GEOMATRIX CONSULTANTS, INC.

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

▲ 5010.64 (FRONT)

CLIENT & PROJECT Private Fuel Storage Facility, LLC
Private Fuel Storage Facility, Skull Valley, Utah

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CALCULATION TITLE (Indicative of the Objective):
Development of soil and foundation parameters in support of dynamic soil-structure interaction analyses.

QA CATEGORY (✓)
☒ I - NUCLEAR SAFETY RELATED
☐ II ☐ III ☐ _____ OTHER

CALCULATION IDENTIFICATION NUMBER				OPTIONAL WORK PACKAGE NO.
J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	
05996.01	Geotechnical	G(P05)-1	N/A	N/A

* APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION * REQUIRED (✓)	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
Chin Man Mok 3/12/97 Chin Man Mok	C.-Y. Chang / 3/31/97 C.-Y. Chang	C.-Y. Chang 3/31/97 C.-Y. Chang	0	N/A		X
Chin Man Mok 6/9/97 Chin Man Mok	C.-Y. Chang 6/9/97 C.-Y. Chang	C.-Y. Chang 6/9/97 C.-Y. Chang	1	0		X

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B PDR

CALCULATION SUMMARY

A5010.62

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CLIENT/PROJECT	QA CATEGORY / CODE CLASS
Private Fuel Storage Facility, LLC Private Fuel Storage Facility, Skull Valley, Utah	I

SUBJECT/TITLE Development of soil and foundation parameters in support of dynamic soil-structure interaction analyses.

OBJECTIVE OF CALCULATION

(1) To develop strain-compatible dynamic soil parameters for SASSI analyses.
(2) To develop dynamic parameters for uncoupled soil spring, dashpot, and mass model.

CALCULATION METHOD/ASSUMPTIONS

(1) 1-D equivalent linear site response analysis using complete program "shake." The purpose is to develop strain-compatible properties corresponding to the design earthquake.
(2) Used weighted average to estimate equivalent homogenous and isotropic soil parameters consistent with strain-compatible properties developed in (1).
(3) Calculate equivalent soil spring, dashpot, and mass parameters to match with the analytical solution of vibration of rigid rectangular footing on homogenous isotropic elastic halspace.

SOURCES OF DATA/EQUATIONS

See List of References on Page 9 of 73.

Computer Programs:

(1) SHAKE: A computer program for earthquake response analysis of horizontally layered sites rev. 96 GMX, Benchmarked against SHAKE91, a commercially available version of the code.

CONCLUSIONS

(1) Strain-compatible dynamic soil parameters were developed for horizontally layered system. These parameters will be used in SASSI analyses. Results are listed on Page 10 of 73 in Section 1.1.
(2) Equivalent soil spring, dashpot, and mass parameters were developed, results are listed on Page 11 of 73 in Section 1.1.

REVIEWER(S) COMMENTS		
	PREPARER: Chin Man Mok	DATE: 3/12/97
	REVIEWER/CHECKER: C.-Y. Chang	DATE: 3/13/97
	INDEPENDENT REVIEWER	DATE

RECORD OF REVISIONS

REV. 0 - ORIGINAL ISSUE

REV. 1 - REV. 1 WAS PERFORMED TO REVISE THE COMPR. WAVE VEL. FOR TWO SOIL LAYERS (SEE PG. 10). DETAILED DESCRIPTION OF REVISION IS AS FOLLOWS:

Revision Item: Dynamic Soil Parameters for SASSI Model

The compressional-wave velocity at the soil layers between Depths of 30 to 120 feet is revised from 2500 to 4000 fps.

Reasons for Revision: Compressional-wave Velocity Inconsistent with Shear-wave Velocity

In the first calculation package, both the compressional-wave velocity (2500 fps) and shear-wave velocity were idealized based on typical values collected in the geophysical survey. However, the Vp-Vs combinations are not physically admissible.

Revision: Increase Compressional-wave Velocity

In order to eliminate the consistency problem. the compressional-wave velocity for these layers were modified such that the corresponding Poisson's ratios are approximately the same as those for the top layers.

typical shear-wave velocity $Vs = 800$ fps for top layers

typical compressional velocity $Vp = 1600$ fps for top layers

corresponding Poisson's ration $$v = \frac{0.5 - \left(\frac{Vs}{Vp}\right)^2}{1 - \left(\frac{Vs}{Vp}\right)^2} \qquad v = 0.333$$

typical shear-wave velocity $Vs = 2000$ fps for soil layers between Depths of 30 to 120 feet

corresponding compressional-wave velocity

$$Vp = Vs \cdot \sqrt{\frac{1 - v}{0.5 - v}} \qquad Vp = 4 \cdot 10^3 \qquad \text{fps}$$

Other Consequences of Revision: Negligible Effects on subsequent analyses

(1) The design response spectra were developed based on the results of seismic hazard analyses. Therefore, they will not be affected by this revision of compressional-wvave velocity.

(2) The idealized model parameters for SSI analyses using the uncoupled soil spring, dashpot, and mass models were estimated based on a set of representative equivalent homogeneous properties. These properties were assumed to be equal to the weighted average of soil parameters within 30 feet below the foundation. Since this revision is for the soil layers below depths of 30 feet, the parameters for the winkler's foundation model are not affected by this change.

CALCULATION SHEET	J.O./W.O./CALCULATION NO. 05996.01-G(P05)-1	REVISION 1	PAGE 3 of 73

PREPARER/DATE	REVIEWER/CHECKER/DATE	INDEPENDENT REVIEWER/DATE
Chin Man Mak 6/9/97	C. J. Cheng 6/9/97	C. J. Cheng 6/9/97

SUBJECT/TITLE	QA CATEGORY/CODE CLASS
Private Fuel Storage Facility, LLC Private Fuel Storage Facility, Skull Valley, Utah	I

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(Revisions, Additions, Deletions, Etc.)

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3	Table of Contents	1	6/09/97	
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Sec.1.2 Page 22	Soil Data Summary			
Sec. 1.3 Page 32	Site Response Analysis			
Sec 1.4 Page 61	Soil Parameters for SASSI Model			
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PRIVATE FUEL STORAGE FACILITY

SKULL VALLEY, UTAH

1.0 AVAILABLE SUBSURFACE INFORMATION

A total of 14 exploratory borings (A-1 to D-4) and 2 seismic refraction/reflection profile surveys were performed over an area of about 2000 ft by 2000 ft in the vicinity of the fuel storage area. Most borings are about 50 feet deep, and a few borings extend to a depth of about 100 feet. Standard Penetration Test (SPT) were performed and samples were collected at 5 feet intervals in these borings. Geotechnical laboratory tests were conducted on these collected samples in shallow depths. These tests included determination of water content, Atterberg Limits, fines percentage, specific gravity, consolidation tests, and unconsolidated-undrained triaxial compression tests. Additional geologic information were provided by Dr. James Baer at Brigham Young University (Stone and Webster, 1997).

2.0 SUBSURFACE CONDITIONS

The fuel storage area generally consists of three soil units: an uppermost clayey silty layer followed by a sandy layer overlying a silty layer.

The uppermost layer is composed primarily of interlayered silt, silty clay, and clayey silt. It extends to a depth of about 25 to 35 feet. They probably represent deeper water facies of Lake Bonneville. They are either dry or damp. The clayey silts and silty clays are commonly slightly to moderately plastic, with some being highly plastic. The SPT blowcounts are mostly between 8 and 20 blows per foot. They are 'stiff' or 'medium dense'. The undrained shear strength of the clayey silt samples collected between depths of 10 to 12 feet is about 2200 to 2400 psf. The maximum past pressure of this clayey silt

is approximately 6000 psf. Due to deformation of the weakly cemented structure of the silt, large secondary consolidation was observed in the consolidation tests. The back-calculated shear-wave velocity of this layer ranges approximately between 690 and 950 feet per second, and the compressional-wave velocity varies from 1140 to 1720 feet per second.

Underlying the clayey silty unit is a layer of very dense, dry, and fine sand. Its thickness varies approximately between 25 to 30 feet. The SPT blowcounts commonly exceed 100 blows per foot. Refusal (blowcounts greater than 100 blows per 6 inches) are often encountered. The back-calculated compressional-wave velocities range approximately between 2180 and 3480 feet per second. The shear-wave velocities vary from about 1680 feet per second to about 2610 feet per second.

Underlain the sand is a layer of very dense silt. Thin layers of fine gravel. coarse sand, and clayey zones are present, indicating a near-shore deposition environment. The shear-wave velocity and the compressional-wave in the zone above water table are believed to be similar to the upper sand layer.

Bedrock was not encountered in the exploratory borings. However, seismic reflection survey data indicates that the depth to bedrock is between 550 feet and 700 feet below the ground surface. According to Dr. James Baer at Brigham Young University, Hickman Knolls may be a detached slide block floating in the unconsolidated valley-fill sediments. His interpretation is based on his gravity survey at the north end of Skull Valley.

Groundwater table was not encountered in the borings. It is believed to be at a depth of about 120 feet where the compressional-wave velocity changes approximately from 2700 to 5600 feet per second.

3.0 SELECTION OF DYNAMIC PARAMETERS

The dynamic behavior of foundation depend primarily on the seismic response of the clayey silty layer underneath the structure. In order to estimate the strain-compatible soil properties of this layer in the event of the design earthquake, one-dimensional site response anlaysis was performed. The dynamic foundation parameters in support of the soil-structure interaction analyses were derived from the results of this one-dimensional site response analysis.

3.1 ONE DIMENSION SITE RESPONSE ANALYSIS

An input rock outcrop motion with peak acceleration of 0.7 g was used and specified at rock outcrop at a depth of 600 feet. A comparison of the response spectrum of this input motion with the design response spectrum is shown on Figure 3-1. The response spectrum of the input motion used resembles the design response spectrum.

The density of the surficial silty clayey soils in the top 30 feet was estimated to be 81 pcf. This value corresponds to the average density of the samples collected at a depth of about 11 feet. There is no density data for soils below this layer. The density values were selected based on typical data published in Naval Facilities Engineering Command (1982), Bowles (1979), and Terzaghi and Peck (1967). In between the depths of 30 and 60 feet, the density of the dry dense sand was assumed to be 115 pcf. For the dense silty soil from a depth of 60 feet to the water table at a depth of 120 feet, the density was assumed to be 120 pcf. This value corresponds to a dry density of about 115 pcf and a moisture content of about 5 %. Below the water table to the bedrock at a depth of 600 feet, the density of the wet dense silty soil was assumed to be 130 pcf. The density of the bedrock was assumed to be 150 pcf. The density profile is summarized in Table 3-1.

TABLE 3-1

Depth (ft)		Density	Type	Water	Remarks
Top	Bottom	(pcf)		Table	
0	30	81	silt/clay	above	from laboratory tests
30	60	115	sand	above	uniform fine sand, close to maximum dry density
60	120	120	silt	above	uniform inorg silt, ~max dry den (~115 pcf), slightly moist (~5%)
120	600	130	silt	below	uniform inorganic silt, close to maximum wet density
600		150	rock	below	typical

The idealized shear-wave velocity profile is shown on Figure 3-2. The maximum past pressure experienced by the uppermost silty clayey layer was about 6000 psf. It is assumed that this maximum pressure was caused by approximately of an additional 80 feet of soils above the current ground surface. The idealized shear-wave velocity in the surficial clayey silty layer (top 30 feet) was estimated based on the assumption of (1) a constant c/p ratio (ratio of cohesion to overburden pressure), (2) a constant G/c ratio (ratio of shear modulus to cohesion), and (3) a shear-wave velocity of 750 feet per second at mid-depth of this layer. For the dense sandy and silty layer below the surficial layer, the shear-wave velocity was estimated based on the assumptions of (1) a shear-wave velocity of 2000 feet per second near the top of this layer, and (2) K_{2max} being linearly proportional to the density of the soil.

The shear modulus reduction curves and the damping ratio curves used are shown on Figures 3-3 through 3-8. The curves for the surfical clayey silty layer (top 30 feet) are based on Vucetic and Dobry (1991) for normally consolidated soil with a plasticity index of 15. The shear modulus reduction curve for the dense sand layer between depths 30 and 60 feet is based on the upper bound curve published by Seed and Idriss (1970). The damping curve for this layer is based on the lower bound curve by Seed and Idriss (1970). For the dense silt between depths of 60 and 120 feet, the shear modulus reduction curve is based on the average of that corresponding to low plasticity clay (Vucetic and Dobry, 1991) and the upper bound curve for sand (Seed and Idriss, 1970). Similarly, the damping curve is based on the average of that corresponding to low plastic clay (Vucetic and Dobry, 1991) and the lower bound curve for sand (Seed and Idriss, 1970). For the

deeper silty soils, the reduction curves and damping curves were derived based on the assumption that the reference strain increases by a factor of 3 for each order of magnitude change in confining pressure (EPRI, 1993). Therefore, the curves (for silts between depths of 60 and 120 feet) were shifted to the right (direction of increasing strain) by a factor of 1.5 for silts between depths of 120 and 300 feet; by a factor of 2 for silts between depths of 300 and 500 feet; and a factor of 2.4 for silts between depths.

The strain-compatible shear-wave velocity and damping ratio profiles determined from the site response analyses are shown on Figures 3-9 and 3-10, respectively.

The site response analyses were conducted using Geomatrix's in-house version of program SHAKE which has been bench marked against the published commercial version, SHAKE.91 (NISEES, 1991)

3.2 IDEALIZED SOIL PROFILE FOR SASSI ANALYSES

Based on the strain-compatible profiles obtained from one-dimensional site response analysis, an idealized horizontally layered soil profile were developed in support of the SSI analyses based on SASSI continuum model. The dynamic properties for this idealized layer are presented in Table 3-2. The compressional-wave velocity profile is estimated based on the back-calculated values from seismic reflection/refraction survey. The damping ratios for compressional-wave is assumed to be the same as those for shear-wave, and are limited to be not greater than 10% (Geomatrix, 1996).

3.3 IDEALIZED MODEL PARAMETERS FOR SSI ANALYSES BASED ON UNCOUPLED SOIL SPRING, DASHPOT, AND MASS MODEL

Based on the soil profile obtained from one-dimensional site response analysis, the representative equivalent shear-wave velocity is estimated to be 515 feet per second. The representative shear-wave damping ratio is about 11 percent. These representative values were computed based on the weighted average of the values within 30 feet below the

foundation. The weighting factors were assumed to decrease linearly with increasing depth, to zero at a depth of 30 feet. The average compressional-wave velocity is selected to be 1500 feet per second. These values correspond to a shear modulus of about 668 ksf, a Young's modulus of about 1915 ksf, a Poisson's ratio of 0.433. Based on Newmark and Rosenblueth (1971) for a surface rectangular foundation of 30 feet by 66 feet, the equivalent dynamic soil parameters were estimated. The estimated parameters are presented in Table 3-3.

3.4 REFERENCES

Bowles, J.E., (1979). "Physical and Geotechnical Properties of Soils," McGraw Hill. Inc.

Electric Power Research Institute, (1993), "Guidelines for Determining Design Basis Ground Motions", EPRITR-102293

Geomatrix Consultants, Inc., (1996), "Recommendations of Site Response Analyses of Vertical Excitation, Richmond-San Rafael Bridge Seismic Retrofit Design"

Geosphere, (1997). "Seismic Survey of Private Fuel Storage Facility", Geosphere Midwest, Midland, MI, January, 1997.

NISEES (1991), SHAKE-91: National Information Service for Earthquake Engineering Software, University of California, Berkeley.

Department of the Navy. (1982), "Soil Mechanics, Design Manual 7.1", Naval Facilities Engineering Command.

Newmark, N.M. and Rosenblueth, E., (1971), "Fundamentals of Earthquake Engineering". Prentice Hall, Inc.

Seed, H.R. and Idriss, I.M., (1970), "Soil Moduli and Damping Factors for Dynamic Response Analyses", Report. No. EERC 70-10, University of California at Berkeley.

Stone and Webster. (1997), "Geotechnical Laboratory Test Report", Stone and Webster Engineering Corporation, Boston, MA, January, 1997.

Terzaghi, K. and Peck, R.B., (1967), "Soil Mechanics in Engineering Practice", John Wiley and Sons. Inc.

Vucetic, M. and Dobry. R., (1991), "Effect of Soil Plasticity on Cyclic Response", Journal of Geotechnical Engineering, Vol. 117, No. 1, Jan 1991, pp 89-107.

Table 3-2
Dynamic Soil Parameters for SASSI model

Depth (ft)		Density	Wave Velocity (fps)		Damping Ratio (%)	
Top	Bottom	(pcf)	Shear	Compressional	Shear	Compressional
0	5	81	637	1500	5	5
5	10	81	520	1500	10	10
10	15	81	469	1500	12	10
15	20	81	353	1500	16	10
20	25	81	327	1500	17	10
25	30	81	280	1500	19	10
30	60	115	1809	4000	4	4
60	120	120	1861	4000	8	8
120	300	130	2080	5600	8	8
300	600	130	2440	5600	8	8
600		150	5000	10000	2	1

Table 3-3

Dynamic Soil Parameter for Spring, Dashpot, and Mass Model

Vertical Vibration Mode:			
Distributed Mass per Area	=	30.0	pcf-sec2
Distributed Vertical Dashpot Constant per Area	=	1.94	kcf-sec
Distributed Vertical Spring Constant per Area	=	59	kcf
Horizontal Vibration Mode:			
Distributed Mass per Area	=	5.5	pcf-sec2
Distributed Horizontal Dashpot Constant per Area	=	0.97	kcf-sec
Distributed Horizontal Spring Constant per Area	=	40	kcf
Rocking Vibration Mode:			
Distributed Mass per Area	=	38.6	pcf-sec2
Distributed Vertical Dashpot Constant per Area	=	1.39	kcf-sec
Distributed Vertical Spring Constant per Area	=	138	kcf

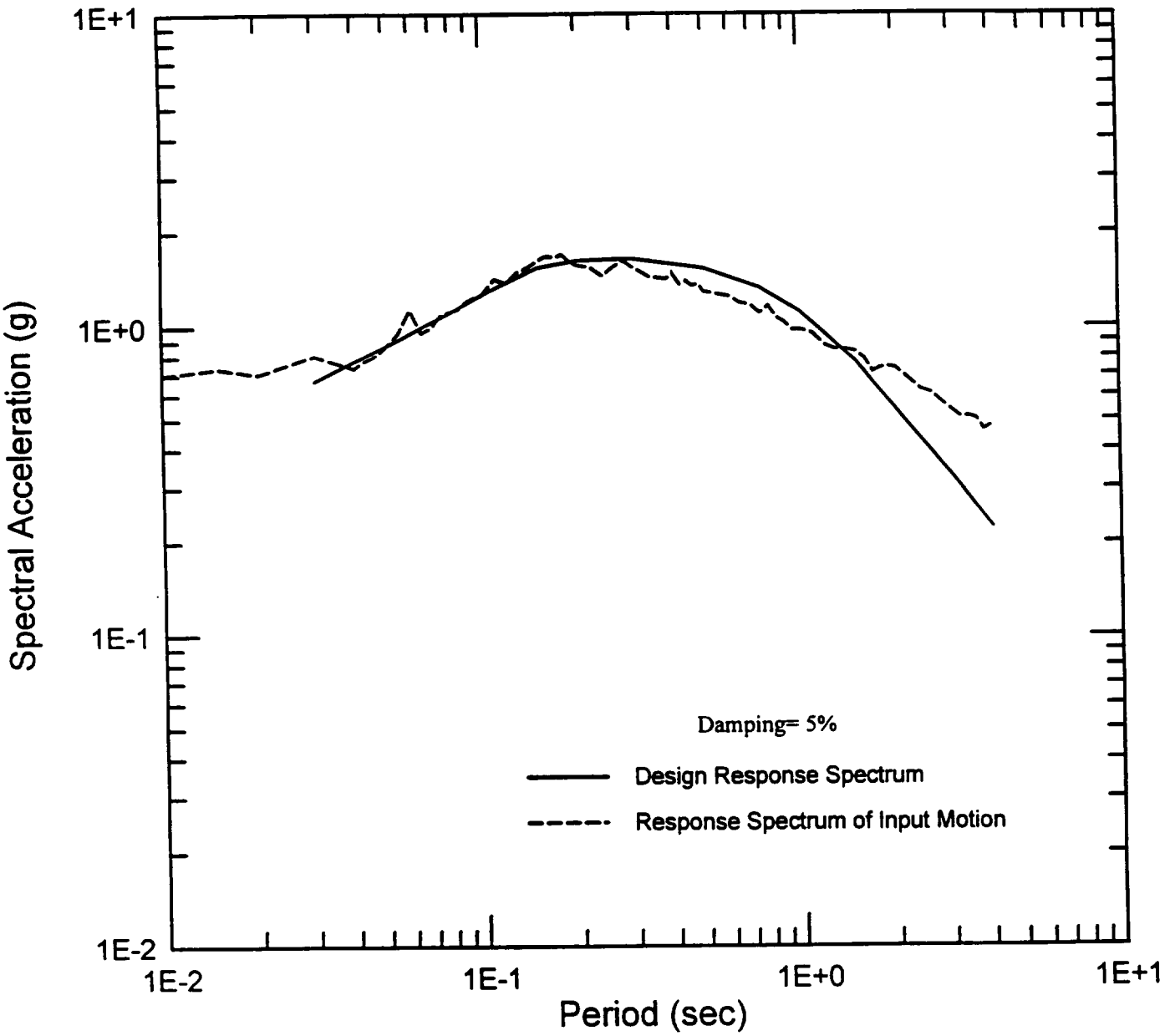


Figure 3-1: Comparison of Response Spectrum of Input Motion with Design Response Spectrum

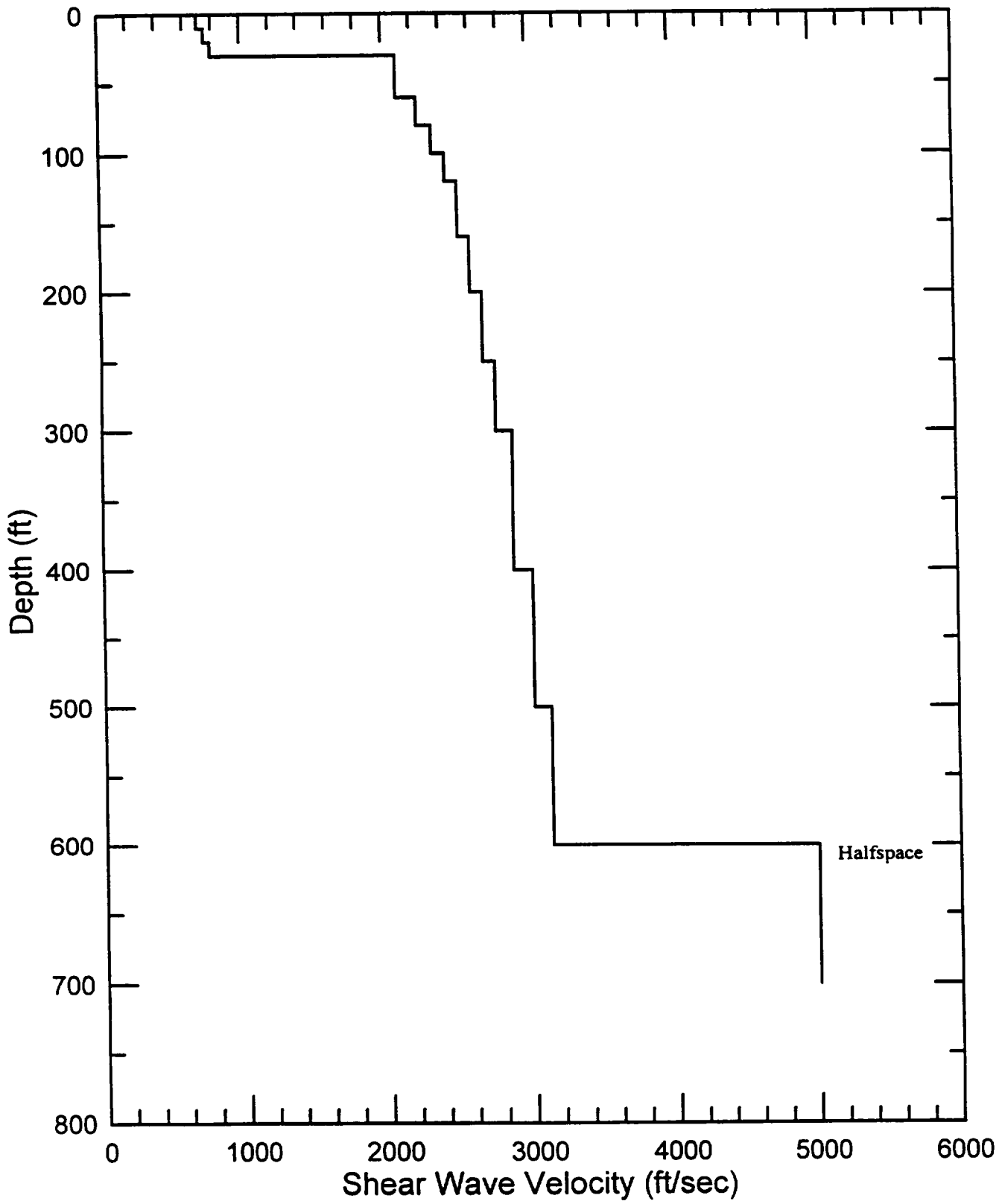


Figure 3-2: Idealized Shear-Wave Profile

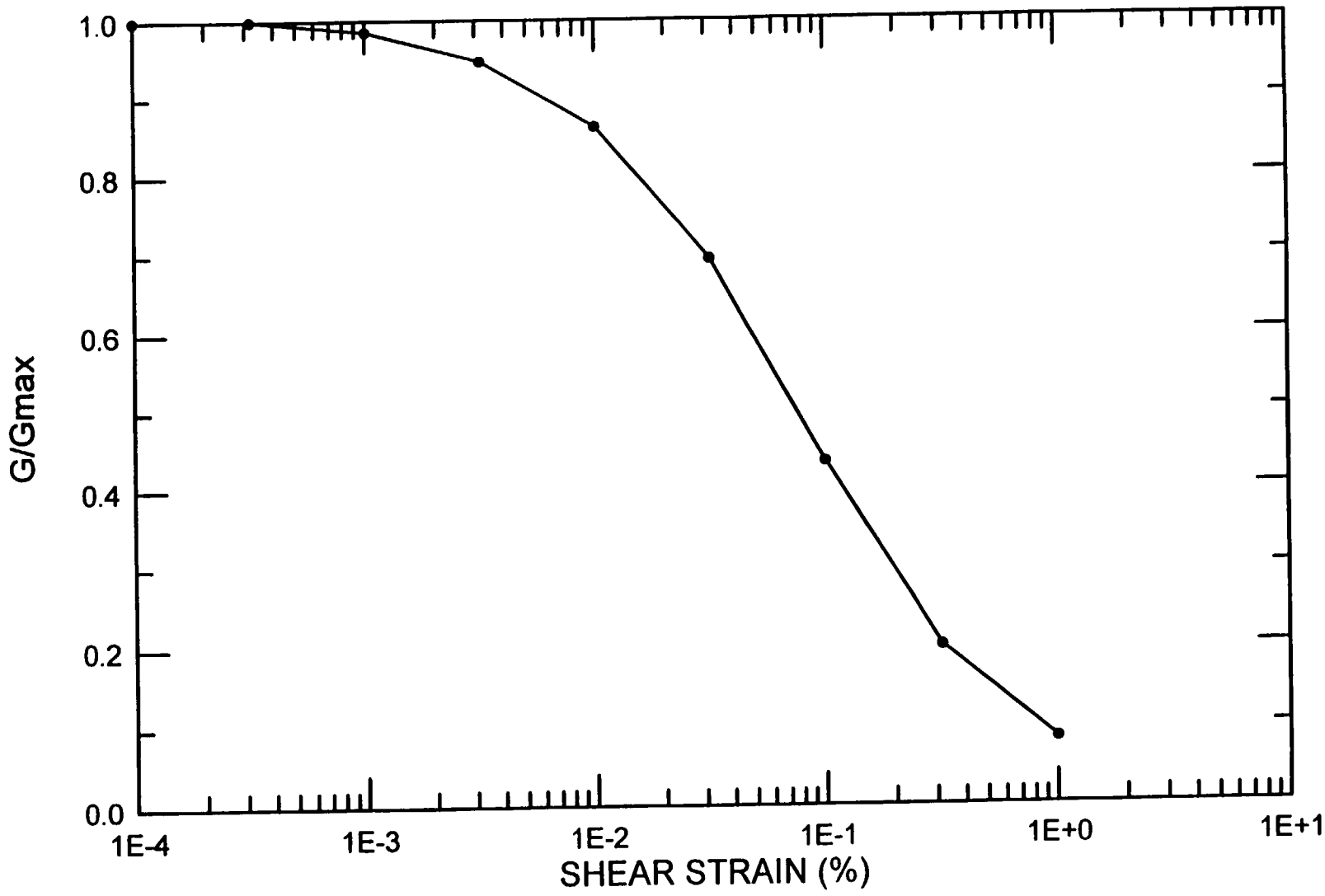


Figure 3-3: Shear Modulus Reduction Curve for Silty Clay (0-30 ft)

