REPORT ON STUDIES OF

SOIL MODULUS AND DAMPING

SAN ONOFRE SITE

June 4, 1985

REPORT ON STUDIES OF SOIL MODULUS AND DAMPING SAN ONOFRE SITE

1.0 INTRODUCTION

This report has been prepared to answer questions which have been raised by the staffs of NRC and LNL regarding the effects of sample disturbance, test conditions, and other possible sources of inacurracy in the determination of the soil moduli and damping at the site of the San Onofre reactors. The evaluation of modulus and damping parameters was done in the early 1970s and documented as the design basis for the SONGS 2 and 3 project. The questions presently being asked were reviewed internally within the Woodward-McNeill organization during that time period. The present report is the result of a review of the files from that period to compile from various sources the reasoning and actions on each point, and to present the results in condensed form for review.

The structures are founded upon the San Mateo Sand Formation, which is very uniform to a depth of about 1,000 feet, based upon borings and geophysical data. Its uniformity is somewhat demonstrated by the narrow range of grain-size distribution shown in Fig. A (SONGS 2 and 3 PSAR, Amendment 11, App. 2E, Attachment 3 [hereinafter, Ref. 1], Fig. B-1) and by the narrow range of in-situ densities shown in Fig. B (SONGS 2 and 3 FSAR, Fig. 2.5-63). The San Mateo sand is somewhat unusual in that, even though it is not cemented (Ref. 1, App. B, p. 4), it supports near-vertical cliffs about 100 feet high in its native state, and at least 35 feet high in its recompacted state.

The tests performed, and to be reported here, were: density tests, in the laboratory on intact Modified California, Pitcher and hand-carved samples, and in the field by sand-cone methods; conventional field crosshole seismic tests; cyclic laboratory tests measuring deformation from platen to platen in most cases and in the middle third of the sample in two cases; controlled field dynamic-response experiments on large-scale (10-ft diameter, 5-ft thickness) concrete foundations and field attenuation tests. The modulus and damping results will be summarized here, along with the derivations of the resulting design curves.

2.0 EFFECTS OF SAMPLE DISTURBANCE

It was recognized early on, and later verified, that the San Mateo Sand had a comparatively high relative density. It was thus expected that the act of sampling would likely yield samples with densities lower than the in-situ densities. That indeed did occur, as shown by the data in Figs. C-1 and C-2 (both from Ref. 1, App. B). Those results show that the field densities tabulated in Figs. B and C-1 average about 123 pcf, which are indeed higher than the laboratory densities (Fig. C-2), which average about 110 pcf. Thus it was clear that there would be some effect on other test data due to this density change due to sampling. That effect was addressed, as will be discussed below after the data are presented. The data will be presented as copies of the figures and tables which have been previously submitted. To prevent confusion, no changes will be made to those figures and tables, although in some cases data points will be added for purposes of comparison or clarification. This means that both Young's and shear moduli, and principal and shear strains will be presented, sometimes mixed on the same sheet, because that was appro-

priate for the use of the original document at that time. Thus, in the material below, it will be necessary to remain continuously aware of which parameter is being presented.

The basic data for modulus as a function of confining pressure and strain are given in Fig. D (Ref. 1, App. B). Those are cyclic triaxial data, for which the strains are computed from the platen-to-platen deformations. It was recognized at that time that the platen-to-platen strains can in some cases be larger than the strain in the soil at some distance away from the platen, near the center of the sample. This is due to boundary conditions at the soil-platen interface, and because the density at the lesser-disturbed center of the sample are probably more nearly equal to those in the field. For that reason, a special (for that time) test was done to evaluate the possible effect. Fig. E (Ref. 1, App. E) shows the test set-up, in which the deformations were simultaneously measured between the platens and between the one-third points along the sample height. The samples were 4-inch diameter by 8-inch high. The results showed that the platen-to-platen strains were about 2.5 times the third-point strains. Because the strain enters into the modulus calculation, it was necessary to multiply the (platen-to-platen) moduli by 2.5, and of course to multiply all strains by 1/2.5 = 0.4, as is demonstrated by lines 1 and 2 in Fig. F (Ref. 1, App. E).

Recognizing that the San Mateo Sand is an unusual material, it was felt prudent to investigate whether dependence of modulus upon the conventional 1/2 power of confining pressure would apply to this material. The results of that study are given in Fig. G (Ref. 1, App. E). A close fit to the data for the range of strains of interest shows that the modulus depends more nearly upon the 2/3 power of the con-

fining pressure, so that value has been used for the project.

The raw data are shown again in Fig. H, along with the resulting corrected modulus-strain curve, recommended for analysis for the SONGS 1 project ("Soil Backfill Conditions" enclosure to letters K. P. Baskin [SCE] to D. M. Crutchfield [NRC] dated 18 April and 1 September 1983 [hereinafter Ref. 2], App. F, Fig. F-1, with data points added).

Attention was next paid to the correction for the effects of laboratory vs. field density. Fig. I shows the data for a confining pressure of 2,000 psf and a principal strain of 0.1 percent. Those data come from Fig. G, and were selected because they evidenced the most scatter. The results, Fig. I, show that in the range of field densities, the sensitivity of shear modulus to density is a second-order effect. Also shown in Fig. I is a normalized calculation using the Seed-Idriss curves (Seed, H. B., and Idriss, I. M.: "Soil Moduli and Damping Factors for Dynamic Response Analysis," Earthquake Engineering Research Center, University of California, Report EERC 70-10, Berkeley, 1970, Fig. 5). Because the data, supported by other data in the literature, show such a small effect for density variation, and because Fig. H, the recommended curve effectively enveloped the data, no further correction was made to the strain- and modulus-corrected curve described above.

The laboratory data for damping are given in Fig. J. (Ref. 1, App. F, Fig. F-2). As is normal for this measurement, the results show considerable scatter, so the results were analyzed for reasonableness as follows. What appeared to be a reasonable average-fit line to the data was drawn, as shown in Fig. J. That line corrected for end effects (i.e.,

strain times 0.4) is shown drawn as "San Mateo Sand" in Fig. K (Ref. 2, App. F, Fig. F-6), for comparison to data from the literature (see Seed and Idriss, 1970, referenced above, Fig. 10) That comparison shows that the selected damping curve lies in the range of, or a little below the data in the literature. Considering the unusual nature of the San Mateo sand the comparison was found to be reasonable, so the curve of Fig. J was used, in the form of Fig. K.

3.0 FIELD EXPERIMENTS

A number of large-scale dynamic response experiments were performed upon 10-foot diameter foundations which were 5 feet thick. The totality of the data and analysis are given in the Units 2 and 3 FSAR, App. 3.7C, to which further reference will be omitted in the rest of this section. The experiments involved setting the slabs into transient motion in various modes, and measuring their responses with respect to time. Setting the slabs into motion was accomplished by pulling on the foundations in the proper direction to excite the desired mode, using various force links designed to break at given loads. The latter allowed the excitation of various strain levels.

The moduli were calculated using the measured amplitudes and the spring constant for that foundation in that mode using the theory of the elastic half-space (see, for example, McNeill. R. L.: "MACHINE FOUNDATIONS, The State of the Art," Proceedings, Specialty Session on Soil Dynamics, Seventh International Conference on Soil Mechanics and Foundation Engineering, 1969). The dampings were calculated using the rate at which the vibrations decayed (See McNeill, reference above).

The results for shear moduli are shown overplotted on the recommended design curve in Fig. L (Ref. 2, App. F, Fig. F-1 with data points added). The points reasonably fit the design curve, and give support for the raising and shifting of the curve described in Section 2 above.

The hysteretic dampings were calculated from the field rocking mode response test by calculating the spatial dampings from the theory of the elastic half-space, then subtracting the results from the measured total dampings. The hysteretic dampings were also estimated by measuring the attenuations of Rayleigh waves, using the methods outlined in FSAR, App. 3.7C. Both sets of results are given in Fig. M (Ref. 2, App. F, Fig. F-6 with ranges of data added) and indicated tht the chosen damping curve is within or slightly below the range of measured field values of damping.

4.0 APPLICATIONS OF PARAMETERS

The stiffness and damping values developed in the references cited above and summarized in Figs. H and K are intended to be used for the calculation of spring and damping constants using the theory of the elastic half-space, at the appropriate strain levels. The fomulations for those constants are given in the attached Fig. N, which comes from FSAR, App. 3.7C. The damping values in Fig. N are for geometric damping, and should have added to them the appropriate value of hysteretic damping from Fig. K. Site-response analyses were done using SHAKE to develop approximate strains for those stiffness parameters. The computations are reported in Ref. 2, App. F, Fig. F-4, from which Fig. O has been taken for ease of reference as to the end results. The corresponding major principal strains for the earthquake are expected to be in the range of about 0.2 percent for purposes of using the equations in Fig. N.

5.0 VARIATION OF PARAMETERS

All of the stiffness data have been plotted in Fig. P to allow an evaluation of the range of variation, but the data have been treated more formally than previously, as discussed below:

The raw data from Fig. H have been corrected for density variation between field and laboratory to 123 pcf using the data typified by Fig. I (which is an example of the broader suite of data), and the results are shown by the solid triangles in Fig. P. A curve, the lower dashed one, has been fitted to those density-corrected data; and that curve has been strain-corrected by the methods of Fig. F to achieve the Laboratory-Corrected curve, which is the upper dashed curve in Fig. P. The density-corrected laboratory data at high strains show a variation of less than 10 percent from the fitted (lower dashed) curve; and the strain-corrected curve from those data agrees exactly with the design curve, except for a small variation (less than 5 percent) at the lower strains which, Fig. O, are below the range of interest to this project.

