GEOTECHNICAL INVESTIGATION REPORT
PROPOSED TELECOMMUNICATIONS FACILITY
SELENIUM
APN: 0314-221-09-0-000

Big Bear, California

Prepared for:
SPECTRUM SERVICES

Prepared by:
GEOBODEN INC.
Irvine, CA 92620

August 13, 2015

Project No. Selenium-1-01

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED TELECMMUNICATIONS FACILITY

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APN: 0314-221-09-0-000

BIG BEAR, CALIFORNIA

SPECTRUM SERVICES

Prepared by:

GEOBODEN INC.
5 Hodgenville
Irvine, California 92620

August 13, 2015

J.N. Selenium-1-01

GEOBODEN INC.
GEOTECHNICAL CONSULTANTS

August 13, 2015 Project No. Selenium-1-01

Ms. Erika Larios
Spectrum Services
4405 East Airport Drive, Suite 100
Ontario, CA. 91761

Subject: Geotechnical Investigation Report
Proposed Telecommunications Facility
Selenium
APN: 0314-221-09-0-000
Big Bear, California

Dear Ms. Larios:

GeoBoden, Inc. is pleased to provide you four (4) 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

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

Cyrus Radvar, G.E.
Principal Geotechnical Engineer

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GEOTECHNICAL INVESTIGATION REPORT

PROPOSED TELECOMMUNICATIONS FACILITY

SELENIUM

APN: 0314-221-09-0-000

BIG BEAR, CALIFORNIA

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GEOTECHNICAL INVESTIGATION REPORT
PROPOSED TELECOMMUNICATIONS FACILITY
SELENIUM
APN: 0314-221-09-0-000
Big Bear, 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 Bear, 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 Verizon Wireless telecommunications facility. The proposed facility will be about 784 square feet in plan dimension. The facility will include one equipment cabinets on a concrete pad inside 8-foot block wall, an antenna which will be about 35 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 antenna 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 located within a mapped liquefaction hazard zone.
- The site is not located within a mapped Alquist-Priolo (AP) Special Study Zone.
- 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 sandy silt, silty sand and sand with silt. The native soils are primarily medium dense.

4.2 GROUNDWATER CONDITIONS

Groundwater was not encountered within our exploratory boring. 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 6 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 antenna tower may be supported on a new typical, large-diameter reinforced concrete drilled pier; Cast-In-Drill-Hole (CIDH) pile. The base reactions for the antenna will be derived from side friction for axial loads, and from passive soil resistance for lateral and over-turning forces. For the proposed drilled pier, 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	3,000

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*	Sandy Silt	-	120	-	-	0.40	-
5 – 40	Sandy Silt & Silty Sand	15-32	125	24	400	0.35	300
Depth of Layer (ft.)	**Ultimate 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	3,000	200	300	500		200	-

* 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 250 psf per foot of pier embedment depth up to a maximum of 2,500 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 2013 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. Base 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 may be expected within the anticipated design depth for the pier. Contractor should be prepared for caving and possible dewatering.

5.2 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 2013 edition of the California Building Code (CBC). Table below, 2013 CBC Seismic Parameters, lists (next) seismic design parameters based on the 2013 CBC methodology, which is based on ASCE/SEI 7-10:

2013 CBC Seismic Design Parameters	Value
Site Latitude (decimal degrees)	34.2723
Site Longitude (decimal degrees)	-116.7956
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	1.950
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.697
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, F_v (Table 1613.3.3(2))	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16-37)	1.950
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-38)	1.045
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-39)	1.300
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-40)	0.697

5.3 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 was not encountered to the maximum explored depth (41.5 feet bgs). The onsite soils are medium dense. It is our opinion the potential for liquefaction at the site is low.

5.4 SHALLOW FOUNDATIONS

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

5.4.1 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.

5.4.2 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.

5.4.3 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 expansive soils should have No. 4 bars (one top and one bottom).

5.5 CONCRETE SLAB ON-GRADE

Concrete slabs will be placed on properly compacted fill as outlined in the following section (Section 5.6). 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 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 18 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid depth.

5.6 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 near 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.”

5.7 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.

5.8 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.

5.9 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), 2013.