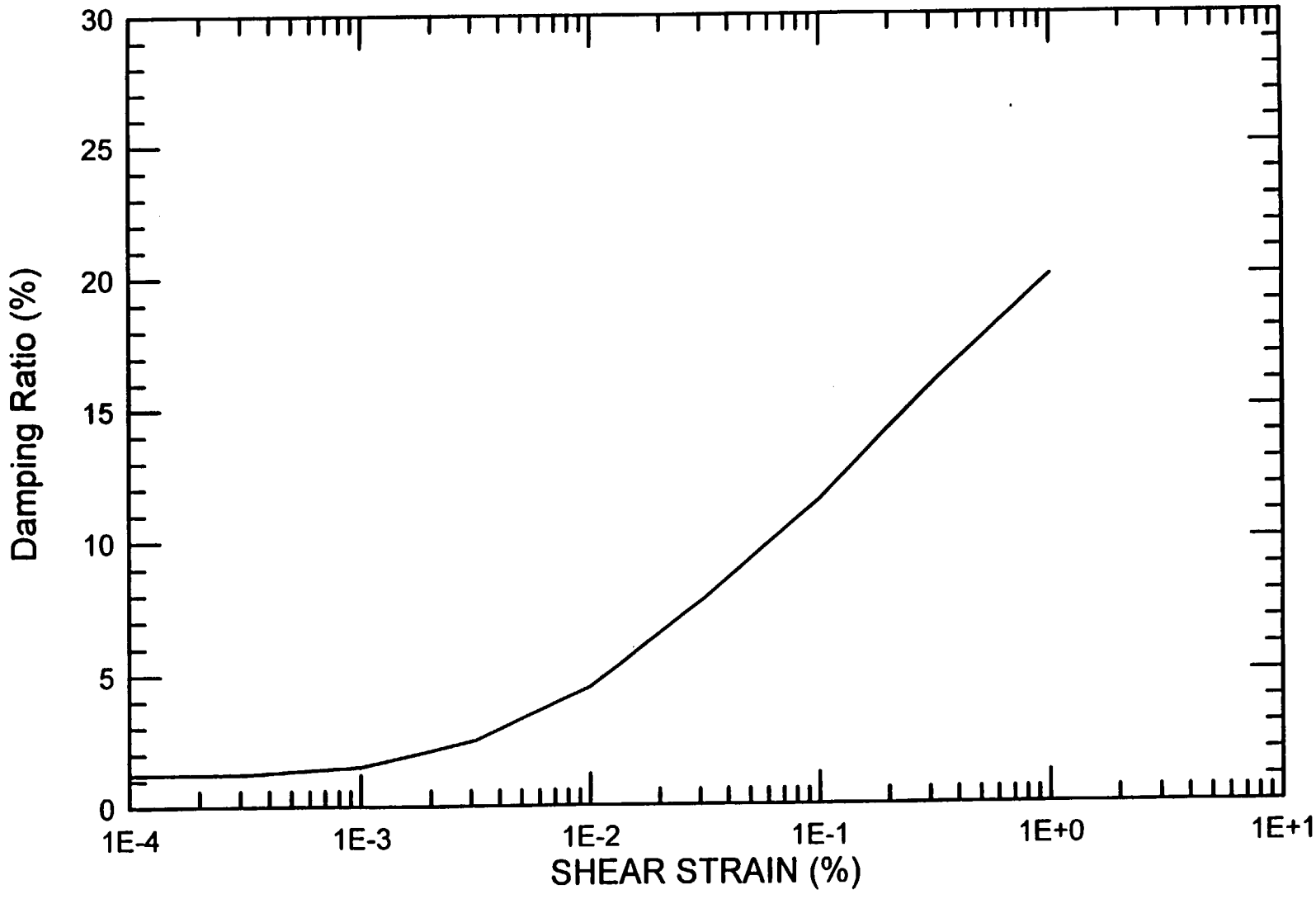


Figure 3-4: Damping Ratio Curve for Silty Clay (0-30 ft)

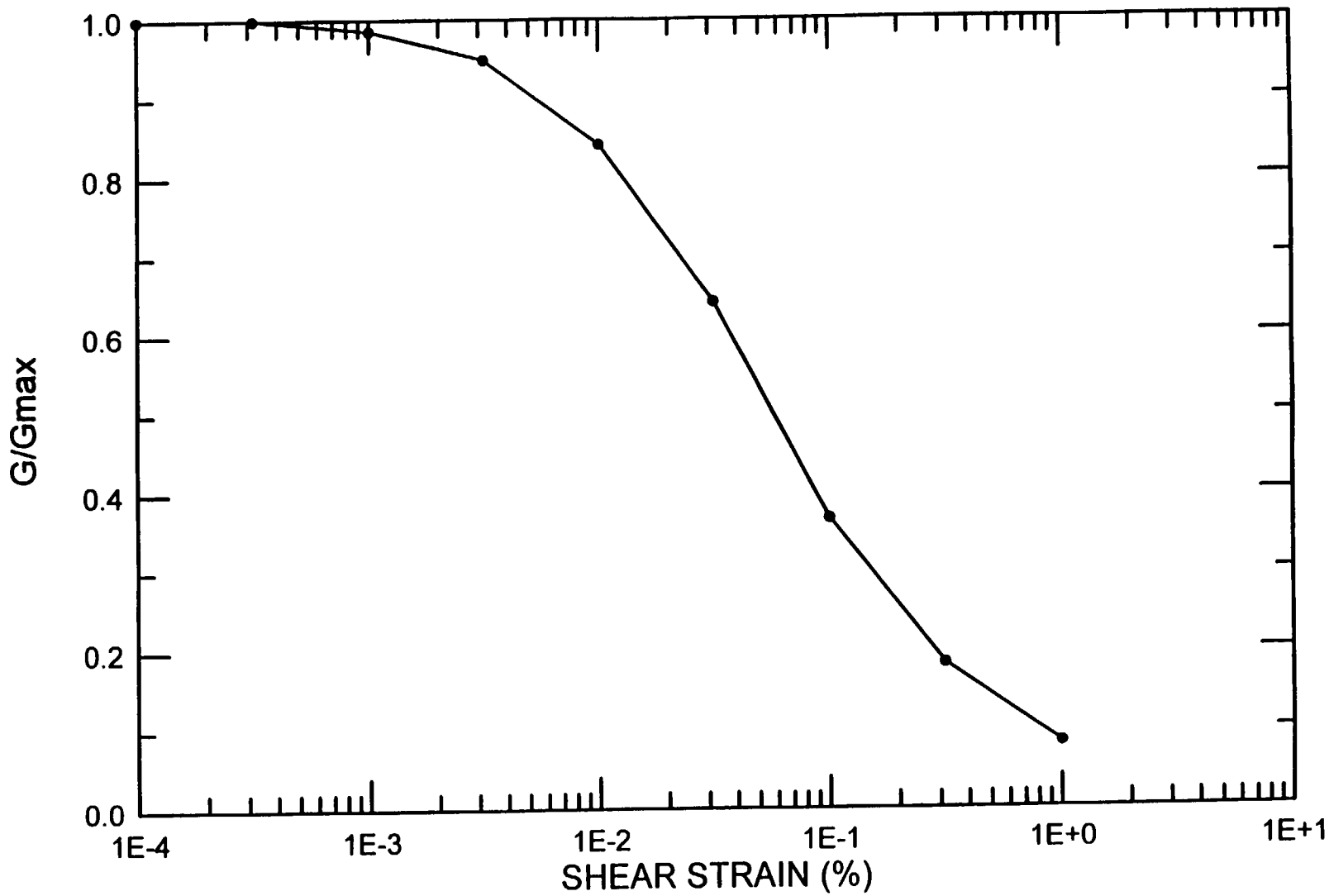


Figure 3-5: Shear Modulus Reduction Curve for Sand (30-60 ft)

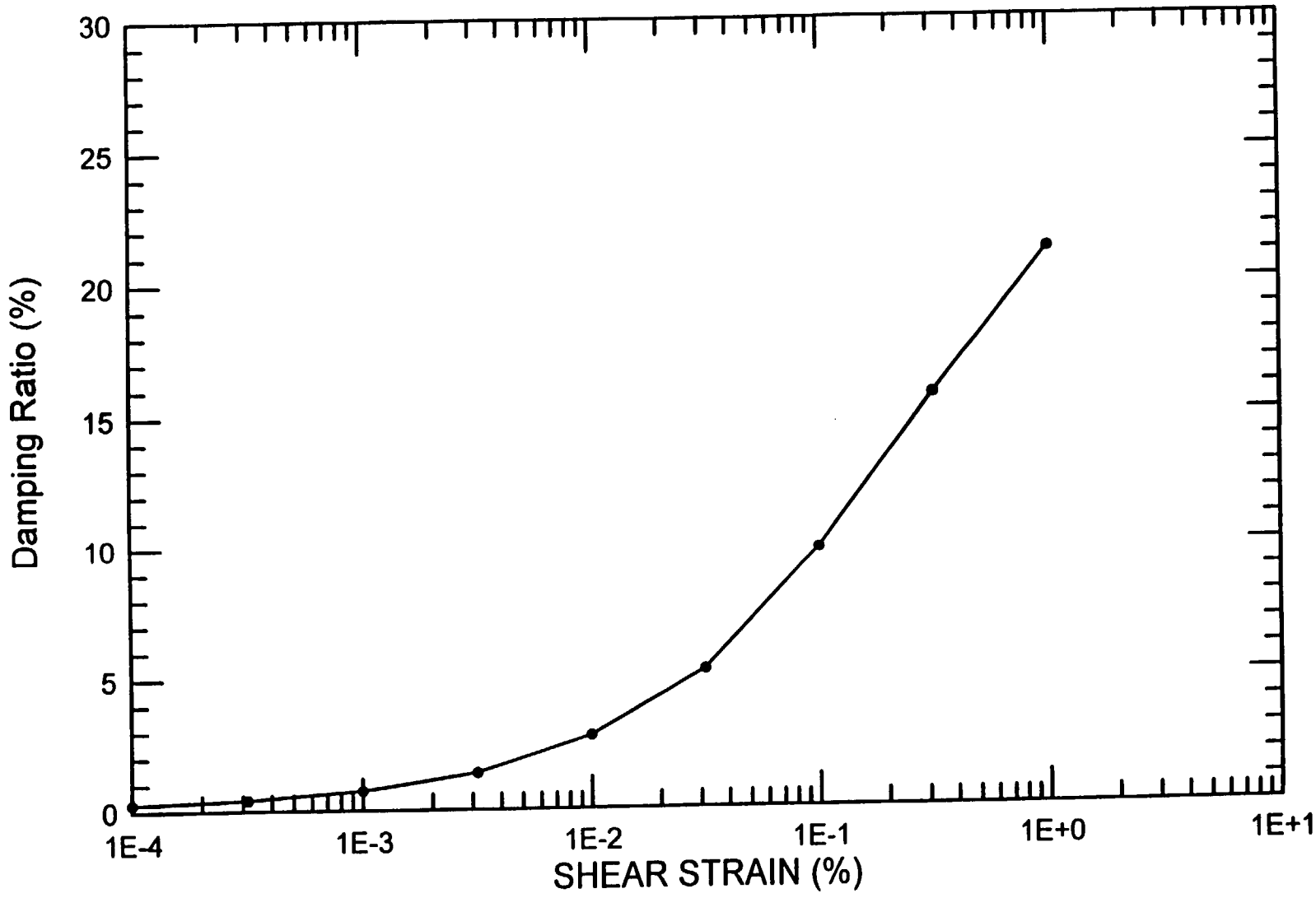


Figure 3-6: Damping Ratio Curve for Sand (30-60 ft)

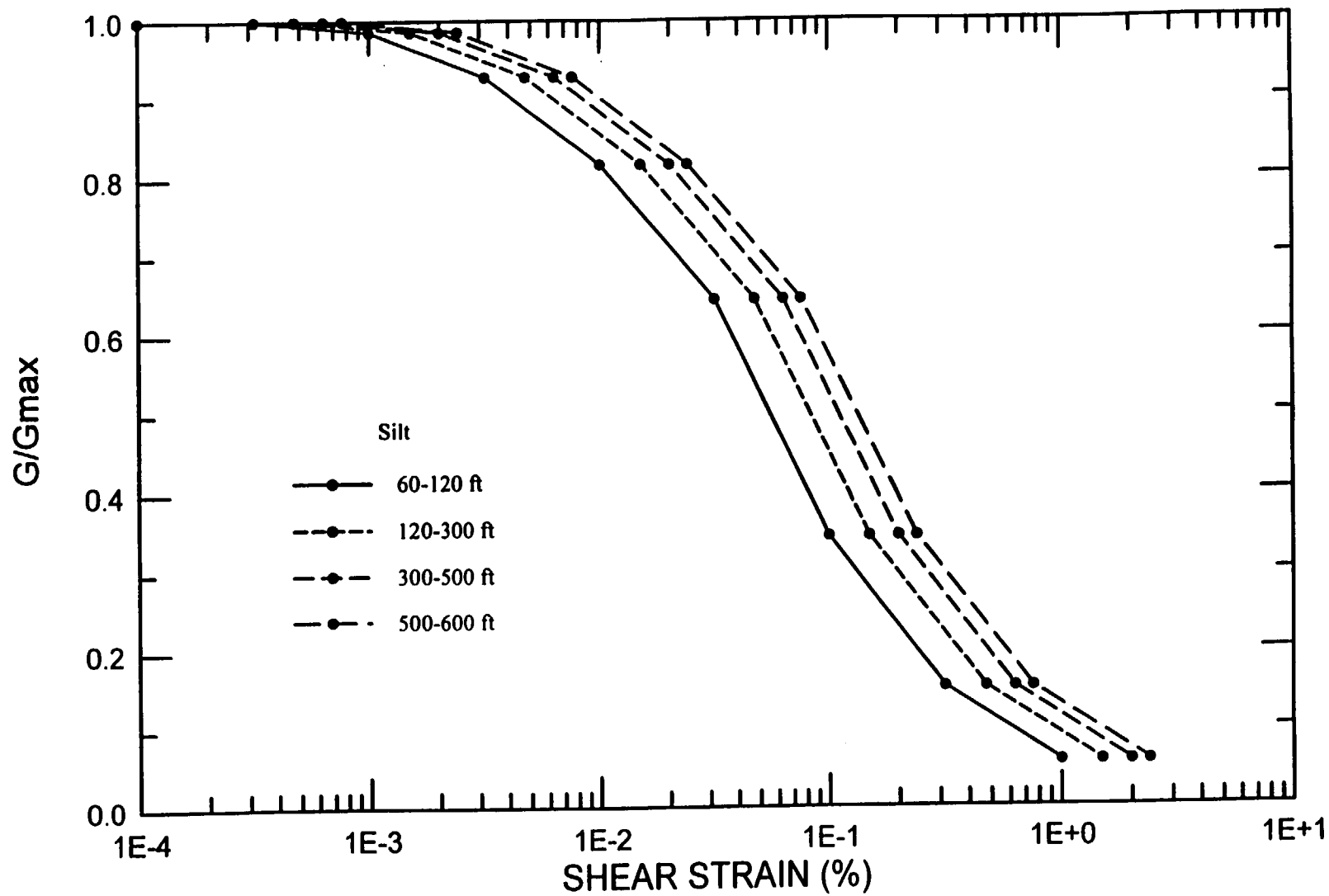


Figure 3-7: Shear Modulus Reduction Curves for Silts

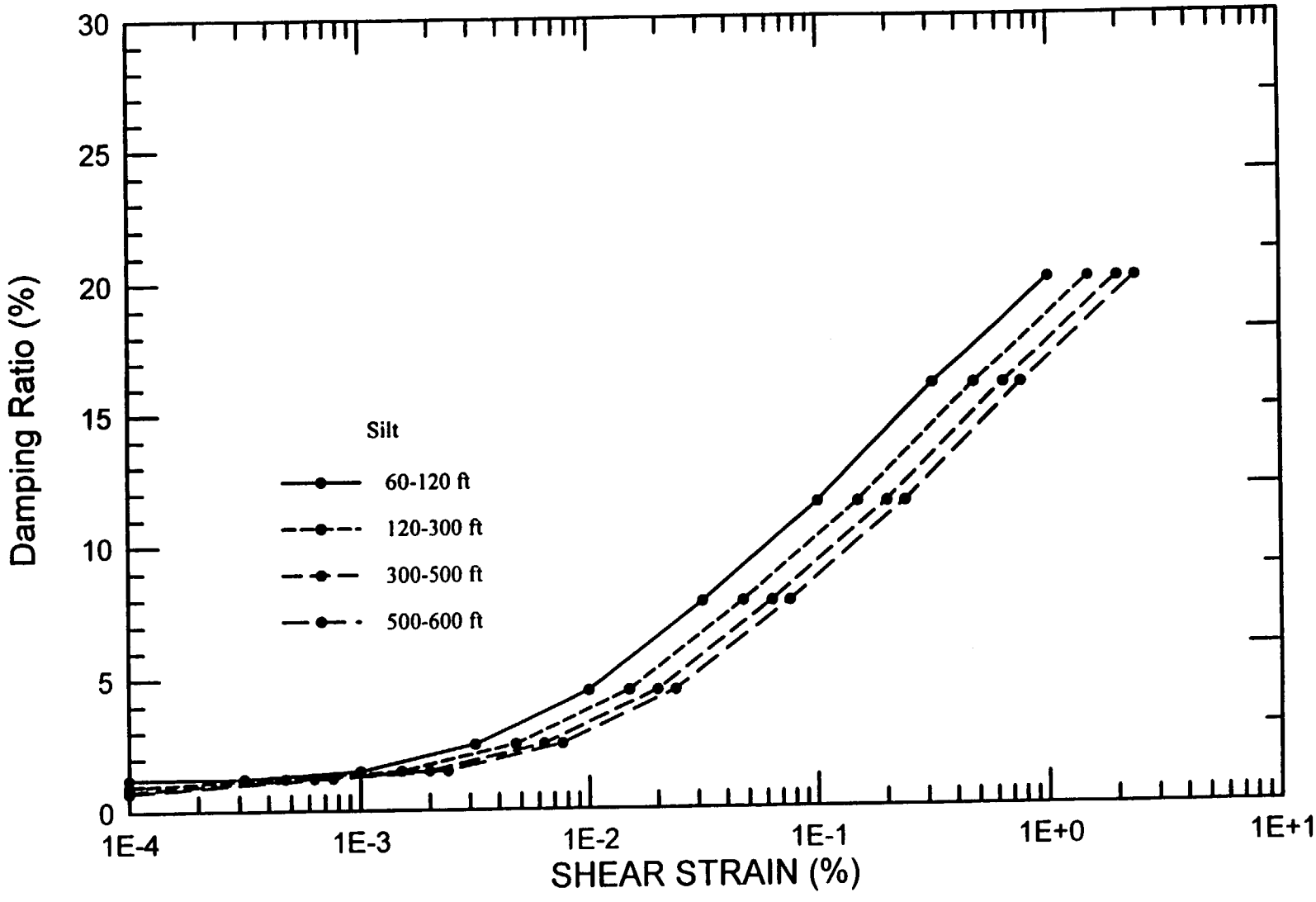


Figure 3-8: Damping Ratio Curves for Silts

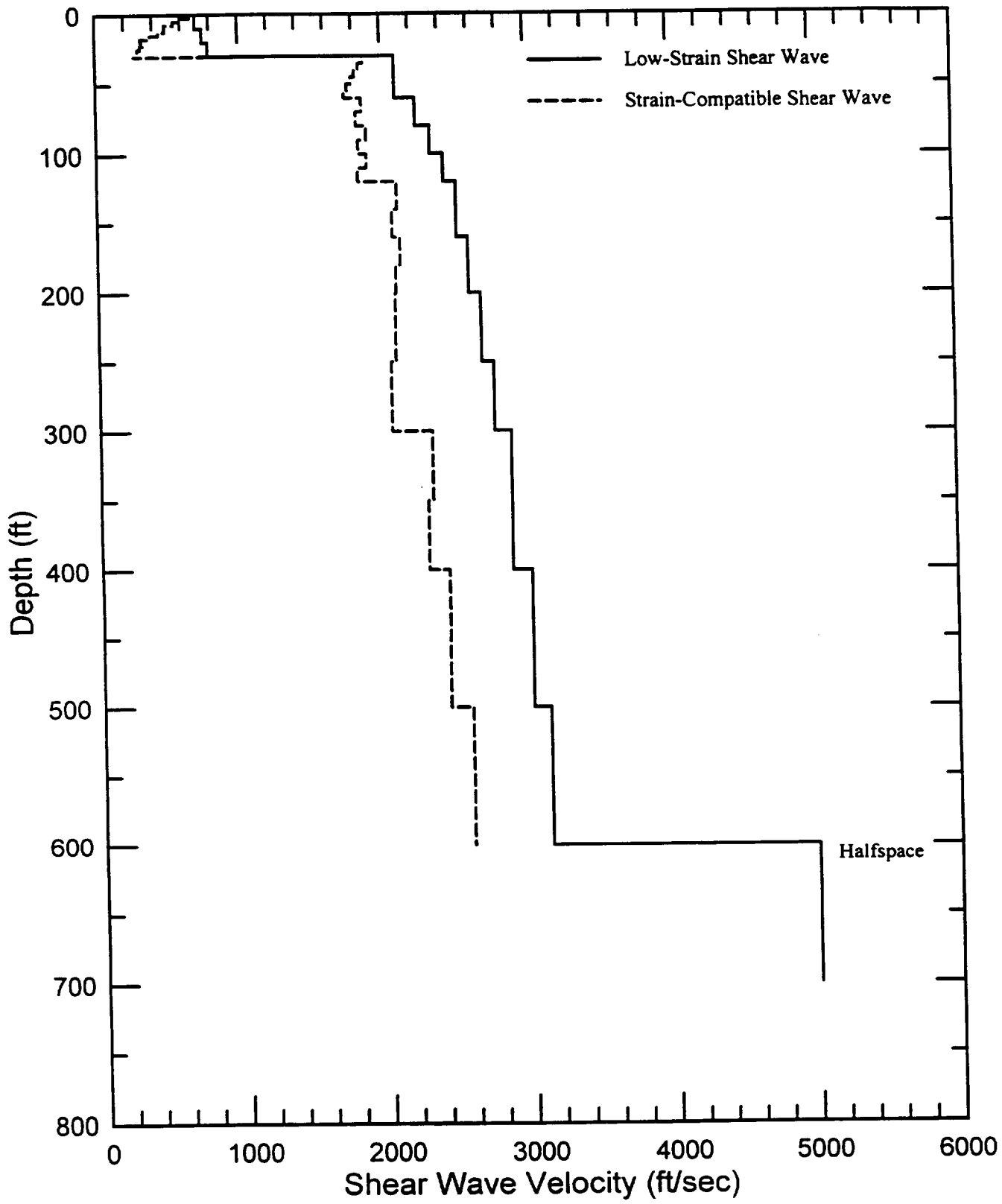


Figure 3-9: Strain-Compatible Shear-Wave Profile

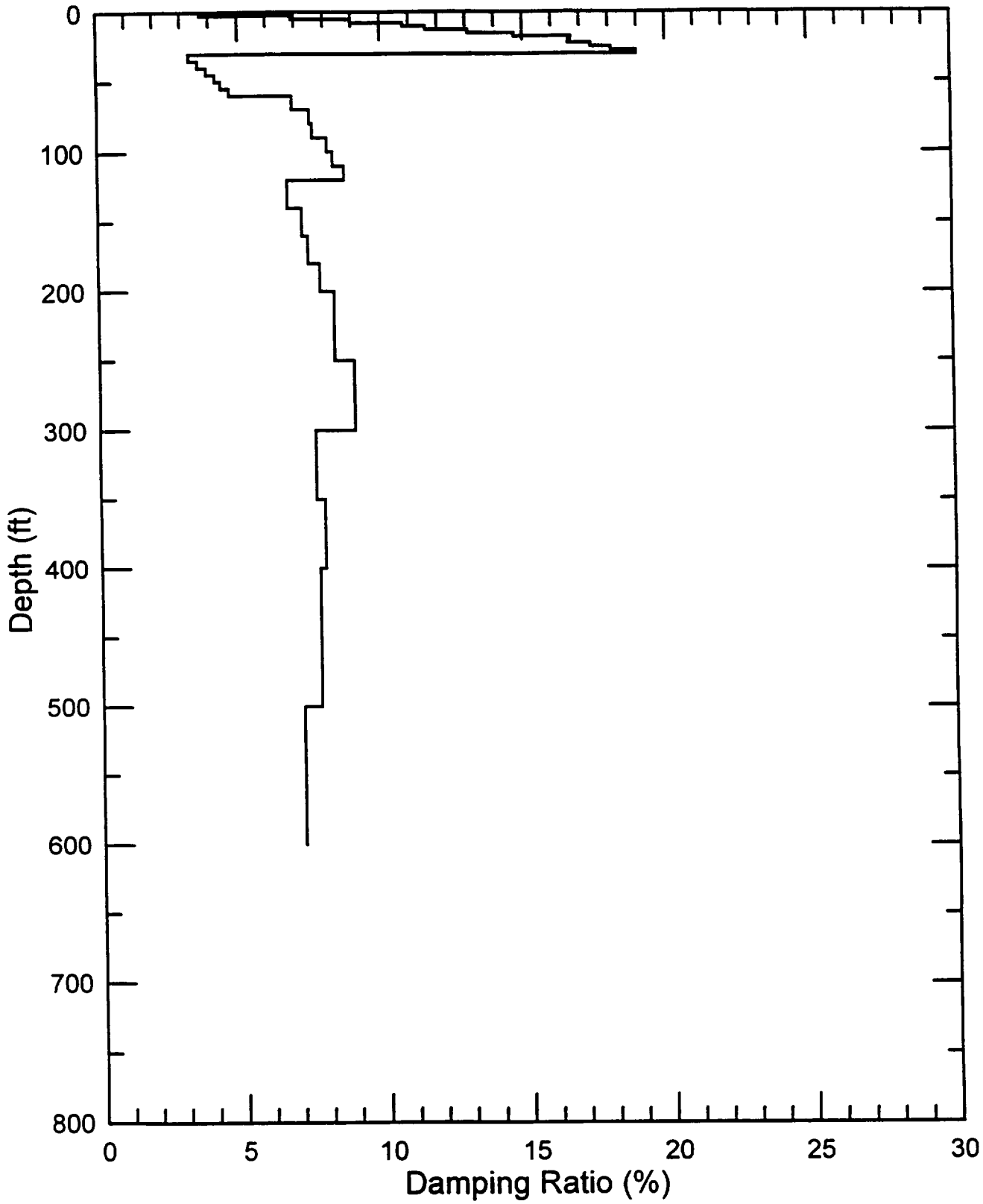


Figure 3-10: Strain-Compatible Damping Ratio Profile

GEOTECHNICAL SITE CHARACTERIZATION

REFERENCES:

(1) Geotechnical Laboratory Test Report, January 1977 by Stone and Webster Engineering Corporation.

