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DYNAMIC MODULI AND DAMPING RATIOS FOR COHESIVE SOILS

by

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Joseph I. Sun, R. Golesorkhi and H. Bolton Seed

1. INTRODUCTION

The forms of the relationships expressing shear modulus and damping ratio as a function of shear strain play an important role in determining the results of ground response analyses. Much information on this aspect of dynamic soil property determination has been presented since the early 1970's. It is the purpose of this report to summarize available data on the dynamic shear moduli and damping factors for cohesive soils under cyclic loading conditions and to present the results in a form which will provide a useful guide in the selection of soil characteristics for analysis purposes. Emphasis will be placed mostly on clays, though limited data for offshore samples and mudstone are also included.

2. GROUND RESPONSE ANALYSIS FOR CLAY SITES

It has long been recognized that local soil conditions can significantly affect the ground response when seismic waves propagate upward through a soil profile. This is especially true for soft clay sites, and ground response analysis techniques have been shown to provide a useful approach to such problems (e.g. Seed and Idriss, 1969, and Seed et al., 1977). More recently in the 1985 Mexico City earthquake, the soft Mexico City clay greatly amplified the ground motions and caused severe damage in certain parts of the city (Rosenblueth, 1985). A simple one-dimensional ground response analysis model, which took into account the dynamic behavior of the Mexico City clay as shown in Fig. 1, has been shown to provide an effective means for predicting the main engineering features of the seismic ground motions in Mexico City (Seed et al., 1987).

Successful application of such procedures for determining ground response is essentially dependent on the incorporation of representative dynamic soil properties in the analyses. Larkin and Donovan (1979) and Martin et al. (1979) have shown that for strain levels which develop under strong shaking conditions, the most important aspects of soil modelling are the forms of the relationships between shear modulus and shear strain.

3. IMPROVEMENTS ON TESTING DEVICES

Since the first comprehensive reports on dynamic soil properties (Seed and Idriss, 1970; Hardin and Drnevich, 1972a and 1972b), much progress has been made in improving dynamic testing apparatus so that the dynamic properties of a specific soil can be measured over a wide strain range using a single piece of equipment.

Thus for example, Hara and Kiyota (1977) introduced the Kjellman-type simple shear device which is capable of testing soils over a strain range from 10^{-3} to 1 percent strain, Isenhower (1979) combined the principles of a resonant column device with a cyclic torsional simple shear device, Kokusho (1980) modified the cyclic triaxial cell so that tests with low levels of excitation can be performed with a minimum of mechanical friction, and Umehara et al. (1982) introduced the resonant-cyclic triaxial testing device.

The primary benefit of such equipment is that it can be used to reduce the number of samples required to determine a modulus attenuation curve for a specific material, thereby eliminating some of the

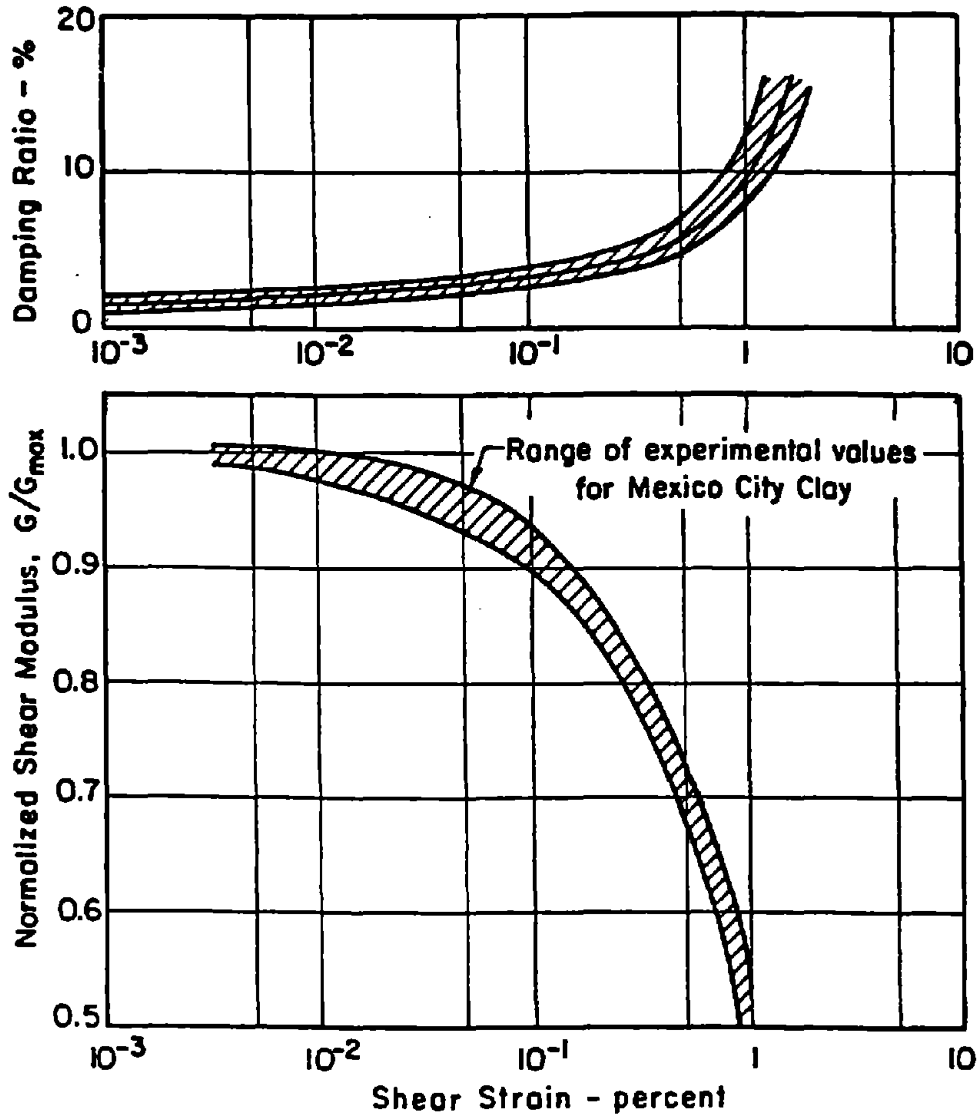

Fig. 1 STRAIN DEPENDENT SHEAR MODULI AND DAMPING RATIOS FOR MEXICO CITY CLAY (after Leon et al., 1974 and Romo and Jaime, 1986)

uncertainties introduced by variations in properties from one sample to another.

4. MODULUS REDUCTION RELATIONSHIPS FOR COHESIVE MATERIALS

Unlike the modulus reduction curves reported for a variety of sands which show a relatively small variation from one sand to another (Fig. 2, after Iwasaki, 1978a), the modulus reduction curves for clays show a much larger scatter, as may be seen from Fig. 3 (after Anderson and Richart, 1976). It is apparent that the modulus reduction curves for clays are highly variable and the rate of modulus reduction with shear strain, which is normally shown on a plot of G/G_{max} vs. strain, where G_{max} is the low strain modulus for a shear strain of the order of 10^{-4} percent, seems to be related to the characteristics of each individual clay.

Modulus reduction relationships for clays have been under investigation in many parts of the world since the early 1970's and test data for undisturbed samples for about 70 cohesive soils, mostly normally consolidated to slightly over consolidated, from the United States, Japan, Canada and New Zealand are summarized in Table 1.

Judging from the wide divergence of modulus reduction relationships reported for clays, it will often be most appropriate to determine the modulus attenuation curve for a clay on a site specific basis. However, for preliminary investigation purposes or when no other information is available, it may well be desirable to have some guidelines on the form of the normalized modulus reduction curve for a clay in relation to its physical properties and other important factors. Such results will be presented in this report.

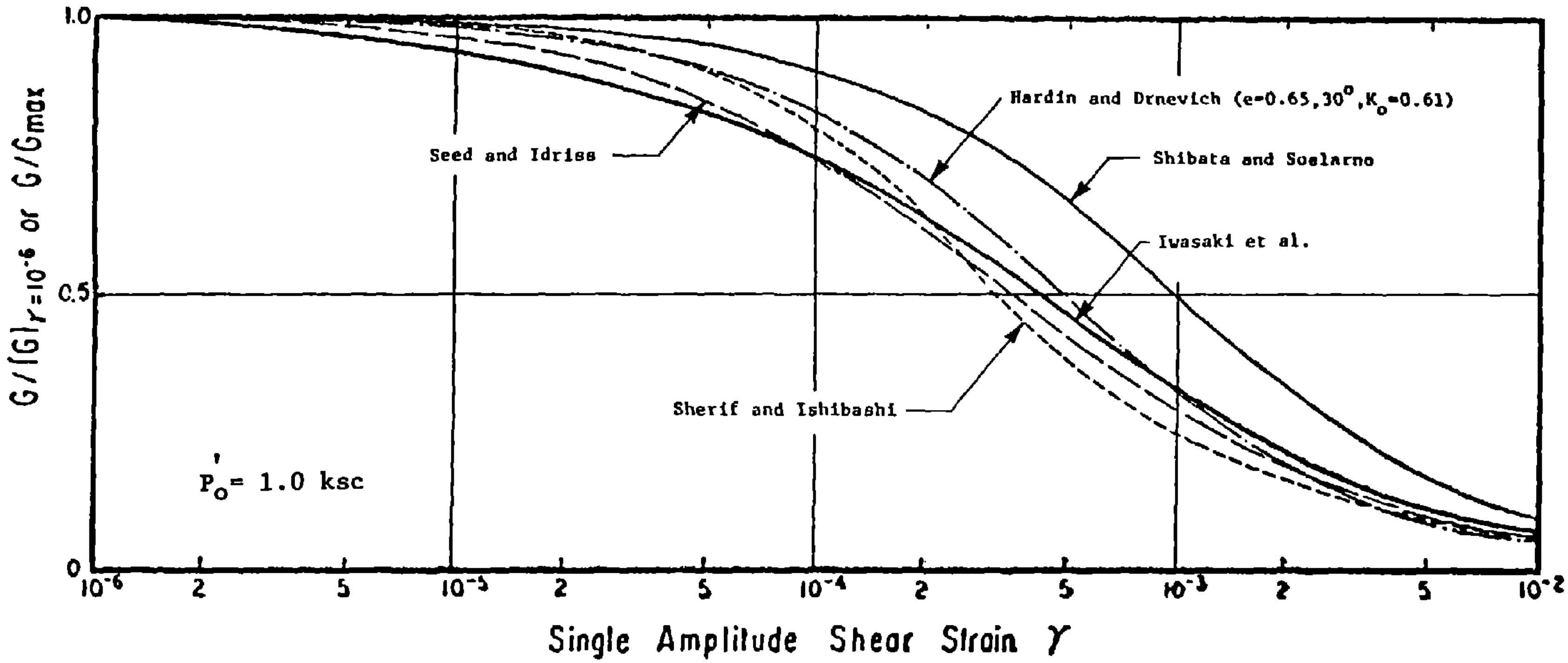

Fig. 2 COMPARISON OF NORMALIZED MODULUS REDUCTION RELATIONSHIPS FOR SANDS (after Iwasaki et al., 1978a)

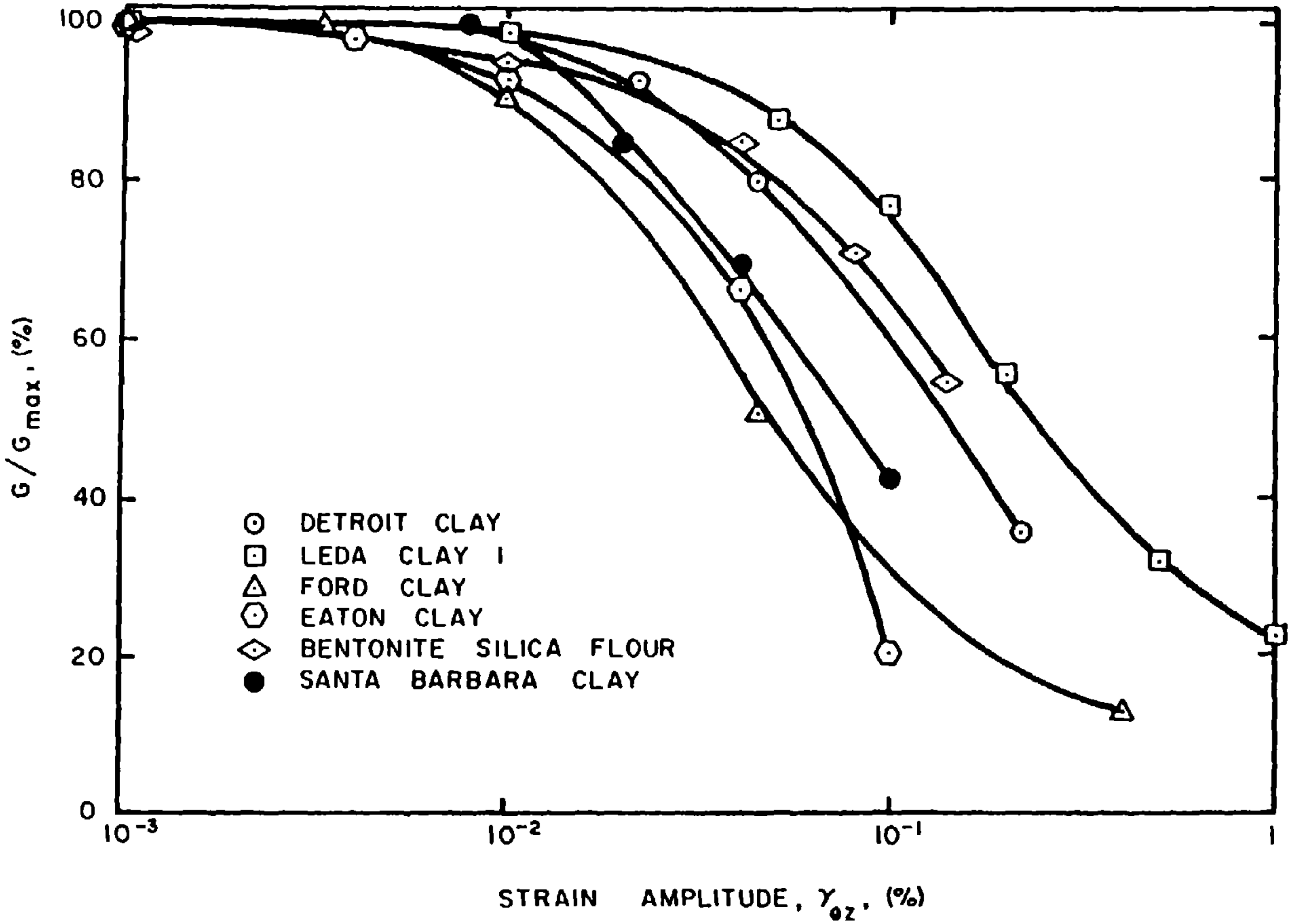

Fig. 3 COMPARISON OF NORMALIZED MODULUS REDUCTION RELATIONSHIPS FOR CLAYS (after Anderson and Richart, 1976)

