SEISMIC CHARACTERISTICS OF SEDIMENTS IN THE NEW MADRID SEISMIC ZONE

A Research Report Submitted To
National Science Foundation

Grant No. BCS-9004086

By
Tzyy-Shiou Chang, Lin Kee Teh, and Yigong Zhang
February 1992

Center for Earthquake Research and Information
Memphis State University
Memphis, Tennessee 38152

IA

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

Page

Page

ABSTRACT

SEISMIC CHARACTERISTICS OF SEDIMENTS IN THE NEW MADRID SEISMIC ZONE

The most critical and essential information required for seismic studies is the determination of dynamic properties and stratification of the underlying soils. According to state-of-the-art geotechnical techniques, strain-dependent nonlinear modulus and damping, which are the important dynamic properties of soils for strong ground motion and liquefaction studies, can be determined only in the laboratory.

A total of 58 samples of soils were collected from the local government agencies and engineering consulting companies. Thirty-five samples of soils in the northern Mississippi embayment region were selected for the dynamic testing program through use of the resonant column techniques for measuring low-strain shear modulus, high-strain modulus degradation characteristics, and damping ratio.

In general, the test results agree with the results of previous studies of soil dynamics: namely, the shear modulus increases with increasing static shear strength (smaller void ratio of sand and higher shear strength S_u of clay) and increasing confining pressure, but decreases with increasing shear strain. For all samples tested, damping ratio decreases as modulus increases. The shear modulus and shear-wave velocity estimated from empirical equations derived in the study have differences ranging from -13% to +12% compared with that estimated from the existing Hardin equation. The values of "n," which represent the effect of confining pressure on shear modulus of granular soils, range remarkably from 0.25

to 0.98 for the soils tested. This wide deviation is reasonable because of the significant difference in constituents and origins of the natural sediments in the study area. However, surprisingly, the average value of each granular soil group ranges from 0.5 to 0.54, essentially the same as recommended by the Hardin equation (n = 0.5)

The high-strain parameters for the tested soils, derived on the basis of Martin-Davidenkov and Ramberg-Osgood models, revealed that the high-strain nonlinear behavior of the soils in the northern Mississippi Embayment region may be somewhat different than that of soils elsewhere. The threshold shear strain, at which shear modulus starts to decrease rapidly with increasing strain, tends to increase with increasing confining pressure. Preliminarily, this infers that for a site subjected to earthquake shaking, the lower portion of the profile below a certain depth would behave like a rigid body.

The empirical equations derived in the study and the tabulated soil properties may be useful for future reference in general soil dynamics and for future seismic studies in the New Madrid seismic zone.

ACKNOLEDGMENTS

The authors express their appreciation to the U.S. Army Corps of Engineers, Memphis district; Hall, Blake and Associates; and Professional Service Industry, Inc. for providing soil samples to the study. The financial support from the National Science Foundation (NSF Grant No BCS-9004086) is also gratefully acknowledged.

Special thanks are also given to Ms. Diane Matthews, Ms. Valinda Stokes, and Mr. Gik Chay Lim for preparing the report. The comments by Professor R. D. Woods of the University of Michigan and Professor T. S. Fry of Memphis State University were essential for finalizing the research report.

CHAPTER 1

INTRODUCTION

The New Madrid seismic zone (NMSZ) has been the focus of attention since the Loma Prieta earthquake that struck San Francisco on Oct 17, 1989, because the NMSZ is considered one of the most active seismic zones in the eastern United States. The rate of occurrence of small to moderate earthquakes in the area is very high, with an average of 150 times per year. The magnitude of these earthquakes range from below threshold of feeling to magnitude of 4 or greater [3, 8]. Based on the results of the probability studies of the New Madrid earthquakes, the chance of occurrence of an earthquake with M_S = 6.3 is high (86-97%) by the year 2035 [8].

Physiographically, the Mississippi Embayment is a part of the Gulf Coastal Plain province. The embayment is separated into two general physiographic areas: the Mississippi Alluvial Plain and the Coastal Plain Upland (Figure 1-1). The New Madrid seismic zone is located at the northern edge of the embayment. It is well known for its seismically active buried faults. These buried faults are associated with numerous earthquakes in the central United States including the catastrophic earthquakes of 1811-1812. The characteristics of the earthquakes that occur in the NMSZ are (1) a long recurrence period, and (2) a low seismic-wave attenuation. The latter has resulted in significantly greater felt and larger damage areas than those subject to earthquakes of the same order in other seismic regions of the United States [8].

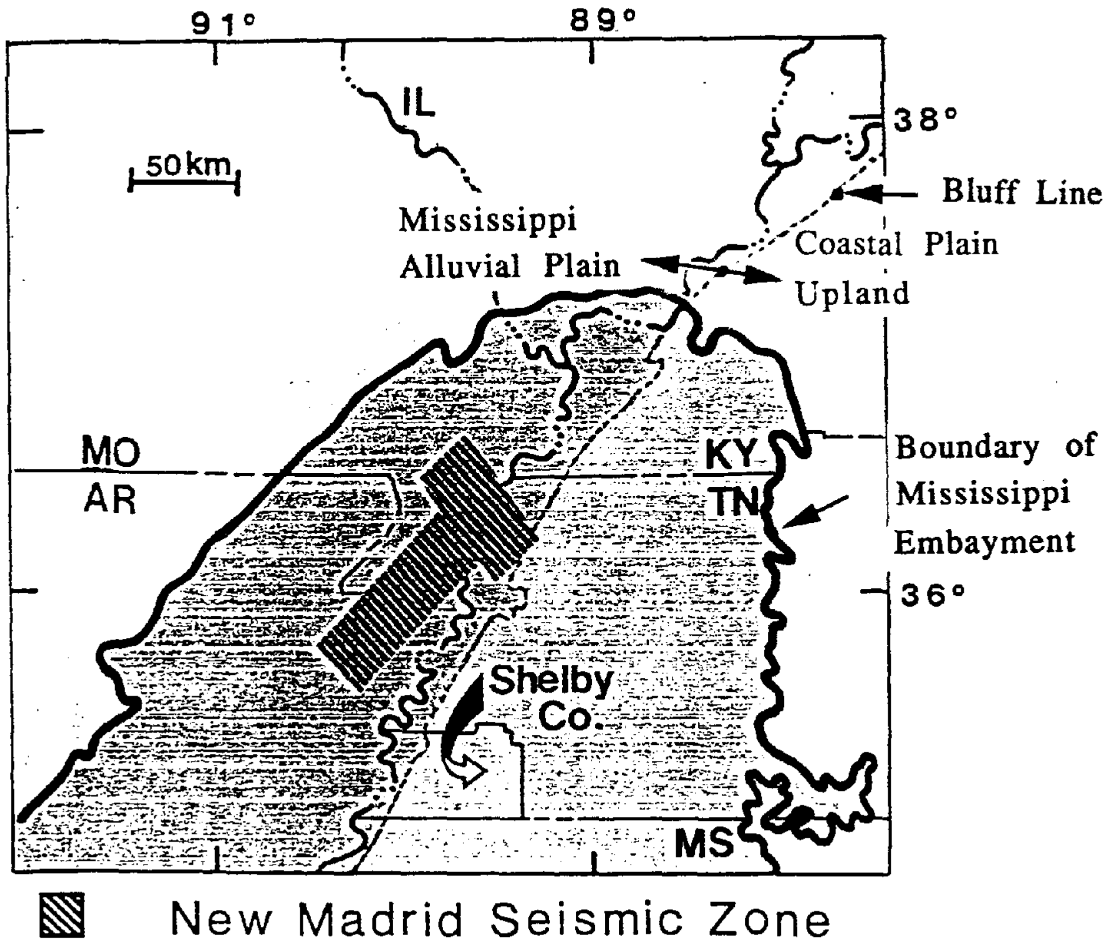

Figure 2-1 Map of Northern Mississippi Embayment

The dynamic properties of the unconsolidated soils in the upper portions of the soil profile are of concern to regional seismic risk assessment and other seismic studies [9, 13, 14, 15]. In the NMSZ areas, the unconsolidated soils are those in and above the Jackson Formation (Upper Claibane) which forms a firm layer within the sediments of the northern Mississippi Embayment. The Coastal Plain Upland, located mostly east of the Mississippi River, has soils that consist of alluvium, loess, and terrace deposits overlying the Jackson Formation. The soils above the Jackson Formation in the Mississippi Alluvial Plain, located mostly west of the Mississippi River, consist of top stratum, upper sand, and lower sand [9].

From the experience of past large earthquakes including the well known 1964 Niigata, Japan, 1985 Mexico City, and 1989 Loma Prieta earthquakes, major causes of failure of the ground supporting the structures are attributed to (1) magnification of ground surface shaking as a result of the coincidence between a natural mode of the soil profile and the dominant frequency of incident seismic waves, and (2) liquefaction of the granular deposits near the ground surface. In order to investigate these two soil dynamics problems, a knowledge of the local subsurface conditions and dynamic properties of the local soils is essential. The ideal way to obtain the elastic properties of in situ soils is through seismic field testing. However, the cost and amount of work involved can be enormously high. Furthermore, according to state-of-the-art geotechnical techniques, strain-dependent nonlinear modulus and damping, which are very important characteristics of soils for strong ground motion studies, can be determined best in the laboratory.

Since the NMSZ is one of the most active seismic zones in the United States, a knowledge of the characteristics of the sediments and the dynamic properties of the soils in the northern Mississippi Embayment region for evaluation of earthquake hazard is essential. In addition, laboratory dynamic testing provides valuable data that are more reliable and accurate for regional hazard evaluation than are estimations of dynamic properties based on empirical equations.

The objectives of this study are

1) To provide a reference table for estimating shear-wave velocity, shear modulus, and damping ratio of various types of natural soils for future seismic studies in NMSZ.
2) To derive or adapt empirical equations for determining the dynamic properties of NMSZ soils by using parameters reflecting the in situ conditions of the natural soil deposits.
3) To study the relationship of dynamic properties between natural sedimentary soil samples and ideal laboratory soil samples.

CHAPTER 2

COLLECTION OF SOIL SAMPLES

The soil samples used in this testing program were collected from local government agencies and private organizations. Government agencies such as the U.S. Army Corps of Engineers and local consulting companies like PSI, and Hall, Blake, and Associates contributed soil samples to this program. The samples are indexed by a six-digit number. The first four digits denote the month and the date the sample was received and the last two digits signify the sequence of the sample received. For example, sample 090601 indicates that this sample was collected on September 6, 1990, and is given a sample number of 1 since this is the first sample received. All of the soil samples came from the northern Mississippi Embayment area except for samples no. 20 and no. 22, which came from a site near the western boundary of the embayment, as indicated in Figure 2-1.

A total of 58 samples were collected from the general NMSZ area from the states of Arkansas and Tennessee. After careful examination and selection, 35 samples were selected for this study. Among these, 12 of the samples originated from the Memphis metropolitan and Shelby County area. Widely dispersed sample locations of data within the northern Mississippi Embayment was not possible because of nonavailability of soil samples from some rural areas.

The soil samples are classified into 3 groups, namely, group A (cohesionless granular soil), group B (cohesive soil), and group C

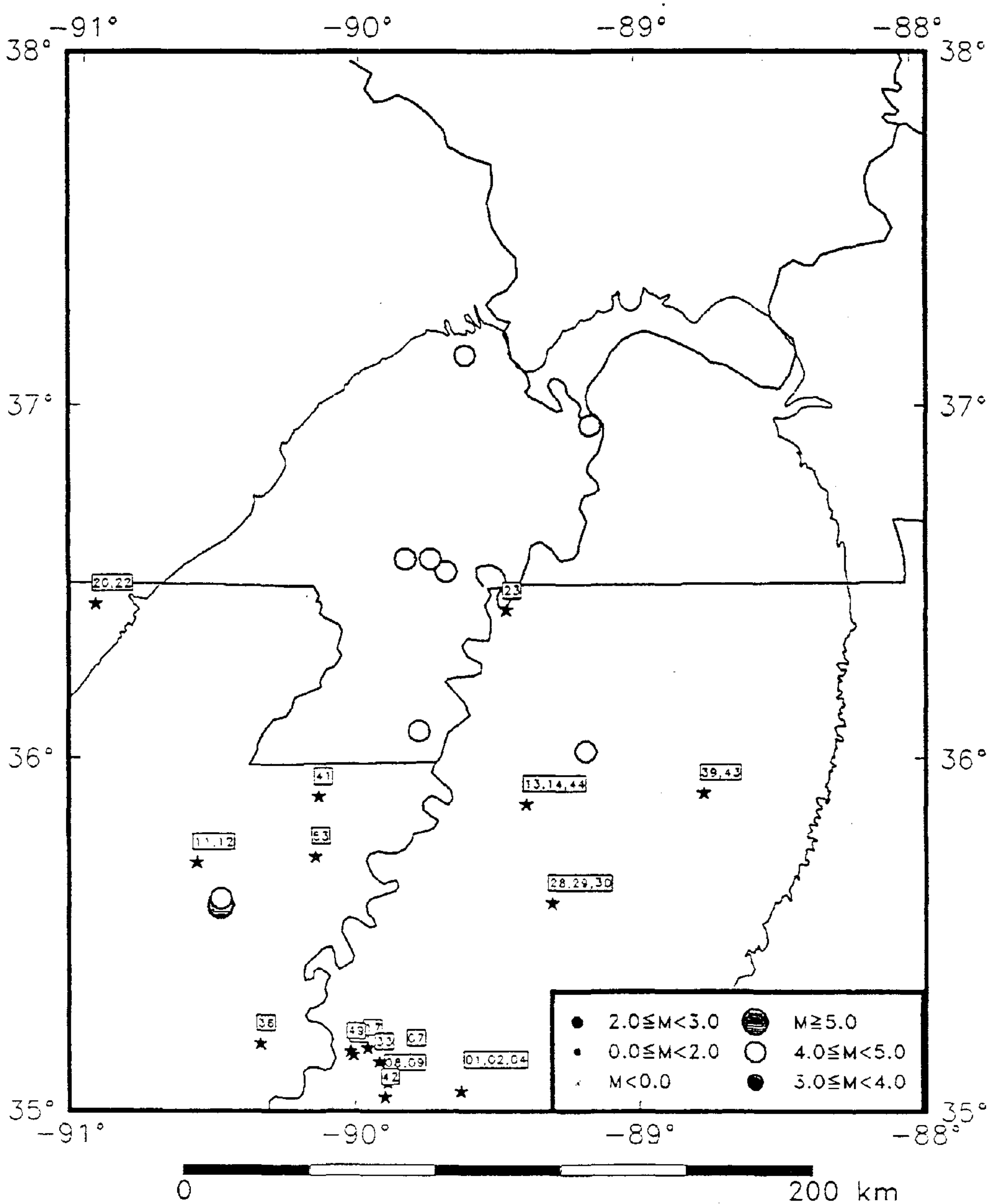

Figure 2-1 Soil Sample Location in Mississippi Embayment

(loess). Within groups A and B, the soil samples were further subdivided into two or more subgroups, as shown in Table 2-1. The exact location of several soil samples are not available and, as a result, only approximate locations are indicated in Table 2-2.

Table 2-1 Classification of Soil Groups used for Dynamic Testing

Soil Groups	Descriptions	Range of Depth of Boring	Number of Samples
A1	Alluvial sand (SP-SM)	5' - 40'	7
A2	Terrace sand and gravel (SP-SW-SM-SC-GP)	5' - 54'	7
A3	Jackson fine sand (SP)	25' - 71'	4
B1	Silty to sandy clay (CL)	1' - 33'	9
B2	Jackson clay (CL-CH)	8' - 25'	4
C	Loess	4.5' - 53.6'	4
		Total	35

Table 2-2 Location, Depth, and Description of Soil Samples

SAMPLE NO.	LOCATION	DEPTH	DESCRIPTION
090601-1 090601-2 090601-3	Wolf River, Collierville, TN (35° 04' 00"N, 89° 37' 30" W)	5'	Very fine brown silty sand (SM)
090602-1 090602-2 090602-3 090602-4	Wolf River, Collierville, TN (35° 04' 00"N, 89° 37' 30" W)	5'	Brown sandy to silty clay (CL)
090604-1 090604-2 090604-3 090604-4	Wolf River, Collierville, TN (35° 04' 00"N, 89° 37' 30" W)	5 '	Brown fine to medium sand (SP)
101507-1 101507-2 101507-3 101507-4	Walnut Grove at Shelby Farm, Memphis, TN (35° 07' 30" N, 89° 49' 45" W)	5'	Brown clayey silt (ML-CL)
101708-1 101708-2 101708-3	Knight & Perkins, Memphis, TN (35° 03' 30" N, 89° 54' 30"W)	30'-40'	Very fine brown silty sand (SP-SM)
101709-1 101709-2 101709-3	Knight & Perkins, Memphis, TN (35° 03' 30" N, 89° 54' 30" W)	30'-40'	Fine reddish brown silty sand (SP-SM)
101711-1 101711-2 101711-3	Trauman, AR (35° 40' 40" N, 90° 35' 30" W)	35'-50'	Dark gray fine sand with trace of chalk (SP)
101712-1 101712-2 101712-3	Trauman, AR (35° 40' 40" N, 90° 35' 30" W)	25'-30'	Gray fine sand (SP)
101713-1 101713-2 101713-3	Halls, TN (35° 50' 45" N, 89° 25' 00" W)	23'	Very fine brown silty sand (SM)
101714	Halls, TN (35° 50' 45" N, 89° 24' 00" W)	27'-28'	Very stiff brown silty clay (CL)
101715	Memphis Downtown Medical Center (35° 08' 30" N, 90° 01' 45" W)	12'-14'	Dark brown sandy to silty clay (CL)
101717	Jackson & Warford, Memphis, TN (35° 09' 35" N, 89° 58' 50" W)	7'-9'	Hard gray clayey silt (ML-CL)

Table 2-2 Continued

SAMPLE NO.	LOCATION	DEPTH	DESCRIPTION
101718 (CJC)	Army Corps of Engineers	24'-25'	Gray clay (CL-CH)
101720	Pocohontas, AR (36° 24' 36" N, 90° 57' 33" W)	14'	Brown silty clay to clayey silt (CL-ML)
101722	Pocohontas, AR (36° 24' 36" N, 90° 57' 33" W)	1'	Gray and brown silty to sandy clay and clayey silt (CL-ML)
101723-1 101723-2 101723-3	Tiptonville, TN (36° 23' 45" N, 89° 29' 10" W)	20'	Brown coarse sand (SP-SW)
111926	Helena, AR (34° 31' 30" N, 90° 37' 58" W)	16'-18'	Dark gray medium silty clay (CL-CH)
111928	Brownsville, TN (35° 34' 20" N, 89° 20' 00" W)	14.5'	Brown clayey to sandy silt (CL-ML)
111929	Brownsville, TN (35° 34' 20" N, 89° 20' 00" W)	4.5'	Brown clayey silt (ML)
111930-1 111930-2 111930-3	Brownsville, TN (35° 34' 20" N, 89° 20' 00" W)	24.5'	Very fine reddish brown silty sand (SP-SM)
121433-1 121433-2 121433-3	MSU Campus, Memphis, TN (35° 07' 00" N, 89° 56' 12" W)	5'	Brown coarse to medium sand with gravel (SM-SW-GP)
010434-1 010434-2 010434-3 010434-4	West Tennessee	69.0'-70.5'	Very fine white sand (SP)
010435	Tribs, West TN	51.0'-53.6'	Hard dark gray clayey silt (ML-CL)
010736	West Memphis,AR (35° 10' 00" N, 90° 21' 40" W)	8.0'-8.8'	Dark gray clay with white chalk (CL-CH)
010837	West TN	14.0'-14.5'	Soft dark brown and gray silty clay (CL)

Table 2-2 Continued

SAMPLE NO.	LOCATION	DEPTH	DESCRIPTION
011439-1 011439-2 011439-3 011439-4	Milan, TN (35° 52' 45" N, 88° 47' 30" W)	44'-71'	Light pink fine silty sand (SP-SM)
011442-1 011442-2	Memphis, TN (35° 01' 20" N, 89° 55' 10" W)	19'-53'	Brown silty coarse to medium terrace sand and gravel (SW-SM-GP)
013143	Milan, TN (35° 52' 45" N, 88° 47' 30" W)	31'-33'	White hard silty to sandy clay (CL)
020144	Halls, TN (35° 50' 45" N, 89° 25' 00" W)	26'-28'	Hard dark brown silty clay (CL)
020145-1 020145-2 020145-3 020145-4	unknown	unknown	Dark brown fine to coarse terrace sand with gravel and silt (SM-SP-GP)
021246-1 021246-2 021246-3	unknown	unknown	Brown coarse to fine sand and gravel (SW-GP)
021549-1 021549-2 021549-3 021549-4	Shelby County Prison, TN (35° 09' 10" N, 90° 02' 40" W)	44'-54'	Light gray silty coarse to fine sand and gravel (SM-SW-GP)
021553-1 021553-2	Osceola, TN (35° 42' 15" N, 90° 10' 00" W)	23.5'-26'	Dark brown silty sand (SM)
CJA20'	Army Corps of Engineers, TN	20'	Gray silty clay (CL-CH)
NMSZ41-3	Manila, AR (35° 52' 10" N, 90° 9' 50" W)	15'	Very fine gray sand (SP)

CHAPTER 3

LABORATORY TESTING PROCEDURES

In this study, the laboratory testing program includes

1) index property tests to obtain the fundamental soil properties,
2) seismic characteristic determinations including low-strain dynamic shear modulus tests, shear-wave velocity tests, and energy dissipation determination (damping ratio), and
3) high-strain modulus degradation characteristic tests.

Each of these testing subprograms will be discussed in detail in the following sections. The seismic characteristics of soils were determined in the laboratory through use of the resonant column test. The applicable theoretical background of that test will be described in the following section.

3.1 Fundamental Index Properties

The basic index properties of the soils are determined in order to describe the fundamental physical properties of the soil tested. Such information is very important for determining the necessary parameters to be used in the dynamic testing as well as for interpretation and application of the dynamic test results. The fundamental properties were determined by using the ASTM standard procedures [1] and/or the soil laboratory manual by Bowles [2]. The basic soil properties determined for the samples include:

1) moisture content,
2) specific gravity,
3) grain size distribution,

4) relative density,

5) shear strength,

6) Atterberg limits,

7) Visual descriptions, and

8) Unified Soil Classsification.

3.2 Shear-Wave Velocity and Damping Ratio at Low Strain

The shear-wave velocity (low-strain dynamic shear modulus) and the energy dissipation characteristics (damping ratio) were determined in this test program under simulated field conditions. These low-strain elastic properties of soils are the fundamental characteristics of soils for seismic studies. For each cohesionless soil sample the dynamic shear modulus tests were conducted for two to four different void ratios. For cohesive soils, a shelby tube sample with water content close to the in situ water content of the samples was used for the texts. The confining pressures for all tests were fixed at 5, 10, 20, 40, and 55 or 60 psi. These pressures simulate in-site condition for depth up to 100 feet.

3.3 Modulus Degradation Characteristics at High Strain

Beyond the elastic range (strain greater than about $10^{-3}\%$), the shear modulus decreases and the damping increases as the strain level increases. This characteristic is significant in seismic studies and other strong motion studies. These tests were conducted at confining pressures of 5, 20, and 55 or 60 psi.

3.4 Theoretical Background of Resonant Column Test

The resonant column technique for testing soil consists of applying a sinusoidal torque to a cylindrical specimen that is identical to a conventional triaxial test specimen [7]. The sinusoidal torque generates shear waves that travel through the specimen and are reflected back from the fixed base. The first mode of undamped natural frequency is obtained by adjusting the frequency of the applied torque when the resonance (maximum response) of the sample is observed. This occurs when the response of the sample (velocity) is 180° out of phase with the applied force, measured at the end of the specimen where the force is applied.

From (1) the measured frequency, (2) known values of specimen size, (3) its mass moment of inertia, and (4) mass moment of inertia of the driving mass, the dynamic shear modulus can be determined [4, 7, 12]. The dynamic shear strain is related to the rotational amplitude at the end of the specimen and the shear damping ratio is affiliated with the resulting motion shear stiffness and magnitude of the sinusoidal force [7].

The dynamic modulus and the damping ratio can be determined from very low dynamic strain levels (on the order of 0.001%) which correspond to the strain levels associated with microseismic activity, traffic vibrations, and machine foundation induced vibrations. Higher strain levels (up to 1%) are associated with earthquake shaking and other strong ground motions resulting from large loadings. The readings at high-strain levels can be obtained in a manner similar to that for low-strain levels except that the power that drives the sample is increased (two times of that previously used starting from low-strain level used) and the frequency is readjusted to obtain the resonance of the sample at various strain levels [7].

The confining pressure can be applied by means of either a liquid confining medium pressurized by regulated compressed air or by direct use of regulated compressed air. In this study, regulated compressed air without a liquid confining medium was used to apply a comfining stress to the sample. The maximum confining pressure that can be used with this apparatus is 100 psi.

The resonant column test apparatus consists of the following individual pieces of equipment.

1) Drnevich Long-Tor resonant column apparatus
2) LVDT readout
3) Oscilloscope
4) Longitudinal charge amplifier
5) Torsional charge amplifier
6) Switch box
7) Digital voltmeter
8) Control box
9) Longitudinal power amplifier
10) Torsional power amplifier
11) Longitudinal oscillator
12) Torsional oscillator
13) Digital frequency monitor

A schematic diagram of a typical test set-up using this equipment is shown in Figure 3-1. A photograph of the Drnevich Long-Tor column apparatus is shown in Figure 3-2.

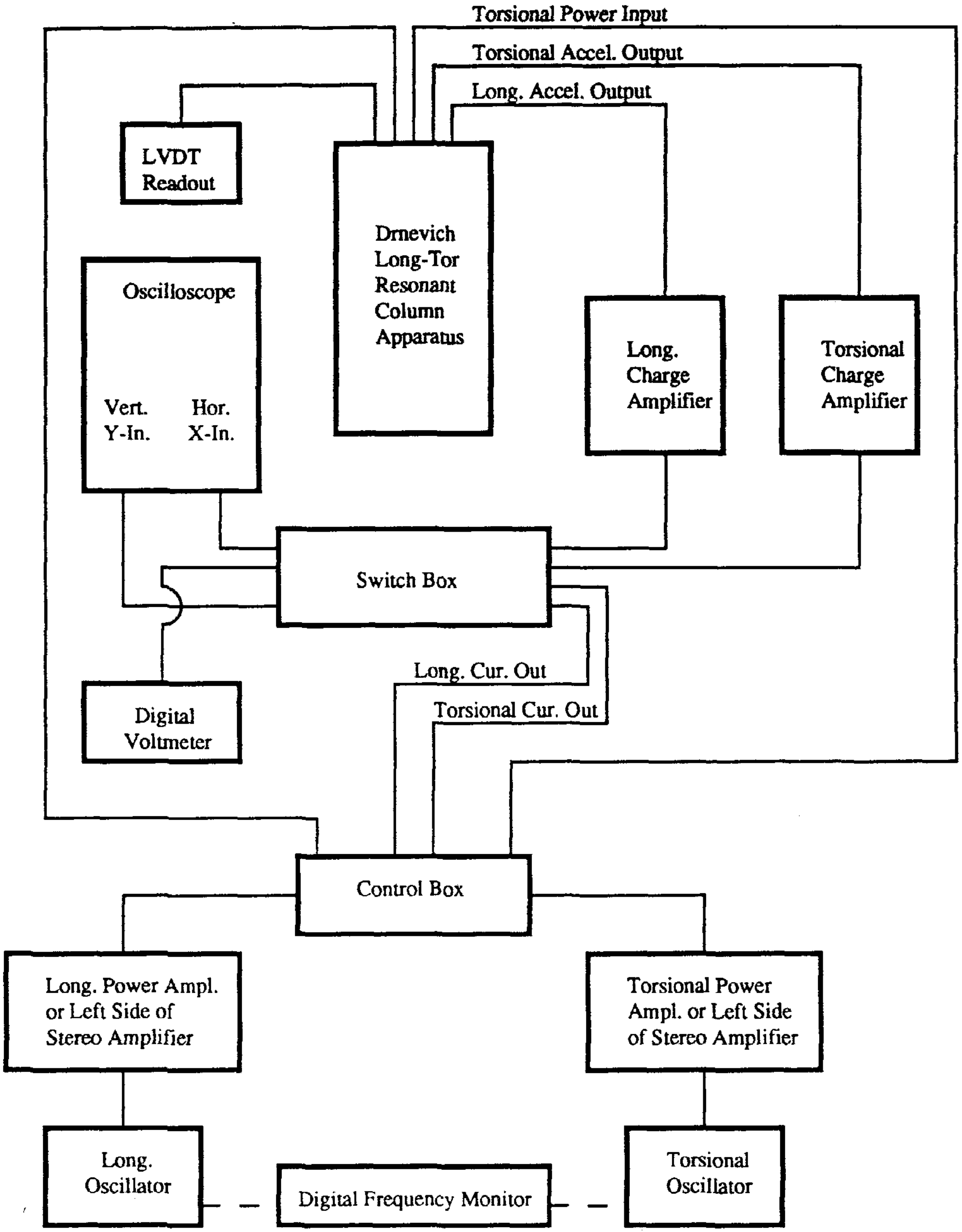

Figure 3-1 Wiring Schematic for Drnevich Long-Tor Resonant Column Apparatus

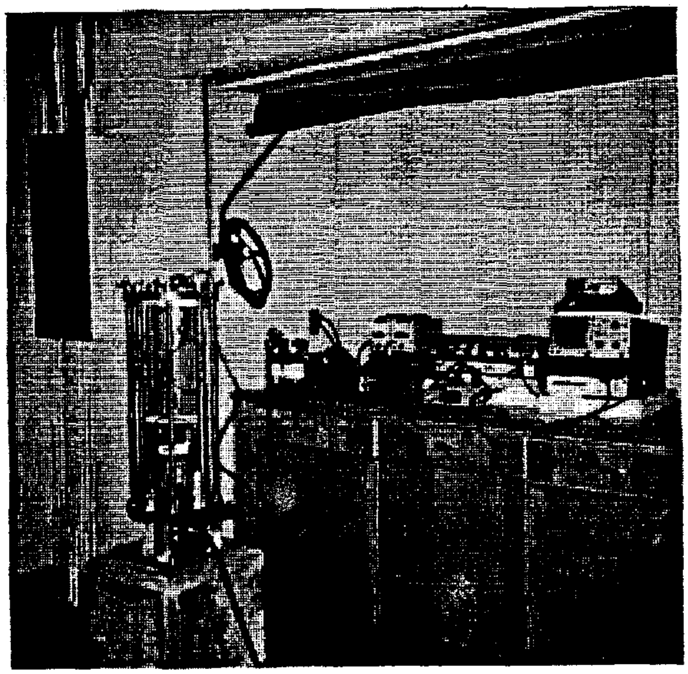

Figure 3-2 Laboratory Set-up of Drnevich Long-Tor Apparatus and Auxiliary Equipment

CHAPTER 4

RESULTS AND DISCUSSIONS OF FUNDAMENTAL STATIC TESTS

The fundamental index properties determined for samples from groups A1, A2, and A3 were specific gravity (G_s), grain size distribution (D_{10}, D_{90}, C_u), relative density (e_{max}, e_{min}), and, in some samples, the water content (w). For samples in groups B1, B2, and C, the fundamental classification tests conducted were moisture content, specific gravity, grain size distribution, shear strength (Q_u, S_u), and Atterberg limits (PI, PL, LL).

4.1 Results of Soil Group A1 (Alluvial Sand)

The samples in this soil group consist of alluvial sand. The average void ratio for the soil samples in group A1 is 0.93 in the loosest state (e_{max}) and 0.48 in the densest state (e_{min}). The grain size distribution curves in Figure 4-1 indicate that samples in group A1 have a large percentage of fine material and have a small variation of particle size, as indicated by the steep curves. An average specific gravity of 2.64 was determined for group A1. This value is reasonable in that the specific gravity for sand ranges from 2.65 to 2.67 [2]. A summary of the results of classification tests for group A1 are summarized in Table 4-1. The samples in this soil group can be described best as "poorly graded fine sand to silty sand (SP-SM)."

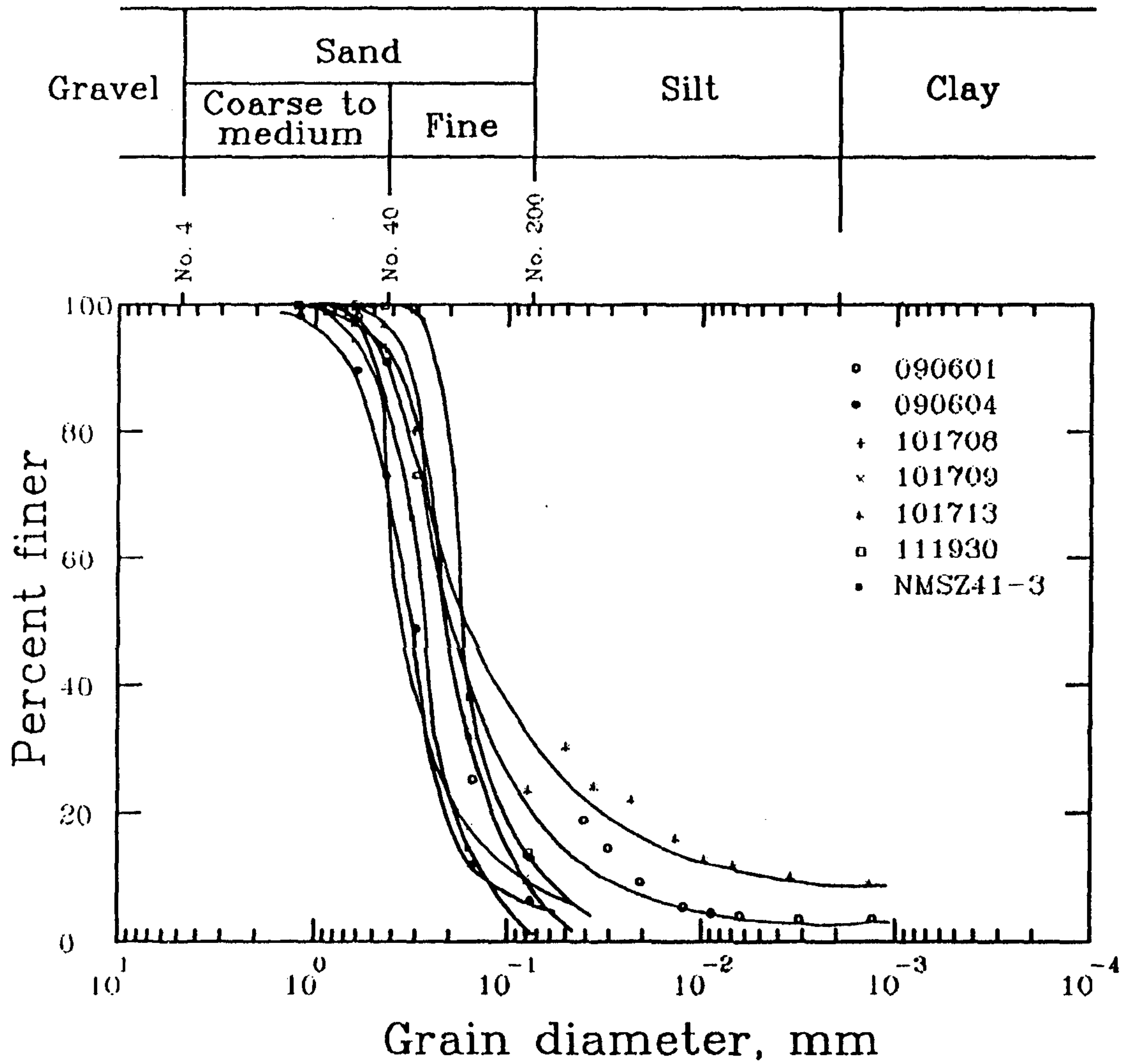

Figure 4-1 Grain Size Distribution Curves of Samples in Soil Group A1

Table 4-1 Soil Group A1: Alluvial Sand (SP-SM)

SAMPLE NO.	SOIL CLASSIFICATION	PARTICLE SHAPE	NATURAL WATER CONTENT	RELATIVE DENSITY		SPECIFIC GRAVITY	% < #200	GRAIN SIZE DISTRIBUTION (mm)			
				e_{max}	e_{min}	G_s		D_{10}	D_{60}	D_{90}	C_u
090601	Very Fine Brown Silty Sand (SM)	angular	_____	0.95	0.47	2.65	27.0	0.030	0.22	0.42	—
090604	Brown Fine to Medium Sand (SP)	angular	_____	0.84	0.45	2.64	6.4	0.140	0.34	0.62	2.4
101708	Very Fine Brown Silty Sand (SP-SM)	angular	5.5%	0.98	0.57	2.66	9.1	0.080	0.22	0.38	2.7
101709	Fine Reddish Brown Silty Sand (SP-SM)	angular	_____	0.96	0.56	2.67	10.1	0.075	0.38	0.50	5.1
101713	Very Fine Brown Silty Sand (SM)	angular	16.0%	0.90	0.40	2.60	23.7	0.004	0.20	0.31	—
111930	Very Fine Reddish Brown Silty Sand (SP-SM)	round	_____	1.11	0.53	2.61	13.8	0.060	0.16	0.22	—
NMSZ41	Very Fine Gray Sand (SP)	round to angular	_____	0.79	0.35	2.65	1.6	0.120	0.28	0.49	2.3

4.2 Results of Soil Group A2 (Terrace Sand and Gravel)

The average values of e_{max} and e_{min} for group A2 are 0.79 and 0.38, respectively. The average value of G_s obtained is 2.60. The particle size in group A2 is mostly coarse material, as shown in the grain size distribution curve in Figure 4-2, with about 10% of the material passing the number 200 sieve. A summary of the classification test results is shown in Table 4-2. A summary description of samples in this soil group is "fairly well-graded fine to coarse sand and silty sand with varying amounts of gravel (SP-SW-SM-GP)."

