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EARTHQUAKE ENGINEERING RESEARCH CENTER

EFFECT OF MULTI-DIRECTIONAL SHAKING ON LIQUEFACTION OF SANDS

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A report on research sponsored by
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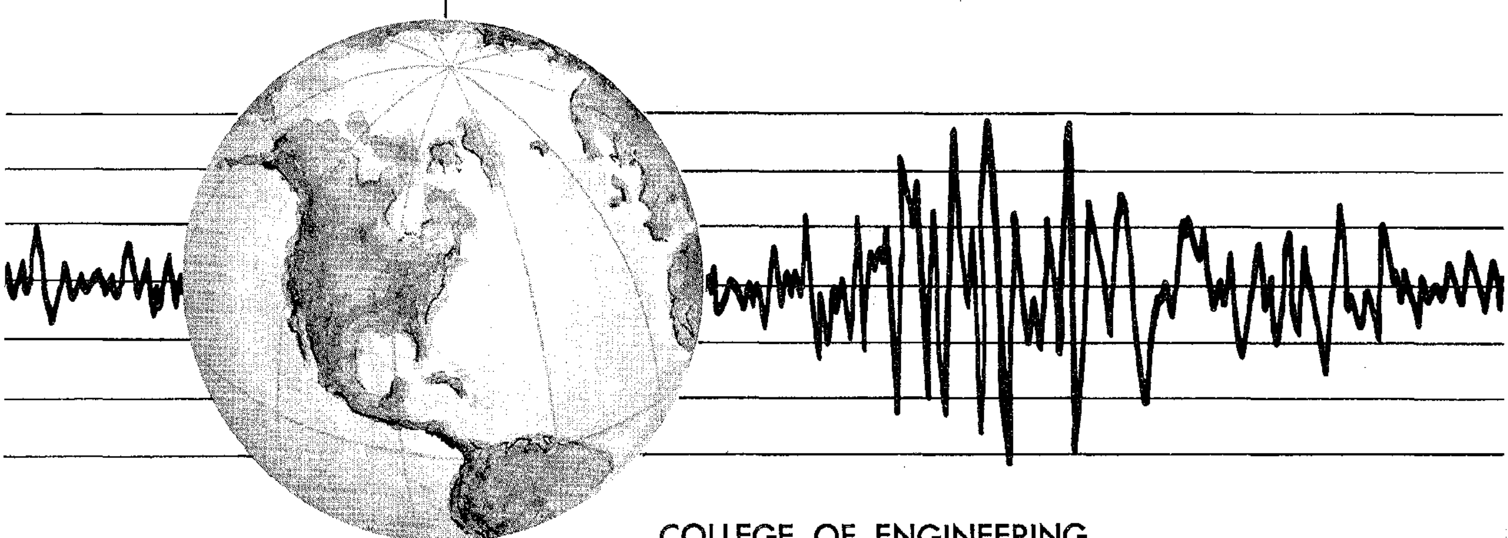

COLLEGE OF ENGINEERING
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H. Bolton Seed,[1] Robert Pyke[2] and Geoffrey R. Martin,[3] Members, ASCE

Introduction

The phenomenon of liquefaction of sands under cyclic loading such as that produced by earthquakes has received much attention in recent years. While the basis for any evaluation of the significance of this phenomenon is the historic evidence that liquefaction has, or has not, caused damage in certain circumstances, our understanding of the problem and our ability to consider situations without direct precedent has been greatly enhanced by the development of analytical procedures for evaluating liquefaction potential. In effect however, these procedures have generally considered only one component of motion, whereas the shaking induced by earthquakes is in fact multi-directional. Recent studies by Pyke, Seed and Chan (1974) have shown that the settlement of dry sand is relatively greater under multi-directional shaking than under 1-directional shaking of similar amplitude and thus it appears that the liquefaction potential of a saturated sand will be increased if this factor is taken into account. The effect of multi-directional shaking on liquefaction potential is examined quantitatively in this paper by combining the results obtained by Pyke (1973) in shaking table and cyclic simple shear tests on dry sand with the model of the

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mechanism of liquefaction proposed by Martin, Finn and Seed (1975). The paper also serves to illustrate the practical application of this model for engineering purposes.

