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August 25, 2016

Via e-mail

NOTICE TO BIDDERS – ADDENDUM NO. 2

FIRE DEPARTMENT REGIONAL TRAINING CENTER, PHASE 2
CONTRACT NO. CS-0671
IN THE CITY OF VERNON, CALIFORNIA

This notice shall be considered as Addendum No. 2 to the Plans and Specifications for the above-mentioned project and modifies the original Contract Documents, as noted below. Portions of the Contract, not specifically mentioned in the Addendum, remain in force.

Referenced Documents:

1. Geotechnical Investigation Report: Attached for reference.

If you have any questions, please contact Lissette Melendez of my staff at (323) 583-8811 extension 246.

Sincerely,

Derek Wieske, P.E.
Director of Public Works, Water, & Development Services

DW/lm
Enclosures
c: Deputy City Clerk
Lissette Melendez
Yesenia Ramirez

GEO-ENVIRONMENTAL, INC.
Caltrans Certified
DBE Firm

GEOTECHNICAL INVESTIGATION REPORT

Proposed Fire Department Training Center
3375 Fruitland Avenue
City of Vernon, Los Angeles County, California

GEI-Project No.: 100-18

Prepared for
Sean Nazarie, P.E.
Director of Engineering
CNC Engineering
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May 8, 2015

Prepared by

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GEOTECHNICAL INVESTIGATION REPORT
PROPOSED FIRE DEPARTMENT TRAINING CENTER
CITY OF VERNON, LOS ANGELES COUNTY, CALIFORNIA

Prepared for

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GEI Project No. 100-18

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May 8, 2015

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1.0 INTRODUCTION

This report presents the results of a site exploration performed by Geo-Environmental, Inc. (GEI) at the site located at 3375 Fruitland Avenue in the City of Vernon, Los Angeles County, California (Figure 1). The exploration was performed to provide geotechnical recommendations for the design and construction of the proposed Fire Department training center. The preparation of this geotechnical investigation report was in general accordance with GEI's Proposal No. 14-124 dated December 15, 2014.

2.0 PURPOSE AND SCOPE

The purpose of this geotechnical investigation was to explore the subsurface conditions at the site and to evaluate the adequacy of the on-site soils to provide satisfactory support for the proposed training center. The scope of GEI's work consisted of the following tasks:

- Data review of information;
- Site reconnaissance, the marking of two (2) boring locations, the drilling of two (2) soil borings to depths ranging between five (5) and 51.5 feet, and the collection of drive and bulk soil samples from the borings,
- Geotechnical engineering analysis; and
- Preparation of a geotechnical report presenting our conclusions and recommendations regarding:
  - *Foundation design parameters;*
  - *Characteristics of the subsurface materials encountered;*
  - *Suitability of onsite soils for use as fill material;*
  - *Thickness and competence of existing fill, if any;*
  - *Nature of bedrock, if encountered;*
  - *Groundwater table;*
  - *Historical high groundwater levels at the site;*
  - *Known earthquake faults and geologic-seismic features in the area;*
  - *Seismic coefficient;*
  - *Earthquake induced dynamic ground compaction;*
  - *Liquefaction potential and mitigative measures;*
  - *Liquefaction induced settlements;*
  - *Structural backfill specifications and compaction using excavated or imported material;*
  - *Removal of loose or undesirable material;*
  - *Floor slab support;*

- *Pavement sections; and*
- *Site preparation for proposed construction.*

3.0 PROPOSED CONSTRUCTION

The project will consist of constructing a modular training center. The new center will be located at the rear of Fire Station No. 1 located at 3375 Fruitland Avenue on an approximately 30,000 square-foot unimproved area of land. The center will consist of various modular props and training areas for the Fire Department.

4.0 SITE GEOLOGY

The lithology that underlies the project site consists primarily of marine and non-marine sedimentary rocks with surficial deposits of alluvial materials. These materials consists of Quaternary Alluvium (Q) of the Pleistocene and Holocene age composed of unconsolidated and semi-consolidated marine lake, playa, and terrace deposits. See Figure 3.

5.0 FAULTING AND SEISMICITY

5.1 SIGNIFICANT FAULTS AND SEISMICITY

A list of active and potentially active faults is presented in Appendix C. This list includes the most significant regional and local Quaternary faults within approximately 100 kilometers (km) of the site. Active and potentially active faults are of Holocene and Quaternary ages, respectively, as defined by the California Division of Mines and Geology (CDMG), Notes 3 and 43 (CDMG, 1981 and 1975), respectively. Faults considered capable of generating earthquake magnitudes of M_l 6.0 or less and lying farther than 160 km (100 miles) from the site are not included because they contribute a minor portion of potential ground shaking at the site. The symbol 'M_l' represents Richter or local magnitude and the symbol 'M_W' represents moment magnitude.

A plot of the epicenters of earthquakes that have occurred in the region between 1800 and 2000, generated using the computer program EQSEARCH (Blake, 2000a), is shown in Appendix C. Historical earthquake data was obtained from an abbreviated version of the CDMG computerized earthquake catalog for the State of California. This catalog has been supplemented by earthquake data from Townley and Allen (1939), and the U.S. Geological Survey's Earthquake Data Base System in Denver, Colorado.

The primary seismic sources for the project site are the Elysian Park Thrust Fault, the Compton Thrust Fault, and the Newport-Inglewood (L.A. Basin and Offshore) Fault. A brief description of the major faults considered relevant to the site is given below:

5.1.1 Elysian Park Thrust Fault

A key issue in estimating the earthquake hazard to the Los Angeles area is the hidden subsurface geometry of blind thrust faults. The Elysian Park Thrust has a poorly known width through the seismogenic crust, and is thus at the center of a current controversy over the maximum magnitude event that it could produce. Competing thin and thick-skinned compressional tectonic models allow very different fault widths within seismogenic depths, leaving poor maximum magnitude estimates. The segmentation of the Elysian Park Thrust Fault system is even less well known; the same is true for the other thrusts along the northern margin of the Los Angeles and San Fernando basins that produced the 1971 San Fernando, 1987 Whittier Narrows, and 1994 Northridge earthquakes. The Elysian Park Thrust Fault Zone lies approximately 0.4 miles of the project site.

5.1.2 Compton Thrust Fault

After the Whittier Narrows (1987, $M_l = 6$) and Northridge (1994, $M_l = 6.7$) earthquakes revealed that blind thrust faults threaten metropolitan Los Angeles, an array of 250 continuously recording GPS stations were deployed to detect and monitor the motion associated with the movement of the blind thrust faults (inclined faults that do not reach the surface) and the surface faults. Gerald Bawden and his colleagues analyzed the data from this network and found that groundwater and oil pumping produce motions at some of the GPS sites that obscure, and in some cases minimize tectonic signals expected from the blind thrust faults. By utilizing only the GPS sites that contain tectonic signals, they determined that the tectonic squeezing (4.4 ± 0.8 mm/year or 0.17 ± 0.03 in/year) across Los Angeles will likely produce earthquakes on either the blind Elysian Park, Puente Hills, or Compton Thrust faults. The Compton Thrust Fault Zone lies approximately 4.2 miles of the project site.

5.1.3 Newport-Inglewood (L.A. Basin and Offshore) Fault

The Newport-Inglewood (L.A. Basin and Offshore) Fault Zones, composed of discontinuous faults and folds that presumably overlie a through-going right-lateral strike-slip fault in the basement rock, trends southeastward from near Santa Monica, across the Los Angeles basin, to Newport Beach. Faults having similar trends and projections occur offshore in San Clemente and in San Diego (the Rose Canyon and La Nacion Fault Zones). Altogether, these various faults constitute a system more than 240 km long that extends into Northern Baja California, Mexico. Individual faults range from seven (7) to 18 km long. For the purpose of computer modeling, the zone is divided into two (2) segments: the Newport-Inglewood and the "offshore zone of deformation." The Newport-Inglewood (L.A. Basin and Offshore) Fault Zones lie approximately between 6.5 and 32.7 miles of the project site.

5.2 SEISMIC DESIGN PARAMETERS

The seismic ground motion values have been determined in accordance with 2013 California Building Code (CBC). The parameters S_S and S_1 are 1.085 and 0.377, respectively, according to 0.2 and 1-second spectral response accelerations. The Site Class is D; the site is assigned Seismic Design Category D. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and for 1-second periods, S_{M1}, (adjusted for site class effects) are 1.156 and 0.620, respectively. For the 5% damped design spectral response acceleration at short periods, S_{DS}, and at 1-second periods, S_{D1}, values of 1.156 and 0.620,

respectively, can be used. These parameters are summarized in Table 1.

Table 1 - Seismic Design Parameters (2013 California Building Code)

Description	Value/Code
Site Latitude (decimal degrees)	33.9977
Site Longitude (decimal degrees)	-118.2051
Site Class	D
0.2 Second Spectral Response, S_S	1.085g
1-Second Spectral Response, S_1	0.377g
Maximum Considered Earthquake Spectral Response Acceleration for Short Periods, S_{MS}	1.156g
Maximum Considered Earthquake Spectral Response Acceleration for 1-Second Periods, S_{M1}	0.620g
5% Damped Design Spectral Response Acceleration at Short Periods, S_{DS}	1.156g
5% Damped Design Spectral Response Acceleration at 1-Second Periods, S_{D1}	0.620g
Seismic Design Category ($S_{Ds>0.50g}$ & $S_{D1>0.20g}$)	D

The peak ground acceleration for the site with 10 percent probability of exceedance in 50 years for alluvium conditions is estimated to be approximately **0.44g** (Reference: Figure 3.3 entitled "Los Angeles 7.5-Minute Quadrangle and Portions of Adjacent Quadrangles").

6.0 FIELD EXPLORATION

6.1 SITE RECONNAISSANCE

A site reconnaissance was performed on April 11, 2014 to verify the condition of the subject site and to determine the suitability of the site for the proposed construction. Surficial conditions encountered were as anticipated.

6.2 DRILLING AND SOIL SAMPLING

Field explorations were conducted on April 14, 2015. A total of two (2) auger borings were drilled within the project site (See Figure 2). Depth of the soil borings ranged approximately between five (5) and 51.5 feet below the existing ground surface.

Relatively undisturbed drive samples were collected from the soil borings at specified depths using a 2.5-inch outside diameter, 18-inch long split-barrel lined with one-inch-high brass rings. The sampler was driven using a 140-pound hammer falling 30 inches. The brass rings were carefully removed from the sampler, transferred to a plastic tube, and sealed at both ends with plastic caps to protect and maintain the in-situ moisture content of the soils. SPT samples were collected from the borings at specified depths using an 18-inch SPT. For SPT sampling, the soil material collected was carefully removed from the sampler and transferred to a plastic bag to protect and to maintain the in-situ moisture content. The blow counts from the SPT sampling were utilized to aid in evaluating the liquefaction potential at the subject site. Bulk samples of the surficial soils were collected for subsequent laboratory analysis to evaluate the granulometry, plasticity, compaction characteristics, consolidation and expansion potential, shear strength parameters, corrosion properties, and Resistance-Value (R-value) of the material.

The soils encountered in the borings were classified in accordance with the Unified Soil Classification Systems as shown on plate A-0 in Appendix A. The logs of the borings are presented in Appendix A.

7.0 INFILTRATION TESTING

Upon completion of the drilling of the borehole to a depth of five (5) feet, infiltration tests were performed in the soil boring in general accordance with the procedures and guidelines prepared by the County of Los Angeles. Prior to the infiltration testing, a 2-inch diameter perforated (slotted) casing with a solid end cap was installed inside the borehole to a depth of five (5) feet below the existing ground surface. The annulus between the borehole's wall and the slotted pipe was filled with filter material (clean sand with grain-size diameters greater than 0.02 inches). The annulus above the test section was backfilled with bentonite (low-permeability) material. The borehole was presoaked to a minimum of one (1) foot above the sealing cap. After the presoaking process, the infiltration testing was conducted for approximately one (1) hour. The time interval for measuring the water drop readings was determined in accordance with the County's guidelines (10 minutes).

All measurements were read to the closest 1/10-inch every 10 minutes. The infiltration tests were terminated after the stabilized rates of the four (4) consecutive readings were within 10 percent. GEI computed the infiltration rates of the on-site soils based on the last reading. The test results are shown in Table 2.

Table 2 – Infiltration Test Results

Boring Location	Infiltration Rate (Measured) (in/hr)	*Infiltration Rate (Final) (in/hr)
B-1	252.75	**23.55**

*Included a Reduction Factor

8.0 LABORATORY TESTING

8.1 INTRODUCTION

Laboratory testing on samples of the soils obtained from the borings was performed in accordance with ASTM specifications. Relatively undisturbed, disturbed SPT, and bulk soil samples obtained from the borings were tested in the laboratory to determine in-situ moisture content and dry density, compaction characteristics (maximum dry density and optimum water content) by modified proctor testing, plasticity/expansive characteristics, grain-size distribution, unconfined compression, corrosion and shear strength. The results of moisture content and dry density testing are indicated on the boring logs in Appendix A. The results of all other laboratory testing are presented below and in Appendix B. Brief descriptions of the laboratory testing performed are presented in the following sections.

8.2 IN-SITU MOISTURE CONTENT AND DRY DENSITY

8.2.1 In-Situ Moisture Content

The in-situ moisture content of the soils was determined in accordance with the ASTM D-2216 Test Method. This method involves obtaining the moist weight of the sample and then drying the sample to obtain its dry weight. The moisture content is calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample, and expressing the result as a percentage.

8.2.2 In-Situ Dry Density

In-situ dry density testing was performed in accordance with the ASTM D-2937 Test Method. This method involves obtaining a relatively undisturbed soil sample by driving a thin-walled cylinder into the soil to determine the dry weight and volume of the sample. Simple computations are then performed to determine the dry unit weight of the soil, generally referred to as the in-situ density of that material. The results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A.

8.3 SIEVE ANALYSIS

Representative samples were dried, weighed, and soaked in water until individual soil particles were separated; then the samples were washed on the No. 200 sieve. The portion of the material retained on the No. 200 sieve was oven-dried and then run through a standard set of sieves in accordance with the ASTM D-422 Test Method. The grain-size distribution data are shown in Figures B-1 and B-2 in Appendix B.

8.4 ATTERBERG LIMITS

Atterberg Limits testing was performed on selected samples to evaluate the plasticity characteristics of the soils in accordance with the ASTM D-4318 Test Method. This method involves determination of the liquid limit and plastic limit of Minus No. 40 U.S. Standard Sieve material.

8.4.1 Liquid Limit

The liquid limit is determined by performing trials in which a portion of the specimen is spread into a brass cup, divided in two (2) with a grooving tool, and allowed to flow together by repeatedly dropping the cup in a standard mechanical device. The multi-point method involves a series of trials over a range of moisture contents. The liquid limit (moisture content at which the soil groove closes for a distance of ½-inch in 25 blows) is determined by the plotted data.

8.4.2 Plastic Limit

The plastic limit is determined by alternately pressing together and rolling a portion of the plastic soil into a 1/8-inch diameter thread until the moisture content is reduced to a point at which the thread crumbles and is no longer able to be pressed together and rerolled into a 1/8-inch diameter thread. The moisture content at this stage is reported as the plastic limit. The Plasticity Index is calculated as the difference between the liquid limit and the plastic limit. Test results are presented below in Table 3 and in Figure B-3 in Appendix B.

Table 3 – Atterberg Limits Test Results

Boring No. and Depth	Liquid Limit (LL) %	Plastic Limit (PL) %	Plasticity Index (PI) %	Plasticity Characteristics
B-2 @ 5 feet	NP	NP	NM	Non-Plastic
B-2 @ 10 feet	NP	NP	NM	Non-Plastic
B-2 @ 15 feet	NP	NP	NM	Non-Plastic
B-2 @ 20 feet	NP	NP	NM	Non-Plastic
B-2 @ 25 feet	NP	NP	NM	Non-Plastic
B-2 @ 30 feet	NP	NP	NM	Non-Plastic
B-2 @ 35 feet	NP	NP	NM	Non-Plastic
B-2 @ 40 feet	NP	NP	NM	Non-Plastic
B-2 @ 45 feet	34	26	8	Low
B-2 @ 50 feet	33	15	18	Medium Plastic

- *Plasticity Index = Liquid Limit – Plastic Limit*
- *NM = Limit could not be measured*

8.5 MODIFIED PROCTOR (COMPACTION)

Compaction testing was performed on representative samples of surficial soils in general accordance with the ASTM D1557 Test Method. This method involves placing soil at a selected moisture-content in five (5) layers into a 4-inch diameter mold with a volume of 1/30 cubic foot. Each layer is compacted with 25 blows of a 10-pound hammer dropped from a distance of 18 inches, thus producing a total compactive effort of approximately 56,000 ft.-lb./ft^3. The resulting dry unit weight is determined. The procedure is repeated at

different moisture contents to establish a relationship between the moisture content and dry unit weight of the soil. When plotted, the data produces a compaction curve, from which the maximum dry density and optimum moisture content of the soil can be determined. The in-situ dry density determined is compared to the maximum dry density for the same material to determine the degree of relative compaction. Results of the Modified Proctor testing performed on selected samples of the subgrade soils are presented in Table 3 and in Figure B-4 in Appendix B.

Table 4 – Compaction Test Results

Boring No.	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Water Content (%)
B-1	0-5	Silty Sand (SM)	120.5	10.0

8.6 Direct Shear

Direct shear testing was performed on a selected sample in accordance with the ASTM D3080 Test Method on relatively undisturbed ring samples. This method consists of placement of the soil sample in a direct shear device, application of a predetermined normal stress, provision for wetting or draining of the soil specimen, or both, consolidation of the specimen under the normal stress, and then shearing the specimen at a constant rate of shearing deformation. The shearing force and horizontal displacements are measured and recorded as the soil specimen is sheared. The shearing is continued well beyond the point of maximum stress until the stress reaches a constant or residual value. Typically, three (3) samples of the same material are sheared at different confining pressures. By plotting the shear stress versus the confining pressure for the samples, a best-fit straight line drawn through the data yields the angle of internal friction and the apparent cohesion of the material. Plot of direct shear testing is presented in Figure B-5 in Appendix B. The results are also summarized in Table 5.