4.3 Results of Soil Group A3 (Jackson Fine Sand)

The water content at the time of testing was determined for all samples in group A3 and the average value was 15.4%. Because all the undisturbed soil samples were sealed and stored appropriately, these water contents should be close to the natural water contents. The average values of void ratio in the loosest state and the densest state are 0.88 and 0.47, respectively. Figure 4-3 indicates that most particles in this soil group consist of fine material, with an average D_{90} size of about 0.4 mm. An average specific gravity of 2.62 was obtained for these soil samples, as shown in Table 4-3. A typical description of samples in this soil group is "poorly graded fine sand (SP)."

Photographs from a scanning electronic microscope (SEM) of one representative samples from each of soil groups A1 (Sample No. NMSZ41), A2 (Sample No. 020146), and A3 (Sample No. 010434) are shown in Figures 4-4, 4-5, and 4-6, respectively.

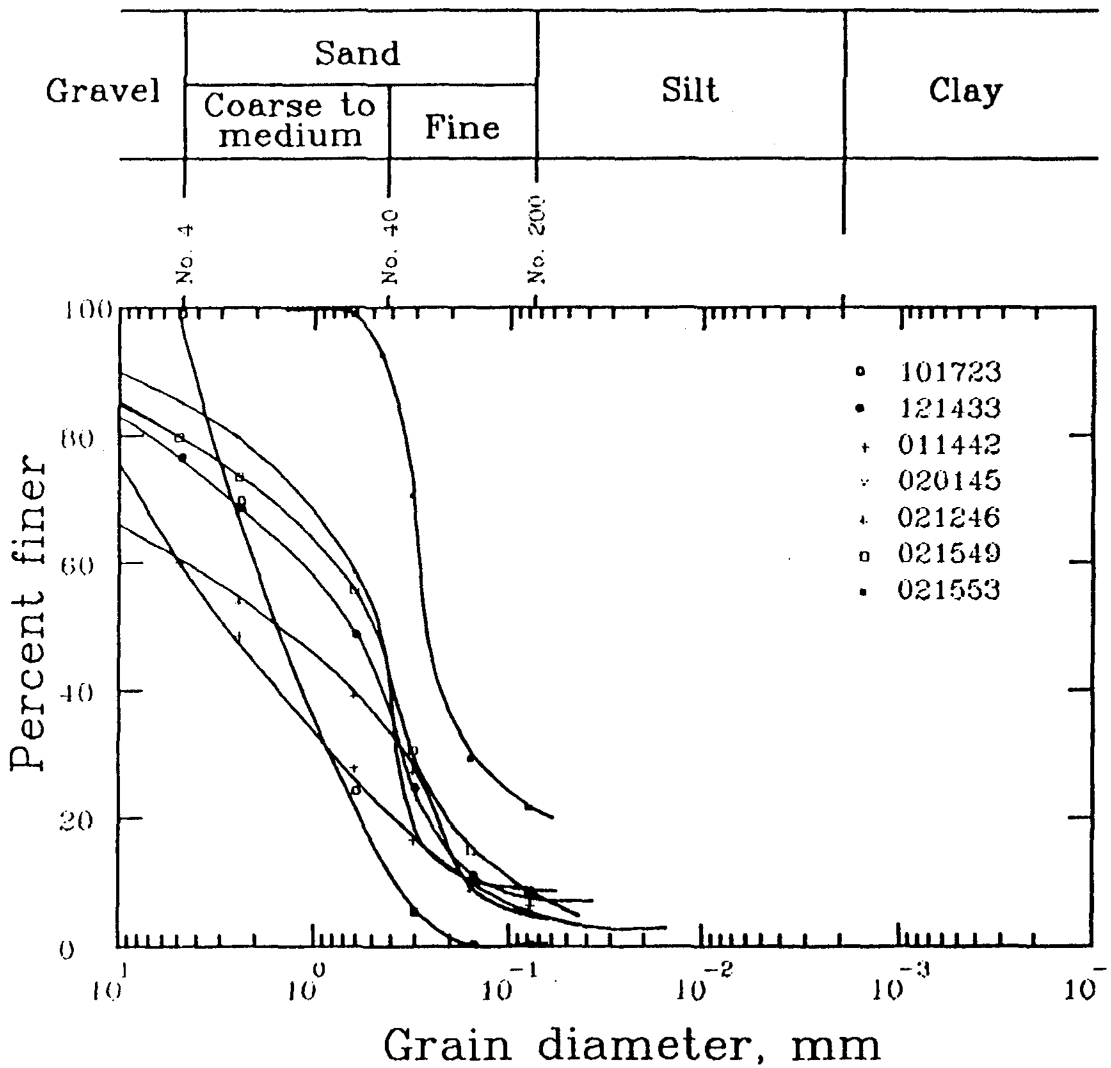

Figure 4-2 Grain Size Distribution Curves of Samples in Soil Group A2

Table 4-2 Soil Group A2: Terrace Sand And Gravel (SP-SW-SM-GP)

SAMPLE NO.	SOIL CLASSIFICATION	PARTICLE SHAPE	NATURAL WATER CONTENT	RELATIVE DENSITY		G_s	% > #4	% < #200	GRAIN SIZE DISTRIBUTION (mm)			
				e_{max}	e_{min}				D_{10}	D_{60}	D_{90}	C_u
101723	Brown Coarse Sand (SP-SW)	round to angular	____	0.67	0.40	2.62	1.1	1.6	0.40	2.0	4.1	5.0
121433	Brown Coarse to Medium Sand with Gravel (SM-SW-GP)	round	____	0.81	0.35	2.60	23.3	8.3	0.12	1.1	__	9.2
011442	Brown Silty Coarse to Medium Terrace Sand and Gravel (SW-SM-GP)	round	2.0%	0.77	0.47	2.59	39.8	10.1	0.15	4.7	__	31.3
020145	Dark Brown Fine to Coarse Terrace Sand with Gravel and Silt (SM-SP-GP)	round	8.7%	0.82	0.43	2.56	14.4	12.2	0.15	0.62	10.0	4.1
021246	Brown Coarse to Fine Sand and Gravel (SW-GP)	round to angular	____	0.71	0.30	2.62	39.5	6.1	0.16	4.5	__	28.1
021549	Light Gray Silty Coarse to Fine Sand and Gravel (SM-SW-GP)	round to angular	15.4%	0.78	0.20	2.57	20.2	11.1	0.09	0.79	__	8.8
021553	Dark Brown Silty Sand (SM)	round to angular	7.6%	0.95	0.53	2.64	__	17.9	__	0.27	0.4	__

G_s = Specific Gravity

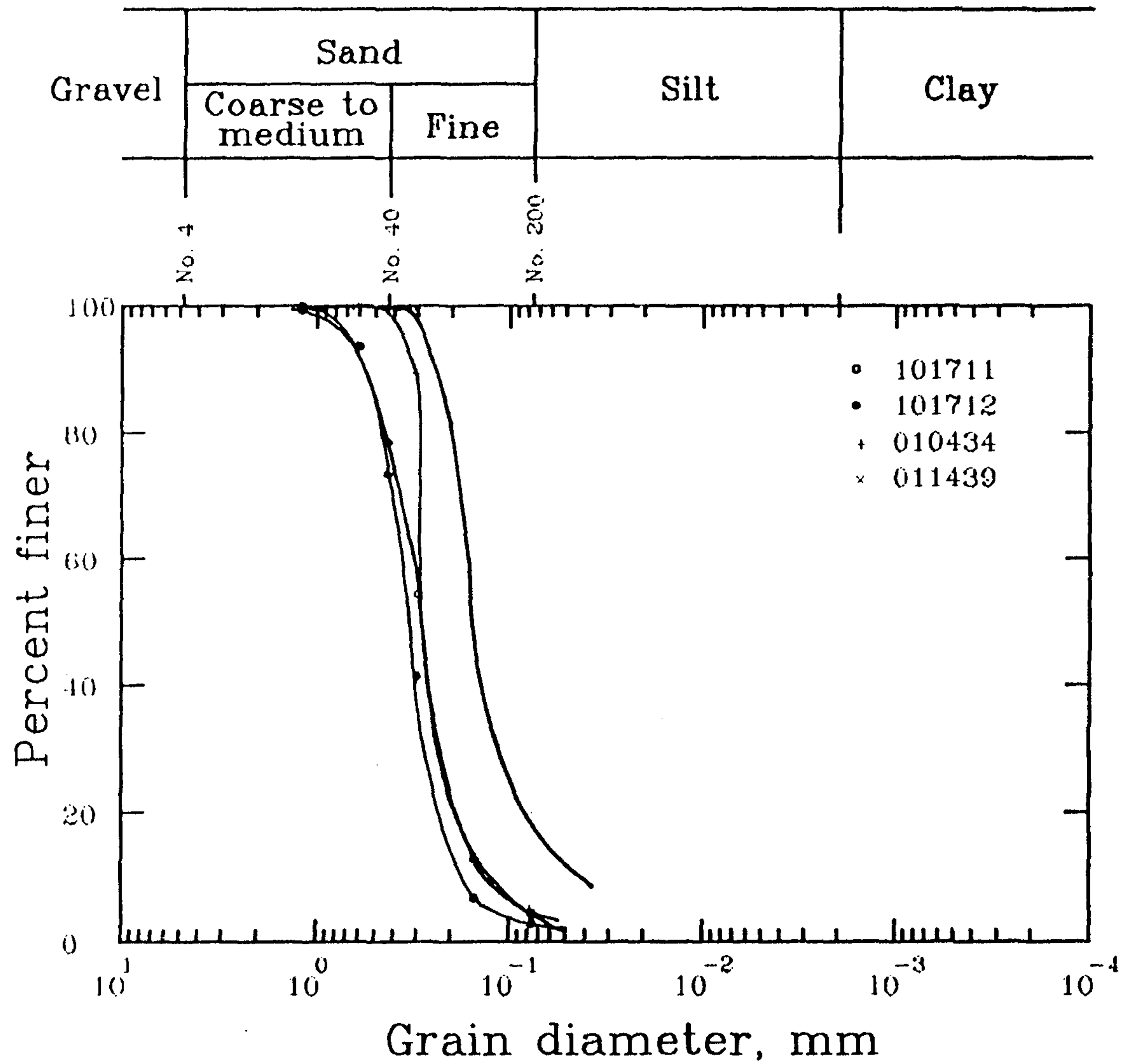

Figure 4-3 Grain Size Distribution Curves of Samples in Soil Group A3

Table 4-3 Soil Group A3: Jackson Fine Sand (SP-SM)

SAMPLE NO.	SOIL CLASSIFICATION	PARTICLE SHAPE	NATURAL WATER CONTENT	RELATIVE DENSITY		SPECIFIC GRAVITY	% < #200	GRAIN SIZE DISTRIBUTION (mm)			
				e_{max}	e_{min}	G_s		D_{10}	D_{60}	D_{90}	C_u
101711	Dark Gray Fine Sand with Trace of Chalk (SP)	round	17.1%	0.71	0.42	2.62	6.1	0.12	0.30	0.55	2.5
101712	Gray Fine Sand (SP)	round to angular	16.8%	0.67	0.38	2.54	3.0	0.17	0.33	0.54	1.9
010434	Very Fine White Sand (SP)	round to angular	19.1%	0.96	0.55	2.66	7.0	0.13	0.29	0.30	2.2
011439	Light Pink Fine Silty Sand (SP-SM)	round to angular	8.5%	1.16	0.54	2.64	15.9	0.04	0.16	0.21	—

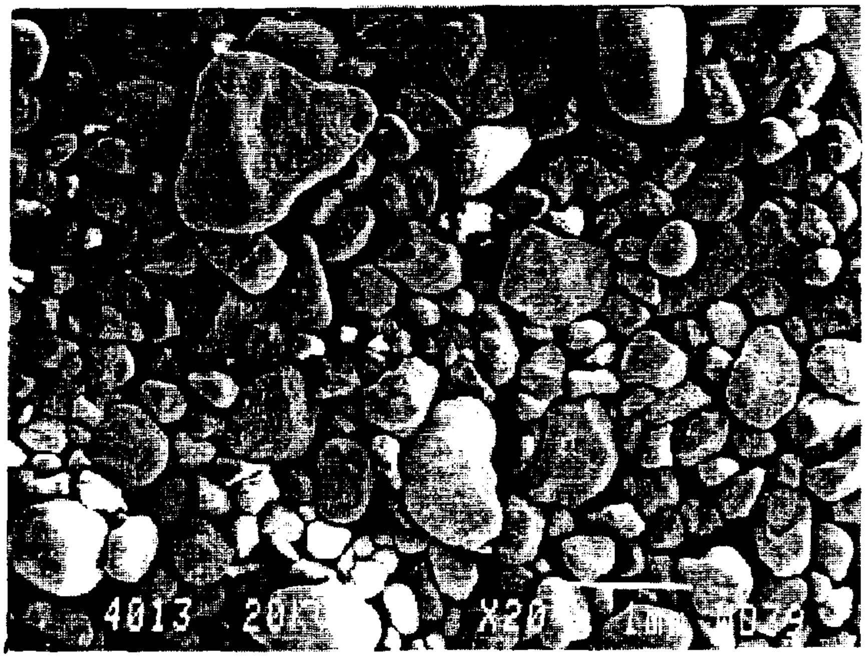

Figure 4-4 Scanning Electronic Microscope of Sample NMSZ41 in Soil Group A1

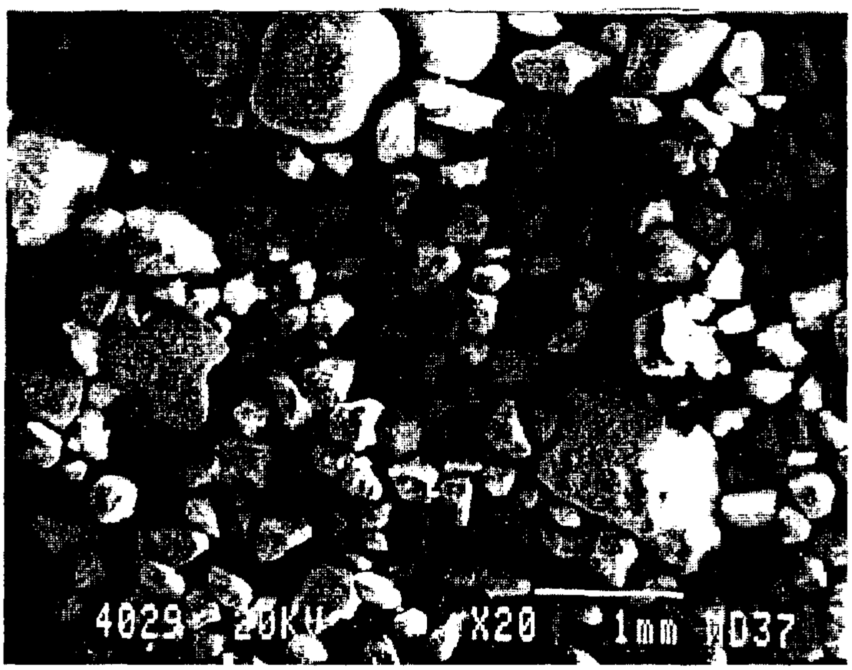

Figure 4-5 Scanning Electronic Microscope of Sample 020146 in Soil Group A2

Figure 4-6 Scanning Electronic Microscope of Sample 010434 in Soil Group A3

4.4 Results of Soil Group B1 (Silty to Sandy Clay)

The grain size distribution curves for samples from this group are plotted in Figure 4-7. These curves indicate that most of the samples consist of fine particles with a small percentage of the particles retained on the number 200 sieve. Table 4-4 presents the summary of the index test results for soil group B1. The average values of specific gravity, plasticity index, and liquid limit obtained are 2.58%, 13.9%, and 32.7%, respectively. The approximate shear strength of the cohesive samples was estimated with a hand penetrometer and ranged from 1 ksf to 4.5 ksf. The liquid limits of the samples in group B1 are all below 50%.

4.5 Results of Soil Group B2 (High Plasticity Fat Clay)

The average natural water content obtained for this group is 23.4%. The value of the specific gravity is remarkably below the general expected values, which has a range of 2.68 to 2.75 (Bowles 1988). The plasticity index and liquid limit determined has an average value of 43.7% and 69.6%, respectively. All the clay samples in soil Group B2 have a liquid limit greater than 50%. The shear strength indicated by a hand penetrometer ranged from 0.95 ksf to 4.25 ksf. The grain size distribution curve is shown in Figure 4-8 and the summary of test results for the individual samples are presented in Table 4-5.

4.6 Results of Soil Group C (Loess)

All the samples in this group consist of loess material. The average water content obtained was 21.2%. The value of average

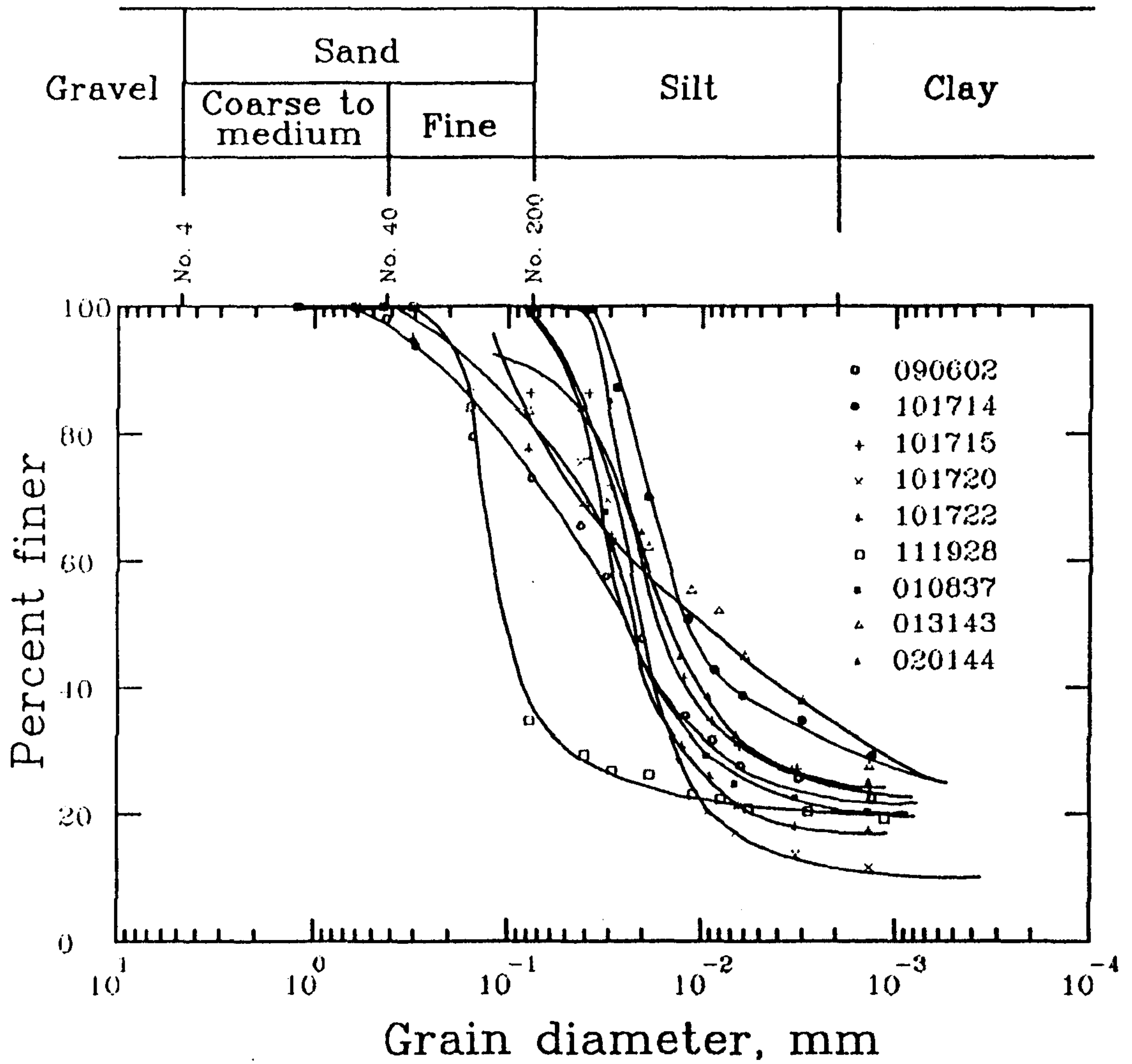

Figure 4-7 Grain Size Distribution Curves of Samples in Soil Group B1

Table 4-4 Soil Group B1: Silty to Sandy Clay (CL-ML)

SAMPLE NO.	SOIL CLASSIFICATION	NATURAL WATER CONTENT	SPECIFIC GRAVITY G_s	% > #200	ATTERBURG LIMITS (%)			SHEAR STRENGTH (KSF)	
					LL	PL	I_p	UCS*	HP*
090602	Brown Sandy to Silty Clay (CL)	———	2.64	26.8	31.6	16.3	15.3	—	———
101714	Very Stiff Brown Silty Clay (CL)	22.3%	2.57	0.6	40.5	24.0	16.5	2.40	3.00
101715	Dark Brown Sandy to Silty Caly (CL)	26.0%	2.53	13.5	36.0	16.7	19.3	0.24	2.75
101720	Brown Silty Caly to Clayey Silt (CL-ML)	19.2%	2.60	0.1	32.2	20.0	12.2	1.60	2.00
101722	Gray and Brown Silty to Sandy Clay and Clayey Silt (CL-ML)	18.1%	2.60	22.1	31.7	16.4	15.3	4.40	4.50
111928	Brown Clayey to Sandy Silt (CL-ML)	15.4%	2.63	66.2	25.8	15.5	10.3	2.00	3.75
010837	Soft Dark Brown and Gray Silty Clay (CL)	32.1%	2.56	0.7	35.7	24.2	11.5	—	1.00
013143	White Hard Silty to Sandy Clay (CL)	11.4%	2.57	16.4	31.8	18.3	13.5	3.50	3.75
020144	Hard Dark Brown Silty Clay (CL)	19.1%	2.51	0.8	29.2	18.1	11.1	—	2.65

*HP: Hand Penetrometer Test
*UCS: Unconfined Compression Test

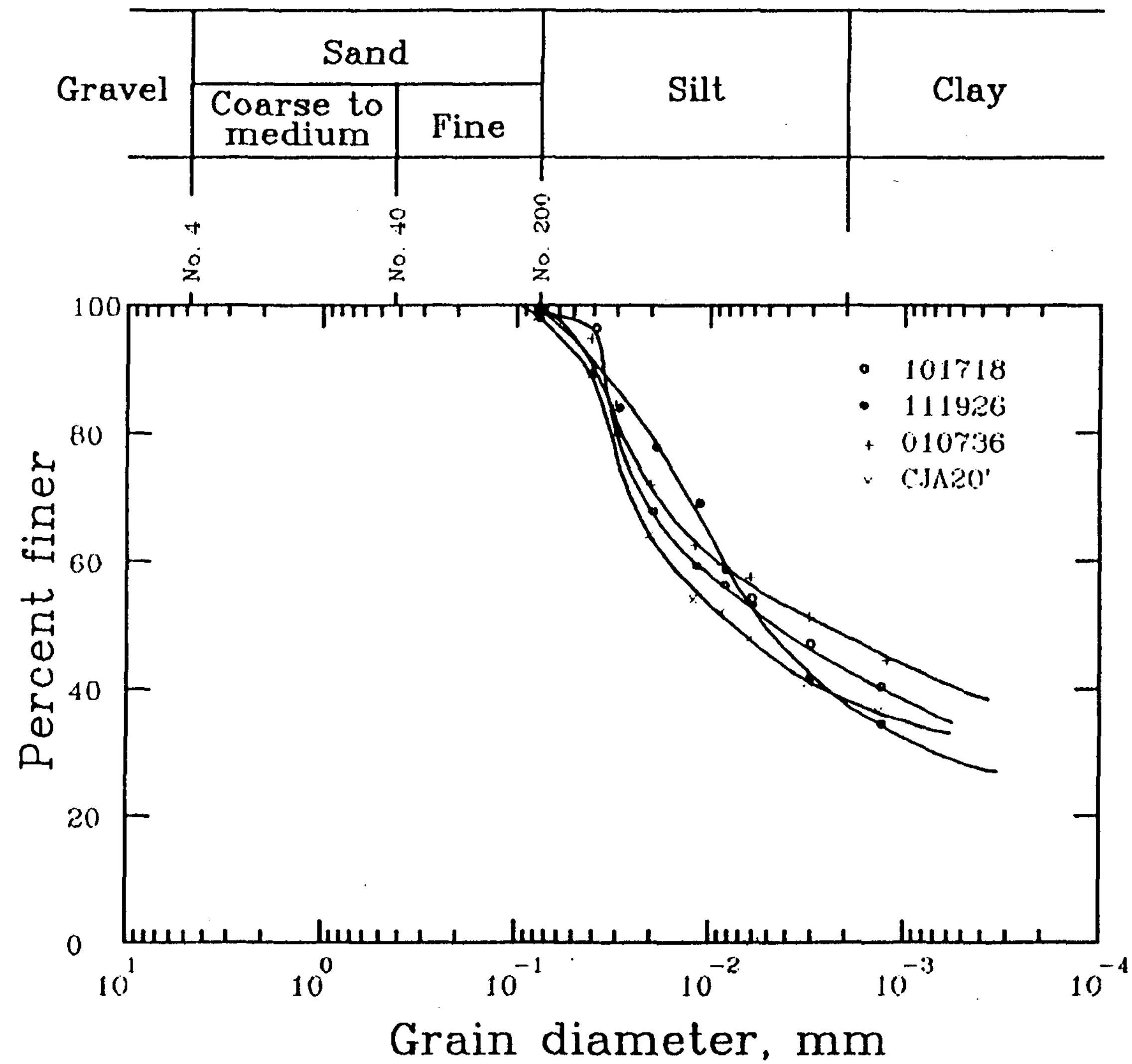

Figure 4-8 Grain Size Distribution Curves of Samples in Soil Group B2

Table 4-5 Soil Group B2: High Plasticity Fat Clay (CL-CH, Jackson Clay)

SAMPLE NO.	SOIL CLASSIFICATION	NATURAL WATER CONTENT	SPECIFIC GRAVITY G_s	% > #200	ATTERBURG LIMITS (%)			SHEAR STRENGTH (KSF)	
					LL	PL	I_p	UCS*	HP*
101718	Gray Clay (CL-CH)	21.6%	2.44	2.0	64.3	24.4	39.9	0.60	2.10
111926	Dark Gray Medium Silty Clay (CL-CH)	23.0%	2.41	0.2	76.6	25.7	50.9	0.70	0.95
010736	Dark Gray Clay with White Chalk (CL-CH)	26.3%	2.46	0.4	84.0	27.8	56.2	0.55	2.00
CJA20'	Gray Silty Clay (CL-CH)	22.7%	2.40	1.8	53.8	26.0	27.8	7.50	4.25

*HP: Hand Penetrometer Test
*UCS: Unconfined Compression Test

specific gravity is 2.58. The samples in this group consist of fine particles, which is typical for loessial soils (Figure 4-9). The average plasticity index determined is 11.1% and the shear strength of the natural soil samples has a range of 2.50 ksf to 3.00 ksf. A summary of the test results for individual samples in this group is presented in Table 4-6.

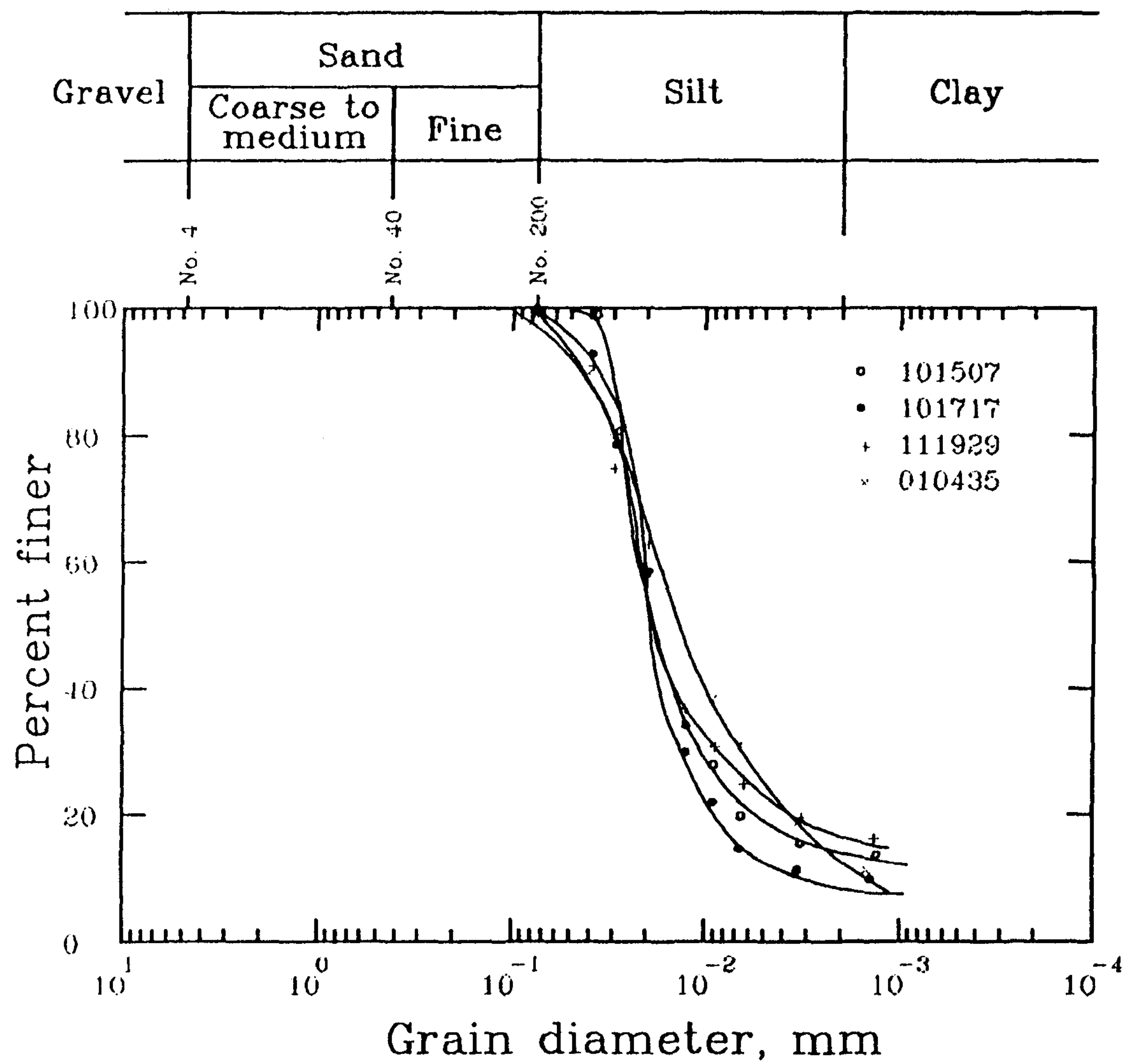

Figure 4-9 Grain Size Distribution Curves of Samples in Soil Group C

Table 4-6 Soil Group C: Silty Loess Soil (ML, ML-CL)

SAMPLE NO.	SOIL CLASSIFICATION	NATURAL WATER CONTENT	SPECIFIC GRAVITY G_s	% > #200	ATTERBURG LIMITS (%)			SHEAR STRENGTH (KSF)	
					LL	PL	I_p	UCS*	HP*
101507	Brown Clayey Silt (ML-CL)	18.9%	2.60	0.2	34.0	24.7	9.3	___	____
101717	Hard Gray Clayey Silt (ML-CL)	22.7%	2.55	0.0	28.3	21.1	7.2	2.8	3.00
111929	Brown Clayey Silt (ML)	21.7%	2.62	0.6	38.9	19.1	19.8	2.5	2.75
010435	Hard Dark Gray Clayey Silt (ML-CL)	21.6%	2.53	2.0	37.6	29.5	8.1	___	2.50

*HP: Hand Penetrometer Test
*UCS: Unconfined Compression Test

CHAPTER 5

LOW STRAIN DYNAMIC PROPERTIES OF SOILS

Hardin and Blake [12] proposed that the shear modulus, G, of soils can be expressed as a function of several parameters.

$$G = f(\sigma_o, e, A, t, H, f, C, \theta, \tau_o, S, T) \quad (5\text{-}1)$$

where σ_o = average effective confining pressure;

e = void ratio;

A = amplitude of shearing strain;

t = secondary effects that are functions of time and magnitude of stress increment;

f = frequency of vibration;

C = grain characteristics;

θ = soil structure;

τ_o = octahedral shearing stress;

S = degree of saturation; and

T = temperature.

Among these parameters, those that are most influential to shear modulus are effective confining pressure (σ_o, environment factor) and void ratio (e, constituent factor). These parameters are known to have substantial effect on sand materials and were examined in this study. The constituent factors to be studied for clay samples are shear strength (s_u) and Atterberg limits (PI, PL, LL). The range of low strain in this study is at or below 0.001%.

5.1 Seismic Characteristics of Soil Groups A1, A2, and A3

5.1.1 Shear Modulus and Shear-Wave Velocity

The shear modulus of soils can be generally expressed as

$$G = f(e)\,\sigma^n \quad \text{for sand} \qquad (5\text{-}2)$$

$$G = f(S_u, PL, LL)\,\sigma^n \quad \text{for clay} \qquad (5\text{-}3)$$

where G = shear modulus
e = void ratio (dimensionless)
σ = confining pressure
n = constant
S_u = shear strength
PL = plastic limits
LL = liquid limits

The variations of shear modulus as a function of confining pressure for a representative sample from each of the soil groups A1, A2, and A3 is shown in Figures 5-1, 5-2, and 5-3, respectively. The data obtained for the corresponding samples for low-strain shear-wave velocity, shear modulus, and damping ratio are shown in Tables 5-1 to 5-3. The rest of the test results are shown in Appendix A. As indicated in Figure 5-1, for the same confining pressure, a higher shear modulus is obtained from denser soils, which generally agrees with the results from previous studies.

