Attachment "A"

Harrington Geotechnical Engineering, Inc.

June 1, 2021 (Revised)

Mr. Marcos Rodriguez
Director of Construction
ROWLAND UNIFIED SCHOOL DISTRICT
1018 South Otterbein Street
Rowland Heights, CA 91748

Subject: Geotechnical Investigation for Design and Construction of New Auto Shop Storage Building (M-10) Rowland High School, 2000 Otterbein Avenue, Rowland Heights, CA

HGEI Project No. 21-01-4178

Dear Mr. Rodriguez:

This report presents the results of a geotechnical investigation performed at your request to establish information on the materials underlying the referenced site and, based thereon, to provide recommendations for design and construction of the new auto shop storage building.

Available preliminary plans and information were used in outlining the scope of the investigation which was conducted in accordance with generally accepted geotechnical engineering practice in this area.

Based on analysis and evaluation of the data obtained it has been concluded that the indicated construction is feasible from a geotechnical standpoint provided the recommendations presented herein are incorporated in the design and construction of the project.

Thank you for this opportunity to be of service again. If you have any questions concerning this report, or if we can be of further assistance, please call at your convenience.

Very truly yours,
HARRINGTON GEOTECHNICAL ENGINEERING, INC.

Joseph L. Welch, P.E. , G.E.
Senior Geotechnical Engineer
JLW:mvp

REGISTERED PROFESSIONAL ENGINEER
JOSEPH L. WELCH
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EXP. 9/30/22
GEOTECHNICAL
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Please visit our website at www.harringtongeotechnical.com

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INTRODUCTION

This report presents the results of a geotechnical investigation conducted at the subject site. The purposes of the investigation were to: 1) determine the types and condition of the soil/bedrock beneath the area; 2) establish static physical and limited chemical properties of the materials; 3) determine groundwater conditions; 4) provide recommendations for designing the new storage building foundation.

SCOPE OF WORK

The scope of work for this preliminary geotechnical investigation consisted of the following:

Field exploration consisting of drilling, sampling and logging was conducted on April 21, 2021 as described in detail in Appendix A.

Selected samples were tested in HEGI's AMRL Accredited Geotechnical Laboratory to develop data necessary for analysis of subsurface conditions and used in the preparation of this report. The laboratory program for this project included: moisture, density, compaction, expansion, shear, consolidation and corrosivity tests. A description of the geotechnical laboratory testing conducted on the samples collected from the site and presentation of the results is found in the Laboratory Procedures & Test Results in Appendix B.

HGEI conducted engineering analysis, constructed figures, and prepared this report depicting the findings and conclusions of the investigation.

SITE LOCATION AND CONDITION

The campus is located at 2000 Otterbein Avenue in Rowland Heights, CA as shown on the Vicinity Map, Figure 1. The proposed auto shop storage building site occupies a portion of a level area in the northwest corner of site adjacent to the existing Building M. The site currently has a reinforced masonry wall on its north and west sides. The intention is to incorporate these walls as part of the building. The structure will have a steel frame that will support the roof.

PROJECT DESCRIPTION

The New Auto Shop Storage Building will be single story structure of steel frame construction. The intention is to incorporate the existing retaining walls on the north and west sides of the proposed storage building.

Foundation loads are not presently available, but are not expected to exceed 2000 pounds per lineal foot.

The subject construction area is shown in Figure 2.

Material Types and Condition

The site is primarily underlain with alluvial deposits, which consists of sand clay, clayey sand, and silty sands interbedded. Refer to the boring logs in Appendix A for detailed descriptions of the material along with unit weights and moisture contents.

Expansion Potential

The surface soil is expansive (E.I. = 30) The 2019 California Building Code (Section 1803A.5.3) categorizes this material as being expansive and special design is required per Section 1808A.6.

Corrosivity

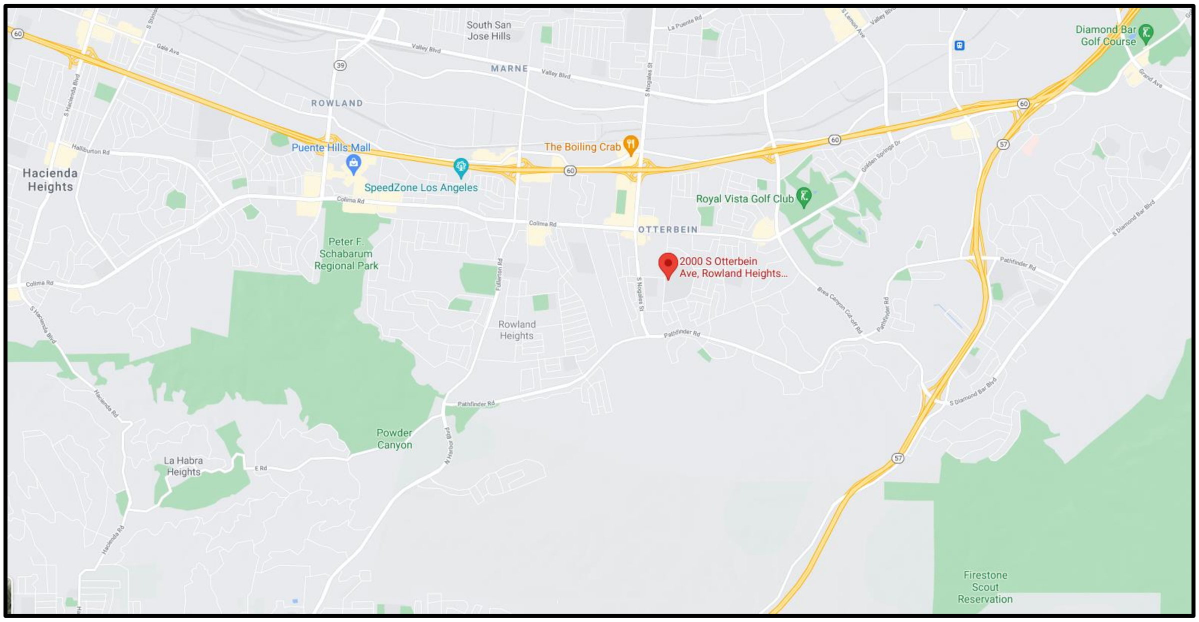
One sample was submitted to a state approved analytical laboratory for corrosivity testing. The results are presented in Table 3 in Appendix A. Harrington Geotechnical Engineering does not practice corrosion engineering and we recommend that a competent corrosion engineer be retained to review the result and recommend any mitigation methods necessary and/or recommend further testing.

These results are only an indicator of soil corrosivity for the sample tested. Other soil found on the site may be more, less, or of a similar corrosive nature. Any imported fill material should also be tested to determine its corrosion potential prior to being accepted for delivery to the site.

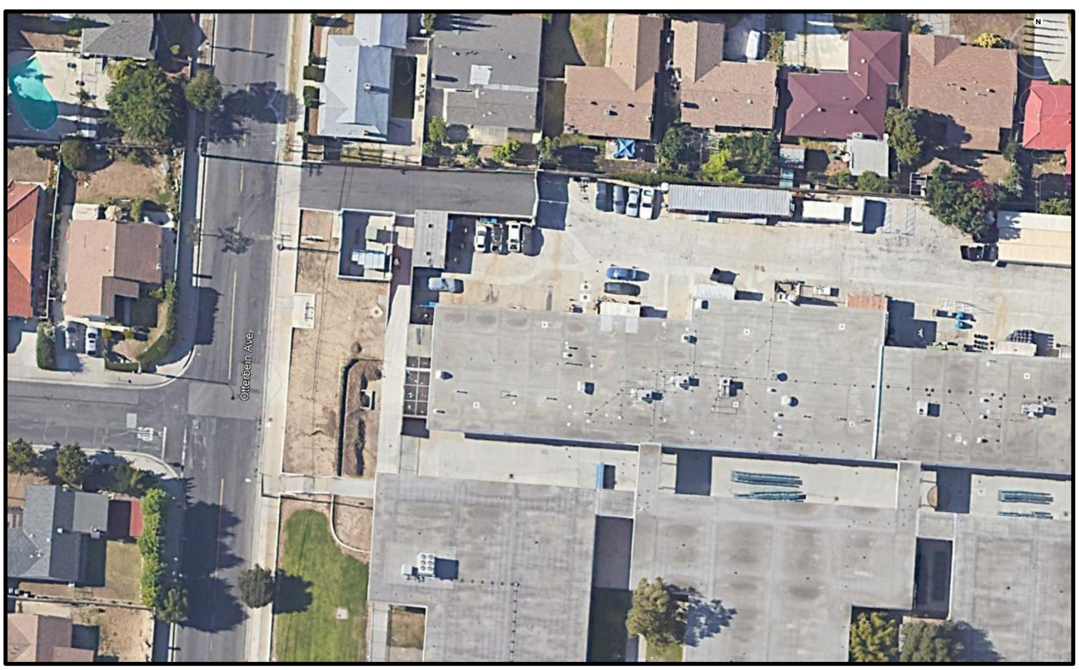
Groundwater Conditions

Groundwater was encountered in the 50-foot-deep borings at a depth of 35 feet. This site is in a mapped liquefaction area with the highest historical groundwater depth at 25 feet. A Liquefaction/dry sand settlement was performed and the results are reported in Appendix D and the Liquefaction Section that follows.

Vicinity Map- Figure 1



Approximate Project Area- Figure 2



Caving
Caving is not expected to be of significant concern during grading and/or construction.

Water-Soluble Sulfate
A negligible amount of water- soluble sulfate was detected in the sample tested.

Liquefaction/Seismically Induced Settlement and Lateral Spreading
The site is located in a potential liquefaction hazard zone as shown on the State of California Earthquake Zones of Required Investigation, La Habra Quadrangle Sheet.

Therefore, a liquefaction/dry sand settlement assessment was conducted using the EQLiquefy & Settle "2" program. Penetration tests and sieve analysis were performed to develop input data for a liquefaction/dry sand settlement analysis.

The analysis indicates a dry sand settlement of 1.67 inches at B-1 and 1.28 inches at B-2. There was no liquefaction settlement under seasonally high groundwater conditions. This is less than the generally accepted allowed settlement of 4 inches for static and seismic. The results of the analyses are presented in Appendix D.

GEOLOGIC HAZARDS AND SEISMIC DESIGN PARAMETERS
Geologic Hazards and Seismic Design Parameters are addressed in the Geologic Hazards Report by Terra Geosciences in Appendix C.

CONCLUSION
Construction of the new auto shop storage building foundation, as presently proposed is considered feasible from a geotechnical engineering standpoint provided the following recommendations are incorporated in the design and construction of the project.

DISCUSSION AND RECOMMENDATIONS
Based on analyses of laboratory test data and evaluation of conditions encountered in the exploratory borings, the following recommendations for design and construction of the building footings and floor slab are being provided subject to confirmation of anticipated conditions during construction.

It is recommended that plans and details be submitted to this office for geotechnical review for compliance with this report. Additional recommendations may be provided based on the review and/or in the course of work if warranted by conditions encountered.

Grading

It is recommended that grading be carried out in accordance with applicable sections of the Grading Specifications in Appendix E and the following site specific recommendations.

There will only be minor changes in grade involved in preparation of the building pad.

Considerable ground disturbance will result from clearing the site of structures, pavement, underground utilities, vegetation, etc. In order to develop increased, uniform support for the new buildings, the following grading procedures are recommended. Some modification may be recommended during the course of work, based on actual conditions encountered.

Prior to grading all vegetation and debris resulting from demolition of existing above-and below-grade structures/utilities should be disposed of off-site in an acceptable manner.

In order to develop increased, uniform support for the buildings and minimize post-construction settlement, it is recommended that the soil throughout the proposed building areas be removed and replaced as uniformly compacted fill. The soil in the new building areas should be over-excavated to a minimum depth of 3 feet deep; the exposed soil should be scarified to a depth of 12 inches, moisture conditioned by aeration or the addition of water as required to 2-3 % above the optimum moisture content, and compacted to a minimum relative compaction of 90% based on the results of compaction tests performed in accordance with ASTM Test Method D1557-12ε1.

Any unsuitable material encountered should be properly disposed of and not incorporated into new fill. The replacement fill material should be placed and spread in thin, loose lifts, and moisture conditioned and compacted as indicated above. Additional compaction tests should be performed as necessary for proper control during grading and to confirm the data in Table 1 of Appendix B.

Imported soil shall be approved by the geotechnical engineer for expansion, sulfate, and strength qualities prior to being transported to the project site. Final acceptance of any imported soil will be based on observation and/or testing of soil actually delivered to the site.

It is recommended that grading operations be monitored by a representative of this office in order to confirm compliance with these recommendations and, in turn, the foundation design recommendations which follow.

Grading should generally be performed in accordance with the Earthwork Specifications in Appendix E. Additional and/or revised recommendations may be presented when the grading plan in submitted for geotechnical review.

Building Foundation Design

An allowable net, dead load+live load bearing pressure of 2500 pounds per square foot is recommended for designing continuous perimeter footings with a minimum plan dimension of 15 inches and depth of 18 inches below the lowest adjacent finished grade. For individual pad footings an allowable net, dead load+live load bearing pressure of 2500 pounds per square foot is also recommended for pad footings with a with minimum width and length of 2 feet and depth of 18 inches below the lowest adjacent finished grade. The pad footing should have its bottom below a 1:1 line projected up from the intersection of the back of the retaining wall and top of the retaining wall footing.

The bearing pressure may be increased by one-third when designing for short duration wind or seismic loads in combination with vertical loads.

A passive bearing pressure of 250 pounds per square foot per foot of embedment below the lowest adjacent finished grade and a friction coefficient of 0.4 may be used to determine resistance to lateral loads. These values may be combined without reduction.

Two No. 4 bars, top and bottom, are recommended as minimum reinforcement for continuous footings. Reinforcement of pad footings, if any, will be governed by structural design.

It is recommended that all footing excavations be examined or tested if necessary and accepted by a representative of the geotechnical engineer prior to placement of reinforcement.

Floor Slab Construction

Unless otherwise required by structural design it is recommended that the floor slab be a full 5-inches-thick and reinforced with No. 4 bars spaced 24 inches apart in both directions.

The subgrade should be moistened prior to placement of the 4 inches of clean sand or gravel. The slab subgrade should be pre-soaked to 1.25 x optimum moisture content to a depth of 12 inches prior to placement of concrete. We have provided this recommendation for pre-saturation, which is a stabilization procedure permitted by Section 1808A.6.4, in lieu of designing foundations in accordance with Section 1808A.6.2.

Existing Retaining Wall

The existing retaining wall can be used as the perimeter wall provided the only new load is the masonry necessary to bring the existing wall to the height of the new proposed wall. There cannot be any additional building load surcharging the existing wall without additional structural analysis of the wall. Preliminary plans indicate there will be an additional surcharge load of 125 psf on the new concrete building floor. Therefore a structural analysis of the wall is required. The existing wall has an allowable bearing capacity of 2,500 psf. The active equivalent soil pressure would be 35 psf/lf and the at rest pressure 50 psf/lf. The backfill condition is level.

A passive bearing pressure of 250 pounds per square foot per foot of embedment below the lowest adjacent finished grade and a friction coefficient of 0.4 may be used to determine resistance to lateral loads. These values may be combined without reduction. The passive bearing pressure and friction coefficient can also be increase by one third for temporary loads such as seismic and wind

The 125 psf surcharge will cause an increase in the active equivalent fluid pressure of 34psf/lf starting 2.5 feet below the ground surface at the back of the wall and extending to the top of the footing.

The 2019 California Building Code (Section 1803.5.12) requires that earthquake forces be included in design of the retaining wall should the retained height equal or exceed 6 feet. In this regard, it is recommended that a dynamic increment of active force equal to 25.3 H^2 acting at 0.6 H above the base of the walls be used. The retained height is measured from the bottom of the footing to the finished soil grade at the rear face of the wall.

Seismic Design

Seismic design values are presented in Geologic Hazards Report by Terra Geosciences. (Appendix C to this report.)

Settlement

Settlement of footings designed as recommended should be ½ inch total and ¼ inch differential. The differential settlement is anticipated to occur over a distance of 20 feet horizontally.

Concrete Quality

A negligible amount of water-soluble sulfate was detected in the surface material and special sulfate-resistant concrete will not be required on this project. The exposure class (ACI 318-19 Table 19.3.1.1) is S0. Concrete may contain Type II cement and should comply with Section 1904A of the 2019 CBC and ACI 318-19, Table 19.3.2.1).

Excavations/and Backfills

It is recommended that the footing excavations be observed/tested by a representative of this office in order to confirm anticipated soil conditions and verify the recommendations presented herein. Backfills should consist of site material placed in appropriate lifts, moisture conditioned to 2-3 % over the optimum moisture content and mechanically compacted to at least 90 percent of maximum dry density. The use of sand is not recommended and jetting should not be permitted.

GENERAL COMMENTS

The services provided under the purview of this report have been performed in accordance with generally accepted geotechnical engineering principals and standards of practice for this area.

The comments and recommendations presented are professional opinions based on our best estimation of project conditions and requirements as indicated by presently available information and data. No further warranty, express or implied, is intended by issuance of this report.

Any unanticipated conditions encountered in the course of construction should be brought to the attention of the geotechnical engineer for evaluation prior to proceeding with work in the area.

This report has been prepared for the specific use of the client indicated for specific application to the proposed construction described herein and shall not be applied to other projects unless approved in writing by this office. Also, the report will be valid for a period of one year from date of issue and will then require updating.

0-0-0

REFERENCES

1. California Department of Conservation, Division of Mines and Geology, 1997 (Revised 2005), Seismic Hazard Zone Report for the La Habra 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 03.

2. California Department of Conservation, California Geological Survey, April 15, 1998, State of California, Earthquake Zones of Required Investigation, La Habra 7.5-Minute Quadrangle, Scale 1:24,000.

3. California Department of Conservation, California Geological Survey, Earthquake Zone App, https://maps.conservation.ca.gov/cgs/EQZApp/.

4. USGS, 2004, Morton, D.M., Bovard, Kelly R., and Alvarez, Rachel M., 2004, Preliminary Digital Geologic Map of the Santa Ana 30'x 60' Quadrangle, Southern California, version 2.0: U.S. Geological Survey Open-File Report 99-0172.

5. California Geological Survey, 2019, Checklist for the Review of Engineering Geology and Seismology Reports for Public Schools, Hospitals, and Essential Service Buildings, Note 48, dated November 2019.

6. California Department of Conservation, Division of Mines and Geology, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication, 117.