Table 1 PHYSICAL PROPERTIES AND NORMALIZED STRAIN DEPENDENT MODULI FOR CLAYS

Identification	LL %	PI %	void ratio	water content %	Shear Strain - percent 0.001	0.003	0.01	0.03	0.1	0.3	1.0	3.0	References
Windsor Clay	52	30	1.36	51	1.00	.996	.950	.815	.492	.252	-	-	Kim & Novak (1981)
Wallaceburg Clay	42	25	1.05	38	1.00	.990	.935	.750	.425	.227		-	
Catham Clay-Silt	29	14	0.75	28	1.00	.975	.860	.635	.330	-	-	-	
Hamilton Clay-Silt	25	12	0.48	17	1.00	.970	.854	.579	.283	-	-	-	
Sarina Clay-Silt	30	14	0.59	23	1.00	.989	.915	.677	.320	-	-	-	
Iona Silty Clay	30	14	0.62	20	1.00	.999	.950	.720	.360	-	-	-	
Port Stanley Clay	35	20	0.58	23	1.00	.995	.907	.653	.347	-	-	-	
Detroit Clay	-	30	1.30	46	1.00	.999	.986	.863	.604	.355 @ 0.22%		-	Anderson and Richart (1976)
FordClay	-	19	0.82	30	1.00	.990	.900	.650	.310	-	-	-	
Eaton Clay	-	20	0.72	27	1.00	.990	.930	.730	.230	.150	-	-	
Leda Clay	-	44	2.19	79	1.00	.999	.990	.920	.770	.460	.220	-	
Santa Barbara Clay	-	44	2.28	80	1.00	.999	.984	.750	.430	-	-	-	
Sample #1	-	51	2.09	71	1.00	.995	.972	.853	.565	.331	.160	-	Hara and Kiyota (1977)
Sample #2	-	33	1.17	43	1.00	.990	.925	.761	.500	.257	.129	-	
Sample #3	-	52	1.31	53	1.00	.990	.925	.745	.483	.269	.160	-	
Sample #4	-	65	1.43	57	1.00	.962	.889	.763	.565	.335	.118	-	
Sample #5	-	79	1.54	62	1.00	.995	.966	.833	.608	.355	.174	-	

Table 1 PHYSICAL PROPERTIES AND NORMALIZED STRAIN DEPENDENT MODULI FOR CLAYS (cont'd)

Identification		LL %	PI %	void ratio	water content %	Shear Strain - percent								Reference
						0.001	0.003	0.01	0.03	0.1	0.3	1.0	3.0	
TEGANUMA CLAY	S-9-1	-	NP	0.93	-	1.00	.952	.857	.675	.396	.175	-	-	Kokusho et al. (1982)
TEGANUMA CLAY	S-9-2	-	14	1.29	-	1.00	.960	.897	.766	.532	.278	.115	.048	
TEGANUMA CLAY	S-1-3	-	38	1.84	-	1.00	.976	.937	.839	.615	.385	.183	.083	
TEGANUMA CLAY	S-8-1	-	41	1.99	-	1.00	.976	.937	.857	.674	.444	.250	.119	
Band PI: 50 to 96	Lower Bd.	-	(50)	(2.69)	-	1.00	.976	.920	.861	.706	.484	.250	.095	
Band PI: 50 to 96	Average	-	(75)	(3.11)	-	1.00	.977	.940	.873	.742	.533	.302	.139	
Band PI: 50 to 96	Upper Bd.	-	(96)	(3.55)	-	1.00	.964	.920	.837	.710	.538	.329	.167	
Plastic Clay - CH		70	41	(1.39)	53	1.00	.970	.960	.870	.640	.390	0.25 @ 0.6%		Koutsoftas & Fischer (1980)
Silty Clay - CL		33	16	(0.87)	32	1.00	.970	.880	.700	.440	.220	0.12 @ 0.6%		
Clay		-	58	-	-	1.00	.977	.944	.788	.500	.235	.133	-	Nishigaki(1971)
Clay		PI= 15 - 30		-	-	1.00	.978	.956	.889	.661	.344	.144	.067	Iwasaki (1978)
Clay		-	-	-	-	1.00	.954	.894	.800	.623	.423	.256	-	Yokota (1980)
Gothenburg Clay		(80)	(35)	(2.03)	(90)	1.00	.988	.966	.841	.600	.43 @ 0.21%		-	Andreasson(1981)
Offshore Silty Clay		(45)	(21)	(0.89)	(30)	1.00	.988	.890	.721	.465	.255	-	-	Stokoe et al. (1980)
Gulf of Alaska		-	-	-	-	1.00	.960	.890	.680	.300	.130	.070	.040	Idriss (1976)
Japanese Clay		-	-	-	-	1.00	.977	.887	.713	.459	.226	.127	-	Ohsaki

Table 1 PHYSICAL PROPERTIES AND NORMALIZED STRAIN DEPENDENT MODULI FOR CLAYS (cont'd)

Identification	LL %	PI %	void ratio	water content %	Shear Strain - percent 0.001	0.003	0.01	0.03	0.1	0.3	1.0	3.0	Reference
Clay Sample #1	-	51	2.09	71	1.00	.999	.923	.775	.473	.308	.154	-	Ohsaki, Hara and Kiyota (1978)
Clay Sample #2	-	33	1.17	43	1.00	.999	.923	.757	.500	.290	.154	-	
Clay Sample #3	-	52	1.31	53	1.00	.990	.880	.710	.461	.265	.136	-	
Clay Sample #4	-	65	1.43	57	1.00	.999	.947	.787	.539	.331	.189	-	
Clay Sample #5	-	79	1.54	62	1.00	.999	.911	.788	.556	.343	.189	-	
Clay Sample #6	-	75	1.63	65	1.00	.999	.927	.742	.450	.254	.154	-	
Sandy-Silt #7	-	12	1.55	59	1.00	.952	.810	.595	.327	.149	.077	-	
Clayey-Silt #8	-	18	0.93	36	1.00	.971	.848	.660	.393	.220	.087	-	
Clayey-Silt #9	-	20	1.27	48	1.00	.976	.867	.688	.417	.202	.087	-	
Clayey-Silt #10	-	28	1.15	44	1.00	.999	.935	.769	.473	.235	.107	-	
Clayey-Silt #11	-	40	1.36	53	1.00	.982	.905	.769	.519	.304	.137	-	
Sample #1	-	NP	-	-	.962	.897	.746	.540	.254	.074	-	-	Zen and Hamada (1978)
Sample #2	-	9	-	-	.963	.925	.838	.676	.400	.140	.038	-	
Sample #3	-	16	-	-	.989	.978	.876	.704	.464	.211	.054	-	
Sample #4	-	25	-	-	.989	.978	.903	.741	.508	.260	.059	-	
Sample #5 to #7	PI=38 to 52		-	-	1.00	.962	.889	.763	.565	.335	.118	-	

Table 1 PHYSICAL PROPERTIES AND NORMALIZED STRAIN DEPENDENT MODULI FOR CLAYS (cont'd)

Identification		LL %	PI %	void ratio	water content %	Shear Strain - percent 0.001	0.003	0.01	0.03	0.1	0.3	1.0	3.0	Reference
Mean Curve-Bay Mud		e = 0.52 to 2.54 (ave=1.62)				1.00	.950	.843	.640	.390	.30 @ 0.15%		-	Stokoe and Lodde (1978)
BAY MUD	Conf.P. 10psi	-	NP	0.63	23	.970	.840	.575	.335	.165	-	-	-	
	20psi	-	NP	0.63	23	.980	.910	.660	.395	.200	-	-	-	
	40psi	-	NP	0.63	23	1.00	.925	.740	.470	.260	-	-	-	
	80psi	-	NP	0.63	62	1.00	.940	.805	.550	.340	-	-	-	
Bay Mud - e < 0.8		-	-	(0.61)	-	.990	.953	.841	.648	.332	.212 @ .15%		-	Lodde (1982)
Bay Mud - e > 1.8		-	-	(2.21)	-	1.00	.976	.911	.800	.537	.419 @ .15%		-	
Bay Mud - H. AFB		-	(40)	(2.48)	(90)	1.00	.992	.975	.889	.664	.369	-	-	Isenhower(1981)
ERTEC	Bay Mud -10'	-	61	3.43	120	1.00	.999	.989	.925	.787	.597	.381 @ 0.7%		ERTEC (1981)
	Bay Mud -20'	-	62	2.98	104	1.00	.994	.918	.797	.633	.451	.286 @ 0.7%		
	Bay Mud -40'	-	52	2.59	89	1.00	.994	.915	.788	.615	.420	.252 @ 0.7%		

Table 1 PHYSICAL PROPERTIES AND NORMALIZED STRAIN DEPENDENT MODULI FOR CLAYS (cont'd)

Identification	LL %	PI %	void ratio	water content %	Shear Strain - percent								Reference
					0.001	0.003	0.01	0.03	0.1	0.3	1.0	3.0	
Silty Clay	79	49	(1.08)	40	1.00	.982	.940	.882	.716	.450	.213	-	Taylor and Parton (1973)
Silty Clay	62	38	(1.24)	46	1.00	.955	.890	.797	.624	.450	.291	-	
Silty Clay	69	38	(1.38)	51	1.00	.903	.769	.558	.353	.208	.099	-	
Osaka Clay D-9	-	82	-	-	1.00	.994	.954	.863	.715	.552	.343	-	Umehara, Zen Higuchi and Ohneda (1982)
Osaka Clay T-28	-	49	-	-	1.00	.994	.965	.848	.556	.304	.135	-	
Japanese Clay	-	76	-	-	.988	.972	.935	.871	.720	.541	.318	-	
Japanese Clay	-	25	-	-	.988	.959	.882	.727	.504	.265	.071	-	
Japanese Clay	-	78	-	-	.961	.938	.888	.817	.659	.459	.226	-	
Japanese Clay	-	76	-	-	.972	.953	.914	.859	.735	.573	.362	-	
Japanese Clay	-	66	-	-	.953	.915	.859	.782	.659	.501	.229	-	
Japanese Clay	-	NP	-	-	.935	.871	.735	.529	.257	.071	.012	-	

Remark: Values in paranthesis were based on average or estimated values.

5. FACTORS INFLUENCING THE MODULUS REDUCTION RELATIONSHIPS OF COHESIVE SOILS

Relationships Between Normalized Modulus Reduction Curves and Plasticity Index

Zen et al. (1978), after extensive testing on laboratory-prepared clay samples with different plasticity indices, first noted the importance of plasticity index on the form of the normalized modulus reduction curves. Fig. 4 shows the results of their studies. It is clear that for clays with higher plasticity indices, the normalized modulus reduction curve gradually moves to the right, showing a slower rate of reduction with increasing shear strain. For the material reported to be non-plastic on the same figure, the modulus reduction relationship resembles that for sands.