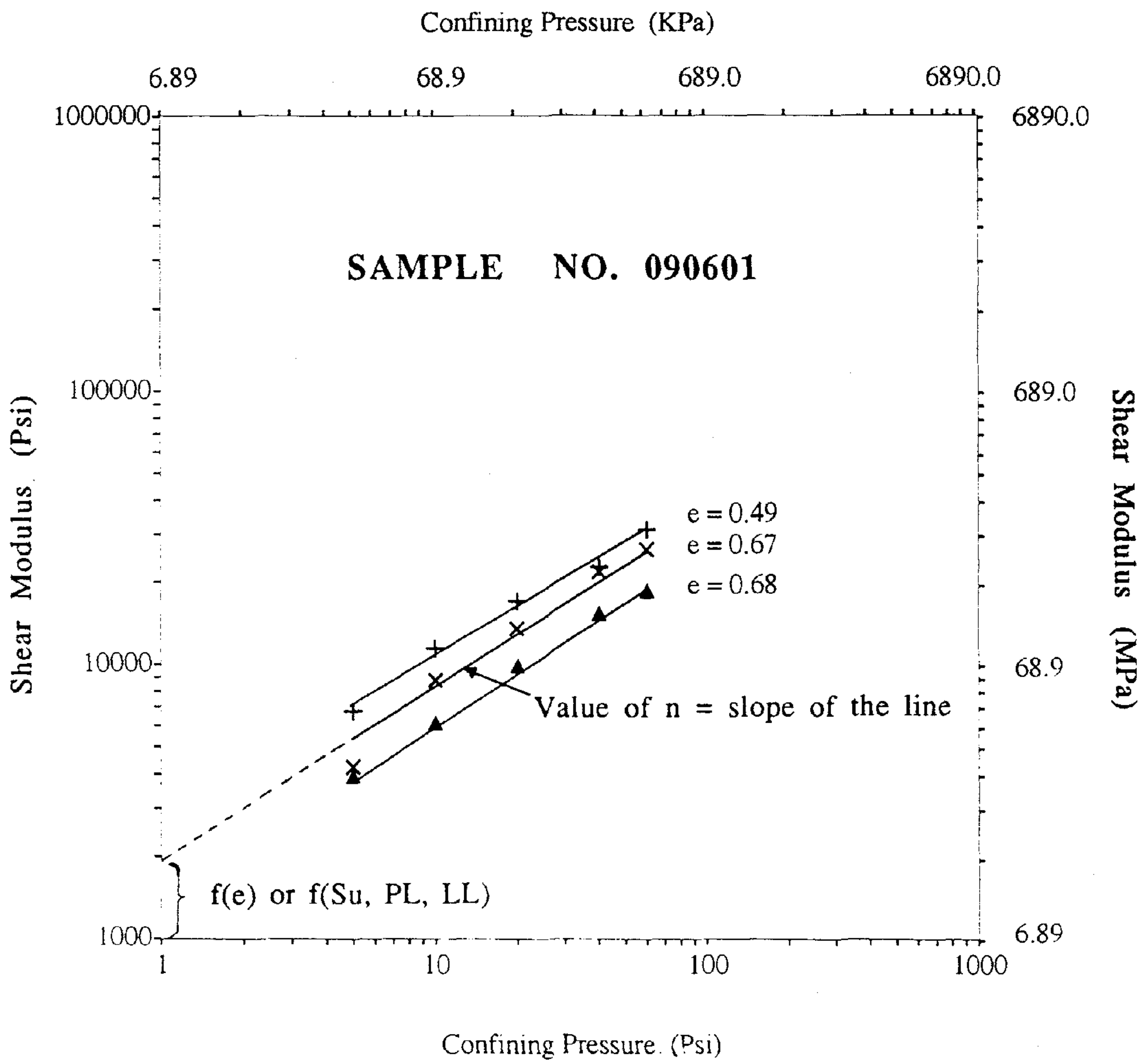

Figure 5-1 Shear Modulus versus Confining Pressure for Sample No. 090601

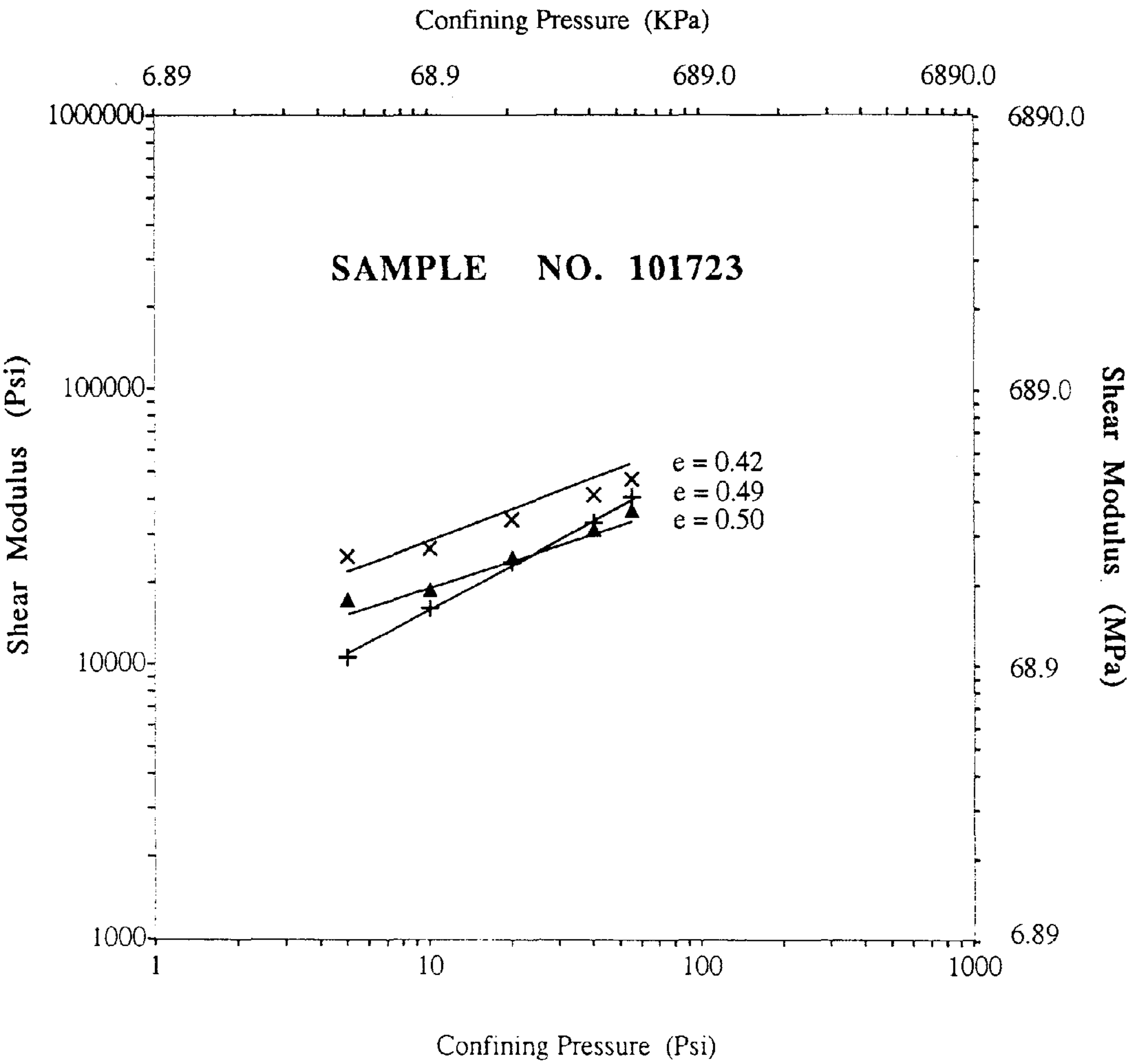

Figure 5-2 Shear Modulus versus Confining Pressure for Sample No. 101723

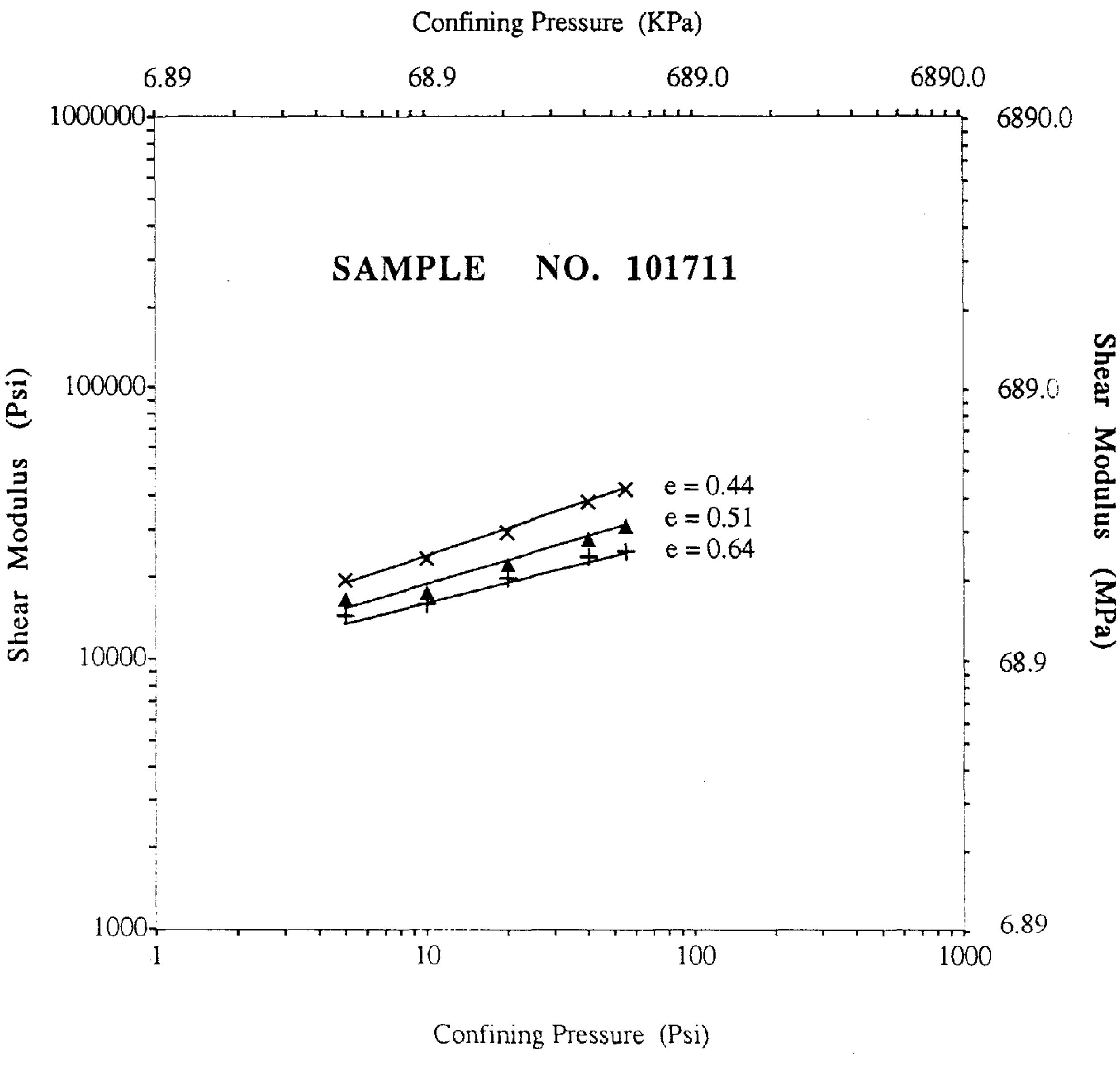

Figure 5-3 Shear Modulus versus Confining Pressure for Sample No. 101711

Table 5-1 Low-Strain Seismic Characteristics for Sample 090601

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.67	Shear Wave Velocity	m/sec	131	188	234	296	324
		ft/sec	430	617	768	971	1063
	Shear Modulus	MPa	29	60	93	150	181
		psi	4196	8681	13456	21702	26188
	Damping Ratio	%	1.85	1.15	0.95	0.63	0.58
0.68	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	130	162	206	256	282
		ft/sec	427	531	676	840	925
	Shear Modulus	MPa	27	42	68	106	128
		psi	3907	6077	9839	15337	18520
	Damping Ratio	%	1.35	1.20	0.95	0.82	0.78
0.49	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	163	211	257.5	296.2	347
		ft/sec	535	692	845	972	1138
	Shear Modulus	MPa	46.5	78.8	117.5	156	214
		psi	6728	11401	17000	22571	30962
	Damping Ratio	%	1.25	0.85	0.71	0.50	0.55

Table 5-2 Low-Strain Seismic Characteristics for Sample 101723

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.42	Shear Wave Velocity	m/sec	312.6	324	365	405	427
		ft/sec	1026	1063	1198	1329	1401
	Shear Modulus	MPa	171.5	183.5	232	285	323
		psi	24813	26550	33567	41235	46733
	Damping Ratio	%	0.68	0.76	0.68	0.63	0.59
0.50	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	274	284.3	326.2	365.2	395.4
		ft/sec	899	933	1070	1198	1297
	Shear Modulus	MPa	119.5	130	171	215	252
		psi	17290	18809	24741	31107	36460
	Damping Ratio	%	0.80	0.72	0.65	0.53	0.52
0.49	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	212.1	262.2	319	376	418
		ft/sec	696	860	1047	1234	1371
	Shear Modulus	MPa	73	111	164	227	281
		psi	105612	16060	23728	32843	40656
	Damping Ratio	%	1.37	1.10	0.86	0.63	0.62

Table 5-3 Low-Strain Seismic Characteristics for Sample 101711

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.44	Shear Wave Velocity	m/sec	278	306	340	387	408
		ft/sec	912	1004	1115	1270	1339
	Shear Modulus	MPa	134	161	201	259	289
		psi	19388	23294	29082	37473	41814
	Damping Ratio	%	2.99	0.95	0.82	0.76	0.88
0.51	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	265	270.9	305.1	341	361.2
		ft/sec	869	889	1001	1119	1185
	Shear Modulus	MPa	114.2	120.3	153.1	189.9	213
		psi	16523	17406	22151.2	27476	308178
	Damping Ratio	%	0.87	0.85	0.73	0.66	0.65
0.64	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	252	264	295	323	331
		ft/sec	827	866	968	1060	1086
	Shear Modulus	MPa	99.6	110	138	164	172
		psi	14411	15915	19966	23728	24886
	Damping Ratio	%	0.90	0.84	0.79	0.73	0.72

The values of "n," which represent the effect of confining pressure, and the values of f(e) or f(Su, PL, LL), which represents the effect of soil constituent, can be determined from the log-log plot as shown in Figure 5-1.

The range of n and the average value of n for each soil group is presented in Table 5-4. Even though the values of n range from 0.25 to 0.98 for the natural soils tested, it can be seen that the average value of n for all the soil groups ranges from 0.5 to 0.54, essentially the same as recommended by the Hardin Equation (n = 0.5). This is an interesting finding that the average behavior of a group of natural sediments is essentially the same as that of a group of ideal laboratory in terms of soil samples effect of confining pressure [5]. The results support Hardin's suggestion: shear modulus of sand is proportional to square root of confining pressure. The relationship of shear modulus and void ratio for all the samples in soil group A was also studied. This was done by nonlinear regression analysis using Hardin's shear modulus equation as a reference equation. The parameters in the shear modulus equation were determined by using subroutine RNLIN [16]. Then graphs were plotted using the obtained equations. The equation for each group and Hardin's equation are shown in Table 5-5. The graphs are shown in Figures 5-4 to 5-6 for soil groups A1 to A3. From Table 5-5, the differences in the estimated shear modulus for groups A1, A2, and A3 using the derived equations in the study and existing Hardin's equation range from about -9% to 12%.

The empirical equations derived in this study for estimating shear modulus of various representative soils in the northern Mississippi embayment region are:

$$G = \frac{2425\ (2.25-e)}{1+e}(\sigma_o)^{0.5} \qquad \text{for group A1} \qquad (5\text{-}4)$$

$$G = \frac{2484\ (2.23-e)}{1+e}(\sigma_o)^{0.5} \qquad \text{for group A2} \qquad (5\text{-}5)$$

$$G = \frac{2986\ (2.15-e)}{1+e}(\sigma_o)^{0.5} \qquad \text{for group A3} \qquad (5\text{-}6)$$

where G = shear modulus in psi

σ_o = confining pressure in psi

Table 5-4 Range of Value of n

Soil Group	Minimum n	Maximum n	Average n
A1	0.27	0.98	0.53
A2	0.32	0.72	0.54
A3	0.25	0.61	0.50

Table 5-5 Comparison of Shear Modulus Equation

Confining Pressure (psi)	Individual Equation	Average Equation	Hardin's Equation
Group A1			
5	$\frac{2305\ (2.34-e)}{1+e}(\sigma_o)^{0.5}$		
10	$\frac{2435\ (2.26-e)}{1+e}(\sigma_o)^{0.5}$		
20	$\frac{2472\ (2.24-e)}{1+e}(\sigma_o)^{0.5}$	$\frac{2425\ (2.25-e)}{1+e}(\sigma_o)^{0.5}$	$\frac{2630\ (2.17-e)}{1+e}(\sigma_o)^{0.5}$
40	$\frac{2468\ (2.21-e)}{1+e}(\sigma_o)^{0.5}$		
60	$\frac{2447\ (2.20-e)}{1+e}(\sigma_o)^{0.5}$		
Group A2			
5	$\frac{2567\ (2.28-e)}{1+e}(\sigma_o)^{0.5}$		
10	$\frac{2477\ (2.24-e)}{1+e}(\sigma_o)^{0.5}$		
20	$\frac{2492\ (2.22-e)}{1+e}(\sigma_o)^{0.5}$	$\frac{2484\ (2.23-e)}{1+e}(\sigma_o)^{0.5}$	$\frac{2630\ (2.17-e)}{1+e}(\sigma_o)^{0.5}$
40	$\frac{2449\ (2.20-e)}{1+e}(\sigma_o)^{0.5}$		
60	$\frac{2433\ (2.20-e)}{1+e}(\sigma_o)^{0.5}$		
Group A3			
5	$\frac{2977\ (2.29-e)}{1+e}(\sigma_o)^{0.5}$		
10	$\frac{2800\ (2.25-e)}{1+e}(\sigma_o)^{0.5}$		
20	$\frac{2697\ (2.21-e)}{1+e}(\sigma_o)^{0.5}$	$\frac{2986\ (2.15-e)}{1+e}(\sigma_o)^{0.5}$	$\frac{2630\ (2.17-e)}{1+e}(\sigma_o)^{0.5}$
40	$\frac{2695\ (2.15-e)}{1+e}(\sigma_o)^{0.5}$		
60	$\frac{3762\ (1.86-e)}{1+e}(\sigma_o)^{0.5}$		

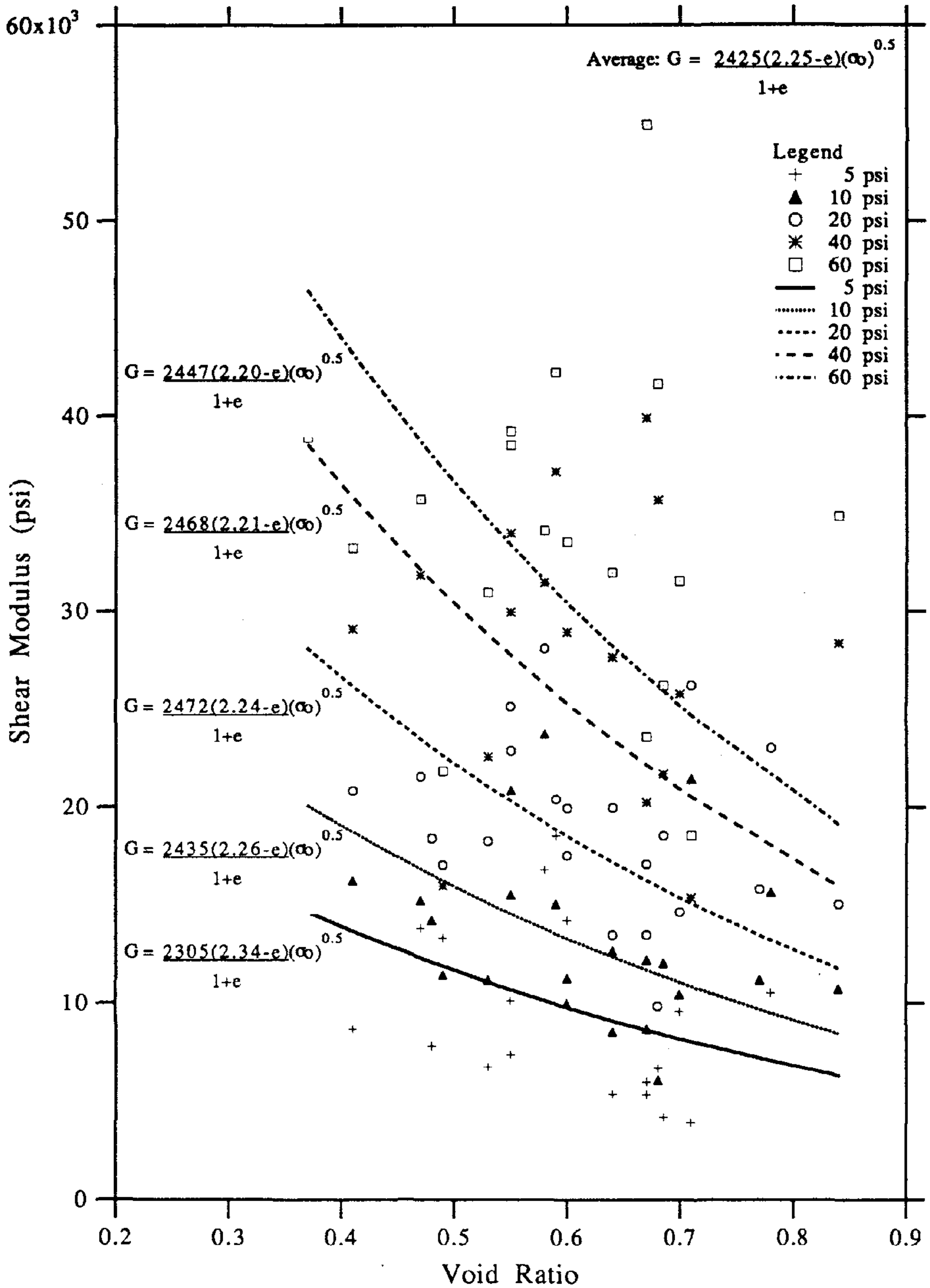

Figure 5-4 Shear Modulus vs Void Ratio for Group A1

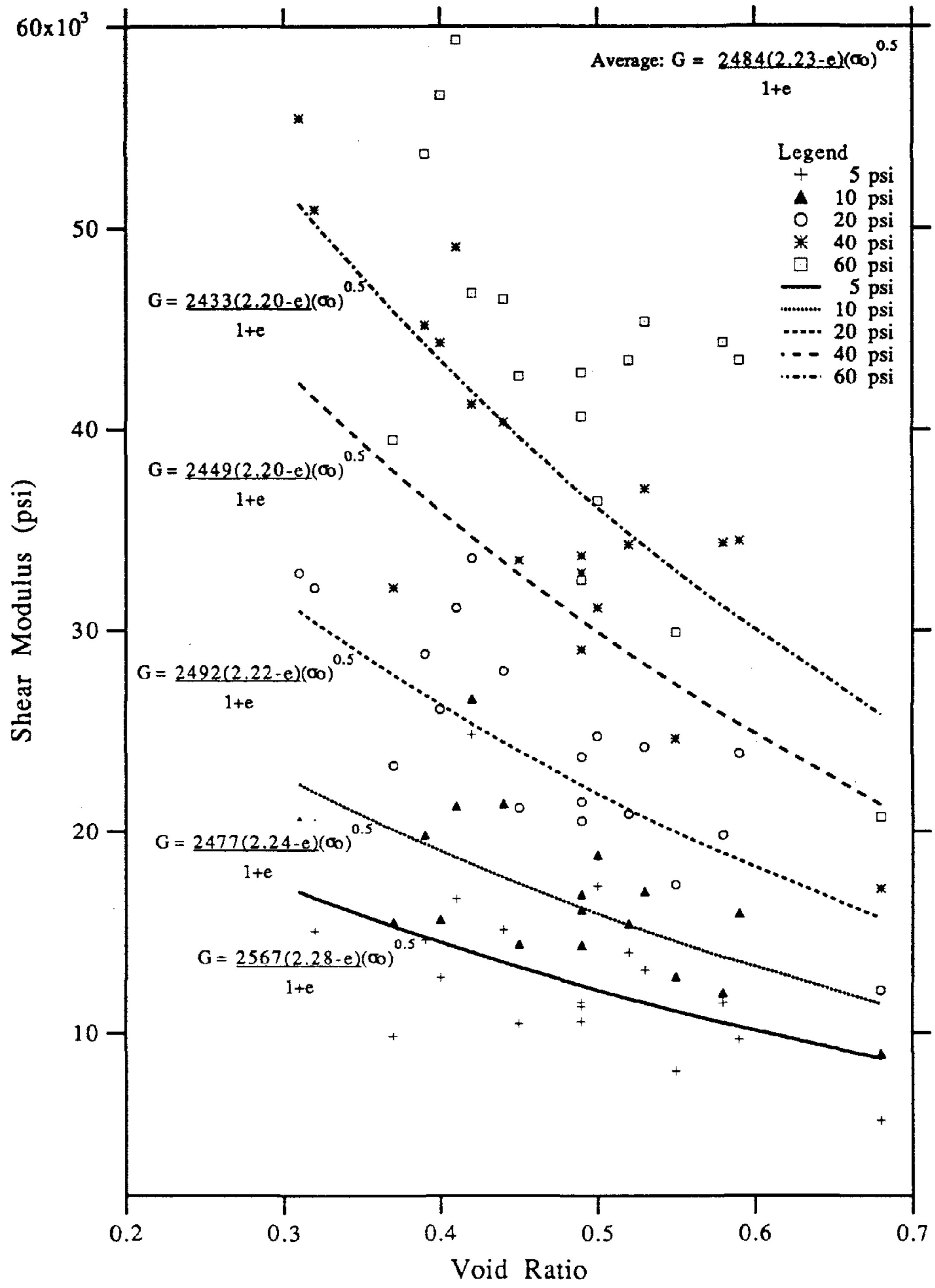

Figure 5-5 Shear Modulus vs Void Ratio for Group A2

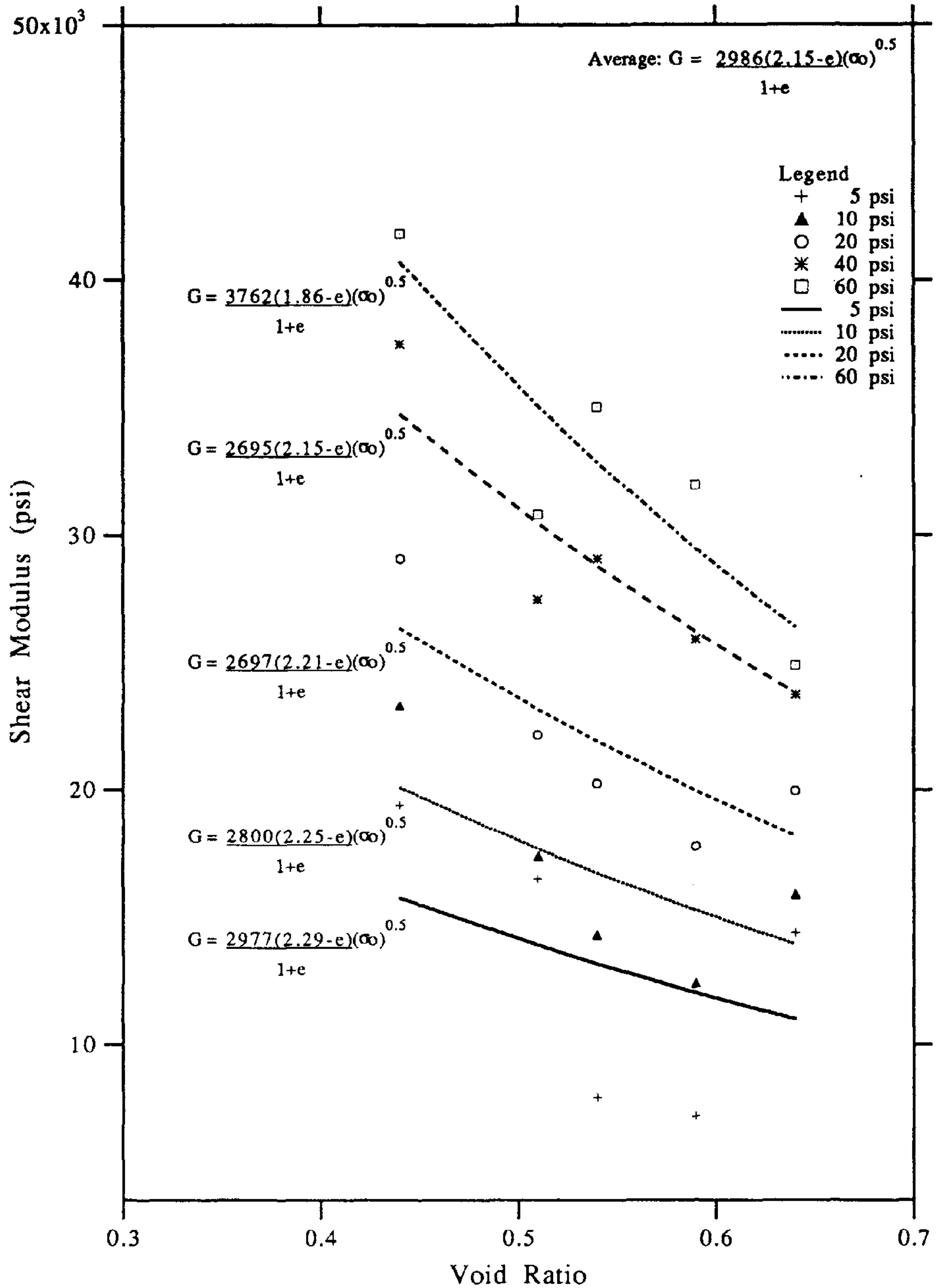

Figure 5-6 Shear Modulus vs Void Ratio for Group A3

The shear-wave velocity is regarded as an important property in the study of strong ground motion. It is well known that the shear-wave velocity is directly influenced by the density of the soils. Thus, this effect was studied by plotting shear-wave velocity as a function of void ratio for the various soil groups in group A and are shown in Figures 5-7 to 5-9. The equations developed to estimate shear-wave velocity of the representative soils are presented below (see also Table 5-6, which also includes Hardin's shear-wave velocity equation for comparison) [12].

$$V_s = [139-(24.1)e](\sigma_o)^{0.25} \quad \text{for group A1} \quad (5\text{-}7)$$

$$V_s = [183-(110.8)e](\sigma_o)^{0.25} \quad \text{for group A2} \quad (5\text{-}8)$$

$$V_s = [207-(132.8)e](\sigma_o)^{0.25} \quad \text{for group A3} \quad (5\text{-}9)$$

where V_s = shear-wave velocity in ft/sec

σ_o = confining pressure in psf

The differences in the estimated shear-wave velocity for soil groups A1, A2, and A3 using the derived average equations and Hardin's equation range form about -13% to 12%. These differences indicate that the average equations derived in the study based on the local soils may be more reliable and accurate than using Hardin equations and other empirical equations for seismic studies in the NMSZ.

5.1.2 Damping Ratio

The damping ratio characteristics of soils are illustrated by plotting the damping ratio against the confining pressure, as shown

