Geotechnical Investigation for Design and Construction of New School Buildings and Associated Improvements at Stanley G. Oswalt Academy, 19501 Shadow Oak Drive, Walnut, CA 91789

Prepared for:

Mr. Marcos Rodriquez, Construction Coordinator
ROWLAND UNIFIED SCHOOL DISTRICT
1830 Nogales Street
Claremont, CA 91711

HGEI Project No. 18-01-3763

December 14, 2018

Harrington Geotechnical Engineering, Inc.

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Mr. Marcos Rodriquez, Construction Coordinator
ROWLAND UNIFIED SCHOOL DISTRICT
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Claremont, CA 91711

Subject: Geotechnical Investigation for Design and Construction of New School Buildings and Associated Improvements at Stanley G. Oswalt Academy, 19501 Shadow Oak Drive, Walnut, CA 91789
HGEI Project No. 18-01-3763

Dear Mr. Rodriquez:

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

Preliminary plans and information provided by Ziemba + Prieto Architects were used in outlining the scope of the investigation and preparing this report. The investigation was performed in accordance with generally accepted geotechnical engineering practice in this area and our Proposal No. P-5123, dated October 18, 2018.

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

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 at this time, please call at your convenience.

Very truly yours,

HARRINGTON GEOTECHNICAL ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER JOSEPH L. WELCH No. GE2239 EXP. 9/30/20 GEOTECHNICAL STATE OF CALIFORNIA

CERTIFIED ENGINEERING GEOLOGIST ALLYSON L. STEINES No. 1848 Expires: 12/20 STATE OF CALIFORNIA

Joseph L. Welch, P.E., G.E.
Senior Geotechnical Engineer

Allyson L. Steines, CEG
Senior Engineering Geologist

Distribution: file, Addressee, Ziemba Architects, Hohbach-Levin Inc., Via E-mail

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INTRODUCTION

This report presents the results of a geotechnical investigation of the subject site. The purposes of the investigation were to: 1) determine the types and condition of the materials underlying the proposed construction areas; 2) establish static physical and limited chemical properties of the materials; 3) determine groundwater conditions; and 4) provide recommendations for design and construction of the new school buildings and associated improvements.

SCOPE OF WORK

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

Review of published and private regional geologic maps and reports (See References).

A field exploration was conducted on November 17, 2018 and consisted of drilling, logging, and sampling sixteen small diameter hollow-stem auger exploratory borings (B-1 to B-16) to depths of up to 16 feet and two additional borings (P-1 and P-2) to depths of 3 feet for infiltration testing. The field exploration is described in detail in Appendix A.

Selected samples were tested in HGEI's AMRL Accredited Geotechnical Laboratory to develop data necessary for analysis of subsurface conditions and used in preparation of this report. A description of the geotechnical laboratory testing conducted for the samples collected at the site and presentation of the results are 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 DESCRIPTION AND HISTORY

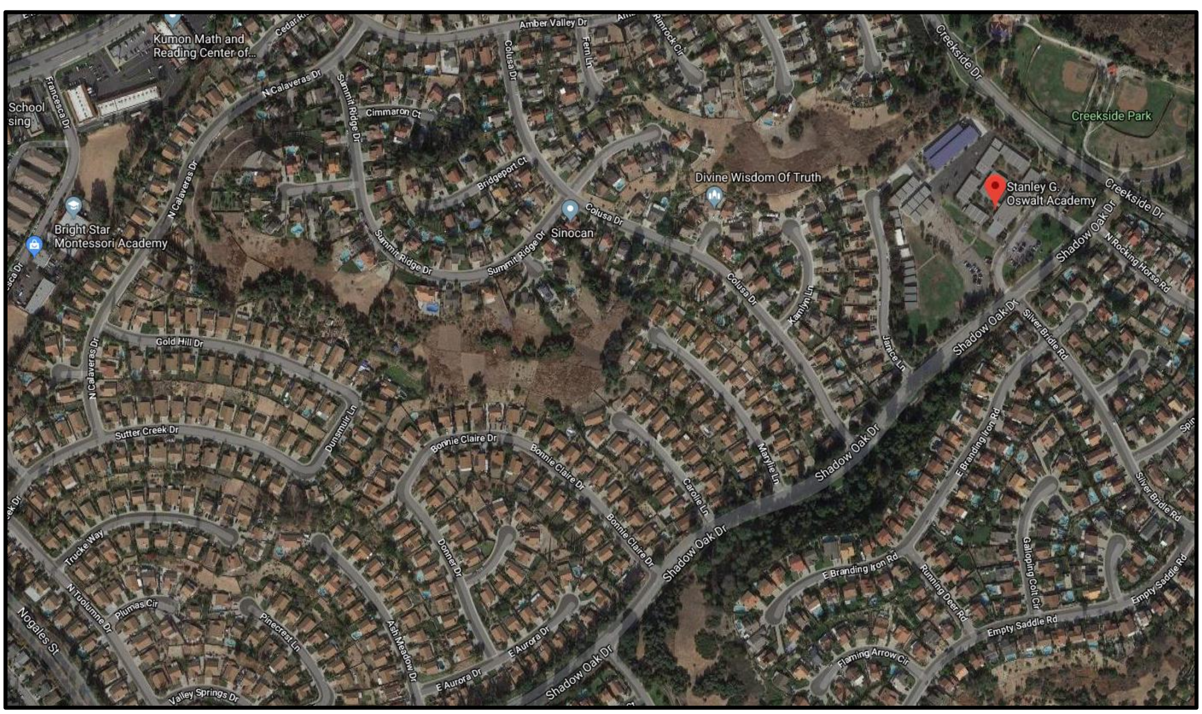
The Stanley G. Oswalt Academy is located at 19501 Shadow Oak Drive in Walnut, CA as shown on the Vicinity Map, Figure 1, which follows. The existing school, situated on a relatively level pad on the northwest corner of Shadow Oak Drive and Creekside Drive, was originally constructed in 1983. The site is bordered to the southeast and southwest by slopes which descend to Shadow Oak Drive and Janice Lane, respectively. The northeast and northwest portions of the property are bordered by slopes which ascend to Creekside Drive and adjacent single-family homes on Margaret Lane, respectively.

Existing school buildings are primarily located on the northerly portion of the property. The current athletic fields and parking lots are located on the south side of the school site. Several permanent and portable buildings, asphalt concrete pavement, concrete flatwork, surface vegetation, trees, and underground utilities occupy the site.

The school site was reportedly rough graded as part of the overall grading for Tentative Tract 36673 in 1980-1981 with geotechnical observation and testing provided by Geolabs Westlake Village (References 11 and 12). The school site was reportedly a cut/fill property with the northerly portion being primarily cut and the southerly portion fill. The descending slopes in the southern portion of the property are constructed of compacted fill and the ascending slopes in the northeast and northwest are a combination of cut and stability fill slopes.

It was documented that remedial removals were made prior to fill placement, slopes were sufficiently stabilized and fill was adequately compacted in accordance with City of Walnut standards at that time. It is our understanding that the geotechnical reports pertaining to the project were approved by the City of Walnut and the existing school was subsequently constructed.

Vicinity Map - Figure 1



PROJECT DESCRIPTION

We understand that the project comprises demolition of several existing buildings and construction of five, new buildings including a covered lunch area, play yard/field areas, parking lot and fire lane, and associated underground utilities and concrete flatwork. The proposed new buildings and associated improvements are shown on Plate A in Appendix A. Minor regrading is anticipated.

One-and two-story buildings with metal stud walls, panelized roof systems and concrete slab-on-grade floors are planned. Typical bearing wall and interior column loads on the order of 4 kips per lineal foot and 70 kips respectively, have been considered in preparation of this report. Revision of the recommendations may be necessary should actual loads exceed these values significantly.

Disposal of storm water via on-site infiltration is proposed, however, the test results obtained indicate the material has a design rate below that which is allowable and infiltration is not feasible.

SUBSURFACE CONDITIONS

Material Types and Condition

Subsurface conditions encountered during this investigation are described in more detail in Appendix A. Logs of the borings are presented on Plates A-1 to A-18 and show the subject site to be capped by a variable layer of fill/alluvium consisting of silty to clayey sands and sandy to silty clays which are underlain by sandstone and siltstone bedrock materials as indicated on the boring logs in Appendix A.

Expansion Potential

Based on the results of laboratory testing (Table 2, Appendix B), the sandy material is non-expansive (E.I. ≤ 20) as defined in section 1803A.5.3 of the 2016 California Building Code and does not require special consideration in design. However the clay material is expansive (E.I. =40) and does require special consideration in design. Recommendations for mitigating post-construction movement due to this characteristic have been incorporated into the design recommendations presented herein and are consistent with the requirements of Section 1808A.6.4 of the 2016 California Building Code.

Water-Soluble Sulfate

A soil sample was delivered to a state approved analytical laboratory for testing to evaluate water-soluble sulfate content. Based on the results of laboratory testing (Table 3, Appendix B) a negligible (S0) exposure category is indicated (ACI 318, Table 4.3.1).

Groundwater Conditions

Groundwater was not encountered in the borings at the time of drilling. The historic high groundwater depth is greater than 50 feet as indicated in the attached Geologic Hazards Report by Terra Geoscience (Appendix D).

GEOLOGIC CONDITIONS AND HAZARDS

Geologic conditions and hazards are addressed in the Geologic Hazards Report by Terra Geosciences (See Appendix D).

Slope Stability and Slope Setback

Conducting slope stability analysis was beyond our scope of work for this project however, the existing fill, cut and stability fill slopes have been in place since the property was developed in 1983 and are anticipated to remain stable. Bedrock bedding attitudes in this area have been documented as dipping to the northwest (References 10 and 11) which are neutral to the cut slope bordering the northeast edge of the property and into the natural/stability fill slope bordering the northwest portion of the property. These conditions are generally considered favorable for slope stability.

For proposed improvements, foundations for structural elements should be setback in accordance with the 2016 California Building Code Setback Details Sections 1808.7.1-1808.7. For descending slopes the setback is H/3 (where H is height of slope) but not to exceed 40 feet and for ascending slopes the setback is H/2 but not to exceed 15 feet. The setback is measured from the bottom of the footing to the face of the slope.

INFILTRATION TESTING

On November 19, 2018, two infiltration tests were conducted at the subject site at the approximate locations shown on the attached plan, Plate A. The areas have a grass surface. Location P-1 was in front of existing building T46 and location P-2 was west of Building T49.

Two test holes were drilled to a depth of 3 feet with an 8-inch-diameter spiral auger. A three-inch-diameter, slotted PVC pipe was centered in each hole and the annular space between the pipe and soil wall in the proposed infiltration zone (approximately 3 feet below grade) was filled with open-graded gravel.

The tests were conducted in accordance with the Boring Percolation Test procedure set forth in Reference 9.

The test data sheets (Plates P-1 and P-2, Appendix E) are attached and indicate a design infiltration rate of 0.04 inch/hour at a depth of 3 feet at location P-1 and P-2. The Storm Water manual requires that subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. The design rate at this site is significantly below the minimum allowable and therefore infiltration is not feasible at these locations.

Based on review of the test results we have concluded that disposal of stormwater using a properly designed on-site disposal system constructed approximately 3 feet below existing grade is not feasible.

CONCLUSION

Based on conditions encountered/established during this investigation, it is our conclusion that the currently planned construction is feasible from a geotechnical engineering standpoint provided the recommendations which follow are implemented during design and construction of the project.

RECOMMENDATIONS

Based on our evaluation of conditions encountered in the field exploration (Appendix A) and the analyses of laboratory test data (Appendix B), we recommend the following input for design and construction of the proposed project. Our recommendations are subject to confirmation of site conditions during grading and 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

In general, all grading should be carried out in accordance with applicable sections of the Standard Specifications contained in Appendix C except as noted in the paragraphs below.

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

Prior to major 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 plus three feet in each direction should be over-excavated to a depth of 2 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.

Soil throughout areas of new concrete flatwork and pavement should be similarly processed to a total depth of 2 feet.

Site material that is free of objectionable amounts of organic matter and/or debris will be suitable for fill material. If additional, imported soil is required it should be similar to site material and should be approved by the geotechnical engineer for expansion, corrosivity, 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.

Fill material, if necessary, should be spread in thin lifts and moisture conditioned and compacted as indicated above.

Seismic Design

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

Foundation and Floor Slab Design

The site materials are expansive (Expansive Index >20) and pre-saturation during grading is recommended to mitigate this condition. We have provided this recommendation for pre-saturation, which is a stabilization procedure permitted by 2016 CBC Section 1808A.6.4.

Conventional spread footings for one-story structures should be at least 12 inches wide and embedded at least 18 inches below the lowest adjacent grade and founded on compacted fill material and may be designed using an allowable, net, dead load plus live load soil bearing pressure of 1,800 pounds per square foot. Conventional spread footings for two-story structures should be at least 15 inches wide and embedded at least 24 inches below the lowest adjacent grade and founded on compacted fill material and may be designed using an allowable, net, dead load plus live load soil bearing pressure of 2,000 pounds per square foot. A one-third increase in bearing may be assumed for short duration wind or seismic loading in combination with vertical loads.

For the purposes of resisting lateral forces, a passive soil pressure of 250 pounds per square foot per foot of depth may be used in design. A coefficient of friction of 0.40 may be used for concrete placed on approved compacted fill. These values may be combined without reduction. Appropriate safety factors must be used.

It is recommended that continuous footings be reinforced with one No. 4 bar, top and bottom. Reinforcement of pad footings, if any, will be governed by structural requirement.

The at-grade floor slabs should be a nominal 4 inches thick and reinforced with No. 4 bars spaced 24 inches apart in both directions.

A moisture vapor retarder installed in accordance with the manufacturer's instructions should be provided below the building slabs.

The slab subgrade should be pre-soaked to 1.3x optimum moisture content to a depth of 12 inches prior to placement of the moisture-vapor barrier.

It is recommended that the geotechnical engineer observe and/or test the foundation excavations in order to verify compliance with the recommendations of the report.

Settlement

Foundation settlement (total and differential) should not exceed 1/2 inch and 1/4 inch, respectively, and will not require special consideration in design provided any disturbed material is removed or compacted as previously recommended.

Retaining Wall Design/Construction

Retaining walls should be founded in compacted fill or bedrock. It is recommended that footings be bottomed at least 24 inches below the lowest adjacent grade. An allowable, net, bearing pressure of 2,000 pounds per square foot may be used for footings at least 24 inches wide and bottomed at the recommended minimum depth. This value may be increased 500 pounds per square foot for each additional foot of width up to a maximum value of 3,000 pounds per square foot.

A one-third increase in bearing may be assumed when considering transient loads due to wind pressure or seismic forces in combination with vertical loads.

Walls backfilled with site materials and free to rotate may be designed using an active lateral earth pressure of 40 pounds per square foot per foot or depth for level backfill and 60 pounds per square foot per foot or depth for 2:1 backfill. Retaining walls restrained from rotating may be designed for an at-rest lateral earth pressure of 60 pounds per square foot/foot of depth for level backfill and 80 pounds per square foot per foot or depth for 2:1 backfill. These values include lateral pressure resulting from soil expansion.

The site is in Seismic Category E and additional earthquake loading on retaining walls higher than 6 feet must be considered. In this regard, an inverted triangular load of 21.9 H^2 acting at 0.6H (H=wall height, feet) above the bottom of the wall may be used.

Passive resistance may be determined using an equivalent fluid density of 250 pounds per cubic foot. A friction coefficient of 0.40 may be used. An appropriate safety factor should be applied.

The retaining wall excavation should be laid back at an inclination no steeper than 1/2H:1V or stability equivalent. Surcharge loading such as stockpile soil, construction materials and/or construction equipment should be kept at least 15 feet away from the top of the slope.

A subdrain consisting of a 4-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 140N geofabric or similar product. Other specialty wall drains (subdrains) can be used in lieu of the pipe and gravel with the approval of the geotechnical engineer. The location of the subdrain outlet should be determined by the civil engineer. Unless water from the subdrain can be drained by gravity flow, a sump/pump will be required.

Waterproofing should be provided if the possibility of efflorescence developing on the wall face is unacceptable.

A 12-inch-wide blanket of granular soil with a sand equivalent of at least 30 should be provided at the backs of the walls and should extend to grade in protected areas, i.e. under floor slabs and/or exterior flatwork. In unprotected areas, the top 36 inches of backfill should consist of site material compacted to at least 90 percent of maximum density to impede surface water infiltration.

Approved site material may be used for wall backfill. The material should be placed in thin lifts, moisture conditioned to near-optimum moisture content and mechanically compacted to at least 90% of the maximum density determined by ASTM Test Method D1557. Continuous benches at least one foot wide should be cut into the existing firm soil during backfilling to interlock the materials.

Light Standards/Signs/Poles

Light Standards/Signs/Poles if included in the design will require piers constructed in fill or bedrock as shown on the attached boring logs. It is recommended that soil conditions be confirmed by a member of our staff during drilling of the pier holes.

Our recommendations/comments pertaining to design and construction of the piers are as follows:

1.1 An allowable, net, vertical bearing pressure of 4,000 psf is applicable to design of 30-inch-diameter piers at least 4 feet deep.

1.2 An allowable lateral bearing pressure of 250 psf/ft., limited to 3,000 psf, is applicable.

1.3 Groundwater was not encountered at the time of drilling the borings and is not expected to be of concern during pier construction.

1.4 A negligible amount of water-soluble sulfate is indicated for the prevalent surface material and special sulfate-resistant concrete will not be required on this project. The exposure class (ACI 318-11, Table 4.2.1.) is S0. Concrete may contain Type II cement and should comply with Section 1904A of the 2016 CBC and ACI 318-11, Table 4.3.1.

1.5 Minor caving/raveling may occur. Concrete should be placed as soon as possible to minimize this occurrence.

Pavement Structural Sections

It is preliminarily recommended that automobile driveways/parking lots (T.I. =4) be paved with a minimum of 3-inches of asphalt concrete and 6-inch Class II aggregate base placed on a minimum of 2-feet of compacted soil in accordance with the California Highway Design Manual, Chapter 630. It is recommended that truck/bus driveways/parking lots (T.I. =6) be paved with a minimum of 4-inches of asphalt concrete and 7-inch Class II aggregate base placed on a minimum of 2-feet of compacted soil in accordance with the California Highway Design Manual, Chapter 630.

Consideration should be given to increasing the asphalt concrete thickness by 1/2 inch in impact areas (drive entrances/exits and at trash enclosures) to avoid premature distress at these locations. The stated thicknesses are minimum; the paving contractor must exercise care to ensure against thickness deficiency.

Unless otherwise specified by others, aggregate base and asphalt concrete should conform to Standard Specifications for Public Works Construction (Green Book) Sections 200-2 and 203. Aggregate base should be compacted to at least 95 percent relative compaction based upon the maximum density determined by ASTM D1557. The Contractor should follow the Grading Specifications presented in Appendix C.

The pavement section discussed above is considered preliminary and should be verified subsequent to rough grading.

Concrete Flatwork

Miscellaneous flatwork should be a nominal 4-inches thick, reinforced at mid-depth with No. 4 bars at 24-inches on center, each way, and provided with adequate control joints. Low slump concrete should be used for all flatwork to further minimize cracking.

It should be noted that due to the expansive characteristic of the site material and normal concrete shrinkage some minor cracking of the miscellaneous flatwork may occur. Additional reinforcement beyond that recommended herein and careful control of concrete slump would be beneficial in reducing such cracking. Also, it is very important that all control joints be caulked and properly maintained to inhibit infiltration of surface water into the soil and thereby minimize expansion.

Concrete Quality

A negligible amount of water-soluble sulfate is indicated for the prevalent surface material and special sulfate-resistant concrete will not be required on this project. The exposure class (ACI

318-11, Table 4.2.1) is S0. Based on this test result concrete may contain Type II cement (Section 1904.2 of the 2013 CBC and ACI 318, Section 4.3, Table 4.3.1). These recommendations should be verified during construction.

Temporary Excavations/Caving

Due to the presence of dense, cohesive soil/bedrock, caving is not expected to be a major concern during construction although minor raveling may occur. The regulations of Cal/OSHA should be followed during performance of all subsurface work and concrete should be placed as soon as possible to minimize this occurrence.

Utility Trench Backfills

Backfill for any trenches associated with this project should consist of site material (the use of imported sand is not recommended) that must be adequately compacted to preclude detrimental settlement. It is recommended, therefore, that backfills for all excavations associated with the project should consist of site material placed in appropriate lifts, moisture conditioned to 2% to 4% above optimum moisture content and compacted to 90% relative compaction based on the maximum dry density obtained in the laboratory in accordance with ASTM test method D1557.

Plan Review

It is recommended that final project plans, details and specifications be submitted to this office for geotechnical review for compliance with the findings and recommendations of this report. Additional recommendations can then be provided if necessary.

Observations and Testing

Grading and compaction operations, foundation construction and trench backfills should be observed and tested by members of our staff so that anticipated soil conditions can be confirmed and the recommendations contained herein validated. If deemed necessary as a result of changed conditions supplemental recommendations may then be provided. Results of those observations and tests should be provided in the final report which should include a statement by the geotechnical engineer concerning the adequacy of the completed work.

Pre-Grade/Construction Meeting

A pre-grade/construction meeting should be attended by the owner's representative, members of the design team, grading contractor, city/county inspector, and a representative from HGEI at the site to review the findings and recommendations of this report and project plans and specifications prior to starting work on the project.

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 observations and our best estimation of project conditions and requirements as indicated by evaluation of presently available information and data. No further warranty, express or implied, is intended by issuance of this report.

This investigation did not include sampling, field measurements or laboratory tests for the presence of any toxic/hazardous substances in the earth materials at the site. However, this does not imply that the site is subject to any unusual geologic, seismic or environmental hazard.

Any unanticipated condition encountered in the course of grading and/or construction should be brought to the attention of the geotechnical engineer for evaluation prior to proceeding with the work.

This report has been developed for the sole use of the client and/or clients authorized representative. These conclusions and recommendations should be verified by a qualified geotechnical engineer based in part upon additional subsurface information obtained during grading and/or foundation construction. No part of the report should be taken out of context, nor utilized without full knowledge and awareness of its intent.

This report is issued on condition that HGEI will be retained to observe the grading, backfilling and foundation construction operations. If another firm provides this service then that firm must review and accept this report, or provide alternate recommendations, and assume responsibility for the project. This report will be valid for a period of one year form date of issue and will then require updating.

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REFERENCES

1. California Department of Conservation, Division of Mines & Geology, 1998, Seismic Hazard Zone Report for the Baldwin Park 7.5 Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 022.