Relationships Between Normalized Modulus Reduction Curves and Confining Pressure

The influence of confining pressure on the normalized modulus reduction relationships for sands has long been recognized (Yoshimi et al., 1977; Iwasaki et al., 1978a; Kokusho, 1980). For clays, however, this influence is not so evident. For instance, Zen et al. (1978) showed that the effect of confining pressure decreases as plasticity index increases, as illustrated in Fig. 5, for laboratory-prepared samples. Stokoe and Lodde (1978) clearly showed the influence of confining pressure on the position of the modulus reduction curve for samples of San Francisco Bay mud having a low void ratio ($e \approx 0.6$) as shown in Fig. 6. However, Isenhower (1979) and Isenhower and Stokoe (1981) reported that confining pressure has a very limited influence on the position of the

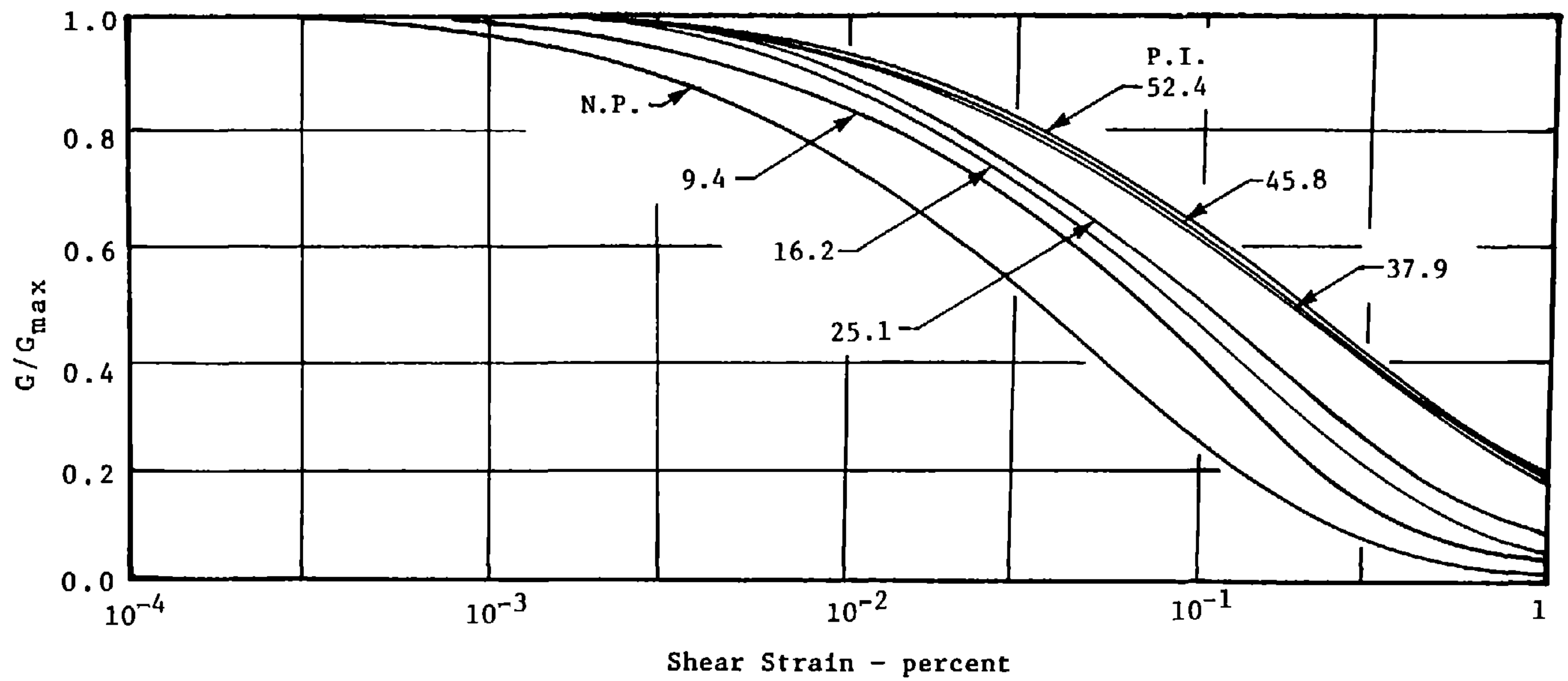

Fig. 4 EFFECT OF PLASTICITY INDEX ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS (modified after Zen et al., 1978)

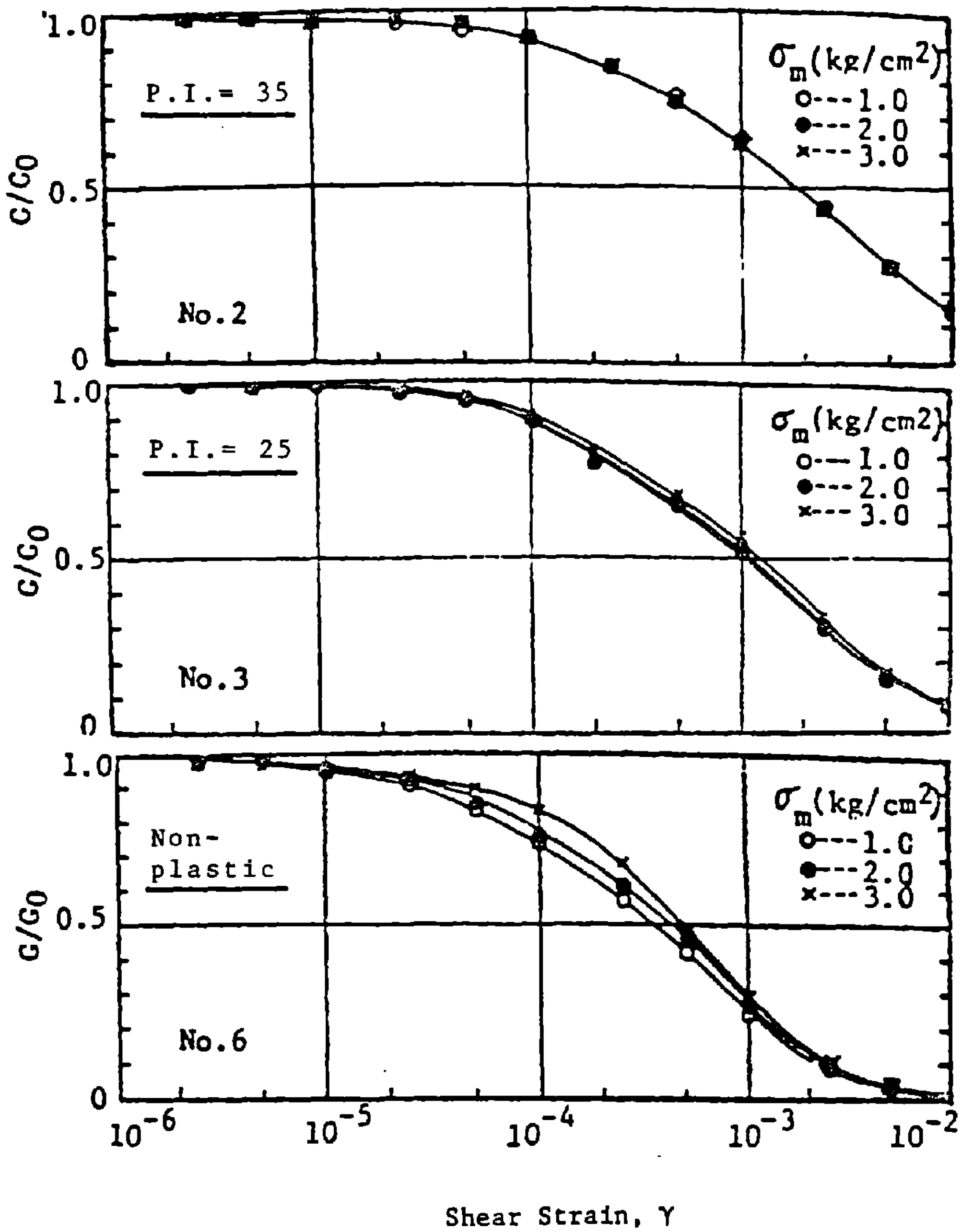

Fig. 5 EFFECT OF MEAN PRINCIPAL STRESS ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH DIFFERENT PLASTICITY INDICES
(after Zen et al., 1978)

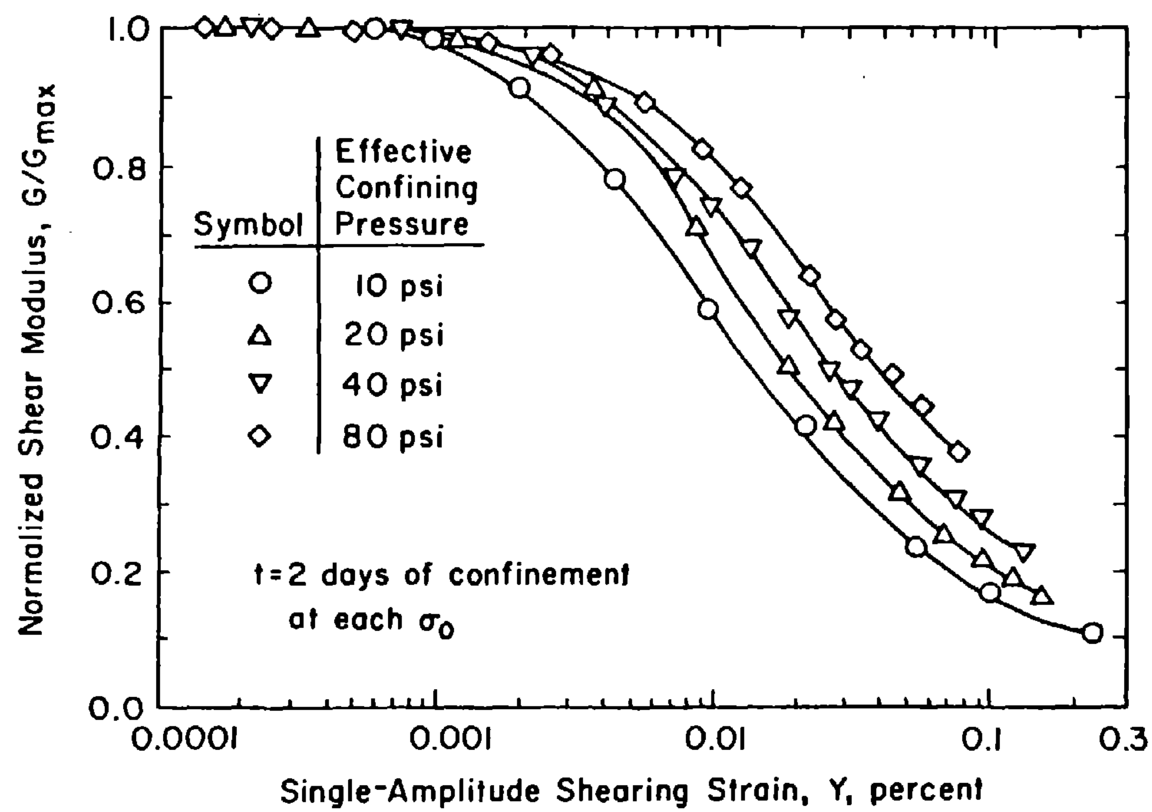

Fig. 6 EFFECT OF CONFINING PRESSURE ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR SAN FRANCISCO BAY MUD WITH A LOW VOID RATIO (after Stokoe and Lodde, 1978)

G/G_{max} versus strain relationship for San Francisco Bay mud, as shown in Fig. 7. The following table lists the approximate influence of confining pressure on the upward shift of G/G_{max} at a strain level of 0.01 percent, for soils with different plasticity indices.

P. I.	Confining Pressure Change	Change in G/G_{max}	Reference
N.P.	10 to 80 psi	≈ 20%	Stokoe and Lodde (1978)
N.P.	1 to 3 ksc	≈ 16%	Zen et al. (1978)
25	50 to 400 kPa	≈ 7%	Kim and Novak (1981)
25	1 to 3 ksc	≈ 5%	Zen et al. (1978)
35	1 to 3 ksc	≈ 0%	Zen et al. (1978)
36	10 to 80 psi	≈ 10%	Stokoe et al. (1980)
30-40	--	≈ 0%	Andreasson (1981)
38-56	7 to 70 psi	≈ 0%	Kokusho et al. (1982)
40	15 to 60 psi	≈ 0%	Isenhower and Stokoe (1981)

It appears from these results that the influence of confining pressure on the normalized modulus reduction curve gradually diminishes as plasticity index increases. The trend is consistent for all data except one offshore silty soil (Stokoe, et al., 1980). In general it appears that the influence of confining pressure is small for clays with plasticity indices exceeding 25 and for shear strains less than 1 percent.

