GEOTECHNICAL INVESTIGATION REPORT
PROPOSED TELECMMUNICATIONS FACILITY
US-CA-5368 BIG PINE
BIG PINE
1001 COUNTY RD

Big Pine, California

Prepared for:

EOCENE ENVIRONMENTAL GROUP

Prepared by:

GEOBODEN INC.

Irvine, CA 92620

June 26, 2025

Project No. Big Pine-1-01

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED TELECMMUNICATIONS FACILITY

US-CA-5368 BIG PINE

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BIG PINE, CALIFORNIA

EOCENE ENVIRONMENTAL GROUP

Prepared by:

GEOBODEN INC.
5 Hodgenville
Irvine, California 92620

June 26, 2025

J.N. Big Pine-1-01

June 26, 2025 Project No. Big Pine-1-01

Attention: Eocene Environmental Group

Subject: Geotechnical Investigation Report
Proposed Telecommunications Facility
US-CA-5368 BIG PINE
Big Pine
1001 COUNTY RD
Big Pine, California

GeoBoden, Inc. is pleased to provide you two (2) copies of the geotechnical report for the proposed telecommunications facility to be constructed at the subject site.

Please do not hesitate to contact us if you have any questions or if we may be of any additional assistance. We look forward to assisting you during the construction of the proposed facility.

Very truly yours,
GEOBODEN INCORPORATED

Shahrokh (Cyrus) E Radvar, G.E.
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
SHAHROKH E. RADVAR
2742
GEOTECHNICAL
STATE OF CALIFORNIA

Copies: 1/Eocene Environmental Group

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED TELECOMMUNICATIONS FACILITY

US-CA-5368 BIG PINE

BIG PINE

1001 COUNTY RD

BIG PINE, CALIFORNIA

TABLE OF CONTENTS

FIGURES

APPENDIXES

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED TELECOMMUNICATIONS FACILITY
US-CA-5368 BIG PINE
BIG PINE
1001 COUNTY RD
Big Pine, California

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation performed by GeoBoden, Inc. (GeoBoden), for a proposed communications facility to be installed in Big Pine, California. The general location of the project is shown on Figure 1, "Vicinity Map".

Based on our project understanding, the project will construct an unmanned telecommunications facility. The facility will include monopine tower which will be about 125 feet in height. Minimal site grading is anticipated to provide a level pad for the proposed facilities. Underground utilities in trenches are planned.

The purpose of this investigation was to provide geotechnical input for the design of the monopine tower foundation. The scope of our services included the following:

- Conducting a seismic hazards screening;
- Coordinating site access;
- Obtaining utility clearances for drilling;
- Performing drilling and sampling at the site;
- Performing laboratory testing of representative samples;
- Engineering analyses; and
- Preparation of this report.

This report summarizes our findings and presents geotechnical recommendations for the design of this communications tower. The boring logs and results of our laboratory testing are contained in Appendix A and B, respectively.

2.0 SEISMIC HAZARDS

As is the case with most of Southern California, the site is located within a highly active seismic area. Based on our review of available information, the seismic hazards for this site are summarized as follows:

- The site is not mapped within liquefaction hazard zone.
- The site is located within an Alquist-Priolo (AP) Special Study Zone. The location of the proposed tower is offset greater than 50 feet from the closest fault.
- The site is located approximately 0.29- km from the Ownes Valley fault. Based on distance to the nearest fault, fault rupture is not anticipated to adversely impact the the proposed telecommunications tower and the associated improvements.
- The site is not located within a mapped landslide hazard zone.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 FIELD INVESTIGATION

A field investigation was conducted at the site to obtain information on the subsurface conditions. The field investigation consisted of drilling one hollow-stem auger boring to a depth of 41.5 feet at the location shown on Figure 2. The field investigation was performed under the supervision of GeoBoden's personnel, who logged the boring and visually classified and collected samples of the subsurface materials encountered in the boring. The boring was backfilled with cuttings from the drilling operation. Final boring logs were prepared from the field logs and are presented in Appendix A.

Drive samples were taken at 5-foot interval using either a Standard Penetration Test (SPT) sampler or a 2.4-inch I.D. ring sampler driven into the bottom of the borehole using a 140-lb hammer dropped a distance of 30 inches. Relatively undisturbed soil samples were retained in a series of brass rings using the ring sampler. Standard Penetration samples were sealed in the field in plastic bags to preserve the natural moisture content. A Bulk sample of the soils was also obtained for additional classification and laboratory testing.

3.2 LABORATORY TESTING

Soil samples obtained from the field investigation were brought to Geotechnical Laboratory. Selected samples were tested to measure physical and engineering properties. Laboratory tests performed included moisture content, unit weight, direct shear, No. 200 Sieve, and chemical analyses. Chemical analyses included pH, soluble sulfates and soluble chlorides. A detailed description of our laboratory testing with the results of the test results is included in Appendix B.

4.0 DISCUSSION OF FINDINGS

The following discussion of findings for the site is based on the results of the field exploration and laboratory testing programs.

4.1 SUBSURFACE CONDITIONS

The site is underlain by native soils consisting of silty sand and sand with silt. The native soils are primarily loose to medium dense.

4.2 GROUNDWATER CONDITIONS

Groundwater was encountered within our exploratory boring at 10 feet. We have reviewed the California Department of Water Resources and Southern District electronic database of historic water level data for the site vicinity. Historically highest groundwater levels in the site vicinity indicate that groundwater has been as shallow as 3 feet below ground surfaces (bgs).

4.3 SOIL ENGINEERING PROPERTIES

Physical tests were performed on the relatively undisturbed samples to characterize the engineering properties of the native soils. Moisture content and dry unit weight determinations were performed on the samples to evaluate the in-situ unit weights of the different materials. Moisture content and dry unit weight results are shown on the boring logs in Appendix A.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 TOWER FOUNDATION

Based on the results of our investigation, the proposed monopine tower may be supported on new typical, large-diameter reinforced concrete drilled piers; Cast-In-Drill-Hole (CIDH) piles. The base reactions for the piles will be derived from side friction for axial loads, and from passive soil resistance for lateral and over-turning forces. For the proposed drilled piers, we computed the allowable capacity of the drilled pier in compression. The soil profile was taken from our field exploration data and the input parameters for our analyses are taken from the results of our laboratory testing and our professional judgment. The results of our analysis of factored axial load capacities (Allowable Axial Capacity) for various sizes of shafts are given in Appendix C of this report.

For a 5-foot diameter drilled shaft, we recommend the following for axial design assuming end bearing and providing for a minimum factor of safety of 3:

AXIAL LOADING

Depth Range (ft.)	Allowable End Bearing Pressure, qa (psf)
0 – 5*	-
5– 40	4,000

For a 5-foot diameter drilled shaft, we recommend a minimum embedment depth of 40 feet.

For the anticipated axial, lateral, and overturning loads, settlement of the pier will be negligible and lateral deflection at the top of pier under the maximum anticipated lateral and over-turning loads is estimated to be ¼ to ½ inch.

We recommend the following for lateral loading design:

LATERAL LOADING

Depth of Layer (ft.)	Soil Type	N-Value Range (bpf)	Unit Weight, γ (pcf)	Internal Friction, (degrees)	Cohesion, c (psf)	Active Rankine Coefficient (Ka)	Passive Pressure EFP (pcf/ft)**
0 – 5*	Silty Sand	-	120	-	-	0.35	-
5 – 40	Silty Sand/ Sand	35-50	125	30	0	0.30	400

Depth of Layer (ft.)	Allowable Unit End Bearing psf	Ultimate Uplift Skin Friction (psf)	Ultimate Compression Skin Friction (psf)	Static Horizontal Modulus of Subgrade reaction (pci)	Cyclic Horizontal Modulus of Subgrade reaction (pci)	Strain @ 50% of Maximum Stress
0 – 5*	-	-	-	-	-	-
5-40	4,000	200	300	1,000	400	-

* The lateral resistance in the upper 5 feet should be ignored for lateral resistance.
*** Up to a maximum passive pressure of 10 times EFP.*

A passive soil resistance of 400 psf per foot of pier embedment depth up to a maximum of 4,000 psf may be assumed for determining the lateral capacity of the pier. A passive soil resistance should be neglected to a depth equal to one pier diameter. Lateral loads applied at the pier head also induce bending moments at depth in the pier. The diameter and/or length of the pier should be increased as necessary to limit lateral pier deflection to a tolerable settlement.

The pier foundation should be designed and constructed in accordance with applicable procedures established by the 2022 California Building Code (CBC) and the American Concrete Institute (ACI). The specifications should be patterned after recommendations included in the "Standards and Specifications for the Drilled Shaft Industry" published by the Association for Drilled Shaft Contractors (ADSC). We recommend that potential foundation contractors be prequalified with a heavy emphasis on local experience as recommended by ADSC. The excavation for the pier shaft should be performed under the observation of GeoBoden to confirm that the pier shaft is in conformance with our recommendations.

For the anticipated subsurface conditions at the site, conventional drilling equipment may be used for excavating the pier shaft. Based on the available information and our local experience, caving and/or seepage are likely to be expected in sandy soils during drilling. Casing may be required to maintain an open shaft for bottom clean-out work, inspection, and installation of reinforcing steel and concrete. The contractor should be prepared to control such caving. The pier shaft should not be left opened for any prolonged period of time. Groundwater is expected within the anticipated design depth for the pier.

