ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-USE/CLASSROOM ADDITIONS
WINTERSBURG PRESBYTERIAN CHURCH
2000 NORTH FAIRVIEW STREET
SANTA ANA, CALIFORNIA

Prepared For:

WINTERSBURG PRESBYTERIAN CHURCH
C/O LA BONTE AND ASSOCIATES
14732 LIVINGSTON STREET
TUSTIN, CALIFORNIA 92780

RECEIVED
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City of Santa Ana

2000 N. FAIRVIEW ST.

Project No. 10-6212

June 30, 2010

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Consulting Geotechnical Engineers

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June 30, 2010
Project No. 10-6212

WINTERSBURG PRESBYTERIAN CHURCH
c/o LA BONTE AND ASSOCIATES
14732 Livingston Street
Tustin, California 92780

Attention: Mr. Larry La Bonte

Subject: REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION
Proposed Multi-use/Classroom Additions
Wintersburg Presbyterian Church
2000 North Fairview Street
Santa Ana, California

Gentlemen:

Presented herewith is the Report of Preliminary Geotechnical Investigation ("Soils Report") prepared by Associated Soils Engineering, Inc. (ASE) for the proposed new additions to the existing church at the subject address (the "Addition"). This work was conducted in accordance with ASE's Proposal No. P10-070, dated May 18, 2010, and your subsequent authorization.

The subject geotechnical investigation was planned and performed based on the provided development information, which included a Phase One Site Plan (Sheet 3) and Existing Floor Plans (Sheet 5), prepared by La Bonte and Associates, which show the location, footprint and maximum design structural loads of the new Addition. Also shown on the Phase One Site Plan were suggested boring locations.

The purpose of this study was to evaluate the subsurface soils conditions at the site, followed by performance of engineering analyses and formulation/assembly of recommendations for the geotechnical design and construction of the proposed new Addition. ASE's study has concluded that the proposed new Addition construction is geotechnically feasible provided that the recommendations and design guidelines with respect to site grading, soil improvements and foundation construction presented in the Soils Report are incorporated in the project plans and design, and implemented during construction. This Soils Report also presents 1) the findings of the geotechnical field investigation, 2) the summary of potential geological/seismic hazard assessment, and 3) the results of laboratory tests performed.

We at ASE appreciate the opportunity to provide our professional services on this important project, and look forward to assisting you during site grading and construction.

If you have any questions or require additional information, please contact the undersigned.

Respectfully submitted,

ASSOCIATED SOILS ENGINEERING, INC.

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Project Engineer

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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the proposed Multi-use/Classroom Additions to Wintersburg Presbyterian Church (the "Addition") at 2000 North Fairview Street, in the City of Santa Ana, California (the "Site"). The approximate location of the Site is shown on the Site Location Map (Figure 1). The purpose of this investigation was to evaluate the general subsurface soil conditions at the Site and provide geotechnical recommendations for the design and construction of Addition. This Soils Report presents the summary of the data collected, and the results of ASE's engineering evaluations/analyses, which provide the basis for the formulation of relevant geotechnical conclusions and recommendations.

1.1 PROJECT DESCRIPTION

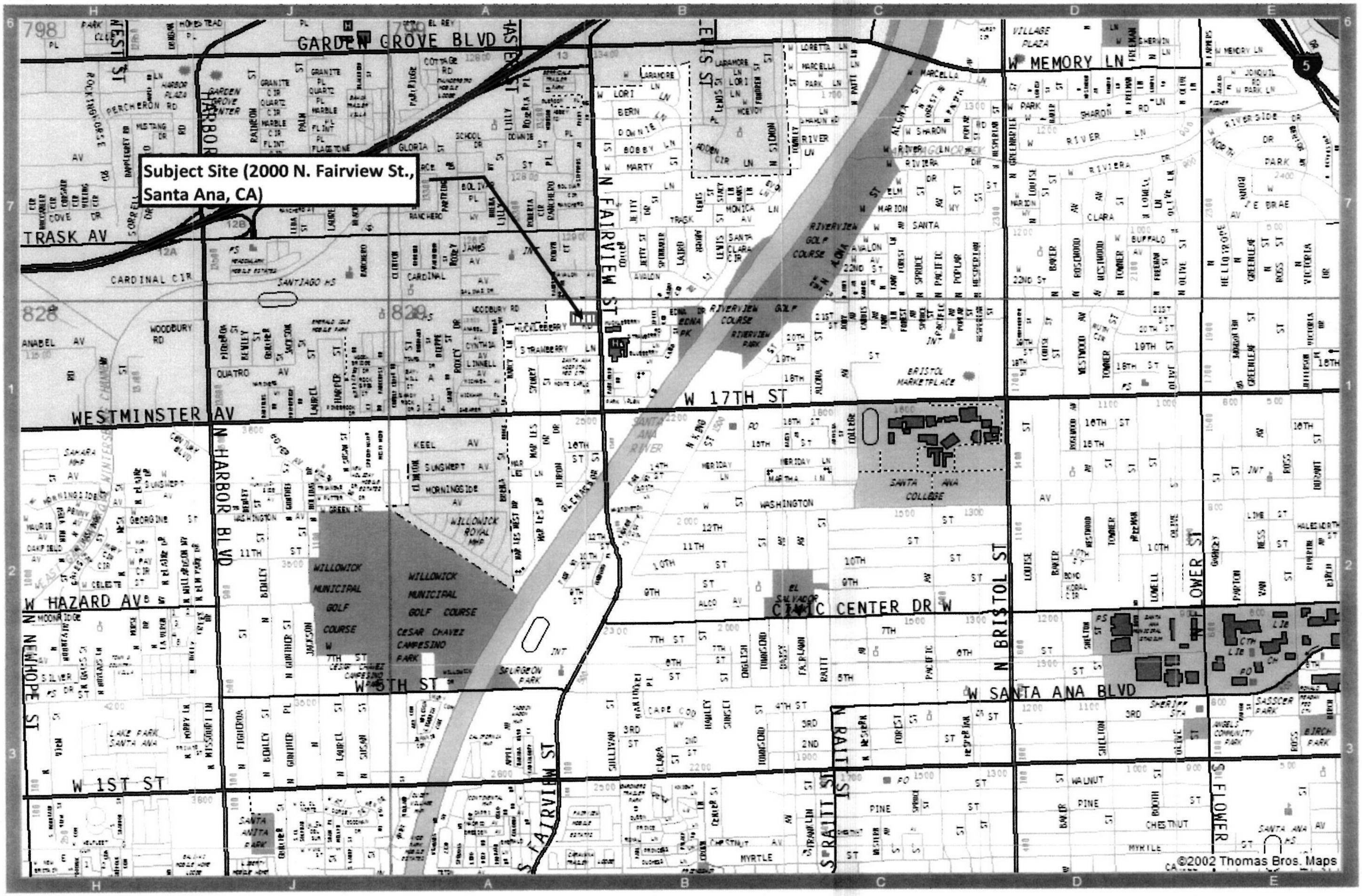
The following provided information is currently understood to be applicable to the subject project.

1.1.1 Building/Development Concepts:

It is ASE's understanding that the planned Addition will include new two -story height addition at the southwesterly corner of, and extending westerly from, the existing church building on site. The new addition will consist of masonry and frame construction and will have interior slab-on-grade construction. The footprint of the new Addition will be an irregular, rectangular shape, with the first floor encompassing 18,052 square feet.

1.1.2 Structural Loading:

It is ASE's understanding that the Addition structure will likely be supported by conventional shallow foundations. Provided plans indicate that the maximum concentrated column load on isolated pad footings (inclusive of both live and dead loads) will be on the order of 185 kips, with a maximum distributed load on continuous footings (dead plus live load) not to exceed 6,500 pounds per linear foot. Tolerable total and differential settlements on the order of 1 inch and 1/4 inch in 30 feet, respectively, have been assumed for the purpose of design.



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Site Location Map

Proj. Name:	**Proposed Multi-Use/Classroom Additions at Wintersburg Presybyterian Church, 2000 N. Fairview St., Santa Ana, CA**
Proj. No.:	**10-6212**
Date:	**June, 2010**

Figure 1

1.2 SCOPE OF EXPLORATION

In accomplishing the subject investigation, ASE's scope of work included the performance of the following tasks:

A. Review of readily available background information, including in-house geotechnical data, geotechnical literature, geologic maps, seismic hazard maps, and literature relevant to the subject site.

B. A geotechnical site reconnaissance to observe the general surficial soil conditions at the site and to select boring locations, followed by notification to Underground Service Alert of the planned boring locations 72 hours prior to drilling.

C. Field exploration consisting of drilling three (3) exploratory borings to depths ranging from 5 feet 7 inches to 51 feet 6 inches below respective existing grades. Field logging and sampling of soils encountered were carried out in each exploratory boring. Locations of the exploratory borings on site are shown on the Boring Location Plan, Plate A.

D. Laboratory testing on representative soil samples for the classification of the materials encountered, and for the determination of the pertinent engineering properties.

E. Engineering analyses of the collected data, including the following aspects:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
- Assessment of geologic/seismic hazards and, if applicable, recommendation of mitigative measures, based on the pertinent criteria required by the California Geological Survey (CGS).
- Determination of the seismic design parameters in accordance with Chapter 16 of the California Building Code, 2007 Edition (2007 CBC).
- Evaluation of the suitability of on-site soils for foundation support, together with the establishment of qualification criteria for on-site or imported fill material.
- Recommendations for site modification and stabilization against the identified seismic hazards.

- Recommendations for design of shallow foundations such as conventional footings including allowable bearing capacity, estimated settlement, and lateral resistance.
- Recommendations for subgrade preparation for slab-on-grade support, including design recommendations.
- Recommendations for the intermediate foundation system such as geopiers in view of the identified seismic hazards.
- Evaluation of the corrosion and expansion potential of the on-site materials.

F. Preparation of this Soils Report presenting the work performed and data acquired, as well as summarizing our conclusions and geotechnical recommendations for the design and construction of the proposed Addition construction.

Please note that ASE's geotechnical investigation did not include any evaluation or assessment of hazardous or toxic materials which may or may not exist on or beneath the site. ASE does not consult in the field of potential site contamination/mitigation.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 LOCATION

The area of proposed Addition is at the southwesterly corner of and extends westward from the Wintersburg Presbyterian Church at 2000 North Fairview Street, in the City of Santa Ana, California. The following information pertaining to site conditions was logged during the course of ASE's field work.

2.2 BOUNDARY CONDITIONS

The site is bound to north by one-story single family residential development. Two-story apartment buildings are south of the property, with a school turf field to the west. The existing church building is immediately east of the area of planned Addition construction, with Fairview Street beyond.

2.3 EXISTING DEVELOPMENT

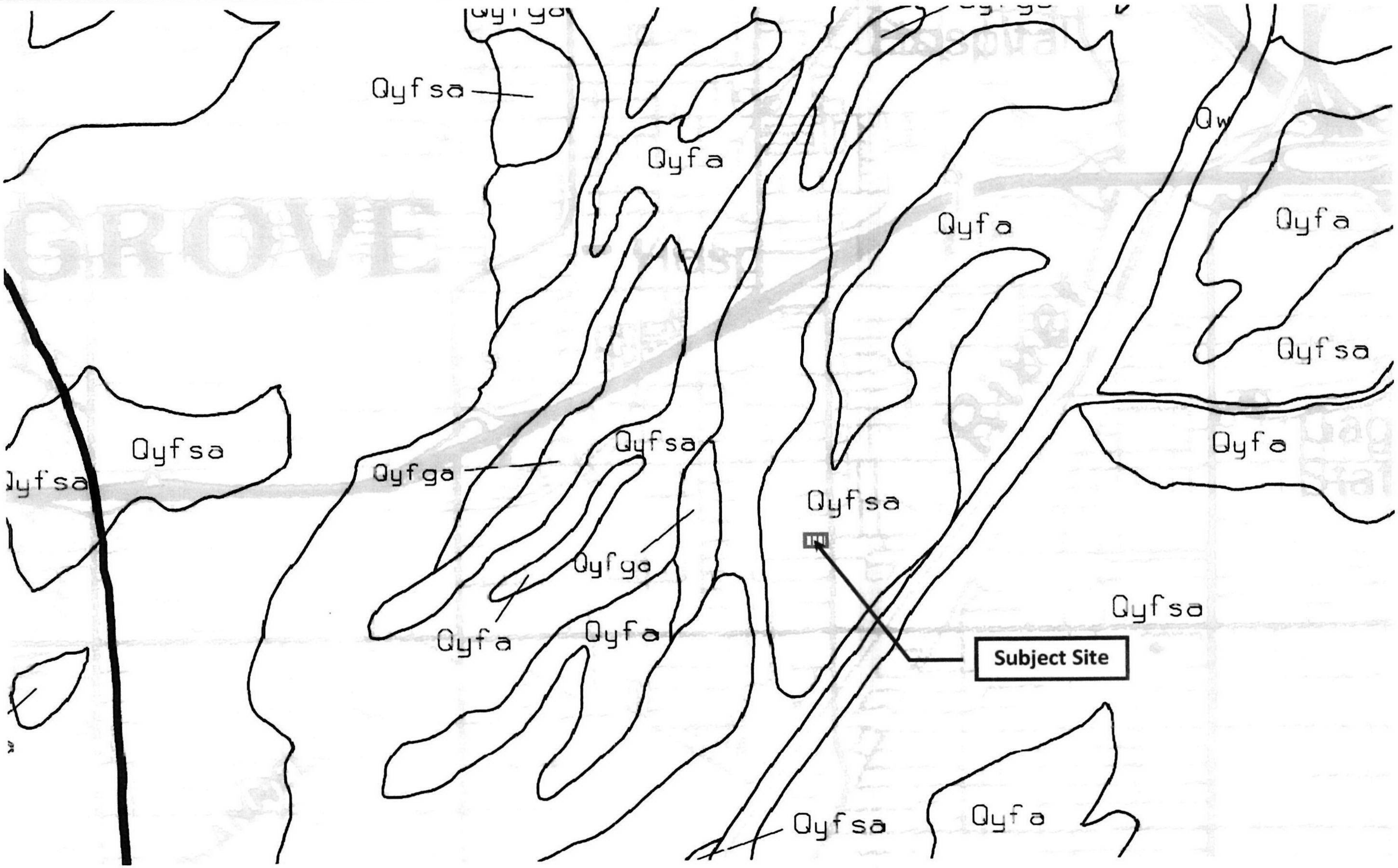
The area of the proposed Addition is generally uniform and level, and consists of mainly of an asphaltic concrete (AC) paved parking lot, along with Portland cement concrete (PCC) flatwork for three (3) basketball courts also stripped for parking. The pavement exhibits slight downward surface gradients to the west and south. Both the existing AC pavement and PCC flatwork visually appear to be in good condition. Some cracking of the AC pavement and PCC curbing was noted around small tree wells.

The existing church building next to the Addition area is one-story construction up to approximately 20 feet high. The Finish Floor of the church is elevated approximately 2.5 feet above the adjacent parking lot grade. A small courtyard area enclosed by masonry walls is located within the area of planned new construction, along with a small caretaker trailer. Small trees, shrubs and ground cover are present in planters adjacent to the church building, along site perimeter fences and in parking lot islands. Some small planter islands containing medium size trees are also present on site.

2.4 SUBSURFACE CONDITIONS

Subsurface exploration disclosed no discernible fill soils at the explored boring locations.

The native earth materials encountered in the exploratory borings consist mainly of Holocene to latest Pleistocene-aged younger alluvial/floodplain deposits (i.e. Qyfsa) up to the maximum explored depth of 51 feet 6 inches. In accordance with CGS (1997, revised 2001), soils within the unit of younger alluvium and floodplain deposits were deposits associated with the Santa Ana River and Santiago Creek. These alluvial soils consist of interstratified silty sands, sands with silt, clean sands, clayey sands with silt, silts, clayey silts, silty clays, clays, and sandy clays with silt. Figure 2, Local Geologic Map, an excerpt from CGS (1997, revised 2001) shows geologic material distribution in the vicinity of the subject site.



(Partial Excerpt of the Quaternary Geologic Map of theAnaheim and Newport Beach 7.5- Minute Quadrangles, California Division of Mines and Geology, Seismic Hazard Zone Report 003, 1997, Revised 2001)

LEGEND

Approximate Site Location

Q_{yfsa} Holocene-age young alluvial fan and floodplain deposits consisting of unconsolidated, loose to moderately dense sand, sandy silt and silt.

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Project:	**Proposed Multi-Use/Classroom Additions at Wintersburg Presybyterian Church, 2000 N. Fairview St., Santa Ana, CA**
Figure 2	**Local Geologic Map**
Proj. No.:	**10-6212** Date: **June, 2010**

Site subsurface soils are, in general, in a damp to wet condition, with some granular and near surface soil layers dry.

Blow counts recorded from advancing Standard Penetration Test ("SPT") sampler and Modified California barrel sampler empirically indicate that the granular strata of on-site alluvial soils are in a loose to dense condition, whereas the fine-grained, cohesive strata (i.e. silts and clays) generally exhibit soft to very stiff consistencies.

More detailed descriptions of soils encountered and conditions observed during the subsurface exploration are shown in the boring logs in Appendix A, together with information including soil classifications, depths and types of soil samples, blow counts, field dry densities and moisture contents, and corresponding laboratory tests performed.

2.5 GROUNDWATER AND CAVING

During ASE's field exploration, groundwater was encountered at depths ranging from 22 feet 3 inches in Boring B-2 to 29 feet 5 inches in Boring B-1. Published data in Seismic Hazard Zone Report 003 for the Anaheim and Newport Beach 7.5-Minute Quadrangles, Orange County, California, published by CGS (1997, revised 2001), indicates that the historic high groundwater level in the subject area is approximately 20 feet below grade. Images reviewed from Google Earth indicate that the subject site is approximately 100 feet above Mean Sea Level (MSL).

Information available from the State of California Department of Water Resources Division of Planning and Local Assistance - Southern Division website (www.water.ca.gov/waterdatalibrary/groundwater/hydrographs/report) indicates that the historic high groundwater level in Well No. 05S10W03K001S, close to the site on the south side of Cardinal Avenue between Roxey Drive and Partridge Street, was 30.9 feet below ground surface elevation on October 30, 2000. The ground surface elevation of the well is 100 feet above MSL. The depth to groundwater for the most recent reading in this well (taken October 27, 2008) was 51.1 feet below existing grade.

Generally, seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations from our observations may occur.

The use of hollow-stem augers during drilling precluded observation of potential caving conditions which may have otherwise occurred in an uncased hole. Caving and/or sloughing were not measured during the extraction of auger stem at the completion of boring operations. However, caving and/or soil sloughing may be likely in excavations greater in dimension than our exploratory borings.

2.6 UTILITIES

No overhead or underground utilities were encountered within the area of ASE's on-site investigation. However, underground and overhead lines are present which service the existing site structures. Irrigation lines are present in planter areas. Overhead lines are present along the northerly and southerly property lines, and along Fairview Street. Light standards are present in the parking lot. Fire hydrants are preset in planters near the northwest and southwest areas of the church exterior. Other utilities, though not known at the time of this report preparation, may be present on site, and should be located and incorporated into site development plans accordingly.

3.0 FAULTING AND SEISMICITY

Santa Ana, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By the definition of the CGS, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults. The subject site is not located within the Alquist-Priolo Earthquake Fault Zone. No evidence of active or potentially active faulting was observed during our investigation.

Several sources were consulted for information pertaining to site seismicity. The majority of data was obtained from the publication by Sadigh, et.al. (1997) which has been incorporated into a digital program (i.e. "EQFAULT") by Blake (2000) that allows for an estimation of peak horizontal acceleration using a data file of approximately 150 digitized California faults. This program compiles various information including the dominant type of faulting within a particular region, the maximum earthquake magnitude each fault is capable of generating, the estimated slip-rate for each fault, and the approximate location of the fault trace. Printouts of the results of the fault search for the subject site are shown in Appendix B.

The subject site is likely to be subject to strong seismic ground shaking during the life of the project. The San Joaquin Hills Fault is the closest known "active" fault to the site and is located approximately 5.0 miles (8.1 km) away. Other nearby active faults includes the Newport-Inglewood (L.A. Basin) Fault and the Whittier Fault, located approximately 7.9 miles (12.7 km) and 11.7 miles (18.8 km) away, respectively.

3.1 DETERMINISTIC ANALYSIS

Based on the referenced literature and deterministic analysis, the San Joaquin Hills Fault approximately 5.0 miles (8.1 km) from the site, would probably generate the most severe site ground motions. A Maximum Probable Earthquake (MPE), i.e. the maximum earthquake that carries a 10 percent exceedance probability in 50 years and a 100-year return period, of 6.6 Mw (moment magnitude as per USGS) has been assessed along the San Joaquin Hills Fault. As shown in Appendix B, estimated peak horizontal ground acceleration (PGA) resulting from a MPE event on the San Joaquin Hills Fault is on the order of 0.406g should this event occur at the fault's closest approach to the site. In addition, approximately 41 active or potentially active faults have been found within 62 miles (100 km) of site.

There are a number of other faults in the Southern California area that are considered "active" and could have an effect on the site in the form of moderate to strong ground shaking, should they be the source of an earthquake. These faults include, but are not limited to, the San Andreas Fault, the San Jacinto Fault, the Whittier-Elsinore Fault and the Newport-Inglewood Fault Zone.

3.2 PROBABILISTIC ANALYSIS

The probabilistic approach incorporates the contributions from all faults and considers the likelihood of the occurrence of earthquakes at any point on the fault. It also incorporates the contributions from earthquakes of various magnitudes, including the maximum credible earthquake, as described by Idriss (1985).

The seismicity of the site was evaluated utilizing probabilistic analysis available from CGS. As described in Cao et.al (2005) and Peterson et al (1996), the CGS analytical framework considers two earthquake sources, i.e. fault sources and area sources, together with geologic/soil characteristics and tectonic movements, for the quantification of PGA of bedrock when subjected to a MPE event. As site-specific ground conditions, e.g. soft rock and alluvium, might attenuate or amplify bedrock-based PGA's, CGS further incorporates recommendations proposed by NEHRP (1994 & 1997) that modify bedrock-based PGA's for both soft rock sites and alluvium sites. For structural design with a typical damping ratio of 5%, two spectral acceleration (S_a) values representing structural periods of 0.2 second (typical of low-rise buildings) and 1.0 second (typical of multi-story buildings) have also been analyzed. As shown in the Appendix B, CGS's probabilistic analysis with a soil classification of "D", as the site is underlain predominantly by younger alluvial deposits soil, indicates that the site is subject to a PGA of 0.379g, a Sa (0.2 sec) of 0.913g, and a S_a (1.0 sec) of 0.461g.

3.3 2007 CBC SEISMIC DESIGN PARAMETERS

The earthquake design requirements listed in the 2007 CBC and other governing standards account for faults classified as "active", in accordance with the most recent fault listing as per the United States Geological Survey (USGS) or the CGS. The proposed structure should be designed and constructed in accordance with applicable portions of Chapter 16 of the 2007 CBC. Construction criteria should follow Seismic Zone 4 and the following seismic design parameters.

The 2007 CBC seismic design criteria for the site had been determined utilizing the Java Ground Motion Parameter Calculator – Version 5.0.8 available from the Seismic Design Values for Buildings webpage on the website of Earthquake Hazard Program of U.S. Geological Survey (http://earthquake.usgs.gov/research/hazmaps/design). A summary of the seismic coefficients is presented in the following table, followed by the design response spectra shown on Figure 3, General Design Response Spectrum.

Please note that conformance to the presented criteria for seismic excitation does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not take place during the occurrence of a MPE. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building may be damaged beyond repair, yet not collapse.

2007 CBC SEISMIC DESIGN PARAMETERS			
Site Latitude:	N 33.7639^0	Site Longitude:	W-117.9043^0
Seismic Parameter		**Recommended Value**	
Site Class [a]		D	
Soil Profile Name [b]		Stiff Soil Profile	
Site Coefficient, Fa [c]		1.0	
Site Coefficient, Fv [d]		1.505	
0.2-Second Spectral Response Acceleration, Ss [e]		1.389 g	
1.0-Second Spectral Response Acceleration, S1 [f]		0.495 g	
Adjusted 0.2-Second Spectral Response Acceleration, SMs [g]		1.389 g	
Adjusted 1.0-Second Spectral Response Acceleration, SM1 [h]		0.746 g	
Design 0.2-Second Spectral Response Acceleration, SDs [i]		0.926 g	
Design 1.0-Second Spectral Response Acceleration, SD1 [j]		0.497 g	
PGA for Site Seismic Hazard Analysis [k]		0.370 g	
Occupancy Category	I or II	III	IV
Seismic Design Category based on SDs [l]	D	D	D
Seismic Design Category based on SD1 [m]	D	D	D

a Per 2007 CBC Table 1613.5.2

b Per 2007 CBC Table 1613.5.2

c Per Java Ground Motion Parameter Calculator from USGS website. Also shown on 2007 CBC Table 1613.5.3 (1).

d Per Java Ground Motion Parameter Calculator from USGS website. Also shown on 2007 CBC Table 1613.5.3 (2).

e Per Java Ground Motion Parameter Calculator from USGS website. Also shown on 2007 CBC Figure 1613.5 (1).

f Per Java Ground Motion Parameter Calculator from USGS website. Also shown on 2007 CBC Figure 1613.5 (2).

g SMs = Fa x Ss

h SM1 = Fv x S1

i SDs = 2/3 x SMs

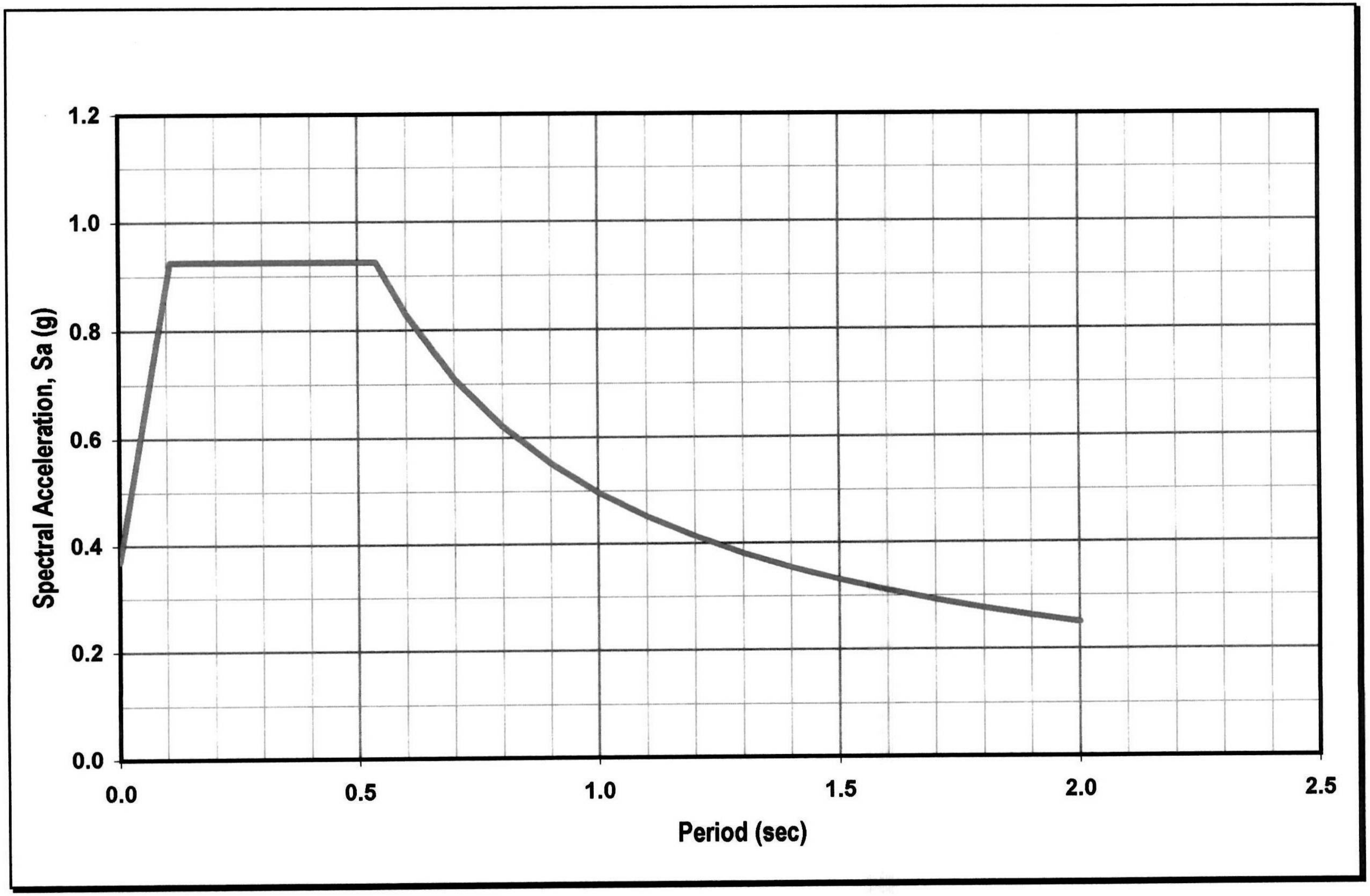
j SD1 = 2/3 x SM1

k PGA = SDs/2.5 per 2007 CBC Section 1802.2.7

l Per 2007 CBC Table 1613.5.6 (1). Also refer to 2007 CBC Section 1613.5.6 for special conditions.

m Per 2007 CBC Table 1613.5.6 (2). Also refer to 2007 CBC Section 1613.5.6 for special conditions.