7. International Code Council (ICC), 2019, California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2

APPENDIX A

FIELD INVESTIGATION

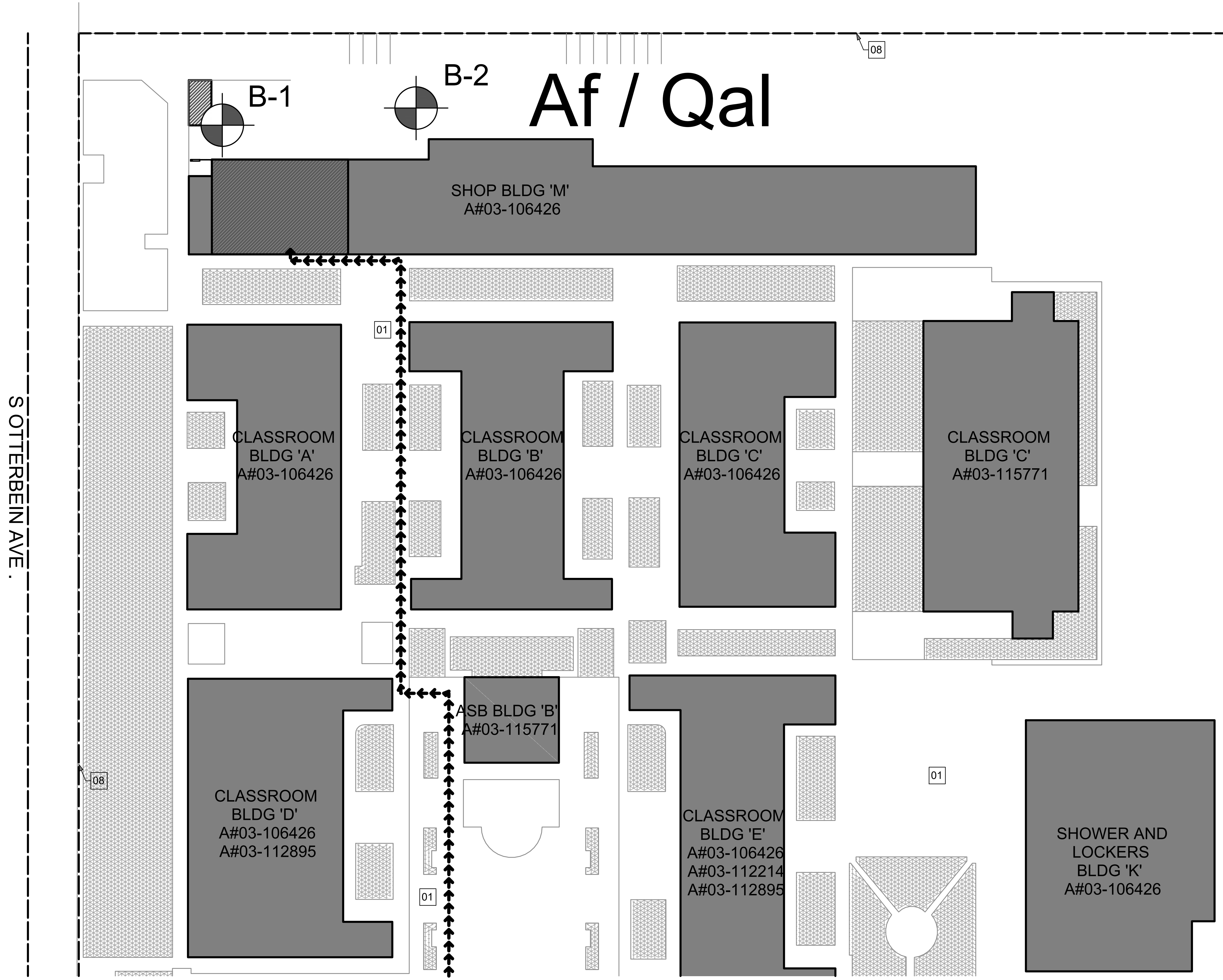
Subsurface exploration consisted of logging and sampling two exploratory borings drilled to a maximum depth of 51 feet using a drilling rig with an 8-inch diameter, hollow-stem, spiral auger. The field work was performed on April 21, 2021. The boring locations are indicated on Plate A and the boring logs are presented on Plates A-1 and A-2. The borings were backfilled with auger cuttings immediately upon completion of sampling.

Caving did not occur due to the type of auger used and materials encountered and no significant difficulty in penetrating the materials to the indicated depths was encountered.

A representative of the geotechnical engineer observed the drilling operations, collected samples of the soil and bedrock, and prepared field logs by visual/tactile examination of the materials. The samples were subsequently examined by the geotechnical engineer in the laboratory and the classifications confirmed or modified on the basis of laboratory test results.

Samples were obtained using a core barrel loaded with 2.42-in.-I.D.x 1-in-long, thin-walled, brass rings. The samples were placed in plastic bags immediately upon removal from the samplers to conserve moisture, labeled for identification and brought to our laboratory for further examination and testing.

Drawing Scale:
1" = 60'



Legend

Approximate Boring Location

Af Artificial Fill

Qal Alluvium

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ORANGE, CA

Project:
Rowland High School
Geotechnical Investigation
Address: 2000 Otterbein Avenue, Rowland Heights, CA
HGEI Project No.: 21-01-4178

Drawing Name:
BORING LOCATION
MAP

DRAWN BY: SBM
CHECKED BY: JLW
DATE: 05/10/21
REVISIONS:

Plate A

\\HGEI-PC\ENGINEERING\ACTIVE PROJECT FILES\4100\4178 2000 OTTERBEIN AVENUE, ROWLAND HEIGHTS, CA\AUTOCAD\BORING LOCATION MAP.DWG

LOG OF BORING B-1

Project: Rowland High School
Job No.: 21-01-4178
Location: 2000 Otterbein Drive, Rowland Heights, CA
Coordinates:

Surface Elev.: Grade
Top of Casing Elev.: N.A.
Drilling Method: RIG
Sampling Method: RING/SPT

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					**ALLUVIUM (Qyf):** SANDY CLAY (CL), reddish brown, moist, stiff, fine to medium grained			
							28	116	17
	5					CLAYEY SAND (SC), very dark gray, moist, medium dense, fine to medium grained	20	108	11
						@7.5' wet to saturated	22	103	18
	10					@10' moist	36	113	12
						SANDY CLAY (CL), gray brown, moist, stiff, fine to medium grained	47	110	17
	15					@15' reddish brown	43	116	13
						SANDY CLAY (CL), mottled light brown/brown, with iron staining, moist, stiff, fine grained	86	108	17
	20						21	89	18
						@22.5' trace gypsum	85	117	16
	25					SAND (SC), red, moist, dense, fine to medium grained	75	111	14
	30						20 50/6"	103	17
	35					@35' saturated	50/6"		21
	40					SILTY SAND (SM), light tan, saturated, dense, fine to medium grained	38 50/4"		23
	45						56		18
	50						83		26

Completion Depth: 51.5
Date Boring Started: 4/21/01
Date Boring Completed: 4/21/21
Logged By: SM
Drilling Contractor:

Remarks:
Groundwater was encountered at 35'.

The stratification lines represent approximate boundaries. The transition may be gradual.

Harrington Geotechnical Engineering, Inc.

PLATE A-1

LOG OF BORING B-2

Project:	Rowland High School	Surface Elev.:	Grade
Job No.:	21-01-4178	Top of Casing Elev.:	N.A.
Location:	2000 Otterbein Drive, Rowland Heights, CA	Drilling Method:	RIG
Coordinates:		Sampling Method:	RING/SPT

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					**ALLUVIUM (Qyf):** CLAYEY SAND (SC), very dark gray, moist to very moist, medium dense, fine to medium grained			
							39	119	12
	5						29	108	18
							23	107	16
	10					SANDY CLAY (CL), very dark gray with iron staining, very moist, medium stiff, fine to medium grained	13	97	20
						SILTY TO CLAYEY SAND (SM), mottled light brown/brown, very moist, dense, fine grained	68	112	16
	15						69	106	17
							77	107	16
	20						20 50/4"	107	17
						@ 22.5' trace gypsum	67	114	16
	25					SILTY TO SANDY CLAY (CL), gray brown with iron staining, moist, siff, moderate fine grained sand pods	47	110	18
	30						33 50/4"	111	18
	35					saturated	47		21
	40					SILTY TO CLAYEY SAND (SM), gray/tan with iron staining, wet, dense, fine to medium grained	75		17
	45						26 50/5"		22
	50					@ 50' medium to coarse grained	33 50/3"		16

Completion Depth:	51.5
Date Boring Started:	4/21/21
Date Boring Completed:	4/21/21
Logged By:	SM
Drilling Contractor:	

Remarks:
Groundwater was encountered at 35'.

The stratification lines represent approximate boundaries. The transition may be gradual.

Harrington Geotechnical Engineering, Inc.

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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 RETAINED ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SAND, 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

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DRAWN BY: BBC | USCS | CHECKED BY: MVD

SOIL CLASSIFICATION CHART

HGEI Project No. 21-01-4178 | **PLATE A-3**

APPENDIX B

LABORATORY PROCEDURES & TEST RESULTS

Sample Processing

The samples collected during the field investigation were examined and classified by the project geotechnical engineer in the laboratory using the visual/tactile method (ASTM D2488-009) and samples were selected for testing. The following is a description of the laboratory testing and presents the results which are incorporated in the previous sections of the report.

Moisture and Density Determination (ASTM D2216-10 & D2937-10)

The core samples were trimmed and weighed and the dry unit weights and field moisture contents were determined. The results are presented on the boring logs in Appendix A.

Compaction Test (ASTM D1557)

A compaction test was performed on a surface soil sample to develop values for use in evaluating existing conditions and initial use during grading performed at the site. Results are presented in Table 1.

Expansion Index Test (ASTM 4829)

An Expansion Index Test was conducted on a sample considered representative of the prevalent surface/near-surface soil to establish data on which to base recommendations for foundation and floor slab design. The test result is presented in Table 2.

Corrosivity Tests (EPA 300.0 & 9045)

One sample was submitted to a state certified analytical laboratory for testing for water-soluble sulfate, chloride, pH and minimum resistivity. Test results are indicated in Table 3.

Particle Size Distribution (Gradation) of Soils Using Sieve Analysis D6913

A grain size analysis was performed on several samples for data needed for the liquefaction analyses. The results are shown on Plates

Consolidation Tests (ASTM D2435)

Consolidation tests were conducted on four samples to determine the settlement characteristic of the materials. Water was added to the samples during the test to determine the effect of increased moisture. Refer to Plates B-1 thru B-4 for results.

Direct Shear Test (ASTM D3080 & D3080 M-11)

Direct shear tests were performed on four samples to determine the static strength of the earth materials. The tests were performed at increased moisture contents and at various confining pressure using a displacement rate of 0.04 in. /min. to establish peak and ultimate strength parameters under adverse conditions of moisture. Refer to Plates B-5 and B-8 for results.

Laboratory Results

TABLE 1- Compaction Test Results (ASTM D1557)			
Sample Id.	**Classification**	**Maximum Dry Density (pcf)**	**Optimum Moisture (%)**
B-1 @ 0'-2'	Clayey Silt (CL) brown	126.0	9.5

TABLE 2 Expansion Index Test Results (ASTM D4829)						
Sample ID	**Moisture Content (%)**		**Dry Density (pcf)**		**Calculated Expansion Index**	**Expansion Potential**
	Initial	**Final**	**Initial**	**Final**		
B-1 @ 0'-2'	10.1	20.0	107.6	104.1	30	Low

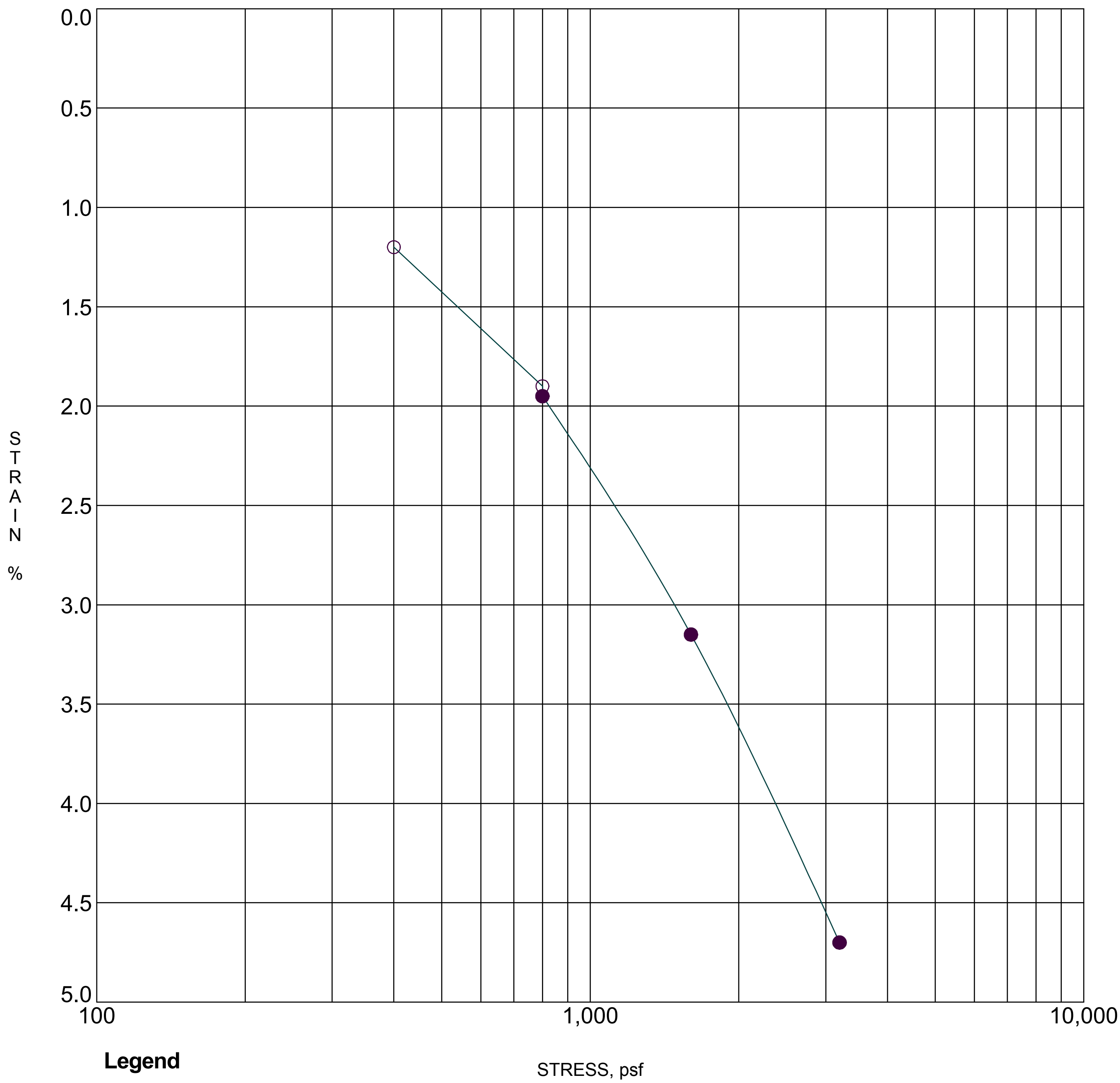
Table 3- Corrosivity Test Results (EPA 300.0 & 9045)				
Sample ID	**Water-Soluble Sulfate (mg/kg)**	**Chloride (mg/kg)**	**pH**	**Resistivity (ohm/cm)**
B-1 @ 0'-2'	39	ND	87.3	2006

ND non-detectable

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 written request to retain the samples for a longer period. (3 months maximum) Note that prolonged storage will result in sample degradation.

0-0-0



Legend

○ Before water

● After water

	Specimen Identification		Classification	DD	MC%
●	**B-1**	**7.5**	**CLAYEY SAND (SC)**	**102**	**22**
⌧					
▲					
★					
⊙					
✿					

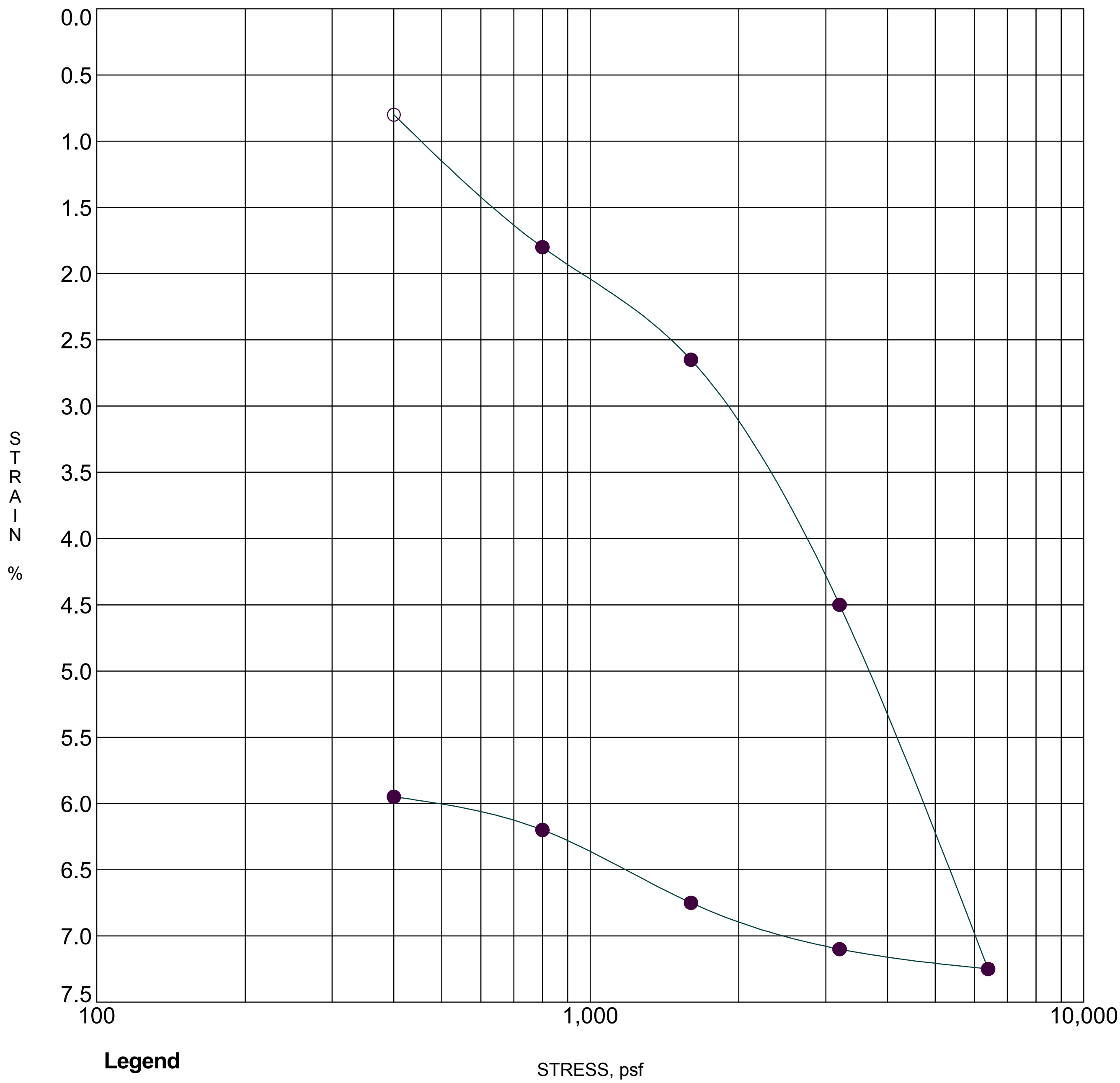
PROJECT **- 2000 Otterbein Avenue, Rowland Heights, CA** JOB NO. **21-01-4178**