The data in Fig. P also include the platen-to-platen and center-third dynamic triaxial data, shown as the solid squares. These were the basis for the corrections made as shown in Fig. F. Of the modulus values measured directly from the center-third of the sample (Points B and D), one point (B) lies on the design curve; and the other (D) lies slightly above (about 10 percent) that curve (the test conditions of test C-D were suspect, in that the platens did not appear to have been properly seated at the start of the test. These data were therefore not relied upon for this project, but are shown here for the sake of completeness).

The field test data are shown in Fig. P as the solid dots. Those data lie within 10 percent of the design curve, except for the two low strain points that lie below the curve by about 15 to 20 percent.

On the basis of these data, the variations shown below were recommended.

Source of Variation	Resulting Variation Stiffness Parameters	
	Large Rigid Foundation	Flexible Structure with Strip and Isolated Foundation
Soil Properties		
due to areal variations at the site	0	± 5 percent
due to ± 30 percent variation in seismic induced strain	± 10 percent	± 10 percent
SSI Methodology	± 10 percent	± 20 percent

6.0 DESIGN VALUES FOR CONTAINMENT

The design shear modulus for the Unit 1 Reactor Building has been calculated at a plane under the structure at a depth of half the radius of the sphere at the ground surface. That depth choice is not particularly important because the modulus does not vary strongly in either the vertical or the horizontal planes, as shown in Fig. Q. That value of shear modulus is 1,390 ksf, for the assumptions described, and using a shear strain of about 0.2 percent from Fig. O.

The design damping should be made up of two parts: a constant value of 12 percent (Fig. K) for hysteresis; and a geometric value, which is dependent on 3-dimensional

modeling of the foundation/soil system computed in 3-dimensional model.

7.0 SUMMARY OF CONCLUSIONS

Based on the foregoing review of data files for laboratory testing and for field testing completed at the San Onofre site, and on a more formal analysis of data as discussed in Section 5, the following conclusions are summarized:

1. The San Mateo Sand is very dense and uniform in horizontal and vertical extent below the site extending about 1,000 feet below plant grade.

2. The dry density of the native soil averages 123 pcf with about ± 3 percent variation on all in-situ density measurements (125 observations) as shown in Fig. B.

3. By formally correcting for density and end effects for laboratory testing and considering the results of the slab response tests, the data scatter about the shear modulus design curve is generally within ± 10 percent as indicated in Fig. P.

4. Considering conclusions 1 through 3 together with the results of site response analysis (Fig. O), the variation in shear modulus tabulated in Section 5 above is considered reasonable and appropriate for the San Onofre site.

5. Considering the comparison of the damping-strain curves for the San Mateo Sand and the damping values reported in the literature for sands together with the results of the field response experiments and attenuation tests

shown in Fig. M, it is concluded that the hysteretic damping curve is both reasonable and appropriate for design and analysis purposes.

6. The uniformity of the site subsurface conditions (i.e., no layering) and the compatibility of the field response experiments results with elastic half space theory (SONGS 2 and 3 FSAR, App. 3.7C) support the use of the relationships for spring constant and spatial damping shown in Fig. N for design and analysis of structures.

7. In the case of the Unit 1 Reactor Building, for which special analysis is being completed, the shear modulus and hysteretic damping parameters for input to the analysis are computed based on conclusions 1 through 5 to be 1390 ksf and 12 percent, respectively.

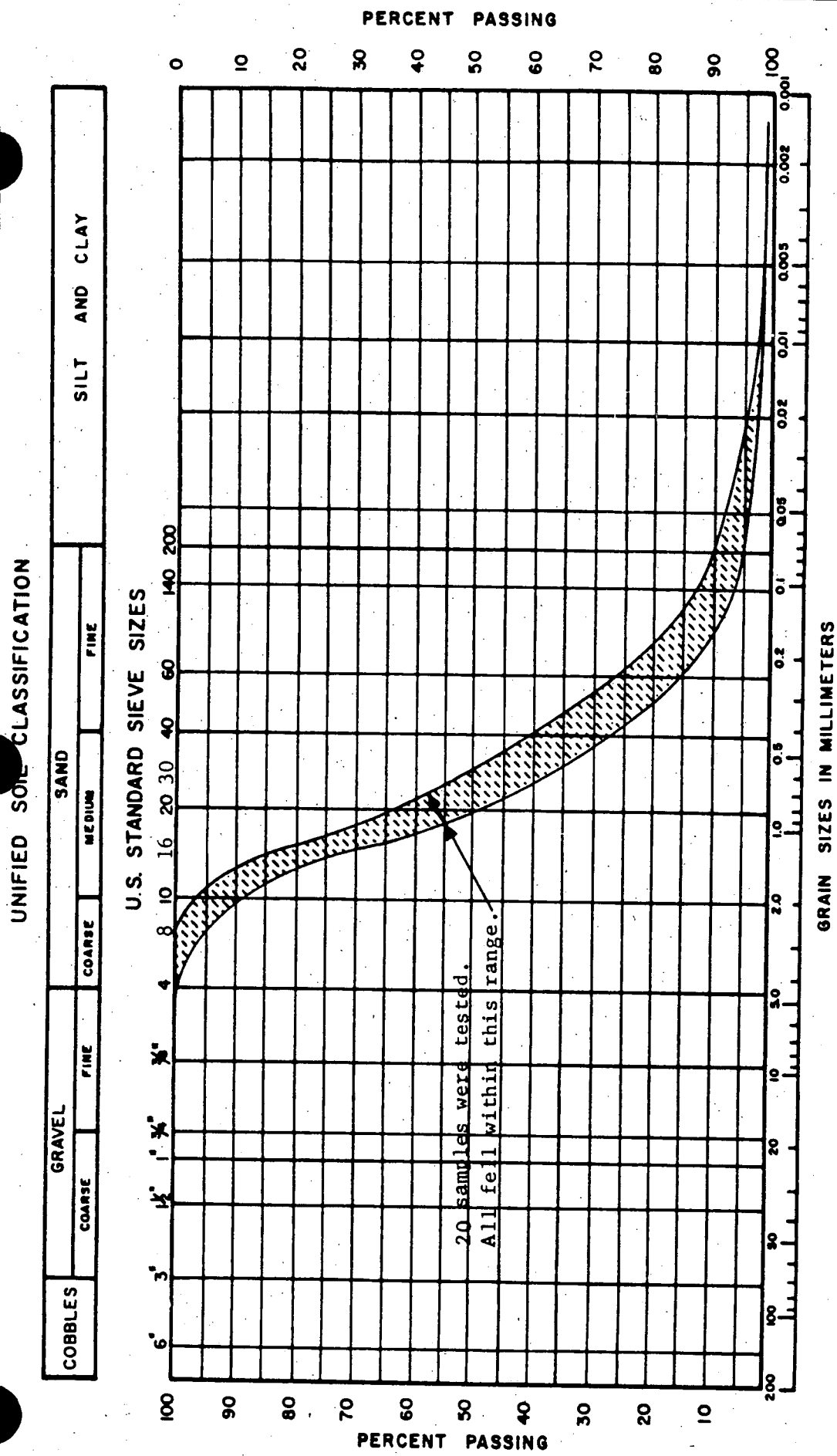
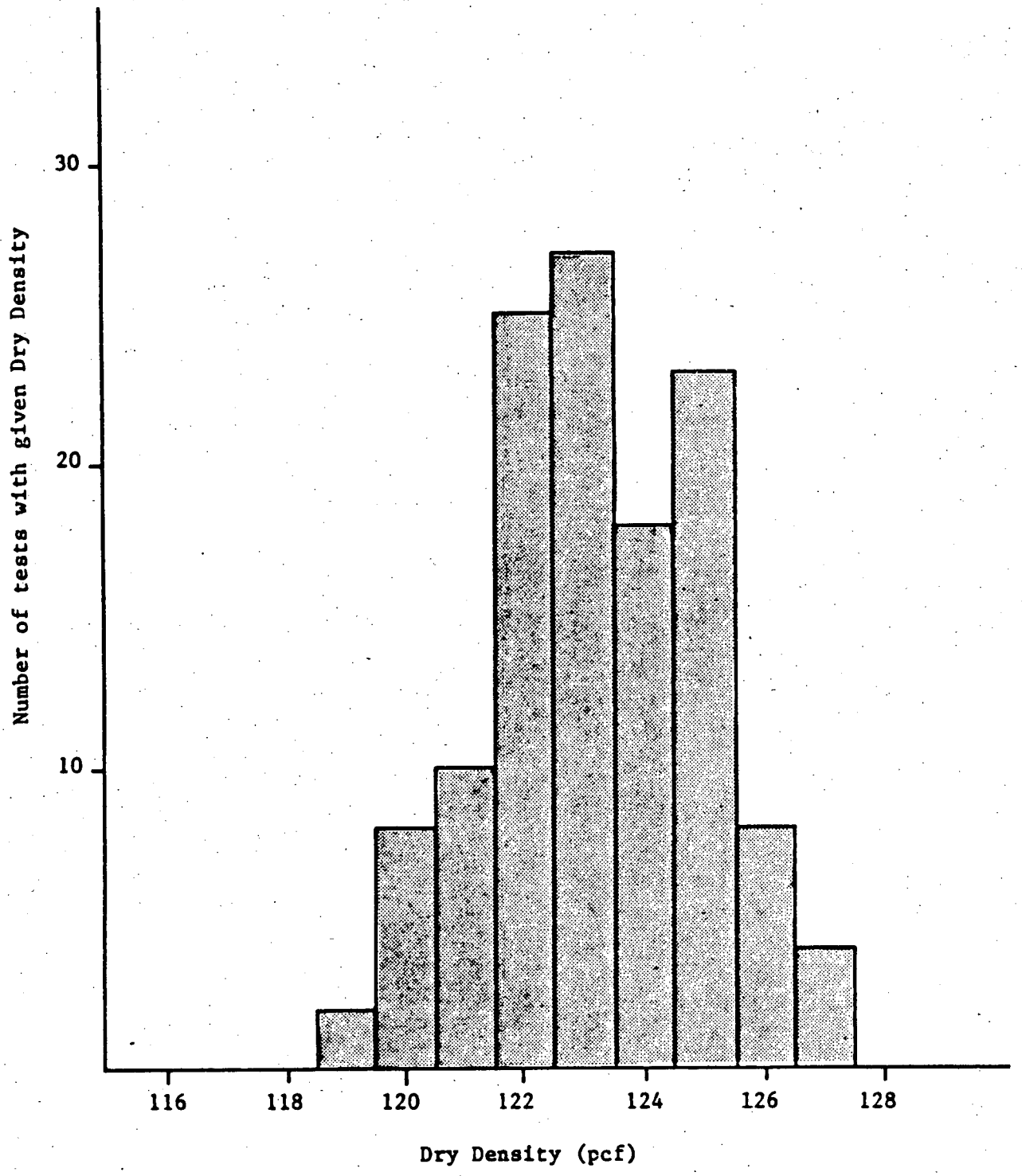