As the PGA assessed from the deterministic ground motion analysis using "EQFAULT", i.e. 0.406g, appears to be more conservative, it is recommended herein to be incorporated in project structural design and planning, if dynamic structural analysis is adopted.



Graph generated by the Java Ground Motion Parameter Calculator – Version 5.0.8 available from the Seismic Design Values for Buildings webpage on the website of Earthquake Hazard Program of U.S. Geological Survey (http://earthquake.usgs.gov/research/hazmaps/design)

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Project:	Proposed Multi-use/Classroom Additions 2000 N. Fairview Street,Santa Ana,CA		
Figure 3	General Design Response Spectrum		
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4.0 SEISMIC HAZARDS

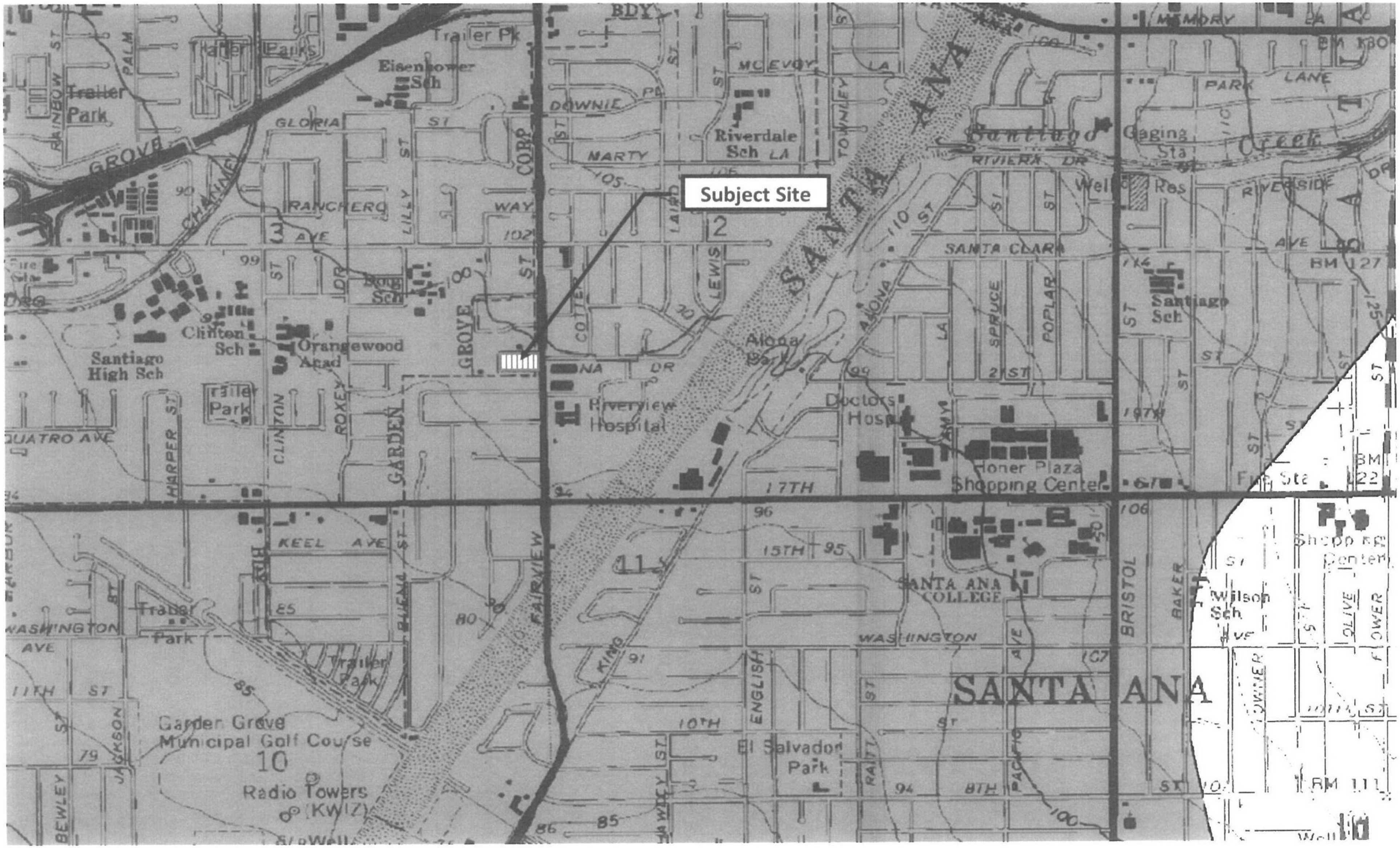
4.1 LIQUEFACTION

As evidenced in Figure 4, Local Seismic Hazard Map, the subject site, as well as the surrounding area, is within an area identified as having a potential for soil liquefaction when subject to a MPE event.

The term "liquefaction" describes a phenomenon in which a saturated cohesionless soil loses strength and acquires a degree of mobility as a result of strong ground shaking during an earthquake. The factors known to influence liquefaction potential include soil type and depth, grain size, relative density, groundwater level, degree of saturation, and both the intensity and duration of ground shaking. The soils to the maximum explored depth of 51 feet 6 inches generally consist of loose to dense granular soils, and soft to very stiff fine-grained soils. Cohesive clayey soils are generally not susceptible to liquefaction.

At the time of ASE's field exploration, groundwater was encountered at depths ranging from 22 feet 3 inches in Boring B-2 to 29 feet 5 inches on Boring B-1. According to CGS (1997, revised 2001), historic high groundwater in the vicinity of the subject site is approximately 20 feet below grade, while the information available from the State of California Department of Water Resources Division of Planning and Local Assistance-Southern District indicates historic high groundwater in the well nearest to the vicinity of the subject site was approximately 30.9 feet below grade recorded in 2000. As such, a high groundwater level at 20 feet deep has been incorporated in ASE's liquefaction analysis.

Liquefaction analysis was performed based on 1) our laboratory data and field data from Boring B-2, 2) PGA from EQFAULT i.e. 0.406g, and 3) high groundwater level of 20 feet deep from CGS (1997, revised 2001). It is recommended in CGS Special Publication 117A (2008) that a factor of safety against liquefaction of 1.30 is deemed the threshold value in determining the liquefaction susceptibility of a certain soil stratum. As presented in Plates L-1 through L-3 in Appendix B, liquefaction analyses performed using the "LIQUEFY2" software indicate that liquefaction-prone



(Partial Excerpt of the State of California Seismic Hazard Zones Anaheim Quadrangle Official Map dated April 15, 1998)

LEGEND

- Approximate Site Location
- Potential Landslide Hazard Area
- Potential Liquefaction Hazard Area

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Figure 4	**Local Seismic Hazard Map**		
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soil strata exist at depths from 21.5 to 23.5 feet, 23.5 to 24.5 feet, and 24.5 to 29 feet in Boring B-2.

As evidenced in Figure 5, Seismically-Induced Surface Manifestation, based on the thickness and depth of the liquefaction-susceptible soil strata encountered, it is likely that surface manifestation such as loss in bearing for foundation and slabs, subsidence with uneven settlement, and/or small-scale lateral spread in localized areas might be experienced at the subject site.

4.2 EARTHQUAKE-INDUCED LANDSLIDES

There is no indication that recent landslides or unstable slope conditions exist on or adjacent to the project site that would otherwise result in an obvious landslide hazard to the proposed development or adjacent properties.

According to CGS (1998) the site is not located within an area identified as having a potential for earthquake-induced landslides as evidenced in Figure 4. Due to the lack of significant relief on or adjacent to the site, the potential for earthquake induced landslides in the future is considered low.

4.3 SEISMIC SETTLEMENTS

Ground accelerations emitted from a seismic event can cause densification of loose soils both above and below the groundwater table that may result in settlements on ground surface due to volumetric compression of soil mass. This phenomenon is often referred to as seismic settlement and commonly takes place in relatively clean sands, as well as soils with low plasticity and less fines. The earth materials on site may undergo seismically-induced settlement during the MPE. While some site soils encountered, consisting predominantly of soft to very stiff fine-grained soils (silts and clays), are not liquefaction-prone, some of the deeper granular soil layers in Boring B-2 have been identified to be potentially liquefiable, as described in the preceding Section 4.1, taking into account the historical high groundwater at 20 feet deep.

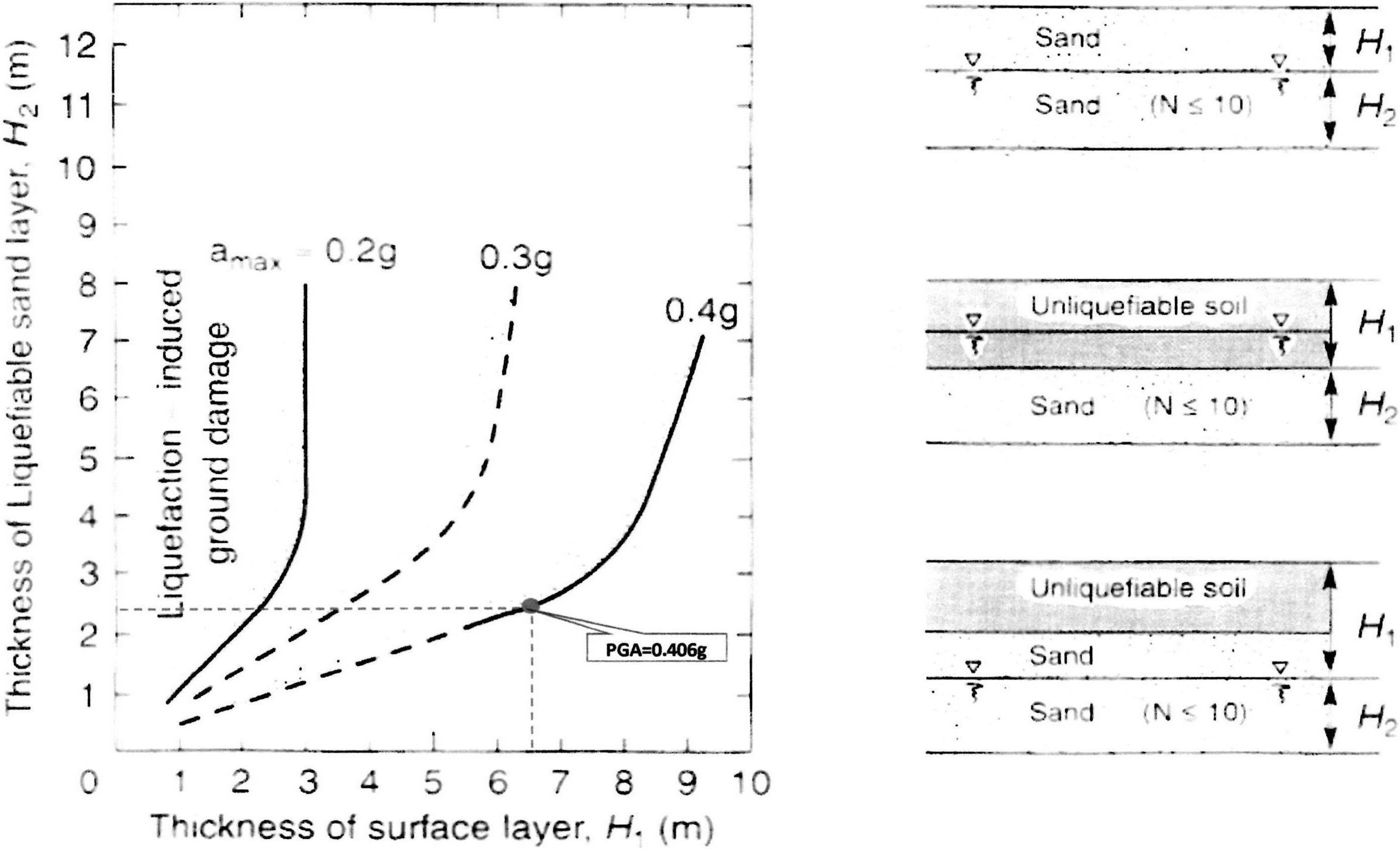

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	Figure 5	**Liquefaction-Induced Surface Manifestation**	
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Based on the procedures developed by Tokimatsu and Seed (1984), total liquefaction-induced settlement ("wet" seismic settlement) at the site has been assessed to be 2.04 inches based on the soil profile encountered in Boring B-2. Differential settlement taking place within the liquefiable soils is typically considered to be on the order of two-thirds (2/3) of the total "wet" seismic settlement, i.e. 1.36 inches near the vicinity of Boring B-2. Such scale of "wet" seismic settlement is anticipated to impose significant impact to the proposed Addition. The calculations of "wet" seismic settlement for saturated soil in Boring B-2 are presented on Plate M-1 in Appendix B.

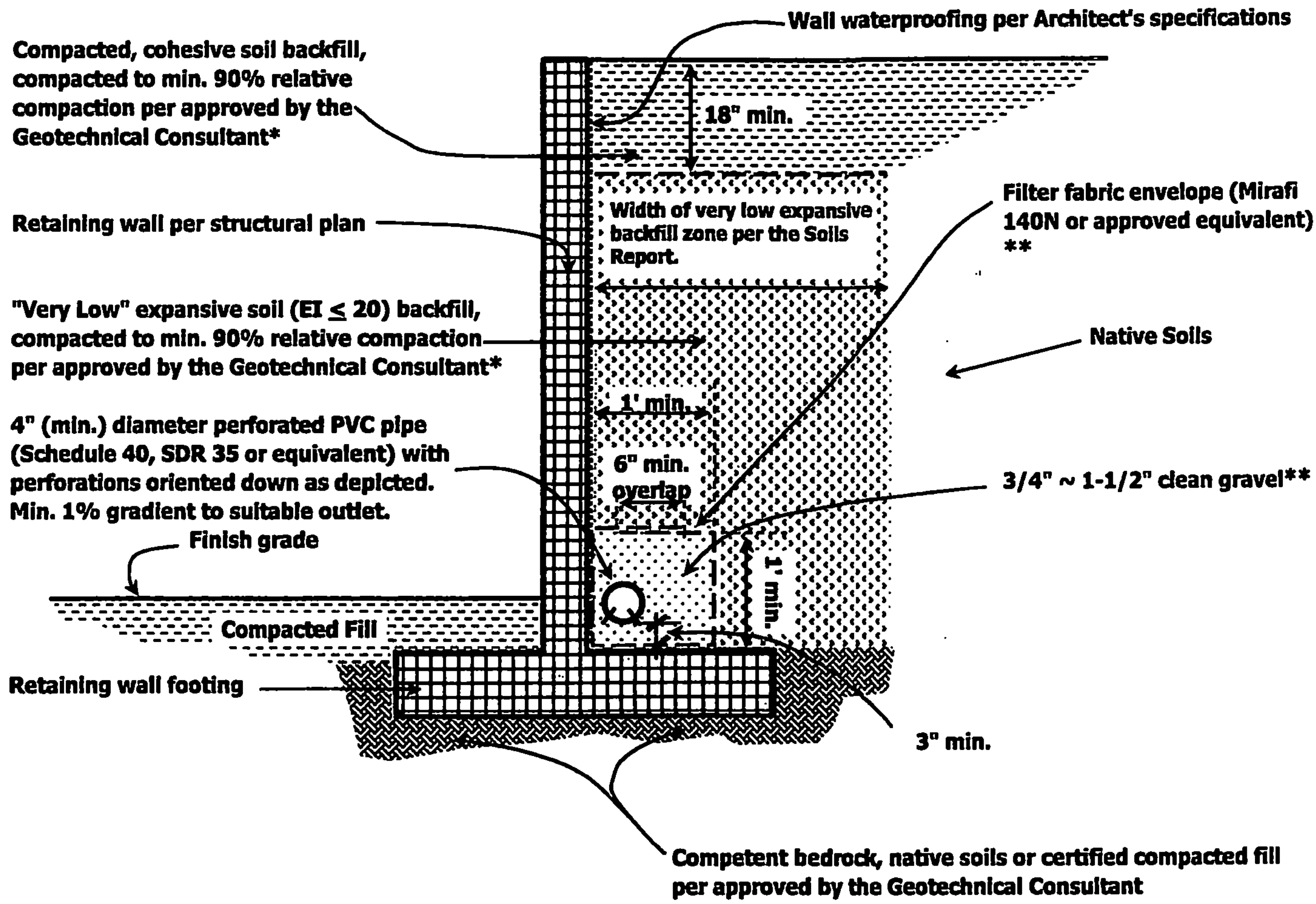
The as-graded soil condition of the site is anticipated to result in subgrade soils generally exhibiting a hard, dense consistency. Settlement of on-site granular soils as a result of seismically-induced densification (i.e. "dry" seismic settlement) has been calculated to be 0.52 inch for soil profile encountered in Boring B-2, as shown on Plate M-1 in Appendix B, in accordance with the procedures recommended by Krinitsky et al. (1993). Such "dry" seismic settlement is expected to affect relatively large pad areas such that the differential settlement over short distances is likely to be low.

4.4 HYDROCONSOLIDATION

Laboratory test results indicate the potential for slight hydroconsolidation in the site soils. For areas on site that are covered with AC pavement or concrete flatwork, or if interceptor systems are installed beneath planter or turf areas to minimize infiltration of moisture into or divert water away from foundation subgrade soils, the potential impact from hydroconsolidation in these areas should be greatly reduced. Hydroconsolidation potential of subgrade soils will also be greatly reduced after implementation of site remedial grading and/or liquefaction mitigation measure with intermediate geopiers.

4.5 SURFACE FAULT RUPTURE

The potential for surface fault rupture due to movement of a primary fault occurring on the site is considered very low.



SPECIFICATIONS FOR CALTRANS CLASS 2 PERMEABLE MATERIAL	
U.S. STANDARD SIEVE SIZE	% PASSING
1"	100
3/4"	90 ~ 100
3/8"	40 ~ 100
No. 4	25 ~ 40
No. 8	18 ~ 33
No. 30	5 ~ 15
No. 50	0 ~ 7
No. 200	0 ~ 3
Sand Equivalent > 75	

* Based on ASTM D-1557-02

** If Caltrans Class 2 permeable material (see gradation to left) is used in place of 3/4" ~ 1-1/2" gravel, filter fabric may be deleted. Caltrans Class 2 permeable material should be compacted to minimum 90 percent relative compaction. Unless otherwise specified, a minimum of 1 cubic foot of gravel should be used for each 1 foot run of drain.

Note: Composite drainage products such as Contech C-Drain, Miradrain or J-Drain may be used as alternative to gravel or Class II. Installation should be performed in accordance with manufacturer's specifications.

Prop. Multi-use / Classroom Additions at
Wintersburg Presbyterian Church
2000 N. Fairview St., Santa Ana, CA

Schematic Not To Scale

Associated Soils Engineering, Inc.
2860 Walnut Avenue
Signal Hill, CA 90755
Tel (562) 426-7990 Fax (562) 426-1842

Project:	~~Prop. Add. & Remodeling to the Museum of Flying, 3100 Airport Ave., Santa Monica, CA~~
Figure 6	RETAINING WALL DRAINAGE DETAILS
Proj. No.:	~~09-6167~~ 10-2212
Date:	~~September, 2009~~ June 2010

4.6 LATERAL SPREADING

Lateral spreading, a phenomenon associating with seismically-induced soil liquefaction, is a display of lateral displacement of soils due to inertial motion and lack of lateral support during or post liquefaction. It is typically exemplified by the formation of vertical cracks on the surface of liquefied soils, and usually takes place on gently sloping ground or level ground with nearby free surface such as drainage or stream channel. Since there is no presence of free surface on or nearby the subject site, the potential for the occurrence of seismically-induced lateral spreading is considered unlikely on the subject site.

4.7 TSUNAMIS AND SEICHES

Due to the elevation of the site and absence of nearby waterfront, hazard from a tsunami is considered very low.

Seiches are rhythmic movements of water within a lake or other enclosed or semi-enclosed body of water, generally caused by earthquakes. Since no lakes or other bodies of water lie on or near the site, the hazard from seiches is not present at the site.

4.8 FLOOD HAZARDS

The subject site is located on the ESRII/FEMA Hazard Awareness site. The subject site is not located within the limits of the 100 year flood plain per FEMA Flood Insurance Rate Map (Map No. 06059C0144J, dated December 3, 2009).

5.0 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

It is ASE's geotechnical opinion that the site may be developed as planned, provided the site grading and foundation design/construction criteria recommended herein are incorporated into the project plans and specifications and implemented during construction.

The major geotechnical considerations affecting the design and construction of the Addition construction is the significant ground settlement potential on site that draws its roots from the following:

1. Soil disturbances as a result of site demolition and clearing operations.
2. Presence of loose, low density soils within the zone of foundation bearing stratum.
3. Presence of foundation of the existing building that need to be taken into account when performing site grading and foundation construction.
4. Potential for soil liquefaction on site when subject to significant seismic events.

All potential soils settlements that might take place on-site due to both seismically-induced ground displacement and statically-induced soils consolidation with respect to different scenarios are summarized in the table on next page.

Category of Settlement	SCENARIOS							
	As-Is Site Conditions		Post-Grading Site Conditions without Ground Modifications[1, 2]		Post-Grading Site Conditions with Compaction Grouting[1, 2]		Site Conditions with Geopiers[3]	
	Shallow Footings	Mat Foundation	Shallow Footings	Mat Foundation	Shallow Footings	Mat Foundation	Shallow Footings	Mat Foundation
"Wet" Seismic Settlement (in.)	2.04[4]	2.04[4]	2.04[4]	2.04[4]	0.0	0.0	0.0	0.0
"Dry" Seismic Settlement (in.)	0.52[4]	0.52[4]	0.38[5]	0.38[5]	0.38[5]	0.38[5]	0.0	0.0
Static Ground Settlement Due to Surcharge Loading (in.)	0.95[6]	8.74[7]	0.38[8]	8.71[7]	0.38[8]	8.69[7]	0.25	6.79[7]
Total Composite Settlement (in.)	3.51	11.30	2.80	11.13	0.76	9.07	0.25	7.64

NOTES:
1. Refer to Section 5.2.3 for remedial grading recommendations;
2. Refer to Section 5.2.2 for compaction grouting recommendations;
3. Refer to Section 5.2.2 for geopier recommendations
4. See Plate M-1;
5. "Dry" seismic settlement as shown on Plate M-1 minus the 0.14" in upper 5'.
6. Refer to Section 5.3.1d for static settlement of shallow footings;
7. Refer to Plates N-1 through N-4 in Appendix C;
8. Post-grading static ground settlement for shallow footings.

Geotechnically, it is conceived that mat foundation is more tolerant to ground displacement than conventional shallow footings due to its higher rigidity and load spread mechanism. For example, while conventional shallow footings are deemed potentially problematic should composite total settlement on-site exceed 2 inches, the mat foundation, if properly designed and constructed, may withstand composite total settlement up to 4 inches. For the adoption of shallow foundation systems such as conventional footings and mat foundation on the subject site, ground modification measures such as compaction grouting is deemed necessary to limit the total composite settlement within the tolerable magnitude for shallow foundation. However, for the structure size of the Addition, the static ground settlement associated with conventional shallow footings will be much less compared to that associated with mat foundation as the latter would likely trigger consolidation in the deeper compressible clayey soils due to deepened stress influence as shown in the table on the next page. As such, mat foundation is not recommended for the subject Addition.

In addition to the consideration of ground modification measures for the support of the Addition on conventional shallow footings if compaction grouting is utilized, it is ASE's opinion that overexcavation and backfilling with properly compacted fill within the upper five (5) feet in the building area, as recommended in the following Section 5.2.2, is essential to reduce unfavorable static foundation displacement as a consequence of settlement of underlying loose soils, and to provide satisfactory bearing stratum for the Addition. The grading recommendations provided herein should be reviewed when final project concept and grading plans become available. It is assumed that the proposed finish grades will be close to existing site grades (± one foot).

The use of deep foundation system such as driven piles or Cast-In-Drilled-Hole (CIDH) caissons, though technically feasible, appears to be a more costly approach. The deep-rooted and relatively thick liquefiable soil strata encountered on site, and underlying soft clayey soils, inevitably compromise the overall capacity of deep foundation due to the anticipated significant downdrag and deep lateral sway, which makes the deep foundation system more costly since greater depth and/or larger foundation stiffness, will likely be required. While no deep foundation design recommendation is provided in this Soils

Report, the Geotechnical Consultant should be consulted if such an approach is preferred by the Owner.

However, the Owner and the Structural Consultant should decide on the preferred foundation system and ground improvement scheme based on 1) the category of structural essence/importance; 2) cost effectiveness; 3) criteria/requirements that are imposed by governing authority.

This firm's findings indicate that the proposed structure, if designed and built according to our recommendations, will be safe from hazards and landslides, settlement or slippage, per Section 111 and such construction will not adversely affect the geologic stability of property outside the building site.

5.1 SITE PREPARATION

5.1.1 Existing Improvements:

Prior to grading operations, it will be necessary to remove designated existing improvements, including any remaining buried obstructions, which may be in the areas of proposed construction. Structure removal should include foundations. Concrete flatwork and asphalt pavements should also be removed from areas of proposed construction. Concrete and asphalt fragments from site demolition operations should be disposed of off-site, unless they can be stockpiled and processed to meet the specifications for Crushed Miscellaneous Base ("CMB"), as outlined in Section 200-2.4 of the latest edition of the "Standard Specifications for Public Works Construction", (i.e. the "Green Book") and reused as granular fill or base material.

5.1.2 Surface Vegetation:

Surface vegetation should be stripped from areas of proposed construction. Stripping should penetrate six inches into surface soils. Any soil contaminated with organic matter (such as root systems or strippings mixed into the soil) should be disposed of off-site or set aside for future use in non-structural landscaped areas. Removal of trees and shrubs should include rootballs and attendant root systems.

5.1.3 Underground Utilities:

Any underground utilities to be abandoned within the zone of proposed construction should be cut off a minimum of 5 feet from the area of the new structure. The ends of cut-off lines should be plugged a minimum of 5 feet with concrete exhibiting minimum shrinkage characteristics to prevent water migration to or from hollow lines. Capping of lines may also be required should the plug be subject to any line pressure.

Alternatively, deep hollow lines may be left in place provided they are filled with lean concrete or 2-sack control density fill (slurry fill). No filled line should be permitted closer than two (2) feet from the bottom of future footings, unless it has been evaluated and approved by the Geotechnical Consultant.

Local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

5.2 SITE GRADING

In view of reducing the adverse effects associated with the development of excessive total or differential settlement underneath the proposed Addition, as well as to ensure uniform bearing competency for the foundations, engineered improvements of near surface on-site soils are recommended in the following sections.

5.2.1 Undocumented Fill/Disturbed Native Soils:

Although not observed in ASE's exploratory borings, any undocumented fill soil, if encountered in the area of proposed Addition, as well as any native soils disturbed during demolition and clearing operations, should be excavated full depth. Lateral extent of overexcavation beyond Addition perimeters, where possible, should be to a minimum distance equal to the depth of fill/loose soil encountered or five (5) feet, whichever is greater.

The exposed excavation bottom should be scarified/reworked to a minimum one (1) foot depth and recompacted to a minimum 90 percent relative compaction at 1 to 3 percentage points above optimum moisture contents prior to backfilling with approved soils as specified in Section 5.2.8. Unless otherwise stated, the measurement of relative compaction in this report should always refer to the latest edition of ASTM D1557 test procedure.

5.2.2 Ground Modification Considerations:

In order to alleviate the potentially significant total and differential settlements that may develop on-site upon the occurrence of the seismically-induced soils liquefaction, it is deemed a feasible geotechnical approach by improving the density/compaction of subsurface soils located between approximate depths of 21.5 to 29 feet deep for building support on shallow conventional footings through compaction grouting which densifies loose soils by displacement. Alternatively, soils may be improved by installation of geopiers which improves the density of soils full depth with a combination of vibro-compaction and vibro-replacement.

Based on ASE's evaluation of site soil characteristics, liquefaction mitigation alternatives, existing site constraints, planned scale of development, availability of specialist geotechnical contractor and cost effectiveness, it is the professional opinion of ASE that compaction grouting appears to be a suitable measure for mitigating liquefaction-induced settlement on site. Relevant information of compaction grouting is shown in Appendix D. In particular, soil strata located at approximate depths between 21.5 and 29 feet on site should be the targeted layers of improvement for the Addition to be supported on shallow footings. Based on the existing density of site soils at depths between 21.5 and 29 feet (see Field Logs of Borings in Appendix A), it has been preliminarily estimated by ASE that, in order to achieve a target relative compaction of 95%, grout slurry volume on the order of ten (10) percent of the existing volume of the targeted improvement zone should be injected. The targeted improvement zone should be extending at least five (5) feet laterally beyond the footprint of the Addition, and should be between 21.5 and 29 feet deep below the existing site grade. Nonetheless, a specialty contractor experienced and knowledgeable in liquefaction mitigation using grouting should be retained for a performance-based, design-build mitigation program.

In order to verify the end result of compaction grouting for liquefaction mitigation, it is recommended that a post-grouting exploration program consisting of verification borings and sampling be implemented. It is recommended that a minimum of three (3) borings be advanced with hollow-stem auger within the area of the Addition. Starting at 21.5 feet deep, Standard Penetration Test ("SPT") sampler should be advanced continuously in 18-inch intervals down to 29 feet deep in order to verify final compaction/resistance of soils within the improved soil strata. As a preliminary benchmark for compaction grouting design/planning consideration, an uncorrected SPT blow count, i.e. the "N" value, of at least <u>28</u> should be registered during the verification boring in the targeted improvement zone.

