities of the peak flow rates through subsequent subareas.

## D.4. INTENSITY-DURATION CURVES

Rainfall intensity (I) is determined using intensity-duration curves which are appropriate for the study watershed.

San Bernardino County has prepared isohyetal maps corresponding to 10-year 1-hour and 100-year 1-hour return frequency precipitation. Point rainfall for intermediate return periods can be determined from Figure D-2. Intensity duration curves for a particular area can be developed using the log-log paper of Figure D-3, plotting the 1-hour point rainfall value for the desired return period, and drawing a straight line through the 1-hour value parallel to the required slope. The slope of the intensity duration curve is assumed to be 0.6 for watersheds in the southwest portion of the County. For desert and mountain watersheds, the slope of the intensity duration curves is assumed to be 0.7. These slope values may be modified if rainfall data record analysis indicates that such modifications are appropriate. Any modifications of the slope values must be approved by the County prior to submittal of a study for County review.

## D.5. RUNOFF COEFFICIENT

The runoff coefficient (C) is the ratio of rate of runoff to the rate of rainfall at an average intensity (I) when the total drainage area is contributing. The selection of the runoff coefficient depends on rainfall intensity, drainage area slope, type and amount of vegetative cover, infiltration capacity of the ground surface, and various other factors.

Since one acre-inch/hour is equal to 1.008 cfs, the rational formula is used to estimate a peak flowrate in cfs. The runoff coefficient is assumed to be a function of the impervious and pervious area fractions, an infiltration rate,



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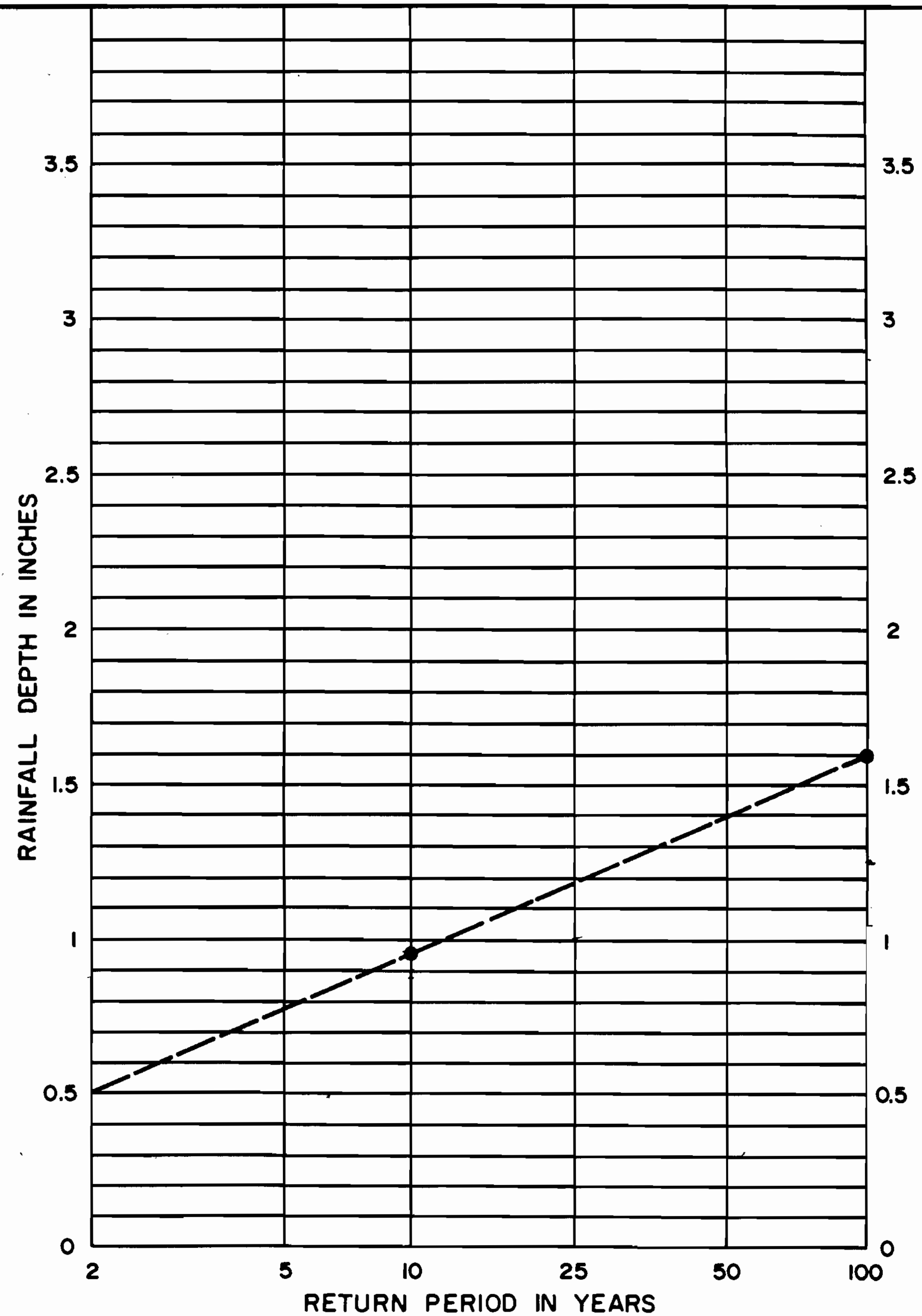


NOTE:

1. FOR INTERMEDIATE RETURN PERIODS PLOT 10-YEAR AND 100-YEAR ONE HOUR VALUES FROM MAPS, THEN CONNECT POINTS AND READ VALUE FOR DESIRED RETURN PERIOD. FOR EXAMPLE GIVEN 10-YEAR ONE HOUR = 0.95" AND 100-YEAR CNE HOUR = 1.60", 25-YEAR ONE HOUR = 1.18".

REFERENCE: NOAA ATLAS 2, VOLUME XI - CAL., 1973

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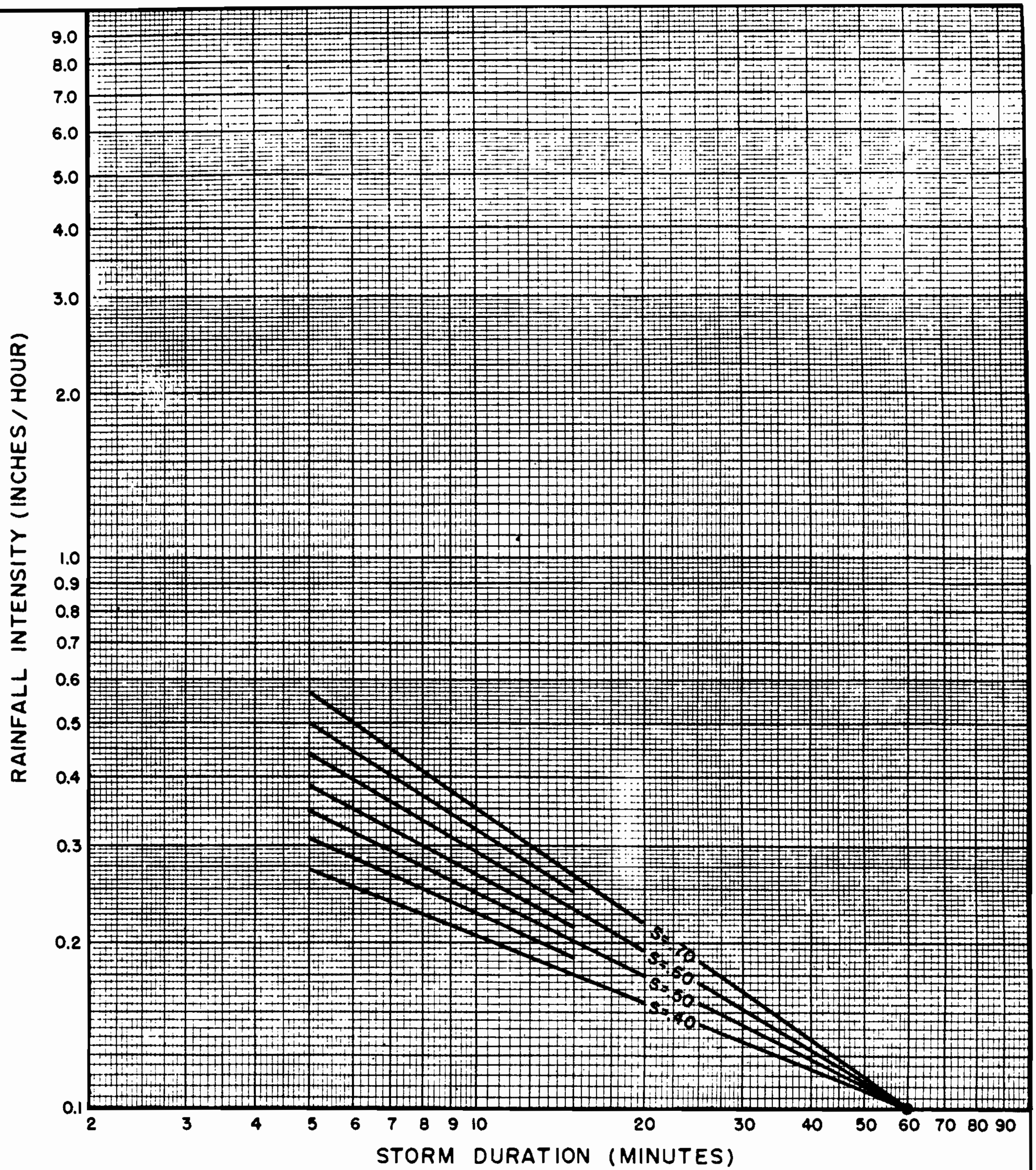
HYDROLOGY MANUAL

RAINFALL DEPTH VERSUS RETURN PERIOD FOR PARTIAL DURATION SERIES




DESIGN STORM FREQUENCY =_____ YEARS

ONE HOUR POINT RAINFALL =_____ INCHES

LOG-LOG SLOPE =_____

PROJECT LOCATION =____________________

**SAN BERNARDINO COUNTY**

HYDROLOGY MANUAL

INTENSITY - DURATION CURVES CALCULATION SHEET

$F_p$, for the pervious area, and the effects of watershed detention. Runoff coefficient curves are developed using the relationship:

$$C = \begin{cases} 0.90\,(a_i + \frac{(I - F_p)a_p}{I}), \text{ for I greater than } F_p; \\ 0.90\,a_i, \text{ for I less than or equal to } F_p \end{cases} \quad (D.3)$$

where the proportion factor of 0.90 is a calibration constant determined by an average fit between the rational method and design storm unit hydrograph (see Section E) peak flow rate estimates, and where

| | | |
|---|---|---|
| $C$ | = | runoff coefficient |
| $I$ | = | rainfall intensity (inches/hour) |
| $F_p$ | = | infiltration rate for pervious areas (inches/hour) (see section C.6.4) |
| $a_i$ | = | ratio of impervious area to total area (decimal fraction) |
| $a_p$ | = | ratio of pervious area to total area (decimal fraction), ($a_p = 1 - a_i$) |

## D.6. PEAK FLOW RATE FORMULA

Combining Equations (D.1) and (D.3), the peak flow estimate for Q is written in simpler terms by

$$Q = .90\,(I - F_m)A \quad (D.4)$$

where $F_m = a_pF_p$ (see section C.6.5), and where in (D.4) it is understood that I is greater than $F_p$ (otherwise $Q = .90\,a_iIA$).

In (D.4), $F_m$ represents the loss rate for the total watershed tributary to the point of concentration. Should the tributary area contain several runoff surfaces, an area-averaged $F_m$ is calculated. Table D.1 illustrates such an area-averaged $F_m$ computation.

GEOTECHNICAL • ENVIRONMENTAL • GEOLOGY • MATERIALS TESTING & INSPECTION • FORENSIC • LABORATORY

SALEM engineering group, inc.

# LIMITED GEOTECHNICAL ENGINEERING INVESTIGATION

## PROPOSED LOADING DOCKS AND PARKING LOT
## 6730 SANTA FE AVENUE E
## HESPERIA, CALIFORNIA

**SALEM PROJECT NO. 3-223-0381**
**MAY 18, 2023**

***PREPARED FOR:***

**MR. GREG REITZ**
**CREDE GROUP**
**18301 VON KARMAN AVENUE, SUITE 510**
**IRVINE, CA 92612**

***PREPARED BY:***

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May 18, 2023 **Project No. 3-223-0381**

Mr. Greg Reitz
**Crede Group**
18301 Von Karman, Suite 510
Irvine, CA 92612

**SUBJECT: LIMITED GEOTECHNICAL ENGINEERING INVESTIGATION**
**PROPOSED LOADING DOCKS AND PARKING LOT**
**6730 SANTA FE AVENUE E**
**HESPERIA, CALIFORNIA**

Dear Mr. Reitz:

At your request and authorization, SALEM Engineering Group, Inc. (SALEM) has prepared this Limited Geotechnical Engineering Investigation report for the Proposed Loading Docks and Parking Lot to be located at the subject site.

The accompanying report presents our findings, conclusions, and recommendations regarding the geotechnical aspects of designing and constructing the project as presently proposed. In our opinion, the proposed project is feasible from a geotechnical viewpoint provided our recommendations are incorporated into the design and construction of the project.

We appreciate the opportunity to assist you with this project. Should you have questions regarding this report or need additional information, please contact the undersigned at (909) 980-6455.

Respectfully Submitted,

**SALEM ENGINEERING GROUP, INC.**

Ibrahim Foud Ibrahim, PE, GE
Senior Managing Engineer
RCE 86724, GE 3222

Clarence Jiang, GE
Senior Geotechnical Engineer
RGE 2477

# TABLE OF CONTENTS

1. PURPOSE AND SCOPE ... 1
2. PROJECT DESCRIPTION ... 1
3. SITE LOCATION AND DESCRIPTION ... 2
4. FIELD EXPLORATION ... 2
5. LABORATORY TESTING ... 2
6. SOIL AND GROUNDWATER CONDITIONS ... 3
   - 6.1 Subsurface Conditions ... 3
   - 6.2 Groundwater ... 3
   - 6.3 Soil Corrosion Screening ... 3
   - 6.4 Percolation Testing ... 4
7. CONCLUSIONS AND RECOMMENDATIONS ... 5
   - 7.1 General ... 5
   - 7.2 Seismic Design Criteria ... 7
   - 7.3 Soil and Excavation Characteristics ... 8
   - 7.4 Materials for Fill ... 8
   - 7.5 Grading ... 9
   - 7.6 Shallow Foundations for loading docks ... 12
   - 7.7 Exterior Concrete Slabs ... 13
   - 7.8 Lateral Earth Pressures and Frictional Resistance ... 14
   - 7.9 Retaining Walls ... 15
   - 7.10 Temporary Excavations ... 15
   - 7.11 Underground Utilities ... 16
   - 7.12 Surface Drainage ... 17
   - 7.13 Pavement Design ... 18
8. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING ... 18
   - 8.1 Plan and Specification Review ... 18
   - 8.2 Construction Observation and Testing Services ... 19
9. LIMITATIONS AND CHANGED CONDITIONS ... 19

**TABLE OF CONTENTS (cont.)**

FIGURES
Figure 1, Vicinity Map
Figure 2, Site Plan

APPENDIX A – FIELD INVESTIGATION
Figures A-1 through A-4, Logs of Exploratory Soil Borings B-1 through B-4
Percolation Test Results, P-1 and P-2

APPENDIX B – LABORATORY TESTING
Direct Shear Results
Gradation Results
Corrosivity Results
Maximum Density and Optimum Moisture Results

APPENDIX C – EARTHWORK AND PAVEMENT SPECIFICATIONS

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# LIMITED GEOTECHNICAL ENGINEERING INVESTIGATION
# PROPOSED LOADING DOCKS AND PARKING LOT
# 6730 SANTA FE AVENUE E
# HESPERIA, CALIFORNIA

## 1. PURPOSE AND SCOPE

This report presents the results of our Limited Geotechnical Engineering Investigation for the Proposed Loading Docks and Parking Lot to be located at 6730 Santa Fe Avenue E in the city of Hesperia, California (see Figure 1, Vicinity Map). The purpose of our limited geotechnical engineering investigation was to investigate the subsurface conditions encountered at the site, and provide conclusions and recommendations relative to the geotechnical aspects of constructing the project as presently proposed.