Figure A

Project: SONGS, Task #4 Job Number: G75I	GRAIN SIZE DISTRIBUTION CURVES	Fig. B-1



SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

RESULTS OF SAND CONE TESTS
ON INSITU SAN MATEO SAND

Figure 2.5-63

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23SE6

Figure B

TABLE B-1

Field Density Data

Test Pit	Test No.	Depth Below Existing Grade	In-Situ Dry Density pcf	In-Situ Water Content %
TP-1	1	5 ft	128.0	2.0
TP-2	1	ground surface	123.5	0.3
TP-2	2	ground surface	125.0	0.3
TP-2	3	9 ft	116.1	2.4
TP-2	4	10 ft	113.7	2.2

Figure C-1

TABLE B-II

Laboratory Density Data

Boring or Test Pit	Sample No.	Depth Below Existing Grade (ft)	Sample Type *	Dry Density pcf	Water Content %
TP-1	1	6	HC	105	4.7
TP-2	1	9	HC	115	3.0
TP-2	2	9	HC	111.9	----
B2	1a	16	PB	106.9	16.9
B2	1b	16	PB	109.6	15.6
B2	1c	16	PB	109.2	15.6
B2	2a	18	PB	106.8	16.1
B2	2b	18	PB	108.6	16.0
B2	2c	18	PB	108.9	15.3
B4	4a	40	PB	107.3	17.1
B4	4b	40	PB	111.8	16.5
B4	C4	18	SP	106.6	2.1
[illegible]	C5	23	SP	100.7	3.0
[illegible]	C6	28	SP	114.2	12.1
[illegible]	C7	33	SP	120.1	10.1

* Key (see Appendix A.2)

PB = Pitcher Barrel
HC = Hand Carved
SP = Standard Penetration

Figure C-2

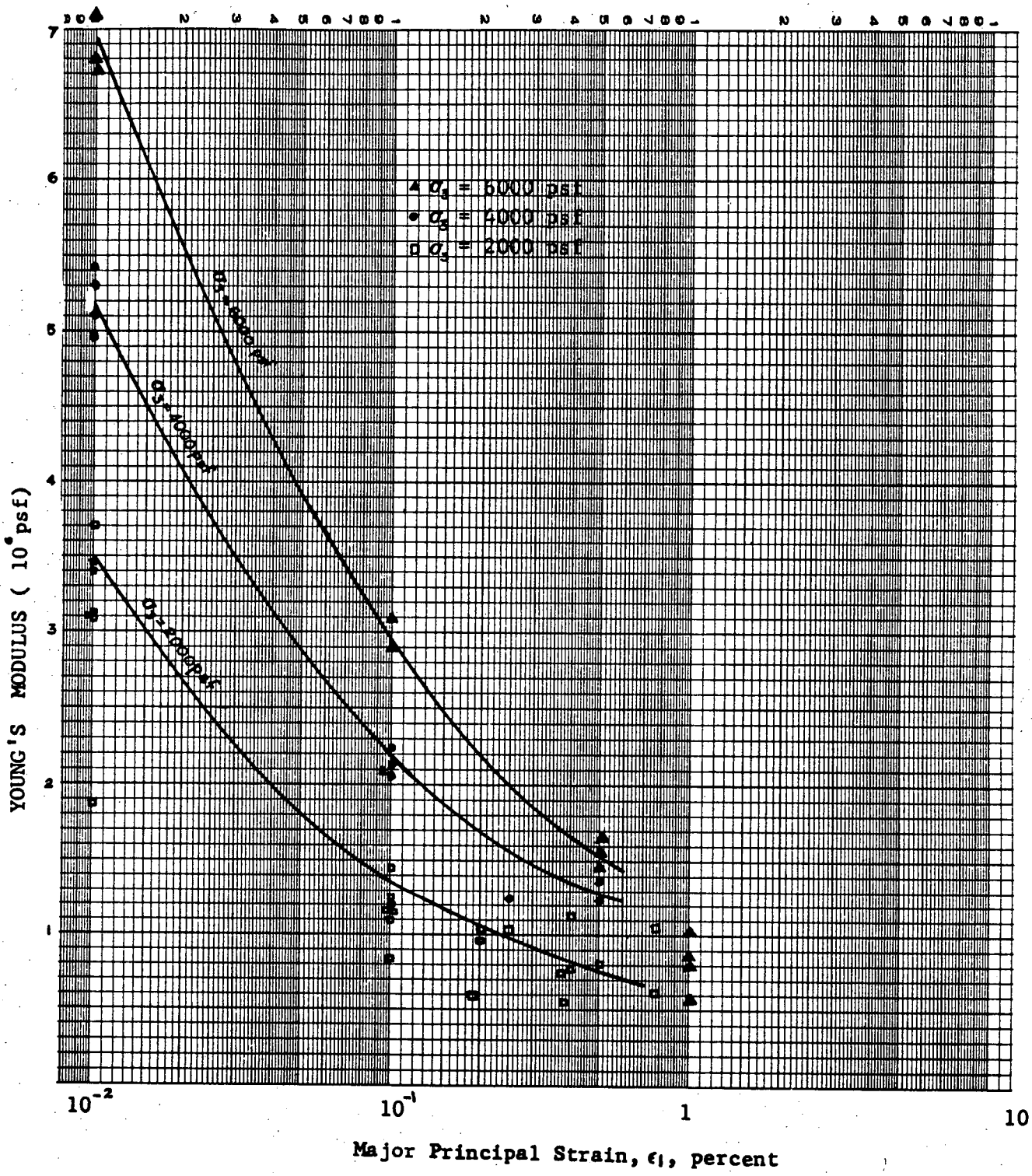
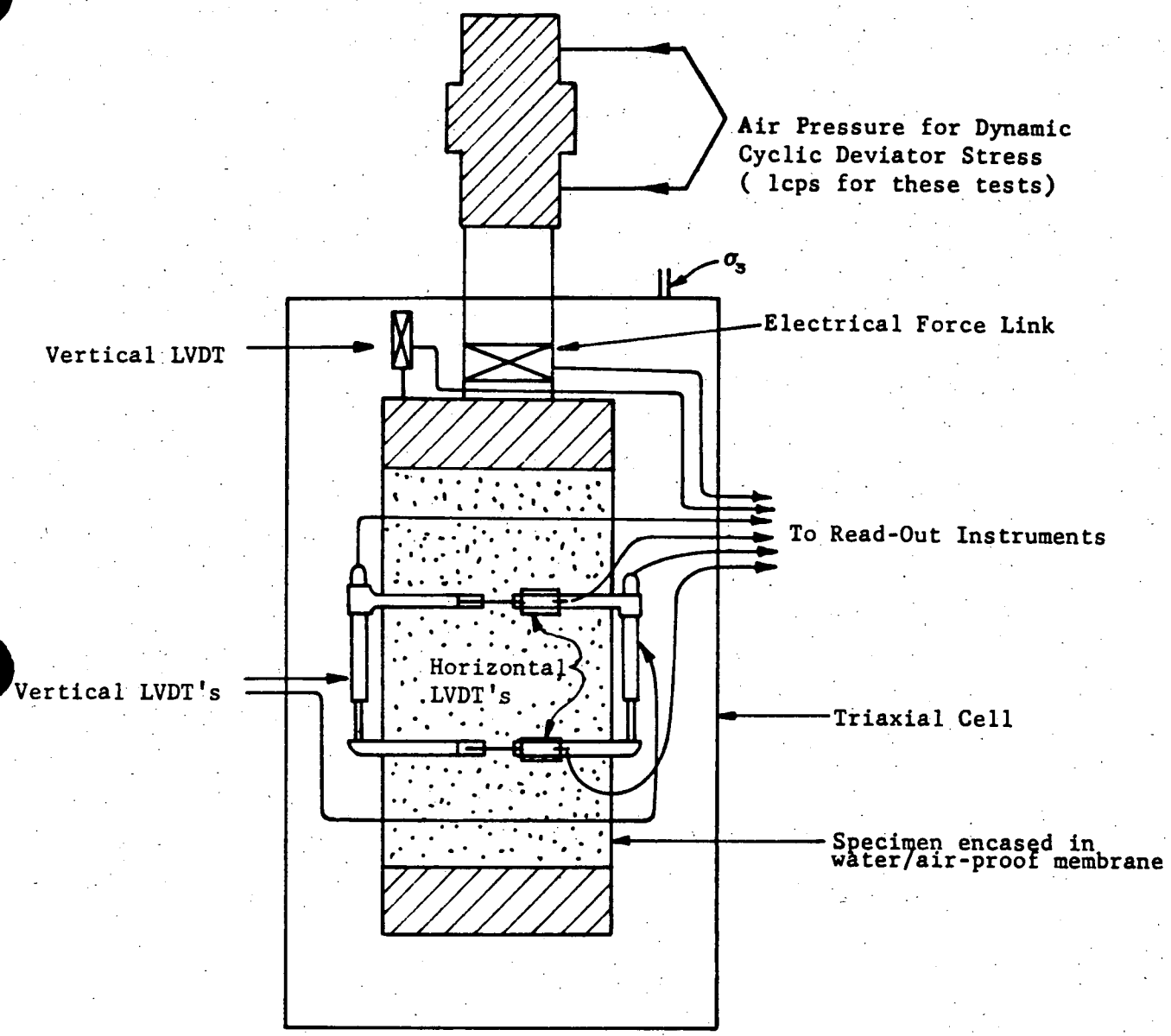


