TORRANCE APARTMENTS
22501 Hawthorne Blvd
City of Torrance, CA

PRELIMINARY SUSMP REPORT
December 16, 2021

Prepared For:

Intracorp Homes
895 Dove, Suite 400
Newport Beach, CA 92660

Prepared By:

URBAN RESOURCE
CONSULTING CIVIL ENGINEERS

Urban Resource Corporation
2923 Saturn Street, Unit H
Brea, CA 92821

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Terry P. Au, P.E.
State of California No. 68466

Preparer (Engineer):			
Title	TERRY AU, PRINCIPAL	PE Registration #	C68466
Company	URBAN RESOURCE CORPORATION		
Address	2923 SATURN STREET, UNIT H, BREA, CA		
Email	TERRY@URBRESOURCE.COM		
Telephone #	949-727-9095		
I hereby certify that this Low Impact Development Plan/SUSMP is in compliance with, and meets the requirements set forth in, Order No. R4-2012-0175, of the Los Angeles Regional Water Quality Control Board.			
Preparer Signature		Date	
Place Stamp Here			

OWNER CERTIFICATION
Preliminary Standard Urban Stormwater Mitigation Plan
for
22501 Hawthorne Blvd – Torrance Apartments
Grading Plan Permit No. ___________

This Preliminary Standard Urban Stormwater Mitigation Plan (SUSMP) for 22501 Hawthorne Blvd Torrance Apartments Project has been prepared for Intracorp Homes by Urban Resource Corporation. This SUSMP is intended to comply with the requirements of the City of Torrance, the County of Los Angeles and the Los Angeles Regional Water Quality Control Board, requiring the preparation of a project specific SUSMP.

I certify under penalty of law that this document and all attachments were prepared under my jurisdiction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system or those persons directly responsible for the gathered information, to the best of my knowledge and belief, the information submitted is true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

The undersigned, while it owns the subject property, is responsible for the implementation of the provisions of this plan and will ensure that this plan is amended as appropriate to reflect up-to-date conditions consistent with the City of Torrance's LID Ordinance, the City of Torrance's Green Streets Policy, and the intent of the stormwater and urban runoff NPDES Permit and Waste Discharge Requirements for the County of Los Angeles, Los Angeles County Flood Control District and the incorporated Cities of Los Angeles County under the jurisdiction of the Los Angeles Regional Water Quality Control Board. A copy of this SUSMP will be maintained at the project site/office.

This SUSMP will be reviewed with the facility operator, facility supervisors, employees, tenants, maintenance and service contractors, or any other party having responsibility for implementing portions of this SUSMP. At least one copy of the approved and certified copy of this SUSMP shall be available on the subject property in perpetuity. Once the undersigned transfers its interest in the property, its successors-in-interest shall bear the aforementioned responsibility to implement and amend the SUSMP.

______________________________ Owner Signature	______________________________ Company
______________________________ Printed Name/Title	______________________________ Company Address
______________________________ Telephone No.	______________________________ Date

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INTRODUCTION

The following site is located at 22501 Hawthorne Blvd in the City of Torrance and is in the County of Los Angeles. The site is bounded by Hawthorne Blvd to the east, W. 226th Street to the south, existing residential to the west and existing retail/commercial/office to the north.

The existing site consists of three retail/commercial buildings, parking lot and landscape improvements with original improvements dating back to 1959. The site imperviousness is estimated to be 99%.

This project is approximately 3.8 acres consisting of one Parcel, Lot 11 of Tract 454. The proposed development is for multi-family residential and commercial retail, including a parking garage, three courtyards, lobby/leasing center, and open plaza. The approximate project imperviousness is 85%.

The existing site topography is relatively flat and generally drains from the northeast to the southwest with site elevations ranging from 82el. to 77el. across the site. Post development conditions will maintain pre-development drainage patterns and development flows will be conveyed via surface flows and pipe flows and outlet into the street gutter via parkway drains (or similar), and into Hawthorne Blvd and/or W. 226th Street. Project peak storm flows are ultimately captured by an existing catch basin in Ocean Avenue, south of W. 226th Street, where existing site drainage currently drains to. Water quality flows will be routed via storm drains lines to the proposed onsite infiltration and storage system.

A small drainage area of approximately 0.2 acres from the existing apartment site along the westerly project boundary drains onto this existing site, and its drainage area flows will be maintained and will flow through wall openings in the proposed boundary wall at the southwest corner of the project. These flows will continue to drain into W. 226th Street and is not treated onsite. Its' drainage subarea is depicted in the LID Plan in Appendix E.

The water quality condition requiring preparation of this SUSMP report is "Redevelopment projects, which are developments that result in creation or addition or replacement of 5,000 square feet or more of impervious surface on a site that was previously developed." This project is a Designated Project per the County of Los Angeles Department of Public Works Low Impact Development Standards Manual, dated February 2014.

APPROACH

The objective of this LID report is to address the performance criteria required and demonstrate compliance with the 2012 MS4 Permit within the Coastal Watersheds of Los Angeles County (CAS004001, Order No. 2012-0175), utilizing the Los Angeles County Department of Public Works' Low Impact Development Standards Manual, dated February 2014. This objective will be satisfied with the proposed following:

1. Implement LID BMPs in accordance with the County of Los Angeles Low Impact Development Standards Manual, to improve water quality and mitigate potential water

quality impacts caused by the development, prior to discharging developed condition storm flows offsite. This development is considered a Designated Project.

To address water quality requirements, the water quality design volume (SWQDv) is calculated utilizing either the 0.75-inch, 24 hour rain event, or the 85th percentile, 24 hour rain event, with the larger SWQDv value governing. Feasibility of BMPs would be analyzed in the following order of priority: 1. Infiltration Systems, 2. Stormwater Capture and Reuse, 3. High Efficiency Biofiltration/Bioretention Systems, 4. Combination of the previous three, if applicable.

Refer to the LID calculations provided in Appendix A and the LID plan in Appendix E for additional details.

STORMWATER QUALITY DESIGN

1. Infiltration: Infiltration/retention for the site is deemed feasible based on the Percolation Test Results report prepared by Geocon West, Inc., dated May 12, 2021. The design infiltrate rate used for design of the drywell system is 1.62in/hr per the report. Per the soils report, groundwater was encountered in boring B2 at a depth of 74 feet below the existing ground surface. The report is included herewith in Appendix B.

Based on the proposed redevelopment (Multi-family Residential/Commercial), the project's typical Pollutants of Concern, per Table 7-3 of the LID Manual, are the following:

- Suspended Solids
- Total Phosphorus
- Total Nitrogen
- Total Kjeldahl Nitrogen
- Cadmium, Total
- Chromium, Total
- Copper, Total
- Lead, Total
- Zinc, Total

Per the County of Los Angeles Public Works Storm Drain database website, the downstream receiving water is Wilmington Drain then Machado Lake (Harbor Park Lake).

The water quality impairments as included in the 303(d) List, applicable TMDLs are listed in the below. The project is not located in or directly adjacent to or discharging directly to a Significant Ecological Area (SEA).

<u>303(d) List Impairment</u>
Wilmington Drain: Ammonia, Coliform Bacteria, Copper, Lead
Machado Lake: Algae, Ammonia, ChemA (tissue), Chlordane, (tissue), DDT (tissue), Diedldrin (tissue), Eutrophic, Odor, PCBs (Polychlorinated biphenyls) (tissue), Trash

Applicable TMDLs
Wilmington Drain: Coliform Bacteria, Copper, Lead
Machado Lake: Algae, Ammonia, ChemA (tissue), Chlordane, (tissue), DDT (tissue), Diedldrin (tissue), Eutrophic, Odor, PCBs (Polychlorinated biphenyls) (tissue), Trash

Treatment in the form of the Torrent Maxwell Plus drywell will be proposed to address stormwater runoff quality from the project improvements. An underground storage system (CMP storage or similar) will be proposed to provide storage for the required water quality treatment volume. The Torrent Maxwell Plus drywell system provides pretreatment for the required project pollutants of concern. The drywell system will be offline, with large storm event flows bypassing the water quality treatment system, and outletting via parkway drains (or similar) into street gutters in Hawthorne Blvd and/or W. 226th Street.

Sizing of the Torrent drywell is based on the mitigation volume for the 85th percentile 24 hour event. The calculation utilizing the Los Angeles County software HydroCalc is included in Appendix A. The Torrent calculations for design and sizing of the drywells are included in Appendix A.

SOURCE CONTROL MEASURES

Source control measures for the project are listed below and BMP fact sheets are included in Appendix D.

- Storm Drain Message and Signage (S-1)
- Landscape Irrigation Practices (S-8)
- Building Materials Selection (S-9)

HYDROMODIFICATION

The project is a redevelopment of a previously developed site in an urbanized area that does not increase the effective impervious area or decrease the infiltration capacity of pervious areas compared to the pre-project conditions. The project will decrease site imperviousness from an estimated imperviousness of 99% to approximately 85% in the developed condition. Furthermore, the project's storm flows will outlet into the street gutters in Hawthorne Blvd and/or W. 226th Street before being captured by an engineered storm drain system. Flows are routed in an engineered storm drain system until it ultimately drains into the Wilmington Drain, which outlets into the Machado Lake.

Therefore, implementation of hydromodification controls are not required for this project.

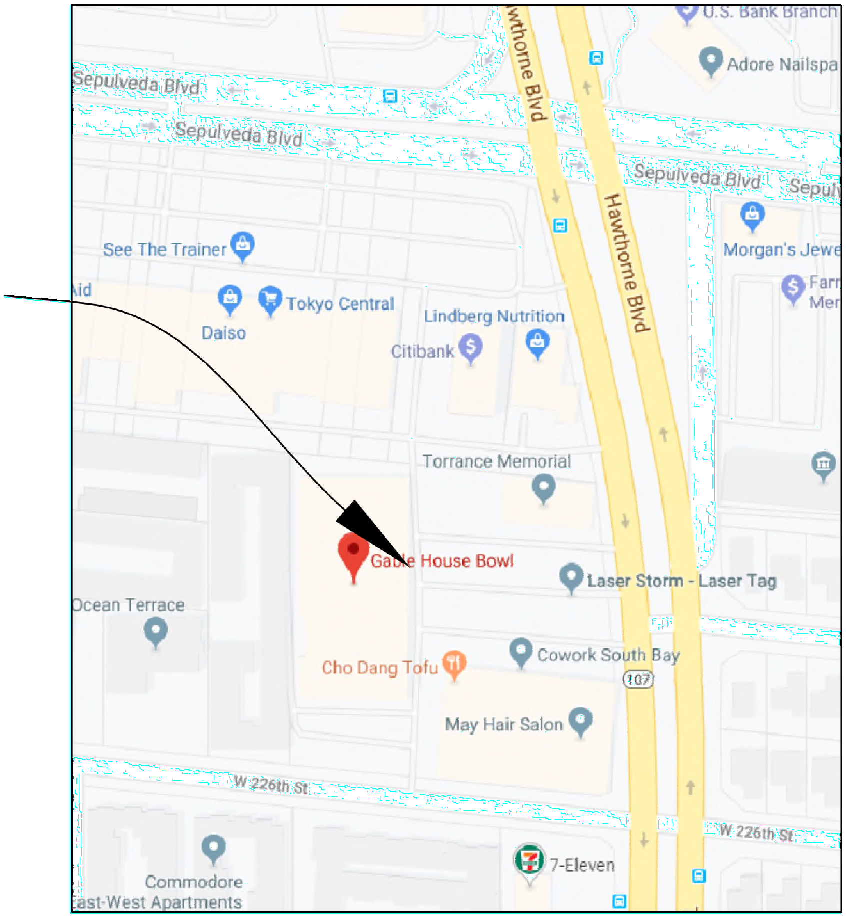
RECOMMENDATIONS

Water quality treatment to meet LID requirements for this project will be provided by a Torrent Maxwell Plus drywell system and an underground storage system The drywell will be sized to meet or exceed the 85th percentile 24 hour storm event, which produces the greater treatment volume per calculations provided in Appendix A. The 85th percentile 24 hour storm event

Stormwater Quality Design Volume ($SWQD_v$) is 8,873 cu-ft. The proposed drywell system will mitigate a volume of 3,139 cu-ft. The remaining water quality volume will be stored in an underground storage system which will be connected to the drywell system for treatment/infiltration. The proposed Torrent Maxwell Plus drywell has an estimated total depth of 50' with a 6' diameter shaft that will contain washed rock between a depth of 18' and 50'. The Maxwell Plus system includes a primary settling chamber for pretreatment. Refer to the calculations included in Appendix A for sizing of the Torrent drywell and additional details. Supporting documents and a LID plan are included in Appendix B and Appendix E, respectively. The proposed drywell detail are also included in Appendix E.

Intracorp Housing will be responsible for the long-term inspection and maintenance of all proposed BMPs. Refer to Appendix C for Operations and Maintenance information.

PROJECT LOCATION



VICINITY MAP

II. REFERENCES

1. Low Impact Development Standards Manual, Los Angeles County Department of Public Works, February 2014.
2. HydroCalc Version 1.0.3, Los Angeles County Department of Public Works, Released 2/21/2018.

APPENDICES

APPENDIX A – LID CALCULATIONS

Peak Flow Hydrologic Analysis

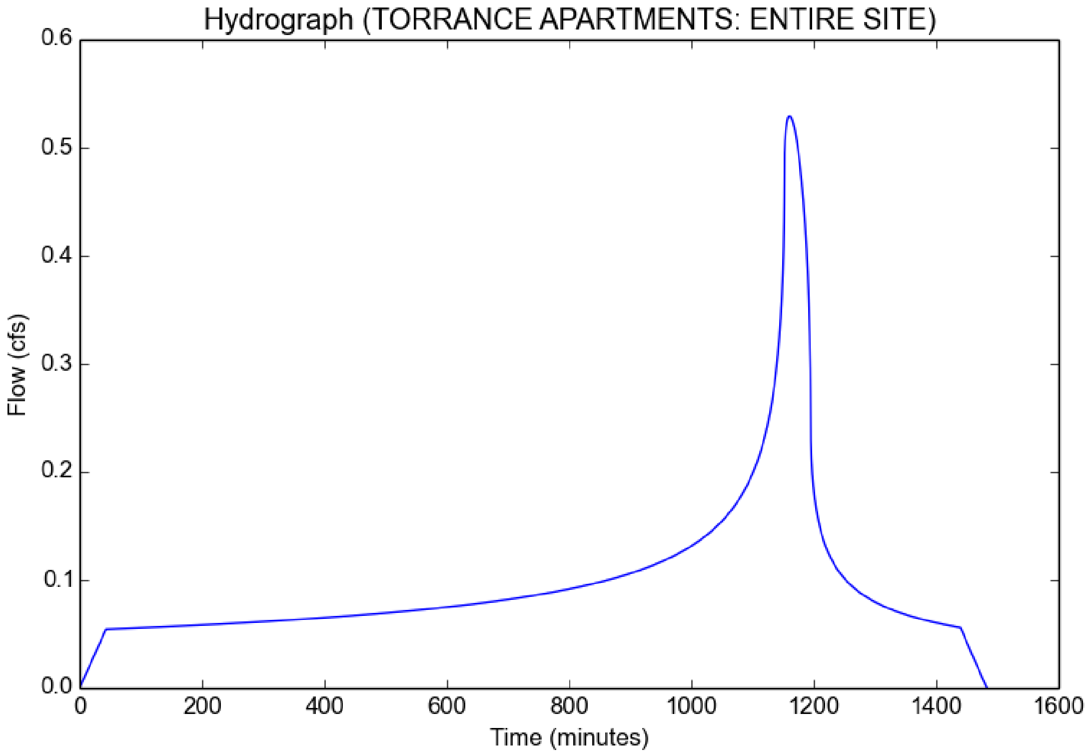
File location: P:/278.737/SUSMP/PRELIMINARY SUSMP REPORT/Hydrocalc_85thPerc24hr_Prelim Calc.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	TORRANCE APARTMENTS
Subarea ID	ENTIRE SITE
Area (ac)	3.81
Flow Path Length (ft)	750.0
Flow Path Slope (vft/hft)	0.005
85th Percentile Rainfall Depth (in)	0.82
Percent Impervious	0.85
Soil Type	10
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	0.82
Peak Intensity (in/hr)	0.178
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.78
Time of Concentration (min)	43.0
Clear Peak Flow Rate (cfs)	0.5288
Burned Peak Flow Rate (cfs)	0.5288
24-Hr Clear Runoff Volume (ac-ft)	0.2014
24-Hr Clear Runoff Volume (cu-ft)	8772.9623



Maxwell® Plus Drainage System Calculations Prepared on December 02, 2021
Project: **Torrance Apartments - Torrance, CA**
Contact: Terry Au at Urban Resource - Brea, CA

Given:

Design Infiltration Rate	1.62	in/hr
Mitigated Volume	8,773	ft^3
Required Drawdown Time	96	hours
Depth to Emergency Overflow	0	ft
Groundwater Depth for Design	74	ft

Proposed:

Drywell Rock Shaft Diameter	6	ft
Primary Chamber Depth	18	ft
Drywell Chamber Depth	18	ft
Rock Porosity	40	%
Depth to Infiltration	14	ft
Drywell Bottom Depth	50	ft

Convert Design Rate from in/hr to ft/sec.

$$1.62 \ \frac{in}{hr} \times \frac{1\ ft}{12\ in} \times \frac{1\ hr}{3600\ sec} = \ 0.000038 \ \frac{ft}{sec}$$

A 6 foot diameter drywell provides 18.85 SF of infiltration area per foot of depth, plus 28.27 SF at the bottom.

For a 50 foot deep drywell, infiltration occurs between 14 feet and 50 feet below grade. This provides 36 feet of infiltration depth in addition to the bottom area. Infiltration area per drywell is calculated below.

$$36\ ft \times \ 18.85 \frac{ft^2}{ft} + \ 28.27\ ft^2 = \ 707 \ \ ft^{\ 2}$$

Combine design rate with infiltration area to get flow (disposal) rate for each drywell.

$$0.000038 \frac{ft}{sec} \times \ 707\ ft^2 = \ 0.02651 \frac{ft^3}{sec}$$

Volume of disposal for each drywell based on various time frames are included below.

96 hrs: 0.0265 CFS x 96 hours x $\frac{3600\ sec}{1\ hr}$ = 9,161 cubic feet of retained water disposed of.

Chamber diameter = 4 feet. Drywell rock shaft diameter = 6 feet.

Volume provided in each primary settling chamber with depth of 18 feet.

$$18\ ft \ \times \ 12.57\ ft^2 \ = \ 226\ ft^3$$

Volume provided in each drywell with chamber depth of 18 feet.

$$18\ ft \ \times \ 12.57\ ft^2 \ + 32\ ft \times \ 28.27\ ft^2 \ \times \ 40\ \% \ = \ 588\ ft^3$$

The MaxWell System is composed of 1 drywell(s) and 1 primary chamber(s).

Total volume provided = 814 ft^3
Total 96 hour infiltration volume = 9,161 ft^3
Total infiltration flowrate = 0.02651 $\frac{ft^3}{sec}$

Based on the total mitigated volume of 8773 CF, after subtracting the volume infiltrated as quickly as it enters the drywell of 2325 CF, the remaining volume is 6448 CF. The storage provided in the drywell system is 814 CF. Therefore 5634 CF can be stored in a separate detention system.

For any questions, please contact Ryan Adaya at 951-202-1037 or via email at RAdaya@TorrentResources.com

Torrent Resources (CA) Incorporated
9950 Alder Avenue
Bloomington, CA 92316
Phone 909-829-0740

HydroCalc Summary

Using the hydrograph produced by the HydroCalc Calculator, the area below the drywell flow disposal rate and the hydrograph curve is estimated as the volume infiltrated in the drywell as it enters. 3 different phases will occur during the 85th percentile storm event. Phase 1 will occur during the beginning of the storm event at the initial increase of flow produced by the storm. When the storm flow is equal to the drywell flow disposal rate, phase 1 ends and phase 2 begins. Phase 2 is when the drywell performs at the flow rate it was design at. Any additional runoff that is produced due to the increase of storm flow will require a detention system. The storm will then hit its peak flow and begin to decrease. When the storm flow decreases to an amount equal to the drywell flow disposal rate, phase 2 ends and phase 3 begins. Phase 3 will occur near the end the storm when the drywell infiltrates the residual runoff until the end of the event.

Phase 1 – Initial Filling of Drywell

From time 0 minutes to 21 minutes, the 85th storm event flowrate that enters the drywell is less than the drywell steady-state infiltration flowrate (flow disposal rate). Therefore, the entire volume entering the drywell from 0 minutes to 21 minutes will infiltrate without overwhelming the drywell. This volume is 16.7 CF.

Time (min)	Incremental Masscurve	Incremental Design Storm Depth (in)	Intensity (in/hr)	Undeveloped Runoff Coefficient (Cu)	Developed Runoff Coefficient (Cd)	Clear Peak Flow Rate (cfs)	Incremental Volume (cu-ft)	Cumulative Volume (cu-ft)	Volume infiltrated by drywell (CF)
20.8	0.007688329	0.00630443	0	0	0	0.0262637	0.31364963	16.3885721	0.31364963
21	0.007762576	0.006365313	0	0	0	0.0265163	0.31668006	16.7052522	0.31812
21.2	0.00783683	0.0064262	0	0	0	0.0267688	0.3197105	17.0249627	0.31812

Phase 2 – Drywell Performing at the Design Rate

From time 21 minutes to 1462 minutes, the flowrate that enters the drywell exceeds the drywell steady-state infiltration flowrate (flow disposal rate). Therefore, the drywell can only infiltrate up to its flow disposal rate which is 0.02651 CFS. Over this period, we multiply the time by the flowrate (and covert as needed) to determine the volume infiltrated in this phase. This volume is 2292.1 CF.

(1462-21) x 60 SEC/MIN x 0.02651 CFS = 2292.1 CF

Time (min)	Incremental Masscurve	Incremental Design Storm Depth (in)	Intensity (in/hr)	Undeveloped Runoff Coefficient (Cu)	Developed Runoff Coefficient (Cd)	Clear Peak Flow Rate (cfs)	Incremental Volume (cu-ft)	Cumulative Volume (cu-ft)	Volume infiltrated by drywell (CF)
1461.8	1	0.82	0.00908811	0.1	0.78	0.027008	0.32565342	8755.88805	0.31812
1462	1	0.82	0.00900081	0.1	0.78	0.0267486	0.32253993	8756.21059	0.31812
1462.2	1	0.82	0.00891355	0.1	0.78	0.0264893	0.31942753	8756.53002	0.31942753

Phase 3 – End of the Storm Event

From time 1462 to 1483 minute (end of storm event), the 85th storm event flowrate that enters the drywell is less than the drywell steady-state infiltration flowrate (flow disposal rate). Therefore, the entire volume entering the drywell from 1462 minutes to 1483 minutes will infiltrate without overwhelming the drywell. This volume is 16.8 CF.

8773 CF - 8756.2 CF = 16.8 CF

Time (min)	Incremental Masscurve	Incremental Design Storm Depth (in)	Intensity (in/hr)	Undeveloped Runoff Coefficient (Cu)	Developed Runoff Coefficient (Cd)	Clear Peak Flow Rate (cfs)	Incremental Volume (cu-ft)	Cumulative Volume (cu-ft)	Volume infiltrated by drywell (CF)
1482.8	1	0.82	8.4239E-05	0.1	0.78	0.0002503	0.00450661	8772.9608	0.00450661
1483	1	0.82	0	0.1	0.78	0	0.00150204	8772.9623	0.00150204
	0	0	0	0	0	0	0	0	0

The total volume infiltrated as it enters the drywell during the 85th percentile storm event is 16.7 + 2292.1 + 16.8 = 2325.6 CF (2325 CF)

HydroCalc Volume Analysis

** (Values from project "Peak Flow Hydrologic Analysis")*

Project: TORRANCE APARTMENTS - Subarea ENTIRE SITE

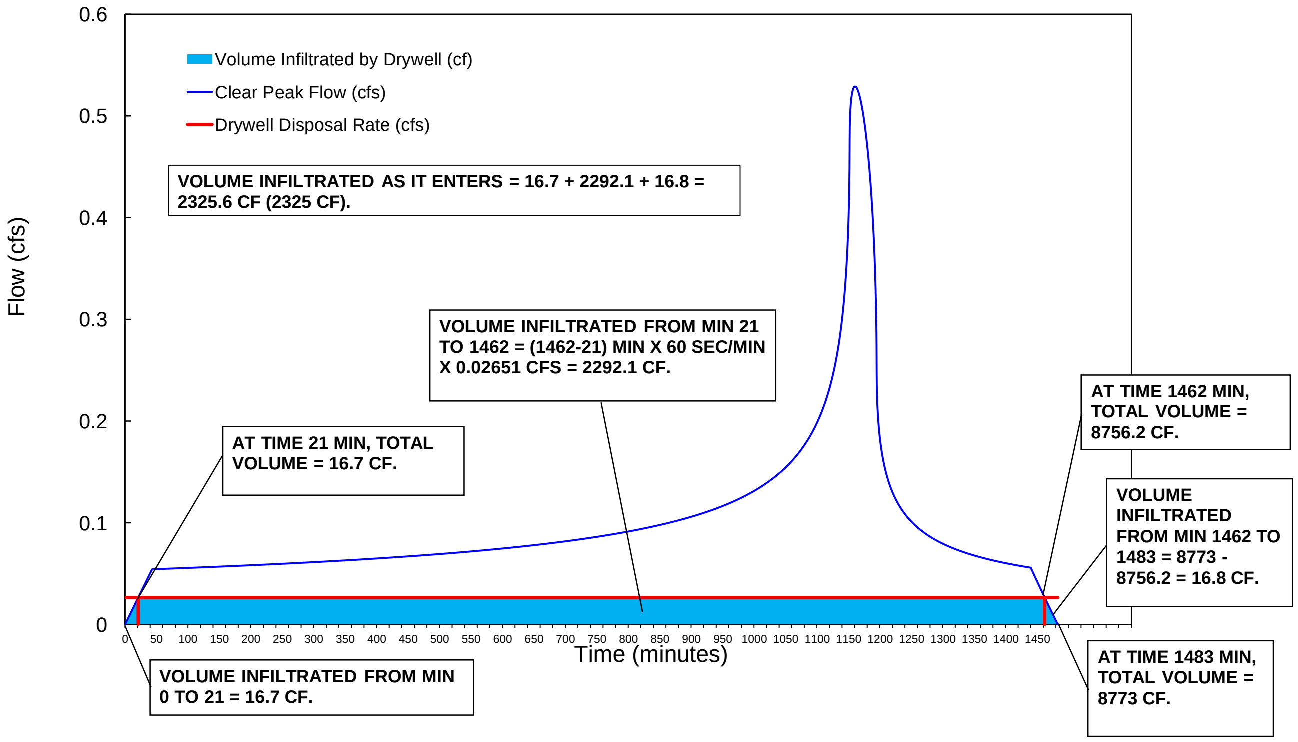
HydroCalc Output Results*

Clear Peak Flow (CFS)	0.5288
24-Hr Clear Runoff Volume (AC-FT)	0.2014
24-Hr Clear Runoff Volume (CF)	8773

Analysis

Drywell Disposal Rate (CFS)	***0.02651***
Total Volume Infiltrated During 1st Phase (CF)	16.7
[2nd Phase] Storm Flow Rate Exceeds Drywell Disposal Rate @ (MIN)	21
Total Volume Infiltrated During 2nd Phase (CF)	2292.1
[3rd Phase] Drywell Disposal Rate Exceeds Storm Flow Rate @ (MIN)	1462
Total Volume Infiltrated During 3rd Phase (CF)	16.8
Total Time of Storm Event (MIN)*	1483
Total Volume Infiltrated as it Enters Drywell (CF)	***2325***
Total Storage within MaxWell System (CF)	814
Remaining Detention Required (CF)	5634

Hydrograph: TORRANCE APARTMENTS - ENTIRE SITE



VOLUME INFILTRATED AS IT ENTERS = 16.7 + 2292.1 + 16.8 = 2325.6 CF (2325 CF).

VOLUME INFILTRATED FROM MIN 21 TO 1462 = (1462-21) MIN X 60 SEC/MIN X 0.02651 CFS = 2292.1 CF.

AT TIME 21 MIN, TOTAL VOLUME = 16.7 CF.

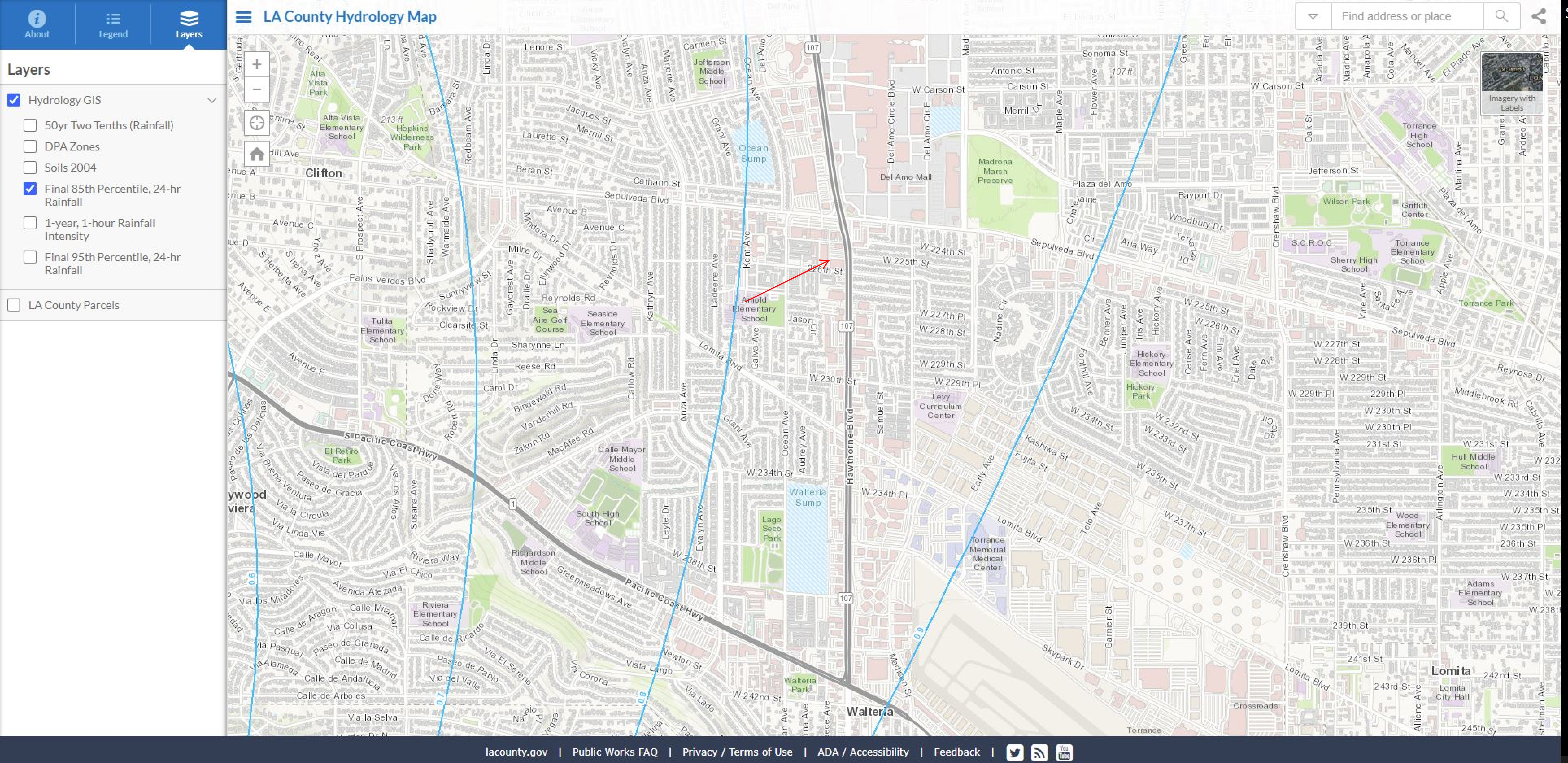
AT TIME 1462 MIN, TOTAL VOLUME = 8756.2 CF.

VOLUME INFILTRATED FROM MIN 1462 TO 1483 = 8773 - 8756.2 = 16.8 CF.

VOLUME INFILTRATED FROM MIN 0 TO 21 = 16.7 CF.

AT TIME 1483 MIN, TOTAL VOLUME = 8773 CF.

APPENDIX B – SUPPORTING DOCUMENTS



LA County Hydrology Map

Layers

- ☑ Hydrology GIS
  - ☐ 50yr Two Tenths (Rainfall)
  - ☐ DPA Zones
  - ☐ Soils 2004
  - ☑ Final 85th Percentile, 24-hr Rainfall
  - ☐ 1-year, 1-hour Rainfall Intensity
  - ☐ Final 95th Percentile, 24-hr Rainfall
- ☐ LA County Parcels

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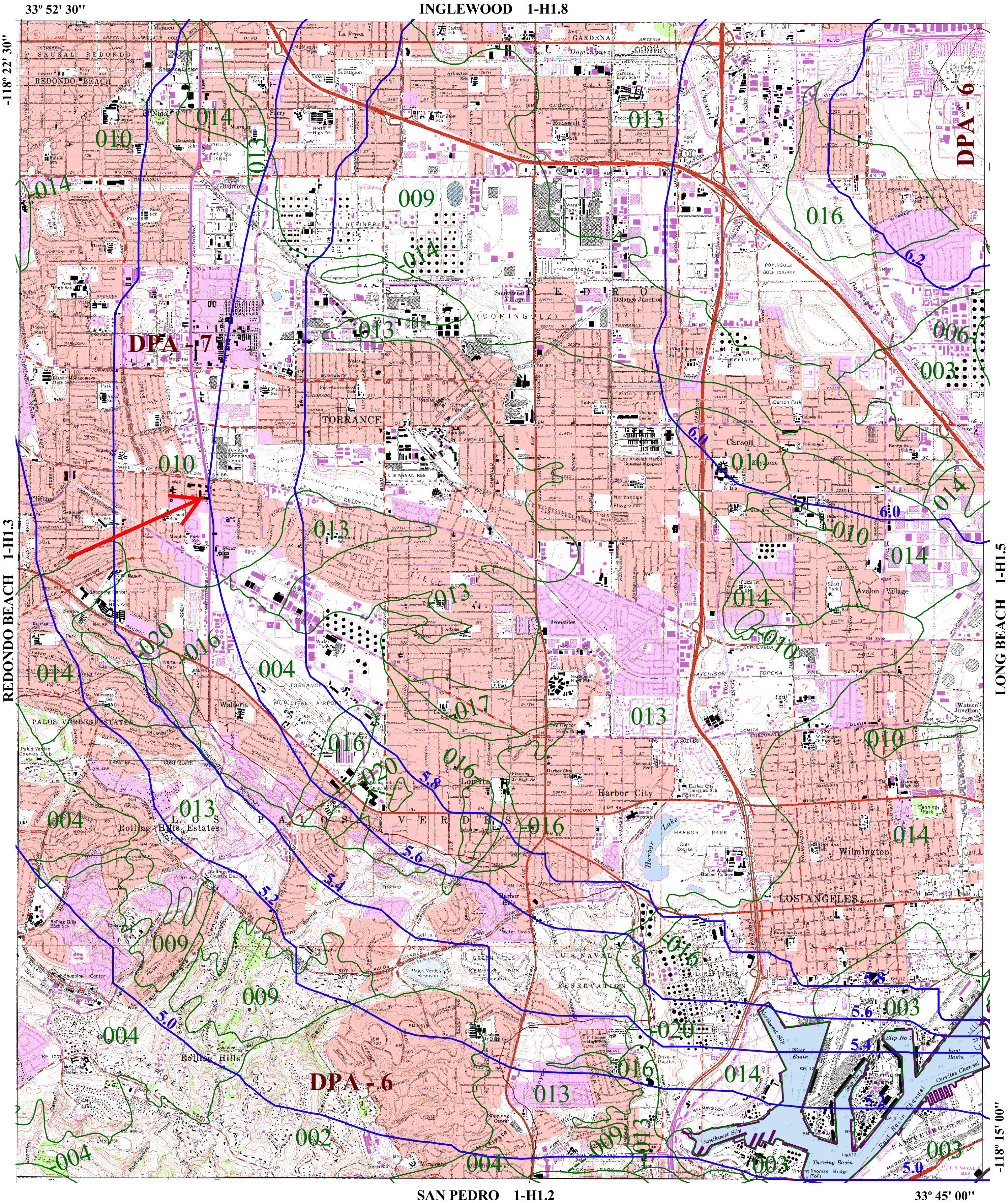


1 0 1 2 Miles

016 SOIL CLASSIFICATION AREA

7.2 INCHES OF RAINFALL

DPA - 6 DEBRIS POTENTIAL AREA

N
W E
S

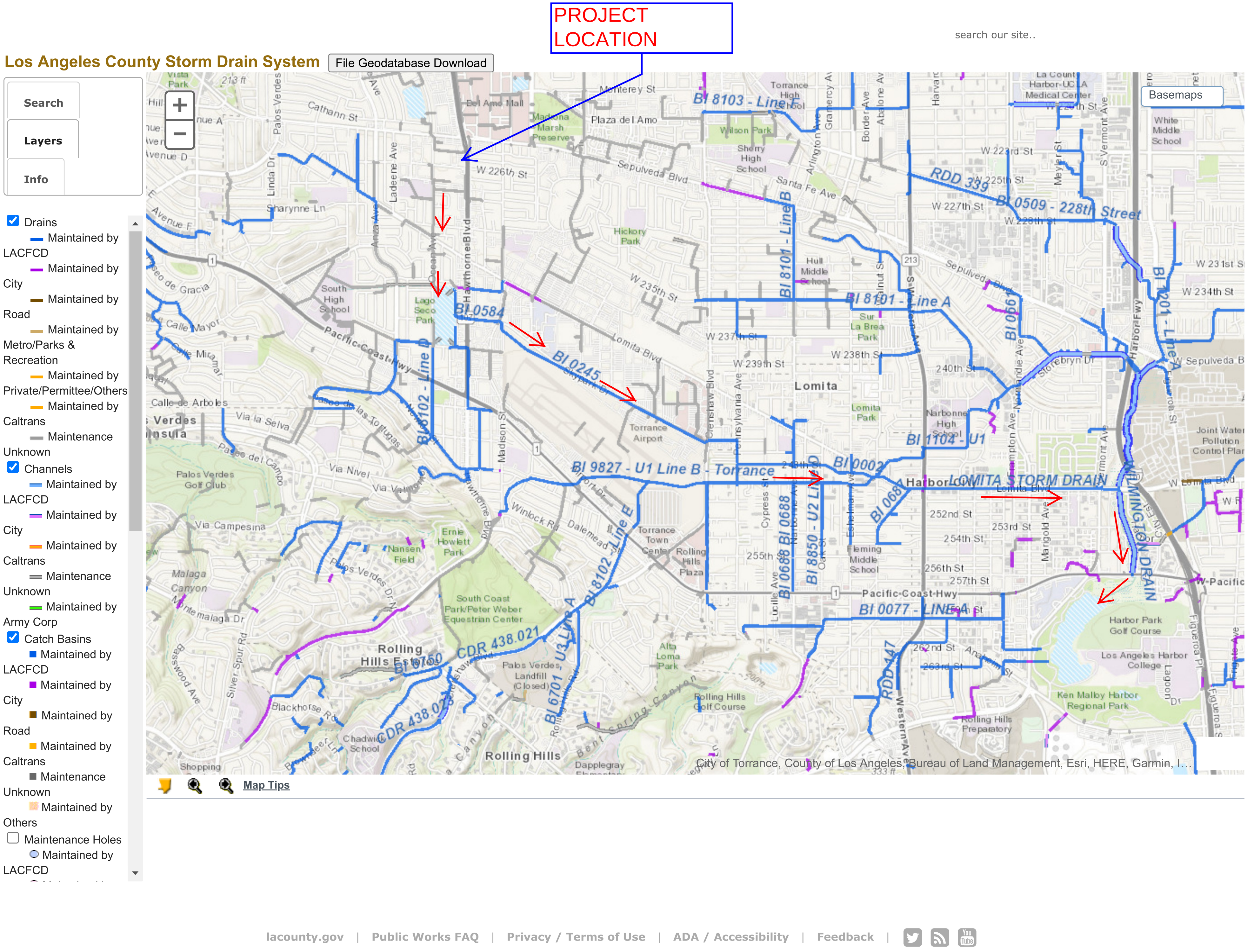
25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878
10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714

TORRANCE

1-H1.4

50-YEAR 24-HOUR ISOHYET

LOS ANGELES COUNTY
PUBLIC WORKS
Public Service That Works



PROJECT LOCATION

search our site..

Los Angeles County Storm Drain System

File Geodatabase Download

Search

Layers

Info

☑ Drains
- ▬ Maintained by LACFCD
- ▬ Maintained by City
- ▬ Maintained by Road
- ▬ Maintained by Metro/Parks & Recreation
- ▬ Maintained by Private/Permittee/Others
- ▬ Maintained by Caltrans
- ▬ Maintenance Unknown

☑ Channels
- ▬ Maintained by LACFCD
- ▬ Maintained by City
- ▬ Maintained by Caltrans
- ▬ Maintenance Unknown
- ▬ Maintained by Army Corp

☑ Catch Basins
- ■ Maintained by LACFCD
- ■ Maintained by City
- ■ Maintained by Road
- ■ Maintained by Caltrans
- ■ Maintenance Unknown
- ■ Maintained by Others

☐ Maintenance Holes
- ○ Maintained by LACFCD

Basemaps

Map Tips

City of Torrance, County of Los Angeles, Bureau of Land Management, Esri, HERE, Garmin, I...

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UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
22501 HAWTHORNE BOULEVARD
TORRANCE, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

INTRACORP HOMES
NEWPORT BEACH, CALIFORNIA

PROJECT NO. W1062-06-02

MAY 12, 2021

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1062-06-02
May 12, 2021

Mr. Joe Francis
Intracorp Homes
895 Dove Street, Suite 400
Newport Beach, CA 92660

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
22501 HAWTHORNE BOULEVARD
TORRANCE, CALIFORNIA

Dear Mr. Francis:

In accordance with your authorization of our proposal dated March 31, 2021, we have performed an updated geotechnical investigation for the proposed multi-family residential development located at 22501 Hawthorne Boulevard in the City of Torrance, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

Petrina Zen

PETRINA ZEN C87489 CIVIL
JELISA THOMAS ADAMS GE3092 GEOTECHNICAL
GERALD KASMAN NO. 2251 CERTIFIED ENGINEERING GEOLOGIST

Petrina Zen	Jelisa Thomas Adams	Gerald A. Kasman
PE 87489	GE 3092	CEG 2251

(EMAIL) Addressee

3303 N. San Fernando Blvd., Suite 100 ■ Burbank, California 91504 ■ Telephone (818) 841-8388 ■ Fax (818) 841-1704

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UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of an updated geotechnical investigation for the proposed multi-family residential development located at 22501 Hawthorne Boulevard in the City of Torrance, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The western portion of the site was initially explored on September 6, 2019 by excavating three 8-inch diameter borings to depths of approximately 65½ and 75½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Supplemental site exploration was performed on April 19, 2021 by excavating five 8-inch diameter borings to depths of approximately 10½ to 35½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 22501 Hawthorne Boulevard in the City of Torrance, California. The site is a square shaped parcel and is currently occupied by three single-story commercial structures and an asphalt paved parking lot. The site is bounded by commercial structures and paved parking areas to the north, by Hawthorne Boulevard to the east, by three-story multi-family residential structures the west, and by West 226th Street to the south. The site is relatively level with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of shrubs and trees located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of constructing three-story, multi-family residential structures consisting of 174 units and a six-story parking structure (see Site Plan, Figure 2). It is generally anticipated that the existing structures will be demolished for the proposed development. However, the existing structure located on the southeast corner of the property will be renovated into approximately 20,000 square foot of retail space, and it is assumed that new foundations may be utilized for support of the proposed improvements. The project will also include courtyard areas, a swimming pool, and surface parking. The development will be constructed at or near present site grade.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed residential and retail structures will be up to 300 kips, and wall loads will be up to 4 kips per linear foot. It is anticipated that column loads for the proposed parking structure will be up to 700 kips, and wall loads will be up to 8 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located on the Torrance Plain, a broad nearly flat alluviated area situated in the southern extreme of the Los Angeles Coastal Plain, northeast of the Palos Verdes hills. The Torrance Plain is a Pleistocene age marine surface only slightly dissected by local streams. Both fresh water and marine fossils are commonly found at shallow depths below the surface of the plain, suggesting that there were periods of complete or nearly complete emergence of the land between periods of subsidence.

Regionally, the Torrance Plain is located within the Peninsular Ranges geomorphic province, which is characterized by elongate northwest-trending mountain ridges separated by straight-sided sediment-filled valleys. The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults, such as the nearby Palos Verdes fault zone, located approximately 3 miles southwest of the site.

4. SOIL AND GEOLOGIC CONDITIONS

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

4.1 Artificial Fill

Artificial fill was encountered in our borings to a maximum depth of 5 feet below existing ground surface. The artificial fill generally consists of yellowish brown, dark gray to brown silty sand to sand and gray to black clay that can be characterized as moist and very loose to medium dense or soft to firm. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Dune Sand

The fill soils are underlain by Pleistocene age (older) dune sand deposits consisting of yellowish brown to reddish brown, poorly- to well-graded sand with minor interbeds of sandy clay, clayey sand, and silty sand with a trace of fine-gravel.The alluvial soils are primarily fine- to coarse-grained, slightly moist to wet and very loose to very dense or stiff to hard, and are fine grained.

5. GROUNDWATER

The site is not located within a groundwater basin. Based on a review of the Seismic Hazard Evaluation for the Torrance Quadrangle (California Division of Mines and Geology [CDMG], 1998), little data for the site vicinity is available. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s.

