Appendix H-1
Geotechnical Report

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

January 31, 2025

EMI Project No. 23-137

Johnson Favaro
5898 Blackwelder Street
Culver City, CA 90232

Attention: Mr. Steve Johnson

Subject: **Geotechnical Report**
Oceanside Beachfront Phase II Project
Oceanside, California

Dear Mr. Johnson:

Attached is our Geotechnical Report for the proposed Beachfront Phase II projects in the City of Oceanside. This report summarizes the geotechnical investigation performed for the project improvements and summarizes the findings, conclusions and recommendations for the design and construction of the geotechnical improvements on the project. The subject report is intended to support the preliminary design submittal for the project. The recommendations provided herein will need to be verified during final design.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions, please do not hesitate to contact us.

Sincerely,

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GEOTECHNICAL REPORT
OCEANSIDE BEACHFRONT PHASE II PROJECT
OCEANSIDE, CALIFORNIA

Prepared for:

Johnson Favaro
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Culver City, CA 90232

Prepared by:

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EMI Project No. 23-137

January 31, 2025

TABLE OF CONTENTS

Section Page

1.0 INTRODUCTION 1

2.0 SITE CONDITIONS 1

3.0 PROJECT DESCRIPTION 3

3.1 Junior Seau Beach Community Center (JSBCC) 3
3.2 Junior Seau Amphitheater and Bandshell 3
3.3 Beachfront Park, Recreation Center and Parking (Rec Center) 4

4.0 GEOTECHNICAL INVESTIGATION 5

5.0 GEOTECHNICAL CONDITIONS 8

5.1 Site Geology 8
5.2 Coastal Bluff Surface Conditions 8
5.3 Seismicity 10
5.4 Tsunami 10
5.5 Subsurface Conditions 12

6.0 GROUNDWATER CONDITIONS 14

7.0 COASTAL BLUFF EROSION EVALUATION 15

8.0 CORROSION EVALUATION 16

9.0 SEISMIC INFORMATION 17

9.1 Seismic Design for Buildings 17
9.2 Liquefaction Potential 18
9.3 Seismically-Induced Settlement 18
9.4 Seismic Slope Instability 18
9.5 Fault Rupture 18

10.0 FOUNDATION RECOMMENDATIONS 19

10.1 Retaining Wall Earth Pressures and Surcharges 19
10.1.1 Lateral Active Earth Pressures 19
10.1.2 Pile Surcharge Load 19
10.2 Spread Footings 21
10.2.1 Allowable Bearing 21
10.2.2 Sliding Resistance 22
10.3 Slab on Grade 22
10.4 Pile Foundations – Drilled Shafts 23
10.4.1 Axial Pile Capacity 23
10.4.2 Lateral Pile Solutions for Drilled Shafts 25
10.5 Global Slope Stability 30

11.0 PAVEMENTS 31

11.1 Subgrade R-Values 31
11.2 Pavement Structural Sections 31
11.3 Pavement Subgrade 31

11.4 Pavement Materials and Construction 32
12.0 CONSTRUCTION CONSIDERATIONS 33
12.1 Construction Advisories 33
12.2 Groundwater Control 33
12.3 Site Preparation 34
12.4 Foundation and Slab Subgrade Preparation 34
12.5 Drilled Shaft Construction Considerations 35
12.6 Engineered Fill Materials and Placement 36
12.7 Grading and Drainage 36
12.8 Retaining Wall Backfill and Drainage 37
12.9 Utility Trenches 37
12.10 Exterior Slab Design Considerations 37
12.11 Geotechnical Observation and Testing 38
13.0 REFERENCES 39

TABLES

Page
Table 1. Exploratory Boring Information 5
Table 2. Idealized Soil Profile 12
Table 3. Groundwater Monitoring Well Readings 14
Table 4. Soil Corrosion Test Results 16
Table 5. Site Coordinates, Shear Wave Velocity, and Peak Ground Acceleration 17
Table 6. Earth Pressure Recommendations 19
Table 7. Spread Footing Data Table 21
Table 8. Lateral Pile Solutions for Free-Head Loading Conditions 25
Table 9. Subgrade R-Values 31
Table 10. Recommended Pavement Structural Sections 31
Table 11. Engineered Fill Gradation Requirements 36

FIGURES

Page
Figure 1. Site Location Map 2
Figure 2. Boring Location Map 7
Figure 3. Regional Geologic Map 9
Figure 4. Tsunami Hzard Map 11
Figure 5. Pile Load Surcharge against Proposed Retaining Walls 20
Figure 6. Allowable Axial Pile Capacities 24
Figure 7. Lateral Pile Solution for 24-in Drilled Shafts – Longitudinal Direction 26
Figure 8. Lateral Pile Solution for 24-in Drilled Shafts – Transverse Direction 27
Figure 9. Lateral Pile Solution for 36-in Drilled Shafts – Longitudinal Direction 28
Figure 10. Lateral Pile Solution for 36-in Drilled Shafts – Transverse Direction 29

APPENDIX

Appendix A. Boring Logs
Appendix B. Laboratory Testing

1.0 INTRODUCTION

This Geotechnical Report presents the findings and conclusions of a geotechnical study conducted by Earth Mechanics, Inc. (EMI) for the development of the Oceanside Beachfront Phase II project located at the intersection of Mission Avenue and North Pacific Street in Oceanside, California. The purpose of the geotechnical study was to obtain information on subsurface soils and conditions and develop design and construction recommendations to assist Johnson Favaro in preparing the Preliminary Design Drawings for the project.

Geotechnical engineering services provided for this project included the following tasks:

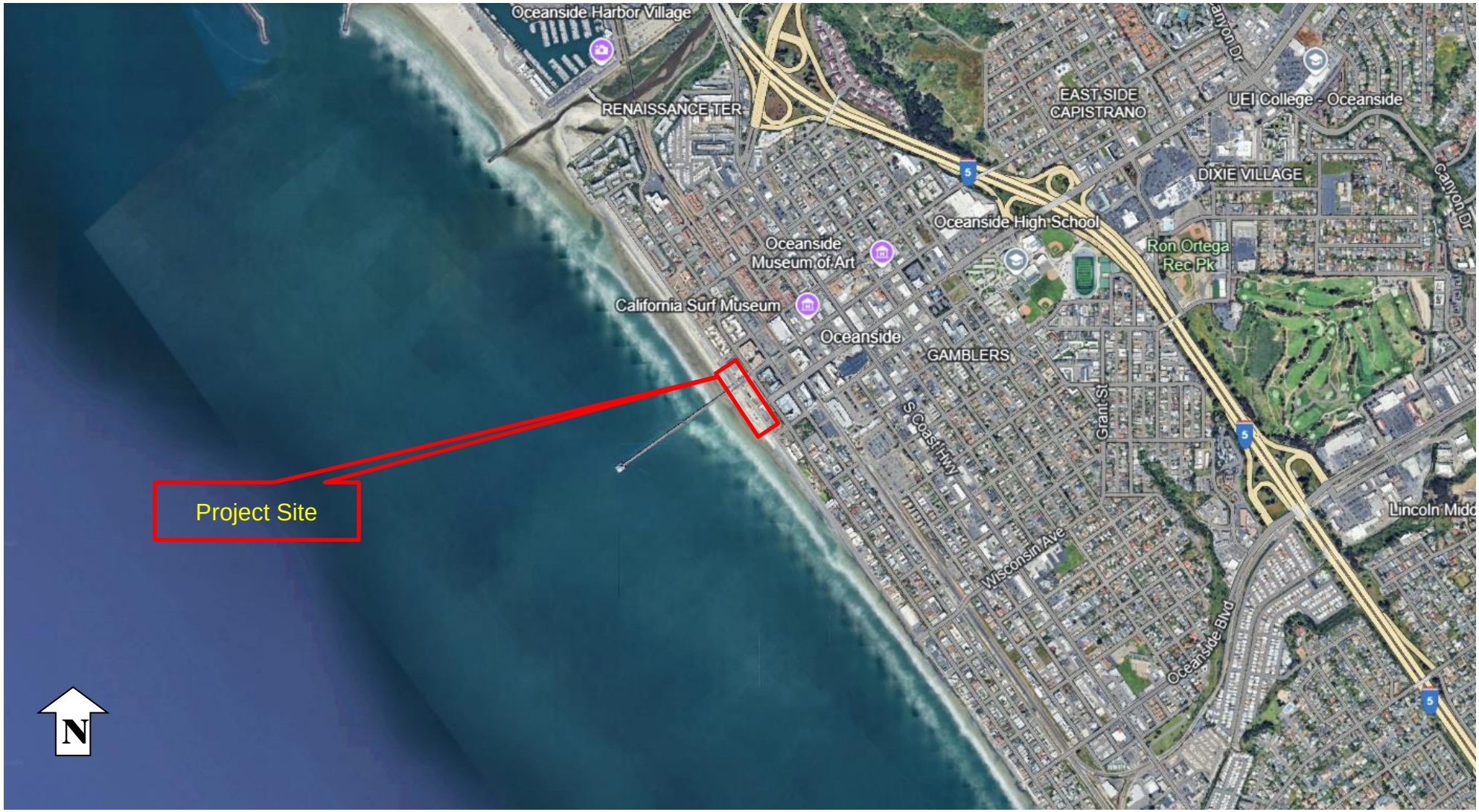
- Collection and review of existing geotechnical information.
- Geotechnical field exploration.
- Laboratory testing of selected subsurface soil samples.
- Seismic evaluation of project site.
- Engineering analysis to develop design and construction recommendations for structure foundations, floor slabs and retaining walls.
- Development of pavement structural sections and construction recommendations.
- Development of recommendations for site preparation and earthwork.
- Preparation of this report presenting our findings, conclusions, and recommendations.

All elevations provided in this report are based on the NAVD 88 datum unless stated otherwise.

2.0 SITE CONDITIONS

The proposed site is located along the waterfront near the intersection of Mission Avenue and North Pacific Street in Oceanside, California. The site is at the base of an approximately 30 ft high bluff that parallels North Pacific Street. The project area is bounded on the east by North Pacific Street, on the west by The Strand, the south by 1st Street and the north by the northern parking lot of the existing Junior Seau Beach Community Center. The Oceanside Pier Bridge (Pier Bridge) runs through the middle of the project between North Pacific Street and the Oceanside Pier. The project location is shown in Figure 1.

The existing project site consists of the Junior Seau Beach Community Center to the north of the Oceanside Pier Bridge, the Oceanside Pier Amphitheater and a surface parking lot located on the south side of the Oceanside Pier Bridge. There are several single-story, temporary and permanent structures for maintenance equipment, restrooms and retail shops are located intermittently throughout the project area surrounding the Pier Bridge and amphitheater. The existing ground surface is at an approximate elevation of +11 to +15 ft along the beachfront. North Pacific Street is at an approximate elevation of +40 to +42 ft at the crest of the bluffs on the eastern side of the project.



Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	**Oceanside Beachfront Phase II Project** Project No. 23-137	**SITE LOCATION MAP** Date: January 2025

Figure 1

3.0 PROJECT DESCRIPTION

Project improvements are proposed generally in three different locations;

- Junior Seau Beach Community Center (JSBCC)
- Junior Seau Pier Amphitheater and Bandshell (Amphitheater)
- Beachfront Park, Recreation Center and Parking (Rec Center)

The project improvements are briefly described in the following sections. In addition to the signature improvements described below, the project includes site retaining walls, an American with Disability Act (ADA) pedestrian ramp and stairs along the bluffs below North Pacific Street, new pavement, lighting, landscape, and hardscape.

The City of Oceanside is also currently conducting another project that will replace the Pier Bridge that spans between North Pacific Street and the Oceanside Pier. That project is being designed under a separate contract but there is coordination between the two design teams. EMI is the geotechnical consultant for the Oceanside Pier Bridge Replacement Project. Geotechnical design and construction recommendations for that project will be provided in a separate Foundation Report specific to that project.

3.1 Junior Seau Beach Community Center (JSBCC)

The JSBCC is an existing two-story building located north of the Pier Bridge. The structure consists of an interior masonry brick two-story structure surrounded by a single-story wood-framed structure that wraps around the western, southern, and eastern sides of the building. No structural work is anticipated for the JSBCC except for an exterior, elevated structural steel mechanical platform proposed to be located at the northeast corner of the existing building. The platform will be constructed using cantilevered steel columns to support the steel platform. The platform will be supported on spread footings.

3.2 Junior Seau Amphitheater and Bandshell

This portion of the project consists of two separate structures: the Amphitheater and the Bandshell. The Bandshell consists of an open stage with an arched concrete canopy that backs up to The Strand. The Amphitheater consists of the stadium style seating area constructed on top of a reinforced concrete building that will house the restrooms, elevator shafts and other storage rooms. The Amphitheater backs to North Pacific Street. The new construction replaces the existing amphitheater, bandshell, and other miscellaneous structures surrounding the existing amphitheater that will be demolished.

The Bandshell stage is elevated approximately 2 ft above the surrounding finish grade (i.e., the Strand) with the canopy over the stage being approximately 25 ft above the stage. The mid-level seating area and cross-aisle of the amphitheater are approximately 11 ft above the Strand with an upper cross-aisle seating area approximately 2.5 ft below North Pacific Street (i.e., approximately 26.5 ft above the Strand).

The eastern wall of the Amphitheater will consist of a braced retaining wall supporting North Pacific Street. The walls of the Amphitheater structure will provide the required lateral bracing for the approximately 30 ft tall retaining wall along North Pacific Street.

The Amphitheater building, Amphitheater retaining walls and bandshell are all proposed to be supported on spread footings. The ground floor of the Amphitheater and Amphitheater building are proposed to be slabs on grade.

A pedestrian ramp will be constructed south of the Amphitheater that will provide ADA access from The Strand to North Pacific Street. The ramp will be constructed in the existing bluffs and will be supported on 24 in to 36 in Cast-In-Drilled-Hole (CIDH) pile foundations (drilled shafts).

3.3 Beachfront Park, Recreation Center and Parking (Rec Center)

This portion of the project consists of a two-story concrete structure with parking at the Strand Level, basketball courts, a landscaped terrace, and enclosed space with classrooms, offices, storage, and back-of-house facilities at the Second Level. Vehicular access is provided to the Second Level via a ramp parallel to First Street.

The Second Level is approximately 11 ft above the Strand and the roof is at approximately the same elevation as North Pacific Street (i.e., approximately 29 ft. above the Strand). Similar to the Amphitheater, the eastern wall of the Recreation Center is an approximately 30 ft tall retaining wall braced by the structure walls that will support North Pacific Street.

The retaining wall and recreation center are proposed to be supported on spread footings. The ground floors are proposed to be slab on grade.

4.0 GEOTECHNICAL INVESTIGATION

A geotechnical field investigation was conducted by EMI in October 2024, consisting of sixteen hollow-stem auger borings. The borings for the subject project improvements were performed concurrently with the borings performed for the Pier Bridge Replacement Project described in Section 3. Boring numbers 006 to 011 were a combination of Cone Penetration Test (CPT) soundings and rotary wash borings conducted at the proposed support locations for the replacement Pier Bridge and are not included in Table 1. Boring logs for the investigations specific to the improvements addressed in the subject report and listed in Table 1 are provided in Appendix A. The approximate locations (latitude and longitude) are summarized in Table 1; the approximate locations are shown on the Exploratory Boring Location Map provided in Figure 2. Boring elevations were obtained from Google Earth as an approximation of the NAVD 88 elevation.

Table 1. Exploratory Boring Information

Boring No.	Boring Type	Latitude	Longitude	Approx. Top of Boring Elevations (ft)	Approx. Bottom of Boring Elevations (ft)	Approx. Groundwater Elevations (ft)[1]
A-24-001	HSA	33.1953	-117.3843	+38	-43.5	0
A-24-002	HSA	33.1951	-117.3842	+39	-42.5	+1.2
A-24-003[2]	HSA	33.1948	-117.3842	+14	-2.5	+5.7
A-24-004	HSA	33.1952	-117.3848	+11	-5.5	NE
A-24-005	HSA	33.1947	-117.3844	+15	-1.5	NE
A-24-012	HSA	33.1945	-117.3836	+37	-13.5	-4
A-24-013	HSA	33.1944	-117.3835	+37	-13.8	-1.3
A-24-014	HSA	33.1940	-117.3832	+39	-11.3	NE
A-24-015	HSA	33.1943	-117.3838	+18	+2.1	+8.5
A-24-016	HSA	33.1940	-117.3835	+16	+0.8	NE
A-24-017	HSA	33.1939	-117.3837	+14	-2.5	NE
A-24-018	HSA	33.1937	-117.3829	+37	+3	+3
A-24-019	HSA	33.1932	-117.3825	+35	-15.2	-10
A-24-021	HSA	33.1934	-117.3829	+14	-36.3	NE
A-24-022[2]	HSA	33.1930	-117.3826	+15	-10.2	+5.6
A-24-024	HSA	33.1931	-117.3831	+12	-13.4	NE

Notes:

1. NE = Not encountered
2. Borings were converted into groundwater monitoring wells

All of the borings for the subject project were performed using hollow-stem auger drilling methods. The hollow-stem auger (HSA) borings were drilled using truck-mounted drill rigs (CME 75) equipped with 8-inch diameter augers. Subsurface soils and conditions were logged and samples of soils were collected for laboratory testing. Bulk samples of near-surface soils were collected. Smaller disturbed and relatively undisturbed soil samples were collected from boreholes generally at five-foot vertical intervals using the Standard Penetration Test (SPT) sampler and the Modified California Drive (MCD) sampler, respectively. The SPT sampler is unlined and has an inside diameter of 1.4 inches and the MCD sampler is lined with a series of 1-inch-tall brass rings with an inside diameter of 2.4 inches.

The SPT and MCD samplers were driven into the soil a total depth of 18 inches or until refusal using a 140-lb hammer falling 30 inches. Automatic trip hammers, used by the drilling contractors, possessed rated efficiencies of 68%, 81% or 83%. The hammer efficiencies for the appropriate drill rig are shown on the header of the individual boring logs in Appendix A. The blowcounts for the last 12 inches or less of penetration were recorded and are shown on the boring logs.

Additionally, two of the borings, A-24-003 and A-24-022, were converted into shallow groundwater monitoring wells after the completion of the respective borings. The wells were constructed with 2-inch diameter Schedule 40 PVC pipe to approximate depth of 15 ft in boring A-24-003 and 25 ft in boring A-24-022. The pipes consist of 5 ft of blank PVC riser to the ground surface and slotted PVC screen encased in a fabric filter sock to the remaining depth. The annular surface of the pipes was filled with approximately ¾-in diameter gravels. A traffic-rated well cover was installed to protect the well heads and to provide access for subsequent groundwater measurements.



LEGEND

A-24-001 (81.5 ft) Hollow-Stem Auger Boring (Depth)

A-24-003 (16.5 ft) Groundwater Monitoring Well was installed after completion of Hollow-Stem Auger Boring (Depth)

0 125 250 ft

SCALE 1" = 125 ft

Note: All boring locations are approximate. All borings were performed on existing paved surface. No borings were drilled on the sandy beach or active wave areas.