5.2 MAT FOUNDATION

Due to presence of shallow ground water, mat foundation is not recommended. We recommend that the new monopine tower be supported on large diameter shafts as recommended in Section 5.1 of this report.

5.3 CBC DESIGN PARAMETERS

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2022 edition of the California Building Code (CBC). Table 1, *2022 CBC Seismic Parameters*, lists (next) seismic design parameters based on the 2022 CBC methodology:

2022 CBC Seismic Parameters

2022 CBC Seismic Design Parameters	Value
Site Latitude (decimal degrees)	37.172394
Site Longitude (decimal degrees)	-118.306361
Site Class Definition	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.704
Mapped Spectral Response Acceleration at 1s Period, S_1	0.614
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.044
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.044
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.363
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.696

5.5 LIQUEFACTION POTENTIAL

For liquefaction to occur, all of three key ingredients are required: liquefaction-susceptible soils, groundwater within a depth of 50 feet or less, and strong earthquake shaking. Soils susceptible to liquefaction are generally saturated loose to medium dense sands and non-plastic silt deposits below the water table.

Groundwater is present at the site. Onsite soils are loose to medium dense. The proposed tower will be supported on deepened shaft foundation. It is our opinion the potential for liquefaction at the site is moderate.

5.6 SHALLOW FOUNDATIONS

Following the site and foundation preparation recommended below, foundation for shallow foundations may be designed as discussed below.

5.6.1 Bearing Capacity and settlement

Shallow foundations may be supported on continuous spread footings and isolated spread footings, and should bear entirely upon competent native soils or properly engineered fill. Continuous and isolated footings should have a minimum width of 14 inches and 24 inches, respectively. All footings should be embedded a minimum depth of 18 inches measured from the lowest adjacent finish grade. Continuous and isolated footings placed on such materials may be designed using a maximum allowable (net) bearing capacity of 2,000 pounds per square foot (psf). The maximum bearing value applies to combined dead and sustained live loads. The allowable bearing pressure may be increased by one-third when considering transient live loads, including seismic and wind forces.

Based on the allowable bearing value recommended above, total settlement of the shallow footings are anticipated to be less than one inch, provided foundation preparations conform to the recommendations described in "Site Preparation and Earthwork" Section of this report. Differential settlement is anticipated to be approximately half the total settlement for similarly loaded footings spaced up to approximately 30 feet apart.

5.6.2 Lateral Load Resistance

Lateral load resistance for the spread footings will be developed by passive soil pressure against sides of footings below grade and by friction acting at the base of the concrete footings bearing on compacted fill. An allowable passive pressure of 250 psf per foot of depth may be used for design purposes. An allowable coefficient of friction 0.35 may be used for dead and sustained live load forces to compute the frictional resistance of the footings constructed directly on compacted fill. Safety factors of 2.0 and 1.5 have been incorporated in development of allowable passive and frictional resistance values, respectively. Under seismic and wind loading conditions, the passive pressure and frictional resistance may be increased by one-third.

5.6.3 Footing Reinforcement

Reinforcement for footings should be designed by the structural engineer based on the anticipated loading conditions. Footings for lightly loaded masonry structures that are supported in low to very low expansive soils should have No. 4 bars (two top and two bottom).

5.7 CONCRETE SLAB ON-GRADE

Concrete slabs will be placed on properly compacted fill as outlined in this report. Moisture content of subgrade soils should be maintained near the optimum moisture content. At the time of the concrete pour, subgrade soils should be firm and relatively unyielding. Any disturbed soils should be excavated and then replaced and compacted to a minimum of 90 percent relative compaction. Slabs should be designed to accommodate very low expansive fill soils. The structural engineer should determine the minimum slab thickness and reinforcing depending upon the expansive soil condition intended use. Unless a more stringent design is recommended by the structural engineer, we recommend a minimum thickness of 4 inches, and reinforcement consisting of No. 3 bars spaced a maximum of 24 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid depth.

5.8 SITE PREPARATION AND EARTHWORK

All site preparation and grading should be observed by experienced personnel reporting to the project Geotechnical Engineer. Our field monitoring services are an essential continuation of our prior studies to confirm and correlate the findings and our prior recommendations with the actual subsurface conditions exposed during construction, and to confirm that suitable fill soils are placed and properly compacted.

The site should be cleared of any debris, organic matter, abandoned utility, and other unsuitable materials. Any existing fill encountered should be excavated and replaced with properly compacted fill or lean concrete to the depth of the fill and to a horizontal distance equal to the depth of excavation (if possible) in order to provide improved foundation support for the proposed facility. Any excavation side slopes should be cut at a gradient no steeper than 1:1(horizontal to vertical), and excavations should not extend below an imaginary 1.5:1 inclined plane projecting below the bottom edge of adjacent existing foundations. All excavations should be observed by GeoBoden to confirm that all unsuitable material is substantially removed from beneath the planned construction prior to placing fill.

Excavations below the final grade level should be properly backfilled using lean concrete or approved fill material compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D1557. The backfill and any additional fill should be placed in loose lifts less than 8 inches thick, moisture conditioned to 2 percent above optimum moisture content, and compacted to 90 percent. Fill materials should be free of construction debris, roots, organic matter, rubble, contaminated soils, and any other unsuitable or deleterious material as determined by the Geotechnical Engineer. The on-site soils are suitable for use as compacted fill, provided the soil is free of any deleterious substance. All import fill material should be approved by the Geotechnical Engineer prior to importing to the site for use as compacted fill.

Unless otherwise noted, all earthwork should be performed in accordance with the latest edition of "Standard Specifications for Public Works Construction."

5.9 UTILITY TRENCHES

It is anticipated that the on-site soils will provide suitable support for underground utilities and piping that may be installed. Any soft and/or unstable material encountered at the bottom of excavations for such facilities should be removed and be replaced with an adequate bedding material.

The on-site soils generally are not considered suitable for bedding or shading of utilities and piping. We recommend that a non-expansive granular material with a sand equivalent greater than 30 be imported for this purpose.

The on-site soils are 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. Trench backfill should be mechanically placed and compacted in thin lifts to at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557. Flooding or jetting for placement and compaction of backfill is not recommended.

5.10 SOLUBLE SULFATES AND SOIL CORROSIVITY

The soluble sulfate, pH, and chloride concentration tests were performed on near-surface collected samples. Corrosion test results are presented in Appendix B. The minimum resistivity tests on near collected bulk sample indicate that the onsite surficial soils are mildly corrosive when in contact with ferrous materials. Typical recommendations for mitigation of the corrosive potential of the soil in contact with building materials are the following:

- Below grade ferrous metals should be given a high quality protective coating, such as an 18 mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar.

- Below grade ferrous metals should be electrically insulated (isolated) from above grade ferrous metals and other dissimilar metals, by means of dielectric fittings in utilities and exposed metal structures breaking grade.

- Steel and wire reinforcement within concrete in contact with the site soils should have at least two inches of concrete cover.

If ferrous building materials are expected to be placed in contact with site soils, it may be desirable to consult a corrosion specialist regarding chosen construction materials, and/or protection design for the proposed structures.

The surficial soils at the site have negligible sulfate attack potential on concrete. As a result, a mix design such as Type II cement should provide resistance against possible sulfate attack.

5.11 CONSTRUCTION OBSERVATION AND FIELD TESTING

Construction observation and field testing services are an essential continuation of our prior studies to confirm and correlate our findings and recommendations with the actual subsurface conditions exposed during construction. We recommend that GeoBoden be present to observe and provide testing during the following construction activities.

- ➢ Site excavations

- Preparation of subgrades for foundations and slab
- Placement of all fill and backfill
- Observations of drilled pier and footing excavations when applicable
- Backfilling of utility trenches when applicable

6.0 GENERAL CONDITIONS

This report presents recommendations pertaining to the proposed development of the subject site as presented to GeoBoden. These recommendations are based on the assumption that the subsurface conditions do not deviate appreciably from those discovered during our geotechnical investigation and the design provided to us is representative of the as-built system. The possibility of different conditions cannot be discounted. It is the responsibility of the Owner to bring any deviations or unexpected conditions observed when our staff or technicians are not on-site during construction to the attention of the Geotechnical Engineer. In event of significant changes in design loads or structural characteristics are made, GeoBoden should be retained to review our original design recommendations and their applicability to the revised design plans. In this way, any required supplemental recommendations can be made in a timely manner.

Although GeoBoden has endeavored to characterize the surface and subsurface conditions at the site, GeoBoden is not responsible for potential problems associated with constructing pier foundations including hole stability and dewatering if any. Constructing the pier foundations under the given site and subsurface conditions is the responsibility of the contractor.