Relationships Between Normalized Modulus Reduction Curves and Void Ratio

Stokoe & Lodde (1978) and Lodde (1982) found that the normalized modulus reduction curves for undisturbed samples taken from the south San Francisco Bay area are dependent on the void ratio of the samples, as shown in Fig. 8. Their data for samples with low ($e < 0.8$) and high

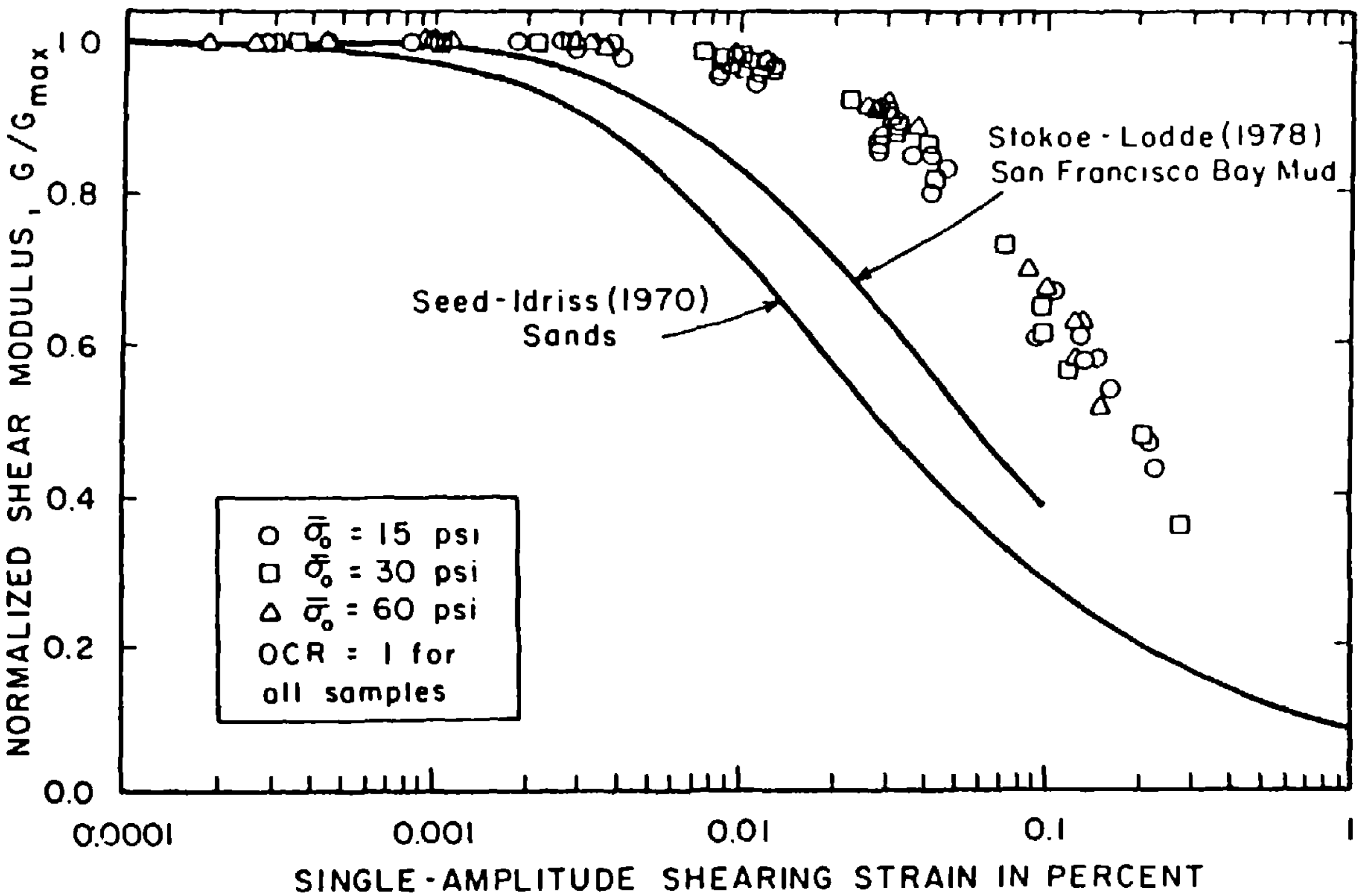

Fig. 7 EFFECT OF CONFINING PRESSURE ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR SAN FRANCISCO BAY MUD WITH HIGH VOID RATIO (after Isenhower and Stokoe, 1981)

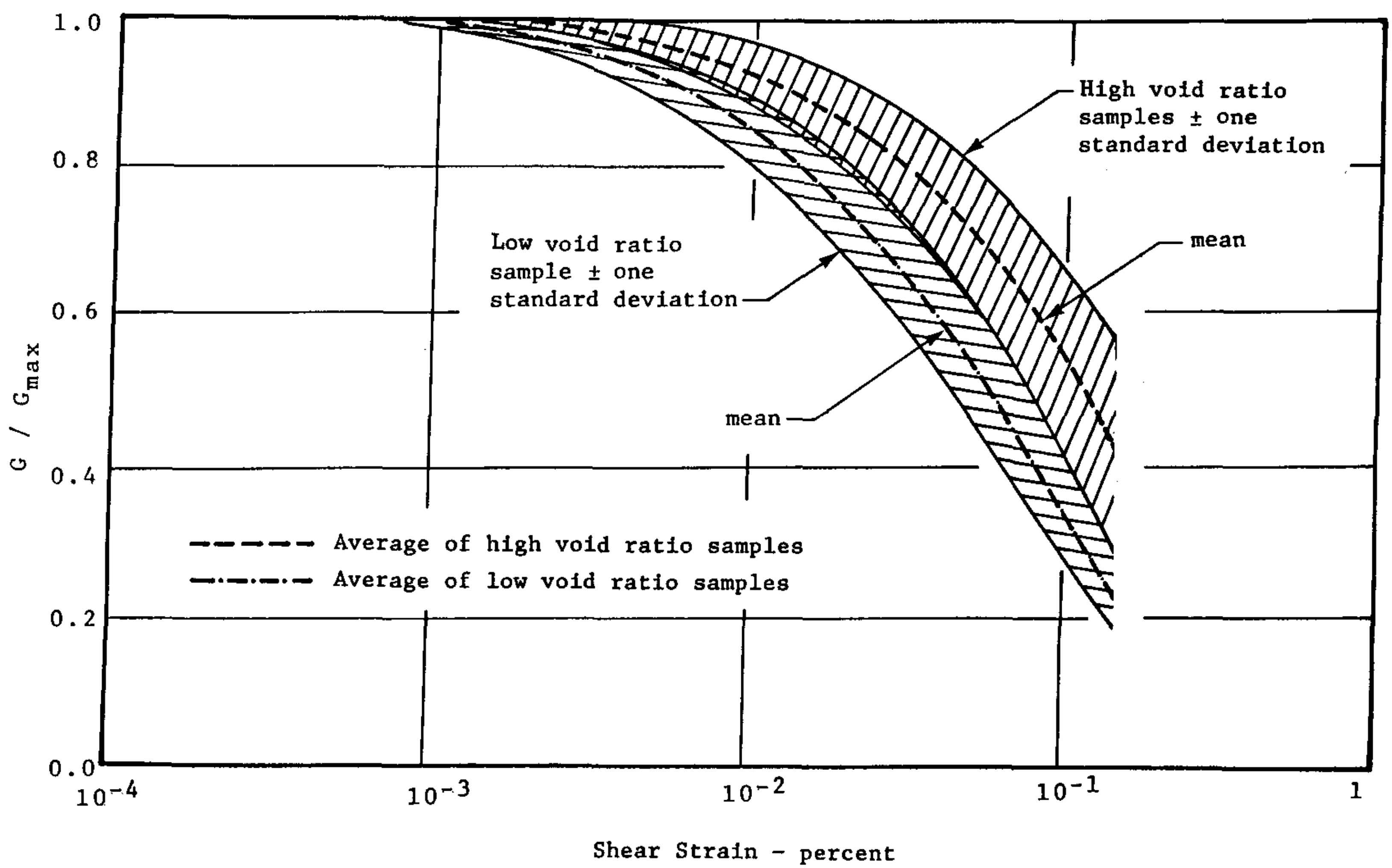

Fig. 8 EFFECT OF VOID RATIO ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR SAN FRANCISCO BAY MUD (reproduced after Lodde, 1982)

(e > 1.8) void ratios are reproduced in Fig. 8. In addition, the compilation of data presented in Table 1 also indicates that the form of the normalized modulus reduction curves for clays depends significantly on the void ratio of the clay, as shown in Fig. 9. However, Isenhower (1979) and Isenhower and Stokoe (1981) showed that for normally consolidated samples of San Francisco Bay mud from Hamilton Air Force Base consolidated to pressures of 15, 30 and 60 psi, there was very little change in the form of the G/G_{max} versus strain relationships (see Fig. 7). Test data presented by Bonaparte and Mitchell (1979) for San Francisco Bay mud samples taken at the same site indicate that an increase in consolidation pressure from 15 to 60 psi would change the void ratio of Bay mud from about 2.5 to 1.5. Thus the data are somewhat conflicting and it appears that the influence of void ratio on the form of the normalized modulus reduction relationship for cohesive soils requires further study.

Relationships Between Normalized Modulus Reduction Curves and Consolidation Stress History

Kokusho et al. (1982) presented data on the normalized modulus reduction curves for samples with plasticity indices greater than 40 tested under different consolidation pressures and at various stages of over-consolidation (Fig. 10). It can be seen that all the curves fall within a reasonably narrow band, despite the different consolidation histories. The differences due to different levels of overconsolidation (OCR = 5 to 15) are not significant, implying that the strain dependent normalized shear modulus is not significantly affected by the consolidation history. This can be more easily seen from Fig. 11 where the effects of three types of consolidation histories are plotted. The effect of long term consolidation (up to about 7 days) and different

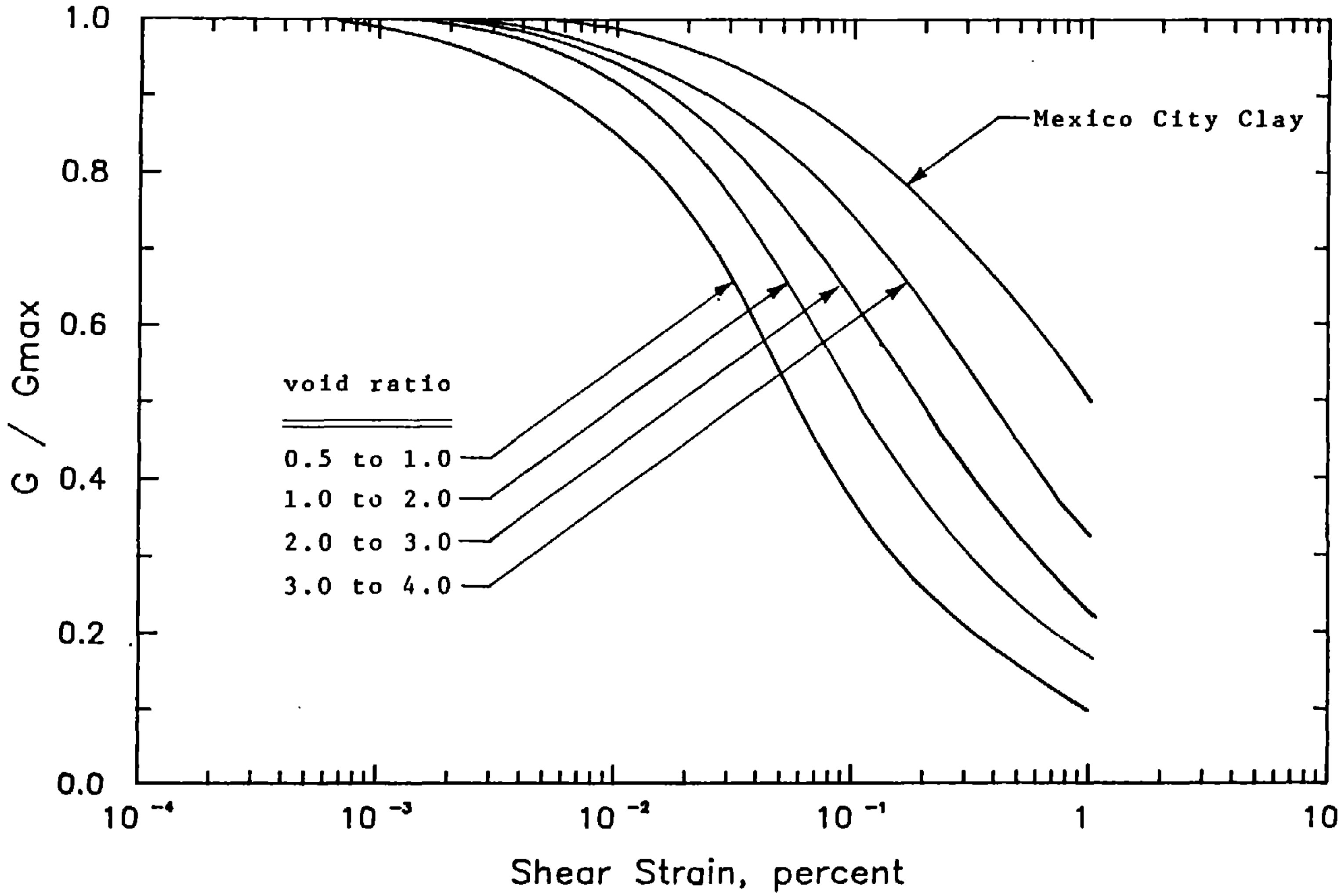

Fig. 9 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH DIFFERENT VOID RATIOS

