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16. ABSTRACT

Introduction: This report presents the results of a seismicity and dynamic response evaluation for embankment and foundation soils for a seismicity and dynamic response evaluation for embankment and foundation soils for a portion of the proposed Interstate Route 15 (P.M. 43.2/48.1) within San Diego County. A contract was awarded for construction of this project in June 1976.

The study was undertaken to supplement the districts pseudostatic analysis which indicated major embankment instability under maximum credible seismic events within the limits of the proposed construction contract. The districts findings precipitated concern for future traffic safety and maintenance of the proposed facility. A more rigorous dynamic analysis was instituted based upon discussions between Eugene Calman the District 11 Materials Engineer and Raymond Forsyth of the Transportation Laboratory. consistent with these discussions, the analysis concentrated on an assumed critical embankment- foundation composite of Station 2218 and is considered representative of other embankment-foundation composites within this locality. Station 2218 is located within the San Luis Rey River Flood Plain about 0.6 mile south of Route 76. A plan view of the project site and location of the study area are shown on the vicinity map, Figure 1.

Pertinent information on geology, seismic setting, fault location and general subsurface soil conditions was obtained from District 11 personnel and utilized as a base from which to develop the work contained herein.

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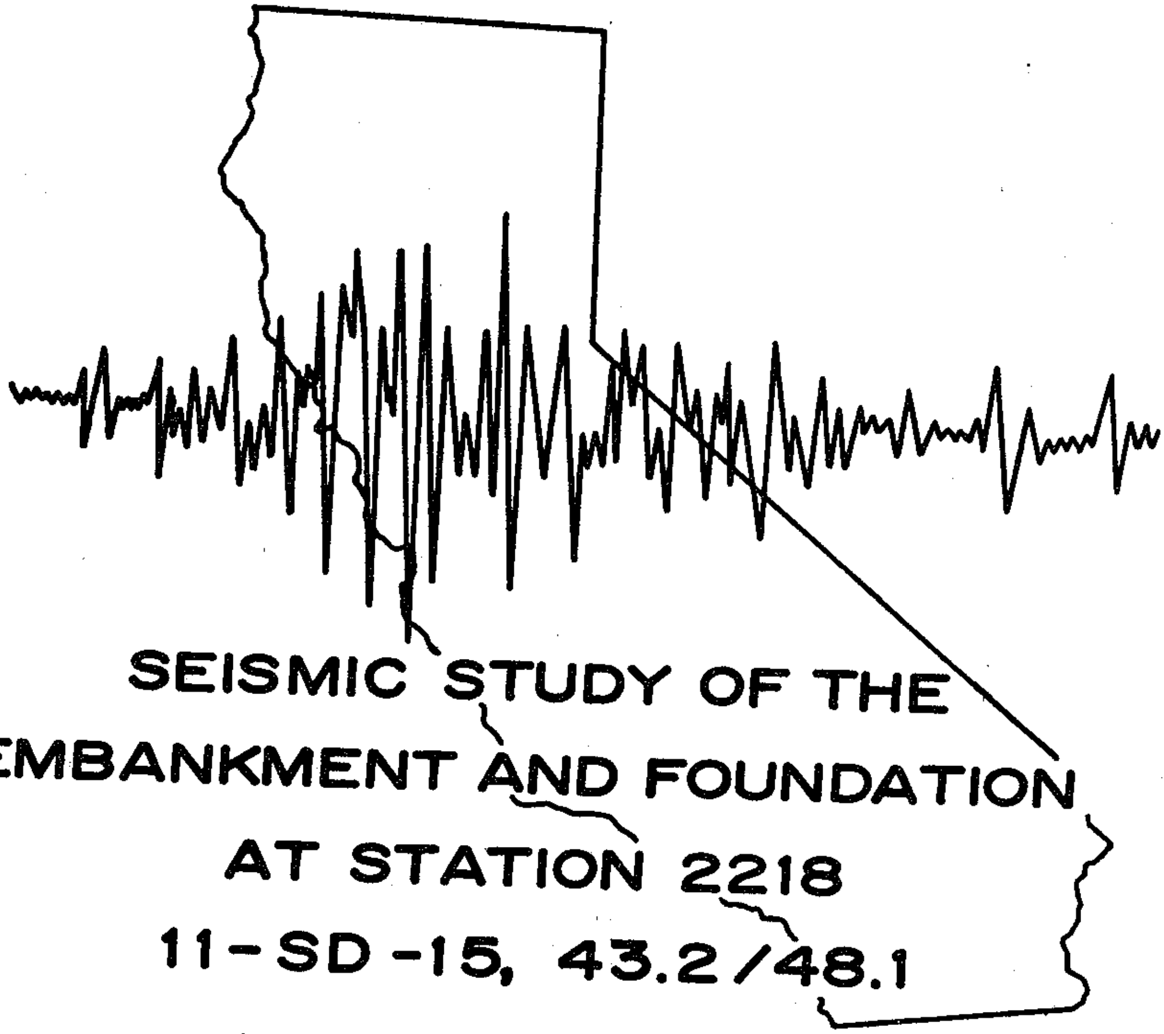

SEISMIC STUDY OF THE EMBANKMENT AND FOUNDATION AT STATION 2218

11-SD-15, 43.2/48.1

Caltrans

CALIFORNIA DEPARTMENT OF TRANSPORTATION

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DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES & ENGINEERING SERVICES
OFFICE OF TRANSPORTATION LABORATORY

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Lilac Road to Rte. 76
11203-095081
Lab Auth 652840

Mr. J. Dekema
District 11 Director of Transportation
4075 Taylor Street, P. O. Box 390
San Diego, California 92112

Dear Sir:

Submitted for your consideration is:

SEISMIC STUDY OF THE EMBANKMENT
AND FOUNDATION AT STATION 2218
11-SD-15-43.2/48.1

Study Made by . . . . Geotechnical Branch

Under the general Direction of . . . . Raymond Forsyth

Work Supervised by . . . . Joseph Hannon

Analysis and Report by . . . . Kenneth Jackura

Very truly yours,

GEORGE A. HILL
Chief, Office of Transportation Laboratory

By
R. A. Forsyth
Chief, Geotechnical Branch

Attachments

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INTRODUCTION

This report presents the results of a seismicity and dynamic response evaluation for embankment and foundation soils for a portion of the proposed Interstate Route 15 (P.M. 43.2/48.1) within San Diego County. A contract was awarded for construction of this project in June 1976.

The study was undertaken to supplement the districts pseudo-static analysis which indicated major embankment instability under maximum credible seismic events within the limits of the proposed construction contract. The districts findings precipitated concern for future traffic safety and maintenance of the proposed facility. A more rigorous dynamic analysis was instituted based upon discussions between Eugene Calman the District 11 Materials Engineer and Raymond Forsyth of the Transporation Laboratory. Consistent with these discussions, the analysis concentrated on an assumed critical embankment-foundation composite of Station 2218 and is considered representative of other embankment-foundation composites within this locality. Station 2218 is located within the San Luis Rey River Flood Plain about 0.6 mile south of Route 76. A plan view of the project site and location of the study area are shown on the vicinity map, Figure 1.

Pertinent information on geology, seismic setting, fault location and general subsurface soil conditions was obtained from District 11 personnel and utilized as a base from which to develop the work contained herein.

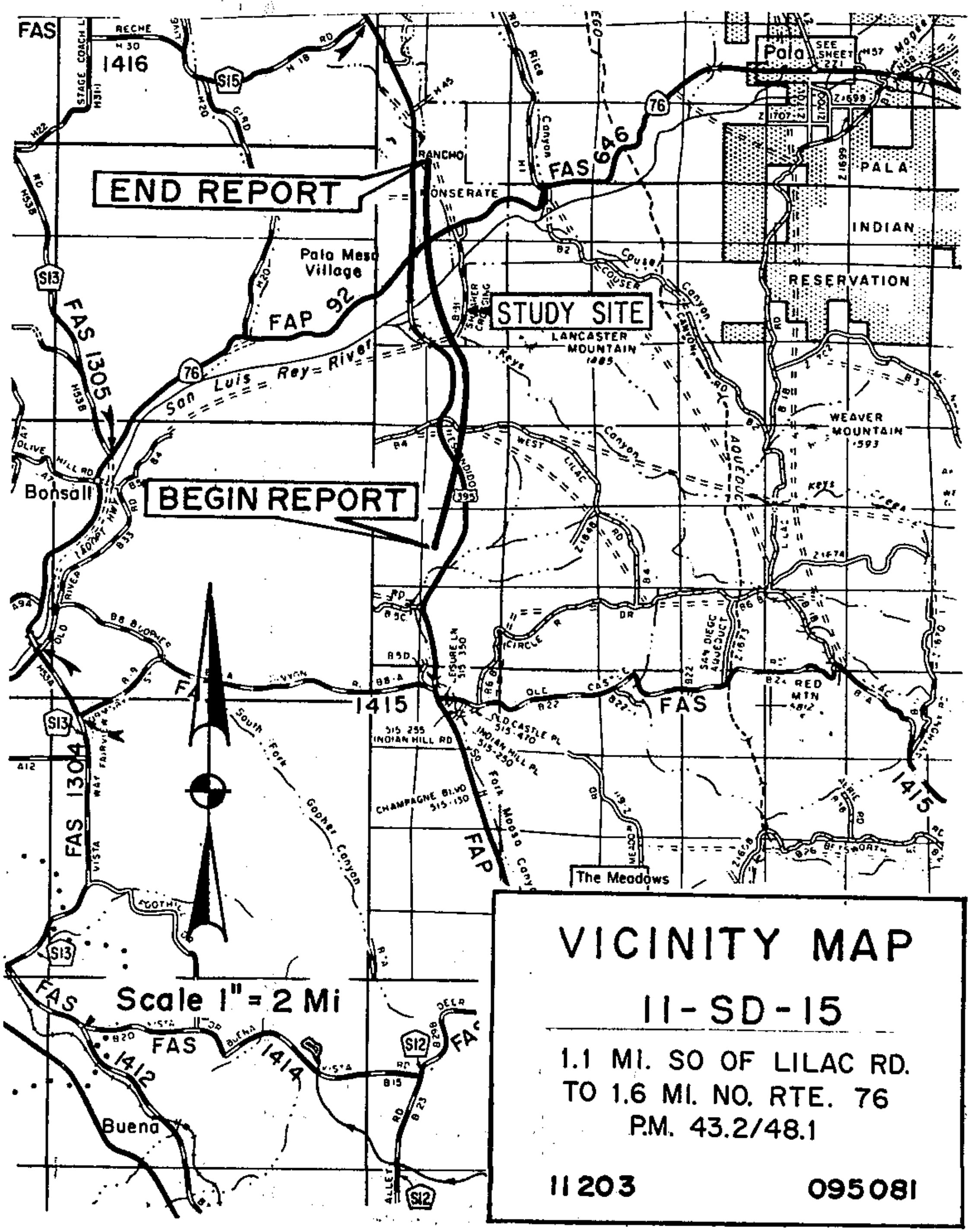

Figure 1, LOCATION OF STUDY SITE
WITHIN LIMITS OF CONTRACT

SCOPE

The investigation included a detailed study of potential liquefaction and seismic densification of subsurface soils, and evaluation of the dynamic stability of a proposed 169 foot high roadway embankment.

Local and regional seismicity and geology were investigated and earthquake motions peculiar to Southern California were studied for site effects. The essential elements of the study involved:

1. Statistically analyzing the seismicity of the area and estimating maximum site bedrock accelerations for a maximum probable earthquake during a design life of 50 years and for a maximum credible earthquake.

2. Investigating susceptibility to liquefaction of foundation soils subjected to the anticipated earthquake intensity levels.

3. Estimating embankment damage potential for the anticipated earthquake intensity levels.

4. Making recommendations pertaining to foundation and embankment treatments to minimize earthquake hazard for a maximum probable event.

CONCLUSIONS

1. The seismicity study indicated that the study area (a 60 mile radius around the site) has a recurrence frequency interval similar to Southern California as a whole. A maximum probable site bedrock acceleration of 0.25g will occur once every 90 years

and a maximum credible site bedrock acceleration of 0.39g will occur once every 300-400 years. These levels of intensity are referred to respectively as: (1) a maximum probable event, and (2) a maximum credible event.