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

FIGURES

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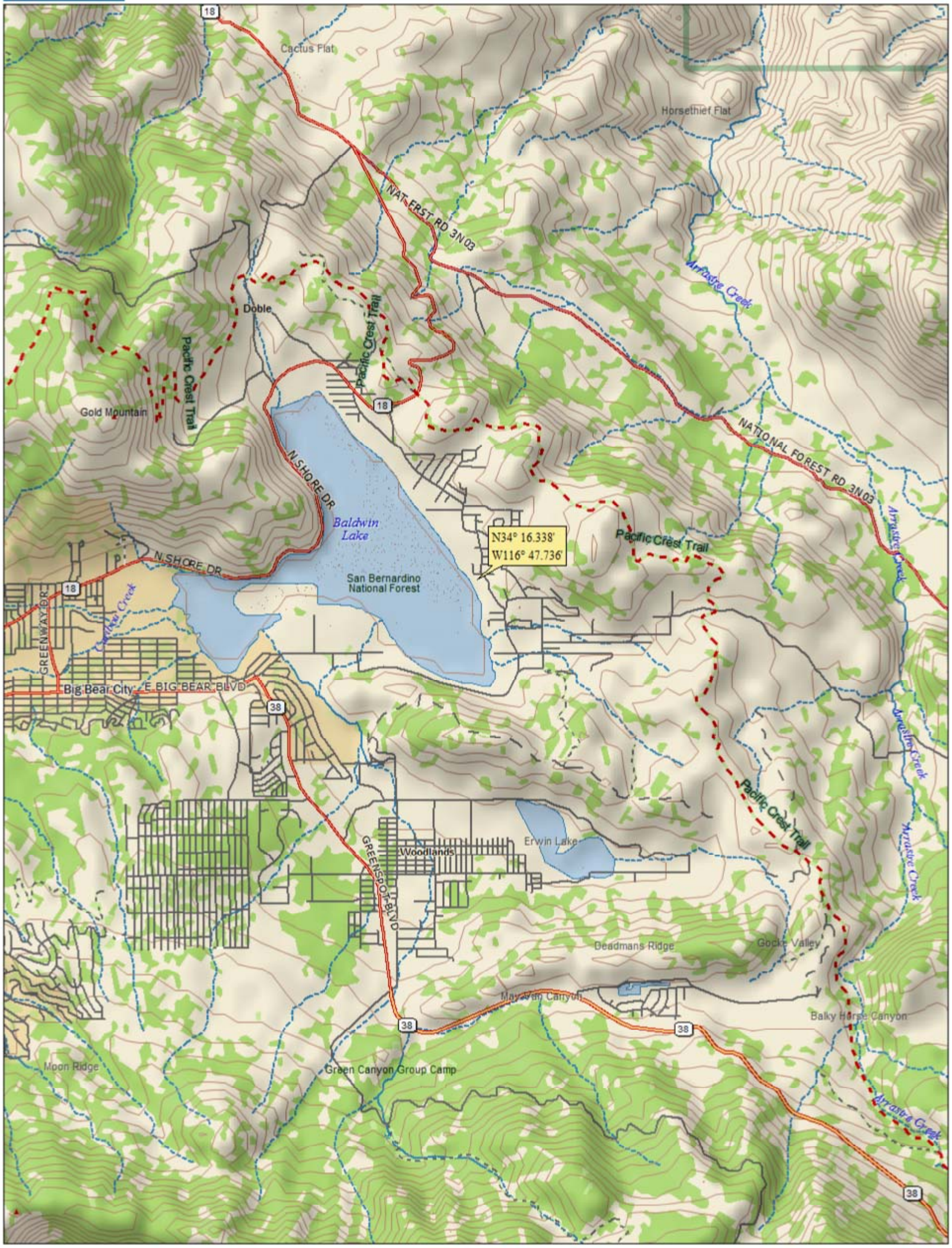

