GeoTek, Inc.
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January 23, 2023
Project No. 3780-SD

Wermers Companies
5120 Shoreham Place, Suite 150
San Diego, CA 92122

Attention: Mr. Patrick Zabrocki

Subject: **Supplemental Infiltration Recommendation Letter**
APN 203-320-20, -02, -48, -51, 40, and -41
Carlsbad Village Drive and Hope Avenue
Carlsbad, California 92008

Dear Mr. Zabrocki:

GeoTek, Inc. (GeoTek) understands that the proposed BMPs located at the subject site are planned to be modular basins and raised planters at the east and west perimeter of the property. Based on verbal conversations with you, it is our understanding that the updated plan is for additional tree well BMPs to manage stormwater along Grand Avenue. Currently, a dirt pathway abuts a section of Grand Avenue near the northwest corner of the site. The proposed improvements consist of widening Grand Avenue with impervious asphalt concrete and tree wells.

Per your request this letter is provided to supplement design recommendations for the stormwater management specific to the proposed BMPs along Grand Avenue.

PERCOLATION TESTING AND INFILTRATION ANALYSIS

For our previous preliminary geotechnical evaluation report (GeoTek, 2022), two percolation borings, P-1 and P-2, were excavated and tested to identify infiltration characteristics of the on-site soil material. To support our updated recommendations in this supplemental letter, three additional percolation borings and tests were prepared with a manual auger boring. The boring was 4-inches in diameter. Percolation testing was conducted in Borings P-3 through P-5 by a representative of GeoTek. The boreholes were allowed to presoak overnight, and testing was performed on the following day. Percolation testing was performed by adding potable water to the borings, recording the initial depth to water, and allowing the water to percolate for 30 minutes, and the resultant depth to water was then measured. In general, the percolation testing was performed for approximately 6 hours to allow rates to stabilize.

For design of shallow infiltration basins, converting percolation rates to infiltration rates via the Porchet method is generally acceptable and appropriate, as this method factors out the sidewall component of the percolation results and represents the bottom conditions of a shallow basin (infiltration). Therefore, the percolation data were converted to infiltration rates via the Porchet method which is consistent with the City of Carlsbad BMP Design Guidelines.

A summary of the infiltration rates, boring depths, and boring locations including our previous test holes are provided in the following table:

TABLE I
INFILTRATION TEST RESULTS

Test No.	Date Tested	Approximate Boring Depth (Inches)	Infiltration Rate (Inches/Hour)
P-1	4/7/2022	48	0.54
P-2	4/7/2022	54	0.55
P-3	1/6/2023	48	0.97
P-4	1/6/2023	65	0.94
P-5	1/6/2023	60	1.26

Copies of the percolation data sheets, and infiltration conversion sheets (Porchet Method) are included in Appendix A. No factors of safety were applied to the rates provided. Over the lifetime of the infiltration areas, the infiltration rates may be affected by sediment build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate in designing the infiltration system.

It should be noted that the infiltration rates provided above were performed in relatively undisturbed on-site soils. Infiltration rates will vary and are mostly dependent on the underlying consistency of the site soils and relative density. Infiltration rates may be impacted by weight of equipment travelling over the soils, placement of engineered fill and other various factors. GeoTek assumes no responsibility or liability for the ultimate design or performance of the storm water facility.

UPDATED STORMWATER INFILTRATION RECOMMENDATIONS

As the current condition allows for percolation of surface waters along Grand Avenue, infiltration by means of tree wells is considered geotechnically suitable provided potential lateral migration of groundwater is reduced by installation of impermeable liners along the sidewalls.

CLOSURE

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

Should you have any questions after reviewing this supplementary letter, please feel free to contact our office at your convenience.

Respectfully submitted,
GeoTek, Inc.

CERTIFIED ENGINEERING GEOLOGIST
Christopher D. Livesey
2733
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
EDWIN R. CUNNINGHAM
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Christopher D. Livesey
CEG, 2733 Exp. 05/31/23
Vice President

Edwin R. Cunningham
RCE 81687, Exp. 03/31/24
Project Engineer

Enclosure:

Figure 1–Geotechnical Map
Appendix A–Percolation/Infiltration Worksheets

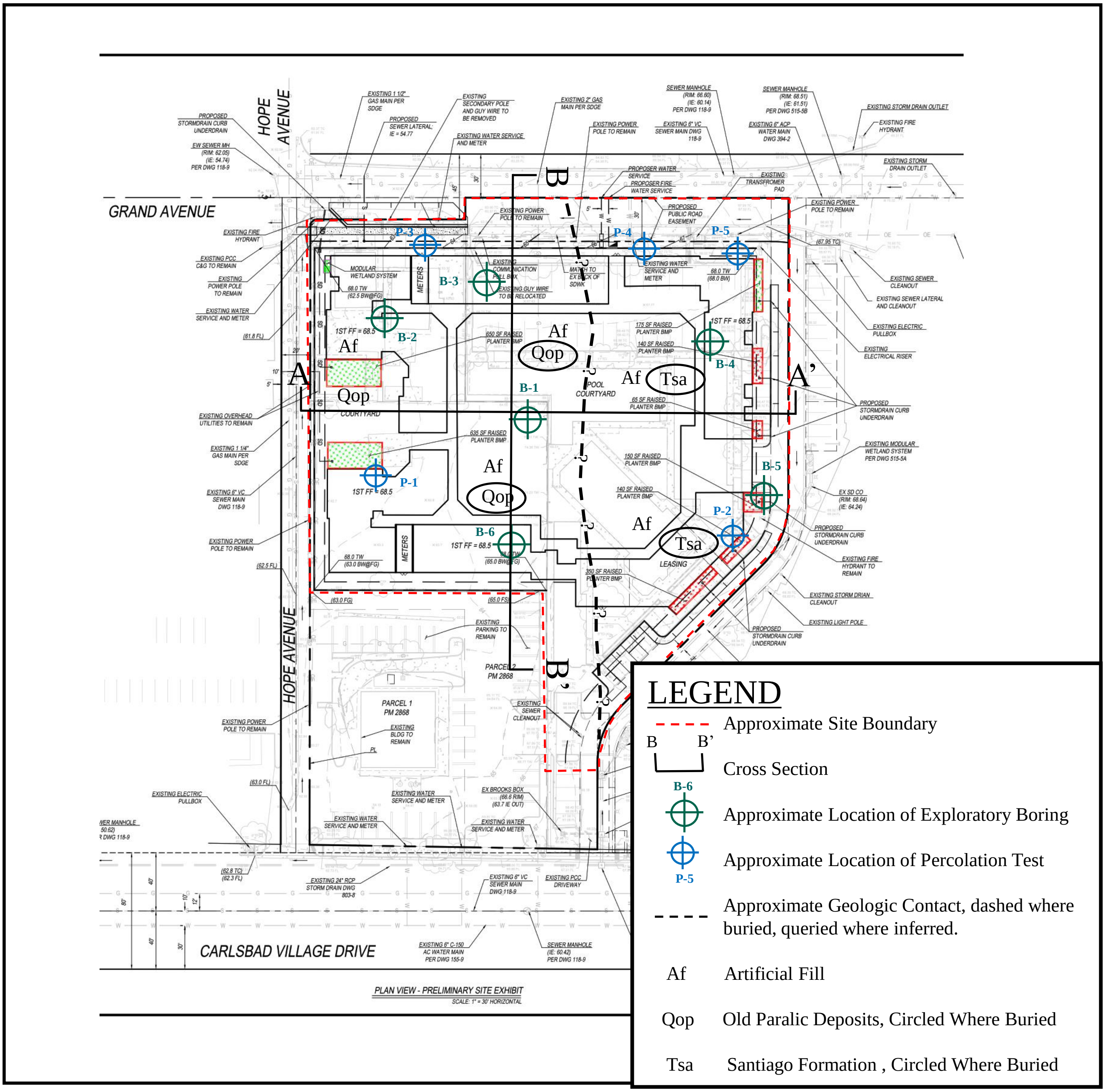
REFERENCES

City of Carlsbad, 2016, "City of Carlsbad BMP Design Manual," Second Update to the February 16, 2016 Manual, Effective January 11, 2023.

City of Carlsbad, 2017, "Engineers Design and Processing Manual," Chapter 2, Section 3.8.

GeoTek, Inc., In-house proprietary information.

Geotek, Inc. 2022, "Preliminary Geotechnical Evaluation, Proposed Hope Apartments, Carlsbad, California," Project No. 3780-SD, dated July 28, 2022.



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Source: Preliminary Site Exhibit, Pasco Laret Suiter & Associates
Scale:

GEOTEK
GEOTECHNICAL | ENVIRONMENTAL | MATERIALS
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FIGURE 2
GEOTECHNICAL MAP
HOPE APARTMENTS
1009 CARLSBAD VILLAGE DRIVE
CARLSBAD, CALIFORNIA

Project No.:	Report Date:	Drawn By:
3780-SD	January 2023	GeoTek, Inc.

APPENDIX A

Percolation/Infiltration Worksheets

PERCOLATION DATA SHEET

Project: Hope Avenue **Job No.:** 3780-SD.

Test Hole No.: P-1 **Tested By:** CDL, **Date:** 4/7/22.

Depth of Hole As Drilled: 48" **Before Test:** 48" **After Test:** 48"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)		Comments
1	7:30	30	48	20	31	11		
2	8:00	30	48	23	34	11		
3	8:30	30	48	23	33	10		
4	9:00	30	48	24	34	10		
5	9:30	30	48	28	35	7		
6	10:00	30	48	24.25	30.75	6.5		
7	10:30	30	48	20.75	28.25	7.5		
8	11:00	30	48	21.50	27.00	5.5		
9	11:30	30	48	20.75	26.25	5.5		
10	12:30	30	48	19.75	24.25	4.5		
11	13:00	30	48	20.25	25.50	5.25		
12	13:30	30	48	18.75	24.00	5.25		
13	14:00	30	48	19.25	24.25	5		

Client:	**Carlsbad Village II, LLC**
Project:	**Hope Avenue Apartments**
Project No:	**3780-SD**
Date:	**4/7/2022**

Boring No. **P-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.25
Test Hole Radius, r =	3.00
Initial Depth to Water, D_O =	19.25
Total Test Hole Depth, D_T =	48

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

$H_O = D_T - D_O$ =	28.75
$H_F = D_T - D_F$ =	23.75
$\Delta H = \Delta D = H_O - H_F$ =	5.00
Havg = $(H_O+H_F)/2$ =	26.25

I_t = 0.54 **Inches per Hour**

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PERCOLATION DATA SHEET

Project: Hope Avenue **Job No.:** 3780-SD.

Test Hole No.: P-2 **Tested By:** CDL, **Date:** 4/7/22.

Depth of Hole As Drilled: 54" **Before Test:** 54" **After Test:** 48"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)		Comments
1	7:15	30	54	24	32	8		
2	7:45	30	54	20	28	8		
3	8:15	30	54	22	29	7		
4	8:45	30	54	23	30	7		
5	9:15	30	54	20.25	28	7.75		
6	9:45	30	54	21.50	27.75	6.25		
7	10:15	30	54	22.50	28.75	6.25		
8	10:45	30	54	21.25	27.75	6.5		
9	11:15	30	54	23.25	29.50	6.25		
10	11:45	30	54	22.75	29.25	6.5		
11	12:15	30	54	21.50	28.50	7		
12	12:45	30	54	20.75	26.25	5.5		
13	13:15	30	54	21.25	27.00	5.75		

Client:	**Carlsbad Village II, LLC**
Project:	**Hope Avenue Apartments**
Project No:	**3780-SD**
Date:	**4/7/2022**

Boring No. **P-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	27.00
Test Hole Radius, r =	3.00
Initial Depth to Water, D_O =	21.25
Total Test Hole Depth, D_T =	54

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

$H_O = D_T - D_O$ =	32.75
$H_F = D_T - D_F$ =	27.00
$\Delta H = \Delta D = H_O - H_F$ =	5.75
$Havg = (H_O+H_F)/2$ =	29.88

I_t = 0.55 **Inches per Hour**

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PERCOLATION DATA SHEET

Project: Hope Apartments **Job No.:** 3780-SD.

Test Hole No.: P-3 **Tested By:** EH, **Date:** 01/06/23.

Depth of Hole As Drilled: 48" **Before Test:** 48" **After Test:** 48"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
1	7:30	30	48	0.5	16	15.5		
2	8:00	30	48	0.5	19	18.5		
3	8:30	30	48	0.5	19.5	19		
4	9:00	30	48	0.5	20	19.5		
5	9:30	30	48	0.5	18	17.5		
6	10:00	30	48	0.5	20	19.5		
7	10:30	30	48	0.5	19.5	19		
8	11:00	30	48	0.5	22.5	22		
9	11:30	30	48	0.5	17.5	17		
10	12:00	30	48	0.5	19.5	19		
11	12:30	30	48	0.5	18	17.5		
12	13:00	30	48	0.5	19.5	19		

Client:	**Carlsbad Village II, LLC**
Project:	**Hope Avenue Apartments**
Project No:	**3780-SD**
Date:	**1/6/2023**

Boring No. **P-3**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	19.50
Test Hole Radius, r =	2.00
Initial Depth to Water, D_O =	0.5
Total Test Hole Depth, D_T =	48

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

$H_O = D_T - D_O$ =	47.50
$H_F = D_T - D_F$ =	28.50
$\Delta H = \Delta D = H_O - H_F$ =	19.00
Havg = $(H_O+H_F)/2$ =	38.00

I_t = 0.97 **Inches per Hour**

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PERCOLATION DATA SHEET

Project: Hope Apartments **Job No.:** 3780-SD.

Test Hole No.: P-4 **Tested By:** EH, **Date:** 01/06/23.

Depth of Hole As Drilled: 65" **Before Test:** 65" **After Test:** 65".

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	∆ In Water Level (Inches)	Rate (minutes per inch)	Comments
1	7:35	30	65	0.5	20	0		
2	8:05	30	65	0.5	26	25.5		
3	8:35	30	65	0.5	25	24.5		
4	9:05	30	65	0.5	26.5	26		
5	9:35	30	65	0.5	25.5	25		
6	10:05	30	65	0.5	27	26.5		
7	10:35	30	65	0.5	24.25	23.75		
8	11:05	30	65	0.5	26.5	26		
9	11:35	30	65	0.5	25	24.5		
10	12:05	30	65	0.5	26	25.5		
11	12:35	30	65	0.5	23.5	23		
12	13:05	30	65	0.5	25.5	25		

Client: Carlsbad Village II, LLC
Project: Hope Avenue Apartments
Project No: 3780-SD
Date: 1/6/2023

Boring No. P-4

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	25.50
Test Hole Radius, r =	2.00
Initial Depth to Water, D_O =	0.5
Total Test Hole Depth, D_T =	65

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

$H_O = D_T - D_O$ =	64.50
$H_F = D_T - D_F$ =	39.50
$\Delta H = \Delta D = H_O - H_F$ =	25.00
$Havg = (H_O+H_F)/2$ =	52.00

I_t = 0.94 **Inches per Hour**

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PERCOLATION DATA SHEET

Project: Hope Apartments **Job No.:** 3780-SD.

Test Hole No.: P-5 **Tested By:** EH, **Date:** 01/06/23.

Depth of Hole As Drilled: 60" **Before Test:** 60" **After Test:** 60".

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Rate (minutes per inch)	Comments
1	7:40	30	60	0.5	25.5	0		
2	8:10	30	60	0.5	31.5	31		
3	8:40	30	60	0.5	30	29.5		
4	9:10	30	60	0.5	31	30.5		
5	9:40	30	60	0.5	29.25	28.75		
6	10:10	30	60	0.5	30	29.5		
7	10:40	30	60	0.5	27.75	27.25		
8	11:10	30	60	0.5	31	30.5		
9	11:40	30	60	0.5	30	29.5		
10	12:10	30	60	0.5	28	27.5		
11	12:40	30	60	0.5	26.25	25.75		
12	13:10	30	60	0.5	29.5	29		

Client:	**Carlsbad Village II, LLC**
Project:	**Hope Avenue Apartments**
Project No:	**3780-SD**
Date:	**1/6/2023**

Boring No. **P-5**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	29.50
Test Hole Radius, r =	2.00
Initial Depth to Water, D_O =	0.5
Total Test Hole Depth, D_T =	60

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

$H_O = D_T - D_O$ =	59.50
$H_F = D_T - D_F$ =	30.50
$\Delta H = \Delta D = H_O - H_F$ =	29.00
$Havg = (H_O+H_F)/2$ =	45.00

I_t = 1.26 **Inches per Hour**

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PRELIMINARY GEOTECHNICAL EVALUATION
PROPOSED HOPE APARTMENTS
APNS 203-320-20, -02, -48. -51, -40, AND -41
CARLSBAD VILLAGE DRIVE AND HOPE AVENUE
CARLSBAD, CALIFORNIA 92008
CT 2022-001/SDP 2022-0006

PREPARED FOR

CARLSBAD VILLAGE II, LLC
3444 CAMINO DEL RIO N, SUITE 202
SAN DIEGO, CALIFORNIA 92108

PREPARED BY

GEOTEK, INC.
1384 POINSETTIA AVENUE, SUITE A
VISTA, CALIFORNIA 92081

PROJECT NO. 3780-SD **JULY 28, 2022**

GeoTek, Inc.
1384 Poinsettia Avenue, Suite A Vista, CA 92081-8505
(760) 599-0509 Office (760) 599-0593 Fax www.geotekusa.com

July 28, 2022
Project No. 3780-SD

Carlsbad Village II, LLC
3444 Camino Del Rio N, Suite 202
San Diego, California 92108

Attention: Mr. Patrick Zabrocki

Subject: **Preliminary Geotechnical Evaluation**
Proposed Hope Apartments
APN 203-320-20, -02, -48, -51, 40, and -41
Carlsbad Village Drive and Hope Avenue
Carlsbad, California 92008

Dear Mr. Zabrocki:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Preliminary Geotechnical Evaluation for the subject project located in the City of Carlsbad, California. This report presents the results of GeoTek's evaluation and provides preliminary geotechnical recommendations for earthwork, foundation design, and construction. Based upon review, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development.

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

Respectfully submitted,
GeoTek, Inc.

