5th Canadian Conf. Earthquake Engineering / Ottawa / 1987 / 5ème Conf. Canadienne Génie Sismique

Effect of anisotropy on dynamic properties of cohesive soils

K.S.Tawfiq & M.S.Aggour
University of Maryland, College Park, USA

ABSTRACT: The two primary dynamic soil properties shear modulus and damping characteristics were evaluated in a resonant column device, for cohesive soil samples obtained at different directions of preferred particle orientation, and subjected to different rates of isotropic stresses. It was found that the effect of particle orientation with regard to the propagated waves was more pronounced at low strain amplitudes than at high strain amplitudes. The parameters 'a' and 'b' that describe the variations of the shear moduli for different directions fit into ellipses. Increasing the isotropic state of stresses on the soil samples decreased the anisotropy dynamic behavior of the samples.

1 INTRODUCTION

Numerous problems in engineering require a knowledge of dynamic soil properties for satisfactory solution. These problems span a wide range of situations involving, at one end of the scale, very small amplitudes of motion, and at the other end high amplitudes of motion involving strong motion earthquakes or nuclear explosions. For pre-failure conditions and in cases where there are no permanent soil deformations, the most important properties to be determined are the soil moduli and damping characteristics. These properties are in turn, dependent on different parameters, some major, some minor, depending on the type of soil and loading conditions.

To evaluate dynamic soil properties three general approaches are usually used, namely laboratory tests, in-situ tests, and empirical correlations. The dynamic moduli and damping characteristics of undisturbed and remolded soil can be determined by a variety of laboratory testing methods. These methods all enable the measurement of soil properties at a wide range of strain levels and under different loading conditions. Useful information can be obtained from commonly used in-situ tests that utilize in-hole geophysical techniques to measure the wave velocity and hence the dynamic moduli of a large mass of soils. There is, however, still a lack of control over test variables and test boundary conditions. Additionally, under field testing conditions propagating waves usually travel through different soil layers at different angles. The velocities of the propagated waves are thus modified according to the directional elastic properties of the soil layers (Bolt, 1970). Researchers such as Stoneley (1955), Buchwald (1961), and Kraut (1963) have studied the effect of anisotropy of the half space medium on the elastic wave propagations. Moony (1974) suggested different methods to generate polarized shear waves in specific directions in field tests so that the elastic properties of the soil deposits correspond to these directions. In laboratory testing techniques, on the other hand, shear waves are propagated in one direction with regard to the particle orientations of the soil specimen, and usually on specimens obtained vertically from the soil deposits.

The existing differences between laboratory and in-situ testing results are an area that is receiving considerable attention. Several investigators have attributed such differences to disturbances of soil specimens that mask the effect of time, stress history and inherent anisotropy on the soil behavior. In the absence of detailed information on sampling and testing conditions, they suggested a factor of 2.5 to represent a reasonable average correction factor for these effects.

The primary objective of the present

study was to investigate the effect of anisotropy on the shear modulus and damping characteristics of cohesive soils. To accomplish such an objective, the soil specimens used were extruded from different directions from compacted soil. The dynamic properties for the different directions of particle orientation were determined at different times and strain levels in the resonant column apparatus. Understanding the effect of soil anisotropy will aid in clarifying the existing differences between laboratory and in-situ results.

2 SOIL TESTED AND SAMPLE PREPARATION

The required samples for this study were prepared from kaolinite clay known as Edger Plastic Kaolin, EPK. This type of clay is a particularly pure commercial kaolinite, and was received in dry, powdered form. From the values of LL and PI, this soil places near the mid-point between clays of low and high plasticity, so it can be considered an average cohesive soil with regard to plasticity. It was necessary to have available a large number of specimens for this study. The requirements were specimens of clay with as high a degree of saturation as possible and with the clay structure duplicated as closely as possible. This includes the void ratio, degree of saturation, particle orientation or fabric, mineralogy, and the composition of the double layer and pore water. Such duplication for large numbers of specimens could only be hoped for in remolded specimens, extruded from compacted soil. The compaction process under specific conditions would provide a variety of particle arrangements ranging from flocculated to nearly parallel orientations. These variations in soil structure, in turn, are controlled by different parameters such as water content, compaction energy, type of compaction, and pore fluid nature. However, the effect of these parameters on the compacted soil structure would vary depending on the sizes and shapes of the clay mineral crystals.

In natural clay, particles usually consist of different sizes and shapes. Moreover, most of the predominant clay particles are far from equidimensional. Thus, there would be many possible arrangements of particles with very irregular and very complex patterns. The patterns of arrangements, however, are less complex in compacted clays than in sedimentary clays because stratifications are less important. Sedimentary clays usually have layers of different sized particles resulting from the conditions of the deposition process. Layering between the compaction lifts and a variation in the soil due to changes in the characteristics of the borrow do, of course, occur. They are, however, of much less importance to the structure than are the sedimentation stratifications in natural clays.

For the purpose of this research the degree of saturation was a matter of concern, and the compacted sample was thought to have as high a degree of saturation as possible. This variable was met by first establishing the moisture density relationship for the EPK clay and then determining the optimum moisture content and degree of saturation lines.

Once the moisture-density-saturation relationship was established using the modified compaction test the required samples were then prepared at 40 percent moisture content with a degree of saturation of 94 percent.