2. California Department of Conservation, California Geological Survey, State of California, Seismic Hazard Zones, Earthquake Zones of Required Investigation, Baldwin Park 7.5 Minute Quadrangle, March 25, 1999, Scale 1:24,000.

3. California Geological Survey, 2011, Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for Public Schools, Hospitals, and Essential Service Buildings dated January 1, 2011.

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

5. California Department of Conservation, Division of Mines and Geology, 1997, Tan, Siang S., "USGS Geologic Map of the Baldwin Park 7.5' Quadrangle Los Angeles County, California: A Digital Database", DMG Open-File Report 98-30, dated 1997.

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7. USGS, Earthquake Hazards Program, 2018, U.S. Seismic Design Maps, https://earthquake.usgs.gov/designmaps/us/application.php, updated 3/19/18.

8. Blake, Thomas F., 2000, FRISKSP (Version 4.00), EQFAULT and EQSEARCH (Version 3.00), Computer Programs for calculating the site to fault distances, Deterministic peak horizontal ground accelerations for a Maximum Magnitude Earthquake, and historic seismicity for an area from selected known faults in the southern California region (Rev. 2000).

9. Administration Manual, GS200.2 6/30/17 County of Los Angeles Department of Public Works, Geotechnical Materials Engineering Division, Guidelines for Design, Investigation, and Reporting, Low Impact Development Stormwater Infiltration.

10. Geolabs Westlake Village, Final Supervised Compacted Fill and Geological Report, Lots 1-50, Tract 39519, Portion of Tentative Tract #36673, City of Walnut, CA. W.O. 6153-19, dated 4/24/1981.

11. Geolabs Westlake Village, Final Supervised Compacted Fill and Geological Report, Proposed School Site, Portion of Tentative Tract #36673, City of Walnut, CA. W.O. 6153-SS, dated 9/24/1981.

12. Ziemba + Prieto Architects, Stanley G. Oswalt Academy, 19501 Shadow Oak Drive, Walnut, CA 91789, Sheets 1, 2.1, 2.2, 3, 4.1, 4.2, 5.1, 5.2, 6, AS1.1, Job No. 150703, dated 8/8/2018.

13. Seaboard Engineering Company, Boundary, Site & Topographic Survey, Oswalt Elementary School, 19501 Shadow Oak Drive, Walnut, CA 91789, Job No. 15-101, dated 4/11/2016.

APPENDIX A

FIELD INVESTIGATION

FIELD INVESTIGATION

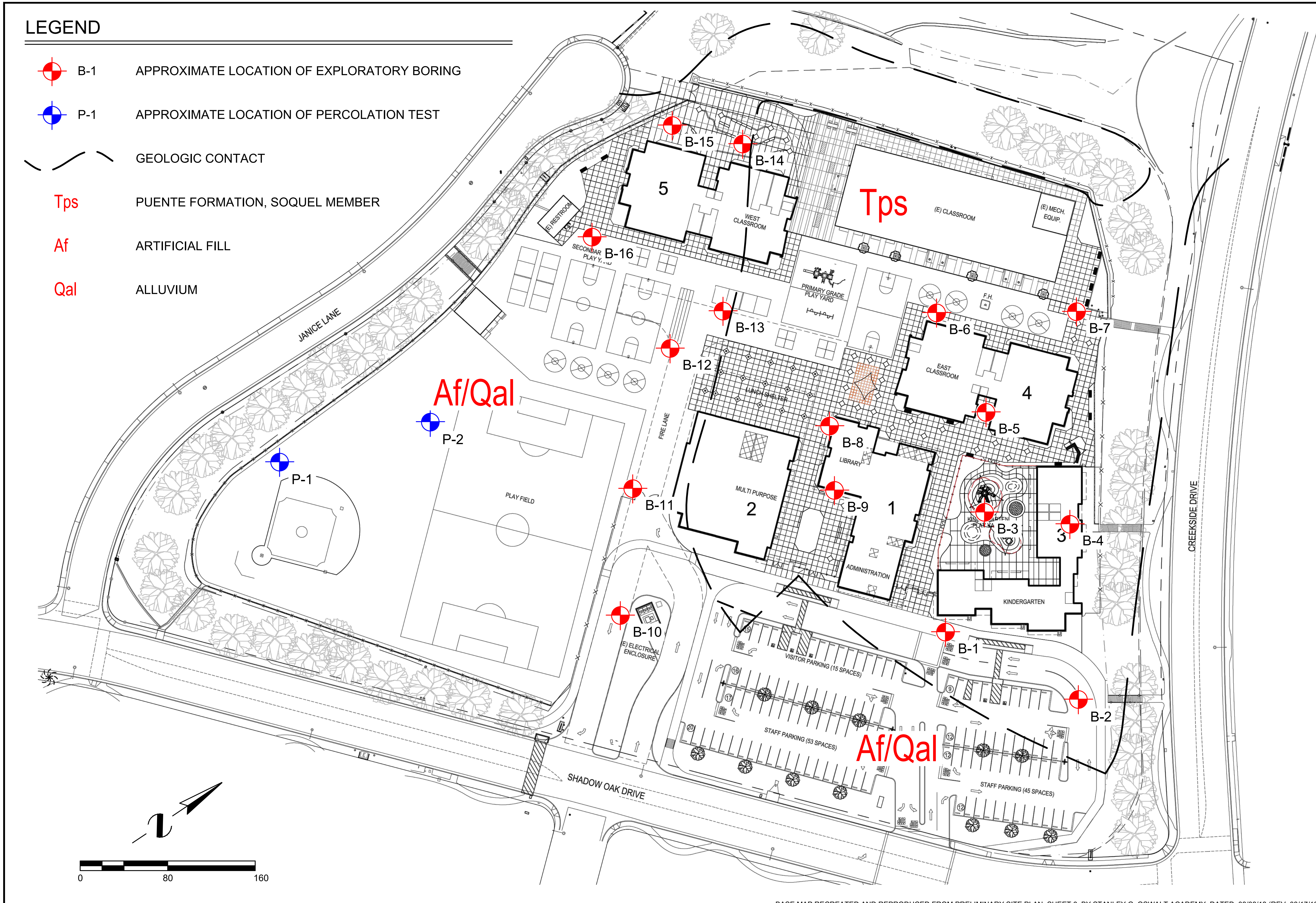
HGEI conducted a field investigation of the subject site on November 17, 2018 consisting of drilling, sampling and logging sixteen small diameter hollow-stem auger exploratory borings (B-1 to B-16) to depths of up to 16 feet and two additional borings (P-1 and P-2) to depths of 3 feet for infiltration testing. These hollow-stem auger borings were drilled using a CME-75, truck-mounted drilling rig, equipped with an automatic hammer (140#/30"), and 8-inch-diameter, continuous-flight auger. The boring locations are indicated on Plate A and the logs of the exploratory borings are presented on Plates A-1 thru A-18. The descriptions represent the prevalent soil types and slightly different material types may be present within the major groupings. Also, the transition from one soil type or condition to another may be gradual rather than abrupt as implied, and differing conditions may exist in unexplored areas.

Unified Soil Classification System Classification Criteria/Symbols are presented on Plate A-19.

A representative of the geotechnical engineer observed the field work, collected samples for transportation to our geotechnical laboratory, and prepared field logs by visual/tactile examination of the materials. Core samples were obtained from the hollow-stem auger borings at discreet intervals using a modified California split-spoon sampler loaded with 2.42" I.D. x 1"-long, thin-wall, brass rings. In addition to the core samples, large bulk samples of the earth materials were collected. Samples were placed in plastic bags immediately upon removal from the sampler to conserve moisture and labeled for identification.

The borings were backfilled with auger cuttings immediately upon completion of sampling.

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



HARRINGTON
GEOTECHNICAL • ENGINEERING • INC.
1590 N. BRIAN STREET, ORANGE CA 92867 T: (714) 637-3093 F: (714) 637-3096

Project: NEW SCHOOL BUILDINGS & IMPROVEMENTS
ROWLAND UNIFIED SCHOOL DISTRICT
Address: 19501 Shadow Oak Drive, Walnut CA 91789
HGEI Project No.: 118-01-3763

Sheet Title: SITE PLAN

DRAWN BY: BBC
CHECKED BY: JLW
DATE: 12/14/18

PLATE A

LOG OF BORING B-1

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					4" AC PAVEMENT/5" AGGREGATE BASE			
						ARTIFICIAL FILL (Af): SILTY SAND (SM), gray	20 50/5"	105	14
						BEDROCK (Tps): SANDSTONE, white/gray, fine grained, very moist to moist, dense			
	5					@5' lenses of gray SILTSTONE, iron stained	22 50/2"	92	26
	10						50/6"	87	7

Completion Depth:	11.0	Remarks: Groundwater was not encountered at time of drilling. Caving did not occur.
Date Boring Started:	11/17/18	
Date Boring Completed:	11/17/18	
Logged By:	JR	
Drilling Contractor:	OWD	

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

Harrington Geotechnical Engineering, Inc.

PLATE A-1

LOG OF BORING B-2

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					1" GRASS/6" TOPSOIL			
						BEDROCK (Tps): SANDSTONE interbedded with SILTSTONE, light brown/gray/orange, very moist to moist, dense, fine grained	70	89	29
	5						35 50/3"	92	16
	10						21 50/3"	91	18

Completion Depth:	11.0	Remarks:
Date Boring Started:	11/17/18	Groundwater was not encountered at time of drilling. Caving did not occur.
Date Boring Completed:	11/17/18	
Logged By:	JR	
Drilling Contractor:	OWD	

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-3

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					0-3" GRASS/TOPSOIL			
						BEDROCK (Tps): SANDSTONE, orange/gray, moist, dense, fine grained, trace roots			
							42	107	5
	5					@ 5' SANDSTONE, thinly interbedded with SILTSTONE, gray/orange, moist, dense, fine grained	48 50/3"	97	14
							50/6"	104	17
	10						50/6"	100	14

Completion Depth:	11.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	MS
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-4

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					**BEDROCK (Tps):** SANDSTONE, gray/white, damp to moist, dense, fine grained			
							32	115	3
	5						28 50/4"	98	5
						@ 7.5' SANDSTONE with occasional thin SILTSTONE interbeds, laminated gray/white/orange, fine grained, moist, hard	80	92	8
	10						18 50/5"	93	4
							28 50/4"	94	14
	15						50/6"	98	8

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	MS
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-5

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					**BEDROCK (Tps):** SILTSTONE thinly interbedded with SANDSTONE, gray/orange, damp to moist, hard, fine grained			
							42 50/1"	102	18
	5					@5' SANDSTONE, gray/white, damp to moist, dense hard	48 50/1"	95	5
						@7.5' SILTSTONE thinly interbedded with SANDSTONE, gray/white, moist, hard, fine grained	42 50/3"	110	5
	10					@10' orange/gray	44 50/1"	84	13

Completion Depth:	11.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	MS
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

PLATE A-5

LOG OF BORING B-6

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					4" AC PAVEMENT/ 9" AGGREGATE BASE			
						BEDROCK (Tps): SANDSTONE thinly interbedded with SILTSTONE, light gray/dark gray, very moist, dense, fine grained	36	85	24
	5					@5' SANDSTONE thinly interbedded with SILTSTONE, light gray/dark gray/orange, moist, dense, fine grained, iron stained	65	88	17
							20 40/3"	102	5
	10								
	15						30 45/3"	96	5

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-7

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					5" AC PAVEMENT			
						BEDROCK (Tps): SANDSTONE, light gray/yellow brown/orange with occasional SILTSTONE interbeds, gray, damp to moist, dense, fine grained, iron oxide staining			
							58	101	3
	5						76	89	19
	10						80	93	10
	15						58	95	20

Completion Depth: 16.0
Date Boring Started: 11/17/18
Date Boring Completed: 11/17/18
Logged By: JR
Drilling Contractor: OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

PLATE A-7

LOG OF BORING B-8

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					**BEDROCK (Tps):** SANDSTONE, light gray with occasional SILTSTONE interbeds, gray, moist, dense, fine grained, fractures, iron oxide staining			
							54	86	18
	5						38	101	9
							62	103	10
	10						24 50/6"	102	7
								92	7
	15							91	18

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	MS
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

PLATE A-8

LOG OF BORING B-9

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					3" GRASS			
						BEDROCK (Tps): SANDSTONE, light gray/yellow brown with occasional SILTSTONE interbeds, gray, moist, dense, fine grained, iron oxide staining, fractures			
							18 50/6"	105	18
	5						70	101	13
							84	97	21
	10						88	104	7
							50/6"	90	6
	15								

Completion Depth:	15.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	MS
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-10

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					4" AC PAVEMENT/2" AGGREGATE BASE			
						ARTIFICIAL FILL (Af): SANDY CLAY (CL) TO CLAYEY SAND (SC), mottled brown/gray, moist, stiff, fine grained	30	107	14
	5					**BEDROCK (Tps):** SANDSTONE, with thin interbeds of SILTSTONE, gray/orange, moist, dense, fine grained, iron stained	27 54/4"	100	12
	10						57	99	15

Completion Depth:	11.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-11

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					3" AC PAVEMENT/4.5" AGGREGATE BASE			
						ARTIFICIAL FILL (Af): SANDY CLAY (CL) to CLAYEY SAND (SC), light brown/dark brown/gray, moist, stiff	31	102	18
	5					@ 5' SILTY CLAY (CL), dark brown, moist, stiff	30	106	15
	10					**ALLUVIUM (Qal):** SILTY CLAY (CL), medium brown, moist, stiff, caliche stringers	14	107	13
	15					**BEDROCK (Tps):** SANDSTONE, gray/white, moist, dense, fine to medium grained	61	108	8

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-12

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					4" AC PAVEMENT/4" AGGREGATE BASE			
						ARTIFICIAL FILL (Af): CLAYEY SAND (SC) TO SANDY CLAY (CL), mottled light brown/dark brown/gray, moist, stiff	27	104	16
							16	105	15
	5					**BEDROCK (Tps):** SANDSTONE, white/yellowish brown, interbedded with SILTSTONE, gray, moist, dense, fine grained	15 50/5"	89	27
							77	79	9
	10								
	15						50/3"		

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-13

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					4" AC PAVEMENT/6" AGGREGATE BASE			
						ARTIFICIAL FILL (Af): SILTY SAND (SM), yellow brown, moist, medium dense, fine to medium grained	31	99	14
	5					**BEDROCK (Tps):** SANDSTONE, yellow brown, moist, dense, fine to medium grained	66	103	6
						@7.5' intermittent SILTSTONE layers	53	91	12
	10								
	15					@15' SILTSTONE, gray brown, interlayered with SANDSTONE, orange, moist, hard iron oxide staining	35	82	32

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

PLATE A-13

LOG OF BORING B-14

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					2" GRASS			
						ARTIFICIAL FILL (Af): SANDY CLAY (CL), red brown, moist, stiff, some organics	24	110	15
	5					**ALLUVIUM (Qal):** CLAYEY SAND (SC), red brown, moist, medium dense, trace pin pores	25	99	15
						@7.5' iron staining			
	10					**BEDROCK (Tps):** SANDSTONE, gray/orange, moist, dense, fine grained	62	93	12
						@12' SANDSTONE interbedded with SILTSTONE, dark gray/light gray/orange, moist, hard, fine grained	30	92	21
	15						52	89	25

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING B-15

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					2" GRASS/2"-4" TOPSOIL			
						ARTIFICIAL FILL (Af): SILTY CLAY (CL), mottled light brown/dark brown/orange, moist, stiff, trace bedrock fragments, trace roots in upper 3'	19	106	9
	5						27	97	11
						ALLUVIUM (Qal): SANDY CLAY (CL), reddish brown, moist, stiff, slight porosity	22	102	12
	10						22	109	12
						CLAYEY SAND (SC), moist, medium dense, fine grained, slight pin porosity	25	97	12
	15						39	103	10

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

PLATE A-15

LOG OF BORING B-16

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					1" GRASS/2" TOPSOIL			
						ARTIFICIAL FILL (Af): SILTY SAND (SM) to SANDY SILT (ML), yellowish brown, damp to moist, stiff, trace bedrock fragments	40	98	9
	5						36	100	10
						@ 7.5' SANDY CLAY (CL) to CLAYEY SAND (SC), mottled brown/tan/orange, moist, stiff	19	86	22
	10					@10' layer of SILTY CLAY, dark brown	34	82	27
						BEDROCK (Tps): SILTSTONE underlain by SANDSTONE, light gray/yellow, moist, dense	22 50/3"	91	15
	15						60	96	10

Completion Depth:	16.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING P-1

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					**ARTIFICIAL FILL (Af):** SANDY SILT (ML) to SILTY SAND (SM), light yellow brown			

Completion Depth:	3.0	Remarks:
Date Boring Started:	11/17/18	Groundwater was not encountered at time of drilling. Caving did not occur.
Date Boring Completed:	11/17/18	
Logged By:	JR	
Drilling Contractor:	OWD	

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

Harrington Geotechnical Engineering, Inc.

LOG OF BORING P-2

Project:	Stanley G. Oswalt Academy	Surface Elev.:	Grade
Job No.:	18-01-3763	Top of Casing Elev.:	N.A.
Location:	19501 Shadow Oak Drive, Walnut, CA	Drilling Method:	HSA
Coordinates:		Sampling Method:	Core Barrel

Elevation, feet	Depth, feet	Sample No.	Sampler Graphics	Symbol / USCS	Recovery %	MATERIAL DESCRIPTION	Blow Counts	Dry Unit Weight, lb/cu ft.	Water Content %
	0					2" GRASS/2" TOPSOIL			
						ARTIFICIAL FILL (Af): SILTY CLAY (CL), light yellowish brown			

Completion Depth:	3.0
Date Boring Started:	11/17/18
Date Boring Completed:	11/17/18
Logged By:	JR
Drilling Contractor:	OWD

Remarks:
Groundwater was not encountered at time of drilling. Caving did not occur.

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	GRAVEL AND GRAVELLY SOILS	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
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	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
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SAND, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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

HARRINGTON
GEOTECHNICAL • ENGINEERING • INC.
1590 NORTH BRIAN STREET, ORANGE, CA. 92867 T: (714) 637-3093 F: (714) 637-3096

DRAWN BY: BBC	USCS	CHECKED BY: MVD
SOIL CLASSIFICATION CHART		
Dec. 2018	HGEI Project No. 18-01-3763	PLATE A-19

APPENDIX B

LABORATORY TEST RESULTS

LABORATORY PROCEDURES & TEST RESULTS

The samples collected during the field investigation were examined and classified by the project geotechnical engineer in the laboratory using the visual/tactile method 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 & D7263-09)

Field Moisture contents were determined for all samples. The core samples were trimmed and weighed and the dry units of the material calculated. Moisture and unit weight data are presented on the boring logs in Appendix A.

Compaction Tests (ASTM D1557-12ε1)

Compaction tests were performed on surface soil samples 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 4829-11)

Expansion Index Tests were conducted samples considered representative of the prevalent surface/near-surface soil site material to establish data on which to base recommendations for foundation and at-grade floor slab design. The test results are presented in Table 2.

Water-Soluble Sulfate Tests (EPA 300.0)

In order to determine the proper cement type for the site, the amount of water-soluble sulfate present in selected samples of the surface material were determined. The test results are presented in Table 3.

Particle Size by Hydrometer Analyses (ASTM D422M)

Hydrometer analyses were performed on selected samples to aid in proper classification of the materials. The results are presented in Table 4.

Consolidation Tests (ASTM D2435/D2435 M-11)

Consolidation tests were performed on several samples to determine the magnitude and rate of consolidation of the soil when subjected to incrementally applied controlled-stress loading. Water was added to the samples during the test to determine the effect of increased moisture. Refer to Plates C-1 thru C-5 for results.

Direct Shear Tests (ASTM D3080/D3080 M-11)

Direct Shear tests were performed on undisturbed specimens to determine the static strength of the soil. The tests were performed at increased moisture contents and at various confining pressures using a displacement rate of 0.0125 in./min. to establish peak and ultimate strength parameters under adverse conditions of moisture. The shear test results (graph) are presented on Plates D-1 through D-5.

TABLE 1
Compaction Test Results (ASTM D1557-12ε1)

Sample Id.	Classification	Maximum Dry Density (pcf)	Optimum Moisture (%)
B-1 @ 0′-3′	Silty Clay (CL), brown	123.0	11.0
B-5 @ 0′-3′	Silty Sand (SM), brown	112.0	14.0
B-15 @3′ 4″	Silty Sand (SM) yellow	117.0	14.0

TABLE 2
Expansion Index Test Results (ASTM D4829-11)

Sample ID	Moisture Content (%)		Dry Density (pcf)		Calculated Expansion Index	Expansion Potential
	Initial	Final	Initial	Final		
B-5 @ 0′-3′	10.8	24.6	96.4	96.5	0	Non-expansive
B-15 @ 3′ 4″	10.6	22.2	104.0	99.8	40	Low

TABLE 3
Water-Soluble Sulfate (EPA 300.0)

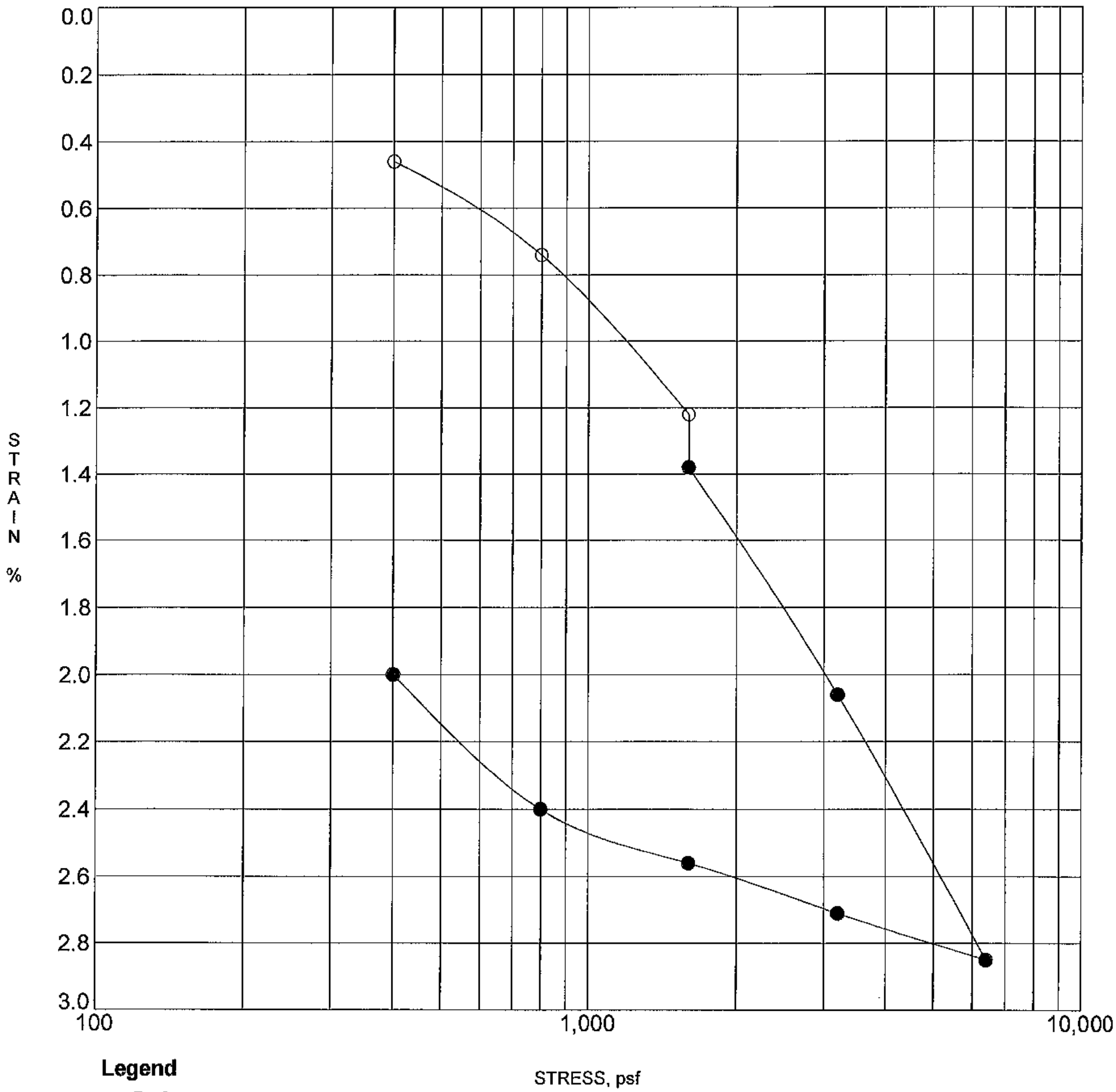
Sample ID	Water-Soluble Sulfate (%)
B-2 @ 0′-3′	0.013
B-5@ 0′-3′	0.0019
B-15@ 3′ 4″	0.016

TABLE 4 Hydrometer Test Results (ASTM D422M)				
Boring No.	**Depth (ft)**	**% sand**	**% silt**	**% clay**
P-1	0-3′	87	6	7
P-2	0-3′	69	10	21

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. Note that prolonged storage will result in sample degradation and may render them unsuitable for testing.