Figure D

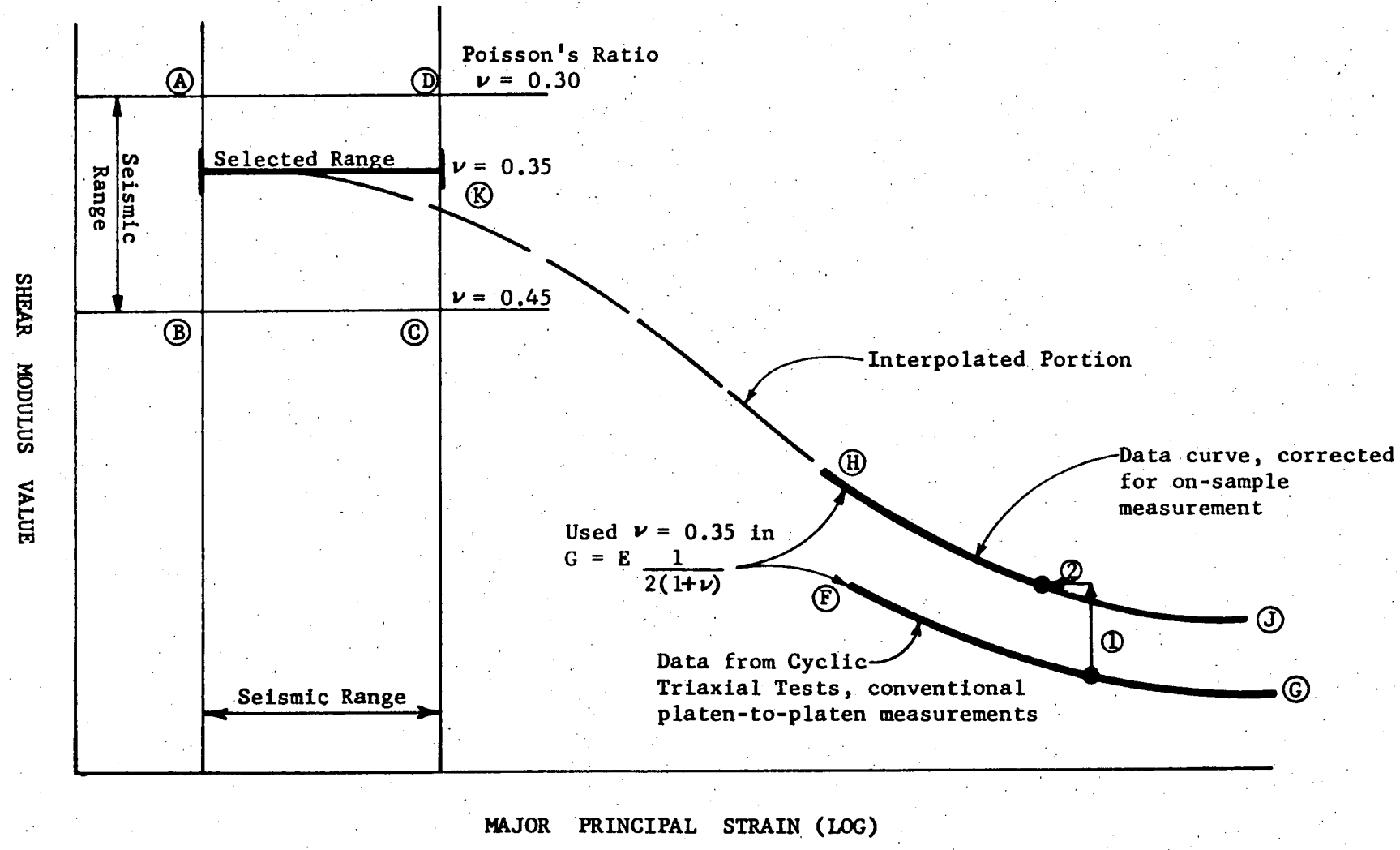
Project: SONGS, Task #4 Job Number: G75I	SUMMARY OF DATA	Fig. D-4



NOTE: •LVDT(Linear Variable Differential Transformer) reads displacement
•Force Link reads load
•All 6 traces are displayed on light sensitive paper with load and displacement as the ordinate and time as the abscissa (similar data presentation as Fig. D-1)
•Pore pressure may be read out also, if appropriate

Figure E

Project: SONGS, Task #4 Job Number: G75I	POISSON'S RATIO TEST Schematic Diagram	Fig. E-1



NOTE: To correct approximately for boundary effects, ① is x2.5, and ② is x0.4, for these materials. Circled letters refer to discussion in text of Appendix E.

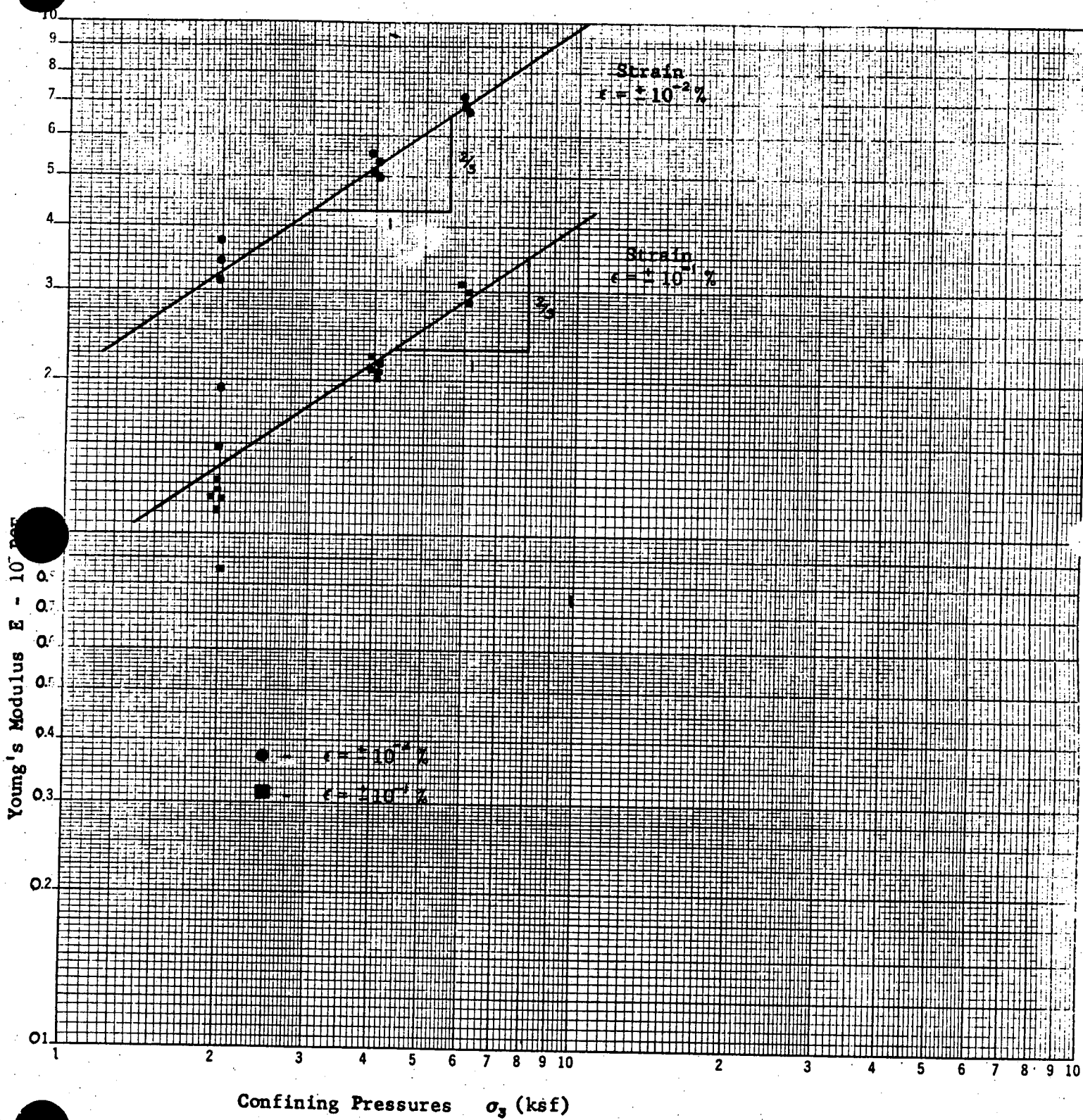
Project: SONGS, Task #4
Job Number: G75I

