APPENDIX 11.4

Geological Data

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November 28, 2001

University of California
Office of Design & Construction
3615A Canyon Crest Drive
Riverside, California 92507
Attention: Mr. George MacMullin

Job No. 01736-3

Subject: Ground Motions
Thermal Energy Storage Expansion
UC Riverside Campus
Riverside, California

Dear Mr. MacMullin:

Attached herewith are the ground motion calculations for the proposed thermal energy storage expansion, located on the campus of University of California, Riverside. This report was based upon a scope of services generally outlined in our proposal letter dated November 16, 2001.

The calculations were prepared by our consulting seismologist, Dr. Norman A. Abrahamson, based upon criteria outlined by the tank manufacturer in a letter dated October 30, 2001 to ProWest PCM, Inc. Spectral accelerations for the Design Basis Earthquake (475-year return period) are a maximum of 1.04g for 5 percent damping and 1.85g for 0.5 percent damping at a period of 0.2 seconds.

The attenuation relations utilized in the calculations were modified to account for near-fault directivity effects. These effects were noted following analysis of strong motion recordings from a seismograph located very close to the fault that ruptured during the 1992 Landers earthquake. Inclusion of the directivity effects results in higher ground motions at longer periods. A discussion of the directivity effects is included as Appendix B of the attached report.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

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Enclosure: Ground Motions by Dr. Norman A. Abrahamson (November 27, 2001)

Distribution: University of California (8)

November 27, 2001

To: Jay Martin
From: Norm Abrahamson
Subject: Report on Ground Motions for the UC Riverside Thermal Energy Storage Tank

The following is the report describing the results of the seismic hazard analysis for the UC Riverside Thermal Energy Storage Tank site. As requested, I've developed the spectra with a return period of 475 years for site condition S_B. The spectra are developed for 5% and 0.5% damping and for periods up to 10 seconds.

Best Regards,

Norm Abrahamson

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Response Spectra for UC Riverside Thermal Energy Storage Tank

Introduction

The scope of work includes an evaluation of ground motions at the site from a probabilistic seismic hazard analysis. The ground motions presented in this report are for a return period of 475 years. The spectra are developed for period up to 10 seconds to cover the convective period of the tank.

Probabilistic Hazard Analysis

Ground motions are developed for the UC Riverside Thermal Energy Storage Tank site using a probabilistic approach. The probabilistic seismic hazard analysis follows the standard approach first developed by Cornell (1968). This approach has been expanded to more fully treat both the randomness (aleatory variability) and the scientific uncertainty (epistemic uncertainty). The mathematical formulation of the hazard analysis used in this study is described in Appendix A.

Seismic Source Characterization

The UC Riverside Thermal Energy Storage Tank site is located about 9 km from the San Jacinto Fault and about 22 km from the San Andreas fault (Figure 1). Due to their high activity rate and close distance to the site, these two faults dominate the seismic hazard at the site.

The mean source parameters used in this study are listed in Table 1. This seismic source characterization is based on the CDMG source model used for the state hazard maps (USGS, CDMG, 1996) and the Working Group on California Earthquake Probabilities (1995) source model. The maximum magnitudes listed in Table 1 are computed using the Wells and Coppersmith (1994) magnitude-area scaling relation for all fault types.

San Jacinto Fault

The San Jacinto Fault is a 236 km long strike-slip fault that is highly segmented. The San Jacinto fault is divided into the five segments listed in Table 1. The site is located closest to the San Bernardino Valley segment at a distance of about 9 km.

The 1995 Working Group source characterization includes several alternatives for cascading the segments to define multiple segment ruptures. The slip-rate assigned to each rupture scenario is the product of the total slip-rate of the fault and the probability that a rupture scenario will occur.

San Andreas Fault

The San Andreas Fault is a predominately right-lateral strike-slip fault extending from Cape Mendocino to Mexico. The northern and southern sections of the fault are divided by the central creeping section which begins south of Hollister and continues to Parkfield. The southern half of the San Andreas Fault is further segmented near San Bernardino at the junction with the San Jacinto Fault. There are four segments of the San Andreas Fault between the creeping section and the San Jacinto Fault junction: Parkfield, Cholame, Carrizo, and Mohave. South of the junction, the San Andreas Fault is segmented into the San Bernardino Mountains and Coachella segments.

The site is located closest to the San Bernardino Mtn segment at a distance of 22 km.

Magnitude Density Function

The magnitude density function describes how the fault slip-rate is distributed in different size earthquakes. In this study, the characteristic model developed by Youngs and Coppersmith (1985) is used. The truncated exponential model is not considered because when it is used for faults for which the activity rate is computed from the slip-rate, it leads to a large over-prediction of the historical rate of moderate magnitude earthquakes.

The minimum magnitude used in the hazard calculation is magnitude 5.0.

Rupture Dimension Relations

Since the attenuation relations (discussed later) are based on the closest distance from the site to any point on the earthquake rupture, the dimensions of the rupture need to be specified for each magnitude. The rupture dimension is modeled using the relations for fault area and fault width developed by Wells and Coppersmith (1994) for all source types. The rupture length is computed by dividing the area by the width.

Attenuation Relations

The site is granitic bedrock corresponding to soil profile type S_B. Few attenuation relations are available for this site category. Therefore four up-to-date attenuation relations for soft-rock site conditions are used in the hazard analysis: Sadigh et al (1997), Abrahamson and Silva (1997), Campbell (1997), and Boore et al (1997). The equal hazard spectra computed using these soft-rock attenuation relations were then scaled for an SB site condition using scale factors computed from the Boore et al (1997) attenuation relation.

For the soft-rock case, an average shear-wave velocity in the top 30 m of 500 m/s is used. This value is consistent with measured shear-eave velocities for sites classified as generic "rock" in attenuation relations. The depth to basement bedrock for the Campbell (1997) attenuation relationship was 1.0 km for soft-rock. All four attenuation relationships are for a spectral damping of 5%.

The four attenuation relations listed above describe the attenuation of the average of the two horizontal components of ground motion. These attenuation relations were adjusted to account for near-fault directivity effects using a modified form of the Somerville et al. (1997) fault-rupture directivity model from Abrahamson (2000) and described in Appendix B. Somerville et al. (1997) developed an empirically-based model quantifying the effects of rupture directivity on horizontal response spectra that can be used to scale the average horizontal component computed from attenuation relations. The Somerville et al. (1997) model comprises two period-dependent scaling factors that may be applied to any ground motion attenuation relationship. One of the factors accounts for the increase in shaking intensity in the average horizontal component of motion due to near-fault rupture directivity effects. The second factor reflects the directional nature of the shaking intensity using two ratios: fault normal (FN) and fault parallel (FP) versus the average (FA) component ratios. The fault normal component is taken as the major principal axis resulting in an FN/FA ratio larger than 1 and the fault parallel component is taken as the minor principal axis with an FP/FA ratio smaller than 1. The two scaling factors depend on whether fault rupture is in the forward or backward direction, and also the length of fault rupturing toward the site.

The ground motions are developed for the horizontal component oriented perpendicular to the strike of the fault. At long periods, the ground motion on the fault normal component will be larger than on the fault parallel component due to directivity effects.

Hazard Results for Soft-Rock Site Condition

Figure 2 shows the computed equal hazard spectra for a return period of 475 years for the fault normal component of motion for a soft-rock site condition for each of the four attenuation relations. The correspond ground motions from the USGS/CDMG 1996 hazard study (Petersen et al., 1996) are also shown for comparison. At a period of 1 second the results of the current study are in good agreement with the USGS/CDMG 1996 results. At short periods, the hazard from this study is lower than the USGS/CDMG 1996 values. The reason for this difference is mostly likely the handling of moderate magnitude events. The USGS/CDMG model smoothes the historical earthquakes over space, whereas, this study has keep the earthquakes concentrated on the fault planes.

The corresponding deaggregated hazard is shown in Figures 3a, and 3b for spectral periods of 0 (peak acceleration) and 2 seconds. These deaggregration (Figure 3a) shows that at peak acceleration, the hazard is dominated by events with magnitudes between 6.5 and 7.5 at distances of 5-10 km which corresponds to the San Jacinto fault. For the longer periods, the deaggregation (Figure 3b) shows that there is a larger range of events contributing to the hazard. In addition to the events on the Jacinto fault, there is also significant contribution to the hazard from more distant events on the San Andreas fault. Events in the magnitude range of 6.5 to 8.0 and distance range of 5 to 50 km have significant contribution to the hazard.

Hard-Rock Spectra

The spectra shown in Figure 2 are for a soft-rock site condition. To account for this difference, the Boore et al (1997) model is used to develop scale factors to scale the soft-rock spectrum. Site class B corresponds to shear-wave velocities of 750-1500 m/s. Using the geometric mean of this range leads to a shear wave velocity of 1060 m/s. Using the Boore et al (1997) attenuation model, the ratio of the 5% damped spectra for Vs=1060 m/s to Vs=500 m/s is shown in Figure 4. To simplify this ratio, a frequency independent factor of 0.8 was adopted.

The average spectrum shown in Figure 2 is scaled by 0.8 and extended out to 10 seconds period using judgment based on experience with long period ground motions. The resulting spectral values are shown in Figure 5 and are listed in Table 2.

Finally, the spectrum a 0.5% damping was developed using the damping scale factors developed by Silva et al (1997). The resulting 0.5% damped spectral values are also listed in Table 2.

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Table 1. Seismic Source Parameters

Fault	Mean Slip-Rate (mm/yr)#	Downdip Width (km)	Length (km)	Mean Char Mag*
San Andreas - Carrizo	34.0	12	145	7.2
San Andreas - Mohave	30.0	12	99	7.1
San Andreas - San Bernardino Mtn (SBM)	11.3	18	107	7.3
San Andreas – Coachella (COA)	11.8	12	95	7.1
San Andreas - SBM + COA	13.0	15	202	7.5
San Jacinto - San Bernardino Vly (SBV)	4.8	15	35	6.7
San Jacinto - San Jacinto Valley (SJV)	3.6	18	42	6.9
San Jacinto – Anza (ANZ)	7.2	18	90	7.2
San Jacinto – SBV + SJV	3.6	16	77	7.1
San Jacinto – SBV + SJV + ANZ	3.6	16	167	7.4
San Jacinto – SJV + ANZ	1.2	17	132	7.4
Elsinore	15.0	15	54	7.0

For faults with multiple segment rupture, the slip-rate for each rupture scenario is given by the total slip-rate of the fault times the probability of the rupture scenario.

* Mean characteristic magnitude is computed using the Wells and Coppersmith (1994) magnitude-area scaling relation for all fault types.

Table 2. Probabilistic Equal Hazard Response for the Fault Normal Component for a Return Period of 475 years.

	Spectral Acc (g)	
Period (sec)	5% Damping	0.5% Damping
0.010	0.441	0.441
0.020	0.441	0.441
0.030	0.441	0.567
0.050	0.600	0.914
0.075	0.740	1.227
0.10	0.835	1.480
0.15	1.000	1.814
0.20	1.039	1.850
0.25	1.008	1.773
0.30	0.970	1.688
0.40	0.880	1.532
0.50	0.790	1.375
0.75	0.599	1.042
1.0	0.486	0.828
1.5	0.348	0.576
2.0	0.268	0.423
3.0	0.176	0.261
4.0	0.125	0.177
5.0	0.094	0.128
6.0	0.073	0.099
7.0	0.059	0.080
8.0	0.048	0.065
10.0	0.033	0.045

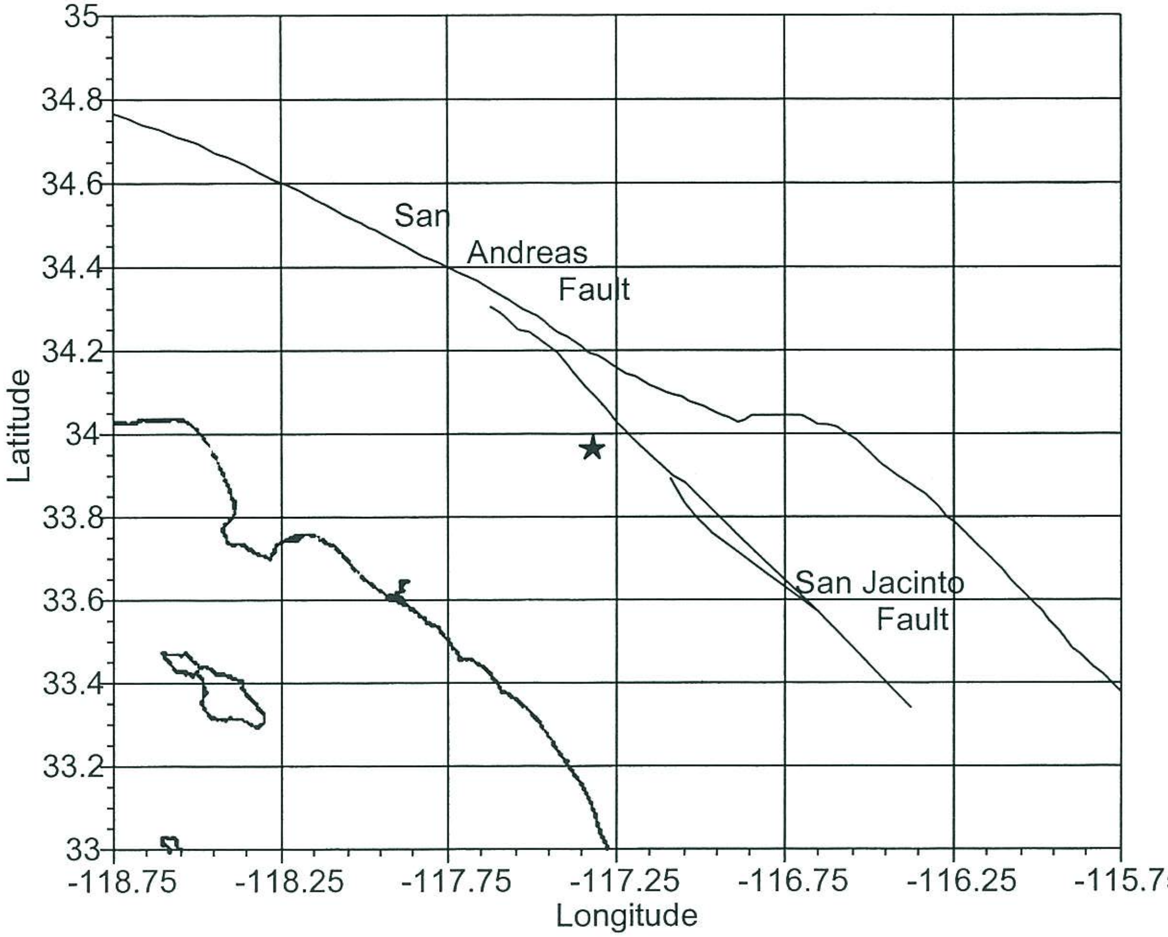

Figure 1. Location of the site and the controlling faults.

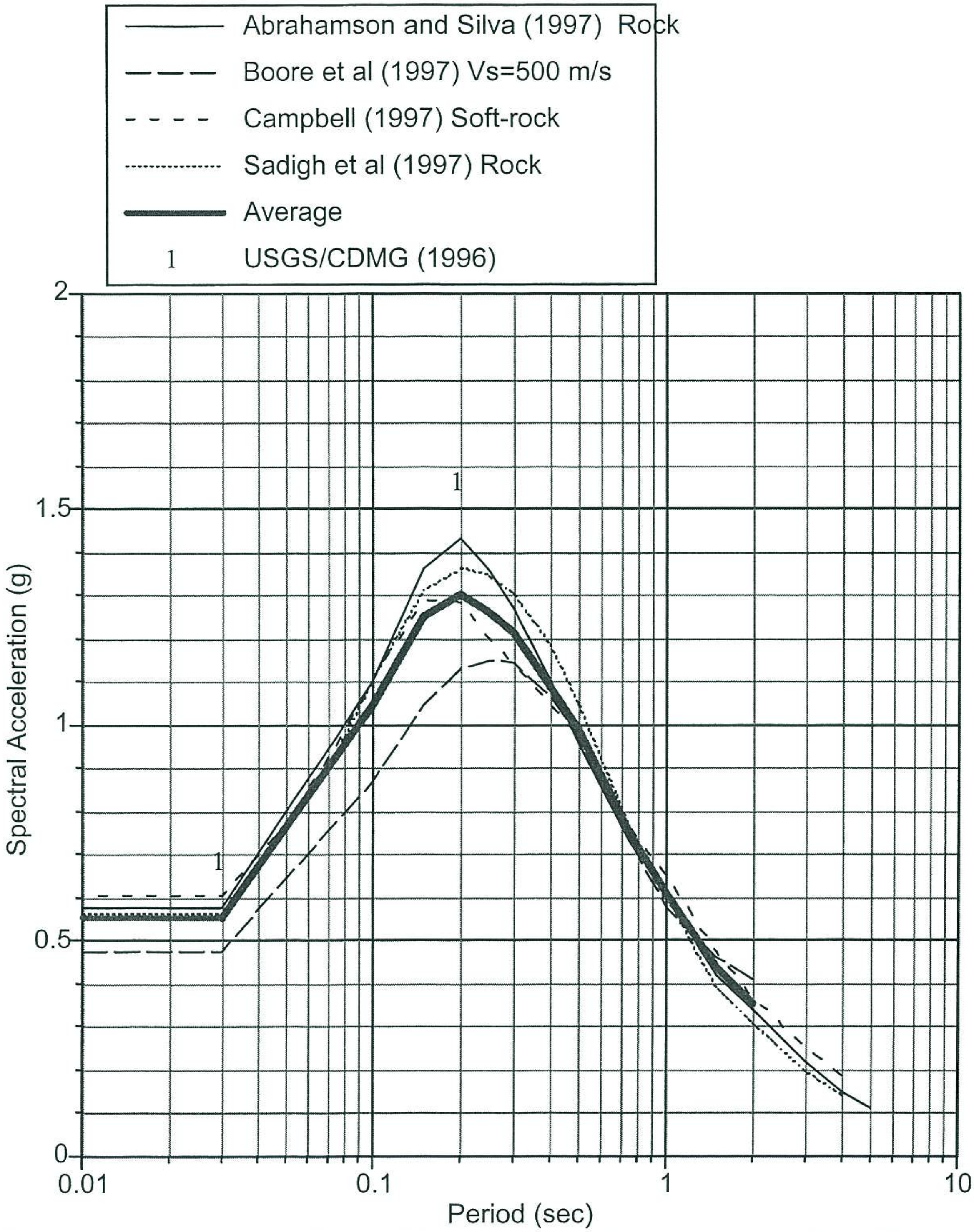

Figure 2. Comparison of the equal hazard spectra for a return period of 475 years for four different attenuation relations for a soft-rock site condition. The values from the USGS/CDMG (1996) hazard map are shown for comparison.

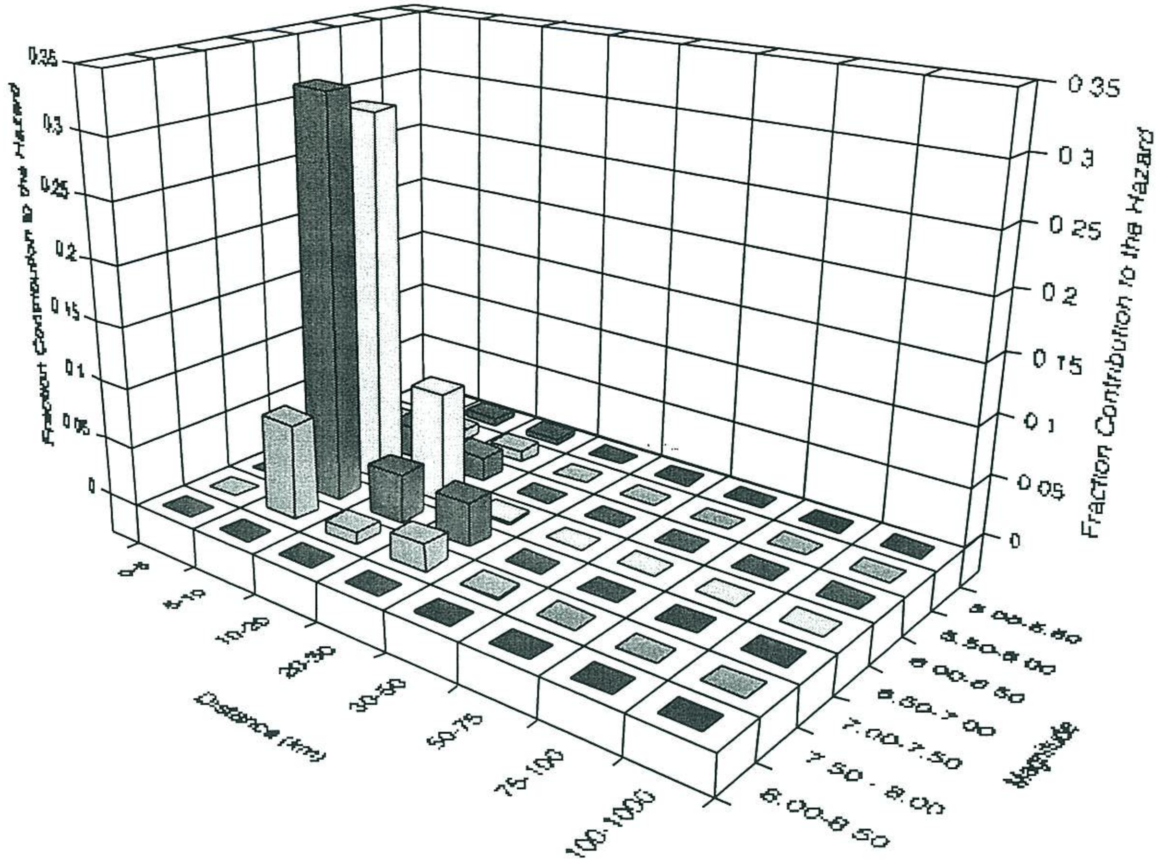

Figure 3a. Deaggregated hazard for peak acceleration for a return period of 475 years.

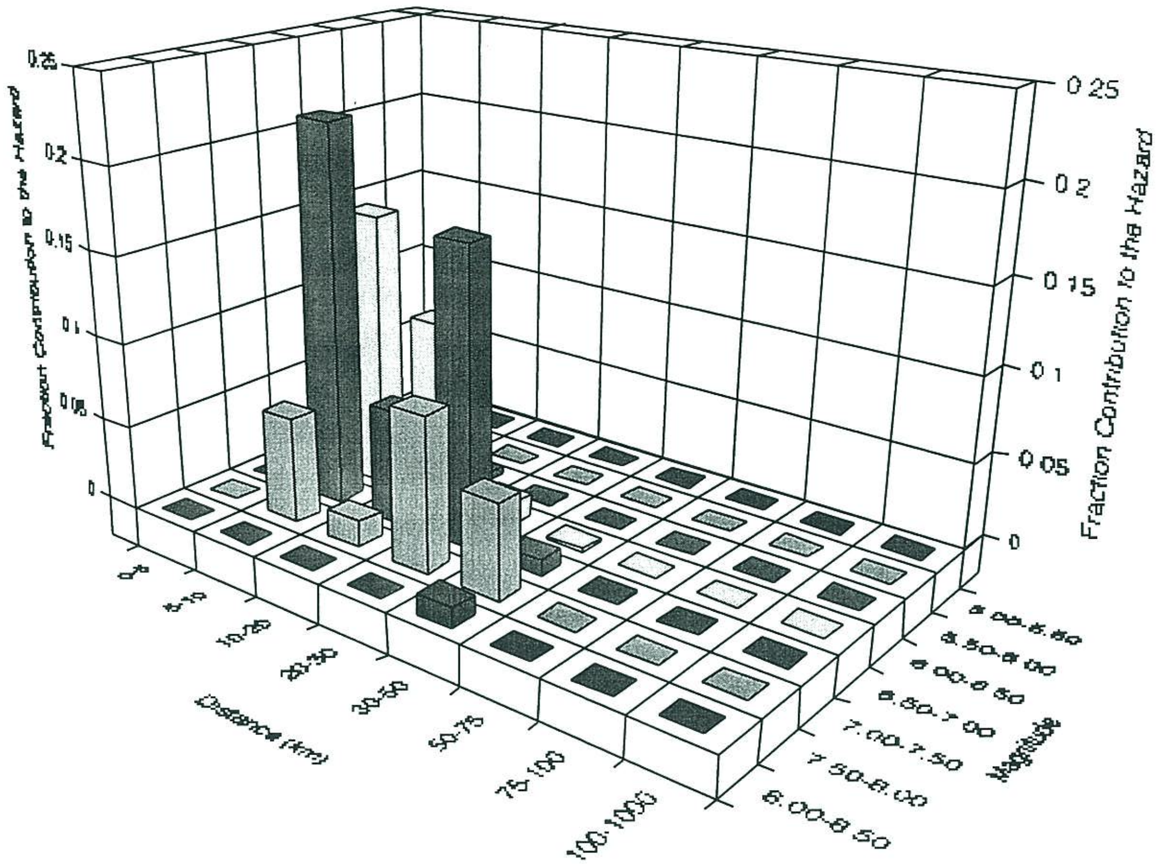

Figure 3b. Deaggregated hazard for a T=2 sec spectral period for a return period of 475 years.

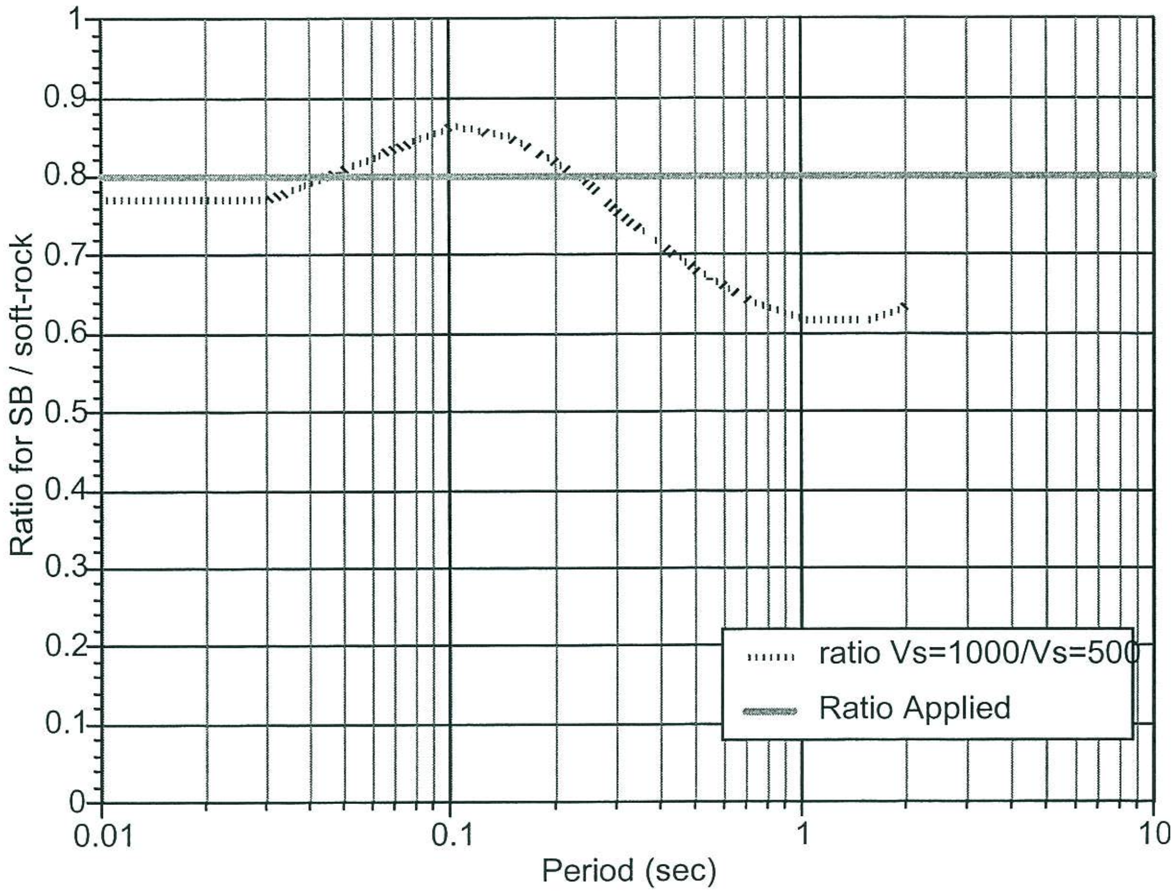

Figure 4. Scale factor for estimating spectra for site class S_B from spectra for generic soft-rock. The ratio for Vs=1000 m/s to Vs=500 m/s is based on the Boore et al (1997) model. For this study, an average value of 0.8 is used for all periods.

