APPENDIX 7.A
MODELING OF DYNAMIC SOIL PROPERTIES

7.A.1 Introduction

7.A.1.1 Scope

This appendix has the following purposes:

- It provides a reference document for other workers engaged in either generic or site-specific studies drawing on both previously published data and new data developed as part of this project;
- It provides the background necessary for interpreting the field and laboratory data for the three reference sites reported in Appendices 8.A and 8.B for use in the analyses reported in Appendix 6.A;
- It documents the development of the generic modulus reduction and damping curves used in Appendix 6.B;
- It extends a previously developed simple nonlinear soil model to include improved modeling of damping at both low and high strains.

7.A.1.2 Definition of Terms

Ideally the relationships between stress and strain that are used in site response analyses would form part of a single integrated model of soil behavior which would accommodate complex three-dimensional cyclic loadings as well as simple monotonic loadings to failure. However, while some progress has been made towards this goal, as a practical matter, simpler relationships which describe only the subset of behavior most applicable to a particular application are commonly used in practice. Thus, in analyses of site response which have generally assumed that motion can be satisfactorily represented by vertically propagating shear waves, there is only a need for appropriate relationships between shear stress and shear strain in horizontal layers. Further, motion is only considered to occur in a single vertical plane so that the possible effects of a second, orthogonal component of motion is neglected. If any consideration is given to vertical motions, these are also decoupled. Some comments on the effects of multi-directional shaking and the selection of properties for use in analysis of vertical motions are included at the end of this appendix but otherwise we consider only simple shear stress-shear strain relationships.

While nonlinear analyses may require only a single relationship that relates shear stress and shear strain under irregular cyclic loadings with appropriate generation of hysteretic damping, construction of such relationships or "models" is non-trivial and, as a practical matter, such models are usually based on the results of laboratory tests conducted using uniform, symmetrical loadings. Such tests also provide the basis for the description of the properties used in "equivalent linear" analyses (Seed and Idriss, 1969). The first two and a quarter cycles of such a test are depicted in Figure 7.A-1. In this and subsequent figures the normalized shear stress is shown on the y axis. This is the shear stress normalized by the applicable maximum shear stress, τ_{max}. The equivalent linear shear modulus is defined as the secant shear modulus after a given number of cycles with a constant value of the cyclic shear strain. In Figure 7.A-1, the modulus is shown after two cycles with a cyclic shear strain of 0.1 percent. The corresponding equivalent hysteretic damping ratio is given by the following expression:

$$D = \frac{1}{4\pi}\frac{\text{area within loop}}{\text{area of triangle}}$$

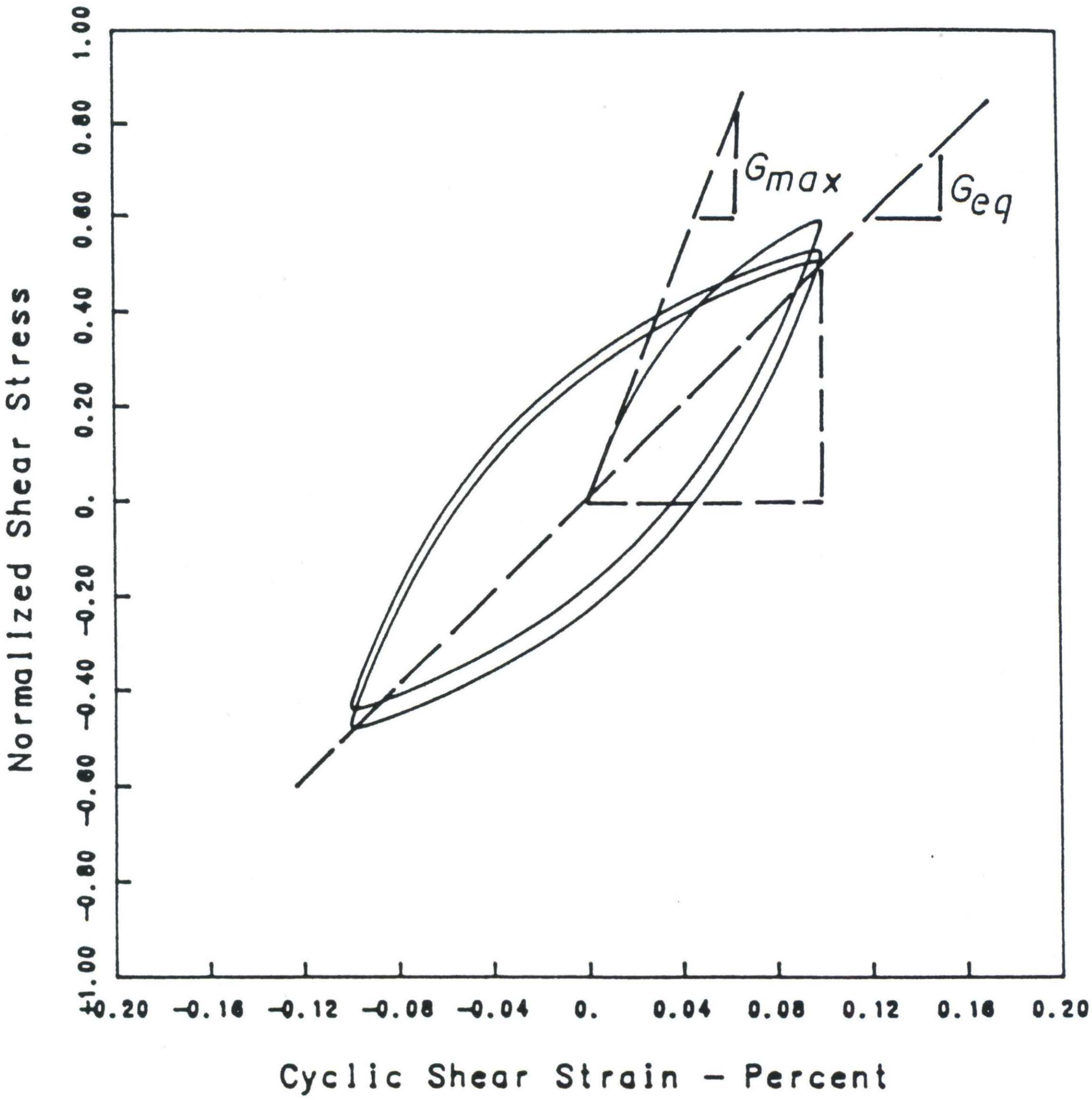

Figure 7.A-1
Typical Shear Stress-Shear Strain Relationship

When plotted at the same scale as shown in Figure 7.A-1, the stress-strain loops at very small strains collapse to a straight line with a slope equal to the maximum shear modulus, G_{max}, which closely approximates the initial tangent modulus on first loading or on reversal under cyclic loading. As the amplitude of the cyclic shear strain increases, the equivalent linear shear modulus decreases and the damping ratio increases. It is frequently convenient to normalize the shear modulus at a given strain level by dividing it by G_{max} and if the resulting quantity G / G_{max} and the damping ratio are plotted against cyclic shear strain, curves such as those shown in Figure 7.A-2 are obtained. Such curves will subsequently be referred to as modulus reduction and damping curves.

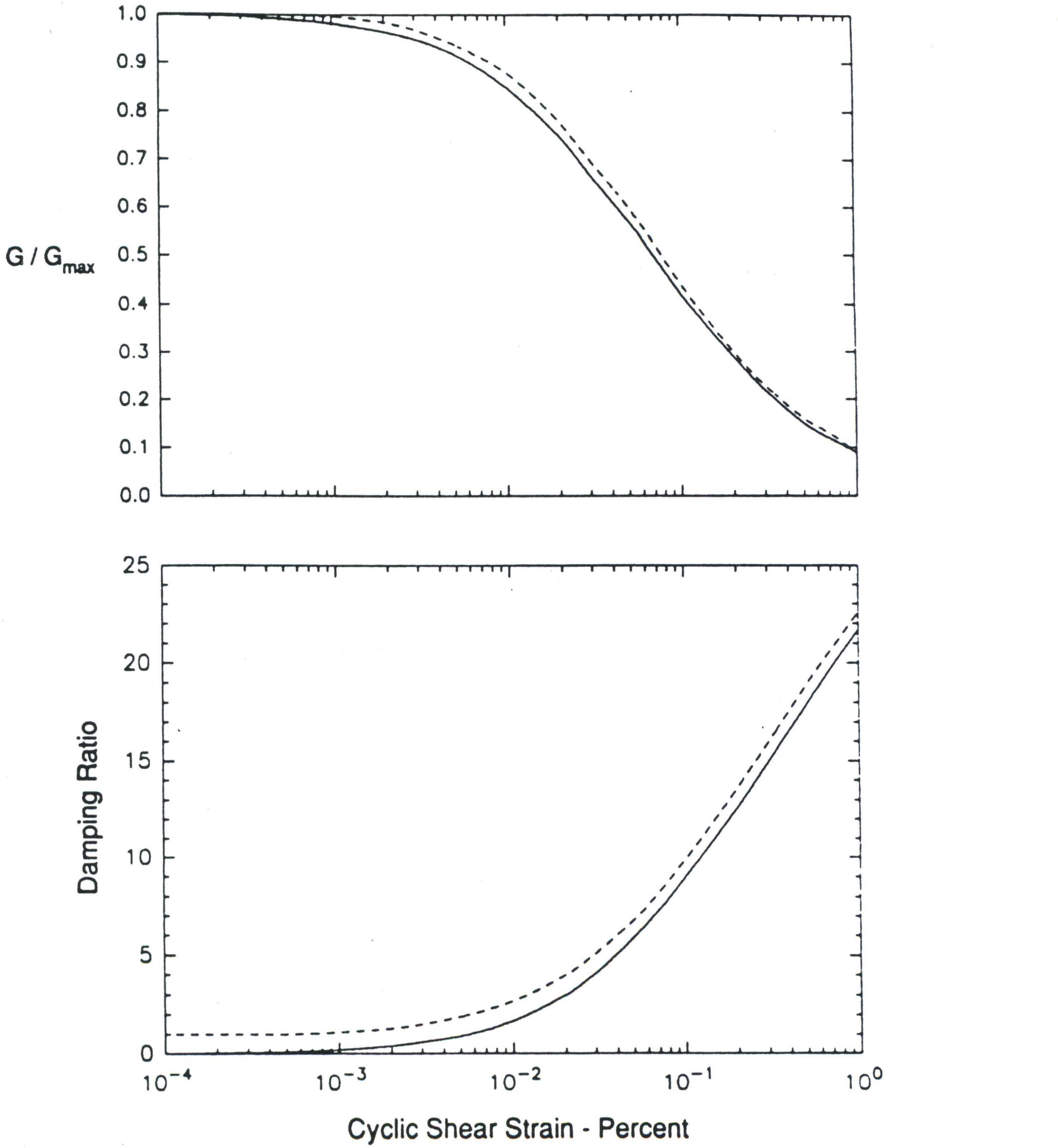

Figure 7.A-2
Typical Modulus Reduction and Damping Curves

Modulus reduction and damping curves of the type shown in Figure 7.A-2 are used directly in equivalent linear analyses in which the modulus reduction factor and the damping ratio are simply "looked up" as a function of the average cyclic shear strain over the duration of excitation for the layer in question. Since the average strains are not known at the outset of the analysis, initial values are assumed and the analysis iterates until "strain compatible" values are found. Conventionally the "average" strain has been taken to be 0.65 times the peak strain in the layer but the question of how to obtain the average strain has recently become more contentious as discussed in Appendix 6.B. In any case, the concept of equivalent linear analyses requires that fixed values of the modulus reduction factor and damping ratio be used throughout the analysis. Thus it is assumed that the modulus reduction factors and damping ratios are independent of the frequency and the number of cycles of loading although the values used are supposed to be obtained using representative loading rates and load histories. These are not unreasonable assumptions in the context of an analytical procedure in which other broader assumptions are made but the notion that the secant shear modulus and the damping ratio are actually independent of the frequency or rate of loading and the number of cycles of loading is not entirely correct.

The assumption that damping in soils is purely hysteretic in nature and is independent of the frequency of loading is not inconsistent with the results of many laboratory studies but gives rise to an interesting inconsistency in commonly used modulus reduction and damping curves. While equivalent linear analyses allow the use of independently derived and possibly inconsistent modulus reduction and damping curves, these curves cannot be completely independent of each other if damping is purely hysteretic,. In particular, at strains less than what is sometimes called the elastic threshold where the modulus reduction factor remains equal to one, soils, in some sense at least, exhibit linear behavior and the damping ratios at these strains might be expected to be zero, as shown by the full lines in Figure 7.A-2. Actual measurements, however, have always shown small but finite values of damping in this strain range, as shown by the broken lines in Figure 7.A-2. A possible explanation for this behavior and further discussion of the effects of the frequency and number of cycles of loading is offered later in this appendix.

While the equivalent linear or secant values of the shear modulus that are derived from an appropriate value of G_{max} and an appropriate modulus reduction curve are in some sense artificial values that make sense only in the context of equivalent linear analyses, it turns out that these values can also be used to define the "backbone" curve used to construct a nonlinear soil model. While, strictly speaking, the backbone curve should be an extension of the shear stress-shear strain relationship for the first quarter cycle of loading as shown in Figure 7.A-1, this curve should be obtained at the correct rate of loading and may be modified by subsequent cyclic loading. Thus, the backbone curve appropriate for use in a model of behavior under rapid cyclic loading and the locus of the end points of stress-strain loops such as those shown in Figure 7.A-1 are not necessarily the same as the shear stress-shear strain curve obtained for a monotonic loading to failure at any fixed rate of loading. It might be noted that in conventional soil mechanics, tests in which specimens are loaded to failure under monotonic loadings are usually called "static" tests, regardless of the rate of loading. Similarly, tests conducted to simulate earthquake loadings are usually referred to as "dynamic" tests although the fact that they are conducted using cyclic loading may be equally or more important than the rate of loading. Thus, the properties required for either equivalent linear or nonlinear analyses are usually referred to as "dynamic properties."

It should also be noted that the concept of equivalent linear analyses does not require the use of a shear strength and that it is possible to iterate to strain-compatible shear moduli and damping ratios which yield computed values of the shear stress which exceed conventional estimates of the shear strengths in some layers. This is not necessarily inconsistent, since conventional estimates of the "static" shear strength may err on the low side, in part because rate of loading effects are neglected, but also because it is normally conservative to err on the low side in estimating shear strengths. It is not, however, conservative to reduce the shear modulus in order to avoid exceeding a low estimate of the shear strength in an equivalent linear analysis of site response or to use a low estimate of the shear strength as the asymptote for the shear stress-shear strain relationship in a nonlinear analysis.

In summary, then, equivalent linear analyses require estimates of G_{max} and modulus reduction and damping curves for each layer used in the analysis and this is also the minimum information needed to construct a nonlinear model. Additional information may be required for nonlinear models which follow rate of loading and cyclic loading effects in greater detail. This information is usually rather specific to the particular nonlinear model that is used but two relatively simple examples are discussed in Section 7.A.5.

7.A.1.3 Measurement Techniques

For screening and preliminary site specific studies it may be adequate to estimate G_{max} or the shear wave velocity, V_s, and to use standard modulus reduction and damping curves of the kind that are presented in Section 7.A.6.

The shear modulus at small strains, G_{max}, and the shear wave velocity V_s, are related by the expression:

$$G_{\max} = \rho v_s^2$$

where ρ = the mass density, so that if either G_{max} or V_s are known or can be estimated, the other can be readily obtained.

Several empirical expressions are available for G_{max} as discussed in Section 7.A.5 and empirical correlations between V_s and various other soil properties are given, for instance by Sykora and Koester (1988) and Dickenson and Seed (1992). Such correlations should be used with great caution, however, since V_s can be affected very significantly by factors such as age and stress history which are not reflected in simple measures or indexes of soil properties.

For final site-specific studies it is imperative that both G_{max} and the modulus reduction and damping curves be measured for the actual soils at the site if uncertainties in these quantities are to be minimized. Three general approaches are available: in situ measurements, laboratory measurements, and backcalculation from observed ground motions.

7.A.1.3.1 In Situ Measurements. While there are obvious advantages to making all measurements of properties in situ, the only parameters that can reliably be measured in the field at present (1992) are the shear wave velocity, V_s, and the compression wave velocity, V_s. While very significant advances have been made in measuring low strain damping in situ in recent years, and in this project as reported in Appendix 8.A, standardized measurement and data reduction techniques have not yet emerged. Techniques to measure shear modulus and damping at larger strains in the field have been suggested from time to time but no viable techniques are yet available.

Presently available techniques for measuring velocities in situ include:

SASW—spectral analysis of surface waves—a good approach for use at shallow depths, particularly in soils in which it is difficult to drill and sample.

Crosshole—requires at least two, preferably three or four, closely spaced borings—likely provides the most accurate and repeatable measurements and allows measurement of shear wave anisotropy which may aid in interpretation of lateral stresses—requires accurate measurement of borehole diameter and drift and in practice usually limited to depths of several hundred feet—covered in part by ASTM Standard D4428.

Downhole—can be conducted in a single boring although measurement of borehole diameter and drift is still desirable—can provide very accurate measurements but more susceptible to measurement and interpretation errors than the crosshole technique—problems resulting from deviation of ray path from the direct source-to-receiver path could be minimized by using two or more downhole receivers—effective depth limited by use of surface sources but use of downhole sources could make technique viable to greater depths.

Seismic Cone—similar in principle to downhole—very fast but usually limited to depths of about 100 feet by capacity of thrusting system and rods.

Suspension Logger—desirably run in uncased borehole which limits demonstration of repeatability but results compare favorably with good crosshole and downhole measurements—samples small volumes of soil at high frequencies and shows more local variability than other techniques—can be used at depths up to 1000 feet.

The field investigation of reference sites reported in Appendix 8.A has indicated that generally similar results can be obtained using each of these techniques but that they all have some limitations. Thus it is desirable to use a minimum of two different measurement techniques on critical sites so that they can serve as a cross check on each other and on the occurrence of gross errors.

Most of the above techniques also allow the possibility of measuring low-strain damping in situ and the results obtained in this study and other recent work in this field should be evaluated in order to maximize the possibility of obtaining usable data on damping in conjunction with the measurement of velocities. It should be noted, however, that differing estimates of damping may be obtained from downhole and crosshole tests because of scattering, reflection and transmission effects, and that the effect of the frequency of excitation may need to be taken into account in comparing different field methods as well as in comparing field and laboratory measurements.