DATE **5/27/21**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE B-1



Legend

○ Before water

● After water

	Specimen Identification		Classification	DD	MC%
●	**B-2**	**10.0**	**SANDY CLAY (CL)**	**96**	**24**
⧗					
▲					
★					
⊙					
✿					

PROJECT **- 2000 Otterbein Avenue, Rowland Heights, CA**

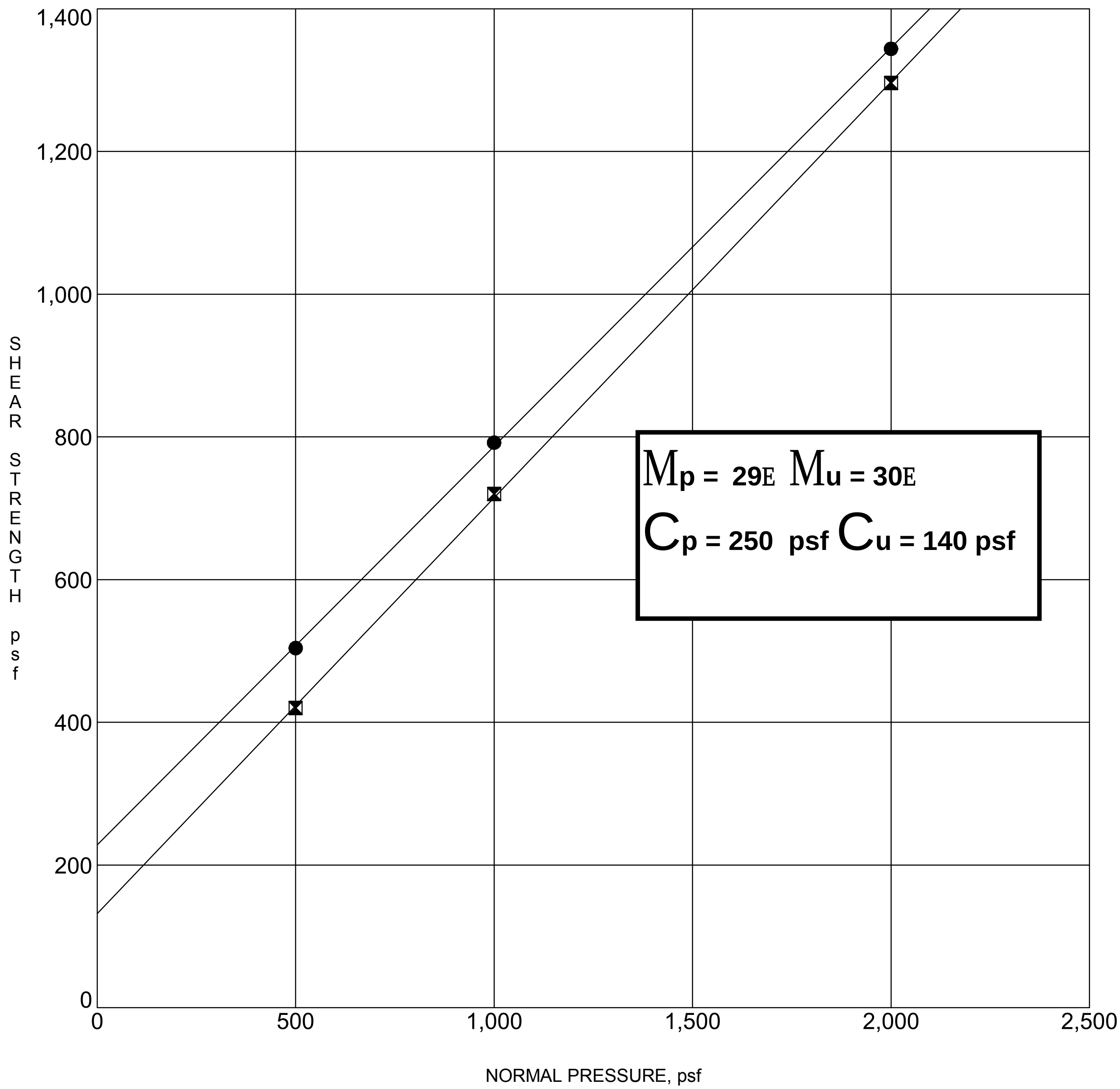
JOB NO. **21-01-4178**

DATE **5/27/21**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE B-2



	Specimen Identification		Classification	DD	MC%
●	**B-1**	**5.0**	**CLAYEY SAND (SC) (Peak)**	**102**	**18**
⧗	**B-1**	**5.0**	**(Ultimate)**		

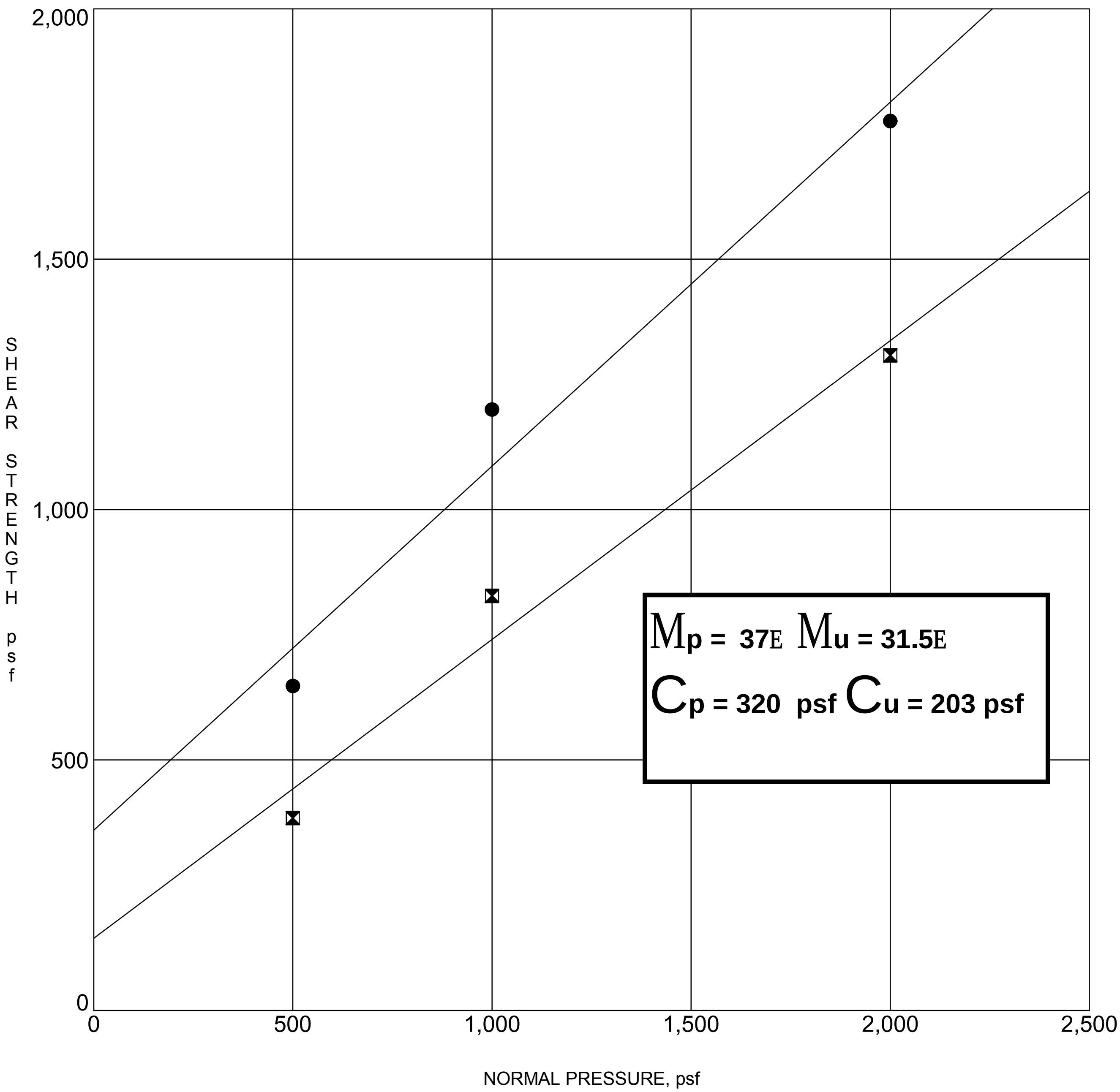
PROJECT **- 2000 Otterbein Avenue, Rowland Heights, CA**

JOB NO. **21-01-4178**
DATE **5/26/21**

SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE B-3



Specimen Identification			Classification	DD	MC%
●	B-2	2.5	CLAYEY SAND (SC) (Peak)	117	15
⧗	B-2	2.5	(Ultimate)		

PROJECT - 2000 Otterbein Avenue, Rowland Heights, CA

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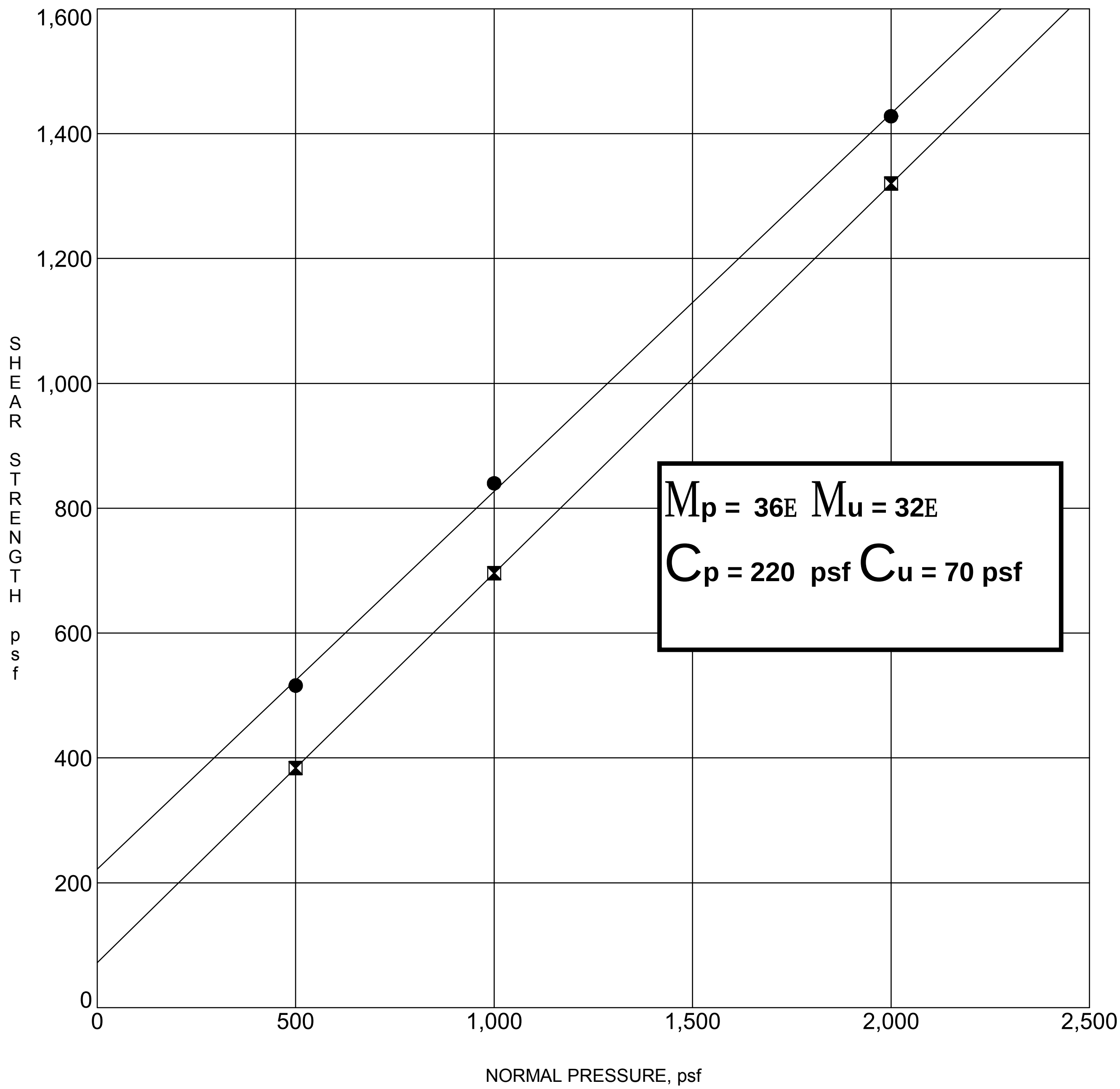
DATE 5/26/21

SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE B-4



Specimen Identification			Classification	DD	MC%
●	**B-2**	**7.5**	**CLAYEY SAND (SC) (Peak)**	**106**	**20**
⧗	**B-2**	**7.5**	**(Ultimate)**		

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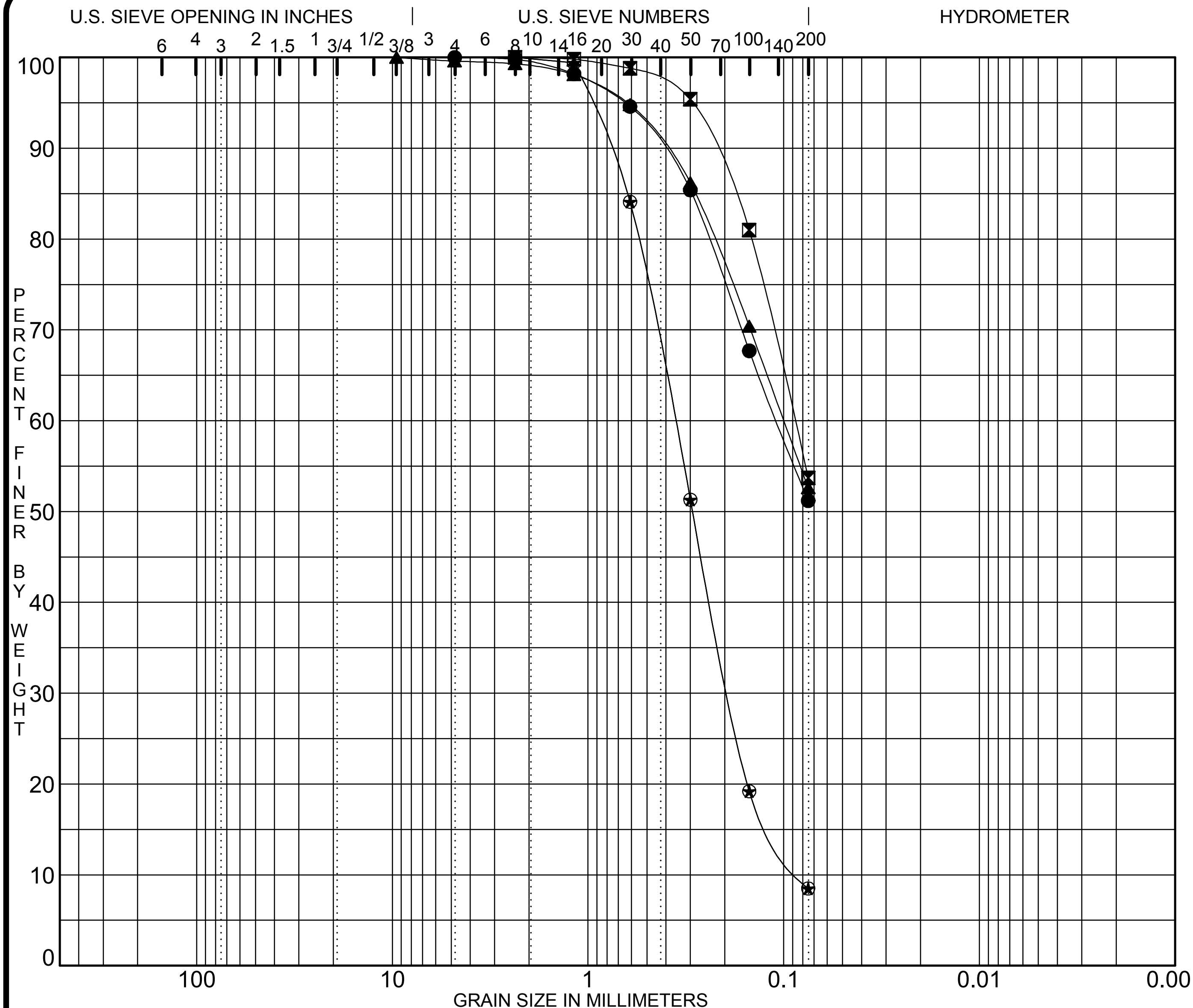
SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE B-5



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-1**	**12.5**							
⧗	**B-1**	**17.5**							
▲	**B-1**	**22.5**							
★	**B-1**	**35.0**						**1.20**	**4.4**
⊙	**B-1**	**45.0**						**1.20**	**4.4**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**12.5**	**4.80**	**0.11**			**0.0**	**48.8**	**51.2**	
⧗	**B-1**	**17.5**	**2.36**	**0.09**			**0.0**	**46.3**	**53.7**	
▲	**B-1**	**22.5**	**9.50**	**0.10**			**0.4**	**47.0**	**52.6**	
★	**B-1**	**35.0**	**2.36**	**0.36**	**0.189**	**0.0827**	**0.0**	**91.5**	**8.5**	
⊙	**B-1**	**45.0**	**2.36**	**0.36**	**0.189**	**0.0827**	**0.0**	**91.5**	**8.5**	

PROJECT **- 2000 Otterbein Avenue, Rowland Heights, CA**

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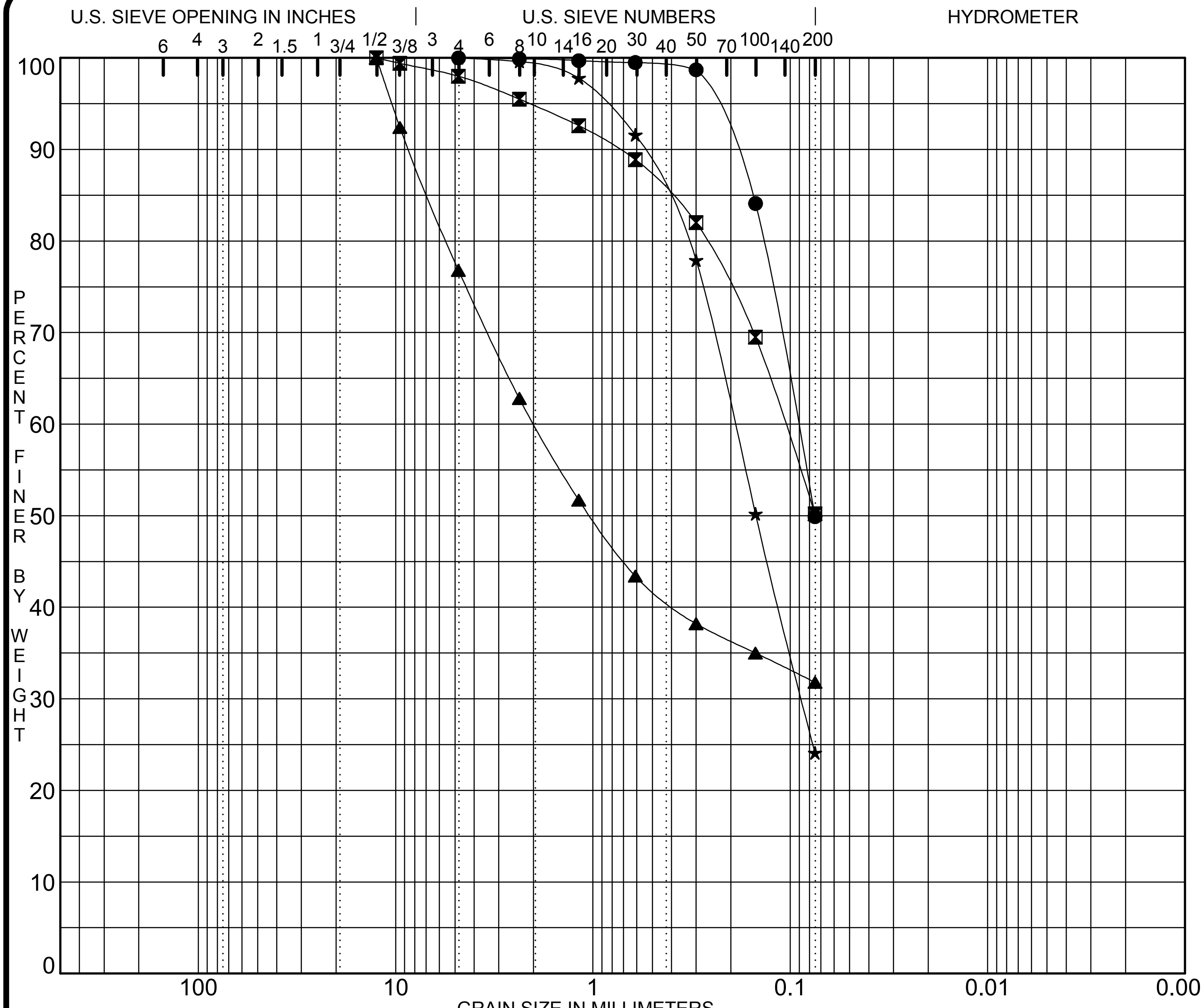
GRADATION CURVES

Harrington Geotechnical Engineering, Inc.

