John R. Byerly
INCORPORATED

GEOTECHNICAL INVESTIGATION

PATRICK HENRY HIGH SCHOOL

WHOLE SITE MODERNIZATION, PHASE II

SAN DIEGO, CALIFORNIA

SAN DIEGO UNIFIED SCHOOL DISTRICT
C/O PJHM ARCHITECTS, INC.

GEOTECHNICAL ENGINEERS • TESTING AND INSPECTION
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AUGUST 27, 2015

PATRICK HENRY HIGH SCHOOL

WHOLE SITE MODERNIZATION, PHASE II

6702 WANDERMERE DRIVE

SAN DIEGO, CALIFORNIA

CLIENT:

SAN DIEGO UNIFIED SCHOOL DISTRICT

C/O PJHM ARCHITECTS, INC.

804 PIER VIEW WAY, SUITE 103

OCEANSIDE, CALIFORNIA 92054-2813

ATTENTION: JAMES BUCKNAM, LEED-AP

RPT. NO.: 2643-b (revised)
FILE NO.: S-13573

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INTRODUCTION

During August and December of 2014, an investigation of the soil conditions underlying the area of the proposed two-story classroom building and administration building addition at the existing Patrick Henry High School was conducted by this firm. The purpose of our investigation was to evaluate the surface and subsurface conditions at the site with respect to safe and economical foundation types, vertical and lateral bearing values, liquefaction and seismic settlement potential, support of concrete slabs-on-grade, and site preparation. Included in the recommendations are the seismic design parameters as required by the 2013 edition of the California Building Code and the ASCE Standard 7-10. Recommendations are also provided for the design of asphalt concrete and portland cement concrete pavement for the proposed parking and driveway areas. The geologic conditions attendant to the site have been evaluated by our consulting engineering geologist, AKW Geotechnical, as required by the California Geological Survey. The engineering geology investigation report is presented herewith as Enclosure 10. Our geotechnical investigation, together with our conclusions and recommendations, is discussed in detail in the following report.

This report has been prepared for the exclusive use of the San Diego Unified School District and their design consultants for specific application to the project described herein. Should the project be modified, the conclusions and recommendations presented in this report should reviewed by the geotechnical engineer. Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, express or implied.

PROJECT DESCRIPTION

For the preparation of this report, we reviewed the project site plan prepared by PJHM Architects, Inc. We understand that planned improvements to the existing Patrick Henry High School will consist of the construction of a two-story classroom building and an addition to the existing Administration building. The two-story classroom building will be constructed in the southeast portion of the high school campus. The west side of the existing administration building will receive an addition. The two-story classroom building will have a footprint area of approximately 20,000 square feet and the administration building addition will be on the order of 1,000 square feet. The two-story classroom building and administration building addition will be of steel-frame and concrete block masonry construction and will incorporate concrete slab-on-grade floors. The new buildings will exert moderate to heavy foundation loads on the underlying soils. As part of the development, a storm water retention basin is planned for an area west of the proposed two-story classroom building. The site for the new structures appear to be at the approximate desired grade, and no significant additional cuts and fills seem likely. The partial site configuration and proposed development are illustrated on Enclosure 1.

SITE CONDITIONS

The existing Patrick Henry High School is located on the north side of Wandermere Drive and west of Park Ridge Boulevard in the city of San Diego. An Index Map showing the general vicinity of the site is presented on the following page. The coordinates of the site are latitude 32.7969° N and longitude 117.05043° W utilizing the North American Datum 1983 (NAD 1983). At the time of our site reconnaissance, we noted that the area of the proposed two-story classroom building was developed with asphalt concrete paved parking, portland cement concrete walkways, and landscaping. The area of the proposed administration building addition is presently occupied by hardscape and raised planters. The area topography is generally flat; the site slopes downward to the west at an average gradient of about 3 percent.

FIELD AND LABORATORY INVESTIGATION

The soils underlying the area of the proposed two-story classroom building and administration building addition were explored by means of six test borings drilled with a truck-mounted flight-auger to depths of up to 22 feet below the existing ground surface. We were unable to place a conventional drill rig in the raised planter area where additional subsurface exploration was needed for the administration building addition. We therefore further explored the soils underlying the administration building addition site by means of a single test boring excavated with hand-auger equipment where refusal occurred at a depth of about 3.5 feet. The approximate locations of the explorations are indicated on Enclosure 1. The soils encountered were examined and visually classified by one of our field engineers. A summary of the soil classifications appears as Enclosure 2. The exploration logs show subsurface conditions at the dates and locations indicated, and may not be representative of other locations and times. The

INDEX MAP

SOURCE DOCUMENT: United States Geological Survey 7.5-Minute Series (Topographic) LA MESA (2012) Quadrangle

TOWNSHIP AND RANGE: T17S R2W SEC 16

LATITUDE: 32.7969N

LONGITUDE: 117.05043W

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stratification lines presented on the logs represent the approximate boundaries between soil types, and the transitions may be gradual. A hollow-stem auger with an outside diameter of 8.5 inches was utilized. The inside diameter of the auger was 4.5 inches.

Bulk and relatively undisturbed samples were obtained at selected levels within the explorations and returned to our laboratory for testing and evaluation. The driving energy or blow counts required to advance the sampler at each sample interval was also noted. Relatively undisturbed soil samples were recovered at various intervals in the borings with a California sampler. The California sampler was a 2.9-inch outside diameter, 2.5-inch inside diameter, split-barrel sampler lined with brass tubes. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was lifted hydraulically and was dropped 30 inches for each blow. Standard penetration tests were performed as Borings 1 and 2 were advanced. The standard penetration test blow counts are shown on the logs for Borings 1 and 2. Standard penetration testing was performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was lifted hydraulically and was dropped 30 inches for each blow. An efficiency value of 1.0 was assumed for the automatic trip hammer. The estimated equivalent standard penetration "N" values are also shown on the boring logs, Enclosure 2

Included in our laboratory testing were moisture/density determinations on all undisturbed samples. Optimum moisture content/maximum dry density relationships were established for typical soil types so that the relative compaction of the subsoils could be determined. Consolidation testing was conducted on selected samples to evaluate the compressibility characteristics of the soil. Expansion index testing was performed on representative samples of soil containing detectable clay. Direct shear tests were conducted on selected samples to determine their strength parameters. The moisture/density data are presented on the boring logs, Enclosure 2. Maximum density test data appear on Enclosure 3. The results of the consolidation and expansion index testing are shown on Enclosures 4 and 5, respectively. The results of the direct shear testing are shown on Enclosure 6. Subgrade soil test data are summarized on Enclosure 7. Chemical testing, comprised of pH, soluble sulfate, chloride, redox potential, and resistivity testing, was also performed. The test results are presented in the "Chemical Test Results" section of this report.

SOIL CONDITIONS

Borings 1, 2, 5, and 7 were drilled through 4.5 to 9.0 inches of asphalt concrete pavement. Artificial fill consisting of dense silty sands with some gravel, clayey fine sands, medium stiff to stiff sandy silts, sandy silts with clay, and clayey silts with sand was encountered in a majority of our test borings to depths of up to 4.0 feet. The fill is associated with previous grading at the site. The natural soils immediately underlying the fill consisted of dense to very dense silty sands with traces of clay, silty sands with gravel and cobbles, and stiff sandy silts. All other underlying natural soils encountered in our test borings generally consisted of dense to very dense silty sands with varying amounts of clay, gravel and cobbles, gravelly sands with cobbles, and stiff to very stiff sandy silts with clay. Refusal to the truck-mounted flight-auger occurred on cobbles in all of our test borings. Based on published geologic reports for this area, very dense soil is considered to extend to a depth of at least 100 feet beneath the site. The depths of fill and depths to refusal are itemized on the following table:

Boring Number	Depth of Fill (ft.)	Depth to Refusal (ft.)
B-1	1.5	18.0
B-2	NA	22.0
B-3	3.0	16.0
B-4	1.5	12.0
B-5	NA	13.0
B-6	2.5	3.5
B-7	4.0	11.0

Neither bedrock nor ground water was encountered at our exploration locations. The near-surface soils encountered in our test borings were determined to have a very low to medium expansion potential in accordance with ASTM D 4829. The soils at the time of our investigation were at elevated moisture contents. If these soils are at similar moisture contents at the time of remedial grading, the soils exposed in the bottom of subexcavation may be unstable (pumping) under the influence of the grading equipment, and stabilization will be needed.

LIQUEFACTION AND DYNAMIC SETTLEMENT

Liquefaction is a phenomenon that occurs when a soil undergoes a transformation from a solid state to a liquefied condition due to the effects of increased pore-water pressure. Loose saturated soils with particle sizes in the medium sand to silt range are particularly susceptible to liquefaction when subjected to seismic groundshaking. Affected soils lose all strength during liquefaction, and foundation failure can occur.

Free ground water was not encountered at our boring locations. Based on ground water data, our consulting engineering geologist estimates that the regional ground water table is at a depth of at least 60 feet below existing grade. Due to the great depth to ground water, we conclude that the potential for liquefaction is low.

It is anticipated that major earthquake ground shaking will occur during the lifetime of the proposed development from the seismically active Rose Canyon fault zone located approximately 8.3 miles west of the site. This fault would create the most significant earthshaking event. Based on an earthquake magnitude of 6.9, a peak horizontal ground acceleration of 0.364g is assigned to the site. To evaluate the potential for seismically induced settlement of the subsoils, the soils were analyzed for relative density. The most effective measurement of relative density of sands with respect to seismic settlement potential is standard penetration resistance. Standard penetration tests were performed as Borings 1 and 2 were advanced to depths of 18 feet and 22 feet, respectively. The standard penetration test "N" values are presented on the boring logs for Borings 1 and 2.

The standard penetration data provided input for the LiquefyPro Version 4.3 program for seismically induced settlement. As indicated in Special Publication 117A (Revised) Release, "Guidelines for Evaluating and Mitigating Seismic Hazards in California, March 2009," a safety factor of 1.3 was used in this analysis. We have assumed that the existing artificial fill will be overexcavated and replaced as engineered fill. The engineered fill was assumed to have an "N" value of 30. The results of this evaluation are shown on Enclosure 9 and reveal a low potential for liquefaction. The analysis also reveals a maximum total potential dynamic settlement of 0.03 inch. This maximum value of potential dynamic settlement is quite small and is not considered significant. It is our opinion that neither liquefaction nor seismically induced dry settlement need be a consideration in the design of the proposed two-story classroom building and administration building addition.

CONCLUSIONS

The artificial fill is non-uniform and undocumented. To assure uniform and acceptable foundation conditions, we recommend that the existing artificial fill within the new building areas be overexcavated and replaced as engineered fill as indicated below under "Site Preparation." Subsequent to site preparation, the structures may be safely founded on conventional continuous and pad footings. The near-surface soils encountered in our test borings have a very low to medium expansion potential in accordance with ASTM D 4829. Inasmuch as soil underlying this site exhibits an expansion index greater than 20, the foundations and slabs for soils should be designed in accordance with "WRI/CRSI Design of Slab-on-Ground Foundations" or "PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils." Recommendations for foundation design and slabs-on-grade are provided below for medium (expansion index of 51 to 90) expansion potential. Our recommendations address the WRI/CRSI foundation design practice. If desired, recommendations for post-tension slab foundations will be provided under separate cover. Foundations should not be allowed to span from cut to fill soil conditions. Detailed recommendations are provided below.

RECOMMENDATIONS

FOUNDATION DESIGN AND SLABS-ON-GRADE

Medium Expansive Soils (EI = 51 to 90)

Foundations constructed for near-surface soils exhibiting a medium expansion potential should also be designed in accordance with the WRI practice as required by Section 1808.6.1 of the CBC. This design practice is considered a minimum. The following recommendations are considered supplemental. A plasticity index of 25 and an unconfined compressive strength of 250 pounds per square foot should be assumed. The soil bearing pressure should not exceed 2,000 pounds per square foot for dead plus live loads. This value may be increased by one-third for wind and seismic loading. Footings should have an embedment depth of at least

18 inches. Reinforcement in the footings should consist of at least four No. 4 bars, two placed near the top and two near the bottom of the footings. Since the application of C_O for soils exhibiting unconfined compressive strengths of less than 6,000 pounds per square foot is permissive in the Code rather than mandatory, we recommend that C_O not be applied to the weighted average plasticity index. The plasticity index presented above should be considered an effective plasticity index inasmuch as the unconfined compressive strengths for the soils are less than 300 pounds per square foot and C_O is undefined.

Slabs should be at least 4 inches in thickness and should be reinforced with at least 6"x6"-W2.9/W2.9 welded wire reinforcement or equivalent. The subgrade soils should be moistened to at least 120 percent of the optimum moisture content to a depth of 18 inches within 48 hours of the placement of the concrete. Geotechnical verification of subgrade moisture conditioning is recommended. Where moisture-sensitive floor coverings are anticipated, the building slab-on-grade floors should be underlain by a moisture vapor retardant membrane, such as 10-mil Stego wrap or equivalent. The moisture vapor retardant membrane should conform to ASTM E 1745-97 (Standard Specification for Plastic Vapor Retarders Used in Contact with Earth or Granular Fill under Concrete Slabs). The moisture vapor retardant membrane should be lapped into the footing excavation to provide full coverage of the subgrade soils. Plumbing protrusions and membrane overlaps should be taped using Stego polyurethane construction grade seaming tape or equivalent to minimize moisture emissions through the membrane. The project superintendent and/or a representative of the geotechnical engineer should inspect the placement of the moisture vapor retardant membrane prior to covering. Installation of the moisture vapor retardant membrane should be performed in accordance with ASTM E 1643-98 (Standard Practice of Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill under Concrete Slabs). Slab concrete should be placed directly on the moisture vapor retardant membrane. To minimize shrinkage cracking and curling, we recommend a maximum water-cement ratio for slab concrete of 0.45.

SEISMIC DESIGN PARAMETERS

The development of the seismic ground motion parameters is described in detail in the engineering geology investigation report performed in our behalf by AKW Geotechnical

(Enclosure 10). In summary, the 2013 California Building Code and the ASCE Standard 7-10 coefficients and factors are provided in the following table:

Factor or Coefficient	***Value***
Latitude	32.7969° N
Longitude	117.05043° W
Mapped S_S	0.890g
Mapped S_1	0.344g
F_a	1.044
F_v	1.456
Final S_{MS}	0.929g
Final S_{M1}	0.501g
Final S_{DS}	0.620g
Final S_{D1}	0.334g
PGA	0.364g
T_L	8 seconds
Site Class	C

LATERAL LOADING

Resistance to lateral loads will be provided by passive earth pressure and basal friction. For footings bearing against compacted fill, passive earth pressure may be considered to develop at a rate of 350 pounds per square foot per foot of depth. Basal friction may be computed at 0.4 times the normal dead load. The resistance from basal friction and passive earth pressure may be combined directly without reduction. The allowable lateral resistance may be increased by one-third for wind and seismic loading.

SITE PREPARATION

We assume that the site will be prepared in accordance with the California Building Code and the current city of San Diego Grading Ordinance. The recommendations presented below are to establish additional grading criteria. These recommendations should be considered preliminary and are subject to modification or expansion based on a geotechnical review of the project foundation and grading plans.

- All areas to be graded should be stripped of organic matter, man-made obstructions, and other deleterious materials. Underground utilities should be removed and relocated or abandoned. All cavities created during site clearing should be cleaned of loose and disturbed soil, shaped to provide access for construction equipment, and backfilled with fill placed and compacted as described below.

- Artificial fill should be removed from all improvement areas. The maximum depth of existing artificial fill encountered in our test borings was 4 feet. The existing artificial fill may extend to greater depths in areas not explored.

- Overexcavation

    - Building areas – To assure uniform soil conditions underlying the building footings, the natural soil should be overexcavated to a depth of at least 2 feet below the bottom of the footings. The exposed surface should be evaluated by the representative of the geotechnical engineer. Natural soil exhibiting a relative compaction of less than 85 percent (ASTM D 1557) should be overexcavated to expose competent natural soil. Competent natural soil is defined as undisturbed material exhibiting a relative compaction of at least 85 percent (ASTM D 1557). The overexcavation should extend beyond the building areas a horizontal distance at least equal to the depth of overexcavation below the final ground surface or 5 feet, whichever distance is greater. A representative of this firm should observe the bottom of all excavations.

    - Pavement and hardscape areas – Subsequent to removal of existing artificial fill, the natural soils below asphalt concrete pavement and portland cement concrete areas should be overexcavated to a depth of 12 inches below existing grade or 12 inches below proposed finished grade, whichever is deeper. Finished grade is defined as the elevation of the top of the subgrade.

- At the time of our subsurface exploration, the upper soils were at elevated moisture contents. If the soils are at similar elevated moisture contents at the time of remedial grading, these soils will need to be dried back to near the optimum moisture content prior to placement as engineered fill. If the soils exposed in the subexcavated surface are at elevated moisture contents, these soils may become unstable (pumping) under the grading equipment and may require stabilization. Stabilization can be accomplished by allowing the soil to dry back to near optimum moisture content. Stabilization can also be accomplished by placing geogrid, such as Tensar BX1100, on the unstable subgrade followed by at least 12 inches of gravel, such as Caltrans Class II aggregate base. If sufficient stabilization has not been attained with 12 inches of aggregate base, an additional 6 inches of aggregate base should be placed.

- Subexcavated surfaces that do not require geogrid/aggregate base stabilization and all other surfaces to receive fill should be scarified to a minimum depth of 8 inches, moisture conditioned to within 2 percent of the optimum moisture content, and densified to a minimum relative compaction of 90 percent (ASTM D 1557).

- The on-site soils should provide adequate quality fill material provided they are free from organic matter and other deleterious materials and are at acceptable moisture contents. Rocks larger than 12 inches in greatest dimension (boulders) should be separated from the fill and placed in landscape areas at depths greater than 2 feet below final grade. Import fill should be inorganic, granular, non-expansive soil free from rocks or lumps greater than 6 inches in maximum dimension and should exhibit a very low expansion potential (expansion index less than 21), negligible sulfate content (less than 1,000 ppm soluble sulfate by weight), and low corrosion potential. Prior to bringing import fill to the site, the contractor should obtain certification to verify that the proposed import meets the State of California Department of Toxic Substance Control (DTSC) environmental standards. Proposed import should be sampled at the source and tested by this firm for expansion index, soluble sulfate content, and corrosion potential.

- All fill should be placed in 8-inch or less lifts, and each lift should be moisture conditioned. Clayey soil should be moisture conditioned to at least 2 percent over optimum moisture content. Fill with no significant clay content should be moisture conditioned to within 2 percent of the optimum moisture content. All engineered fill should be densified to a minimum relative compaction of 90 percent (ASTM D 1557).

The surface of the site should be graded to provide positive drainage away from the structures. Drainage should be directed to established swales and then to appropriate drainage structures to minimize the possibility of erosion. Water should not be allowed to pond adjacent to footings.

SHRINKAGE AND SUBSIDENCE

Volume change in going from cut to fill conditions is anticipated where near-surface grading will occur. Assuming the fill will be compacted to an average relative compaction of 93 percent, an average cut-fill shrinkage of 10 percent is estimated. Further volume loss will occur through subsidence during preparation of the natural ground surface. Although the contractor's methods and equipment utilized in preparing the natural ground will have a significant effect on the amount of natural ground subsidence that will occur, our experience indicates as much as 0.10 foot of subsidence in areas prepared to receive fill should be anticipated. These values are exclusive of losses due to stripping or removal of subsurface obstructions.