The scope of this investigation included a field exploration, percolation testing, laboratory testing, engineering analysis, and the preparation of this report. Our field exploration was performed on May 8, 2023, and included drilling of four (4) small-diameter soil borings to a maximum depth of 10 feet at the site. Additionally, two (2) percolation tests were performed at depths of approximately 3 and 4¾ feet below ground surface to determine the infiltration rates. The approximate locations of the soil borings and percolation tests are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation and exploratory boring logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical properties for engineering analyses. Appendix B presents the laboratory test results in tabular and graphic format. The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. If project details vary significantly from those described herein, SALEM should be contacted to determine the necessity for review and possible revision of this report. Earthwork and Pavement Specifications are presented in Appendix C. If text of the report conflict with the specifications in Appendix C, the recommendations in the text of the report have precedence.

## 2. PROJECT DESCRIPTION

Based on the site plans provided to us, we understand that the proposed development of the site will include construction of two (2) concrete loading docks and an asphaltic concrete (AC) parking lot. Each loading dock will have 4 depressed loading bays. A loading dock, 80 feet by 100 feet, will be located on the northeast side of the existing building, and another loading dock, 85 feet by 100 feet, will be located at the southeast end of the existing building. The parking lot will be located to the east of the existing building.

As the site area is relatively flat with no major changes in grade, we anticipate that cuts and fills during earthwork will be limited to providing positive site drainage. In the event that changes occur in the nature or design of the project, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions of our report are modified. The site configuration and locations of proposed improvements are shown on the Site Plan, Figure 2.

## 3. SITE LOCATION AND DESCRIPTION

The site is located northwest of the intersection of Jenny Street and Santa Fe Avenue E in the city of Hesperia, California (see Vicinity Map, Figure 1). The address of the site is 6730 Santa Fe Avenue E.

The subject site is irregular in shape and encompasses approximately 6.11 acres. The northern half of the site is vacant and will not be developed. The southern half of the site is occupied by a 21,831 square-foot sheet metal 67industrial building surrounded by associated asphalt concrete pavement and unpaved/non-landscaped land. An annex structure currently exists at the east corner of the industrial building. A steel frame structure is located in the north corner of the southern half of the site. A chain-linked fence surrounds the site. The southern half of the site is relatively flat with no major changes in grade.

## 4. FIELD EXPLORATION

Our field exploration consisted of site surface reconnaissance and subsurface exploration. The exploratory test borings (B-1 through B-4) were drilled on May 8, 2023, and were advanced with a 3-inch diameter hand auger. Exterior asphalt for B-1 and B-4 was cored using a coring machine prior to drilling. The test borings were extended to a maximum depth of approximately 10 feet below existing grade. Drilling was limited to 8 feet in boring B-4 due to auger refusal on hard soil conditions. The approximate locations of our test borings are shown on the Site Plan, Figure 2.

The materials encountered in the test borings were visually classified in the field, and logs were recorded by a field engineer and stratification lines were approximated on the basis of observations made at the time of drilling. Visual classification of the materials encountered in the test borings were generally made in accordance with the Unified Soil Classification System (ASTM D2488).

A soil classification chart and key to sampling is presented on the Unified Soil Classification Chart, in Appendix "A." The logs of the test borings are presented in Appendix "A." The Boring Logs include the soil type, color, moisture content, dry density, and the applicable Unified Soil Classification System symbol. The location of the test borings were determined by measuring from features shown on the Site Plan, provided to us. Hence, accuracy can be implied only to the degree that this method warrants. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted. Soil samples were obtained from the test borings at the depths shown on the logs of borings. Bag samples were recovered and placed in a sealed bag to preserve their natural moisture content. Upon completion of the exploration, the borings were backfilled with soil cuttings, and then patched with concrete patch (where applicable),

## 5. LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory-testing program was formulated with emphasis on the evaluation of natural moisture, density, shear strength, maximum density and optimum moisture determination, and gradation of the materials encountered.

In addition, chemical tests were performed to evaluate the corrosivity of the soils to buried concrete and metal. Details of the laboratory test program and the results of laboratory test are summarized in Appendix "B." This information, along with the field observations, was used to prepare the final boring logs in Appendix "A."

## 6. SOIL AND GROUNDWATER CONDITIONS

### 6.1 Subsurface Conditions

The subsurface conditions encountered appear typical of those found in the geologic region of the site. In general, the soils within the depth of our borings consisted predominately of silty sand. The exterior surface within our test borings B-1 and B-4 consisted of approximately 2 to 3¼ inches of asphalt concrete (AC) underlain by approximately 2 to 3¼ inches of aggregate base (AB).

Fill soils may be present on site between our boring locations since the site was graded for the current development. The consistency of the fills should be verified during site construction. Prior to fill placement, Salem Engineering Group, Inc. should inspect the bottom of the excavation to verify no additional excavation will be required. Verification of the fill soils and the extent of fill should be determined during site grading.

The soils were classified in the field during the drilling and sampling operations. The stratification lines were approximated by the field engineer on the basis of observations made at the time of drilling. The actual boundaries between different soil types may be gradual and soil conditions may vary. For a more detailed description of the materials encountered, the Boring Logs in Appendix "A" should be consulted. The Boring Logs include the soil type, color, moisture content, and the applicable Unified Soil Classification System symbol. The locations of the test borings were determined by measuring from feature shown on the Site Plan provided to us. Hence, accuracy can be implied only to the degree that this method warrants.

### 6.2 Groundwater

The test boring locations were checked for the presence of groundwater during and after the drilling operations. Free groundwater was not encountered during our investigation. Based on regional groundwater data near the site vicinity, the historically highest groundwater depth is estimated to be greater than 50 feet below ground surface. It should be recognized that water table elevations may fluctuate with time, being dependent upon seasonal precipitation, irrigation, land use, localized pumping, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

### 6.3 Soil Corrosion Screening

Excessive sulfate in either the soil or native water may result in an adverse reaction between the cement in concrete and the soil. The 2014 Edition of ACI 318 (ACI 318) has established criteria for evaluation of sulfate and chloride levels and how they relate to cement reactivity with soil and/or water.

A soil sample was obtained from the project site and was tested for the evaluation of the potential for concrete deterioration or steel corrosion due to attack by soil-borne soluble salts and soluble chloride. The water-soluble sulfate concentration in the saturation extract from the soil sample was detected to be less than 807 mg/kg. ACI 318 Tables 19.3.1.1 and 19.3.2.1 outline exposure categories, classes, and concrete requirements by exposure class. ACI 318 requirements for site concrete based upon soluble sulfate are summarized in Table 6.3 below.

**TABLE 6.3**
**WATER SOLUBLE SULFATE EXPOSURE REQUIREMENTS**

| Water-Soluble Sulfate ($SO_4$) in Soil, %by Weight | Exposure Severity | Exposure Class | Maximum w/cm Ratio | Min. Concrete Compressive Strength | Cementitious Materials Type |
|---|---|---|---|---|---|
| 0.0807 | Not Severe | S0 | N/A | 2,500 psi | No Restriction |

The water-soluble chloride concentration detected in saturation extract from the soil samples was 32 mg/kg. This level of chloride concentration is considered to be mildly corrosive. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, applicable manufacturer's recommendations for corrosion protection of buried metal pipe be closely followed.

## 6.4 Percolation Testing

Two percolation tests (P-1 and P-2) were performed. Results of the falling head tests are presented in the attachments to this report. The approximate locations of the percolation tests are shown on the attached Site Plan, Figure 2.

The boreholes were advanced to the depths shown on the percolation test worksheets. The holes were pre-saturated before percolation testing commenced. Percolation rates were measured by filling the test holes with clean water and measuring the water drops at a certain time interval. The difference in the percolation rates are reflected by the varied type of soil materials at the bottom of the test holes. The test results are shown on the table below.

**TABLE 6.4**
**PERCOLATION TEST RESULTS**

| Test No. | Depth (feet) | Tested Infiltration Rate[1] (inch/hour) | Factor of Safety[2] | Design Infiltration Rate (inch/hour) | Soil Type[3] |
|---|---|---|---|---|---|
| P-1 | 4¾ | **1.12** | **2.25** | **0.50** | Silty SAND (SM) |
| P-2 | 3 | **2.32** | **2.25** | **1.03** | Poorly graded SAND (SP) |

[1] **Tested infiltration Rate = ($\Delta$H 60 r) / ($\Delta$t(r + 2$H_{avg}$))**
[2] **Based on Worksheets H, $S_A$ = 1.5 and $S_B$ = 1.5**
[3] **At bottom of test hole.**

The FS should be verified by the civil engineer based on Worksheets H: Factor of Safety and Design Infiltration Rate and Worksheet provided in the San Bernardino County Stormwater Program, Technical Guidance Document for Water Quality Management Plans (WQMP).

The soil infiltration or percolation rates are based on tests conducted with clear water. The infiltration/percolation rates may vary with time as a result of soil clogging from water impurities. The soils may also become less permeable to impermeable if the soil is compacted. Thus, periodic maintenance consisting of clearing the bottom of the drainage system of clogged soils should be expected. The infiltration/percolation rate may become slower if the surrounding soil is wet or saturated due to prolonged rainfalls. Additional percolation tests should be conducted at bottom of the drainage system during construction to verify the infiltration/percolation rate.

The scope of our services did not include a groundwater study and was limited to the performance of percolation testing and soil profile description, and the submitted data only. Our services did not include those associated with septic system design. Neither did services include an Environmental Site Assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater, or atmosphere; or the presence of wetlands. Any statements, or absence of statements, in this report or on any boring logs regarding odors, unusual or suspicious items, or conditions observed, are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous and/or toxic assessment.

The geotechnical engineering information presented herein is based upon professional interpretation utilizing standard engineering practices. The work conducted through the course of this investigation, including the preparation of this report, has been performed in accordance with the generally accepted standards of geotechnical engineering practice, which existed in the geographic area at the time the report was written. No other warranty, express or implied, is made. Please be advised that when performing percolation testing services in relatively small diameter borings, that the testing may not fully model the actual full scale long term performance of a given site. This is particularly true where percolation test data is to be used in the design of large infiltration system such as may be proposed for the site.

The measured percolation rate includes dispersion of the water at the sidewalls of the boring as well as into the underlying soils. Subsurface conditions, including percolation rates, can change over time as fine-grained soils migrate. It is not warranted that such information and interpretation cannot be superseded by future geotechnical engineering developments. We emphasize that this report is valid for the project outlined above and should not be used for any other sites.

## 7. CONCLUSIONS AND RECOMMENDATIONS

### 7.1 General

7.1.1 Based upon the data collected during this investigation, and from a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed construction at the site as planned, provided the recommendations contained in this report are incorporated into the project design and construction. Conclusions and recommendations provided in this report are based on our review of available literature, analysis of data obtained from our field exploration and laboratory testing program, and our understanding of the proposed development at this time.

7.1.2 The primary geotechnical constraints identified in our investigation is the presence of potentially compressible soils at the site. Recommendations to mitigate the effects of these soils are provided in this report.

7.1.3 The scope of this investigation did not include subsurface exploration within the existing building and structure areas during field exploration. As such, subsurface soil conditions and materials present below the existing site structures are unknown and may be different than those noted within this report. The presence of potentially unacceptable fill materials, undocumented fill, and/or loose soil material that may be present below existing site features shall be taken into consideration. Our firm shall be present at the time of demolition activities to verify soil conditions are consistent with those identified as part of this investigation.

7.1.4 No significant fill soils were encountered during this investigation. Fill soils may be present on site between our boring locations since the site was graded for the current development. Verification of the fill soil and the extent of fill should be determined during site grading. Undocumented/uncompacted fill materials are not suitable to support any future structures and should be excavated and replaced with Engineered Fill. Prior to fill placement, SALEM should inspect the bottom of the excavation to verify the fill condition.

7.1.5 Site demolition activities shall include removal of all surface obstructions not intended to be incorporated into final site design. In addition, underground buried structures and/or utility lines encountered during demolition and construction should be properly removed and the resulting excavations backfilled with Engineered Fill. It is suspected that possible demolition activities of the existing structures may disturb the upper soils. After demolition activities, it is recommended that disturbed soils be removed and/or recompacted.

7.1.6 Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. The upper 4 to 6 inches of the soils containing vegetation, roots, and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. The stripped vegetation will not be suitable for use as Engineered Fill or within 5 feet of building pads, loading docks or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

7.1.7 SALEM shall review the project grading and foundation plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required. If SALEM is not provided plans and specifications for review, we cannot assume any responsibility for the future performance of the project.

7.1.8 SALEM shall be present at the site during site demolition and preparation to observe site clearing/demolition, preparation of exposed surfaces after clearing, and placement, treatment and compaction of fill material.

7.1.9 SALEM's observations should be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. Moisture content of footings and slab subgrade should be tested immediately prior to concrete placement. SALEM should observe foundation excavations prior to placement of reinforcing steel or concrete to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report.

### 7.2 Seismic Design Criteria

7.2.1 For seismic design of the structures, and in accordance with the seismic provisions of the 2022 CBC, our recommended parameters are shown below. These parameters were determined using California's Office of Statewide Health Planning and Development (OSHPD) Seismic Design Map Tool Website (https://seismicmaps.org/) in accordance with the 2022 CBC. The Site Class was determined based on the soils encountered during our field exploration.