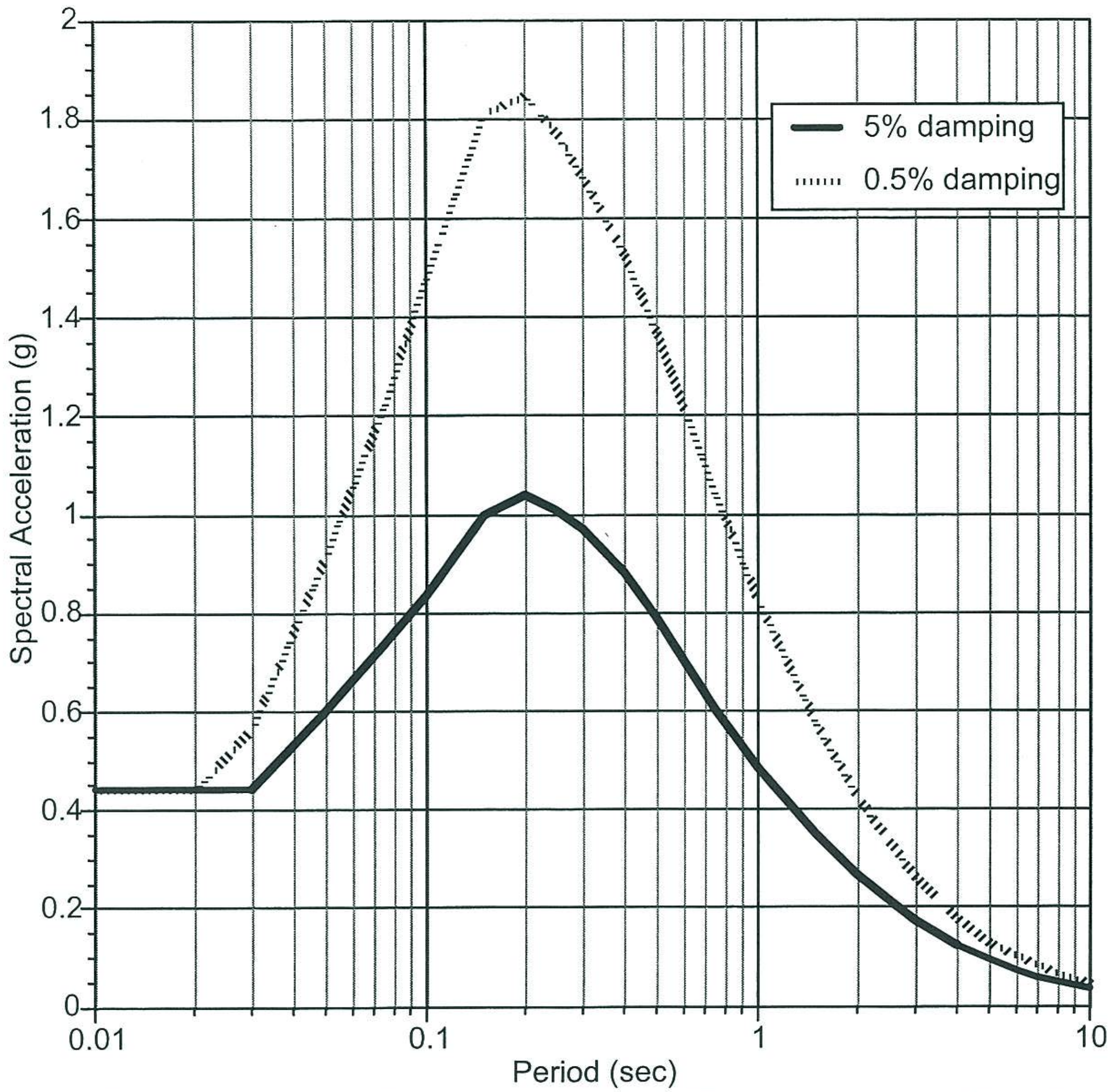

Figure 5. Response spectra for a return period of 475 years for site condition S_B.

APPENDIX A
PROBABILISTIC SEISMIC HAZARD ASSESSMENT MATHEMATICAL FORMULATION

The probabilistic seismic hazard analysis follows the standard approach first developed by Cornell (1968). The main change from the original work is that more parameters are randomized (a more complete description of the aleatory variables) and epistemic uncertainty is considered. In particular, the aleatory variability in the ground motion was not considered in the original work. The ground motion aleatory variability has a large effect on the hazard and can not be ignored.

The basic methodology involves computing how often a specified level of ground motion will be exceeded at the site. The hazard analysis computes the annual number of events that produce a ground motion parameter, A, that exceeds a specified level, "a". This number of events per year, ν, is also called the "annual frequency of exceedance". The inverse of ν is called the "return period".

The calculation of the annual frequency of exceedance, ν, involves the rate of earthquakes of various magnitudes, the rupture dimension of the earthquakes, the location of the earthquakes relative to the site, and the attenuation of the ground motion from the earthquake rupture to the site.

The annual rate of events from the i^{th} source that produce ground motions that exceed "a" at the site is the product of the probability that the ground motion exceeds the test value given that an earthquake has occurred on the i^{th} source and the annual rate of events with magnitude greater than m_{min} on the i^{th} source.

$$\nu_i(A>a) = N_i(m_{min})P_i(A>a|E_i(m \geq m_{min}) \qquad \text{(A-1)}$$

where $N_i(m_{min})$ is the annual number of events with magnitude greater than m_{min} on the i^{th} source and $E_i(m_{min})$ indicates that an event with magnitude ≥ m_{min} has occurred on the i^{th} source

For multiple seismic sources, the total annual rate of events with ground motions that exceed z at the site is just the sum of the annual rate of events from the individual sources (assuming that the sources are independent).

$$\nu(A>a) = \sum_{i=1}^{N_{source}} \nu_i(A>a) \qquad \text{(A-2)}$$

Hazard for Fault Sources

Fault sources are modeled by multiple-planes which allows changing the strike of the fault. For planar sources (e.g. known faults), we need to consider the finite dimension and location of the rupture in order to compute the closest distance. Specifically, we need to randomize the rupture length, rupture width, rupture location along strike, rupture location down dip, and hypocenter location along the rupture length (for strike-slip faults). (Since rupture width and length are correlated, it is easier to consider the rupture area and rupture width and then back calculate the rupture length.)

The general form of the conditional probability for the i^{th} fault is given by:

$$P_i(A>a \mid E_i(m \geq m_{min})) = \int_{RA=0}^{\infty} \int_{RW=0}^{\infty} \int_{Ex=0}^{1} \int_{Ey=0}^{1} \int_{x=0}^{1} \int_{m=M_{min}}^{M_{max\,i}} f_{m_i}(m)\; f_{RA_i}(m) f_{RW_i}(m)\; f_{Ex}(Ex)\; f_{Ey}(Ey)$$

$$f_{x_i}(x)\; P_i(A>a \mid m, r(Ex,Ey,RA,RW),x)\, dm\, dx\, dEy\, dEx\;\; dRW\, dRA$$

where $f_{RW}(m)$, $f_{RA}(m)$, $f_{Ex}(Ex)$, $f_{Ey}(Ey)$, $f_x(x)$, and $f_m(m)$ are probability density functions for the rupture width, rupture area, rupture location along strike, rupture location down dip, hypocenter location in the rupture plane, and magnitude, respectively. The models used for these probability density functions are described later.

For the fault normal component (FN), the probability of exceeding the ground motion "a" for a given magnitude, m, and closest distance, r, and hypocenter location, x, is given by

$$P(A > a|m,r,x) = 1 - \Phi\left(\frac{\ln(a) - \ln(Sa_{FN}(m,r,x))}{\sigma(m)}\right)$$

Where $Sa(m,r,x)$ and $\sigma(m)$ are the median and standard deviation of the ground motion from the attenuation relations for the fault normal component as described in appendix B, and $\Phi()$ is the normal probability integral given by

$$\Phi(z) = \int_{-\infty}^{z} \frac{1}{\sqrt{2\pi}}\, e^{-u^2/2}\, du$$

Probability of Exceedance

The annual rate of events given in Eq (A-2) is not probability; it can exceed 1. To convert the annual rate of events to a probability, we consider the probability that the ground motion exceeds test level "a" at least once during a specified time interval.

At this step, a common assumption is that the occurrence of earthquakes is a Poisson process. That is, there is no memory of past earthquakes, so the chance of an earthquake occurring in a given year does not depend on how long it has been since the last earthquake. (Non-Poisson models are discussed later.) If the occurrence of earthquakes is a Poisson process then the occurrence of peak ground motions is also a Poisson process. For a Poisson process, the probability of an event (e.g. ground motion exceeding z) occurring n times in time interval t is given by

$$p_n(t) = \exp(-\nu t)\,(\nu t)^n/n! \qquad \text{(A-6)}$$

The probability that at least one event occurs (e.g. $n \geq 1$) is 1 minus the probability that no events occur:

$$P(n\geq 1,t) = 1 - p_o(t) = 1 - \exp(\,-\nu t\,) \qquad \text{(A-7)}$$

So the probability of at least one occurrence of ground motion level z in t years is given by

$$P(A>a,t) \;=\; 1 - \exp(\,-\nu(A>a)t\,) \qquad \text{(A-8)}$$

For t=1 year, this probability is the annual hazard.

Aleatory and Epistemic Uncertainty

The basic part of the hazard calculation is computing the integrals in Eq (A-3). All of the aleatory variables are inside of the hazard integral. The randomness of the seismic source variables is characterized by the probability density functions which are discussed below. The randomness of the attenuation relation is accounted for in the probability of exceeding the ground motion, "a", for a given magnitude and closest distance.

Epistemic (scientific) uncertainty is considered by using alternative models and/or parameter values for the probability density functions, attenuation relation, and activity rate. For each alternative model, we recalculate the hazard and compute alternative hazard curves. Epistemic uncertainty is typically handled using a logic tree approach for specifying the alternative models for the density function, attenuation relation, and activity rates.

Activity Rate

There are two approaches to estimating the fault activity rate: historical seismicity and geologic (and geodetic) information.

If historical seismicity catalogs are used to estimate the activity rate, then the estimate of $N(m^L)$ is usually based on fitting the truncated exponential model (discussed below) to the historical data. Maximum likelihood procedures are generally preferred over least-squares for estimating the activity rate and the b-value.

When using geologic information on slip-rates of faults, the activity rate is computed by balancing the energy build-up estimated from geologic evidence with the total energy release of earthquakes. Knowing the dimension of the fault, the slip-rate, and the rigidity of the fault, we can balance the long term seismic moment so that the fault is in equilibrium. (e.g. Youngs and Coppersmith, 1985).

The seismic energy release is balanced by requiring the build up of seismic moment to be equal to the release of seismic moment in earthquakes. The build up of seismic moment is computed from the long term slip-rate. The seismic moment, Mo (in dyne cm), is given by

$$Mo = \mu A D \tag{A-9}$$

where μ is the rigidity of the crust, A is the area of the fault (in cm^2), and D is the average displacement (slip) on the fault surface (in cm). The annual rate of build up of seismic moment is given by

$$Mo = \mu A S \tag{A-10}$$

where S is the slip-rate in cm/year. The seismic moment released during an earthquake is given by

$$\log_{10} Mo = 1.5 M + 16.05 \tag{A-11}$$

where M is the moment magnitude of the earthquake.

To balance the moment build up and the moment release, the annual moment rate from the slip-rate is set equal to the sum of the moment released in all of the earthquakes that are expected to occur each year.

$$\mu AS = N(m^L) \int_{m=M^L}^{m^U} f_m(m)\, 10^{(1.5m + 16.05)}\, dm \qquad \text{(A-12)}$$

Given the slip-rate, fault area, and magnitude density function, the activity rate, $N(m^L)$ is given by:

$$N(m^L) = \frac{\mu AS}{\int_{m=M^L}^{m^U} f_m(m)\, 10^{(1.5m + 16.05)} dm}$$

Magnitude Density Distribution

The magnitude density distribution describes the relative number of large magnitude and moderate magnitude events that occur on the seismic source. Two alternative magnitude density functions are considered: the truncated exponential model and the characteristic model.

The truncated exponential model is the standard Gutenberg-Richter model that is truncated at the minimum and maximum magnitudes and renormalized so that it integrates to unity. The density function for the truncated exponential model is given by

$$f_m(m) = \frac{\beta \exp(-\beta(m-m^L))}{1-\beta \exp(-\beta(m^U-m^L))} \qquad \text{(A-14)}$$

where β is ln(10) times the b-value. Regional estimates of the b-value are usually used with this model.

The characteristic model assumes that more of the seismic energy is released in large magnitude events than for the truncated exponential model. That is, there are fewer small magnitude events for every large magnitude event for the characteristic model than for the truncated exponential model. There are different models for the characteristic model. Two commonly used models are the characteristic model as defined by Youngs and Coppersmith (1985) and the "maximum magnitude" characteristic model. In this paper, we will call these two models the characteristic model and maximum magnitude model, respectively.

The density function for the generalized form of the Youngs and Coppersmith characteristic model is given by

$$f_m(m) = \begin{cases} \dfrac{\beta \exp(-\beta(m-m^L))}{1-\beta \exp(-\beta(m^U-\Delta m_2-m^L))} \dfrac{1}{1+c} & \text{for } m < m^U-\Delta m_2 \\ \dfrac{\beta \exp(-\beta(m^U-\Delta m_1-\Delta m_2-m^L))}{1-\beta \exp(-\beta(m^U-\Delta m_2-m^L))} \dfrac{1}{1+c} & \text{for } m \geq m^U-\Delta m_2 \end{cases} \qquad \text{(A-15)}$$

where

$$c = \frac{\beta \exp(-\beta(m^U-\Delta m_1-\Delta m_2-m^L))}{1-\beta \exp(-\beta(m^U-\Delta m_2-m^L))} \Delta m_2 \qquad \text{(A-16)}$$

The density function for this model is shown in Figure A-1. In the Youngs and Coppersmith model, Δm_1=1.0 and Δm_2=0.5.

Comparing the examples of the truncated exponential and characteristic density functions shown in Figure A-1, we see that the density functions themselves are similar at small magnitudes. However, when the geologic moment-rate is used to set the annual rate of events, $N(m^L)$, then there is a large impact on $N(m^L)$ depending on the selection of the magnitude density function. Figure A-2 shows the comparison of the magnitude recurrence relation for the truncated exponential and characteristic models (using the Youngs and Coppersmith value for Δm_1 and Δm_2) when they are constrained to have the same total moment rate. The characteristic model has many fewer moderate magnitude events than the truncated exponential model (about a factor of 10 difference).

Recent studies have found that the characteristic model does a better job of matching observed seismicity than the truncated exponential (Geomatrix, 1992, Woodward-Clyde, 1994) when the total moment rate is constrained by the geologic slip-rate.

Rupture Dimension Density Functions

For the rupture area and rupture width, the density function are determined from regression models which give the rupture area and rupture width as a function of magnitude. For this project, the Wells and Coppersmith (1994) empirical models for rupture area and rupture width are used:

$$\log_{10} (RA) = -3.49 + 0.91\ M \pm 0.24 \qquad \text{(A-17)}$$

$$\log_{10} (W) = -1.01 + 0.32\ M \pm 0.15 \qquad \text{(A-18)}$$

The density functions, $f_{RA}(m)$ and $f_{RW}(m)$ are log normal distributions centered about the median values given by Eq. (A-15) and (A-16). These distributions are truncated at $\pm 2\sigma$ in the hazard calculations.

Rupture Location Density Functions

The center of the rupture location is parameterized in terms of the normalized fault length and fault width. Ex is the fraction of the fault length (measure along strike) and Ey is the fraction of the fault width (measured down dip). The location of the center of the rupture plane is assumed to be uniformly distributed over the fault plane. The resulting density functions for $f_{Ex}(E_x)$ and $f_{Ey}(E_y)$ are unity.

Hypocenter Location Density Function

For a given rupture dimension (length and width) and rupture location, the location of the hypocenter along strike is parameterized in terms of the normalized rupture length. The location of the hypocenter is assumed to be uniformly distributed over the rupture plane. The resulting density function for $f_x(x)$ is unity. In the hazard analysis, a total of 10 hypocenter locations evenly spaced along the rupture length are used for each magnitude, rupture location, and rupture dimension.

Appendix B Directivity Effects Model

Introduction

The empirical attenuation relations were developed for the average horizontal component without regard to the direction of rupture. Somerville et al (1997) developed an empirically based model quantifying the effects of rupture directivity on horizontal response spectra that can be used to scale the average horizontal component from attenuation relations. There are two effects of rupture directivity on long period response spectral values that are modeled by Somerville. First, there is an increase in the average horizontal component for cases of rupture coming toward the site and there is a decrease in the average horizontal motion for rupture running away from the site. Second, there is a systematic difference in the two horizontal components of motion when they are oriented parallel and perpendicular to the strike of the fault. At long periods, the fault normal component is larger than the fault parallel component. This increase in the fault normal component has also been studied by Geomatrix (1995).

In this project the a modified form of the Somerville et al. (1997) model has been used to characterize the two parts of the directivity effect.

Somerville et al (1997) Model

Somerville et al (1997) provides scale factors to account for directivity effects for the horizontal components. The Somerville et al model for the difference in the two horizontal components (fault normal and fault parallel) for strike-slip earthquakes is given by

$$\ln (FN/Ave\ H) = \cos(2\phi)\ [C_1 + C_2 \ln(R_{rup}+1) + C_3 (M-6)] \qquad \text{(B-1)}$$

for M>6 and $\phi < 45$; it is zero otherwise. The coefficients for these models are listed in Table B-1. This model is used without modification in this study.

Somerville et al also provides a model for the effect of rupture direction on the average horizontal component. This model was modified for use on this project as described below.

Modifications of the Somerville et al (1997) model

There are several aspects of the empirical model for the average horizontal component scale factors developed by Somerville et al that needed to be modified to make the model applicable to a probabilistic hazard analysis.

(A) Distance dependence

As published, the model is independent of distance. The data set used in the analysis includes recordings at distances of 0 to 50 km. A distance dependent taper function was applied to the model that reduces the effect to zero for distances greater than 60km.

$$T_d(r) = \begin{cases} 1 & \text{for } r < 30 \text{ km} \\ 1 - (r-30)/30 & \text{for } 30 \text{ km} < r < 60 \text{ km} \\ 0 & \text{for } r > 60 \text{ km} \end{cases} \qquad \text{(B-2)}$$

(B) Magnitude dependence

As published, the model is applicable to magnitudes greater than 6.5. A magnitude taper was applied that reduces the effect to zero for magnitudes less than 6.0.

$$T_m(m) = \begin{cases} 1 & \text{for } m \geq 6.5 \\ 1 - (m-6.5)/0.5 & \text{for } 6 \leq m < 6.5 \\ 0 & \text{for } m < 6 \end{cases} \qquad \text{(B-3)}$$

(C) Saturation With x cos(θ)

The empirical model uses a form that increases a constant rate as x increases from 0 to 1. There is little empirical data with x cos(θ) values greater than 0.6, particularly for rupture distances less than 20 km. The short distance data suggest that there may be a saturation of the directivity effect as a function of x cos(θ).The extrapolation of the model to larger x cos(θ) values is not well constrained. To evaluate this extrapolation, three separate groups applied their seismological numerical modeling methods to generate synthetic time histories for a range of x cos(θ) values. The numerical modeling results indicated that the directivity effect saturates for x cos(θ) > 0.4, As a result, the functional form of the directivity model was changed to include saturation with x cos(θ). The coefficients of the model were based on the empirical data, and not on the synthetics.

Based on the trends in the numerical simulations, the form of the directivity function is modified to reach a maximum at x cos(θ) =0.4. The model was developed for a spectral period of 3 seconds because this is an important period for the bridge. The slope is greater than the Somerville model, but it flattens out at a lower level . The hazard calculation is sensitive to the model values at large x cos(θ) (greater than 0.9) so this change results in a reduction of the ground motion.

The T=3 second value is used to guide the adjustment of the model at all periods. The resulting model is given by

$$y_{Dir}(x,\theta,T) = \begin{cases} C_1(T) + 1.88\, C_2(T)\, x \cos(\theta) & \text{for } x \cos(\theta) \leq 0.4 \\ C_1(T) + 0.75\, C_2(T) & \text{for } x \cos(\theta) > 0.4 \end{cases} \tag{B-4}$$

where $C_1(T) + C_2(T)$ are from Somerville et al and are listed in Table A-4a.

5. Reduction of the Standard Deviation

Including the directivity effect should results in a reduction of the standard deviation of the attenuation relation. The standard deviation of the data < 20 km including the directivity was compared to the standard deviation of the published model. At T=3 seconds, there is a reduction of about 0.05 natural log units. The period dependence of the reduction is approximated by the period dependence of the slope of the directivity effect. To account for the reduction in the standard deviation due to including the directivity effect as part of the model, the standard deviations for the published attenuation relations were modified for use in the hazard analysis using the following relation:

$$\sigma'(M,T) = \sigma\,(M,T) - 0.05\, C_2(T)/1.333 \tag{B-5}$$

where $C_2(T)$ is given in Table B-1.

Final Directivity Model

The following model is used for the average horizontal component for strike-slip faults

$$\ln Sa_{dir}\,(M,r,x,\theta,T) = \ln Sa(M,r) + y_{Dir}(x,\theta,T)\, T_d(r)\, T_m\,(m) \tag{B-6}$$

where Sa(M,r) is an empirical attenuation relation without directivity.

C·H·J· INCORPORATED

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November 8, 2001

University of California
Office of Design & Construction
3615A Canyon Crest Drive
Riverside, California 92507
Attention: Mr. George MacMullin

Job No. 01736-3

Subject: Subsurface Investigation
Proposed Thermal Energy Storage Tanks
University of California
Riverside, California

Reference: Geotechnical Investigation
Thermal Energy Storage Expansion and Satellite Plant
University of California
Riverside, California
Report Prepared by C.H.J., Incorporated
Dated August 31, 2001, Job No. 01736-3

Dear Mr. MacMullin:

As requested, we have conducted a subsurface investigation of the proposed thermal energy storage tanks located on the campus of the University of California, Riverside. As you know, this firm recently completed a geotechnical investigation of the eastern proposed tank (referenced above), referred to as the "new tank" in this report. That investigation excluded the western proposed tank, referred to as the "future tank" in this report. The purpose of this subsurface investigation was to obtain the subsurface data needed for planning for the future tank, and to obtain additional subsurface data for the new tank. A 20-scale Preliminary Grading Plan, prepared by Bechard Long & Associates, Inc., was used during our investigation. Both proposed tanks will have a bottom elevation of 1,146.9 feet, which will require a cut depth of approximately 40 feet below existing ground at the center of each tank.

SCOPE OF SERVICES

The scope of services provided during this subsurface investigation included the following:

- Review of previous geotechnical investigations conducted by C.H.J., Incorporated in the site vicinity, including an investigation for the nearby TES tank constructed in 1993 (C.H.J., Inc., December 31, 1992)

- Placement of three exploratory hollow stem auger borings on the future tank site
- Placement of seven exploratory percussion (air track) borings on the new and future tank sites
- Logging and sampling of the exploratory borings
- Placement of three seismic refraction lines to quantitatively evaluate the rippability of the subsurface materials (bedrock) at the location of the future tank
- Evaluation of the subsurface data to develop site-specific recommendations for site excavation

EXPLORATORY BORINGS

The subsurface conditions underlying the future tank site were explored by means of three exploratory hollow stem auger borings drilled to a maximum depth of 55.5 feet below the existing ground surface. The borings were drilled with a CME 55 truck-mounted drill rig. The approximate locations of the hollow stem auger borings are indicated on the enclosed Plat (Appendix "A"). Continuous logs of the subsurface conditions, as encountered within the exploratory auger borings, were recorded at the time of drilling by a staff geologist from this firm. Bulk samples of typical soil types obtained were returned to the laboratory for classification.

The subsurface conditions underlying both the new and future tank sites were also explored by means of seven exploratory percussion borings drilled to a maximum depth of 59.0 feet below the existing ground surface. The percussion borings were drilled with an Ingersoll-Rand ECM 370 drill rig with a 3.5 inch diameter bit. The approximate locations of the percussion borings are indicated on the enclosed Plat (Appendix "A"). Continuous logs of the subsurface conditions, as encountered within the exploratory percussion borings, were recorded at the time of drilling by a staff geologist from this firm. Penetration rates were recorded in the field for correlation with rock hardness and estimates of rippability. A chart relating rock penetration rates to rock density and estimated rippability was provided to us by the drilling contractor (Baxter Drilling) and modified slightly for inclusion in this report (Appendix "B").

The exploratory boring logs are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

Two of the three auger borings did not encounter refusal to the anticipated pad elevation of the proposed future tank. Boring No. 10 encountered refusal in bedrock at 36.5 feet. In general, auger borings encounter refusal in granitic bedrock at a velocity of approximately 4,500 fps. All of the auger borings drilled at the new tank site during the previous investigation (C.H.J., Inc., August 31, 2001) encountered refusal at depths of 21 to 25 feet.

The four percussion borings drilled at the future tank site (PB-1 through PB-4) encountered penetration rates in bedrock of less than 17 seconds per foot (spf). The correlation chart (Appendix "B") indicates that bedrock with a penetration rate of less than 18 spf can be expected to be rippable. Three of these borings were drilled to below the elevation of the proposed pad. The fourth was terminated at a shallow depth (24 feet) due to time constraints.

The three percussion borings drilled at the new tank site (PB-5 through PB-7) encountered slighly slower penetration rates in bedrock (up to 19 spf at 25 feet deep in PB-7), falling into the probable non-rippable range according to the correlation chart. Two of these borings were drilled to below the elevation of the proposed pad. The third was terminated at a shallow depth (36 feet) due to time constraints.

SEISMIC REFRACTION

Based on the proposed tank pad elevation of 1,145 feet, a cut of approximately 40 feet in depth can be expected at center of the future tank. The results of a seismic refraction survey conducted on the new tank during our referenced geotechnical investigation (C.H.J., Inc., August 31, 2001) found marginally rippable to non-rippable bedrock at a relatively shallow depth (20 to 30 feet) below ground surface at the new tank location. A seismic refraction survey was conducted to determine the seismic velocity profile of the subsurface rock at the location of the future tank and the expected depths of rippable materials with large excavation equipment.

Seismic refraction surveying is a method of geophysical exploration in which a seismic wave is generated at a fixed point and the travel time of that wave is recorded by audio detectors (geophones) placed at known distances from the source. The velocity of the seismic wave is then calculated from the wave arrival time at each geophone (Dobrin, 1976). The time-travel data is then utilized to calculate a profile of seismic velocity vs. depth.

The seismic velocity can be utilized to estimate the rippability of subsurface materials. A chart included in the Caterpillar Performance Handbook (1992) correlates seismic velocity of different rock types to rippability by a D8L or D9N bulldozers utilizing single- or multi-shank rippers. This chart indicates granitic rock, such as that which underlies the site, is rippable to a velocity up to 6,800 feet per second (fps). Granitic rock of between 6,800 and 8,000 fps is considered to be marginally rippable, and velocities of greater than 8,000 fps are considered to be non-rippable. Our experiences with similar sites in this area indicate the velocities in the Caterpillar Performance Handbook (1992) are approximately 1,500 fps too fast for reasonable production rates.

For this investigation, three seismic refraction lines were performed at the locations shown on Enclosure "A-1" (Lines S-4, S-5 and S-6). These lines were 203 to 270 feet in length, including offset end-shot points. A 16-pound sledgehammer was used as an energy source to produce the seismic waves, and twelve 14-Hz geophones (with 60 percent damping) were spaced at 7 to 20 foot intervals along the lines to detect both the direct and refracted waves. The seismic wave arrivals were recorded on a 12-channel Geometrics SmartSeis® model signal enhancement refraction/reflection seismograph. Lines S-4 and S-6 were measured in a 12-channel configuration; Line S-5 was measured with a 24-channel configuration by collecting two sets of data and combining them. The 24-channel configuration doubles the data collected and allows for more precise interpretation of the data. Field conditions were very noisy due to suspected electromechanical equipment operating in the area. Five shot points were utilized along each seismic line spread using offset, forward, reverse, and intermediate locations in order to obtain sufficient data for velocity analysis and depth modeling purposes. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrival times. These seismic wave arrival times are digitally recorded on the in-board seismograph computer and subsequently transferred to a disk. The data disk was then downloaded into our office computer for further processing, analyzing, and printing.