Groundwater was encountered in boring B2 at a depth of 74 feet below the existing ground surface. Considering the depth to groundwater encountered in our borings, groundwater is not anticipated to be encountered during construction. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.21).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2021a; CGS, 2021b; CDMG, 1999) or a city-designated Fault Hazard Management Zone (City of Torrance, 2010) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Palos Verdes Hills Fault Zone located approximately 1.3 miles to the southwest (CGS, 2021b, Ziony and Jones, 1989). Other nearby active faults include the Redondo Canyon Fault, the Cabrillo Fault, the Newport-Inglewood Fault Zone, and the Santa Monica Fault located approximately 2.8 miles to the west-southwest, 3.7 miles to the south, 6.8 miles to the east-northeast, and 16 miles to the west of the site, respectively. The active San Andreas Fault Zone is located approximately 51 miles northeast of the site (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	64	E
Long Beach	March 10, 1933	6.4	26	ESE
Tehachapi	July 21, 1952	7.5	89	NNW
San Fernando	February 9, 1971	6.6	41	N
Whittier Narrows	October 1, 1987	5.9	23	NE
Sierra Madre	June 28, 1991	5.8	36	NE
Landers	June 28, 1992	7.3	113	ENE
Big Bear	June 28, 1992	6.4	91	ENE
Northridge	January 17, 1994	6.7	29	NNW
Hector Mine	October 16, 1999	7.1	130	ENE
Ridgecrest	July 5, 2019	7.1	140	NNE

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

6.3 Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.836g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.664g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.836g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.128g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.224g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.752g*	Section 1613.2.4 (Eqn 16-39)

Note:

*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

The table on the following page presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.81g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.891g	Section 11.8.3 (Eqn 11.8-1)

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.87 magnitude event occurring at a hypocentral distance of 6.28 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.66 magnitude occurring at a hypocentral distance of 11.1 kilometers from the site.

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

6.4 Liquefaction Potential

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

The State of California Seismic Hazard Zone Map for the Torrance Quadrangle (CDMG, 1999) indicates that the site is not located in an area designated as having a potential for liquefaction. In addition, a review of the City of Torrance Safety Element (Torrance, 2010) indicates that the site is not located within an area identified as having a potential for liquefaction. Soils encountered in our borings were generally medium dense to dense and stiff to hard and well consolidated. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is low.

6.5 Slope Stability

The topography at the site and the immediate vicinity is relatively level. The site is not located within a City of Torrance Landslide Hazard Zone (Torrance, 2010). According to the County of Los Angeles Safety Element (Leighton, 1990), the site is not located within an area identified as a "Hillside Area" or an area identified as having a potential for slope instability. Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is not located within an inundation hazard area. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area; therefore, tsunamis are not considered a significant hazard at the site.

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2019; LACDPW, 2019b).

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (Cal GEM) Well Finder Website (CalGEM, 2021), the site is located within the Torrance Oil Field. Additionally, the Eyer Brothers Well #4, a plugged oil and gas production well operated by George E. & Lola Vardas, is located within the limits of the property. According to the CalGEM well finder, the well is located approximately 120 feet west of Hawthorne Boulevard and 250 feet north of W. 226th Street. CalGEM should be contacted to verify that the abandonment was performed in accordance with current standards. Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map. Undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

Since the site is within the Torrance Oil Field, there could be a potential for methane and other volatile gases to occur at the site which may require a permanent methane gas control system beneath the proposed buildings. Should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The City of Torrance Local Hazard Mitigation Plan indicates that there are no recognized past subsidence events in the Torrance area (City of Torrance, 2016). There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

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

7.1.3 Based on these considerations, it is recommended that the upper 5 feet of existing site soils within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. The excavation should extend laterally a minimum distance of three feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Where the recommended lateral over-excavation cannot be performed, such as adjacent to a property line or an existing foundation, foundations should be deepened as necessary to derive support in the undisturbed alluvial soils found at and below a depth of 5 feet. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.4 The proposed structures and improvements may be supported on a conventional foundation system deriving support in newly placed engineered fill and/or the competent alluvial soils found at and below a depth of 5 feet. It is the intent of the Geotechnical Engineer to allow building foundations to derive support in both engineered fill and competent alluvial soils for this project if conditions warrant such an occurrence. Alluvial soils exposed in foundation excavations should be properly compacted for foundation support. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete. Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.5 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.1.6 Where proposed foundations will be deeper than an existing foundation, the new foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

7.1.7 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Performing open excavations adjacent to or deeper than the existing foundation system could potentially remove lateral support and/or undermine the existing foundation. Excavation for construction of new foundations immediately adjacent to existing foundations may require special excavation measures in order to maintain lateral support of the existing adjacent foundation. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.17).

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structures, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvium at a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in Preliminary Pavement Recommendations section of this report (see Section 7.11).

7.1.10 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.20).

7.1.11 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Due to the granular nature of the soils, moderate to excessive caving is anticipated in unshored excavations. The contractor should be aware that formwork may be required to prevent caving of shallow spread foundation excavations.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.2.4 The upper 5 feet of existing site soils encountered during the investigation are considered to have a "very low" to "low" expansive potential (EI = 0 & 21) and are classified as "non-expansive" to "expansive" in accordance with the 2019 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the foundations and slabs will derive support in materials with a "low" expansion potential.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the upper site soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figures B62 and B63) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that corrosion-resistant ABS pipes (or equivalent) be utilized in lieu of cast-iron for subdrains and retaining wall drains beneath the structure.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B62 and B63) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Grading

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

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

7.4.3 Grading should commence with the removal of existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. Existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.4 As a minimum, it is recommended that the upper 5 feet of existing earth materials within the proposed building areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities. Where excavation and compaction is to be conducted, the excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Where the recommended lateral excavation cannot be performed due to property line constraints and/or the presence of existing structures, conventional foundations should be deepened as necessary to derive support in the undisturbed alluvial soils found at or below a depth of 5 feet.

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

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

7.4.7 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Performing open excavations adjacent to or deeper than the existing foundation system could potentially remove lateral support and/or undermine the existing foundation. Excavation for construction of new foundations immediately adjacent to existing foundations may require special excavation measures in order to maintain lateral support of the existing adjacent foundation. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.4.8. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.11).

7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structures, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvium at a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.4.10 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B62 and B63).

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

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 15 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent.

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

7.6 Foundation Design

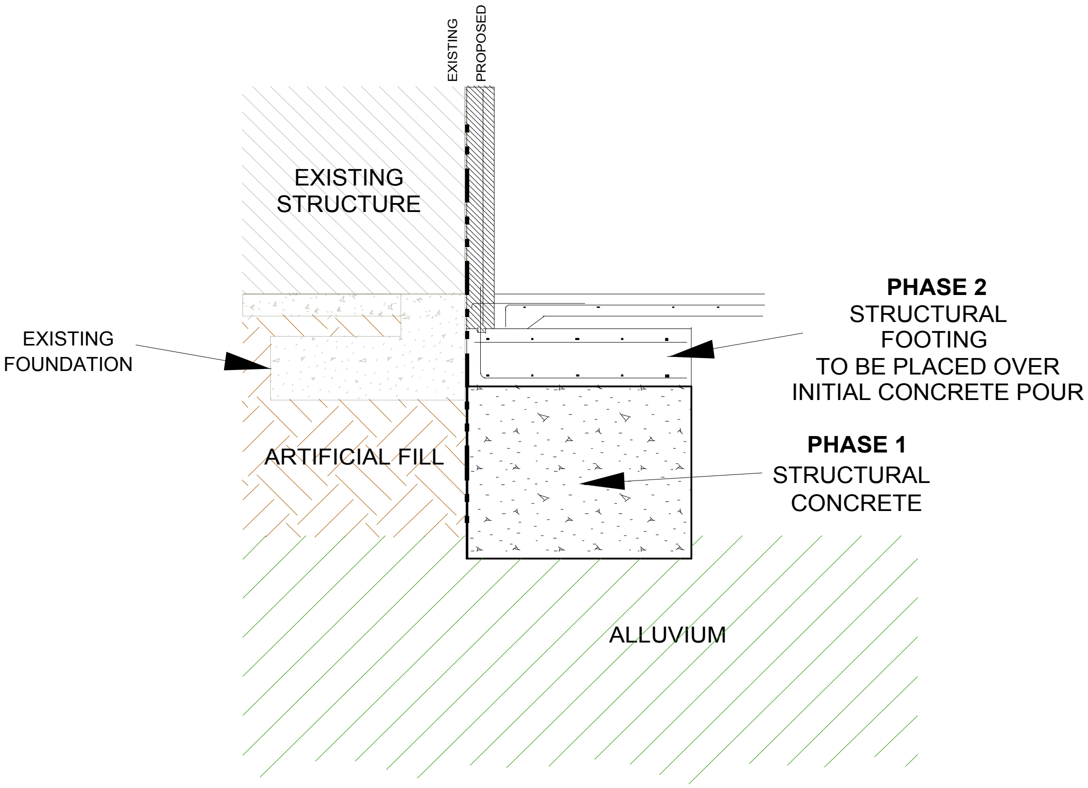
7.6.1 A conventional shallow spread foundation system may be utilized for support of the proposed structures and improvements provided foundations derive support in newly placed engineered fill and/or the competent alluvial soils found at and below a depth of 5 feet. It is the intent of the Geotechnical Engineer to allow building foundations to derive support in both engineered fill and competent alluvial soils for this project if conditions warrant such an occurrence. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to placing steel or concrete.

7.6.2 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.6.3 Where proposed foundations will be deeper than an existing foundation, the new foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

7.6.4 The client should be aware that special excavation measures, such as slot cutting or shoring, will be required to construct foundations along the property line or adjacent to existing foundations. Recommendations for temporary excavations are provided in Section 7.17.

7.6.5 Foundations along the property line or adjacent to existing foundations may be constructed in two phases using special excavation measures to create temporary excavations and quickly restore the majority of the support. The first phase of foundation construction will be to excavate a temporary excavation. The lower portion of the excavation, once approved by Geocon, can be backfilled with structural concrete up to the desired bottom of foundation depth. The project structural engineer should determine if the Phase 1 concrete pour requires any reinforcing and/or a key between the two pours. The excavation should be backfilled on the same day the excavation is opened. The second phase of the foundation construction will be to place the reinforced structural foundation on top of the previously placed Phase 1 concrete. The two-part foundation construction is illustrated on the following page.



7.6.6 If two-part foundation construction is used along the property line, the structural footing will be bounded laterally by artificial fill and, therefore, passive pressure along the sides of the foundations cannot be utilized. Resistance to lateral loads should be provided via structural connections to other portions of the structure.

7.6.7 Continuous footings may be designed for an allowable bearing capacity of 2,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.8 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.9 The allowable soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

7.6.10 In order to minimize settlements to less than ¾ inch between existing and proposed foundations, a reduced bearing capacity is being recommended. Continuous footings may be designed for an allowable bearing capacity of 1,800 pounds per square foot (psf) and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials. Isolated spread foundations may be designed for an allowable bearing capacity of 1,800 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.6.11 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

7.6.12 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

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

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

7.6.15 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

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

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

7.7 Foundation Settlement

7.7.1 The maximum expected static settlement for the residential and retail structures with assumed column loads of 300 kips and wall loads of 4 kips per linear foot, supported on a conventional foundation system designed with a maximum bearing pressure of 4,000 psf, and deriving support in the recommended bearing materials is estimated to be less than 1¼ inches and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ⅔ inch over a distance of 20 feet.

7.7.2 The maximum expected static settlement for the retail structure with assumed column loads of 300 kips and wall loads of 4 kips per linear foot, supported on a conventional foundation system designed with a maximum bearing pressure of 1,800 psf, and deriving support in the recommended bearing materials is estimated to be less than ¾ inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

7.7.3 The maximum expected static settlement for the parking structure with assumed column loads of 700 kips and wall loads of 8 kips per linear foot, supported on a conventional foundation system designed with a maximum bearing pressure of 4,000 psf, and deriving support in the recommended bearing materials is estimated to be less than 1½ inches and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¾ inch over a distance of 20 feet.

7.7.4 If side by side construction is planned for the residential structures and parking structure, it is recommended that the parking structure be constructed prior to the adjacent residential structure in order to allow the majority of the static settlement to occur in the parking structure. This will help to minimize differential settlements between the two structures. Additional settlement analyses should be performed once the foundation loading configuration for the proposed structures is established to further evaluate the potential for differential settlement between the residential structure and parking structure. The utilization of a lesser bearing value, or increasing the thickness of engineered fill below the foundations, would further reduce the anticipated settlements and could be evaluated once the design becomes more finalized.

7.7.5 It is recommended that a seismic separation or flexible connection be utilized where the adjacent structures abut. The design of the connection is at the discretion of the project structural engineer and should take into account potential differential settlements between structures.

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

7.8 Miscellaneous Foundations

7.8.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structures, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvium at a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

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

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

7.9 Lateral Design

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

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against competent alluvium or newly placed engineered fill may be computed as an equivalent fluid having a density of 240 pounds per cubic foot (pcf) with a maximum earth pressure of 2,400 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-on-Grade

7.10.1 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The finished subgrade must observed and be approved in writing prior to placement of a vapor retarder, reinforcing steel, or concrete.

7.10.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.10.3 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.10.4 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

7.11 Preliminary Pavement Recommendations

7.11.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.11.2 The following pavement sections are based on an assumed R-Value of 35. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	9.0

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

7.11.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

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

7.12 Retaining Wall Design

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

7.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* section of this report (see Section 7.6).

7.12.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 40 pcf.

7.12.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 61 pcf.

7.12.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 93 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.12.6 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

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

7.13 Retaining Wall Drainage

7.13.1 Retaining walls not designed for hydrostatic pressure should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 5). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.13.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 6). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.13.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

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

7.14 Swimming Pool

7.14.1 The proposed swimming pools should be designed as free-standing structures deriving support in newly placed engineered fill and/or the competent alluvial soils found at or below a depth of 5 feet.

7.14.2 Swimming pool foundations and walls may be designed in accordance with the foundation design recommendations below and *Retaining Wall Design* section of this report (see Section 7.12). The proposed pools should be constructed utilizing an expansive soils design, and a hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

7.14.3 If a spa is proposed it should be constructed independent of the swimming pool and must not be cantilevered from the swimming pool shell.

7.14.4 A reinforced concrete mat foundation may be utilized for support of the proposed swimming pool. The mat foundation for the pool may derive support in the undisturbed alluvial soils found at and below a depth of 5 feet. If necessary, these miscellaneous improvements may derive support in a combination of newly placed engineered fill and competent alluvium found at and below a depth of 5 feet.

7.14.5 It is anticipated that the proposed mat foundation will impart an average pressure of less than 1,500 psf. The recommended maximum allowable bearing value is 1,500 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.14.6 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch be utilized for the design of the mat foundation bearing on competent alluvial soils. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

Where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width in feet

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

7.14.8 Based on the soil overburden load that will be removed during excavation of the swimming pool, anticipated settlements are expected to be small. We estimate the total settlements for a mat foundation to be less than ½ inch, with differential settlements on the order of ¼ inch over a horizontal distance of 40 feet.

7.14.9 Foundation excavations should be observed by Geocon, prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* section of this report (see Sections 7.6 and 7.12).

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

7.15.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.13).

7.15.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.16 Elevator Piston

7.16.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation , especially if the drilling is performed subsequent to the foundation construction.

7.16.2 Casing will be required since caving is expected in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.16.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.17 Temporary Excavations

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

7.17.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 8 feet in height. A uniform slope does not have a vertical portion.

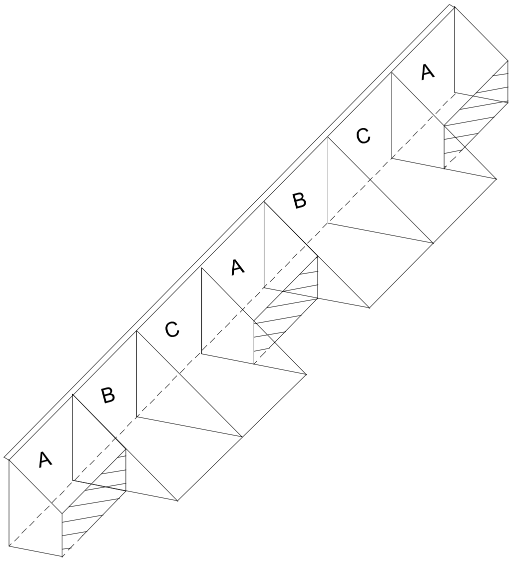
7.17.3 Performing continuous vertical excavations along property lines or adjacent to an existing structure could remove support from the property and/or structure which is not acceptable. Continuous vertical excavations along the public right-of-way should not exceed 2 feet in height. If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for slot cutting and shoring are provided in Sections 7.18 and 7.19 of this report.

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

7.18 Slot Cutting

7.18.1 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. Where slot-cutting is used for foundation construction, the proposed construction techniques should be discussed with the structural engineer so that appropriate modifications can be made to the foundation design, such as additional reinforcing or details for doweling.

7.18.2 It is recommended that the initial temporary excavation along the property line be sloped back at a uniform 1:1 (H:V) slope gradient or flatter for excavation of the existing soils to the necessary depth. The temporary excavation should not extend below the surcharge area of any adjacent foundations. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation. The temporary slope may then be excavated using the slot-cutting (see illustration below).



7.18.3 Alternate "A" slots of 8 feet in width may be worked. The remaining earth buttresses ("B" and "C" slots) should also be 8 feet in width. The wall, foundation, or backfill should be completed in the "A" slots to a point where support of the offsite property and/or any existing structures is restored before the "B" slots are excavated. After completing the wall, foundation, or backfill in the "B" slots, finally the "C" slots may be excavated. Slot-cutting is not recommended for vertical excavations greater than 5 feet in height. A slot-cut calculation is provided below, and assumes no surcharge loads will be acting on the excavation. If surcharge loads will be present, the slot-cut calculation should be revised as necessary.

Slot Cut Calculation

Input:

Height of Slots	(H)	5.0	feet
Unit Weight of Soils	(γ)	125.0	pcf
Friction Angle of Soils	(ϕ)	31.0	degrees
Cohesion of Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Design Equations

$b = H/(\tan \alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin \alpha)$

$R_1 = d*[W*(\cos\alpha)*(\tan \phi)+(c*b)]$

$R_2 = 2*[(0.5*H*b)*c]$

FS = Resistance Force/Driving Force

$FS = (R_1+R_2)/(F_1)$

Surcharge Pressure:

Line Load	(q_L)	0.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Failure Angle (α) degrees	Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet^2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force lbs	**Allowable Width of Slots*** (d) feet
45	5.0	13	1562.5	1104.9	1371.0	2500.0	8.0
46	4.8	12	1508.9	1085.4	1324.9	2414.2	8.0
47	4.7	12	1457.1	1065.6	1280.7	2331.3	8.0
48	4.5	11	1406.9	1045.5	1238.5	2251.0	8.0
49	4.3	11	1358.3	1025.1	1197.9	2173.2	8.0
50	4.2	10	1311.1	1004.4	1159.1	2097.7	8.0
51	4.0	10	1265.3	983.3	1121.8	2024.5	8.0
52	3.9	10	1220.8	962.0	1086.1	1953.2	8.0
53	3.8	9	1177.4	940.3	1051.8	1883.9	8.0
54	3.6	9	1135.2	918.4	1019.0	1816.4	8.0
55	3.5	9	1094.1	896.2	987.4	1750.5	8.0
56	3.4	8	1053.9	873.7	957.2	1686.3	8.0
57	3.2	8	1014.7	851.0	928.2	1623.5	8.0
58	3.1	8	976.4	828.0	900.5	1562.2	8.0
59	3.0	8	938.8	804.7	873.9	1502.2	8.0
60	2.9	7	902.1	781.3	848.4	1443.4	8.0
61	2.8	7	866.1	757.5	824.0	1385.8	8.0
62	2.7	7	830.8	733.5	800.6	1329.3	8.0
63	2.5	6	796.1	709.4	778.3	1273.8	8.0
64	2.4	6	762.1	685.0	757.0	1219.3	8.0
65	2.3	6	728.6	660.3	736.7	1165.8	8.0
66	2.2	6	695.7	635.5	717.3	1113.1	8.0
67	2.1	5	663.2	610.5	698.9	1061.2	8.0
68	2.0	5	631.3	585.3	681.4	1010.1	8.0
69	1.9	5	599.8	559.9	664.7	959.7	8.0
70	1.8	5	568.7	534.4	649.0	909.9	8.0

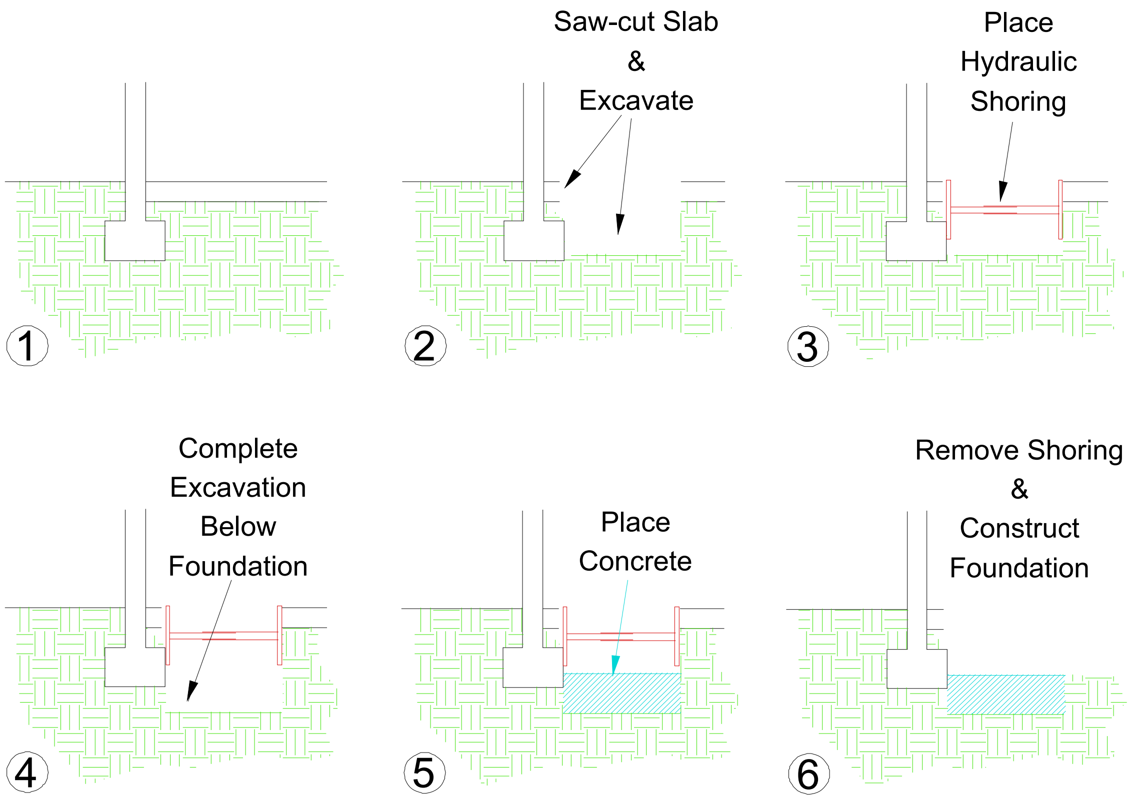
* Width of Slots to achieve a minimum of 1.25 Factor of Safety, w ith a Maximum Allow able Slot Width of 8-feet.

Critical Slot Width w ith Factor of Safety equal or exceeding 1.25:

d_{allow} = 8.0 feet

7.19 Trench Shoring

7.19.1 To protect the existing footings, hydraulic trench shoring may be implemented where excavations will extend below existing foundations. The excavation may be conducted adjacent to the foundation but should not extend below the foundation until the shoring is installed. Once shoring is installed the excavation can be completed. Once the concrete is placed to an elevation that is slightly above the bottom of the existing adjacent foundation, the shoring may be removed and the new foundation constructed. See illustration below.



7.19.2 It is recommended that an equivalent fluid pressure based on the table below, be utilized for design of hydraulic shoring.

HEIGHT OF SHORED EXCAVATION (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 5	31	61

7.19.3 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, the at-rest pressure should be considered for design purposes.

7.19.4 A qualified engineer should be retained to review and prepare a shoring plan in accordance with the shoring manufacture's specifications.

7.19.5 Additional active pressure should be added for a surcharge condition due to adjacent structures and should be designed for each condition as the project progresses.

7.19.6 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

7.19.7 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\ (z) = \ \sigma_H(z)cos^2(1.1\theta)$$

where x is the distance from the face of the excavation/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z, θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z.

7.20 Stormwater Infiltration

7.20.1 During the September 6, 2019 site exploration, boring B2 was utilized to perform percolation testing. The boring was drilled and advanced to the depth listed in the table below. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with filter pack. The boring was then filled with water to pre-saturate the soils. After pre-saturating the soils, the casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figure 7.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B2	Sand and Clay (SP/CL/SW)	50-60	4.86	1.62

7.20.2 Based on the test method utilized (Boring Percolation Test), the reduction factor RF_t may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor RF_s may be taken as 1.0.

7.20.3 The results of the percolation testing indicate that soils at the location and depths listed in the table above are conductive to infiltration, and it is our opinion that the site is suitable for infiltration of stormwater at the locations tested above.

7.20.4 It is our further opinion that infiltration of stormwater and will not induce excessive hydro-consolidation at the location of percolation testing (see Figures B25 through B54), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any. If infiltration is planned for any location other than where the above testing was performed, additional onsite and laboratory testing may be required.

7.20.5 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 15 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.20.6 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum 2-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.20.7 The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.21 Surface Drainage

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

7.21.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.21.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

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

7.22 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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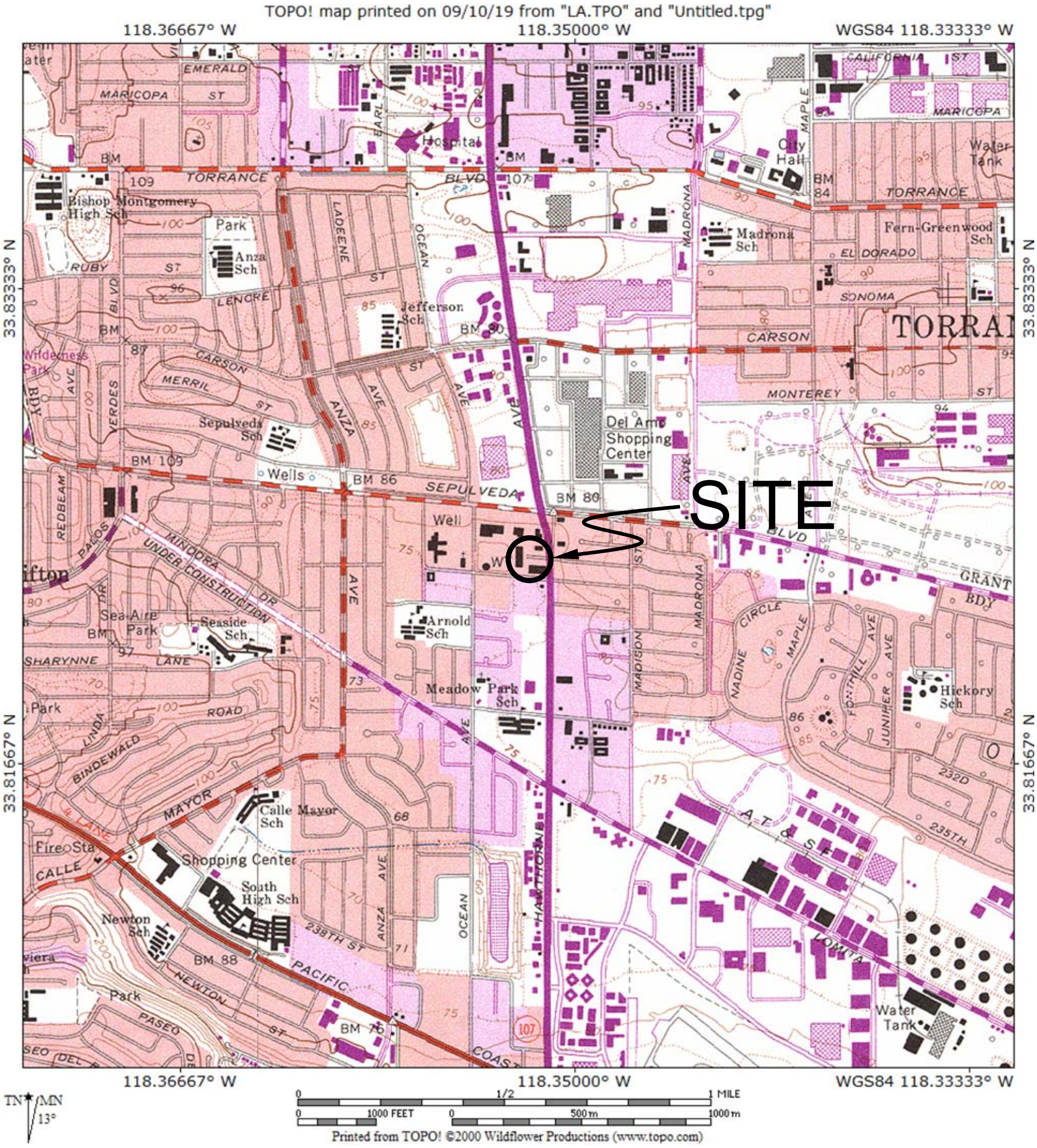

U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, TORRANCE, CA QUADRANGLE

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DRAFTED BY: RA
CHECKED BY: SFK

VICINITY MAP

22501 HAWTHORNE BOULEVARD
TORRANCE, CALIFORNIA

MAY 2021
PROJECT NO. W1062-06-02
FIG. 1

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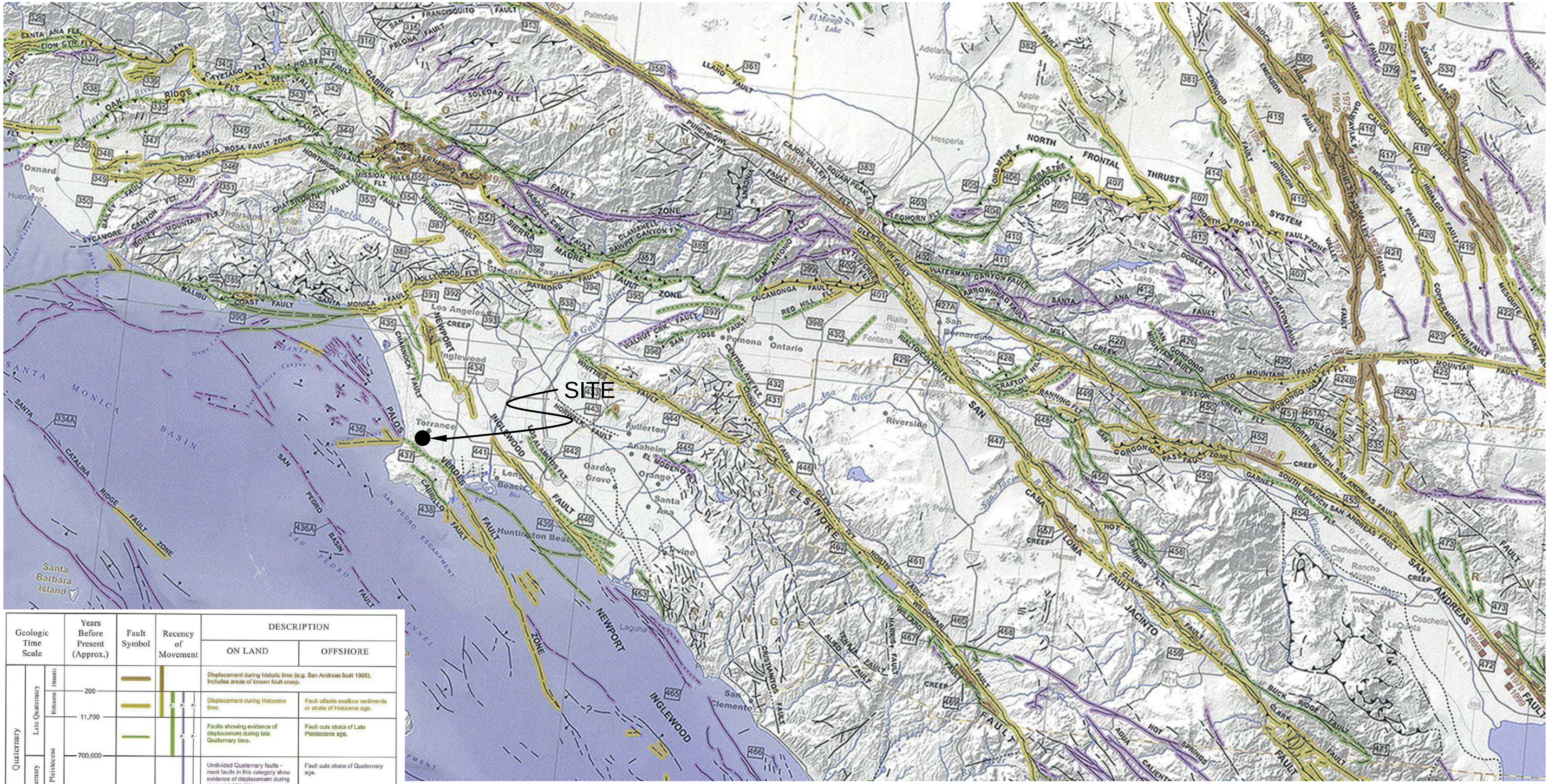
SITE PLAN

22501 HAWTHORNE BOULEVARD
TORRANCE, CALIFORNIA

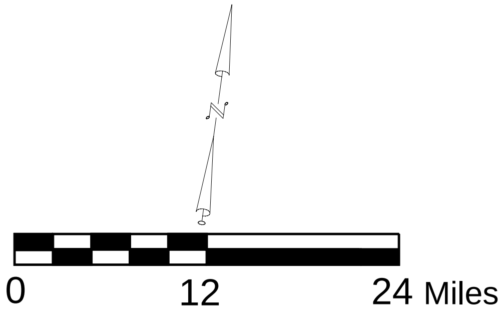
MAY 2021	PROJECT NO. W1062-06-02	FIG. 2

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



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

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



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REGIONAL FAULT MAP

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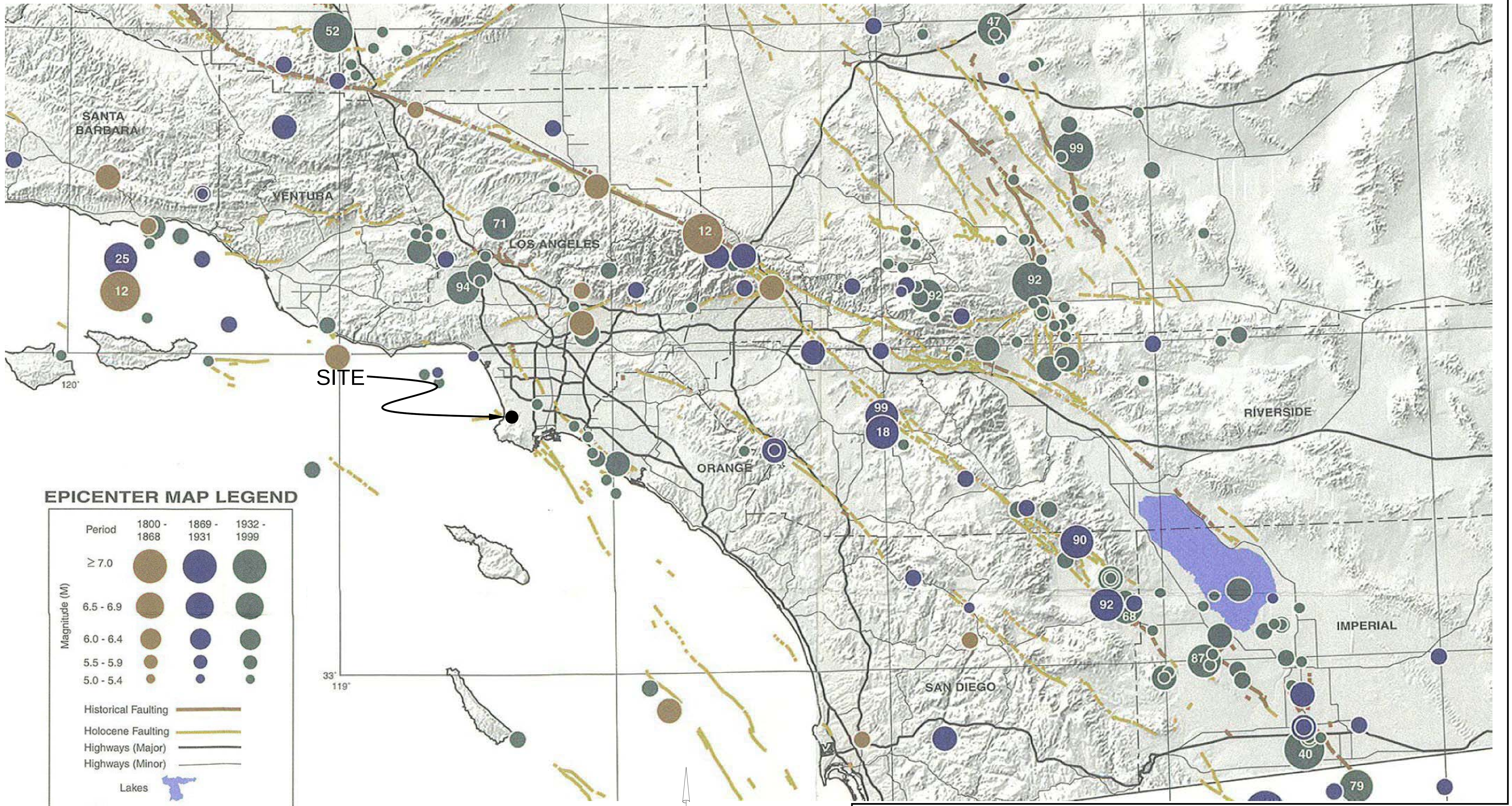
PROJECT NO. W1062-06-02
FIG. 3



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

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REGIONAL SEISMICITY MAP

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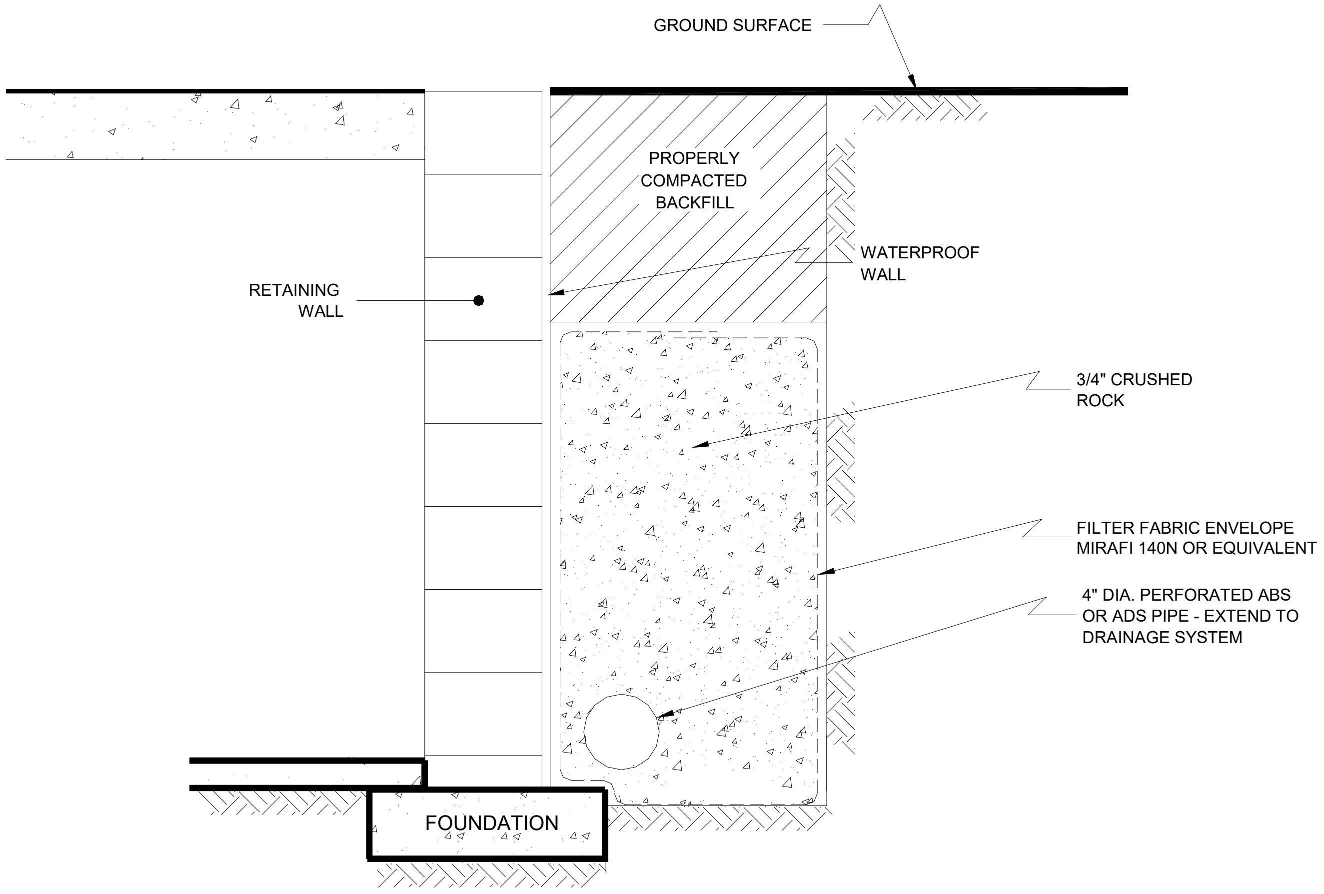
MAY 2021 | PROJECT NO. W1062-06-02 | FIG. 4



NO SCALE

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RETAINING WALL DRAIN DETAIL

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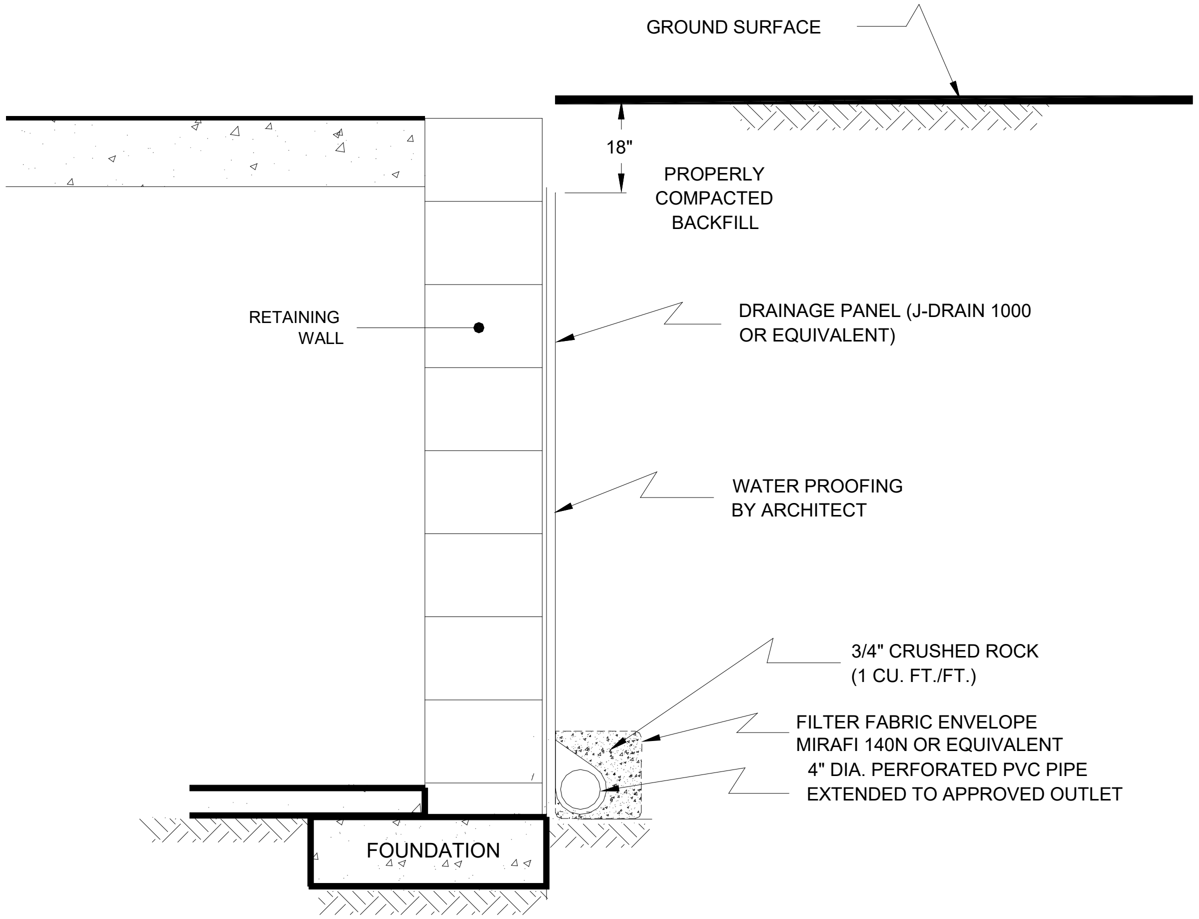
MAY 2021 | PROJECT NO. W1062-06-02 | FIG. 5



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RETAINING WALL DRAIN DETAIL

22501 HAWTHORNE BOULEVARD
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MAY 2021 | PROJECT NO. W1062-06-02 | FIG. 6

BORING PERCOLATION TEST FIELD LOG

Date: 9/6/2019
Project Number: W1062-06-02
Project Location: Gable House - Torrance
Earth Description: SP/CL/SW
Tested By: JAO
Liquid Description: Clear Clean Tap Water
Measurement Method: Sounder

Boring/Test Number: B2
Diameter of Boring: 8 inches
Diameter of Casing: 2 inches
Depth of Boring: 60 feet
Depth to Invert of BMP: 50 feet
Depth to Water Table: 74 feet
Depth to Initial Water Depth (d_1): 600 inches

Start Time for Pre-Soak: 10:30 AM
Start Time for Standard: 11:30 AM

Water Remaining in Boring (Y/N): No
Standard Time Interval Between Readings: 10 min

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	11:31 AM	11:41 AM	10	55.2	
2	11:43 AM	11:53 AM	10	53.6	
3	11:56 AM	12:06 PM	10	52.8	
4	12:10 PM	12:20 PM	10	50.5	
5	12:25 PM	12:35 PM	10	52.2	
6	12:38 PM	12:48 PM	10	47.8	Stabilized Readings
7	12:52 PM	1:02 PM	10	49.9	Achieved with Readings
8	1:06 PM	1:16 PM	10	50.4	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
Test Section Height, h: 120.0 inches

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

A = **3066** in^2

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

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

Reading 6	V = **2401** in^3	Percolation Rate =	**4.70** inches/hour
Reading 7	V = **2509** in^3	Percolation Rate =	**4.91** inches/hour
Reading 8	V = **2533** in^3	Percolation Rate =	**4.96** inches/hour

Measured Percolation Rate = **4.86** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 1
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

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

Total Reduction Factor = 3

Design Infiltration Rate

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

Design Infiltration Rate = **1.62** inches/hour

FIGURE 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was initially explored on September 6, 2019 by excavating three 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 65½ and 75½ feet below the existing ground surface. Supplemental site exploration was performed on April 19, 2021 by excavating five 8-inch diameter borings to depths of approximately 10½ to 35½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by $2^{3}/_{8}$-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples at proposed footing depths were also obtained. Percolation testing was performed in boring B2.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A8. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

PROJECT NO. W1062-06-02

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 9/16/19

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 5" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, moist, yellowish brown, fine-grained.			
2								
4								
	B1@5' ■				**OLDER DUNE SAND** Silty Sand, loose, moist, brown, fine-grained.	8	98.0	6.7
6				SM				
8					Sand, medium dense, moist to wet, yellowish brown, fine-grained, some clay.			
10	B1@10' ■					27	107.1	18.6
12								
14								
	B1@15' ■					63	109.8	14.5
16								
18				SP				
20	B1@20' ■				- slightly moist, light brown to brown	58	104.1	6.8
22								
24								
	B1@25' ■				- light brown	50 (5)	103.8	4.9
26								
28								

W1062-06-02 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 3

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ☒ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◩ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. W1062-06-02

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 9/16/19

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30' ■				- very dense, some medium-grained	50 (5)	111.1	4.1
32								
34								
	B1@35' ■				- dense, fine-grained	50 (6)	95.9	6.1
36								
38	B1@37.5' ■					50 (6)	96.8	6.5
40	B1@40' ■			SP	- trace medium-grained	50 (6)	100.3	3.7
42								
44								
	B1@45' ■				- light brown, fine-grained	50 (5)	99.5	4.3
46								
48								
50	B1@50' ■					50 (5)	92.8	6.1
52								
54				CL	Clay, hard, moist, grayish brown, some oxidation mottles, some fine-grianed sand.			
	B1@55' ■					50 (5)	108.7	19.9
56					Sand, dense, moist, olive brown, trace clay.			
58				SP				

W1062-06-02 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **9/16/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@60' ■			CL	Sandy Clay, hard to dense, moist, olive brown with orange mottles, fine-grained.	52	105.2	19.7
62								
64				SP	Sand, dense, moist to wet, brown, some oxidation, medium-to coarse-grained, some clay.			
	B1@65' ■					50 (6)	121.7	10.9

Total depth of boring: 65.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

W1062-06-02 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 3 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. W1062-06-02

BORING 2

ELEV. (MSL.) **--** DATE COMPLETED **9/16/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, moist, yellowish brown, fine-grained.			
2								
				SP	**OLDER DUNE SAND** Sand, medium dense, moist, brown, some clay, fine-grained.			
4								
	B2@5'			SM	Silty Sand, loose, moist, dark yellowish brown, fine-grained, some medium-grained.	6	88.0	23.3
6								
8				CL	Sandy Clay, stiff, moist, dark yellowish brown, fine- to medium-grained.			
10	B2@10'					35	118.3	15.1
12								
				SC	Clayey Sand, medium dense, moist, dark yellowish brown, fine-grained.			
14								
	B2@15'					52	122.1	10.5
16								
18								
20	B2@20'				- decrease in clay	46		9.1
22								
				SP	Sand, medium dense, slighlty moist, light brown, fine-grained.			
24								
	B2@25'					40	101.5	4.3
26								
28								

W1062-06-02 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 3

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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GEOCON

PROJECT NO. W1062-06-02

BORING 2

ELEV. (MSL.) **--** DATE COMPLETED **9/16/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'				- moist, fine- to medium-grained	52	106.9	6.6
32								
	B2@32.5'				- dense, fine- to medium-grained, trace coarse-grained	50 (6)	102.5	5.3
34				SP				
	B2@35'				- fine- to medium-grained	50 (4)	99.2	6.5
36								
38	B2@37.5'					50 (6)	102.8	8.0
					Sand with Clay, dense to hard, moist, light olive brown to reddish brown, fine- to medium-grained, abundant oxidation.			
40	B2@40'			SP-SC		50 (4)	99.2	3.5
42								
					Sand, dense, moist, light brown, fine-grained.			
44								
	B2@45'					50 (6)	102.7	3.5
46								
48								
				SP				
50	B2@50'					50 (6)	92.2	6.4
52								
54								
	B2@55'				- fine- to medium-grained	73	106.2	3.2
56								
58								
				CL	Clay, hard, moist, reddish brown, some medium-grained sand.			