2. Seismically induced ground motion at the site would be the most intense as a result of rupture on the Elsinore Fault, some ±9 miles distant from the site. Richter magnitudes of 6.0 and 7.0 on the Elsinore Fault are considered equivalent to the maximum probable event and maximum credible event, respectively.

3. Results of a finite element analysis of embankment behavior under dynamic loading indicated negligible damage for the maximum probable event and potential slope failures with likely road closure for a maximum credible event.

4. A computerized wave propagation solution (SHAKE 3) of the foundation soils indicated liquefaction does not appear likely under the maximum credible event. Localized liquefaction, however, outside the embankment area may develop and in turn propagate to the embankment toe resulting in potential toe failure.

All liquefaction testing and analyses assumed isotropic stress conditions and existing overburden conditions (no embankment), hence results for the purposes of embankment behavior are conservative.

RECOMMENDATIONS

From information contained in the Preliminary Materials Report, Supplemental Materials Report, and information developed by this study, the essential points for discussion are as follows:

1. Blanket rolling of the foundation as recommended in the Preliminary Materials Report will not aid significantly in reducing settlement for the anticipated fill height of 169 feet. However, the resulting strength increase for initial embankment construction would be beneficial for a rapidly placed fill. Due to the soft nature of the upper foundation soils, it is suggested that heave stakes be placed and observed during filling operations. Also, placement of several piezometers at the 15 foot depth on centerline will enable a closer observation of excess transient pore pressures during embankment placement and assist determination of a safe rate for placement. Permeability rates may be sufficiently high to allow rapid construction, but this should be substantiated. Excess pore pressure values less than 50 percent of overburden pressures should maintain adequate static stability. Blanket rolling will have little effect on dynamic stability.

2. The analysis indicated the proposed placement of a 15 foot thick permeable rock zone in the base of the fill in lieu of embankment soil will not materially reduce the liquefaction potential of subsurface soils. Placement of the rock zone will serve purposes other than countering liquefaction, such as drainage for high river stages or resistance to wave action.

3. Slope stability under dynamic loading is adequate for soil strength parameters as discussed in the text. If, however, material of a cohesive nature is encountered during excavation, added stability of the embankment slope can be realized by blending the cohesive soil with the decomposed granite for the outer 15 or 20 feet. A blended soil should have a minimum cohesion value of 800 psf and Ø no less than 28 degrees.

SITE CONDITIONS

The geologic and seismic setting as described below was derived, in part, from the District 11 Materials Report and Supplemental Material Memo of May 13, 1976. This information will update the reader on terrain, depositional characteristics, fault locations, and potential earthquake magnitude.

Geological Setting

The project is in valley and ridge terrain on the western edge of the predominantly granitic Peninsular Ranges. Project materials consist mostly of disintegrated granite and granitic rock with lesser amounts of metamorphic rock, compacted sediments, alluvium, slope wash and soils.

Bonsall Tonalite, a medium-grained granitic rock with abundant inclusions of older, darker, softer metamorphic material, outcrops occasionally between Stations 2070 and 2155+. Much of the material has altered to decomposed granite, and much of the rock is broken and weathered.

The Indian Mountain Leucogranodiorite outcrops between Stations 2070 and 2220±. It is a hard, light-colored granitic rock, or dike rock, also with darker inclusions. Dikes are up to 20 ft. thick. Decomposed granite outcrops over much of the area between Stations 2070 and 2220 and locally between 2250 and 2320±. It ranges from a few feet to over 80 feet in thickness consisting primarily of gravelly to silty sand with some hard floaters.

River terrace deposits, consisting of compacted gravelly-silty to clayey sands over 30 feet thick, occur from Station 2185 to

2205+. Unconsolidated alluviums from one to ten feet thick occur in smaller tributary valleys throughout the project. The materials vary from bouldery or gravelly sand to sandy clay. Flood plain and channel deposits of the San Luis Rey River occur between Stations 2209 and 2253±. They consist of from 9 to over 72 feet of loose to compact gravelly to silty sands with a few local zones of silt, silty clay, sandy clay and clayey sand. Residual soils are one to 15 feet in thickness and vary from gravelly sand to clay with abundant cobbles and boulders.

The dominant regional fault pattern is comprised of a series of northwest-trending faults. Of these, the Elsinore, San Jacinto, San Andreas, and Rose Canyon Faults are considered most capable of producing relatively high seismic accelerations within the above project limits. These faults are located ±9 miles northeast, 32 miles northeast, ±55 miles northeast, and 20 miles southwest of the project, respectively.

Seismic Setting and Activity

Southern California lies on the Circum-Pacific Belt, one of the most seismically active zones of the world. The Southern California region has been subjected to at least 31 earthquakes of Richter magnitude 6.0 or greater during the period of 1912-1975. At least 3 earthquakes may have approached 8.0 or greater since 1917.

No evidence to date has been uncovered to indicate activity in the Southern California region is changing from established patterns, and specifically within the general vicinity of the seismic study area encompassing the project site.

This study area, a 60-mile radius around the site, is compared to Southern California seismicity as a whole on Figure 2. In general, the study area appears to have the same or slightly higher activity than the entire Southern California region for magnitude ranging from 4 to 7. No earthquakes greater than 7.0 were recorded within the study area between the years of historical study.

Specific data for Route 15 was combined with the Allen and Housner data and the average utilized in estimating frequency intervals of various bedrock acceleration levels as illustrated on Figure 3.

The return periods utilized a probability concept that assumes all locations within the study area will be vulnerable to the expected range of bedrock acceleration levels.

Maximum bedrock accelerations (0.35 to 0.4g) at the site are expected to be generated by the Elsinore Fault due to its proximity (9± miles). Other fault systems are not expected to produce ground motion as intense as the Elsinore due to their relative remoteness from the project site. Table I lists the major faults within a 70-mile radius of the site and expected magnitude and bedrock accelerations.

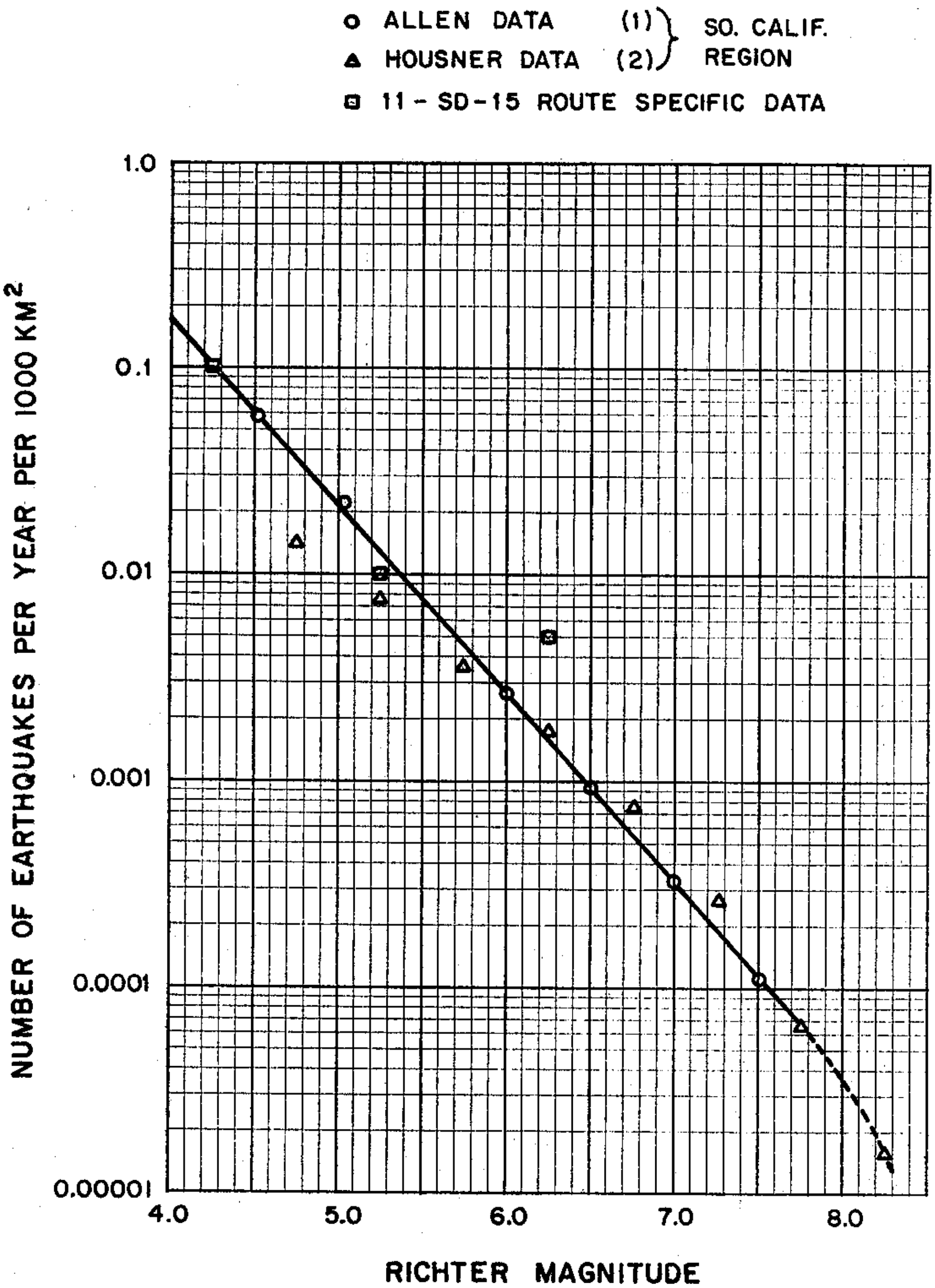

Figure 2, NUMBER OF EARTHQUAKES PER YEAR PER 1000 KM^2 EQUAL TO OR GREATER THEN THE GIVEN MAGNITUDE

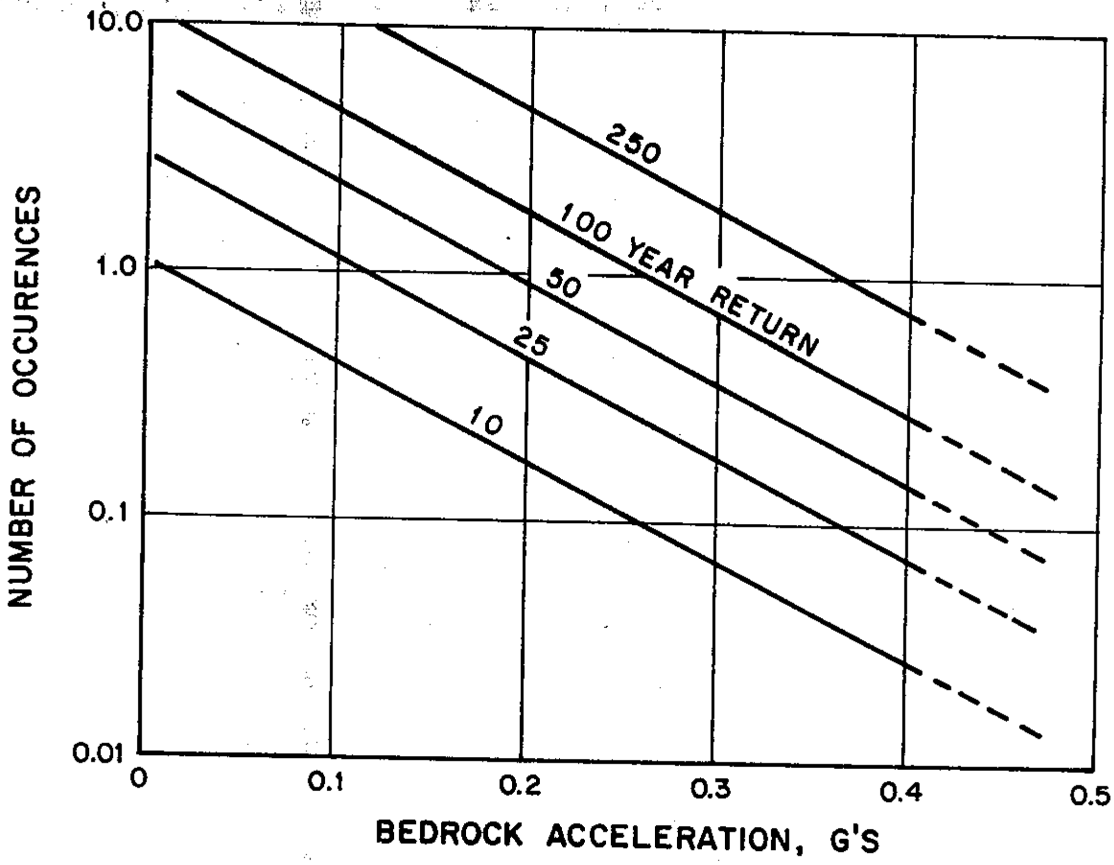

Figure 3, OCCURENCES OF BEDROCK ACCELERATIONS WITHIN SAN DIEGO ROUTE 15 STUDY AREA.
(STUDY AREA ENCOMPASSES FAULT ACTIVITY WITHIN 60 MILE RADIUS OF PROJECT SITE).