The data on the paper record and/or display screen were used to analyze the arrival time of the seismic waves at each geophone station in the form of a wiggle trace, or wave travel-time curve, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes using the computer programs **SIP** (**S**eismic refraction **I**nterpretation **P**rogram), developed by Rimrock Geophysics (1995), and **SeisOpt™@2D** (Optim, 2000). **SIP** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on the time-distance graphs and is better suited for layered media. In addition, the computer program **SeisOpt™@2D** was also used for comparative purposes as it models the subsurface velocity gradient in discrete velocity "cells". This program is an automatic refraction interpretation package that performs velocity model optimization and visualization using repeated forward modeling. Test velocity models are created, through which travel times are calculated and are compared with the observed data, and are optimized. The optimal solution is the velocity model with the minimum travel time error between the calculated and observed data. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

Line S-4 was performed along the road along the top (south) margin of the proposed pad. This is the area of anticipated deepest cut. The time-distance data for Line S-4 are included as Enclosure "C-1". A three-layer velocity vs. depth profile is included as Enclosure "C-2". A velocity gradient model is included as Enclosure "C-3". The velocity vs. depth profile (Enclosure "C-2") shows the depth to very hard (non-rippable) bedrock (10,200 fps) with a D-9 or equivalent as approximately 40 feet (east end) to approximately 25 feet (west end). The velocity gradient model (Enclosure "C-3") shows a depth of approximately 25 feet to hard (marginal to non-rippable) bedrock (6,000 fps) at the west end of the line, deepening to greater than 40 feet at the east end of the line.

Line S-5 (24-channel) was performed perpendicular to S-4 through the center of the proposed tank. The time-distance data for Line S-5 are included as Enclosure "C-4". A three-layer velocity vs. depth profile is included as Enclosure "C-5". A velocity gradient model is included as Enclosure "C-6". The velocity vs. depth profile (Enclosure "C-5") shows the depth to very hard (non-rippable) bedrock (12,600 fps) with a D9 or equivalent as at least approximately 50 feet . The velocity gradient model (Enclosure "C-6") shows a depth of approximately 40 feet to hard bedrock (6,000 to 7,000 fps) that is expected to be marginally rippable to non-rippable. The 40 foot depth is approximately coincident with the proposed tank bottom (Enclosure "C-6").

Line S- 6 was performed parallel to S-1 through the center of the proposed tank. The time-distance data for Line S-6 are included as Enclosure "C-7". A three-layer velocity vs. depth profile is included as Enclosure "C-8". A velocity gradient model is included as Enclosure "C-9". The velocity vs. depth profile (Enclosure "C-8") shows the depth to very hard (non-rippable) bedrock (8,700 fps) with a D-9 or equivalent as at least 40 feet. The velocity gradient model (Enclosure "C-9") shows a depth of approximately 35 feet to hard bedrock (6,000+ fps) that is expected to be marginally rippable to non-rippable.

The velocity profiles generated during this investigation vary in part due to a gradual increase in velocity with depth observed in all of the lines. In this case, the velocity gradient models are expected to be more accurate in that they do not depend on a sharp contrast in layer velocities. Based on the data that were collected and interpreted, it is our expectation that the depth to marginally rippable to non-rippable rock (6,000 to 7,000 fps) is approximately 35 to 40 feet beneath most of the future tank site. Therefore, it is our expectation that relatively small amounts of dense bedrock that is non-rippable with a D9 or equivalent will be encountered during grading of the tank site. Hard bedrock that is encountered may require alternate methods of excavation such as jack hammering or blasting.

It should be noted that the seismic refraction method, under ideal conditions, is accurate to within about 10 percent. Due to the noisy field conditions encountered, the expected velocities and depths should be considered approximate only.

No outcrops or core-stones are present at the ground surface at the proposed tank site. However, hard core-stones are visible in the hillside southwest of the site, and may exist in the subsurface in the vicinity of the proposed future tank. These core-stones, if encountered, may be non-rippable and may also require alternate methods of excavation.

CONCLUSIONS

The data obtained during this investigation yielded minor apparent conflicts with respect to the anticipated depth to non-rippable bedrock at the future tank site. Only one of three hollow stem auger holes drilled encountered refusal at an elevation higher than pad grade (1,146 feet). Hollow stem auger borings normally encounter refusal at a seismic velocity of about 4,500 fps, well within the rippable range for large bulldozers. The seismic refraction data show an unexpectedly high velocity for relatively shallow bedrock (6,000 fps), which should result in refusal to the auger borings. The penetration rates obtained in the percussion borings do not reflect any bedrock that is hard enough to be considered non-rippable with a D9.

Based on the preponderance of the evidence obtained and reviewed during this investigation, it is our expectation that the depth to marginally rippable to non-rippable rock (6,000 to 7,000 fps) is approximately 35 to 40 feet beneath most of the future tank site. Therefore, it is our expectation that relatively small amounts of dense bedrock that is non-rippable with a D9 or equivalent will be encountered during grading of the tank site. Hard bedrock that is encountered may require alternate methods of excavation such as jack hammering or blasting.

RECOMMENDATIONS

It is our expectation that relatively small amounts of dense bedrock that is non-rippable with a D9 or equivalent will be encountered during grading of the future tank site. Hard bedrock that is encountered may require alternate methods of excavation such as jack hammering or blasting. An economic decision should be made regarding the need for pre-blasting of the future tank site. This decision will depend in part upon the cost of pre-blasting the future tank site versus the cost of blasting or jack hammering the non-rippable rock that may be encountered during grading.

All conclusions and recommendations regarding the anticipated rippability of the new tank site included in our previous investigation (C.H.J., Inc., August 31, 2001) remain valid.

All recommendations included in our previous investigation (C.H.J., Inc., August 31, 2001) remain valid except for the following:

Temporary cut slopes in the colluvium/residual soil should be no steeper than 3/4(h) to 1(v) up to a maximum height of 20 feet.

CLOSURE

We trust that this report provides the requested subsurface information. If you should have any questions or require additional information, please do not hesitate to contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

Jay J. Martin, E.G. 1529
Senior Geologist

REGISTERED GEOLOGIST
JAY J. MARTIN
No. 1529
CERTIFIED ENGINEERING GEOLOGIST
3/31/02
STATE OF CALIFORNIA

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Senior Vice President

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No. 443
Exp. 3-31-05
GEOTECHNICAL
STATE OF CALIFORNIA

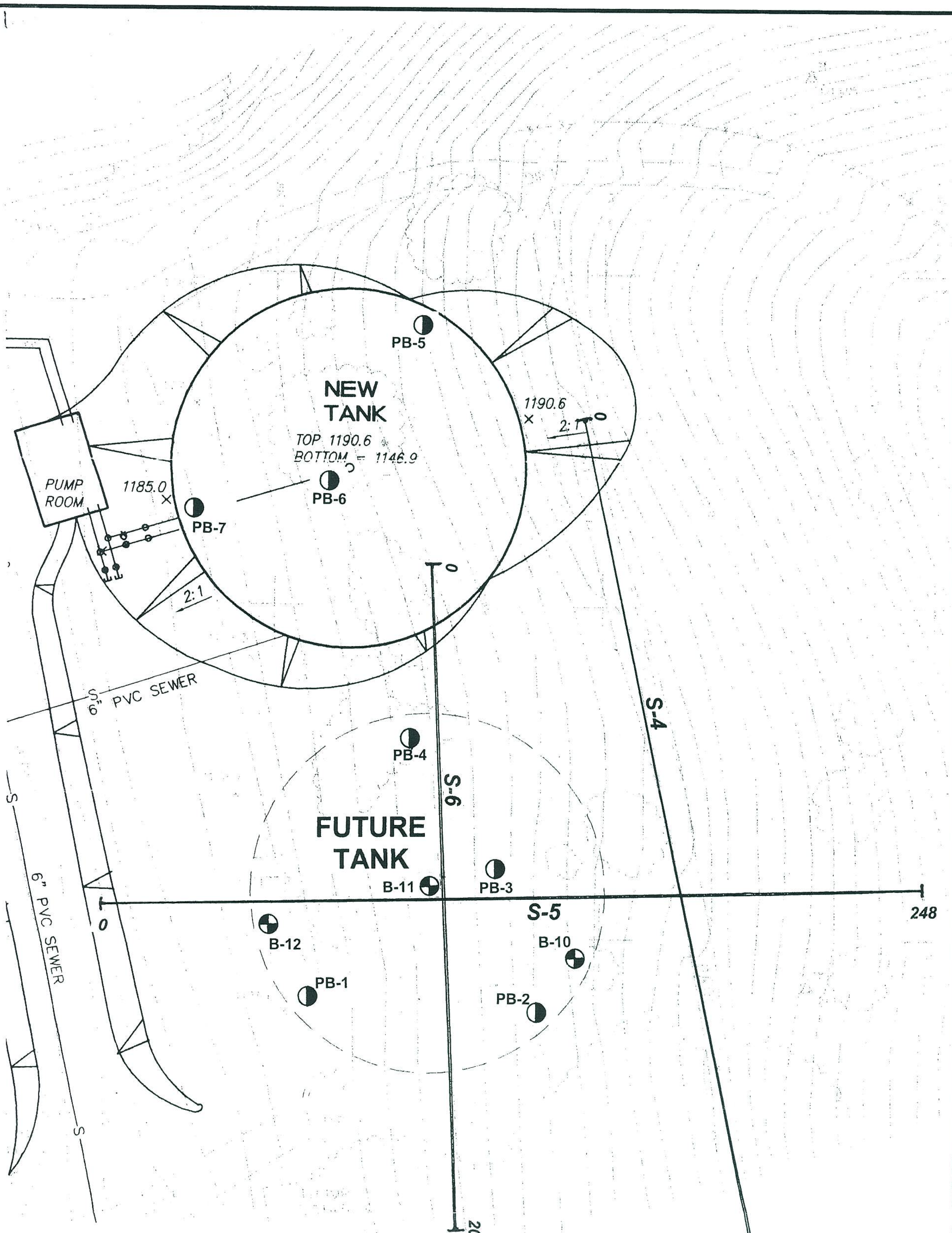
JJM/RJJ:jm/tlw

Enclosures: Appendix "A" - Geotechnical Map
Appendix "B" - Exploratory Boring Logs
Appendix "C" - Seismic Refraction Data and Interpretations

Distribution: University of California (6)

APPENDIX "A"

GEOTECHNICAL MAP



Legend

- B-12 Hollow stem auger borings
- PB-7 Percussion borings
- 0 S-6 203 Seismic refraction line

Scale 1"=30'

N

PLAT		
FOR: UNIVERSITY OF CALIFORNIA, RIVERSIDE	PROPOSED FUTURE TANK THERMAL ENERGY STORAGE EXPANSION UNIVERSITY OF CALIFORNIA, RIVERSIDE RIVERSIDE, CALIFORNIA	ENCLOSURE "A-1"
DATE: NOVEMBER 2001		JOB NUMBER 01736-3

C.H.J., INCORPORATED

APPENDIX "B"

LOGS OF EXPLORATORY BORINGS

Enclosure "B" (1of2)
Job No. 01736-3

KEY TO LOGS OF PERCUSSION BORINGS

Estimated Rippability vs. Penetration Rate*

For Ingersoll-Rand ECM 370 Drill - 3.5" Bit
(Modified from M.J. Baxter Drilling Company)

Characterization	Penetration Rate (sec/ft)	Estimated Rippability
Soft - Medium	10 - 18	Rippable
Medium - Hard	18-25	Probable Non-Rippable
Hard	25+	Non-Rippable
Very Hard	35	Blasting

* From historical data compiled over the years

Enclosure "B" (2of2)
Job No. 01736-3

SOIL CLASSIFICATION CHART

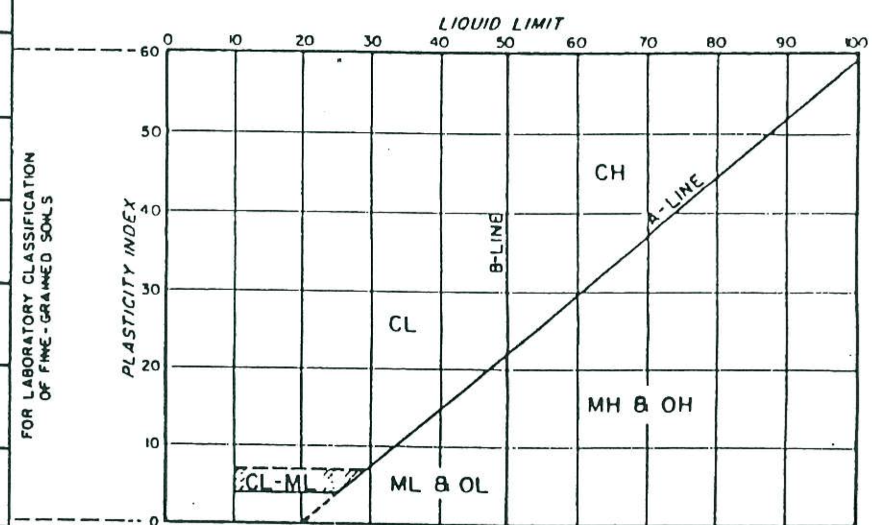
MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF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

GRADATION CHART

MATERIAL SIZE		PARTICLE SIZE			
		LOWER LIMIT		UPPER LIMIT	
		MILLIMETERS	SIEVE SIZE	MILLIMETERS	SIEVE SIZE
SAND	FINE	.074	#200 x	0.42	#40 x
	MEDIUM	0.42	#40 x	2.00	#10 x
	COARSE	2.00	#10 x	4.76	#4 x
GRAVEL	FINE	4.76	#4 x	19.1	3/4" •
	COARSE	19.1	3/4" •	76.2	3" •
COBBLES		76.2	3"	304.8	12"
BOULDERS		304.8	12	914.4	36"

x US STANDARD • CLEAR SQUARE OPENINGS

PLASTICITY CHART



UNIFIED SOIL CLASSIFICATION SYSTEM

C·H·J· INCORPORATED

LOG OF BORING 10

Date Drilled: 10/31/01

Client: University of California

Equipment: CME 55 Drill Rig

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,195±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0–8		(SM) Silty Sand, fine with medium, brown	Native						
8–12		(SM) Silty Sand, fine, red brown							
12–16		(SM) Silty Sand, fine to coarse with clay and gravel to 1/2", dark brown	Bedrock						
16–35		(SM) Silty Sand, fine to coarse with gravel to 1/3", brown							

Depth scale: 5, 10, 15, 20, 25, 30

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 11/8/01

C·H·J·

THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-1a

LOG OF BORING 10

Date Drilled: 10/31/01

Client: University of California

Equipment: CME 55 Drill Rig

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,195±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to coarse with gravel to 1/3", brown							
		END OF BORING							
40									
45									
50		BEDROCK AT 12.0' REFUSAL IN BEDROCK AT 36.5' NO FILL NO FREE GROUNDWATER							
55									
60									
65									

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING 11

Date Drilled: 10/31/01

Client: University of California

Equipment: CME 55 Drill Rig

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,186±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine with medium, light brown	Native						
3		(SM) Silty Sand, fine with clay, red brown							
11		(SM) Silty Sand, fine to medium with coarse and gravel to 1/3", dark brown	Bedrock						
16		(SM) Silty Sand, fine to coarse with gravel to 1/3", gray brown							

Depth scale: 5, 10, 15, 20, 25, 30

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING 11

Date Drilled: 10/31/01 | Client: University of California

Equipment: CME 55 Drill Rig | Driving Weight / Drop: N/A

Surface Elevation (ft): 1,186± | Logged by: R.M. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
35–57		(SM) Silty Sand, fine to coarse with gravel to 1/3", gray brown							
57		END OF BORING							
		BEDROCK AT 11.0' REFUSAL IN BEDROCK AT 57.0' NO FILL NO FREE GROUNDWATER							

Depth scale: 40, 45, 50, 55, 60, 65

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 11/8/01

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RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-2b

LOG OF BORING 12

Date Drilled: 10/31/01

Client: University of California

Equipment: CME 55 Drill Rig

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,179±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0–8		(SM) Silty Sand, fine with medium, red brown	Native						
8–15		(SM) Silty Sand, fine to medium with coarse and gravel to 1/3", brown	Bedrock						
15–35		(SM) Silty Sand, fine to coarse with gravel to 1/3", gray brown							

Depth scale: 5, 10, 15, 20, 25, 30

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 11/8/01

C·H·J·

THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-3a

LOG OF BORING 12

Date Drilled: 10/31/01

Client: University of California

Equipment: CME 55 Drill Rig

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,179±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to coarse with gravel to 1/3", gray brown							
40									
45									
50									
55									
		END OF BORING							
60		BEDROCK AT 8.0' REFUSAL IN BEDROCK AT 55.5' NO FILL NO FREE GROUNDWATER							
65									

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-1

Date Drilled: 11/6/01

Client: University of California

Equipment: Ingersoll-Rand 370 Percussion

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,179±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5									
						9.2			
9		(SM) Silty Sand, fine to coarse, gray	Bedrock						
10									
						10.4			
15									
						13.2			
20									
22		(SP-SM) Sand, fine to coarse with silt and gravel to 1/3", gray				10.2			
25									
						11.6			
30									
						12.6			

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-1

Date Drilled: 11/6/01
Client: University of California
Equipment: Ingersoll-Rand 370 Percussion
Driving Weight / Drop: N/A
Surface Elevation (ft): 1,179±
Logged by: R.M.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES: DRIVE	SAMPLES: BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP-SM) Sand, fine to coarse with silt and gravel to 1/3", gray				12.6			
40									
45									
50						13.6			
55									
		END OF BORING							
60		BEDROCK AT 9.0' NO REFUSAL NO FILL NO FREE GROUNDWATER							
65									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-2

Date Drilled: 11/6/01

Client: University of California

Equipment: Ingersol-Rand 370 Percussion

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,192±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5									
						11.2			
10									
		(SP-SM) Sand, fine to coarse with silt, gray	Bedrock			13.6			
15									
						13.0			
20									
25									
						10.4			
30									
						11.6			

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

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THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-5a

LOG OF BORING PB-2

Date Drilled: 11/6/01 | Client: University of California

Equipment: Ingersol-Rand 370 Percussion | Driving Weight / Drop: N/A

Surface Elevation (ft): 1,192± | Logged by: R.M. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP-SM) Sand, fine to coarse with silt, gray							
						12.2			
40									
						11.6			
45									
						13.2			
50									
						13.4			
55									
		END OF BORING							
60		BEDROCK AT 12.0' NO REFUSAL NO FILL NO FREE GROUNDWATER							
65									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-3

Date Drilled: 11/6/01

Client: University of California

Equipment: Ingersol-Rand 370 Percussion

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,189±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5						10.6			
10						13.6			
15						11.8			
18.5		(SP-SM) Sand, fine to coarse with silt, gray	Bedrock						
20									
24		END OF BORING							
25									
30		BEDROCK AT 18.5' NO REFUSAL NO FILL NO FREE GROUNDWATER							

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-4

Date Drilled: 11/6/01
Client: University of California
Equipment: Ingersol-Rand 370 Percussion
Driving Weight / Drop: N/A
Surface Elevation (ft): 1,183±
Logged by: R.M.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5						10.3			
10						10.2			
15		(SP) Sand, fine to coarse, gray	Bedrock			11.0			
20						16.2			
25						14.4			
30									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

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THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-7a

LOG OF BORING PB-4

Date Drilled: 11/6/01

Client: University of California

Equipment: Ingersol-Rand 370 Percussion

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,183±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP) Sand, fine to coarse, gray				15.6			
40						13.2			
45									
						13.6			
50									
						13.4			
55									
		END OF BORING							
60		BEDROCK AT 15.0' NO REFUSAL NO FILL NO FREE GROUNDWATER							
65									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

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THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-7b

LOG OF BORING PB-5

Date Drilled: 11/6/01 | Client: University of California

Equipment: Ingersol-Rand 370 Percussion | Driving Weight / Drop: N/A

Surface Elevation (ft): 1,192± | Logged by: R.M. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5						11.4			
7		(SP) Sand, fine to coarse, gray	Bedrock						
10									
12						14.2			
15									
17						9.4			
20									
25						12.6			
30						11.6			

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

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THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-8a

LOG OF BORING PB-5

Date Drilled: 11/6/01 | Client: University of California

Equipment: Ingersol-Rand 370 Percussion | Driving Weight / Drop: N/A

Surface Elevation (ft): 1,192± | Logged by: R.M. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP) Sand, fine to coarse, gray							
						11.8			
40									
						14.2			
45									
						11.8			
50									
						16.0			
55									
		END OF BORING BEDROCK AT 7.0' NO REFUSAL NO FILL NO FREE GROUNDWATER							
60									
65									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-6

Date Drilled: 11/6/01
Client: University of California
Equipment: Ingersol-Rand 370 Percussion
Driving Weight / Drop: N/A
Surface Elevation (ft): 1,184±
Logged by: R.M.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5						6.3			
10									
						12.2			
15									
						9.8			
		(SP) Sand, fine to coarse, gray	Bedrock						
20									
						11.2			
25									
						12.4			
30									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

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THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-9a

LOG OF BORING PB-6

Date Drilled: 11/6/01

Client: University of California

Equipment: Ingersol-Rand 370 Percussion

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,184±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP) Sand, fine to coarse, gray							
						12.0			
40									
						13.4			
45									
						13.6			
50									
						14.0			
55									
		END OF BORING							
60		BEDROCK AT 18.0' NO REFUSAL NO FILL NO FREE GROUNDWATER							
65									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-7

Date Drilled: 11/6/01

Client: University of California

Equipment: Ingersol-Rand 370 Percussion

Driving Weight / Drop: N/A

Surface Elevation (ft): 1,178±

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to coarse, brown	Native						
5									
10						9.8			
15						9.4			
16		(SP) Sand, fine to coarse, gray	Bedrock						
20						13.0			
25						19.0			
30						18.5			

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

LOG OF BORING PB-7

Date Drilled: 11/6/01

Equipment: Ingersol-Rand 370 Percussion

Surface Elevation (ft): 1,178±

Client: University of California

Driving Weight / Drop: N/A

Logged by: R.M.

Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	PENETRATION (Seconds/Foot)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP) Sand, fine to coarse, gray							
		END OF BORING							
40		BEDROCK AT 16.0' NO REFUSAL NO FILL NO FREE GROUNDWATER							
45									
50									
55									
60									
65									

BORING LOG W/PENETRATION DATA 01736-3.GPJ CHJ.GDT 11/8/01

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THERMAL ENERGY STORAGE TANKS
RIVERSIDE, CALIFORNIA

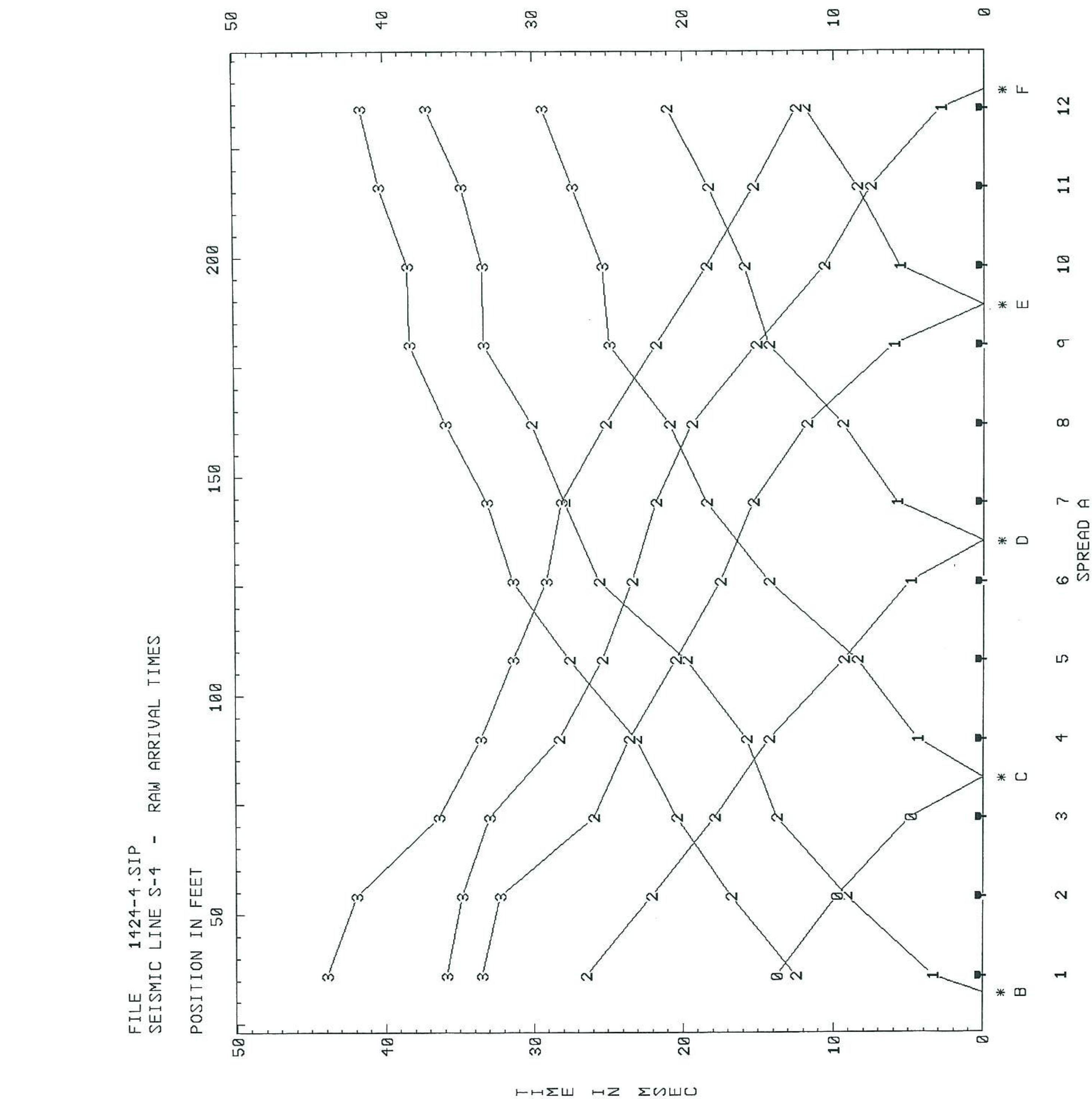
Job No. 01736-3

Enclosure B-10b

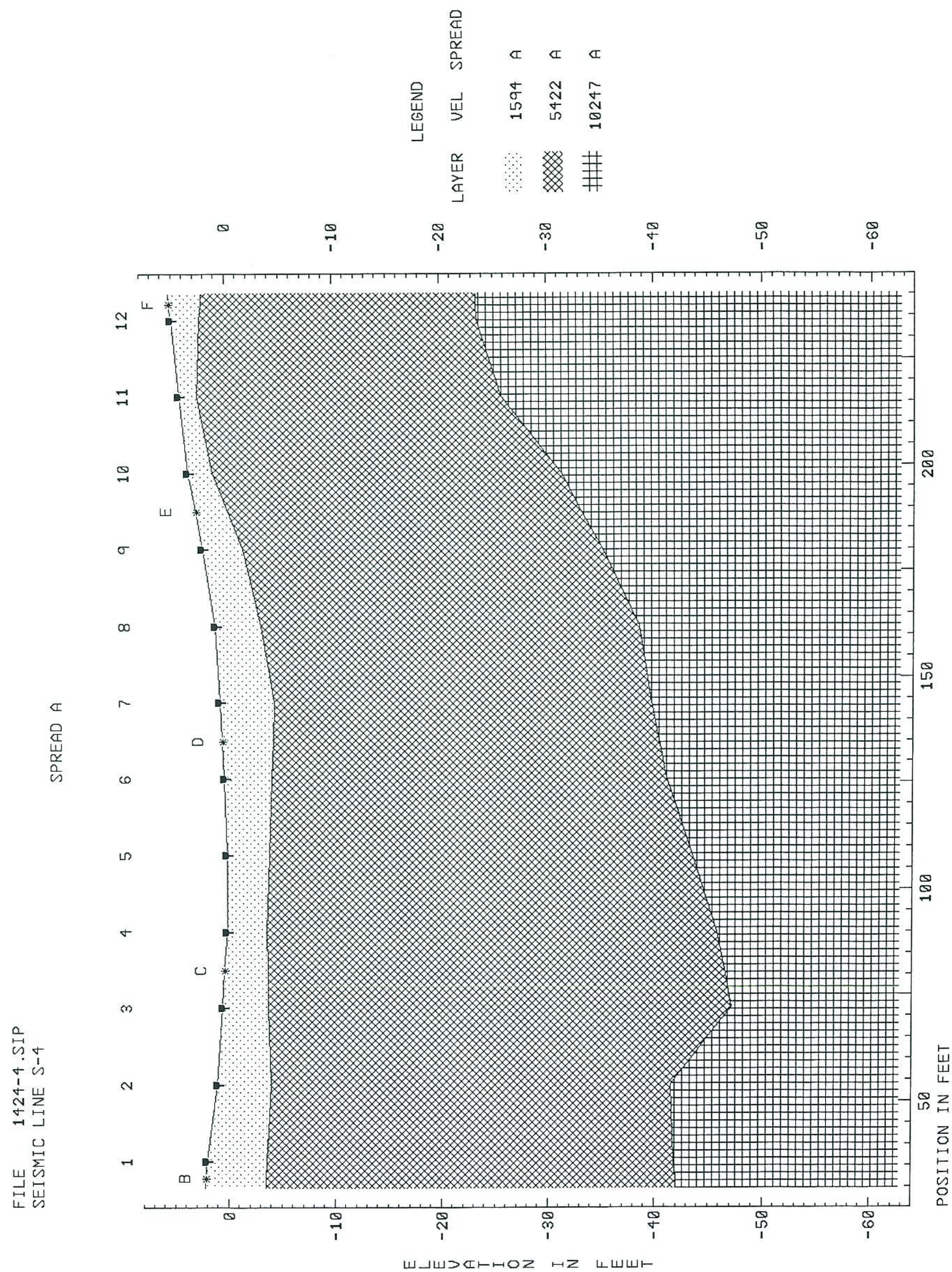
APPENDIX "C"

