



Converse Consultants

Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

July 14, 2020

Updated September 17, 2020

Mr. Shawn Hanumanthu
Senior Project Manager
Cumming Corporation
523 West 6th Street, Suite 1001
Los Angeles, California 90014

Subject: **UPDATED GEOTECHNICAL LETTER REPORT**
Athletic Field Ramp Rehabilitation Project
La Mirada High School
13520 Adelfa Drive, La Mirada, California 90638
Norwalk-La Mirada Unified School District
Converse Project No. 20-31-234-01
DSA Application No. #03-120869

Dear Mr. Hanumanthu:

In accordance with your request and approval, Converse Consultants (Converse) has updated this geotechnical letter report for the subject project. It is our understanding that the District is planning to rehabilitate the existing ADA ramp with retaining walls on the slope between levels adjacent to the existing football stadium within La Mirada High School.

Based on our field exploration, laboratory testing, geologic evaluation, and geotechnical analysis and review of our previous geotechnical report dated October 24, 2019, which was prepared for the proposed football stadium project within La Mirada High School and understanding of the planned site development, it is our opinion that the proposed rehabilitation project is feasible from a geotechnical standpoint, provided the conclusions and recommendations in this geotechnical letter report are incorporated into the project plans, specifications, and are followed during site construction.

We appreciate the opportunity to be of continued service to Norwalk-La Mirada Unified School District. If you have any questions or require additional information, please call the undersigned at (626) 930-1275.

CONVERSE CONSULTANTS

Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President / Principal Engineer

Dist: 1/Addressee via Email
Encl: Appendix A, Field Exploration
Appendix B, Laboratory Testing Program



1.0 SITE AND PROJECT DESCRIPTION

The proposed project is located at 13520 Adelfa Drive in La Mirada, Los Angeles County, California. The subject site is relatively flat to gently sloping with surface elevation of approximately 198 to 212 feet relative to mean-sea-level (MSL). The site is bounded by Foster Road to the north, Adelfa Drive to the west, and by La Mirada Golf Course to south and east. The site coordinates are: North latitude: 33.90910 degrees, West longitude: -118.00144 degrees. The project site location is presented in Drawing No. 1, *Site Location Map*.

We understand that the proposed project entails rehabilitation of existing ADA ramp and other related improvements at the football stadium within La Mirada High School as shown on Drawing No. 2, *Boring Location Map*. The structural loads are anticipated to be low to moderate.

2.0 FOUNDATION RECOMMENDATIONS

2.1 Shallow Foundations

2.1.1 Vertical Capacity

The proposed improvement can be supported by conventional shallow footing. Conventional spread footings founded on compacted fill soils may be designed for a net bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live-loads. The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.

Based on the review of the as built plans, we estimate that existing foundation for the retaining wall has a net bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live-loads.