Background Information

The analytical procedures currently used for evaluating liquefaction potential generally follow those suggested by Seed and Idriss (1967, 1971). In these procedures the average cyclic shear stresses induced at various points in the ground are evaluated using one-dimensional or plane strain finite element models. These stresses are then compared with the uniform cyclic shear stresses causing liquefaction of representative elements of soil at different depths as determined by laboratory tests; that the test specimens be truly representative in this procedure is extremely important (Seed, 1976). If the occurrence of liquefaction is not predicted, then a factor of safety equal to the stress required to cause liquefaction divided by the stress induced by the earthquake may be determined for each point in the deposit.

Cyclic triaxial tests have generally been used to evaluate the resistance to liquefaction of laboratory samples. It is believed, however, that the loading to which elements of soil are subjected in earthquakes is better reproduced in cyclic simple shear tests. Comparative studies have indicated that for normally consolidated sands the shear stresses causing liquefaction under simple shear conditions are less than those causing liquefaction in cyclic triaxial tests where the isotropic consolidation stress in the triaxial tests is equal to the vertical consolidation stress in the simple shear tests. Seed and Peacock (1971) have suggested that the stress ratio τ_h/σ'_v causing liquefaction under cyclic simple shear conditions might be related to the stress ratio $\sigma_{dc}/2\sigma_a$ causing liquefaction in cyclic triaxial tests by a correction factor, c_r, where

$$\frac{\tau_h}{\sigma'_v} = c_r \frac{\sigma_{dc}}{2\sigma'_a}$$

Recent studies at the University of California, Berkeley, in which Monterey No. 0 sand was used in both shaking table and cyclic triaxial tests indicate values in the order of 0.63 to 0.65 for this correction factor (DeAlba et al, 1975).

In the application of these procedures only one component of motion has normally been considered in as much as the induced shear stresses are computed using only one component of motion and the laboratory tests have involved cyclic loading in one direction only. The basis for assigning the amplitude and duration of the component of motion used in the analysis is variable, but commonly the peak acceleration used is estimated on the basis of the greater of the two horizontal components of recorded motions. The relative amplitudes of the two horizontal components of recorded motions of course vary, being a function not only of any preferred direction of motion in either bedrock or soil deposits but also the orientation of the recording instrument. For design purposes it is usual to assume two equal components, but this will be conservative if the values used are the greatest that might occur in any direction.

The effect of vertical motion is usually disregarded in studies of saturated soils because it is believed that the transient vertical inertia forces will be carried primarily by the pore water, and cause little change in the effective stresses carried by the soil grains or in the residual pore water pressures. This is a consequence of the fact that for a fully saturated soil the stiffness of the pore fluid is usually at least an order of magnitude greater than the stiffness of the soil skeleton, and vertical accelerations greater than 1g are required to produce any significant tendency for volume reduction in the soil grain structure.

The Effect of Multi-Directional Shaking on the Settlement of Dry Sand

Because the phenomenon of liquefaction is related to the tendency for dry or drained granular materials to compact under cyclic loading, it should be possible to gain at least a qualitative estimate of the effect of multi-directional shaking on liquefaction by studying the results of tests on dry sand.

Tests in which dry Monterey No. 0 sand was subjected to unidirectional and multi-directional shaking under simple shear conditions have been reported by Pyke, Seed and Chan (1974). Two basic patterns of motion were used in these tests. The first was a combination of two sinusoidal components with a phase difference of 90 degrees so that a circular resultant motion, termed gyratory shear, was obtained. The second was a pair of randomly generated motions (see Fig. 1) which could be run with the peaks of the two components either in or out of phase. The loading was acceleration or stress controlled and it was found that for any of the circumstances described above the settlement caused by the combined horizontal motions was approximately equal to the sum of the settlements caused by the two compnnents if these were run separately.

The results of the tests using random motions are summarized in Fig. 2 where the settlement in 10 cycles of loading is shown as a function of the stress ratio, τ_h/σ'_v , where τ_h is the maximum horizontal shear stress and σ'_v is the applied vertical stress. These tests confirmed the previous finding by Silver and Seed (1971b) that for a given cyclic shear strain the induced settlement is independent of the vertical stress; however, if a test is run under stress controlled conditions, the cyclic shear strains and hence the settlements increase with increasing values of the stress ratio. For a given value of the stress ratio it may be seen that the settlement caused by the combined motion is approximately equal to the sum of the settlements caused separately by the X and Y components. However, because the stress-settlement relationship is non-linear, the stress ratio causing a given settlement for the combined motions is typically only about 20 percent less than the stress-ratio which causes the same settlements under a single component.