7.A.1.3.2 Laboratory Measurements. A variety of laboratory tests are available for measuring both G_{max} and the modulus reduction and damping curves at larger strains. In earlier work the strain ranges over which most test equipment could operate reliably was limited and different tests would be used to measure G_{max} and to obtain modulus reduction and damping curves. This has lead on some occasions to obvious inconsistencies but better quality work usually shows relatively good agreement between the results of different types of laboratory tests and newer equipment is generally capable of operating over wider strain ranges.

The most common laboratory tests are:

Resonant Column—Usually applied in torsion on solid cylinders but can be used for longitudinal excitation also—usually limited to small strains but can develop intermediate strain levels using softer or smaller diameter test specimens—covered in part by ASTM Standard D4015.

Cyclic Triaxial—Involves axial loading of solid cylinders—shear modulus and damping are not measured directly but are interpreted from Youngs modulus and damping—traditionally used at larger strains but can be extended to smaller strains with special equipment and instrumentation—covered in part by ASTM Standard D3999.

Cyclic Simple Shear—Loading conditions more closely approximate those in the field but it is more difficult to set up undisturbed samples—again used more at larger strains but can be extended to smaller strains with special equipment and instrumentation.

Cyclic Torsional Shear—Can be conducted using solid or hollow cylinders—hollow cylinders give greatest control over the stress state but make testing of undisturbed samples very difficult—perhaps the easiest configuration for testing over a very wide strain range.

Again, no single test is free of limitations and it is desirable to test the same soil in two or more kinds of apparatus as a cross-check. While it is traditional to present the results of these tests just as values of G_{max} and modulus reduction and damping curves, the stress-strain curves should also be recorded and presented even when the intent is to conduct only equivalent linear analyses since the stress-strain loops can provide important clues regarding the validity of the data. When nonlinear analyses are to be run it is even more critical to record and present the stress-strain loops and to track the development of any excess pore pressures.

7.A.1.3.3 Backcalculation from Observations of Ground Motion. An important third source of data on dynamic properties is provided by backcalculation from observations of ground motions at vertical arrays or close by soil/rock station pairs. This technique will not usually be practical as part of a specific site investigation unless the site is located in a highly seismic area, however, estimates of both the G_{max} profile and the modulus reduction and damping curves that can be obtained by backcalculation can provide important insights into the uncertainty associated with various field and laboratory measurement techniques if detailed investigations are performed at the same site. In non-seismic areas it is not possible to obtain data on the strain dependence of dynamic properties but observation of microtremors in vertical arrays can provide a cross-check on other measurements of the G_{max} or V_s profile.

7.A.1.4 Sources of Uncertainty

While it is very easy to make both random and systematic gross errors in both field and laboratory measurements, these can be minimized by: (1) using properly trained and experienced personnel; (2) following appropriate quality assurance procedures; and (3) always using two or more different procedures to obtain key measurements. However, there are additional sources of uncertainty which from a practical point of view often cannot be eliminated. It is of the greatest importance that the engineers involved in both the measurement of properties and their use in analyses understand the sources of these uncertainties so that they can work towards minimizing them and, where they cannot be reduced to the level at which they are no longer significant, accounting for them in conducting and interpreting the results of the analyses. These sources of uncertainty include:

7.A.1.4.1 Inherent Variability of Soil Deposits. When soil profiles are divided by geologists or engineers into layers that might be assigned short descriptions such as silty clay or gravelly sand, the soils within these layers are rarely homogeneous. This can be readily seen if the grain size curves or the Atterberg limits for a number of samples from one layer are plotted together.

Also changes of soil type can often be seen within the length of a sample that has been taken from within a broadly defined layer that might otherwise be assumed to be homogeneous. This is especially true if borings have been logged principally from cuttings. Ideally, small diameter continuous borings and/or cone soundings should be made prior to drilling larger diameter borings to obtain samples for more sophisticated laboratory tests, both to quantify the variability of the deposit and to optimize locations for sampling.

As a general rule, finer grained soils that have been deposited in a low-energy environment will be more uniform than coarser grained soils deposited in high-energy environments. Residual soils and other soils with variable cementation may also show high variability even though cuttings and samples appear to be uniform. Thus an understanding of the regional and site geology and depositional history is important to planning and executing studies of soil properties.

7.A.1.4.2 Inherent Anisotropy. Even soils which are apparently homogeneous usually exhibit at least some degree of inherent anisotropy as a result of the impact of their method of deposition and subsequent stress history on the soil fabric. Commonly, but not always, this leads to the strength and stiffness being lower for shearing on horizontal planes as opposed to vertical planes. Inherent anisotropy is one of the contributors to shear wave velocity varying with direction and to differences in behavior in different types of laboratory tests.

7.A.1.4.3 Anisotropic In Situ Stresses. Additional anisotropy of in situ soil properties can result from an anisotropic state of stress being superimposed on inherent anisotropy. Virtually all soils are subjected to an anisotropic state of stress in the field with the lateral effective stress being about half the vertical effective stress for normally consolidated soils but increasing with the degree of overconsolidation. Because the shear wave velocity is a function of the effective stresses only in the direction of wave propagation and the direction of particle motion, but not in the third direction (Roesler, 1979; Stokoe et al., 1985; Yan and Byrne, 1991), measured shear wave velocities may vary with the direction and polarity of the waves that are generated.

7.A.1.4.4 Drilling and Sampling Disturbance. While it is obvious that samples may suffer from disturbance as a result of poor drilling, sampling and sample handling practices, it is not so widely recognized that some disturbance is inevitable as a result of release of the in situ stresses. This problem becomes more severe with increasing depth and even if it is possible to establish the in situ state of stress and replicate it in the laboratory, it may not be possible to eliminate the effects of unloading and reloading the sample (see, for instance, Thomann and Hryciw, 1992). It is well established that the shear modulus tends to increase and the damping decrease with the time that samples are consolidated under sustained pressure in the laboratory and it has been suggested that extrapolation of laboratory results to the geologic age of the soil provides the best estimate of the in situ behavior (see, for instance, Anderson and Stokoe, 1978), but this procedure does not necessarily compensate correctly for the differences in the stress states in the field and in the laboratory and for the stress path followed in the sampling and reconsolidation process.

7.A.1.4.5 Limitations of Laboratory Equipment. No laboratory equipment is capable of fully representing the in situ stress state and deformation conditions including the three-dimensional nature of earthquake ground motion. Thus, all laboratory testing devices represent a compromise of one kind or another and it should not be surprising that there are some differences between results of cyclic loading tests performed in different kinds of apparatus (see, for instance, Pyke, 1978; Bhatia et al., 1985; Talesnick and Frydman; 1991).

7.A.1.5 Integration and Interpretation of Data

It should be apparent from the foregoing discussion that no data, be it acquired in the field or the laboratory, should automatically be used in analyses without some thought being given to the necessity for integrating it with data from other sources and making judgments as to the most appropriate values for use in analyses. Nonetheless, substantial progress has been made in the care with which both field and laboratory tests are conducted so that it is now generally possible to obtain reasonable agreement between the values of V_s or G_{max} measured in the laboratory and the field. Indeed, this is a basic check that should always be made to check that the samples as reconsolidated and tested in the laboratory are representative of field conditions. Further research and development work in this area is still required, however, as definitive guidance on the optimum procedures for reconsolidating samples is lacking, especially for overconsolidated soils. In this respect, it is worth noting that measurement of G_{max} in the field using both SH and SV waves and then checking G_{max} in the laboratory in both horizontal and vertical directions using "bender elements" (see, for instance, Dyvik and Madshus, 1985; Thomann and Hryciw, 1990), might provide a high level of confidence that field conditions have been satisfactorily re-established. Modulus reduction and damping curves obtained from such samples would then represent the best estimate that can be made short of the development of techniques for making measurements at large strains in the field, but these estimates would still need to be verified by cross-checking with forward or backcalculation of observed behavior at well instrumented field sites.

7.A.2 Sources of Literature

In common with many other fields of study, the most detailed information regarding both equipment and results is provided only in research reports and Ph.D. theses which might not be formally published. Even in these documents important details, particularly with regard to problems or ideas that might not have worked out, may not be included and it is invariably useful to contact the workers involved to discuss their published and on-going work. More formal publications include:

(1) Papers in the principal geotechnical engineering journals: *The Journal of Geotechnical Engineering* of ASCE, *The Geotechnical Testing Journal* of ASTM, *Géotechnique*, *The Canadian Geotechnical Journal*, and *Soils and Foundations.*

(2) The proceedings of two specialty conferences organized by the Geotechnical Engineering Division of the ASCE:

"Earthquake Engineering and Soil Dynamics," Pasadena, 1978.

"Earthquake Engineering and Soil Dynamics II, Recent Advances in Ground-Motion Evaluation," Park City, 1988.

(3) The proceedings of sessions sponsored by the Geotechnical Engineering Division at ASCE national conventions:

"Soil Sampling and Its Importance to Dynamic Laboratory Testing," Chicago, 1978.

"Measurement and Use of Shear Wave Velocity for Evaluating Dynamic Properties," Denver, 1985.

"Richart Commemorative Lectures," Detroit, 1985.

"Advances in the Art of Testing Soils Under Cyclic Conditions," Detroit, 1985.

"Recent Advances in Instrumentation, Data Acquisition and Testing in Soil Dynamics," Orlando, 1991.

(4) Special Technical Publications of the American Society for Testing and Materials:

"Dynamic Geotechnical Testing," STP No. 654, 1978.

"Soil Specimen Preparation for Laboratory Testing," STP 599, 1976.

"Laboratory Shear Strength of Soil," STP 740, 1981.

"Advanced Triaxial Testing of Soil and Rock," STP 977, 1988.

"Geophysical Applications for Geotechnical Investigations," STP 1101, 1990.

(5) The proceedings of two conferences organized by the University of Missouri, Rolla:

International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St Louis, 1981.

Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St Louis, 1991.

(6) The proceedings of two recent national and international conferences published by the Earthquake Engineering Research Institute:

Fourth U.S. National Conference on Earthquake Engineering, Palm Springs, California, May 20–24, 1990.

Fourth International Conference on Seismic Zonation, Stanford, California, August 25–29, 1991.

(7) The proceedings of other recent international conferences including:

International Conferences on Soil Mechanics and Foundation Engineering:

11th, San Francisco, August 1985.

12th, Rio de Janeiro, August 1989.

World Conference on Earthquake Engineering:

8th, San Francisco, June 1984.

9th, Tokyo, August 1988.

10th, Madrid, July 1992.

International Conferences on Soil Dynamics and Earthquake Engineering:

1st, Southhampton, U.K., August 1983.

2nd, Aboard Queen Elizabeth 2, June 1985.

3rd, Princeton, New Jersey, June 1987.

4th, Mexico City, October 1989.

(8) Other special publications, including:

"Soils Under Cyclic and Transient Loading," Proceedings of the International Symposium on Soils Under Cyclic and Transient Loading, Swansea, A. A. Balkema, 1980.

"Earthquake Geotechnical Engineering," Proceedings of Discussion Session on Influence of Local Conditions on Seismic Response. Twelfth International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Japanese Society of Soil Mechanics and Foundation Engineering, 1989.

7.A.3 Previous Summary Papers

The first significant work on dynamic properties was conducted in the 1960's and much of the early work is summarized in two major reviews:

H. B. Seed, and I. M. Idriss, "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. EERC 70-10, University of California, Berkeley, December 1970. (Also Section 5, "Soil Behavior Under Earthquake Loading Conditions," Report to U.S. Atomic Energy Commission, SW-AA, January 1972.)

Hardin, B. O., Drnevich, V. P., "Shear Modulus and Damping in Soils: Measurements and Parameters Effects," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 98, No. SM6, 1972, pp. 603–624.

Hardin, B. O., Drnevich, V. P., "Shear Modulus and Damping in Soils: Design Equations and Curves," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 98, No. GT7, 1972, pp. 667–692.

While the Hardin and Drnevich papers provided a more detailed treatment of the subject, giving expressions for the variation of shear modulus and damping with a number of parameters, the Seed and Idriss report, which gave typical ranges for modulus reduction and damping curves for sands and clays, was generally easier to follow and found very wide use in practice.

The Seed and Idriss report was subsequently extended to cover gravelly soils in the following paper:

Seed H. B., Wong, R. T., Idriss, I. M., Tokimatsu, K., "Moduli and Dynamic Factors for Dynamic Analyses of Cohesionless Soils," *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 11, 1986, pp. 1016–1032.

In both the Seed and Idriss report and the Hardin and Drnevich papers the modulus reduction curves for clays fell more sharply than the curves for sand, however, it turns out that this early conclusion was in error and that clayey soils are generally more elastic (that is, they show a lower rate of decrease of secant shear modulus with increasing shear strain) than sands. Revised modulus reduction and damping curves for clays were thus presented by Seed and his co-workers and by Dobry and Vucetic as follows:

Dobry, R, Vucetic, M., "Dynamic Properties and Seismic Response of Soft Clay Deposits," Proceedings of the International Symposium on Geotechnical Engineering of Soft Soils, Mexico City, 1987, Vol. 2, pp. 51–87.

Sun, J. I., Golesorkhi, R., Seed, H. B., "Dynamic Moduli and Damping Ratios for Cohesive Soils," Report No. EERC-88/15, University of California, Berkeley, August 1988.

Vucetic, M., Dobry, R., "Effects of Soil Plasticity on Cyclic Response," *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No.1, 1991, pp. 89–107.

While the preceding papers trace the evolution of practice in the United States of America, notable contributions have been made by a number of workers in Japan. Earlier work and significant new contributions were reported by Kokusho and his co-workers in the following papers covering sands, gravels and clays:

Kokusho, T., "Cyclic Triaxial Test of Dynamic Soil Properties for Wide Strain Range," *Soils and Foundations*, Vol. 20, No. 2, 1980, pp. 45–60.

Kokusho, T., Esashi, Y., "Cyclic Triaxial Test On Sands and Coarse Materials," Proceedings Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 1981, Vol. 1, pp. 673–676.

Kokusho, T., Yoshida, Y., Esashi, Y., "Dynamic Properties of Soft Clay For Wide Strain Range," *Soils and Foundations*, Vol. 22, No. 4, 1982, pp. 1–18.

Other reviews with more emphasis on the implementation of dynamic properties in analyses include:

Richart, F. E., "Some Effects of Dynamic Soil Properties on Soil-Structure Interaction," *Journal of Geotechnical Engineering Division*, ASCE, Vol. 101, No. GT12, 1975, pp. 1197–1240.

Wood, D. M., "Laboratory Investigations of the Behaviour of Soils under Cyclic Loading: A Review," *Soil Mechanics —Transient and Cyclic Loads*, Edited by G. N. Pande and O. C. Zienkiewicz, John Wiley & Sons, 1982.

Ishihara, K., "Evaluation of Soil Properties for Use in Earthquake Response Analysis," *Geomechanical Modelling in Engineering Practice*, Edited by R. Dungar and J. A. Studer, A. A. Balkema, 1986.

Review papers emphasizing in situ measurements include:

Woods, R. D., "Measurement of Dynamic Soil Properties," Earthquake Engineering and Soil Dynamics, Proceedings of the ASCE Geotechnical Engineering Specialty Conference, Pasadena, 1978, Vol. 1, pp. 91–178.

Stokoe, K. H., "Field Measurement of Dynamic Soil Properties," Proceedings of the Second ASCE Conference on Civil Engineering and Nuclear Power, Knoxville, Tennessee, Vol. 2, pp. 1–31.

Woods, R. D., "Field and Laboratory Determination of Soil Properties at Low and High Strains," Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, 1991, Vol. 2, pp. 1727–1741.

Review papers which emphasize integration of field and laboratory data and static and dynamic properties include:

Jamiolkowski, M., et al., "New Developments in Field and Laboratory Testing of Soils," Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Vol. 1, pp. 57–153, 1985.

Jamiolkowski, M., Leroueil, S., LoPresti, D. C. F., "Design Parameters from Theory to Practice," Theme Lecture, Geo-Coast '91 Conference, Port and Harbour Research Institute, Yokohama, 1991.

More recent Japanese work which emphasize comparisons of field and laboratory data are summarized in:

Kokusho, T., "In Situ Dynamic Soil Properties and Their Evaluations," Proceedings of the Eighth Asian Regional Conference on Soil Mechanics and Foundation Engineering, 1987, Vol. 2, pp. 215–240.

Kokusho, T., "Seismic Response of Soil Layer and Its Dynamic Properties," Preprint, Tenth World Conference on Earthquake Engineering, Madrid, 1992.

And, a recent summary co-sponsored by the Electric Power Research Institute is published as:

Dobry, R., et al., "Low and High Strain Cyclic Material Properties," Proceedings NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization, Report No. NP-7337, Electric Power Research Institute, Palo Alto, Vol. 1, June 1991.

7.A.4 Summary of Factors Affecting Modulus Reduction and Damping Curves

It is clear from the preceding review that the primary variation in modulus reduction and damping curves is with soil type, coarser, cohesionless soils showing a greater reduction in modulus with increasing shear strain than fine-grained, cohesive soils. This variation is summarized in Figure 7.A-3 which is based on the publications of Seed and Idriss and their co-workers but is also consistent with the findings of many other workers whose contributions are cited in Sections 7.A.3 and 7.A.7.

The published data on damping are more confusing, in part because of measurement errors in much of the earlier work, so rather than showing an equivalent figure for damping based on published data, companion damping curves are presented in Section 7.A.6, following the description of a simple model that generates consistent modulus reduction and damping curves. The general trend indicated by the published data for damping at intermediate to high strains are, however, compatible with the trend shown for the modulus reduction factors, namely, that coarser, cohesionless soils show greater values of damping with increasing shear strain than fine-grained, cohesive soils.

