APPENDIX D

Geology and Soils Information

GEOTECHNICAL REPORT

KAISER PERMANENTE - IRWINDALE SPECIALTY MOB
12761 SHABARUM AVENUE
IRWINDALE, CALIFORNIA

GEOBASE, INC.

GEOTECHNICAL REPORT

KAISER PERMANENTE - IRWINDALE SPECIALTY MOB
12761 SHABARUM AVENUE
IRWINDALE, CALIFORNIA

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I. INTRODUCTION

1.1 General

Kaiser Foundation Health Plan, Inc. is planning the construction of the Kaiser Permanente - Irwindale Specialty Medical Office Building (MOB), located at 12761 Shabarum Avenue, in the City of Irwindale, California. The site location is shown on the Site Location Map, Figure A-1, Appendix A. GEOBASE, INC. (GEOBASE) was retained by Kaiser Foundation Health Plan, Inc. to complete a geotechnical report for the proposed development.

In April 2016, GEOBASE was retained by Kaiser Foundation Hospital, Inc. to complete a geotechnical evaluation of the site for use in preliminary design and planning of the proposed development. The results of this evaluation were presented in a report titled "Geotechnical Evaluation, 12761 Shabarum Avenue, Irwindale, California" (GEOBASE, 2015).

For the geotechnical report, we were provided with:

- Architectural site plan, elevations and overall floor plans.
- Structural Foundation Plans, Sheet No.'s SX-1.01 and S2.23, showing preliminary foundation layouts and column loads.

This geotechnical report incorporates the results of both field and laboratory testing. These results are discussed with reference to the proposed developments, as shown on the above noted plans. Both general and specific recommendations pertinent to suitable site development and foundation design, respectively, are provided. Construction guidelines related to the geotechnical aspects of the project are also addressed.

1.2 Objective of the Geotechnical Report

The objective of the geotechnical report is to provide recommendations pertinent to suitable site development and foundation design. These recommendations are to satisfy the requirements of the regulating agencies and assist with final design and construction of the project, as planned.

1.3 Scope of Services

To achieve the objective of the geotechnical report, stated above, the services provided during the course of the geotechnical evaluation and subsequent analyses included:

- Review of available published and unpublished geotechnical, geological, and seismological reports and maps pertinent to the site;

- Review of previous soils reports and related documents (see references);
- Field exploration program consisting of advancing four (4) Cone Penetration Tests (CPT's);
- Field testing consisting of two (2) geophysical survey lines, utilizing multi-channel array surface wave (MASW) methods.
- Evaluation of data obtained from the above and previous borings at the site;
- Engineering analyses; and,
- Preparation of this report describing the field investigation, summarizing the results of field testing and engineering analyses, and providing appropriate preliminary recommendations for site development and foundation design.

II. REVIEW OF AVAILABLE REPORTS

The J. Byer Group, Inc. completed the geotechnical report (May 11, 1998), liquefaction potential (January 27, 1999) and addendum (March 1999) for the existing Jacmar Food Service building and associated facilities. Liquefaction potential was considered "very low" and these reports were approved by the Los Angeles County Department of Public Works. The grading operations, including observations and testing were carried out at the building pad, utility trenches and parking lots, and reported in the compaction reports dated May 4 and November 8, 1999.

A reconnaissance report for the Irwindale Sand and Gravel Mines was found online and at the Los Angeles County Department of Public Works office. This report addressed, in general terms, potential of slope instability and landslides resulting from the mining operation that extends from ground surface to approximately 180 feet deep; however, no landslide or slope failures were identified within the project site.

The project site is not mapped within an area susceptible to landslides, FEMA flood zones, subsidence or current State of California Earthquake zones; however, it is mapped within a potentially liquefiable area.

III. SITE AND PROJECT DESCRIPTIONS

3.1 Site Description

The project site is roughly rectangular shaped and is currently occupied by the Jacmar Food Service warehouse building. The site is bounded by Durbin Pit Mine to the north, 605 Freeway to the west, and commercial developments to the south and east.

As can be observed on Figure A-1, page 2 of 2, Appendix a, north of the project site north property line, at the Durbin Pit Mine Property, based on visual observations, a level area in the order of thirty (30) feet wide is followed by a descending slope up to forty (40) feet high; this slope maximum gradient is visually estimated in the order of 2H:1V (Horizontal:Vertical). At the toe of this slope a level area, approximately sixty (60) feet wide can be observed. North of the areas described in the aforementioned is the pit mine excavation.

Asphaltic concrete paved parking areas and driveways surround the existing building to its west, east and south, and a concrete slab/pavement was noted to the south, adjacent to the loading dock. To the north, the existing structure is located at/very near the property line. The site appears to be relatively flat with drainage appearing to be directed towards the southeast. A billboard sign was observed at the southwest corner of the property. A retaining wall, approximately ten (10) feet high, was also observed along the southern property line, and the adjacent property, to the south, is approximately five (5) feet lower in elevation.

3.2 Project Description

The layout of the proposed site development, Parking Structure and MOB are shown on the Site and Boring Locations Plan, Figure A-2, Appendix A.

The Parking Structure is planned at the northeast corner of the site. It is anticipated to be a five (5) level structure, approximately 26,000 square feet in plan area, with the lower floor elevation at approximately 317 feet above mean-sea-level (amsl).

The three (3) storey MOB is proposed to be located within the western half of the site. The plan area of this structure, including the plaza, is approximately 38,000 square feet. Finish floor elevation of the lower level is planned at 322.5 feet amsl, five and one-half (5.5) feet higher than the lower level floor elevation of the parking structure. In this respect, the northern portion of the east wall of the MOB adjoins the southern half of the west wall of the parking structure.

A retaining wall is planned along the north property line. A driveway leading to the Parking Structure and at-grade parking are proposed along the south, west and north sides of the MOB.

Maximum dead-plus-live column loads for the MOB and Parking Structure are 335 and 1487 kips, respectively, as given by the structural engineer.

IV. SITE INVESTIGATION

4.1 Field Program

The field investigation was carried out on March 12 and 16, 2015, and consisted of two (2) geophysical lines and advancing four (4) CPT's, at the approximate locations shown on the Site, CPT, Boring and Geophysical Survey Locations Plan, Figure A-2, Appendix A. The CPT's were located in the field utilizing a roll-a-tape. Therefore, the CPT locations should be considered accurate only to the degree implied by the method used.

The CPT's were advanced to a maximum depth of forty-two (42) feet and refusal was obtained at all CPT locations. The CPT Plots are presented in Appendix B, Figures B-1 thru B-5, inclusive. The Cone Penetration Tests (CPT) were performed in accordance with ASTM D 3441. The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil, and a continuous record of cone and friction resistance versus depth is obtained in digital form at the ground surface. A specially designed truck is used to transport and house the test equipment and to provide a ten (10) ton reaction to the thrust of the hydraulic rams. Near-continuous CPT records provide: approximate correlations with soil classification; relatively accurate definition of the thickness of various soil layers; subsoils data for seismic settlement analyses; and, engineering properties of the subsoils for static settlement analyses.

Two (2) geophysical survey lines utilizing multi-channel array surface wave (MASW) methods were completed to obtain the shear wave velocity profile of the subsoils. The results are summarized on Figures A-3 and A-4. A discussion of field procedures, geophysical techniques, data processing and interpretation, and the results of the geophysical survey are given in Appendix B.

Borings from the previous investigation (J.Byer Group, Inc., May 11, 1998) are presented herein as Figures B-7 thru B-16, inclusive, Appendix B.

4.2 Laboratory Testing

The laboratory test results (J.Byer Group, Inc., May 11, 1998) are presented on the Log of Borings, Figures B-7 thru B-16, inclusive, Appendix B, where applicable, and in Appendix C.

V. SUBSURFACE CONDITIONS

5.1 Subsoil Conditions

An approximately three (3) to four (4) inch layer of asphaltic concrete overlying a five (5) to six (6) inch layer of aggregate base was encountered at the CPT locations.

The generalized stratigraphic profile at CPT locations consists of four (4) feet of fill soils (silty sand with gravels) overlying sands to silty sands with varying amounts of gravels. With respect to the fill soils, the existing Jacmar building pad compaction report indicates tests have exceeded ninety (90) percent relative compaction from approximate elevations of 316 to 323 feet and lateral extent was at least five (5) feet beyond existing building limits. At the CPT-3 location, the CPT test results indicate that the silty sands to sandy silts in the upper eight (8) feet below ground surface are in a "loose" to "medium dense" state. At other locations and in general, the subsoils are inferred to have a "medium dense" to "very dense" state to a maximum depth of forty-two (42) feet below ground surface, where refusal was encountered.

It should be noted that the native silty sands and sands at the site can be friable, which may require form-work to construct footings and measures to maintain temporary cut slope stability.

5.2 Groundwater Conditions

CPT's advanced by GEOBASE, INC. (GEOBASE) and previous borings drilled by J. Byer Group at the site, advanced to a maximum depth of forty-two (42) and twenty-five (25) feet, respectively, did not encounter groundwater to the total depth drilled; however, groundwater conditions may be altered by geologic detail between borings, by seasonal and meteorological variations, and by construction activity.

Historic highest groundwater level contours, shown on the Baldwin Park Quadrangle, Plate 1.2 of the Seismic Hazard Zone Report 022 prepared by the California Geological Survey published in 1998, indicate a historic high groundwater level of twenty-five (25) feet below existing grade at the site location; this plate is reproduced herein as Figure A-5, Appendix A. It should be noted that the Durbin Mine Pit, to the north of the site, had been excavating for the mining operation that resulted in groundwater level draw-down in excess of 100 feet below adjacent ground surface.

VI. SEISMICITY

6.1 Site Coordinates

The site latitude and longitude are 34.0815 degrees north and 117.9965 degrees west, respectively.

6.2 Site Classification

The soil classification procedure recommended by CBC 2013, subsection 1613.3.2, which references ASCE 7-10, Chapter 20, was adhered to.

The Cone Penetration Tests (CPT) and geophysical survey test results provided average shear wave velocities of 340 m/s within the top 100 feet. The shear wave profiles of the CPT's and geophysical survey presented on Figure A-6 show good correlations. To develop seismic design criteria, the subsoils within the top 100 feet at the site are judged to be Site Class D.

6.3 Seismic Design Criteria

Based on CBC 2013, subsection 1616.10.2, which references and modifies ASCE 7-10, subsection 11.4.7:

1. Site-specific, site response analysis will be required if the structure is located in site Class F soils, unless the exception to Section 20.3.1 of ASCE 7-10 is applicable.

2. Site-specific Ground-Motion Hazard Analysis (GMHA) will be required, provided that: the structure is on a site with S_1 greater than or equal to 0.6g and time-history analysis of the structure is being performed; and, the structure is seismically isolated and/or uses damping systems.

3. For buildings assigned to Seismic Design Category E or F, or when required by the building official, a GMHA shall be performed in accordance with ASCE 7, Chapter 21, as modified by Section 1803A.6 of the CBC 2013.

Based on the above criteria, since the structure is assigned to Seismic Design Category E (see subsection 6.3.1.2), a site-specific GMHA was completed. The following subsections present the seismic design parameters based on the mapped parameters and the site specific GMHA.

6.3.1 Mapped Seismic Design Parameters

6.3.1.1 Mapped Accelerations Response Spectra

Mapped, risk-targeted maximum considered earthquake, MCE_R, spectral response accelerations for 0.2 and 1.0 second periods are provided in maps published in the ASCE 7-10, which is the reference used in the CBC 2013. These maps are prepared by the USGS and the California portion of the map was prepared jointly with the CGS. These maps use results of seismic hazard analyses from both probabilistic and deterministic procedures, and are applicable to Site Class B and five (5) percent of critical damping. The mapped site accelerations are adjusted for site class effects using parameters Fa and Fv, which are functions of site class and mapped site spectral accelerations.

The mapped design horizontal spectral accelerations were evaluated in accordance with ASCE 7-10, using the US Seismic Design Maps Application (USGS, 2016) available at the USGS website: http://geohazards.gov/designmaps/us/application.php. This web application requires the inputs of site location (coordinates) and site soil classification.

The project site is Site Class D and coefficient values Fa and Fv of 1.0 and 1.5, respectively, are obtained for the site. Mapped MCE_R accelerations obtained for the project site are summarized in Table I, below.

TABLE I
MCE_R MAPPED ACCELERATIONS

PERIOD (SECONDS)	MAPPED ACCELERATION PARAMETERS (g)	SITE CLASS D	
		MCE_R ACCELERATIONS ADJUSTED FOR SITE CLASS EFFECTS (g)	RISK COEFFICIENTS
0.2	S_s: 2.241	2.241	$C_{RS} = 0.993$
1.0	S_1: 0.781	1.172	$C_{R1} = 1.013$

Based on Table I, the mapped spectral response accelerations, adjusted for Site Class D, S_{MS} and S_{M1} are 2.241 and 1.172g, respectively.

6.3.1.2 Seismic Design Category

The mapped spectral response acceleration parameter at one (1) second period (S_1) is 0.781g which is greater than 0.75g and the building is not considered to be Risk Category IV. Therefore, a Seismic Design Category E should be used for the design of the proposed structure per Section 1613.3.5 of CBC 2013.

6.3.1.3 Design Spectra Based on Mapped Parameters

Section 11.4.5 of ASCE 7-10 describes a procedure to obtain a design response spectra curve for use in cases where a design response spectrum is required by the ASCE 7-10 standard, and site-specific ground motion procedures are not used. This procedure is based on the use of the mapped spectral response accelerations adjusted for site class effects, in the determination of the design response spectra curve. Using this procedure, numerical values of the design spectral response accelerations based on the mapped parameters for the project site are provided in Table II, below.

TABLE II
MAPPED DESIGN RESPONSE SPECTRUM

Period (Seconds)	Mapped Design Spectral Response Acceleration (g)
0.00	0.598
0.105	1.494
0.20 (S_{DS})	1.494
0.52	1.494
1.00 (S_{D1})	0.781
2.00	0.391
3.00	0.260
4.00	0.195
5.00	0.156

6.3.2 Site-Specific Ground Motion Procedures - Ground Motion Hazard Analysis (Site-Specific GMHA Parameters)

6.3.2.1 General

As part of the GMHA, probabilistic and deterministic spectral response accelerations corresponding to the risk-targeted Maximum Considered Earthquake (MCE_R) are determined. The MCE_R ground motions are defined as the maximum level of earthquake ground shaking that is considered as reasonable to design normal structures against collapse.

The site-specific MCE_R spectral response acceleration at any period is taken as the lesser of the spectral response accelerations obtained using the probabilistic and deterministic methods of GMHA. The design spectral response acceleration at any period is then determined as two-thirds (2/3) of the site-specific MCE_R spectral response acceleration; however, the site specific design response spectrum should not be taken less than eighty (80) percent of the design spectral response acceleration determined from the general procedure

(ASCE 7-10, Figure 11.4-1), which is based on the mapped spectral response accelerations.

The CBC 2013 (reference ASCE 7-10) procedure for the determination of the site-specific GMHA includes:

- Determination of mapped MCE_R parameters.
- Use of the Next Generation Attenuation (NGA) relationships in the calculation of the probabilistic and deterministic response spectra.
- Use of the 2008 USGS fault model in the seismic hazard evaluations.
- Use of the risk coefficient of earthquake loading in the calculation of probabilistic response spectra
- Use of the eighty-four (84) percentile values in the determination of the characteristic earthquakes corresponding to the faults in the calculation of deterministic response spectra.
- Use of the maximum rotated horizontal component in the determination of the probabilistic and deterministic response spectra.

6.3.2.2 Probabilistic MCE_R Ground Motions

The probabilistic spectral response accelerations shall be taken as the spectral response accelerations in direction of maximum horizontal response represented by a five (5) percent damped acceleration response spectrum that is expected to achieve one (1) percent probability of collapse within a fifty (50) year period. Method 1 or 2 may be used to determine the ordinates of the probabilistic ground-motion response spectrum per ASCE 7-10, Section 21.2.1; in the current analysis, Method 1 was used.

The probabilistic seismic risk analysis is based on the premise that moderate to large earthquakes occur on mappable Quaternary faults and that the occurrence rate of earthquakes on each fault is proportional to the Quaternary fault-slip-rate. This analysis assumes that earthquakes are distributed uniformly and therefore does not consider when the last earthquake occurred on the fault. The length of rupture of the fault as a function of earthquake magnitude is accounted for, and ground motion estimates at a site are made using the magnitude of the earthquake and the closest distance from the site to the rupture zone. The probabilistic risk analysis has explicitly taken into account uncertainties associated with:

- The earthquake magnitude;
- The rupture length given magnitude;

- The location of rupture zone on the fault;
- The maximum possible magnitude of earthquakes; and,
- The acceleration at the site given magnitude of earthquake and distance from the rupture zone to the site.

Probabilistic seismic hazard analyses were performed using the computer program "2008 Interactive Deaggregations" available on the USGS website. The 2008-updates of the source and attenuation models of the NSHMP (Petersen and others, 2008) are used for the determination of the response spectra in this program. The program provides seismic-hazard deaggregations for the response spectra at periods: 0.0 s; 0.1 s; 0.2 s; 0.3 s; 0.5 s; 1.0 s; 2.0 s; 3.0 s; 4.0 s; and, 5.0 s.

For each of these periods, the program provides the average of response spectra obtained from the three NGA attenuation relationships recommended to be used by the CBC 2013 to evaluate the attenuation of earthquake energy with distance from the source. These NGA attenuation relationships are proposed by Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008). Method 1, as described in ASCE 7-10, Section 21.2.1.1, was used to determine the probabilistic (MCE_R) ground-motion response spectrum by multiplying risk coefficients to the USGS NSHMP NGA probabilistic results. The value of risk coefficients, C_R, was determined at 0.2 second period, C_{RS} = 0.993, and at one (1) second period, C_{R1} = 1.013, from Figures 22-17 and 22-18 of ASCE 7-10, respectively. The risk coefficients for the various periods were determined as shown in Table III:

TABLE III
SEISMIC RISK COEFFICIENTS (C_R)

Periods	C_R
$T \leq 0.2s$	C_{RS} = 0.993
$T \geq 1.0s$	C_{R1} = 1.013
$0.2s < T < 1.0s$	Linear Interpolation

In order to convert the spectral response obtained from the program on the USGS website to their maximum horizontal component, the result obtained for each period from the aforementioned software was multiplied by the appropriate factor to convert it to that corresponding to the maximum rotated component. Table IV presents the conversion factors used for the various periods as suggested by proposal SDPRG-1R4 (2009), Table I, page 35.

TABLE IV
FACTORS USED TO CONVERT SPECTRAL ACCELERATIONS OBTAINED FROM THE NGA RELATIONSHIPS TO THOSE CORRESPONDING TO MAXIMUM ROTATED COMPONENT

Period (Seconds)	Factor
PGA	1.1
0.1	1.1
0.2	1.1
0.3	1.1
0.5	1.2
1.0	1.3
2.0	1.3
4.0+	1.4

The probabilistic spectral response accelerations corresponding to the average spectra obtained from the aforementioned three attenuation relationships, and used for the determination of the site-specific MCE_R response spectra at the project site are shown in Figure A-7, Appendix A and an estimated shear-wave velocity of 340 m/s was used in the probabilistic seismic hazard analyses.

6.3.2.3 Deterministic MCE_R Spectra

The CBC 2013 specifies the deterministic MCE_R response acceleration at each period as the eighty-fourth (84) percentile of the largest five (5) percent damped spectral response acceleration computed at that period for characteristic earthquakes on all known active faults within the region. The spectral accelerations should correspond to the maximum rotated component of ground motion; however, the ordinate of the deterministic MCE_R ground motion response spectrum should not be taken less than the corresponding ordinate of a lower limit MCE_R response spectrum curve determined as a function of the coefficients F_a and F_v, assuming that the values of S_s and S_1 are 1.5 and 0.6, respectively.

For the project site coordinates, provided in Figure A-1, Appendix A, a search was carried out using the USGS/CGS 2008 National Seismic Hazard Maps (NSHM) -Source Parameters, and faults with characteristics that produce the strongest earthquakes at the project site were selected. Based on these results, the faults that have the largest influence on the site seismicity are the Raymond, Sierra Madre Connected and Elsinore faults. These faults and their corresponding parameters are provided in Table V.

TABLE V
FAULT PARAMETERS USED FOR THE DETERMINISTIC ANALYSIS

Fault Name	Distance from Site (Km)	Hanks Magnitude (M)	Fault Type	Preferred Dip (Degree)	Rupture Top (Km)
Raymond	8.19	6.5	SS	79	0
Sierra Madre Connected	8.75	7.2	Reverse	51	0
Elsinore; W+GI+T+J+CM	10.73	7.85	SS	84	0

Peak ground accelerations and response spectra corresponding to the characteristic earthquake for each of the aforementioned faults were determined using the average of the three (3) attenuation relationships discussed in subsection 6.3.2.2 and recommended by the CBC 2013. The Microsoft Excel spreadsheet prepared by L. Atiq and available at the website: http://peer.berkeley.edu/products/rep_nga_models.htm was used to obtain the response spectra corresponding to the characteristic earthquakes. Using this spreadsheet, the eighty-four (84) percentile (sigma plus one standard deviation) values of the spectral responses were selected. Since the CBC 2013 requires use of the maximum rotated horizontal component to be used in the analysis, the result obtained for each period from the aforementioned software was multiplied by the appropriate factor to convert it to that corresponding to the maximum rotated component. Table IV, subsection 6.3.2.2, presents the conversion factors used for the various periods as suggested by proposal SDPRG-1R4 (2009), Table I, page 35. As noted previously, a shear wave velocity of 340 m/s was used in the determination of characteristic earthquakes for each of the faults.

Figure A-8, Appendix A, shows spectral response accelerations of the characteristic earthquakes, which correspond to the specified MCE_R accelerations. This figure also shows the specified lower limits of the MCE_R spectral accelerations, obtained as described in the ASCE 7-10 standard.

By comparing the ordinates of the specified MCE_R spectral response accelerations from the faults governing maximum ground motions at the site with the corresponding ordinates from the specified lower limits of the acceleration response spectra curve, the response spectra from the deterministic method were obtained and are also shown in Figure A-8, Appendix A.

6.3.2.4 Site-Specific MCE_R Spectra

The site-specific MCE_R spectral response acceleration at any period, S_{AM}, is taken as the lesser of the spectral response accelerations obtained from the probabilistic and deterministic methods. The MCE_R probabilistic and deterministic spectra obtained as described in subsections 6.3.2.2 and 6.3.2.3, respectively, are presented in Figure A-9, Appendix A. The site-specific MCE_R spectra defined as the lesser of the probabilistic and deterministic spectra is also shown in Figure A-9, Appendix A.

6.3.2.5 Site-Specific Design Spectra

The ASCE 7-10 specifies the design spectral response acceleration at any period as two-thirds (2/3) of the site specific MCE_R spectral response acceleration; however, the design spectral response acceleration at any period should not be taken less than eighty (80) percent of the design spectral response acceleration determined using the mapped parameters for the site (see subsection 6.3.1).

The site-specific design response spectrum based on two-thirds (2/3) of site-specific MCE_R spectral response accelerations, together with the response spectra curve obtained as eighty (80) percent of the spectra based on mapped parameters for the project site are shown in Figure A-10, Appendix A. The site-specific design response spectra curve for the project site is also shown in Figure A-10, Appendix A, as the greater of the two spectra curves. Numerical values of the site-specific design spectral response accelerations for the project site are provided in Table VI.

TABLE VI
SITE-SPECIFIC DESIGN RESPONSE SPECTRA

Period (Seconds)	Site-specific Design Spectral Response Acceleration (g)
0.00	0.478
0.02	0.615
0.05	0.821
0.075	0.993
0.105	1.195
0.20	1.195
0.30	1.195
0.52	1.195
0.75	0.833
1.00	0.686
1.50	0.525
2.00	0.408
3.00	0.277
4.00	0.196
5.00	0.163

6.3.2.6 Design Acceleration Parameters

The CBC 2013/ASCE 7-10 specifies the design response spectrum at short period, S_{DS} as the design spectrum at the period of 0.2 second; however, this value should not be less than ninety (90) percent of the design spectra obtained at any period larger than 0.2 second. Also, the CBC 2013/ASCE 7-10 specifies S_{D1} as the greater of the design response spectrum at one (1) second or twice the spectrum at two seconds. The parameters S_{MS} and S_{M1} can be taken as 1.5 times S_{DS} and S_{D1}, respectively. These values shall not be less than eighty (80) percent of values determined in mapped parameters, subsection 6.3.1.

Based on the above, and the values of site-specific design response spectra provided in Table VI, the design acceleration parameters are obtained as follows:

S_{DS} = 1.20g
S_{D1} = 0.82g

6.3.2.7 Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Accelerations

From Figure 22-7 of ASCE 7-10, PGA = 0.792g is multiplied by the site coefficient F_{PGA} = 1.0 (Table 11.8-1) to obtain the mapped MCE Geometric Mean Peak Ground Acceleration (PGA_M). For Site Class D, $PGA_M = F_{PGA}$ x PGA. Therefore, PGA_M = 0.792g may be used for evaluation of liquefaction, lateral spreading, seismic settlement and soil-related issues.

6.4 Earthquake Effects

6.4.1 Liquefaction

Liquefaction occurs when the pore pressures generated within a soil mass equals the overburden pressure. This results in a loss of strength and the soil then possesses a certain degree of mobility.

Factors considered to evaluate liquefaction potential include groundwater conditions, soil type, particle size distribution, earthquake magnitude and acceleration, and soil density obtained through the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). Soils subject to liquefaction comprise saturated fine grained sands to coarse silts. Coarser-grained soils are considered free-draining and therefore dissipate excess pore pressures, while fine-grained soils posses undrained shear strength.

The Seismic Hazard Zones Map from the CDMG for the Baldwin Park Quadrangle, released in March 25, 1999 indicates that the project site is located within an area where historic occurrences of liquefaction, or local geological, geotechnical and groundwater conditions would indicate permanent ground displacement due to liquefaction (Figure A-11, Appendix A). The CPT's and geophysical survey field testing results indicate that the subsoils are in a "very dense" state below twenty-five (25) feet below existing grade and the published historic highest groundwater level is also at twenty-five (25) feet below existing grade (Figure A-5, Appendix A). Based on the aforementioned, it is judged that the subsoils at the project site posses a very low potential for liquefaction.

Liquefaction analyses results using a PGA_M of 0.792g (subsection 6.3.2.7) are provided together with the seismic settlement analyses in Appendix D.

6.4.2 Seismically Induced Settlements

Based on an examination of the subsoils conditions, seismic settlement analyses were conducted at CPT-1 and CPT-2 locations. For these analyses, a PGA_M of 0.792g based on the maximum considered earthquake geometric means peak ground acceleration, described in subsection 6.3.2.7, and an earthquake magnitude of 6.68 was used based on the USGS deaggregation (mean) results. Seismic settlements for the saturated sands were estimated using the Ishihara and Yoshimine (1992) Method and for the unsaturated sands using the Tokimatsu and Seed (1987) Method.

Based on our evaluation of the analyses results at the CPT locations, presented in Appendix D, seismic settlement at the site is anticipated to be negligible.

6.4.3 Tsunamis, Inundation, Seiche and Flooding

A tsunami is a sea wave generated by a submarine earthquake, landslide, or volcanic event. The site is not located within a coastal area. Therefore, a tsunami hazard at the site is considered very low.

A seiche is an earthquake-induced wave in a confined body of water, such as a lake, reservoir, or bay. Resulting oscillations could cause waves up to tens of feet high, which in turn could cause extensive damage along the shoreline. The most serious consequence of a seiche would be the overtopping and failure of a dam. Based on the disclosure report, the site is located within a dam failure innundation area.