0-0-0



Legend

○ Before water

● After water

	Specimen Identification		Classification	DD	MC%
●	**B-1**	**5.0**	**SILTSTONE BEDROCK**	**96**	**23**
⊠					
▲					
★					
⊙					
✿					

PROJECT **Stanley Oswalt Academy - 19501 Shadown Oak Drive, Walnut, CA**

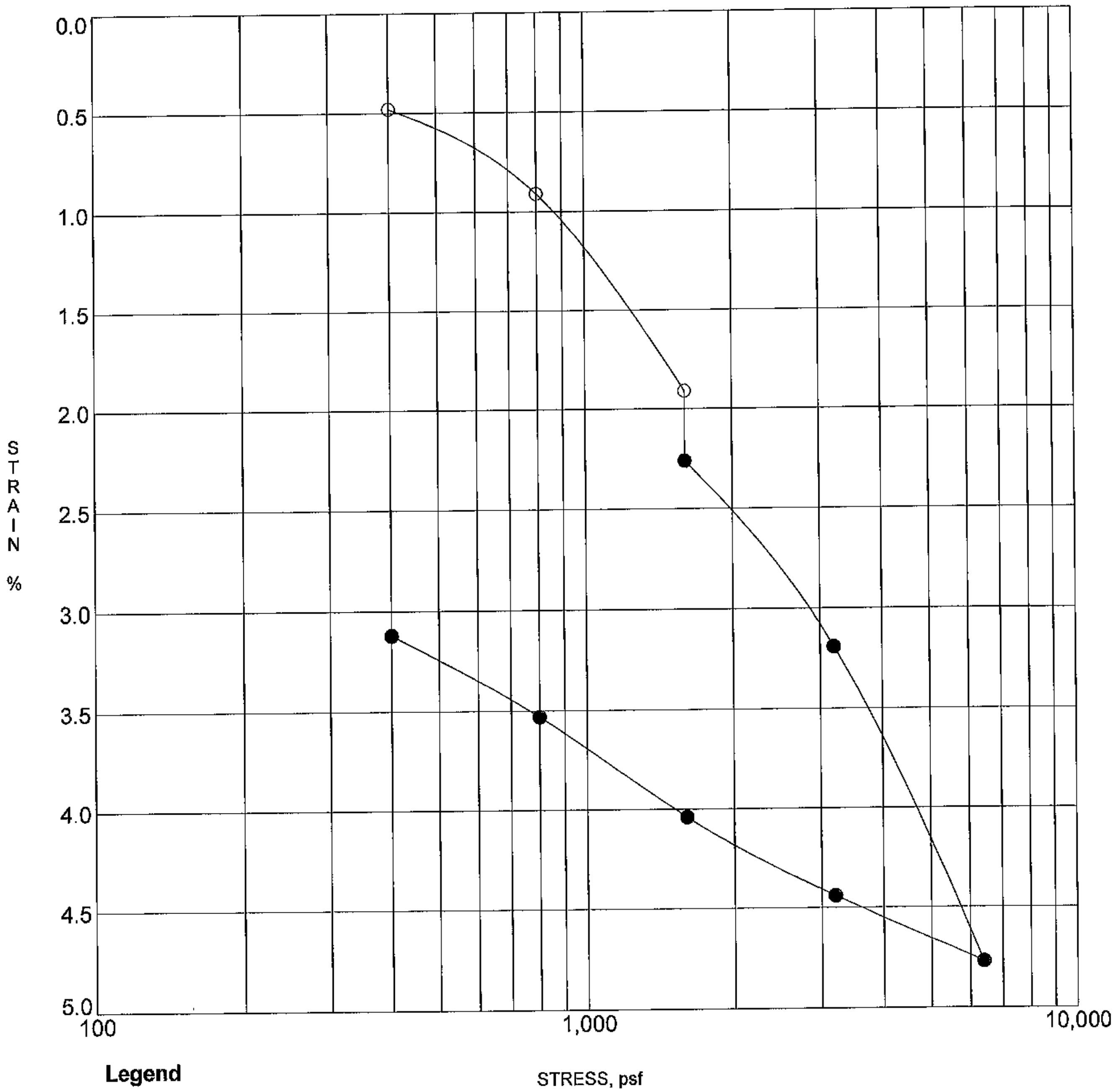
JOB NO. **18-01-3763**

DATE **12/13/18**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE C-1



Legend

○ Before water

● After water

	Specimen Identification		Classification	DD	MC%
●	**B-5**	**5.0**	**SANDSTONE BEDROCK**	**89**	**28**
⧗					
▲					
★					
⊙					
✪					

PROJECT **Stanley Oswalt Academy - 19501 Shadown Oak Drive, Walnut, CA**

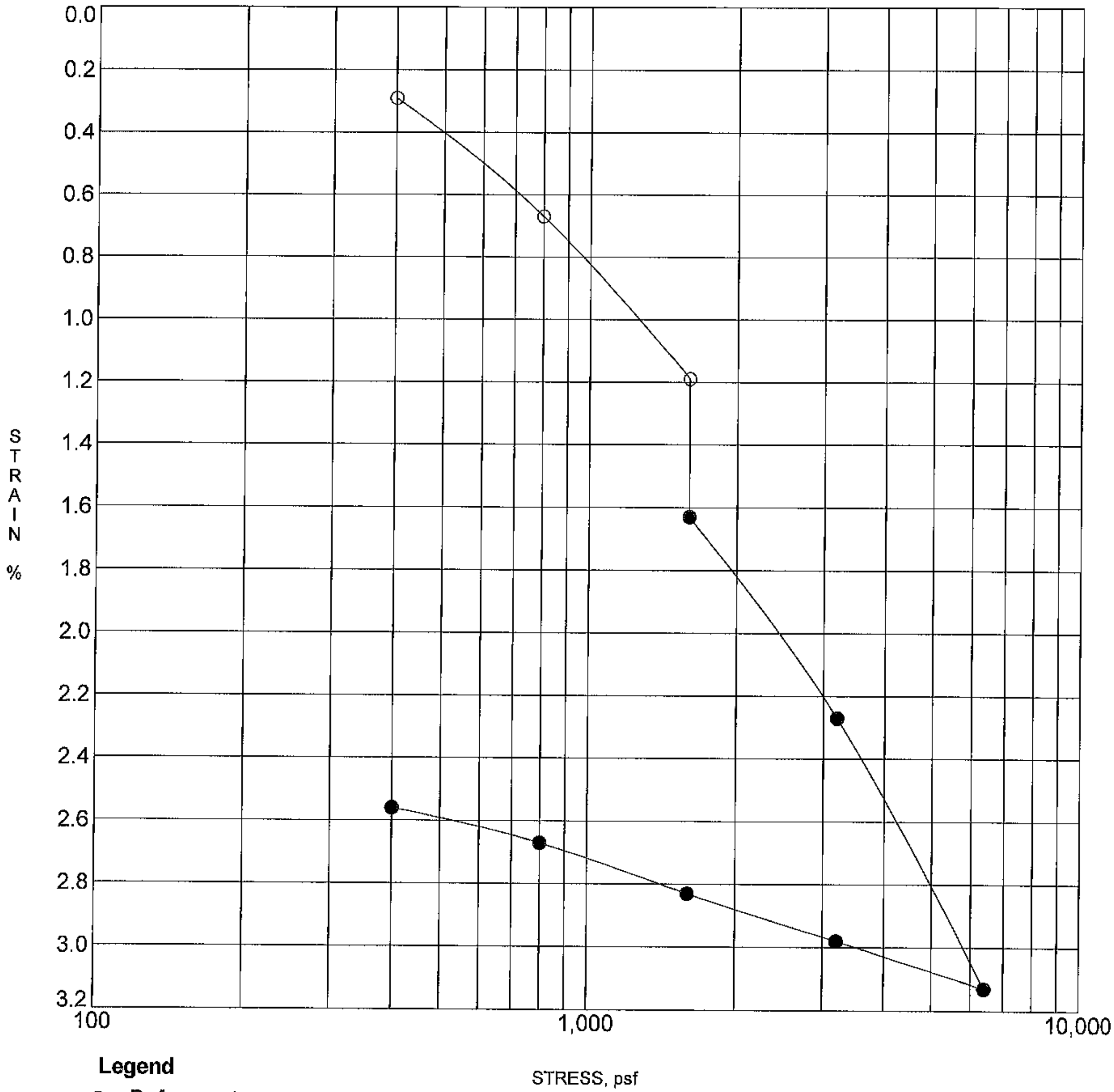
JOB NO. **18-01-3763**

DATE **12/13/18**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE C-2



Legend

○ Before water

● After water

	Specimen Identification		Classification	DD	MC%
●	**B-10**	**5.0**	**SANDSTONE/SILTSTONE BEDROCK**	**103**	**19**
⊠					
▲					
★					
⊙					
✿					

PROJECT **Stanley Oswalt Academy - 19501 Shadown Oak Drive, Walnut, CA**

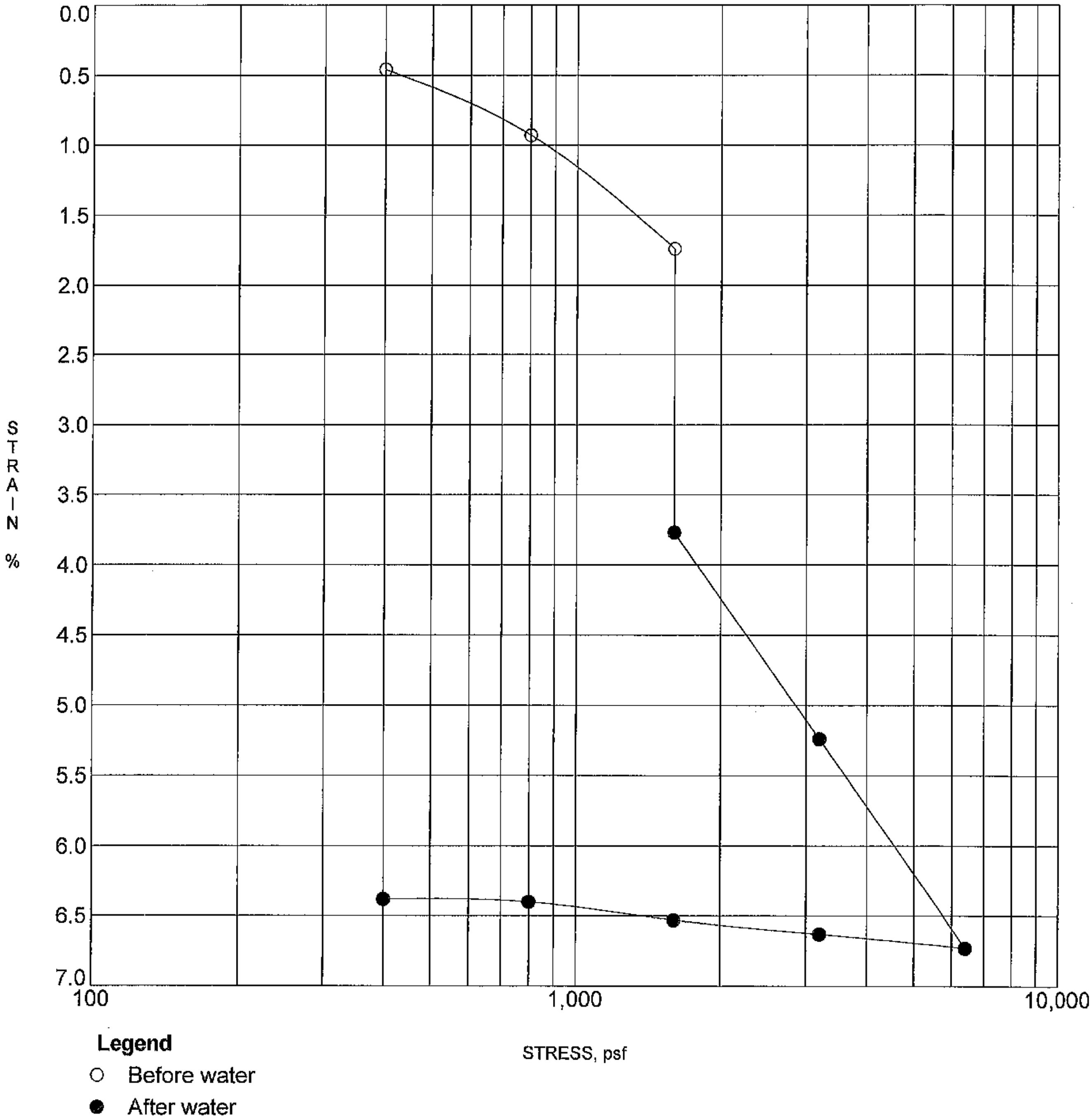
JOB NO. **18-01-3763**

DATE **12/13/18**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE C-3



	Specimen Identification		Classification	DD	MC%
●	**B-13**	**5.0**	**SANDSTONE BEDROCK**	**100**	**19**
⊠					
▲					
★					
⊙					
✪					

PROJECT **Stanley Oswalt Academy - 19501 Shadown Oak Drive, Walnut, CA**

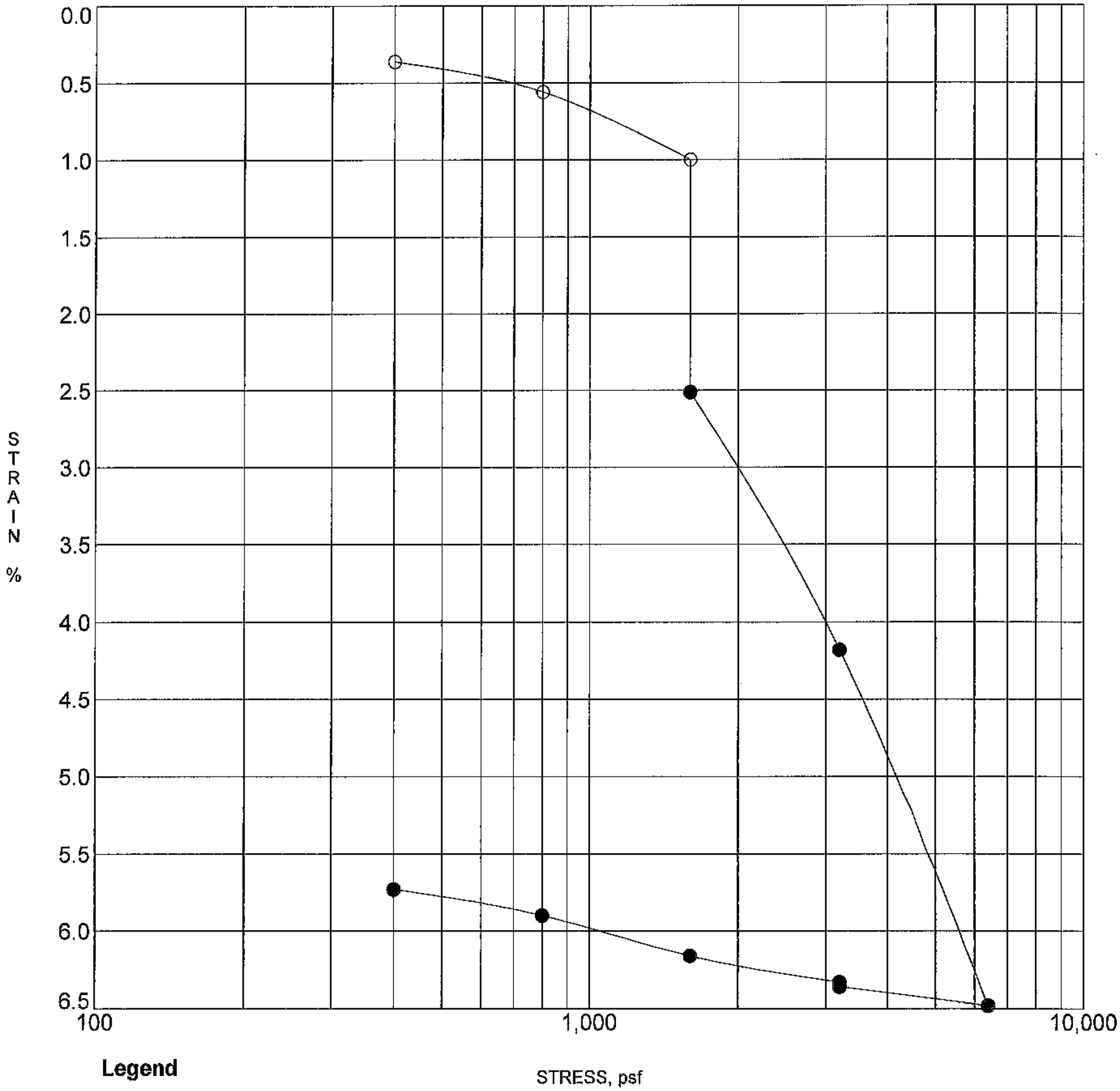
JOB NO. **18-01-3763**

DATE **12/13/18**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE C-4



Legend

○ Before water

● After water

	Specimen Identification		Classification	DD	MC%
●	**B-15**	**7.5**	**SANDY CLAY (CL)**	**103**	**18**
⊠					
▲					
★					
⊙					
✪					

PROJECT **Stanley Oswalt Academy - 19501 Shadown Oak Drive, Walnut, CA**

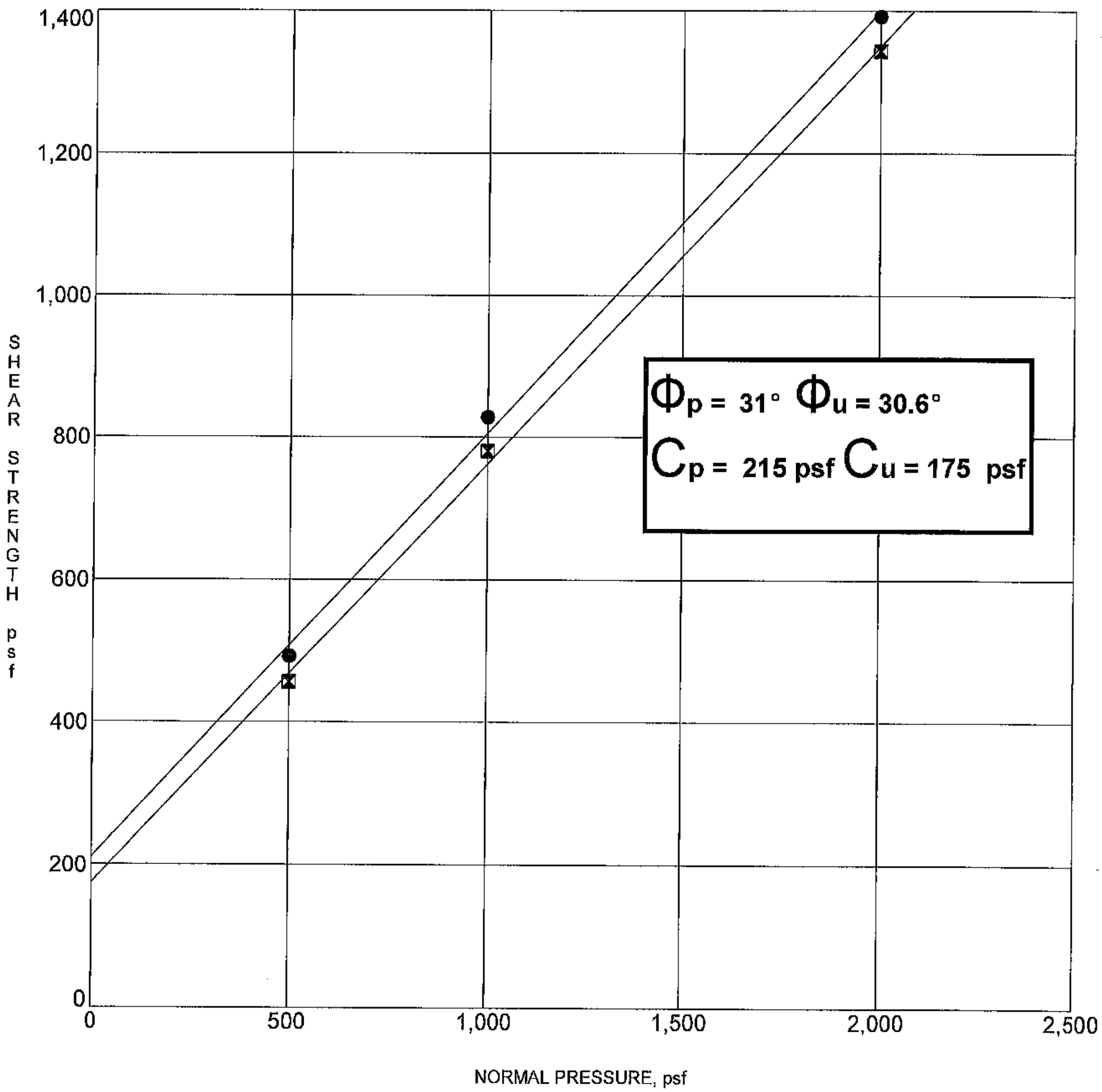
JOB NO. **18-01-3763**

DATE **12/13/18**

CONSOLIDATION TEST

Harrington Geotechnical Engineering, Inc.