Table 5 – Direct Shear Test Results

Boring No. and Depth	Peak Cohesion (psf)	Peak Friction Angle (degrees)
B-1 @ 3.5 feet	489	36

8.7 Resistance Value (R-Value)

R-Value testing of the subgrade soils was performed in the laboratory on selected samples in accordance with California Test Method No. 301. This method involves six (6) steps: (1) preparation of materials for testing, (2) compaction, exudation pressure determination, and measurements of R-Value test specimens, (3) determination of the expansion pressure of R-Value test specimens, (4) measurement of the horizontal pressure and displacement by means of the stabilometer, (5) calculation of the moisture content and density of R-Value test specimens, and (6) determination of the R-Value of the material. The results of the R-Value testing are shown in Table 6.

Table 6 – R-Value Test Results

Boring No. and Depth	By Exudation (TI = 5.0)	By Expansion (TI = 5.0)
B-1 @ 0-5 feet	69	63

8.8 CORROSION POTENTIAL

Corrosivity testing was performed on a selected sample to assess the corrosion potential of the on-site soils. Corrosion test was performed on a soil sample collected at B-1. A soil is considered corrosive if one or more of the following conditions exist (per Caltrans Test Methods):

- Sulfate concentration is greater than or equal to 2,000 ppm (SC>2,000)
- Chloride concentration is greater than or equal to 500 ppm (CC>500)
- pH is less than 5.5 (pH<5.5)
- Minimum resistivity is less than or equal to 1,000 ohm/cm (MR<1,000)

Results of the corrosion test (resistivity, pH, sulfate, and chloride) are described in the following sections:

8.8.1 Sulfate Content

Testing of the sulfate content of the subgrade soils was performed in the laboratory in accordance with California Test Method (CT) No. 417. This method is used for determination of the sulfate content of soils and waters. In this test method, the sulfate ion is precipitated with barium chloride, in an acidic medium, to barium sulfate crystals of uniform size. The barium sulfate present in suspension is determined by a measurement of its turbidity and comparison with a known standardization curve. The results are used to determine the corrosive nature of the environment for concrete structures, as well as for other purposes. The soil sulfate content detected in the soils was 99 ppm. We recommend utilizing Type I Portland cement for concrete in contact with the subgrade soils.

8.8.2 Chloride Content

Determination of the chloride content in the subgrade soils was performed in the laboratory in accordance with CT No. 422. The test method is divided into two (2) parts for determination of the chloride content of waters and the water-soluble chloride content of soils. Part 2 was performed to determine the chloride content of the subgrade soils at the subject site. The chloride content of the soils tested was 198 ppm.

8.8.3 Minimum Resistivity and pH

Testing of the subgrade soils was performed in the laboratory to determine the resistivity and pH. Factors that contribute to corrosion include the presence of soluble salts, soil resistivity, soil pH, and the presence of oxygen.

The resistivity of the soil indicates the relative quantity of soluble salts, while the pH of the soil indicates the degree of acidity or alkalinity. The laboratory resistivity and pH measurements for soil samples are

determined using this test method. The pH of the soils was determined to be 7.2. The minimum resistivity detected was 4,700 ohm-cm.

In general, metal pipes or structural members to be buried below grade should be properly coated or wrapped and sealed with corrosion resistant tar, enamel, or plastic tape for protection against the onsite soils. As appropriate, a qualified corrosion engineer should be consulted for a more in-depth evaluation of the corrosive nature of the onsite soils and for any special corrosion protection design that may be required.

9.0 SITE CONDITIONS

9.1 SUBSURFACE CONDITIONS

Subsurface soils encountered in the borings drilled at the site consisted of mixtures of fine- to coarse-grained sands and silts with occasional clay deposits. These alluvial deposits varied in thickness, density, and consistency. In general, the upper (between 0-5 feet deep) subgrade soils generally existed in a medium dense state. Underlying these soils, the materials encountered generally existed in a medium dense to dense state. Moisture contents varied between 3.0 and 23.3%. The dry unit weight of the onsite soils varied between 96.4 and 120.5 pounds per cubic foot (pcf).

9.2 GROUNDWATER

Groundwater was not encountered at soil boring B-2 drilled up to 51.5 feet below the existing ground surface. **Historically, the highest groundwater table depth measured within the project site has been between 30 and 40 feet below the existing ground surface (Plate 1.2 entitled "Historically Highest Ground Water Contours and Borehole Log Data Locations, South Gate Quadrangle)**. The depth of the groundwater table may fluctuate depending on environmental changes such as heavy rains, injection of water in nearby areas, or dewatering operations in surrounding project site.

10.0 SOIL LIQUEFACTION

10.1 METHOD OF ANALYSIS

A soil liquefaction analysis was conducted in accordance with the guidelines outlined in the CDMG (1997) Special Publication 117 and recommended procedures for its implementation (S.C.E.C., 1999). The project site is located in an area prone to liquefaction during a strong ground shaking generated from an earthquake.

Soil liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose-to-medium dense deposits of saturated soils. Thus, three (3) conditions are required for liquefaction to occur: (1) a cohesionless soil of loose-to-medium density; (2) a saturated condition; and (3) rapid large-strain cyclic loading, normally generated by strong earthquake motions.

10.2 Impact of Soil Liquefaction

Loose, saturated sandy and silty soils tend to liquefy if subjected to strong ground shaking such as that generated by an earthquake. Soil liquefaction results in a very significant loss of shear strength in these materials, sometimes causing uncontrolled lateral flow and large seismic deformations. Consequently, soil liquefaction often causes severe damage to structures during earthquakes. A large amount of the damage in the Simi and San Fernando Valleys during the Northridge earthquake occurred because of liquefaction in loose, saturated sandy silty soils underlying numerous structures, both single-family homes and multi-storied buildings (EERC, 1994).

10.3 Evaluation of Soil Liquefaction Potential

GEI evaluated soil liquefaction potential using an updated version of the so-called "simplified procedure" (Seed et al., 1982; updated by Seed and coworkers (Seed et al, 1983, 1985, 1986a,b, 1988, 1990 and 2003) and the participants of the National Center for Earthquake Engineering Research (NCEER) Workshop on Liquefaction (Youd and Idriss, 1997), as well as the guidelines for evaluating seismic hazards in California (Special Publication 117, by the California Department of Conservation, CDMG, 1997). The simplified procedure is based on the results of field testing (SPT data) and an empirical boundary between levels of ground shaking that have (and have not) produced liquefaction during past earthquakes.

The analysis is based upon the soil data collected during GEI's field exploration and the seismic hazard evaluation presented in the seismicity section of this report. **An estimated peak horizontal ground acceleration at the surface of 0.44g and a magnitude of M 6.7 were used in the evaluation of liquefaction potential. The water table level was assumed to be at 30 feet below the existing ground surface.** The potential for liquefaction was analyzed for exploratory boring B-2 drilled at the site.

The liquefaction study indicated that the soils layers existing between depths of approximately 41.5 and 51.5 feet below the ground surface have factors of safety of less than 1.3 against occurrence of liquefaction. These soils layers may, therefore be considered susceptible to liquefaction and liquefaction related seismic hazards (e.g., dynamic settlement) during the design seismic event.

10.4 Seismic Settlement

As a result of liquefaction, the proposed foundation of the proposed training center may be subjected to several hazards including liquefaction-induced settlements. In order to estimate the amount of post-earthquake settlement, the method proposed by Wu and Seed (2004) was used in which the seismically induced cyclic stress ratios and corrected N-values are related to the volumetric strain of the soil. The amount of settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density of the soils. Under the current conditions, a maximum post-earthquake settlement of approximately 1.7 inches was calculated for the site. Based on the guidelines presented in California Division of Mines and Geology (CDMG) Special Publication 117 (1997a) and considering the relatively non-uniform subsurface stratigraphy across the site, we estimate a maximum localized differential settlement approximately between 0.8 and 1.3 inches.

In addition, dry soils (soils above the assumed groundwater table) with low density of softer consistency tend to

undergo a certain degree of compaction during a seismic event. Earthquake shaking often induces significant cyclic shear strain in a soil mass which responds to the vibration by undergoing volumetric changes. Volumetric changes in dry soils take place primarily through changes in the void ratio (usually contraction in loose or normally consolidated soft soils, and dilation in dense or overconsolidated stiff soils) and secondarily through particle reorientation. Such volumetric changes are generally non-recovered. The likely settlement induced by dynamic compaction of relatively dry soil layers at the site (i.e., within the upper 30 feet) was calculated using the method proposed by Wu and Seed (2004). Under the current conditions, the site soils up to a depth of approximately 30 feet are estimated to undergo a post-earthquake settlement of up to approximately one (1) inch. Considering the relatively non-uniform susbsurface stratigraphy across the site, we estimate a maximum localized differential settlement approximately between 0.5 and 0.75 inches.

11.0 CONCLUSIONS AND RECOMMENDATIONS

11.1 General Conclusions

Based on our field observations and the results of laboratory tests performed on soil samples obtained from the site, it is our opinion that the subject site is suitable for support of the proposed training center building, provided the recommendations presented in this report are implemented in the design of foundation system and into the construction phase of the development. The primary geotechnical issue is the potential for soil liquefaction at depths between 41.5 and 51.5 feet below the existing grade in the event of strong ground shaking during an earthquake.

To mitigate the potential problems due to soil liquefaction, we recommend constructing a two (2) feet thick bi-axial geogrid-reinforced engineered fill under the foundation system of the building to minimize earthquake-induced differential settlements. The proposed reinforcement should consist of two (2) layers of high-tensile strength bi-axial geogrid reinforcement. The geogrids should be placed at approximately 12 and 24 inches below the bottom of the foundation system (footings) [See Figure 4]. The geogrid reinforcement is highly effective and delivers immediate soil geogrid stress transfer ensuring minimal movement of soil structure. Consequently, the differential settlements induced in the foundation system, which generally cause more damage to the building structure than uniform settlements, are greatly minimized. The layers of the bi-axial geogrid reinforcement should be overlapped at least 12 inches.

A shallow spread footings foundation system for the proposed training center building must be designed to withstand the estimated ground deformation associated with soil liquefaction during a major earthquake within the allowable stress limits of the structure. Liquefaction of the soils could lead to settlement of up to 1.7 inches in the potentially liquefiable stratum resulting in approximately 0.8-inch settlement at the spread footings level. (Geogrid reinforced fill should prevent the occurrence of unacceptable level of differential settlement). Although liquefaction of the soils at deeper depths could lead to temporary reduction in the bearing capacity of the affected soils, the potential for gross bearing failure of the soils beneath the conventional spread footings, in our opinion, is low.

The deformation associated with soil liquefaction could also disrupt safe performance of the underground pipes and utilities. The disruption from ground displacements on underground pipes and utilities may be reduced by providing flexible joints at connections and at the proposed structure. All gas lines should be

equipped with automated emergency shut-off valves.

The existing fill materials are generally low in expansion potential and consequently do not require special measures to be undertaken in the design of footings or slab-on-grade to withstand expansion pressure.

11.2 Site Preparation and Earthwork

11.2.1 Cleaning and Grubbing

Prior to grading, the areas to be developed should be cleared of all debris. All buried obstructions, such as old footings, abandoned utilities, and tree roots should be removed. All deleterious material generated during the clearing operations should be removed from the site. Live or active utilities should be located, protected, and/or rerouted.

Prior to excavating for the development, all organic materials and vegetation (if any) should be removed from the construction area and disposed of the site. If any cesspools, septic tanks, or buried structures are encountered, they should be removed in their entirety. Any utilities, whether active or inactive, should be identified and removed from the site, or relocated per project plans and specifications. Any cavities resulting from the removal of any existing buried structures, foundations, or utility lines should be properly backfilled as recommended in the Subgrade Preparation section of this report, and compacted as recommended in the Placement and Compaction of Fills section of this report. However, as an alternative, these cavities can be backfilled with two-sack cement slurry (two (2) sacks of cement Type I or II per cubic yard with a maximum slump of 5.0 inches).

11.2.2 Construction of Engineered Fill for Building Structure

Prior to constructing the engineered fill, all deleterious material and debris should be removed from the site. The bottom of the excavation should be observed and approved by GEI, and any unsuitable material exposed should be removed as required. The bottom of the excavations should be prepared in accordance with Section 11.2.3. The excavations should extend five (5) feet laterally beyond the building layout (i.e., a 1:1 (H:V) projection from the edge of the footings/slabs) to a minimum distance equal either to the total depth of overexcavation of the building pad or to the depth of overexcavation below the bottom of the footings/slabs, whichever is greater.

The first layer of high-tensile strength bi-axial geogrid should be placed on top of the approved bottom of the excavations. The backfill soils should be processed to remove deleterious material, reworked (uniform mixture and well-moisture conditioned) and placed on top of the first layer of geogrid. The second layer of high-tensile strength bi-axial geogrid should be placed within the engineered fill approximately 12 inches below the bottom of the foundation system (See Figure 4). We recommend that fill materials be placed in 8-inch thick loose lifts and compacted to at least 95 percent of the maximum dry density, with a water content above optimum, as determined in accordance with ASTM Test Method D1557.

Observation and testing during earthwork should be performed by GEI to verify that no deleterious materials are present in the subgrade soils, the required degree of relative compaction at water content above optimum has been achieved and that the geogrid reinforcement has been placed properly.

Based on the results of our exploratory soil borings and laboratory testing, the soils encountered within the upper five (5) feet generally exhibit moisture contents well below the optimum moisture content for that material. The low moisture content may cause problems in the achievement of the required relative compaction in the soils using typical rubber-tired equipment or steel track equipment. However, if saturated soils are found during grading operations, drying of such soils will be required to minimize pumping conditions and to facilitate compaction. The use of heavy equipment is not recommended if saturated soils are encountered within the proposed depths of recompaction for the building pad and pavement areas. The use of this equipment could cause "pumping" in the soils; therefore, we strongly recommend use of light-to-medium-sized equipment, such as small vibratory or roller to minimize the pumping conditions if saturated materials are encountered in any of the excavations for the building pad and pavement areas. If the pumping conditions are induced by this type of equipment, the stabilization of the subgrade soils by using geotextile fabric and crushed rock will probably be required as an alternative.

11.2.3 Preparation of Over-Excavation Bottom

All exposed subgrade soil surfaces, including the over-excavation bottom for the building pad, should be observed by a representative of GEI prior to the placement of fill. If the exposed bottoms of the subgrade soils are predominantly "cohesionless" (sandy material), they should be scarified to a depth of approximately six (6) inches, moisture-conditioned above the optimum moisture content, and recompacted to dry densities corresponding to at least 95 percent of their maximum densities in accordance with ASTM D1557.

11.2.4 Excavation for Utility Trenches

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill. This is especially important for deeper fills. We recommend that all known utilities be shown on the grading plan. The actual depths of removal will need to be determined during grading in the field by a GEI's Soils Engineer.

11.2.5 Protection of Excavations

Temporary construction excavations may be made vertically without shoring to a depth of five (5) feet below adjacent grade. For deeper cuts, the slopes should be shored or sloped back 1:1 (H:V). No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the toe of the excavation or five (5) feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site facilities should be properly shored to maintain support of adjacent elements. All excavations and shoring systems should meet the minimum requirements given in the most current California Occupational Safety and Health Administration (Cal/OSHA) Standards.

11.2.6 Difficulty of Excavations

Due to the nature of the existing soils, difficulty in excavation operations within the project site is not expected. Use of medium-to-heavy equipment (such as loaders or equivalent) is recommended to perform the operations. A representative of GEI should be present during the excavations.

11.2.7 Potential for Caving

The existing soil materials at the site could cave-in. Flattened excavation slopes should be sufficient to mitigate caving of shallow excavations, although deeper excavations will require some form of external stabilization, such as shoring or bracing. Maintenance of adequate moisture content within the removed and recompacted soil materials is recommended to improve excavation stability.

11.2.8 Fill Materials

Fill materials should consist of onsite soils and/or an approved import material. The onsite soils encountered during the geotechnical investigation are acceptable for use as fill material for this project. Before being used as fill, onsite soils should be cleaned of all vegetation and materials with any dimension larger than four (4) inches. <u>If import material is required for use as fill for this project, it should be predominantly granular (35 percent or less passing the no. 200 sieve), non-expansive (EI less than 20 and/or PI less than 12), and should be free of organic or inorganic debris, contamination, and materials with any dimension larger than four (4) inches.</u> Import material should be tested and approved by GEI prior to importation to the job site. *GEI should be notified a minimum of two (2) working days prior to the scheduled importation of the soil to the project site.*

If the Contractor intends to use crushed or inert demolition debris (concrete, brick, and/or asphalt) for fill material, these processing requirements should be followed:

- *If the crushed material meets gradation, R-Value, and durability requirements for Crushed Miscellaneous Base (CMB) per Greenbook ("Standard Specifications for Public Works Construction"), it can be used as a base course.*

- *If the inert debris is crushed to a well-graded mixture with a maximum particle size of one (1) inch, even if it does not meet CMB criteria, the crushed material may be used directly as non-expansive fill, without further blending.*

- *Inert debris up to a maximum size of four (4) inches may also be used in fills, provided it is thoroughly blended with soil to form a well-graded mixture. In general, at least two (2) volumes of soil will be required for each volume of debris.*

The use and blending of inert materials into the fills should be monitored by GEI's Soils Engineer and/or his representative.

11.2.9 Placement and Compaction of Fills

Structural fill should be used for the support of all foundation elements and may only be placed on approved surfaces/subgrades prepared in accordance with Section 11.2.3 of this report. Fill should be placed in loose lifts, moisture-conditioned above the optimum moisture content, and mechanically compacted. If imported soil is used within the building, water or electrical equipment pads, it should be placed in lifts of uniform thickness across the entire pad (i.e., existing onsite soils should not be used for a portion of the building pad while import fill is used for other portions of the building pad).