Figure A2,
Log of Boring 2, Page 2 of 3

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. W1062-06-02

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 9/16/19

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60'		CL		51	128.2	10.5
62							
			SW	Sand, dense, moist, well-graded, brown.			
64							
	B2@65'				50 (5)	118.2	6.6
66							
68							
70	B2@70'				50 (6)	111.7	4.7
72							
74		▼					
	B2@75'			- wet, medium- to coarse-grained	50 (6)	114.7	12.9

Total depth of boring: 75.5 feet
Fill to 2.5 feet.
Groundwater encountered at 74 feet.
Percolation testing performed.
Backfiled with soil cuttings and tamped.
Asphalt patched.

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

Figure A2,
Log of Boring 2, Page 3 of 3

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. W1062-06-02

BORING 3

ELEV. (MSL.) **--** DATE COMPLETED **9/16/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, moist, brown, fine-grained.			
2								
					OLDER DUNE SAND Silty Sand, very loose, moist, brown, fine-grained.			
4								
	B3@5'			SM		2	95.2	13.4
6								
8					Clayey Sand, medium dense, moist, brown, fine-grained.			
10	B3@10'			SC		33	112.7	13.6
12								
					Silty Sand, medium dense, brown, fine-grained.			
14								
	B3@15'			SM		40	102.9	18.0
16								
18					Sand, dense, moist, brown, fine-grained.			
20	B3@20'					61	105.6	5.2
22								
				SP				
24								
	B3@25'				- light brown	50 (6")	95.1	10.6
26								
28								

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

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. W1062-06-02

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 9/16/19

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30' ■ BULK ☒ 30-35'					50 (4")		
32								
	B3@32.5' ■				- fine- to medium-grained	50 (6")	105.6	5.2
34				SP				
	B3@35' ■				- very dense	50 (5")	97.9	9.2
36								
38	B3@37.5' ■				- fine-grained	50 (3")	107.3	3.7
					Clay, hard, moist, light brown.			
40	B3@40' ■					50 (5")	70.3	52.6
				CL				
42								
					Sand, dense, slightly moist, light brown, fine-grained.			
44								
	B3@45' ■					50 (5")	97.1	5.0
46								
48								
50				SP				
	B3@50' ■				- light yellowish brown to light brown	50 (5")	86.2	8.7
52								
54								
	B3@55' ■				- light brown	50 (4")	95.8	5.6
56								
58								

Figure A3,
Log of Boring 3, Page 2 of 3

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

BORING 3

ELEV. (MSL.) **--** DATE COMPLETED **9/16/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3@60' ■					50 (6")	89.3	10.2
62								
				SP				
64								
	B3@65' ■				- moist, dark brown, fine- to medium-grained, some clay	50 (5")	121.4	12.0

Total depth of boring: 65.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfiled with soil cuttings and tamped.
Asphalt patched.

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

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

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. W1062-06-02

BORING 4

ELEV. (MSL.) **--** DATE COMPLETED **4/19/21**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 3" SAND: 5.5"** **ARTIFICIAL FILL** Sand, poorly graded, medium dense, moist, brown, medium-grained, trace clay.			
2								
	B4@2.5'			SP-SM	**OLDER DUNE SAND** Sand with Silt, loose, moist, brown, fine-grained.	13		
4								
	B4@5'				- medium dense, some medium-grained	29	115.7	9.0
6								
				SP	Sand, poorly graded, medium dense, moist, light brown, fine- to medium-grained.			
8	B4@7.5'					28	111.1	8.6
10	B4@10' BULK 10-15'					33	107.3	7.8
12				SP-SC	Sand with Clay, medium dense, moist, yellowish brown, fine- to medium-grained.			
	B4@12.5'					33	113.2	10.7
14								
	B4@15'				- some light brown mottles	41	106.4	11.0
16								
				SP	Sand, poorly graded, medium dense, slightly moist, light brown, fine-grained, some medium-grained.			
18	B4@17.5'					47	101.2	7.8
20	B4@20'					48	99.7	6.3
22								
24								
	B4@25'				- very dense	50 (5.5")	100.7	4.5
26								
28								

W1062-06-02 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. W1062-06-02

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 4/19/21

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30' ■				- increase in medium-grained	50 (4.5")	103.9	5.0
32				SP				
34								
	B4@35' ■					50 (6")	97.2	6.3

Total depth of boring: 35.5 feet
Fill to 2.75 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A4,
Log of Boring 4, Page 2 of 2

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 4/19/21

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 2" BASE: NONE** **ARTIFICIAL FILL** Sand, poorly graded, loose, moist, brown, fine- to medium-grained.			
2	B5@2.5'				Clay, firm, moist, gray, trace oil/tar deposits. - some fine- to medium-grained	21	100.9	22.7
4					**OLDER DUNE SAND** Sand, loose, moist, dark brown, fine-grained.			
	B5@5'					12	105.0	7.2
6								
8	B5@7.5'			SP	- moist to wet, yellowish brown	14	101.9	13.9
10	B5@10' BULK 10-15'				- medium dense, light brown	25	104.8	17.0
12					Sandy Clay, stiff, moist, yellowish brown, fine- to medium-grained.			
	B5@12.5'					24	106.0	19.5
14				CL				
	B5@15'					28	107.6	19.9
16					Sand, medium dense, moist, light yellowish brown, fine-grained, some medium-grained.			
18	B5@17.5'			SP		49	103.9	11.8
20	B5@20'				- dense, light brown, decrease in medium-grained	62	103.7	7.2

Total depth of boring: 20.5 feet
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface patched.

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

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1062-06-02 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. W1062-06-02

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 4/19/21

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 4.5" BASE: NONE** **ARTIFICIAL FILL** Sand, poorly graded, medium dense, moist, very dark brown, fine- to medium-grained.			
2								
	B6@2.5'				- very loose, light yellowish brown	6	96.0	22.7
					Clay, soft, moist, black.			
4					**OLDER DUNE SAND** Sand, poorly graded, very loose, moist, dark yellowish brown, fine-grained.			
	B6@5'					3		
6								
8	B6@7.5'			SP		5	100.6	8.3
10	B6@10' BULK 10-15'				- loose, moist to wet, trace silt	9	108.5	19.4
12					Sand with Clay, poorly graded, medium dense, moist, yellowish brown, fine- to medium-grained.			
	B6@12.5'					41		
14				SP-SC				
	B6@15'				- decrease in clay	36	103.5	14.1
16								
					Sand, poorly graded, medium dense, slightly moist, light brown, fine-grained, some medium-grained.			
18	B6@17.5'					49	99.4	6.4
20	B6@20'				- very dense	50 (6")	101.5	5.1
22								
				SP				
24								
	B6@25'					50 (6")		
26								
28								

Figure A6,
Log of Boring 6, Page 1 of 2

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. W1062-06-02

BORING 6

ELEV. (MSL.) --   DATE COMPLETED 4/19/21

EQUIPMENT **HOLLOW STEM AUGER**   BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B6@30'				- increase in medium-grained, trace coarse-grained	50 (5.5")		
32				SP				
34								
	B6@35'				- decrease in medium-to coarse-grained	50 (5.5")	96.7	7.4

Total depth of boring: 35.5 feet
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1062-06-02 BORING LOGS.GPJ

Figure A6,
Log of Boring 6, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. W1062-06-02

BORING 7

ELEV. (MSL.) -- DATE COMPLETED 4/19/21

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5' ⊠				**AC: 3" BASE: NONE** **ARTIFICIAL FILL** Sand, poorly graded, loose, moist, dark brown, fine- to medium-grained.			
2								
	B7@2.5' ■				- light brown, trace fine gravel	19	110.7	12.0
4					**OLDER DUNE SAND** Sand, poorly graded, loose, moist, brown to dark yellowish brown, fine-grained.			
	B7@5' ■					12	105.2	105.2
6				SP				
8	B7@7.5' ■				- moist to wet, light brown, trace silt	11		
10	B7@10' ■			SP-SC	Sand with Clay, medium dense, moist, brown, fine-grained, some medium-grained.	24		

Total depth of boring: 10.5 feet
Fill to 3.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A7,
Log of Boring 7, Page 1 of 1

W1062-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◣ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. W1062-06-02

BORING 8

ELEV. (MSL.) **--** DATE COMPLETED **4/19/21**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 2.5" BASE: NONE** **ARTIFICIAL FILL** Sand, poorly graded, loose, moist, brown and dark brown, fine- to medium-grained, some coarse-grained.			
2	B8@2.5'					13	113.2	7.8
4					**OLDER DUNE SAND** Sand, poorly graded, very loose, moist, yellowish brown, fine-grained.			
	B8@5'					5		
6				SP				
8	B8@7.5'				- moist to wet, trace to some silt	16	106.9	14.6
				SC	Clayey Sand, medium dense, moist, yellowish brown, fine- to medium-grained.			
10	B8@10'					35	117.7	12.2

Total depth of boring: 10.5 feet
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A8,
Log of Boring 8, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

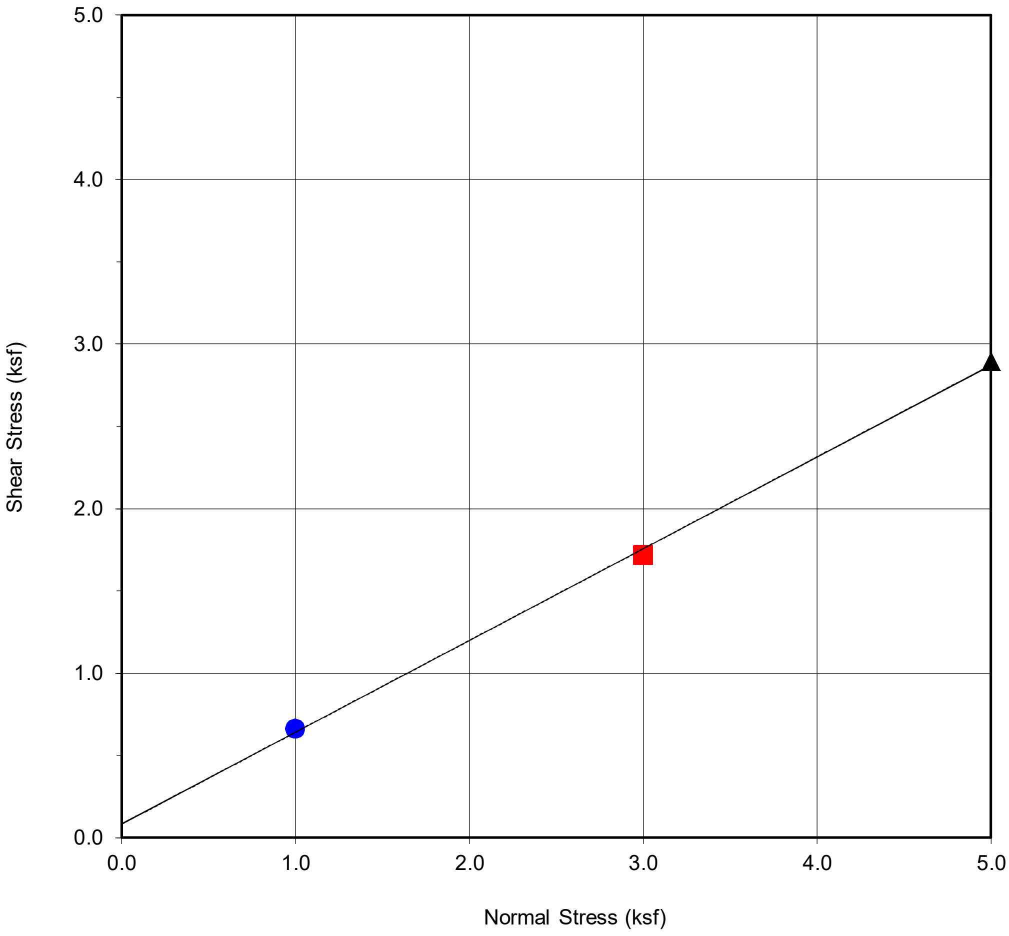
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, compaction, consolidation characteristics, expansive index, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B63. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B5**
Sample No.	**B5@0-5**
Depth (ft)	**0-5**
Sample Type:	Ring

Soil Identification:		
Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	82	29.2
Ultimate	82	29.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.66	■ 1.72	▲ 2.89
Shear Stress @ End of Test (ksf)	○ 0.66	□ 1.72	△ 2.89
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.0	9.2	9.2
Initial Dry Density (pcf)	115.1	114.8	115.0
Initial Degree of Saturation (%)	52.3	53.0	53.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.0	15.6	15.5

GEOCON

DIRECT SHEAR TEST RESULTS

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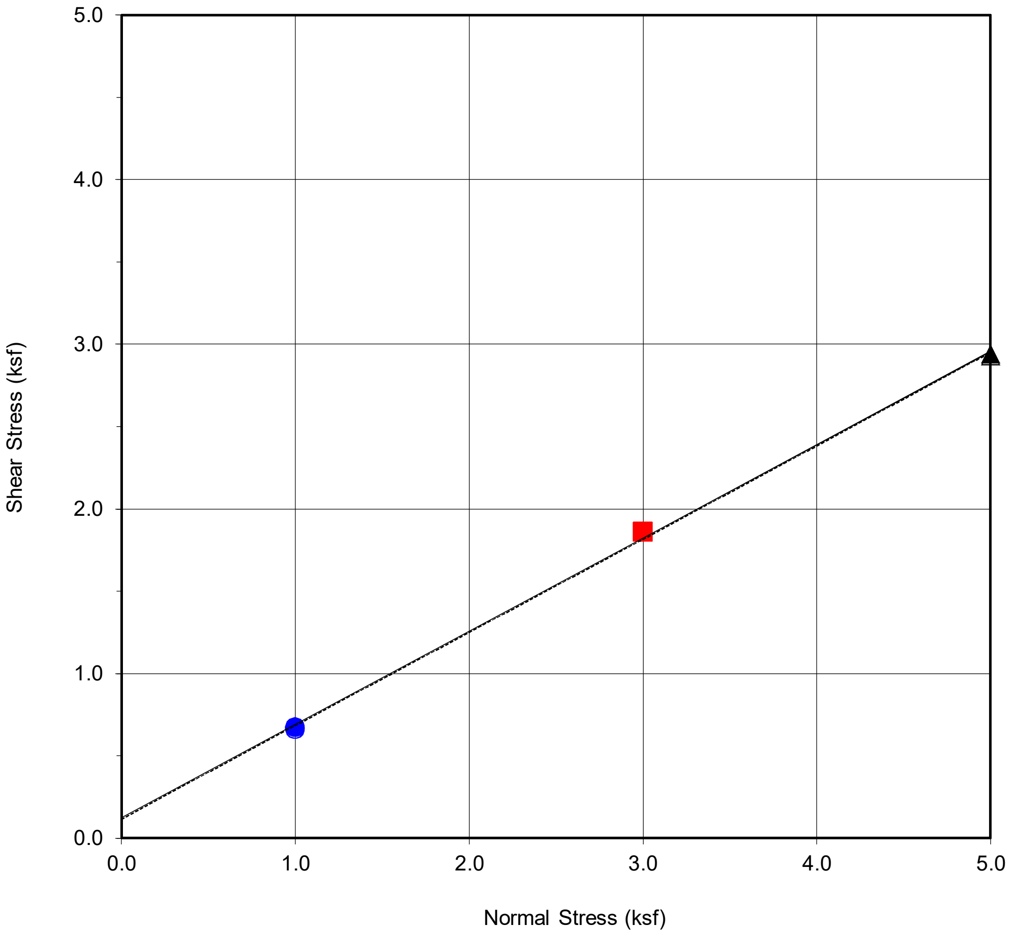
Figure B1



Boring No.	**B8**
Sample No.	**B8@0-5**
Depth (ft)	**0-5**
Sample Type:	Ring

Soil Identification:		
Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	123	29.6
Ultimate	115	29.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.67	■ 1.86	▲ 2.94
Shear Stress @ End of Test (ksf)	○ 0.66	□ 1.86	△ 2.93
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.3	12.2	16.0
Initial Dry Density (pcf)	106.9	105.9	102.7
Initial Degree of Saturation (%)	53.1	55.7	67.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.5	16.3	16.7

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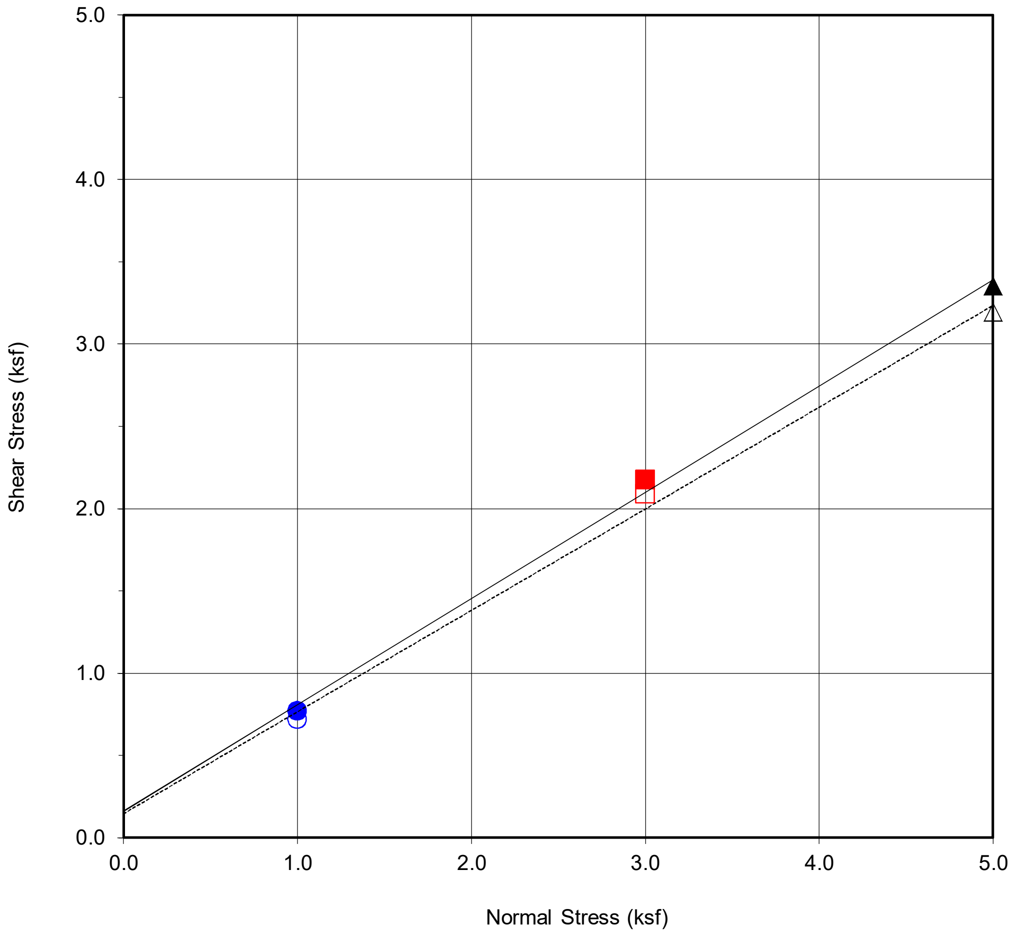
Figure B2



Boring No.	**B4**
Sample No.	**B4@2.5'**
Depth (ft)	**2.5'**
Sample Type:	Ring

Soil Identification:		
Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	161	32.9
Ultimate	146	31.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.77	■ 2.17	▲ 3.35
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.09	△ 3.19
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.5	9.0	9.0
Initial Dry Density (pcf)	96.4	101.5	102.2
Initial Degree of Saturation (%)	44.9	36.6	37.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.5	18.0	17.7

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DIRECT SHEAR TEST RESULTS

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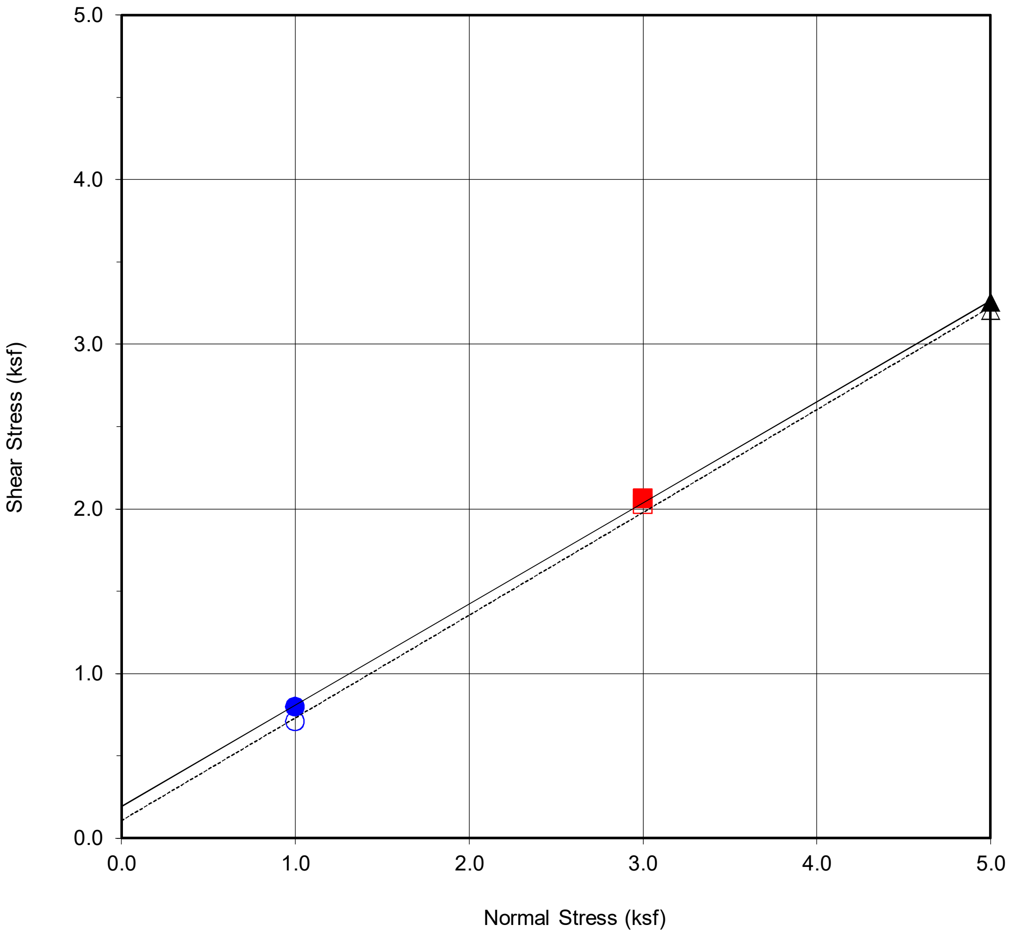
Figure B3



Boring No.	**B1**
Sample No.	**B1@5**
Depth (ft)	**5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.80	■ 2.06	▲ 3.25
Shear Stress @ End of Test (ksf)	○ 0.71	□ 2.02	△ 3.20
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.3	9.9	12.0
Initial Dry Density (pcf)	99.4	98.1	96.3
Initial Degree of Saturation (%)	36.2	37.2	43.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.3	21.0	19.6

Soil Identification:

Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	193	31.6
Ultimate	107	32.0

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DIRECT SHEAR TEST RESULTS

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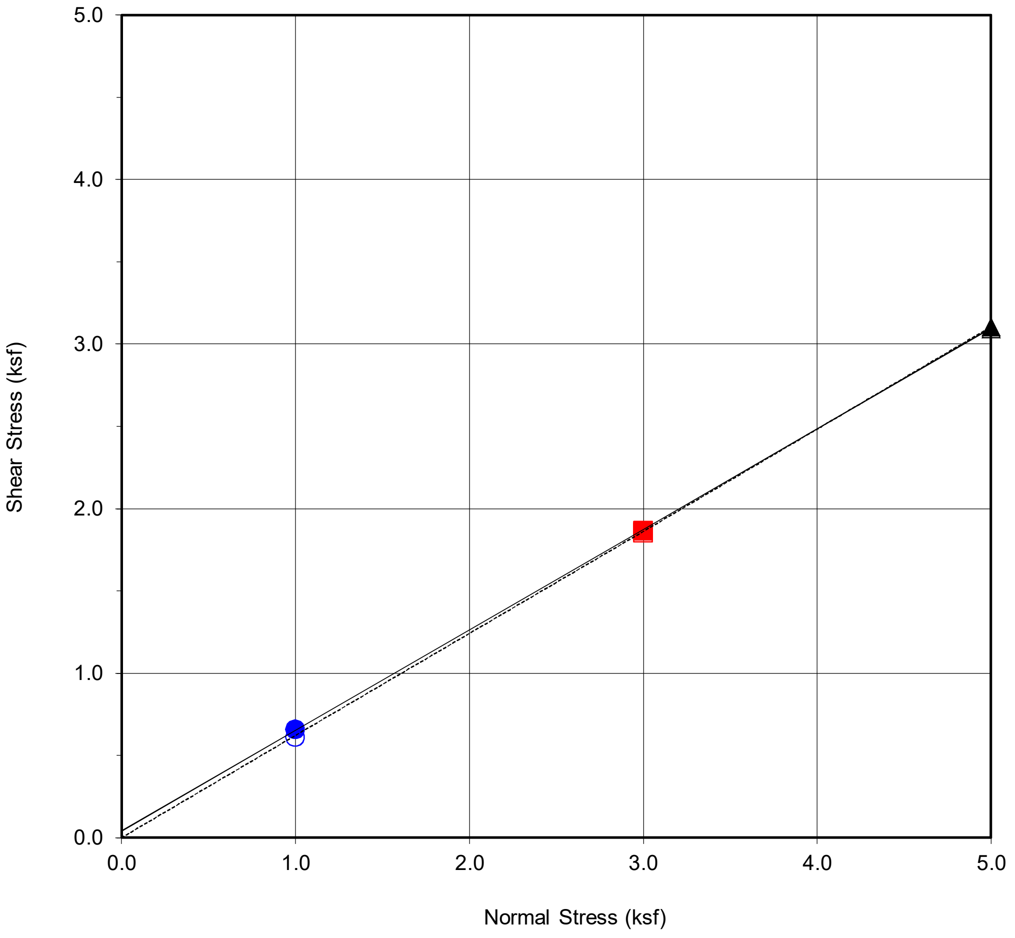
Figure B4



Boring No.	**B3**
Sample No.	**B3@5'**
Depth (ft)	**5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.66	■ 1.86	▲ 3.10
Shear Stress @ End of Test (ksf)	○ 0.61	□ 1.85	△ 3.09
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.4	13.4	14.6
Initial Dry Density (pcf)	94.4	94.2	94.6
Initial Degree of Saturation (%)	42.7	45.9	50.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.9	18.3	17.4

Soil Identification:

Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	40	31.4
Ultimate	0	31.8

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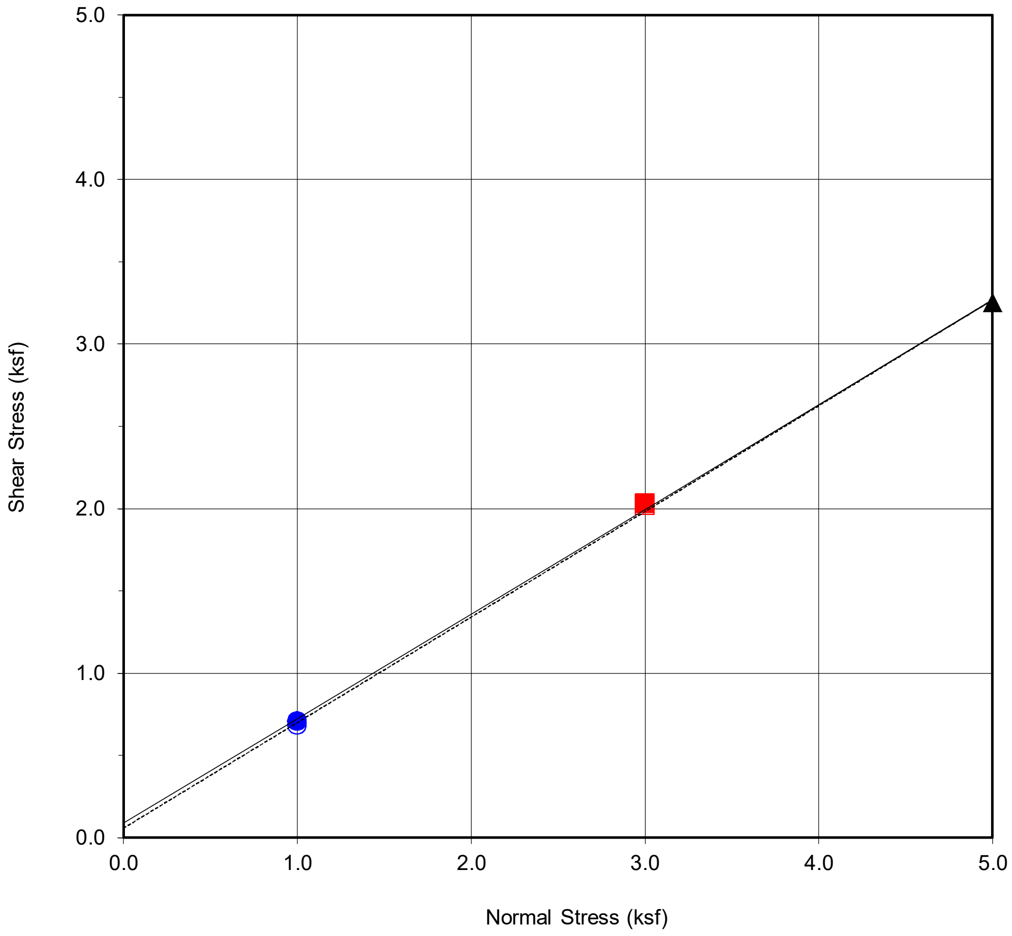
Figure B5



Boring No.	**B6**
Sample No.	**B6@5'**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:		
Dark Yellowish Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	88	32.5
Ultimate	58	32.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.71	■ 2.03	▲ 3.25
Shear Stress @ End of Test (ksf)	○ 0.68	□ 2.02	△ 3.25
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.8	11.4	11.1
Initial Dry Density (pcf)	94.2	92.9	94.0
Initial Degree of Saturation (%)	37.0	37.7	37.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.9	18.8	18.9

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DIRECT SHEAR TEST RESULTS

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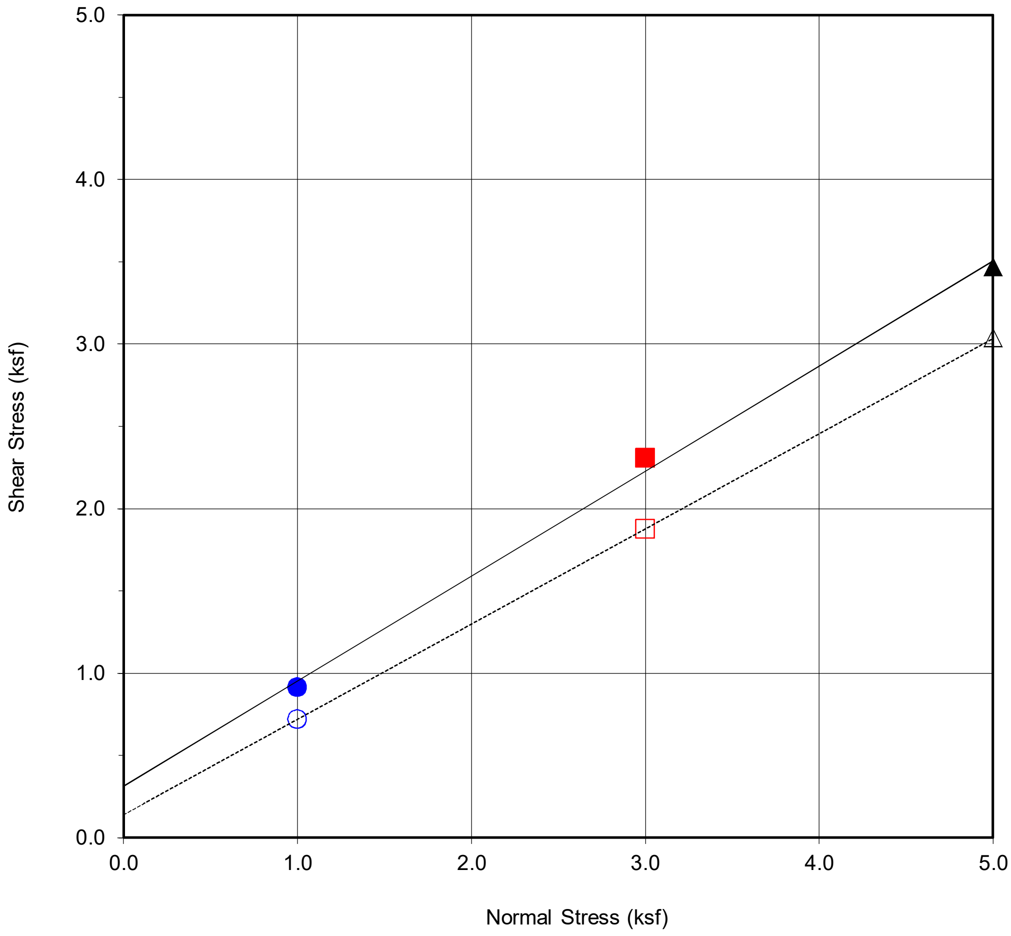
Figure B6



Boring No.	**B5**
Sample No.	**B5@7.5**
Depth (ft)	**7.5**
Sample Type:	Ring

Soil Identification:

Yellowish Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	311	32.6
Ultimate	139	30.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.91	■ 2.30	▲ 3.47
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.87	△ 3.04
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	19.3	25.4	22.4
Initial Dry Density (pcf)	102.8	97.3	98.6
Initial Degree of Saturation (%)	81.5	93.8	85.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.4	18.3	17.5

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DIRECT SHEAR TEST RESULTS

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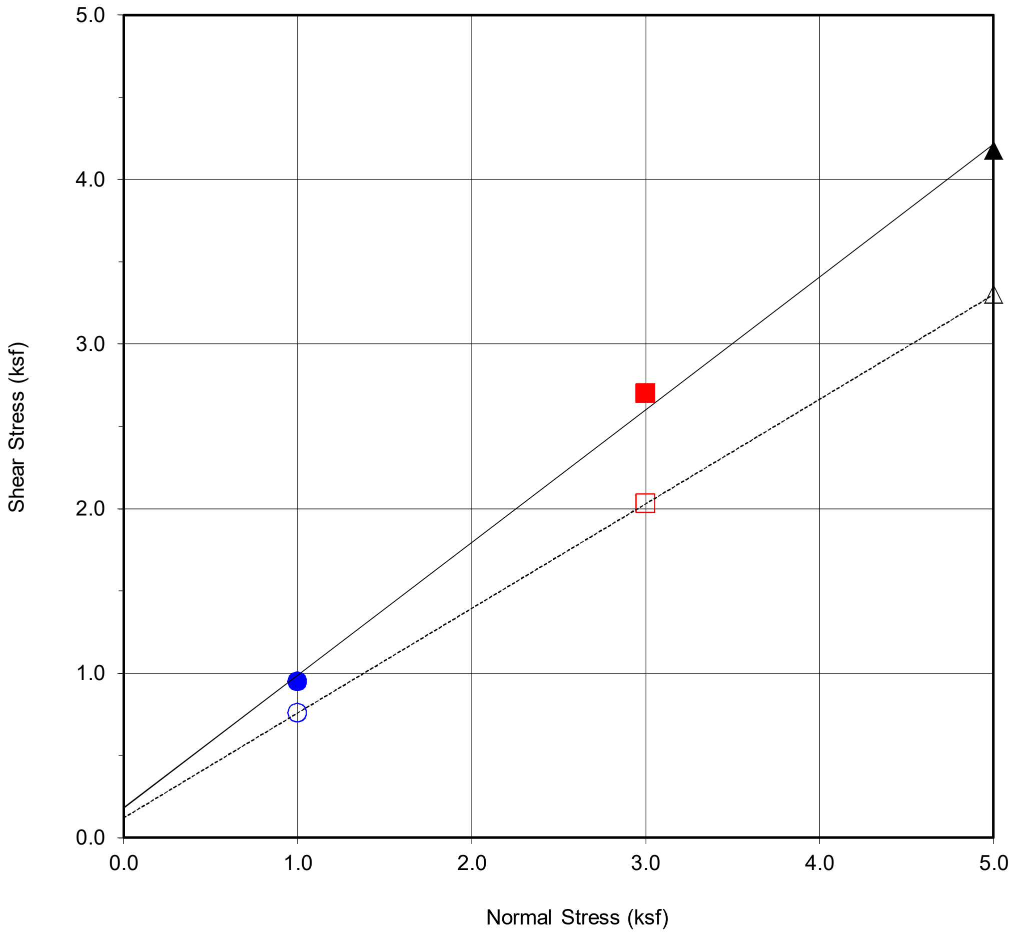
Figure B7



Boring No.	**B7**
Sample No.	**B7@7.5'**
Depth (ft)	**7.5'**
Sample Type:	Ring

Soil Identification:		
Light Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	180	38.9
Ultimate	120	32.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.95	■ 2.70	▲ 4.18
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.03	△ 3.30
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	23.1	20.1	19.1
Initial Dry Density (pcf)	103.8	110.0	113.6
Initial Degree of Saturation (%)	100.1	102.1	106.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.7	18.7	15.9

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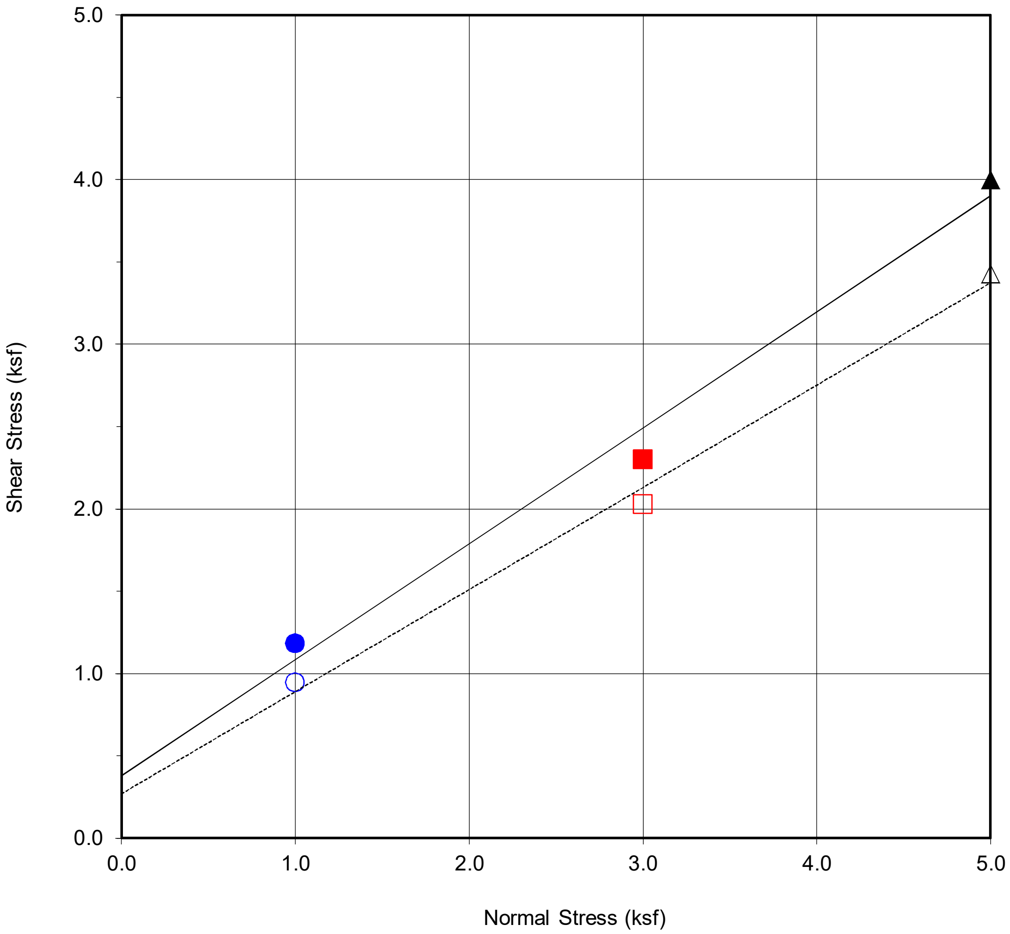
Figure B8



Boring No.	**B1**
Sample No.	**B1@10**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:		
Yellowish Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	378	35.2
Ultimate	269	31.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.18	■ 2.30	▲ 4.00
Shear Stress @ End of Test (ksf)	○ 0.94	□ 2.02	△ 3.43
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	20.4	21.1	20.2
Initial Dry Density (pcf)	107.5	105.0	105.8
Initial Degree of Saturation (%)	97.1	93.9	91.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.5	20.8	19.6