ASPHALT CONCRETE AND PORTLAND CEMENT CONCRETE PAVEMENT

Representative samples of upper soils at the site have been tested for relevant subgrade properties. A Traffic Index of 5.0 was assumed for interior parking and driveway areas for conventional vehicular traffic and fire lanes, and a Traffic Index of 6.0 was assumed where heavier truck and bus traffic will be accommodated. It is our understanding that the maximum weight of a tandem-axle fire truck is 68 kips. It is anticipated that a single fire truck will visit the site approximately twice a year. In conjunction with the test data shown on Enclosure 7, we believe the sections presented on the following table should provide durable pavement.

		"R"	Thickness (Inches)	
Location	**TI**	**Value**	**Asphalt Concrete**	**Aggregate Base**
Pavement areas for conventional passenger cars, light trucks, and fire lanes	5.0	15	2.5	9.5
Pavement areas for bus and heavier trucks	6.0	15	3.0	11.5

Location	TI	"R" Value	Thickness (Inches) Portland Cement Concrete
Pavement areas for conventional passenger cars and light trucks	5.0	15	4.5
Pavement areas for bus and heavier trucks	6.0	15	7.0

Aggregate base is geotechnically required for the PCC pavement sections to assure an unyielding subgrade. We recommend a minimum of 5 inches of aggregate base placed over the 12 inches of compacted subgrade soil. The design engineer may wish to provide some level of reinforcement to minimize the width of shrinkage cracks.

Prior to the placement of aggregate base, we recommend that the final subgrade surface be scarified to a depth of at least 12 inches, moisture conditioned to within 2 percent of the optimum moisture content, and compacted to a minimum relative compaction of at least 90 percent (ASTM D 1557).

Concrete should be proportioned for a maximum slump of 4 inches and to achieve a minimum compressive strength of 3,000 psi at 28 days. If additional workability is desired, a plasticizing or water-reducing admixture should be utilized in lieu of increasing the water content. Control joints for the 4.5-inch-thick pavement should be spaced no more than 13.5 feet on-center each way. The control joints for the 7-inch-thick pavement should be spaced no more than 21 feet on-center each way. Control joints should be established either by hand groovers, plastic inserts, or saw-cutting as soon as the concrete can be cut without dislodging aggregate. Cutting the control joints the day after the concrete pour will likely result in uncontrolled shrinkage cracks. Concrete should not be placed in hot and windy weather. Water curing should commence immediately after the final finishing and should continue for at least 7 days.

The above designs are preliminary and for estimating purposes only. We recommend that during the process of rough grading, observation and additional testing of the actual subgrade soils should be performed. Final pavement design sections can then be determined. The foregoing pavement sections assume that utility trench backfill below all proposed pavement areas will be compacted to at least 90 percent relative compaction. The upper 12 inches of subgrade below asphalt concrete pavement areas should be compacted to at least

90 percent relative compaction. Aggregate base should be densified to at least 95 percent relative compaction. Suggested specifications for aggregate base material are presented on Enclosure 8.

CHEMICAL TEST RESULTS

The chemical test results from a sample taken from Boring 7 between a depth of 0.5 foot and 2.5 feet are shown on the following table:

Analysis	Result	Units
Saturated Resistivity	1350	ohm-cm
Chloride	ND (Not Detected)	ppm
Sulfate	32	ppm
pH	8.0	pH units
Redox Potential	158	mV

The soil tested in Boring 7 exhibited negligible soluble sulfate content; therefore, sulfate-resistant concrete will not be required for this project.

The soil exhibits a low saturated resistivity and may be corrosive to buried ferrous-metal pipes. All other tested parameters are consistent with low corrosion potential. If buried ferrous metal pipe is to be utilized, we recommend further sampling and testing be performed during construction. If the test results continue to indicate a high corrosion potential, recommendations for corrosion protection should be obtained from a corrosion engineer.

FOUNDATION AND GRADING PLAN REVIEW

The project foundation and grading plans should be reviewed by the geotechnical engineer. Additional recommendations may be required at that time.

CONSTRUCTION OBSERVATIONS

All grading operations, including the preparation of the ground surface, should be observed and compaction tests performed by this firm. No fill should be placed on any prepared surface until that surface has been evaluated by the geotechnical engineer. All footing excavations should be observed by the geotechnical engineer prior to placement of forms or reinforcing steel.

The conclusions and recommendations presented in this report are based upon the field and laboratory investigation described herein, and represent our best engineering judgment. Should conditions be encountered in the field that appear different from those described in this report, we should be contacted immediately in order that appropriate recommendations might be prepared.

Respectfully submitted,

JOHN R. BYERLY, INC.

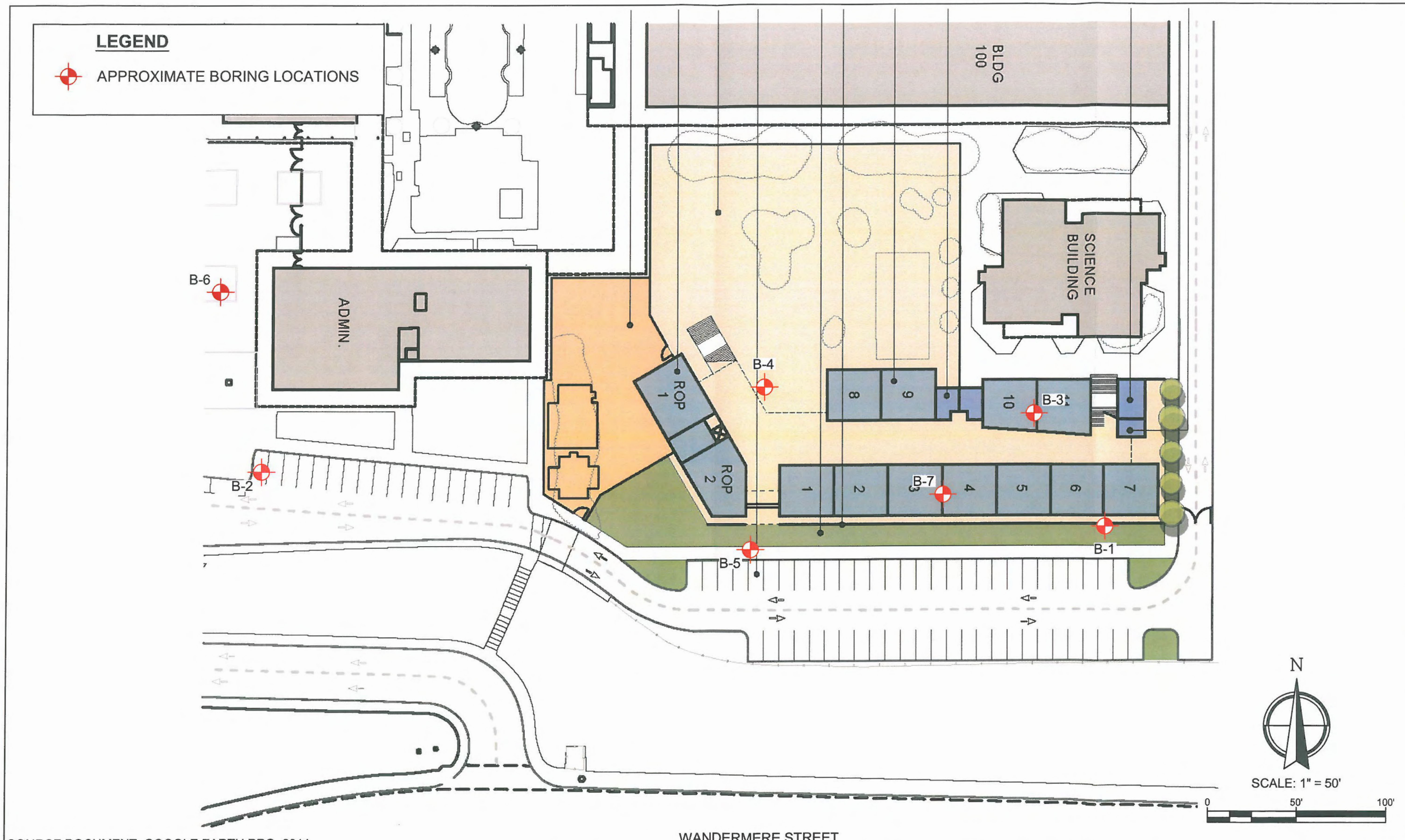
Michael L. Lozano
Staff Engineer

John R. Byerly, Geotechnical Engineer
President

REGISTERED PROFESSIONAL ENGINEER
JOHN R. BYERLY
No. GE000179
Exp. Date 6-30-17
GEOTECHNICAL
STATE OF CALIFORNIA

JRB:MLL:jet

Enclosures:
(1) Plot Plan
(2) Test Boring Logs
(3) Maximum Density Determinations
(4) Consolidation Test Results
(5) Expansion Index Test Results
(6) Direct Shear Test Results
(7) Subgrade Soil Tests
(8) Specifications for Aggregate Base
(9) Liquefaction and Dynamic Settlement Analysis
(10) Engineering Geology Investigation



SOURCE DOCUMENT: GOOGLE EARTH PRO, 2014

John R. Byerly INCORPORATED	GEOTECHNICAL INVESTIGATION TWO-STORY CLASSROOM BUILDING AND ADMINISTRATION BUILDING ADDITION	PATRICK HENRY HIGH SCHOOL 6702 WANDERMERE STREET SAN DIEGO, CALIFORNIA	Enclosure 1 Rpt. No.: 2643-b File No.: S-13573

Boring 1

Boring Date: 8/25/14
Surface Elevation:
Drilling Method: Flight-auger

Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	USCS	Description
0						GWT not encountered	AS	8.5 inches of asphalt concrete
		30	118	11.6	91		SC	Gray-brown clayey fine sand, moist and dense (FILL)
							ML	Light brown to gray-brown sandy silt, moist and stiff (ORIGINAL GROUND)
		40	125	12.2	94		SM	Light brown to yellow-brown silty fine to coarse sand with clay, moist and dense
5		50/8"	110	14.9	92		ML	Dark gray-brown sandy silt with clay, moist and very stiff
	51							
							SM	Brown silty fine to coarse sand with some gravel and cobbles, damp and very dense
10		50/4"	132	6.2	96			
	300							
15		50/1/2"	137	5.4	99			
	300							
							END	Refusal on Cobbles at 18.0 Feet No Free Groundwater Encountered
20								
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT From Myra\S-13573 (Rpt. No. 2643-a).log Date: 8/27/2015

LOG OF BORING

John R. Byerly, Inc.	Patrick Henry High School San Diego, California	Enclosure 2, Page 1 Report No.: 2643-b File No.: S-13573

Boring 2

Boring Date: 8/25/14
Surface Elevation:
Drilling Method: Flight-auger

Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	Soil	Description
0						GWT not encountered	AS	7.0 inches of asphalt concrete
		37	113	10.1	89		SM	Gray-brown silty fine to medium sand, moist and medium dense (ORIGINAL GROUND)
		50/11"	120	8.7	91		SM	Gray-brown to dark gray-brown silty fine to coarse sand with gravel, moist asnd dense
5		50/7 1/2"	130	2.8	94		SM	Brown silty fine to coarse sand with gravel and cobbles, dry and dense
	120							
10		50/3"	131	5.9	95			- becoming damp and very dense at 10.0 feet
	120							
							SP	Light brown gravelly sand with cobbles, dry and very dense
15		50/5"	135	2.4	98			
	300							
								- with seams of light olive fine to coarse sand with gravel at 18.0 feet
20		50/1 1/2"	137	2.2	99			
	300							
							END	Refusal on Cobbles at 22.0 Feet No Free Groundwater Encountered
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT From Myra\S-13573 (Rpt. No. 2643-a).log Date: 8/27/2015

LOG OF BORING

Boring 3

Boring Date: 8/25/14
Surface Elevation:
Drilling Method: Flight-auger

Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	USCS	Description
0						GWT not encountered	ML	Gray to gray-brown clayey silt with sand, moist and stiff (FILL)
		50/11"	105	13.8	88			
		41	107	11.2	90		ML	Gray-brown to dark gray-brown sandy silt, dry and stiff (ORIGINAL GROUND)
5		50/10"	112	15.1	94		ML	Dark gray-brown sandy silt with clay, moist and very stiff
		50/7 1/2"	115	9.0	90		SM	Brown silty fine to medium sand, moist and dense
10		50/5"	130	5.4	94		SM	Brown silty fine to coarse sand with gravel and cobbles, damp and dense
15		50/1"	135	4.1	98			- becoming very dense at 15.0 feet
							END	Refusal on Cobbles at 16.0 Feet No Free Groundwater Encountered
20								
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT From Myra\S-13573 (Rpt. No. 2643-a).log Date: 8/27/2015

LOG OF BORING

John R. Byerly, Inc.	Patrick Henry High School San Diego, California	Enclosure 2, Page 3 Report No.: 2643-b File No.: S-13573

Boring 4

Boring Date: 8/25/14
Surface Elevation:
Drilling Method: Flight-auger

Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	Soil	Description
0						GWT not encountered	SM	Gray-brown silty fine to medium sand with some gravel, damp and dense (FILL)
		50/8"	117	6.5	92			
2							SM	Light gray-brown to gray-brown silty fine to coarse sand with gravel and cobbles, moist and dense (ORIGINAL GROUND)
		50/8"	118	8.8	93		SM	Dark gray-brown silty fine to medium sand with clay, moist and dense
4								
		50/10"	127	2.9	92		SM	Brown silty fine to coarse sand with gravel and cobbles, dry and dense
6								
		50/6 1/2"	131	2.3	95			- becomming very dense at 7.0 feet
8								
10							SM	Brown silty fine to medium sand with gravel and cobbles, damp and very dense
		50/2"	135	3.3	98			
12							END	Refusal on Cobbles at 12.0 Feet No Free Groundwater Encountered
14								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT From Myra\S-13573 (Rpt. No. 2643-a).log Date: 8/27/2015

LOG OF BORING

John R. Byerly, Inc.

Patrick Henry High School
San Diego, California

Enclosure 2, Page 4
Report No.: 2643-b
File No.: S-13573

Boring 5

Boring Date: 8/26/14
Surface Elevation:
Drilling Method: Flight-auger

Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	Soil Type	Description
0						GWT not encountered	AS	9.0 inches of asphalt concrete
		50/6 1/2"	115	12.0	89		SC	Dark gray to dark gray-brown clayey fine sand, moist and medium dense (ORIGINAL GROUND)
2								
		50/10"	108	11.9	90		ML	Dark gray to dark gray-brown sandy silt with some clay, dry and stiff
4								
		50/10"	105	15.9	88		ML	Olive-gray sandy silt with clay, moist and stiff
6							SM	Brown silty fine to coarse sand with gravel and cobbles, damp and very dense
		50/6"	132	3.9	96			
8								
10		50/2"	137	3.1	99			
12								
							END	Refusal on Cobbles at 13.0 Feet No Free Groundwater Encountered
14								

LOG OF BORING

Boring 6

Boring Date: 8/26/14
Surface Elevation:
Drilling Method: Flight-auger

Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	Soil	Description
0						GWT not encountered	ML	Dark yellow-brown sandy silt, moist and medium stiff (FILL)
		---	104	13.4	87			
2							SM	Light gray-brown to gray-brown silty fine to coarse sand with gravel and cobbles, damp and dense (ORIGINAL GROUND)
							end	Refusal on Cobbles at 3.5 Feet No Free Groundwater Encountered
4								
6								
8								
10								
12								
14								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT From Myra\S-13573 (Rpt. No. 2643-a).log Date: 8/27/2015

LOG OF BORING

Boring 7

Boring Date: 8/25/14
Surface Elevation:
Drilling Method: Flight-auger

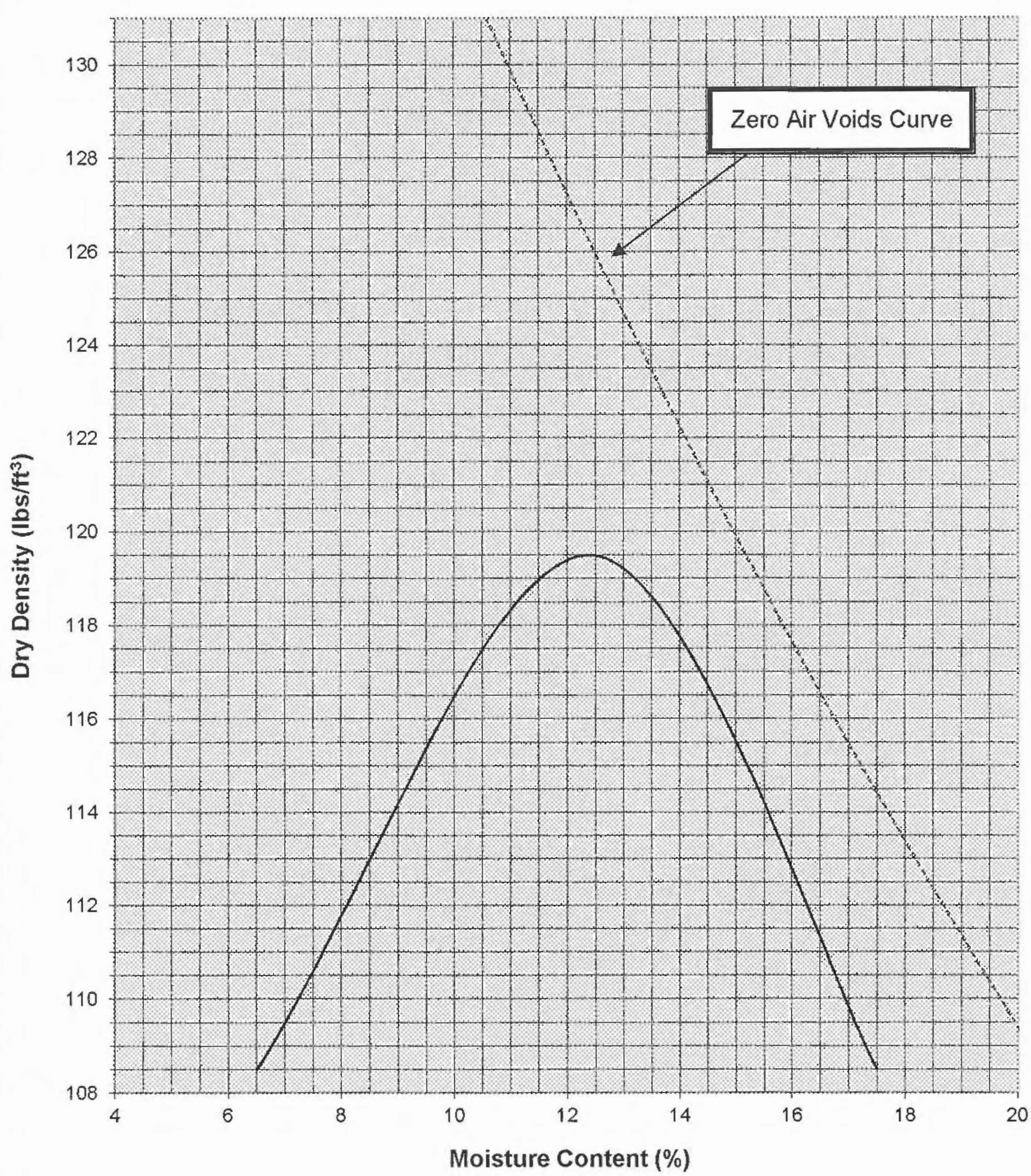
Depth	Std. Pen. "N" Value	CA Sampler Blows/Ft	Dry Density (PCF)	Moisture Content (%)	Rel. Compaction (%)	Water Table	USCS	Description
0						GWT not encountered	AS	4.5 inches of asphalt concrete
							ML	Dark gray-brown sandy silt with clay, moist and stiff (FILL)
		16	101	14.5	85			
2								
		35	118	12.6	91		SC	Brown clayey fine sand, very moist and dense (FILL)
4							SM	Brown silty fine to coarse sand with a trace of clay, moist and very dense (ORIGINAL GROUND)
		50/7 1/2"	126	8.1	95			
6								
		50/8 1/2"	130	5.0	94		SM	Light gray silty fine to coarse sand with gravel and cobbles, damp and dense
8								
10								- becoming very dense at 10.0 feet
		50/1/2"	137	4.4	99			
							END	Refusal on Cobbles at 11.0 Feet No Free Groundwater Encountered
12								
14								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\PROJECT From Myra\S-13573 (Rpt. No. 2643-a).log Date: 8/27/2015