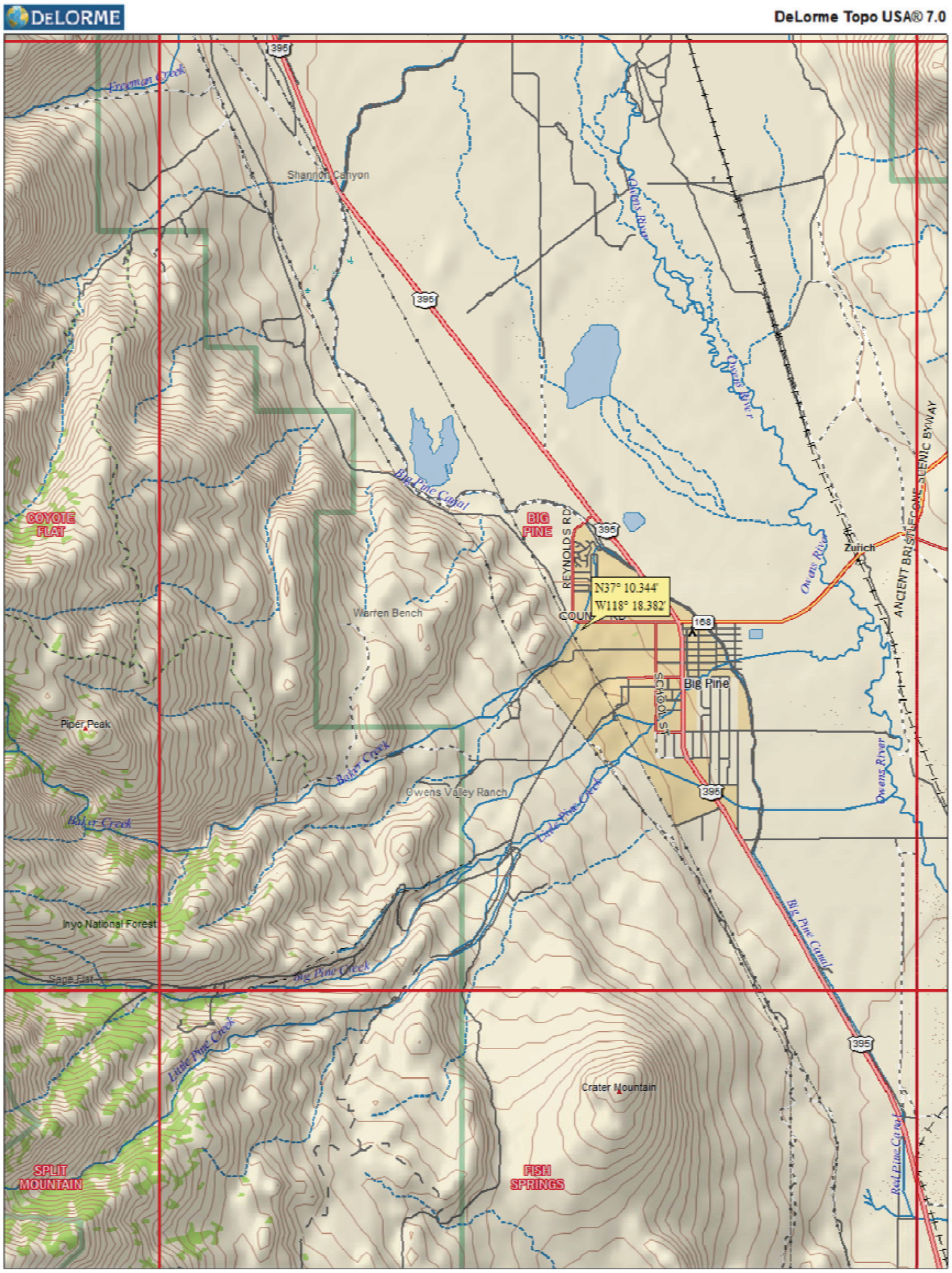
Professional judgments presented in this report are based on evaluations of the information available, on GeoBoden's understanding of foundation design, and GeoBoden's general experience in the field of geotechnical engineering. GeoBoden does not guarantee the interpretations made, only that the engineering work and judgment rendered meet the standard of care of the geotechnical profession at this time.

7.0 REFERENCES

California Building Code (CBC), 2022.

NAVFAC 7.02 "Foundations & Earth Structures", Naval Facilities Engineering Command, Revalidated by Change 1 September 1986.

FIGURES



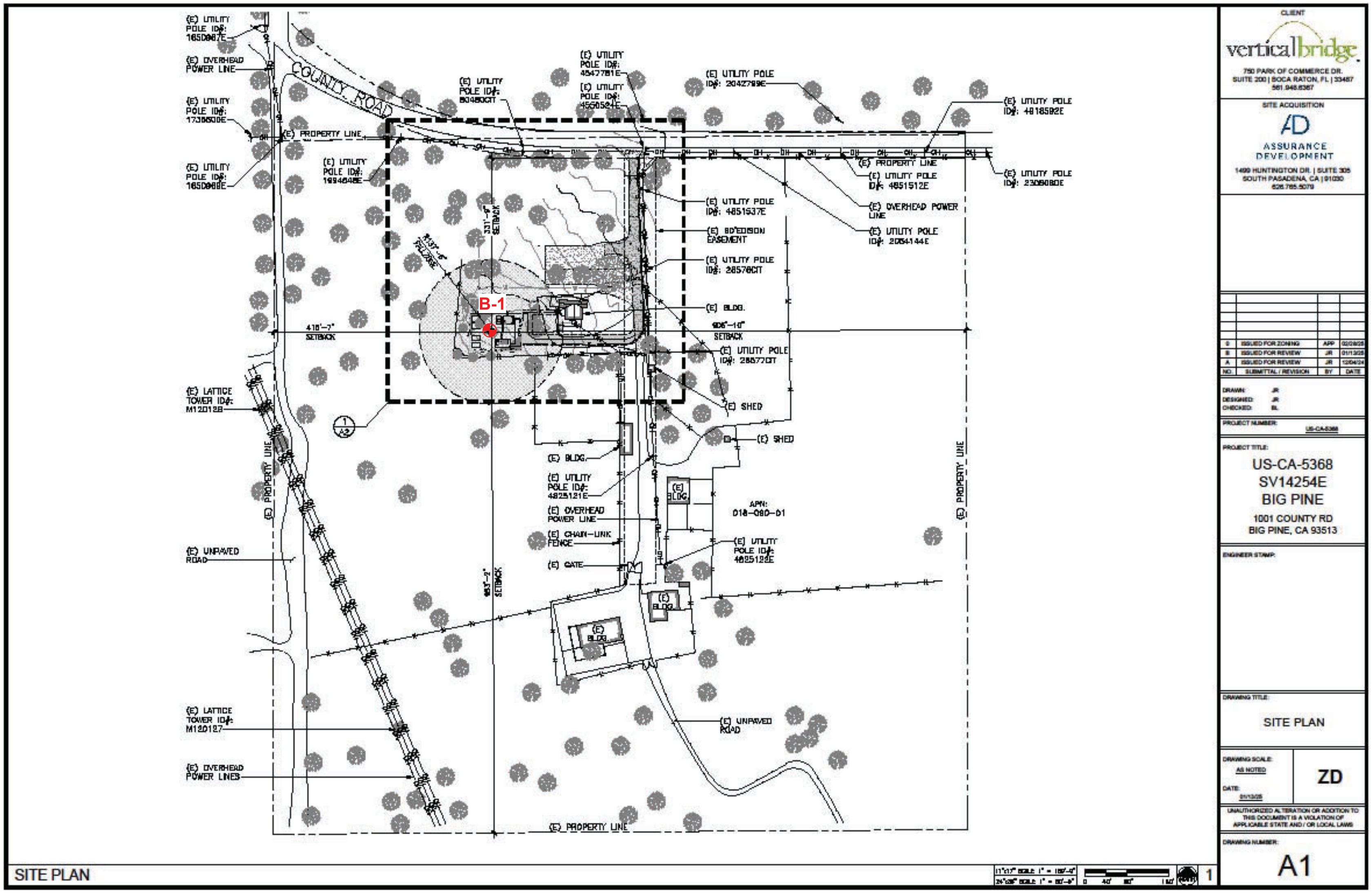
Data use subject to license.

MN (11.8° E)

0 ½ 1 1½ 2 mi

Data Zoom 11-4

GEOBODEN INC. Geotechnical Consultants	SITE LOCATION MAP Proposed Telecommunications Facility US-CA-5368 BIG PINE Big Pine 1001 COUNTY RD Big Pine, California	Figure By S.R.	Project No. Pine-1-01
		Map No. XX	Figure No. 1
		Date 06-26-25	