On the basis of these results it seems reasonable to postulate that for saturated sands tested under undrained conditions the pore pressures will increase approximately twice as fast under two equal components of shaking as compared with shaking under one component only. The cyclic shear stress causing liquefaction in a given number of cycles under multi-directional shaking however, would only be slightly less than the cyclic shear stress causing liquefaction for shaking in one direction only; this reduction may be in the order of 20 percent, but it should be noted that the stress ratio, τ_h/σ'_v used to characterize the shaking table test conditions is not directly comparable with the stress ratio, τ_h/σ'_v , used in liquefaction analyses because the histories of effective stress and cyclic shear strain will be different under drained and undrained conditions.

While these conclusions might be accepted as sufficient for practical purposes, it is instructive to examine the relationship between drained and undrained behavior in more detail, in order to provide a quantitative evaluation of the effect of multi-directional shaking on liquefaction.

Model of the Mechanism of Pore Pressure Development Leading to Initial Liquefaction

A model by which the behavior of sands under undrained cyclic loading can be computed from data obtained in tests on dry sand has been suggested by Martin, Finn and Seed (1975). This model may be illustrated as shown in Fig. 3. As a consequence of any applied cyclic stress or strain under undrained conditions, the structure of a cohesionless soil tends to become more compact, with a resulting transfer of stress to the pore water and a reduction in stress on the soil grains. As a result of the reduction in stress, the soil grain structure rebounds to the extent required to keep the volume constant and this interplay of volume reduction and soil structure rebound on successive cycles determines the magnitude of the increase in pore water pressure increase in the soil.

If the magnitude of the vertical strain due to reduction in volume of the structure of a soil element in any one cycle is $\Delta\varepsilon_{vd}$ and the deformation modulus for one-dimensional or constrained unloading is designated E_r, then the stress release required to maintain constant volume will be

$$\Delta\sigma = E_r \cdot \Delta\varepsilon_{vd} \tag{1}$$

and the corresponding increase in pore water pressure will be

$$\Delta u = E_r \cdot \Delta\varepsilon_{vd} \tag{2}$$

The vertical strain per cycle can be determined by cyclic loading tests on dry sand and the rebound modulus by a rebound test on the dry sand. Thus the change in pore pressure can be computed directly from these data.

That such a mechanism provides a reasonable model of the behavior of saturated sands subjected to cyclic stress or strain applications can be illustrated by simple shear tests on dry sands. Indeed if a dry sand is subjected to cyclic loading in a constant volume test, the relaxation of vertical effective stress is similar to that which occurs when the constant volume condition is maintained in the presence of water (Pickering, 1973).

It should be noted that equation (2) assumes that the stiffness of the pore fluid is significantly greater than that of the soil skeleton. Should this not be true because of incomplete saturation or any other reason the full expression given by Martin, Finn and Seed (1975) should be used.

In order to perform computations using the model of soil behavior described above it is clearly necessary to have available data which provides the values of settlement per cycle consistent with the previous strain history, together with data describing the modulus on unloading. However, it is also necessary to have data concerning the shear modulus at the appropriate levels of effective stress and previous strain history in order to model a stress-controlled cyclic loading condition because it is necessary to compute the cyclic shear strain in order to obtain the settlement per cycle. The results of tests conducted to develop such data for Monterey No. 0 sand are presented in the following section.

Data on the Characteristics of Dry Sand Under Cyclic Loading

In order to develop a full set of data on the characteristics of dry Monterey No. 0 sand under cyclic loading conditions, a series of tests was performed using the NGI type cyclic simple shear device previously described by Silver and Seed (1971a). Tests were conducted at three strain levels and three vertical stress levels for each of three relative densities. While the tests were intended to have constant cyclic shear strains, the shear modulus tended to increase as a result of the cyclic loading and because of some flexibility in the loading system, the cyclic shear strains actually measured on the sample decreased as the modulus increased. This may be seen from the results of a typical test presented in Fig. 4, which shows the variation of shear stress, shear strain, shear modulus and cumulative vertical settlement with increasing number of cycles.

In order to eliminate the effect of the varying cyclic shear strain and to present the data in a form convenient for further analysis, the settlement per cycle is shown in Fig. 5 as a function of cyclic shear strain for the test results shown in Fig. 4 and for a number of other tests conducted at 60 percent relative density. The path of each test is shown by a dashed line and values of the total settlement corresponding to the various data points and curves are marked on each path.