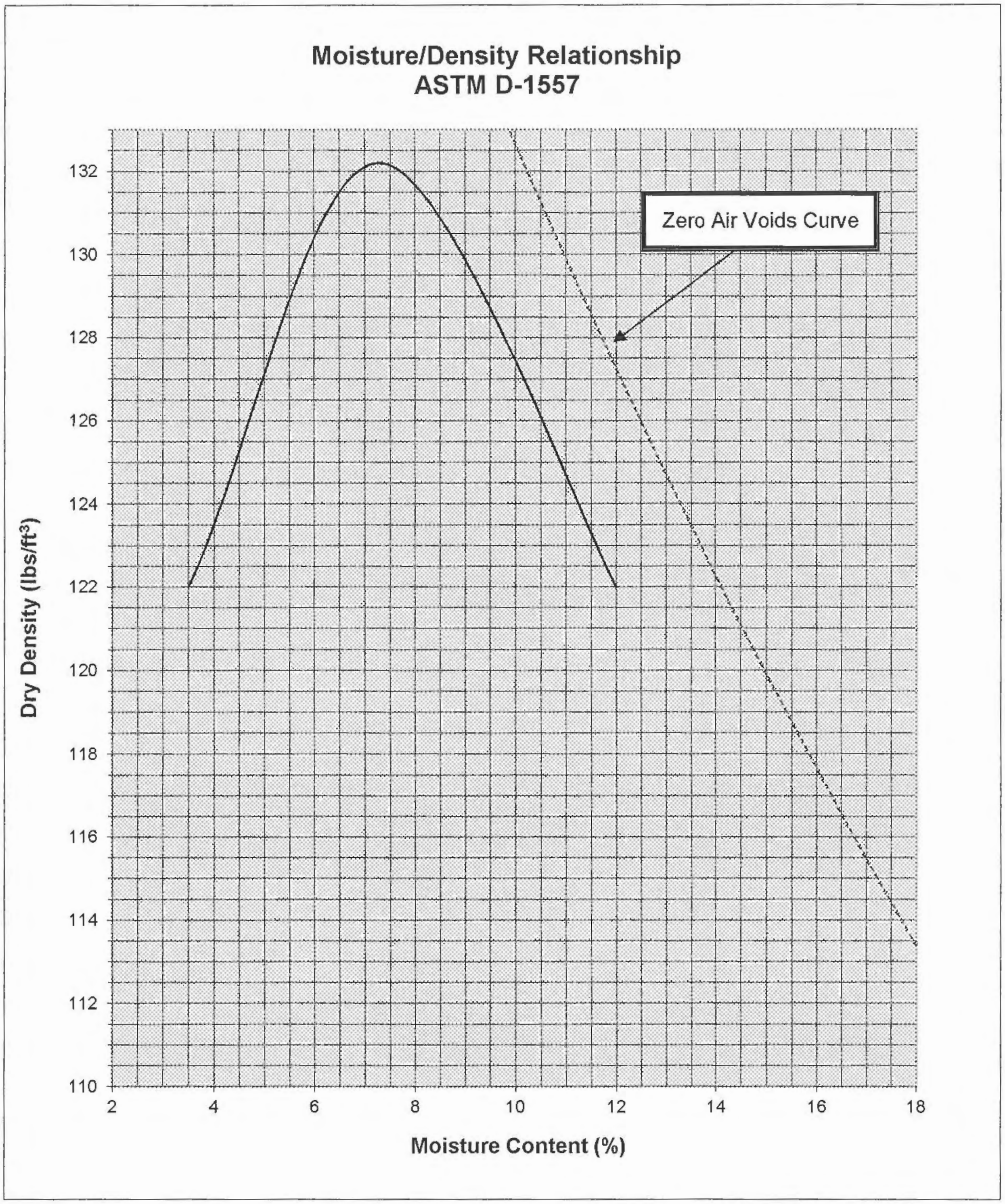
LOG OF BORING

John R. Byerly, Inc.	Patrick Henry High School San Diego, California	Enclosure 2, Page 7 Report No.: 2643-b File No.: S-13573

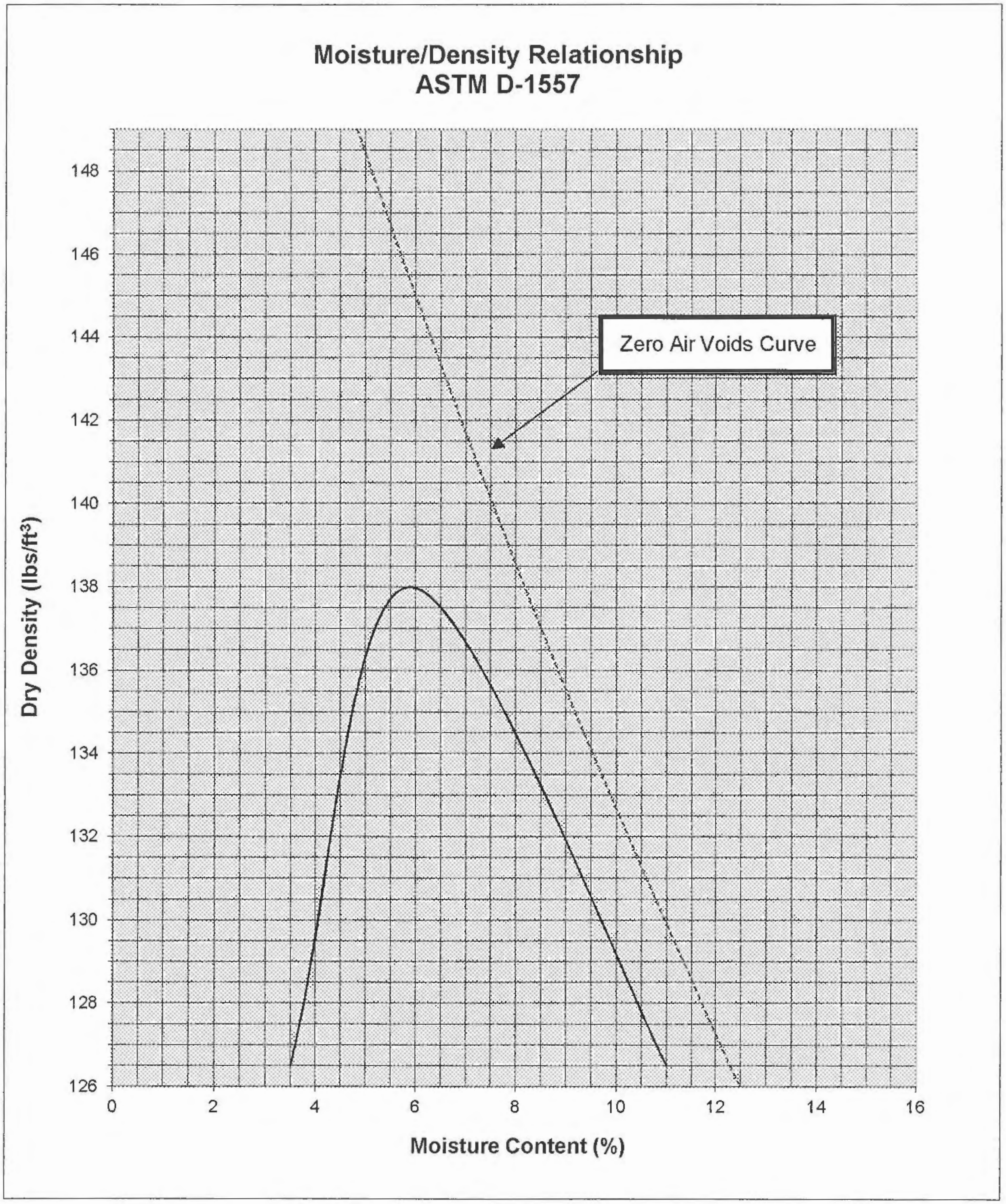
Moisture/Density Relationship
ASTM D-1557



Boring No.	B-1 (Sample ID No. 1)
Depth (ft.)	2.0
Optimum Moisture (%)	12.4
Maximum Dry Density (pcf)	119.5
Soil Classification	Light brown to gray-brown sandy silt (ML)

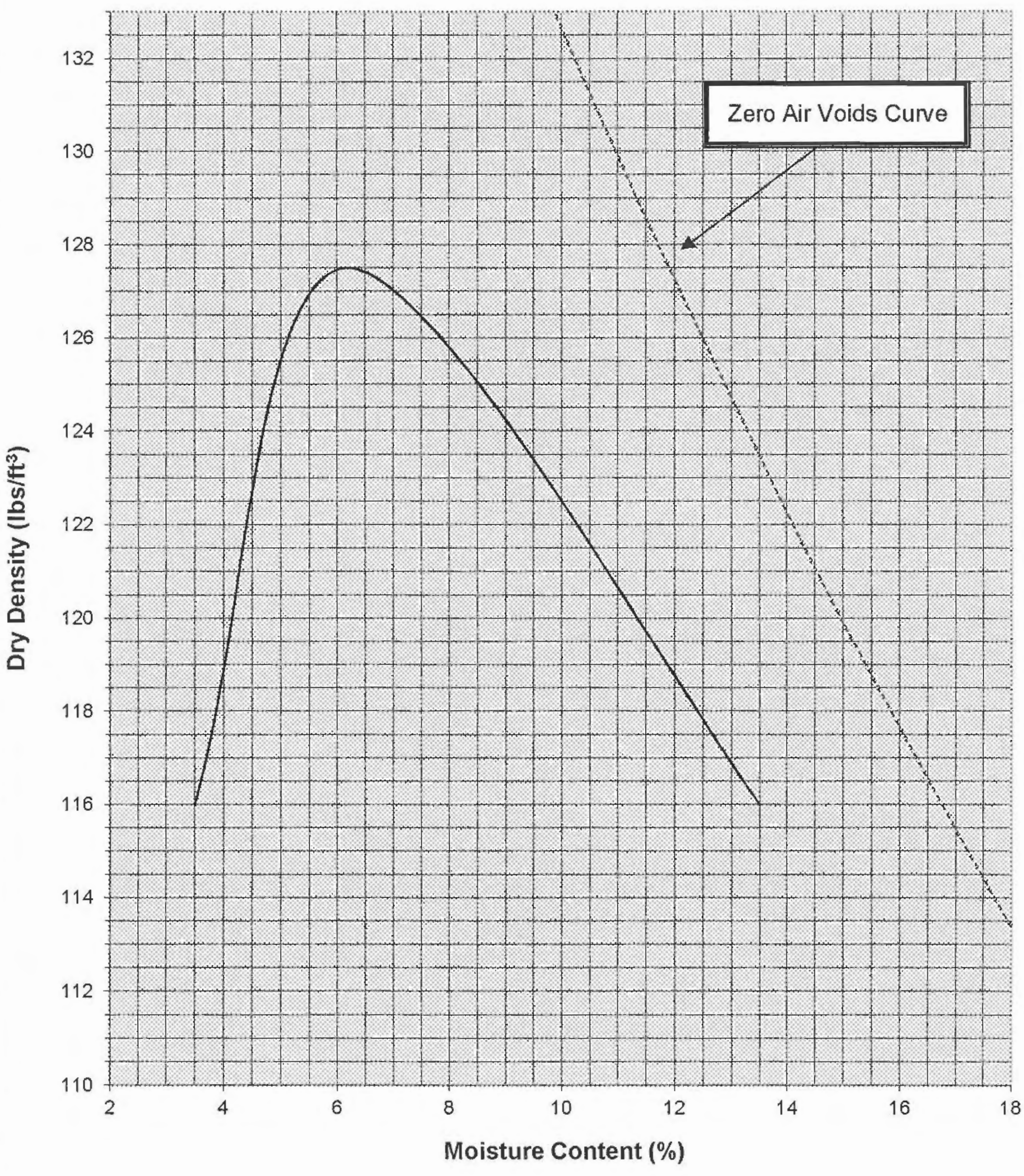


Boring No.	B-2 (Sample ID No. 2)
Depth (ft.)	4.0
Optimum Moisture (%)	Gray-brn to dark brown silty fine
Maximum Dry Density (pcf)	to coarse sand with gravel (SM)
Soil Classification	

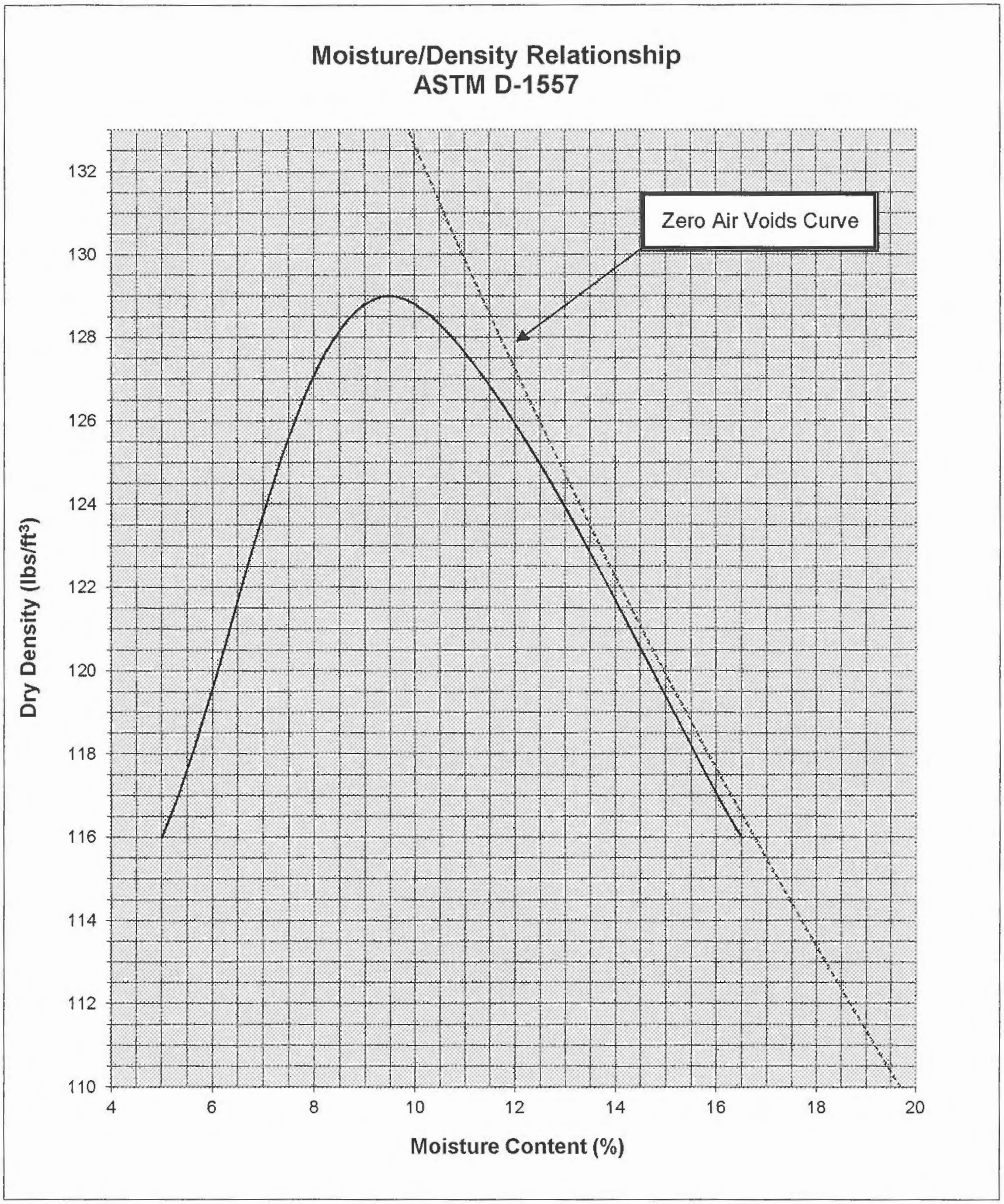


Boring No.	B-2 (Sample ID No. 3)
Depth (ft.)	6.0
Optimum Moisture (%)	5.9
Maximum Dry Density (pcf)	138.0
Soil Classification	Brown silty fine to coarse sand w/gravel and cobbles (SM)

Moisture/Density Relationship
ASTM D-1557



Boring No.	B-3 (Sample ID No. 4)
Depth (ft.)	7.0
Optimum Moisture (%)	6.2
Maximum Dry Density (pcf)	127.5
Soil Classification	Brown silty fine to medium sand (SM)



Boring No.	B-5 (Sample ID No. 5)
Depth (ft.)	2.0
Optimum Moisture (%)	9.5
Maximum Dry Density (pcf)	129.0
Soil Classification	Dark gray to dark gray-brown clayey fine sand (SC)