In view of the existence of significant liquefaction potential within the on-site alluvial deposits layers, it is recommended that intermediate geopier ground modification measure such as vibro piers and rammed aggregate piers may also be considered. Brief descriptions of these ground modification techniques, excerpted from respective specialty contractor's product information, are shown in Appendix D. Through different ways of "pre-inducing" liquefaction or soil densification by means of high-frequency ramming and vibratory motion simultaneously with aggregate replacement of the vibration-created void space in the liquefiable soil layers, the resultant soil matrix (a combination of densified site soils and well compacted aggregates) is anticipated to exhibit a final composite shear strength sufficiently strong in resisting the excessive seismic loading during a seismic event, thus permitting the new structure to be supported by conventional shallow footings. Concrete, bricks and asphaltic concrete (AC) derived from site demolition operation may be reprocessed into materials compatible to the aggregate utilized for geopier application, or to the Crushed Miscellaneous Base (CMB) per Section 200-2.4 of the latest edition of the Standard Specifications for Public Works Construction, i.e. the "Green Book", and re-used. In addition, the loose, shallow bearing stratum will be densified by the intrusion of geopiers thus eliminating the need of site remedial grading within the ground modifications area. If geopiers are selected for improvement of site soils and liquefaction mitigation, specialty contractor knowledgeable in the design and installation of geopiers should be retained for the design build work as proprietary equipment and technology are oftentimes specific to the specialty contractor.

Although no ground improvement is allowed to go beyond the property lines due to 1) the rather limited scale and distance of the existing developments on the adjoining properties and 2) the possible presence of shallow foundation on neighboring structures, it is ASE's opinion that ground modification involving compaction grouting or geopiers limited within the subject property lines is anticipated to provide satisfactory performance in mitigating the concerned liquefaction hazard.

5.2.3 Remedial Grading:

a) With the Adoption of Compaction Grouting:

In order to provide acceptable support for structure foundations and slabs, it is recommended that the site soils within the building pad for the Addition be overexcavated uniformly to a minimum depth of <u>five (5) feet</u> below existing site grade, or three (3) feet below the bottom of the lowest footing, whichever is deeper, and replaced with properly compacted fill such that the Addition is supported on a layer of re-engineered, compacted fill. The excavation bottom should be near uniform. The overexcavation should extend laterally to a minimum distance of five (5) feet beyond building perimeters, where possible.

b) With the Adoption of Geopiers:

In order to provide acceptable support for structure foundation, it is recommended that the site soils be overexcavated uniformly to a minimum depth of <u>one (1) foot</u> below the footing bottoms and replaced with properly compacted fill such that the Addition footings are supported on a layer of re-engineered, compacted fill. The excavation bottoms should be near uniform.

The overexcavation should extend laterally to a minimum distance of one (1) foot beyond footing edges, where possible.

Soils beneath building interior slabs should also be overexcavated uniformly to a minimum depth of one (1) foot below slab bottoms, and replaced with suitable site or import, very-low expansive granular soil compacted to minimum 90 percent relative compaction at 1 to 3 percentage points above optimum moisture contents.

Soils exposed at excavation bottom to a depth of six (6) inches should be reworked and recompacted to exhibit a minimum 90 percent relative compaction at 1 to 3 percentage points above optimum moisture content prior to receiving fill placement or footing construction. The exposed excavation bottoms should be observed, tested, and approved by the Geotechnical Consultant prior to placing compacted fill. In case of the presence of localized loose soils, the overexcavation needs to be deepened accordingly to remove the loose soil condition. However, this deepened overexcavation may be terminated when the exposed native, undisturbed soils exhibit a natural relative compaction greater than 85 percent, subject to the testing and inspection by the representative from the Geotechnical Consultant.

However, this overexcavation and recompaction requirement may be omitted if 1) the post-geopier-installation compaction testing on the surficial soils show a minimum 90 percent relative compaction has been reached within subgrade soils, and 2) permission from the local governing authority is obtained.

The Geotechnical Consultant should be provided with appropriate foundation details and staking during grading to verify that depths and/or locations of the recommended overexcavation are adequate. For areas on site that grading recommendations stipulated in both Sections 5.2.1 and 5.2.3 apply, the more stringent ones should govern.

The depth of overexcavation should be reviewed by the Geotechnical Consultant during the actual construction. Any subsurface obstruction, buried structural elements, and unsuitable material encountered during grading, should be immediately brought to the attention of the Geotechnical Consultant for proper exposure, removal and processing, as recommended.

5.2.4 Temporary Excavation:

Excavations of site soils 4 feet or deeper should be temporarily shored or sloped in accordance with Cal OSHA requirements.

a) Temporary Sloping:

In areas where excavations deeper than 4 feet are not adjacent to existing structures or public right-of-ways, sloping procedures may be utilized for temporary excavations. It is recommended that temporary slopes in both fill and native soils be graded no steeper than 1.5:1 (H:V) for excavations up to 10 feet in depth. The above temporary slope criteria is based on level soil conditions behind temporary slopes with no surcharge loading (structures, traffic) within a lateral distance behind the top of slope equivalent to the slope height.

It is recommended that excavated soils be placed a minimum lateral distance from top of slope equal to the height of slope. A minimum setback distance equivalent to the slope height should be maintained between the top of slope and heavy excavation/grading equipment.

Should running sand conditions be experienced during excavation operations, flattening of cut slope faces, or other special procedures may be required to achieve stable, temporary slopes.

Soil conditions should be reviewed by the Geotechnical Consultant as excavation progresses to verify acceptability of temporary slopes. Final temporary cut slope design will be dependent upon the soil conditions encountered, construction procedures and schedule.

b) Temporary Shoring:

Temporary shoring will be required for those excavations where temporary sloping as specified above is not feasible.

Temporary cantilever shoring, if used, should be designed to resist an active earth pressure of 35 pounds per cubic foot (pcf) equivalent fluid pressure (EFP) for level soil conditions behind shoring. The resultant lateral deflection of shoring and surficial settlement immediately behind shoring are estimated to be on the order of one (1) to one and one half (1½) percent of the shored excavation depth. Should this ground deformation be intolerable to the existing structure, ASE should be consulted for more detailed analysis and further recommendations.

The design of shoring should also include surcharge loading effects of existing structures and anticipated traffic, including delivery and construction equipment, when loading is within a distance from the shoring equal to the depth of excavation.

In addition to the above, a minimum uniform lateral pressure of 100 pounds per square foot in the upper ten feet of shoring should be incorporated in the design when normal traffic is permitted within ten feet of the shoring.

c) Slot Cutting Adjacent to Existing Structures:

Prior to any excavations the footing systems of the existing structure should be researched. It would not be permitted to excavate site soils adjacent to or below existing footing foundation. "A-B" slot cutting grading procedures may be utilized to accomplish the required overexcavation for areas adjacent to existing building that might otherwise be undermined by the grading operation on the subject site. As a general guideline, slot cutting would be necessary for overexcavation located within a lateral distance from the existing structure or public right-of-ways equivalent to one (1) times the excavation depth.

While the maximum width and sequence of slot-cuts should be evaluated in the field during grading operations based on conditions exposed during initial site grading adjacent to the existing structures, for preliminary planning purpose, the width per slot should not exceed ten (10) feet. Increase of length per cut slot is possible upon inspection and evaluation of actual exposed slot cut condition by the Geotechnical Consultant during site grading. Care shall be exercised such that no soil is removed from underneath any existing shallow foundation.

5.2.5 Exterior Concrete Slab/Flatwork Support:

For the purpose of reducing future unsightly and uneven movements and cracks, it is recommended that the upper one (1) foot of soils below exterior concrete flatwork or hardscapes should consist predominantly of very low-expansive, suitable site, import or blended material (EI not greater than 20), compacted to minimum 90 percent relative compaction at 1 to 3 percentage points <u>above</u> optimum moisture contents. Prior to placement of the above recommended fill layer, the upper six (6) inches of exposed native subgrade should be reworked to 90 percent relative compaction at 1 to 3 percentage points <u>above</u> optimum moisture contents.

The above subgrade preparation for exterior slab/flatwork may be omitted if geopier ground modification is adopted.

5.2.6 New Fills:

If any non-structural area is to receive new fills as part of any planned grade alteration, the upper eighteen (18) inches of site soils should be reworked and recompacted to a minimum 90 percent relative compaction at 1 to 3 percentage points <u>above</u> optimum moisture content prior to receiving any new fills, where required, to achieve finish grade elevations outside building pad areas.

5.2.7 Suitable Soils and Imported Soils:

Any soil re-used or imported as fill for the completion of grading operations should consist of predominantly very low-expansive material exhibiting an EI not greater than 20, and should be exhibiting a relatively uniform gradation, free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials. Unless otherwise approved by the Geotechnical Consultant, fill materials should also comply with the following soil corrosivity criteria with respect to the desired concrete and reinforcement protection.

Corrosivity Criteria for Select Fill and General Fill			
Soluble Sulfate (% by weight) (1)	Soluble Chloride (ppm) (2)	Resistivity Value (ohm-cm) (3)	pH-Value (3)
≤ 0.1	≤ 100	≥ 2000	7.3 ~ 8.8

(1) California Test Method 417.
(2) California Test Method 422.
(3) California Test Method 532.

All potential import material must be approved by the Geotechnical Consultant or his representative, prior to its use and arrival on site.

5.2.8 Backfilling and Compaction Requirements:

Existing site soils at their present state and composition, unless indicated otherwise, are considered suitable for re-use as fill during site grading at designated areas and depths within the footprint of the building, non-structural or landscape areas, and backfilling of utility trenches, provided they are 1) free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials, 2) are not environmentally contaminated, and 3) adequately moisture conditioned to permit achieving the required compaction. No nesting of large particles (2 to 4-inch size) should be permitted during backfilling operations.

On-site soils and import materials approved for use as fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to 1 to 3 percentage points above optimum moisture content, and compacted to a minimum 90 percent relative compaction.

5.2.9 Tests and Observations:

All grading, compaction, and backfill operations should be performed under the observation of and testing by the Geotechnical Consultant's field representative. An adequate number of field tests should be taken to ensure compliance with this report and local ordinances.

If it is determined during grading that site soils require overexcavation to greater depths for obtaining proper support for the proposed structure, this additional work should be performed in accordance with the recommendations of the Geotechnical Consultant.

Imported fill soils should be examined by a representative of this office, and tested as necessary for evaluating their suitability for use as fill prior to being hauled to the site. Final acceptance of any imported soil will be based upon review and testing of the soil actually delivered to the site.

5.3 FOUNDATION DESIGN

It is ASE's opinion that, with the adoption of compaction grouting or geopier ground modification mentioned in Section 5.2.2, conventional continuous spread footings and isolated pad footings bearing on compacted fill, may be used to provide foundation support for the Addition at the subject site, provided site grading recommendations presented in Section 5.2 are incorporated in project planning and design, and implemented during site construction.

Presented below are the recommended geotechnical design and construction criteria for shallow footing foundation with raised floor and patio slab-on-grade.

5.3.1 Conventional Shallow Footing Foundation:

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting the proposed Addition construction, it is recommended that the following tabulated minimum footing embedments, widths and reinforcements for various footing types be adopted.

***Minimum* Footing Dimension & Reinforcement**					
Continuous Spread Footing			**Isolated Pad Footing**		
Depth (1) (in)	**Width (in)**	**Reinforcement (2)**	**Depth (1) (in)**	**Width (in)**	**Reinforcement (2)**
24	15	Four #4 bars – two near the top and two near the bottom	24	36	Four #4 bars – two near the top and two near the bottom, applied bi-axially

(1) Footing embedment measured from the nearest adjacent lowest soils grade.
(2) Based strictly from geotechnical point of view.

A grade beam at least 12-inches wide should be provided across large entrances/openings exceeding 15 feet in span to minimize differential movements that might otherwise be experienced by the slabs. The same minimum reinforcement recommended for the adjacent footings should be applied to the grade beam as well.

Foundation design details such as concrete strength, reinforcements, etc. should be established by the Structural Consultant.

b) Allowable Soils Bearing Capacity:

For footings complying with the minimum dimension requirements stipulated in Section 5.3.1 a) above, the allowable soils bearing capacities, inclusive of both dead and live loads, should be as per tabulated below:

Allowable Soils Bearing Capacity (psf)		**Increase per 12-inch Increment in Footing Width (psf)**	**Increase per 12-inch Increment in Footing Depth (psf)**	**Maximum Composite Ceiling Value (psf)**
Continuous Spread Footing	**Isolated Pad Footing**			
2,000	2,000	200	500	4,000

The above allowable bearing capacities may be increased by one-third (1/3) when subject to short-term, transient loading induced by wind or seismic activities.

It is recommended that new footings for the Addition should be kept a minimum distance away from the edges of the existing building footings equivalent to the depth of overexcavation (i.e. the distance "Z") such that the above tabulated allowable soils bearing capacity could be utilized without reduction from the effects of footing overlapping. Should this condition not be met due to structural design reason, a reduction factor of 50% should be applied to the above tabulated allowable soils bearing capacity for new footings abutting against the existing building footings (i.e. the distance "0"). For new footings located at a distance between "Z" and "0" from the existing building footings, the reduction factor can be linearly interpolated between "100%" and "50%" accordingly.

Although research of the existing building footings was not part of ASE's scope of work, for structural design consideration, the allowable load-bearing values for Class "4" soils as stipulated in Section 1804 of 2007 CBC may be considered for the existing building footings. However, the same reduction factor criteria as aforementioned should also be applied to the existing building footings.

c) Lateral Resistance:

Resistance to lateral loads can be assumed to be provided by passive lateral earth pressure and by friction acting on structural components in permanent contact with the subgrade soils.

Lateral resistance on the sides of new foundations may be computed using a passive lateral earth pressure of 210 pcf EFP for footings embedded into properly compacted fill, subject to a maximum of 2100 psf. An ultimate coefficient of friction on the order of 0.35 may also be used for structural dead load acting between foundation footings and the supporting soils. The above passive lateral earth pressure may be used in conjunction with the ultimate coefficient of friction in calculating composite lateral resistance, provided the passive lateral earth pressure value is reduced by one-third (1/3).

The same reduction factor for distance consideration mentioned in Section 5.3.1b above should also be applied to the passive lateral earth pressure values mentioned in this section. However, no reduction to the ultimate coefficient of friction is deemed necessary.

d) Settlements:

Total static settlements resulting from compression of subgrade soils for conventional footings designed and constructed in accordance with the above criteria, and supporting maximum provided column and wall loads mentioned in Section 1.1.2 above, are not anticipated to exceed one (1) inch. Upon implementation of site grading as per recommended in Section 5.2.3 above, the total static settlements for conventional footings supporting the provided loads may be reduced to less than 1/2 inch. A differential settlement on the order of 1/4 inch is anticipated between similarly loaded adjacent isolated pad footings, as well as for continuous wall footings over a distance of approximately 30 feet. A differential settlement on the order of 1/4 inch is also anticipated between adjacent column and/or wall footings that support the maximum provided loads. Differential settlements on the order of 1/4 to 1/3 inch are also anticipated between the new and existing construction.

The composite settlements accounting for both seismically-induced soil displacement and statically-induced soil consolidation in a worst-case scenario are anticipated to be on the order of 3.5 inches, with a corresponding composite differential settlement around 2.33 inches. After the implementation of the suggested ground modification alternatives of compaction grouting or geopier, the composite total and differential settlements are anticipated to be reduced to less than one (1) inch and 1/2 inch, respectively, thus rendering the adoption of conventional shallow footing foundation for the support of Addition to be geotechnically feasible.

Please be reminded that ASE should be contacted for further evaluation and recommendations, as necessary, should final design structural loads exceed the maximum loads provided in the above analyses by more than 10 percent.

5.3.2 Retaining Walls:

Cantilevered retaining walls should be designed for an "active" lateral earth pressure value of 35 pcf EFP for approved granular backfill soils and level backfill conditions. An "at-rest" lateral earth pressure value of 55 pcf EFP for approved granular backfill and level backfill conditions should be used for top-restrained retaining walls. Retaining walls subject to uniform surcharge loads should be designed for an additional uniform lateral pressure equal to one-third (1/3) and one-half (1/2) of the anticipated surcharge pressure over the full retained height of the retaining wall (measuring from the top of wall to the heel of wall footing) for cantilevered and top-restrained wall fixity conditions, respectively. Footings should be reinforced as recommended by the Structural Consultant. Appropriate back drainage should be provided to avoid excessive build-up of hydrostatic wall pressures.

Retaining Wall Design Parameter	Value
Allowable Bearing Capacity	2,000 psf (1)
Active Pressure [granular backfill: level]	55 pcf EFP (2)
At-rest Pressure [granular backfill: level]	35 pcf EFP (2)
Passive Pressure (per foot of depth)	210 pcf (3)
Coefficient of Friction	0.35 (3)
Minimum Footing Depth	24 inches
Minimum Footing Width	15 inches
Minimum Reinforcement	Four No. 4 rebar - 2 near top and 2 near bottom

(1) Based on compliance with the above mentioned earthwork recommendations.

(2) Design values assuming a drained condition with very low-expansive materials (EI less than or equal to 20) within the backfill zone and no surcharge loading conditions.

(3) Passive lateral resistance may be combined with frictional resistance provided the passive lateral earth pressure is reduced by 1/3. See Section 5.3.1c.

The Geotechnical Consultant should be on-site during slope cutting and retaining wall construction to inspect the slope conditions, to evaluate the stability of slope cuts and, if necessary, to provide additional remedial or mitigative recommendations.

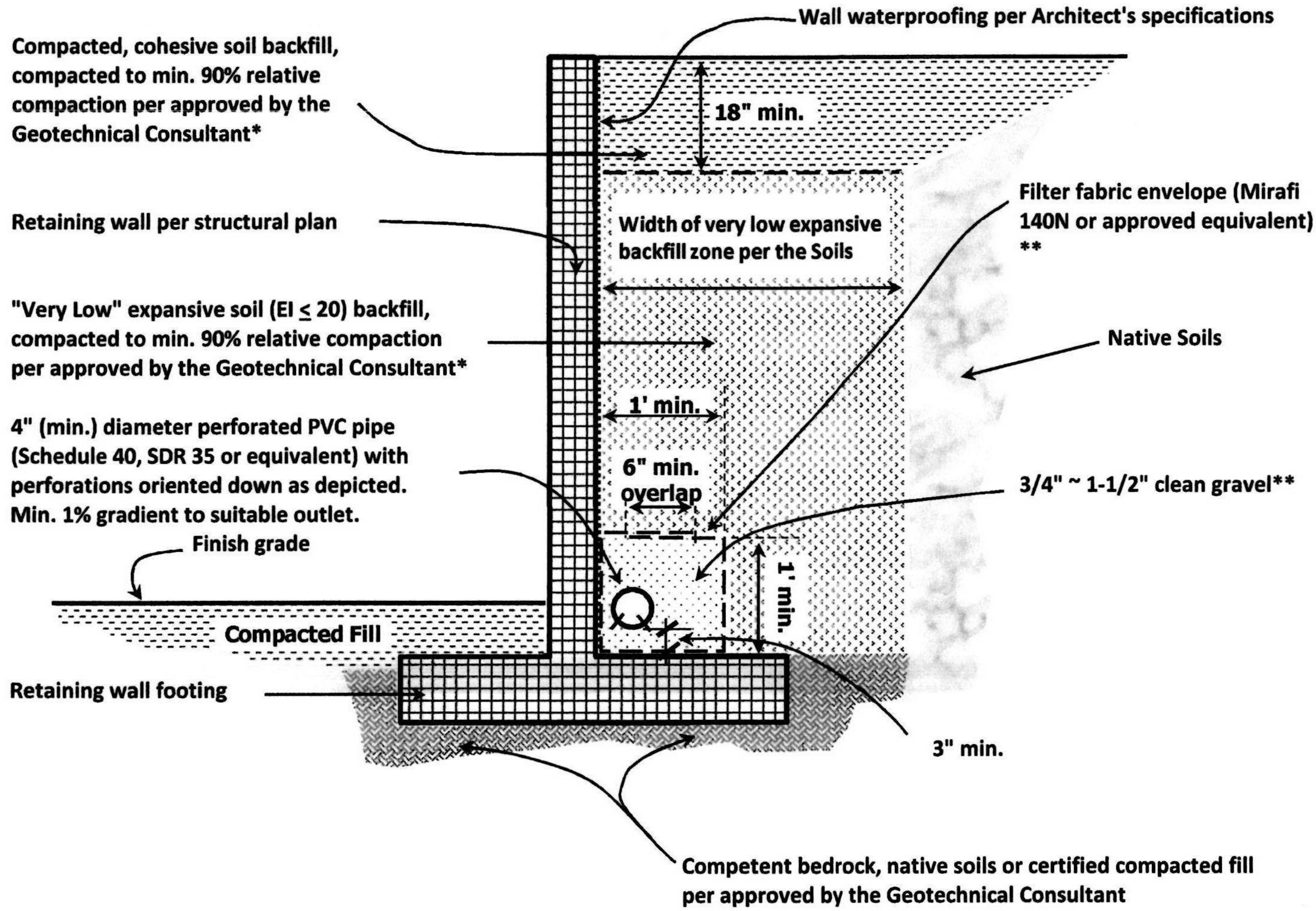
Preferably, the backfill should consist of approved very low-expansive material (i.e. EI not greater than 20) and should be compacted to a minimum relative compaction of 90 percent. The width of the very low-expansive backfill zone should be a minimum of one (1) foot measured from the rear side of the stem of the retaining wall, or the space between the rear side of the stem and the heel of the retaining wall, or one-half (1/2) of the retained height of the retaining wall, whichever is greater. Flooding or jetting of backfill should not be permitted. Granular backfill should be capped with 18 inches (minimum) of relatively impervious fill to seal the backfill and prevent saturation. Figure 6, Retaining Wall Drainage Details, illustrates the general configuration and requirements for retaining wall drainage. Should any conflict noticed between recommendations stated in this report and those shown in Figure 6, the fore should govern. Other retaining wall drainage alternatives may be considered but should first be reviewed and approved by the Geotechnical Consultant prior to implementation.

Should the space behind the new retaining wall be too tight to implement the above recommended backfill effort, as an alternative, 1.5-sack control density fill may be used in lieu of regular soil backfill, provided that the integrity and functionality of wall backdrain is protected and maintained.

It should be noted that the use of heavy compaction equipment in close proximity to retaining structures can result in wall pressures exceeding design values and corresponding wall movement greater than that normally associated with the development of active or at-rest conditions. In this regard, the contractor should take appropriate precautions during the backfill placement.

5.3.3 Footing/Foundation Observation:

All footing/foundation excavations should be observed by the Geotechnical Consultant's representative to verify minimum embedment depths and competency of bearing soils. Such observations should be made prior to placement of any reinforcing steel or concrete.



SPECIFICATIONS FOR CALTRANS CLASS 2 PERMEABLE MATERIAL	
U.S. STANDARD SIEVE SIZE	% PASSING
1"	100
3/4"	90 ~ 100
3/8"	40 ~ 100
No. 4	25 ~ 40
No. 8	18 ~ 33
No. 30	5 ~ 15
No. 50	0 ~ 7
No. 200	0 ~ 3
Sand Equivalent > 75	

* Based on ASTM D-1557-02

** If Caltrans Class 2 permeable material (see gradation to left) is used in place of 3/4" ~ 1-1/2" gravel, filter fabric may be deleted. Caltrans Class 2 permeable material should be compacted to minimum 90 percent relative compaction. Unless otherwise specified, a minimum of 1 cubic foot of gravel should be used for each 1 foot run of drain.

Note: Composite drainage products such as Contech C-Drain, Miradrain or J-Drain may be used as alternative to gravel or Class II. Installation should be performed in accordance with manufacturer's specifications.

Schematic Not To Scale

Associated Soils Engineering, Inc.
2860 Walnut Avenue
Signal Hill, CA 90755
Tel (562) 426-7990 Fax (562) 426-1842

Project:	Proposed Multi-Use/Classroom Additions at Wintersburg Presybyterian Church, 2000 N. Fairview St., Santa Ana, CA
Figure 6	Retaining Wall Drainage Details
Proj. No.: 10-6212	Date: June, 2010

5.4 EXPANSIVE SOILS

Laboratory testing results on a near surface soil sample indicates a "Very Low" soil expansion potential (i.e. EI = 2) as defined in 2007 CBC. It is ASE's opinion that the very low soil expansion potential is not anticipated to be of significant structural concern. Nonetheless, it may be desirable that the soil expansion potential be re-evaluated through additional testing during or after rough grading operations to verify the design adequacy of foundation against the re-tested soil expansion potential as heterogeneity within soil mass is not uncommon.

5.5 SLABS-ON-GRADE

Concrete floor slabs in buildings and exterior concrete flatwork should be supported on properly compacted soils as recommended in the Site Grading section (i.e. Section 5.2) of this report. The slab subgrade soils should also be proof-rolled just prior to construction to provide a firm, unyielding surface, especially if the subgrade has been disturbed or loosened by the passage of construction traffic. Final compaction and testing of slab subgrade should be performed just prior to placement of concrete.

For structural design of concrete slabs, a coefficient of subgrade reaction ("k") on the order of 180 pounds per square inch per inch (psi/in) may be used. Interior and exterior slabs should be properly designed and reinforced for the construction and service loading conditions. To minimize slab distress due to seismically-induced ground movement, geotechnically, it would be prudent to provide a minimum actual slab thickness of four (4) inches with minimum reinforcement consisting of No. 3 reinforcing bars spaced at 24 inches on centers each way, or 6 x 6 W1.4 x W1.4 welded wire mesh placed at mid-slab, or equivalent. The structural details, such as slab thickness, concrete strength, amount and type of reinforcements, joint spacing, etc., should be established by the Structural Consultant in accordance with pertinent sections in 2007 CBC.

The interior slabs should be underlain by an impermeable membrane (minimum 8-mil-thick visqueen) topped with two inches of clean, coarse sand (i.e. ASTM C33 concrete sand). For slab areas that are more sensitive to moisture migration, a minimum 2-inch-thick layer of free-draining coarse sand functioning as capillary break should further underlie the visqueen. The capillary break materials should meet the following specifications:

Sieve Size	Percent Passing
1/2-inch	100
No. 16	50 – 85
No. 200	<15
Sand Equivalent	<50

5.6 ASPHALTIC CONCRETE (AC) FLEXURAL PAVEMENT DESIGN

The finish grade at the subject site is anticipated to be underlain by compacted structural fill consisting of site soils. For preliminary pavement design purposes, an R-Value of 40 has been assumed considering the site soils as subgrade soils. Three (3) traffic indices ("TI") of 4.5, 5.5 and 7.0, together with the assumed R-Value, have been utilized for the development of preliminary recommendations for the pavement sections. Analyses performed in accordance with the current edition of the Caltrans Highway Design Manual, and assuming compliance with site preparation recommendations, it is recommended that the following AC pavement structural sections be used:

Traffic Index (TI)	***Pavement Section Alternatives***		***Remark***
	AC(1) (inches)	***AB(2) (inches)***	
4.5	3.0	4.0	For auto parking stalls.
5.5	3.0	5.5	For auto circulation aisles.
7.0	4.0	7.0	For fire lanes and truck access ways/entry and exits.

(1) Asphalt Concrete;

(2) CAB or CMB, Green Book sections 200-2.2 and 200-2.4, respectively, compacted to at least 95% relative compaction.

Please be reminded that the above preliminary pavement section recommendations have been established based purely on procedures stipulated in Caltrans Manual.

Local government authority should be consulted for minimum pavement section requirements and, if more stringent than that recommended by ASE, be complied with.

It is recommended that R-Value testing be performed on representative soil samples after rough grading operations on the upper 2 feet to confirm/modify applicability of the above pavement sections.

The aggregate base should conform to the Crushed Aggregate Base ("CAB") or Crushed Miscellaneous Base ("CMB") per Sections 200-2.2 and 200-2.4 of the Green Book requirements, respectively. The base course should be compacted to a minimum relative compaction of 95% at 1 to 3 percentage points above the optimum moisture content. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

The asphalt concrete pavement should be compacted to 95% of the unit weight as tested in accordance with the Hveem procedure. The asphalt concrete material shall conform to Type III, Class C2 or C3, of the Green Book. All subgrade and aggregate base materials should be proof-rolled by heavy rubber tire equipment to verify that the subgrade and base grade are in a non-yielding condition.

If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

5.7 PORTLAND CEMENT CONCRETE PAVEMENTS

The following concrete pavement sections are based on load safety factors of 1.0 and 1.1, and a modulus of subgrade reaction ("k" Value) of 180 pounds per cubic inch for site soils compacted as subgrade material, and the design procedures presented in the Portland Cement Association bulletin "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984. A design service life of 20 years was assumed for the design of the Portland cement concrete pavement section.

Concrete Flexural Strength, psi (1)	Pavement Thickness, Inches (2), (4)	Pavement Thickness, Inches (3), (4)
600	5.5	6.5
650	5.0	6.0

(1) Represents 90-day flexural strength.
(2) Load Safety Factor = 1.0 (Auto Parking Stalls)
(3) Load Safety Factor = 1.1 (Truck Traffic Areas/Entry and Exits)
(4) Assumes no PCC shoulder or curb.

The Structural Consultant should establish the design details of the concrete pavement section, including reinforcements, concrete strength, and joint and load transfer requirements.

It is recommended that edges of concrete pavements which are not adjacent to existing buildings, or are adjacent to planter areas, be downturned a minimum of 12 inches or be constructed with curbing to prevent water infiltration to subgrade soils. If edges are downturned or curbing is constructed, the above pavement thicknesses should be decreased by one inch.

The upper one-foot of exposed subgrade soils beneath concrete pavements should be further compacted to a minimum 95 percent relative compaction at 1 to 3 percentage points above optimum moisture contents. Subgrade soils should exhibit a firm, unyielding surface in addition to the recommended compaction. Final compaction and testing of pavement subgrade should be performed just prior to placement of aggregate base and/or concreting. Other pertinent subgrade preparation measures stipulated in the "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984, or required by the jurisdictional municipal authorities should be followed accordingly.