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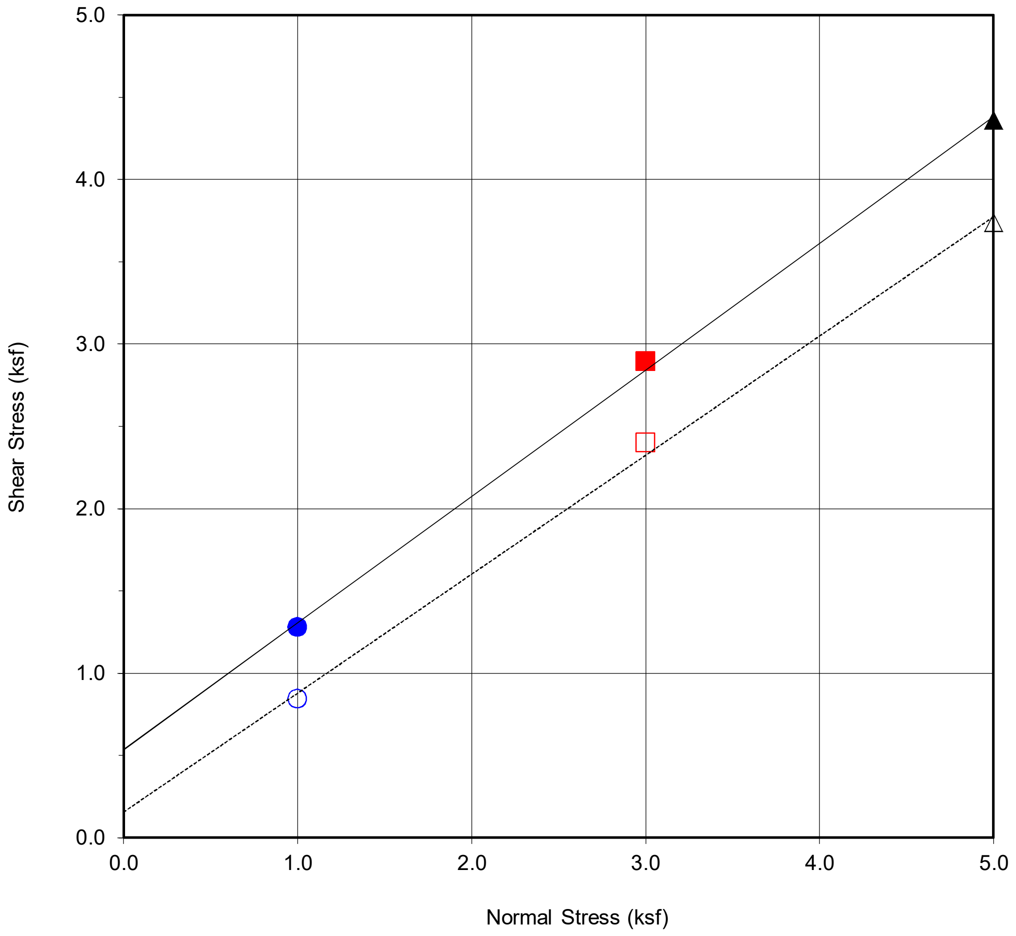
Figure B9



Boring No.	**B6**
Sample No.	**B6@12.5'**
Depth (ft)	**12.5'**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.28	■ 2.89	▲ 4.36
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.40	△ 3.74
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.6	15.4	16.0
Initial Dry Density (pcf)	105.1	104.9	108.7
Initial Degree of Saturation (%)	69.6	68.5	78.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.2	18.1	18.0

Soil Identification:

Dark Yellowish Brown Sand with Clay (SP-SC)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	534	37.6
Ultimate	156	35.9

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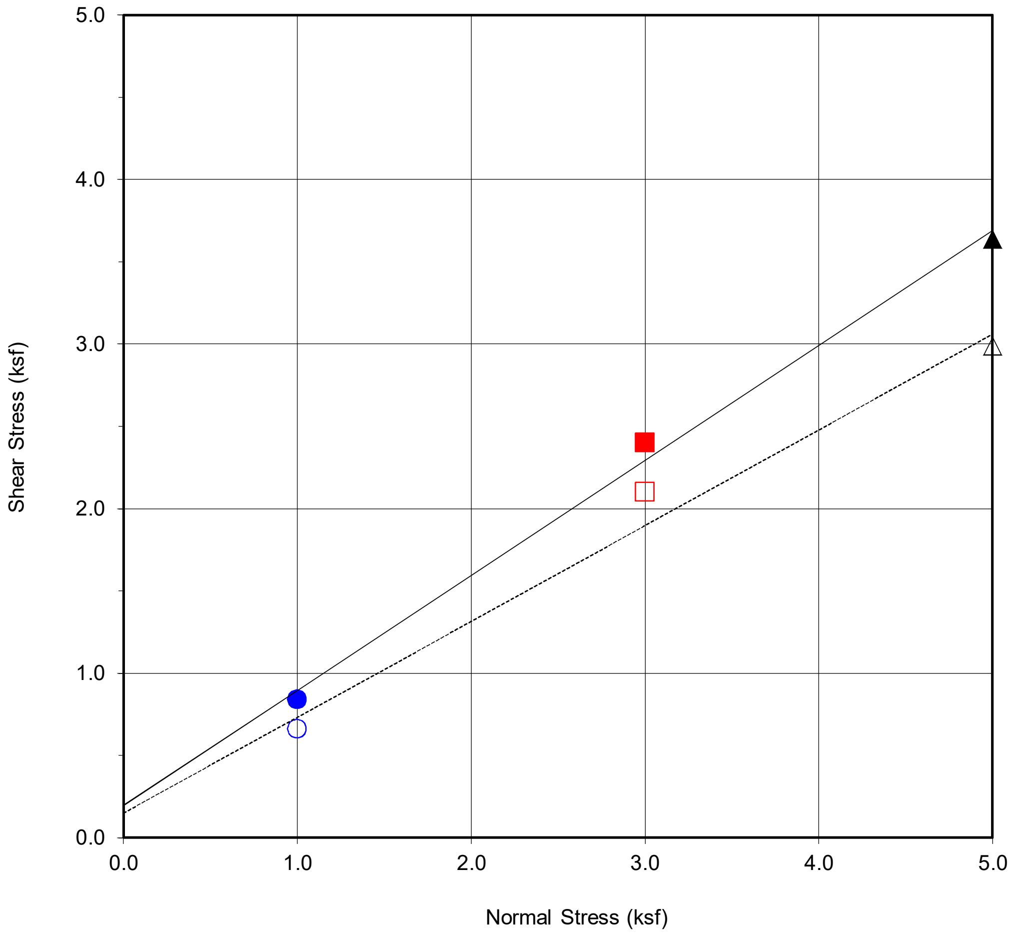
Figure B10



Boring No.	**B4**
Sample No.	**B4@12.5**
Depth (ft)	**12.5**
Sample Type:	Ring

Soil Identification:

Yellowish Brown Sand with Clay (SP-SC)

Strength Parameters

	C (psf)	ϕ (°)
Peak	195	35.0
Ultimate	150	30.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.84	■ 2.40	▲ 3.64
Shear Stress @ End of Test (ksf)	○ 0.66	□ 2.10	△ 2.99
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.7	12.9	12.4
Initial Dry Density (pcf)	103.7	107.8	109.3
Initial Degree of Saturation (%)	67.9	61.7	61.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.8	18.1	17.2

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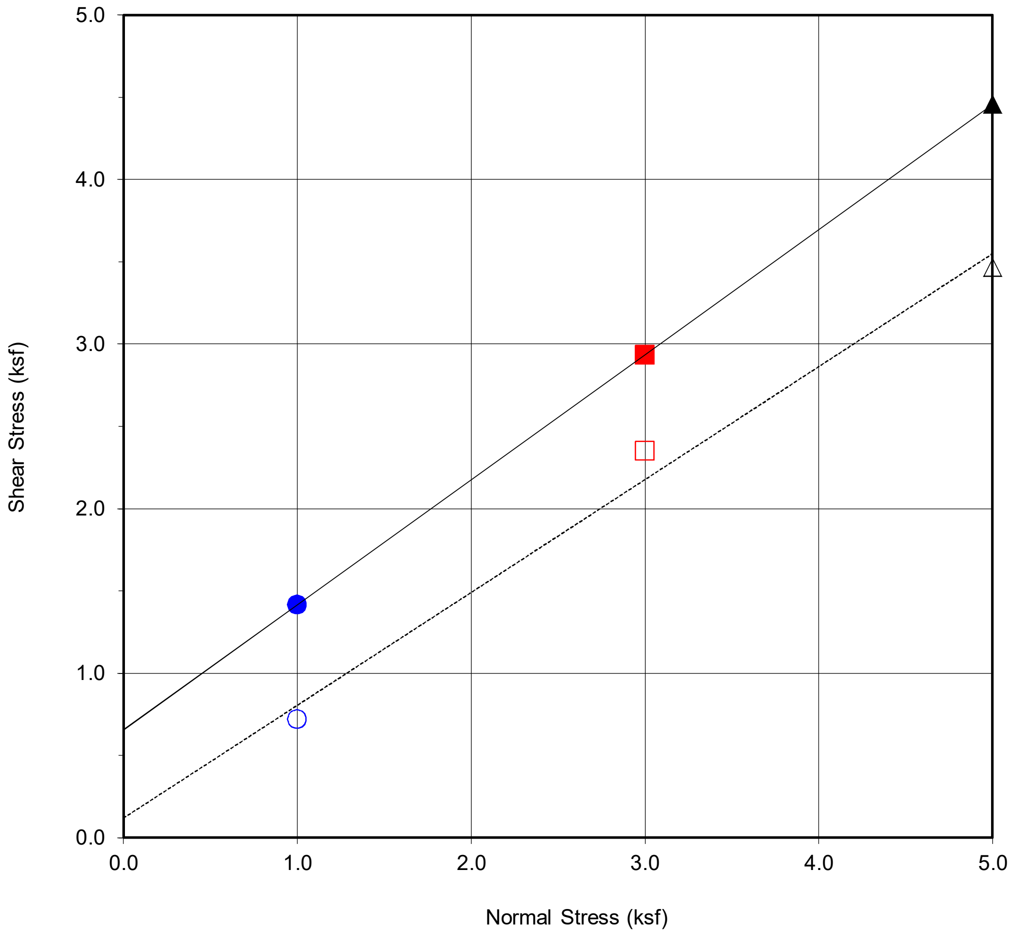
Figure B11



Boring No.	**B1**
Sample No.	**B1@15**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:

Yellowish Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	653	37.3
Ultimate	119	34.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.42	■ 2.93	▲ 4.46
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.35	△ 3.46
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	18.5	18.3	18.6
Initial Dry Density (pcf)	102.9	106.5	107.3
Initial Degree of Saturation (%)	78.3	84.8	87.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.8	19.7	18.1

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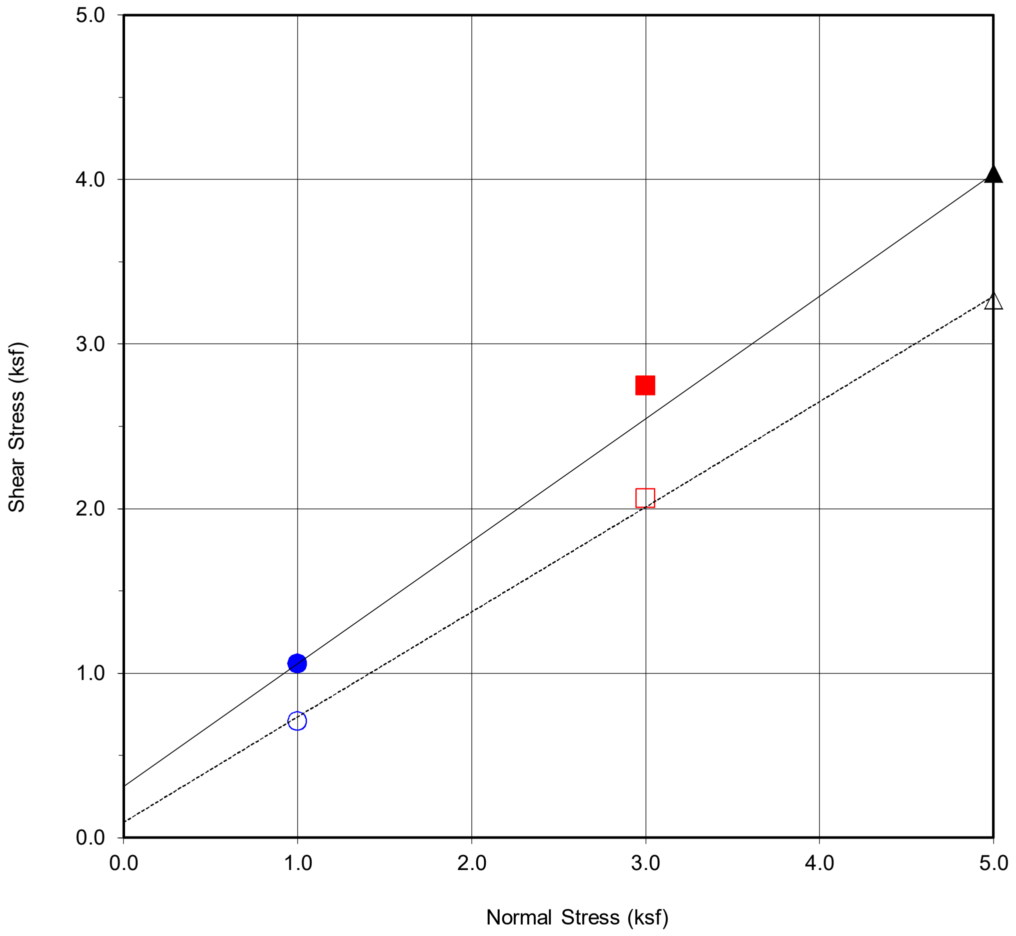
Figure B12



Boring No.	**B3**
Sample No.	**B3@15**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:

Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	310	36.7
Ultimate	93	32.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.06	■ 2.74	▲ 4.04
Shear Stress @ End of Test (ksf)	○ 0.71	□ 2.06	△ 3.27
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	18.0	18.1	17.0
Initial Dry Density (pcf)	103.4	102.7	100.4
Initial Degree of Saturation (%)	77.2	76.2	67.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.2	18.9	19.7

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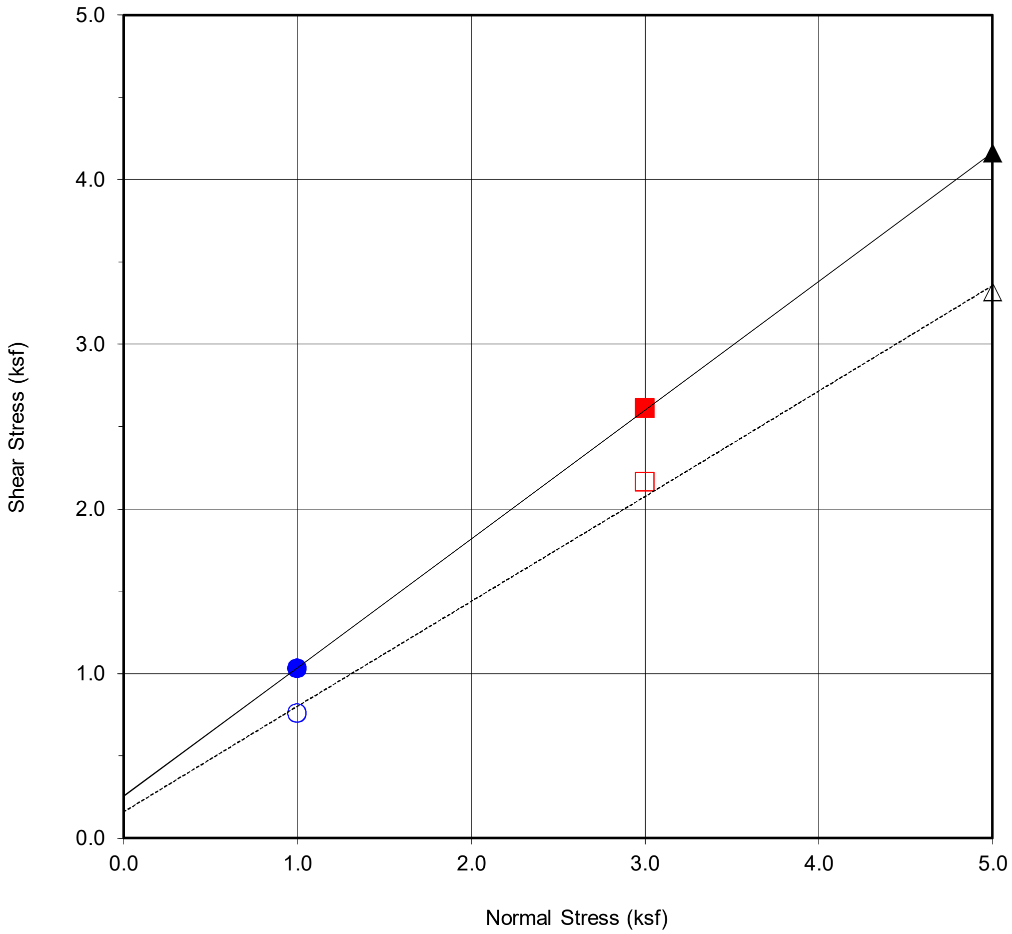
Figure B13



Boring No.	**B1**
Sample No.	**B1@25**
Depth (ft)	**25**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.03	■ 2.61	▲ 4.16
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.16	△ 3.32
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.3	9.8	9.9
Initial Dry Density (pcf)	98.5	95.2	98.8
Initial Degree of Saturation (%)	39.1	34.4	37.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.3	28.1	22.6

Soil Identification:

Light Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	253	38.0
Ultimate	159	32.6

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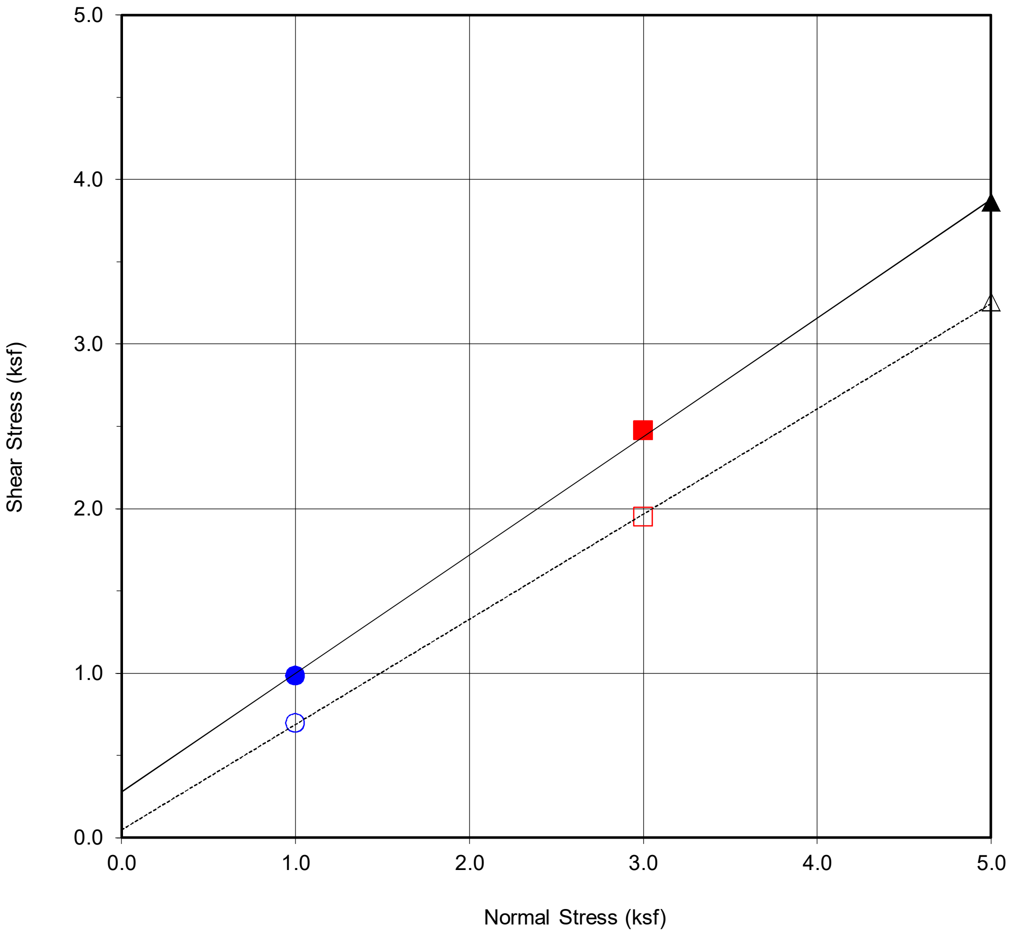
Figure B14



Boring No.	**B3**
Sample No.	**B3@25'**
Depth (ft)	**25**
Sample Type:	Ring

Soil Identification:

Light Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	277	35.8
Ultimate	47	32.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	■ 2.47	▲ 3.86
Shear Stress @ End of Test (ksf)	○ 0.70	□ 1.95	△ 3.25
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.9	10.6	11.5
Initial Dry Density (pcf)	93.4	93.8	92.7
Initial Degree of Saturation (%)	30.0	35.9	37.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	24.9	25.1	25.1

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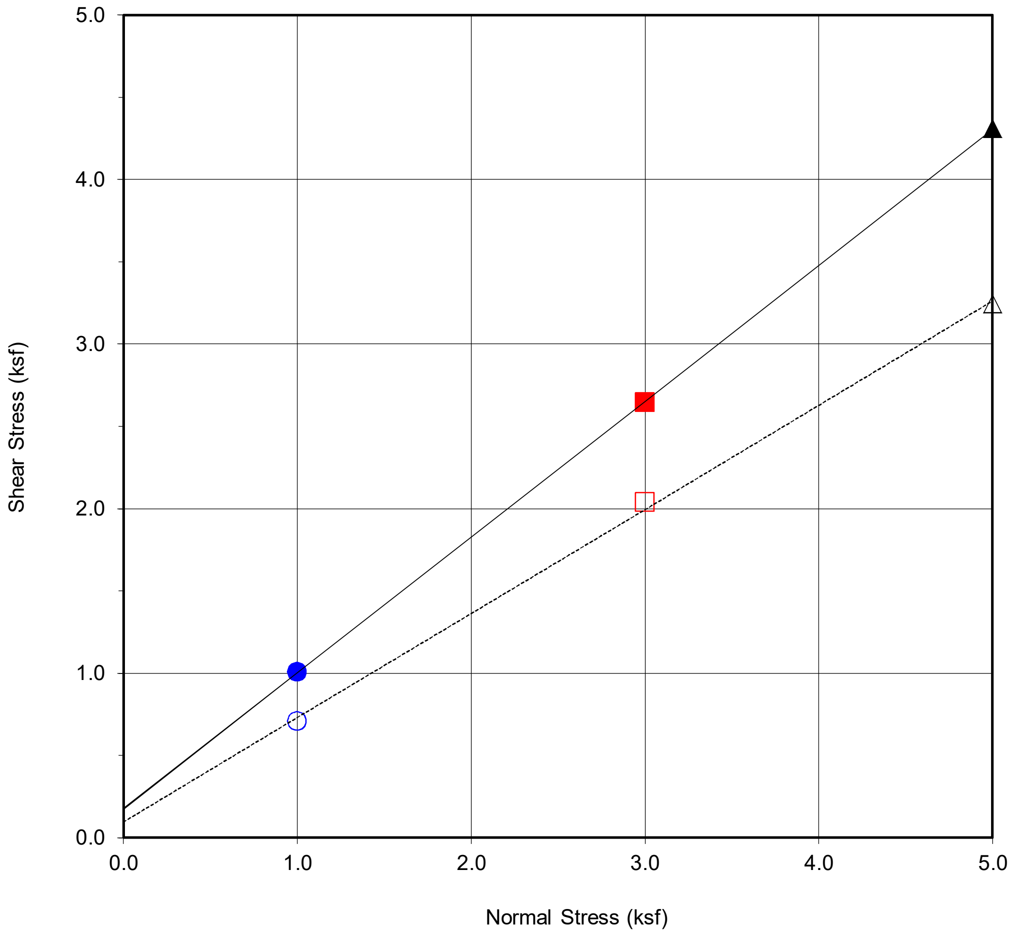
Figure B15



Boring No.	**B2**
Sample No.	**B2@32.5'**
Depth (ft)	**32.5**
Sample Type:	Ring

Soil Identification:		
Light Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	176	39.5
Ultimate	95	32.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.01	■ 2.65	▲ 4.31
Shear Stress @ End of Test (ksf)	○ 0.71	□ 2.04	△ 3.24
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	5.3	6.2	5.5
Initial Dry Density (pcf)	101.8	101.3	102.1
Initial Degree of Saturation (%)	21.7	25.2	22.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.1	18.7	18.4

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DIRECT SHEAR TEST RESULTS

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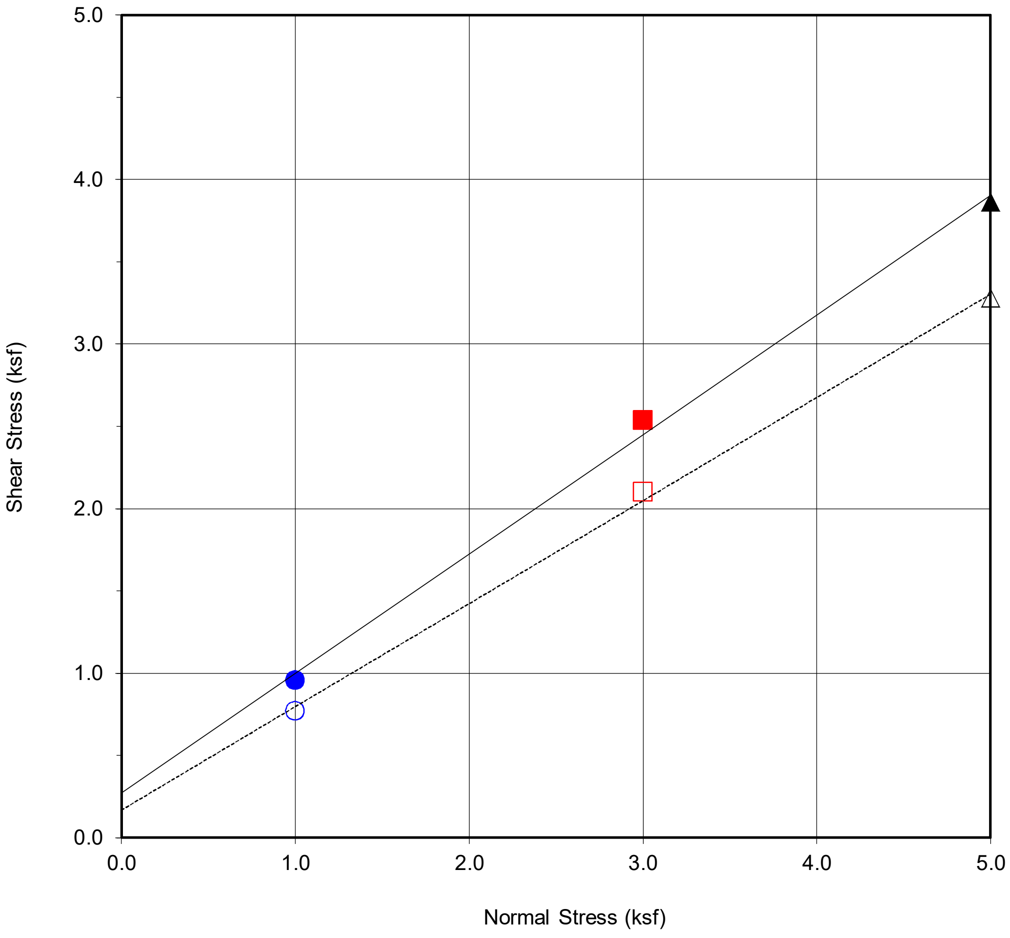
Figure B16



Boring No.	**B1**
Sample No.	**B1@35**
Depth (ft)	**35**
Sample Type:	Ring

Soil Identification:

Light Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	271	36.0
Ultimate	168	32.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.96	■ 2.53	▲ 3.86
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.10	△ 3.28
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	19.1	17.0	12.7
Initial Dry Density (pcf)	85.3	86.0	91.2
Initial Degree of Saturation (%)	52.7	47.7	40.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	29.2	27.8	26.2

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DIRECT SHEAR TEST RESULTS

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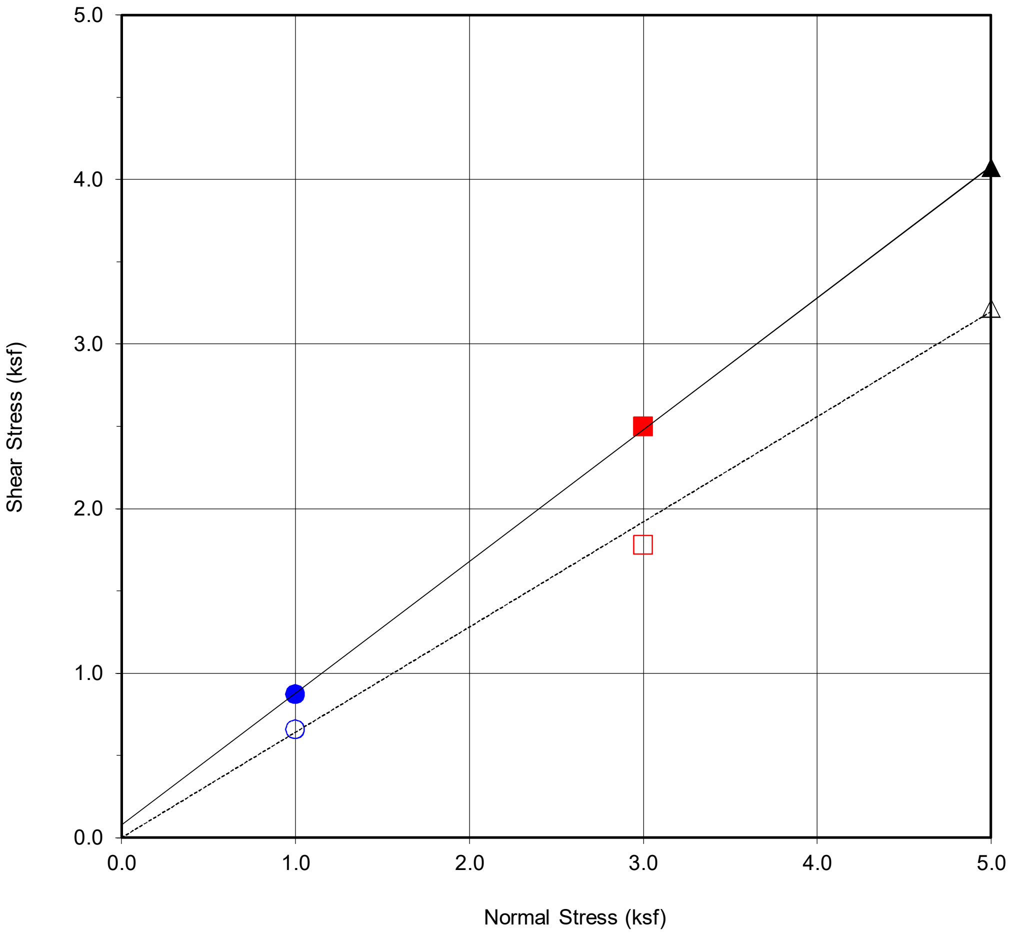
Figure B17



Boring No.	**B3**
Sample No.	**B3@35**
Depth (ft)	**35**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.87	🟥 2.50	▲ 4.07
Shear Stress @ End of Test (ksf)	○ 0.66	□ 1.78	△ 3.22
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.7	8.2	9.2
Initial Dry Density (pcf)	94.4	96.4	99.1
Initial Degree of Saturation (%)	29.7	29.7	35.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.7	30.3	20.1

Soil Identification:

Light Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	76	38.7
Ultimate	0	32.6

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DIRECT SHEAR TEST RESULTS

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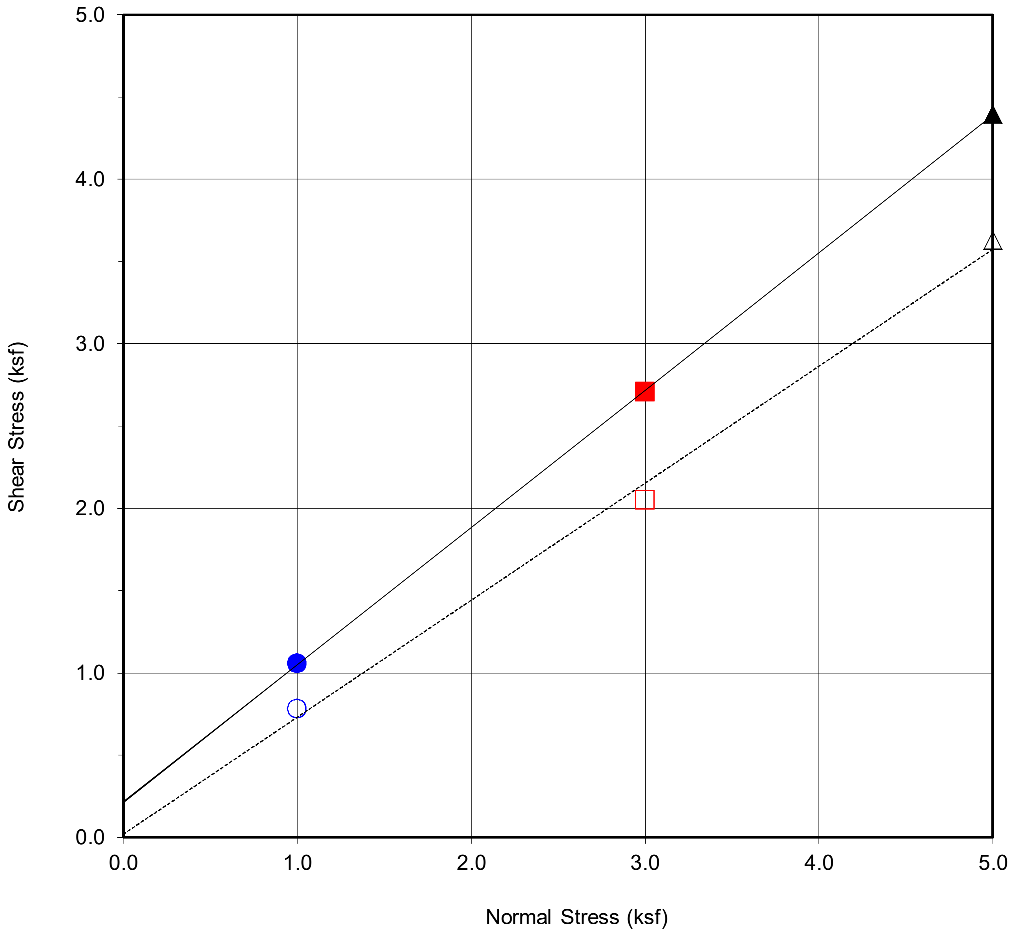
Figure B18



Boring No.	**B1**
Sample No.	**B1@40**
Depth (ft)	**40**
Sample Type:	Ring

Soil Identification:		
Light Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	214	39.9
Ultimate	19	35.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.06	■ 2.71	▲ 4.40
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.05	△ 3.63
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.2	10.1	10.5
Initial Dry Density (pcf)	94.9	92.8	93.1
Initial Degree of Saturation (%)	28.5	33.5	34.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	25.7	25.2	25.4

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DIRECT SHEAR TEST RESULTS

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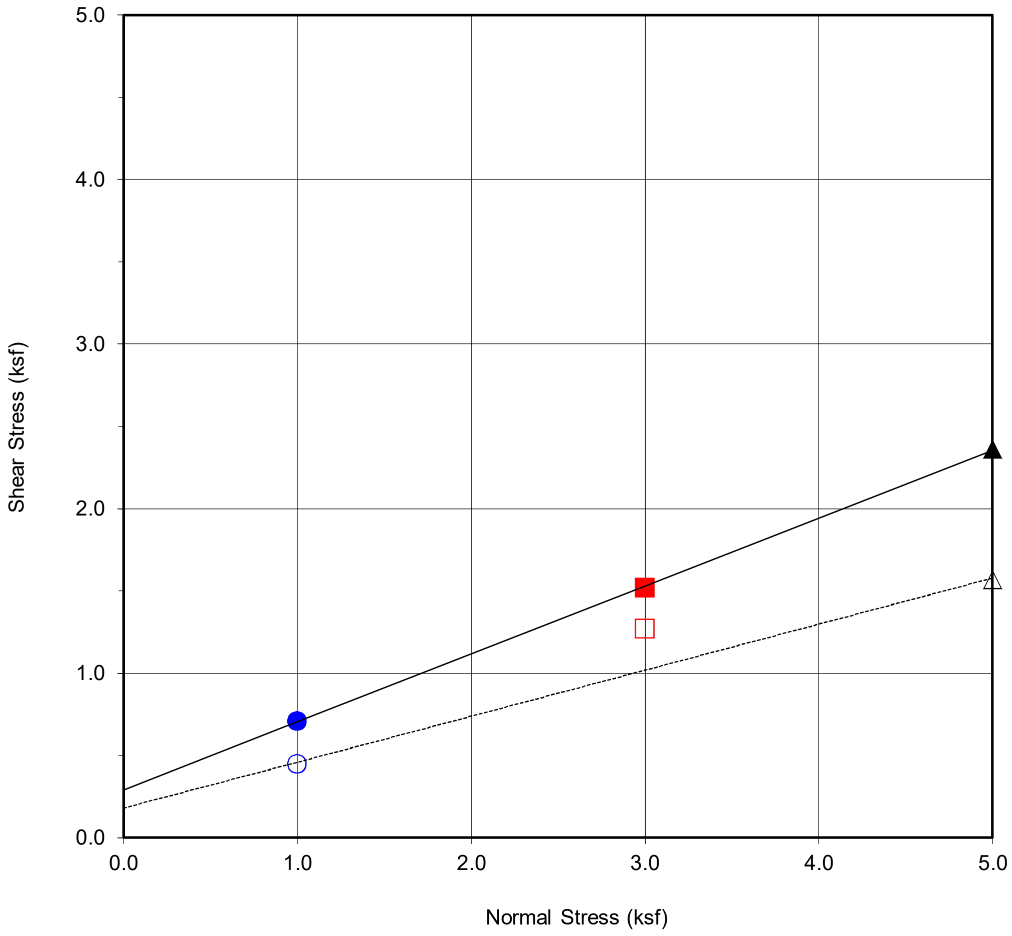
Figure B19



Boring No.	**B3**
Sample No.	**B3@40**
Depth (ft)	**40**
Sample Type:	Ring

Soil Identification:		
Light Brown Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	289	22.4
Ultimate	180	15.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.71	■ 1.52	▲ 2.36
Shear Stress @ End of Test (ksf)	○ 0.45	□ 1.27	△ 1.57
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	52.6	53.2	55.1
Initial Dry Density (pcf)	70.3	70.2	68.4
Initial Degree of Saturation (%)	101.5	102.4	101.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	55.3	53.5	53.9

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DIRECT SHEAR TEST RESULTS

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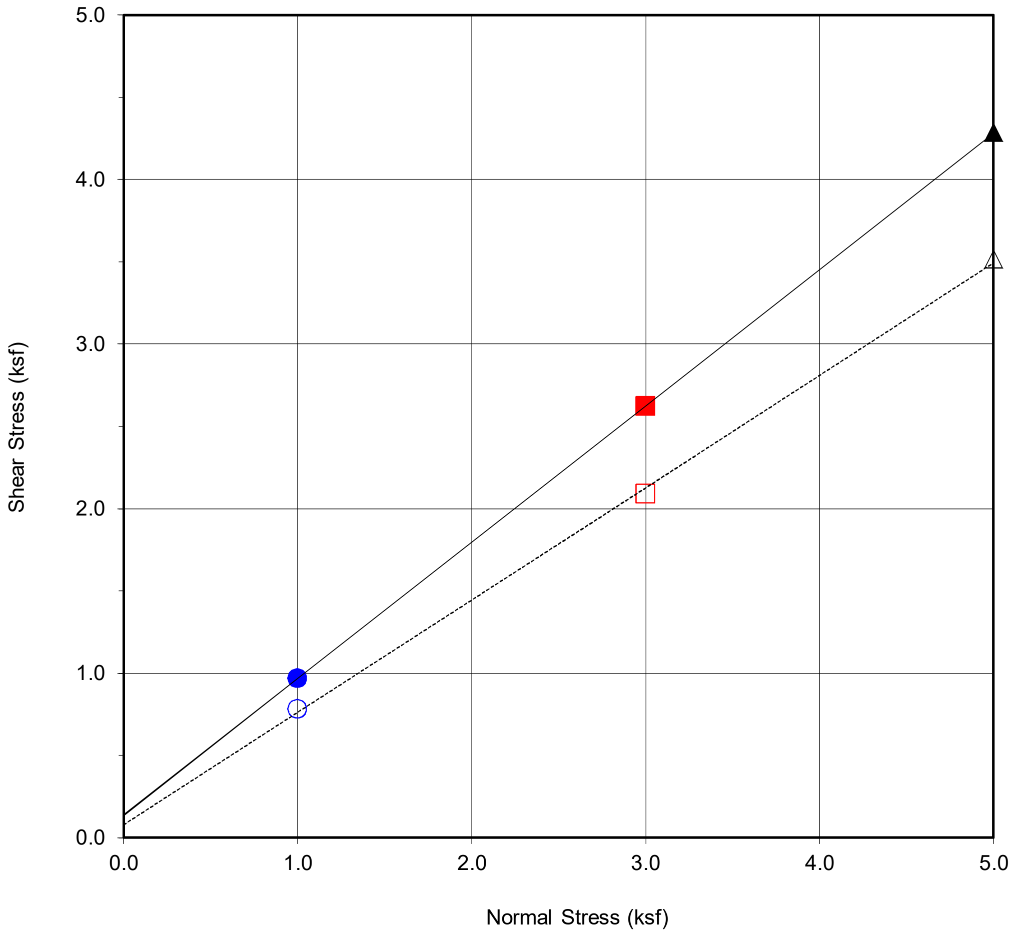
Figure B20



Boring No.	**B1**
Sample No.	**B1@45**
Depth (ft)	**45**
Sample Type:	Ring

Soil Identification:

Light Brown Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	138	39.7
Ultimate	78	34.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 2.62	▲ 4.28
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.09	△ 3.51
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.7	8.4	16.6
Initial Dry Density (pcf)	92.8	94.8	90.8
Initial Degree of Saturation (%)	45.5	29.3	52.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	25.8	24.1	24.6

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DIRECT SHEAR TEST RESULTS

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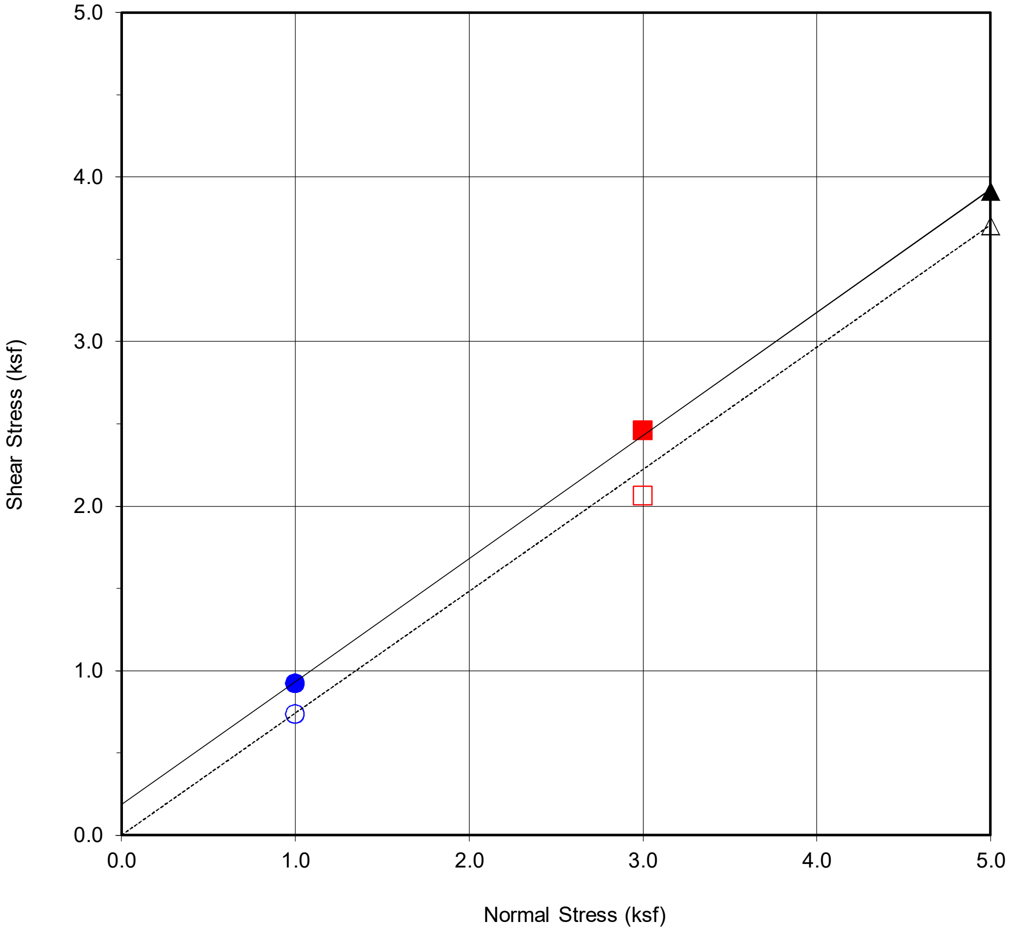
Figure B21



