



GHD Geotechnical Investigation

SFVAMC - PICU Expansion and Renovation
San Francisco, California



November 2015



November 5, 2015

Paige Smith
POLYTECH ASSOCIATES INC.
235 Pine Street, 17th Floor
San Francisco, CA 94104

RE: Geotechnical Investigation, SFVAMC - PICU Expansion and Renovation

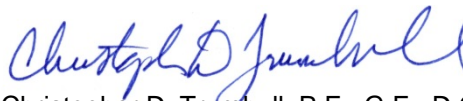
Dear Ms. Smith,

GHD Inc (GHD) is pleased to present the attached report containing the results of our geotechnical investigation for the proposed San Francisco VA Medical Center (SFVAMC) - Psychiatric Intensive Care Unit (PICU) Expansion and Renovation project in San Francisco, California. It is our understanding that Building 203 will be expanded and renovated. The southwest corner of the "A" wing of the building will be expanded to the south with 1,100 square feet of new PICU space. The western portion (8,000 square feet) of the "A" wing of the existing PICU facility will be renovated. It is our understanding that structural improvements are not required for the renovation portion and therefore geotechnical input is not needed.

The accompanying report presents our findings, conclusions, and recommendations developed from our geotechnical investigation. Contained in the report are geotechnical design criteria and recommendations for design and construction of the proposed building addition as well as earthwork recommendations. The results of the subsurface exploration and laboratory testing programs, which form the basis of our recommendations, are also included in the report. On the basis of our investigation, the site is suitable, from a geotechnical perspective, to receive the planned improvements provided the recommendations included in the report are incorporated into the design and construction of the project.

If you have any questions regarding the information contained in this report, or if we may be of further assistance, please do not hesitate to contact us.

Sincerely,
GHD Inc


Christopher D. Trumbull, P.E., G.E., D.GE
Senior Geotechnical Engineer




Kyle Jermstad, E.I.T.
Staff Engineer

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Distribution

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1. Introduction

This report presents the findings, conclusions, and recommendations developed from our geotechnical investigation. The investigation was conducted in accordance with the Standard Form of Agreement Between Architect and Consultant signed July 2, 2015.

1.1 Project Description

It is our understanding that Building 203 will be expanded and renovated. The southwest corner of the "A" wing of the building will be expanded to the south with 1,100 square feet of new PICU space. The western portion (8,000 square feet) of the "A" wing of the existing PICU facility will be renovated. We understand individual column loads are 160 kips (120 kips dead load and 40 kips live load), according to Mr. Robert Graff of Degenkolb Engineers. There are no uplift or lateral loads, as they are being applied to the existing structure. It is also our understanding that structural improvements of the existing building are not required and therefore geotechnical input is not needed.

1.2 Purpose and Scope of Work

The purpose of this investigation was to evaluate the suitability of the project site, from a geotechnical perspective, for the proposed improvements. The main objectives of the investigation were to characterize the subsurface materials, perform engineering analyses, develop geotechnical recommendations and criteria to be used for design and construction, and document our findings, conclusions, and recommendations in this report.

The scope of our geotechnical investigation included the following:

- A review of published geologic and geotechnical material pertaining to the site vicinity;
- One boring drilled to an approximate depth of 41½ feet to obtain sufficient information to evaluate the subsurface conditions and provide foundation recommendations for the expansion;
- Performing geotechnical laboratory testing on select soil samples collected from the borings;
- Engineering analyses to develop geotechnical design criteria and recommendations for the proposed project; and
- Preparation of this report.

2. Field Exploration and Laboratory Testing

2.1 Field Exploration

One boring was drilled to an approximate depth of 41½ feet below ground surface (bgs) on August 14, 2015 at the approximate location shown in Figures A-1 and A-2. The boring was located in the field based on a land survey by others. The boring was drilled under the supervision of Kyle Jermstad of GHD utilizing a truck-mounted CME-55 drill rig equipped with 6-inch solid flight augers. Samples were collected using both Modified California and SPT samplers driven by an automatic hammer with a weight of 140 pounds and a drop of 30 inches.

The number of blows required for each 6-inch increment of drive were recorded and the cumulative blow count for the 12 inches of drive (following the first 6 inches of “seating” drive), or fraction thereof where resistance was encountered, is presented in the logs of borings. The blow counts presented in the logs are uncorrected and shown as they were recorded in the field. Both the samples and drill cuttings were visually classified in the field based on the Unified Soil Classification System (USCS) in general accordance with ASTM D2488.

The subsurface conditions encountered are summarized in Section 3.2. Logs of the borings were initially prepared based on the field logging and visual examination of the soil samples in the field. Considering the results of the laboratory testing, the soil classifications were finalized in general accordance with ASTM D2478.

The soil boring key and the logs of borings are presented in Appendix B.

Previous explorations by others performed in the project area include the following:

- Eight borings drilled by Woodward-Lundgren & Associates in 1971 to a maximum depth of 43 feet below ground surface; and
- One boring performed by Fugro in 2011 to a maximum depth of 45.4 feet below ground surface.

The soil boring key, logs of borings, and laboratory results performed by others are presented in Appendix D.

2.2 Laboratory Testing

Laboratory testing was conducted on disturbed soil samples recovered during the site investigation.

Tests conducted include the following:

- Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock by Mass (ASTM D2216);
- Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318);
- Standard Test Method for Particle-Size Analysis of Soils (ASTM D422);
- Method of Testing Soil and Waters for Sulfate Content (CTM 417);

- Method of Testing Soil and Waters for Chloride Content (CTM 422);
- Method for Estimating the Service Life of Steel Culverts (CTM 643);
- Redox Potential ASTM D1498m; and
- Sulfides (AWWA C105/A25.5)

Geotechnical laboratory test results are presented in Appendix C.

3. Site and Subsurface Conditions

3.1 Site Conditions

The site is located at the VA Medical Center at the northwest corner of the San Francisco Peninsula. The proposed addition is located at the southwest corner of the A-wing of Building 203, the Psychiatric Intensive Care Unit. At the time of our exploration the flat site was occupied by an at-grade concrete loading dock and air conditioning units.

3.2 Subsurface Conditions

3.2.1 General Geology and Faulting

The area is mapped as being primarily composed of Quaternary dune sand (Qs) overlying Franciscan Complex. The nearest active fault to the site is the San Andreas Fault, which is just less than 4½ miles to the west. The San Andreas Fault is an active dextral strike-slip fault with Holocene and Historic movement. According to the Alquist-Priolo Earthquake Fault Zone Act, the surrounding project area is not within a Special Studies Zone; however, the property is bordered by a California Geological Survey Earthquake Induced Landslide Study Zone. Strong ground shaking at the site should be expected during an earthquake.

3.2.2 Subsurface Materials

Six inches of concrete existed at the ground surface. The results of our field exploration and laboratory analyses indicate the subsurface materials below the concrete generally consist of medium dense SAND (SP) to an approximate depth of 22½ feet. Medium dense to dense clayey SAND (SC) was encountered at an approximate depth of 22½ feet to 38 feet. Franciscan Formation was encountered below the sand to the maximum depth explored, approximately 41½ feet bgs. The subsurface conditions are in general agreement with the materials presented on boring logs by others in the close proximity. Details of subsurface materials are presented in the GHD logs of borings as well as previous borings by others presented in Appendix B.

3.2.3 Groundwater Conditions

The depth to groundwater was measured by GHD at 30½ feet bgs at the time of drilling in the recent boring. Although free groundwater was not noted by Fugro in its 2011 Boring B-1, seepage was noted at 21 feet. It should be noted that the borings may not have been left open long enough to establish equilibrium groundwater conditions. In addition, fluctuations in groundwater level could occur due to changes in season, variations in rainfall, pumping, or other factors.

4. Conclusions

The site is suitable for the proposed improvements provided the recommendations presented herein are incorporated into the design and construction of the project.

4.1 Foundation Support and Settlement

Based on our subsurface exploration, the sandy bearing materials are moderately strong and are not questionable. The proposed addition foundation can be supported on a deep foundation deriving support from the native sandy soil. For foundations and subgrade designed and prepared as recommended in this report, differential settlements are expected to be no greater than 1/2 inch over a horizontal distance of 50 feet. Total settlements of less than 1/2 inch are anticipated.

4.2 Expansion Potential

Expansive soils are defined as soils that undergo large volume changes (shrink or swell) due to variations in moisture content and meet four provisions in the 2012 International Building Code (IBC). Such volume changes may cause damaging settlement and/or heave of foundations, slabs-on-grade, pavements, etc. The sandy soil encountered does not meet the IBC provisions and has a low expansion potential.