PLATE B-6



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-2**	**15.0**							
⧗	**B-2**	**25.0**							
▲	**B-2**	**35.0**							
★	**B-2**	**45.0**							

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**15.0**	**4.80**	**0.09**			**0.0**	**50.1**	**49.9**	
⧗	**B-2**	**25.0**	**12.50**	**0.11**			**2.0**	**47.8**	**50.2**	
▲	**B-2**	**35.0**	**12.50**	**1.98**			**23.4**	**44.8**	**31.8**	
★	**B-2**	**45.0**	**4.80**	**0.19**	**0.088**		**0.0**	**75.9**	**24.1**	

PROJECT **- 2000 Otterbein Avenue, Rowland Heights, CA**

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GRADATION CURVES

Harrington Geotechnical Engineering, Inc.

PLATE B-7

APPENDIX C

GEOLOGIC HAZARDS REPORT

TERRA GEOSCIENCES

GEOLOGIC HAZARDS REPORT

PROPOSED AUTO SHOP EXPANSION PROJECT

JOHN A. ROWLAND HIGH SCHOOL

2000 SOUTH OTTERBEIN AVENUE

ROWLAND HEIGHTS, LOS ANGELES COUNTY, CALIFORNIA

Project No. 152781-3

April 29, 2021

Prepared for:

Harrington Geotechnical Engineering, Inc.
1590 North Brian Street
Orange, CA 92867-3406

Harrington Geotechnical Engineering, Inc.
1590 North Brian Street
Orange, CA 92867-3406

Attention: Mr. Don Harrington, Jr.

Regarding: Geologic Hazards Report
Proposed Auto Shop Expansion Project
John A. Rowland High School
2000 South Otterbein Avenue
Rowland Heights, Los Angeles County, California
HGEI Project No. 21-01-4178

At your request, this firm has prepared a geologic hazards report for the proposed auto shop expansion project, as referenced above. The purpose of this study was to evaluate the existing geologic conditions of the property and any corresponding potential geologic and/or seismic hazards, with respect to the proposed development from a geologic standpoint. Previous geophysical studies (shear-wave surveys) were performed at this school site by our firm (Terra Geosciences 2015a and 2015b) for purposes of determining the Site Classification, for seismic design purposes.

This report has been prepared utilizing the suggested “Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings” (CGS Note 48, 2019), along with the Geologic portion of the “*Factors to Be Included in the Geological and Environmental Hazards Report*,” which is included as Appendix H of the “School Site Selection and Approval Guide,” prepared by the School Facility Planning Division, California Department of Education, and the Geohazard Reports requirements outlined by the DSA (2016). The scope of services provided for this evaluation included the following:

- **Review of available published and unpublished geologic/seismic data in our files pertinent to the site, including the provided site-specific boring logs.**
- **Evaluation of the local and regional tectonic setting and historical seismic activity, including performing a site-specific CBC ground motion analysis.**
- **Preparation of this report presenting our findings, conclusions, and recommendations from a geologic standpoint.**

Accompanying Maps, Illustrations, and Appendices

Plate 1 - Regional Geologic Map
Plate 2 - Google™ Earth Imagery Map
Plate 3 - Site Plan
Appendix A - Site-Specific Ground Motion Analysis
Appendix B - References

PROJECT SUMMARY

We understand that this report will be appended to your current geotechnical investigation, therefore, some descriptive sections such as site description, proposed development, etc., have been purposely omitted as they have been described in detail in your referenced report. No grading plans were available for this evaluation, and no field or subsurface exploration was performed by this firm. Only a review of available geologic and geotechnical data in our files was undertaken including your site-specific exploratory boring logs that were drilled on April 21, 2021.

GEOLOGIC SETTING

The subject property is located in southwestern California, within a natural geomorphic province known as the Peninsular Ranges, which stretch approximately 1,500-kilometers from southern California in the United States to the southern tip of Mexico's Baja California Peninsula. The rocks within this province are dominated by Mesozoic granitic rocks, derived from the same massive batholith which forms the core of the Sierra Nevada Mountains in California.

The Peninsular Ranges is generally characterized by steep elongated ranges and valleys that trend northwesterly-southeasterly and is divided into a series of fault-bounded blocks each of which has a set of uniform characteristics internally. The northern end of the Peninsular Ranges includes the Los Angeles Basin, which is a northwest-trending alluvial lowland plain about 50 miles long and 20 miles wide. The Los Angeles Basin is, in turn, comprised of several structural blocks or subdivisions which are separate by major zones of faulting or flexures in the basement rock terrain.

More specifically, the site is included within the Northeastern Block, which is a triangular-shaped wedge approximately 35 miles long from northwest to southeast. The basement rocks are exposed along the north end of the Puente and San Jose Hills and are cut by northwest to northeast trending faults that break through to the surface through the super adjacent rocks. This block is generally bounded by the Cucamonga Fault to the north, the Whittier Fault to the southwest, and the Chino Fault to the east. The block contains a very thick (as much as 13,000 feet) sequence of Miocene volcanic and sedimentary rock, as partially exposed in the San Jose and Puente Hills.

EARTH MATERIALS

Locally, the subject construction area site has been mapped by Dibblee (2001) to be surficially mantled by late Pleistocene age older alluvial deposits, generally comprised of alluvial gravel and sand, as shown on Plate 1 (map symbol Qae). These deposits are in turn underlain at depth by Miocene age sedimentary bedrock of the Puente Formation (Soquel Sandstone facies), generally comprised of bedded medium-grained arkosic sandstone, which includes silty clay shale. Subsurface exploration by HGEI (2021) within the proposed construction area encountered predominantly interbedded fine- to medium-grained sandy clay, fine- to medium-grained clayey sand, and fine- to medium-grained silty sand, to a depth of at least 51½ feet. These earth materials were noted to be in a medium dense/medium stiff to dense/stiff condition. Groundwater was noted to have been encountered locally as shallow as 35 feet in depth.

FAULTING

There are at least forty-three major late Quaternary active/potentially active faults that are located within a 100-kilometer (62 mile) radius of the site (Blake, 1989-2000a). Of these, there are no known active faults that traverse the site based on available published literature. The subject site is not located within a State of California "Alquist-Priolo Earthquake Fault Zone" for fault rupture hazard (California Geological Survey, 2018 and C.D.M.G., 1991). The nearest such mapped hazard zone is associated with the active Whittier Fault (northern segment of the Elsinore Fault Zone) located approximately 2.6± miles to the southwest, as shown below on Figure 1.

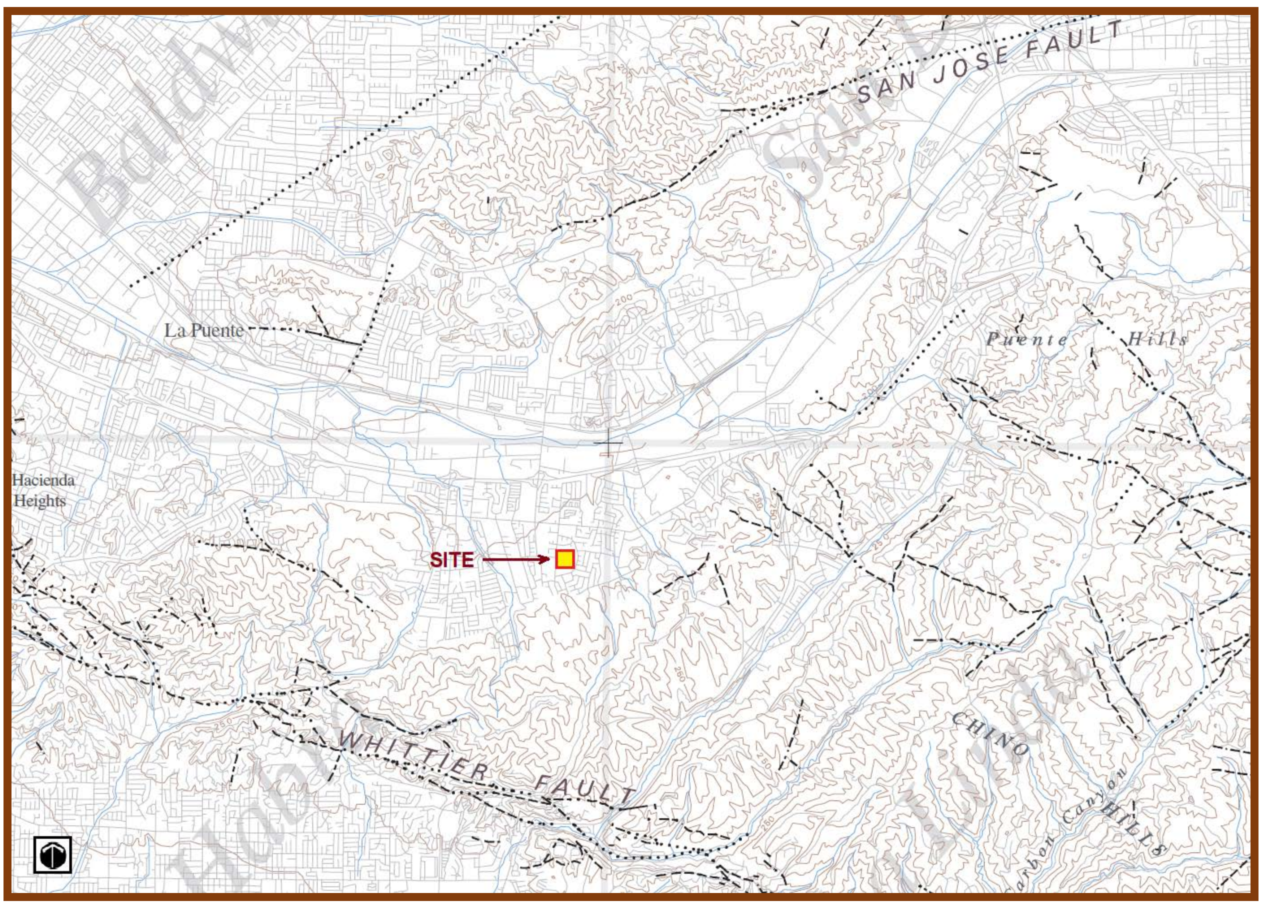


FIGURE 1- Fault Map (Morton and Miller, 2006; Sheet 2 of 4).

The Whittier Fault is a 38-kilometer long right-lateral, strike-slip fault with an estimated maximum moment magnitude of M_W 6.9, and an associated slip-rate of 2.5 ±1 mm/year (Cao et al., 2003 and Petersen et al., 2008). However, for seismic design purposes, we are considering that a cascading effect of rupture will occur along the entire length of the Elsinore Fault Zone (which includes the Whittier, Glen Ivy, Temecula, Julian, and Coyote Mountain Faults segments collectively) rather than just the Whittier segment. Based on the recently published rupture-model data (Petersen et al., 2008), the total rupture area of these combined faults is 3,841.7 square kilometers and has an associated Maximum Moment Magnitude (M_W) of 7.8.

Another nearby significant fault is the San Jose Fault approaching within 3.4±-miles to the north (see Figure 1 above), which is a 20-kilometer long left-lateral, reverse/oblique fault with an estimated maximum moment magnitude of M_W 6.7, and an associated slip-rate of 0.5 ±0.5 mm/year (Wills et al., 2007 and Petersen et al., 2008). At this time, this fault has not been mapped as being active (C.G.S., 2018). Both the Whittier and San Jose Faults were used in the site-specific seismic ground motion analysis for this study, as presented within Appendix A.

GROUNDWATER

The subject school site lies within the southern fringes of the San Gabriel Valley Groundwater Basin, which is generally bordered on the north by the Raymond Fault and the San Gabriel Mountains, consolidated sedimentary bedrock hills (including the Repetto, Merced, and Puente Hills) along the south and west, with the Chino and San Jose Faults forming the eastern boundary (California Department of Water Resources, 2004). According to data provided by the California Department of Water Resources (2021b), there are no nearby water wells. The closest measured well is located approximately one mile to the north along the San Jose Creek area, which would not be representative for the site.

Based on data presented by the California Division of Mines and Geology (1997), a high groundwater level of 25± feet is shown within the local alluvial deposits that includes the subject site. Additionally, groundwater was encountered locally within the exploratory borings drilled within the subject study area at a depth of at 35 feet (HGEI, 2021).

GROUND MOTION ANALYSIS

According to California Geological Survey Note 48 (CGS, 2019), a site-specific ground motion analysis is required for the subject site (CBC, 2019, Section 1613A and also as required by ASCE 7-16, Chapter 21), the detailed results of which are presented within Appendix A. Additionally, seismic shear-wave surveys were also performed prior to the school construction remodel, during previous ground motion studies that were performed by this firm (Terra Geosciences, 2015a and 2015b). These surveys were to aid in determining the Site Classification and VS_{30} input values for the site-specific ground motion analysis. The locations of the shear-wave survey lines (SW-1 through SW-3) are shown on Plates 3 and 4, for reference. The results of these surveys yielded seismic shear-wave velocities of **1,268.1 ft/sec** (SW-1), **1,107.7 ft/sec** (SW-2), and **1,115.3 ft/sec** (SW-3). For this project, we have elected to use the lower, most conservative value of 1,107.7 ft/sec for the input value in our ground motion analysis.

Geographically, the proposed construction area is located at Longitude -117.88468 and Latitude 33.98345 (World Geodetic System of 1984 coordinates). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps (OSHPD, 2021) and the California Building Code criteria (CBC, 2019), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017). The results of this site-specific analysis have been summarized and are tabulated below:

TABLE 1 – SUMMARY OF SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_S	**1.843g**
S_1	**0.649g**
F_a	**1.0**
F_v	**1.7**
S_{DS}	**1.240g**
S_{D1}	**0.87g**
S_{MS}	**1.865g**
S_{M1}	**1.298g**
T_L	**8 Seconds**
MCE_G PGA	**0.84g**
Shear-Wave Velocity (V100)	**1,107.7 ft/sec**
Site Classification	**D**
Risk Category	**III**

HISTORIC SEISMICITY

A computerized search, based on Southern California historical earthquake catalogs, has been performed using the programs EQSEARCH (Blake, 1989-2018) and the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2021). The following table and discussion summarize the known historic seismic events (≥M4.0) that have been estimated and/or recorded during this time period of 1800 to April 2021 within a 100-kilometer (62-mile) radius of the site.

TABLE 2 - HISTORIC SEISMIC EVENTS; 1800-2021 (100-kilometer radius)

4.0 - 4.9	448
5.0 - 5.9	59
6.0 - 6.9	15
7.0 - 7.9	0
8.0+	0

It should be noted that pre-instrumental seismic events (generally before 1932) have been estimated from isoseismal maps (Toppozada, et al., 1981 and 1982). These data have been compiled generally based on the reported intensities throughout the region, thus focusing in on the most likely epicentral location. Instrumentation beyond 1932 has greatly increased the accuracy of locating earthquake epicenters. A summary of the historic earthquake data is as follows:

- The closet recorded notable earthquake (M4.0+) was approximately one mile to the southwest (January 1, 1976, M4.4).

- The nearest estimated significant historic earthquake epicenter (pre-1932) was approximately seven miles to the northwest (December 25, 1903, M5.0).

- The nearest recorded significant historic earthquake epicenter was approximately four miles southwest of the site, being a M5.1 event, which occurred on March 21, 2014.

- The largest estimated historical earthquake magnitude within a 62-mile radius of the site is a M6.9 event of December 8, 1812 (approximately 30 miles northeast).

- The largest recorded historical earthquake was the M6.4 Long Beach event, located approximately 25 miles to the southwest (March 11, 1933).

- The largest estimated ground acceleration (based on the attenuation relationship of Boore et al., 1997) estimated to have been experienced at the site was 0.165g which resulted from the M6.3 earthquake event of July 11, 1855, located approximately 15 miles to the northwest.

An Earthquake Epicenter Map which includes magnitudes 4.0 and greater for a 100-kilometer (62-mile) radius (blue circle) has been included below as Figure 2. This map was prepared using the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2021a) of instrumentally recorded events from the period of 1932 to April 2021, in turn overlain on Google™ Earth imagery (2021). The blue circle shown on figure 2 below, represents the earthquake search radius of 100-kilometers (62 miles) from the site, which is located as the small blue dot in the center of the circle.