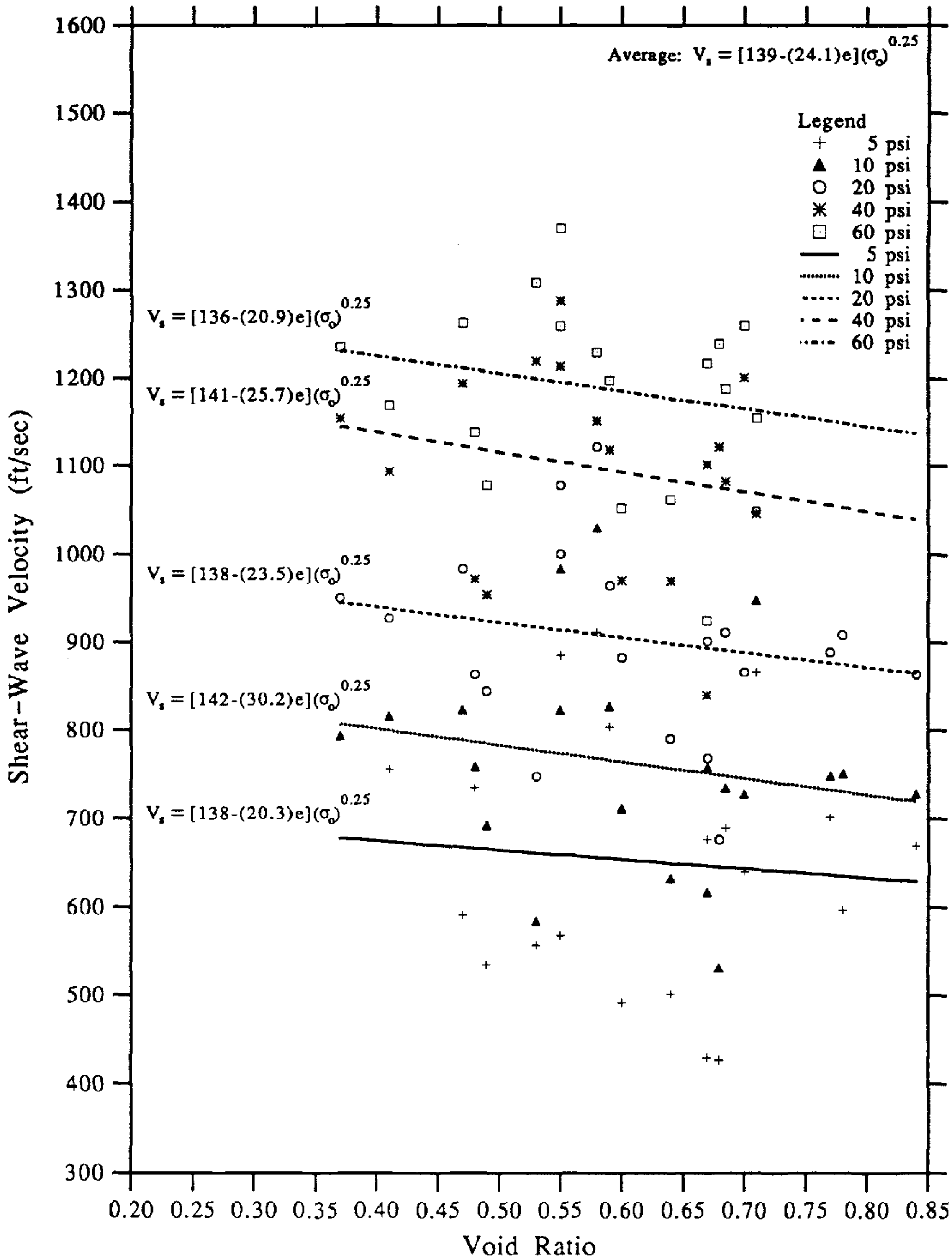

Figure 5.7 Shear-Wave Velocity vs Void Ratio for Group A1

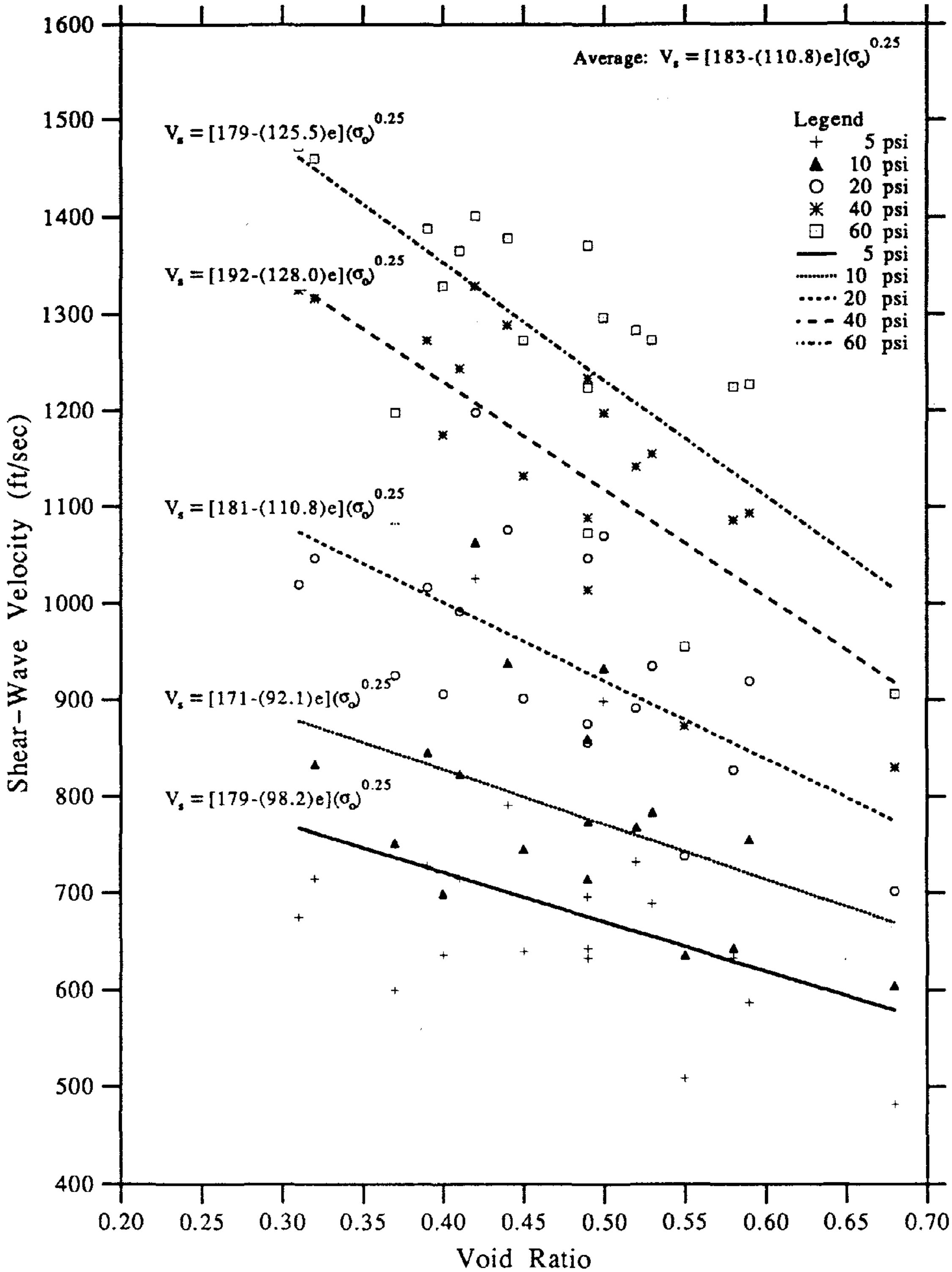

Figure 5-8 Shear-Wave Velocity vs Void Ratio for Group A2

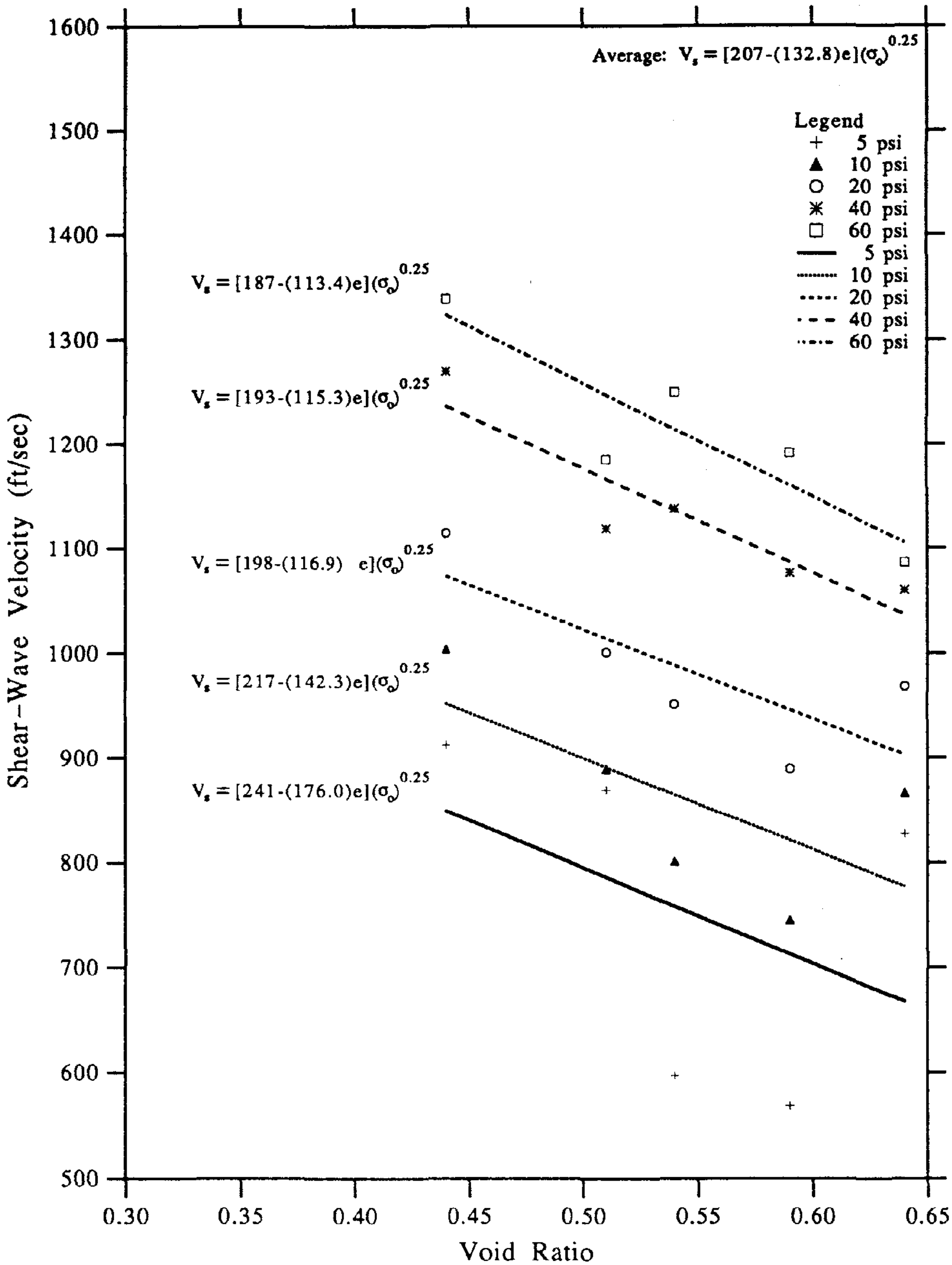

Figure 5.9 Shear-Wave Velocity vs Void Ratio for Group A3

Table 5-6 Comparison of Shear-Wave Velocity Equation

Confining Pressure (psi)	Individual Equation	Average Equation	Hardin's Equation
Group A1			
5	$[138-(20.3)e](\sigma_0)^{0.25}$		
10	$[142-(30.2)e](\sigma_0)^{0.25}$		
20	$[138-(23.5)e](\sigma_0)^{0.25}$	$[139-(24.1)e](\sigma_0)^{0.25}$	$[170-(78.2)e](\sigma_0)^{0.25}$
40	$[141-(25.7)e](\sigma_0)^{0.25}$		
60	$[136-(20.9)e](\sigma_0)^{0.25}$		
Group A2			
5	$[179-(98.2)e](\sigma_0)^{0.25}$		
10	$[171-(92.1)e](\sigma_0)^{0.25}$		
20	$[181-(110.5)e](\sigma_0)^{0.25}$	$[183-(110.8)e](\sigma_0)^{0.25}$	$[170-(78.2)e](\sigma_0)^{0.25}$
40	$[192-(128.0)e](\sigma_0)^{0.25}$		
60	$[191-(125.5)e](\sigma_0)^{0.25}$		
Group A3			
5	$[241-(176.0)e](\sigma_0)^{0.25}$		
10	$[217-(142.3)e](\sigma_0)^{0.25}$		
20	$[198-(116.9)e](\sigma_0)^{0.25}$	$[207-(132.8)e](\sigma_0)^{0.25}$	$[170-(78.2)e](\sigma_0)^{0.25}$
40	$[193-(115.3)e](\sigma_0)^{0.25}$		
60	$[187-(113.4)e](\sigma_0)^{0.25}$		

in Figures 5-10 to 5-12. The graphs of damping ratio plotted as function of confining pressure illustrates that as confining pressure is increased the damping ratio decreases. This agrees with the general trend reported in previous studies in soil dynamics. The damping ratios for soil samples of group A1 range from 0.22% to 4.5%, with an average value of 1.12%, as shown in Table 5-7. Results of damping ratio for groups A2 and A3 are found in Table 5-7 also.

5.2 Seismic Characteristics of Soil Groups B1 and B2

5.2.1 Shear Modulus and Shear-Wave Velocity

The graph of shear modulus as a function of confining pressure for one of the samples in soil groups B1 and B2 is shown in Figures 5-13 and 5-14, respectively. The data for the graphs are presented in Tables 5-8 and 5-9 for soil groups B1 and B2, respectively. The rest of the test results of soil group B1 and B2 are shown in Appendix A. In general, confining pressure has less influence on shear modulus of clay samples than on cohesionless samples.

5.2.2 Damping Ratio

The damping ratios as a function of confining pressure are illustrated in Figures 5-15 and 5-16 for group B1 and B2, respectively. These graphs also demonstrate that for increasing confining pressure, the damping ratio decreases. The ranges and average values of damping ratio are displayed in Table 5-7.

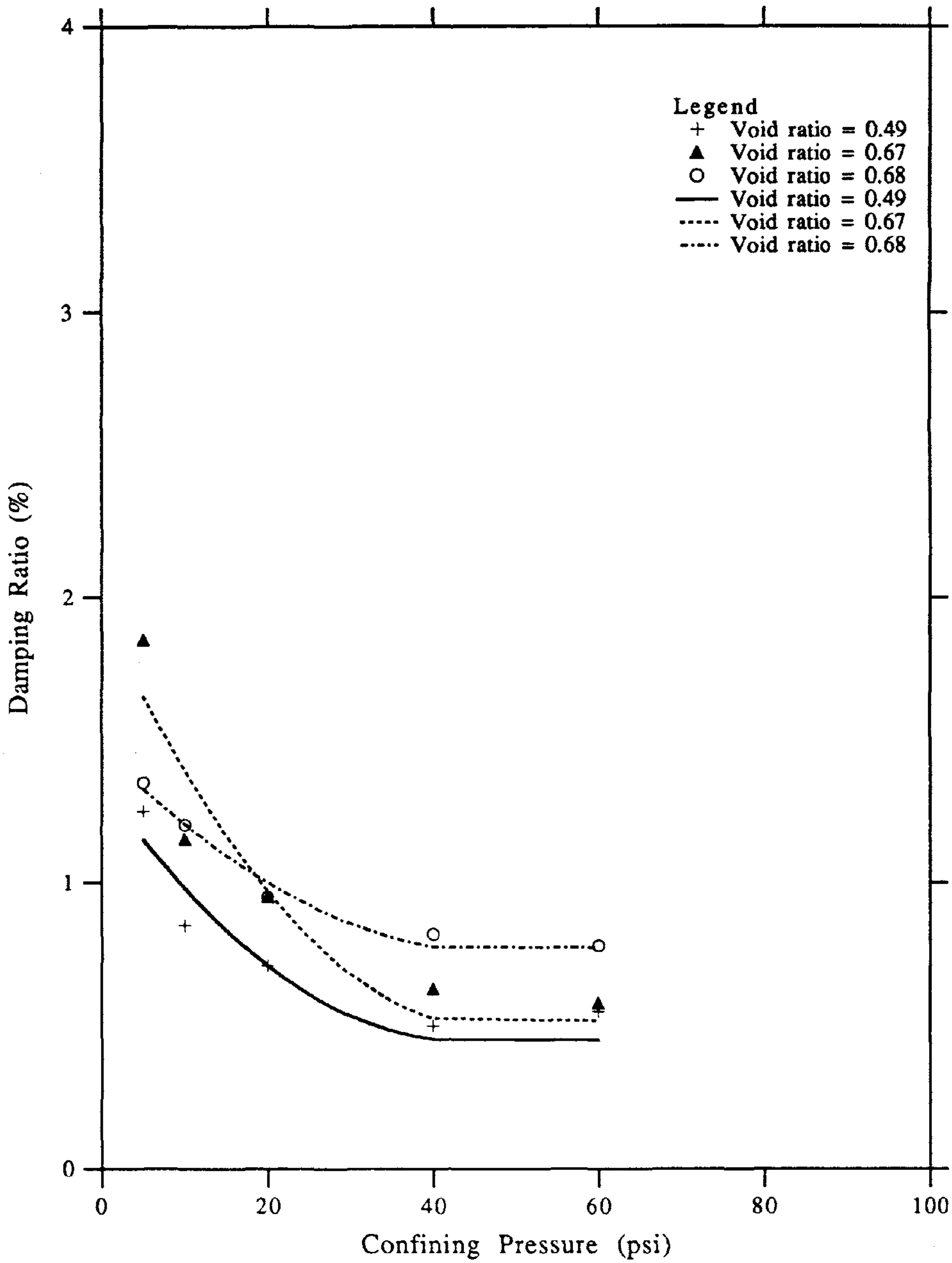

Figure 5-10 Damping Ratio vs Confining Pressure For Sample 090601

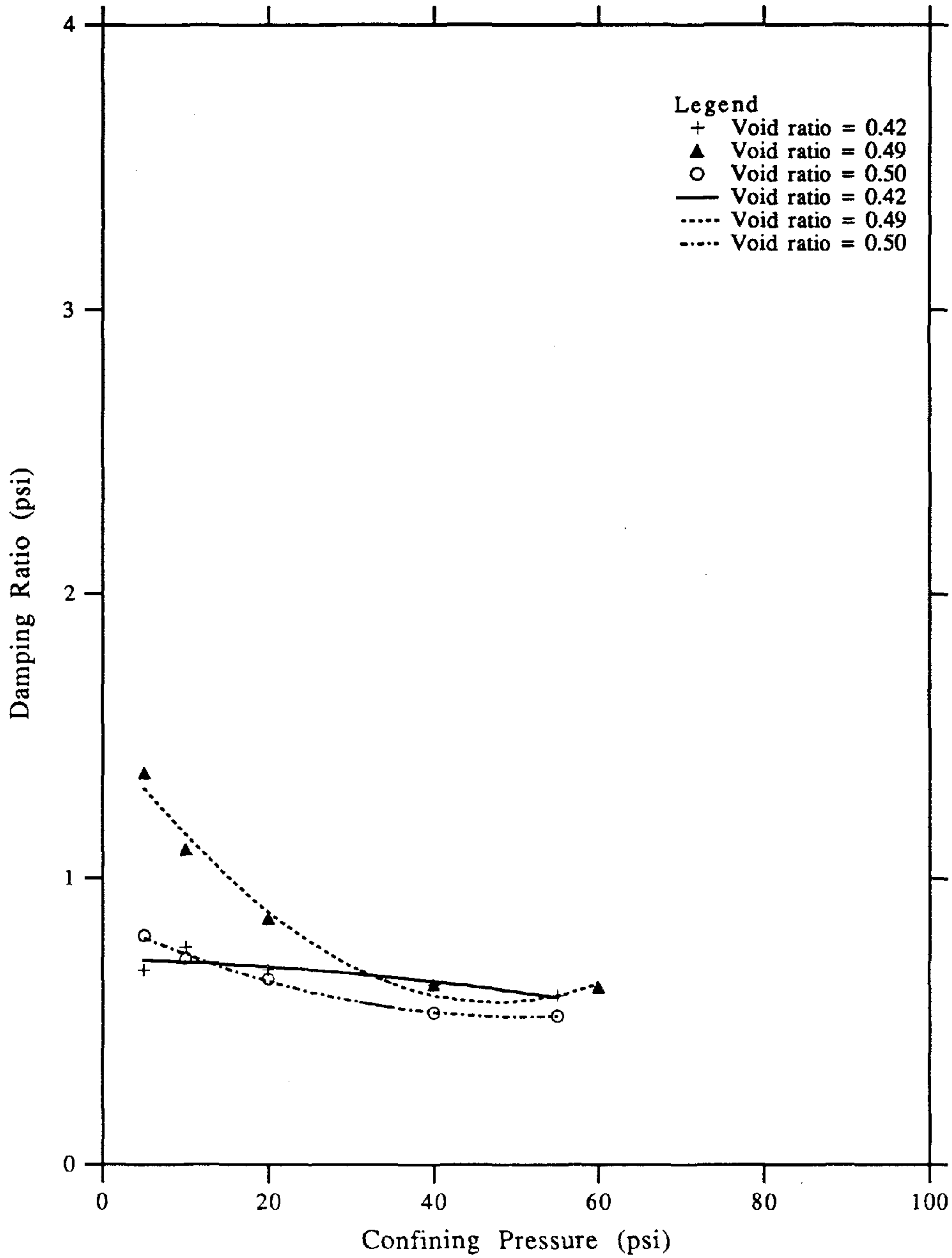

Figure 5-11 Damping Ratio vs Confining Pressure for Sample 101723

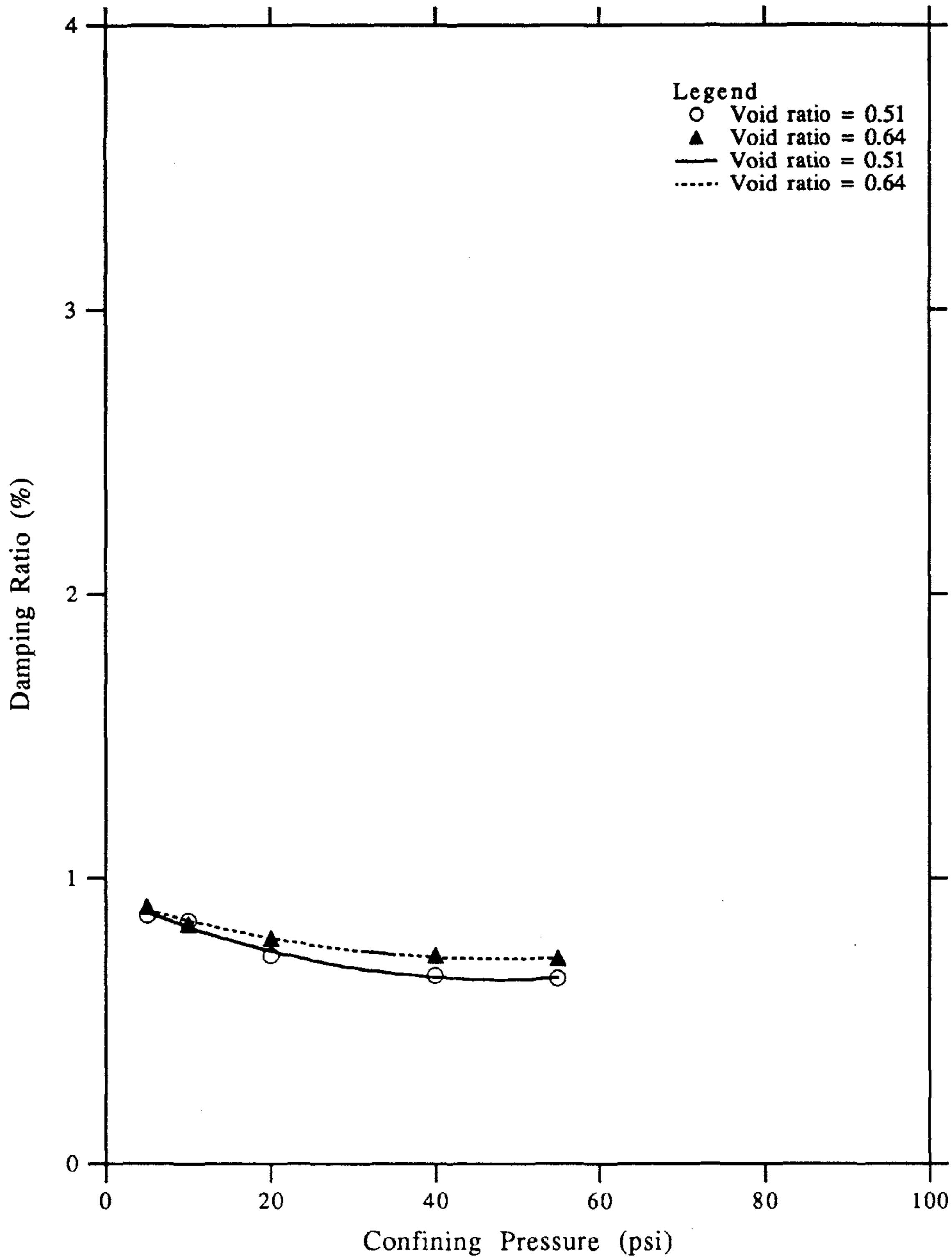

Figure 5-12 Damping Ratio vs Confining Pressure for Sample 101711

Table 5-7 Summary of Resulting Damping Ratio (5 psi to 60 psi)

Soil Group	Range of Damping Ratio (%)	Average Damping Ratio (%) at ~30 psi
A1	0.22-4.50	1.12
A2	0.53-7.25	1.83
A3	0.23-3.95	0.84
B1	1.15-8.50	3.10
B2	2.15-6.50	3.69
C	0.86-3.03	1.52

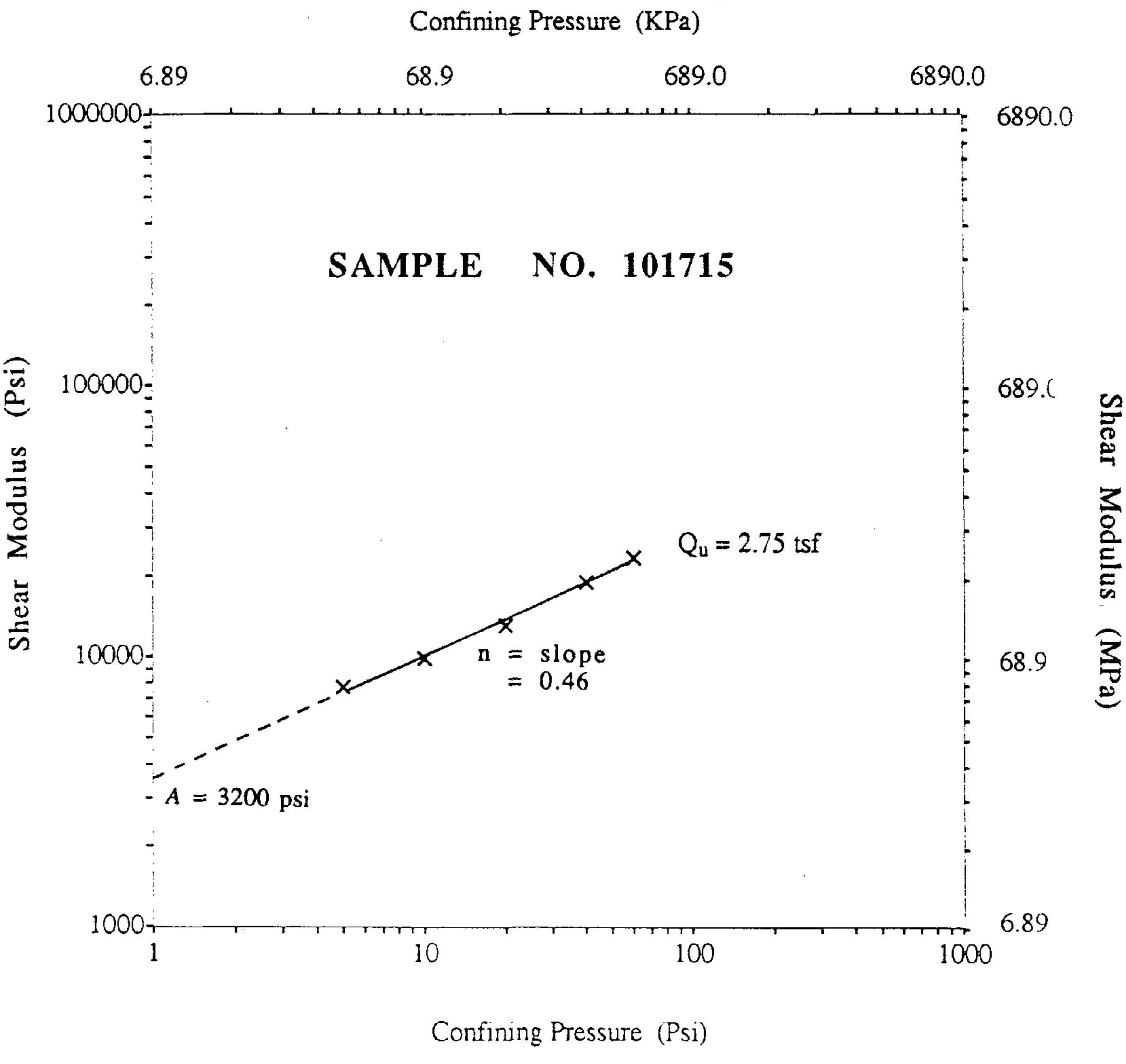

Figure 5-13 Shear Modulus versus Confining Pressure for Sample No. 101715

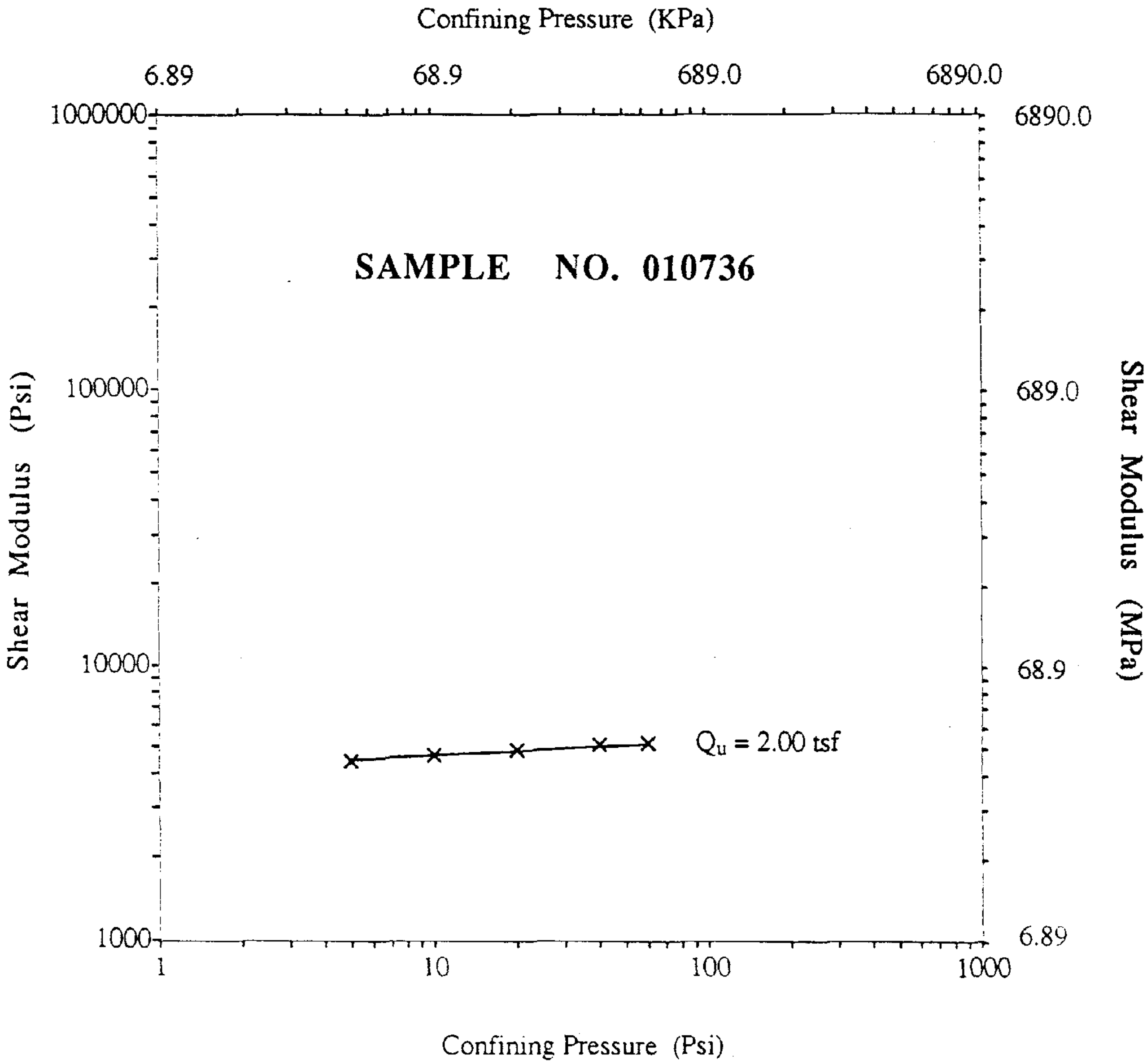

Figure 5-14 Shear Modulus versus Confining Pressure for Sample No. 010736

Table 5-8 Low-Strain Seismic Characteristics for Sample 101715

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
2.25-2.75	Shear Wave Velocity	m/sec	140	159	183	219	241
		ft/sec	459	522	600	719	791
	Shear Modulus	MPa	53	68	90	131	160
		psi	7669	9839	13022	18954	23150
	Damping Ratio	%	2.10	1.65	1.45	1.22	1.15

Table 5-9 Low-Strain Seismic Characteristicsfor Sample 010736

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
2	Shear Wave Velocity	m/sec	128	130.5	134.5	137	138
		ft/sec	420	428	441	449	453
	Shear Modulus	MPa	30.5	32	33.6	35	35.5
		psi	4413	4630	4862	5064	5137
	Damping Ratio	%	5.50	5.60	5.50	5.50	5.40

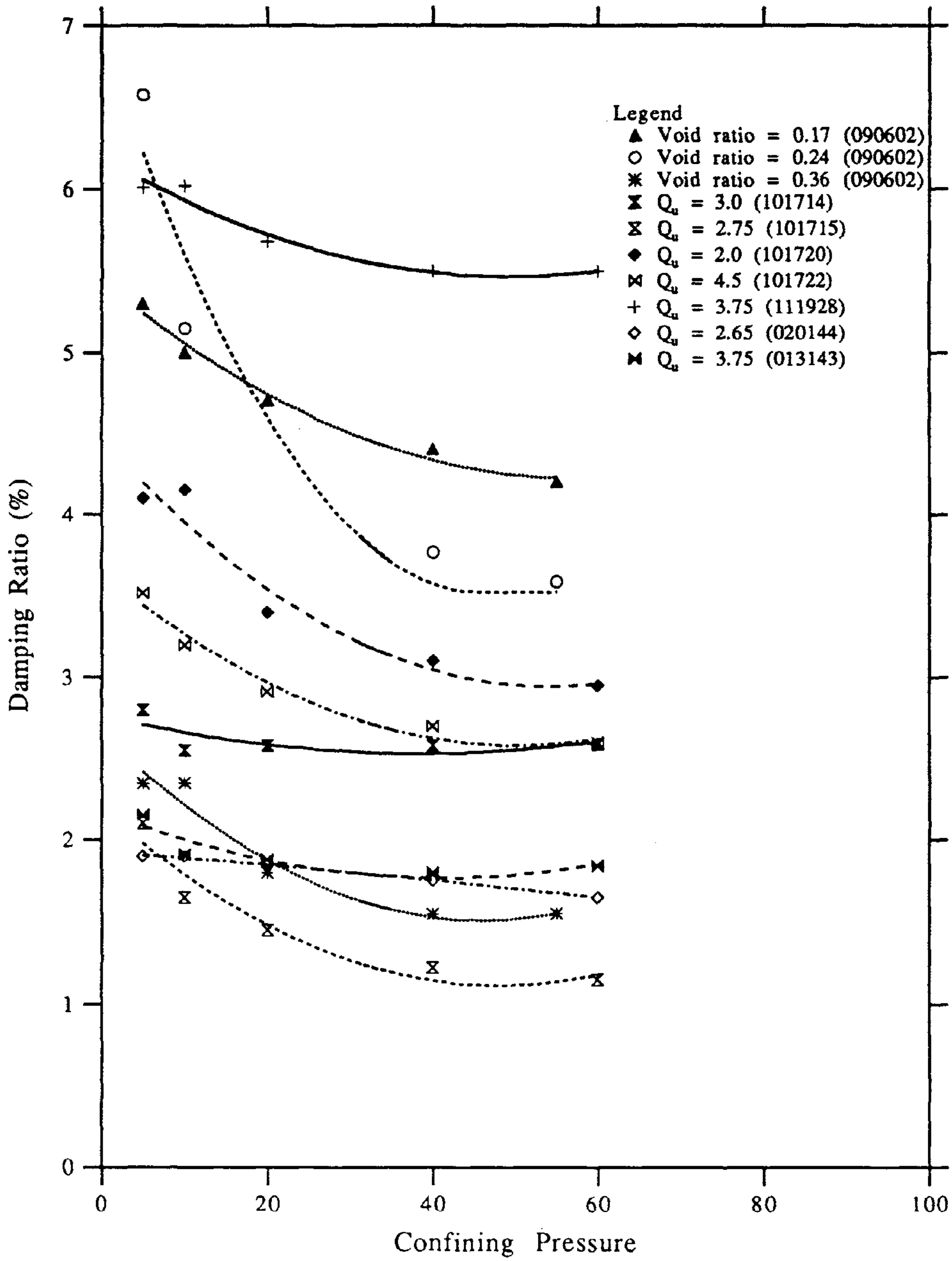

Figure 5-15 Damping Ratio vs Confining Pressure for Group B1

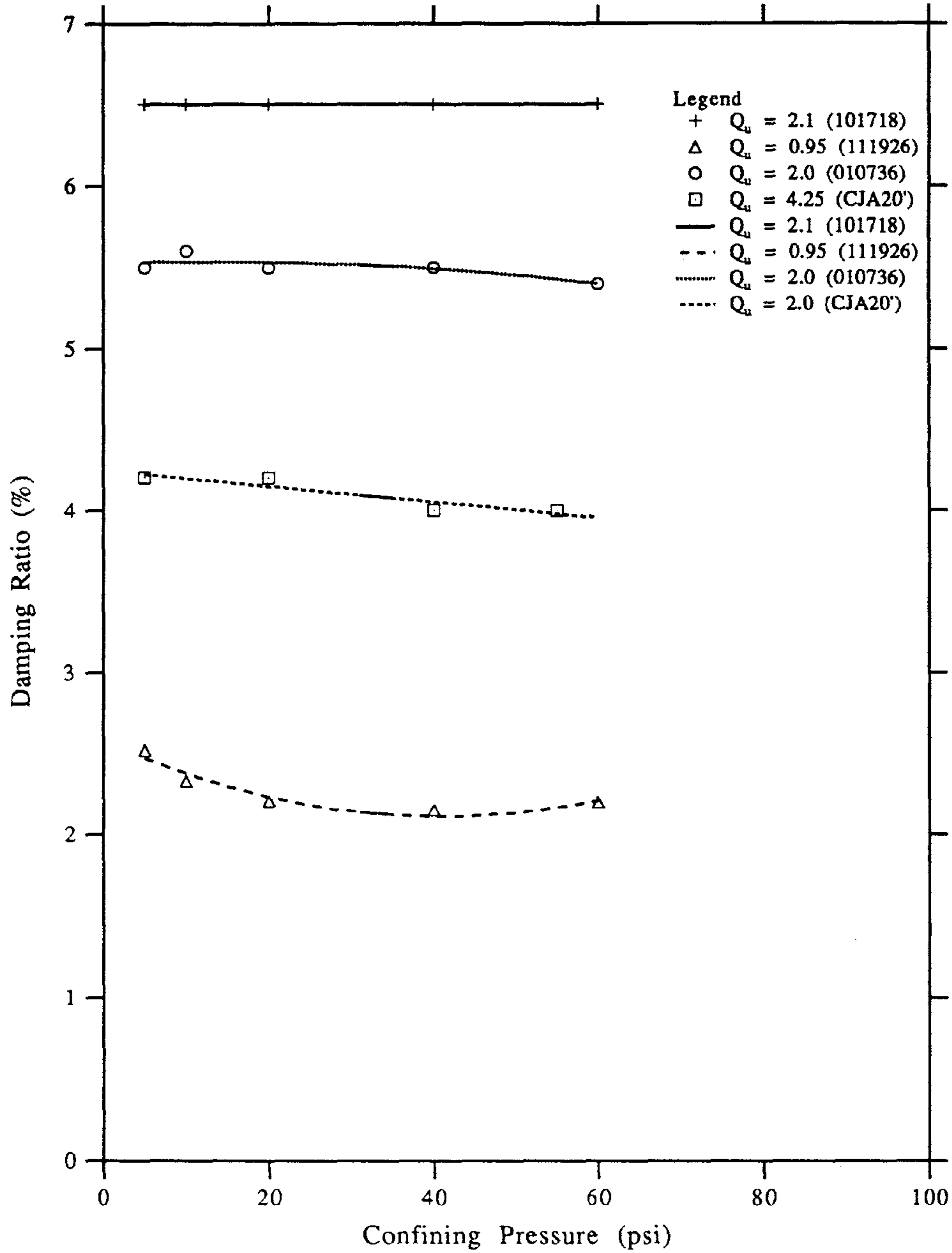

Figure 5-16 Damping Ratio vs Confining Pressure for Group B2

5.3 Empirical Equations for Estimating Shear Modulus of Clay Soils in Groups B1 and B2

As indicated in Equation 5-10, the *A* parameter represents the total effect of all influential factors affecting modulus of clay soils besides confining pressure. The value of parameter *A* can be determined at confining pressure of 1 psi on the graph of shear modulus as a function of confining pressure in log-log scale, as shown in Figure 5-13. The value of *A* parameter of this soil sample is 3200 psi and n value is 0.46. Equation 5-3 can be expressed as

$$G = f(S_u, PL, LL)\,\sigma^n$$

$$= A\,\sigma^n \tag{5-10}$$

where A = shear modulus at confining pressure of 1 psi on the plot of shear modulus against confining pressure

On the basis of Equation 5-10, clay samples are divided into two groups, clay with <2% sand (pure silty clay) and clay with 15-22% sand (silty to sandy clay), in order to study the effect of shear strength and sand contents on parameter *A*. The following equations for the two groups were derived from the plots in Figure 5-17.

$$G = f(S_u, PL, LL)\,\sigma^n$$

$$= (900 + 1845\,S_u)\,\sigma^n \quad \text{for silty clay with <2\% sand} \tag{5-11}$$

$$G = (420 + 940\,S_u)\,\sigma^n \quad \text{for silty clay with 15-22\% sand} \tag{5-12}$$

where S_u = shear strength in ksf

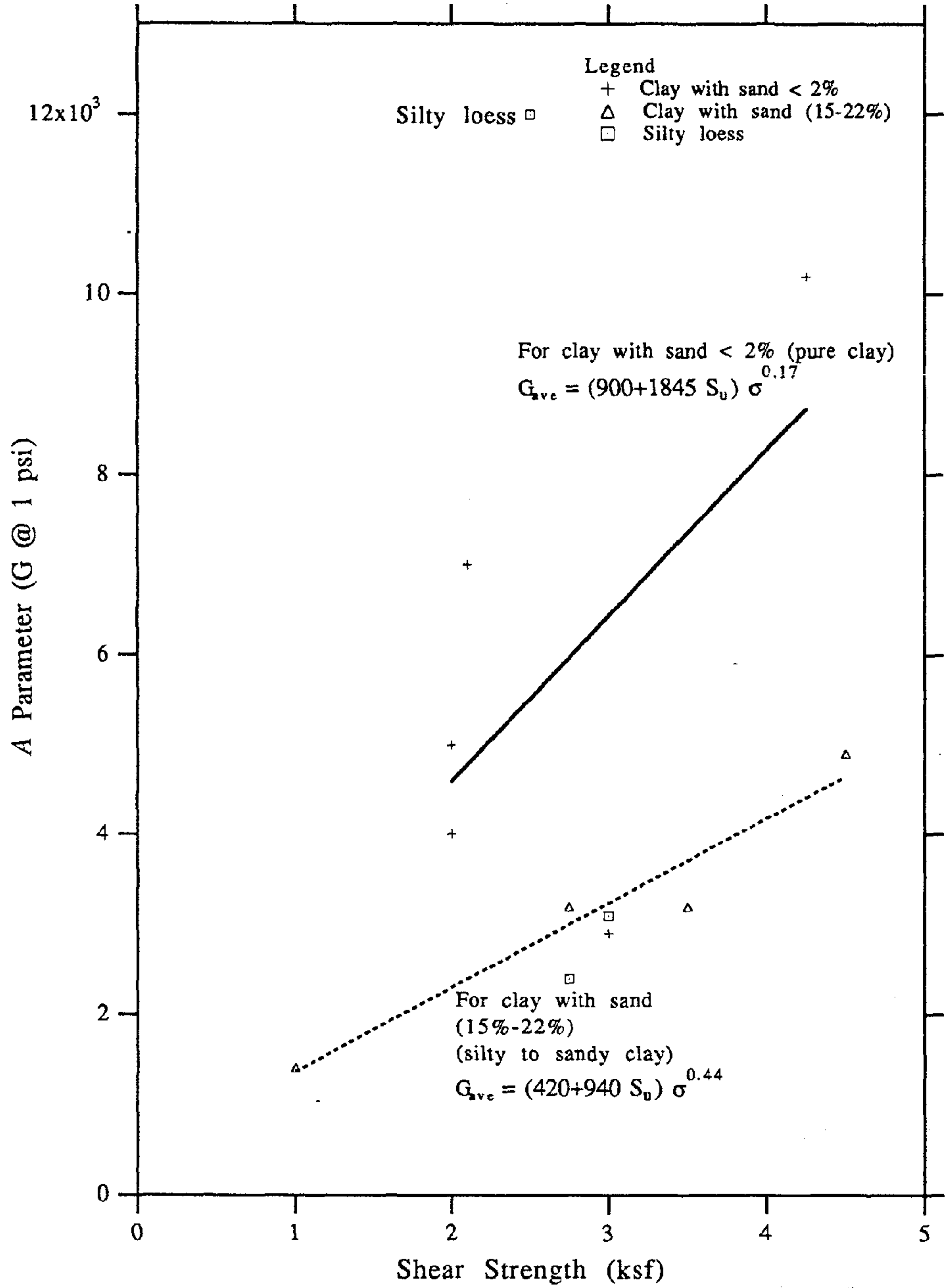

Figure 5-17 "A" Parameter vs Q_u

These equations indicate that as the shear strength aggrandizes, the parameter, *A*, also becomes greater. For pure silty clay, the value of parameter, *A*, is large but the value of n (slope of graph of shear modulus confining pressure on log-log plot) is small. On the other hand, value of *A* is small and n is large for sandy clay. This indicates that confining pressure has more influence on shear modulus of sandy clay than clay with little amount of sand. Equations 5-11 and 5-12 have not been compared with other appropriate existing empirical equations and are considered preliminary. However, these equations may be useful for roughly estimating the shear modulus of cohesive soils in the northern Mississippi embayment region.