SEISMIC REFRACTION DATA AND INTERPRETATIONS

ENCLOSURE: "C-1"
JOB NO.:01736-3



ENCLOSURE: "C-2"
JOB NO.: 01736-3

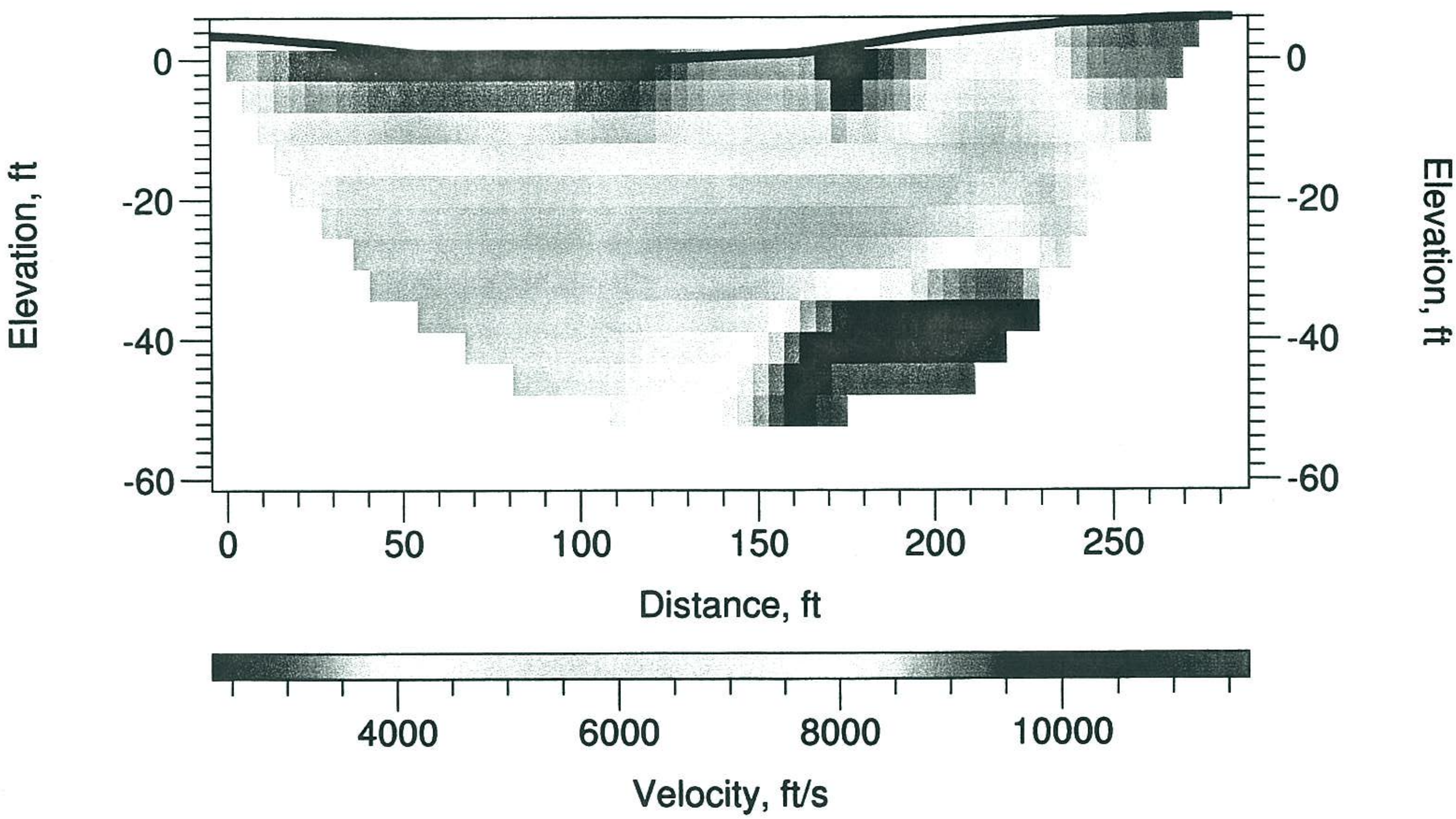


ENCLOSURE: "C-3"
JOB NO.: 01736-3

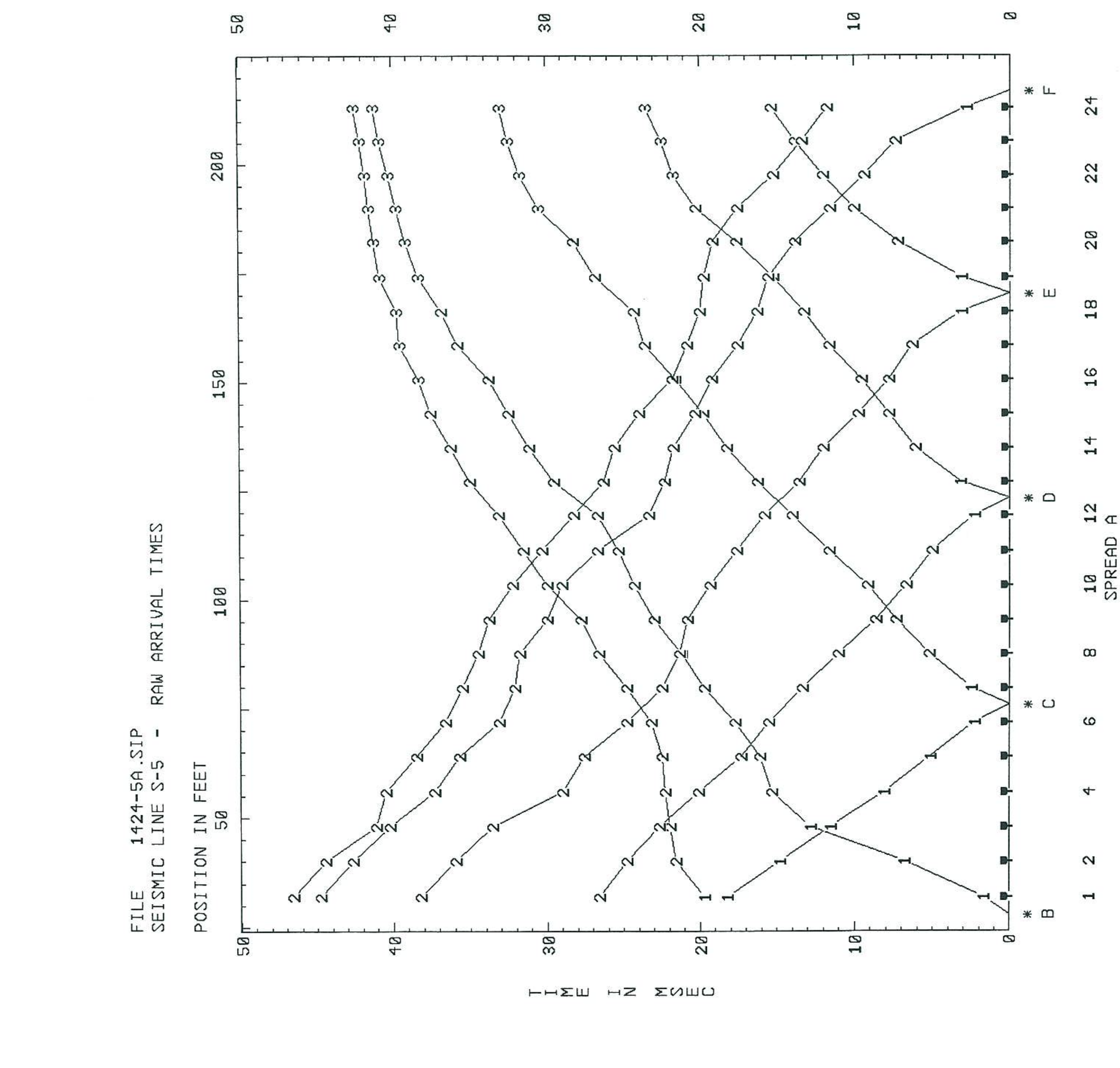
SEISMIC LINE S-4

South 78° West →

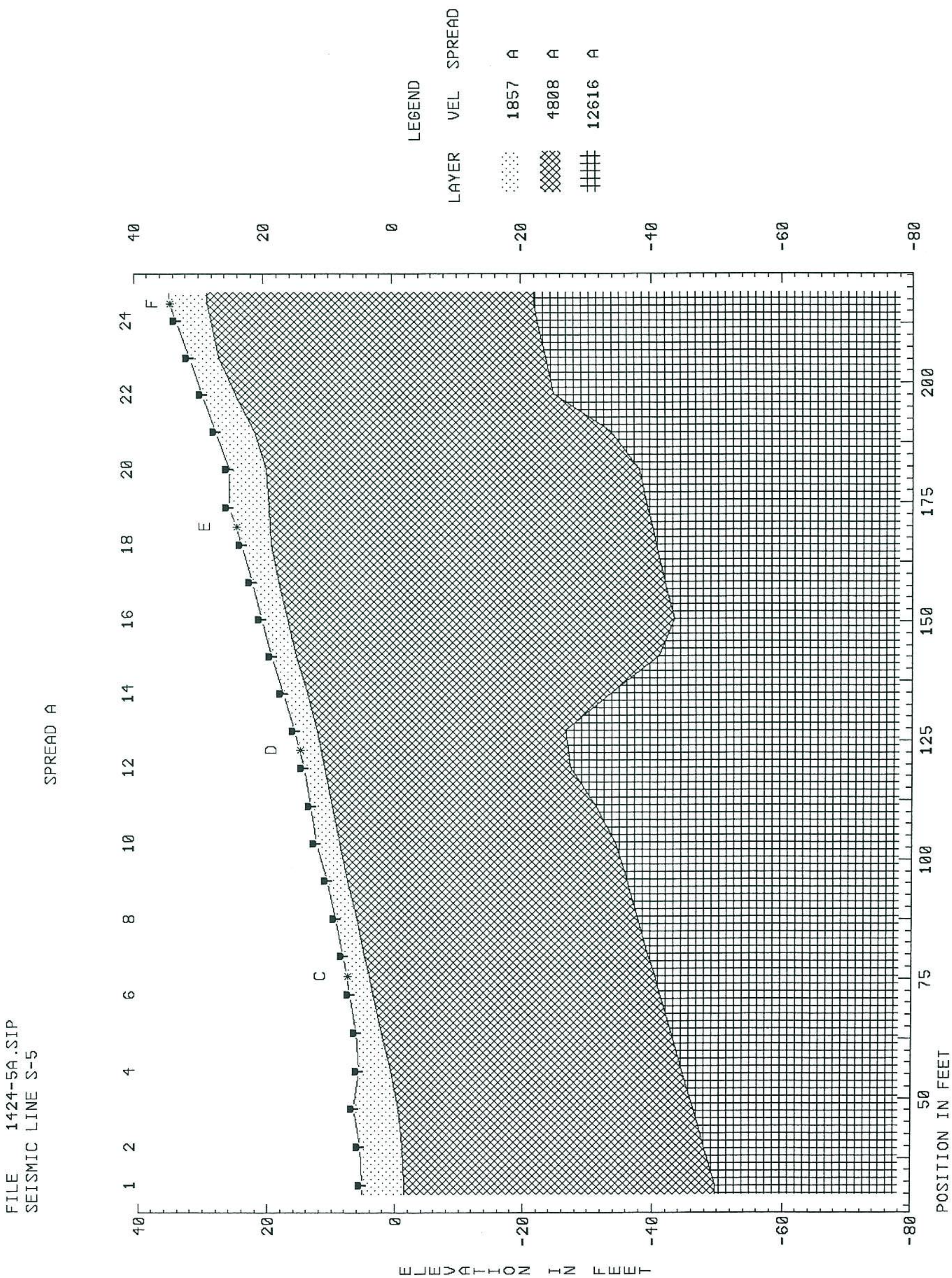
Velocity Gradient Model



ENCLOSURE: "C-4"
JOB NO.:01736-3



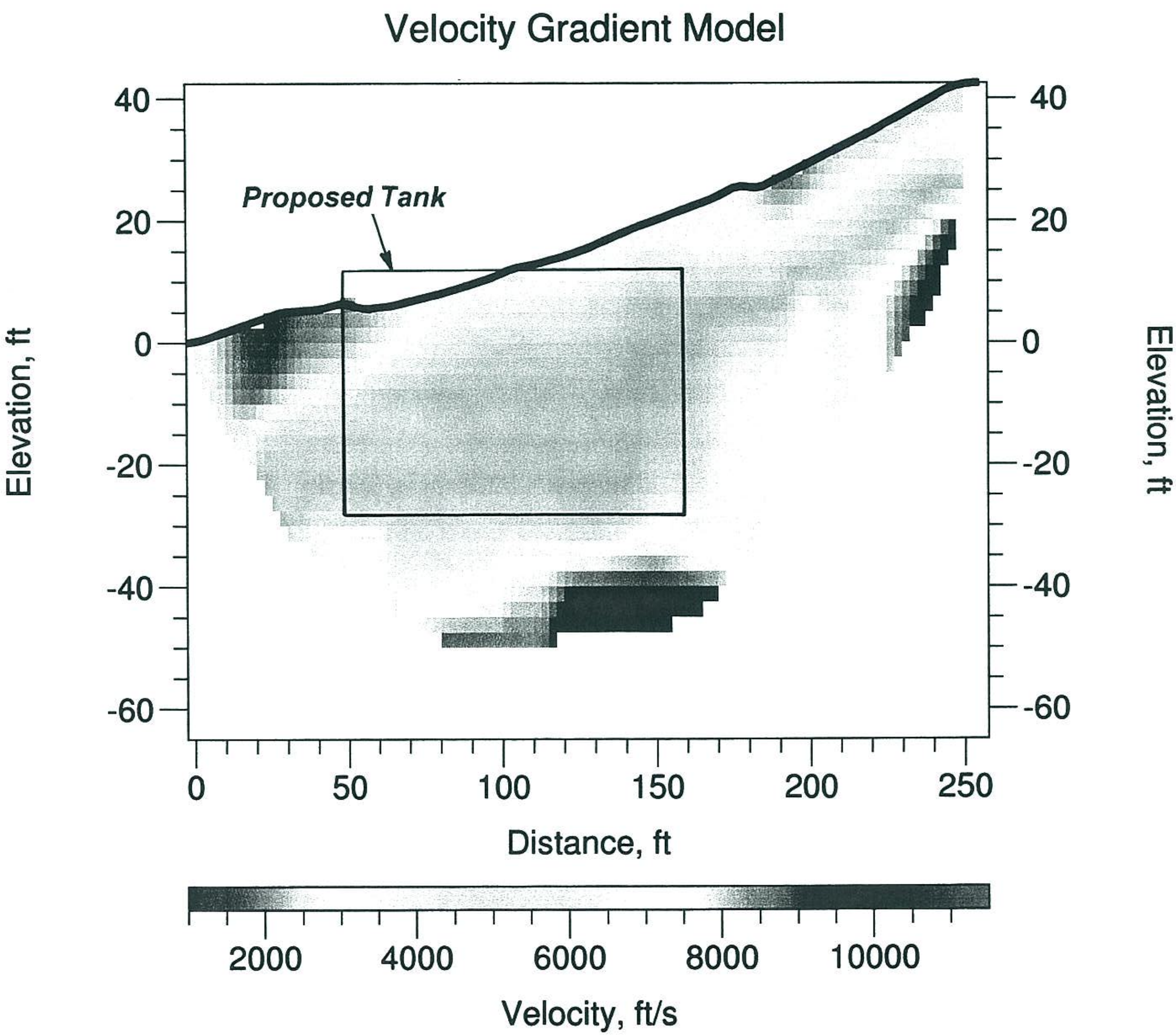
ENCLOSURE: "C-5"
JOB NO.: 01736-3



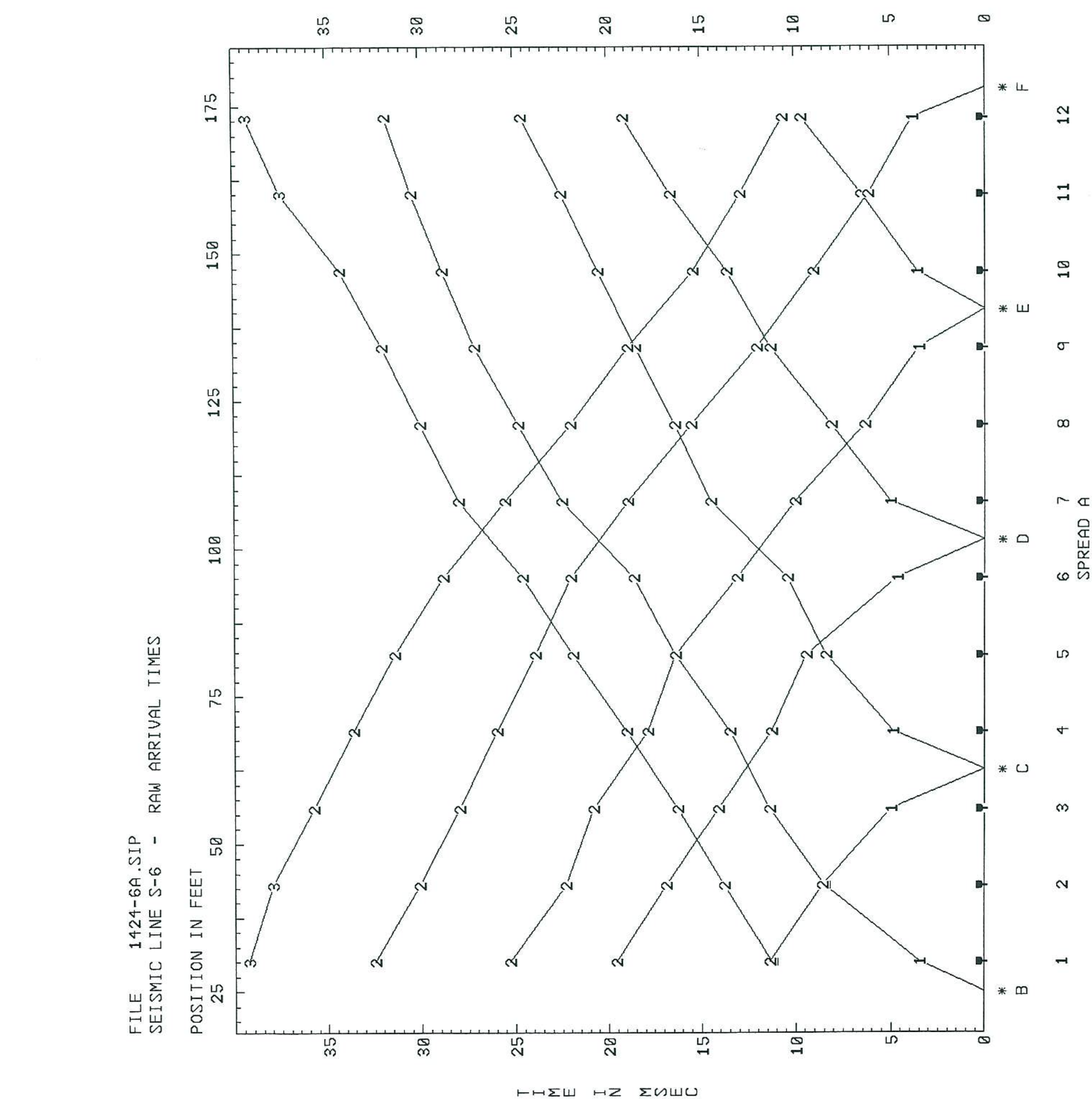
ENCLOSURE: "C-6"
JOB NO.: 01736-3

SEISMIC LINE S-5

← North - South →

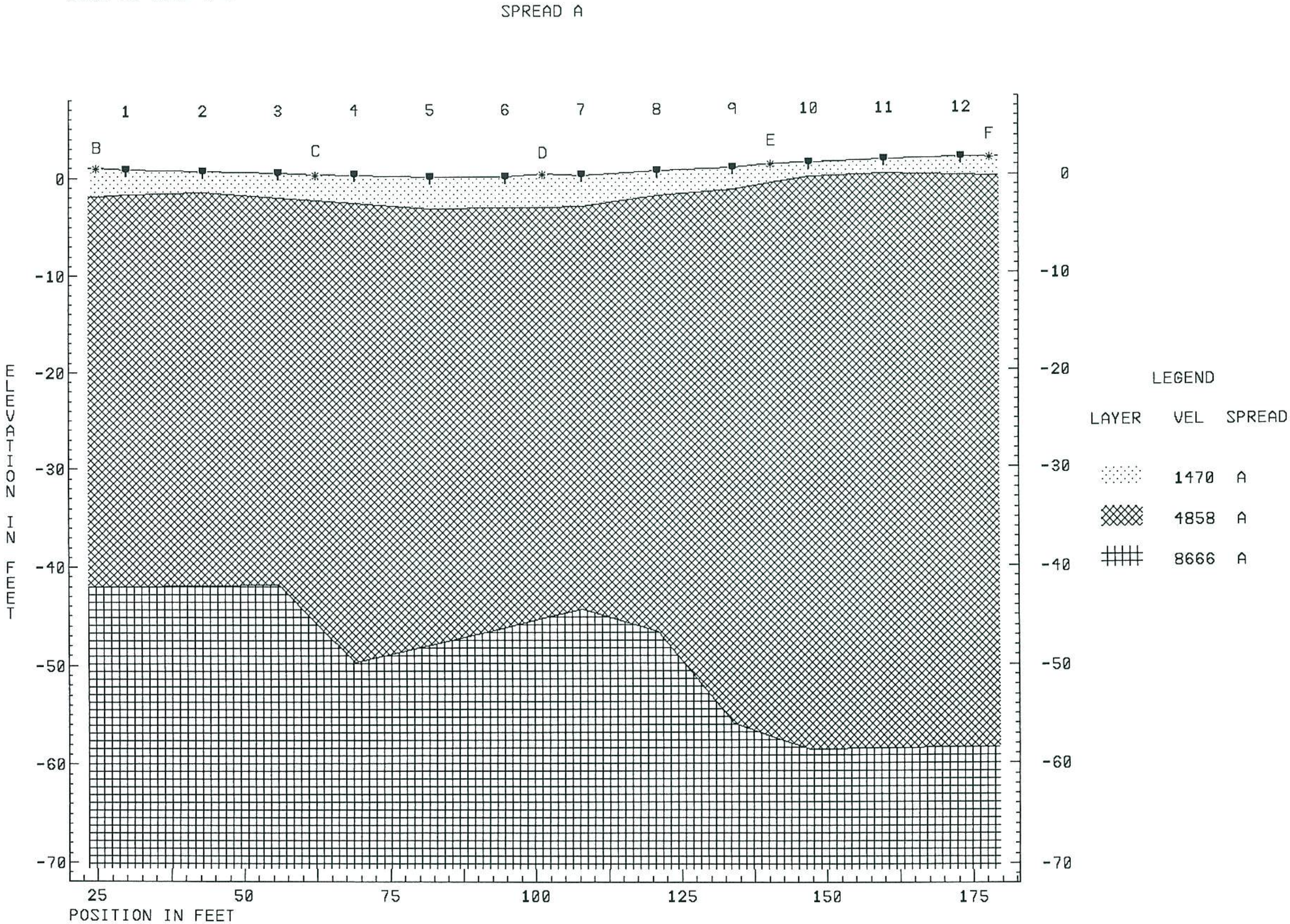


ENCLOSURE: "C-7"
JOB NO.:01736-3



ENCLOSURE: "C-8"
JOB NO.: 01736-3

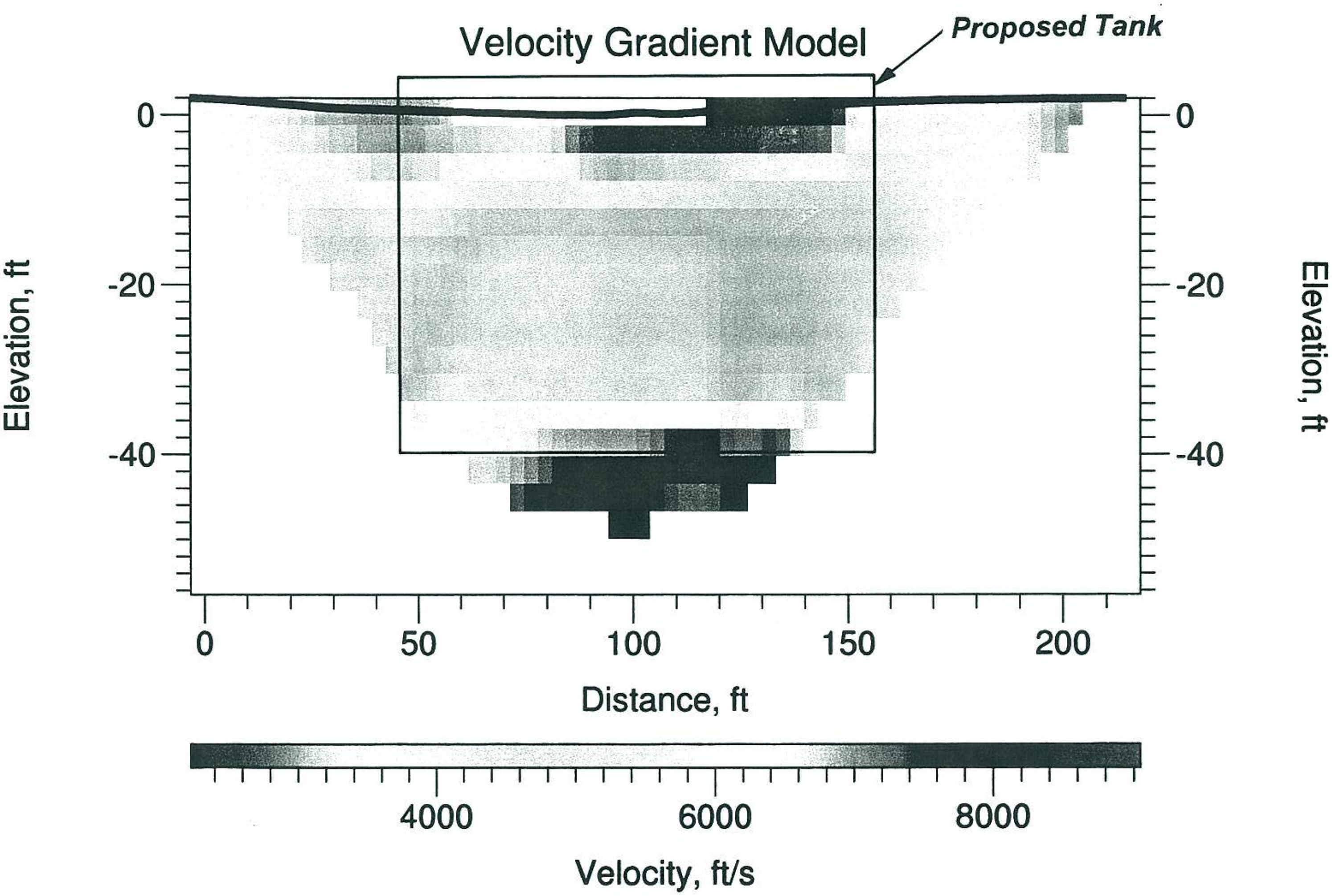
FILE 1424-6A.SIP
SEISMIC LINE S-6



ENCLOSURE: "C-9"
JOB NO.: 01736-3

SEISMIC LINE S-6

← East - West →



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GEOTECHNICAL INVESTIGATION
THERMAL ENERGY STORAGE EXPANSION AND SATELLITE PLANT
UNIVERSITY OF CALIFORNIA
RIVERSIDE, CALIFORNIA

PREPARED FOR
UNIVERSITY OF CALIFORNIA, RIVERSIDE
JOB NO. 01736-3

C·H·J· INCORPORATED

P.O. Box 231, Colton, CA 92324-0231 • 1355 E. Cooley Dr., Colton, CA 92324-3954 • Phone (909) 824-7210 • Fax (909) 824-7209

August 31, 2001

University of California, Riverside
Office of Design and Construction
3615A Canyon Crest Drive
Riverside, California 92521-0116
Attention: Ms. Tricia Thrasher

Job No. 01736-3

Dear Ms. Thrasher:

Attached herewith is the Geotechnical Investigation report, prepared for the proposed Thermal Energy Storage Expansion and Satellite Plant, to be constructed on the campus of the University of California in Riverside, California.