Earth Mechanics, Inc.
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Oceanside Beachfront Phase II Project

EMI Project No. 23-137

Date: January 2025

Boring Location Map

Figure 2

5.0 GEOTECHNICAL CONDITIONS

5.1 Site Geology

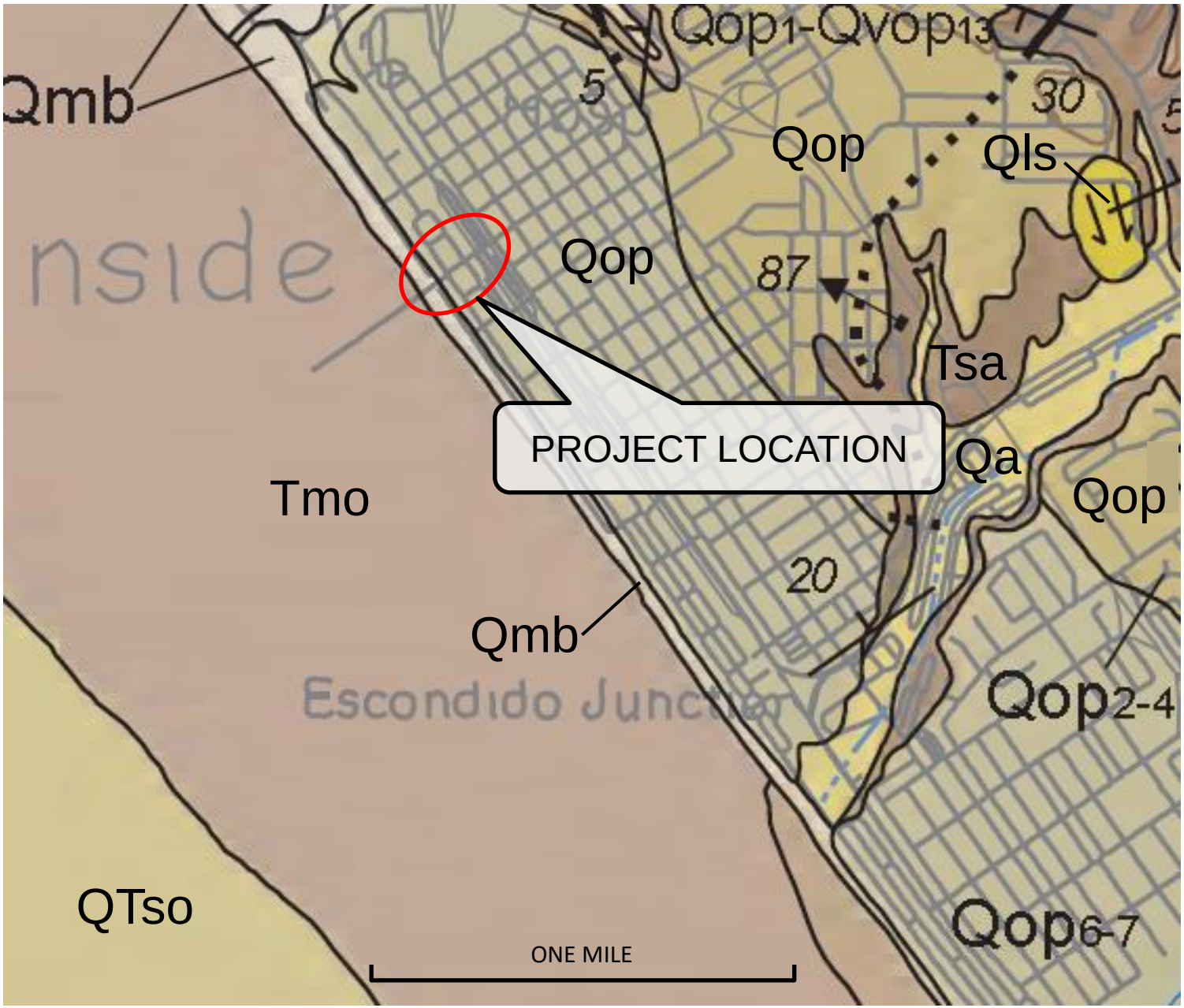
The project area is located in the western portion of the Peninsular Ranges physiographic province, in the Coastal Plain subregion. The Coastal Plain region, ranging from approximately 1 to 12 miles wide, is bounded by the Pacific Ocean to the west, and the Peninsular Ranges to the east. It is characterized by broad, planar mesas gently sloping to the west, incised by deep canyons. The Peninsular Ranges are a group of northwest-southeast trending mountains and valleys between the San Andreas fault on the east and the offshore area called the Continental Borderland. Bedrock in the Peninsular Ranges is predominantly composed of Mesozoic-age granitics. The region surrounding Oceanside, including the offshore Continental Borderland area, is transected by a series of long, mostly northwest-trending, strike-slip fault systems.

Published geological mapping (Kennedy et al., 2007) indicates that surficial deposits at the project site consist of consolidated relatively flat lying, old paralic deposits (map symbol: Qop). From the coastal bluffs west to the ocean, surficial soils consist of unconsolidated beach sands (map symbol: Qmb) overlying bedrock most likely of the Santiago Formation (map symbol: Tsa). Regional Geologic Map of the project area is shown in Figure 3.

5.2 Coastal Bluff Surface Conditions

The marine beach deposits (Qmb) underlying the western portion of the project site generally consist of unconsolidated silts, sands and gravels. The coastal bluffs above the beach (in the eastern portion of the project site) are composed of older Quaternary paralic deposits (Qop) consisting of interfingered strandline, beach, estuarine and colluvial deposits. The paralic deposits encountered in the borings performed as part of this investigation primarily consist of dense to very dense, sands, silty sands and sandy silts, gravel, and minor sandy clay. The bluffs are historically capable of standing near vertical due to cementation of the deposits. Prior to development surrounding the Oceanside Pier, the greatest detriment to bluff stability prior to the narrowing of the beach in the mid 1920's, was subaerial erosion. After the narrowing of the beach, between late December 1940 and early January 1941, a series of storms originating in the Aleutian Islands moved into southern California and considerably damaged beachfront and bluff-top property (Kuhn and Shepard 1979). The project area has since been covered with hardscape and landscape associated with the pier construction and surrounding development and is now largely resistant to subaerial and storm related erosion.

Slope Stability: The bluffs have not historically been susceptible to landslides and are expected to be globally stable even during the design seismic event. The bluffs will be modified as part of the development and retaining walls will be proposed along proposed cuts. The bluff materials are expected to be easily excavated with conventional earth moving equipment.



N

EXPLANATION

Qa	Alluvial deposits
Qls	Landslide deposits
Qmb	Marine beach deposits
Qop	Old paralic deposits
Qvop	Very old paralic deposits
QTso	Offshore sediments & sedimentary rocks
Tmo	Offshore sedimentary rocks
Tsa	Santiago Formation

SOURCE: Kennedy et al., 2007

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Oceanside Beachfront Phase II Project

Project No. 23-137

REGIONAL GEOLOGIC MAP

Date: January 2025

Figure 3

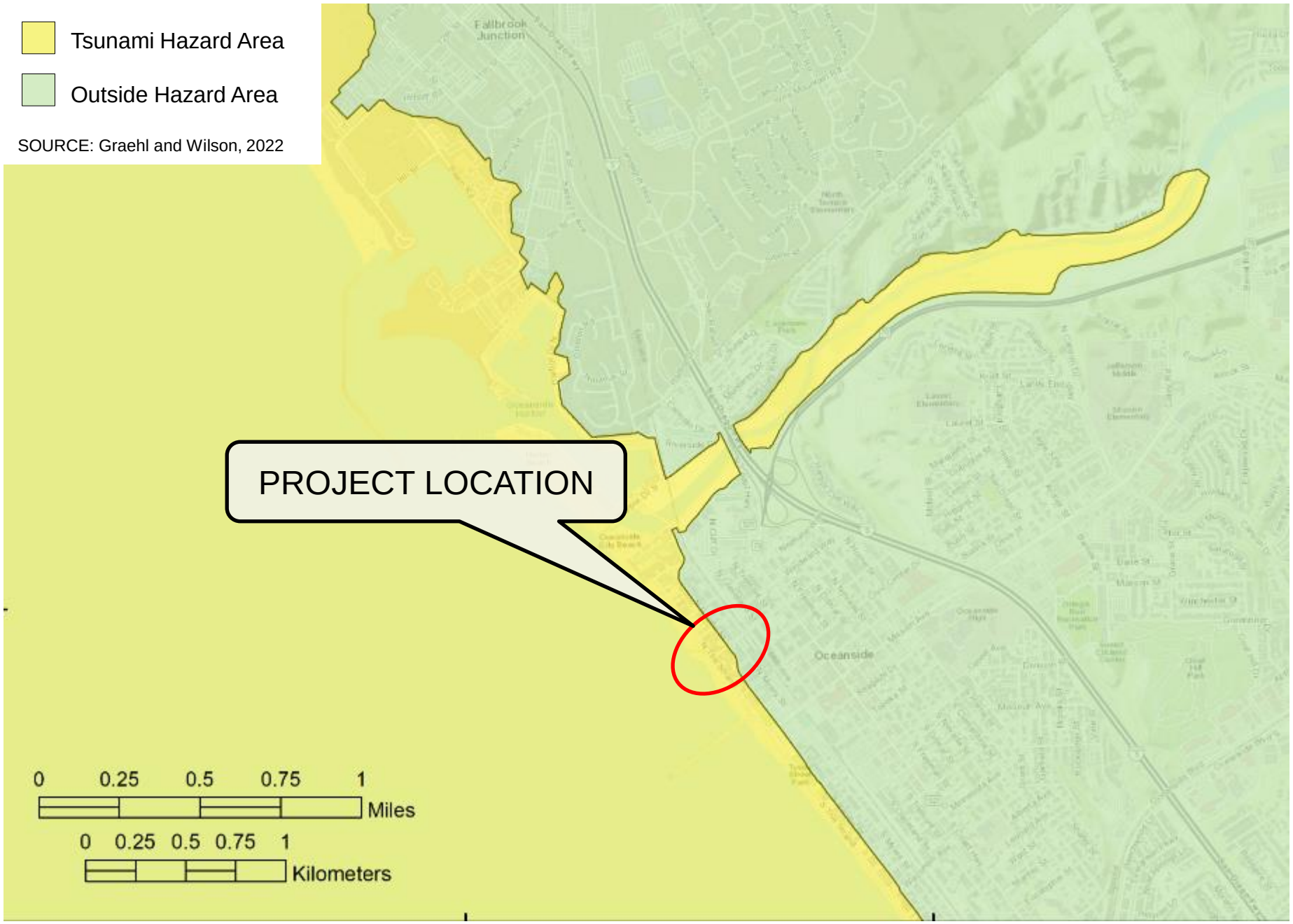
5.3 Seismicity

There are no mapped active faults crossing the site (Jennings, 2010; USGS, 2024) and the California Geological Survey (2024a) has not identified (unevaluated) an Earthquake Fault Zone at the site. The project site is located in a seismically active region and is subject to shaking from both local and distant earthquakes. The closest active faults include the Oceanside section of the Newport-Inglewood-Rose Canyon structural zone (USGS, 2024). The north-south trending Newport-Inglewood-Rose Canyon structural zone is located approximately 2.4 miles east of the project. A right lateral strike-slip fault, the fault is considered to be capable of producing a magnitude 6.9 to 7.1 earthquake (Field et. al, 2013; Cao et al, 2003).

The most significant recent earthquake in the region was the 1986 ML 5.8 event which occurred offshore about 32 miles southwesterly of Oceanside at a depth of about 5.5 miles. The earthquake caused one death and caused damage throughout San Diego County and southern Orange County (SCEDC, 2024).

5.4 Tsunami

A tsunami is a powerful displacement of the ocean as a result of an earthquake. The lower portion of the project site at the base of the coastal bluffs is identified as being within a tsunami hazard area (Graehl and Wilson, 2022). Structures within this zone could be subjected to strong currents, severe erosion and debris impacts in the event of a tsunami. Figure 3 is a map showing the Tsunami Hazard area.



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Oceanside Beachfront Phase II Project

Project No. 23-137

TSUNMAI HAZARD MAP

Date: January 2025

Figure 4

5.5 Subsurface Conditions

Idealized soil profiles and design strength parameters for geotechnical analyses and foundation design were developed using the available subsurface information and are presented in Table 2. The shear strength parameters for coarse-grained soils were estimated using direct shear test results (ultimate) and correlations with field blowcounts (Caltrans Geotechnical Manual – Soil Correlations, 2021b). In locations where a discrepancy was observed between blowcount correlations and direct shear test results, the design strength parameters were selected using the blowcount correlations considering that the blowcount correlations provide the best indication of in-situ soil strength. The shear strength parameters for fine-grained soils were estimated using pocket penetrometer (PP) test results (ultimate) supplemented with correlations with field blowcounts (Caltrans Geotechnical Manual – Soil Correlations, 2021b). In Table 2, a factor of 0.65 was used to convert Modified California Drive (MCD) sampler blowcounts to Standard Penetration Test (SPT) sampler blowcounts.

The available subsurface information indicates that the site soils are composed of a thin layer of compact alluvium overlaying very dense poorly sorted sand and gravel with small amounts of clay binder to the explored depth. The idealized soil profile is presented in Table 2.

Table 2. Idealized Soil Profile

Approximate Elevation (feet)	Predominant Soil Type	Range of SPT N_{60} Blowcount (Blows/ft)	Friction Angle (degrees)	Cohesion or Undrained Shear Strength (psf)	Total Unit Weight (pcf)
Amphitheater and Recreation Center Retaining Walls Along North Pacific Street and ADA Ramp					
+40 to +13	Poorly-graded Sand / Poorly-graded Sand with Silt / Silty Sand	15 to >50 Average = 40	33	150	120
+13 to -30	Poorly-graded Sand / Silty Sand / Clayey Sand / Poorly-graded Sand with Silt	48 to >50 Average >50	38	0	125
-30 to -50	Silt with Sand / Sandy Lean Clay with Gravel / Sandy Silt	28 to >50 Average >50	0	4,000	125
-50 to -80	Clayey Gravel with Sand	>50	38	0	125
Junior Seau Beach Community Center Foundations					
+13 to +2	Poorly-graded Sand / Silty Sand	8 to 15 Average = 11	30	100	120
+2 to -20	Poorly-graded Sand / Silty Sand / Poorly-graded Sand with Silt	(27) to >50 Average > 50	38	0	125

Approximate Elevation (feet)	Predominant Soil Type	Range of SPT N_{60} Blowcount (Blows/ft)	Friction Angle (degrees)	Cohesion or Undrained Shear Strength (psf)	Total Unit Weight (pcf)
Junior Seau Pier Amphitheater and Bandshell Foundations					
+18 to +5	Poorly-graded Sand / Silty Sand	7 to 11 Average = 9	30	100	120
+5 to -10	Poorly-graded Sand / Clayey Sand / Poorly-graded Sand with Silt	(29) to >50 Average > 50	38	0	125
Beachfront Park, Recreation Center and Parking (Rec Center) Foundations					
+15 to +2	Poorly-graded Sand / Silty Sand	9 to 11 Average = 10	30	100	120
+2 to -15	Clayey Sand / Poorly-graded Sand / Poorly-graded Sand with Silt	>50	38	0	125
* Values in ( ) are equivalent SPT blowcounts, converted from MCD sampler blowcounts					

6.0 GROUNDWATER CONDITIONS

During the field investigation by EMI for the project, groundwater was encountered in the borings and CPT soundings between the approximate elevations of -10 and +8.5 ft (depth of 8.3 to 38.3 ft below existing grade at the time of geotechnical explorations). Additionally, as described in Section 4, two groundwater monitoring wells were installed on-site to monitor groundwater depth. The following table presents the measured groundwater depth at the time of the investigation and an additional reading obtained during the preparation of the subject report.

Table 3. Groundwater Monitoring Well Readings

Well No.	Latitude (degrees)	Longitude (degrees)	Approx. Elevation of Top of PVC Pipe (ft)	Measured Depth to Water from Top of Pipe Oct 3, 2024 (ft)	Approx. Water Surface Elevation Oct 3, 2024 (ft)	Measured Depth to Water from Top of Pipe Jan 16, 2025 (ft)	Approx. Water Surface Elevation Jan 16, 2025 (ft)
A-24-003	33.1948	-117.3842	+13	8.3	+4.7	9.2	+3.8
A-24-022	33.1930	-117.3826	+15	9.4	+5.6	10.1	+4.9

A design groundwater table was placed at the elevation of +8.5 ft for liquefaction potential assessment and foundation design. The design groundwater was placed at an elevation generally consistent with high tide as the groundwater at the project site is expected to fluctuate with the tide due to the proximity of the site to the Pacific Ocean. This is a conservative design groundwater elevation/depth, but it is not expected to result in costly or unnecessary overdesign.

Variations in groundwater elevation will occur daily as tide rises and falls with there likely being a time lag between the elevation of groundwater in the ground and the tide elevation along the shore. The groundwater conditions encountered during construction will likely be different at any given time from the design groundwater and the contractor should not base their construction means and methods based on the design groundwater. Groundwater impacts on construction are addressed in Section 12.0

7.0 COASTAL BLUFF EROSION EVALUATION

The project site is adjacent to approximately 30 ft high coastal bluff that parallels North Pacific Street. The only location where the existing bluff slope is not covered with landscaping or hardscape is between the existing Amphitheater seating area and 1st Street. In this location, bluff slope is covered with sparse to no vegetation and shallow erosion gullies on the order of a couple of feet in depth.

As mentioned in Section 5.2, the coastal bluffs are historically capable of standing near vertical due to cementation of the deposits. Furthermore, the project area has been covered with hardscape and landscape associated with the pier construction and surrounding development and is now largely resistant to significant subaerial and storm related erosion.

Based on the information provided by the city, the existing coastal bluff is susceptible to erosion and shallow instability. It is our understanding that there are no previously conducted erosion studies for the coastal bluff. Therefore, it is assumed that erosion would occur at a rate of 1.5 to 2 inches per year where exposed bluffs are not covered by landscaping or hardscape. Based on this, total erosion of 10 to 12 ft is anticipated for a design life of 75 years. This erosion rate is only incorporated into the foundation design for the pedestrian ramp which is located south of the proposed Amphitheater.

8.0 CORROSION EVALUATION

Representative soil samples collected from throughout the project area were tested to determine corrosivity including minimum resistivity, pH, soluble sulfate content, and soluble chloride content. Nine soil samples were tested for corrosivity; results are summarized in Table 4.

Table 4. Soil Corrosion Test Results

Boring No.	Sample Depth (ft)	Sample Elevation (ft)	Groundwater El. During Drilling (ft)	USCS Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
A-24-001	5	+33	0	Silty Sand (SM)	2800	9.3	396	39
A-24-004	10	+1	NE	Poorly Graded Sand (SP)	3300	9.1	187	138
A-24-012	5	+32	-4	Silty Sand (SM)	1800	7.7	261	263
A-24-014	10	+29	NE	Silty Sand (SM)	3100	8.6	365	154
A-24-015	5	+13	+8.5	Poorly Graded Sand (SP)	4400	8.1	161	168
A-24-017	5	+9	NE	Poorly Graded Sand (SP)	12000	8.7	561	47
A-24-019	5	+30	-10	Silty sand (SM)	1000	8.4	472	547
A-24-021	10	+4	NE	Silty Sand with Gravel (SM)	1200	8.4	1172	250
A-24-024	5	+7	NE	Poorly Graded Sand (SP)	5200	7.6	139	237

Note: NE = Not Encountered

According to the Caltrans Corrosion Guidelines V3.2 (Caltrans, 2021c), soils are considered corrosive if the pH is 5.5 or less, chloride concentration is 500 parts per million (ppm) or greater, or sulfate concentration is 1,500 ppm or greater. Based on the Caltrans criteria and close proximity to the marine environments, EMI recommends that the on-site soils at the locations of improvements addressed in this report be considered corrosive.

9.0 SEISMIC INFORMATION

9.1 Seismic Design for Buildings

The 2022 California Building Code (CBC) Seismic Design Parameters have been generated using the ASCE Hazard Tool. This web-based software application calculates seismic design parameters in accordance with ASCE 7-16 and 2022 CBC. The 2022 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than or equal 0.2.

However, Supplement 3 of Section 11.4.8 of ASCE 7-16 includes an exception from performing such analysis for specific structures on Site Class D sites. A site-specific ground motion analysis is not required where the value of the parameter S_{M1} determined using Equation 11.4-2 is increased by 50% for all applications of S_{M1}. Increasing S_{M1} by 50% results in an increase in the value of S_{D1} by 50%. Based on our understanding of the proposed structures, it is our assumption that the exception in ASCE 7-16 Section 11.4.8 (Supplement 3) applies to the proposed structures. However, the structural engineer should verify the applicability of this exception.

Based on this exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in the 2022 CBC.

Table 5. Site Coordinates, Shear Wave Velocity, and Peak Ground Acceleration

Description	Values
2022 California Building Code Site Classification	D
Site Latitude (degrees)	33.1937
Site Longitude (degrees)	-117.3834
S_S Mapped Spectral Acceleration for a 0.2-Second Period	1.08
S_1 Mapped Spectral Acceleration for a 1-Second Period	0.394
F_a Site Coefficient for a 0.2-Second Period	1.068
F_v Site Coefficient for a 1-Second Period	1.906
S_{DS} Design Spectral Acceleration for a 0.2-Second Period	0.769
S_{D1} Design Spectral Acceleration for a 1-Second Period	0.751[1]

Note:
1. Per ASCE 7-16 (Supplement 3) – Section 11.4.8, the S_{D1} value was increased by 50%

Based on the ASCE Hazard Tool, using the American Society of Civil Engineers (ASCE 7-16) standard, the peak ground acceleration (PGA_M) at the project site is expected to be 0.535 g. Based on the USGS Unified Hazard Tool, the project site has a mean magnitude of 6.69.