To obtain specimens for the dynamic properties measurements having different directions with regard to the preferred particle orientation, a tube of 1.5 in. diameter and 8 in. in height was used. The tube was first lubricated inside and outside to eliminate friction with the soil and then pushed into the soil in the mold. Three different directions were chosen, vertical, 45 degrees to the horizontal, and horizontal. With regard to the particle orientation the directions are 90°, 45°, and 0°. The horizontal specimens were obtained by extracting the soil sample from the mold and then a plastic extension 6 in. in diameter and 3 in. in height and having a hole of 1.5 in. in diameter on the side was placed on the top edge of the mold, the specimens were then obtained by pushing the metal tube from the side hole into the compacted soil.

It should be noted here that the compacted soil specimens are susceptible to the thixotropic effect and this in turn would alter the mechanical behavior of the soil specimens with time. Therefore, it was necessary to avoid this effect when measuring the dynamic soil properties of the compacted samples. Based on studies done by different researchers (Moretto, 1946; Skempton and Northery, 1952; Seed and Chen, 1957; Mitchell, 1960), the thixotropy behavior of compacted soils depends on the strain levels at which the specimens are tested and water contents which affect the particle orientations.

Mitchell (1960) postulated that for high strain levels the effect of thixotropy is less pronounced and for a higher water content the interparticle attractive forces are too weak, therefore the effect of thixotropy would decrease. Gray and Kashmeeri (1971) performed resonant column tests on compacted soil samples and found an increase in the thixotropic shear ratio with time. However, with an increasing water content the ratio would increase and the rate of this increment would decrease as the water content approached the liquid limit of the soil. Therefore, for the samples used in this research the water content and the degree of saturation were as high as possible and the storage period exceeded two months before the testing program was conducted.

3 TEST EQUIPMENT AND PROCEDURES

The resonant technique was employed in this investigation for the measurement of the dynamic soil properties and damping characteristics of cohesive soil. The resonant column device used was of the "Drnevich" type in which soil specimens obtained from three different directions, vertical, horizontal and 45° with respect to the preferred particle orientations, were all subjected to the same parameters, so that a comparison of the dynamic soil properties for each direction could be made.

After mounting the soil specimen, assembling the resonant column apparatus, and before applying any confining pressure on the specimen, torsional excitation was applied to obtain initial values for the dynamic soil moduli and damping. These initial readings were taken as a reference for further readings when applying stress increments. For vertical specimens shear waves were applied tangential to the soil particles, and for horizontal specimens shear waves were applied perpendicular to the soil particles. For the inclined specimens shear waves acted at 45° to the particle orientations.

Upon completion of the initial readings, the desired confining pressure was applied to the soil specimen. The pressure was controlled by a pressure regulator and monitored through a column of mercury for accurate readings. Immediately after applying the stress increment, a torsional excitation with a low amplitude was applied to the soil specimen and the frequency of the excitation force was adjusted until the system's resonant frequency was obtained. The phase relationship describing resonance was established by observing the Lissajous figure formed on the X-Y oscilloscope. The frequency was then measured with a digital electronic frequency meter and recorded along with the input, output, and LVDT readings. The excitation force was then increased by varying the dial of the attenuater to obtain a higher amplitude and the frequency was adjusted accordingly to establish another resonance condition for the system. The sequence of the excitation forces applied was 10, 25, 50, 75, 100, and 125 MV (rms). At the first stage the strain levels ranged from 0.002 to 0.02 percent. However, as the time increased the excitation forces for the torsional motion were increased. That was to eliminate disturbing the soil specimen when applying high strain levels. The duration of the vibrations applied for each mode was approximately 15 seconds. The time sequence for successive readings of the torsional motion was 1, 10, 30, 60, and 120 minutes after increasing the confining pressure. Thereafter measurements were made about twice a day. Most of the tests were continued for eight days although some specimens were tested up to 42 days at a constant stress increment.

Upon completion of recording the data at one confining pressure, either the pressure was removed and the device was disassembled or the pressure was changed to the next pressure in the sequence. Because the amplitudes of vibration were relatively low, vibrations at a previous pressure had no effect on moduli and damping measurements at the following confining pressure; therefore a new measurement sequence could be conducted at the higher confining pressure on the same specimen. The confining pressure sequence used in this program was 5, 10, 20 and 30 psi. Some of the specimens were subjected to one stress increment and others were subjected to different stress increments and each stress increment was equal to 10 psi.

4 RESULTS

As an example of the data obtained, Figure 1 shows the variation of the shear modulus verses the shear strain for a vertical specimen subjected to a confining pressure of 5 psi. The dynamic soil moduli were recorded at different times and strain levels. In the figure the dynamic soil moduli increased as the time level increased and decreased as the strain amplitude increased. The rate of