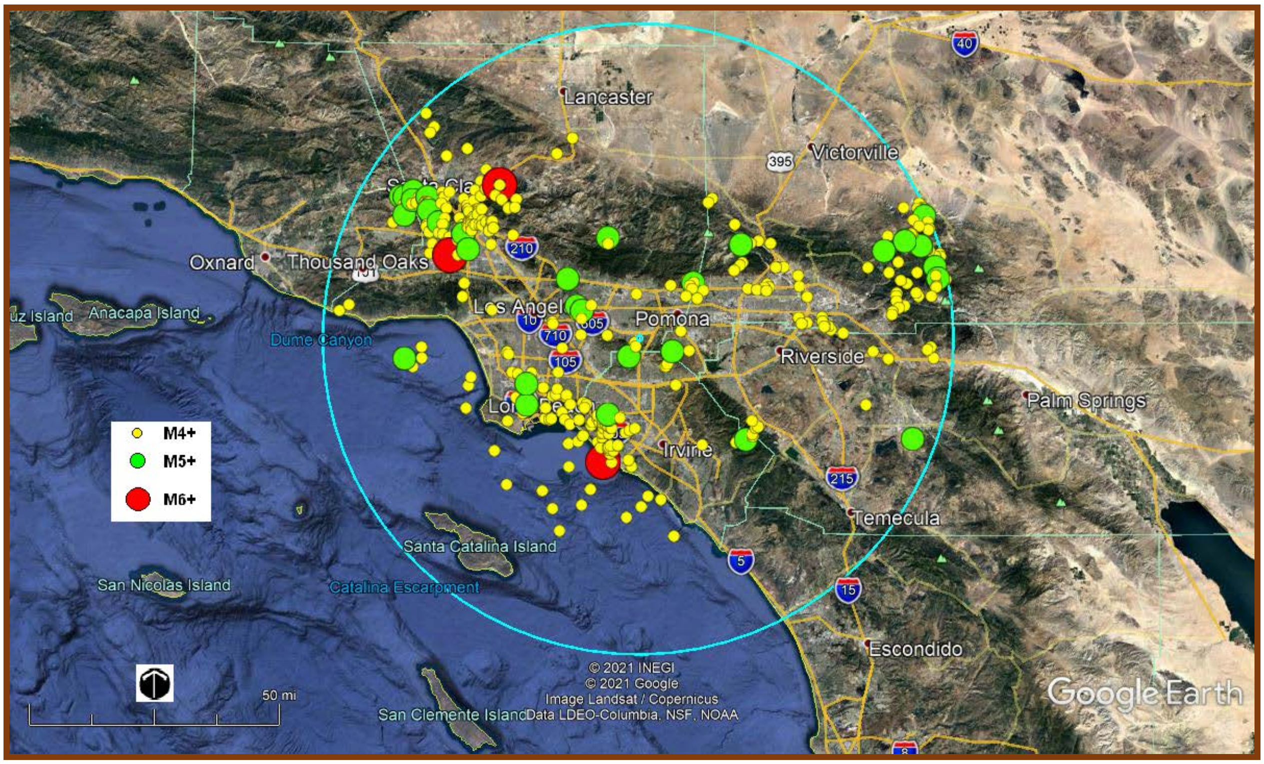


FIGURE 2- Earthquake Epicenter Map showing events of M4.0+ within a 100-kilometer radius.

SECONDARY SEISMIC HAZARDS

Secondary permanent or transient seismic hazards generally associated with severe ground shaking during an earthquake are ground rupture, liquefaction, seiches or tsunamis, ground lurching/lateral spreading, flooding (water storage facility failure), landsliding, rockfalls, and seismically-induced settlement. These hazards are discussed below.

Ground Rupture

Ground rupture is generally considered most likely to occur along pre-existing faults. Since no known active faults are believed to traverse the subject site, the probability of ground rupture is considered very low-nil.

Liquefaction

In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soils from repeated disturbances of saturated cohesionless soil that can result in the settlement of buildings, ground failures, or other related hazards. The main factors contributing to this phenomenon are: 1) cohesionless, granular soils having relatively low densities (usually of Holocene age); 2) shallow groundwater (generally less than 50 feet); and 3) moderate-high seismic ground shaking. Mapping by the California Geological Survey (2018) indicates the subject site to be included within a liquefaction zone, as shown on Plate 2, for reference. Based on the exploratory boring data (HGEI, 2021), groundwater was locally encountered at a depth of at 35 feet within the alluvial sediments. Therefore, there may to be a potential for liquefaction to occur during a large seismic event and impact the proposed development.

Seismically-Induced Settlement

Seismically-induced settlement generally occurs within areas of loose, granular soils. Based on the generally fine- to medium-grained and medium dense/medium stiff to dense/stiff nature of the alluvial sediments underlying the site, as encountered by HGEI (2021), the potential for seismically-induced settlement is considered to be low, but cannot be completely ruled out.

Ground Lurching/Lateral Spreading

Ground lurching is the horizontal movement of soil, sediments, or fill located on relatively steep embankments or scarps as a result of seismic activity, forming irregular ground surface cracks. The potential for lateral spreading or lurching is highest in areas underlain by soft, saturated materials, especially where bordered by steep banks or adjacent hard ground. Due to the relatively flat-lying nature of the project area with no exposed slopes locally (the slope farther to the north is comprised of native sedimentary bedrock) and the dense, consolidated sedimentary bedrock underlying the site and surroundings, the potential for ground lurching and/or lateral spreading at the study area appears to be low.

Seiches/Tsunamis

Based on the far distance of large, open bodies of water and the elevation of the site with respect to sea level, the possibility of seiches/tsunamis is considered nil. Additionally, mapping by the California Geological Survey (2014) does not indicate the site to be located within a tsunami inundation zone.

Landsliding

Due to the relatively low-lying relief of the site, landsliding due to seismic shaking is considered nil. Additionally, the site is not shown to be located within a zone that has a potential for earthquake-induced landsliding, as shown on Plate 2 (C.G.S., 2018).

Flooding (Water Storage Facility Failure)

Since no water storage facility (i.e., water tank, dam, etc.) is located above the site, the potential for flooding, caused by water storage facility failure, is considered nil.

Rockfalls

Since no large rock outcrops are present at or adjacent to the site, the possibility of rockfalls during seismic shaking is nil.

FLOODING

According to the Federal Emergency Management Agency (FEMA), the subject site is not located within the boundaries of a 100-year flood (Community Panel No. 06037C1875F, September 26, 2008). This portion of the campus is shown to be located within "Zone X," which is defined as “Area of Minimal Flood Hazard.”

OTHER GEOLOGIC HAZARDS

There are other potential geologic hazards not necessarily associated with seismic activity that occur statewide. These hazards include; natural hazardous materials (such as methane gas, hydrogen-sulfide gas, and tar seeps); Radon-222 gas (EPA, 1993); naturally occurring asbestos; volcanic hazards (Martin, 1982); and regional subsidence. Of these hazards, there are none that appear to impact the site.

CONCLUSIONS AND RECOMMENDATIONS

General:

Based on our review of available pertinent published and unpublished geologic/seismic literature (including the site-specific boring log data) and our field reconnaissance, construction of the proposed auto shop expansion project appears to be feasible from a geologic standpoint, providing our recommendations are considered during planning and construction.

Conclusions:

1. Based on available published geologic data and review of the provided boring logs, the site is mantled by late Pleistocene age older alluvial deposits, generally comprised of alluvial gravel and sand, in turn underlain at depth by Miocene age sedimentary bedrock of the Puente Formation (Soquel Sandstone facies), generally comprised of bedded medium-grained arkosic sandstone. More specifically, subsurface exploration by HGEI indicates the subject construction area to be underlain by predominantly interbedded fine- to medium-grained sandy

clay, fine- to medium-grained clayey sand, and fine- to medium-grained silty sand, to a depth of at least 51½ feet, that are in a medium dense/medium stiff to dense/stiff condition.

2. Based on subsurface exploration by HGEI, groundwater was locally encountered at a depth of 35 feet. Mapping by the California Geological Survey indicates groundwater levels to be a shallow as 25± feet in the general region. Shallow groundwater conditions are not anticipated to be encountered during construction within the subject site.

3. Based on our literature research, no active faults are known to traverse the subject site. The nearest mapped active fault by the State of California is the Whittier Fault, located approximately 2.6±-miles to the southwest. The San Jose Fault, located 3.4±-miles to the north-northwest, is not currently zoned as active.

4. Based on review of available pertinent geologic data, there appears to be a potential for liquefaction to occur in the event of a large seismogenic event in the region. Additionally, the potential for secondary seismic settlement may also be a possibility, although is considered to be low. There do not appear to be any other potential permanent or transient secondary seismic hazards, as previously discussed, that would affect the proposed development.

5. The primary geologic hazard that exists at the site is that of ground shaking, which accounts for nearly all earthquake losses. Moderate to severe ground shaking could be anticipated during the life of the proposed development.

Recommendations:

1. For seismic design purposes, we are considering that a cascading effect of rupture will occur along the entire length of the Elsinore Fault Zone (which includes the Whittier, Glen Ivy, Temecula, Julian, and Coyote Mountain fault segments collectively) rather than just the singular Whittier Fault segment (which has an estimated maximum moment magnitude of M_W 6.9). This type of cascading rupture event has an associated Maximum Moment Magnitude (M_W) of 7.8. At this time, the San Jose Fault is considered to be capable of producing a Maximum Moment Magnitude earthquake of M_W 6.7. Although the Whittier Fault has a greater seismogenic potential with respect to the site, both the San Jose and Whittier faults were analyzed for the site-specific ground-motion analysis as presented within Appendix A.

2. It is recommended that all structures be designed to at least meet the current 2019 CBC edition and the ASCE Standard 7-16, where applicable; however, it should be noted that the building code is described as a minimum design condition and is often the maximum level to which structures are designed. Structures that are built to minimum code are designed to remain standing after an earthquake in order for

occupants to safely evacuate, but then may have to ultimately be demolished (Larson and Slosson, 1992). It is the responsibility of both the property owner and project structural engineer to determine the risk factors with respect to using CBC minimum design values for the proposed school facilities. This information should be carefully reviewed prior to construction.

3. The potential for liquefaction and seismically-induced settlement should be properly evaluated by the project Geotechnical Engineer, if warranted. A high groundwater level of at least 25 feet should be used for analysis along with a site acceleration of 0.84g, as presented within Appendix A.

CLOSURE

Our conclusions and recommendations are based on a review of available existing geologic/seismic data and the provided site-specific provided subsurface exploratory boring logs. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. It is assumed that all the conclusions and recommendations outlined in this report are understood and followed. If any portion of this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geologist / Geophysicist
CEG 1459 / PGP 1002

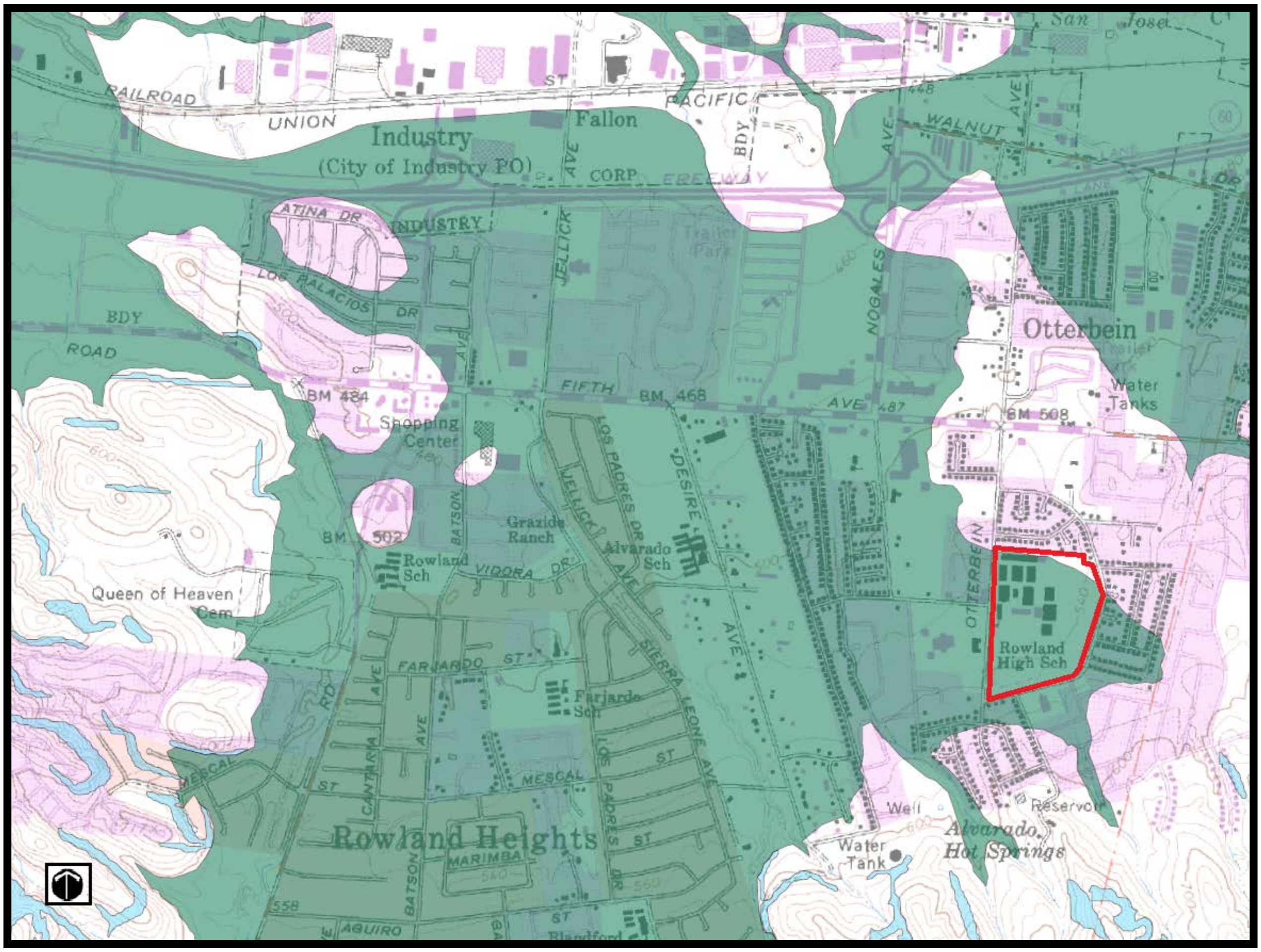
REGIONAL GEOLOGIC MAP

BASE MAP: Dibblee (2001); Dibblee Foundation Map #DF-74, Scale 1: 24,000; School site outlined in blue.

PARTIAL LEGEND

Qae	**OLDER ALLUVIAL DEPOSITS**	Slightly elevated and locally dissected alluvial gravel and sand (late Pleistocene).
Tmss	**PUENTE FORMATION**	Soquel Sandstone Facies: Bedded arkosic sandstone, medium-grained, includes silty clay shale (Miocene).
	GEOLOGIC CONTACT	Solid where well located, dashed where inferred, dotted where concealed.
	FAULT	Solid where well located, dashed where inferred, dotted where concealed.

SEISMIC HAZARDS ZONE MAP



BASE MAP: C.G.S., 2018, La Habra 7.5' Seismic Hazard Zones Map, Scale 1: 24,000; Site outlined in red.

JOHN A. ROWLAND HIGH SCHOOL

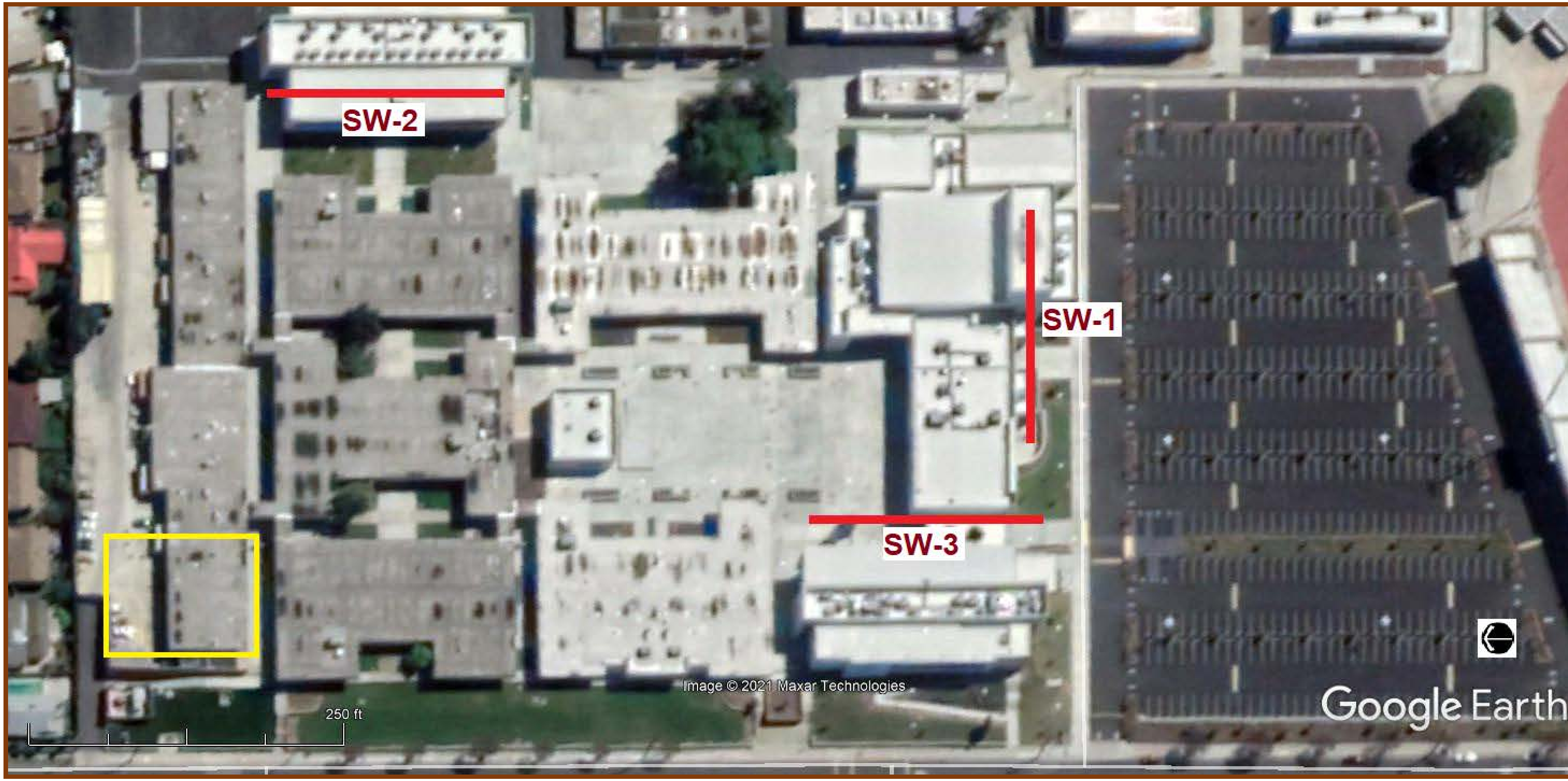
ROWLAND HEIGHTS, LOS ANGELES COUNTY, CALIFORNIA