PLATE C-5



	Specimen Identification		Classification	DD	MC%
●	**B-1**	**10.0**	**SILTSTONE BEDROCK (Peak)**	**83**	**30**
⊠	**B-1**	**10.0**	**(Ultimate)**		

PROJECT **Stanley Oswalt Academy - 19501 Shadow Oak Drive, Walnut, CA**

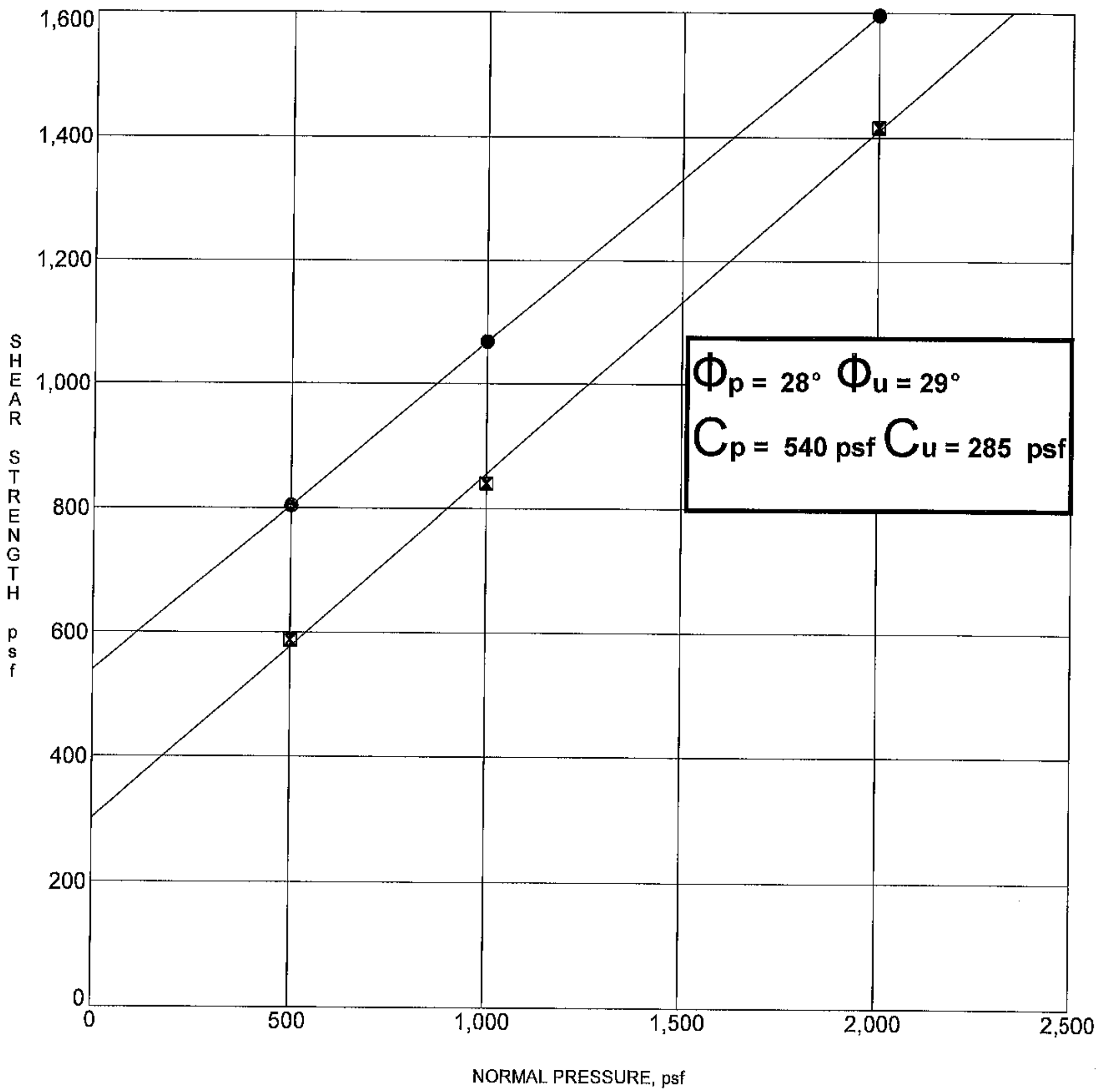
JOB NO. **18-01-3763**

DATE **12/11/18**

SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE D-1



	Specimen Identification		Classification	DD	MC%
●	B-5	7.5	**SILTSTONE/SANDSTONE BEDROCK (Peak)**	**90**	**32**
⊠	B-5	7.5	**(Ultimate)**		

PROJECT **Stanley Oswalt Academy - 19501 Shadow Oak Drive, Walnut, CA**

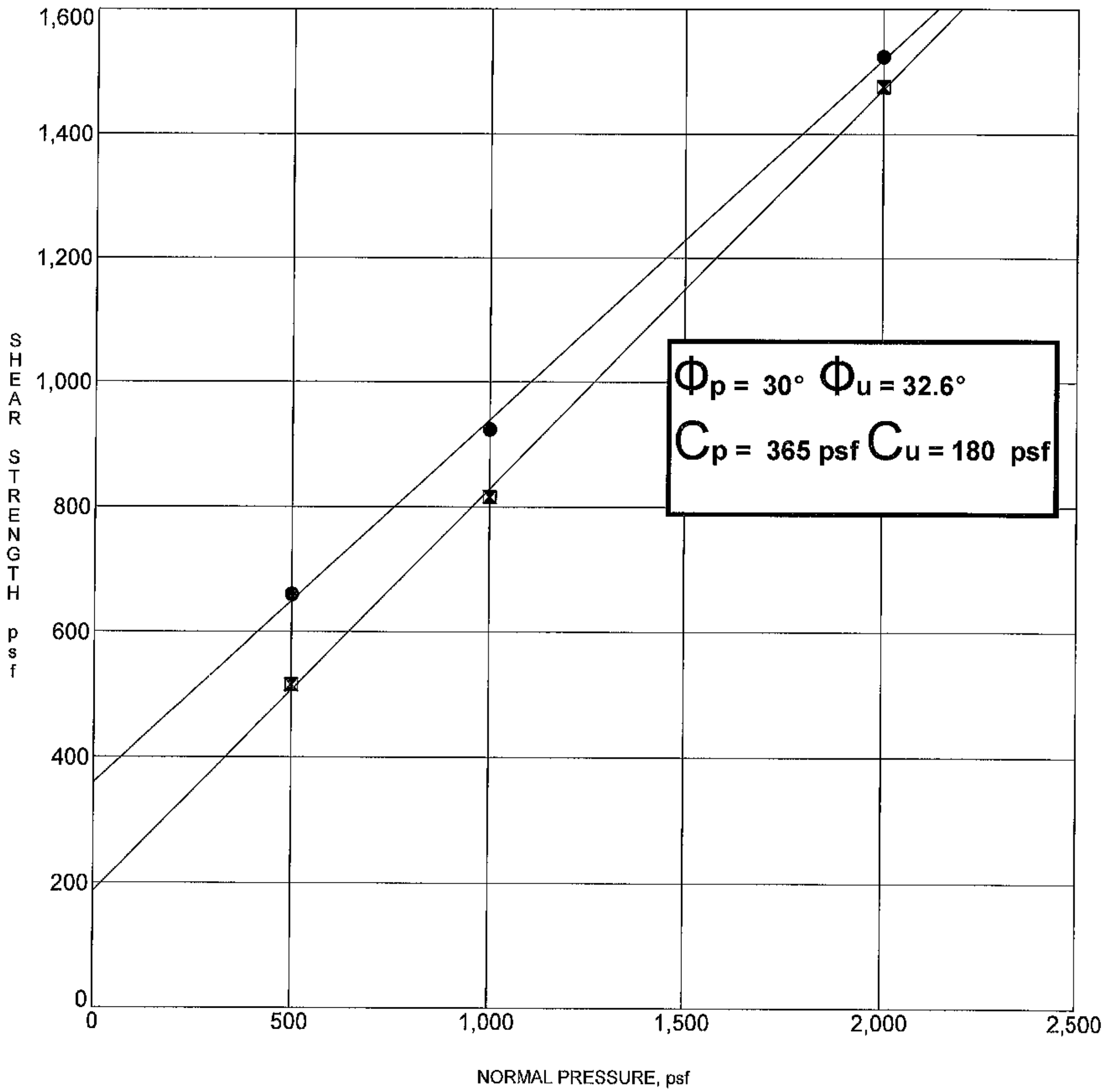
JOB NO. **18-01-3763**

DATE **12/13/18**

SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE D-2



	Specimen Identification		Classification	DD	MC%
●	**B-10**	**10.0**	**SANDSTONE/SILTSTONE (Peak)**	**95**	**26**
⊠	**B-10**	**10.0**	**(Ultimate)**		

PROJECT **Stanley Oswalt Academy - 19501 Shadow Oak Drive, Walnut, CA**

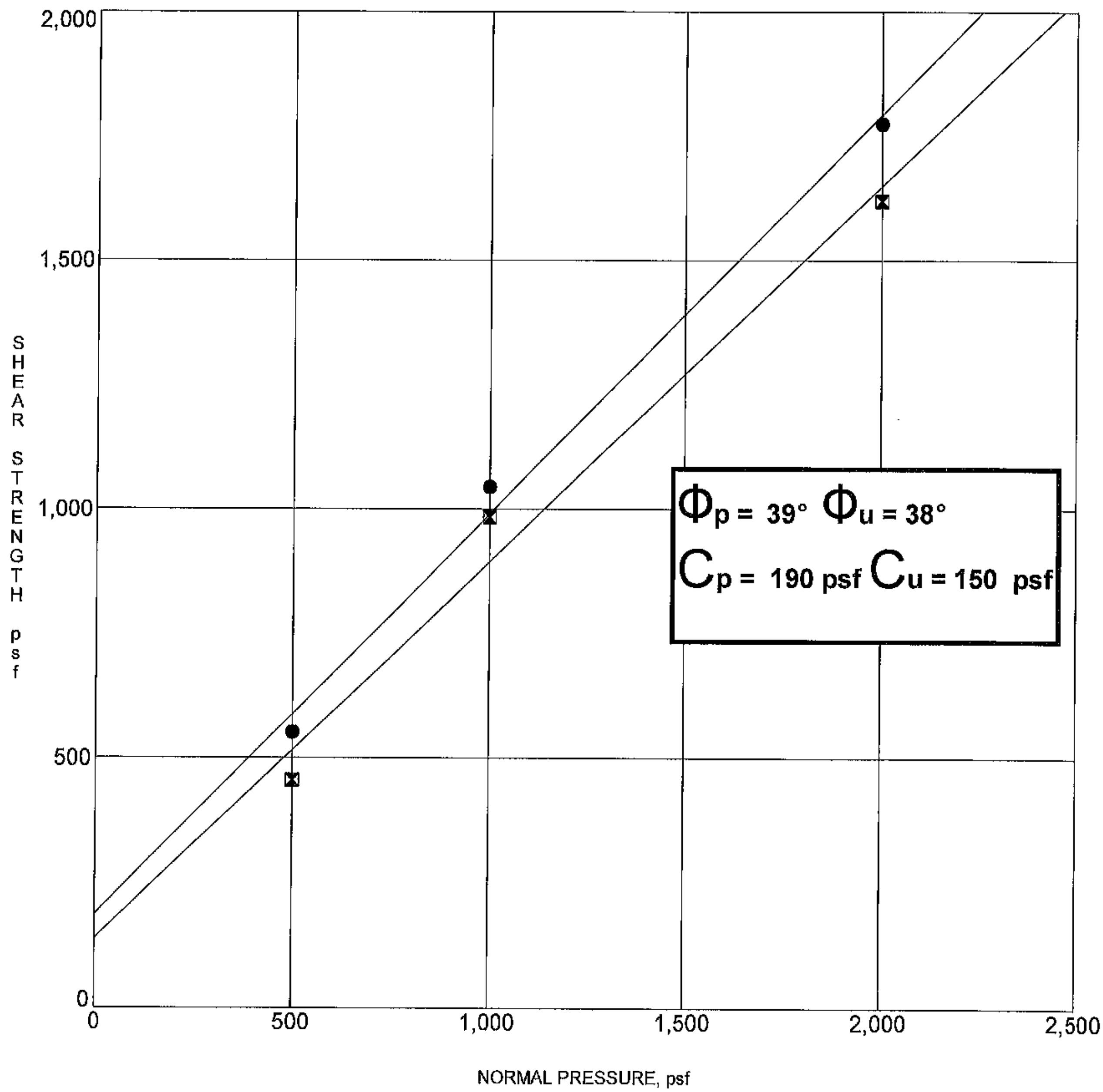
JOB NO. **18-01-3763**

DATE **12/13/18**

SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE D-3



	Specimen Identification		Classification	DD	MC%
●	B-13	7.5	SANDSTONE/SILTSTONE BEDROCK (Peak)	88	25
⊠	B-13	7.5	(Ultimate)		

PROJECT **Stanley Oswalt Academy - 19501 Shadow Oak Drive, Walnut, CA**

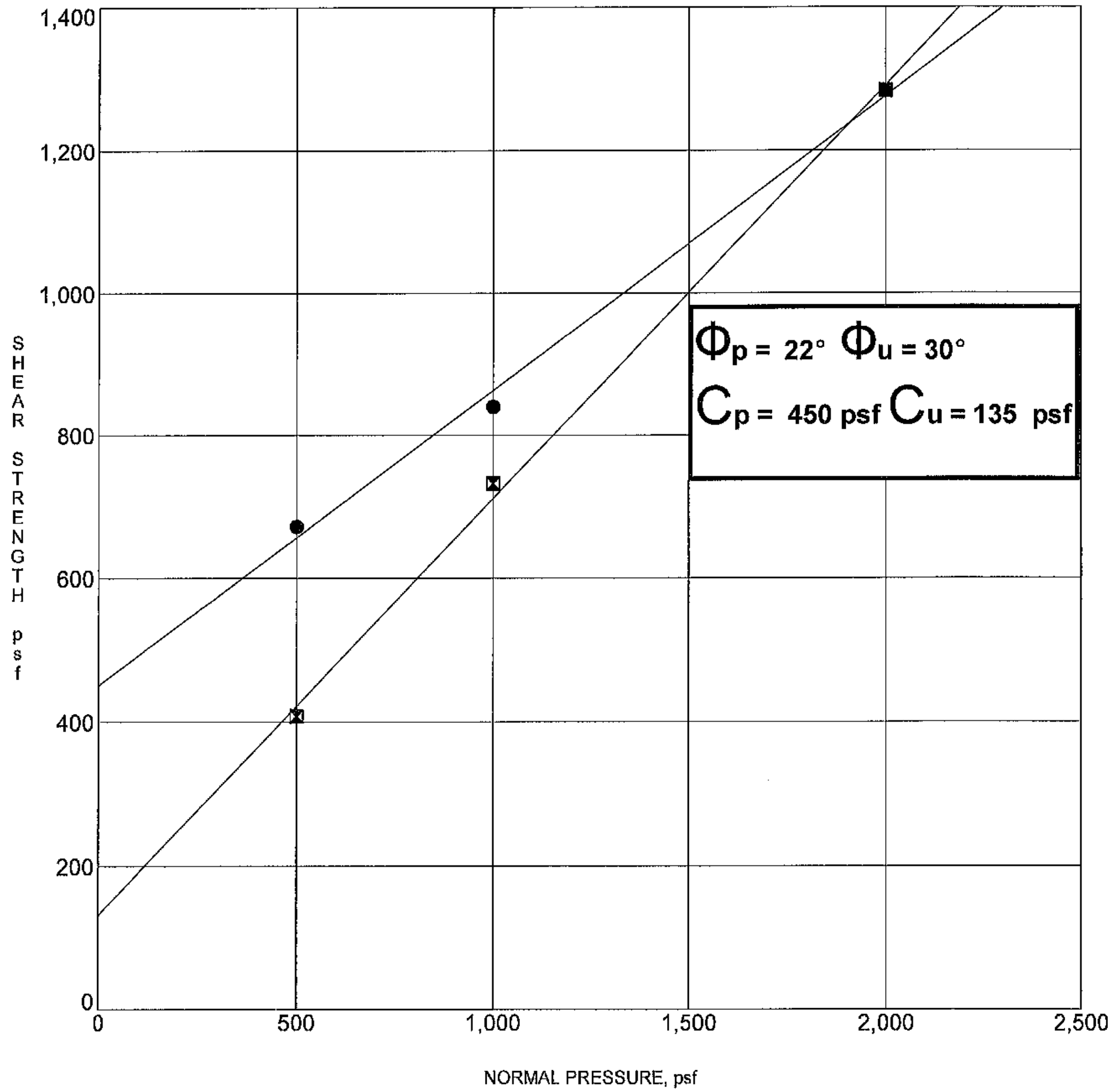
JOB NO. **18-01-3763**

DATE **12/13/18**

SHEAR TEST DIAGRAM

Harrington Geotechnical Engineering, Inc.

PLATE D-4



	Specimen Identification		Classification	DD	MC%
●	**B-15**	**10.0**	**SANDY CLAY (CL) (Peak)**	**104**	**20**
⊠	**B-15**	**10.0**	**(Ultimate)**		

PROJECT **Stanley Oswalt Academy - 19501 Shadow Oak Drive, Walnut, CA**

JOB NO. **18-01-3763**

DATE **12/13/18**

SHEAR TEST DIAGRAM

Harrington GeotEchnical Engineering, Inc.

PLATE D-5

APPENDIX C

STANDARD GRADING SPECIFICATIONS

STANDARD 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 for 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-inch-diameter, perforated, Schedule 40 PVC pipe surrounded by one cubic foot per linear foot of gravel and filter fabric. Canyon subdrains should consist of 8-inch-diameter, perforated, Schedule 40 PVC pipe surrounded by nine cubic feet per linear foot of approved gravel wrapped with 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, Figure 1. 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.

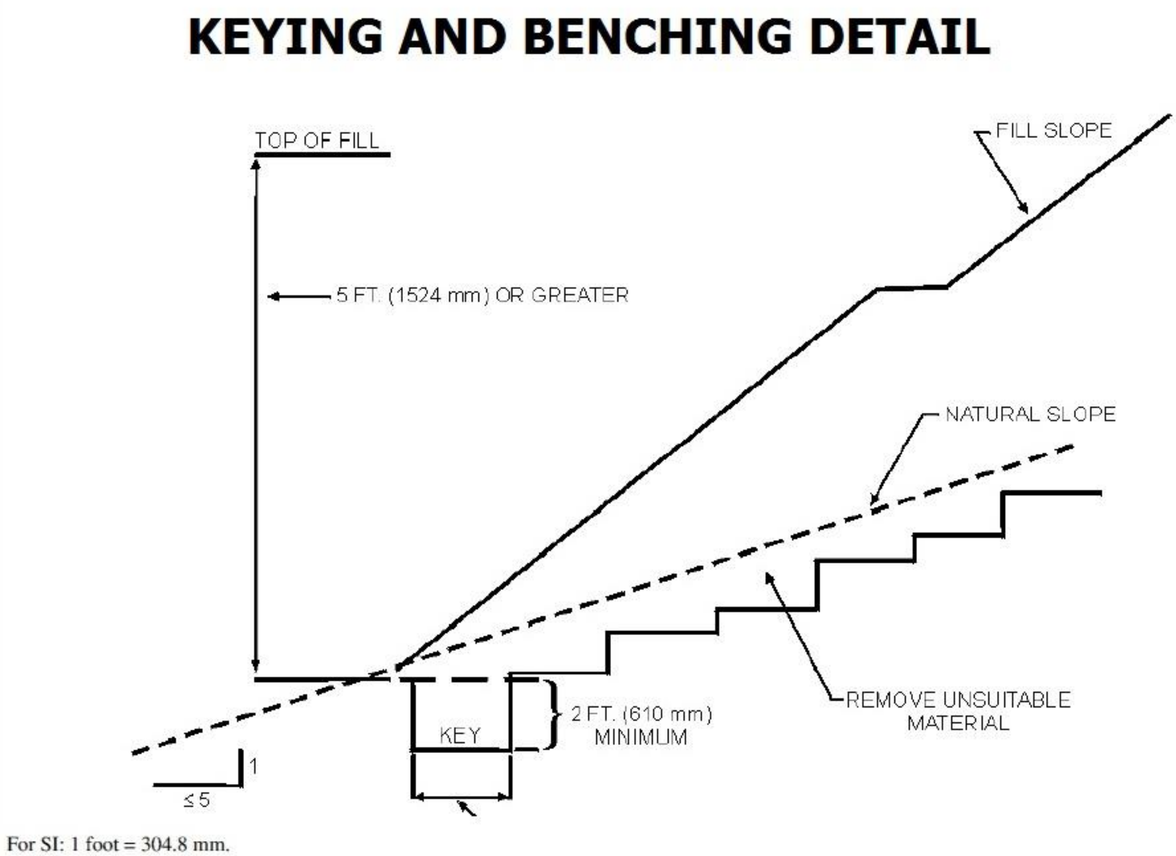


FIGURE 1

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

7.1. 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-foot vertical intervals during backfill placement.

7.2. 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

8.1. 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.