(2) Seismic Survey. January 1997 by Geosphere Midwest.

GENERAL SITE PROFILE

0→25~ft.

- Mainly interlayered silt, silty clay, and clayey silt
- Silty clays and clayey silts are commonly slightly to moderately plastic, some are highly plastic.
- SPT N-value mostly between 8-20 bl/ft.
- Significant secondary consolaidation due to weakly => 'stiff'= or medium dense materials cemented silt
- Po'≈ 3 tsf
- Deeper water facies of Lake Bonneville
- Su = 2.2 to 2.4 ksf for Z lab samples at 10 to 12 ft. deep

25~35→~120

- SPT N-Values commonly exceed 100 bl/ft. and refusal conditions are often encountered
- Upper 25 to 30 ft — very dense, dry, and fine sand
- Below 30 ft — very dense silt with thin layers of gravel and coarse sand, occassionally => near-shore deposition environment, also with claying zones but not signigicant.

Water Table	=	believed to be at ≈ 120 ft. deep
Bedrock	=	between 550 and 700 ft. below ground surface
Hickman Knolls	=	Dr. James Baer at BYU → likely on detached slide block

Summary of Density Data

Boring	Sample	Depth	Test	Description	Dry Unit Weight (pcf)	Water Content (%)	Bulk Density (pcf)
B-4	U-3D	10.4	UU	silty CLAY/clayey SIL	67.1	27.4	85.5
C-2	U-2D	11.1	UU	clayey SILT	57.9	35.6	78.5
C-1	U-3B	10.8	C	clayey SILT	64.7	30.3	84.3
C-1	U-3C	11.2	C	clayey SILT	55.8	38.9	77.5
C-1	U-3D	11.4	C	clayey SILT	51.7	46.7	75.8
C-2	U-2C	10.9	C	clayey SILT	64.9	27.6	82.8
C-2	U-2E	11.7	C	clayey SILT	57.5	39.7	80.3
average					59.9	35.2	80.7

Summary of Density Data										
Boring	Sample	Depth	Test	Description	Dry Unit Weight (pcf)	Water Content (%)	Bulk Density (pcf)	Po (psf)	H1 (I/H) (ft)	H2 (J-C) (ft)
C-1	U-3B	10.8	C	clayey SILT	64.7	30.3	84.3	6600	78.3	67.5
C-1	U-3C	11.2	C	clayey SILT	55.8	38.9	77.5	6600	85.2	74.0
C-1	U-3D	11.4	C	clayey SILT	51.7	46.7	75.8	6200	81.7	70.3
C-2	U-2C	10.9	C	clayey SILT	64.9	27.6	82.8	6400	77.3	66.4
C-2	U-2E	11.7	C	clayey SILT	57.5	39.7	80.3			
average					58.9	36.6	80.2	6450.0	80.6	69.5

Density					
Depth (ft)		Soil	Density	Water	Source
Top	Bottom	Type	(pcf)	Table	
0	30	Silt/Clay	81	above	lab tests
30	60	Sand	115	above	DM7, uniform fine sand, ~max density, dry
60	120	Silt	120	above	DM7, uniform inorganic silt, ~max dry density (115%), slightly moist (~5%)
120	600	Silt	130	below	DM7, uniform inorganic silt, ~max wet density
600		rock	150	below	Typical

Shear-wave Velocity

Depth (ft)	Density (pcf)	Sigv' (psf)	Po' (psf)	Vs (fps)	
0	81	0	5600	681	
2	81	162	5762	690	
4	81	324	5924	700	→ 705 @ 5
6	81	486	6086	710	
8	81	648	6248	719	
10	81	810	6410	728	
12	81	972	6572	737	
14	81	1134	6734	746	→ 751 @ 15
16	81	1296	6896	755	
18	81	1458	7058	764	
20	81	1620	7220	773	
22	81	1782	7382	781	
24	81	1944	7544	790	→ 794 @ 25
26	81	2106	7706	798	
28	81	2268	7868	807	
Silty Clay 30	81	2430	8030	815	
30	115	2430	8030	2002	
35	115	3005	8605	2037	
40	115	3580	9180	2070	
45	115	4155	9755	2102	→ 2102 @ 45
50	115	4730	10330	2132	
55	115	5305	10905	2161	
60	115	5879	11479	2189	
Sand 60	120	5880	11480	2189	
65	120	6480	12080	2217	
70	120	7080	12680	2241	→ 2244 @ 70
75	120	7680	13280	2270	
80	120	8280	13880	2295	
85	120	8880	14480	2320	
90	120	9480	15080	2343	→ 2343 @ 90
95	120	10080	15680	2366	
100	120	10680	16280	2389	
105	120	11280	16880	2410	
110	120	11880	17480	2432	→ 2432 @ 110
115	120	12480	18080	2452	
Silt 120	120	13079	18679	2472	
120	130	13080	18680	2472	
130	130	13756	19356	2494	
140	130	14432	20032	2516	→ 2516 @ 140
150	130	15108	20708	2537	
160	130	15784	21384	2557	
170	130	16460	22060	2577	
180	130	17136	22736	2597	→ 2597 @ 180
190	130	17812	23412	2616	
200	130	18488	24088	2635	
210	130	19164	24764	2653	
220	130	19840	25440	2671	→ 2680 @ 225
230	130	20516	26116	2688	
240	130	21192	26792	2706	
250	130	21868	27468	2722	
260	130	22544	28144	2739	
270	130	23220	28820	2755	→ 2763 @ 275
280	130	23896	29496	2771	
290	130	24572	30172	2787	
Silt 300	130	25248	30848	2803	
310	130	25924	31524	2818	
320	130	26600	32200	2833	
330	130	27276	32876	2848	
340	130	27952	33552	2862	
350	130	28628	34228	2876	→ 2876 @ 350
360	130	29304	34904	2891	
370	130	29980	35580	2904	
380	130	30656	36256	2918	
390	130	31332	36932	2932	
400	130	32008	37608	2945	
410	130	32684	38284	2958	
420	130	33360	38960	2971	
430	130	34036	39636	2984	
440	130	34712	40312	2997	
450	130	35388	40988	3009	→ 3009 @ 450
460	130	36064	41664	3021	
470	130	36740	42340	3034	
480	130	37416	43016	3046	
490	130	38092	43692	3057	
Silt 500	130	38768	44368	3069	
510	130	39444	45044	3081	
520	130	40120	45720	3092	
530	130	40796	46396	3104	
540	130	41472	47072	3115	
550	130	42148	47748	3126	→ 3126 @ 550
560	130	42824	48424	3137	
570	130	43500	49100	3148	
580	130	44176	49776	3159	
590	130	44852	50452	3169	
Silt 600	130	45527	51127	3180	
600	150	45529	51129	3180	→ 5000

Shear-wave Velocity				
Depth	Density	Sigv'	Pv'	Vs
(ft)	(pcf)	(psf)	(psf)	(fps)
0	81	0	5600	681
2	81	162	5762	690
4	81	324	5924	700
6	81	486	6086	710
8	81	648	6248	719
10	81	810	6410	728
12	81	972	6572	737
14	81	1134	6734	746
16	81	1296	6896	755
18	81	1458	7058	764
20	81	1620	7220	773
22	81	1782	7382	781
24	81	1944	7544	790
26	81	2106	7706	798
28	81	2268	7868	807
28	81	2430	8030	815
30	115	2430	8030	2002
35	115	3085	8685	2037
40	115	3580	9180	2070
45	115	4155	9755	2102
50	115	4750	10350	2132
55	115	5365	10965	2161
60	115	5979	11479	2189
60	120	5880	11480	2189
65	120	6480	12080	2217
70	120	7080	12680	2244
75	120	7680	13280	2270
80	120	8280	13880	2295
85	120	8880	14480	2320
90	120	9480	15080	2343
95	120	10080	15680	2366
100	120	10680	16280	2389
105	120	11280	16880	2410
110	120	11880	17480	2432
115	120	12480	18080	2452
120	120	13079	18679	2472
120	130	13080	18680	2472
130	130	13756	19356	2494
140	130	14432	20032	2516
150	130	15108	20708	2537
160	130	15784	21384	2557
170	130	16460	22060	2577
180	130	17136	22736	2597
190	130	17812	23412	2616
200	130	18488	24088	2635
210	130	19164	24764	2653
220	130	19840	25440	2671
230	130	20516	26116	2688
240	130	21192	26792	2706
250	130	21868	27468	2722
260	130	22544	28144	2739
270	130	23220	28820	2755
280	130	23896	29496	2771
290	130	24572	30172	2787
300	130	25248	30848	2803
310	130	25924	31524	2818
320	130	26600	32200	2833
330	130	27276	32876	2848
340	130	27952	33552	2862
350	130	28628	34228	2876
360	130	29304	34904	2891
370	130	29980	35580	2904
380	130	30656	36256	2918
390	130	31332	36932	2932
400	130	32008	37608	2945
410	130	32684	38284	2958
420	130	33360	38960	2971
430	130	34036	39636	2984
440	130	34712	40312	2997
450	130	35388	40988	3009
460	130	36064	41664	3021
470	130	36740	42340	3034
480	130	37416	43016	3046
490	130	38092	43692	3057
500	130	38768	44368	3069
510	130	39444	45044	3081
520	130	40120	45720	3092
530	130	40796	46396	3104
540	130	41472	47072	3115
550	130	42148	47748	3126
560	130	42824	48424	3137
570	130	43500	49100	3148
580	130	44176	49776	3159
590	130	44852	50452	3169
600	130	45527	51127	3180
600	150	45529	51129	3180

The unit weight of dry soil ($S_r = 0\%$) is

$$\gamma_d = (1 - n)\gamma_s \tag{6.5}$$

and of saturated soil ($S_r = 100\%$) is

$$\gamma = (1 - n)\gamma_s + n\gamma_w = \gamma_s - n(\gamma_s - \gamma_w) \tag{6.6}$$

The unit weight of the principal solid constituents of soils is given in Table 6.2. For sand grains the average unit weight is usually about 2.65 gm/cm³. For clay particles the unit weight varies from 2.5 to 2.9 with a statistical average of approximately 2.7.

Given in Table 6.3 are the porosity and the saturated unit weight of typical soils. For sandy soils the weight of dry soil has also been included. The weights have been computed on the assumption that

Table 6.3
Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State

Description	Porosity, n (%)	Void ratio, e	Water content, w (%)	Unit weight grams/cm³ γ_d	Unit weight grams/cm³ γ	Unit weight lb/ft³ γ_d	Unit weight lb/ft³ γ
1. Uniform sand, loose	46	0.85	32	1.43	1.89	90	118
2. Uniform sand, dense	34	0.51	19	1.75	2.09	109	130
3. Mixed-grained sand, loose	40	0.67	25	1.59	1.99	99	124
4. Mixed-grained sand, dense	30	0.43	16	1.86	2.16	116	135
5. Glacial till, very mixed-grained	20	0.25	9	2.12	2.32	132	145
6. Soft glacial clay	55	1.2	45	-	1.77	-	110
7. Stiff glacial clay	37	0.6	22	-	2.07	-	129
8. Soft slightly organic clay	66	1.9	70	-	1.58	-	98
9. Soft very organic clay	75	3.0	110	-	1.43	-	89
10. Soft bentonite	84	5.2	194	-	1.27	-	80

w = water content when saturated, in per cent of dry weight.
γ_d = unit weight in dry state.
γ = unit weight in saturated state.

the value of γ_s is 2.65 gm/cm³ for sandy soils and 2.70 gm/cm³ for clays. The tabulated values should be considered only as approximations. Before final computations are made on a given job, the actual unit weight of the soil should always be determined.

Problems

1. A sample of saturated clay weighed 1526 gm in its natural state, and 1053 gm after drying. Determine the natural water content. If the unit weight of the solid constituents was 2.70 gm/cm³, what was the void ratio? the porosity? the weight per cubic foot?

Ans. $w = 45.0\%$; $e = 1.22$; $n = 0.55$; $\gamma = 111$ lb/ft³.

2. A sample of hardpan had a weight of 129.1 gm and a volume of 56.4 cm³ in its natural state. Its dry weight was 121.5 gm. The unit weight of the solid constituents was found to be 2.70 gm/cm³. Compute the water content, the void ratio, and degree of saturation.

Ans. $w = 6.3\%$; $e = 0.25$; $S_r = 0.67$.

3. The unit weight of a sand backfill was determined by field measurements to be 109 lb/ft³. The water content at the time of the test was 8.6%, and the unit weight of the solid constituents was 2.60 gm/cm³. In the laboratory the void ratios in the loosest and densest states were found to be 0.642 and 0.462, respectively. What were the void ratio and the relative density of the fill?

Ans. $e = 0.616$; $D_r = 0.14$.

4. A dry quartz sand weighs 96 lb/ft³. What is its unit weight when saturated?

Ans. $\gamma = 122$ lb/ft³.

5. A sample of silty clay was found, by immersion in mercury, to have a volume of 14.88 cu cm. Its weight at the natural water content was 28.81 gm and after oven drying was 24.83 gm. The unit weight of solid constituents was 2.70 gm/cm³. Calculate the void ratio and the degree of saturation of the sample.

Ans. $e = 0.617$; $S_r = 0.701$.

6. Given the values of porosity n for the soils in Table 6.3, check the values of water content w and unit weight γ (lb/ft³). For soils 1–5, $\gamma_s = 2.65$ gm/cm³; for soils 6–10, $\gamma_s = 2.70$ gm/cm³.

ART. 7 CONSISTENCY AND SENSITIVITY OF CLAYS

Consistency and Sensitivity of Undisturbed Soils

The consistency of clays and other cohesive soils is usually described as *soft*, *medium*, *stiff*, or *hard*. The most direct quantitative measure

From DM7 (1982)

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146(3)	90	155(3)	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148(4)	125	156(4)	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

From Bowles (1979)

Table 6-1 Typical values of e_{max}, e_{min}, and unit weight for several soils

	Loose			Dense		
	Dry unit weight	e_{max}†	ϕ‡	Dry unit weight	e_{min}†	ϕ‡
Gravel	16.0–18.0	0.62–0.44	32–36	18.0–20.0	0.44–0.30	35–50
Coarse sand	15.0–17.5	0.73–0.50	32–38	17.5–19.6	0.50–0.33	35–48
Clayey sand	14.0–16.5	0.86–0.58	28–32	16.5–18.5	0.58–0.40	35–40
Silty sand	12.6–15.5	1.05–0.68	28–32	15.5–17.5	0.68–0.49	32–38
Fine sand	14.0–18.5	0.86–0.40	27–33	15.5–18.0	0.68–0.44	33–39
Sandy gravel	15.0–18.0	0.73–0.44	30–38	18.0–22.0	0.44–0.18	36–45
Gravelly sand	15.0–18.0	0.73–0.44	30–38	18.0–22.5	0.44–0.16	36–50
Silt	14.0–15.5	0.86–0.68	20–30	15.5–17.5	0.68–0.49	25–32

† Depends on G_s.
‡ Use higher values for angular particles

mold, the void ratio is easily computed. The densest state is obtained by vibrating a confined weight of sand and measuring the volume. Table 6-1 gives some ranges of void ratios and other data for several soils as an indication of the values one might expect to obtain. Tabulated values such as these are acceptable for preliminary design but should never be used for any final design.

The *relative density* is a measure of the in situ void ratio e_n related to the laboratory values of the maximum and minimum void ratios as

$$D_r = \frac{e_{max} - e_n}{e_{max} - e_{min}} \tag{6-1}$$

Relative density can also be expressed in terms of the maximum (γ_{max}), minimum (γ_{min}), and in situ (γ_n) dry unit weights as

$$D_r = \frac{\gamma_{max}}{\gamma_n}\frac{\gamma_n - \gamma_{min}}{\gamma_{max} - \gamma_{min}} \tag{6-2}$$

This equation is preferable to Eq. (6-1) due to the greater ease of determining unit weights and because it does not require a determination of the specific gravity. Considerable importance was attributed to the relative density by early proponents, who attempted to relate various soil properties such as void ratio, angle of internal friction, and, thus, indirectly settlement and strength characteristics to this index property. Relative density is sometimes used at present in liquefaction studies (Sec. 14-7) as a field compaction specification requirement, and to assess the competence of in situ granular materials for foundations.

The major reason for using relative density is that undisturbed sampling of in situ cohesionless sands and gravels is nearly impossible and, as a consequence, penetrometer testing is widely used. A large data base presently exists—albeit with considerable scatter—relating penetration tests to relative density. Table 6-2 gives some simple field identification tests which may be used to estimate D_r.

Table 6-2 Terms and field identification in relative density

Soil state†	D_r	Field identification
Very loose	0–0.20	Easily indented with finger, thumb, or fist
Loose	0.20–0.40	Somewhat less easily indented with fist. Easily shoveled
Medium compact	0.40–0.70	Shoveled with difficulty
Compact	0.70–0.90	Requires pick to loosen for shoveling by hand
Very compact	0.90–1.00	Requires blasting or heavy equipment to loosen

† Not all authorities agree on either the terminology describing the soil state or the value of D_r to which it applies. These values are as good as any proposed and with the subjective nature of the index property may be used with confidence.

Considerable research, with the latest reported in ASTM (1973), indicates that D_r is not a very reliable soil index property. It is quite possible for two sands with identical values of in situ void ratios e_n and D_r to have significantly different engineering behavior due to grain shape, cementation, confinement, and stratification resulting from deposition and stress history.

Since D_r depends on laboratory determination of γ_{max} and γ_{min}, or the corresponding void ratios, a large error may result in not accurately determining both of these values. Generally, a statistical determination of γ_{max} will produce a rather consistent (average or standard deviation) value about 0.45 kN/m³ too small, and conversely for the minimum unit weight a value about 0.45 kN/m³ too large. This is illustrated in Ex. 6-1.

Example 6-1 A medium coarse, gravelly sand was tested in the author's laboratory by a group of 10 students. Each student did three tests each for the maximum and minimum unit weights and reported the extreme (not the average) values obtained. The sand was returned to the source container and well mixed for further use. The data were as follows (the Δ-values were later computed for obtaining the standard deviation σ):

Test	1	2	3	4	5	6	7	8	9	10
γ_{min}	14.36	14.58	15.02	15.12	14.78	14.51	14.40	15.08	15.15	14.43
Δ	0.0	0.22	0.66	0.76	0.42	0.15	0.04	0.72	0.79	0.07
γ_{max}	18.57	18.70	18.86	18.39	18.50	18.32	18.45	18.42	18.49	18.75
Δ	0.29	0.16	0.0	0.47	0.36	0.54	0.41	0.44	0.37	0.11

REQUIRED Compute the standard deviation and assess the error in D_r if the in situ value of $\gamma_n = 16.0$ kN/m³.

SOLUTION The standard deviation is computed based on the maximum and minimum values of unit weight, not on the average, since the definition of D_r is based on the extreme values. With this concept, the Δ values are obtained as

FAX MESSAGE

To: Stan Macie
John Donnell
Stone & Webster
Englewood, CO

From: Robert Youngs
Geomatrix Consultants
San Francisco, Calif.

Fax No: 303-741-7806

Date: February 15, 1997

Subject: Skull Valley ISFSI

==========

Dear Stan:

The following are the revised response spectra for the Stansbury Fault

Stansbury 84th-percentile Horizontal

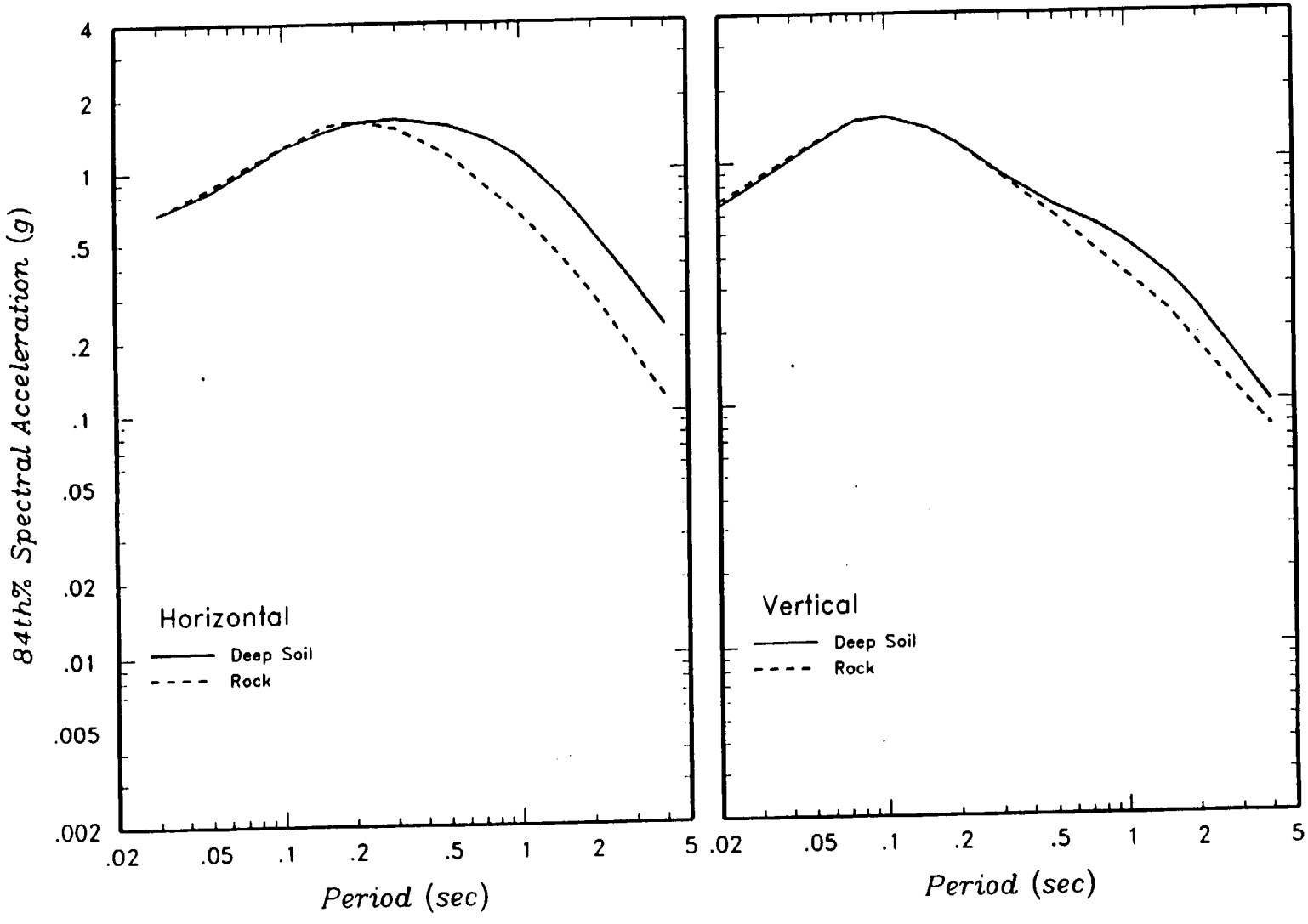
T	Deep Soil	Rock	Envelope
0.03	0.669	0.670	0.670
0.05	0.831	0.868	0.868
0.075	1.049	1.081	1.081
0.1	1.256	1.271	1.271
0.15	1.471	1.549	1.549
0.2	1.598	1.631	1.631
0.3	1.653	1.510	1.653
0.5	1.542	1.161	1.542
0.75	1.336	0.828	1.336
1	1.125	0.649	1.125
1.5	0.772	0.424	0.772
2	0.539	0.301	0.539
3	0.328	0.172	0.328
4	0.224	0.114	0.224

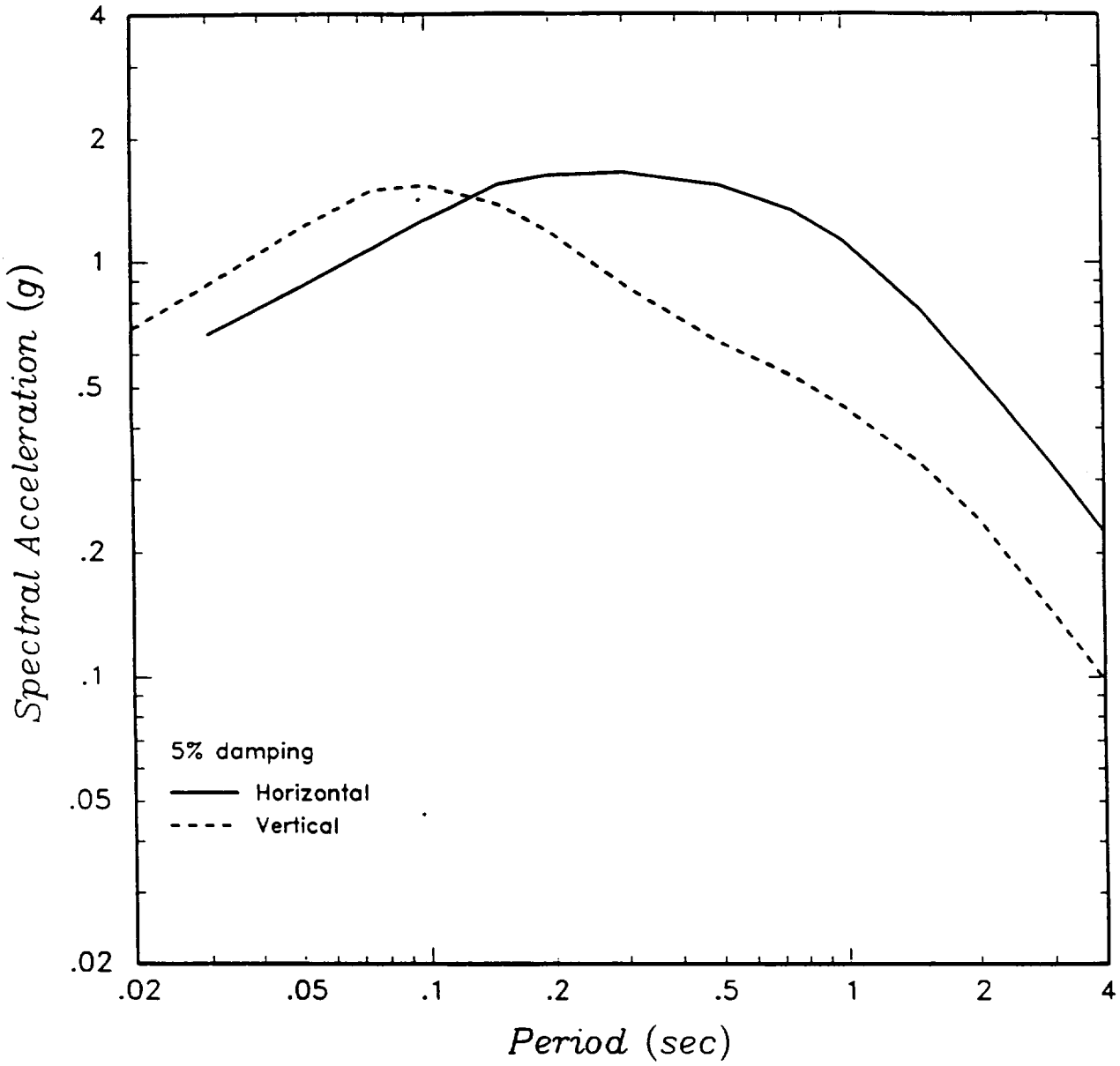
Stansbury 84th-percentile Vertical

T	Deep Soil	Rock	Envelope
0.02	0.658	0.689	0.689
0.05	1.175	1.202	1.202
0.075	1.483	1.497	1.497
0.1	1.537	1.541	1.541
0.15	1.379	1.366	1.379
0.2	1.179	1.167	1.179
0.3	0.879	0.864	0.879
0.5	0.641	0.581	0.641
0.75	0.532	0.414	0.532
1	0.448	0.330	0.448
1.5	0.328	0.236	0.328
2	0.244	0.170	0.244
3	0.144	0.107	0.144
4	0.098	0.077	0.098