LEGEND

B-1 — NUMBER AND APPROXIMATE LOCATION OF BORING

GEOBODEN INC.
Geotechnical Consultants

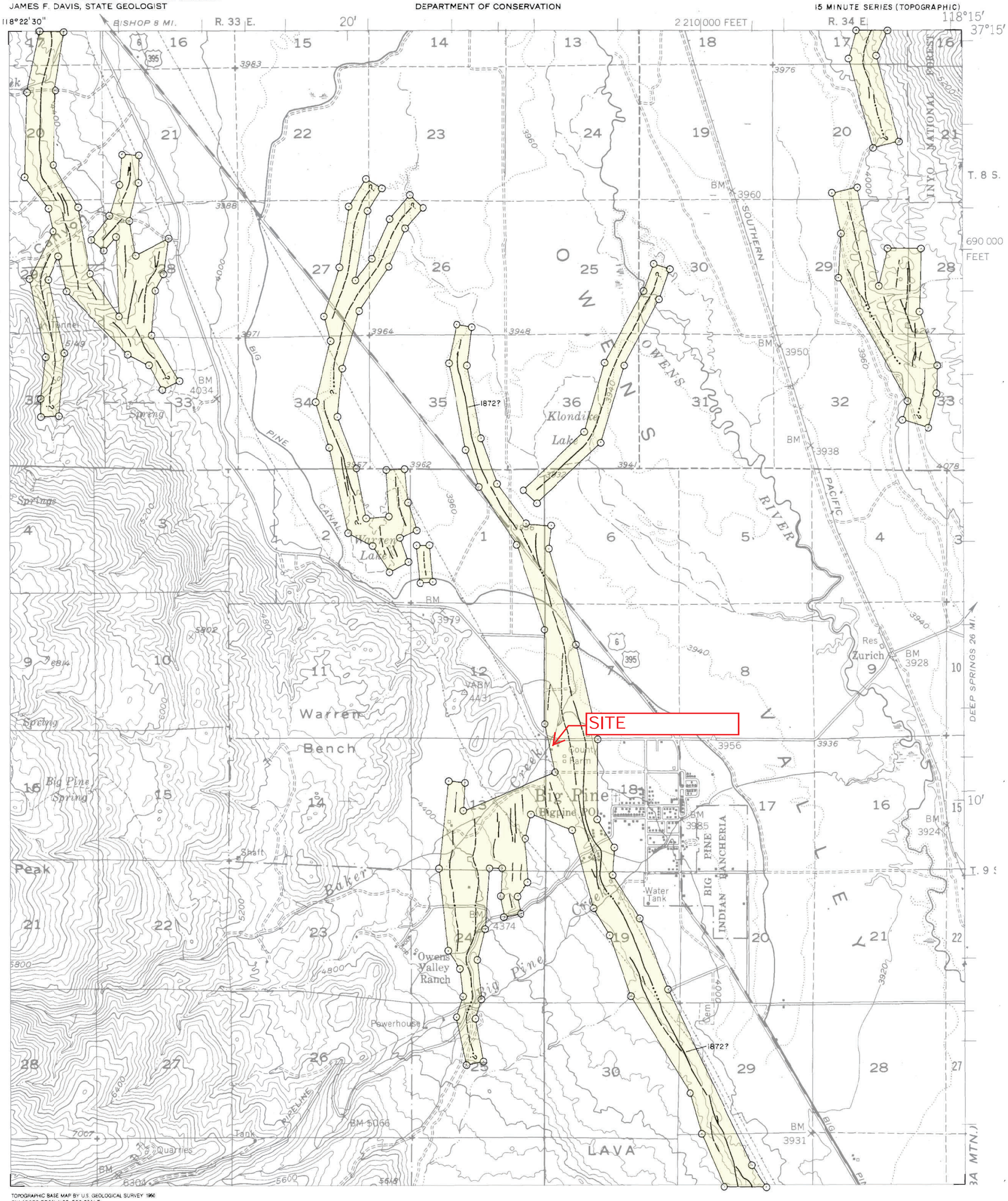
BORING LOCATION PLAN
Proposed Telecommunications Facility
US-CA-5368 BIG PINE
Big Pine
1001 COUNTY RD
Big Pine, California

Figure By S.R.	Project No. Big Pine-1-01
Map No. XX	Figure No.
Date 06-26-25	2

CALIFORNIA DIVISION OF MINES AND GEOLOGY
JAMES F. DAVIS, STATE GEOLOGIST

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF CONSERVATION

NE 1/4 BIG PINE QUADRANGLE
CALIFORNIA
15 MINUTE SERIES (TOPOGRAPHIC)



TOPOGRAPHIC BASE MAP BY U.S. GEOLOGICAL SURVEY 1950
ENLARGED FROM 1:62,500 SCALE

SCALE 1:24,000

CONTOUR INTERVAL 80 FEET
DATUM IS MEAN SEA LEVEL

MAP EXPLANATION

Potentially Active Faults

Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

STATE OF CALIFORNIA
SPECIAL STUDIES ZONES

Delineated in compliance with
Chapter 7.5, Division 2 of the California Public Resources Code
(Alquist-Priolo Special Studies Zones Act)

NE 1/4 BIG PINE QUADRANGLE

REVISED OFFICIAL MAP

Effective: January 1, 1985

James F. Davis State Geologist

REFERENCES USED TO COMPILE FAULT DATA

NE 1/4 Big Pine Quadrangle

Bateman, P.C., 1965, Geology and tungsten mineralization of the Bishop district, California: U.S. Geological Survey Professional Paper 470, 208 p., 11 plates.

Bryant, W.A., 1984, Owens Valley and White Mountains frontal fault zones, Big Pine area, Inyo County: California Division of Mines and Geology Fault Evaluation Report FER-159 (unpublished).

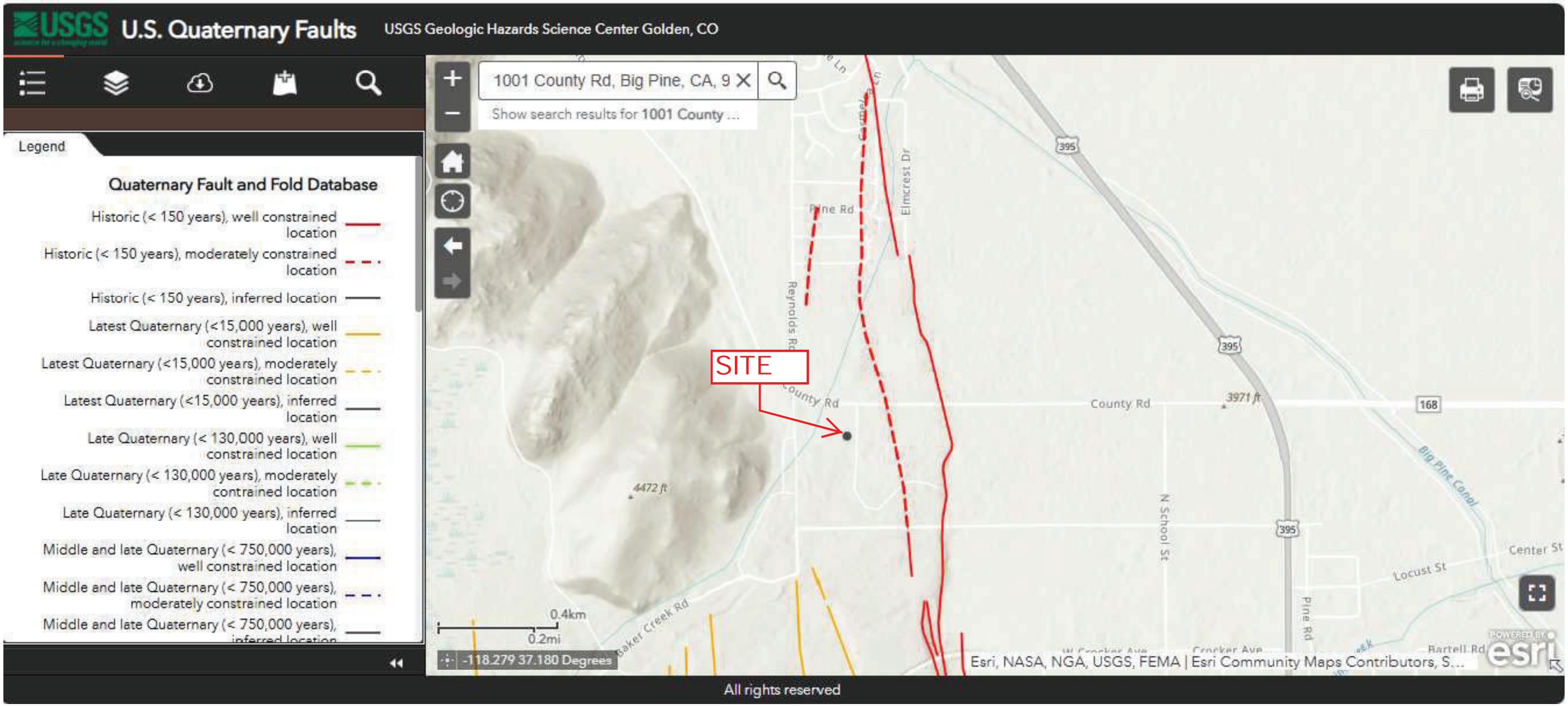
Bryant, W.A., 1984, Evidence of recent faulting along the Owens Valley, Round Valley, and White Mountains fault zones, Inyo and Mono Counties, California: California Division of Mines and Geology Open-File Report 84-54 SAC, 2 Plates.

Envicom, 1976, Seismic safety element for the general plan, Inyo-Mono Association of Governmental Entities: Unpublished consulting report for Inyo and Mono Counties, 129 p., 2 appendices, 9 plates.

For additional information on faults in this map area, the rationale used for zoning, and additional references consulted, refer to unpublished Fault Evaluation Reports on file at the DMG office in Pleasant Hill.

IMPORTANT – PLEASE NOTE

1) *This map may not show all faults that have the potential for surface fault rupture, either within the special studies zones or outside their boundaries.*
2) Faults shown are the basis for establishing the boundaries of the special studies zones.
3) The identification and location of these faults are based on the best available data. However, the quality of data used is varied. Traces have been drawn as accurately as possible at this map scale.
4) Fault information on this map is not sufficent to serve as a substitute for the geologic site investigations (special studies) required under Chapter 7.5 of Division 2 of the California Public Resources Code.