APPENDIX D

GEOLOGIC HAZARDS REPORT BY TERRA GEOSCIENCES

TERRA GEOSCIENCES

GEOLOGIC HAZARDS REPORT

PROPOSED MODERNIZATION PROJECT

STANLEY G. OSWALT ACADEMY

19501 SHADOW OAK DRIVE

CITY OF WALNUT, LOS ANGELES COUNTY, CALIFORNIA

Project No. 183175-1

November 24, 2018

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 Modernization Project
Stanley G. Oswalt Academy, 19501 Shadow Oak Drive
City of Walnut, Los Angeles County, California
HGEI Project No. 18-01-3763

INTRODUCTION

At your request, this firm has prepared a geologic hazards report for the proposed modernization 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. 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, 2013), 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.**
- **Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.**
- **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 - Seismic Hazard Zone Map
Plate 3 - Google™ Earth Imagery Map
Plate 4 - Seismic Line Location Map
Appendix A - Shear-Wave Survey
Appendix B - Site-Specific Ground Motion Analysis
Appendix C - References

PROJECT SUMMARY

Based on the information that has been provided, we understand that various improvements to the existing school campus are proposed. Additionally, we understand that this report will be appended into the geotechnical report prepared for the site by Harrington Geotechnical Engineering, Inc. (HGEI), therefore, some descriptive sections such as site description, proposed development, etc., have been purposely omitted as they are described in detail in the main report.

No grading or detailed site plans were available for this evaluation, and no field or sub-surface exploration was performed by this firm. Only a field site reconnaissance, performance of a geophysical seismic shear-wave survey, and a review of available geologic and geotechnical data in our files were undertaken. The proposed development area has been mapped by Tan (2000) to be underlain by late Miocene age sedimentary bedrock of the Puente Formation (Soquel Member). Artificial fill materials created from previous site grading may also be present locally.

Based on a preliminary computerized seismic analysis, using the U.S. Seismic Design Maps web application (U.S.G.S., 2017), the mapped spectral response acceleration parameter at the one-second period (S_1=0.762g), was found to be greater than or equal to 0.75g which requires the site to be assigned to Seismic Design Category “E” (CBC, 2016, Section 1613A.3.5). This category classification requires that a site-specific ground motion analysis be performed (C.G.S. Note 48, item 16). The detailed results of this site-specific analysis are presented within Appendix B for reference.

Additionally, to aid in providing applicable data for the site-specific ground motion analysis, a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods was performed in order to assess the one-dimensional average shear-wave velocity structure beneath the subject construction area to a depth of at least 100 feet. This survey line was performed within the north-central portion of the school campus (as shown on Plates 3 and 4), proximal to all of the proposed development areas, which provided the necessary survey line length. Photographic views of the survey traverse have been included within Appendix A for visual and reference purposes. The resultant shear wave velocity (V_S) within the upper 100 feet (30 meters) was then used to both determine the Site Classification (ASCE, 2010, Table 20.3-1 and CBC, 2016, Table 31F-6-1) of the subject project study area as well as being used for the V_S input value of the seismic analysis. The detailed results of this survey are presented within Appendix A for reference.

The location of the seismic shear-wave traverse (Seismic Line SW-1) is presented on a captured Google™ Earth image (Google™ Earth, 2018), as presented Plate 3. In addition, the survey line was also transposed onto a partial copy of the provided 40-scale Site Plan (Sheet No. AS1.1), prepared by Ziemba + Prieto Architects, Burbank, California, dated July 26, 2018, as presented on the Seismic Line Location Map (see Plate 4), for reference.

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 subject school 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

Geologic mapping by Tan (2000) and by Morton and Miller (2006), as shown on the Regional Geologic Map (see Plate 1), indicate the subject site to be underlain by Miocene age sedimentary bedrock of the Puente Formation, more specifically the Soquel Member (map symbol Tpsq), generally described as being gray to yellowish-gray, massive to well-bedded, medium- to coarse-grained, poorly sorted sandstone interbedded with matrix-supported pebbly sandstone. Subsurface exploration by HGEI (2018) within the proposed construction areas indicate that the site is underlain by fine-grained sandstone with thin interbeds of siltstone to a depth of at least 16 feet. Localized artificial fill may be present across the proposed construction area.

FLOODING

According to the Federal Emergency Management Agency (2008), the subject school site is not shown to be located within the boundaries of a 100-year flood zone (Community Panel No. 06037C1725F, September 26, 2008). The site is shown to be located within "Zone X" which is defined as “Area of Minimal Flood Hazard”.

GROUNDWATER

The subject site lies within semi-consolidated sedimentary bedrock of the San Jose Hills and is generally considered to be non-water bearing. Additionally, the site does not lie within a designated State of California groundwater basin. During on-site subsurface exploration by HGEI (2018), groundwater was not encountered within any of the exploratory borings to a depth of at least 16 feet. Mapping by the California Geological Survey (1998) does not indicate any groundwater depth contouring in the region implying that no known historical shallow groundwater conditions are locally present within the vicinity of the subject site. Additionally, the City of Walnut (2018) does not indicate the site to be located within a zone of potential liquefaction (Figure PS-4, Seismic Hazards), indicating that groundwater is greater than 50 feet in depth.

FAULTING

There are at least forty-four 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 (CGS, 2018). The nearest such “mapped” hazard zone is associated with the active Whittier Fault (northern segment of the Elsinore Fault Zone) located 5⅓± miles to the southwest (C.D.M.G., 1991).

The Whittier Fault is a 38-kilometer long right-lateral, strike-slip fault capable of producing an earthquake with an estimated maximum moment magnitude of M_W 6.9, and has 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 singular Whittier Fault 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 with an associated Maximum Moment Magnitude (M_W) of 7.8.

The nearest mapped significant fault is the San Jose Fault approaching within 2,500± feet to the north (see Plate 1), 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). Additionally, the City of Walnut (2018) does not include the site to be located within any seismic hazard zone associated with the San Jose Fault (Figure PS-4, Seismic Hazards).

Both the San Jose and Whittier faults were analyzed for the site-specific ground-motion. Although the Whittier Fault has a greater seismogenic potential, the closer San Jose Fault was found to control over most of the spectrum.

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., 2018). 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 November 2018 within a 100-kilometer (62-mile) radius of the site.

TABLE 1 - HISTORIC SEISMIC EVENTS; 1800-2018 (100-kilometer radius)

Richter Magnitude (M)	No. of Events
4.0 - 4.9	504
5.0 - 5.9	68
6.0 - 6.9	16
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 closest recorded notable earthquake epicenter (magnitude 4.0 or greater) is the M4.2 event of January 1, 1976, located approximately four miles to the south.
- The nearest estimated significant historic earthquake epicenter (pre-1932) was approximately seven miles to the west (December 25, 1903, M5.0).
- The nearest recorded significant historic earthquake epicenter was an M5.1 event, located approximately seven miles southwest of the site (March 29, 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 27 miles northeast).
- The largest recorded historical earthquake was the M6.7 (M_W6.4) Northridge event, located approximately 40 miles to the northwest (January 17, 1994).
- The largest estimated ground acceleration estimated to have been experienced at the site was 0.175g which resulted from the M6.3 event of July 7, 1855, located approximately14 miles to the west.

An Earthquake Epicenter Map which includes magnitudes 4.0 and greater for a 100-kilometer (62-mile) radius has been included below as Figure 1, for reference. This map was prepared using the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2018) of instrumentally recorded events from the period of 1932 to November 2018, superimposed on a captured Google™ Earth image (Google™ Earth, 2018).

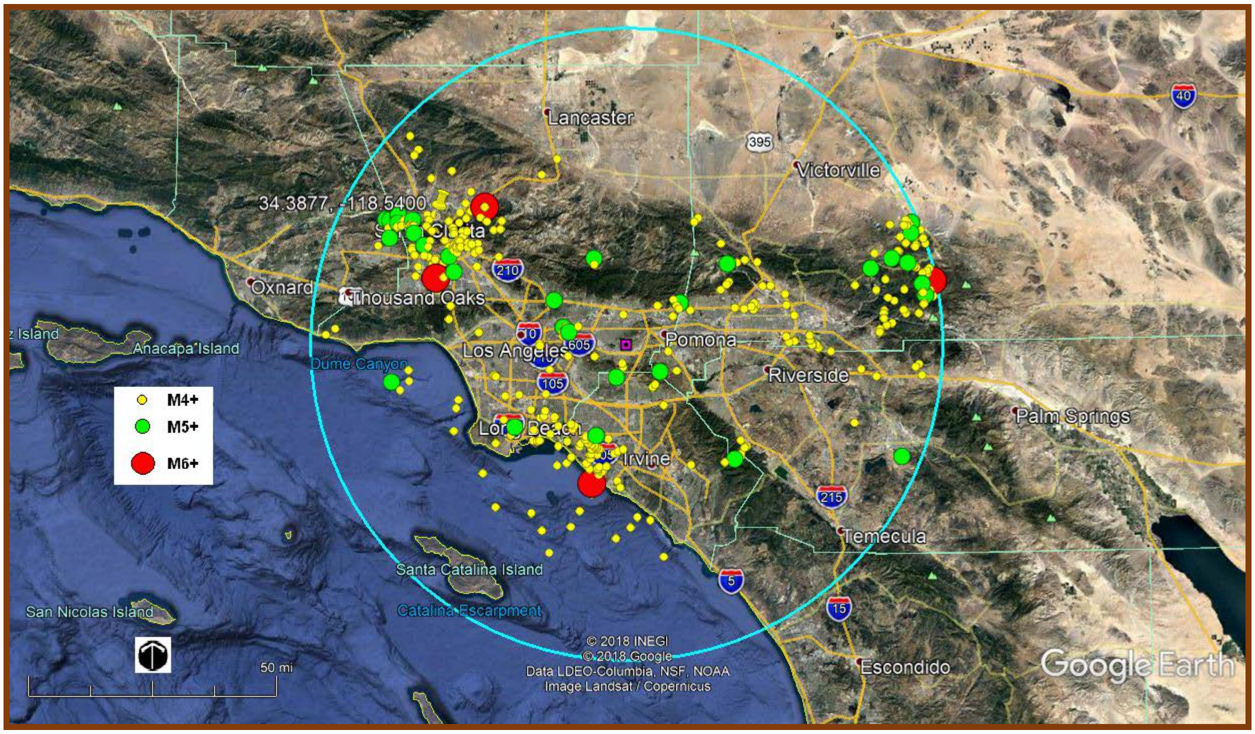


FIGURE 1- Earthquake Epicenter Map; Events of M4.0+ within a 100-kilometer radius (blue circle).

SITE-SPECIFIC GROUND MOTION ANALYSIS

According to California Geological Survey Note 48 (CGS, 2013), a site-specific ground motion analysis is required for the subject project since the Seismic Design Category has been determined to be "E" (CBC 1613A.3.5 and also as required by ASCE 7-10, Section 11.4), the detailed results of which are presented within Appendix B. Additionally, a seismic shear-wave survey was conducted for this study by our firm as presented within Appendix A of this report for purposes of determining the Site Classification and V_{S30} input values for the ground motion analysis. Geographically, the proposed construction area is located at Longitude -117.8800 and Latitude 34. 0270 (World Geodetic System of 1984 coordinates). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the U.S.G.S. Design Maps (U.S.G.S., 2017) and the California Building Code criteria (CBC, 2016), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-10 Standard (2010). The results of this site-specific analysis have been summarized and are tabulated below:

TABLE 2 – SUMMARY OF SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_S	2.147g
S_1	0.762g
F_a	1.0
F_v	1.5
S_{DS}	1.240g
S_{D1}	0.960g
S_{MS}	1.860g
S_{M1}	1.440g
T_L	8 Seconds
MCE_G PGA	0.76g
Site Class	D
Seismic Design Category	E

SECONDARY SEISMIC HAZARDS

Secondary permanent or transient seismic hazards that are generally associated with severe ground shaking during an earthquake include ground rupture, liquefaction, seiches or tsunamis, flooding (water storage facility failure), ground lurching/later spreading, 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. Based on the underlying semi-consolidated sedimentary bedrock and absence of shallow groundwater, there does not appear to be a potential for liquefaction to impact the proposed development. Additionally, City of Walnut (2018) does not indicate the site to be located within a zone of potential liquefaction (Figure PS-4, Seismic Hazards).

Seismically-Induced Settlement

Seismically-induced settlement generally occurs within areas of loose, granular soils. Since the subject project area is underlain by near-surface semi-consolidated sedimentary bedrock (HGEI, 2018), the potential for seismically-induced settlement is considered nil.

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 potential for ground lurching and/or lateral spreading at the study area appears to be nil.

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.

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. The City of Walnut (2018) indicates that the City is not located within the limits of flooding due to catastrophic failure of dams or levees.

Landsliding

Due to the relatively low-lying relief of the site, landsliding due to seismic shaking 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.

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; naturally occurring asbestos; volcanic hazards; 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), the proposed modernization project appears to be feasible from a geologic standpoint, providing our recommendations are considered during planning and construction.

Conclusions:

1. Based on available published surficial geologic mapping, the subject site is shown to be underlain by the Soquel Member of the Puente Formation (Miocene age), generally described as being gray to yellowish-gray, massive to well-bedded, medium- to coarse-grained, poorly sorted sandstone interbedded with matrix-supported pebbly sandstone. Based on the subsurface exploration performed by HGEI, the construction areas are underlain to be sedimentary bedrock generally comprised of fine-grained sandstone with thin interbeds of siltstone to a depth of at least 16 feet, with possible localized areas of surficial artificial fill.

2. Based on subsurface exploration by HGEI, groundwater was not encountered to a depth of at least 16 feet. The site lies within semi-consolidated sedimentary bedrock of the San Jose Hills that is generally considered to be non-water bearing. Groundwater, if present, is expected to be greater than 50 feet in depth.

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 5⅓± miles to the southwest. The nearer San Jose Fault, located 2,500± feet to the north, is not currently zoned as active.

4. There are no permanent or transient secondary seismic hazards that are expected to occur within the project study area based on our study and review of available published literature.

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.

2. It is recommended that all structures be designed to at least meet the current California Building Code provisions in the latest CBC edition (2016) and the ASCE Standard 7-10 (2010), where applicable. However, it should be noted that the building code is intended as a minimum construction design and is often the maximum level to which structures are designed. Essential structures (such as the subject school site) that are built to minimum code are designed to at least remain operational after an earthquake. 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 development.

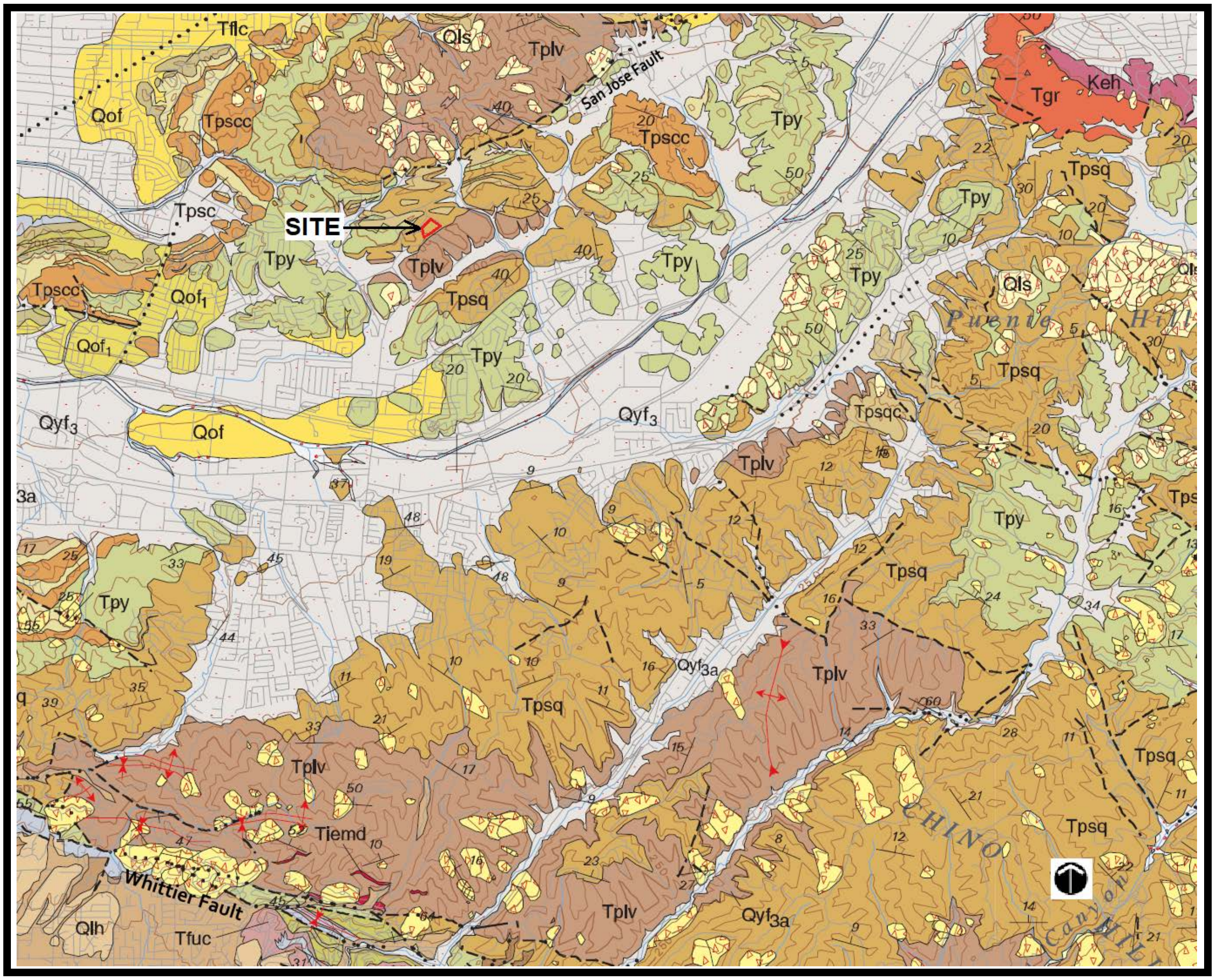
CLOSURE

Our conclusions and recommendations are based on an interpretation of available existing geologic/seismic data. 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. If 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
Certified Engineering Geologist, CEG 1459
Professional Geophysicist, PGP 1002

REGIONAL GEOLOGIC MAP

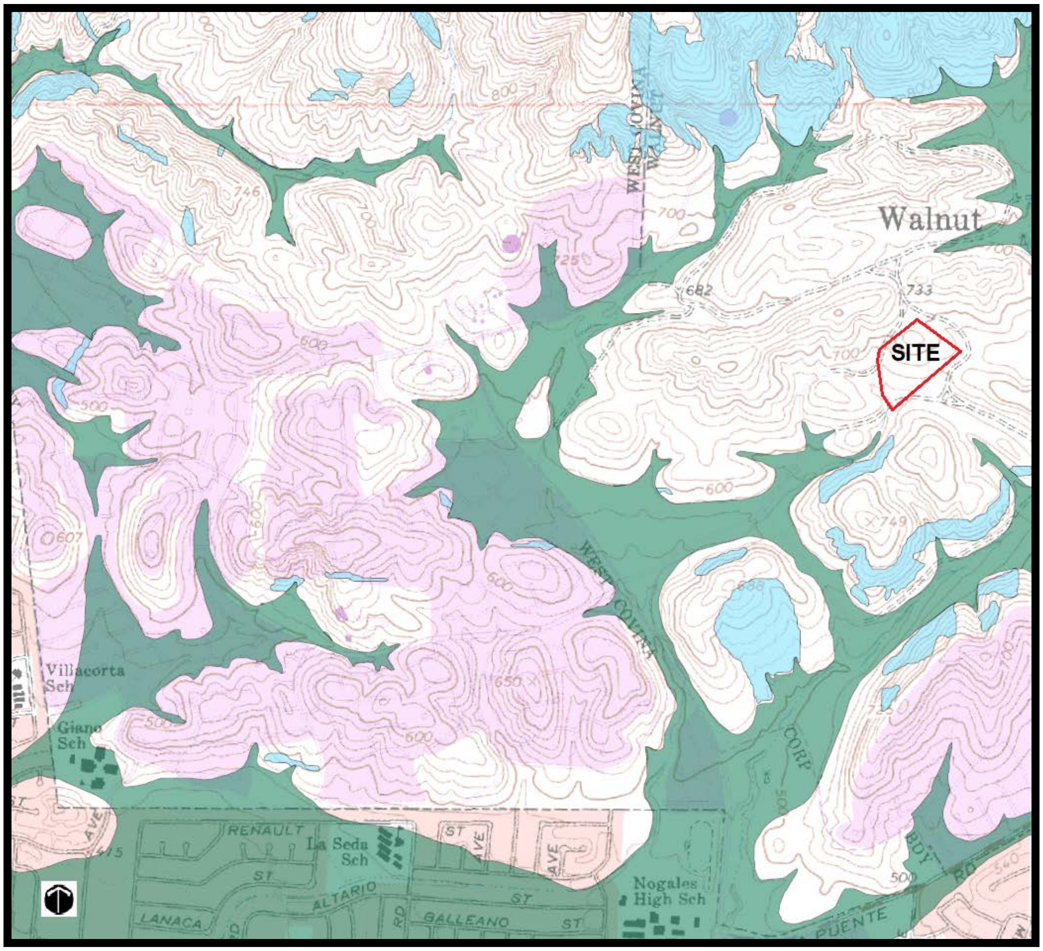


BASE MAP: Morton and Miller (2006), U.S.G.S. OFR 2006-1217, Scale 1” = 1.5± miles.

PARTIAL LEGEND

Tpsq	**PUENTE FORMATION**	Soquel Member; massive to well-bedded, medium- to coarse-grained, poorly sorted sandstone (Miocene).
Tplv	**PUENTE FORMATION**	La Vida Member; Light-gray to black, massive to well-bedded, generally friable siltstone. (Miocene).
	GEOLOGIC CONTACT	Solid where well to approximately located, dashed where poorly located or inferred.
	FAULT	Solid where well to approximately located, dashed where poorly located or inferred.