After soil type, the most significant factors affecting modulus reduction and damping curves are confining pressure for cohesionless soils and plasticity (as indicated by the plasticity index, or P.I.) for cohesive soils. Selected data on the variation of modulus reduction factors and damping ratios with confining pressure are shown in Figures 7.A-4 and 7.A-5. These data suggest that laboratory tests on freshly deposited clean sands (which are sometimes referred to as "baby" sands) show a greater sensitivity to confining pressure than natural soils but that there is still a systematic variation of modulus reduction factors and damping ratios with confining pressure for natural silty and clayey sands. Typical variations of the modulus reduction factors and damping ratios with P.I. for cohesive soils are shown by Sun et al. (1988) and Vucetic and Dobry (1991) and in Section 7.A.6. Since the shear modulus decreases more slowly with increasing shear strain for soils with a higher P.I., a confusion in terminology arises since more "plastic" soils in fact show more "elastic" stress-strain behavior.

Damping at low strains does not, however, follow quite the same pattern as modulus reduction factors and damping at higher strains. While low strain damping decreases with increasing confining pressure as shown in Figure 7.A-6, data published by Kokusho (1982) and some of the data developed in this study suggest that for fine-grained soils, while damping at high strains decreases with increasing P.I., low strain damping increases with increasing P.I. These variations are relatively minor, however, compared to what appears to be a major increase with either the grain size or the uniformity of the soil. This increase in low strain damping for coarser or less homogeneous soils is greater than might be inferred from their more nonlinear modulus reduction curves and smaller reference strains and comparisons of field and laboratory measurements of damping that are reported in Section 8 indicate that such soils may exhibit even greater damping in the field. Such differences may result in part from the fact that field measurements are typically made at somewhat higher frequencies than laboratory measurements, but may also result from back-scattering and other wave-propagation effects which cause the apparent damping in the field to be greater than the "intrinsic" damping which is measured using relatively homogeneous samples in the laboratory.

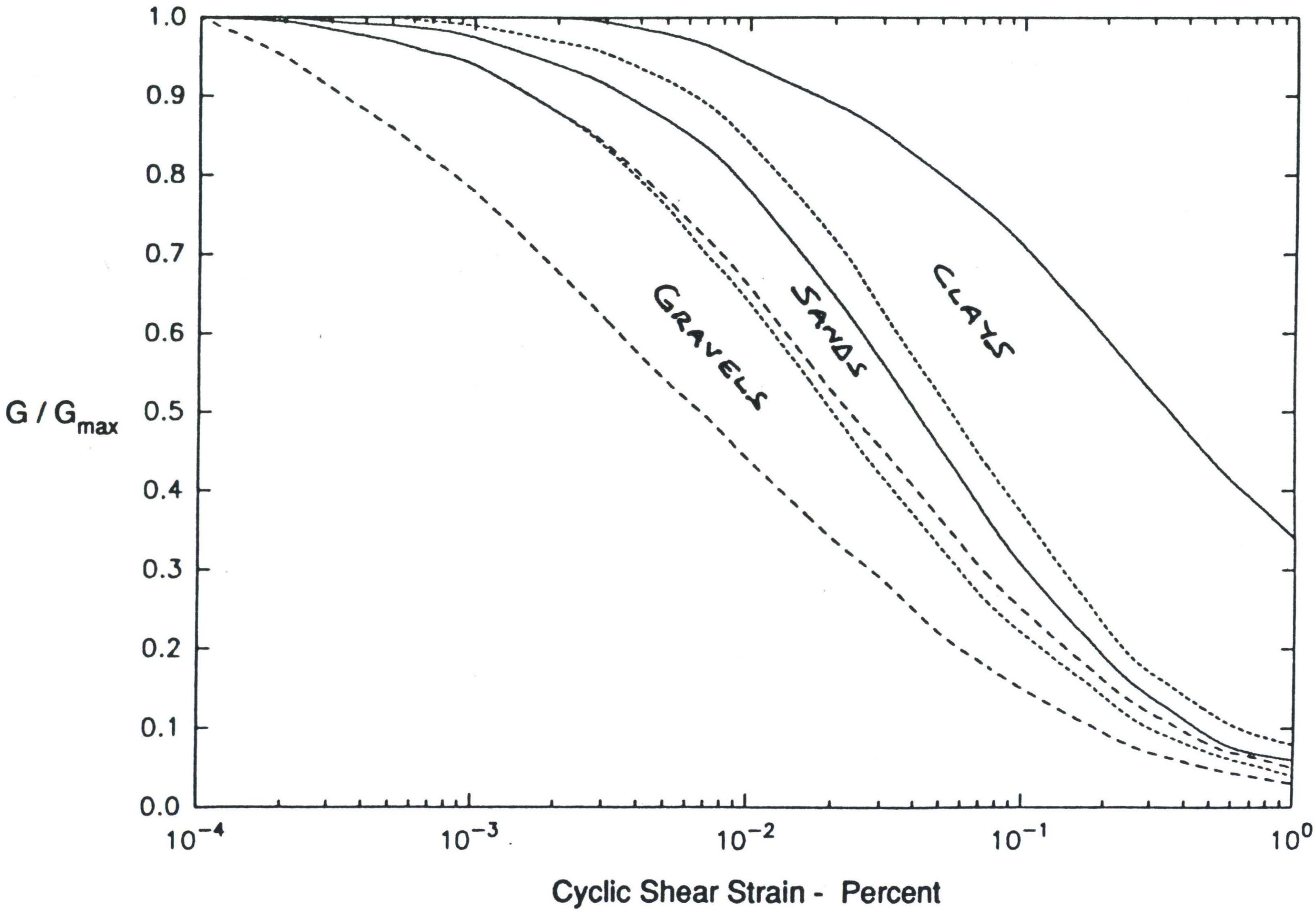

Figure 7.A-3
Typical Ranges for Modulus Reduction Curves

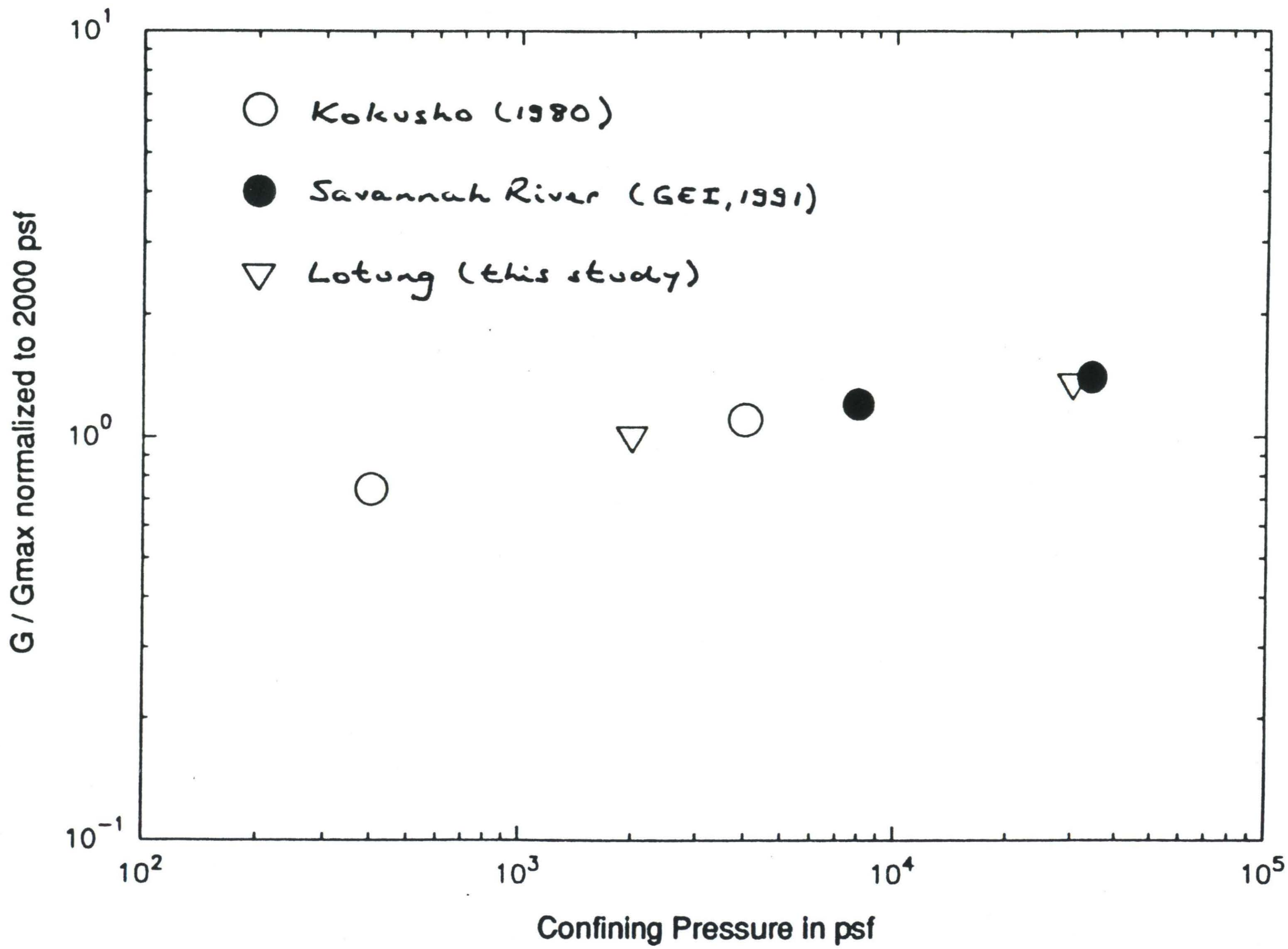

Figure 7.A-4
Variation of Modulus Reduction Factors with Confining Pressure

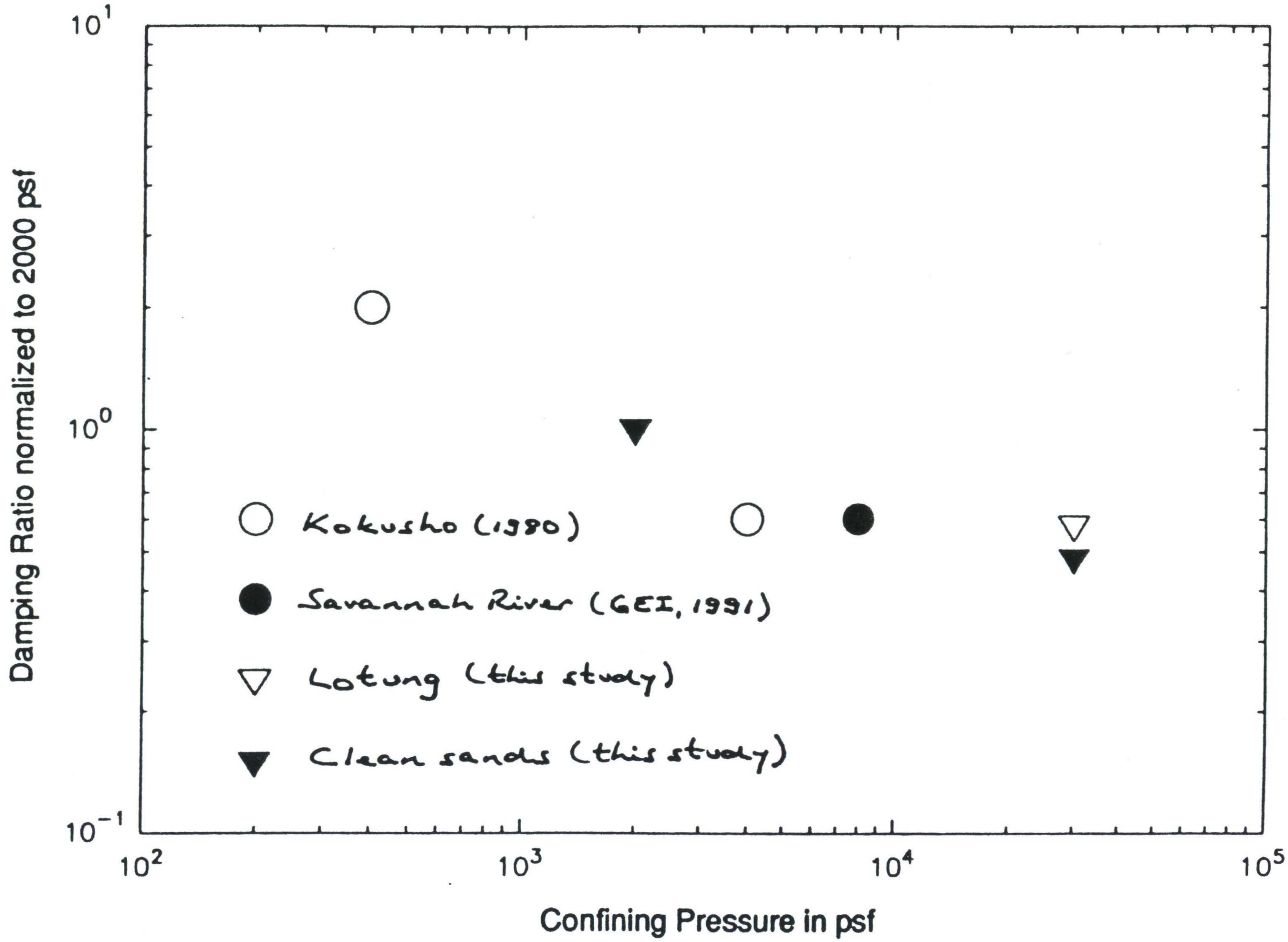

Figure 7.A-5
Variation of Damping Ratio with Confining Pressure

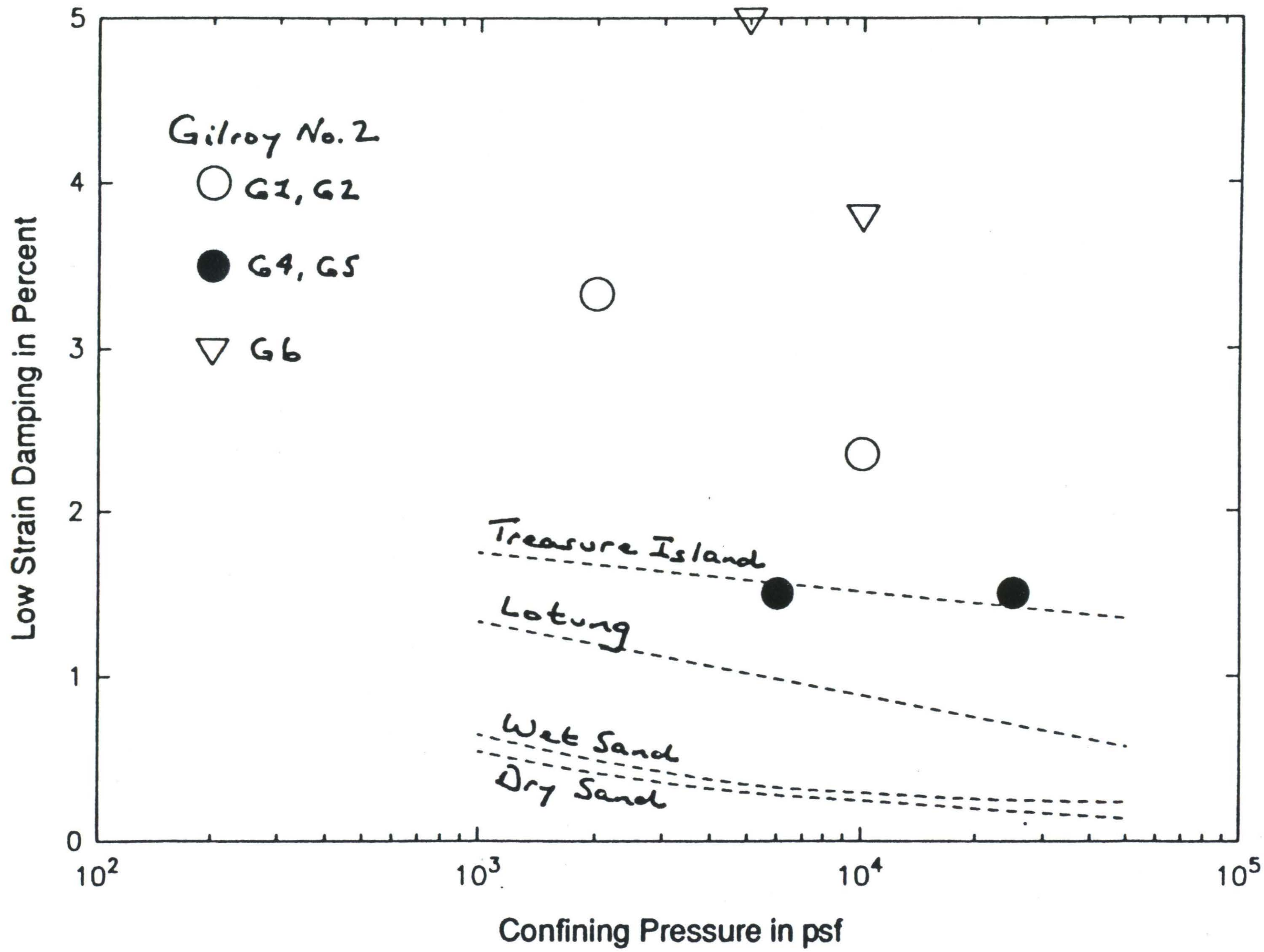

Figure 7.A-6
Variation of Low Strain Damping with Confining Pressure

In comparison with basic soil type and confining pressure or plasticity, all other factors appear to secondary in nature (see, for instance, Iwasaki and Tatsuoka, 1977; Tatsuoka et al., 1979; Yu and Richart, 1984; Alarcon et al.; 1989) although in some special situations these other factors may become important. For instance, the effect of the frequency of loading is generally not important over the range of frequencies used in laboratory tests but may become important when comparing the results of laboratory tests with field tests, as noted above. Rate of loading effects also appear to be significant in generating damping at low strains as discussed in the following section. As discussed in Section 7.A.6, the effect of overconsolidation on modulus reduction and damping curves may be obscured by other factors and deserves further study.

7.A.5 An Improved Nonlinear Soil Model

7.A.5.1 Introduction

A simple hyperbolic model of nonlinear shear stress-shear strain behavior is developed in this section in order to: (1) provide a basis for generating consistent modulus reduction and damping curves; (2) study a possible mechanism for including low strain damping in such models; and (3) study mechanisms for modeling the degradation and degeneration of the shape of stress-strain loops which may significantly affect the damping at higher strain levels. This model is a one-dimensional model in the sense that it only attempts to model the shear stress-shear strain relationship and does not attempt to model coupled shear and compression behavior under generalized three-dimensional loadings. Examples of more complete three-dimensional models are given by Prevost (1989) and Li et al. (1992).