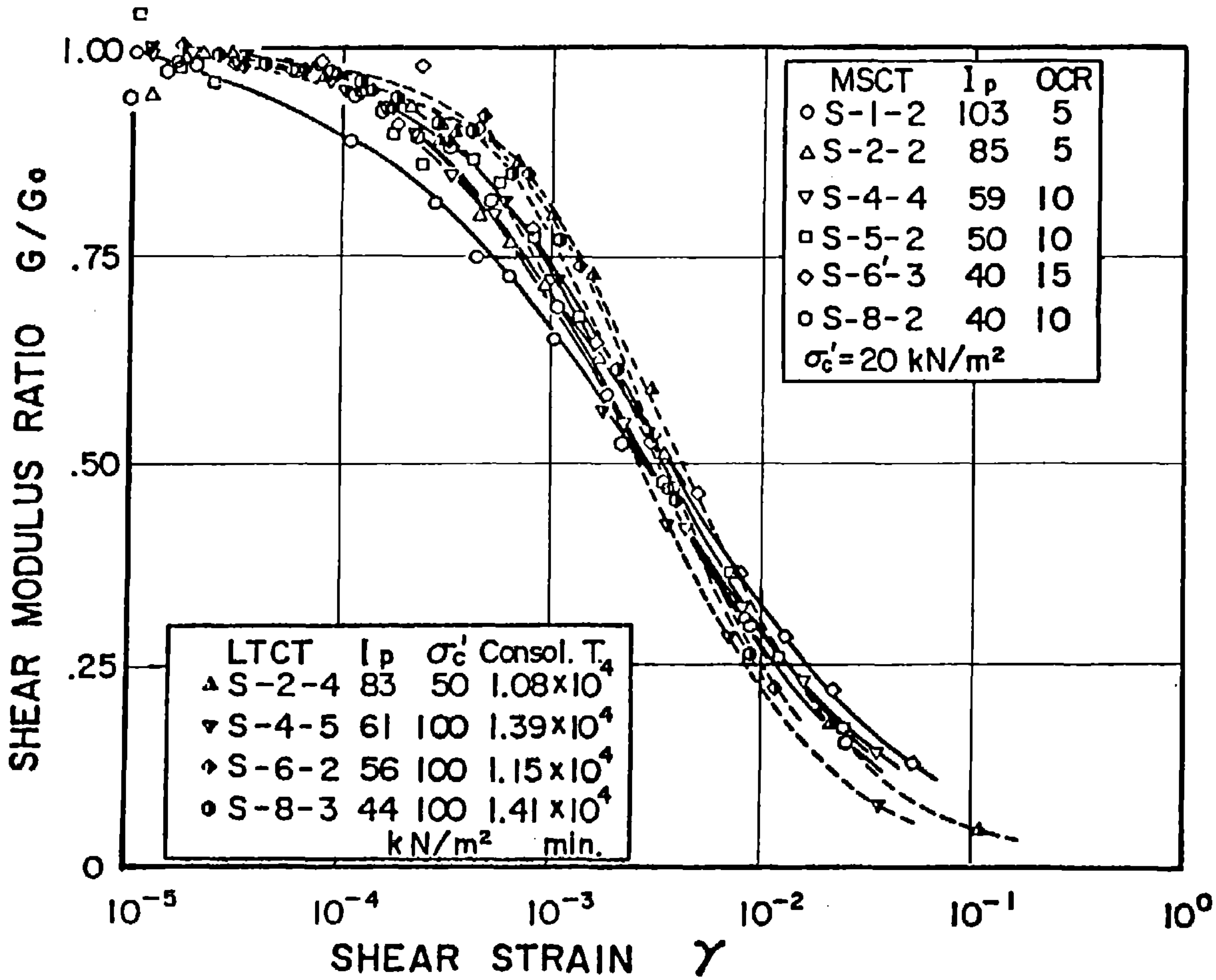

Fig. 10 EFFECT OF OVER-CONSOLIDATION AND LONG TERM CONSOLIDATION ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS (after Kokusho et al., 1982)

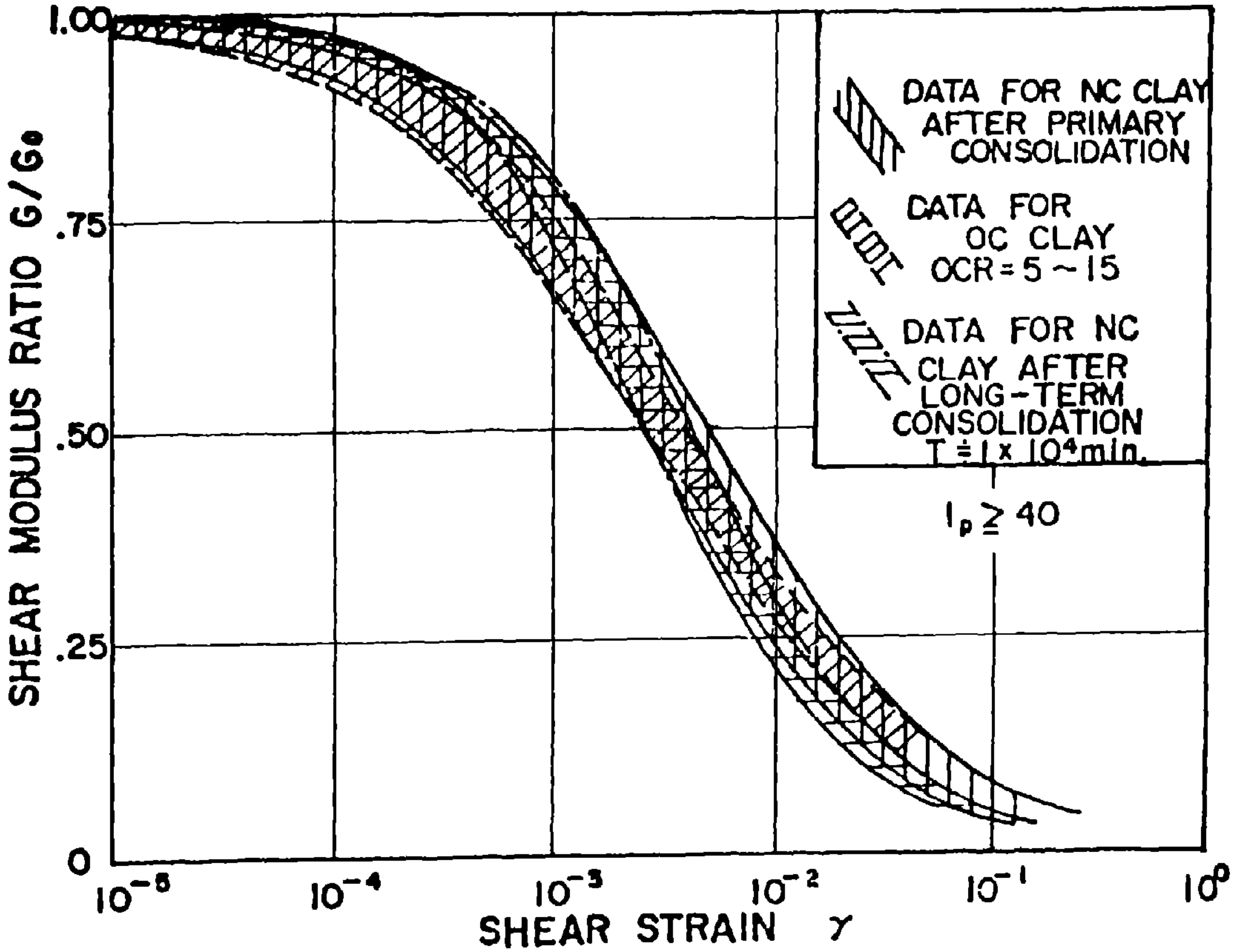

Fig. 11 COMPARISON OF NORMALIZED MODULUS REDUCTION RELATIONSHIP OF CLAYS FOR THREE DIFFERENT CONSOLIDATION STATES (after Kokusho et al., 1982)

degree of over-consolidation is to increase both the small strain and large strain moduli at about the same rate, thus leaving their ratio (G/G_{max}) essentially unchanged. This finding suggests that the current practice of combining the in-situ small strain modulus for a clay measured by a seismic survey with the laboratory measured modulus reduction curve determined for a range of strains by cyclic loading tests on good quality undisturbed samples may not be unreasonable for estimating the in-situ large strain moduli for cohesive soils.

Relationships Between Normalized Modulus Reduction Curves and Duration of Confinement

Taniguchi et al. (1978) reported the effect of duration of confinement on the normalized modulus reduction curves of a sandy silt with a plasticity index of 38. Durations of confinement investigated ranged between 15 minutes and 24 hours. It was concluded that the effect of duration of confinement, within the range investigated, on the normalized modulus reduction curve was almost negligible in the strain range of 10^{-4} to 10^{-2} percent. Zen et al. (1986) also showed similar findings for soils having plasticity indices ranging from 40 to 90.

Relationships Between Normalized Modulus Reduction Curves and Frequency of Loading

Aggour et al. (1987) used random excitations with different cutoff frequencies to study the effect of frequency of loading on the modulus reduction curves for cohesive soils. The results of these studies are shown in Fig. 12. It appears that frequency of loading has a significant effect on the modulus reduction relationship, with higher frequencies producing a slower rate of modulus reduction for frequencies in excess of

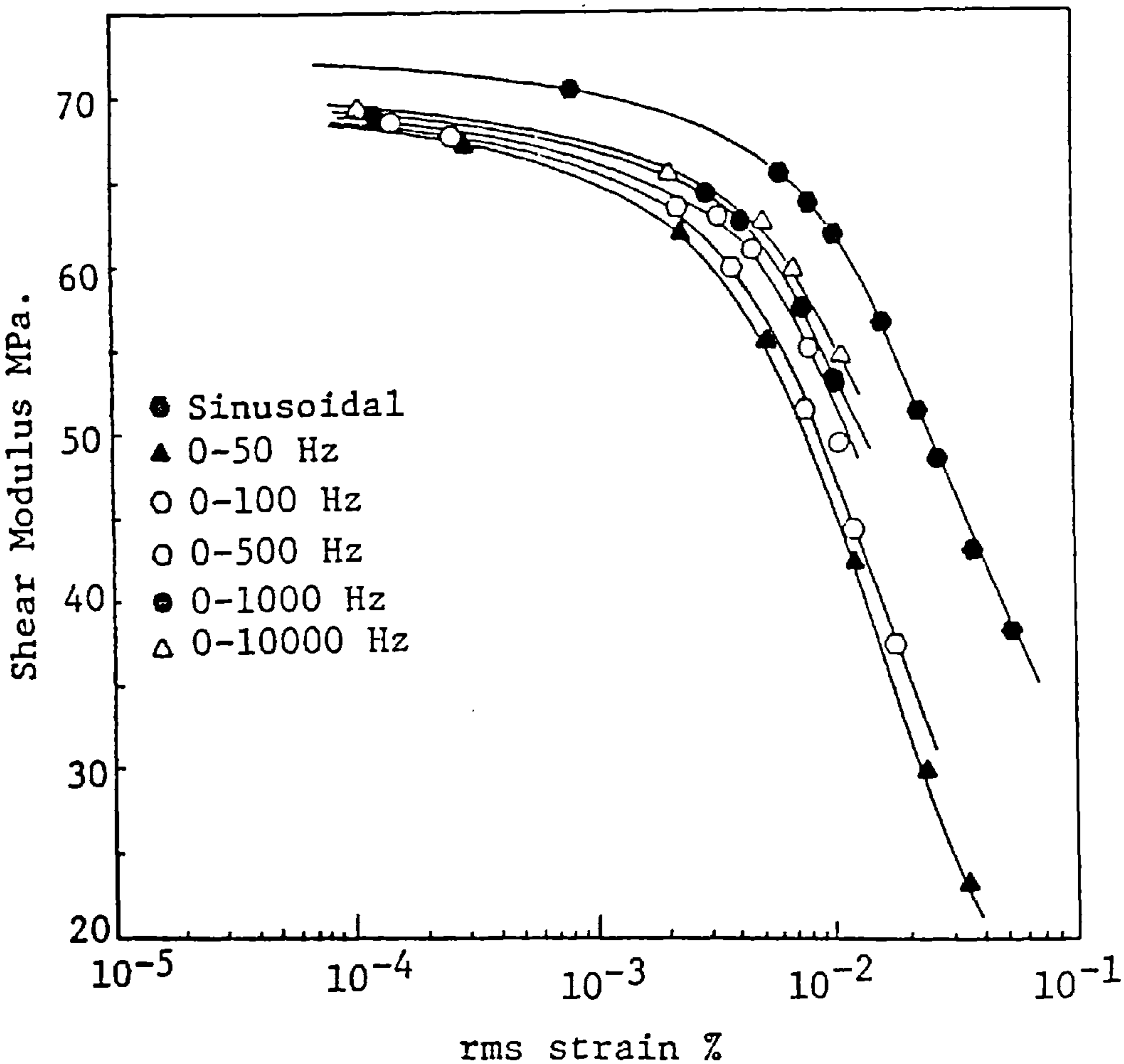

Fig. 12 EFFECT OF FREQUENCY OF LOADING ON NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS (after Aggour et al., 1987)

50 Hz. However, for the frequency range of interest for most earthquakes say 0.1 Hz to 30 Hz, the effect of frequency is negligible for strains less than 0.1%.

6. NORMALIZED MODULUS REDUCTION CURVES FOR COHESIVE MATERIALS WITH VARIOUS PLASTICITY INDICES

Based on the above examination of the effects of factors which may influence the form of the normalized modulus reduction relationships for cohesive soils, it would appear that plasticity index seems to be by far the most dominant and consistent factor.

Accordingly, all of the data presented in Table 1 were separated into five groups on the basis of plasticity index values as indicated below, and plotted to show the range of modulus reduction curves for each group and the average curve for each group.