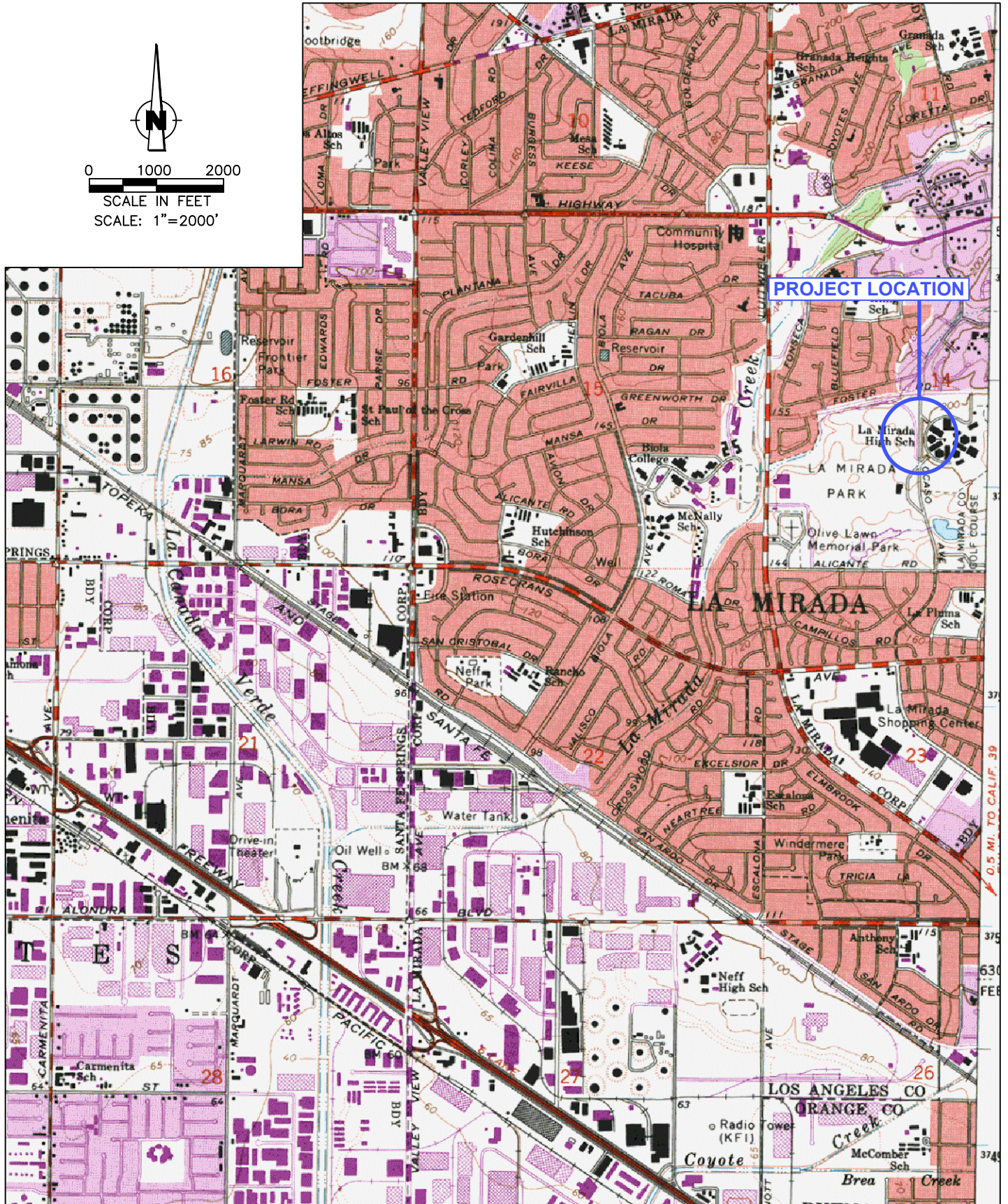
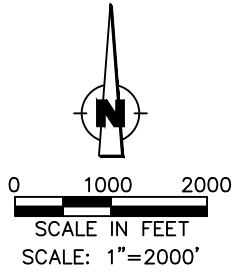
2.1.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.3 may be assumed with normal dead load forces. An allowable passive earth pressure of 150 psf per foot of depth up to a maximum of 2,000 psf may be used for footings poured against properly compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

2.1.3 Settlement

The static settlement of structures supported on continuous and/or spread footings founded on compacted fill and native soil will depend on the actual footing dimensions