5.4 Seismic Characteristics of Loess (Soil Group C)

Figure 5-17 also indicates that there is no clear correlation between *A* and Q_u for silty material (loess). An example graph of damping ratio versus void ratio for group C is shown in Figure 5-18, while the range and average of damping ratio for this group is shown in Table 5-7. The test results of loess soils were inadequate for making any conclusions or developing empirical equations. No existing empirical equations and very few test results are available for comparisons.

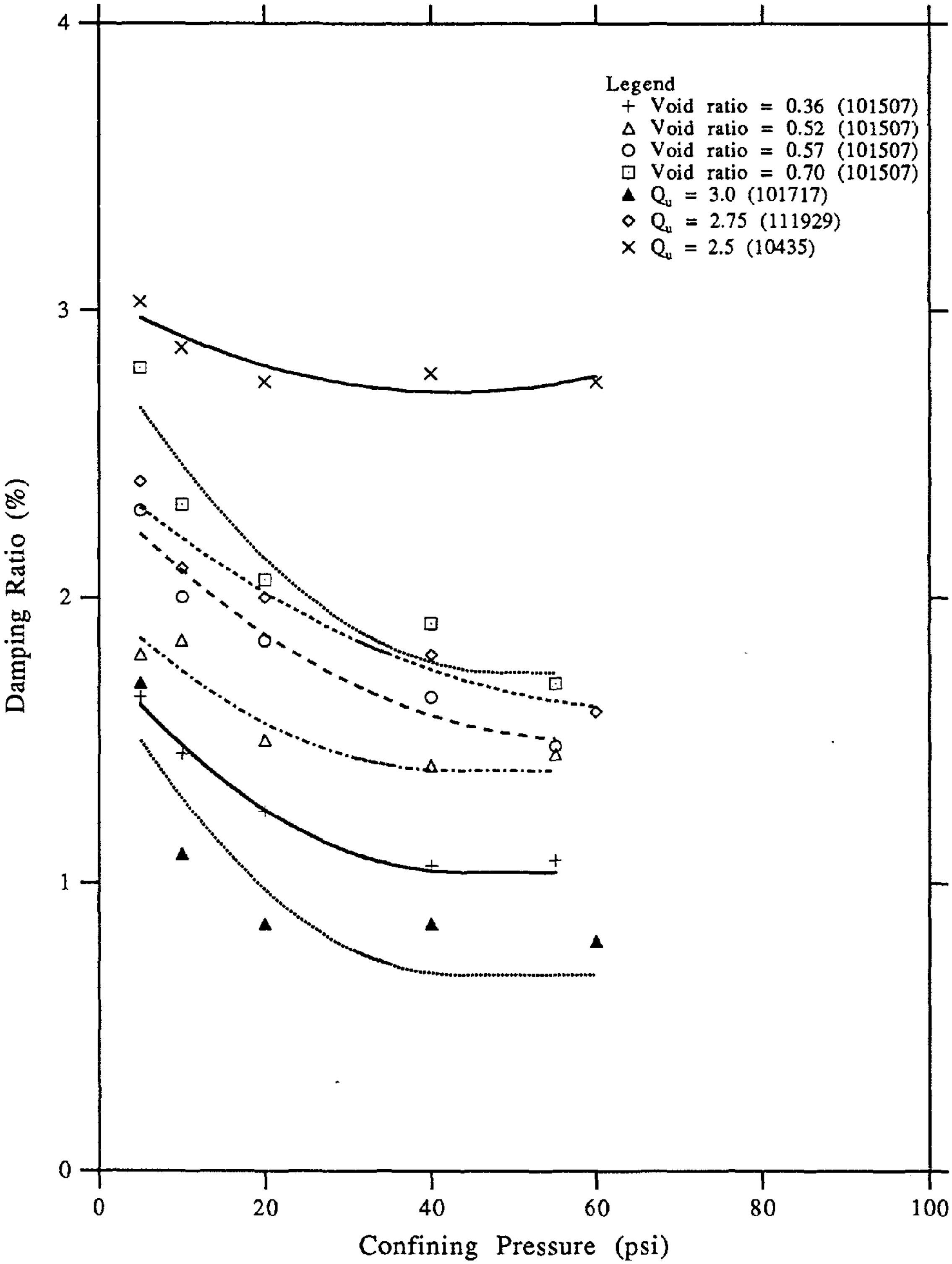

Figure 5-18 Damping Ratio vs Confining Pressure for Group C

CHAPTER 6

HIGH-STRAIN DYNAMIC PROPERTIES OF SOILS

For a large magnitude earthquake, the natural soil strata are often subjected to vibration displacement beyond the elastic range, that is, high strain in the nonlinear range. Thus, it is important to determine the dynamic properties of soils under high-strain loading conditions for strong ground motion studies and nonlinear soil dynamics studies. After determining the low-strain dynamic data of the soil samples, the high-strain properties can be obtained without removing the sample from the test set-up. The high-strain modulus degradation characteristics of the soil samples were obtained by increasing the power that drives the specimen and by the readings being measured at resonance. This extension of the tests was repeated at confining pressures of 5, 10, and 55 or 60 psi.

6.1 High-Strain Dynamic Properties of Soils

The parameters that are used to study the dynamic properties of soils such as modulus ratio $G(\gamma)/G_o$, the damping ratio, and the shear strain amplitude are plotted on the same graph in order to study their relationships. The modulus ratio, $G(\gamma)/G_o$, is the ratio between the shear modulus at high strain and the shear modulus in the elastic range (less than 10^{-5}). For each soil sample subject to a range of confining pressures, modulus strain degradation curves are plotted. See Figures 6-1 to 6-18. The range and average of the results are indicated on these graphs. The graphs demonstrate that as shear