Group No.	Plasticity Index
1	5 to 10
2	10 to 20
3	20 to 40
4	40 to 80
5	over 80

The results are shown in Figs. 13 to Fig. 17. Fig. 18 shows a summary of the average curves for each group together with the curve for Mexico City clay. The curves follow the same general pattern as that observed by Zen et al. (1984) for artificially prepared laboratory samples, shown in Fig. 19, suggesting that sample disturbance may not have a significant effect on the form of these normalized curves. A similar observation were reported for sandy gravel samples from Japan where the

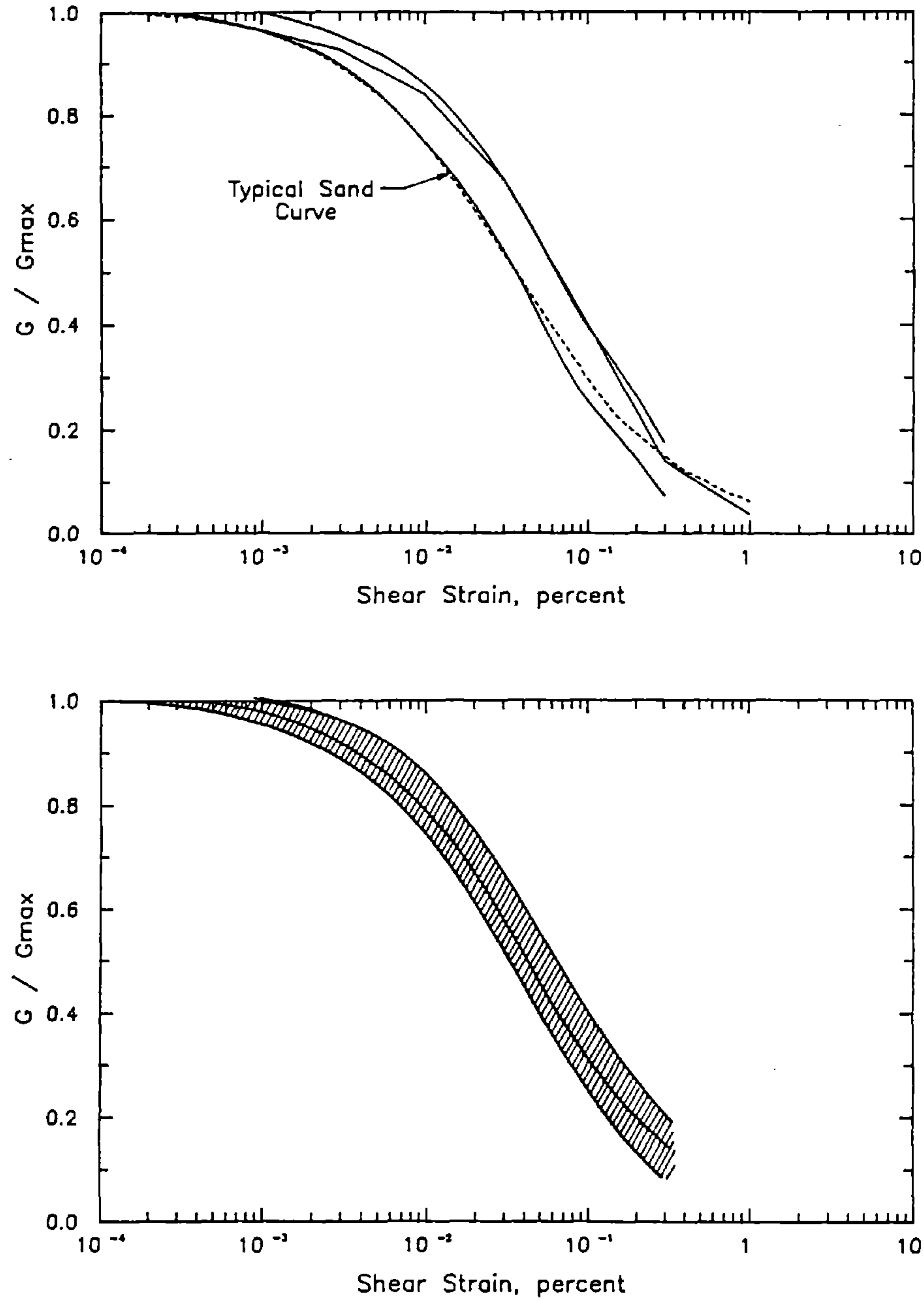

Fig. 13 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH PLASTICITY INDEX BETWEEN 5 TO 10

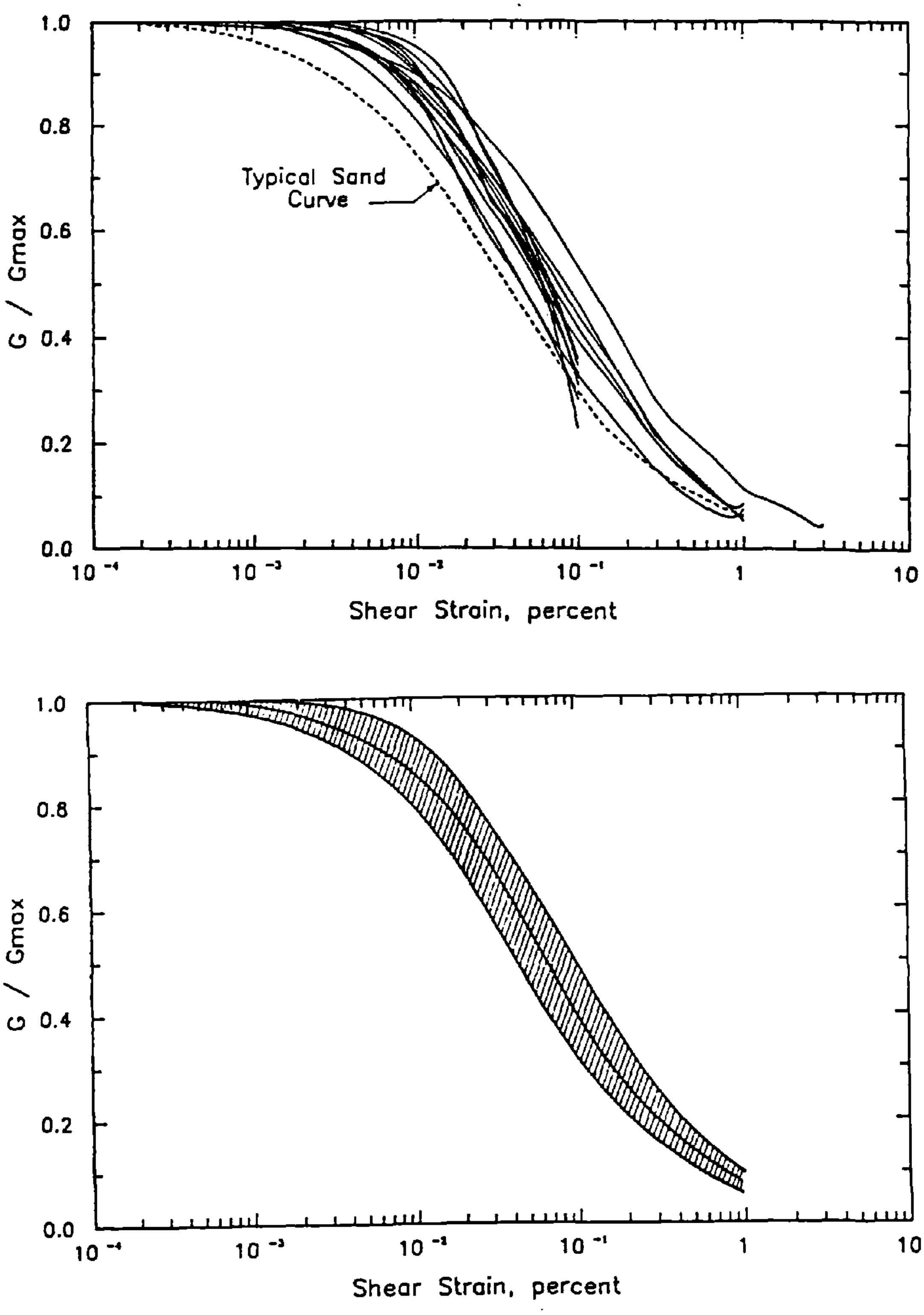

Fig. 14 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH PLASTICITY INDEX BETWEEN 10 TO 20

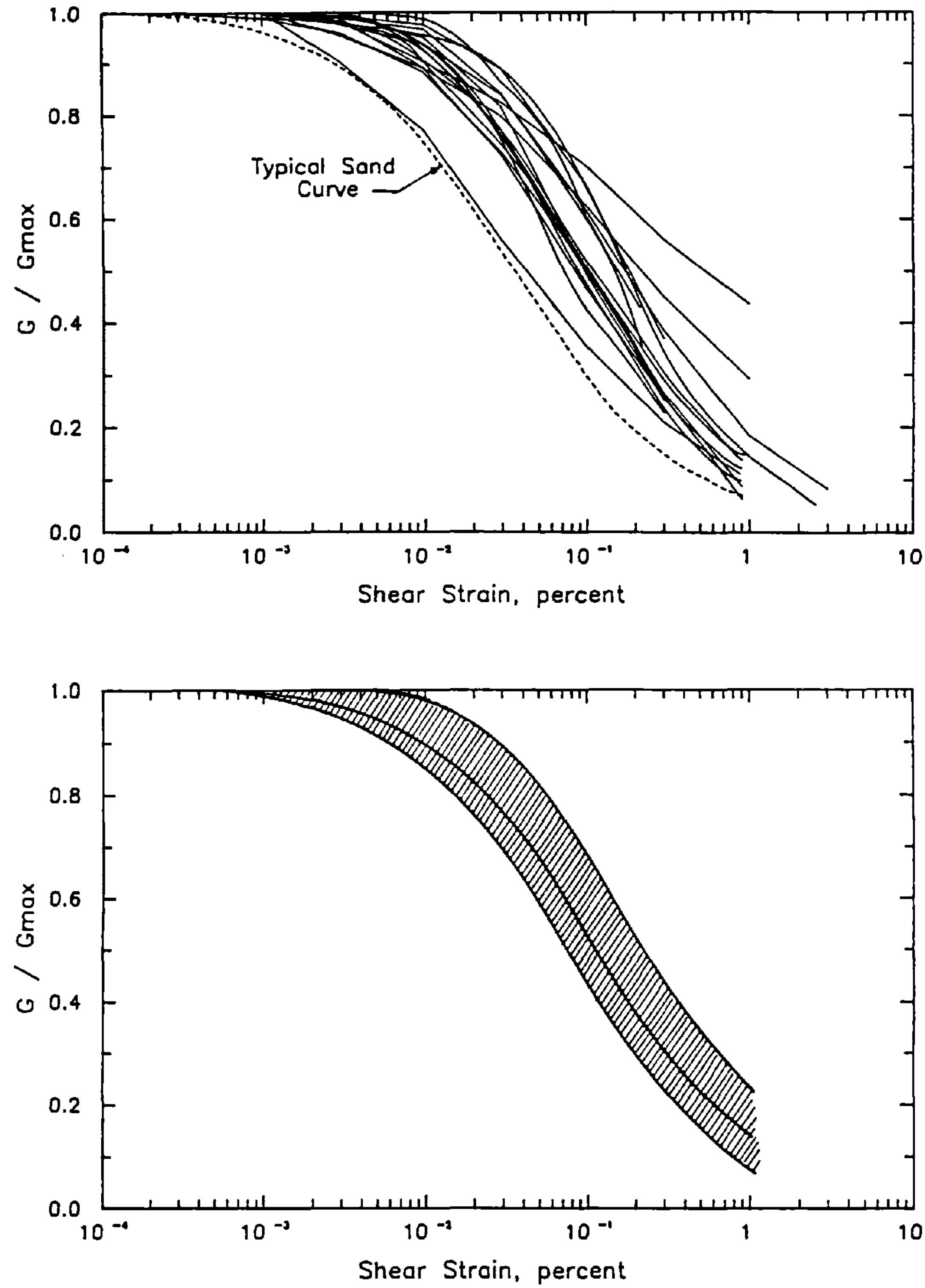

Fig. 15 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH PLASTICITY INDEX BETWEEN 20 TO 40

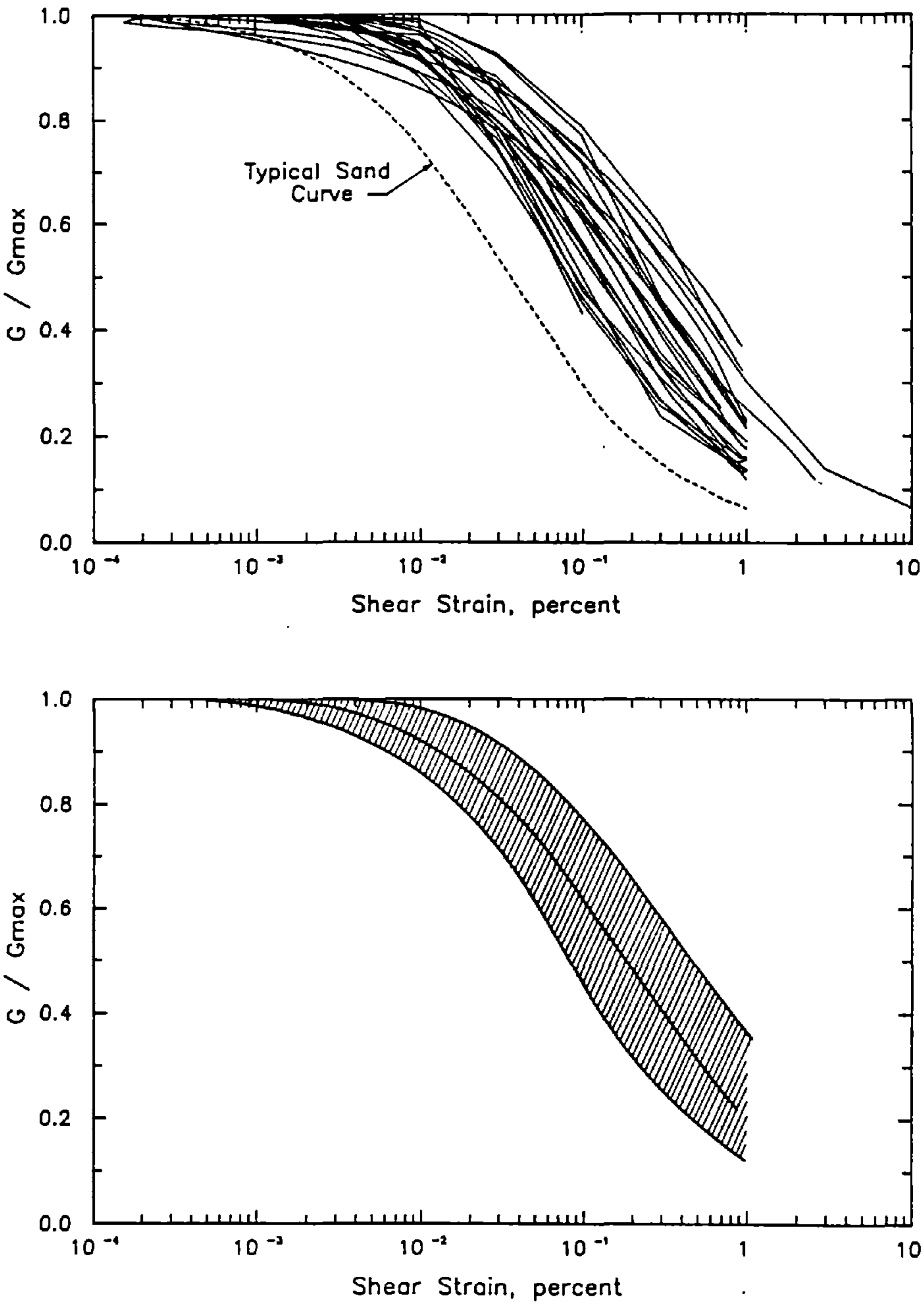

Fig. 16 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH PLASTICITY INDEX BETWEEN 40 TO 80