TABLE I

Tabulation of Large Fault System maximum potential magnitude, and associated site bedrock accelerations.

FAULT	FAULT DISTANCE TO SITE (Mile)	MAXIMUM CREDIBLE EVENT (Richter Magnitude)	MAXIMUM CREDIBLE SITE BEDROCK ACCELERATION (g's)
ELSINORE	8-10	7.0	0.39
ROSE CANYON	20	7.0	0.21
SAN JACINTO	32	7.5	0.18
LA NACION	35	6.8	0.10
SAN ANDREAS			
(Central Segment)	55	8.3	0.12
(South Segment)	55	7.5	0.08
SAN CLEMENTE	70	7.7	0.07

Information on Table I was obtained from the Materials Report Supplemental Memo. From the data contained on Figures 2 and 3 and Table I, it is reasonable to assume that a maximum earthquake (Richter magnitude on the Elsinore Fault of M=7.0±) will occur once every 300-400 years, an M=6.0± once every 75-100 years, and an M=5.4± once every 15-20 years.

Associated average bedrock acceleration levels at the site are estimated at 0.39, 0.25 and 0.1 g, respectively.

These estimates are statistical and are based on various assumptions. Thus, they must be used with the degree of caution that will yield realistic yet conservative estimates. Since every structure cannot be economically designed for the most destructive seismic event, the statistical approach provides a limit on expected damage potential and reasonable earthquake selection for the design life period.

Soil and Groundwater Profiles

Sample borings were made in June 1976 by District 11 personnel to obtain undisturbed samples for dynamic testing and permit relative density determinations. Standard penetration resistance tests (STP) for estimating relative density measurements were only marginally successful due to the limitations of the field mobil rig. However, three field STP measurements were obtained at depths of 7, 10 and 14 feet. Relative density values were estimated at 65, 40 and 90%, respectively. The boring legend and the boring profile record are illustrated on Figures 4(a) and 4(b) respectively.

Table II illustrates relative density measurements from Office of Structures penetration resistance tests conducted in 1974 at Station 2214 using a 1 inch diameter probe and a 25 lb. hammer with an 18 inch drop. Comparison of these data for the upper 29 feet to equivalent standard penetration resistance test values are made based on the Bridge Department's Engineering Geology Handbook. Results indicate loose soils to 13 feet with estimated relative densities of 30%. The silt-clay content is quite high, hence results are probably more indicative of consistency or softness than relative density. From 13-20 feet relative densities average 50% with the exception of a 1-foot thick strata between the 16-17 foot depth, where relative densities are estimated at 60-70% and indicate the boundary of the more competent engineering zone.

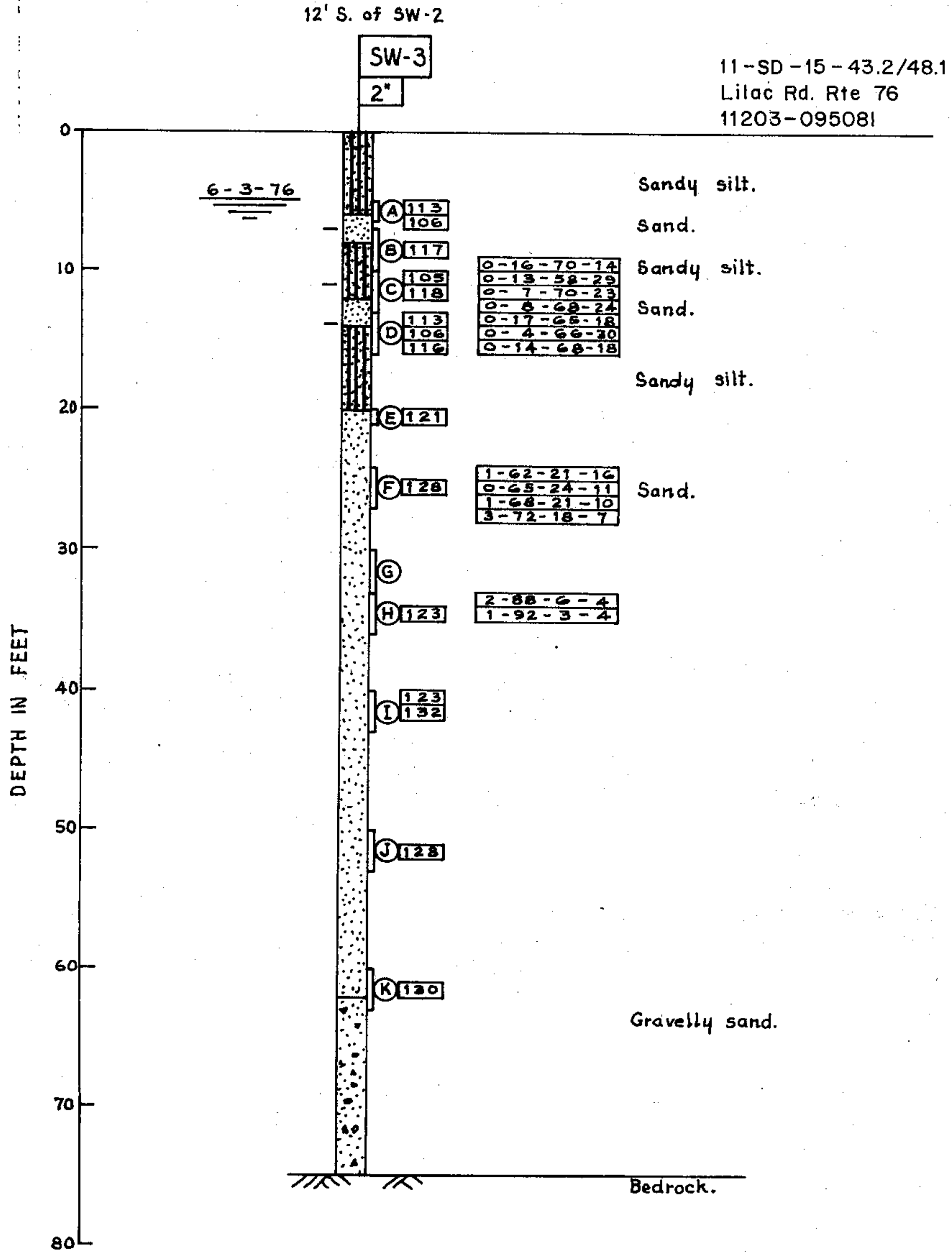

Figure 4(b) BORING RECORD OF HOLE SW-3

CROSS-SECTION & PROFILE SHEETS

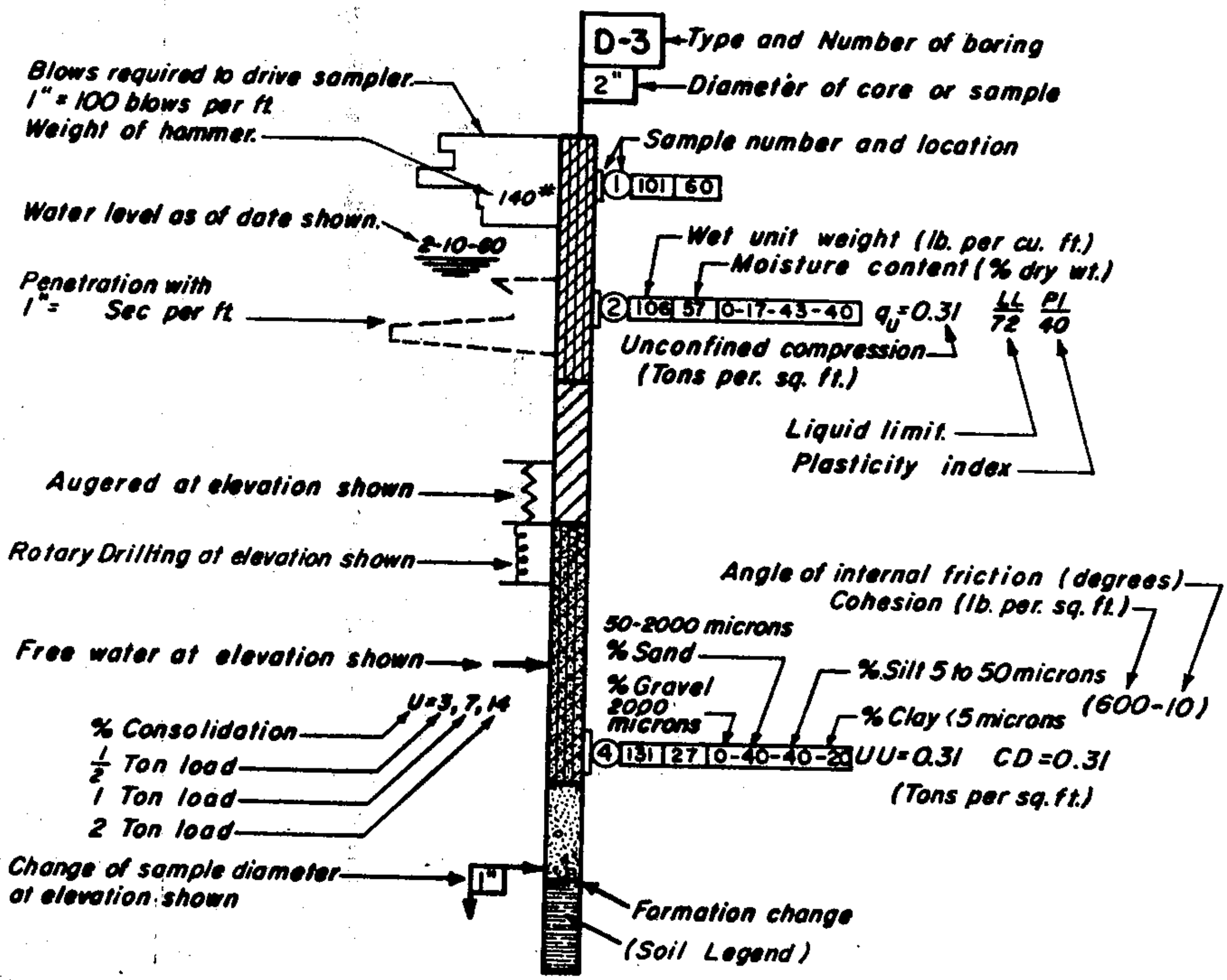

Figure 4(a) BORING LEGEND

TABLE II

Estimation of field relative densities using 1-inch probe penetration test results.

DEPTH (Feet)	1" PROBE (BPF)	STANDARD PENETRATION EQUIVALENT*	RELATIVE DENSITY (%)
0-13	12-34	5	30
13-16	54-68	5-10	40-60
16-17	86		70
17-20	46-66	5-10	40-60
20-29	88-143	10-15	60-70

*From Bridge Department Engineering Geology Handbook.

Subsurface soils, descriptions of which are noted on Figure 4b, consist of soft sandy-silt, clayey-silt, and loose silty sand mixtures to a depth of approximately 26 feet. Below 26 feet to a depth of approximately 63 feet, clean dense sands predominated. From 63 to 75 feet (bedrock) sandy-gravel was assumed based on the drilling difficulty encountered at these depths.