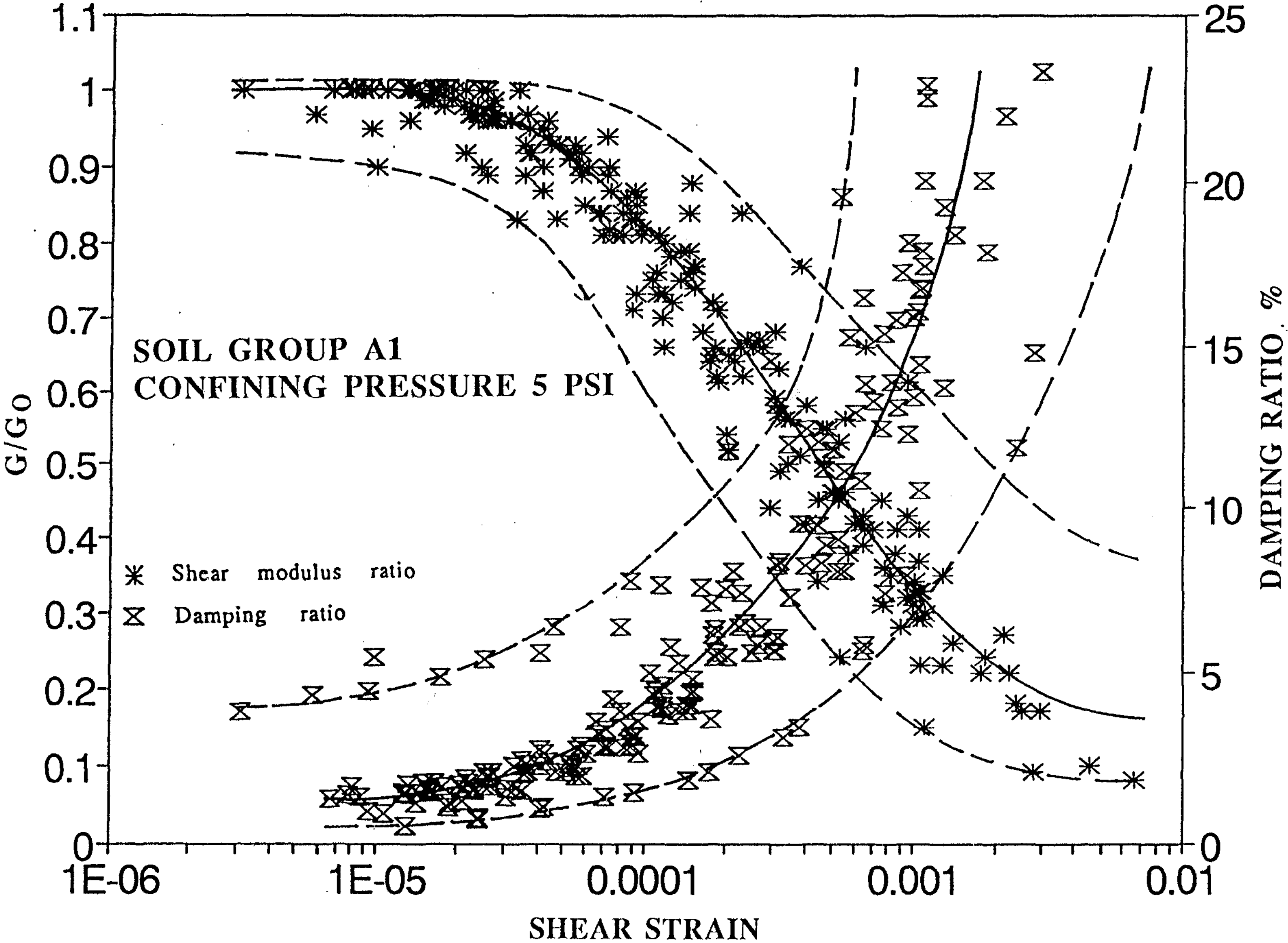

Figure 6-1 Shear Modulus Degradation Curves for Soil Group A1 at 5 psi

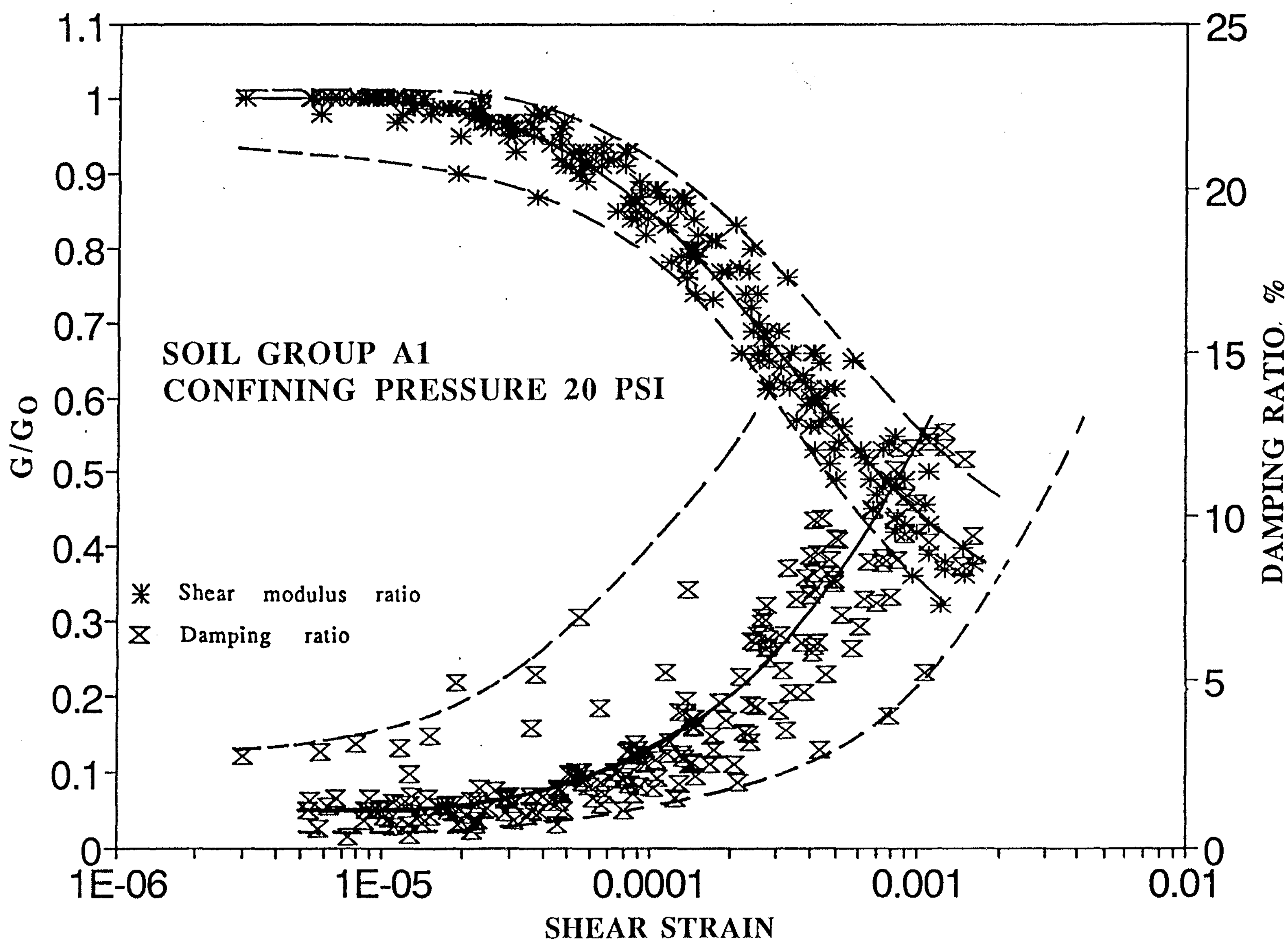

Figure 6-2 Shear Modulus Degradation Curves for Soil Group A1 at 20 psi

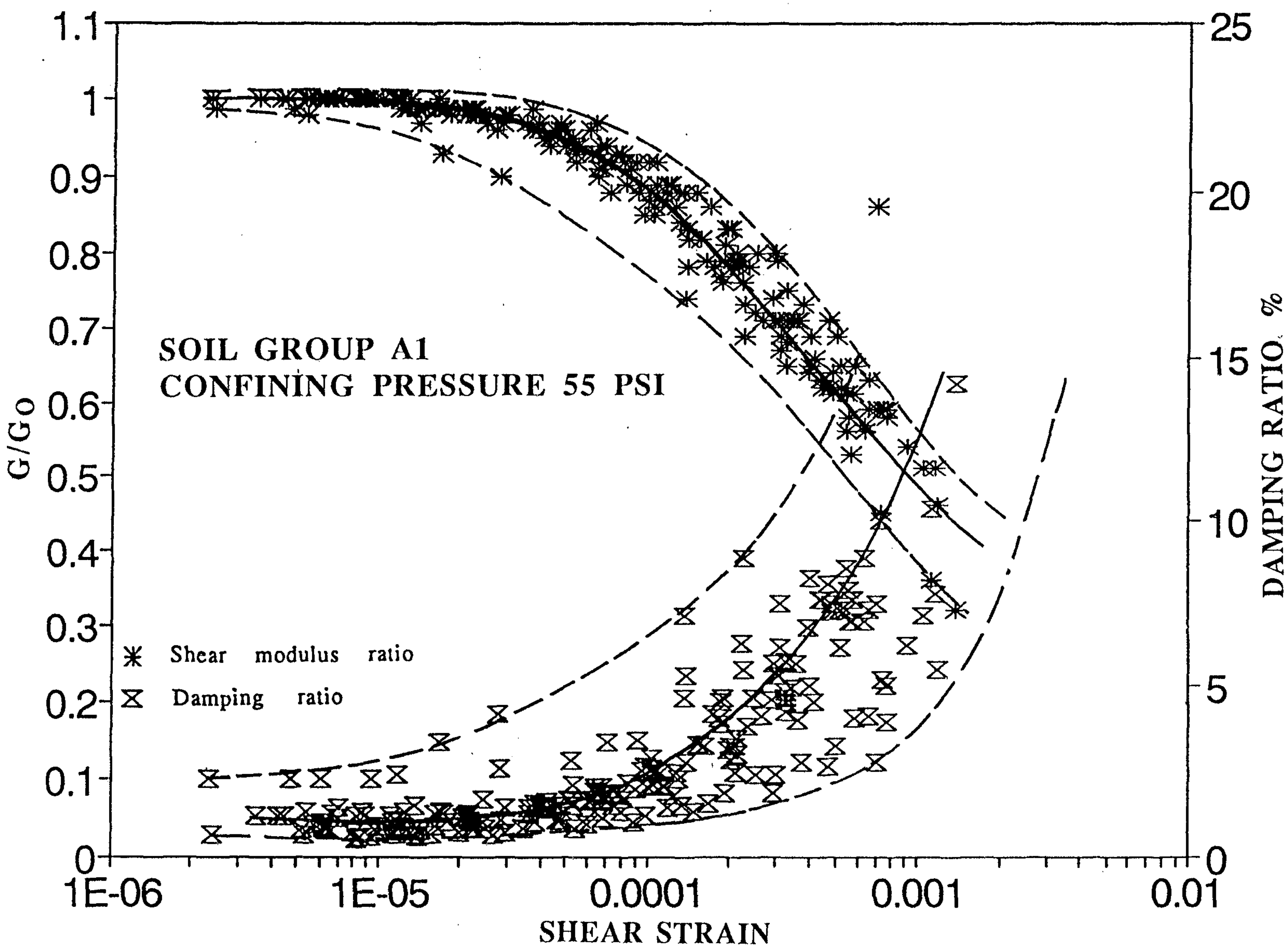

Figure 6-3 Shear Modulus Degradation Curves for Soil Group A1 at 55 psi

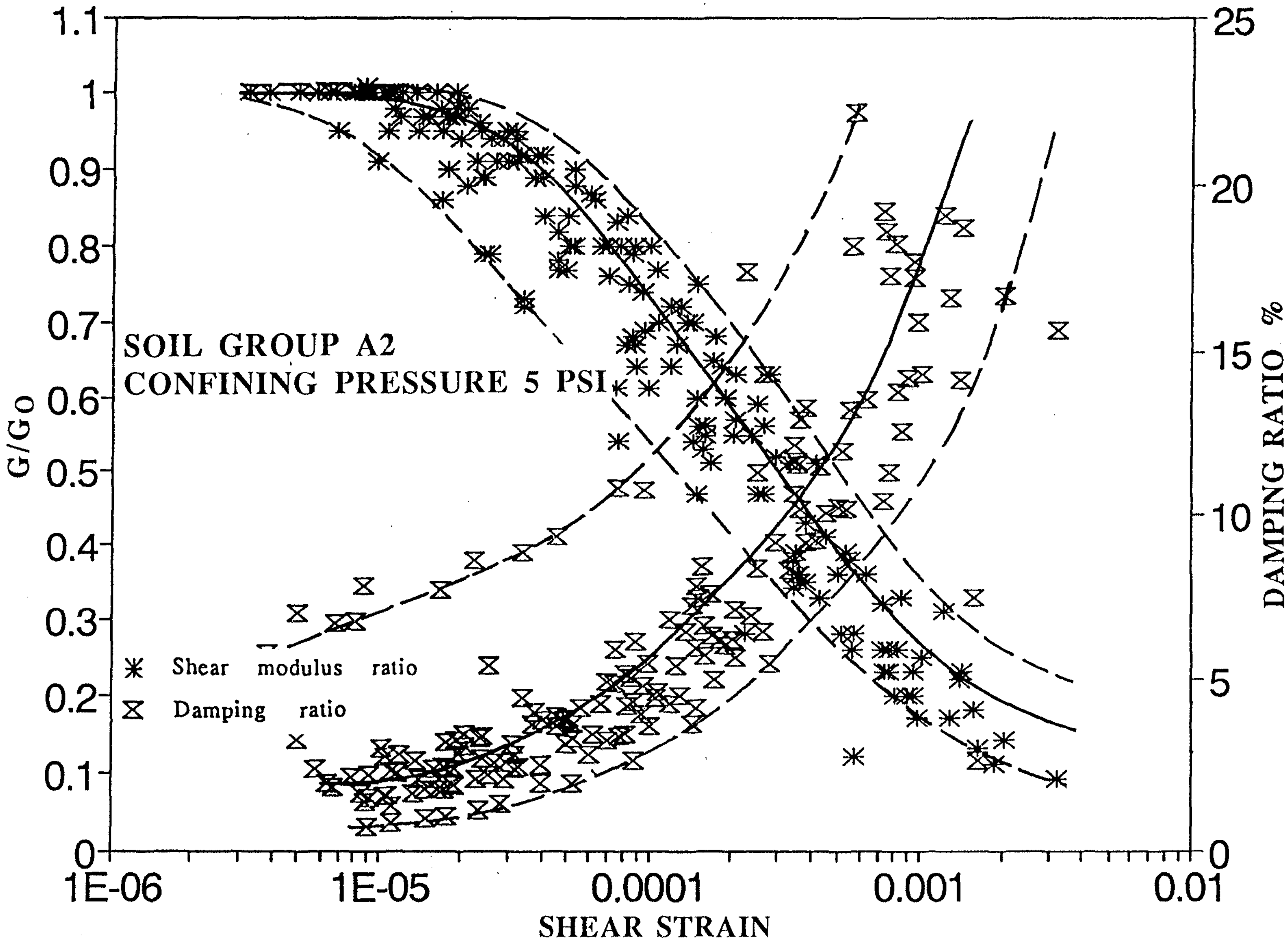

Figure 6-4 Shear Modulus Degradation Curves for Soil Group A2 at 5 psi

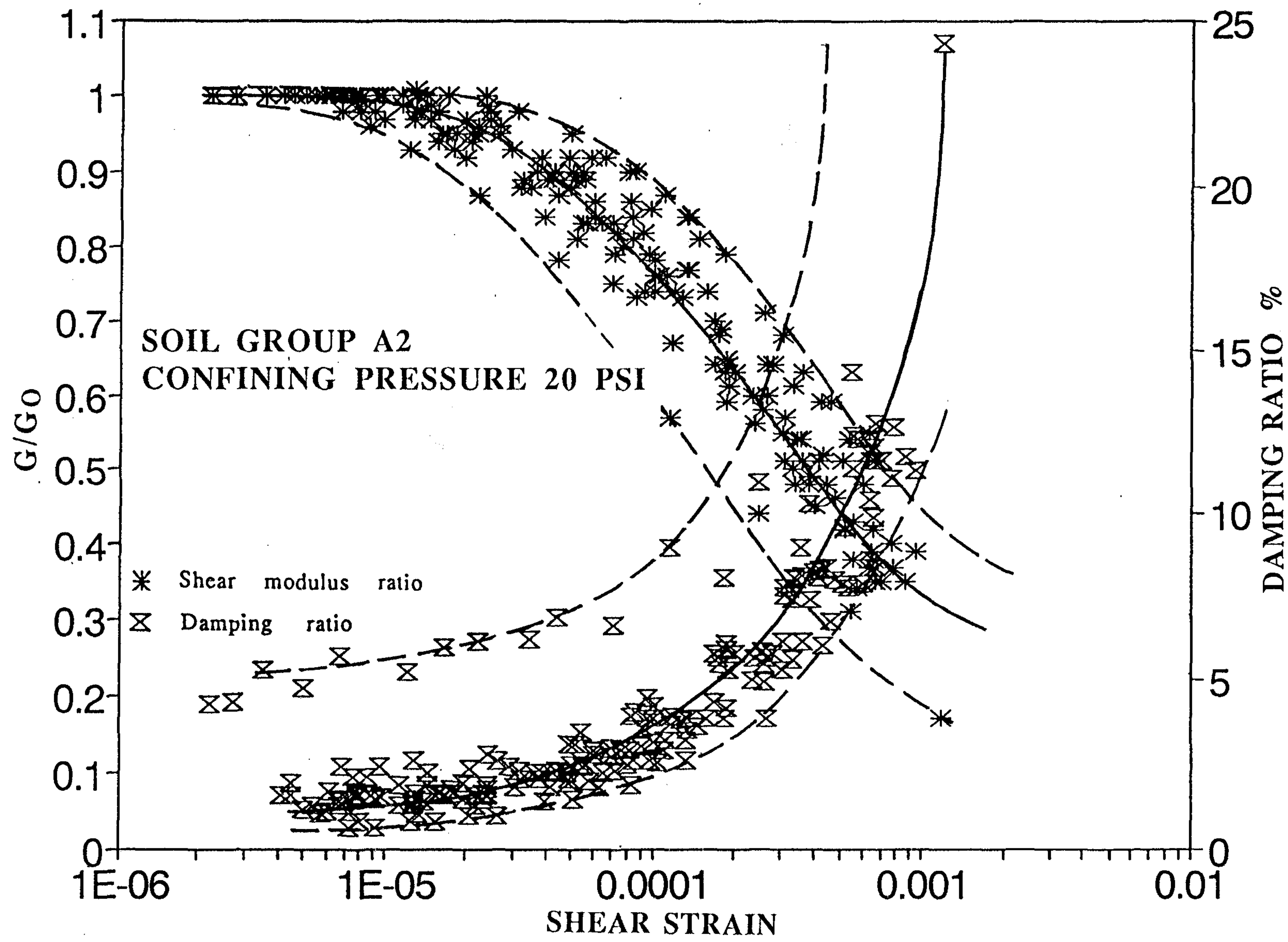

Figure 6-5 Shear Modulus Degradation Curves for Soil Group A2 at 20 psi

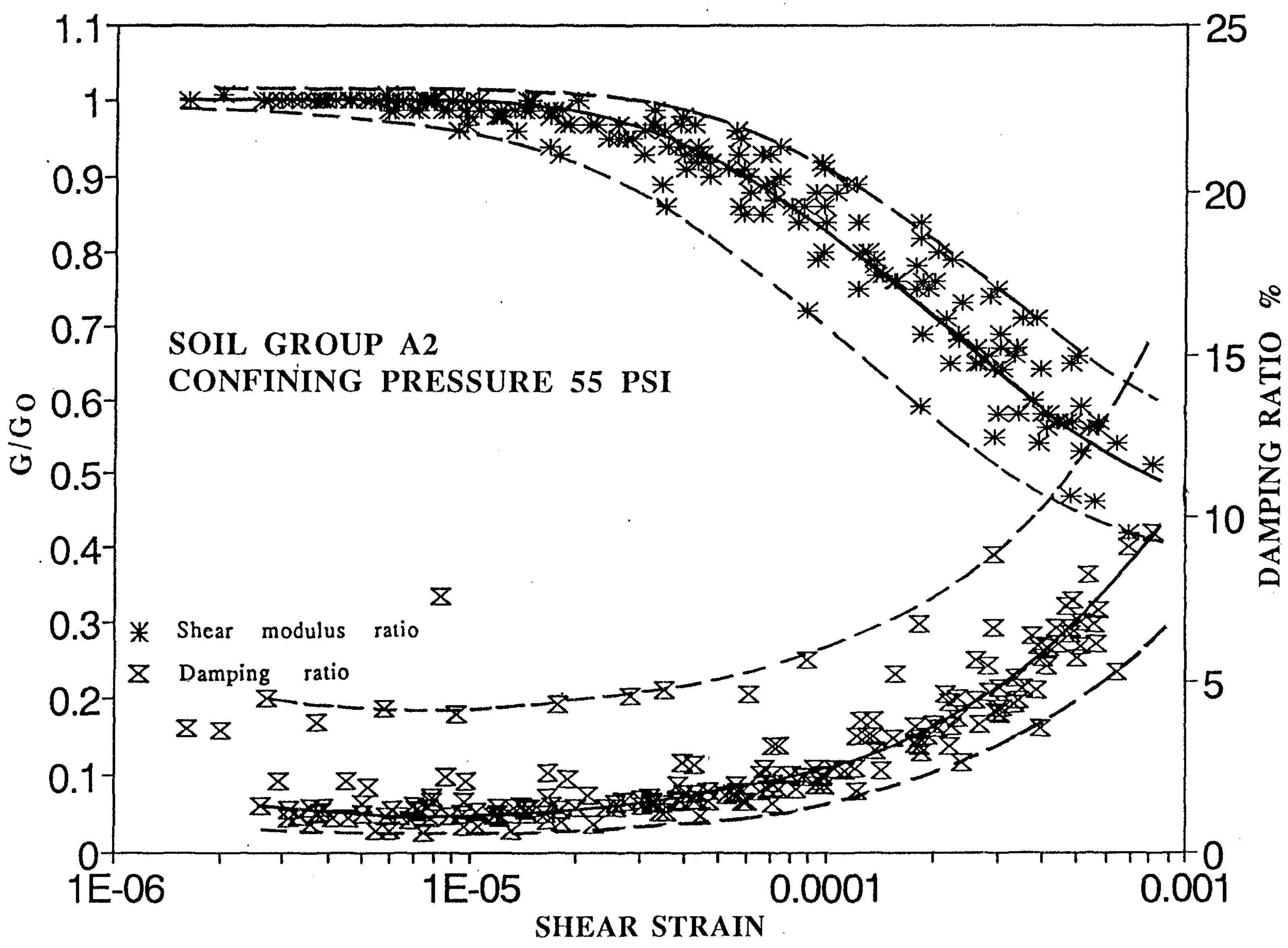

Figure 6-6 Shear Modulus Degradation Curves for Soil Group A2 at 55 psi

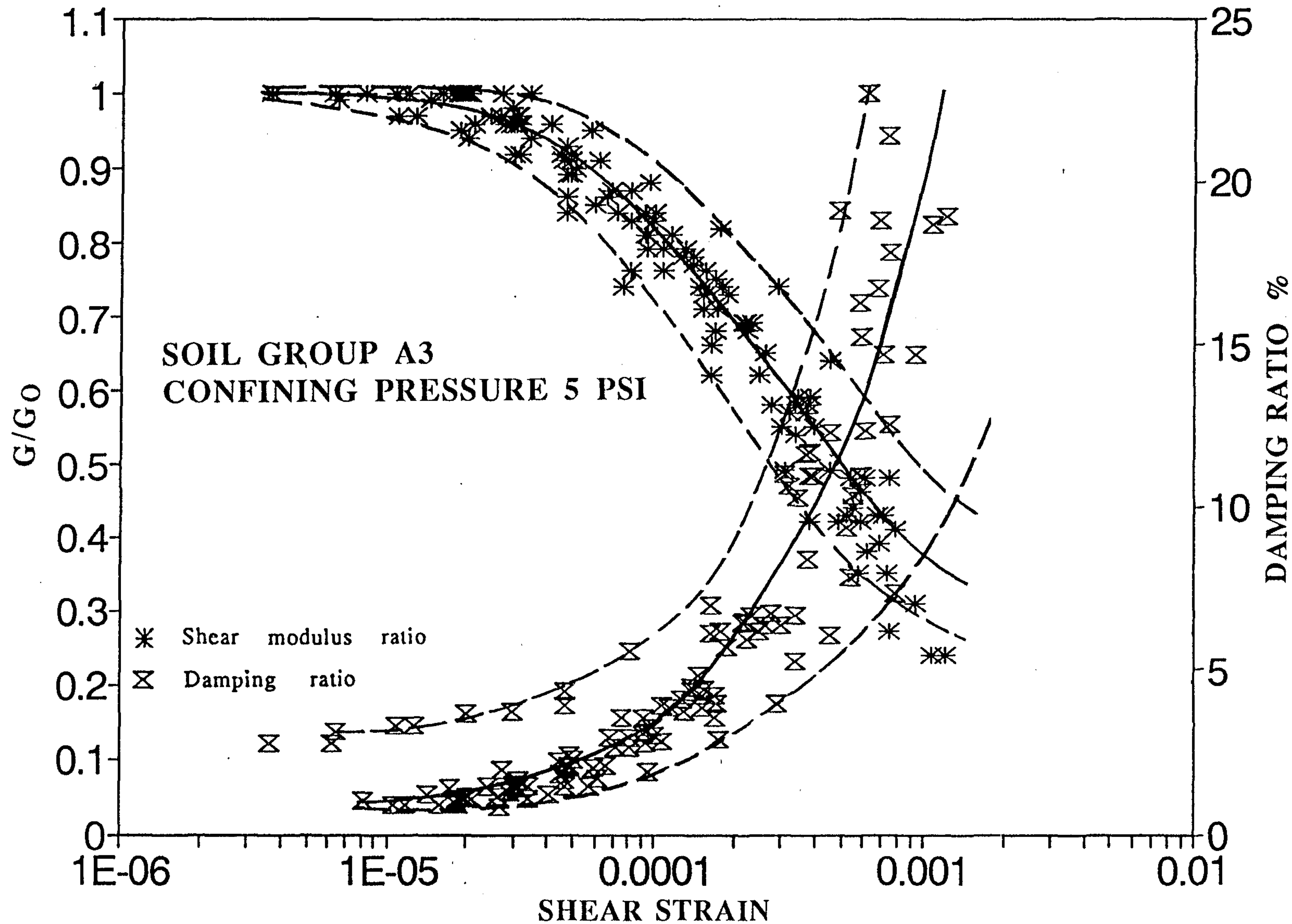

Figure 6-7 Shear Modulus Degradation Curves for Soil Group A3 at 5 psi

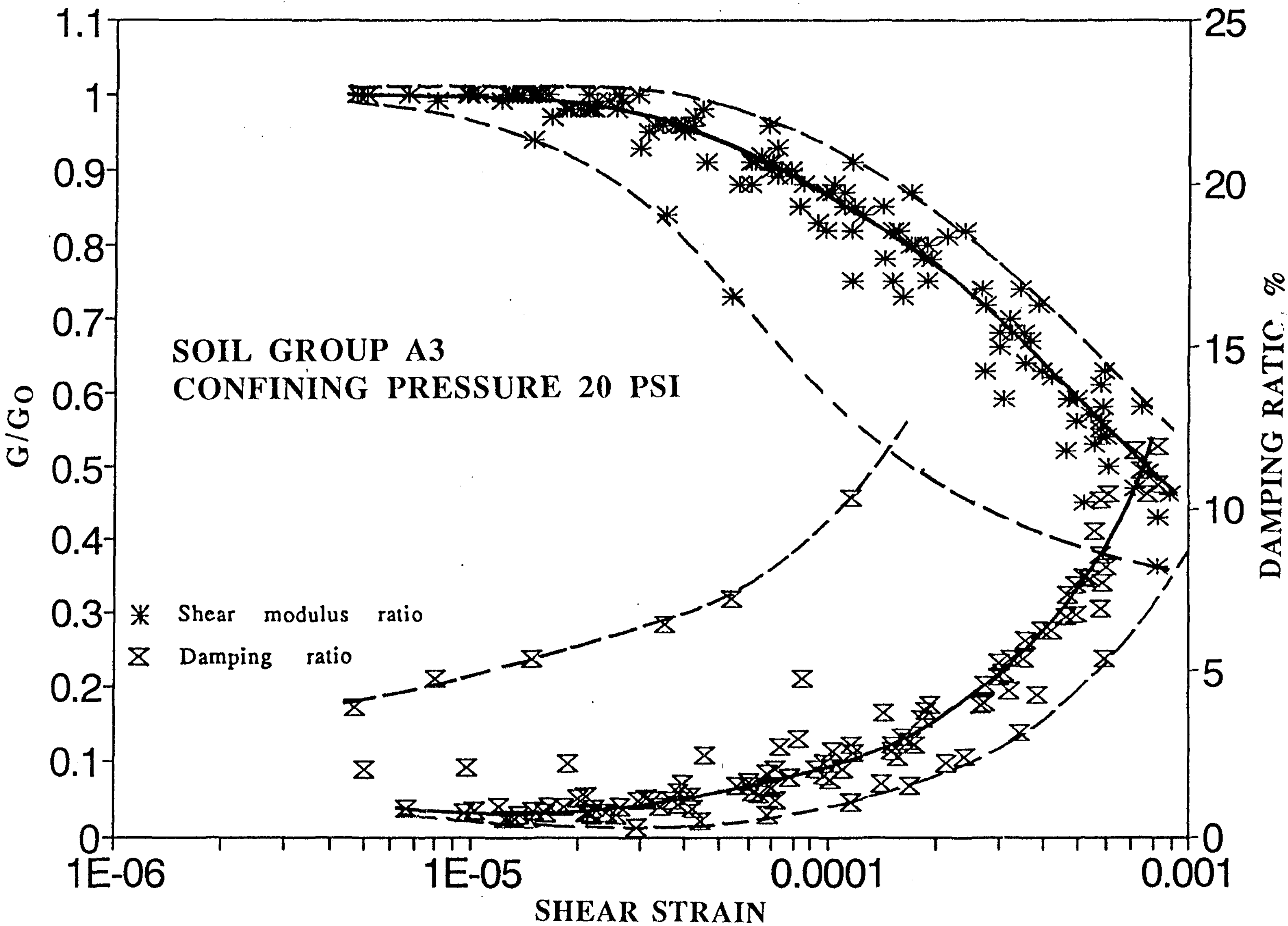

Figure 6-8 Shear Modulus Degradation Curves for Soil Group A3 at 20 psi

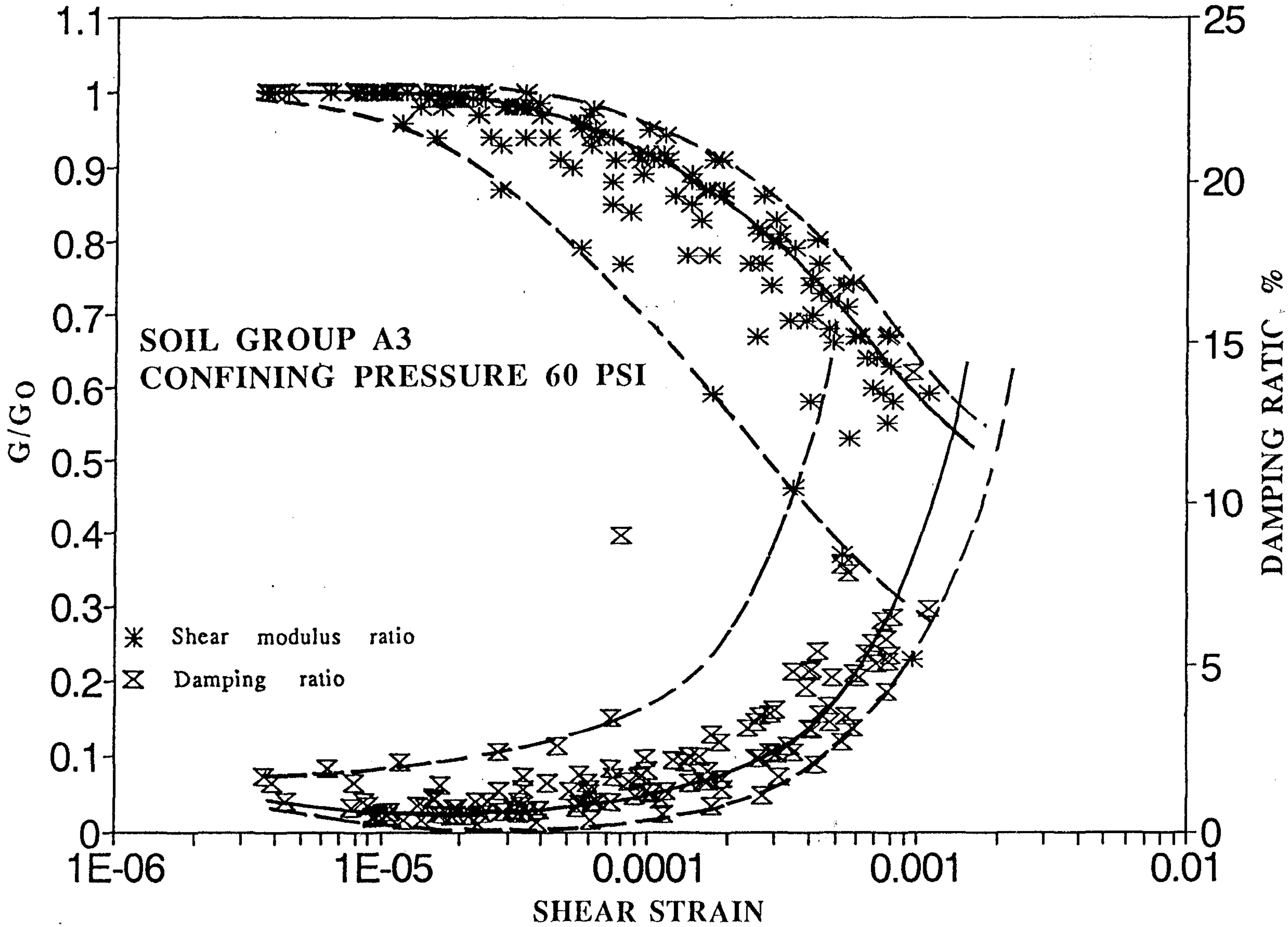

Figure 6-9 Shear Modulus Degradation Curves for Soil Group A3 at 60 psi

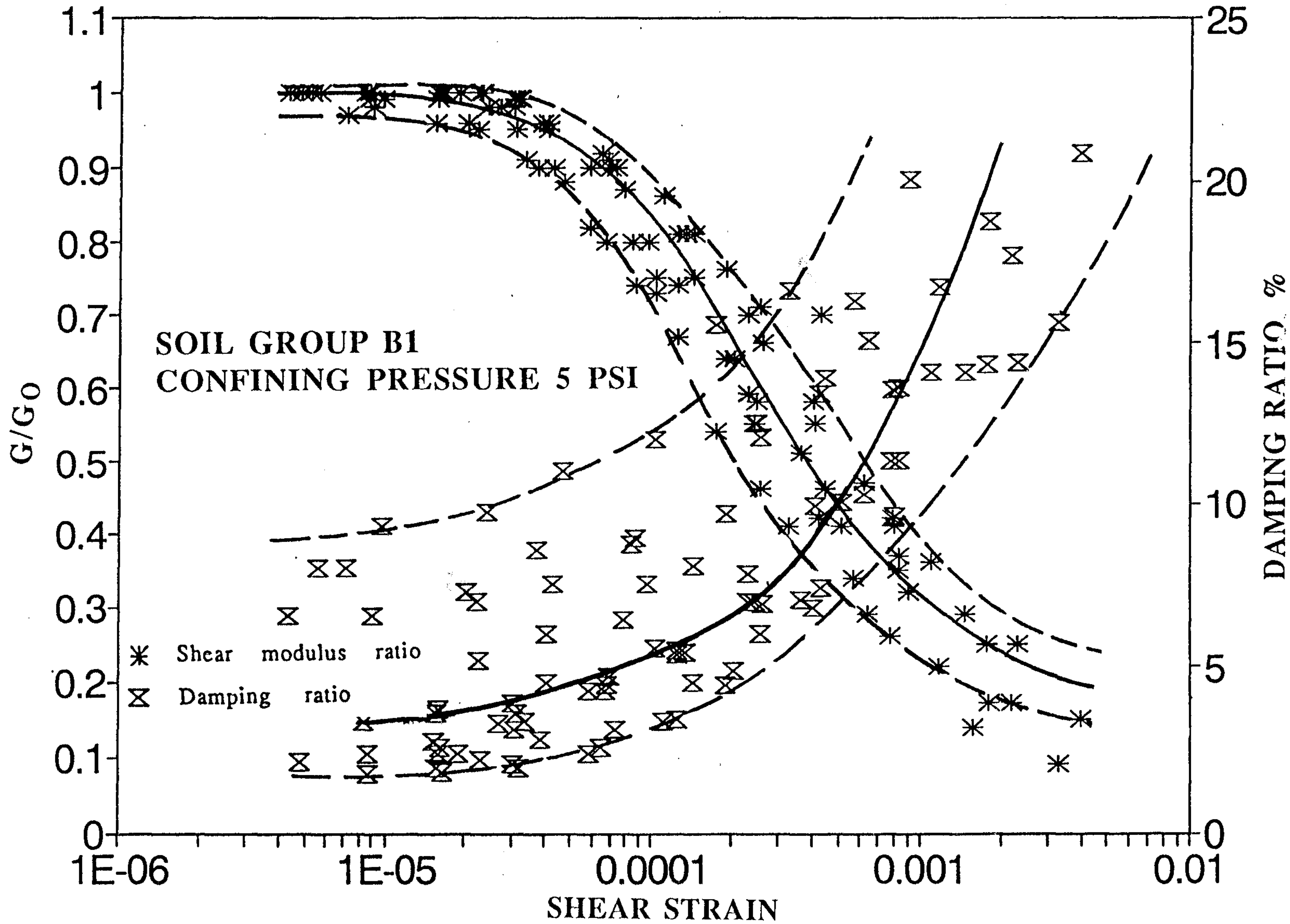

Figure 6-10 Shear Modulus Degradation Curves for Soil Group B1 at 5 psi

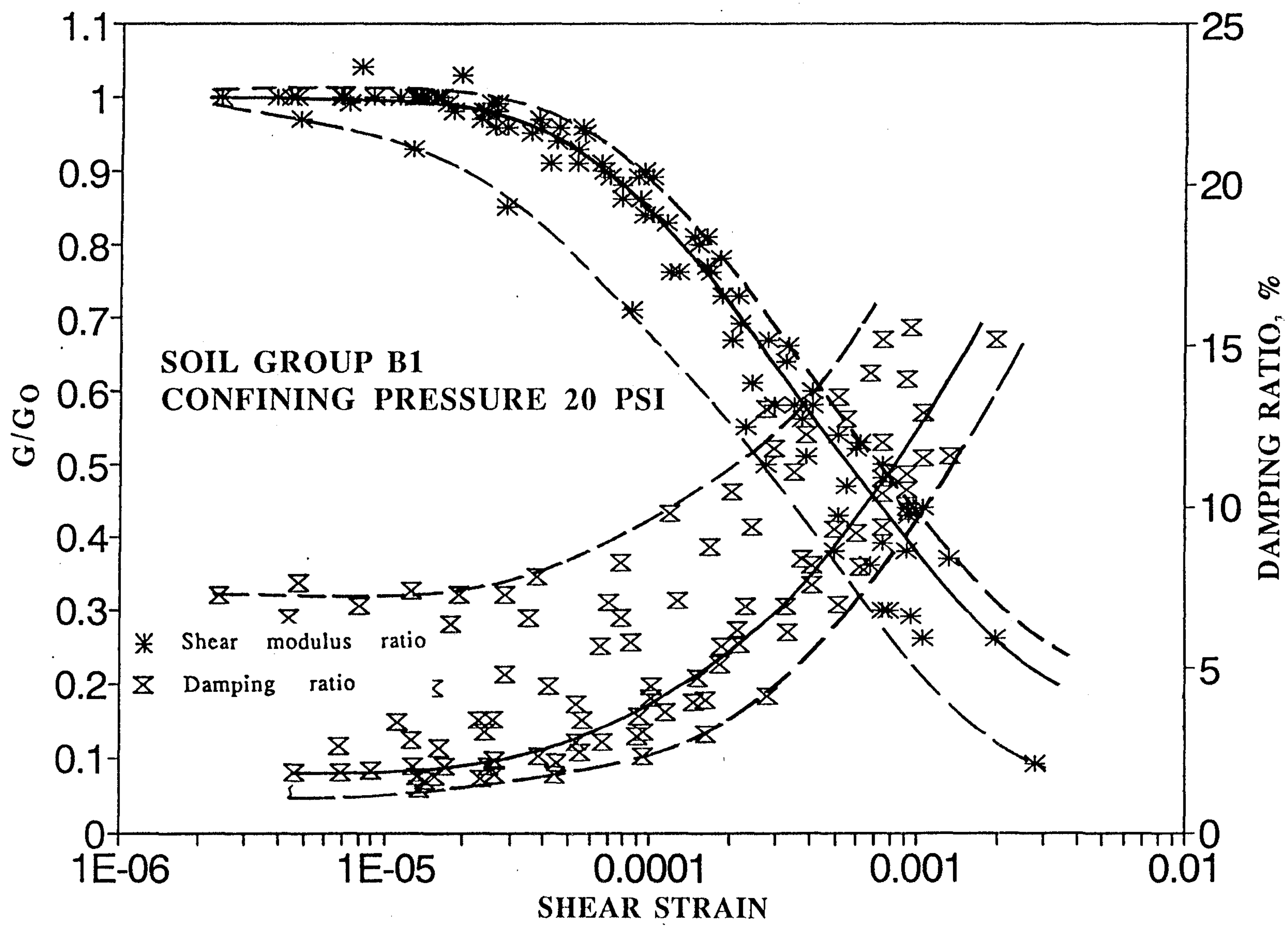

Figure 6-11 Shear Modulus Degradation Curves for Soil Group B1 at 20 psi

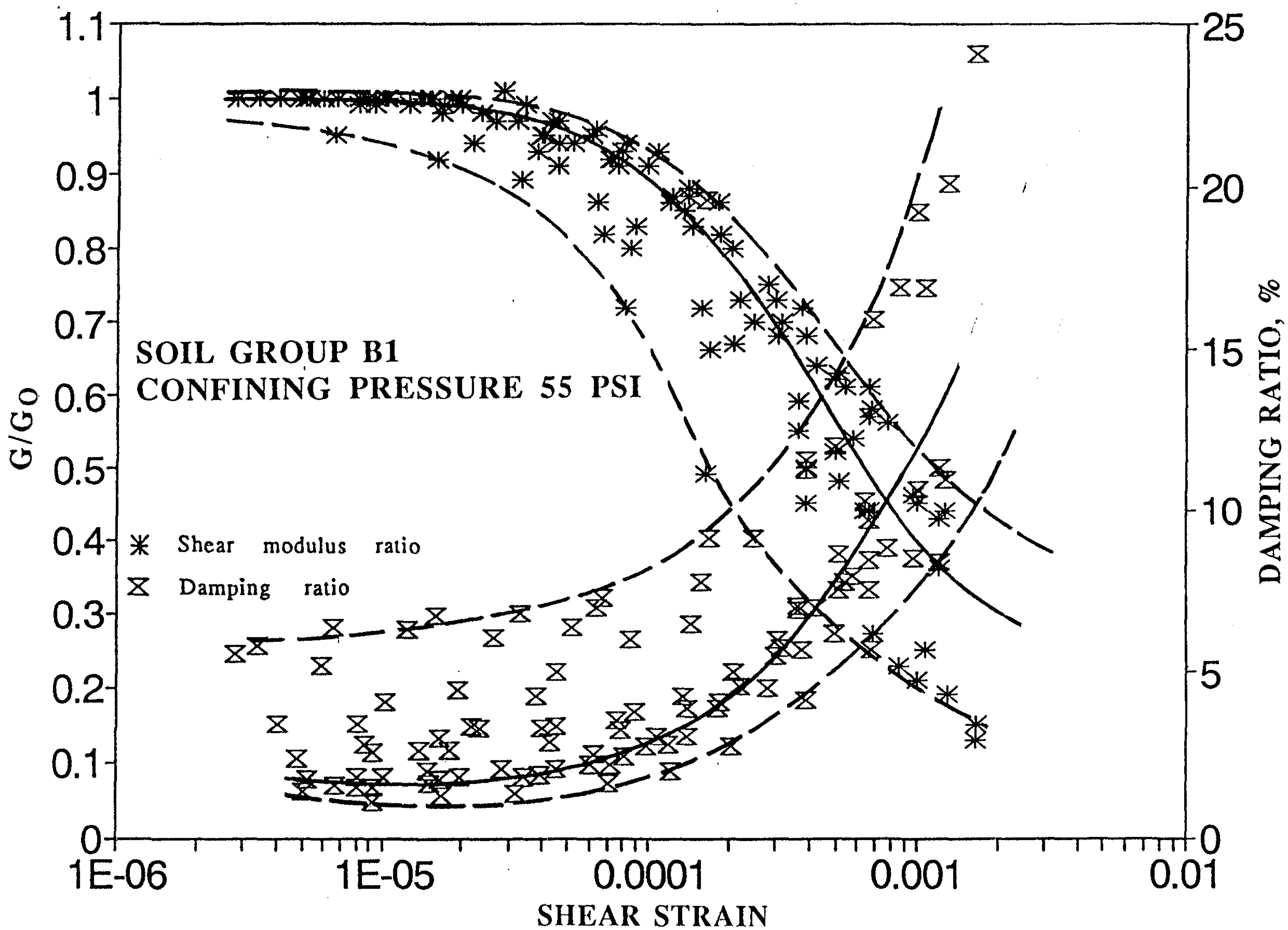

Figure 6-12 Shear Modulus Degradation Curves for Soil Group B1 at 55 psi

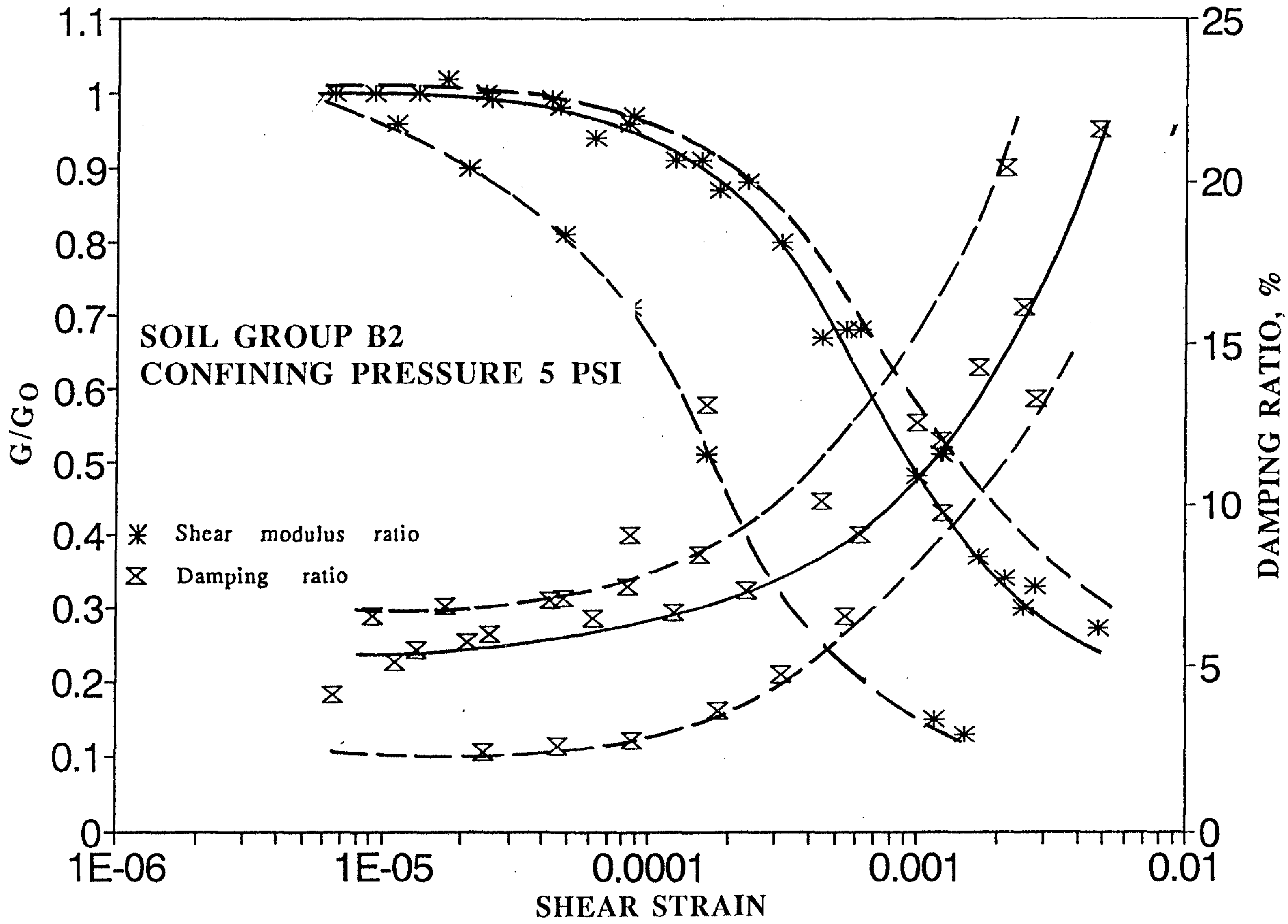

Figure 6-13 Shear Modulus Degradation Curves for Soil Group B2 at 5 psi

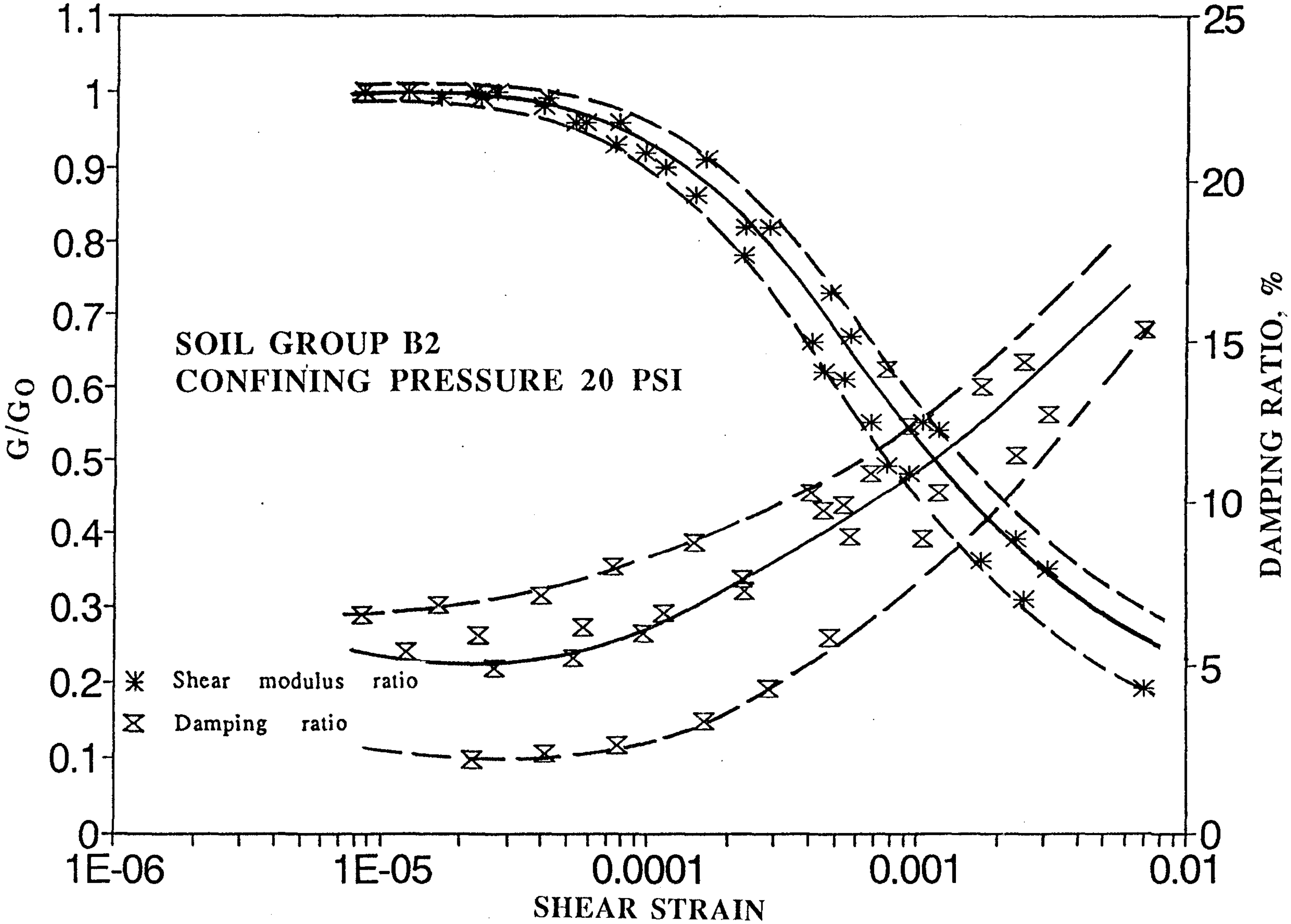

Figure 6-14 Shear Modulus Degradation Curves for Soil Group B2 at 20 psi

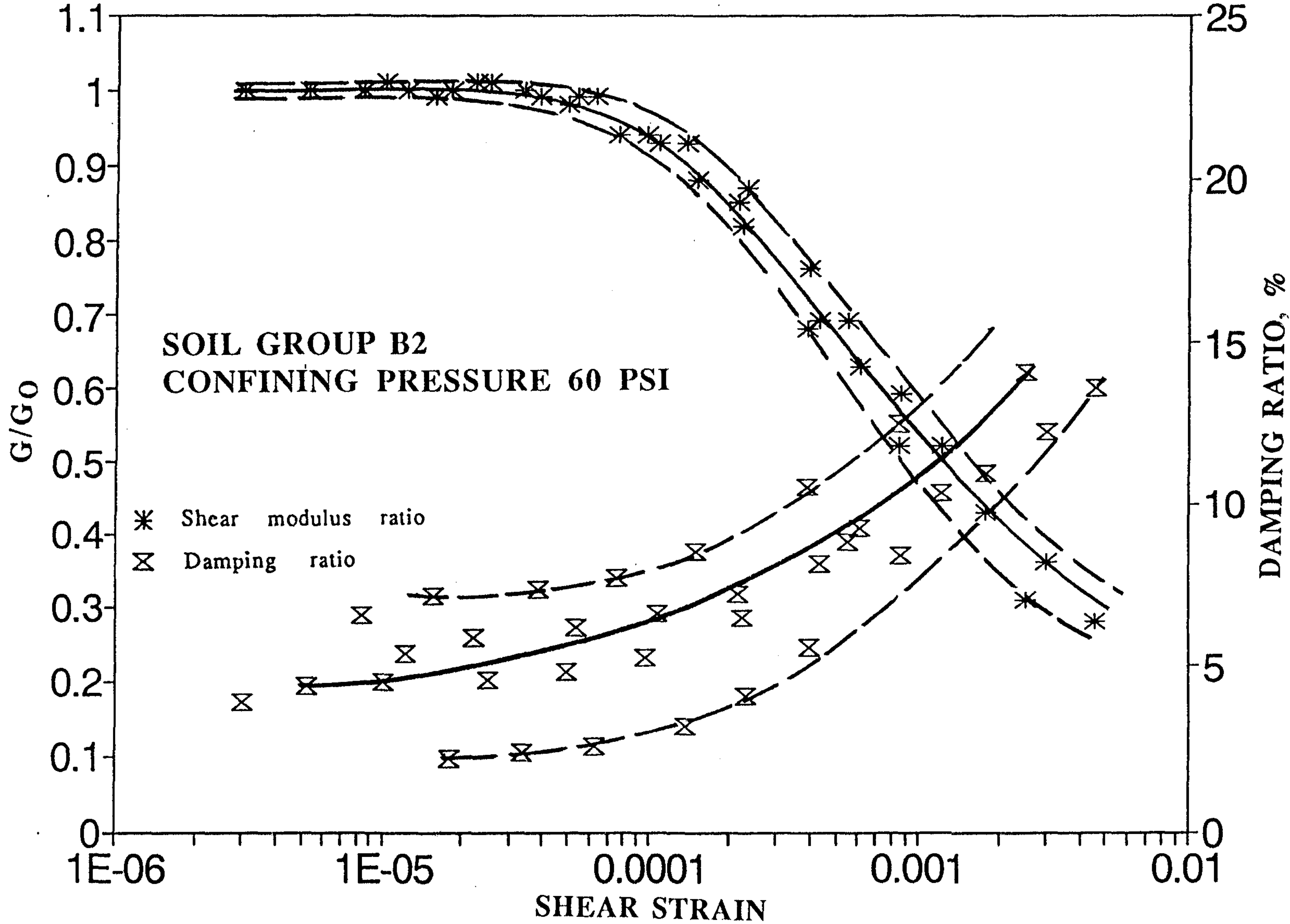

Figure 6-15 Shear Modulus Degradation Curves for Soil Group B2 at 60 psi

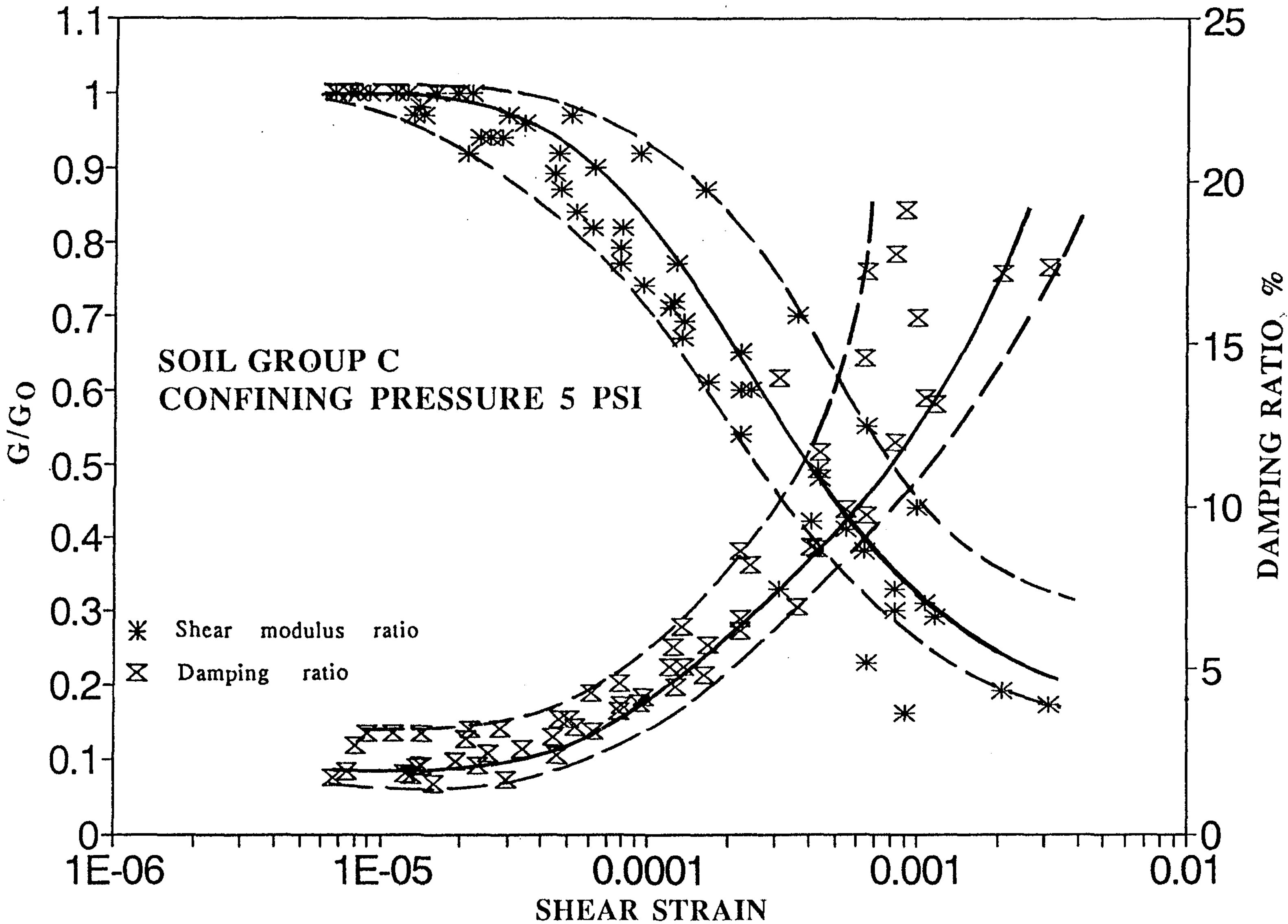

Figure 6-16 Shear Modulus Degradation Curves for Soil Group C at 5 psi

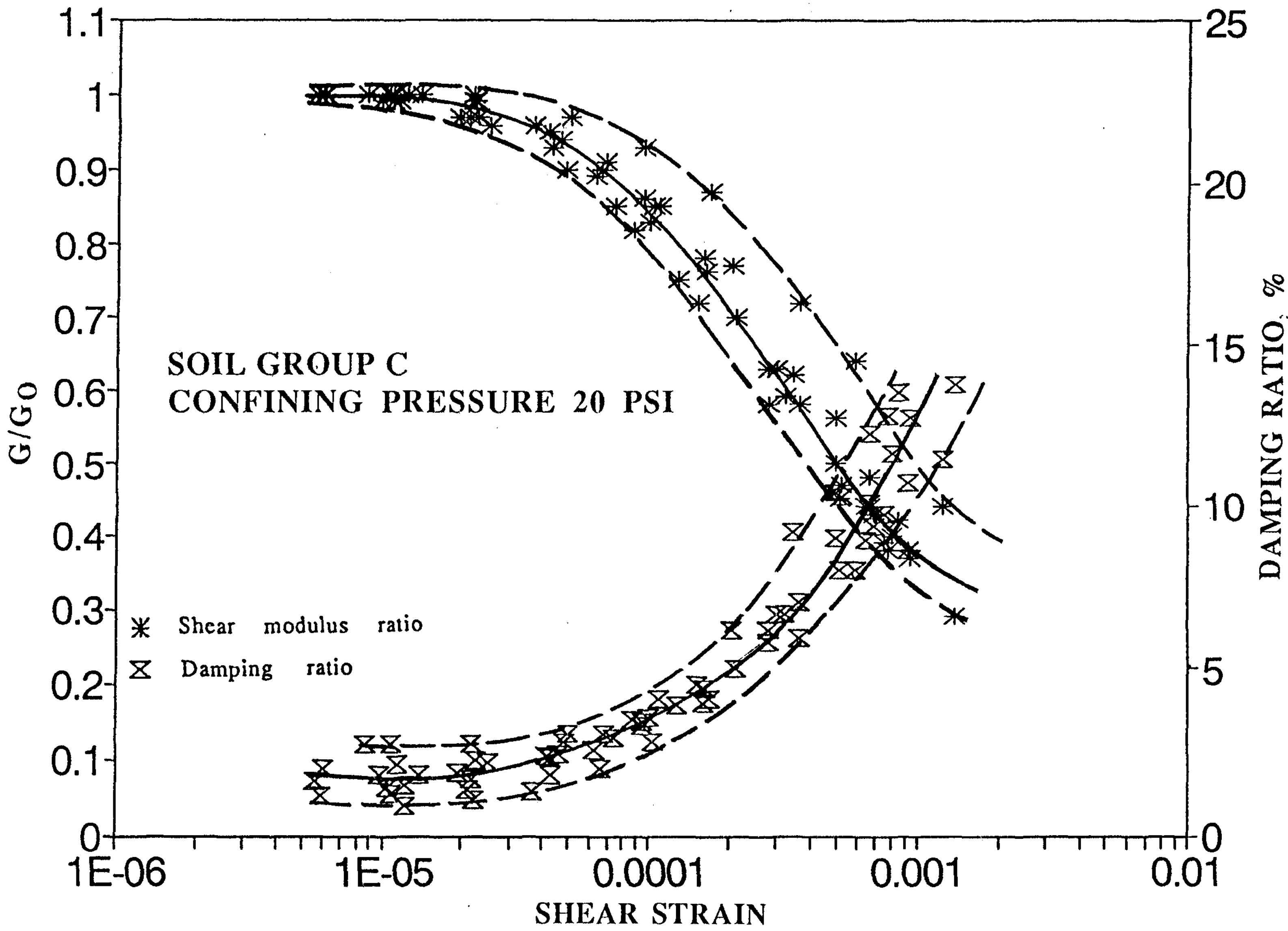

Figure 6-17 Shear Modulus Degradation Curves for Soil Group C at 20 psi

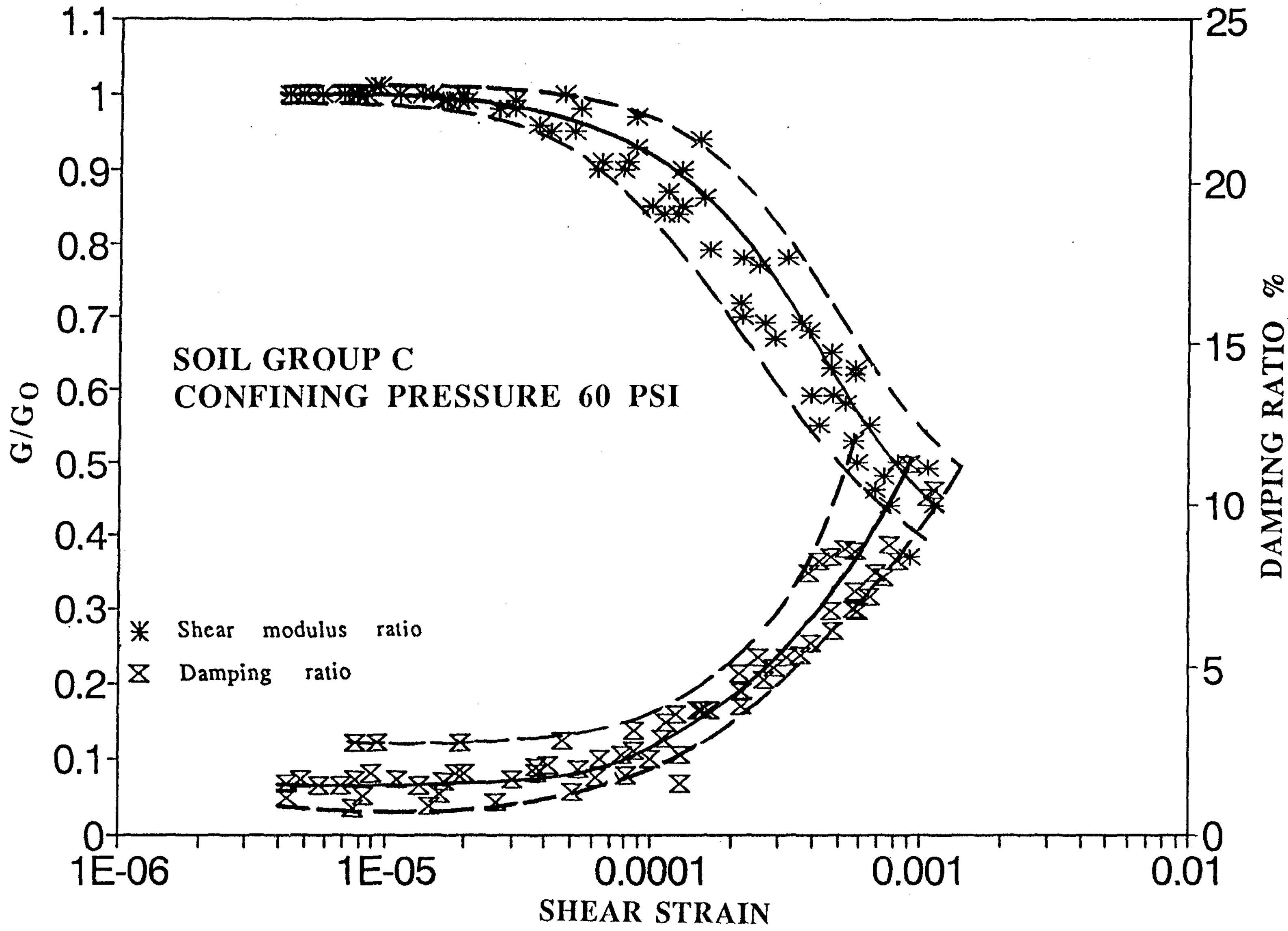

Figure 6-18 Shear Modulus Degradation Curves for Soil Group C at 60 psi

strain increases, the damping ratio increases and modulus ratio decreases. These clear modulus-strain degradation characteristics agree well with the findings from previous studies.

Under cyclic loading, soil exhibit significant nonlinear stress-strain behavior. A typical hysteresis loop for a soil is depicted in Figure 6-19. Two models that are commonly used to describe the hysteretic behavior of soils are the Martin-Davidenkov and Ramberg-Osgood models [10, 11, 12]. The Martin-Davidenkov model can be expressed by the equation

$$\frac{G(\gamma)}{G_o} = 1 - \left[\frac{\left(\frac{\gamma}{\gamma_r}\right)^{2B}}{1 + \left(\frac{\gamma}{\gamma_r}\right)^{2B}} \right]^A \tag{6.1}$$

where $G(\gamma)$ = modulus at any shear strain γ

G_o = low-amplitude shear modulus at shear strain of

γ_r = reference strain

A, B = constants

for which reference strain is defined as $\gamma_r = \frac{\tau_{max}}{G_o}$

where τ_{max} = shear strength, S_u

The Ramberg-Osgood model for secant modulus can be expressed as

$$\frac{G(\gamma)}{G_o} = \frac{1}{1 + \alpha \left| \frac{1}{C_1} \frac{\gamma}{\gamma_r} \frac{G(\gamma)}{G_o} \right|^{R-1}} \tag{6.2}$$

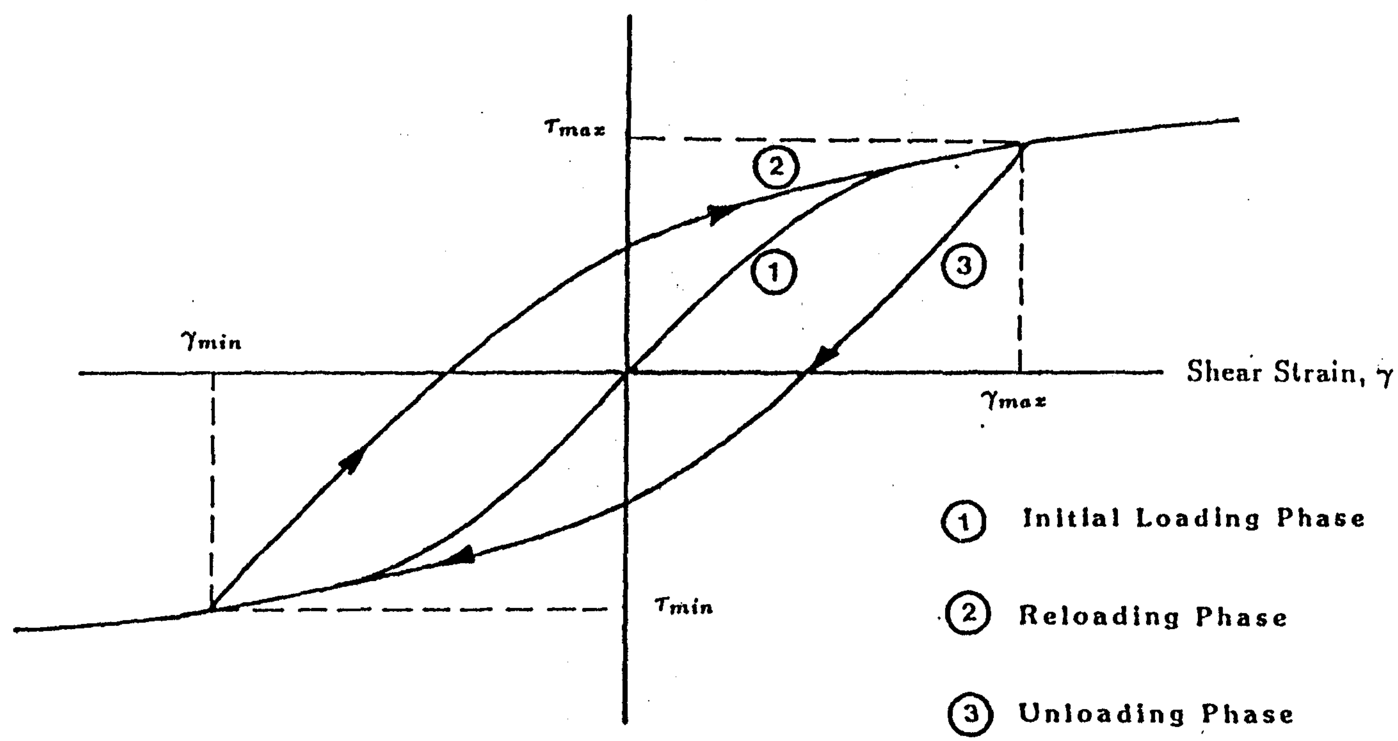

Figure 6-19 Loading Paths of Hysteresis Loop

The parameters A and B are used to describe the detailed nonlinear behavior of soils in the Martin-Davidenkov model. The variable soil parameters used in the Ramberg-Osgood model are C_1, α, and R. From the results of the high-strain tests, as shown in Figures 6-1 to 6-18, the parameters for the two models can be determined by using nonlinear regression analysis. An example of the results of the type of analogs is summarized in Table 6-1 for soil group A1. The results for other soil groups are presented in Appendix B. The parameters for both models on the basis of average behavior of each soil group is included in Table 6-2. Table 6-2 provides (1) accurate and reliable average nonlinear behavior coefficients for a representative sample of sediments in the northern Mississippi Embayment region for seismic studies in the NMSZ, and (2) a comparison and examination of the existing models for describing the nonlinear behavior of soils. Comparison with another study has shown that the high-strain nonlinear behavior of the soils in the northern Mississippi Embayment region may be somewhat different than that of soils elsewhere. More data are required to further study the regional differences and their effects on seismic studies in the earthquake-prone areas [6, 10, 11, 17].

6.2 Threshold Elastic Strain of Soils

The range of elastic strain of soils is very important in a nonlinear strong ground motion study. All the results indicate that the modulus ratio versus strain curves shift to the upper right corner on the plot (Figure 6-20). This implies that there is an elongation of the elastic

Table 6-1 Parameters of Soil Group A1 for Martin-Davidenkov Model

Sample No.	Void Ratio (e)	Confining Pressure (psi)	High-Strain Parameters A	B	γ_o
090601	0.67	5	4.8440	0.3914	0.0003772
		20	1.5486	0.3831	0.0003423
		55	2.2050	0.3421	0.0002711
	0.68	5	29.3613	0.2704	0.0000005744
		20	1.7986	0.3774	0.0004032
		60	2.0530	0.3729	0.0002507
	0.49	5	3.1259	0.3673	0.00007244
		20	1.8381	0.4119	0.0002624
		55	1.7692	0.3841	0.0004537
090604	0.60	5	2.2078	0.3234	0.00007121
		20	--	--	--
		55	0.5809	1.1173	0.00004030
	0.55	5	26.9950	0.3303	0.00000172
		20	2.6027	0.4061	0.0001372
		55	--	--	--
	0.64	5	3.4154	0.2787	0.0001180
		20	1.3774	0.4103	0.0004933
		55	24.8288	0.2576	0.00000082
	0.47	5	--	--	--
		20	5.6564	0.3061	0.00001745
		55	35.4322	0.2465	0.0000003125
101708	0.84	5	2.3047	0.4150	0.00008822
		20	3.9227	0.3217	0.00004862
		60	3.3377	0.2905	0.00008243
	0.70	5	--	--	--
		20	0.28718	0.3417	0.00009785
		60	7.2487	0.2673	0.00001444
	0.59	5	0.6141	0.6432	0.0003796
		20	9.3381	0.2178	0.000002036
		60	0.7137	0.4823	0.001393

Table 6-1 Continued

Sample No.	Void Ratio (e)	Confining Pressure (Psi)	High-Strain Parameters		
			A	B	γ_o
101709	0.58	5	4.3590	0.3492	0.00003914
		20	--	--	--
		55	--	--	--
	0.71	5	3.4414	0.3473	0.00006616
		20	1.8673	0.4851	0.0002625
		55	0.7084	0.7682	0.0008845
	0.77	5	0.6388	0.9364	0.0004283
		20	1.8756	0.4077	0.0002126
		55	3.3719	0.3470	0.00009485
101713	0.41	5	1.1674	0.45782	0.00054924
		20	1.9734	0.40804	0.00029197
		55	--	--	--
	0.53	5	4.7556	0.34923	0.00030336
		20	20.373	0.30256	0.0000026338
		55	13.594	0.27865	0.0000049236
	0.48	5	2.0669	0.39426	0.0001434
		20	1.73913	0.41026	0.00022499
		55	1.36131	0.43109	0.0003853
111930	0.55	5	5.3821	0.3009	0.00002191
		20	2.5645	0.3737	0.0001335
		55	1.9292	0.4186	0.0002707
	0.67	5	1.7737	0.4217	0.0001480
		20	--	--	--
		60	--	--	--
	0.69	5	2.3802	0.4547	0.0000776
		20	--	--	--
		55	--	--	--
	0.78	5	4.9732	0.3203	0.00001953
		20	--	--	--
		55	--	--	--

Table 6-1 Continued

Sample No.	Void Ratio (e)	Confining Pressure (Psi)	High-Strain Parameters		
			A	B	γ_o
NMSZ41	0.64	6	0.7954	0.48488	0.0006433
		20	1.2029	0.3784	0.0005311
		55	0.5412	0.7283	0.001979
	0.60	6	0.8998	0.3999	0.0005150
		20	2.0157	0.3525	0.0002497
	0.37	5	4.9005	0.3649	0.00002963
		20	6.5704	0.3174	0.000016335
		55	2.5540	0.3128	0.0001455

Table 6-2 High-Strain Parameters for Average Behavior of Various Soil Groups

Sample Group	Confining Pressure (psi)	Reference Strain $\gamma_o \times 10^{-3}$ ($\gamma_o = \tau/G_o$)	Martin-Davidenkov		Ramberg-Osgood		
			A	B	R	C_1	α
A_1	5	0.324	1.412	0.380	2.35	0.8	1.149
	20	0.652	1.362	0.325	1.99	0.8	0.999