CERTIFIED ENGINEERING GEOLOGIST
Christopher D. Livesey
2733
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
EDWIN R. CUNNINGHAM
No. C 81687
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Project Engineer

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions of the project site. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site.
- Excavation of six (6) exploratory borings and collection of soil samples for subsequent laboratory testing.
- Excavation of two auger drilled test holes for subsequent percolation testing.
- Laboratory testing of the soil samples collected during the field investigation.
- Compilation of this geotechnical report which presents GeoTek's findings of pertinent site geotechnical conditions and geotechnical recommendations for site development.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The subject property is located adjacent to the northeast corner of Carlsbad Village Drive and (future extension) of Hope Avenue, in the City of Carlsbad, California (see Figure 1). The project site can be readily identified as 1009 Carlsbad Village Drive, but incudes the broader area of San Diego Assessor's Parcel Numbers 203-320-20, -02, -48, -51, 40, and -41. The subject site is bounded to the north by Grand Avenue, to the east by The Lofts Apartments, to the south by a restaurant (Carl's Jr.), and to the west by Hope Avenue. The eastern portion of the site is occupied by a two-story motel (Carlsbad Village Inn) and a parking lot, the northwest portion of the site is occupied by one to two-story residential structures, and the southwest is a vacant lot with what appears to be two slab-on-grade foundations. The grades on the west half and east half are generally flat, however an approximate four foot tall retaining wall separates the grades from each side. The west half is at an approximate elevation of 63 feet elevation and the east half is at an approximate elevation of 69 feet.

2.2 PROPOSED DEVELOPMENT

Based on the preliminary layout plan provided by Pasco Laret Suiter and Associates, proposed improvements include a 156-residential unit 4-story structure over a two-level subterranean podium parking structure. A courtyard, perimeter flatwork, and stormwater BMP planters are also shown. Associated improvements are anticipated to consist of wet and dry utilities and offsite public road improvements as well as on-site parking and pavement/hardscaping improvements.

It is anticipated that the residential buildings will be of wood frame construction and the subterranean podium-style parking structure is anticipated to be constructed of reinforced concrete. For the purposes of this report, it is assumed preliminary design dead loads for the garage columns are 360 kips with a live load of 110 kips. Once actual loads are known that information should be provided to GeoTek to determine if modifications to the recommendations presented in this report are warranted.

As site planning progresses and additional or revised plans become available, they should be provided to GeoTek for review and comment. If plans vary significantly, additional geotechnical field exploration, laboratory testing and engineering analyses may be necessary to provide specific earthwork recommendations and geotechnical design parameters for actual site development plans.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

GeoTek's field study, conducted on April 6th and 7th 2022, consisted of a site reconnaissance and excavation of six (6) exploratory borings advanced with a conventional CME-75 hollow-stem auger drilling rig mounted on a rubber tired truck. Boring depths ranged from between 16 and 50 feet below existing grade. Excavation of two (2) additional borings, P-1 and P-2, to depths of approximately 5 feet below grade, were performed for percolation testing. A representative from GeoTek visually logged the borings in accordance with the Unified Soil Classification System (USCS), collected relatively undisturbed and loose bulk soil samples for laboratory analysis, and transported the samples to GeoTek's laboratory. Percolation tests were performed the following day. Approximate locations of the exploratory borings and percolation test holes are presented on the Geotechnical Map, Figure 2. A description of material encountered in the test borings is included in Appendix A.

3.2 PERCOLATION TESTING

Two percolation borings (Borings P-1 and P-2) were excavated to depths approximately 4 to 4.5 feet below the existing ground surface. The boring bottom and side walls were scarified and cleaned as feasible of potential drilling fines adhered to the boring walls. The test hole was then filled with potable water to pre-soak. Following overnight pre-soaking, the test holes were filled with water and the drop in water level was recorded every 30 minutes. The test was continued for a minimum of twelve readings and the final reading was used in the calculation of the infiltration rate. The field data was converted to an infiltration rate via the Porchet method. Over the lifetime of the storm water disposal areas, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. The rates presented below do not include a factor of safety, the BMP designer should include appropriate factors of safety in their design.

INFILTRATION TEST RESULTS		
Test No.	Approximate Boring Depth (Inches)	Infiltration Rate (Inches per hour)
P-1	48	0.54
P-2	54	0.55

Copies of the percolation data sheets and infiltration conversion sheets (Porchet Method) are included in Appendix A.

3.3 LABORATORY TESTING

Laboratory testing was performed on bulk soil samples collected during the field explorations. The purpose of the laboratory testing was to evaluate their physical and chemical properties for use in engineering design and analysis. Results of the laboratory testing program, along with a brief description and relevant information regarding testing procedures, are included in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The subject property is located in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends roughly 975 miles from the north and northeasterly adjacent the Transverse Ranges geomorphic province to the peninsula of Baja California. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zones trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province. The Newport-Inglewood-Rose Canyon Fault zone meanders the southwest margin of the province. No faults are shown in the immediate site vicinity on the map reviewed for the area.

4.2 EARTH MATERIALS

A brief description of the earth materials encountered during the current subsurface exploration is presented in the following sections. Based on the field observations and review of published geologic maps the subject site is locally underlain by artificial fill (Af) over Quaternary-age Paralic Deposits (Qop) over Tertiary-age Santiago Formation (Tsa).

4.2.1 Artificial Fill (Map Symbol Af)

Artificial fill was encountered in all borings between one and six feet below existing grades. The fill soils along the west half were as shallow as one to two feet and consisted of reddish to light brown silty sand (SM soil type based upon the Unified Soil Classification System) that may have been disturbed due to demolition and construction. The fills in the eastern half were as deep as six feet and consisted of silty medium to coarse sand (SP soil type) consistent with decomposed granite fill.

4.2.2 Quaternary-age Old Paralic Deposits (Map Symbol Qop)

Old Paralic Deposits were encountered in test borings B-1, B-3 and B-6 at approximate depths between two and twenty feet below the ground. The formational material consisted of medium dense to dense, reddish brown, moist to wet, silty fine to medium sand (SM soil type).

4.2.3 Tertiary-age Santiago Formation (Map Symbol Tsa)

Santiago Formation was encountered in all borings with exception to B-2 which was terminated due to utility conflicts. Santiago Formation was encountered at depths between six and total depths explored (50 feet) and consisted of very dense, light gray, wet, silty fine sandstone (excavates as SM soil type). It should be noted that a significant depth to Santiago Formation was observed between the western and eastern half of the site. The western half encountered Santiago Formation at depths of 20 feet, whereas the eastern half encountered Santiago Formation at near the surface to a depth of 6 feet. The stratigraphical difference between the Santiago Formation between the western and eastern halves of the site are attributed to transgressional depositional episode against a paleo bluff (ancient shoreline and bluff). This interpretation is consistent with the historic geology and depositional environment of the geology setting of the Old Paralic Deposits.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during the recent site exploration. If encountered during earthwork construction, surface water on this site will most likely be the result of precipitation. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was encountered during exploration of the subject site. Based on the anticipated depth of excavation, groundwater is anticipated to be a factor in site design, development and post construction. The following table presents tabulated groundwater data. Data has been obtained by direct measurement during field exploration and research review of the adjacent property (The Lofts), and readily available data.

Summary of Groundwater Data				
Reference ID	Date	Surface Elevation (ft)	Depth (ft)	Elevation (ft)
Boring B-1	4/6/22	63	10	53
Boring B-3	4/6/22	69	10	59
Boring B-4	4/6/22	69	19	50
Boring B-5	4/6/22	69	5	64
Boring B-6	4/6/22	63	11	52
The Lofts (1044 Carlsbad Village Drive)	----	~74	10.5	~63.5

4.4 EARTHQUAKE HAZARDS

4.4.1 Surface Fault Rupture

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is not in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone or a Special Studies Zone (Bryant and Hart, 2007). No faults transecting the site were identified on the readily available geologic maps reviewed. The nearest known active fault is the Newport Inglewood-Rose Canyon fault located about five miles to the southwest of the site.

4.4.2 Liquefaction/Seismic Settlement

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, consolidation and settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain-size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

The liquefaction potential and seismic settlement potential on this site is considered negligible due to the density of the underlying Santiago Formation materials and consideration of proposed design (subterranean podium-style parking structure).

4.4.3 Other Seismic Hazards

The potential for landslides and rockfall is considered negligible, due to the low gradient topographic setting of the site. The potential for secondary seismic hazards such as seiche and tsunami is remote due a review of California Department of Conservation, Geologic Survey, Tsunami Inundation San Luis Rey Quadrangle, 2009.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Development of the site appears feasible from a geotechnical viewpoint provided that the following recommendations are incorporated in the design and construction phases of the development. The following sections present general recommendations for currently anticipated site development plans.

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Carlsbad, the 2019 (or current) California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix C outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix C.

5.2.2 Site Clearing and Preparation

Site preparation should start with removal of deleterious materials, vegetations, and trees/shrubs in the proposed improvement areas. These materials should be disposed of properly off site. Any existing underground improvements, utilities and trench backfill should also be removed or be further evaluated as part of site development operations.

5.2.3 Remedial Grading

Prior to placement of fill materials and in all structural areas, the upper variable, potentially compressible materials should be removed. Removals should include at a minimum all fills. Based on the explored locations, an average removal depth of 3 feet from existing grades may be anticipated. However, considering the proposed subterranean podium style parking structure design, excavation for the parking structure is anticipated to remove all unsuitable soils.

The bottom of the removals should be observed by a GeoTek representative prior to processing the bottom for receiving placement of compacted fills. Depending on actual field conditions encountered during grading, locally deeper and/or shallower areas of removal may be necessary. Prior to fill placement, if fills are needed to reach design grades, the bottom of all removals should be scarified to a minimum depth of six (6) inches, moisture conditioned to slightly above optimum moisture content, and then compacted to at least 90% of the soil's maximum dry density as determined by ASTM D1557 test procedures. The resultant voids from remedial grading/over-excavation should be filled with materials placed in general accordance with Section 5.2.6 Engineered Fill of this report.

5.2.4 Cut/Fill Transition Lots

Grading may result in a cut/fill transition at the proposed building pad finish grades. If a geologic contact of Formational material against fills is encountered at finish pad grades, the cut portion should be over-excavated a minimum of five feet below pad grades, or two feet below the base of proposed footings, whichever is deeper, and be replaced with engineered fill. Cut/fill transitions may occur across ancillary or detached buildings outside of the subterranean parking structure footprint. Depending on the proposed design, an alternative to overexcavating across the entire site, where small fills are needed, could be to compact the fill material to 95 percent compaction relative to ASTM D1557. GeoTek should be contacted for additional considerations on such a case.

5.2.5 Cut Lots

Lots wholly excavated in a cut condition exposing sandstone of the Santiago Formation may remain as cut. This will be the case for the subterranean (basement) parking structure.

5.2.6 Engineered Fill

Onsite materials are generally considered suitable for reuse as engineered fill provided, they are free from vegetation, roots, debris, and rock/concrete or hard lumps greater than six (6) inches in maximum dimension. The earthwork contractor should have the proposed excavated materials to be used as engineered fill at this project approved by the soils engineer prior to placement.

Engineered fill materials should be moisture conditioned to at or above optimum moisture content and compacted in horizontal lifts not exceeding 8 inch in loose thickness to a minimum relative compaction of 90% as determined by ASTM D1557 test procedures.

5.2.7 Excavation Characteristics

Excavations in the onsite materials can generally be accomplished with heavy-duty earthmoving or excavating equipment in good operating condition. Exploratory borings were advanced with relative ease, however when driving the samples, blow counts indicated dense and very dense silty sandstones. A rippability survey was not performed as part of the scope of work under this report. If desired, a rippability survey can be provided. This report should be reviewed by the grading contractors solicited for grading construction, as hallow stem auger boring and excavation with track hoe equipment is not equivalent. Advancement of a boring may be more readily performed compared to a bucket excavator.

5.2.8 Temporary Basement Excavation

Depending on the actual design of the basement footprint, excavation of the basement may be feasible by sloping the excavations. Based on preliminary discussions with the client, a soldier beam and wood lagging with tie-back anchors are preferred for the basement excavation.

It is extremely difficult to predict accurately the amount of deflection of a shored excavation. It should be realized that some deflection will occur. It is estimated that this deflection may be on the order of 1-inch at the top of the shored excavation. If greater than expected deflection occurs during construction, additional bracing may be necessary to minimize adjacent area settlement. If it is desired to reduce the deflection of the shoring, a greater lateral earth pressure (such as at-rest earth pressures) may be used in the shoring design with an increased stiffness of the system.

Soldier pile installations consisting of a concrete encased steel H-beams should be observed by the project geotechnical consultant to verify excavations are drilled into anticipated conditions,

pile excavations are properly prepared and cleaned out, dimensions are achieved, and specific installation procedures are followed. The shoring to be constructed at the site should be surveyed and monitored on a regular basis for any movement. If any significant movement is observed during shoring and construction operations, it should be brought to the immediate attention of the project general contractor, shoring contractor and geotechnical consultant for appropriate corrective measures.
It is recommended that during design of the shoring GeoTek be contacted for review of geotechnical design parameters.

Soldier Piles

Soldier piles installed to support earth pressures are anticipated to be concrete encased H piles, designed by the project structural engineer or shoring engineer. Other reasonable shoring options might be sheet piling and/or secant or tangent drilled piers.

The excavation for the proposed basement is anticipated to expose bedrock materials of the Santiago Formation. Santiago Formation bedrock is also expected to be encountered at the base of some of the excavations. As old paralic deposits and artificial fill overly the bedrock in this portion of the project site, measures to prevent caving should be considered during excavation.

The drilling contractor should be made aware of the presence of bedrock and that appropriate heavy-duty drilling equipment in good working order and/or special drilling techniques will be required. It should be realized that the ability of any particular contractor to excavate the materials encountered will vary based on factors that may or may not be considered in the presented evaluation. All methods available to evaluate rock hardness and associated rippability are interpretive to some extent. As such, experience and judgment are primary factors in such evaluations.

For design of cantilevered shoring, lateral at-rest or active earth pressures may be suitable with a static lateral pressure equal to that developed by a fluid with a density of 40 pounds per cubic foot (pcf) for the active condition and 65 pcf for the at-rest condition for retained material with level backfill. The actual pressure distribution to be used for design should be determined by the structural/shoring engineer. For braced excavations, the shoring could be designed based on a uniform pressure distribution with a pressure value of 22.0 H psf, where H is the wall height in feet.

The above equivalent fluid weights do not include other superimposed loading conditions such as vehicular traffic, hydrostatic (water table), structures, construction materials, seismic conditions, etc. Applicable surcharge loads should be considered and applied by the structural/shoring engineer. The project structural/shoring engineer should design the shoring system using a suitable factor of safety and it should be designed for the lowest adjacent grade.

For the design of soldier piles, an ultimate lateral bearing value (passive value) of 300 pounds per square foot per foot of depth, to a maximum value of 4,500 psf, may be assumed for material below the level of excavation to determine soldier pile depth and spacing. The effective width of the soldier pile can be assumed to be twice the solder pile diameter for passive pressure calculations. However, passive resistance should be ignored within the upper foot due to possible disturbance. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed earth material. The construction of the shoring system should be monitored continuously, and adjacent structures/improvements should be observed for any potential lateral and vertical movement.

Lagging

Design of lagging is the purview of the shoring designer. Lagging should be installed at a maximum 5-foot vertical unsupported cut as the excavation is advanced. Field conditions including earth material classification and seepage during construction may determine if this height of vertical cut needs to be reduced to less than 5 feet. Friable soils were noted in the boring logs and indicates caving or sluffing soil may be encountered during excavation of lagging. The upper one foot of the lagging should be grouted or slurry–filled to assist in diverting surface water from migrating behind the shoring walls.

The lagging should be backfilled immediately as the excavation is advanced in order to minimize the voids created between the lagging and vertical cut and also to reduce the potential for ground subsidence behind the wall. The lagging material should be designed considering it may serve as a permanent installation.

5.2.9 Shrinkage and Bulking

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

Shrinkage is not anticipated to be a factor in quantities estimating, as the site, based on the proposed basement construction will likely be an export site. For excavations in the formational material (Old Paralics and Santiago Formation) silty sandstone, a bulking factor of 10 percent may be considered. Subsidence should not be a factor on the subject site due to the presence of near surface formational material.

5.2.10 Trench Excavations and Backfill

Temporary excavations within the onsite materials should be stable at 1:1 inclinations for short durations during construction, and where cuts do not exceed 10 feet in height. Temporary cuts to a maximum height of 4 feet can be excavated vertically.

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

Utility trench backfill should be compacted to at least 90% relative compaction of the maximum dry density as determined by ASTM D1557 test procedures. Under-slab trenches should also be compacted to project specifications.

Onsite materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than 6± inches are removed.

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

5.3 DESIGN RECOMMENDATIONS

5.3.1 Stormwater Infiltration

Many factors control infiltration of surface waters into the subsurface, such as consistency of native soils and bedrock, geologic structure, fill consistency, material density differences, and existing groundwater conditions. Current site plans indicate several modular basins located on the east and west perimeter of the property, which are shown on Figure 2.

A review of the site conditions and proposed development was performed in general accordance with the City of Carlsbad BMP design manual. The scope of stormwater evaluation was performed to identify infiltration characteristics. As required by the City of Carlsbad BMP design manual, the following bullet points describe required considerations and some optional considerations. The BMPs were evaluated each based on required considerations and all were found to be limiting infiltration by the same restrictive consideration, therefore, to present a simple discussion the following discussion regards all BMPs, unless where specifically discussed.

5.3.1a. Based on a review of www.geotracker.com, environmental impacted sites are not reported within 100 feet of the site.

5.3.1b. Based on a review of Geotracker.com and a reconnaissance of the properties surrounding the site, which were found to be residential, there was not an industrial active building that may pose a lack of source control within 100 feet of the site.

5.3.1c. Based on the surrounding existing development and the understanding that the proposed project will be supported by a municipal sanitation system, the BMPs are not located within 50 feet of septic tanks or leach fields.

5.3.1d. Based on a review of the proposed improvements, the BMPs are not designed within 10 feet of structural retaining walls (basement).

5.3.1e. Based on a review of the proposed improvements, the BMPs are anticipated to be designed within 10 feet of sewer utilities.

5.3.1f. Based on a review of the geologic information for the site and the site specific evaluation that identified shallow dense bedrock within two feet of the surface. Infiltration of surface waters will develop a shallow perched groundwater condition within 10 feet of the BMPs.

5.3.1g. Based on a review of the topography of the site, hydric soils are not prone to exist. However, based on the shallow bedrock of the site and in low gradient proposed areas, hydric soils have the potential to develop due to infiltration of surface waters.

5.3.1h. Based on the shallow bedrock, hazards due to liquefiable soils is considered to be low.

5.3.1i. Based on the proposed design, the BMPs are not located within 1.5 times the height of an adjacent steep slope (basement).

5.3.1j. Based on the site specific study and conclusion, the site is within a predominantly type D soil.