Undisturbed 2-inch California samples were obtained where possible, for moisture-density, gradation, and laboratory triaxial cyclic loading tests. Results of the moisture-density and gradations are also recorded on the soil boring profile, Figure 4b. Static triaxial strength tests and compressibility characteristics were not performed. However, it is conjectured that the soft nature of the upper 20-25 feet could produce instability and compressibility problems if rapidly loaded. The high sand-silt contents and pervious boundaries, however, should promote drainage and full soil strength should be mobilized soon after embankment construction.

Groundwater at the site was at a depth of approximately 5 feet during the exploration in early June, 1976. Since the site is located in the flood plain area of the San Luis Rey River, groundwater levels are assumed to fluctuate seasonally and possibly rise periodically to ground surface.

Geophysical Surveys

Subsurface seismic surveys were performed for estimating damage potential of various design earthquakes, and to obtain shear wave velocities for use in computerized solutions of ground surface motions.

For the first 20 feet, crosshole shear wave velocities were obtained using a high speed recorder to record energy waves created by a 2-inch diameter closed tube being driven into the soil using a 140 lb. hammer. Two holes were utilized, one for the geophone or energy pickup, the other hole approximately 15 feet away from the energy source or hammer driven tube. Beyond 20 feet of depth, the soil was too dense for utilization of the closed tube, hence, a four-bladed drag bit was substituted as the wave source generator.

Shear wave velocities were of excellent definition almost down to bedrock, or a depth of approximately 75 feet.

Figure 5 illustrates the measured shear wave velocities (broken lines). The soil profile is also shown to illustrate material type.

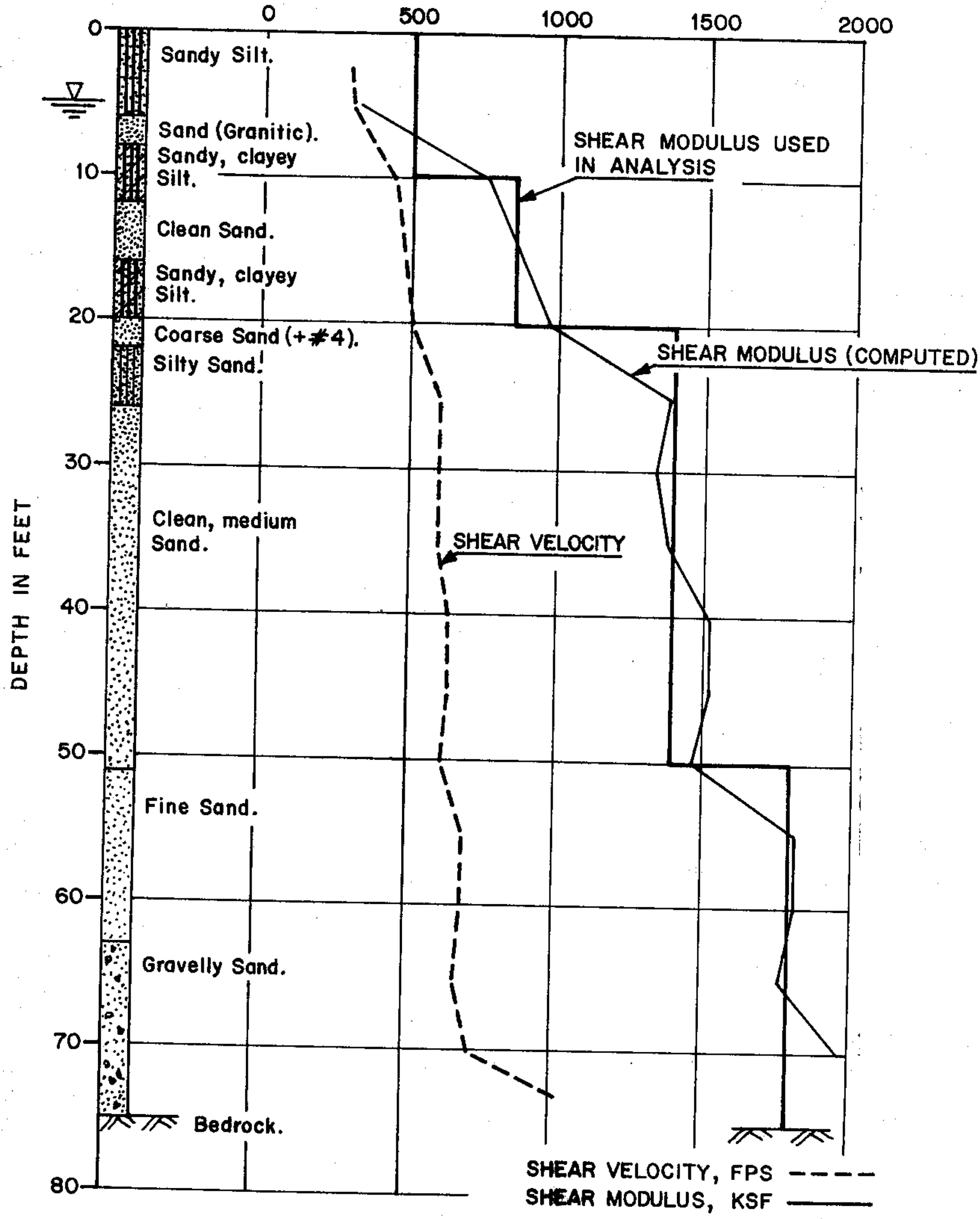

Figure 5, ILLUSTRATION OF SOIL PROFILE AND CORRESPONDING SHEAR WAVE VELOCITIES. SOLID LINES ARE THE SHEAR MODULUS VALUES BASED ON THE SHEAR VELOCITY

LABORATORY STATIC AND DYNAMIC TESTS

Dynamic Tests

A number of the 2-inch undisturbed California samples of the foundation material were tested for liquefaction susceptibility by subjecting them to various cyclic stress loading conditions. All specimens were saturated by inundation, then backpressured to 3 K_g/cm^2 (KSC), and isotropically consolidated to the field effective confining pressure within a triaxial compression apparatus.

Symmetric pulsating deviator stress levels were applied until liquefaction or 20% axial strain developed (failure). Results of the cyclic loading tests are illustrated on Figure 6.

Figure 6 illustrates the corrected normalized shear stress levels versus number of cycles required to produce failure. The three predominant foundation soil types to 38 feet in depth are depicted along with their generalized depth intervals. Samples below 38 feet were considered less susceptible to liquefaction than the overlying soil due to greater relative densities and greater effective overburden pressures and therefore not tested.

Dynamic test results indicated the sandy-silts and silty sands to 26 feet did not liquefy but failed by large strain appplitudes. The strain amplitude (sum of compressive and extensive strain) of 20% was considered failure for the purposes of this analysis. Below 26 feet, the medium sands exhibited much greater strengths even though failure was induced by liquefaction.

Static Tests

For estimating the stress-strain, modulus and damping characteristics of the proposed embankment material, several triaxial

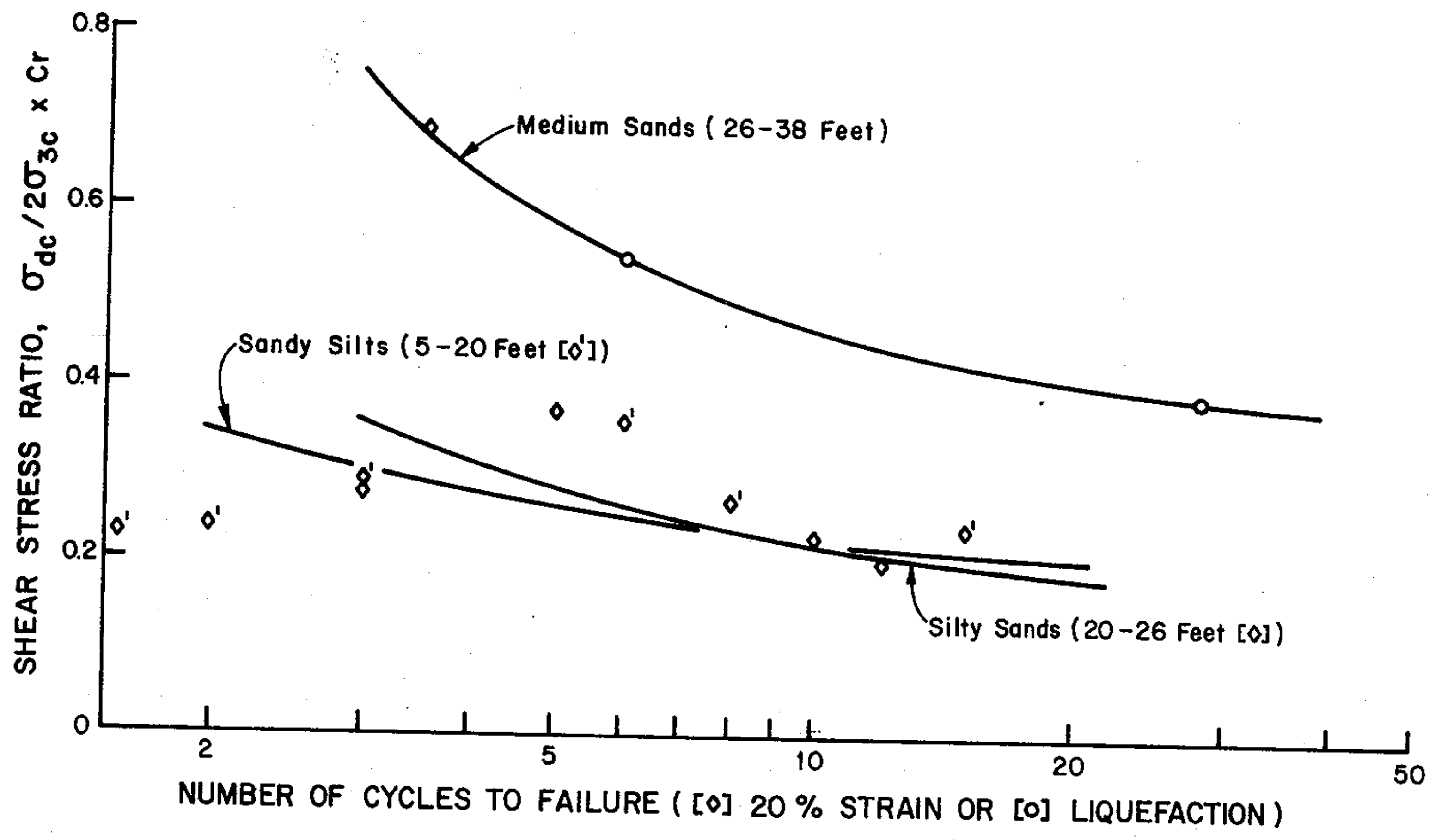

Figure 6, ILLUSTRATION OF THE CORRECTED NORMALIZED SHEAR STRESS RATIO VERSUS NUMBER OF CYCLES TO PRODUCE FAILURE tests of the borrow were compacted to 90% relative compaction using Test Method Calif. 216. The soil is a silty sand (decomposed granite) and native to the Route 15 site. Static triaxial tests were conducted in a consolidated-drained state, unsaturated and stressed to 20% strain. Mohr failure envelopes indicated an average c of 500 psf and a Ø angle of 34 degrees. Figure 7(a) illustrates the peak normal and shear stress at failure (alpha envelope).

Figure 7(b) illustrates the initial tangent modulus values necessary for input into the computer to estimate the static stress state within the embankment.

A remolded sample was also tested in the resonant column apparatus to estimate the initial shear modulus necessary in the dynamic evaluation. Results were not commensurate with known values of similar soils and densities, therefore shear modulus and damping characteristics were estimated by soil type, and densities recorded in currently available literature.

STUDY RESULTS

Earthquake Selection

In order to evaluate the effects of seismic excitation, several digitized earthquake motions were chosen from a selection of recorded earthquake motions in the Southern California region. The two earthquake motions selected, Castaic and San Fernando, where bedrock recordings having periods and accelerations near that needed for our study.