FIELD AND LABORATORY VALUES

Fig. E-2

Figure F

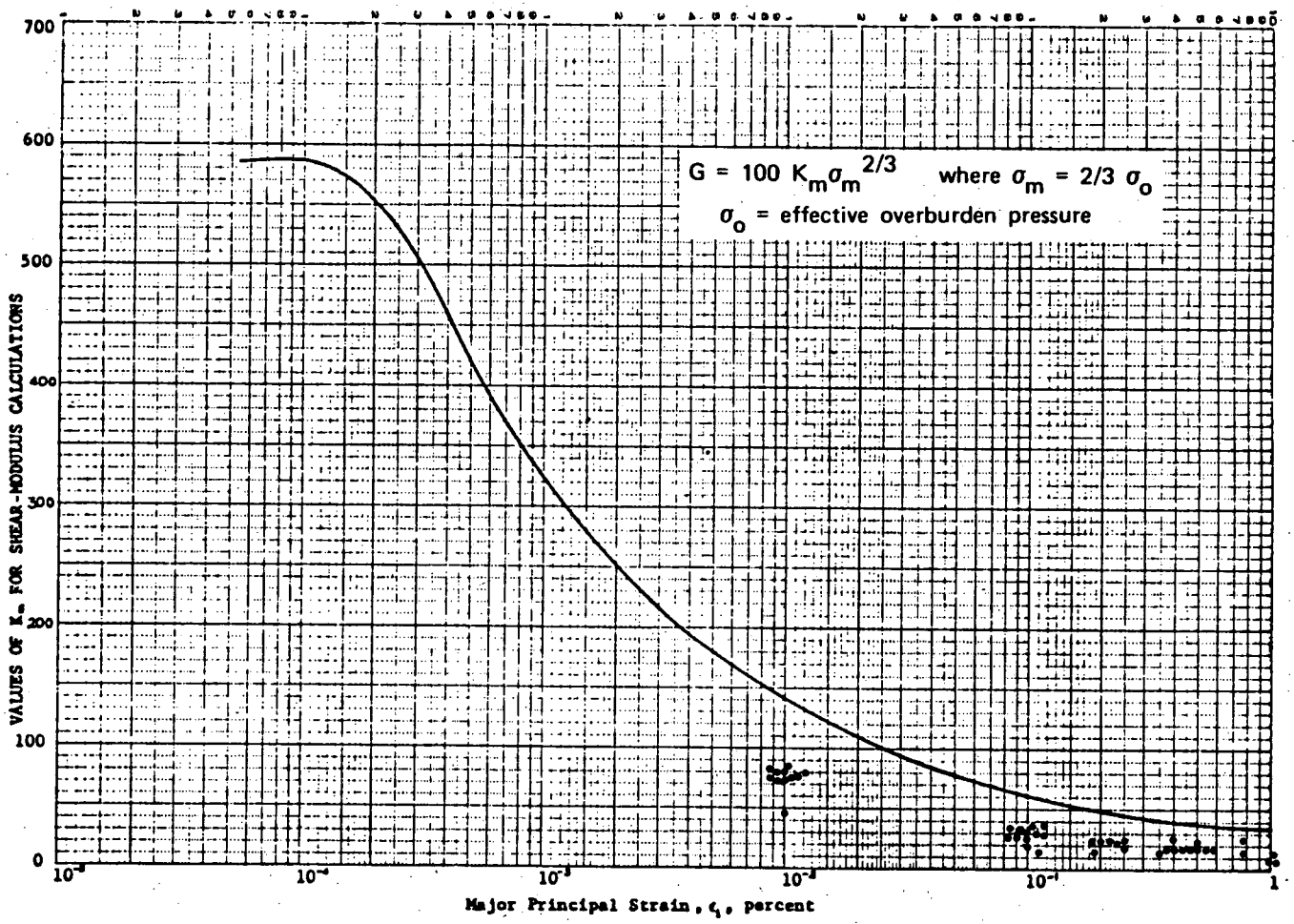
WOODWARD-McNEILL & ASSOCIATES



Project: SONGS, Task #4 Job Number: G75I	DEPENDENCE ON CONFINEMENT	Fig. E-3

WOODWARD-McNEILL & ASSOCIATES

Figure G



• Based on laboratory data (platen to platen data)

Figure H

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	VARIATION OF SHEAR MODULUS WITH STRAIN FOR SAN MATEO SAND	Fig. F-1

WOODWARD–CLYDE CONSULTANTS

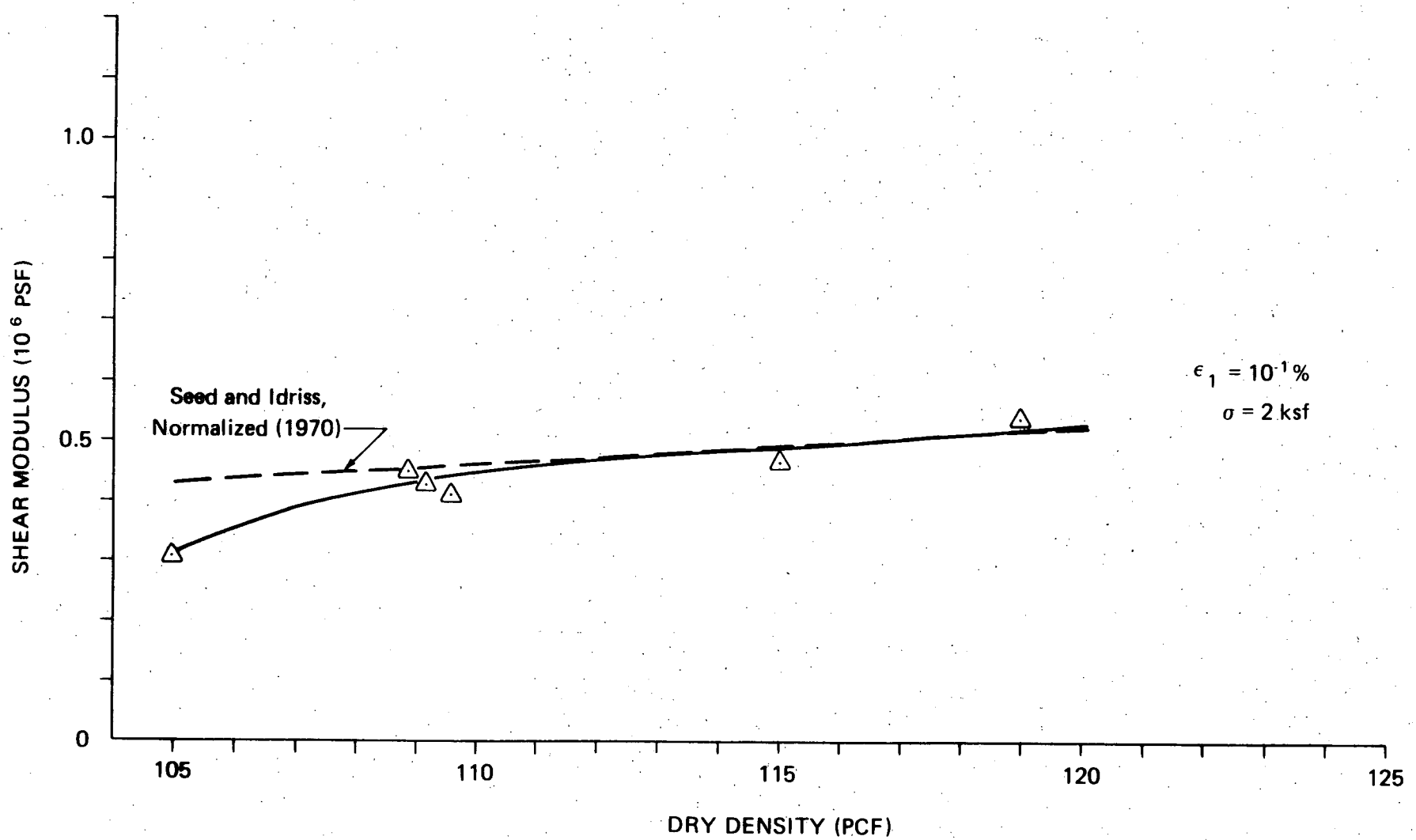


Figure 1

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	SHEAR MODULUS VS. DENSITY	Fig. 1