9.2 Liquefaction Potential

Liquefaction is a phenomenon whereby saturated granular soils lose their inherent shear strength due to increased pore water pressures, which may be induced by cyclic loading such as that caused by an earthquake. Low density granular soils, shallow groundwater, and long duration/high acceleration seismic shaking are some of the factors favorable to cause liquefaction.

The liquefaction analysis was performed using the recent site-specific borings, and the design groundwater elevations provided in Section 6.0. The liquefaction potential of saturated, coarse-grained materials below the design groundwater table was evaluated using the SPT and CPT data following the procedures outlined by Youd et al. (2001).

Based on our evaluation, potentially liquefiable soils were encountered only in one boring near elevation +7 ft. Potentially liquefiable layers were not encountered in any of the other borings. Based on this, liquefiable layers are considered isolated and discontinuous. Therefore, the liquefaction potential is considered low within the project site.

9.3 Seismically-Induced Settlement

The seismically-induced settlement was estimated using SPT blowcounts recorded in the borings with the empirical method outlined by Tokimatsu and Seed (1987). The analysis indicates that the seismically-induced settlement is in the order of 1 inch from one boring and less than ¼ inch from all other borings. The seismically-induced differential settlement is expected to be less than ½ inch over a distance of about 40 ft.

9.4 Seismic Slope Instability

Since liquefaction potential is considered low, seismic instability of the slopes is not considered a design issue. Therefore, the coastal bluff slopes below North Pacific Street are expected to be globally stable during the design earthquake. The global stability evaluation of the bluffs surrounding the proposed project improvements is described in Section 10.

9.5 Fault Rupture

Surface rupture usually occurs along traces of known active or potentially active faults. However, many historic seismic events, including the 1994 Northridge Earthquake, have occurred on faults without surface expression (blind faults) that were not previously known to exist or to be active.

The CGS has established criteria for faults as active, potentially active, or inactive based on the evidence of historical surface displacement. The California Geological Survey (CGS, 2024b) and United States Geological Survey (2024) indicate that no active or potentially active faults are known to traverse the site, and the site is not located within a currently designated Earthquake Fault Zone. The likelihood of fault rupture at the project site is considered low.

10.0 FOUNDATION RECOMMENDATIONS

Based on the recently completed borings, site soils are conducive to support the structures and retaining walls on spread footings provided the site has been prepared in accordance with the requirements presented in the Earthwork section of this report.

10.1 Retaining Wall Earth Pressures and Surcharges

10.1.1 Lateral Active Earth Pressures

Design earth pressures for the retaining walls were estimated using the trial wedge analysis assuming a level and 2H:1V sloping backfill behind the wall. For walls free to move laterally at the top, the flexible wall active earth pressures provided in Table 6 for a free draining backfill should be used. For walls that are required to remain rigid, the rigid wall at-rest earth pressures provided in Table 6 should be used. For the seismic condition, the incremental seismic earth pressure should be added to the static earth pressure for the appropriate wall rigidity to get the total seismic earth pressure.

Surcharge pressures due to vehicle live loads and structures on the wall backfill should be added to the above lateral earth pressures, where applicable for static evaluations. The live load from traffic surcharge can be modeled as a uniform vertical load of 240 psf. The vertical surcharge should be converted to uniform horizontal earth pressures as presented in Table 6 for flexible or rigid walls.

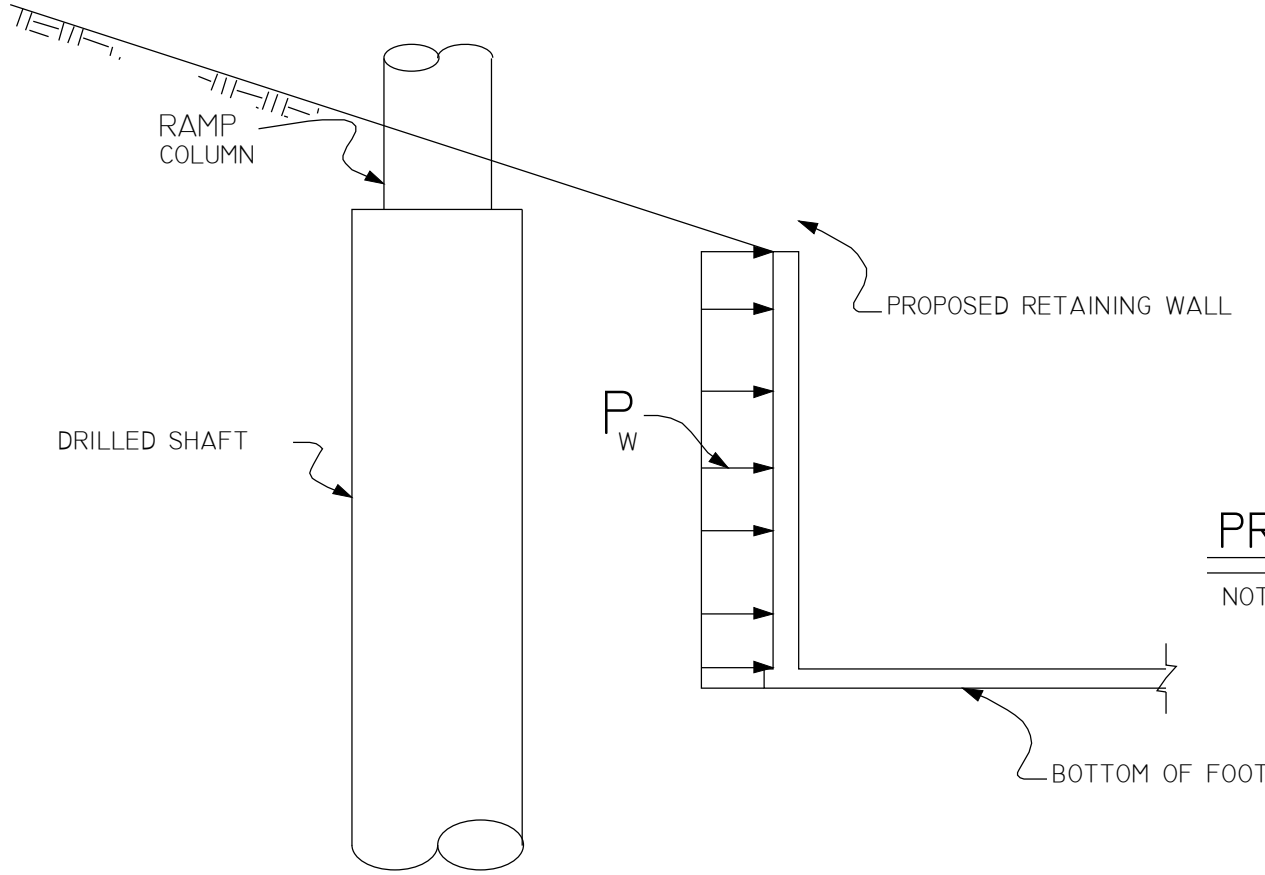
Table 6. Earth Pressure Recommendations

Wall Fixity	Static Earth Pressure	Incremental Seismic Earth Pressure
Active (Flexible Wall)	40 psf/ft for level backfills 63 psf/ft for 2H:1V sloping backfills	15 psf/ft for level backfills 40 psf/ft for 2H:1V sloping backfills
At-Rest (Rigid Wall)	60 psf/ft for level backfill	80 psf/ft for level backfill
Surcharge Loads (Flexible Wall)	(0.33 x surcharge) psf for level backfills (0.53 x surcharge) for 2H:1V sloping backfills	N/A
Surcharge Loads (Rigid Wall)	(0.5 x surcharge) psf for level backfills	N/A
Note: Live load from traffic surcharge can be modeled as a uniform vertical load of 240 psf and converted to a uniform horizontal load using the appropriate static earth pressure coefficient for the wall fixity.		

The above recommended values presented do not include hydrostatic forces. Backdrains should be installed behind the walls to relieve hydrostatic pressure. Additional recommendations may be necessary if such conditions are to be included in the design.

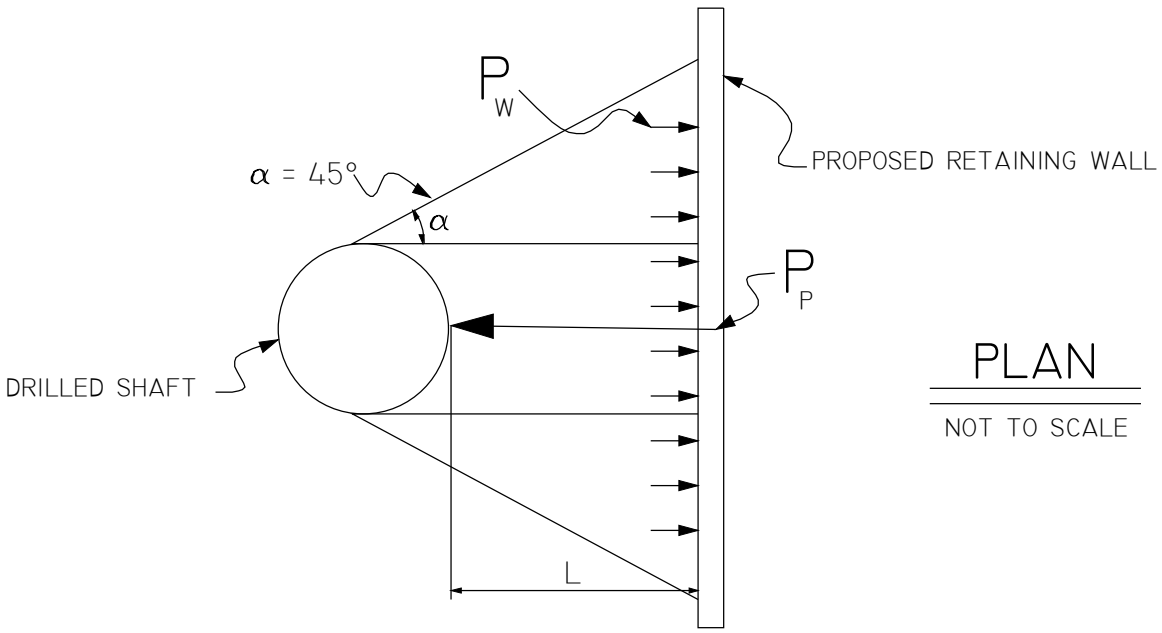
10.1.2 Pile Surcharge Load

Drilled shafts supporting the ADA ramp described in Section 3 are proposed behind a portion of the Amphitheater retaining wall. Any drilled shafts located within a distance of 7D (D is diameter of the drilled shaft) from the back of the retaining walls will impose a lateral surcharge against the back of the retaining wall. The lateral surcharge load from the CIDH piles can be estimated using the methodology outlined in Figure 5.



PROFILE

NOT TO SCALE



PLAN

NOT TO SCALE

P_P = SOIL REACTION AGAINST PILE (OUTPUT FROM LPILE) (LB/IN OF HEIGHT)
P_W = PRESSURE APPLIED AGAINST WALL (LB/IN OF HEIGHT)
D = DIAMETER OF DRILLED SHAFT (IN)
L = CLEAR DISTANCE FROM CIDH TO BACK OF WALL (IN)

$$P_W\ (LB/IN^2) = \frac{P_P\ (LB/IN)}{D + 2(L + 0.5D)}$$

P_P = 200 LB/IN FOR D = 24 IN
P_P = 400 LB/IN FOR D = 36 IN

OCEANSIDE BEACHFRONT PHASE II PROJECT

PILE LOAD SURCHARGE AGAINST PROPOSED RETAINING WALL

FIGURE 5

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

Project No. 23-137 | Date: JANUARY 2025

10.2 Spread Footings

The modifications to the Junior Seau Beach Community Center, the new Junior Seau Amphitheater and Bandshell, the new Beachfront Park Recreation Center and associated retaining walls are all proposed to be supported on spread footings.

10.2.1 Allowable Bearing

In general, where spread footings are proposed, site soils consisted of a near-surface layer of medium dense soil with occasional loose pockets underlain by very dense layers. Due to the potential for shallow groundwater to be encountered during construction, deep remedial excavations to replace the near-surface layer to obtain higher soil bearing capacities is not recommended during preliminary design. However, if required, higher bearing capacities for spread footings can be achieved with remedial earthwork if the preliminary recommendations provided result in oversized footings. Revised recommendations, if necessary, would be based on the footing dimensions and embedment determined during final design.

The allowable bearing pressure estimates listed in Table 7 should be used for preliminary design and are based on range of footing dimensions and minimum footing depths below the finished grade. The allowable bearing pressures apply to dead loads and design live load conditions. The design bearing pressures may be increased by one-third when considering total loads that include wind or seismic conditions. The weight of the foundation concrete below grade may be neglected in dead load computations.

The static settlement for the spread footings is limited to 1.0-inch. The estimated differential static settlement is limited to 0.5 inches over the distance of 40 ft. It is recommended that minimum 1-ft of remedial overexcavation should be performed below the bottom of the footings and backfilled with engineered fill materials. On-site materials are considered suitable to be used as engineered fill materials provided the on-site materials are processed to have maximum particle size of 1 inch.

Table 7. Spread Footing Data Table

Foundation Type	Footing Width (ft)	Minimum Embedment below Finished Grade (ft)	Allowable Bearing Pressure (psf)	Static Settlement (inch)	Static Differential Settlement
Spread Footing	Up to 25	1.5	2,000	1	½ inch over 40 ft
Note: Minimum 1-ft of engineered fill materials is recommended below the bottom of the spread footings					

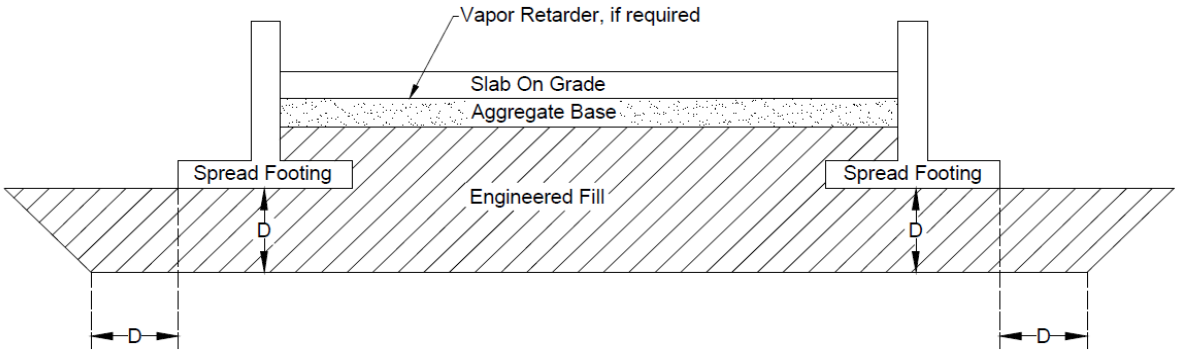
Foundations should be reinforced as necessary to reduce the potential for distress caused by differential foundation movement. Foundation excavations should be observed by the geotechnical engineer. If the soil conditions encountered differ significantly from those presented in this report, supplemental recommendations will be required.

10.2.2 Sliding Resistance

Lateral loads can be resisted by friction acting along the base of the footing and passive lateral earth pressure. To determine the allowable frictional resistance along the base of a footing, a coefficient of friction of 0.40 between the concrete footing and the soil below the footing is recommended. The passive lateral earth pressure can be determined using an equivalent fluid density of 360 pcf, assuming level ground in front of the footing. If the ground surface in front of the wall is exposed earth, not covered by hardscape or sidewalk, the passive pressure for the top foot of ground should be neglected. The full passive resistance developed using the earth pressure against the face of the footing will be mobilized at a displacement equal to 2% of the height between the bottom of footing elevation and finished grade elevation above the footing (H); linear interpolation can be used for any displacement less than 2% of H. The sliding resistance of footings may be determined using 100% of the passive pressure and 100% of the friction along the base of the footing.

10.3 Slab on Grade

Based on the information presented in the project plans, the interior floors are proposed to be slab-on-grade. As shown in the figure below, the slab-on-grade should be supported on engineered fill materials extending to a minimum depth of D = 1 ft below the bottom of the spread footings. A minimum 4 inches of aggregate base should be placed below the bottom of the slab on grade.



For the design of the floor slabs, an unadjusted vertical modulus of subgrade reaction K_{V1} of 200 pounds per cubic inch is estimated for medium dense sands below the floor slabs. The estimated K_{V1} value given above is based on data from a 1-foot by 1-foot square plate and should be reduced for larger dimensions. For a square footing of width B, the adjusted modulus K_{BxB} can be taken as:

$$K_{BxB} = K_{V1}\left[\frac{(B+1)}{2B}\right]^2$$

For a rectangular slab having a long dimension of L (ft) and a short dimension of B (ft), a value of modulus of subgrade reaction $K_{L\times B}$ can be obtained using the following equation:

$$K_{LxB} = K_{BxB}\left[\frac{1+0.5\frac{B}{L}}{1.5}\right]$$

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. The slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

To control the cracking, saw cut control joints should be placed in the slab and sealed with water-proof, nonextruding compressible compound recommended for heavy duty concrete pavement and wet environment.

10.4 Pile Foundations – Drilled Shafts

As described in Section 3, an ADA pedestrian ramp is proposed south of the proposed Amphitheater which will provide pedestrian access between North Pacific Street and The Strand. A portion of the ramp is proposed to be supported at grade while the majority of the ramp will be have a multiple-span bridge configuration supported on single CIDH pile bents. The CIDH piles will be installed through existing exposed bluff materials. The bluff face is a sparsely vegetated slope with visible erosion gullies along the face.

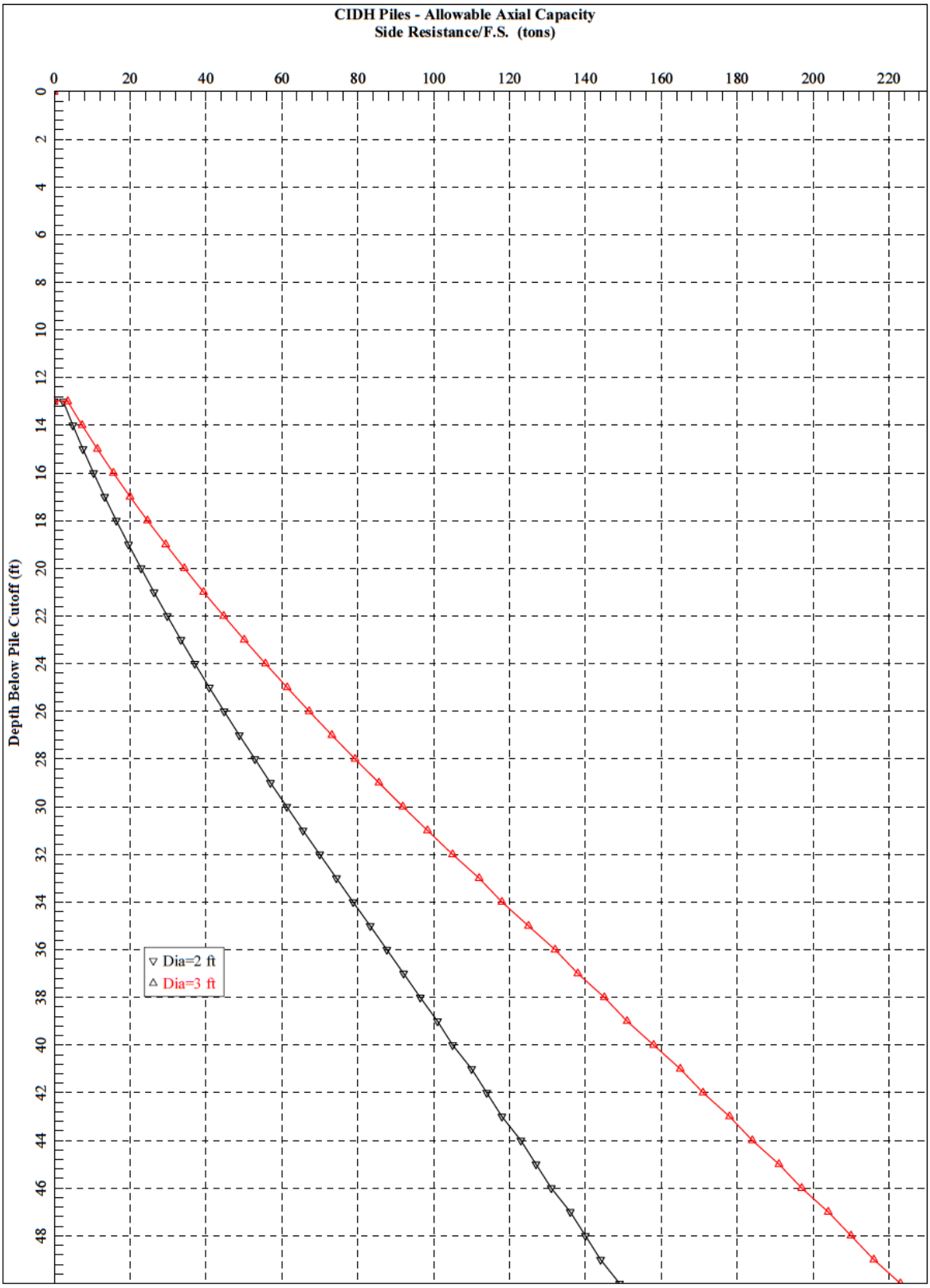
Erosion: As discussed in Section 7.0, approximately 10 to 12 ft of erosion is conservatively considered in the coastal bluff materials not covered by landscape or hardscape. It is assumed that the coast bluff is generally inclined at about 1:1 (H:V) angle. Based on this anticipated erosion, pile would lose up to 10 to 12 ft vertically of soil resistance along the piles over a 75 year life-span of the structure. Potential erosion of the bluff was considered in the axial and lateral pile capacity calculations by ignoring 12 ft (potential erosion depth along the pile) of soil resistance below pile cut-off elevations.