7.A.5.2 Simple Hyperbolic Model

A simple model of soil behavior under simple shear loading conditions can be constructed by assuming that the shape of the shear stress-shear strain relationship can be represented by a hyperbola which has an initial slope equal to G_{max}, the shear modulus at small strains, and is asymptotic to τ_{max}, the "shear strength". The quantity

$$\gamma_r = \frac{\tau_{\max}}{G_{\max}} \qquad \text{7.A-1}$$

defined by Hardin and Drnevich (1972) as the reference strain turns out to be useful both in expressing the stress-strain relationship in mathematical form and as a measure of the relative values of G_{max} and τ_{max}. Soils with larger values of γ_r have greater "shear strengths" relative to their small strain modulus and show more elastic stress-strain behavior than soils with smaller values of γ_r. Thus gravelly soils have low values of γ_r and more plastic clays have high values.

The quantity τ_{max} is referred to in this discussion as the "shear strength", using quotation marks to indicate that while the asymptote of the stress-strain relationship should be related to the shear strength of the soil under the appropriate loading conditions, it is in fact very difficult to measure the shear strength under the right loading conditions and, further, the stress-strain relationship for many soils may deviate from a true hyperbolic shape at larger strains so that a better fit to observed stress-strain relationships at small to moderate strains can be obtained by treating τ_{max} as a fitting parameter, rather than deriving it independently as the shear strength.

The quantity G_{max} is best obtained by careful measurement of the shear wave velocity profile at the site in question but for parametric studies it may be useful to study the variation of G_{max} with confining pressure using either of two standard formula. One such formula due to Hardin (1978) is an update of the earlier expression given by Hardin and Black (1968). The revised expression is:

$$G_{\max} = Ap_a F(e) OCR^k \left(\frac{\sigma'_m}{p_a} \right)^n \qquad \text{7.A-2}$$

Where:

e	=	void ratio
p_a	=	atmospheric pressure
σ'_m	=	mean effective stress
n	=	an exponent which has a value in the order of 0.5
OCR	=	overconsolidation ratio
k	=	an exponent whose value varies from 0 if the P.I. is zero to 0.5 if the P.I. equals 100 or more and
F(e)	=	$(0.3 + 0.7e^2)^{-1}$

Hardin suggested a constant value of 625 for the parameter A but values ranging from 400 to 700 are found in practice.

An alternate expression can be obtained by putting the expression given by Seed and Idriss (1970) in non-dimensional form as follows:

$$G_{max} = K_G p_a \left(\frac{\sigma'_m}{p_a} \right)^{ng} OCR^{nocr} \qquad 7.A\text{-}3$$

Where:

K_G	=	a dimensionless parameter equivalent to the K_{2max} of Seed and Idriss
p_a	=	atmospheric pressure
σ'_m	=	mean effective stress
ng	=	an exponent
OCR	=	overconsolidation ratio
nocr	=	a further exponent similar to Hardin's k

Typical values of the parameter K_G are shown in Figure 7.A-7. The exponent, ng, is commonly taken to be equal to 0.5 for cohesionless soils but this value and the use of the mean effective stress are in fact simplifications. Careful measurements of shear wave velocities under controlled conditions in the laboratory suggest that the shear wave velocity is in fact a function of only the normal stress in the direction of wave propagation and the normal stress in the direction of particle motion. The shear modulus turns out to a function of the product of these two stresses both raised to something like the 0.25 power, so that use of the mean stress raised to the 0.5 power, where the mean stress is defined as $\sigma'_v(1 + k_o)/2$, is a good approximation for many conditions. For cohesionless soils the exponent ng increases at low confining pressures (below 1000 psf) and for cohesive soils the exponent approaches unity.

In older work the shear modulus of cohesive soils has often been reported in terms of the undrained shear strength, S_u, as G_{max}/S_u, but this introduces the significant difficulties involved in measuring S_u and it is preferable to treat data from all soils in a consistent fashion as a function of the effective stresses and consolidation history.

The effect of overconsolidation on the shear modulus at small strains is in fact unclear. Jamiolkowski (personal communication, 1992) is adamant that G_{max} is independent of mechanical overconsolidation if the parameter K_G is made a function of the void ratio as is done in the Hardin expression but contrary evidence can also be found. In any event, it is clear from a number of studies (for instance, Weiler, 1988) that G_{max} does not increase as rapidly with increasing OCR as does the undrained shear strength, so that ratio G_{max}/S_u decreases with increasing OCR and the reference strain, which is proportional to the inverse of G_{max}/S_u, must tend to increase with increasing OCR.

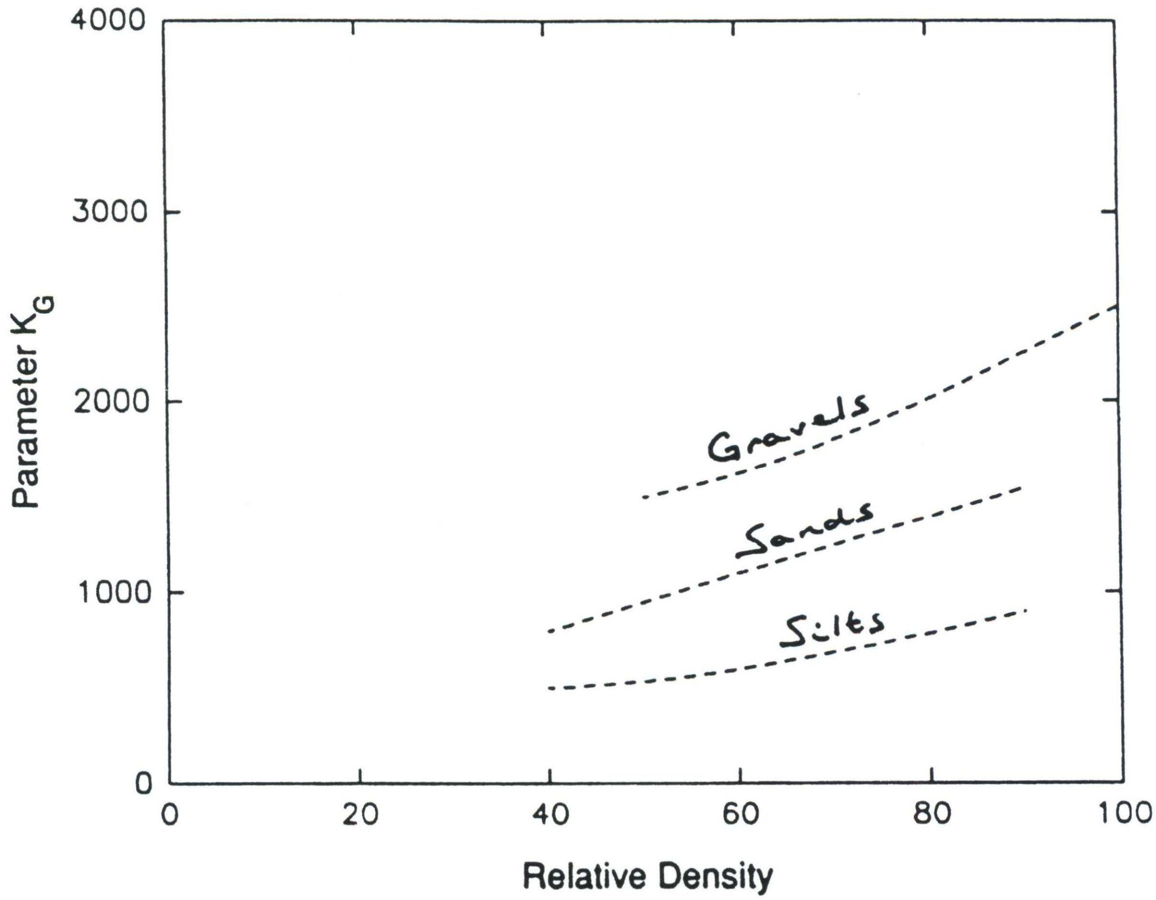

Figure 7.A-7
Variation of Parameter K_G with Soil Type

While the quantity τ_{max} may best be viewed as a fitting parameter, it is convenient for modeling purposes to use the conventional Mohr-Coulomb failure criterion to express τ_{max} as a function of the vertical effective stress:

$$\tau_{\max} = c + \sigma_v' \tan\phi \qquad \text{7.A-4}$$

Where:

c = cohesion
ϕ = angle of friction
σ_v' = vertical effective stress

The angle of friction tends to decrease with increasing confining pressure and it is sometimes convenient to calculate ϕ as:

$$\phi = \phi_1 - \Delta\phi \log\left(\frac{\sigma_v}{p_a}\right) \qquad \text{7.A-5}$$

Where:

ϕ_1 = angle of friction at one atmosphere of pressure
$\Delta\phi$ = reduction in angle of friction per log cycle of pressure

Given appropriate values of G_{max} and τ_{max} and hence the reference strain, γ_r, the relationship between shear stress and shear strain for initial loading can be written as:

$$\tau = \tau_{\max}\left(\frac{\frac{\gamma}{\gamma_r}}{1+\frac{\gamma}{\gamma_r}}\right) \qquad 7.A\text{-}6$$

and a simple model for cyclic loading can be obtained by applying the Cundall-Pyke hypothesis (Pyke,1979) in which the shear stress following a load reversal is given by:

$$\tau = \tau_c + \tau_{\max}\left(\frac{\frac{\gamma-\gamma_c}{\gamma_r}}{1+\frac{|\gamma-\gamma_c|}{c\gamma_r}}\right) \qquad 7.A\text{-}7$$

in which:

τ_c = shear stress at last reversal
γ_c = shear strain at last reversal
and:

$$c = \left|\pm 1 - \frac{\tau_c}{\tau_{\max}}\right| \qquad 7.A\text{-}8$$

where the first term is negative for unloading and positive for reloading. The factor c therefore has values which range from a minimum of 0 to a maximum of 2, as opposed to the alternate Masing hypothesis which uses a constant factor of 2. The stress-strain loops that are obtained using the Cundall-Pyke hypothesis are more realistic in several respects than those obtained using the Masing hypothesis and the resulting model allows the development of permanent strains which the Masing hypothesis does not.

It should be emphasized that the use of a hyperbolic shape is an approximation and a convenience. Other, more complex shapes could be adopted and cycled in accordance with the Cundall-Pyke hypothesis but a hyperbolic shape provides a good fit to much laboratory data on the variation of the secant, or equivalent linear, shear modulus with shear strain up to cyclic shear strains of 0.1 to 1 percent, beyond which the use of equivalent linear procedures is questionable and more elaborate models are require for nonlinear analyses. If the hyperbola is fitted to the relationship between secant shear modulus and cyclic shear strain after a given number of cycles of loading, this can be used both directly in equivalent linear analyses and as a basis for constructing a nonlinear soil model.

For use in equivalent linear analyses the basic hyperbolic relationship can be re-written as:

$$\frac{G}{G_{\max}} = \frac{1}{1+\frac{\gamma}{\gamma_r}} \qquad 7.A\text{-}9$$

which indicates that the reduction of secant shear modulus with increasing shear strain is solely a function of the reference strain. Thus, soils which show larger values for the reference strain, such as sands under higher confining pressures, more plastic clays and overconsolidated soils in general, show relatively less reduction of G_{max} with increasing cyclic shear strain.

7.A.5.3 Modeling of Low Strain Damping

While relationships between the variation of modulus with shear strain and the variation of damping with shear strain have commonly been developed and presented as if the modulus and damping are independent quantities, this cannot in fact be true if damping in soils is purely hysteretic, as is also commonly believed. As discussed in Section 7.A.1, if the damping in soils is purely hysteretic, then any model that describes the shear stress-shear strain relationship must also define the area of the stress-strain loops and hence the damping ratio. However, this creates a conflict with observed behavior in that even carefully calibrated laboratory measurements show finite values of damping at low strains where the secant modulus is constant. This observation has sometimes led to the suggestion that there may be small amounts of viscous damping in soils which are masked by hysteretic damping at larger strains, but that conclusion is not supported by measurements which show that small as well as large strain damping is largely independent of the frequency of loading. While Joyner et al. (1981) suggested a scheme for including frequency independent damping in nonlinear analyses, other nonlinear analysis schemes have usually included small amounts of viscous damping in an attempt to approximate the observed low strain damping at the frequency of greatest interest.

It turns out, however, that the low strain damping observed in the laboratory can be explained by including rate-of-loading effects in the stress-strain model in conjunction with recognizing that in most laboratory tests cyclic loads are applied using a sinusoidal loading function under which the rate of strain is markedly different at zero-crossings and at the ends of the loops. This point was originally made by Vucetic (1986) who suggested that it may explain why the ends of stress-strain loops are typically somewhat rounded. Thus, even at very low strains, the difference in the value of G_{max} at zero-crossings and at the ends of the loops may be sufficient to introduce hysteresis. Because the extreme points on the stress-strain loops are reached at a constant, infinitesimally slow, rate-of-strain, regardless of the frequency of the cyclic loading, there may not, however, be a significant change in the secant modulus.

Such a rate-of-strain effect can be introduced by defining a strain rate parameter, v_G, where, as suggested by Isenhower and Stokoe (1981), v_G equals the change in modulus per ten-fold increase in the strain rate normalized to the modulus at a shear strain rate of 0.01 percent per second. Then:

$$G^*_{\max} = G_{\max}\delta_G\delta^*_G\left(1 + v_G \log_{10}\frac{\dot{\gamma}}{0.0001}\right) \qquad \text{7.A-10}$$

where $\dot{\gamma}$ = shear strain (absolute) per second. A similar parameter, v_τ, can be defined to quantify rate-of-loading effects on τ_{max}.

Existing data on strain rate effects on shear modulus are limited, but data presented by Isenhower and Stokoe suggest that values of v_G in the order of 4 percent may be appropriate for San Francisco Bay Mud. Assuming for the moment that v_τ equals v_G, this value yields a low strain damping value of about 2 percent which is in the right order for Bay Mud. Because the low strain damping generated by the model invariably turns out to be equal to about half the value of the strain rate parameter over the range of interest, it is not in fact necessary to measure the rate of loading effect directly. If the appropriate value of low strain damping is known, the strain rate parameter can easily be set to obtain the desired damping. The stress-strain relationships and the modulus reduction and damping curves obtained using this approach are relatively insensitive to the frequency of loading and to the value assumed for v_τ. While further study of the strain-rate effect on both G_{max} and τ_{max} is required, the preliminary results obtained in this study appear to offer a rational explanation of both the observation of rounded ends on stress-strain loops obtained using sinusoidal loadings and finite values of damping at low strains. Further, because available data suggest that strain-rate effects increase with increasing plasticity, this approach offers an explanation of why low strain damping may increase with increasing P.I. while damping at higher strains decreases with increasing P.I.

7.A.5.4 Modeling of High Strain Damping

One of the reasons that a simple hyperbolic model may no longer be adequate at large cyclic shear strains is that for most natural soils the shape of shear stress-shear strain loops for saturated soils tends to both degrade (flatten) and degenerate (change shape) with increasing number of cycles of loading. It turns out, however, that a good fit to observed shear stress-shear strain loops can still be obtained using a simple hyperbola as the backbone curve if G_{max} and τ_{max} are multiplied by degradation and degeneration indices and updated values of G_{max} and τ_{max} are calculated as:

$$G^*_{\max} = G_{\max}\delta_G\delta^*_G \qquad \text{7.A-11}$$

$$\tau^*_{\max} = \tau_{\max}\delta_\tau\delta^*_\tau \qquad \text{7.A-12}$$

where δ_G is the degradation index as defined by Idriss et al. (1978), δ_G* is a multiplier that operates within each cycle of loading, and δ_τ and δ_τ* are similar multipliers operating on τ_{max}. These degradation and degeneration indices must somehow be made a function of the amplitude and number of cycles of loading and can, if desired, be made a function of the sustained and transient excess pore pressures. Examples of possible functions for modeling the degradation and degeneration of stress-strain loops for clays using a total stress approach and for sands using an effective stress approach are given below.

7.A.5.5 Total Stress Modeling of Degradation and Degeneration

The degradation index, δ, as used by Idriss et al. (1978) is a measure of the degradation of the secant shear modulus with the number of cycles of loading at a given cyclic shear strain.

$$\delta = \frac{G_n}{G_1} \qquad \text{7.A-13}$$

Where:

G_1 = secant shear modulus on first cycle
G_n = secant shear modulus on the n^{th} cycle

For irregular cyclic loadings the degradation index after each half cycle of loading is given by:

$$d_n = d_{n-1}\left\{1+0.5(d_{n-1})^{1/t}\right\}^{-t} \qquad \text{7.A-14}$$

Where:

δ = the present value of δ
δ_{n-1} = the previous value of δ
t = a parameter determined experimentally which describes the variation of δ with the number of cycles of loading at a given cyclic shear strain. Typical values of the parameter, t, as a function of P.I. are given by Vucetic (1992).

While the degradation index, δ, was defined by Idriss et al. to be a measure of the degradation of the secant modulus, it can be applied to both G_{max}, the shear modulus at small strains, and τ_{max}, the shear strength in order to degrade the stress-strain relationship at an appropriate rate. Ideally, however, the degradation of G_{max}, and τ_{max}, might be monitored separately in laboratory tests and different degradation indices would be applied to each of these quantities. Moreover, laboratory data sometimes indicate that the tangent shear modulus at the end of each stress-strain loop degrades little, if at all, but the shape of the hysteresis loops degenerates and the tangent modulus within each loop drops to a value rather lower than the secant modulus measured through the ends of the loops. For these materials, more realistic stress-strain loops are obtained if G_{max}, and τ_{max}, are reduced as a function of a modified degradation index, δ^*, which is dependent on the strain level:

$$\delta^* = 1-(1-\delta)\left[1-\left(\frac{|\gamma-\gamma_0|}{g\gamma_r}\right)^e\right] \qquad 7.A\text{-}15$$

Where:

γ_0 = the average of the shear strains at the last two zero-crossings of the shear stress
e = a fitting parameter which controls the rate at which degradation is activated within each cycle
g = a fitting parameter which determines the shear strain at which degradation is no longer effective

Typical shear stress-shear strain loops generated using such a model are shown in Figure 7.A-8 for four different values of P.I., selecting values of G_{max} and τ_{max} to give appropriate values of γ_r. Note that the example for a P.I. of 10 shows some degeneration of the shape of the loops which will impact the damping at higher strains and approaches the behavior of a cohesionless soil.