For example, from the test data shown in Fig. 4, the following values may be read off directly:

Table 1. Material Characteristics Determined From Test Data in Fig. 4.

(1) Total Prior Vertical Settlement -percent	(2) Settlement per Cycle -percent	(3) Cyclic Shear Strain -percent	(4) Cyclic Shear Stress -psf	(5) Shear Modulus -psf
0	0.105	0.235	480	2×10^5
0.1	0.055	0.220	480	2.2×10^5
0.2	0.032	0.202	600	3.0×10^5
0.3	0.015	0.187	690	3.7×10^5
0.4	0.005	0.170	730	4.3×10^5

Values of settlement per cycle versus cyclic shear strain corresponding to different values of prior total vertical settlement from the above table are plotted as open circles in Fig. 5. Similar results from other tests with different initial conditions, but for the same relative density of test specimens, are also plotted in Fig. 5 and contours have been drawn through points of equal total settlement. Thus the settlement per cycle can be determined for any level of cyclic shear strain on the first cycle of loading (shown as total settlement ε_r equal to zero) and for the next cycle of loading following accumulation of various total settlements. These results further confirm the findings of Silver and Seed (1971b) and Youd (1972) that settlement is a function of cyclic shear strain and the previous strain history, but is independent of the vertical stress. In the previous studies the previous strain history was indicated simply by the number of cycles but the total settlement is found to be a more convenient indicator of the previous strain history because it combines in a single parameter the effects of both the number of load cycles and their amplitudes.

The data for shear modulus obtained in these tests can be reduced and plotted in a similar manner as shown in Fig. 4, Table 1 and Fig. 6. In the latter figure the equivalent linear shear modulus is plotted as a function of cyclic shear strain. Again the test results from Fig. 4 are shown as open circles and the path of each test is shown by a dashed line with the total settlement being used as an indicator of previous strain history. As is well known, the shear modulus decreases with increasing cyclic shear strain. For dry sand, the shear modulus also appears to increase with previous cyclic strain history. The results are illustrated for one vertical stress only. The full results showed that the shear modulus varied approximately with the square root of the mean confining stress although this was less true at higher strain levels and lower confining pressures (Pyke, 1973).

Data on the one-dimensional loading and unloading stress-strain relationship for Monterey No. 0 sand were also obtained by cycling the vertical load once before the cyclic shear stresses were applied, and by observing the vertical strain on unloading at the end of the cyclic load test. A typical result is shown in Fig. 7. The rebound on unloading following cyclic shear stress applications was always somewhat greater than that observed in a single cycle of vertical load application, but the difference in results was not very large during the first two-thirds of the unloading process. Thus for many analytical purposes, the virgin unloading curve may be considered to provide an adequate approximation to the behavior of the soil under cyclic loading conditions.

It is of interest to note that similar data were also extracted from the shaking table tests which have been referred to previously. Good agreement was found between the shear moduli measured in the two types of test. Good agreement was also found between the settlement on the first cycle in the two types of test but the settlement per cycle appeared to be greater for later cycles in the shaking table tests. It may be that the wire-reinforced rubber membrane which contains the sample in the simple shear device restricted the settlement of the cap in these tests. The presence of this membrane may also limit the accuracy of the one-dimensional stress-strain relationships. However, it is believed that the general nature of the results is correct.

The above results demonstrate that the behavior of sands at the relatively small strain levels which are normally associated with cyclic loading is sensitive to previous strain history. The results are also sensitive to the method of sample preparation (Pyke, 1973) and the results reported here are for samples prepared by raining dry sand at a controlled rate of deposition. These aspects of the behavior of dry sands also appear to be significant in the evaluation of the liquefaction characteristics of saturated sands.

Use of Test Data to Compute Pore Pressure Generation and Development of Initial Liquefaction in Cyclic Load Tests on Saturated Samples

1. 1-Directional Shaking

The data obtained on dry Monterey No. 0 sand may now be used to predict the undrained behavior of this sand by using the model of soil behavior described previously. The computational procedures can best be illustrated by an example which can be worked manually. For this purpose it is helpful to present the three required sets of data on separate plots:

(1) Data on Settlement Per Strain Cycle

The settlement data from Fig. 5 is reproduced in Fig. 8 with additional contours of total settlement interpolated.