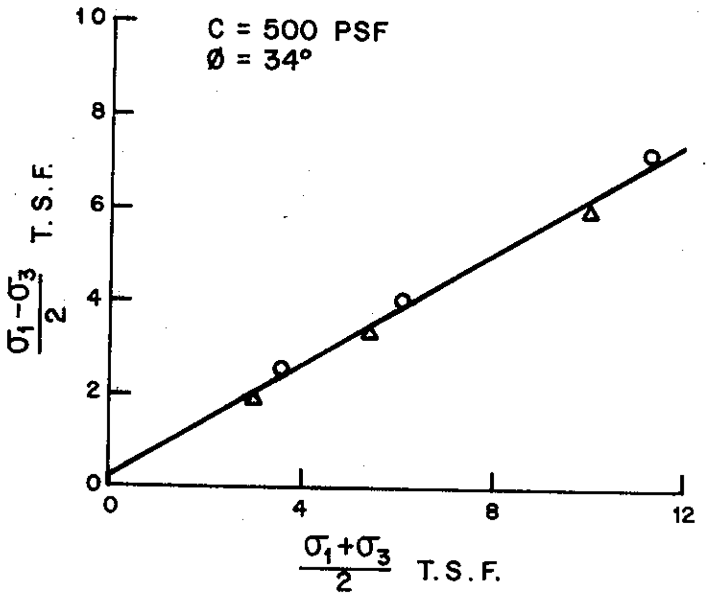

Figure 7(a), FAILURE ENVELOPE FOR EMBANKMENT SOILS

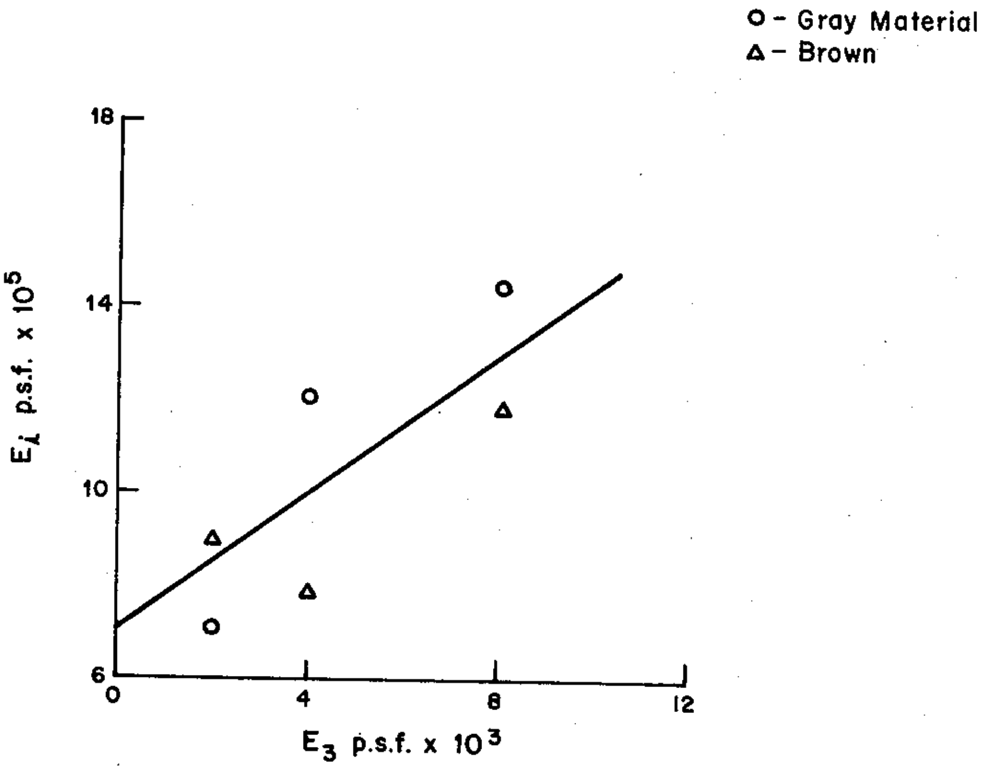

Figure 7(b), INITIAL TANGENT MODULUS VERSUS CONFINING PRESSURE FOR EMBANKMENT SOILS

The earthquake periods and accelerations were modified, as required, to conform to specific site bedrock values for a rupture of magnitude 6.0 on the Elsinore Fault. The associated site bedrock acceleration was estimated at 0.25g (probability of one every 90 years) and was selected as the maximum probable the site would experience in a 50-year design life. Analysis of fault ruptures for faults other than the Elsinore were not investigated due to their minor influence at the site as compared to ruptures on the Elsinore Fault (see Table I). Table III lists the two earthquakes used in the computer analysis of site damage potential and their recorded predominant period and maximum acceleration and the adjusted predominant period and maximum acceleration which is defined as the maximum probable earthquake.

A maximum credible earthquake was also investigated using a simplified procedure, in conjunction with a computer analysis. This event, a magnitude 7 earthquake, on the Elsinore Fault (Table I) has an expected return period of once every 300-400 years. Maximum site bedrock acceleration that can reasonably be expected is 0.39g.

TABLE III

Tabulation of recorded and adjusted earthquake period and acceleration values. Adjusted values represent site response for rupture on Elsinore Fault equivalent to a Richter magnitude 6.0.

EARTHQUAKE	RECORDED		ADJUSTED	
	PERIOD	ACCELERATION	PERIOD	ACCELERATION
Castaic	.30 sec.	0.27g	.35 sec.	0.25g
San Fernando	.15 sec.	0.55g	.35 sec.	0.25g

Ground Motion Analysis

Most current methods for determining the effects of local soil conditions on ground response during an earthquake are based on the upward propagation of shear waves from the underlying rock foundation. Analytical procedures which incorporate non-linear soil behavior have yielded results in general agreement with field observations, particularly at sites where bedrock depth changes gradually over a large area. The development of these procedures has been extremely valuable to engineers in that it provides an analytical method for estimating the amplitude and frequency characteristics of ground surface motion at a specified site prior to actual seismic excitation.

The ground response investigation presented herein extensively utilized the computer program, "SHAKE 3" (1). The program, developed by Schnabel, Lysmer, and Seed, computes the response associated with the upward propagation of shear waves through a system of homogeneous, viscoelastic layers of infinite horizontal extent. The basis of the program is the continuous solution to the wave equation adopted for use with transient motions through the Fast Fourier Transform Alogarithm. The nonlinearity of the shear modulus and damping is accounted for by the use of equivalent linear soil properties using an iterative procedure to obtain strain values compatible with the modulus and damping.

The two earthquake records, modified to the specified design values, were used as the bedrock motions applied to the soil profile (Figure 8) representing the general site soil conditions. Output motion data included peak accelerations at ground surface and at soil layer boundaries and soil layer stress-time histories at 12.5 feet and 25 feet.

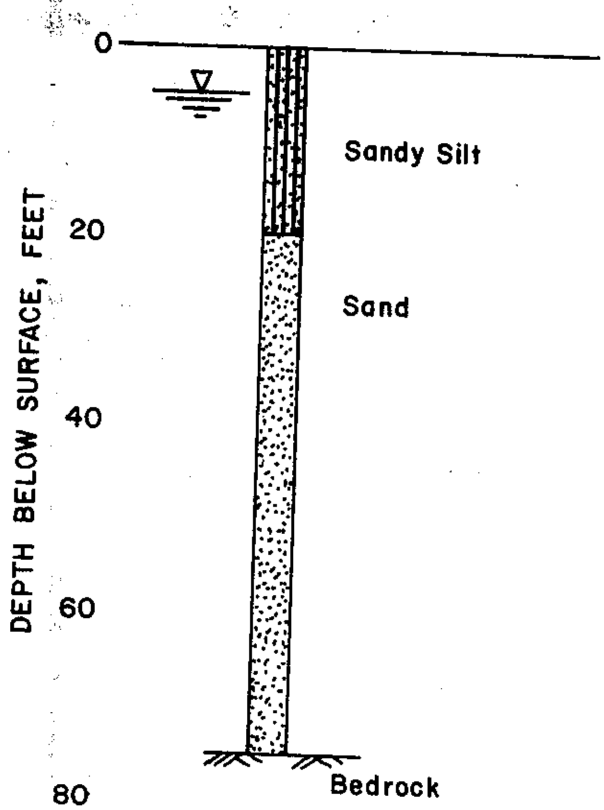

Figure 8, SOIL PROFILE USED FOR GROUND MOTION ANALYSIS

Computer analysis indicated final soil periods for the Castaic and San Fernando Quakes of 1.3 and 1.1 seconds, respectively. The input bedrock acceleration level of 0.25g was amplified slightly at ground surface to 0.26g for the Castaic and attenuated at ground surface to 0.21g for the San Fernando.

Liquefaction and Seismic Densification

Maximum Probable Event

Utilizing results from the ground motion analysis (no embankment) the shear stress ratio (maximum average shear stress divided by the effective overburden pressure, $\tau_{av}/\bar{\sigma}_o$) was estimated from the shear stress time histories of the two earthquakes.

The maximum average shear stress was taken as the computed average of the four highest shear stress pulses for the Castaic earthquake and the ten highest stress pulses for the San Fernando earthquake. These averages and their associated shear stress pulses were then compared to the laboratory cyclic test results at the same number of stress pulse cycles.

Figures 9 and 10 illustrate the soil profile, maximum ground surface acceleration, and the computed shear stress ratios for the Castaic and San Fernando earthquakes, respectively (dashed line) versus the shear stress ratio (solid line) required for failure as determined from the laboratory tests shown previously on Figure 6.

Liquefaction did not develop within the soil depth interval between 5-26 feet and failure was defined as a result of excessive axial strain equivalent to 20%. Liquefaction, however, did develop within the medium sand strata soil depth interval

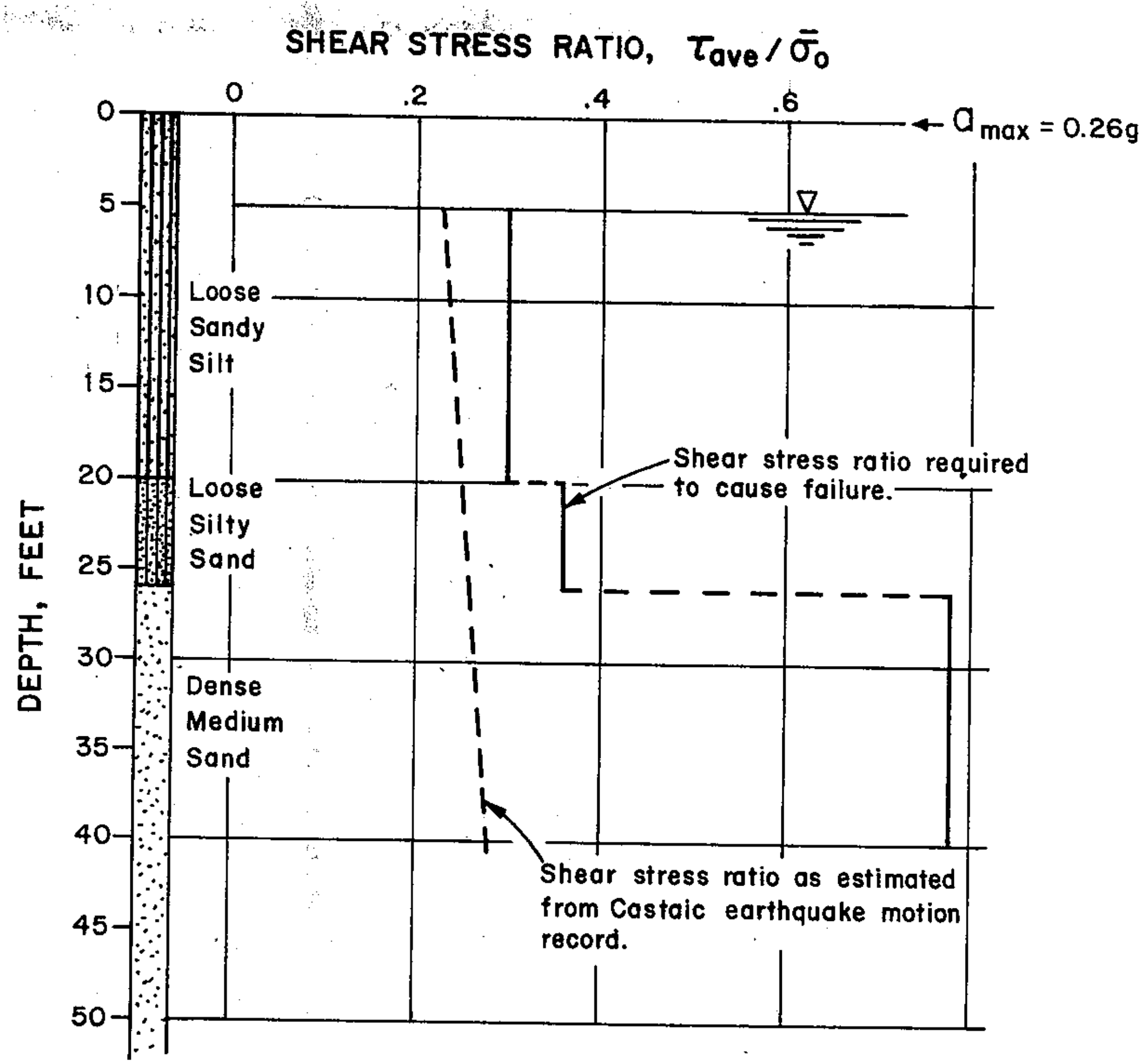

Figure 9, COMPARISON OF COMPUTERIZED SHEAR STRESS RATIO FOR CASTAIC EARTHQUAKE TO SHEAR STRESS RATIO REQUIRED TO PRODUCE FAILURE