5.8 SITE DRAINAGE

Per Section 1803.3 of 2007 CBC, a minimum 5% descending gradient away from the building for a minimum distance of 10 feet should be incorporated for earth grade placed adjacent to the foundation. This descending gradient may be reduced to 2% for any impervious areas, such as concrete paved walkways, within the 10-foot zone. For areas where the 10-foot drainage distance is not attainable, alternative measure such as concrete-lined swales having a minimum 2% gradient may be adopted to divert the water away from the building, provided that the minimum 5% gradient is maintained in the distance between the building footprint and the diversion measure such as swales. For more specific site drainage guidelines, the Project Civil Consultant should refer to the pertinent sections in 2007 CBC.

Any planter areas to be placed adjacent to structure perimeters should be provided with solid bottoms and a drainage pipe, to divert water away from foundation and slab subgrade soils. Excessive moisture variations in site soils could result in significant volume changes and movement.

5.9 SOIL CORROSIVITY EVALUATION

Soils corrosivity tests were performed on representative samples of site soil submitted to Quartech Consultants, Inc. (QCI Job No. 10-064-06c, June 11, 2010). These tests were meant to determine the corrosive potential of on-site soils to proposed concrete foundations and underground metal conduits. The soils corrosivity test results are presented on the attached Plate H in Appendix A.

5.9.1 Concrete Corrosion:

Disintegration of concrete may be attributed to the chemical reaction of soil sulfates and hydrated lime and calcium aluminate within the cement. The severity of the reaction resulting in expansion and disruption of the cement is primarily a function of the concentration of the soluble sulfates and the water-cement ratio of the concrete.

A soluble sulfate content of 0.0480% by weight has been recorded from corrosivity testing conducted on on-site soils, as indicated in Plate H in Appendix A. Per Table 4.2.1 of ACI 318-08, soils exhibiting soluble sulfate content less than 0.1% by weight are classified as having "Not Applicable" sulfate exposure and "SO" sulfate exposure category. As such, for structural features to be in direct contact with on-site soils, the tested "SO" sulfate exposure category indicates that there should be no special geotechnical restriction on the type of Portland cement or water-cement ratio to be used, as per stipulated in Table 4.3.1 of ACI 318-08.

5.9.2 Metal Corrosion:

In the evaluation of soil corrosivity to metal, the hydrogen ion concentrates (pH) and the electrical resistivity of the site and backfill soils are the principal variables in determining the service life of ferrous metal conduit. The pH of soil and water is a measure of acidity or alkalinity, while the resistivity is a measure of the soils resistance to the flow of electrical current.

Currently available design charts indicate that corrosion rates decrease with increasing resistivities and increasing alkalinities. It can also be noted that for alkaline soils, the corrosion rate is more influenced by resistivity than by pH.

The resistivity value of 1,400 ohm-cm, as well as a pH-value of 7.22, classify the on-site soils tested to be severely corrosive to buried ferrous metals. Based on California Test 643, the year to perforation for 18-gauge steel in contact with soils of similar resistivity and pH-value is approximately <u>24</u> years for the severely corrosive on-site soils. In lieu of additional testing, alternative piping materials, i.e. plastic piping, may be used instead of metal if longer service life is desired or required.

A soluble chloride content of 72 ppm recorded in our limited laboratory test is considered low to the threshold values of 100 and 200 ppm per Federal Highway Administration Standards (FHWA), 2002 and Caltrans Standards, 1999, respectively. Therefore, <u>no</u> special measures in terms of rebar protection against chloride corrosion will be required as a result of the low soluble chloride content tested. The compliance with the corrosivity criteria stipulated in Section 5.2.6 will ensure that no other particular reinforcement protection measure will be needed for foundations and structural elements in contact with fill materials.

5.10 UTILITY TRENCHES

All trenches should be backfilled with approved fill material compacted to relative compaction of not less than 90 percent of maximum dry density. Care should be taken during backfilling to prevent utility line damage.

The on-site soils may be used for backfilling utility trenches from one foot above the top of pipe to the surface, provided the material is free of organic matter and deleterious substances. Any soft and/or loose materials or fill encountered at pipe invert should be removed and replaced with properly compacted fill or adequate bedding material.

On-site soils may be suitable for bedding or shading of utilities. Site and imported soils for pipe bedding should consist of non-expansive granular soils. Bedding materials should consist of sand with a Sand Equivalent value (California Test Method 217) not less than 30.

If sandy soils are used for trench backfill, the backfill should be topped with a minimum 2-foot thick cap of compacted fine-grained soil. Also, a minimum 10-foot length of trench at the entrance and exist points of structures should be backfilled with fine-grained soils to serve as a plug to prevent water migration into structure foundation support zones.

The walls of temporary construction trenches may not be stable when excavated nearly vertical due to potential for caving. Shoring of excavation walls or flattening of slopes will be required if excavation depths greater than 4 feet are necessary.

Trenches should be located so as not to impair the bearing capacity of soils or cause settlement under foundations. As a guide, trenches parallel to foundations should be clear of a 45-degree plane extending outward and downward from the edge of the foundations.

All work associated with trenches, excavations and shoring must conform to the State of California Safety Code.

5.11 FIELD PERCOLATION TEST DATA

Initial seepage rates obtained during the "Sand Soil Criteria Test" in Boring B-3 after overnight pre-soaking did qualify on-site soils to be "Sandy Soils". Percolation tests were therefore performed using the "Sandy" method (i.e. one hour test) in accordance with County of Orange Department of Health procedures modified to test the cross sectional zone of typical soils within the level of anticipated groundwater infiltration (e.g. approximately 4 inches to 5 feet below existing grade).

Field percolation tests were conducted on June 4, 2010. Linear plot of the field percolation test data indicates a percolation test rate of approximately 0.207 minutes per inch (mpi) for clean water. The minimum acceptable percolation rate for design of leach field type drainage systems for sewage water as per the Uniform Plumbing Code, 1985 Edition, is 60 mpi. Field percolation test data is presented on the attached Plate J.

Tabulated below are the results of percolation testing conducted at the location of Boring B-3. Also included is the percolation rate for clean water and sewage water presented in gallons per square foot per day as determined from the percolation test rate results.

		Percolation Rates (Gal/Sq. Ft./Day)	
Boring No.	**Percolation Test Rate (Minutes/Inch)**	**Clean Water**	**Sewage * Water**
B-3	0.207	867.2	216.8

* A factor of 4.0 has been applied in deriving the sewage water percolation rate from the corresponding clean water percolation rate. The Civil Consultant may adopt a more stringent factor if deemed appropriate per the project performance requirement.

The clear water rate presented above is anticipated to be the fastest rate that can be absorbed by the site soils at the boring location. However, with time and depending on the degree of saturation of soils and other factors, the percolation rate may reduce to the slowest tabulated rate which is typical for sewage disposal fields.

a) Construction Notes:

- The degree of compactive effort in the upper 1 to 1.5 feet of soils above any filter material should be sufficient to obtain a minimum 90 percent relative compaction. As any greater compactive efforts in the soil strata of water retention system construction may cause the percolation rates to reduce substantially, it is not advisable to impose significant structural loading in these areas, from a geotechnical viewpoint.
- The rate of water transmission from the filter material to the soil will be limited the porosity characteristics of the fabric wrap around the filter material.

5.12 PLAN REVIEW, OBSERVATIONS AND TESTING

All excavations should be observed by a representative of this office to verify minimum embedment depths, competency of bearing soils and that the excavations are free of loose and disturbed materials. Such observations should be made prior to placement of any fill, reinforcing steel or concrete. All grading and fill compaction should be performed under the observation of and testing by a Geotechnical Consultant or his representative.

As foundation and grading plans are completed, they should be forwarded to the Geotechnical Consultant for review of conformance with the intent of these recommendations.

6.0 CLOSURE

This report has been prepared for the exclusive use of Wintersburg Presbyterian Church and their design consultants for use in design and construction of the proposed Multi-use/Classroom Additions to the existing church complex. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties.

The Owner or their representatives are responsible for ensuring the information and recommendations contained in this report are brought to the attention of the project engineers and architects, incorporated into the project plans, and implemented by project contractors. This report should be named on project grading plans as a part of the project specifications.

We request and recommend notification should any of the following occur:

1. Final plans for site development indicate utilization of areas not originally proposed for construction.
2. Structural loading conditions vary from those utilized for evaluation and preparation of this report.
3. The site is not developed within 12 months following the date of this report.

If changes or delays do occur, this office should be notified and provided with finalized plans of site development for our review to enable us to provide the necessary recommendations for additional work and/or updating of the report. Any charges for such review and necessary recommendations would be at the prevailing rate at the time of performing review work.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions existing across the area of study are similar to those encountered in the test excavations. However, no warranty is expressed or implied as to the conditions at locations or depths other than those excavated. Should conditions encountered during construction differ significantly from those described in this report, this office should be contacted immediately for recommendations prior to continuation of work.

Our findings and recommendations were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment. We make no other warranty, either express or implied.

These recommendations are, however, dependent on the aforementioned assumption of uniformity and upon proper quality control of engineered fill and foundations. Geotechnical observations and testing should be provided on a continuous basis during grading at the site to confirm preliminary design assumptions and to verify conformance with the intent of our recommendations. If parties other than Associated Soils Engineering, Inc., are engaged to provide geotechnical services during construction, they must be informed that they will be required to assume complete responsibility for the geotechnical phase of the project by either concurring with the recommendations in this report or providing alternative recommendations.

This concludes our scope of services as indicated in our proposal dated May 18, 2010, however, our report is subject to review by the controlling authorities for the project. Any further geotechnical services that may be required of our office to respond to questions/comments of the controlling authorities after their review of the report will be performed on a time-and-expense basis as per our current fee schedule. We would not proceed with any response to report review comments/questions without authorization from your office.

We appreciate your business and are prepared to assist you with construction-related services.

APPENDIX-A

The following Appendix contains the substantiating data and laboratory test results to complement the engineering evaluations and recommendations contained in the report.

Plate A	Boring Location Plan
Plates B-1 through B-3	Logs of Borings
Plates C-1 through C-5	Consolidation Test Results
Plates D-1 through D-3	Direct Shear Test Results
Plates G-1 through G-8	Particle Size Test Results
Plate H	Results of Soil Corrosivity Tests
Plate J	Field Percolation Test Results

SITE EXPLORATION

On June 3, 2010, field explorations were performed by drilling test borings at the approximate three (3) locations indicated on the attached Boring Location Plan, Plate A. The exploratory borings were drilled by Choice Drilling, Inc., utilizing a truck mounted, rotary drilling rig equipped with 8-inch diameter continuous flight, hollow-stem rotary augers. The borings extended to depths of 5 feet 7 inches to 51 feet 6 inches from the existing grades.

Continuous observations of the materials encountered in the borings were recorded in the field. The soils were classified in the field by visual and textural examination and these classifications were supplemented by obtaining bulk soil samples for future examination in the laboratory. Relatively undisturbed samples of soils were extracted in a Modified California barrel sampler lined with 2.375-inch diameter by one-inch high rings and tipped with tapered cutting shoe. Additional samples were obtained in a Standard Penetration sampler in accordance with specifications outlined in ASTM D1586. All samples were secured in moisture-resistant bags as soon as taken to minimize the loss of field moisture while being transported to the laboratory and awaiting testing.

Upon completion of exploration, the borings were backfilled with excavated materials and compacted by tamping, with existing pavement patched with cold patch asphalt.

Description of the soils encountered, depth of samples, field density and moisture content of tested samples, respective laboratory tests performed, as well as Standard Penetration Test ("N" Values) and Modified California barrel sampler blow counts are presented in the attached Logs of Borings.

LABORATORY TESTS

After samples were visually classified in the laboratory, a testing program that would provide sufficient data for our evaluation was established.

MOISTURE CONTENT AND DENSITY TESTS

The undisturbed soil retained within the rings of the Modified California barrel sampler was tested in the laboratory to determine in-place dry density and moisture content. Test results are presented on the Logs of Borings (see attached "B" Plates).

CONSOLIDATION AND DIRECT SHEAR TESTS

Consolidation (ASTM D-2435) and direct shear (ASTM D-3080) tests were performed on selected relatively undisturbed and remolded samples to determine the settlement characteristics and shear strength parameters of various soil samples, respectively. The results of these tests are shown graphically on the appended “C” and “D” Plates.

GRAIN SIZE ANALYSIS

Grain size analysis tests were performed in accordance with ASTM D422 test specifications on selected soils to determine the particle size distribution of various soils sampled in the boring analyzed for site liquefaction potential. The results of these tests are shown graphically on the attached "G" Plates.

SOIL CORROSIVITY

Tests of soluble sulfate and chloride contents were performed in accordance with California Test Methods 417 and 422, respectively, to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. Resistivity and pH-value tests were performed in accordance with California Test Method 643 to assess the degree of corrosivity of the subgrade soils with regard to ferrous metal piping. The test results are shown on Plate H.

MAXIMUM DENSITY TEST

A maximum density test was conducted in accordance with ASTM D1557-07, Method A, using 5 equal layers, 25 blows each layer, 10-pound hammer, 18 inch drop in a 1/30 cubic foot mold. The results are as follows:

BORING NO.	DEPTH, FEET	MAXIMUM DRY DENSITY, PCF	OPTIMUM MOISTURE CONTENT, %	MATERIAL CLASSIFICATION
B-1	0.75-5	127.0	10.0	SM

EXPANSION TEST

An expansion test was performed on a soil sample to determine the swell characteristics. The expansion test was conducted in accordance with ASTM D4829-03 test procedures. The expansion sample was remolded to approximately 90 percent relative compaction at near optimum moisture content subjected to 144 pounds per square foot surcharge load and was saturated.

LOCATION	MOLDED DRY DENSITY, PCF	MOLDED MOIST. CONTENT, %	% SATURATION	EXPANSION INDEX	EXPANSION CLASSIFICATION
Boring B-1 @ 0.75'-5'	115.5	9.9	54.0	2	Very Low

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-1

Project:	*Proposed Multi-use/Classroom Additions-Santa Ana*
Location:	*2000 N. Fairview Street*
Project No.	*10-6212*

Dates(s) Drilled:	**6/3/2010**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling Incorporated**	Total Depth:	**35 Feet**
Rig Make/Model:	**Mobile Drill B-61**	Hammer Type:	**Automatic**
Drilling Method:	**Hollw-stem Auger**	Hammer Weight/Drop:	**140 Lb./30 Inches**
Hole Diameter:	**8 Inches**	Surface Elevation:	**Unknown**

Comments: Groundwater encountered at 29' 5". Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0						ASPHALTIC CONCRETE PAVEMENT: 3.0"				
						SM	AGGREGATE BASE MATERIAL: 4.5" (resembles remnant AC layer)				MAX DENSITY EXPANSION REMOLD SHEAR CORROSIVITY TESTS
				13(Ring)			SILTY SAND: Light olive brown to light olive gray,trace yellowish brown,moist,fine-grained sand	94.3	9.3		CONSOL,SHEAR @ 2 feet
5	5			10(Ring)		SM	same as above,becoming pale olive with trace yellowish brown	95.1	14.3		
						SM	SILTY SAND: Grayish brown,trace yellowish brown,very moist,fine-grained sand				
10	10			6/6"(Ring)				81.1	23.9		
				7/6"(Ring)		SP	SAND: Light gray with dark gray,damp,fine-grained sand,trace gravel	93.6	5.5		CONSOL
						SP	SAND: Light gray to pale yellow,damp,fine to medium-grained sand				
15	15			20(Ring)				99.1	4.2		CONSOL

SOILS ENGINEERING, INC.

FIELD LOG OF BORING B - 1

Project: ***Proposed Multi-use/Classroom Additions-Santa Ana***

Location: ***2000 N. Fairview Street*** | Project No. ***10-6212***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
				CL	SILTY CLAY: Dark grayish brown with yellowish brown,moist to very moist				
20	20	15(Ring)				103.6	22.7		
				SP-SM	SAND WITH SILT: Gray,wet,fine to medium-grained sand				
25	25	37(Ring)				110.6	18.3		
				CL	SILTY CLAY: Dark gray,trace light olive brown,very moist to wet				
30	30	5/6"(Ring)				93.1	31.9		
		7/6"(Ring)		ML	CLAYEY SILT: Dark gray,trace olive yellow,very moist	100.3	25.9		
		6/6"(Ring)		ML	same as above,wet	88.8	31.5		
35	35	9/6"(Ring)		CL	SILTY CLAY: Very dark gray,wet	83.7	37.6		

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-2

Project:	***Proposed Multi-use/Classroom Additions-Santa Ana***
Location:	***2000 N. Fairview Street***
Project No.	***10-6212***

Dates(s) Drilled:	**6/3/2010**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling Incorporated**	Total Depth:	**51 Feet 6 Inches**
Rig Make/Model:	**Mobile Drill B-61**	Hammer Type:	**Automatic**
Drilling Method:	**Hollw-stem Auger**	Hammer Weight/Drop:	**140 Lb./30 Inches**
Hole Diameter:	**8 Inches**	Surface Elevation:	**Unknown**

Comments: Groundwater encountered at 22' 3". Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			ASPHALTIC CONCRETE PAVEMENT: 3.0"				
			SM	AGGREGATE BASE MATERIAL: 6.25"				
		7/6"(Ring)	SM	SILTY SAND: Light olive brown,moist,fine-grained sand,with layer Fine to Medium Sand with Silt (SP-SM)	104.0	9.9		
		8/6"(Ring)	SP		98.0	3.4		
				SILTY SAND: Olive,moist,fine-grained sand				
				SAND: Light yellowish brown,damp,fine to medium-grained sand				
5	5	3/6"(Ring)	SP	same as above,becoming white,dry	91.4	1.5		CONSOL,SHEAR
		5/6"(Ring)	SP	SAND: White,damp,fine to coarse-grained sand *insufficient sample for density	*	2.8		
			SP					
		4(SPT)		SAND: Light yellowish brown,damp,fine to medium-grained sand,trace gravel		4.7		GRAIN SIZE
			SP	SAND: Pale yellow,trace olive yellow,moist,fine-grained sand				
10	10	4/6"(Ring)			97.9	11.4		
		5/6"(Ring)	SP	SAND: Pale olive,damp to moist,fine to medium-grained sand,lens Silty Clay (CL)	95.4	8.1		
			SP	SAND: Pale yellow,damp,fine to medium-grained sand				
		4/6"(Ring)			97.0	5.4		
		5/6"(Ring)	SM	SILTY SAND: Olive gray with light gray,moist,fine-grained sand	92.8	18.5		
			SP	SAND: Olive to pale olive,moist,fine to medium-grained sand,trace gravel +moisture sample from upper 6" of SPT				
15	15	+				12.4		

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B - 2

Project: ***Proposed Multi-use/Classroom Additions-Santa Ana***

Location: ***2000 N. Fairview Street*** | Project No. ***10-6212***

DEPTH (Ft.)	ELEVATION (MSL)	TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
		4(SPT)	CL	SILTY CLAY: Dark olive gray,trace light olive brown,very moist to wet		27.4		GRAIN SIZE
		2/6"(Ring)	CL	same as above,becoming olive brown	100.3	24.2		CONSOL
		4/6"(Ring)	CL	SILTY CLAY: Gray and light olive brown,very moist to wet,with calcareous concretions	96.8	26.9		
20	20	4(SPT)	CL	SANDY CLAY WITH SILT: Olive,very moist,fine to medium-grained sand		20.5		GRAIN SIZE
			SC-SM	CLAYEY SAND WITH SILT: Olive gray,wet,fine to medium-grained sand				
		5/6"(Ring)			111.0	20.0		GRAIN SIZE
		5/6"(Ring)	SP-SM	SAND WITH SILT: Olive gray,wet,fine to medium-grained sand	109.7	17.5		
25	25	16(SPT)	SP	SAND: Olive,with layers yellowish brown,wet,fine to medium-grained sand		19.8		GRAIN SIZE
		17(Ring)	SP	same as above,with lens Silty Clay (CL)	106.7	18.1		
30	30	6(SPT)	CL	SILTY CLAY: Olive gray grading to dark gray,wet,with thin layer Olive with Dark Gray Silty Fine Sand (SM) at 31 feet		33.3		
						20.9 (SM)		
		8(Ring)	CL	same as above,becoming very dark gray with light olive brown	88.6	33.3		
35	35	2/6"(SPT)	CL	same as above,becoming dark gray		35.1		GRAIN SIZE

FIELD LOG OF BORING B - 2

ASSOCIATED ASE SOILS ENGINEERING, INC.

Project: ***Proposed Multi-use/Classroom Additions-Santa Ana***

Location: ***2000 N. Fairview Street*** | Project No. ***10-6212***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
		2/6"(SPT)	CL	SILTY CLAY: Grayish brown with light olive brown,wet		34.2		
		12/6" (Ring)	CL	SILTY CLAY: Dark gray,wet	89.6	32.3		
		13/6" (Ring)	CL	CLAY: Grayish brown with light olive brown,very moist,with thin layer Silty Fine Sand with Clay (SM)	95.7	23.9		GRAIN SIZE
			CL					
40	40	19(SPT)	CL	SILTY CLAY: Dark gray,wet		26.6		
				SILTY CLAY: Olive gray with dark gray and yellowish brown,very moist,with thin layer Light Olive Brown with Olive Silty Fine Sand (SM)		22.5 (SM)		
		60(Ring)	SP	SAND: Olive with light gray,wet,fine to medium-grained sand	105.2	19.1		
45	45	28(SPT)	SP-SM	SAND WITH SILT: Olive with light gray,wet,fine to medium-grained sand,with layer Dark Gray Silty Clay (CL)		20.5 35.5 (CL)		GRAIN SIZE
		67(Ring)	SP-SM	same as above	114.7	14.9		
50	50	15/6" (SPT)	SP-SM	same as above,becoming gray		21.2		
		11/6" (SPT)	CL	SILTY CLAY: Very dark gray,wet		42.7		

SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-3

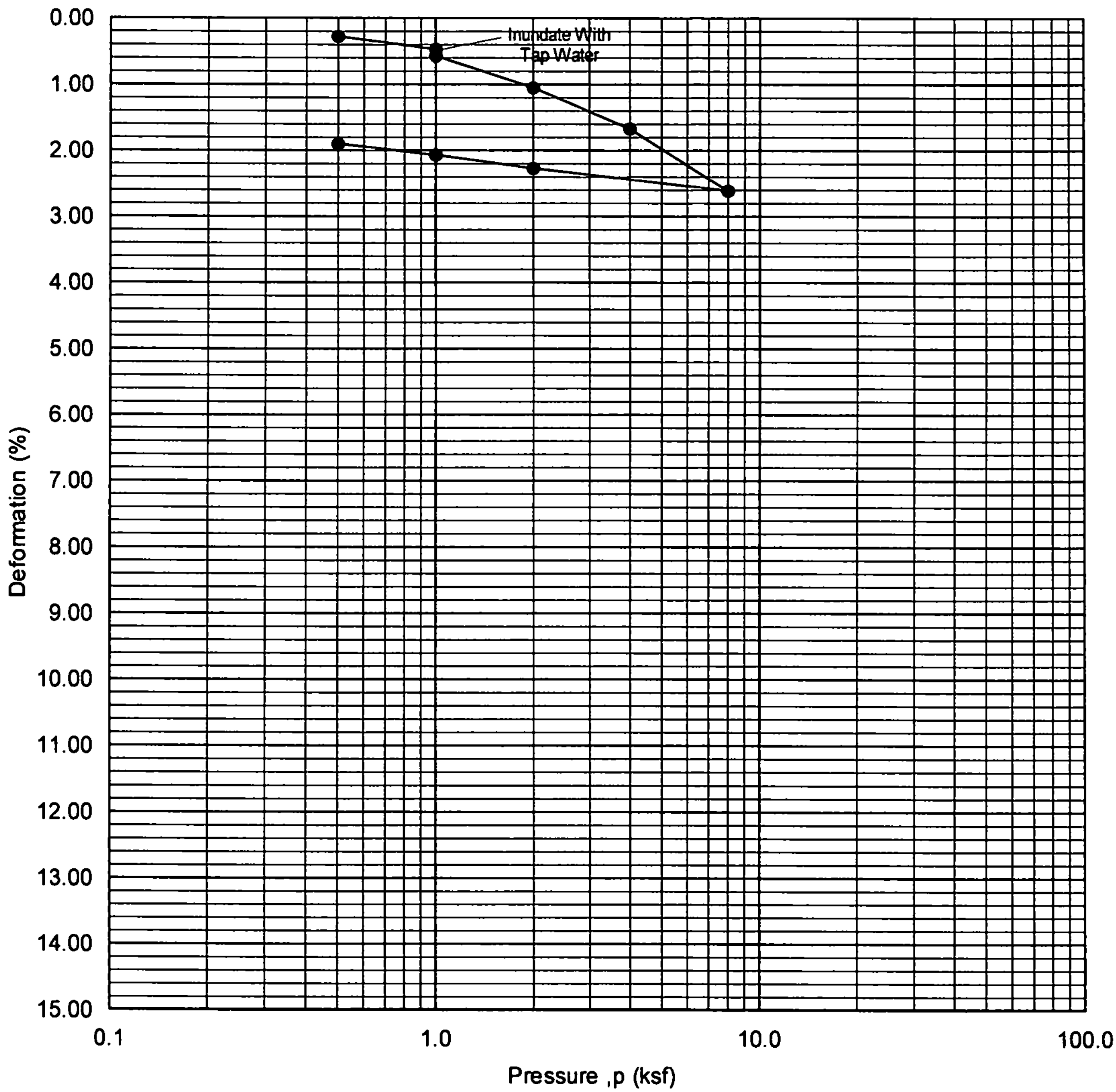
Project:	*Proposed Multi-use/Classroom Additions-Santa Ana*
Location:	*2000 N. Fairview Street*
Project No.	*10-6212*

Dates(s) Drilled:	**6/3/2010**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling Incorporated**	Total Depth:	**5 Feet 7 Inches**
Rig Make/Model:	**Mobile Drill B-61**	Hammer Type:	**Automatic**
Drilling Method:	**Hollw-stem Auger**	Hammer Weight/Drop:	**140 Lb./30 Inches**
Hole Diameter:	**8 Inches**	Surface Elevation:	**Unknown**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0				ASPHALTIC CONCRETE PAVEMENT: 3.0"				
				SM	AGGREGATE BASE MATERIAL: 4.0"				
		13/6" (Ring)			SILTY SAND: Light olive brown to light yellowish brown,damp,fine to medium-grained sand	115.1	3.1		
		33/6" (Ring)		SP	SAND: Pale olive,dry to damp,fine to coarse-grained sand,with scattered gravel *insufficient sample for density	*	2.4		
		16(Ring)		SP	SAND: Pale olive,damp to moist,fine to medium-grained sand,with layers Dark Grayish Brown Silty Clay (CL) and Olive Silty Fine to Medium Sand (SM)	100.4	8.4		
5	5			ML	SILT: Gray with yellowish brown,moist				

NOTE:Boring backfilled with 7 inches of compacted site soils. Five (5) feet length of slotted PVC pipe placed in boring with annular area backfilled with pea gravel. Pea gravel (2.0") placed at bottom of pipe and percolation test performed after overnight presoak.