	55	1.030	1.225	0.335	1.89	0.8	1.135
A2	5	0.266	1.303	0.365	1.92	0.8	1.241
	20	0.560	1.172	0.325	2.32	0.8	1.460
	55	0.895	1.680	0.200	2.00	0.8	0.852
A3	5	0.307	1.646	0.275	1.98	0.8	1.564
	20	0.642	1.867	0.205	2.04	0.8	0.663
	60	1.131	1.557	0.290	2.19	0.8	1.006
B1	5	1.095	0.819	0.405	2.38	0.4	2.308
	20	0.760	0.920	0.470	2.56	0.4	1.430
	55	0.467	1.629	0.340	2.31	0.4	0.326
B2	5	3.843	0.654	0.500	2.35	0.4	1.731
	20	2.597	0.880	0.430	2.48	0.4	4.423
	60	2.309	0.934	0.425	2.34	0.4	1.444
C	5	0.706	1.049	0.385	2.43	0.5	1.637
	20	0.430	1.514	0.305	2.27	0.5	0.537
	60	0.245	2.636	0.275	2.13	0.5	0.152

Table 6-3 High-Strain Parameters (Typical soil results from refs. 11 and 17)

Soil Type	High-Strain Parameters
Sand	α = 1.0 C = 0.8 R = 3.0
Clay	α = 1.8 C = 0.5 R = 2.5

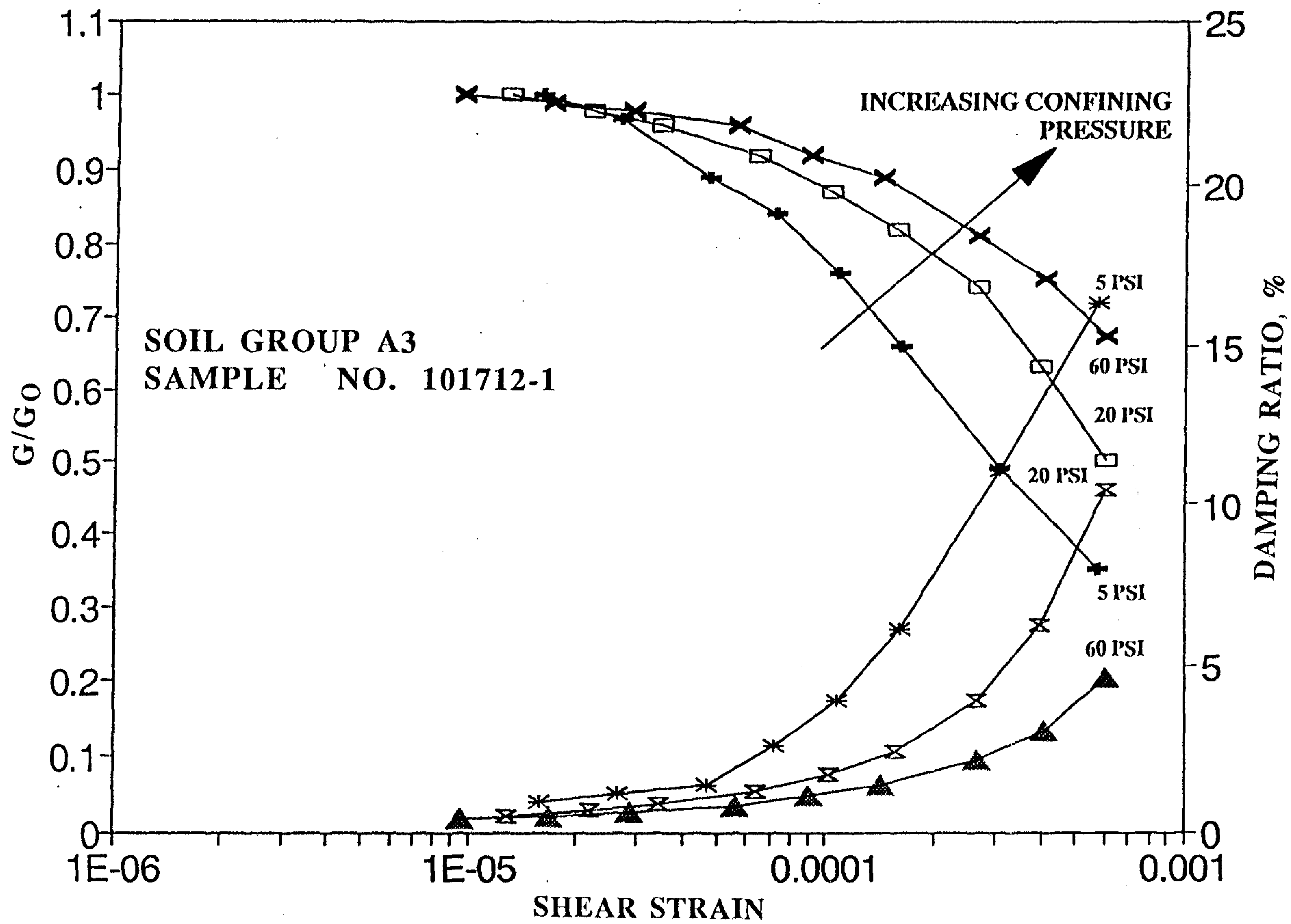

Figure 6-20 Shear Modulus Degradation Curves for Soil Group A3 (Sample 101712-1)

range as confining pressure increases. On the basis of this finding, the threshold of elastic strain is indicated at about 90% of modulus ratio (G/G_0) from a plot of modulus ratio and damping ratio as a function of shear strain. See Figure 6-21 for one of the samples from soil group A2. A significant decrease in shear modulus is observed beyond this threshold elastic strain. Also, the flat plateau of modulus ratio curve is more defined as the confining pressure increases. In the study, the shear-strain amplitude corresponding to $G/G_0 = 0.9$ is defined as "threshold elastic strain" and is obtained for soils at each of the confining pressures.

Figure 6-22 is a plot of threshold elastic strain versus confining pressure for samples in soil groups A, whereas Figure 6-23 shows the same plot for one sample in each of the soil groups B1, B2, and C. The figures show that for increasing confining pressure, the threshold elastic strain also increases. Preliminarily, this infers that for a site subjected to earthquake shaking, the lower portion of the soil profile below a certain depth would behave like a rigid body. For studies about threshold elastic strain, these findings should be confirmed with more tests results at higher pressures.

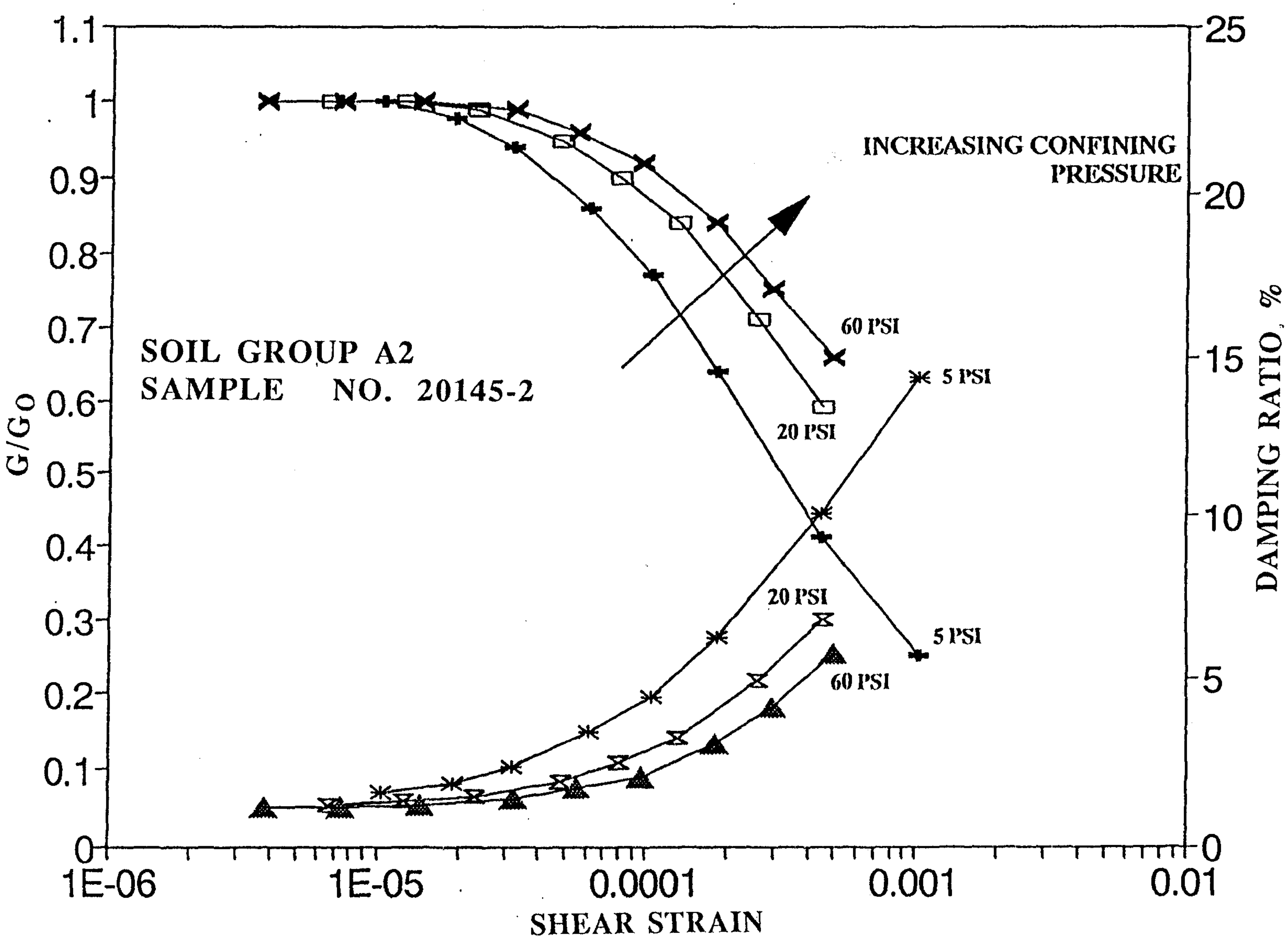

Figure 6-21 Shear Modulus Degradation Curves for Soil Group A2 (Sample 20145-2)

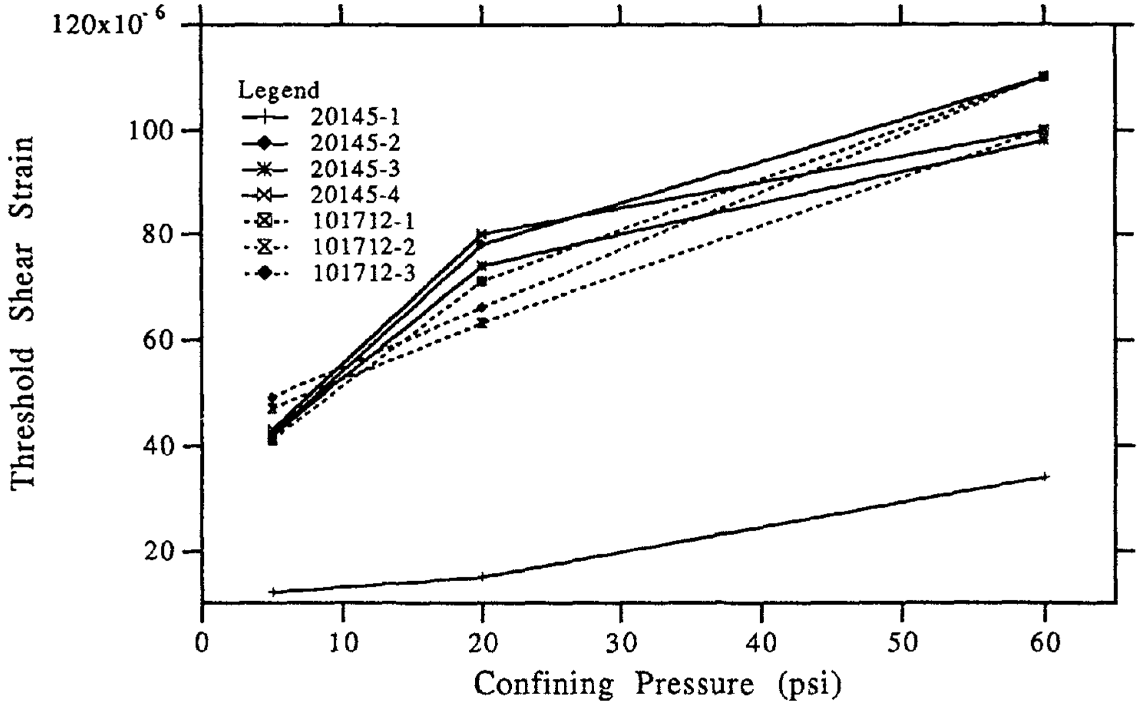

Figure 6-22 Shear Strain vs Confining Pressure for Samples 20145 and 101712

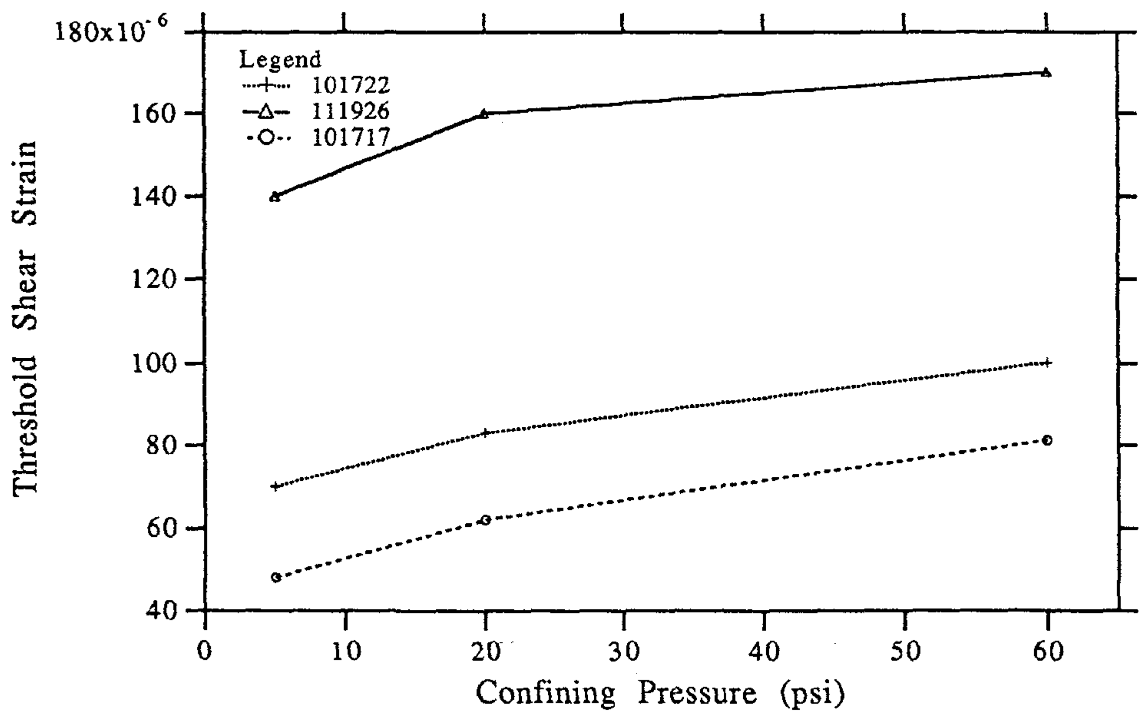

Figure 6-23 Shear Strain vs Confining Pressure for Samples 101722, 111926 and 101717

CHAPTER 7

CONCLUSIONS AND FUTURE STUDIES

The failure of structures and earthworks related to the soil profile of a site under earthquake shaking has been seen from historical events such as the Niigata earthquake and Mexico City earthquake. Thus, acquiring actual and realistic data for understanding seismic characteristics of soils, including damping ratio, and low-and high-strain characteristics of soils in an earthquake-prone region, is essential and more reliable for seismic studies. The research conducted provided seismic characteristics of soils of the sediments in the northern Mississippi Embayment.

7.1 Conclusions

Based on the results of the testing program, the following conclusions can be drawn.

1. A systematic dynamic testing program for determining seismic characteristics of sediments provides an accurate and reliable data base for regional seismic studies. The tabulated data developed in the study include shear-wave velocity, shear modulus, and modulus degradation characteristics of the representative soil groups in the NMSZ under various simulated site conditions. These data are constructed for future reference and applications. Empirical equations for estimating shear-wave velocity and shear modulus of soils in the northern Mississippi Embayment region are also derived.

2. Results of dynamic testing on natural sediments generally agree with results from previous tests on selected ideal soils. The shear modulus and shear-wave velocity increases with decreasing void ratio and increasing confining pressure. On the other hand, damping ratio decreases as confining pressure increases.

3. Because of a significant difference in constituents of natural sediments, the results of shear modulus versus void ratio and shear-wave velocity versus void ratio are scattered. However, the average values of "n" of each soil group (n = 0.5 to 0.54) are essentially the same as recommended by the existing Hardin's equation (n = 0.5). Our results support the suggestion by the Hardin equation concerning the effect of confining pressure on shear modulus of sand: shear modulus of sand is proportional to the square root of confining pressure.

4. The differences in the estimated shear modulus and shear-wave velocity of soils using the empirical equations derived in the study and the existing Hardin's equation range from -13% to 12%.

5. The nonlinear behavior of sediments in the NMSZ may be somewhat different than that of the soils in other regions, as revealed in this study by both the Ramberg-Osgood and Martin-Davidenkov models. The average nonlinear characteristics of each representative soil group are documented in a tabular format for future reference and applications.

6. Threshold elastic shear strain tends to increase with increasing confining pressure for all the soils tested. Preliminarily, this infers that for a site subjected to earthquake shaking, the lower portion of the profile under a certain depth would behave like a rigid body.

7.2 Future Studies

Future studies regarding the seismic characteristics of the sediments in the northern Mississippi Embayment are listed as follows:

1. Seismic characteristics of loess (silty material) are not clearly defined in this study. This material may behave either like a cohesive soil or like a cohesionless soil, depending upon the grain size distribution of the loess. Future studies are required to comprehend the seismic characteristics of loess material that is widespread in the New Madrid seismic zone.

2. High-strain dynamic properties of soils under high pressures are necessary for a simpler and better modeling of soil profiles for strong ground motion studies.

3. Nonlinear behavior of soils in the northern Mississippi Embayment should be compared with that of soils in other earthquake-prone areas to investigate their effect on regional seismic studies.

REFERENCES

1. American Society for Testing and Materials, (1989). Annual book of ASTM standards, Philadelphia, Pennsylvania.

2. Bowles, J. E. (1978). Engineering properties of soils and their measurement 2nd ed. McGraw-Hill Book Co., New York.

3. Center for Earthquake Research and Information, Memphis State University, Seismic Network Records.

4. Chang, T. S. (1986). "Dynamic behavior of cemented sand," Dissertation presented to University of Michigan, Ann Arbour, Michigan, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

5. Chang, T. S., and Woods, R. D., (1987). "Effect of confining pressure on shear modulus of cemented sand," Development in geotechnical engineering 43, International Conference on Soil Dynamics and Earthquake Engineering, Princeton University, New Jersey, 193-208.

6. Dobry, R., and Vucetic M. (1987). "Dynamic properties and seismic response of soft clay deposits," Proceedings International Symposium on Geotechnical Engineering of Soft Soils, Mexico City, August 1987, Vol. 2, 51-87.

7. "Drnevich Long-Tor resonant column apparatus operating manual," Soil Dynamics Instruments, Inc., Lexington, Kentucky, January 1990.