SEISMIC HAZARDS ZONE MAP



BASE MAP: C.G.S. (2018), Baldwin Park 7.5' Quadrangle; Seismic Hazard Zones, Scale 1"=2,000±'

PROPOSED MODERNIZATION PROJECT

STANLEY G. OSWALT ACADEMY

19501 SHADOW OAK DRIVE

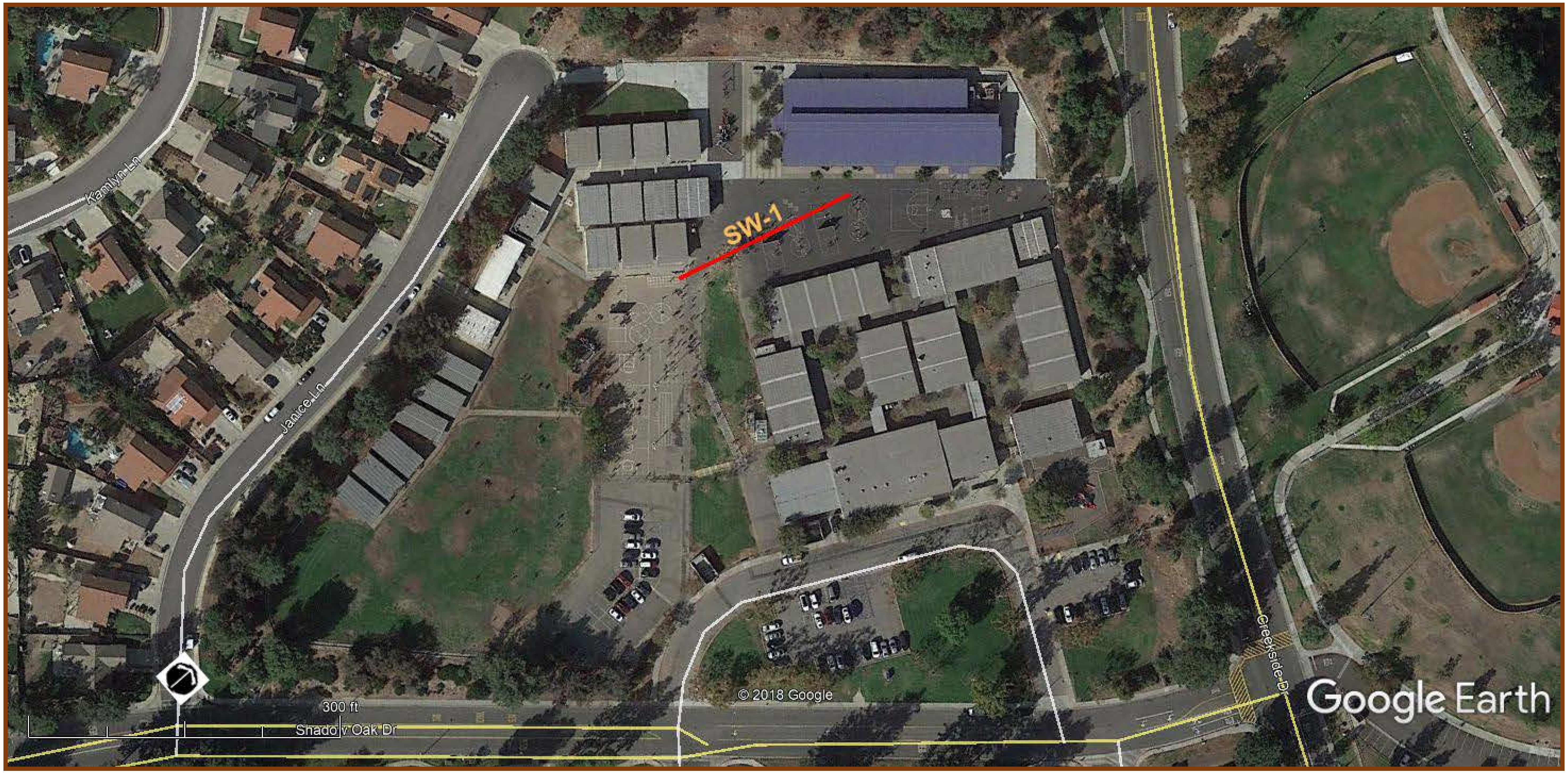
CITY OF WALNUT, CALIFORNIA

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.

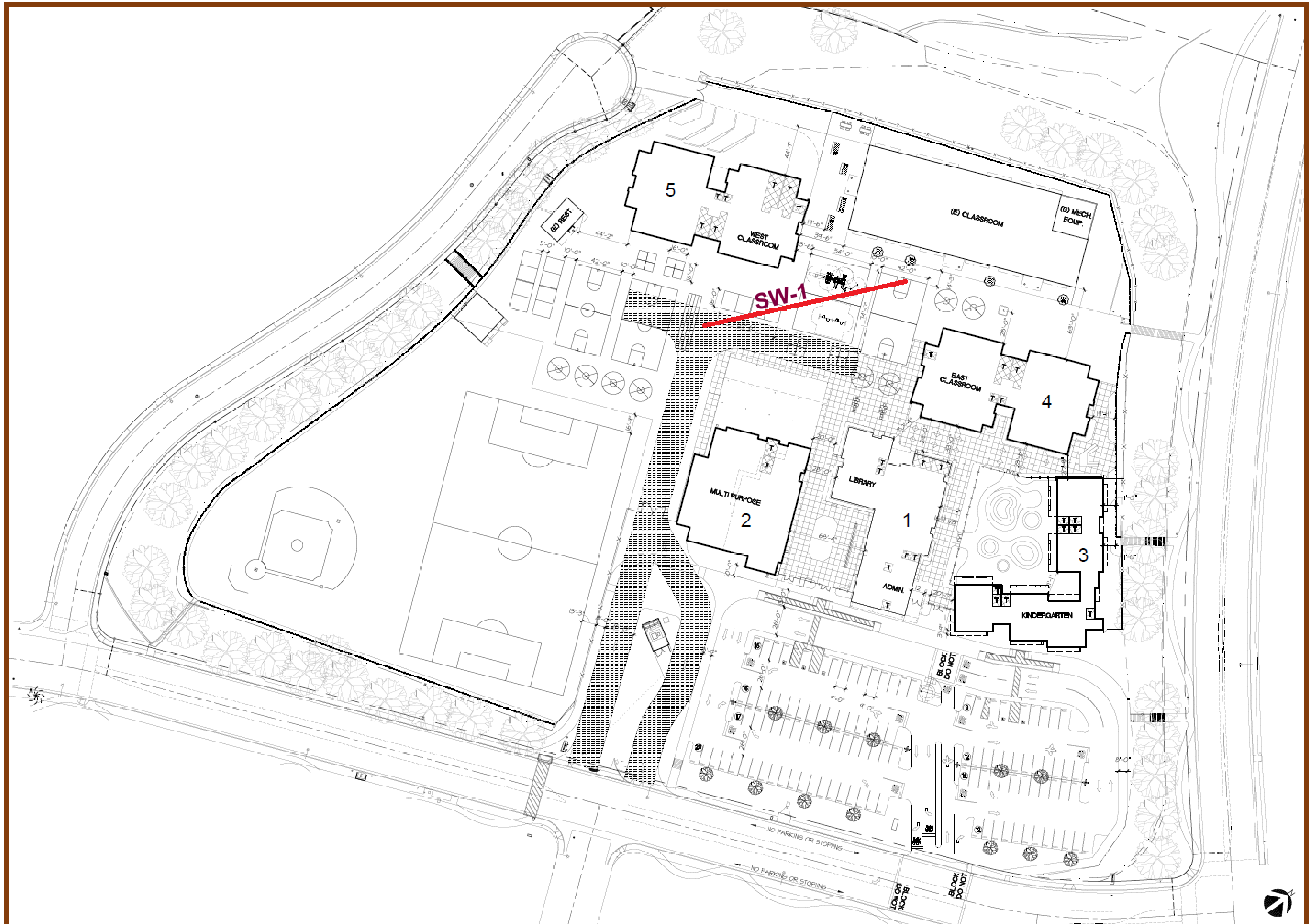
Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would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

GOOGLE™ EARTH IMAGERY MAP



Base map from Google™ Earth (2018); Seismic Line SW-1 indicated by red line.

SEISMIC LINE LOCATION MAP



Base Map modified from Site Plan (Sheet AS1.1) prepared by Ziemba + Prieto Architects, Burbank, California, dated July 26, 2018; Shear-Wave survey line (SW-1) shown in red.

APPENDIX A

SHEAR-WAVE SURVEY

SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse was performed within the north-central portion of the subject school campus, proximal to all of the proposed development areas, which has been approximately located on the Google™ Earth Imagery Map (see Plate 3) and the Survey Line Location Map (see Plate 4), for reference. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM survey lines used the same linear geometry array that consisted of a 184-foot long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals. For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 30-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple shots (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of any artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 30 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Processing

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2009). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within this appendix.

Data Analysis

Data acquisition went very smoothly and the quality was considered to be very good. The seismic model data indicates that the average shear-wave velocity beneath the survey traverse has several velocity layers which gradually increase with depth. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **716.6** feet per second as shown on the Shear-Wave Model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "S_D" (stiff/dense soil profile), which has a velocity range from 600 to 1,200 ft/sec (CBSC, 2016 & ASCE, 2010; Table 20.3-1). The "weighted average" velocity is computed from a formula that is used by the ASCE (2010; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100). This formula is as follows:

$$V100' = 100/[(t1/v1) + (t2/v2) + ... + (tn/vn)]$$

Where t1, t2, t3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The shear-wave model displays these calculated layers and associated velocities (feet/second) to the maximum obtained depth of 252 feet (shaded area on shear-wave model) where locally sampled. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model are also included within this appendix for reference purposes.

Limitations

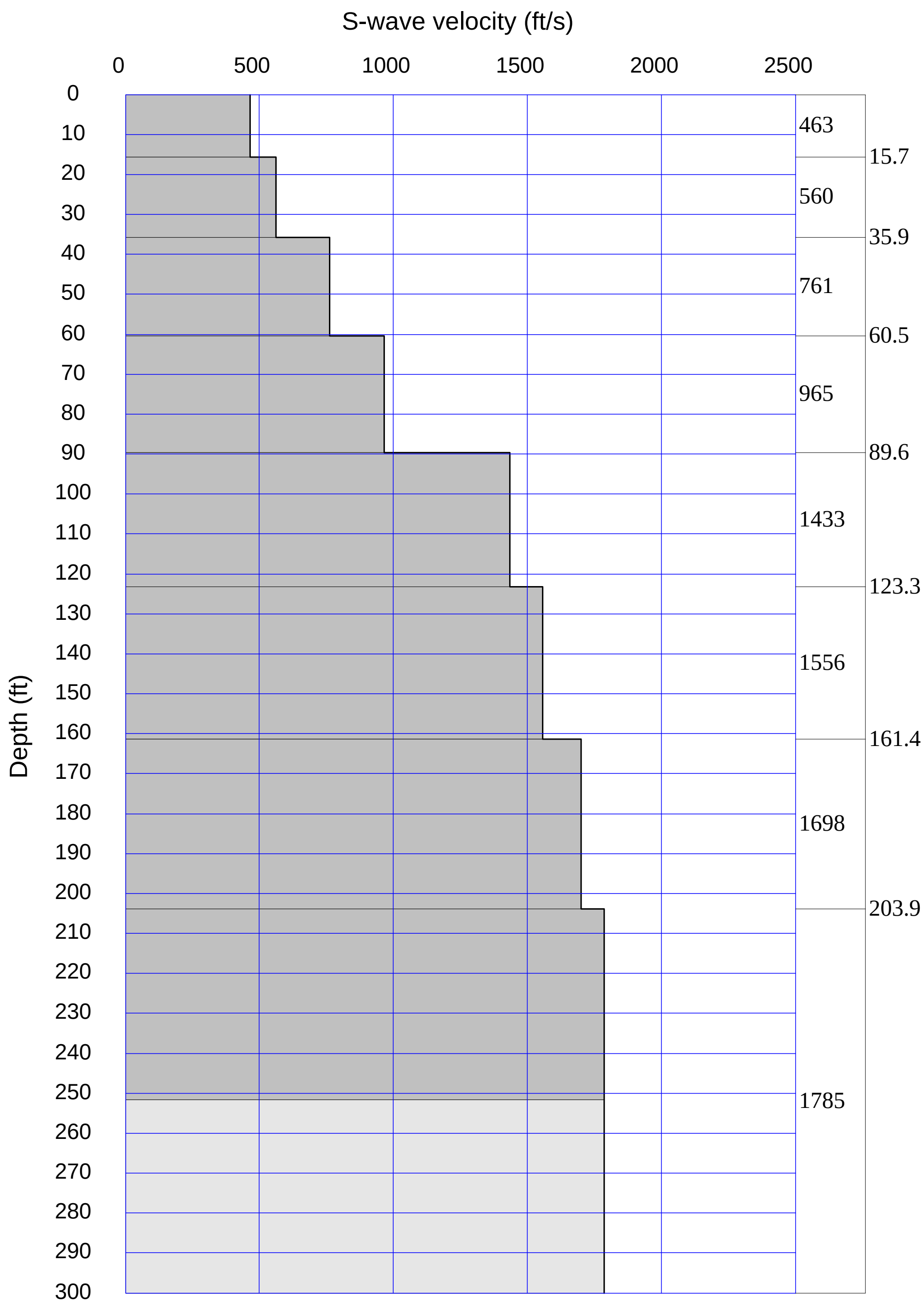
This survey was performed using "state of the art" geophysical equipment, techniques, and computer software. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. Compared with traditional borehole shear-wave surveys of which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent.

View looking southwesterly along Seismic Line SW-1.

View looking northeasterly along Seismic Line SW-1.

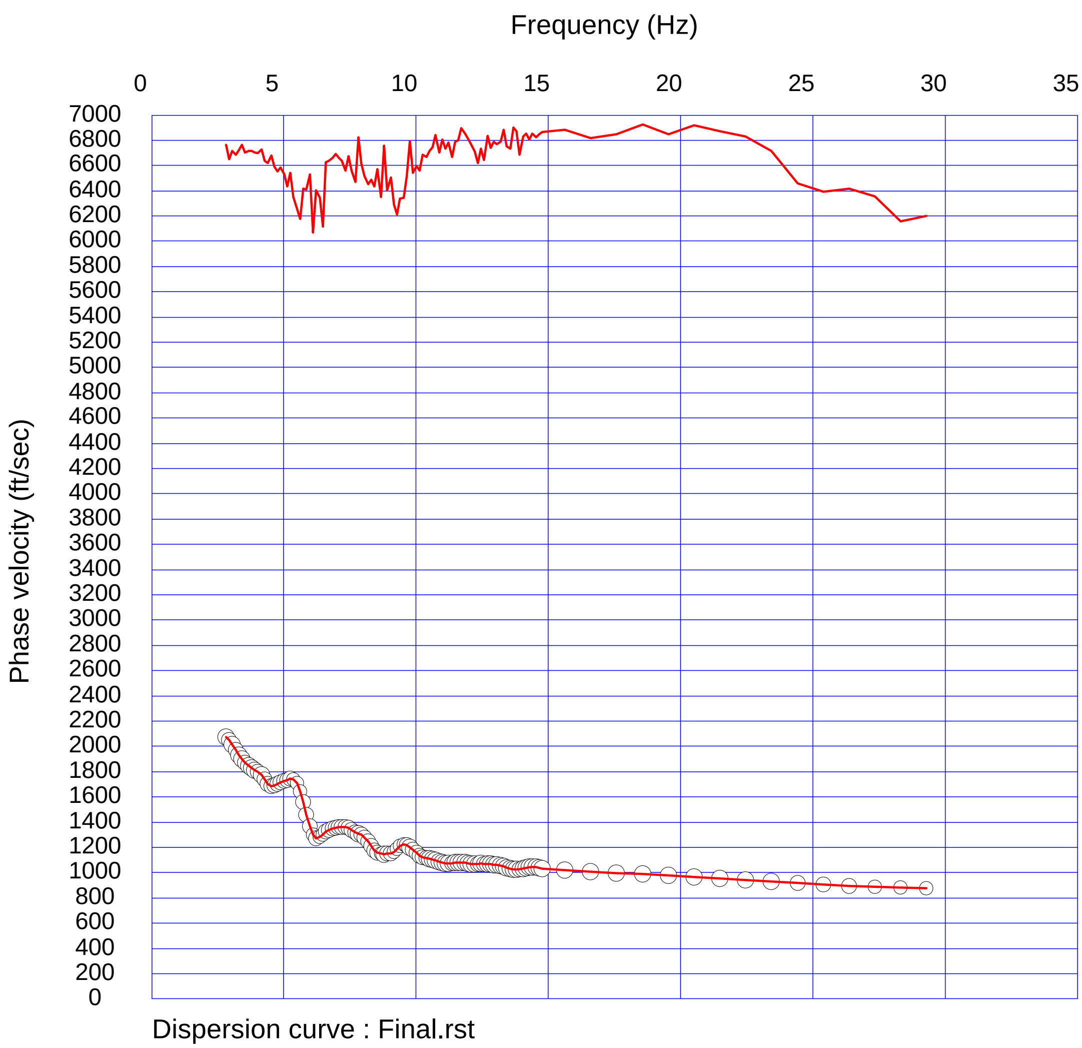
SEISMIC LINE SW-1

SHEAR-WAVE MODEL

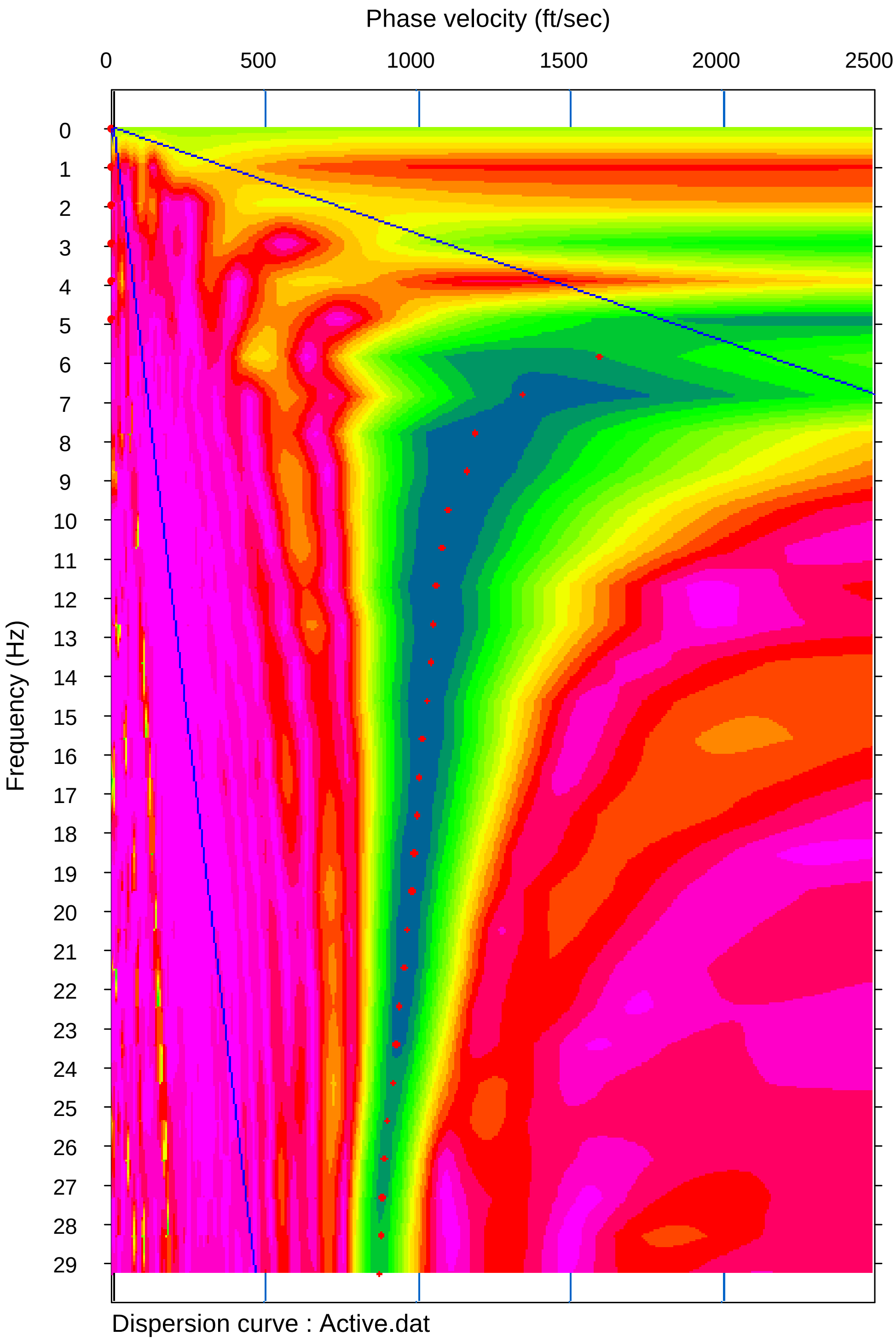


S-wave velocity model (inverted): Final.rst **Average Vs 100ft = 716.6 ft/sec**

SEISMIC LINE SW-1

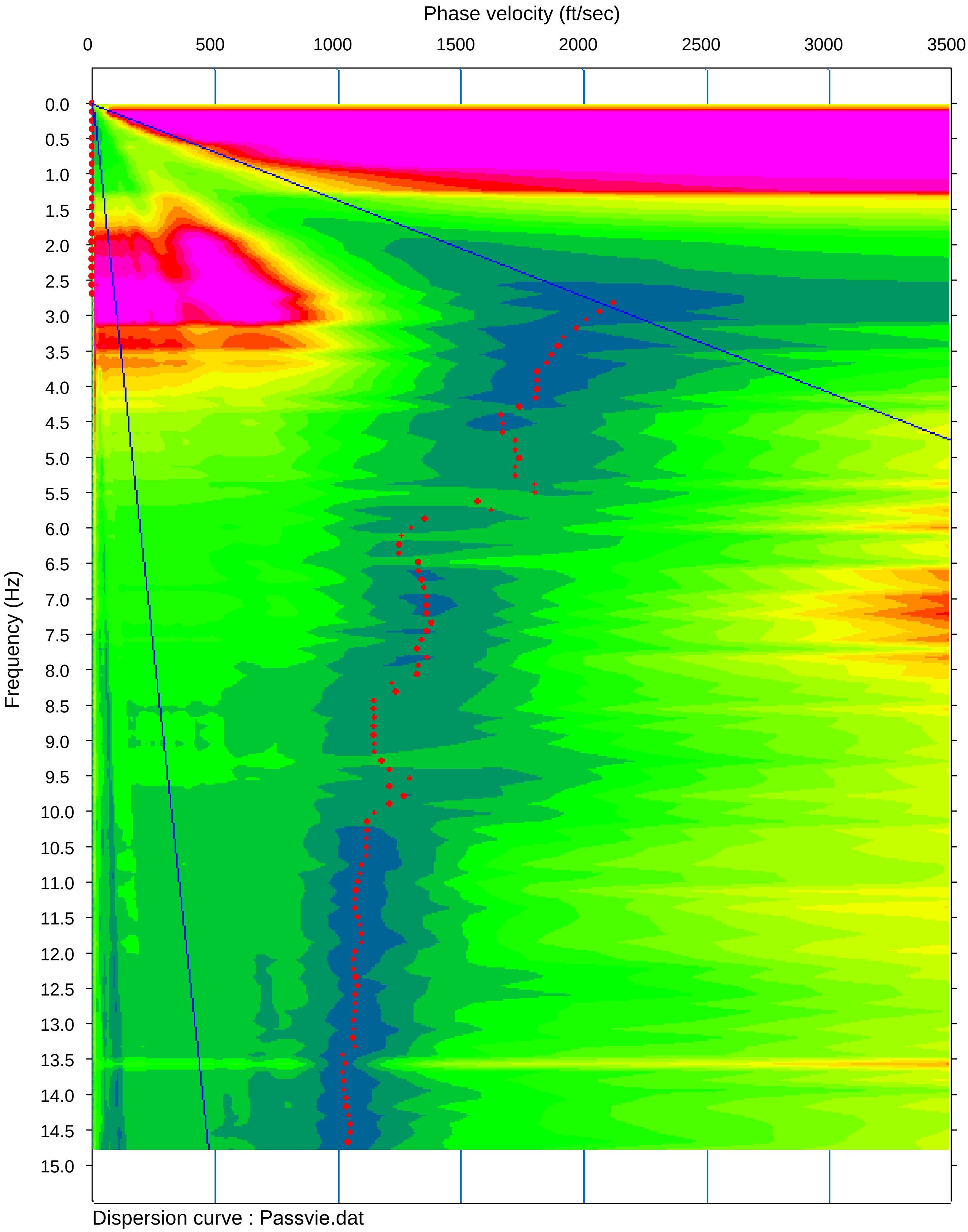


SEISMIC LINE SW-1



ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



PASSIVE DISPERSION CURVE

APPENDIX B

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 7-10 Standard (2010) is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

- ***Mapped Spectral Acceleration Parameters (CBC 1613A.3.1)***- 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), a value of 2.147g for the 0.2 second period (S_s) and 0.762g for the 1.0 second period (S_1) was calculated (ASCE 7 Figures 22-1, 22-2 and CBC 1613A.3.1).