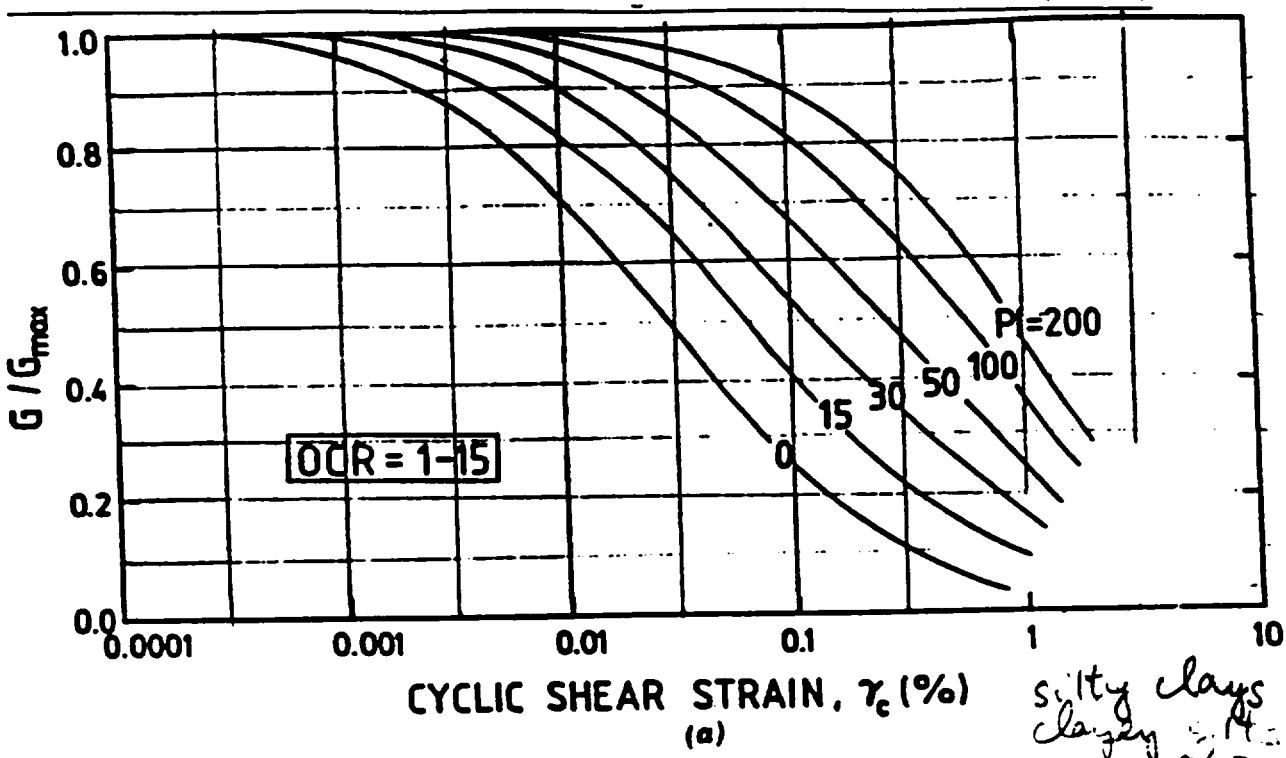
2 Plots are attached. I will be sending the draft geophysical review shortly.

Bob Youngs


84th% Spectral Acceleration (g)
4
2
1
.5
.2
.1
.05
.02
.01
.005
.002
Horizontal
Deep Soil
Rock
.02
.05
.1
.2
.5
1
2
5
Period (sec)
Vertical
Deep Soil
Rock
.02
.05
.1
.2
.5
1
2
5
Period (sec)

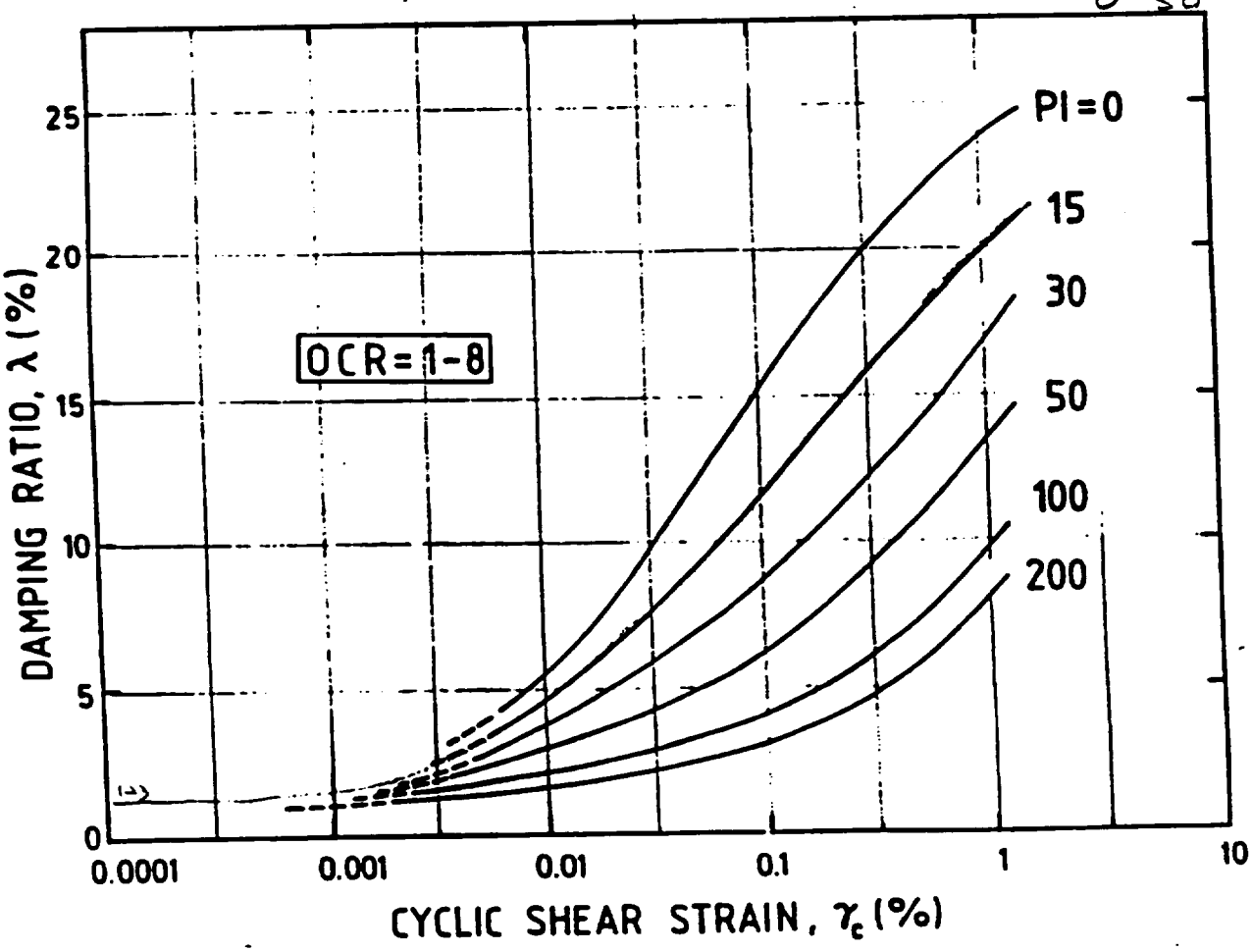


Recommended envelope 84th percentile spectra



(a)

silty clays
clayey silt
0-30



From Vucetic and Dobry (1991)

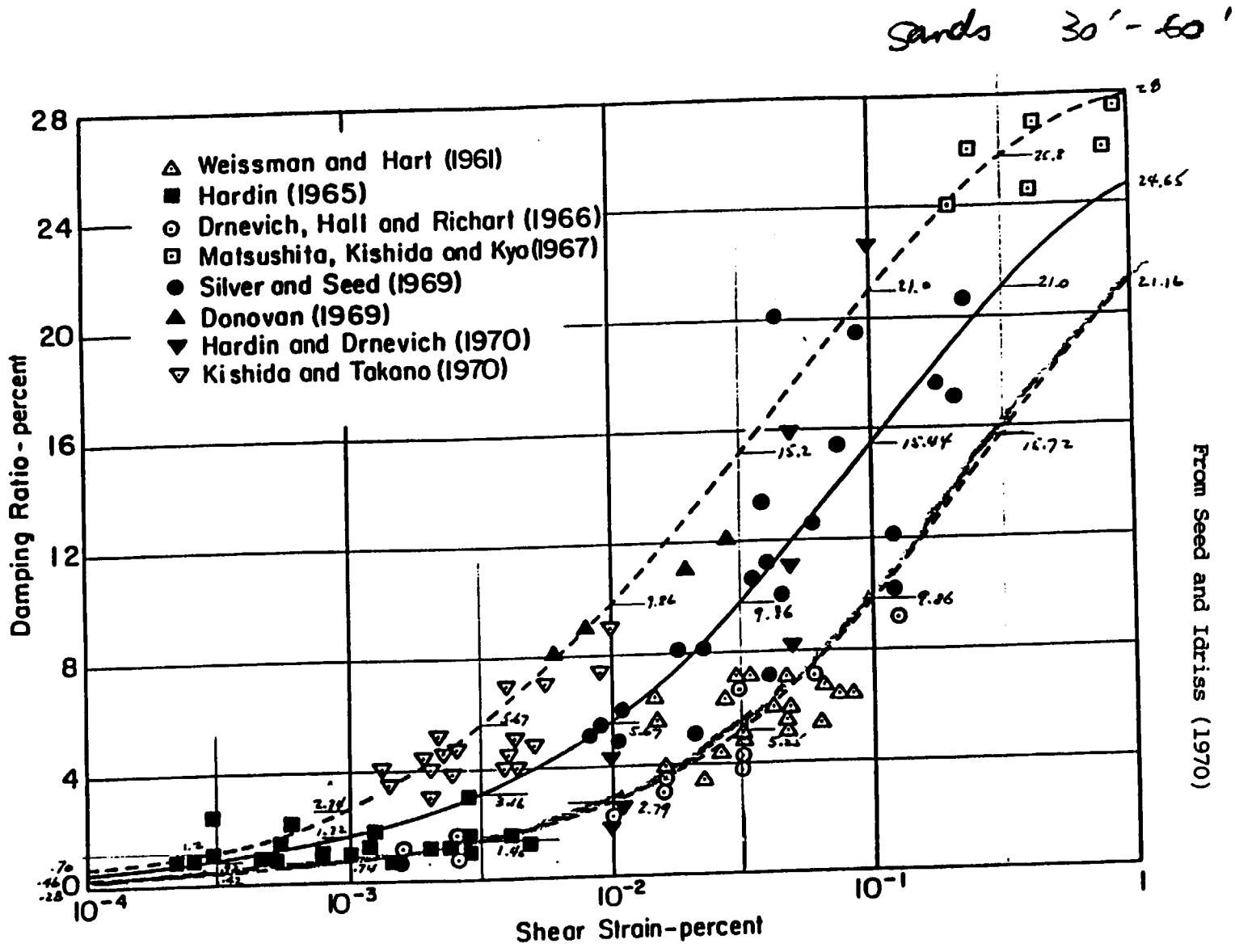


FIG. 9 DAMPING RATIOS FOR SANDS

Sand 30-60

From Seed & Idriss (1970)

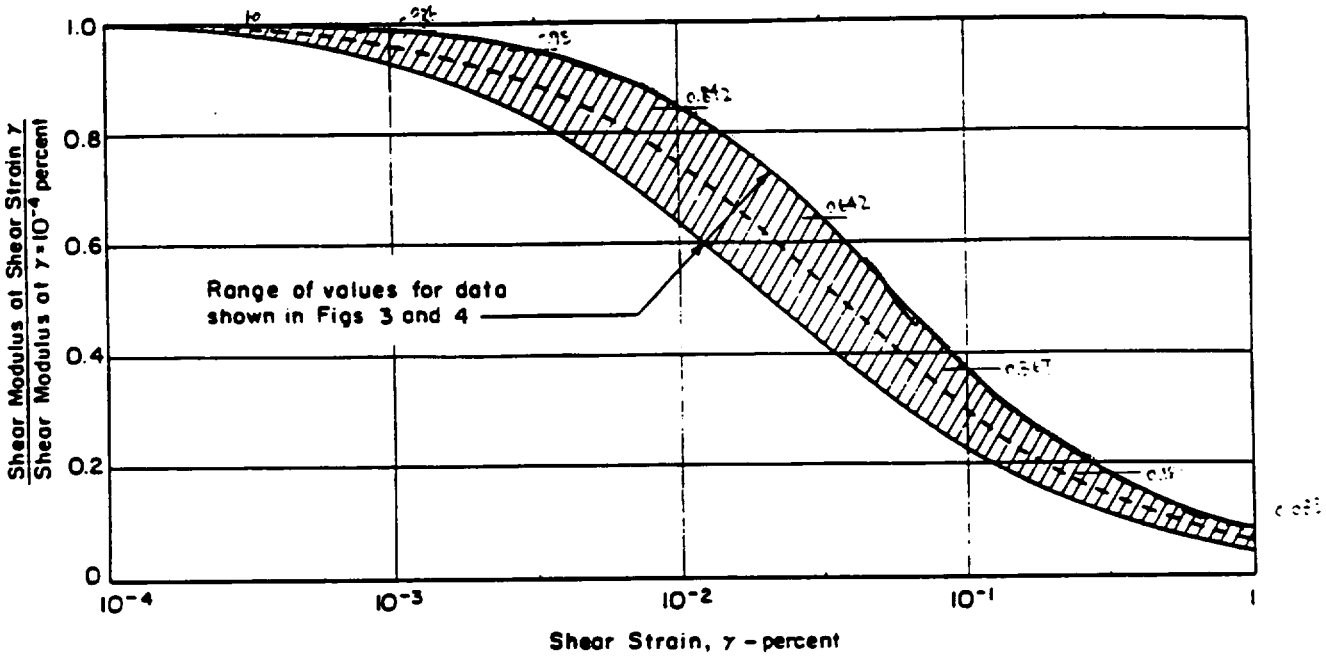


FIG. 5 VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SANDS
(after Seed and Idriss, 1970)

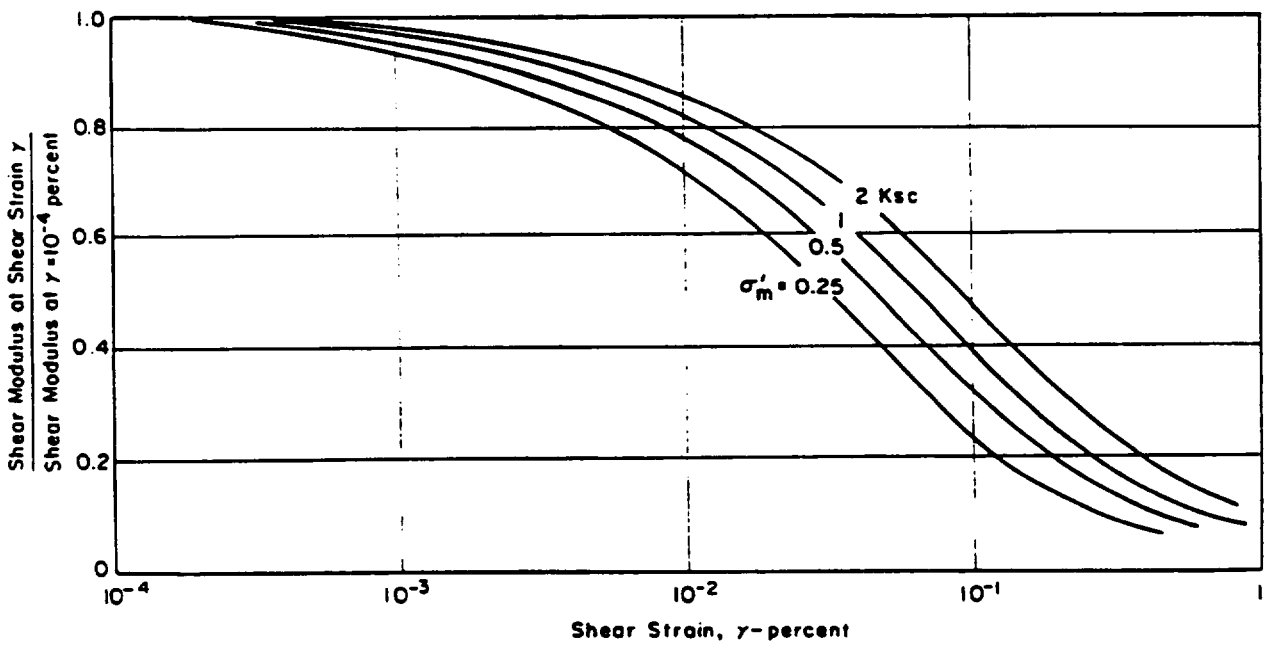
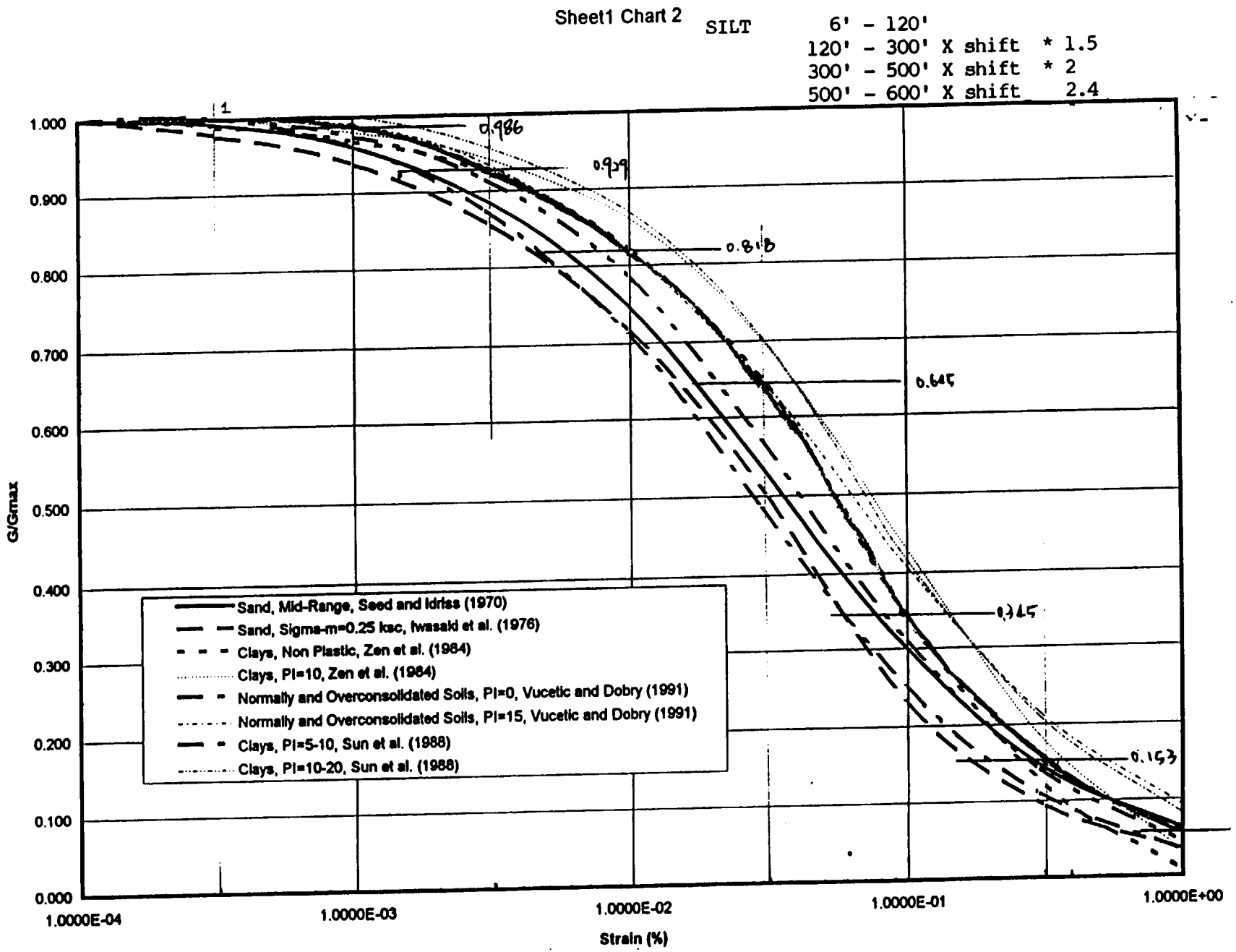
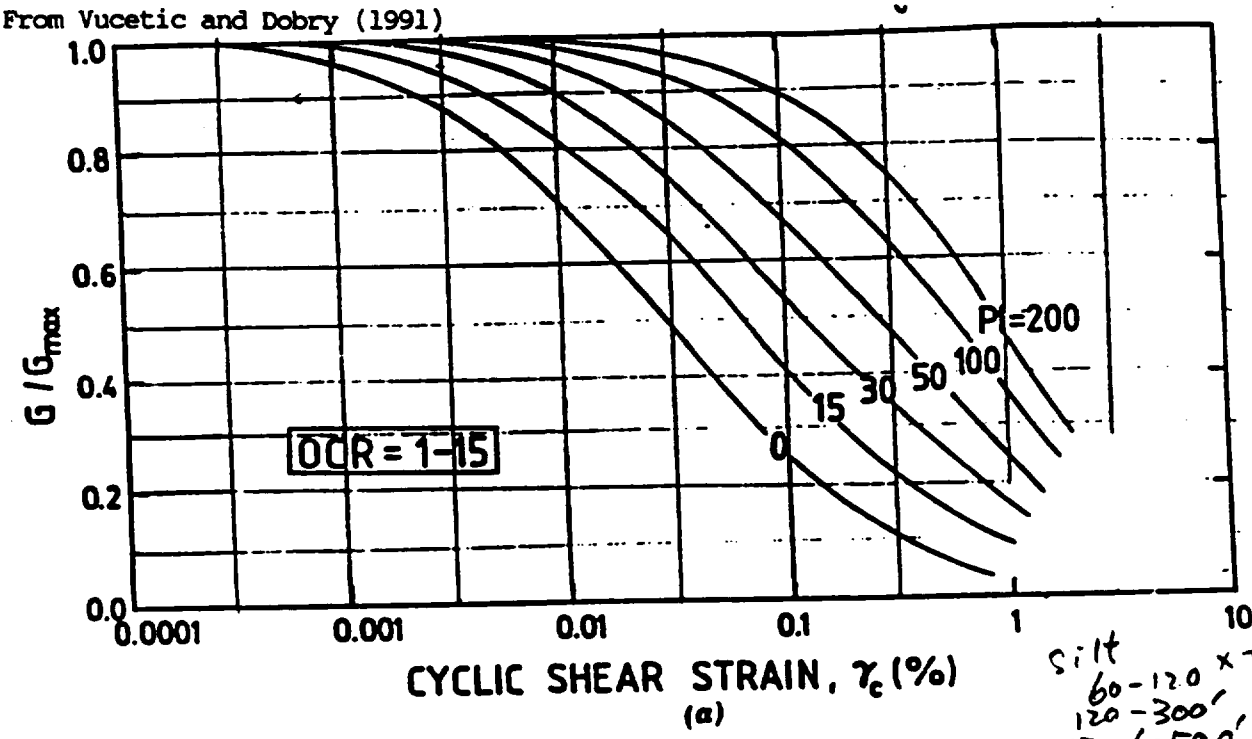


FIG. 6 VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SANDS
(after Iwasaki et al., 1976)

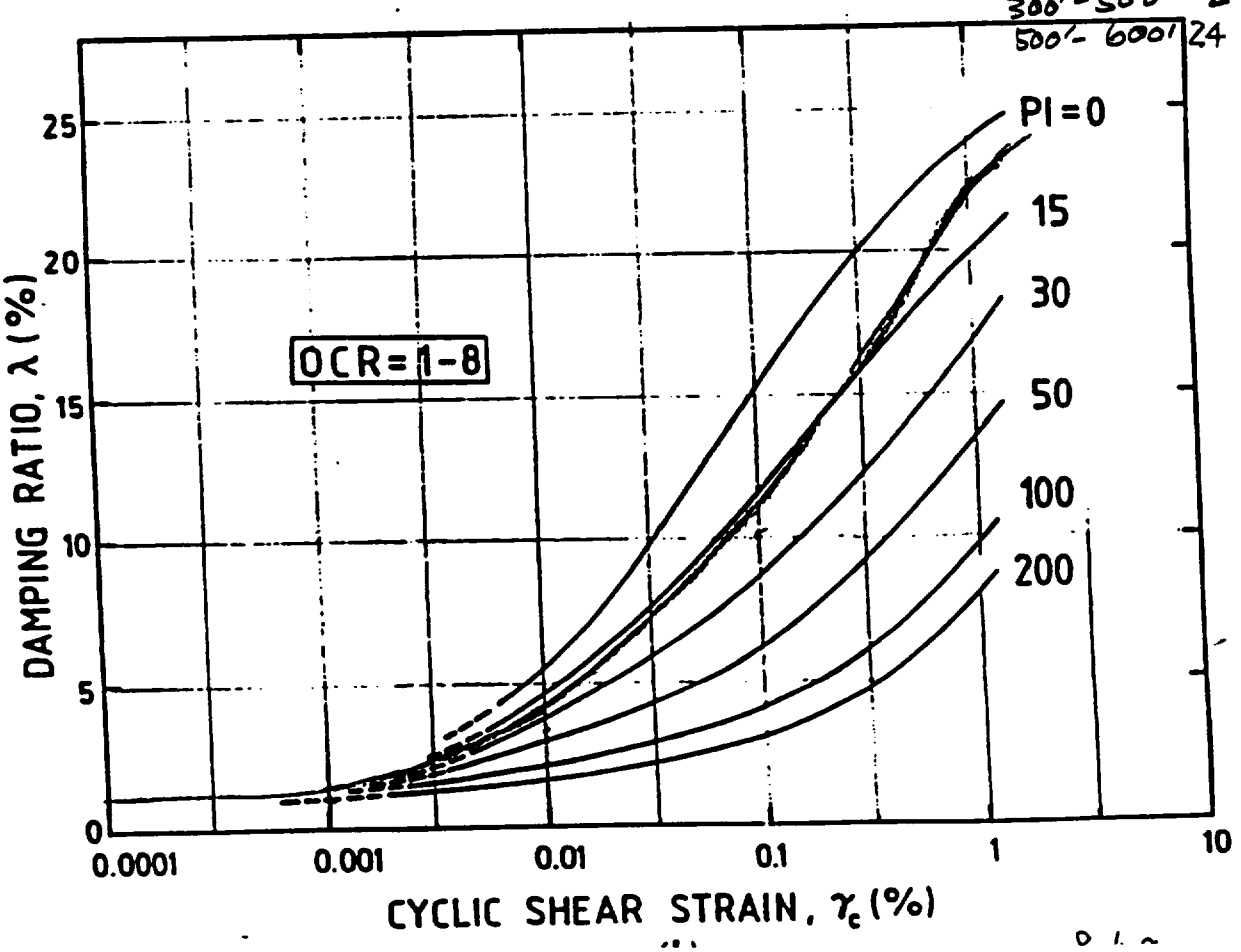

Sheet1 Chart 2
SILT
6' - 120'
120' - 300' X shift * 1.5
300' - 500' X shift * 2
500' - 600' X shift 2.4
G/Gmax
Strain (%)
1.000
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
1.0000E-04
1.0000E-03
1.0000E-02
1.0000E-01
1.0000E+00
1
0.986
0.939
0.818
0.645
0.345
0.153
0.058
Sand, Mid-Range, Seed and Idriss (1970)
Sand, Sigma-m=0.25 ksc, Iwasaki et al. (1976)
Clays, Non Plastic, Zen et al. (1984)
Clays, PI=10, Zen et al. (1984)
Normally and Overconsolidated Soils, PI=0, Vucetic and Dobry (1991)
Normally and Overconsolidated Soils, PI=15, Vucetic and Dobry (1991)
Clays, PI=5-10, Sun et al. (1988)
Clays, PI=10-20, Sun et al. (1988)