Based on outline numbers 5.3.1d, e, f, and g, the DMA's for the site are classified as restricted for infiltration. As the DMAs are considered to be restricted design infiltration rates are not considered necessary.

Table D.1-1: Considerations for Geotechnical Analysis of Infiltration Restrictions

Restriction Element		Is Element Applicable? (Yes/No)
Mandatory Considerations	BMP is within 100' of Contaminated Soils	No
	BMP is within 100' of Industrial Activities Lacking Source Control	No
	BMP is within 100' of Well/Groundwater Basin	No
	BMP is within 50' of Septic Tanks/Leach Fields	No
	BMP is within 10' of Structures/Tanks/Walls	No
	BMP is within 10' of Sewer Utilities	Yes
	BMP is within 10' of Groundwater Table	Yes
	BMP is within Hydric Soils	No
	BMP is within Highly Liquefiable Soils and has Connectivity to Structures	No
	BMP is within 1.5 Times the Height of Adjacent Steep Slopes (≥25%)	No
	County Staff has Assigned "Restricted" Infiltration Category	No
Optional Considerations	BMP is within Predominantly Type D Soil	Yes
	BMP is within 10' of Property Line	Yes
	BMP is within Fill Depths of ≥5' (Existing or Proposed)	Yes
	BMP is within 10' of Underground Utilities	
	BMP is within 250' of Ephemeral Stream	No
	Other (Provide detailed geotechnical support)	
Result	Based on examination of the best available information, I have **not identified any restrictions** above.	☐ Unrestricted
	Based on examination of the best available information, I have **identified one or more restrictions** above.	☒ Restricted

Existing fills and new fills of 5 to 6 feet are in the eastern half of the site and associated BMPs

Table D.1-1 is divided into Mandatory Considerations and Optional Considerations. Mandatory

Based on the restricted category of the DMA, the proposed basin should be designed for filtration and all sides, including the bottom, should be designed with an impermeable liner to mitigate the potential for groundwater mounding to develop and/or migrate laterally and impact the proposed design improvements.

5.3.2 Hydrological Soil Classification

Summary of Mapped Soil Conditions

The United States Department of Agriculture, Natural Resource Conservation Service, Web Soil Survey (WSS), an internet based map service, classifies the majority of the site (approximately 85% based on area) as MIC Marina loamy coarse sand, 2-9% slopes. The interpretative unit (MIC) is classified as a hydrological Group B.

The WSS classifies map units based on topography, weather, typical soil section in the upper 40 inches, hydrological properties (slope gradient, drainage class, infiltration rates, runoff potential, flood potential) and interpretative groups (land capability classification, hydrologic soil group, hydric soil rating).

The WSS uses the National Soil Survey Handbook (NSSH) and its eDirectives to provide national continuity of soil classifications related to the agricultural industry. Classification is based on laboratory testing of field samples, direct testing in the field, and interpretations from aerial and satellite photography. Samplings and laboratory analyses are performed on select sites and extrapolated beyond the sampled locations.

The NSSH states that "increased mapping has been performed by remote spatial interpretations in lieu of updating surveys based on new or supplemental laboratory data."

The WSS provides the location of data points on their interpretive maps. Data sets are predominately concentrated in agricultural areas and are sparsely available in urban and suburban areas (if at all). A review of the WSS data set was performed. The closest data sample identified is located at the approximate location of El Mirlo Drive, Oceanside, California. That data point is approximately 8 miles northeast of the subject site, in a different geologic unit (Kt-Tonalite/granitics) and presumably obtained prior to the existing development of the residential tract homes at the stated location. The survey methodology on the WSS for the site is noted to be based on aerial photography dated September 13, 2021.

The WSS has classified the site improvement area as a hydrological Group B. It should be noted that the soil classification in the WSS are based on taxonomy principally for agricultural purposes. Classification of soils presented on the logs utilize the Unified Soil Classification Standard, as per industry standards. GeoTek's findings result in inconsistencies between the site and information provided on the WSS. These inconsistencies include:

The WSS classifies the site as a hydrological Group B, which is defined by eDirective 630, Chapter 7 as:

> Group B—Soils in this group have moderately low runoff potential when thoroughly wet. Group B soils typically have less than 10 to 20 percent clay and 50 to 90 percent sand.
>
> The limits on the diagnostic physical characteristics of group B are as follows…..Soils that are deeper than 40 inches to a water impermeable layer and a water table are in group B if the saturated hydraulic conductivity of all soil layers within 40 inches of the surface is between 0.57 and 1.42 inches per hour.

Based on GeoTek's site specific study, the site has formational material Bedrock between 20 to 40 inches below the ground.

> Following the flow chart of Table 7-1: depth to high groundwater is anticipated to be great than 40 inches. Ksat depth rancge is between 0 and 40 inches, limited by near surface bedrock (formational material). As a result, the site is classified as a Group D.

The WSS National Engineering Handbook provides a table summarizing the criteria for assignment of hydrological soil groups in Table 7-1. This table has been presented herein and highlights the criteria that identifies the site, specific to our findings (noted in yellow high lighter):

Table 7-1 (NEH, 2009)

Depth to water impermeable layer 1/	Depth to high water table 2/	K_{sat} of least transmissive layer in depth range	K_{sat} depth range	HSG 3/
<50 cm [<20 in]	—	—	—	D
50 to 100 cm [20 to 40 in]	<60 cm [<24 in]	>40.0 µm/s (>5.67 in/h)	0 to 60 cm [0 to 24 in]	A/D
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in/h)	0 to 60 cm [0 to 24 in]	B/D
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in/h)	0 to 60 cm [0 to 24 in]	C/D
		≤1.0 µm/s (≤0.14 in/h)	0 to 60 cm [0 to 24 in]	D
	≥60 cm [≥24 in]	>40.0 µm/s (>5.67 in/h)	0 to 50 cm [0 to 20 in]	A
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in/h)	0 to 50 cm [0 to 20 in]	B
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in/h)	0 to 50 cm [0 to 20 in]	C
		≤1.0 µm/s (≤0.14 in/h)	0 to 50 cm [0 to 20 in]	D
>100 cm [>40 in]	<60 cm [<24 in]	>10.0 µm/s (>1.42 in/h)	0 to 100 cm [0 to 40 in]	A/D
		>4.0 to ≤10.0 µm/s (>0.57 to ≤1.42 in/h)	0 to 100 cm [0 to 40 in]	B/D
		>0.40 to ≤4.0 µm/s (>0.06 to ≤0.57 in/h)	0 to 100 cm [0 to 40 in]	C/D
		≤0.40 µm/s (≤0.06 in/h)	0 to 100 cm [0 to 40 in]	D
	60 to 100 cm [24 to 40 in]	>40.0 µm/s (>5.67 in/h)	0 to 50 cm [0 to 20 in]	A
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in/h)	0 to 50 cm [0 to 20 in]	B
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in/h)	0 to 50 cm [0 to 20 in]	C
		≤1.0 µm/s (≤0.14 in/h)	0 to 50 cm [0 to 20 in]	D
	>100 cm [>40 in]	>10.0 µm/s (>1.42 in/h)	0 to 100 cm [0 to 40 in]	A
		>4.0 to ≤ 10.0 µm/s (>0.57 to ≤1.42 in/h)	0 to 100 cm [0 to 40 in]	B
		>0.40 to ≤4.0 µm/s (>0.06 to ≤0.57 in/h)	0 to 100 cm [0 to 40 in]	C
		≤0.40 µm/s (≤0.06 in/h)	0 to 100 cm [0 to 40 in]	D

1/ An impermeable layer has a K_{sat} less than 0.01 µm/s [0.0014 in/h] or a component restriction of fragipan; duripan; petrocalcic; orstein; petrogypsic; cemented horizon; densic material; placic; bedrock, paralithic; bedrock, lithic; bedrock, densic; or permafrost.

2/ High water table during any month during the year.

3/ Dual HSG classes are applied only for wet soils (water table less than 60 cm [24 in]). If these soils can be drained, a less restrictive HSG can be assigned, depending on the K_{sat}.

5.3.3 Foundation Design Criteria

Preliminary foundation design criteria, in general conformance with the 2019 CBC, are presented herein. Based on conversations with you, conventional, post-tension, mat slap and tie-downs are being considered. These are typical design criteria and are not intended to supersede the design by the structural engineer. Updated or revised foundations may be needed based on updated design and can be provided upon request. Independent of foundation selection the following recommendations should be considered.

- Groundwater will need to be addressed due to the subterranean parking garage design elevations. A temporary dewatering system will be anticipated to be needed to handle the influx of groundwater anticipated. A permanent system may also be considered.
- The structural engineer should take into account the bouncy force when designing for the subterranean basement if a permanent system is not designed.
- Waterproofing of the retaining walls and subterranean foundation should be addressed by the architect and/or structural engineer.

Based on visual classification of materials encountered onsite and plasticity index of the soils as verified by laboratory testing, site soils are anticipated to exhibit a "very low" (EI ≤ 20) expansion index and "low" (21≤EI<50) expansion index design parameters are provided for conservancy. Additional laboratory testing should be performed at the time of supplemental geotechnical evaluations and upon completion of site grading to verify the expansion potential and plasticity index of the subgrade soils. If not, the more conservative foundation design category should be utilized (low expansive condition).

- An allowable bearing capacity of 4,500 pounds per square foot (psf) may be used for design of continuous and perimeter footings that meet the depth and width requirements in the table above. This value may be increased by 300 pounds per square foot for each additional 12 inches in depth and 200 pounds per square foot for each additional 12 inches in width to a maximum value of 6,500 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).

- Based on experience in the area, structural foundations may be designed in accordance with the 2019 CBC to withstand a total settlement of 1-inch and maximum differential settlement of one-half of the total settlement over a horizontal distance of 40 feet.

- The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 4,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.33 may be used with dead load forces. passive pressure and frictional resistance can be combined without reduction.

- A grade beam should be utilized across large entrances. The depth and the width of the grade beam should be the same as the adjoining footings.

5.3.4 Post Tension Foundation Recommendations

Presented below are post-tensioned (PT) foundation design parameters for the proposed structures at the site. Following site grading, it is anticipated that the upper building pad soils will have a "very low" and "low" expansion index potential. These parameters are in general conformance with *Design of Post-Tensioned Slabs-on-Ground*, Third Edition *with 2008 Supplement* (PTI, 2008). These recommendations are minimal recommendations and are not intended to supersede the design by the project structural engineer.

PT Design for Very Low Expansive Soils

Based upon the Post Tensioning Institute (PTI) "Design of Post-Tensioned Slabs-on-Ground (3rd edition), soils having a "very low" (0-20) expansion potential can be considered "non-active". Since the *2019 California Building Code* (CBC) indicates Post Tensioning Institute (PTI) design methodology is intended for expansive soils conditions, which do not apply to soils having a "very low" (0-20) expansion potential, no e_m or y_m parameters as used in the PTI methodology are provided for soils having a "very low" (0-20) expansion potential.

For "non-active" soils (soils having a "very low" expansion potential), foundation recommendations can be consistent with a Building Research Advisory Board (BRAB) Type II foundation system, which is "lightly reinforced against shrinkage and temperature cracking". This type of foundation system can be reinforced with either steel reinforcement bars or post-tensioned cables. Post-tensioning for this type of foundation system should utilize the recommended design procedure by the referenced PTI manual and 2019 CBC. All reinforcing (steel or post-tensioning) should be properly designed and specified by the structural engineer.

PT Design for Very Low Expansive Soils

Post-tensioned slab foundation design parameters for structures constructed on soils having a "low" expansion index potential for this project are as follows:

GEOTECHNICAL RECOMMENDATIONS FOR POST-TENSIONED SLABS	
Foundation Design Parameter	"Low" Expansion Potential (EI≤50) LL≤38; PI≤15; Material Passing #200 Sieve = 40%; Clay Fines = 10%
Edge Moisture Variation Distance, e_m - Edge Lift (swelling) - Center Lift (shrinkage)	 4.8 ft 9.0 ft
Soil Differential Movement, y_m - Edge Lift (swelling) - Center Lift (shrinkage)	 ≈0.67 in ≈0.29 in
Exterior Perimeter Beam Embedment	12 inches*
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 100% to a depth of 12 inches

*Required depth of perimeter beam/stiffening rib per structural calculations may govern.
The following assumptions were used to generate e_m and y_m values: Thornthwaite Moisture Index = -20; constant suction value = 3.9pF; post-equilibrium case assumed with wet (swelling) cycle going from 3.9pF to 3.0pF and drying (shrinking) cycle going from 3.9pF to 4.5pF.

Post-tensioned slabs should be designed in accordance with the *2019 CBC* and PTI design methodology.

The bottom of the perimeter edge beam/deepened footing should be designed to resist tension forces using either cable or conventional reinforcement, per the structural engineer.

It should be noted that the above recommendations are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

5.3.5 Conventional Foundation Recommendations

Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented below. Following site grading, the site soils are anticipated as having a "very low" (EI≤20) and "low" (21≤EI<50) expansion index in accordance with ASTM D 4829. These are minimal recommendations and are not intended to supersede the design by the project structural engineer.

The conventional foundation elements for the proposed structures should bear entirely in engineered fill soils. Foundations should be designed in accordance with the 2019 or current applicable version of the CBC.

A summary of GeoTek's preliminary foundation design recommendations is presented in the table below:

Design Parameter	Category I "Very Low" Expansion Index	Category II "Low" Expansion Index
Foundation Depth or Minimum Perimeter Beam Depth (below lowest adjacent grade)	1-story = 12 Inches 2-story = 18 Inches 3-story = 24 Inches 4-story = 30 Inches	1-story = 18 Inches 2-story = 18 Inches 3-story = 24 Inches 4-story = 30 Inches
Perimeter or Continuous Beam Foundations Minimum Width (Inches)*	1-story = 12 Inches 2-story = 15 Inches 3-story = 18 Inches 4-story = 21 Inches	1-story = 12 Inches 2-story = 15 Inches 3-story = 18 Inches 4-story = 21 Inches
Isolated Square or Column Foundations Minimum Width (Inches)*	1-story = 24 Inches 2-story = 30 Inches 3-story = 36 Inches 4-story = 42 Inches	1-story = 24 Inches 2-story = 30 Inches 3-story = 36 Inches 4-story = 42 Inches
Minimum Slab Thickness (actual)[1]	4 – Actual	4 – Actual
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab, or No. 3 bars at 24-inch centers	6" x 6" – W2.9/W2.9 welded wire fabric placed in middle of slab, or No. 3 bars at 18-inch centers.
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom	Two No. 5 reinforcing bars, one placed near the top and one near the bottom
Effective Plasticity Index	<15	$15 \leq X \leq 20$
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum of 110% of the optimum moisture content to a depth of at least 18 inches prior to placing concrete

*Code minimums per Table 1809.7 of the 2019 CBC should be complied with.

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

5.3.6 Mat Slab Foundation

The mat slab foundation for the subterranean parking garage should have a minimum embedment depth of 24 inches below lowest adjacent grade and may be designed using an allowable bearing capacity of 4,500 psf. The recommended allowable soil bearing pressures may be increased by one-third for temporary seismic or wind loading. Reinforcement within the mat foundation should be determined by the structural engineer.

For resistance to lateral loads, an allowable coefficient of friction of 0.33 between the base of the foundation elements. In addition, an allowable passive earth resistance equal to an equivalent fluid weight of 300 pounds per cubic foot (pcf) for bedrock acting against the foundations may be used to resist lateral forces. The top foot of passive resistance for foundations should be neglected unless confined by pavement or slab.

A modulus of subgrade reaction (k-value) of 250 pounds per cubic inch (pci) may be considered for design.

5.3.7 Under Slab Moisture Membrane

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2019 CBC Section 1907.1

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g., stake penetrations, tears, punctures from walking on the vapor retarder placed atop the underlying aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture that thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6-mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeability) to achieve the desired performance level.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek does not practice in the field of moisture vapor transmission evaluation/migration since that practice is not a geotechnical discipline. Therefore, GeoTek recommends that a qualified person, such as the flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the building be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person (or persons) should provide recommendations relative to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate. In addition, the recommendations in this report and GeoTek's services in general are not intended to address mold prevention; since GeoTek, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

5.3.8 Miscellaneous Foundation Recommendations

- To reduce moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.

- Spoils from the footing excavations should not be placed in the slab-on-grade areas unless properly moisture-conditioned, compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

5.3.9 Foundation Setbacks

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least 7 feet and need not exceed 40 feet.

- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall stem. This applies to the existing retaining walls along the perimeter if they are to remain.

- The bottom of any existing foundations for structures should be deepened to extend below a 1:1 projection upward from the bottom of the nearest excavation.

5.3.10 Seismic Design Parameters

The site is located at approximately 33.1632, degrees west latitude and -117.3443 degrees north longitude. Site spectral accelerations (Ss and S1), for 0.2 and 1.0 second periods for a risk targeted two (2) percent probability of exceedance in 50 years (MCER) were determined using the web interface provided by SEAOC/OSHPD (https://seismicmaps.org) to access the USGS Seismic Design Parameters. A risk category of II has been utilized as an input design parameter. Due to the very apparent density of the underlying bedrock, a Site Class "C" is considered appropriate for this site. The results, based on ASCE 7-16 and the 2019 CBC, are presented in the following table.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.064g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.385g
Site Coefficient for Site Class "C", F_a	1.2
Site Coefficient for Site Class "C", F_v	1.5
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 0.2 Second, S_{MS}	1.276g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 1.0 Second, S_{M1}	0.578g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.851g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.385g
Site Modified Peak Ground Acceleration (PGA_M)	0.562g
Seismic Design Category	D

5.3.11 Soil Sulfate Content and Corrosivity

Sulfate content test results indicate water soluble sulfate is less than 0.1 percent by weight, which is considered "S0" as per Table 19.3.1.1 of ACI 318-14. Based upon the test results, no special recommendations for concrete are required for this project due to soil sulfate exposure.

5.3.12 Preliminary Pavement Design

Traffic indices have not been provided during this stage of site planning. In addition, site conditions have not been graded to a final design to evaluate specific pavement subgrade conditions. Therefore, the minimum structural sections provided below are based on a preliminary laboratory R-Value of 25 and the assumed traffic indices.