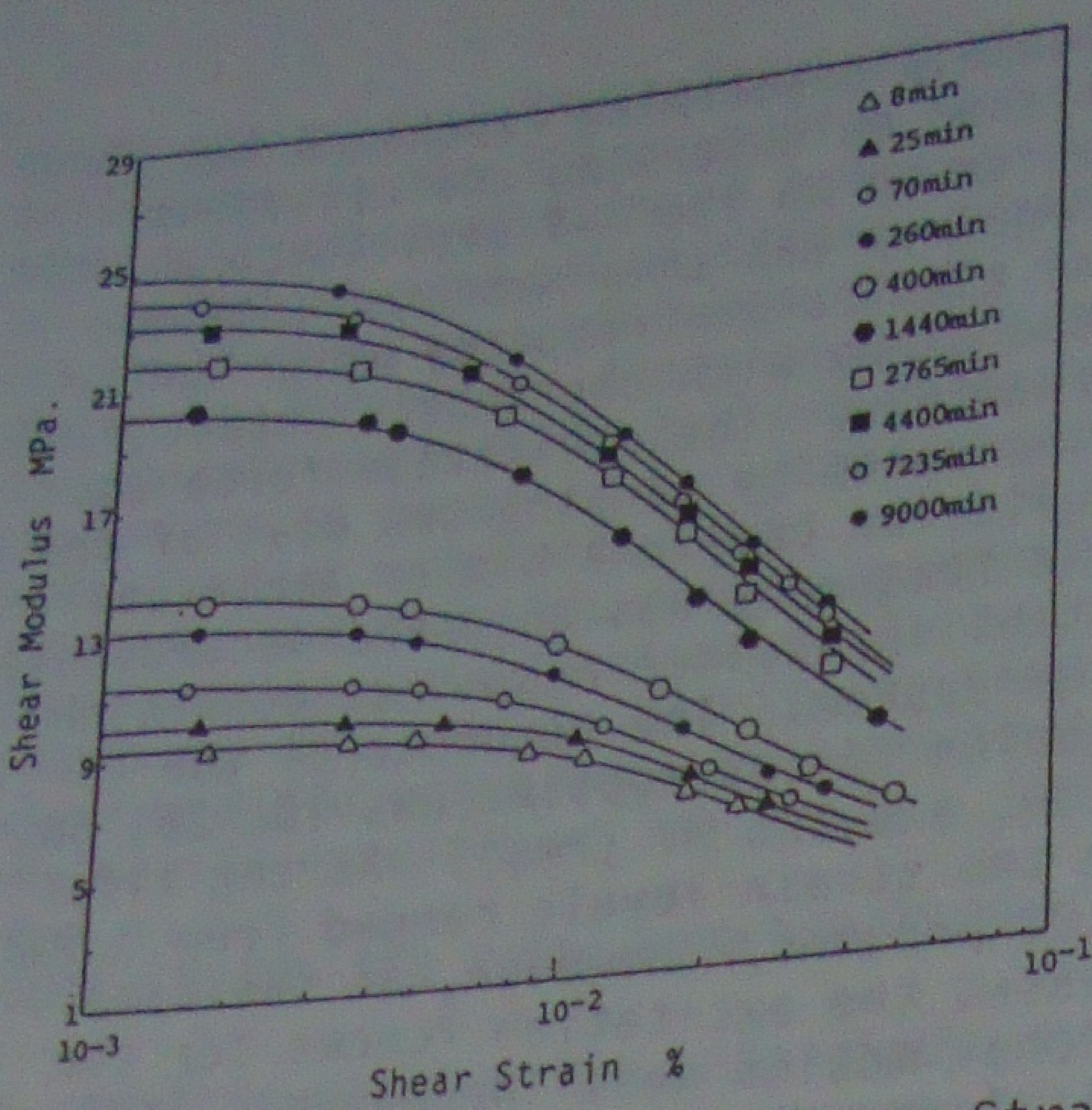

Figure 1 Shear Modulus vs. Shear Strain

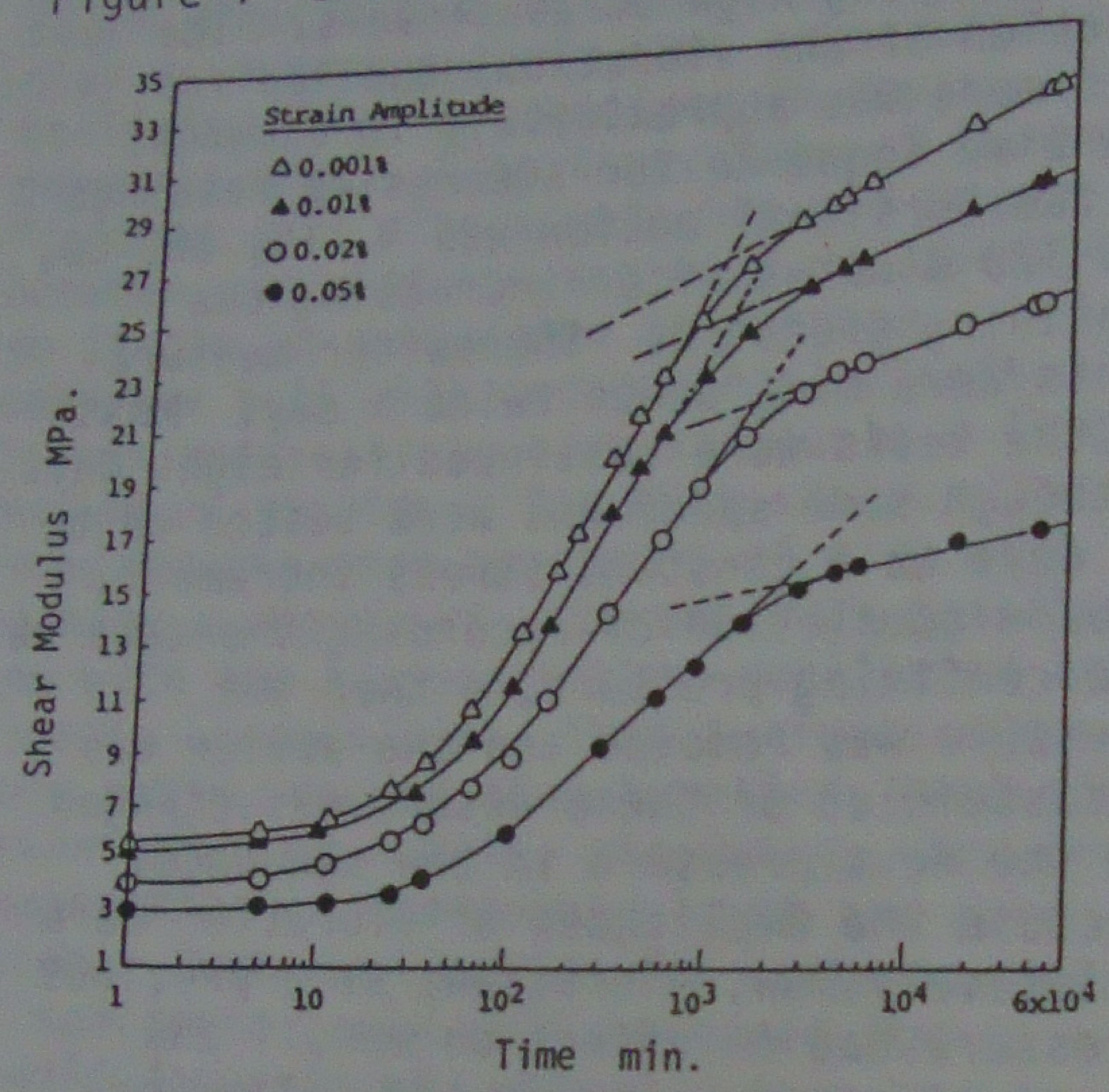

Figure 2 Shear Modulus vs. Time

increment in the value of the moduli was high during the first 1440 min, then the rate decreased as the duration of the confining pressure increased. This increase in the stiffness of the soil specimen for the first 24 hours was due to primary consolidation as compared with the increase in stiffness in the last few readings due to the secondary consolidation stage as shown in Figure 2. In the secondary consolidation stage, the increase in shear modulus values increased at a constant rate under a constant effective stress. It has been established (Hardin and Black, 1968; Afifi and Woods, 1971; and Anderson and Stokoe, 1978) that this portion of the shear modulus vs. time curve can be represented by a straight line in a shear modulus vs. log time plot. Based on that, the secondary increase in the values of the shear modulus can be quantitatively expressed by the slopes of the straight lines. For different strain amplitudes, confining pressures, and directions of particle orientation with regard to wave propagation, there were different values for the slopes of the straight lines, and different values for the point of interception with the vertical axis as shown in Figure 3. It was found that the values of the slopes denoted as 'a' and the values of the points of interception with the shear modulus axis denoted as 'b' can fit into curves that depict the variation of these constants with the different parameters under study. Examples of these curves are shown in a normalized form in Figures 4 and 5. Normalized values for 'a' and 'b' were obtained by dividing the values of 'a' and 'b' for all conditions by the 'a' and 'b' values for vertical specimens at strain amplitudes equal to 1×10^{-3} percent. It was also found that an ellipse can fit the values of 'a' and 'b' for different directions, at which the major axis for the ellipse of constant 'a' is horizontal