4.3 Slope Stability

Landslides occur due to multiple factors, including slope inclination, bedrock geology, geologic structure, geomorphology, weathering, vegetation, and precipitation. Due to the relatively flat inclination of the site, landslide potential is considered negligible.

4.4 Groundwater and Wet Weather Earthwork

Groundwater was encountered at 30½ feet bgs during GHD's recent drilling and is not anticipated to be encountered during construction. At the time of our field exploration, the existing near surface soils were dry to moist. These soils could become unworkable during and shortly after periods of rainfall and require drying to reach a workable moisture content.

4.5 Ground Shaking

The site vicinity is located in an area generally characterized as having high seismicity. Using the USGS Seismic Hazard Tool Website considering the site location, ASCE7-10/NEHRP, and Type D soils, the Peak Ground Acceleration (PGA) is 0.77g. Strong ground shaking at the site should be expected during an earthquake.

4.5.1 Liquefaction

Liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil (sand, gravel, and some silts) during earthquake shaking, causing the soil to experience a partial to complete loss of shear strength. Such a loss of shear strength can result in settlement and/or horizontal movement (lateral spreading) of the soil mass.

One boring was drilled to a depth of 41½ feet at the proposed expansion site to characterize the subsurface materials regarding liquefaction. We performed a liquefaction analysis of each boring in following guidance from Boulanger and Idriss, 2004, using the LiqIT v 4.7 software. Nearby borings by others were considered as well.

For the project site, the PGA expected is 0.77g (ASCE 7-10, Type D soil) and a design moment magnitude from the San Andreas Fault of 7.9. The seismic information, sample depth, blow count, estimated unit weight, fines contents, soil types, and groundwater depth were entered into software. For the soil layers that were susceptible to liquefaction, the liquefaction induced settlement was estimated using the procedures by Ishihara and Yoshimine (1992). Procedures by Iwasaki et al (1978) were used to estimate the probability of liquefaction.

The results of the liquefaction analysis indicate factors of safety against liquefaction above two for all soil layers. Based on the procedures by Iwasaki, the potential for liquefaction is zero. In addition, based on the Seismic Hazards Zones, City and County of San Francisco (California Division of Mines and Geology, 2000), the site is not in a location zoned with historical liquefaction; therefore, the potential for ground settlement due to liquefaction is low.

4.6 Corrosion

Soils corrosivity analysis is important for estimating and mitigating the deterioration of buried ferrous metals and concrete. We performed corrosion testing on a representative sample from boring B-1 at a depth of 2 feet bgs as an indicator of the corrosive properties of the soil. Test results are summarized below in Table 1 and presented in detail in Appendix C.

Table 1 Soil Corrosion Results

Boring No.	Depth (ft.)	pH	Minimum Resistivity (ohm-cm)	Water Soluble Sulfates (ppm)	Water Soluble Chlorides (ppm)	Redox Potential (mV)	Points
B-1	2	8.89	5,360	23.6	8.0	179.00	3

According to ACI 318, a sulfate concentration less than 1,000 parts per million is considered “not applicable” (i.e., no mitigation required). A water soluble chloride content of less than 500 ppm is generally non-corrosive to reinforced concrete.

To evaluate the potential for external corrosion potential on ductile iron pipe from soil, the 10-point system in C105/A21.5 (ANSI/AWWA, 1999) was used. As shown in Table 1 above, a point value based on the values in Table 2 below. The long life of historical unprotected pipe in soil with less than 10 points indicates a non-corrosive environment (AWWA 2005).

The provided preliminary corrosion test results are only an indicator of potential soil corrosivity for the sample tested at the selected depth interval. It is possible that corrosion potential can vary by sample location and depth.

Our scope of services does not include corrosion engineering; therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design any mitigation that may be required.

Table 2 10-Point Soil Corrosion Evaluation System

10-point soil test evaluation for iron pipe	
Soil Characteristics	Points*
Resistivity— Ωcm^\dagger	
<1,500	10
$\geq 1,500$ –1,800	8
>1,800–2,100	5
>2,100–2,500	2
>2,500–3,000	1
>3,000	0
pH	
0–2	5
2–4	3
4–6.5	0
6.5–7.5	0‡
7.5–8.5	0
>8.5	3
Redox potential— mV	
>+100	0
+50 – +100	3.5
0 – +50	4
Negative	5
Sulfides	
Positive	3.5
Trace	2
Negative	0
Moisture	
Poor drainage, continuously wet	2
Fair drainage, generally moist	1
Good drainage, generally dry	0
<p>*10 points: corrosive to iron pipe; protection is indicated. \daggerBased on water-saturated soil box. This method is designed to obtain the lowest and most accurate resistivity reading. \ddaggerIf sulfides are present and low (<100 mV) or negative redox-potential results are obtained, three points should be given for this range.</p>	

5. Recommendations

5.1 Site Preparation and Earthwork

5.1.1 Site Preparation

General site preparation should include the stripping of any surface vegetation including the root zone. Existing utilities less than 5 feet below foundations should be evaluated by GHD geotechnical staff in the field on a case-by-case basis.

5.1.2 Earthwork

5.1.2.1 General Subgrade Preparation

Any soft or loose areas should be stabilized. Stabilization may be accomplished by excavating to firm, native material and replacing with engineered fill or using woven geosynthetics or geogrids. Proof-rolling and final verification of stabilization should be conducted under the observation of a GHD geotechnical representative. Upon completion of subgrade preparation, engineered fill should be placed as described below.

5.1.2.2 Engineered Fill

Engineered fill should consist of a homogenous mixture of soil and rock free of vegetation, organic material, and rubble. Highly plastic or organic soils should not be used for engineered fill but may be placed in landscape areas.

We anticipate that the materials generated from on-site excavations will be suitable for use as engineered fill. Imported material to be used as engineered fill should meet the specifications listed below in Table 3 after compaction.

Table 3 Import Fill Specifications

Direct Shear (ASTM D3080)	Atterberg Limits (ASTM D4318)	Particle Size (ASTM C136 or D422)
30°	PI < 15 LL < 40	100% passing the 6 inch sieve minimum of 85% passing the 2-1/2 inch sieve maximum of 30% passing the #200 sieve

GHD geotechnical staff should observe and approve import fill material in writing prior to the material being brought on site. Engineered fill material should not contain rocks greater than 6 inches in largest dimension. Rocks placed in fill should be surrounded by a well-compacted soil matrix to prevent “nesting” and the creation of voids within the fill.

5.1.2.3 Compaction

Engineered fill should be moisture conditioned as necessary, placed in horizontal loose lifts not exceeding 8 inches in thickness, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557 for fills less than 5 feet in thickness. In areas to receive paved improvements, or fills thicker than 5 feet, fill should be compacted to 95 percent of ASTM D1557. Placement of fill material should be verified by a GHD geotechnical representative on a continuous basis.

5.1.2.4 Trench Backfill

Trench backfill should meet the Engineered Fill specifications detailed above. Trench backfill should be placed in lifts not exceeding 12 inches in thickness and compacted to 90 percent of ASTM D1557 by mechanical means only (no jetting). In areas to receive paved improvements, 95 percent of ASTM D1557 is required. Pipe bedding shall conform to the pipe manufacturer's recommendations.

5.1.3 Temporary Slopes/Shoring

Temporary slopes and shoring should conform to OSHA standards. Shored excavations should be constructed from the top down in cuts not exceeding 5 vertical feet in depth. Excavation of subsequent cuts should not be performed until shoring of the adjacent upper cut has been completed. Protection of workers and adjacent structures, shoring design, and the stability of all temporary slopes should be contractually established as solely the responsibility of the contractor.

Foundation excavations for new structures may be near existing foundations. In order to minimize impacts on the existing facilities during excavation, GHD recommends that trenching be located outside an imaginary 1.5:1 (H:V) plane from the base of the existing foundation in firm native undisturbed soil. In the event that this recommendation is not practical, the designer shall incorporate trench shoring or structural improvements such as sheet piling to protect the existing adjacent foundations. Trench support shall be designed by a Professional Engineer registered in the State of California and shall consider adjacent surcharge.