According to the Federal Emergency Management Agency (FEMA), September 26, 2008, Flood Insurance Rate Map, Los Angeles County and incorporated areas, California, the proposed project site is located in Zone X, areas determined to be outside of the 0.2% annual chance floodplain (Figure A-12, Appendix A).

6.4.4 Surface Rupture

The site is not located within any of the Alquist-Priolo Earthquake Fault Zones, as shown on the Seismic Hazards Zones Map, Figure A-11, Appendix A. The likelihood of direct surface fault rupture at the site is considered very low based on the presently known tectonic framework. Cracking due to shaking from distant events is not considered a significant hazard, although it is a possibility at any site.

6.4.5 Seismically Induced Landsliding

The project site is not mapped in a potential earthquake-induced landslide area (Figure A-11, Appendix A). The closest portion of the site to the Durbin Pit Mine excavation is in excess of 200 feet (Figure A-1, page 2 of 2, Appendix A). Therefore, the hazard associated with earthquake-induced landsliding is considered low.

6.4.6 Lateral Spreading

Seismically induced lateral spreading involves primarily movement of earth materials due to ground shaking. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The potential for liquefaction at the site is considered very low. Therefore, the potential for lateral spreading at the subject site is considered very low.

6.4.7 Subsidence

Subsidence is ground settlement as a result of lowering of the groundwater table or oil extraction. Such settlements generally extend over a large area and can result in damage to the structures within the area. The subject site is not mapped within a subsidence susceptibility area and the subsoils are very dense; however, due to drawn-down water level from the Durban Quarry Pit excavation, the project site may be susceptible to subsidence.

VII. CONCLUSIONS

It is our opinion that the site is suitable for the proposed development. The following presents conclusions which may influence design and construction decisions:

- Based on geophysical survey results, Figures A-3 and A-4, Appendix A, the upper ten (10) feet below existing grade are in a "medium dense" state. Below ten (10) feet to a depth of twenty-five (25) feet, they are judged to be "medium dense" to "dense", and below twenty-five (25) feet they are "very dense". The on-site soils are non-expansive.

- Ground improvement within the upper ten (10) feet below existing grade, e.g. removal and recompaction, is recommended.

- The project site is Site Class D since the average shear wave velocity for the upper one hundred (100) feet is 340 meters per second (m/s) less than 360 m/s.

- Groundwater was not encountered at the site to the depth of exploration and is judged to be in excess of fifty (50) feet at this time. Published historic highest groundwater level is twenty-five (25) feet below existing grade.

- The project site is mapped in a liquefiable zone; however, field exploration test results and associated analyses indicate that the subsoils at the site are not liquefiable.

- Seismically-induced settlement is negligible.

- The flood insurance rate map (FIRM) prepared by the Federal Emergency Management Agency (FEMA), map number 06037C1700F, effective date September 26, 2008 show the site to be in Zone X. Zone X is an area determined to be outside of the 0.2 percent annual chance of flood plain.

VIII. SITE DEVELOPMENT RECOMMENDATIONS

8.1 General

The proposed development, outlined in subsection 3.2, is feasible from a geotechnical engineering standpoint. Project plans and specifications should take into account the appropriate geotechnical features of the site and conform to the recommendations of the geotechnical report.

8.2 Clearing

Existing structures and their foundations, concrete, asphaltic concrete, surface vegetation, trash and debris should be cleared and removed from the site. The existing fill soils, where removed, may be re-used as structural fill provided that they do not contain any deleterious materials or particles over six (6) inches in largest dimension. Topsoil and soils with organic inclusions are not considered suitable for reuse as structural fill, but it may be stockpiled for future use.

Underground facilities such as utilities, pipes or underground storage tanks may exist at the site. Removal of underground tanks is subject to state law as regulated by County or City Health and/or Fire Department agencies. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects.

Septic tanks should be removed in their entirety. Cesspools or seepage pits should be pumped of their contents and backfilled with a two-sack sand-cement slurry. Any wells, if encountered during construction, should be exposed and capped in accordance with the requirements of the regulating agencies.

Depressions resulting from the removal of foundation of existing structures, buried pipes, obstructions and/or tree roots should be backfilled with properly compacted material.

8.3 Subgrade Preparation

8.3.1 Building Pad

Within the building pads, all undocumented fills, if encountered, should be removed and replaced as properly compacted fill. The upper ten (10) to twelve (12) feet, below existing grade, of the subsoils are considered "loose" based on CPT and shear wave velocity test results. These soils should be removed and replaced as properly compacted fills, as discussed in subsection 8.4.2. The lateral extent of overexcavation beyond building/footing limits should be at least equal to the depth of fill.

Construction activities and exposure to the environment can cause deterioration of the subgrade. Therefore, it is recommended that the final condition of the subgrade soils be observed and/or tested by GEOBASE field personnel, immediately, prior to slab-on-grade construction.

8.3.2 Pavement Areas, Walkways and Patios

The subsoils beneath the pavement areas, walkways, driveways and patios should be overexcavated two (2) feet to facilitate construction of a compacted fill blanket. The lateral extent of overexcavation should be at least equal to the depth of fill.

Construction activities and exposure to the environment can cause deterioration of the subgrade. Therefore, it is recommended that the final condition of the subgrade soils be observed and/or tested by GEOBASE field personnel immediately prior to construction.

8.4 Fill Placement

8.4.1 Preparation of Surface Soils

Prior to placing any fill, the exposed surface soils should be scarified to a minimum depth of eight (8) to ten (10) inches, moisture-conditioned (wetted or dried) to at least optimum moisture content and compacted to a minimum of ninety (90) percent relative compaction based on ASTM D 1557.

8.4.2 Compaction

Cohesive soils should be placed in loose lifts not exceeding six (6) inches, moisture-conditioned to approximately two (2) to four (4) percentage points above optimum moisture content, and compacted to the minimum relative compaction listed in Table VII below.

TABLE VII
COMPACTION REQUIREMENTS

Type of Fill/Area	Relative Compaction (ASTM D 1557) Minimum Percent
Fills within building pad area	95
Other structural fill	90

Granular fill materials should be placed in loose lifts of six (6) to eight (8) inches, moisture-conditioned to at least optimum moisture content, and compacted to the minimum relative compaction listed in the preceding table.

8.5 Fill Material

The on-site soils are non-expansive and may be reused as compacted fill provided they are free of organics, deleterious materials, debris and particles over six (6) inches in largest dimension.

Any soils imported to the site for use as fill for subgrade materials should be predominantly granular and non expansive (Expansion Index less than 20) and should contain sufficient fines (approximately twenty [20] percent) so as to be relatively impermeable when compacted. The imported soils should be approved by GEOBASE prior to importing. Laboratory testing required for approval of import sources may require forty-eight (48) hours. GEOBASE should be notified of import locations a minimum of seventy-two (72) hours prior to its proposed use.

8.6 Surface Drainage

To enhance future site performance, it is recommended that all pad drainage be collected and directed away from proposed structures to disposal areas. For soils areas, we recommend that a minimum of five (5) percent gradient away from foundation elements be maintained. All roof drains should be connected to solid pipes discharging to the curb or other suitable area drains. It is important that drainage be directed away from foundations and that proper drainage patterns be established at the time of construction and maintained throughout the life of the structures.

Landscape areas within ten (10) feet of the building perimeter should consist of planters that have sealed bottoms and bottom drains to prevent infiltration of water into the adjacent foundation soils. The surface of the ground in these areas should also be maintained at a minimum gradient of five (5) percent towards surface area drains

Care should be exercised in controlling surface runoff onto permanent and temporary slopes. The area back of slope crests should be graded such that water will not be allowed to flow freely onto the slope face. If excavations of temporary slopes are carried out in the rainy season, appropriate erosion protection measures may be required to minimize erosion of the slope cuts.

8.7 Temporary Excavations

Temporary construction excavations are anticipated for construction of utility trenches, footings and overexcavation. Temporary construction excavations in soils may be made vertically without shoring to a depth of approximately four (4) feet below adjacent surrounding grade. For deeper cuts in soils, the slopes should be properly shored or sloped back at least 1H:1V (Horizontal:Vertical) or flatter. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the toe of excavation unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at forty-five (45) degrees below the edge of any nearby adjacent existing site facilities including underground pipelines, should be properly shored to maintain foundation support of the adjacent structures and utilities.

All excavations and shoring systems should meet, as a minimum, the requirements given in the State of California Occupational Safety and Health Standards. Stability of temporary slopes is the responsibility of the contractor.

8.8 Trench Backfill

It is our opinion that utility trench backfill could be placed and compacted by mechanical means. Jetting or flooding of backfill material is not recommended. If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, other methods of utility trench compaction may also be appropriate, as approved by the geotechnical engineer at the time of construction. All backfill should be compacted to a minimum of ninety (90) and ninety-five (95) percent relative compaction per ASTM D 1557 for outside and inside building limits, respectively.

8.9 Code Section 111

Relative to the County of Los Angeles Code Section 111, the proposed development will not adversely affect the site or adjacent properties. Further, based on the investigation and analyses reported herein, including review of available reports (Section II), the subject property will be free of potential geologic and geotechnical hazards such as settlement, liquefaction, landsliding and fault rupture.

IX. FOUNDATION RECOMMENDATIONS

9.1 General

The following recommendations have been formulated from visual, physical and analytical considerations of existing site conditions and are believed to be applicable for the proposed development.

The on-site soils are not expansive. The following recommendations are based on non-expansive surface soils. Foundation and slab reinforcement configurations should meet, as a minimum, the requirements of the governing agencies and/or CBC 2013.

9.2 Foundation Alternatives

The results of the site investigation indicate that the foundations for the proposed developments may be influenced by the relatively compressible nature of the subsoil layers encountered in the upper ten (10) to twelve (12) feet below existing grade.

9.3 Footings

Footings based on native soils or properly compacted fill soils may be used to support the proposed structures. Footings should have a minimum width of twenty-four (24) inches and should be placed a minimum three (3) feet below the lowest adjacent grade.

9.3.1 Soil Bearing Pressures

Spread and continuous footings based on native soils and/or compacted fills, as described in subsection 8.4 may be designed for an allowable dead-plus-live load bearing pressure of 5,000 psf. The aforementioned allowable bearing capacities are based on the assumption that the bases of footings are a minimum three (3) feet below lowest adjacent grade. These bearing pressure may be increased by one-third (1/3) for short-term wind or seismic loads. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed the above-mentioned allowable bearing values.

Footings placed closer than one (1) width apart should be structurally tied, e.g. parking structure along gridline P3.

Surcharge of one foundation by another depends on horizontal and vertical locations of the two foundations relative to each other. This condition is anticipated, at this time, along the west wall of the parking structure between grid lines P3 and P6. In general terms, surcharge may be computed assuming pressure distribution with a plane descending at a 1H:1V (Horizontal:Vertical) from the edge of the upper footings. Footings on top of one another should be connected. Scaled sections should be provided to the geotechnical engineer for evaluation.

Foundations adjacent to descending slopes must meet slope set-back requirements per CBC 2013, subsection 1808.7.2 and Figure 1808.7.1.

Recommendations for footing for minor structures are outlined in subsection 9.4.

9.3.2 Footings Adjacent to Trenches or Existing Footings

Where footings are located adjacent to utility trenches, they should extend below a one-to-one plane projected upward from the inside bottom corner of the trench. Footing excavations adjacent to the footings of existing buildings should be carried out such that the existing footings are not undermined.

9.3.3 Settlement

Total static settlement of footing foundations constructed as described above are not anticipated to exceed one (1.0) inch and differential settlement is not anticipated to exceed one-half (½) inch. Maximum footing widths of sixteen (16) feet were used to estimate the aforementioned settlements.

9.3.4 Lateral Load Resistance

Lateral loads (wind or seismic) against structures may be resisted by friction between the bottom of foundations and the supporting soils. A friction coefficient of 0.4 is recommended for compacted fill and/or undisturbed native soils. An allowable lateral bearing pressure equal to an equivalent fluid weight of 200 pounds per cubic foot to a maximum of 5,000 pounds per square foot acting against the foundations may also be used, provided the foundations are poured tight against compacted fill/native soils. The frictional resistance and lateral resistance of the soils may be combined without reduction in determining total lateral resistance.

9.3.5 Footing Observations

All foundation excavations should be observed by GEOBASE prior to the placement of forms, reinforcement, or concrete, for verification of conformance with the intent of these recommendations and confirmation of the bearing capacities. All loose or unsuitable material should be removed prior to the placement of concrete. Materials from footing excavations should not be spread in slab-on-grade areas unless compacted.

9.4 Footings for Minor Structures

Spread or continuous footings may be used for the support of minor structures (minor retaining walls, and free-standing walls) that are structurally separated from the parking structure and MOB. These footings may be underlain by a minimum of two (2) feet of properly compacted fill, as outlined in subsection 8.3.2, provided that the risk of future maintenance can be tolerated. Alternatively, all undocumented fills, where encountered, should be removed and replaced as properly compacted fill. Footings shall be reinforced in accordance with the recommendations of the structural engineer.

For the support of minor structures that are structurally separated from the MOB and Parking Structure, footings may be designed for an allowable dead-plus-live load bearing pressure of 1,500 psf. These structures may be designed for the presumptive design parameters outlined in the California Building Code, CBC 2013.

The above bearing pressures may also be increased by one-third (1/3) for short-term wind or seismic loads. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed the above-mentioned allowable bearing values. Footings placed closer than one (1) width apart should be structurally tied.

All foundation excavations should be observed by GEOBASE prior to the placement of forms, reinforcement, or concrete, for verification of conformance with the intent of these recommendations and confirmation of the bearing capacities. All loose or unsuitable material should be removed prior to the placement of concrete. Materials from footing excavations should not be spread in subgrade areas unless compacted.

9.5 Pole Foundations

Pole foundations may be designed for an allowable passive pressure of 400 pounds per square foot per foot of depth starting at two (2) feet below adjacent grade. The maximum passive pressure should not exceed 5,000 psf.

9.6 Basement Walls and Retaining Walls

9.6.1 *Earth Pressures*

Wall backfill is anticipated to consist of "very low" expansive soils. The walls should be designed to resist lateral pressures imposed by the surrounding soils and surcharge loads. It is recommended that for static loading condition: walls which are free to rotate at the top (at least 0.01radian deflection) should be designed to resist a lateral pressure imposed by an equivalent fluid weighing thirty-five (35) pounds per cubic feet; and, walls that are structurally braced against movement at the top should be designed to resist a lateral pressure equivalent to that imposed by a fluid weighing fifty-six (56) pounds per cubic foot. In addition, a uniform pressure equal to one-third (1/3) and one-half (½) of any vertical pressure adjacent to the basement wall should be assumed to act on the free and braced walls, respectively. These aforementioned pressures assume that positive drainage will be provided as recommended in subsection 9.6.2.

For seismic loading conditions, where appropriate, the dynamic loading increment of active earth pressures against basement walls should be taken as twenty-two (22) psf per foot of height distributed in an inverted triangular distribution.

9.6.2 *Wall Backfill*

The wall backfill should be well drained to relieve possible hydrostatic pressures on the wall. A pre-fabricated drainage system such as Miradrain, Eakadrain or equivalent, installed in accordance with the manufacturer's recommendations, may be used. Alternatively, the wall should be designed to withstand hydrostatic pressures.

The basement walls below existing grade should be waterproofed to prevent moisture build-up on the interior sides of the walls as a result of water migration from the soils in contact with the walls. The water proofing should be applied for the full height of the basement walls and walls below existing grade, and meet as a minimum the requirements of the CBC 2013.

9.7 Ultimate Values

The recommended design values presented in this report are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values may be multiplied by the factors given in Table VIII:

TABLE VIII
LOAD FACTORS FOR ULTIMATE DESIGN

Foundation Loading	Ultimate Design Loading
Bearing Value (without increase)	3
Passive Pressure	1.33
Coefficient of Friction	1.25

In no event, however, should the foundation sizes be reduced from those required for support of dead-plus-live loads when using working stress values.

9.8 Floor Slabs

Concrete slab-on-grade may be used. The subgrade for the floor slab should be prepared in accordance with the recommendations provided in subsections 8.3 and 8.4.

Slab-on-grade floors should be designed by the Structural Engineer using applicable CBC requirements and designed for the intended use and loading, including temperature and shrinkage stresses. Thickness of floor slabs should be at least five (5) inches actual and determined by the project Structural Engineer for the project loading and service conditions. Slabs in moisture sensitive areas should be damproofed in accordance with CBC 2013, subsection 1805.2.

X. PAVEMENT RECOMMENDATIONS

10.1 Asphaltic Concrete Pavement

An R-value of sixty-five (65) was used for the design (Osborne, 1985). The following alternative preliminary minimum pavement sections may be used. The traffic index assumed in Table IX, below, ***should be confirmed by the Civil Engineer*** and R-value tests should be performed during grading, prior to finalizing the pavement section.

TABLE IX
ASPHALTIC CONCRETE PAVEMENT SECTIONS

PAVEMENT UTILIZATION	TRAFFIC INDEX	ASPHALTIC CONCRETE (INCHES)	CRUSHED AGGREGATE BASE (INCHES)
Automobile parking areas	5	3	4
Truck and bus loading/unloading areas and driveways	6	4	4

The upper twelve (12) inches of subgrade soils, below the aggregate base, should be scarified, moisture conditioned and recompacted to a minimum of ninety-five (95) percent relative compaction, at to slightly above optimum moisture content, based on ASTM D 1557.

The crushed aggregate base must meet CALTRANS "Class 2 Aggregate Base" specifications and should be compacted to at least ninety-five (95) percent relative compaction based on ASTM D 1557. Asphaltic concrete should be compacted to at least ninety five (95) percent of the density obtained with the California Kneading Compactor (CAL 304).

10.2 Rigid Pavement

A Portland Cement concrete (PCC) pavement may also be used. In the design of the PCC pavement section shown in Table X, below, the following design parameters were used:

- Modulus of subgrade reaction of the soil, k (R-Value = 65) -- 250 pci
- Modulus of rupture of concrete, MR -- 550 psi
- Traffic Category, TC -- C

- Average daily truck traffic, ADTT -- 100

The traffic category and average daily truck traffic should be confirmed by the civil engineer.

Based on the design parameters presented above, the following rigid pavement section, calculated in general conformance with the procedure recommended by ACI 330R-01, may be used.

TABLE X
PCC PAVEMENT SECTION

PAVEMENT UTILIZATION	PCC Minimum Thickness (inches)
Truck loading/unloading areas (TC = C)	6

The upper twelve (12) inches of subgrade soils below the PCC should be scarified, moisture conditioned and recompacted to a minimum of ninety-five (95) percent relative compaction, at to slightly above optimum moisture content, based on ASTM D 1557.

The PCC pavement reinforcement should be designed by the structural engineer for shrinkage, temperature stresses and loading conditions. A thickened edge should be constructed on the outside of the concrete pavements subject to wheel loads. Control joints should be included in the design of the PCC by the structural engineer at a maximum spacing of fifteen (15) feet each way.

XI. SOIL CORROSIVITY -- IMPLICATIONS

Corrosivity series (electrical conductivity, pH, chloride and water soluble sulfate tests) tests should be conducted during grading. Alternatively, Type V Portland cement should be used for the construction of concrete structures in contact with the subgrade soils and metals in contact with the subgrade soils should be protected.

XII. PLAN REVIEW, OBSERVATIONS AND TESTING

Post-investigation services are an important and integrated part of this investigation and should be carried out by GEOBASE. The project foundation and grading plans, and specifications should be forwarded to GEOBASE for review for conformance with the intent of the soils recommendations.

Geotechnical observations of excavation bases should be carried out prior to fill placement. Observations and testing of all fill placement should be carried out on a continuous basis to verify the design assumptions and conformance with the intent of the recommendations. Observations of footing bases should be carried out prior to concrete pour.

XIII. LIMITATIONS

This investigation was performed in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is intended for use by the client and its representatives, and with regard to the specific project discussed herein. Any changes in the design or location of the proposed new structure, however slight, should be brought to our attention so that we may determine how they may affect our conclusions. The conclusions and recommendations contained in this report are based on the data relating only to the specific project and location discussed herein. This report does not relate any conclusions or recommendations about the potential for hazardous and/or contaminated materials existing at the site.

The analyses and recommendations submitted in this report are based upon the observations noted during drilling of the borings, interpretation of laboratory test results, and geological evidence. This report does not reflect any variations which may occur away from the borings and which may be encountered during construction. If conditions observed during construction are at variance with the preliminary findings, we should be notified so that we may modify our conclusions and recommendations, or provide alternate recommendations, if necessary.

The recommendations presented herein assume that the plan review, observations and testing services, outlined in Section XII of the report, will be provided by GEOBASE. During execution of the aforementioned services, GEOBASE can finalize the report recommendations based on observations of actual subsurface conditions evident during construction. GEOBASE cannot assume liability for the adequacy of the recommendations if another party is retained to observe construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project, and incorporated into the plans and specifications. In this respect, it is recommended that we be allowed the opportunity to review the project plans and the specifications for conformance with the geotechnical recommendations.

This office does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for other than our own personnel on the site. Therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein to be unsafe.

This report is subject to review by the appropriate regulating agencies.

Respectfully submitted
GEOBASE, INC.

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R.C.E. 82460
Associate Engineer

J-M. Chevallier, P.E., G.E.
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Managing Principal

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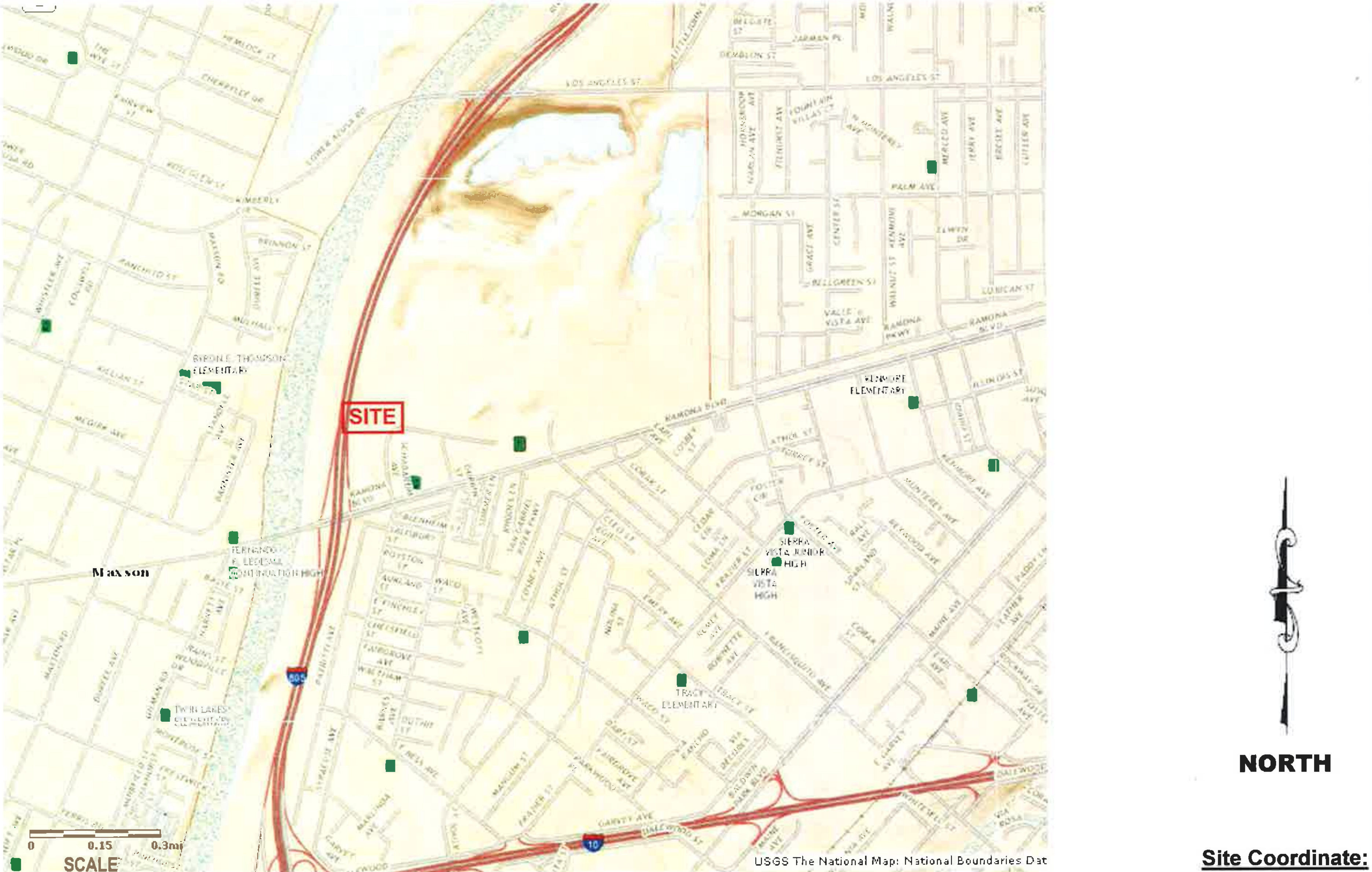
APPENDIX A



Site Coordinate:
Lat: 34.0815
Lon: -117.9965

Source: Arcgis.com, ESRI, USGS The National Map: National Boundaries Dataset, National Elevation Dataset, Geographic Names Information System, National Hydrography Dataset, National Land Cover Database, National Structures Dataset, and National Transportation Dataset; U.S. Census Bureau - TIGER/Line; HERE Road Data

GEOBASE

SITE LOCATION MAP

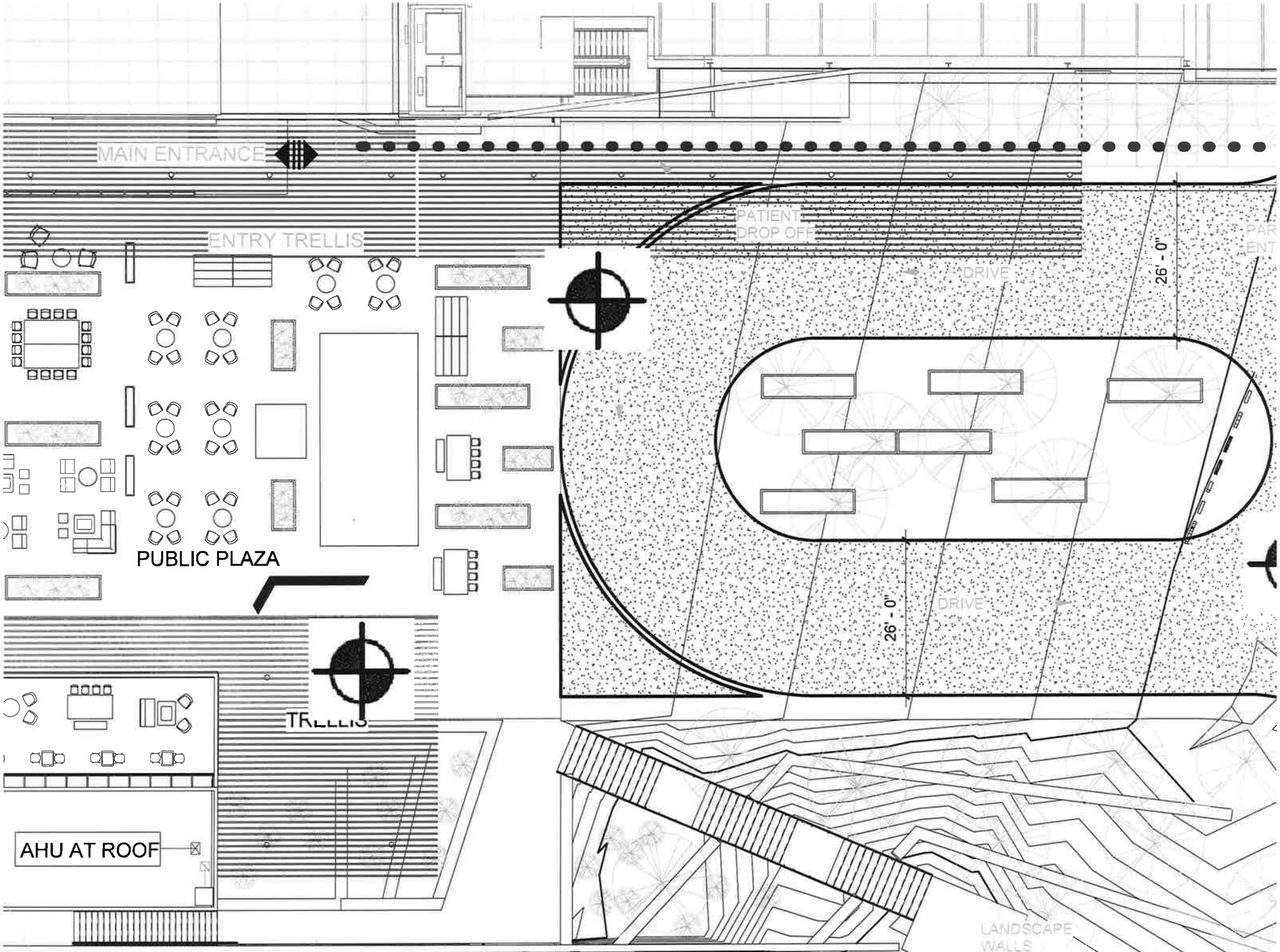
Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