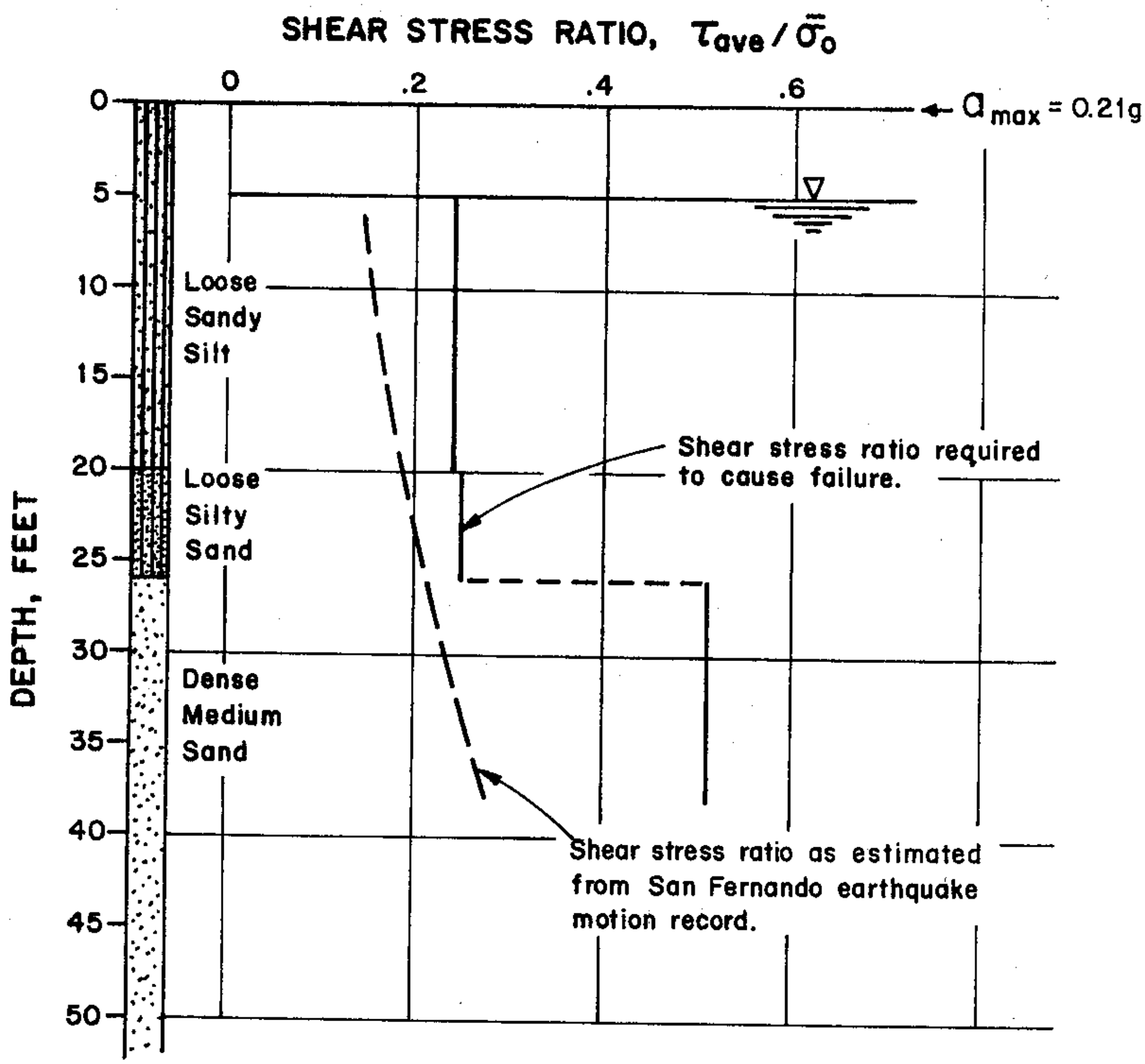

Figure 10, COMPARISON OF COMPUTERIZED SHEAR STRESS RATIO FOR SAN FERNANDO EARTHQUAKE TO SHEAR STRESS RATIO REQUIRED TO PRODUCE FAILURE between 26-38 feet, at an average axial strain equivalent to approximately 10%. Due to the greater relative density of the medium sand, significantly greater stress ratios were required to induce failure conditions in this strata than the less dense soils above.

Thus, Figures 9 and 10 indicate that the postulated design earthquake will not induce liquefaction for the depth interval 5-38 feet. Susceptibility to liquefaction below this depth is not anticipated due to the effect of greater overburden and relative densities similar to or greater than the medium sand deposit.

Figure 11 was developed to illustrate the theoretical field shear stress ratio as computed based on Seed and Idriss's simplified procedure (2). The results are slightly more conservative than those estimated from the two design earthquakes but still does not exceed stress ratio levels necessary to produce failure conditions or liquefaction.

For estimating seismic densification, the grain size, relative density, and pore pressure ratios, were utilized and settlement was based on information contained in Figure 12. Cyclic testing of retrieved soil samples from the Route 15 site indiated pore pressure ratios of 0.8 to 0.9 developed for the majority of the tests from 5-26 feet after 3 or 4 cycles. Grain size analysis indicated D_{50} sizes of 0.02 mm and 0.15 mm for depth intervals of 0-20 feet and 20-50 feet, respectively. Utilizing this information, total seismic settlement is anticipated to be less than 1.5 inches based on an average pore pressure ratio of 0.9, average D_{50} grain size of 0.10 mm and an average relative density of 50%. Maximum depth of affected soil is estimated at 50 feet.

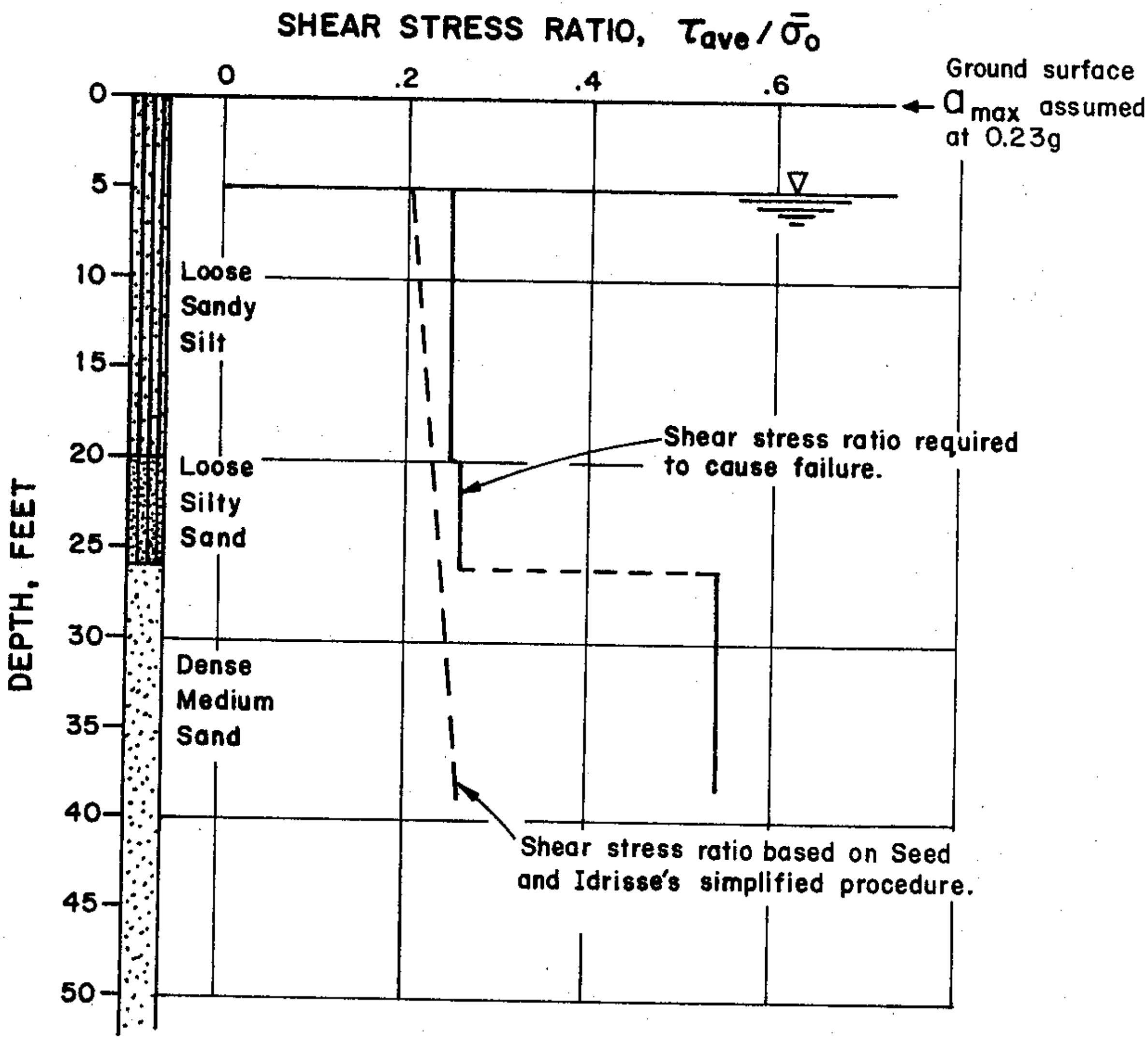

Figure 11, COMPARISON OF SIMPLIFIED FIELD SHEAR STRESS RATIO TO SHEAR STRESS RATIO REQUIRED TO PRODUCE FAILURE

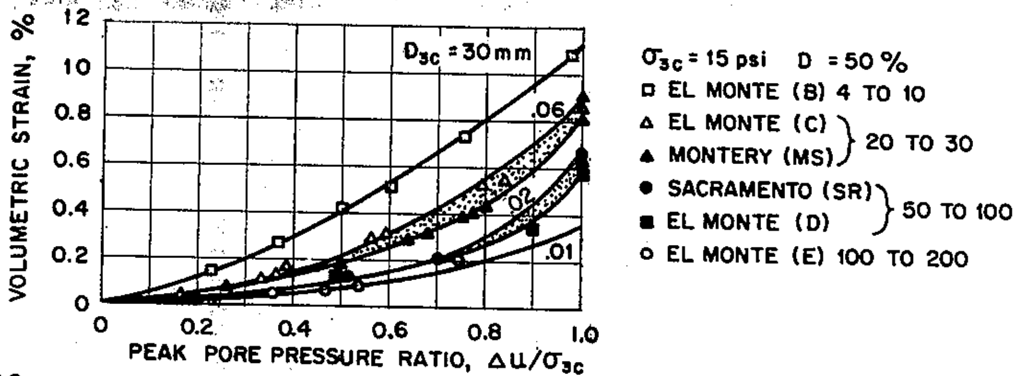

Figure 12a, EFFECT OF GRAIN SIZE ON RECONSOLIDATION VOLUMETRIC STRAIN
(LEE AND ALBAISA - 1974)