(2) Data on Shear Modulus of Sand

Because the data for shear modulus become too unweildy for hand calculation if the effects of previous strain history are included, the shear modulus data for the sand is reproduced in Fig. 9 for several vertical stress levels and for the fifth cycle of loading in the cyclic simple shear tests. It may be noted, as found in many previous investigations, that the shear modulus increases with increasing confining pressure but decreases with increasing amplitude of cyclic shear strain.

(3) Rebound Curve on Unloading

The unloading portion of the stress-strain curve from Fig. 6 is replotted in Fig. 10, showing the vertical strains that correspond to decreases in the vertical stress from the maximum value. Again, only the virgin unloading curve is shown and no attempt has been made to take account of the effects of cyclic loading, since the two curves are essentially parallel during the first two-thirds of the unloading process.

Suppose now it is desired to determine the rate of development of pore water pressures and initial liquefaction under undrained conditions in a stress-controlled cyclic load test involving the application of cyclic shear stresses of 200 psf to an element of Monterey No. 0 sand consolidated to 60 percent relative density under a vertical stress of 1600 psf. The corresponding value of the cyclic stress ratio, τ_h/σ'_v , is 0.125.

The computation begins by entering the data shown in Fig. 9 to obtain the cyclic shear strain caused by a cyclic shear stress of 200 psf when the effective vertical stress is 1600 psf. This is most easily done by determining the locus of all points which have a shear stress of 200 psf (shown by the broken line in Fig. 9) and then locating the intercept of this locus with the modulus curve for the required vertical stress. For the designated conditions, this is shown by the point marked 1 in Fig. 9, and the strain developed may be read off directly as 0.027 percent. At this stage the total settlement will be equal to zero and the settlement during the first cycle corresponding to the developed strain of 0.027 percent may be read off directly for these conditions from the data presented in Fig. 8. The resulting settlement per cycle, designated by the point 1 in Fig. 8, may be seen to be 0.008 percent. This value becomes the total settlement after the first cycle and the decrease in vertical effective stress, (which will be equal to the increase in pore water pressure during the first cycle) required to produce a rebound of 0.008 percent may be read from Fig. 10. The necessary reduction, marked by point 1 in Fig. 10, may be seen to be 350 psf. Note that by plotting Fig. 10 in this manner it is not actually necessary to determine the tangent modulus and perform the multiplication indicated in equation (1).

The series of computations described above are listed in the first row of the results shown in Table 2. At the end of the first cycle, the computed

Table 2. Example of Computation of Pore Pressure Development During One-Directional Shaking

Cycle Number (1)	Effective Vertical Stress, -psf (2)	Cyclic Shear Strain, -percent (3)	Settlement per Cycle, -percent (4)	Total Settlement, -percent (5)	Pore Water Pressure, u -psf (6)	$\frac{u}{\sigma'_v}$ (7)
1	1600	0.027	0.008	0.008	350	0.22
2	1250	0.035	0.01	0.018	630	0.39
3	970	0.044	0.013	0.031	850	0.53
4	750	0.063	0.018	0.049	1080	0.68
5	520	0.14	0.043	0.092	1400	0.88
6	200	>0.50	>0.052	>0.144	1600	1.00

value of pore water pressure is 350 psf and the corresponding value of the effective vertical stress is 1250 psf. The same procedure is then repeated for additional cycles. The results obtained for each subsequent cycle are indicated by points marked 2, 3, 4 and 5 in Figs. 8, 9 and 10 and tabulated in Table 2. It may be seen that initial liquefaction, that is, the condition for which the excess pore water pressure becomes equal to the initial effective stress, occurred on the sixth cycle.

2. <u>2-directional shaking</u>

The computation illustrated above may readily be modified to predict the probable behavior of sand under two or multi-directional shaking for undrained conditions. In the shaking table tests on dry sand it was found that the effect of a second component of shaking, equal to the first, was to approximately double the settlements caused by a stress-controlled loading, see Fig. 2. While there are some small differences in the effects of multi-directional shaking on both settlement per cycle and soil moduli as compared with one component acting alone (Pyke, 1973), the effect of multi-directional shaking can be represented simply by doubling the settlement per cycle which is measured under one-directional shaking conditions. The computations of the rate of development of initial liquefaction, using the same stress conditions as for the example in Table 2, but for two-directional shaking producing twice the settlements per cycle indicated in Fig. 8, are presented in Table 3. It may be seen that for these conditions initial liquefaction develops after only about 3 - 1/2 cycles. The rates of development of pore pressures for the test conditions represented by Tables 2 and 3 are plotted in Fig. 11.