USGS
U.S. Quaternary Faults
USGS Geologic Hazards Science Center Golden, CO
Legend
Quaternary Fault and Fold Database
Historic (< 150 years), well constrained location
Historic (< 150 years), moderately constrained location
Historic (< 150 years), inferred location
Latest Quaternary (<15,000 years), well constrained location
Latest Quaternary (<15,000 years), moderately constrained location
Latest Quaternary (<15,000 years), inferred location
Late Quaternary (< 130,000 years), well constrained location
Late Quaternary (< 130,000 years), moderately contrained location
Late Quaternary (< 130,000 years), inferred location
Middle and late Quaternary (< 750,000 years), well constrained location
Middle and late Quaternary (< 750,000 years), moderately constrained location
Middle and late Quaternary (< 750,000 years),
1001 County Rd, Big Pine, CA, 9
Show search results for 1001 County ...
SITE
Reynolds Rd
Pine Rd
Elmcrest Dr
County Rd
N School St
4472 ft
3971 ft
395
168
Baker Creek Rd
Big Pine Canal
Locust St
Center St
Pine Rd
Crocker Ave
Bartell Rd
0.4km
0.2mi
-118.279 37.180 Degrees
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esri
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APPENDIX A
BORING LOGS

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

PROPOSED TELECOMMUNICATIONS FACILITY
US-CA-5368 BIG PINE
BIG PINE
1001 COUNTY RD
BIG PINE, CALIFORNIA

Prior to drilling, the proposed boring was located in the field by measuring from existing site features.

A total of one exploratory boring was drilled. GeoBoden of Irvine, California, performed the drilling The approximate boring location is shown on Figure 2.

Depth-discrete soil samples were collected at selected intervals from the exploratory boring using a 2 ½ -inch inside diameter (I.D.) modified California Split-barrel sampler fitted with 12 brass ring of 2 ½ inches in O.D. and 1-inch in height and one brass liner (2 ½ -inch O.D. by 6 inches long) above the brass rings. The sampler was lowered to the bottom of the boreholes and driven 18 inches into the soil with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the lower 12 inches is shown on the blow count column of the boring logs.

After removing the sampler from the boreholes, the sampler was opened and the brass rings and liner containing the soil were removed and observed for soil classification. Brass rings containing the soil were sealed in plastic canisters to preserve the natural moisture content of the soil. A Bulk sample of near surface soil was collected from exploratory boring and placed in plastic bags. Soil samples and bulk sample collected from exploratory boring were labeled, and submitted to the laboratory for physical testing.

Standard Penetration Tests (SPTs) were also performed. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140-pound hammer falling 30 inches. The number of blows required to drive the SPT sampler the lower 12 inches of the sampling interval is recorded on the blow count column of the boring logs.

An engineer recorded the soil classifications and descriptions on field logs using the Unified Soil Classification System as described by the American Society for Testing and Materials (ASTM) D 2488-90, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." The final boring logs were prepared from the field logs and are presented in this Appendix.

At the completion of the sampling and logging, the exploratory boring was backfilled with the drilled cuttings.

GEOBODEN, INC.

BORING NUMBER B-1

CLIENT Eocene Environmental Group

PROJECT NAME Proposed Telecommunications Facility

PROJECT NUMBER Big Pine-1-01

PROJECT LOCATION Big Pine, CA

DATE STARTED 6/21/25 **COMPLETED** 6/21/25

GROUND ELEVATION **HOLE SIZE** 6 inches

DRILLING CONTRACTOR GeoBoden, Inc.

GROUND WATER LEVELS:

DRILLING METHOD HSA

∇ **AT TIME OF DRILLING** 10.00 ft

LOGGED BY S.R. **CHECKED BY**

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SILTY SAND (SM): light yellowish brown, moist										
5			MC R-1		20		107	8				24
8		POORLY-GRADED SAND w. SILT (SP-SM): brown, moist										
10	∇		MC R-2		17		109	12				11
15			MC R-3		23		107	8				
20			SS R-4		22							
23		SILTY SAND (SM): pale yellow, moist										
25			SS S-5		26							
30			SS S-6		28							
35												

(Continued Next Page)

CLIENT Eocene Environmental Group
PROJECT NAME Proposed Telecommunications Facility
PROJECT NUMBER Big Pine-1-01
PROJECT LOCATION Big Pine, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
35		SILTY SAND (SM): pale yellow, moist *(continued)*	SS S-8		31							
		POORLY-GRADED SAND (SP): light brown, moist										
40			SS S-9		33							

Bottom of borehole at 41.5 feet.

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

PROPOSED TELECOMMUNICATIONS FACILITY
US-CA-5368 BIG PINE
BIG PINE
1001 COUNTY RD
BIG PINE, CALIFORNIA

Laboratory tests were performed on selected samples to assess the engineering properties and physical characteristics of soils at the site. The following tests were performed:

- Moisture content and dry density
- Direct shear
- No. 200 Wash Sieve
- Corrosion potential

Test results are summarized on laboratory data sheets or presented in tabular form in this appendix.

Moisture Density Tests

The field moisture contents, as a percentage of the dry weight of the soils, were determined by weighing samples before and after oven drying. The dry density, in pounds per cubic foot, was also determined fir all relatively undisturbed ring samples collected. These analyses were performed in accordance with ASTM D 2937. The results of these determinations are shown on the boring logs in Appendix A.

Direct Shear

Direct shear tests were performed on undisturbed samples of on-site soils. A different normal stress was applied vertically to each soil sample ring which was then sheared in a horizontal direction. The resulting shear strength for the corresponding normal stress was measured at a maximum constant rate of strain of 0.005 inches per minute. The direct shear results are shown graphically on a laboratory data sheet included in this appendix.

No. 200 Wash Sieve

A quantitative determination of the percentage of soil finer than 0.075 mm was performed on selected soil samples by washing the soil through the No. 200 sieve. Test procedures were performed in accordance with ASTM Method D1140. The results of the tests are shown on the boring logs in Appendix A.

Corrosion Potential

The selected soil sample in the near surface was tested to determine the corrosivity of the site soil to steel and concrete. The soil samples were tested for soluble sulfate (Caltrans 417), soluble chloride (Caltrans 422), and pH and minimum resistivity (Caltrans 643). The results of the corrosion tests are summarized in Table B-1.

TABLE B-1 (Corrosion Test Results)

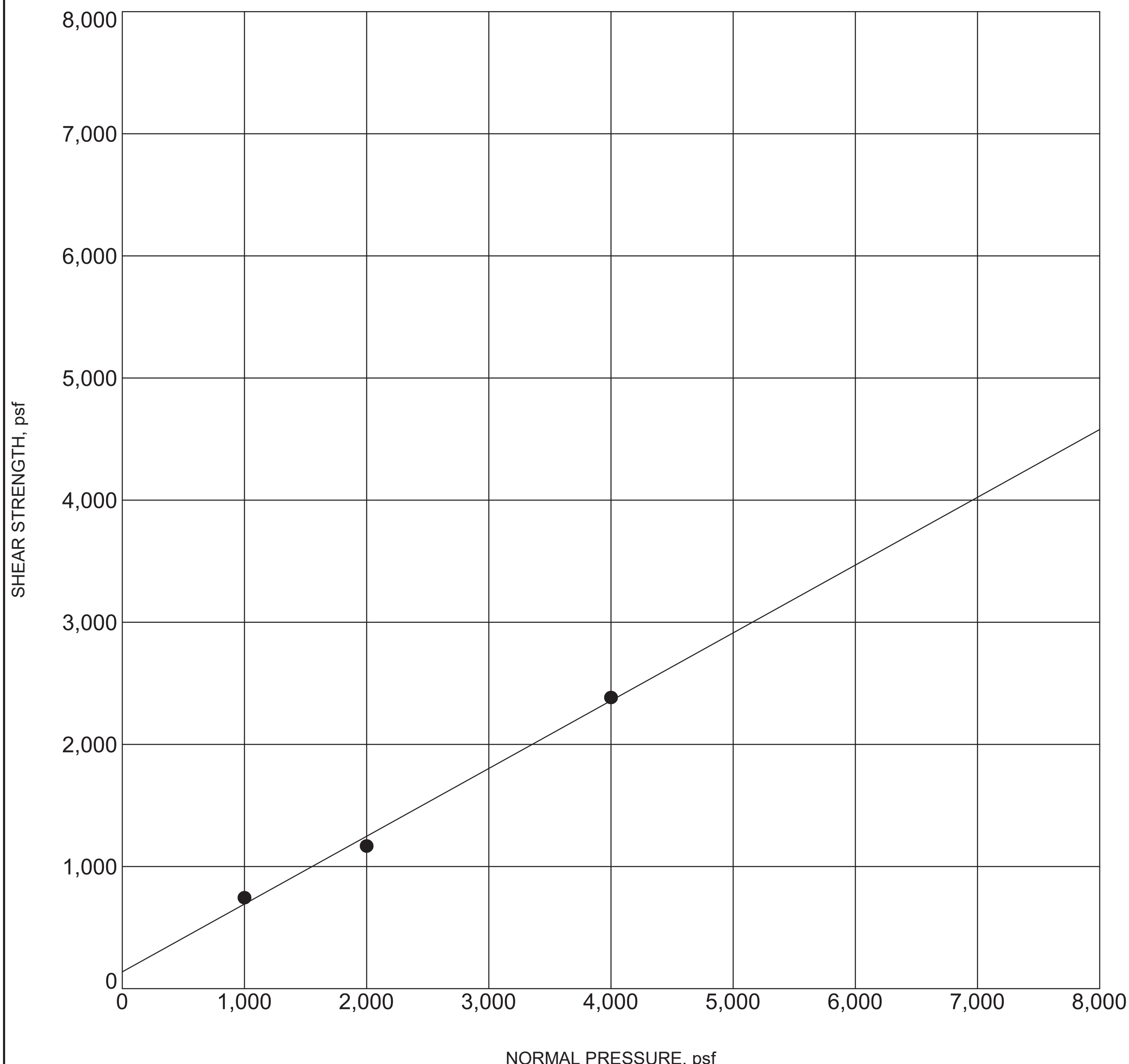
Boring No.	Depth (ft)	Chloride Content (Calif. 422)	Sulfate Content (Calif. 417) % by Weight	pH (Calif. 643)	Resistivity (Calif. 643) Ohm*cm
B-1	0-5	43	0.0117	7.1	2,042

GEOBODEN, INC.