10.4.1 Axial Pile Capacity

Axial pile capacities were estimated using SHAFT v2017 (Ensoft, 2019) for the 24 in and 36 in diameter piles considering skin friction along the drilled shafts alone and no end bearing. Due to anticipated erosion of the coastal bluff materials over the design period, the upper 12 ft below pile cutoff of skin friction is ignored in the capacity evaluations. No group reduction factors were applied to the axial pile capacity estimates based on the single pile support configuration. However, due to the zig-zag configuration of the ramp, piles are spaced in close proximity in some locations. Group action may need to be considered during final design if adjacent piles are spaced closer than 2.5 diameters.

The allowable pile capacity curves for both 24 in and 36 in diameter drilled shafts are presented in Figure 6. A factor of safety of 2.0 was considered in the allowable pile capacity calculations. The pile capacities are based on soil resistance only and may be further limited by the pile-head connection details and the structural material strength.

Figure 6. Allowable Axial Pile Capacities



10.4.2 Lateral Pile Solutions for Drilled Shafts

Lateral single-pile analyses were performed for free-head loading condition using the idealized soil profiles provided in Table 2 and the computer program LPILE v2019 (Ensoft, 2021). The LPILE generated p-y curves for sandy soils were estimated using the API criteria (API, 2000). Due to the potential erosion, upper 12 ft of soil resistance was ignored in the lateral pile analysis.

Even though proposed piles are single shafts, the piles are proposed adjacent to each other in some locations due to zig-zag nature of the ramp. Based on the draft plans, there are three rows of piles parallel to North Pacific Street. A minimum center-to-center pile spacing of 3D was considered between the rows in the group efficiency factor calculations. Based on pile spacing of 3D, a group efficiency factor of 0.65 were considered in the transverse direction (perpendicular to the ramp) and 1.0 were considered in the longitudinal direction (along the ramp).

The resulting pile shear capacity and maximum bending moment caused by lateral pile-head deflections are provided in Figure 7 and Figure 8 for 24-inch diameter drilled shafts and in Figure 9 and Figure 10 for 36-inch drilled shafts. The solutions presented are entirely based on soil resistance and linear pile properties. Therefore, these values may be limited by the flexural strength (plastic moment) of the piles and pile-head connection details. Lateral pile solutions are provided for pile-head deflections from 0.25 to 2 inches, and linear interpolation can be used for intermediate pile-head deflections.

Table 8. Lateral Pile Solutions for Free-Head Loading Conditions

Support Location (Loading Direction)	Pile Type	Pile Head Deflection (inch)	Pile Head Shear (kips)	Maximum Moment (in-kip)	Depth to Max. Moment from Pile Top (ft)
Pedestrian Ramp (Longitudinal Direction - Along Ramp)	24-in CIDH	0.25	2.9	284	15.0
		0.5	5.8	567	15.0
		1	11.6	1125	15.0
		2	22.7	2178	15.0
Pedestrian Ramp (Transverse Direction - Perpendicular to Ramp)	24-in CIDH	0.25	2.5	261	15.0
		0.5	5.0	520	15.0
		1	9.9	1027	15.5
		2	19.4	1973	16.0
Pedestrian Ramp (Longitudinal Direction - Along Ramp)	36-in CIDH	0.25	8.1	1013	16.5
		0.5	16.2	2021	16.5
		1	32.1	4000	16.5
		2	62.6	7736	17.0
Pedestrian Ramp (Transverse Direction - Perpendicular to Ramp)	36-in CIDH	0.25	6.7	912	17.0
		0.5	13.5	1817	17.0
		1	26.7	3588	17.0
		2	52.1	6893	17.5

Figure 7. Lateral Pile Solution for 24-in Drilled Shafts – Longitudinal Direction

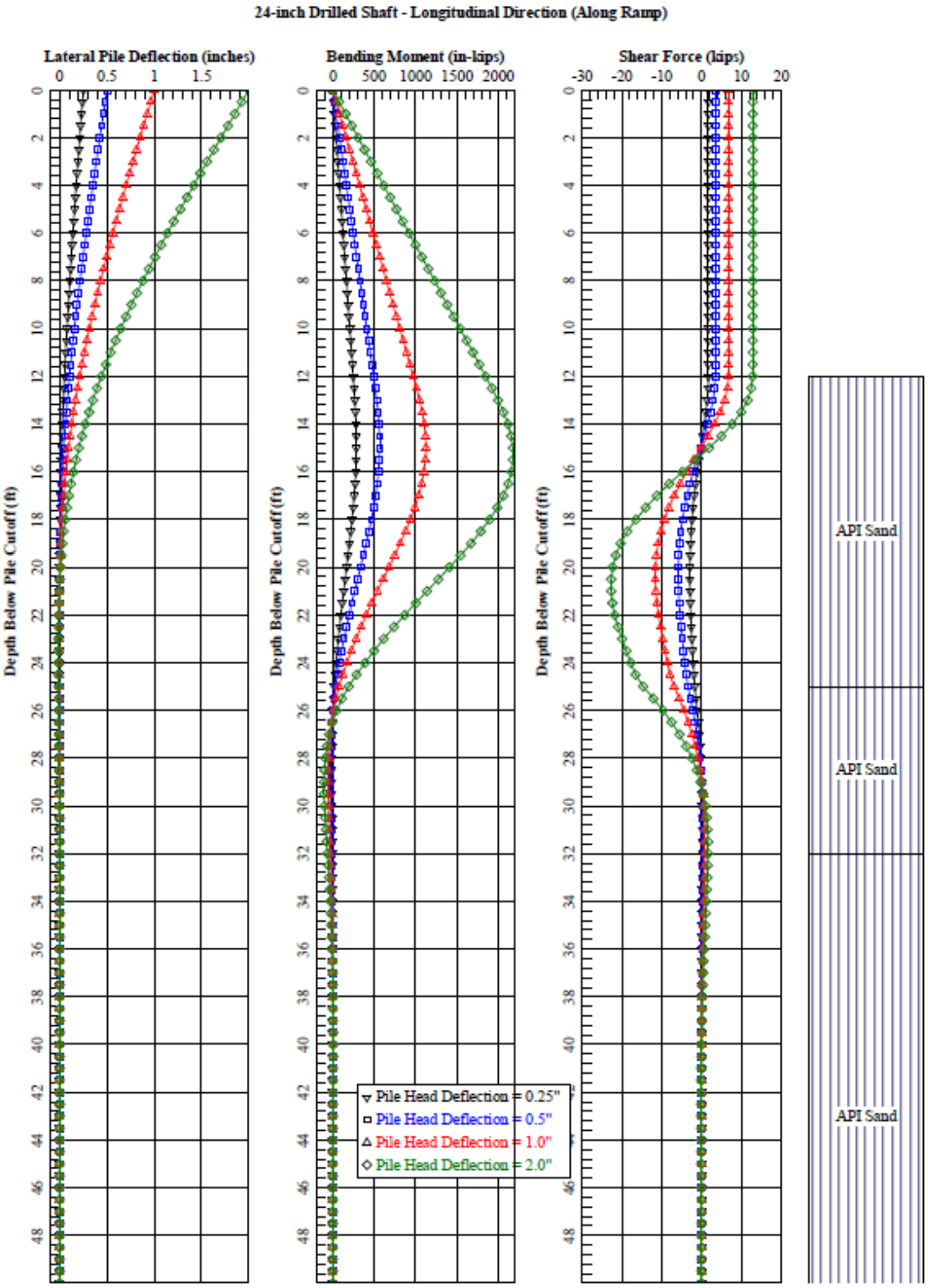


Figure 8. Lateral Pile Solution for 24-in Drilled Shafts – Transverse Direction

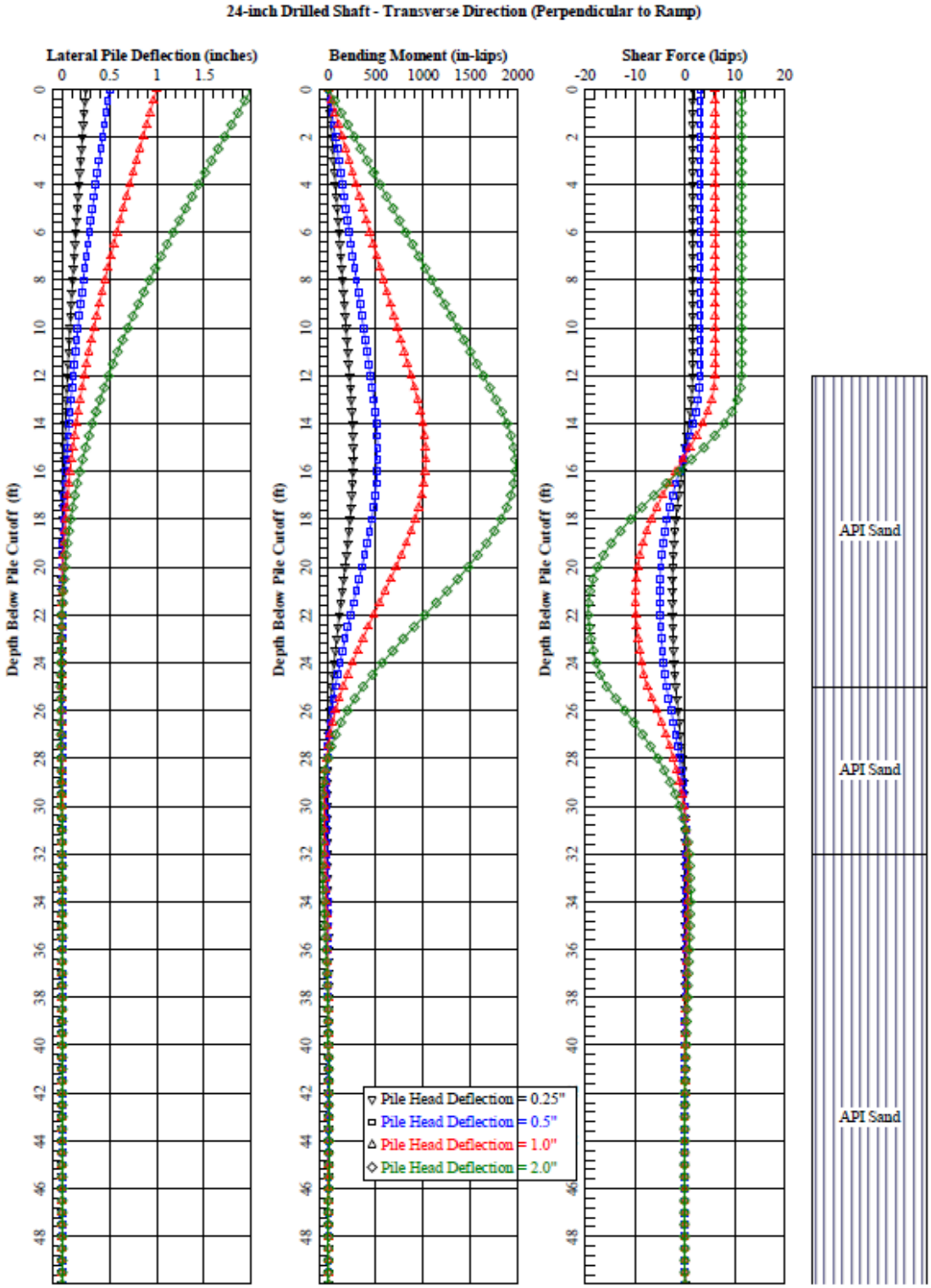


Figure 9. Lateral Pile Solution for 36-in Drilled Shafts – Longitudinal Direction

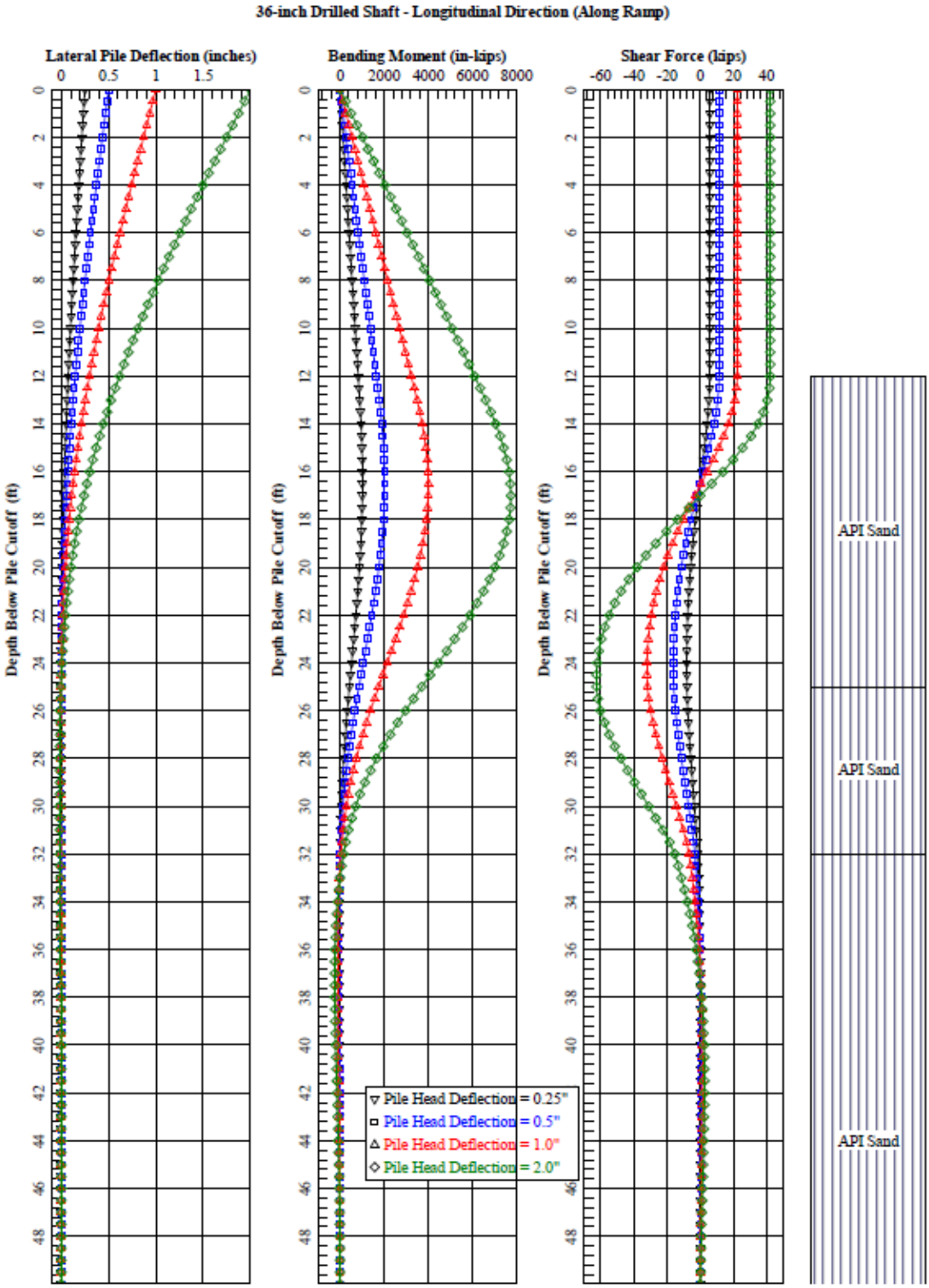
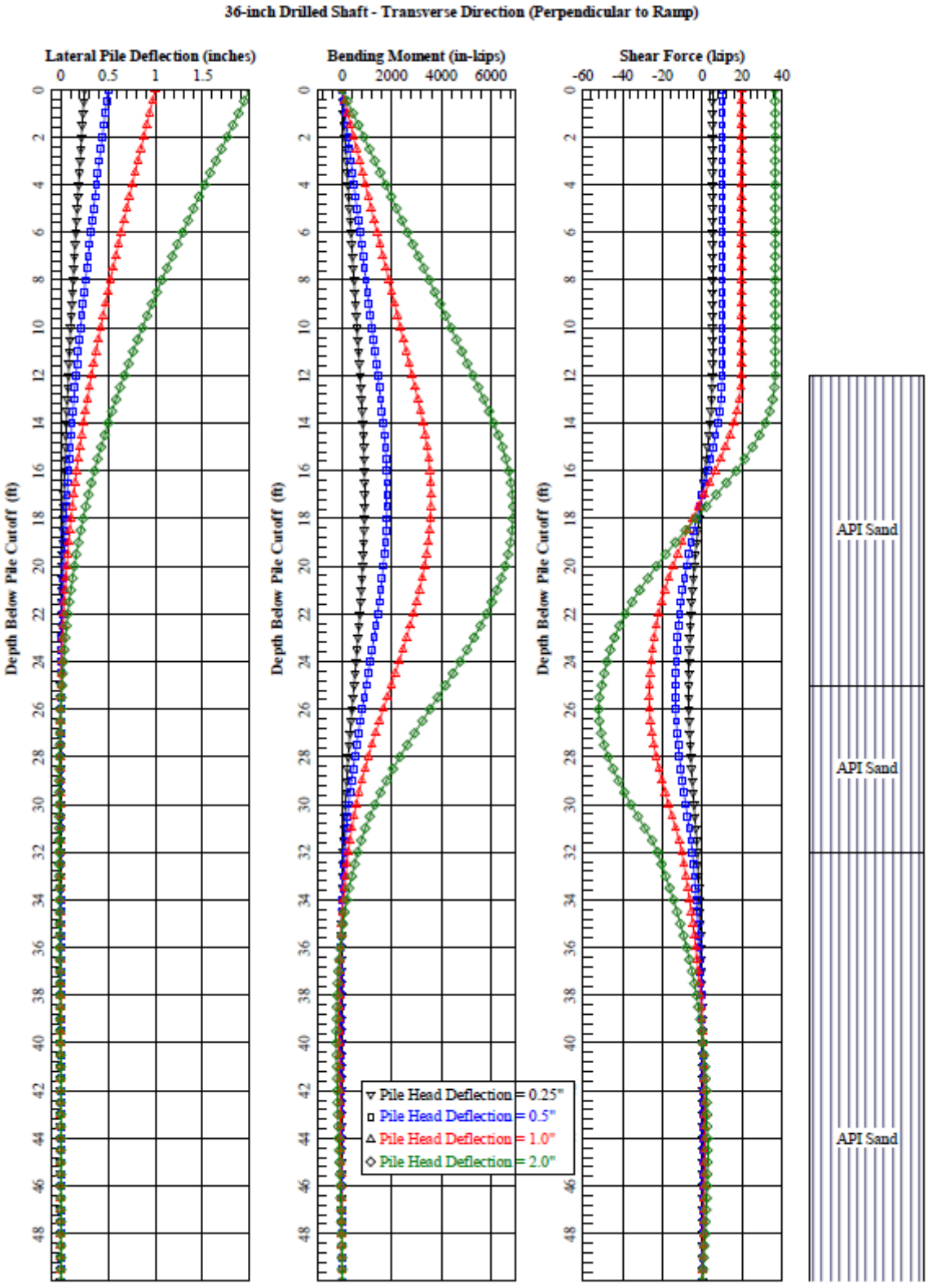


Figure 10. Lateral Pile Solution for 36-in Drilled Shafts – Transverse Direction



10.5 Global Slope Stability

Retaining walls are proposed parallel to the proposed ADA pedestrian ramp at the toe of the existing bluff. Global stability analyses for the exposed portion of the bluff and the retaining walls were conducted for both static and pseudo-static conditions for potential deep-seated failures below the retaining wall footing. A design wall height of 12 feet was considered in the analysis. Behind the retaining wall, a back slope of 2H:1V was considered in the analysis. The analysis was performed using the computer program Slide2 (Rocscience, 2020).