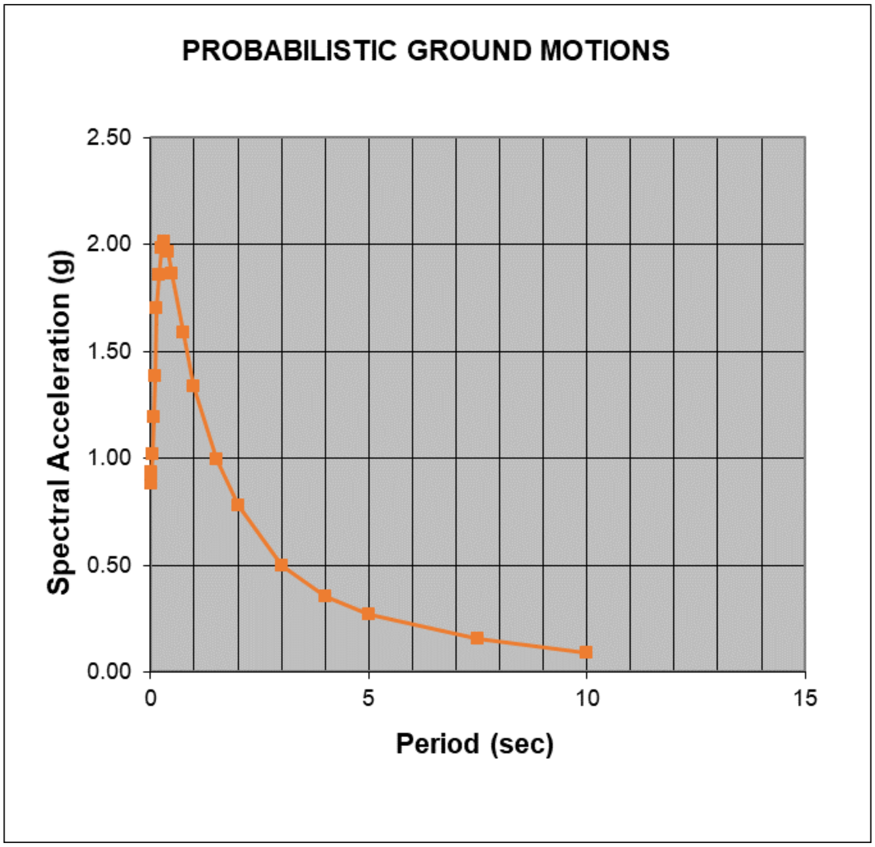
- ***Site Coefficients (CBC 1613A.3.3)***- Based on CBC Tables 1613A.3.3(1) and 1613A.3.3(2), the site coefficient F_a = 1.0 and F_v = 1.5, respectively.

- ***Site Classification (CBC 1613A.3.2)***- Based on the site-specific shear-wave value of 218.4 m/sec (716.6 ft/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 stiff soil with average shear-wave velocities of 180 to 360 meters/second.

- ***Seismic Design Category (CBC 1613A.3.5)***- Based on the proposed construction within the existing school facilities (Risk Category III; CBC Table 1604.5) and using the U.S. Seismic Design Maps web application (U.S.G.S., 2016), the mapped spectral response acceleration parameter at the one-second period (S_1=0.762g), was found to be greater than 0.75g which requires the site to be assigned to Seismic Design Category **"E."**

- ***Probabilistic (MCE_R) Ground Motions (ASCE 7 Section 21.2.1)***- Per Section 21.2.1, 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; U.S.G.S., 2016). The selected Earthquake Rupture Forecast (ERF) was UCERF2 along with a Probability of Exceedance of 2% in 50 Years. The average of three Next Generation Attenuation Relations (2008 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2008), Campbell & Bozorgnia (2008) and Boore & Atkinson (2008). 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 three NGA attenuation relationships Chiou & Youngs (2008), Campbell & Bozorgnia (2008), and Boore & Atkinson (2008).
-

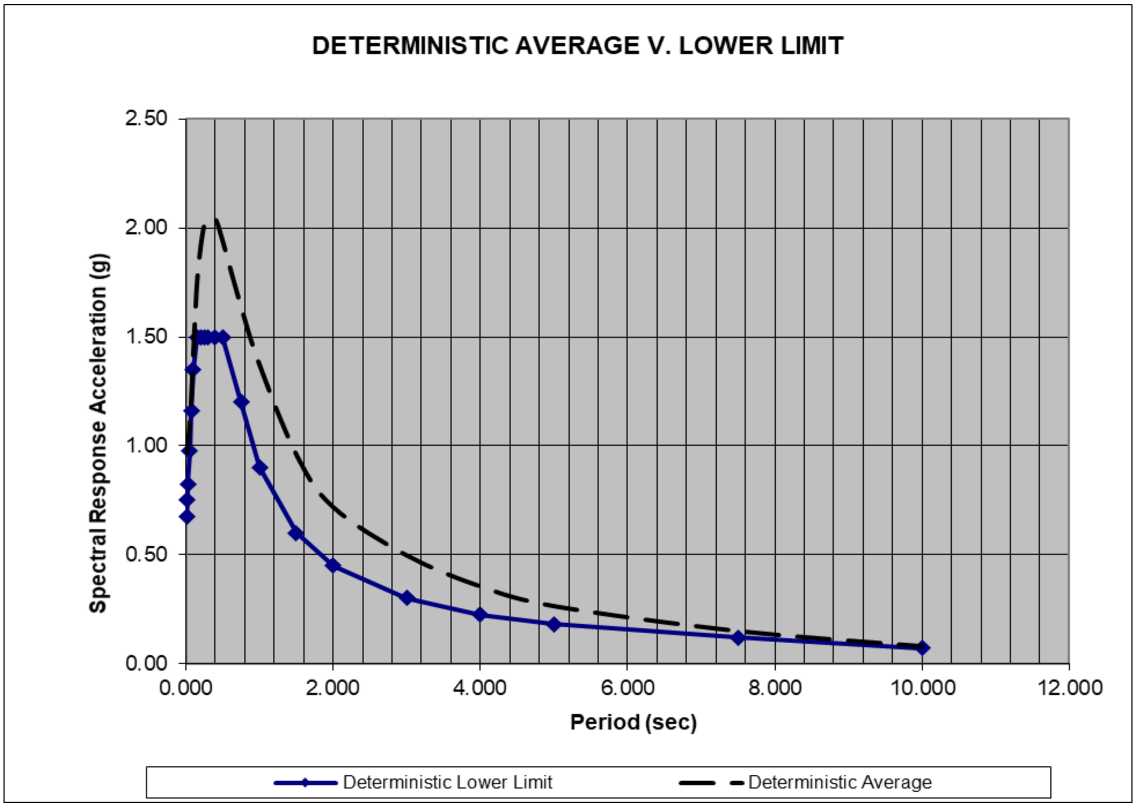
Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (U.S.G.S, 2016), discussions with the California Geologic Survey (CGS), and based on the length and maximum magnitude of each of the segments of the Whittier Fault Zone, the largest moment magnitude (M) for this fault is 7.8, considering a cascading event along the entire fault zone. At this time, the San Jose Fault is considered to be capable of producing a Maximum Moment Magnitude earthquake of M_W 6.7. Both the San Jose and Whittier faults were analyzed for the site-specific ground-motion. Although the Whittier Fault has a greater seismogenic potential, the closer San Jose Fault was found to control over most of the spectrum.

The spectral accelerations revealed by the computations were multiplied by 1.8 to provide the MCE_R values. Following is a summary of the Deterministic Spectral Response Acceleration Values and Comparison with Deterministic Lower Limit.

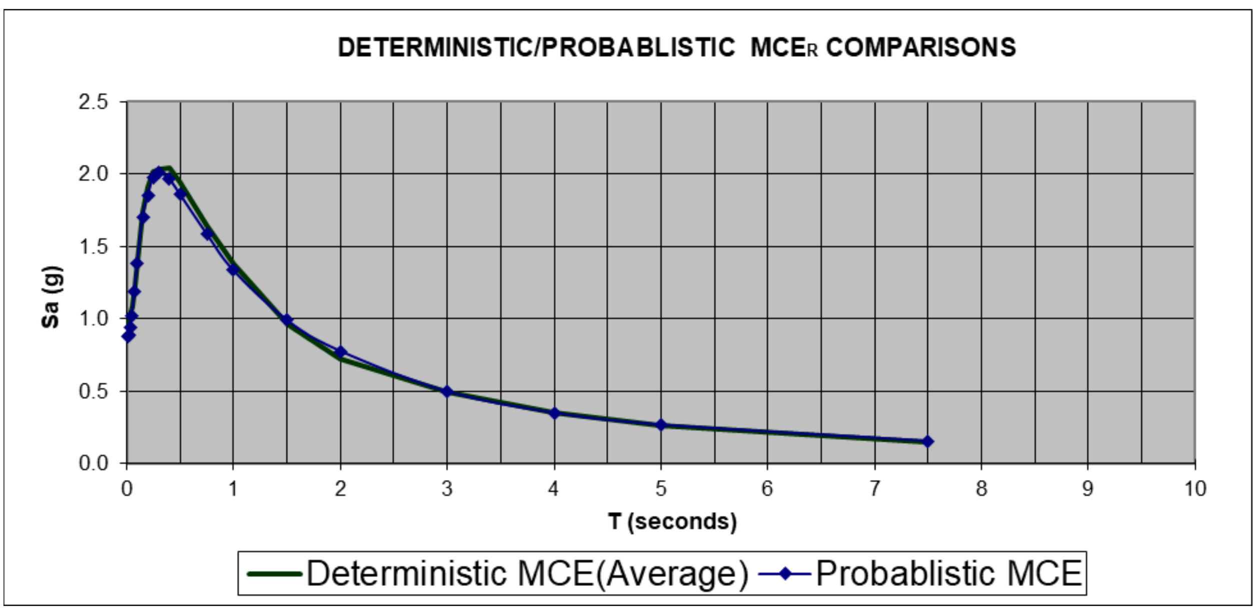
Deterministic Summary and Comparison with Deterministic Lower Limit - Section 21.2.2

T	Whittier Fault $S_{a(Average)}$	San Jose Fault S_a (Average)	***S_a (Average) *1.8***	***Max. Rotated S_a***	Lower Limit	Value	Method
0.010	0.36	0.45	0.81	0.97	0.68	0.97	Deterministic
0.020	0.36	0.46	0.83	0.98	0.75	0.98	Deterministic
0.030	0.37	0.48	0.86	1.02	0.83	1.02	Deterministic
0.050	0.39	0.51	0.91	1.09	0.98	1.09	Deterministic
0.075	0.45	0.57	1.03	1.23	1.16	1.23	Deterministic
0.100	0.51	0.66	1.18	1.41	1.35	1.41	Deterministic
0.150	0.63	0.80	1.45	1.74	1.50	1.74	Deterministic
0.200	0.68	0.88	1.58	1.91	1.50	1.91	Deterministic
0.250	0.72	0.92	1.65	2.01	1.50	2.01	Deterministic
0.300	0.73	0.93	1.67	2.04	1.50	2.04	Deterministic
0.400	0.72	0.92	1.66	2.04	1.50	2.04	Deterministic
0.500	0.69	0.88	1.58	1.94	1.50	1.94	Deterministic
0.750	0.60	0.74	1.33	1.64	1.20	1.64	Deterministic
1.000	0.51	0.62	1.11	1.38	0.90	1.38	Deterministic
1.500	0.40	0.43	0.78	0.96	0.60	0.96	Deterministic
2.000	0.32	0.32	0.58	0.72	0.45	0.72	Deterministic
3.000	0.22	0.19	0.40	0.50	0.30	0.50	Deterministic
4.000	0.16	0.13	0.28	0.36	0.23	0.36	Deterministic
5.000	0.12	0.09	0.21	0.27	0.18	0.27	Deterministic
7.500	0.07	0.04	0.12	0.15	0.12	0.15	Deterministic
10.000	0.03	0.02	0.06	0.08	0.07	0.08	Deterministic
PGA	0.36	0.45	0.81	g			

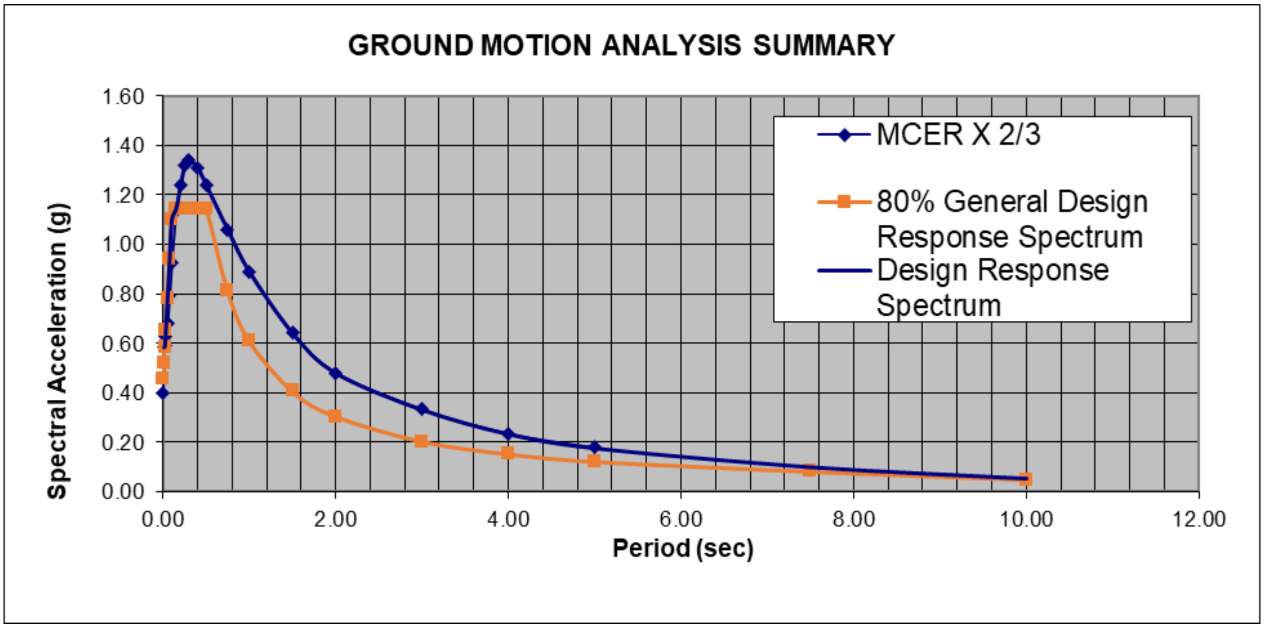
The average spectral accelerations are indicated graphically in the following diagram which also includes the Deterministic Lower Limit. This graph indicates that the deterministic MCE_R values are greater than the Deterministic Lower Limit values.



♦ <u>***Site Specific MCE$_R$ (ASCE 7 Section 21.2.3)***</u>- 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 are plotted in the following diagram:



♦ <u>***Design Response Spectrum (ASCE 7 Section 21.3)***</u>- In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3 S_{aM}$, 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:



♦ <u>***Design Acceleration Parameters (ASCE 7 Section 21.4)***</u>- Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as the spectral acceleration, S_a, 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 spectral acceleration, S_a, at a period of 1 s or two times the spectral acceleration, S_a, at a period of 2 sec. 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 11.4.3 for S_{MS}, and S_{M1} and Section 11.4.4 for S_{DS} and S_{D1}.

- ***Site Specific Design Parameters***- For the 0.2 second period, a value of 1.24g (S_{DS}) was computed based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.34g occurring at T=0.30. This was multiplied by 0.9 to produce a value of 1.21g which is less than 1.24g, confirming 1.24g as the applicable value. A value of 0.960g (S_{D1}) for the 1.0 second period was also calculated (ASCE 7-10, 21.4). For the MCE_R 0.2 second period, a value of 1.860g (S_{MS}) was computed, along with a value of 1.440g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-10, 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 0.74g. 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 0.81g. The site-specific MCE_G peak ground acceleration was calculated to be 0.74g, which was determined by using the lesser of the probabilistic (0.74g) or the deterministic (0.81g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 0.774g x 0.80 = 0.62g).

SEISMIC DESIGN PARAMETERS SUMMARY

Project:	Stanley G. Oswalt Academy	Lattitude:	34.027
Project #:	183175-1	Longitude:	-117.88
Date:	11/23/18		

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-10

Mapped Acceleration Parameters per ASCE 7-10, Chapter 22

S_s= 2.147 Figure 22-1

S_1= 0.762 Figure 22-2

Site Class per Table 20-3.1

Site Class= D

Site Coefficients per 1613A.3.3

F_a= 1 Table 11.4-1

F_v= 1.5 Table 11.4-2

Mapped Design Spectral Response Acceleration Parameters

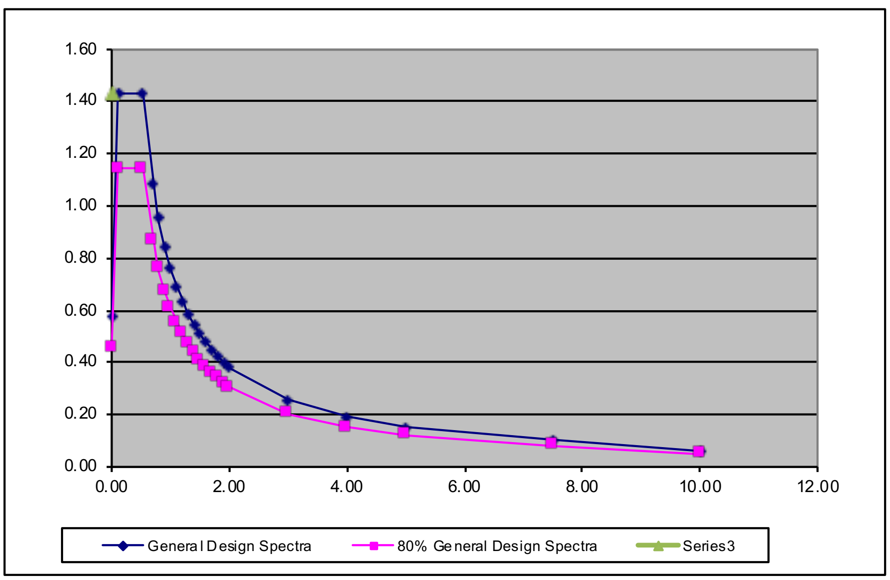
S_{Ms}=	2.147	Equation 11.4-1
S_{M1}=	1.143	Equation 11.4-2

S_{DS}=	1.4313333	Equation 11.4-3
S_{D1}=	0.762	Equation 11.4-4

T_0=	0.106	sec	
T_S=	0.532	sec	
T_L=	8	sec	From Fig 22-12
PGA	0.774	g	
F_{PGA}=	1.00		From Table 11.8-1
C_{RS}=	0.996		Figure 22-17
C_{R1}=	1.012		Figure 22-18

Response Spectra

Period (T)	Sa	.8XSa
0.00	0.57	0.458
0.11	1.43	1.145
0.53	1.43	1.145
0.70	1.09	0.871
0.80	0.95	0.762
0.90	0.85	0.677
1.00	0.76	0.610
1.10	0.69	0.554
1.20	0.64	0.508
1.30	0.59	0.469
1.40	0.54	0.435
1.50	0.51	0.406
1.60	0.48	0.381
1.70	0.45	0.359
1.80	0.42	0.339
1.90	0.40	0.321
2.00	0.38	0.305
3.00	0.25	0.203
4.00	0.19	0.152
5.00	0.15	0.122
7.50	0.10	0.081
10.00	0.06	0.049



ASCE 7-10 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

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

"Presented data are based upon Deterministic & Probablistic Analyses using NGA Relationships of Chiou & Youngs (2008), Abrahamsom et.al. (2008) and Boore et.al.(2008)"

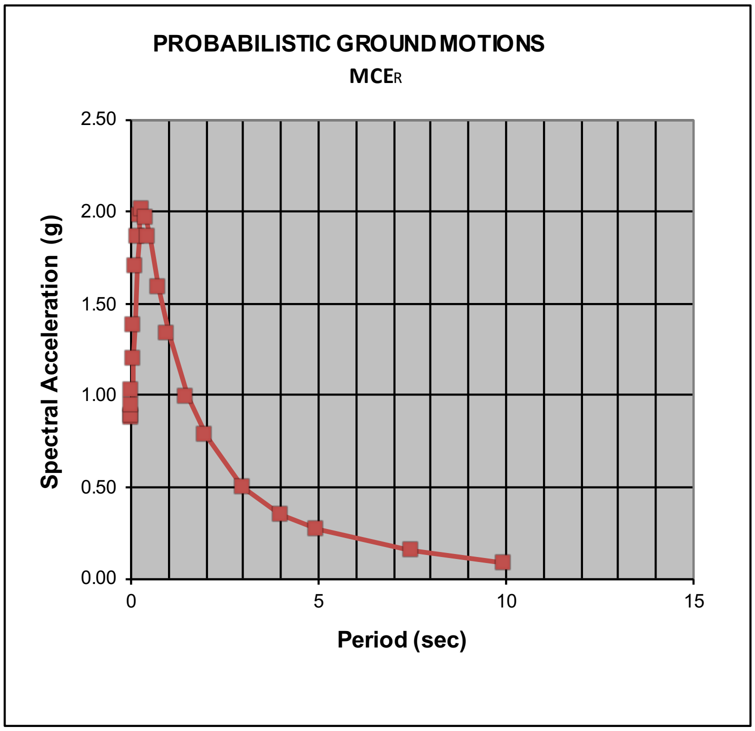
Probabilistic MCE_R per 21.2.1.1

OpenSHA data

2% Probability Of Exceedance in 50 years

"Presented data are based upon Deterministic & Probablistic Analyses using NGA Relationships of Chiou & Youngs (2008), Abrahamsom et.al. (2008) and Boore et.al.(2008)"