This report was based upon a scope of services generally outlined in our proposal letter dated August1, 2001, and other written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

Jay J. Martin, E.G.
Senior Geologist

TAD/TRW/JJM/RJJ:jjm/tlw

Distribution: University of California, Riverside (6)
Burkett & Wong (1)

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UNIVERSITY OF CALIFORNIA
RIVERSIDE, CALIFORNIA
PREPARED FOR
UNIVERSITY OF CALIFORNIA, RIVERSIDE
JOB NO. 01736-3

INTRODUCTION

During August of 2001, a geotechnical investigation was conducted for the proposed Thermal Energy Storage Expansion and Satellite Plant. The purpose of this investigation was to explore and evaluate the soils engineering/geologic conditions in the area of the proposed Thermal Energy Storage Expansion and Satellite Plant and to provide appropriate geologic and geotechnical engineering recommendations for design and construction of the proposed structures.

To orient our investigation, we were furnished with an electronic copy of a 50- scale Concept Design, dated August 3, 2001 and prepared by Bechard Long and Associates, Inc. The approximate location of the site is shown on the attached Index Map (Appendix "A").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this geotechnical investigation included the following:

- Review of published and unpublished literature and maps, and previous investigations conducted by C.H.J., Incorporated in the site vicinity
- Review and analysis of stereoscopic aerial photographs flown in 1931, 1957, 1974, 1990, 1995, and 2001 for past land use and geotechnical hazards
- Geologic reconnaissance to assist in characterization and interpretation of subsurface site conditions, identification of geologic hazards, and placement of exploratory borings
- Placement of nine exploratory borings on the site
- Logging and sampling of the exploratory borings for testing and evaluation

- Placement of 3 seismic refraction lines to quantitatively evaluate the rippability of the subsurface materials (bedrock) at the location of the proposed tank
- Laboratory testing on selected samples
- Evaluation of the geotechnical data to develop site-specific recommendations for site grading, foundation design, and mitigation of potential geotechnical constraints

PROJECT CONSIDERATIONS

It is our understanding that the proposed project will include construction of a 94-foot diameter reinforced concrete storage tank and a satellite plant consisting of two structures that will house the chillers, pumps, and cooling towers. It is anticipated that the Satellite Plant will utilize conventional spread foundations and slab-on-grade type construction.

The project grading plan was not available at the time of our investigation. However it was indicated that the bottom of the Thermal Energy Storage Expansion tank will be founded at elevation 1,145 feet.

SITE DESCRIPTION

The subject site consists of two separate areas on the campus of UCR. The area of the proposed Thermal Energy Storage Expansion site is located on the hillside east of East Campus Drive, south of Computing and Communication Services buildings and Parking Lot 9. Vacant hillside terrain is located south of the proposed tank. Several structures are located west of the proposed storage tank across East Campus Drive.

The site of the proposed Satellite Plant is located east of East Campus Drive and the Herbarium. Several modular structures, including the Archaeology Lab, were located on the site of the proposed satellite plant.

The area of the proposed Thermal Energy Storage Expansion is presently unoccupied. A partially buried steel pipe was observed trending east/west in the area of the proposed Thermal Energy Storage Expansion. It is anticipated that underground utilities exist in the area of the proposed structure.

Vegetation on the site of the proposed Thermal Energy Storage Expansion consisted of small trees, seasonal weeds, and grasses. Landscaped areas surrounded the existing structures on the proposed satellite plant. Vegetation in the area of the proposed Satellite Plant consisted of landscaped areas with trees and small shrubs.

The topography of the proposed storage tank consists of a north-northwest facing slope near the base of a low-lying bedrock hill to the south. The site is nearly planar, with a slope of approximately 15 percent to the north-northwest. The proposed satellite plant is located on a deeply dissected alluvial fan. Off site to the east, the alluvial apron has been dissected and eroded by a north- to south-trending, steep-walled ravine.

No other surface features pertinent to this investigation were noted.

FIELD INVESTIGATION

The soil conditions underlying the subject site were explored by means of nine exploratory borings drilled to a maximum depth of 50 feet below the existing ground surface with a truck-mounted drill rig equipped for soil sampling. Four of the borings were placed at the tank site and five at the proposed satellite plant. The approximate locations of our exploratory borings are indicated on the enclosed Plat (Appendix "A").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Relatively undisturbed samples were obtained by driving a split-spoon ring sampler ahead of the borings at selected levels. The number of hammer blows required to advance the sampler a total of 12 inches was converted to equivalent SPT N_{60} data and recorded on the boring logs. Undisturbed as well as bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our equivalent SPT N_{60} data, are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

LABORATORY INVESTIGATION

Included in our laboratory testing program were field moisture content determinations on all samples returned to the laboratory and field dry densities on all undisturbed samples. Optimum moisture content/maximum dry density relationships were established for typical soil types for relative compaction and recompaction evaluations. The results are included on the boring logs. An expansion index determination was performed as per Uniform Building Code (UBC) Standard Test Method 18-2 on a selected sample of clay-bearing soil. Samples of probable foundation subgrade soil were delivered to Del Mar Analytical Laboratory for soluble sulfate testing.

A direct shear test was performed on a selected sample for slope stability, bearing capacity and lateral earth pressure evaluations. A sand equivalent test was performed on a selected sandy sample for pipe bedding evaluations. Sieve analysis were conducted on selected samples for classification purposes. The laboratory test results are presented in Appendix "C".

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The site is located on the Perris Block, a portion of the Peninsular Ranges Geomorphic Province. The Perris Block is a fault-bounded region of relative tectonic stability, a mass of relatively high land composed of crystalline bedrock thinly and discontinuously mantled by sedimentary material (Woodford and others, 1971). A Geologic Index Map (Morton and Cox, 1994) is included as Enclosure "A-3".

The area of the proposed Thermal Energy Storage Expansion is located on a thin mantle of colluvial material overlying Cretaceous granitic rock. The area of the proposed Satellite Plant is located on an alluvial fan emanating from the Box Springs Mountains located south of the site. At the site, the alluvial fan is comprised of brown silty sands and sands. Published geologic mapping (Enclosure "A-3") shows the surficial materials at the Satellite Plant as older alluvium.

A north-trending, steep-sided, minor drainage tributary to the University Wash is present off site to the east.

Aerial photographs flown from 1957 to 1995 indicate that the surficial soils in the area of the proposed Thermal Energy Storage Expansion have been disturbed by plowing for past agricultural uses. Aerial photographs also show that fill materials have been placed in the northern portion of the proposed satellite plant associated with the previous development of the site.

Our exploratory borings indicate that colluvium and residual soils blanket the proposed Thermal Energy Storage Expansion to depths of approximately 8 to 14 feet. Fill was encountered in Boring No. 3 to a depth of 1 foot. Bedrock was encountered below those depths, and it is anticipated that bedrock will be encountered in significant cuts of the proposed Thermal Energy Storage Expansion during grading. The blanketing material is generally comprised of fine silty sands. Our equivalent Standard Penetration Test (SPT) and density data indicate that the colluvial/residual soils are loose to very dense, grading more dense with depth. The bedrock is in a very dense state.

Fill was encountered on the proposed satellite plant in Boring Nos. 6 and 7 to depths of 4.5 feet. The proposed satellite plant is underlain by older alluvial soils comprised of silty sands and sands with silt to depths ranging from 9.5 to 23.5 feet. Weathered granitic bedrock was encountered in all borings on the proposed satellite plant below the alluvium. Based on equivalent SPT and density data, the older alluvium encountered at the proposed Satellite Plant is in place in a loose to medium dense state.

Groundwater was not encountered within any of our exploratory borings to the maximum depths attained.

Refusal in granitic bedrock was experienced within exploratory Boring Nos. 1 through 4 of the proposed Thermal Energy Storage Expansion at depths ranging from approximately 21 to 25 feet below the existing ground surface. The rippability of the bedrock at the Thermal Energy Storage Expansion is addressed later in this report. Refusal was not experienced within exploratory Boring Nos. 5 through 9 placed within the site of the proposed Satellite Plant.

All borings experienced slight caving upon removal of the augers.

Our laboratory testing indicated that the soils tested have a "very low" expansion potential in accordance with UBC Standard Test Method 18-2.

Results of soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attack.

A more detailed description of the subsurface soil conditions encountered within our exploratory borings is presented on the attached boring logs (Appendix "B").

FAULTING

The tectonics of the Southern California area are dominated by the interaction of the North American Plate and the Pacific Plate, which are apparently sliding past each other in a transform motion. Although some of the motion may be accommodated by rotation of crustal blocks such as the western Transverse Ranges (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and to be accommodating most of the transform motion between the Pacific Plate and the North American Plate. However, some of the plate motion is apparently also partitioned out to the other northwest-trending, strike-slip faults that are thought to be related to the San Andreas system, such as the San Jacinto fault and the Elsinore fault. Local compressional or extensional strain resulting from the transform motion along this boundary is accommodated by left-lateral, reverse, and normal faults such as the Cucamonga fault, the Crafton Hills fault zone, and the blind thrust faults of the Los Angeles Basin (Matti and others, 1992; Morton and Matti, 1993).

The Box Springs fault is shown by Rogers (1966) as a buried trace beneath Pleistocene-age alluvium approximately 1/2 mile northeast of the site. Although this fault is readily visible as a bedrock feature southeast of the site, it is considered to be inactive.

The San Jacinto fault zone, a system of northwest-trending, right-lateral, strike-slip faults is present across the San Jacinto Valley and through the San Timoteo Badlands, approximately 5 miles northeast of the site. The San Jacinto fault is the closest known active fault to the proposed storage tank and is considered to be the most important fault to the site with respect to the hazard of seismic shaking. More large historic earthquakes have occurred on the San Jacinto fault than any other fault in Southern California (Working Group on California Earthquake Probabilities, 1988).

Based on the data of Matti and others (1992), the portion of the San Jacinto fault adjacent to the site may be accommodating much of the motion between the Pacific Plate and the North American Plate in this area. Matti and others (1992) suggest this motion is transferred to the San Andreas fault in the Cajon Pass region by "stepping over" to parallel fault strands which include the Glen Helen fault. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 43 percent (±17 percent) probability of a major earthquake on the San Jacinto Valley segment of the San Jacinto fault for the 30 year interval from 1994 to 2024.

The San Andreas fault zone is located along the southwest margin of the San Bernardino Mountains, approximately 14 miles northeast of the site. The toe of the mountain front in the San Bernardino area roughly demarcates the presently active trace of the San Andreas fault, which is characterized by youthful fault scarps, vegetational lineaments, springs, and offset drainages. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 28 percent (±13 percent) probability to a major earthquake occurring on the San Bernardino Mountains segment of the San Andreas fault between 1994 and 2024.

The southern margin of the San Gabriel Mountains is coincident with a series of east-west trending, predominantly reverse and thrust faults known as the Transverse Ranges frontal fault system. The San Fernando fault of this system ruptured during the 1971 magnitude (**M**) 6.7 San Fernando earthquake. The Cucamonga fault of this system is located at the base of the San Gabriel Mountains, approximately 16 miles northwest of the site. Evidence of recent activity on this fault includes fresh scarps, sag ponds, and disrupted Holocene alluvium (Dutcher and Garrett, 1963; Yerkes, 1985; Morton and Yerkes, 1987).

The Elsinore fault zone is present approximately 18 miles southwest of the site. The Elsinore fault zone is composed of multiple *en echelon* and diverging fault traces and splays into the Whittier and Chino faults to the north. Although a zone of overall right-lateral deformation consistent with the regional plate tectonics, traces of the Elsinore fault zone form the graben of the Elsinore and Temecula Valleys. Holocene surface rupture events have been documented for several principal strands of the Elsinore fault zone (Saul, 1978; Rockwell and others, 1986; Wills, 1988).

HISTORICAL EARTHQUAKES

A map of recorded earthquake epicenters is included as Enclosure "A-4". This map includes the Cal Tech database for earthquakes with magnitudes of 4.0 or greater from 1977 through 2000.

The San Jacinto fault is the most seismically active fault in Southern California, although it has no record of producing great events comparable to those that occurred on the San Andreas fault during the Fort Tejon earthquake of 1857 and the San Francisco earthquake of 1906 (Working Group on California Earthquake Probabilities, 1988). Between 1899 and 1990, seven earthquakes of **M** 6 .0 or greater have occurred along the San Jacinto fault. Two of these earthquakes, an estimated **M** 6.7 1 in 1899 and a **M** 6.8 in 1918, took place in the San Jacinto Valley, east of the site. Two others, an estimated **M** 6.5 in 1899 and a **M** 6.2 in 1923, took place in the San Bernardino Valley, north of the site (Working Group on California Earthquake Probabilities, 1988).

The only large historical earthquake that can definitely be attributed to the Elsinore fault was a **M** 6.0 event in 1910 in the Temescal Valley area. This event caused damage to structures from Corona to Wildomar (Weber, 1977). Since 1932, four **M** 4.0+ earthquakes have occurred along the Elsinore fault zone in the Santiago Peak area (Weber, 1977).

No large earthquakes have occurred on the San Bernardino Mountains segment of the San Andreas fault within the regional historical time frame. Using dendrochronological evidence, Jacoby and others (1987) inferred that a great earthquake on December 8, 1812 ruptured the northern reaches of this segment. Recent trenching studies have revealed evidence of rupture on the San Andreas fault at Wrightwood occurred within this time frame (Fumal and others, 1993). Comparison of rupture events at the Wrightwood and Pallett Creek, and analysis of reported intensities at the coastal missions, led Fumal and others (1993) to conclude that the December 8, 1812 event ruptured the San Bernardino Mountains segment of the San Andreas fault largely to the southeast of Wrightwood, possibly extending into the San Bernardino Valley.

Surface rupture occurred on the Mojave segment of the San Andreas fault in the great 1857 Fort Tejon earthquake. The Coachella Valley segment of the San Andreas fault was responsible for the 1948 **M** 6.5 earthquake in the Desert Hot Springs area and for the 1986 **M** 5.6 earthquake in the North Palm Springs area.

No significant historical earthquakes have been specifically attributed to the Box Springs fault or the Cucamonga fault in the general area of the site.

SEISMIC ANALYSIS

The precise relationship between magnitude and recurrence interval of large earthquakes for a given fault is not known due to the relatively short time span of recorded seismic activity. As a result, a number of assumptions must be made to quantify the ground shaking hazard at a particular site.

Seismic hazard evaluations can be conducted from both a probabilistic and a deterministic standpoint. The probabilistic method is prescribed by current codes and was utilized to estimate the seismic hazard to the site during this investigation.

PROBABILISTIC HAZARD ANALYSIS:

The probabilistic analysis of seismic hazard is a statistical analysis of seismicity of all known regional faults attenuated to a particular geographic location. The results of a probabilistic seismic hazard analysis are presented as the annual probability of exceedance of a given strong motion parameter for a particular exposure time (Johnson and others, 1992).

For this report, the probabilistic analysis computer program FRISKSP (Blake, 2000) was used to analyze the location of the site under the criteria for rock sites by Campbell (1997, 2000) in relation to seismogenic faults within a 62-mile (100 km) radius of the site. The fault database utilized is published by the California Division of Mines and Geology (Petersen and others, 1998). The FRISKSP program assumes that significant earthquakes occur on mappable faults and that the occurrence rate of earthquakes on a fault is proportional to the estimated slip rate of that fault. Potential earthquake magnitudes are correlated to expected fault rupture areas and the resultant maximum ground acceleration at the site is computed. From the summation of the accelerations from all the potential sources, the total average annual expected number of occurrences of an acceleration greater than each of the values requested is calculated (Blake, 2000). The resultant graph of probability of exceedance vs. acceleration (Enclosure "D-1") indicates that a peak ground acceleration of 0.52g has a 10 percent probability of exceedance in 50 years. This corresponds to the Design Basis Earthquake as defined in the California Building Code (1998) and has a statistical return period of 475 years.

SEISMIC ZONE:

Figure 16A-2 presented in the 1998 California Building Code (CBC) places the portion of Riverside County west of 115° 30', which includes the site, within Seismic Zone 4. A Seismic Zone Factor "Z" of 0.40 is assigned to Seismic Zone 4.

SOIL PROFILE TYPE:

Based on SPT blow counts and extrapolation of geologic data to a depth of 100 feet beow ground surface, the appropriate classification for the proposed storage tank is S_B, rock, and the proposed satellite plant is S_C, very dense soil and soft rock soil profile type.

NEAR-SOURCE EFFECTS:

The seismic hazard to this site is dominated by the adjacent San Jacinto fault. For the purpose of near-source effect evaluation, maps of near-source zones in California including a classification table for the faults involved were prepared by the California Division of Mines and Geology to be used with the 1997 UBC (International Conference of Building Officials, 1997). The adjacent San Jacinto segment of the San Jacinto fault is classified as a Type "B" fault by the California Division of Mines and Geology (Petersen and others, 1998). Due to the potential for cascading (multiple-segment rupture) of the San Jacinto fault, the appropriate classification for the San Jacinto fault is Type "A". The applicable near-source acceleration factor N_A, as defined in the 1997 UBC, is 1.08, and the near-source velocity factor N_V is 1.36.

RIPPABILITY

Refusal to drilling with a hollow-stem auger was encountered at the location of the Thermal Energy Storage Expansion tank at depths of approximately 21 to 25 feet below ground surface. Based on the proposed tank pad elevation of 1,145 feet, a cut of approximately 35 feet in depth can be expected at the north side of the proposed pad. Cut depths on the south side of the pad are expected to be approximately 55 feet deep. A seismic refraction survey was conducted to determine the seismic velocity profile of the subsurface rock and the expected depths of rippable materials with large excavation equipment.

Seismic refraction surveying is a method of geophysical exploration in which a seismic wave is generated at a fixed point and the travel time of that wave is recorded by audio detectors (geophones) placed at known distances from the source. The velocity of the seismic wave is then calculated from the wave arrival time at each geophone (Dobrin, 1976). The time-travel data is then utilized to calculate a profile of seismic velocity vs. depth.

The seismic velocity can be utilized to estimate the rippability of subsurface materials. A chart included in the Caterpillar Performance Handbook (1992) correlates seismic velocity of different rock types to rippability by a D8L or D9N bulldozers utilizing single- or multi-shank rippers. This chart indicates granitic rock, such as that which underlies the site, is rippable to a velocity up to 6,800 feet per second (fps). Granitic rock of between 6,800 and 8,000 fps is considered to be marginally rippable, and velocities of greater than 8,000 fps are considered to be non-rippable. Our experiences with similar sites in this area indicate the velocities in the Caterpillar Performance Handbook (1992) are approximately 1,500 fps too fast for reasonable production rates.

For this investigation, three seismic refraction lines were performed at the locations shown on Enclosure "A-2" (Lines S-1, S-2 and S-3). These lines were 225 to 260 feet in length, including offset end-shot points. A 16-pound sledgehammer was used as an energy source to produce the seismic waves, and twelve 14-Hz geophones (with 60 percent damping) were spaced at 15 to 20 foot intervals along the lines to detect both the direct and refracted waves. The seismic wave arrivals were recorded on a 12-channel Geometrics SmartSeis® model signal enhancement refraction/reflection seismograph. Field conditions were very noisy due to suspected electromechanical equipment operating in the area. Three to five shot points were utilized along each seismic line spread using offset, forward, reverse, and intermediate locations in order to obtain sufficient data for velocity analysis and depth modeling purposes. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrival times. These seismic wave arrival times are digitally recorded on the in-board seismograph computer and subsequently transferred to a disk. The data disk was then downloaded into our office computer for further processing, analyzing, and printing.

The data on the paper record and/or display screen were used to analyze the arrival time of the seismic waves at each geophone station in the form of a wiggle trace, or wave travel-time curve, for quality control purposes in the field. All of the recorded data was transferred to our office computer for further processing, analyzing, and printing purposes using the computer programs **SIP** (**S**eismic refraction **I**nterpretation **P**rogram), developed by Rimrock Geophysics (1995), and **SeisOpt™@2D** (Optim, 2000). **SIP** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on the time-distance graphs and is better suited for layered media. In addition, the computer program **SeisOpt™@2D** was also used for comparative purposes as it models the subsurface velocity gradient in discrete velocity "cells". This program is an automatic refraction interpretation package that performs velocity model optimization and visualization using repeated forward modeling. Test velocity models are created, through which travel times are calculated and are compared with the observed data, and are optimized. The optimal solution is the velocity model with the minimum travel time error between the calculated and observed data. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

Line S-1 was performed along the road along the top (south) margin of the proposed pad. This is the area of anticipated deepest cut. The time-distance data for Line S-1 are included as Enclosure "E-1". A three-layer velocity vs. depth profile is included as Enclosure "E-2". A velocity gradient model is included as Enclosure "E-3". The velocity vs. depth profile (Enclosure "E-2") shows the depth to very hard (non-rippable) bedrock (9,400 fps) with a D-9 or equivalent as approximately 30 feet (west end) to approximately 60 feet (east end). The velocity gradient model (Enclosure "E-3") shows a depth of approximately 30 feet to hard bedrock (6,000 fps) that is expected to be marginally rippable to non-rippable.

Line S-2 was performed perpendicular to S-1 through the center of the proposed tank. The time-distance data for Line S-2 are included as Enclosure "E-4". A three-layer velocity vs. depth profile is included as Enclosure "E-5". A velocity gradient model is included as Enclosure "E-6". The velocity vs. depth profile (Enclosure "E-4") shows the depth to very hard (non-rippable) bedrock (10,200 fps) with a D9 or equivalent as approximately 20 feet (south end) to approximately 80 feet (north end). The velocity gradient model (Enclosure "E-6") shows a depth of approximately 20 to 30 feet to hard bedrock (6,000 fps) that is expected to be marginally rippable to non-rippable.

Line S-3 was performed parallel to S-1 through the center of the proposed tank. The time-distance data for Line S-3 are included as Enclosure "E-7". A three-layer velocity vs. depth profile is included as Enclosure "E-8". A velocity gradient model is included as Enclosure "E-9". The velocity vs. depth profile (Enclosure "E-8") shows the depth to very hard (non-rippable) bedrock (9,600 fps) with a D-9 or equivalent as at least 40 feet. The velocity gradient model (Enclosure "E-9") shows a depth of approximately 25 feet (west end) and approximately 35 feet (east end) to hard bedrock (6,000 fps) that is expected to be marginally rippable to non-rippable.

The velocity profiles generated during this investigation vary in part due to a gradual increase in velocity with depth observed in all of the lines. In this case, the velocity gradient models are expected to be more accurate in that they do not depend on a sharp contrast in layer velocities. Based on the data that were collected and interpreted, it is our expectation that the depth to marginally rippable to non-rippable rock (6,000 to 7,000 fps) is relatively shallow beneath most of the proposed tank site (20 to 30 feet below ground surface). It should be noted that the seismic refraction method, under ideal conditions, is accurate to within about 10 percent. Due to the noisy field conditions encountered, the expected velocities and depths should be considered approximate only. The results of the seismic refraction survey suggest that dense bedrock that is non-rippable with a D9 or equivalent may be encountered during grading of the tank site. Hard bedrock that is encountered may require alternate methods of excavation such as jack hammering or blasting.

No outcrops or core-stones are present at the ground surface at the proposed tank site. However, hard core-stones are visible in the hillside southwest of the site, and may exist in the subsurface in the vicinity of the proposed tank. These core-stones, if encountered, may be non-rippable and may also require alternate methods of excavation.

GROUNDWATER AND LIQUEFACTION

The site is underlain by crystalline bedrock at a relatively shallow depth which is considered to be essentially non-water bearing. However, the bedrock is overlain by more permeable alluvial/colluvial soils, a condition conducive to localized perching of water at the soil/bedrock interface. Application of landscape water on site can be expected to aggravate this condition. Given the geomorphology of the site, it is likely that the soil/bedrock interface is inclined too steeply to perch significant amounts of water. However, landscape water application should be limited to the amount actually necessary for sustained plant growth.

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid (Matti and Carson, 1991). Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (less than 50 feet in depth; 2) presence of unconsolidated sandy alluvium, typically Holocene in age; and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur. Based upon the data reviewed during this evaluation, only one of the three geologic conditions for increased liquefaction susceptibility (strong ground shaking) is expected to exist on the site. Therefore, liquefaction is not considered to be a potential hazard to the site.

FLOODING AND EROSION

No evidence of significant flooding of the site was observed during our geologic field reconnaissance or on the aerial photographs reviewed.

On-site materials are susceptible to erosion by running water. Finish graded areas should be protected from the effects of runoff.

SLOPE STABILITY

Data from the exploratory borings revealed a relatively thick (approximately 8 to 14 feet) accumulation of colluvium/residual soil overlying the bedrock at the site of the proposed Thermal Energy Storage Expansion. Thickened soil profiles are commonly associated with slope failures, resulting from increased rates of erosion of unstable soils. Possible evidence for landsliding was observed on the hillside as a concave slope. However, other north-facing hillsides in the area show a similar concave morphology, which suggests that the concave surface is a result of weathering processes and not landsliding. The bedrock at the site is generally massive and dense, with no out-of-slope structures developed that could result in increased susceptibility to landsliding. Based on geomorphology observed during the geologic field reconnaissance and the aerial photographs reviewed, the potential for existing landsliding on the site is considered to be very low. However, geologic in-grading observations should be conducted by the Engineering Geologist.

Rounded bedrock boulders were observed on the hillsides in the vicinity of the site. These boulders may be potentially unstable, particularly in the event of a large earthquake. Based on potential rockfall paths to the site, the potential hazard from rock falls to the site is negligible.

CONCLUSIONS

On the basis of our field and laboratory investigations, it is the opinion of this firm that the proposed development is feasible from geotechnical standpoint, provided the recommendations contained in this report are implemented during design, grading, and construction.

Based upon our field investigation and test data, it is our opinion that the existing soils will not, in their present condition, provide uniform or adequate support for the proposed Satellite Plant. Our equivalent SPT and density data indicate that the surficial fills overly loose to medium dense soils in the northern portion of the proposed Satellite Plant while the south portion is underlain by dense older alluvium. These conditions may cause unacceptable differential and/or overall settlement upon application of the anticipated foundation loads.

To provide adequate and uniform support for the proposed Satellite Plant, the existing fills and unsuitable, native soils will need to be removed and that the building pad area be further subexcavated as necessary and recompacted to provide a uniform compacted fill mat beneath footings and slabs. A compacted fill mat will provide a dense, uniform, high-strength soil layer to distribute the foundation loads over the underlying soils. Conventional spread foundations, either individual spread footings and/or continuous wall footings, may be utilized in conjunction with the compacted fill mat.

Based upon our exploratory boring and seismic refraction data, and proposed tank pad grades, excavation for the proposed Thermal Energy Storage Expansion is expected to encounter marginally rippable to non-rippable rock (6,000 to 7,000 fps). Hard bedrock that is encountered may require alternate methods of excavation such as jack hammering or blasting. These methods may not be feasible on campus due to the presence of people and sensitive equipment.

No evidence of active faulting on or immediately adjacent to the site was observed during the geologic field reconnaissance or on the aerial photographs reviewed.

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed structures.

No evidence for landsliding was observed at the location of the proposed Satellite Plant. Based on geomorphology observed during the geologic field reconnaissance and the aerial photographs reviewed, the potential for existing landsliding on the tank site is considered to be very low. The potential for landsliding at the tank site should be addressed by the Engineering Geologist during geologic in-grading observations.