7.A.5.6 Effective Stress Modeling of Degradation and Degeneration

A simple scheme suggested by Seed, Martin and Lysmer (1976) to compute the excess pore pressures that may develop in saturated cohesionless materials when they are subjected to cyclic loading under undrained conditions may be used to model degradation and degeneration of shear stress-shear strain loops for cohesionless soils. Using this scheme, the excess pore pressures are computed at the end of each half-cycle and, if the low strain shear modulus and/or the shear strength have been specified to be a function of the mean effective stress, they will be recomputed at the end of each half-cycle.

The Seed et al. procedure for computing excess pore pressures relies on the fact that the curve of normalized excess pore pressure, u/σ_m' versus the normalized number of cycles of loading, N/N_L, where N_L is the number of cycles causing initial liquefaction or failure, has a characteristic shape as shown in the Seed et al. paper.

The number of cycles, N, or the number of cycles to initial liquefaction or failure, N_L, is always the number of uniform cycles of loading in laboratory tests and N_L is not constant but varies with the amplitude of cyclic shear stress that is applied. There are in fact any number of pairs of values N_L and τ_{av}, the uniform or average cyclic shear stress, which define the loading conditions causing initial liquefaction or failure. Both N_L and τ_{av} must therefore be defined for each element or layer. Normally N_L will be set to one value and τ_{av} will be computed for each element as a function of the initial mean effective stress.

Because the actual history of shear stress in any element is irregular, the equivalent number of uniform cycles with an amplitude of τ_{av} must be computed in order to use the curve of excess pore pressure. This is done by saving the peak shear stress in each half cycle of the shear stress defined by successive zero-crossings and using the weighing procedure described by Seed et al. (1975). This weighting procedure requires use of a curve based on the shape of the τ_{av} versus N_L line which defines initial liquefaction or failure, adjusted so that a cycle of stress with an amplitude equal to τ_{av} is given a weight of unity. Cycles with larger or smaller amplitudes are then equivalent to more or less than one cycle of the uniform or average cyclic shear stress.

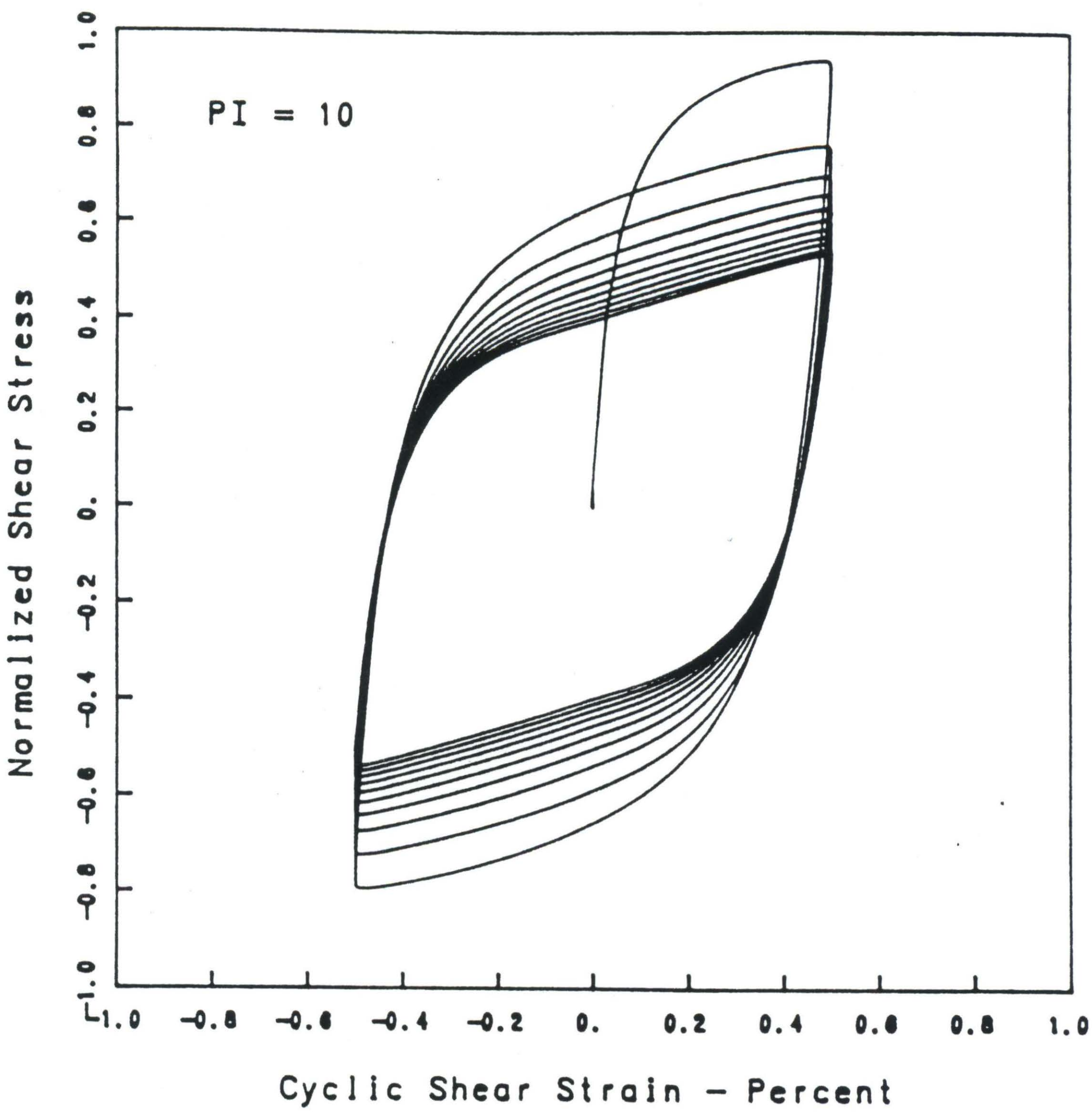

Figure 7.A-8a
Typical Stress-Strain Loops for Clay

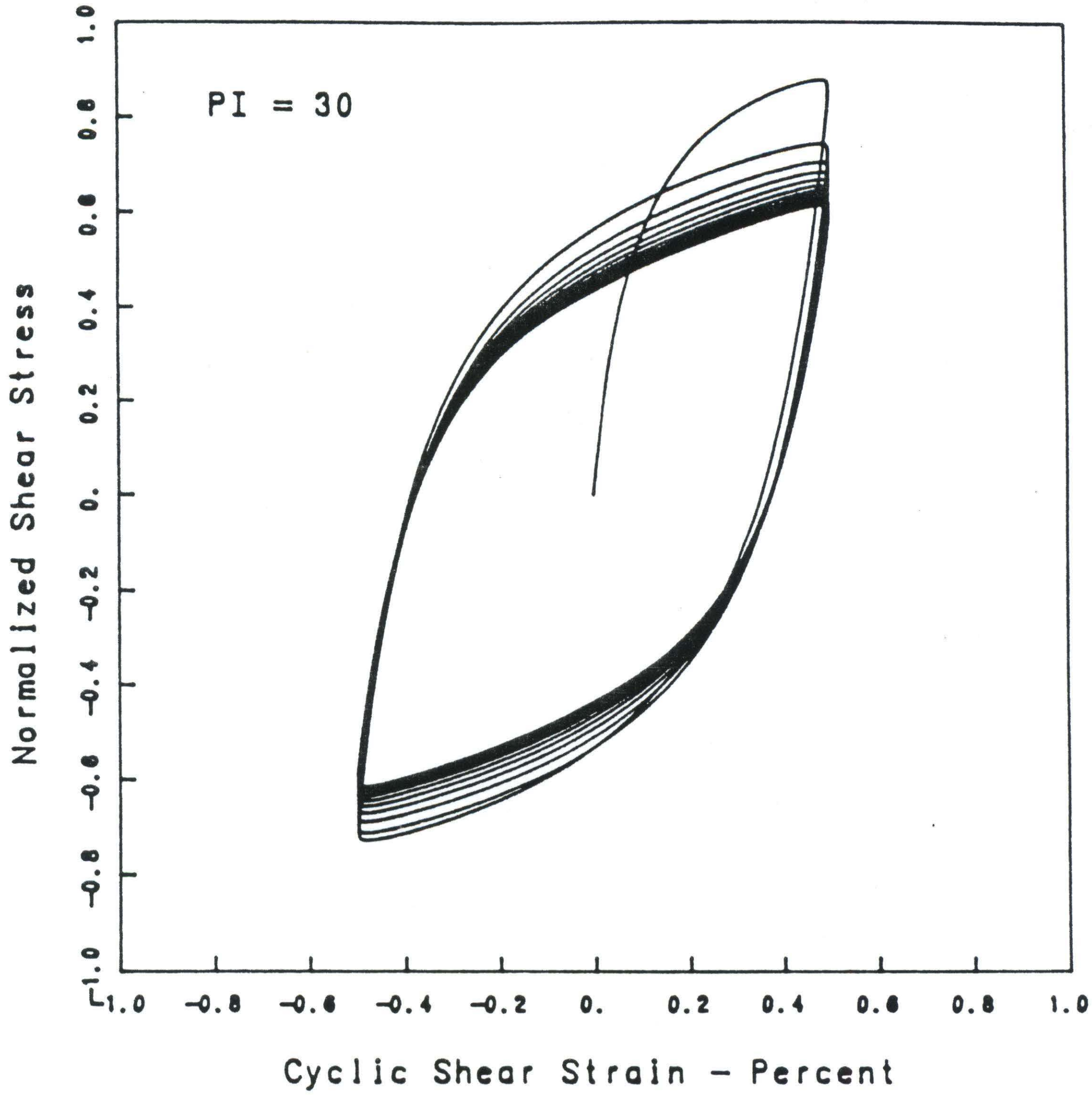

Figure 7.A-8b
Typical Stress-Strain Loops for Clay

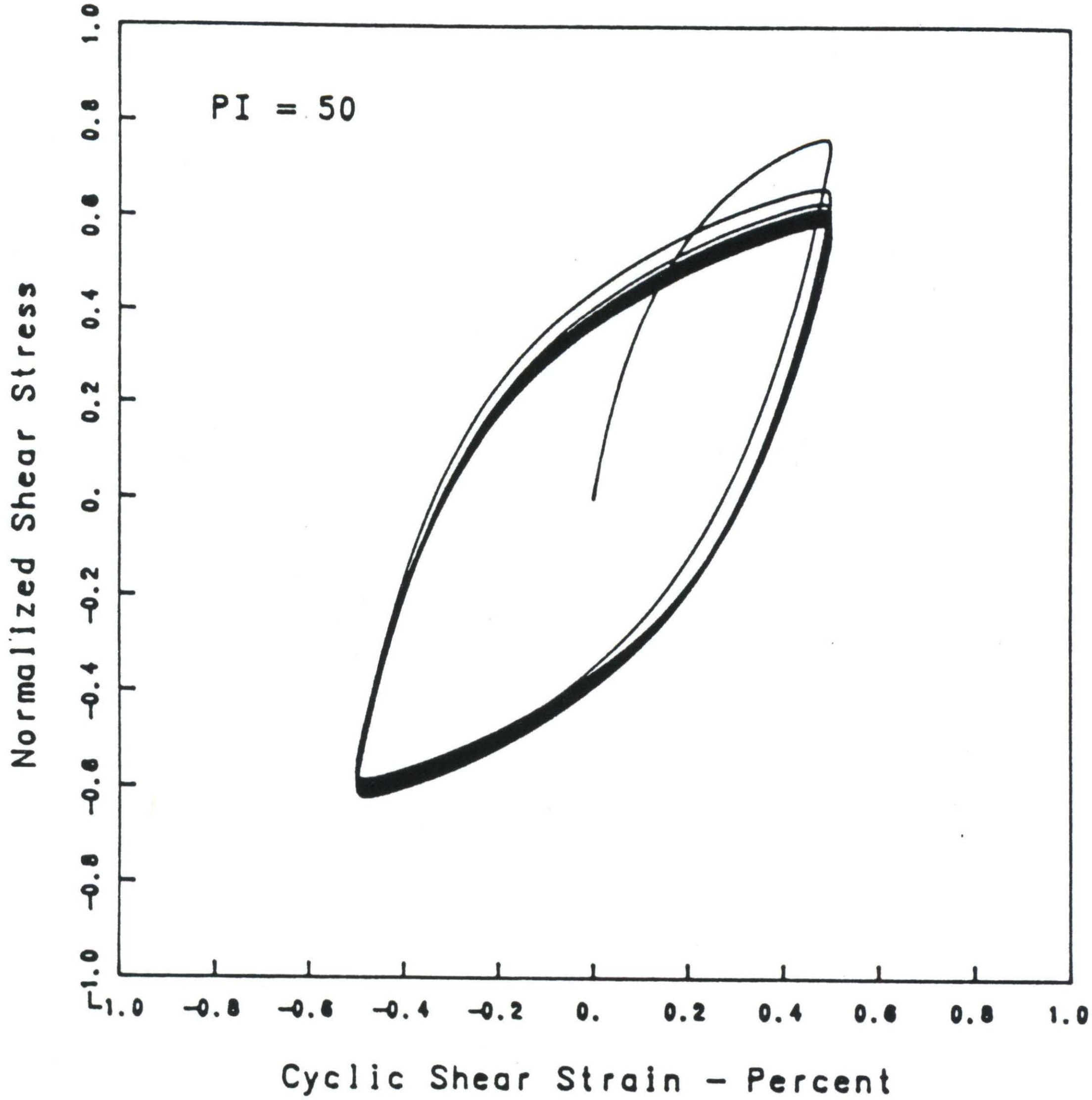

Figure 7.A-8c
Typical Stress-Strain Loops for Clay

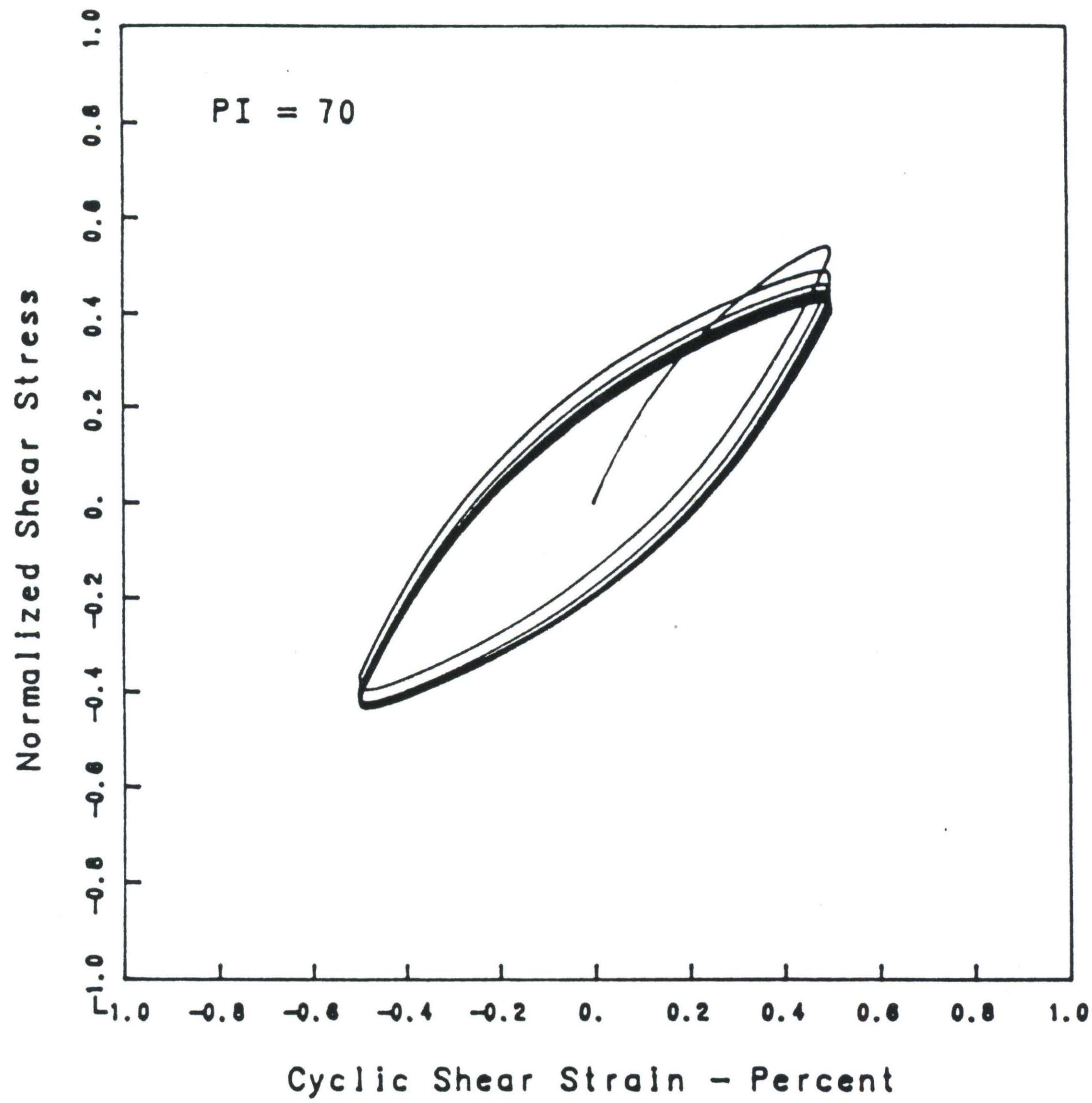

Figure 7.A-8d
Typical Stress-Strain Loops for Clay

As noted previously, when the use of pore pressure generation curves is specified in conjunction with stress-dependent values for G_{max} and τ_{max}, the values of G_{max} and τ_{max} will be recomputed each half-cycle as a function of the current value of the excess pore pressure. Note that half-cycles are defined by crossings of the zero shear strength axis and that the excess pore pressures computed by the Seed et al, procedure are the peak or sustained excess pore pressures. Additionally, there will be transient fluctuations of the excess pore pressures within each cycle which increase as the relative density increases and the confining pressure decreases. These transient decreases in excess pore pressures within each half-cycle are what cause the characteristic degenerated shape of the stress-strain relationship obtained for silts and sands subjected to undrained cyclic loading.