Subgrade soils placed in over-excavation areas should be compacted to a minimum of 95 percent relative compaction as determined by the ASTM D1557 Test Method. Fill soils placed in other areas (such as sidewalks and hardscaped areas) should be compacted to a minimum of 90 percent relative compaction as determined by the ASTM D1557 Test Method. Any aggregate base (AB) should be moisture-conditioned between optimum and two (2) percent above optimum moisture content, and compacted to a minimum of 95 percent relative compaction. Fill compaction should be tested and recorded by a representative of GEI. Densification by flooding or jetting is not permitted. Compacted fill soils should not be allowed to dry out and should be kept moist (between optimum moisture content and two (2) to three (3) percent above the optimum moisture content) until covered by subsequent construction.

11.2.10 Lift Thickness and Compaction Equipment

The optimum lift thickness can be determined depending upon the type of equipment used for compaction. The following table shows the required uncompacted lift thickness for the backfill material depending upon the type of equipment used in the field.

Table 8 – Uncompacted Lift Thickness for Backfill

Type of Equipment	Lift Thickness (in.)
Small Vibratory Hand-Held Compactors	4 to 6
Medium-Sized Vibratory Compactors or Static Rollers	6 to 8
Scrapers and Heavy Equipment (Loaders and Dozers)	8 to 12

11.2.11 Trench Backfill

Trench excavations for utility pipes may be backfilled with onsite soils under the observation of a representative of GEI. After utility pipes have been laid and properly bedded, the space around the pipe should be backfilled with clean sand (having a sand equivalent of 30 or greater) or gravel to a depth of approximately one (1) foot over the top of the pipe, before the controlled backfill is placed. Controlled backfill should be moisture-conditioned, placed, and compacted in accordance with the recommendations presented in Section 11.2.9 of this report.

If backfill areas are located where mechanical compaction of soil backfill is impractical due to space constraints, two-sack cement slurry may be substituted for compacted backfill. The slurry should contain two (2) sacks of cement per cubic yard and should have a maximum slump of five (5) inches.

11.2.12 Earthwork Monitoring – Construction Observation and Testing

A representative of GEI should observe all soil improvement operations and foundation installations, as well as near-surface soil treatment, including excavations, subgrade preparation, and fill placement activities. In addition, in-place field density testing should be performed during fill placement and compaction to evaluate the overall compaction of the soils. Soils that do not meet the minimum compaction requirements should be reworked and tested prior to placement of any additional fill.

11.3 FOUNDATION DESIGN

We recommend that the shallow foundation system for the proposed training center building consist of conventional spread footings, which can tolerate the anticipated uniform settlement that might occur as a result of soil liquefaction in the event of a major earthquake as discussed above in Section 10. The conventional spread footings should be placed on top of the reinforced engineered fill prepared in accordance with recommendations contained in Section 11.2.2 of this report.

The bottom of the interior footings should extend to a minimum depth of 12 inches below the finished floor level; exterior footings should extend to a minimum depth of 18 inches below the lowest adjacent finished grade. These footings may be designed for an allowable bearing pressure of 3,000 pounds per square foot (psf). The recommended maximum allowable bearing capacity includes a factor of safety of at least three (3) against shear failure and can be increased by one third (1/3) to resist transient load such as those caused by earthquake and winds. Continuous perimeter footings should be used and founded at least 18 inches below lowest adjacent ground surface. Footings should be reinforced with at least two (2) no. 4 bars at the top and also at the bottom.

For properly constructed footings placed in engineered fill, the total static settlement caused by structural loads typical of the wood frame structures is estimated to be less than one (1) inch. Most of the settlement of the proposed structure is expected to occur as the loads are applied during construction or shortly thereafter.

A passive resistance of 250 pounds per square foot (psf) per foot of embedment and a coefficient of friction of 0.4 for use with the dead loads may be utilized in computing lateral capacity. The coefficient of friction should be applied to net dead normal loads only for design against lateral forces.

11.4 CONCRETE SLAB CONSTRUCTION

Concrete slabs should have a minimum thickness of four (4) inches actual and should be reinforced with 6" x 6" - (#6 x #6) W2.9 x W2.9 welded wire mesh or steel bars no. 3 spaced at 18 inches on centers each way. All slab reinforcement should be properly supported to ensure the desired placement at mid-height. In areas where moisture-sensitive floor coverings are anticipated over the slab, we recommend the use of a polyethylene vapor barrier with a minimum of 6.0 mil thickness to be placed beneath the slab. The moisture barrier should be overlapped or sealed at splices and covered top and bottom by a 1-inch to 2-inch minimum layer of clean, moist (not saturated) sand to aid in concrete curing and to minimize potential punctures.

11.5 PAVEMENT DESIGN

For the proposed pavement design, we utilized an R-value of 63 and a Traffic Index (TI) of five (5). These values were used to calculate the required pavement sections using the procedures and design criteria described in the California Division of Highways Design Method (*Highway Design Manual, May 7, 2012*). The results of the calculations are presented in Table 9.

Table 9 – Recommended Section Thicknesses

R-Value	TI	Recommended Alternatives			
		Construction			
		Alternative 1		Alternative 2	Alternative 3
		AC	AB*	FDAC	PCC**
63	5.0	3.0	4.0	6.0	4.0

Design Life of Alternative 1= 20 years, FDAC = Full Depth Asphalt Concrete
**Class II Aggregate Base (R-Value of 80)*
***PCC = Portland Cement Concrete*
NOTES: For Alternative 3: Contraction transverse joints should be spaced not more than 15 feet on centers to control shrinkage cracking. PCC section should be reinforced with steel bars no. 3 spaced at 12 inches on centers each way. Reinforcement should be placed mid-height of the PCC slab.
Life expectancy may increment depending on several factors such as traffic application, environment conditions (temperature and moisture), subgrade properties (corrosion), future maintenance, and rehabilitation.

11.6 Drainage

The area should remain adequately drained toward the street or to an approved drainage system. Inlet structures should be maintained to ensure their proper function.

To enhance future site performance, it is recommended that water be collected and directed away from structures to disposal areas. It is important that drainage patterns established at the time of fine grading be maintained throughout the life of the structures. Significant variations in the moisture condition of the subgrade soils due to summer heat may cause distress to the foundation system and cracking of the concrete slabs.

11.7 Exterior Concrete and Masonry Flatwork

Exterior concrete and masonry flatwork should be supported on at least two (2) feet of compacted non-expansive fill. This includes exterior sidewalks, stamped concrete, non-traffic pavement, pavers, etc. Prior to placement of concrete, the subgrade should be prepared as recommended in this report.

11.8 Plan Review and Construction Observation and Testing

GEI should be retained to review the finished grading earthwork specifications for conformance with the intent of our recommendations. The review will enable GEI to modify recommendations should final design conditions be different then presently understood.

During construction, GEI should provide field observation and testing to check that the excavation, site preparation, and fill placement conform to the intent of our recommendations, as well as to the project plans and specifications. This would allow GEI to develop supplemental recommendations as appropriate for the actual soil conditions encountered and the specific construction techniques employed by the Contractor.

12.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GEI's efforts were prepared exclusively for use by CNC Engineering and their consultants in designing the proposed training center and other improvements. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed expansion, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

This report presents recommendations for the subject site based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the referenced reports and during our recent site investigation. The possibility of different local soil conditions cannot be discounted. It is the responsibility of CNC Engineering or its representative to bring any deviations or unexpected conditions observed during construction to the attention of GEI. In this way, any required supplemental recommendations can be made with a minimum of delay to the project.

If necessary, construction should be observed and tested by GEI during the following construction stages.

- *Excavation, subgrade preparation, and fill soil compaction;*
- *Foundation excavation and construction;*
- *Trench backfilling; and*
- *When any unusual subsurface soil conditions are encountered.*

It is the responsibility of the property owner or his/her authorized representative to review the recommendations made by GEI and to authorize the Contractor to perform such work as required to comply with such recommendations. The Contractor and/or CNC Engineering shall have the responsibility to inform GEI of the starting date of construction and the anticipated period during which testing and/or observation by GEI will be needed. Any backfill performed without testing as specified by GEI and the latest County of Los Angeles Building Code cannot be approved by GEI.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time, and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, expressed or implied, is included or intended in our report.

13.0 REFERENCES

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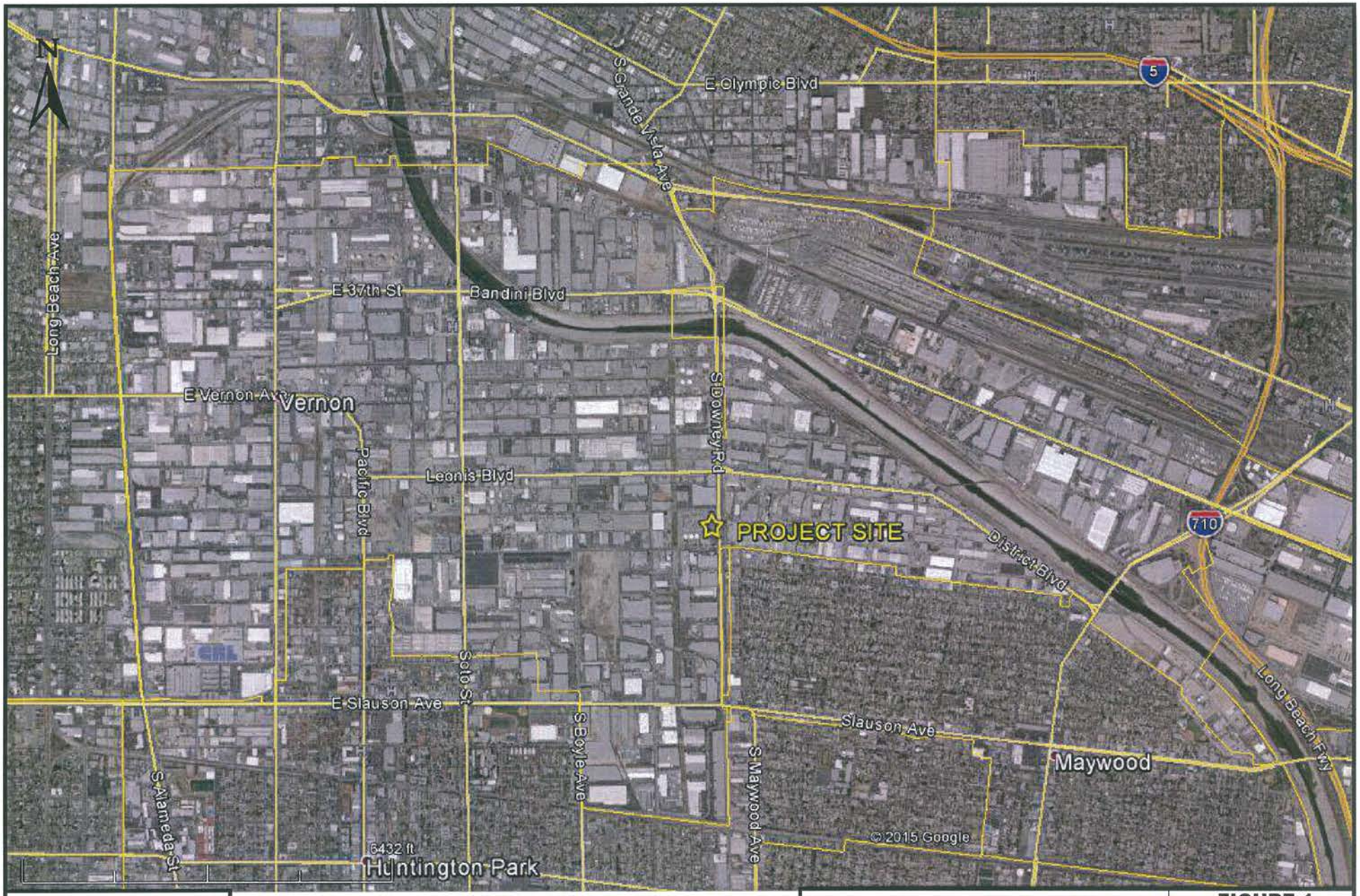
ILLUSTRATIONS



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Caltrans/DSA/City of Los Angeles/AASHTO Certified

SBE/CBE/MBE/DBE Firm

VICINITY MAP

Geotechnical Investigation Report
Proposed Fire Department Training Center
City of Vernon, Los Angeles County, California

FIGURE 1	
DRAWN BY:	RCF
APPROVED BY:	FHS
PROJECT NO:	100-18
DATE:	5/5/2015

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LEGEND:

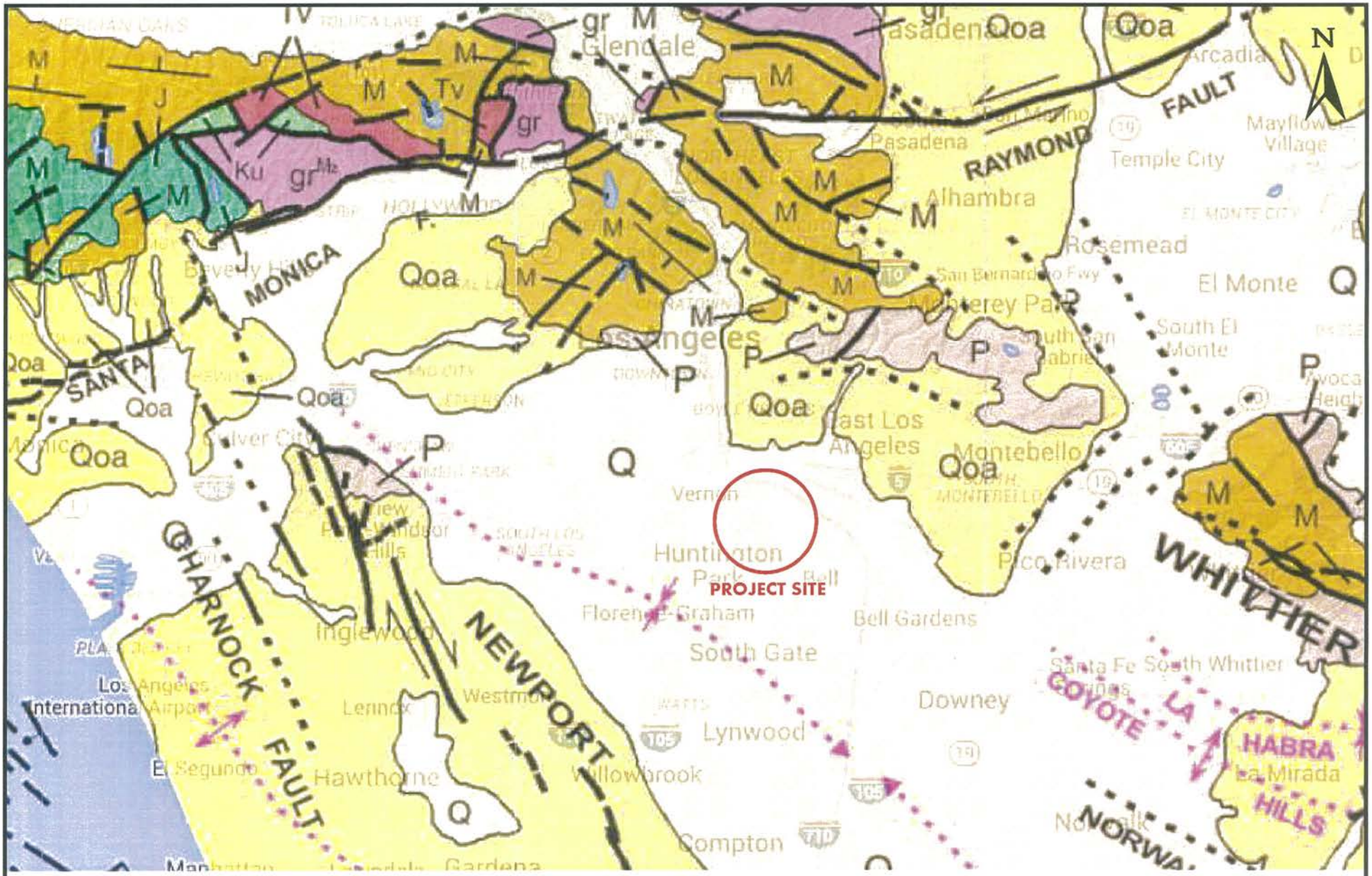
- **B-1** *(Soil Boring used for Percolation Testing)*
- **B-2**

BORING LOCATION MAP

Geotechnical Investigation Report
Proposed Vernon Fire Department Training Center Project
City of Vernon, Los Angeles County, California