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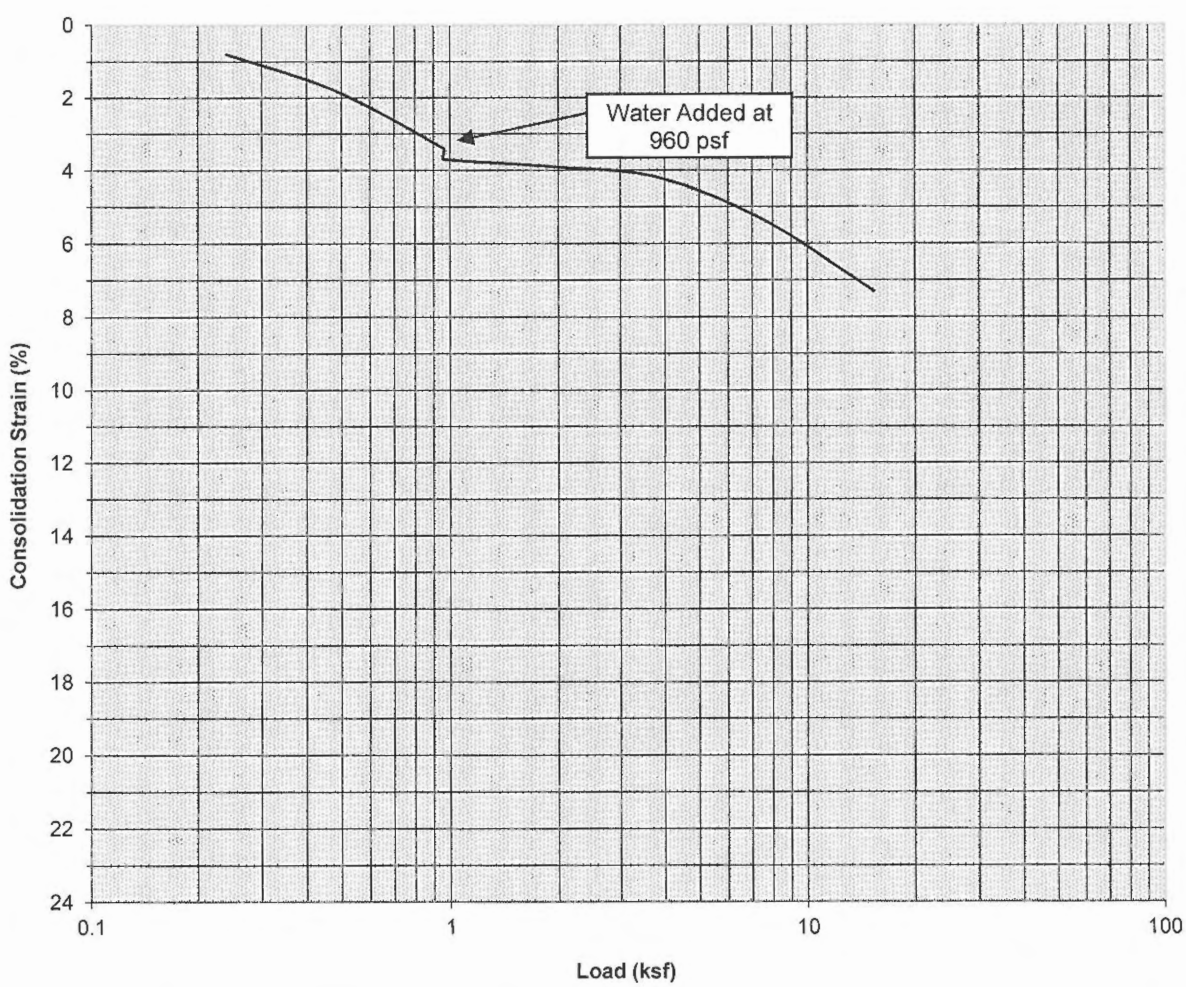
Consolidation Test Results



Classification: SM

Boring Number:	B-1	Initial Moisture Content (%)	12.2
Depth (ft)	3.0	Final Moisture Content (%)	18.1
Specimen Diameter (in)	2.4	Initial Dry Density (pcf)	125
Specimen Thickness (in)	1.0		

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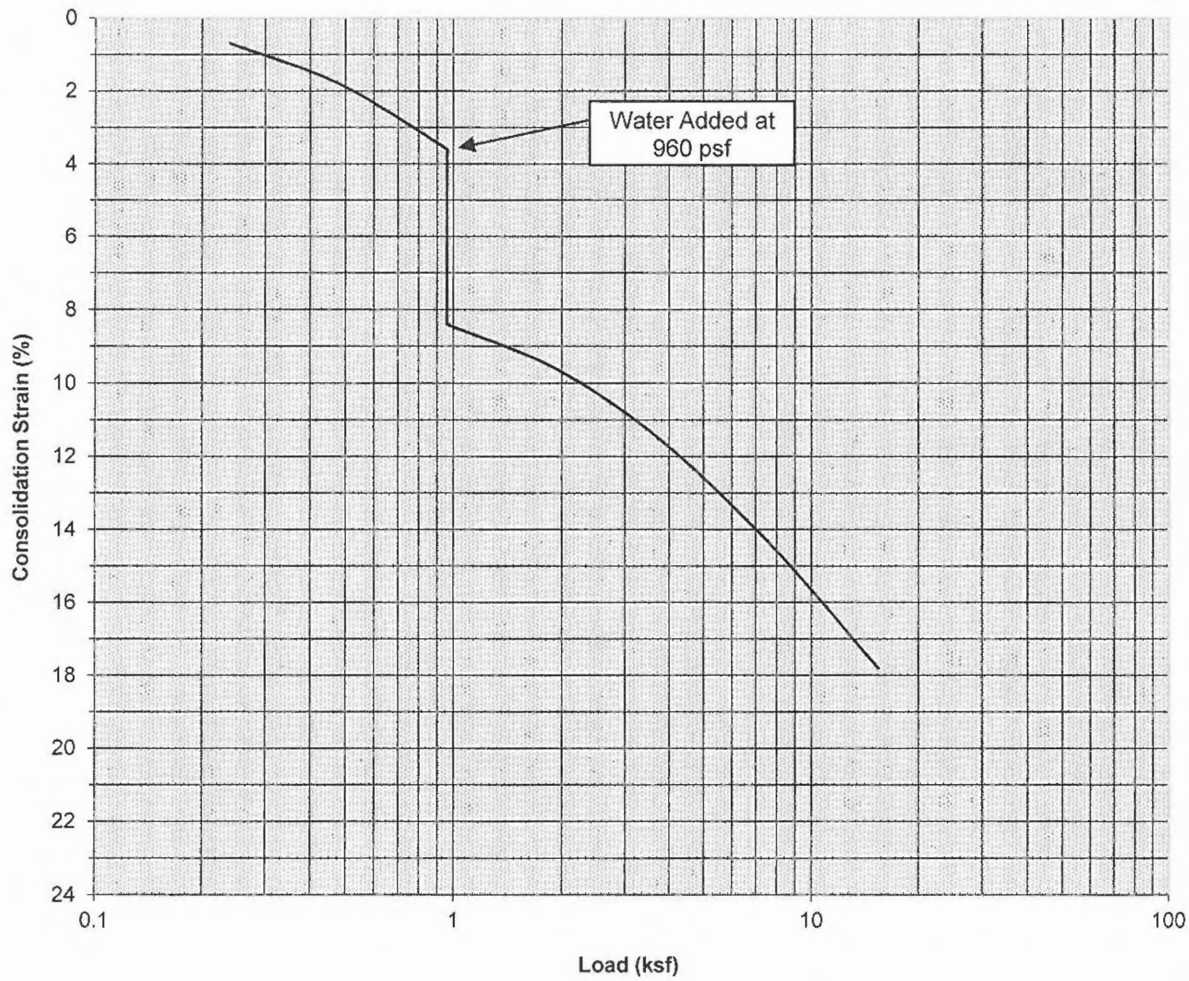
Consolidation Test Results



Classification: SM

Boring Number:	B-2	Initial Moisture Content (%)	8.7
Depth (ft)	3.0	Final Moisture Content (%)	18.6
Specimen Diameter (in)	2.4	Initial Dry Density (pcf)	120
Specimen Thickness (in)	1.0		

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EXPANSION INDEX TEST DATA

Test Description

The sample was moistened so that the as-compacted moisture content was between 49 and 51 percent saturation. The moistened sample was compacted into a 4-inch-diameter mold in two layers; each layer was compacted by 15 blows of a 5.5-pound hammer falling 12 inches. The sample was trimmed to a thickness of 1 inch and placed in a consolidometer loaded with 12.63 pounds. The sample was then submerged in distilled water, and the expansion to constant volume was noted.

Test Boring No.	Depth (ft.)	Compaction Moisture Content (%)	Dry Density (pcf)	Expansion Index	Expansion Potential
B-3	1.0-3.0	11.7	105	81	Medium
B-6	0.0-2.5	9.1	112	1	Very low

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DIRECT SHEAR TESTS

Test Boring No.	Depth of Sample (Ft.)	Angle of Internal Friction (°)	Cohesion (PSF)
B-2	3.0	35	120

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RESULTS OF SUBGRADE SOIL TESTS

California Department of Transportation Test Methods 202, 217, & 301
ASTM Designations C136 and D2419

PROJECT: Patrick Henry High School

										Percent Passing Sieve Size:								
Sample No.	**Location**	**3"**	**2½"**	**2"**	**1½"**	**1"**	**3/4"**	**1/2"**	**3/8"**	**No. 4**	**No. 8**	**No. 16**	**No. 30**	**No. 50**	**No. 100**	**No. 200**	**Sand Equiv.**
1	B-1 at 1.5'-5.0'									100	97	93	89	78	55	42	16
2	B-5 at 1.5'-9.5'									100	98	94	89	78	50	29	14

STABILOMETER "R" VALUE

Sample No.		1	
Moisture Content (%)	17.7	18.6	19.5
Dry Density (lbs./cu. ft.)	110.4	109.1	107.8
Exudation Pressure (psi)	773	499	248
Expansion Pressure (psf)	207.840	173.200	125.570
"R" Value	20	17	14
"R" Value at 300 PSI Exudation		15	

Enclosure 7
Rpt. No.: 2643-b
File No.: S-13573

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SUGGESTED SPECIFICATIONS FOR CLASS II BASE

Sieve Size	Percent Finer Than
1 Inch	100
3/4 Inch	90 - 100
No. 4	35 - 60
No. 30	10 - 30
No. 200	2 - 9
Sand Equivalent (Minimum)	25
"R" Value (minimum) at 300 psi Exudation	78

Enclosure 8
Rpt. No.: 2643-b
File No.: S-13573

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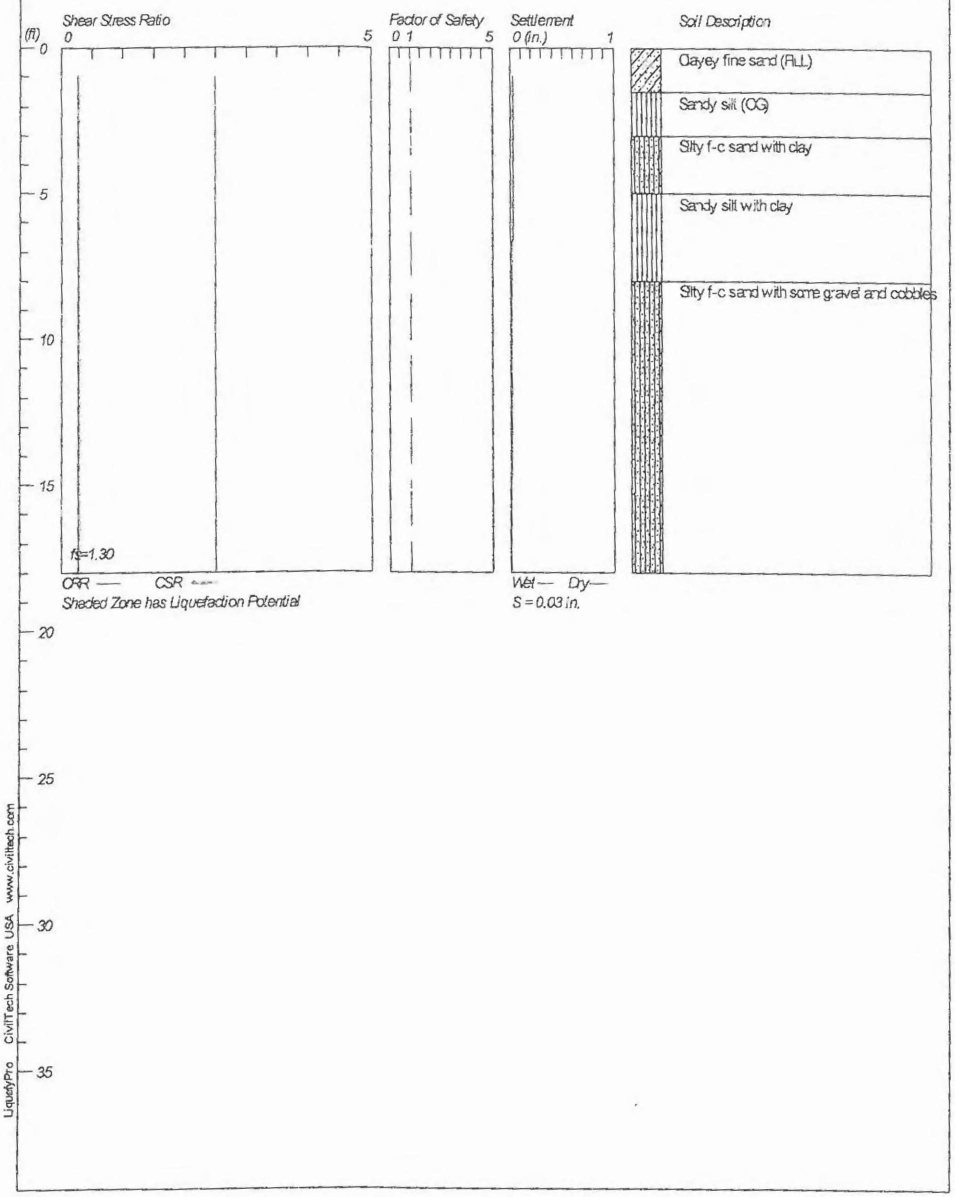
LIQUEFACTION ANALYSIS

SAN DIEGO UNIFIED SCHOOL DISTRICT - PATRICK HENRY HIGH SCHOOL

Hole No.=B-1 Water Depth=60.0 ft Surface Elev.=604 feet above MSL Magnitude=6.9

Acceleration=0.364g



John R. Byerly, Inc. S-13573

****

LIQUEFACTION ANALYSIS CALCULATION SHEET
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Input File Name: C:\Liquefy4\S-13573.1.liq
Title: SAN DIEGO UNIFIED SCHOOL DISTRICT - PATRICK HENRY HIGH SCHOOL
Subtitle: S-13573

Surface Elev.=604 feet above MSL
Hole No.=B-1
Depth of Hole= 18.0 ft
Water Table during Earthquake= 60.0 ft
Water Table during In-Situ Testing= 60.0 ft
Max. Acceleration= 0.36 g
Earthquake Magnitude= 6.9
User defined factor of safty (applied to CSR) User fs=1.3
fs=user, Plot one CSR (fs=user)

Hammer Energy Ratio, Ce=1
Borehole Diameter, Cb=1
Sampeling Method, Cs=1
SPT Fines Correction Method: Stark/Olson et al.*
Settlement Analysis Method: Ishihara / Yoshimine*
Fines Correction for Liquefaction: Stark/Olson et al.*
Fine Correction for Settlement: Post-Liq. Correction *
Average Input Data: Smooth*
* Recommended Options

Input Data:

Depth ft	SPT	Gamma pcf	Fines %
1.0	30.0	130.0	70.0
3.0	30.0	130.0	30.0
4.0	30.0	130.0	30.0
6.0	51.0	126.4	70.0
11.0	300.0	140.2	25.0
16.0	300.0	144.4	25.0

Output Results:
Settlement of saturated sands=0.00 in.
Settlement of dry sands=0.03 in.
Total settlement of saturated and dry sands=0.03 in.
Differential Settlement=0.013 to 0.017 in.

Depth ft	CRRm	CSRfs w/fs	F.S.	S_sat. in.	S_dry in.	S_all in.
1.00	2.48	0.30	5.00	0.00	0.03	0.03
2.00	2.48	0.30	5.00	0.00	0.02	0.02
3.00	2.48	0.30	5.00	0.00	0.02	0.02
4.00	2.48	0.30	5.00	0.00	0.02	0.02
5.00	2.48	0.30	5.00	0.00	0.02	0.02
6.00	2.48	0.30	5.00	0.00	0.02	0.02
7.00	2.48	0.30	5.00	0.00	0.02	0.02
8.00	2.48	0.30	5.00	0.00	0.02	0.02
9.00	2.48	0.30	5.00	0.00	0.02	0.02

10.00	2.48	0.30	5.00	0.00	0.01	0.01
11.00	2.48	0.30	5.00	0.00	0.01	0.01
12.00	2.48	0.30	5.00	0.00	0.01	0.01
13.00	2.48	0.29	5.00	0.00	0.01	0.01
14.00	2.48	0.29	5.00	0.00	0.01	0.01
15.00	2.48	0.29	5.00	0.00	0.01	0.01
16.00	2.48	0.29	5.00	0.00	0.00	0.00
17.00	2.48	0.29	5.00	0.00	0.00	0.00
18.00	2.48	0.29	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units Depth = ft, Stress or Pressure = tsf (atm), Unit Weight = pcf, Settlement = in.

CRRm	Cyclic resistance ratio from soils
CSRfs	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRfs
S_sat	Settlement from saturated sands
S_dry	Settlement from dry sands
S_all	Total settlement from saturated and dry sands
NoLiq	No-Liquefy Soils

****

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Input File Name: C:\Liquefy4\S-13573.1.liq
Title: SAN DIEGO UNIFIED SCHOOL DISTRICT - PATRICK HENRY HIGH SCHOOL
Subtitle: S-13573

Input Data:

Surface Elev.=604 feet above MSL
Hole No.=B-1
Depth of Hole=18.0 ft
Water Table during Earthquake= 60.0 ft
Water Table during In-Situ Testing= 60.0 ft
Max. Acceleration=0.36 g
Earthquake Magnitude=6.9
User defined factor of safty (applied to CSR) User fs=1.3
fs=user, Plot one CSR (fs=user)

Hammer Energy Ratio, Ce=1
Borehole Diameter, Cb=1
Sampeling Method, Cs=1
SPT Fines Correction Method: Stark/Olson et al.*
Settlement Analysis Method: Ishihara / Yoshimine*
Fines Correction for Liquefaction: Stark/Olson et al.*
Fine Correction for Settlement: Post-Liq. Correction *
Average Input Data: Smooth*
* Recommended Options

Depth ft	SPT	Gamma pcf	Fines %
1.0	30.0	130.0	70.0
3.0	30.0	130.0	30.0
4.0	30.0	130.0	30.0
6.0	51.0	126.4	70.0
11.0	300.0	140.2	25.0
16.0	300.0	144.4	25.0

Output Results: (Interval = 1.00 ft)

CSR Calculation:

Depth ft	gamma pcf	sigma tsf	gamma' pcf	sigma' tsf	rd	CSR	fs (user)	CSRfs w/fs
1.00	130.0	0.065	130.0	0.065	1.00	0.23	1.3	0.30
2.00	130.0	0.130	130.0	0.130	1.00	0.23	1.3	0.30
3.00	130.0	0.195	130.0	0.195	0.99	0.23	1.3	0.30
4.00	130.0	0.260	130.0	0.260	0.99	0.23	1.3	0.30
5.00	128.2	0.325	128.2	0.325	0.99	0.23	1.3	0.30
6.00	126.4	0.388	126.4	0.388	0.99	0.23	1.3	0.30
7.00	129.2	0.452	129.2	0.452	0.98	0.23	1.3	0.30
8.00	131.9	0.517	131.9	0.517	0.98	0.23	1.3	0.30
9.00	134.7	0.584	134.7	0.584	0.98	0.23	1.3	0.30
10.00	137.4	0.652	137.4	0.652	0.98	0.23	1.3	0.30

11.00	140.2	0.721	140.2	0.721	0.97	0.23	1.3	0.30
12.00	141.0	0.792	141.0	0.792	0.97	0.23	1.3	0.30
13.00	141.9	0.862	141.9	0.862	0.97	0.23	1.3	0.29
14.00	142.7	0.933	142.7	0.933	0.97	0.23	1.3	0.29
15.00	143.6	1.005	143.6	1.005	0.97	0.23	1.3	0.29
16.00	144.4	1.077	144.4	1.077	0.96	0.23	1.3	0.29
17.00	144.4	1.149	144.4	1.149	0.96	0.22	1.3	0.29
18.00	144.4	1.221	144.4	1.221	0.96	0.22	1.3	0.29