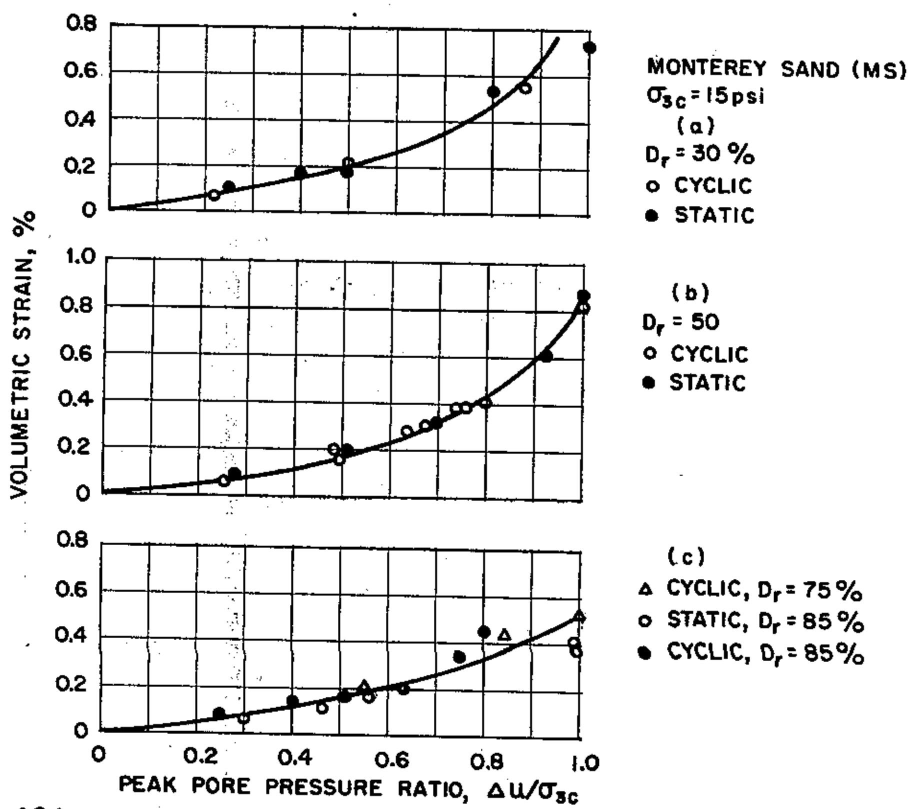

Figure 12b, RECONSOLIDATION VOLUMETRIC STRAIN FOLLOWING STATIC AND CYCLIC PORE PRESSURE INCREASES
(LEE AND ALBAISA - 1974)

Maximum Credible Event

A computerized analysis of the maximum credible event was not conducted. However, extending findings of the maximum probable event enabled some understanding of the damage potential associated with the maximum credible. The maximum credible event, equivalent to a Richter magnitude, M=7.0 on the Elsinore Fault, would have an average site bedrock acceleration of 0.39g or approximately 55% higher than the maximum probable of 0.25g. The maximum credible event was estimated to induce shear stress ratios by the same percentage, hence, causing failure for the upper 26 feet of soil by virtue of excessive straining. Areal liquefaction, which is the more dangerous of the two situations, would still not be produced in the soil depth from 26-38 feet or below.

Seismic densification would not develop excessive volumetric straining much beyond the estimate discussed earlier. Seismic densification is estimated at less than 2 inches.

Slope Stability

Maximum Probable Event

For analyzing embankment stability, a finite element computer method of analysis (FEM) was conducted. Program 4CST and TWIST, capable of determining static stresses and strains for instantansous loading, were utilized. Both drained and undrained conditions can be analyzed for plane stress and plane strain conditions with these programs.

Subsequent to the development of the static stress condition, a dynamic (FEM) analysis was instituted. Stresses and strains under earthquake loading were determined by use of computer program LUSH. Superimposing 65 percent of the peak dynamic stress on the static stress condition enable estimation of the overall stress state. Analysis for elemental failure was determined by comparing the static plus dynamic stresses to the peak allowable stresses as determined from the static triaxial tests on the embankment borrow material. The ratio of the peak stress states (greater than 1.0 for stability) then gave an estimation of embankment stability under dynamic loading.

Figure 13 illustrates the embankment and the numbered elements. The embankment height is 169 feet with a top width of approximately 225 feet and side slope ratio of two to one. The embankment was modeled as a homogeneous fill consisting of a silty sand (DG) with strength parameters, as discussed earlier.

For static stability the foundation material was assumed to fully consolidate to the overburden pressures immediately upon material placement. Although this is an oversimplification, it does characterize soil behavior for the completed structure. Instantaneous load application without the advantage of consolidation resulted in failure of the embankment due to foundation failure within the compressible silts.

The results of the dynamic analysis for the maximum probable earthquake are indicated on the left half of centerline of Figure 14. This portion of the figure illustrates those elements (shaded areas) that were overstressed during the excitation motion. Since overstressing is limited to areas outside the embankment foundation composite, dynamic stability is considered acceptable.

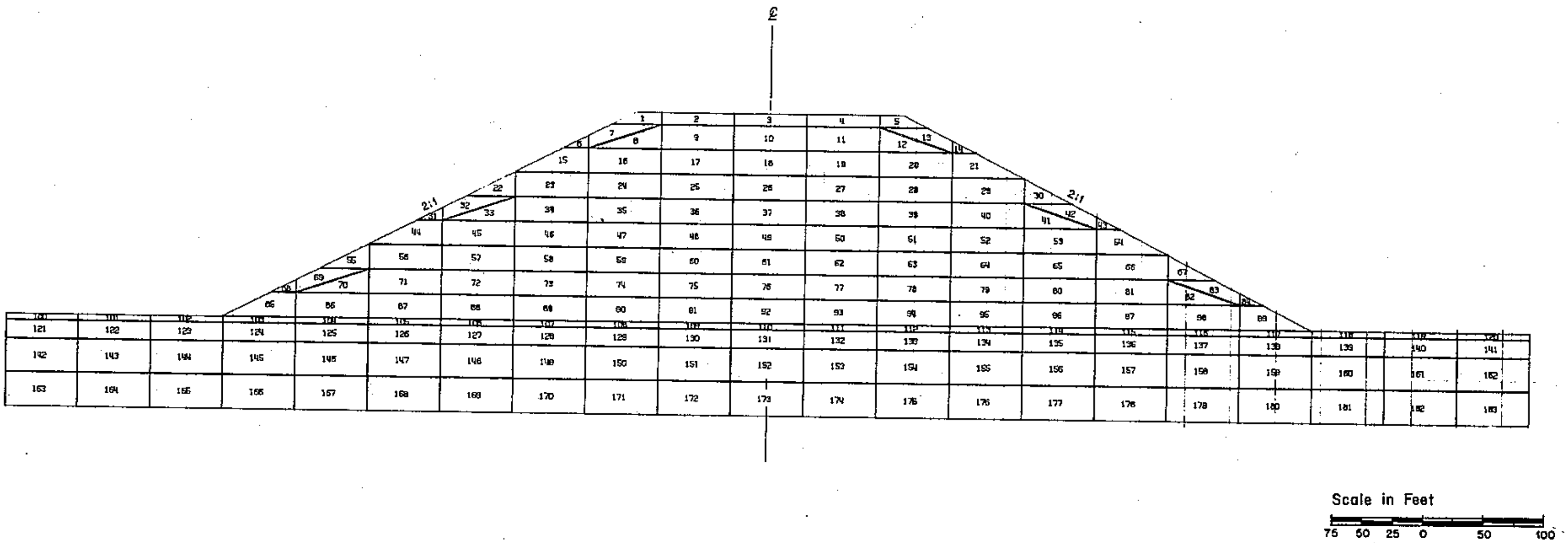

Figure 13, EMBANKMENT AND FOUNDATION GEOMETRY UTILIZED IN FINITE ELEMENT ANALYSIS. NUMBERS ARE FOR ELEMENTAL REFERENCE.

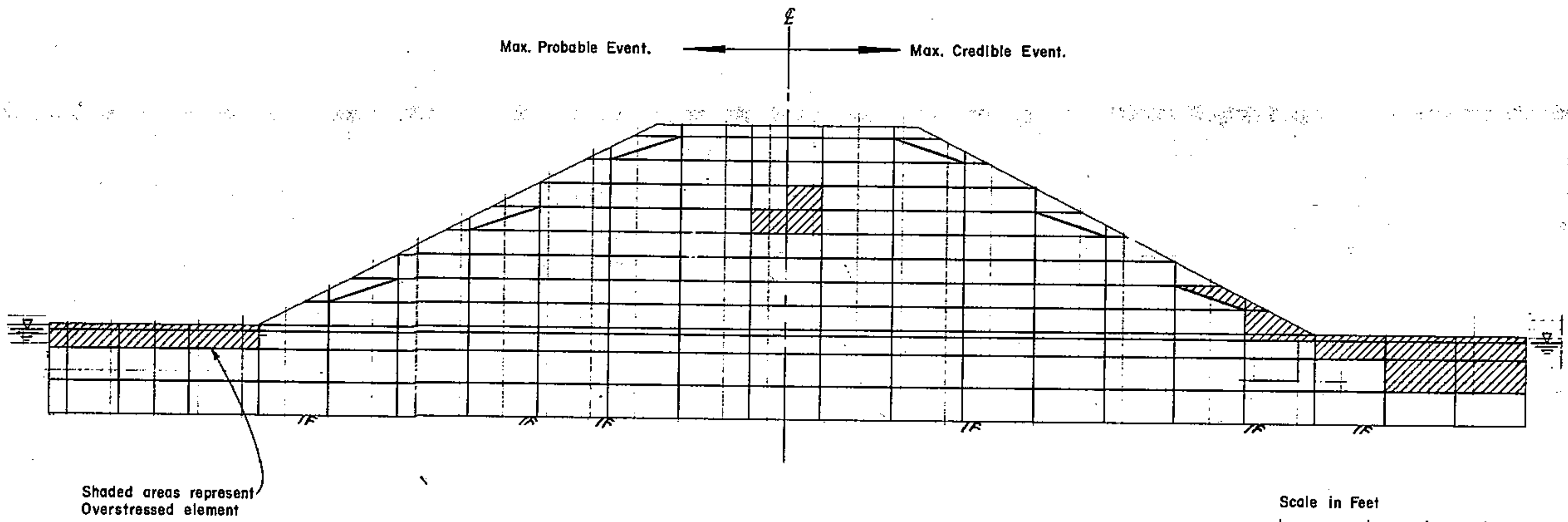

Figure 14, FINITE ELEMENT GRID OF EMBANKMENT AND FOUNDATION ILLUSTRATING ELEMENTS OVERSTRESSED UNDER MAXIMUM PROBABLE EVENT (LEFT HALF) AND MAXIMUM CREDIBLE EVENT (RIGHT HALF)

Maximum Credible Event

Using the known static stress state and increasing bedrock excitations to a maximum of 0.39g, strength ratios for the maximum credible event were estimated. The method of compiling data is identical to that previously discussed under the maximum probable event. The increased dynamic stresses can be easily compared to those for the lesser event (left half) and areas of overstressing identified. It should be pointed out that strength ratios of 0.9-1.0 or so do not always constitute a failure mode. Progressive failure within the embankment is not likely if the overstressed elements are surrounded by elements with strength ratios greater then 1.0.

During this event, additional elemental overstressing occurred near the toe of the embankment as a consequence of high embankment stresses and foundation failure. This, however, appears confined to the region around the toe and thus does not appear detrimental to the integrity of the structure as a whole.

Additional information on the dynamic response of the elements is presented on Figures 15a, b and c. These figures illustrate the dynamic shear stress-time history of motion for elements 10, 92 and 145 respectively during the earthquake record typifying the maximum credible event. Peak stress used in the analysis is the maximum recorded shear stress value recorded during any portion of the record. For this particular record, maximum values were obtained early in the motion and assigned an arbitrary negative sign due to the excursion following below the zero event line. The absolute value of the peak shear stress is then utilized in developing stress-strength ratios for estimating overstressed elements.