The transient fluctuations of the excess pore pressures can be modelled using three additional parameters, e, f, and g, where the e and g parameters are the same as that used in the simple degradation model—that is, they control the sharpness and width of the transient decreases in pore pressure within each half-cycle—and f is the ratio of the transient excess pore pressure to the sustained excess pore pressure.

The shear stress-shear strain loops obtained using such a model for a typical saturated sand at three different confining pressures subjected to a cyclic shear strain of 0.5 percent are shown in Figure 7.A-9.

7.A.6 Standard Modulus Reduction and Damping Curves

The simple model developed in the previous section can now be used to develop modulus reduction and damping curves that are compatible with each other and consistent with the better published data.

Figures 7.A-10 and 7.A-11 show modulus reduction and damping curves for a wide range of soils in terms of the reference strain, γ_r. While the reference strain is not a very familiar index of soil behavior, it turns out to be a very appropriate index for this purpose and it is not difficult to gain a sense of the relationship between reference strain and soil type. Gravelly soils have reference strains as low as 0.01 percent and highly plastic clays have reference strains approaching 1.0 percent. Sandy soils at moderate confining pressures typically have reference strains in the order of 0.1 percent and it turns out that the modulus reduction and damping curves for 0.1 percent that are shown in Figures 7.A-10 and 7.A-11 are similar to the upper bound of the modulus reduction curves and the lower bound of the damping curves given by Seed and Idriss (1970), which have been found to be generally representative of many silty and clayey sands (I.M.Idriss, personal communication, 1992). It might be noted that the families of curves shown in Figures 7.A-10 and 7.A-11 generally show more linear behavior than was the case in the earliest published studies and that this observation is especially applicable to clayey soils.

The sensitivity of the modulus reduction and damping curves for dry or moist sands to the initial confining pressure is shown in Figures 7.A-12 and 7.A-13. Freshly deposited clean sands may show greater variation with confining pressure than that shown, and silty or clayey sands may show less variation.

Corresponding curves for saturated sands are shown in Figures 7.A-14 and 7.A-15. While there are only minor differences in the modulus reduction factors, note that the damping at high strains is markedly reduced for saturated sands, consistent with the experimental data of Matasovic and Vucetic (1992).

Modulus reduction and damping curves for clays are shown in Figures 7.A-16 and 7.A-17 as a function of the plasticity index. These curves are generally consistent with the curves of both Seed et al. (1988) and Vucetic and Dobry (1991).

Finally, modulus reduction and damping curves suitable for generic site response studies in Eastern North America are shown in Figures 7.A-18 and 7.A-19. It is intended that these curves represent soils in the general range of gravelly sands to low plasticity silty or sandy clays. They should not be applied to either very gravelly or very clayey deposits. As is the case with the curves presented in Figures 7.A-10 and 7.A-11, they do not account for the effects of degradation and degeneration of stress-strain loops, so that for materials which develop any excess pore pressures under the levels of loading in question, the damping ratios at larger strains likely err in the conservative direction.

While generic curves of the kind given in Figures 7.A-10 through 7.A-19 are suitable for preliminary studies, soil-specific modulus reduction and damping curves and their variation with depth should be obtained for final site-specific studies of site response. In this connection there are two potentially important factors which remain unresolved: (1) the effect of overconsolidation; and (2) the effect of sample disturbance on measured modulus reduction and damping curves. Vucetic and Dobry (1991), for instance, report that overconsolidation is not a significant factor but it is possible that the effects of overconsolidation are masked by: (1) sample disturbance in testing natural soils, and (2) the fact that many laboratory tests reported as being overconsolidated have been conducted using isotropic overconsolidation rather than overconsolidation with no lateral strain, such as occurs in the field. Since the ratio G_{max}/S_u for clayey soils decreases with increasing OCR, the reference strain, which is proportional to the inverse of G_{max}/S_u, must increase, and therefore one would expect that overconsolidated soils might show increasingly linear behavior with increasing OCR. It is generally believed that the effect of sample disturbance is to move the modulus reduction and damping curves in the opposite direction, that is, to make them more nonlinear (for example, Anderson and Stokoe, 1978), so that OCR and sample disturbance would act in opposite directions. However, it can also be argued that since sample disturbance might be expected to reduce G_{max} more than τ_{max}, that disturbance should increase the reference strain and lead to more elastic behavior (Larkin and Taylor, 1982). It is not possible to resolve these issues with currently available data but this might be done in a carefully planned laboratory test program in which the effects of overconsolidation, stress release and mechanical disturbance are simulated under controlled conditions.