Rounded bedrock boulders were observed on the hillsides in the vicinity of the site. These boulders may be potentially unstable, particularly in the event of a large earthquake. Based on potential rockfall paths to the site, the potential hazard from rock falls to the site is negligible.

No evidence of recent flooding of the site or surrounding area was observed.

Shallow bedrock at the site presents a possibility of localized perched groundwater conditions and seepage occurring at soil-bedrock interfaces exposed on slope faces.

The lack of significant static groundwater and the presence of granitic bedrock precludes liquefaction as a hazard at the site.

RECOMMENDATIONS

SEISMIC DESIGN CONSIDERATIONS:

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed structures. Therefore, the proposed structures should be designed accordingly.

Figure 16A-2 presented in the 1998 California Building Code (CBC) places the portion of Riverside County west of 115° 30', which includes the site, within Seismic Zone 4. A Seismic Zone Factor "Z" of 0.40 is assigned to Seismic Zone 4.

The appropriate site classification for the proposed Thermal Energy Storage Expansion is S_B, rock, and the proposed Satellite Plant is S_C, very dense soil and soft rock soil profile type.

The site is subject to a near-source acceleration factor N_A of 1.08 and a near-source velocity factor N_V of 1.36, as defined in the 1997 UBC.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the soils engineer. An on-site, pre-job meeting with the owner, the contractor, and the soils engineer should occur prior to all grading-related operations. Operations undertaken at the site without the soils engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the UBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped of significant vegetation and other deleterious materials. These materials should be removed from the site for disposal.

Any existing uncontrolled fill or pockets of loose disturbed soils encountered during construction should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. Any roots or other deleterious materials encountered at this time should be removed prior to replacing the soil.

Cavities created by removal of subsurface obstructions, such as irrigation lines, should be thoroughly cleaned of loose soil, organic matter, and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for site fill.

A minimum removal of at least the upper three feet of existing soils be removed from the Satellite Plant site. The open excavation should be observed by the Engineering Geologist to determine areas with existing fills and unsuitable soils requiring removal. Based upon data obtained from our exploratory borings it appears at this time that as a minimum the upper 9 ± feet of soils will need to be removed from the building pad area to a distance of 5 feet laterally beyond the building lines at the bottom of the excavation. The actual depth of removal should be determined at the time of grading by the Engineering Geologist who should observe and approve all subexcavation bottoms prior to fill placement. The southern portion of the Satellite Plant building pad areas should also be subexcavated to minimize differential fill depths.

Following approval of the bottom, subexcavated areas should be scarified to a depth of approximately 12 inches. The scarified soils should then be brought to near optimum moisture content and recompacted to a relative compaction of at least 90 percent prior to refilling the subexcavation to grade as properly compacted fill.

PREPARATION OF FILL AREAS:

Prior to placing fill, and after the open subexcavation have been observed by the Engineering Geologist, the surfaces of all areas to receive fill should be scarified to a depth of approximately 12 inches. The scarified soils should be brought to near optimum moisture content and recompacted to a relative compaction of at least 90 percent (ASTM D 1557-91).

PREPARATION OF FOOTING AREAS:

All footings for the Satellite Plant structures should rest upon a minimum of 18 inches of properly compacted fill material. In areas where the required thickness of compacted fill is not accomplished by the mandatory removals or additional removals, the footing areas should be further subexcavated to provide both the minimum recommended compacted fill thickness below the footings base grade and to limit the differential fill depth below the structure to less than ten feet with the subexcavation extending at least 5 feet beyond the footing lines. The bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to near optimum moisture content and recompacted to at least 90 percent relative compaction (ASTM D 1557-91) prior to refilling the excavation to grade as properly compacted fill.

The proposed Thermal Energy Storage Expansion may bear directly into observed and approved bedrock.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material provided they are free from roots, other organic matter, and deleterious materials. Unless approved by the soils engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in fills.

Import fill, if required, should be inorganic, non-expansive granular soils free from rocks or lumps greater than 6 inches in maximum dimension. Sources for import fill should be observed and approved by the soils engineer prior to their use.

Fill should be spread in near-horizontal layers, approximately 8 inches in thickness. Thicker lifts may be approved by the soils engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift shall be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum moisture content and 3 percent above, and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D 1557-91.

To minimize settlement potential fills over ten feet in depth should be compacted to at least 95% relative compaction and at least 2% over optimum moisture (ASTM D 1557)

Based upon the relative compaction of the soils determined during this investigation and the relative compaction anticipated for compacted fill soils, we estimate an average compaction shrinkage of approximately 5 to 10 percent. Therefore, 1.05 cubic yards to 1.10 cubic yards of in-place soil material would be necessary to yield one cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.1 feet. These values are exclusive of losses due to stripping, or the removal of other subsurface obstructions, if encountered, and may vary due to differing conditions within the project boundaries and the limitations of this investigation.

The deeper bedrock excavation may slightly bulk or may generate considerable quantities of oversize material requiring special handling.

Values presented for shrinkage and subsidence are estimates only. Final grades should be adjusted, and/or contingency plans to import or export material should be made to accommodate possible variations in actual quantities during site grading.

EXPANSIVE SOILS:

Since the clayey materials tested from this site exhibited a "very low" potential for expansion in accordance with UBC Standard Test Method 18-2. Specialized construction procedures, specifically to resist expansive soil forces are not anticipated at this time. Requirements for reinforcing steel to satisfy structural criteria are not affected by this recommendation. Additional evaluation of soils for expansion potential should be conducted by the soils engineer during the grading operation.

FOUNDATION DESIGN:

If the site is prepared as recommended, the proposed Satellite Plant structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 18 inches of compacted fill. Footings should be a minimum of 12 inches wide and should be established at a minimum depth of 12 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum safe soil-bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads. This allowable bearing pressure may be increased by 300 psf for each additional foot of width and by 700 psf for each additional foot of depth to a maximum safe soil bearing pressure of 4,500 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading.

The proposed Thermal Energy Storage Expansion will be deeply founded entirely within massive crystalline bedrock. Based upon UBC values, a maximum allowable foundation pressure of 12,000 psf may be utilized.

For footings thus designed and constructed, we would anticipate a maximum settlement of less than 1/2 inch. Differential settlement between similarly loaded adjacent footings is expected to be approximately half the total settlement.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 350 psf per foot of depth. Base friction may be computed at 0.35 times the normal load. Base friction and passive earth pressure may be combined without reduction.

For preliminary retaining wall design purposes, a lateral active earth pressure developed at a rate of 40 psf per foot of depth should be utilized for unrestrained conditions. This value should be verified prior to construction when the backfill materials and conditions have been determined and is applicable only to level, properly drained backfill with no additional surcharge loadings. If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters.

For the Thermal Energy Storage Expansion founded in bedrock, the lateral bearing of 1,200 psf per foot of embedment in bedrock and the sliding coefficient of 0.70 provided in the UBC may be utilized. For braced earth-retaining structures, such as the Thermal Energy Storage Expansion, "at rest" lateral earth pressures of 60 and 80 psf per foot of depth may be utilized for level and 3(h) to 1(v) backfills, respectively.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill. A lean sand cement slurry may be considered to backfill void spaces or narrow areas difficult to properly compact.

SLABS-ON-GRADE:

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil. The soil should be compacted to 90 percent relative compaction (ASTM D 1557). The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane will reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete.

FILL SLOPE CONSTRUCTION:

Fill slopes should be constructed in accordance with current Uniform Building Code requirements in regard to benching and drainage and should be constructed no steeper than 2(h) to 1(v). Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slopes to provide dense, erosion-resistant surfaces.

CUT SLOPE CONSTRUCTION:

The proposed tank may require permanent cut slopes up to approximately 60 feet in maximum height. Cut slopes in the colluvium/residual soil should be no steeper than 2(h) to 1(v). Cut slopes in the bedrock should be no steeper than 1 (h) to 1(v) up to maximum height of 50 feet. Cut slopes higher than 30 feet should be evaluated by the Engineering Geologist and Geotechnical Engineer prior to and during construction.

For cut slopes higher than 30 feet, UBC terraces are not expected to be needed. However, a concrete paved interceptor drain should be provided above the top of the cut slope for the tank to protect the upper soils from erosion.

TEMPORARY SLOPE CONSTRUCTION:

Temporary cut slopes may be needed during grading of the tank. The cooluvium/residual soil is generally classified as "Type B" soils according to CAL/OSHA (1993). Accordingly, the steepest inclination allowed by CAL/OSHA (1993) for simple temporary slopes up to 20 feet in height in "Type B" soils is 1(h) to 1(v).

The bedrock is expected to be classified as "Stable Rock" according to CAL/OSHA. Since temporary slopes in the bedrock are expected to be higher than 20 feet, the maximum inclination of temporary slopes in the bedrock should be 1/2(h) to 1(v). Temporary slopes may need to be scaled to remove potentially unstable rocks generated during grading. All temporary slopes should be observed by the Engineering Geologist during grading.

POTENTIAL EROSION:

The potential for erosion should be mitigated by proper drainage design. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion. Graded areas should be planted or otherwise protected from erosion by wind or water.

SOLUBLE SULFATES:

Results of the soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attack, as per Table 4.3.1 of the American Concrete Institute Manual of Concrete Practice (2000). No special measures, such as specific cement types, water-cement ratios, etc., will be needed for this "negligible" exposure to sulfate attack. Additional soluble sulfate testing should be conducted during construction on the actual soils encountered.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the soils engineer. The presence of the soils engineer's field representative will be for the purpose of providing observation and field testing, and will not include any supervising or directing of the actual work of the contractor, his employees, or agents. Neither the presence of the soils engineer's field representative nor the observations and testing by the soils engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the soils engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

C.H.J., Incorporated has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable soils engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the proposed storage tanks the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of C.H.J., Incorporated. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appear different than those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

Respectfully submitted,
C.H.J., INCORPORATED

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Senior Vice President

Jay J. Martin, E.G. 1529
Senior Geologist

TAD/TRW/JJM/RJJ:jjm/tlw

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Aero Tech Photography, February 15, 2001, Black and white aerial photograph nos. 1-45, 1-46 and 1-47.

Fairchild Camera, September, 1931, Black and white aerial photographs, flight no. C-1470, frame nos. B:76, B:77 and B:78.

Riverside County Flood Control and Water Conservation District, December 20, 1957, Black and white aerial photograph numbers 30 and 31.

Riverside County Flood Control and Water Conservation District, May 24, 1974, Black and white aerial photograph numbers 87 and 88.

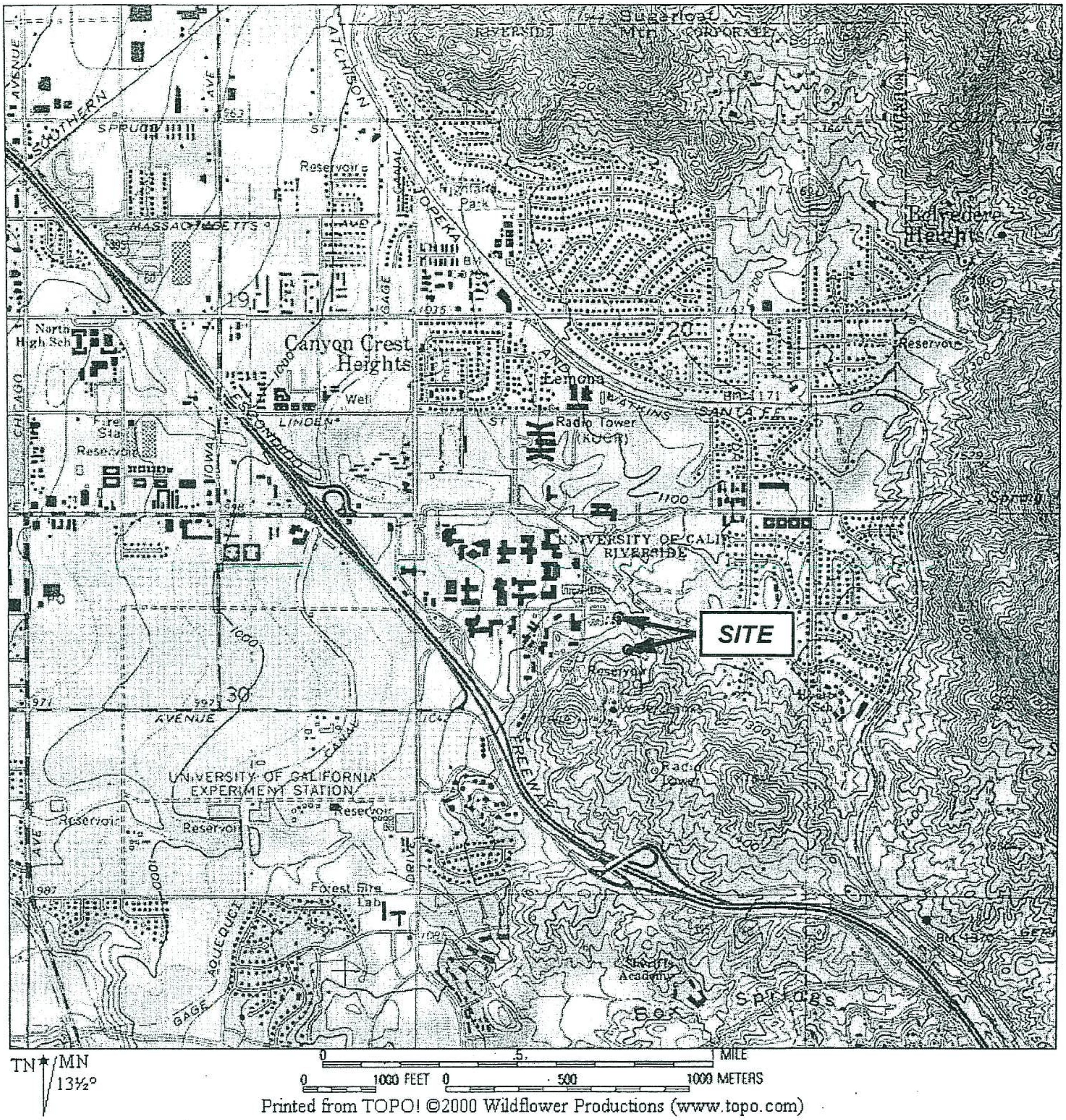
Riverside County Flood Control and Water Conservation District, January 23, 1990, Black and white aerial photograph numbers 3-16 and 3-17.

Riverside County Flood Control and Water Conservation District, February 1, 1995, Black and white aerial photograph numbers 3-16 and 3-17.

C·H·J·

APPENDIX "A"

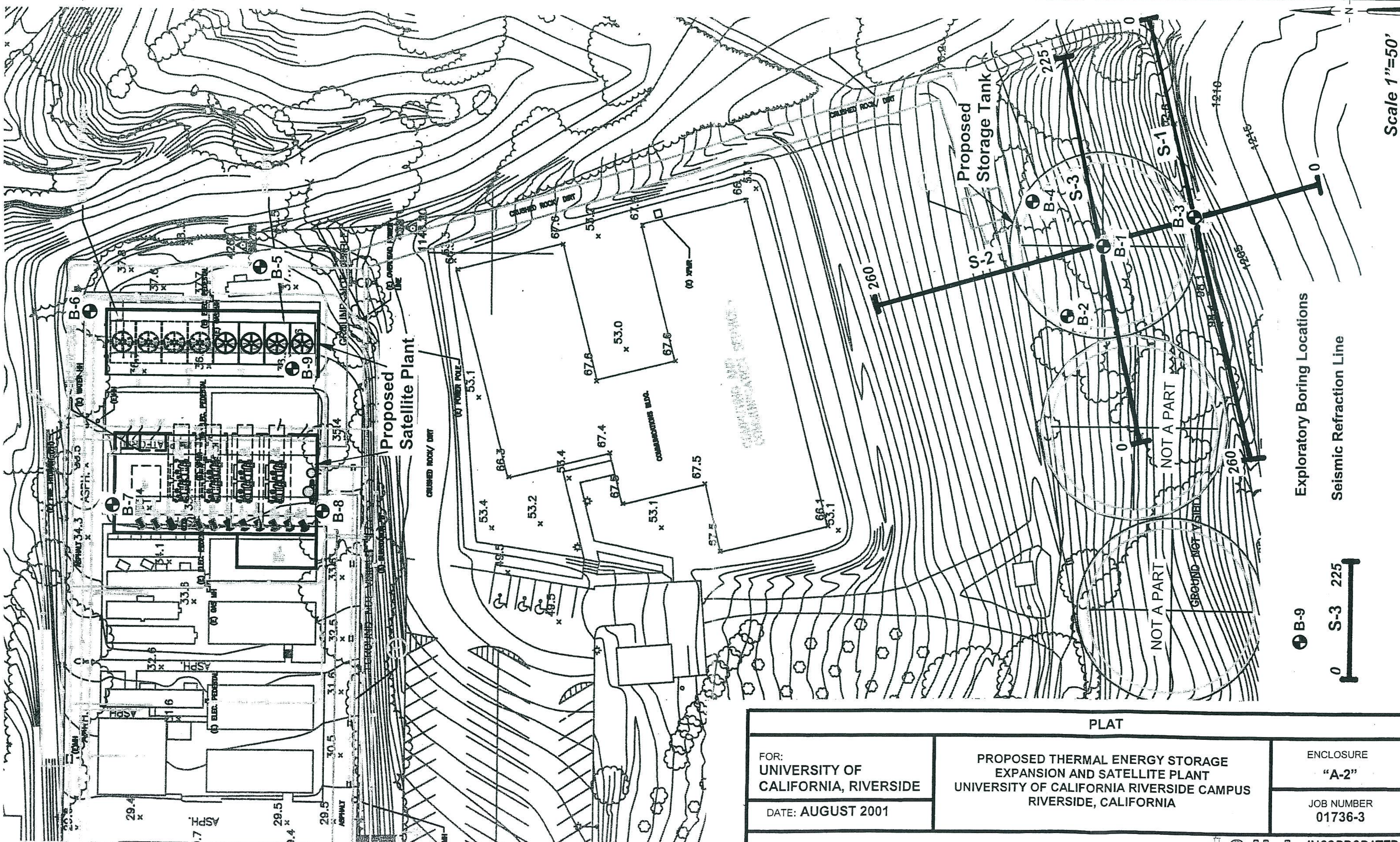
GEOTECHNICAL MAPS



INDEX MAP

FOR: UNIVERSITY OF CALIFORNIA, RIVERSIDE	PROPOSED THERMAL ENERGY STORAGE EXPANSION AND SATELLITE PLANT UNIVERSITY OF CALIFORNIA RIVERSIDE CAMPUS RIVERSIDE, CALIFORNIA	ENCLOSURE "A-1"
DATE: AUGUST 2001		JOB NUMBER 01736-3

C.H.J., INCORPORATED



PLAT

FOR:
UNIVERSITY OF CALIFORNIA, RIVERSIDE

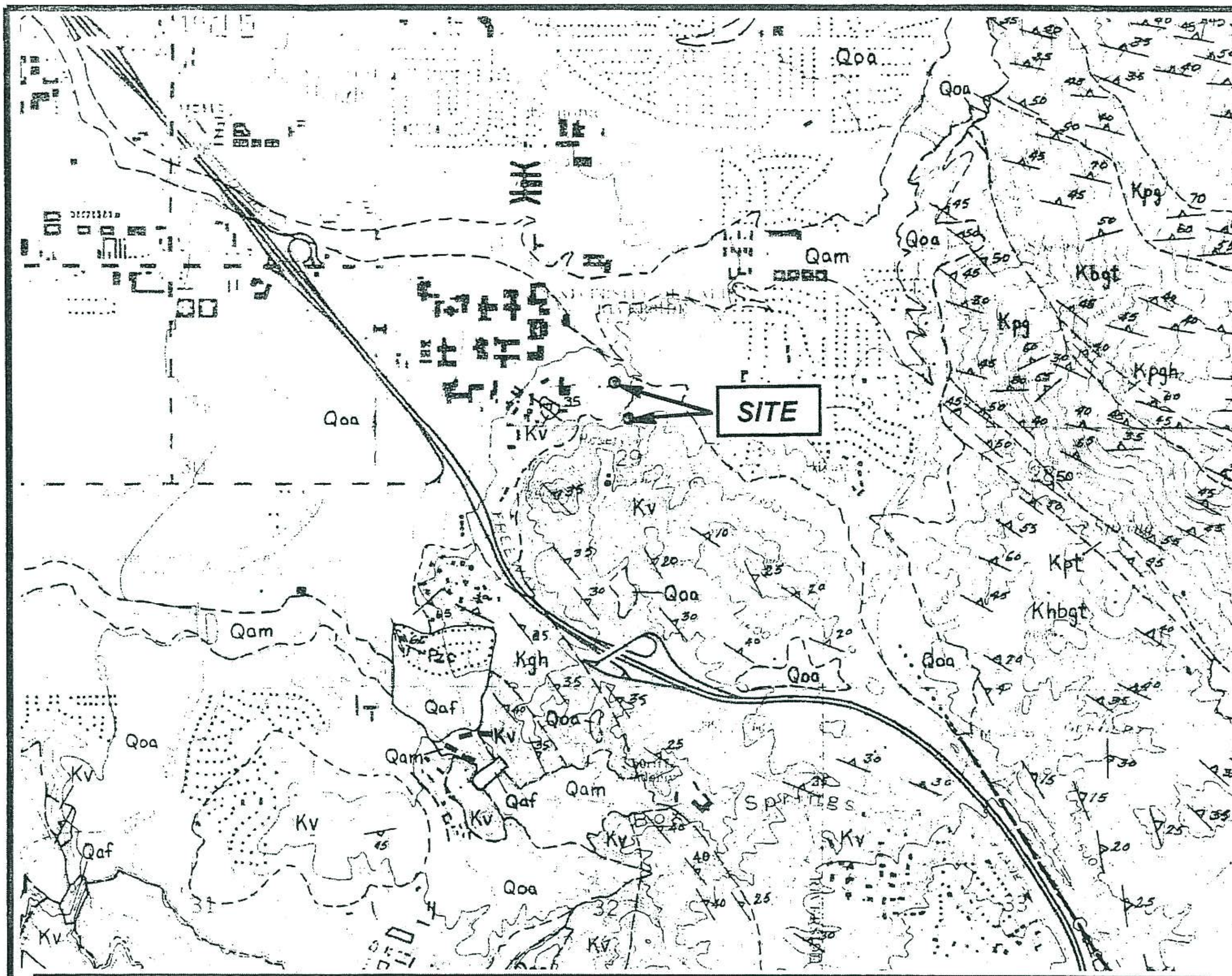
DATE: AUGUST 2001

PROPOSED THERMAL ENERGY STORAGE EXPANSION AND SATELLITE PLANT
UNIVERSITY OF CALIFORNIA RIVERSIDE CAMPUS
RIVERSIDE, CALIFORNIA

ENCLOSURE
"A-2"

JOB NUMBER
01736-3

C.H.J., INCORPORATED



Legend:

Qaf	Artificial fill (Holocene)
Qam	Alluvium of minor streams (Holocene)
Qoa	Older alluvium (Pleistocene)
Kbth	Heterogeneous biotite tonalite
Kpg	Porphyritic granodiorite

Val Verde tonalite and associated rocks (Cretaceous)-

Kv	Val Verde tonalite of Osborn (1939)
Khbgt	Hornblende-biotite granodiorite and tonalite

Metamorphic rocks (Paleozoic?)-

Pzc	Calc-silicate hornfels

85

Contact- Dashed where approximately located. Arrows indicated dip

60 Strike and dip of inclined foliation

Base Map: D.M. Morton and B.F. Cox (1994)

GEOLOGIC INDEX MAP

FOR:
UNIVERSITY OF CALIFORNIA, RIVERSIDE

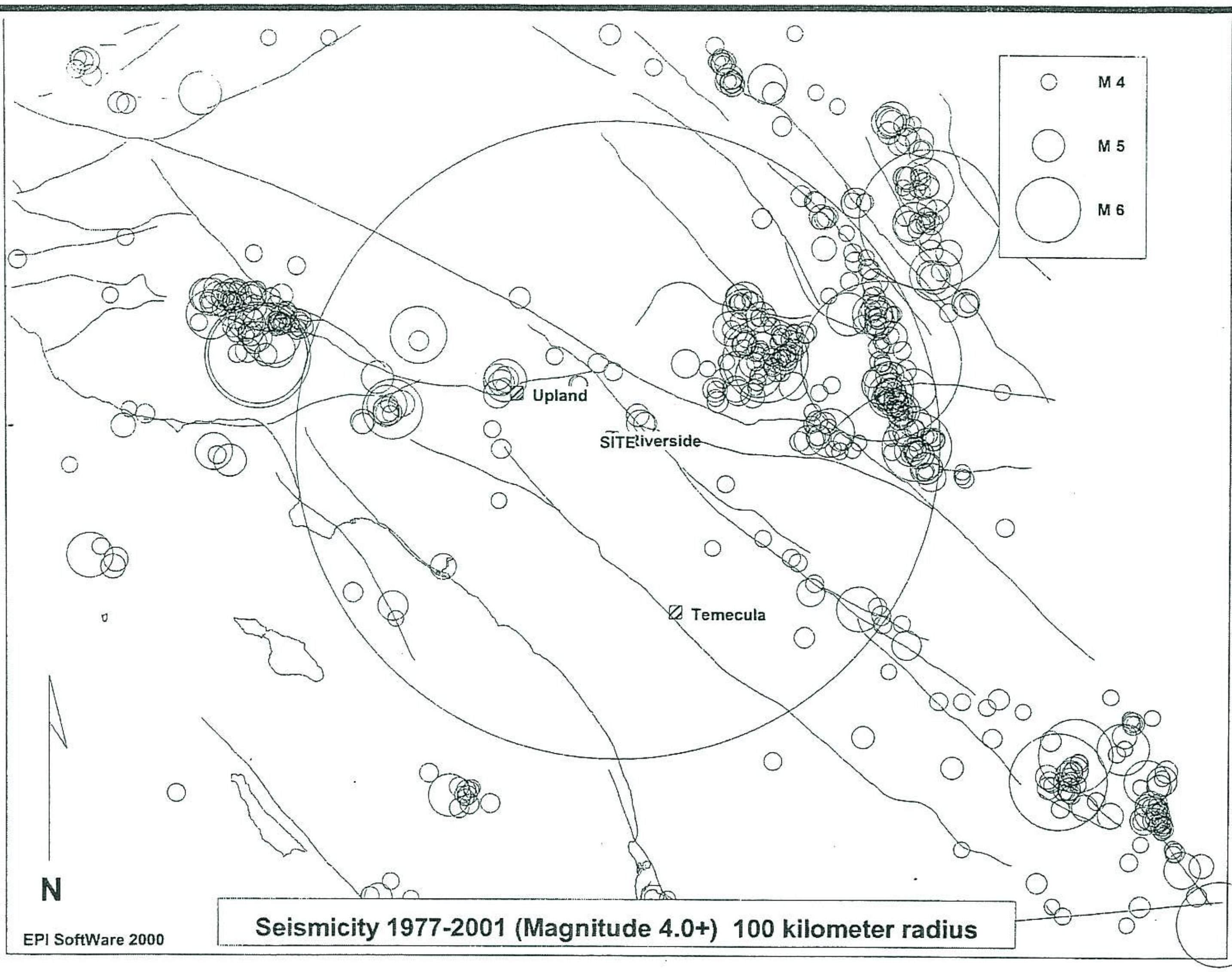
DATE: AUGUST 2001

PROPOSED THERMAL ENERGY STORAGE EXPANSION AND SATELLITE PLANT
UNIVERSITY OF CALIFORNIA RIVERSIDE CAMPUS
RIVERSIDE, CALIFORNIA

ENCLOSURE
"A-3"

JOB NUMBER
01736-3

C.H.J., INCORPORATED



SITE LOCATION: 33.970 LAT. -117.322 LONG.