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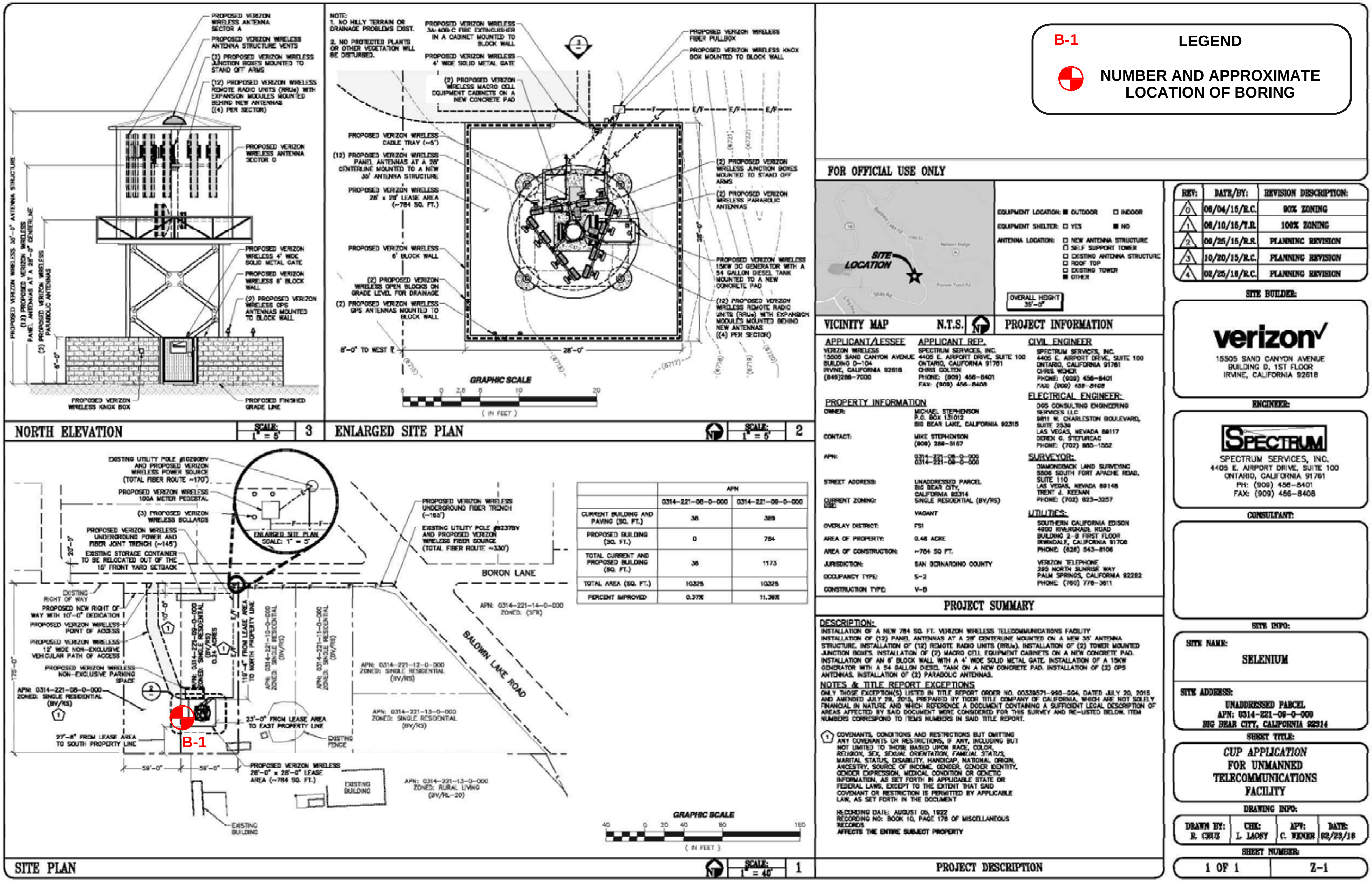
GEOBODEN INC. Geotechnical Consultants	**SITE LOCATION MAP** **Proposed Telecommunications Facility** **Selenium** **APN: 0314-221-09-0-000** **Big Bear, California**	Figure By S.R.	Project No. Selenium-1-01
		Map No. XX	Figure No. 1
		Date 08-13-15	



GEOBODEN INC. Geotechnical Consultants	BORING LOCATION PLAN Proposed Telecommunications Facility Selenium APN: 0314-221-09-0-000 Big Bear, California	Figure By S.R.	Project No. Selenium-1-01
		Map No. XX	Figure No.
		Date 08-13-15	2

APPENDIX A
BORING LOGS

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

PROPOSED TELECOMMUNICATIONS FACILITY
SELENIUM
APN: 0314-221-09-0-000
BIG BEAR, 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 using a hollow-stem auger drill rig equipped with 8-inch outside diameter (O.D.) augers. 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.