5.2 Foundations – Helical Piles

It is our understanding that Building 203 is supported on deep foundations. In order to maintain a similar settlement potential, we recommend the expansion be supported by a deep foundation as well. Provided within this section are recommendations for bearing capacity of helical pile deep foundations. The bearing stratum consists of dense sand and the top helix should be at least 15 feet below the existing slab. The piles should have a round shaft with a wall thickness of at least 0.30 inches. Helical piles should be manufactured by Chance Civil Construction or equivalent.

Select configurations and capacities are presented below in Table 4.

Table 4 Helical Pile Capacities

Individual Helix Configuration	Shaft Diameter (in)	Depth to Top Helix (ft)	Allowable Vertical Capacity (kips)	Installation Torque Range (ft-lbs)
10/12/14/14	3.5	15	82	10,000
12/14/14/14	3.5	15	88	10,000

To avoid a capacity reduction due to group effects, individual piles should be spaced at least three times the diameter of the largest helix, or a minimum of 42 inches for the configurations above. If closer pile spacing is necessary, GHD should be contacted for appropriate group reduction values.

The installation torque of each pile should be monitored and recorded during installation by a GHD geotechnical representative to confirm the minimum torque to achieve design capacity is reached and that the maximum torque rating is not exceeded. The helical piles should be designed and installed in accordance with the manufacturer's specifications and recommendations. Load testing should be performed on each helical pile to confirm that design loadings are supported.

Total settlement should be less than ½ inch and differential settlement across the structure should be less than ½ inch.

5.3 Seismic Design

The seismic design criteria for the site (37.78162°N, 122.50579°W), listed in the table below, were developed in accordance with ASCE 7-10 and 2009 NEHRP based on the subsurface information obtained from our geotechnical investigation.

Table 5 Seismic Design Criteria

Parameter	Recommended Value	Reference (ASCE/SEI 7-10)
Site Class	D	Table 20.3-1
Mapped MCE spectral response at short period (S_s)	1.955 g	Figure 22-1
Mapped MCE spectral response at 1 sec period (S_1)	0.917 g	Figure 22-2
Site coefficient (F_a)	1.0	Table 11.4-1
Site coefficient (F_v)	1.5	Table 11.4-2
MCE spectral response acceleration for short period (S_{MS})	1.955 g	Equation 11.4-1
MCE spectral response acceleration for 1 sec period (S_{M1})	1.375 g	Equation 11.4-2
Design Spectral Acceleration for short period (S_{DS})	1.303 g	Equation 11.4-3
Design Spectral Acceleration for 1 sec period (S_{D1})	0.917 g	Equation 11.4-4

5.4 Surface Drainage and Erosion Control

Drainage around the receiving structure should be constructed in a way such that soils near the foundation do not become saturated. Surfaces within 10 feet of structures should be sloped a minimum of 2 percent to direct water away and prevent ponding. We recommend the surface drainage be designed in accordance with the latest edition of the IBC.

Erosion control measures should be implemented for exposed surfaces, which may be subject to soil erosion during periods of intensive rainfall. In general, all construction surfaces should be graded to drain to prevent water from ponding.

5.5 Plan Review and Construction Observation

Our conclusions and recommendations are contingent upon GHD being retained to review project plans and specifications during the construction document phase to evaluate if they are consistent with our recommendations. They are also contingent upon GHD being retained to provide intermittent observation and appropriate field and laboratory testing during site preparation and grading, excavation, helical pile installation, and fill placement and compaction to evaluate if the subsurface conditions are as anticipated. If the subsurface conditions are observed to be different from those described in this report, we should be notified immediately so that the changed conditions can be evaluated and our recommendations revised, if appropriate. The recommendations in this report are contingent upon our notification and review of changed conditions. These services are performed on an as-requested basis and are in addition to this geotechnical investigation. We cannot provide comment on conditions, situations, or stages of construction that we are not notified to observe.

6. References

- Idriss, I. M., and Boulanger, R. W., *Soil Liquefaction During Earthquakes.* Monograph MNO-12,
- Iwasaki T, Tokida K, Tatsuko F, Yasuda S (1978), *A Practical Method For Assessing Soil Liquefaction Potential Based On Case Studies At Various Sites In Japan*, Proc. 2nd. Int. Conf. on Microzonation, San Francisco, Vol.2. Earthquake Engineering Research Institute, Oakland, CA, 2008.
- Ishihara, K., and Yoshimine, M., 1992. *Evaluation of settlements in sand deposits following liquefaction during earthquakes*, *Soils and Foundations* 32(1), 173–88.
- California Division of Mines and Geology (2000), *Seismic Hazards Zones, City and County of San Francisco*,
- California Department of Transportation (Caltrans). 2003. *Corrosion Guidelines*.
- Boulanger and Idriss, 2004, *Evaluating the Potential For Liquefaction Or Cyclic Failure Of Silts And Clays*
- U.S. Geologic Survey, June 12, 2014, *U.S. Seismic Design Maps*,
<http://earthquake.usgs.gov/designmaps/us/application.php>.
- Chance Civil Construction, *Design, Installation and Testing of Helical Piles & Anchors*
- USGS Scientific Investigations Map 2918, *Geologic Map of San Francisco County, CA* .KMZ
- INTERNATIONAL CODE COUNCIL, INC. May 30, 2014, *2015 International Building Code*

7. Limitations

This Geotechnical Investigation ("Report"):

- Has been prepared by GHD Inc ("GHD") for POLYTECH ASSOCIATES INC. under the professional supervision of those senior partners and/or senior staff whose seals and signatures appear herein.
- May only be used and relied on by POLYTECH ASSOCIATES INC., which is responsible to ensure that all relevant parties to the project, including designers, contractors, subcontractors, etc., are made aware of this report in its entirety.
- Must not be copied to, used by, or relied on by any person other than POLYTECH ASSOCIATES INC. without the prior written consent of GHD; and
- May only be used for the purpose of engineering design of the proposed structures at the project site described in this report (and must not be used for any other purpose).

GHD and its servants, employees and officers otherwise expressly disclaim responsibility to any person other than POLYTECH ASSOCIATES INC. arising from or in connection with this Report.

To the maximum extent permitted by law, all implied warranties and conditions in relation to the services provided by GHD and the Report are excluded unless they are expressly stated to apply in this Report.

The services undertaken by GHD in connection with preparing this Report:

- In regard to site exploration and testing
 - Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.
- In regard to limitations
 - Our scope of services was limited to the proposed work described in this report, and did not address other items or areas.
 - The geotechnical investigation upon which this report is based was conducted for the proposed structures at the project site described in this report. The conclusions and recommendations contained in this report are not valid for other structures and/or project sites. If the proposed project is modified or relocated, or if the subsurface conditions found during construction differ from those described in this report, GHD should be

provided the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations need revision.