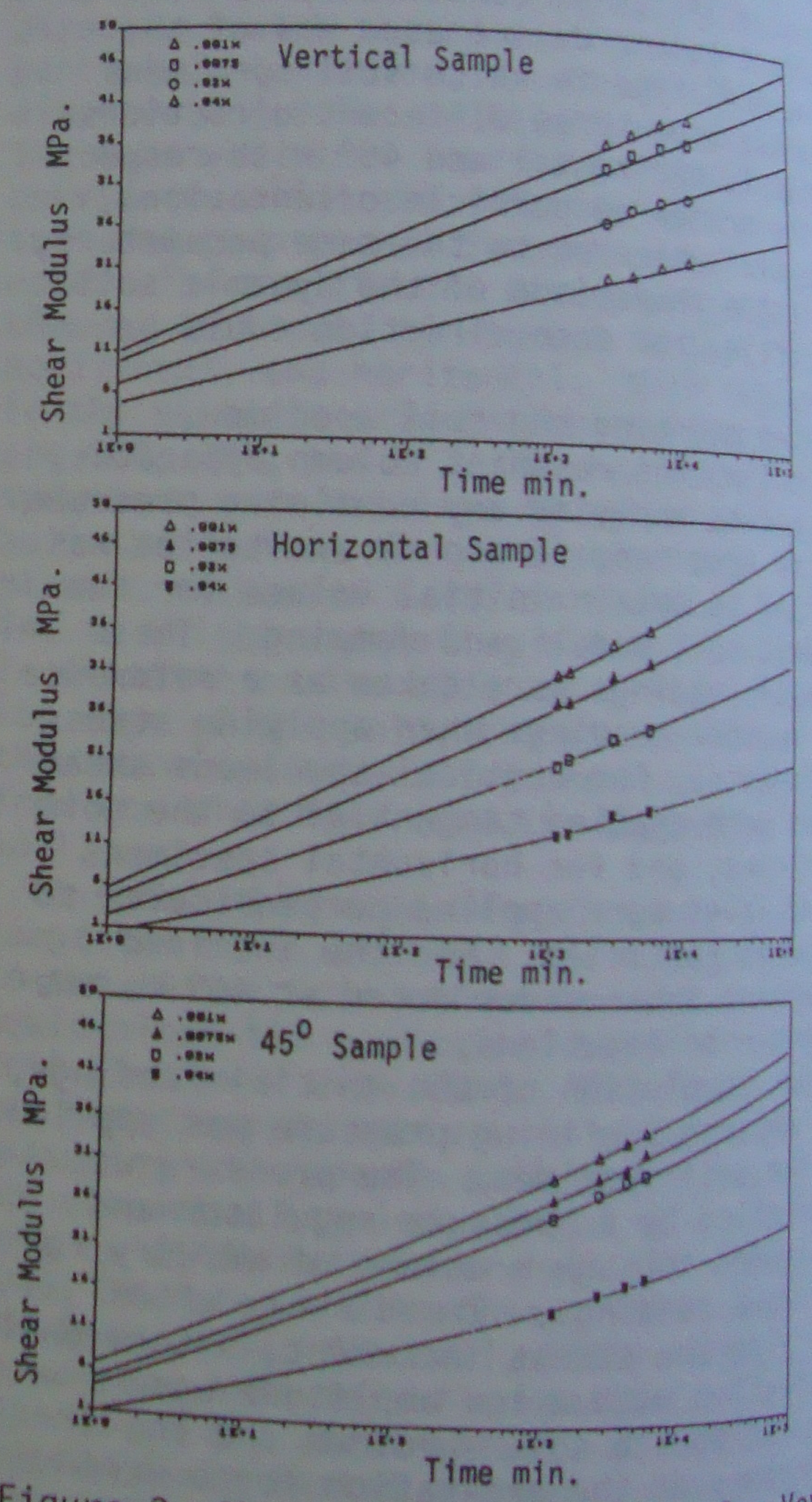

Figure 3 Shear Modulus vs. Time for Vertical, Horizontal, and 45° Samples.