**TABLE 7.2.1**
**SEISMIC DESIGN PARAMETERS**

| Seismic Item | Symbol | Value | ASCE 7-16 or 2022 CBC Reference |
|---|---|---|---|
| Site Coordinates (Datum = NAD 83) | | 34.3730 Lat<br>-117.3211 Lon | |
| Site Class | -- | D-Default | ASCE 7 Table 20.3 |
| Risk Category | -- | II | CBC Table 1604.5 |
| Site Coefficient for PGA | $F_{PGA}$ | 1.2 | ASCE 7 Table 11.8-1 |
| Peak Ground Acceleration (adjusted for Site Class effects) | $PGA_M$ | 0.685g | ASCE 7 Equation 11.8-1 |
| Seismic Design Category | SDC | D | ASCE 7 Table 11.6-1 & 2 |
| Mapped Spectral Acceleration (Short period - 0.2 sec) | $S_S$ | 1.5 g | CBC Figure 1613.2.1(1-10) |
| Mapped Spectral Acceleration (1.0 sec. period) | $S_1$ | 0.6 g | CBC Figure 1613.2.1(1-10) |
| Site Class Modified Site Coefficient | $F_a$ | 1.2 | CBC Table 1613.2.3(1) |
| Site Class Modified Site Coefficient | $F_v$ | *1.7 | CBC Table 1613.2.3(2) |
| MCE Spectral Response Acceleration (Short period - 0.2 sec) $S_{MS} = F_a S_S$ | $S_{MS}$ | 1.8 g | CBC Equation 16-20 |
| MCE Spectral Response Acceleration (1.0 sec. period) $S_{M1} = F_v S_1$ | $S_{M1}$ | *1.53 g | CBC Equation 16-21 |
| Design Spectral Response Acceleration $S_{DS}=⅔S_{MS}$ (short period - 0.2 sec) | $S_{DS}$ | 1.2 g | CBC Equation 16-22 |
| Design Spectral Response Acceleration $S_{D1}=⅔S_{M1}$ (1.0 sec. period) | $S_{D1}$ | *1.02 g | CBC Equation 16-23 |
| Short Term Transition Period ($S_{D1}/S_{DS}$), seconds | $T_S$ | 0.85 | ASCE 7-16, Section 11.4.6 |
| Long Period Transition Period (seconds) | $T_L$ | 12 | ASCE 7-16, Figure 22-14 |

*** Determined per ASCE Table 11.4-2 for use in calculating $T_S$ only.**

7.2.2 Site Specific Ground Motion Analysis was not included in the scope of this investigation. Per ASCE 11.4.8, structures on Site Class D with $S_1$ greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific motion analysis may not be required based on Exceptions listed in ASCE 11.4.8. The Structural Engineer should verify whether

Exception No. 2 of ASCE 7-16, Section 11.4.8, is valid for the site. In the event that a site specific ground motion analysis is required, SALEM should be contacted for these services.

7.2.3 Conformance to the criteria in the above table 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.

**7.3 Soil and Excavation Characteristics**

7.3.1 Based on the soil conditions encountered in our soil borings, the onsite soils can be excavated with moderate effort using conventional heavy-duty earthmoving equipment.

7.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements. Temporary excavations are further discussed in a later Section of this report.

7.3.3 The near surface soils identified as part of our investigation are, generally, slightly moist to moist due to the absorption characteristics of the soil. Earthwork operations may encounter very moist unstable soils which may require removal to a stable bottom. Exposed native soils exposed as part of site grading operations shall not be allowed to dry out and should be kept continuously moist prior to placement of subsequent fill.

**7.4 Materials for Fill**

7.4.1 Excavated soils generated from cut operations at the site are suitable for use as general Engineered Fill in structural areas provided they do not contain deleterious matter, debris, organic material, or rock material larger than 3 inches in maximum dimension.

7.4.2 Import soil shall be well-graded, slightly cohesive silty fine sand or sandy silt, with relatively impervious characteristics when compacted. A clean sand or very sandy soil is not acceptable for this purpose. This material should be approved by the Engineer prior to use and should typically possess the soil characteristics summarized below in Table 7.4.2.

**TABLE 7.4.2**
**IMPORT FILL REQUIREMENTS**

| | |
|---|---|
| Minimum Percent Passing No. 200 Sieve | 15 |
| Maximum Percent Passing No. 200 Sieve | 50 |
| Minimum Percent Passing No. 4 Sieve | 70 |
| Maximum Particle Size | 3" |
| Maximum Plasticity Index | 10 |
| Maximum CBC Expansion Index | 15 |

7.4.3 The preferred materials specified for Engineered Fill are suitable for most applications with the exception of exposure to erosion. Project site winterization and protection of exposed soils during the construction phase should be the sole responsibility of the Contractor, since they have complete control of the project site.

7.4.4 Proposed import materials should be sampled, tested, and approved by SALEM prior to its transportation to the site.

7.4.5 Environmental characteristics and corrosion potential of import soil materials should also be considered.

### 7.5 Grading

7.5.1 A representative of our firm shall be present during all site clearing and grading operations to test and observe earthwork construction. This testing and observation is an integral part of our service as acceptance of earthwork construction is dependent upon compaction of the material and the stability of the material. The Geotechnical Engineer may reject any material that does not meet compaction and stability requirements. Further recommendations of this report are predicated upon the assumption that earthwork construction will conform to recommendations set forth in this section as well as other portions of this report.

7.5.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance.

7.5.3 Site preparation should begin with removal of existing surface/subsurface structures, underground utilities (as required), any existing uncertified fill, and debris. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with Engineered Fill in accordance with the recommendations of this report.

7.5.4 Site demolition activities shall include removal of all surface obstructions not intended to be incorporated into final site design. In addition, underground buried structures and/or utility lines encountered during demolition and construction should be properly removed and the resulting excavations backfilled with Engineered Fill. After demolition activities, it is recommended that disturbed soils be removed and/or recompacted.

7.5.5 Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth to remove organic-rich topsoil. The upper 2 to 6 inches of the soils containing, vegetation, roots and other objectionable organic matter encountered at the time of grading should be stripped and removed from the surface. Deeper stripping may be required in localized areas. In addition, existing concrete and asphalt materials shall be removed from areas of proposed improvements and stockpiled separately from excavated soil material. The stripped vegetation, asphalt and concrete materials will not be suitable for use as Engineered Fill or within 5 feet of building pads, loading docks, or within pavement areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas or exported from the site.

7.5.6 Tree root systems in proposed improvement areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots greater than ½ inch in diameter.

Tree roots removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill of tree root excavations is not permitted until all exposed surfaces have been inspected and the Soils Engineer is present for the proper control of backfill placement and compaction. Burning in areas which are to receive fill materials shall not be permitted.

7.5.7 No significant fill soils were encountered in our test borings. Fill soil may be present onsite since the site was previously graded for the current development. Undocumented and uncompacted fill materials are not suitable to support any future structures and should be excavated and replaced with Engineered Fill. The actual depth of the overexcavation and recompaction should be determined by our field representative during construction.

7.5.8 To minimize post-construction soil movement and provide uniform support for the proposed loading docks, overexcavation and recompaction within the proposed loading dock areas should be performed to a minimum depth of **two (2) feet** below existing grade or **one (1) foot** below footing bottom, whichever is deeper. The overexcavation and recompaction should also extend laterally to a minimum of 3 feet beyond the outer edges of the proposed footings except in areas where lateral extension is restricted by existing footings.

7.5.9 Slot cuts, braced shorings or shields may be used for supporting vertical excavations near existing structures. Therefore, in order to comply with the local and state safety regulations, a properly designed and installed shoring system would be required to accomplish planned excavations and installation.

7.5.10 Within pavement areas, it is recommended that scarification, moisture conditioning, and recompaction be performed to at least **12 inches** below existing grade or finish grade, whichever is deeper. In addition, the upper 12 inches of final pavement subgrade – whether completed at-grade, by excavation, or by filling – should be uniformly moisture-conditioned to near the optimum moisture content and compacted to at least 95% relative compaction

7.5.11 Prior to placement of fill soils, the upper 10 to 12 inches of native subgrade soils should be scarified, moisture-conditioned to no less than optimum moisture content, and recompacted to a minimum of 95% of the maximum dry density based on ASTM Test Method D1557 latest edition.

7.5.12 All Engineered Fill (including scarified ground surfaces and backfill) should be placed in thin lifts to allow for adequate bonding and compaction (typically 6 to 8 inches in loose thickness).

7.5.13 Engineered Fill soils should be placed, moisture conditioned to no less than optimum moisture content, and compacted to at least 95% relative compaction.

7.5.14 An integral part of satisfactory fill placement is the stability of the placed lift of soil. If placed materials exhibit excessive instability as determined by a SALEM field representative, the lift will be considered unacceptable and shall be remedied prior to placement of additional fill material. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

7.5.15 Final pavement subgrade should be finished to a smooth, unyielding surface. We further recommend proof-rolling the subgrade with a loaded water truck (or similar equipment with high contact pressure) to verify the stability of the subgrade prior to placing aggregate base.

7.5.16 The most effective site preparation alternatives will depend on site conditions prior to grading. We should evaluate site conditions and provide supplemental recommendations immediately prior to grading, if necessary.

7.5.17 We do not anticipate groundwater or seepage to adversely affect construction if conducted during the drier months of the year (typically summer and fall). However, groundwater and soil moisture conditions could be significantly different during the wet season (typically winter and spring) as surface soils become wet; perched groundwater conditions may develop. Grading during this time period will increase the chances of encountering wet materials resulting in possible excavation and fill placement difficulties.

Project site winterization consisting of placement of aggregate base and protecting exposed soils during construction should be performed. If the construction schedule requires grading operations during the wet season, we can provide additional recommendations as conditions warrant.

7.5.18 Wet soils may become non conducive to site grading as the upper soils yield under the weight of the construction equipment. Therefore, mitigation measures should be performed for stabilization.

Typical remedial measures include: discing and aerating the soil during dry weather; mixing the soil with dryer materials; removing and replacing the soil with an approved fill material or placement of slurry, crushed rocks or aggregate base material; or mixing the soil with an approved lime or cement product.

The most common remedial measure of stabilizing the bottom of the excavation due to wet soil condition is to reduce the moisture of the soil to near the optimum moisture content by having the subgrade soils scarified and aerated or mixed with drier soils prior to compacting. However, the drying process may require an extended period of time and delay the construction operation.

To expedite the stabilizing process, slurry or crushed rock may be utilized for stabilization provided this method is approved by the owner for the cost purpose. If the use of slurry or crushed rock is considered, it is recommended that the upper soft and wet soils be replaced by 6 to 24 inches of 2-sack slurry or ¾-inch to 1-inch crushed rocks. The thickness of the slurry or rock layer depends on the severity of the soil instability. The recommended 6 to 24 inches of slurry or crushed rock material will provide a stable platform. It is further recommended that lighter compaction equipment be utilized for compacting the crushed rock.

A layer of geofabric is recommended to be placed on top of the compacted crushed rock to minimize migration of soil particles into the voids of the crushed rock, resulting in soil movement. Although it is not required, the use of geogrid (e.g. Tensar NX750) below the slurry or crushed rock will enhance stability and reduce the required thickness of crushed rock necessary for stabilization. Our firm should be consulted prior to implementing remedial measures to provide appropriate recommendations.

### 7.6 Shallow Foundations for loading docks

7.6.1 The site is suitable for use of conventional shallow foundations consisting of continuous footings and isolated pad footings bearing in properly compacted Engineered Fill.

7.6.2 The bearing wall footings considered for the structure should be continuous with a minimum width of 15 inches and extend to a minimum depth of 18 inches below the lowest adjacent soil grade. Isolated column footings should have a minimum width of 24 inches and extend a minimum depth of 18 inches below the lowest adjacent soil grade. Footing depth should be measured at the time of footing trench excavation not to include any future material (e.g. base, concrete, asphalt, etc.) over the subgrade.

7.6.3 The bottom of footing excavations should be maintained free of loose and disturbed soil. Footing concrete should be placed into a neat excavation.

7.6.4 New foundations planned directly adjacent to existing foundations should extend at a minimum to the bottom of new foundations or the depths specified above, whichever is greater

7.6.5 Footings proportioned as recommended above may be designed for the maximum allowable soil bearing pressures shown in the table below.

| Loading Condition | Allowable Bearing |
|---|---|
| Dead Load Only | 2,000 psf |
| Dead-Plus-Live Load | 2,500 psf |
| Total Load, Including Wind or Seismic Loads | 3,325 psf |

7.6.6 For design purposes, total settlement due to static and seismic loadings on the order of 1½ inches may be assumed for shallow footings. Differential settlement due to static and seismic loadings, along a 30-foot exterior wall footing or between adjoining column footings, should be ¾ inches, producing an angular distortion of 0.002. Most of the settlement is expected to occur during construction as the loads are applied. However, additional post-construction settlement may occur if the foundation soils are flooded or saturated. The footing excavations should not be allowed to dry out any time prior to pouring concrete.

7.6.7 Resistance to lateral footing displacement can be computed using an allowable coefficient of friction factor of 0.45 acting between the base of foundations and the supporting subgrade.

7.6.8 Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 350 pounds per cubic foot acting against the appropriate vertical native footing faces. The frictional and passive resistance of the soil may be combined provided that a 50 percent reduction of the frictional resistance factor is used when determining the total lateral resistance. An increase of one-third is permitted when using the alternate load combination that includes wind or earthquake loads.

7.6.9 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

7.6.10 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Prior to placing rebar reinforcement, foundation excavations should be evaluated by a representative of SALEM for appropriate support characteristics and moisture content. Moisture conditioning may be required for the materials exposed at footing bottom, particularly if foundation excavations are left open for an extended period.

### 7.7 Exterior Concrete Slabs

7.7.1 The upper 24 inches of the slab subgrade should be recompacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557, and the slab should be underlain by at least 6 inches of crushed aggregate base (CAB) compacted to a minimum relative compaction of 95 percent.

7.7.2 Slabs should have a minimum thickness of 5 inches, and a minimum compressive strength of 4,000 psi. Slabs should be reinforced as a minimum with No. 4 reinforcement bars at 18 inches on center, each way. Thicker slabs and/or additional reinforcement may be required by the structural engineer based on the anticipated loading.

7.7.3 Concrete slabs may be designed utilizing an allowable bearing pressure of 1,000 psf for dead-plus-live loads. This value may be increased by one-third for short duration loads, such as wind or seismic.

7.7.4 The subgrade should be kept in a moist condition until time of slab placement. Slabs subject to structural loading may be designed utilizing a modulus of subgrade reaction K of 200 pounds per square inch per inch. The K value was approximated based on inter-relationship of soil classification and bearing values (Portland Cement Association, Rocky Mountain Northwest).

7.7.5 It is recommended that utility trenches within the structure be compacted, as specified in our report, to minimize the transmission of moisture through the utility trench backfill.

7.7.6 Ponding of water should not be allowed adjacent to the slabs. Over-irrigation in landscaped areas adjacent to the slabs should be prevented.

7.7.7 Proper finishing and curing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

### 7.8 Lateral Earth Pressures and Frictional Resistance

7.8.1 Active, at-rest and passive unit lateral earth pressures against footings and walls are summarized in the table below:

| **Lateral Pressures**<br>**Drained and Level Backfill Conditions** | **Equivalent Fluid Pressure, pcf** |
|---|---|
| Active Pressure | 33 |
| At-Rest Pressure | 52 |
| Passive Pressure | 350 |
| **Related Parameters** | |
| Allowable Coefficient of Friction | 0.45 |
| In-Place Soil Density ($lbs/ft^3$) | 120 |

7.8.2 Active pressure applies to walls, which are free to rotate. At-rest pressure applies to walls, which are restrained against rotation. The preceding lateral earth pressures assume sufficient drainage behind retaining walls to prevent the build-up of hydrostatic pressure.

7.8.3 The top one-foot of adjacent subgrade should be deleted from the passive pressure computation.

7.8.4 A safety factor consistent with the design conditions should be included in the usage of the values in the above table.

7.8.5 For stability against lateral sliding, which is resisted solely by the passive pressure, we recommend a minimum safety factor of 1.5.

7.8.6 For stability against lateral sliding, which is resisted by the combined passive and frictional resistance, a minimum safety factor of 2.0 is recommended.