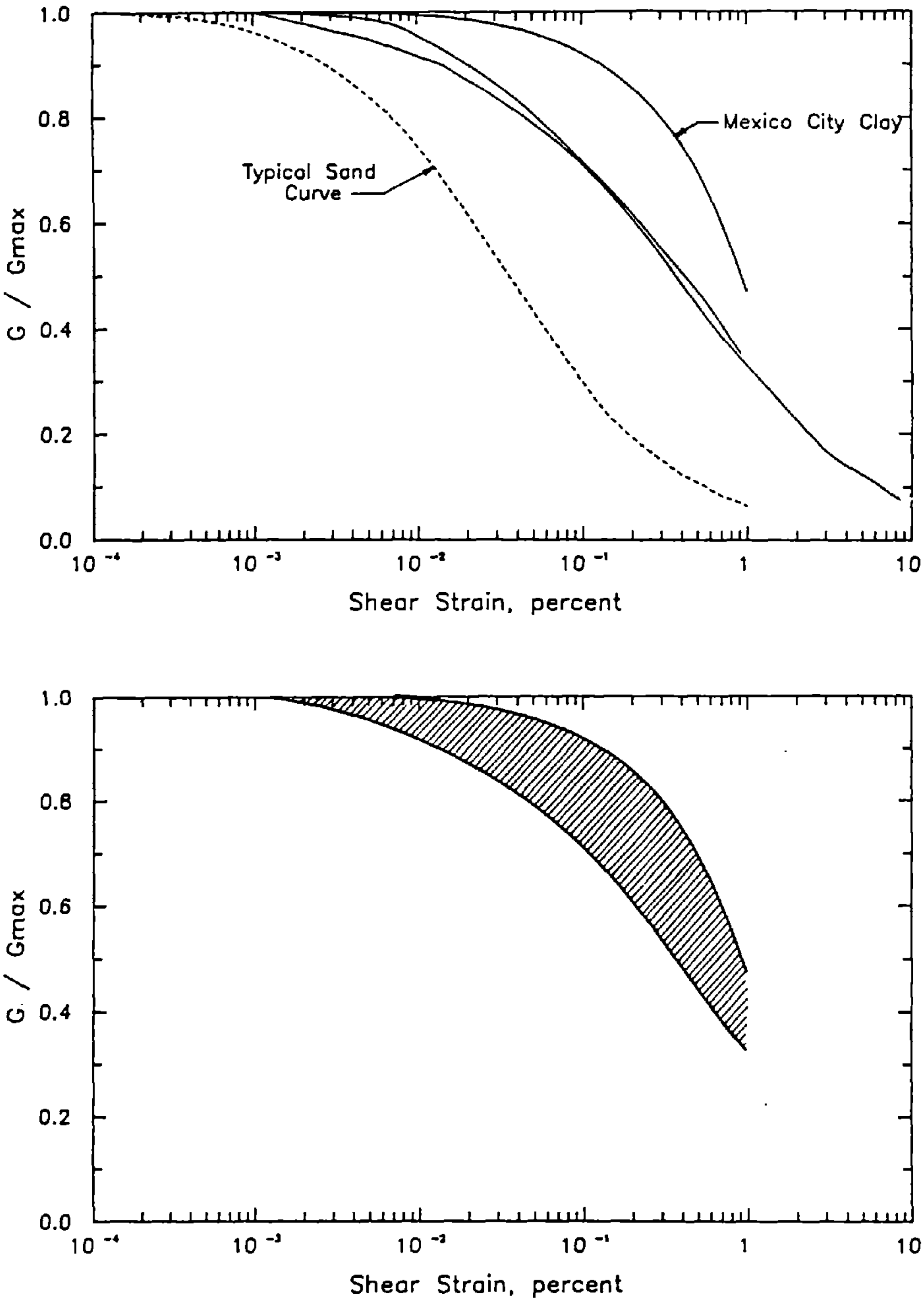

Fig. 17 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH PLASTICITY INDEX OVER 80

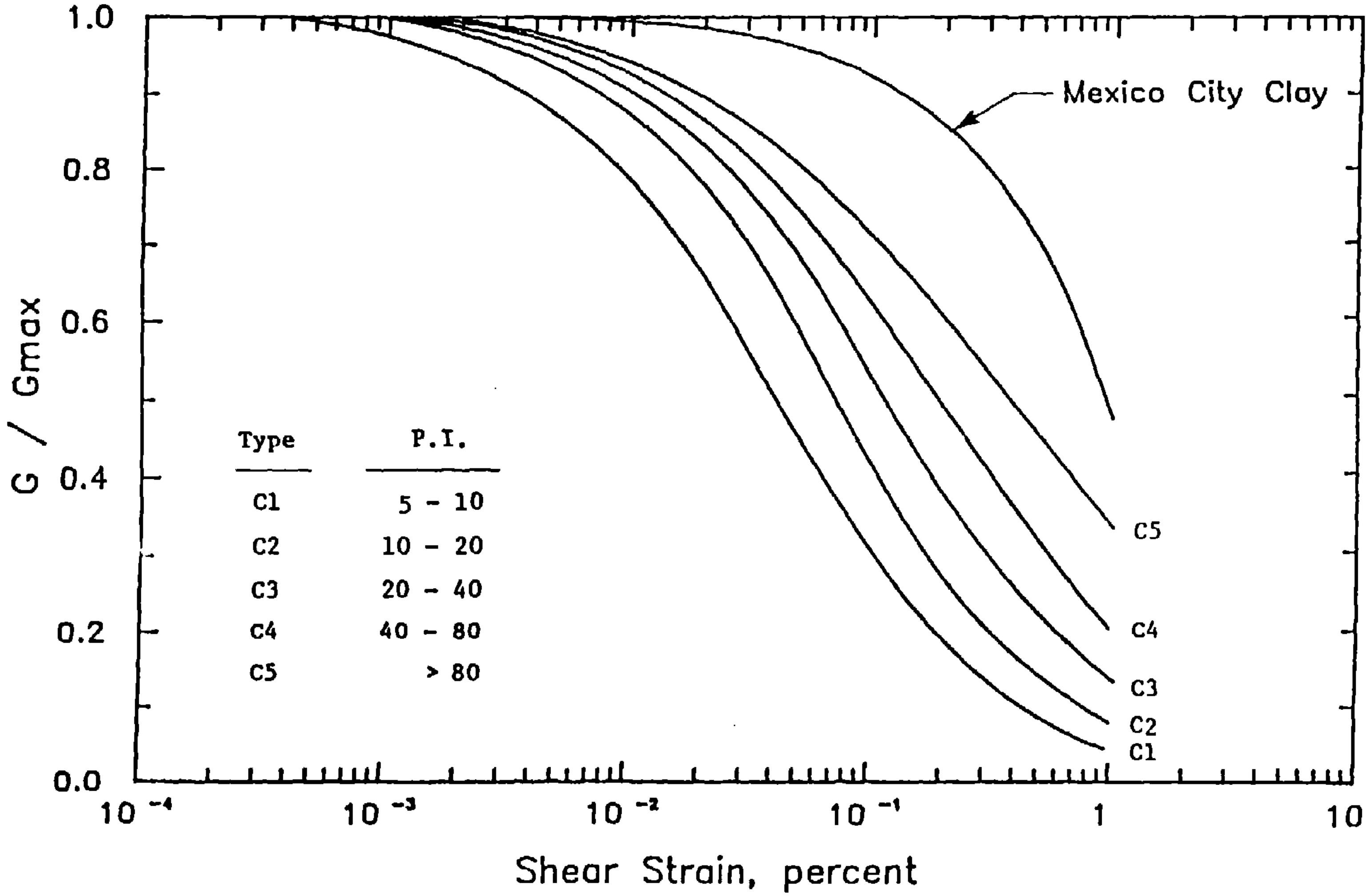

Fig. 18 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR CLAYS WITH DIFFERENT PLASTICITY INDICES

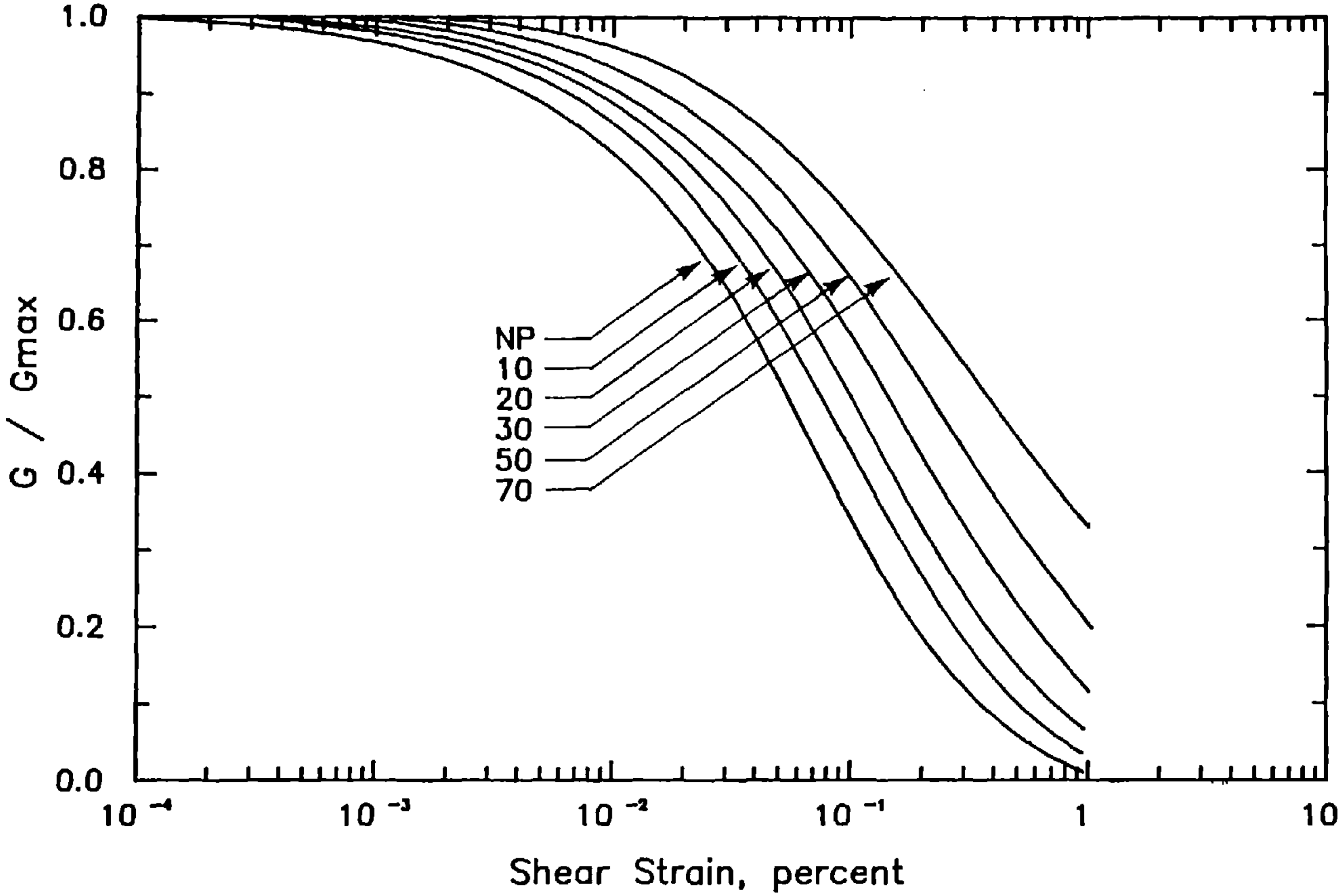

Fig. 19 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR LABORATORY PREPARED CLAY SAMPLES WITH DIFFERENT PLASTICITY INDICES (after Zen et al., 1984)

normalized modulus reduction relationship for samples obtained by ground freezing techniques was found to agree well with that determined for laboratory-reconstituted samples of the same material (Tamaoki et al., 1986 and Hatanaka and Suzuki, 1986). It is also interesting to note that plasticity index has a more profound effect on the location of the normalized modulus reduction curve for clays of low plasticity than for highly plastic clays. The same trend was also noted by Kokusho et al. (1982).