FIGURE 2

DRAWN BY:	RCF
APPROVED BY:	FHS
PROJECT NO:	100-18
DATE:	4/14/2015



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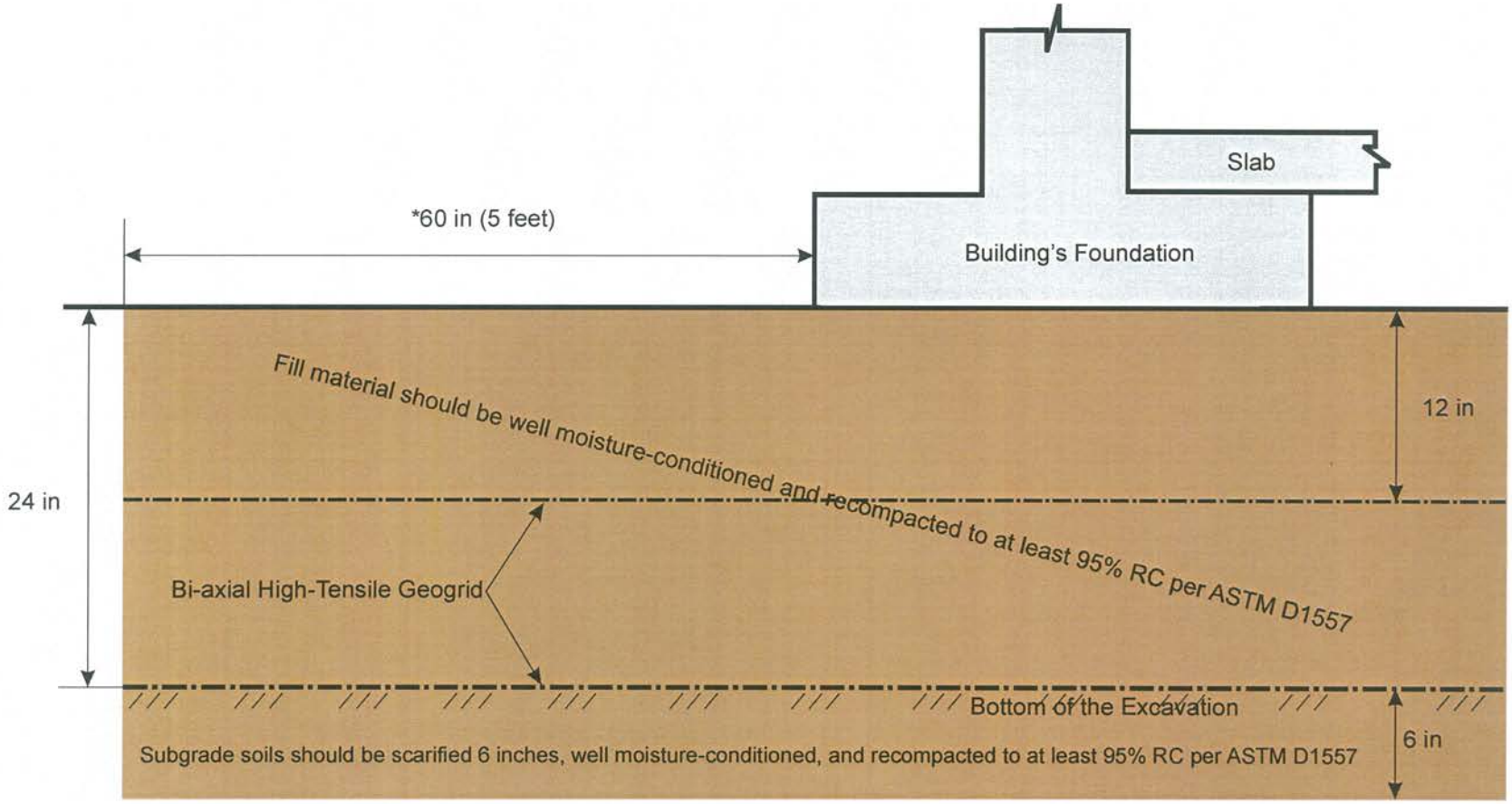
GEOLOGIC MAP

Geotechnical Investigation Report
Proposed Vernon Fire Department Training Center Project
City of Vernon, Los Angeles County, California

FIGURE 3	
DRAWN BY:	RCF
APPROVED BY:	FHS
PROJECT NO:	100-18
DATE:	4/14/2015



*The excavations should extend five (5) feet laterally beyond the building layout (i.e., a 1:1 [H:V]) projection from the edge of the footings/slab to a minimum distance equal either to the total depth of overexcavation of the building pad or to the depth of overexcavation below the bottom of the footings/slabs, whicever is greater.

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REINFORCED ENGINEERING FILL

Geotechnical Investigation Report
Proposed Vernon Fire Department Training Center Project
City of Vernon, Los Angeles County, California

FIGURE 4	
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PROJECT NO:	100-18
DATE:	4/14/2015

APPENDIX A

BORING LOGS

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Figure: A-0

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LOG OF BOREHOLE B-1

Date Drilled: 4/14/15 | Logged by: DT | Project Manager: FHS

Equipment: 8" Hollow Stem Auger | Driving Weight and Drop: 140 lbs @ 30" drop

Surface Elevation(ft): Ground Surface | Depth to Water(ft): Not Encountered

ELEVATION(ft)	DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVE	SAMPLES BULK	SAMPLE NO.	BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
			SILTY SAND (SM), dark olive brown, dry to damp, medium dense to dense, fine-grained sand, non-plastic fines							
	1									
	2									
	3		- Light olive brown							
			- Mostly fine grained sand				15	4.4	96.4	EI CORR COMP R-VAL
	4									
	5									
			Borehole terminated at 5 feet below the existing ground surface. No groundwater was encountered during drilling operations. Upon completion of the drilling of the borehole, infiltration tests were performed in general accordance with the procedures and guidelines prepared by the County of Los Angeles.							

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Proposed Fire Department Training Center
City of Vernon, CA
GEI Project No.: 100-18

Figure A-1

LOG OF BOREHOLE B-2

Date Drilled: 4/14/15 | Logged by: DT | Project Manager: FHS

Equipment: 8" Hollow Stem Auger | Driving Weight and Drop: 140 lbs @ 30" drop

Surface Elevation(ft): Ground Surface | Depth to Water(ft): Not Encountered

ELEVATION(ft)	DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVE	SAMPLES BULK	SAMPLE NO.	SPT BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
	1		**SILTY SAND (SM)**, olive brown, dry, loose, fine- to medium-grained sand, non plastic fines							
	2									
	3									
	4									
	5		- Medium dense	X			13	3.2	106.0	AL GS DS
	6									
	7									
	8									
	9									
	10		**POORLY GRADED SAND with SILT (SP-SM)**, olive brown, moist, medium dense, mostly medium-grained sand	X			14	12.8	---	AL GS
	11									
	12									
	13									
	14									
	15		**WELL-GRADED SAND with SILT (SW-SM)**, olive brown, moist, very dense, medium-grained sand	X			52	10.6	120.5	AL GS
	16									
	17									
	18									
	19									
	20		**POORLY GRADED SAND with SILT (SP-SM)**, olive brown, moist, medium dense, fine- to coarse-grained sand, 6 inches of gravelly sand	X			24	10.4	---	AL GS
	21									
	22									
	23									
	24									
	25		- Fine to medium-grained sand, dense	X			40	8.6	111.1	AL GS
	26									
	27									
	28									
	29									

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City of Vernon, CA
GEI Project No.: 100-18

Figure A-2a

LOG OF BOREHOLE B-2

Date Drilled: 4/14/15 | Logged by: DT | Project Manager: FHS

Equipment: 8" Hollow Stem Auger | Driving Weight and Drop: 140 lbs @ 30" drop

Surface Elevation(ft): Ground Surface | Depth to Water(ft): Not Encountered

ELEVATION(ft)	DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVE	SAMPLES BULK	SAMPLE NO.	SPT BLOWS/FOOT	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
	31		**SILTY SAND (SM)**, dark olive brown, very moist, medium dense, fine-grained sand	X			19	23.3	---	AL GS
	32									
	33									
	34									
	35		**POORLY GRADED SAND with SILT (SP-SM)**, olive brown, damp, very dense, fine- to medium-grained sand	X			57	3	108.3	AL GS
	36									
	37									
	38									
	39									
	40		**SILTY SAND (SM)**, olive brown, very moist, dense, fine- to medium-grained sand	X			38	17.0	---	AL GS
	41									
	42									
	43									
	44									
	45		- Dark olive brown, medium dense, mostly fine-grained sand, low plastic fines	X			16	18.9	110.8	AL GS
	46									
	47									
	48									
	49									
	50		**SANDY CLAY (CL)**, strong brown to dark brown, very moist, very stiff, medium plastic fines, fine-grained sand	X			17	20.8	---	AL GS
	51									

Borehole terminated at 51.5 feet below the existing ground surface.
No groundwater was encountered during drilling operations.
Borehole was backfilled and compacted with with soil cuttings.

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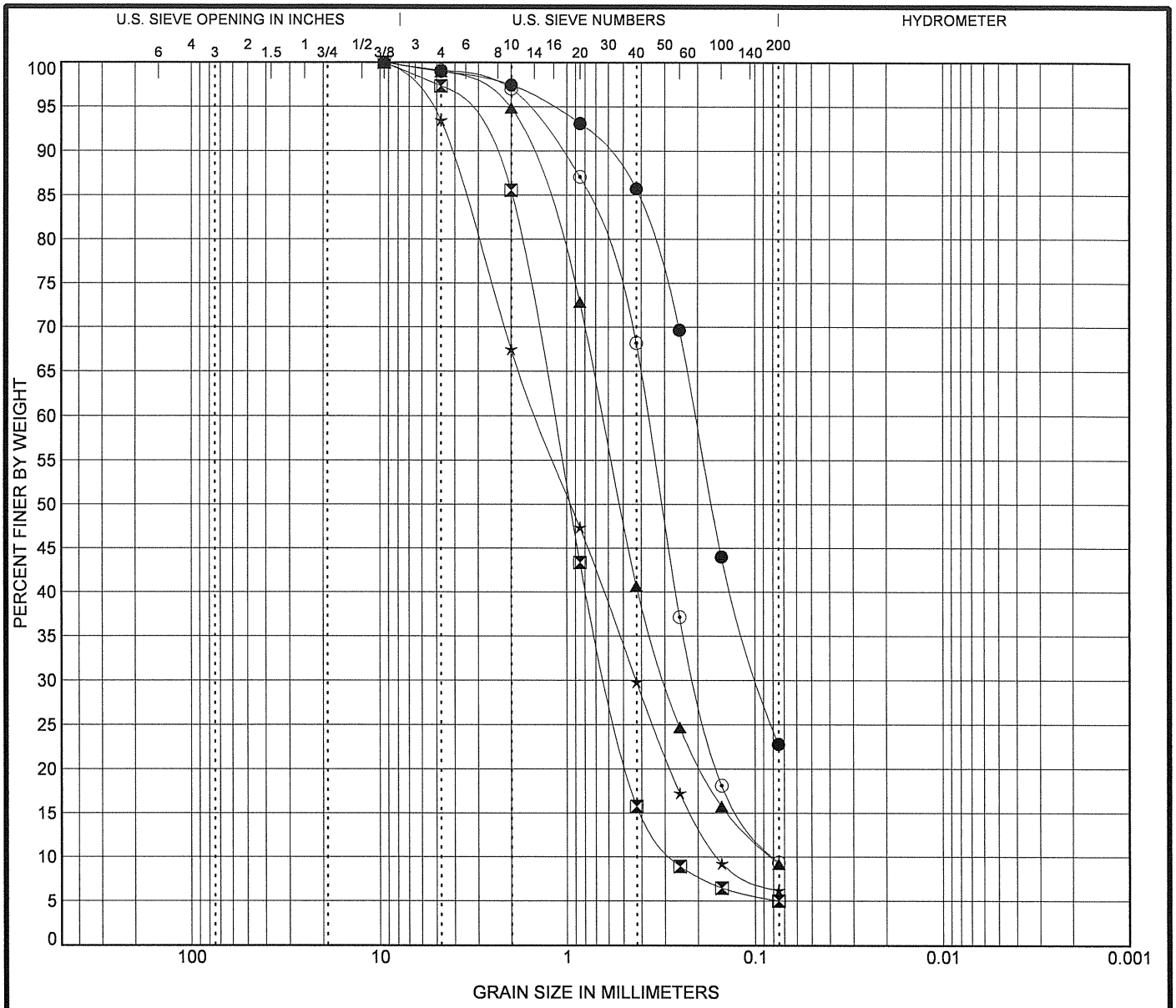
Proposed Fire Department Training Center
City of Vernon, CA
GEI Project No.: 100-18

Figure A-2b

APPENDIX B

LABORATORY TEST RESULTS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	B-2 @ 5.0 ft.	SILTY SAND (SM)	NP	NP	NP		
⊠	B-2 @ 10.0 ft.	POORPLY GRADED SAND with SILT (SP-SM)	NP	NP	NP	1.14	4.38
▲	B-2 @ 15.0 ft.	WELL-GRADED SAND with SILT (SW-SM)	NP	NP	NP	1.69	7.89
★	B-2 @ 20.0 ft.	POORPLY GRADED SAND with SILT (SP-SM)	NP	NP	NP	0.80	9.25
⊙	B-2 @ 25.0 ft.	POORPLY GRADED SAND with SILT (SP-SM)	NP	NP	NP	1.46	4.69

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2 @ 5.0 ft.	9.5	0.206	0.095		1.0	76.2	22.8	
⊠	B-2 @ 10.0 ft.	9.5	1.191	0.608	0.272	2.7	92.3	5.0	
▲	B-2 @ 15.0 ft.	9.5	0.645	0.298	0.082	1.0	89.8	9.2	
★	B-2 @ 20.0 ft.	9.5	1.455	0.428	0.157	6.5	87.3	6.2	
⊙	B-2 @ 25.0 ft.	9.5	0.37	0.206	0.079	0.9	89.7	9.4	

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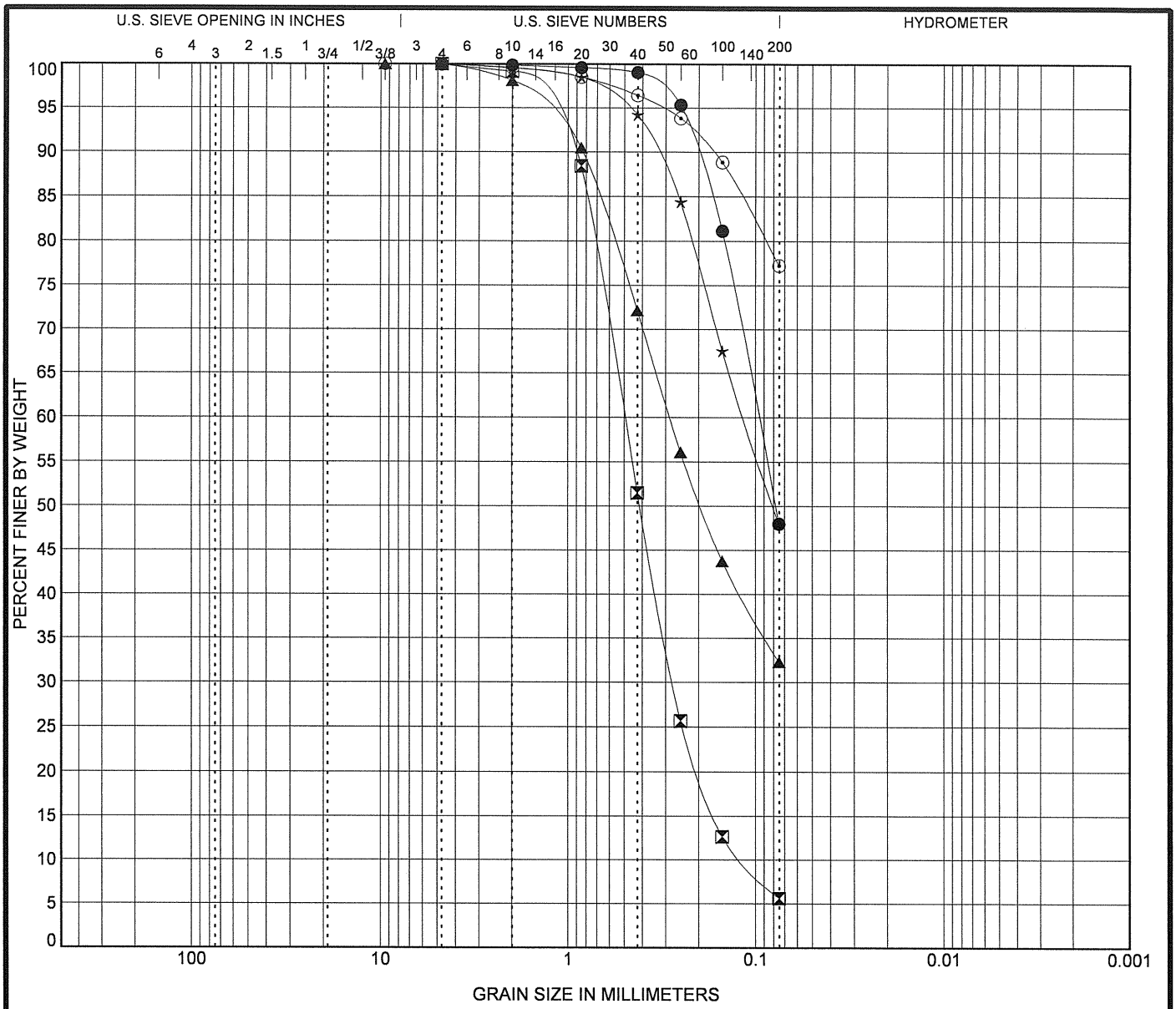
GRAIN SIZE DISTRIBUTION

Project: Proposed Fire Department Training Center

Location: City of Vernon, CA

GEI Project No.: 100-18

Figure B-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	B-2 @ 30.0 ft.	SILTY SAND (SM)	NP	NP	NP		
⊠	B-2 @ 35.0 ft.	POORPLY GRADED SAND with SILT (SP-SM)	NP	NP	NP	1.30	4.32
▲	B-2 @ 40.0 ft.	SILTY SAND (SM)	NP	NP	NP		
★	B-2 @ 45.0 ft.	SILTY SAND (SM)	34	26	8		
⊙	B-2 @ 50.0 ft.	SANDY CLAY (CL)	33	15	18		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2 @ 30.0 ft.	4.75	0.096			0.0	52.0	48.0	
⊠	B-2 @ 35.0 ft.	4.75	0.499	0.273	0.115	0.0	94.4	5.6	
▲	B-2 @ 40.0 ft.	9.5	0.285			0.1	67.6	32.3	
★	B-2 @ 45.0 ft.	4.75	0.115			0.0	52.0	48.0	
⊙	B-2 @ 50.0 ft.	9.5				0.1	22.7	77.2	