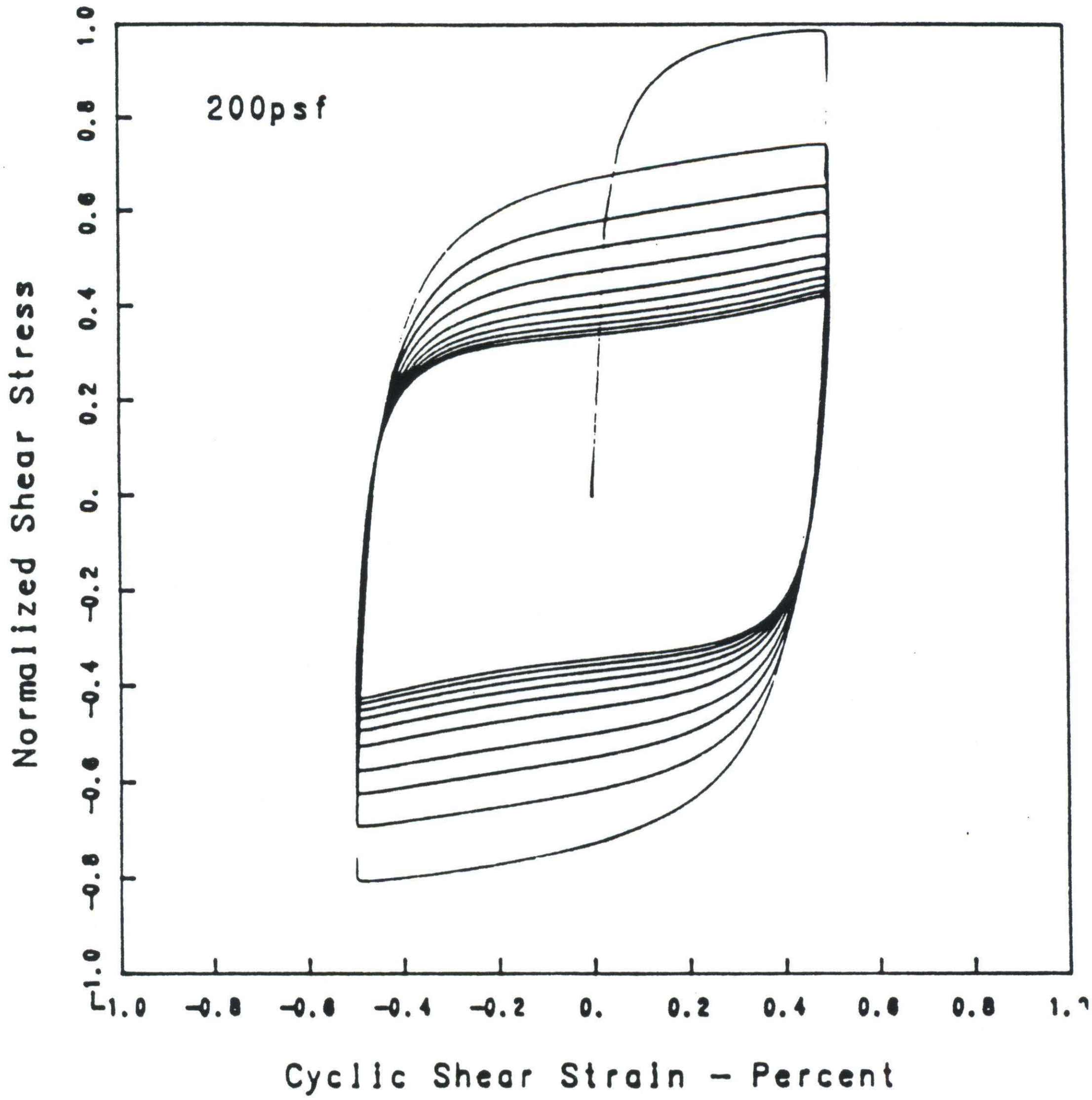

Figure 7.A-9a
Typical Stress-Strain Loops for Sand

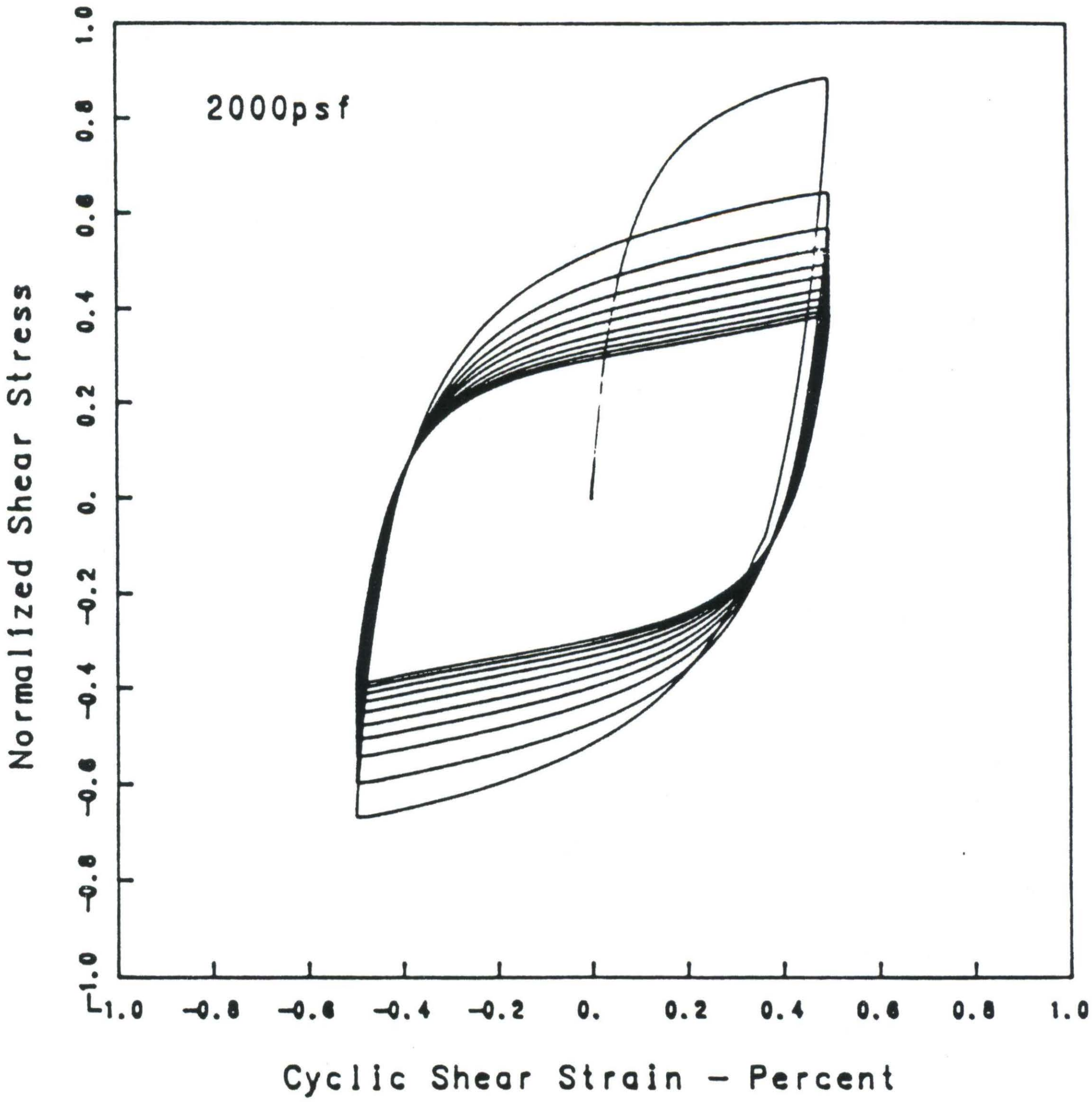

Figure 7.A-9b
Typical Stress-Strain Loops for Sand

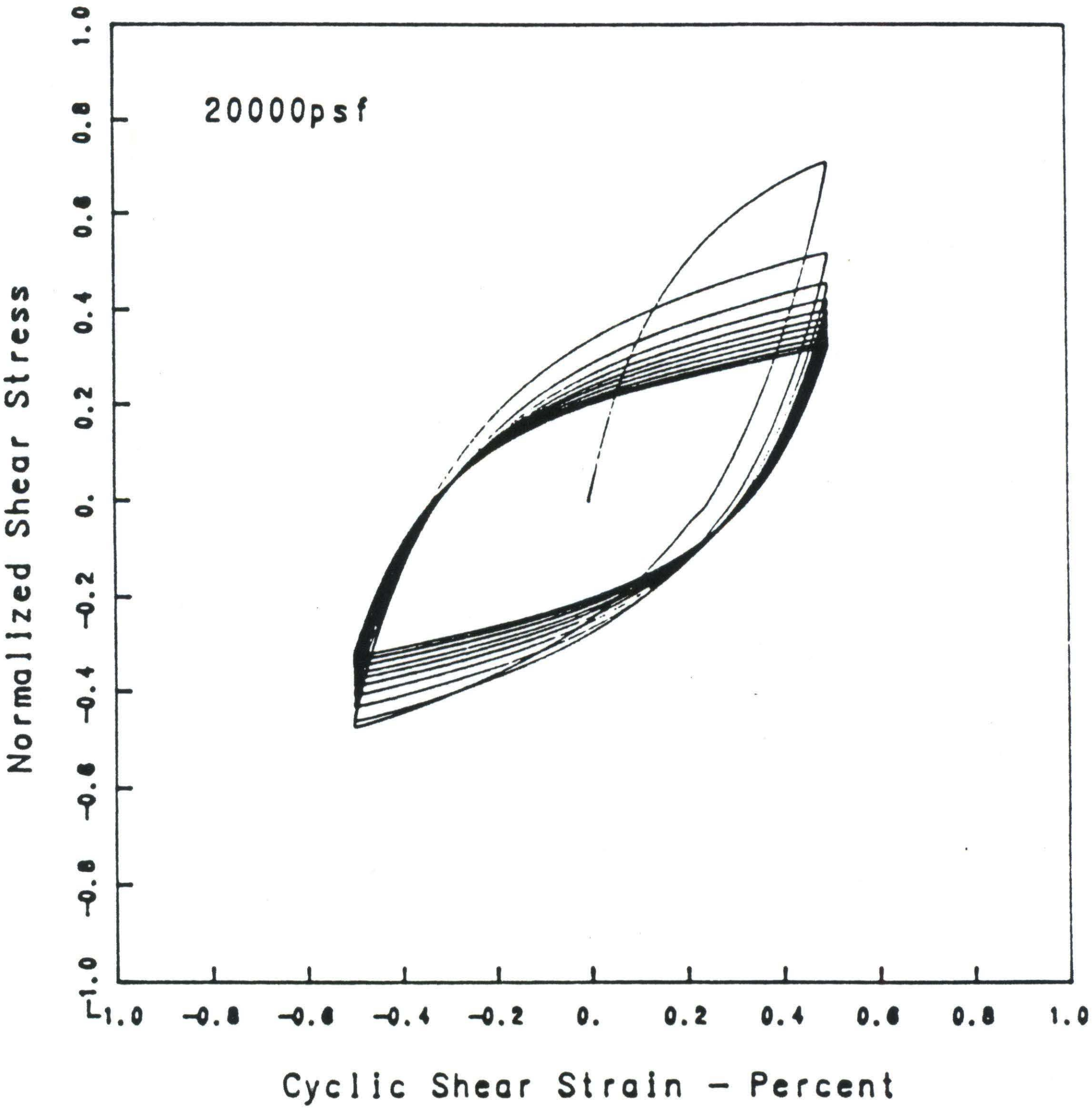

Figure 7.A-9c
Typical Stress-Strain Loops for Sand

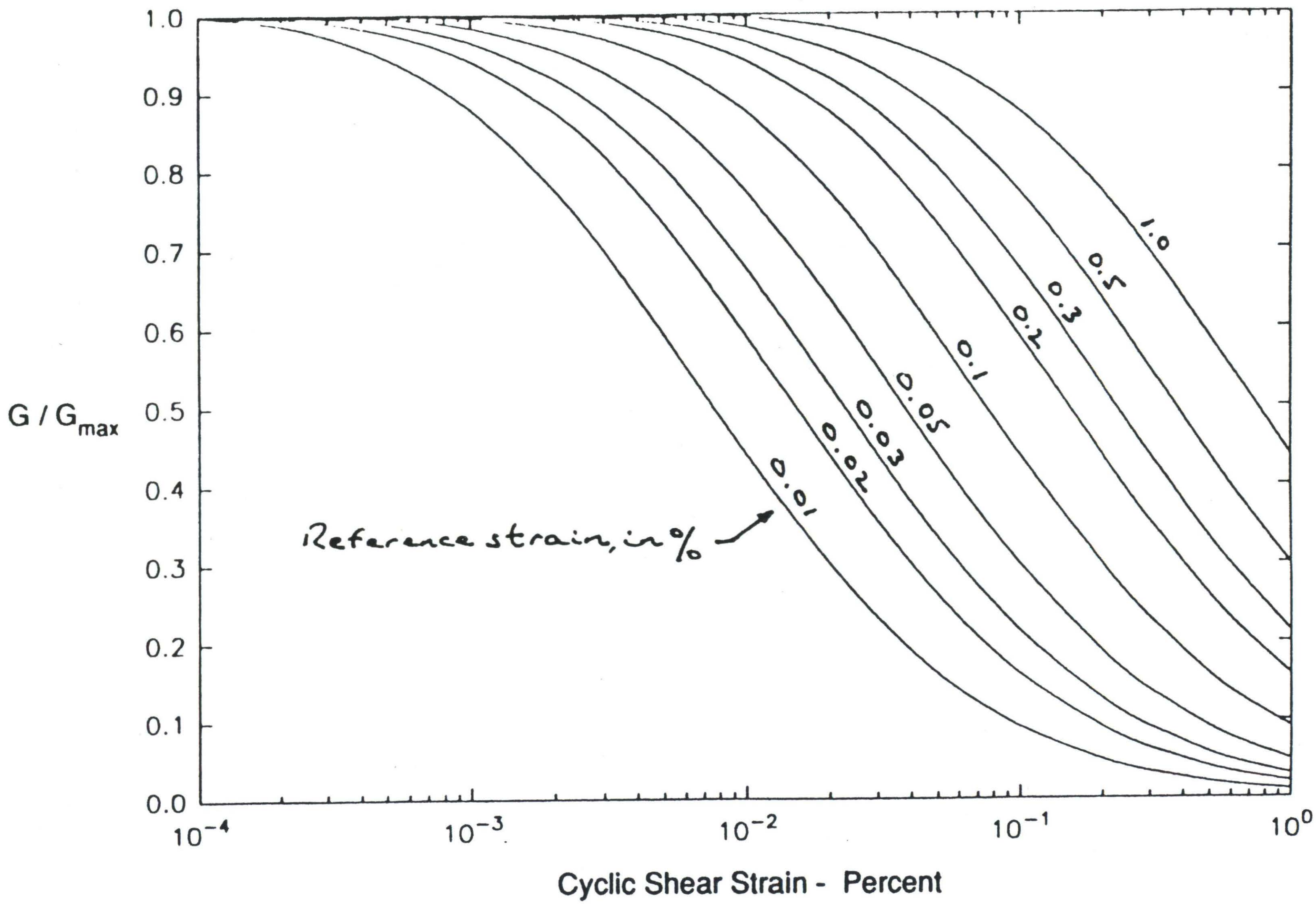

Figure 7.A-10
Modulus Reduction Curves as a Function of Reference Strain

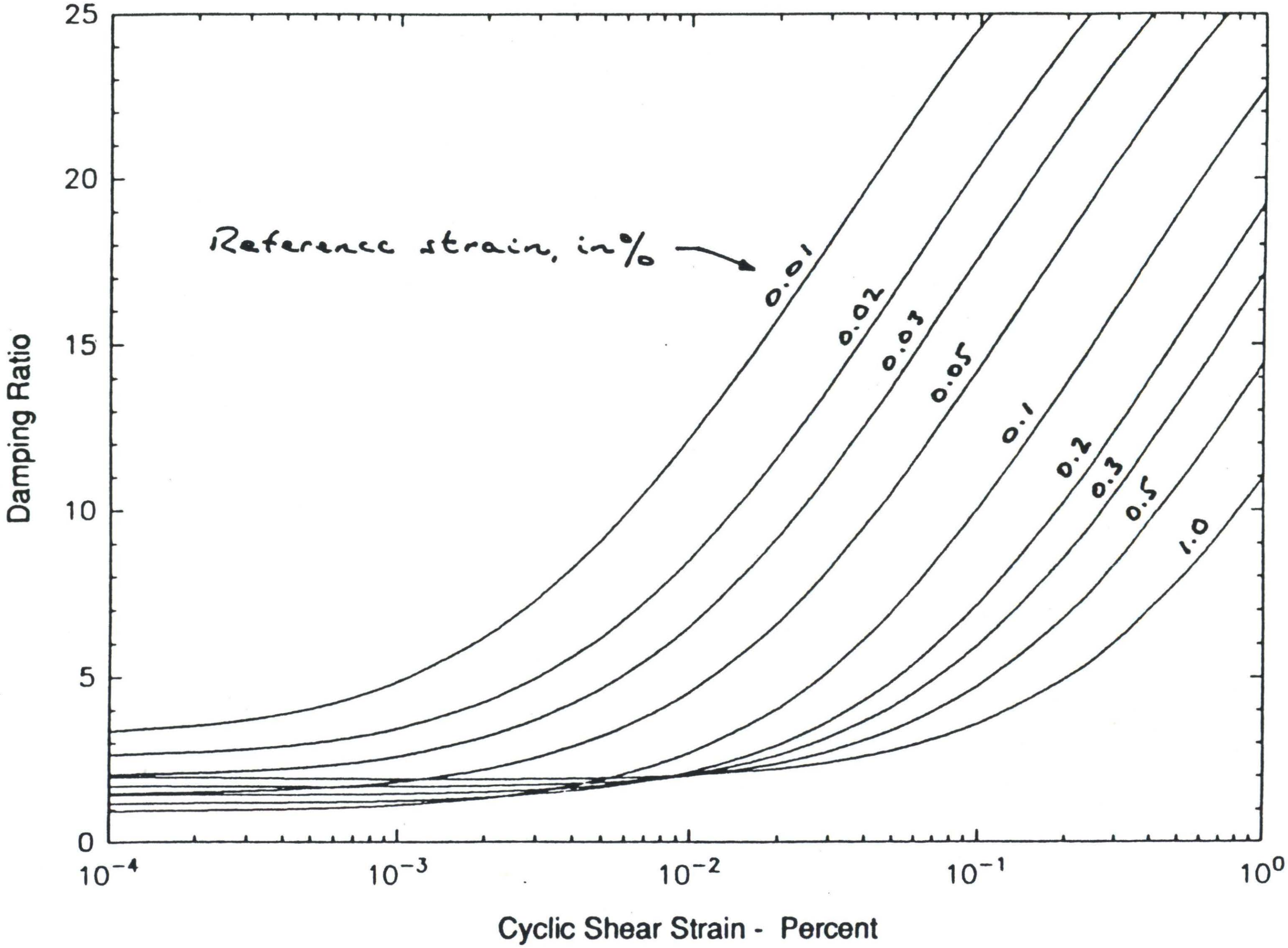

Figure 7.A-11
Damping Curves as a Function of Reference Strain

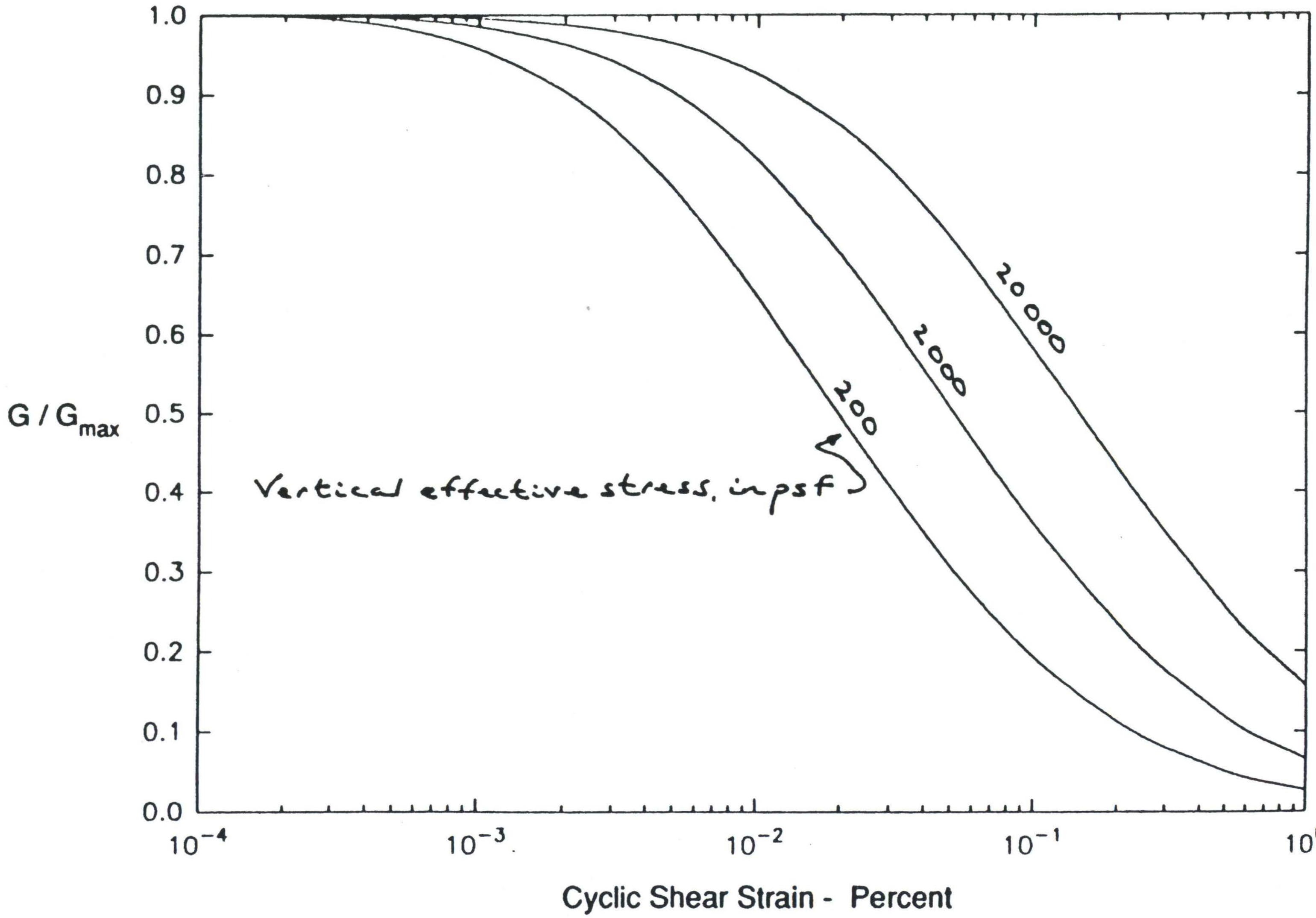

Figure 7.A-12
Modulus Reduction Curves for Dry Sands

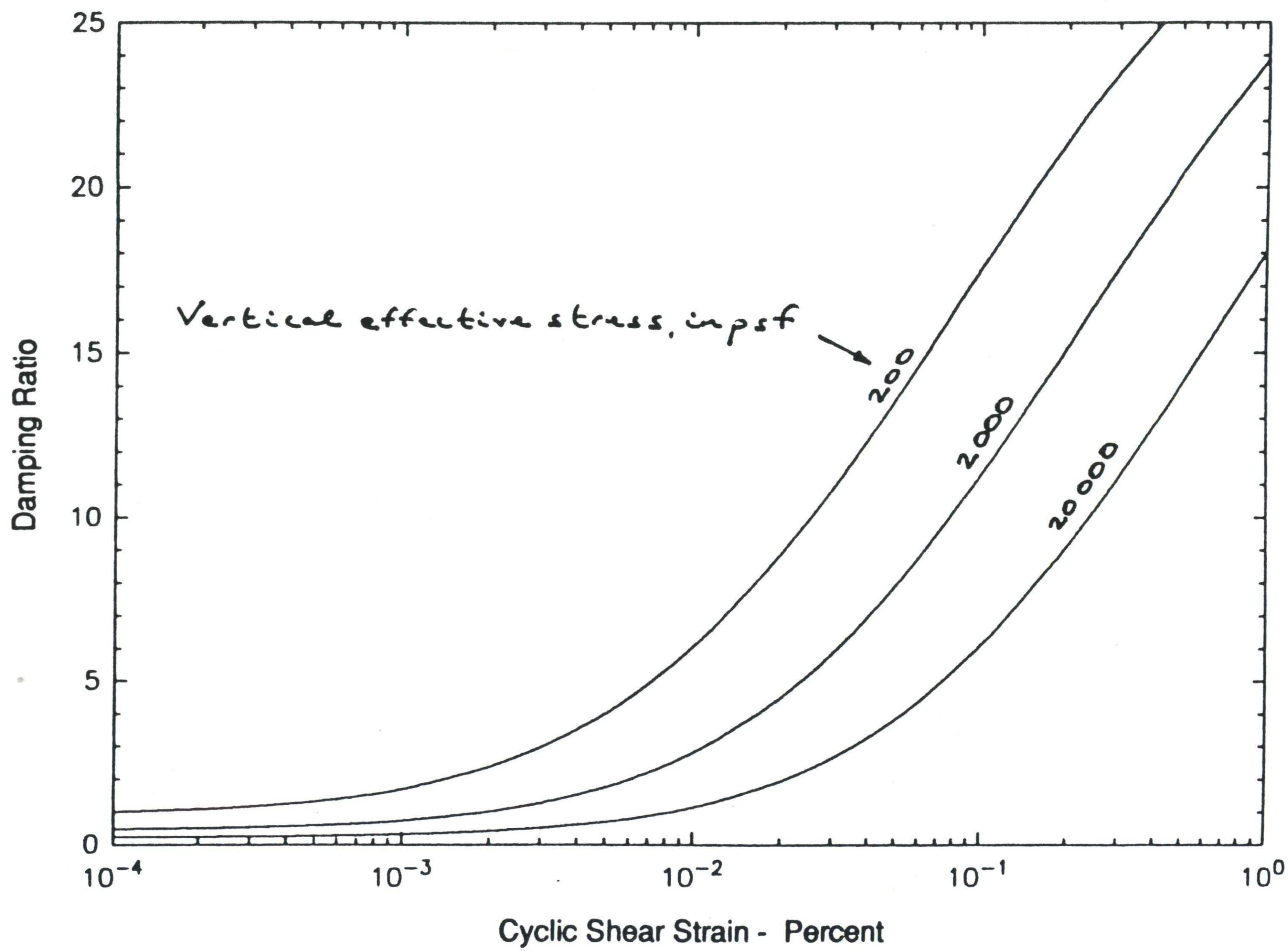

Figure 7.A-13