Slope stability analyses were conducted for the static condition including a 2-foot soil surcharge to represent traffic loading at the crest of the slope. Stability analysis for the seismic condition was performed using the pseudo-static approach with a seismic coefficient of 0.178, which is equal to one-third PGA.

According to the results of the analyses, the existing slopes meet the minimum required factor-of-safety for deep-seated failure of 1.5 for the static condition and 1.1 for the pseudo-static condition.

11.0 PAVEMENTS

11.1 Subgrade R-Values

Two near-surface soil samples collected by EMI (from upper 5 ft of the existing subgrade within the existing parking lot) were to determine the R-values. The results of these tests are presented in Table 9.

Table 9. Subgrade R-Values

Boring No.	Soil Type	R-Value
A-24-003	Poorly graded Sand (SP)	76
A-24-015	Poorly graded Sand with Silt (SP-SM)	77

R-values of near-surface soils are 76 and 77. Near surface soil conditions throughout the project site consist of predominantly coarse-grained soils. R-values of near-surface soils are expected to be within the range indicated by the R-value testing.

11.2 Pavement Structural Sections

The minimum flexible pavements for parking lots and drive ways were selected from the City of Oceanside Grading Regulation Manual. A 20-year design life was considered.

The recommended pavement structural sections are presented in Table 10.

Table 10. Recommended Pavement Structural Sections

Location	Design Life (Years)	Recommended Minimum Flexible Pavement Sections
Parking areas	20	3” AC / 4” AB
On-site Drive Ways and Delivery Areas		3” AC / 10” AB
Note: AC = Asphalt Concrete; AB = Aggregate Base (Class 2)		

11.3 Pavement Subgrade

If the R-value of subgrade exposed during construction comes into question, the exposed soils should be sampled and tested during grading to verify the R-value. In locations where structural pavement sections will be constructed atop import fill (Select) materials, the Select Material placed within four feet of the grading plane should have a minimum R-value of 40 and a Plasticity Index of less than or equal to 12%.

Prior to placing the pavements, the exposed subgrade should be scarified to minimum depth of 8 inches, moisture conditioned and compacted to at least 90% relative compaction based on the maximum densities determined by ASTM D1557. Special care is required when compacting

soils near buried utility lines, drainage devices, and other sensitive buried structures, to avoid damaging the existing facilities.

11.4 Pavement Materials and Construction

All materials should meet the Standard Specifications for Public Works Construction (Greenbook). Aggregate base materials should meet the gradation and quality requirement of Crushed Aggregate Base (¾ inch maximum) in the Greenbook.

Preventative maintenance should be planned and provided through an on-going pavement management program to enhance future pavement performance. Preventative maintenance may consist of both localized maintenance (such as crack sealing or patching) and global maintenance (such as surface sealing or mill and overlay) and are intended to slow the rate of pavement deterioration.

12.0 CONSTRUCTION CONSIDERATIONS

12.1 Construction Advisories

If variations in subsurface soils or conditions from those described herein are observed during construction, the geotechnical designer should be contacted to verify the applicability of the recommendations. Recommendations in this report may require modification based on actual subsurface conditions encountered during construction.

All work should be performed in accordance with the Greenbook or California Building Code (CBC) or City of Oceanside Grading Regulations Manual, except as indicated in the Special Provisions prepared for the project.

Earthwork on the project should be observed and evaluated by geotechnical engineer. The evaluation of earthwork should include observation and testing of engineered fill, subgrade preparation, foundation soils, and other geotechnical conditions exposed during the construction of the project.

The Contractor should conform to all applicable occupational safety and health standards, rules, regulations, and orders established by the State of California. In addition, other State, County, or Municipal regulations may supersede the recommendations presented herein and should be satisfied. The contractor is responsible for the safety of workers in the field during construction.

Design of temporary shoring and temporary cut or fill slopes shall be made the contractor's responsibility during construction. Temporary cut slopes should be in accordance with all applicable codes and regulations including State, local, and OSHA requirements. If a shoring design plan is required, qualified geotechnical personnel should review the plan and provide necessary geotechnical recommendations.

Heavy construction equipment should not be used immediately adjacent to shoring due to large lateral pressures induced by such equipment unless the shoring is designed to accommodate such pressures. Proper measures should be taken to prevent damage to adjacent existing structures and utilities. Excavated soil or construction materials should not be stockpiled adjacent to shoring or open excavations. Stockpiled soil and construction materials should be set back a distance at least equal to the height of the excavation.

12.2 Groundwater Control

During the field explorations performed by EMI and groundwater well installed within the project site, groundwater was encountered between elevations -10 and +8.5 ft (depth of 8.3 to 38.3 ft below existing grade at the time of geotechnical explorations). Therefore, groundwater should be expected to be encountered during construction. Due to the proximity of the site to the Pacific Ocean, groundwater is anticipated to be affected by tidal fluctuations and may vary throughout the day. Groundwater elevations around the site may also be subject to seasonal rainfall fluctuation and runoff amountand other manmade conditions. Therefore, the groundwater elevation during construction may be higher or lower than the elevation encountered during the field explorations.

It shall be made the contractor's responsibility to control subsurface and surface water. The contractor should dewater the site as necessary, if groundwater is encountered. Contractors should also be cognizant that any dewatering activities could induce ground subsidence which affects adjacent surface and subsurface structures and utilities. Water should not be allowed to stand in any excavations. If excavations become flooded, at-least the bottom 8 inches of soil should be removed and replaced, and re-compacted to a minimum 90 percent relative compaction. Additional removals may be required at the discretion of the resident engineer or geotechnical personnel. Any seepage or groundwater removed from an excavation should be tested and disposed of in compliance with all applicable local, state and federal requirements.

12.3 Site Preparation

Strip and remove existing foundations, pavements, debris, vegetation, and other deleterious materials from proposed building and pavement areas. Demolition of the existing buildings and/or any other structures should include complete removal of the existing foundations. Exposed surfaces should be free of mounds and depressions which could prevent uniform compaction. The site should be graded to create a relatively level surface to receive the engineered fill beneath proposed building structures.

If fill materials are encountered within the proposed building areas, those fill materials should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction. Furthermore, any existing utilities, if encountered, should be removed or relocated and the excavation thoroughly cleaned prior to backfill placement and/or construction.

12.4 Foundation and Slab Subgrade Preparation

Spread footings for proposed retaining walls and buildings will be founded below existing grades within in-situ soils. The minimum remedial overexcavation depth of 1-ft is recommended for proposed spread footings and floor slabs. The recommended minimum overexcavation depth is measured from the footing bottom. The horizontal limits of the overexcavation should extend laterally, from the outside edges of the spread footings, a minimum distance equal to the overexcavation depth.

Upon completion of the recommended minimum remedial overexcavations, the overexcavation bottoms should be scarified to about eight inches deep, conditioned to near optimum moisture content, and compacted to at least 90% relative compaction based on maximum densities determined by ASTM D1557. The overexcavation bottom should be inspected by qualified geotechnical personnel to confirm that the subgrade is firm and unyielding. After the processed overexcavation bottom has been inspected and accepted, engineered fill materials should be used to backfill between the bottom of the overexcavation and bottom of the footing and floor slabs as shown in Section 10.2. Engineered fill materials should be placed in loose lifts not exceeding eight inches thick, conditioned to near optimum moisture content, and compacted to a minimum relative compaction of 90% based on maximum densities determined by ASTM D1557.

If soft or wet ground conditions are encountered during construction (overexcavation bottom is not firm and unyielding), the soft or wet bottom will require stabilization prior to placing the engineered fill materials. If the required compaction cannot be achieved due to soft or wet

subgrade, then the subgrade bottom will require stabilization. Stabilization can be accomplished by overexcavating and placing a combination of gravel, aggregate base, geogrid, or geofabric. One conventional approach is to overexcavate the subgrade soils (generally 18 to 24 inches deep), place woven fabric on the overexcavated bottom, and place aggregate base in lifts on top of the fabric to create a stable subgrade. When using this approach, EMI recommends constructing test areas with various overexcavation depths to determine a suitable depth. A second approach to mitigating pumping conditions is to scarify/aerate the soils in the upper one to two feet to reduce moisture content. However, this is a sensitive operation because scarifying and recompacting the surface soils can cause pumping of the underlying soils. This approach requires time in the schedule to scarify and dry out the upper soils and therefore may not be a feasible alternative. Cold, rainy, and cloudy weather will cause soil to retain moisture. Also, the size and weight of the construction equipment influences whether pumping will occur.

12.5 Drilled Shaft Construction Considerations

Project specifications for the drilled shafts should reference Section 49-3 of the Caltrans Standard Specification (2024). Loose soils should be cleaned from the bottom of the drilled excavations. Pile borings should be inspected and approved by the geotechnical engineer prior to the installation of reinforcement. Extreme care in drilling, placement of steel, and the pouring of concrete is essential to avoid excessive disturbance of pile boring walls. Concrete placement by pumping or tremie tube to the bottom of the pile borings will be required. Sufficient space should be provided in the pile reinforcing cage during fabrication to allow the insertion of a tremie tube for concrete placement.

Drilled shaft excavations should be continuously monitored where they are located adjacent to existing structure footings. If caving soils are encountered and the hole becomes oversized during excavation, temporary casing should be used to prevent compromising soil supporting the existing structure foundation.

Exploratory borings performed for this project encountered difficult drilling conditions through cemented bluff materials and marine deposits due to the presence of dense to very dense sand, gravels, and cobbles. The contractor should anticipate that penetration will be slow; casing (if used) installation into these materials will also be difficult. Hard drilling should be anticipated.

Groundwater was encountered between elevations -10 and +8.5 ft during drilling of the recent field investigations. Actual groundwater elevations may be different during construction due to conditions described in Section 12.2. If groundwater is encountered during pile excavation, a “wet” method of construction will be required. As a standard Caltrans practice for “wet” construction, PVC tubings must be installed within the reinforcement cage of the CIDH pile for gamma-ray testing per Caltrans Memo-To-Designers 3-1 (2014b).

If caving occurs above groundwater, a temporary casing may be used to prevent any damages to the adjacent existing footings. Casings should have an outer diameter equal to or exceeding the pile diameter. Temporary casing should be pulled as the concrete is being poured while always maintaining at least a 5 ft head of concrete inside the casing. The contractor can choose to use a “wet” method of construction to control caving. The use of driven or vibrated temporary casing must be approved by the City of Oceanside due to the potential for ground disturbance of the

surrounding bluffs. In the event that any boring becomes bell-shaped and cannot be advanced due to severe caving, all loose material should be removed from the bottom of the boring and the caved region filled with a low strength sand-cement slurry. Drilling may continue when the slurry has reached its initial set.

Upon reaching the specified tip elevations, shaft bottoms should be machine cleaned or air-vacuumed. The pile reinforcing cage should be installed and the concrete pumped, immediately after drilling is completed.

12.6 Engineered Fill Materials and Placement

All fill materials should be free of organic and inorganic materials, vegetation, debris and fragments larger than 6 inches in size. Pea gravel or other similar materials should not be used as fill or backfill materials without prior approval from the geotechnical engineer.

Engineered fill materials should have minimum sand equivalent of 20, plasticity index of less than 12 and should comply with the following gradation requirements:

Table 11. Engineered Fill Gradation Requirements

Sieve Size	Percentage Passing
3”	100
No. 4	50-100
No. 200	10-40

On-site materials may be used as engineered fill materials provided on-site materials are processed to meet the engineered fill material specifications. Imported materials to be used as engineered fill materials should meet the engineered fill material specifications.

Engineered fill materials should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift. Fill lifts should not exceed 8 inches loose thickness.

12.7 Grading and Drainage

Positive drainage should be provided during construction and maintained throughout the life of the development. Infiltration of water into utility trenches or foundation excavations should be prevented during construction. Planters and other surface features which could retain water in areas adjacent to the building or pavements should be sealed or eliminated. In areas where sidewalks or paving do not immediately adjoin the structure, we recommend that protective slopes be provided with a minimum grade of approximately 5 percent for at least 10 feet from perimeter walls. Backfill against footings, exterior walls, and in utility and sprinkler line trenches should be well compacted and free of all construction debris to reduce the possibility of moisture infiltration.

We recommend a minimum horizontal setback distance of 10 feet from the perimeter of any building and the high-water elevations such as storm-water retention basin.

Roof drainage should discharge into splash blocks or extensions when the ground surface beneath such features is not protected by exterior slabs or paving. Sprinkler systems and landscaped irrigation should not be installed within 5 feet of foundation walls.

12.8 Retaining Wall Backfill and Drainage

Engineered fill materials should be used as backfill material behind the retaining wall. Backfill should be placed in loose lifts not exceeding 8 inches in thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The relative compaction should be based on the maximum density determined by ASTM 1557. Jetting or flooding to compact backfill is not recommended. Heavy compaction equipment, such as vibratory rollers, dozers, or loaders, should not be used adjacent to the retaining wall in order to avoid damaging the wall due to large lateral earth pressures.

Backdrains should be installed behind the retaining wall to relieve hydrostatic pressure. Backdrains should be constructed in accordance with the wall plans.

12.9 Utility Trenches

It is anticipated that the on-site soils and fill materials will provide suitable support for underground utilities and pipes. Any soft and/or unsuitable material encountered at the bottom of excavations should be removed and be replaced with an adequate bedding material. A non-expansive granular material with a sand equivalent greater than 30 should be used as bedding materials, unless allowed or specified otherwise by the utility manufacturer.

On-site materials are considered suitable for backfill of utility and pipe trenches from one foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances. Engineered fill materials should be used for trench backfill in structural areas.

Trench backfill should be mechanically placed and compacted to a minimum relative compaction of 90% based on maximum densities determined by ASTM D1557. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and expansion index requirements of engineered fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended.

12.10 Exterior Slab Design Considerations

Exterior slabs-on-grade, exterior architectural features, and utilities founded on, or in backfill may experience some movement due to the volume change of the backfill. To reduce the potential for damage caused by movement, we recommend:

- Exterior slabs should be supported directly on subgrade fill with no, or very low expansion potential. Subgrade soils beneath exterior slabs should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted to a minimum relative compaction of 90% based on maximum densities determined by ASTM

D1557. The moisture content and compaction of subgrade soils should be maintained until slab construction

- Placement of effective control joints on relatively close centers and isolation joints between slabs and other structural elements
- Provide adequate drainage in areas adjoining the slabs
- Design the slabs to allow vertical movement between the exterior slabs and adjoining structural elements

12.11 Geotechnical Observation and Testing

Qualified geotechnical personnel should perform inspections and testing during the following stages of construction:

- Grading operations, including temporary and permanent excavations, remedial excavations, and placement of compacted fill.
- Excavations and remedial earthwork for structure foundations.
- Spread footing and drilled shaft construction.
- Excavations for utility trenches and other drainage structures.
- Placement of bedding and backfill in utility and culvert trenches.
- Removal or support of buried utilities or structures.
- Shoring installation, if necessary.
- When any unusual conditions are encountered.

For areas where cuts and temporary excavations are planned near existing structures, the need for monitoring vertical and horizontal movement of the existing structures and utilities should be evaluated by the Resident Engineer, construction manager, and contractor.

The exposed subgrade and each lift of compacted fill should be tested, evaluated, and reworked as necessary until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 2,500 square feet of compacted fill in the building areas and 5,000 square feet in pavement areas. One density and water content test for every 50 linear feet of compacted utility trench backfill.

13.0 REFERENCES

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APPENDIX A

BORING LOGS

LOG OF BORING NO. A-24-001

Grade Elevation:	**38 ft**				
Boring Depth:	**81.5 ft**	Driller:	**2R Drilling**	SHEET	**1 of 4**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**9-30-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 43.08" / Longitude: -117° 23' 3.48"	
Logged By:	**NK**	Groundwater Reading:	**38.0 ft on 9-30-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					SILTY SAND (SM); brown; moist; mostly coarse to fine SAND; little nonplastic fines.				
5		1	27		Dense.		9		CR
10		2	48		Poorly-graded SAND with SILT (SP-SM); dense; brown; moist; mostly coarse to fine SAND; few nonplastic fines.		4	108	
15		3	38		Poorly-graded SAND (SP); very dense; yellowish brown; moist; mostly medium to fine SAND; trace nonplastic fines.				
20		4	58		Dense; light brown; coarse to fine SAND; hard drilling and rig chattered.		2	98	
25									

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

(CONTINUED) LOG OF BORING NO. A-24-001

Date Drilled:	9-30-24	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 4**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25									
		5	43		Very dense; few fine, angular GRAVEL, max. 3/8 in. dia.. Poorly-graded SAND (SP) *(continued)*.				
30		6	REF						
35		7	42		SILTY SAND (SM); very dense; light brown; moist; mostly medium to fine SAND; little nonplastic fines.				
40		8	79/10		1% fine GRAVEL, max. 1/5 in. dia.; 77% SAND; 22% fines.		28	99	W
45		9	42		Poorly-graded SAND with SILT (SP-SM); very dense; light brown; moist; few fine GRAVEL, max. 1/5 in. dia.; mostly coarse to fine SAND; few nonplastic fines.				
50		10	50/3 "		Poorly-graded SAND (SP); very dense; light brown; moist; mostly coarse to fine SAND; trace nonplastic fines.		25	106	

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Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-001

Date Drilled:	9-30-24	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **3 of 4**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
					Poorly-graded SAND (SP) *(continued)*.				
55		11	50/5 "		SILTY SAND (SM); very dense; light brown; moist; mostly medium to fine SAND; little nonplastic fines.				
60		12	86/9		Poorly-graded SAND (SP); very dense; light brown; moist; mostly medium to fine SAND; trace nonplastic fines.		17	109	
65		13	50/3 "		Few fine GRAVEL, max. 1/5 in. dia.; coarse to fine SAND.				
70		14	REF						
75		15	47		SILTY SAND (SM); very dense; brown; moist; mostly medium to fine SAND; some nonplastic fines.				