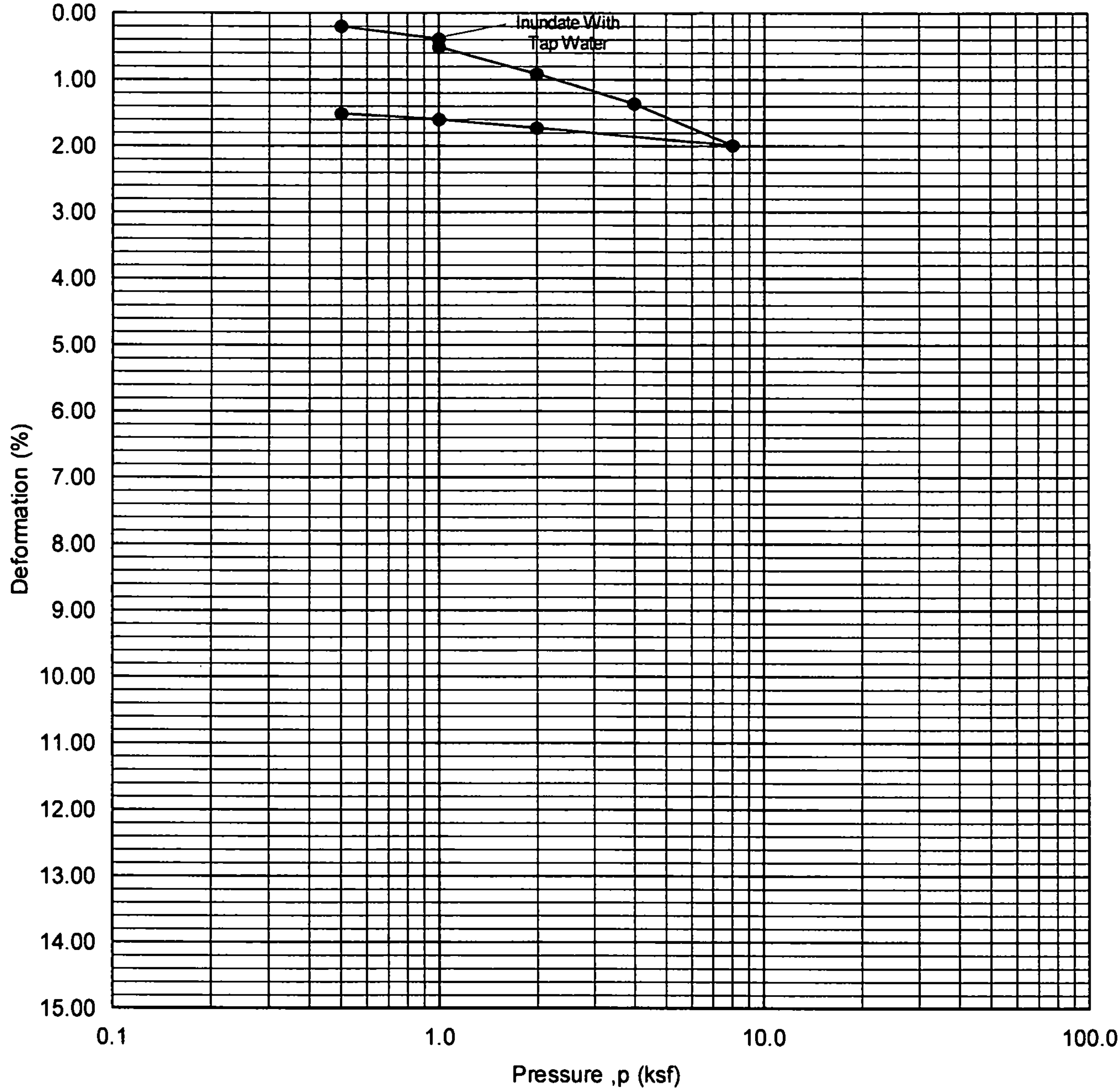


Boring No. : B-1
Depth (ft.) : 2.0
Sample Type: Silty Fine Sand

Dry Density (pcf) = 94.3
Moisture (%) = 9.3

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-1

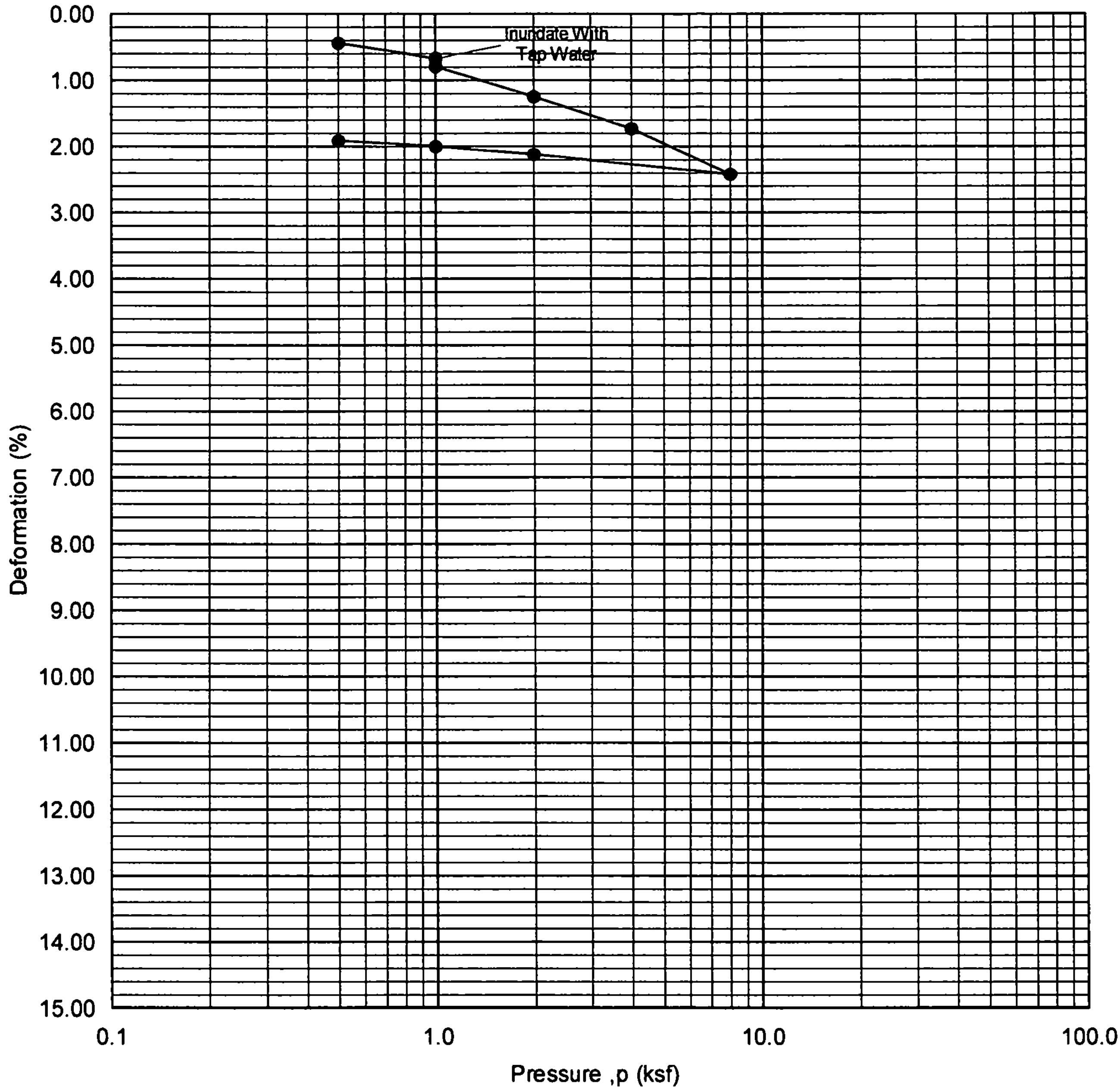


Boring No. : B-1
Depth (ft.) : 10.5
Sample Type: Fine Sand with trace Gravel

Dry Density (pcf) = 93.6
Moisture (%) = 5.5

Project Name: Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-2

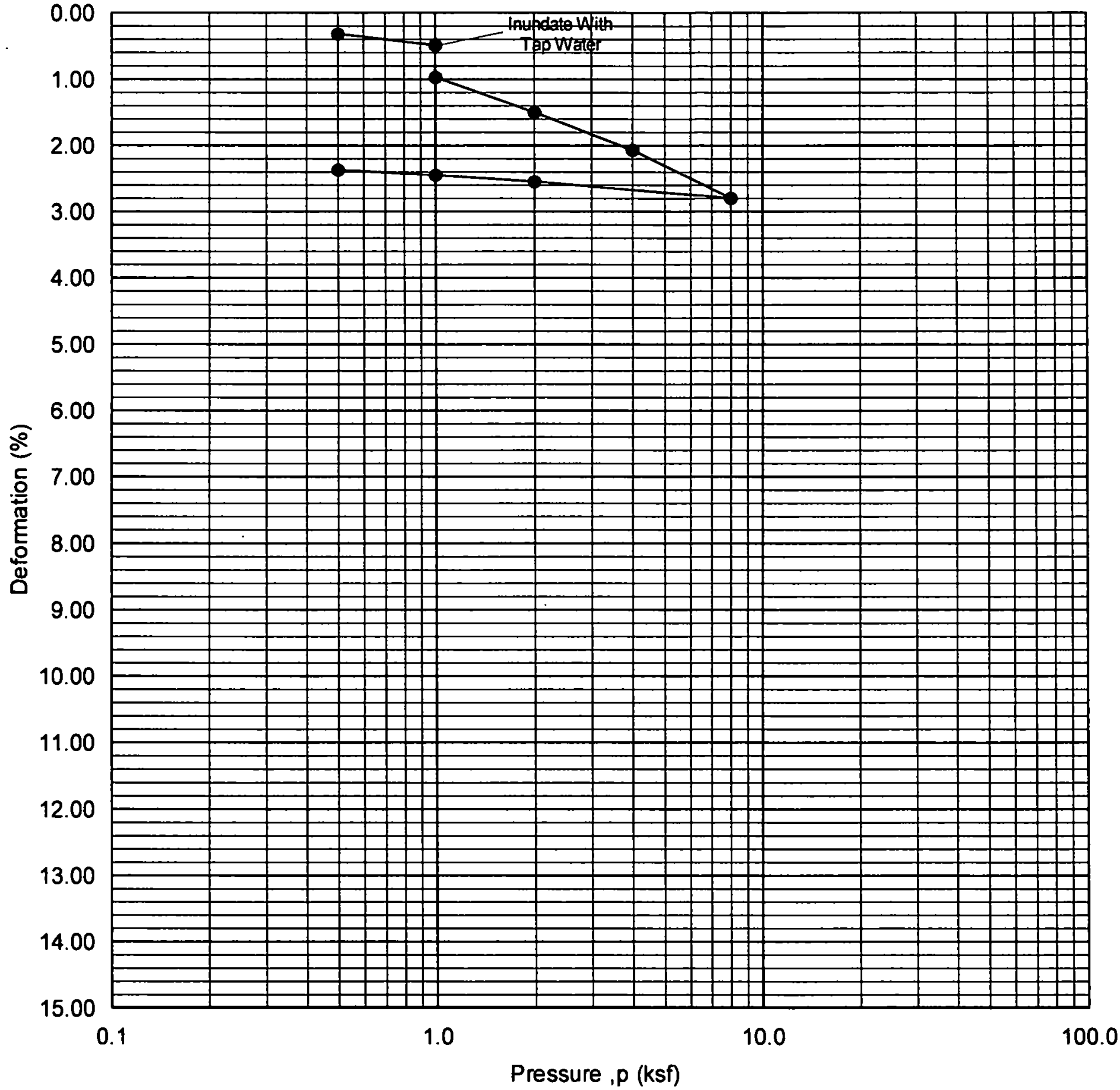


Boring No. : B-1
Depth (ft.) : 15.0
Sample Type: Fine to Medium Sand

Dry Density (pcf) = 99.1
Moisture (%) = 4.2

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-3

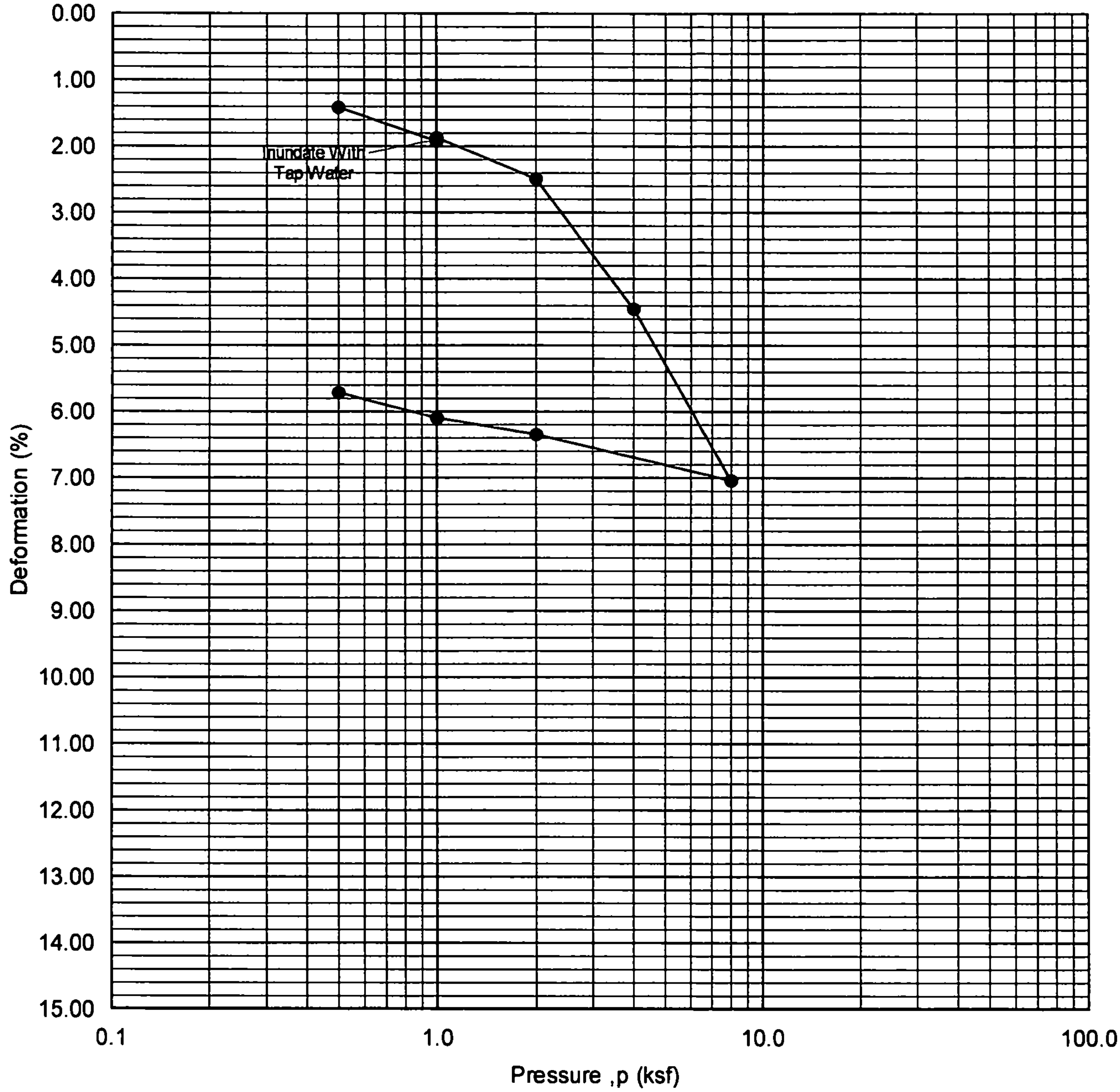


Boring No. : B-2
Depth (ft.) : 5.0
Sample Type: Fine to Medium Sand

Dry Density (pcf) = 91.4
Moisture (%) = 1.5

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-4

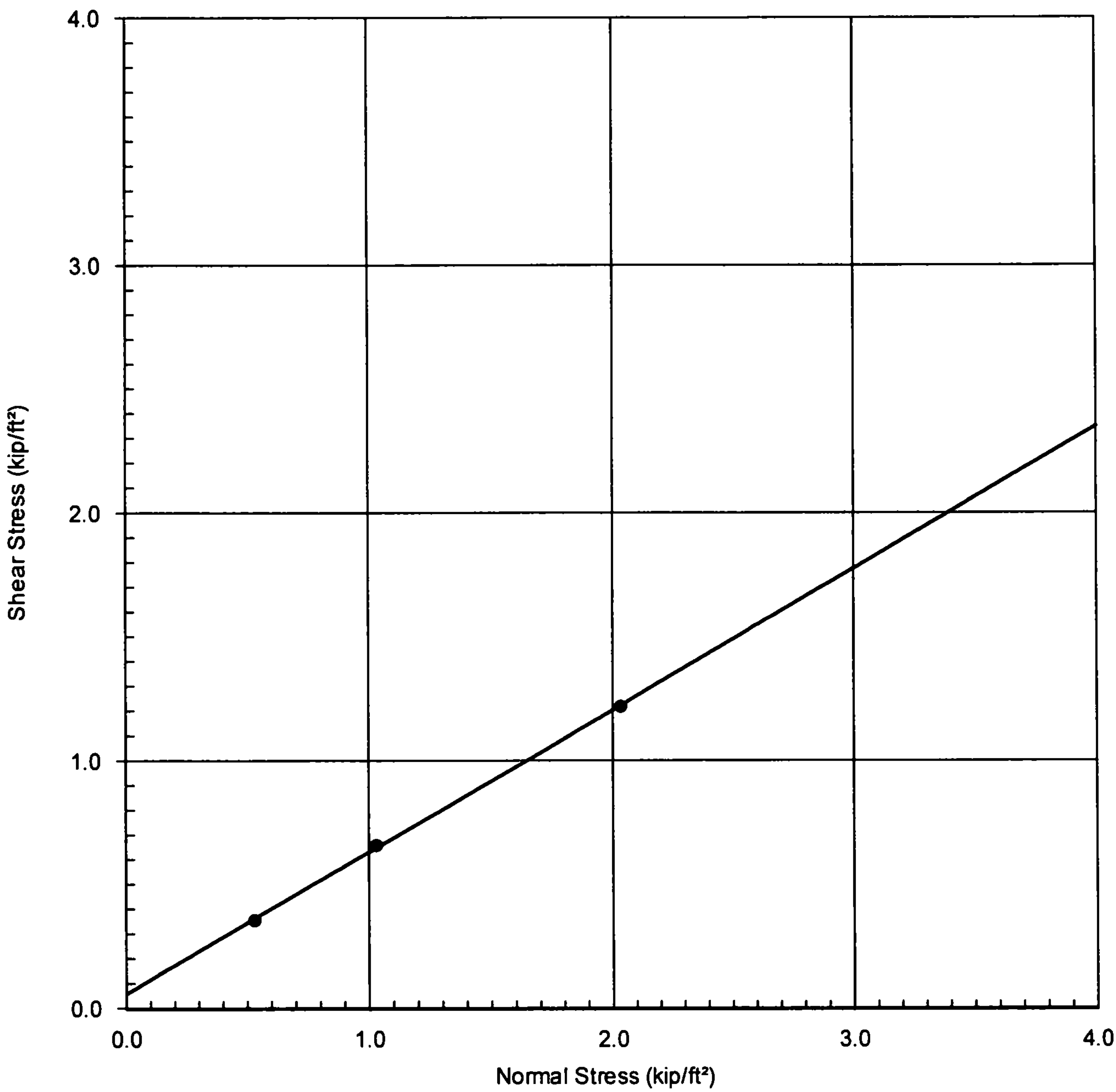


Boring No. : B-2
Depth (ft.) : 18.0
Sample Type: Silty Clay

Dry Density (pcf) = 100.3
Moisture (%) = 24.2

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-5

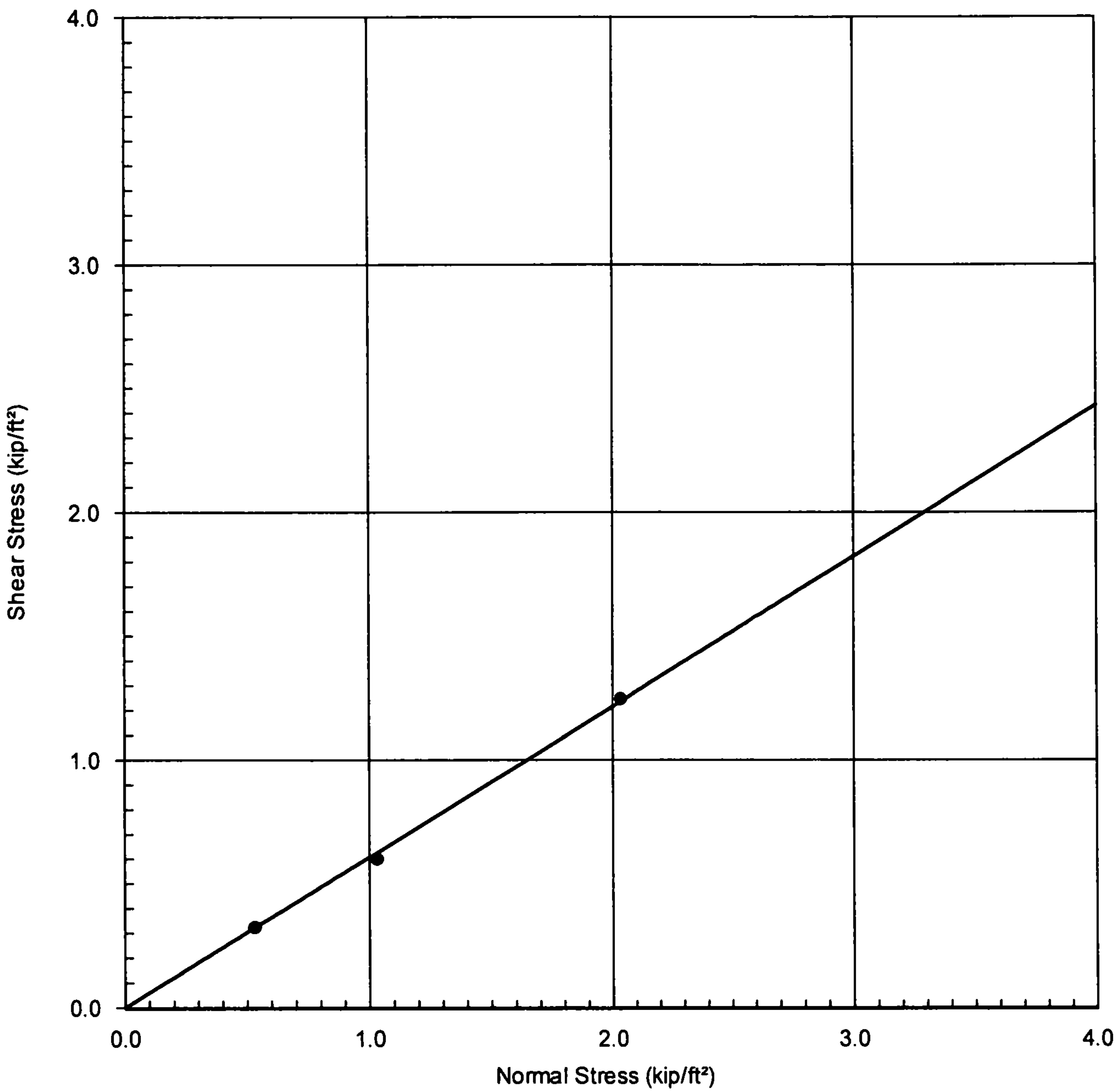


Boring No. : B-1
Depth (ft.) : 2.0
Sample : Relatively Undisturbed
Sample Type : Silty Fine Sand

Cohesion(C) = 55 psf
Friction (ϕ) = 29.5°
Dry Density (pcf) = 94.3
Moisture (%) = 9.3

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-1

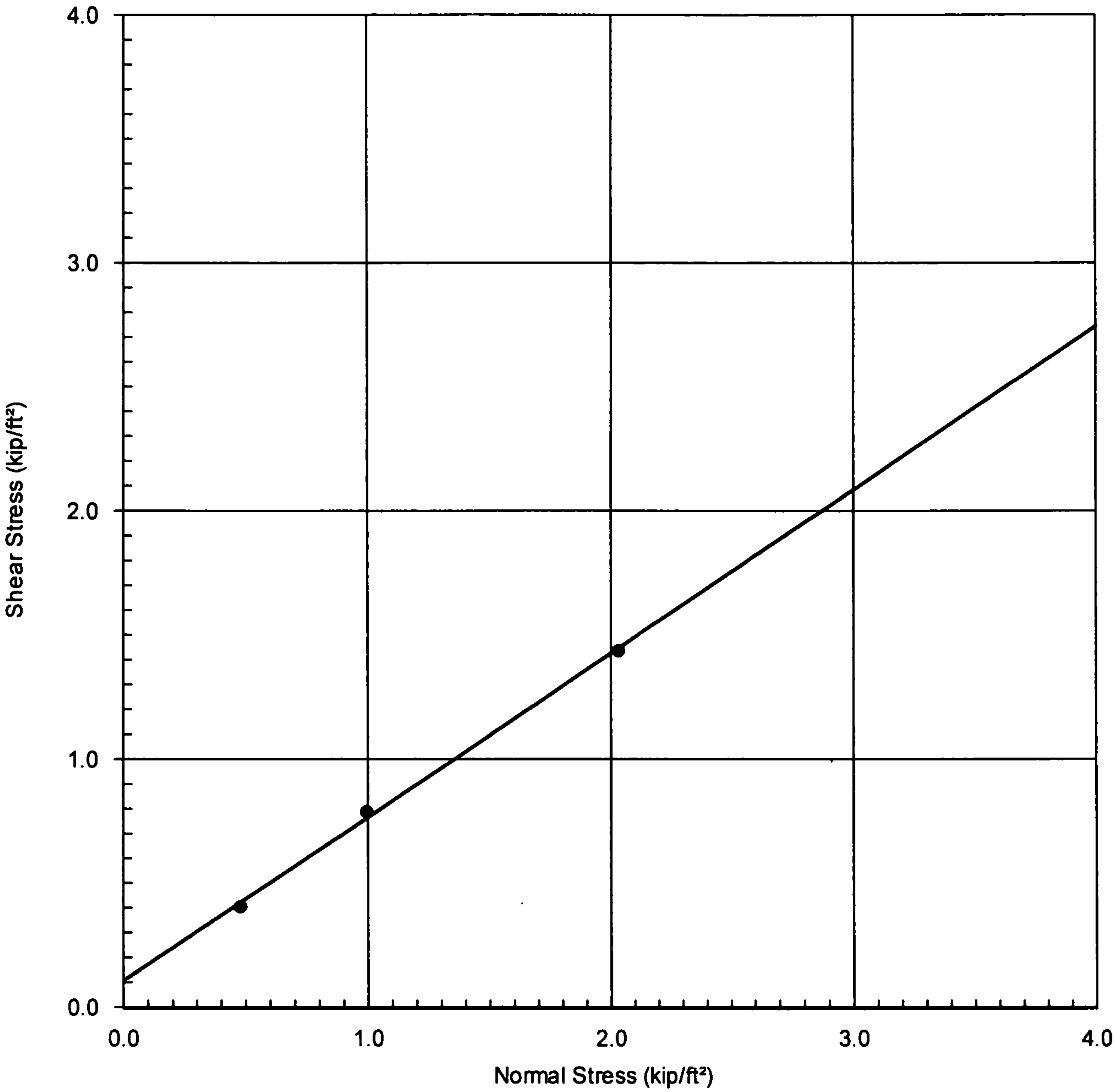


Boring No. : B-2
Depth (ft.) : 5.0
Sample : Relatively Undisturbed
Sample Type : Fine to Medium Sand

Cohesion(C) = 0 psf
Friction (ϕ) = 31.5°
Dry Density (pcf) = 91.4
Moisture (%) = 1.5

Project Name: Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-2

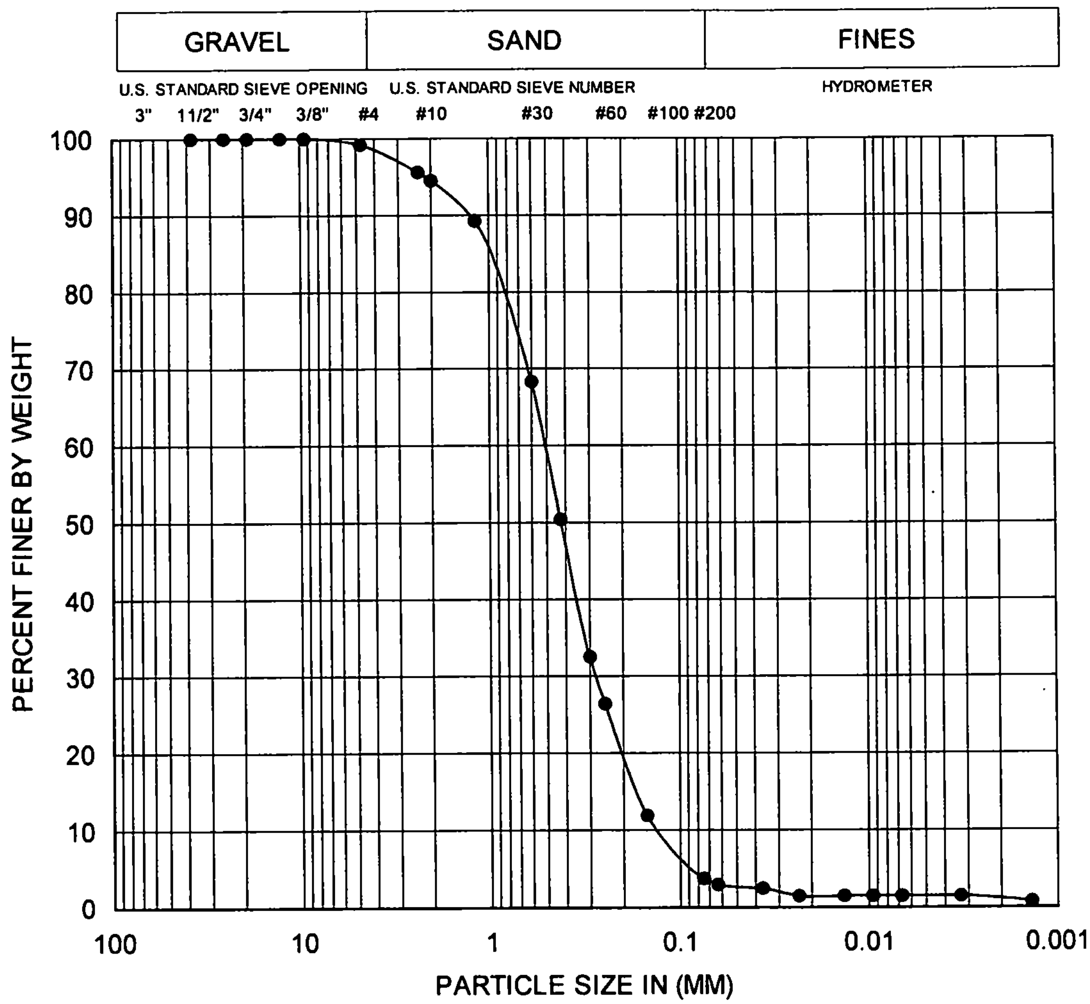


Boring No. : B-1
Depth (ft.) : 0.75-5
Sample : Remolded (90% of Maximum Density)
Sample Type : Silty Fine Sand

Cohesion(C) = 105 psf
Friction (ϕ) = 33°
Dry Density (pcf) = 114.3
Moisture (%) = 10.0