Boring No.	**B3**
Sample No.	**B3@50**
Depth (ft)	**50**
Sample Type:	Ring

Soil Identification:		
Light Brown Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	186	36.8
Ultimate	0	36.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.92	■ 2.46	▲ 3.91
Shear Stress @ End of Test (ksf)	○ 0.73	□ 2.06	△ 3.70
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.1	8.7	10.1
Initial Dry Density (pcf)	85.7	88.1	85.8
Initial Degree of Saturation (%)	25.3	25.7	28.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	28.8	27.8	27.9

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DIRECT SHEAR TEST RESULTS

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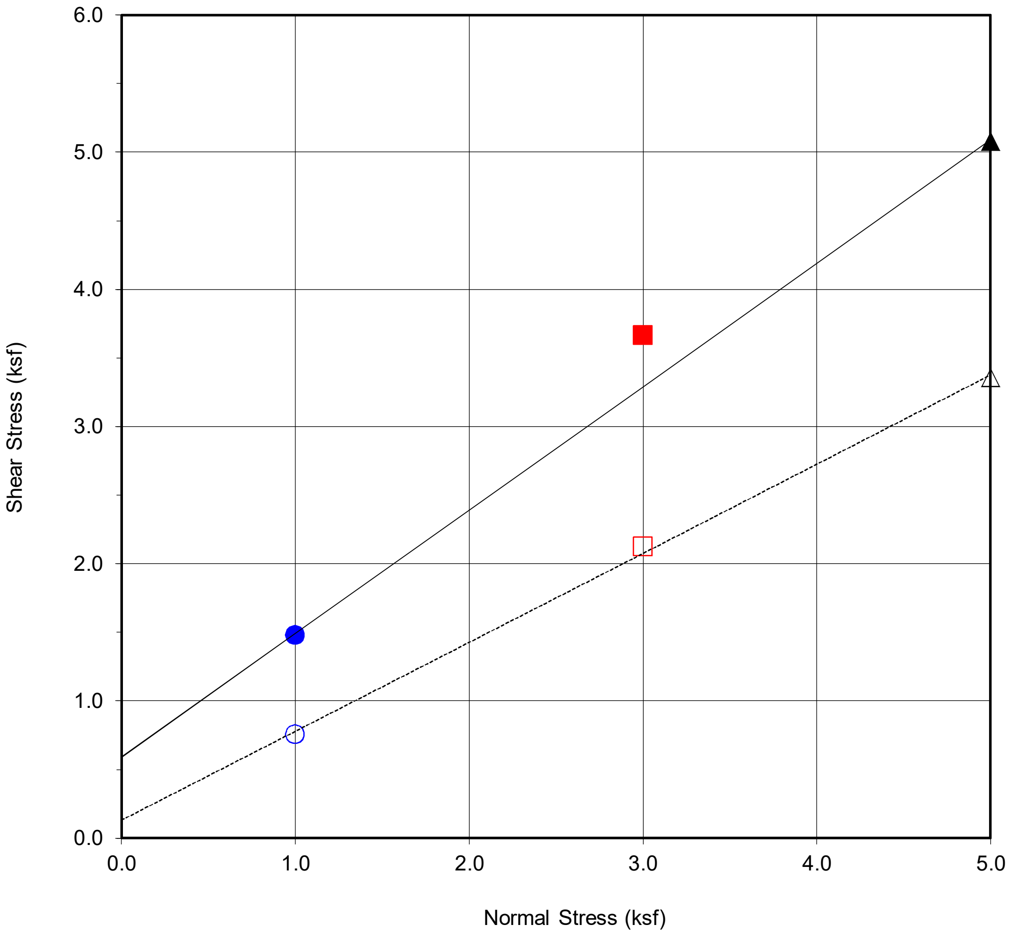
Figure B22



Boring No.	**B1**
Sample No.	**B1@55**
Depth (ft)	**55**
Sample Type:	Ring

Soil Identification:

Grayish Brown Clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	590	42.0
Ultimate	131	33.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.48	■ 3.66	▲ 5.08
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.12	△ 3.35
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.1	13.3	13.1
Initial Dry Density (pcf)	117.1	121.0	118.9
Initial Degree of Saturation (%)	68.0	91.2	84.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.0	15.0	15.3

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DIRECT SHEAR TEST RESULTS

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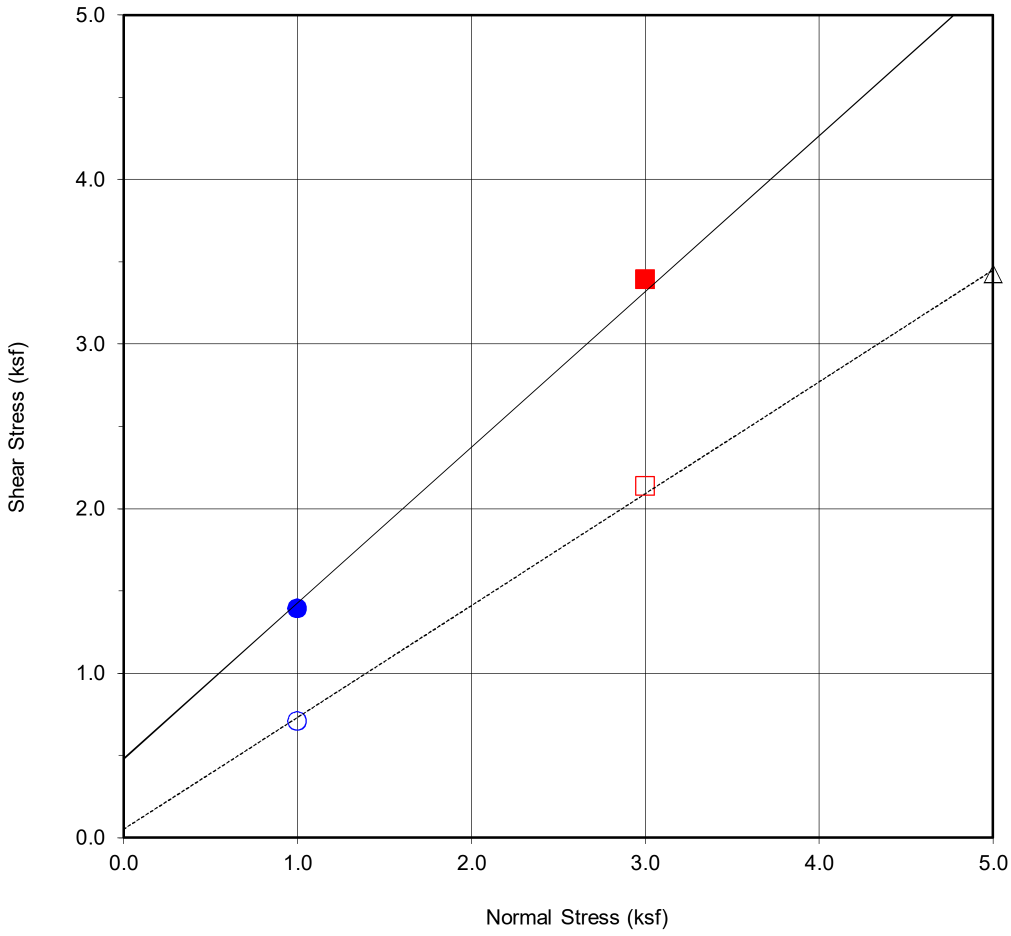
Figure B23



Boring No.	**B3**
Sample No.	**B3@65**
Depth (ft)	**65**
Sample Type:	Ring

Soil Identification:

Dark Brown Sand (SP)

Strength Parameters

	C (psf)	φ (°)
Peak	479	43.4
Ultimate	51	34.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.39	■ 3.39	▲ 5.18
Shear Stress @ End of Test (ksf)	○ 0.71	□ 2.14	△ 3.43
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.2	12.0	12.2
Initial Dry Density (pcf)	119.8	120.7	122.0
Initial Degree of Saturation (%)	80.6	81.4	86.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	-0.8	12.9	12.6

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DIRECT SHEAR TEST RESULTS

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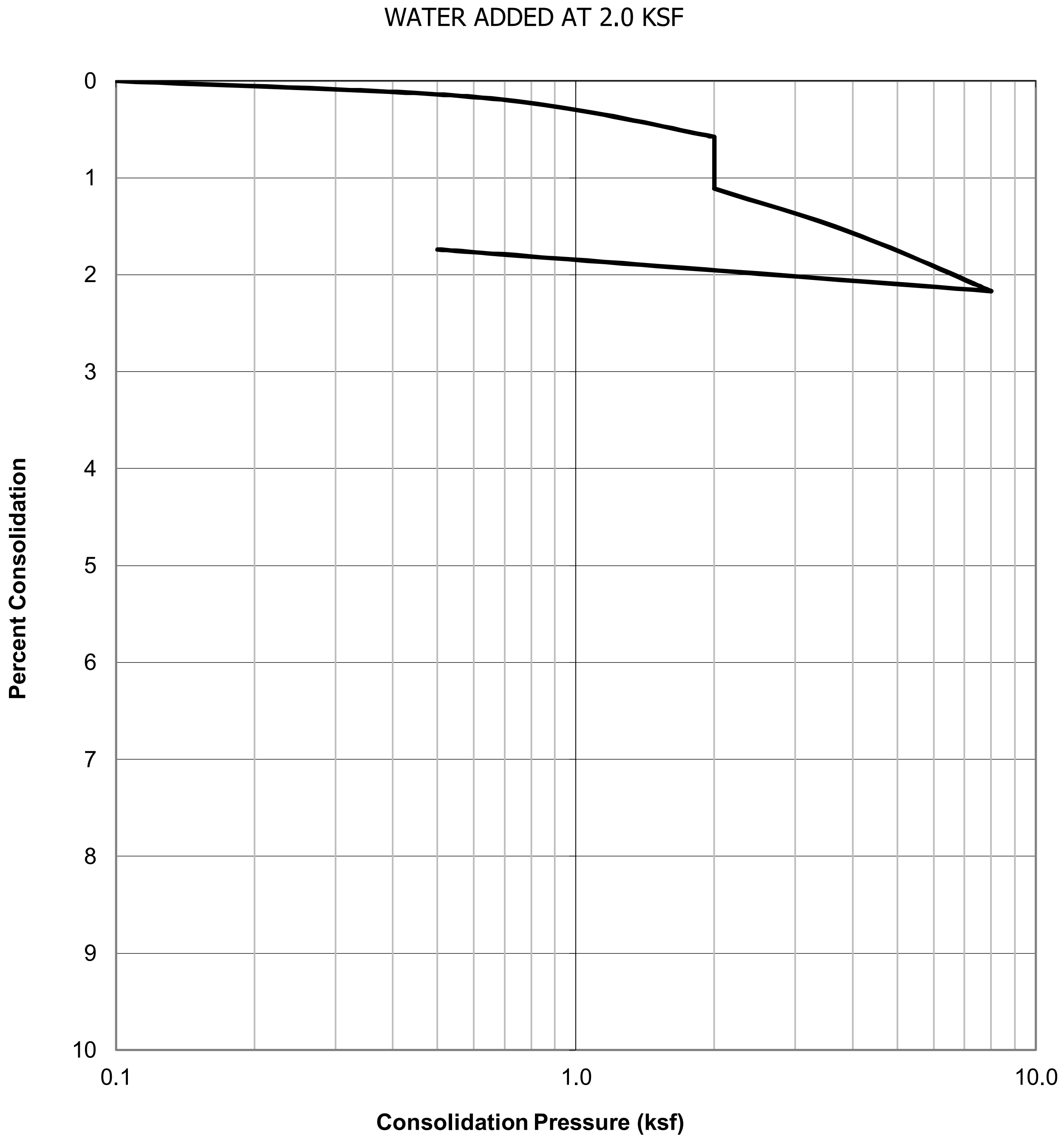
Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@5	Brown Sand with Silt (SP-SM)	106.2	11.8	16.8

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CONSOLIDATION TEST RESULTS

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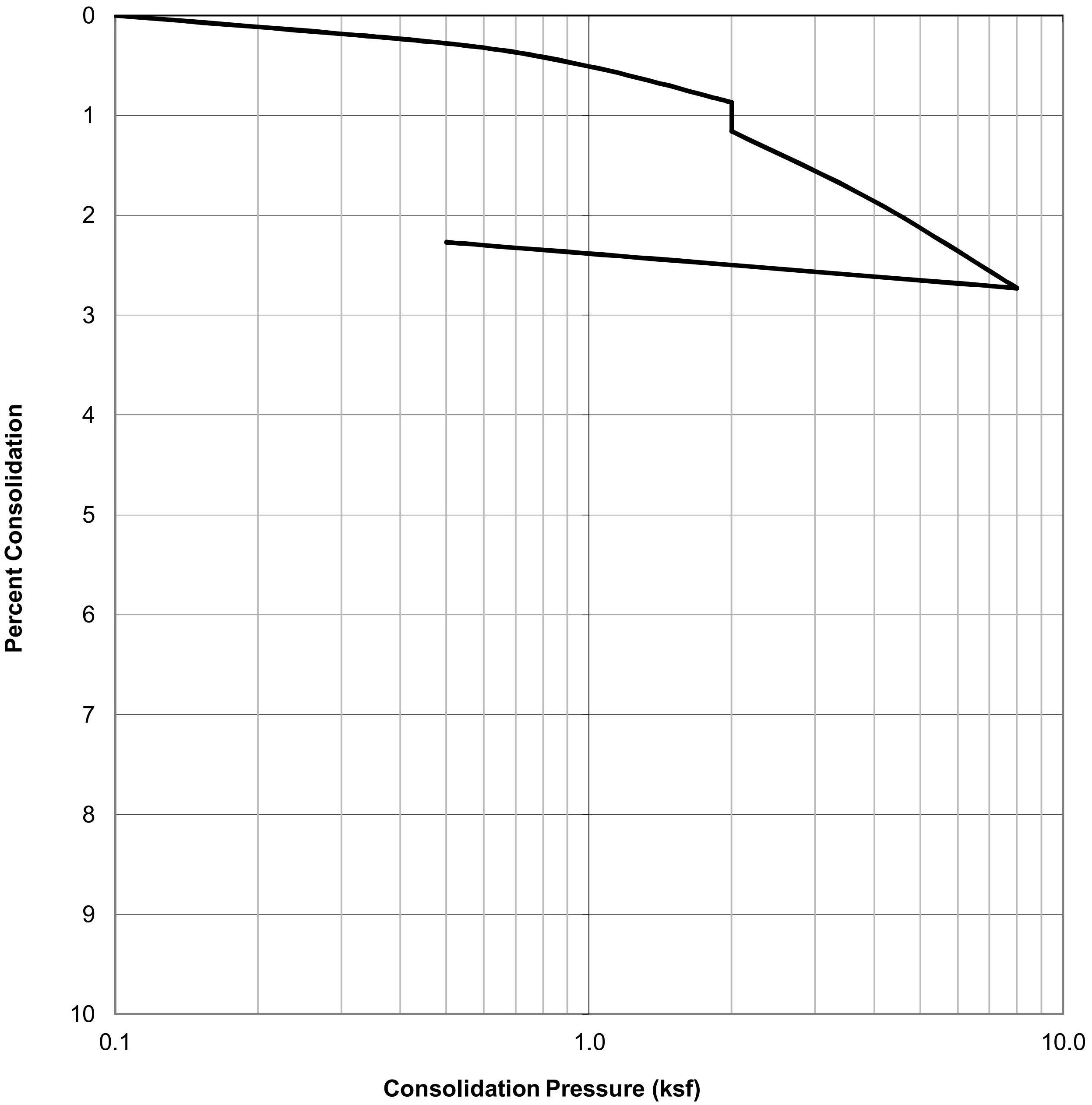
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@5	Dark Yellowish Brown Sand (SP)	96.3	7.6	12.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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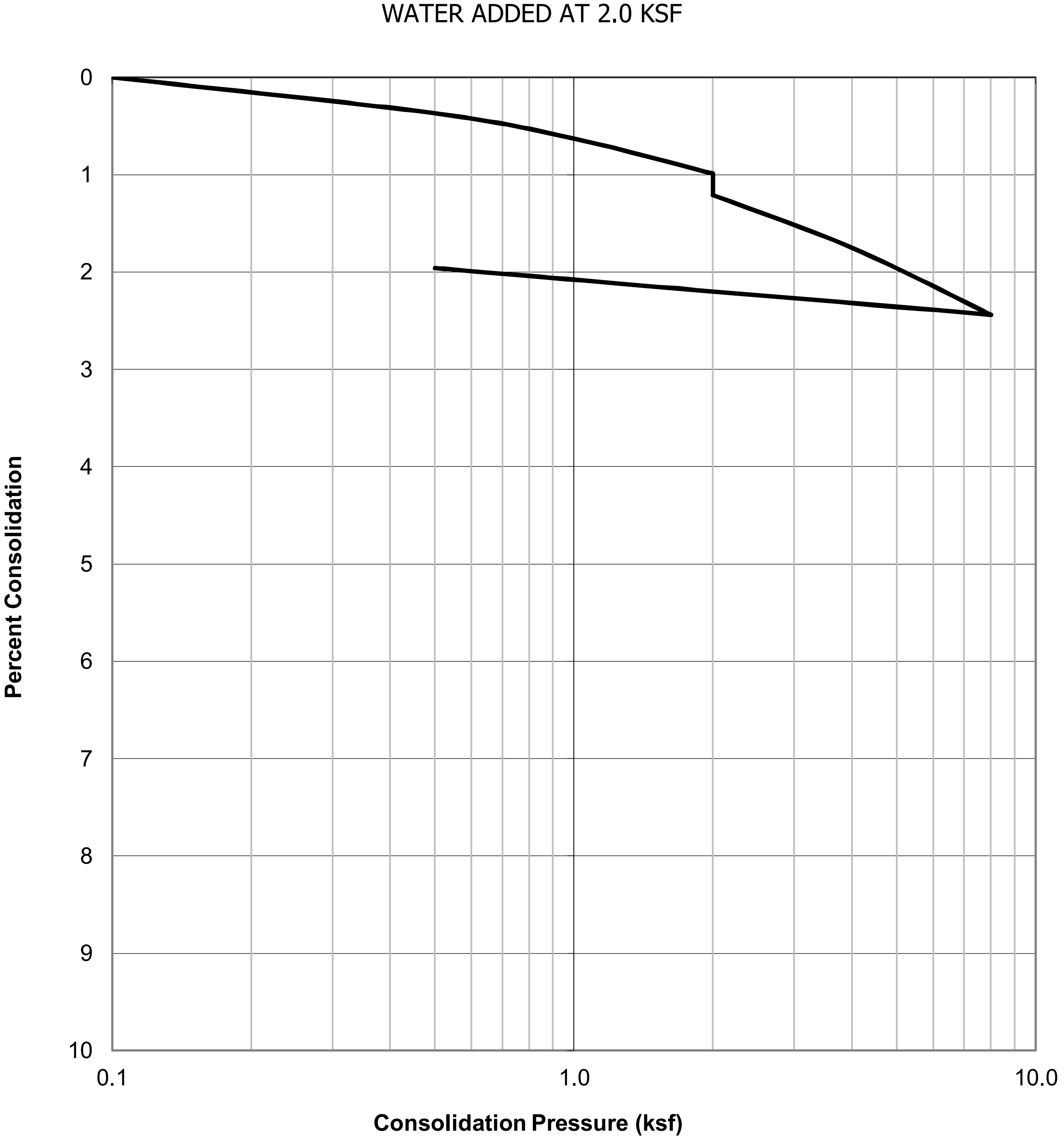
Figure B26



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@5	Yellowish Brown Sand (SP)	102.8	10.2	16.7

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ASTM D-2435

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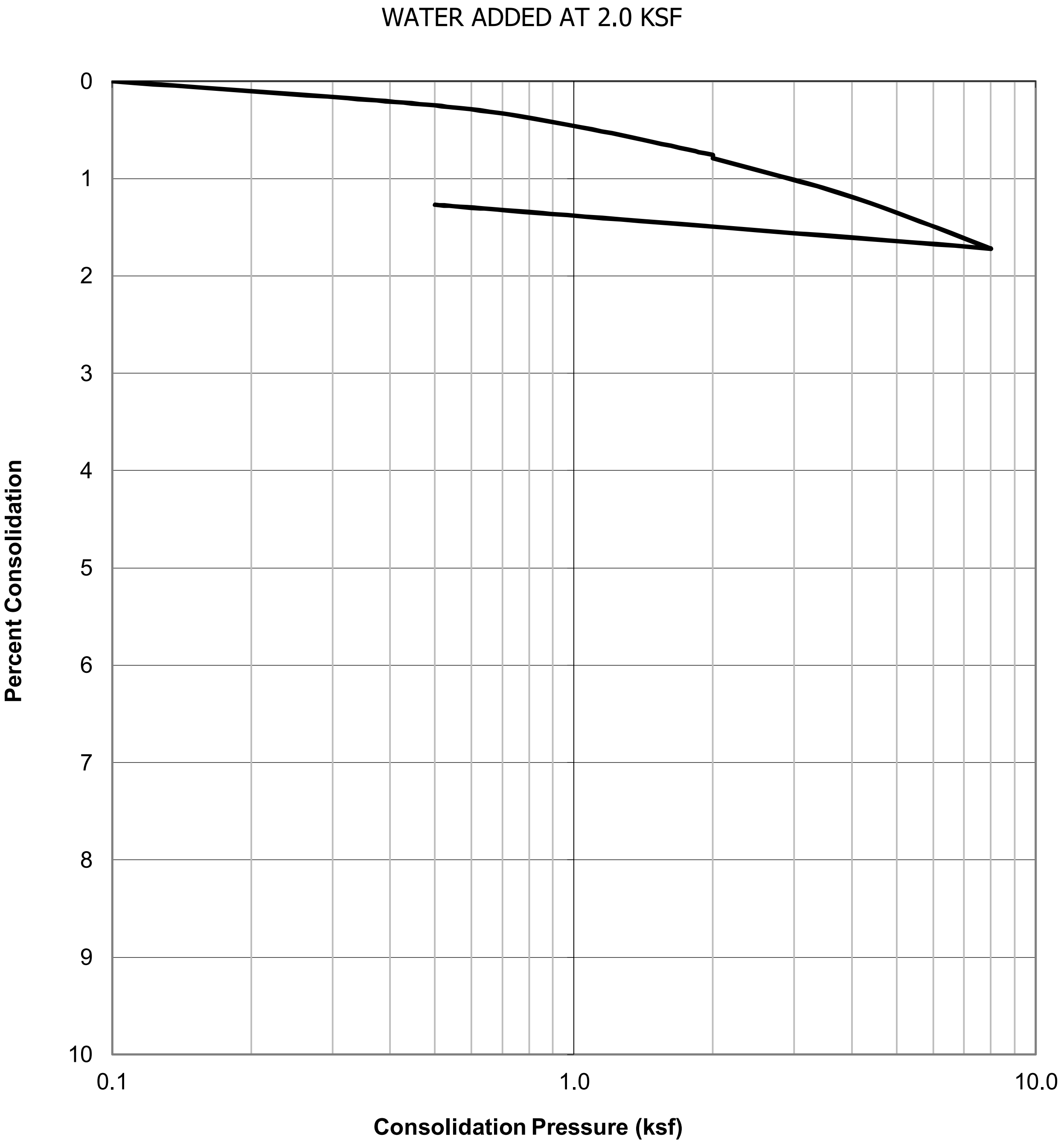
Figure B27



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@7.5	Yellowish Brown Sand (SP)	106.1	21.5	17.6

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CONSOLIDATION TEST RESULTS

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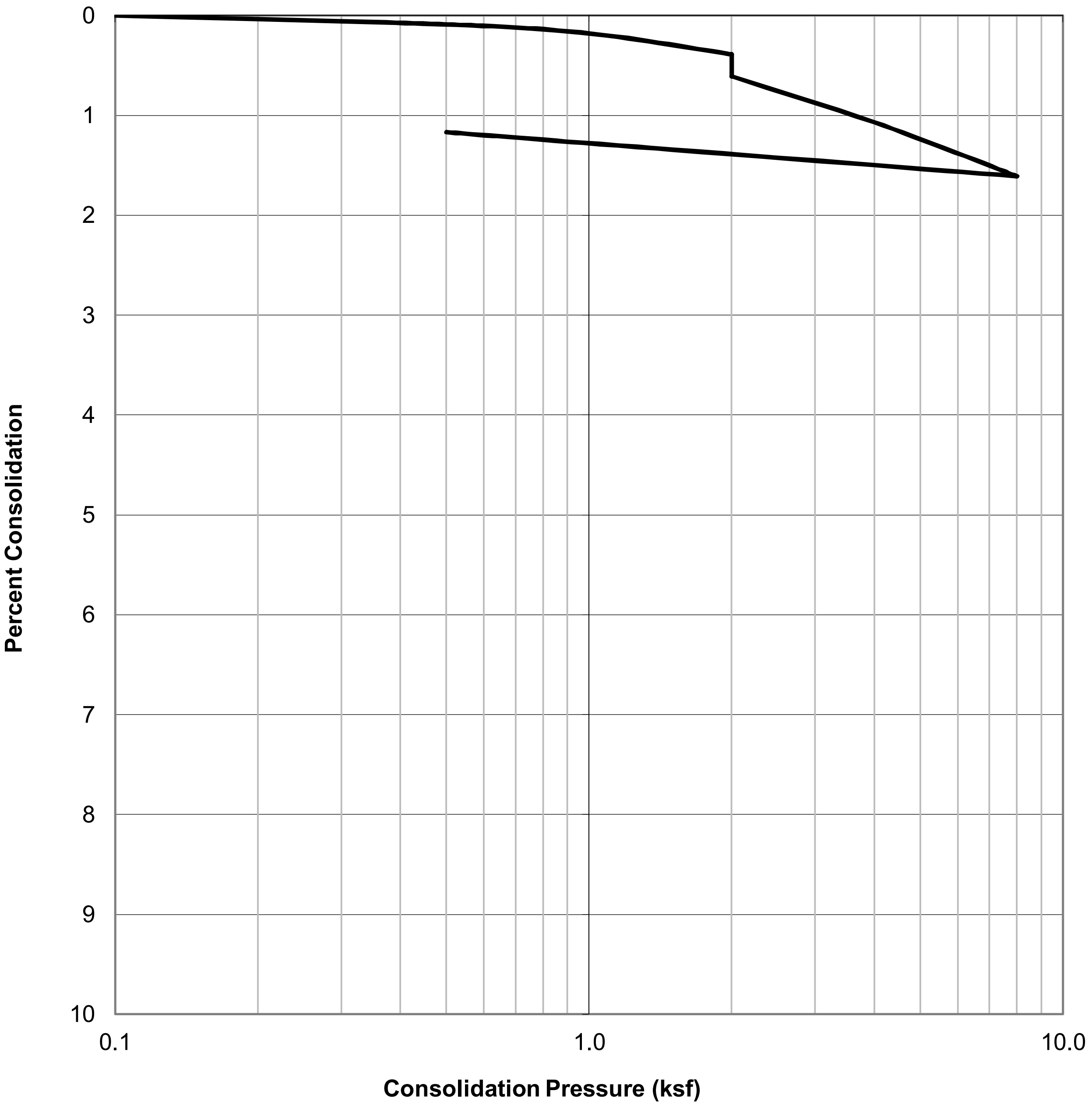
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@7.5	Dark Yellowish Brown Sand (SP)	98.1	10.6	20.9

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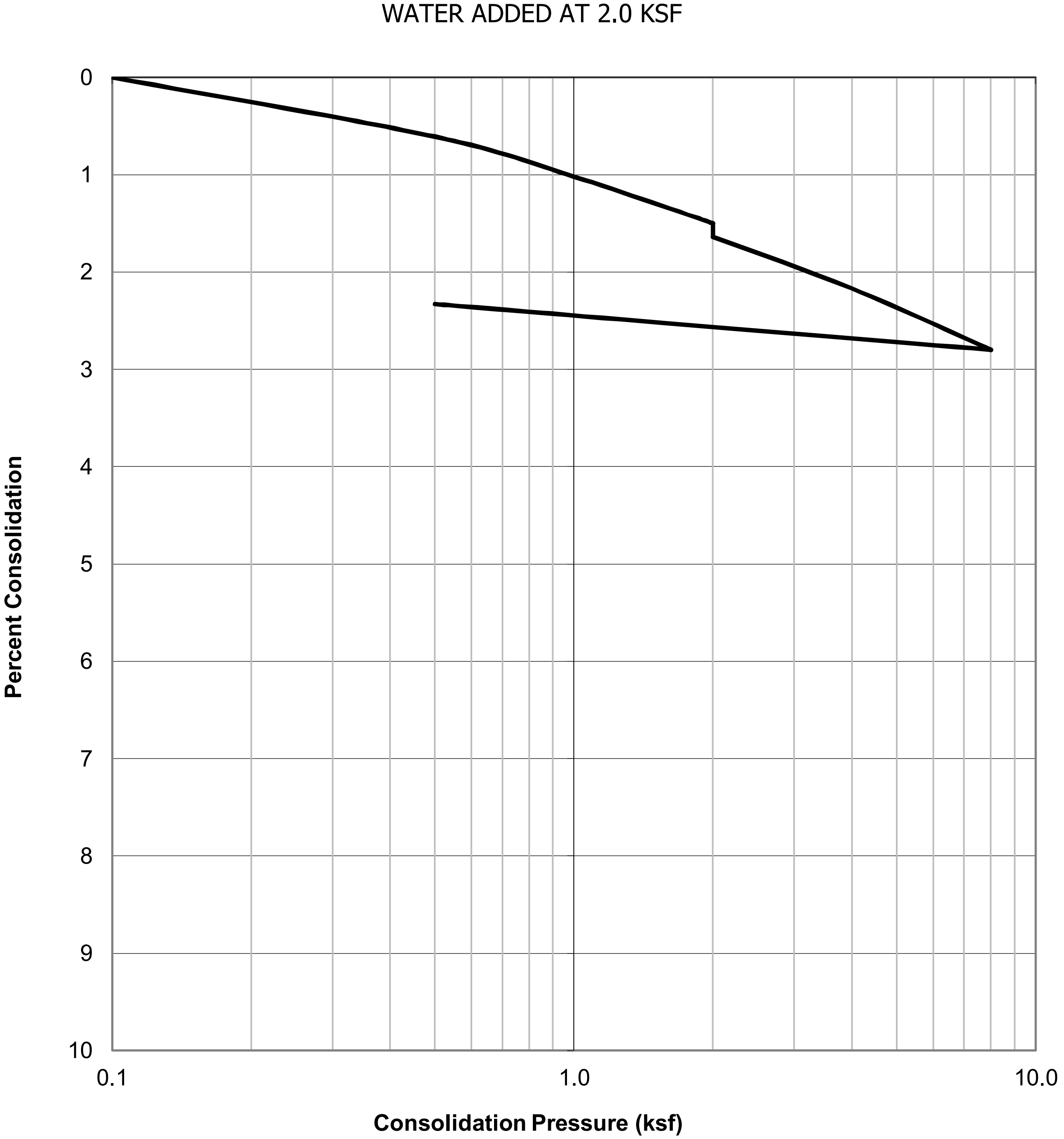

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@7.5	Light Brown Sand (SP)	103.2	19.4	19.6

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CONSOLIDATION TEST RESULTS

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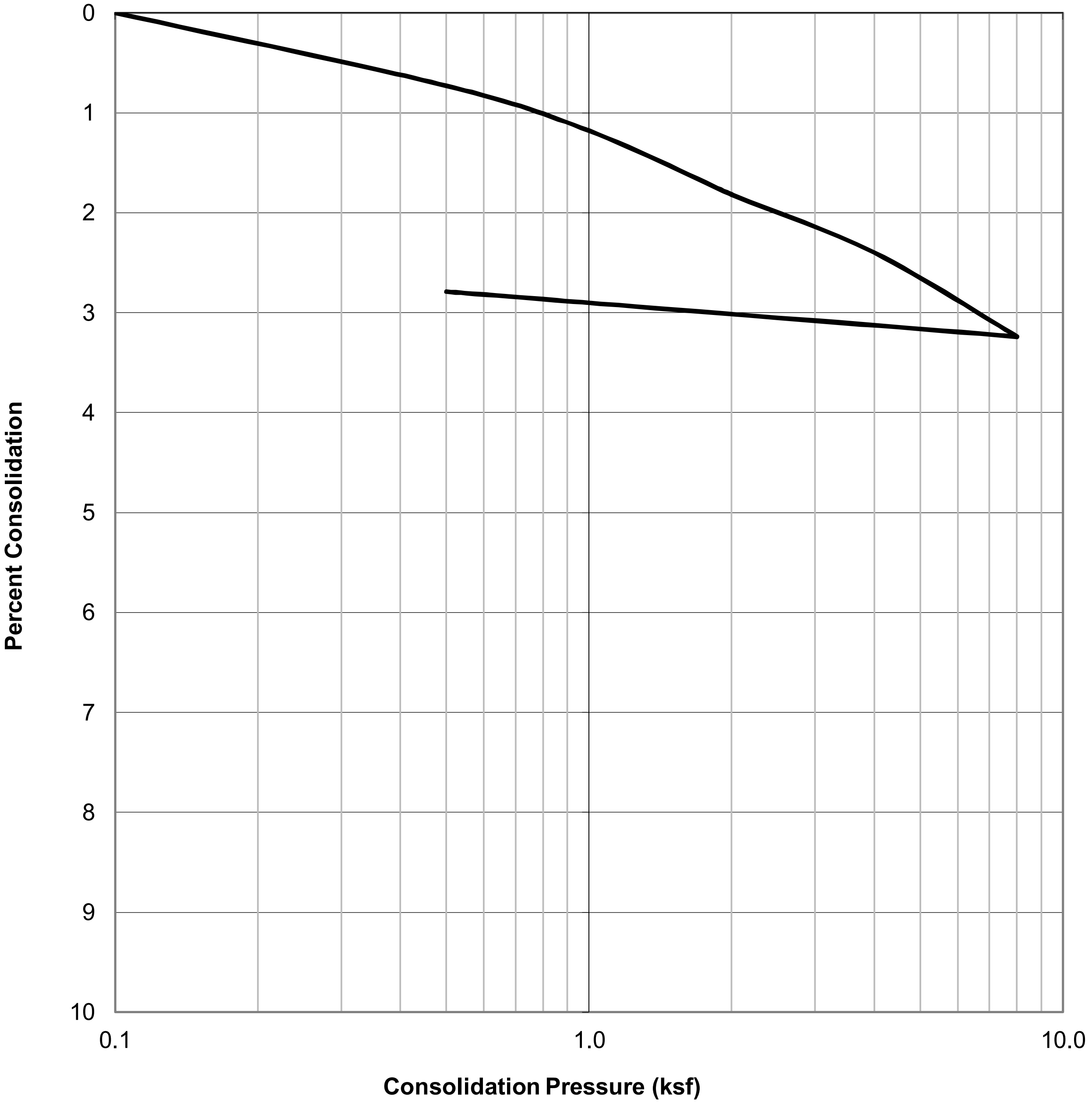
Figure B30

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@10	Dark Yellowish Brown Sand (SP)	109.9	20.4	22.4

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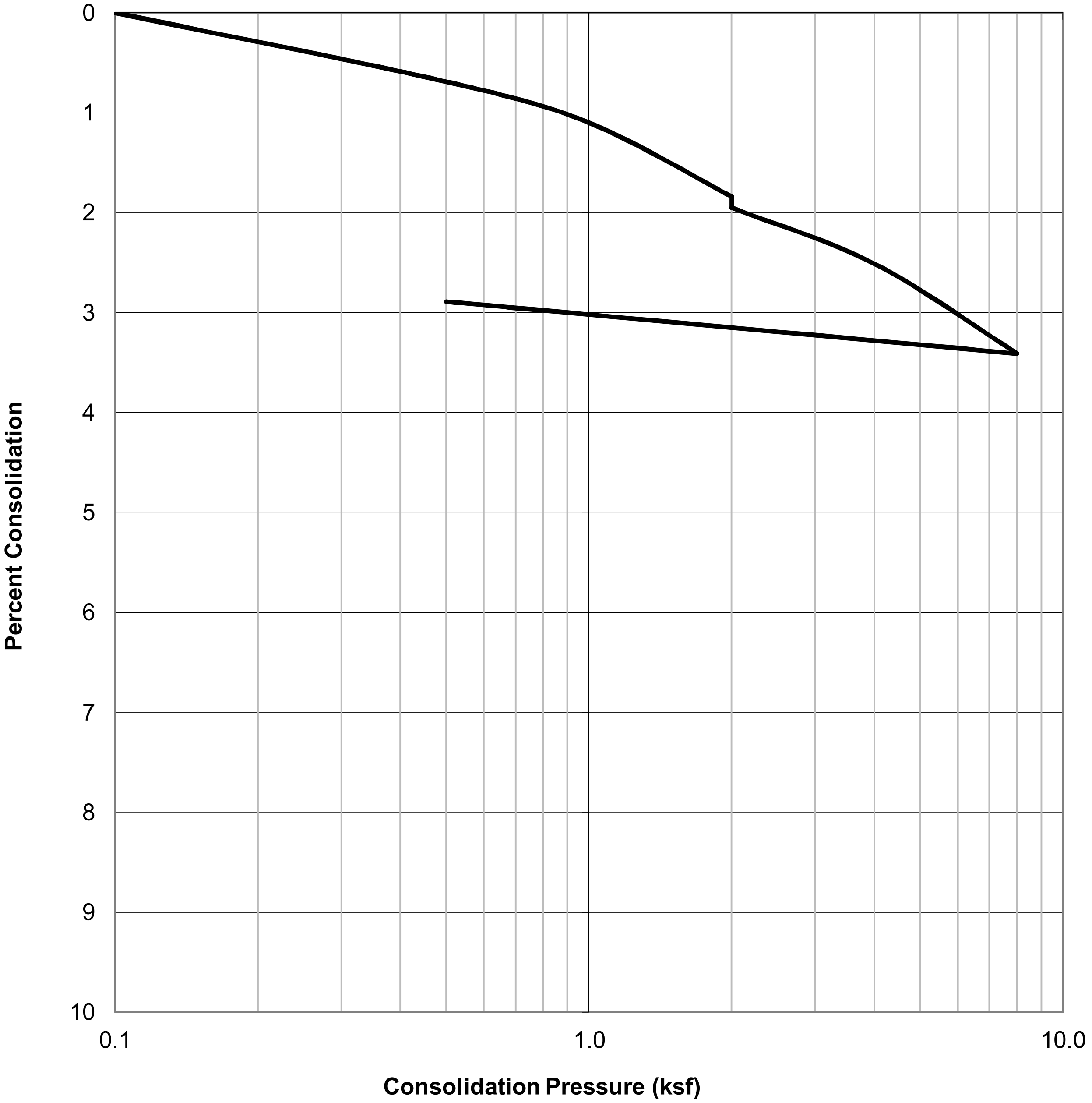
Figure B31

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@10	Brown Sand with Clay (SP-SC)	108.1	17.7	17.0

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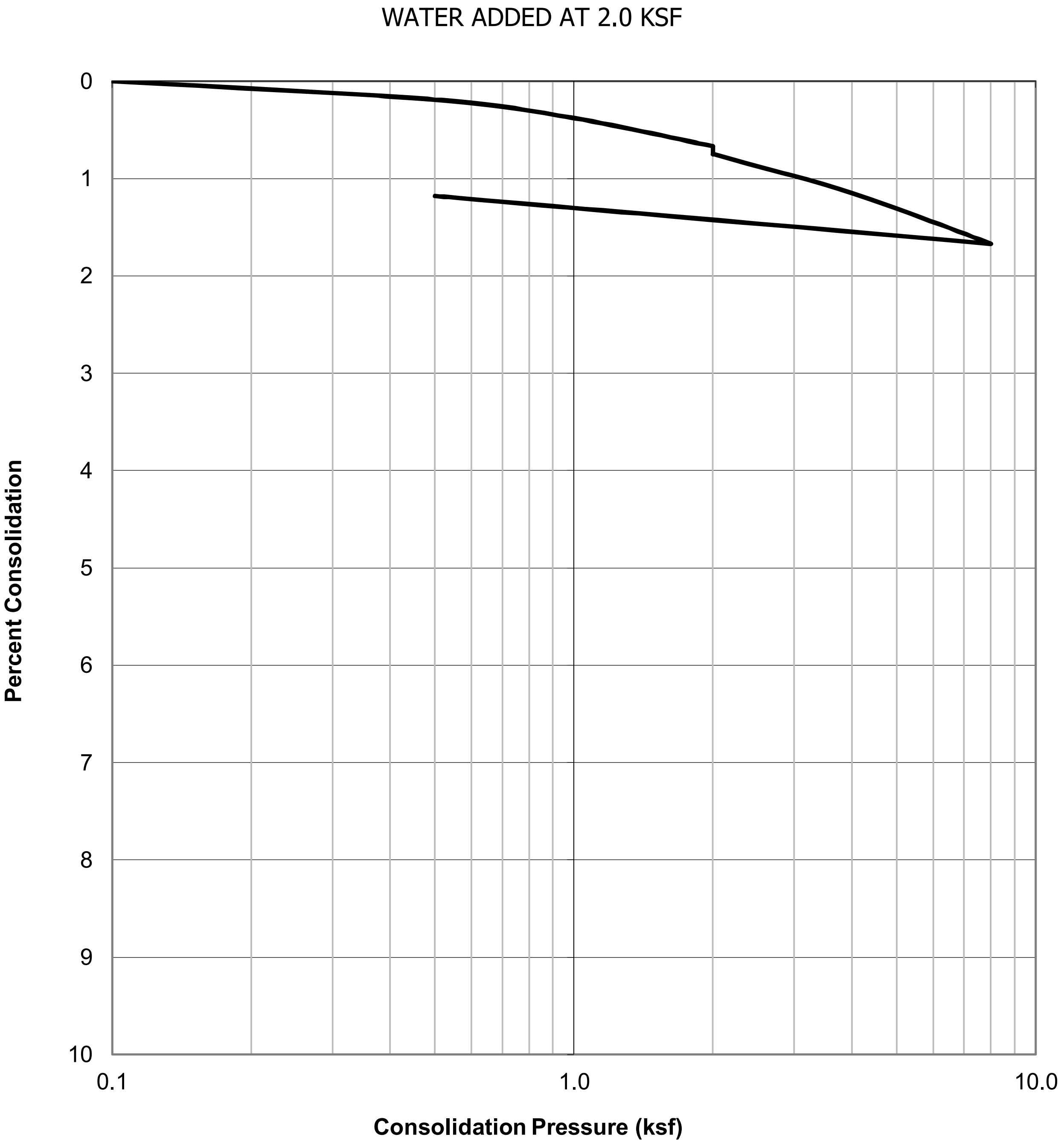
Figure B32



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@12.5	Yellowish Brown Sand with Clay (SP-SC)	114.1	13.5	15.7

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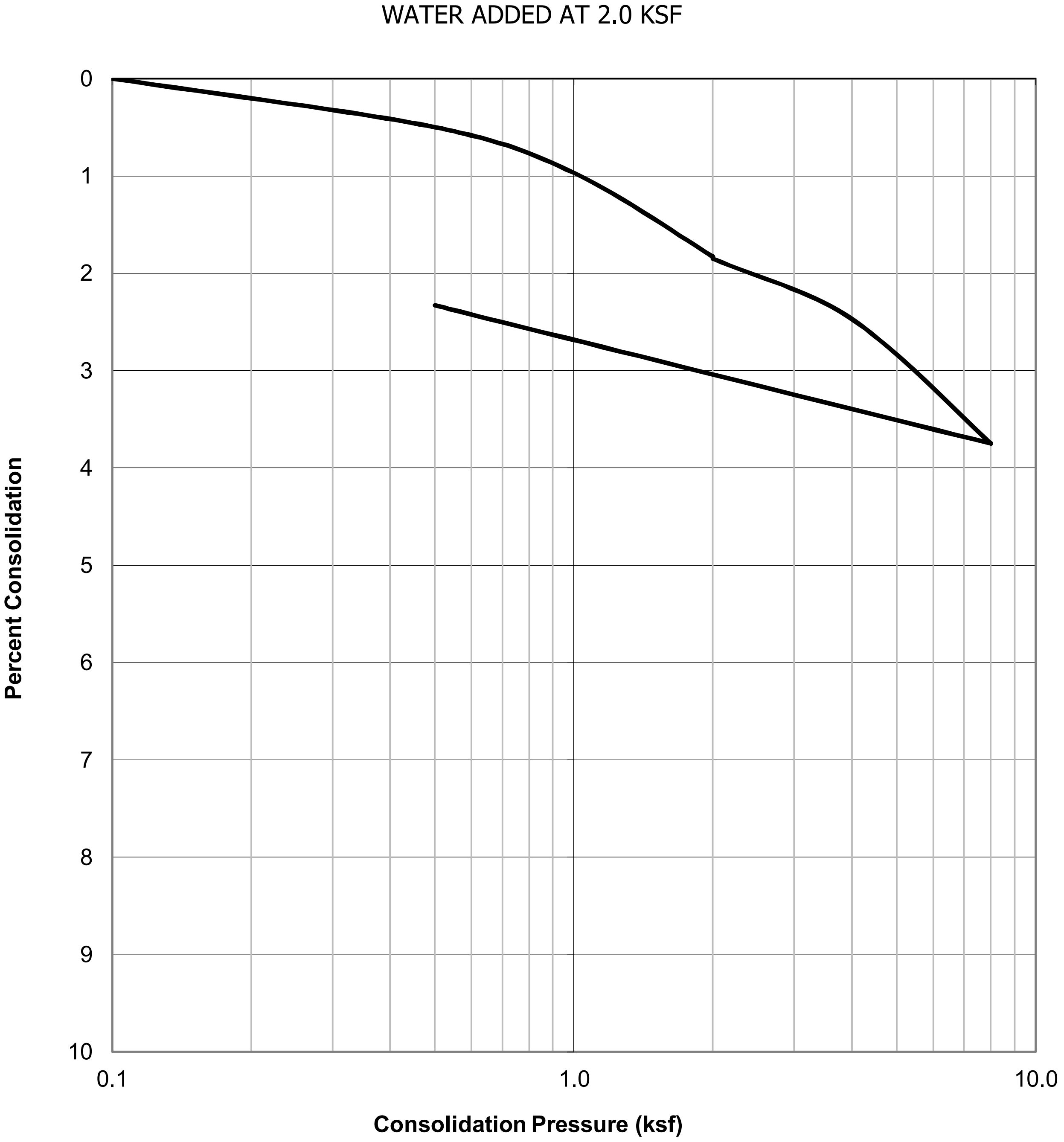
Figure B33