DIRECT SHEAR TEST

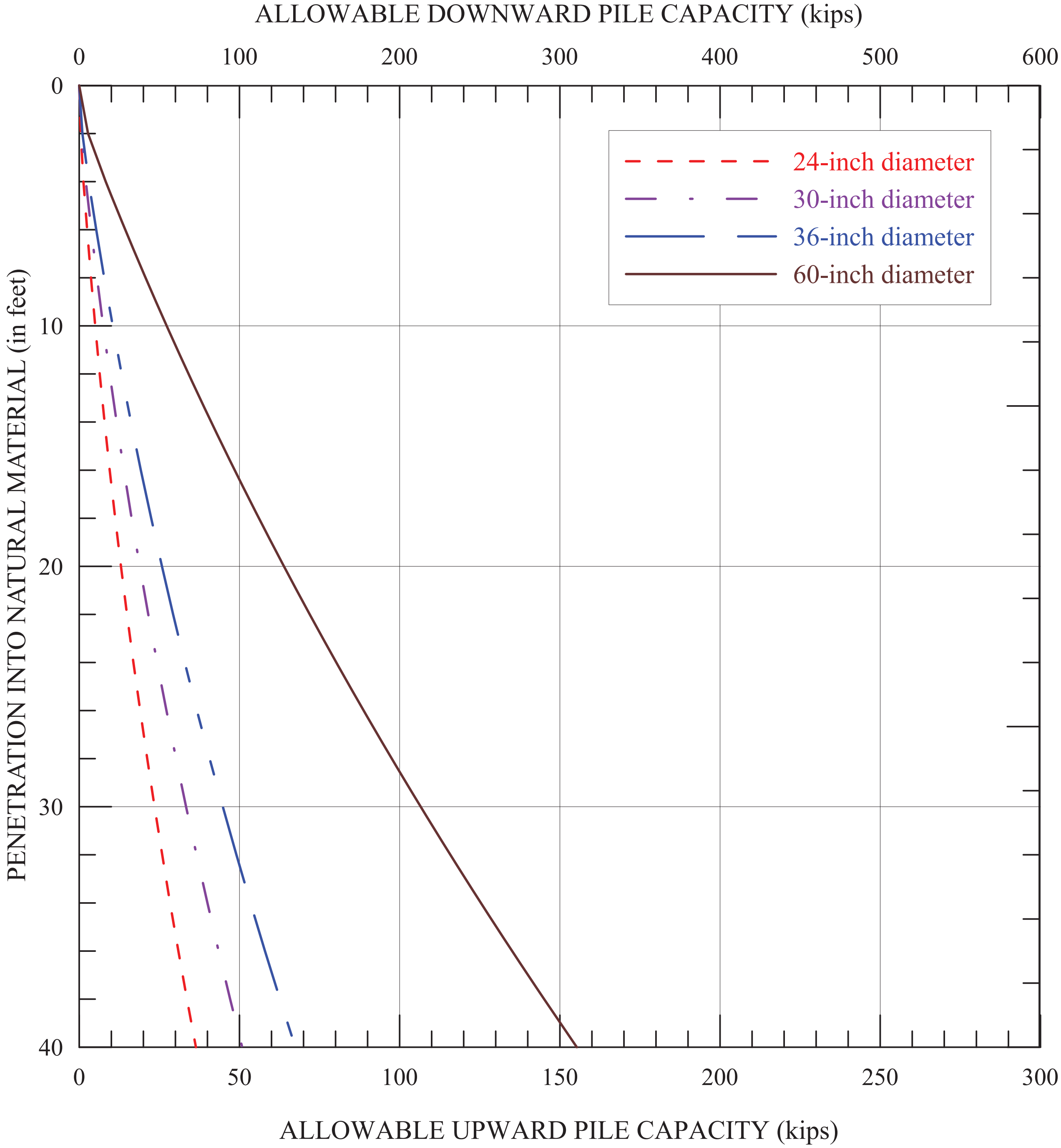
CLIENT Eocene Environmental Group **PROJECT NAME** Proposed Telecommunications Facility

PROJECT NUMBER Big Pine-1-01 **PROJECT LOCATION** Big Pine, CA



	Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-1**	**15.0**	**POORLY-GRADED SAND w. SILT (SP-SM)**	**107**	**8**	**137.0**	**29**

APPENDIX C
AXILE PILE CAPACITY



NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 3 pile diameters on centers.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lesser values by the strength of the piles.

DRILLED PILE CAPACITIES

Proposed Telecommunications Facility US-CA-5368 BIG PINE Big Pine 1001 COUNTY RD Big Pine, California	GEOBODEN, INC.	DRILLED PILE CAPACITY Project No. Pine-1-01 Figure C-1

PILE CAPACITY CALCULATIONS

24-INCH DIAMETER/SIZE PILE

JOB NO.: Big Pine-1-01 BY: SR DESCRIPTION: Drilled Pile Capacity
CLIENT: Eocene Environmental Group DATE: 6/26/2025

			Provide if section is not circular:			Values used in calculations:		
PILE DIAMETER/SIZE:	24	in	SIDE AREA:		ft^2/ft	SIDE AREA:	6.3	ft^2/ft
OVERBURDEN PRESSURE @ PILE TOP:	0	psf	TIP AREA:		ft^2	TIP AREA:	3.1	ft^2

FACTORS OF SAFETY

FRICTION:	2	Depth Increment (ft):	2.00	Downdrag Depth (ft):	(if any)
BEARING:	3	δ/ϕ:	0.75	Downdrag Force (kip):	

NOTE: *Based on NAVFAC 7.02 "Foundations & Earth Structures". For depths > 20 pile diameters, the same overburden pressure is used in calcs.*
For C_A/C, δ/ϕ, K_{HC}, and N_q values see tables and Graph on Left

Layer parameters and depths are from bottom of pile cap.

Layer No.	Layer Depth (ft)	Bottom Layer Depth (ft)	c (psf)	φ (deg)	δ (deg)	δ (deg) used in calcs	γ' (psf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$
1	0	5	0	0	0	0	120	0	0.35	7	0
2	5	41.5	0	31	23.25	23.25	120	0	0.3	7	0
3											
4											
5											
6											
7											
8											

Penetration below pile cap (ft)	c (psf)	φ (deg)	γ' (pcf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$	δ (deg)	σ'₀ (psf) at mid-layer (psf)	Friction at mid-layer (psf)	Allowable Friction (psf)	ALLOWABLE DOWNWARD CAPACITY Friction (kips)	End Bear (kips)	Total (kips)	w/drag (kips)	ALL. UPWARD Total (kips)	w/drag (kips)
0	0	0	120	0	0.35	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.35	7	0	0	120	0	0	0	1	1	1	0	0
4	0	0	120	0	0.35	7	0	0	360	0	0	0	3	3	3	0	0
6	0	31	120	0	0.3	7	0	23.25	600	77	39	0	4	5	5	0	0
8	0	31	120	0	0.3	7	0	23.25	840	108	54	1	6	7	7	1	1
10	0	31	120	0	0.3	7	0	23.25	1080	139	70	2	8	10	10	1	1
12	0	31	120	0	0.3	7	0	23.25	1320	170	85	3	10	13	13	2	2
14	0	31	120	0	0.3	7	0	23.25	1560	201	101	4	11	16	16	2	2
16	0	31	120	0	0.3	7	0	23.25	1800	232	116	6	13	19	19	3	3
18	0	31	120	0	0.3	7	0	23.25	2040	263	131	7	15	22	22	4	4
20	0	31	120	0	0.3	7	0	23.25	2280	294	147	9	17	26	26	5	5
22	0	31	120	0	0.3	7	0	23.25	2520	325	162	11	18	30	30	6	6
24	0	31	120	0	0.3	7	0	23.25	2760	356	178	14	20	34	34	7	7
26	0	31	120	0	0.3	7	0	23.25	3000	387	193	16	22	38	38	8	8
28	0	31	120	0	0.3	7	0	23.25	3240	418	209	19	24	42	42	9	9
30	0	31	120	0	0.3	7	0	23.25	3480	449	224	21	26	47	47	11	11
32	0	31	120	0	0.3	7	0	23.25	3720	479	240	24	27	52	52	12	12
34	0	31	120	0	0.3	7	0	23.25	3960	510	255	28	29	57	57	14	14
36	0	31	120	0	0.3	7	0	23.25	4200	541	271	31	31	62	62	16	16
38	0	31	120	0	0.3	7	0	23.25	4440	572	286	35	33	67	67	17	17
40	0	31	120	0	0.3	7	0	23.25	4680	603	302	38	34	73	73	19	19