Project Name: Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-3

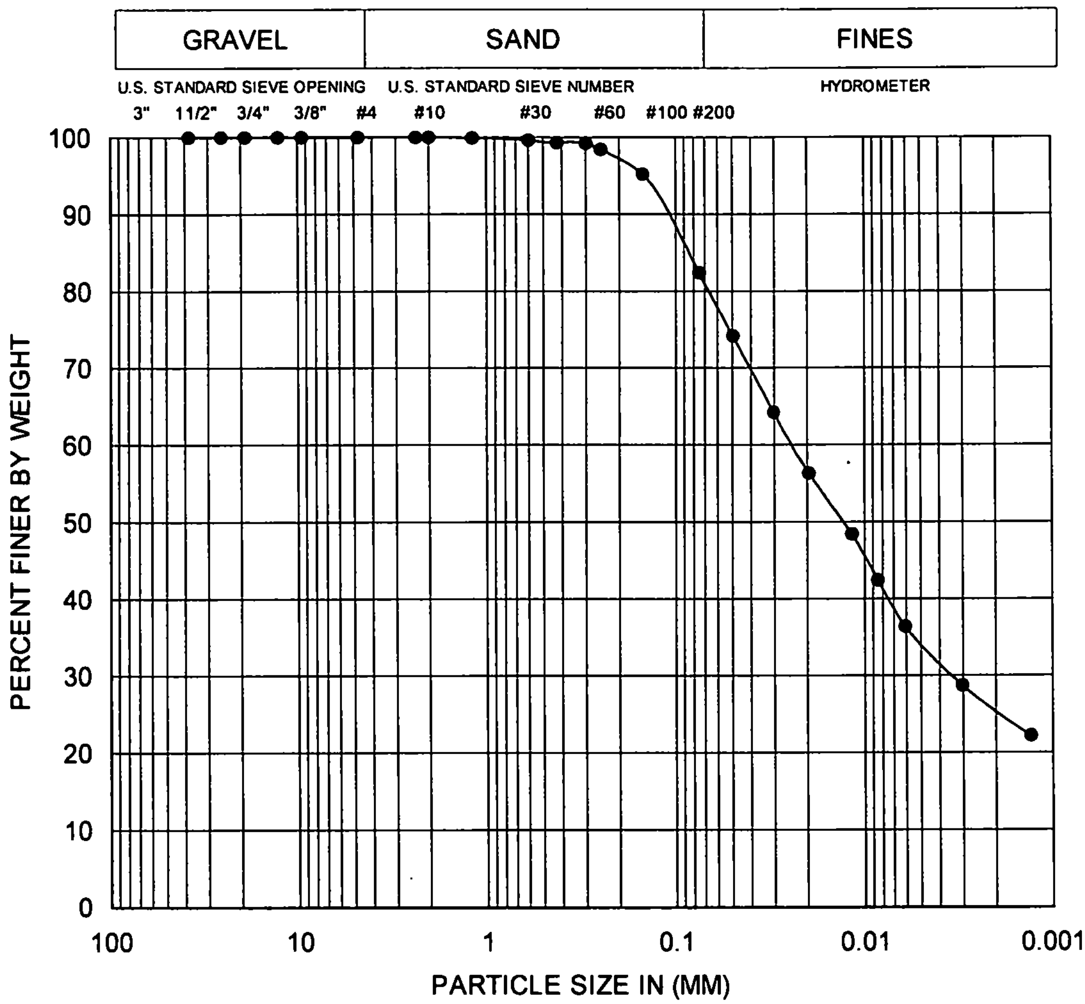


Boring No. : B-2
Depth (Ft.) : 7.0
Sample Type: Fine to Medium Sand (SP)

% Gravel: 0.8
% Sand: 95.9
% Silt: 2.3
% Clay: 1.4

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

PLATE G-1

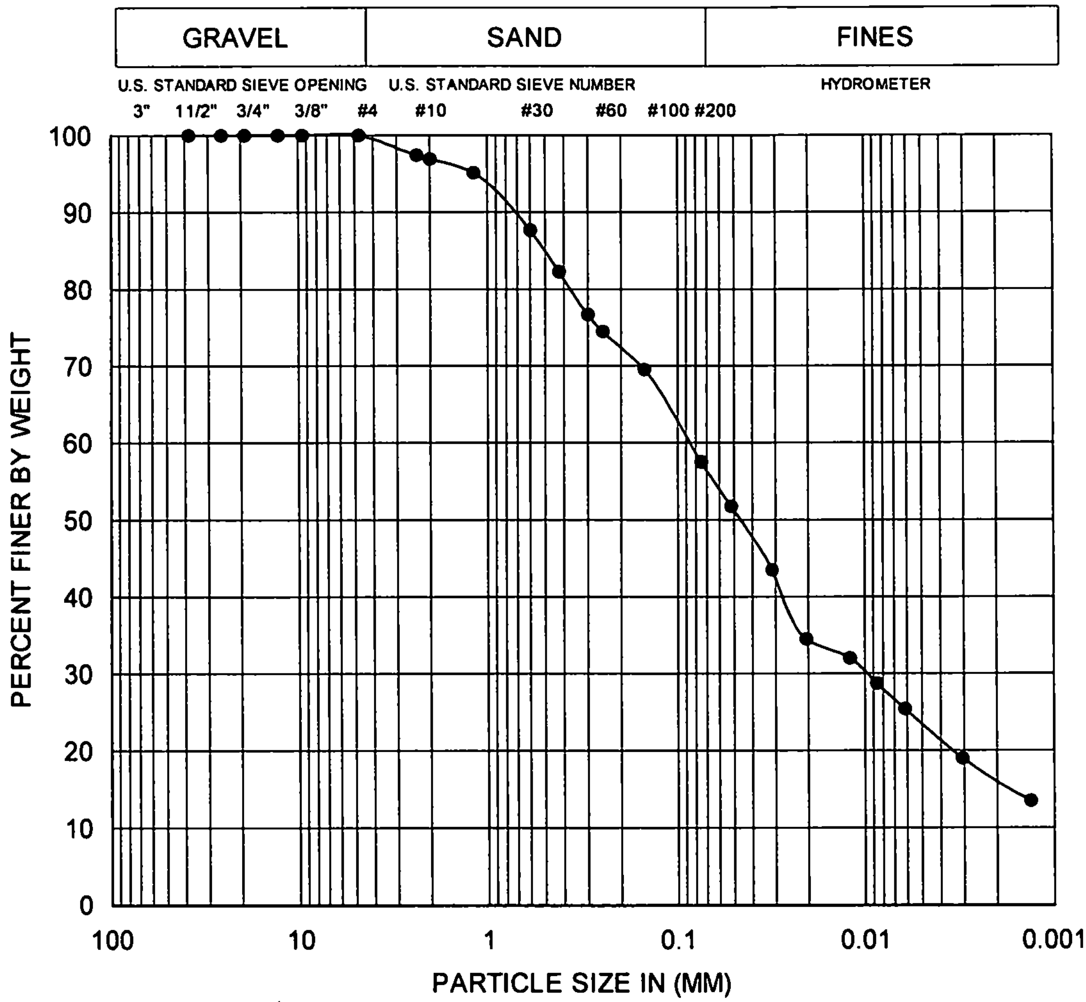


Boring No. : B-2
Depth (Ft.) : 15.5
Sample Type: Silty Clay (CL)

% Gravel: 0.0
% Sand: 17.6
% Silt: 48.8
% Clay: 33.6

Project Name: Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

PLATE G-2

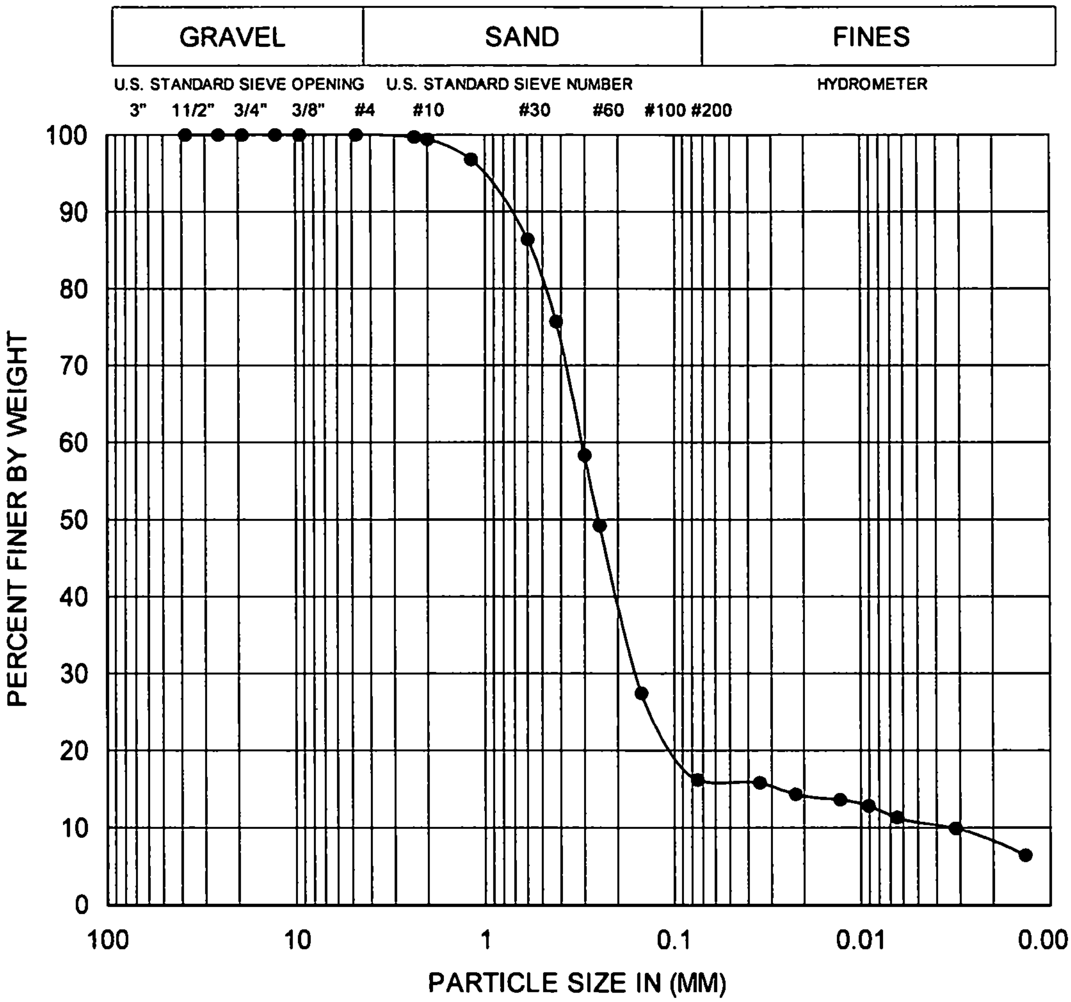


Boring No. : B-2
Depth (Ft.) : 20.0
Sample Type: Fine to Medium Sandy Clay with Silt (CL)

% Gravel: 0.0
% Sand: 42.5
% Silt: 34.5
% Clay: 23.0

Project Name: Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

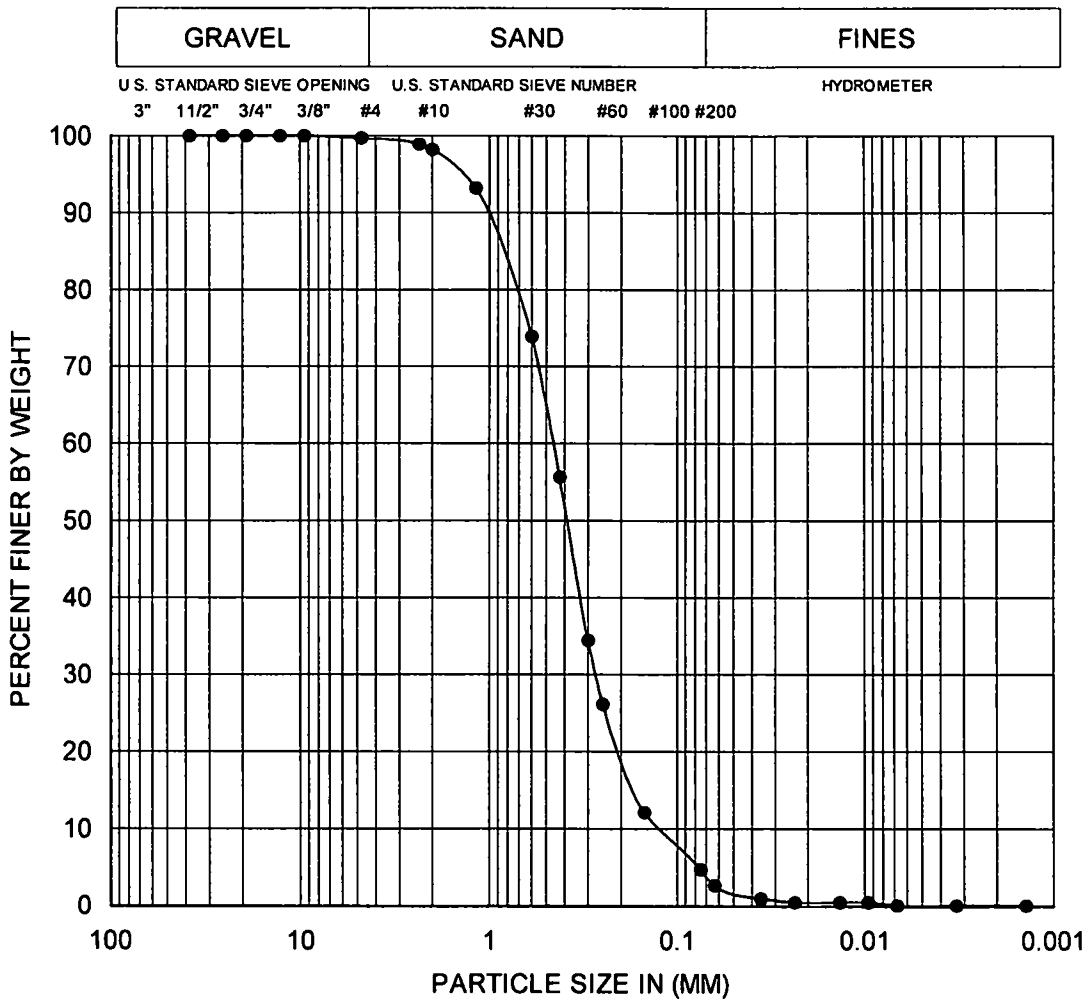
PLATE G-3



Boring No. : B-2
Depth (Ft.) : 23.0
Sample Type: Silty,Clayey Fine to Medium Sand (SC-SM)

% Gravel: 0.0
% Sand: 83.8
% Silt: 5.5
% Clay: 10.7

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

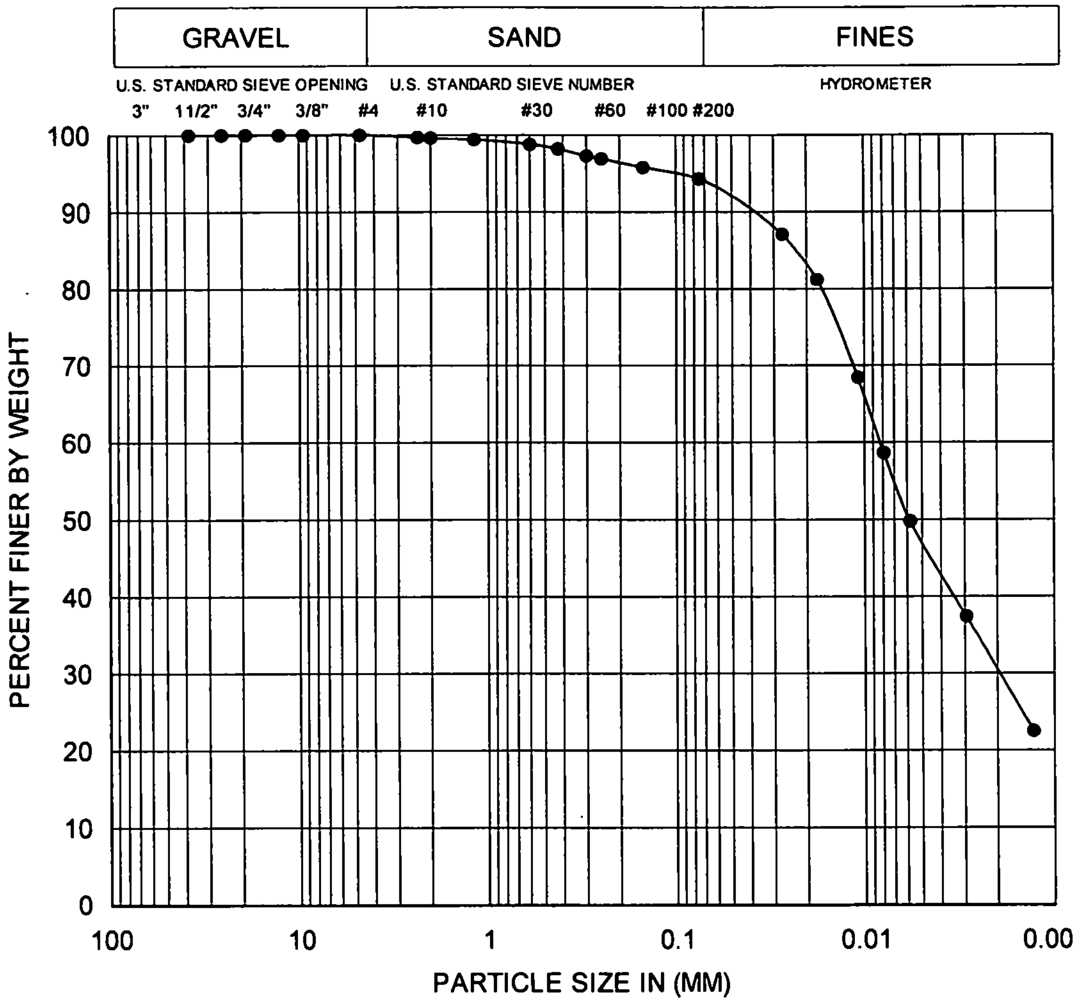


Boring No. : B-2
Depth (Ft.) : 25.0
Sample Type: Fine to Medium Sand (SP)

% Gravel: 0.3
% Sand: 95.0
% Silt: 4.7
% Clay: 0.0

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

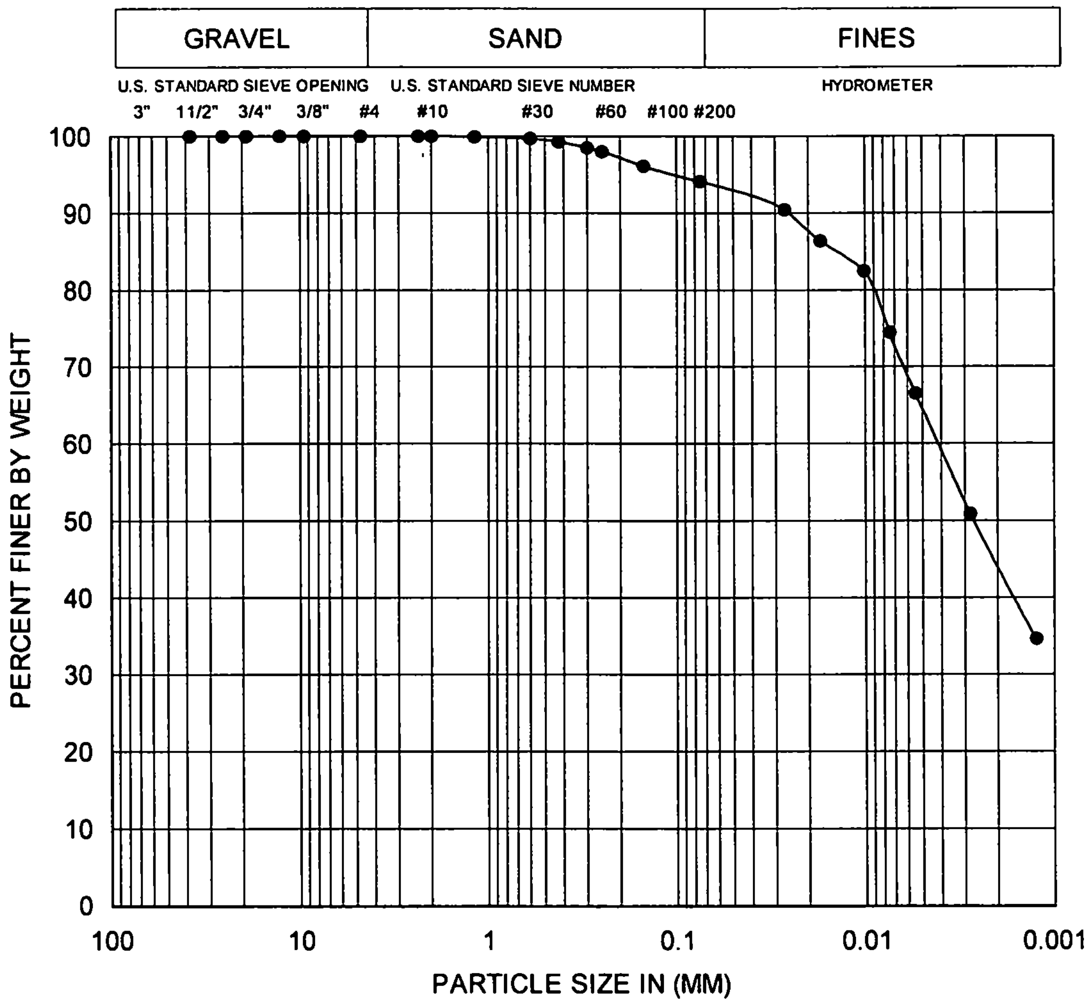
PLATE G-5



Boring No. : B-2
Depth (Ft.) : 35.0
Sample Type: Silty Clay (CL)

% Gravel: 0.0
% Sand: 5.7
% Silt: 48.0
% Clay: 46.3

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

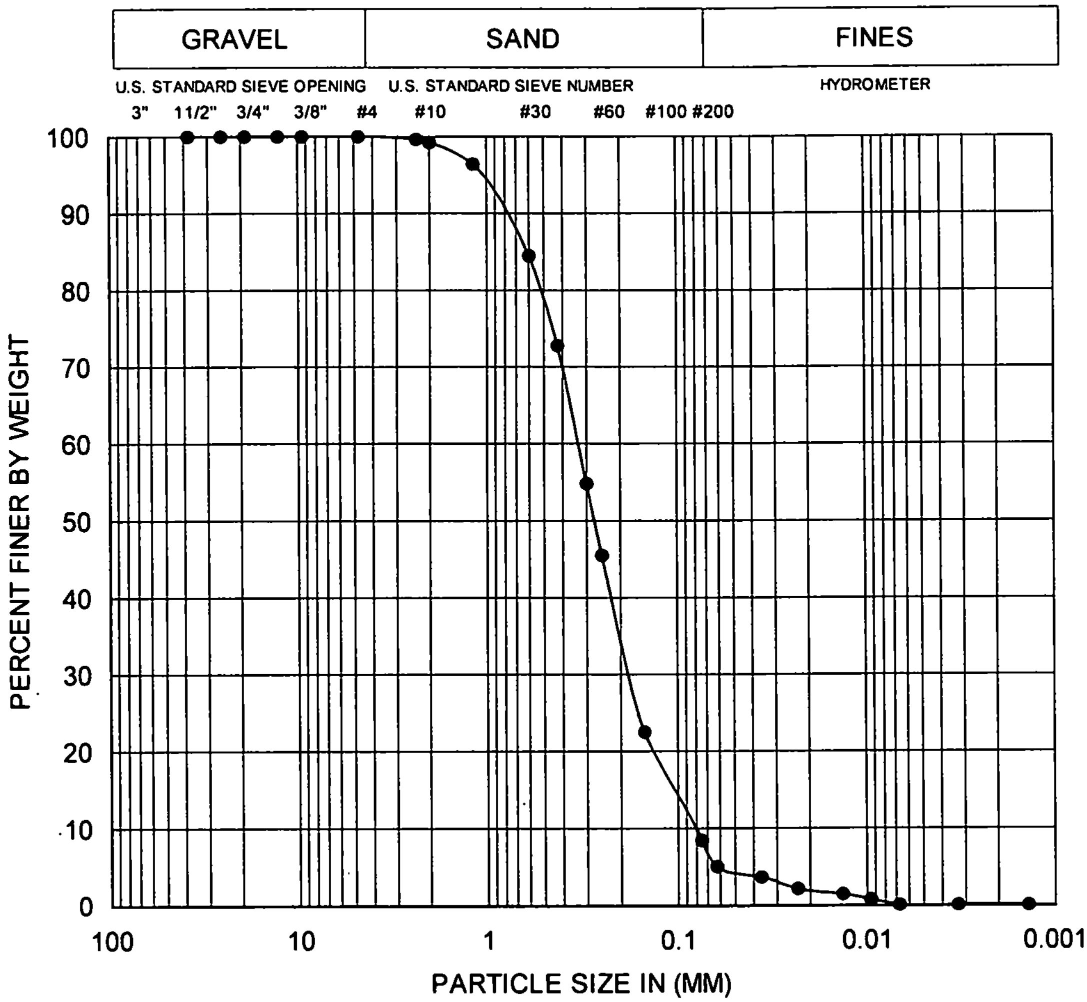


Boring No. : B-2
Depth (Ft.) : 38.5
Sample Type: Clay (CL)

% Gravel: 0.0
% Sand: 5.9
% Silt: 30.3
% Clay: 63.8

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

PLATE G-7



Boring No. : B-2
Depth (Ft.) : 45.0
Sample Type: Fine to Medium Sand with Silt (SP-SM)

% Gravel: 0.0
% Sand: 91.6
% Silt: 8.4
% Clay: 0.0

Project Name:Multi-use/Classroom Additions-2000 N. Fairview Street	Project No.: 10-6212
ASSOCIATED SOILS ENGINEERING, INC.	PARTICLE-SIZE CURVE (ASTM D422)

PLATE G-8

Client Name: Associated Soils Engineering, Inc.
Project No.: ASE 10-6212
For: La Bonte and Associates
Job Name: Wintersberg Presb. Church Additions
Address: 2000 N. Fairview St., Santa Ana, CA

QCI Project No.:10-064-06c
Date: June 11, 2010
Summarized by: ABK

Sample ID	Depth	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 (% By Weight)	Resistivity CT-532 (643) (ohm-cm)
B-1	9" – 5'	7.22	72	0.0480	1,400

PERCOLATION DATA SHEET

Project: Proposed Multi-Use/Classroom Additions-Wintersburg Presbyterian Church – 2000 North Fairview Street, Santa Ana, California Job No.: 10-6212
Test Hole No.: B-3 Date Excavated: 06/03/10 Depth of Test Hole: 5' 0"
Soil Classification: Silty Fine Sand / Fine to Medium Sand / Fine to Coarse Sand
Check for Sandy Soil Criteria Tested By: Gary L. Martin Date: 06/04/10
Presoak: √ Actual Percolation Tested By: Gary L. Martin Date: 06/04/10
(2" of Gravel on Bottom)

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)
1	7:49 8:14	25	-4.0	-56.25	52.25
2	8:15 8:40	25	-4.0	-56.0	52.0

USE NORMAL SANDY (CIRCLE ONE) SOIL CRITERIA

Time	Time Interval (Min.)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Percolation Rate (Min./Inches)
8:41 8:51	10	10	-4.0	-52.75	48.75	0.205
8:51:30 9:01:30	10	20	-4.0	-52.5	48.5	0.206
9:02 9:12	10	30	-4.0	-52.0	48.0	0.208
9:12:30 9:22:30	10	40	-4.0	-52.0	48.0	0.208
9:23 9:33	10	50	-4.0	-51.5	47.5	0.211
9:33:30 9:43:30	10	60	-4.0	-51.5	47.5	0.211

APPENDIX B

SITE FAULTING/SEISMICITY DATA AND SEISMIC HAZARD QUANTIFICATION

EQFAULT – Deterministic Estimation of Peak Acceleration from Digitized Faults

Probabilistic Seismic Assessment Utilizing CGS's Analysis

PLATES L-1 THROUGH L-3	LIQUEFACTION ANALYSIS
PLATE M-1	DYNAMIC SETTLEMENT EVALUATION

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*                      *
*    E Q F A U L T     *
*                      *
*    Version 3.00      *
*                      *
************************
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 10-6212

DATE: 06-09-2010

JOB NAME: Proposed Multi-use/Classroom Additions-Wintersburg
Presbyterian Church-2000 N. Fairview Street,Santa Ana,CA
CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\Cgsflte.dat

SITE COORDINATES:
SITE LATITUDE: 33.7639
SITE LONGITUDE: 117.9043

SEARCH RADIUS: 62 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\Cgsflte.dat

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN JOAQUIN HILLS	5.0(8.1)	6.6	0.406	X
NEWPORT-INGLEWOOD (L.A.Basin)	7.9(12.7)	7.1	0.290	IX
WHITTIER	11.7(18.8)	6.8	0.196	VIII
PUENTE HILLS BLIND THRUST	11.7(18.9)	7.1	0.288	IX
NEWPORT-INGLEWOOD (Offshore)	11.9(19.2)	7.1	0.222	IX
CHINO-CENTRAL AVE. (Elsinore)	16.5(26.6)	6.7	0.176	VIII
ELSINORE (GLEN IVY)	16.6(26.7)	6.8	0.144	VIII
PALOS VERDES	18.3(29.5)	7.3	0.172	VIII
SAN JOSE	19.1(30.7)	6.4	0.124	VII
UPPER ELYSIAN PARK BLIND THRUST	23.9(38.5)	6.4	0.096	VII
SIERRA MADRE	25.9(41.7)	7.2	0.150	VIII
CUCAMONGA	26.9(43.3)	6.9	0.119	VII
RAYMOND	27.2(43.8)	6.5	0.089	VII
CLAMSHELL-SAWPIT	29.1(46.8)	6.5	0.082	VII
VERDUGO	29.1(46.9)	6.9	0.109	VII
HOLLYWOOD	30.9(49.7)	6.4	0.070	VI
ELSINORE (TEMECULA)	33.1(53.2)	6.8	0.068	VI
CORONADO BANK	34.2(55.1)	7.6	0.112	VII
SANTA MONICA	36.0(58.0)	6.6	0.068	VI
SAN JACINTO-SAN BERNARDINO	39.6(63.7)	6.7	0.050	VI
MALIBU COAST	40.5(65.2)	6.7	0.063	VI
SIERRA MADRE (San Fernando)	42.0(67.6)	6.7	0.060	VI
SAN JACINTO-SAN JACINTO VALLEY	42.3(68.1)	6.9	0.054	VI
SAN ANDREAS - Cho-Moj M-1b-1	43.1(69.3)	7.8	0.099	VII
SAN ANDREAS - Mojave M-1c-3	43.1(69.3)	7.4	0.076	VII
SAN ANDREAS - 1857 Rupture M-2a	43.1(69.3)	7.8	0.099	VII
SAN ANDREAS - Whole M-1a	43.1(69.3)	8.0	0.112	VII
SAN ANDREAS - SB-Coach. M-2b	43.2(69.5)	7.7	0.092	VII
SAN ANDREAS - SB-Coach. M-1b-2	43.2(69.5)	7.7	0.092	VII
SAN ANDREAS - San Bernardino M-1	43.2(69.5)	7.5	0.081	VII
SAN GABRIEL	43.9(70.7)	7.2	0.064	VI
NORTHRIDGE (E. Oak Ridge)	44.5(71.6)	7.0	0.070	VI
CLEGHORN	45.5(73.2)	6.5	0.036	V
ANACAPA-DUME	47.9(77.1)	7.5	0.092	VII
SANTA SUSANA	51.5(82.9)	6.7	0.045	VI
ROSE CANYON	52.2(84.0)	7.2	0.051	VI
NORTH FRONTAL FAULT ZONE (West)	52.6(84.6)	7.2	0.065	VI
SAN JACINTO-ANZA	56.7(91.3)	7.2	0.046	VI
HOLSER	57.2(92.1)	6.5	0.033	V
ELSINORE (JULIAN)	57.7(92.9)	7.1	0.042	VI

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SIMI-SANTA ROSA	58.5(94.1)	7.0	0.049	VI

****

-END OF SEARCH- 41 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JOAQUIN HILLS FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 5.0 MILES (8.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4060 g

PROBABILISTIC SEISMIC ASSESSMENT UTILIZING CGS's ANALYSIS

Project No.: **10-6212**

Project Site Coordinates: *Longitude* - **W -117.9043^0**

Latitude - **N 33.7639^0**

Project Site Soil Classification: **Alluvium**

TABLE OF DESIGN GROUND MOTIONS

(CGS Probabilistic Analysis)

Soil Type / Design Acceleration (G)	Firm Rock(1)	Soft Rock(2)	Alluvium(2)
PGA(3)	0.332	0.346	0.379
S_a (0.2 second)(4)	0.796	0.837	0.913
S_a (1.0 second)(4)	0.3	0.374	0.461

(1) Classified by NEHRP (FEMA, 1997) as rocks having a shear wave velocity no less than 760 meters per second.