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GRAIN SIZE DISTRIBUTION

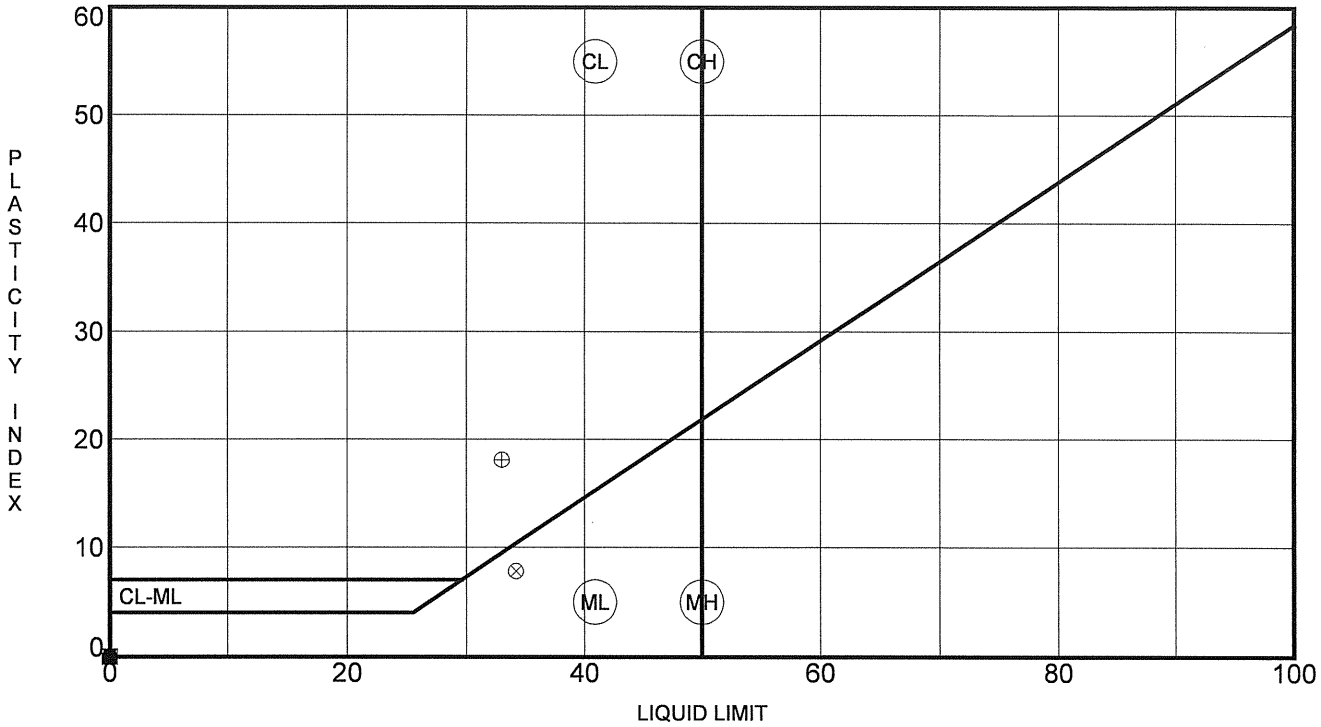
Project: Proposed Fire Department Training Center

Location: City of Vernon, CA

GEI Project No.: 100-18

Figure B-2

US_GRAIN_SIZE 100-18.GPJ LAGNNN01.GDT 5/7/15



	Specimen Identification		LL	PL	PI	Fines	Classification
●	B-2	5.0	NP	NP	NP	23	SILTY SAND (SM)
⊠	B-2	10.0	NP	NP	NP	5	POORPLY GRADED SAND with SILT (SP-SM)
▲	B-2	15.0	NP	NP	NP	9	WELL-GRADED SAND with SILT (SW-SM)
★	B-2	20.0	NP	NP	NP	6	POORPLY GRADED SAND with SILT (SP-SM)
⊙	B-2	25.0	NP	NP	NP	9	POORPLY GRADED SAND with SILT (SP-SM)
✿	B-2	30.0	NP	NP	NP	48	SILTY SAND (SM)
○	B-2	35.0	NP	NP	NP	6	POORPLY GRADED SAND with SILT (SP-SM)
△	B-2	40.0	NP	NP	NP	32	SILTY SAND (SM)
⊗	B-2	45.0	34	26	8	48	SILTY SAND (SM)
⊕	B-2	50.0	33	15	18	77	SANDY CLAY (CL)

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ATTERBERG LIMITS' RESULTS

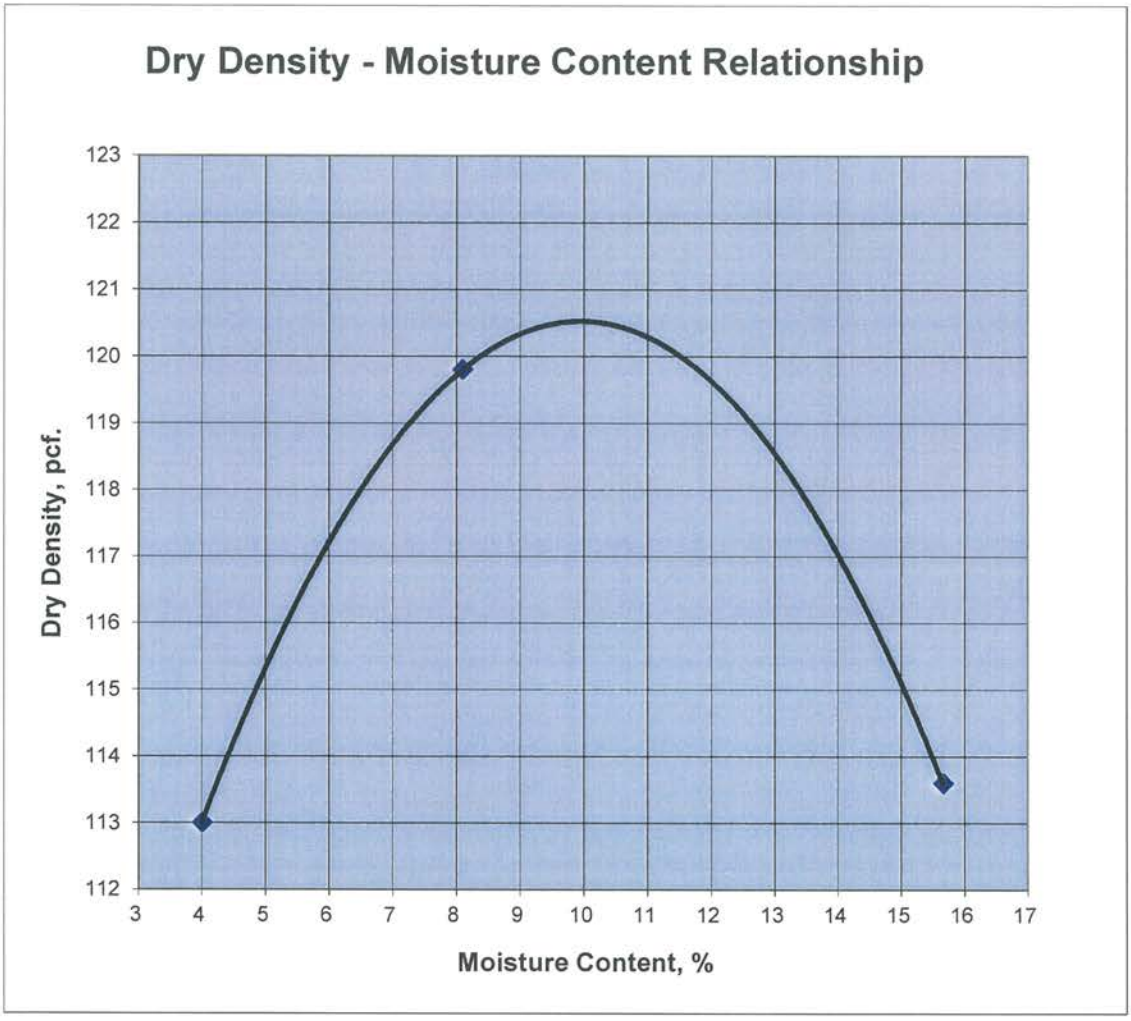
Project: Proposed Fire Department Training Center

Location: City of Vernon, CA

GEI Project No.: 100-18

Figure B-3

US ATTERBERG LIMITS 100-18.GPJ LAGNNN01.GDT 5/7/15



Sample No.: B-1 @ 0-5 feet

Soil Type: SILTY SAND (SM)

Method of Compaction: ASTM D 1557-00

Maximum Dry Density: 120.5 pcf

Optimum Moisture Content: 10.0 %

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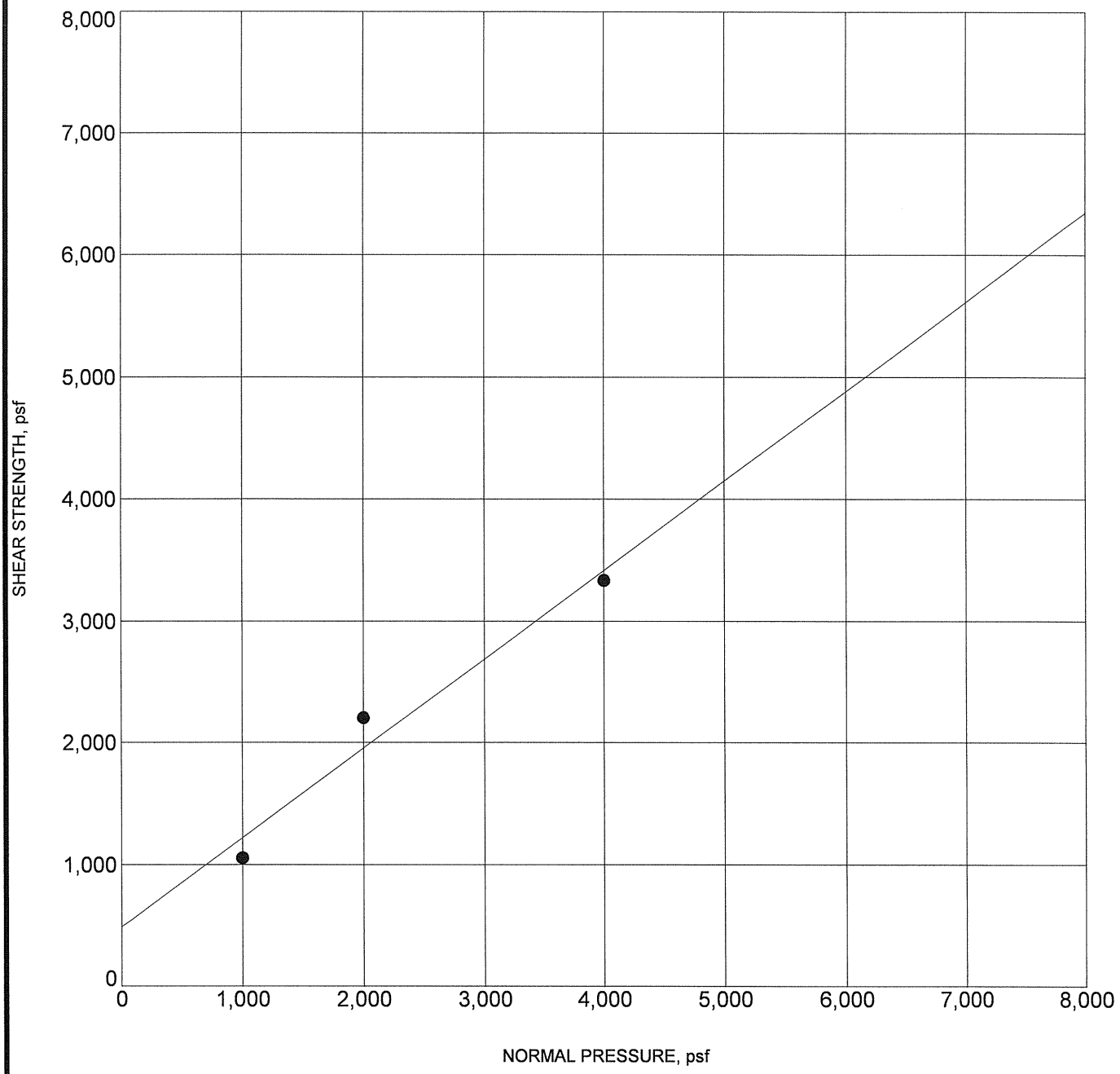
City of Vernon
Los Angeles County, California

GEI-Project No: 100-18

Figure B-4



	Specimen Identification	Classification	γ_d	MC%	c	ϕ
●	**B-2 @ 5.0 ft.**	**SILTY SAND (SM)**	**96.4**	**4.4**	**489**	**36**

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DIRECT SHEAR TEST

Project: Proposed Fire Department Training Center

Location: City of Vernon, CA

GEI Project No.: 100-18

Figure B-5

US DIRECT SHEAR 100-18.GPJ LAGNNN01.GDT 5/7/15

APPENDIX C

FAULTING AND SEISMICITY

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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: 100-18

DATE: 05-04-2015

JOB NAME: Fire Department Training Center

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.9977
SITE LONGITUDE: 118.2051

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 5) Boore et al. (1997) Horiz. - SOIL (310)
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: cd_2drp
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 0.0

100-18 EQFAULT

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ELYSIAN PARK THRUST	0.4(0.6)	6.7	0.588	X
COMPTON THRUST	4.2(6.8)	6.8	0.437	X
NEWPORT-INGLEWOOD (L.A.Basin)	6.5(10.5)	6.9	0.299	IX
HOLLYWOOD	8.5(13.7)	6.4	0.236	IX
RAYMOND	8.6(13.9)	6.5	0.246	IX
VERDUGO	9.7(15.6)	6.7	0.253	IX
WHITTIER	10.7(17.3)	6.8	0.204	VIII
SANTA MONICA	13.0(20.9)	6.6	0.195	VIII
SIERRA MADRE	14.5(23.4)	7.0	0.222	IX
PALOS VERDES	15.0(24.1)	7.1	0.188	VIII
CLAMSHELL-SAWPIT	17.1(27.6)	6.5	0.151	VIII
SAN JOSE	18.2(29.3)	6.5	0.144	VIII

MALIBU COAST	19.0(30.5)	6.7	0.155	VIII
NORTHRIDGE (E. Oak Ridge)	19.3(31.1)	6.9	0.170	VIII
SIERRA MADRE (San Fernando)	20.0(32.2)	6.7	0.149	VIII
SAN GABRIEL	22.6(36.3)	7.0	0.131	VIII
CHINO-CENTRAL AVE. (Elsinore)	22.7(36.5)	6.7	0.136	VIII
ANACAPA-DUME	28.0(45.1)	7.3	0.158	VIII
SANTA SUSANA	28.0(45.1)	6.6	0.110	VII
CUCAMONGA	28.5(45.8)	7.0	0.134	VIII
NEWPORT-INGLEWOOD (Offshore)	32.7(52.6)	6.9	0.094	VII
HOLSER	33.1(53.3)	6.5	0.091	VII
ELSINORE-GLEN IVY	34.1(54.8)	6.8	0.086	VII
SAN ANDREAS - 1857 Rupture	36.8(59.2)	7.8	0.137	VIII
SAN ANDREAS - Mojave	36.8(59.2)	7.1	0.095	VII
OAK RIDGE (Onshore)	38.4(61.8)	6.9	0.101	VII
SIMI-SANTA ROSA	39.5(63.6)	6.7	0.089	VII
SAN JACINTO-SAN BERNARDINO	43.4(69.9)	6.7	0.068	VI
SAN CAYETANO	44.0(70.8)	6.8	0.086	VII
SAN ANDREAS - Southern	44.3(71.3)	7.4	0.096	VII
SAN ANDREAS - San Bernardino	44.3(71.3)	7.3	0.091	VII
CLEGHORN	47.7(76.7)	6.5	0.057	VI
SAN ANDREAS - Carrizo	51.6(83.0)	7.2	0.077	VII
CORONADO BANK	52.8(85.0)	7.4	0.084	VII
ELSINORE-TEMECULA	54.9(88.3)	6.8	0.060	VI
SAN JACINTO-SAN JACINTO VALLEY	55.5(89.3)	6.9	0.062	VI
SANTA YNEZ (East)	55.9(90.0)	7.0	0.065	VI
NORTH FRONTAL FAULT ZONE (West)	57.1(91.9)	7.0	0.078	VII
OAK RIDGE(Blind Thrust Offshore)	58.3(93.8)	6.9	0.073	VII
VENTURA - PITAS POINT	58.7(94.5)	6.8	0.069	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.
CHANNEL IS. THRUST (Eastern)	60.8(97.8)	7.4	0.092	VII
MONTALVO-OAK RIDGE TREND	63.8(102.7)	6.6	0.058	VI
M.RIDGE-ARROYO PARIDA-SANTA ANA	64.0(103.0)	6.7	0.061	VI
RED MOUNTAIN	67.6(108.8)	6.8	0.062	VI
GARLOCK (West)	69.2(111.3)	7.1	0.058	VI

100-18 EQFAULT

PLEITO THRUST	69.9(112.5)	7.2	0.074	VII
BIG PINE	73.1(117.6)	6.7	0.045	VI
HELENDALE - S. LOCKHARDT	74.1(119.2)	7.1	0.055	VI
ROSE CANYON	74.8(120.4)	6.9	0.049	VI
SANTA CRUZ ISLAND	75.1(120.8)	6.8	0.057	VI
SAN JACINTO-ANZA	75.9(122.2)	7.2	0.057	VI
ELSINORE-JULIAN	80.7(129.8)	7.1	0.052	VI
NORTH FRONTAL FAULT ZONE (East)	81.4(131.0)	6.7	0.051	VI
WHITE WOLF	82.3(132.4)	7.2	0.065	VI
LENWOOD-LOCKHART-OLD WOMAN SPRGS	84.6(136.1)	7.3	0.055	VI
PINTO MOUNTAIN	85.0(136.8)	7.0	0.047	VI
NORTH CHANNEL SLOPE	88.0(141.6)	7.1	0.059	VI
SANTA YNEZ (West)	88.5(142.5)	6.9	0.043	VI
GARLOCK (East)	89.5(144.1)	7.3	0.053	VI
GRAVEL HILLS - HARPER LAKE	94.1(151.5)	6.9	0.041	V
JOHNSON VALLEY (Northern)	94.6(152.2)	6.7	0.037	V
LANDERS	95.1(153.1)	7.3	0.051	VI
SANTA ROSA ISLAND	97.5(156.9)	6.9	0.049	VI
BLACKWATER	98.3(158.2)	6.9	0.040	V
SAN ANDREAS - Coachella	99.5 (160.2)	7.1	0.044	VI