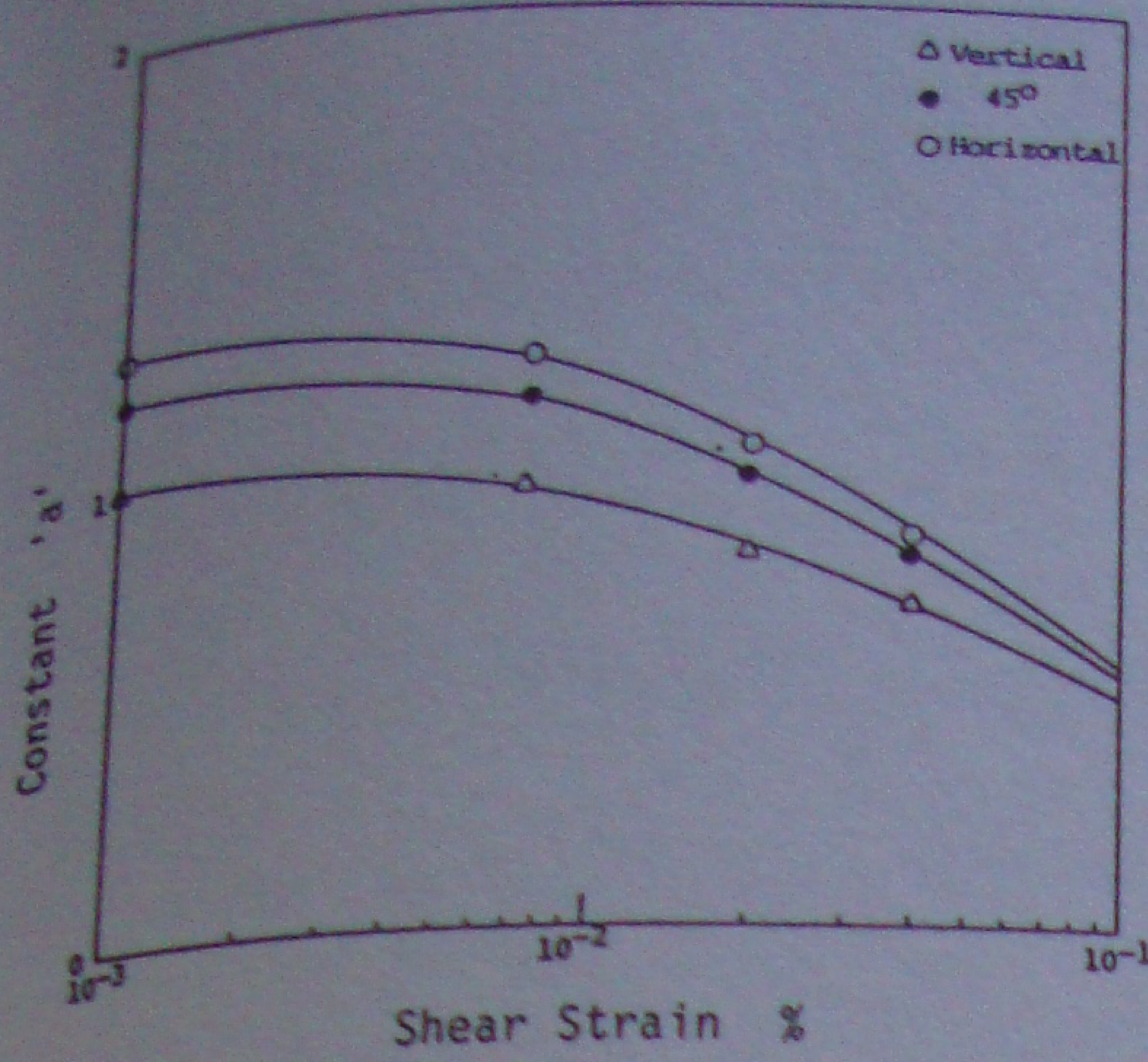

Figure 4 Normalized Values of Constant 'a'.

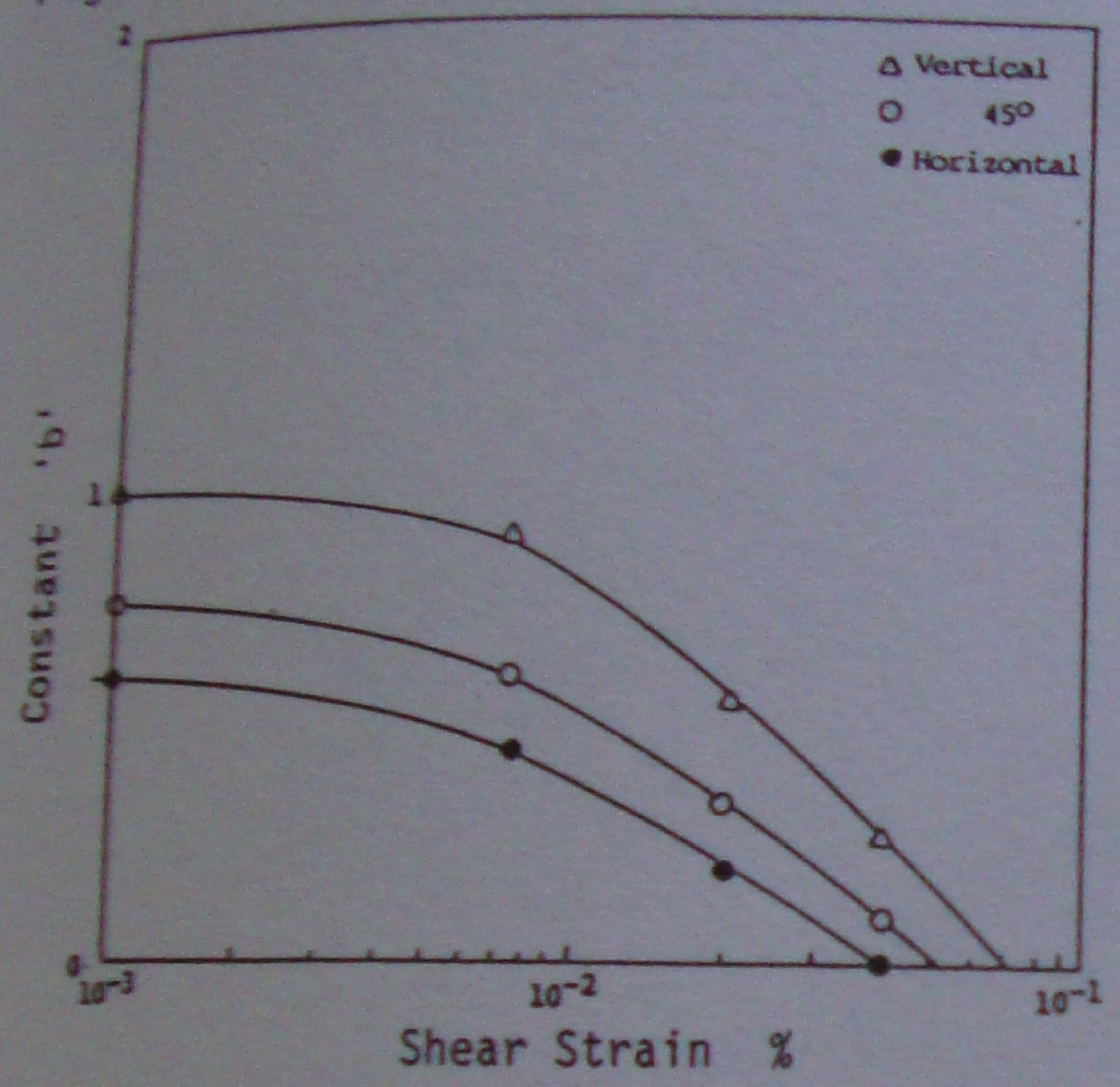

Figure 5 Normalized Values of Constant 'b'.

and the major axis for the ellipse of constant 'b' is vertical as shown in Figure 6.

For the case of damping, Figure 7 shows the damping values plotted as a function of the strain for the vertical, horizontal and the 45° specimens. As the figure shows, the damping values for all three cases are almost the same, i.e., soil anisotropy has no effect on the damping values within the range of variables tested.