Table 3. Example Computation Multi-Directional Shaking

Cycle Number (1)	Effective Vertical Stress, -psf (2)	Cyclic Shear Strain, -percent (3)	Settlement per Cycle, -percent (4)	Total Settlement, -percent (5)	Pore Water Pressure, u -psf (6)	$\frac{u}{\sigma'_v}$ (7)
1	1600	0.027	0.016	0.016	575	0.36
2	1025	0.042	0.025	0.041	980	0.61
3	620	0.105	0.064	0.105	1475	0.91
4	125	>0.50	>0.039	>0.144	1600	1.00

In order to compare the effects of 1 and 2 directional shaking on the development of pore water pressures and initial liquefaction over a wide range of cyclic stress conditions, computations similar to those described above have been made for conditions representing 1-directional and multi-directional shaking over a range of values of the stress ratio, τ_h/σ'_v . A short computer program was written to expedite the work and to allow inclusion of the effect of strain history on the shear modulus. Analytical functions were fitted to the data for settlement per cycle, shear modulus and the modulus on unloading in the manner suggested by Martin, Finn and Seed (1975).

The results for two simulated tests with the same stress ratio, τ_h/σ'_v , but with the settlement per cycle doubled in the second test, are shown in Fig. 12. Both the excess pore pressure and the cyclic shear strain are plotted as a function of the number of cycles of loading. It may be seen in both Figs. 11 and 12 that the excess pore pressures increase approximately twice as fast for the test condition representative of multi-directional shaking and that initial liquefaction occurs in about half the number of cycles that are required for one-directional shaking.

In Fig. 13 these results plus additional points are plotted in the conventional manner showing the number of cycles required to cause initial liquefaction as a function of the stress-ratio, τ_h/σ'_v . The results of the computations shown in Tables 2 and 3 are shown by the triangular points in this figure. Again, for a given stress-ratio, the number of cycles required to cause initial liquefaction under multi-directional shaking is about half that under one-dimensional shaking; however, because the relationship between shear stress and number of cycles to liquefaction is non-linear, there is only a small difference between the stress ratios causing liquefaction in a given number of cycles. In the range from 5 to 20 cycles, the shear stresses causing liquefaction under multi-directional shaking are in fact only 10 to 20 percent less than those under one-directional shaking.

It is also interesting to note that the shear stress ratios causing liquefaction under one-directional shaking computed by this procedure are somewhat less than those determined experimentally by DeAlba, Seed and Chan (1975) in shaking table tests on saturated samples of Monterey No. 0 sand. It may be that closer agreement could be obtained by making refinements to the computational model used, particularly with regard to the modulus on unloading; however, the general form of the results is very similar to that obtained experimentally. Thus it seems reasonable to believe that the model can be used with a high degree of confidence for evaluating effects such as that studied herein.

On the basis of the results shown in Fig. 13, it would be possible to account for the effect of multi-directional shaking in the analyses of liquefaction potential either by adjusting the stress ratio that causes liquefaction in a given number of cycles or by altering the number of uniform cycles that is taken to be equivalent to the irregular loading in the field. A procedure for obtaining the equivalent number of uniform cycles for individual components of motion and typical values for this number have been given by Seed et al, 1975. The effect of multi-directional shaking could be taken into account by adding together the equivalent number of uniform cycles obtained for each component; however, this would only be an approximation, as the effect of two motions applied successively is not the same as the effect of those motions applied concurrently. A generally simpler and sufficiently accurate approach appears to be to conduct analyses considering only one component of motion and to apply a small reduction to the stresses estimated to cause liquefaction on the basis of laboratory tests in order to account for the presence in the field of a second horizontal component.

Conclusions

Both qualitative use of the results of shaking table tests on dry sand and the results of a quantitative evaluation using data from cyclic simple shear tests indicate that the shear stresses causing liquefaction under multi-directional shaking with two equal components are 10 to 20 percent less than the shear stresses causing liquefaction under one-directional shaking. Since in practice it is unlikely that a second component of motion would be equal to the single component used for design purposes, it is suggested that a reduction of 10 percent in the shear stresses causing liquefaction is a suitable general procedure for accounting for the effects of multi-directional shaking. Combining this factor with the correction factor which should be applied to cyclic triaxial test results in order to obtain the shear stresses causing liquefaction under simple shear conditions with uni-directional shaking (DeAlba et al, 1975) an overall correction factor of about 0.57 is obtained on the basis of studies conducted on normally consolidated samples of Monterey No. 0 sand.