From Vucetic and Dobry (1991)



(a)

silt
60-120 x silt
120-300' 1.5
300'-500' 2
500'-600' 24



From Vucetic and Dobry (1991)

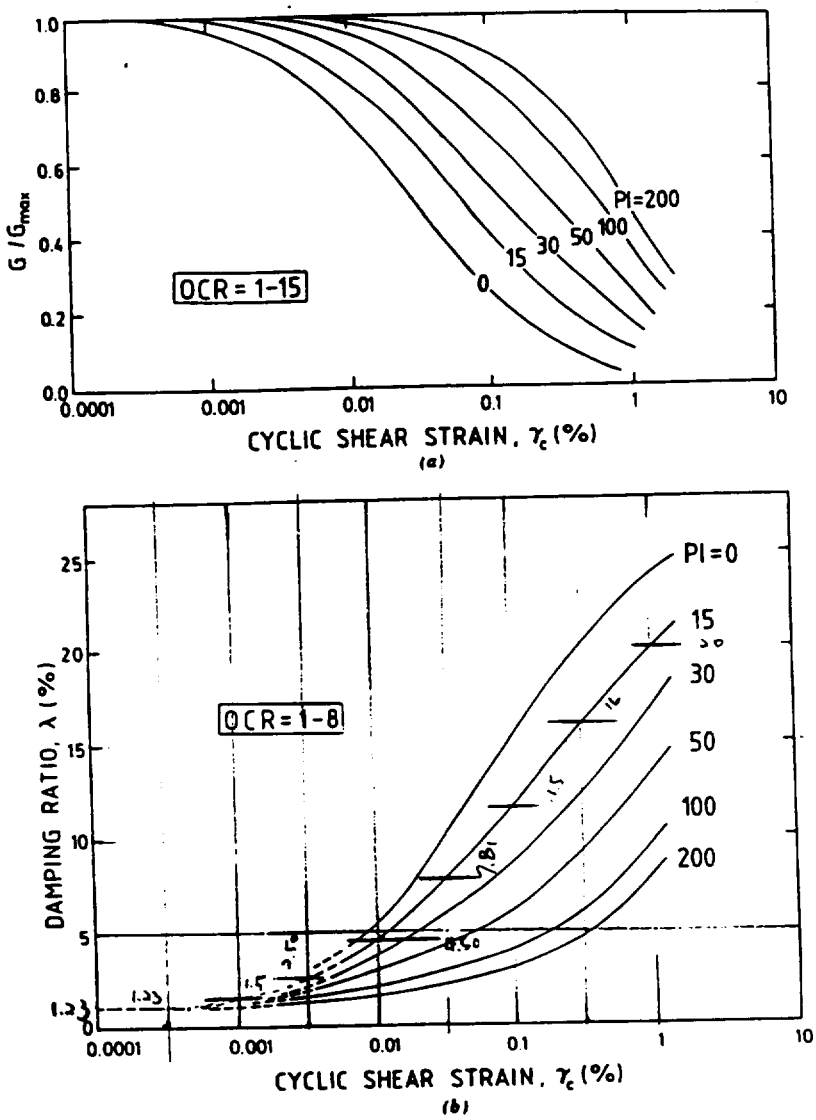
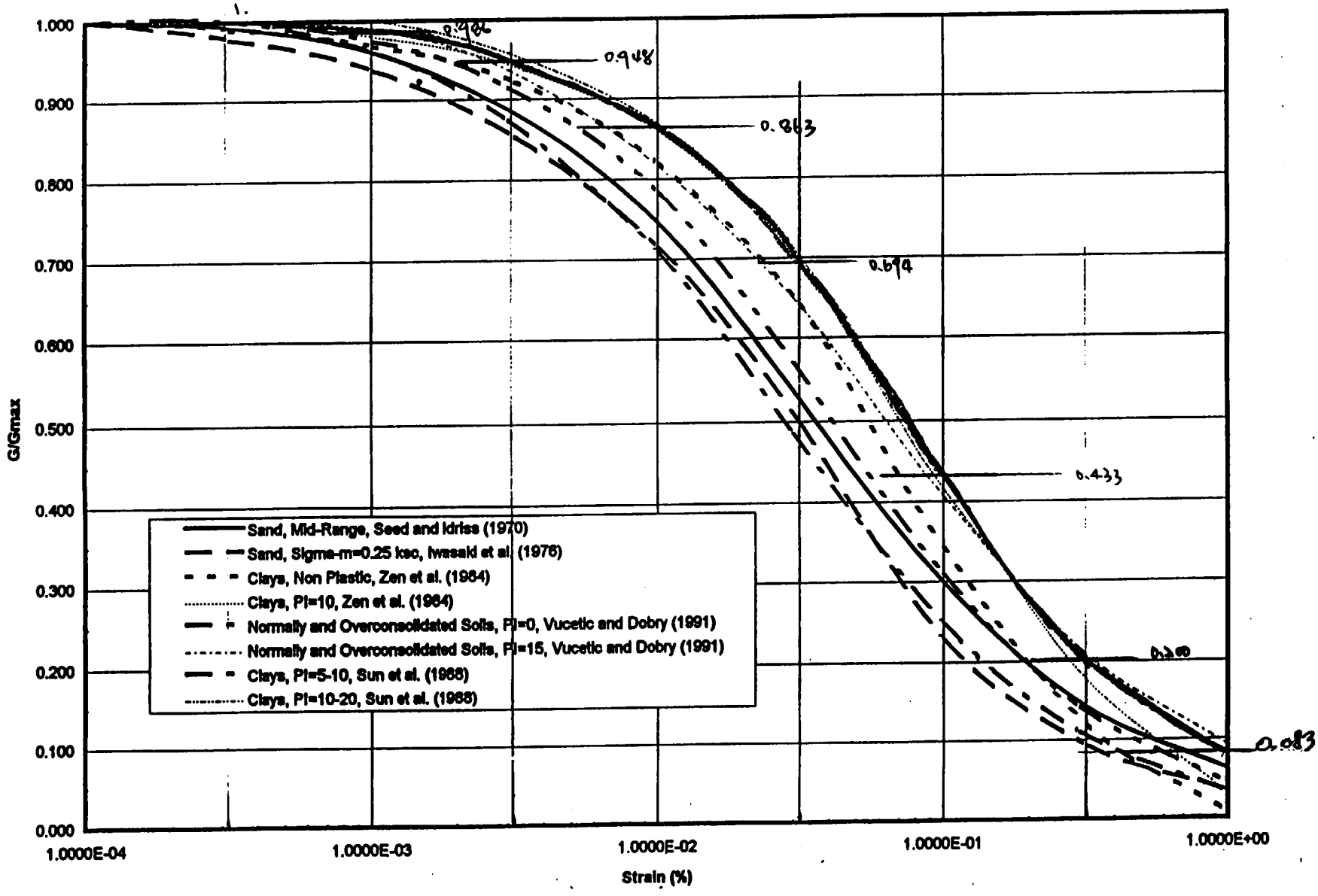


FIG. 6. Relations between G/G_{max} versus γ_c and λ versus γ_c Curves and Soil Plasticity for Normally and Overconsolidated Soils

Sheet1 Chart 2

silty clays, clayey silt 0-30 ft

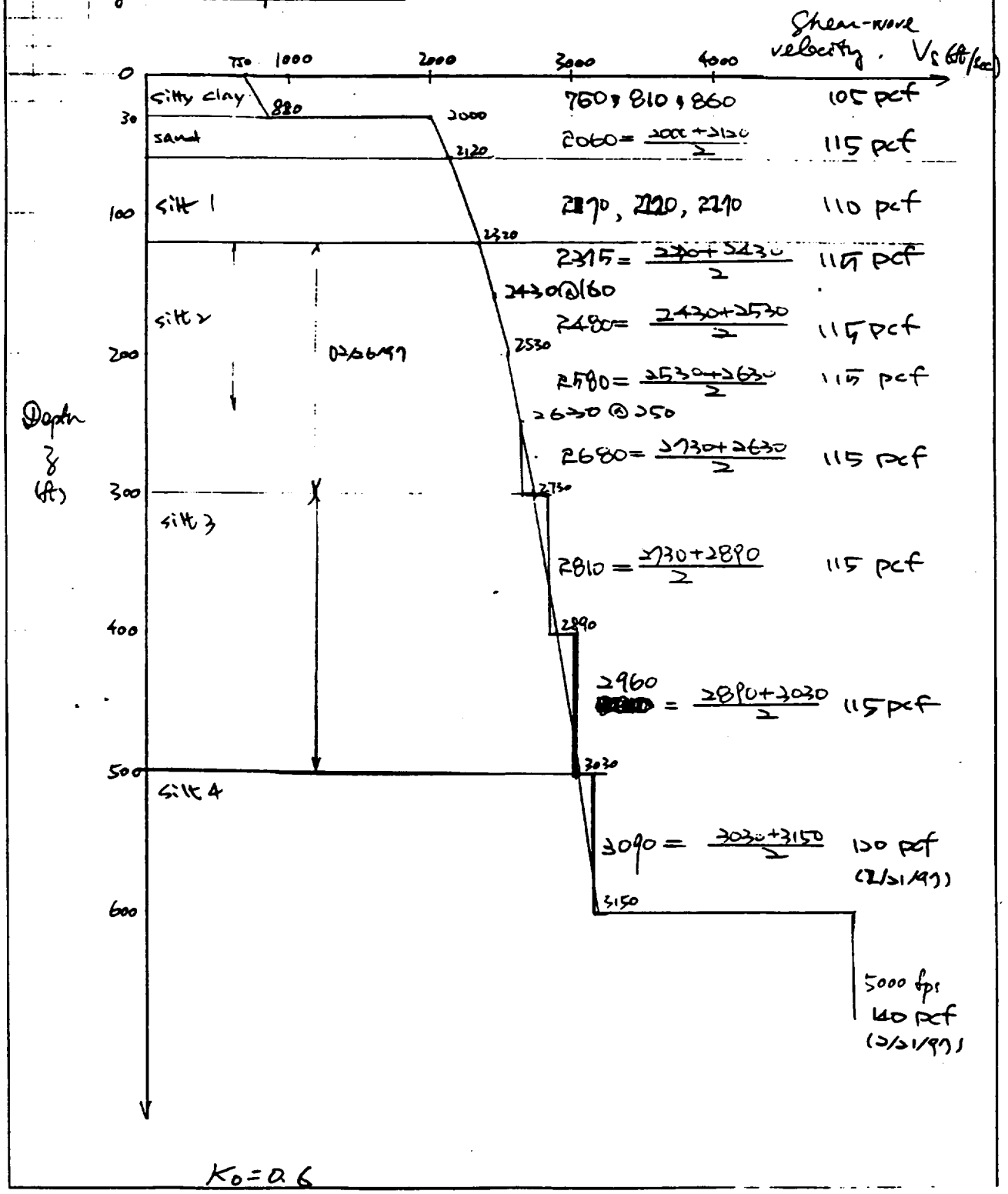


I:\PA\380\GEO-RPT1.DOC

By C.C. CHIN Checked By Chin Man Mak

Date 3/12/97 Date 3/12/97

Idealized Vs-profile



INPUT FILE FOR "SHAKE" PROGRAM [SCHNABEL, 1972]

SKLH101.INP

```
32768      0.6
   8
   6    1   10  100.    Design Curves for Skull Valley
   9 100. Shear Modulus for Silty Clay   0- 30 ft
 0.000100  0.000316  0.001000  0.003160  0.010000  0.031600  0.100000  0.316000
 1.000000
    1.000     1.000     0.986     0.948     0.863     0.694     0.433     0.200
    0.083
   9   1. Damping for Silty Clay         0- 30 ft
 0.000100  0.000316  0.001000  0.003160  0.010000  0.031600  0.100000  0.316000
 1.000000
     1.23      1.23      1.50      2.50      4.50      7.81      11.5      16.0
     20.0
   9 100. Shear Modulus for Sand        30- 60 ft
 0.000100  0.000316  0.001000  0.003160  0.010000  0.031600  0.100000  0.316000
 1.000000
    1.000     1.000     0.986     0.950     0.842     0.642     0.367     0.183
    0.083
   9   1. Damping for Sand              30- 60 ft
 0.000100  0.000316  0.001000  0.003160  0.010000  0.031600  0.100000  0.316000
 1.000000
     0.28      0.42      0.74      1.40      2.79      5.25      9.86      15.72
    21.16
   9 100. Shear Modulus for Silt        60-120 ft
 0.000100  0.000316  0.001000  0.003160  0.010000  0.031600  0.100000  0.316000
 1.000000
    1.000     1.000     0.986     0.929     0.818     0.645     0.345     0.153
    0.058
   9   1. Damping for Silt              60-120 ft
 0.000100  0.000316  0.001000  0.003160  0.010000  0.031600  0.100000  0.316000
 1.000000
     1.23      1.23      1.50      2.50      4.50      7.81      11.5      16.0
     20.0
   9 100. Shear Modulus for Silt       120-300 ft
 0.000100  0.000474  0.001500  0.004740  0.015000  0.047400  0.150000  0.474000
 1.500000
    1.000     1.000     0.986     0.929     0.818     0.645     0.345     0.153
    0.058
   9   1. Damping for Silt             120-300 ft
 0.000100  0.000474  0.001500  0.004740  0.015000  0.047400  0.150000  0.474000
 1.500000
     0.95      1.23      1.50      2.50      4.50      7.81      11.5      16.0
     20.0
   9 100. Shear Modulus for Silt       300-500 ft
 0.000100  0.000632  0.002000  0.006320  0.020000  0.063200  0.200000  0.632000
 2.000000
    1.000     1.000     0.986     0.929     0.818     0.645     0.345     0.153
    0.058
   9   1. Damping for Silt             300-500 ft
 0.000100  0.000632  0.002000  0.006320  0.020000  0.063200  0.200000  0.632000
 2.000000
     0.79      1.23      1.50      2.50      4.50      7.81      11.5      16.0
     20.0
   9 100. Shear Modulus for Silt       500-600 ft
 0.000100  0.000759  0.002400  0.007589  0.024000  0.075895  0.240000  0.758947
 2.400000
    1.000     1.000     0.986     0.929     0.818     0.645     0.345     0.153
    0.058
   9   1. Damping for Silt             500-600 ft
 0.000100  0.000759  0.002400  0.007589  0.024000  0.075895  0.240000  0.758947
 2.400000
     0.72      1.23      1.50      2.50      4.50      7.81      11.5      16.0
     20.0
   2
   1   15   25                    Skull Valley Horizontal MA= 0.7 g
   1    1    4        10.              0.05      0.081      705.          1.0
   2    1    4        10.              0.05      0.081      751.          1.0
   3    1    4        10.              0.05      0.081      794.          1.0
   4    2    6        30.              0.05      0.115     2102.          1.0
   5    3    2        20.              0.05      0.120     2244.          1.0
   6    3    2        20.              0.05      0.120     2343.          1.0
```

Page 1

```
                                                SKLH101.INP
   7    3    2        20.                0.05       0.120      2432.          1.0
   8    4    2        40.                0.01       0.130      2516.          1.0
   9    4    2        40.                0.01       0.130      2597.          1.0
  10    4    1        50.                0.01       0.130      2680.          1.0
  11    4    1        50.                0.01       0.130      2763.          1.0
  12    5    2       100.                0.01       0.130      2876.          1.0
  13    5    2       100.                0.01       0.130      3009.          1.0
  14    6    1       100.                0.01       0.130      3126.          1.0
  15                                     0.01       0.150      5000.          1.0
   1
1638432768     0.020      0(4e17.7)
      1.0                    25.0     1
olmh101.acc
   3
  36    0
   4
      20      2.0       0.65
   5
   1    3    5    7    9   11   13   15   17   19
   0    1    1    1    1    1    1    1    1    1
   1    0    0    0    0    0    0    0    0    0
   5
  21   23   25   27   29   31   33   35   36   36
   1    1    1    1    1    1    1    1    1    0
   0    0    0    0    0    0    0    0    0    1
   0
```

SKLH101.OUT

```
*********************************************
*********************************************
**                                         **
** SHAKE -- A COMPUTER PROGRAM FOR         **
**          EARTHQUAKE RESPONSE ANALYSIS   **
**          OF HORIZONTALLY LAYERED SITES  **
**                                         **
**                                         **
** MS-DOS VERSION - CONVERTED TO IBM-PC BY **
**                  Shyh-Shiun Lai, WCC    **
**                  January 1985           **
**                                         **
** (Modified to Use 16384 Points and 100   **
**  Soil Layers, S.J. Chiou, August 1995)  **
**                                         **
*********************************************
*********************************************
Output file name : sklh101.out
Start time : 1997/02/26 -- 11:50:59.91

 MAX. NUMBER OF TERMS IN FOURIER TRANSFORM =     32768
 NECESSARY LENGTH OF BLANK COMMON X        =    204819
 EARTH PRESSURE AT REST FOR SAND           =     0.600
1******   OPTION  8  ***  READ RELATION BETWEEN SOIL PROPERTIES AND STRAIN

   CURVES FOR RELATION STRAIN VERSUS SHEAR MODULUS AND DAMPING
```

MP FACTR	MATL TYPE	STRAIN (%)	MOD RED CO	STRAIN (%)	DA
	1	0.000100	1.000	0.000100	
1.2					
	1	0.000316	1.000	0.000316	
1.2					
	1	0.001000	0.986	0.001000	
1.5					
	1	0.003160	0.948	0.003160	
2.5					
	1	0.010000	0.863	0.010000	
4.5					
	1	0.031600	0.694	0.031600	
7.8					
	1	0.100000	0.433	0.100000	
11.5					
	1	0.316000	0.200	0.316000	
16.0					
	1	1.000000	0.083	1.000000	
20.0					

MP FACTR	MATL TYPE	STRAIN (%)	MOD RED CO	STRAIN (%)	DA
	2	0.000100	1.000	0.000100	
0.3					
	2	0.000316	1.000	0.000316	
0.4					
	2	0.001000	0.986	0.001000	
0.7					
	2	0.003160	0.950	0.003160	
1.4					
	2	0.010000	0.842	0.010000	

SKLH101.OUT

2.8	2	0.031600	0.642	0.031600
5.3	2	0.100000	0.367	0.100000
9.9	2	0.316000	0.183	0.316000
15.7	2	1.000000	0.083	1.000000
21.2				

MP FACTR	MATL TYPE	STRAIN (%)	MOD RED CO	STRAIN (%)	DA
1.2	3	0.000100	1.000	0.000100	
1.2	3	0.000316	1.000	0.000316	
1.5	3	0.001000	0.986	0.001000	
2.5	3	0.003160	0.929	0.003160	
4.5	3	0.010000	0.818	0.010000	
7.8	3	0.031600	0.645	0.031600	
11.5	3	0.100000	0.345	0.100000	
16.0	3	0.316000	0.153	0.316000	
20.0	3	1.000000	0.058	1.000000	

MP FACTR	MATL TYPE	STRAIN (%)	MOD RED CO	STRAIN (%)	DA
0.9	4	0.000100	1.000	0.000100	
1.2	4	0.000474	1.000	0.000474	
1.5	4	0.001500	0.986	0.001500	
2.5	4	0.004740	0.929	0.004740	
4.5	4	0.015000	0.818	0.015000	
7.8	4	0.047400	0.645	0.047400	
11.5	4	0.150000	0.345	0.150000	
16.0	4	0.474000	0.153	0.474000	
20.0	4	1.500000	0.058	1.500000	

MP FACTR	MATL TYPE	STRAIN (%)	MOD RED CO	STRAIN (%)	DA
0.8	5	0.000100	1.000	0.000100	
1.2	5	0.000632	1.000	0.000632	
1.5	5	0.002000	0.986	0.002000	
2.5	5	0.006320	0.929	0.006320	
4.5	5	0.020000	0.818	0.020000	
	5	0.063200	0.645	0.063200	

SKLH101.OUT

7.8	5	0.200000	0.345	0.200000
11.5	5	0.632000	0.153	0.632000
16.0	5	2.000000	0.058	2.000000
20.0				

MP FACTR	MATL TYPE	STRAIN (%)	MOD RED CO	STRAIN (%)	DA
	6	0.000100	1.000	0.000100	
0.7	6	0.000759	1.000	0.000759	
1.2	6	0.002400	0.986	0.002400	
1.5	6	0.007589	0.929	0.007589	
2.5	6	0.024000	0.818	0.024000	
4.5	6	0.075895	0.645	0.075895	
7.8	6	0.240000	0.345	0.240000	
11.5	6	0.758947	0.153	0.758947	
16.0	6	2.400000	0.058	2.400000	
20.0					

1****** OPTION 2 *** READ SOIL PROFILE

NEW SOIL PROFILE NO. 1 IDENTIFICATION Skull Valley Horizontal MA

wave velocity scaling factor = 0.10000E+01
0NUMBER OF LAYERS 15 DEPTH TO BEDROCK 600.00
NUMBER OF FIRST SUBMERGED LAYER 25 DEPTH TO WATER LEVEL 120.00

LAYER	TYPE	MAX-MOD SHEAR VEL	THICKNESS SVMAX	DEPTH	EFF. PRESS.	MODULUS	DAMPING	U NIT WEIGTH
1	1	1250.280	2.50	1.25	0.101	1250.280	0.0500	0.0810
		705.000	705.000					
2	1	1250.280	2.50	3.75	0.304	1250.280	0.0500	0.0810
		705.000	705.000					
3	1	1250.280	2.50	6.25	0.506	1250.280	0.0500	0.0810
		705.000	705.000					
4	1	1250.280	2.50	8.75	0.709	1250.280	0.0500	0.0810
		705.000	705.000					
5	1	1418.760	2.50	11.25	0.911	1418.760	0.0500	0.0810
		751.000	751.000					
6	1	1418.760	2.50	13.75	1.114	1418.760	0.0500	0.0810
		751.000	751.000					
7	1	1418.760	2.50	16.25	1.316	1418.760	0.0500	0.0810
		751.000	751.000					
8	1	1418.760	2.50	18.75	1.519	1418.760	0.0500	0.0810
		751.000	751.000					
9	1	1585.879	2.50	21.25	1.721	1585.879	0.0500	0.0810
		794.000	794.000					
10	1	1585.879	2.50	23.75	1.924	1585.879	0.0500	0.0810
		794.000	794.000					
11	1	1585.879	2.50	26.25	2.126	1585.879	0.0500	0.0810
		794.000	794.000					
12	1	1585.879	2.50	28.75	2.329	1585.879	0.0500	0.0810
		794.000	794.000					
13	2	15780.015	5.00	32.50	2.718	15780.015	0.0500	0.1150
		2102.000	2102.000					
14	2	15780.015	5.00	37.50	3.293	15780.015	0.0500	

SKLH101.OUT

```
0.1150          2102.000  2102.000
15      2   15780.015        5.00      42.50        3.868     15780.015      0.0500
0.1150          2102.000  2102.000
16      2   15780.015        5.00      47.50        4.443     15780.015      0.0500
0.1150          2102.000  2102.000
17      2   15780.015        5.00      52.50        5.017     15780.015      0.0500
0.1150          2102.000  2102.000
18      2   15780.015        5.00      57.50        5.592     15780.015      0.0500
0.1150          2102.000  2102.000
19      3   18765.973       10.00      65.00        6.480     18765.973      0.0500
0.1200          2244.000  2244.000
20      3   18765.973       10.00      75.00        7.680     18765.973      0.0500
0.1200          2244.000  2244.000
21      3   20458.318       10.00      85.00        8.880     20458.318      0.0500
0.1200          2343.000  2343.000
22      3   20458.318       10.00      95.00       10.080     20458.318      0.0500
0.1200          2343.000  2343.000
23      3   22042.076       10.00     105.00       11.280     22042.076      0.0500
0.1200          2432.000  2432.000
24      3   22042.076       10.00     115.00       12.480     22042.076      0.0500
0.1200          2432.000  2432.000
25      4   25556.934       20.00     130.00       13.756     25556.934      0.0100
0.1300          2516.000  2516.000
26      4   25556.934       20.00     150.00       15.108     25556.934      0.0100
0.1300          2516.000  2516.000
27      4   27228.979       20.00     170.00       16.460     27228.979      0.0100
0.1300          2597.000  2597.000
28      4   27228.979       20.00     190.00       17.812     27228.979      0.0100
0.1300          2597.000  2597.000
29      4   28997.266       50.00     225.00       20.178     28997.266      0.0100
0.1300          2680.000  2680.000
30      4   30821.178       50.00     275.00       23.558     30821.178      0.0100
0.1300          2763.000  2763.000
31      5   33393.754       50.00     325.00       26.938     33393.754      0.0100
0.1300          2876.000  2876.000
32      5   33393.754       50.00     375.00       30.318     33393.754      0.0100
0.1300          2876.000  2876.000
33      5   36553.742       50.00     425.00       33.698     36553.742      0.0100
0.1300          3009.000  3009.000
34      5   36553.742       50.00     475.00       37.078     36553.742      0.0100
0.1300          3009.000  3009.000
35      6   39451.672      100.00     550.00       42.148     39451.672      0.0100
0.1300          3126.000  3126.000
36    BASE                                                    116460.         0.010
0.1500           5000.

 PERIOD = 0.90 FROM AVERAGE SHEARVEL. =   2673.

MAXIMUM AMPLIFICATION =  61.95
FOR FREQUENCY         =   1.27 C/SEC.
    PERIOD            =   0.79 SEC.

1******   OPTION  1  ***  READ INPUT MOTION

 EARTHQUAKE  -

16384 ACCELERATION VALUES AT TIME INTERVAL    0.0200

    THE VALUES ARE LISTED ROW BY ROW AS READ FROM CARDS
    TRAILING ZEROS ARE ADDED TO GIVE A TOTAL OF32768 VALUES

 MAXIMUM ACCELERATION =  0.70000
 AT TIME              = 60.36 SEC

 THE VALUES WILL BE MULTIPLIED BY A FACTOR =  1.000
 TO GIVE NEW MAXIMUM ACCELERATION          =  0.70000

 MEAN SQUARE FREQUENCY =       3.38 C/SEC.
```

SKLH101.OUT

MAX ACCELERATION = 0.70000 FOR FREQUENCIES REMOVED ABOVE 25.00 C/SEC.