8. Johnston, A. C., and Nava, S. J. (1985). "Recurrence rates and probability estimates for the New Madrid seismic zone," Journal of Geophysical Research, V. 90, No. B7.

9. M & H Engineering & Memphis State University (1974). Regional earthquake risk study," Technical report for Mississippi-Arkansas-Tennessee council of Governments, Memphis Delta Development District.

10. Martin, P. P. (1976). "Non linear methods for dynamic analysis of ground response," Dissertation presented to University of California, Berkeley, California, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

11. Prange, B. "Dynamic response and wave propagation in soils," Editor, Prange, B., Institute of Soil and Rock Mechanics, Rotterdam, Netherlands, 1978.

12. Richart, F. E., Jr., Hall, J. R., and Woods, R. D., (1970). Vibration of soils and foundations. Prentice-Hall, Inc., Englewood Cliff, New Jersey, 414.

13. Seed, H. B., and Idriss, I. M., (1970). "Soil moduli and damping factors for dynamic response analysis," Report No. EERC 70-10, University of California, Berkerly.

14. Seed, H. B., Whitman, R. V., Dezfulian, H. Dobry, R. and Idriss, I. M., (1972). "Soil conditions and building damage in 1967 Caracas earthquake", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM8.

15. Sharma, S., and Kovacs, W. D. (1982). "Preliminary microzonation of the Memphis, Tennessee", Bulletin of the Seismological Society of America, Vol. 72, No. 3, 1011-1024.

16. "User's manual of Fortran subroutines for statistical analysis," Version 1.0, International Mathematical and Statistical Libraries, Inc., Houston, Texas, April 1987.

17. Woods, R. D. Course notes for course CE 648, University of Michigan, Ann Arbour, Michigan, Spring, 1988.

Appendix A

Low-Strain Seismic Characteristics for Sample 090604

Void Ratio	Confining Pressure	Unit					
	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
0.6	Shear Wave Velocity	m/sec	150	217	269	341	365
		ft/sec	492	712	883	1119	1198
	Shear Modulus	MPa	37.2	77.7	121	191	221
		psi	5382.3	11242.0	17506.8	27634.7	31975.2
	Damping Ratio	%	1.65	0.95	0.78	0.55	0.60
0.55	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	173	251	305	372	399
		ft/sec	568	823	1001	1220	1309
	Shear Modulus	MPa	51	107	158	235	271
		psi	7378.9	15481.2	22860.1	34000.8	39209.4
	Damping Ratio	%	1.33	0.91	0.70	0.55	0.60
0.64	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	153	193	241	296	321
		ft/sec	502	633	791	971	1053
	Shear Modulus	MPa	37	59	93	140	163
		psi	5353.3	8536.4	13455.6	20255.8	23583.5
	Damping Ratio	%	1.75	1.30	1.01	0.88	0.75
0.47	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	180	251	300	364	385
		ft/sec	591	823	984	1194	1263
	Shear Modulus	MPa	54	105	149	220	247
		psi	7812.9	15191.8	21557.9	31830.5	35737.0
	Damping Ratio	%	1.55	0.90	0.79	0.69	0.80

Low-Strain Seismic Characteristics for Sample 101708

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.84	Shear Wave Velocity	m/sec	204	222	263	319	352
		ft/sec	669	728	863	1047	1155
	Shear Modulus	MPa	62	74	104	154	189
		psi	8970.4	10706.6	15047.2	22281.4	27345.3
	Damping Ratio	%	4.50	3.95	3.30	2.75	2.35
0.70	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	195	222	264	330	362
		ft/sec	640	728	866	1083	1188
	Shear Modulus	MPa	55	72	101	158	192
		psi	7957.6	10417.3	14613.1	22860.1	27779.4
	Damping Ratio	%	1.50	1.26	1.15	0.98	0.90
0.59	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	245	252	294	351	375
		ft/sec	804	827	965	1152	1230
	Shear Modulus	MPa	98	104	141	200	232
		psi	14179.1	15047.2	20400.5	28936.8	33566.7
	Damping Ratio	%	3.85	3.59	2.80	2.45	2.26

Low-Strain Seismic Characteristics for Sample 101709

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.58	Shear Wave Velocity	m/sec	278	314	342	393	418
		ft/sec	912	1030	1122	1289	1371
	Shear Modulus	MPa	128	164	194	257	292
		psi	18519.6	23728.2	28068.7	37183.8	42247.8
	Damping Ratio	%	1.35	1.25	1.18	1.20	1.20
0.71	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	264	289	320	366	384
		ft/sec	866	948	1050	1201	1260
	Shear Modulus	MPa	123	148	181	240	264
		psi	17796.2	21413.3	26187.8	34724.2	38196.6
	Damping Ratio	%	1.40	1.30	1.35	1.20	1.17
0.77	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	214	228	271	336	381
		ft/sec	702	748	889	1102	1250
	Shear Modulus	MPa	68	77	109	168	216
		psi	9838.5	11140.7	15770.6	24306.9	31251.8
	Damping Ratio	%	1.67	1.61	1.52	1.45	1.35

Low–Strain Seismic Characteristics for Sample 101713

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.41	Shear Wave Velocity	m/sec	230.4	248.7	282.6	333.3	356.3
		ft/sec	756	816	927	1094	1169
	Shear Modulus	MPa	95.2	112	144	201	230
		psi	13773.9	16204.6	20834.5	29081.5	33277.4
	Damping Ratio	%	1.35	1.27	1.21	1.17	1.17
0.53	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	169.8	178	228	291	329
		ft/sec	557	584	748	955	1079
	Shear Modulus	MPa	69.9	77	126	207	266
		psi	10113.4	11140.7	18230.2	29949.6	38486.0
	Damping Ratio	%	1.81	1.55	1.21	1.19	1.13
0.48	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	224	231.4	263.1	245.7	286.4
		ft/sec	735	759	863	806	940
	Shear Modulus	MPa	91.8	98	127	110	151
		psi	13282.0	14179.1	18374.9	15915.3	21847.3
	Damping Ratio	%	1.65	1.50	1.45	1.50	1.41

Low-Strain Seismic Characteristics for Sample 111930

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.55	Shear Wave Velocity	m/sec	270	300	329	370	384
		ft/sec	886	984	1079	1214	1260
	Shear Modulus	MPa	115.9	144	173.7	217.5	235.9
		psi	16768.9	20834.5	25131.6	31468.8	34131.0
	Damping Ratio	%	0.89	0.82	0.75	0.72	0.73
0.67	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	206	231	275	336	371
		ft/sec	676	758	902	1102	1217
	Shear Modulus	MPa	66	84	118	178	218
		psi	9549.2	12153.5	17072.7	25753.8	31541.2
	Damping Ratio	%	1.28	1.15	1.01	0.95	0.93
0.685	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	210	224	278	342	378
		ft/sec	689	735	912	1122	1240
	Shear Modulus	MPa	72.5	82.9	128	196	241
		psi	10489.6	11994.3	18519.6	28358.1	34868.9
	Damping Ratio	%	1.20	1.25	1.15	0.92	0.85
0.78	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	182	229	277	338	373
		ft/sec	597	751	909	1109	1224
	Shear Modulus	MPa	68.2	108	159	240	296
		psi	9867.5	15625.9	23004.8	34724.2	42826.5
	Damping Ratio	%	1.6	1.25	1.15	1.00	0.95

Low-Strain Seismic Characteristics for Sample NMSZ41

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.64	Shear Wave Velocity	m/sec	-	-	-	-	-
		ft/sec	-	-	-	-	-
	Shear Modulus	MPa	46.24	87.34	138.05	246.72	287.86
		psi	6690.2	12636.7	19973.7	35696.5	41648.8
	Damping Ratio	%	0.83	0.62	0.55	0.61	0.65
0.6	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	-	-	-	-	-
		ft/sec	-	-	-	-	-
	Shear Modulus	MPa	41.37	68.95	137.9	275.8	379.22
		psi	5985.6	9976.0	19952.0	39903.9	54867.1
	Damping Ratio	%	0.52	0.43	0.32	0.27	0.22
0.37	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	179	242	290	352	377
		ft/sec	587	794	951	1155	1237
	Shear Modulus	MPa	60	110	159	233	269
		psi	8681.1	15915.3	23004.8	33711.4	38920.0
	Damping Ratio	%	1.50	1.10	0.90	0.75	0.69

Low-Strain Seismic Characteristics for Sample 121433

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.88	Shear Wave Velocity	m/sec	217.3	235.3	293.94	377.8	424.6
		ft/sec	713	772	964	1240	1393
	Shear Modulus	MPa	80.1	94.1	147.1	243.7	308.4
		psi	11589.2	13614.8	21283.0	35259.5	44620.6
	Damping Ratio	%	1.50	1.35	1.20	1.10	1.05
0.44	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	241	286	328	393	420
		ft/sec	791	938	1076	1289	1378
	Shear Modulus	MPa	104.5	148	193	279	321
		psi	15119.5	21413.3	27924.1	40366.9	46443.6
	Damping Ratio	%	5.50	4.70	4.22	4.00	3.66
0.37	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	183	229	282	331	365
		ft/sec	600	751	925	1086	1198
	Shear Modulus	MPa	68	107	161	222	273
		psi	9838.5	15481.2	23294.2	32119.9	39498.8
	Damping Ratio	%	2.14	1.75	1.35	1.19	1.15

Low-Strain Seismic Characteristics for Sample 011442

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.49	Shear Wave Velocity	m/sec	193	218	261	332	373
		ft/sec	633	715	856	1089	1224
	Shear Modulus	MPa	78	99	142	233	296
		psi	11285.4	14323.7	20545.2	33711.4	42826.5
	Damping Ratio	%	1.58	1.45	1.30	1.10	1.23
1.65	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	242	281	339	425	482
		ft/sec	794	922	1112	1394	1581
	Shear Modulus	MPa	73	99	143	228	298
		psi	10561.9	14323.7	20689.8	32988.0	43115.9
	Damping Ratio	%	3.30	1.50	1.55	1.25	1.10

Low–Strain Seismic Characteristics for Sample 020145

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.49	Shear Wave Velocity	m/sec	196	236	267	309	327
		ft/sec	643	774	876	1014	1073
	Shear Modulus	MPa	79.5	116.5	148.5	200.5	224.6
		psi	11502.4	16855.7	21485.6	29009.2	32496.1
	Damping Ratio	%	7.01	6.02	5.45	4.24	4.30
0.45	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	195	227	275	345	388
		ft/sec	640	745	902	1132	1273
	Shear Modulus	MPa	72.3	99.3	146.6	231.2	294.7
		psi	10460.7	14367.1	21210.7	33451.0	42638.4
	Damping Ratio	%	1.60	1.42	1.30	1.25	1.16
0.58	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	193	196	252	331	373
		ft/sec	633	643	827	1086	1224
	Shear Modulus	MPa	79.3	82.6	137.1	237.3	306.3
		psi	11473.5	11950.9	19836.2	34333.6	44316.8
	Damping Ratio	%	1.85	1.75	1.45	1.27	1.06
0.52	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	223	234	272	348	391
		ft/sec	732	768	892	1142	1283
	Shear Modulus	MPa	96.5	106.5	144.5	236.9	300.2
		psi	13962.0	15408.9	20906.9	34275.7	43434.2
	Damping Ratio	%	1.70	1.50	1.44	1.37	1.30

Low-Strain Seismic Characteristics for Sample 021246

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.4	Shear Wave Velocity	m/sec	194	213	276	358	405
		ft/sec	636	699	906	1175	1329
	Shear Modulus	MPa	88	108	180	306	391
		psi	12732.2	15625.9	26043.2	44273.4	56571.5
	Damping Ratio	%	1.56	1.45	1.15	.85	.93
0.32	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	218	254	319	401	445
		ft/sec	715	833	1047	1316	1460
	Shear Modulus	MPa	104	141	222	352	433
		psi	15047.2	20400.5	32119.9	50928.8	62648.3
	Damping Ratio	%	1.85	1.65	1.29	1.05	.95
0.31	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	205.6	246	311	404	449
		ft/sec	675	807	1020	1325	1473
	Shear Modulus	MPa	99	142	227	383	472
		psi	14323.7	20545.2	32843.3	55414.0	68290.9
	Damping Ratio	%	1.65	1.40	1.10	.95	.85

Low-Strain Seismic Characteristics for Sample 021549

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.22	Shear Wave Velocity	m/sec	145.2	165.8	221	270	296
		ft/sec	476	544	725	886	971
	Shear Modulus	MPa	48	63	110	165	200
		psi	6944.8	9115.1	15915.3	23872.9	28936.8
	Damping Ratio	%	7.25	3.64	2.42	1.86	1.61
0.53	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	210	239	285	352	388
		ft/sec	689	784	935	1155	1273
	Shear Modulus	MPa	90.6	117.6	167.4	256.3	313.4
		psi	13108.4	17014.9	24220.1	37082.6	45344.0
	Damping Ratio	%	3.23	2.59	1.97	1.05	1.00
0.39	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	222	258	310	388	423
		ft/sec	728	846	1017	1273	1388
	Shear Modulus	MPa	101	137	199	312	371
		psi	14613.1	19821.7	28792.2	45141.5	53677.8
	Damping Ratio	%	1.96	1.70	1.60	1.52	1.35
0.41	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	218	251	302	379	416
		ft/sec	715	823	991	1243	1365
	Shear Modulus	MPa	115	147	215	339	410
		psi	16638.7	21268.6	31107.1	49047.9	59320.5
	Damping Ratio	%	2.20	1.80	1.60	1.25	1.20

Low-Strain Seismic Characteristics for Sample 021553

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.55	Shear Wave Velocity	m/sec	155	194	225	266	291
		ft/sec	509	636	738	873	955
	Shear Modulus	MPa	56.2	88.3	120.1	170.1	206.5
		psi	8131.3	12775.6	17376.6	24610.8	29877.3
	Damping Ratio	%	2.85	2.35	2.15	1.95	1.87
0.68	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	147	184	214	253	276
		ft/sec	482	604	702	830	906
	Shear Modulus	MPa	39.5	62	83.5	118.5	143.4
		psi	5715.0	8970.4	12081.1	17145.1	20747.7
	Damping Ratio	%	2.96	2.50	2.40	2.10	2.00
0.59	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	179	230	280	333	374
		ft/sec	587	755	919	1093	1227
	Shear Modulus	MPa	67	110	165	238	300
		psi	9693.8	15915.3	23872.9	34434.8	43405.3
	Damping Ratio	%	2.30	1.60	1.40	1.08	1.04

Low-Strain Sesismic Characteristics for Sample 101712

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.40	Shear Wave Velocity	m/sec	185	249	294	348	380
		ft/sec	607	817	965	1142	1247
	Shear Modulus	MPa	58	106	148	208	248
		psi	8391.7	15336.5	21413.3	30094.3	35881.7
	Damping Ratio	%	.95	.65	.55	.41	.41
0.54	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	182	244	290	347	381
		ft/sec	597	801	951	1138	1250
	Shear Modulus	MPa	55	99	140	201	242
		psi	7957.6	14323.7	20255.8	29081.5	35013.6
	Damping Ratio	%	1.05	.70	.55	.44	.42
0.59	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	173	227	271	328	363
		ft/sec	568	745	889	1076	1191
	Shear Modulus	MPa	50	86	123	179	221
		psi	7234.2	12442.8	17796.2	25898.5	31975.2
	Damping Ratio	%	1.1	0.8	0.62	0.47	0.44

Low-Strain Sesismic Characteristics for Sample 010434

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.63	Shear Wave Velocity	m/sec	154	192	228	274	302
		ft/sec	505	630	748	899	991
	Shear Modulus	MPa	51	78.5	110.4	159.5	195.5
		psi	7378.9	11357.7	15973.1	23077.1	28285.8
	Damping Ratio	%	0.90	0.45	0.32	0.23	0.19
0.57	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	191	240	282.3	331.2	361.5
		ft/sec	627	787	926	1087	1186
	Shear Modulus	MPa	62.5	98.5	136.8	187.5	224
		psi	9042.8	14251.4	19792.8	27128.3	32409.3
	Damping Ratio	%	0.90	0.65	0.55	0.49	0.46
0.69	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	203.4	238.2	278.8	332	360
		ft/sec	667	781	915	1089	1181
	Shear Modulus	MPa	64.5	88.5	121.5	172.5	201.9
		psi	9332.1	12804.6	17579.1	24958.0	29211.7
	Damping Ratio	%	3.95	3.40	2.75	1.95	1.65
0.72	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	172	222	262	315	348
		ft/sec	564	728	860	1033	1142
	Shear Modulus	MPa	53.6	88.5	124.1	181.5	220.5
		psi	7755.1	12804.6	17955.3	26260.2	31902.9
	Damping Ratio	%	1.45	0.90	0.65	0.44	0.37
0.62	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	179.5	238.5	279	332.5	364.6
		ft/sec	589	782	915	1091	1196
	Shear Modulus	MPa	60	106.5	145.5	208	250.5
		psi	8681.1	15408.9	21051.6	30094.3	36243.4
	Damping Ratio	%	1.05	0.75	0.57	0.41	0.39

Low-Strain Sesismic Characteristics for Sample 011439

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
0.74	Shear Wave Velocity	m/sec	129	160.2	191.5	234.6	261
		ft/sec	423	526	628	770	856
	Shear Modulus	MPa	32.4	50.1	71.5	108.5	134
		psi	4687.8	7248.7	10344.9	15698.2	19387.7
	Damping Ratio	%	1.10	0.80	0.65	0.48	0.46
0.56	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	204	228	264	326	356
		ft/sec	669	748	866	1070	1168
	Shear Modulus	MPa	82.8	102.8	138.5	211.6	253.5
		psi	11979.9	14873.5	20038.8	30615.2	36677.4
	Damping Ratio	%	2.70	2.45	2.05	1.65	1.44
0.68	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	186	206	255	315	344
		ft/sec	610	676	837	1033	1129
	Shear Modulus	MPa	67.5	83.5	125.2	194.5	234.5
		psi	9766.2	12081.1	18114.5	28141.1	33928.4
	Damping Ratio	%	1.05	0.90	0.71	0.59	0.60
0.71	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	179	208	254	314	341
		ft/sec	587	682	833	1030	1119
	Shear Modulus	MPa	70	95	142	219	260
		psi	10127.9	13745.0	20545.2	31685.8	37617.9
	Damping Ratio	%	1.25	1.00	0.81	0.65	0.61

Low–Strain Sesismic Characteristics for Sample 090602

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.36	Shear Wave Velocity	m/sec	188	186	239	323	362
		ft/sec	617	610	784	1060	1188
	Shear Modulus	MPa	70.7	69.5	116	213	280
		psi	10229.2	10055.6	16783.4	30817.7	40511.6
	Damping Ratio	%	2.35	2.35	1.80	1.55	1.55
0.24	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	323	259	284	296	318
		ft/sec	1060	850	932	971	1043
	Shear Modulus	MPa	117.8	159.1	190.4	211	245
		psi	17043.8	23019.3	27547.9	30528.4	35447.6
	Damping Ratio	%	6.57	5.15	7.25	3.77	3.59
0.17	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	101	112	131	155	168
		ft/sec	331	367	430	509	551
	Shear Modulus	MPa	27.5	33.4	45.6	64.6	75.9
		psi	3978.8	4832.5	6597.6	9346.6	10981.5
	Damping Ratio	%	5.30	5.00	4.70	4.40	4.20
0.13	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	165	182	209	238	250
		ft/sec	541	597	686	781	820
	Shear Modulus	MPa	75	91	121	158	176
		psi	10851.3	13166.3	17506.8	22860.1	25464.4
	Damping Ratio	%	8.50	7.20	6.50	5.90	5.70

Low-Strain Sesismic Characteristics for Sample 101714

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
3.5	Shear Wave Velocity	m/sec	111	109	111	118	119
		ft/sec	364	358	364	387	390
	Shear Modulus	MPa	25	23.9	24.9	27.6	28.2
		psi	3617.1	3458.0	3602.6	3993.3	4080.1
	Damping Ratio	%	2.80	2.55	2.58	2.58	2.59

Low-Strain Sesismic Characteristics for Sample 101715

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
3.5	Shear Wave Velocity	m/sec	140	159	183	219	241
		ft/sec	459	522	600	719	791
	Shear Modulus	MPa	53	68	90	131	160
		psi	7668.3	9838.5	13021.6	18953.6	23149.5
	Damping Ratio	%	2.10	1.65	1.45	1.22	1.15

Low–Strain Sesismic Characteristics for Sample 101720

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
2.0	Shear Wave Velocity	m/sec	143	161	178	204	218
		ft/sec	469	528	584	669	715
	Shear Modulus	MPa	61	78	95	127	145
		psi	8825.7	11285.4	13745.0	18374.9	20979.2
	Damping Ratio	%	4.10	4.15	3.40	3.10	2.95

Low–Strain Sesismic Characteristics for Sample 101722

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
4.5	Shear Wave Velocity	m/sec	143	161	178	204	218
		ft/sec	469	528	584	669	715
	Shear Modulus	MPa	61	78	95	127	145
		psi	8825.7	11285.4	13745.0	18374.9	20979.2
	Damping Ratio	%	4.10	4.15	3.40	3.10	2.95

Low-Strain Sesismic Characteristics for Sample 111928

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
3.75	Shear Wave Velocity	m/sec	195	215	228	258	275
		ft/sec	640	705	748	846	902
	Shear Modulus	MPa	102.5	120.1	138.5	178.5	200.1
		psi	14830.1	17376.6	20038.8	25826.1	28951.3
	Damping Ratio	%	6.01	6.02	5.68	5.50	5.50

Low–Strain Sesismic Characteristics for Sample 010837

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi
1	Shear Wave Velocity	m/sec	110.1	122.1	135.1
		ft/sec	361	401	443
	Shear Modulus	MPa	27.8	33.8	41.6
		psi	4022.2	4890.3	6018.9
	Damping Ratio	%	2.21	2.45	2.45

Low-Strain Seismic Characteristics for Sample 013143

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
[3.5]	Shear Wave Velocity	m/sec	239	253	262	274	283.2
		ft/sec	784	830	860	899	929
	Shear Modulus	MPa	165	186.5	201	220	239
		psi	23872.9	26983.6	29081.5	31830.5	34579.5
	Damping Ratio	%	2.15	1.91	1.88	1.80	1.84

Low-Strain Sesismic Characteristics for Sample 020144

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
2.63	Shear Wave Velocity	m/sec	194.5	198	216	239.5	255.5
		ft/sec	638	650	709	786	838
	Shear Modulus	MPa	112.5	116	135.1	168.5	193
		psi	16277.0	16783.4	19546.8	24379.3	27924.1
	Damping Ratio	%	1.90	1.90	1.85	1.75	1.65

Low–Strain Sesismic Characteristics for Sample 101718

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
2.0-2.1	Shear Wave Velocity	m/sec	144	146	150	151	150
		ft/sec	472	479	492	495	492
	Shear Modulus	MPa	53	55	58	58	59
		psi	7668.3	7957.6	8391.7	8391.7	8536.4
	Damping Ratio	%	6.50	6.50	6.50	6.50	6.50

Low–Strain Sesismic Characteristics for Sample 111926

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
1.25	Shear Wave Velocity	m/sec	103	108.5	113.5	121.9	125.6
		ft/sec	338	356	372	400	412
	Shear Modulus	MPa	24.03	26.5	29.9	35.5	37.5
		psi	3476.8	3834.1	4326.1	5136.3	5425.7
	Damping Ratio	%	2.52	2.33	2.21	2.15	2.20

Low-Strain Sesismic Characteristics for Sample CJA20

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	20 psi	40 psi	55 psi
[7.5]	Shear Wave Velocity	m/sec	155	221	230	237
		ft/sec	509	725	755	778
	Shear Modulus	MPa	65	133.8	146.7	155.2
		psi	9404.5	19358.7	21225.2	22455.0
	Damping Ratio	%	4.20	4.20	4.00	4.00

Low–Strain Sesismic Characteristics for Sample 101507

Void Ratio	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	55 psi
0.52	Shear Wave Velocity	m/sec	227.9	228.4	275.2	346.5	378
		ft/sec	748	749	903	1137	1240
	Shear Modulus	MPa	88	88.9	129.2	206	245.5
		psi	12732.2	12862.4	18693.2	29804.9	35520.0
	Damping Ratio	%	1.80	1.85	1.50	1.41	1.45
0.57	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	210	242	268	306	319
		ft/sec	689	794	879	1004	1047
	Shear Modulus	MPa	72	96	118	154	167
		psi	10417.3	13889.7	17072.7	22281.4	24162.3
	Damping Ratio	%	2.30	2.00	1.85	1.65	1.48
0.70	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	178	207	236	274	286
		ft/sec	584	679	774	899	938
	Shear Modulus	MPa	55.5	75.2	98.2	133.1	148
		psi	8030.0	10880.3	14208.0	19257.5	21413.3
	Damping Ratio	%	2.80	2.32	2.06	1.91	1.70
0.35	Confining Pressure		5 psi	10 psi	20 psi	40 psi	55 psi
	Shear Wave Velocity	m/sec	214	248	276	320	339
		ft/sec	702	814	906	1050	1112
	Shear Modulus	MPa	103.6	139.6	174.1	234	262
		psi	14989.3	20197.9	25189.5	33856.1	37907.3
	Damping Ratio	%	1.65	1.45	1.25	1.06	1.08
0.47	Confining Pressure		5 psi	10 psi	20 psi	40 psi	60 psi
	Shear Wave Velocity	m/sec	179.5	238.5	279	332.5	364.6
		ft/sec	589	782	915	1091	1196
	Shear Modulus	MPa	60	106.5	145.5	208	250.5
		psi	8681.1	15408.9	21051.6	30094.3	36243.4
	Damping Ratio	%	1.05	0.75	0.57	0.41	0.39

Low–Strain Sesismic Characteristics for Sample 101717

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
4.0	Shear Wave Velocity	m/sec	138.2	171.4	202.2	242.2	278.1
		ft/sec	453	562	663	795	912
	Shear Modulus	MPa	54.5	83.6	117	167.6	221.3
		psi	7885.3	12095.6	16928.1	24249.1	32018.6
	Damping Ratio	%	1.70	1.10	0.86	0.86	0.80

Low–Strain Sesismic Characteristics for Sample 111929

Unconfined compression strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
3-3.25	Shear Wave Velocity	m/sec	138.2	171.4	202.2	242.2	278.1
		ft/sec	453	562	663	795	912
	Shear Modulus	MPa	54.5	83.6	117	167.6	221.3
		psi	7885.3	12095.6	16928.1	24249.1	32018.6
	Damping Ratio	%	2.40	2.10	2.00	1.80	1.60

Low-Strain Sesismic Characteristics for Sample 010435

Unconfined Comp. Strength (ksf)	Confining Pressure	Unit	5 psi	10 psi	20 psi	40 psi	60 psi
2.5	Shear Wave Velocity	m/sec	188	192	200	206	215
		ft/sec	617	630	656	676	705
	Shear Modulus	MPa	105	111	120	129	140
		psi	15191.8	16059.9	17362.1	18664.3	20255.8
	Damping Ratio	%	3.03	2.87	2.75	2.78	2.75

Appendix B

High-Strain Seismic Characteristics for Soil Group A2

Sample No.	Void Ratio (e)	Confining Pressure (psi)	High-Strain Parameters		
			A	B	γ_o
101723	0.42	5	16.6775	0.2812	0.000001133
		20	1.6784	0.38312	0.00015224
		55	0.90028	0.4658	0.000502
	0.50	5	--	--	--
		20	1.5530	0.4087	0.0002318
		55	2.54113	0.3146	0.0001269
	0.49	5	7.6846	0.3529	0.000006944
		20	1.80135	0.3948	0.0001383
		60	1.32342	0.4163	0.0003999
121433	0.88	5	1.9193	0.4702	0.00008798
		20	5.6008	0.3160	0.000015555
		60	5.8575	0.2967	0.00001872
	0.44	5	--	--	--
		20	11.3035	0.2819	0.000001298
		60	--	--	--
	0.37	5	--	--	--
		20	4.8243	0.2771	0.00001019
		60	1.7690	0.3522	0.0001677
011442	0.49	5	1.1804	0.5048	0.0002413
		20	21.9567	0.2773	0.000001133
		60	--	--	--
	1.65	5	9.2973	0.3682	0.000007905
		20	--	--	--
		60	--	--	--
020145	0.49	5	2.8605	0.37315	0.0003729
		20	2.4817	0.3179	0.00005861
		55	--	--	--
	0.45	5	2.75884	0.4074	0.00006759
		20	6.0093	0.3156	0.000025092
		55	--	--	--
	0.58	5	0.84368	0.6052	0.0004971
		20	4.8014	0.3317	0.00004141
		60	--	--	--

High–Strain Seismic Characteristics for Soil Group A2 (Continued)

Sample No.	Void Ratio (e)	Confining Pressure (Psi)	High–Strain Parameters		
			A	B	γ_o
020145	0.52	5	1.3763	0.5281	0.00021025
		20	--	--	--
		60	3.6258	0.3142	0.00008249
021246	0.40	5	1.9455	0.47200	0.000096891
		20	6.0890	0.3145	0.00001194
		55	3.3350	0.32319	0.00006239
	0.32	5	2.1234	0.37815	0.00005254
		20	2.4694	0.3246	0.00005739
		60	1.90479	0.3578	0.0001735
	0.31	5	3.1961	0.3685	0.00002679
		20	6.7376	0.27986	0.0000055055
		55	5.91393	0.27729	0.00001128
021549	0.22	5	1.9961	0.3939	0.00004044
		20	0.87341	0.54975	0.0003541
		60	1.1855	0.44025	0.0003991
	0.53	5	1.7081	0.33357	0.00005759
		20	1.0556	0.4091	0.0003788
		60	1.7551	0.3587	0.0002756
	0.39	5	1.8479	0.4329	0.00008323
		20	1.9922	0.3633	0.00012182
		60	6.3116	0.2841	0.00001423
	0.41	5	1.8381	0.3999	0.000062682
		20	1.9831	0.3580	0.0001097
		60	2.7003	0.3109	0.000088113
021553	0.55	5	--	--	--
		20	16.8711	0.2872	0.000001746
		60	4.2377	0.2874	0.000033956
	0.68	5	2.7558	0.3693	0.00006364
		20	16.7027	0.3027	0.000002221
		60	--	--	--
	0.59	5	2.9986	0.3629	0.00004858
		10	4.1386	0.3297	0.000031226
		20	2.6758	0.3459	0.00011444
		40	1.3699	0.4144	0.0004449
		60	4.5792	0.2955	0.00004809

High-Strain Seismic Characteristics for Soil Group A3

Sample No.	Void Ratio (e)	Confining Pressure (psi)	High-Strain Parameters A	B	γ_o
101711	0.44	5	3.2079	0.33004	0.000061552
		20	1.6046	0.40663	0.00033789
		55	2.1469	0.4085	0.0004186
	0.51	5	1.6796	0.3954	0.0001604
		20	1.0658	0.4716	0.0004313
		55	3.2070	0.2883	0.0000549
	0.64	5	2.3590	0.3774	0.00009942
		20	2.07370	0.39921	0.0001356
		55	1.58182	0.37975	0.0001195
101712	0.40	5	4.7165	0.3842	0.00002711
		20	0.6268	0.7502	0.0009761
		60	2.2821	0.3241	0.0002979
	0.54	5	3.2683	0.3312	0.0000575
		20	1.7112	0.3904	0.0002975
		60	1.9338	0.3405	0.0004061
	0.59	5	1.7643	0.4339	0,0001960
		20	1.9274	0.3939	0.0002271
		60	1.7581	0.3734	0.0004810
010434	0.63	5	1.0459	0.5886	0.00004215
		20	4.5901	0.3429	0.00007173
		55	2.0894	0.3735	0.0005022
	0.57	5	1.1046	0.6076	0.0003469
		20	--	--	--
		60	--	--	--
	0.69	5	2.0523	0.5098	0.00004849
		20	--	--	--
		55	2.8803	0.2261	0.00007341
	0.72	5	0.7894	0.6252	0.0005720
		20	1.8445	0.3819	0.0002717
		60	2.0060	0.3353	0.0004210
	0.62	5	4.3700	0.372	0.00003610
		20	2.4097	0.3620	0.0001646
		60	2.0606	0.3333	0.0003716

High-Strain Seismic Characteristics for Soil Group A3 (Continued)

Sample No.	Void Ratio (e)	Confining Pressure (Psi)	High-Strain Parameters		
			A	B	γ_o
011439	0.74	5	0.9931	0.3655	0.00071671
		20	1.3497	0.4685	0.0005442
		60	2.1996	0.3170	0.0008841
	0.56	5	1.6523	0.42498	0.0001218
		20	2.03335	0.3448	0.0001633
		60	3.6693	0.2638	0.00006222

High-Strain Seismic Characteristics for Soil Group B1

Sample No.	Shear Strength (KSF)	Confining Pressure (psi)	High-Strain Parameters		
			A	B	γ_o
090602	0.36	50	6.3863	0.3752	0.00001433
		20	12.4979	0.28347	0.000004327
		55	--	--	--
	0.24	5	25.9574	0.3336	0.000001024
		20	1.2820	0.3651	0.0001716
		55	2.4620	0.3416	0.00009060
	0.17	5	3.5366	0.4591	0.00005702
		20	1.9099	0.4996	0.0001500
		55	1.3279	0.5139	0.0002468
	0.13	5	0.9792	0.4720	0.0005029
		20	10.10	0.3930	0.00001292
		55	0.6528	0.6069	0.0005172
101714	3.0	5	--	--	--
		20	0.9333	0.7181	0.00002914
		55	4.0195	0.5934	0.00001857
101715	2.75	5	12.1461	0.3461	0.000006052
		20	11.0928	0.2798	0.000004787
		55	1.7242	0.3794	0.0003620
101720	2.0	5	4.7641	0.3585	0.00003905
		20	2.1727	0.3761	0.0001474
		60	1.5201	0.4363	0.0003562
101722	4.5	5	2.60537	0.45868	0.00011572
		20	8.41337	0.3242	0.00001506
		60	2.0809	0.4045	0.0002623
111928	3.75	5	1.93495	0.50188	0.00009666
		20	1.5316	0.4517	0.0002612
		60	1.2830	0.5240	0.0003847
010837	1.0	5	5.8877	0.34327	0.00002743
		20	30.181	0.3529	0.000002354
		60	3.9450	0.3539	0.00009809

High-Strain Seismic Characteristics for Soil Group B1 (Continued)

Sample No.	Shear Strength (KSF)	Confining Pressure (psi)	High-Strain Parameters		
			A	B	γ_o
013143	3.75	5	--	--	--
		20	2.7146	0.40184	0.0001016
		60	1.2891	0.53183	0.0003714
020144	2.65	5	2.61777	0.4052	0.0001262
		20	--	--	--
		60	--	--	--

High-Strain Seismic Characteristics for Soil Group B2

Sample No.	Shear Strength (KSF)	Confining Pressure (psi)	High-Strain Parameters		
			A	B	γ_o
101718	2.1	5	--	--	--
		20	6.2907	0.3137	0.00002914
		60	8.6364	0.3186	0.00001857
111926	0.95	5	4.2123	0.47200	0.000096891
		20	2.5808	0.3045	0.00001194
		55	14.9537	0.32319	0.00001039
010736	2.0	5	0.6837	0.6636	0.001915
		20	2.7446	0.31033	0.0001879
		60	15.7908	0.2850	0.000005893
CJA20	4.25	5	2.4679	0.3988	0.00004420
		20	4.6938	0.3504	0.00005765
		55	--	--	--

High-Strain Seismic Characteristics for Soil Group C

Sample No.	Shear Strength (KSF)	Confining Pressure (psi)	High-Strain Parameters		
			A	B	γ_o
101507	0.52	5	1.1487	0.6111	0.0001751
		20	2.0766	0.4012	0.0001913
		55	--	--	--
	0.57	5	5.8373	0.3281	0.00001200
		20	7.4589	0.3174	0.00001044
		55	--	--	--
	0.70	5	3.8786	0.3226	0.00002722
		20	--	--	--
		55	--	--	--
	0.36	5	2.4986	0.3428	0.00006871
		20	1.9826	0.3848	0.0001631
		55	1.9942	0.3968	0.0001914
101717	3.0	5	1.9496	0.5528	0.0001145
		20	1.7777	0.4592	0.0002117
		60	2.1791	0.3728	0.0002157
111929	2.75	5	18.3306	0.3300	0.000002970
		20	2.7739	0.3794	0.00009444
		60	4.2178	0.4160	0.00006578
010435	2.5	5	5.8087	0.3702	0.00004716
		20	2.2056	0.3993	0.0002811
		55	--	--	--