CLIENT Spectrum Services
PROJECT NAME Proposed Telecommunications Facility
PROJECT NUMBER Selenium-1-01
PROJECT LOCATION Big Bear, CA
DATE STARTED 6/26/15 **COMPLETED** 6/26/15
GROUND ELEVATION **HOLE SIZE** 8 inches
DRILLING CONTRACTOR GeoBoden, Inc.
GROUND WATER LEVELS:
DRILLING METHOD HSA
AT TIME OF DRILLING ---
LOGGED BY S.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	SANDY SILT (ML): light olive gray, moist, ~30% fine sand, ~70% fines										
5		MC R-1		32		104	16				67
	SILTY SAND to SANDY SILT (SM/ML): greenish gray, moist, ~50% fines, ~50% sand										
10		MC R-2		27		107	14				
	POORLY-GRADED SAND w. SILT (SP-SM): light greenish gray, moist, ~10% fines, ~90% fine sand										
15		MC R-3		36							12
	SANDY SILT (ML): olive gray, moist, ~30% fine sand, ~70% fines										
20		MC R-4		30							64
	SILTY SAND (SM): olive gray, moist, ~30% fines, ~70% fine sand										
25		SS S-5		18							
30		SS S-6		23							
	POORLY-GRADED SAND w. SILT (SP-SM): olive, moist, ~10% fines, ~90% sand										
35											

(Continued Next Page)

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BORING NUMBER B-1

GEOBODEN, INC.

CLIENT Spectrum Services

PROJECT NAME Proposed Telecommunications Facility

PROJECT NUMBER Selenium-1-01

PROJECT LOCATION Big Bear, 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		POORLY-GRADED SAND w. SILT (SP-SM): olive, moist, ~10% fines, ~90% sand *(continued)*	SS S-7		27							
40			SS S-8		32							

Bottom of borehole a 41.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with cuttings.

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

PROPOSED TELECOMMUNICATIONS FACILITY
SELENIUM
APN: 0314-221-09-0-000
BIG BEAR, 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)

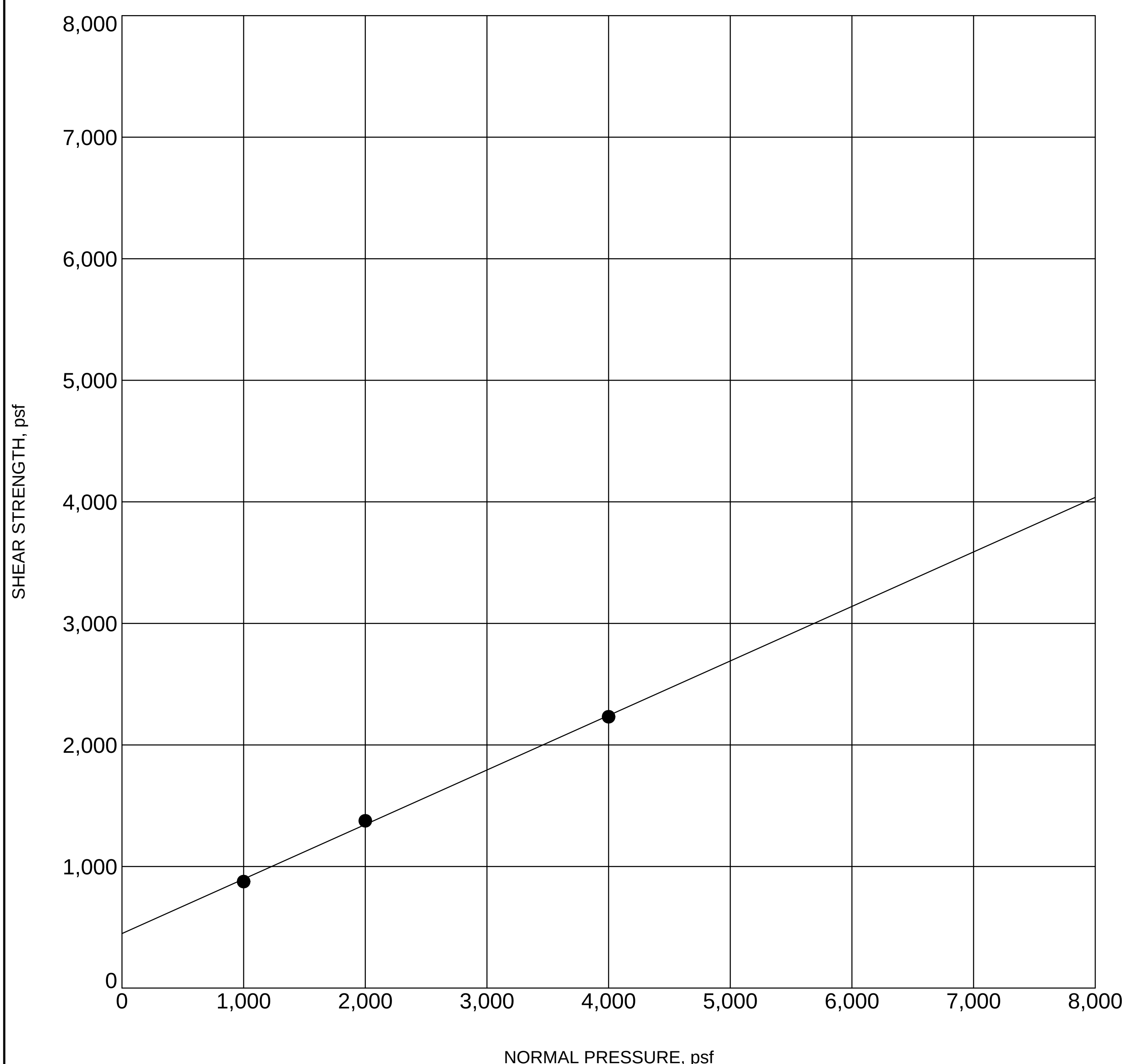
Boring No.	Depth (ft)	Chloride Content (Calif. 422) ppm	Sulfate Content (Calif. 417) % by Weight	pH (Calif. 643)	Resistivity (Calif. 643) Ohm*cm
B-1	0-5	96	0.0187	7.4	1,497