LEGEND

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

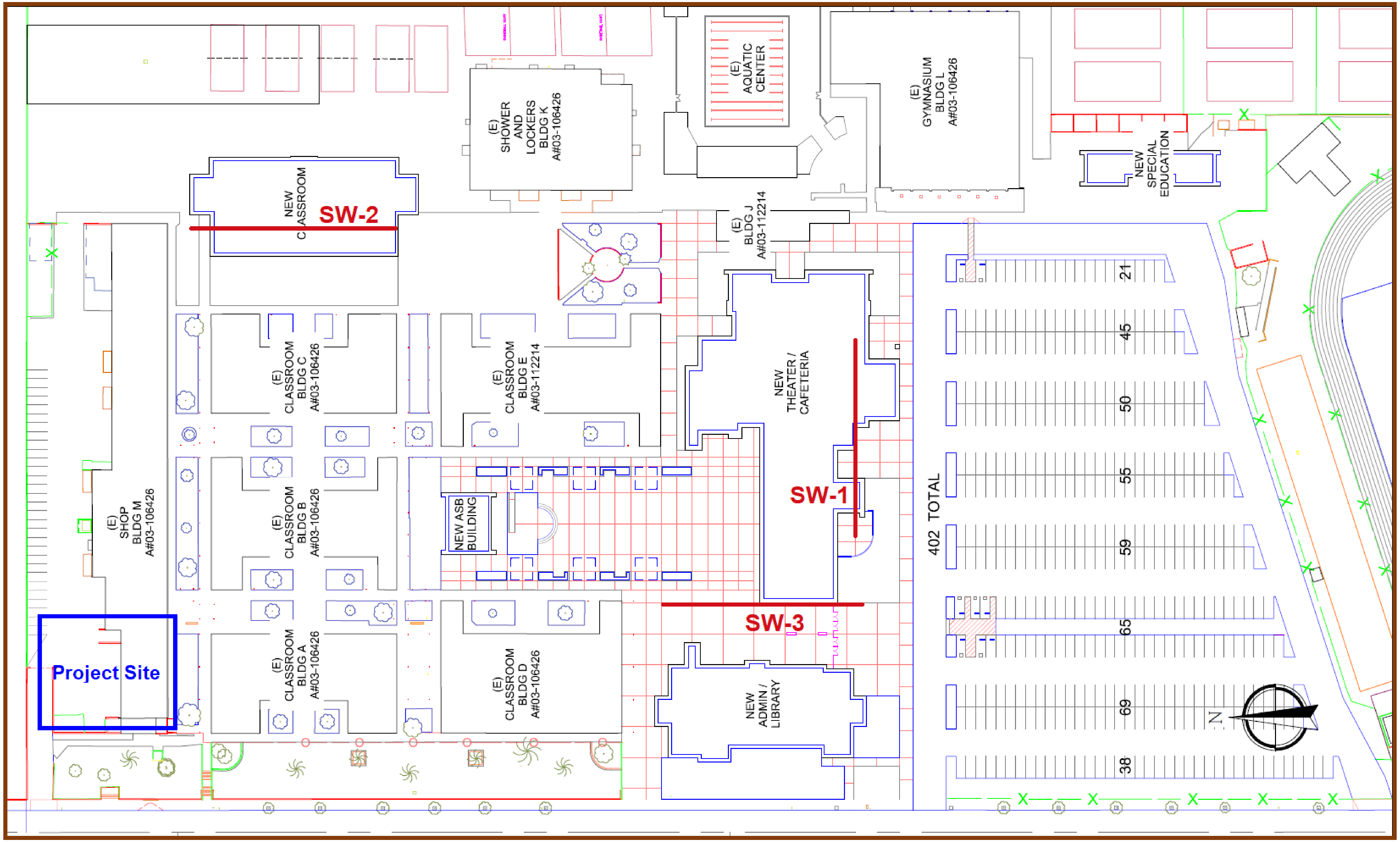
Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

GOOGLE™ EARTH IMAGERY MAP



Base Map: Google™ Earth (2021); Seismic shear-wave traverses SW-1 through SW-3 shown as red lines, project site outlined in yellow.

SITE PLAN



BASE MAP: Site Plan prepared by WLC Architects, Inc. (partial modified copy); Seismic shear-wave traverses SW-1 through SW-3 shown as red lines, project site outlined in blue.

APPENDIX A

SITE-SPECIFIC GROUND MOTION ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS

A detailed summary of the site-specific ground motion analysis, which follows Section 21 of the ASCE Standard 7-16 (2017) and the 2019 California Building Code is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

- **Mapped Spectral Acceleration Parameters (CBC 1613A.2.1)**-

  Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **1.843g** for the 0.2 second period (S_S) and **0.649** for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613A.2.1).

- **Site Classification (CBC 1613A.2.2 & ASCE 7-16 Chapter 20)**-

  Based on the selected conservative site-specific measured shear-wave value of 1,107.7 feet/second (337.6 m/sec), the soil profile type used should be Site Class **"D."** This Class is defined as having the upper 100 feet (30 meters) of the subsurface being underlain by very dense soil/soft rock with average shear-wave velocities of 600 to 1,200 feet/second (180 to 360 meters/second).

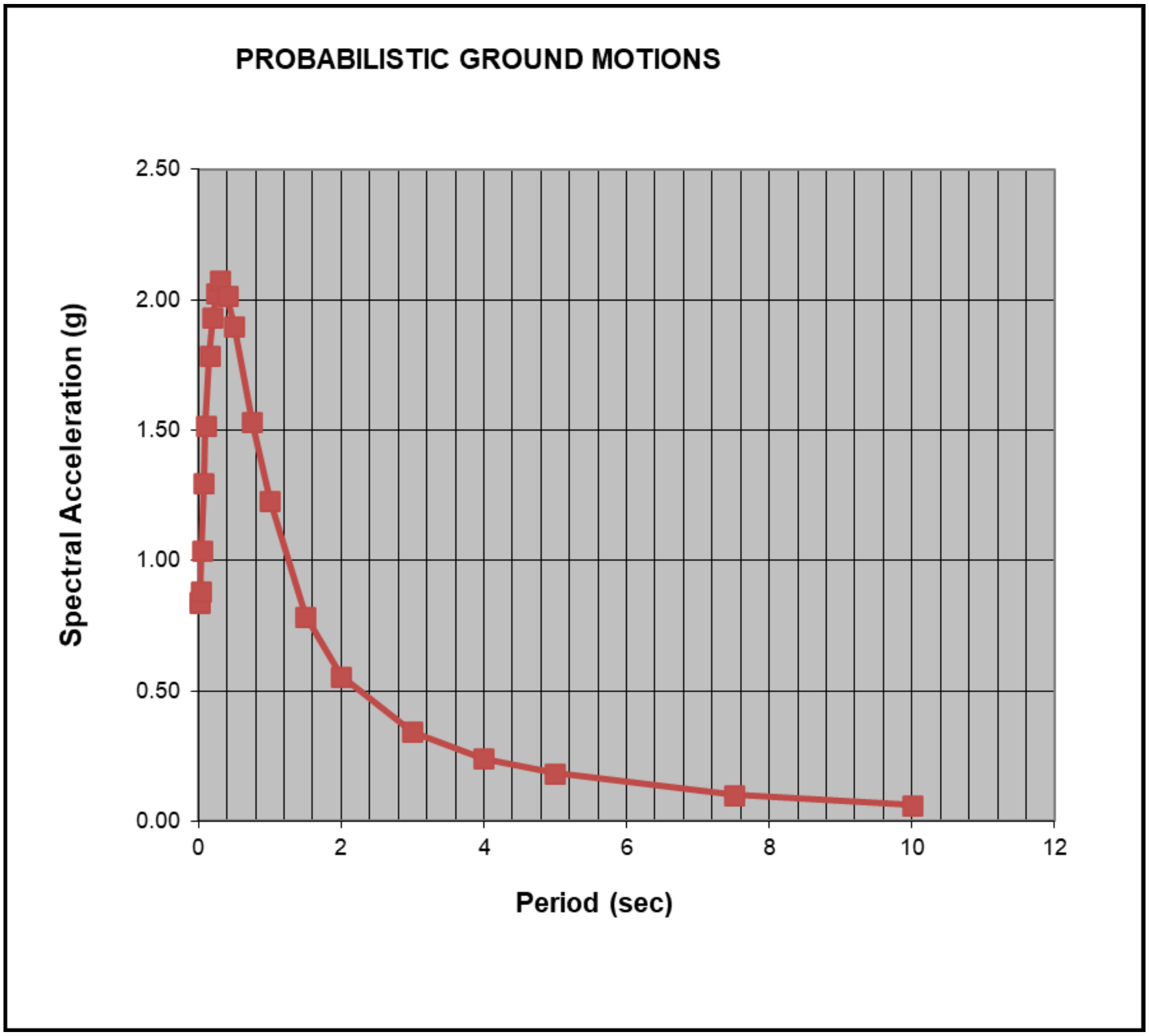
- **Site Coefficients (CBC 1613A.2.3)**-

  Based on CBC Tables 1613A.2.3(1) and 1613A.2.3(2), the site coefficient $\mathbf{F_a = 1.0}$ and $\mathbf{F_v = 1.7}$, respectively.

- **Probabilistic (MCE_R) Ground Motions (ASCE 7 Section 21.2.1)**-

  Per Section 21.2.1.1 (**Method 1**), the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

  The probabilistic analysis included the use of the Open Seismic Hazard Analysis (OpenSHA). The selected Earthquake Rupture Forecast (ERF) was UCERF3 along with a Probability of Exceedance of 2% in 50 Years. The average of four Next Generation Attenuation West-2 Relations (2014 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2014), Abrahamsom et al. (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Campbell & Bozorgnia (2014). The Probabilistic Risk Targeted Response Spectrum was determined as the product of the ordinates of the probabilistic response spectrum and the applicable risk coefficient (C_R). These values were then modified to produce a spectrum based upon the maximum rotated components of ground motion. The resulting MCE_R Response Spectrum is indicated below:



- ***Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)***-

  The deterministic MCE_R response acceleration at each period shall be calculated as an 84th-percentile 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. Analyses were conducted using the average of four Next Generation Attenuation West-2 Relations (2014 NGA), including Chiou & Youngs (2014), Abrahamsom et al. (2014), Boore et al. (2014), and Campbell & Bozorgnia (2014).

  Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013), published geologic data, and based on the length and maximum magnitude of the Whittier Fault Zone (combined segments), a moment magnitude (M_W) used for this fault was 7.8. Additionally, the nearby San Jose Fault was also used for this analysis (M_W 6.7).

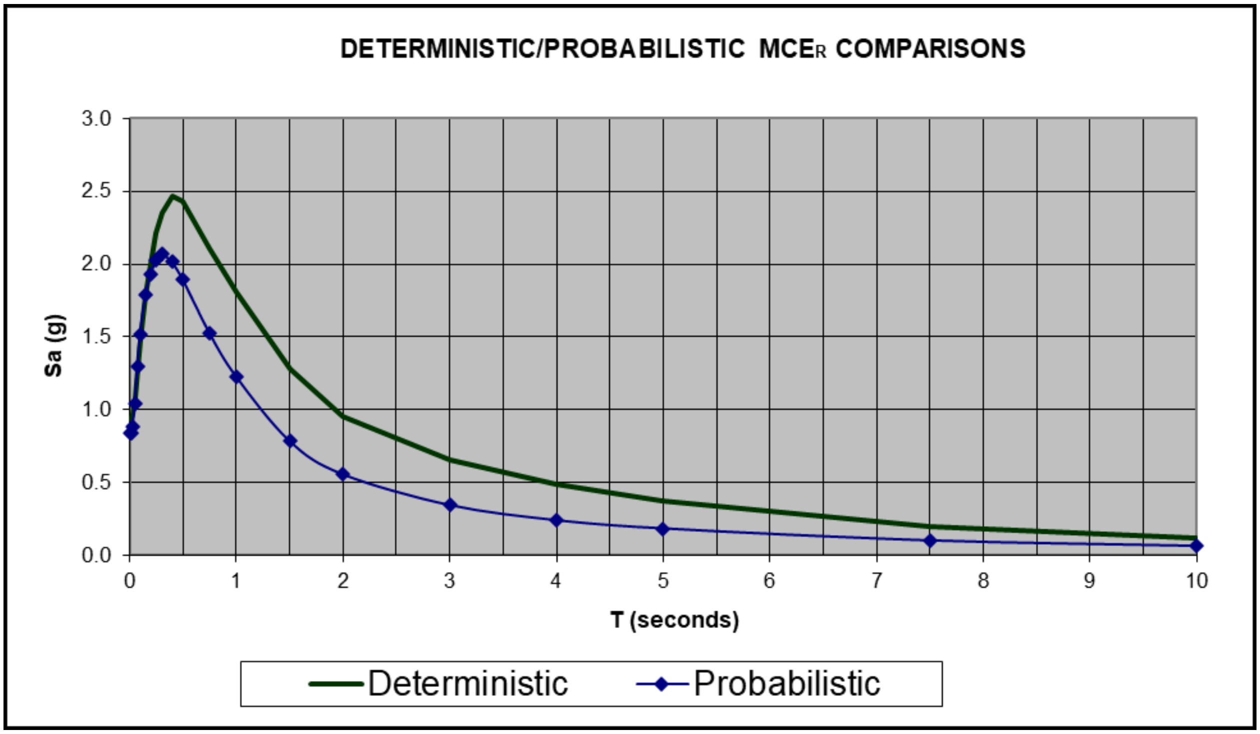
- **Site Specific MCE_R (ASCE 7 Section 21.2.3)**-

The site-specific MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2. The deterministic ground motions were compared with the probabilistic ground motions that were determined in accordance with Section 21.2.1. These results are tabulated below:

Comparison of Deterministic MCE_R Values with Probabilistic MCE_R Values - Section 21.2.3

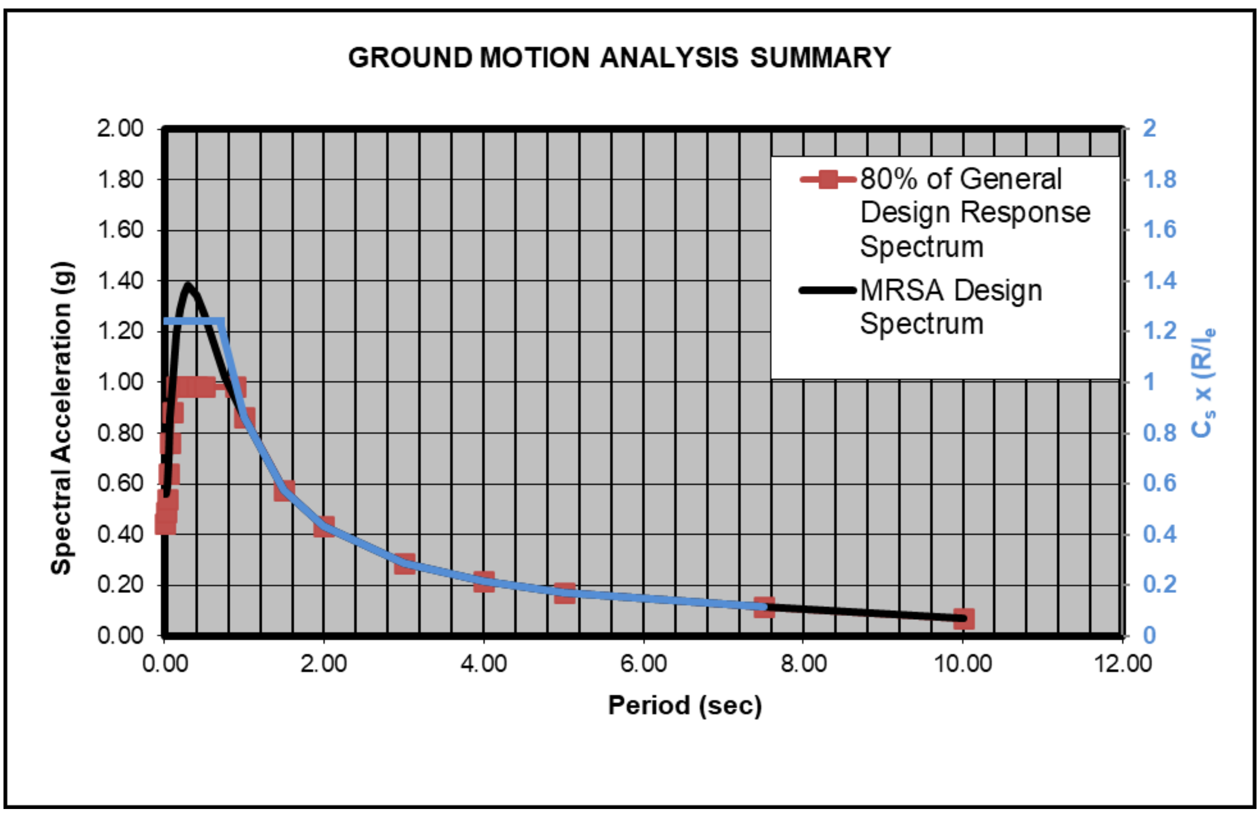
Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.94	0.84	0.84	Probabilistic Governs
0.020	0.94	0.84	0.84	Probabilistic Governs
0.030	0.97	0.88	0.88	Probabilistic Governs
0.050	1.09	1.04	1.04	Probabilistic Governs
0.075	1.29	1.30	1.29	Deterministic Governs
0.100	1.48	1.52	1.48	Deterministic Governs
0.150	1.79	1.79	1.79	Probabilistic Governs
0.200	2.00	1.93	1.93	Probabilistic Governs
0.250	2.21	2.02	2.02	Probabilistic Governs
0.300	2.35	2.07	2.07	Probabilistic Governs
0.400	2.47	2.02	2.02	Probabilistic Governs
0.500	2.43	1.90	1.90	Probabilistic Governs
0.750	2.10	1.53	1.53	Probabilistic Governs
1.000	1.81	1.23	1.23	Probabilistic Governs
1.500	1.28	0.79	0.79	Probabilistic Governs
2.000	0.95	0.56	0.56	Probabilistic Governs
3.000	0.66	0.34	0.34	Probabilistic Governs
4.000	0.49	0.24	0.24	Probabilistic Governs
5.000	0.38	0.18	0.18	Probabilistic Governs
7.500	0.19	0.10	0.10	Probabilistic Governs
10.000	0.12	0.06	0.06	Probabilistic Governs

These comparisons are plotted in the following diagram:



- ***Design Response Spectrum (ASCE 7 Section 21.3)***-

  In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3S_{aM}$, where S_{aM} is the MCE_R spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of S_a. These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



- **Design Acceleration Parameters (ASCE 7 Section 21.4)**-

  Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the products of Sa * T for periods between 1 and 5 seconds. The parameters S_{MS}, and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with Section 11.4.4 for S_{MS}, and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

- **Site Specific Design Parameters** -

  For the 0.2 second period (S_{DS}), a value of 1.24g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.38g occurring at T=0.30 seconds. This was multiplied by 0.9 to produce a value of 1.24g making this the applicable value. A value of 0.87g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 1.865g (S_{MS}) was computed, along with a value of 1.298g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

- **Site-Specific MCE_G Peak Ground Accelerations (ASCE 7 Section 21.5)**-

  The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.84g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.84g. The site-specific MCE_G peak ground acceleration was calculated to be **0.84g**, which was determined by using the lesser of the probabilistic (0.84g) or the deterministic (0.84g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 0.87g x 0.80 = 0.70g).