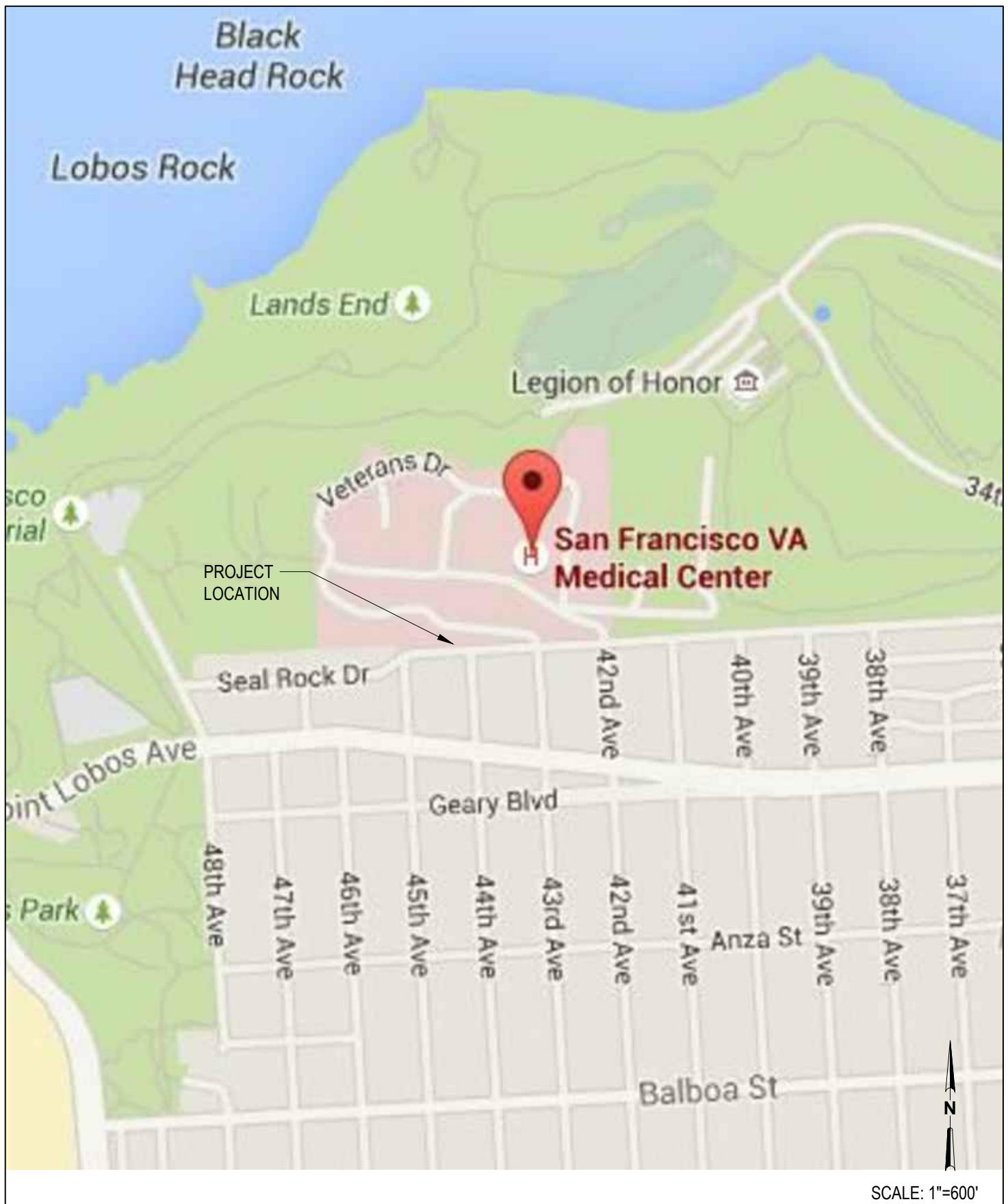
- Did not include evaluation or investigation of the presence or absence of wetlands.
- Did not include a fault investigation.

GHD expressly disclaims responsibility for any error in, or omission from, this Report arising from or in connection with any of the Assumptions being incorrect. There is no warranty, either expressed or implied. GHD accepts no liability regarding completeness or accuracy of the information presented and/or provided to us, or any conclusions and decisions which may be made by the client or others regarding the subject site/project. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observations of construction.

Subject to the paragraphs in this section of the Report, the interpretations of data, findings, conclusions, recommendations and professional opinions in this Report are based on the information reviewed, site conditions encountered, and samples collected during our field exploration and were developed in accordance with generally accepted geotechnical engineering principles and practices and as prescribed by the client. This Report is considered valid for the proposed project for a period of two years from the report date provided that the site conditions and development plans remain unchanged. With the passage of time, changes in the conditions of a property can occur due to natural processes or the works of man on this or adjacent properties. Legislation or the broadening of knowledge may result in changes in applicable standards. Depending on the magnitude of any changes, GHD may require that additional studies (at additional cost) be performed and that an updated report be issued. Additional studies may disclose information which may significantly modify the findings of this report. GHD will retain untested samples collected during our field investigation for a period not to exceed 60 days unless other arrangements are made with the client. After a period of two years from the report date, GHD expressly disclaims responsibility for any error in, or omission from, this Report arising from or in connection with those opinions, conclusions and any recommendations.

Appendix A

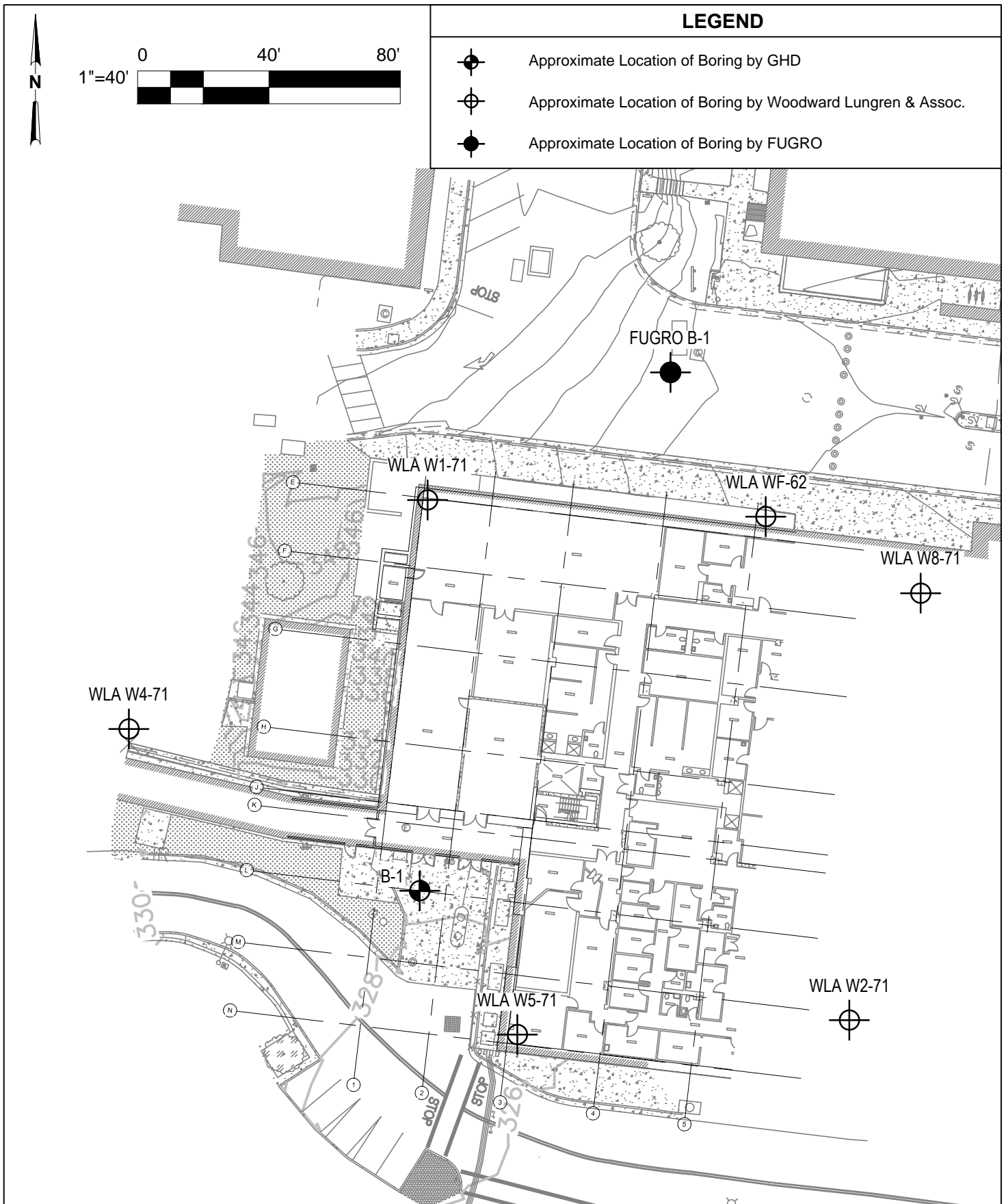
Figures



Vicinity Map

SFVAMC - PICU Expansion & Renovation
San Francisco, CA
Project Number: 11109034

FIGURE
A-1



Boring Location Map **SFVAMC - PICU Expansion & Renovation** San Francisco, CA Project Number: 11109034

FIGURE
A-2

Appendix B

Logs of Borings

EMPIRICAL CORRELATIONS WITH STANDARD PENETRATION RESISTANCE N VALUES*

	N Value * (Blows/ft)	Consistency	Unconfined Compressive Strength (tons/sq ft)		N Value * (Blows/ft)	Relative Density
FINE GRAINED SOIL	0 - 2	Very Soft	<0.25	COARSE GRAINED SOIL	0 - 4	Very Loose
	3 - 4	Soft	0.25 - 0.50		5 - 10	Loose
	5 - 8	Medium Stiff	0.50 - 1.00		11 - 30	Medium Dense
	9 - 15	Stiff	1.00 - 2.00		31 - 50	Dense
	16 - 30	Very Stiff	2.00 - 4.00		>50	Very Dense
	>30	Hard	>4.00			

*ASTM D 1586; number of blows of 140 pound hammer falling 30 inches to drive a 2-inch-O.D., 1.4-inch-I.D. sampler one foot.