DIRECT SHEAR TEST

GEOBODEN, INC.

CLIENT Spectrum Services | **PROJECT NAME** Proposed Telecommunications Facility

PROJECT NUMBER Selenium-1-01 | **PROJECT LOCATION** Big Bear, CA



	Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-1**	**10.0**	**SANDY SILT (ML)**	**107**	**14**	**448.0**	**24**

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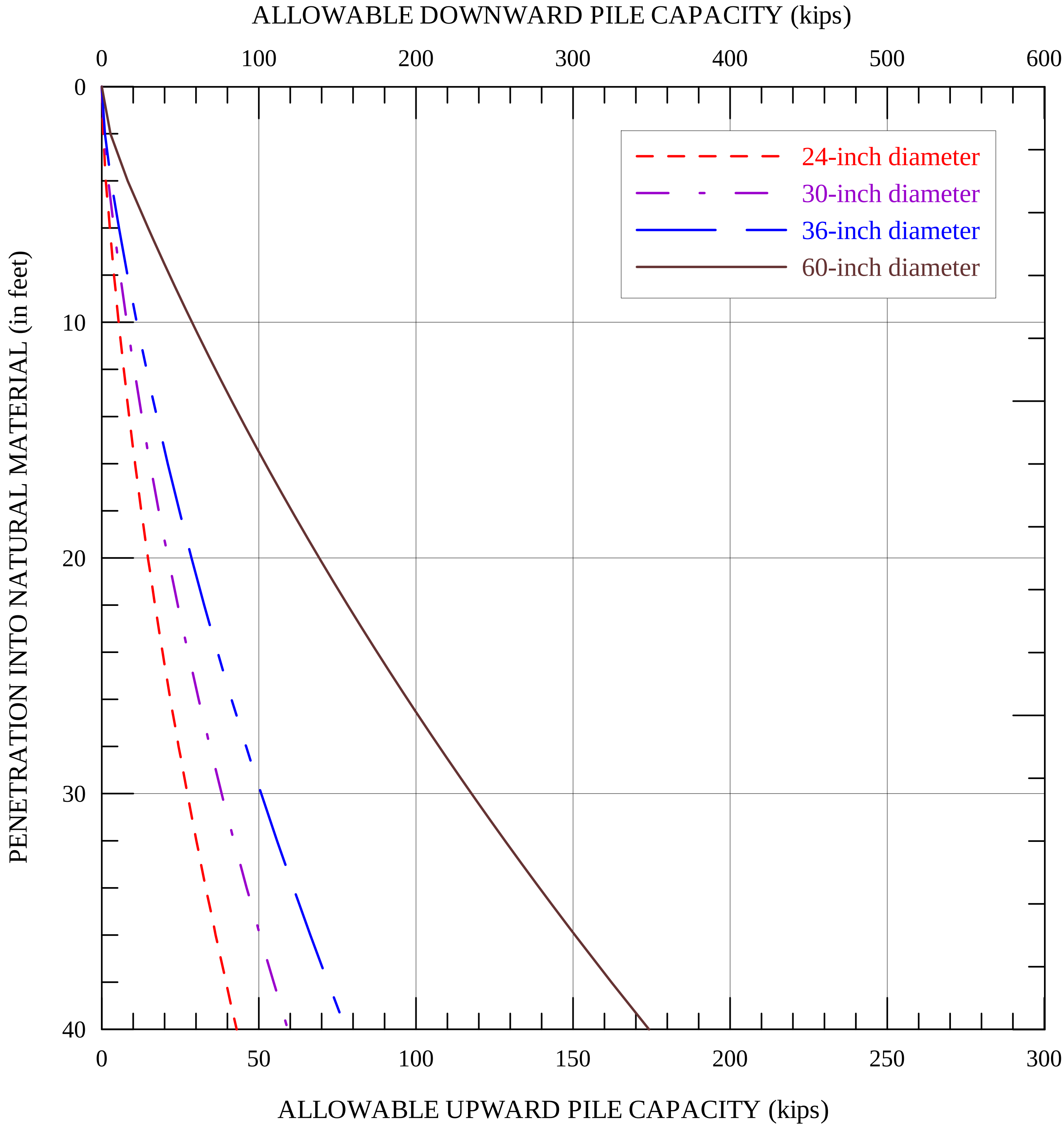
APPENDIX C
AXILE PILE CAPACITY