7.8.7 For lateral stability against seismic loading conditions, we recommend a minimum safety factor of 1.1.

7.8.8 For dynamic seismic lateral loading the following equation shall be used:

| **Dynamic Seismic Lateral Loading Equation** |
|---|
| Dynamic Seismic Lateral Load = $\frac{3}{8}\gamma K_h H^2$ |
| Where: $\gamma$ = In-Place Soil Density |
| $K_h$ = Horizontal Acceleration = $\frac{2}{3} PGA_M$ |
| H = Wall Height |

## 7.9 Retaining Walls

7.9.1 Retaining and/or below grade walls should be drained with either perforated pipe encased in free-draining gravel or a prefabricated drainage system. The gravel zone should have a minimum width of 12 inches wide and should extend upward to within 12 inches of the top of the wall. The upper 12 inches of backfill should consist of native soils, concrete, asphaltic-concrete or other suitable backfill to minimize surface drainage into the wall drain system. The gravel should be completely wrapped in nonwoven polypropylene geotextiles (filter fabric) to minimize migration of soil particles into the voids of the crushed rock.

7.9.2 Prefabricated drainage systems, such as Miradrain®, Enkadrain®, or an equivalent substitute, are acceptable alternatives in lieu of gravel provided they are installed in accordance with the manufacturer's recommendations. If a prefabricated drainage system is proposed, our firm should review the system for final acceptance prior to installation.

7.9.3 Drainage pipes should be placed with perforations down and should discharge in a non-erosive manner away from foundations and other improvements. The top of the perforated pipe should be placed at or below the bottom of the adjacent floor slab or pavements. The pipe should be placed in the center line of the drainage blanket and should have a minimum diameter of 4 inches. Slots should be no wider than 1/8-inch in diameter, while perforations should be no more than ¼-inch in diameter.

7.9.4 If retaining walls are less than 5 feet in height, the perforated pipe may be omitted in lieu of weep holes on 4 feet maximum spacing. The weep holes should consist of 2-inch minimum diameter holes (concrete walls) or unmortared head joints (masonry walls) and placed no higher than 18 inches above the lowest adjacent grade. Two 8-inch square overlapping patches of geotextile fabric (conforming to the CalTrans Standard Specifications for "edge drains") should be affixed to the rear wall opening of each weep hole to retard soil piping.

7.9.5 During grading and backfilling operations adjacent to any walls, heavy equipment should not be allowed to operate within a lateral distance of 5 feet from the wall, or within a lateral distance equal to the wall height, whichever is greater, to avoid developing excessive lateral pressures. Within this zone, only hand operated equipment ("whackers," vibratory plates, or pneumatic compactors) should be used to compact the backfill soils.

## 7.10 Temporary Excavations

7.10.1 We anticipate that the majority of the near surface site soils will be classified as Cal-OSHA "Type C" soil when encountered in excavations during site development and construction. Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and make appropriate recommendations where necessary.

7.10.2 It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements. 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.

7.10.3 Temporary excavations and slope faces should be protected from rainfall and erosion. Surface runoff should be directed away from excavations and slopes.

7.10.4 Open, unbraced excavations in undisturbed soils should be made according to the slopes presented in the following table:

**RECOMMENDED EXCAVATION SLOPES**

| Depth of Excavation (ft) | Slope (Horizontal : Vertical) |
|---|---|
| 0-5 | 1:1 |
| 5-10 | 2:1 |

7.10.5 If, due to space limitation, excavations near property lines or existing structures are performed in a vertical position, slot cuts, braced shorings or shields may be used for supporting vertical excavations. Therefore, in order to comply with the local and state safety regulations, a properly designed and installed shoring system would be required to accomplish planned excavations and installation. A Specialty Shoring Contractor should be responsible for the design and installation of such a shoring system during construction.

7.10.6 Braced shorings should be designed for a maximum pressure distribution of 30H, (where H is the depth of the excavation in feet). The foregoing does not include excess hydrostatic pressure or surcharge loading. Fifty percent of any surcharge load, such as construction equipment weight, should be added to the lateral load given herein. Equipment traffic should concurrently be limited to an area at least 3 feet from the shoring face or edge of the slope.

7.10.7 The excavation and shoring recommendations provided herein are based on soil characteristics derived from the borings within the area. Variations in soil conditions will likely be encountered during the excavations. SALEM Engineering Group, Inc. should be afforded the opportunity to provide field review to evaluate the actual conditions and account for field condition variations not otherwise anticipated in the preparation of this recommendation. Slope height, slope inclination, or excavation depth should in no case exceed those specified in local, state, or federal safety regulation, (e.g. OSHA) standards for excavations, 29 CFR part 1926, or Assessor's regulations.

### 7.11 Underground Utilities

7.11.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than 3-inches in maximum dimension. Trench backfill utilizing native soils should be placed in loose lifts not exceeding 8-inches and compacted to 95% relative compaction.

7.11.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to approximately 6 to 12 inches above the crown of the pipe. Pipe bedding and backfill material should conform to the requirements of the governing utility agency.

7.11.3 It is suggested that underground utilities crossing beneath new or existing structures be plugged at entry and exit locations to the building or structure to prevent water migration. Trench plugs can consist of on-site clay soils, if available, or sand cement slurry. The trench plugs should extend 2 feet beyond each side of individual perimeter foundations.

7.11.4 The contractor is responsible for removing all water-sensitive soils from the trench regardless of the backfill location and compaction requirements. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

**7.12 Surface Drainage**

7.12.1 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 to important engineering properties. Proper drainage should be maintained at all times.

7.12.2 The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than 5 percent for a minimum distance of 10 feet.

7.12.3 Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building and drainage gradients maintained to carry all surface water to collection facilities and off site. These grades should be maintained for the life of the project. Ponding of water should not be allowed adjacent to the structure. Over-irrigation within landscaped areas adjacent to the structure should not be performed.

7.12.4 Roof drains should be installed with appropriate downspout extensions out-falling on splash blocks so as to direct water a minimum of 5 feet away from the structures or be connected to the storm drain system for the development.

### 7.13 Pavement Design

7.13.1 Based on site soil conditions and laboratory testing, an R-value of 40 was used for the preliminary flexible asphaltic concrete pavement design. The R-value may be verified during grading of the pavement areas.

7.13.2 The pavement design recommendations provided herein are based on the State of California Department of Transportation (CALTRANS) design manual. The following table shows the recommended pavement sections for various traffic indices.

**TABLE 7.13.2**
**ASPHALT CONCRETE PAVEMENT**

| Traffic Index | Asphaltic Concrete | Clean Crushed Aggregate Base* | Compacted Subgrade* |
|---|---|---|---|
| 5.0 (Vehicle Parking and Drive Areas) | 3.0" | 4.0" | 12.0" |
| 6.0 (Occasional Truck Areas) | 3.0" | 6.0" | 12.0" |
| 7.0 (Heavy Truck Areas) | 4.0" | 7.0" | 12.0" |

***95% compaction based on ASTM D1557 Test Method***

7.13.3 The following recommendations are for light-duty, medium-duty and heavy-duty Portland Cement Concrete pavement sections.

**TABLE 7.13.3**
**PORTLAND CEMENT CONCRETE PAVEMENT**

| Traffic Index | Portland Cement Concrete* | Clean Crushed Aggregate Base** | Compacted Subgrade** |
|---|---|---|---|
| 5.0 (Light Duty) | 5.0" | 4.0" | 12.0" |
| 6.0 (Medium Duty) | 6.0" | 4.0" | 12.0" |
| 7.0 (Heavy Duty) | 7.0" | 6.0" | 12.0" |

*** Minimum Compressive Strength of 4,000 psi, Minimum Reinforcement of No. 4 bars at 18 inches o.c. each way***
***** 95% compaction based on ASTM D1557 Test Method***

## 8. PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

### 8.1 Plan and Specification Review

8.1.1 SALEM should review the project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

### 8.2 Construction Observation and Testing Services

8.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

8.2.2 SALEM should be present at the site during site preparation to observe site clearing, preparation of exposed surfaces after clearing, and placement, treatment and compaction of fill material.

8.2.3 SALEM's observations should be supplemented with periodic compaction tests to establish substantial conformance with these recommendations. Moisture content of footings and slab subgrade should be tested immediately prior to concrete placement. SALEM should observe foundation excavations prior to placement of reinforcing steel or concrete to assess whether the actual bearing conditions are compatible with the conditions anticipated during the preparation of this report.

## 9. LIMITATIONS AND CHANGED CONDITIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the test borings drilled at the approximate locations shown on the Site Plan, Figure 2. The report does not reflect variations which may occur between borings. The nature and extent of such variations may not become evident until construction is initiated. If variations then appear, a re-evaluation of the recommendations of this report will be necessary after performing on-site observations during the excavation period and noting the characteristics of such variations.

The findings and recommendations presented in this report are valid as of the present and for the proposed construction. If site conditions change due to natural processes or human intervention on the property or adjacent to the site, or changes occur in the nature or design of the project, or if there is a substantial time lapse between the submission of this report and the start of the work at the site, the conclusions and recommendations contained in our report will not be considered valid unless the changes are reviewed by SALEM and the conclusions of our report are modified or verified in writing. The validity of the recommendations contained in this report is also dependent upon an adequate testing and observations program during the construction phase.

Our firm assumes no responsibility for construction compliance with the design concepts or recommendations unless we have been retained to perform the on-site testing and review during construction. SALEM has prepared this report for the exclusive use of the owner and project design consultants.

SALEM does not practice in the field of corrosion engineering. It is recommended that a qualified corrosion engineer be consulted regarding protection of buried steel or ductile iron piping and conduit or, at a minimum, that manufacturer's recommendations for corrosion protection be closely followed. Further, a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of concrete slabs and foundations in direct contact with native soil. The importation of soil and or aggregate materials to the site should be screened to determine the potential for corrosion to concrete and buried metal piping.

The report has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No other warranties, either express or implied, are made as to the professional advice provided under

If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (909) 980-6455.

Respectfully Submitted,

**SALEM ENGINEERING GROUP, INC.**

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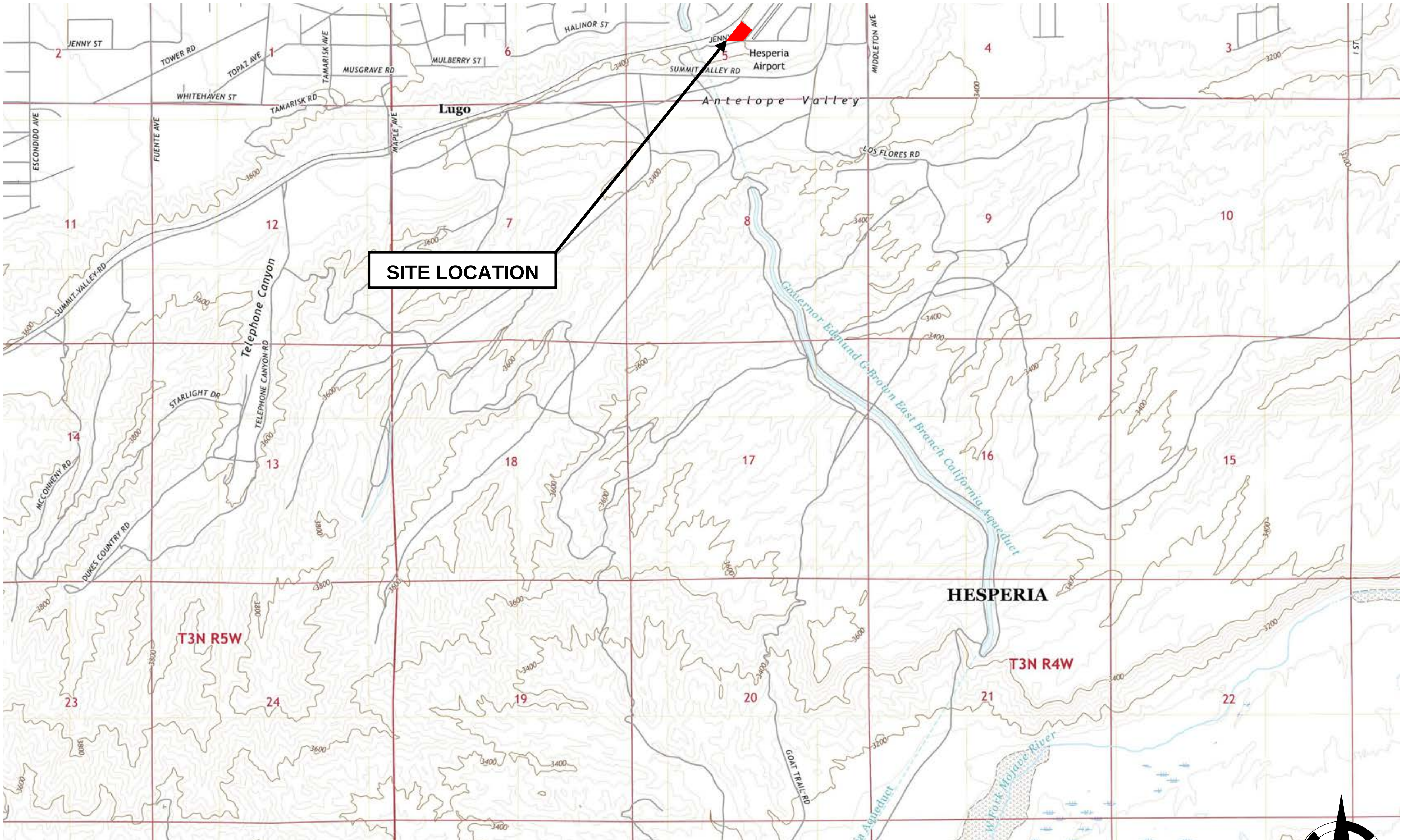


Source Image: U.S. Geological Survey, Silverwood Lake, California, [https://ngmdb.usgs.gov/topoview](https://ngmdb.usgs.gov/topoview) (2021)