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Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-001

Date Drilled:	**9-30-24**	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **4 of 4**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
80		16	50		SILT with SAND (ML); olive brown; moist; 26% medium to fine SAND; 74% low plasticity fines.		27	97	W
					Bottom of borehole at 81.5 ft bgs				
85									
90									
95									
100									
105									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-002

Grade Elevation:	**39 ft**				
Boring Depth:	**81.5 ft**	Driller:	**2R Drilling**	SHEET	**1 of 4**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 83%	
Date Drilled:	**9-30-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 42.36" / Longitude: -117° 23' 3.12"	
Logged By:	**WD**	Groundwater Reading:	**37.8 ft on 9-30-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					SILTY SAND (SM); olive to olive gray; moist; mostly medium to fine SAND; little nonplastic fines.				
5		1	23		Dense; olive yellow.		14		
10		2	39		Poorly-graded SAND (SP); dense; olive yellow; moist; mostly medium to fine SAND; trace nonplastic fines.		2	103	DS
15		3	25		Olive.				
20		4	54				2		
25									

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Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

(CONTINUED) LOG OF BORING NO. A-24-002

Date Drilled:	**9-30-24**	Comments: Hammer Energy Efficiency: 83% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 4**
Logged By:	**WD**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25		5	20		Poorly-graded SAND with SILT (SP-SM); medium dense; light olive gray; moist; mostly coarse to fine SAND; few nonplastic fines.				
30		6	96/11		SILTY SAND (SM); very dense; light yellowish brown; moist; mostly coarse to fine SAND; little nonplastic fines.				
35		7	37		Light yellowish brown to olive brown; moist; 13% coarse to fine, angular GRAVEL, max. 1 in. dia.; 73% SAND; 14% fines.		20		W
40		8	78/11		Poorly-graded SAND with SILT (SP-SM); very dense; olive brown; wet; few fine GRAVEL, max. 1/2 in. dia.; mostly coarse to fine SAND; few nonplastic fines.		13	115	
45		9	70						
50		10	50/3 "		Poorly-graded SAND (SP); very dense; olive brown; wet; mostly coarse to fine SAND; trace nonplastic fines.		27	79	

(continued)

(CONTINUED) LOG OF BORING NO. A-24-002

Date Drilled:	**9-30-24**	Comments: Hammer Energy Efficiency: 83% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **3 of 4**
Logged By:	**WD**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
					Poorly-graded SAND (SP) *(continued)*.				
55		11	85/11		Poorly-graded SAND with SILT (SP-SM); very dense; olive brown; wet; mostly medium to fine SAND; few nonplastic fines.				
60		12	50/4 "		Olive.				
65		13	59		Light olive gray; few fine GRAVEL, max. 1/4 in. dia..				
70		14	50		SILT with SAND (ML); olive brown; moist; 26% fine SAND; 74% low plasticity fines.		28	97	W
75		15	20						

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-002

Date Drilled:	**9-30-24**	Comments: Hammer Energy Efficiency: 83% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **4 of 4**
Logged By:	**WD**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
80					SILT with SAND (ML) *(continued)*. 27% SAND; 73% fines.				
		16	35				31	92	W
					Bottom of borehole at 81.5 ft bgs				
85									
90									
95									
100									
105									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-003

Grade Elevation:	**14 ft**				
Boring Depth:	**16.5 ft**	Driller:	**2R Drilling**	SHEET	**1 of 1**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 83%	
Date Drilled:	**10-3-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 41.28" / Longitude: -117° 23' 3.12"	
Logged By:	**WD**	Groundwater Reading:	**8.3 ft on 10-3-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND (SP); olive brown; moist; 1% fine GRAVEL, max. 3/4 in. dia.; 94% medium to fine SAND; 5% nonplastic fines.				
		0					3		W
5					Loose.				
		1	6				7		
10					Medium dense; dark gray; wet.				
		2	38				21	99	
15					Poorly-graded SAND with SILT (SP-SM); very dense; olive gray; wet; mostly medium to fine SAND; few nonplastic fines.				
		3	41				22		
					Bottom of borehole at 16.5 ft bgs				
20									
25									

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

LOG OF BORING NO. A-24-004

Grade Elevation:	**11 ft**				
Boring Depth:	**15.8 ft**	Driller:	**2R Drilling**	SHEET **1 of 1**	
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-1-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 42.72" / Longitude: -117° 23' 5.28"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND (SP); light brown; moist; mostly medium to fine SAND; trace nonplastic fines.				
5		1	11		Medium dense.		25		
10		2	31		Few fine GRAVEL, max. 1/5 in. dia.; coarse to fine SAND.		20	105	CR, DS
15		3	50/3 "		Very dense.		22		
					Bottom of borehole at 15.8 ft bgs				
20									
25									

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

LOG OF BORING NO. A-24-005

Grade Elevation:	**15 ft**				
Boring Depth:	**16.5 ft**	Driller:	**2R Drilling**	SHEET **1 of 1**	
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-2-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 40.92" / Longitude: -117° 23' 3.84"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND (SP); light gray; moist; mostly medium to fine SAND; trace nonplastic fines.				
5		1	7		Loose.		2		
10		2	50		Dense.		20	103	
15		3	83		Very dense; few fine GRAVEL, max. 3/8 in. dia.; coarse to fine SAND.		21		
					Bottom of borehole at 16.5 ft bgs				
20									
25									

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

LOG OF BORING NO. A-24-012

Grade Elevation:	**37 ft**				
Boring Depth:	**50.5 ft**	Driller:	**2R Drilling**	SHEET	**1 of 2**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-1-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 40.2" / Longitude: -117° 23' 0.96"	
Logged By:	**NK**	Groundwater Reading:	**41.0 ft on 10-1-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					SILTY SAND (SM); light brown; moist; mostly medium to fine SAND; little nonplastic fines.				
5		1	21		Medium dense.		9		CR
10		2	35		Poorly-graded SAND (SP); medium dense; light brown; moist; few fine GRAVEL, max. 1/5 in. dia.; mostly medium to fine SAND; trace nonplastic fines.		2	113	DS
15		3	37		Dense.				
20		4	39		Medium dense.		4	107	
25									

(continued)

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Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-012

Date Drilled:	**10-1-24**	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25					Very dense. Poorly-graded SAND (SP) *(continued)*.				
		5	66						
30									
		6	90				7	108	
35		7	50/4 "		Coarse to fine SAND.				
40									
		8	90/10				7	116	
45		9	REF						
50		10	REF						
					Bottom of borehole at 50.5 ft bgs				

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-013

Grade Elevation:	**37 ft**				
Boring Depth:	**50.8 ft**	Driller:	**2R Drilling**	SHEET	**1 of 2**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 83%	
Date Drilled:	**10-1-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 39.84" / Longitude: -117° 23' 0.6"	
Logged By:	**WD**	Groundwater Reading:	**38.3 ft on 10-1-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND with SILT (SP-SM); olive; moist; mostly fine SAND; few nonplastic fines.				
5		1	16		Medium dense.				
10		2	37		Poorly-graded SAND (SP); medium dense; yellowish brown; moist; trace fine GRAVEL, max. 1/5 in. dia.; mostly coarse to fine SAND; trace nonplastic fines.		1	105	DS
15		3	34		Poorly-graded SAND with SILT (SP-SM); dense; yellowish brown; moist; mostly coarse to fine SAND; few nonplastic fines.				
20		4	51		Poorly-graded SAND (SP); dense; olive; moist; mostly fine SAND; trace nonplastic fines.		3	95	
25									

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EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-013

Date Drilled:	**10-1-24**	Comments: Hammer Energy Efficiency: 83% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**WD**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25					Very dense; light olive gray. Poorly-graded SAND (SP) *(continued)*.				
		5	73						
30					Olive gray.				
		6	71				7	101	
35					Light olive gray.				
		7	94/11						
40									
		8	50/5 "						
45					Wet.				
		9	63				16		
50									
		10	50/3 "						
					Bottom of borehole at 50.8 ft bgs				

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-014

Grade Elevation:	**39 ft**				
Boring Depth:	**50.3 ft**	Driller:	**2R Drilling**	SHEET **1 of 2**	
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-2-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 38.4" / Longitude: -117° 22' 59.52"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					SILTY SAND (SM); reddish brown; moist; mostly medium to fine SAND; little nonplastic fines.				
5		1	12		Medium dense.				
10		2	69		Very dense.		6	119	CR
15		3	28		Poorly-graded SAND with SILT (SP-SM); dense; reddish brown; moist; mostly medium to fine SAND; few nonplastic fines.				
20		4	50		Poorly-graded SAND (SP); dense; light brown; moist; mostly medium to fine SAND; trace nonplastic fines.		4	103	
25									

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-014

Date Drilled:	**10-2-24**	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25					Very dense. Poorly-graded SAND (SP) *(continued)*.				
		5	76						
30					Trace fine GRAVEL, max. 3/4 in. dia.; coarse to fine SAND; hard drilling and rig chattered.				
		6	93				3	87	
35		7	REF		Few GRAVEL, max. 1/5 in. dia.; hard drilling and rig chatter continued.				
40		8	REF		Hard drilling and rig chatter continued.				
45		9	REF		CLAYEY SAND (SC); very dense; light brown to olive brown; moist; few fine GRAVEL, max. 3/4 in. dia.; mostly coarse to fine SAND; little low plasticity fines; hard drilling and rig chatter continued.		14		
50		10	REF		Poorly-graded SAND (SP); very dense; light brown; moist; few fine GRAVEL, max. 3/8 in. dia.; mostly coarse to fine SAND; trace nonplastic fines.				
					Bottom of borehole at 50.3 ft bgs				

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-015

Grade Elevation:	**18 ft**				
Boring Depth:	**15.9 ft**	Driller:	**2R Drilling**	SHEET	**1 of 1**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 83%	
Date Drilled:	**10-1-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 39.48" / Longitude: -117° 23' 1.68"	
Logged By:	**WD**	Groundwater Reading:	**9.5 ft on 10-1-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND with SILT (SP-SM); dark brown; moist; mostly medium to fine SAND; few nonplastic fines.				
		0					6		
5					Poorly-graded SAND (SP); loose; dark grayish brown; moist; mostly medium to fine SAND; trace nonplastic fines.				
		1	5				4		CR
10					Very dense.				
		2	90						
15					Wet.				
		3	50/5 "				19	103	
					Bottom of borehole at 15.9 ft bgs				
20									
25									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-016

Grade Elevation:	**16 ft**				
Boring Depth:	**15.2 ft**	Driller:	**2R Drilling**	SHEET **1 of 1**	
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-2-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 38.4" / Longitude: -117° 23' 0.6"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND (SP); light brown; moist; mostly coarse to fine SAND; trace nonplastic fines.				
5		1	53		Very dense; hard drilling and rig chattered.		6		
10		2	50/3 "		SILTY SAND with GRAVEL (SM); very dense; olive brown; moist; 21% fine, angular GRAVEL, max. 1/2 in. dia.; 63% coarse to fine SAND; 16% nonplastic fines; hard drilling and rig chatter continued.		7	129	W
15		3	REF		Poorly-graded GRAVEL with SILT and SAND (GP-GM); very dense; olive brown; moist; mostly fine GRAVEL, max. 3/4 in. dia.; little coarse to fine SAND; few nonplastic fines.		6		
					Bottom of borehole at 15.2 ft bgs				
20									
25									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-017

Grade Elevation:	**14 ft**				
Boring Depth:	**16.5 ft**	Driller:	**2R Drilling**	SHEET	**1 of 1**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-2-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 38.04" / Longitude: -117° 23' 1.32"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND (SP); light brown; moist; mostly medium to fine SAND; trace nonplastic fines.				
5		1	8		Medium dense.		4		CR
10		2	33		Hard drilling and rig chattered.		16	105	
15		3	68		Very dense.		14		
					Bottom of borehole at 16.5 ft bgs				
20									
25									

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

LOG OF BORING NO. A-24-018

Grade Elevation:	**37 ft**				
Boring Depth:	**34.0 ft**	Driller:	**2R Drilling**	SHEET	**1 of 2**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-1-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 37.32" / Longitude: -117° 22' 58.44"	
Logged By:	**NK**	Groundwater Reading:	**34.0 ft on 10-1-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND with SILT (SP-SM); reddish brown; moist; mostly medium to fine SAND; few nonplastic fines.				
5		1	52		Very dense.				
10		2	26		Medium dense; light brown; moist.		8	116	DS
15		3	58		Poorly-graded SAND (SP); very dense; light brown; moist; mostly medium to fine SAND; trace nonplastic fines.				
20		4	49		Dense; trace fine GRAVEL, max. 1/2 in. dia.; coarse to fine SAND.		4	105	
25									

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-018

Date Drilled:	**10-1-24**	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25									
		5	50/3 "		Very dense; hard drilling and rig chattered. Poorly-graded SAND (SP) *(continued)*.				
30					Encountered COBBLES; hard drilling and rig chatter continued.				
					Encountered refusal in potential COBBLE layers. Bottom of borehole at 34.0 ft bgs				
35									
40									
45									
50									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-019

Grade Elevation:	**35 ft**				
Boring Depth:	**50.2 ft**	Driller:	**2R Drilling**	SHEET	**1 of 2**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 83%	
Date Drilled:	**10-2-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 35.52" / Longitude: -117° 22' 57"	
Logged By:	**WD**	Groundwater Reading:	**45.0 ft on 10-2-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					SILTY SAND (SM); yellowish brown; moist; 2% fine GRAVEL, max. 3/8 in. dia.; 67% coarse to fine SAND; 31% nonplastic fines.				
		0					7		W
5					Medium dense; medium to fine SAND; little fines.				
		1	11				11		CR
10					Poorly-graded SAND (SP); dense; olive; moist; mostly coarse to fine SAND; trace nonplastic fines.				
		2	38				1	95	
15									
		3	30						
20					Very dense.				
		4	56				1	87	
25									

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Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

(CONTINUED) LOG OF BORING NO. A-24-019

Date Drilled:	**10-2-24**	Comments: Hammer Energy Efficiency: 83% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**WD**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25		5	35		Light olive gray; hard drilling and rig chattered. Poorly-graded SAND (SP) *(continued)*.		5		
30		6	50/5 "		Poorly-graded SAND with GRAVEL (SP); very dense; olive gray; moist; little coarse to fine GRAVEL, max. 3 in. dia.; mostly coarse to fine SAND; trace nonplastic fines; encountered COBBLES; hard drilling and rig chatter continued.				
35		7	50/5 "		Hard drilling and rig chatter continued.				
40		8	REF		CLAYEY SAND with GRAVEL (SC); very dense; pale olive; moist; little coarse to fine GRAVEL, max. 3 in. dia.; mostly coarse to fine SAND; little low plasticity fines; encountered COBBLES; hard drilling and rig chatter continued.		14	125	
45		9	REF		Wet; medium to fine SAND; some fines; encountered COBBLES; hard drilling and rig chatter continued.				
50		10	REF		Bottom of borehole at 50.2 ft bgs				

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-021

Grade Elevation:	**14 ft**				
Boring Depth:	**50.3 ft**	Driller:	**2R Drilling**	SHEET	**1 of 2**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-3-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 36.24" / Longitude: -117° 22' 58.44"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND with GRAVEL (SP); light brown; moist; little fine GRAVEL, max. 3/8 in. dia.; mostly coarse to fine SAND; trace nonplastic fines.				
5		1	50		Very dense; hard drilling and rig chattered.				
10		2	REF		SILTY SAND with GRAVEL (SM); very dense; light brown; moist; 37% fine GRAVEL, max. 1/2 in. dia.; 43% coarse to fine SAND; 20% nonplastic fines; hard drilling and rig chatter continued.		10	129	CR, W
15		3	REF		Olive brown; hard drilling and rig chatter continued.				
20		4	50/2 "		Little GRAVEL, max. 3/4 in. dia.; hard drilling and rig chatter continued.		9	135	
25									

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-021

Date Drilled:	**10-3-24**	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25		5	REF		Poorly-graded SAND (SP); very dense; light brown; moist; few fine GRAVEL, max. 3/8 in. dia.; mostly coarse to fine SAND; trace nonplastic fines.				
30		6	REF		Poorly-graded SAND with SILT (SP-SM); very dense; light brown; moist; trace fine GRAVEL, max. 1/2 in. dia.; mostly coarse to fine SAND; few nonplastic fines; hard drilling and rig chattered.		13	121	
35		7	REF		SANDY SILT with GRAVEL (ML); very dense; olive brown; moist; little fine GRAVEL, max. 3/8 in. dia.; little coarse to fine SAND; mostly nonplastic fines; hard drilling and rig chatter continued.				
40		8	50/1 "		CLAYEY SAND with GRAVEL (SC); very dense; olive brown; moist; 28% fine GRAVEL, max. 1/2 in. dia.; 41% coarse to fine SAND; 31% low plasticity fines; hard drilling and rig chatter continued.		13	138	W
45		9	REF		Poorly-graded SAND with SILT (SP-SM); very dense; light brown; moist; few fine GRAVEL, max. 3/8 in. dia.; mostly coarse to fine SAND; few nonplastic fines; hard drilling and rig chatter continued.				
50		10	REF		Poorly-graded GRAVEL with SILT and SAND (GP-GM); very dense; light brown; moist; mostly coarse to fine GRAVEL, max. 2 in. dia.; little coarse to fine SAND; few nonplastic fines.		9	119	
					Bottom of borehole at 50.3 ft bgs				

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-022

Grade Elevation:	**15 ft**				
Boring Depth:	**25.2 ft**	Driller:	**2R Drilling**	SHEET **1 of 2**	
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 83%	
Date Drilled:	**10-3-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 34.8" / Longitude: -117° 22' 57.36"	
Logged By:	**WD**	Groundwater Reading:	**9.4 ft on 10-3-24**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND with SILT (SP-SM); dark yellowish brown; moist; mostly medium to fine SAND; few nonplastic fines.				
5		1	8		Medium dense.				
10		2	50/5 "		Poorly-graded GRAVEL with SILT and SAND (GP-GM); very dense; olive; wet; mostly coarse to fine GRAVEL, max. 3 in. dia.; little coarse to fine SAND; few nonplastic fines; encountered COBBLES; hard drilling and rig chattered.		3	135	
15		3	65		Poorly-graded SAND with GRAVEL (SP); very dense; olive; wet; little coarse to fine GRAVEL, max. 2 in. dia.; mostly coarse to fine SAND; trace nonplastic fines; hard drilling and rig chatter continued.		10		
20		4	50/5 "		CLAYEY SAND with GRAVEL (SC); very dense; olive; wet; 35% coarse to fine GRAVEL, max. 2 in. dia.; 45% coarse to fine SAND; 20% low plasticity fines; hard drilling and rig chatter continued.		12	132	W
25									

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Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-022

Date Drilled:	**10-3-24**	Comments: Hammer Energy Efficiency: 83% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**WD**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25			REF		Bottom of borehole at 25.2 ft bgs				
30									
35									
40									
45									
50									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

LOG OF BORING NO. A-24-024

Grade Elevation:	**12 ft**				
Boring Depth:	**25.4 ft**	Driller:	**2R Drilling**	SHEET	**1 of 2**
Borehole Diameter:	**8"**	Type of Rig:	**CME 75**	Comments: Hammer Energy Efficiency: 81%	
Date Drilled:	**10-3-24**	Hammer Data:	**140 lbs automatic hammer drops 30-inch**	Latitude: 33° 11' 35.16" / Longitude: -117° 22' 59.16"	
Logged By:	**NK**	Groundwater Reading:	**Not Encountered**	Boring Backfill: Soil Cuttings with Concrete Cap	

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
0					ASPHALT Existing Pavement.				
					Poorly-graded SAND (SP); light brown; moist; mostly medium to fine SAND; trace nonplastic fines.				
5		1	7		Loose.		3		CR
10		2	47		Dense; hard drilling and rig chattered.				
15		3	50/5 "		Very dense; trace coarse to fine GRAVEL, max. 1 in. dia.; coarse to fine SAND.		20		
20		4	REF		CLAYEY SAND with GRAVEL (SC); very dense; light brown; moist; little fine GRAVEL, max. 3/4 in. dia.; mostly coarse to fine SAND; little low plasticity fines; hard drilling and rig chattered.		11	133	
25									

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137 | Date: 1-14-25

(CONTINUED) LOG OF BORING NO. A-24-024

Date Drilled:	**10-3-24**	Comments: Hammer Energy Efficiency: 81% Boring Backfill: Soil Cuttings with Concrete Cap	SHEET **2 of 2**
Logged By:	**NK**		

Depth (ft)	Sample Type	Sample	Blows/foot	Graphic Log	GEOTECHNICAL DESCRIPTION	Drilling Method	Moisture (%)	Dry Density (pcf)	Test/ Results
25		5	REF		Poorly-graded SAND with GRAVEL (SP); very dense; light brown; moist; little fine GRAVEL, max. 3/8 in. dia.; mostly coarse to fine SAND; trace nonplastic fines.				
					Bottom of borehole at 25.4 ft bgs				
30									
35									
40									
45									
50									

EMI BORING LOG WITH DRILLING METHOD OCEANSIDE BEACHFRONT BORINGS.GPJ EMI CALTRANS 2025 V1.0.GLB 1/31/25

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Oceanside Beachfront Phase II Project