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 Selenium APN: 0314-221-09-0-000 Big Bear, California	GEOBODEN, INC.	DRILLED PILE CAPACITY Project No. Selenium-1-01 Figure C-1

PILE CAPACITY CALCULATIONS

24-INCH DIAMETER/SIZE PILE

JOB NO.: Selenium BY: SR DESCRIPTION: Drilled Pile Capacity
CLIENT: Spectrum DATE: 8/13/2015

			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	0.4	7	0
2	5	41.5	400	24	18	18	125	0	0.5	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	0.4	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.4	7	0	0	120	0	0	0	1	1	1	0	0
4	0	0	120	0	0.4	7	0	0	360	0	0	0	3	3	3	0	0
6	400	24	125	0	0.5	7	0	18	610	99	50	1	4	5	5	0	0
8	400	24	125	0	0.5	7	0	18	860	140	70	2	6	8	8	1	1
10	400	24	125	0	0.5	7	0	18	1110	180	90	3	8	11	11	1	1
12	400	24	125	0	0.5	7	0	18	1360	221	110	4	10	14	14	2	2
14	400	24	125	0	0.5	7	0	18	1610	262	131	6	12	17	17	3	3
16	400	24	125	0	0.5	7	0	18	1860	302	151	8	14	21	21	4	4
18	400	24	125	0	0.5	7	0	18	2110	343	171	10	15	25	25	5	5
20	400	24	125	0	0.5	7	0	18	2360	383	192	12	17	29	29	6	6
22	400	24	125	0	0.5	7	0	18	2610	424	212	15	19	34	34	7	7
24	400	24	125	0	0.5	7	0	18	2860	465	232	18	21	39	39	9	9
26	400	24	125	0	0.5	7	0	18	3110	505	253	21	23	44	44	10	10
28	400	24	125	0	0.5	7	0	18	3360	546	273	24	25	49	49	12	12
30	400	24	125	0	0.5	7	0	18	3610	586	293	28	26	54	54	14	14
32	400	24	125	0	0.5	7	0	18	3860	627	314	32	28	60	60	16	16
34	400	24	125	0	0.5	7	0	18	4110	668	334	36	30	66	66	18	18
36	400	24	125	0	0.5	7	0	18	4360	708	354	41	32	73	73	20	20
38	400	24	125	0	0.5	7	0	18	4610	749	374	45	34	79	79	23	23
40	400	24	125	0	0.5	7	0	18	4860	790	395	50	36	86	86	25	25

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)

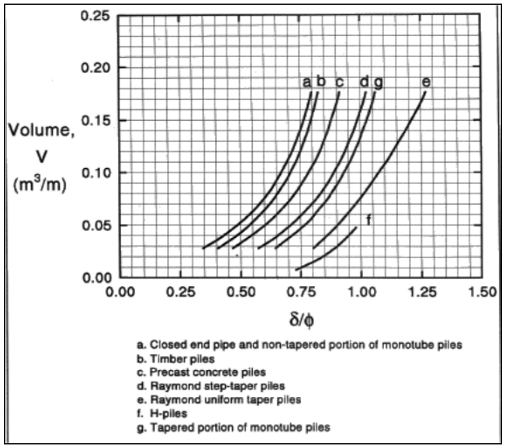
Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

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.: Selenium BY: SR DESCRIPTION: Drilled Pile Capacity
CLIENT: Spectrum DATE: 8/13/2015