SITE LOCATION MAP



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La Mirada High School New Football Stadium Access Ramp Project
 13520 Adelfa Drive
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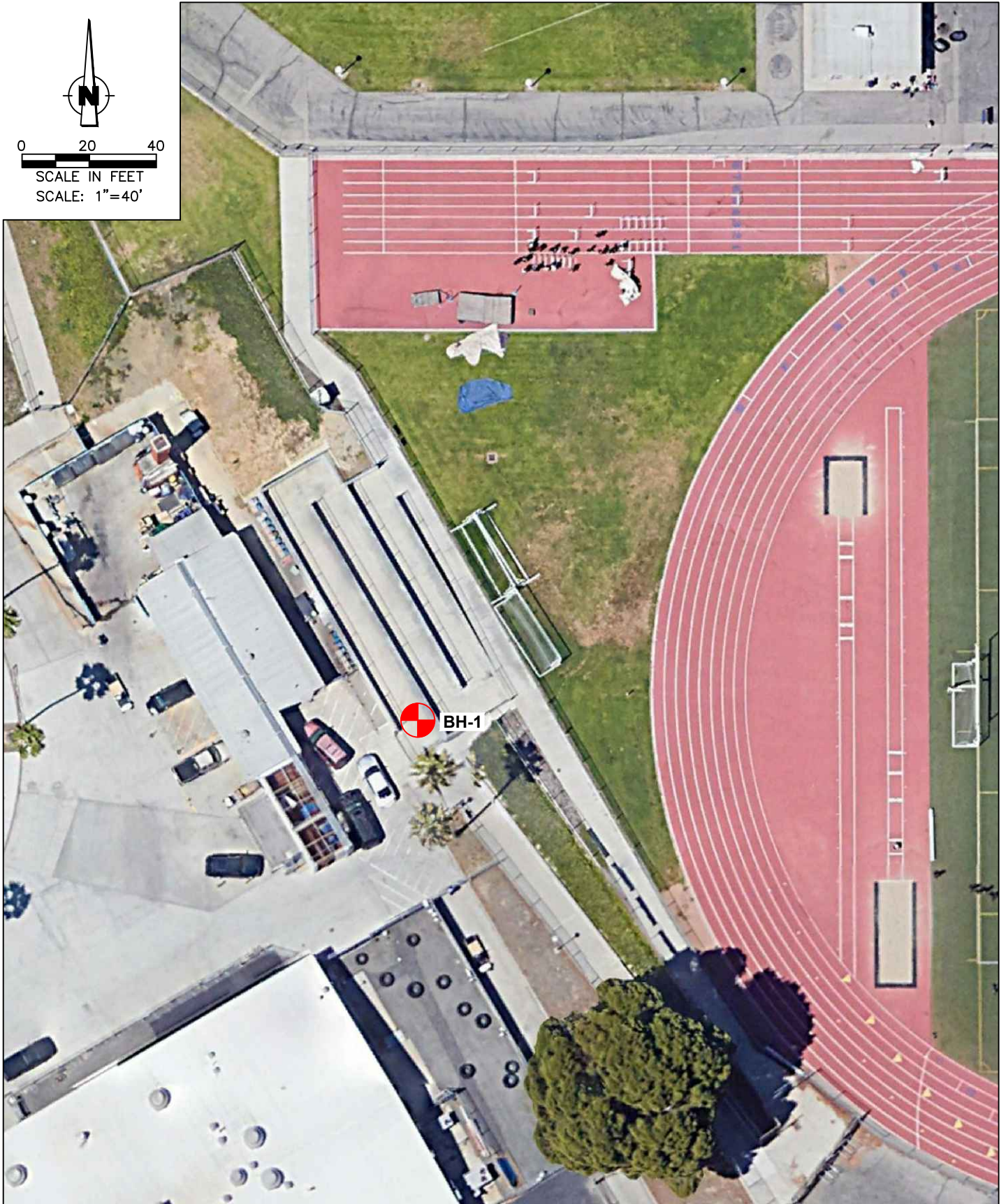
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DRAWING NO.
 1



0 20 40

SCALE IN FEET
SCALE: 1"=40'



BORING LOCATION MAP



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DRAWING NO.
2

and the imposed vertical loads. Based on the maximum allowable net bearing pressures, static settlement is anticipated to be less than 1.0 inch. Differential settlement is expected to be up to one-half (1/2) of the total settlement over a 30-foot span.

2.1.4 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by thirty-three percent (33%) for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by thirty-three percent (33%) for lateral loading due to wind or seismic forces.

2.2 Modulus of Subgrade Reaction

For the subject project, design of the structures supported on compacted fill subgrade prepared in accordance with the recommendations provided in this report may be based on a soil modulus of subgrade reaction of (k_s) of 125 pounds per square inch per inch.

2.3 Lateral Earth Pressure

The following provisional design values may be used for retaining walls design. The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of soil behind the wall, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following earth pressures are recommended for vertical walls with no hydrostatic pressure.