1****** OPTION 3 *** READ WHERE OBJECT MOTION IS GIVEN

OBJECT MOTION IN LAYER NUMBER 36 OUTCROPPING

1****** OPTION 4 *** OBTAIN STRAIN COMPATIBLE SOIL PROPERTIES

MAXIMUM NUMBER OF ITERATIONS = 20
MAXIMUM ERROR IN PERCENT = 2.00
FACTOR FOR EFFECTIVE STRAIN IN TIME DOMAIN = 0.65

EARTHQUAKE -
SOIL PROFILE - Skull Valley Horizontal MA

ITERATION NUMBER 1
THE CALCULATION HAS BEEN CARRIED OUT IN THE TIME DOMAIN WITH EFF. STRAIN = .65* MAX. STRAIN

G	LAYER	TYPE G USED	DEPTH ERROR	EFF. STRAIN NEW Vs	NEW DAMP.	DAMP USED	ERROR	NEW
27	1	1250.280	1.3 -14.1	0.00838 659.868	0.042	0.050	-19.3	1095.3
31	2	1250.280	3.8 -37.4	0.02508 601.502	0.071	0.050	30.0	910.1
64	3	1250.280	6.3 -58.3	0.04162 560.282	0.087	0.050	42.5	789.6
97	4	1250.280	8.8 -79.6	0.05787 526.117	0.097	0.050	48.7	696.2
28	5	1418.760	11.3 -88.4	0.06493 547.167	0.101	0.050	50.6	753.1
97	6	1418.760	13.8 -104.8	0.07836 524.756	0.107	0.050	53.4	692.6
68	7	1418.760	16.3 -120.2	0.09113 506.040	0.112	0.050	55.4	644.1
39	8	1418.760	18.8 -134.3	0.10307 490.673	0.116	0.050	57.0	605.6
44	9	1585.879	21.3 -133.1	0.10199 520.055	0.116	0.050	56.8	680.3
65	10	1585.879	23.8 -142.6	0.11083 509.757	0.119	0.050	58.0	653.6
74	11	1585.879	26.3 -151.0	0.11861 501.188	0.122	0.050	58.9	631.8
82	12	1585.879	28.8 -158.1	0.12525 494.202	0.124	0.050	59.6	614.3
98	13	15780.015	32.5 -26.9	0.01364 1865.921	0.035	0.050	-44.7	12434.4
20	14	15780.015	37.5 -29.9	0.01513 1844.582	0.037	0.050	-36.1	12151.7
72	15	15780.015	42.5 -32.5	0.01653 1825.957	0.039	0.050	-29.4	11907.5
35	16	15780.015	47.5 -34.9	0.01787 1809.573	0.040	0.050	-24.0	11694.8
90	17	15780.015	52.5 -37.1	0.01913 1795.067	0.042	0.050	-19.7	11508.0
32	18	15780.015	57.5 -39.1	0.02031 1782.155	0.043	0.050	-16.1	11343.1
62	19	18765.973	65.0 -38.7	0.01906 1905.440	0.064	0.050	21.3	13530.5
	20	3	75.0	0.02218	0.068	0.050	26.4	13102.5

SKLH101.OUT

96	18765.973	-43.2	1875.063					
	21	3	85.0	0.02303	0.069	0.050	27.5	14168.5
65	20458.318	-44.4	1949.846					
	22	3	95.0	0.02556	0.072	0.050	30.5	13848.6
29	20458.318	-47.7	1927.706					
	23	3	105.0	0.02615	0.073	0.050	31.2	14844.1
07	22042.076	-48.5	1995.788					
	24	3	115.0	0.02883	0.075	0.050	33.7	14520.8
56	22042.076	-51.8	1973.938					
	25	4	130.0	0.02890	0.064	0.010	84.3	18384.8
83	25556.934	-39.0	2133.962					
	26	4	150.0	0.03256	0.067	0.010	85.1	17927.6
19	25556.934	-42.6	2107.257					
	27	4	170.0	0.03313	0.068	0.010	85.2	19029.5
12	27228.979	-43.1	2171.051					
	28	4	190.0	0.03510	0.069	0.010	85.6	18793.1
99	27228.979	-44.9	2157.529					
	29	4	225.0	0.03926	0.073	0.010	86.2	19524.5
72	28997.266	-48.5	2199.110					
	30	4	275.0	0.04672	0.078	0.010	87.1	19946.5
53	30821.178	-54.5	2222.748					
	31	5	325.0	0.05073	0.072	0.010	86.1	22642.3
79	33393.754	-47.5	2368.194					
	32	5	375.0	0.05669	0.075	0.010	86.7	22084.6
64	33393.754	-51.2	2338.846					
	33	5	425.0	0.05472	0.074	0.010	86.5	24369.3
85	36553.742	-50.0	2456.849					
	34	5	475.0	0.05608	0.075	0.010	86.6	24233.7
70	36553.742	-50.8	2450.004					
	35	6	550.0	0.05256	0.068	0.010	85.2	27623.9
32	39451.672	-42.8	2615.767					

VALUES IN TIME DOMAIN

LAYER	TYPE	THICKNESS FT	DEPTH FT	MAX STRAIN PRCNT	MAX STRESS PSF	TIME SEC
1	1	2.5	1.3	0.01289	141.17	44.74
2	1	2.5	3.8	0.03858	351.17	44.74
3	1	2.5	6.3	0.06403	505.65	44.74
4	1	2.5	8.8	0.08903	619.94	44.74
5	1	2.5	11.3	0.09990	752.34	44.74
6	1	2.5	13.8	0.12056	835.10	44.74
7	1	2.5	16.3	0.14020	903.15	44.74
8	1	2.5	18.8	0.15857	960.35	44.74
9	1	2.5	21.3	0.15691	1067.56	44.74
10	1	2.5	23.8	0.17051	1114.54	44.74
11	1	2.5	26.3	0.18248	1153.03	44.74
12	1	2.5	28.8	0.19269	1183.86	44.74
13	2	5.0	32.5	0.02099	2610.09	44.74
14	2	5.0	37.5	0.02327	2827.72	44.74
15	2	5.0	42.5	0.02544	3028.85	44.74
16	2	5.0	47.5	0.02749	3214.63	44.74
17	2	5.0	52.5	0.02942	3386.16	44.74
18	2	5.0	57.5	0.03125	3544.50	44.74
19	3	10.0	65.0	0.02932	3967.60	60.56
20	3	10.0	75.0	0.03413	4471.37	60.56
21	3	10.0	85.0	0.03543	5020.37	60.56
22	3	10.0	95.0	0.03932	5444.85	60.56
23	3	10.0	105.0	0.04024	5972.70	60.54
24	3	10.0	115.0	0.04436	6441.21	60.54
25	4	20.0	130.0	0.04447	8175.59	60.54
26	4	20.0	150.0	0.05009	8979.65	60.54
27	4	20.0	170.0	0.05096	9698.33	60.54
28	4	20.0	190.0	0.05399	10146.99	60.54
29	4	50.0	225.0	0.06040	11793.28	60.62
30	4	50.0	275.0	0.07188	14337.19	60.60
31	5	50.0	325.0	0.07805	17672.07	60.62
32	5	50.0	375.0	0.08722	19261.73	60.62

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SKLH101.OUT

ITERATION NUMBER 15
THE CALCULATION HAS BEEN CARRIED OUT IN THE TIME DOMAIN WITH EFF. STRAIN = .65* MAX. STRAIN

LAYER	TYPE	DEPTH	EFF. STRAIN	NEW DAMP.	DAMP USED	ERROR	NEW G	G USED	ERROR	NEW Vs
1	1	1.3	0.00636	0.037	0.037	0.2	1120.7	1121.204	0.0	667.497
2	1	3.8	0.02290	0.069	0.069	0.2	926.8	927.648	-0.1	607.003
3	1	6.3	0.04606	0.090	0.090	0.2	760.9	762.498	-0.2	550.011
4	1	8.8	0.08056	0.108	0.108	0.2	602.5	604.310	-0.3	489.436
5	1	11.3	0.10148	0.116	0.115	0.2	610.1	611.947	-0.3	492.482
6	1	13.8	0.14956	0.131	0.130	0.2	498.6	500.727	-0.4	445.237
7	1	16.3	0.22905	0.147	0.147	0.3	376.2	379.911	-1.0	386.720
8	1	18.8	0.38148	0.167	0.166	0.1	256.6	257.536	-0.4	319.395
9	1	21.3	0.38002	0.166	0.166	0.1	287.4	288.421	-0.3	338.047
10	1	23.8	0.47155	0.174	0.174	0.1	252.7	253.758	-0.4	316.950
11	1	26.3	0.58677	0.181	0.181	0.2	217.4	219.150	-0.8	294.042
12	1	28.8	0.74755	0.190	0.189	0.4	178.4	181.972	-2.0	266.375
13	2	32.5	0.01278	0.033	0.033	0.0	12612.9	12614.044	0.0	1879.260
14	2	37.5	0.01490	0.036	0.036	0.0	12192.8	12192.035	0.0	1847.697
15	2	42.5	0.01708	0.039	0.039	0.0	11818.1	11815.845	0.0	1819.087
16	2	47.5	0.01932	0.042	0.042	-0.1	11480.5	11476.974	0.0	1792.915
17	2	52.5	0.02160	0.044	0.044	-0.1	11173.8	11169.227	0.0	1768.805
18	2	57.5	0.02393	0.047	0.047	-0.1	10893.4	10887.954	0.1	1746.473
19	3	65.0	0.02308	0.069	0.069	0.0	12990.1	12989.964	0.0	1866.998
20	3	75.0	0.02809	0.075	0.075	0.0	12435.8	12434.559	0.0	1826.732
21	3	85.0	0.02962	0.076	0.076	0.0	13395.0	13392.806	0.0	1895.877
22	3	95.0	0.03505	0.081	0.081	-0.1	12644.0	12636.825	0.1	1841.963
23	3	105.0	0.03625	0.083	0.083	-0.1	13428.4	13418.930	0.1	1898.237
24	3	115.0	0.04212	0.087	0.087	-0.1	12567.5	12555.923	0.1	1836.379
25	4	130.0	0.03261	0.067	0.067	-0.1	17921.6	17916.295	0.0	2106.904
26	4	150.0	0.03818	0.072	0.072	-0.1	17315.2	17309.258	0.0	2070.957
27	4	170.0	0.04050	0.074	0.074	-0.1	18206.7	18198.822	0.0	2123.598
28	4	190.0	0.04649	0.078	0.078	-0.1	17642.0	17634.799	0.0	2090.405
29	4	225.0	0.05537	0.083	0.083	-0.1	17529.9	17514.449	0.1	2083.754
30	4	275.0	0.06853	0.090	0.090	0.0	16920.2	16913.063	0.0	2047.199
31	5	325.0	0.05919	0.076	0.076	0.0	21868.2	21867.824	0.0	2327.359
32	5	375.0	0.06528	0.079	0.079	0.0	21257.8	21261.363	0.0	2294.646