(2) Modification factors from PGA reflecting local site soils conditions are per NEHRP (FEMA, 1997), which are ground acceleration-dependent.

(3) Per Cao et al. (2003), it is defined as the peak ground acceleration for the subject site that carries a 10% probability of being exceeded in 50 years.

(4) Spectra acceleration derived from respective PGA with a 5% damping ratio incorporated.

LIQTEST

```
*****************************
*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
*****************************
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EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 10-6212 DATE: 06-21-2010

JOB NAME: Proposed Multi-use/Classroom Additions-2000 N. Fairview Street

SOIL-PROFILE NAME: 6212B2.LDW

BORING GROUNDWATER DEPTH: 22.30 ft

CALCULATION GROUNDWATER DEPTH: 20.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.60 Mw

SITE PEAK GROUND ACCELERATION: 0.406 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.00

N60 HAMMER CORRECTION FACTOR: 1.30

MAGNITUDE SCALING FACTOR METHOD: Idriss (1997, in press)

Magnitude Scaling Factor: 1.387

rd-CORRECTION METHOD: NCEER (1997)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 3.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

NCEER [1997] Method   LIQUEFACTION ANALYSIS SUMMARY

File Name: LIQTEST.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.014	0.014	9	8.44	*	*	*	*	*	**

PLATE L-1

1	0.75	0.043	0.043	9	8.44	*	*	*	*	*	**
1	1.25	0.071	0.071	9	8.44	*	*	*	*	*	**
1	1.75	0.100	0.100	9	8.44	*	*	*	*	*	**
1	2.25	0.128	0.128	9	8.44	*	*	*	*	*	**
2	2.75	0.155	0.155	6	0.01	*	*	*	*	*	**
2	3.25	0.181	0.181	6	0.01	*	*	*	*	*	**
2	3.75	0.206	0.206	6	0.01	*	*	*	*	*	**
2	4.25	0.232	0.232	6	0.01	*	*	*	*	*	**
2	4.75	0.257	0.257	6	0.01	*	*	*	*	*	**
2	5.25	0.283	0.283	6	0.01	*	*	*	*	*	**
2	5.75	0.308	0.308	6	0.01	*	*	*	*	*	**
2	6.25	0.334	0.334	6	0.01	*	*	*	*	*	**
2	6.75	0.359	0.359	6	0.01	*	*	*	*	*	**
2	7.25	0.385	0.385	6	0.01	*	*	*	*	*	**
2	7.75	0.410	0.410	6	0.01	*	*	*	*	*	**
2	8.25	0.436	0.436	6	0.01	*	*	*	*	*	**
2	8.75	0.461	0.461	6	0.01	*	*	*	*	*	**
2	9.25	0.487	0.487	6	0.01	*	*	*	*	*	**
2	9.75	0.512	0.512	6	0.01	*	*	*	*	*	**
2	10.25	0.538	0.538	6	0.01	*	*	*	*	*	**
2	10.75	0.563	0.563	6	0.01	*	*	*	*	*	**
2	11.25	0.589	0.589	6	0.01	*	*	*	*	*	**
2	11.75	0.614	0.614	6	0.01	*	*	*	*	*	**
2	12.25	0.640	0.640	6	0.01	*	*	*	*	*	**
2	12.75	0.665	0.665	6	0.01	*	*	*	*	*	**
2	13.25	0.690	0.690	6	0.01	*	*	*	*	*	**
3	13.75	0.717	0.717	7	6.80	*	*	*	*	*	**
3	14.25	0.744	0.744	7	6.80	*	*	*	*	*	**
4	14.75	0.772	0.772	4	0.01	*	*	*	*	*	**
4	15.25	0.799	0.799	4	0.01	*	*	*	*	*	**
5	15.75	0.829	0.829	4	~	*	*	*	*	*	**
5	16.25	0.860	0.860	4	~	*	*	*	*	*	**
5	16.75	0.891	0.891	4	~	*	*	*	*	*	**
5	17.25	0.921	0.921	4	~	*	*	*	*	*	**
5	17.75	0.952	0.952	4	~	*	*	*	*	*	**
5	18.25	0.983	0.983	4	~	*	*	*	*	*	**
5	18.75	1.014	1.014	4	~	*	*	*	*	*	**
5	19.25	1.045	1.045	4	~	*	*	*	*	*	**
5	19.75	1.076	1.076	4	~	*	*	*	*	*	**
6	20.25	1.107	1.099	4	~	~	~	~	~	~	~~
6	20.75	1.138	1.115	4	~	~	~	~	~	~	~~
6	21.25	1.170	1.131	4	~	~	~	~	~	~	~~

NCEER [1997] Method LIQUEFACTION ANALYSIS SUMMARY

File Name: LIQTEST.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C_N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r_d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
7	21.75	1.202	1.147	7	3.20	0.906	11.2	0.119	0.949	0.262	0.63
7	22.25	1.235	1.165	7	3.20	0.906	11.2	0.119	0.948	0.265	0.62
7	22.75	1.268	1.183	7	3.20	0.906	11.2	0.119	0.947	0.268	0.62
7	23.25	1.302	1.200	7	3.20	0.906	11.2	0.119	0.946	0.271	0.61
8	23.75	1.334	1.217	7	1.04	0.900	9.0	0.097	0.945	0.273	0.49
8	24.25	1.367	1.234	7	1.04	0.900	9.0	0.097	0.943	0.276	0.49
9	24.75	1.399	1.250	16	0.02	0.874	18.1	0.189	0.942	0.278	0.94
9	25.25	1.430	1.266	16	0.02	0.874	18.1	0.189	0.941	0.280	0.94
9	25.75	1.462	1.282	16	0.02	0.874	18.1	0.189	0.940	0.283	0.93
9	26.25	1.493	1.298	16	0.02	0.874	18.1	0.189	0.939	0.285	0.92
9	26.75	1.525	1.314	16	0.02	0.874	18.1	0.189	0.938	0.287	0.91
9	27.25	1.556	1.330	16	0.02	0.874	18.1	0.189	0.936	0.289	0.91

9	27.75	1.588	1.346	16	0.02	0.874	18.1	0.189	0.935	0.291	0.90
9	28.25	1.619	1.362	16	0.02	0.874	18.1	0.189	0.934	0.293	0.90
9	28.75	1.651	1.378	16	0.02	0.874	18.1	0.189	0.933	0.295	0.89
10	29.25	1.681	1.392	6	~	~	~	~	~	~	~~
10	29.75	1.711	1.406	6	~	~	~	~	~	~	~~
10	30.25	1.740	1.420	6	~	~	~	~	~	~	~~
10	30.75	1.770	1.434	6	~	~	~	~	~	~	~~
10	31.25	1.799	1.448	6	~	~	~	~	~	~	~~
10	31.75	1.829	1.462	6	~	~	~	~	~	~	~~
10	32.25	1.858	1.476	6	~	~	~	~	~	~	~~
10	32.75	1.888	1.490	6	~	~	~	~	~	~	~~
10	33.25	1.917	1.504	6	~	~	~	~	~	~	~~
10	33.75	1.947	1.518	6	~	~	~	~	~	~	~~
10	34.25	1.976	1.532	6	~	~	~	~	~	~	~~
10	34.75	2.006	1.546	6	~	~	~	~	~	~	~~
10	35.25	2.035	1.560	6	~	~	~	~	~	~	~~
10	35.75	2.065	1.574	6	~	~	~	~	~	~	~~
10	36.25	2.094	1.587	6	~	~	~	~	~	~	~~
10	36.75	2.124	1.601	6	~	~	~	~	~	~	~~
10	37.25	2.153	1.615	6	~	~	~	~	~	~	~~
11	37.75	2.183	1.629	19	~	~	~	~	~	~	~~
11	38.25	2.213	1.643	19	~	~	~	~	~	~	~~
11	38.75	2.242	1.657	19	~	~	~	~	~	~	~~
11	39.25	2.272	1.671	19	~	~	~	~	~	~	~~
11	39.75	2.302	1.685	19	~	~	~	~	~	~	~~
11	40.25	2.331	1.699	19	~	~	~	~	~	~	~~
11	40.75	2.361	1.713	19	~	~	~	~	~	~	~~
11	41.25	2.390	1.727	19	~	~	~	~	~	~	~~
11	41.75	2.420	1.741	19	~	~	~	~	~	~	~~
11	42.25	2.450	1.755	19	~	~	~	~	~	~	~~
11	42.75	2.479	1.770	19	~	~	~	~	~	~	~~
12	43.25	2.510	1.784	38	0.05	0.747	37.0	Infin	0.822	0.305	NonLiq

NCEER [1997] Method

LIQUEFACTION ANALYSIS SUMMARY

File Name: LIQTEST.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
12	43.75	2.541	1.800	38	0.05	0.747	37.0	Infin	0.818	0.305	NonLiq
12	44.25	2.572	1.816	38	0.05	0.747	37.0	Infin	0.814	0.304	NonLiq
12	44.75	2.604	1.832	38	0.05	0.747	37.0	Infin	0.810	0.304	NonLiq
13	45.25	2.636	1.848	29	0.64	0.719	27.7	0.307	0.806	0.303	1.41
13	45.75	2.669	1.865	29	0.64	0.719	27.7	0.307	0.802	0.303	1.41
13	46.25	2.702	1.883	29	0.64	0.719	27.7	0.307	0.798	0.302	1.41
13	46.75	2.735	1.900	29	0.64	0.719	27.7	0.307	0.794	0.301	1.41
13	47.25	2.768	1.917	29	0.64	0.719	27.7	0.307	0.789	0.301	1.42
13	47.75	2.801	1.935	29	0.64	0.719	27.7	0.307	0.785	0.300	1.42
13	48.25	2.833	1.952	29	0.64	0.719	27.7	0.307	0.781	0.299	1.42
13	48.75	2.866	1.969	29	0.64	0.719	27.7	0.307	0.777	0.299	1.43
13	49.25	2.899	1.987	29	0.64	0.719	27.7	0.307	0.773	0.298	1.43
13	49.75	2.932	2.004	29	0.64	0.719	27.7	0.307	0.769	0.297	1.44
13	50.25	2.965	2.021	29	0.64	0.719	27.7	0.307	0.765	0.296	1.44
13	50.75	2.998	2.039	29	0.64	0.719	27.7	0.307	0.761	0.295	1.44
14	51.25	3.030	2.055	22	~	~	~	~	~	~	~~

DYNAMIC SETTLEMENT OF SATURATED SOIL

BORING B-2

(Historical High Groundwater of 20 feet)

(Tokimatsu and Seed, 1984)

Depths, (ft)	Layer Thickness, (ft)	N1 (60) Corrected	Induced Stress Ratio*	Volumetric Strain, (%)	Dynamic Settlement, (inch)
21.5-23.5	2.0	11.2	0.267	2.4	0.58
23.5-24.5	1.0	9.0	0.275	2.8	0.34
24.5-29.0	5.5	18.1	0.287	1.7	1.12

Dynamic Settlement of Saturated Soil = ***2.04 inches***

* - average value

DYNAMIC SETTLEMENT OF DRY SOIL

Fundamentals of Earthquake Resistant Construction by Krinitzsky, Gould and Edinger (1984)

BORING B-2

Depths (feet)	Layer Thickness (feet)	N	Ap/g	Δ H/H (%)	Dynamic Settlement (inches)
0.75-2.5	1.75	9	0.406	0.25	0.05
2.5-13.5	11.0	6	0.406	0.30	0.40
13.5-14.5	1.0	7	0.406	0.286	0.03
14.5-15.5	1.0	4	0.406	0.334	0.04

Dynamic Settlement of Dry Soil = 0.52 inches

Dynamic Settlement of Dry Soil = 0.38 inches

(After completion of remedial grading-Compaction Grouting alternate)

APPENDIX C

SCHMERTMANN SETTLEMENT ANALYSIS OF MAT FOUNDATION

ASSOCIATED SOILS ENGINEERING, INC.

PROJECT: Multi-Use/Classroom Addition @ Wintersburg Presbyterian Church

LOCATION: 2000 N. Fairview Street, Santa Ana, CA

Schmertmann Settlement

CONDITION 1 **Center; 200' x 90' mat (B/L=1)'; without compaction grouting** 2

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	15.0	15.0	4.0	3	20	40	7.50	0.083	0.167	0.0625
2	15.0	21.5	6.5	4.0	5	1	2.5	18.25	0.203	0.262	0.6818
3	21.5	29.0	7.5	16.0	3	80	160	25.25	0.281	0.324	0.0152
4	29.0	43.0	14.0	4.0	5	1	2.5	36.00	0.400	0.420	2.3520
5	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.493	0.0141
6	51.0	51.5	0.5	22.0	5	2	5	51.25	0.569	0.477	0.0477
7			0.0			0	0	0.00	0.000	0.100	0.0000
8			0.0			0	0	0.00	0.000	0.100	0.0000
9			0.0			0	0	0.00	0.000	0.100	0.0000
10			0.0			0	0	0.00	0.000	0.100	0.0000
11			0.0			0	0	0.00	0.000	0.100	0.0000
12			0.0			0	0	0.00	0.000	0.100	0.0000
13			0.0			0	0	0.00	0.000	0.100	0.0000
14			0.0			0	0	0.00	0.000	0.100	0.0000
15			0.0			0	0	0.00	0.000	0.100	0.0000
16			0.0			0	0	0.00	0.000	0.100	0.0000
17			0.0			0	0	0.00	0.000	0.100	0.0000
18			0.0			0	0	0.00	0.000	0.100	0.0000
19			0.0			0	0	0.00	0.000	0.100	0.0000
20			0.0			0	0	0.00	0.000	0.100	0.0000
										Sum =	3.1732

Settlement:	0.729	feet
	8.74	inches
	22.21	cm

CONDITION 2 **Corner; 200' x 90' mat (Plane Strain); without compaction grouting** 1

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	15.0	15.0	4.0	3	20	40	7.50	0.083	0.225	0.0844
2	15.0	21.5	6.5	4.0	5	1	3.5	18.25	0.203	0.261	0.4844
3	21.5	29.0	7.5	16.0	3	80	160	25.25	0.281	0.284	0.0133
4	29.0	43.0	14.0	4.0	5	1	3.5	36.00	0.400	0.320	1.2800
5	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.357	0.0102
6	51.0	51.5	0.5	22.0	5	2	7	51.25	0.569	0.371	0.0265
7			0.0			0	0	0.00	0.000	0.200	0.0000
8			0.0			0	0	0.00	0.000	0.200	0.0000
9			0.0			0	0	0.00	0.000	0.200	0.0000
10			0.0			0	0	0.00	0.000	0.200	0.0000
11			0.0			0	0	0.00	0.000	0.200	
12			0.0			0	0	0.00	0.000	0.200	
13			0.0			0	0	0.00	0.000	0.200	
14			0.0			0	0	0.00	0.000	0.200	
15			0.0			0	0	0.00	0.000	0.200	
16			0.0			0	0	0.00	0.000	0.200	
17			0.0			0	0	0.00	0.000	0.200	
18			0.0			0	0	0.00	0.000	0.200	
19			0.0			0	0	0.00	0.000	0.200	
20			0.0			0	0	0.00	0.000	0.200	
										Sum =	1.8988

Settlement:	0.436	feet
	5.23	inches
	13.29	cm

DIFFERENTIAL =	-3.51	inches

ASSOCIATED SOILS ENGINEERING, INC.

PROJECT: Multi-Use/Classroom Addition @ Wintersburg Presbyterian Church

LOCATION: 2000 N. Fairview Street, Santa Ana, CA

Schmertmann Settlement

CONDITION 1 **Center; 200' x 90' mat (B/L=1)'; with site grading & w/o compaction grouting** **2**

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	5.0	5.0	25.0	3	125	250	2.50	0.028	0.122	0.0024
2	5.0	15.0	10.0	4.0	3	20	40	10.00	0.111	0.189	0.0472
3	15.0	21.5	6.5	4.0	5	1	2.5	18.25	0.203	0.262	0.6818
4	21.5	29.0	7.5	16.0	3	80	160	25.25	0.281	0.324	0.0152
5	29.0	43.0	14.0	4.0	5	1	2.5	36.00	0.400	0.420	2.3520
6	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.493	0.0141
7	51.0	51.5	0.5	22.0	5	2	5	51.25	0.569	0.477	0.0477
8			0.0			0	0	0.00	0.000	0.100	0.0000
9			0.0			0	0	0.00	0.000	0.100	0.0000
10			0.0			0	0	0.00	0.000	0.100	0.0000
11			0.0			0	0	0.00	0.000	0.100	0.0000
12			0.0			0	0	0.00	0.000	0.100	0.0000
13			0.0			0	0	0.00	0.000	0.100	0.0000
14			0.0			0	0	0.00	0.000	0.100	0.0000
15			0.0			0	0	0.00	0.000	0.100	0.0000
16			0.0			0	0	0.00	0.000	0.100	0.0000
17			0.0			0	0	0.00	0.000	0.100	0.0000
18			0.0			0	0	0.00	0.000	0.100	0.0000
19			0.0			0	0	0.00	0.000	0.100	0.0000
20			0.0			0	0	0.00	0.000	0.100	0.0000
										Sum =	3.1604

Settlement:		
	0.726	feet
	8.71	inches
	22.12	cm

CONDITION 2 **Corner; 200' x 90' mat (Plane Strain); with site grading & w/o compaction grouting** **1**

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	5.0	5.0	25.0	3	125	250	2.50	0.028	0.208	0.0042
2	5.0	15.0	10.0	4.0	3	20	40	10.00	0.111	0.233	0.0583
3	15.0	21.5	6.5	4.0	5	1	3.5	18.25	0.203	0.261	0.4844
4	21.5	29.0	7.5	16.0	3	80	160	25.25	0.281	0.284	0.0133
5	29.0	43.0	14.0	4.0	5	1	3.5	36.00	0.400	0.320	1.2800
6	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.357	0.0102
7	51.0	51.5	0.5	22.0	5	2	7	51.25	0.569	0.371	0.0265
8			0.0			0	0	0.00	0.000	0.200	0.0000
9			0.0			0	0	0.00	0.000	0.200	0.0000
10			0.0			0	0	0.00	0.000	0.200	0.0000
11			0.0			0	0	0.00	0.000	0.200	
12			0.0			0	0	0.00	0.000	0.200	
13			0.0			0	0	0.00	0.000	0.200	
14			0.0			0	0	0.00	0.000	0.200	
15			0.0			0	0	0.00	0.000	0.200	
16			0.0			0	0	0.00	0.000	0.200	
17			0.0			0	0	0.00	0.000	0.200	
18			0.0			0	0	0.00	0.000	0.200	
19			0.0			0	0	0.00	0.000	0.200	
20			0.0			0	0	0.00	0.000	0.200	
										Sum =	1.8769

Settlement:		
	0.431	feet
	5.17	inches
	13.14	cm

DIFFERENTIAL =	-3.54	inches

ASSOCIATED SOILS ENGINEERING, INC.

PROJECT: Multi-Use/Classroom Addition @ Wintersburg Presbyterian Church

LOCATION: 2000 N. Fairview Street, Santa Ana, CA

Schmertmann Settlement

CONDITION 1 **Center; 200' x 90' mat (B/L=1)'; with site grading & compaction grouting** **2**

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	5.0	5.0	25.0	3	125	250	2.50	0.028	0.122	0.0024
2	5.0	15.0	10.0	4.0	3	20	40	10.00	0.111	0.189	0.0472
3	15.0	21.5	6.5	4.0	5	1	2.5	18.25	0.203	0.262	0.6818
4	21.5	29.0	7.5	30.0	3	150	300	25.25	0.281	0.324	0.0081
5	29.0	43.0	14.0	4.0	5	1	2.5	36.00	0.400	0.420	2.3520
6	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.493	0.0141
7	51.0	51.5	0.5	22.0	5	2	5	51.25	0.569	0.477	0.0477
8			0.0			0	0	0.00	0.000	0.100	0.0000
9			0.0			0	0	0.00	0.000	0.100	0.0000
10			0.0			0	0	0.00	0.000	0.100	0.0000
11			0.0			0	0	0.00	0.000	0.100	0.0000
12			0.0			0	0	0.00	0.000	0.100	0.0000
13			0.0			0	0	0.00	0.000	0.100	0.0000
14			0.0			0	0	0.00	0.000	0.100	0.0000
15			0.0			0	0	0.00	0.000	0.100	0.0000
16			0.0			0	0	0.00	0.000	0.100	0.0000
17			0.0			0	0	0.00	0.000	0.100	0.0000
18			0.0			0	0	0.00	0.000	0.100	0.0000
19			0.0			0	0	0.00	0.000	0.100	0.0000
20			0.0			0	0	0.00	0.000	0.100	0.0000
										Sum =	3.1533

Settlement:		
	0.724	feet
	8.69	inches
	22.07	cm

CONDITION 2 **Corner; 200' x 90' mat (Plane Strain); with site grading & compaction grouting** **1**

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	5.0	5.0	25.0	3	125	250	2.50	0.028	0.208	0.0042
2	5.0	15.0	10.0	4.0	3	20	40	10.00	0.111	0.233	0.0583
3	15.0	21.5	6.5	4.0	5	1	3.5	18.25	0.203	0.261	0.4844
4	21.5	29.0	7.5	30.0	3	150	300	25.25	0.281	0.284	0.0071
5	29.0	43.0	14.0	4.0	5	1	3.5	36.00	0.400	0.320	1.2800
6	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.357	0.0102
7	51.0	51.5	0.5	22.0	5	2	7	51.25	0.569	0.371	0.0265
8			0.0			0	0	0.00	0.000	0.200	0.0000
9			0.0			0	0	0.00	0.000	0.200	0.0000
10			0.0			0	0	0.00	0.000	0.200	0.0000
11			0.0			0	0	0.00	0.000	0.200	
12			0.0			0	0	0.00	0.000	0.200	
13			0.0			0	0	0.00	0.000	0.200	
14			0.0			0	0	0.00	0.000	0.200	
15			0.0			0	0	0.00	0.000	0.200	
16			0.0			0	0	0.00	0.000	0.200	
17			0.0			0	0	0.00	0.000	0.200	
18			0.0			0	0	0.00	0.000	0.200	
19			0.0			0	0	0.00	0.000	0.200	
20			0.0			0	0	0.00	0.000	0.200	
										Sum =	1.8707

Settlement:		
	0.430	feet
	5.15	inches
	13.09	cm

DIFFERENTIAL = -3.53 inches

ASSOCIATED SOILS ENGINEERING, INC.

PROJECT: Multi-Use/Classroom Addition @ Wintersburg Presbyterian Church

LOCATION: 2000 N. Fairview Street, Santa Ana, CA

Schmertmann Settlement

CONDITION 1 — **Center; 200' x 90' mat (B/L=1)'; without site grading & with geopiers** — 2

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	15.0	15.0	25.0	3	125	250	7.50	0.083	0.167	0.0100
2	15.0	21.5	6.5	4.0	5	20	50	18.25	0.203	0.262	0.0341
3	21.5	29.0	7.5	35.0	3	175	350	25.25	0.281	0.324	0.0070
4	29.0	43.0	14.0	4.0	5	1	2.5	36.00	0.400	0.420	2.3520
5	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.493	0.0141
6	51.0	51.5	0.5	22.0	5	2	5	51.25	0.569	0.477	0.0477
7			0.0			0	0	0.00	0.000	0.100	0.0000
8			0.0			0	0	0.00	0.000	0.100	0.0000
9			0.0			0	0	0.00	0.000	0.100	0.0000
10			0.0			0	0	0.00	0.000	0.100	0.0000
11			0.0			0	0	0.00	0.000	0.100	0.0000
12			0.0			0	0	0.00	0.000	0.100	0.0000
13			0.0			0	0	0.00	0.000	0.100	0.0000
14			0.0			0	0	0.00	0.000	0.100	0.0000
15			0.0			0	0	0.00	0.000	0.100	0.0000
16			0.0			0	0	0.00	0.000	0.100	0.0000
17			0.0			0	0	0.00	0.000	0.100	0.0000
18			0.0			0	0	0.00	0.000	0.100	0.0000
19			0.0			0	0	0.00	0.000	0.100	0.0000
20			0.0			0	0	0.00	0.000	0.100	0.0000
										Sum =	2.4648

Settlement:		
	0.566	feet
	6.79	inches
	17.25	cm

CONDITION 2 — **Corner; 200' x 90' mat (Plane Strain); without site grading & with geopiers** — 1

	500	psf
Foundation Pressure	0.250	tsf
Footing Width (B)	90.0	feet
Footing Depth	1.0	feet
Soil Unit Weight	120.0	pcf
Time of Interest	15.0	years

Initial Overburden	0.060	tsf
Net Foundation Pressure	0.190	tsf
C1 =	0.84	
Ct =	1.44	

LAYER	Depth (ft)		Δ Z	SPT-N	Soil	Qc	E	Middle	Z/B	Iz	(Iz/E)*Z
	Top	Bottom	(ft)		Type	(tsf)	(tsf)	Depth (ft)			
1	0.0	15.0	15.0	25.0	3	125	250	7.50	0.083	0.225	0.0135
2	15.0	21.5	6.5	4.0	5	20	70	18.25	0.203	0.261	0.0242
3	21.5	29.0	7.5	35.0	3	175	350	25.25	0.281	0.284	0.0061
4	29.0	43.0	14.0	4.0	5	1	3.5	36.00	0.400	0.320	1.2800
5	43.0	51.0	8.0	28.0	3	140	280	47.00	0.522	0.357	0.0102
6	51.0	51.5	0.5	22.0	5	2	7	51.25	0.569	0.371	0.0265
7			0.0			0	0	0.00	0.000	0.200	0.0000
8			0.0			0	0	0.00	0.000	0.200	0.0000
9			0.0			0	0	0.00	0.000	0.200	0.0000
10			0.0			0	0	0.00	0.000	0.200	0.0000
11			0.0			0	0	0.00	0.000	0.200	
12			0.0			0	0	0.00	0.000	0.200	
13			0.0			0	0	0.00	0.000	0.200	
14			0.0			0	0	0.00	0.000	0.200	