Recommended Values of C_A/C (NAVFAC 7.2-196 Fig. 2)

Pile Type	Consistency of Soil	Cohesion, C (psf)	C_A/C
Timber & Concrete	Very Soft	0-250	0-1
	Soft	250-500	1-0.96
	Med. Stiff	500-1000	0.96-0.75
	Stiff	1000-2000	0.75-0.475
	Very Stiff	2000-4000	0.475-0.325
Steel	Very Soft	0-250	0-1
	Soft	250-500	1-0.92
	Med. Stiff	500-1000	0.92-0.7
	Stiff	1000-2000	0.7-0.36
	Very Stiff	2000-4000	0.36-0.1875

Earth Pressure Coefficients K_{HC} and K_{HT} (NAVFAC 7.2-194 Fig.1)

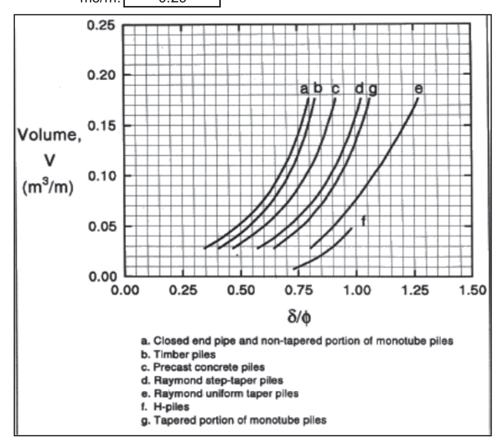
Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

Friction Angle - δ (NAVFAC 7.2-194 Fig.1)

Pile Type	δ
Steel	20
Concrete	3/4 φ
Timber	3/4 φ

Driven Piles (FHWA Driven Pile Foundations Fig 9.10 Page 9-29)

m3/m: 0.29



PILE CAPACITY CALCULATIONS

30-INCH DIAMETER/SIZE PILE

JOB NO.: Big Pine-1-01 BY: SR DESCRIPTION: Drilled Pile Capacity

CLIENT: Eocene Environmental Group DATE: 6/26/2025

PILE DIAMETER/SIZE: 30 in

OVERBURDEN PRESSURE @ PILE TOP: 0 psf

Provide if section is not circular:
SIDE AREA: ft^2/ft
TIP AREA: ft^2

Values used in calculations:
SIDE AREA: 7.9 ft^2/ft
TIP AREA: 4.9 ft^2

FACTORS OF SAFETY
FRICTION: 2
BEARING: 3

Depth Increment (ft): 2.00
δ/φ: 0.75

Downdrag Depth (ft): (if any)
Downdrag Force (kip):

NOTE: *Based on NAVFAC 7.02 "Foundations & Earth Structures". For depths > 20 pile diameters, the same overburden pressure is used in calcs.*
For C_A/C, δ/ϕ, K_{HC}, and N_q values see tables and Graph on Left

Layer parameters and depths are from bottom of pile cap.

Layer No.	Layer Depth (ft)	Bottom Layer Depth (ft)	c (psf)	φ (deg)	δ (deg)	δ (deg) used in calcs	γ' (psf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$
1	0	5	0	0	0	0	120	0	0.35	7	0
2	5	41.5	0	31	23.25	23.25	120	0	0.3	7	0
3											
4											
5											
6											
7											
8											

Penetration below pile cap (ft)	c (psf)	φ (deg)	γ' (pcf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$	δ (deg)	σ'_0 (psf) at mid-layer (psf)	Friction at mid-layer (psf)	Allowable Friction (psf)	ALLOWABLE DOWNWARD CAPACITY Friction (kips)	End Bear (kips)	Total (kips)	w/drag (kips)	ALL. UPWARD Total (kips)	w/drag (kips)
0	0	0	120	0	0.35	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.35	7	0	0	120	0	0	0	1	1	1	0	0
4	0	0	120	0	0.35	7	0	0	360	0	0	0	4	4	4	0	0
6	0	31	120	0	0.3	7	0	23.25	600	77	39	1	7	7	7	0	0
8	0	31	120	0	0.3	7	0	23.25	840	108	54	1	10	11	11	1	1
10	0	31	120	0	0.3	7	0	23.25	1080	139	70	3	12	15	15	1	1
12	0	31	120	0	0.3	7	0	23.25	1320	170	85	4	15	19	19	2	2
14	0	31	120	0	0.3	7	0	23.25	1560	201	101	5	18	23	23	3	3
16	0	31	120	0	0.3	7	0	23.25	1800	232	116	7	21	28	28	4	4
18	0	31	120	0	0.3	7	0	23.25	2040	263	131	9	23	33	33	5	5
20	0	31	120	0	0.3	7	0	23.25	2280	294	147	12	26	38	38	6	6
22	0	31	120	0	0.3	7	0	23.25	2520	325	162	14	29	43	43	7	7
24	0	31	120	0	0.3	7	0	23.25	2760	356	178	17	32	49	49	9	9
26	0	31	120	0	0.3	7	0	23.25	3000	387	193	20	34	54	54	10	10
28	0	31	120	0	0.3	7	0	23.25	3240	418	209	23	37	60	60	12	12
30	0	31	120	0	0.3	7	0	23.25	3480	449	224	27	40	67	67	13	13
32	0	31	120	0	0.3	7	0	23.25	3720	479	240	31	43	73	73	15	15
34	0	31	120	0	0.3	7	0	23.25	3960	510	255	35	45	80	80	17	17
36	0	31	120	0	0.3	7	0	23.25	4200	541	271	39	48	87	87	19	19
38	0	31	120	0	0.3	7	0	23.25	4440	572	286	43	51	94	94	22	22
40	0	31	120	0	0.3	7	0	23.25	4680	603	302	48	54	102	102	24	24

Recommended Values of C_A/C (NAVFAC 7.2-196 Fig. 2)

Pile Type	Consistency of Soil	Cohesion, C (psf)	C_A/C
Timber & Concrete	Very Soft	0-250	0-1
	Soft	250-500	1-0.96
	Med. Stiff	500-1000	0.96-0.75
	Stiff	1000-2000	0.75-0.475
	Very Stiff	2000-4000	0.475-0.325
Steel	Very Soft	0-250	0-1
	Soft	250-500	1-0.92
	Med. Stiff	500-1000	0.92-0.7
	Stiff	1000-2000	0.7-0.36
	Very Stiff	2000-4000	0.36-0.1875

Earth Pressure Coefficients K_{HC} and K_{HT} (NAVFAC 7.2-194 Fig.1)

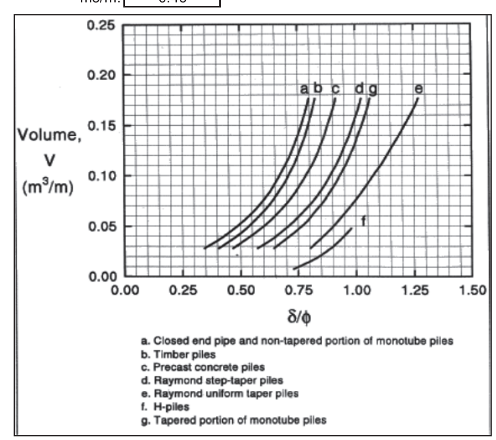
Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

Friction Angle - δ (NAVFAC 7.2-194 Fig.1)

Pile Type	δ
Steel	20
Concrete	3/4 φ
Timber	3/4 φ

Driven Piles (FHWA Driven Pile Foundations Fig 9.10 Page 9-29)

m3/m: 0.46



PILE CAPACITY CALCULATIONS

36-INCH DIAMETER/SIZE PILE

JOB NO.: Big Pine-1-01 BY: SR DESCRIPTION: Drilled Pile Capacity
CLIENT: Eocene Environmental Group DATE: 6/26/2025

PILE DIAMETER/SIZE:	36	in	Provide if section is not circular: SIDE AREA:		ft^2/ft	Values used in calculations: SIDE AREA:	9.4 ft^2/ft
OVERBURDEN PRESSURE @ PILE TOP:	0	psf	TIP AREA:		ft^2	TIP AREA:	7.1 ft^2

FACTORS OF SAFETY

FRICTION:	2	Depth Increment (ft):	2.00	Downdrag Depth (ft):	(if any)
BEARING:	3	δ/φ:	0.75	Downdrag Force (kip):	

NOTE: *Based on NAVFAC 7.02 "Foundations & Earth Structures". For depths > 20 pile diameters, the same overburden pressure is used in calcs. For C_A/C, δ/ϕ, K_{HC}, and N_q values see tables and Graph on Left*

Layer parameters and depths are from bottom of pile cap.