MINIMUM LOCATION QUALITY: C

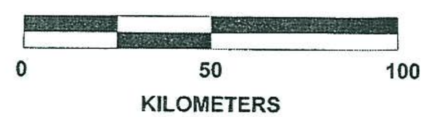
TOTAL # OF EVENTS ON PLOT: 558

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 259

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

4.0- 4.9 : 229
5.0- 5.9 : 27
6.0- 6.9 : 2
7.0- 7.9 : 1
8.0- 8.9 : 0

CLOSEST EVENT: 4.8 ON WEDNESDAY, OCTOBER 02, 1985 LOCATED APPROX. 9 KILOMETERS NORTHEAST OF THE SITE



EARTHQUAKE EPICENTER MAP

FOR: UNIVERSITY OF CALIFORNIA, RIVERSIDE	PROPOSED THERMAL ENERGY STORAGE EXPANSION AND SATELLITE PLANT UNIVERSITY OF CALIFORNIA RIVERSIDE CAMPUS RIVERSIDE, CALIFORNIA	ENCLOSURE "A-4"
DATE: AUGUST 2001		JOB NUMBER 01736-3

C.H.J., INCORPORATED

APPENDIX "B"

EXPLORATORY LOGS

KEY TO LOGS

LEGEND OF SAMPLE DATA:

Dist.	Indicates Disturbed Ring Sample
DS	Direct Shear Test (ASTM D 3080)
Exp.	Expansion Test (UBC Standard Test Method 18-2)
MDC	Maximum Dry Density - Optimum Moisture Content Determination (ASTM D 1557)
N.R.	Indicates No Recovery of Sample
Ring	Indicates Undisturbed Ring Sample. Undisturbed Ring Samples are obtained with a "California Sampler" (3.00" O.D. and 2.42" I.D.) driven by an auto hammer with a 140-pound weight falling 30 inches. The blows per foot are corrected to equivalent SPT N_{60} values.
SA	Sieve Analysis (ASTM D 442)
SE	Sand Equivalent Test (ASTM D 2419)
SPT	Indicates Standard Penetration Test. The SPT N_{60}-value is the corrected number of blows required to drive an SPT sampler 12 inches using a 140 pound weight falling 30 inches. The SPT sampler is 2" O.D. and 1-3/8" I.D.
SS	Soluble Sulfate Test (EPA Method 300.0)

ENGINEERING PROPERTIES FROM SPT BLOWS

Relationship of Penetration Resistance to Relative Density for Cohesionless Soils*
(After Mitchell and Katti, 1981)

No. of SPT Blows (N_{60})	Descriptive Relative Density	Approx. Relative Density (%)
<4	Very Loose	0-15
4-10	Loose	15-35
10-30	Medium Dense	35-65
30-50	Dense	65-85
>50	Very Dense	85-100

* At an effective overburden pressure of 1 ton per square foot (100 kPa)

SOIL CLASSIFICATION CHART

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

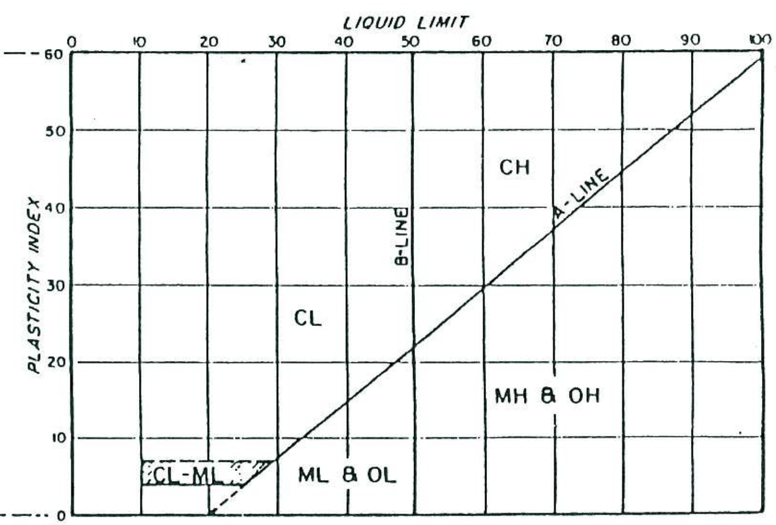
GRADATION CHART

MATERIAL SIZE		PARTICLE SIZE			
		LOWER LIMIT		UPPER LIMIT	
		MILLIMETERS	SIEVE SIZE	MILLIMETERS	SIEVE SIZE
SAND	FINE	.074	#200 x	0.42	#40 x
	MEDIUM	0.42	#40 x	2.00	#10 x
	COARSE	2.00	#10 x	4.76	#4 x
GRAVEL	FINE	4.76	#4 x	19.1	3/4" •
	COARSE	19.1	3/4" •	76.2	3" •
COBBLES		76.2	3"	304.8	12"
BOULDERS		304.8	12"	914.4	36"

x US STANDARD • CLEAR SQUARE OPENINGS

PLASTICITY CHART

FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS



UNIFIED SOIL CLASSIFICATION SYSTEM

C·H·J· INCORPORATED

Enclosure "B" (2of2)
Job No. 01736-3

LOG OF BORING 1

Date Drilled: 8/10/01
Client: University of California
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lb/30 in
Surface Elevation (ft): 1,288±
Logged by: R.M.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium and coarse, light brown					2.3		
						10	2.8	99	Ring
5						64/11"	2.4	116	Ring
		(SM) Silty Sand, fine to medium with coarse, light brown					3.2		
10						40/4"	3.6	106	Ring
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray brown	Bedrock				2.7		
15						40/4"	2.3	110	Ring
20						40/3"	N.R.	N.R.	Ring
25						40/2"	N.R.	N.R.	Ring
		END OF BORING							
30		BEDROCK AT 13.0' REFUSAL AT 25.0' ON BEDROCK NO FILL NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

LOG OF BORING 2

Date Drilled: 8/10/01 | Client: University of California

Equipment: CME 55 Drill Rig | Driving Weight / Drop: 140 lb/30 in

Surface Elevation (ft): 1,183± | Logged by: R.M. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to medium with coarse, light brown			X		2.3		
				X		26	3.9	111	Ring
5		(SM) Silty Sand, fine with medium and clay, light brown			X		5.6		
				X		40/5"	5.5	103	Ring
10		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, light gray	Bedrock		X		1.8		
				X		40/4"	1.3	111	Ring
15				X		40/2"	N.R.	N.R.	Ring
20		END OF BORING							
25									
30		BEDROCK AT 9.0' REFUSAL AT 21.0' ON BEDROCK NO FILL NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

LOG OF BORING 3

Date Drilled: 8/15/01 | Client: University of California

Equipment: CME 55 Drill Rig | Driving Weight / Drop: 140 lb/30 in

Surface Elevation (ft): 1,199± | Logged by: T.D. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine with medium, light red brown	Fill		X		2.7		
				X		6	4.0	111	Ring
3.5		(SM) Silty Sand, fine to medium with coarse, red brown	Native		X		3.2		
5				X		40/5"	2.5	137	Ring
10				X		40/3.5"	5.9	120	Ring
14		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray	Bedrock						
15				X		40/1.5"	1.1	120	Ring
20				X		40/1"	0.8	114	Ring
25				X		40/3"	0.8	113	Ring
25		END OF BORING							
30		BEDROCK AT 14.0' REFUSAL AT 25.0' ON BEDROCK FILL TO 3.5' NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-3

LOG OF BORING 4

Date Drilled: 8/15/01 | Client: University of California

Equipment: CME 55 Drill Rig | Driving Weight / Drop: 140 lb/30 in

Surface Elevation (ft): 1,181± | Logged by: T.D. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium, red brown					2.9		
						26	4.8	117	Ring
5									
		(SM) Silty Sand, fine to coarse, gray brown					3.7		
						40	2.8	116	SA, SE, MDC
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray	Bedrock				2.9		
10									
						40/2"	2.6	110	Ring
15									
						40/2"	2.6	110	Ring
20									
		END OF BORING				40/1"	N.R.	N.R.	Ring
25									
30		BEDROCK AT 8.0' REFUSAL AT 22.0' ON BEDROCK NO FILL NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

LOG OF BORING 5

Date Drilled: 8/15/01 | Client: University of California

Equipment: CME 55 Drill Rig | Driving Weight / Drop: 140 lb/30 in

Surface Elevation (ft): 1,138± | Logged by: T.D. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium, light red brown			X		2.3		
				X		32	1.7	124	Ring
5				X		20	2.2	117	Ring
		(SM) Silty Sand, fine to coarse, brown to red brown			X		2.5		SA, MDC
10				X		62/9"	5.5	123	Ring
15				X		40/4"	6.7	127	Ring
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, red brown to gray brown	Bedrock		X		2.7		
20				X		40/2"	3.1	110	Ring
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray			X		2.5		
25				X		40/2.5"	2.2	111	Ring
30				X		40/1.5"	N.R.	N.R.	Ring
		END OF BORING BEDROCK AT 18.5', NO REFUSAL, NO FILL, NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-5

LOG OF BORING 6

Date Drilled: 8/15/01 | Client: University of California

Equipment: CME 55 Drill Rig | Driving Weight / Drop: 140 lb/30 in

Surface Elevation (ft): 1,137± | Logged by: T.D. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium, red brown	Fill				3.7		
						74	4.6	132	Ring
5		(SM) Silty Sand, fine to medium with coarse, red brown	Native				4.1		
						19	3.8	117	Ring
10									
						49	3.0	116	Ring
15									
						40/3"	6.3	124	Ring
20			Rock						
						40/0.5"	3.2	112	Ring
25		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray	Bedrock						
						40/3"	5.9	109	Ring
30									
						40/3"	2.8	105	Ring

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-6a

LOG OF BORING 6

Date Drilled: 8/15/01
Client: University of California
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lb/30 in
Surface Elevation (ft): 1,137±
Logged by: T.D.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray							
						40/2"	1.7	115	Ring
40									
						40/1.5"	1.5	112	Ring
45									
		END OF BORING				40/1"	N.R.	N.R.	Ring
50									
55									
60		BEDROCK AT 24.5' NO REFUSAL FILL TO 4.5' NO FREE GROUNDWATER							
65									

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-6b

LOG OF BORING 7

Date Drilled: 8/15/01
Client: University of California
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lb/30 in
Surface Elevation (ft): 1,135±
Logged by: T.D.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium, red brown	Fill		X		5.1		
				X		63	6.7	133	Ring
5		(SM) Silty Sand, fine with medium and clay, red brown	Native		X		5.1		
				X		17	5.4	115	Exp., SS, SA Ring
10				X		40/5"	7.6	119	Ring
		(SP-SM) Sand, fine to coarse with silt and clay, light red brown			X		6.1		
15				X		40/6"	5.4	126	Ring
20				X		40/6"	6.5	126	Ring
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt, gray brown	Bedrock		X		2.0		
25				X		40/4.5"	2.3	112	Ring
30				X		40/2"	N.R.	N.R.	Ring
		END OF BORING BEDROCK AT 23.0', NO REFUSAL, FILL TO 4.5', NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-7

LOG OF BORING 8

Date Drilled: 8/15/01
Client: University of California

Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lb/30 in

Surface Elevation (ft): 1,135±
Logged by: T.D.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium and clay, red brown			X		5.6		SA, SS, MDC, DS
				X		40/4"	6.1	126	Ring
5									
				X		40/6"	6.3	116	Ring
10									
				X		40/5.5"	7.2	114	Ring
		(SP-SM) Weathered Granitic Bedrock recovered as Sand, fine to coarse with silt and clay, gray brown	Bedrock		X		3.7		
15									
				X		40/3"	3.2	107	Ring
20									
				X		40/1.5"	3.0	115	Ring
25									
				X		40/2"	N.R.	N.R.	Ring
		END OF BORING							
30		BEDROCK AT 12.5' NO REFUSAL NO FILL NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-8

LOG OF BORING 9

Date Drilled: 8/15/01
Client: University of California
Equipment: CME 55 Drill Rig
Driving Weight / Drop: 140 lb/30 in
Surface Elevation (ft): 1,136±
Logged by: T.D.
Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (SPT N_{60} Equiv.)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine with medium, red brown			X		5.8		
				X		49	5.1	120	Ring
				X		74/10.5"	5.2	121	Ring
10		(SM) Weathered Granitic Bedrock recovered as Silty Sand, fine to coarse with clay and gravel to 1/2", dark gray brown			X		5.8		SA, MDC
				X		40/2.5"	2.2	110	Ring
			Bedrock		X		2.3		
15				X		40/1.5"	3.3	113	Ring
20				X		40/1.5"	N.R.	N.R.	Ring
25				X		40/4.5"	1.6	118	Ring
30				X		40/1.5"	3.5	109	Ring
		END OF BORING BEDROCK AT 13.0', NO REFUSAL, NO FILL, NO FREE GROUNDWATER							

BOREHOLE_LOG 01736-3.GPJ CHJ.GDT 8/28/01

C·H·J·

STORAGE TANK & SATELLITE PLANT
RIVERSIDE, CALIFORNIA

Job No. 01736-3

Enclosure B-9

APPENDIX "C"

LABORATORY TESTING

C·H·J·

Enclosure "C-1"
Job No. 01736-3

TEST DATA SUMMARY

OPTIMUM MOISTURE - MAXIMUM DENSITY RELATION:

ASTM D 1557-91

Boring No.	Depth of Sample (ft.)	Classification	Optimum Moisture (Percent)	Maximum Dry Density (pcf)
B4	55.0-80.0	Silty Sand, fine to coarse, brown (SM)	8.0	133.0
B5 & B9	7.5 & 9.5	Silty Sand, fine to coarse, brown (SM)	7.5	135.5
B8	0.0-12.5	Silty Sand, fine with medium and clay, red brown (SM)	8.5	132.5

DIRECT SHEAR TEST: Remolded to 90% Relative Compaction: (Ultimate, Saturated)

ASTM D 3080

Boring No.	Depth of Sample (ft.)	Angle of Internal Friction (^{0})	Apparent Cohesion (PSF)
8	0.0	27	50

TEST DATA SUMMARY

EXPANSION INDEX:

UBC Standard Test Method 18-2

Boring No.	Depth of Sample (ft.)	Initial Moisture (%)	Final Moisture (%)	Degree of Saturation (%)	Expansion Index	Expansion Potential
7	4.5	10.5	17.9	46.0	4	"very low"

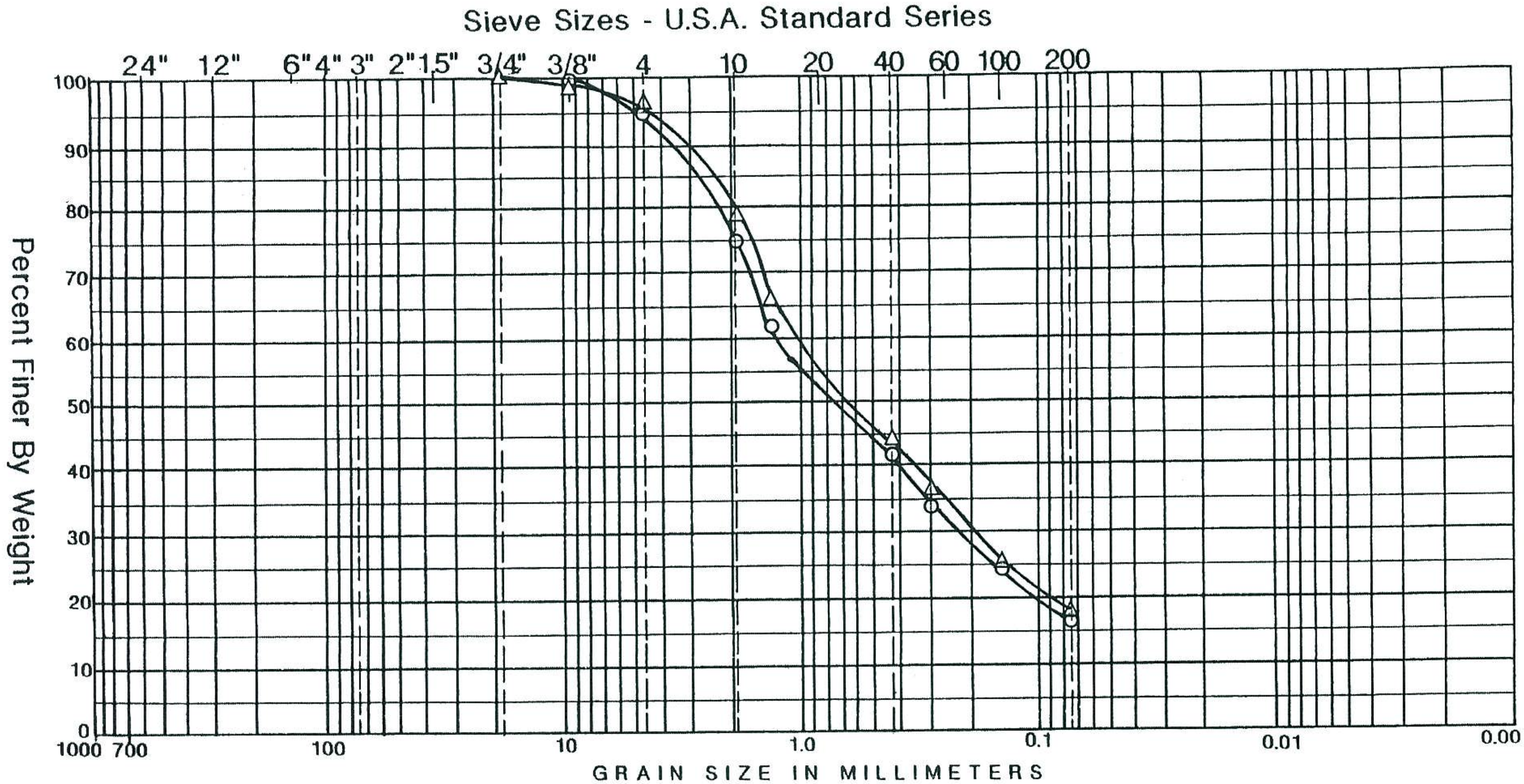
SOLUBLE SULFATES:

EPA Test Method 300.0

Boring No.	Depth of Sample (Ft.)	Result (%)	Exposure*
7	4.5	0.0075	"negligible"
8	0.0	0.0025	"negligible:

* Based on criteria from American Concrete Institute Manual of Concrete Practice, 2000 (Part 3), Table 4.3.1.

ENCLOSURE: "C-3"
JOB NO.: 01736-3



Cobbles & Boulders	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Boring	Depth	Classification		
O	4	5.5'-8.0'	SM	Silty Sand, fine to coarse	SE=38
Δ	5 & 9	7.5' & 9.5'	SM	Silty Sand, fine to coarse	

Gradation Curves

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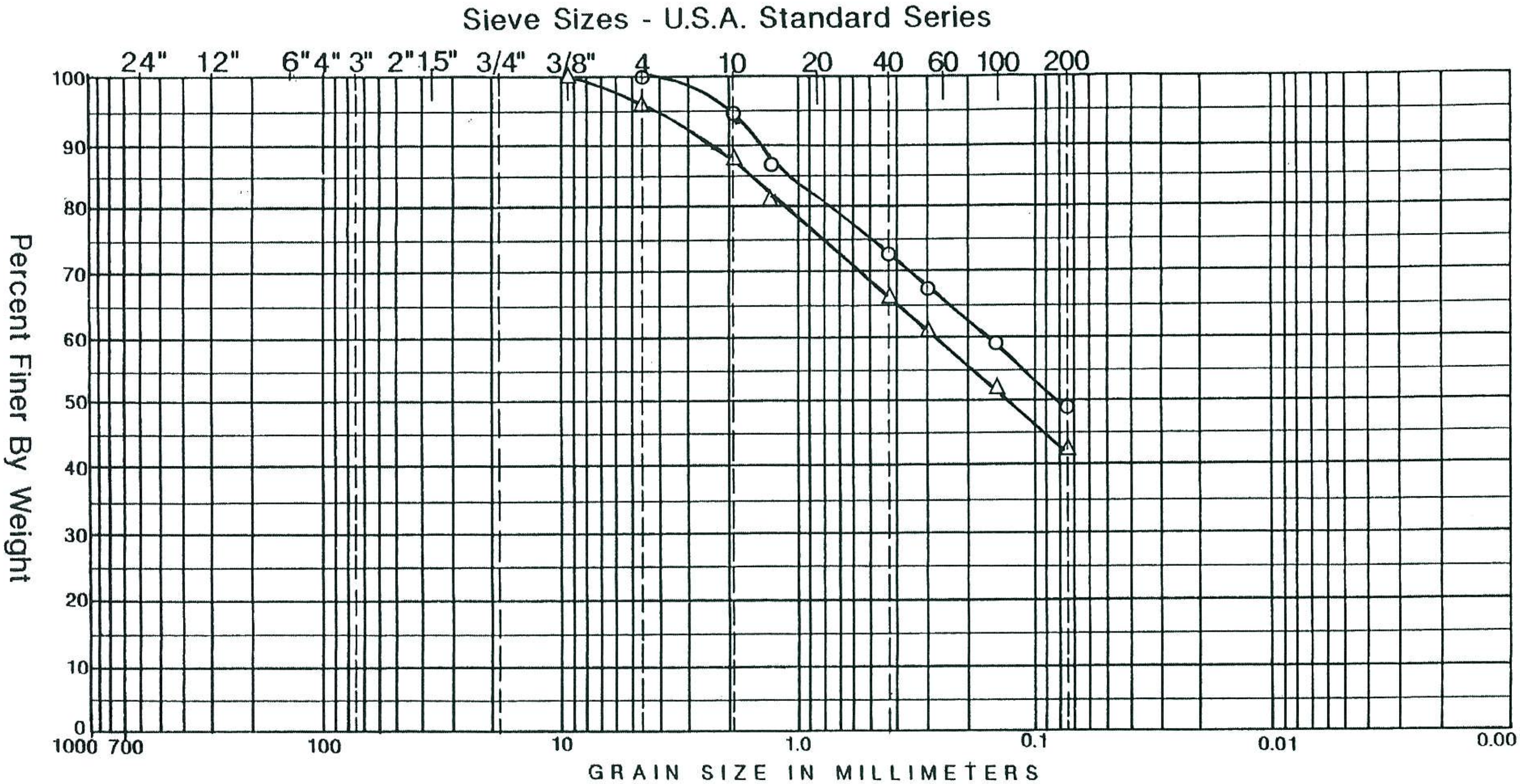
ENCLOSURE: "C-4"
JOB NO.: 01736-3



Cobbles & Boulders	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Boring	Depth	Classification	
O	7	4.5'	SM	Silty Sand, fine with medium and clay
Δ	8	0.0' - 12.5'	SM	Silty Sand, fine with medium and clay

Gradation Curves

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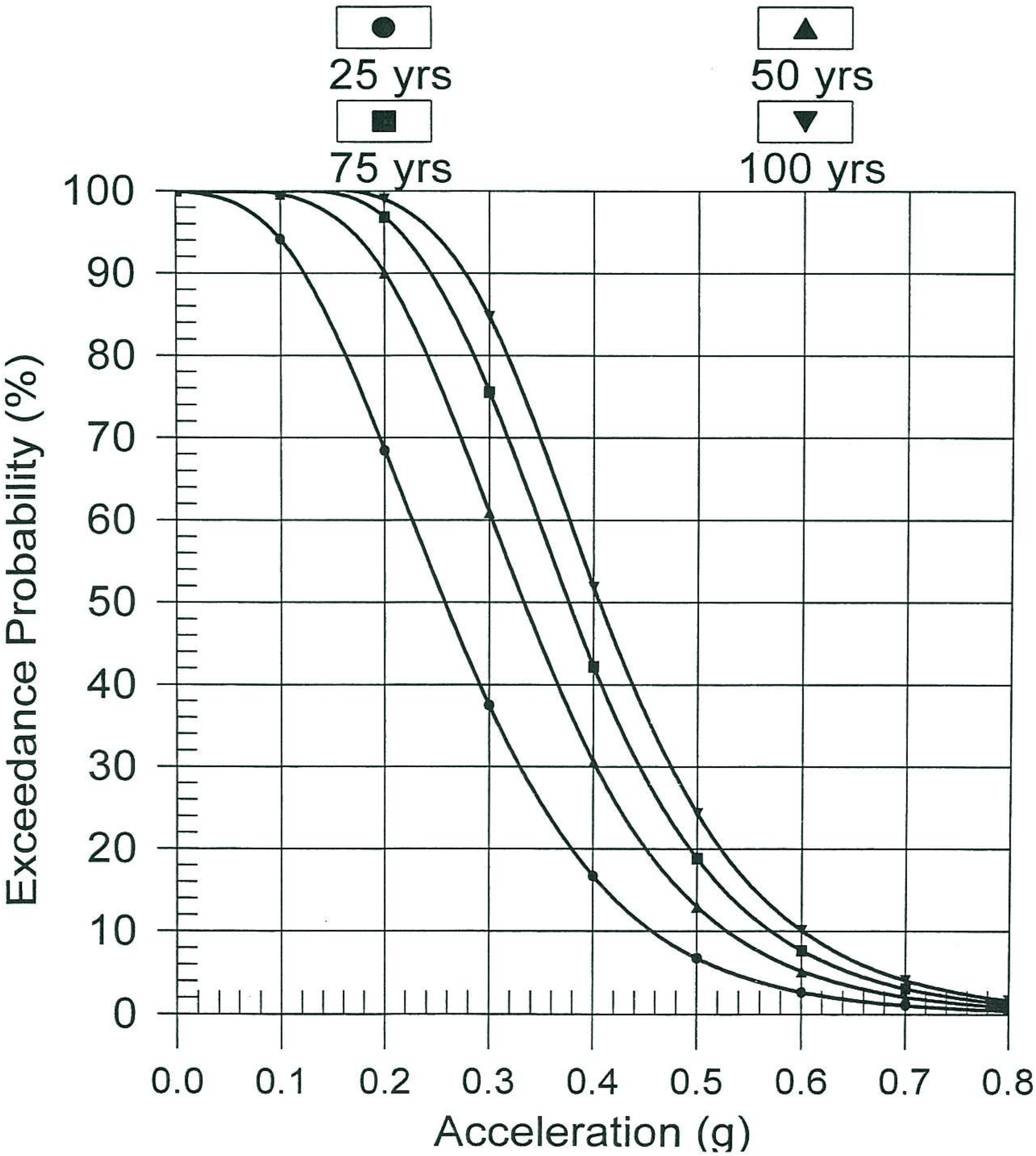
APPENDIX "D"