CSR is based on water table at 60.0 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma'	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
1.00	30.00	1.00	0.75	0.065	1.70	38.25	70.0	7.20	45.45	2.00
2.00	30.00	1.00	0.75	0.130	1.70	38.25	50.0	7.20	45.45	2.00
3.00	30.00	1.00	0.75	0.195	1.70	38.25	30.0	6.00	44.25	2.00
4.00	30.00	1.00	0.75	0.260	1.70	38.25	30.0	6.00	44.25	2.00
5.00	40.50	1.00	0.75	0.325	1.70	51.64	50.0	7.20	58.84	2.00
6.00	51.00	1.00	0.75	0.388	1.60	61.39	70.0	7.20	68.59	2.00
7.00	100.80	1.00	0.75	0.452	1.49	112.44	61.0	7.20	119.64	2.00
8.00	150.60	1.00	0.75	0.517	1.39	157.04	52.0	7.20	164.24	2.00
9.00	200.40	1.00	0.85	0.584	1.31	222.91	43.0	7.20	230.11	2.00
10.00	250.20	1.00	0.85	0.652	1.24	263.39	34.0	6.96	270.35	2.00
11.00	300.00	1.00	0.85	0.721	1.18	300.24	25.0	4.80	305.04	2.00
12.00	300.00	1.00	0.85	0.792	1.12	286.60	25.0	4.80	291.40	2.00
13.00	300.00	1.00	0.85	0.862	1.08	274.60	25.0	4.80	279.40	2.00
14.00	300.00	1.00	0.85	0.933	1.04	263.93	25.0	4.80	268.73	2.00
15.00	300.00	1.00	0.95	1.005	1.00	284.28	25.0	4.80	289.08	2.00
16.00	300.00	1.00	0.95	1.077	0.96	274.62	25.0	4.80	279.42	2.00
17.00	300.00	1.00	0.95	1.149	0.93	265.85	25.0	4.80	270.65	2.00
18.00	300.00	1.00	0.95	1.221	0.90	257.88	25.0	4.80	262.68	2.00

CRR is based on water table at 60.0 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.9:

Depth ft	sigC' tsf	CRR7.5 tsf	Ksigma	CRRv	MSF	CRRm	CSRfs w/fs	F.S. CRRm/CSRfs
1.00	0.04	2.00	1.00	2.00	1.24	2.48	0.30	5.00
2.00	0.08	2.00	1.00	2.00	1.24	2.48	0.30	5.00
3.00	0.13	2.00	1.00	2.00	1.24	2.48	0.30	5.00
4.00	0.17	2.00	1.00	2.00	1.24	2.48	0.30	5.00
5.00	0.21	2.00	1.00	2.00	1.24	2.48	0.30	5.00
6.00	0.25	2.00	1.00	2.00	1.24	2.48	0.30	5.00
7.00	0.29	2.00	1.00	2.00	1.24	2.48	0.30	5.00
8.00	0.34	2.00	1.00	2.00	1.24	2.48	0.30	5.00
9.00	0.38	2.00	1.00	2.00	1.24	2.48	0.30	5.00
10.00	0.42	2.00	1.00	2.00	1.24	2.48	0.30	5.00
11.00	0.47	2.00	1.00	2.00	1.24	2.48	0.30	5.00
12.00	0.51	2.00	1.00	2.00	1.24	2.48	0.30	5.00
13.00	0.56	2.00	1.00	2.00	1.24	2.48	0.29	5.00
14.00	0.61	2.00	1.00	2.00	1.24	2.48	0.29	5.00
15.00	0.65	2.00	1.00	2.00	1.24	2.48	0.29	5.00
16.00	0.70	2.00	1.00	2.00	1.24	2.48	0.29	5.00
17.00	0.75	2.00	1.00	2.00	1.24	2.48	0.29	5.00
18.00	0.79	2.00	1.00	2.00	1.24	2.48	0.29	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 tsf	(N1)60	Fines %	d(N1)60	(N1)60s
1.00	-	-	-	38.25	70.0	4.86	43.11
2.00	-	-	-	38.25	50.0	3.87	42.12
3.00	-	-	-	38.25	30.0	2.56	40.81
4.00	-	-	-	38.25	30.0	2.56	40.81
5.00	-	-	-	51.64	50.0	3.87	55.51
6.00	-	-	-	61.39	70.0	4.86	66.24
7.00	-	-	-	100.00	61.0	4.45	104.45
8.00	-	-	-	100.00	52.0	3.98	103.98
9.00	-	-	-	100.00	43.0	3.45	103.45
10.00	-	-	-	100.00	34.0	2.85	102.85
11.00	-	-	-	100.00	25.0	2.19	102.19
12.00	-	-	-	100.00	25.0	2.19	102.19
13.00	-	-	-	100.00	25.0	2.19	102.19
14.00	-	-	-	100.00	25.0	2.19	102.19
15.00	-	-	-	100.00	25.0	2.19	102.19
16.00	-	-	-	100.00	25.0	2.19	102.19
17.00	-	-	-	100.00	25.0	2.19	102.19
18.00	-	-	-	100.00	25.0	2.19	102.19

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine*

Depth ft	CSRfs w/fs	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsv in.	S in.

Settlement of Saturated Sands=0.000 in.
dsz is per each segment: dz=0.05 ft
dsv is per each print interval: dv=1 ft
S is cumulated settlement at this depth

Settlement of Dry Sands:

Depth ft	sigma' tsf	sigC' tsf	(N1)60s	CSRfs w/fs	Gmax tsf	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsv in.	S in.
17.95	1.22	0.79	102.19	0.29	1857.3	1.9E-4	0.0273	0.0086	0.90	0.0078	9.3E-5	0.000	0.000
17.00	1.15	0.75	102.19	0.29	1804.3	1.9E-4	0.0294	0.0093	0.90	0.0084	1.0E-4	0.002	0.002
16.00	1.08	0.70	102.19	0.29	1746.7	1.8E-4	0.0281	0.0089	0.90	0.0080	9.6E-5	0.002	0.004
15.00	1.01	0.65	102.19	0.29	1687.3	1.7E-4	0.0268	0.0085	0.90	0.0076	9.2E-5	0.002	0.006
14.00	0.93	0.61	102.19	0.29	1626.1	1.7E-4	0.0255	0.0081	0.90	0.0073	8.7E-5	0.002	0.007
13.00	0.86	0.56	102.19	0.29	1562.9	1.6E-4	0.0243	0.0077	0.90	0.0069	8.3E-5	0.002	0.009
12.00	0.79	0.51	102.19	0.30	1497.5	1.6E-4	0.0231	0.0073	0.90	0.0066	7.9E-5	0.002	0.011
11.00	0.72	0.47	102.19	0.30	1429.4	1.5E-4	0.0218	0.0069	0.90	0.0062	7.5E-5	0.002	0.012
10.00	0.65	0.42	102.85	0.30	1361.9	1.4E-4	0.0206	0.0065	0.90	0.0059	7.0E-5	0.001	0.014

9.00	0.58	0.38	103.45	0.30	1291.4	1.3E-4	0.0194	0.0061	0.90	0.0055	6.6E-5	0.001	0.015
8.00	0.52	0.34	103.98	0.30	1217.6	1.3E-4	0.0214	0.0068	0.90	0.0061	7.3E-5	0.001	0.016
7.00	0.45	0.29	104.45	0.30	1140.0	1.2E-4	0.0197	0.0062	0.90	0.0056	6.7E-5	0.001	0.018
6.00	0.39	0.25	66.24	0.30	907.7	1.3E-4	0.0217	0.0069	0.90	0.0062	7.4E-5	0.001	0.019
5.00	0.32	0.21	55.51	0.30	782.5	1.2E-4	0.0210	0.0066	0.90	0.0060	7.2E-5	0.001	0.021
4.00	0.26	0.17	40.81	0.30	632.2	1.2E-4	0.0208	0.0066	0.90	0.0059	7.1E-5	0.001	0.022
3.00	0.20	0.13	40.81	0.30	547.5	1.1E-4	0.0223	0.0071	0.90	0.0063	7.6E-5	0.002	0.024
2.00	0.13	0.08	42.12	0.30	451.7	8.7E-5	0.0160	0.0051	0.90	0.0045	5.5E-5	0.001	0.025
1.00	0.07	0.04	43.11	0.30	321.9	6.1E-5	0.0096	0.0030	0.90	0.0027	3.3E-5	0.001	0.026

Settlement of Dry Sands=0.026 in.
dsz is per each segment: dz=0.05 ft
dsv is per each print interval: dv=1 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Dry Sands=0.026 in.
Differential Settlement=0.013 to 0.017 in.

Units	Depth = ft, Stress or Pressure = tsf (atm), Unit Weight = pcf, Settlement = in.

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT)
fc	Friction from CPT testing
Gamma	Total unit weight of soil
Gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [tsf]
sigma'	Effective vertical stress [tsf]
sigC'	Effective confining pressure [tsf]
rd	Stress reduction coefficient
CSR	Cyclic stress ratio induced by earthquake
fs	User request factor of safety, apply to CSR
w/fs	With user request factor of safety inside
CSRfs	CSR with User request factor of safety
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksigma	Overburden stress correction factor for CRR7.5
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksigma
MSF	Magnitude scaling factor for CRR (M=7.5)
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
F.S.	Factor of Safety against liquefaction F.S.=CRRm/CSRfs
Cebs	Energy Ratio, Borehole Dia., and Sample Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction

dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after seattlement fines corrections
ec	Volumetric strain for saturated sands
ds	Settlement in each Segment dz
dz	Segment for calculation, dz=0.050 ft
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for dry sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

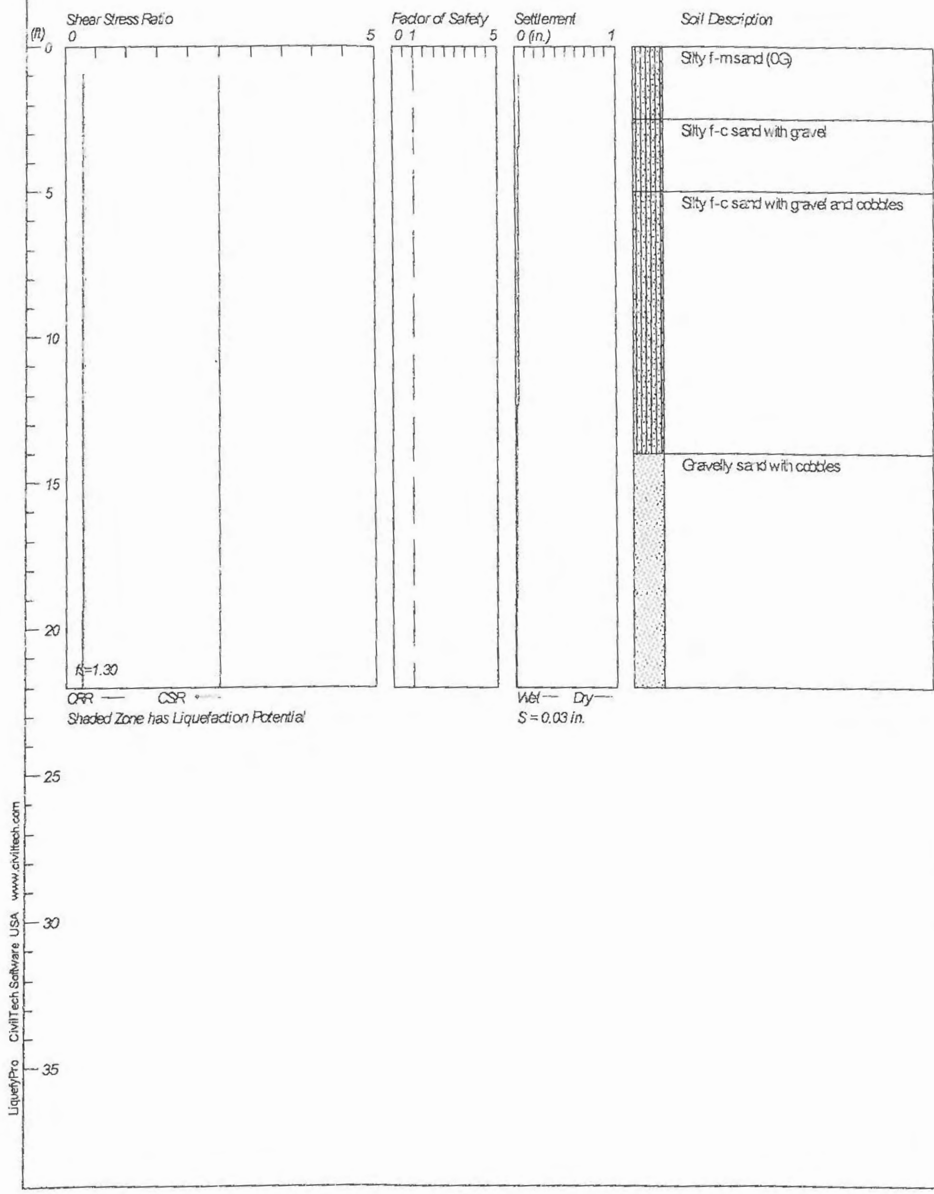
NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

LIQUEFACTION ANALYSIS

SAN DIEGO UNIFIED SCHOOL DISTRICT - PATRICK HENRY HIGH SCHOOL

Hole No.=B-2 Water Depth=60.0 ft Surface Elev.=602 feet above MSL Magnitude=6.9
Acceleration=0.364g



John R. Byerly, Inc. S-13573

Enclosure 9, Page 9
Rpt. No.: 2643-b
File No.: S-13573

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Input File Name: C:\Liquefy4\S-13573.2.liq
Title: SAN DIEGO UNIFIED SCHOOL DISTRICT - PATRICK HENRY HIGH SCHOOL
Subtitle: S-13573

Surface Elev.=602 feet above MSL
Hole No.=B-2
Depth of Hole= 22.0 ft
Water Table during Earthquake= 60.0 ft
Water Table during In-Situ Testing= 60.0 ft
Max. Acceleration= 0.36 g
Earthquake Magnitude= 6.9
User defined factor of safty (applied to CSR) User fs=1.3
fs=user, Plot one CSR (fs=user)

Hammer Energy Ratio, Ce=1
Borehole Diameter, Cb=1
Sampeling Method, Cs=1
SPT Fines Correction Method: Stark/Olson et al.*
Settlement Analysis Method: Ishihara / Yoshimine*
Fines Correction for Liquefaction: Stark/Olson et al.*
Fine Correction for Settlement: Post-Liq. Correction *
Average Input Data: Smooth*
* Recommended Options

Input Data:

Depth ft	SPT	Gamma pcf	Fines %
1.0	30.0	130.0	30.0
3.0	30.0	130.0	25.0
4.0	30.0	130.0	25.0
6.0	120.0	133.6	25.0
11.0	120.0	138.7	25.0
16.0	300.0	138.2	1.0
21.0	300.0	140.0	1.0

Output Results:
Settlement of saturated sands=0.00 in.
Settlement of dry sands=0.03 in.
Total settlement of saturated and dry sands=0.03 in.
Differential Settlement=0.017 to 0.022 in.

Depth ft	CRRm	CSRfs w/fs	F.S.	S_sat. in.	S_dry in.	S_all in.
1.00	2.48	0.30	5.00	0.00	0.03	0.03
2.00	2.48	0.30	5.00	0.00	0.03	0.03
3.00	2.48	0.30	5.00	0.00	0.03	0.03
4.00	2.48	0.30	5.00	0.00	0.03	0.03
5.00	2.48	0.30	5.00	0.00	0.03	0.03
6.00	2.48	0.30	5.00	0.00	0.03	0.03
7.00	2.48	0.30	5.00	0.00	0.03	0.03
8.00	2.48	0.30	5.00	0.00	0.02	0.02

9.00	2.48	0.30	5.00	0.00	0.02	0.02
10.00	2.48	0.30	5.00	0.00	0.02	0.02
11.00	2.48	0.30	5.00	0.00	0.02	0.02
12.00	2.48	0.30	5.00	0.00	0.02	0.02
13.00	2.48	0.29	5.00	0.00	0.02	0.02
14.00	2.48	0.29	5.00	0.00	0.02	0.02
15.00	2.48	0.29	5.00	0.00	0.01	0.01
16.00	2.48	0.29	5.00	0.00	0.01	0.01
17.00	2.48	0.29	5.00	0.00	0.01	0.01
18.00	2.48	0.29	5.00	0.00	0.01	0.01
19.00	2.48	0.29	5.00	0.00	0.01	0.01
20.00	2.48	0.29	5.00	0.00	0.00	0.00
21.00	2.48	0.29	5.00	0.00	0.00	0.00
22.00	2.48	0.29	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units Depth = ft, Stress or Pressure = tsf (atm), Unit Weight = pcf, Settlement = in.

CRRm	Cyclic resistance ratio from soils
CSRfs	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRfs
S_sat	Settlement from saturated sands
S_dry	Settlement from dry sands
S_all	Total settlement from saturated and dry sands
NoLiq	No-Liquefy Soils

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Input File Name: C:\Liquefy4\S-13573.2.liq
Title: SAN DIEGO UNIFIED SCHOOL DISTRICT - PATRICK HENRY HIGH SCHOOL
Subtitle: S-13573

Input Data:

Surface Elev.=602 feet above MSL
Hole No.=B-2
Depth of Hole=22.0 ft
Water Table during Earthquake= 60.0 ft
Water Table during In-Situ Testing= 60.0 ft
Max. Acceleration=0.36 g
Earthquake Magnitude=6.9
User defined factor of safty (applied to CSR) User fs=1.3
fs=user, Plot one CSR (fs=user)

Hammer Energy Ratio, Ce=1
Borehole Diameter, Cb=1
Sampeling Method, Cs=1
SPT Fines Correction Method: Stark/Olson et al.*
Settlement Analysis Method: Ishihara / Yoshimine*
Fines Correction for Liquefaction: Stark/Olson et al.*
Fine Correction for Settlement: Post-Liq. Correction *
Average Input Data: Smooth*
* Recommended Options

Depth ft	SPT	Gamma pcf	Fines %
1.0	30.0	130.0	30.0
3.0	30.0	130.0	25.0
4.0	30.0	130.0	25.0
6.0	120.0	133.6	25.0
11.0	120.0	138.7	25.0
16.0	300.0	138.2	1.0
21.0	300.0	140.0	1.0

Output Results: (Interval = 1.00 ft)

CSR Calculation:

Depth ft	gamma pcf	sigma tsf	gamma' pcf	sigma' tsf	rd	CSR	fs (user)	CSRfs w/fs
1.00	130.0	0.065	130.0	0.065	1.00	0.23	1.3	0.30
2.00	130.0	0.130	130.0	0.130	1.00	0.23	1.3	0.30
3.00	130.0	0.195	130.0	0.195	0.99	0.23	1.3	0.30
4.00	130.0	0.260	130.0	0.260	0.99	0.23	1.3	0.30
5.00	131.8	0.325	131.8	0.325	0.99	0.23	1.3	0.30
6.00	133.6	0.392	133.6	0.392	0.99	0.23	1.3	0.30
7.00	134.6	0.459	134.6	0.459	0.98	0.23	1.3	0.30
8.00	135.6	0.526	135.6	0.526	0.98	0.23	1.3	0.30
9.00	136.7	0.594	136.7	0.594	0.98	0.23	1.3	0.30

10.00	137.7	0.663	137.7	0.663	0.98	0.23	1.3	0.30
11.00	138.7	0.732	138.7	0.732	0.97	0.23	1.3	0.30
12.00	138.6	0.801	138.6	0.801	0.97	0.23	1.3	0.30
13.00	138.5	0.871	138.5	0.871	0.97	0.23	1.3	0.29
14.00	138.4	0.940	138.4	0.940	0.97	0.23	1.3	0.29
15.00	138.3	1.009	138.3	1.009	0.97	0.23	1.3	0.29
16.00	138.2	1.078	138.2	1.078	0.96	0.23	1.3	0.29
17.00	138.6	1.147	138.6	1.147	0.96	0.22	1.3	0.29
18.00	138.9	1.217	138.9	1.217	0.96	0.22	1.3	0.29
19.00	139.3	1.286	139.3	1.286	0.96	0.22	1.3	0.29
20.00	139.6	1.356	139.6	1.356	0.95	0.22	1.3	0.29
21.00	140.0	1.426	140.0	1.426	0.95	0.22	1.3	0.29
22.00	140.0	1.496	140.0	1.496	0.95	0.22	1.3	0.29