SKLH101.OUT

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    33     5        425.0          0.06020          0.077        0.077        0.0        23844.0
12   23846.148       0.0    2430.222
    34     5        475.0          0.06074          0.077        0.077        0.0        23795.4
53   23797.898       0.0    2427.746
    35     6        550.0          0.05966          0.071        0.071        0.0        26873.1
99   26878.502       0.0    2579.978
```

VALUES IN TIME DOMAIN

LAYER	TYPE	THICKNESS FT	DEPTH FT	MAX STRAIN PRCNT	MAX STRESS PSF	TIME SEC
1	1	2.5	1.3	0.00978	109.60	47.88
2	1	2.5	3.8	0.03523	326.50	47.88
3	1	2.5	6.3	0.07086	539.21	47.88
4	1	2.5	8.8	0.12395	746.88	47.88
5	1	2.5	11.3	0.15612	952.49	47.88
6	1	2.5	13.8	0.23010	1147.41	47.88
7	1	2.5	16.3	0.35239	1325.71	47.88
8	1	2.5	18.8	0.58690	1506.07	47.90
9	1	2.5	21.3	0.58464	1680.63	47.88
10	1	2.5	23.8	0.72547	1833.28	47.88
11	1	2.5	26.3	0.90272	1963.38	47.88
12	1	2.5	28.8	1.15007	2052.77	47.88
13	2	5.0	32.5	0.01967	2480.80	47.86
14	2	5.0	37.5	0.02292	2795.10	47.86
15	2	5.0	42.5	0.02628	3105.73	47.86
16	2	5.0	47.5	0.02972	3412.17	47.86
17	2	5.0	52.5	0.03324	3713.89	47.86
18	2	5.0	57.5	0.03681	4010.36	47.86
19	3	10.0	65.0	0.03551	4613.26	60.74
20	3	10.0	75.0	0.04322	5375.04	60.74
21	3	10.0	85.0	0.04556	6103.18	60.74
22	3	10.0	95.0	0.05392	6817.43	60.74
23	3	10.0	105.0	0.05578	7489.83	60.74
24	3	10.0	115.0	0.06480	8143.91	60.74
25	4	20.0	130.0	0.05017	8990.69	60.74
26	4	20.0	150.0	0.05874	10171.16	60.74
27	4	20.0	170.0	0.06231	11344.33	60.72
28	4	20.0	190.0	0.07152	12618.25	60.70
29	4	50.0	225.0	0.08518	14932.25	60.70
30	4	50.0	275.0	0.10544	17840.29	60.68
31	5	50.0	325.0	0.09106	19912.97	60.68
32	5	50.0	375.0	0.10043	21348.29	60.66
33	5	50.0	425.0	0.09262	22085.02	60.66
34	5	50.0	475.0	0.09344	22235.62	60.66
35	6	100.0	550.0	0.09178	24665.40	59.20

PERIOD = 1.12 FROM AVERAGE SHEARVEL. = 2142.

MAXIMUM AMPLIFICATION = 9.83
FOR FREQUENCY = 1.01 C/SEC.
PERIOD = 0.99 SEC.

1****** OPTION 5 *** COMPUTE MOTION IN NEW SUBLAYERS

EARTHQUAKE -
SOIL DEPOSIT - Skull Valley Horizontal MA

LAYER PUNCHED CARDS	DEPTH FT	MAX. ACC. G	TIME SEC	MEAN SQ. FR. C/SEC	QUIET ZONE	ACC. RATIO ACC. RECORD
OUTCR. 4096	0.0	1.08854	47.86	1.87		0.000

SKLH101.OUT

WITHIN 0	5.0	1.08502	47.86	1.85	0.000
WITHIN 0	10.0	1.07028	47.86	1.78	0.000
WITHIN 0	15.0	1.03948	47.86	1.67	0.000
WITHIN 0	20.0	0.95406	47.86	1.55	0.000
WITHIN 0	25.0	0.81421	47.86	1.58	0.000
WITHIN 0	30.0	0.75008	44.74	1.90	0.000
WITHIN 0	40.0	0.74530	44.74	1.88	0.000
WITHIN 0	50.0	0.72941	44.74	1.80	0.000
WITHIN 0	60.0	0.70179	44.74	1.68	0.000

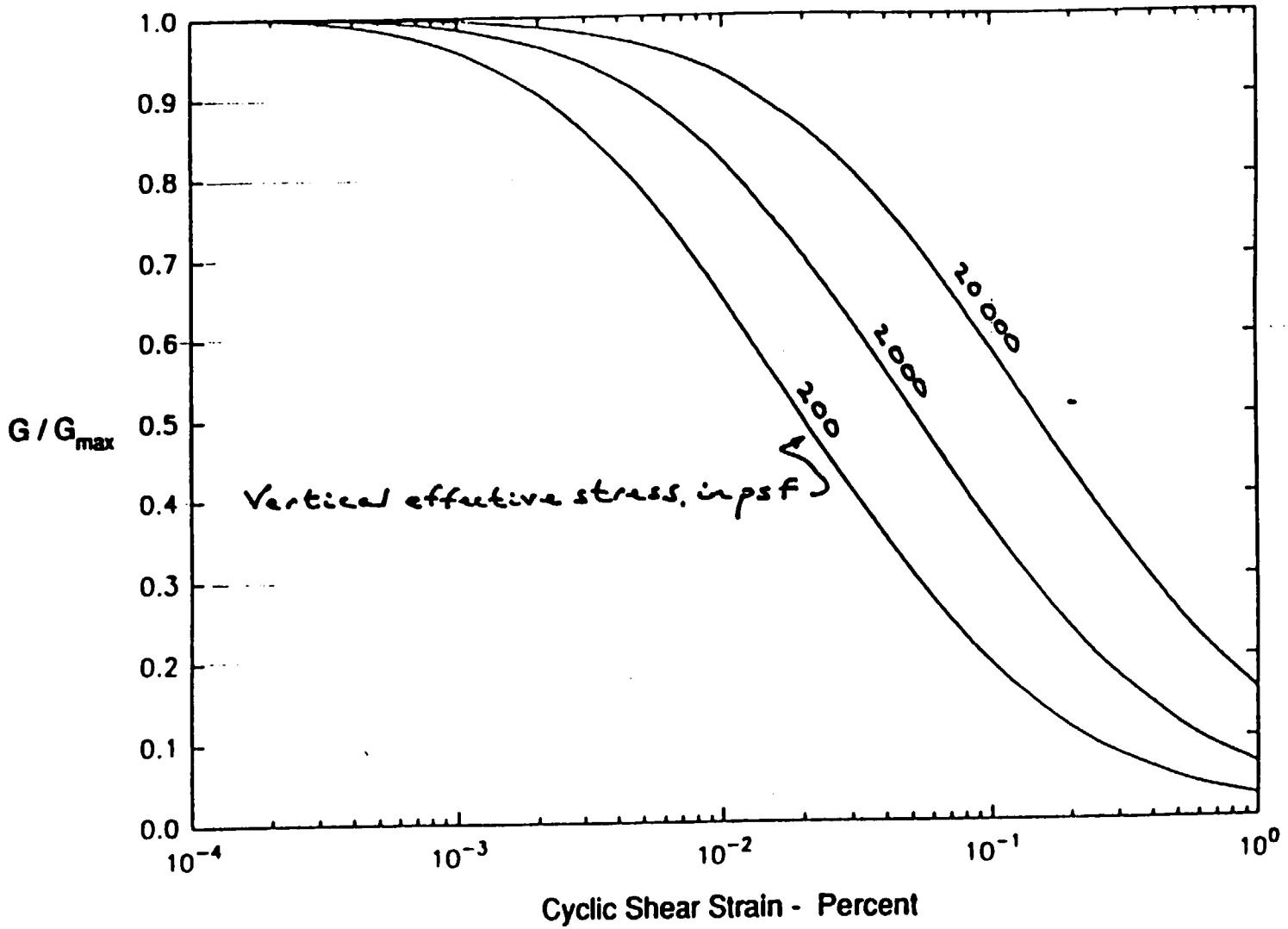
1****** OPTION 5 *** COMPUTE MOTION IN NEW SUBLAYERS

EARTHQUAKE -
SOIL DEPOSIT - Skull Valley Horizontal MA

LAYER PUNCHED CARDS	DEPTH FT	MAX. ACC. G	TIME SEC	C/SEC	MEAN SQ. FR. QUIET ZONE	ACC. RATIO ACC. RECORD
WITHIN 0	80.0	0.63969		44.76	1.47	0.000
WITHIN 0	100.0	0.55861		44.76	1.31	0.000
WITHIN 0	120.0	0.56109		60.68	1.26	0.000
WITHIN 0	160.0	0.58194		60.68	1.35	0.000
WITHIN 0	200.0	0.56036		60.66	1.51	0.000
WITHIN 0	300.0	0.53660		48.06	1.84	0.000
WITHIN 0	400.0	0.53326		48.02	2.17	0.000
WITHIN 0	500.0	0.51139		35.44	2.65	0.000
WITHIN 0	600.0	0.53183		60.36	2.95	0.000
OUTCR. 4096	600.0	0.70000		60.36	3.38	0.000

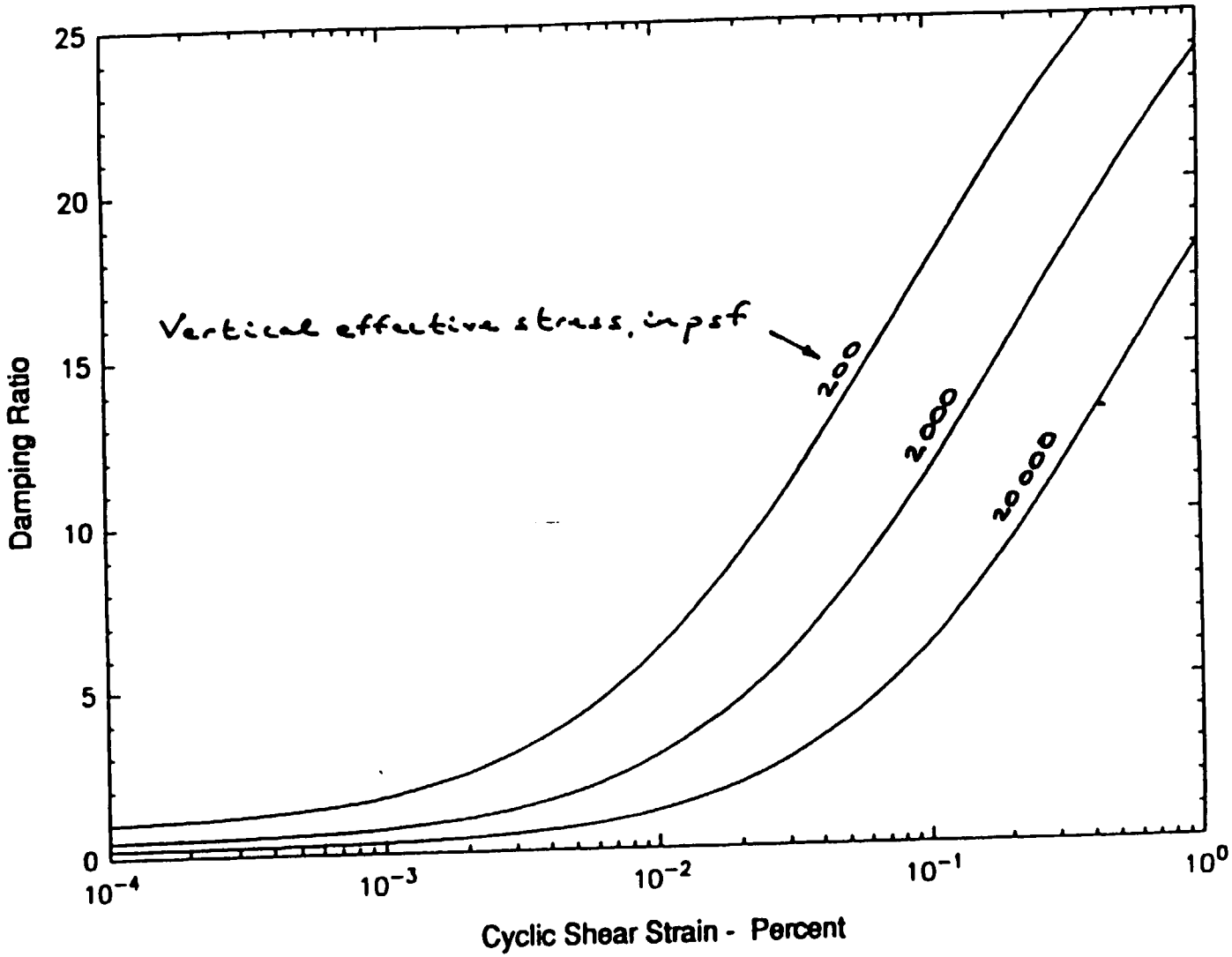
SKLH101A

											AVE Vs	AVE G	AVE D
1	1	1.3	0.00636	0.037	0.037	0.2	1120.799	1121.204	0	667.497	637	1024	0.053
2	1	3.8	0.0229	0.069	0.069	0.2	926.854	927.648	-0.1	607.003			
3	1	6.3	0.04606	0.09	0.09	0.2	760.978	762.498	-0.2	550.011	520	682	0.099
4	1	8.8	0.08056	0.108	0.108	0.2	602.588	604.31	-0.3	489.436			
5	1	11.3	0.10148	0.116	0.115	0.2	610.111	611.947	-0.3	492.482	469	554	0.124
6	1	13.8	0.14956	0.131	0.13	0.2	498.669	500.727	-0.4	445.237			
7	1	16.3	0.22905	0.147	0.147	0.3	376.203	379.911	-1	386.72	353	316	0.157
8	1	18.8	0.38148	0.167	0.166	0.1	256.616	257.536	-0.4	319.395			
9	1	21.3	0.38002	0.166	0.166	0.1	287.464	288.421	-0.3	338.047	327	270	0.170
10	1	23.8	0.47155	0.174	0.174	0.1	252.704	253.758	-0.4	316.95			
11	1	26.3	0.58677	0.181	0.181	0.2	217.495	219.15	-0.8	294.042	280	198	0.186
12	1	28.8	0.74755	0.19	0.189	0.4	178.491	181.972	-2	266.375			
13	2	32.5	0.01278	0.033	0.033	0	12612.93	12614.04	0	1879.26	1809	11695	0.040
14	2	37.5	0.0149	0.036	0.036	0	12192.8	12192.04	0	1847.697			
15	2	42.5	0.01708	0.039	0.039	0	11818.13	11815.85	0	1819.087			
16	2	47.5	0.01932	0.042	0.042	-0.1	11480.52	11476.97	0	1792.915			
17	2	52.5	0.0216	0.044	0.044	-0.1	11173.83	11169.23	0	1768.805			
18	2	57.5	0.02393	0.047	0.047	-0.1	10893.46	10887.95	0.1	1746.473			
19	3	65	0.02308	0.069	0.069	0	12990.12	12989.96	0	1866.998	1861	12910	0.079
20	3	75	0.02809	0.075	0.075	0	12435.84	12434.56	0	1826.732			
21	3	85	0.02962	0.076	0.076	0	13395.09	13392.81	0	1895.877			
22	3	95	0.03505	0.081	0.081	-0.1	12644.08	12636.83	0.1	1841.963			
23	3	105	0.03625	0.083	0.083	-0.1	13428.45	13418.93	0.1	1898.237			
24	3	115	0.04212	0.087	0.087	-0.1	12567.54	12555.92	0.1	1836.379			
25	4	130	0.03261	0.067	0.067	-0.1	17921.61	17916.3	0	2106.904	2080	17468	0.080
26	4	150	0.03818	0.072	0.072	-0.1	17315.29	17309.26	0	2070.957			
27	4	170	0.0405	0.074	0.074	-0.1	18206.74	18198.82	0	2123.598			
28	4	190	0.04649	0.078	0.078	-0.1	17642.01	17634.8	0	2090.405			
29	4	225	0.05537	0.083	0.083	-0.1	17529.93	17514.45	0.1	2083.754			
30	4	275	0.06853	0.09	0.09	0	16920.29	16913.06	0	2047.199			
31	5	325	0.05919	0.076	0.076	0	21868.27	21867.82	0	2327.359	2440	24085	0.075
32	5	375	0.06528	0.079	0.079	0	21257.84	21261.36	0	2294.646			
33	5	425	0.0602	0.077	0.077	0	23844.01	23846.15	0	2430.222			
34	5	475	0.06074	0.077	0.077	0	23795.45	23797.9	0	2427.746			
35	6	550	0.05966	0.071	0.071	0	26873.2	26878.5	0	2579.978			



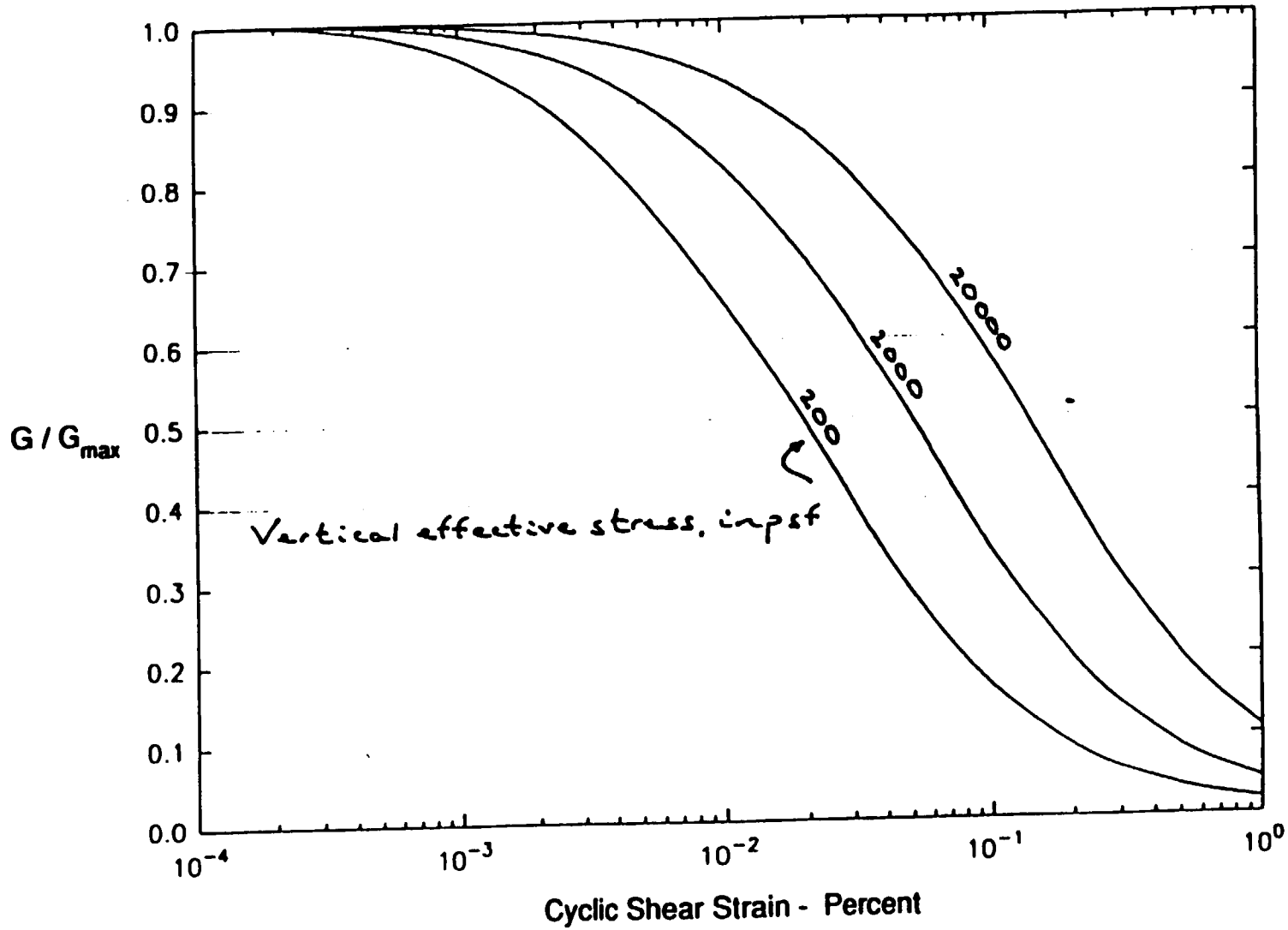
From EPRI (1993)

Figure 7.A-12
Modulus Reduction Curves for Dry Sands



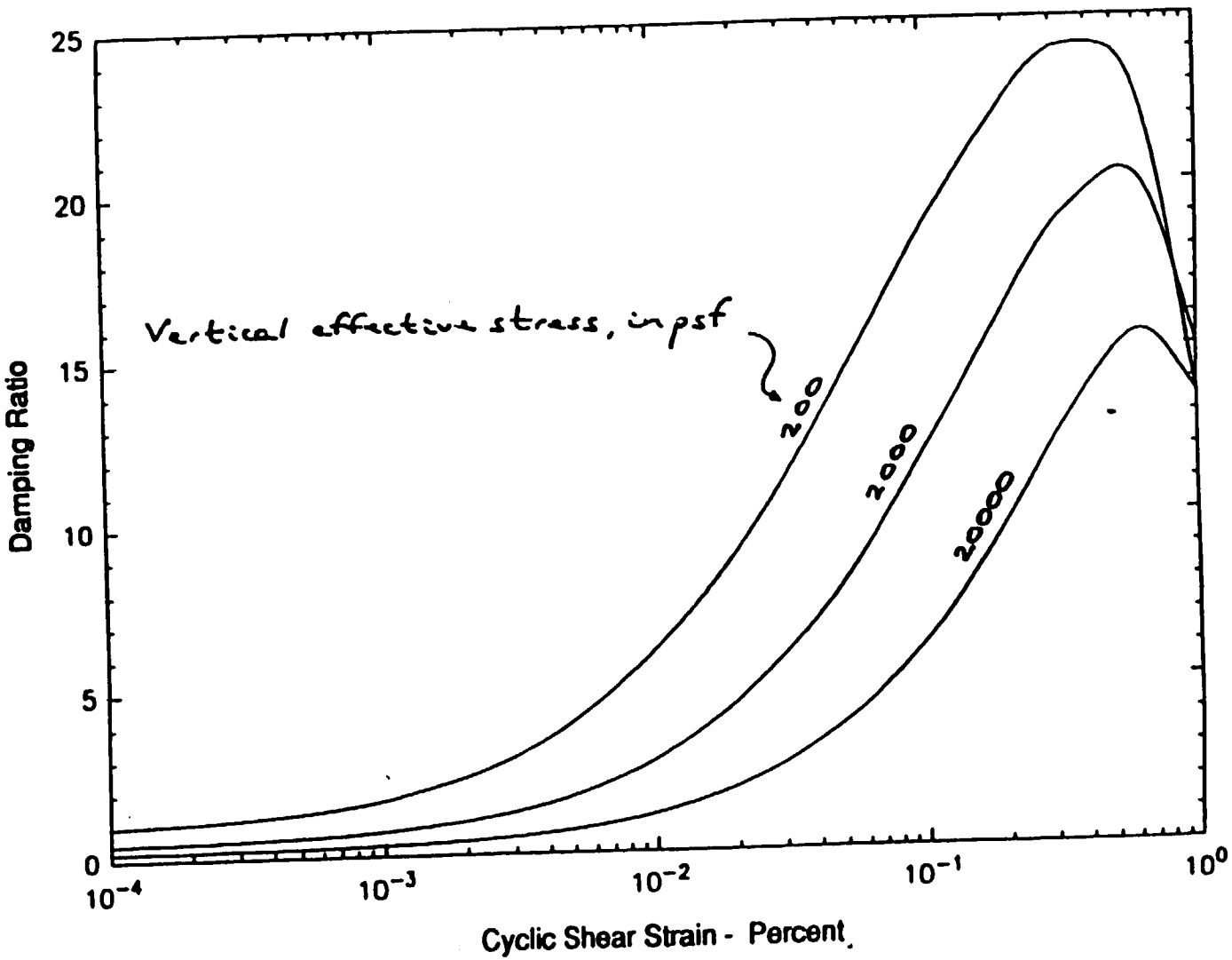
From EPRI (1993)

Figure 7.A-13
Damping Curves for Dry Sands



From EPRI (1993)

Figure 7.A-14
Modulus Reduction Curves for Saturated Sands



From EPRI (1993)

Figure 7.A-15
Damping Curves for Saturated Sands

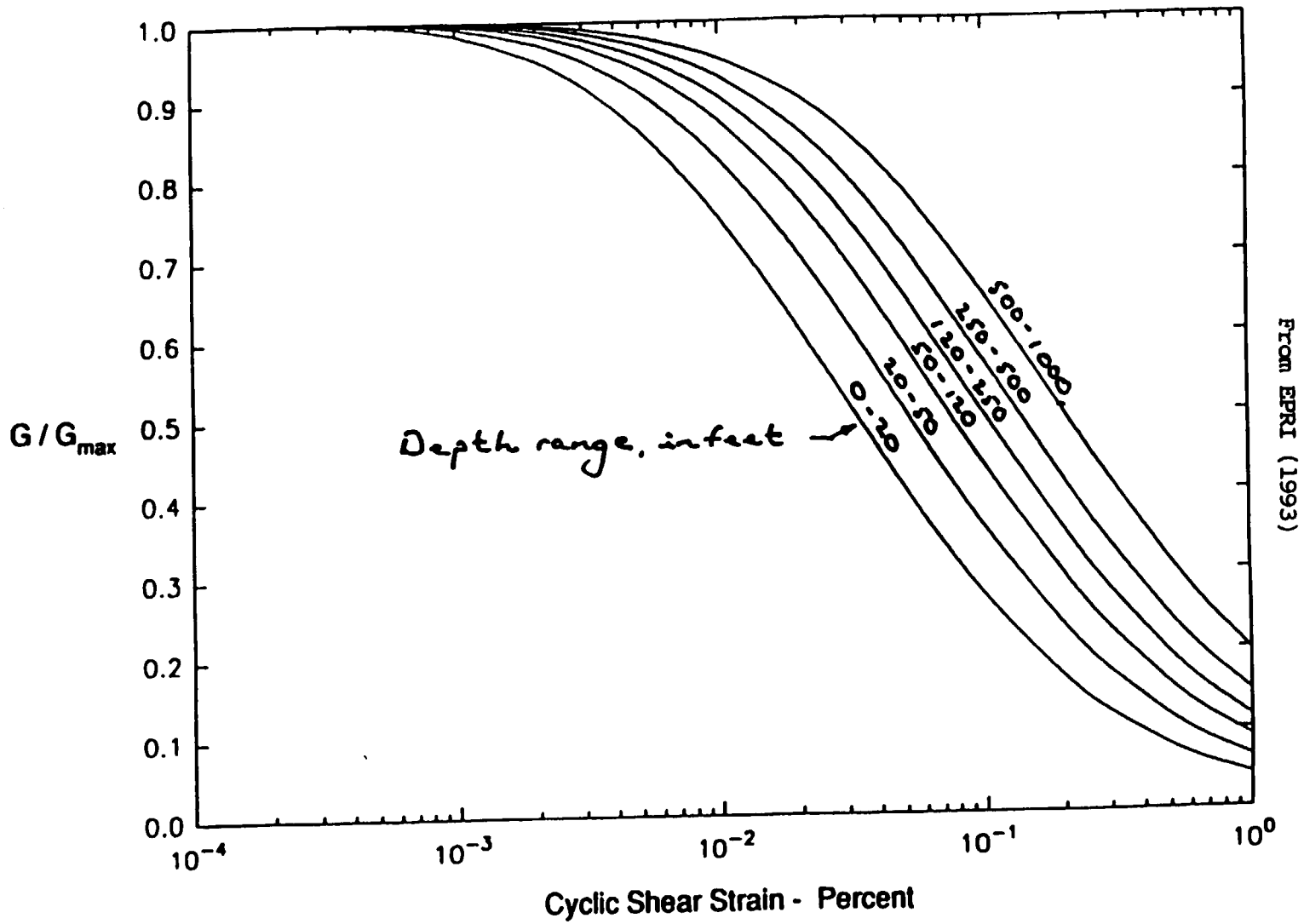
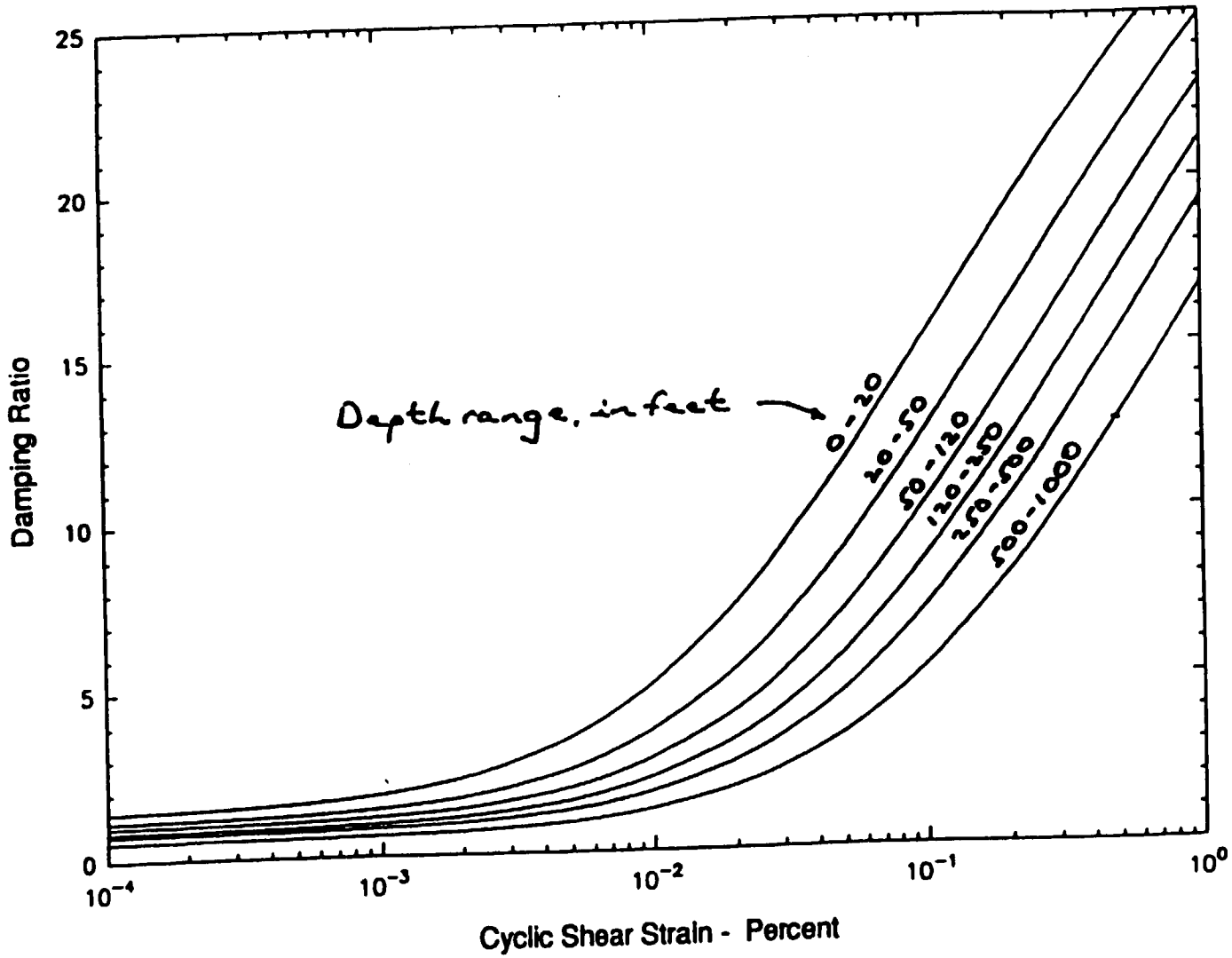


Figure 7.A-18
Modulus Reduction Curves for Generic ENA Sites



From EPRI (1993)

Figure 7.A-19
Damping Curves for Generic ENA Sites

ITERATION NUMBER 15
THE CALCULATION HAS BEEN CARRIED OUT IN THE TIME DOMAIN WITH EFF. STRAIN = .65* MAX. STRAIN

LAYER	TYPE	DEPTH	EFF. STRAIN		NEW DAMP.		DAMP USED		ERROR		NEW G	G USED	ERROR	NEW Vs	
1	1	1.3	0.00636		0.037		0.037		0.2		1120.799	1121.204	0.0	667.497	
2	1	3.8	0.02290	5 ft	0.069	0.053	0.069		0.2	1024	926.854	927.648	-0.1	607.003	637
3	1	6.3	0.04606		0.090		0.090		0.2		760.978	762.498	-0.2	550.011	
4	1	8.8	0.08056	10	0.108	0.099	0.108		0.2	682	602.588	604.310	-0.3	489.436	520
5	1	11.3	0.10148		0.116		0.115		0.2		610.111	611.947	-0.3	492.482	
6	1	13.8	0.14956	15	0.131	0.124	0.130		0.2	554	498.669	500.727	-0.4	445.237	469
7	1	16.3	0.22905		0.147		0.147		0.3		376.203	379.911	-1.0	386.720	
8	1	18.8	0.38148	20	0.167	0.157	0.166		0.1	316	256.616	257.536	-0.4	319.395	353
9	1	21.3	0.38002		0.166		0.166		0.1		287.464	288.421	-0.3	338.047	
10	1	23.8	0.47155	25	0.174	0.170	0.174		0.1	270	252.704	253.758	-0.4	316.950	327
11	1	26.3	0.58677		0.181		0.181		0.2		217.495	219.150	-0.8	294.042	
12	1	28.8	0.74755	30	0.190	0.186	0.189		0.4	198	178.491	181.972	-2.0	266.375	280
13	2	32.5	0.01278		0.033		0.033		0.0		12612.927	12614.044	0.0	1879.260	
14	2	37.5	0.01490		0.036		0.036		0.0		12192.804	12192.035	0.0	1847.697	
15	2	42.5	0.01708		0.039		0.039		0.0		11818.133	11815.845	0.0	1819.087	
16	2	47.5	0.01932		0.042	0.040	0.042		-0.1	11695	11480.517	11476.974	0.0	1792.915	1809
17	2	52.5	0.02160		0.044		0.044		-0.1		11173.828	11169.227	0.0	1768.805	
18	2	57.5	0.02393	60	0.047		0.047		-0.1		10893.462	10887.954	0.1	1746.473	
19	3	65.0	0.02308		0.069		0.069		0.0		12990.115	12989.964	0.0	1866.998	
20	3	75.0	0.02809		0.075		0.075		0.0		12435.843	12434.559	0.0	1826.732	
21	3	85.0	0.02962		0.076		0.076		0.0		13395.094	13392.806	0.0	1895.877	
22	3	95.0	0.03505		0.081	0.079	0.081		-0.1	12910	12644.083	12636.825	0.1	1841.963	1861
23	3	105.0	0.03625		0.083		0.083		-0.1		13428.454	13418.930	0.1	1898.237	
24	3	115.0	0.04212	120	0.087		0.087		-0.1		12567.538	12555.923	0.1	1836.379	
25	4	130.0	0.03261		0.067		0.067		-0.1		17921.611	17916.295	0.0	2106.904	
26	4	150.0	0.03818		0.072		0.072		-0.1		17315.287	17309.258	0.0	2070.957	
27	4	170.0	0.04050		0.074		0.074		-0.1		18206.738	18198.822	0.0	2123.598	
28	4	190.0	0.04649		0.078	0.080	0.078		-0.1	17468	17642.014	17634.799	0.0	2090.405	2080
29	4	225.0	0.05537		0.083		0.083		-0.1		17529.932	17514.449	0.1	2083.754	
30	4	275.0	0.06853	300	0.090		0.090		0.0		16920.289	16913.063	0.0	2047.199	
31	5	325.0	0.05919		0.076		0.076		0.0		21868.268	21867.824	0.0	2327.359	
32	5	375.0	0.06528		0.079		0.079		0.0		21257.836	21261.363	0.0	2294.646	
33	5	425.0	0.06020		0.077	0.075	0.077		0.0	24085	23844.012	23846.148	0.0	2430.222	2440
34	5	475.0	0.06074		0.077		0.077		0.0		23795.453	23797.898	0.0	2427.746	
35	6	550.0	0.05966	600	0.071		0.071		0.0		26873.199	26878.502	0.0	2579.978	

SUMSASSI.XLS

Depth Top	Depth Bottom	Density	Vs	Damping
0	5	81	637	0.053
5	10	81	520	0.099
10	15	81	469	0.124
15	20	81	353	0.157
20	25	81	327	0.170
25	30	81	280	0.186
30	60	115	1809	0.040
60	120	120	1861	0.079
120	300	130	2080	0.080
300	600	130	2440	0.075
600		150	5000	0.000

SK I

LAYER	TYPE	DEPTH	EFF. STRAIN (%)	NEW DAMP.	NEW G	NEW Vs		shape	weight	contribution G (ksf)	D
1	1	1.3	0.00636	0.037	1120.799	667.497		28.7	0.16	179.30	0.59
2	1	3.8	0.0229	0.069	926.854	607.003		26.2	0.15	135.36	1.01
3	1	6.3	0.04606	0.09	760.978	550.011		23.7	0.13	100.53	1.19
4	1	8.8	0.08056	0.108	602.588	489.436		21.2	0.12	71.21	1.28
5	1	11.3	0.10148	0.116	610.111	492.482		18.7	0.10	63.60	1.21
6	1	13.8	0.14956	0.131	498.669	445.237		16.2	0.09	45.03	1.18
7	1	16.3	0.22905	0.147	376.203	386.72		13.7	0.08	28.73	1.12
8	1	18.8	0.38148	0.167	256.616	319.395		11.2	0.06	16.02	1.04
9	1	21.3	0.38002	0.166	287.464	338.047		8.7	0.05	13.94	0.81
10	1	23.8	0.47155	0.174	252.704	316.95		6.2	0.03	8.73	0.60
11	1	26.3	0.58677	0.181	217.495	294.042		3.7	0.02	4.49	0.37
12	1	28.8	0.74755	0.19	178.491	266.375		1.2	0.01	1.19	0.13
13	2	32.5	0.01278	0.033	12612.927	1879.26					
14	2	37.5	0.0149	0.036	12192.804	1847.697			sum G	668.13	
15	2	42.5	0.01708	0.039	11818.133	1819.087			sum D		10.53
16	2	47.5	0.01932	0.042	11480.517	1792.915			Vs (fps)	515.37	
17	2	52.5	0.0216	0.044	11173.828	1768.805					
18	2	57.5	0.02393	0.047	10893.462	1746.473					
19	3	65	0.02308	0.069	12990.115	1866.998					
20	3	75	0.02809	0.075	12435.843	1826.732					
21	3	85	0.02962	0.076	13395.094	1895.877					
22	3	95	0.03505	0.081	12644.083	1841.963					
23	3	105	0.03625	0.083	13428.454	1898.237					
24	3	115	0.04212	0.087	12567.538	1836.379					
25	4	130	0.03261	0.067	17921.611	2106.904					
26	4	150	0.03818	0.072	17315.287	2070.957					
27	4	170	0.0405	0.074	18206.738	2123.598					
28	4	190	0.04649	0.078	17642.014	2090.405					
29	4	225	0.05537	0.083	17529.932	2083.754					
30	4	275	0.06853	0.09	16920.289	2047.199					
31	5	325	0.05919	0.076	21868.268	2327.359					
32	5	375	0.06528	0.079	21257.836	2294.646					
33	5	425	0.0602	0.077	23844.012	2430.222					
34	5	475	0.06074	0.077	23795.453	2427.746					
35	6	550	0.05966	0.071	26873.199	2579.978					

Subject SKULL VALLEY NSF

By Chin Man Mok Checked By C.C. CHIN

Date 2/26/97 Date 2/26/97

Properties of Equivalent homogeneous isotropic medium:

$V_p = 1500$ fps $\beta_p = 10\%$

$V_s = 515$ fps $\beta_s = 11\%$

$G = 668130$ psf $E = 1915$ ksf

$\mu = 0.433$ $\rho = 81$ pcf.

These equivalent properties were computed by weighted average described in the analysis outline.

Subject SKULL VALLEY NSF

By Chin Man Mok Checked By C. C. CHIN

Date 2/26/97 Date 2/26/97

SUMMARY of analysis results.

dof	mass/area (pcf-sec^2)	dashpot constant/area (kcf-sec)	spring constant/area (kcf)
Vertical	30	1.94	59
Horizontal	5.5	0.97	40
Rocking	38.6	1.39	138

Subject SKULL VALLEY NSF

By Chin Man Mok Checked By C.C. CHIN

Date 2/26/97 Date 2/26/97

ESTIMATION OF DYNAMIC PARAMETERS

Analysis is based on equations published by Newmark and Rosenblueth (1971). Copies of the equations are attached on Page 10/11 in this section 1.5

1. Vertical d.o.f.

$h = 0.27\sqrt{A}$

$A = 30 \times 64 = 1920 \ ft^2$

$h = 0.27\sqrt{1920} = 11.83$

$M = Ah\rho$

$m = \frac{M}{A} = h\rho = 81 \times \frac{11.83}{32.2}$ psf/ft/s²

$= 30 \ \frac{lb\text{-}sec^2}{ft^3}$ or $0.03 \ kcf\text{-}sec^2$ #

$K_v = \frac{E\sqrt{A}\,C_s}{1-\nu^2}$

$C_s = 1.1$ for $\frac{L}{B} = 2.13$

$\therefore k_v = \frac{K_v}{A} = \frac{E}{\sqrt{A}}\frac{C_s}{1-\nu^2} = \frac{1915 \times 1.1}{\sqrt{1920}\,(1-0.433^2)} = 59 \ kcf$ #

$C = 5.42\sqrt{K_v \rho h^3}$

$c = \frac{C}{A} = \frac{5.42}{A}\sqrt{k_v A \rho h^3}$

$= \frac{5.42}{1920}\sqrt{59 \times 1920 \times \frac{0.081}{32.2} \times 11.83^3}$

$= 1.94 \ kcf\text{-}sec$

Subject SKULL VALLEY NSF

By Chin Man Mok Checked By C. C. Chen

Date 2/26/97 Date 2/26/97

2. Horizontal dof

$h = 0.05\sqrt{A}$

$= 0.05\sqrt{1920} = 2.191$ ft.

$M = Ah\rho$

$m = \frac{M}{A} = h\rho = \frac{0.081 \times 2.191}{32.2} = 0.0055$ kcf-sec^2

or 5.51 pcf-sec^2

$K_h = \frac{E\sqrt{A}\,k_T}{1-\nu^2}$

$k_T = 0.75$ for $\nu = 0.433$

$\therefore k_h = \frac{K_h}{A} = \frac{1915 \times 0.75}{\sqrt{1920}\,(1-0.433^2)} = 40.3$ kcf

$C = 41.1\sqrt{K\rho h^3}$

$c = \frac{C}{A} = \frac{41.1}{A}\sqrt{k_h A \rho h^3}$

$= \frac{41.1}{1920}\sqrt{40.3 \times 1920 \times \frac{0.081}{32.2} \times 2.191^3}$

$= 0.969$ kcf-sec

By Chin Man Mok Checked By C.C. CHIN

Date 2/26/97 Date 2/26/97

3. Rocking d.o.f.

$h = 0.35\sqrt{A}$

$= 0.35\sqrt{1920} = 15.34$ ft

$M = Ah\rho$

$m = \frac{M}{A} = h\rho = \frac{0.081 \times 15.34}{32.2} = 0.0386$ kcf-sec^2

≈ 38.6 pcf-sec^2

$K_r = \frac{EI\,k_\phi}{\sqrt{A}\,(1-\nu^2)} = \frac{ELB^3 k_\phi}{12\sqrt{BL}\,(1-\nu^2)} = \frac{1915 \times 64 \times 30^3 \times 2.57}{12\sqrt{1920}\,(1-0.433^2)}$

$\doteq 19906560$ kip-ft/rad

$(= 19906024)$

$k_\phi = 2.57$

$\text{Moment} = \int_{-\frac{B}{2}}^{\frac{B}{2}} k_r L x^2 \phi\, dx = \frac{k_r L \phi}{3}\left(x^3\right)_{-\frac{B}{2}}^{\frac{B}{2}}$

$= \frac{k_r L \phi}{3}\,\frac{2B^3}{8} = \frac{k_r L B^3}{12}\phi$

plus ?

$\therefore k_r \frac{LB^3}{12} = \frac{ELB^3 \times 2.57}{12\sqrt{BL}\,(1-\nu^2)}$

$= \frac{2.57\,E}{\sqrt{BL}\,(1-\nu^2)} = \frac{1915 \times 2.57}{\sqrt{1920}\,(1-0.433^2)} = 138.24$ kcf

$C = 0.97\sqrt{K\rho h^5}$

$C = 0.97\sqrt{19906560 \times \frac{0.081}{32.2} \times 15.34^5}$

$= 200054.3$ kip-ft-sec/rad

$c = \frac{C}{I} = \frac{200054.3 \times 12}{64 \times 30^3} = 1.39$ kcf-sec

(ref. moment $= \int_{-\frac{B}{2}}^{\frac{B}{2}} c L x^2 \dot{\phi}\, dx = c \cdot I\dot{\phi}$ where $I = \frac{LB^3}{12}$)

From Newmark and Rosenblueth (1971)

velocity of Love waves lies between these two shear velocities and is a function of the frequency. Using primes to denote the stratum, the velocity of Love waves v_l can be found from

$$\mu\left(1 - \frac{v_l^2}{v_s^2}\right)^{1/2} - \mu'\left(\frac{v_l^2}{v_s'^2} - 1\right)^{1/2} \tan \kappa H\left(\frac{v_l^2}{v_s'^2} - 1\right)^{1/2} = 0 \qquad (3.57)$$

where $\kappa = \omega/v_l$. We see that as $\kappa \to 0$, or when we deal with long waves, $v_l \to v_s$, and as $\kappa \to \infty$, for short waves, $v_l \to v_s'$.

Solutions are available for Rayleigh, Love, and other types of waves under a variety of stratification conditions. In many such solutions the velocity of wave propagation is a function of the wave frequency. When this happens, unless we are dealing with sinusoidal, steady-state conditions, we find that the shape of a disturbance changes as it travels along the medium in question. Sharp disturbances become trains of waves, each train containing oscillations of essentially equal frequency. Further, the velocity of a group of waves under these conditions differs from the velocity of an individual wave. This type of dispersion does not necessarily combine in additive manner with the dispersion due to internal damping[5] and accounts partly for the increase in duration of earthquake motions with focal distance.

3.14 *Group Velocity*

We have seen that in viscoelastic materials, wave velocities are functions of the frequency of the waves. Even in a perfectly elastic solid, Love waves, among others, travel with a velocity that depends on the frequency and hence, ordinarily, on wavelength. The phenomenon, known as dispersion, gives rise to reinforcement and interference of waves having nearly the same velocities. This causes the appearance of clusters of waves of essentially equal wavelengths. The location of these clusters in space moves with a velocity, called *group velocity*, that differs from the velocities of the waves.

Some idea of the effect of dispersion in this context may be gleaned from the study of the combination of two one-dimensional waves of the same amplitude but slightly different frequencies and velocities. Let us consider, then, the combined wave

$$x = a \sin \kappa(X - vt) + \sin(\kappa + \Delta\kappa)[X - (v + \Delta v)t]$$

where $\kappa v = \omega$, the circular frequency. This we can write in the form

$$x = 2a \sin\left(\frac{2\kappa + \Delta\kappa}{2} X - \frac{2\omega + \Delta\omega}{2} t\right) \cos\left(\frac{\Delta\kappa}{2} X - \frac{\Delta\omega}{2} t\right)$$

The sine function in this expression represents a wave with a frequency and a length equal to the averages of the original waves. The cosine function is a

[5] This remark is proved for waves traveling along a linearly damped cylindrical rod (Hunter, 1960).

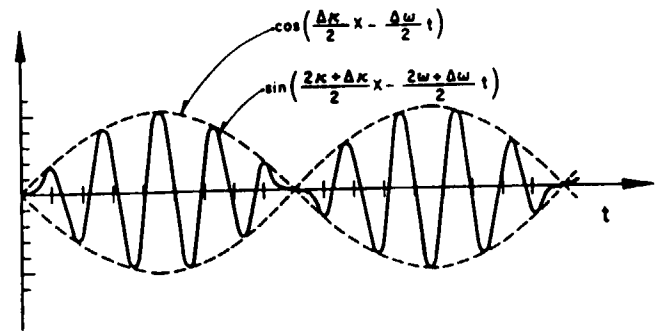


Figure 3.15. Combined wave.

very long wave that envelops the motions corresponding to the first factor, as shown by the dashed line in Fig. 3.15. This envelope moves in the direction of the waves with a velocity equal to $(\Delta\omega/2)/(\Delta\kappa/2)$ or $\Delta\omega/\Delta\kappa$. In the limit, when there is a continuous spectrum of wave frequencies, we may write for the group velocity

$$v_g = \frac{d\omega}{d\kappa} = v + \kappa\frac{dv}{d\kappa}$$

or, introducing the symbol $\Lambda = 2\pi/\kappa$ for wavelength,

$$v_g = v - \Lambda\frac{dv}{d\Lambda}$$

Only when v does not depend on the wavelength does the group velocity coincide with the wave velocity, and no clusters develop.

It can be shown that, when dispersed waves undergo reflection and refraction at an interface, the angles that the corresponding paths form with the interface are functions of the individual wave velocities as for nondispersed waves, while the velocities of transmission of energy follow the law of the group velocities.[6]

3.15 *Soil-Foundation Interaction*

The same contact stresses between soil and foundation that may be held responsible for earthquake effects on structures also cause deformations in the soil, especially in the vicinity of every structural foundation. The phenomenon constitutes one form of dynamic soil-structure interaction. It is also known in the literature as "energy feedback to the ground," "foundation yielding," and "foundation compliance." It has received considerable attention with a

[6] A more thorough explanation of the matter of group velocity, based on a Fourier integral representation of dispersed waves, is found in Bullen (1953), pp. 58–66, 93–95, and 107–108.

From Newmark and Rosenblueth (1971)

view to application both to seismic problems and to the study of vibrations of machine foundations. Yet no entirely satisfactory solution is available for cases other than circular foundations, even under the assumption of perfectly elastic soil behavior.