PRELIMINARY ASPHALT PAVEMENT STRUCTURAL SECTION			
Design Criteria	Traffic Index (TI)	Asphaltic Concrete (AC) Thickness (inches)	Aggregate Base (AB) Thickness (inches)
Driveway or Perimeter Private	5.0	4.0	4.0
Grand Avenue (Offsite Public Right of Way)	5.0	4.0	4.0
Grand Avenue (Offsite Public Right of Way)	6.0	4.0	8.0

Actual structural pavement design is to be determined by the geotechnical engineer's testing (R-Value) of the exposed subgrade. Thus, the actual R-Value of the subgrade soils can only be determined at the completion of grading for street subgrades and the above values are subject to change based laboratory testing of the as-graded soils near subgrade elevations.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. Pavement base should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum dry density as determined by ASTM D 1557 test procedures

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

5.3.13 Portland Cement Concrete (PCC)

It is anticipated that Portland Cement Concrete (PCC) pavements will be utilized. Based on the City of Carlsbad minimum design guidelines for driveways, the following recommended minimum PCC pavement section is provided for these areas:

Ground floor of the parking structure
6 Inches Portland Cement Concrete (PCC) over
Santiago Formational Material Subgrade

Driveway into the parking structure or other structural surface pavement
7.5 Inches Portland Cement Concrete (PCC) over
6 Inches Aggregate Base (AB) over
12-inches subgrade compacted to 95% per ASTM D 1557

For the PCC options, it is recommended concrete having a minimum 28-day flexural strength (or modulus of rupture (MOR)) of 650 psi be used. A "pavement"-type concrete mix (not a "slab"-type) concrete mix should be use. Air-entrainment (5 ± 2 percent) of the concrete should be provided. Sulfate resistant concrete is not required. A maximum joint spacing of 12 feet is also recommended. Reinforcement of the concrete should be provided as recommended by the structural engineer.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Preliminary grading plans are not yet available. Retaining wall foundations embedded a minimum of 18 inches into engineered fill or dense formational materials should be designed using an allowable bearing capacity of 4,500 pounds per square foot (psf) may be used for design of continuous and perimeter footings that meet the depth and width requirements in the table above. This value may be increased by 300 pounds per square foot for each additional 12 inches in depth and 200 pounds per square foot for each additional 12 inches in width to a maximum value of 6,500 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads). Passive pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 4,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.33 may be used with dead load forces. Passive pressure and frictional resistance can be combined without reduction.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in the table below for specific slope gradients of retained materials.

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Select Backfill*
Level	40
2:1	65

*Select backfill should consist of approved materials with an EI≤20 and should be provided throughout the active zone.

The above equivalent fluid weights do not include other superimposed loading conditions such as expansive soil, vehicular traffic, structures, seismic conditions or adverse geologic conditions.

5.4.2 Restrained Retaining Walls

Any retaining wall that will be restrained prior to placing backfill or walls that have male or reentrant corners should be designed for at-rest soil conditions using an equivalent fluid pressure of 65 pcf (select backfill), plus any applicable surcharge loading. For areas having male or reentrant corners, the restrained wall design should extend a minimum distance equal to twice the height of the wall laterally from the corner, or as otherwise determined by the structural engineer.

5.4.3 Seismic Earth Pressures on Retaining Walls

As required by the 2019 CBC, walls with a retained height greater than six feet are required to include an incremental seismic earth pressure in the wall design. Based upon review, a wall with a retained height of up to approximately 10 feet is planned at the site.

Based on the planned site wall heights and an $S_{DS}/2.5$ value of 0.340g, the following incremental seismic earth pressures may be used in the design of site walls greater than six feet in height:

Wall Scenario	Additional Equivalent Fluid Pressure (PCF)
Level Unrestrained	$18H^2$
2:1 Sloping Backfill Unrestrained	$29H^2$
Restrained	$28H^2$

The point of application of the incremental seismic earth pressure is at 1/3H, where H is the retained height.

5.4.4 Wall Backfill and Drainage

Wall backfill should include a minimum one (1) foot wide section of ¾ to 1-inch clean crushed rock (or approved equivalent). The rock should be placed immediately adjacent to the back of wall and extend up from the backdrain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. If the walls are designed using the "select" backfill design parameters, then the "select" materials shall be placed within the active zone as defined by a 1:1 (H:V) projection from the back of the retaining wall footing up to the retained surface behind the wall. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs.

The backfill materials should be placed in lifts no greater than 8-inches in thickness and compacted to a minimum of 90% of the maximum dry density as determined in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC, or approved equivalent) in a minimum of one (1) cubic foot per lineal foot of 3/8 to one (1) inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and be directed (via a solid outlet pipe) to an appropriate disposal area.

As an alternative to the drain, rock and fabric, a pre-manufactured wall drainage product (example: Mira Drain 6000 or approved equivalent) may be used behind the retaining wall. The wall drainage product should extend from the base of the wall to within two (2) feet of the ground surface. The subdrain should be placed in direct contact with the wall drainage product.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

6. CONCRETE FLATWORK

6.1 GENERAL CONCRETE FLATWORK

6.1.1 Exterior Concrete Slabs and Sidewalks

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness with 6" x 6" – W1.4/W1.4 welded wire fabric, placed in the middle of slab. It is recommended that control joints be placed in two directions spaced the numeric equivalent roughly 24 times the thickness of the slab in inches (e.g., a 4-inch slab would have control joints at 96 inch [8 feet] centers). These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices typically utilized in construction.

Presaturation of flatwork subgrade should be verified to be a minimum of 100% of the soils optimum moisture to a depth of 12 inches for soils having a "very low" expansive index potential. Subgrade having a "low" expansion index potential should be verified to be moisture conditioned to a minimum of 110% of the soils optimum moisture at a depth of 12 inches below subgrade.

7. POST CONSTRUCTION CONSIDERATIONS

7.1 LANDSCAPE MAINTENANCE AND PLANTING

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

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

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be warranted and advisable. GeoTek could discuss these issues, if desired, when plans are made available.

7.2 DRAINAGE

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings. Site drainage should conform to Section 1804.4 of the 2019 CBC. Roof gutters and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter pan or directly to the storm drain system. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

7.3 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

GeoTek recommends that site grading, specifications, retaining wall/shoring plans and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. Additional recommendations may be necessary based on these reviews. It is also recommended that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek's representative perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Observe temporary shoring construction such as soldier beam excavation, basement excavation, and tie-back installation.
- Evaluate the suitability of on-site and import materials for fill placement and collect soil samples for laboratory testing when necessary.
- Observe the fill for uniformity during placement including utility trenches.

- Observe and test the fill for field density and relative compaction.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

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

8. LIMITATIONS

The scope of this evaluation is limited to the area explored that is shown on the Geotechnical Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of proposed construction as indicated to us by the client. The scope is based on GeoTek's understanding of the project and the client's needs, GeoTek's proposal (Proposal No. P-0300822-SD) dated March 11th, 2022, and geotechnical engineering standards normally used on similar projects in this region.

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

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

9. SELECTED REFERENCES

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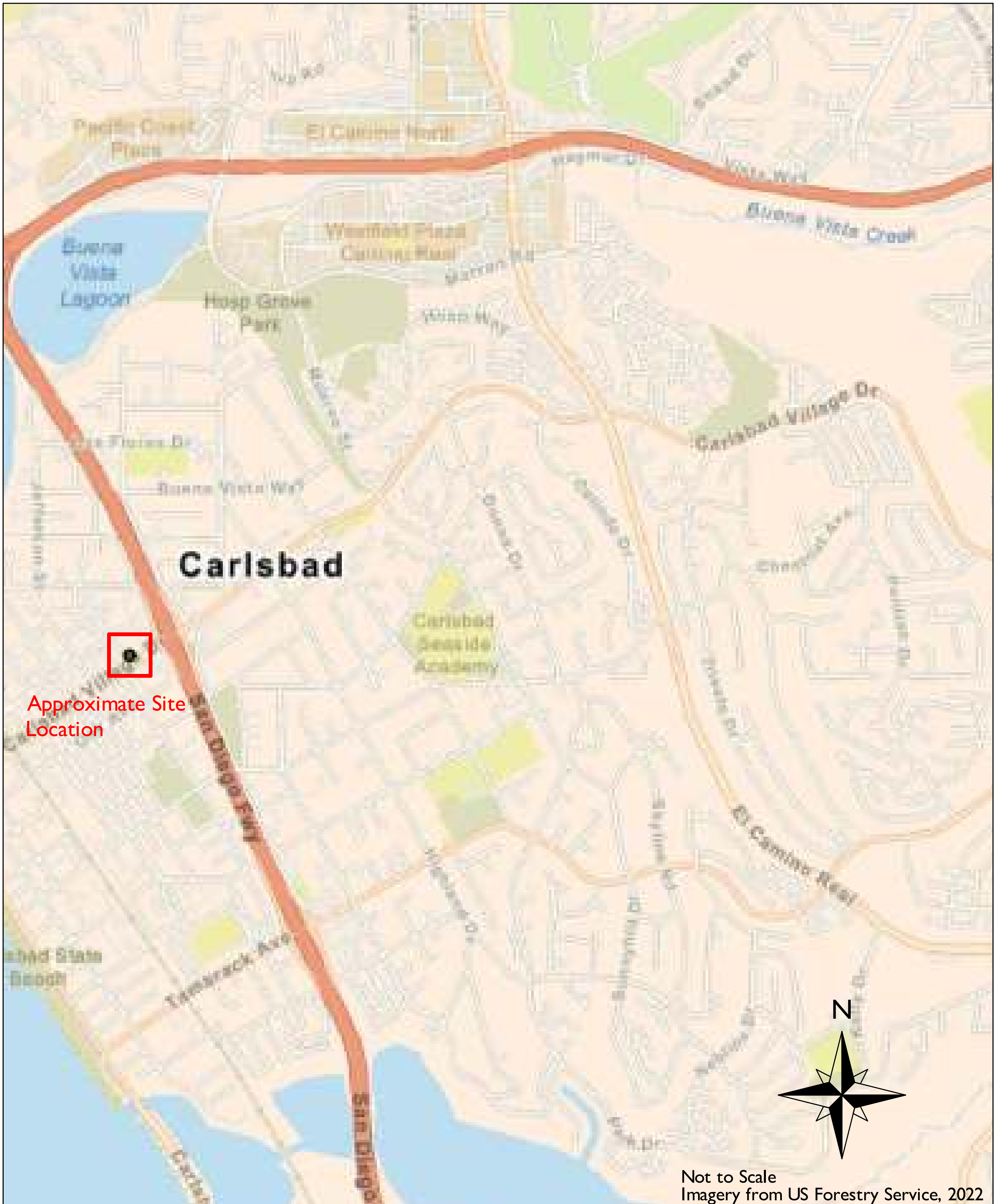


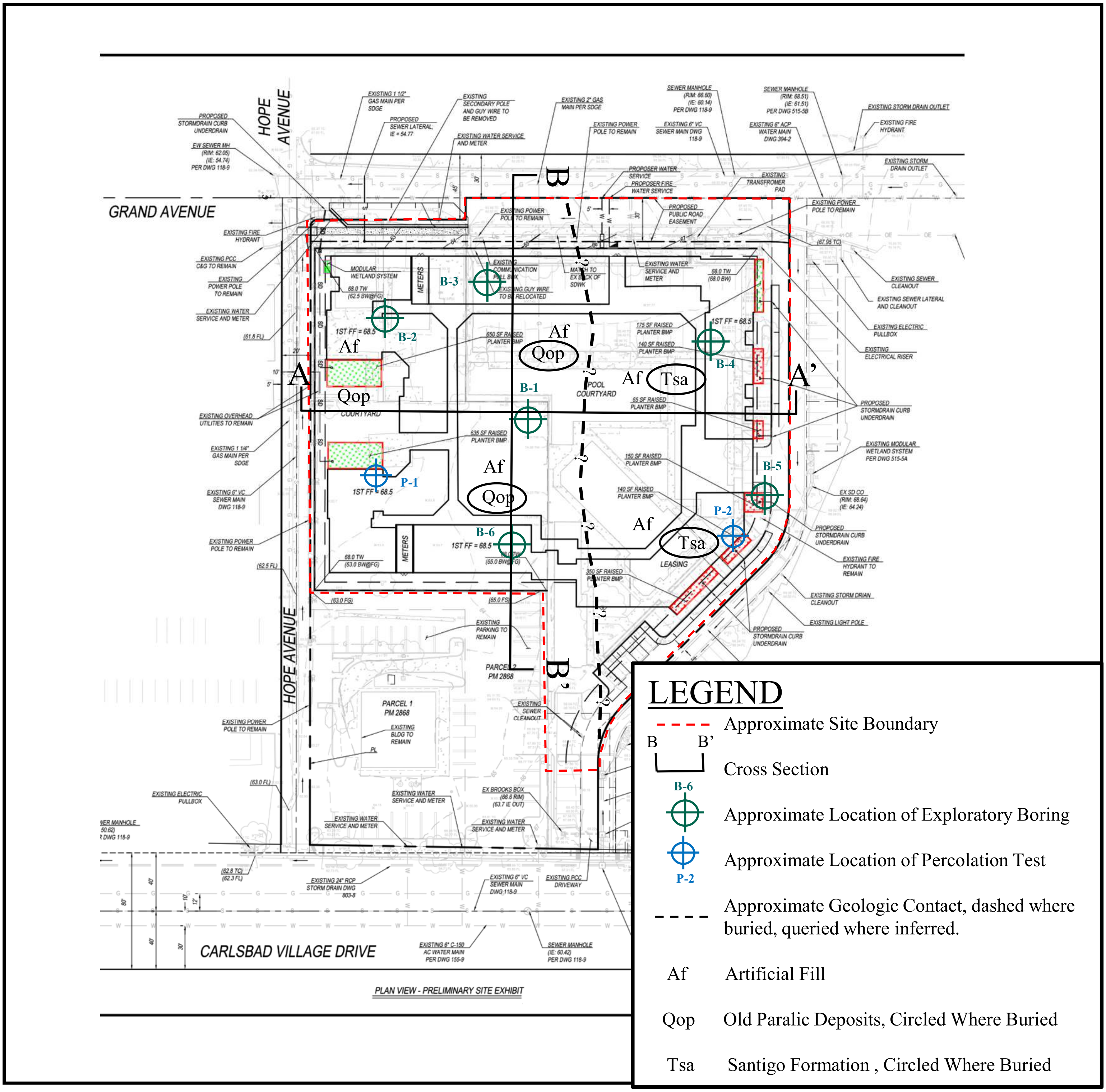
Carlsbad Village II, LLC
950 Carlsbad Village Drive
APNs: 203-320-20, -02, -48, -51, -40, -41
Carlsbad, California

PN: 3780-SD | DATE: April 2022

Figure 1

Site Location Map

GEOTEK
1384 Poinsettia Avenue, Suite A
Vista, California 92081

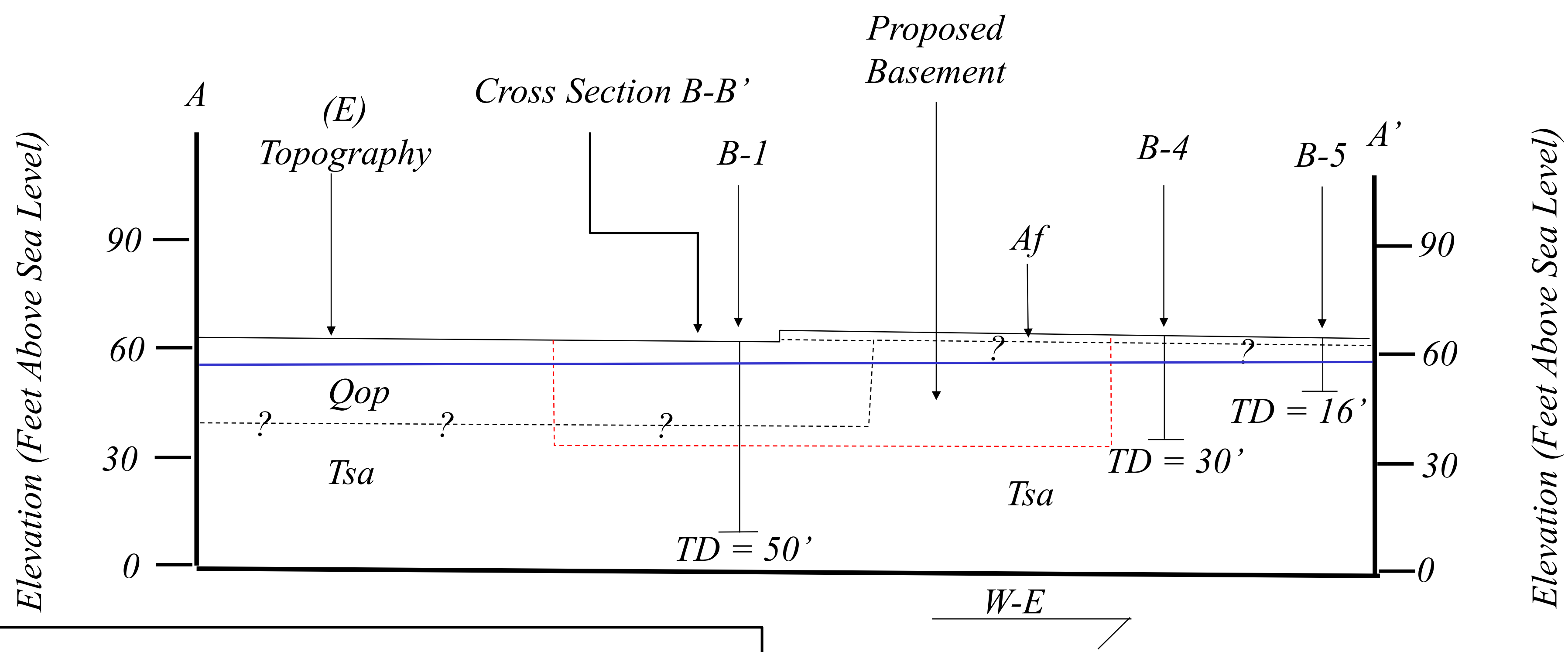


Source: Preliminary Site Exhibi, Pasco Laret Suiter & Associates
Scale: **1" = ~90'**

GEOTEK
GEOTECHNICAL | ENVIRONMENTAL | MATERIALS
1384 Poinsettia Avenue, Suite A, Vista, CA 92081
(760) 599-0509 (phone) / (760) 599-0593 (FAX)

FIGURE 2
GEOTECHNICAL MAP
HOPE APARTMENTS
1009 CARLSBAD VILLAGE DRIVE
CARLSBAD, CALIFORNIA

Project No.:	Report Date:	Drawn By:
3780-SD	5/9/22	CDL



LEGEND

- - - *Approximate Geologic Contact, dashed where buried, queried where inferred.*