CSR is based on water table at 60.0 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma'	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
1.00	30.00	1.00	0.75	0.065	1.70	38.25	30.0	6.00	44.25	2.00
2.00	30.00	1.00	0.75	0.130	1.70	38.25	27.5	5.40	43.65	2.00
3.00	30.00	1.00	0.75	0.195	1.70	38.25	25.0	4.80	43.05	2.00
4.00	30.00	1.00	0.75	0.260	1.70	38.25	25.0	4.80	43.05	2.00
5.00	75.00	1.00	0.75	0.325	1.70	95.63	25.0	4.80	100.43	2.00
6.00	120.00	1.00	0.75	0.392	1.60	143.79	25.0	4.80	148.59	2.00
7.00	120.00	1.00	0.75	0.459	1.48	132.87	25.0	4.80	137.67	2.00
8.00	120.00	1.00	0.75	0.526	1.38	124.05	25.0	4.80	128.85	2.00
9.00	120.00	1.00	0.85	0.594	1.30	132.30	25.0	4.80	137.10	2.00
10.00	120.00	1.00	0.85	0.663	1.23	125.27	25.0	4.80	130.07	2.00
11.00	120.00	1.00	0.85	0.732	1.17	119.21	25.0	4.80	124.01	2.00
12.00	156.00	1.00	0.85	0.801	1.12	148.12	20.2	3.65	151.77	2.00
13.00	192.00	1.00	0.85	0.871	1.07	174.90	15.4	2.50	177.40	2.00
14.00	228.00	1.00	0.85	0.940	1.03	199.90	10.6	1.34	201.25	2.00
15.00	264.00	1.00	0.95	1.009	1.00	249.67	5.8	0.19	249.86	2.00
16.00	300.00	1.00	0.95	1.078	0.96	274.47	1.0	0.00	274.47	2.00
17.00	300.00	1.00	0.95	1.147	0.93	266.07	1.0	0.00	266.07	2.00
18.00	300.00	1.00	0.95	1.217	0.91	258.37	1.0	0.00	258.37	2.00
19.00	300.00	1.00	0.95	1.286	0.88	251.29	1.0	0.00	251.29	2.00
20.00	300.00	1.00	0.95	1.356	0.86	244.74	1.0	0.00	244.74	2.00
21.00	300.00	1.00	0.95	1.426	0.84	238.67	1.0	0.00	238.67	2.00
22.00	300.00	1.00	0.95	1.496	0.82	233.02	1.0	0.00	233.02	2.00

CRR is based on water table at 60.0 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.9:

Depth ft	sigC' tsf	CRR7.5 tsf	Ksigma	CRRv	MSF	CRRm	CSRfs w/fs	F.S. CRRm/CSRfs
1.00	0.04	2.00	1.00	2.00	1.24	2.48	0.30	5.00
2.00	0.08	2.00	1.00	2.00	1.24	2.48	0.30	5.00
3.00	0.13	2.00	1.00	2.00	1.24	2.48	0.30	5.00
4.00	0.17	2.00	1.00	2.00	1.24	2.48	0.30	5.00
5.00	0.21	2.00	1.00	2.00	1.24	2.48	0.30	5.00
6.00	0.25	2.00	1.00	2.00	1.24	2.48	0.30	5.00
7.00	0.30	2.00	1.00	2.00	1.24	2.48	0.30	5.00
8.00	0.34	2.00	1.00	2.00	1.24	2.48	0.30	5.00
9.00	0.39	2.00	1.00	2.00	1.24	2.48	0.30	5.00
10.00	0.43	2.00	1.00	2.00	1.24	2.48	0.30	5.00
11.00	0.48	2.00	1.00	2.00	1.24	2.48	0.30	5.00
12.00	0.52	2.00	1.00	2.00	1.24	2.48	0.30	5.00
13.00	0.57	2.00	1.00	2.00	1.24	2.48	0.29	5.00
14.00	0.61	2.00	1.00	2.00	1.24	2.48	0.29	5.00

15.00	0.66	2.00	1.00	2.00	1.24	2.48	0.29	5.00
16.00	0.70	2.00	1.00	2.00	1.24	2.48	0.29	5.00
17.00	0.75	2.00	1.00	2.00	1.24	2.48	0.29	5.00
18.00	0.79	2.00	1.00	2.00	1.24	2.48	0.29	5.00
19.00	0.84	2.00	1.00	2.00	1.24	2.48	0.29	5.00
20.00	0.88	2.00	1.00	2.00	1.24	2.48	0.29	5.00
21.00	0.93	2.00	1.00	2.00	1.24	2.48	0.29	5.00
22.00	0.97	2.00	1.00	2.00	1.24	2.48	0.29	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 tsf	(N1)60	Fines %	d(N1)60	(N1)60s
1.00	-	-	-	38.25	30.0	2.56	40.81
2.00	-	-	-	38.25	27.5	2.38	40.63
3.00	-	-	-	38.25	25.0	2.19	40.44
4.00	-	-	-	38.25	25.0	2.19	40.44
5.00	-	-	-	95.63	25.0	2.19	97.81
6.00	-	-	-	100.00	25.0	2.19	102.19
7.00	-	-	-	100.00	25.0	2.19	102.19
8.00	-	-	-	100.00	25.0	2.19	102.19
9.00	-	-	-	100.00	25.0	2.19	102.19
10.00	-	-	-	100.00	25.0	2.19	102.19
11.00	-	-	-	100.00	25.0	2.19	102.19
12.00	-	-	-	100.00	20.2	1.80	101.80
13.00	-	-	-	100.00	15.4	1.41	101.41
14.00	-	-	-	100.00	10.6	0.99	100.99
15.00	-	-	-	100.00	5.8	0.55	100.55
16.00	-	-	-	100.00	1.0	0.10	100.10
17.00	-	-	-	100.00	1.0	0.10	100.10
18.00	-	-	-	100.00	1.0	0.10	100.10
19.00	-	-	-	100.00	1.0	0.10	100.10
20.00	-	-	-	100.00	1.0	0.10	100.10
21.00	-	-	-	100.00	1.0	0.10	100.10
22.00	-	-	-	100.00	1.0	0.10	100.10

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine*

Depth ft	CSRfs w/fs	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsv in.	S in.

Settlement of Saturated Sands=0.000 in.
dsz is per each segment: dz=0.05 ft
dsv is per each print interval: dv=1 ft
S is cumulated settlement at this depth

Settlement of Dry Sands:

Depth ft	sigma' tsf	sigC' tsf	(N1)60s	CSRfs w/fs	Gmax tsf	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsv in.	S in.
21.95	1.49	0.97	100.10	0.29	2042.0	2.1E-4	0.0314	0.0099	0.90	0.0089			

1.1E-4	0.000	0.000									
	21.00	1.43	0.93	100.10	0.29	1996.0	2.1E-4	0.0305	0.0096	0.90	0.0087
1.0E-4	0.002	0.002									
	20.00	1.36	0.88	100.10	0.29	1946.5	2.0E-4	0.0295	0.0093	0.90	0.0084
1.0E-4	0.002	0.004									
	19.00	1.29	0.84	100.10	0.29	1895.8	2.0E-4	0.0285	0.0090	0.90	0.0081
9.7E-5	0.002	0.006									
	18.00	1.22	0.79	100.10	0.29	1843.8	1.9E-4	0.0276	0.0087	0.90	0.0078
9.4E-5	0.002	0.008									
	17.00	1.15	0.75	100.10	0.29	1790.5	1.9E-4	0.0297	0.0094	0.90	0.0085
1.0E-4	0.002	0.010									
	16.00	1.08	0.70	100.10	0.29	1735.6	1.8E-4	0.0284	0.0090	0.90	0.0081
9.7E-5	0.002	0.012									
	15.00	1.01	0.66	100.55	0.29	1681.6	1.8E-4	0.0271	0.0086	0.90	0.0077
9.3E-5	0.002	0.014									
	14.00	0.94	0.61	100.99	0.29	1625.3	1.7E-4	0.0258	0.0082	0.90	0.0073
8.8E-5	0.002	0.016									
	13.00	0.87	0.57	101.41	0.29	1566.4	1.6E-4	0.0245	0.0078	0.90	0.0070
8.4E-5	0.002	0.017									
	12.00	0.80	0.52	101.80	0.30	1504.8	1.6E-4	0.0233	0.0074	0.90	0.0066
7.9E-5	0.002	0.019									
	11.00	0.73	0.48	102.19	0.30	1440.0	1.5E-4	0.0220	0.0070	0.90	0.0063
7.5E-5	0.002	0.020									
	10.00	0.66	0.43	102.19	0.30	1370.4	1.4E-4	0.0208	0.0066	0.90	0.0059
7.1E-5	0.001	0.022									
	9.00	0.59	0.39	102.19	0.30	1297.6	1.4E-4	0.0196	0.0062	0.90	0.0056
6.7E-5	0.001	0.023									
	8.00	0.53	0.34	102.19	0.30	1221.1	1.3E-4	0.0218	0.0069	0.90	0.0062
7.4E-5	0.001	0.025									
	7.00	0.46	0.30	102.19	0.30	1140.0	1.2E-4	0.0201	0.0064	0.90	0.0057
6.9E-5	0.001	0.026									
	6.00	0.39	0.25	102.19	0.30	1053.4	1.1E-4	0.0183	0.0058	0.90	0.0052
6.2E-5	0.001	0.027									
	5.00	0.33	0.21	97.81	0.30	946.2	1.0E-4	0.0166	0.0053	0.90	0.0047
5.7E-5	0.001	0.029									
	4.00	0.26	0.17	40.44	0.30	630.2	1.2E-4	0.0209	0.0066	0.90	0.0059
7.1E-5	0.001	0.030									
	3.00	0.20	0.13	40.44	0.30	545.8	1.1E-4	0.0224	0.0071	0.90	0.0064
7.7E-5	0.002	0.031									
	2.00	0.13	0.08	40.63	0.30	446.4	8.8E-5	0.0163	0.0052	0.90	0.0046
5.6E-5	0.001	0.033									
	1.00	0.07	0.04	40.81	0.30	316.1	6.2E-5	0.0098	0.0031	0.90	0.0028
3.4E-5	0.001	0.034									

Settlement of Dry Sands=0.034 in.
dsz is per each segment: dz=0.05 ft
dsv is per each print interval: dv=1 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Dry Sands=0.034 in.
Differential Settlement=0.017 to 0.022 in.

Units	Depth = ft, Stress or Pressure = tsf (atm), Unit Weight = pcf, Settlement = in.

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT)
fc	Friction from CPT testing
Gamma	Total unit weight of soil
Gamma'	Effective unit weight of soil

Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [tsf]
sigma'	Effective vertical stress [tsf]
sigC'	Effective confining pressure [tsf]
rd	Stress reduction coefficient
CSR	Cyclic stress ratio induced by earthquake
fs	User request factor of safety, apply to CSR
w/fs	With user request factor of safety inside
CSRfs	CSR with User request factor of safety
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksigma	Overburden stress correction factor for CRR7.5
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksigma
MSF	Magnitude scaling factor for CRR (M=7.5)
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
F.S.	Factor of Safety against liquefaction F.S.=CRRm/CSRfs
Cebs	Energy Ratio, Borehole Dia., and Sample Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after seattlement fines corrections
ec	Volumetric strain for saturated sands
ds	Settlement in each Segment dz
dz	Segment for calculation, dz=0.050 ft
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for dry sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

AKW GEOTECHNICAL

GEOTECHNICAL CONSULTANTS

Project No. M1065-01
December 9, 2014

Mr. John Byerly
John R. Byerly, Inc.
Bloomington, California 92316

Subject: ENGINEERING GEOLOGY INVESTIGATION
TWO STORY CLASSROOM BUILDING
AND ADMINISTRATION BUILDING ADDITION
PATRICK HENRY HIGH SCHOOL
6702 WANDERMERE DRIVE
SAN DIEGO, CALIFORNIA

Dear Mr. Byerly:

In accordance with your authorization, we have performed an engineering geology investigation for the proposed two-story classroom and administration building addition at Patrick Henry High School in San Diego, California. The accompanying report presents results of our investigation and includes conclusions and recommendations pertaining to the geologic aspects of the improvements and renovations on the site as presently proposed. John R. Byerly, Inc. will be presenting the primary geotechnical report for the subject site. It is our understanding that this engineering geology report will be included as an appendix in your report.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,
AKW GEOTECHNICAL

Ernest W. Roumelis
CEG 2385

PROFESSIONAL GEOLOGIST ★ STATE OF CALIFORNIA ★ ERNEST W. ROUMELIS No. 2385 CERTIFIED ENGINEERING GEOLOGIST

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ENGINEERING GEOLOGY INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the findings of our engineering geology investigation for the proposed improvements at Patrick Henry High School. The purpose of the study was to address potential site geologic hazard conditions for the proposed structures in general accordance with California Geological Survey (CGS) Note 48 (CGS, 2013) and the 2013 California Building Code.

To prepare this report we conducted a review of readily available published and unpublished reports, maps and documents pertinent to the proposed site additions. We performed a geologic field reconnaissance of the site and the surrounding area concurrently with the site investigation by John R. Byerly, Inc.

2. SITE AND PROJECT CONDITIONS

2.1. EXISTING SITE CONDITIONS

The proposed two-story classroom and administration building extension will be located in the south eastern portion of the existing Patrick Henry High School campus. The school campus is located at 6702 Wandermere Drive, in the city of San Diego, San Diego County, California (see *Site Location Map*, Enclosure 1). The existing school was founded in 1968. The coordinates of the site are latitude 32.7969° N and longitude 117.05043° W, utilizing the North American Datum (NAD) from 1983. The site is located in the La Mesa 7.5 Minute USGS Quadrangle. The current topography of the site varies between 600 and 595 feet above Mean Sea Level (MSL) and slopes gently towards the northwest. The center of the site lies approximately 599 feet above Mean Sea Level (MSL). Patrick Henry High School is administered by the San Diego Unified School District.

2.2. PROPOSED DEVELOPMENT

We understand that the proposed development will consist of a two-story classroom building to be constructed in the southeast portion of the campus and that the west side of the current administration building is to receive an addition. The classroom will have a building footprint of approximately 20,000 square feet and the administration building addition will be on the order of 1,000 square feet. We anticipate the new buildings will consist of steel-frame and concrete block masonry construction, will exert light to moderate foundation loads on the underlying soils, and will each incorporate a concrete slab-on-grade floor. As part of the development, a storm water retention basin is planned for an area west of the proposed two-story classroom building. New parking areas paved with asphalt concrete will be developed. Although no specific grading plans were available for our review, we are assuming that significant cuts, cut slopes, fills, fill slopes, and/or retaining walls are not proposed with the development of the site based on the existing site topography. A schematic site plan prepared by PJHM Architects, Inc. (2014) modified by John R. Byerly, Inc. (2014) was used for this report.

2.3. AERIAL PHOTOGRAPH AND TOPOGRAPHIC MAP REVIEW

Vintage stereoscopic aerial photographs of the site and vicinity from the years between 1953 and 2005 were reviewed for our investigation. Vintage USGS topographic maps from the years between 1942 and 2012 as well as three-dimensional computer-aided photography flown between the years of 1995 and 2013 and presented by Google Earth (Google, 2013) were reviewed for this report. No lineaments were observed in the aerial photos in the vicinity of the school site.

3. GEOLOGY

3.1. GEOLOGIC SETTING

The site is located in the southern portion of the Peninsular Ranges Geomorphic province, within the San Diego Embayment. The Peninsular Ranges province extends southeastward from the area east of Los Angeles Basin to beyond the Mexican border and is subdivided into several structural units, such as the Santa Ana Mountains, the San Diego Embayment, the Perris, and San Jacinto Mountain blocks. The Peninsular Ranges province is generally characterized by northwest oriented valleys and mountain ranges bounded by major right lateral strike-slip fault zones. The San Andreas Fault zone constitutes the eastern provincial boundary, the Newport-Inglewood fault constitutes the western provincial boundary, while the San Jacinto and Elsinore Fault zones are located within the center of the province. Rocks of the Peninsular Ranges are typically Cretaceous igneous and marine sedimentary and Paleozoic to Mesozoic metasedimentary rocks. Tertiary marine and non-marine sedimentary and volcanic rock along with Quaternary sediment lies unconformably on either the Cretaceous sedimentary or the older basement rock.

The San Diego Embayment consists of marine and non-marine sedimentary rocks deposited in several pulses of marine inundation and regression. The proposed project site is located on Eocene-aged conglomerate and siltstone deposits. The earth materials encountered on the subject site are described in more detail in a subsequent section of this report. The geologic units onsite are possible Artificial Fill, the Stadium Conglomerate and the Mission Valley Formation and (see Regional Geologic Map, Enclosure 2). Metamorphic bedrock underlies these units at depth. These units are discussed below.

3.2. GEOLOGIC UNITS

Artificial Fill

Artificial fill was encountered in all of the borings during the site investigation by Byerly (2014). The site lies within the existing portion of the Patrick Henry High School campus. The fill varies in thickness between 1.5 feet and 4 feet, and was likely placed during school site grading in the late 1960's (see Site Geologic Map, Enclosure 3). Any existing fill encountered should be evaluated by the project Geotechnical Engineer for competency with respect to any proposed settlement sensitive structures.

Stadium Conglomerate

The Eocene-age Stadium Conglomerate consists of a massive cobble rich formational sandstone unit. Where encountered, the unit consists of dense to very dense, brown silty fine to coarse sand with cobbles. The Stadium Conglomerate constitutes bedrock and underlies the site to the maximum depth of exploratory borings B-2 and B-6 placed on the site for the current investigation [approximately 22 feet (6.7 meters) bgs] (see Site Geologic Map, Enclosure 3). High blow counts from the borings and nearby surface exposures indicate the Stadium Conglomerate corresponds to Site Class C based on the NEHRP Site Class descriptions presented in Section 11.4.2 of the 2013 CBC.

Mission Valley Formation

The Eocene-age Mission Valley Formation consists of a near shore marine deposit, light olive to gray brown, fine to medium grained sandstone with cobble conglomerate interbeds (up to 35% of total mass) and interbreeds of dark gray claystone and is in conformable contact with the Stadium Conglomerate. Where encountered, the unit consists of dense to very dense, brown to dark grayish brown sandy silt to silty fine sand with occasional lenses of gravel and cobble. The Mission Valley Formation constitutes bedrock and underlies the site to the maximum depth of exploratory borings B-1, B-3, B-4, B-5 and B-7 placed on the site for the current investigation [approximately 18 feet (5.5 meters) bgs] (see Site Geologic Map, Enclosure 3). High blow counts from the borings and nearby surface exposures indicate the Mission Valley Formation corresponds to Site Class C based on the NEHRP Site Class descriptions presented in Section 11.4.2 of the 2013 CBC.