Damping Curves for Dry Sands

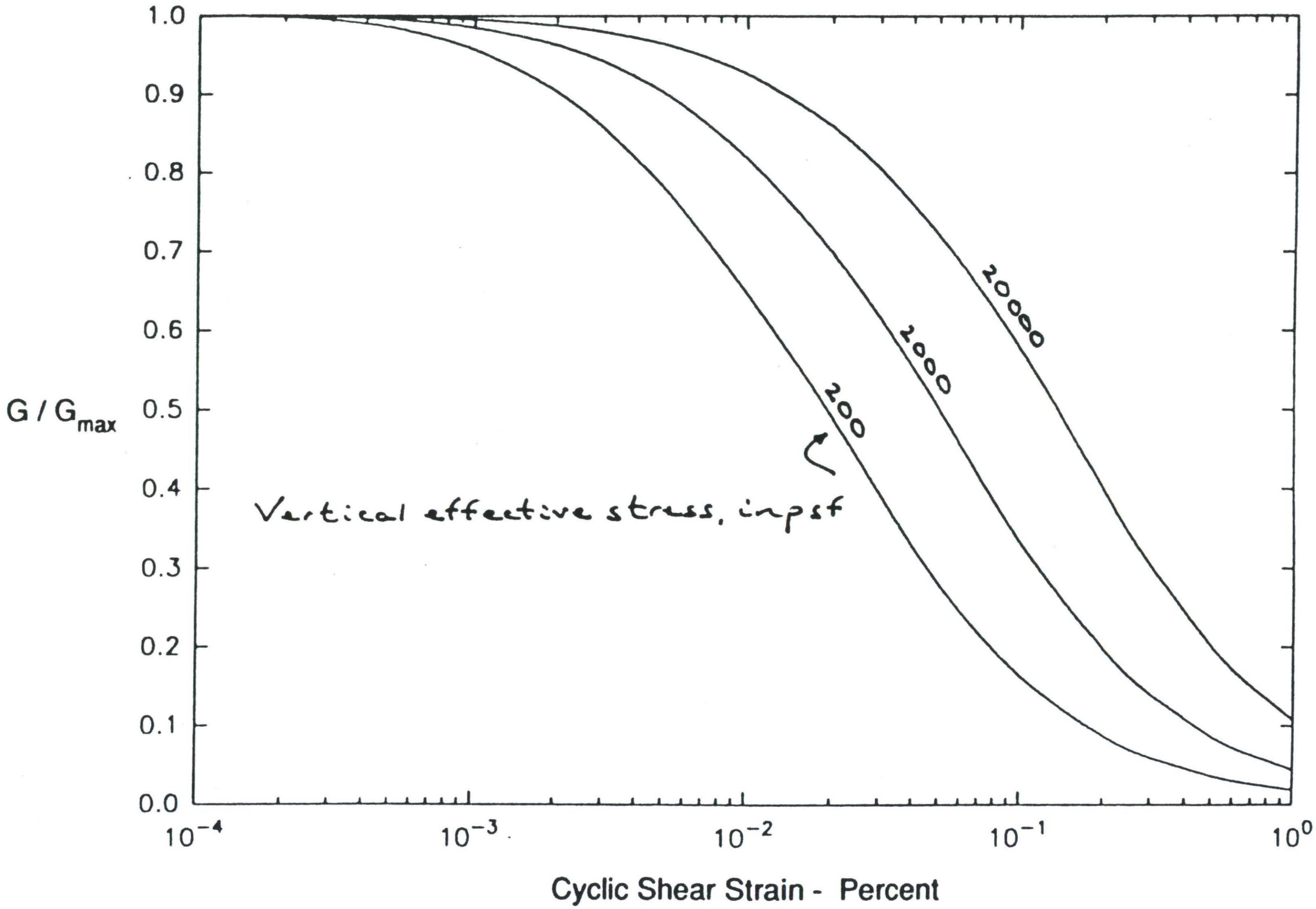

Figure 7.A-14
Modulus Reduction Curves for Saturated Sands

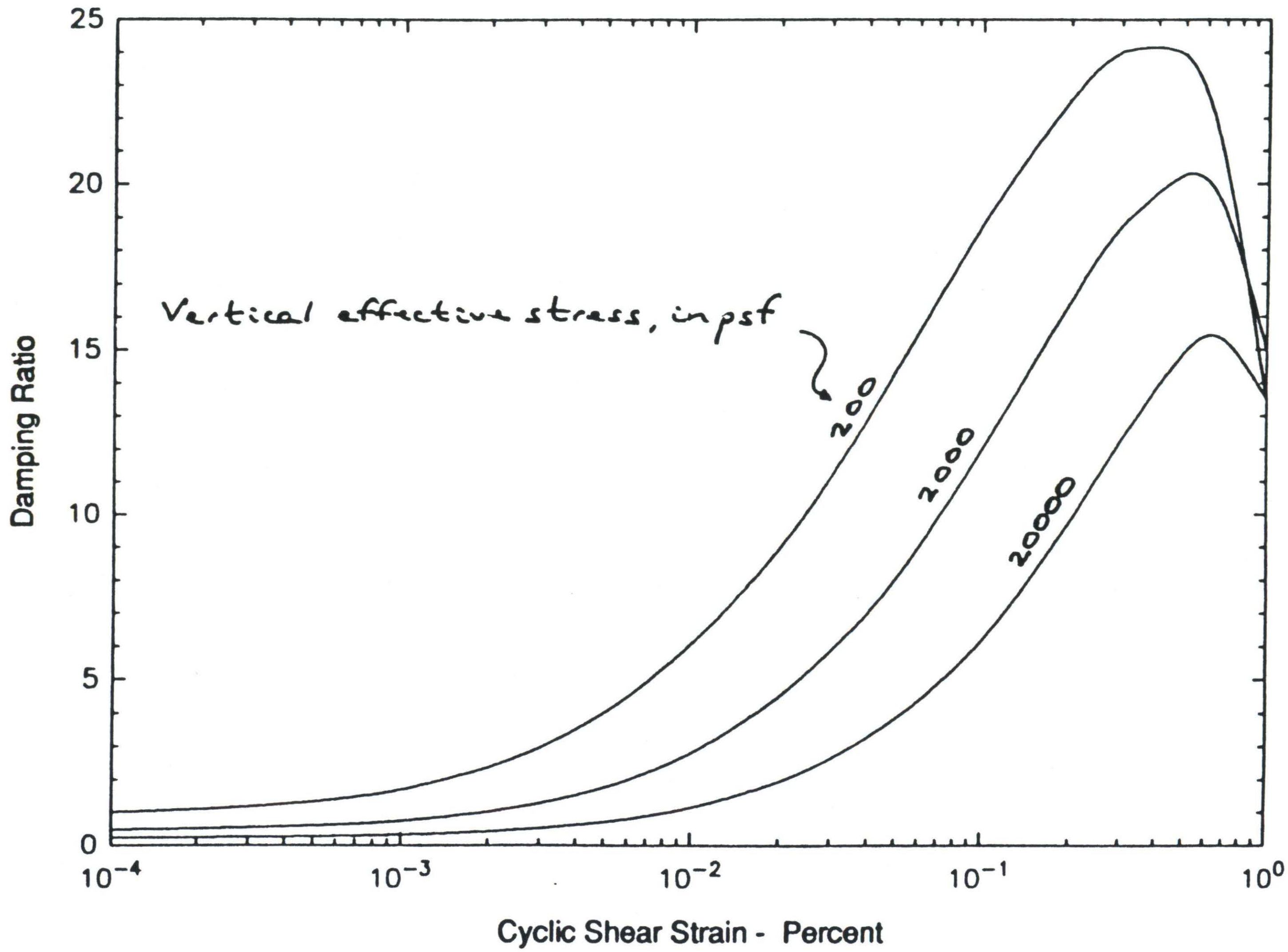

Figure 7.A-15
Damping Curves for Saturated Sands

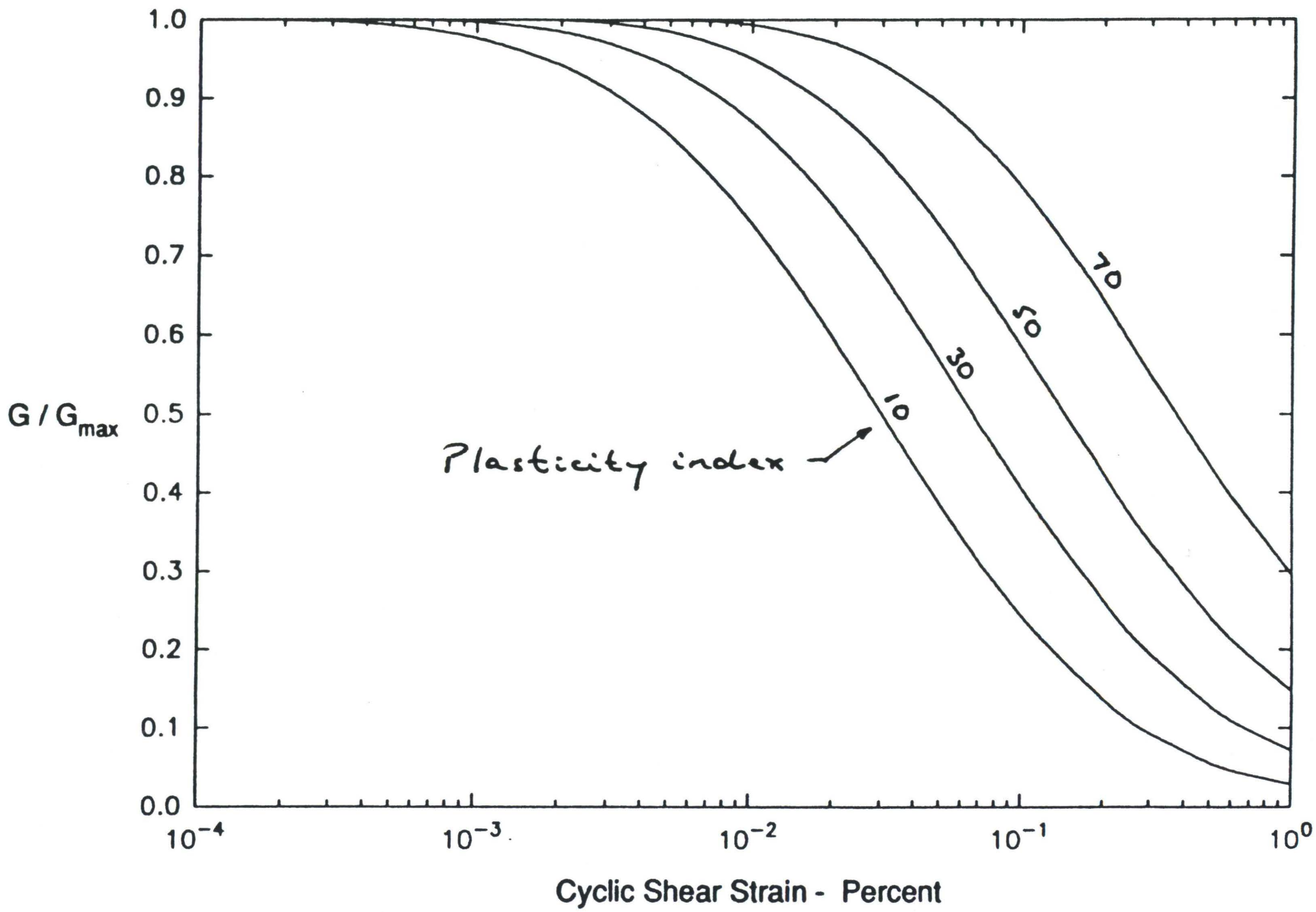

Figure 7.A-16
Modulus Reduction Curves for Clays

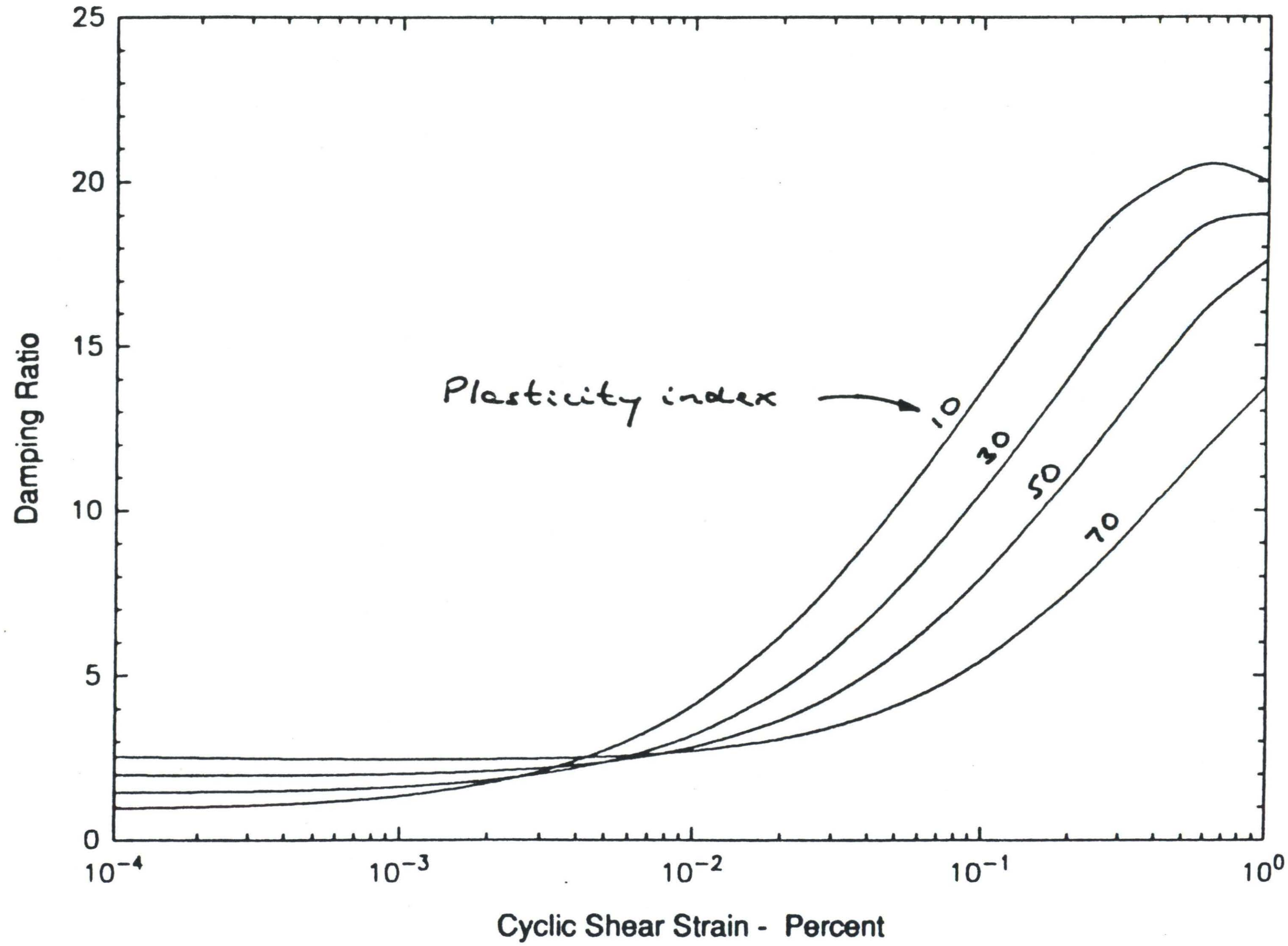

Figure 7.A-17
Damping Curves for Clays

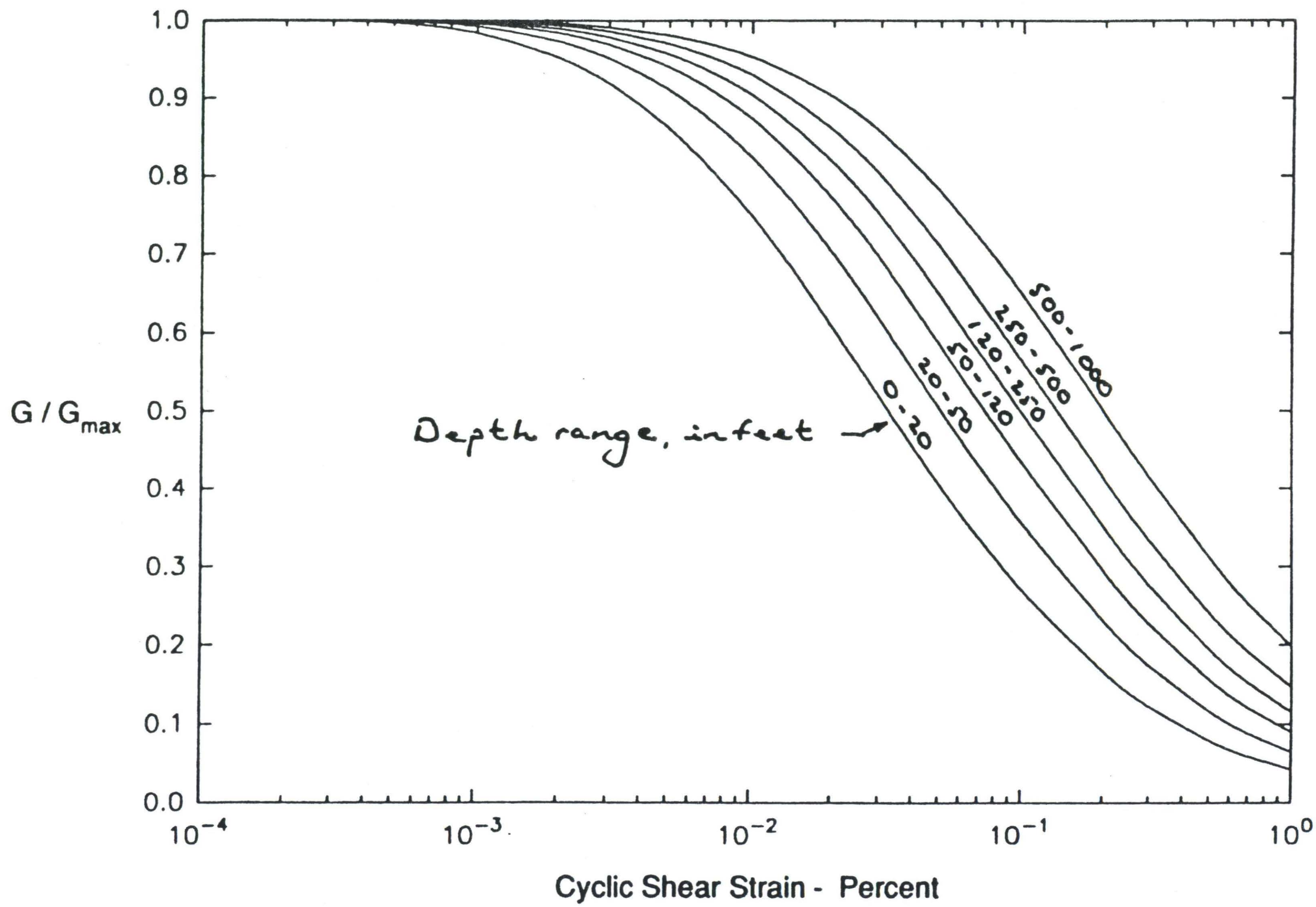

Figure 7.A-18
Modulus Reduction Curves for Generic ENA Sites

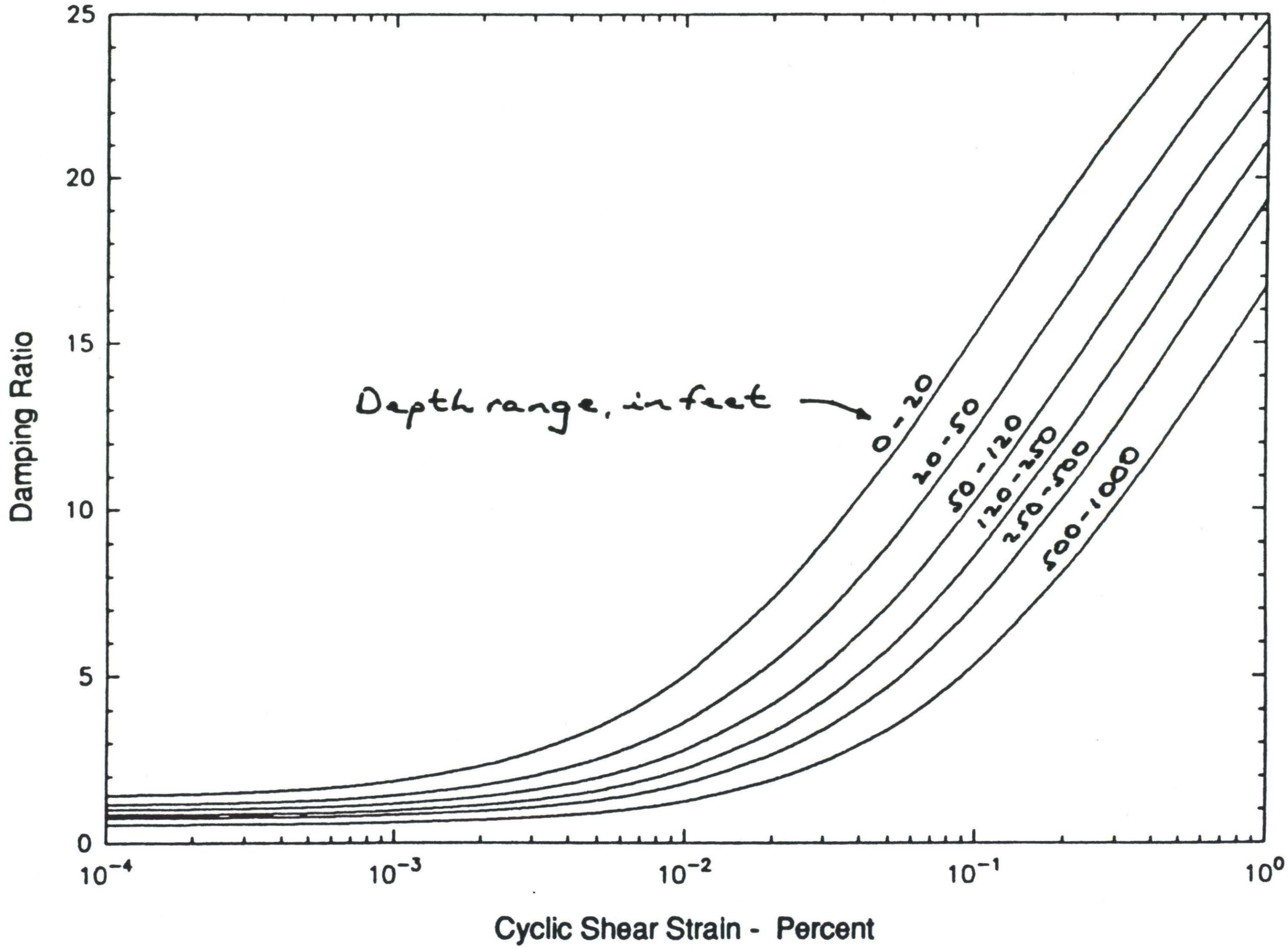

Figure 7.A-19
Damping Curves for Generic ENA Sites

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