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@12.5	Yellowish Brown Sandy Clay (CL)	104.9	21.1	21.4

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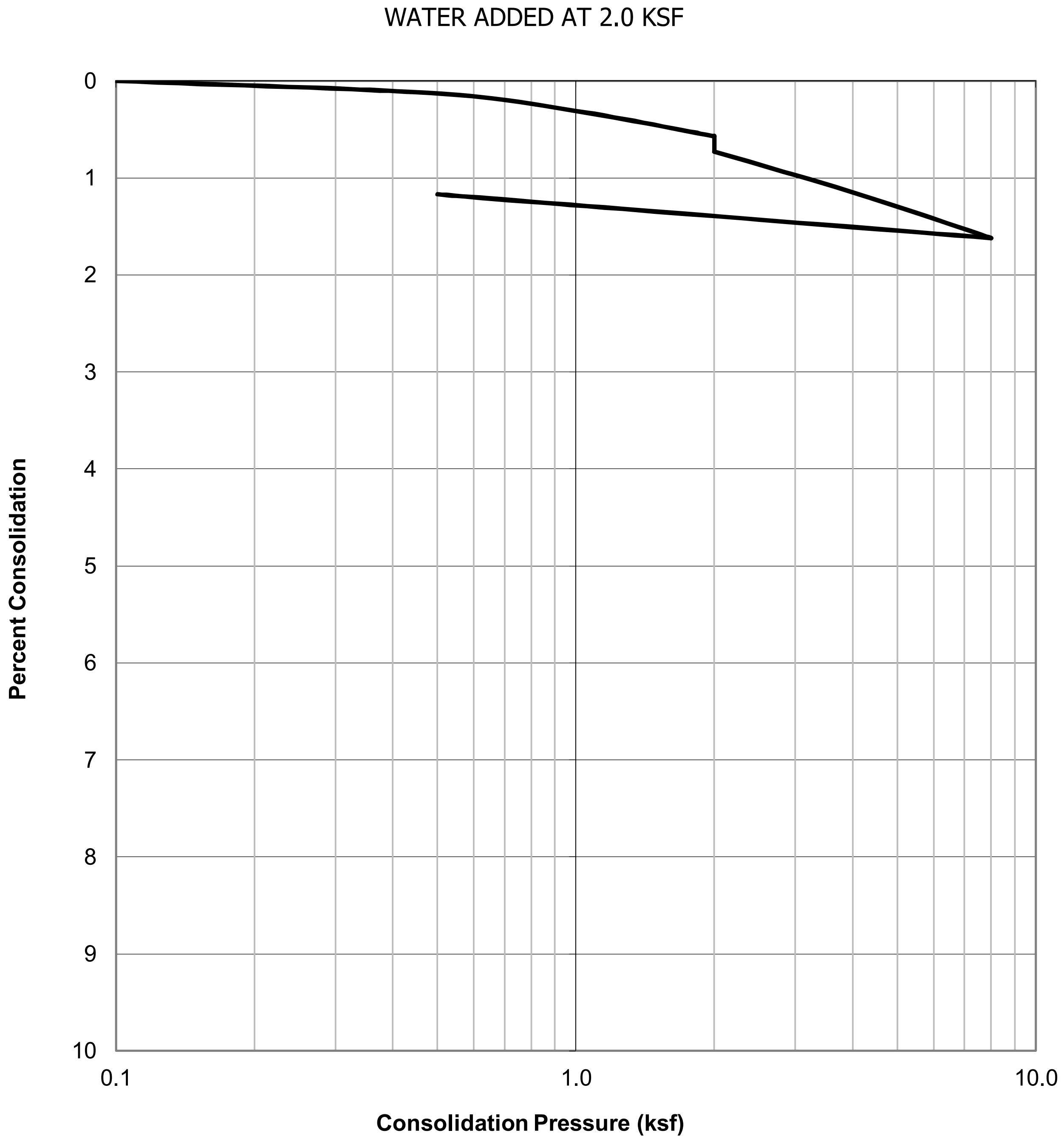
Figure B34



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@17.5	Yellowish Brown Sand (SP)	100.0	11.4	21.4

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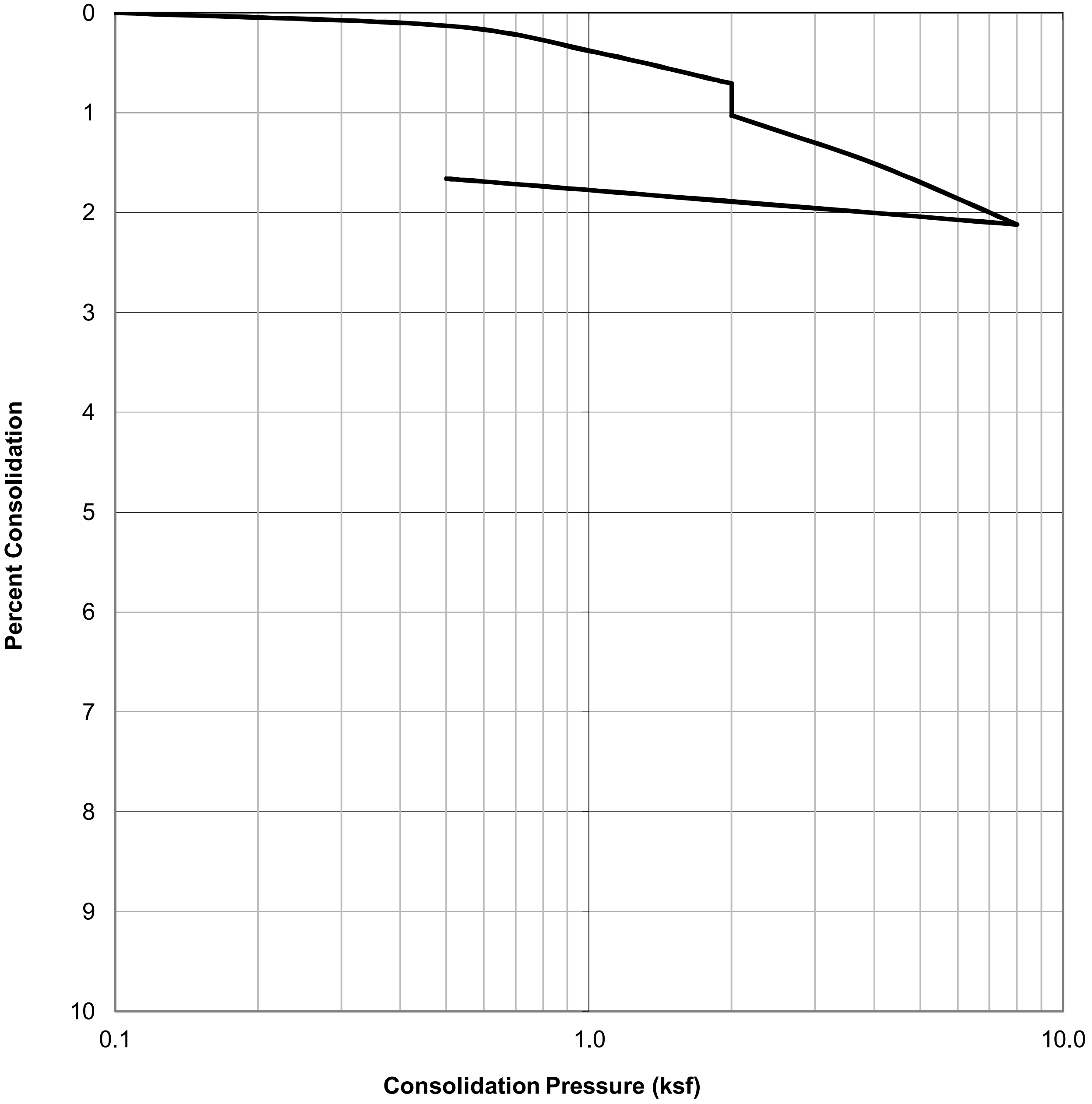
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@20	Light Brown Sand (SP)	92.2	11.7	26.6

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CONSOLIDATION TEST RESULTS

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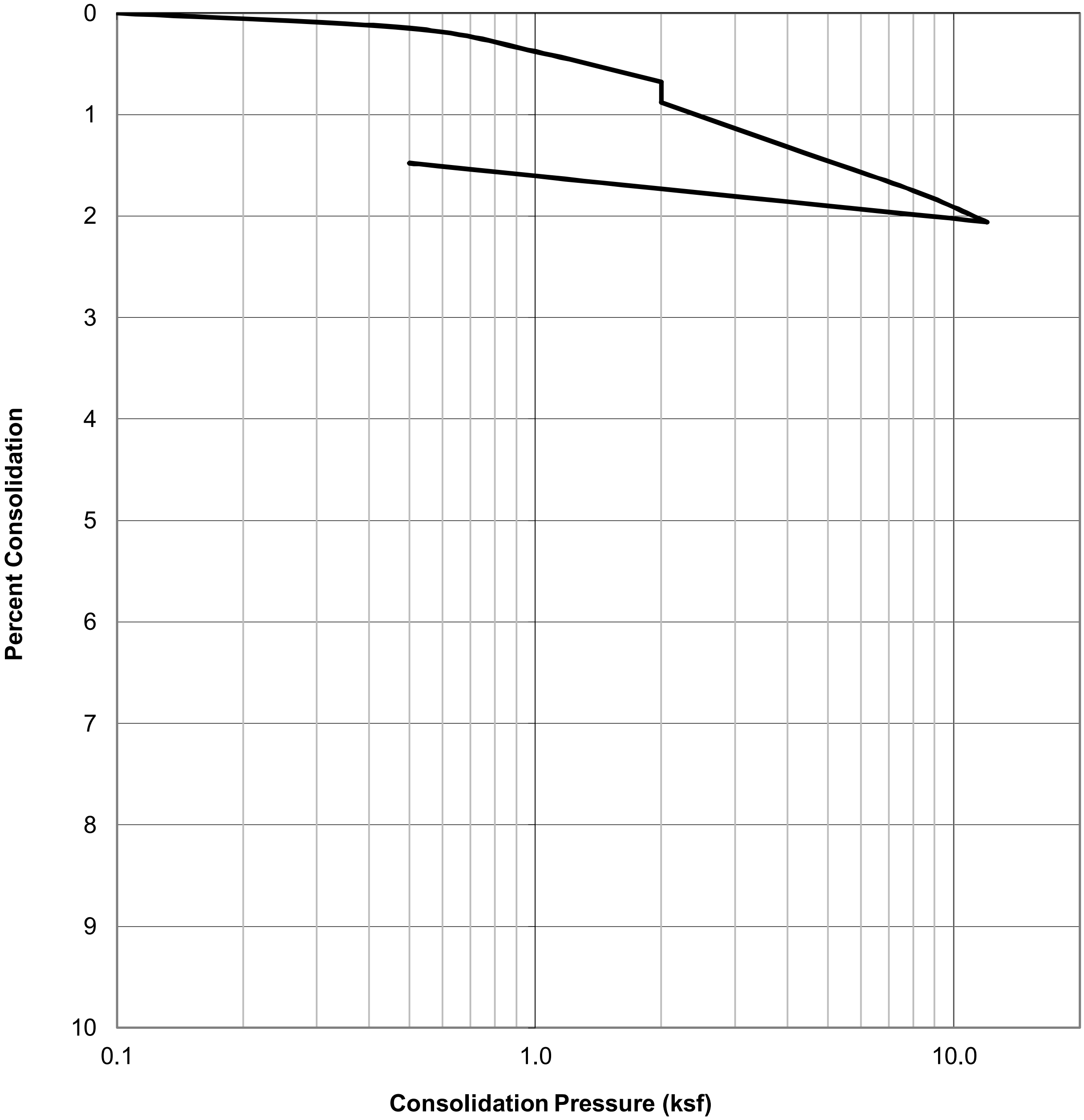
Figure B36

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@25	Light Brown Sand (SP)	104.6	6.0	20.7

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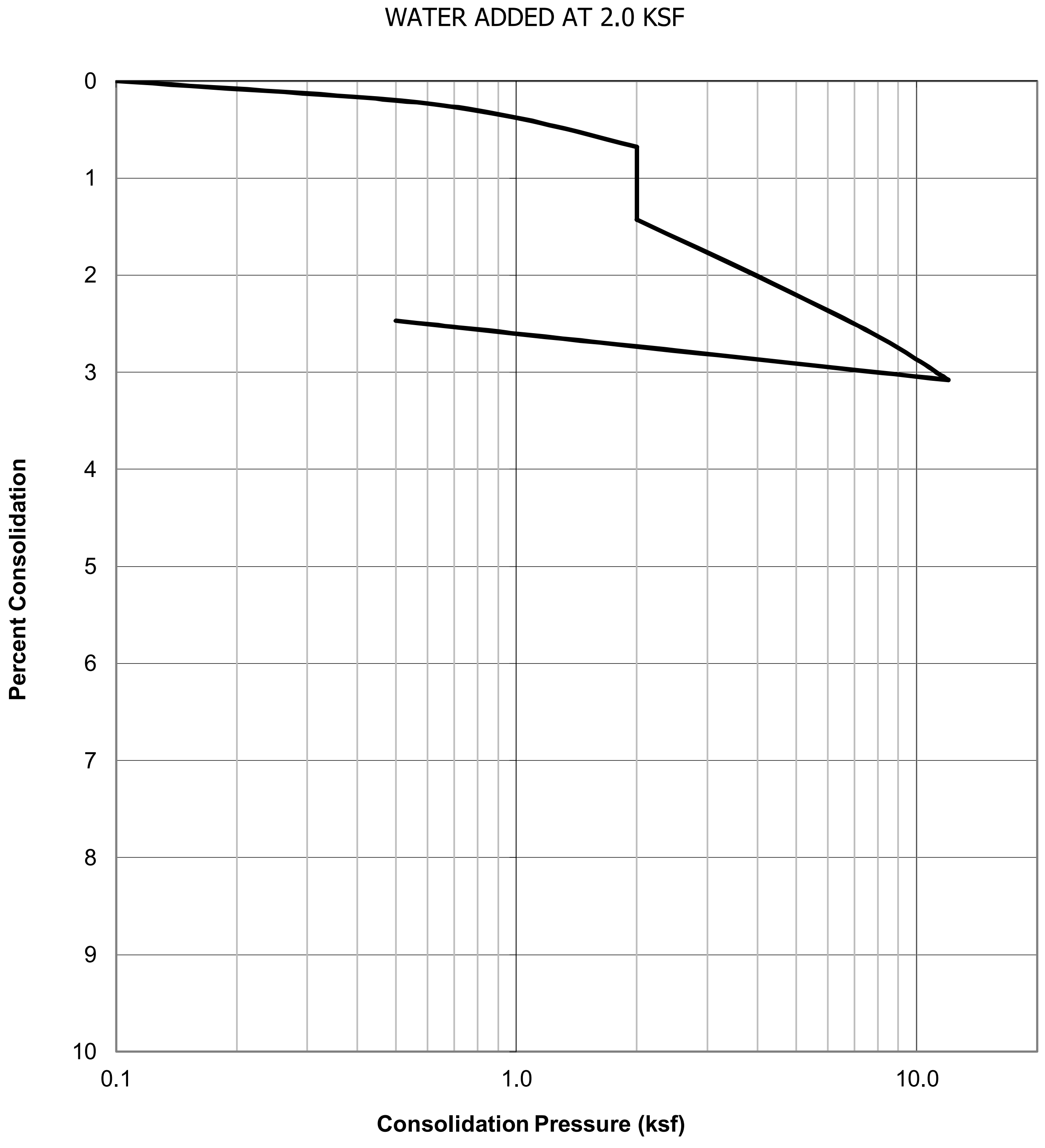

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Light Brown Sand (SP)	96.4	7.4	24.3

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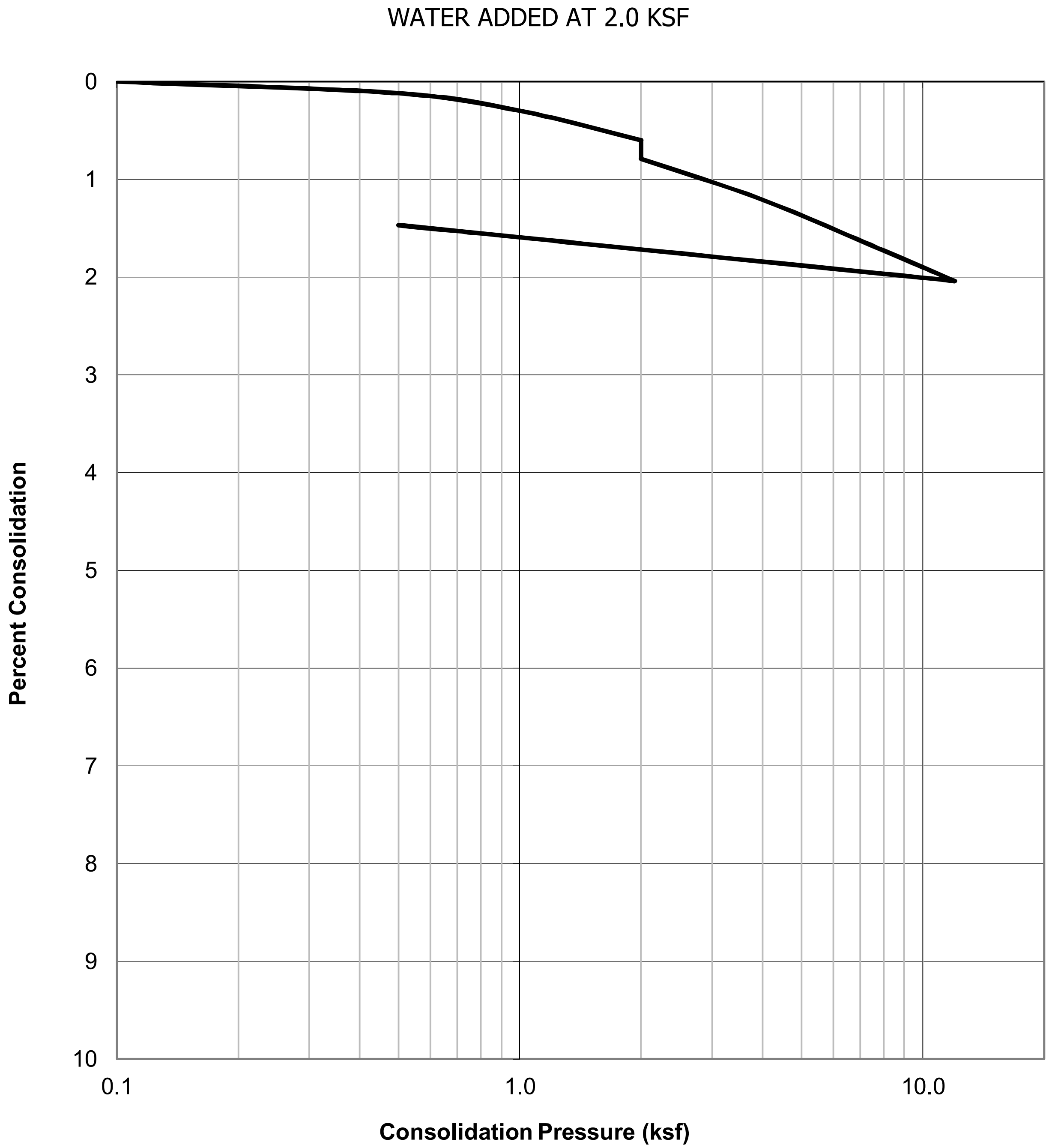
Figure B38



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@25	Light Brown Sand (SP)	95.9	6.5	21.7

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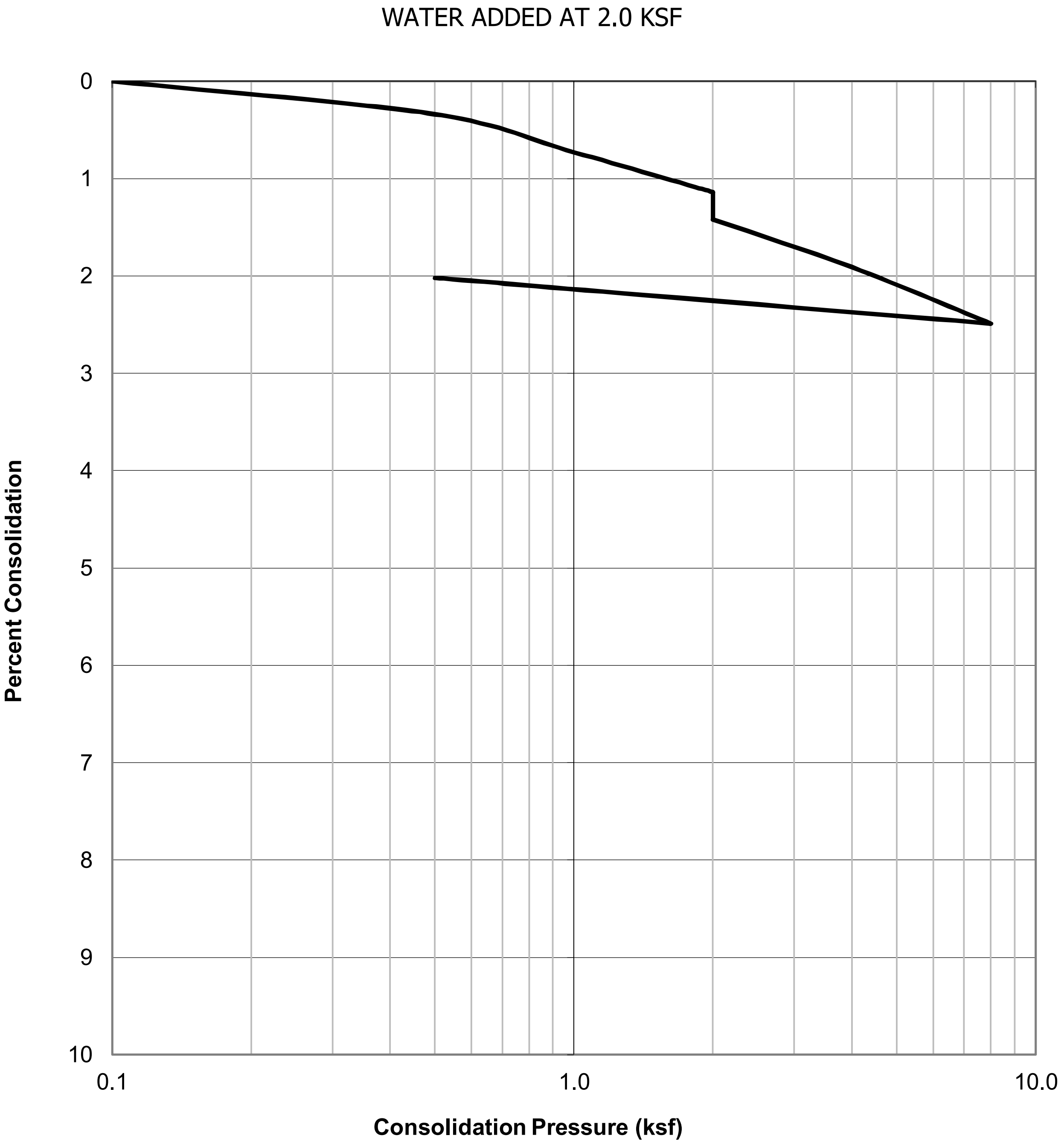
Figure B39



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@25	Light Brown Sand (SP)	94.8	6.4	24.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1062-06-02

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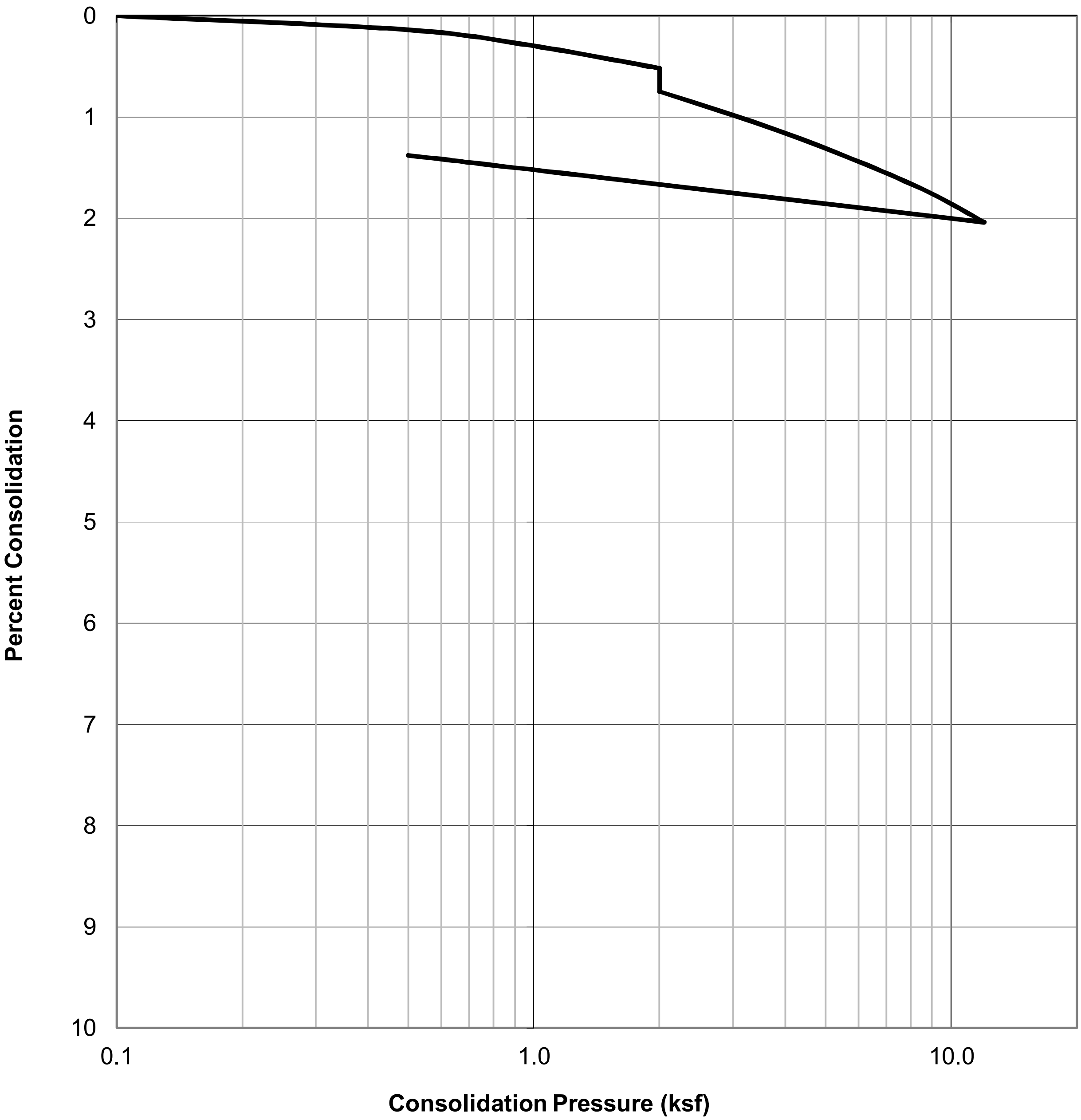
Figure B40

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@30	Light Brown Sand (SP)	106.8	6.5	20.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1062-06-02

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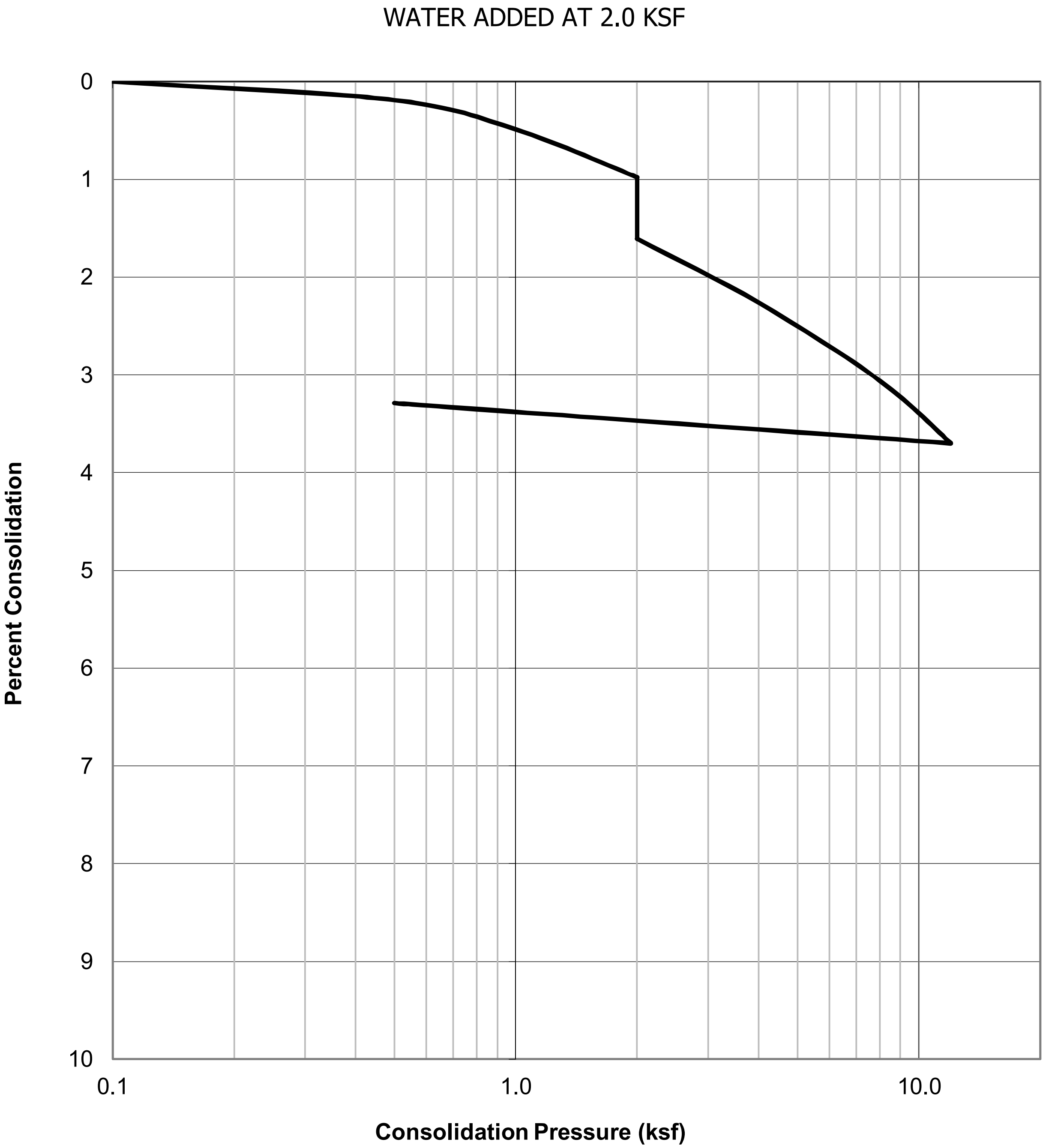

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@30	Light Brown Sand (SP)	100.9	6.6	18.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1062-06-02

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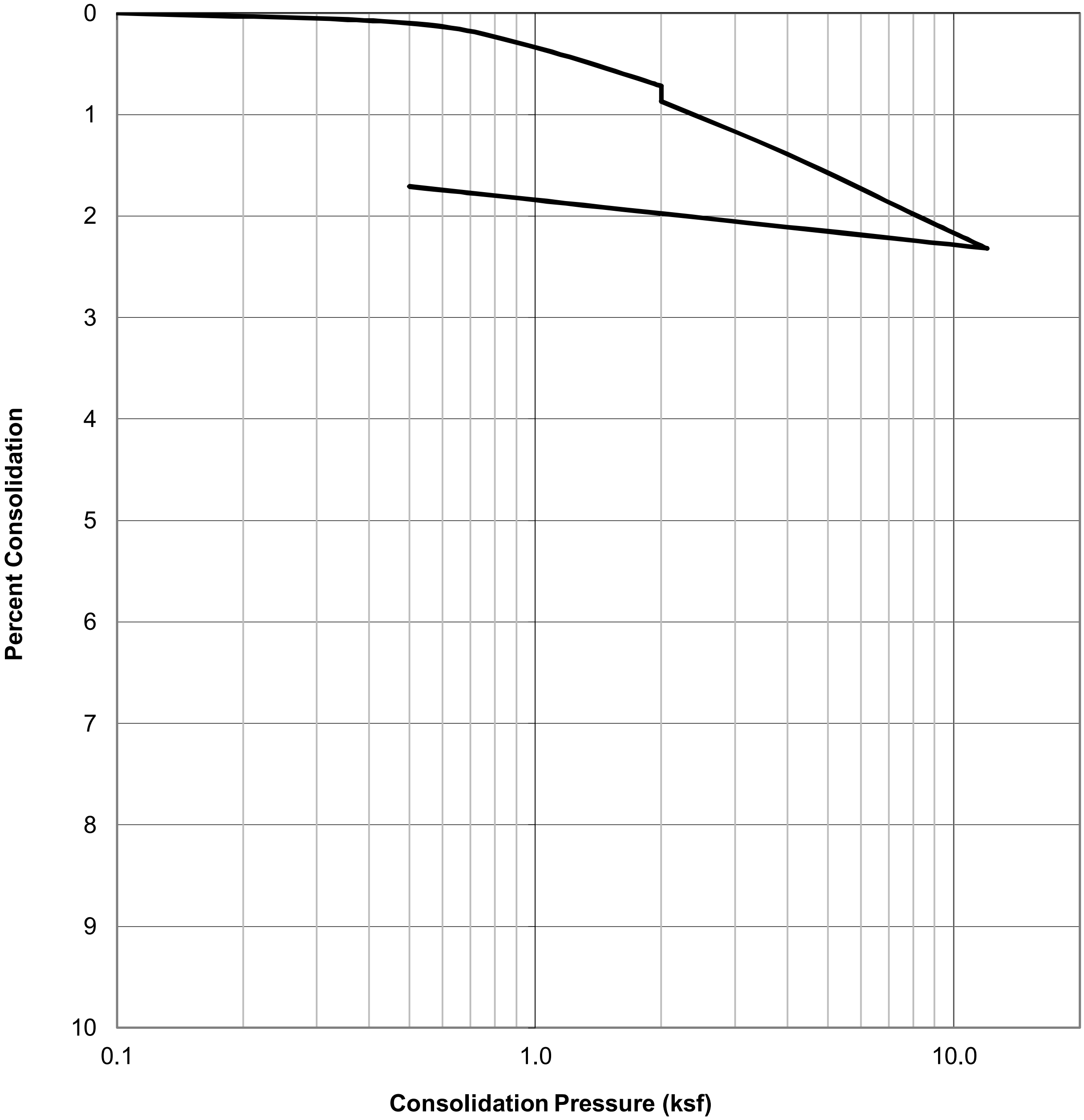
Figure B42

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@30	Light Brown Sand (SP)	97.7	6.1	21.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1062-06-02

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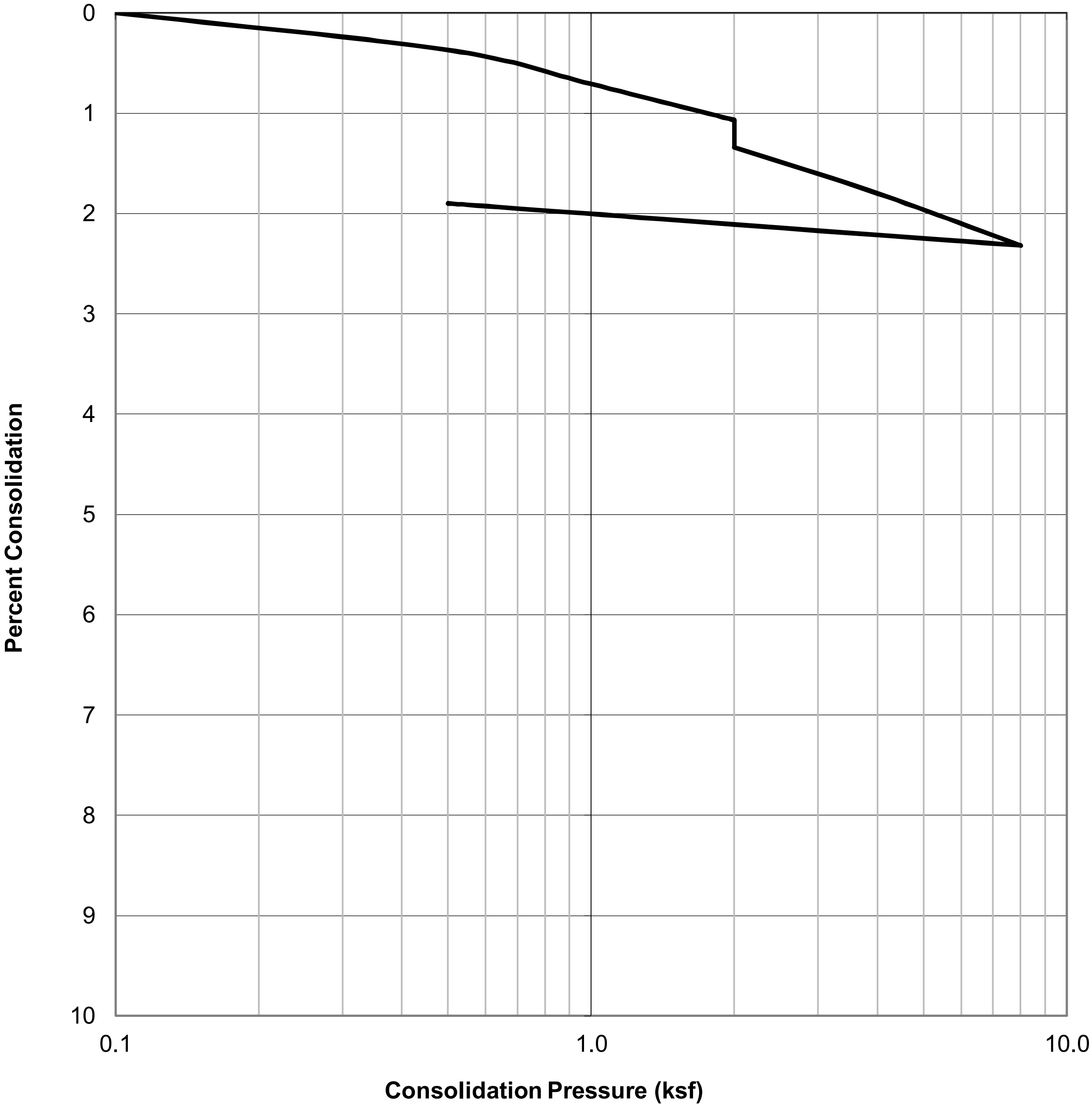
Figure B43

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@30	Light Brown Sand (SP)	102.2	4.3	19.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1062-06-02

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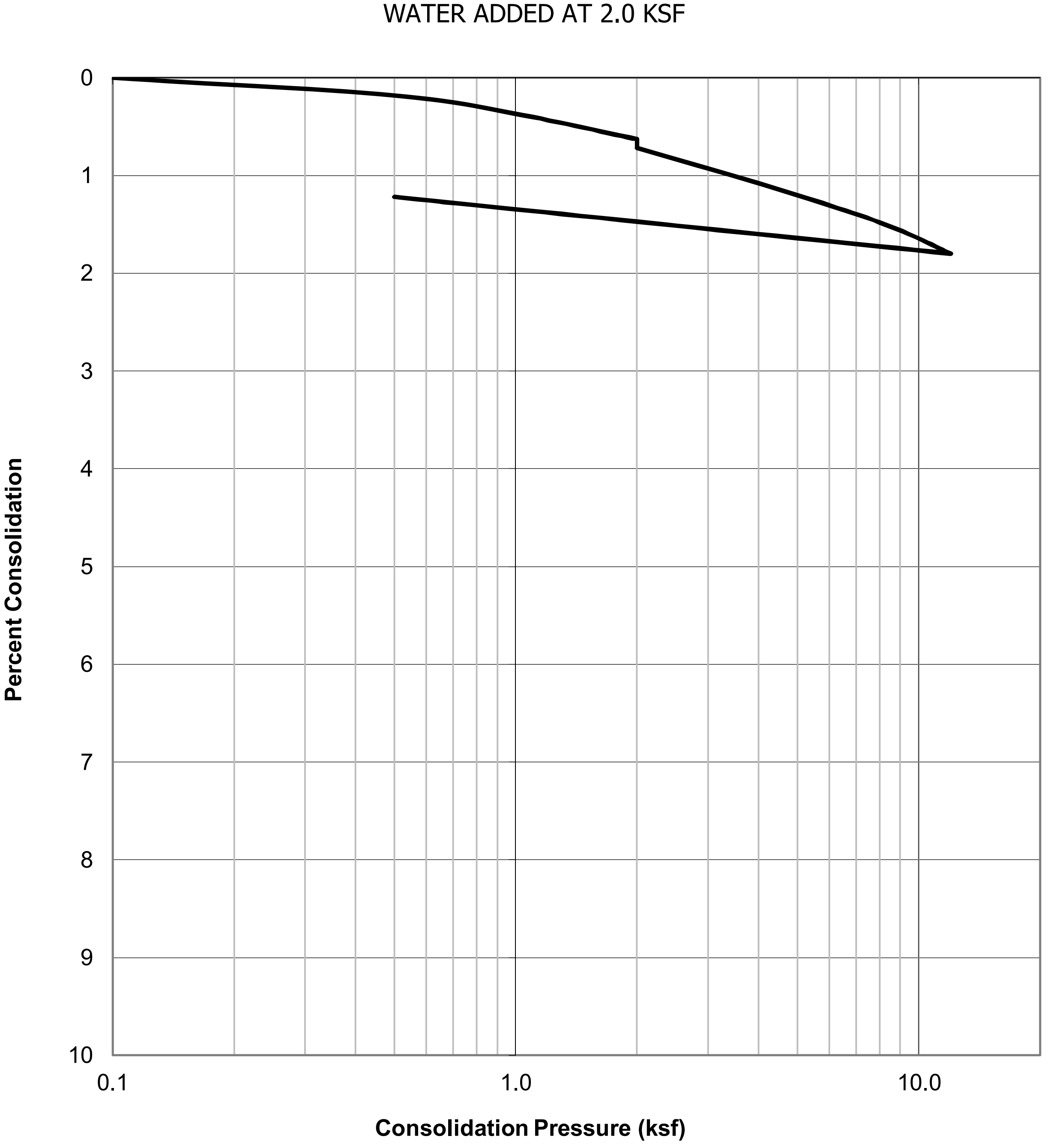
Figure B44



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Light Brown Sand (SP)	98.1	7.5	23.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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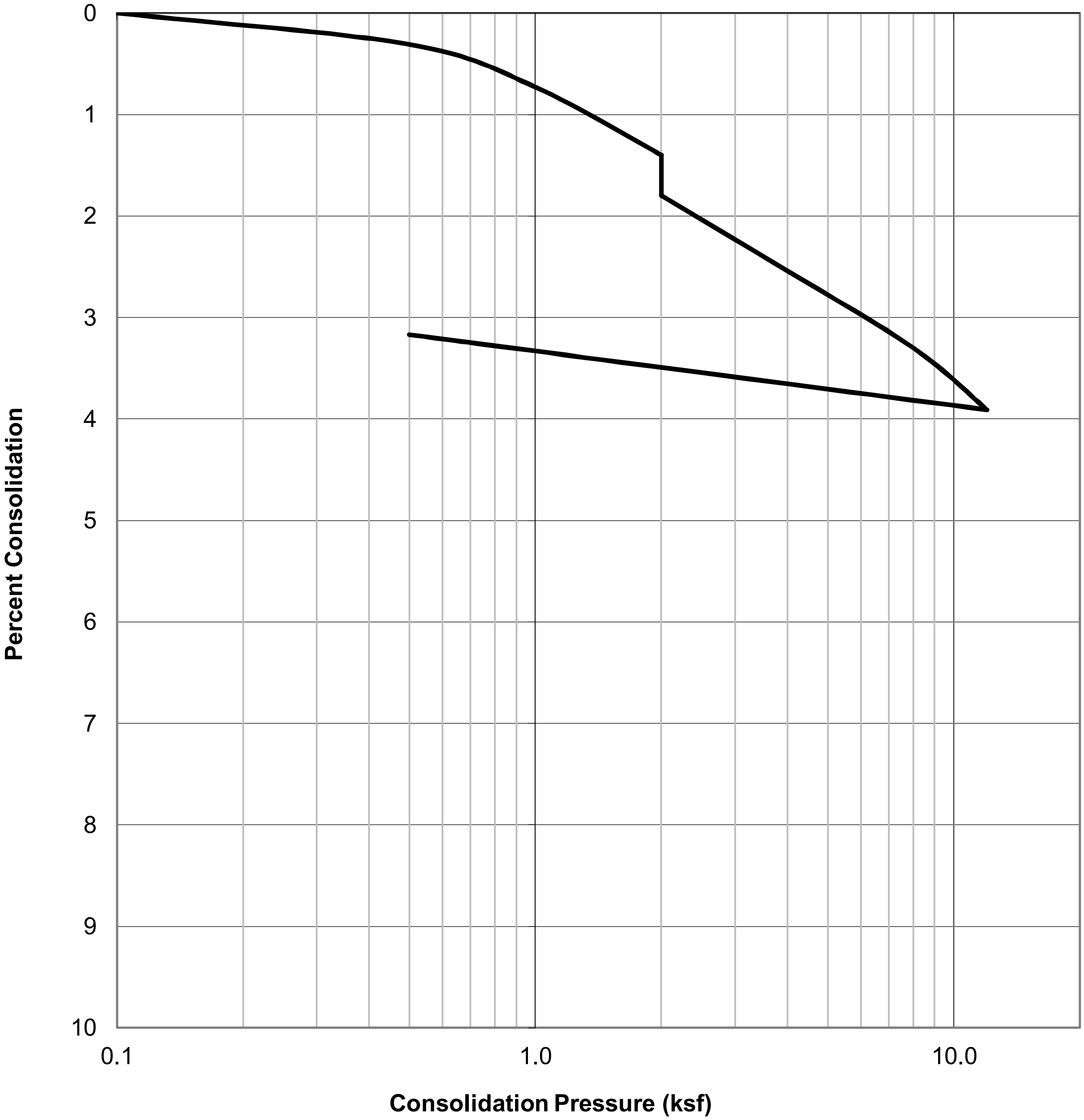
Figure B45