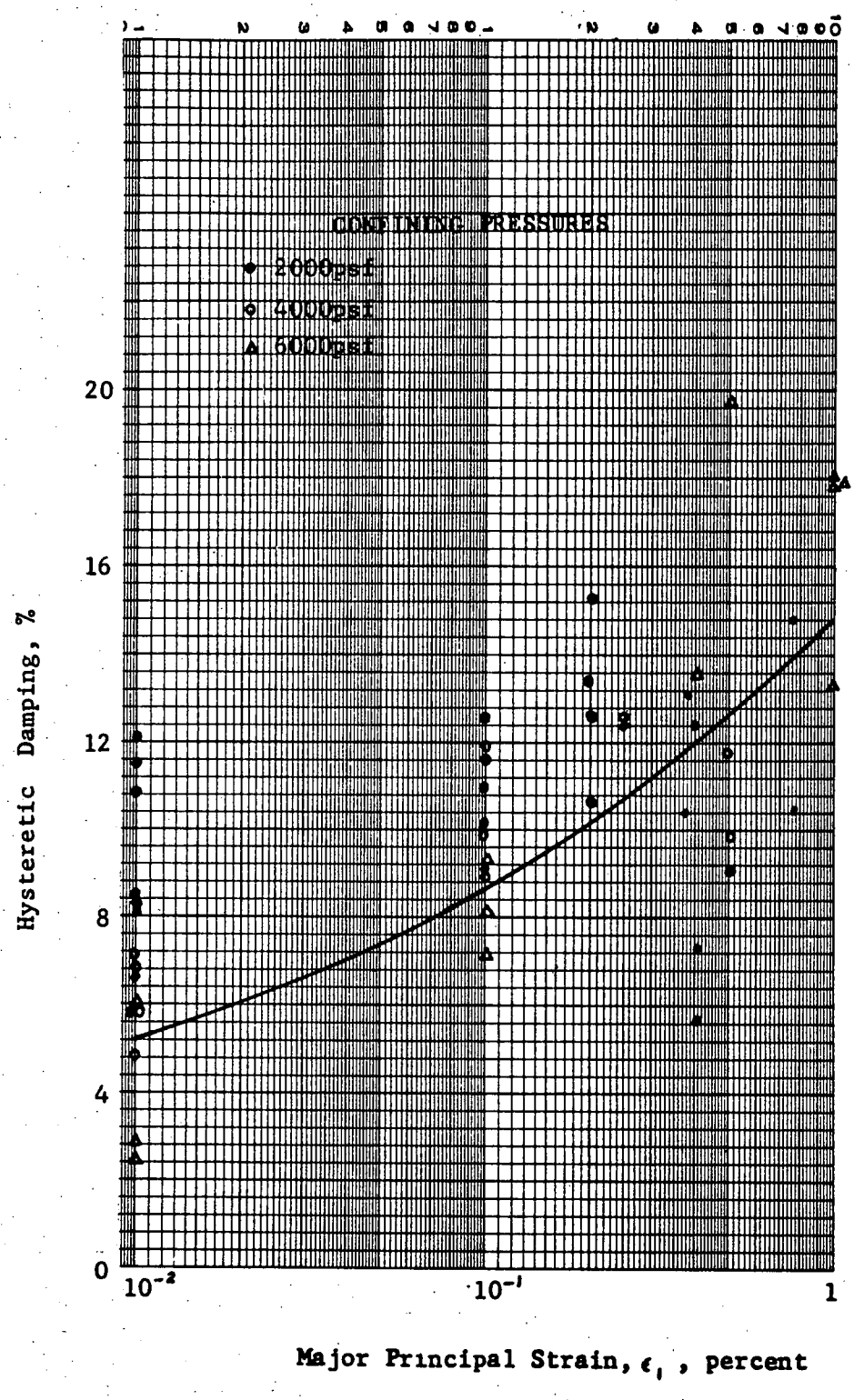


Figure J

Project: SONGS, Task #4 Job Number: G75I	SUMMARY OF DATA	Fig. F-2

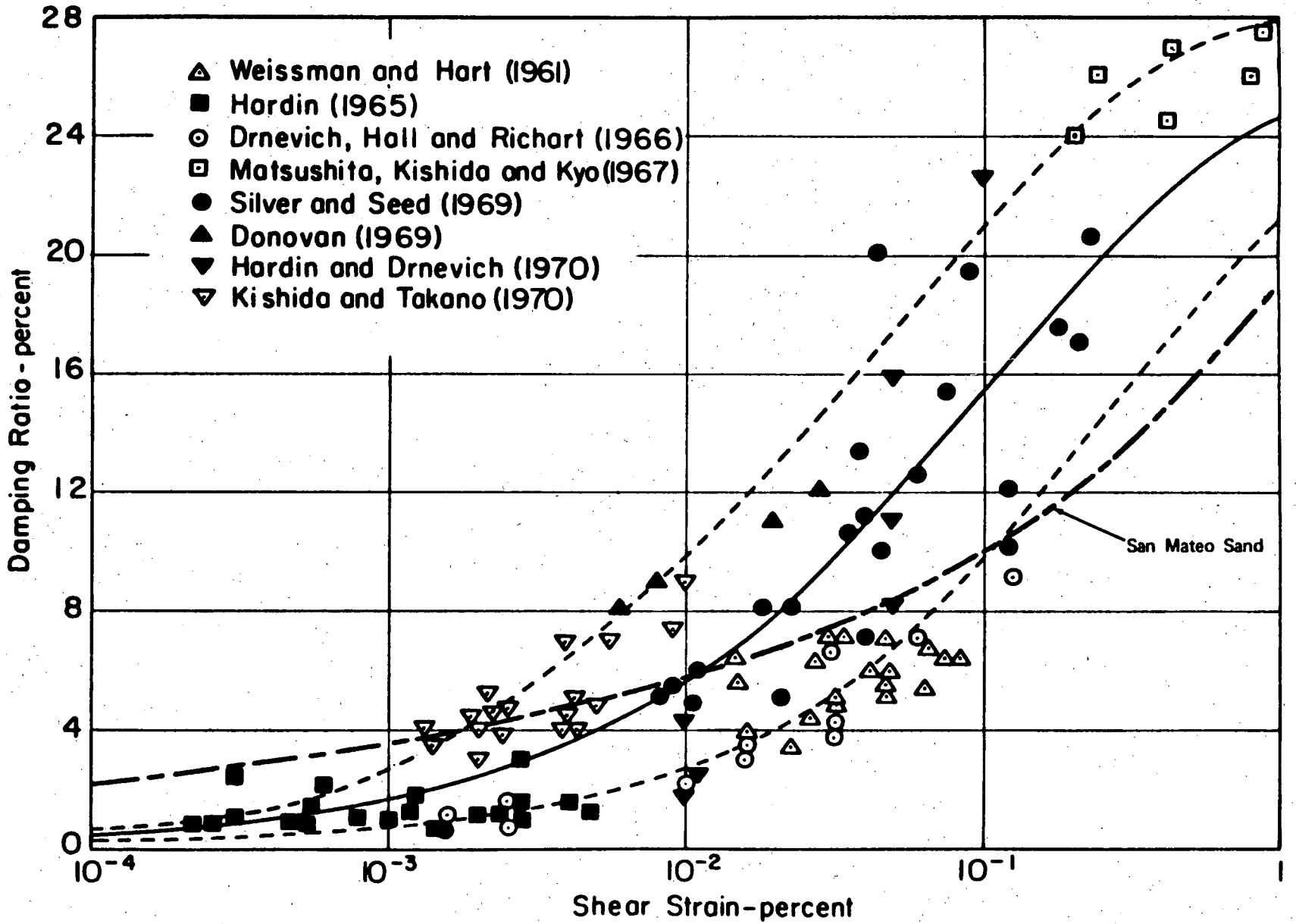
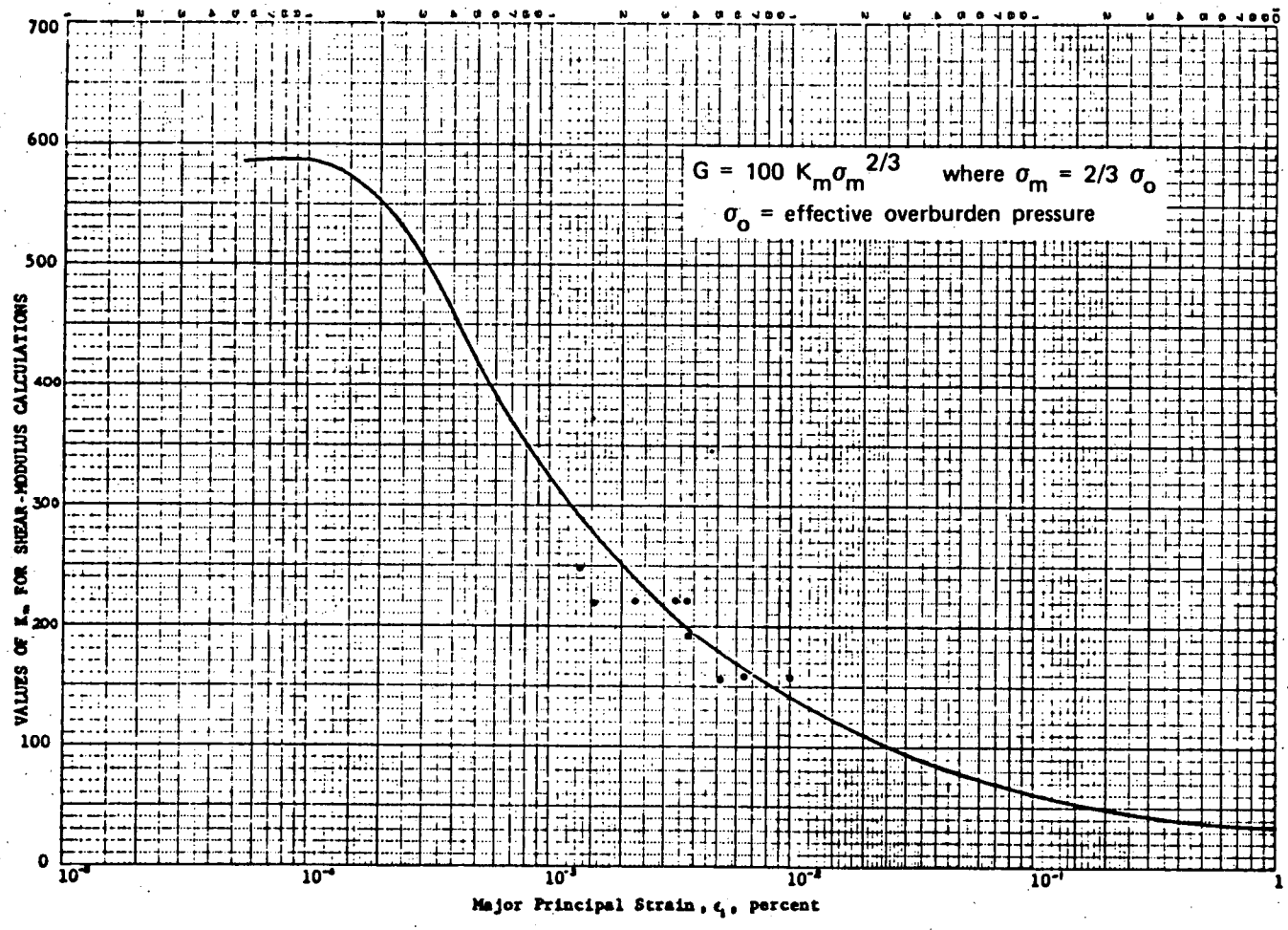