15			0.0			0	0	0.00	0.000	0.200	
16			0.0			0	0	0.00	0.000	0.200	
17			0.0			0	0	0.00	0.000	0.200	
18			0.0			0	0	0.00	0.000	0.200	
19			0.0			0	0	0.00	0.000	0.200	
20			0.0			0	0	0.00	0.000	0.200	
										Sum =	1.3605

Settlement:		
	0.312	feet
	3.75	inches
	9.52	cm

DIFFERENTIAL = -3.04 inches

APPENDIX D

INFORMATION OF COMPACTION GROUTING AND GEOPIER GROUND MODIFICATIN TECHNIQUES

HAYWARD BAKER INC.

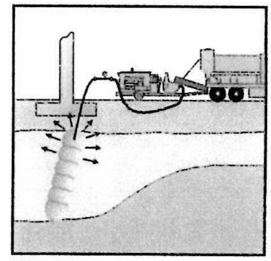
COMPACTION GROUTING

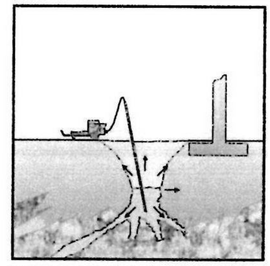
Compaction grouting improves a wide range of ground conditions by displacement, for a variety of site improvement and remedial applications.

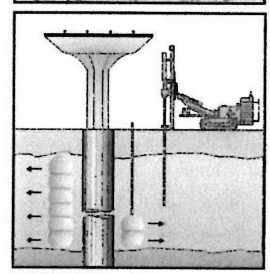
Compaction grouting was used to seal this 160-foot diameter sinkhole that extended down to the Floridan aquifer.

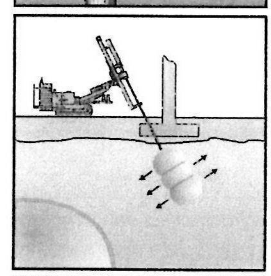
When a properly designed compaction grout is injected into loose soils, homogeneous grout bulbs are formed that displace, densify and thus strengthen the surrounding soil. The technique was originally developed in the 1950's as a remedial measure for the correction of building settlement, and used almost exclusively for that purpose for many years. Over the past twenty years, however, compaction grouting technology has evolved to treat a wide range of subsurface conditions for new and remedial construction. These include rubble fills, poorly placed fills, loosened or collapsible soils, sinkhole sites, and liquefiable soils.

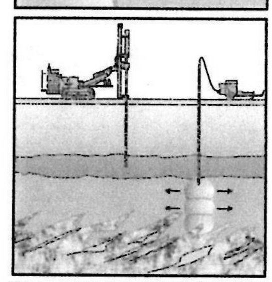
Hayward Baker's compaction grouting techniques, which include the internationally respected Denver System, offer an economic advantage over conventional approaches such as removal and replacement or piling. Compaction grouting can be accomplished where access is difficult and space is limited. Since compaction grouting's effectiveness is independent of structural connections, the technique is readily adaptable to existing foundations.

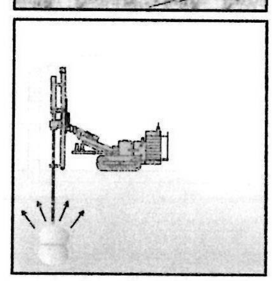












HAYWARD BAKER
Geotechnical Construction

KELLER

Compaction Grouting Technology...

Compaction grouting improves ground conditions by displacement. A very viscous (low-mobility), aggregate grout is pumped in stages to displace and densify the surrounding soils. By sequencing the grouting work from primary to secondary to tertiary locations, this densification process can be performed to achieve significant improvement. Hayward Baker's compaction grouting capability, spanning more than 25 years, is enhanced by the control features provided by the Denver System: batching-on-demand, and specialized, high pressure injection.

The grout mix must have specific characteristics: a very low mobility (low slump) mixture that is 'pumpable' but, upon installation, exhibits an internal friction enabling it to remain intact and displace the surrounding soil without fracturing it.

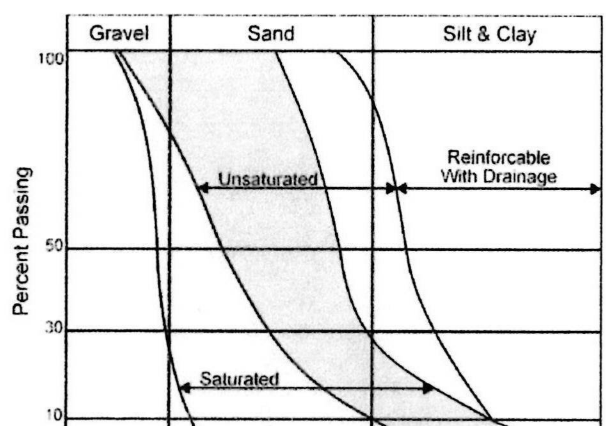
Site Investigation

For successful compaction grouting, comprehensive knowledge of subsurface conditions is important. In order to prepare a suitable program, a geotechnical engineering consultant will develop a site investigation report, which will generally contain site geology and history, soil gradation, and the in situ horizontal permeability of each treatment stratum. Type and condition of nearby structures and utilities, together with plan and elevation locations, will further assist program development.

Geotechnical Considerations

Conditions necessary for optimum compaction grouting results:

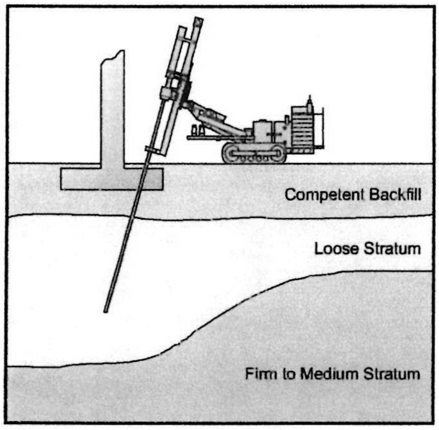
1. The in situ vertical stress in the treatment stratum must be sufficient to enable the grout to displace the soil horizontally (if uncontrolled heave of the ground surface occurs, densification will be minimized).
2. When compaction grout is injected into saturated soils, a pore pressure increase occurs as a result of ground displacement. This increased pressure must dissipate for effective densification to take place. Therefore, the grout injection rate should be slow enough to allow pore pressure dissipation. Sequencing of grout injection is also important.
3. Compaction grouting can usually be effective in most silts and sands, provided that the soil is not near saturation.
4. Soils that lose strength during remolding (saturated, fine-grained soils; sensitive clays) should be avoided.
5. Greater displacement will occur in weaker soil strata. Excavated grout bulbs confirm that compaction grouting focuses improvement where it is most needed.
6. Collapsible soils can usually be treated effectively by adding water during drilling prior to compaction grout injection.
7. Stratified soils, particularly thinly stratified soils, can be cause for difficult or reduced improvement capability.

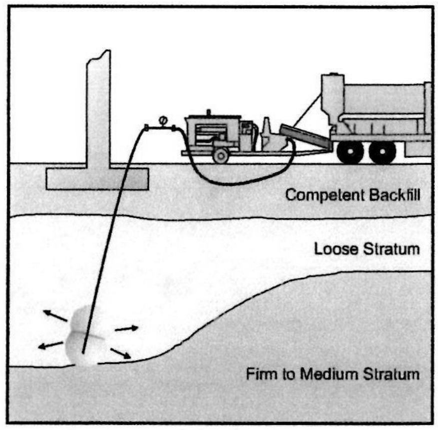


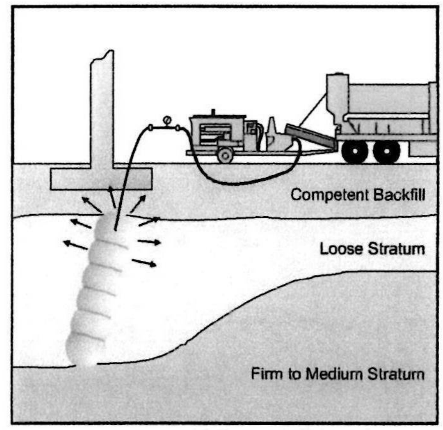
Range of soils that will show improvement by post-testing. Compaction grouting can also be used to reinforce soils beyond this guideline, provided that drainage is enhanced.

"The design and application of compaction grouting is always site-specific, considering the entire above- and below-ground conditions."

Compaction Grouting Delivery Methods







Installation of grout pipe:

- *Drill or drive casing*
- *Location very important*
- *Record ground information from casing installation*

Initiation of grouting:

- *Typically bottom up, but can be top down*
- *Grout quality important*
- *Pressure and/or volume of grout is usually limited*
- *Slow, uniform stage injection*

Continuation of grouting:

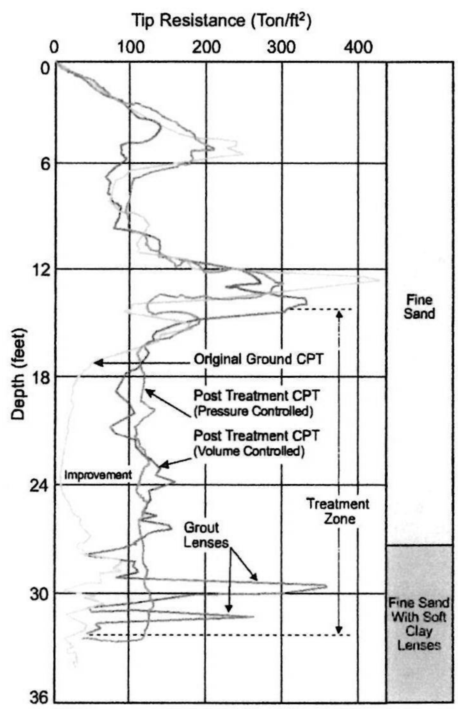
- *On-site batching can aid control*
- *Grout quality important*
- *Pressure, grout quantity and indication of heave are controlling factors*
- *Sequencing of plan injection points very important*

Improvement Conditions

Typically greater than 1,500 psf overburden stress is required to maximize densification. Limited densification can be achieved with less overburden. This stress can come from overburden soils, surcharge loads and/or foundation loads. When densification is the primary intent, a replacement ratio and pressure criterion is applied to each stage of compaction grouting. This ratio is determined based on the existing density, the soil density range, and the amount of displacement necessary to affect the improvement.

$$\text{Replacement Ratio (RR)} = \frac{\text{CG Volume}}{\text{Treatment Volume}} \simeq 5 \text{ to } 15\% \text{ (typical)}$$

Experience has proven that treatment spacing should not exceed 6 to 10 ft. From this, a compaction grouting volume can be calculated. The maximum pressure criterion prevents fracture and ground heave and compensates for stiff zones in the treatment area. Vertical stages are usually set at 2- to 3-ft intervals; tighter grid spacing will generally lead to better results.



Cone Penetrometer Test results, such as the ones illustrated above for volume cut-off and pressure cut off, show the degree of improvement achieved by compaction grouting.

Quality Control/Quality Assurance

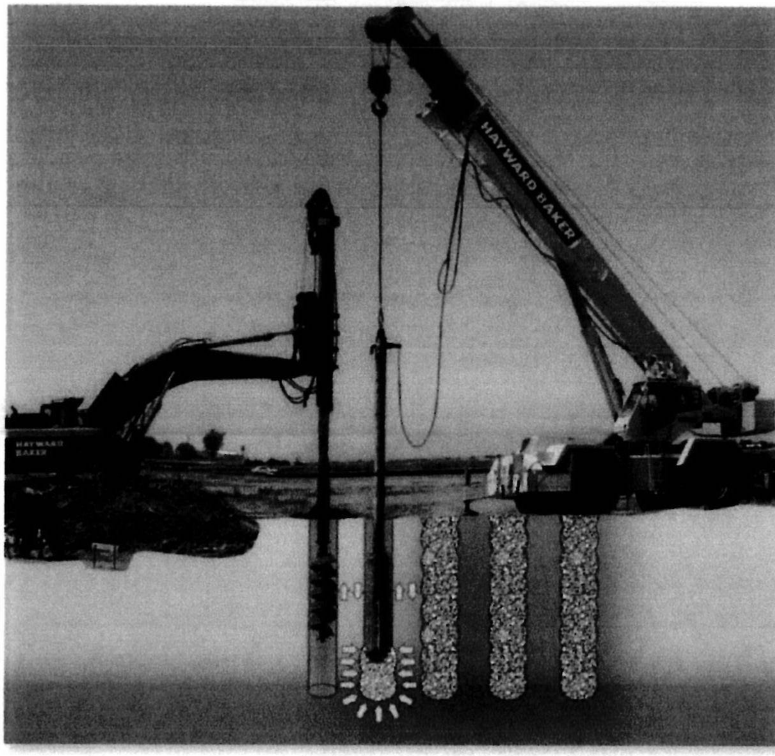
Quality control includes procedural inspection and documentation of the work activity, testing to ensure proper mix design and injection rates, and verification of ground improvement where applicable. Ground improvement can be assessed by Standard Peneteration Testing, Cone Penetrometer Testing, or other similar methods. Data recording of important grouting parameters has been utilized on sensitive projects.

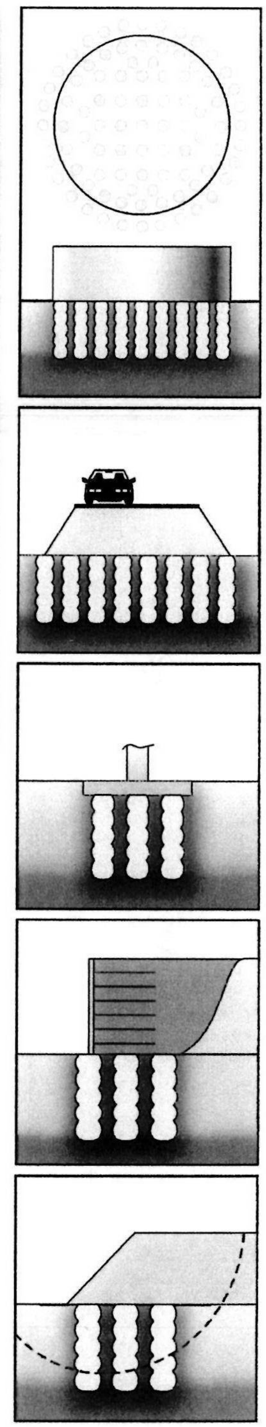
HAYWARD BAKER INC.

VIBRO PIERS

Aggregate Piers for Shallow Foundations

With over 60 years of experience on thousands of projects, Hayward Baker can mobilize quickly to install a Vibro Pier system that is often more cost-effective than other foundation systems.





Vibro Piers, also known as aggregate piers, are typically installed to intermediate depths of 5 to 20 feet for the support of new loads. Suited for light to heavy loads, on large or small projects, Vibro Piers are quick to install and very effective at reinforcing the surrounding soil.

Vibro Piers reinforce the ground to increase bearing capacity, reduce settlement, increase global stability and decrease seismic deformations. Vibro Pier technology utilizes a powerful down hole vibrator to compact select aggregate in lifts. The vibratory energy and ramming action of the vibrator causes the dense aggregate to interlock and form a stiff pier that engages the surrounding soil, providing reinforcement and increased shear resistance.

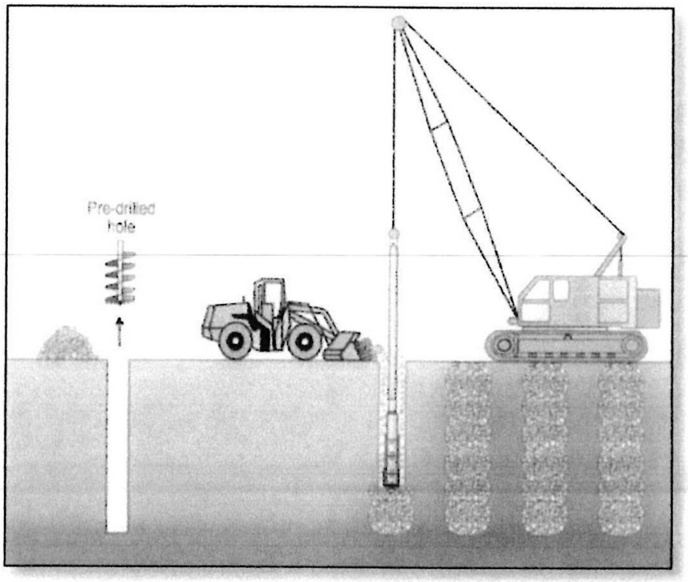
As North America's largest geotechnical contractor, Hayward Baker has the resources to design, build and warranty your project. The vibrators are manufactured in-house, ensuring that performance and reliability are the best in the industry. Hayward Baker's network of regional offices and strategically-located, full-service equipment yards means fast mobilization and reduced start-up costs.

HAYWARD BAKER
Geotechnical Construction

KELLER

Vibro Pier Technology...

Vibro Piers incorporate the best aspects of the deep vibratory densification technique with the most cost-effective equipment to install aggregate reinforcement for the support of new loads. The technique was specifically developed as a fast and economical treatment for poorly placed fills and shallow cohesive, mixed and layered soils.



Vibro Pier construction utilizes a pre-drilled hole which stays open during pier construction. A graded, crushed aggregate is then added and compacted in lifts.

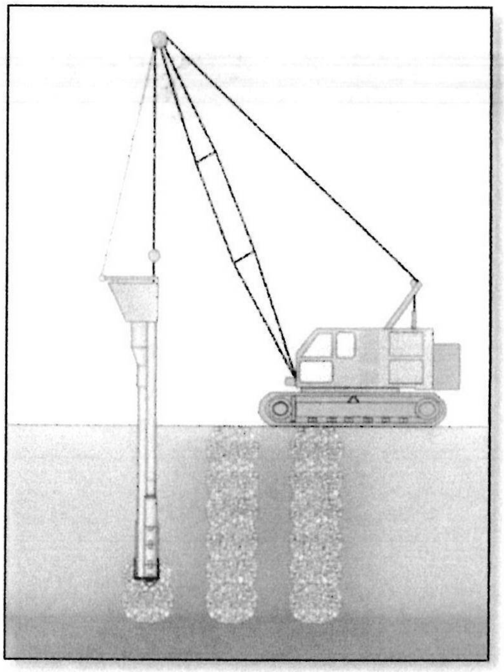
Construction Process

Typical construction begins with pre-drilling the pier location to create a full-depth hole with a diameter that is equal to the final pier design diameter. In soft soils, a slightly smaller diameter may be used due to pier enlargement during compaction.

Aggregate is then introduced to the hole and compacted in lifts by repetitive ramming with a powerful, specially-designed vibrator.

The technique will yield reinforced ground conditions to increase bearing capacity and shear resistance, and reduce settlement from new loads. Anchor bars are incorporated during pier construction when tension resistance is required.

For soils in which the pre-drilled hole will not stay open, the bottom feed process can be used to avoid the need for casing. In the bottom feed process, aggregate is fed through a tremie pipe attached to the vibrator.



With the bottom feed process, the aggregate is conveyed through a tremie pipe to the vibrator tip. The method eliminates the need for casing in unstable soils.

For seismic applications, Vibro Piers can be very effective in reducing dynamic settlement. If loose granular layers are present, the process is a very effective densification technique, reducing the liquefaction potential.

Application

Vibro Piers are suited for support of lightly to heavily loaded structures where soil conditions are soft to medium stiff. Structures that have been successfully supported by Vibro Piers include:

- *Multi-story buildings*
- *Commercial centers*
- *Parking structures*
- *Retaining walls*
- *Warehouses*
- *Wind turbine towers*
- *Storage tanks*
- *Roadway embankments*
- *Schools*
- *Slopes*

4. REDUCTION IN THE POTENTIAL FOR LIQUEFACTION WITHIN GEOPIER-REINFORCED SOIL LAYERS

The installation of Geopier foundation elements results in a significant reduction in the potential for soil liquefaction within Geopier-reinforced soil layers. Geopier foundation elements reduce the potential for soil liquefaction in four ways (Figure 3).

1. Geopier aggregate is compacted to a density sufficient to preclude liquefaction of the aggregate and of the matrix soil in the primary reinforced zone, extending 6 inches from the Geopier cavity perimeter. Thus, the percentage of non-liquefiable bearing material below Geopier-supported footings is generally 50 to 70 percent of the footing area.

2. During installation, horizontal stress within the adjacent soil mass is increased by ramming Geopier aggregate into the cavity. Horizontal stress has been shown by Ferguson et al. (1993) and Handy (1998) to range between two times the preconstruction in-situ lateral pressure and the passive earth pressure at a distance of about three feet from the Geopier element perimeter. After installation, the maximum principal stress (σ_1') within the soil mass adjacent to the Geopier is oriented horizontally and may be 2 to 5 times the overburden vertical stress at that depth. For a soil deposit of a given relative density (usually characterized by standard penetration test N-values), the available resistance to cyclic shearing (τ) is directly proportional to the maximum principal stress (Seed and Idriss, 1982). Thus, if the principal stress increases by a factor of two, the available cyclic shear resistance also increases by a factor of two.

3. Because Geopier elements are stiffer than the surrounding soil, Geopier elements will absorb a greater percentage of shear stresses that occur within the soil deposit during seismic loading. Assuming that shear stresses induced in the soil mass and Geopier elements are proportional to stiffness, the ratio of the applied shear stress resisted by the unimproved matrix soil (τ_s) to the free-field shear stress (τ) induced by the earthquake could be expressed as:

$$\frac{\tau_s}{\tau} = \frac{1}{(1-R_a+R_a R_s)} \qquad \textit{Eq. 1.}$$

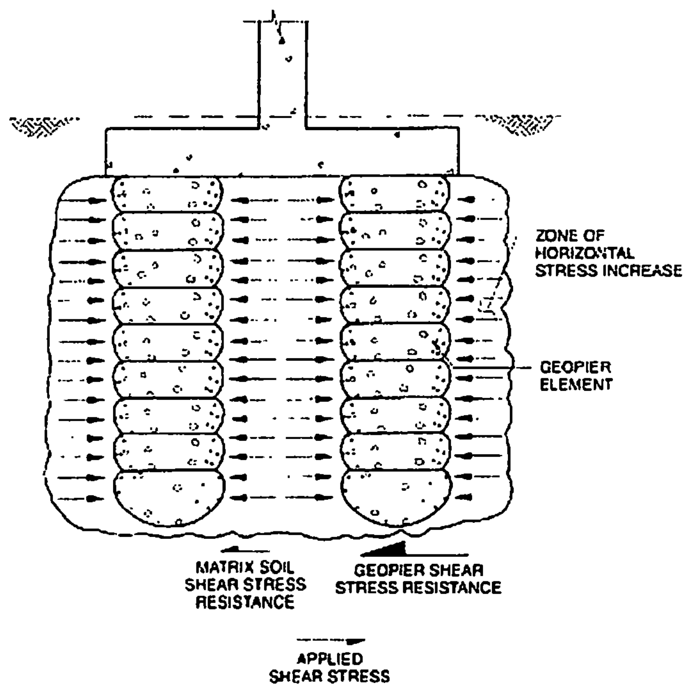
where R_a is the percent area coverage of Geopiers elements below the footing and R_s is the ratio of the stiffness of the compacted Geopier aggregate to the stiffness of the native unimproved soil. Depending on the nature of existing soil deposits, the ratio of the stiffness of Geopier elements to existing soil has been found to range from 8 to 35. This stiffness ratio is even greater for soil that exhibits liquefaction potential.

Using Eq.1, if Geopier elements and the associated primary reinforced zone (6 inches from the Geopier cavity perimeter) cover 60 percent of the footing area and exhibit a stiffness ratio of 10, the shear stress that should occur within the foundation soil will be limited to 16 percent of the average shear stress applied by seismic shaking. If Geopier elements and the associated primary reinforced zone cover 70 percent of the footing footprint area and exhibit a stiffness ratio of 25, the shear stress that should occur within the foundation soil will only be 6 percent of the average shear stress applied by seismic shaking. This reduction in applied shear stress is significant in computing the reduction in liquefaction susceptibility of the Geopier-reinforced soil.

4. Depending on the gradations of the existing soil materials and the Geopier stone, Geopier elements may serve as a drainage path for the dissipation of excess pore water pressure and act as gravel drains.

In summary, the installation of Geopier foundation elements is considered to significantly reduce the potential for soil liquefaction in Geopier-reinforced soil layers. This reduced liquefaction potential results in a significant decrease in the potential for bearing capacity failure and excessive settlement during and following major seismic events. The ductility of Geopier-supported foundation systems allows the Geopier elements to deform with the soil mass and thus provides for greater post-earthquake integrity.

Figure 3. Reduction of Liquefaction Potential.



5. SPECIAL PROVISION FOR USE OF GEOPIER SOIL REINFORCEMENT FOR SEISMIC CONSIDERATIONS

The use of Geopier-supported footings will reduce earthquake-induced shear stress on foundation bearing soils to about 6 to 16 percent of the original shear stress applied by seismic shaking. This will significantly reduce the potential for soil liquefaction and the associated potential for large footing movements of the foundation system.

APPENDIX E - LIST OF REFERENCES

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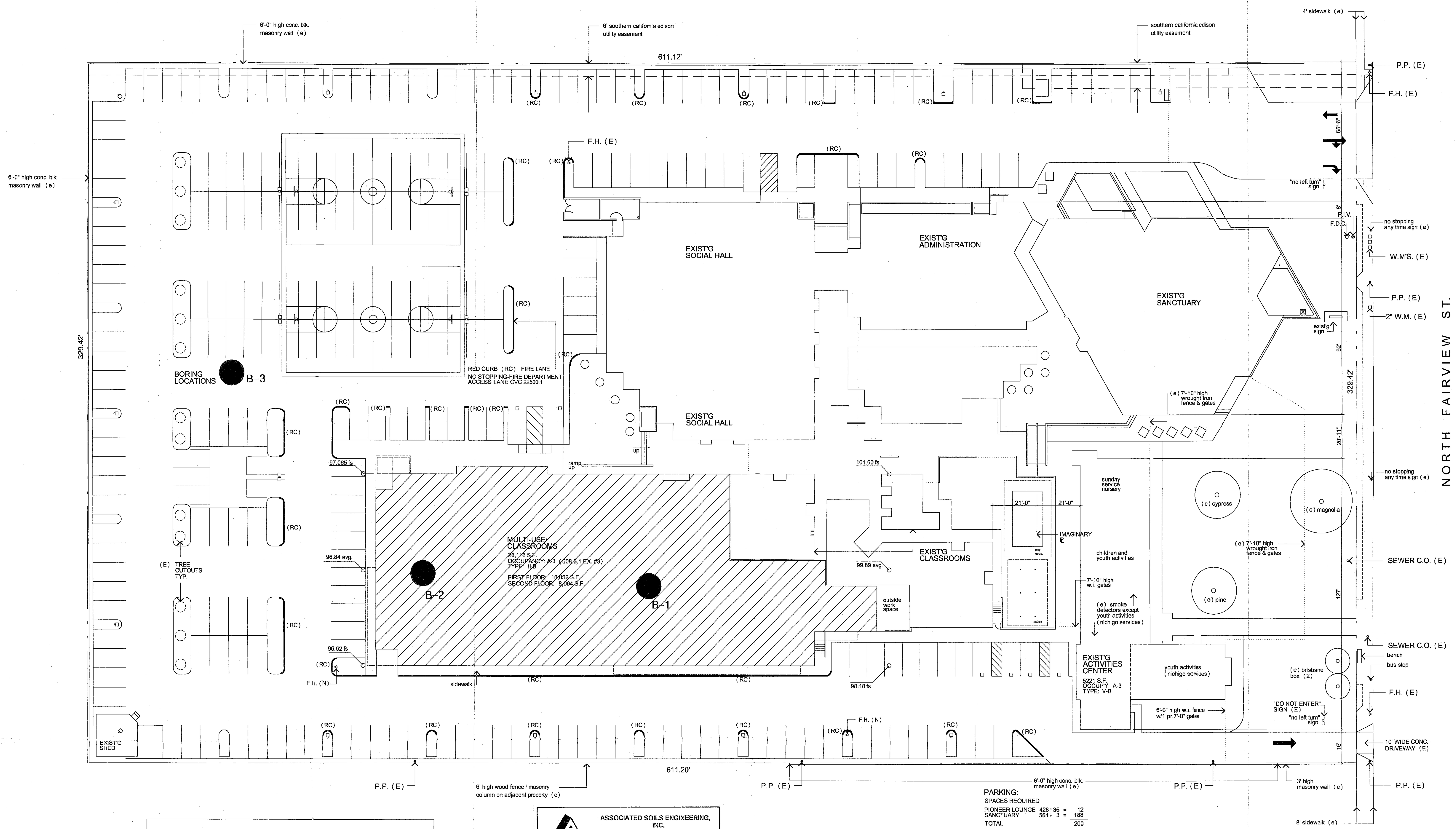

SOILS ENGINEER

THE MAX. LOADS ARE:
WALL = DL 5.5 K/LF
LL 1.0 K/LF
COL. = DL 100K
LL 85K

ASSOCIATED SOILS ENGINEERING, INC.
2860 WALNUT AVENUE
SIGNAL HILL, CA 90755
Ph: (562) 426-7990

BORING LOCATION PLAN

Project No.: 10-6212	Date: June,2010	Plate: A

● Denotes Approximate Boring Location

PARKING:

SPACES REQUIRED

PIONEER LOUNGE	428 ÷ 35 =	12
SANCTUARY	564 ÷ 3 =	188
TOTAL		200

SPACES PROVIDED

REGULAR	260
HANDICAPPED	8
TOTAL	268

LEGEND:

POT ⟶ PATH OF TRAVEL
(e) EXISTING
(RC) RED CURB
PARKING LOT LIGHTS

NOTES:

PANIC HARDWARE ON ALL EXTERIOR DOORS (OFFSET PUSH BARS)
LIGHTS OVER ALL EXTERIOR DOORS

LABONTE' AND ASSOCIATES
ARCHITECTURE . PLANNING
14732 LIVINGSTON ST. TUSTIN CA. 92780 714 838-6236

N

PHASE ONE SITE PLAN

1" = 20' - 0"

WINTERSBURG PRESBYTERIAN CHURCH FEASIBILITY SCHEMATICS