Layer No.	Layer Depth (ft)	Bottom Layer Depth (ft)	c (psf)	φ (deg)	δ (deg)	δ (deg) used in calcs	γ' (psf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$
1	0	5	0	0	0	0	120	0	0.35	7	0
2	5	41.5	0	31	23.25	23.25	120	0	0.3	7	0
3											
4											
5											
6											
7											
8											

Penetration below pile cap (ft)	c (psf)	φ (deg)	γ' (pcf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$	δ (deg)	σ'₀ (psf) at mid-layer (psf)	Friction at mid-layer (psf)	Allowable Friction (psf)	ALLOWABLE DOWNWARD CAPACITY Friction (kips)	End Bear (kips)	Total (kips)	w/drag (kips)	ALL. UPWARD Total (kips)	w/drag (kips)
0	0	0	120	0	0.35	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.35	7	0	0	120	0	0	0	2	2	2	0	0
4	0	0	120	0	0.35	7	0	0	360	0	0	0	6	6	6	0	0
6	0	31	120	0	0.3	7	0	23.25	600	77	39	1	10	11	11	0	0
8	0	31	120	0	0.3	7	0	23.25	840	108	54	2	14	16	16	1	1
10	0	31	120	0	0.3	7	0	23.25	1080	139	70	3	18	21	21	2	2
12	0	31	120	0	0.3	7	0	23.25	1320	170	85	5	22	26	26	2	2
14	0	31	120	0	0.3	7	0	23.25	1560	201	101	7	26	32	32	3	3
16	0	31	120	0	0.3	7	0	23.25	1800	232	116	9	30	38	38	4	4
18	0	31	120	0	0.3	7	0	23.25	2040	263	131	11	34	45	45	6	6
20	0	31	120	0	0.3	7	0	23.25	2280	294	147	14	38	52	52	7	7
22	0	31	120	0	0.3	7	0	23.25	2520	325	162	17	42	59	59	9	9
24	0	31	120	0	0.3	7	0	23.25	2760	356	178	20	46	66	66	10	10
26	0	31	120	0	0.3	7	0	23.25	3000	387	193	24	49	74	74	12	12
28	0	31	120	0	0.3	7	0	23.25	3240	418	209	28	53	81	81	14	14
30	0	31	120	0	0.3	7	0	23.25	3480	449	224	32	57	90	90	16	16
32	0	31	120	0	0.3	7	0	23.25	3720	479	240	37	61	98	98	18	18
34	0	31	120	0	0.3	7	0	23.25	3960	510	255	42	65	107	107	21	21
36	0	31	120	0	0.3	7	0	23.25	4200	541	271	47	69	116	116	23	23
38	0	31	120	0	0.3	7	0	23.25	4440	572	286	52	73	125	125	26	26
40	0	31	120	0	0.3	7	0	23.25	4680	603	302	58	77	135	135	29	29

Recommended Values of C_A/C (NAVFAC 7.2-196 Fig. 2)

Pile Type	Consistency of Soil	Cohesion, C (psf)	C_A/C
Timber & Concrete	Very Soft	0-250	0-1
	Soft	250-500	1-0.96
	Med. Stiff	500-1000	0.96-0.75
	Stiff	1000-2000	0.75-0.475
	Very Stiff	2000-4000	0.475-0.325
Steel	Very Soft	0-250	0-1
	Soft	250-500	1-0.92
	Med. Stiff	500-1000	0.92-0.7
	Stiff	1000-2000	0.7-0.36
	Very Stiff	2000-4000	0.36-0.1875

Earth Pressure Coefficients K_{HC} and K_{HT} (NAVFAC 7.2-194 Fig.1)

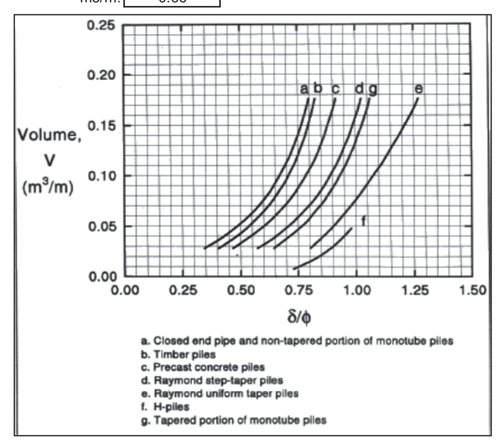
Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

Friction Angle - δ (NAVFAC 7.2-194 Fig.1)

Pile Type	δ
Steel	20
Concrete	3/4 φ
Timber	3/4 φ

Driven Piles (FHWA Driven Pile Foundations Fig 9.10 Page 9-29)

m3/m: 0.66



PILE CAPACITY CALCULATIONS

60-INCH DIAMETER/SIZE PILE

JOB NO.: Big Pine-1-01
CLIENT: Eocene Environmental Group
BY: SR
DATE: 6/26/2025
DESCRIPTION: Drilled Pile Capacity

PILE DIAMETER/SIZE: 60 in
OVERBURDEN PRESSURE @ PILE TOP: 0 psf

Provide if section is not circular:
SIDE AREA: ft^2/ft
TIP AREA: ft^2

Values used in calculations:
SIDE AREA: 15.7 ft^2/ft
TIP AREA: 19.6 ft^2

FACTORS OF SAFETY
FRICTION: 2
BEARING: 3

Depth Increment (ft): 2.00
δ/ϕ: 0.75

Downdrag Depth (ft): (if any)
Downdrag Force (kip):

NOTE: *Based on NAVFAC 7.02 "Foundations & Earth Structures". For depths > 20 pile diameters, the same overburden pressure is used in calcs.*
For C_A/C, δ/ϕ, K_{HC}, and N_q values see tables and Graph on Left

Layer parameters and depths are from bottom of pile cap.

Layer No.	Layer Depth (ft)	Bottom Layer Depth (ft)	c (psf)	ϕ (deg)	δ (deg)	δ (deg) used in calcs	γ' (psf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$
1	0	5	0	0	0	0	120	0	0.35	7	0
2	5	41.5	0	31	23.25	23.25	120	0	0.3	7	0
3											
4											
5											
6											
7											
8											

Penetration below pile cap (ft)	c (psf)	ϕ (deg)	γ' (pcf)	c_A/c	K_{HC}	N_q	$N_{cs/cc}$	δ (deg)	σ'_0 (psf) at mid-layer (psf)	Friction at mid-layer (psf)	Allowable Friction (psf)	ALLOWABLE DOWNWARD CAPACITY Friction (kips)	End Bear (kips)	Total (kips)	w/drag (kips)	ALL. UPWARD Total (kips)	w/drag (kips)
0	0	0	120	0	0.35	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.35	7	0	0	120	0	0	0	5	5	5	0	0
4	0	0	120	0	0.35	7	0	0	360	0	0	0	16	16	16	0	0
6	0	31	120	0	0.3	7	0	23.25	600	77	39	1	27	29	29	1	1
8	0	31	120	0	0.3	7	0	23.25	840	108	54	3	38	41	41	1	1
10	0	31	120	0	0.3	7	0	23.25	1080	139	70	5	49	55	55	3	3
12	0	31	120	0	0.3	7	0	23.25	1320	170	85	8	60	68	68	4	4
14	0	31	120	0	0.3	7	0	23.25	1560	201	101	11	71	82	82	5	5
16	0	31	120	0	0.3	7	0	23.25	1800	232	116	15	82	97	97	7	7
18	0	31	120	0	0.3	7	0	23.25	2040	263	131	19	93	112	112	9	9
20	0	31	120	0	0.3	7	0	23.25	2280	294	147	23	104	128	128	12	12
22	0	31	120	0	0.3	7	0	23.25	2520	325	162	28	115	144	144	14	14
24	0	31	120	0	0.3	7	0	23.25	2760	356	178	34	126	160	160	17	17
26	0	31	120	0	0.3	7	0	23.25	3000	387	193	40	137	178	178	20	20
28	0	31	120	0	0.3	7	0	23.25	3240	418	209	47	148	195	195	23	23
30	0	31	120	0	0.3	7	0	23.25	3480	449	224	54	159	213	213	27	27
32	0	31	120	0	0.3	7	0	23.25	3720	479	240	61	170	232	232	31	31
34	0	31	120	0	0.3	7	0	23.25	3960	510	255	69	181	251	251	35	35
36	0	31	120	0	0.3	7	0	23.25	4200	541	271	78	192	270	270	39	39
38	0	31	120	0	0.3	7	0	23.25	4440	572	286	87	203	290	290	43	43
40	0	31	120	0	0.3	7	0	23.25	4680	603	302	96	214	311	311	48	48

Recommended Values of C_A/C (NAVFAC 7.2-196 Fig. 2)

Pile Type	Consistency of Soil	Cohesion, C (psf)	C_A/C
Timber & Concrete	Very Soft	0-250	0-1
	Soft	250-500	1-0.96
	Med. Stiff	500-1000	0.96-0.75
	Stiff	1000-2000	0.75-0.475
	Very Stiff	2000-4000	0.475-0.325
Steel	Very Soft	0-250	0-1
	Soft	250-500	1-0.92
	Med. Stiff	500-1000	0.92-0.7
	Stiff	1000-2000	0.7-0.36
	Very Stiff	2000-4000	0.36-0.1875

Earth Pressure Coefficients K_{HC} and K_{HT} (NAVFAC 7.2-194 Fig.1)

Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

Friction Angle - δ (NAVFAC 7.2-194 Fig.1)

Pile Type	δ
Steel	20
Concrete	3/4 ϕ
Timber	3/4 ϕ

Driven Piles (FHWA Driven Pile Foundations Fig 9.10 Page 9-29)

m3/m: 1.82