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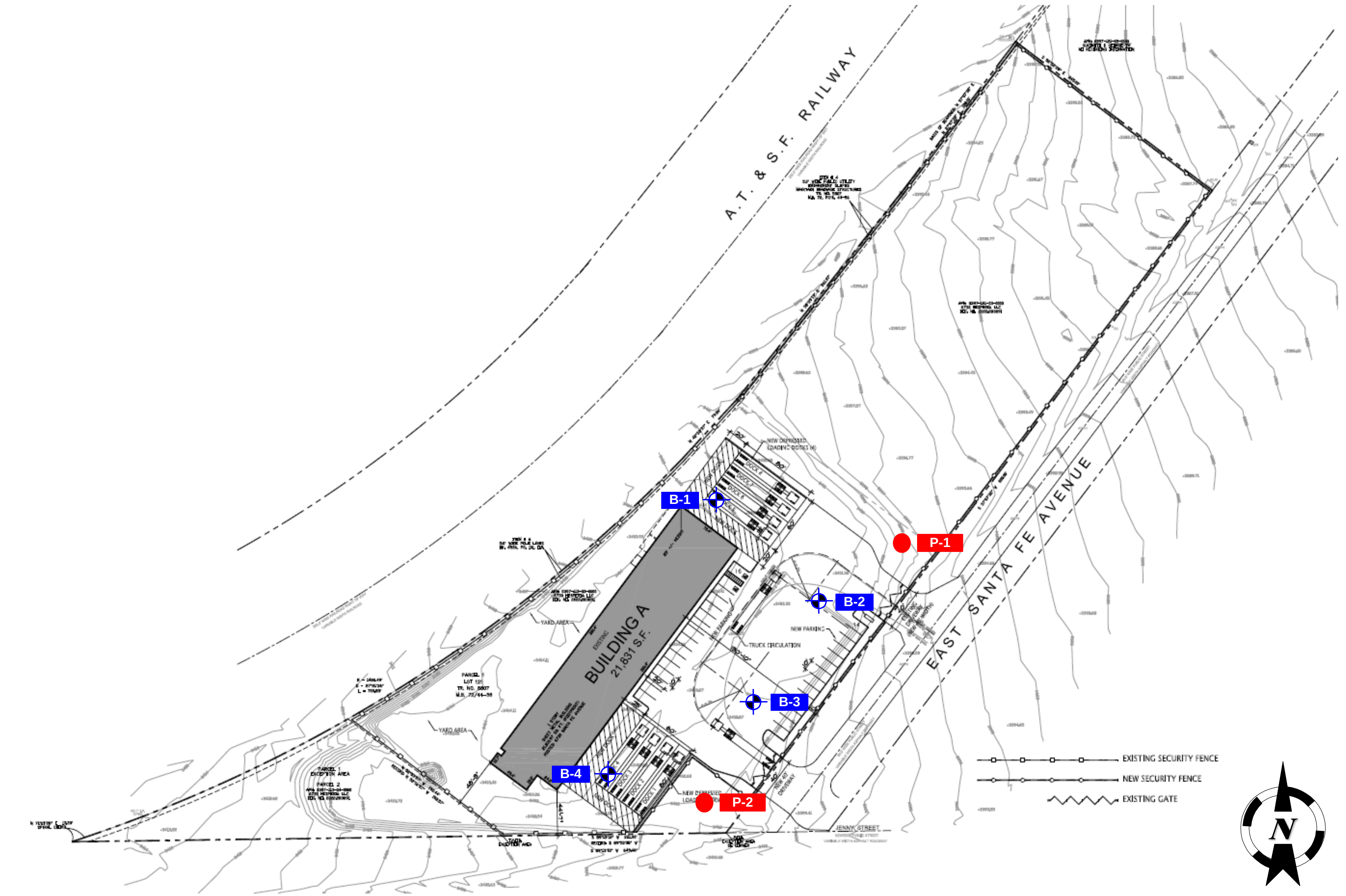
## VICINITY MAP

**LIMITED GEOTECHNICAL ENGINEERING INVESTIGATION**
**Proposed Loading Docks and Parking Lot**
**6730 Santa Fe Avenue E**
**Hesperia, California**

| SCALE: | DATE: |
|---|---|
| NOT TO SCALE | 05/2023 |
| DRAWN BY: JC | APPROVED BY: CJ |
| PROJECT NO. 3-223-0381 | FIGURE NO. 1 |

SALEM engineering group, inc.




# SITE PLAN

**LIMITED GEOTECHNICAL ENGINEERING INVESTIGATION**
**Proposed Loading Docks and Parking Lot**
**6730 Santa Fe Avenue E**
**Hesperia, California**

| SCALE: | DATE: |
|---|---|
| NOT TO SCALE | 05/2023 |
| DRAWN BY: JC | APPROVED BY: CJ |
| PROJECT NO. 3-223-0381 | FIGURE NO. 2 |

**LEGEND:**

B-1 Soil Boring Locations
P-1 Percolation Locations

All Locations Approximate

SALEM engineering group, inc.

APPENDIX

# A

## APPENDIX A
## FIELD EXPLORATION

Fieldwork for our investigation (drilling) was conducted on May 8, 2023, and included a site visit, subsurface exploration, percolation testing, and soil sampling. The locations of the exploratory borings and percolation tests are shown on the Site Plan, Figure 2. Boring logs for our exploration are presented in figures following the text in this appendix. Borings were located in the field using existing reference points. Therefore, actual boring locations may deviate slightly.

In general, the test borings were advanced with a 3-inch diameter hand auger. Surface asphalt for borings B-1 and B-4 was cored using a coring machine prior to drilling. The test borings were extended to a maximum depth of 10 feet below existing grade. Subsurface soil samples were obtained from ring samples and the auger cuttings at the depths shown on the logs of borings.

Subsurface conditions encountered in the exploratory borings were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The logs depict soil and geologic conditions encountered and depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

**Test Boring:** B-1 **Page** 1 **Of:** 1

**Project Number:** 3-223-0381

**Date:** 05/08/2023

**Client:** Crede Group

**Project:** Proposed Loading Docks and Parking Lot

**Location:** 6730 Santa Fe Avenue E, Hesperia, California

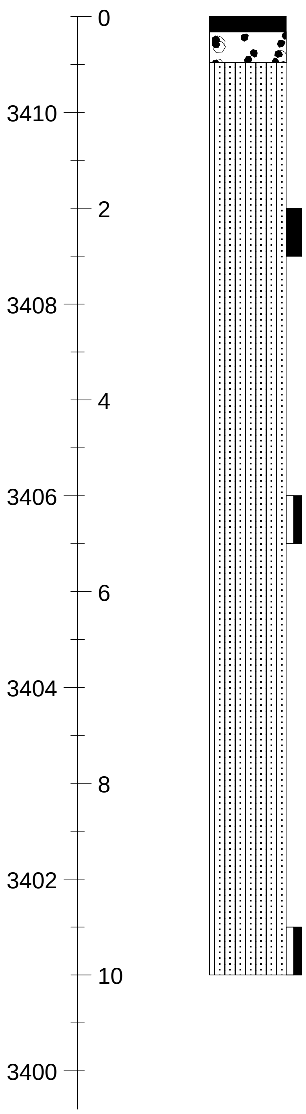
**Drilled By:** SALEM **Logged By:** CC

**Drill Type:** N/A **Elevation:** 3,411'

**Auger Type:** 3 in. Hand Auger **Initial Depth to Groundwater:** N/A

**Hammer Type:** 35 lb - Manual Drop **Final Depth to Groundwater:** N/A

ELEVATION/ DEPTH (feet) — SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA




| USCS | Soil Description | N-Values blows/ft. | Moisture Content % | Dry Density, PCF | Remarks |
|---|---|---|---|---|---|
| AC | Asphalt Concrete = 2 in. | | | | |
| AB | Aggregate Base = 3.25 | | | | |
| SM | Silty SAND<br>Moist; brown; fine to coarse grain sand. | | | | |
| | | | 5.5 | 105.8 | |
| | Grades as above; reddish brown. | | | | |
| | Grades as above. | | 7.9 | - | |
| | Grades as above; trace gravel. | | 5.6 | - | |
| | End of boring at 10 feet BSG. | | | | |

**Notes:**

**Figure Number A-1**

**SALEM** engineering group, inc.

**Test Boring:** B-2 **Page** 1 **Of:** 1

**Project Number:** 3-223-0381

**Date:** 05/08/2023

**Client:** Crede Group

**Project:** Proposed Loading Docks and Parking Lot

**Location:** 6730 Santa Fe Avenue E, Hesperia, California

**Drilled By:** SALEM **Logged By:** CC

**Drill Type:** N/A **Elevation:** 3,409'

**Auger Type:** 3 in. Hand Auger **Initial Depth to Groundwater:** N/A

**Hammer Type:** 35 lb - Manual Drop **Final Depth to Groundwater:** N/A

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | N-Values blows/ft. | Moisture Content % | Dry Density, PCF | Remarks |
|---|---|---|---|---|---|---|---|
| 0 | | SM | Silty SAND<br>Slightly moist; light brown; fine to coarse grain sand. | | | | |
| 3408 | | | | | 3.6 | - | |
| 2 | | | Grades as above; brown. | | 4.4 | - | |
| 3406 | | | | | | | |
| 4 | | | | | | | |
| 3404 | | | Grades as above; moist; fine to medium grain sand. | | 6.9 | - | |
| | | | End of boring at 5 feet BSG. | | | | |
| 6 | | | | | | | |
| 3402 | | | | | | | |
| 8 | | | | | | | |
| 3400 | | | | | | | |
| 10 | | | | | | | |
| 3398 | | | | | | | |

**Notes:**

**Figure Number A-2**

**Test Boring:** B-3 **Page** 1 **Of:** 1

**Project Number:** 3-223-0381

**Date:** 05/08/2023

**Client:** Crede Group

SALEM engineering group, inc.

**Project:** Proposed Loading Docks and Parking Lot

**Location:** 6730 Santa Fe Avenue E, Hesperia, California

**Drilled By:** SALEM **Logged By:** CC

**Drill Type:** N/A **Elevation:** 3,412'

**Auger Type:** 3 in. Hand Auger **Initial Depth to Groundwater:** N/A

**Hammer Type:** 35 lb - Manual Drop **Final Depth to Groundwater:** N/A

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | N-Values blows/ft. | Moisture Content % | Dry Density, PCF | Remarks |
|---|---|---|---|---|---|---|---|
| 3412 — 0 | | SM | Silty SAND<br>Moist; brown; fine to coarse grain sand. | | | | |
| | | | Grades as above. | | 5.1 | - | |
| 3410 — 2 | | | | | | | |
| | | | Grades as above. | | 5.8 | - | |
| | | | End of boring at 3 feet BSG. | | | | |
| 3408 — 4 | | | | | | | |
| 3406 — 6 | | | | | | | |
| 3404 — 8 | | | | | | | |
| 3402 — 10 | | | | | | | |

**Notes:**

**Figure Number A-3**

**SALEM** engineering group, inc.

**Test Boring:** B-4 **Page** 1 **Of:** 1

**Project Number:** 3-223-0381

**Date:** 05/08/2023

**Client:** Crede Group

**Project:** Proposed Loading Docks and Parking Lot

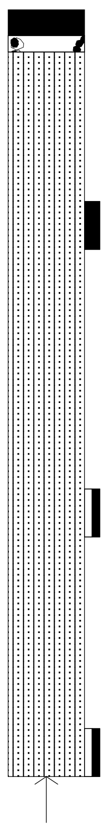
**Location:** 6730 Santa Fe Avenue E, Hesperia, California

**Drilled By:** SALEM **Logged By:** CC

**Drill Type:** N/A **Elevation:** 3,412'

**Auger Type:** 3 in. Hand Auger **Initial Depth to Groundwater:** N/A

**Hammer Type:** 35 lb - Manual Drop **Final Depth to Groundwater:** N/A



| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | N-Values blows/ft. | Moisture Content % | Dry Density, PCF | Remarks |
|---|---|---|---|---|---|---|---|
| 3412 — 0 | | AC | Asphalt Concrete = 3.25 in. | | | | |
| | | AB | Aggregate Base = 2 in. | | | | |
| | | SM | Silty SAND<br>Moist; reddish brown; fine to coarse grain sand; trace gravel. | | | | |
| 3410 — 2 | | | | | 8.6 | 115.8 | |
| 3408 — 4 | | | | | | | |
| | | | Grades as above; brown; less silt. | | 7.2 | - | |
| 3406 — 6 | | | | | | | |
| | | | Grades as above; light brown. | | 6.6 | - | |
| 3404 — 8 | | | Refusal at 8 feet BSG due to hard soil. | | | | |
| 3402 — 10 | | | | | | | |

**Notes:**

**Figure Number A-4**

# KEY TO SYMBOLS

**Symbol Description**

**Strata symbols**

Asphaltic Concrete

Description not given for:
"AG"

Silty sand

**Misc. Symbols**

Drill rejection

**Soil Samplers**

California sampler

Auger

**Notes:**

**Granular Soils**
**Blows Per Foot (Uncorrected)**

| | MCS | SPT |
|---|---|---|
| Very loose | <5 | <4 |
| Loose | 5-15 | 4-10 |
| Medium dense | 16-40 | 11-30 |
| Dense | 41-65 | 31-50 |
| Very dense | >65 | >50 |

**Cohesive Soils**
**Blows Per Foot (Uncorrected)**

| | MCS | SPT |
|---|---|---|
| Very soft | <3 | <2 |
| Soft | 3-5 | 2-4 |
| Firm | 6-10 | 5-8 |
| Stiff | 11-20 | 9-15 |
| Very Stiff | 21-40 | 16-30 |
| Hard | >40 | >30 |

MCS = Modified California Sampler
SPT = Standard Penetration Test Sampler

# Percolation Test Worksheet

**Project:** **Proposed Loading Docks and Parking Lot**
**6730 Santa Fe Avenue E**
**Hesperia, California**

**Job No.:** **3-223-0381**
**Date Drilled:** **5/8/2023**
**Soil Classification:** **Silty SAND (SM)**

Hole Radius: 3 in.
Pipe Dia.: 3 in.

**Test Hole No.:** **P-1**
**Tested by:** **CC**
**Drilled Hole Depth:** **4.75** ft.

**Presoaking Date:** **5/8/2023**
**Test Date:** **5/8/2023**

Total Depth of Hole: 57 in.
Pipe Stick up: 0.25 ft.

| Time Start | Time Finish | Depth of Test Hole (ft)# | Refill-Yes or No | Elapsed Time (hrs:min) | Initial Water Level# (ft) | Final Water Level# (ft) | Δ Water Level (in.) | Δ Min. | Meas. Perc Rate (min/in) | Initial Height of Water (in) | Final Height of Water (in) | Average Height of Water (in) | Infiltration Rate, It (in/hr) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 8:25 | 8:50 | 5.0 | Y | 0:25 | 1.52 | 2.64 | 13.44 | 25 | 1.9 | 41.8 | 28.3 | 35.0 | 1.32 |
| 8:51 | 9:16 | 5.0 | Y | 0:25 | 1.60 | 2.63 | 12.36 | 25 | 2.0 | 40.8 | 28.4 | 34.6 | 1.23 |
| 9:17 | 9:27 | 5.0 | Y | 0:10 | 2.06 | 2.44 | 4.56 | 10 | 2.2 | 35.3 | 30.7 | 33.0 | 1.19 |
| 9:27 | 9:37 | 5.0 | N | 0:10 | 2.44 | 2.77 | 3.96 | 10 | 2.5 | 30.7 | 26.8 | 28.7 | 1.18 |
| 9:37 | 9:47 | 5.0 | N | 0:10 | 2.77 | 3.05 | 3.36 | 10 | 3.0 | 26.8 | 23.4 | 25.1 | 1.14 |
| 9:48 | 9:58 | 5.0 | Y | 0:10 | 1.64 | 2.05 | 4.92 | 10 | 2.0 | 40.3 | 35.4 | 37.9 | 1.13 |
| 9:58 | 10:08 | 5.0 | N | 0:10 | 2.05 | 2.41 | 4.32 | 10 | 2.3 | 35.4 | 31.1 | 33.2 | 1.12 |
| 10:08 | 10:18 | 5.0 | N | 0:10 | 2.41 | 2.73 | 3.84 | 10 | 2.6 | 31.1 | 27.2 | 29.2 | 1.13 |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | **Infiltration Rate** | | | **1.12** |

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# Percolation Test Worksheet

**Project:** **Proposed Loading Docks and Parking Lot**
**6730 Santa Fe Avenue E**
**Hesperia, California**

**Job No.:** **3-223-0381**
**Date Drilled:** **5/8/2023**
**Soil Classification:** **Poorly graded SAND (SP)**

Hole Radius: 3 in.
Pipe Dia.: 3 in.
Total Depth of Hole: 36 in.