PROBABILITY OF EXCEEDANCE VS. ACCELERATION

PROBABILITY OF EXCEEDANCE

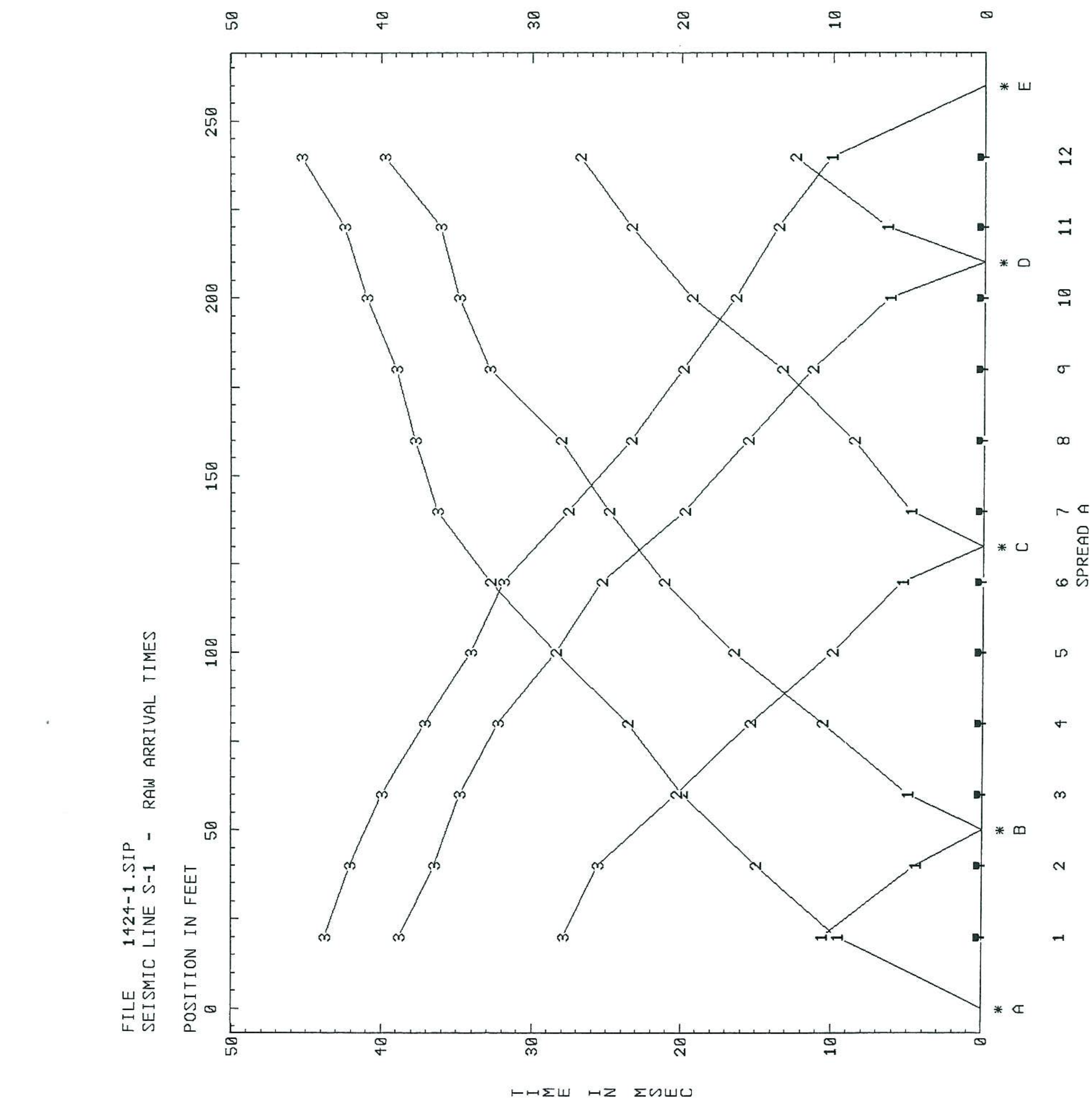
CAMPBELL (1997, 2000) Hard Rock



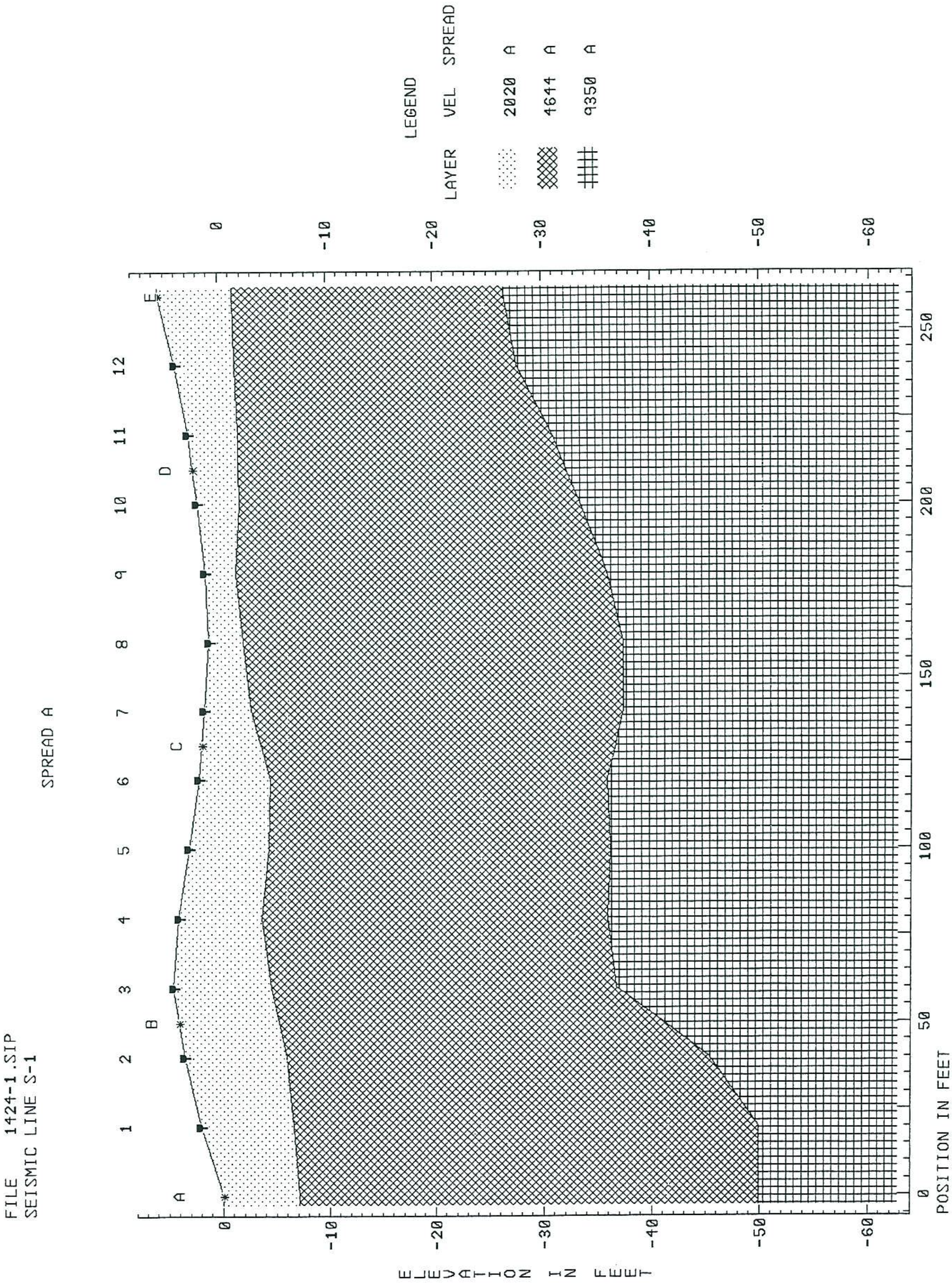
APPENDIX "E"

SEISMIC REFRACTION DATA AND INTERPRETATIONS

ENCLOSURE: "E-1"
JOB NO.: 01736-3



ENCLOSURE: "E-2"
JOB NO.: 01736-3

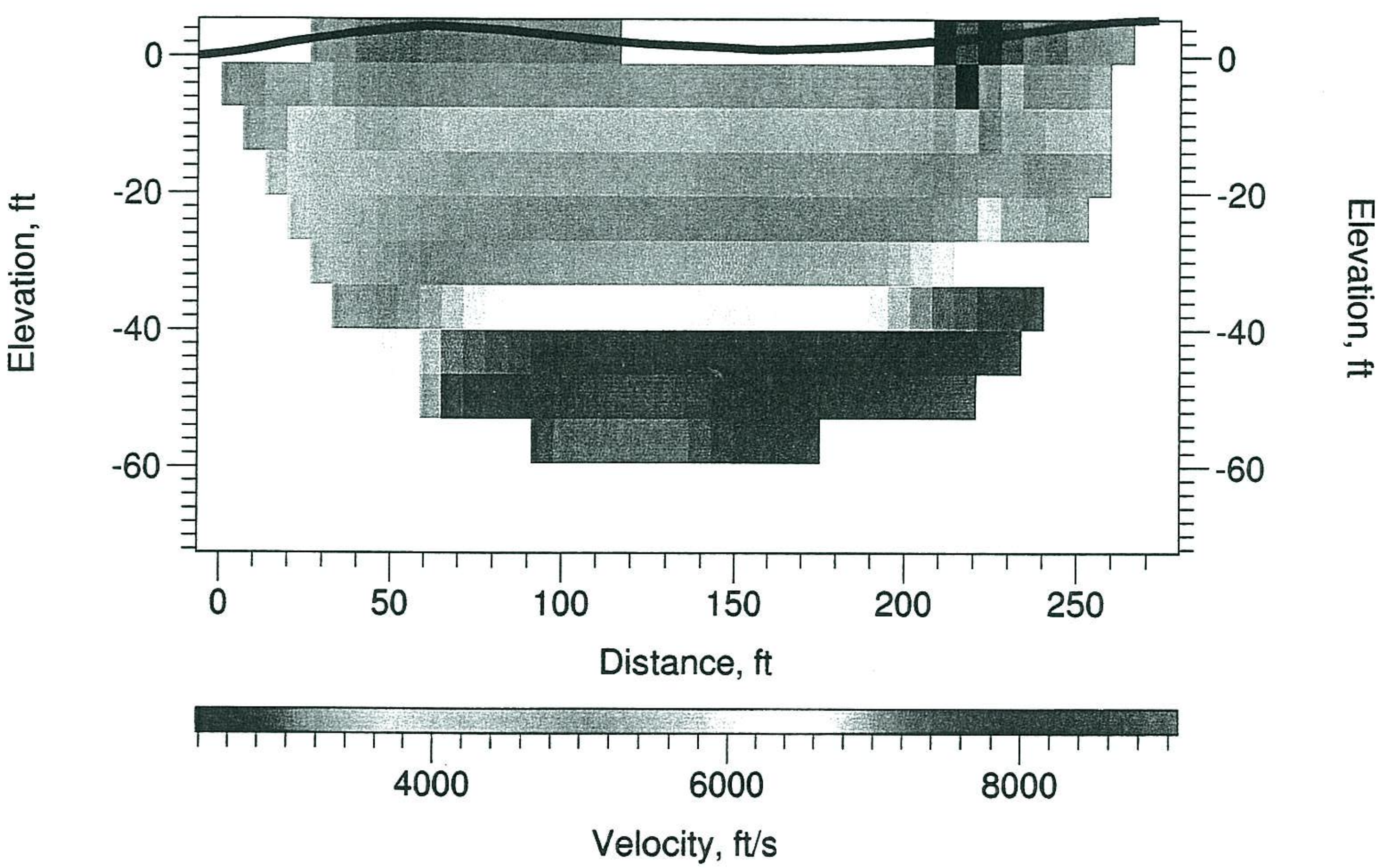


ENCLOSURE: "E-3"
JOB NO.: 01736-3

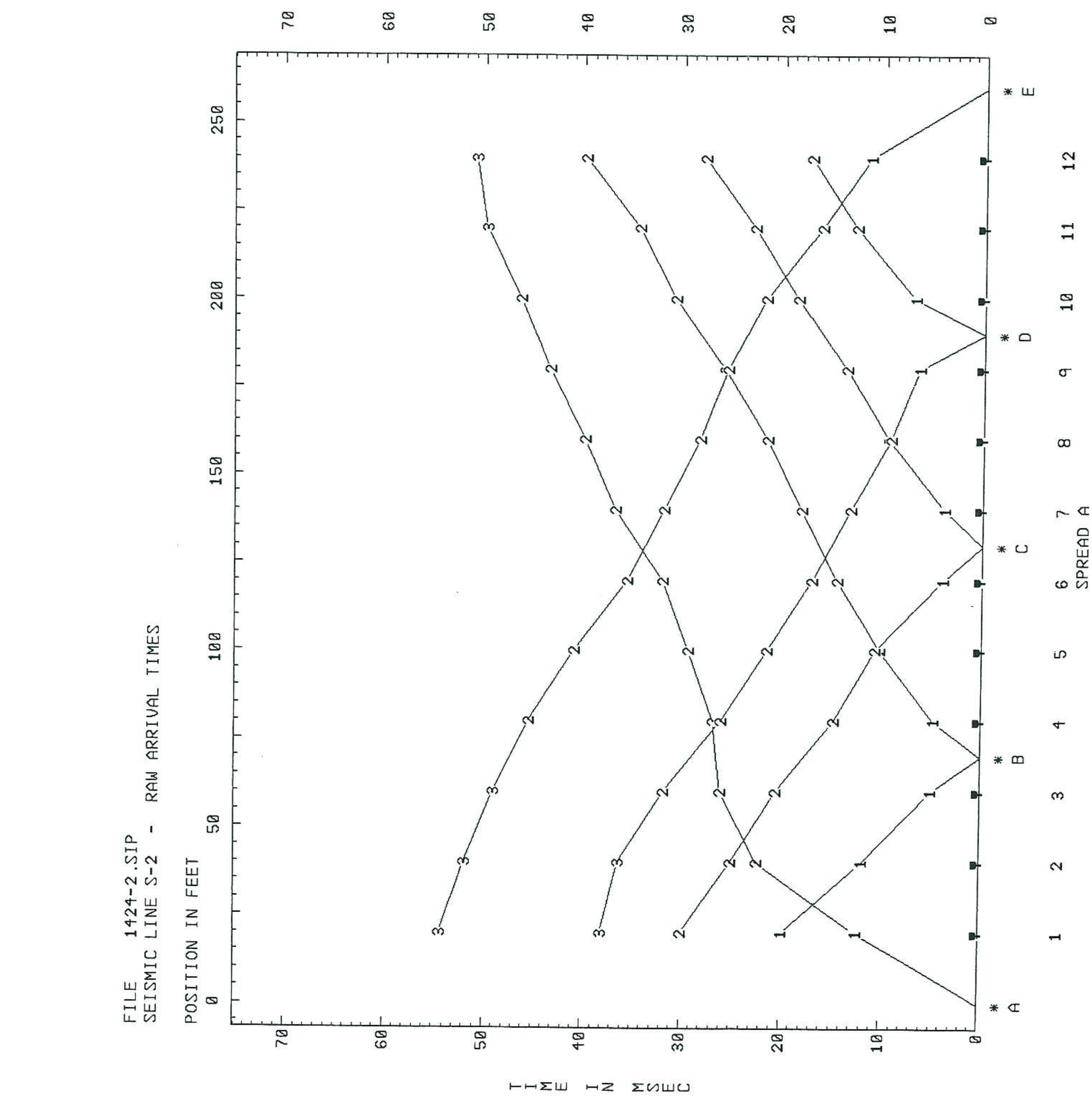
SEISMIC LINE S-1

South 77° West →

Velocity Gradient Model

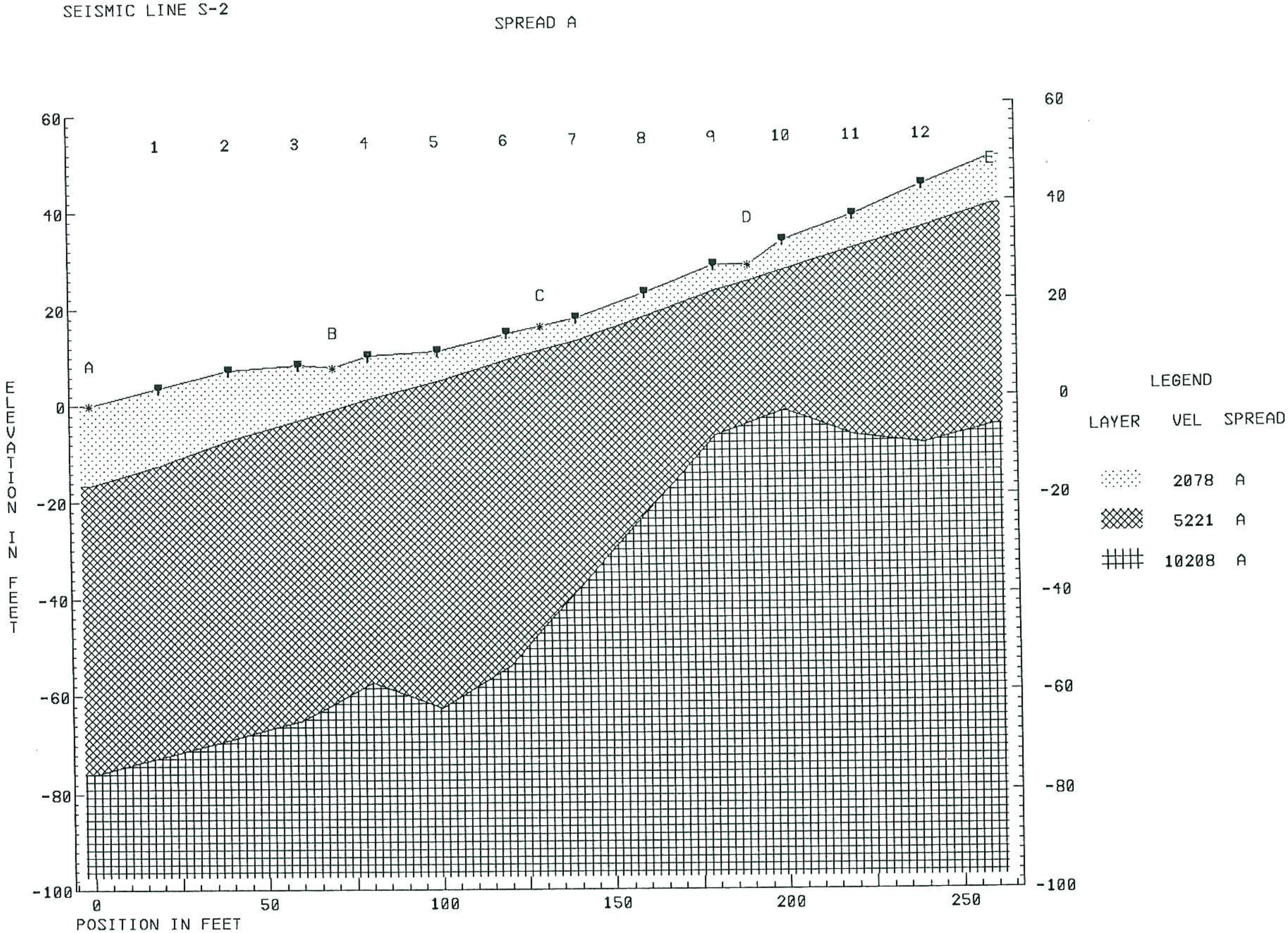


ENCLOSURE: "E-4"
JOB NO.: 01736-3



ENCLOSURE: "E-5"
JOB NO.: 01736-3

FILE 1424-2.SIP
SEISMIC LINE S-2

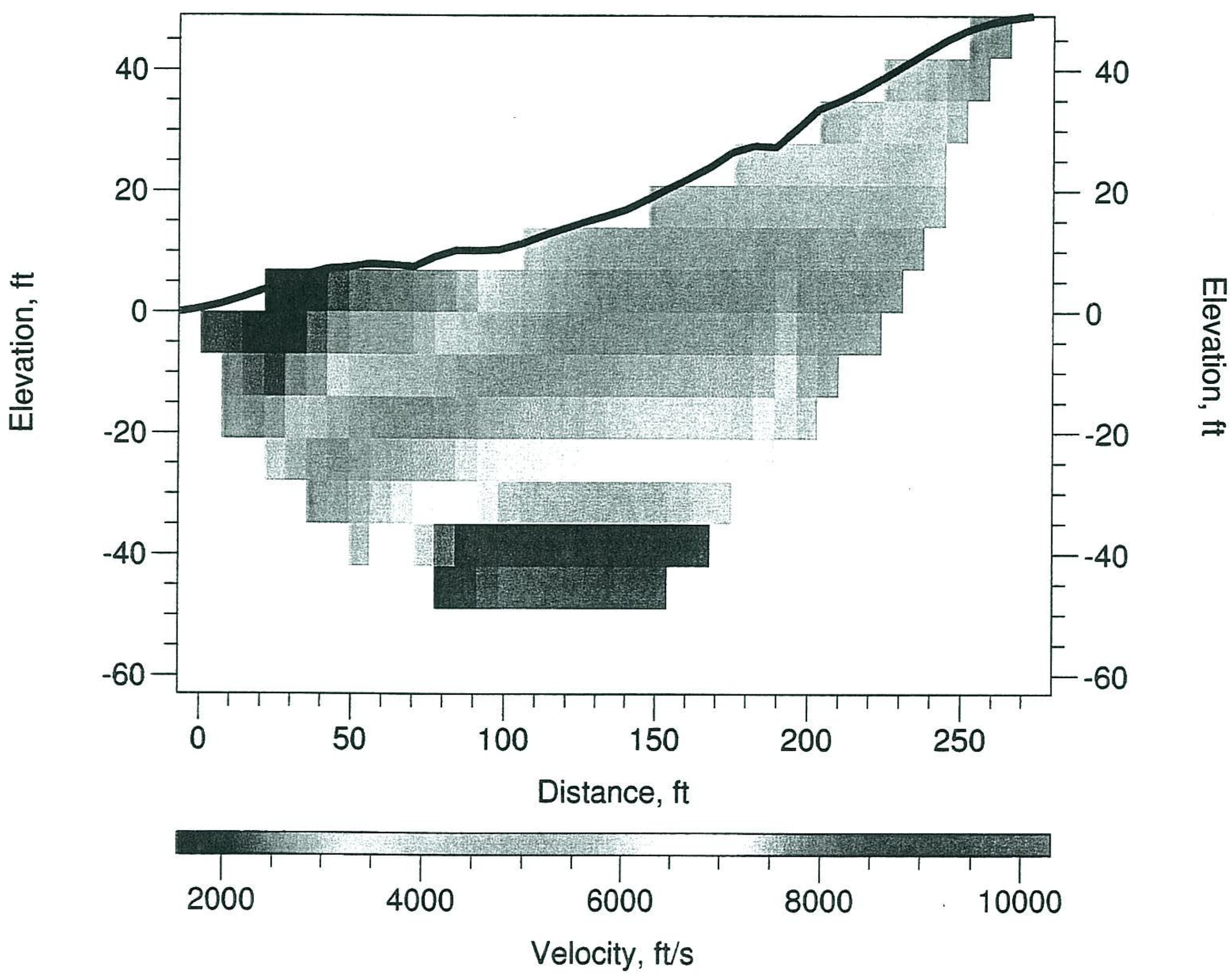


ENCLOSURE: "E-6"
JOB NO.: 01736-3

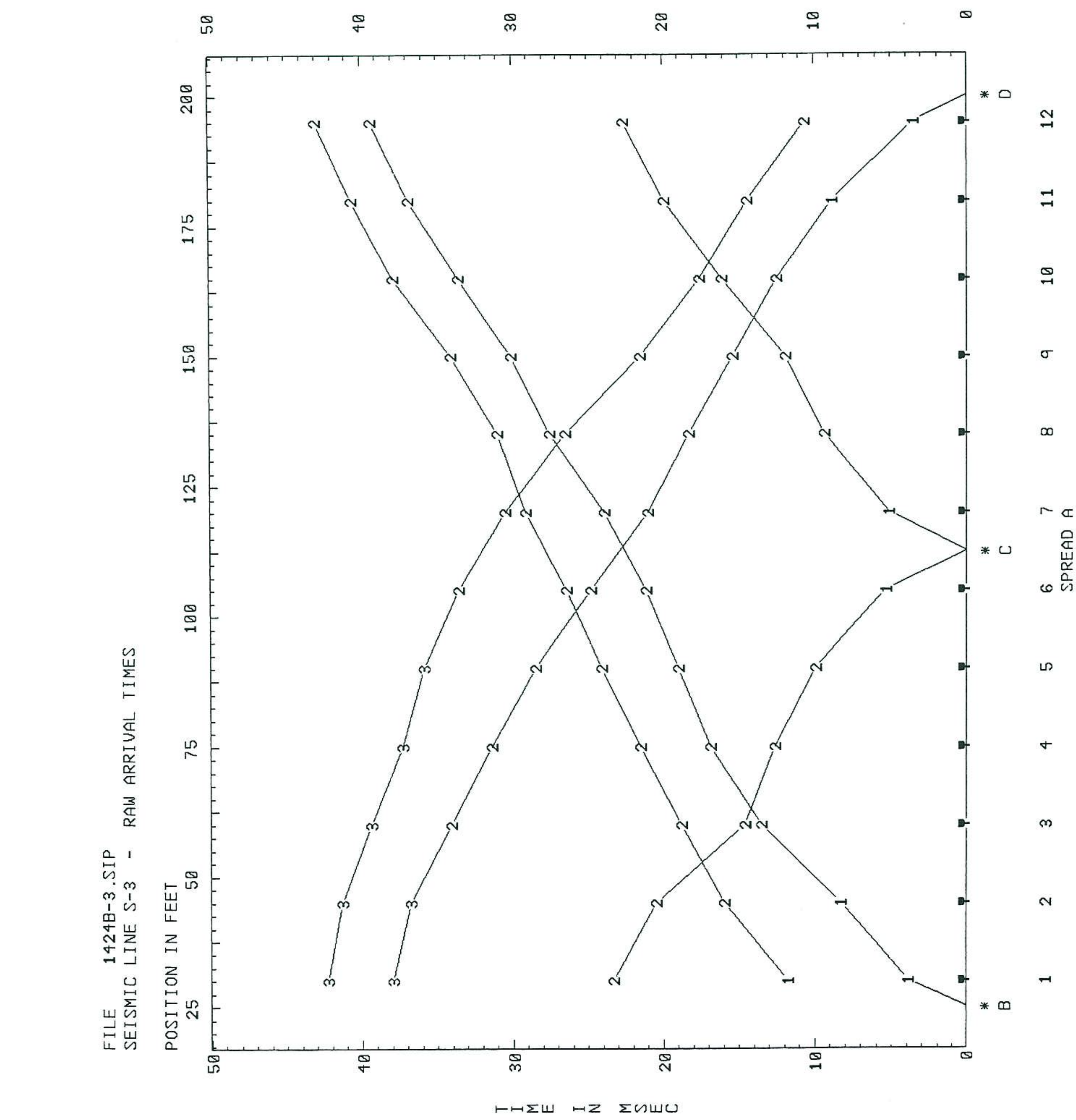
SEISMIC LINE S-2

South 16° East →

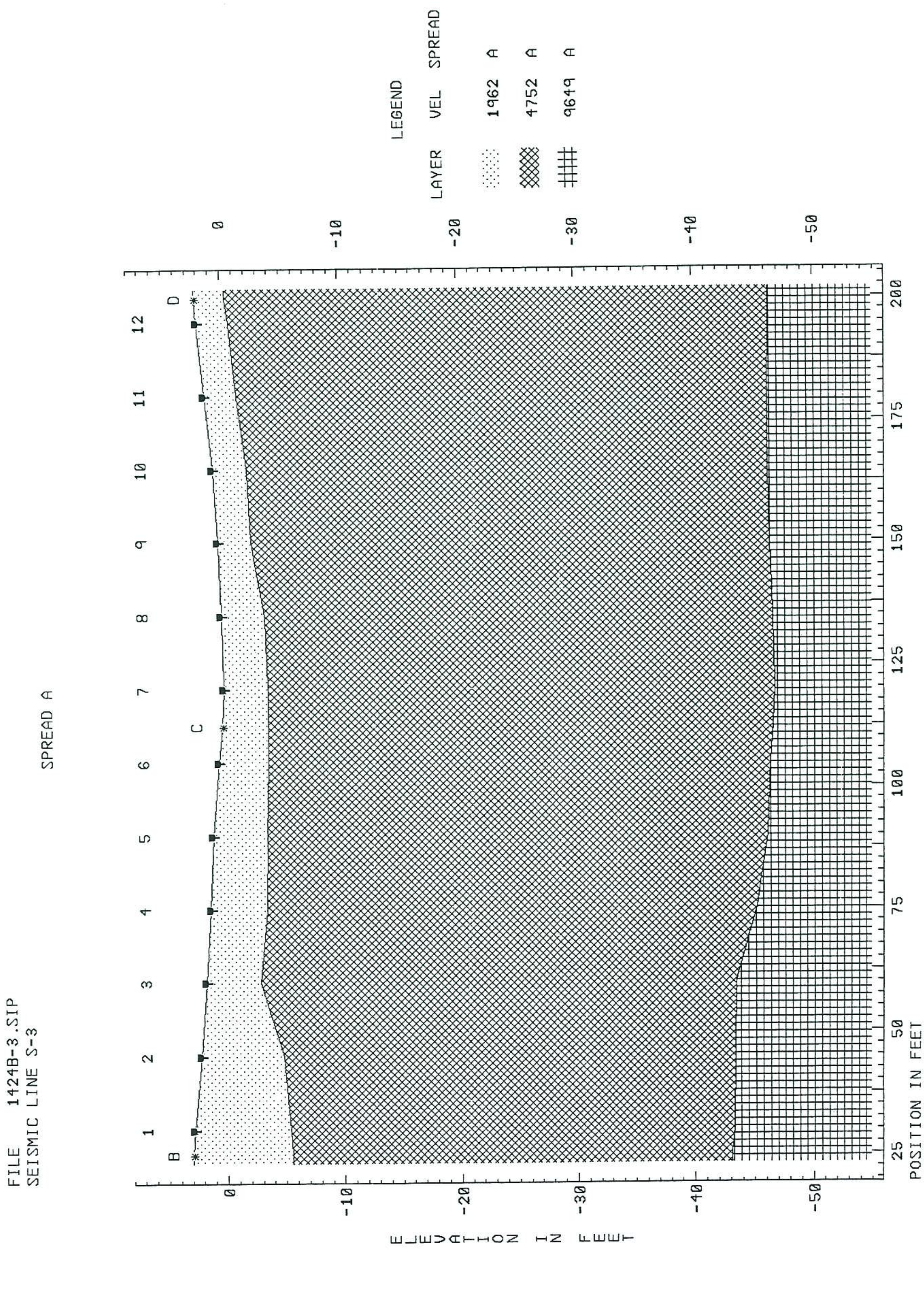
Velocity Gradient Model



ENCLOSURE: "E-7"
JOB NO.: 01736-3



ENCLOSURE: "E-8"
JOB NO.: 01736-3



ENCLOSURE: "E-9"
JOB NO.: 01736-3

SEISMIC LINE S-3

North 78° East →

Velocity Gradient Model