****

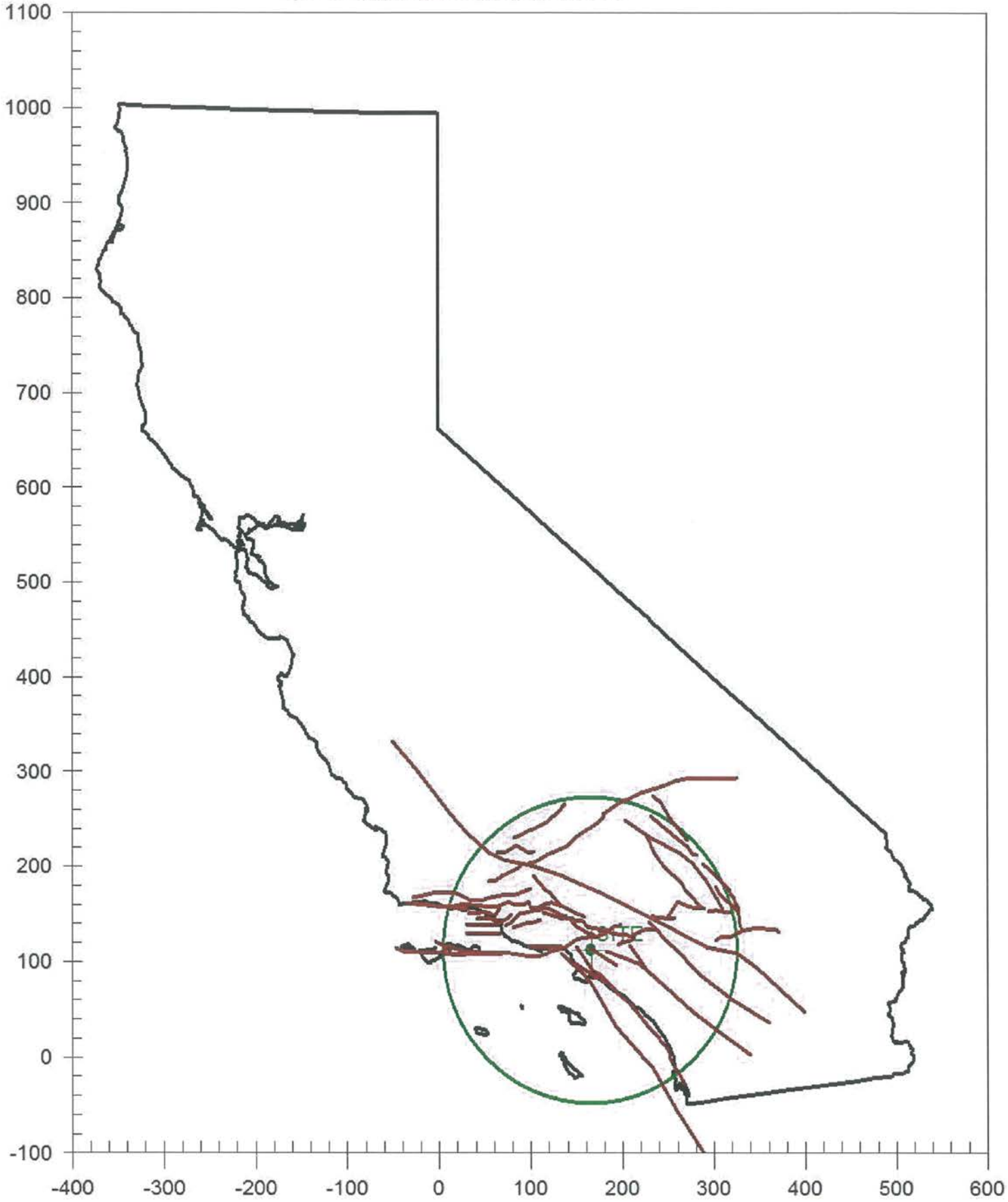
-END OF SEARCH- 65 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELYSIAN PARK THRUST FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.4 MILES (0.6 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5884 g