**Test Hole No.:** **P-2**
**Tested by:** **CC**
**Drilled Hole Depth:** **3.0** ft.

**Presoaking Date:** **5/8/2023**
**Test Date:** **5/8/2023**

Pipe Stick up: 1.75 ft.

| Time Start | Time Finish | Depth of Test Hole (ft)# | Refill-Yes or No | Elapsed Time (hrs:min) | Initial Water Level# (ft) | Final Water Level# (ft) | Δ Water Level (in.) | Δ Min. | Meas. Perc Rate (min/in) | Initial Height of Water (in) | Final Height of Water (in) | Average Height of Water (in) | Infiltration Rate, It (in/hr) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 8:45 | 9:10 | 4.8 | Y | 0:25 | 2.40 | 3.75 | 16.20 | 25 | 1.5 | 28.2 | 12.0 | 20.1 | 2.70 |
| 9:11 | 9:36 | 4.8 | Y | 0:25 | 2.62 | 3.81 | 14.28 | 25 | 1.8 | 25.6 | 11.3 | 18.4 | 2.58 |
| 9:37 | 9:47 | 4.8 | Y | 0:10 | 2.70 | 3.23 | 6.36 | 10 | 1.6 | 24.6 | 18.2 | 21.4 | 2.50 |
| 9:47 | 9:57 | 4.8 | N | 0:10 | 3.23 | 3.62 | 4.68 | 10 | 2.1 | 18.2 | 13.6 | 15.9 | 2.42 |
| 9:57 | 10:07 | 4.8 | N | 0:10 | 3.62 | 3.91 | 3.48 | 10 | 2.9 | 13.6 | 10.1 | 11.8 | 2.35 |
| 10:08 | 10:18 | 4.8 | Y | 0:10 | 3.00 | 3.43 | 5.16 | 10 | 1.9 | 21.0 | 15.8 | 18.4 | 2.33 |
| 10:18 | 10:28 | 4.8 | N | 0:10 | 3.43 | 3.76 | 3.96 | 10 | 2.5 | 15.8 | 11.9 | 13.9 | 2.32 |
| 10:28 | 10:38 | 4.8 | N | 0:10 | 3.76 | 4.02 | 3.12 | 10 | 3.2 | 11.9 | 8.8 | 10.3 | 2.38 |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | **Infiltration Rate** | **2.32** |

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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), Caltrans, or other suggested procedures. Selected samples were tested for in-situ moisture content, density, shear strength, maximum density and optimum moisture content, gradation, and corrosivity of the material encountered. The results of the laboratory tests are summarized in the following figures.

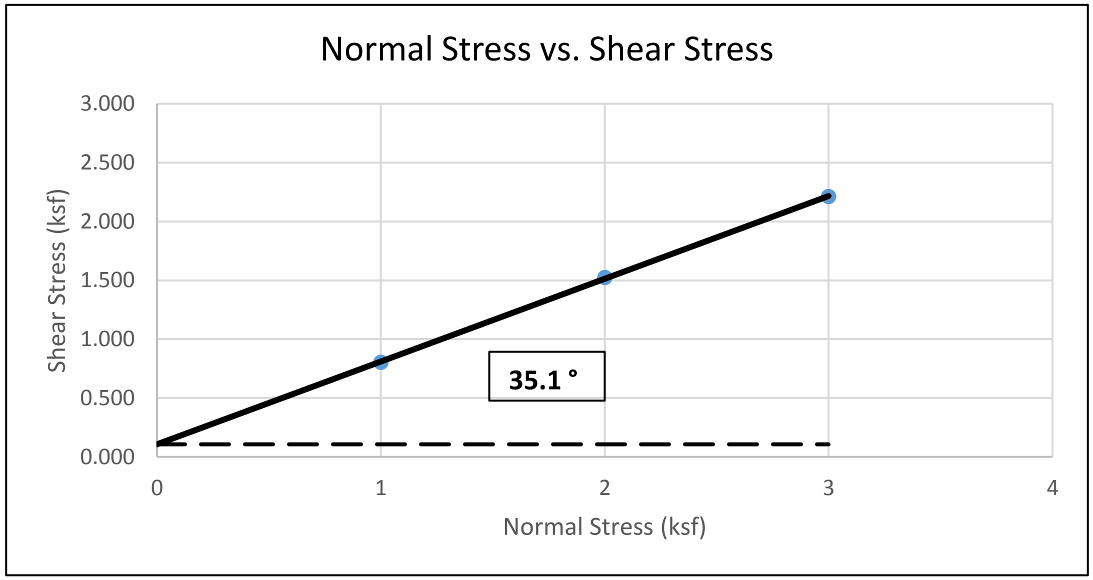
# Direct Shear Test (ASTM D3080)

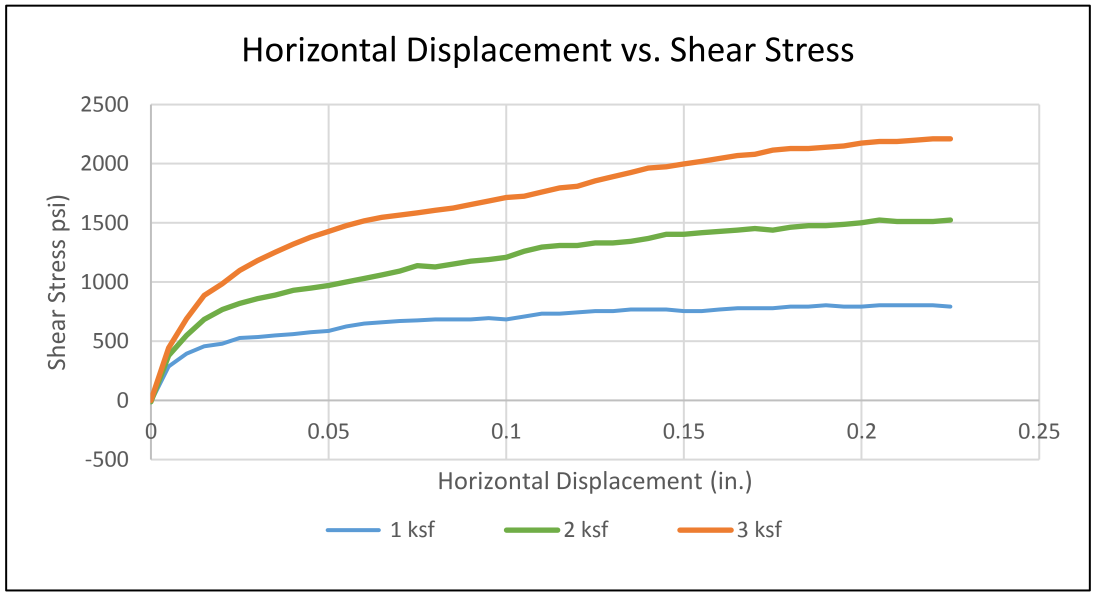
Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA
Project Number: 3-223-0381
Client: Crede Group
Sample Location: B-1 @ 2'
Sample Type: Undisturbed Ring
Soil Classification: Silty SAND (SM)
Tested By: M. Noorzay
Reviewed By: CJ
Date: 5/11/2023
Equipment Used: Geomatic Direct Shear Machine

| | Sample 1 | Sample 2 | Sample 3 |
|---|---|---|---|
| Normal Stress (ksf) | 1.000 | 2.000 | 3.000 |
| Shear Rate (in/min) | 0.004 | | |
| Peak Shear Stress (ksf) | 0.804 | 1.524 | 2.210 |
| Residual Shear Stress (ksf) | 0.000 | 0.000 | 0.000 |

| | | | |
|---|---|---|---|
| Initial Height of Sample (in) | 1.000 | 1.000 | 1.000 |
| Height of Sample before Shear (in.) | 1 | 1 | 1 |
| Diameter of Sample (in) | 2.416 | 2.416 | 2.416 |
| Initial Moisture Content (%) | 5.4 | | |
| Final Moisture Content (%) | 14.5 | 13.6 | 13.4 |
| Dry Density (pcf) | 108.5 | 110.2 | 108.8 |

| Peak Shear Strength Values | |
|---|---|
| **Slope** | 0.70 |
| **Friction Angle** | 35.1 |
| **Cohesion (psf)** | 106 |

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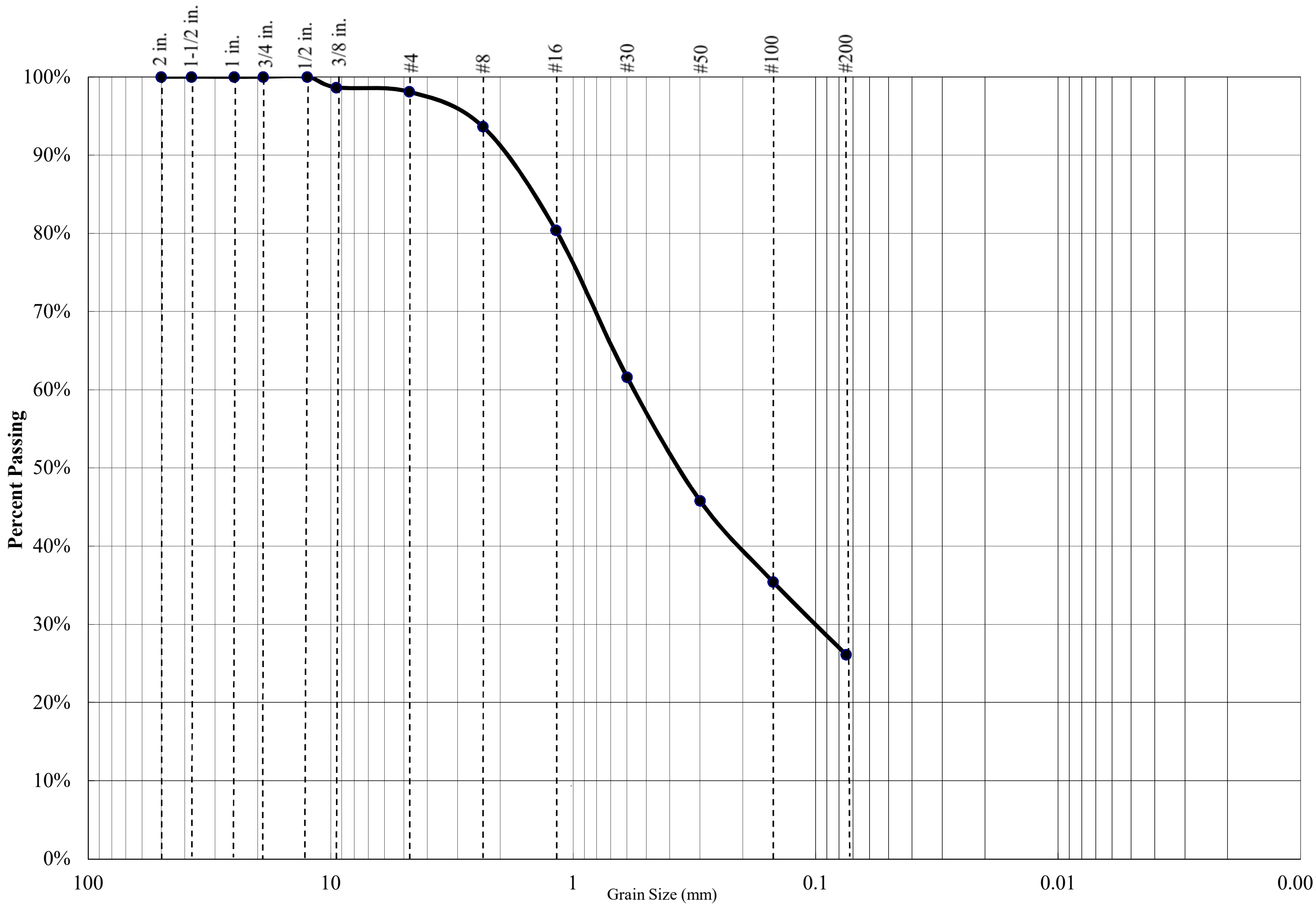
## PARTICLE SIZE DISTRIBUTION DIAGRAM

### GRADATION TEST - ASTM C136




| Percent Gravel | Percent Sand | Percent Silt/Clay |
|---|---|---|
| 2% | 72% | 26% |

| Sieve Size | Percent Passing |
|---|---|
| 3/4 inch | 100.0% |
| 1/2 inch | 100.0% |
| 3/8 inch | 98.7% |
| #4 | 98.1% |
| #8 | 93.6% |
| #16 | 80.4% |
| #30 | 61.6% |
| #50 | 45.8% |
| #100 | 35.4% |
| #200 | 26.1% |

| Atterberg Limits | | |
|---|---|---|
| PL= | LL= | PI= |

| Coefficients | | | | |
|---|---|---|---|---|
| D85= | | D60= | | D50= |
| D30= | | D15= | | D10= |
| $C_u$= | N/A | $C_c$= | N/A | |

| USCS CLASSIFICATION |
|---|
| Silty SAND (SM) |

**Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA**

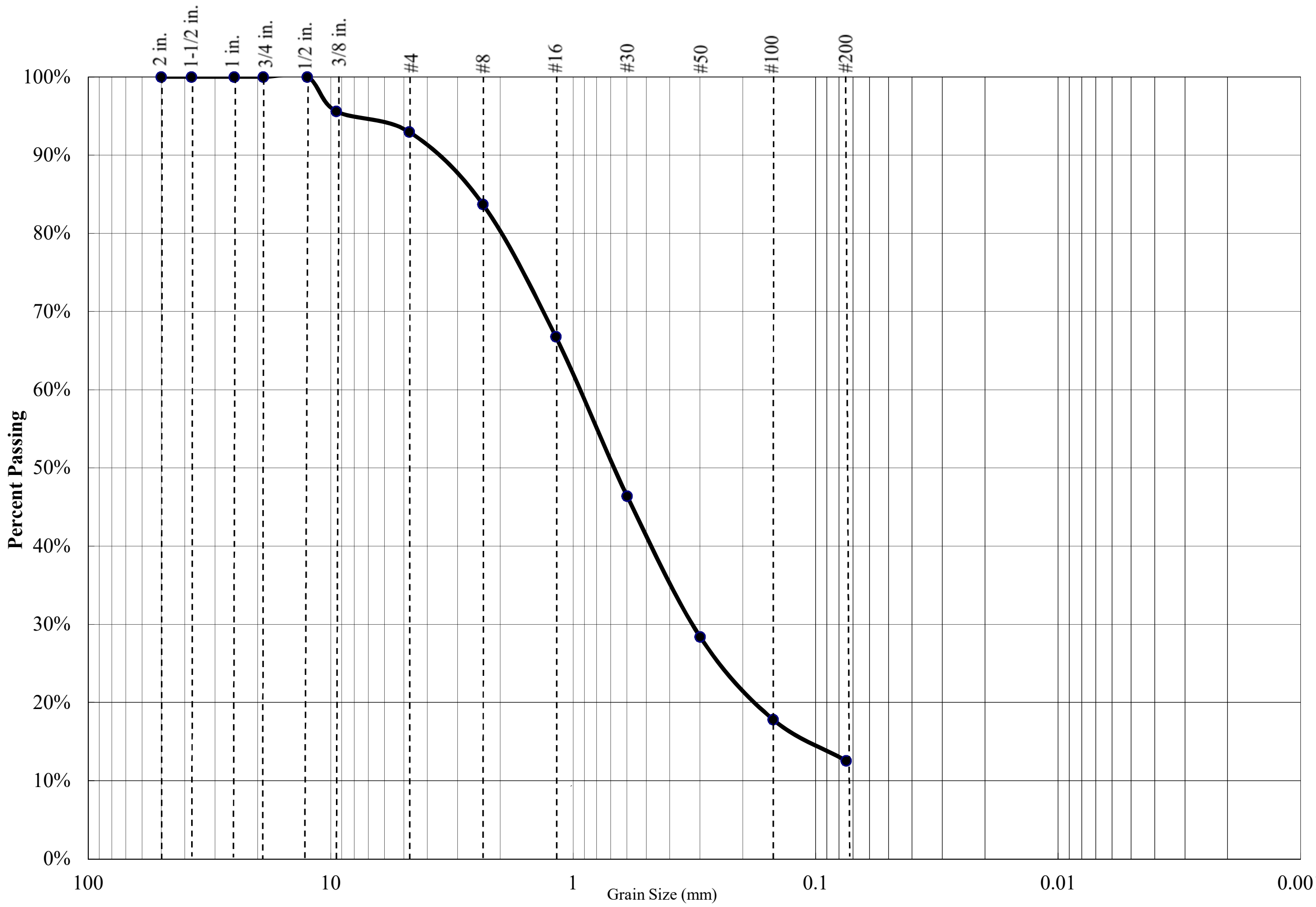
**Project Number: 3-223-0381**

**Boring: B-1 @ 2'**

SALEM engineering group, inc.