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@35	Light Brown Sand (SP)	94.2	6.5	23.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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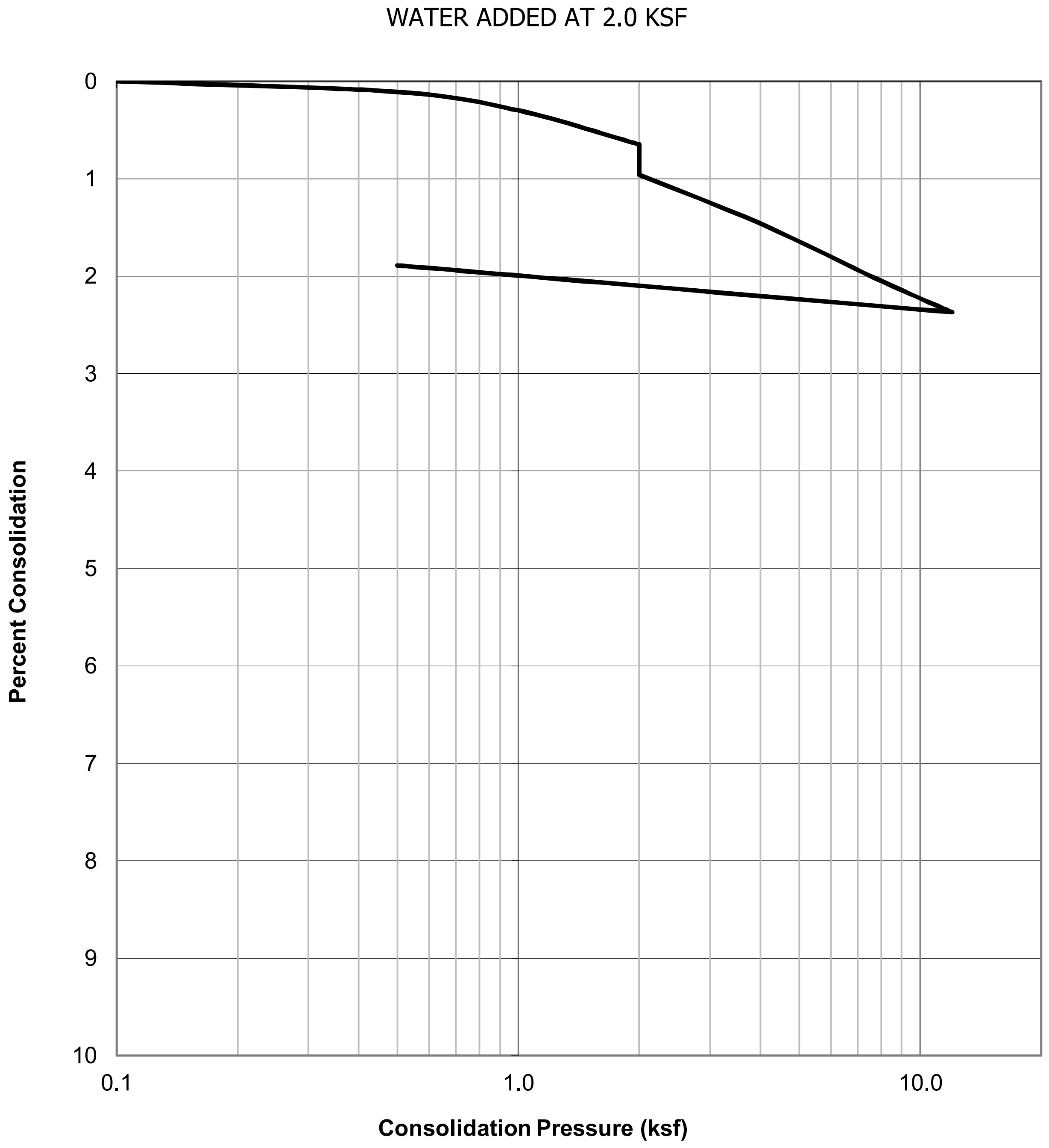
Figure B46



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@35	Light Brown Sand (SP)	97.9	4.4	20.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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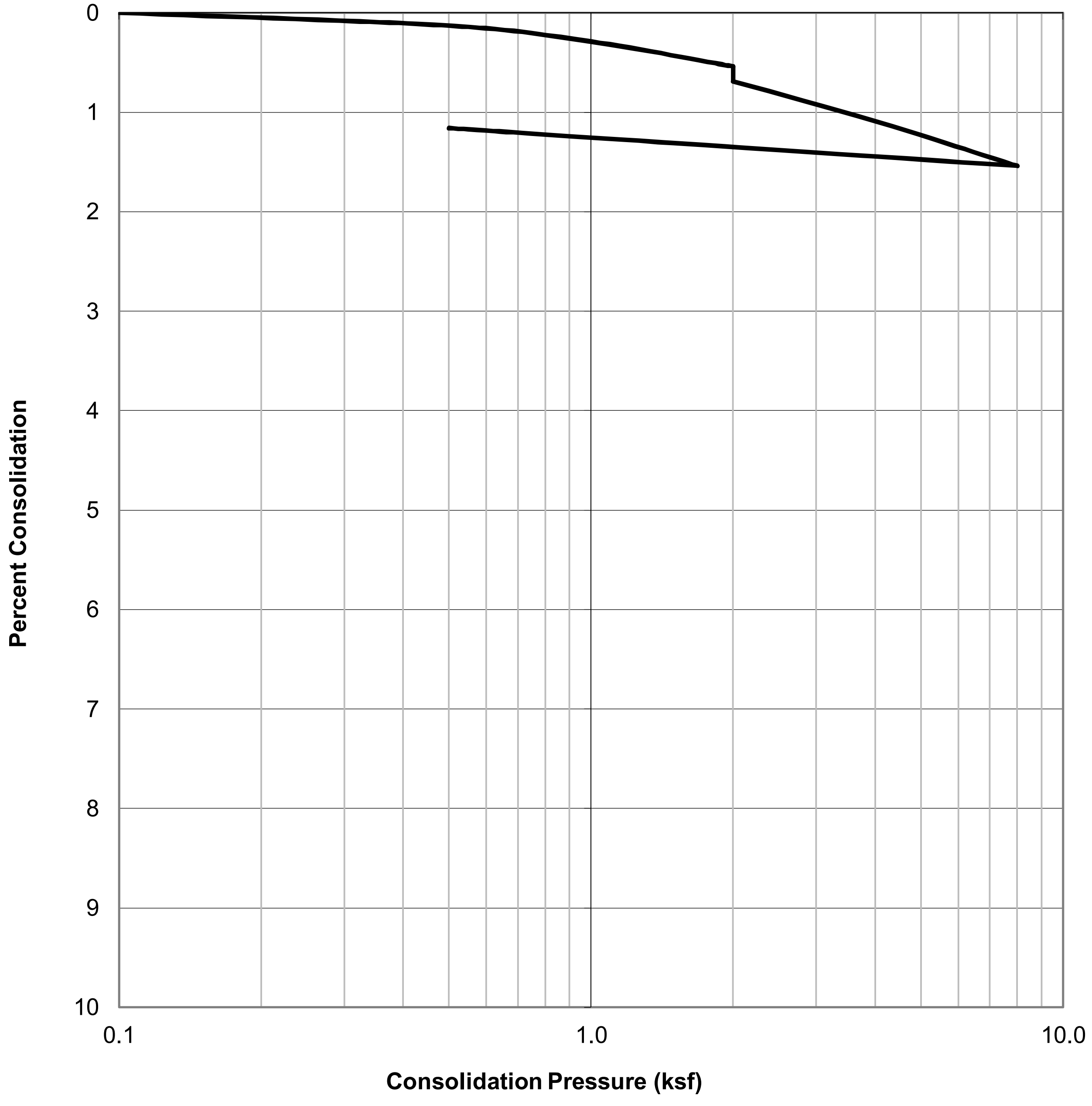
Figure B47

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@40	Light Brown Sand (SP)	99.6	4.9	24.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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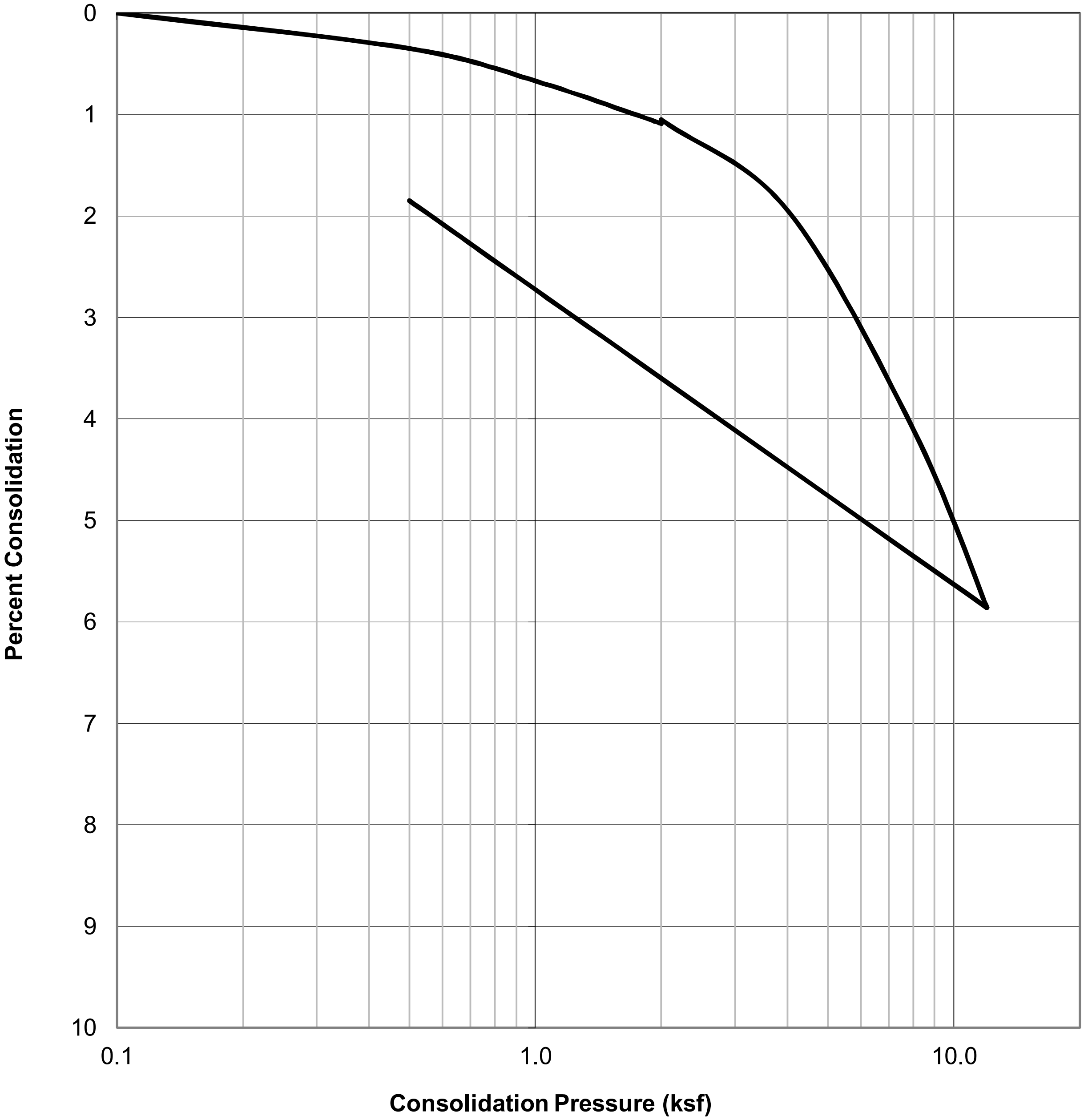
Figure B48

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@40	Light Brown Clay (CL)	70.7	52.1	55.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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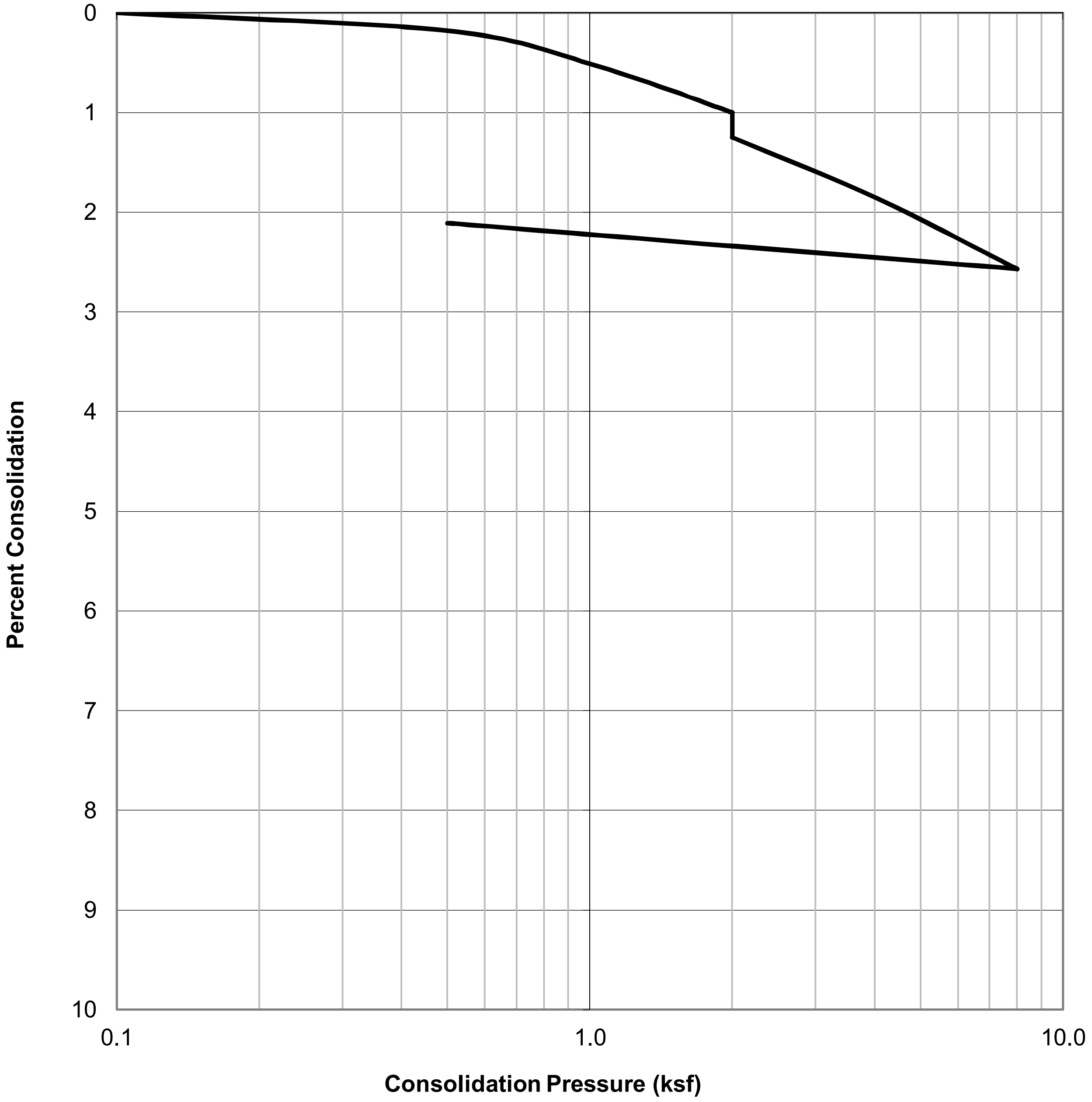
Figure B49

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@50	Light Brown Sand (SP)	88.6	9.5	30.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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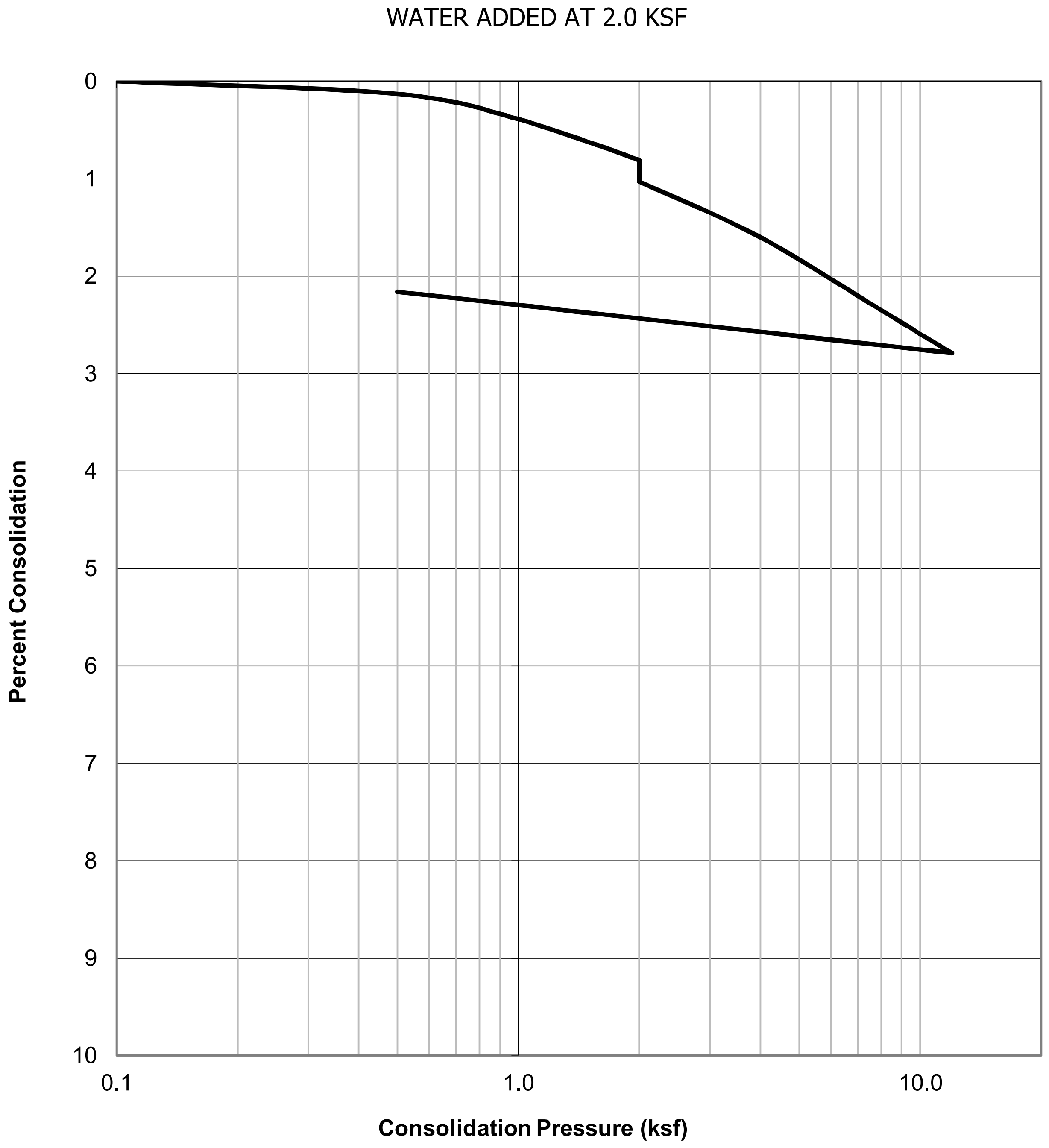
Figure B50



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@50	Light Brown Sand (SP)	88.9	7.1	26.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1062-06-02

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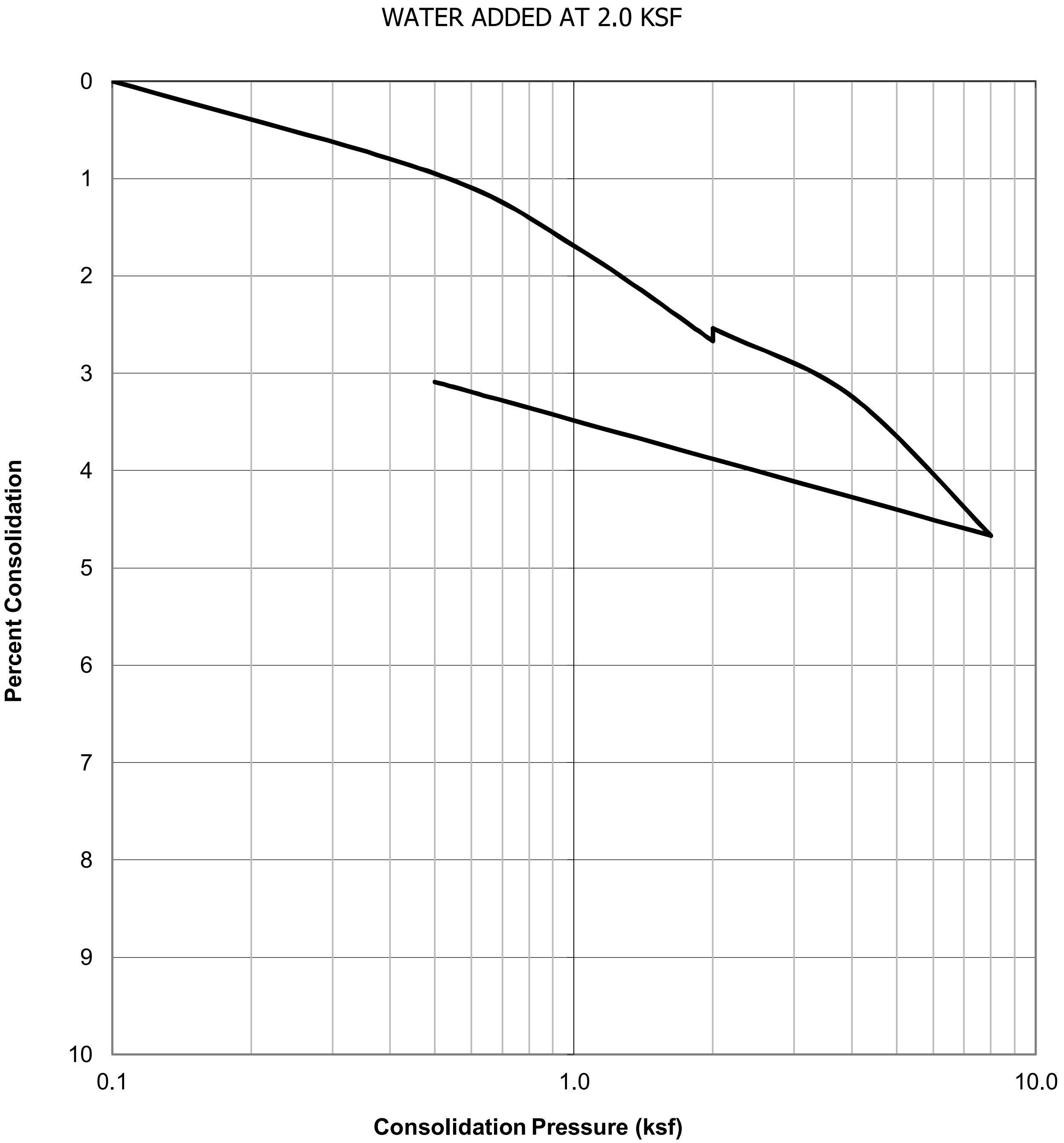
Figure B51



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@55	Grayish Brown Clay (CL)	98.5	25.8	25.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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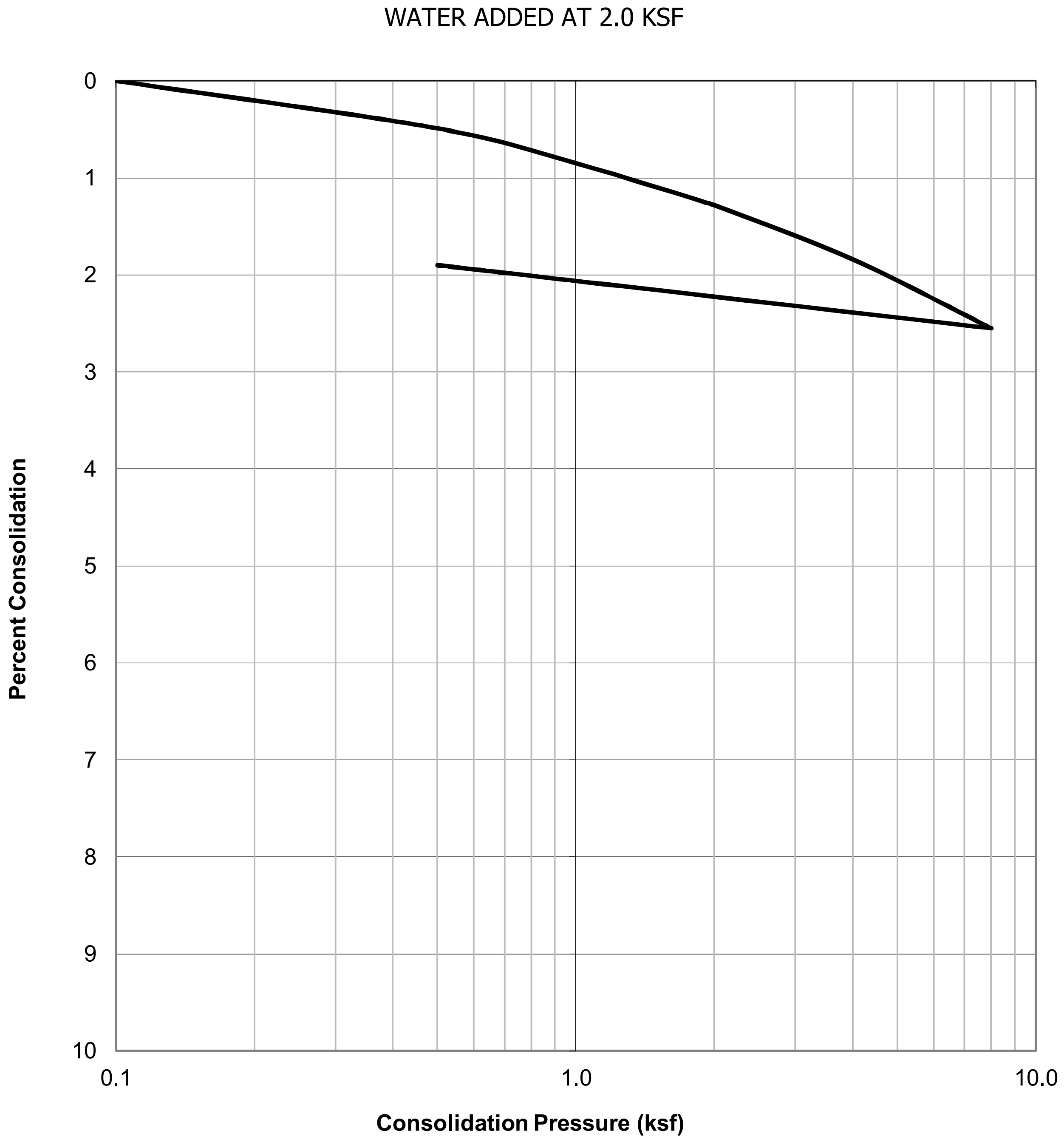
Figure B52



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@60	Olive Brown Sandy Clay (CL)	106.2	19.1	21.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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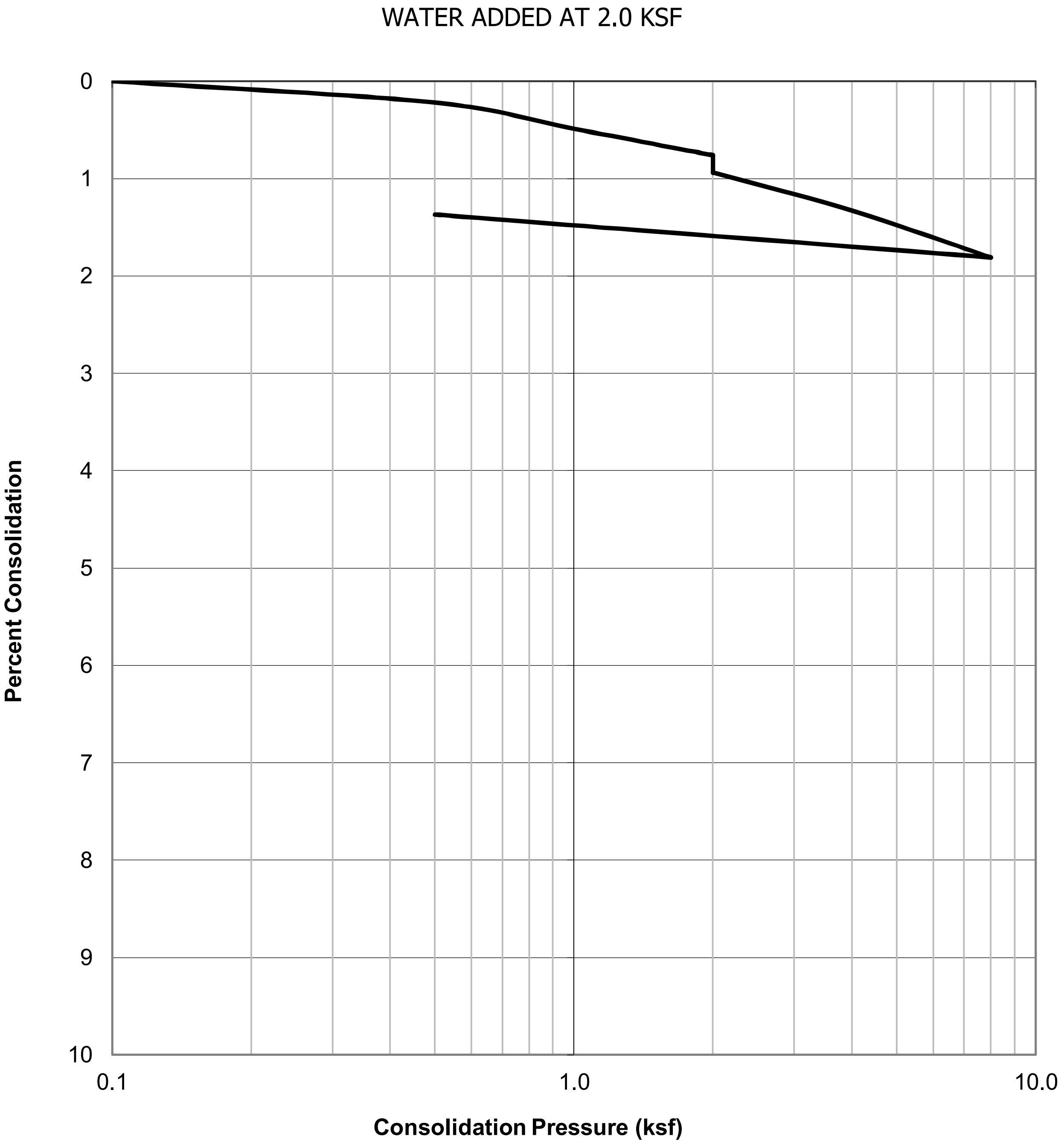
Figure B53



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@60	Light Brown Sand (SP)	87.9	10.2	30.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1062-06-02

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Figure B54

B5@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	590.9	612.5
Wt. of Mold	(gm)	176.5	176.5
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	395.6	612.5
Dry Wt. of Soil + Cont.	(gm)	372.4	382.3
Wt. of Container	(gm)	95.6	176.5
Moisture Content	(%)	8.4	14.1
Wet Density	(pcf)	125.0	131.3
Dry Density	(pcf)	115.3	115.2
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	65.4	69.7
Degree of Saturation	(%) [S_{meas}]	49.5	77.0

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/6/2021	10:00	1.0	0	0.37
5/6/2021	10:10	1.0	10	0.3695
Add Distilled Water to the Specimen				
5/7/2021	10:00	1.0	1430	0.3904
5/7/2021	11:00	1.0	1490	0.3904

Expansion Index (EI meas) =	20.9
Expansion Index (Report) =	**21**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

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

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: PZ

Project No.: W1062-06-02

22501 Hawthorne Boulevard
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Figure B55

B8@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	609.6	621.8
Wt. of Mold	(gm)	195.3	195.3
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	933.5	621.8
Dry Wt. of Soil + Cont.	(gm)	908.7	380.1
Wt. of Container	(gm)	633.5	195.3
Moisture Content	(%)	9.0	12.2
Wet Density	(pcf)	125.0	128.5
Dry Density	(pcf)	114.7	114.5
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	66.2	65.5
Degree of Saturation	(%) [S_{meas}]	52.1	70.9

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/6/2021	10:00	1.0	0	0.3371
5/6/2021	10:10	1.0	10	0.3368
Add Distilled Water to the Specimen				
5/7/2021	10:00	1.0	1430	0.3333
5/7/2021	11:00	1.0	1490	0.3333

Expansion Index (EI meas) =	-3.5
Expansion Index (Report) =	**0**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3

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

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

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Project No.: W1062-06-02

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Figure B56

B3@30-35

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	0.0	589.1
Wt. of Mold	(gm)	0.0	0.0
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	300.0	589.1
Dry Wt. of Soil + Cont.	(gm)	#DIV/0!	#DIV/0!
Wt. of Container	(gm)	0.0	0.0
Moisture Content	(%)	#DIV/0!	#DIV/0!
Wet Density	(pcf)	0.0	177.5
Dry Density	(pcf)	#DIV/0!	#DIV/0!
Void Ratio		#DIV/0!	#DIV/0!
Total Porosity		#DIV/0!	#DIV/0!
Pore Volume	(cc)	#DIV/0!	#DIV/0!
Degree of Saturation	(%) [S_{meas}]	#DIV/0!	#DIV/0!

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
9/20/2019	10:00	1.0	0	0.4385
9/20/2019	10:10	1.0	10	0.4382
Add Distilled Water to the Specimen				
9/21/2019	10:00	1.0	1430	0.4287
9/21/2019	11:00	1.0	1490	0.4287

Expansion Index (EI meas) =	-9.5
Expansion Index (Report) =	**0**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2016 California Building Code, Section 1803.5.3

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

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

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Figure B57

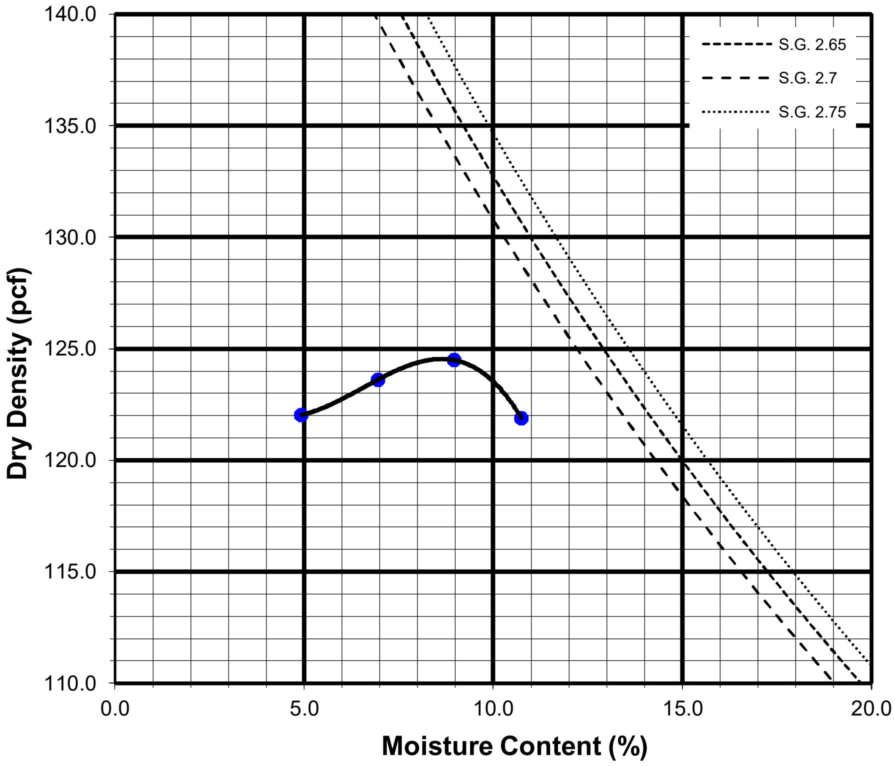
Sample No:

B2@0-5	Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6293	6345	6335	6230		
Weight of Mold	(g)	4296	4296	4296	4296		
Net Weight of Soil	(g)	1997	2049	2039	1934		
Wet Weight of Soil + Cont.	(g)	530.4	631.9	703.6	566.2		
Dry Weight of Soil + Cont.	(g)	502.1	587.9	644.7	544.1		
Weight of Container	(g)	95.0	97.2	96.4	95.3		
Moisture Content	(%)	7.0	9.0	10.7	4.9		
Wet Density	(pcf)	132.2	135.7	135.0	128.0		
Dry Density	(pcf)	123.6	124.5	121.9	122.0		

Maximum Dry Density (pcf) **124.5**

Optimum Moisture Content (%) **8.5**



Preparation Method: A

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: PZ

GEOCON

Project No.: W1062-06-02

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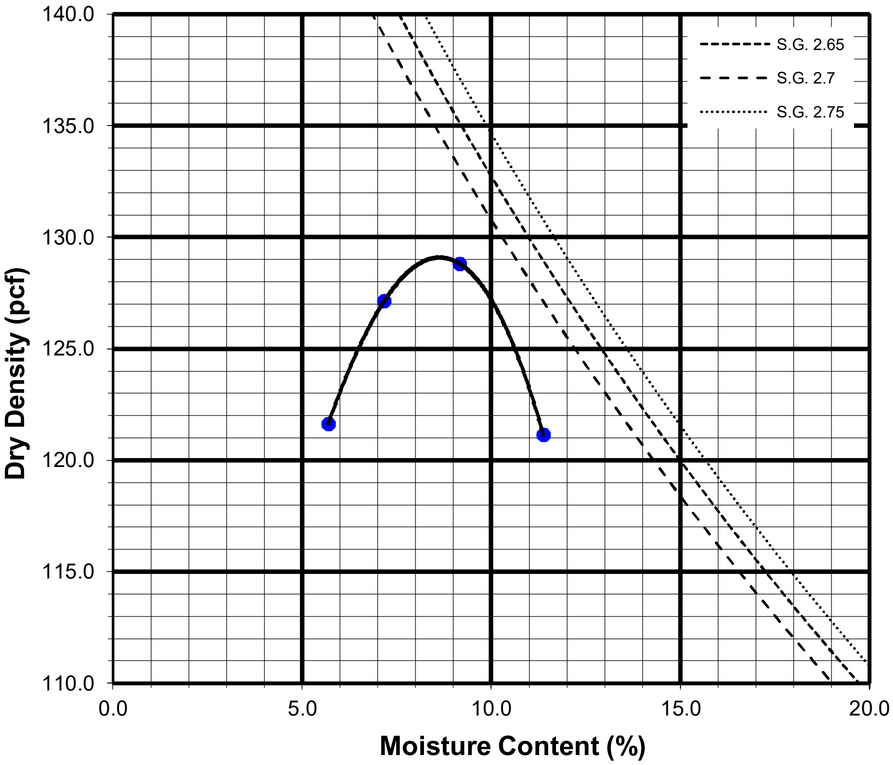
Figure B58

Sample No:

B5@0-5'	Brown Sand (SP)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6416	6350	6330	6234		
Weight of Mold	(g)	4292	4292	4292	4292		
Net Weight of Soil	(g)	2124	2058	2038	1942		
Wet Weight of Soil + Cont.	(g)	2505.2	2470.7	2417.6	2322.2		
Dry Weight of Soil + Cont.	(g)	2326.6	2332.8	2209.4	2217.5		
Weight of Container	(g)	378.5	408.9	378.8	378.4		
Moisture Content	(%)	9.2	7.2	11.4	5.7	0.0	0.0
Wet Density	(pcf)	140.6	136.2	134.9	128.6		
Dry Density	(pcf)	128.8	127.1	121.1	121.6	0.0	0.0

Maximum Dry Density (pcf)	**128.9**	**Optimum Moisture Content (%)**	**8.2**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: PZ

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Project No.: W1062-06-02

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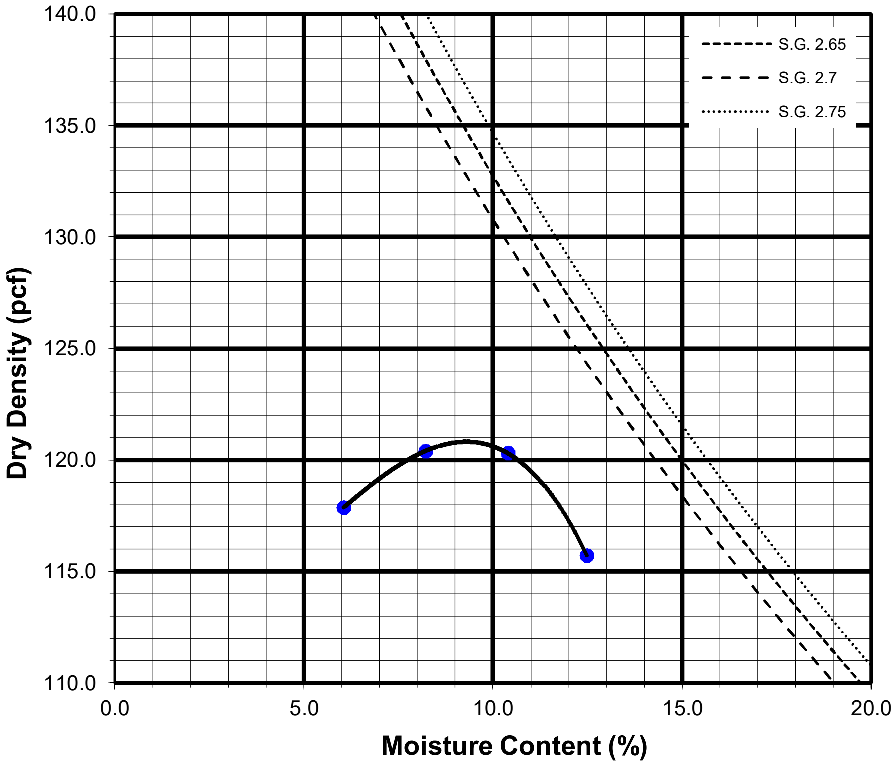
Figure B59

Sample No:

B8@0-5'	Brown Sand (SP)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6258	6298	6260	6180		
Weight of Mold	(g)	4292	4292	4292	4292		
Net Weight of Soil	(g)	1966	2006	1968	1888		
Wet Weight of Soil + Cont.	(g)	2346.2	2387.0	2379.9	2268.5		
Dry Weight of Soil + Cont.	(g)	2127.9	2197.9	2230.4	2160.8		
Weight of Container	(g)	378.3	378.9	410.2	378.6		
Moisture Content	(%)	12.5	10.4	8.2	6.0		
Wet Density	(pcf)	130.2	132.8	130.3	125.0		
Dry Density	(pcf)	115.7	120.3	120.4	117.9		

Maximum Dry Density (pcf)	**121.2**	**Optimum Moisture Content (%)**	**9.3**



Preparation Method: A

GEOCON

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: PZ

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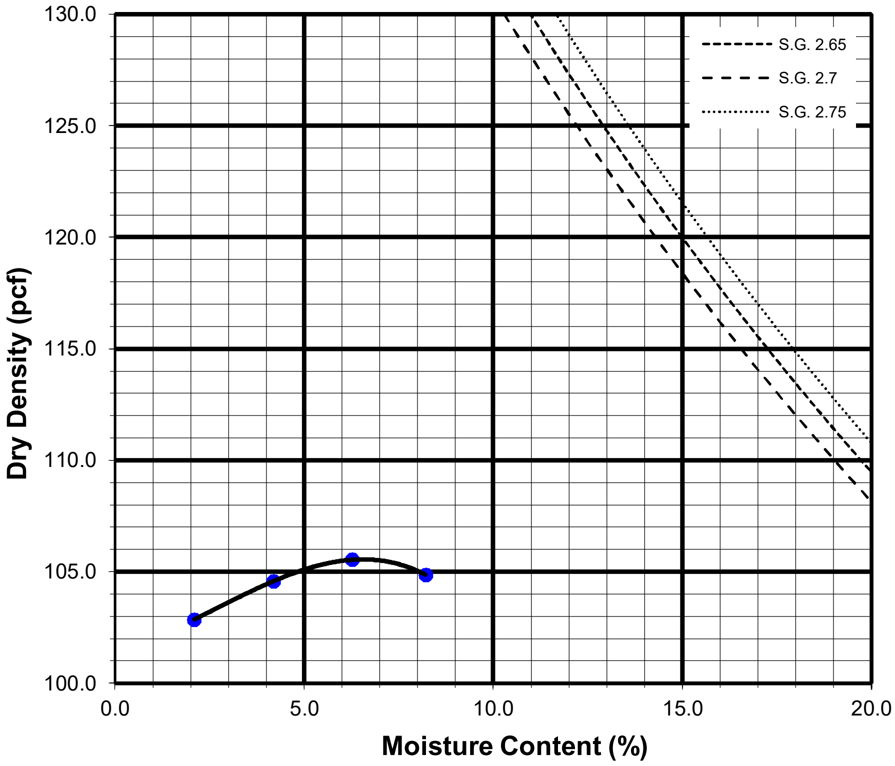
Figure B60

Sample No:

B3@30-35	Light Brown Sand (SP)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	5942	5990	6010	5882		
Weight of Mold	(g)	4296	4296	4296	4296		
Net Weight of Soil	(g)	1646	1694	1714	1586		
Wet Weight of Soil + Cont.	(g)	2019.1	1854.2	2112.6	1931.5		
Dry Weight of Soil + Cont.	(g)	1953.0	1767.2	1983.5	1900.4		
Weight of Container	(g)	377.8	378.6	411.1	409.6		
Moisture Content	(%)	4.2	6.3	8.2	2.1		
Wet Density	(pcf)	109.0	112.1	113.5	105.0		
Dry Density	(pcf)	104.6	105.5	104.9	102.9		

Maximum Dry Density (pcf)	**105.5**	**Optimum Moisture Content (%)**	**6.0**



Preparation Method: A

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: PZ

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Project No.: W1062-06-02

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Figure B61

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS

CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B5@0-5'	8.7	2400 (Moderately Corrosive)
B8@0-5'	8.6	7000 (Moderately Corrosive)
B5@10-15'	7.3	3200 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS

EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B5@0-5'	0.042
B8@0-5'	0.024
B5@10-15'	0.004

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS

CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ$_4$)**	**Sulfate Exposure***
B5@0-5'	0.001	S0
B8@0-5'	0.000	S0
B5@10-15'	0.003	S0

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CORROSIVITY TEST RESULTS

Checked by: PZ

Project No.: W1062-06-02

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Figure B62

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS

CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B6@10-15'	7.2	6300 (Moderately Corrosive)
B2@20	7.8	33000 (Mildly Corrosive)
B3@30-35	9.5	110 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS

EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B6@10-15'	0.001
B2 @ 20	0.156
B3 @ 30-35	0.011

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS

CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B6@10-15'	0.002	S0
B2 @ 20	0.000	S0
B3 @ 30-35	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: PZ

Project No.: W1062-06-02

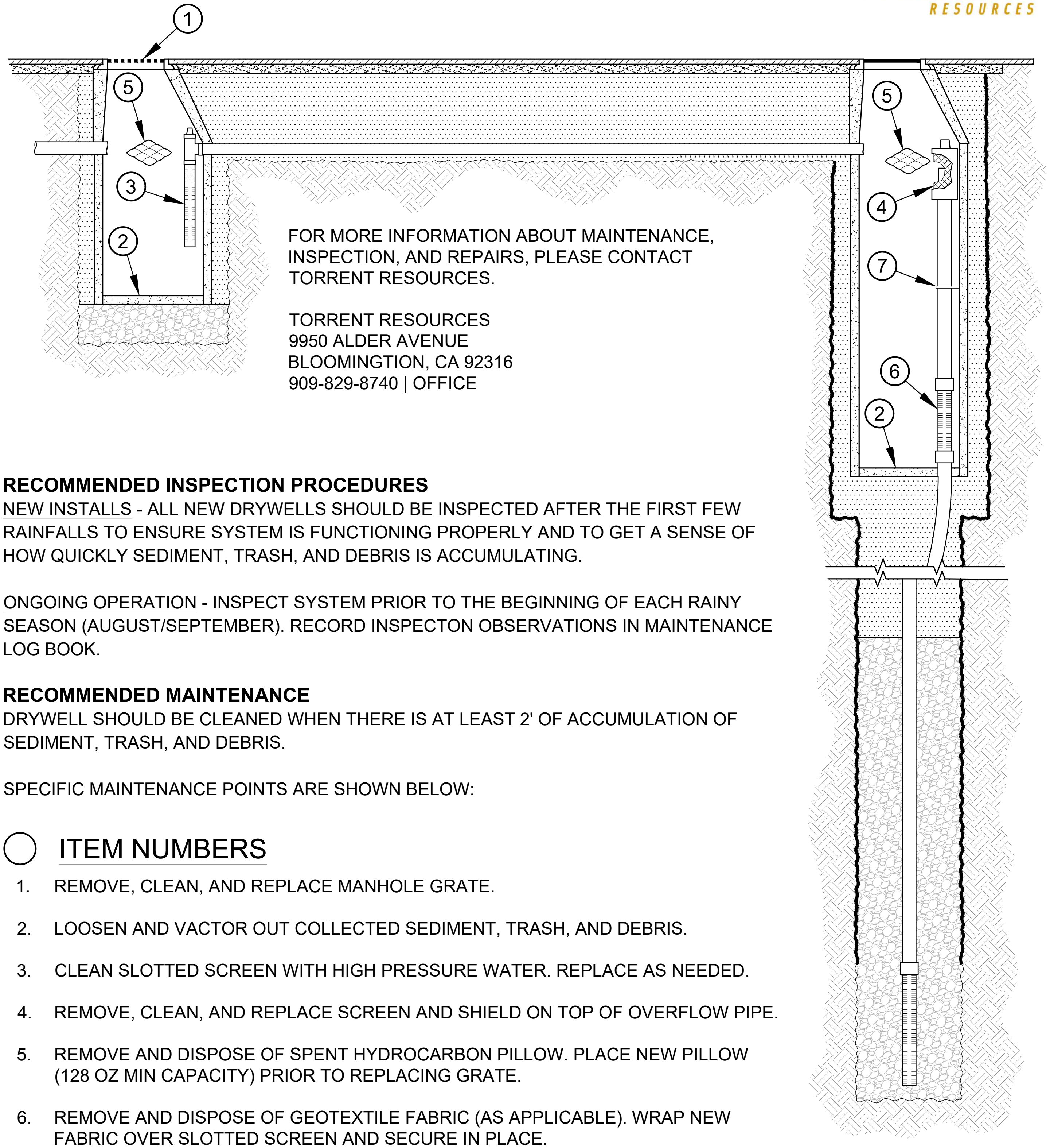
22501 Hawthorne Boulevard
Torrance, California

May 2021

APPENDIX C – MAINTENANCE INFORMATION/SAMPLE COVENANT AND AGREEMENT

INSPECTION AND MAINTENANCE PROCEDURES FOR MAXWELL PLUS DRYWELL

TORRENT®
RESOURCES



FOR MORE INFORMATION ABOUT MAINTENANCE, INSPECTION, AND REPAIRS, PLEASE CONTACT TORRENT RESOURCES.