Project Number: 23-137

Date: 1-14-25

APPENDIX B

LABORATORY TESTING

TABLE B-1 SUMMARY OF LABORATORY TEST RESULTS

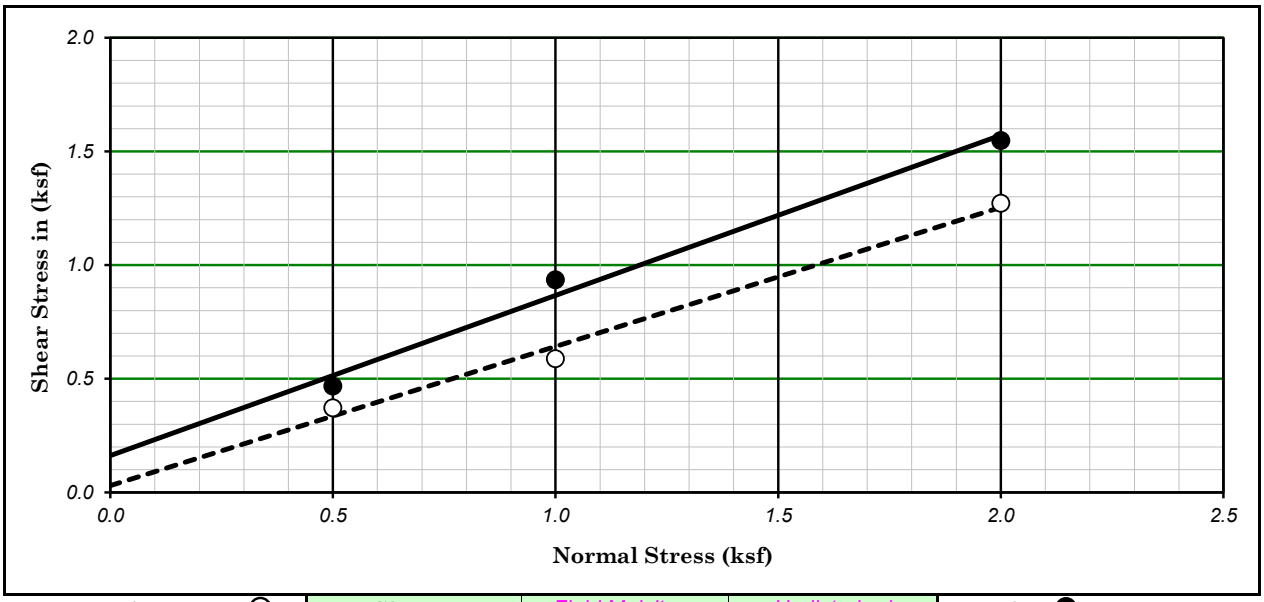
Project No. : *23-137* **Project Name :** Johnson Favaro, Oceanside Beachfront

Boring No .	Sample No.	Sample Depth (ft)	Soil Identification (group symbol) ASTM D2488/D2487	Moisture Content ASTM D2216 (%)	Total Unit Weight ASTM D2937 (pcf)	Pocket Penetrometer (tsf)	Torvane Shear (tsf)	Grain Size Distribution GR:SA:FI (%)	Sand Equivalent (CT-217)	Atterberg Limits ASTM D4318 (LL/PL/PI)	Soil-Minimum Resistivity CT-643 (ohm-cm)	Soil-pH CT-643	Soil-Soluble Sulfate Content CT-417 (ppm)	Soil-Moisture Free Chloride Content CT-422 (ppm)
A-24-001	*S-1*	5	SM	9.4							2800	9.3	396	39
A-24-001	*D-2*	10	SP-SM	4.0	112.1									
A-24-001	*D-4*	20	SP	1.7	100.1									
A-24-001	*D-8*	40	SM	28.4	127.6			1:77:22						
A-24-001	*D-10*	50	SP	25.0	132.0									
A-24-001	*D-12*	60	SP	17.1	127.7									
A-24-001	*D-16*	80	ML	27.1	123.8			0:26:74						
A-24-002	*S-1*	5	SM	14.0										
A-24-002	*D-2*	10	SP	2.1	105.6									
A-24-002	*D-4*	20	SP	2.1										
A-24-002	*S-7*	35	SM	20.0				13:73:14						
A-24-002	*D-8*	40	SP-SM	13.3	129.8									
A-24-002	*D-10*	50	SP	26.5	99.9									
A-24-002	*D14*	70	ML	27.9	124.2			0:26:74						
A-24-002	*D-16*	80	ML	30.6	120.0			0:27:73						
A-24-003	*B-0*	0	SP	2.7				1:94:5						
A-24-003	*S-1*	5	SP	6.6										
A-24-003	*D-2*	10	SP	20.6	119.8									
A-24-003	*S-3*	15	SP-SM	21.5										
A-24-004	*S-1*	5	SP	25.0										
A-24-004	*D-2*	10	SP	19.8	126.4						3300	9.1	187	138
A-24-004	*S-3*	15	SP	21.5										
A-24-005	*S-1*	5	SP	1.8										
A-24-005	*D-2*	10	SP	19.7	122.8									
A-24-005	*S-3*	15	SP	21.1										
A-24-012	*S-1*	5	SM	8.9							1800	7.7	261	263
A-24-012	*D-2*	10	SP	2.5	115.5									
A-24-012	*D-4*	20	SP	4.2	111.1									
A-24-012	*D-6*	30	SP	6.7	115.2									
A-24-012	*D-8*	40	SP	6.7	124.1									
A-24-013	*D-2*	10	SP	1.4	106.2									
A-24-013	*D-4*	20	SP	2.8	97.3									
A-24-013	*D-6*	30	SP	7.4	108.8									
A-24-013	*S-9*	45	SP	16.1										

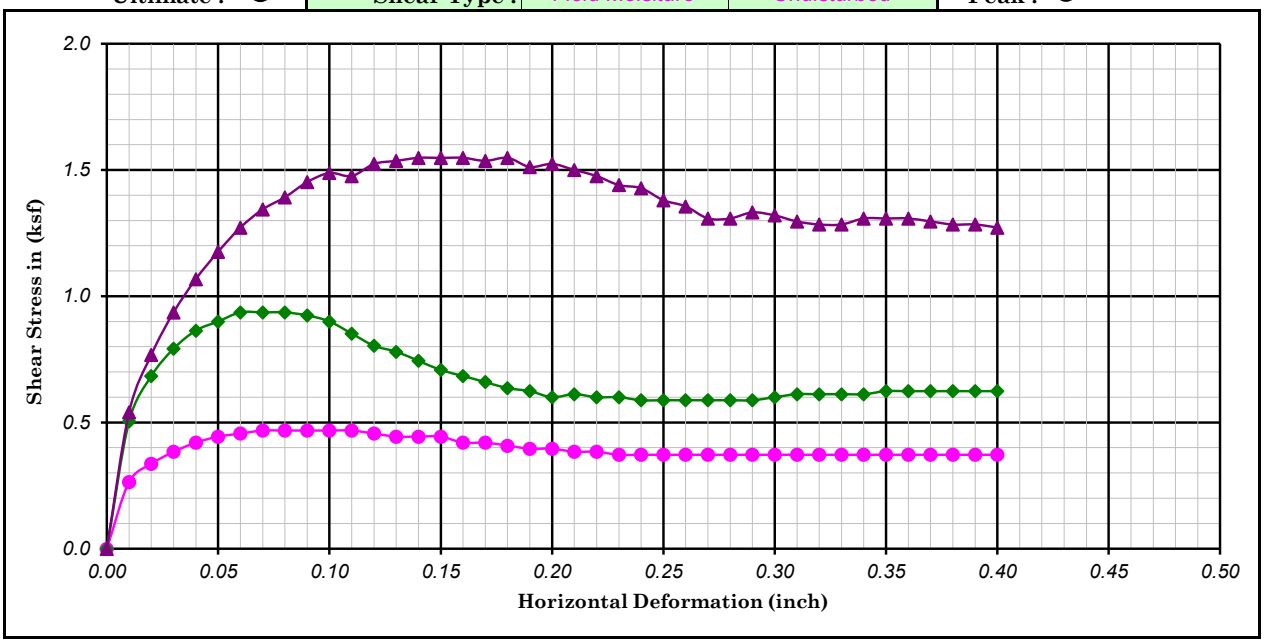
TABLE B-1 SUMMARY OF LABORATORY TEST RESULTS

Project No. : *23-137* **Project Name :** Johnson Favaro, Oceanside Beachfront

Boring No .	Sample No.	Sample Depth (ft)	Soil Identification (group symbol) ASTM D2488/D2487	Moisture Content ASTM D2216 (%)	Total Unit Weight ASTM D2937 (pcf)	Pocket Penetrometer (tsf)	Torvane Shear (tsf)	Grain Size Distribution GR:SA:FI (%)	Sand Equivalent (CT-217)	Atterberg Limits ASTM D4318 (LL/PL/PI)	Soil-Minimum Resistivity CT-643 (ohm-cm)	Soil-pH CT-643	Soil-Soluble Sulfate Content CT-417 (ppm)	Soil-Moisture Free Chloride Content CT-422 (ppm)
A-24-014	*D-2*	10	SM	5.8	125.6						3100	8.6	365	154
A-24-014	*D-4*	20	SP	4.5	107.9									
A-24-014	*D-6*	30	SP	3.4	90.1									
A-24-014	*S-9*	45	SC	13.8										
A-24-015	*B-0*	0	SP-SM	5.9										
A-24-015	*S-1*	5	SP	4.3							4400	8.1	161	168
A-24-015	*D-3*	15	SP	18.6	121.8									
A-24-016	*S-1*	5	SP	6.1										
A-24-016	*D-2*	10	SM	6.7	137.6			21:63:16						
A-24-016	*S-3*	15	GP-GM	5.6										
A-24-017	*S-1*	5	SP	3.8							12000	8.7	561	47
A-24-017	*D-2*	10	SP	16.4	122.0									
A-24-017	*S-3*	15	SP	13.7										
A-24-018	*D-2*	10	SP-SM	8.3	125.4									
A-24-018	*D-4*	20	SP	4.3	109.6									
A-24-019	*B-0*	0	SM	6.9				2:67:31						
A-24-019	*S-1*	5	SM	11.4							1000	8.4	472	547
A-24-019	*D-2*	10	SP	1.1	96.4									
A-24-019	*D-4*	20	SP	1.4	87.7									
A-24-019	*S-5*	25	SP	5.2										
A-24-019	*D-8*	40	SC	14.5	142.7									
A-24-021	*D-2*	10	SM	10.1	142.5			37:43:20			1200	8.4	1172	250
A-24-021	*D-4*	20	SM	9.1	147.5									
A-24-021	*D-6*	30	SP-SM	12.8	136.0									
A-24-021	*D-8*	40	SC	12.8	155.2			28:41:31						
A-24-021	*D-10*	50	GP-GM	8.9	129.4									
A-24-022	*D-2*	10	GP-GM	2.9	139.0									
A-24-022	*S-3*	15	SP	10.0										
A-24-022	*D-4*	20	SC	12.5	148.0			35:45:20						
A-24-024	*S-1*	5	SP	2.9							5200	7.6	139	237
A-24-024	*S-3*	15	SP	20.1										
A-24-024	*D-4*	20	SC	10.7	147.3									



Ultimate : ○ | Shear Type : Field Moisture | Undisturbed | Peak : ●



Boring No. : A-24-002		Strength Intercept (C) :	0.16	(ksf)	Peak	0.03	(ksf)	Ultimate
Sample No. : D-2			7.76	(kPa)		1.44	(kPa)	
Depth (ft/m) : 10.0	3.05	Friction Angle (ϕ) :	35.17	Degree		31.47	Degree	
Description : Olive yellow, Poorly graded SAND (SP)					Shear Rate (inch/minute) :		0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	2.11	104.29	16.41	0.62	0.50	23.94	0.47	22.41	0.37	17.81
◆	1.84	105.72	16.64	0.59	1.00	47.88	0.94	44.82	0.59	28.15
▲	1.62	106.00	16.68	0.59	2.00	95.76	1.55	74.12	1.27	60.90

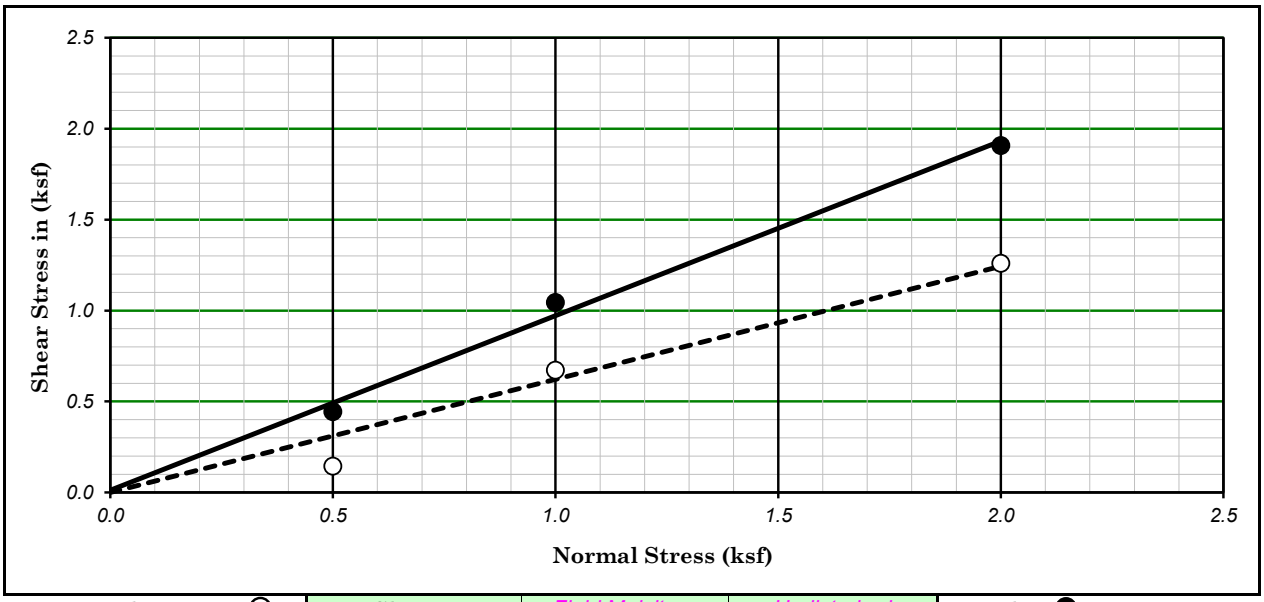
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Johnson Favaro, Oceanside Beachfront

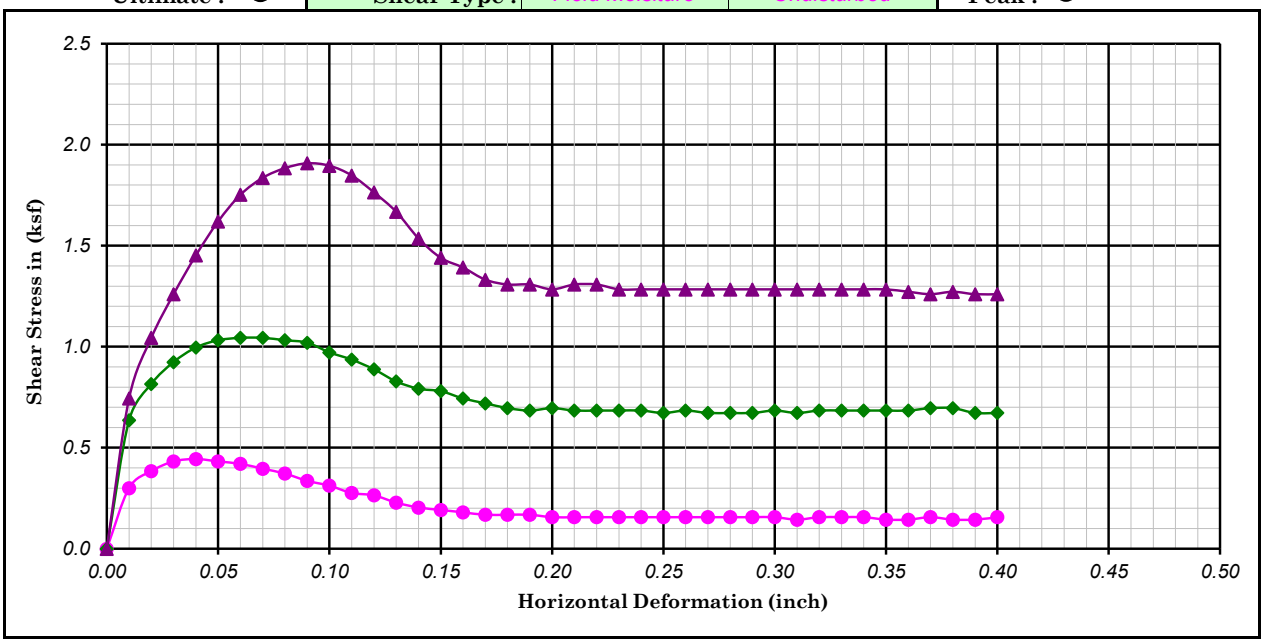
DIRECT SHEAR TEST

(ASTM D-3080)

Project No. : 23-137 | Date : 11/26/04 | Figure No.



Ultimate : ○ | Shear Type : Field Moisiture | Undisturbed | Peak : ●



Boring No. : A-24-004		Strength Intercept (C) :	0.01	(ksf)	Peak	0.00	(ksf)	Ultimate
Sample No. : D-2			0.57	(kPa)		0.00	(kPa)	
Depth (ft/m) : 10.0	3.05	Friction Angle (ϕ) :	43.83	Degree		31.87	Degree	
Description : Light brown, Poorly graded SAND (SP)					Shear Rate (inch/minute) :		0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	19.85	105.30	16.57	0.60	0.50	23.94	0.44	21.26	0.14	6.89
◆	19.84	107.33	16.89	0.57	1.00	47.88	1.04	49.99	0.67	32.18
▲	18.56	108.43	17.07	0.55	2.00	95.76	1.91	91.36	1.26	60.33

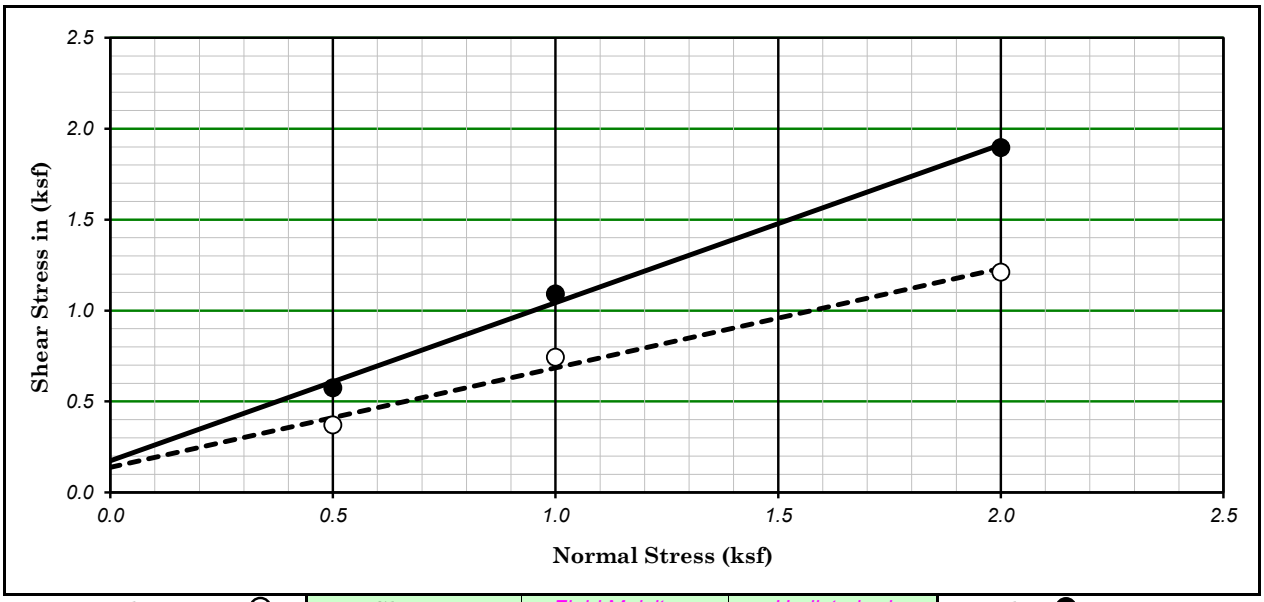
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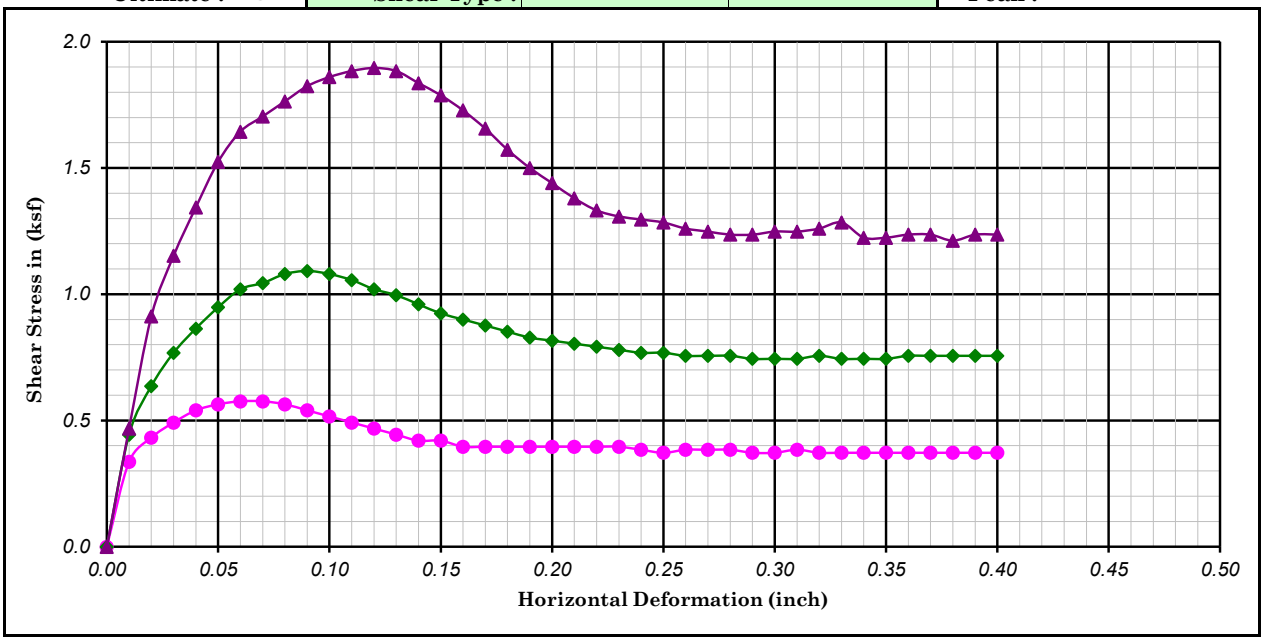
DIRECT SHEAR TEST

(ASTM D-3080)

Project No. : 23-137 | Date : 11/26/04 | Figure No.