7. NORMALIZED MODULUS REDUCTION CURVES FOR OFFSHORE MATERIALS AND MUDSTONES

Seismic ground response analyses are often required for the seismic design of offshore drilling platforms. However, limited test data on the dynamic properties of offshore materials are available. Fig. 20 shows some of the normalized modulus reduction curves available in the literature. The results shown in Fig. 20 include data reported by Anderson (1980) for silty clay samples ranging in depth from 12 to 121 meters; data reported by Idriss et al. (1976) for samples of clay from the Gulf of Alaska; and data reported by Stokoe et al. (1980) for a clay from an offshore site in southern California. These results show the same general trends as those presented in Fig. 18.

Finally, for completeness purposes, the shear modulus reduction relationships for a mudstone reported by Hara and Kiyota (1977) is shown in Fig. 21. The mudstone tested had a shear wave velocity of about 1500 fps.

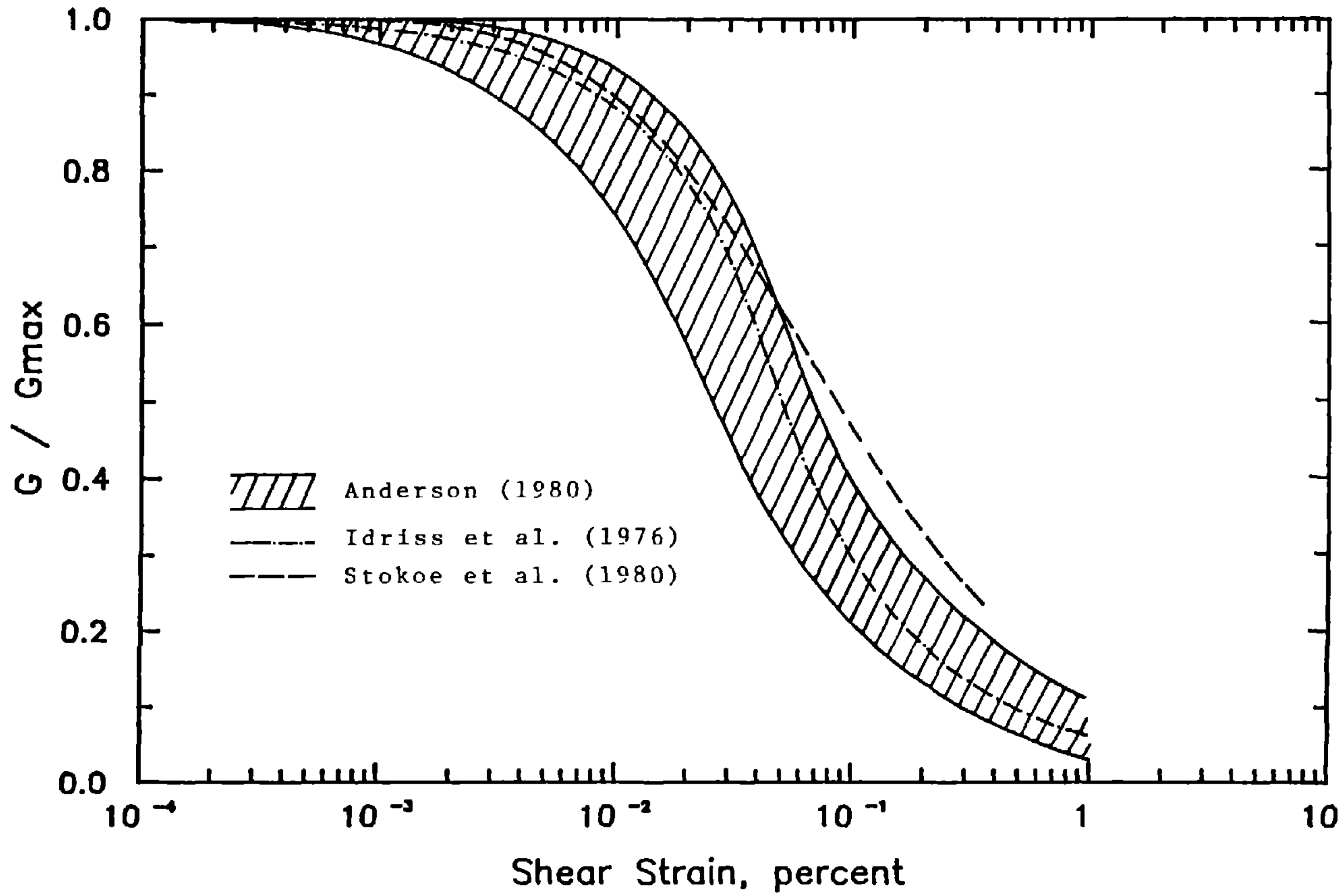

Fig. 20 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR OFFSHORE SILTY CLAY SAMPLES

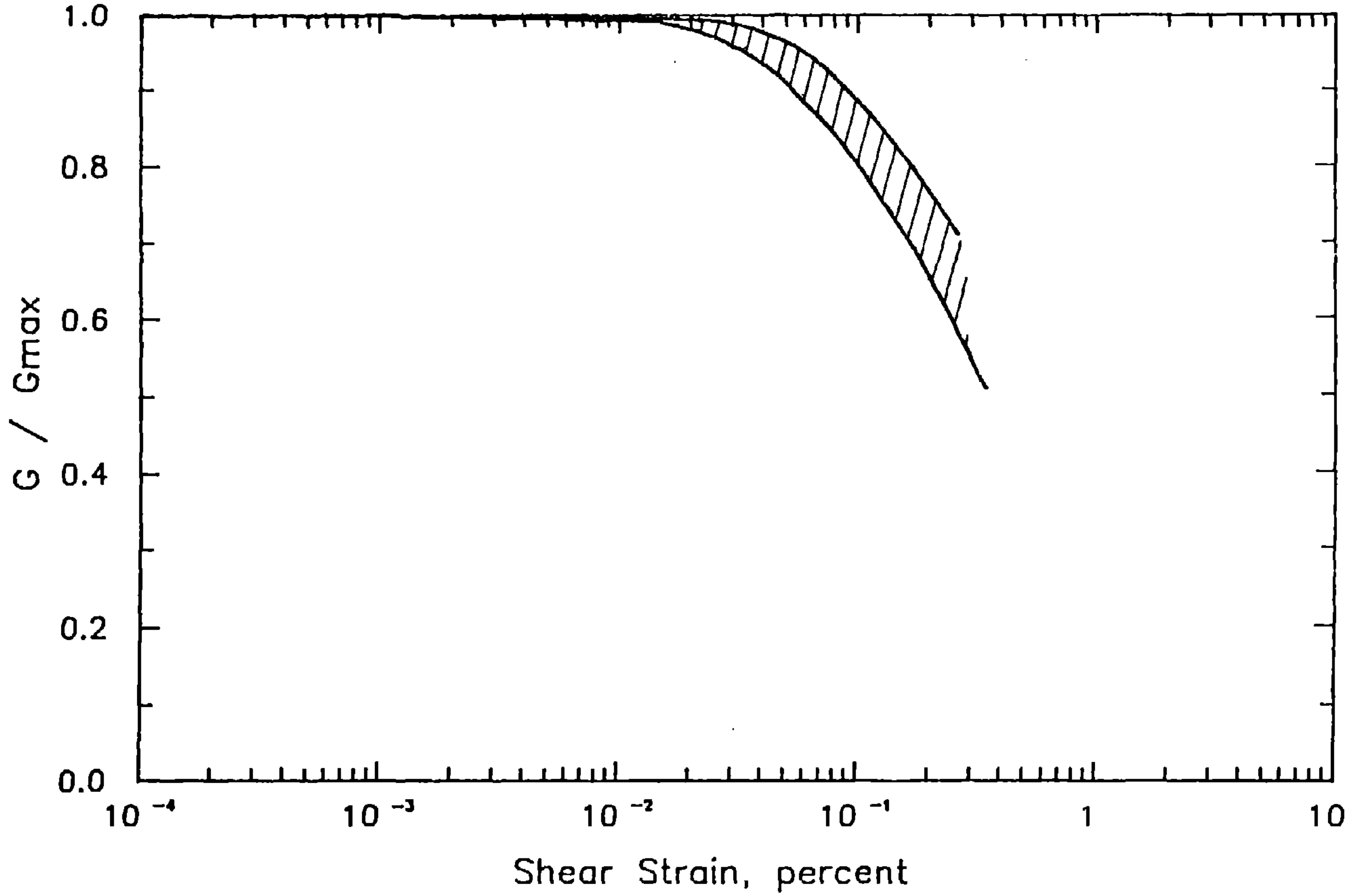

Fig. 21 NORMALIZED MODULUS REDUCTION RELATIONSHIP FOR MUDSTONE HAVING SHEAR WAVE VELOCITY OF 1500 FPS (after Hara and Kiyota, 1977)

8. DAMPING RATIO RELATIONSHIP WITH SHEAR STRAIN

Reported values for the damping characteristics of cohesive soils have not changed significantly over the years from the range indicated by Seed and Idriss in 1970, as may be seen from Fig. 22. Kokusho (1982) has suggested that damping ratio values may be related to the plasticity index of a soil, but the trend is not clear at this time.

9. CONCLUSION

A review of the factors influencing the normalized modulus attenuation curves for cohesive soils shows that the form of this relationship is not significantly affected by consolidation stress history, duration of confinement, frequency of loading (for earthquake frequencies) and sample disturbance up to moderate strain levels. Confining pressure may influence the form of the modulus reduction curves for low plasticity soils but it has very little influence on the G/G_{max} vs. strain curves for soils having a plasticity index in excess of 25. However, the form of the relationship is significantly influenced by the plasticity index of a soil and the results shown in Fig. 19 are believed to provide a useful guide to the use of such relationships in engineering practice. In addition, it appears that void ratio may be a significant secondary factor to be considered in selecting a modulus reduction curve for analysis purposes.

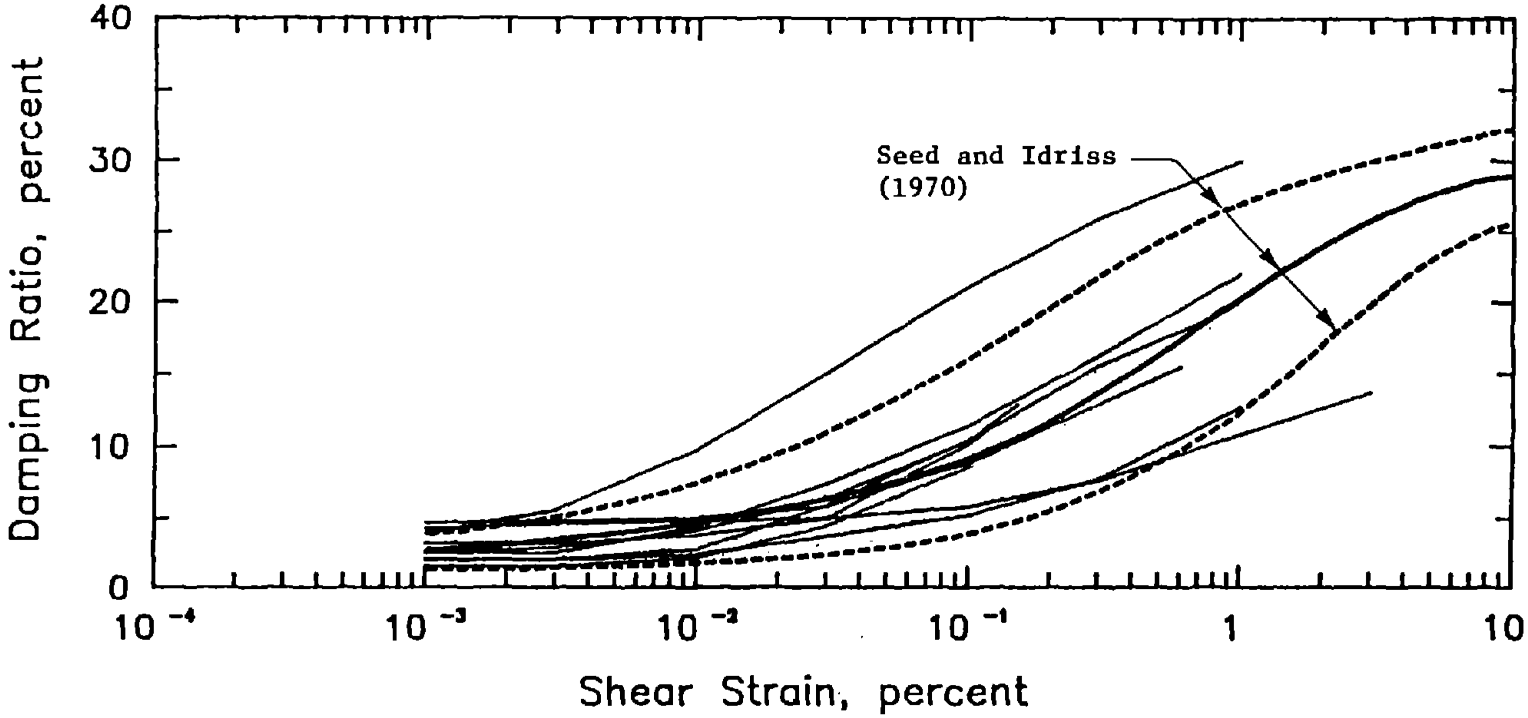

Fig. 22 STRAIN DEPENDENT DAMPING RATIOS FOR CLAYS

ACKNOWLEDGEMENT

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