4. SEISMIC HAZARDS

4.1. FAULTING

The site is not located within or adjacent to an Alquist-Priolo Earthquake Fault Zone (EFZ) (Hart and Bryant, 1997, 1999, 2003; Bryant and Hart, 2007). The boundary of the closest Alquist-Priolo EFZ is located approximately 8.3 miles (13.4 kilometers) west of the site associated with the Rose Canyon fault (Treiman, 2002). The City of San Diego also considers the Rose Canyon fault as active. Enclosure 5 shows the locations of these nearby faults with respect to the site.

A fault table of the active or potentially active faults within 62 miles (100 kilometers) of the site was generated by EQFAULT (Blake, 2000a) and was reviewed for this investigation. However, due to the limitations of the data base utilized by Blake, all of the fault distances were determined by individual measurements from more precise geologic maps, including the State's Alquist-Priolo EFZ maps, California Geological Survey (2013b, 2010), Morton and Miller (2006), Jennings et al. (2010), and USGS (2013). The faults discussed below are considered to represent the closest and most significant faults to the site with the potential to induce ground surface rupture and/or generate strong ground motion in the event of a moderately sized or larger earthquake. The faults or fault zones located closest to the site are discussed briefly below.

<u>Mission Gorge Fault</u>

The Mission Gorge Fault is a potentially active (USGS, 2013) east west trending fault along the alignment and slightly north of Mission Gorge Road. The Mission Gorge Fault is located approximately 1.5 miles (2.4 km) north of the site. No additional information for the Mission Gorge Fault was available for our review in the USGS Database (USGS, 2008). The fault is not listed as active by the City of San Diego General Plan (2008) and not designated as an Alquist-Priolo EFZs. If any future publications document earthquake activity or potential ground shaking hazard for the Mission Gorge fault, please contact our office for an updated hazard assessment with respect to the proposed development.

<u>La Nacion Fault Zone</u>

The La Nacion Fault Zone is a north-trending fault east west trending fault west of the site and is listed as active by the City of San Diego. The La Nacion Fault Zone is located approximately 2.7 miles (4.3 km) west-southwest of the site. The La Nacion Fault Zone is considered to be capable of producing an earthquake of at least M_W 6.7 (City of San Diego, 2008). The fault is not designated as an Alquist-Priolo EFZs.

The Rose Canyon Fault Zone

The Rose Canyon Fault Zone consists of a series of north-trending right lateral strike-slip faults located just west of Interstate 5 in the Coronado Island area. Various discontinuous strands exhibit strike-slip, oblique slip, normal and reverse type motions as they transect thru the San Diego metropolitan area and San Diego Bay. The Rose Canyon Fault Zone is located approximately 8.3 miles (13.4 kilometers) west of the site. The Rose Canyon Fault Zone is considered to be capable of producing an earthquake of at least M_W 6.9, is documented in the 2008 Update for the National Seismic Hazard Maps (USGS, 2008) and is designated as having potential surface fault rupture hazard by an Alquist-Priolo EFZs (Treiman, 2002).

4.2. SEISMICITY

The recorded history of earthquakes prior to the seismograph is sparse and inconsistent. The oldest seismographs (or recordable earthquake devices) originated in Italy in the mid-1800s. The modern seismograph was developed in Japan in 1880 (Richter, 1958). Electromagnetic seismometers (calibrated seismographs) were developed between 1928 and 1930. Townley and Allen (1939) documented earthquakes along the Pacific Coast of the U.S. between 1769 and 1928. The systematic recording of large earthquakes in California began in 1932-1933 by the U.S. Coast and Geodetic Survey (Richter, 1958). As part of our investigation, we reviewed earthquake data recorded between A.D. 1800 and 2013 by searching the USGS database (USGS, 2014). The nearest significant earthquake epicenter to the site is approximately 2.9 miles (4.6 kilometers) west of the site. The Magnitude 5.0 earthquake occurred on May 25, 1803, in proximity to the Mission Gorge fault and reportedly generated accelerations of 0.22g.

5. GROUND MOTION

The 2013 California Building Code, Section 11.4 ground motion values were generated using the U.S Geological Survey (2013b) "US Seismic Design Maps" website and tool (Version 3.1.0). The site coordinates input to the USGS program are 32.7969 ° N and longitude 117.05043 ° W, NAD 1983. The mapped MCE ground motion parameter, **S_S, is 0.890**g from Figure 22-1 of ASCE 7-10 (American Society of Civil Engineers, 2010). The mapped MCE ground motion parameter, **S_1, is 0.344g** from Figure 22-2 of ASCE 7-10. S_1, therefore, is less than 0.75g.

The interpolated Site Coefficient, **Fa, is 1.044** from Table 11.4-1 of the ASCE 7-10, based on S_S greater than 0.75g and Site Class C. The interpolated Site Coefficient, **Fv, is 1.456** from Table 11.4-2 of the ASCE 7-10, based on S_1 greater than 0.30g and Site Class C. The Section 11.4.3 Adjusted MCER spectral response acceleration parameter, **S_{MS}, is 0.929g**. The Section 11.4.3 Adjusted MCER spectral response acceleration parameter, **S_{M1}, is 0.501g**. The Section 11.4.4 Design spectral response acceleration parameter, S_{DS}, **is 0.620g**. The Section 11.4.4 Design spectral response acceleration parameter, S_{D1}, **is 0.334g.** The Long-period Transition Period, T_L, **is 8** seconds from Figure 22-12 of ASCE 7-10.

The proposed structure on the site is expected to belong to Occupancy Category III. Based on the S_1 parameter being less than 0.75g and the Occupancy Category being III, the proposed building would be assigned to **Seismic Design Category D** per the 2013 CBC.

In lieu of a site-specific ground motion study, the Peak Ground Acceleration, PGA, for the site is 0.345g from Figure 22-7 of ASCE 7-10. From Table 11.8-1 of ASCE 7-10, the interpolated Site Coefficient, F_{PGA}, is 1.055, based on a PGA greater than 0.3g and Site Class C. The mapped MCE Geometric Mean Peak Ground Acceleration, PGA_M, is 1.055 times the Peak Ground Acceleration or **PGA_M = 0.364g** utilizing Equation 11.8-1 from ASCE 7-10. The Geotechnical Engineer of Record should determine whether or not liquefaction settlement potential could affect proposed settlement sensitive structures.

Using Method 1 of Section 21.2.1.1 of ASCE 7-10, **C_{RS} is 0.981** from Figure 22-17 of ASCE 7-10; **C_{R1} is 1.050** from Figure 22-18 of ASCE 7-10.

6. GROUNDWATER

Groundwater was not encountered in any of the exploratory borings placed on the site to the maximum depth of approximately 22 feet (6.5 meters) bgs. Shallow perched groundwater may be present in the ball field areas of the school campus located immediately northwest of the site (Leighton, 2011). The Stadium Conglomerate and Mission Valley Formation units are not considered water bearing units so reliable groundwater data is not available. Data from two nearby water district wells located approximately 3.3 mile (5.3 km) west of the site (Well No.'s 327860N1171058W001 and 327859N1171070W001 just east of Qualcomm Stadium) indicate that recent high groundwater depths of 78 feet (23.7 meters) below ground surface (bgs) in 2010 and depths of 108 feet (33.0 meters) bgs respectively in 2008 in an alluvium filled canyon. Several environmental wells were identified during a database search (GEOTRACKER) in the northeastern corner of the campus near the intersection of Navajo Road and Park Ridge Blvd. Logs of the wells indicate shallow groundwater, however then do not distinguish between perched nuisance water and the phreatic surface. Based on the elevation of well casings, the topographic relief, and the estimated depth of fill and alluvium near the site, we feel that groundwater could be located between 60 and 70 feet below the ground surface. In addition, Lake Murray reservoir is located approximately 0.67 mile (1.0 km) southeast of the site and probably controls local groundwater conditions. The surface elevation of Lake Murray is approximately 526 feet above mean sea level and so approximately 70 feet (22.5 meters) below the elevation of the site. We feel that 60 feet (18.3 meters) bgs is a reasonably conservative depth estimate to the phreatic surface. Therefore, shallow groundwater will not pose any hazard to the proposed development.

7. LIQUEFACTION AND LATERAL SPREADING

The California Geological Survey has not conducted Seismic Hazards Mapping for the La Mesa 7.5 Minute Quadrangle, therefore the site is not located in a Seismic Hazard Liquefaction Zone as defined by the State's Seismic Hazard Mapping Act. Ground water was not encountered in any of the exploratory borings placed on the site during the current geotechnical investigations (Byerly, 2014). Due to the dense nature of shallow formational material, and the lack of groundwater surface, we feel the potential for liquefaction at the site is to be considered low. No building or grading plans were available for our review. It is our understanding that no significant slopes will be constructed, therefore lateral spreading is not anticipated to be a concern.

8. LANDSLIDE AND SLOPE STABILITY

The California Geological Survey has not conducted Seismic Hazards Mapping for the La Mesa 7.5 Minute Quadrangle. No areas have been designated as "zones of required investigation for earthquake induced landslides" as defined by the State's Seismic Hazard Mapping Act. Slope stability hazards are not expected to affect the proposed site development.

9. SUBSIDENCE AND INFLATION

Subsidence is a regional lowering of the ground surface. Inflation is a regional rising of the ground surface. Subsidence and inflation can result from either tectonic or non-tectonic stress changes. Tectonically-induced subsidence or tectonically-induced inflation is the result of extension or compression (respectively) of the crust. Non-tectonic subsidence or non-tectonic inflation of the ground surface is commonly associated with the removal or addition (respectively) of fluids from either an aquifer (ground water) or a reservoir (oil, gas, steam, *et cetera*).

Based on the dense nature of Eocene aged materials beneath the site, and the lack of groundwater withdrawal, the site is not anticipated to experience any appreciable amount of regional subsidence.

10. FLOODING

The City of San Diego (2008) has not designated the vicinity of the school site as lying within a 100 year floodplain. The site is not located within a flood hazard area as defined by the CBC.

10.1. Seismically Induced Flooding

No water reservoirs were observed near the site at the time of this investigation (Google, 2014). The site is not located within or near an established Dam Inundation Hazard Zone as described in the City of San Diego General Plan (2008). Due to the local topographic expression, seismically induced flooding of the site is not considered to be a potential hazard to the proposed structure.

10.2. Tsunamis

A tsunami is a seismically generated ocean wave. Due to the location of the site with respect to the Pacific Ocean, tsunamis are not a hazard to the site.

11. VOLCANIC ACTIVITY

Volcanic activity in California has usually been associated with former subduction zone tectonism or extensional tectonism that permits mantle-derived basalt to extrude onto the surface. Volcanic activity can also be associated with hot spot plumes emanating from the mantle, like Hawaii. Jennings (1994) did not show recent volcanic eruptions in the vicinity of the site. Since a significant source of recent volcanism is not located in the vicinity of the site, volcanic activity is not anticipated on or near the site during the lifetime of the proposed structure.

12. CONCLUSIONS AND RECOMMENDATIONS

The proposed two-story classroom and administration building extension will be located in the south eastern portion of the existing Patrick Henry High School campus. The school campus is located at 6702 Wandermere Drive, in the city of San Diego, San Diego County, California (see *Site Location Map*, Enclosure 1). The existing school was founded in 1968. The coordinates of the site are latitude 32.7969° N and longitude 117.05043° W, utilizing the North American Datum (NAD) from 1983. The current topography of the site varies between 595 and 600 feet above Mean Sea Level (MSL) and slopes gently towards the northwest.

The geologic units onsite are Artificial Fill associated with school site grading and Eocene age bedrock of the Stadium Conglomerate and Mission Valley Formations. The bedrock units encountered consist of dense to very dense, silty fine to coarse sand with cobbles. The Stadium Conglomerate and Mission Valley Formations underlie the site to the maximum depth of the exploratory borings placed on the site for the current investigation [approximately 22 feet (6.7 meters) bgs]. The fill varies in thickness between 1.5 feet and 4 feet should be evaluated by the project Geotechnical Engineer for competency with respect to any proposed settlement sensitive structures. This material encountered on site suggests that the upper 100 vertical feet of earth material corresponds to Site Class C based on the Site Class descriptions presented in Section 11.4.2 of the 2013 CBC.

The site is not located within or adjacent to an Alquist-Priolo Earthquake Fault Zone (EFZ) (Hart and Bryant, 1997, 1999, 2003; Bryant and Hart, 2007). The boundary of the closest Alquist-Priolo EFZ is located approximately 8.3 miles (13.4 kilometers) west of the site associated with the Rose Canyon fault (Treiman, 2002). The City of San Diego also considers the Rose Canyon fault as active. The Rose Canyon fault is capable of generating a magnitude 6.9 earthquake (USGS, 2008). The nearest significant earthquake epicenter to the site is approximately 2.9 miles (4.6 kilometers) west of the site. The Magnitude 5.0 earthquake occurred on May 25, 1803, in proximity to the Mission Gorge fault and reportedly generated accelerations of 0.22g.

The mapped MCE ground motion parameter, S_S, is 0.890g from Figure 22-1 of ASCE 7-10 (American Society of Civil Engineers, 2010). The mapped MCE ground motion parameter, S_1, is 0.344g from Figure 22-2 of ASCE 7-10. S_1, therefore, is less than 0.75g. The interpolated Site Coefficient, Fa, is 1.044 from Table 11.4-1 of the ASCE 7-10, based on S_S greater than 0.75g and Site Class C. The interpolated Site Coefficient, Fv, is 1.456 from Table 11.4-2 of the ASCE 7-10, based on S_1 greater than 0.30g and Site Class C. The Section 11.4.3 Adjusted MCER spectral response acceleration parameter, S_{MS}, is 0.929g. The Section 11.4.3 Adjusted MCER spectral response acceleration parameter, S_{M1}, is 0.501g. The Section 11.4.4 Design spectral response acceleration parameter, S_{DS}, is 0.620g. The Section 11.4.4 Design spectral response acceleration parameter, S_{D1}, is 0.334g. The Long-period Transition Period, T_L, is 8 seconds from Figure 22-12 of ASCE 7-10. The proposed structure on the site is expected to belong to Occupancy Category III. Based on the S1 parameter being less than 0.75g and the Occupancy Category being III, the proposed building would be assigned to **Seismic Design Category D** per the 2013 CBC. In lieu of a site-specific ground motion study, the Peak Ground Acceleration, PGA, for the site is 0.345g from Figure 22-7 of ASCE 7-10. From Table 11.8-1 of ASCE 7-10, the interpolated Site Coefficient, FPGA, is 1.055, based on a PGA greater than 0.3g and Site Class C. The mapped MCE Geometric Mean Peak Ground Acceleration, PGA_M, is 1.055 times the Peak Ground Acceleration or $PGA_M = 0.364g$ utilizing Equation 11.8-1 from ASCE 7-10.

Groundwater was not encountered in any of the exploratory borings placed on the site to the maximum depth of approximately 22 feet (6.5 meters) bgs. Shallow perched groundwater may be present in the ball field areas of the school campus located immediately northwest of the site. The Stadium Conglomerate and Mission Valley Formation units are not considered water bearing units so reliable groundwater data is not available. Several environmental wells were identified during a database search (GEOTRACKER) in the northeastern corner of the campus near the intersection of Navajo Road and Park Ridge Blvd. Logs of the wells indicate shallow groundwater, however then do not distinguish between perched nuisance water and the phreatic surface. Based on the elevation of well casings, the topographic relief, and the estimated depth of fill and alluvium near the site, we feel that groundwater could be located between 60 and 70 feet below the ground surface. In addition, Lake Murray reservoir is located approximately 0.67 mile (1.0 km) southeast of the site and probably controls local groundwater conditions. The surface elevation of Lake Murray is approximately 526 feet above mean sea level and so approximately 70 feet (22.5 meters) below the elevation of the site. We feel that 60 feet (18.3 meters) bgs is a reasonably conservative depth estimate to the phreatic surface. Therefore, shallow groundwater will not pose any hazard to the proposed development.

Due to the dense nature of subsurface formational material and the lack of groundwater, the potential for liquefaction at the site is to be considered "low."

The California Geological Survey has not conducted Seismic Hazards Mapping for the La Mesa 7.5 Minute Quadrangle. No areas have been designated as "zones of required investigation for earthquake induced landslides" as defined by the State's Seismic Hazard Mapping Act and no slopes are proposed. Slope stability hazards are not expected to affect the proposed structure on the site.

Based on the dense nature of Eocene aged materials beneath the site, and the lack of groundwater withdrawal, the site is not anticipated to experience any appreciable amount of regional subsidence.

The City of San Diego (2008) has not designated the vicinity of the school site as lying within a 100 year floodplain. The site is not located within a flood hazard area as defined by the CBC.

No water reservoirs were observed near the site at the time of this investigation (Google, 2014). The site is not located within or near an established Dam Inundation Hazard Zone as described in the City of San Diego General Plan (2008). Due to the local topographic expression, seismically induced flooding of the site is not considered to be a potential hazard to the proposed structure.

Due to the location of the site with respect to the Pacific Ocean, tsunamis are not a hazard to the site.

Since a significant source of recent volcanism is not located in the vicinity of the site, volcanic activity is not anticipated on or near the site during the lifetime of the proposed structure.

The grading plan for the proposed development should be reviewed and approved by the project engineering geologist before initiating grading on the site.

APPENDIX A
TECHNICAL REFERENCES

Presented below is a list of appropriate and current geology and seismology references pertinent to the project site-specific conditions. Regional or "standard of practice" references that generally pertain to this type of report are omitted for brevity. Please contact our office for a full reference list.

1. Bryant, W.A. and Hart, E.W., (2007). "Fault-rupture hazard zones in California, Alquist-Priolo Earthquake Fault Zoning Act with index to Earthquake Fault Zones maps, interim revision." California Resources Agency, Department of Conservation, Geological Survey Special Publication, SP 42, 42 p.

2. Byerly, John R., Inc., (2014). Preliminary Geotechnical Investigation, Rancho Cucamonga High School, Proposed Two Story Classroom Building and Addition to Administration Building, 6702 Wandermere Drive, San Diego, California, (report in progress), Project Number 13573.

3. California Geological Survey (1995). "Alquist-Priolo Earthquake Fault Zones Map (formerly Special Studies Zone Map), for the Cucamonga Peak Quadrangle, Official Maps." Department of Conservation, Division of Mines and Geology, Scale: 1" = 2,000'.

4. California Geological Survey (2009). "Guidelines for Evaluating and Mitigating Seismic Hazards in California." California Resources Agency, Department of Conservation, Geological Survey Revised Special Publication SP 117A, Website: http://www.conservation.ca.gov /cgs/shzp/webdocs/ Documents/sp117.pdf

5. California Geological Survey (2010). "2010 Geologic Map and Fault Activity Map of California", California Resources Agency, Department of Conservation, Geological Survey Website: http://www.quake.ca.gov/gmaps/ FAM/faultactivitymap.html

6. California Geological Survey (2013a). "Checklist for the review of engineering geology and seismology reports for California public schools, hospitals, and essential services buildings." California Resources Agency, Department of Conservation, Geological Survey Note 48, Report Dated October 21, 2013 Webpage: http://www.conservation.ca.gov /cgs/information/publications/ cgs_notes/note_48/Documents/Note_48.pdf

7. California Geological Survey (2013b). "Search For Regulatory Maps Webpage", California Resources Agency, Department of Conservation, California Geological Survey Website: http://www.quake.ca.gov/gmaps/WH/regulatorymaps.htm

8. City of San Diego (2008) "City of San Diego General Plan", City URL: http://www.sandiego.gov/planning/genplan/index.shtml

9. Google (2013). "Google Earth" (Version 7.1.2.2041)

10. Holzer, Thomas L (1984). Man-induced Land Subsidence. Geological Society of America, Boulder, Colorado, 221p.

11. Jennings, C.W., Bryant, W. A., Patel, M, Sander, E., Thompson, J., Wanish, B., Fonseca, M., (2010). "An Explanatory Text to Accompany the Fault activity map of California" California Resources Agency, Department of Conservation website, accessed at http://www.conservation.ca.gov/cgs/cgs_history/Documents/FAM_phamplet.pdf

12. Kennedy, M, P., and Tan, S.S., (2004), "Geologic map of the San Diego 30' X 60' quadrangles, California." U.S. Department of the Interior, Geological Survey Open-File Report, OFR 2004-1361, Version 1.0, Scale: 1" ≈ 1.5 miles.