# PARTICLE SIZE DISTRIBUTION DIAGRAM

## GRADATION TEST - ASTM C136




| Percent Gravel | Percent Sand | Percent Silt/Clay |
|---|---|---|
| 7% | 80% | 13% |

| Sieve Size | Percent Passing |
|---|---|
| 3/4 inch | 100.0% |
| 1/2 inch | 100.0% |
| 3/8 inch | 95.6% |
| #4 | 93.0% |
| #8 | 83.7% |
| #16 | 66.8% |
| #30 | 46.4% |
| #50 | 28.4% |
| #100 | 17.8% |
| #200 | 12.6% |

| Atterberg Limits | | |
|---|---|---|
| PL= | LL= | PI= |

| Coefficients | | | | |
|---|---|---|---|---|
| D85= | | D60= | | D50= |
| D30= | | D15= | | D10= |
| $C_u$= | N/A | $C_c$= | N/A | |

| USCS CLASSIFICATION |
|---|
| Silty SAND (SM) |

**Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA**

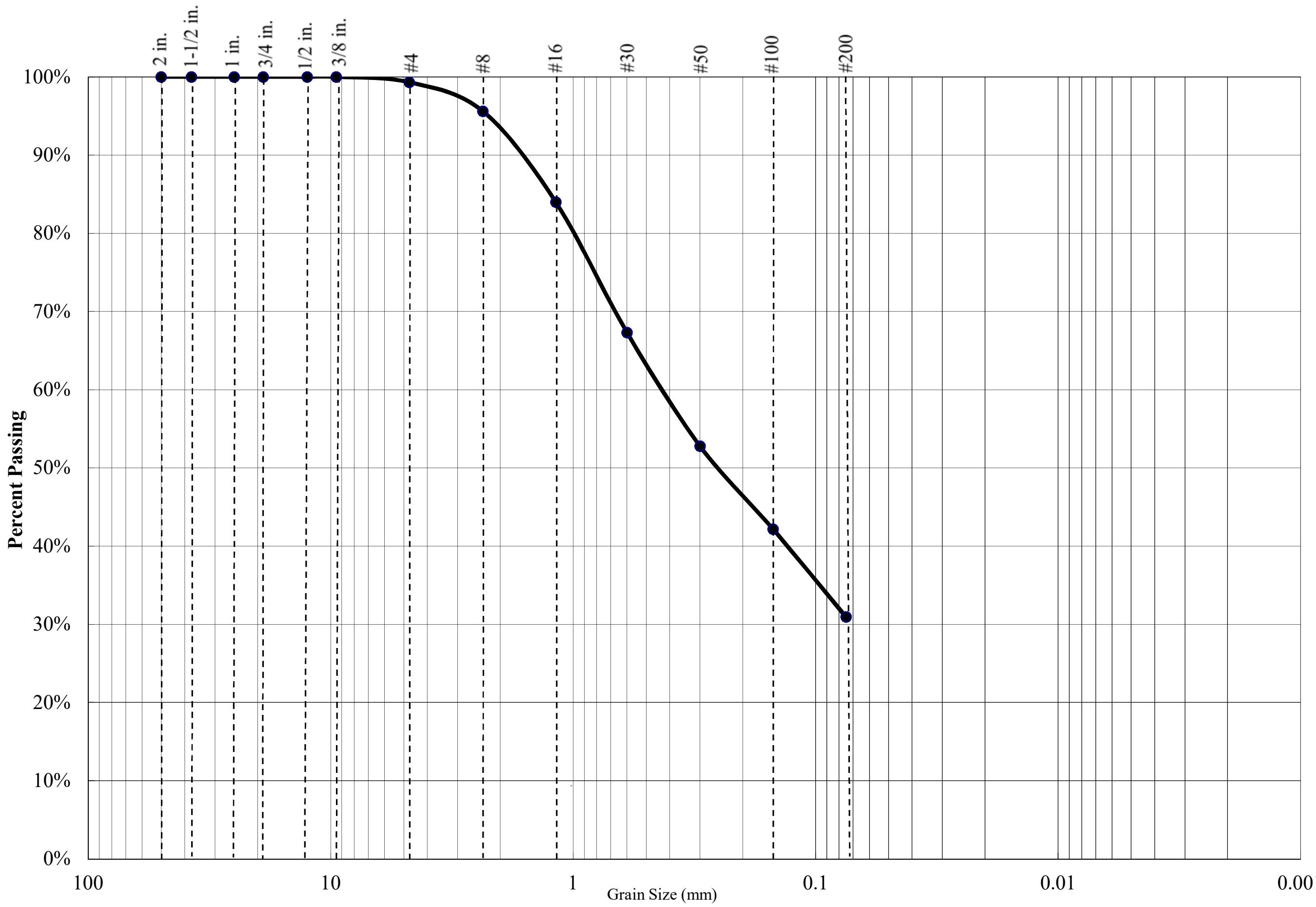
**Project Number: 3-223-0381**

**Boring: B-1 @ 10'**

SALEM engineering group, inc.

# PARTICLE SIZE DISTRIBUTION DIAGRAM

## GRADATION TEST - ASTM C136




| Percent Gravel | Percent Sand | Percent Silt/Clay |
|---|---|---|
| 1% | 68% | 31% |

| Sieve Size | Percent Passing |
|---|---|
| 3/4 inch | 100.0% |
| 1/2 inch | 100.0% |
| 3/8 inch | 100.0% |
| #4 | 99.3% |
| #8 | 95.6% |
| #16 | 84.0% |
| #30 | 67.3% |
| #50 | 52.8% |
| #100 | 42.2% |
| #200 | 30.9% |

| Atterberg Limits | | |
|---|---|---|
| PL= | LL= | PI= |

| Coefficients | | | |
|---|---|---|---|
| D85= | | D60= | D50= |
| D30= | | D15= | D10= |
| $C_u$= | N/A | $C_c$= N/A | |

| USCS CLASSIFICATION |
|---|
| Silty SAND (SM) |

**Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA**

**Project Number: 3-223-0381**

**Boring: B-2 @ 5'**

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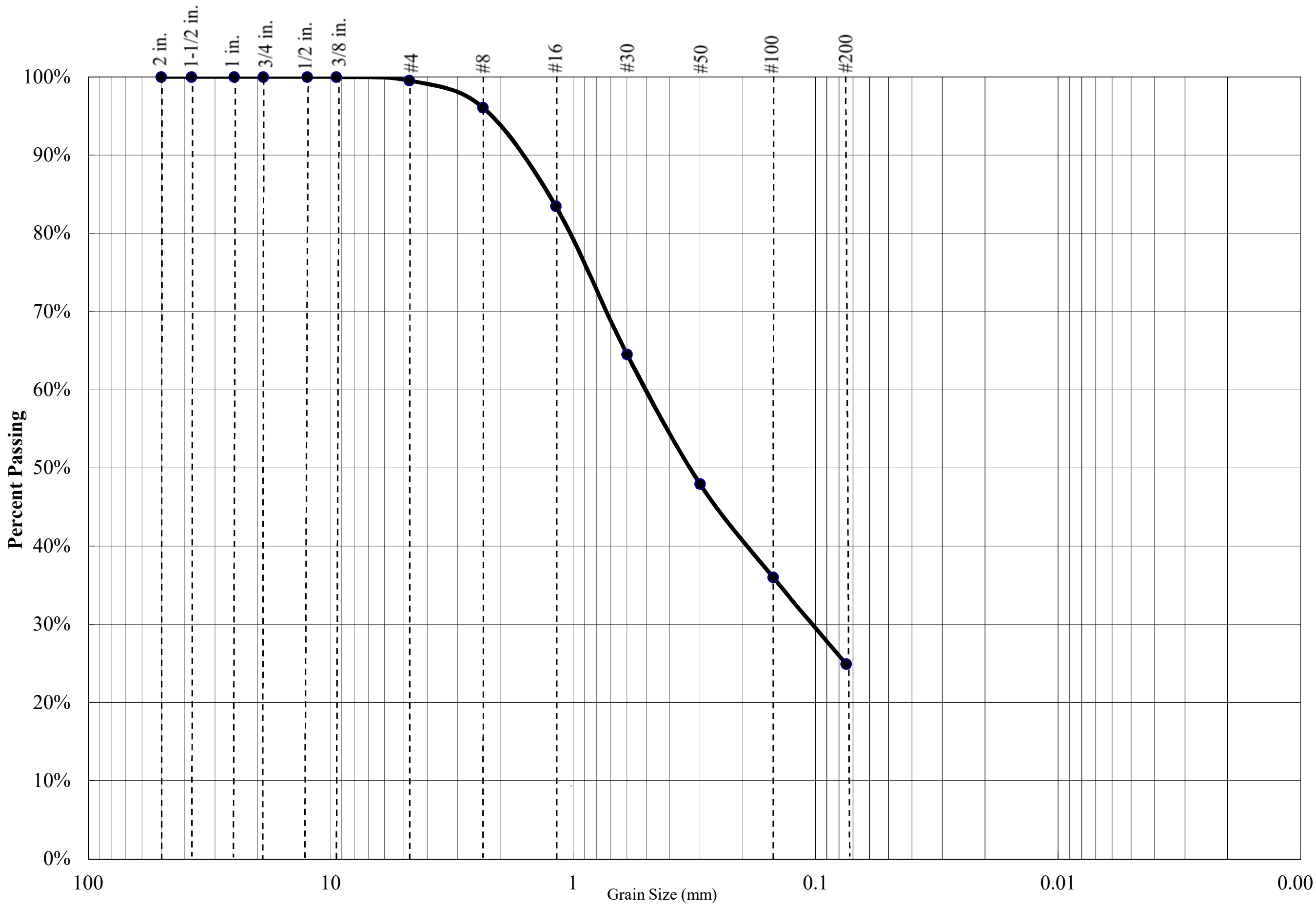
## PARTICLE SIZE DISTRIBUTION DIAGRAM

### GRADATION TEST - ASTM C136




| Percent Gravel | Percent Sand | Percent Silt/Clay |
|---|---|---|
| 0% | 75% | 25% |

| Sieve Size | Percent Passing |
|---|---|
| 3/4 inch | 100.0% |
| 1/2 inch | 100.0% |
| 3/8 inch | 100.0% |
| #4 | 99.5% |
| #8 | 96.1% |
| #16 | 83.5% |
| #30 | 64.5% |
| #50 | 47.9% |
| #100 | 36.0% |
| #200 | 24.9% |

| Atterberg Limits | | |
|---|---|---|
| PL= | LL= | PI= |

| Coefficients | | | | |
|---|---|---|---|---|
| $D_{85}$= | | $D_{60}$= | | $D_{50}$= |
| $D_{30}$= | | $D_{15}$= | | $D_{10}$= |
| $C_u$= | N/A | $C_c$= | N/A | |

| USCS CLASSIFICATION |
|---|
| Silty SAND (SM) |

**Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA**

**Project Number: 3-223-0381**

**Boring: B-3 @ 1'**

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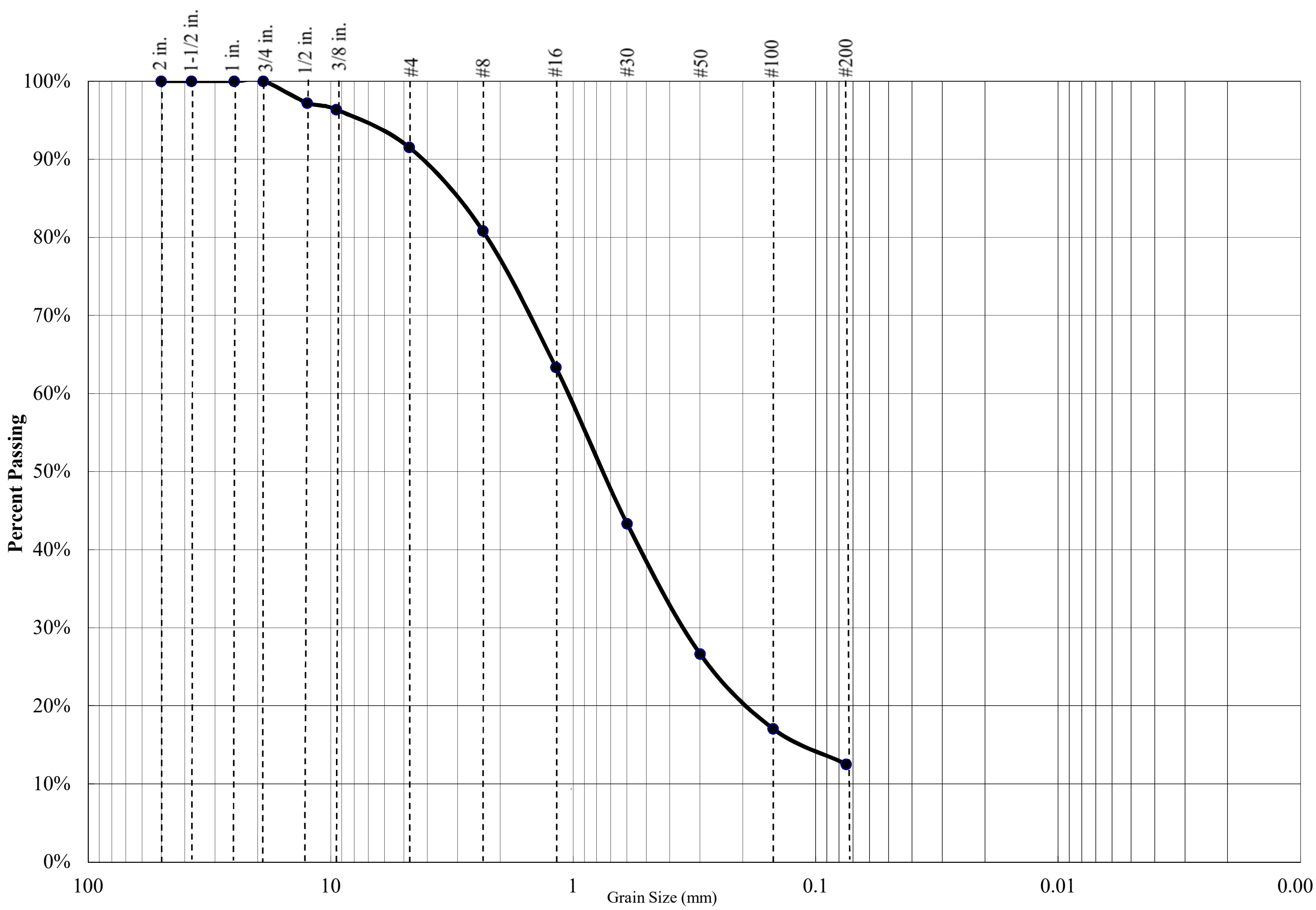
## PARTICLE SIZE DISTRIBUTION DIAGRAM

### GRADATION TEST - ASTM C136




| Percent Gravel | Percent Sand | Percent Silt/Clay |
|---|---|---|
| 8% | 79% | 13% |

| Sieve Size | Percent Passing |
|---|---|
| 3/4 inch | 100.0% |
| 1/2 inch | 97.2% |
| 3/8 inch | 96.4% |
| #4 | 91.5% |
| #8 | 80.8% |
| #16 | 63.4% |
| #30 | 43.3% |
| #50 | 26.7% |
| #100 | 17.1% |
| #200 | 12.5% |

| Atterberg Limits | | |
|---|---|---|
| PL= | LL= | PI= |

| Coefficients | | | | | |
|---|---|---|---|---|---|
| $D_{85}$= | | $D_{60}$= | | $D_{50}$= | |
| $D_{30}$= | | $D_{15}$= | | $D_{10}$= | |
| $C_u$= | N/A | $C_c$= | N/A | | |

| USCS CLASSIFICATION |
|---|
| Silty SAND (SM) |

**Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA**

**Project Number: 3-223-0381**

**Boring: B-4 @ 5'**

SALEM engineering group, inc.