Figure K

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	DAMPING RATIOS FOR SANDS (after Seed and Idriss 1972) COMPARED TO THE SAN MATEO SAND DAMPING RATIOS	Fig. F-6

WOODWARD–CLYDE CONSULTANTS



• Based on field slab response tests

Figure L

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521-001A	VARIATION OF SHEAR MODULUS WITH STRAIN FOR SAN MATEO SAND	Fig. F-1

WOODWARD–CLYDE CONSULTANTS

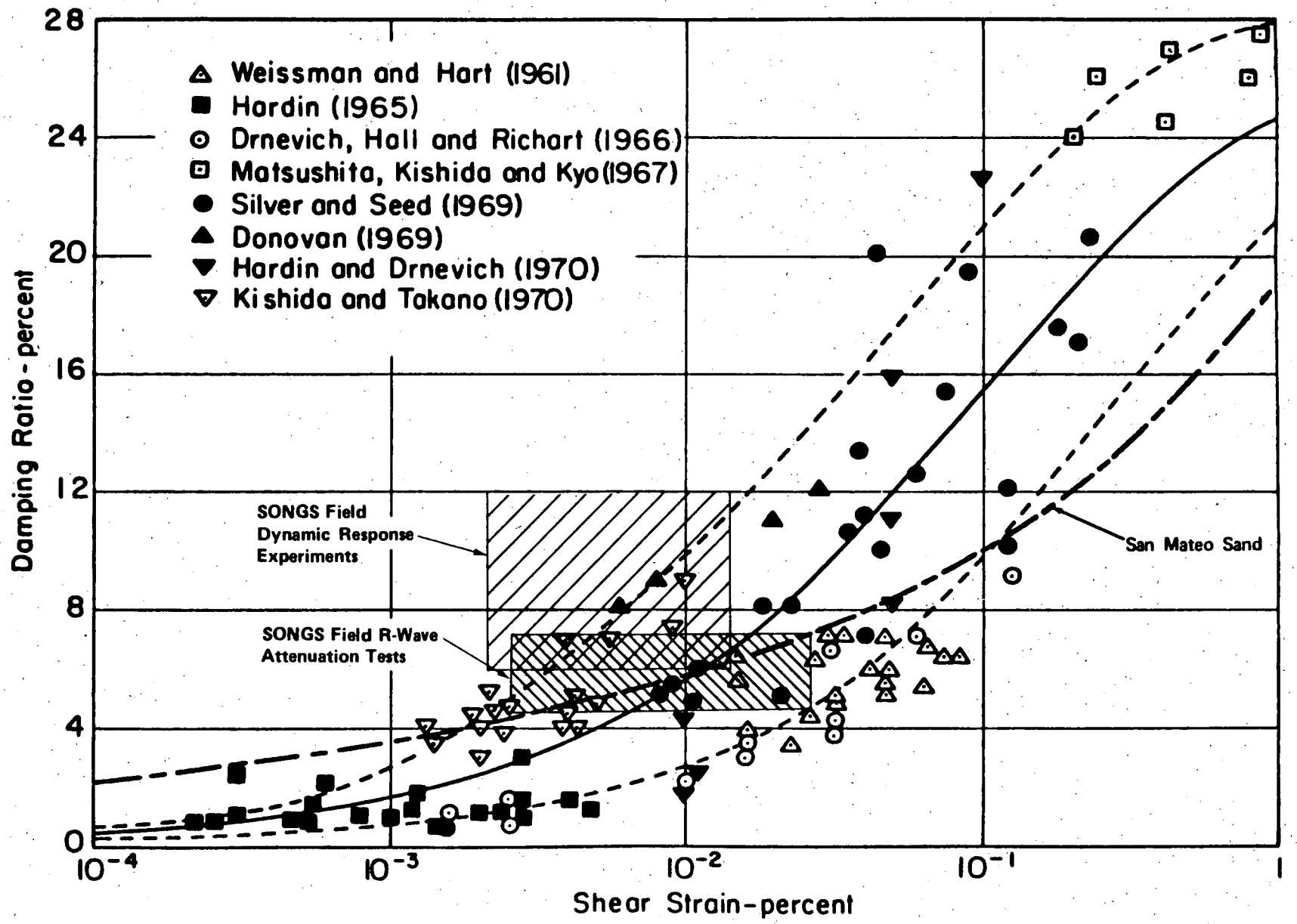


Figure M

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	DAMPING RATIOS FOR SANDS (after Seed and Idriss 1972) COMPARED TO THE SAN MATEO SAND DAMPING RATIOS	Fig. F-6

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TABLE I

DESIGN PARAMETERS

Parameters	Mode of Motion: VERTICAL TRANSLATION	HORIZONTAL TRANSLATION	ROCKING	TWISTING
Inertia	m, mass of foundation and machine	m, mass of foundation and machine	I_r, mass moment of inertia about rocking axis	I_t, mass moment of inertia about twist axis
Radius	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt[4]{\frac{BL^3}{3\pi}}$	$r = \sqrt[4]{\frac{BL(B^2 + L^2)}{6\pi}}$
Inertia Ratio	$B_v = \frac{(1-\nu)m}{4\rho r_e^3}$	$B_h = \frac{(7-8\nu)m}{32(1-\nu)\rho r_e^3}$	$B_r = \frac{3(1-\nu)I'_r}{8\rho r_e^5}$	$B_t = \frac{I_t}{\rho r_e^5}$
Effective Inertia for Design	m	m	$I'_r = \eta_r I_r$ See Appendix E for value of η_r	I_t
Stiffness Coefficient	$k_v = \frac{4GrC}{(1-\nu)}$	$k_h = \frac{32(1-\nu)\ GrC}{(7-8\nu)}$	$k_r = \frac{8Gr^3C}{3(1-\nu)}$	$k_t = \frac{16Gr^3C}{3}$
Geometric Damping	$D_v = \frac{0.425}{\sqrt{B_v}}$	$D_h = \frac{0.288}{\sqrt{B_h}}$	$D_r = \frac{0.15}{(1+B_r)\sqrt{B_r}}$	$D_t = \frac{0.50}{1+2B_t}$
General Case $C = C_1\ C_2$	$C_1 = 0.81$ C_2 = See Fig. 8	$C_1 = 1.0$ C_2 = See Fig. 8	$C_1 = 0.66$ C_2 = See Fig. 8	$C_1 = 0.41$ C_2 = See Fig. 8
Containment Structure Value of C	1.08	1.09	0.60	insufficient data

* For rectangular shaped foundations, calculation should be based on equations of pgs. 350 and 351, Richart, Hall and Woods, "Vibrations of Soil and Foundation."

$r_e = 0.6r$ for translation modes

$r_e = 0.8r$ for rotational modes

Note: for square or rectangular footing-

B = width of foundation in plan (parallel to axis of rotation)

L = length of foundation in plan (perpendicular to axis of rotation)