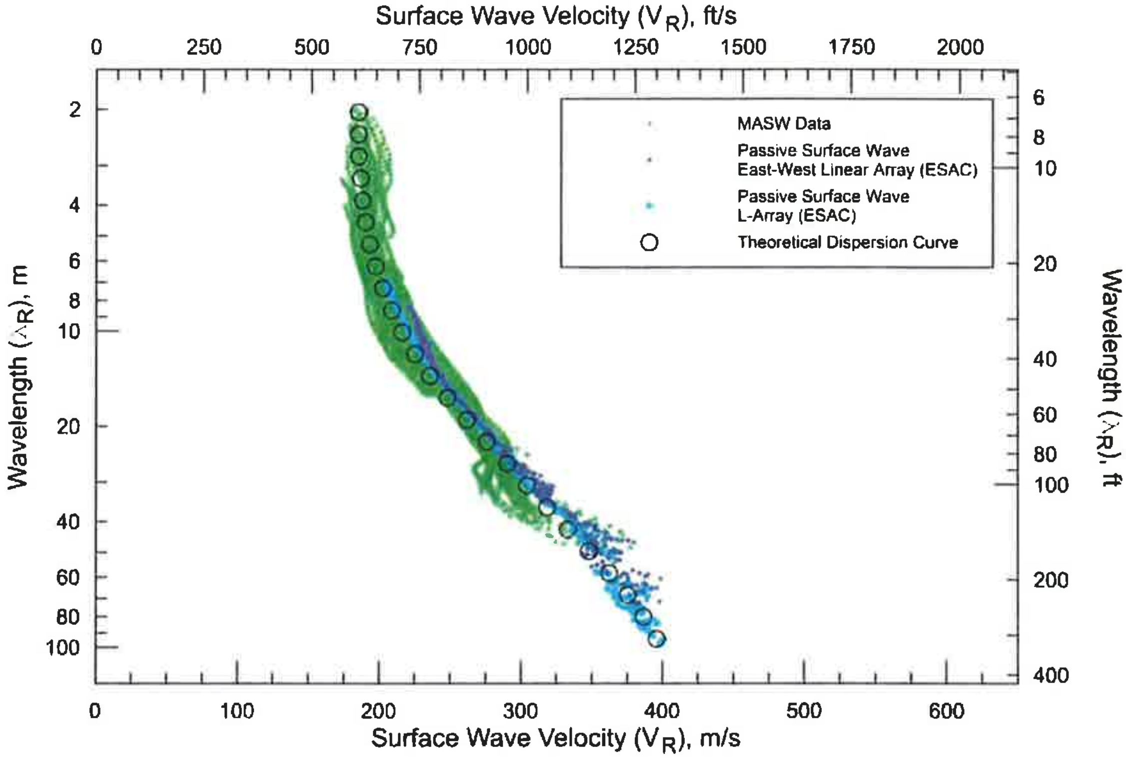
C.314.70.02

FIGURE A-1

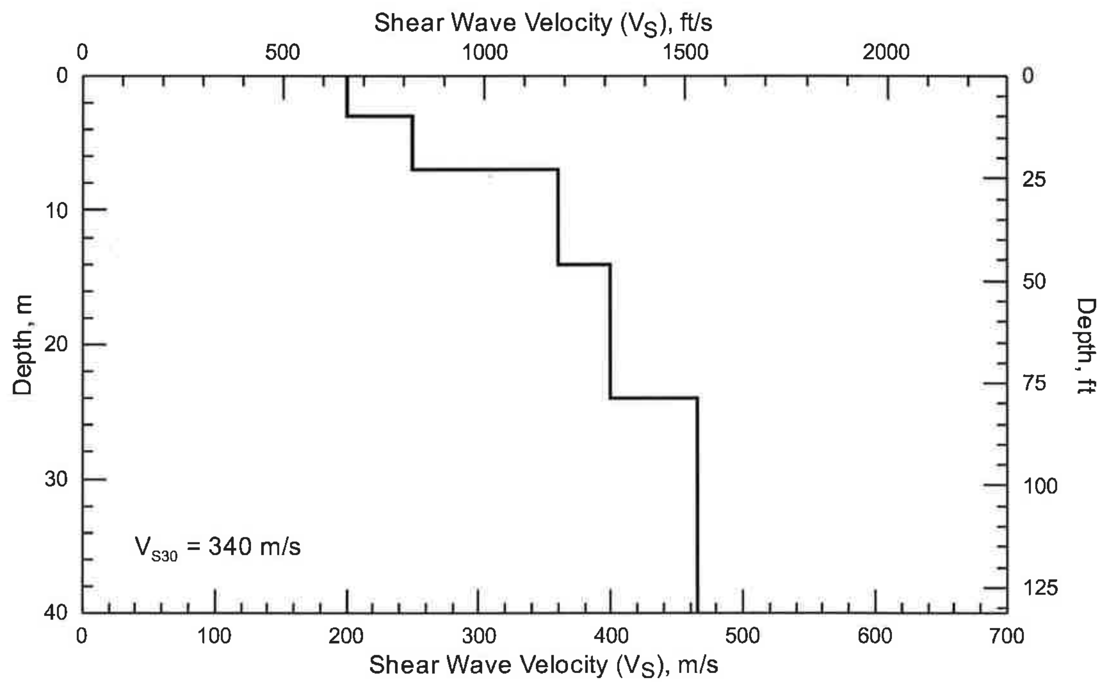
Note: This drawing is part of GEOBASE INC.'s report C.314.70.02 dated April 2016 and should be read with the entire report for evaluation.

GEOBASE	**SITE LOCATION MAP** Kaiser Permanente – Irwindale Specialty MOB 12761 Schabarum Ave Irwindale, California C.314.70.02 FIGURE A-1


MAIN ENTRANCE
ENTRY TRELLIS
PATIENT DROP OFF
DRIVE
26' - 0"
DRIVE
26' - 0"
PUBLIC PLAZA
AHU AT ROOF
LANDSCAPE WALLS

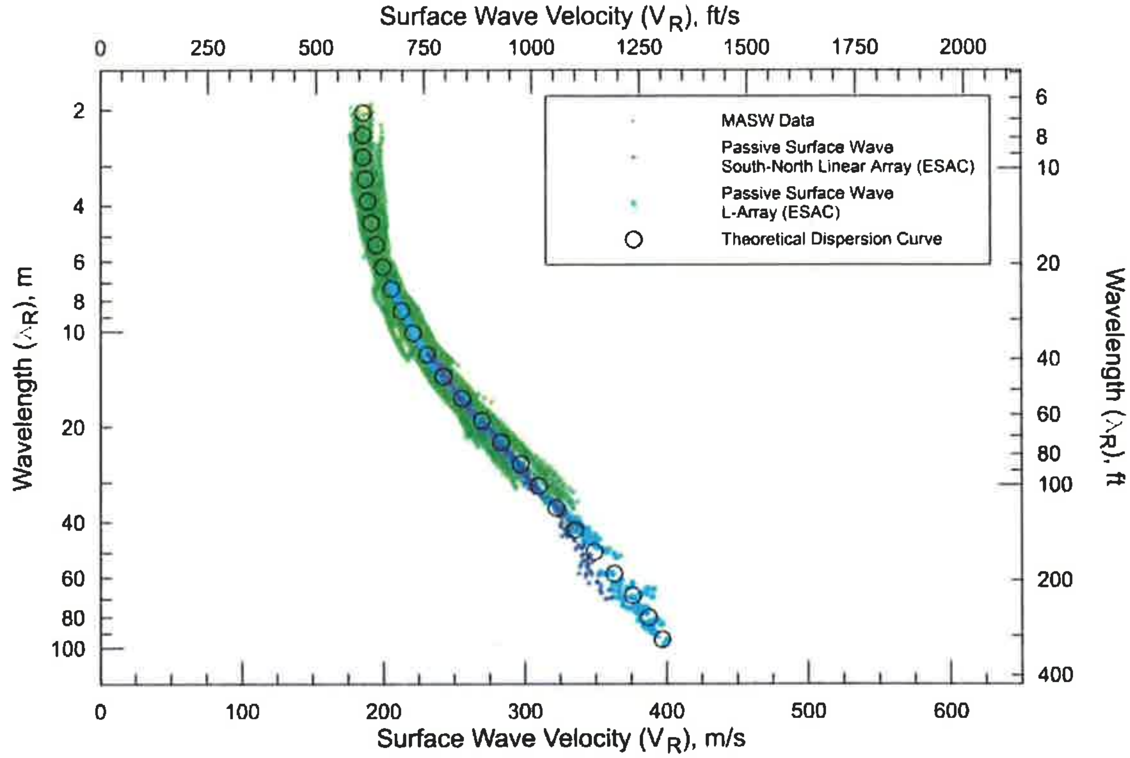


Comparison of Field Experimental Data and Theoretical Dispersion Curve from Active and Passive Surface Wave Arrays

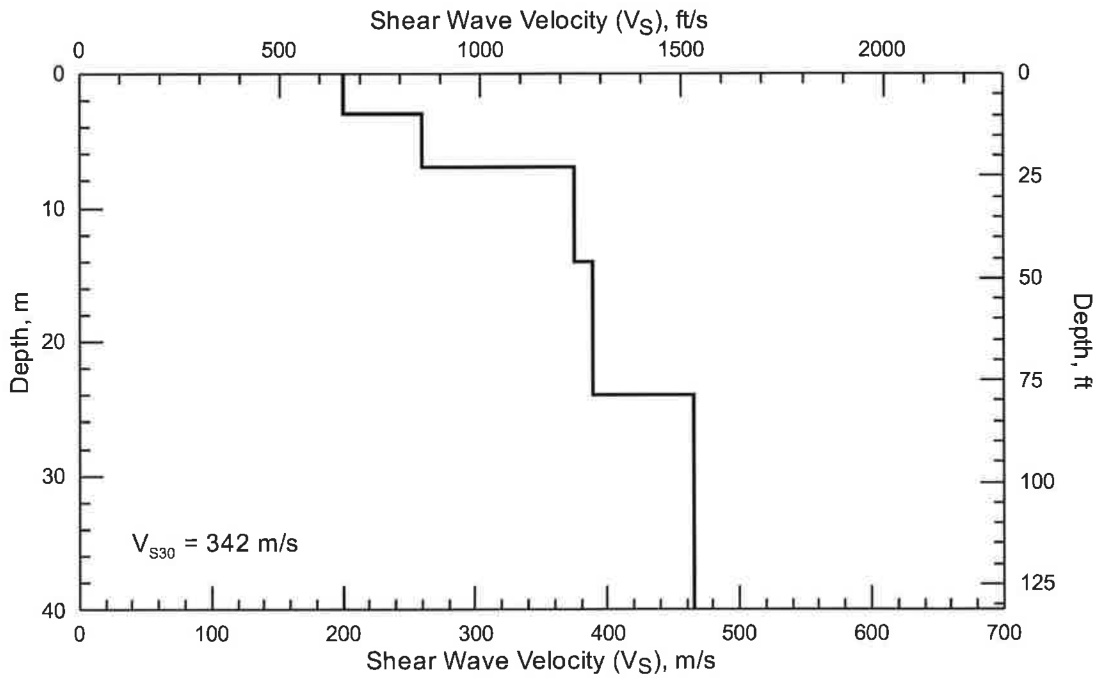


V_S Model from Active and Passive Surface Wave Arrays

GEOVision geophysical services	FIGURE 2 VELOCITY MODEL FOR ACTIVE (ARRAY 2) AND PASSIVE (ARRAY 1) SURFACE WAVE ARRAYS
Project No.: 15087	12761 SHABARUM AVENUE IRWINDALE, CALIFORNIA
Date: MAR. 19, 2015	
Drawn By: D. CARPENTER	
Approved By: A. MARTIN	PREPARED FOR GEOBASE, INC.



Comparison of Field Experimental Data and Theoretical Dispersion Curve from Active and Passive Surface Wave Arrays



V_S Model from Active and Passive Surface Wave Arrays

GEOVision geophysical services	FIGURE 3 VELOCITY MODEL FOR ACTIVE (ARRAY 3) AND PASSIVE (ARRAY 1) SURFACE WAVE ARRAYS
Project No.: 15087 Date: MAR. 19, 2015	12761 SHABARUM AVENUE IRWINDALE, CALIFORNIA
Drawn By: D. CARPENTER Approved By: A. MARTIN	PREPARED FOR GEOBASE, INC.

Explanations:

30 — Depth to ground water in feet

● Borehole Site

Note:

Data obtained from Plate 1.2 Historically Highest Groundwater Contours and Borehole Log Data Locations, Baldwin Park Quadrangle, Seismic Hazard Zone Report 022, 1998.

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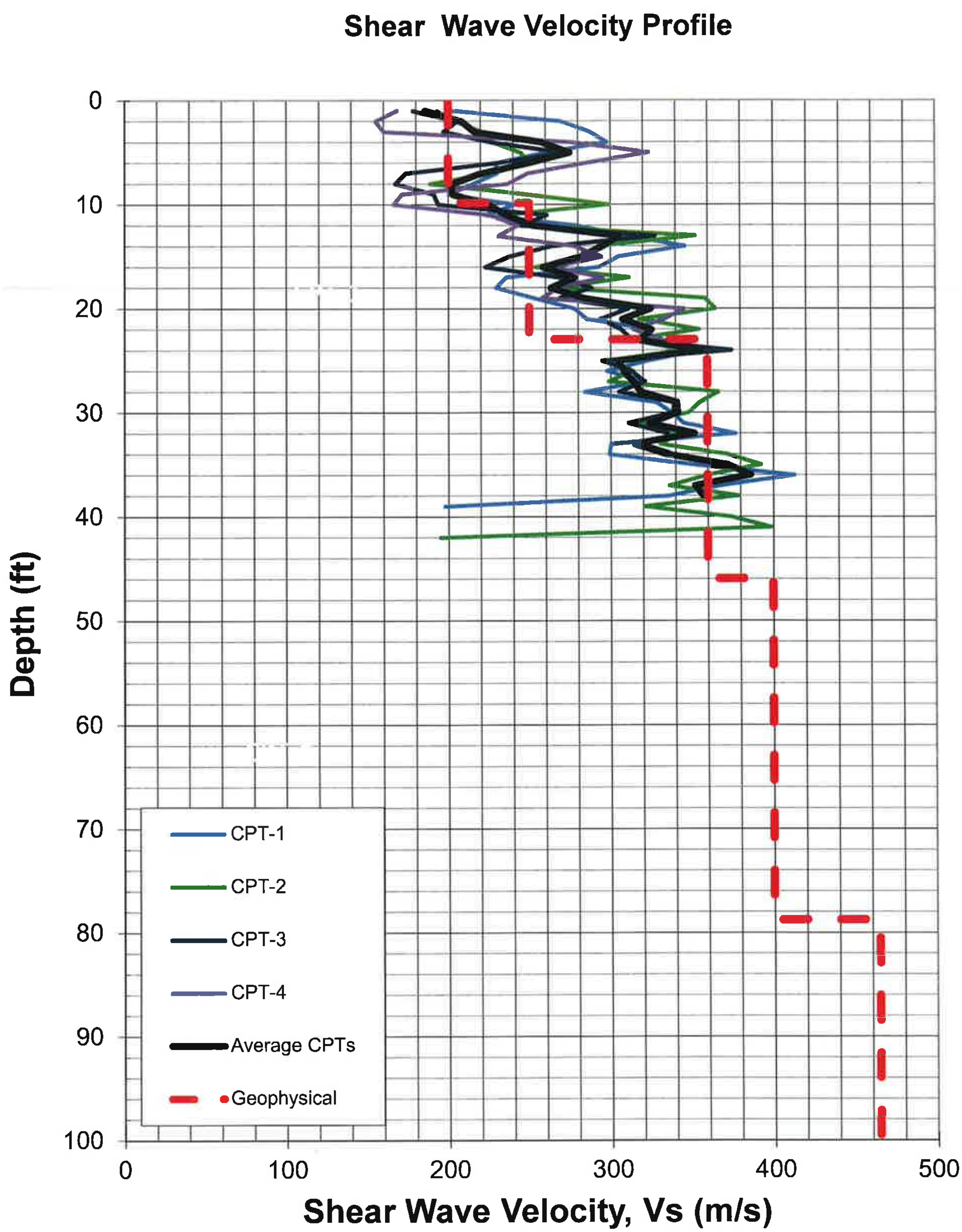
HISTORIC HIGHEST GROUNDWATER LEVELS
Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-5

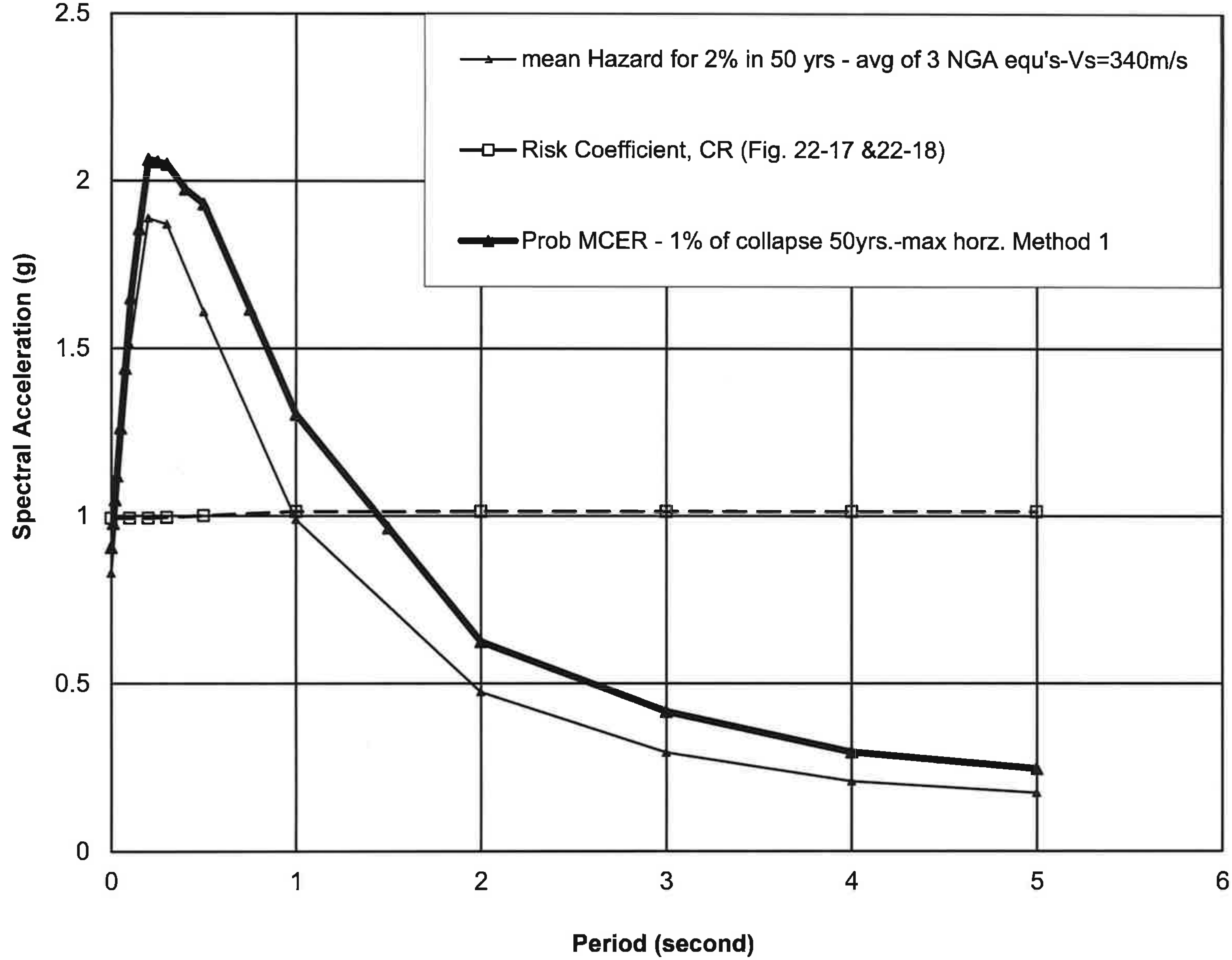


GEOBASE

CPTs AND GEOPHYSICAL SURVEY SHEAR WAVE PROFILES
Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-6

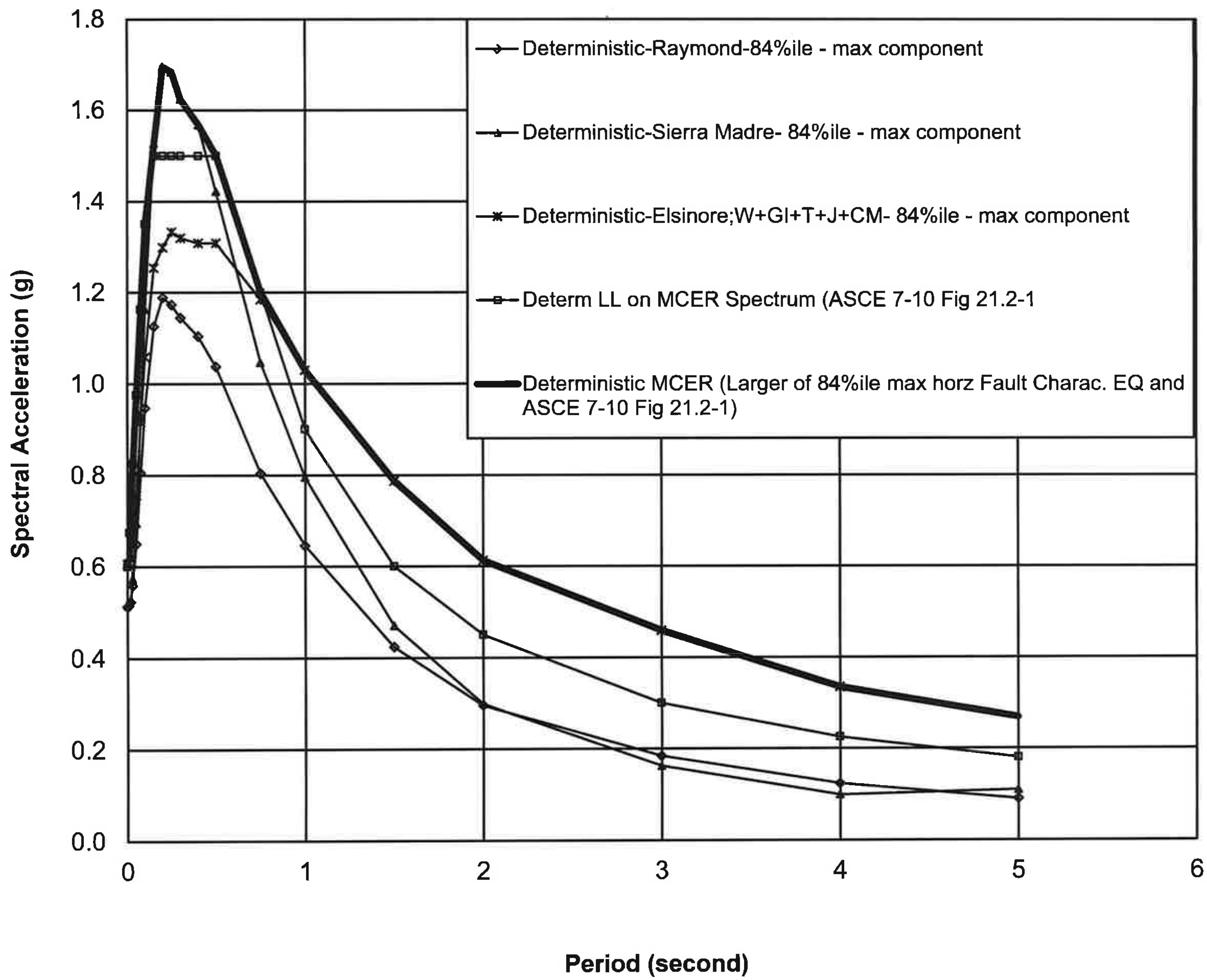


GEOBASE

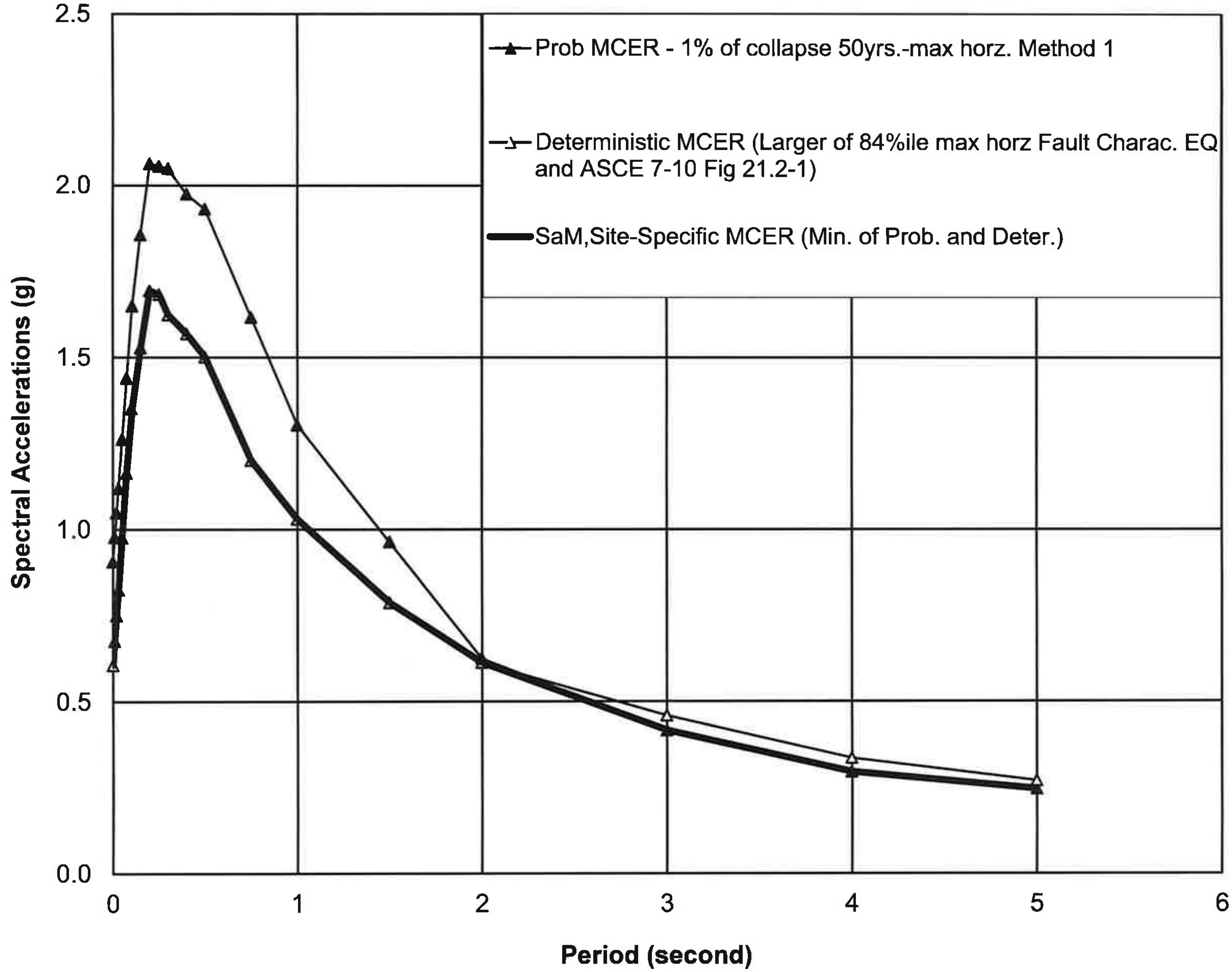
PROBABILISTIC MCER RESONSE SPECTRA
Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-7