—— *Approximate Groundwater Elevation*

Af *Artificial Fill*

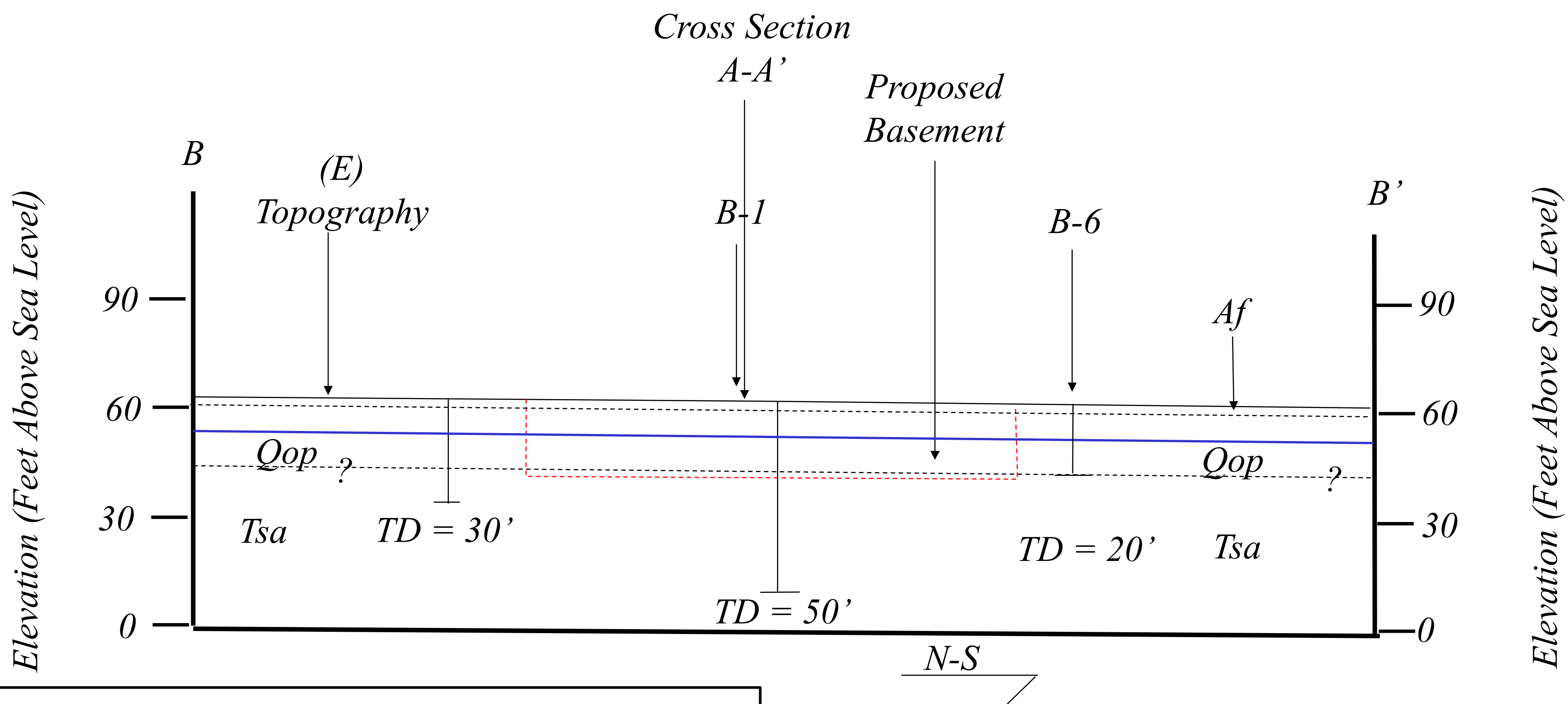
Qop *Old Paralic Deposits*

Tsa *Santiago Formation*

GEOTEK

Cross Section A-A' **Hope Apartments**

May 2022 *PN: 3780-SD* **Figure 3**



LEGEND

- - - *Approximate Geologic Contact, dashed where buried, queried where inferred.*

— *Approximate Groundwater Elevation*

Af *Artificial Fill*

Qop *Old Paralic Deposits*

Tsa *Santiago Formation*

GEOTEK

Cross Section B-B'

Hope Apartments

May 2022 PN: 3780-SD

Figure 4

APPENDIX A

LOGS OF EXPLORATION

AND

PERCOLATION WORKSHEETS

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

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

Bulk Samples (Small)
These samples are normally small bags of earth materials less than 10 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Ring Samples

B – BORING/TRENCH LOG LEGEND

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

SOILS

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

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip

C: Contact line

………..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
———	Thick solid line denotes end of boring/trench

(Additional denotations and symbols are provided on the log of borings/trenches)

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Carlsbad Village II, LLC	**DRILLER:**	Baja Exploration
PROJECT NAME:	Hope Apartments	**DRILL METHOD:**	6" Dia Hollowstem Auger
PROJECT NO.:	3780-SD	**HAMMER:**	140lbs/30in
LOCATION:	Carlsbad, CA	**ELEVATION:**	63 feet
LOGGED BY:	MRF		
OPERATOR:	Manny		
RIG TYPE:	CME-75		
DATE:	4/6/2022		

BORING NO.: B-1

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
	Large Bulk		BB-1	SP	**Artificial Fill (Af)** Fine SAND, brown, dry, soft, roots			
				SM	**Old Paralic Deposits (Qop)** Silty fine to medium SAND, light brown, dry, loose, trace gravels			
5	Ring	18 18 33	R-1	SM	Silty fine to medium SAND, brown to reddish brown, dry, very dense	3.2	107.9	
10	SPT	30 50/9	S-1	SM	Silty fine to medium SAND, brown to reddish brown, moist, very dense	11.2		
15	Ring	50/6	R-2	SM	Silty fine to medium SAND, brown to reddish brown, moist, very dense	16.5	113.5	
				▽	Water on sampling rod			
20	SPT	50/5	S-2		**Santiago Formation (Tsa)** Mud coming out of cuttings Silty fine to medium SANDSTONE, gray, wet, very dense	19.8		
25	Ring	50/4	R-3		Silty fine SANDSTONE, gray, wet, dense, well cemented			SH
30	SPT	50/6	S-3		Silty fine SANDSTONE, gray, wet, dense	11.2		

Laboratory Testing: Water Content (%), Dry Density (pcf), Others

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Carlsbad Village II, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	MRF
PROJECT NAME:	Hope Apartments	**DRILL METHOD:**	6" Dia Hollowstem Auger	**OPERATOR:**	Manny
PROJECT NO.:	3780-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	Carlsbad, CA	**ELEVATION:**	63 feet	**DATE:**	4/6/2022

BORING NO.: B-1 Cont.

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
35	Ring	50/3	R-4		Silty fine SANDSTONE, gray, wet, very dense	16.4	117.9	
40	SPT	50/5	S-4		Silty fine to medium SANDSTONE, wet, very dense, friable	15.8		
45	Ring	50/2	R-2		Silty fine SANDSTONE, grayish brown, moist, cemented very dense	10.6	131.9	
50	SPT	50/2	S-5		Clayey fine SANDSTONE, brownish gray, moist, very dense	13.6		

HOLE TERMINATED AT 50.1 FEET

Groundwater encountered at 17 feet during drilling
Static Groundwater Observed at 3pm is 10.3'
Static Groundwater Observed On 4/7 at 7am is 9.8'

55

60

LEGEND

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

Lab testing:

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Carlsbad Village II, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Hope Apartments	**DRILL METHOD:**	6" Dia Hollowstem Auger	**OPERATOR:**	Manny
PROJECT NO.:	3780-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	Carlsbad, CA	**ELEVATION:**	63 feet	**DATE:**	4/6/2022

BORING NO.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Planter - 2 inches of mulch			
					Artificial Fill (Af)			
				SM	Silty fine SAND, dark reddish brown, moist, loose			
					Hand Auger: 1st Hole - Encounter direct burial wire 2nd Hole - 18 inches east, plastic pipe 3rd Hole - 18 inches east, vcp			
5					**Hole Abandoned**			
10								
15								
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT: Carlsbad Village II, LLC	**DRILLER:** Baja Exploration	**LOGGED BY:** CDL
PROJECT NAME: Hope Apartments	**DRILL METHOD:** 6" Dia Hollowstem Auger	**OPERATOR:** Manny
PROJECT NO.: 3780-SD	**HAMMER:** 140lbs/30in	**RIG TYPE:** CME-75
LOCATION: Carlsbad, CA	**ELEVATION:** 69 feet	**DATE:** 4/6/2022

BORING NO.: B-3

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Asphalt 3 inches over subgrade			
	Large Bulk		BB-1	SM	**Artificial Fill (Af)** Silty medium to coarse SAND (Decomposed Granite), brown, moist			MD, SH SR
					Silty fine SAND, dark brown, moist, brick fragments			
5	Ring	15 18 23	R-1	SM	**Old Paralic Deposits (Qop)** Silty fine SAND, dark reddish brown, moist Silty fine SAND, dark reddish brown, moist, faint palsofacies			
10	SPT	11 13 15	S-1	SM	Silty fine to medium SAND, reddish brown, moist, dense, outside sampler and stem is wet	8.1		
15	Ring	27 28 45	R-2	SP	Poorly graded coarse SAND, black, wet, very dense Poorly graded medium SAND, brown, wet, very dense, trace well round large gravel Large well round gravel in cuttings, driling slows, standing rig			SH
20	SPT	50/5	S-2		**Santiago Formation (Tsa)** Silty fine SANDSTONE, light gray, wet, very dense, sluff	23.3		
25	Ring	50/2	R-3		Silty fine to medium SANDSTONE, gray brown, wet, very dense	14.9	115.5	
					Silty fine SANDSTONE, pale gray, moist, very dense			
30	SPT	50/5	S-3		**HOLE TERMINATED AT 30.5 FEET** Groundwater encountered at 10 feet			

LEGEND

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

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Carlsbad Village II, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Hope Apartments	**DRILL METHOD:**	6" Dia Hollowstem Auger	**OPERATOR:**	Manny
PROJECT NO.:	3780-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	Carlsbad, CA	**ELEVATION:**	69 feet	**DATE:**	4/6/2022

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Asphalt 2 inches over subgrade			
					Artificial Fill (Afu)			
				SM	Silty medium to coarse SAND (Decomposed Granite), reddish brown, moist, large gravel			
				SM	Silty fine SAND, dark brown			
5	SPT	9	S-1	SM	Silty fine to medium SAND, dark brown, moist			
		10			**Santiago Formation (Tsa)**			
		18			Silty fine SANDSTONE, light brownish gray, moist, medium dense	8.1		
10	Ring	15	R-1		Silty fine SANDSTONE, olive brown, moist, oxidized staining			
		50/5						SH
					Well rounded GRAVEL in cuttings and increased moisture			
15	SPT	50/2	S-2		Silty fine SANDSTONE, olive brown, moist, low sample recovery	11.1		
20	Ring	50/2	R-2		Mud cuttings, no recovery			
					Very hard drilling, slow advancement, footings might be difficult to excavate			
					inside of auger plugged			
25	SPT	50/6	S-3		Silty fine SANDSTONE, pale olive brown, moist to very moist	7.5		
					Silty fine SANDSTONE, pale olive brown, moist to very moist	7.9		
					HOLE TERMINATED AT 30.5 FEET			
30	SPT	50/5	S-4		Groundwater encountered at 19 feet			

LEGEND

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

Lab testing:

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

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Carlsbad Village II, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Hope Apartments	**DRILL METHOD:**	6" Dia Hollowstem Auger	**OPERATOR:**	Manny
PROJECT NO.:	3780-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	Carlsbad, CA	**ELEVATION:**	69 feet	**DATE:**	4/6/2022

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
	Large Bulk				Asphalt 3 inches over subgrade			
			BB-1		**Artificial Fill (Afu)**			
				SM	Silty medium to coarse SAND with gravel (Decomposed Granite), reddish brown			SR
				SM	Silty fine SAND, dark brown, moist			
5	Ring	8 17	R-1	SM	Silty fine SAND, dark brown, moist			
		35			**Santiago Formation (Tsa)** Silty fine SANDSTONE, light blue gray, very moist, very dense	10.6	121.7	
10	SPT	12 16 18	S-1		Clayey SANDSTONE, mottled olive gray and olive brown, very moist, very dense, sample is friable	4.4		
					Increased density, harder drilling			
15	SPT	30 50/5	S-2		Sitly fine SANDSTONE, olive gray, moist, friable, some oxidized stained fractures	8.0		
					HOLE TERMINATED AT 16 FEET			
					Percured groundwater at 10 feet?			
20								
25								
30								

LEGEND

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

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

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Carlsbad Village II, LLC	**DRILLER:**	Baja Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Hope Apartments	**DRILL METHOD:**	6" Dia Hollowstem Auger	**OPERATOR:**	Manny
PROJECT NO.:	3780-SD	**HAMMER:**	140lbs/30in	**RIG TYPE:**	CME-75
LOCATION:	Carlsbad, CA	**ELEVATION:**	63 feet	**DATE:**	4/6/2022

BORING NO.: B-6

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Artificial Fill (Af)			
			BB-1 0'-20'	SP	Fine SAND, brown, dry, soft, roots			RV
					Old Paralic Deposits (Qop)			
				SM	Silty fine to medium SAND, light brown, dry, loose, trace gravels			
5	SPT	10 23 30	S-1	SM	Silty fine to medium SAND, light brown			
10	SPT	50/5	S-2	SM	Silty fine to medium SAND, light reddish brown, wet, very dense			
					Cuttings show increase in moisture			
15	SPT	50/6	S-3		**Santiago Formation (Tsa)** Silty fine SANDSTONE, olive gray, very moist			
20	SPT	50/3	S-4		Silty fine SANDSTONE, olive gray, very moist			
					HOLE TERMINATED AT 20 FEET			
					Groundwater encountered at 10 feet Backfilled with soil cuttings			
25								
30								

LEGEND

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

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

PERCOLATION DATA SHEET

Project: Hope Avenue **Job No.:** 3780-SD.

Test Hole No.: P-1 **Tested By:** CDL, **Date:** 4/7/22.

Depth of Hole As Drilled: 48" **Before Test:** 48" **After Test:** 48"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)		Comments
1	7:30	30	48	20	31	11		
2	8:00	30	48	23	34	11		
3	8:30	30	48	23	33	10		
4	9:00	30	48	24	34	10		
5	9:30	30	48	28	35	7		
6	10:00	30	48	24.25	30.75	6.5		
7	10:30	30	48	20.75	28.25	7.5		
8	11:00	30	48	21.50	27.00	5.5		
9	11:30	30	48	20.75	26.25	5.5		
10	12:30	30	48	19.75	24.25	4.5		
11	13:00	30	48	20.25	25.50	5.25		
12	13:30	30	48	18.75	24.00	5.25		
13	14:00	30	48	19.25	24.25	5		

Client:	**Carlsbad Village II, LLC**
Project:	**Hope Avenue Apartments**
Project No:	**3780-SD**
Date:	**4/7/2022**

Boring No. **P-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.25
Test Hole Radius, r =	3.00
Initial Depth to Water, D_O =	19.25
Total Test Hole Depth, D_T =	48

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

$H_O = D_T - D_O$ =	28.75
$H_F = D_T - D_F$ =	23.75
$\Delta H = \Delta D = H_O - H_F$ =	5.00
Havg = $(H_O+H_F)/2$ =	26.25

I_t = 0.54 **Inches per Hour**

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PERCOLATION DATA SHEET

Project: Hope Avenue **Job No.:** 3780-SD.

Test Hole No.: P-2 **Tested By:** CDL, **Date:** 4/7/22.

Depth of Hole As Drilled: 54" **Before Test:** 54" **After Test:** 48"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)		Comments
1	7:15	30	54	24	32	8		
2	7:45	30	54	20	28	8		
3	8:15	30	54	22	29	7		
4	8:45	30	54	23	30	7		
5	9:15	30	54	20.25	28	7.75		
6	9:45	30	54	21.50	27.75	6.25		
7	10:15	30	54	22.50	28.75	6.25		
8	10:45	30	54	21.25	27.75	6.5		
9	11:15	30	54	23.25	29.50	6.25		
10	11:45	30	54	22.75	29.25	6.5		
11	12:15	30	54	21.50	28.50	7		
12	12:45	30	54	20.75	26.25	5.5		
13	13:15	30	54	21.25	27.00	5.75		

Client: Carlsbad Village II, LLC

Project: Hope Avenue Apartments

Project No: 3780-SD

Date: 4/7/2022

Boring No. P-2

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	27.00
Test Hole Radius, r =	3.00
Initial Depth to Water, D_O =	21.25
Total Test Hole Depth, D_T =	54

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

$H_O = D_T - D_O$ =	32.75
$H_F = D_T - D_F$ =	27.00
$\Delta H = \Delta D = H_O - H_F$ =	5.75
Havg = $(H_O+H_F)/2$ =	29.88

I_t = 0.55 **Inches per Hour**

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APPENDIX B

RESULTS OF LABORATORY TESTING

SUMMARY OF LABORATORY TESTING

Identification and Classification

Soils were identified visually in general accordance with the standard practice for description and identification of soils (ASTM D 2488). The soil identifications and classifications are shown on the Logs of Exploration in Appendix A.

Moisture Density Modified Proctor

Laboratory testing was performed on one sample collected during the subsurface exploration for compaction characteristics. The laboratory maximum dry density and optimum moisture content for the soil was determined in general accordance with ASTM Test Method D 1557 procedures. The test results are graphically presented in Appendix B.

Full Corrosion Suite

A full corrosion series was performed in general accordance with several ASTM Test Methods. The samples were obtained from Test Pit TP-6 and TP-7 and tested by Project X Engineering.

Atterberg Limits

The tests were performed in general accordance with ASTM D 4318. The test results are presented in Appendix B.

Percent of Soil Passing No 200 Sieve

The amount of soil finer than No. 200 sieve was determined for two sandy samples collected from the site. The tests were performed in general accordance with ASTM D 1140. The test results are presented in Appendix B.