Ultimate : ○ | Shear Type : Field Moisture | Undisturbed | Peak : ●



Boring No. : A-24-012		Strength Intercept (C) :	0.17	(ksf)	Peak	0.14	(ksf)	Ultimate
Sample No. : D-2			8.33	(kPa)		6.61	(kPa)	
Depth (ft/m) : 10.0	3.05	Friction Angle (ϕ) :	41.00	Degree		28.67	Degree	
Description : Light brown, Poorly graded SAND (SP)					Shear Rate (inch/minute) :		0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	2.46	113.93	17.93	0.48	0.50	23.94	0.58	27.58	0.37	17.81
◆	2.87	114.57	18.03	0.47	1.00	47.88	1.09	52.28	0.74	35.62
▲	2.96	112.40	17.69	0.50	2.00	95.76	1.90	90.78	1.21	58.03

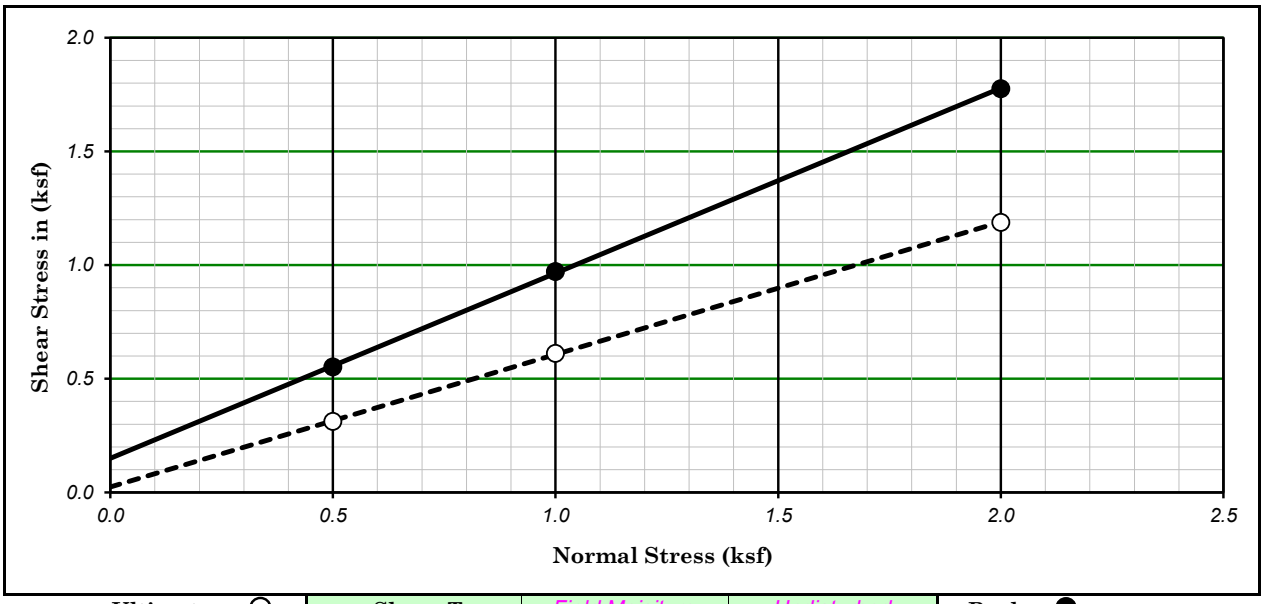
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Geotechnical and Earthquake Engineering

Johnson Favaro, Oceanside Beachfront

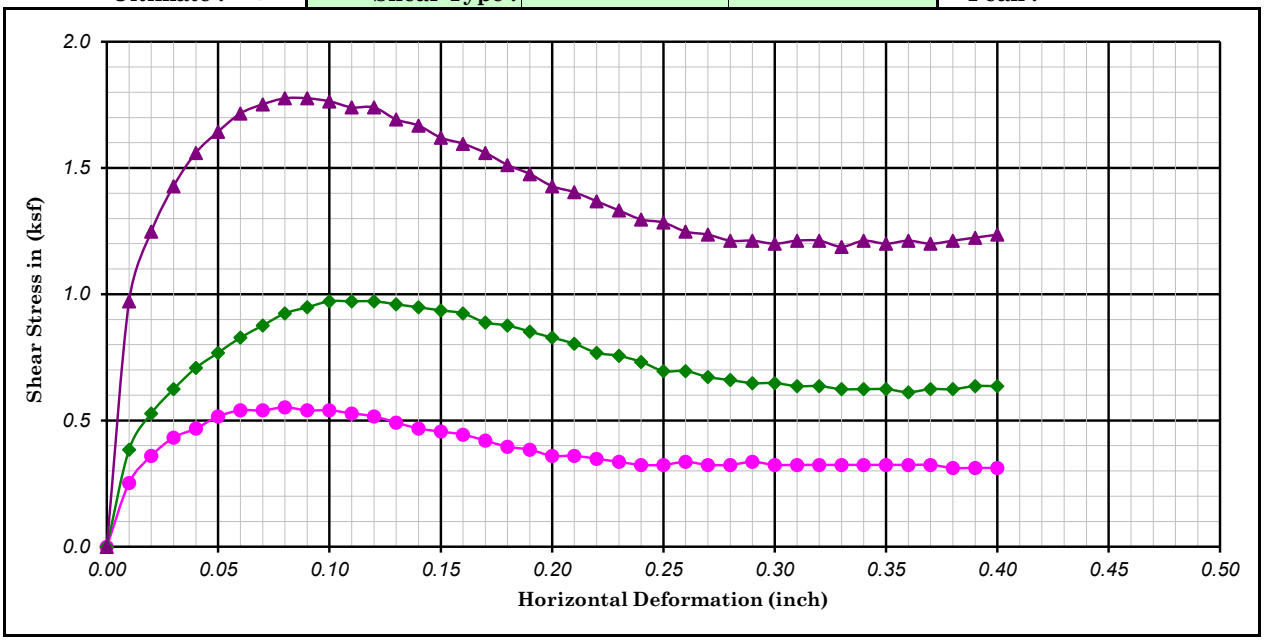
DIRECT SHEAR TEST

(ASTM D-3080)

Project No. : 23-137 | Date : 11/26/04 | Figure No.



Ultimate : ○ | Shear Type : Field Moisture | Undisturbed | Peak : ●



Boring No. : A-24-013		Strength Intercept (C) :	0.15	(ksf)	Peak	0.02	(ksf)	Ultimate
Sample No. : D-2			7.18	(kPa)		1.15	(kPa)	
Depth (ft/m) : 10.0	3.05	Friction Angle (ϕ) :	39.16	Degree		30.24	Degree	
Description : Yellowish brown, Poorly graded SAND (SP)					Shear Rate (inch/minute) :	0.02		

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	1.43	103.45	16.28	0.63	0.50	23.94	0.55	26.43	0.31	14.94
◆	1.17	107.98	17.00	0.56	1.00	47.88	0.97	46.54	0.61	29.30
▲	1.22	105.81	16.65	0.59	2.00	95.76	1.78	85.03	1.19	56.88

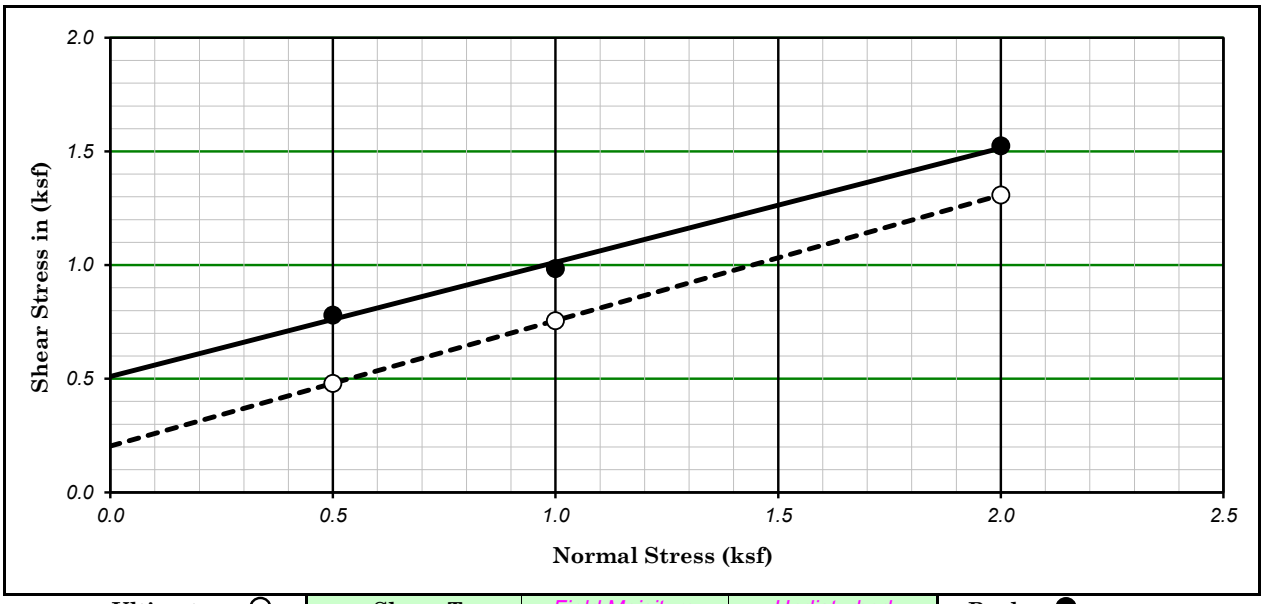
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Johnson Favaro, Oceanside Beachfront

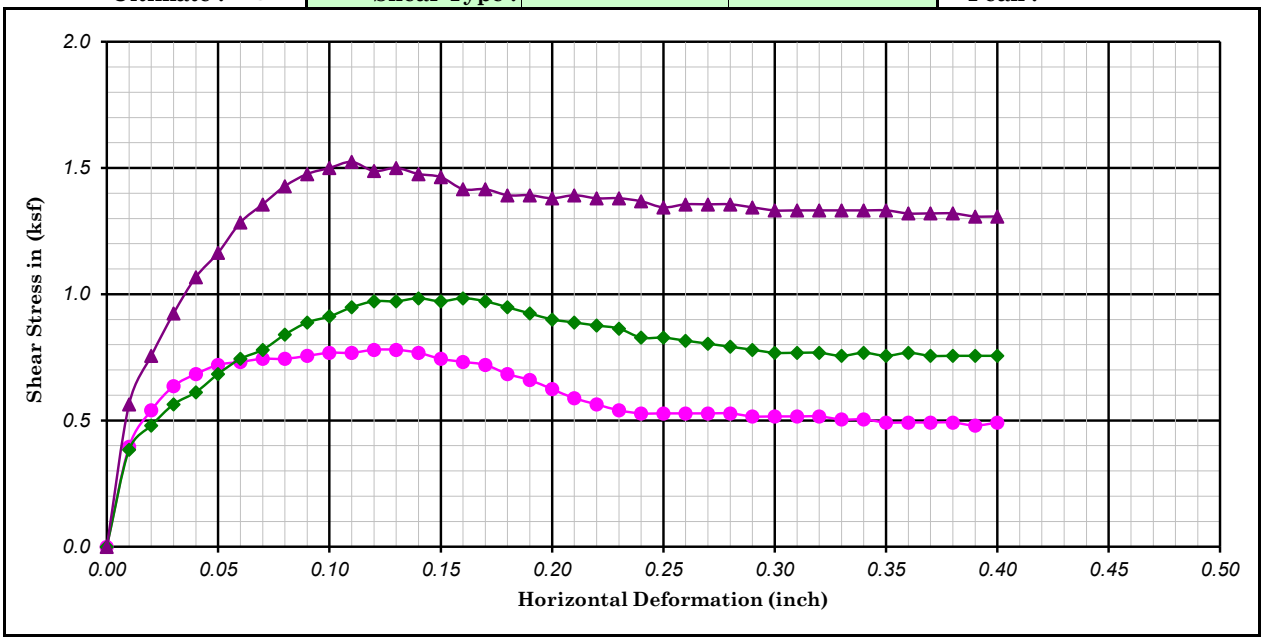
DIRECT SHEAR TEST

(ASTM D-3080)

Project No. : 23-137 | Date : 11/26/04 | Figure No.



Ultimate : ○ | Shear Type : Field Moisture | Undisturbed | Peak : ●



Boring No. : A-24-018		Strength Intercept (C) :	0.51	(ksf)	Peak	0.20	(ksf)	Ultimate
Sample No. : D-2			24.42	(kPa)		9.77	(kPa)	
Depth (ft/m) : 10.0	3.05	Friction Angle (ϕ) :	26.67	Degree		28.90	Degree	
Description : Light brown, Poorly graded SAND with SILT (SP-SM)					Shear Rate (inch/minute) :		0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	8.29	119.13	18.75	0.41	0.50	23.94	0.78	37.35	0.48	22.98
◆	6.54	114.03	17.95	0.48	1.00	47.88	0.98	47.11	0.76	36.20
▲	5.26	112.62	17.73	0.50	2.00	95.76	1.52	72.97	1.31	62.63

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Johnson Favaro, Oceanside Beachfront

DIRECT SHEAR TEST

(ASTM D-3080)

Project No. : 23-137 | Date : 11/26/04 | Figure No.

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

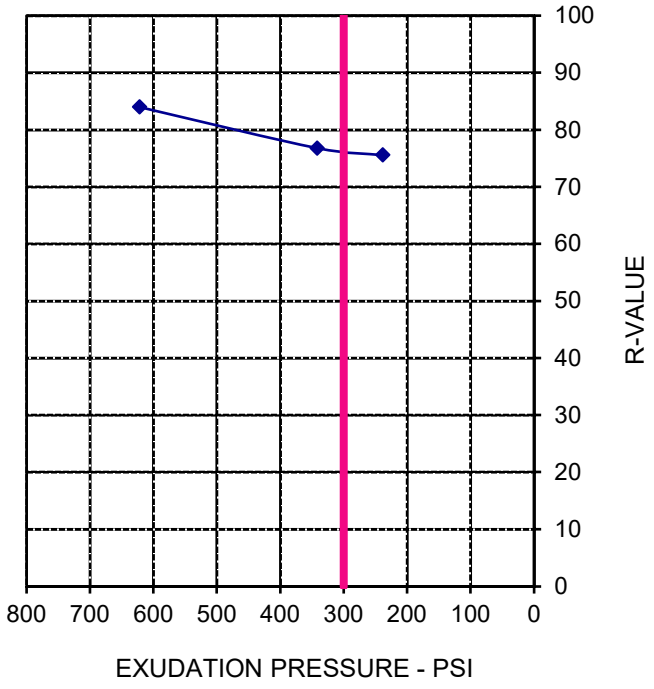
R-VALUE TEST DATA

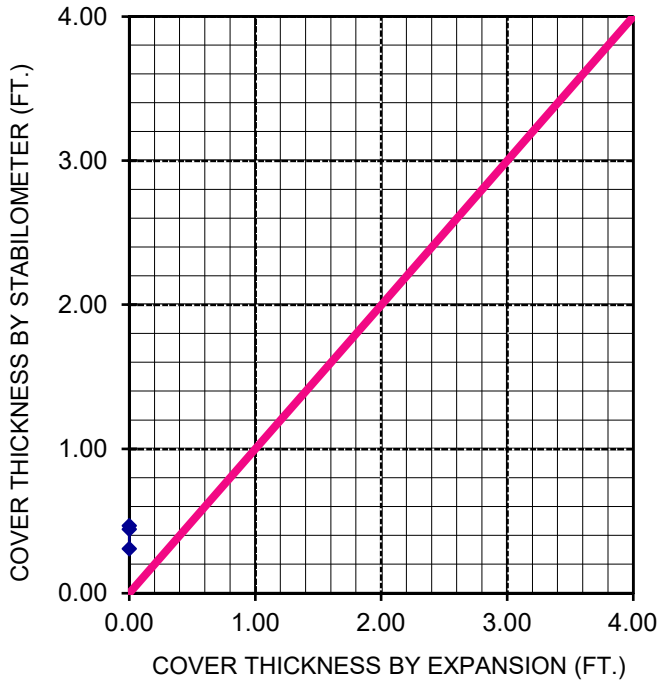
ASTM D2844

Project Name:	Johnson Favaro, Oceanside Beachfront	Tested By:	ST	Date:	12/02/24
Project Number:	23-137	Computed By:	KM	Date:	12/03/24
Boring No.:	A-24-003	Checked By:	AP	Date:	12/09/24
Sample No.:	B-0	Depth (ft.):	-		
Location:	N/A				
Soil Description:	Poorly-graded Sand				

Mold Number	E	D	F	
Water Added, g	71	63	50	
Compact Moisture(%)	9.9	9.2	8.0	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	239	342	622	
Sample Height, Inches	2.6	2.3	2.6	
Gross Weight Mold, g	3049	2906	2952	
Tare Weight Mold, g	1963	1953	1867	
Net Sample Weight, g	1086	953	1084	
Expansion, inchesx10^{-4}	0	0	0	
Stability 2,000 (160 psi)	10/21	8/16	6/12	
Turns Displacement	5.80	5.97	6.14	
R-Value Uncorrected	74	79	83	
R-Value Corrected	76	77	84	
Dry Density, pcf	115.1	115.0	117.0	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.47	0.44	0.31	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE	By Exudation:	76
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	76
Remarks	Gf = 1.34, and 0.2 % Retained on the ¾" *Not Applicable	





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R-VALUE TEST DATA

ASTM D2844

Project Name: Johnson Favaro, Oceanside Beachfront
Project Number: 23-137
Boring No.: A-24-015
Sample No.: B-0 Depth (ft.): -
Location: N/A
Soil Description: Poorly-graded Sand with Silt

Tested By: ST Date: 12/02/24
Computed By: KM Date: 12/03/24
Checked By: AP Date: 12/09/24

Mold Number	A	C	B	
Water Added, g	52	42	30	
Compact Moisture(%)	11.3	10.3	9.2	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	133	369	648	
Sample Height, Inches	2.4	2.5	2.5	
Gross Weight Mold, g	3054	3064	3045	
Tare Weight Mold, g	1965	1967	1965	
Net Sample Weight, g	1089	1097	1080	
Expansion, inchesx10^{-4}	0	0	0	
Stability 2,000 (160 psi)	12/23	10/19	8/14	
Turns Displacement	5.34	5.26	5.16	
R-Value Uncorrected	74	78	83	
R-Value Corrected	72	78	83	
Dry Density, pcf	123.5	120.6	120.0	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.53	0.42	0.32	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE		
	By Exudation:	77
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	77
Remarks	Gf = 1.34, and 0.1 % Retained on the ¾" *Not Applicable	