GEOBASE	**DETERMINISTIC MCER RESONSE SPECTRA** **Kaiser Permanente – Irwindale Specialty MOB** **12761 Schabarum Ave** **Irwindale, California**	
	C.314.70.02	**FIGURE A-8**

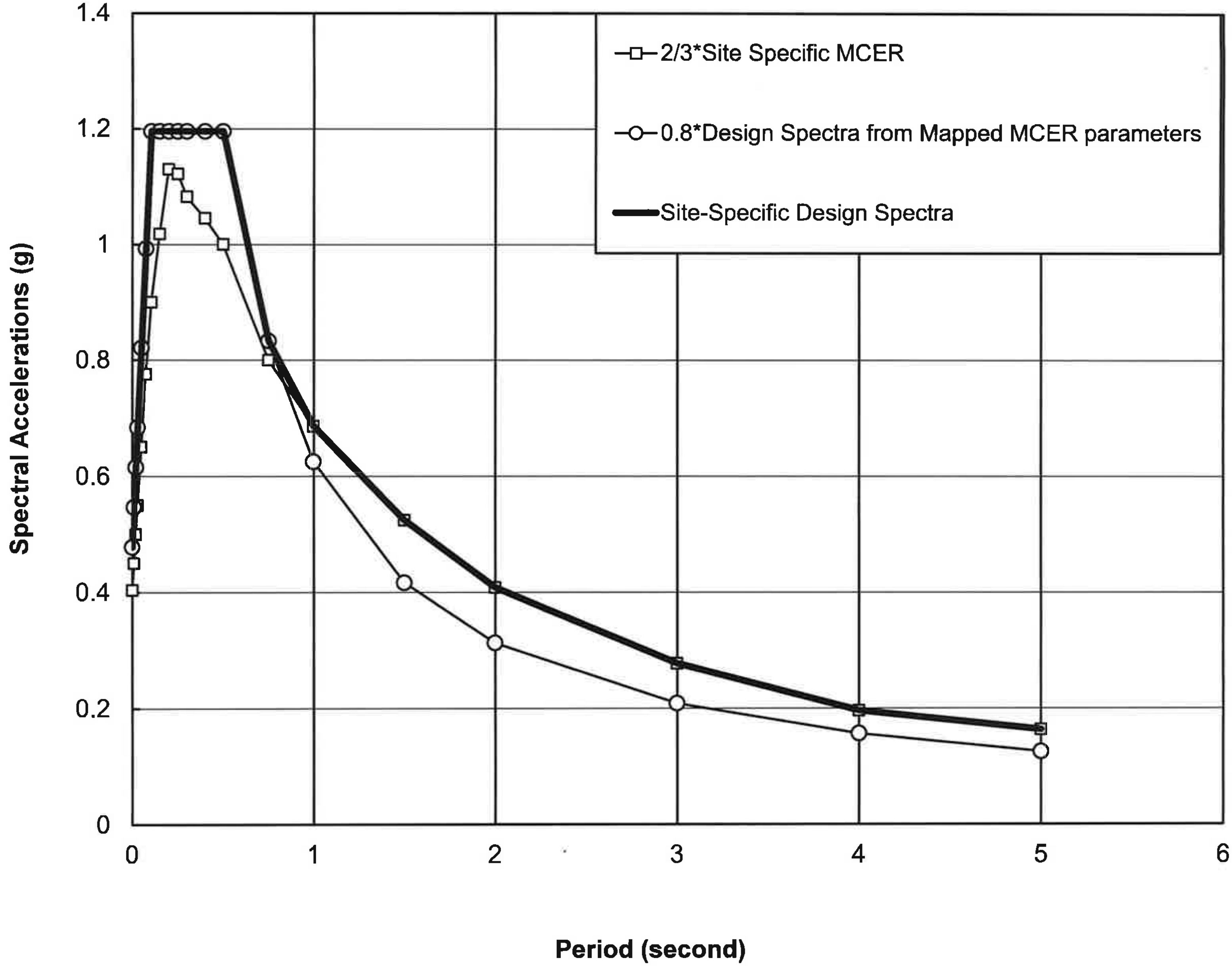


GEOBASE

SITE-SPECIFIC MCER RESONSE SPECTRA
Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-9

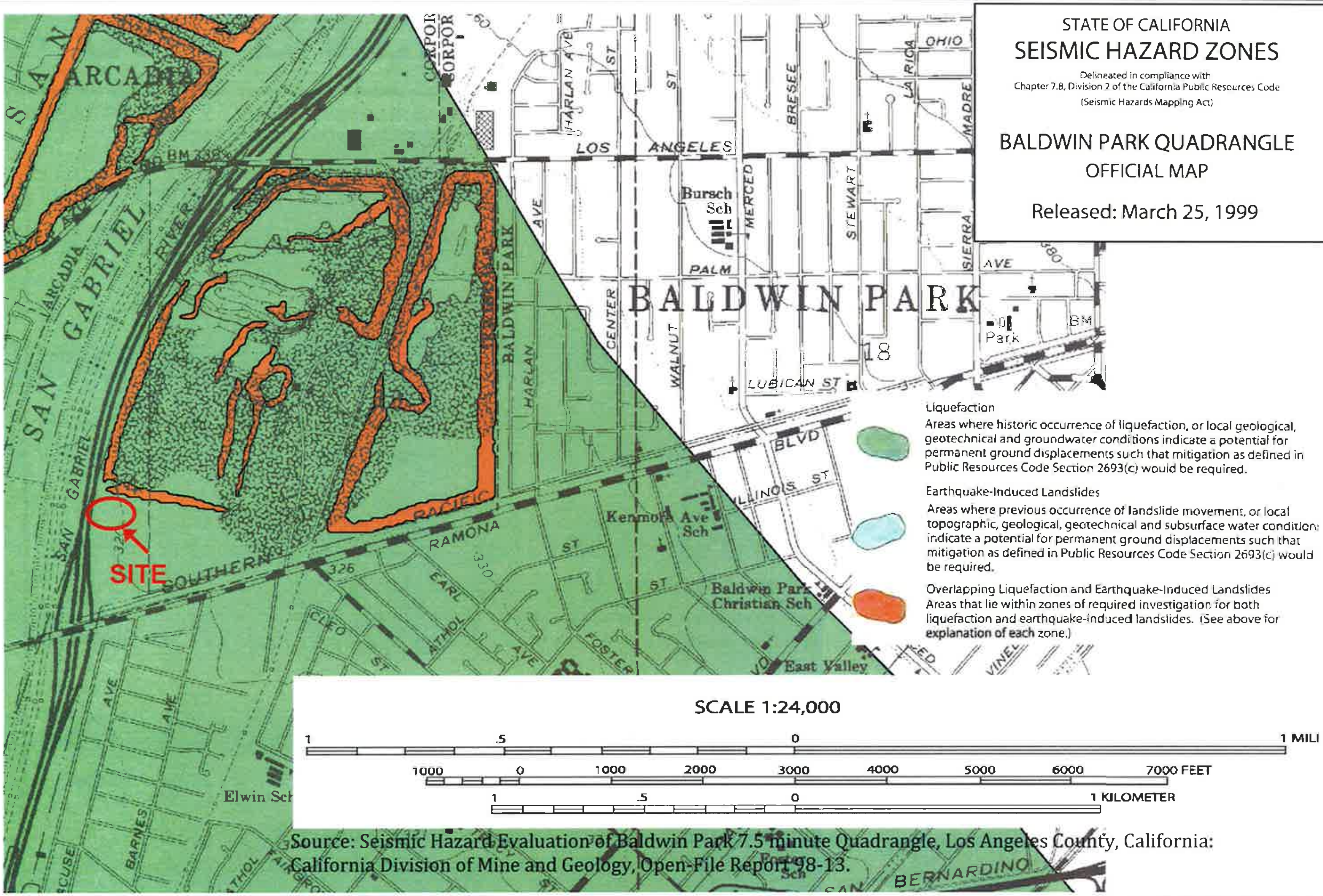


GEOBASE

SITE-SPECIFIC DESIGN RESONSE SPECTRA
Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-10



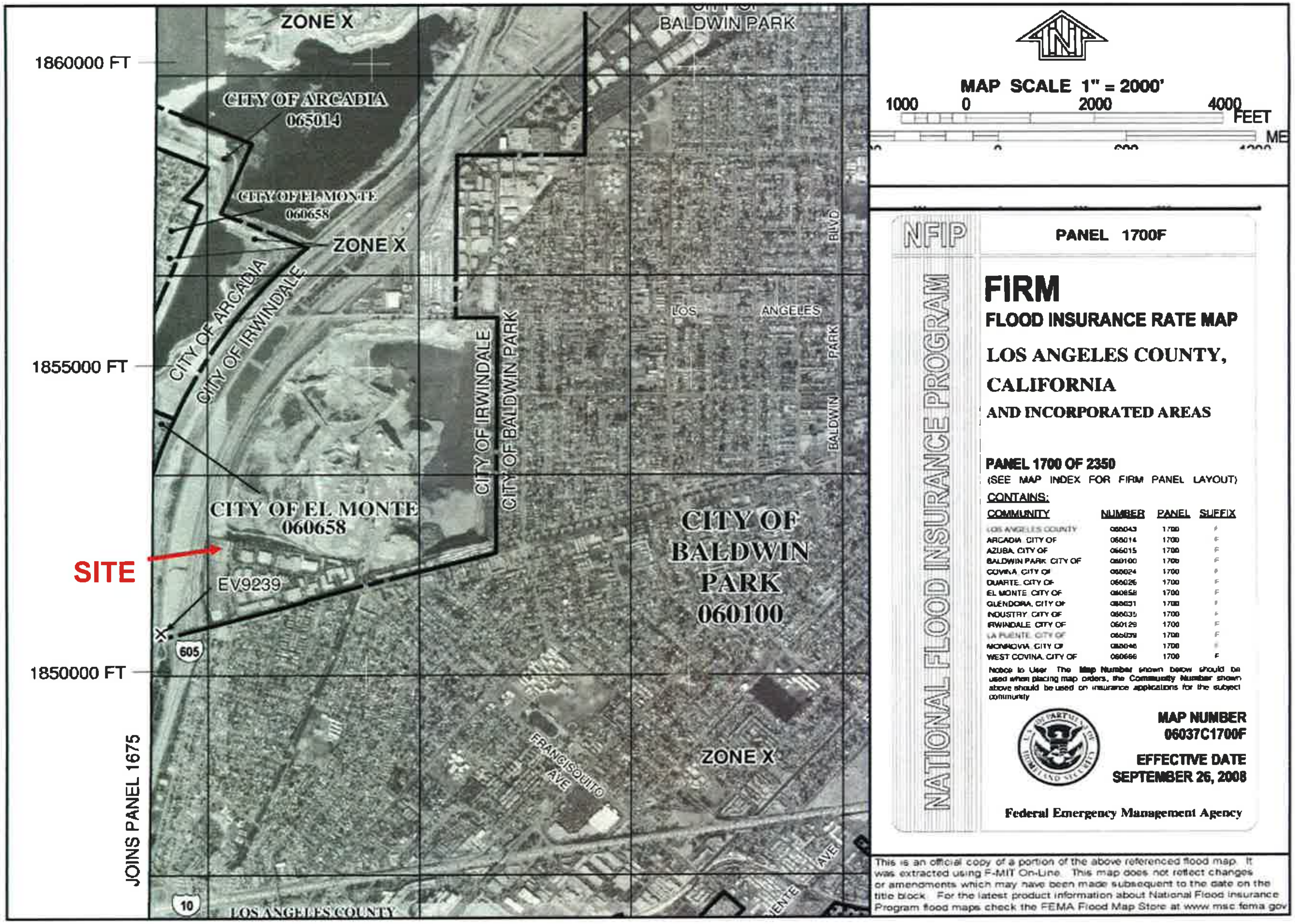
GEOBASE

SEISMIC HAZARD ZONES MAP

Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-11



The SITE is in Zone X – Area determined to be outside of 0.2% annual chance floodplain.

GEOBASE

FEMA FLOOD MAP

Kaiser Permanente – Irwindale Specialty MOB
12761 Schabarum Ave
Irwindale, California

C.314.70.02

FIGURE A-12

APPENDIX B

The terms and symbols used on the Log of Borings to summarize the results of the field investigation and subsequent laboratory testing are described in the following:

It should be noted that materials, boundaries, and conditions have been established only at the boring locations, and are not necessarily representative of subsurface conditions elsewhere across the site.

A. PARTICLE SIZE DEFINITION (ASTM D2487 AND D422)

Boulder	-- larger than 12-inches	Sand, medium	-- No.40 to No. 10 sieves
Cobble	-- 3-inches to 12-inches	Sand, fine	-- No.200 to No. 40 sieves
Gravel, coarse	-- 3/4-inch to 3-inches	Silt	-- 5μm to No. 200 sieves
Gravel, fine	-- No.4 sieve to 3/4 -inch	Clay	-- smaller than 5 μm
Sand, coarse	-- No.10 to No.4 sieve		

B. SOIL CLASSIFICATION

Soils and bedrock are classified and described according to their engineering properties and behavioral characteristics. The soil of each stratum is described using ASTM D2487 and D2488.

The following adjectives may be employed to define percentage ranges by weight of minor components:

trace	--	1-10%	some	--	20-35%
little	--	10-20%	"and" or "y"	--	35-50%

The following descriptive terms may be used for stratified soils:

parting	--	0 to 1/16-in. thickness;	layer	--	½-in. to 12-in. thickness;
seam	--	1/16 to ½-in. thickness;	stratum	--	greater than 12-in. thickness.

C. SOIL DENSITY AND CONSISTENCY

The density of coarse grained soils and the consistency of fine grained soils are described on the basis of the Standard Penetration Test:

COARSE GRAINED SOILS		FINE GRAINED SOILS		
DENSITY	SPT BLOWS PER FOOT	ESTIMATED CONSISTENCY	SPT BLOWS PER FOOT	ESTIMATED RANGE OF UNCONFINED COMPRESSIVE STRENGTH (TSF)
very loose	less than 4	very soft	less than 2	less than 0.25
loose	5 to 10	soft	2 to 4	0.25 to 0.50
medium	11 to 30	firm (medium)	5 to 8	0.50 to 1.0
dense	31 to 50	stiff	9 to 15	1.0 to 2.0
very dense	over 50	very stiff	16 to 30	2.0 to 4.0
		hard	over 30	over 4.0

D. STANDARD PENETRATION TEST (SPT) -- D1586

The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, 2-inches outside diameter and 1 3/8-inches inside diameter, is driven eighteen (18) inches. The sampler is seated in the first six (6) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

E. ABBREVIATION OF LABORATORY TEST DESIGNATIONS

C	Consolidation	pH	pH
CBR	California Bearing Ratio	pp	Pocket Penetrometer
Ch	Water Soluble Chlorides	PS	Particle Size
DS	Direct Shear	RV	R-Value
EI	Expansion Index	SE	Sand Equivalent
ER	Electrical Resistivity	SG	Specific Gravity
k	Permeability	SO_4	Water Soluble Sulfates
MD	Moisture	TX	Triaxial Compression
MP	Modified Proctor Compaction Test	TV	Torvane Shear
O	Organic Content	U	Unconfined Compression

F. STRATIFICATION LINES

The stratification lines indicated on the boring logs and profiles represent the ***approximate*** boundary between material types and the transition may be gradual.

SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

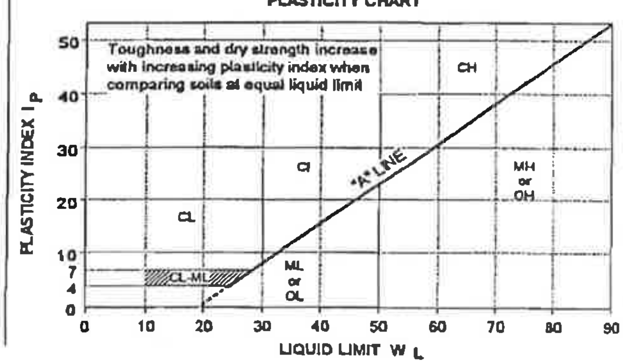
MAJOR DIVISION			GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils	Strong color or odor and often fibrous texture	
COARSE-GRAINED SOILS (More than half by weight larger than No. 200 sieve size)	GRAVELS (More than half coarse fraction larger than No. 4 sieve size)	CLEAN GRAVELS	GW		Well-graded Gravels, Gravel-Sand mixtures (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3	
			GP		Poorly-graded Gravels and Gravel-Sand mixtures (<5% fines)	Not meeting all above requirements	
		DIRTY GRAVELS	GM		Silty Gravels, Gravel-Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p<4$	
			GC		Clayey Gravels, Gravel-Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p>7$	
	SANDS (More than half coarse fraction smaller than No. 4 sieve size)	CLEAN SANDS	SW		Well-graded Sands, Gravelly Sands (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3	
			SP		Poorly-graded Sands or Gravelly Sands (<5% fines)	Not meeting all above requirements	
		DIRTY SANDS	SM		Silty Sands, Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p<4$	
			SC		Clayey Sands, Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p>7$	
FINE-GRAINED SOILS (More than half by weight passes No. 200 sieve size)	SILTS Below "A" line on plasticity chart; negligible organic content		ML		Inorganic Silts and very fine Sands, Rock Flour, Silty Sands of slight plasticity	$W_L<50$	See chart below
			MH		Inorganic Silts micaceous or diatomaceous, fine Sandy or Silty soils	$W_L>50$	
	CLAYS Above "A" line on plasticity chart; negligible organic content		CL		Inorganic Clays of low plasticity, Gravelly, Sandy, or Silty Clays, lean Clays	$W_L<30$	
			CI		Inorganic Clays of medium plasticity, Silty Clays	$W_L>30, <50$	
			CH		Inorganic Clays of high plasticity, fat Clays	$W_L>50$	
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart		OL		Organic Silts and organic Silty Clays of low plasticity	$W_L<50$	
			OH		Organic Clays of high plasticity	$W_L>50$	

The soil of each stratum is described using ASTM D2487 and D2488 modified slightly so that an inorganic clay of "medium plasticity" is recognized.

ADDITIONAL SOIL CLASSIFICATION

- Fill Soil
- Ss Sandstone
- Cs Claystone
- Ms Siltstone

PLASTICITY CHART



GEOBASE

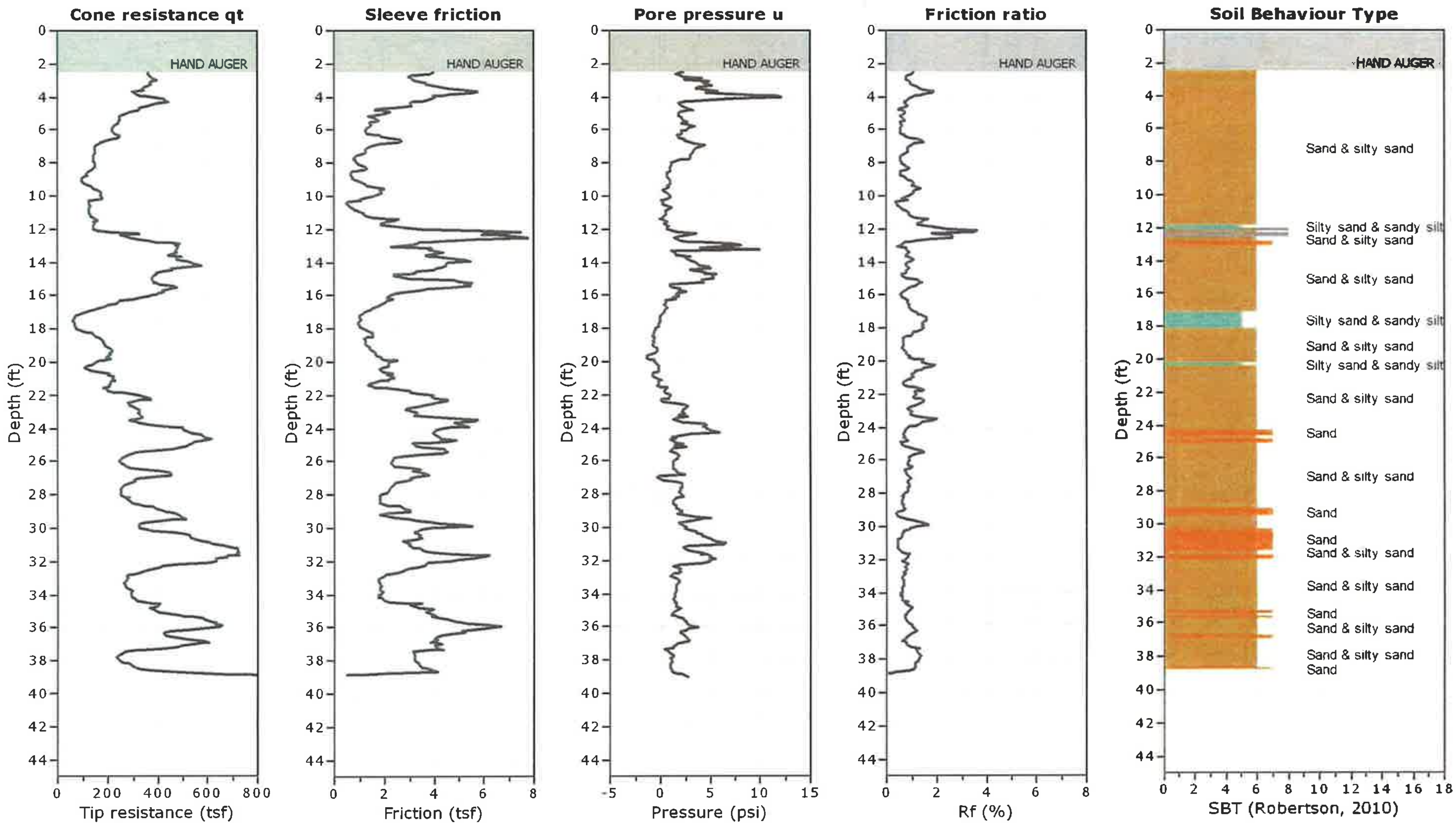
EXPLANATION OF TERMS AND SYMBOLS USED

Figure B-1

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

Project: GEOBASE, Inc.
Location: 12761 Schabarum Ave Irwindale, CA

CPT: CPT-1
Total depth: 39.04 ft, Date: 3/16/2015
Cone Type: Vertek

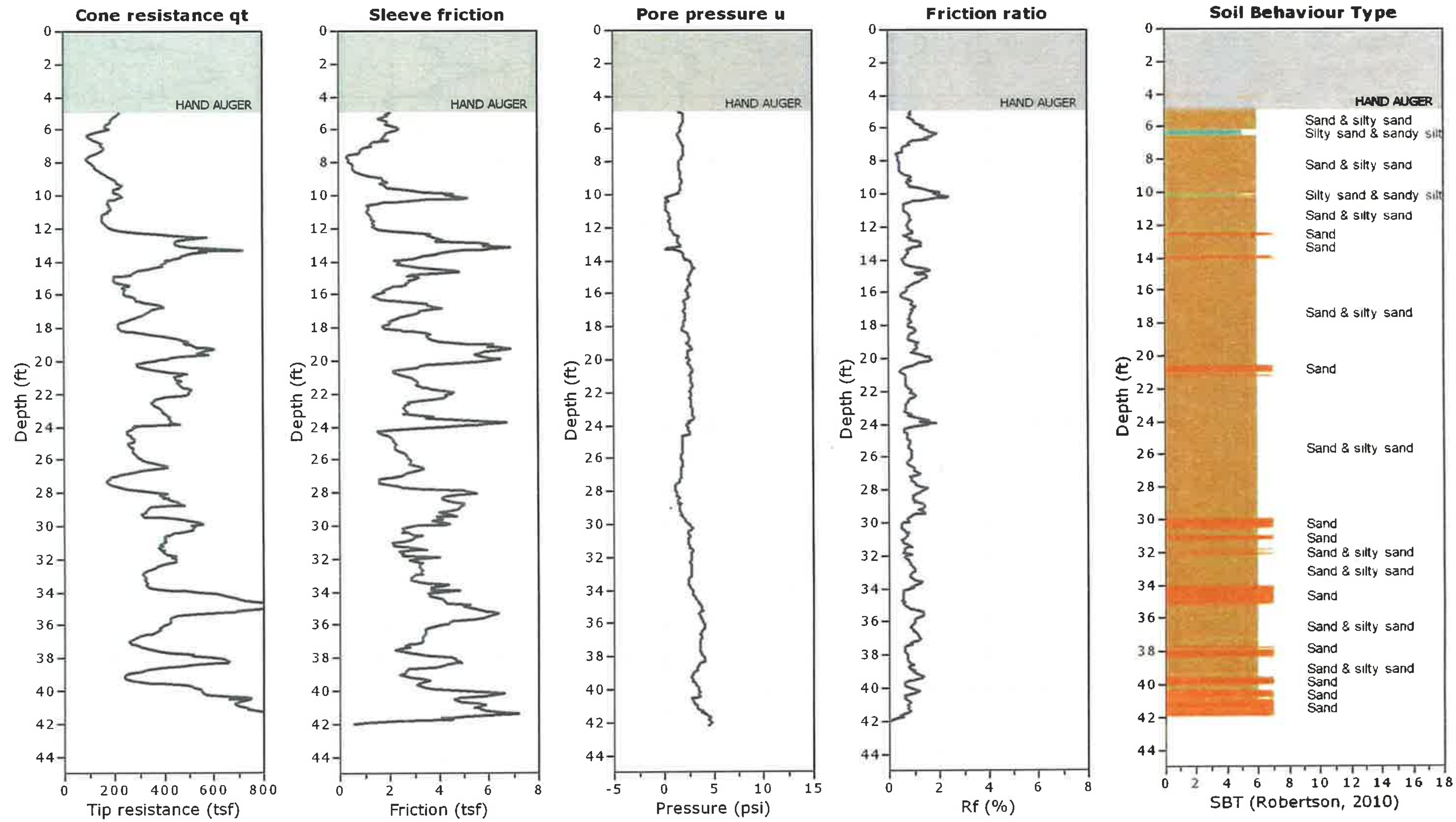


CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 4/3/2015, 2:06:33 PM
Project file: C:\GeobaseIrwindale3-15\CPeT Data\Plot Data\Plots w-ha.cpt