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

MAJOR DIVISIONS		SYMBOLS	DESCRIPTIONS
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOIL	CLEAN GRAVELS	GW WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		LITTLE OR NO FINES	GP POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES	GM SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		APPRECIABLE AMOUNT OF FINES	GC CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOIL	CLEAN SANDS	SW WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		LITTLE OR NO FINES	SP POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
FINE GRAINED SOIL	MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SM SILTY SANDS, SAND - SILT MIXTURES
		APPRECIABLE AMOUNT OF FINES	SC CLAYEY SANDS, SAND - CLAY MIXTURES
	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
	SILTS AND CLAYS		OL ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
			MH INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
HIGHLY ORGANIC SOIL	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	CH INORGANIC CLAYS OF HIGH PLASTICITY
			OH ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
			PT PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

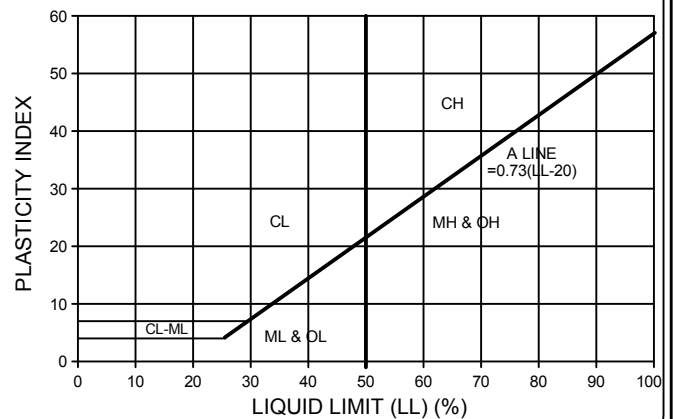
PARTICLE SIZE IDENTIFICATION

BOULDERS	>12 in
COBBLES	3 - 12 in
GRAVEL: COARSE	3/4 - 3 in
GRAVEL: FINE	No.4 - 3/4 in
SAND: COARSE	No.10 - No.4
SAND: MEDIUM	No.40 - No.10
SAND: FINE	No.200 - No.40
SILT	0.002 mm - No.200
CLAY	<0.002 mm

WELL GRADED - HAVING WIDE RANGE OF GRAIN SIZES AND APPRECIABLE AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES

POORLY GRADED - PREDOMINATELY ONE GRAIN SIZE, OR HAVING A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING

PLASTICITY CHART



SAMPLE SYMBOLS

	SPT (1.375 I.D.)
	California (2.0-inch I.D.)
	California Modified (2.5-inch I.D.)
	No Recovery
	Shelby Tube
	Auger Sample

WELL SYMBOLS

	Cement Grout
	Bentonite
	Filter Sand
	Screen in filter sand
	Slough
	RX (Bedrock)

WATER LEVEL SYMBOLS

	Water level at time of drilling.
	Water level measured at a specified time after drilling and sampling or well completion.

GENERAL NOTES

- Soil classifications are based on the Unified Soil Classification System. Soil descriptions and stratum lines are interpretive, and actual changes may be gradual. Field descriptions may have been modified to reflect results of laboratory tests.
- Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations.
- Abbreviations:

CD = TX-CD	NR = No Recovery
CN = Consolidation	PR = Permeability
CR = Corrosivity	RV = R-Value
CU = TX-CU	TC = Cyclic Triaxial
DS = Direct Shear	UC = Unconfined Compression
EI = Expansion Index	UU = TX-UU (quick)
	Z = Ziplock



Soil Boring Key
 VA Medical Center San Francisco-PICU Expansion & Renovation
 San Francisco, CA
 Project Number: 11109034

**FIGURE
B-1**

Start Date: 8/14/15		Finish Date: 8/14/15		Total Depth Drilled (ft bgs): 40.0
Drilling Method: 8-inch Hollow Stem Auger		Drilling Contractor: Taber Drilling		Arbitrary Ground Surface Elevation: 328.30
Drill Rig: CME-55		Hammer Type: Automatic Trip		Hammer Weight / Drop: 140 / 30
Logged By: Kyle Jermstad	Reviewed By: C. Trumbull	Borehole Backfill: spoils, concrete		Groundwater Depth (ft): 30.5 ATD
Remarks:				

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	USCS Classification	Graphic Log	Sample Type	Sample/Run No.	Blows/6"	Uncorrected N Value	Pocket Pen (tsf)	Torvane (tsf)	Water Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index
328		Concrete													
		Brown SAND (SP), fine-grained, medium dense, moist													
	2				MC	1-1A 1-1	8 9 14	23							
326					MC	1-2A 1-2	3 6 13	19							
324	4				MC	1-3A 1-3	5 8 10	18			5	105			
322	6														
320	8														
318	10		SP		MC	1-4A 1-4	4 5 8	13							
316	12														
314	14														
		becomes dense			MC	1-5A 1-5	7 12 18	30			4	108			
312	16				SPT	1-6	6 10 13	23					2		
310	18														
308	20	becomes medium dense			MC	1-7A 1-7	5 8 12	20			14	105			
306	22	Brown clayey SAND (SC), fine-to coarse-grained, medium dense, low plasticity, moist.	SC												
304	24														



Log of Boring B-1

SFVAMC - PICU Expansion & Renovation
San Francisco, CA
Project Number: 11109034

Sheet 1 of 2

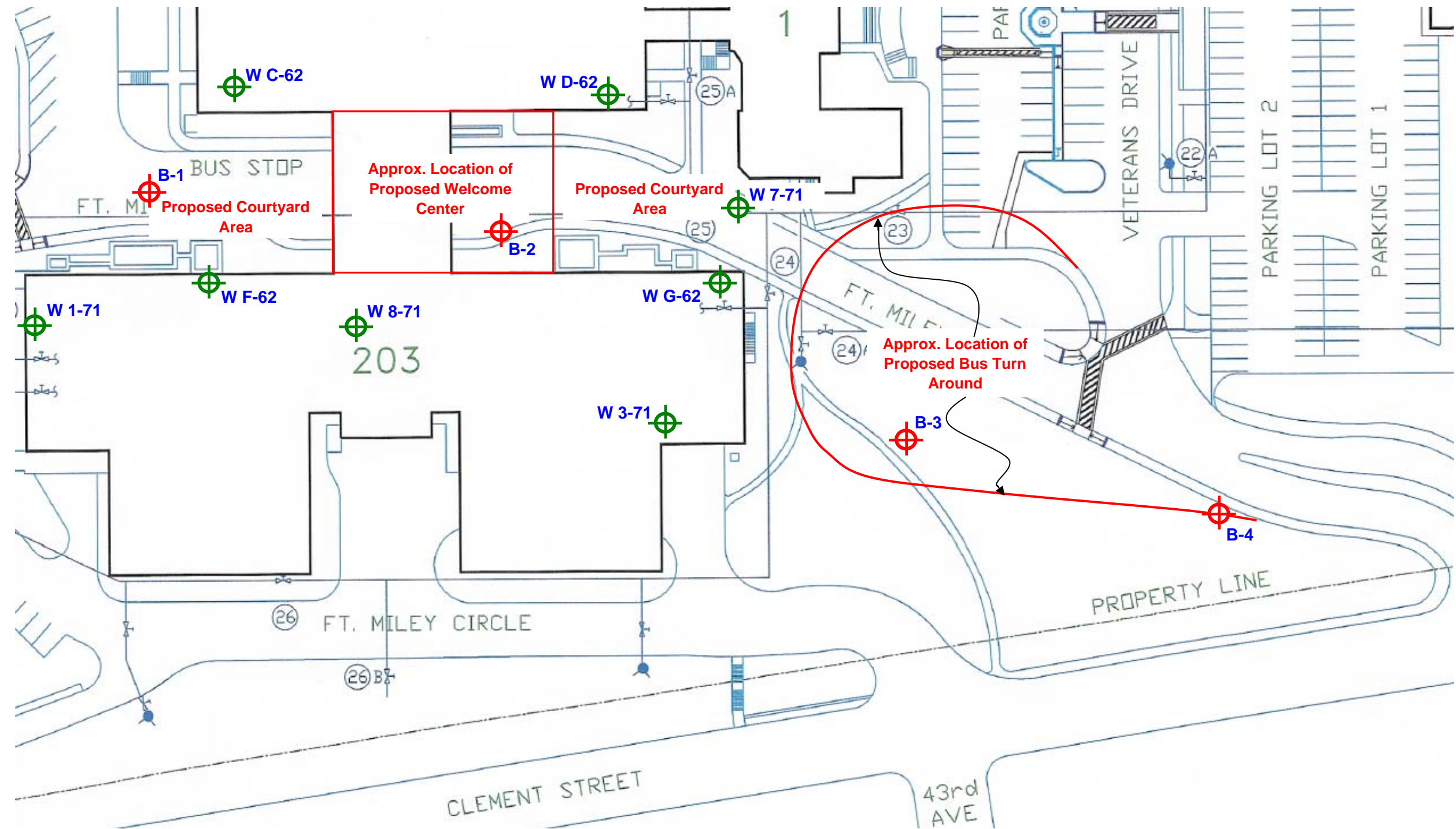
Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	USCS Classification	Graphic Log	Sample Type	Sample/Run No.	Blows/6"	Uncorrected N Value	Pocket Pen (tsf)	Torvane (tsf)	Water Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index
302	26	Brown clayey SAND (SC), fine-to coarse-grained, medium dense, low plasticity, moist.			MC	1-8A 1-8	5 7 10	17	4.0	2.5			43	30	17
300	28														
298	30	with gravel, 3/8" diameter, rounded, very dense, wet.			MC	1-9A 1-9	11 17 33	50	3.5	5.5					
296	32		SC												
294	34	Auger hit isolated rock													
292	36	Becomes gray			SPT	1-10	39 18 43	61	4.5+			28			
290	38	Rig chatter, slow auger advancement Shale (Franciscan Formation), extremely to very weak, completely to highly weathered, gray.													
288	40		RX		SPT	1-11	8 22 15	37	4.5+	3.0					
286	42	Boring terminated at 41.5ft bgs. Groundwater encountered at 30.5ft bgs.													
284	44														
282	46														
280	48														
278	50														
276	52														
274	54														