Direct Shear

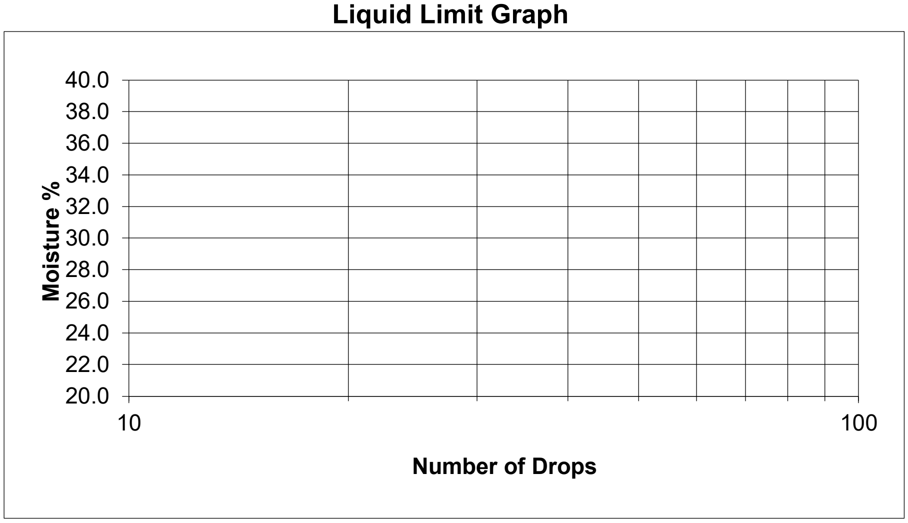
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080 procedures. The rate of deformation is approximately 0.35 inches per minute. The samples were sheared under varying confining loads to determine the coulomb shear strength parameters, angle of internal friction and cohesion. One test was performed on a bulk sample that was remolded to approximately 90 percent of the maximum dry density as determined by ASTM D 1557. The results of the testing are graphically presented in Appendix B.

ATTERBERG LIMITS DATA

Field Classification	Light Brown Silty Sand	Job No.	3780-SD
Sample Number	B-5 BB-1	Client	Carlsbad Village II, LLC
Sample Type	Big Bulk	Project	Hope Apartments
Location	950 Carlsbad Village Drive		
Tested by:	CH		

	Plastic Limit		Liquid Limit			
Number of Blows			15	20	28	37
Determination	1	2	1	2	3	4
Dish						
Wt. of Dish + Wet Soil	0.00	0.00	0.00	0.00	0.00	0
Wt. of Dish + Dry Soil	0.00	0.00	0.00	0.00	0.00	0
Wt. of Moisture	0.00	0.00	0.00	0.00	0.00	0.00
Wt. of Dish	0.85	0.85	0.86	0.86	0.86	0.86
Wt. of Dry Soil	-0.85	-0.85	-0.86	-0.86	-0.86	-0.86
Moisture Content %	0.0	0.0	0.0	0.0	0.0	0.0

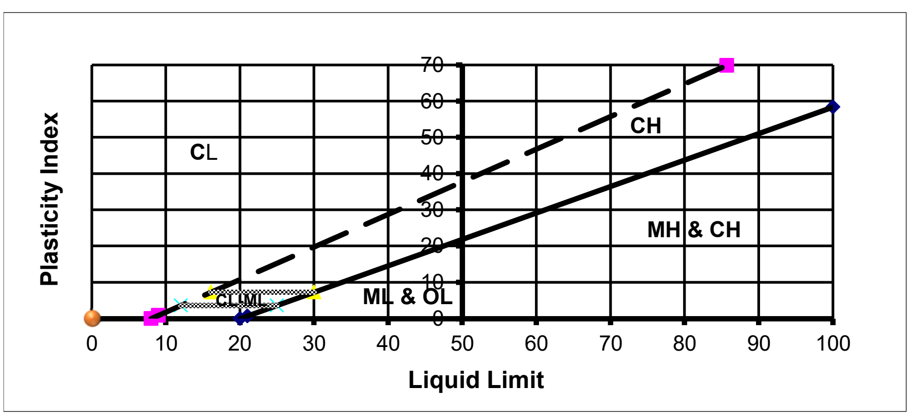
Liquid Limit Graph



Liquid Limit
0

Plastic Limit
0

Plasticity Index
0



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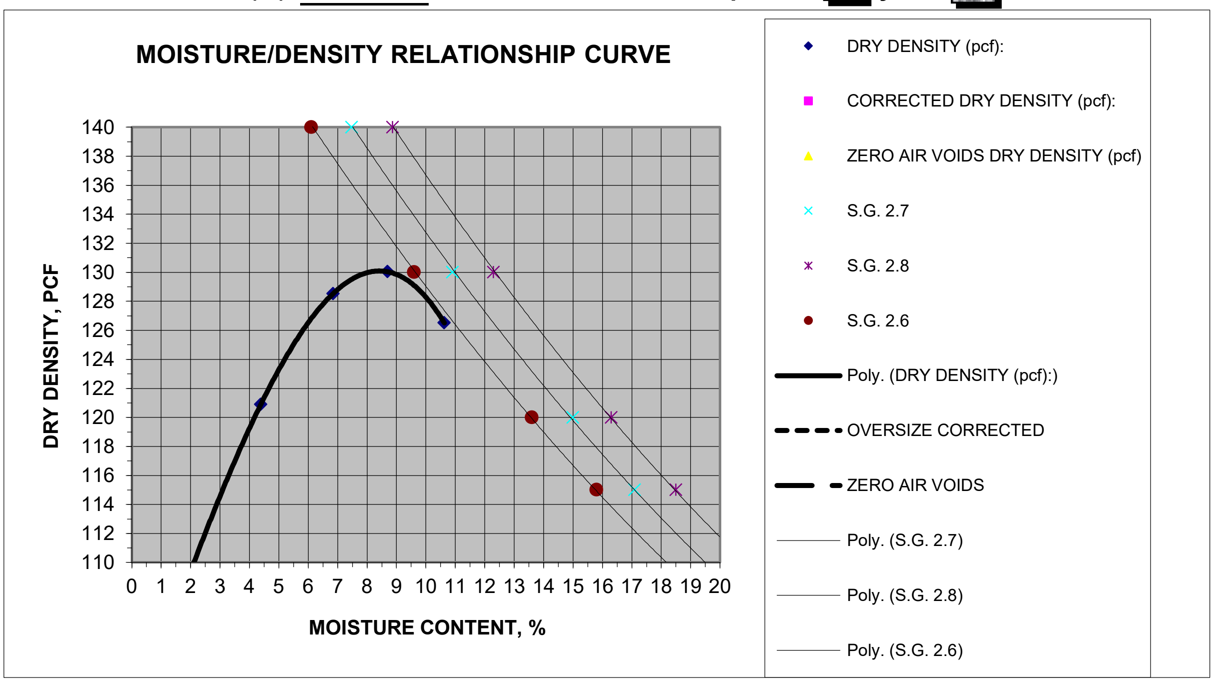
MOISTURE/DENSITY RELATIONSHIP

Client: Carlsbad Village II, LLC
Job No.: 3780-SD
Project: Hope Apartments
Lab No.: Corona
Location: -
Material Type: Light brown silty sand
Material Supplier: -
Material Source: -
Sample Location: B3 @ .-20'
-
Sampled By: CL
Date Sampled: -
Received By: MP
Date Received: -
Tested By: RL
Date Tested: 5/2/2022
Reviewed By: DA
Date Reviewed: 5/5/2022

Test Procedure: ASTM D1557 **Method:** A
Oversized Material (%): 0.1 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	130.0	@ Optimum Moisture, %	8.5
Corrected Maximum Dry Density, pcf		@ Optimum Moisture, %	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Atterberg Limits:

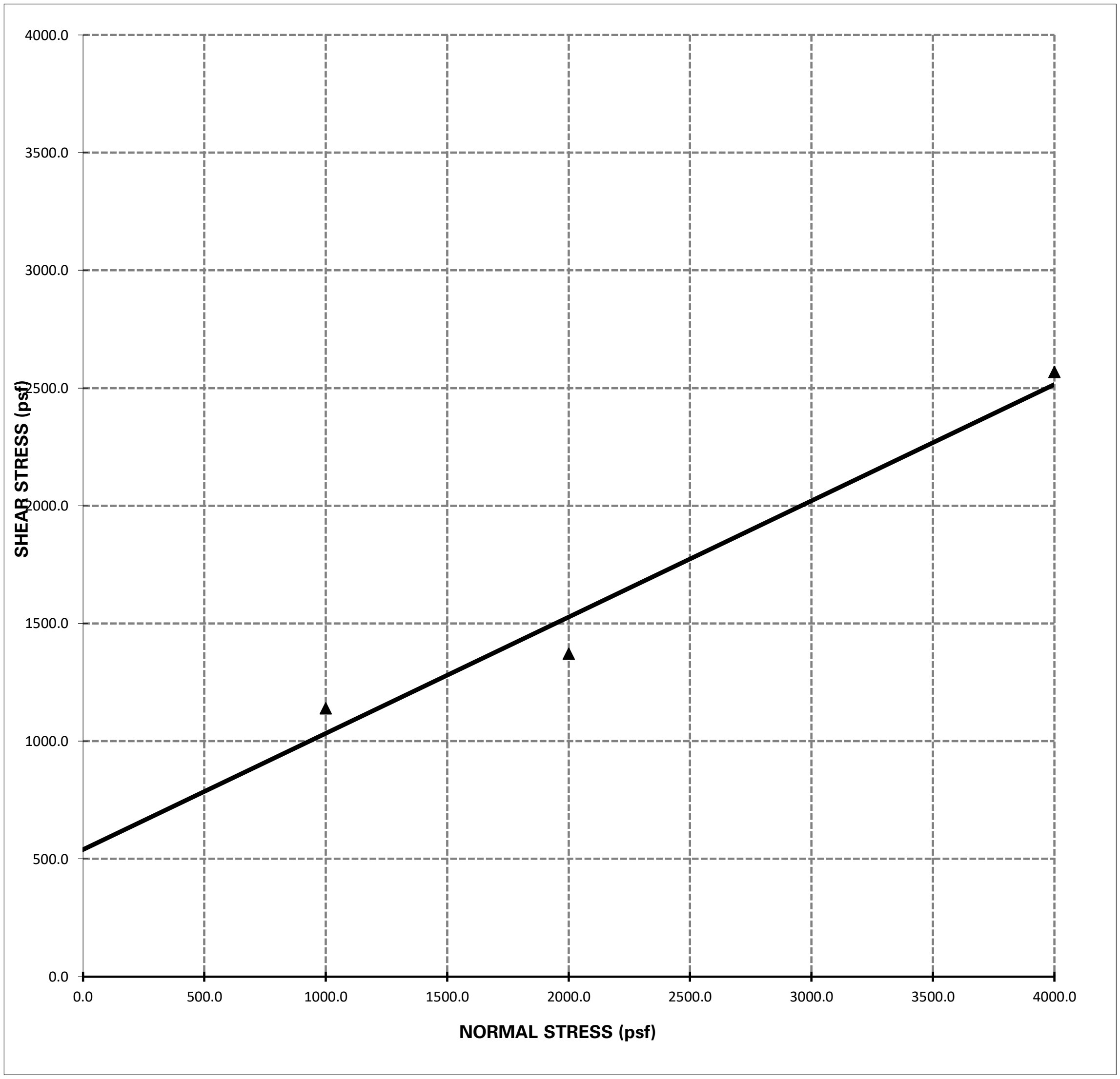
	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification: ____________

DIRECT SHEAR TEST

Project Name:	Hope Apartments	***Sample Location:***	B-3 @ 0-3 feet
Project Number:	3780-SD	***Date Tested:***	5/3/2022



Shear Strength: **Φ = 26 °** **C = 540 psf**

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

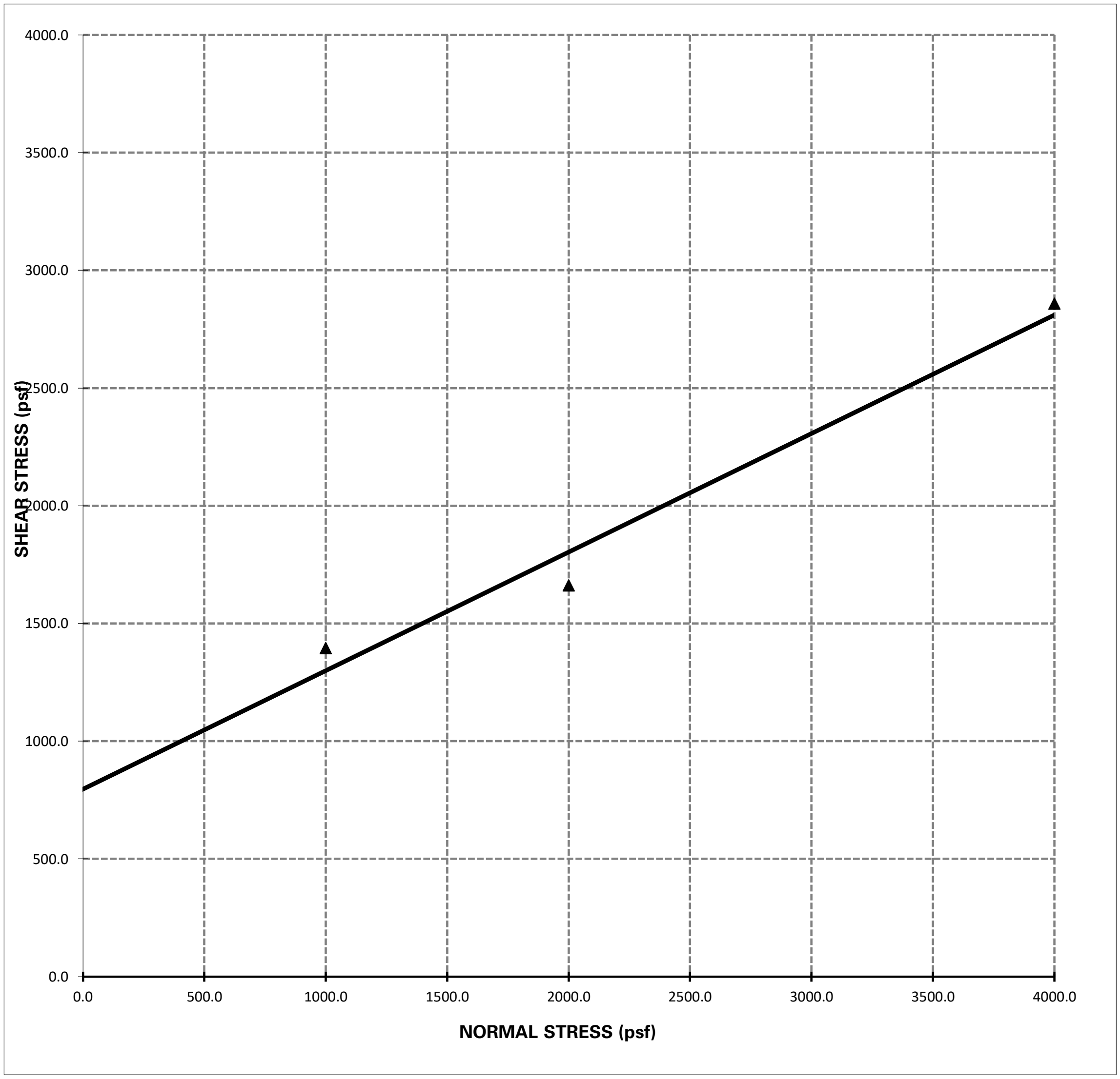
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.35 in/min.

DIRECT SHEAR TEST

Project Name: Hope Apartments
Project Number: 3780-SD
Sample Location: B-3 @ 0-3 feet
Date Tested: 5/3/2022

PEAK VALUE



Shear Strength: Φ = 27 ° C = 797 psf

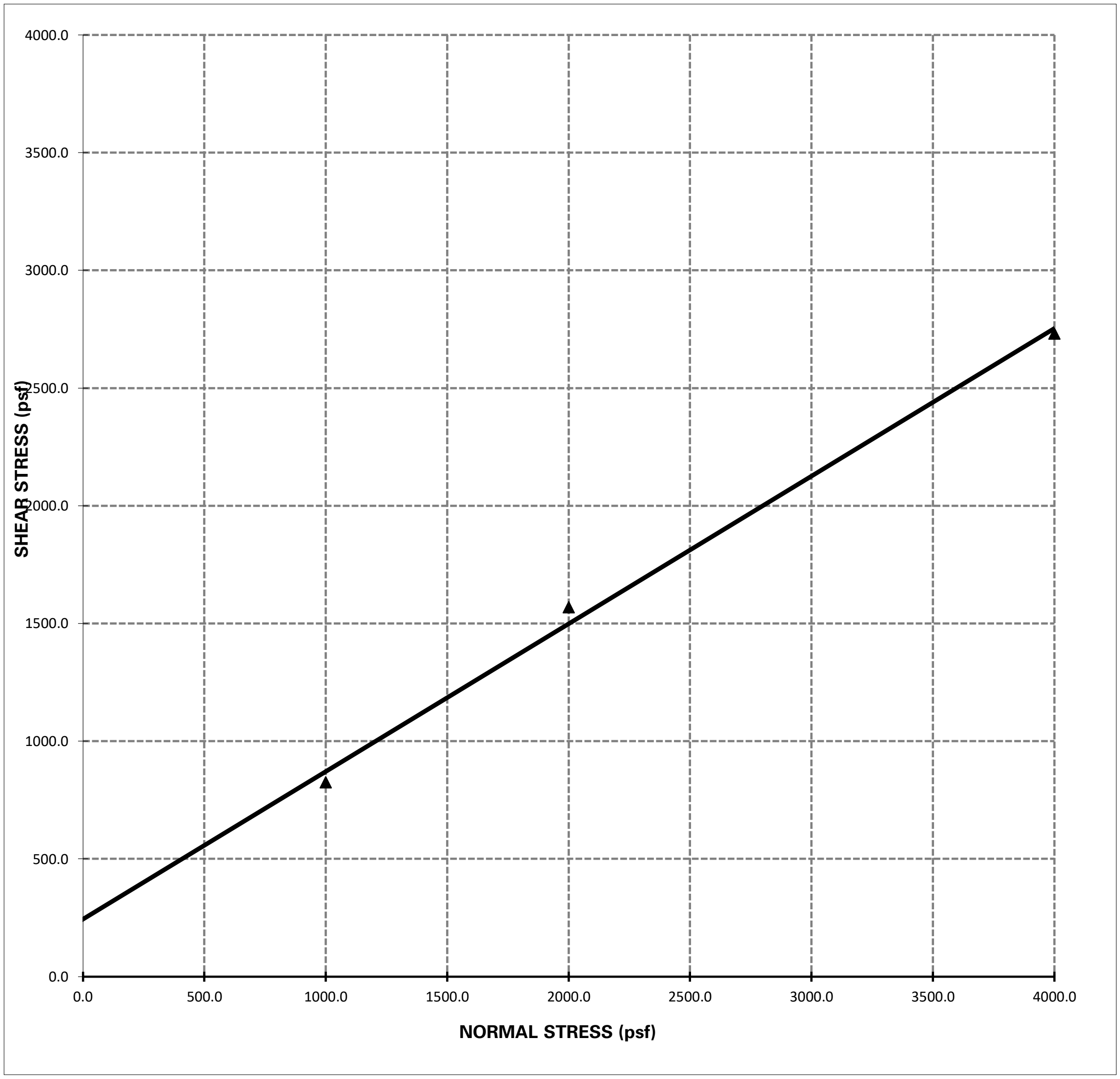
Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