SEISMIC DESIGN PARAMETERS SUMMARY

Project: Rowland High School
Project #: 152781-3
Date: 4/22/21

Lattitude: 33.98345
Longitude: -117.88468

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	1.843	Figure 22-1
S_1=	0.649	Figure 22-2

Site Class per Table 20.3-1

Site Class=	D - Stiff Soil

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1	Table 11.4-1	=	1	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.7	Table 11.4-2	=	2.50	For Site Specific Analysis per ASCE7-16 21.3

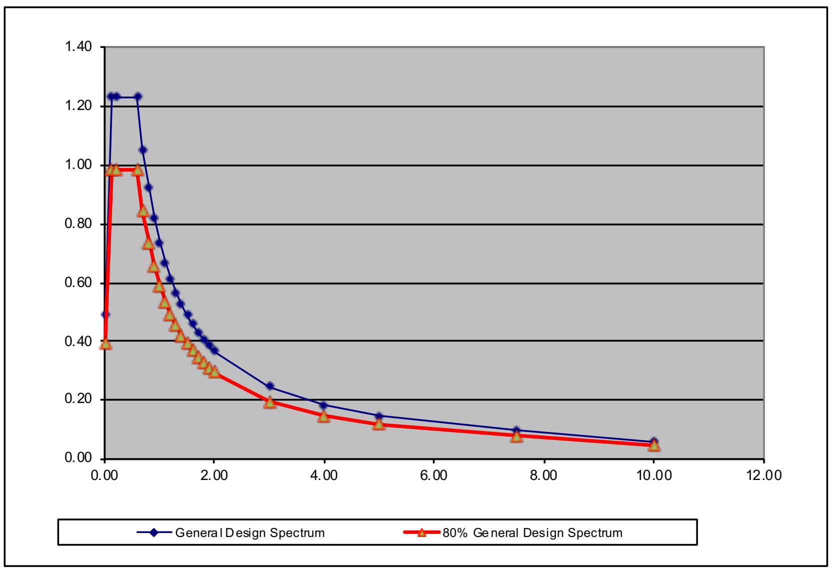
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	1.843	Equation 11.4-1	1.843	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	1.103	Equation 11.4-2	1.623	For Site Specific Analysis per ASCE7-16 21.3

T_0=	0.120	sec	
T_S=	0.599	sec	
T_L=	8	sec	From Fig 22-12
PGA	0.793	g	
F_{PGA}=	1.1		From Table 11.8-1
C_{RS}=	0.901		Figure 22-17
C_{R1}=	0.903		Figure 22-18

S_{DS}=	1.229	Equation 11.4-3
S_{D1}=	0.736	Equation 11.4-4

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.49	0.39
0.12	1.23	0.98
0.20	1.23	0.98
0.60	1.23	0.98
0.70	1.05	0.84
0.80	0.92	0.74
0.90	0.82	0.65
1.00	0.74	0.59
1.10	0.67	0.53
1.20	0.61	0.49
1.30	0.57	0.45
1.40	0.53	0.42
1.50	0.49	0.39
1.60	0.46	0.37
1.70	0.43	0.35
1.80	0.41	0.33
1.90	0.39	0.31
2.00	0.37	0.29
3.00	0.25	0.20
4.00	0.18	0.15
5.00	0.15	0.12
7.50	0.10	0.08
10.00	0.06	0.05



ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

Use Maximum Rotated Horizontal Component?* (Y/N) y

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

PROBABILISTIC MCER per 21.2.1.1

Earthquake Rupture Forecast - UCERF3 **Method 1**

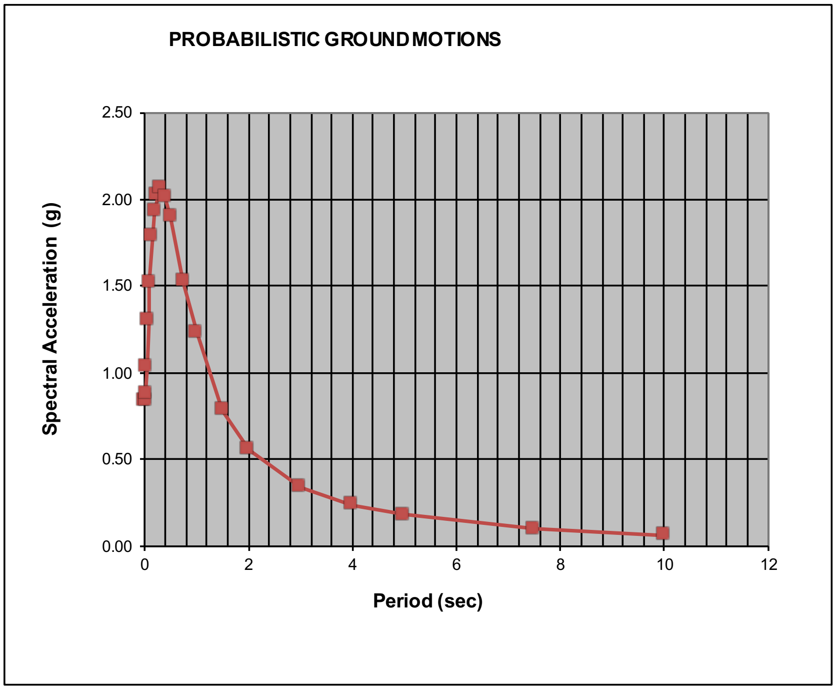
Field, E.H., T.H. Jordan, and C.A. Cornell (2003), OpenSHA: A Developing Community-Modeling Environment for Seismic Hazard Analysis, Seismological Research Letters, 74, no. 4, p. 406-419.

OpenSHA data

2% Probability Of Exceedance in 50 years

Maximum Rotated Horizontal Component determined per ASCE7-16

T	Sa 2% in 50	MCER
0.01	0.93	0.84
0.02	0.94	0.84
0.03	0.98	0.88
0.05	1.15	1.04
0.08	1.44	1.30
0.10	1.69	1.52
0.15	1.98	1.79
0.20	2.15	1.93
0.25	2.25	2.02
0.30	2.30	2.07
0.40	2.24	2.02
0.50	2.10	1.90
0.75	1.70	1.53
1.00	1.36	1.23
1.50	0.87	0.79
2.00	0.62	0.56
3.00	0.38	0.34
4.00	0.27	0.24
5.00	0.20	0.18
7.50	0.11	0.10
10.00	0.07	0.06



S_s=	2.15	1.93
S_1=	1.36	1.23
PGA	0.84 g	

Risk Coefficients:			
C_{RS}	0.901	Figure 22-18	Get from Mapped Values
C_{R1}	0.903	Figure 22-19	
Fa=	1	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Input Parameters		Whittier Fault	San Jose Fault
Fault			
M	= Moment magnitude	7.8	6.7
R_{RUP}	= Closest distance to coseismic rupture (km)	4.2	5.5
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	4.2	5.5
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	4.2	5.5
U	= Unspecified Faulting Flag (Boore et.al.)	0	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0	1
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0	0
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	0	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	0	0
δ	= Average dip of rupture plane (degrees)	90	74
V_{S30}	= Average shear-wave velocity in top 30m of site profile	337.6	337.6
$F_{Measured}$		1	1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.25	0.25
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	1.9	1.9
Site Class		D	D
W (km)	= Fault rupture width (km)	14.6	16.5
F_{AS}	= 0 for mainshock; 1 for aftershock	0	0
σ	=Standard Deviation	1	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

T	Whittier Fault	San Jose Fault	Maximum $S_{a\,(Average)}$	Corrected* S_a (per ASCE7-16)	Scaled $S_{a(Average)}$	Controlling Fault
0.010	0.85	0.74	0.85	0.94	0.94	Whittier Fault
0.020	0.86	0.75	0.86	0.94	0.94	Whittier Fault
0.030	0.88	0.76	0.88	0.97	0.97	Whittier Fault
0.050	0.99	0.83	0.99	1.09	1.09	Whittier Fault
0.075	1.17	1.00	1.17	1.29	1.29	Whittier Fault
0.100	1.35	1.17	1.35	1.48	1.48	Whittier Fault
0.150	1.63	1.43	1.63	1.79	1.79	Whittier Fault
0.200	1.82	1.59	1.82	2.00	2.00	Whittier Fault
0.250	1.99	1.69	1.99	2.21	2.21	Whittier Fault
0.300	2.09	1.74	2.09	2.35	2.35	Whittier Fault
0.400	2.14	1.68	2.14	2.47	2.47	Whittier Fault
0.500	2.07	1.54	2.07	2.43	2.43	Whittier Fault
0.750	1.70	1.18	1.70	2.10	2.10	Whittier Fault
1.000	1.39	0.91	1.39	1.81	1.81	Whittier Fault
1.500	0.97	0.56	0.97	1.28	1.28	Whittier Fault
2.000	0.70	0.37	0.70	0.95	0.95	Whittier Fault
3.000	0.47	0.21	0.47	0.66	0.66	Whittier Fault
4.000	0.33	0.13	0.33	0.49	0.49	Whittier Fault
5.000	0.25	0.09	0.25	0.38	0.38	Whittier Fault
7.500	0.13	0.04	0.13	0.19	0.19	Whittier Fault
10.000	0.08	0.02	0.08	0.12	0.12	Whittier Fault
PGA	0.84	0.69	0.84		0.84	g

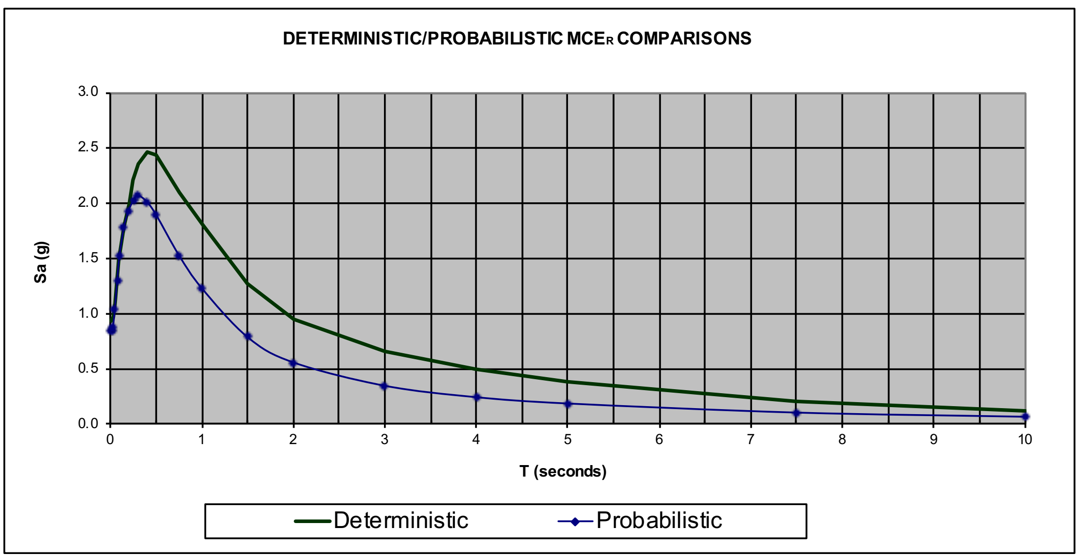
Max Sa=	2.47	
Fa =	1.00	Per ASCE7-16 21.2.2
1.5XFa=	1.5	
Scaling Factor=	1.00	

* Correction is the adjustment for Maximum Rotated Value if Applicable

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.94	0.84	0.84	ProbabilisticGoverns
0.020	0.94	0.84	0.84	ProbabilisticGoverns
0.030	0.97	0.88	0.88	ProbabilisticGoverns
0.050	1.09	1.04	1.04	ProbabilisticGoverns
0.075	1.29	1.30	1.29	Deterministic Governs
0.100	1.48	1.52	1.48	Deterministic Governs
0.150	1.79	1.79	1.79	ProbabilisticGoverns
0.200	2.00	1.93	1.93	ProbabilisticGoverns
0.250	2.21	2.02	2.02	ProbabilisticGoverns
0.300	2.35	2.07	2.07	ProbabilisticGoverns
0.400	2.47	2.02	2.02	ProbabilisticGoverns
0.500	2.43	1.90	1.90	ProbabilisticGoverns
0.750	2.10	1.53	1.53	ProbabilisticGoverns
1.000	1.81	1.23	1.23	ProbabilisticGoverns
1.500	1.28	0.79	0.79	ProbabilisticGoverns
2.000	0.95	0.56	0.56	ProbabilisticGoverns
3.000	0.66	0.34	0.34	ProbabilisticGoverns
4.000	0.49	0.24	0.24	ProbabilisticGoverns
5.000	0.38	0.18	0.18	ProbabilisticGoverns
7.500	0.19	0.10	0.10	ProbabilisticGoverns
10.000	0.12	0.06	0.06	ProbabilisticGoverns



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	2/3*MCE_R	80% General Design Response Spectrum (per ASCE 7-16 23.3-1)	Design Response Spectrum	TXSa
0.01	0.56	0.44	0.56	
0.02	0.56	0.49	0.56	
0.03	0.59	0.54	0.59	
0.05	0.69	0.64	0.69	
0.08	0.86	0.76	0.86	
0.10	0.99	0.89	0.99	
0.15	1.19	0.98	1.19	
0.20	1.29	0.98	1.29	
0.25	1.35	0.98	1.35	`
0.30	1.38	0.98	1.38	
0.40	1.34	0.98	1.34	
0.50	1.27	0.98	1.27	
0.75	1.02	0.98	1.02	
1.00	0.82	0.87	0.87	0.87
1.50	0.52	0.58	0.58	0.87
2.00	0.37	0.43	0.43	0.87
3.00	0.23	0.29	0.29	0.87
4.00	0.16	0.22	0.22	0.87
5.00	0.12	0.17	0.17	0.87
7.50	0.07	0.12	0.12	
10.00	0.04	0.07	0.07	

Highest value of S_a for any period exceeding 0.2 sec.=	1.38
90%of Highest Value =	1.24
Maximum TXSa from T=1s-5s =	0.87

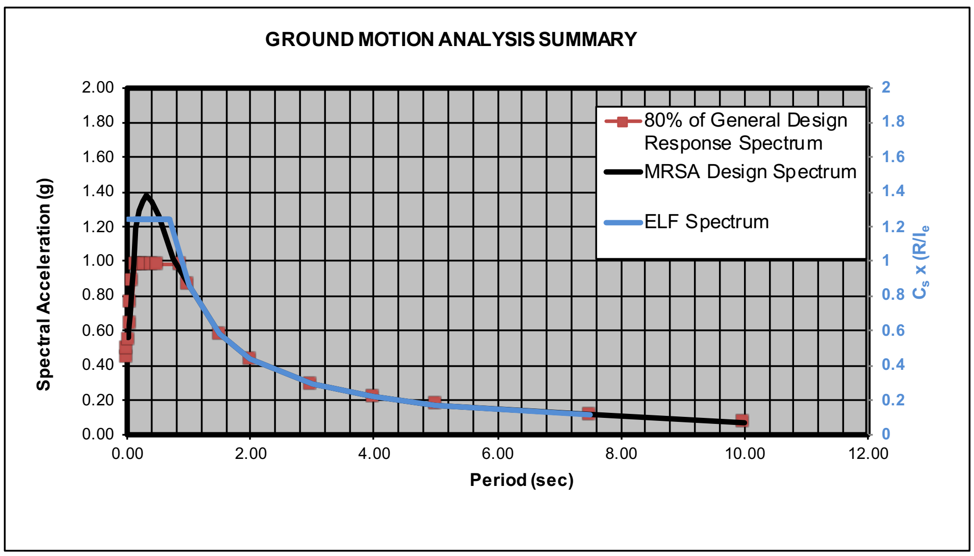
S_{DS}=	1.24
S_{D1}=	0.87
T_s =	0.70

S_{MS}=	1.865
S_{M1}=	1.298

PGA Determination:

Site Coefficient F_{PGA}=	1.1	
Mapped PGA=	0.79	Figure 22-7
PGA_M =	0.87	g

Deterministic PGA =	0.84	g
Probabilistic PGA =	0.84	g
Lesser of Deterministic/Probabilistic =	0.84	g
80% of $PGA_{M=}$	0.70	g
MCE_G PGA=	0.84	g



APPENDIX B

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REFERENCES

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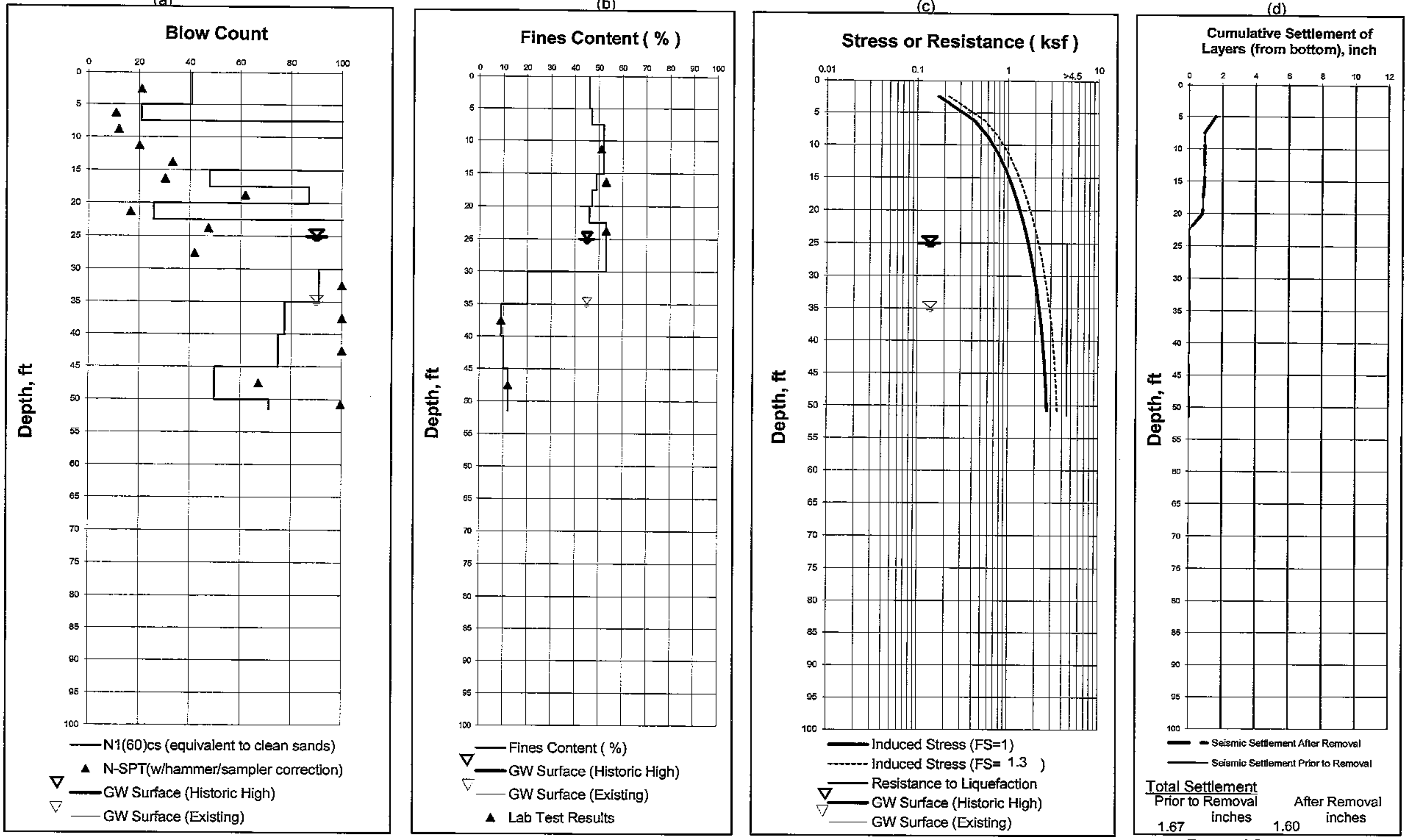
APPENDIX D

LIQUEFACTION ANALYSES

COMPUTER PROGRAM: EQLique&Settle"2"

Location.......... B-1
Elevation (MSL) (ft) 530

NOTE: If the total settlement is very small (e.g.<0.05"), it will not be seen due to the scale used, and should be reported as "negligible".