Log of Boring B-1


SFVAMC - PICU Expansion & Renovation
San Francisco, CA
Project Number: 11109034


Sheet 2 of 2

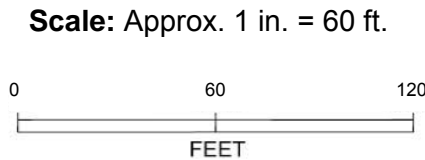


SOURCE: This base map was provided by the Department of Veterans Affairs, dated September 18, 2001.

LEGEND

 **B-1** Approximate Location of Boring by Fugro (2011)

 **W 1-71** Approximate Location of Previous Borings near the proposed improvements by Others.



SITE PLAN
SFVAMC - Welcome Center
San Francisco, California

APPENDIX A FIELD EXPLORATIONS

The field exploration program consisted of a surface reconnaissance and a subsurface exploration program. As a part of the geotechnical exploration for the project, four exploratory borings, designated B-01 through B-04, were conducted on April 12, 2011 to a maximum depth of about 51.5 feet. The borings were drilled with a track-mounted drill rig equipped with hollow stem auger drilling equipment and automatic hammer.

Representative soil samples were obtained from the boring using a Modified California split-barrel drive sampler (outside diameter of 3.0 inches, inside diameter of 2.5 inches) with liners, and a Standard Penetration Test (SPT) split-barrel drive sampler (outside diameter of 2.0 inches, inside diameter of 1.375 inches). All samples were transmitted to our laboratory for evaluation and appropriate testing. The sampler types are indicated in the "Sampler" column of the boring log as designated in Plate A-1.

Resistance blow counts were obtained with the samplers by dropping a 140-pound hammer through a 30-inch free fall using a wire-line safety hammer. The sampler was driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches. Due to the large diameter of the Modified California sampler, the blow counts for this sampler are not standard penetration resistance values. In order to convert these values to approximate standard penetration resistance values, the indicated blow counts should be multiplied by a factor of about 0.6.

Upon completion of our field explorations, the borings were backfilled with neat cement grout. The logs of the borings, as well as a key for the classification of the soil (Plate A-1) and Terms and Definitions Used for Rock (Plate A-2), are included as part of this appendix. The boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

The approximate locations of the exploratory borings are shown on the Site Plan, Plate 2.

CLASSIFICATION AND MATERIAL SYMBOLS

MAJOR DIVISIONS PER ASTM D2488-06			MAJOR GROUP NAMES AND MATERIAL SYMBOLS		
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS	Clean gravels less than 5% fines	GW		Well-Graded GRAVEL
			GP		Poorly Graded GRAVEL
		Gravels with more than 12% fines	GM		SILTY GRAVEL
			GC		CLAYEY GRAVEL
	SANDS	Clean sand less than 5% fines	SW		Well-Graded SAND
			SP		Poorly Graded SAND
		Sands with more than 12% fines	SM		SILTY SAND
			SC		CLAYEY SAND
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid Limit Less than 50%		ML		SILT
			CL		Lean CLAY
			OL		ORGANIC SILT
	SILTS AND CLAYS Liquid Limit Greater than 50%		MH		Elastic SILT
			CH		Fat CLAY
			OH		ORGANIC CLAY
HIGHLY ORGANIC SOILS		PT		Peat or Highly Organic Soils	
Notes: Classification of soils on the boring logs is in general accordance with ASTM D2488, or D2487 if appropriate laboratory data are available. The geologic formation is noted in bold font at the top of interpreted interval on the boring logs.			OTHER MATERIAL SYMBOLS		
				Debris or Mixed Fill	
				Pavement with Aggregate Base	

SAMPLER TYPE

Note: Refer to text of report for additional details or other sampler types.

BLOW COUNT

Number of blows required to drive sampler each of three 6-in. intervals, as measured in the field (uncorrected). An SPT hammer (140 lb., falling 30-in.) was used unless otherwise noted on the boring log. For example:

Blow Count	Description
5 7 8	5, 7, and 8 blows for first, second, and third interval, respectively.
35 50/3"	35 blows for the first interval. 50 blows for the first 3 inches of the second interval. Lack of third value implies that driving was stopped 3 inches into the second interval.
WOH WOH 5	"WOH" indicates that the weight of the hammer was sufficient to advance the sampler over the first two intervals. 5 blows were required to advance the sampler over the third interval.

N-VALUE

The N-Value represents the blowcount for the last 12 inches of the sample drive if three 6-inch intervals were driven. N-value presented is independant of impact energy. If 50 hammer blows were insufficient to drive through either the second or the third interval, the total number of blows and total length driven are reported (excluding the first interval). "ref" (refusal) indicates that 50 blows were insufficient to drive through the first 6-inch interval.

Parenthesis indicate that an approximate correction has been applied for non-SPT drive samplers. For example, a factor of 0.63 is commonly used to adjust blow counts obtained using a 3-inch outside diameter modified California sampler to correspond to Standard Penetration Test.

UNDRAINED SHEAR STRENGTH

A value of undrained shear strength is reported. The value is followed by a letter code indicating the type of test that was performed, as follows:

U - Unconfined Compression
Q - Unconsolidated Undrained Triaxial
T - Torvane
P - Pocket Penetrometer
M - Miniature Vane
F - Field Vane
R - R-value

OTHER TESTS

Field or laboratory tests without a dedicated column on the boring log are reported in the Other Tests column. A letter code is used to indicate the type of test. For certain tests, a value representing the test result is also provided. Typical letter codes are as follows. Additional codes may be used. Refer to the report text and the laboratory testing results for additional information.

k - Permeability (cm/s)
Consol - Consolidation
Gs - Specific Gravity
MA - Particle Size Analysis
EI - Expansion Index
OVM - Organic Vapor Meter

WATER LEVEL SYMBOLS

▽ Initial water level
▼ Final water level
~ Seepage encountered

INCREASING MOISTURE CONTENT

↓ Dry
Moist
Wet

CONSISTENCY OF COHESIVE SOIL







CONSISTENCY	UNDRAINED SHEAR STRENGTH (KIPS PER SQUARE FOOT)
Very Soft	< 0.25
Soft	0.25 to 0.50
Medium Stiff	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	> 4.0

Note: In absence of test data, consistency has been estimated based on manual observation.

APPARENT DENSITY OF COHESIONLESS SOIL

APPARENT DENSITY	N-VALUE
Very Loose	0 to 4
Loose	5 to 9
Medium Dense	10 to 29
Dense	30 to 49
Very Dense	> 49

MATERIAL SYMBOLS

	Sandstone
	Siltstone
	Claystone
	Basalt
	Boulders and Cobbles
	Other (refer to boring log)

Note: Composite or additional symbols may be shown on boring log. Refer to material description on boring log.