# CHEMICAL ANALYSIS

## $SO_4$ - Modified CTM 417 & Cl - Modified CTM 417/422

Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA
Project Number: 3-223-0381
Date Sampled: 5/8/2023 Date Tested: 5/11/2023
Sampled By: CC Tested By: M. Noorzay
Soil Description: Brown Silty SAND (SM)

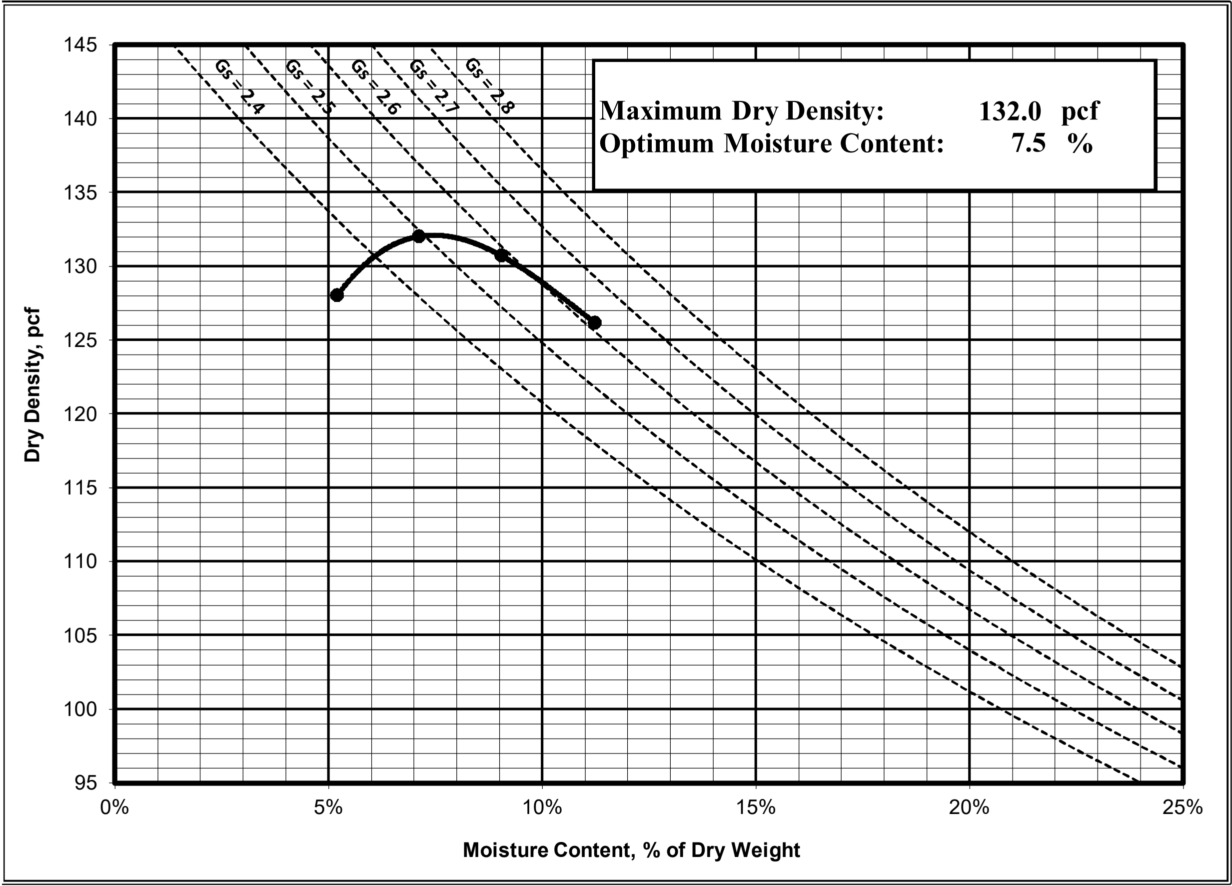
| Sample Number | Sample Location | Soluble Sulfate $SO_4$-S | Soluble Chloride Cl | pH |
|---|---|---|---|---|
| 1a. | B-2 @ 0'-5' | 840 mg/kg | 32 mg/kg | 7.5 |
| 1b. | B-2 @ 0'-5' | 780 mg/kg | 31 mg/kg | 7.5 |
| 1c. | B-2 @ 0'-5' | 800 mg/kg | 32 mg/kg | 7.5 |
| **Average:** | | **807 mg/kg** | **32 mg/kg** | **7.5** |

SALEM engineering group, inc.

# Laboratory Compaction Curve
# ASTM D1557

Project Name: Proposed Loading Docks & Parking Lot - Hesperia, CA
Project Number: 3-223-0381
Date Sampled: 5/8/2023 Date Tested: 5/11/2023
Sampled By: CC Tested By: M. Noorzay
Sample Location: B-2 @ 0'-5'
Soil Description: Brown Silty SAND (SM)
Test Method: Method B

| | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Weight of Moist Specimen & Mold, (g) | 6316.7 | 6418.3 | 6435.0 | 6401.9 |
| Weight of Compaction Mold, (g) | 4280.2 | 4280.2 | 4280.2 | 4280.2 |
| Weight of Moist Specimen, (g) | 2036.5 | 2138.1 | 2154.8 | 2121.7 |
| Volume of Mold, ($ft^3$) | 0.0333 | 0.0333 | 0.0333 | 0.0333 |
| Wet Density, (pcf) | 134.7 | 141.4 | 142.5 | 140.3 |
| Weight of Wet (Moisture) Sample, (g) | 200.0 | 200.0 | 200.0 | 200.0 |
| Weight of Dry (Moisture) Sample, (g) | 190.1 | 186.7 | 183.4 | 179.8 |
| Moisture Content, (%) | 5.2% | 7.1% | 9.1% | 11.2% |
| Dry Density, (pcf) | 128.0 | 132.0 | 130.7 | 126.2 |

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APPENDIX

# C

**APPENDIX C**
**GENERAL EARTHWORK AND PAVEMENT SPECIFICATIONS**

When the text of the report conflicts with the general specifications in this appendix, the recommendations in the report have precedence.

**1.0 SCOPE OF WORK:** These specifications and applicable plans pertain to and include all earthwork associated with the site rough grading, including, but not limited to, the furnishing of all labor, tools and equipment necessary for site clearing and grubbing, stripping, preparation of foundation materials for receiving fill, excavation, processing, placement and compaction of fill and backfill materials to the lines and grades shown on the project grading plans and disposal of excess materials.

**2.0 PERFORMANCE:** The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of SALEM Engineering Group, Incorporated, hereinafter referred to as the Soils Engineer and/or Testing Agency. Attainment of design grades, when achieved, shall be certified by the project Civil Engineer. Both the Soils Engineer and the Civil Engineer are the Owner's representatives. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary adjustments until all work is deemed satisfactory as determined by both the Soils Engineer and the Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Soils Engineer, Civil Engineer, or project Architect.

No earthwork shall be performed without the physical presence or approval of the Soils Engineer. The Contractor shall notify the Soils Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork.

The Contractor shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the Contractor shall defend, indemnify and hold the Owner and the Engineers harmless from any and all liability, real or alleged, in connection with the performance of work on this project, except for liability arising from the sole negligence of the Owner or the Engineers.

**3.0 TECHNICAL REQUIREMENTS**: All compacted materials shall be densified to no less than 95 percent of relative compaction (90 percent for clay soils) based on ASTM D1557 Test Method (latest edition) or as specified in the technical portion of the Soil Engineer's report. The location and frequency of field density tests shall be determined by the Soils Engineer. The results of these tests and compliance with these specifications shall be the basis upon which satisfactory completion of work will be judged by the Soils Engineer.

**4.0 SOILS AND FOUNDATION CONDITIONS**: The Contractor is presumed to have visited the site and to have familiarized himself with existing site conditions and the contents of the data presented in the Geotechnical Engineering Report. The Contractor shall make his own interpretation of the data contained in the Geotechnical Engineering Report and the Contractor shall not be relieved of liability for any loss sustained as a result of any variance between conditions indicated by or deduced from said report and the actual conditions encountered during the progress of the work.

**5.0 DUST CONTROL:** The work includes dust control as required for the alleviation or prevention of any dust nuisance on or about the site or the borrow area, or off-site if caused by the Contractor's operation either during the performance of the earthwork or resulting from the conditions in which the Contractor leaves the site. The Contractor shall assume all liability, including court costs of codefendants, for all claims related to dust or wind-blown materials attributable to his work. Site preparation shall consist of site clearing and grubbing and preparation of foundation materials for receiving fill.

**6.0 CLEARING AND GRUBBING:** The Contractor shall accept the site in this present condition and shall demolish and/or remove from the area of designated project earthwork all structures, both surface and subsurface, trees, brush, roots, debris, organic matter and all other matter determined by the Soils Engineer to be deleterious. Such materials shall become the property of the Contractor and shall be removed from the site.

Tree root systems in proposed improvement areas should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots greater than 1 inch in diameter. Tree roots removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill of tree root excavations is not permitted until all exposed surfaces have been inspected and the Soils Engineer is present for the proper control of backfill placement and compaction. Burning in areas which are to receive fill materials shall not be permitted.

**7.0 SUBGRADE PREPARATION:** Surfaces to receive Engineered Fill and/or building or slab loads shall be prepared as outlined above, scarified to a minimum of 12 inches, moisture-conditioned as necessary, and recompacted to 95 percent relative compaction (90 percent for clay soils).

Loose soil areas and/or areas of disturbed soil shall be moisture-conditioned as necessary and recompacted to 95 percent relative compaction (90 percent for clay soils). All ruts, hummocks, or other uneven surface features shall be removed by surface grading prior to placement of any fill materials. All areas which are to receive fill materials shall be approved by the Soils Engineer prior to the placement of any fill material.

**8.0 EXCAVATION:** All excavation shall be accomplished to the tolerance normally defined by the Civil Engineer as shown on the project grading plans. All over-excavation below the grades specified shall be backfilled at the Contractor's expense and shall be compacted in accordance with the applicable technical requirements.

**9.0 FILL AND BACKFILL MATERIAL:** No material shall be moved or compacted without the presence or approval of the Soils Engineer. Material from the required site excavation may be utilized for construction site fills, provided prior approval is given by the Soils Engineer. All materials utilized for constructing site fills shall be free from vegetation or other deleterious matter as determined by the Soils Engineer.

**10.0 PLACEMENT, SPREADING AND COMPACTION:** The placement and spreading of approved fill materials and the processing and compaction of approved fill and native materials shall be the responsibility of the Contractor. Compaction of fill materials by flooding, ponding, or jetting shall not be permitted unless specifically approved by local code, as well as the Soils Engineer. Both cut and fill shall be surface-compacted to the satisfaction of the Soils Engineer prior to final acceptance.

**11.0 SEASONAL LIMITS:** No fill material shall be placed, spread, or rolled while it is frozen or thawing, or during unfavorable wet weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until the Soils Engineer indicates that the moisture content and density of previously placed fill is as specified.

**12.0 DEFINITIONS -** The term "pavement" shall include asphaltic concrete surfacing, untreated aggregate base, and aggregate subbase. The term "subgrade" is that portion of the area on which surfacing, base, or subbase is to be placed.

The term "Standard Specifications": hereinafter referred to, is the most recent edition of the Standard Specifications of the State of California, Department of Transportation. The term "relative compaction" refers to the field density expressed as a percentage of the maximum laboratory density as determined by ASTM D1557 Test Method (latest edition).

**13.0 PREPARATION OF THE SUBGRADE -** The Contractor shall prepare the surface of the various subgrades receiving subsequent pavement courses to the lines, grades, and dimensions given on the plans. The upper 12 inches of the soil subgrade beneath the pavement section shall be compacted to a minimum relative compaction of 95 percent (90 percent for clay soils) based upon ASTM D1557. The finished subgrades shall be tested and approved by the Soils Engineer prior to the placement of additional pavement courses.

**14.0 AGGREGATE BASE -** The aggregate base material shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base material shall conform to the requirements of Section 26 of the Standard Specifications for Class II material, ¾-inch or 1½-inches maximum size. The aggregate base material shall be compacted to a minimum relative compaction of 95 percent based upon ASTM D1557. The aggregate base material shall be spread in layers not exceeding 6 inches and each layer of aggregate material course shall be tested and approved by the Soils Engineer prior to the placement of successive layers.

**15.0 ASPHALTIC CONCRETE SURFACING -** Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at a central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades, and dimensions shown on the plans. The viscosity grade of the asphalt shall be PG 64-10, unless otherwise stipulated or local conditions warrant more stringent grade. The mineral aggregate shall be Type A or B, ½ inch maximum size, medium grading, and shall conform to the requirements set forth in Section 39 of the Standard Specifications. The drying, proportioning, and mixing of the materials shall conform to Section 39. The prime coat, spreading and compacting equipment, and spreading and compacting the mixture shall conform to the applicable chapters of Section 39, with the exception that no surface course shall be placed when the atmospheric temperature is below 50 degrees F. The surfacing shall be rolled with a combination steel-wheel and pneumatic rollers, as described in the Standard Specifications. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.