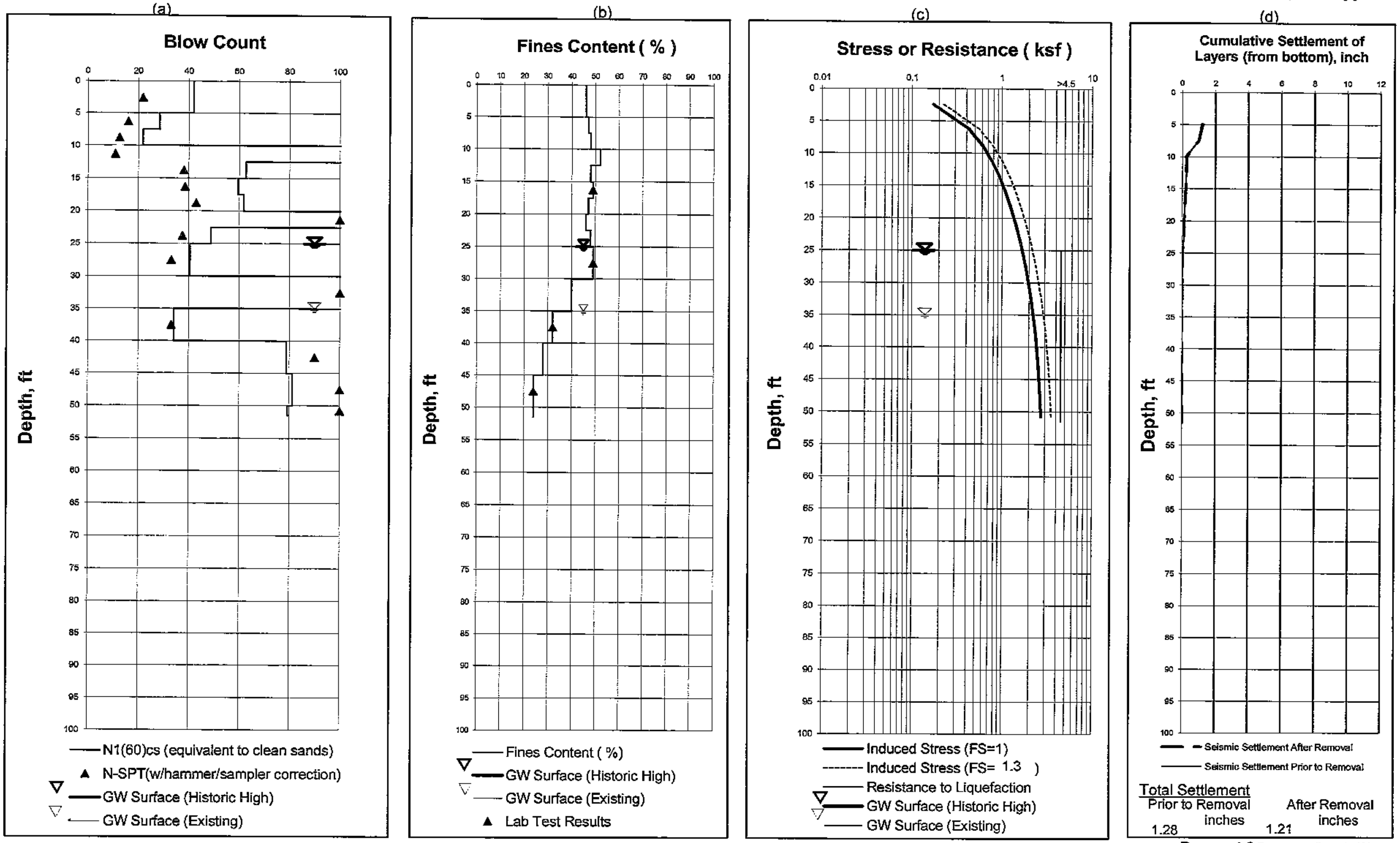
Removal & Recomp. Depth (ft) = 3

PROJECT: Rowland High School Building (M-10) Rowland Heights, Los Angeles County, CA Rowland Unified School District	Weighted Ground Accel. (M=7.5) = 0.93 g Site Magnitude = 7.8	**Liquefaction Potential and Seismic Settlements Based on Boring Data** **HARRINGTON GEOTECHNICAL INC** **Geotechnical Engineering Consultants**	Job No.: 21-01-4178 Date: 5-25-2021 Figure No. 2

COMPUTER PROGRAM: EQLique&Settle"2"

Location.......... B-2
Elevation (MSL) (ft) 530

NOTE: If the total settlement is very small (e.g.<0.05"), it will not be seen due to the scale used, and should be reported as "negligible".



Total Settlement

Prior to Removal	After Removal
1.28 inches	1.21 inches

Removal & Recomp. Depth (ft) = 3

PROJECT: Rowland High School Building (M-10) Rowland Heights, Los Angeles County, CA Rowland Unified School District	Weighted Ground Accel. (M=7.5) = 0.93 g Site Magnitude = 7.8	Liquefaction Potential and Seismic Settlements Based on Boring Data HARRINGTON GEOTECHNICAL INC Geotechnical Engineering Consultants	Job No.: 21-01-4178 Date: 5-25-2021 Figure No. 2

APPENDIX E

GRADING SPECIFICATIONS

APPENDIX E

Grading Specifications

These specifications present generally accepted standards and minimum grading (earthwork) requirements for the development of the subject project. These specifications shall be the project guidelines for earthwork except where specifically superseded in the geotechnical report(s) for the subject project; including the approved grading plan; and/or approved grading permit.

The Project Geotechnical Engineer and Project Engineering Geologist should be properly notified for an opportunity to review the following recommendations in order to comment on the suitability of the recommendations on the proposed development.

1. General

1.1. The Contractor shall be responsible for the satisfactory completion of all earthwork (including grading of constructed fills and cuts) in accordance with the project plans and specifications.

1.2. The Project Geotechnical Engineer and Project Engineering Geologist or their authorized representatives shall perform observations, testing services and geotechnical consultation throughout the duration of the project.

1.3. It is the Contractor's responsibility to prepare the ground surface to receive the fill to the satisfaction of the Project Geotechnical Engineer and to place, spread, mix and compact the fill materials in accordance with the project specifications and as required by the Project Geotechnical Engineer. The Contractor shall also remove all material considered by the Project Geotechnical Engineer to be unsuitable for use in the construction of compacted fills.

1.4. The Contractor shall have suitable and sufficient equipment in operation to handle the volume of fill material being placed and provide support equipment to properly compact the material in accordance with project specifications. When necessary, equipment will be shut down temporarily in order to permit proper compaction of fills by support equipment.

2. Site Preparation

2.1. Excessive vegetation and all deleterious material shall be removed from the fill areas and disposed of offsite of the grading operation. Existing earth materials determined by the Project Geotechnical Engineer as being unsuitable (incompatible) for placement in compacted fill areas shall be removed and disposed of offsite of the grading operation. When applicable, the Contractor may obtain the approval of the Project Geotechnical Engineer and the controlling authorities for the project to dispose of the above-described materials, or a portion thereof, in designated areas onsite.

2.2. The exposed surfaces in areas to receive fill shall be scarified to a depth specified by the geotechnical report or a nominal 6 inches as determined by the Project Geotechnical Engineer; moisture conditioned as necessary; and compacted. In areas where it is necessary to obtain the approval of the controlling agency prior to placing fill, it will be the Contractor's responsibility to arrange the required inspections.

2.3. Any underground structures, e.g. cesspools, cisterns, septic tanks, wells, pipelines, etc., encountered during the grading operation are to be removed or relocated and the ground prepared for fill (cut) in a proper manner as recommended by the Project Geotechnical Engineer and/or the controlling agency for the project.

3. Subdrains

3.1. All subdrains should be constructed below the fill areas. Horizontal subdrains should be constructed below sloping fill areas at approximate 30 feet vertical intervals. Typical subdrains (less than 300 linear feet in length) should of constructed of 4 inches diameter perforated Schedule 40 PVC pipe surrounded by one cubic foot per linear foot of gravel and filter fabric. Canyon subdrains should of constructed of 8 inches diameter perforated Schedule 40 PVC pipe surrounded by nine cubic feet per linear foot of gravel and filter fabric.

4. Compacted Fills/Fill Slopes

4.1. All material imported to the grading operation should be reviewed by the Project Geotechnical Engineer for compatibility prior to placement as fill. Laboratory testing of import materials may be required as recommended by the Project Geotechnical Engineer. Import materials deemed unacceptable for placement of fill should be removed from the fill areas and disposed of offsite of the grading operation.

4.2. All rock or rock fragments less than 8 inches in size should be incorporated into fill in a manner which will prevent nesting and the rock/rock fragments are completely surrounded with compacted fill.

4.3. All rocks greater than 8 inches in size shall be removed from the project site or placed in accordance with the recommendations of the Project Geotechnical Engineer and controlling agency code in areas designated as suitable for rock disposal.

4.4. All fill materials shall be placed in thin loose lifts, moisture conditioned as necessary and compacted in accordance with project specifications. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to obtain a nearly uniform moisture condition and a nearly uniform blend of materials.

4.5. All wet materials proposed for placement in fill areas should be moisture conditioned as necessary (either air dried or mechanically mixed). The Project Geotechnical Engineer may recommend removal of materials deemed too wet for placement of fill.

4.6. All fills shall be compacted to minimum project standards in compliance with the testing methods specified in the geotechnical report and in accordance with recommendations of the Project Geotechnical Engineer. Unless otherwise specified, the compaction standard shall be ASTM D1557 (latest approved standard).

4.7. All proposed slopes receiving fill (or ground sloping in excess of a ratio of five horizontal to one vertical), the fill shall be keyed and benched through all unsuitable topsoil, colluvium, alluvium, or creep-prone material into competent bedrock in accordance with the recommendations and approval of the Project Geotechnical Engineer or Project Engineering Geologist.

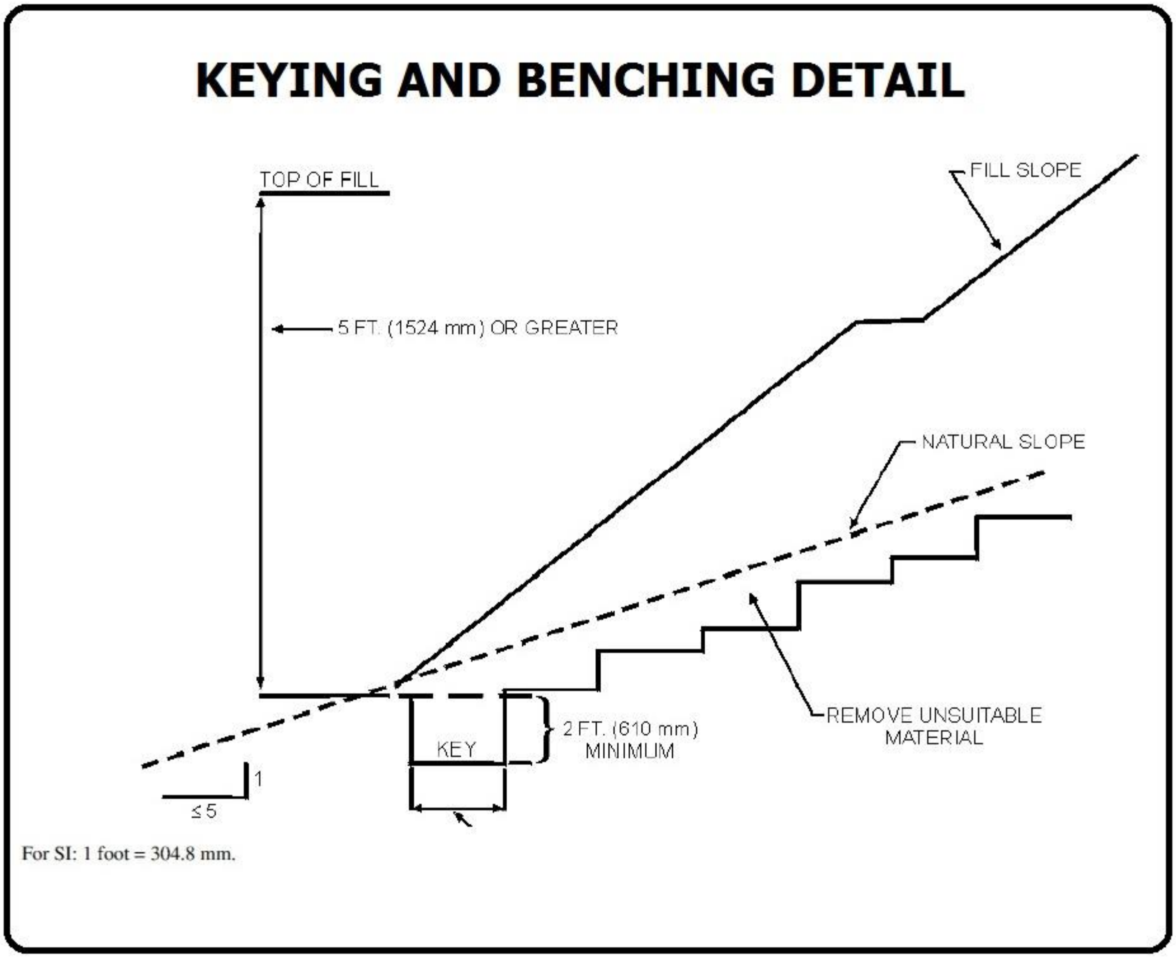
4.8. All drainage terraces for proposed fill slopes shall be constructed in compliance with the approved Grading Plan and/or the recommendations of the Project Civil Engineer. The preparation of the ground for construction of the drainage terraces should be reviewed for suitability by the Project Geotechnical Engineer.

4.9. All fill slopes (including buttresses and stabilization fills) shall be graded to a ratio not to exceed two horizontal to one vertical. The Contractor shall be required to obtain the specified minimum relative compaction out to the proposed finish slope face of slope. This may be achieved by both overbuilding the slope and cutting back to expose the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure which produces the designated result.

5. Keying and Benching

5.1. All fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep-prone material into bedrock or other firm material, and the transition shall be stripped of all unsuitable materials prior to placing fill. See the Keying and Benching Detail. The cut portion should be completed and then evaluated by the Project Engineering Geologist prior to placement of fill. The minimum dimensions of the key should be determined by the Project Engineering Geologist. All keys should include a subdrain as specified in Section 3.

Figure D1 Keying and Benching Detail



6. Cut Slopes

6.1. All cut slopes shall be inspected by the Project Engineering Geologist. The Contractor should notify the Project Engineering Geologist when cut slopes are started. If, during the course of grading, previously unforeseen and/or unanticipated adverse or potentially adverse geologic conditions are encountered, the Engineering Geologist and Geotechnical Engineer shall investigate, analyze and make recommendations for mitigation of these conditions.

6.2. All cut slopes shall be graded to a ratio not to exceed two horizontal to one vertical.

6.3. All drainage terraces for proposed cut slopes and shall be constructed in compliance with the approved Grading Plan and/or the recommendations of the Project Civil

Engineer. The preparation of the ground for construction of the drainage terraces should be reviewed for suitability by the Project Geotechnical Engineer.

7. Retaining Wall Backfill

The retaining wall backfill should include a 12" wide blanket of granular soil (with a sand equivalent of at least 30) above a constructed subdrain and extend to within 3 feet of finished grade. The top 3 feet of backfill should consist of site material compacted to at least 90 percent relative compaction to impede surface water infiltration. Benches at least 2 feet wide should be cut into the excavation slope (backcut) at 2 feet vertical intervals during backfill placement.

The subdrain should consist of a 3-inch-diameter, perforated, Schedule 40 PVC or ABS SDR-35 pipe surrounded by one cubic foot/foot of 3/4-inch gravel wrapped in Mirafi 140 N Geofabric or similar product. An adequate outlet for the subdrain should be provided and the location of the subdrain outlet should be reviewed by the project geotechnical engineer (engineering geologist) for suitability.

8. Utility Trench Backfills

Backfill for utility trenches should consist of site material that must be adequately compacted to preclude detrimental settlement. It is recommended, therefore, that backfills placed below the building foundation and to a distance of five feet outside thereof, and/or below concrete flatwork, be placed in appropriate lifts, moisture conditioned as necessary and mechanically compacted as to at least 90 percent of maximum dry density. Import materials (including sand) should be reviewed by the Project Geotechnical Engineer for suitability.

9. Grading Observations

9.1. Grading operations shall be observed by the Project Geotechnical Engineer (Geotechnical Technician) and where required, the Project Engineering Geologist.

9.2. All field density tests shall be made by the Geotechnical Technician to establish the relative compaction and moisture content of the fill in accordance with project specifications. Density tests shall generally be performed at (minimum) intervals not to exceed of 2 vertical feet or 1,000 cubic yards of material placed.

9.3. All field density testing of fill placed during the grading operation shall conform to the minimum project specifications. When test results indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction (or outside the acceptable moisture range); the fill shall be reworked until the required density and/or moisture content has been attained; or the material shall be removed. No additional fill shall be placed over an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements and that lift has been approved by the Project Geotechnical Engineer.