GRAIN SIZE FOR CRYSTALLINE IGNEOUS AND METAMORPHIC ROCK

DESCRIPTION	AVERAGE CRYSTAL DIAMETER
Very Coarse Grained or Pegmatic	Greater of equal to 3/8 inch
Coarse-Grained	3/16 to 3/8 inch
Medium-Grained	1/32 to 3/16 inch
Fine-Grained	1/250 to 1/32 inch
Aphanitic	Less than 1/250 inch

WEATHERING OF INTACT ROCK

DESCRIPTION	CHARACTERISTICS
Fresh	No discoloration, not oxidized.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from fractures; some feldspar crystals are dull.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty", feldspar crystals are "cloudy".
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay.

BEDDING SPACING

DESCRIPTION	THICKNESS/SPACING
Massive	Greater than 10 feet.
Very Thickly Bedded	3 to 10 feet.
Thickly Bedded	1 to 3 feet.
Moderately Bedded	4 inches to 1 foot.
Thinly Bedded	1 to 4 inches.
Very Thinly Bedded	1/4 to 1 inch.
Laminated	Less than 1/4 inch.

HARDNESS

DESCRIPTION	CRITERIA
Extremely Hard	Cannot be scratched with a pocket knife or sharp pick. Can only be chipped with repeated heavy hammer blows.
Very Hard	Cannot be scratched with a pocket knife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Can be scratched with a pocket knife or sharp pick with difficulty (heavy pressure). Breaks with heavy hammer blows.
Moderately Hard	Can be scratched with a pocket knife or sharp pick with light or moderate pressure. Core breaks with moderate hammer blows.
Moderately Soft	Can be grooved 1/6-inch deep with a pocket knife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily with a pocket knife or sharp pick with light pressure. Can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Can be readily indented, grooved or gouged with fingernail, or carved with a pocketknife. Breaks with light manual pressure.

FRACTURE DENSITY

DESCRIPTION	OBSERVED FRACTURE DENSITY
Unfractured	No fractures.
Very Slightly Fractured	Core lengths greater than 3 feet.
Slightly Fractured	Core lengths mostly from 1 to 3 feet.
Moderately Fractured	Core lengths mostly 4 inches to 1 foot.
Intensely Fractured	Core lengths mostly from 1 to 4 inches.
Very Intensely Fractured	Mostly chips and fragments.



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	MATERIAL DESCRIPTION	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
							4.5" ASPHALT CONCRETE (AC)							
							10" AGGREGATE BASE (AB)							
							Poorly-graded SAND with SILT (SP-SM): brown, moist, fine to medium, subrounded to subangular fine to coarse gravel							
5			9 13 12	16	18 18"		- medium dense							
			4 7 9	13	18 18"		- little to no gravel	119	6					
10			2 6 4	12	18 18"		Poorly-graded SAND (SP): loose to medium dense, brown, moist, fine to medium grained, trace silt							
			2 6 4	9	18 18"				5	1				
15			2 3 2	5	18 18"									
							SILTY SAND (SM): loose, dark brown, moist, fine grained							
20			1 1 2	3	18 18"		- seepage at 21 ft							
							CLAYEY SAND (SC): loose to medium dense, dark brown mottled strong brown, moist, lenses of SANDY CLAY (CL)							
25			4 8 5	8	18 18"									
			4 14 21	23	18 18"		- olive brown mottled strong brown	123	12	29			1.3 P	
30			9 13 17	20	11 18"									
			5 12 16	28	18 18"		Lean CLAY (CL): stiff, dark greenish gray, moist, medium plasticity, appears to be decomposed bedrock							
35			8 17 23	26	18 18"		FRANCISCAN FORMATION (SHALE AND SHEARED ROCK): decomposed, very soft, dark greenish gray, claylike						3.5 P 3.3 Q	

Continued

BORING DEPTH: 45.4 ft
 BACKFILL: Grout
 DEPTH TO WATER: Seepage at 21'
 FIELDWORK DATE: April 12, 2011
 DRILLING METHOD: 8-in. dia. Hollow Stem Auger

HAMMER TYPE: Automatic Trip
 RIG TYPE: CME 55
 DRILLED BY: Paul Britton
 LOGGED BY: VAC
 CHECKED BY:

LOG OF BORING NO. B-01
 SFVAMC Welcome Center
 San Francisco, California



DEPTH, ft	MATERIAL SYMBOL	SAMPLER TYPE	BLOW COUNT OR PRESSURE, psi	N VALUE OR RQD%	RECOVERY	LOCATION:	DRY UNIT WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX	UNDRAINED SHEAR STRENGTH, S_u ksf	OTHER TESTS
			7 12 15	27	18 18	FRANCISCAN FORMATION (SHALE AND SHEARED ROCK): decomposed, very soft, dark greenish gray, claylike							
45			50/4.5	Ref/4.5	4.5 4.5	- less weathered, moderately soft							
						Boring End @ 45.4'							
						NOTE: Terms and symbols defined on Plates A-1 and A-2.							

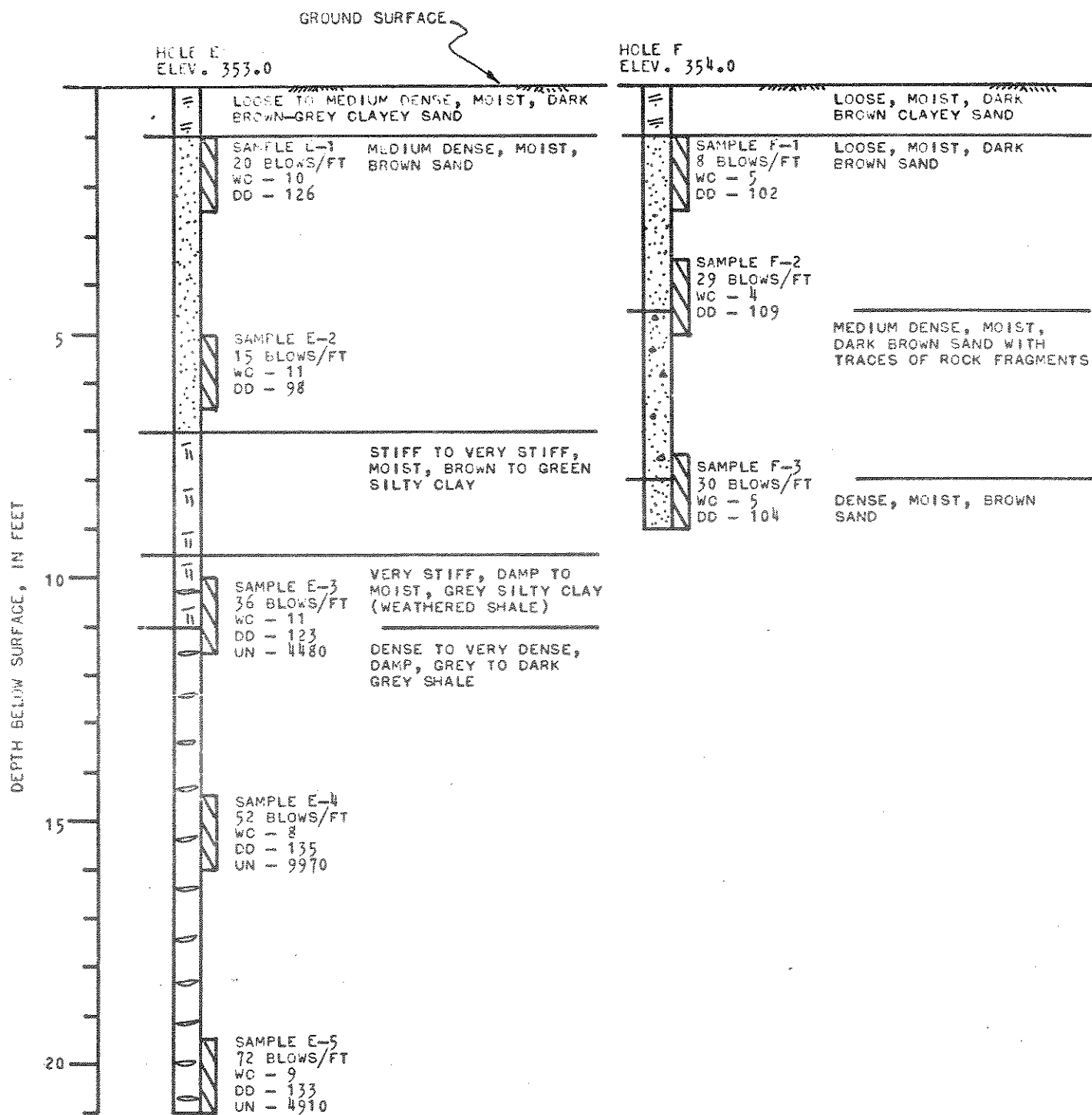


FIG. 11 - LOGS OF BORINGS

Project: V. A. HOSPITAL ADDITION			Log of Boring No. 1		
Date Drilled: <u>12-22-70</u>			Hammer Weight: <u>140 LBS.</u>		
Type of Boring: <u>6" AUGER</u>			Remarks: _____		