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	0.4	7	0
2	5	41.5	400	24	18	18	125	0	0.5	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	0.4	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.4	7	0	0	120	0	0	0	1	1	1	0	0
4	0	0	120	0	0.4	7	0	0	360	0	0	0	4	4	4	0	0
6	400	24	125	0	0.5	7	0	18	610	99	50	1	7	8	8	0	0
8	400	24	125	0	0.5	7	0	18	860	140	70	2	10	12	12	1	1
10	400	24	125	0	0.5	7	0	18	1110	180	90	3	13	16	16	2	2
12	400	24	125	0	0.5	7	0	18	1360	221	110	5	16	21	21	3	3
14	400	24	125	0	0.5	7	0	18	1610	262	131	7	18	26	26	4	4
16	400	24	125	0	0.5	7	0	18	1860	302	151	9	21	31	31	5	5
18	400	24	125	0	0.5	7	0	18	2110	343	171	12	24	36	36	6	6
20	400	24	125	0	0.5	7	0	18	2360	383	192	15	27	42	42	8	8
22	400	24	125	0	0.5	7	0	18	2610	424	212	18	30	48	48	9	9
24	400	24	125	0	0.5	7	0	18	2860	465	232	22	33	55	55	11	11
26	400	24	125	0	0.5	7	0	18	3110	505	253	26	36	62	62	13	13
28	400	24	125	0	0.5	7	0	18	3360	546	273	30	38	69	69	15	15
30	400	24	125	0	0.5	7	0	18	3610	586	293	35	41	76	76	17	17
32	400	24	125	0	0.5	7	0	18	3860	627	314	40	44	84	84	20	20
34	400	24	125	0	0.5	7	0	18	4110	668	334	45	47	92	92	23	23
36	400	24	125	0	0.5	7	0	18	4360	708	354	51	50	101	101	25	25
38	400	24	125	0	0.5	7	0	18	4610	749	374	57	53	109	109	28	28
40	400	24	125	0	0.5	7	0	18	4860	790	395	63	56	118	118	31	31

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)

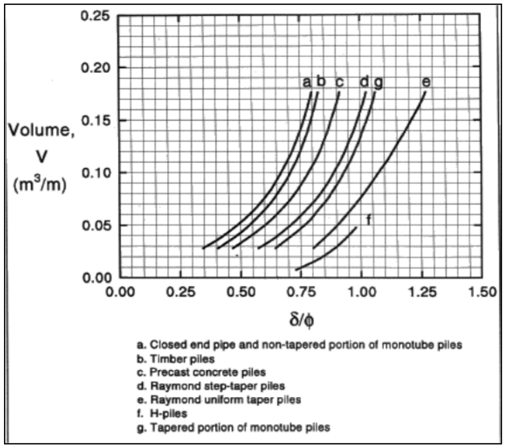
Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

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.: Selenium | BY: SR | DESCRIPTION: Drilled Pile Capacity

CLIENT: Spectrum | DATE: 8/13/2015

			Provide if section is not circular:			Values used in calculations:		
PILE DIAMETER/SIZE:	36	in	SIDE AREA:		ft^2/ft	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	0.4	7	0
2	5	41.5	400	24	18	18	125	0	0.5	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	0.4	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.4	7	0	0	120	0	0	0	2	2	2	0	0
4	0	0	120	0	0.4	7	0	0	360	0	0	0	6	6	6	0	0
6	400	24	125	0	0.5	7	0	18	610	99	50	1	10	11	11	0	0
8	400	24	125	0	0.5	7	0	18	860	140	70	2	14	16	16	1	1
10	400	24	125	0	0.5	7	0	18	1110	180	90	4	18	22	22	2	2
12	400	24	125	0	0.5	7	0	18	1360	221	110	6	22	28	28	3	3
14	400	24	125	0	0.5	7	0	18	1610	262	131	8	27	35	35	4	4
16	400	24	125	0	0.5	7	0	18	1860	302	151	11	31	42	42	6	6
18	400	24	125	0	0.5	7	0	18	2110	343	171	15	35	49	49	7	7
20	400	24	125	0	0.5	7	0	18	2360	383	192	18	39	57	57	9	9
22	400	24	125	0	0.5	7	0	18	2610	424	212	22	43	65	65	11	11
24	400	24	125	0	0.5	7	0	18	2860	465	232	27	47	74	74	13	13
26	400	24	125	0	0.5	7	0	18	3110	505	253	31	51	83	83	16	16
28	400	24	125	0	0.5	7	0	18	3360	546	273	36	55	92	92	18	18
30	400	24	125	0	0.5	7	0	18	3610	586	293	42	60	102	102	21	21
32	400	24	125	0	0.5	7	0	18	3860	627	314	48	64	112	112	24	24
34	400	24	125	0	0.5	7	0	18	4110	668	334	54	68	122	122	27	27
36	400	24	125	0	0.5	7	0	18	4360	708	354	61	72	133	133	30	30
38	400	24	125	0	0.5	7	0	18	4610	749	374	68	76	144	144	34	34
40	400	24	125	0	0.5	7	0	18	4860	790	395	75	80	156	156	38	38