Table No. 1, Lateral Earth Pressures

Backfill Slope (H:V)	Cantilever Wall Equivalent Fluid Pressure (psf)	Restrained Wall Equivalent Fluid Pressure (psf)
Level	40 (triangular pressure distribution)	60 (triangular pressure distribution)

The recommended lateral pressures assume that the walls are fully back-drained to prevent build-up of hydrostatic pressure. Weep holes or equivalent drainage system can be used to prevent build-up of hydrostatic pressure. Additionally, adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. Waterproofing membranes should be added to the subterranean wall levels for moisture sensitive areas to mitigate moisture migration through the walls. The 2-inch diameter weep holes at 6 feet on center and permeable drainage materials wrapped in filter fabric installed behind walls are adequate for drainage behind walls. During construction, representative of geotechnical engineer will verify the drainage construction behind the wall.

Cantilever retaining walls greater than 6 feet, as measured from the surface, should be designed to resist additional earth pressure caused by seismic ground shaking. A



corrosivity study is desired by the design team, a corrosion engineer can be consulted for appropriate mitigation procedures and construction design.

3.0 CONSTRUCTION CONSIDERATIONS

3.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Temporary sloped excavation is feasible if performed in accordance with the slope ratios provided in Section 3.2, *Temporary Excavations*. Existing utilities should be accurately located and either protected or removed as required. For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor, as necessary, to protect the workers in the excavation.

3.2 Temporary Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations (if necessary) may be constructed according to the slope ratios presented in Table No. 2, *Slope Ratios for Temporary Excavations*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand may have to be constructed at a flatter gradient than presented in the following table:

Table No. 3, Slope Ratios for Temporary Excavations

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	vertical
4 – 8	1:1
8+	1.5:1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within 5 feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of 6 feet of stockpiled soils placed at least 5 feet from the trench edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.



3.3 Slot Cut Recommendations

Temporary excavations during possible improvements should not extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of the existing utility lines or structures. The remedial grading excavations should not cause loss of bearing and/or lateral support for adjacent utilities or structures.

If remedial grading excavations extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of adjacent off-site utility lines or structure foundations, shoring or slot cutting shall be employed. "A-B-C" slot cuts exposing native sandy soils may be excavated with maximum 8 feet wide and 8 feet height sections to prevent the existing utility lines or off-site structures from becoming unstable. Backfill should be accomplished in the shortest period of time possible and in alternating sections.

The ABC slot cutting method could be a possible option as an alternative to shoring for excavation less than 8 feet or with cohesive soils. In general, for structures it is not recommended for slot cutting if the height of excavation exceeds more than 8 feet or into sandy soils and with surcharging load.

3.4 Geotechnical Services During Construction

This letter has been prepared to aid in the site preparation and site grading plans and specifications, and to assist the architect, civil and structural engineers in the design of the proposed structure. It is recommended that Converse be provided an opportunity to review final design drawings and specifications to verify that the recommendations of this report have been properly implemented.

Recommendations presented herein are based upon the assumption that adequate earthwork monitoring will be provided by the geotechnical engineer of record. Excavation bottoms should be observed by a geotechnical engineer or his/her representative prior to the placement of compacted fill. Structural fill and backfill should be placed and compacted during continuous observation and testing. Footing excavations should be observed prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.



Appendix A

Field Exploration



APPENDIX A: FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface conditions were noted, and the approximate locations of the borings were determined. The exploratory borings were approximately located using existing boundary and other features as a guide and should be considered accurate only to the degree implied by the method used. The various field study methods performed are discussed below.

Exploratory Borings

One (1) exploratory boring (BH-1) was drilled within the project site on June 30th, 2020. Boring was advanced using hand augering with a 4-inch diameter auger to depth of 21.5 feet below the existing ground surface (bgs). Boring was visually logged by a Converse Geologist and sampled at regular intervals and at changes in subsurface soils. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

Ring samples of the subsurface materials were obtained at frequent intervals in the exploratory borings using a drive sampler (2.4-inches inside diameter and 3.0-inches outside diameter) lined with sample rings. Samples are retained in brass rings (2.4-inches inside diameter and 1.0-inch in height). The central portion of the samples were retained and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk sample of typical soil types was also obtained.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. Changes in material conditions that occur between driven samples are indicated in the logs at the top of the next drive sample. A key to soil symbols and terms is presented as Figure No. A-1, *Soil Classification Chart*. The log of the exploratory boring is presented in Figure No. A-2, *Log of Borings*.



SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SAMPLE TYPE

	STANDARD PENETRATION TEST Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
	DRIVE SAMPLE 2.42" I.D. sampler.
	DRIVE SAMPLE No recovery
	BULK SAMPLE
	GRAB SAMPLE
	GROUNDWATER WHILE DRILLING
	GROUNDWATER AFTER DRILLING

BORING LOG SYMBOLS

LABORATORY TESTING ABBREVIATIONS		
TEST TYPE		
(Results shown in Appendix B)		
CLASSIFICATION		
Plasticity	pi	
Grain Size Analysis	ma	
Passing No. 200 Sieve	wa	
Sand Equivalent	se	
Expansion Index	ei	
Compaction Curve	max	
Hydrometer	h	
STRENGTH		
Pocket Penetrometer		p
Direct Shear		ds
Direct Shear (single point)		ds*
Unconfined Compression		uc
Triaxial Compression		tx
Vane Shear		vs
Consolidation		c
Collapse Test		col
Resistance (R) Value		r
Chemical Analysis		ca
Electrical Resistivity		er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS



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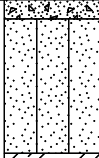


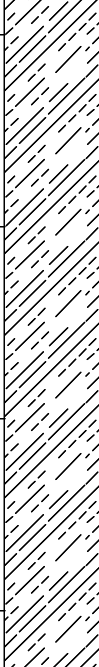








Figure No.
 A-1

Log of Boring No. BH-1

Dates Drilled: 6/30/2020 Logged by: PAF Checked By: RAM

Equipment: HAND AUGER Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 208 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS		SAMPLES		BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.		DRIVE	BULK				
		CEMENT CONCRETE RAMP 6.5 INCHES <u>FILL (Af):</u> SILTY SAND (SM): few gravel and clay, light brown.							ma, max ei
5		OLDER SURFICIAL SEDIMENTS (Qoa) claystone/siltstone-very thinly bedded, moist, light gray-brown					14	97	ds
10		CLAYSTONE/SILTSTONE slightly moist, large pieces of claystone, light brown					10	98	
15		becomes stiff, light brown					18	88	
20		light brown to reddish-brown					19	90	
		End of boring at 21.5 feet below ground surface. Groundwater not encountered. Borehole backfilled with grout and patched with quick set concrete on 6/30/20.							



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Figure No.
 A-2

Appendix B

Laboratory Testing Program



APPENDIX B: LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the *in-situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on one (1) selected sample. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curve is shown in Drawing No. B-1, *Grain Size Distribution Results*.

Maximum Dry Density Test

One (1) laboratory maximum dry density-moisture content relationship test was performed on a representative bulk sample of the upper 5 feet of soil material. The testing was conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Drawing No. B-2, *Moisture-Density Relationship Results*.

Direct Shear

A Direct shear test was performed on one (1) relatively undisturbed sample at soaked moisture conditions. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.01 inch/minute. Shear deformation was recorded until a maximum of about 0.50-inch shear displacement was achieved. Peak strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing No. B-3, *Direct Shear Test Results*, and the following table:



Table No. B-1, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-1	5	Claystone/Siltstone	28	90

Expansion Index Test

One (1) representative bulk sample was tested to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with ASTM D4829 Standard. Test results are presented in the following table:

Table No. B-2, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-1	1-5	Silty Sand with Clay and Gravel	8	Low

Soil Corrosivity

One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and soluble sulfate. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL in Arcadia, California. The test results received from EGL are included in the following table:

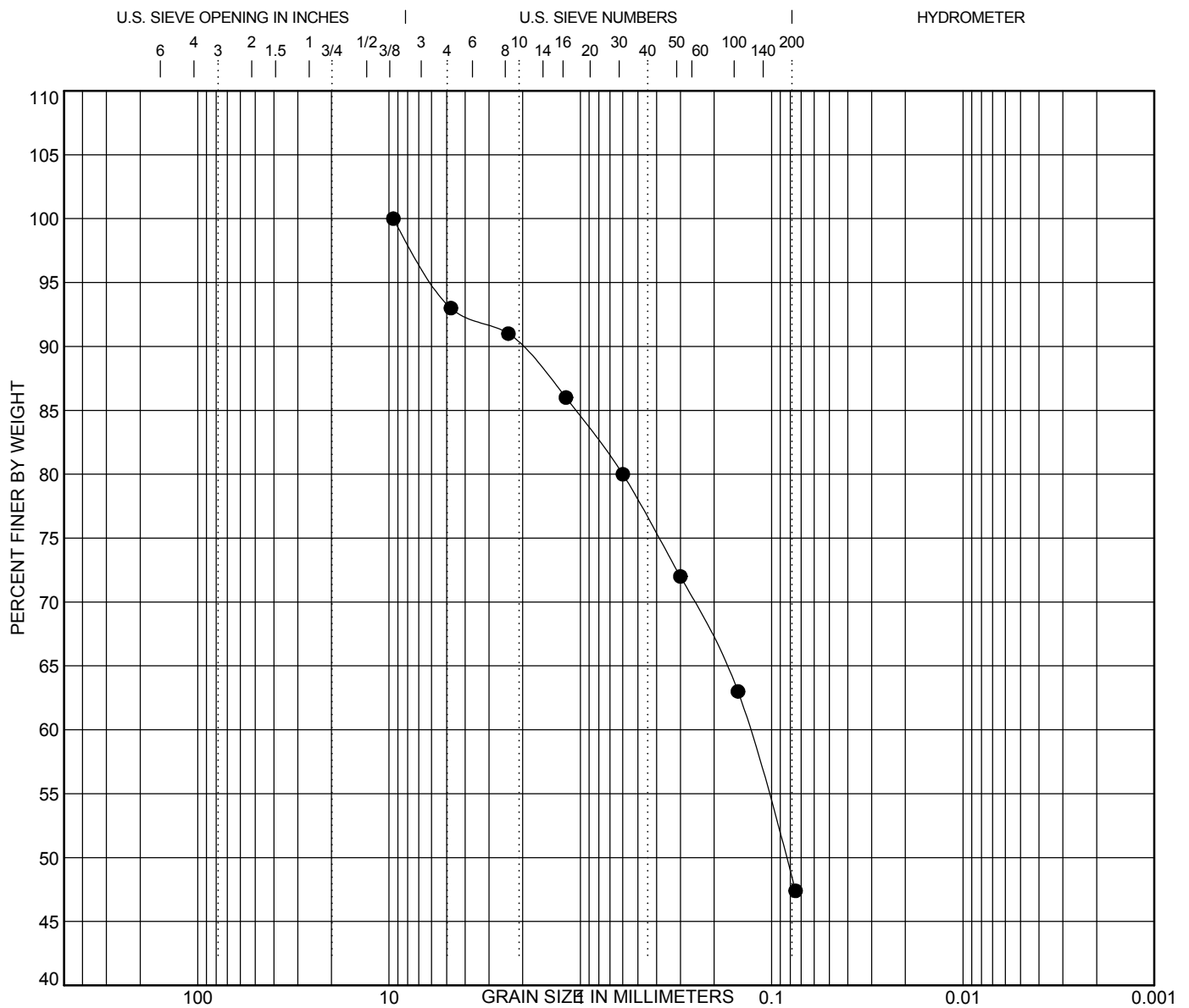
Table No. B-3, Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (% by weight)	Saturated Resistivity (Caltrans 643) Ohm-cm
BH-1	0-5	8.44	150	0.007	1,200

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period of time.





COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring No.	Depth (ft)	Description					LL	PL	PI	Cc	Cu
● BH-1	0 - 5	Silty Sand (SM) with Gravel									
Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay		
● BH-1	0	9.5	0.131			7.0	45.6	47.4			

GRAIN SIZE DISTRIBUTION RESULTS

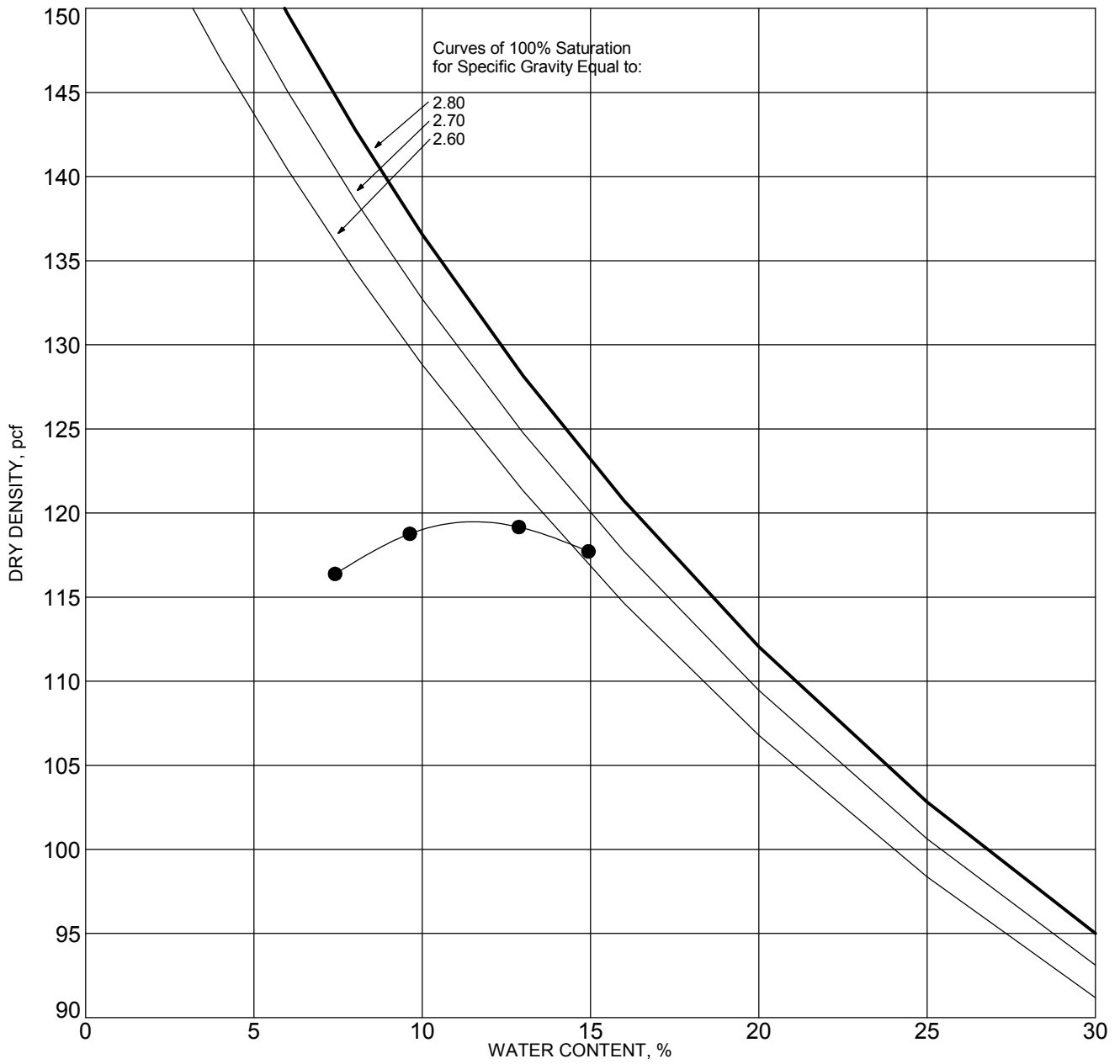


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Figure No.
 B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-1	0 - 5	Silty Sand (SM)	D1557 Method A	11.5	119.5

NOTE:

MOISTURE-DENSITY RELATIONSHIP RESULTS

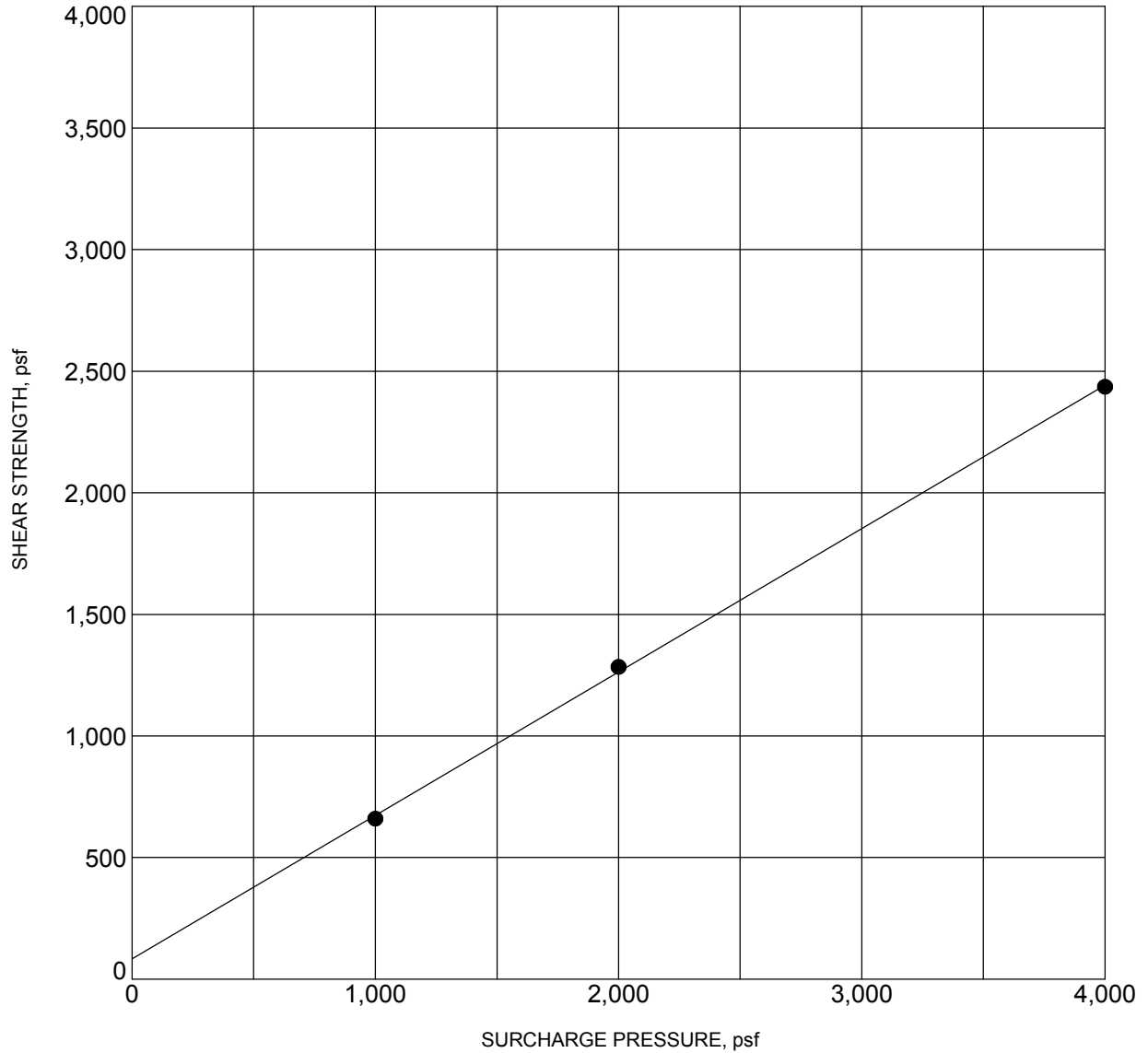


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Figure No.
 B-2



BORING NO. :	BH-1	DEPTH (ft) :	5
DESCRIPTION :	Claystone/ Siltstone		
COHESION (psf) :	90	FRICTION ANGLE (degrees):	28
MOISTURE CONTENT (%) :	14.3	DRY DENSITY (pcf) :	97.0

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS



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Figure No.
 B-3