13. Leighton Consulting, Inc, (2011), Geotechnical Investigation, Proposed Patrick Henry High School Stadium and Ball Field Improvements, Project No. 603068-002, report dated June 6, 2011.

14. United States Geological Survey (1942). "1942 7.5 Minute Series Topographic Map, La Mesa Quadrangle."

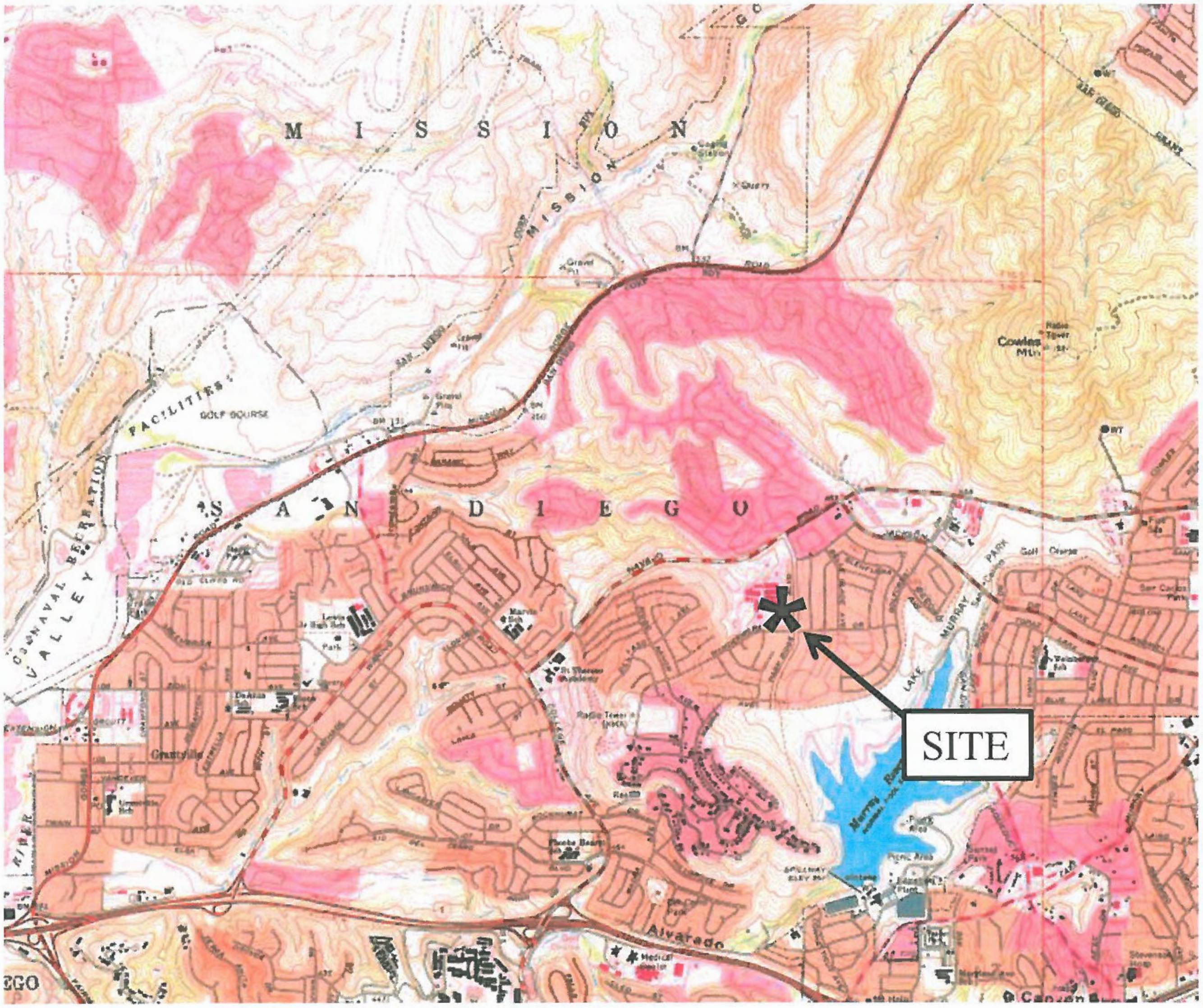
15. United States Geological Survey (1967). "1967 7.5 Minute Series Topographic Map, La Mesa Quadrangle."

16. United States Geological Survey (1967). "1975 Photo-Revised 7.5 Minute Series Topographic Map, La Mesa Quadrangle."

17. United States Geological Survey (2012). "2014 7.5 Minute Series Topographic Map, La Mesa Quadrangle."

18. United States Geological Survey (2012b). "US Seismic Design Maps Tool", Version 3.1.0, last updated 11 July 2013. U.S. Department of the Interior, Geological Survey, Webpage: http://geohazards.usgs.gov/designmaps/us/application.php. (including revisions based on pending ASCE 7-10 April 2013 Erratum).

19. United States Geological Survey (2013). Quaternary Fault and Fold Database of the Untied States., Webpage: http://earthquake.usgs.gov/hazards/qfaults/



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SOURCE: 1975 USGS 7.5 MINUTE SERIES
TOPOGRAPHIC MAP
LA MESA QUADRANGLE
SAN DIEGO COUNTY, CALIFORNIA

NO SCALE

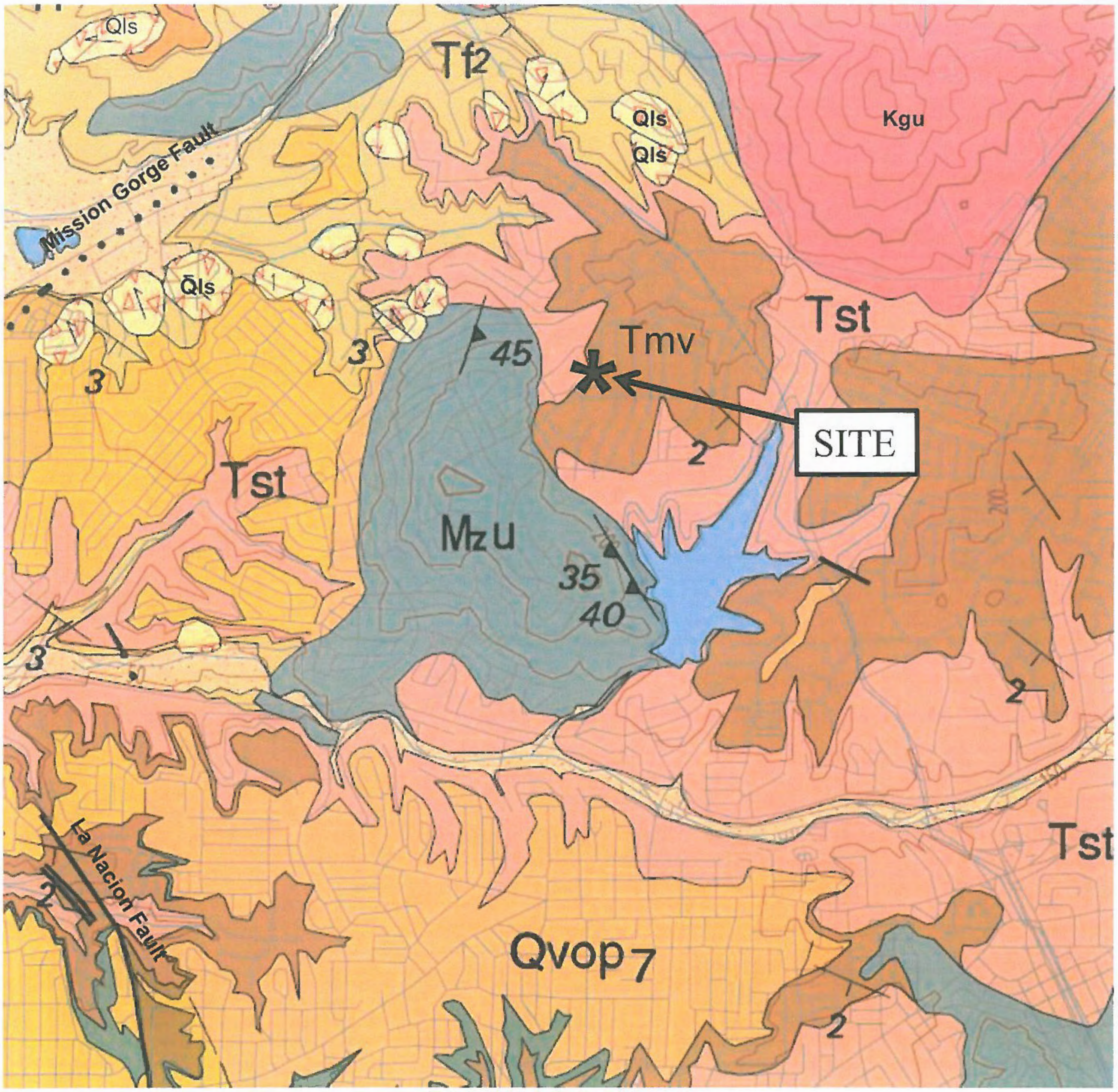
AKW GEOTECHNICAL

P.O. BOX 891173, TEMECULA, CA 92589
PHONE (951) 265-9849

SITE LOCATION MAP

ENGINEERING GEOLOGY INVESTIGATION
PATRICK HENRY HIGH ACHOOL
PROPOSED TWO-STORY CLASSROOM
AND ADMINISTRATION BUILDING
6702 WANDERMERE STREET, SAN DIEGO, CALIFORNIA

EWR	DATE 12-9-2014	PROJECT NO. M1065-01	ENCLOSURE 1



GEOLOGIC MAP LEGEND

Qls	Landslide deposits (Holocene & Late Pleistocene)	Kgu	Granodiorite and tonalite undivided (Cretaceous)
Qyc	Young colluvial deposits (Holocene & Late Pleistocene)	MzU	Metasedimentary and metavolcanic undivided (Mesozoic)
$Qvop_8$	Very old paralic deposits (mid to early Pleistocene)	3	Strike and dip of bed
Tmv	Mission Valley Formation (mid Eocene) sandstone	45	Strike and dip of metamorphic foliation
Tst	Stadium Conglomerate (mid Eocene)		Fault (dotted where buried)

SOURCE: Kennedy & Tan, 2005,
Geologic Map of the San Diego 30' x 60'
Quadrangle, USGS Open File Report 2004-1361

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SCALE FEET

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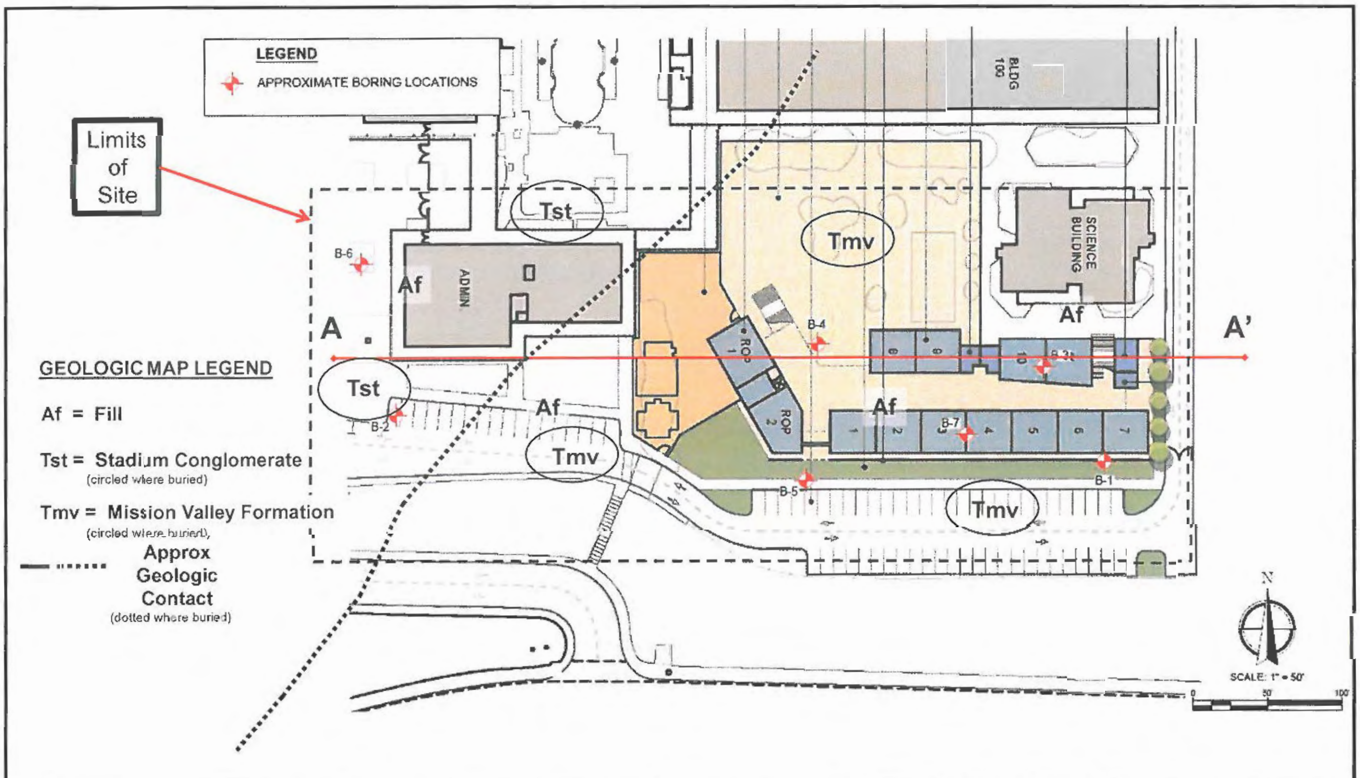
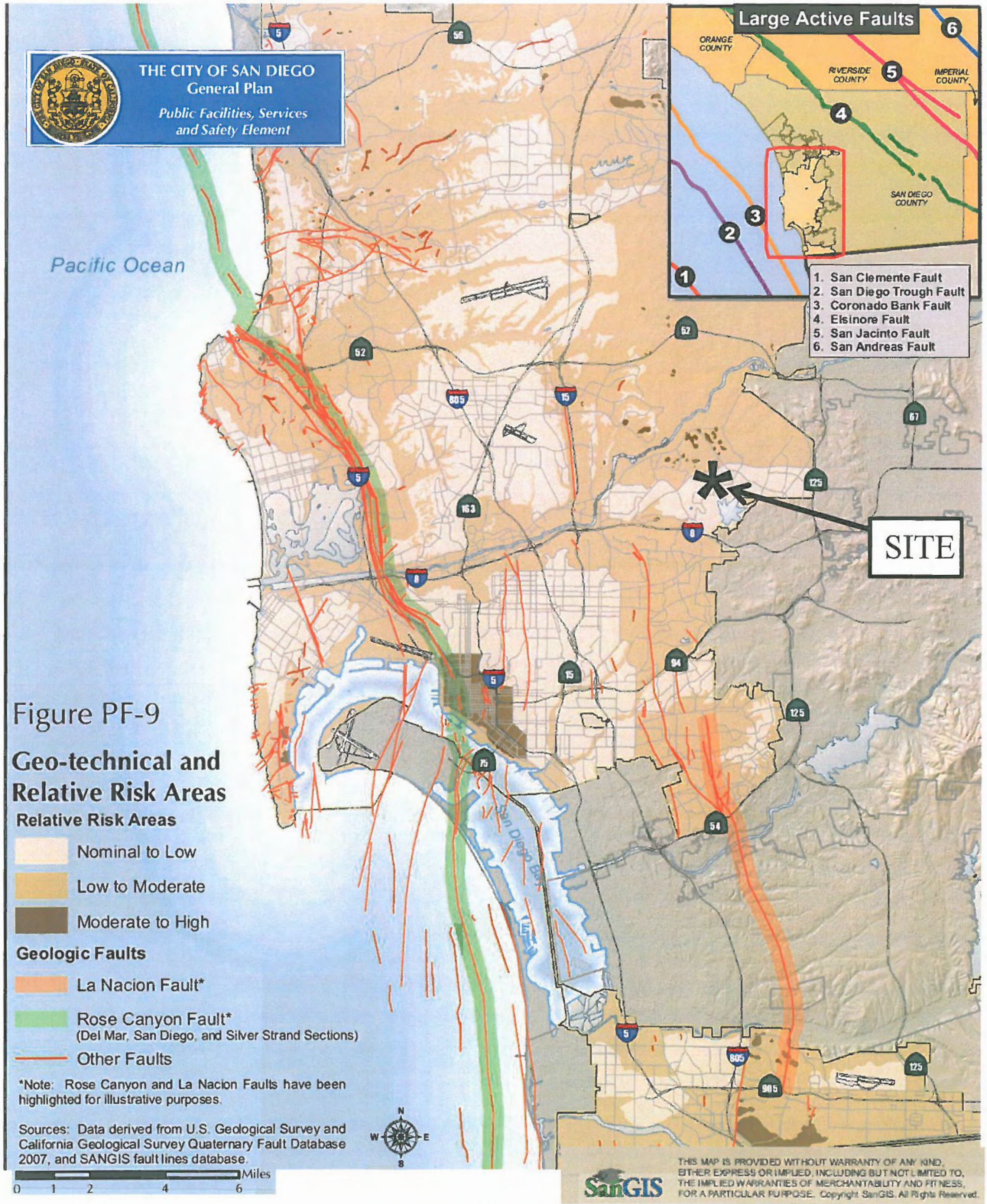
REGIONAL GEOLOGIC MAP

ENGINEERING GEOLOGY INVESTIGATION
PATRICK HENRY HIGH ACHOOL
PROPOSED TWO-STORY CLASSROOM
AND ADMINISTRATION BUILDING
6702 WANDERMERE STREET, SAN DIEGO, CALIFORNIA

EWR	DATE 12-9-2014	PROJECT NO. M1065-01	ENCLOSURE 2



AKW GEOTECHNICAL P.O. BOX 891173, TEMECULA, CA 92589 PHONE (951) 265-9849	**SITE GEOLOGIC MAP** ENGINEERING GEOLOGY INVESTIGATION PATRICK HENRY HIGH ACHOOL PROPOSED TWO-STORY CLASSROOM AND ADMINISTRATION BUILDING 6702 WANDERMERE STREET, SAN DIEGO, CALIFORNIA
EWR — DATE 12-9-2014	PROJECT NO. M1065-01 — ENCLOSURE 3



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REGIONAL FAULT MAP

ENGINEERING GEOLOGY INVESTIGATION
PATRICK HENRY HIGH ACHOOL
PROPOSED TWO-STORY CLASSROOM
AND ADMINISTRATION BUILDING
6702 WANDERMERE STREET, SAN DIEGO, CALIFORNIA

EWR	DATE 12-9-2014	PROJECT NO. M1065-01	ENCLOSURE 1