TORRENT RESOURCES
9950 ALDER AVENUE
BLOOMINGTION, CA 92316
909-829-8740 | OFFICE

RECOMMENDED INSPECTION PROCEDURES

NEW INSTALLS - ALL NEW DRYWELLS SHOULD BE INSPECTED AFTER THE FIRST FEW RAINFALLS TO ENSURE SYSTEM IS FUNCTIONING PROPERLY AND TO GET A SENSE OF HOW QUICKLY SEDIMENT, TRASH, AND DEBRIS IS ACCUMULATING.

ONGOING OPERATION - INSPECT SYSTEM PRIOR TO THE BEGINNING OF EACH RAINY SEASON (AUGUST/SEPTEMBER). RECORD INSPECTON OBSERVATIONS IN MAINTENANCE LOG BOOK.

RECOMMENDED MAINTENANCE

DRYWELL SHOULD BE CLEANED WHEN THERE IS AT LEAST 2' OF ACCUMULATION OF SEDIMENT, TRASH, AND DEBRIS.

SPECIFIC MAINTENANCE POINTS ARE SHOWN BELOW:

○ ITEM NUMBERS

1. REMOVE, CLEAN, AND REPLACE MANHOLE GRATE.
2. LOOSEN AND VACTOR OUT COLLECTED SEDIMENT, TRASH, AND DEBRIS.
3. CLEAN SLOTTED SCREEN WITH HIGH PRESSURE WATER. REPLACE AS NEEDED.
4. REMOVE, CLEAN, AND REPLACE SCREEN AND SHIELD ON TOP OF OVERFLOW PIPE.
5. REMOVE AND DISPOSE OF SPENT HYDROCARBON PILLOW. PLACE NEW PILLOW (128 OZ MIN CAPACITY) PRIOR TO REPLACING GRATE.
6. REMOVE AND DISPOSE OF GEOTEXTILE FABRIC (AS APPLICABLE). WRAP NEW FABRIC OVER SLOTTED SCREEN AND SECURE IN PLACE.
7. CHECK TO MAKE SURE OVERFLOW PIPE BRACKET IS SECURE. TIGHTEN OR ADD SCREWS AS NEEDED

DISPOSE OF WASTE IN ACCORDANCE WITH LOCAL LAWS AND REQUIREMENTS.

ALL MAINTENANCE ACTIVITIES, INSPECTION OBSERVATIONS, AND REPAIRS SHALL BE RECORDED IN THE MAINTENANCE LOG BOOK.

REVISED 4/6/18

Inspection & Maintenance Log

Location: ____

Date	Description of Maintenance Performed/Comments	Performed By

RECORDING REQUESTED BY
AND MAIL TO:

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
BUILDING AND SAFETY DIVISION
900 S. FREMONT AVENUE, 3RD FLOOR
ALHAMBRA, CA 91803-1331

Space above this line is for Recorder's use

COVENANT AND AGREEMENT
REGARDING THE MAINTENANCE OF LOW IMPACT DEVELOPMENT (LID) & NATIONAL POLLUTANTS DISCHARGE ELIMINATION SYSTEM (NPDES) BMPs

The undersigned, ______________________________________ ("Owner"), hereby certifies that it owns the real property described as follows ("Subject Property"), located in the County of Los Angeles, State of California:

LEGAL DESCRIPTION

ASSESSOR'S ID #__________________TRACT NO.__________________LOT NO.________________________

ADDRESS: ____

____

Owner is aware of the requirements of the County of Los Angeles' Green Building Standards Code, Title 31, Section 4.106.4 (LID), and National Pollutant Discharge Elimination System (NPDES) permit. The following post-construction BMP features have been installed on the Subject Property:

☐ Porous pavement
☐ Cistern/rain barrel
☐ Infiltration trench/pit
☐ Bioretention or biofiltration
☐ Rain garden/planter box
☐ Disconnect impervious surfaces
☐ Dry Well
☐ Storage containers
☐ Landscaping and landscape irrigation
☐ Green roof
☐ Other ____

____

The location, including GPS x-y coordinates, and type of each post-construction BMP feature installed on the Subject Property is identified on the site diagram attached hereto as Exhibit 1.

Owner hereby covenants and agrees to maintain the above-described post-construction BMP features in a good and operable condition at all times, and in accordance with the LID/NPDES Maintenance Guidelines, attached hereto as Exhibit 2.

Owner further covenants and agrees that the above-described post-construction BMP features shall not be removed from the Subject Property unless and until they have been replaced with other post-construction BMP features in accordance with County of Los Angeles' Green Building Standards Code, Title 31 and NPDES permit.

Owner further covenants and agrees that if Owner hereafter sells the Subject Property, Owner shall provide printed educational materials to the buyer regarding the post-construction BMP features that are located on the Subject Property, including the type(s) and location(s) of all such features, and instructions for properly maintaining all such features.

Owner makes this Covenant and Agreement on behalf of itself and its successors and assigns. This Covenant and Agreement shall run with the Subject Property and shall be binding upon owner, future owners, and their heirs, successors and assignees, and shall continue in effect until the release of this Covenant and Agreement by the County of Los Angeles, in its sole discretion.

Owner(s):

By:________________________________ Date:________________________________

By:________________________________ Date:________________________________

(PLEASE ATTACH NOTARY)

REFERENCE

PLAN CHECK NO.: __________________________________ DISTRICT OFFICE NO.:______________________

ATTACHMENTS

APPENDIX D – BMP FACT SHEETS

S-1: Storm Drain Message and Signage

Purpose

Waste material dumped into storm drain inlets can adversely impact surface and ground waters. In fact, any material discharged into the storm drain system has the potential to significantly impact downstream receiving waters. Storm drain messages have become a popular method of alerting and reminding the public about the effects of and the prohibitions against waste disposal into the storm drain system. The signs are typically stenciled or affixed near the storm drain inlet or catch basin. The message simply informs the public that dumping of wastes into storm drain inlets is prohibited and/or that the drain ultimately discharges into receiving waters.

General Guidance

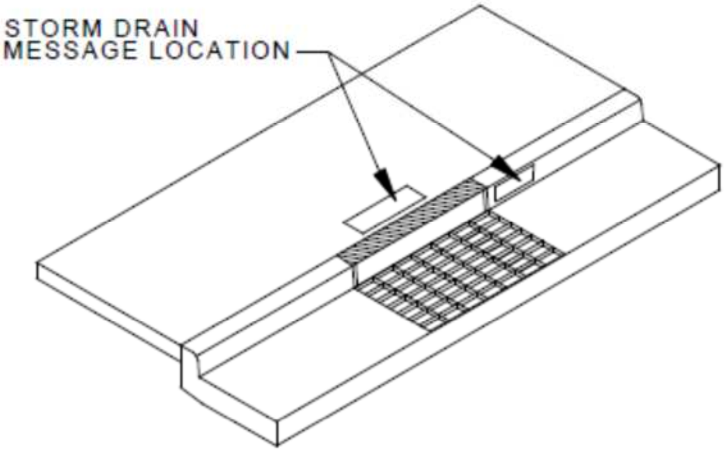
- The signs must be placed so they are easily visible to the public.
- Be aware that signs placed on sidewalk will be worn by foot traffic.

Design Specifications

- Signs with language and/or graphical icons that prohibit illegal dumping, must be posted at designated public access points along channels and streams within the project area. Consult with Los Angeles County Department of Public Works (LACDPW) staff to determine specific signage requirements for channels and streams.
- Storm drain message markers, placards, concrete stamps, or stenciled language/icons (e.g., “No Dumping – Drains to the Ocean”) are required at all storm drain inlets and catch basins within the project area to discourage illegal or inadvertent dumping. Signs should be placed in clear sight facing anyone approaching the storm drain inlet or catch basin from either side (see Figure D-1 and Figure D-2). LACDPW staff should be contacted to determine specific requirements for types of signs and methods of application. A stencil can be purchased for a nominal fee from LACDPW Building and Safety Office by calling (626) 458-3171. All storm drain inlet and catch basin locations must be identified on the project site map.

Maintenance Requirements

Legibility and visibility of markers and signs should be maintained (e.g., signs should be repainted or replaced as necessary). If required by LACDPW, the owner/operator or homeowner’s association shall enter into a maintenance agreement with the agency or record a deed restriction upon the property title to maintain the legibility of placards and signs.



NOTES:

1. STORM DRAIN MESSAGE SHALL BE APPLIED IN SUCH A WAY AS TO PROVIDE A CLEAR, LEGIBLE IMAGE.
2. STORM DRAIN MESSAGE SHALL BE PERMANENTLY APPLIED DURING THE CONSTRUCTION OF THE CURB AND GUTTER USING A METHOD APPROVED BY THE LOCAL AGENCY.

Figure D-1. Storm Drain Message Location – Curb Type Inlet

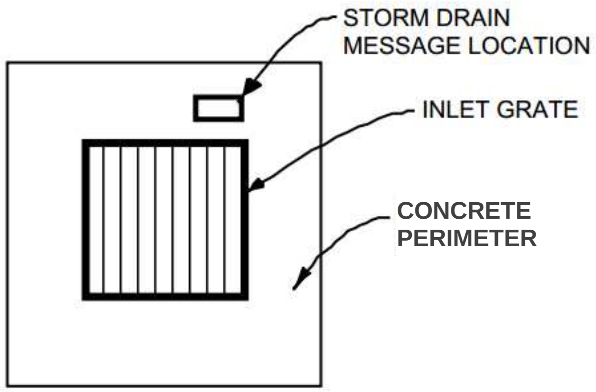


Figure D-2. Storm Drain Message Location – Catch Basin/Area Type Inlet

S-8: Landscape Irrigation Practices

Purpose

Irrigation runoff provides a pathway for pollutants (i.e., nutrients, bacteria, organics, sediment) to enter the storm drain system. By effectively irrigating, less runoff is produced resulting in less potential for pollutants to enter the storm drain system.

General Guidance

- Do not allow irrigation runoff from the landscaped area to drain directly to storm drain system.
- Minimize use of fertilizer, pesticides, and herbicides on landscaped areas.
- Plan sites with sufficient landscaped area and dispersal capacity (e.g., ability to receive irrigation water without generating runoff).
- Consult a landscape professional regarding appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth.

Design Specifications

- Choose plants that minimize the need for fertilizer and pesticides.
- Group plants with similar water requirements and water accordingly.
- Use mulch to minimize evaporation and erosion.
- Include a vegetative boundary around project site to act as a filter.
- Design the irrigation system to only water areas that need it.
- Install an approved subsurface drip, pop-up, or other irrigation system.[1] The irrigation system should employ effective energy dissipation and uniform flow spreading methods to prevent erosion and facilitate efficient dispersion.
- Install rain sensors to shut off the irrigation system during and after storm events.
- Include pressure sensors to shut off flow-through system in case of sudden pressure drop. A sudden pressure drop may indicate a broken irrigation head or water line.
- If the hydraulic conductivity in the soil is not sufficient for the necessary water application rate, implement soil amendments to avoid potential geotechnical hazards (i.e., liquefaction, landslide, collapsible soils, and expansive soils).

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[1] If alternative distribution systems (e.g., spray irrigation) are approved, the County will establish guidelines to implement these new systems.

- For sites located on or within 50 feet of a steep slope (15% or greater), do not irrigate landscape within three days of a storm event to avoid potential geotechnical instability.[2]
- Implement Integrated Pest Management practices.

For additional guidelines and requirements, refer to the Los Angeles County Department of Health Services.

Maintenance Requirements

Maintain irrigation areas to remove trash and debris and loose vegetation. Rehabilitate areas of bare soil. If a rain or pressure sensor is installed, it should be checked periodically to ensure proper function. Inspect and maintain irrigation equipment and components to ensure proper functionality. Clean equipment as necessary to prevent algae growth and vector breeding. Maintenance agreements between LACDPW and the owner/operator may be required. Failure to properly maintain building and property may subject the property owner to citation.

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[2] As determined by the City of Los Angeles, Building and Safety Division

S-9: Building Materials Selection

Purpose

Building materials can potentially contribute pollutants of concern to stormwater runoff through leaching. For example, metal buildings, roofing, and fencing materials may be significant sources of metals in stormwater runoff, especially due to acidic precipitation. The use of alternative building materials can reduce pollutant sources in stormwater runoff by eliminating compounds that can leach into stormwater runoff. Alternative building materials may also reduce the need to perform maintenance activities (i.e., painting) that involve pollutants of concern, and may reduce the volume of stormwater runoff. Alternative materials are available to replace lumber and paving.

Design Specifications

Lumber

Decks and other house components constructed using pressure-treated wood that is typically treated using arsenate, copper, and chromium compounds are hazardous to the environment. Pressure-treated wood may be replaced with cement-fiber or vinyl.

Roofs, Fencing, and Metals

Minimizing the use of copper and galvanized (zinc-coated) metals on buildings and fencing can reduce leaching of these pollutants into stormwater runoff. The following building materials are conventionally made of galvanized metals:

- Metal roofs;
- Chain-link fencing and siding; and
- Metal downspouts, vents, flashing, and trim on roofs.

Architectural use of copper for roofs and gutters should be avoided. As an alternative to copper and galvanized materials, coated metal products are available for both roofing and gutter application. Vinyl-coated fencing is an alternative to traditional galvanized chain-link fences. These products eliminate contact of bare metal with precipitation or stormwater runoff, and reduce the potential for stormwater runoff contamination. Roofing materials are also made of recycled rubber and plastic.

Green roofs may be an option. Green roofs use vegetation such as grasses and other plants as an exterior surface. The plants reduce the velocity of stormwater runoff and absorb water to reduce the volume of stormwater runoff. One potential problem with using green roofs in the Los Angeles County area is the long, hot and dry summers, which may kill the plants if they are not watered. See the Green Roof Fact Sheet (RET-7) in Appendix E.

Pesticides

The use of pesticides around foundations can be reduced through the use of alternative barriers. Sand barriers can be applied around foundations to deter termites, as they cannot tunnel through sand. Metal shields also block termites from tunneling. Additionally, diatomaceous earth can be used to repel or kill a wide variety of other pests.

Maintenance Requirements

The integrity of structural elements that are subject to damage (e.g., signs) must be maintained by the owner/operator as required by local codes and ordinances. Maintenance agreements between LACDPW and the owner/operator may be required. Failure to properly maintain building and property may subject the property owner to citation.

RET-4: Dry Well

Description

A dry well is a bored, drilled, or driven shaft or hole whose depth is greater than its width. A dry well may either be a small excavated pit filled with aggregate or a prefabricated storage chamber or pipe segment. Dry well design and function are similar to infiltration trenches in that they are designed to temporarily store and subsequently infiltrate stormwater runoff. In particular, dry wells can be used to reduce the volume of stormwater runoff from building roofs. While generally not a significant source of stormwater runoff pollution, roofs are one of the most important sources of new or increased stormwater runoff volume from land development sites. Dry wells can be used to indirectly enhance water quality by reducing the volume of stormwater runoff to be treated by other downstream stormwater quality control measures.

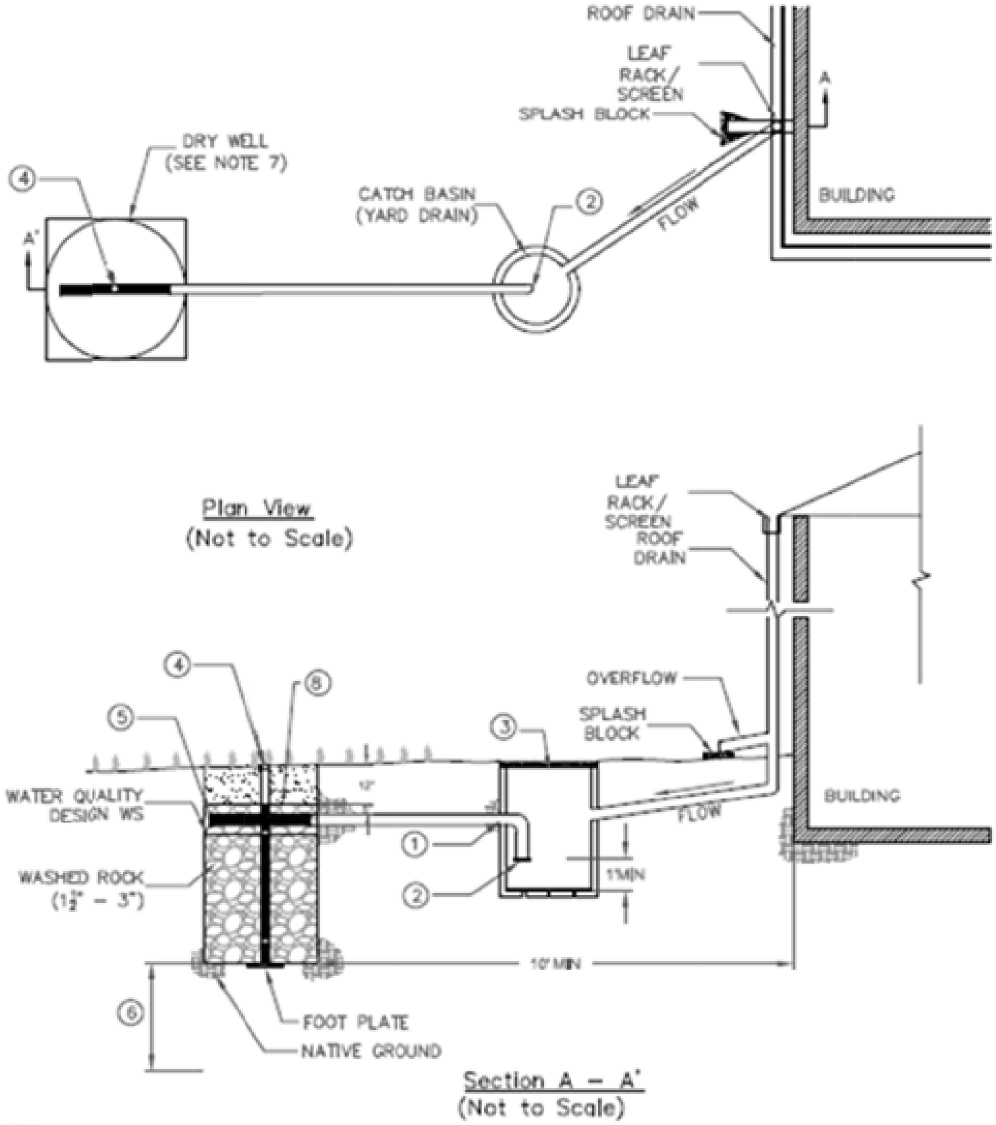
A schematic of a typical dry well is presented in Figure E-4.

LID Ordinance Requirements

Dry wells can be used to meet the on-site retention requirements of the LID Ordinance. Dry wells will prevent pollutants in the SWQDv from being discharged off-site.

Advantages

- Requires minimal space to install
- Low installation costs
- Provide groundwater recharge
- Reduces peak stormwater runoff flows during small storm events



NOTES:

① MINIMUM 4" – 6" DIAMETER PVC PIPE. INSTALL AT FLAT SLOPE.

② INSTALL FINE MESH SCREEN AT INLET TO DRY WELL. SET INLET ELEVATION AT 1' MINIMUM ABOVE CATCH BASIN BOTTOM.

③ CATCH BASIN (YARD DRAIN) INSTALLED WITH A SOLID LID FLUSH WITH GROUND SURFACE.

④ 4–6" VERTICAL PERFORATED PVC INSPECTION WELL WITH SCREW LID (NUT DOWN) FLUSH WITH GROUND SURFACE.

⑤ CAP END OF 4–6" HORIZONTAL PERFORATED PVC DISPERSION PIPE.

⑥ MINIMUM 10' ABOVE SEASONAL HIGH GROUNDWATER TABLE AND 3' ABOVE BEDROCK.

⑦ DRY WELL CONFIGURATION MAY VARY (E.G. PRE-FAB MAY BE CIRCULAR).

⑧ CHOKING STONE LAYER SHALL BE PLACED ON TOP OF THE DRY WELL TO SEPARATE IT FROM THE TOPSOIL AND PREVENT CLOGGING.

Figure E-4. Dry Well Schematic

Disadvantages

- Is not appropriate for areas with low permeability soils or high groundwater levels
- May not be appropriate for industrial sites or locations with contaminated soils or where spills may occur because of the potential threat to groundwater contamination
- Cannot receive untreated stormwater runoff except from rooftops
- Requires complete reconstruction for failed dry wells
- Is not suitable for fill sites or on steep slopes

General Constraints and Implementation Considerations

- Dry wells can be integrated into open space buffers and other landscape areas.
- The potential for groundwater contamination must be carefully considered,. Dry wells are not suitable for sites that:
  - Use or store chemicals or hazardous materials, unless they are prevented from entering the well; or
  - Un-remediated “brownfield sites” where there is known groundwater or soil contamination
- Dry wells should be sited away from tree drip lines and kept free of vegetation.
- If the corrected in-situ infiltration rate exceed 2.4 in/hr, then stormwater runoff may need to be fully-treated with an upstream stormwater quality control measure prior to infiltration to protect groundwater quality.
- Dry wells cannot be located on sites with a slope greater than 20 percent (5:1).
- Pretreatment to remove sediment is required to protect dry wells from high sediment loads.
- If a yard drain is proposed as part of the design, it must be designed so that any standing water in the catch basin will infiltrate within 96 hours.
- If possible, the entire tributary area of the dry well should be stabilized before construction begins. If this is not possible, all flows should be diverted around the dry well to protect it from sediment loads during construction or the top two inches of soil from the dry well bottom should be removed after the site has been stabilized. Excavated material should be stored such that it cannot be washed back into the dry well if a storm occurs during construction.
- The equipment used to construct the dry well should have extra wide low-pressure tires. Construction traffic should not enter the dry well because it can compact soil, which reduces infiltration capacity. If heavy equipment is used on the base of the dry well, the infiltrative capacity may be restored by tilling or aerating prior to placing the infiltrative bed.

- Clean, washed gravel should be placed in the excavated dry well in lifts and lightly compacted with a plate compactor. Use of unwashed gravel can result in clogging.
- A geomembrane liner should be installed generously with overlapping seams on sides, bottom, and one foot below the surface of the dry well.
- Once construction is complete, stabilize the entire tributary area to the dry well before allowing stormwater runoff to enter it.
- An observation well must be installed to check water levels, retention time, and evidence of clogging.
- Accessibility for maintenance during dry and wet weather conditions must be provided.

Design Specifications

The following sections provide design specifications for dry wells.

Geotechnical

Due to the potential to contaminate groundwater, cause slope instability, impact surrounding structures, and potential for insufficient infiltration capacity, an extensive geotechnical site investigation must be conducted during the site planning process to verify site suitability for a dry well. All geotechnical investigations must be performed according to the most recent GMED Policy GS 200.1. Soil infiltration rates and the groundwater table depth must be evaluated to ensure that conditions are satisfactory for proper operation of a dry well. The project applicant must demonstrate through infiltration testing, soil logs, and the written opinion of a licensed civil engineer that sufficiently permeable soils exist on-site to allow the construction of a properly functioning dry well.

Dry wells are appropriate for soils with a minimum corrected in-situ infiltration rate of 0.3 in/hr. The geotechnical report must determine if the proposed project site is suitable for a dry well and must recommend a design infiltration rate (see “Design Infiltration Rate” under the “Sizing” section). The geotechnical investigation should be such that a good understanding is gained as to how the stormwater runoff will move through the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water.

Pretreatment

Pretreatment is important for all structural stormwater quality control measures, but it is particularly important for retention facilities. Pretreatment refers to design features that provide settling of large particles before stormwater runoff enters a stormwater quality control measure in order to reduce the long-term maintenance burden. Pretreatment should be provided to reduce the sediment load entering a dry well in order to maintain the infiltration rate of the dry well. To ensure that dry wells are effective, the project applicant must incorporate pretreatment devices that provide sediment reduction (e.g., vegetated swales, vegetated filter strips, sedimentation manholes, and proprietary devices).

Setbacks

Dry wells must be sited following the setbacks from the most recent GMED Policy GS 200.1.

Geometry

- Dry well configurations vary, but generally have length and width top dimensions close to a square. Prefabricated dry wells are often circular.
- The filter bed media layers must have the following composition and thickness, unless they are prefabricated dry wells:
    - Top layer: 2 inches of pea gravel
    - Middle layer: 3 to 5 feet of washed 2- to 6-inch gravel; void spaces should be approximately 30 to 40 percent
    - Bottom layer: 6 inches of sand or geomembrane liner equivalent.
- Gravel media and prefabricated dry wells have porosities of 30 to 40 percent and 80 to 95 percent, respectively.
- If a dry well receives stormwater runoff from an underground pipe (i.e., stormwater runoff does not enter the top of the dry well from the ground surface), a fine mesh screen should be installed at the inlet. The inlet elevation should be 18 inches below the ground surface (i.e., below 12 inches of surface soil and 6 inches of dry well media).

Sizing

Dry wells are sized using a simple sizing method where the SWQDv must be completely infiltrated within 96 hours. Dry wells provide stormwater runoff storage in the voids of the rock fill.

Step 1: Determine the SWQDv

Dry wells must be designed to capture and retain the SWQDv (see Section 6 for SWQDv calculation procedures).

Step 2: Determine the design infiltration rate

Determine the corrected in-situ infiltration rate (f_{design}) of the native soil using the procedures described in the most recent GMED Policy GS 200.1.

Step 3: Calculate the surface area

Determine the required size of the infiltration surface by assuming the SWQDv will fill the available void spaces of the gravel storage layer. The maximum depth of stormwater runoff that can be infiltrated within the maximum retention time (96 hrs) is calculated using the following equation:

$$d_{max} = \frac{f_{design}}{12} \times t$$

Where:

d_{max} = Maximum depth of water that can be infiltrated within the required drawdown time [ft];
f_{design} = Design infiltration rate [in/hr]; and
t = Maximum retention time (max 96 hrs) [hr].

Select the dry well depth (d_t) such that:

$$d_t \leq \frac{d_{max}}{n_t}$$

Where:

d_t = Depth of dry well fill [ft];
d_{max} = Maximum depth of water that can be infiltrated within the maximum retention time [ft]; and
n_t = Dry well fill porosity.

Calculate the infiltrating surface area (bottom of the dry well) required:

$$A = \frac{SWQDv}{d_t \times n_t}$$

Where:

A = Surface area of the bottom of the dry well [ft^2];
SWQDv = Stormwater quality design volume [ft^3]; and
d_t = Depth of dry well fill [ft]; and
n_t = Dry well fill porosity.

Flow Entrance and Energy Dissipation

Energy dissipation controls, constructed of sound materials such as stones, concrete, or proprietary devices that are rated to withstand the energy of the influent flow, must be installed at the inlet to the dry well. Consult with LACDPW for the type and design of energy dissipation structure.

Drainage

The specifications for designing drainage systems for dry wells are presented below:

- The bottom of dry well must be native soil that is over-excavated at least one foot in depth with the soil replaced uniformly without compaction. Amending the excavated soil with two to four inches (~15 to 30 percent) of coarse sand is recommended.
- The use of vertical piping, either for distribution or infiltration enhancement, is prohibited. This application may be classified as a Class V Injection Well per 40 CFR Part 146.5(e)(4).
- The infiltration capacity of the subsurface layers should be sufficient to ensure a maximum retention time of 96 hours. An observation well must be installed to allow observation of retention time.

Hydraulic Restriction Layer

The entire infiltrative area, including the side walls must lined with a geomembrane liner to prevent soil from migrating into the top layer and reducing storage capacity. The specifications of the geomembrane liner are presented in Table E-7. The entire well area, including the sides, must be lined with a geomembrane liner prior to placing the media bed. Provide generous overlap at the seams.

Table E-7. Geomembrane Liner Specifications for Dry Wells

Parameter	Test Method	Specifications
Material		Nonwoven geomembrane liner
Unit weight		8 oz/yd^3 (minimum)
Filtration rate		0.08 in/sec (minimum)
Puncture strength	ASTM D-751 (Modified)	125 lbs (minimum)
Mullen burst strength	ASTM D-751	400 lb/in^2 (minimum)
Tensile strength	AST D-1682	300 lbs (minimum)
Equiv. opening size	US Standard Sieve	No. 80 (minimum)

Observation Well

The observation well is a vertical section of perforated PVC pipe, four- to six-inch diameter, installed flush with the top of the dry well on a footplate and with a locking, removable cap. The observation well is needed to monitor the infiltration rate in dry well and is useful for marking the location of the dry well.

Vegetation

- Dry wells must be kept free of vegetation.

- Trees and other large vegetation should be planted away from dry well such that drip lines do not overhang the infiltration area.

Restricted Construction Materials

Use of pressure-treated wood or galvanized metal at or around a dry well is prohibited.

Maintenance Access

The dry well must be safely accessible during wet and dry weather conditions if it is publicly-maintained. If the dry well becomes plugged and fails, access is needed to excavate the dry well and replace the filter bed media. To prevent damage and compaction, access must able to accommodate a backhoe working at “arm’s length” from the dry well.

Maintenance Requirements

Maintenance and regular inspections are important for proper function of dry wells. The following are general maintenance requirements:

- Conduct regular inspection and routine maintenance for pretreatment devices.
- Inspect dry well and its observation well frequently to ensure that water infiltrates into the subsurface completely within maximum retention time of 96 hours. If water is present in the observation well more than 96 hours after a major storm, the dry well may be clogged. Maintenance activities triggered by a potentially clogged facility include:
    - Check for debris/sediment accumulation and remove sediment (if any) and evaluate potential sources of sediment and vegetative or other debris (e.g., embankment erosion, channel scour, overhanging trees, etc). If suspected upstream sources are outside of the County's jurisdiction, additional pretreatment operations (e.g., trash racks, vegetated swales, etc.) may be necessary.
    - Assess the condition of the top aggregate layer for sediment buildup and crusting. Remove the top layer of pea gravel and replace. If slow draining conditions persist, the entire dry well may need to be excavated and replaced.
- Eliminate standing water to prevent vector breeding.
- Remove and dispose of trash and debris as needed, but at least prior to the beginning of the wet season.

A summary of potential problems that may need to be addressed by maintenance activities is presented in Table E-8.

The County requires execution of a maintenance agreement to be recorded by the property owner for the on-going maintenance of any privately-maintained stormwater quality control measures. The property owner is responsible for compliance with the maintenance agreement. A sample maintenance agreement is presented in Appendix H.

Table E-8. Dry Well Troubleshooting Summary

Problem	Conditions When Maintenance Is Needed	Maintenance Required
Trash and Debris	Trash and debris > 5 ft^3/1,000 ft^2	Remove and dispose of trash and debris.
Contaminants and Pollution	Any evidence of oil, gasoline, contaminants, or other pollutants	Remove any evidence of visual contamination.
Erosion/Sediment Accumulation	Undercut or eroded areas at inlet structures	Repair eroded areas and re-grade if necessary.
	Accumulation of sediment, debris, and oil/grease in pretreatment devices	Remove sediment, debris, and/or oil/grease.
	Accumulation of sediment, debris, and oil/grease on surface or inlet	Remove sediment, debris, and/or oil/grease.
Water Drainage Rate	Standing water, or by inspection of observation wells	Remove the top layer of the dry well bottom and replace if necessary.

APPENDIX E – LID PLAN

LEGEND

▬ WATER QUALITY TREATMENT AREA/DRAINAGE BOUNDARY

—SD— PROPOSED LOW FLOW STORM DRAIN LINE

▦ PROPOSED UNDERGROUND STORAGE SYSTEM FOR WQ VOLUME (CMP STORAGE OR SIMILAR)

● PRIMARY SETTLING CHAMBER ● DRYWELL TORRENT MAXWELL PLUS DRYWELL SYSTEM WITH PRIMARY SETTLING CHAMBER

● PRELIMINARY STORM FLOW OUTLET LOCATION INTO STREET GUTTER

→ DIRECTION OF LOW (LOW FLOW PIPE). LARGE STORM FLOW WILL OUTLET INTO HAWTHORNE BLVD AND/OR W. 226TH ST.

GRAPHIC SCALE
20 0 10 20 40
(IN FEET)
1 inch = 20 ft.

P:\278.737\SUSMP\737-PREL-SUSMP.dwg
PLOT DATE - Dec 09, 2021 - 4:22PM

PLAN SET: A
DATE: 12-1-21

ENGINEER'S SEAL

TERRY P. AU
C 68466
EXP. 9-30-23

PREPARED BY:

URBAN RESOURCE
CONSULTING CIVIL ENGINEERS

2923 SATURN STREET, UNIT H
BREA, CA 92821
PHONE: 949-727-9095

PRELIMINARY LID PLAN FOR TRACT 454 LOT 11 22501 HAWTHORNE BLVD, TORRANCE

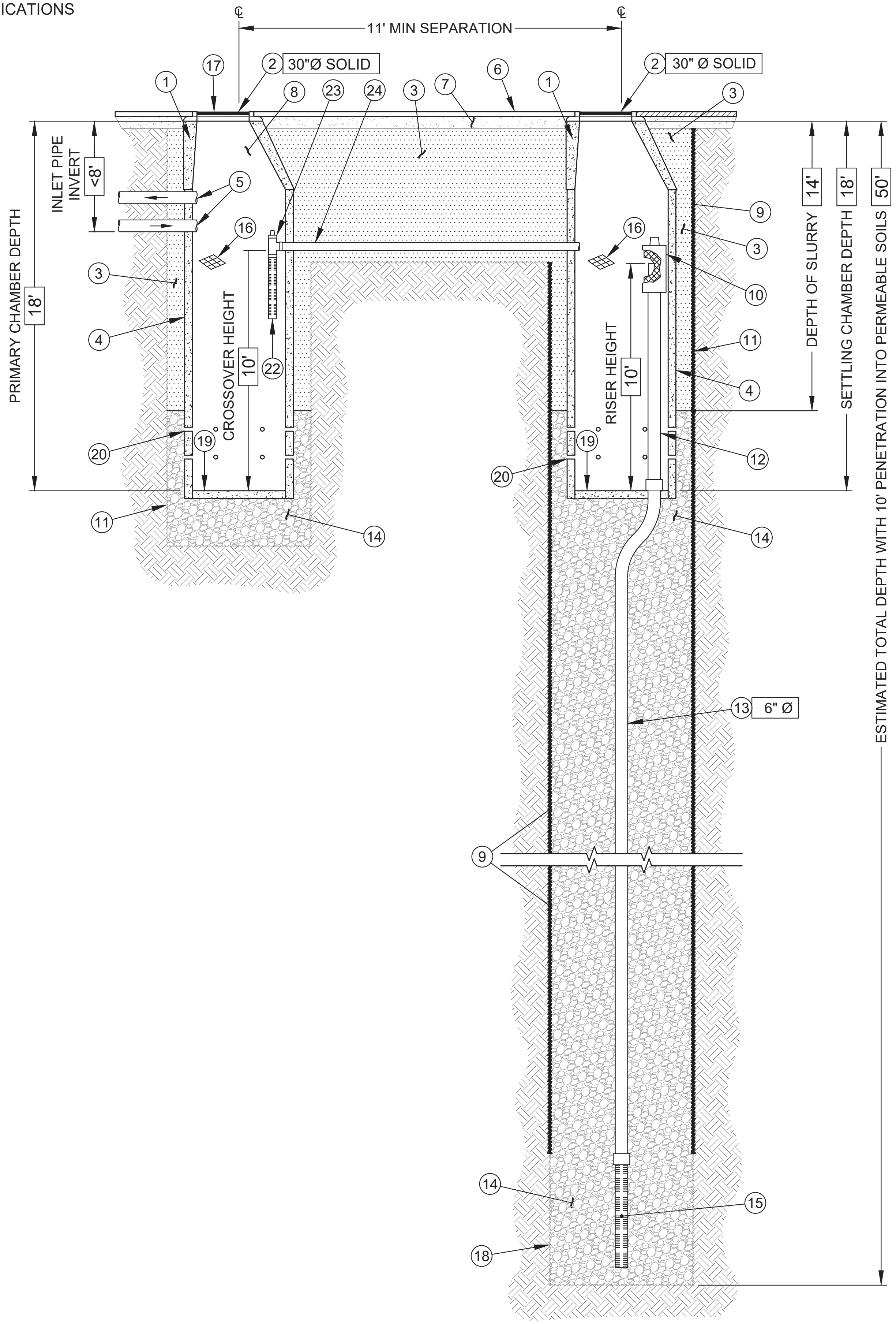
PLAN CONTROL
INTRACORP

The MaxWell® Plus

DRAINAGE SYSTEM DETAILS AND SPECIFICATIONS

Torrance Apartments

Torrance, CA



○ ITEM NUMBERS

1. **MANHOLE CONE** - MODIFIED FLAT BOTTOM.
2. **BOLTED RING & GRATE/COVER** - DIAMETER & TYPE AS SHOWN. CLEAN CAST IRON WITH WORDING **"STORM WATER ONLY"** IN RAISED LETTERS. **BOLTED IN 2 LOCATIONS** AND SECURED TO CONE WITH MORTAR. RIM ELEVATION ±0.02' OF PLANS.
3. **STABILIZED BACKFILL** - TWO-SACK SLURRY MIX.
4. **PRE-CAST LINER** - 4000 PSI CONCRETE 48" ID. X 54" OD. **CENTER IN HOLE** AND ALIGN SECTIONS **TO MAXIMIZE BEARING SURFACE.**
5. **INLET PIPE/OUTLET PIPE (BY OTHERS). SEE SEPARATE PLAN FOR INVERT ELEVATIONS.**
6. **GRADED BASIN OR PAVING** (BY OTHERS).
7. **COMPACTED BASE MATERIAL**, IF REQUIRED (BY OTHERS).
8. **FREEBOARD DEPTH VARIES** WITH INLET PIPE ELEVATION. INCREASE PRIMARY AND SECONDARY CHAMBER DEPTHS AS NEEDED TO MAINTAIN ALL INLET PIPE ELEVATIONS ABOVE RISER PIPE.
9. **NON-WOVEN GEOTEXTILE SLEEVE** - MIRAFI 140 NL. MIN. 6 FT Ø. HELD APPROX. 10 FEET OFF THE BOTTOM OF EXCAVATION.
10. **PUREFLO® DEBRIS SHIELD** - ROLLED 16 GA. STEEL X 24" LENGTH WITH VENTED ANTI-SIPHON AND INTERNAL 0.265" MAX. SWO FLATTENED EXPANDED STEEL SCREEN X 12" LENGTH. **FUSION BONDED EPOXY COATED**.
11. **MIN. 6' Ø DRILLED SHAFT.**
12. **RISER PIPE** - SCH. 40 PVC MATED TO DRAINAGE PIPE AT BASE SEAL.
13. **DRAINAGE PIPE** - ADS HIGHWAY GRADE OR SCH. 40 PVC WITH TRI-A COUPLER. SUSPEND PIPE DURING BACKFILL OPERATIONS. DIAMETER AS NOTED.
14. **ROCK** - WASHED, SIZED BETWEEN 3/8" AND 1-1/2".
15. **FLOFAST® DRAINAGE SCREEN** - SCH. 40 PVC 0.120" SLOTTED WELL SCREEN WITH 32 SLOTS PER ROW/FT. OVERALL LENGTH VARIES, UP TO 120" WITH TRI-B COUPLER.
16. **ABSORBENT** - HYDROPHOBIC PETROCHEMICAL SPONGE. MIN. 128 OZ. CAPACITY. TYPICAL, 2 PER CHAMBER.
17. **FABRIC SEAL** - U.V. RESISTANT GEOTEXTILE - **TO BE REMOVED BY CUSTOMER** AT PROJECT COMPLETION. GRATED ONLY.
18. **MIN. 6' Ø DRILLED SHAFT.**
19. **BASE SEAL** - CONCRETE SLURRY.
20. **6 PERFORATIONS MINIMUM PER FOOT, 2 ROWS MINIMUM.**
21. NOT USED.
22. **INTAKE SCREEN** - 4" Ø SCH. 40 PVC 0.120" MODIFIED SLOTTED WELL SCREEN WITH 32 SLOTS PER ROW/ FT. 48" OVERALL LENGTH WITH TRI-CEND CAP.
23. **VENTED ANTI-SIPHON INTAKE WITH FLOW REGULATOR.**
24. **CONNECTOR PIPE** - 4" Ø SCH. 40 PVC.

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