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)

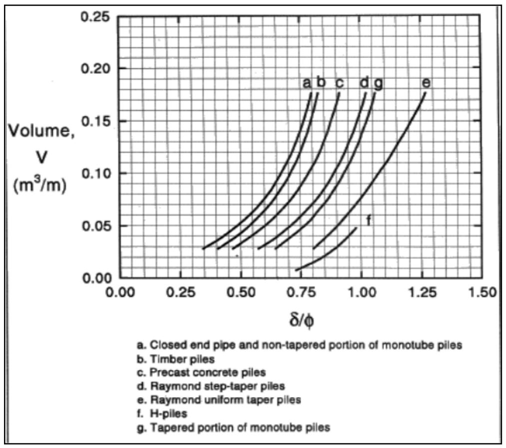
Pile Type	K_{HC}	K_{HT}
Driven Single H-Pile	0.5-1.0	0.3-0.5
Driven Single Displacement pile	1.0-1.5	0.6-1.0
Driven Single Displacement Tapered pile	1.5-2.0	1.0-1.3
Driven Jetted Pile	0.4-0.9	0.3-0.6
Drilled Pile (less than 24" diameter)	0.7	0.4

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.: Selenium   BY: SR   DESCRIPTION: Drilled Pile Capacity
CLIENT: Spectrum   DATE: 8/13/2015

			Provide if section is not circular:			Values used in calculations:	
PILE DIAMETER/SIZE:	60	in	SIDE AREA:		ft^2/ft	SIDE AREA:	15.7 ft^2/ft
OVERBURDEN PRESSURE @ PILE TOP:	0	psf	TIP AREA:		ft^2	TIP AREA:	19.6 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	0.4	7	0
2	5	41.5	400	24	18	18	125	0	0.5	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	0.4	7	0	0	0	0	0	0	0	0	0	0	0
2	0	0	120	0	0.4	7	0	0	120	0	0	0	5	5	5	0	0
4	0	0	120	0	0.4	7	0	0	360	0	0	0	16	16	16	0	0
6	400	24	125	0	0.5	7	0	18	610	99	50	2	28	30	30	1	1
8	400	24	125	0	0.5	7	0	18	860	140	70	4	39	43	43	2	2
10	400	24	125	0	0.5	7	0	18	1110	180	90	7	51	57	57	3	3
12	400	24	125	0	0.5	7	0	18	1360	221	110	10	62	72	72	5	5
14	400	24	125	0	0.5	7	0	18	1610	262	131	14	74	88	88	7	7
16	400	24	125	0	0.5	7	0	18	1860	302	151	19	85	104	104	9	9
18	400	24	125	0	0.5	7	0	18	2110	343	171	24	97	121	121	12	12
20	400	24	125	0	0.5	7	0	18	2360	383	192	30	108	138	138	15	15
22	400	24	125	0	0.5	7	0	18	2610	424	212	37	120	157	157	18	18
24	400	24	125	0	0.5	7	0	18	2860	465	232	44	131	175	175	22	22
26	400	24	125	0	0.5	7	0	18	3110	505	253	52	142	195	195	26	26
28	400	24	125	0	0.5	7	0	18	3360	546	273	61	154	215	215	30	30
30	400	24	125	0	0.5	7	0	18	3610	586	293	70	165	235	235	35	35
32	400	24	125	0	0.5	7	0	18	3860	627	314	80	177	257	257	40	40
34	400	24	125	0	0.5	7	0	18	4110	668	334	90	188	279	279	45	45
36	400	24	125	0	0.5	7	0	18	4360	708	354	101	200	301	301	51	51
38	400	24	125	0	0.5	7	0	18	4610	749	374	113	211	324	324	57	57
40	400	24	125	0	0.5	7	0	18	4860	790	395	126	223	348	348	63	63

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