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

Project: GEOBASE, Inc.
Location: 12761 Schabarum Ave Irwindale, CA

CPT: CPT-2
Total depth: 42.16 ft, Date: 3/16/2015
Cone Type: Vertek



CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 4/3/2015, 2:06:07 PM
Project file: C:\GeobaseIrwindale3-15\CPeT Data\Plot Data\Plots w-ha.cpt

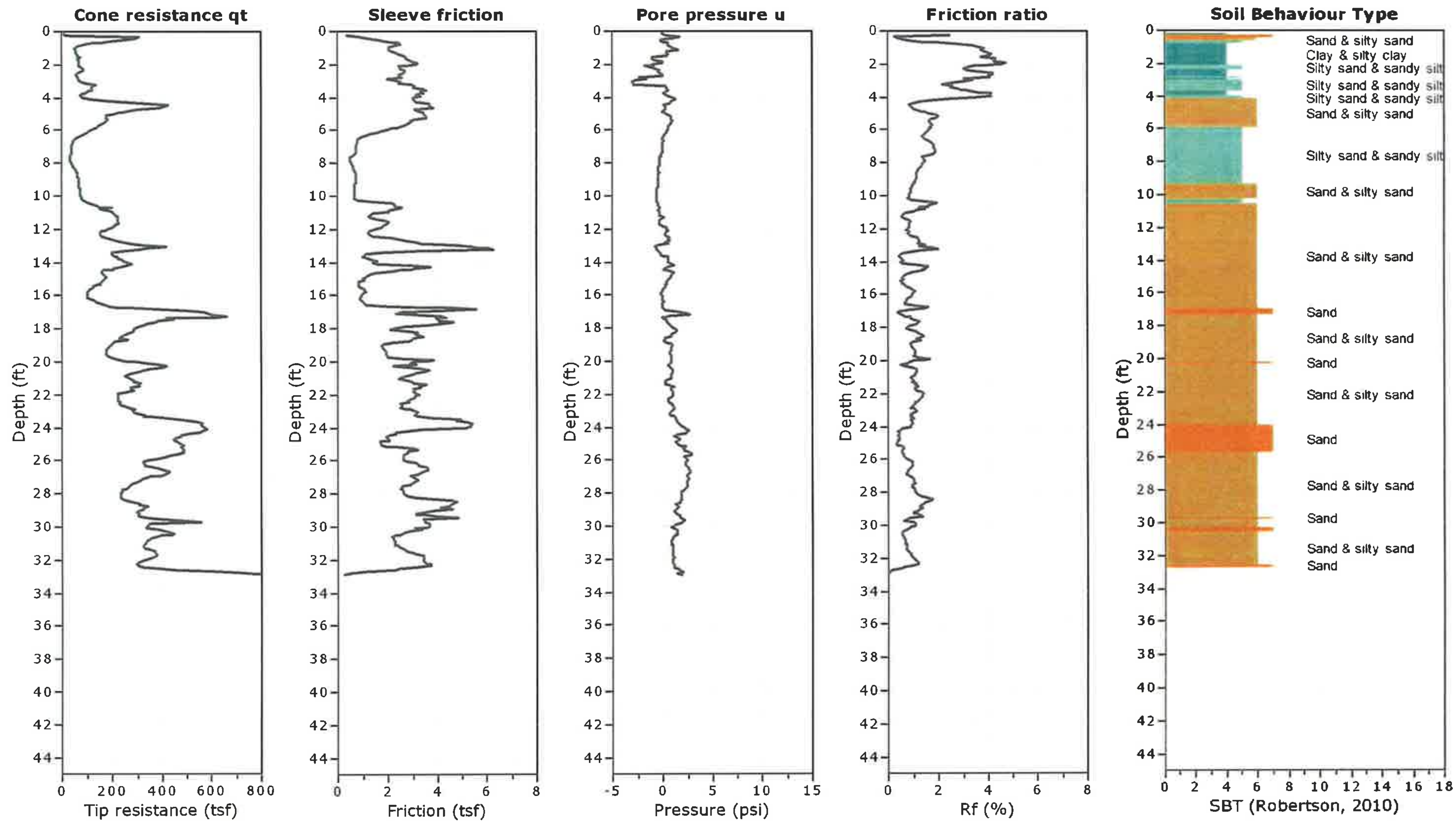
C.314.70.00

FIGURE B-3

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

Project: GEOBASE, Inc.
Location: 12761 Schabarum Ave Irwindale, CA

CPT: CPT-3
Total depth: 32.99 ft, Date: 3/16/2015
Cone Type: Vertek



CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 4/3/2015, 2:05:08 PM
Project file: C:\GeobaseIrwindale3-15\CPeT Data\Plot Data\Plots w-ha.cpt

0

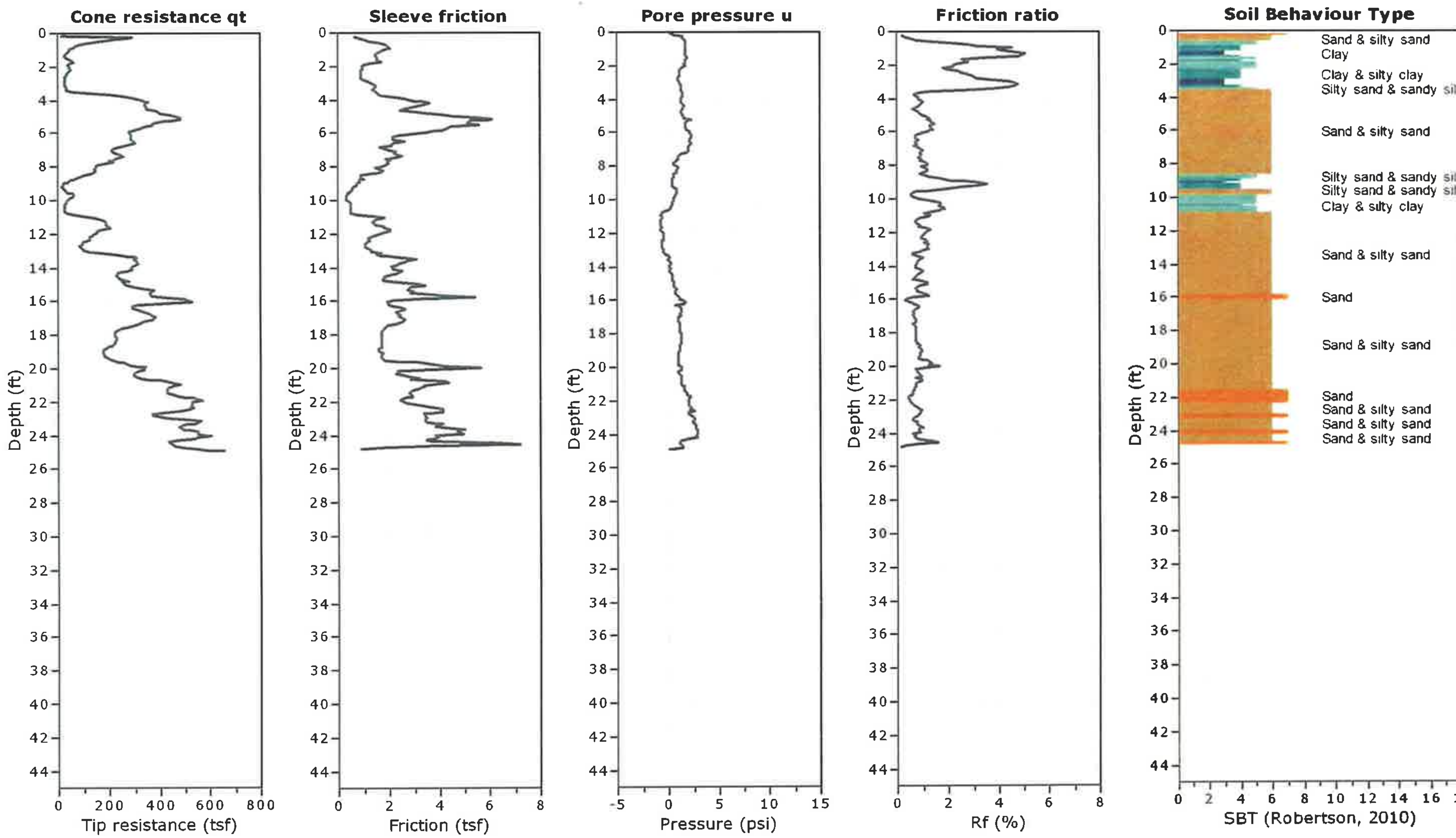
C.314.70.00

FIGURE B-4

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

Project: GEOBASE, Inc.
Location: 12761 Schabarum Ave Irwindale, CA

CPT: CPT-4
Total depth: 24.95 ft, Date: 3/16/2015
Cone Type: Vertek



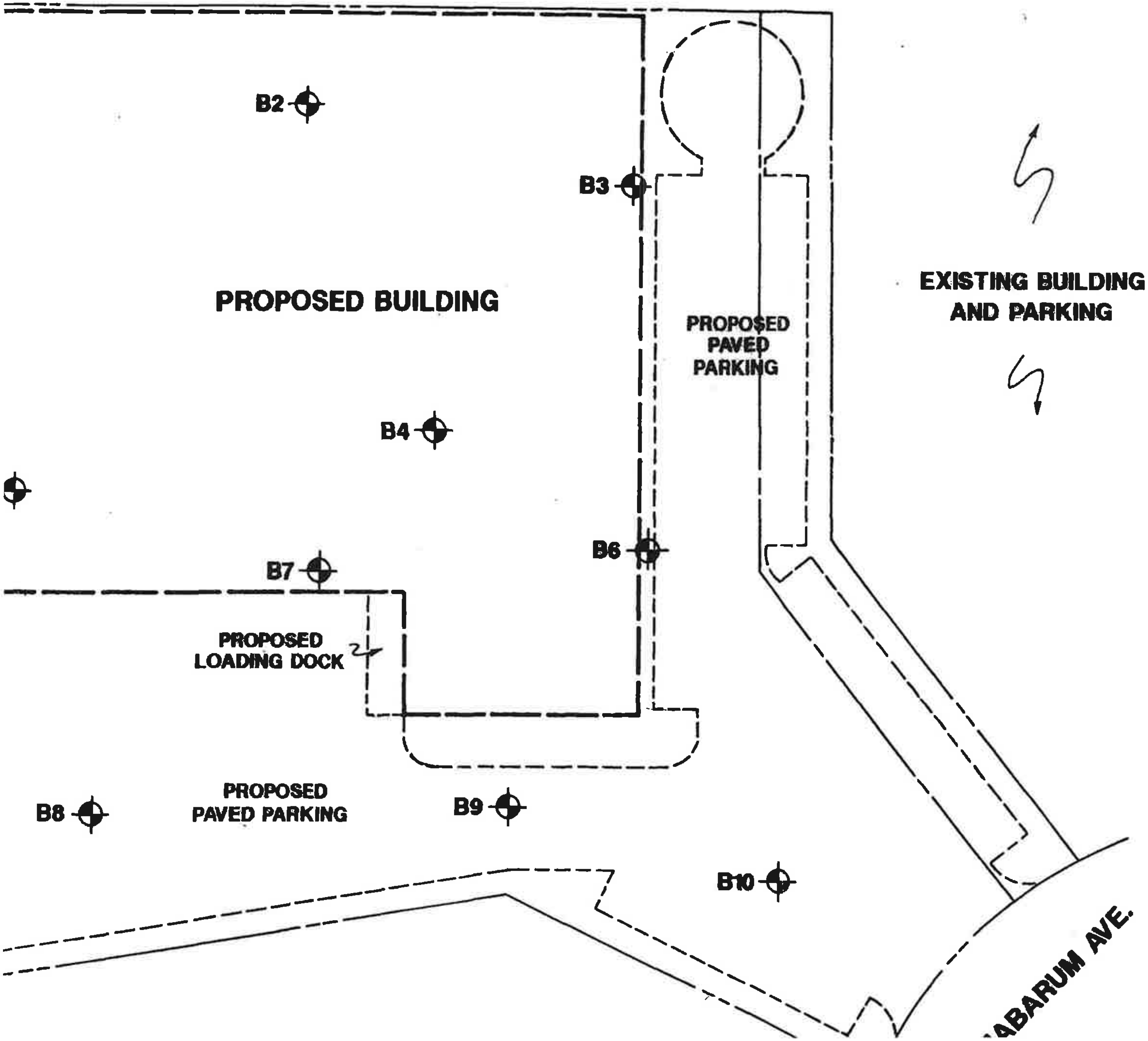
CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 4/3/2015, 2:00:05 PM
Project file: C:\GeobaseIrwindale3-15\CPeT Data\Plot Data\Plots w-ha.cpt

FIGURE B-5

CONSULTANT: JET

MAY 11, 1998

REFERENCE: PLOT PLAN PROV



LEGEN

B10 LOCATION AN

N

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A GEOTECHNICAL CONSULTING FIRM
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818•549•9959 Tel 818•543•3747 Fax

LOG OF BORING 1

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/24/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Silty Sand, light gray-brown, slightly moist, medium dense, cobbles to 6 inches, some asphalt to 1 inch
				1	
2	7	2.3	108.4	2	**ALLUVIUM:** Sand, light gray, moist, dense, poorly graded, fine/medium grained
				3	
				4	
5	6	5.3	107.8	5	
				6	
				7	
				8	
				9	
10	6	2.0	103.4	10	
				11	Gravelly Sand, light gray to gray to light brown, moist, dense, rounded cobbles up to 4 inch rounded, medium to very coarse
				12	
				13	Sandy Gravel, light gray to dark gray to light brown, moist, dense, rounded cobbles up to 8 inches
				14	
				15	
				16	
				17	
				18	slight caving
				19	
				20	*End at 20 Feet; Slight Caving at 18 Feet; No Water; Fill to 2 Feet.*

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LOG OF BORING 2

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/24/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Gravelly Sand, light gray to light brown, moist, medium dense to dense, rounded cobbles to 6 inches
				1	
				2	
				3	**ALLUVIUM:** Sandy Gravel, gray, moist, dense
				4	
				5	
6	5	4.3	103.6	6	Sand, light gray, moist, dense, poorly graded, medium grained
				7	
				8	
				9	gray to light brown, medium to coarse grained
10	5	8.4	107.1	10	
				11	
				11½	Silty sand, dark gray brown, very moist
				12	
				12½	Sand, light gray to light brown, moist, dense, some gravel
				13	Gravelly Sand, light gray to gray to brown, moist, dense, rounded cobbles up to 4 inches
				14	
				14½	Sandy Gravel, gray to gray brown, moist, dense, rounded cobbles up to 8 inches
				15	
				16	*End at 15 Feet; No Caving; No Water; Fill to 3 Feet.*

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LOG OF BORING 3

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/24/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**ALLUVIUM:** Gravelly Sand, brown, dry, cobbles to 4 inches, dense
				1	
				2	
				3	
				4	
5	7	3.7	117.2	5	
				6	
				7	
				8	Sand, brown, moist, dense, cobbles to 4 inches
				9	
				10	
				11	
				12	Sand, gray, moist, cobbles to 4 inches
				13	Sand, brown, moist, gravel to 3 inches
				14	
15	7	5.7	96.0	15	
				16	
				17	
				18	
				19	
20	9	4.0	115.0	20	*End at 20 Feet; No Water; No Caving; No Fill.*

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LOG OF BORING 4

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/24/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Silty Sand, gray and brown, gravel to 3 inches
				1	
2	2	2.2	95.2	2	**ALLUVIUM:** Sand, brown and gray, slightly moist, dense, gravel to 2 inches
				3	
				4	
5	4	3.7	95.0	5	
				6	
				7	
				8	
				9	Gravelly and coarse Sand, brown, moist, gravel to 2 inches
10	3	4.5	121.0	10	
				11	
				12	
				13	
				14	
15	9	4.0	109.6	15	Sand, brown, moist, gravel to 3 inches
				16	
				17	
				18	
				19	
20	5	4.3	101.2	20	gravel to 2 inches
				21	
				22	
				23	
				24	
25	11	3.2	110.2	25	Gravelly Sand, brown, moist, gravel to 3 inches *End at 25 Feet, No Water; No Caving; Fill to 2 Feet.*

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LOG OF BORING 5

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/24/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Silty Sand, dark gray brown, moist, medium dense
				1	
2	4	1.5	105.9	2	**ALLUVIUM:** Sand, light gray to gray, moist, dense, rounded cobbles to 4 inches
				3	
				4	
5	2	2.3	113.9	5	gray to light brown
				6	
				7	
				8	
				9	Clayey Gravel, dark brown, very moist, dense
10	8	21.1	97.7	10	
				11	
				12	
				13	Gravelly Sand, light gray to light brown, moist, dense, rounded cobbles to 4 inches
				14	
				15	
				16	
				17	
				18	
				19	Sandy Gravel, light gray to light brown, dense, rounded cobbles to 8 inches, slight caving
				20	
				21	
				22	
				23	
				24	
				25	*End at 25 Feet; No Caving; No Water; Fill to 2 Feet.*

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LOG OF BORING 6

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/27/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	FILL: Silty Sand, light gray to light brown, slightly moist, medium dense
				½	Sand, light gray, slightly moist, medium dense, poorly graded, medium grained
				1	
				1½	
				2	
				3	**ALLUVIUM:** Gravelly Sand, gray, moist, dense, rounded gravel to 2½ inches
				4	
				4½	Sand, light gray to gray to light brown, moist, medium dense to dense, poorly graded
5	4	6.7	90.3	5	
				6	
				7	
				8	
				9	Gravelly Sand, gray to light gray, moist, medium dense, rounded gravel to 2 inches
10	2	6.6	102.9	10	Sand, light gray to light brown, moist, dense with rounded gravel to 1 inch
				11	
				12	
				13	
				14	
				14½	Sandy Gravel, light brown to gray, moist, dense, rounded rock cobbles to 7 inches
15	10	4.7	95.5	15	
				16	
				17	
				18	
				19	slight to moderate caving
				20	
				21	
				22	*End at 22 Feet; Slight to Moderate Caving at 19 Feet; No Water; Fill to 3 Feet.*

LOG OF BORING 7

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JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/27/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Sand, light gray, dry, slightly loose to medium dense, poorly graded, medium grained
				1	
				1½	**ALLUVIUM:** Sand, light gray to light brown, moist, medium dense
				2	
				3	
4	4	6.1	93.1	4	
				5	
				6	Gravelly Sand, light gray, moist, dense
				7	
8	2	5.0	106.4	8	
				9	
				10	
				11	gravel up to 1½ inches
12	10	5.2	109.4	12	
				13	
				14	
				15	Gravelly Sand, gray to light brown, moist, dense, rounded cobbles to 4 inches, some clay
				16	
				17	
				18	
				19	
				20	Sandy Gravel, gray to brown, moist, dense, cobbles to 8 inches
				22	
				23	
				24	
				25	*End at 25 Feet; No Water; No Caving; Fill to 1½ Feet*

LOG OF BORING 8

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512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/27/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Gravelly Sand, light gray, dry, loose, rounded gravel to 3 inches
				1	
				1½	**ALLUVIUM:** Sandy Gravel, light gray to light brown, slightly moist, medium dense, cobbles to 6 inches
				2	
				3	
				4	
				5	Gravelly Sand, light gray to light brown, moist, dense, gravel to 2 inches
				6	
				7	
				8	
				9	
				10	*End at 10 Feet; No Water; No Caving; Fill to 1½ Feet.*

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A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

LOG OF BORING 9

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/27/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Sand, light gray, dry, slightly loose
				1	
				2	**ALLUVIUM:** Gravelly Sand, slightly moist, light gray to brown, dense
3	3	6.0	104.1	3	moist, rounded gravel to 1 inch
				4	
				5	
6	6	2.6	119.3	6	
				7	
				8	
				9	
				10	*End at 10 Feet; No Water; No Caving; Fill to 1½ Feet.*

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LOG OF BORING 10

JB: 17726-B CLIENT: JACMAR

CONSULTANT: JET DATE LOGGED: 4/27/98

REPORT DATE: 5/11/98

Sample Depth (feet)	Blows Per Foot	Moisture Content %	Dry Density (pcf)	Depth (feet)	LITHOLOGIC DESCRIPTION
				0	**FILL:** Sandy Gravel, light gray, dry, slightly loose
				1	
				2	
				3	
4	1	8.8	87.7	4	**ALLUVIUM:** Sand, light gray to light brown, slightly moist, medium dense
				5	
				6	dense, with cobbles up to 4 inches
				7	
				8	
				9	
10	9	NR	NR	10	*End at 10 Feet; No Water; No Caving; Fill to 4 Feet.*

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REPORT

SURFACE WAVE MEASUREMENTS

12761 SHABARUM AVE.
IRWINDALE, CALIFORNIA

GEO*Vision* Project No. 15087

Prepared for

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Report 15087-01

April 1, 2015

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1 INTRODUCTION

In-situ seismic measurements using active and passive surface wave techniques were performed at 12761 Shabarum Avenue in Irwindale, California on March 12, 2015. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of 30 m (98.4 ft), or more, for UBC/IBC site classification. The active surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive surface wave technique consisted of the array microtremor ("L" array) method.

The average shear wave velocity of the upper 30 m (V_{S30}) is used in the NEHRP provisions and the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 1994). The average shear wave velocity of the upper 100 ft (V_{S100}) is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_S30 > 1500$ m/s (UBC) or $V_S100 > 5{,}000$ ft/s (IBC)
Class B – rock – $760 < V_S30 \leq 1500$ m/s (UBC) or $2{,}500 < V_S100 \leq 5{,}000$ ft/s (IBC)
Class C – very dense soil and soft rock – $360 < V_S30 \leq 760$ m/s (UBC)
or $1{,}200 < V_S100 \leq 2{,}500$ ft/s (IBC)
Class D – stiff soil – $180 < V_S30 \leq 360$ m/s (UBC) or $600 < V_S100 \leq 1{,}200$ ft/s (IBC)
Class E – soft soil – $V_S30 < 180$ m/s (UBC) or $V_S100 < 600$ ft/s (IBC)
Class F – soils requiring site-specific evaluation

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain a 30 m (100 ft) S-wave velocity sounding. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to 30 m and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the refraction microtremor method of Louie (2001) or the array microtremor technique can be used to extend the depth of investigation at sites that have adequate noise levels.

This report contains the results of the active and passive surface wave measurements conducted along three arrays at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Interpretation and results are presented in Section 5. Section 6 presents our conclusions. References and our professional certification are presented in Sections 7 and 8, respectively.

2 OVERVIEW OF THE SURFACE WAVE METHODS

A discussion of active and passive surface wave methods is provided in the technical note included as Appendix A. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the refraction and array microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh waves when propagating in a layered medium. The phase velocity, V_R, depends primarily on the material properties (V_S, mass density and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. Waves of different wavelengths, λ, (or frequencies, f) sample different depths. As a result of the variance in the shear stiffness of the layers, waves with different wavelengths travel at different phase velocities; hence, dispersion. A surface wave dispersion curve, or dispersion curve for short, is the variation of V_R with λ or f.

The SASW and MASW methods are in-situ seismic methods for determining shear wave velocity (V_S) profiles [Stokoe et al., 1994; Stokoe et al., 1989; Park et al., 1999a and 1999b, Foti, 2000]. Surface wave techniques are non-invasive and non-destructive, with all testing performed on the ground surface at strain levels in the soil in the elastic range (< 0.001%). SASW testing consists of collecting surface wave phase data in the field, generating the dispersion curve and then using iterative forward or inverse modeling to calculate the shear stiffness profile. MASW testing consists of collecting multi-channel seismic data in the field and applying a wavefield transform to obtain the dispersion curve and data modeling.

A detailed description of the SASW field procedure is given in Joh [1996]. A vertical dynamic load is used to generate horizontally-propagating Rayleigh waves. The ground motions are monitored by two, or more, vertical receivers and recorded by the data acquisition system capable of performing both time and frequency-domain calculations. Theoretical, as well as, practical considerations, such as attenuation, necessitate the use of several receiver spacings to generate the dispersion curve over the wavelength range required to evaluate the stiffness profile. To minimize phase shifts due to differences in receiver coupling and subsurface variability, the source location is reversed.

After the time-domain motions from the two receivers are converted to frequency-domain records using the Fast Fourier Transform, the cross power spectrum and coherence are calculated. The phase of the cross power spectrum, ϕ_w (f), represents the phase differences between the two receivers as the wave train propagates past them. It ranges from $-\pi$ to π in a wrapped form and must be unwrapped through an interactive process called masking. Phase jumps are specified, near-field data (wavelengths longer than three times the distance from the source to first receiver) and low-coherence data are removed. The experimental dispersion curve is calculated from the unwrapped phase angle and the distance between receivers by:

$$V_R = f * d_2/(\Delta\phi/360°),$$

Where V_R is Rayleigh wave phase velocity, f is frequency, d_2 is the distance between receivers and $\Delta\phi$ is the phase difference in degrees.

WinSASW V1, a program developed at the University of Texas at Austin, or WinSASW V2 (Joh, 2002) is used to reduce SASW data and interpret the dispersion curve.

A detailed description of the MASW method is given by Park, 1999a and 1999b. Ground motions are recorded by 24 or more geophones spaced 1 to 2 m apart and aligned in a linear array and connected to a seismograph. A wavefield transform, such as the f-k or τ-p transform, is applied to the time history data to isolate the surface wave dispersion curve. PICKWIN95, software developed by Oyo Corporation is typically used to process the MASW data and obtain the dispersion curve.

The refraction microtremor technique is a passive surface wave technique developed by Dr. John Louie at University of Nevada, Reno. A detailed description of this technique can be found in Louie, 2001. The refraction microtremor method differs from the more established array microtremor technique in that it uses a linear receiver array rather than a triangular or circular array. Unlike the SASW method, which uses an active energy source (i.e. hammer), the microtremor technique records background noise emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. Refraction microtremor field procedures consist of laying out a linear array of 24 or more, 4.5 to 8 Hz geophones and recording 10, or more, 15 to 60 second noise records. These noise records are reduced using the software package SeisOpt® ReMi™ v2.0 by Optim™ Software and Data Services. This package is used to generate and combine the slowness (p) – frequency (f) transform of the noise records. The surface wave dispersion curve is picked at the lower envelope of the surface wave energy identified in the p-f spectrum.

A detailed discussion of the array microtremor method can be found in Okada, 2003. This technique uses 4 to 48 receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle and "L" shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. Receivers typically consist of 1 to 4.5 Hz geophones. The triangle array, which consists of several embedded equilateral triangles, is often used as it provides good results with a relatively small number of geophones. With this array the outer side of the triangle should be at least equal to the desired depth of investigation. The "L" array is useful at sites located at the corner of perpendicular intersecting streets. Typically 10 to 20, 30-second noise records are acquired for analysis. The surface wave dispersion curve is estimated by calculating the spatial autocorrelation (SPAC) function for the time-history data. A first-order Bessel function is fit to the SPAC function to obtain the dispersion curve (phase velocity at each frequency). PICKWIN95, software developed by Oyo Corporation is typically used to process the array microtremor data and obtain the dispersion curve.

The active and passive surface wave techniques complement one another as outlined below:

- SASW/MASW techniques image the shallow velocity structure which cannot be imaged by the microtremor technique and is needed for an accurate V_S30/V_S100' estimate.
- Microtremor techniques work best in noisy environments where SASW/MASW depth investigation may be limited.
- In a noisy environment, the microtremor technique will usually extend the depth of an SASW/MASW sounding.

- The degree of fit in the overlapping portion of the dispersion curves from the two techniques provides a level of confidence in the results.

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled. Typically, WinSASW V1 or V2 is used to model the data, whereby through iterative forward and/or inverse modeling, a V_S profile is found whose theoretical dispersion curve is a close fit to the field data.