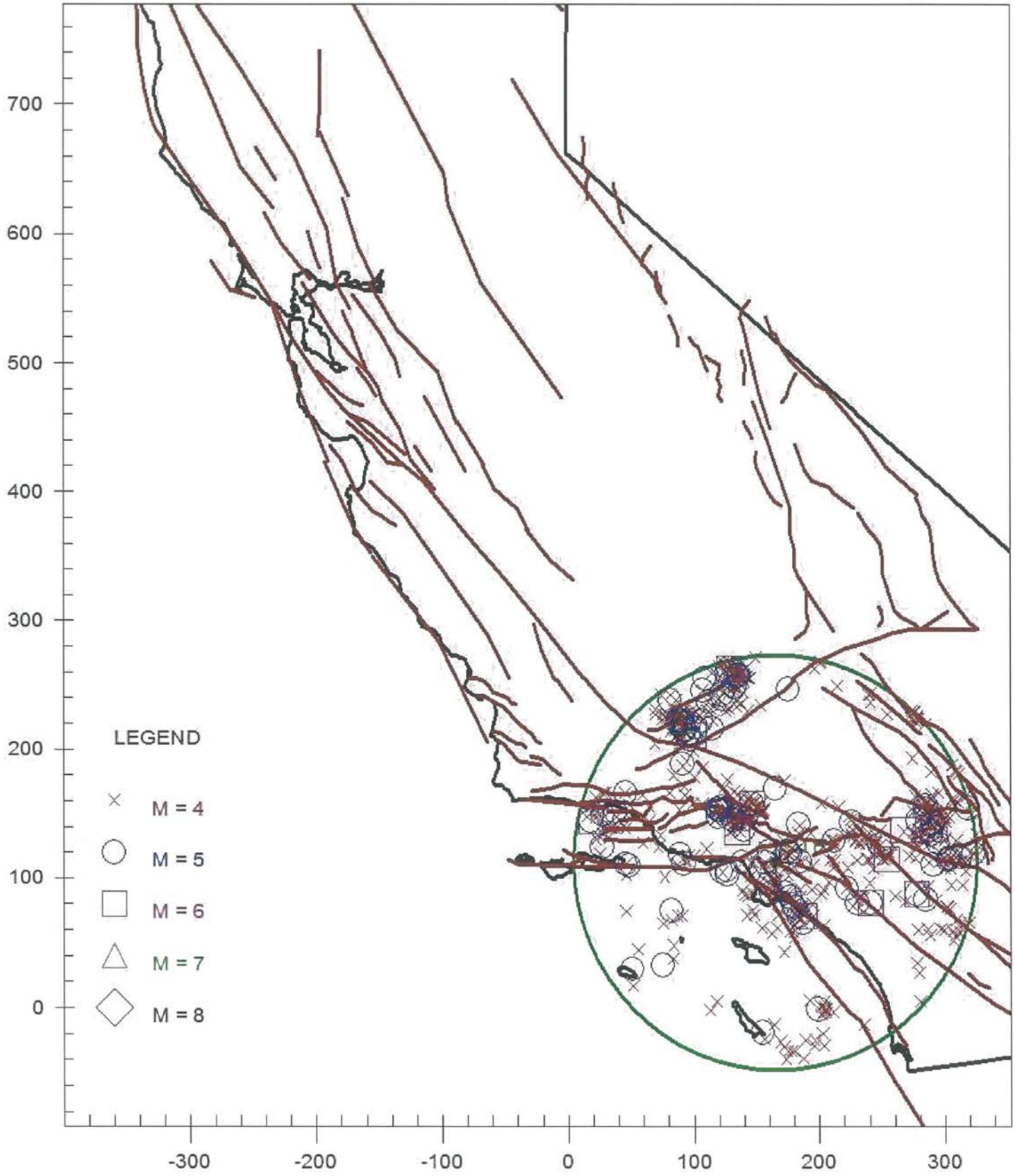
CALIFORNIA FAULT MAP

Fire Department Training Center



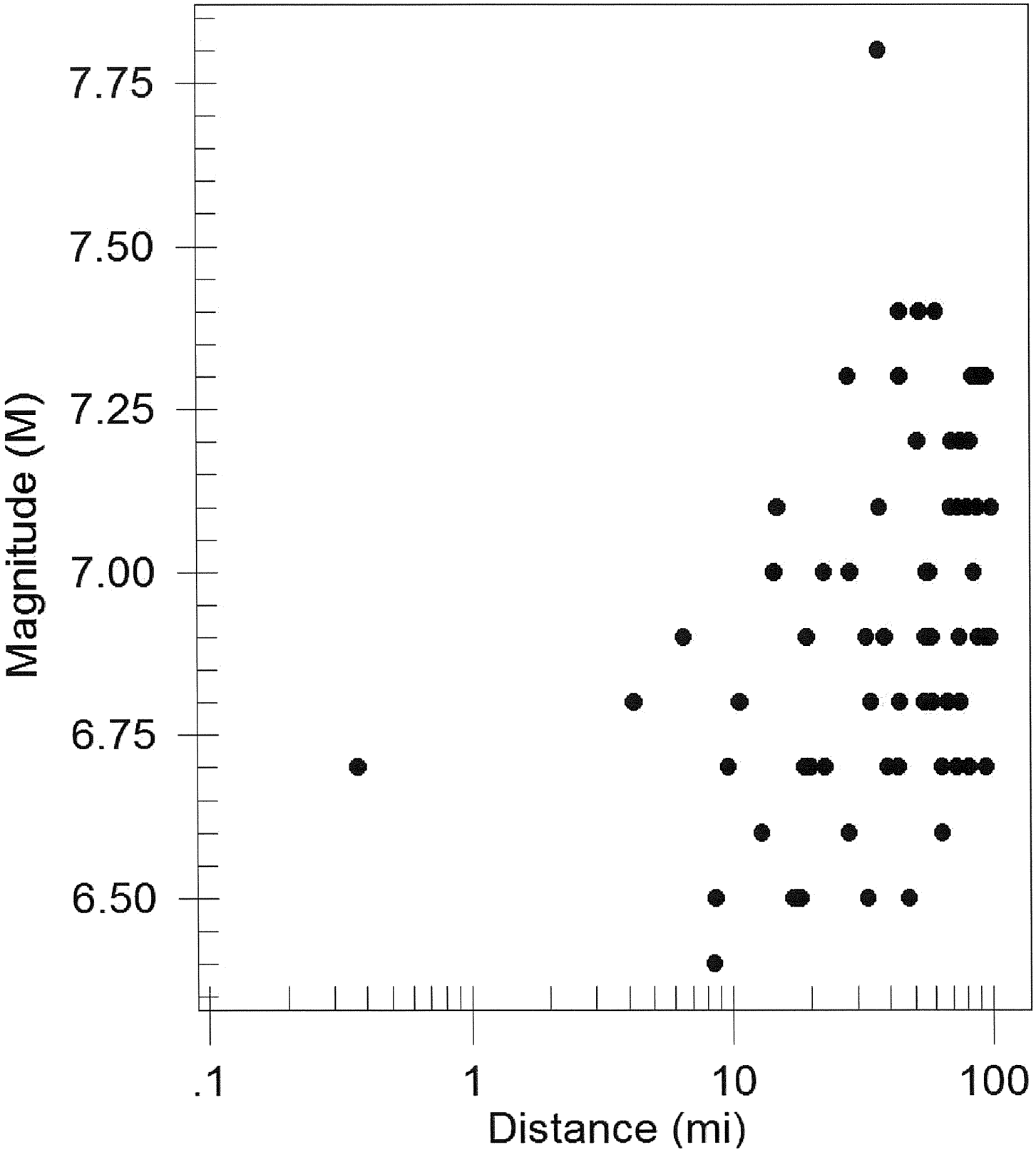
EARTHQUAKE EPICENTER MAP

Fire Department Training Center



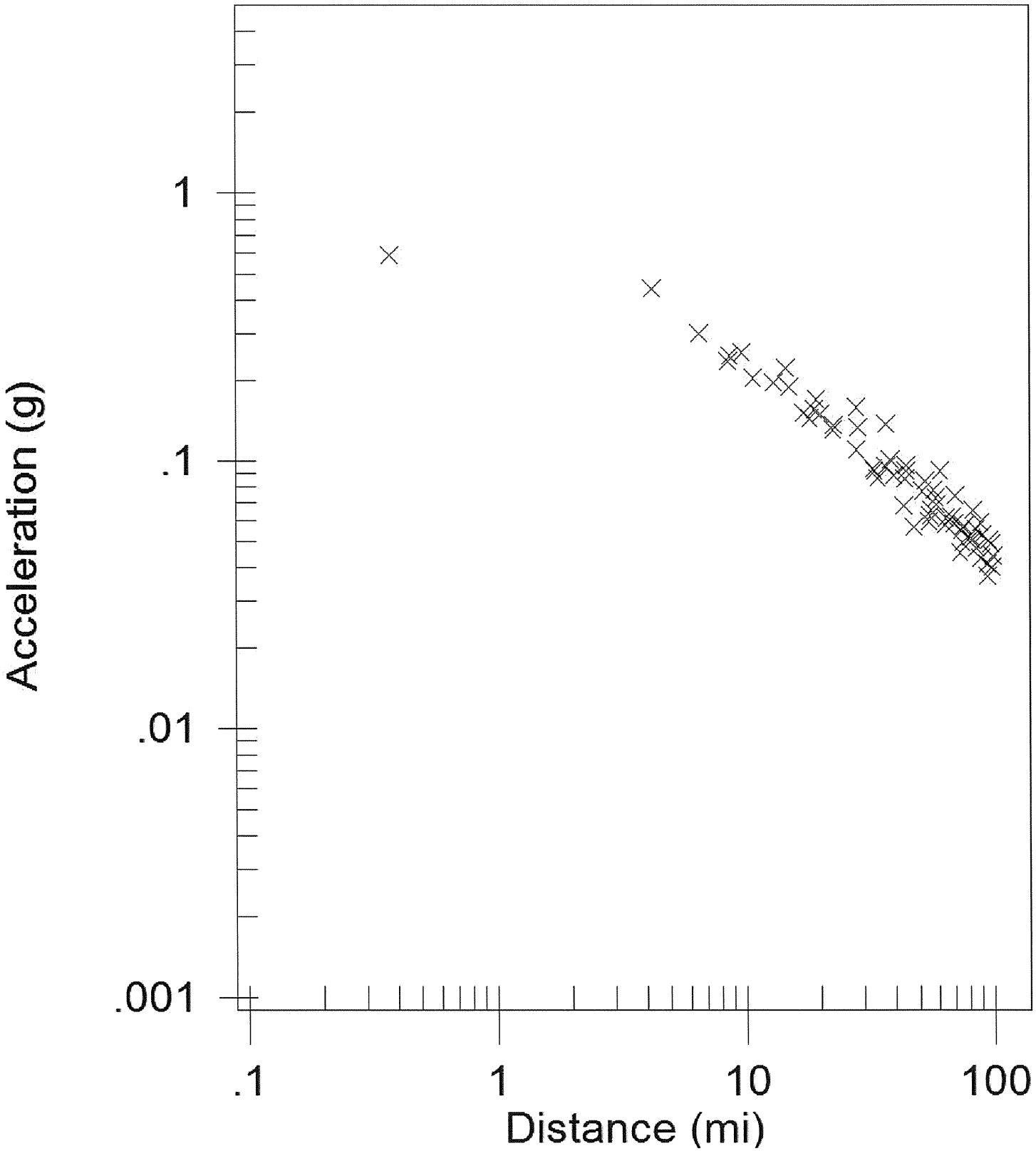
EARTHQUAKE MAGNITUDES & DISTANCES

Fire Department Training Center



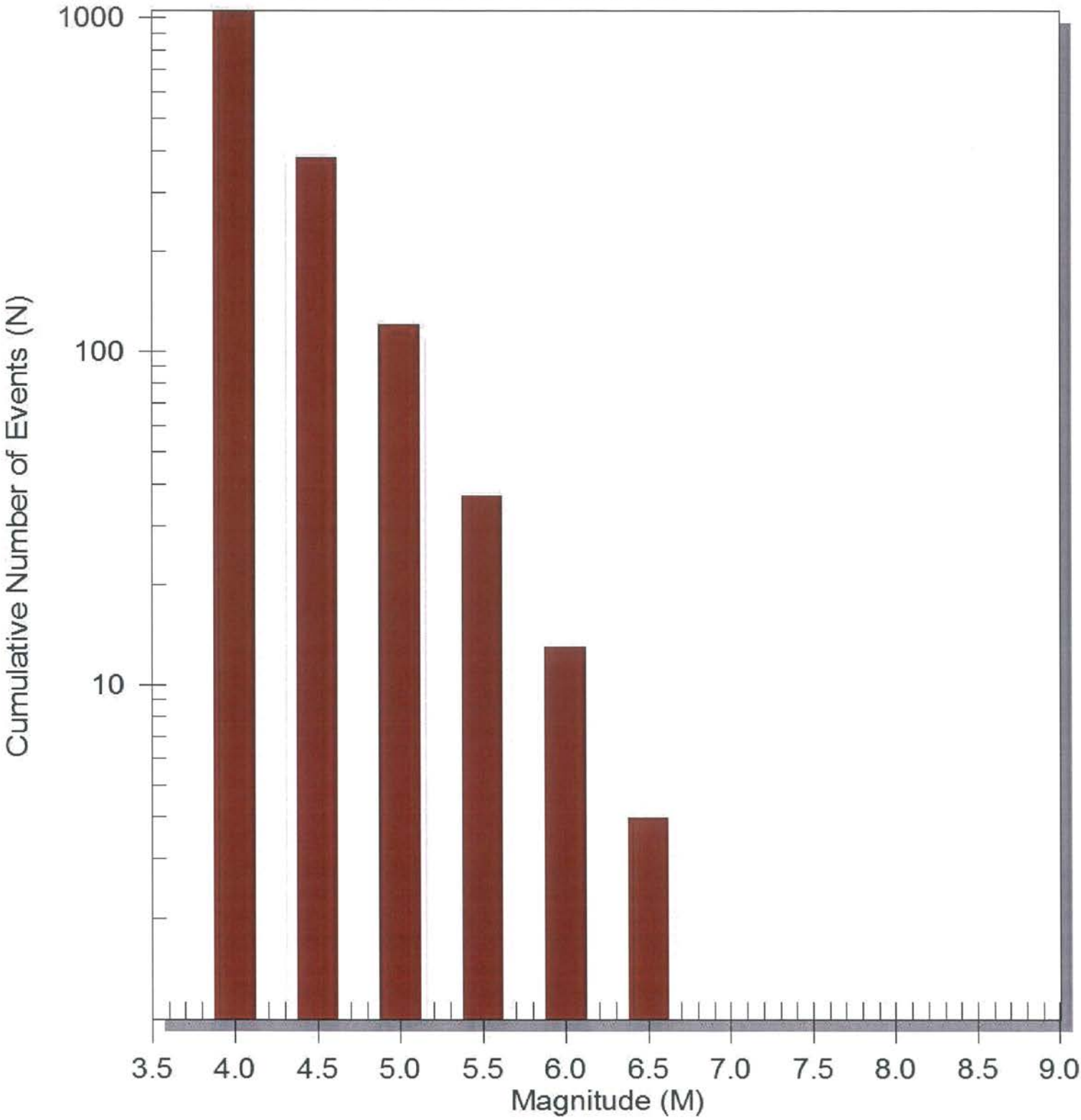
MAXIMUM EARTHQUAKES

Fire Department Training Center



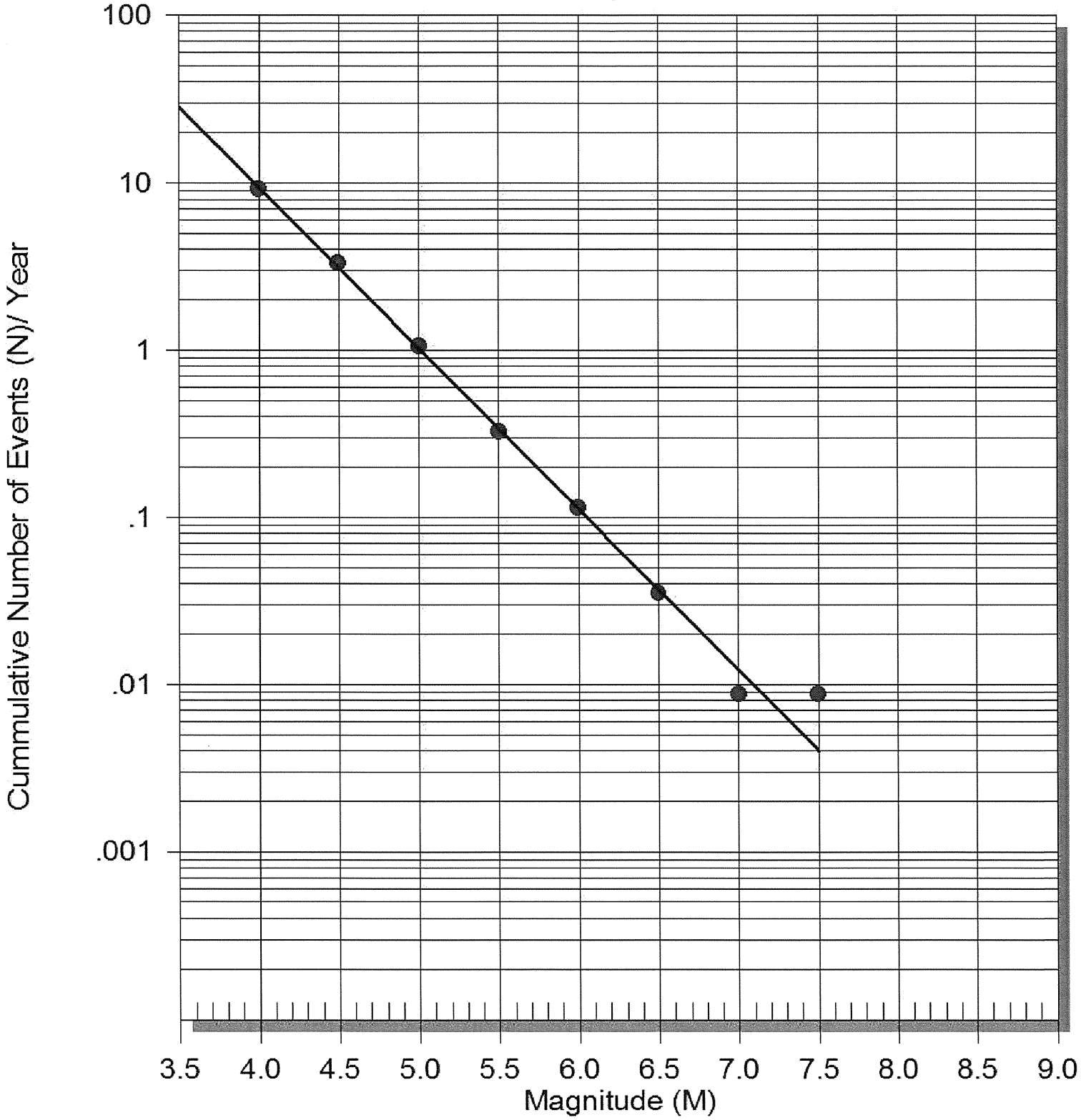
Number of Earthquakes (N) Above Magnitude (M)

Fire Department Training Center



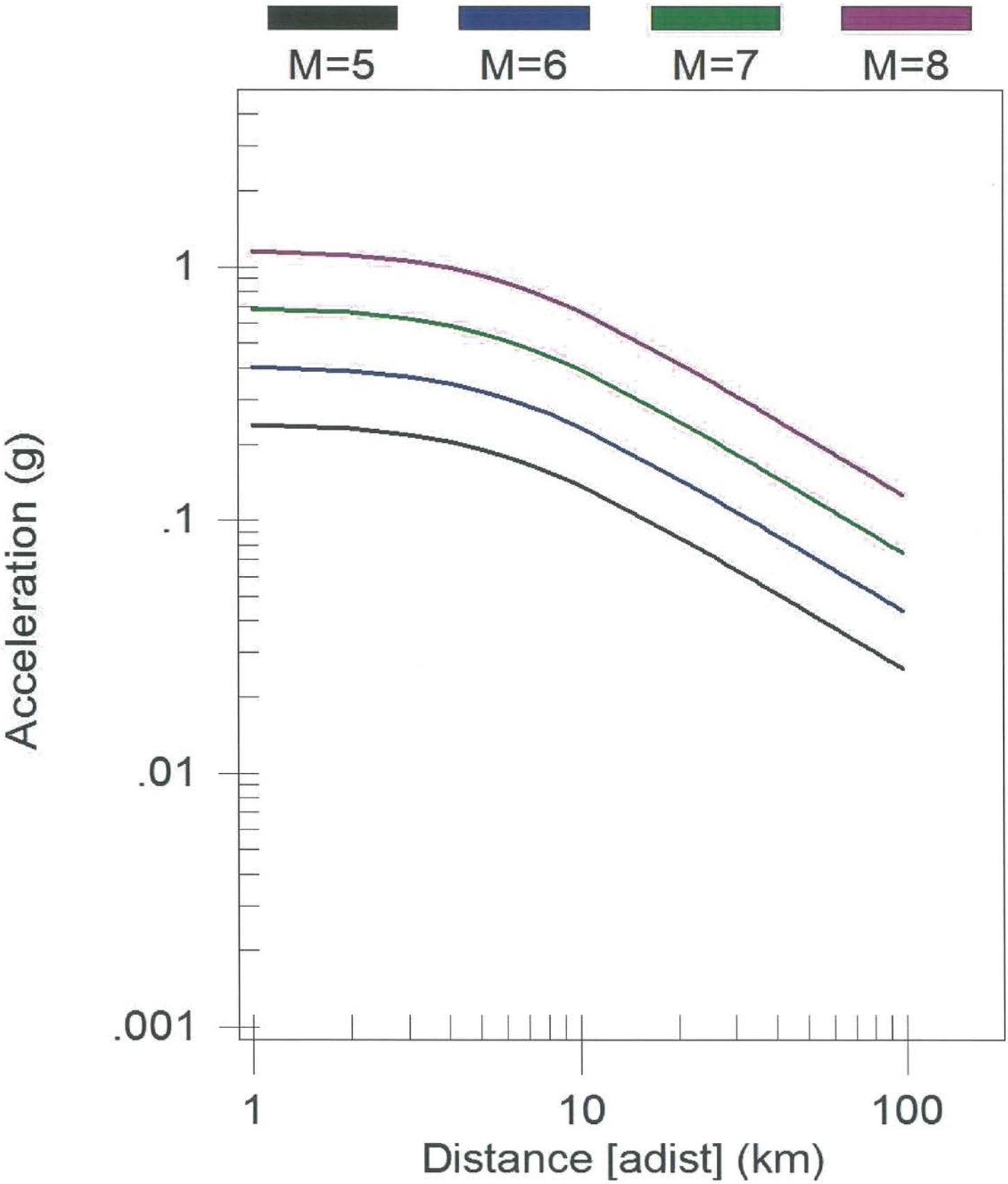
EARTHQUAKE RECURRENCE CURVE

Fire Department Training Center



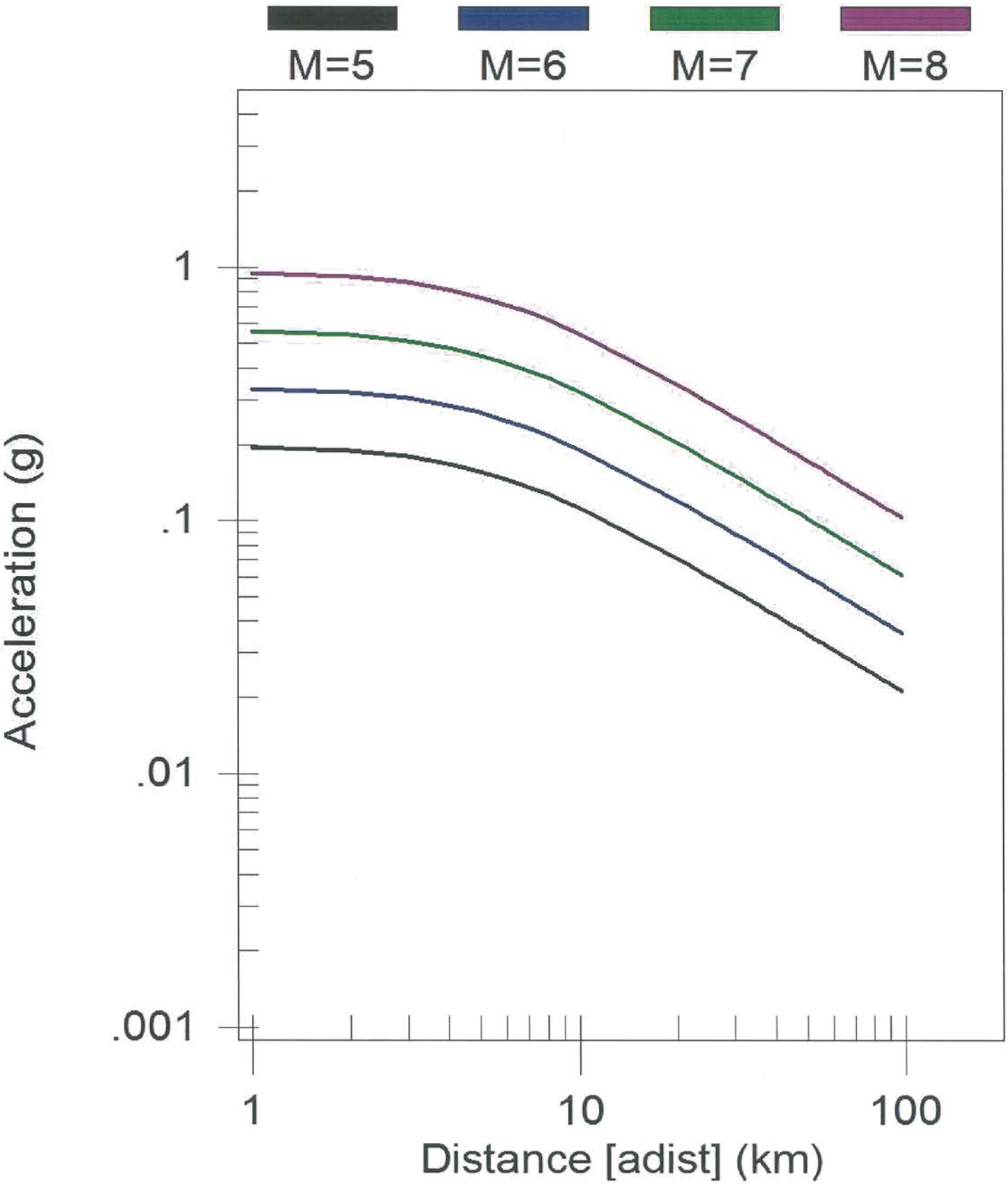
BLIND-THRUST FAULTS

5) Boore et al. (1997) Horiz. - SOIL (310)



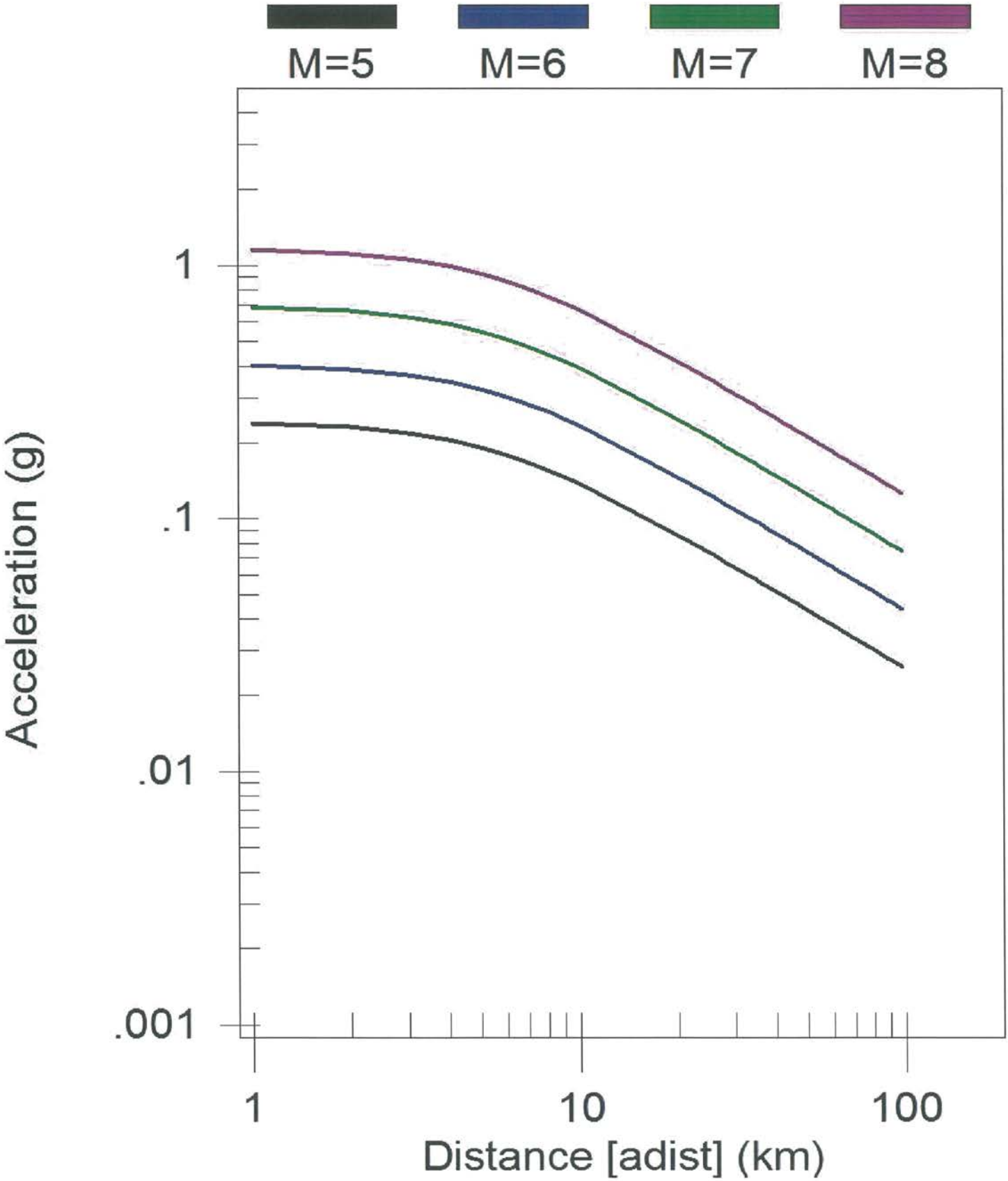
STRIKE-SLIP FAULTS

5) Boore et al. (1997) Horiz. - SOIL (310)



DIP-SLIP FAULTS

5) Boore et al. (1997) Horiz. - SOIL (310)