A rigid body resting on soil has six degrees of freedom: for example, an up-and-down motion, torsion about a vertical axis, two degrees in rocking, and two degrees of horizontal translation. Suppose that the responses in all modes were known for a massless body subjected either to an instantaneous pulse or to a harmonic, steady-state disturbance along each component. Then appropriate use of either convolution integrals, Laplace (Sandi, 1960), or Fourier (Monge and Rosenberg, 1964) transforms would permit calculation of the responses of any structure of linear behavior resting on a rigid foundation supported in turn by a soil of linear behavior.

Most of the solutions available concern a rigid plate, either circular or rectangular, resting on an isotropic, homogeneous, linearly elastic halfspace, under steady-state vibration and have been obtained assuming that the distribution of contact stresses is the same as under static loading, independently of the frequency of vibration. Actually the distribution of contact stresses depends on the frequency. Lysmer (1965) has succeeded in solving the problem of a rigid plate under steady-state vertical oscillation taking into account the proper distribution of contact stresses. To this end he has taken the solution for a flexible plate that applies a vibratory uniform pressure on the ground (Sung, 1953). By subtracting the effects of a smaller concentric plate he has obtained the responses to a ring that applies uniformly distributed vibratory pressures; by replacing the rigid plate with a set of 20 concentric rings and equating their vertical displacements at every instant he has obtained a numerical solution.

Using a somewhat similar approach, Elorduy (1967) has developed a method applicable to the vibrations of a rigid plate of arbitrary shape resting on an elastic halfspace. He makes use of the known solution for the free-field effects of a vertical (Pekeris, 1955) or a horizontal (Chao, 1960) concentrated pulse applied at a point of the free surface of the elastic halfspace. He then solves two sets of simultaneous equations to satisfy the boundary condition at the base of the plate. Elorduy's application to rectangular plates is beset with the simplifying assumption that the phase lag between force and displacement is the same at all points of contact between the plate and the halfspace. Nevertheless, his solution for the oscillations of a square plate agrees well with the solution due to Kobori (1962), which was obtained by a different procedure.

Elorduy's approach, after removing the simplifying assumption and incorporating an explicit consideration of coupling between vertical and horizontal displacements, can give results as accurate as desired for plates of arbitrary shape. However, as in Lysmer's treatment, the method gives rise to sets of very ill-conditioned equations in some range of the variables. This difficulty was obviated by Robertson (1966) through a transformation of the integral equation from which these sets of equations are derived. He was thus able to arrive at the exact solution for the vertical oscillations of a rigid circular plate on an

elastic halfspace. His method can be adapted to the analysis of the rocking, torsional, and translational oscillations of rigid circular plates and to the vibrations of infinitely long rigid band plates. However, it is not applicable in any form to finite square or rectangular plates.

Tajimi (1969) has been able to solve the problem of rocking and translational oscillations of a rigid, circular, cylindrical pier embedded in an elastic stratum when both the stratum and the pier rest on an elastic halfspace.

A comparison of the exact solution for a rigid circular plate on an elastic halfspace with the solution based on the same distribution as under static conditions shows that the latter is satisfactory up to and somewhat beyond the resonant frequency, but not much beyond. For very high frequencies the solution obtained by assuming a static pressure distribution even predicts an equivalent negative damping, which makes it unacceptable. In the study of the vibration of machine foundations, such high frequencies are often of interest; in problems of earthquake-resistant design this is not necessarily the case. Since many problems have been solved only under the simplifying assumption in question, we shall retain it in the presentation of some solutions.

Our lack of concern with very high frequencies stems from the following consideration. It is well known that soil-foundation interaction may affect the fundamental mode and period of vibration appreciably but that its effects are small on the second mode and period and negligible on the higher harmonics. As an illustration consider a flexural two-mass system. Let the flexibilities be concentrated at the base and at the first mass, the masses be equal to each other, the flexibilities also be equal to each other, and the masses be equally spaced. If we introduce a spring at the foundation to simulate rocking, with the same flexibility as the spring elements at the joints, the fundamental period will increase 36 percent while the second natural period increases 8 percent. Indeed, it follows from the orthogonality of natural modes that if the fundamental mode of vibration of a building is a straight line, there can be no base overturning moment in any of the higher modes (Bielak, 1969) and hence these are not affected by the possibility of interaction with rocking motion of the base. Since the fundamental mode is almost always approximately straight, interaction can rarely have an important effect on the higher modes and periods. [This conclusion is apparently contradicted in papers by Parmelee (1967 and 1969), but the corresponding solutions fail to take into account vibration in other natural modes when analyzing the response in any given mode.]

Now, the fundamental period of the soil-structure system is not smaller than that of an infinitely rigid structure resting on the same soil and having the same masses and geometry as the structure in question. Because the second natural period in buildings is of the order of one half to one third of the fundamental (except when soil-foundation interaction is such as to make the fundamental mode much more significant than the harmonics), we are not interested in an accurate evaluation of the phenomenon of foundation compliance much beyond a frequency equal to about twice the first resonant frequency associated with a rigid block resting on soil, and usually not much beyond the first resonant frequency.

From Newmark and Rosenblueth (1971)

In principle, once the solutions were available for instantaneous pulses or steady-state disturbances, integral transforms would solve every problem of interest. The approach would be impractical, however, and would preclude analyzing nonlinear structures. A more attractive even if only approximate treatment replaces the soil with a virtual mass fixed to the foundation, a massless spring, and a massless dashpot in parallel with the spring. The three parameters must be defined for every degree of freedom and may be so placed as to include correctly coupling between the various degrees. In this manner we have no difficulty in applying the standard methods of analysis for multidegree systems to a new system, whose degrees of freedom include those of the structure proper plus six of the foundation, and we may even deal with nonlinear structural behavior.

A rigorous treatment of this sort would require having two of the parameters in every degree of freedom vary with the frequency of vibration because we would have to adjust for two quantities at each frequency: the amplitude of response and its phase shift with respect to a harmonic excitation. If, as proposed, we take the parameters as independent of frequency, we must fulfill certain conditions. In a simple system, as we saw in Chapter 1, the response at low frequency is essentially sensitive to the spring constant. Hence, if our model is to cover a range of low frequencies, the spring stiffnesses must coincide with the values derived from static loading. (In a real soil this is to be interpreted as a rapid, quasistatic loading in which consolidation and creep are not given the opportunity to occur to an appreciable extent.) In the ranges of the resonant frequencies the dynamic magnifications of responses are sensitive only to the percentages of damping; these ranges will fix the dashpot constants. For high frequencies, only the masses are significant. Lysmer points out that, as the frequency of excitation tends to infinity, the wavelengths of the disturbances emanating from the foundation tend to zero; hence the virtual masses must also tend to zero, and if we wish our solution to hold for all possible frequencies, we must take the virtual mass in every natural mode as zero.

Reasoning along these lines and adjusting to the exact solution we mentioned earlier for the vertical oscillations of a circular plate, so as to minimize the error in the amplitude of the responses to a harmonic force applied at the center of the plate, Lysmer proposes the following parameters for this degree of freedom

$$K = \frac{4}{1-\nu}\mu r \tag{3.58}$$

$$C = \frac{0.85Kr}{v_s} \tag{3.59}$$

where K is the spring constant, ν and μ are Poisson's ratio and modulus of rigidity, r is the radius of the plate, C the dashpot constant, and v_s the velocity of shear waves in the soil ($\sqrt{\mu/\rho}$). The spring constant in Eq. 3.58 is that for static loading. The dashpot constant in Eq. 3.59 is chosen such that in the entire range of possible Poisson ratios, $0 \le \nu \le 0.5$ and forcing frequencies $0 \le \omega \le \infty$, the computed amplitude of the response does not differ from the exact

solution by more than about 30 percent; in the range of greatest interest, it differs by less than 20 percent. The phase change between the force and the response is automatically approximated also in a rough manner.

The model described is the simplest that replaces the soil with a small number of elements having parameters independent of the frequency and yet gives the correct order of magnitude of the responses. But the condition that the model be acceptable for very high frequencies causes a loss of accuracy in the lower frequency range, and this loss is unnecessary in the analysis of responses to earthquakes. By introducing a virtual mass of soil we have one additional parameter that permits a better adjustment over a limited range of frequencies. When we do this, the computed responses will be smaller than in the absence of the virtual mass if we retain the dashpot constant as given by Eq. 3.59. Hence we must compensate by adopting a smaller dashpot constant. The following constants (Nieto, Rosenblueth, and Rascón, 1965) give response amplitudes that check with the "exact" solution [which assumes the same contact stress distribution as under static loading (Sezawa, 1927a; Reissner, 1936; Arnold, Bycroft, and Warburton, 1955; Richart, 1962)] within a few percent at least up to forcing frequencies equal to twice that of resonance: K as in Eq. 3.58, the virtual mass equal to that of a cylindrical body of soil having the same base as the plate and a height h equal to 0.27 times the square root of the base area A (Fig. 3.16), and a dashpot constant

$$C = \frac{0.64Kr}{v_s}$$

The latter can be put in the more convenient form

$$C = \frac{1.35Kh}{v_s} \tag{3.60}$$

A comparison with the "exact" solution is shown in Fig. 3.17.

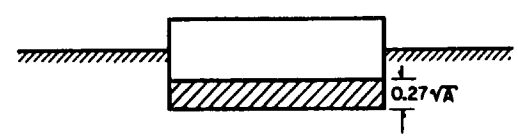


Figure 3.16. Virtual mass in vertical oscillations of circular plate.

Using a similar type of adjustment together with available information on spring constants and solutions for circular and rectangular rigid plates, Table 3.1 has been constructed (Nieto, Rosenblueth, and Rascón, 1965; Barkan, 1962). It is a partial list of stiffnesses, virtual masses, and dashpot constants for various degrees of freedom of plates of these shapes.

The positions of the springs and dashpots are important to reflect the proper coupling between various degrees of freedom. Owing to symmetry, in circular and rectangular plates with uniformly distributed mass, there is coupling only between the rocking and transverse-displacement degrees. In plates of other

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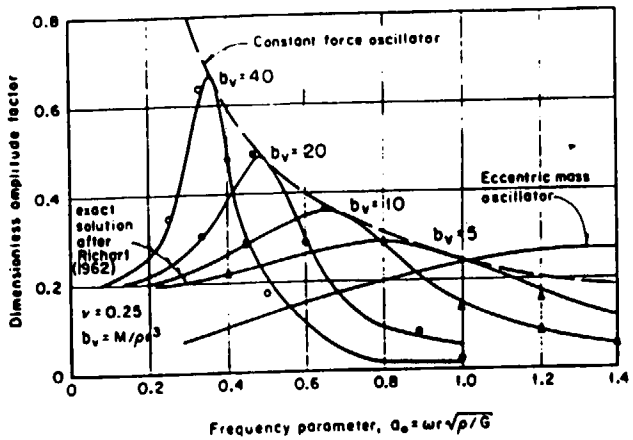

Figure 3.17. Comparison of responses of circular plates to vertical excitation.

TABLE 3.1. STIFFNESSES, VIRTUAL MASSES, AND DASHPOT CONSTANTS

Degree of freedom	Height of soil prism	Dashpot constant	Stiffness: Circular base	Stiffness: Rectangular base*
Vertical	$0.27\sqrt{A}$	$5.42\sqrt{K\rho h^3}$	$4\mu r/(1-\nu)$	$E\sqrt{A}c_z/(1-\nu^2)$
Horizontal	$0.05\sqrt{A}$	$41.1\sqrt{K\rho h^3}$	$5.8\pi\mu r(1-\nu^2)/(2-\nu)^2$	$E\sqrt{A}k_T/(1-\nu^2)$
Rocking†	$0.35\sqrt{A}$	$0.97\sqrt{K\rho h^3}$	$2.7\mu r^3(\nu=0)$	$EIk_\phi/\sqrt{A}(1-\nu^2)$
Torsion	$0.25\sqrt{A}$	$3.76\sqrt{K\rho h^3}$	$16\mu r^3/3$	$1.5EJk_T/\sqrt{A}(1-\nu^2)$

Aspect ratio	c_z	k_T: $\nu = 0.1$	0.2	0.3	0.4	0.5	k_ϕ‡
1	1.06	1.00	0.938	0.868	0.792	0.704	1.984
1.5	1.07	1.01	0.942	0.864	0.770	0.692	2.254
2.0	1.09	1.02	0.945	0.870	0.784	0.686	2.510
3.0	1.13	1.05	0.975	0.906	0.806	0.700	2.955
5.0	1.22	1.15	1.050	0.950	0.850	0.732	3.700
10.0	1.41	1.25	1.160	1.040	0.940	0.940	4.981

*Coefficients c_z, k_T, and k_ϕ tabulated in subsequent columns.
†Take moments of inertia with respect to axis at soil-foundation interface.
‡Rocking parallel to long side.

shapes or with other mass distributions, there may be coupling with other degrees of freedom or among all six of them. The same situation sometimes stems from asymmetric distribution of stiffnesses in the superstructure.

Comparisons (Nieto, Rosenblueth, and Rascón, 1965) are shown in Figs. 3.18–3.20 between the response amplitudes obtained from the models described in Table 3.1 and the "exact" solutions for steady-state harmonic excitation (Sung, 1953; Richart, 1962). We notice that the agreement for horizontal vibrations is comparable to that for vertical oscillations in Fig. 3.17. Agreement is adequate for torsional and rocking motion throughout most of the range of excitation frequencies covered in the figures, except in the neighborhood of the

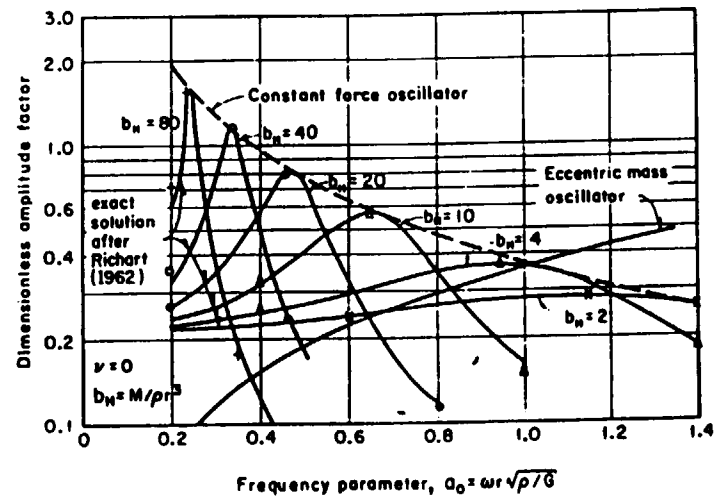

Figure 3.18. Comparison of responses of circular plates to horizontal excitation.

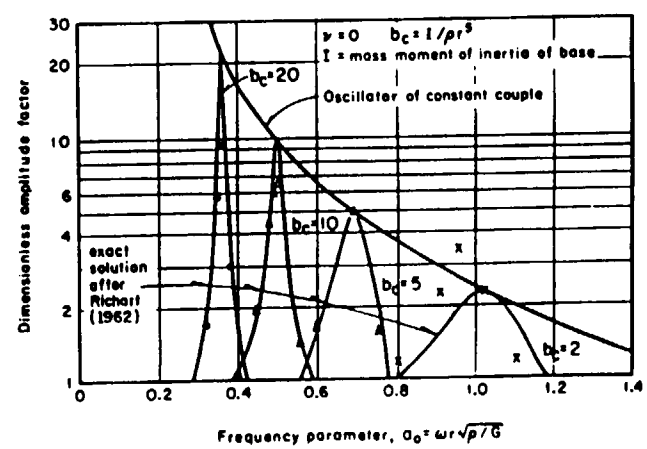

Figure 3.19. Comparison of responses of circular plates to rocking excitation.

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From Newmark and Rosenblueth (1971)

From Newmark and Rosenblueth (1971)

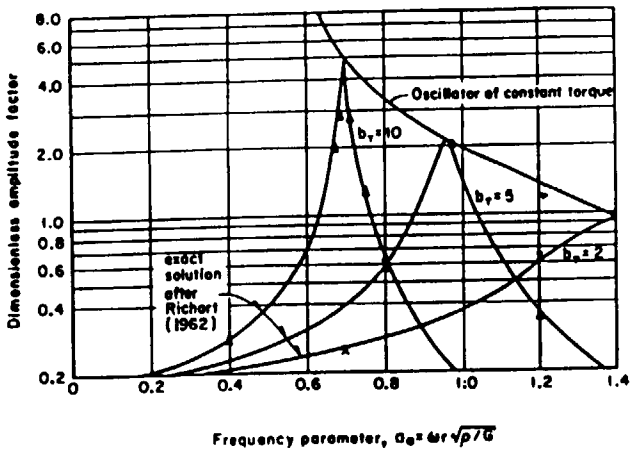

Figure 3.20. Comparison of responses of circular plates to torsional excitation.

resonant values when these are very small or very large. The discrepancy is important in these short intervals and should not be disregarded in the analysis of machine foundations or in the calculation of responses to earthquakes having well-defined, prevailing frequencies when these frequencies lie close to the rocking or torsional natural frequencies of the machine foundations. For most purposes in earthquake-resistant design, however, these discrepancies may well be overlooked because they affect only the contributions of short intervals in the entire range of significant frequencies of the motion.

Matters would improve if we varied one or two parameters in the models as a function of frequency. No doubt this should be done in the cases of narrow-band excitation that we quoted in the foregoing paragraph. Apparently, we could always proceed in this manner when using modal analysis. By trial and error or iteration we could find the values of parameters giving the best adjustment in the neighborhood of the natural frequencies of the soil-structure system and recompute these frequencies in terms of those parameters. But modal analysis does not apply strictly when we include soil-structure interaction because the combined system lacks classical natural modes. Hence, if we resort to modal analysis at all, great refinements are unwarranted. And if we wish to attain great accuracy there will be little advantage in adopting the simplified models proposed in this article, and we shall do well to return to the "exact" solutions. These allow us to compute the *transfer functions* of the system (its responses to instantaneous pulses), from which we can find the effects of various types of earthquakes on systems of linear behavior, as will be done in Chapters 9 and 10.

Ordinarily, analysis of pronouncedly nonlinear systems with soil-structure interaction will be formulated validly in terms of the models that Table 3.1 proposes, since nonlinearity will ensure that a vast range of frequencies will enter into play.

For other shapes of foundation the constants K for vertical oscillations are obtained readily by making reasonable assumptions about the contact pressure distribution, using charts (Newmark, 1947) to find the settlement of various points as though the foundation were flexible and to compute the foundation's average contact pressure and average settlement. Ordinarily the ratio of the two will give a satisfactory approximation to K. For example, under a circular plate subjected to a central vertical load the obviously wrong assumption that the contact pressure is uniform gives an error of only 5 percent (Timoshenko and Goodier, 1951). The spring constants that correspond to rocking oscillations can be obtained in similar fashion, while those for torsional and horizontal motions require integration of Cerrutti's equation for displacements at the ground surface. Once K has been obtained, the data in Table 3.1 can be used as a guide to estimate the dashpot constant and the virtual mass of soil. Studies are needed to allow reasonable estimates to be made of these parameters for deep, compensated foundations and for foundations on piles.

Numerical solutions have been obtained using high-speed computers for specific two-dimensional cases using lumped-parameter models and finite elements (Parmelee, 1969; Wilson, 1969). Some solutions correspond to surface foundations on a halfspace; others correspond to a foundation on a soil layer that in turn rests on a bedrock halfspace (Whitman, 1969), to partially compensated foundations (J. K. Minami and Sakurai, 1969), to a circular pier in a layered halfspace (Tajimi, 1969), and to foundations on point bearing piles (Penzien, Scheffey, and Parmelee, 1964; Kobori, Minai, and Inoue, 1969). Essentially the same remarks apply as the ones made on the problem of multiple wave reflection (Section 3.5) concerning "radiation damping" and the correct specification of boundary conditions where the soil or rock is assumed to terminate.

PROBLEMS[7]

3.1*. Compute the fundamental period of a cylindrical chimney stack of steel with circular cross section 6 ft in diameter, whose height is 90 ft, and whose thickness is $\frac{1}{4}$ in. (Fig. 3.21). Neglect shear deformations, rotary inertia, damping, gravity effects, and soil-foundation interaction.

Ans. 0.406 sec.

3.2. The unit weight and modulus of elasticity of a soil formation are 2.0 ton/m³ and 2×10^5 ton/m². Compute the velocities of dilatational, rotational, and Rayleigh waves in this material. Assume that Poisson's relation applies.

Ans. $v_p = 1085$ m/sec, $v_s = 626$ m/sec, $v_r = 576$ m/sec.

3.3*. A 30-m layer of the material specified in Problem 3.2 rests on what may be idealized as a semiinfinite rock formation having a unit weight of 2.8 ton/m³, a modulus of elasticity of 3×10^6 ton/m², and a Poisson's ratio of 0.25. Compute the

[7] Solution of problems marked with an asterisk is lengthy.