5 CONCLUSION

Laboratory tests were conducted on soil specimens that were obtained from different directions with regard to the particle orientation for the determination of the dynamic properties. The values of the dynamic shear modulus obtained during the secondary consolidation stage were found to vary with the particle orientation direction, the strain

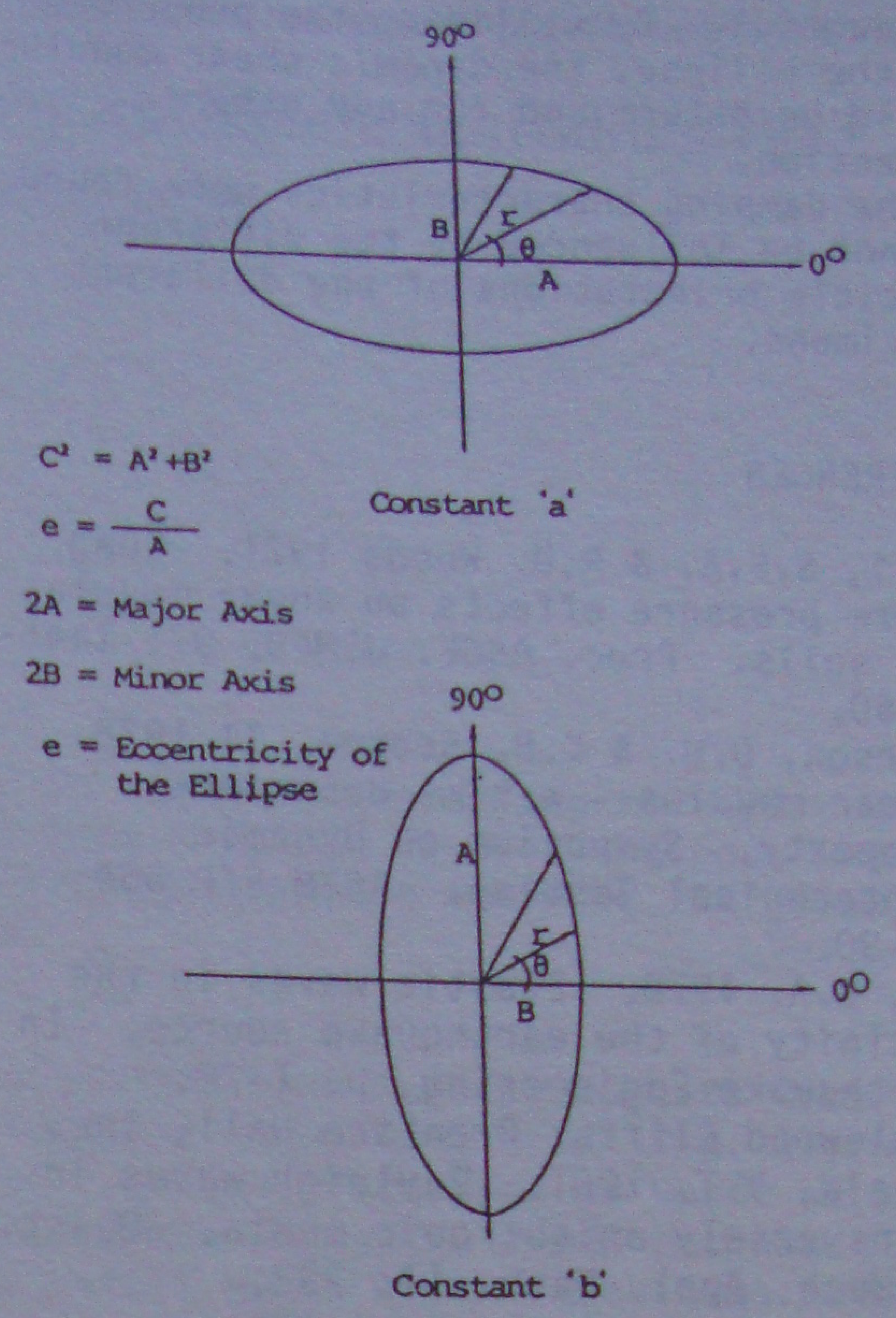

Figure 6 Variations in the Values of 'a' and 'b'.