Figure N

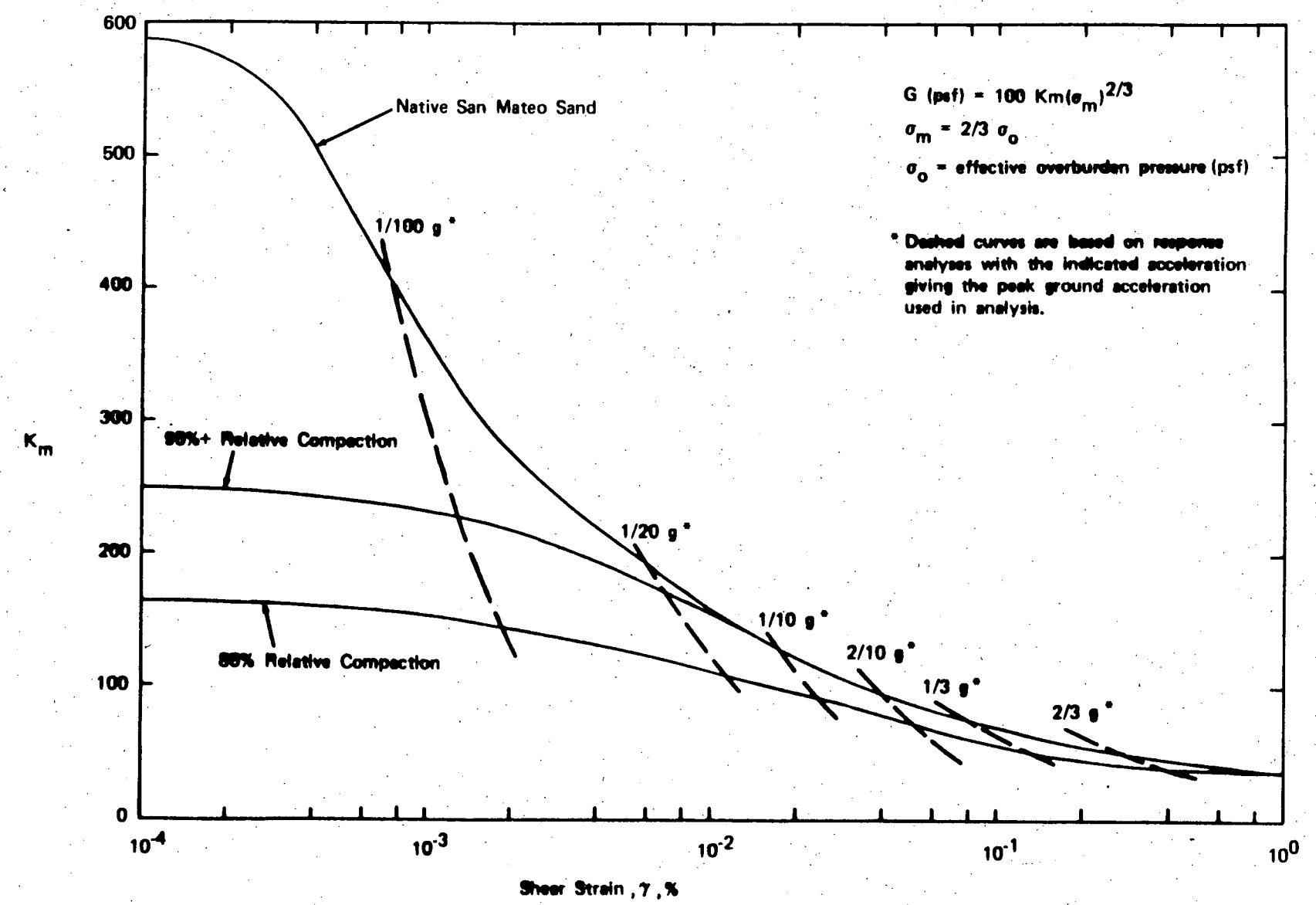
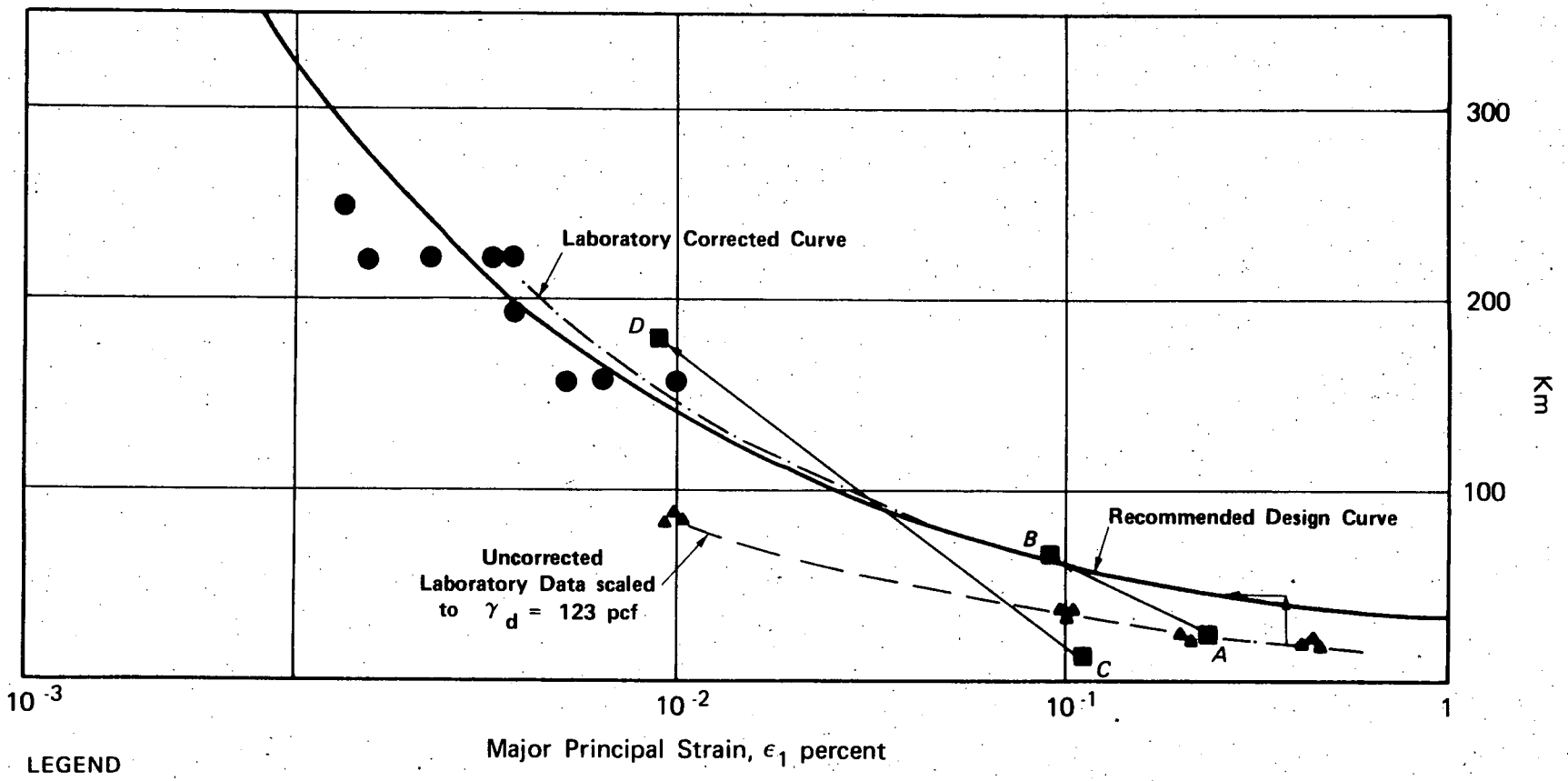


Figure O

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	MODULUS AS A FUNCTION OF STRAIN AND PEAK GROUND ACCELERATION	Fig. F-4

WOODWARD—CLYDE CONSULTANTS



LEGEND

- ● Field Slab response tests from Figure L
- ▲ Laboratory modulus tests scaled for density to 123 pcf
- ■ Laboratory modulus tests for which central 1/3 strain measurements were made

A&C represent platen–to–platen measurements
B&D represent central 1/3 measurements

$G = 100\ K_m \sigma_m^{2/3}$ where $\sigma_m = 2/3\ \sigma_o$

σ_o = effective overburden pressure

Project: SONGS RE-EVALUATION **Project No.** 413521		**Fig.** P

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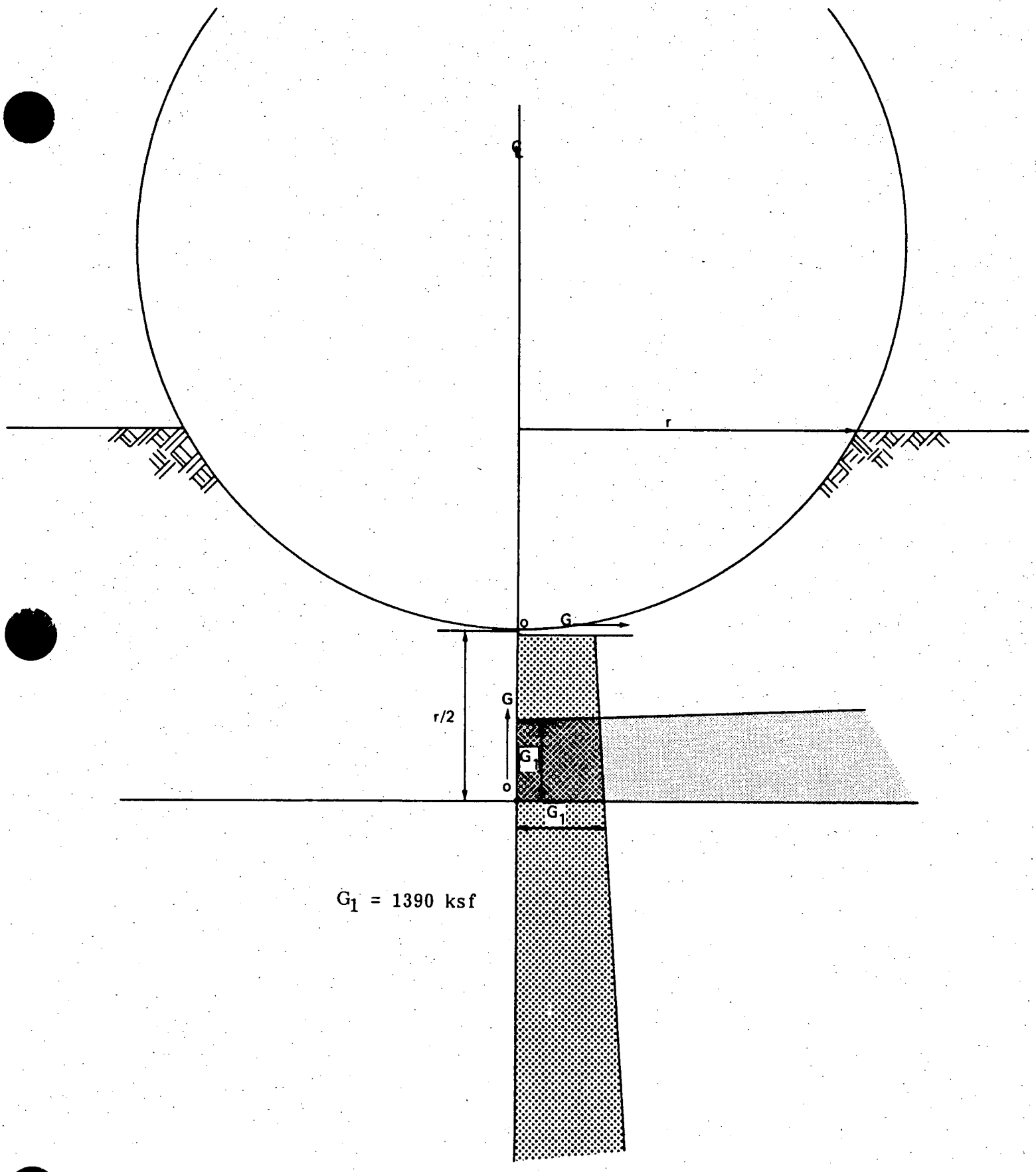

Figure Q