The final model profile is assumed to represent actual site conditions. Several options exist for forward modeling: a formulation that takes into account only fundamental-mode Rayleigh wave motion (called the 2-D solution) and one that includes all stress waves and incorporates receiver geometry (3-D solution) [Roesset et al., 1991].

The theoretical model used to interpret the dispersion assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good "global" estimate of the material properties along the array. The results may be more representative of the site than a borehole "point" estimate.

Based on our experience at other sites, the shear wave velocity models determined by surface wave testing are within 20% of the velocities that would be determined by other seismic methods [Brown, 1998]. The average velocity of the upper 30 m or 100 ft, however, is much more accurate than this, often to better than 5%, because it is less sensitive to the layering in the model.

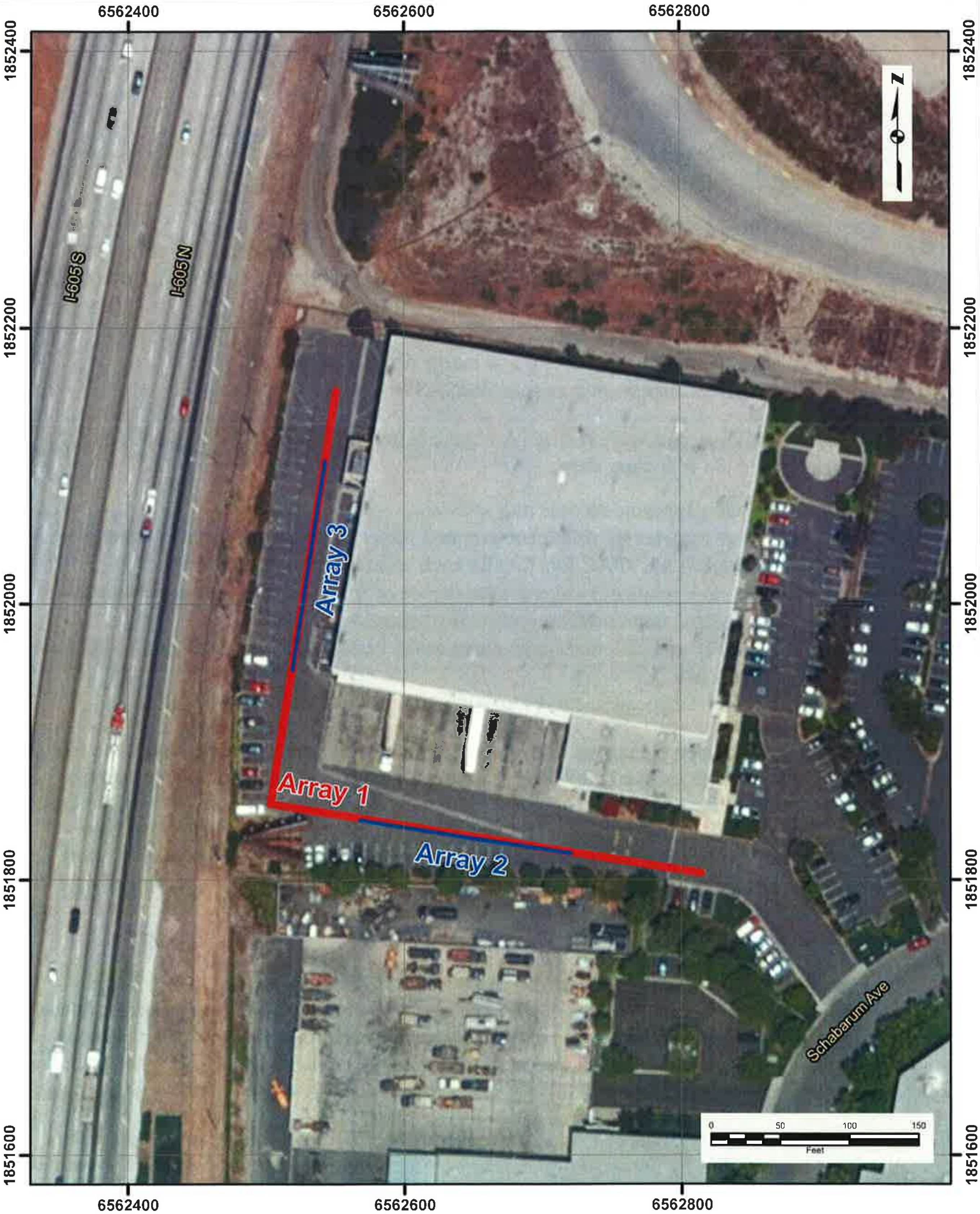
3 FIELD PROCEDURES

The surface wave soundings were established where possible and are shown in Figure 1. Active surface wave data were acquired using the MASW technique. Passive surface wave data were acquired using the array microtremor method.

A typical MASW field layout is shown in Appendix A. MASW equipment used during this investigation consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, seismic cable with 10-foot takeouts, a 3 lb hammer, a 10 lb sledgehammer, an accelerated weight drop and an aluminum plate. MASW data were acquired along a linear array of 48 geophones spaced 1 m (3.3 ft) apart (Arrays 2 and 3). Shot points were located 1, 5, 10 and 20 m (3.3, 16.4, 32.8 and 65.6 ft) from the end geophone locations and multiple shot points were located in the interior of the array. The 3 lb hammer and 10 lb sledgehammer were used for the 1 m offset source locations and the center shot. The accelerated weight drop was used for the 1, 5, 10 and 20 m offset source locations. Data from the transient impacts (hammers) were averaged 10 times to improve the signal-to-noise ratio. Photographs of typical MASW equipment are presented in Appendix A. All field data were saved to hard disk and documented on field data acquisition forms.

Array microtremor measurements were made along an “L” shaped array of 48, 4.5 Hz geophones with 4 m (13.1 ft) geophone spacing along the array (Array 1). A typical field layout is shown in Appendix A. The passive surface wave array consisted of two Geometrics Geode signal enhancement seismographs that were used to record forty, 30 sec noise records using a 2 ms sample rate. Data were stored on a laptop computer for later processing and field geometry and associated files names were documented in field data acquisition forms.

Linear array microtremor measurements were made concurrently with the “L” array using the same geophones and spacing as stated above. Each leg of the “L” array was separated to process an east-west and south-north set of array microtremor measurements for an effective result of two linear soundings.



Legend

- Active Surface Wave Array (MASW)
- Passive Surface Wave Array (L-Array)

NOTES:
1. California State Plane Coordinate System, NAD 83 Zone V (0405), US Survey Feet
2. Image Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

GEOVision geophysical services

Date:	3/19/2015
GV Project:	15087
Developed by:	D Carpenter
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	15087-1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
12761 SHABARUM AVENUE
IRWINDALE, CALIFORNIA

PREPARED FOR
GEOBASE, INC.

4 DATA REDUCTION AND MODELING

The MASW data were reduced using the software Seismic Pro Surface V6.0 developed by Geogiga using the following steps:

- Input seismic record into software.
- Enter receiver spacing, geometry and wavelength restrictions, as necessary.
- Apply wavefield transform to seismic record to convert the data to phase velocity – frequency space.
- Identify and pick dispersion curve.
- Repeat for all shot records and merge dispersion curves.
- Convert dispersion curves to WinSASW format for modeling.

The array microtremor data were reduced using the software PICKWIN95 developed by Oyo Corporation using the following steps:

- Input all seismic records into software.
- Enter receiver spacing, geometry and wavelength restrictions, as necessary.
- Calculate the SPAC function for each seismic record and average.
- For each frequency calculate the degree of fit of a first-order Bessel function to the SPAC function for a multitude of phase velocities.
- Identify and pick dispersion curve as the best fit of the Bessel function for each frequency.
- Convert dispersion curves to WinSASW format for modeling.

The surface wave dispersion curves from the active and passive surface wave data were used for modeling. An iterative forward modeling process was used to generate an S-wave velocity model for the sounding. During this process an initial velocity model was generated based on general characteristics of the dispersion curve. The theoretical dispersion curve was then generated using the 2-D modeling algorithm (fundamental mode Rayleigh wave dispersion module) and compared to the field dispersion curve. Adjustments were then made to the thickness and velocities of each layer and the process repeated until an acceptable fit to the field data is obtained.

Data inputs into the modeling software included layer thickness, S-wave velocity, P-wave velocity and mass density. P-wave velocity and mass density only have a very small influence (i.e. less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is impacted by the location of the bedrock, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.8 to 2.0 g/cc were used in the profile for subsurface soils. Variation in mass density has a negligible effect on surface wave dispersion within the normal range encountered in geotechnical engineering. During data modeling, the compression wave velocity, V_P, of unsaturated soils was estimated using a Poisson's ratio, ν, of 0.30 and the relationship:

$$V_P = V_S\,[(2(1-\nu))/(1-2\nu)]^{0.5}$$

Depth to groundwater at this site is unknown. However, groundwater is expected to be encountered between 15 m (50 ft) and 30 m (98.4 ft) below ground surface. The saturated zone was fixed at a nominal depth of 17 m (55.8 ft) and assigned a P-wave velocity of 1,500 m/s (4921 ft/s) to 1,550 m/s (5,085 ft/s). The presence of water at a depth of greater than 17 m (55.8 ft) would have a negligible effect on the value of V_{S30}.

5 INTERPRETATION AND RESULTS

The fit of the theoretical dispersion curve to the experimental data collected at the site and the modeled V_S profiles for Array 1 and Array 2 are presented in Figure 2. The fit of the theoretical dispersion curve to the experimental data collected at the site and the modeled V_S profiles for Array 1 and Array 3 are presented in Figure 3. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The V_S profiles used to match the field data is provided in tabular form as Table 1 and Table 2.

Table 1 Velocity Model for Array 1 and Array 2

Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m/s	ft/s	m/s	ft/s
0	0.0	3	9.8	200	656	374	1,228
3	9.8	4	13.1	250	820	468	1,534
7	23.0	7	23.0	360	1,181	674	2,210
14	45.9	3	9.8	400	1,312	748	2,455
17	55.8	7	23.0	400	1,312	1,500	4,921
24	78.7	> 16	> 52	465	1,526	1,550	5,085

Approximate depth of investigation is 40 m.

Table 2 Velocity Model for Array 1 and Array 3

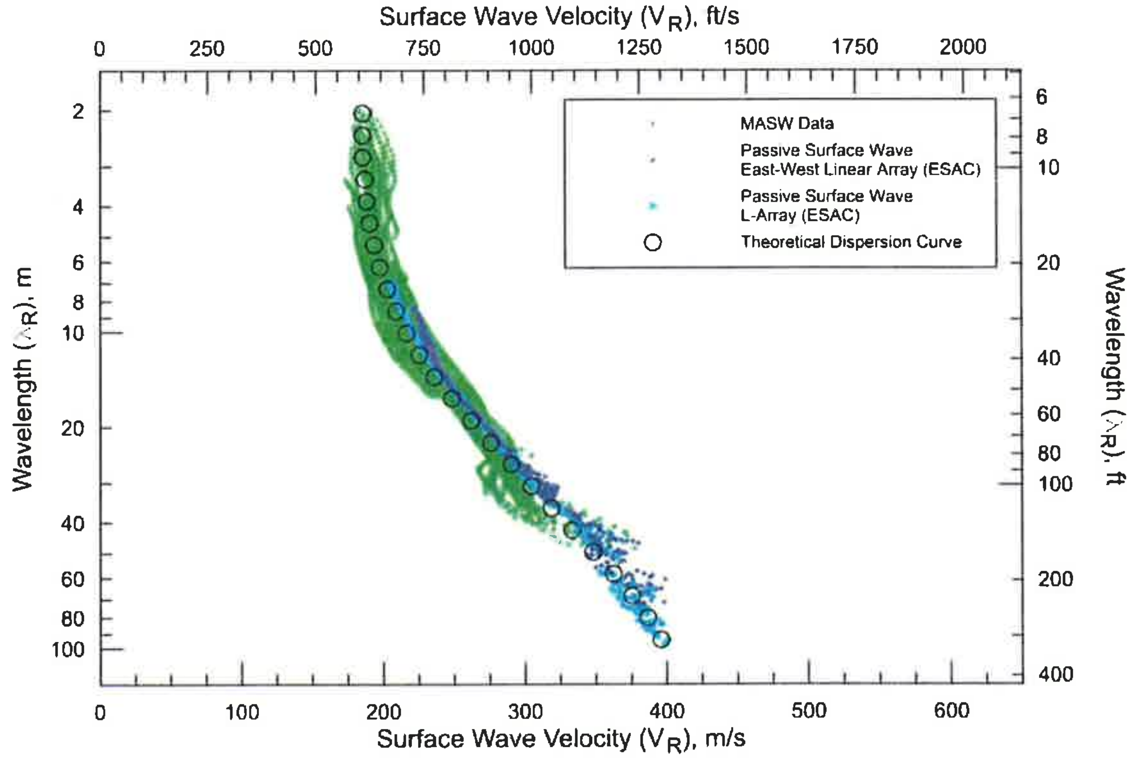
Depth to Top of Layer		Layer Thickness		S-Wave Velocity		Inferred P-Wave Velocity	
m	ft	m	ft	m/s	ft/s	m/s	ft/s
0	0.0	3	9.8	200	656	374	1,228
3	9.8	4	13.1	260	853	486	1,596
7	23.0	7	23.0	375	1,230	702	2,302
14	45.9	3	9.8	390	1,280	730	2,394
17	55.8	7	23.0	390	1,280	1,500	4,921
24	78.7	> 16	> 52	465	1,526	1,550	5,085

Approximate depth of investigation is 40 m.

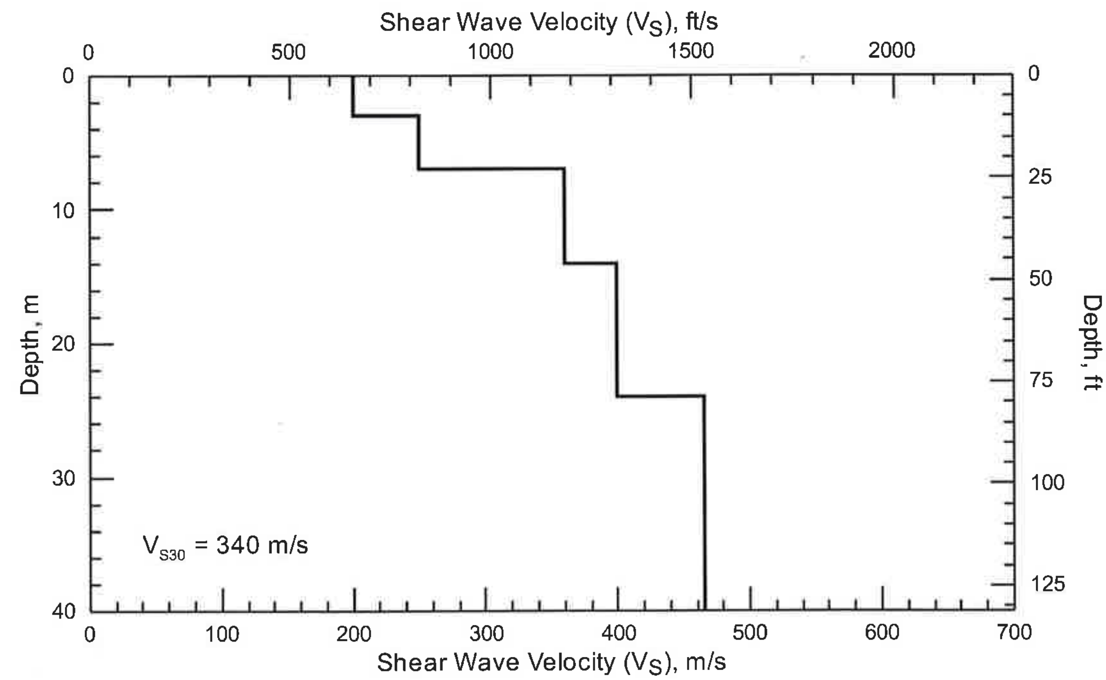
The surface wave phase velocities from the array microtremor measurements ("L" shaped array and linear arrays) are in good agreement with those from the MASW data in the region of overlapping wavelengths. Minor differences in the surface wave dispersion curves between the different techniques likely result from lateral velocity variation. The estimated depth of investigation for the combined active and passive surface wave sounding is about 40 m (131.2 ft).

The shear wave velocity profile for Array 1 and Array 2 consists of about 3 m (9.8 ft) of soft sediment or fill material with an S-wave velocity of about 200 m/s (656 ft/s). Below a depth of 3 m (9.8 ft), S-wave velocity increases from about 250 m/s (820 ft/s) to 360 m/s (1,181 ft/s) at a depth of approximately 7 m (23 ft). Below a depth of 7 m (23 ft), S-wave velocity gradually increases from about 360 m/s (1,181 ft/s) to 400 m/s (1,312 ft/s) at a depth of approximately 14 m (45.9 ft). At a depth of approximately 24 m (78.7 ft), S-wave velocity increases to about 465 m/s (1,526 ft/s).

The shear wave velocity profile for Array 1 and Array 3 consists of about 3 m (9.8 ft) of soft sediment or fill material with an S-wave velocity of about 200 m/s (656 ft/s). Below a depth of 3 m (9.8 ft), S-wave velocity increases from about 260 m/s (853 ft/s) to 375 m/s (1,230 ft/s) at a depth of approximately 7 m (23 ft). Below a depth of 7 m (23 ft), S-wave velocity increases slightly from about 375 m/s (1,181 ft/s) to 390 m/s (1,280 ft/s) at a depth of approximately 14 m (45.9 ft). At a depth of approximately 24 m (78.7 ft), S-wave velocity increases to about 465 m/s (1,526 ft/s).

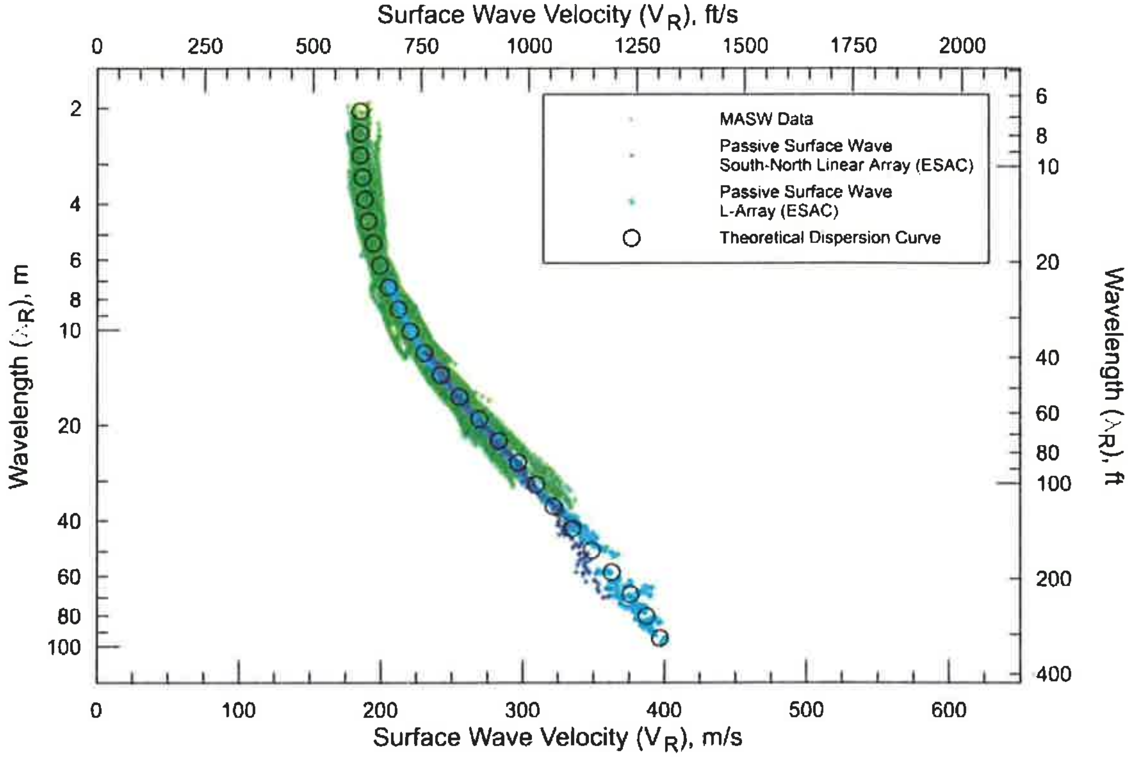


Comparison of Field Experimental Data and Theoretical Dispersion Curve from Active and Passive Surface Wave Arrays

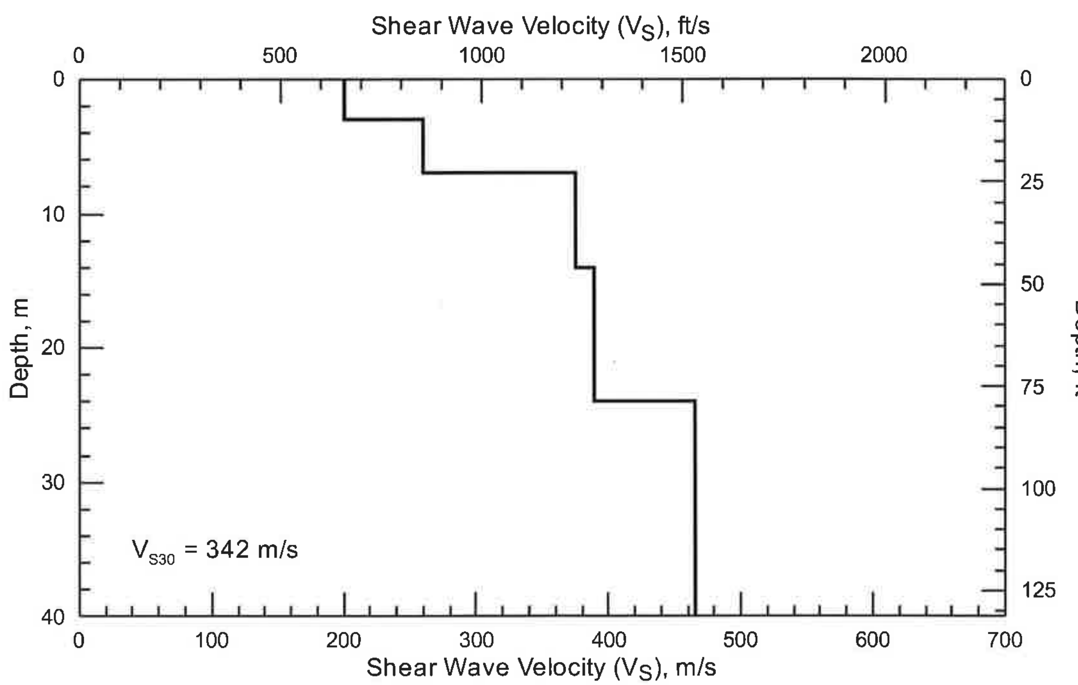


V_S Model from Active and Passive Surface Wave Arrays

GEOVision geophysical services	FIGURE 2 VELOCITY MODEL FOR ACTIVE (ARRAY 2) AND PASSIVE (ARRAY 1) SURFACE WAVE ARRAYS
Project No.: 15087 Date: MAR. 19, 2015	12761 SHABARUM AVENUE IRWINDALE, CALIFORNIA
Drawn By: D. CARPENTER Approved By: A. MARTIN	PREPARED FOR GEOBASE, INC.



Comparison of Field Experimental Data and Theoretical Dispersion Curve from Active and Passive Surface Wave Arrays



V_S Model from Active and Passive Surface Wave Arrays

GEOVision geophysical services	FIGURE 3 VELOCITY MODEL FOR ACTIVE (ARRAY 3) AND PASSIVE (ARRAY 1) SURFACE WAVE ARRAYS
Project No.: 15087 Date: MAR. 19, 2015	12761 SHABARUM AVENUE IRWINDALE, CALIFORNIA
Drawn By: D. CARPENTER Approved By: A. MARTIN	PREPARED FOR GEOBASE, INC.

6 CONCLUSIONS

Active and passive surface wave measurements using the MASW and array microtremor ("L" array) techniques were made at 12761 Shabarum Avenue in Irwindale, California to characterize shear-wave velocity of the upper 30 m or more (98.4 ft). The locations of the active and passive surface wave arrays are presented in Figure 1. The shear wave velocity depth profiles determined by these methods are presented as Figures 2 and 3 and in Tables 1 and 2.

V_{S30} is approximately 340 m/s (1,114 ft/s) beneath the surface wave arrays (Arrays 1 and 2). V_{S30} is approximately 342 m/s (1,124 ft/s) beneath the surface wave arrays (Arrays 1 and 3). Therefore, according to the Uniform and International Building Codes, the area in the vicinity of the surface wave arrays is classified as Class D, stiff soil.

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Prepared by

4/1/2015

David Carpenter
Senior Staff Geophysicist
GEO*Vision* Geophysical Services

Date

Reviewed and approved by

4/1/2015

Antony Martin
California Professional Geophysicist, P. GP 989
GEO*Vision* Geophysical Services

Date

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

APPENDIX A

ACTIVE AND PASSIVE SURFACE WAVE TECHNIQUES

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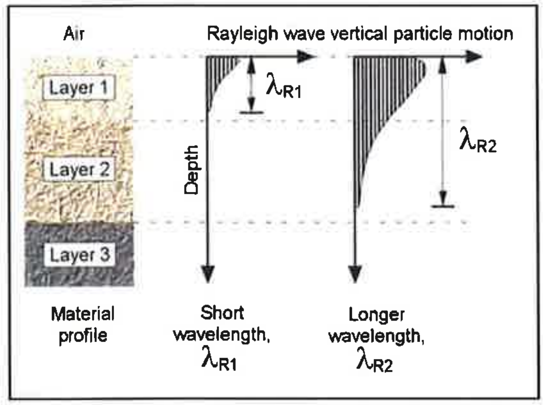
Overview

Active and passive surface wave techniques are relatively new in-situ seismic methods for determining shear wave velocity (V_S) profiles. Testing is performed on the ground surface, allowing for less costly measurements than with traditional borehole methods. The basis of surface wave techniques is the dispersive characteristic of Rayleigh waves when traveling through a layered medium. Rayleigh wave velocity is determined by the material properties (primarily shear wave velocity, but also to a lesser degree compression wave velocity and material density) of the subsurface to a depth of approximately 1 to 2 wavelengths. As shown in the adjacent diagram, longer wavelengths penetrate deeper and their velocity is affected by the material properties at greater depth. Surface wave testing consists of measuring the surface wave dispersion curve at a site and modeling it to obtain the corresponding shear wave velocity profile.



Active Surface Wave Techniques

Active surface wave techniques measure surface waves generated by dynamic sources such as hammers, weight drops, electromechanical shakers, vibroseis and bulldozers. These techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods.