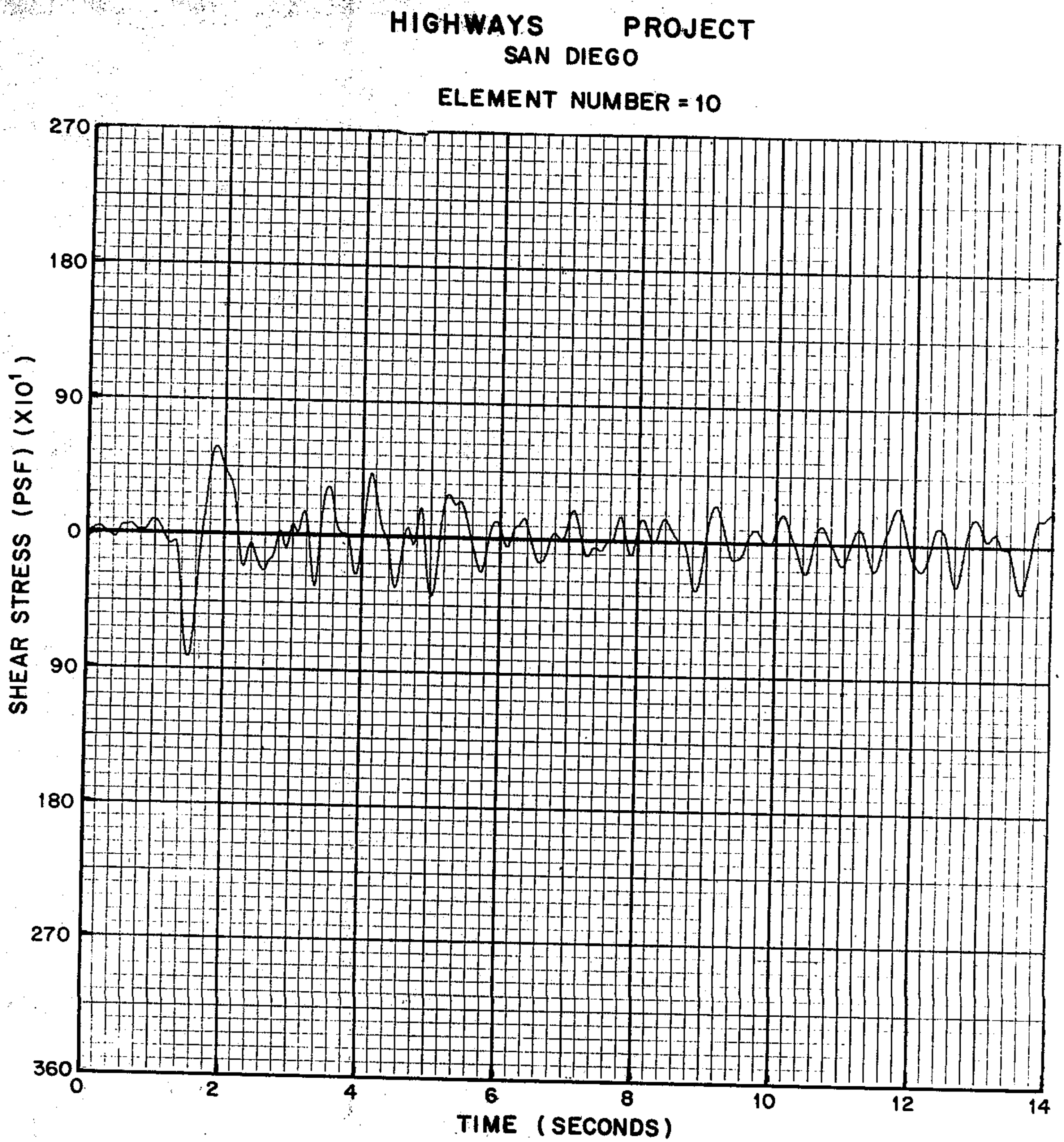

Figure 15 (a) SHEAR STRESS-TIME HISTORY OF MOTION FOR ELEMENT 10 DURING MAXIMUM CREDIBLE EVENT.
(DYNAMIC STRESS LEVELS ONLY)

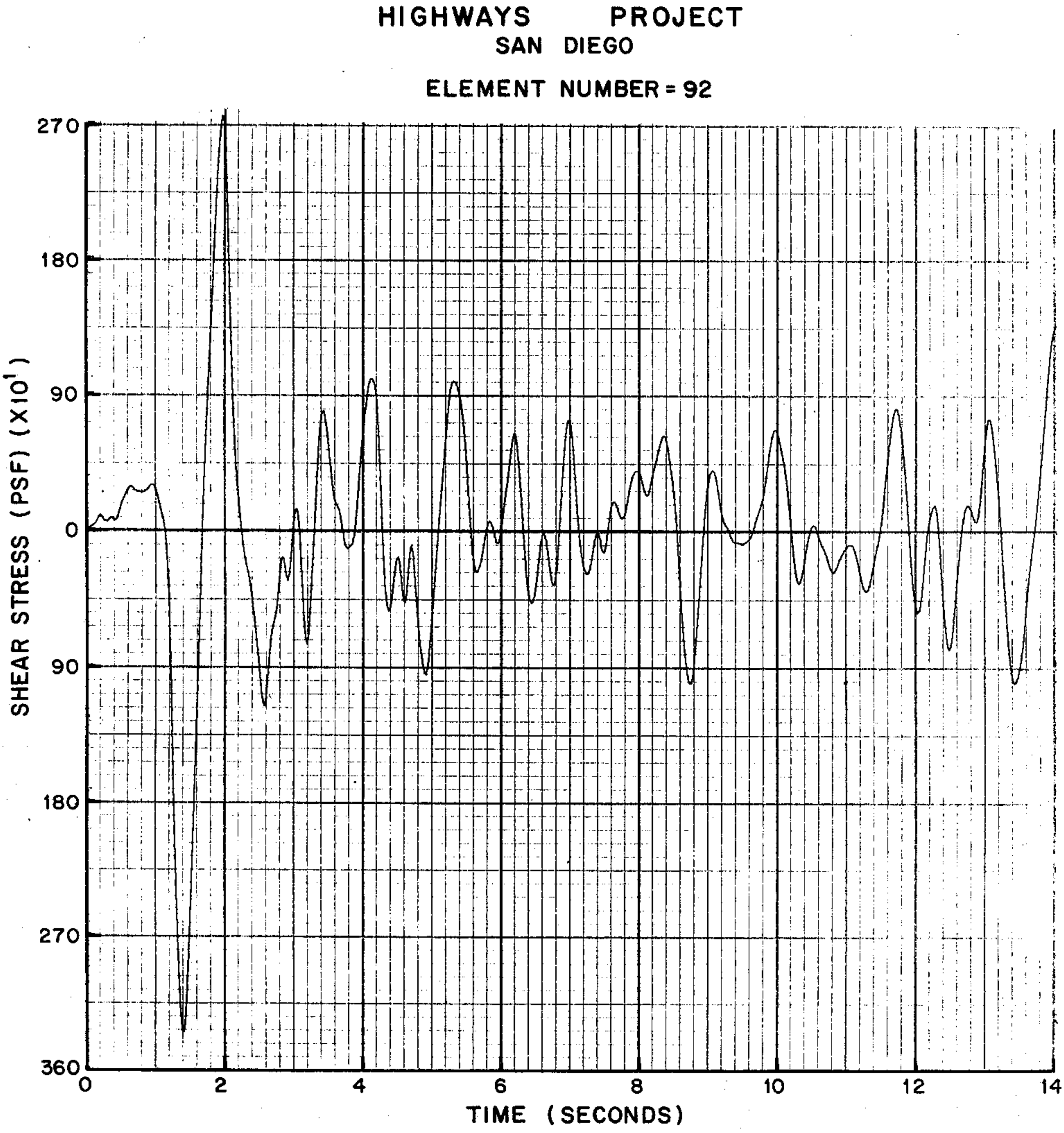

Figure 15 (b) SHEAR STRESS-TIME HISTORY OF MOTION FOR ELEMENT 92 DURING MAXIMUM CREDIBLE EVENT. (DYNAMIC STRESS LEVELS ONLY)

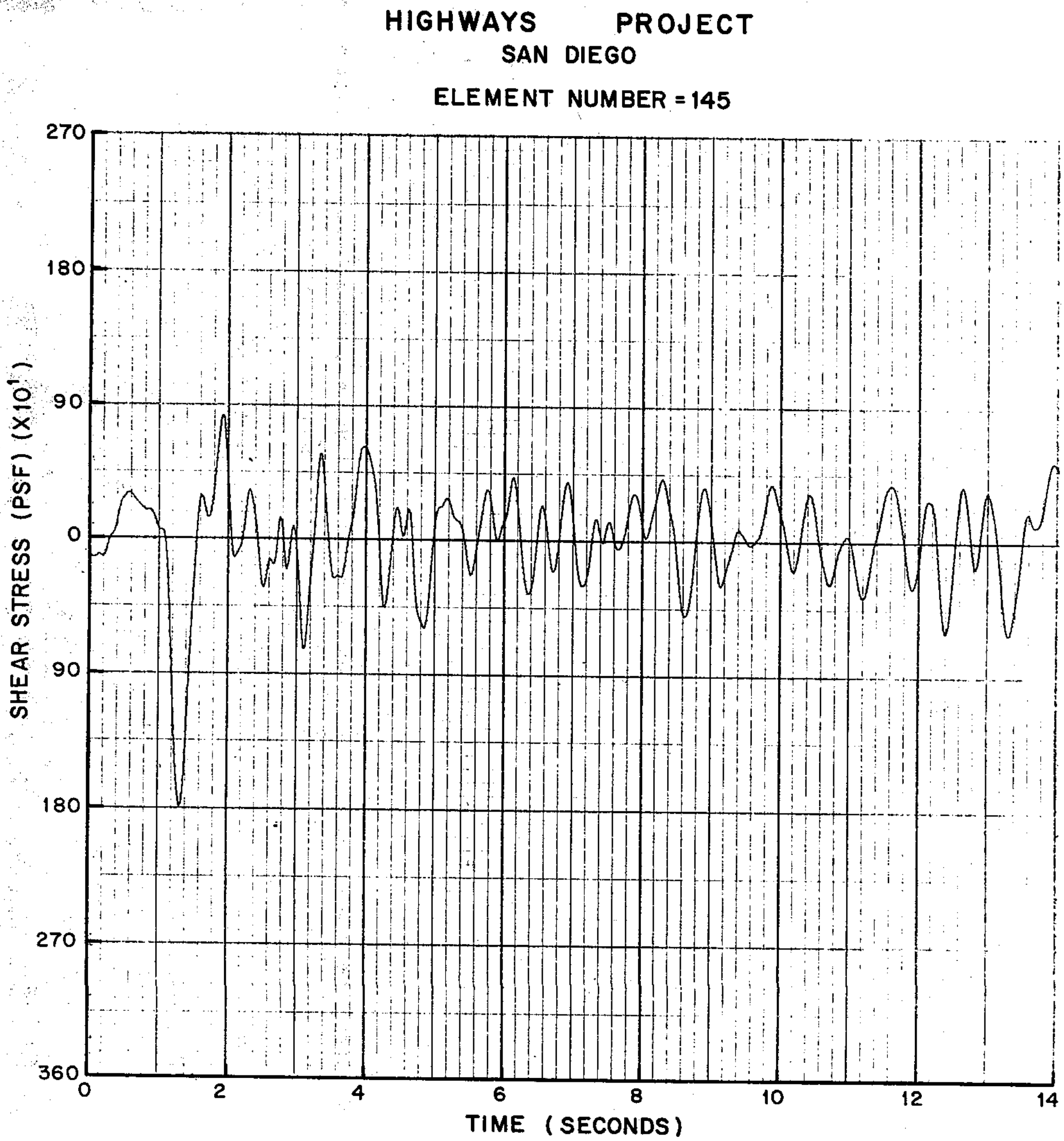

Figure 15 (c) SHEAR STRESS-TIME HISTORY OF MOTION FOR ELEMENT 145 DURING MAXIMUM CREDIBLE EVENT. (DYNAMIC STRESS LEVELS ONLY)

REFERENCES

1. Schnabel, P. B., Lysmer, J. and Seed, H. B., SHAKE, A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites, University of California, Berkeley, Report No. EERC 72-12, 1972.

2. Seed, H. Bolton and Idriss, I. M., A Simplified Procedure for Evaluating Soil Liquefaction Potential, November 1970, Report EERC 70-9, University of California, Berkeley.