Project Name:	Hope Apartments	***Sample Location:***	B-1 @ 25 feet
Project Number:	3780-SD	***Date Tested:***	4/22/2022



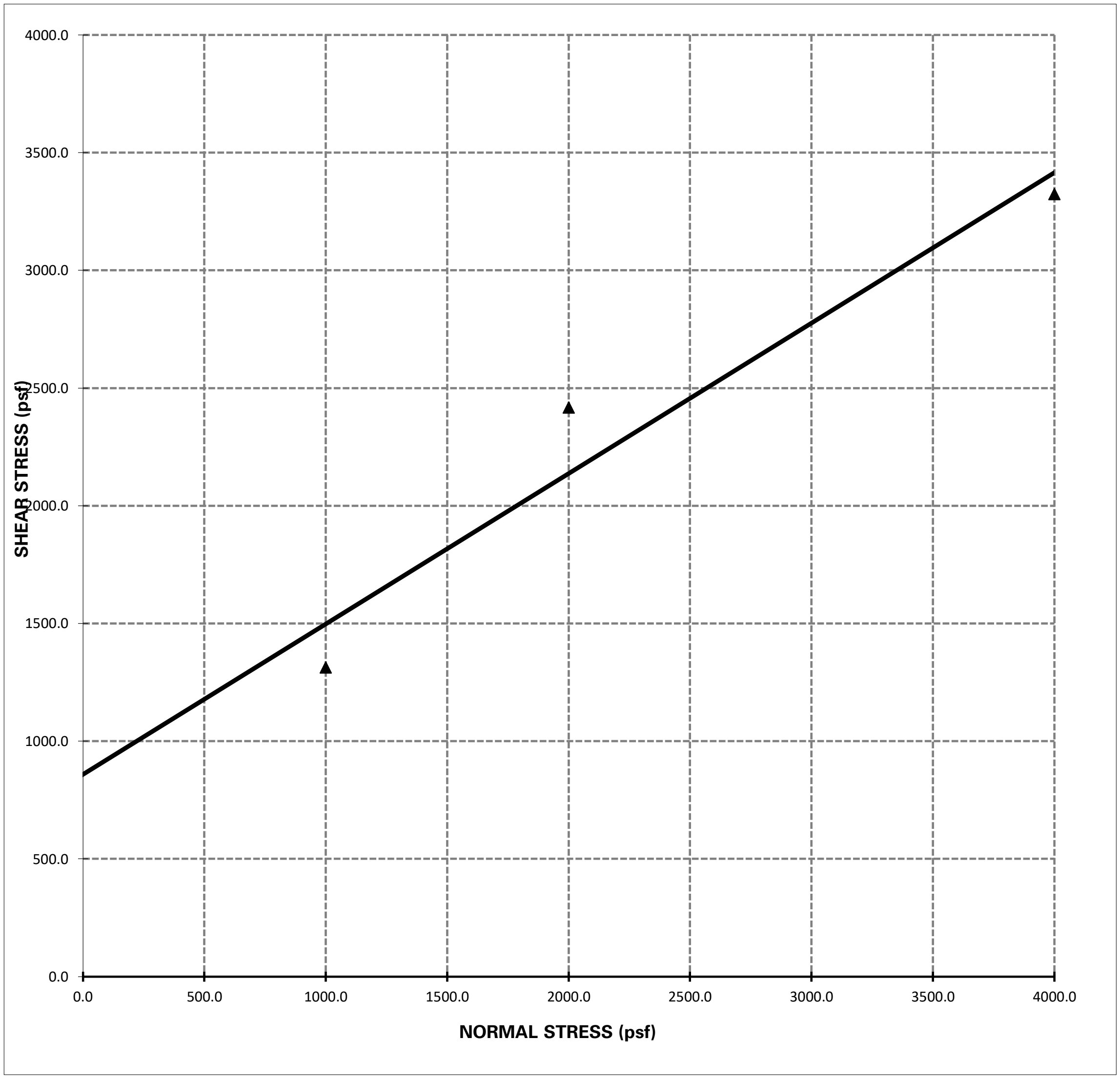
Shear Strength: Φ = 32.1 ° C = 244.00 psf

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

Project Name:	Hope Apartments	***Sample Location:***	B-1 @ 25 feet
Project Number:	3780-SD	***Date Tested:***	4/22/2022

PEAK VALUE



Shear Strength: Φ = **32.6 °** **C = 859.50 psf**

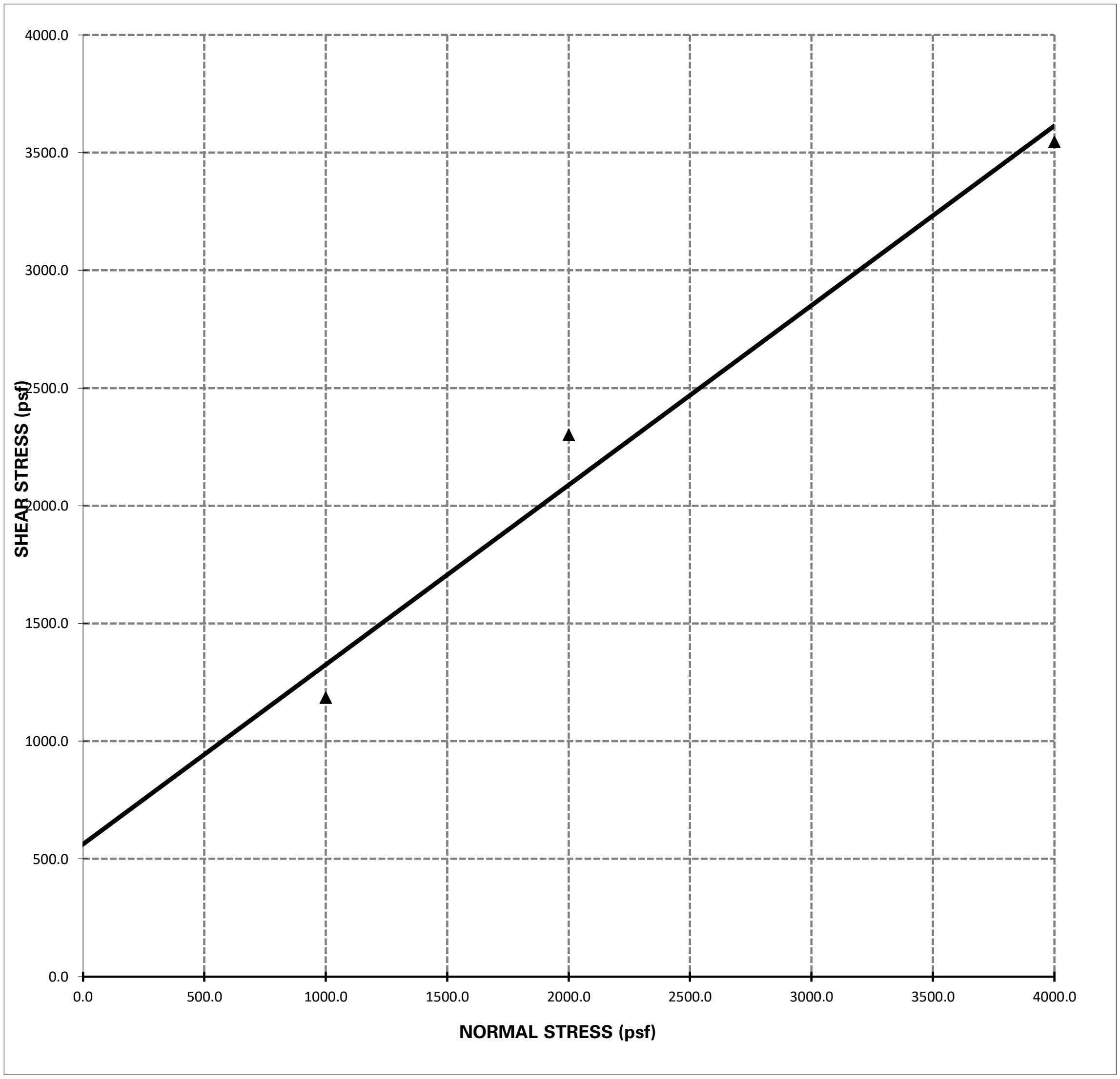
Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

Project Name:	Hope Apartments	***Sample Location:***	B-3 @ 15 feet
Project Number:	3780-SD	***Date Tested:***	4/22/2022



Shear Strength: Φ = **37.4 °** **C = 562.50 psf**

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.

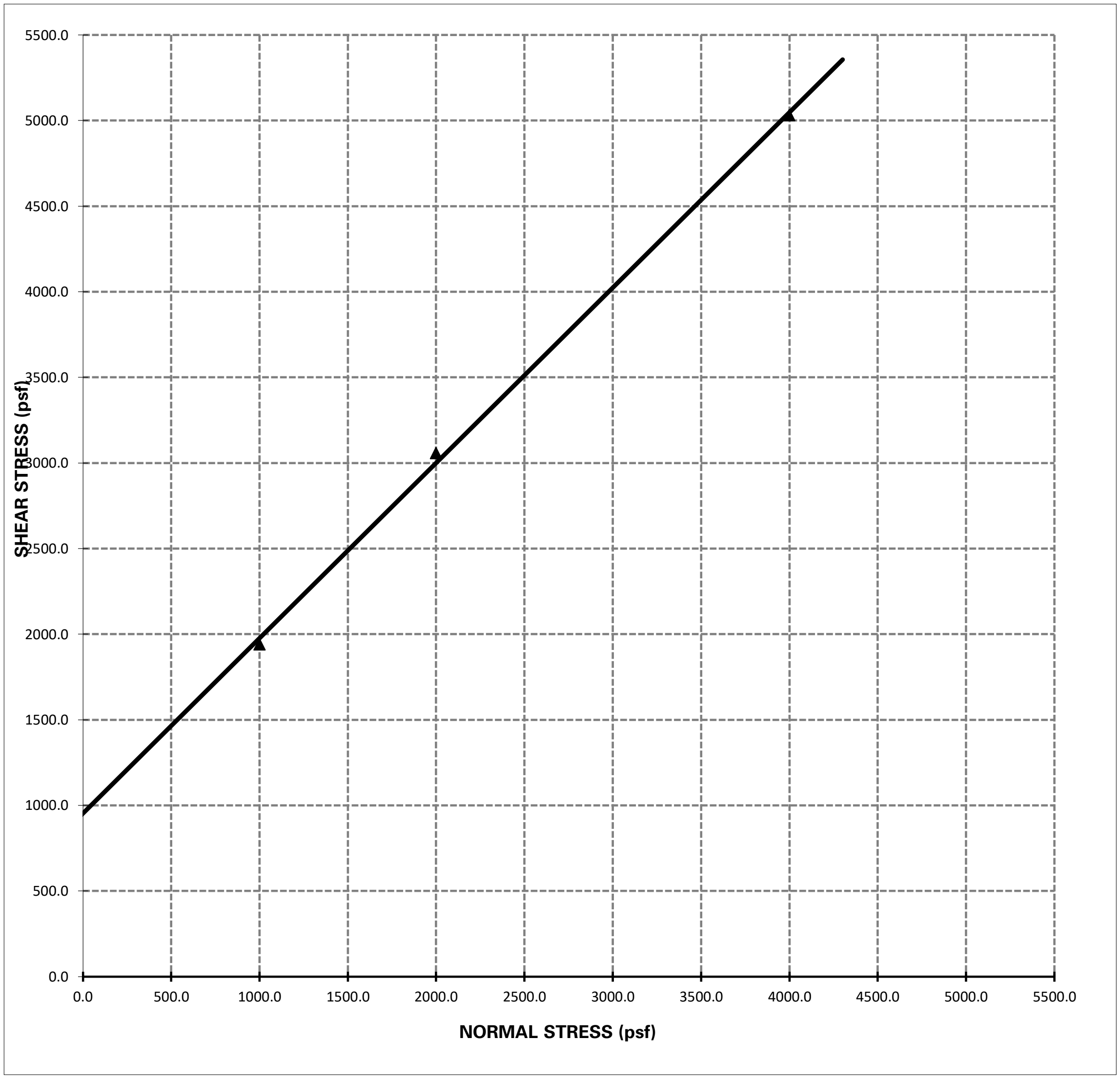
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name: Hope Apartments
Project Number: 3780-SD
Sample Location: B-3 @ 15 feet
Date Tested: 4/22/2022

PEAK VALUE

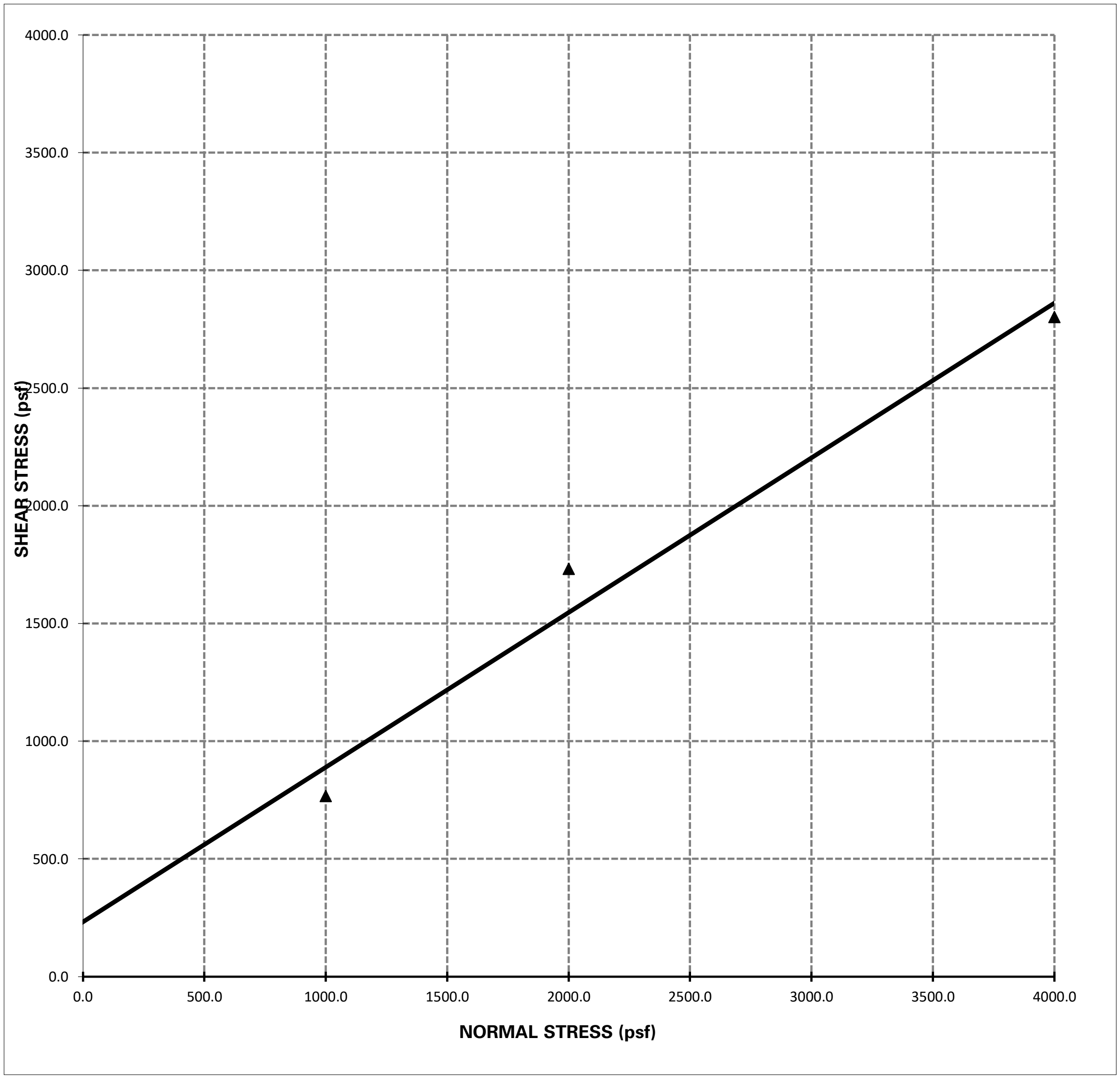


Shear Strength: $\Phi =$ **45.7 °** **C = 953.50 psf**

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name:	Hope Apartments	***Sample Location:***	B-4 @ 10 feet
Project Number:	3780-SD	***Date Tested:***	4/22/2022



Shear Strength: $\Phi =$ **33.3 °** **C = 232.50 psf**

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.