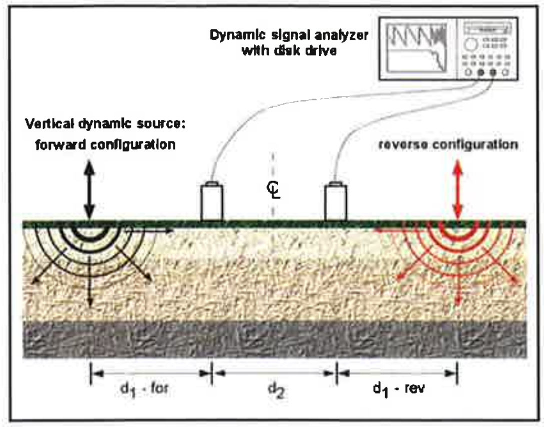
Hammer Energy Sources

Accelerated Weight Drop

Electromechanical Shaker

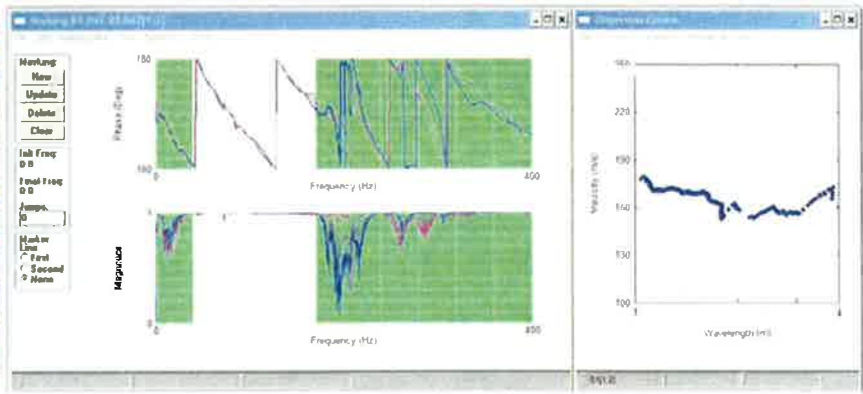
Bulldozer Energy Source

The SASW method is optimized for conducting V_S depth soundings. A dynamic source is used to generate surface waves of different wavelengths (or frequencies) which are monitored by two or more receivers at known offsets. An expanding receiver spread and optimized source-receiver geometry are used to minimize near field effects, body wave signal and attenuation. A dynamic signal analyzer is typically used to calculate the phase and coherence of the cross spectrum of the time history data collected at a pair of receivers. During data analysis, an interactive masking process is used to discard low quality data and to unwrap the phase spectrum, as shown in the figure below. The dispersion curve (Rayleigh wave phase velocity versus frequency or alternatively wavelength) is calculated from the unwrapped phase spectrum.



SASW Setup

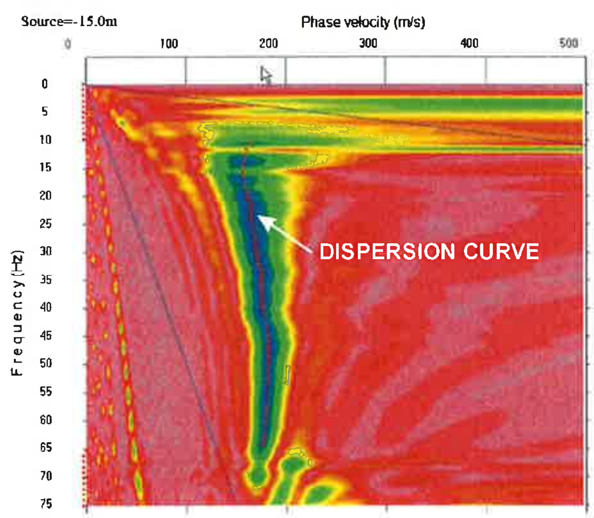
HP Dynamic Signal Analyzer



Masking of Wrapped Phase Spectrum and Resulting Dispersion Curve

The MASW field layout is similar to that of the seismic refraction technique. Twenty four, or more, geophones are laid out in a linear array with 1 to 2m spacing and connected to a multi-channel seismograph as shown below. This technique is ideally suited to 2D V_S imaging, with data collected in a roll-along manner similar to that of the seismic reflection technique. The source is offset at a predetermined distance from the near geophone usually determined by field testing. The Rayleigh wave dispersion curve is obtained by a wavefield transformation of the seismic record such as the f-k or τ-p transforms. These transforms are very effective at isolating surface wave energy from that of body waves. The dispersion curve is picked as the peak of the surface wave energy in slowness (or velocity) – frequency space as shown. One advantage of the MASW technique is that the wavefield transformation may not only identify the fundamental mode but also higher modes of surface waves. At some sites, particularly those with large velocity inversions, higher surface wave modes may contain more energy than the fundamental mode.

MASW Field Setup

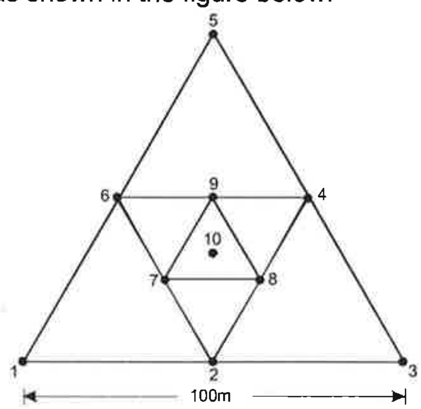


Wavefield Transform of MASW data

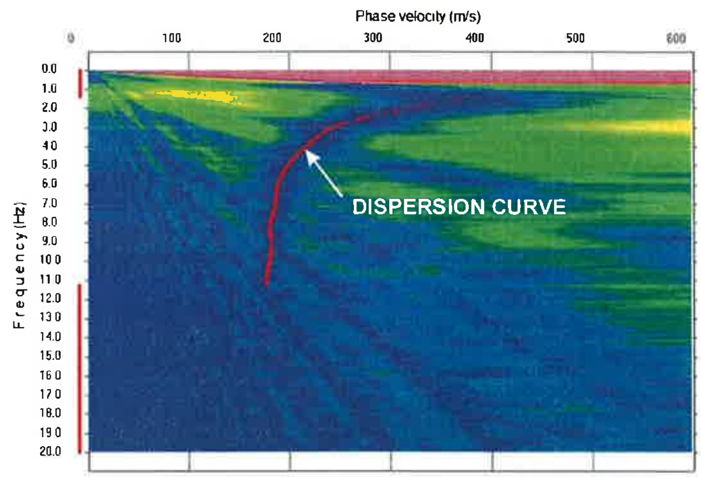
Passive Surface Wave Techniques

Passive surface wave techniques measure noise; surface waves from ocean wave activity, traffic, factories, etc. These techniques include the array microtremor and refraction microtremor (REMI) techniques.

The array microtremor technique typically uses 7 or more 4.5- or 1-Hz geophones arranged in a two-dimensional array. The most common arrays are the triangle, circle, semi-circle and "L" arrays. The triangle array, which consists of several embedded equilateral triangles, is often used as it provides good results with a relatively small number of geophones. With this array the outer side of the triangle should be at least as long as the desired depth of investigation. Typically, fifteen to twenty 30-second noise records are acquired for analysis. The spatial autocorrelation (SPAC) technique is one of several methods that can be used to estimate the Rayleigh wave dispersion curve. A first order Bessel function is fit to the SPAC function to determine the phase velocity for particular frequency. The image shown below shows the degree of fitness of the Bessel function to the SPAC function for a wide range of phase velocity and frequency. The dispersion curve, is the peak (best fit), as shown in the figure below.



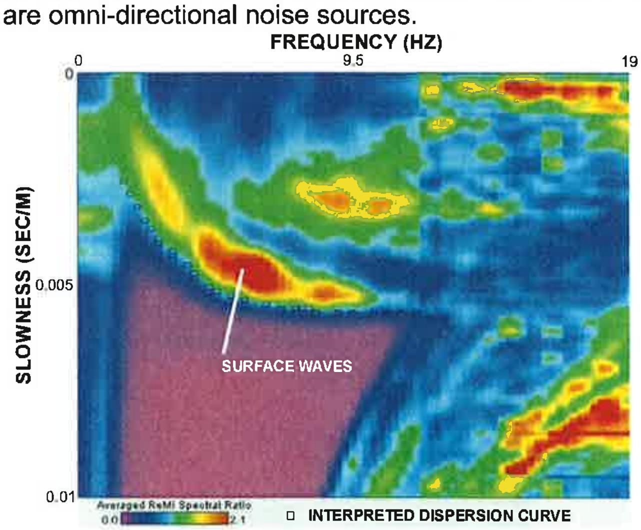
Triangle Array Geometry



Dispersion Curve from Array Microtremor Measurements

The refraction microtremor (REMI) technique uses a field layout similar to the seismic refraction method (hence its name). Twenty-four, 4.5 Hz geophones are laid out in a linear array with a spacing of 6 to 8m and fifteen to twenty 30-second noise records are acquired. A slowness-frequency (p-f) wavefield transform is used to separate Rayleigh wave energy from that of other waves. Because the noise field can originate from any direction, the wavefield transform is conducted for multiple vectors through the geophone array, all of which are summed. The dispersion curve is defined as the lower envelope of the Rayleigh wave energy in p-f space. Because the lower envelope is picked rather than the energy peak (energy traveling along the profile is slower than that approaching from an angle), this technique may be somewhat more subjective than the others, particularly at low frequencies. The SPAC technique can also be used to extract the surface wave dispersion curve from linear array microtremor data providing there are omni-directional noise sources.

Refraction Microtremor Array Layout



Wavefield Transform of REMI Data

Depth of Investigation
Active surface wave investigations typically use various sized sledge hammers to image the shear wave velocity structure to depths of up to 15m. Weight drops and electromechanical shakers can often be used to image to depths of 30m. Bulldozers and vibroseis trucks can be used to image to depths as great as 100m. Passive surface wave techniques can often image shear wave velocity structure to depths of over 100m, given sufficient noise sources and space for the receiver array. Large passive arrays, utilizing long-period seismometers with GPS clocks have been used to image shear wave velocity structure to depths of several kilometers.

Combined Active and Passive Surface Wave Testing
The combined use of active and passive techniques may offer significant advantages on many investigations. It can be very costly to mobilize large energy sources for 30m/100ft active surface wave soundings. In urban environments, the combined use of active and passive surface wave techniques can image to these depths without the need for large energy sources. We have found that dispersion curves from active and passive surface wave techniques are generally in good agreement, making the combined use of the two techniques viable. It is not recommended that passive surface wave techniques be applied alone for UBC/IBC site classification investigations. Microtremor techniques do not generally characterize near surface velocity, which may have a significant impact of the average shear wave velocity of the upper 30m or 100ft and so should always be used in conjunction with SASW or MASW. An SASW sounding to a depth of 30m requires at least a 60m linear array. If sufficient space is not available for this, it may be possible to use a 45m triangle array on the site or place a 100-200m long REMI array along an adjacent sidewalk or an "L" array at an adjacent street intersection.

Microtremor Measurements along Sidewalk

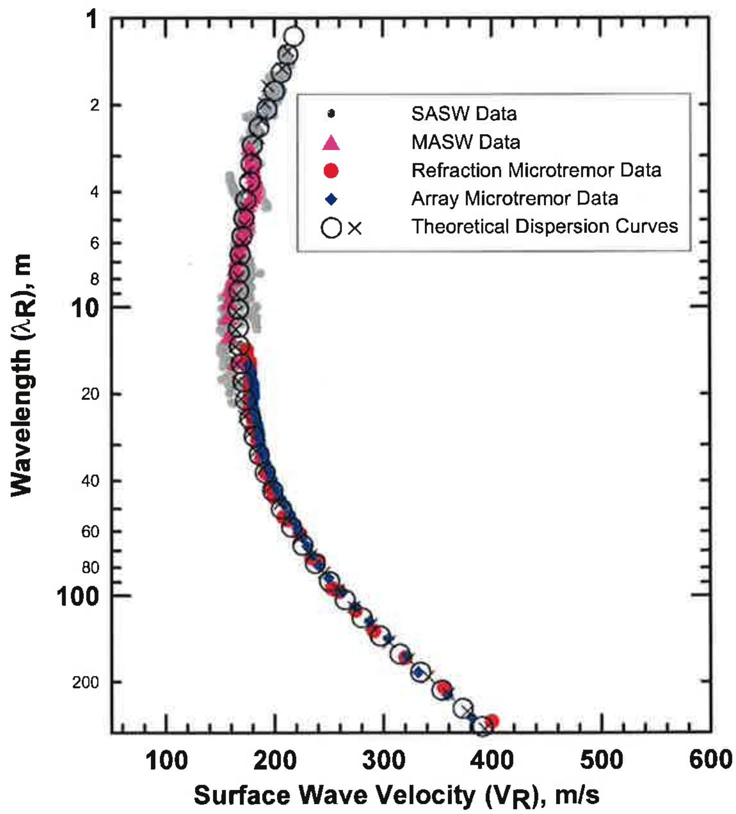
Modeling
There are several options for interpreting surface wave dispersion curves, depending on the accuracy required in the shear wave velocity profile. A simple empirical analysis can be done to estimate the average shear wave velocity profile. For greater accuracy, forward modeling of fundamental-mode Rayleigh wave dispersion as well as full stress wave propagation can be performed using several software packages. A formal inversion scheme may also be used. With many of the analytical approaches, background information on the site can be incorporated into the model and the resolution of the final profile may be quantified.

Applications
Active and passive surface wave testing can be used to obtain V_S profiles for:

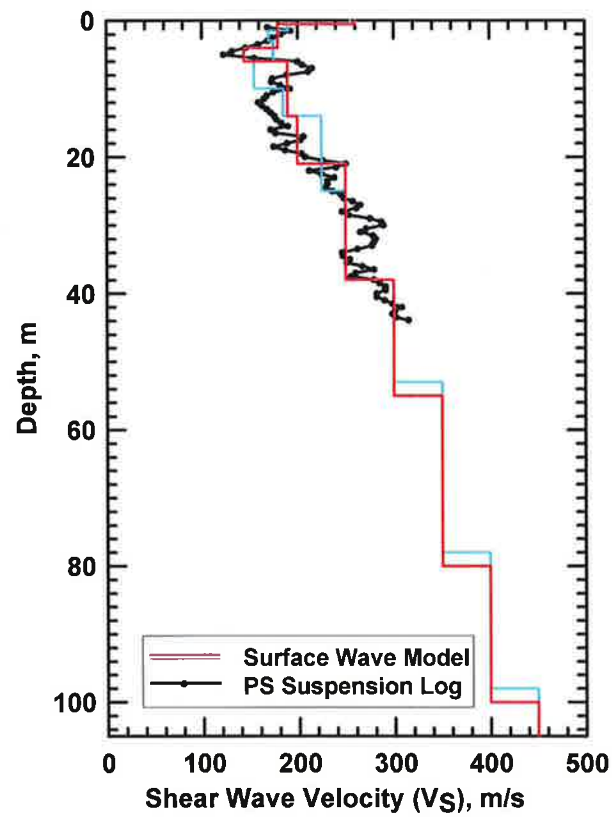
- UBC/IBC site classification for seismic design
- Earthquake site response
- Seismic microzonation
- Liquefaction analysis
- Soil compaction control
- Mapping subsurface stratigraphy
- Locating potentially weak zones in earthen embankments and levees

Case History

The figures below show the surface wave dispersion curves and alternative shear wave velocity models for a site in Los Angeles, California. All of the previous figures illustrating SASW, MASW, array and refraction microtremor techniques were from this site. The dispersion curves from all four methods are shown on the left along with the theoretical dispersion curves for alternative S-wave velocity versus depth models on the right. Conditions at this site were very poor for active surface wave techniques because of the presence of very low velocity hydraulic fill. In fact, with active surface wave techniques it was only possible to image to a depth of about 12.5m with energy sources typically capable of imaging to 30m. There is excellent agreement in the dispersion curves generated from all of the methods over the overlapping wavelength ranges. The minor differences probably result from variable velocity of the hydraulic fill within the sampling volume of the specific methods. Two Vs versus depth models were generated to illustrate the difficulty modeling the highly variable, near surface velocity structure evident in the PS log. The two surface wave models yielded similar values for the average shear-wave velocity of the upper 30m (V_S30), 201 and 202 m/s, illustrating that Vs30 is much more tightly constrained than the actual layer thicknesses and velocities in the models. V_S30 estimated from the PS log (194 m/s) is within 4% of that estimated from the two surface wave models (201 and 202 m/s). The small differences in V_S30 between the two methods may easily result from the different sampling regimes (borehole versus large area) rather than errors in either of the methods.



Field Data and Theoretical Dispersion Curve



V_S Model

In contrast to borehole measurements which are point estimates, surface wave testing is a global measurement, that is, a much larger volume of the subsurface is sampled. The resulting profile is representative of the subsurface properties averaged over distances of up to several hundred feet. Although surface wave techniques do not have the layer sensitivity or accuracy (velocity and layer thickness) of borehole techniques; the average velocity over a large depth interval (i.e. the average shear wave velocity of the upper 30m or 100ft) is very well constrained. Because surface wave methods are non-invasive and non-destructive, it is relatively easy to obtain the necessary permits for testing. At sites that are favorable for surface wave propagation, active and passive surface wave techniques allow appreciable cost and time savings.

APPENDIX C

The J. Byer Group, May 1998

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the fill and alluvium were obtained from the borings and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring lined barrel sampler conforming to ASTM D-3550 with successive drops of the Kelly bar. Experience has shown that sampling causes some disturbance of the sample, however the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D-2937. The moisture content of the samples was determined using the procedures outlined in ASTM D-2216. The results are shown on the Log of Borings.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill was determined by remolding bulk samples of the alluvium using the procedures outlined in ASTM D 1557, a five-layer standard. Remolded samples were prepared at 90 percent of the maximum density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Soil Type	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
7	2	Sand	112.0	17.5	Nil

Expansion Test

To find the expansiveness of the soil, a swell test was performed using the procedures outlined in ASTM D-4829. Based upon the testing, the earth materials are not expansive.

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Shear-Tests

Shear tests were performed on the soil samples of future compacted fill and alluvium using the procedures outlined in ASTM D-3080 and a strain controlled, direct shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inches per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the "Shear Test Diagrams".

Consolidation

Consolidation tests were performed on insitu samples of the alluvium. Results are graphed on the "Consolidation Curves".

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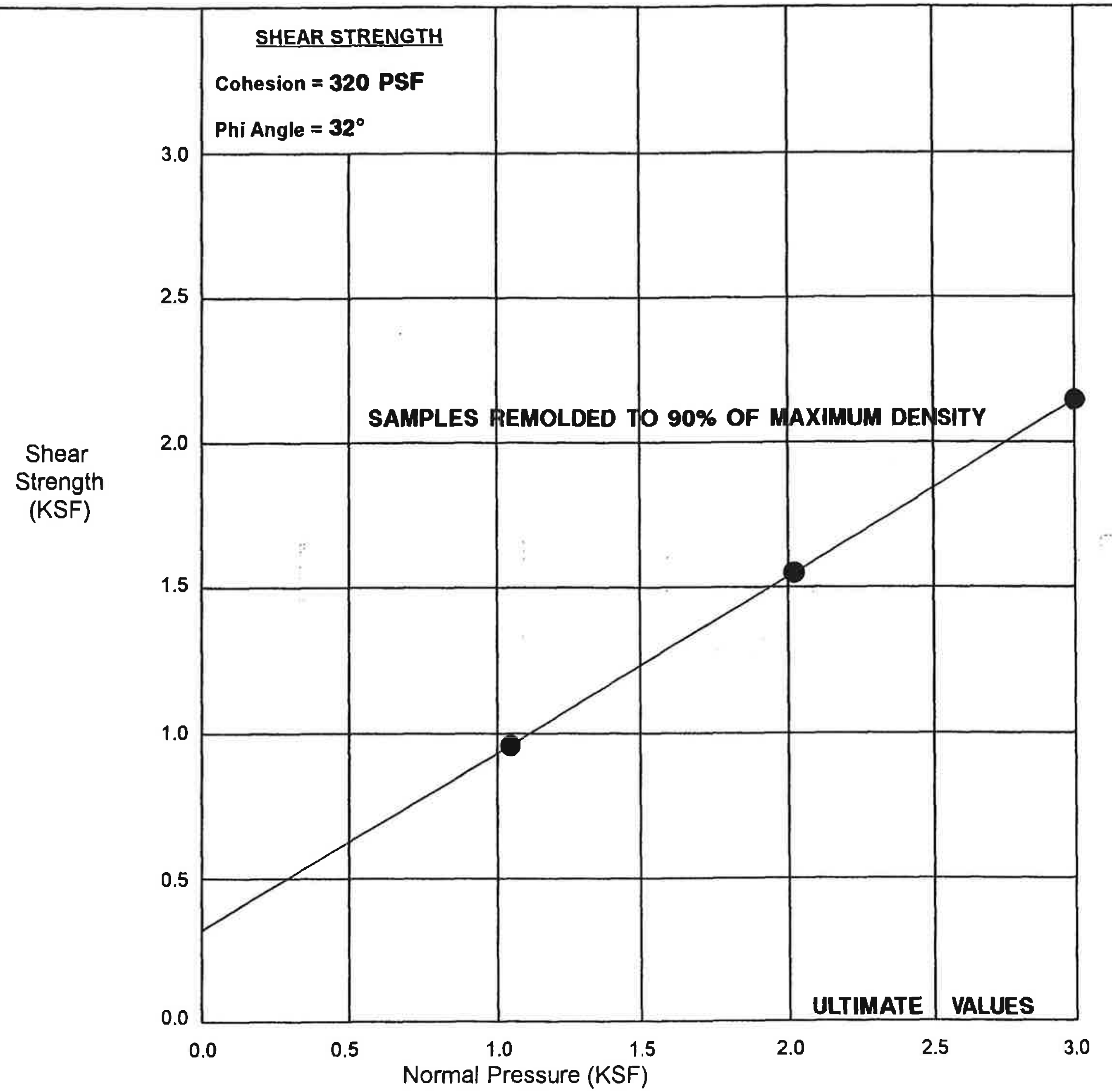
818•549•9959 Tel 818•543•3747 Fax

SHEAR TEST DIAGRAM #1

JB: 17726-B Jacmar

SAMPLE: Future Fill



○ Direct Shear (Field Moisture)

● Direct Shear (Saturated)

Moisture Content (%) = **24.4**

Dry Density (pcf) = **100.8**

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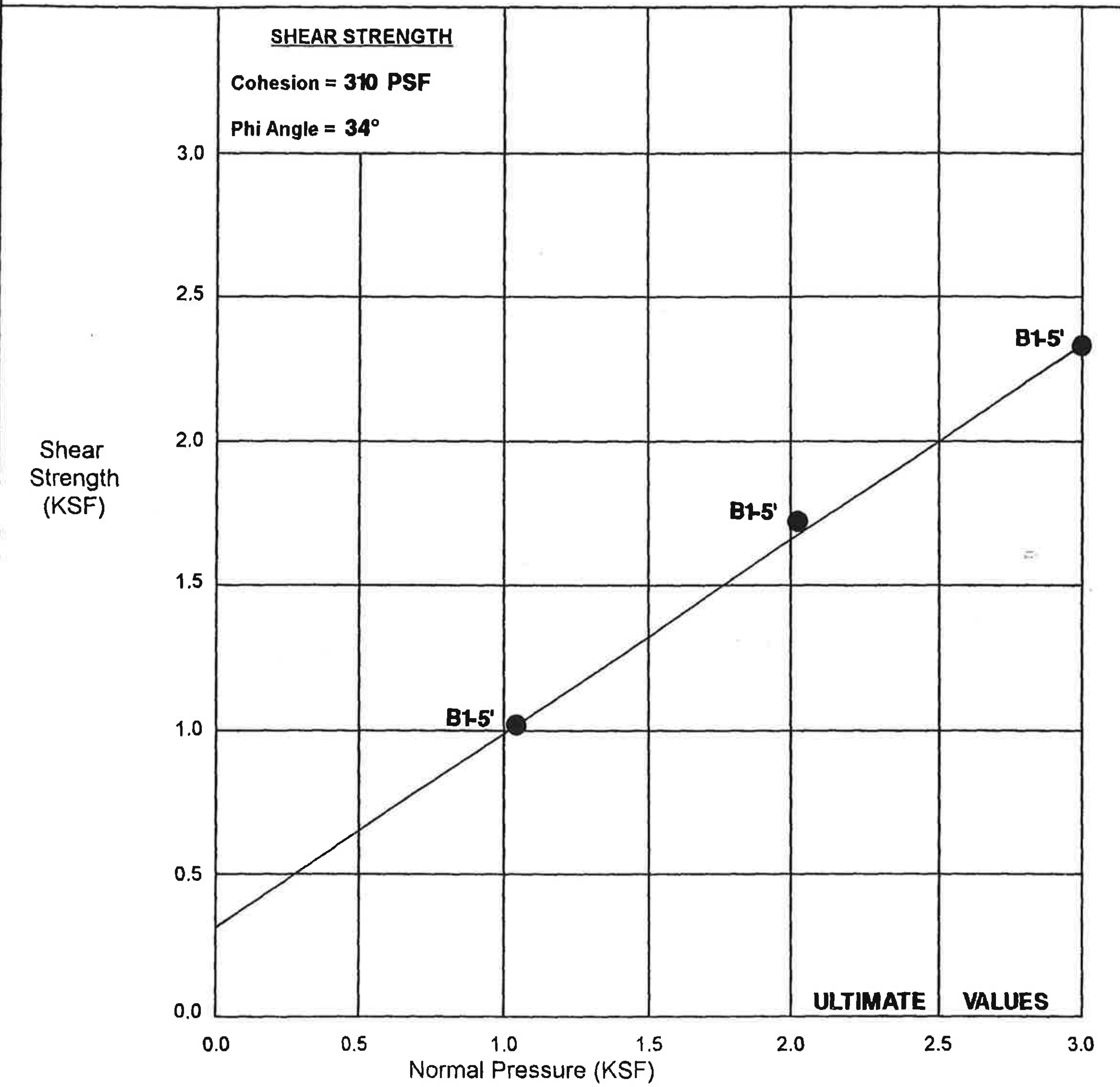
SHEAR TEST DIAGRAM

JB: 17726-B Jacmar

SAMPLE: Alluvium



○ Direct Shear (Field Moisture)

● Direct Shear (Saturated)

Moisture Content (%) = **20.0**

Dry Density (pcf) = **107.8**

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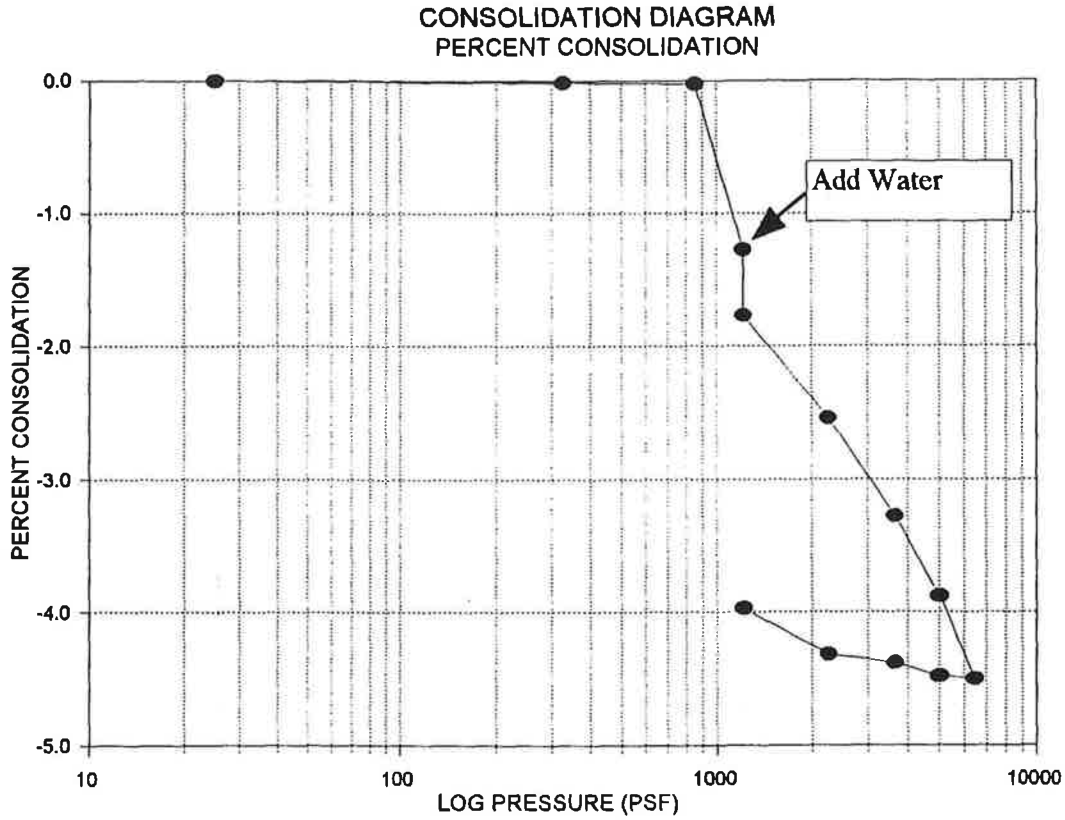
CONSOLIDATION DIAGRAM #1