	Sa	
T	2% in 50	MCE_R
0.01	0.88	0.88
0.02	0.89	0.89
0.03	0.94	0.94
0.05	1.02	1.02
0.08	1.20	1.19
0.10	1.39	1.38
0.15	1.71	1.70
0.20	1.86	1.86
0.25	1.96	1.98
0.30	1.99	2.01
0.40	1.94	1.97
0.50	1.84	1.86
0.75	1.57	1.59
1.00	1.32	1.34
1.50	0.98	0.99
2.00	0.77	0.78
3.00	0.49	0.50
4.00	0.35	0.35
5.00	0.27	0.27
7.50	0.15	0.16
10.00	0.09	0.09



Risk Coefficients:

C_{RS}	0.996	Figure 22-17
C_{R1}	1.012	Figure 22-18

S_s=	1.86	1.86
S_1=	1.57	1.59
PGA	0.76	g

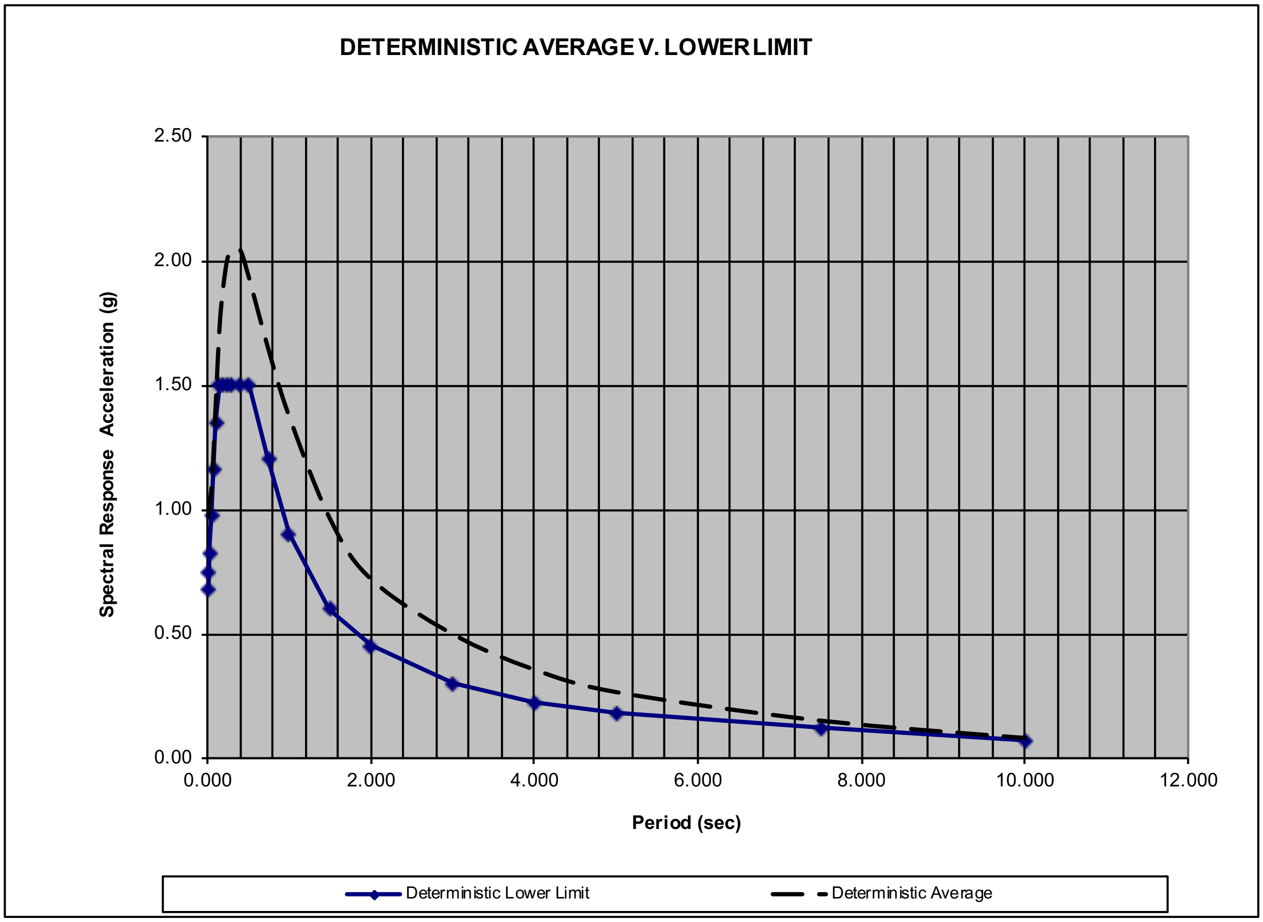
DETERMINISTIC MCE per 21.2.2

Input Parameters		Whittier Fault	San Jose Fault
M	= Moment magnitude	7.8	6.7
R_{RUP}	= Closest distance to coseismic rupture (km)	8.7	0.76
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	8.7	0.76
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	8.7	0.76
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	1	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	0	0
δ	= Average dip of rupture plane (degrees)	84	74
V_{S30}	= Average shear-wave velocity in top 30m of site profile	218.4	218.4
$F_{Measured}$		1	1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (m)	400	400
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	2	2
W (km)		16	16
F_{AS}		0	0
HW Taper		0	0
σ		0	0

"Presented data are based upon Deterministic & Probablistic Analyses using NGA Relationships of Chiou & Youngs (2008), Abrahamsom et.al. (2008) and Boore et.al.(2008)"
**Computed per Boore et. Al. (2011)

Deterministic Summary and Comparison with Deterministic Lower Limit - Section 21.2.2

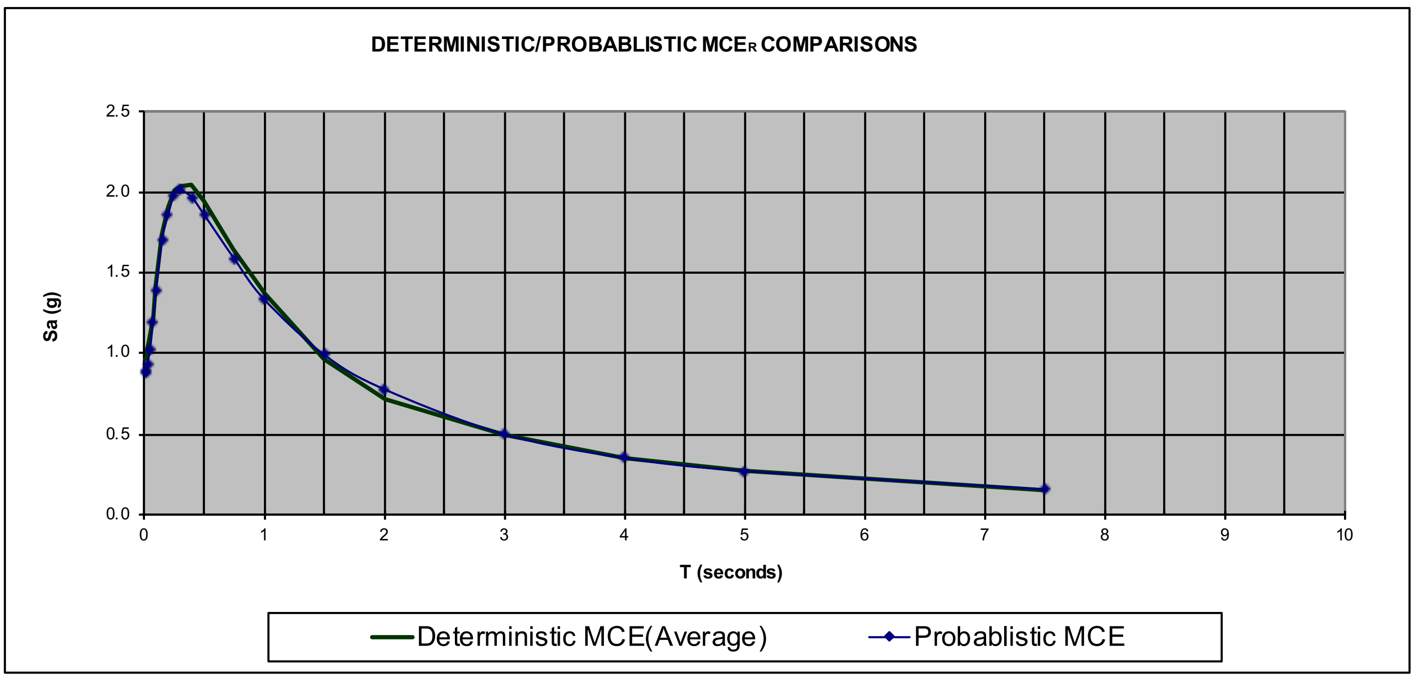
T	Whittier Fault $S_{a(Average)}$	San Jose Fault $S_{a\ (Average)}$	$S_{a\ (Average)}$ *1.8	Max. Rotated S_a	Lower Limit	Value	Method
0.010	0.36	0.45	0.81	0.97	0.68	0.97	Deterministic
0.020	0.36	0.46	0.83	0.98	0.75	0.98	Deterministic
0.030	0.37	0.48	0.86	1.02	0.83	1.02	Deterministic
0.050	0.39	0.51	0.91	1.09	0.98	1.09	Deterministic
0.075	0.45	0.57	1.03	1.23	1.16	1.23	Deterministic
0.100	0.51	0.66	1.18	1.41	1.35	1.41	Deterministic
0.150	0.63	0.80	1.45	1.74	1.50	1.74	Deterministic
0.200	0.68	0.88	1.58	1.91	1.50	1.91	Deterministic
0.250	0.72	0.92	1.65	2.01	1.50	2.01	Deterministic
0.300	0.73	0.93	1.67	2.04	1.50	2.04	Deterministic
0.400	0.72	0.92	1.66	2.04	1.50	2.04	Deterministic
0.500	0.69	0.88	1.58	1.94	1.50	1.94	Deterministic
0.750	0.60	0.74	1.33	1.64	1.20	1.64	Deterministic
1.000	0.51	0.62	1.11	1.38	0.90	1.38	Deterministic
1.500	0.40	0.43	0.78	0.96	0.60	0.96	Deterministic
2.000	0.32	0.32	0.58	0.72	0.45	0.72	Deterministic
3.000	0.22	0.19	0.40	0.50	0.30	0.50	Deterministic
4.000	0.16	0.13	0.28	0.36	0.23	0.36	Deterministic
5.000	0.12	0.09	0.21	0.27	0.18	0.27	Deterministic
7.500	0.07	0.04	0.12	0.15	0.12	0.15	Deterministic
10.000	0.03	0.02	0.06	0.08	0.07	0.08	Deterministic
PGA	0.36	0.45	0.81	g			



DETERMINISTIC AVERAGE V. LOWER LIMIT
Spectral Response Acceleration (g)
2.50
2.00
1.50
1.00
0.50
0.00
0.000
2.000
4.000
6.000
8.000
10.000
12.000
Period (sec)
Deterministic Lower Limit
Deterministic Average

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probablistic MCE_R Values (S_a) per 21.2.3
"Presented data are based upon Deterministic & Probablistic Analyses using NGA Relationships of Chiou & Youngs (2008), Abrahamsom et.al. (2008) and Boore et.al.(2008)"

Period T	Determisitic MCE_R	Probablistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.97	0.88	0.88	ProbablisticGoverns
0.020	0.98	0.89	0.89	ProbablisticGoverns
0.030	1.02	0.94	0.94	ProbablisticGoverns
0.050	1.09	1.02	1.02	ProbablisticGoverns
0.075	1.23	1.19	1.19	ProbablisticGoverns
0.100	1.41	1.38	1.38	ProbablisticGoverns
0.150	1.74	1.70	1.70	ProbablisticGoverns
0.200	1.91	1.86	1.86	ProbablisticGoverns
0.250	2.01	1.98	1.98	ProbablisticGoverns
0.300	2.04	2.01	2.01	ProbablisticGoverns
0.400	2.04	1.97	1.97	ProbablisticGoverns
0.500	1.94	1.86	1.86	ProbablisticGoverns
0.750	1.64	1.59	1.59	ProbablisticGoverns
1.000	1.38	1.34	1.34	ProbablisticGoverns
1.500	0.96	0.99	0.96	Deterministic Governs
2.000	0.72	0.78	0.72	Deterministic Governs
3.000	0.50	0.50	0.50	ProbablisticGoverns
4.000	0.36	0.35	0.35	ProbablisticGoverns
5.000	0.27	0.27	0.27	Deterministic Governs
7.500	0.15	0.16	0.15	Deterministic Governs
10.000	0.08	0.09	0.08	Deterministic Governs



DESIGN RESPONSE SPECTRUM per Section 21.3

Period	MCE_R X 2/3	80% General Design Response Spectrum	Design Response Spectrum
0.00	0.40	0.46	0.46
0.010	0.59	0.52	0.59
0.020	0.59	0.59	0.59
0.030	0.62	0.65	0.65
0.050	0.68	0.78	0.78
0.075	0.79	0.94	0.94
0.100	0.92	1.10	1.10
0.150	1.13	1.15	1.15
0.200	1.24	1.15	1.24
0.250	1.32	1.15	1.32
0.300	1.34	1.15	1.34
0.400	1.31	1.15	1.31
0.500	1.24	1.15	1.24
0.750	1.06	0.81	1.06
1.000	0.89	0.61	0.89
1.500	0.64	0.41	0.64
2.000	0.48	0.30	0.48
3.000	0.33	0.20	0.33
4.000	0.23	0.15	0.23
5.000	0.18	0.12	0.18
7.500	0.10	0.08	0.10
10.000	0.05	0.05	0.05

DESIGN ACCELERATION PARAMETERS per Section 21.4

Highest value of S_a for any period exceeding 0.2 sec.=	1.34
90%of Highest Value =	1.21
2 X Sa @ T= 2 sec.=	0.96

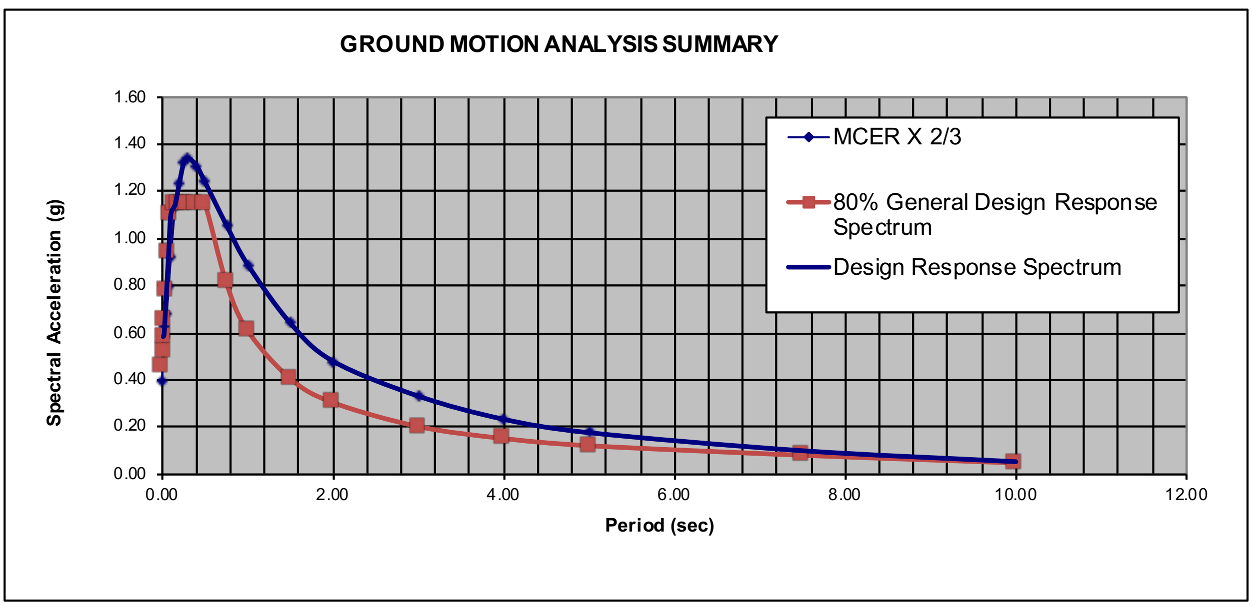
S_{DS}=	1.24
S_{D1}=	0.96

S_{MS}=	1.86
S_{M1}=	1.44

PGA Determination:

Site Coefficient F_{PGA}=	1.000	
Mapped PGA=	0.774	Figure 22-7
PGA_M =	0.774	

Deterministic PGA =	0.81	g
Probabalistic PGA =	0.76	g
Lesser of Deterministic/Probabilistic =	0.76	g
80% of $PGA_{M=}$	0.62	g
MCE_G PGA=	0.76	g



APPENDIX C

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REFERENCES

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APPENDIX E

PERCOLATION TEST RESULTS

PERCOLATION TEST

Project Location: StanleyG. Oswalt Academy
19501 Shadow Oak Drive
Walnut, CA

Boring/Test Number: P-1

Project No.: 18-06-3763
Earth Description: Silty Sand
Tested by: Chris
Liquid Description: Tap Water
Measurement Method: Sounder

Diameter of Boring: 8″
Depth of Boring: 3.0′
Depth to Invert of Proposed BMP: 3.0′
Depth to Current Water Table Per Ref. 1: 40′
Depth to Initial (Pre-Soak) Water Surface: 1′
Depth to Final Water: 1.1′

Time Interval: Standard
Start Time for Pre-Soak: 9:00 A.M.
Start Time for Standard: 9:20 A.M.
Date: 11/19/2018

Water Remaining in Boring (Y/N): Y
Time Interval Between Readings: 10 min.

Reading Number	Time Start/End (hh:mm)	Elapsed Δ Time (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/Comments
1	9:23 9:53	30	1.2	2.4	Area Area = $\Pi(8)(24-\Delta d/2)+\Pi(4)^2$ $\Pi(8)(24-1.2/2)+\Pi(16) = 638.4$
2	9:53 10:23	30	7.2	14.4	
3	10:23 10:53	30	1.2	2.4	Measured Percolation Rate: MPR $= \dfrac{\Delta d \Pi(4)^2(2)}{\text{Area}} =$ MPR $= \dfrac{1.2 \times \Pi \times 16 \times 2}{638.4} = 0.19$ in/hr
4	10:53 11:23	30	1.2	2.4	Design Infiltration Rate: Measured Percolation Rate/CF
5	11:23 11:53	30	2.4	4.8	
6	11:53 12:23	30	0.0	0.0	CF = $RF_1 RF_v RF_s = 2 \times 1.5 \times 1.5 = 4.5$
7	12:23 12:53	30	1.2	2.4	Measured Infiltration = 0.19 in/hr
8	12:53	30	1.2	2.4	

Reading Number	**Time Start/End (hh:mm)**	**Elapsed ∆ Time (mins)**	**Water Drop During Standard Time Interval ∆d (inches)**	**Percolation Rate for Reading (in/hr)**	**Soil Description/Notes/Comments**
	1:23				Design Infiltration Rate (DIR)= 0.19/4.5 =0.04 in/hr DIR must be greater than 0.3in/hr
9	1:23 1:53	30	1.2	2.4	
10	1:53 2:23	30	1.2	2.4	P-1 is not acceptable
11	2:23 2:53	30	1.2	2.4	
12	2:53 3:23	30	1.2	2.4	

Table P-1

Harrington Geotechnical Engineering, Inc.

PERCOLATION TEST

Project Location:	StanleyG. Oswalt Academy 19501 Shadow Oak Drive Walnut, CA	**Boring/Test Number: P-2**
Project No.:	18-06-3763	
Earth Description:	Silty Clay	**Diameter of Boring:** 8″
Tested by:	Chris	**Depth of Boring:** 3.0′
Liquid Description:	Tap Water	**Depth to Invert of Proposed BMP: 3**.0′
Measurement Method:	Sounder	**Depth to Current Water Table Per Ref. 1:** 40′
		Depth to Initial (Pre-Soak) Water Surface: 1′
Time Interval: Standard		**Depth to Final Water:** 1.1′
Start Time for Pre-Soak:	9:00 A.M.	**Water Remaining in Boring (Y/N):** Y
Start Time for Standard:	9:25 A.M.	**Time Interval Between Readings:** 10 min.
Date:	11/19/2018	

Reading Number	Time Start/End (hh:mm)	Elapsed Δ Time (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/Comments
1	9:28	30	3.6	7.2	Measured Percolation Rate: Area = Π(8)(24- Δd/2)+Π(4)^2 Π(8)(24-1.2/2) +Π(16) =638.4 MPR = $\frac{\Delta d \Pi(4)^2(2)}{\text{Area}}$ = MPR = $\frac{1.2 x \Pi x\ 16 x 2}{638.4}$ = 0.19 in/h
	9:58				
2	9:58	30	9.6	19.2	
	10:28				
3	10:28	30	2.4	4.8	
	10:58				
4	10:58	30	2.4	4.8	Design Infiltration Rate: Measured Percolation Rate/CF CF = $RF_1 RF_V RF_S$ = 2x1.5x1.5 = 4.5 Measured Infiltration = 0.19 in/hr
	11:28				
5	11:28	30	1.2	2.4	
	11:58				
6	11:58	30	1.2	2.4	
	12:28				
7	12:28	30	1.2	2.4	
	12:58				
8	12:58	30	1.2	2.4	

Reading Number	**Time Start/End (hh:mm)**	**Elapsed ∆ Time (mins)**	**Water Drop During Standard Time Interval ∆d (inches)**	**Percolation Rate for Reading (in/hr)**	**Soil Description/Notes/Comments**
	1:28				Design Infiltration Rate (DIR)= 0.19/4.5 =0.04 in/hr DIR must be greater than 0.3in/hr
9	1:28 1:58	30	1.2	2.4	
10	1:58 2:28	30	1.2	2.4	P-2 is not acceptable
11	2:28 2:58	30	1.2	2.4	
12	2:58 3:28	30	1.2	2.4	

Table P-2