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
Surface Elevation: 353						
<div style="display: flex; flex-direction: column; align-items: center;"> <div style="margin-bottom: 20px;">5</div> <div style="margin-bottom: 20px;">10</div> <div style="margin-bottom: 20px;">15</div> <div style="margin-bottom: 20px;">20</div> <div style="margin-bottom: 20px;">25</div> <div style="margin-bottom: 20px;">30</div> </div>	<div style="margin-bottom: 20px;">1</div> <div style="margin-bottom: 20px;">2</div> <div style="margin-bottom: 20px;">3</div> <div style="margin-bottom: 20px;">4</div> <div style="margin-bottom: 20px;">5</div> <div style="margin-bottom: 20px;">6</div>	<div style="margin-bottom: 20px;">4</div> <div style="margin-bottom: 20px;">22</div> <div style="margin-bottom: 20px;">20</div> <div style="margin-bottom: 20px;">26</div> <div style="margin-bottom: 20px;">24</div> <div style="margin-bottom: 20px;">19</div>	<p style="text-align: center; margin: 0;">S A N D (SP)</p> <p style="text-align: center; margin: 0;">LOOSE, MOIST, DARK-BROWN, MEDIUM TO FINE</p> <hr style="border: 0.5px solid black; margin: 10px 0;"/> <p style="text-align: center; margin: 0;">S A N D (SP)</p> <p style="text-align: center; margin: 0;">MEDIUM DENSE, MOIST, BROWN, FINE TO MEDIUM</p> <div style="margin-top: 100px; text-align: center;"> <p style="margin: 0;">BECOMING SILTY</p> </div>	<div style="margin-bottom: 20px;">3</div> <div style="margin-bottom: 20px;">6</div> <div style="margin-bottom: 20px;">3</div> <div style="margin-bottom: 20px;">4</div>	<div style="margin-bottom: 20px;">103</div> <div style="margin-bottom: 20px;">107</div> <div style="margin-bottom: 20px;">103</div> <div style="margin-bottom: 20px;">110</div>	

Job No. S-12302	WOODWARD-LUNDGREN & ASSOCIATES	W1-71
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Project:
V. A. HOSPITAL ADDITION

Log of Boring No. 1

Depth, Ft.	Samples	Blows/Ft.	DESCRIPTION	Moisture Content, %	Dry Density pcf	Unconfined Compressive Strength, psf
CONTINUED:						
35	7	40	SAND (SP) MEDIUM DENSE, MOIST, BROWN, FINE TO MEDIUM, BECOMING SILTY ▽ WATER 12-23-70			
40	8	35	SILTY CLAY (CL) VERY MEDIUM STIFF TO STIFF, WET, BROWN STIFF	18	114	7270
45	9	35	SANDY CLAY (CL-SC) VERY STIFF, MOIST, BROWN ↓ CONSIDERABLE ROCK FRAGMENTS	9	135	4390
50	10	70/3"	SANDSTONE VERY DENSE, GREEN, WEATHERED			
55			SANDSTONE VERY DENSE, GREEN			
60	11	70/3"	SHALE VERY DENSE, BLUE-GREEN			
65			← BOTTOM OF HOLE @ 61'			

1) The material shown on this sheet is reproduced from a Soil Investigation Report dated 5-27-71, prepared by Woodward-Lundgren and Associates. A copy of this report is available for review at the architect's office.

2) It is included for general information only.

NOTES

Project: V. A. HOSPITAL ADDITION		Log of Boring No. 1	
Drawn By: J. L. PEREIRA	Checked By: J. L. PEREIRA	Scale: 1" = 10'	Sheet: 1 of 4
Depth (Feet)	Description	Soil Type	Remarks
0 - 10	SAND (SP)		
10 - 20	LOESS, HEAVY, SUB-CRUSTAL, MEDIUM TO FINE		
20 - 30	SAND (SP)		
30 - 40	MEDIUM GRADE, HEAVY, SAND, FINE TO MEDIUM		
40 - 50			
50 - 60			
60 - 70			
70 - 80			
80 - 90			
90 - 100			
100 - 110			
110 - 120			
120 - 130			
130 - 140			
140 - 150			
150 - 160			
160 - 170			
170 - 180			
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360 - 370			
370 - 380			
380 - 390			
390 - 400			
400 - 410			
410 - 420			
420 - 430			
430 - 440			
440 - 450			
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Appendix C

Laboratory Test Results

LABSUM TEST 11109034 SFVAMC.GPJ 11/5/15

Boring ID	Depth (ft)	Description	Water Content (%)	Dry Density (pcf)	Maximum Size (mm)	%<#200 Sieve	Liquid Limit	Plastic Limit	Plasticity Index	Other Tests
B-1	2.0	Brown SAND (SP)								Corrosion
B-1	6.0	Brown SAND (SP)	4.8	105.3						
B-1	16.0	Brown SAND (SP)	4.0	108.1						
B-1	16.5	Brown SAND (SP)			0.6	2				
B-1	21.0	Brown SAND (SP)	14.4	105.3						
B-1	25.5	Brown clayey SAND (SC)					30	13	17	
B-1	26.0	Brown clayey SAND (SC)			4.75	43				
B-1	35.0	Dark gray clayey SAND (SC)			25	28				



Summary of Laboratory Results

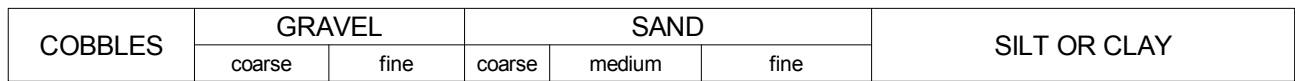
SFVAMC - PICU Expansion & Renovation

San Francisco, CA

Project Number: 11109034

FIGURE

C-1-1

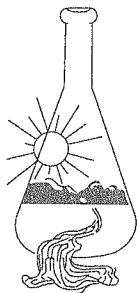
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SFVAMC - PICU Expansion & Renovation

San Francisco, CA

Project Number: 11109034

C-1-1



Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/26/2015

Date Submitted 08/21/2015

To: Bryon Iseger
GHD
4080 Plaza Goldorado Cir. B
Cameron Park, CA 95682

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 11109034-SFVAC Site ID : 1-1 @ 2-2.5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70310-146632.

EVALUATION FOR SOIL CORROSION

Soil pH	8.89		
Moisture	4.0	%	
Minimum Resistivity	5.36	ohm-cm (x1000)	
Chloride	8.0 ppm	00.00080	%
Sulfate	23.6 ppm	00.00236	%
Redox Potential	(+) 179	mv	
Sulfides	Presence - NEGATIVE		

METHODS

pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
Redox Potential ASTM G-200, Sulfides AWWA C105/A25.5

SUNLAND ANALYTICAL LAB
11419 Sunrise Gold Circle, # 10
Rancho Cordova, CA 95742
(916) 852-8557

INVOICE
=====

GHD
4080 Plaza Goldorado Cir. B
Cameron Park, CA 95682

Inv.No. 90310

Date 08/26/2015
Terms: NET 30, 30+ 15%

Customer P.O.#

Requestor: Iseger

* Please indicate Invo.# on remittance

ATTENTION ACCOUNTS PAYABLE

SUN NOS.	SAMPLE LOCATION	ANALYSIS	PRICE
-----	-----	-----	-----
146632	11109034-SFVAC 1-1 @ 2-2.5 FT	CTP.5	203.00
***** Total *****			203.00

GHD Inc

4080 Plaza Goldorado Circle
Suite B
Cameron Park, CA 95682 USA
T: 1 530 677 5515

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