JB: 17726-B Jacmar

CONSULTANT: JET

EARTH MATERIAL: Alluvium

LOCATION: B7-4'



Dry Density	93.1 pcf	Specific Gravity	2.65
Initial Moisture	6.1%	Initial Void Ratio	0.78
Initial % Saturation	20.8%	C'c	0.04

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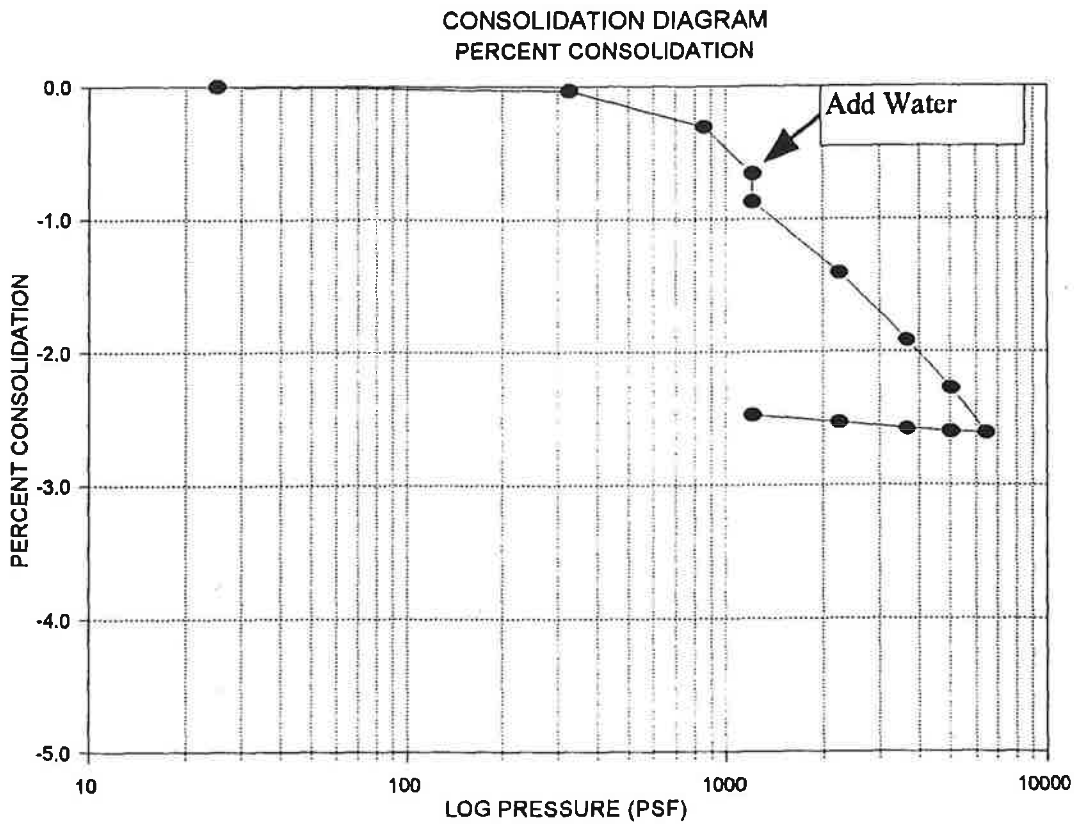
CONSOLIDATION DIAGRAM #2

JB: 17726-B Jacmar

CONSULTANT: JET

EARTH MATERIAL: Alluvium

LOCATION: B2-6'



Dry Density	103.6 pcf	Specific Gravity	2.65
Initial Moisture	4.3%	Initial Void Ratio	0.60
Initial % Saturation	19.1%	C'c	0.03

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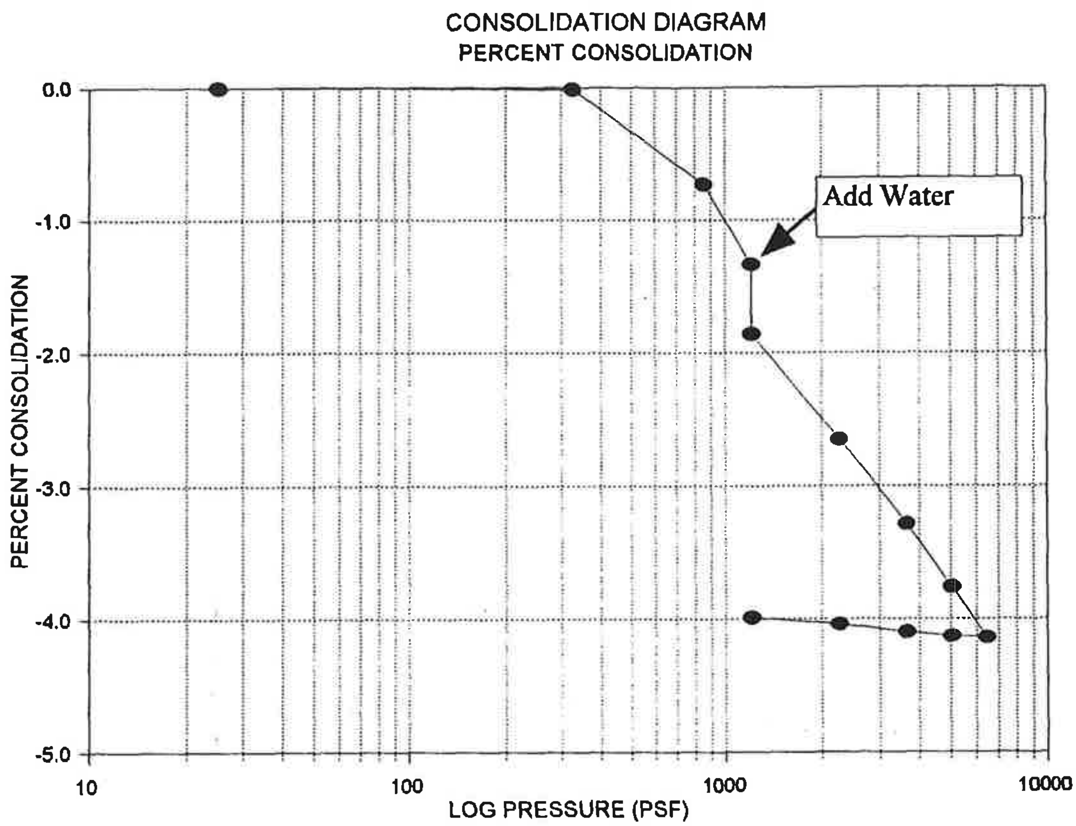
CONSOLIDATION DIAGRAM #3

JB: 17726-B Jacmar

CONSULTANT: JET

EARTH MATERIAL: Alluvium

LOCATION: B9-6'



Dry Density	119.3 pcf	Specific Gravity	2.65
Initial Moisture	2.6%	Initial Void Ratio	0.39
Initial % Saturation	17.8%	C'c	0.03

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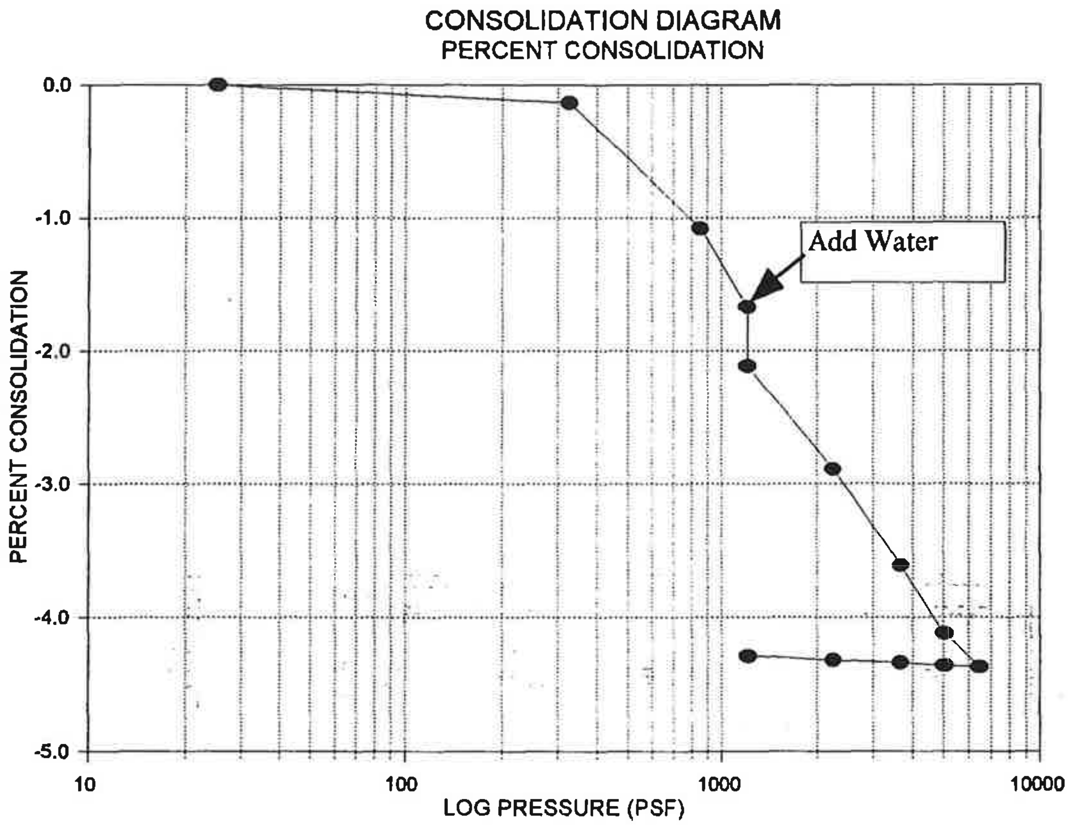
CONSOLIDATION DIAGRAM #4

JB: 17726-B Jacmar

CONSULTANT: JET

EARTH MATERIAL: Alluvium

LOCATION: B1-10'



Dry Density	103.4 pcf	Specific Gravity	2.65
Initial Moisture	2.0%	Initial Void Ratio	0.60
Initial % Saturation	8.8%	C'c	0.03

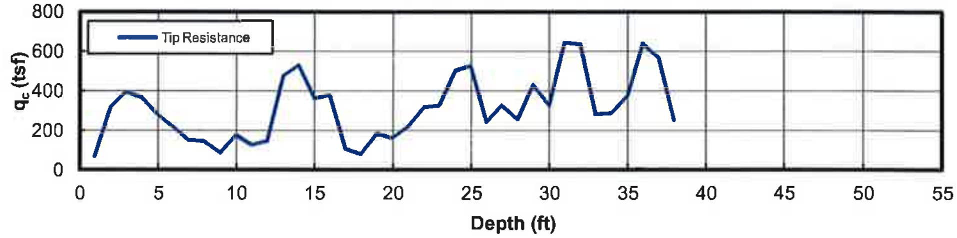
APPENDIX D

Liquefaction and Seismic Settlement Analyses

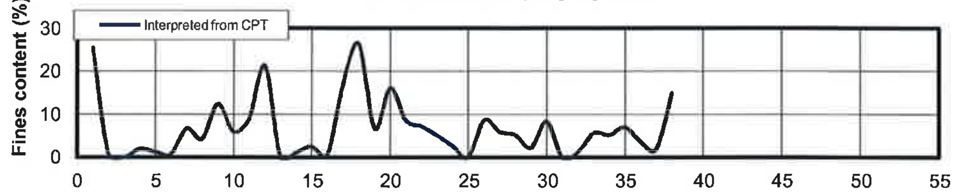
- CPT-1
- CPT-2

Kaiser Irwindale MOB
CPT-1 (2016)

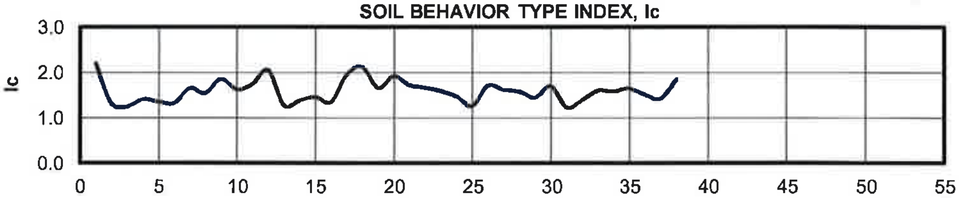
CPT TIP RESISTANCE



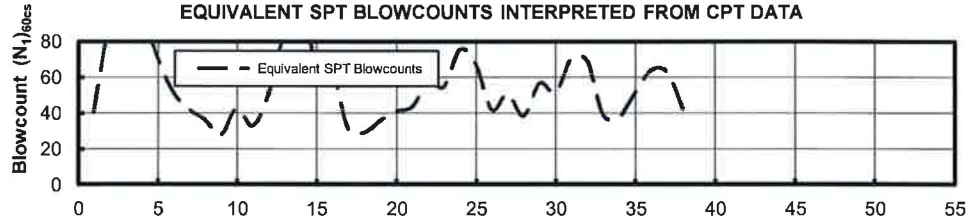
INTERPRETED FINES CONTENT



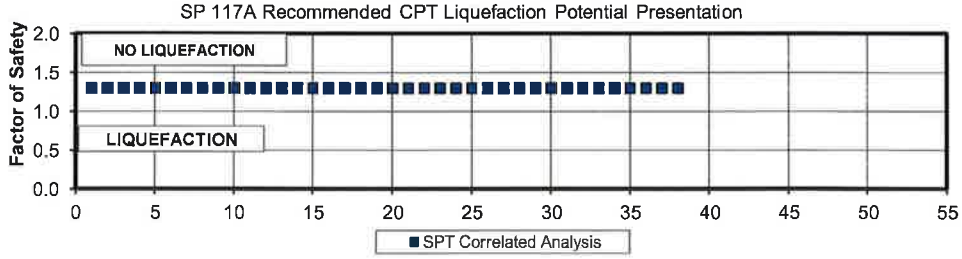
SOIL BEHAVIOR TYPE INDEX, Ic



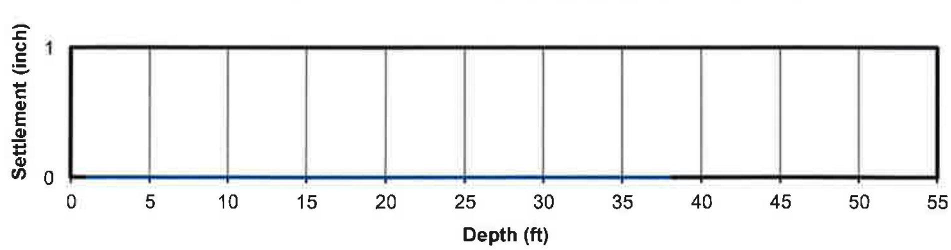
EQUIVALENT SPT BLOWCOUNTS INTERPRETED FROM CPT DATA



FACTOR OF SAFETY AGAINST TRIGGERING LIQUEFACTION
SP 117A Recommended CPT Liquefaction Potential Presentation



SEISMIC SETTLEMENT OF SATURATED SAND
USING SPT-CORRELATED ANALYSIS (Ishihara and Yoshimine, 1992)



RIRVANDALE MOB
.70.02

not liq NOT LIQUEFIABLE MATERIAL
not liq no liquefaction too dense to liquefy

not defined CPT method does not define this condition, use Chinese criteria

STEP 1: ASSUME n = 1.0 (eg. clay)

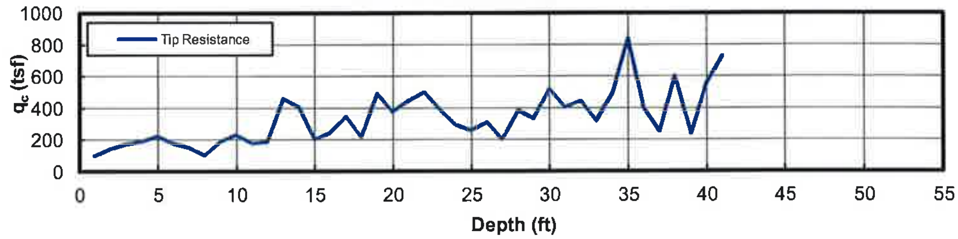
STEP 2: IF I_c < 2.6 ... OTHERWISE RECALCULATE Q ... (eg. clean sands)

STEP 3: IF RECALCULATED I_c > 2.6, RECALCULATE C_N AND Q with n = 0.7 (eg. silts and sandy silts)

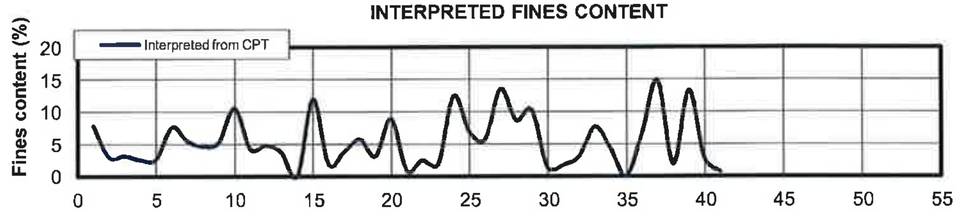
Row	Location Relative to GWT	CPT-based SPT N60	CPT-based volumetric % Strain (Calculated)	Factor of Safety against liquefaction, CPT Based	FS for Seismic settlement
1	ABOVE GW	18	23.4	ABOVE GW	2.00
2	ABOVE GW	54	0.9	ABOVE GW	2.00
3	ABOVE GW	85	0.0	ABOVE GW	2.00
4	ABOVE GW	85	1.9	ABOVE GW	2.00
5	ABOVE GW	48	1.3	ABOVE GW	2.00
6	ABOVE GW	37	0.8	ABOVE GW	2.00
7	ABOVE GW	31	6.6	ABOVE GW	2.00
8	ABOVE GW	28	4.3	ABOVE GW	2.00
9	ABOVE GW	20	12.4	ABOVE GW	2.00
10	ABOVE GW	33	3.9	ABOVE GW	2.00
11	ABOVE GW	27	9.8	ABOVE GW	2.00
12	ABOVE GW	38	21.2	ABOVE GW	2.00
13	ABOVE GW	77	0.0	ABOVE GW	2.00
14	ABOVE GW	82	1.1	ABOVE GW	2.00
15	ABOVE GW	88	2.5	ABOVE GW	2.00
16	ABOVE GW	84	0.6	ABOVE GW	2.00
17	ABOVE GW	27	17.1	ABOVE GW	2.00
18	ABOVE GW	23	26.3	ABOVE GW	2.00
19	ABOVE GW	38	6.6	ABOVE GW	2.00
20	ABOVE GW	40	16.1	ABOVE GW	2.00
21	ABOVE GW	48	8.6	ABOVE GW	2.00
22	ABOVE GW	88	7.1	ABOVE GW	2.00
23	ABOVE GW	85	5.1	ABOVE GW	2.00
24	ABOVE GW	82	2.6	ABOVE GW	2.00
25	ABOVE GW	84	0.0	ABOVE GW	2.00
26	BELOW GW	52	0.5	not liq	2.00
27	BELOW GW	88	5.9	not liq	2.00
28	BELOW GW	81	5.1	not liq	2.00
29	BELOW GW	77	2.2	not liq	2.00
30	BELOW GW	70	0.3	not liq	2.00
31	BELOW GW	100	0.0	not liq	2.00
32	BELOW GW	100	1.1	not liq	2.00
33	BELOW GW	57	5.6	not liq	2.00
34	BELOW GW	57	5.1	not liq	2.00
35	BELOW GW	78	7.0	not liq	2.00
36	BELOW GW	100	3.5	not liq	2.00
37	BELOW GW	100	1.9	not liq	2.00
38	BELOW GW	81	14.9	not liq	2.00

Kaiser Irwindale MOB
CPT-2 (2016)

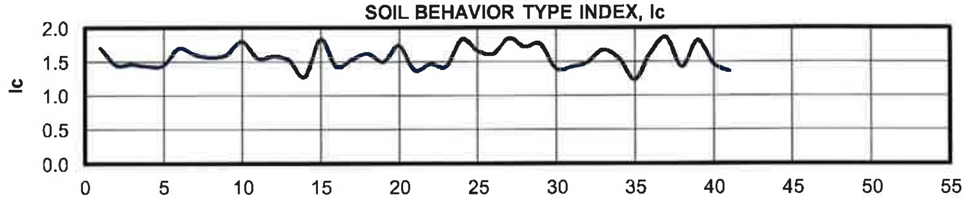
CPT TIP RESISTANCE



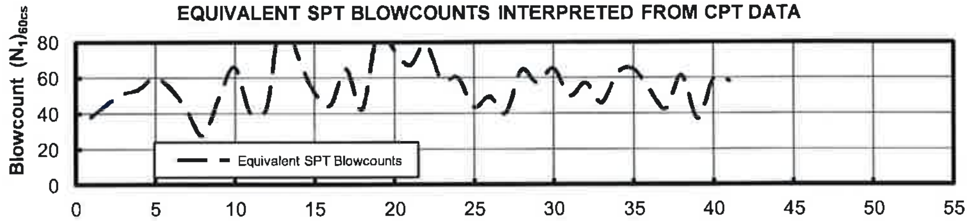
INTERPRETED FINES CONTENT



SOIL BEHAVIOR TYPE INDEX, Ic

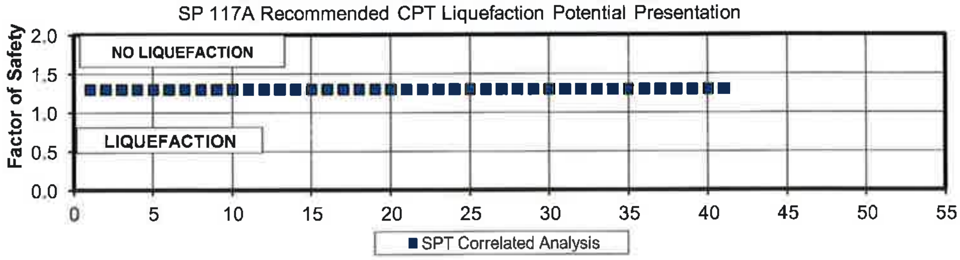


EQUIVALENT SPT BLOWCOUNTS INTERPRETED FROM CPT DATA



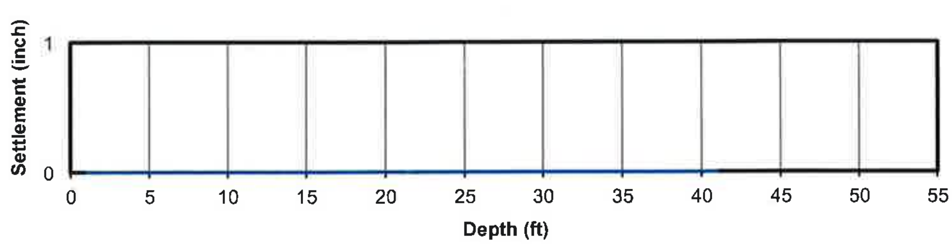
FACTOR OF SAFETY AGAINST TRIGGERING LIQUEFACTION

SP 117A Recommended CPT Liquefaction Potential Presentation



SEISMIC SETTLEMENT OF SATURATED SAND

USING SPT-CORRELATED ANALYSIS (Ishihara and Yoshimine, 1992)



IRWINDALE MOB
70.02
2011

not liq NOT LIQUEFIABLE MATERIAL
no liq no liquefaction, too dense to liquefy

not defined CPT method does not define this condition; use Chinese criteria

STEP 1: ASSUME n = 1.0 (eg. clay) | STEP 2: If Ic > 2.6 "not liq"; OTHERWISE RECALCULATE Cq AND Q with n = 0.5 (eg. clean sands) | STEP 3: IF RECALCULATED Ic > 2.6, RECALCULATE Cq AND Q with n = 0.7 (eg. silts and sandy silts)

Row	Location Relative to GWT	CPT-based SPT N60	Calculated $(N_1)_{60}$: $(N_1)_{60} = N_m C_N C_E$	CPT-based estimate % fines (Calculated)	Factor of Safety against liquefaction, CPT Based (SP117 interpretation)	FS for Seismic settlement	column Number
1	ABOVE GW	22	44	[illegible]	ABOVE GW	2.00	14
2	ABOVE GW	27	54	3.0	ABOVE GW	2.00	14
3	ABOVE GW	32	64	3.1	ABOVE GW	2.00	14
4	ABOVE GW	35	70	2.5	ABOVE GW	2.00	14
5	ABOVE GW	41	75	2.6	ABOVE GW	2.00	14
6	ABOVE GW	38	83	7.6	ABOVE GW	2.00	14
7	ABOVE GW	31	48	3.5	ABOVE GW	2.00	14
8	ABOVE GW	21	30	4.6	ABOVE GW	2.00	11
9	ABOVE GW	38	52	5.3	ABOVE GW	2.00	14
10	ABOVE GW	32	87	10.6	ABOVE GW	2.00	14
11	ABOVE GW	34	42	4.2	ABOVE GW	2.00	14
12	ABOVE GW	37	44	4.8	ABOVE GW	2.00	14
13	ABOVE GW	87	88	3.7	ABOVE GW	2.00	14
14	ABOVE GW	67	73	0.0	ABOVE GW	2.00	14
15	ABOVE GW	47	49	12.0	ABOVE GW	2.00	14
16	ABOVE GW	44	45	1.9	ABOVE GW	2.00	14
17	ABOVE GW	86	65	3.8	ABOVE GW	2.00	14
18	ABOVE GW	44	42	5.7	ABOVE GW	2.00	14
19	ABOVE GW	82	85	3.1	ABOVE GW	2.00	14
20	ABOVE GW	82	74	8.9	ABOVE GW	2.00	14
21	ABOVE GW	78	87	0.9	ABOVE GW	2.00	14
22	ABOVE GW	82	78	2.5	ABOVE GW	2.00	14
23	ABOVE GW	70	59	1.8	ABOVE GW	2.00	14
24	ABOVE GW	88	57	12.4	ABOVE GW	2.00	14
25	ABOVE GW	54	43	6.9	ABOVE GW	2.00	14
26	BELOW GW	63	50	5.7	not liq	2.00	14
27	BELOW GW	48	37	13.5	not liq	2.00	14
28	BELOW GW	64	64	8.7	not liq	2.00	14
29	BELOW GW	75	56	10.2	not liq	2.00	14
30	BELOW GW	81	67	1.4	not liq	2.00	14
31	BELOW GW	71	51	1.7	not liq	2.00	14
32	BELOW GW	85	58	3.1	not liq	2.00	14
33	BELOW GW	87	47	7.6	not liq	2.00	14
34	BELOW GW	85	65	4.5	not liq	2.00	14
35	BELOW GW	100	68	0.0	not liq	2.00	14
36	BELOW GW	83	55	6.6	not liq	2.00	14
37	BELOW GW	61	40	14.9	not liq	2.00	14
38	BELOW GW	100	65	1.9	not liq	2.00	14
39	BELOW GW	58	36	13.3	not liq	2.00	13
40	BELOW GW	100	63	2.7	not liq	2.00	14
41	BELOW GW	100	82	0.7	not liq	2.00	14