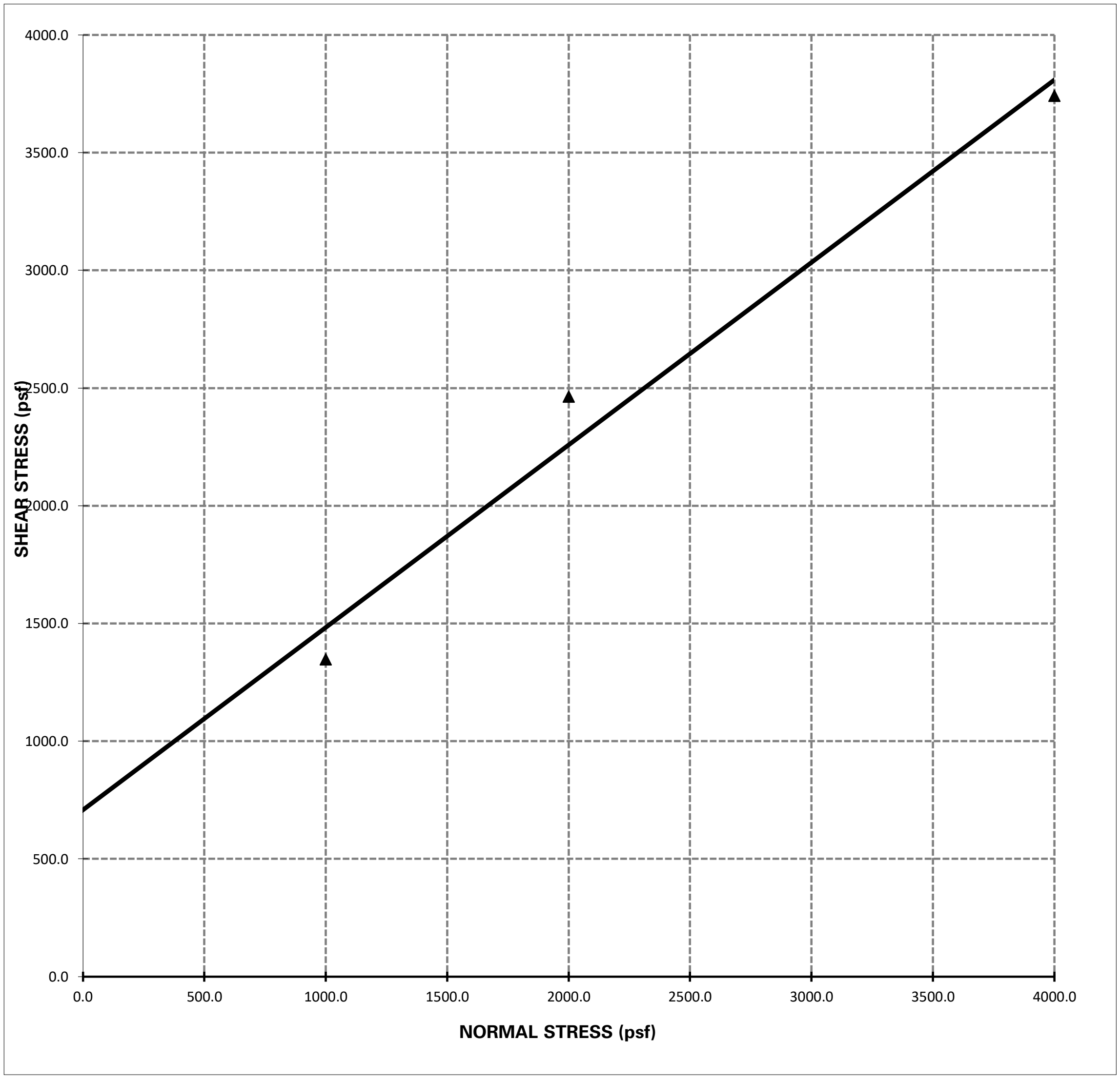
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

Project Name:	Hope Apartments	***Sample Location:***	B-4 @ 10 feet
Project Number:	3780-SD	***Date Tested:***	4/22/2022

PEAK VALUE



Shear Strength: Φ = **37.8 °** **C = 709.00 psf**

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

• ANALYSIS
• DESIGN

LaBelle • Marvin

PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

• SOILS, ASPHALT TECHNOLOGY

April 21, 2022

Mr. Chris Livesey
GeoTek Inc.
1384 Poinsettia Avenue Suite A
Vista, CA 92081-8505

Project No. 48199

Dear Mr. Livesey:

Laboratory testing of the bulk soil sample delivered to our laboratory on 4/19/2022 has been completed.

Reference:	W.O. # 3780-SD
Project:	1006 Carlsbad Village Drive
Sample:	B-6, BB-1 @ 0'-~~2'~~ 0'-20'

Data sheets and graphical presentations are transmitted herewith for your use and information. Any untested portion of the samples will be retained for a period of sixty (60) days prior to disposal. The opportunity to be of service is appreciated, and should you have any questions, kindly call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:tw
Enclosures

2700 S. GRAND AVENUE • SANTA ANA, CA 92705-5404 • (714) 546-3468 • FAX (714) 546-5841
INFO@LABELLEMARVIN.COM

LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 48199

DATE: 4/21/2022

BORING NO. B-6, BB-1 @ 0'-2' 0'-20'

1006 Carlsbad Village Drive

W.O.# 3780-SD

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	7	8	9
Water added, grams	71	50	35
Initial Test Water, %	12.0	9.9	8.5
Compact Gage Pressure,psi	45	130	340
Exudation Pressure, psi	190	431	744
Height Sample, Inches	2.55	2.49	2.48
Gross Weight Mold, grams	3113	3084	2895
Tare Weight Mold, grams	1950	1946	1770
Sample Wet Weight, grams	1163	1138	1125
Expansion, Inches x 10exp-4	0	15	28
Stability 2,000 lbs (160psi)	43 / 103	25 / 53	16 / 30
Turns Displacement	4.13	3.90	3.73
R-Value Uncorrected	25	56	74
R-Value Corrected	25	56	74
Dry Density, pcf	123.4	126.0	126.7

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.77	0.45	0.27
G. E. by Expansion	0.00	0.50	0.93

Equilibrium R-Value	**42** by EXUDATION	Examined & Checked: 4 /21/ 22
REMARKS:	Gf = 1.25 0.1% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 48199

DATE: 4 /21/ 2022

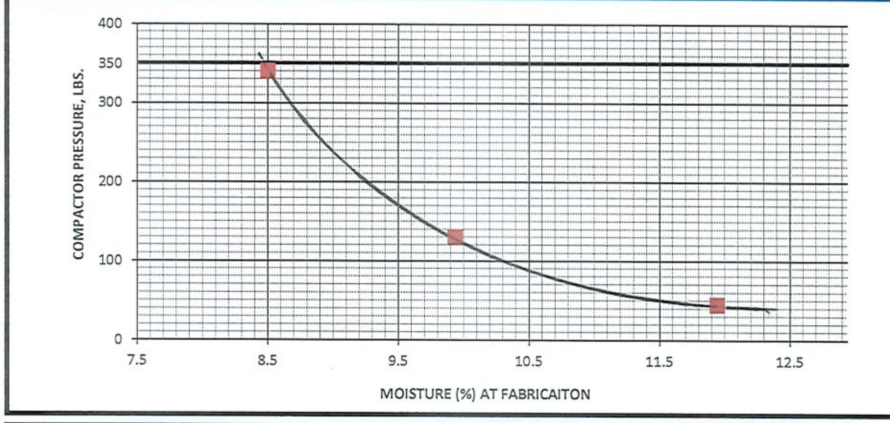
REMARKS:

BORING NO. B-6, BB-1 @ 0'-2' 0'-20'

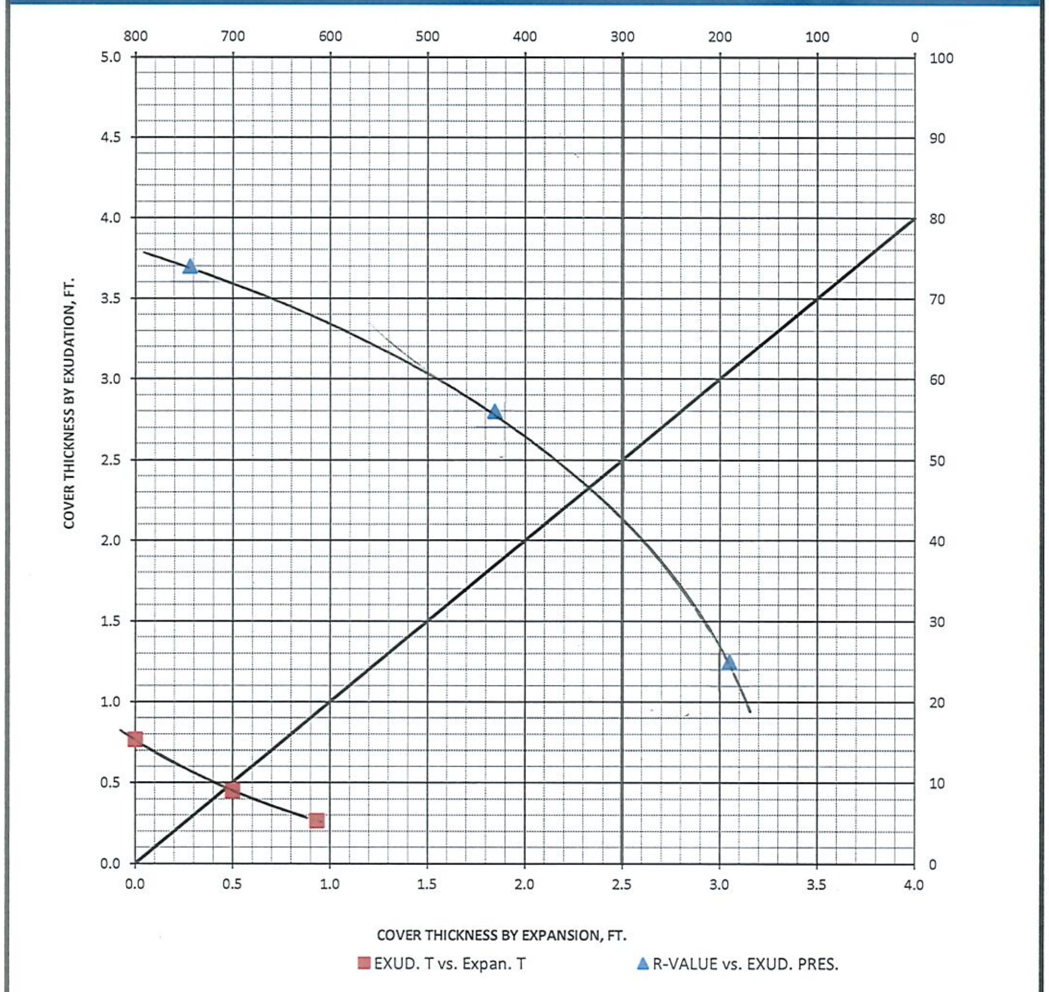
1006 Carlsbad Village Drive

W.O.# 3780-SD

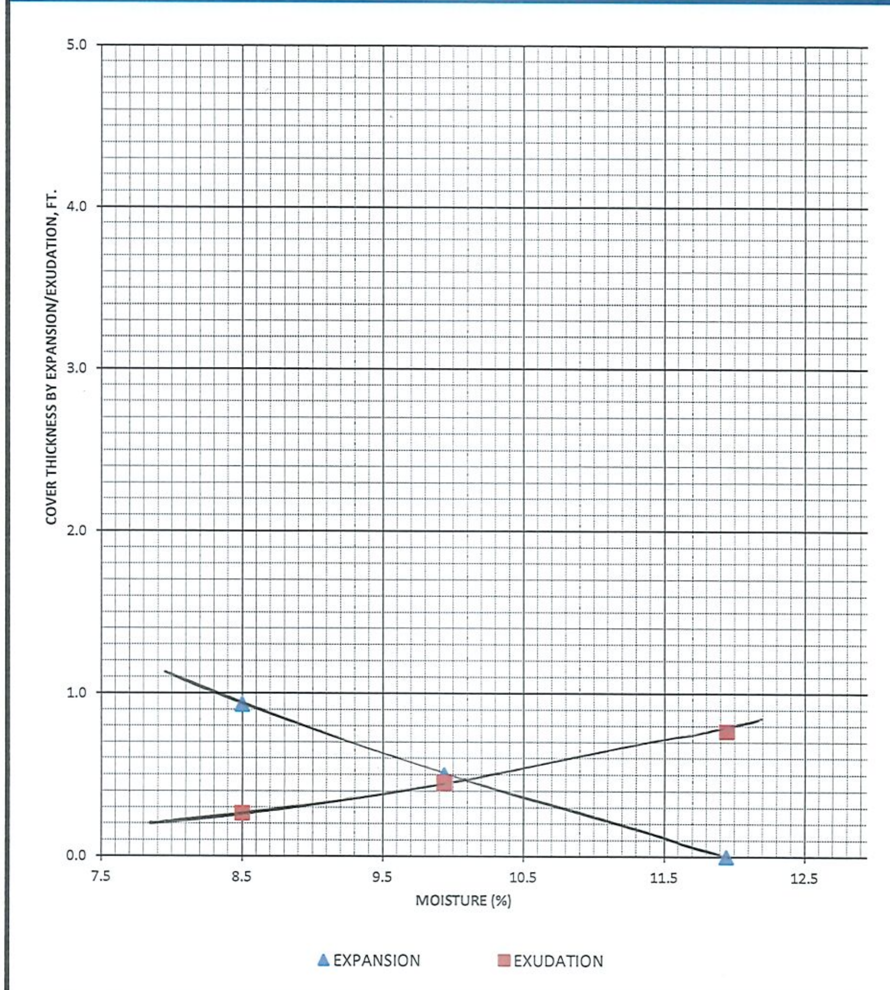
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



Results Only Soil Testing
for
Hope Apartments

April 18, 2022

Prepared for:

Chris Livesey
GeoTek, Inc.
1384 Poinsettia Ave, Suite A
Vista, CA, 92081
clivesey@geotekusa.com

Project X Job#: S220414J
Client Job or PO#: 3780-SD

Respectfully Submitted,

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

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

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: Hope Apartments
Client Job Number: 3780-SD
Project X Job Number: S220414J
April 18, 2022

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-3 BB-1	0-3	9.7	0.0010	3.4	0.0003	308,200	8,710	9.0	105	0.12	0.2	8.9	ND	93.1	4.2	21.9	8.3	0.7	0.2
B-5 BB-1	0-3	16.2	0.0016	8.0	0.0008	10,720	4,556	9.3	116	0.12	0.0	0.7	ND	73.4	4.5	21.1	10.0	2.1	2.3

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

Project X
Corrosion Engineering
Corrosion Control - Soil, Water, and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number	S220414 J GEOTEK 3780-SD Hope 2 Full

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	GeoTek, Inc.	Contact Name:	Chris Livesey	Phone No:	949-338-9233
Mailing Address:	1384 Poinsetta Ave, Ste A, Vista, CA 92081	Contact Email:	clivesey@geotekusa.com		
Accounting Contact:	Accounts Payable	Invoice Email:	ap@geotekusa.com; lwhite@geotekusa.com		
Client Project No:	3780-SD	Project Name:	Hope Apartments		
P.O. #:	Vista				

(Business Days) Turn Around Time:	3-5 Day Standard	[illegible]	[illegible]
	✓		

Results By: ☐ Phone ☐ Fax ☑ Email

Date & Received by :

Special Instructions:

ANALYSIS REQUESTED (Please circle)

Analysis	Group	Caltrans	AASHTO / SM	Default Method
Soil Resistivity	Full Corrosion Series – Geo Quad	Caltrans CTM643	AASHTO T288	ASTM G187
pH	Full Corrosion Series – Geo Quad	Caltrans CTM643	AASHTO T289	ASTM G51
Sulfate	Full Corrosion Series – Geo Quad	Caltrans CTM417	AASHTO T290	ASTM D4327
Chloride	Full Corrosion Series – Geo Quad	Caltrans CTM422	AASHTO T291	ASTM D4327
Redox Potential	Full Corrosion Series		SM 2580B	ASTM G200
Sulfide	Full Corrosion Series			SM 4500-S2
Ammonia	Full Corrosion Series		SM 4500-NH3	ASTM D6919
Nitrate	Full Corrosion Series		SM 4500-NO3	ASTM D4327
Flouride	Full Corrosion Series			ASTM D4327
Phosphate	Full Corrosion Series			ASTM D4327
Lithium	Full Corrosion Series			ASTM D6919
Sodium	Full Corrosion Series			ASTM D6919
Potassium	Full Corrosion Series			ASTM D6919
Magnesium	Full Corrosion Series			ASTM D6919
Calcium	Full Corrosion Series			ASTM D6919
BiCarbonate	Full Corrosion Series			SM 2320B
Full Corrosion Series				
Soil Corrosivity Evaluation Report	Reports (*Req: Min 3 Samples, site map, and groundwater info)			
Water Corrosivity Mini Report	Reports (*Req: Min 3 Samples, site map, and groundwater info)			
Moisture Content				ASTM D2216
Total Alkalinity				SM 2320B
Thermal Resistivity				
Metallurgical Analysis				
Langelier Index				
Puckorius Index				
XRF Elemental Analysis				
Water Hardness				

	#	SAMPLE ID - BORE #	DESCRIPTION	DEPTH (ft)	DATE COLLECTED	Full Corrosion Series	Other analyses
190	1	B-3 BB-1		0-3'		✓	
191	2	B-5 BB-1		0-3'		✓	
	3						
	4						
	5						
	6						
	7						
	8						
	9						
	10						
	11						
	12						
	13						
	14						
	15						
	16						

APPENDIX C

GENERAL EARTHWORK GRADING GUIDELINES

GENERAL GRADING GUIDELINES

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

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the California Building Code, CBC (2019) and the guidelines presented below.

Preconstruction Meeting

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

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.
6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will

be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.

7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

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

Treatment of Existing Ground

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

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to

obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.

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

Slope Construction

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

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

JOB SAFETY

General

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

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

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

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

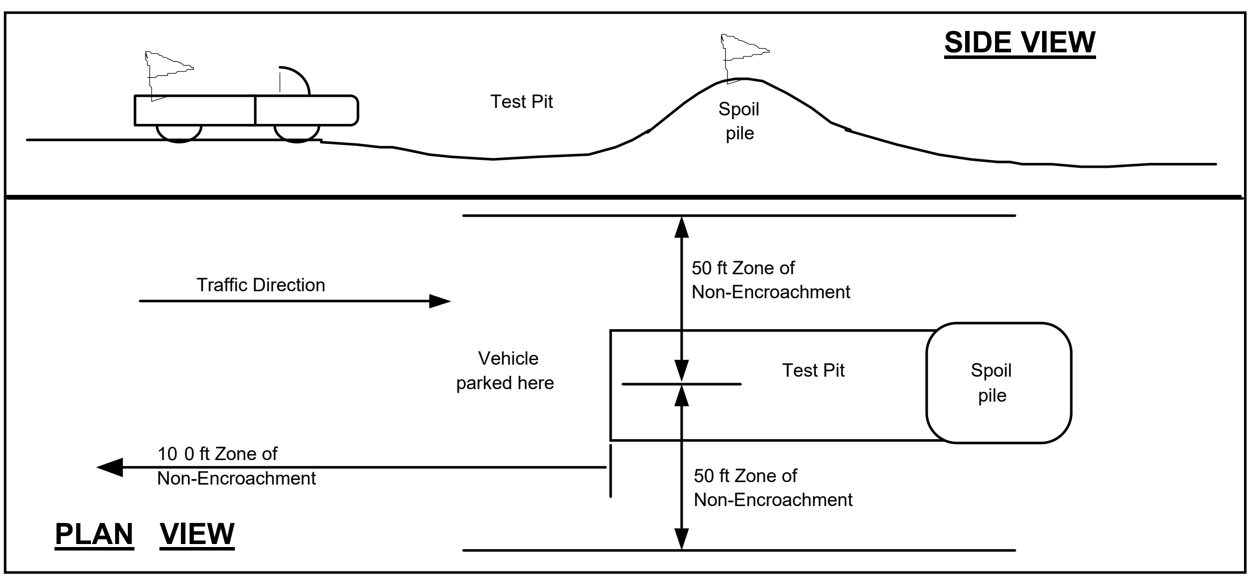
Test Pits Location, Orientation and Clearance

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

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

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

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

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

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