APPENDIX D

SOIL LIQUEFACTION ANALYSIS

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

JOB NUMBER: 100-18 DATE: 05-06-2015

JOB NAME: Training Center

SOIL-PROFILE NAME: LIQ10018.LDW

BORING GROUNDWATER DEPTH: 30.00 ft

CALCULATION GROUNDWATER DEPTH: 30.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 6.70 Mw

SITE PEAK GROUND ACCELERATION: 0.440 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.15

SAMPLER SIZE CORRECTION FACTOR: 1.00

N60 HAMMER CORRECTION FACTOR: 0.95

MAGNITUDE SCALING FACTOR METHOD: R.Seed (1998, pers. comm)

Magnitude Scaling Factor: 1.212

rd-CORRECTION METHOD: Idriss (1998, in press)

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 — PAGE 1

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	0.013	0.013	13	~	*	*	*	*	*	**
1	0.75	0.038	0.038	13	~	*	*	*	*	*	**
1	1.25	0.063	0.063	13	~	*	*	*	*	*	**
1	1.75	0.087	0.087	13	~	*	*	*	*	*	**
1	2.25	0.113	0.113	13	~	*	*	*	*	*	**
1	2.75	0.138	0.138	13	~	*	*	*	*	*	**
1	3.25	0.163	0.163	13	~	*	*	*	*	*	**
1	3.75	0.188	0.188	13	~	*	*	*	*	*	**
1	4.25	0.213	0.213	13	~	*	*	*	*	*	**
1	4.75	0.238	0.238	13	~	*	*	*	*	*	**
1	5.25	0.263	0.263	13	~	*	*	*	*	*	**
1	5.75	0.288	0.288	13	~	*	*	*	*	*	**
1	6.25	0.313	0.313	13	~	*	*	*	*	*	**
2	6.75	0.338	0.338	14	~	*	*	*	*	*	**
2	7.25	0.363	0.363	14	~	*	*	*	*	*	**
2	7.75	0.388	0.388	14	~	*	*	*	*	*	**
2	8.25	0.413	0.413	14	~	*	*	*	*	*	**
2	8.75	0.438	0.438	14	~	*	*	*	*	*	**
2	9.25	0.463	0.463	14	~	*	*	*	*	*	**
2	9.75	0.488	0.488	14	~	*	*	*	*	*	**
2	10.25	0.513	0.513	14	~	*	*	*	*	*	**
2	10.75	0.538	0.538	14	~	*	*	*	*	*	**
2	11.25	0.563	0.563	14	~	*	*	*	*	*	**
3	11.75	0.587	0.587	52	~	*	*	*	*	*	**
3	12.25	0.612	0.612	52	~	*	*	*	*	*	**
3	12.75	0.637	0.637	52	~	*	*	*	*	*	**
3	13.25	0.662	0.662	52	~	*	*	*	*	*	**

3	13.75	0.687	0.687	52	~	*	*	*	*	*	**
3	14.25	0.712	0.712	52	~	*	*	*	*	*	**
3	14.75	0.737	0.737	52	~	*	*	*	*	*	**
3	15.25	0.762	0.762	52	~	*	*	*	*	*	**
3	15.75	0.787	0.787	52	~	*	*	*	*	*	**
3	16.25	0.812	0.812	52	~	*	*	*	*	*	**
4	16.75	0.837	0.837	24	~	*	*	*	*	*	**
4	17.25	0.862	0.862	24	~	*	*	*	*	*	**
4	17.75	0.887	0.887	24	~	*	*	*	*	*	**
4	18.25	0.912	0.912	24	~	*	*	*	*	*	**
4	18.75	0.937	0.937	24	~	*	*	*	*	*	**
4	19.25	0.962	0.962	24	~	*	*	*	*	*	**
4	19.75	0.987	0.987	24	~	*	*	*	*	*	**
4	20.25	1.012	1.012	24	~	*	*	*	*	*	**
4	20.75	1.037	1.037	24	~	*	*	*	*	*	**
4	21.25	1.062	1.062	24	~	*	*	*	*	*	**

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 2

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
5	21.75	1.087	1.087	40	~	*	*	*	*	*	**
5	22.25	1.112	1.112	40	~	*	*	*	*	*	**
5	22.75	1.137	1.137	40	~	*	*	*	*	*	**
5	23.25	1.162	1.162	40	~	*	*	*	*	*	**
5	23.75	1.187	1.187	40	~	*	*	*	*	*	**
5	24.25	1.212	1.212	40	~	*	*	*	*	*	**
5	24.75	1.237	1.237	40	~	*	*	*	*	*	**
5	25.25	1.262	1.262	40	~	*	*	*	*	*	**
5	25.75	1.287	1.287	40	~	*	*	*	*	*	**
5	26.25	1.312	1.312	40	~	*	*	*	*	*	**
6	26.75	1.337	1.337	19	8.39	*	*	*	*	*	**
6	27.25	1.362	1.362	19	8.39	*	*	*	*	*	**
6	27.75	1.387	1.387	19	8.39	*	*	*	*	*	**
6	28.25	1.412	1.412	19	8.39	*	*	*	*	*	**
6	28.75	1.437	1.437	19	8.39	*	*	*	*	*	**
6	29.25	1.462	1.462	19	8.39	*	*	*	*	*	**

6	29.75	1.487	1.487	19	8.39	*	*	*	*	*	**
6	30.25	1.512	1.505	19	8.39	0.834	25.7	0.280	0.861	0.247	1.37
6	30.75	1.537	1.514	19	8.39	0.834	25.7	0.280	0.858	0.249	1.36
6	31.25	1.562	1.523	19	8.39	0.834	25.7	0.280	0.855	0.251	1.35
7	31.75	1.587	1.533	57	0.27	0.809	50.7	Infin	0.852	0.252	NonLiq
7	32.25	1.612	1.542	57	0.27	0.809	50.7	Infin	0.849	0.254	NonLiq
7	32.75	1.637	1.552	57	0.27	0.809	50.7	Infin	0.846	0.255	NonLiq
7	33.25	1.662	1.561	57	0.27	0.809	50.7	Infin	0.843	0.257	NonLiq
7	33.75	1.687	1.570	57	0.27	0.809	50.7	Infin	0.840	0.258	NonLiq
7	34.25	1.712	1.580	57	0.27	0.809	50.7	Infin	0.837	0.260	NonLiq
7	34.75	1.737	1.589	57	0.27	0.809	50.7	Infin	0.834	0.261	NonLiq
7	35.25	1.762	1.599	57	0.27	0.809	50.7	Infin	0.831	0.262	NonLiq
7	35.75	1.787	1.608	57	0.27	0.809	50.7	Infin	0.828	0.263	NonLiq
7	36.25	1.812	1.617	57	0.27	0.809	50.7	Infin	0.825	0.265	NonLiq
8	36.75	1.837	1.627	38	10.41	0.786	43.1	Infin	0.822	0.266	NonLiq
8	37.25	1.862	1.636	38	10.41	0.786	43.1	Infin	0.820	0.267	NonLiq
8	37.75	1.887	1.646	38	10.41	0.786	43.1	Infin	0.817	0.268	NonLiq
8	38.25	1.912	1.655	38	10.41	0.786	43.1	Infin	0.814	0.269	NonLiq
8	38.75	1.937	1.664	38	10.41	0.786	43.1	Infin	0.811	0.270	NonLiq
8	39.25	1.962	1.674	38	10.41	0.786	43.1	Infin	0.808	0.271	NonLiq
8	39.75	1.987	1.683	38	10.41	0.786	43.1	Infin	0.805	0.272	NonLiq
8	40.25	2.012	1.693	38	10.41	0.786	43.1	Infin	0.802	0.273	NonLiq
8	40.75	2.037	1.702	38	10.41	0.786	43.1	Infin	0.799	0.274	NonLiq
8	41.25	2.062	1.711	38	10.41	0.786	43.1	Infin	0.796	0.274	NonLiq
9	41.75	2.087	1.721	18	7.94	0.765	23.0	0.233	0.793	0.275	1.03
9	42.25	2.112	1.730	18	7.94	0.765	23.0	0.233	0.790	0.276	1.02
9	42.75	2.137	1.740	18	7.94	0.765	23.0	0.233	0.788	0.277	1.02
9	43.25	2.162	1.749	18	7.94	0.765	23.0	0.233	0.785	0.277	1.02

NCEER [1997] Method — LIQUEFACTION ANALYSIS SUMMARY — PAGE 3

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
9	43.75	2.187	1.758	18	7.94	0.765	23.0	0.233	0.782	0.278	1.01
9	44.25	2.212	1.768	18	7.94	0.765	23.0	0.233	0.779	0.279	1.01
9	44.75	2.237	1.777	18	7.94	0.765	23.0	0.233	0.776	0.279	1.01
9	45.25	2.262	1.787	18	7.94	0.765	23.0	0.233	0.773	0.280	1.01

9	45.75	2.287	1.796	18	7.94	0.765	23.0	0.233	0.770	0.281	1.01
9	46.25	2.312	1.805	18	7.94	0.765	23.0	0.233	0.767	0.281	1.00
10	46.75	2.337	1.815	17	7.71	0.746	21.6	0.214	0.765	0.282	0.92
10	47.25	2.363	1.824	17	7.71	0.746	21.6	0.214	0.762	0.282	0.92
10	47.75	2.388	1.834	17	7.71	0.746	21.6	0.214	0.759	0.283	0.92
10	48.25	2.413	1.843	17	7.71	0.746	21.6	0.214	0.756	0.283	0.92
10	48.75	2.438	1.852	17	7.71	0.746	21.6	0.214	0.753	0.283	0.91
10	49.25	2.463	1.862	17	7.71	0.746	21.6	0.214	0.750	0.284	0.91
10	49.75	2.488	1.871	17	7.71	0.746	21.6	0.214	0.748	0.284	0.91
10	50.25	2.513	1.881	17	7.71	0.746	21.6	0.214	0.745	0.285	0.91
10	50.75	2.538	1.890	17	7.71	0.746	21.6	0.214	0.742	0.285	0.91
10	51.25	2.563	1.899	17	7.71	0.746	21.6	0.214	0.739	0.285	0.91

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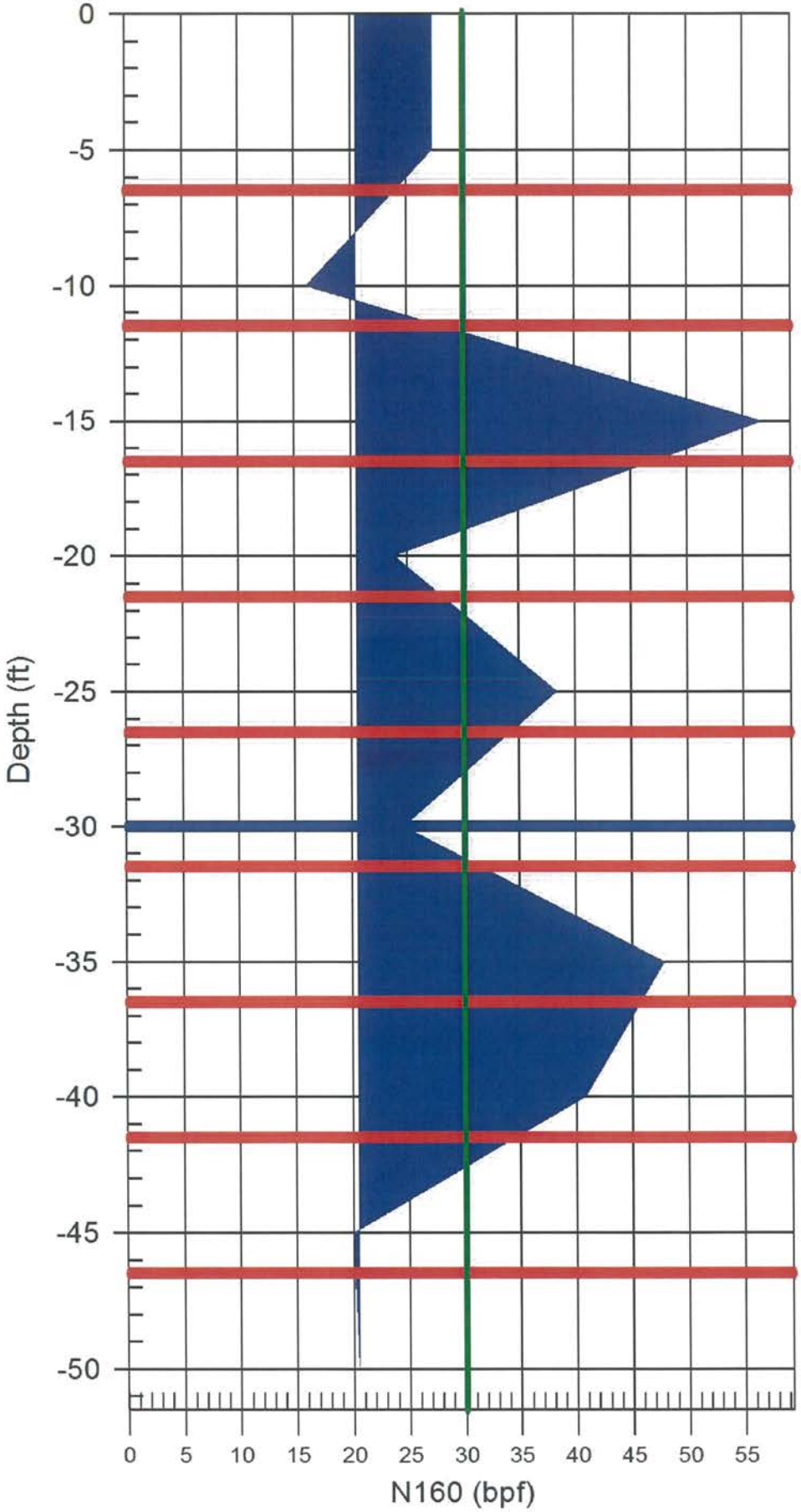
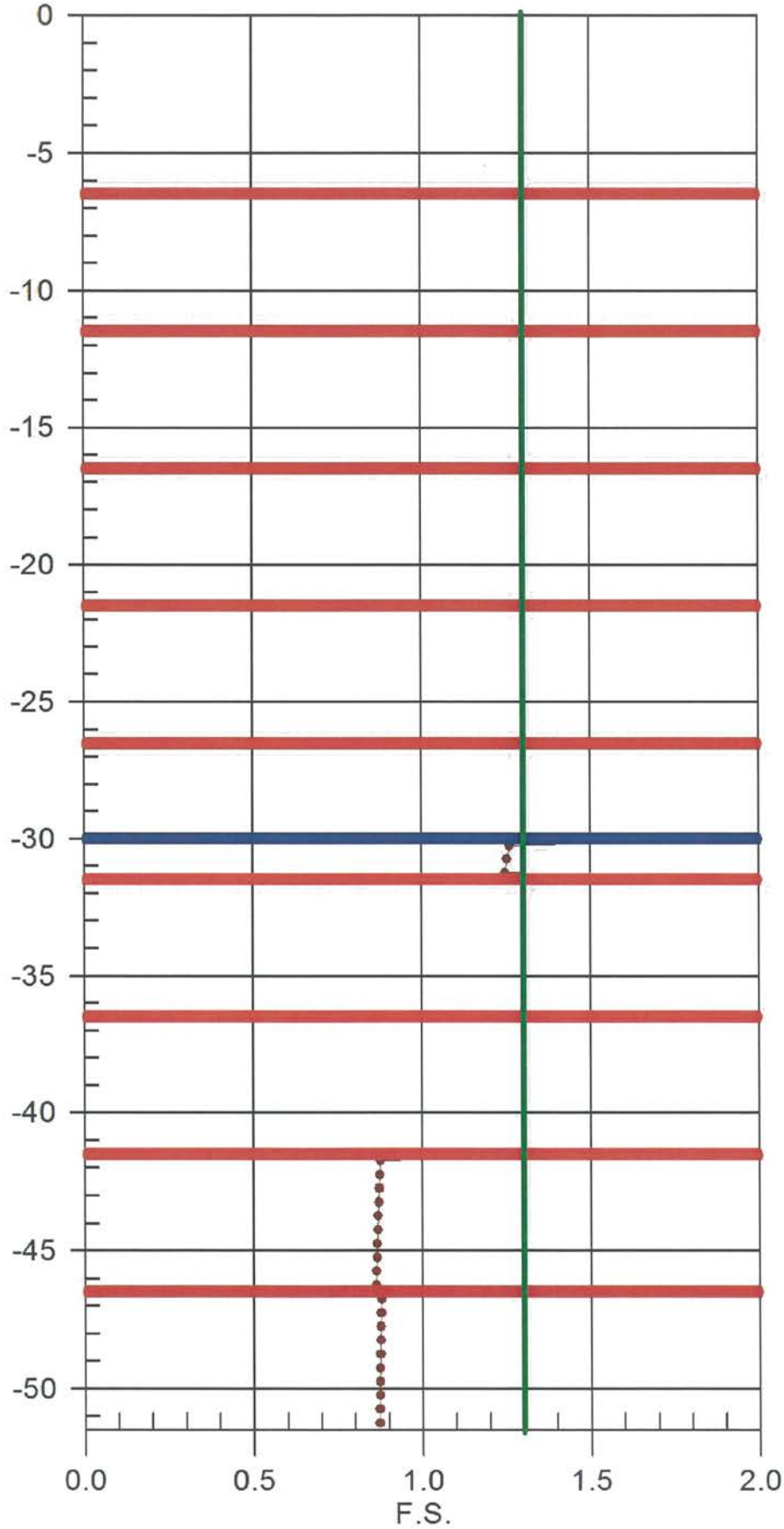
Fire Department Training Center

(N1)60cs vs. Depth



Fire Department Training Center

Factor of Safety vs. Depth



APPENDIX E

SITE PHOTOGRAPHS

Photo 1: Borehole being prepared for the performance of the percolation testing.

Photo 2: Picture showing Rotary-Wash Drill Rig used for drilling up to 51.5 feet below the existing ground surface.