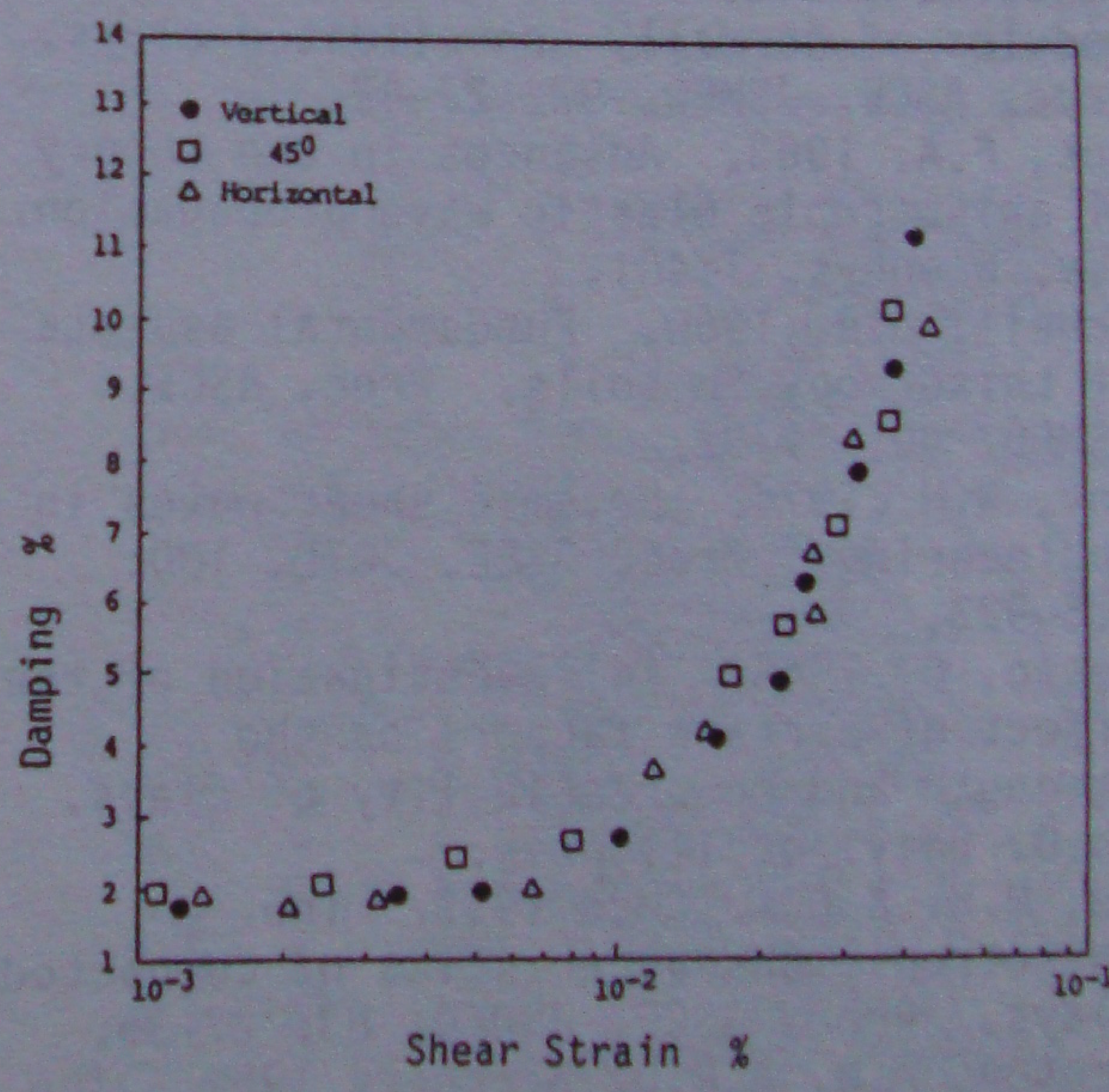

Figure 7 Damping vs. Shear Strain

amplitude, and the confining pressures.

A method was presented to determine the effect of anisotropy on the dynamic shear modulus. In the method, the 'a' and 'b' constants that represent the slopes and the point of interception with the shear modulus axis in the G vs. log t relationship were first determined for two specimens, one with the particle orientation vertical and the other horizontal. By utilizing the properties of the ellipse, the dynamic shear modulus could be determined for any other direction.

The damping characteristics were found to not be influenced by the different particle orientations of the different specimens.

REFERENCES

Afifi, S.E.A. & R.D. Woods 1971. Long term pressure effects on shear modulus of soils. Proc. ASCE. JSMFD. 97: 1445-1460.

Anderson, D.G. & K.H. Stokoe, II 1978. Shear modulus - a time dependent property. Symposium on Dynamic Geotechnical Testing. ASTM STP 654: 66-90.

Bolt, B.A. 1970. Elastic waves in the vicinity of the earthquake source. In Earthquake Engineering, p. 1-20. Englewood Cliffs: Prentice Hall, Inc.

Buchwald, V.T. 1961. Rayleigh waves in transversely anisotropic media. Quart. J. Mech. Appl. Math. 14: 293.

Gray, D.H. & N.A. Kashmeeri 1971. Thixotropic Behavior of Compacted Clays. Proc. ASCE. JSMFD. 97: 193-207.

Hardin, B.O. & W.L. Black 1968. Vibration modulus of normally consolidated clays. Proc. ASCE. JSMFD. 92: 27-42.

Kraut, F.A. 1963. Advances in the theory of anisotropic elastic wave propagation. Rev. Geophys. 1:401.

Mitchell, J.R. 1960. Fundamental aspects of thixotropy in soils. Proc. ASCE. JSMFD. 86: 19-52.

Moony, N.M. 1974. Seismic shear waves in engineering. Proc. ASCE. JGTD. 100: 905-923.

Moretto, O. 1946. An investigation of the effect of certain factors on the strength and compressibility of clays. Ph.D. Univ. of Illinois.

Seed, H.B. & C.K. Chen 1957. The thixotropic characteristics of compacted clays. Proc. ASCE. JSMFD. 83: 27-35.

Skempton, A.W. & R.D. Northery 1952. The sensitivity of clays. Geotechnique 3: 30-35.

Stoneley, R. 1955. On the dynamical theory of diffraction. Trans. Camb. Phil. Soc. 9: 243.