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Appendix 1 - Notation

The following symbols are used in this paper:

c_r correction facotr from cyclic triaxial test to field conditions

τ_h maximum horizontal cyclic shear stress

σ_{dc} cyclic deviator stress

σ'_{vc} vertical consolidation stress

σ'_a isotropic consolidation stress

σ'_v vertical effective stress

D_r relative density

ε_{rd} vertical strain or settlement under drained conditions

E_r tangent modulus in one-dimensional unloading

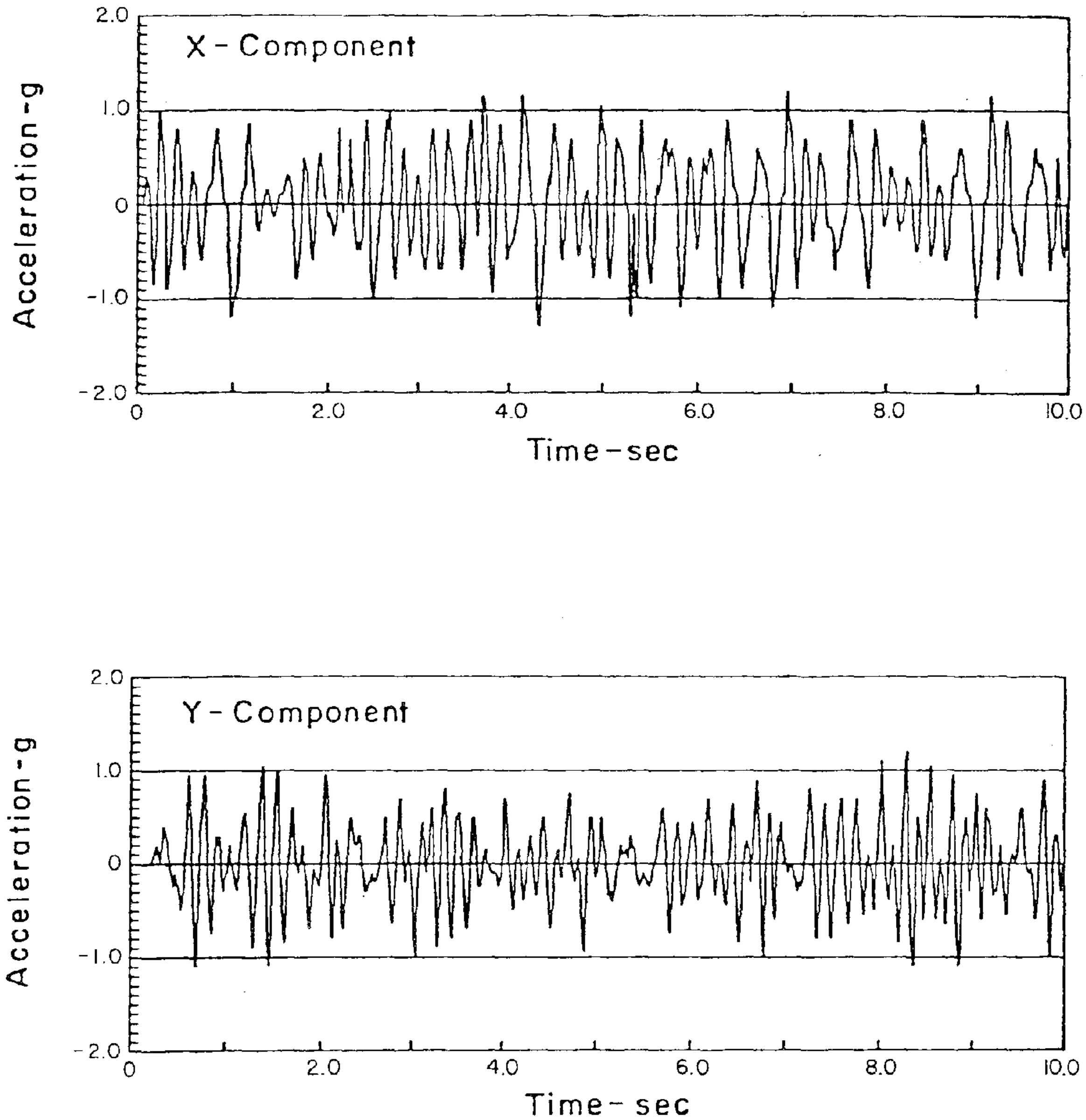

Fig. 1 RANDOM ACCELERATION HISTORIES

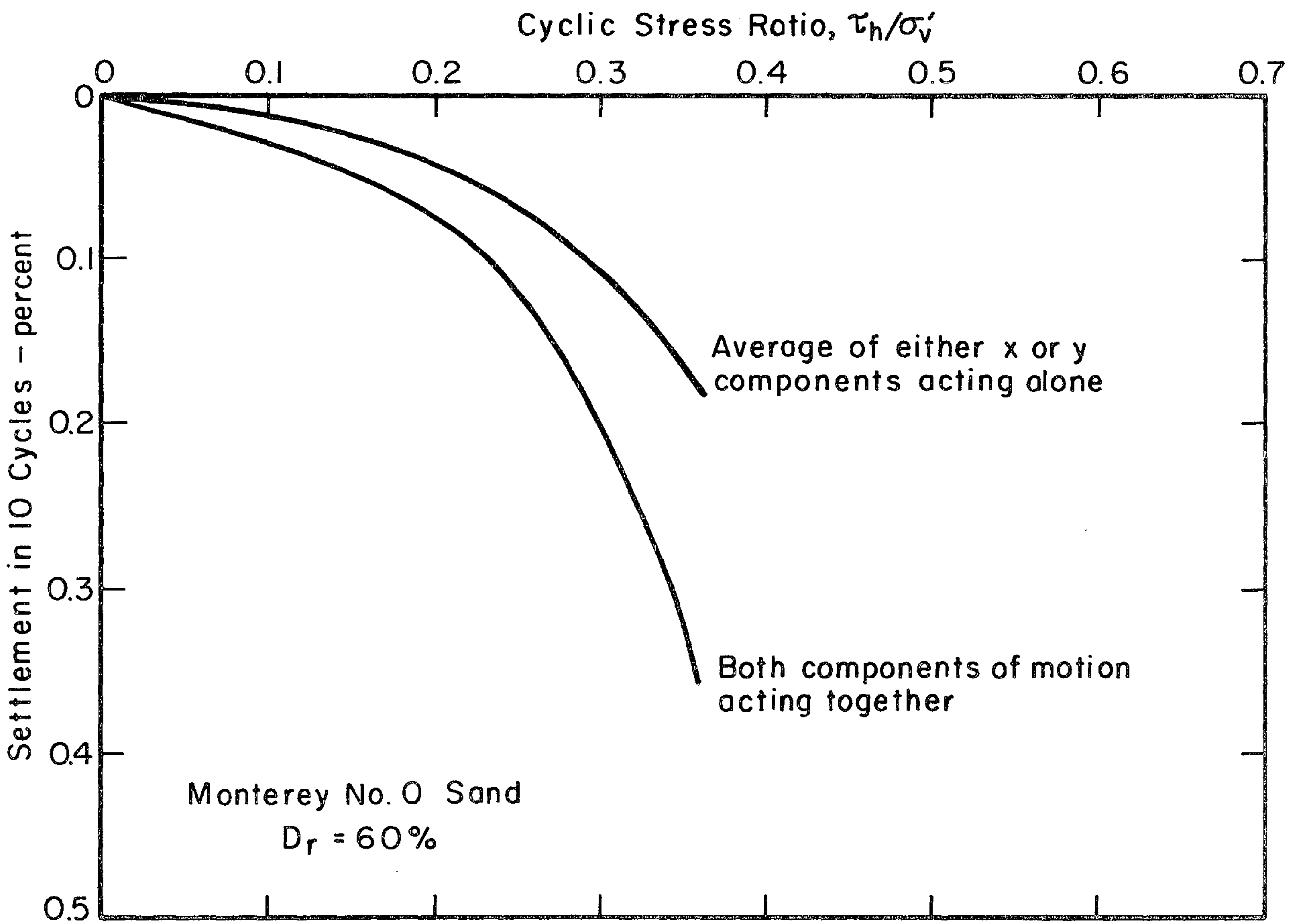

Fig. 2 SETTLEMENT OF DRY SAND UNDER UNIDIRECTIONAL AND MULTIDERECTIONAL SHAKING

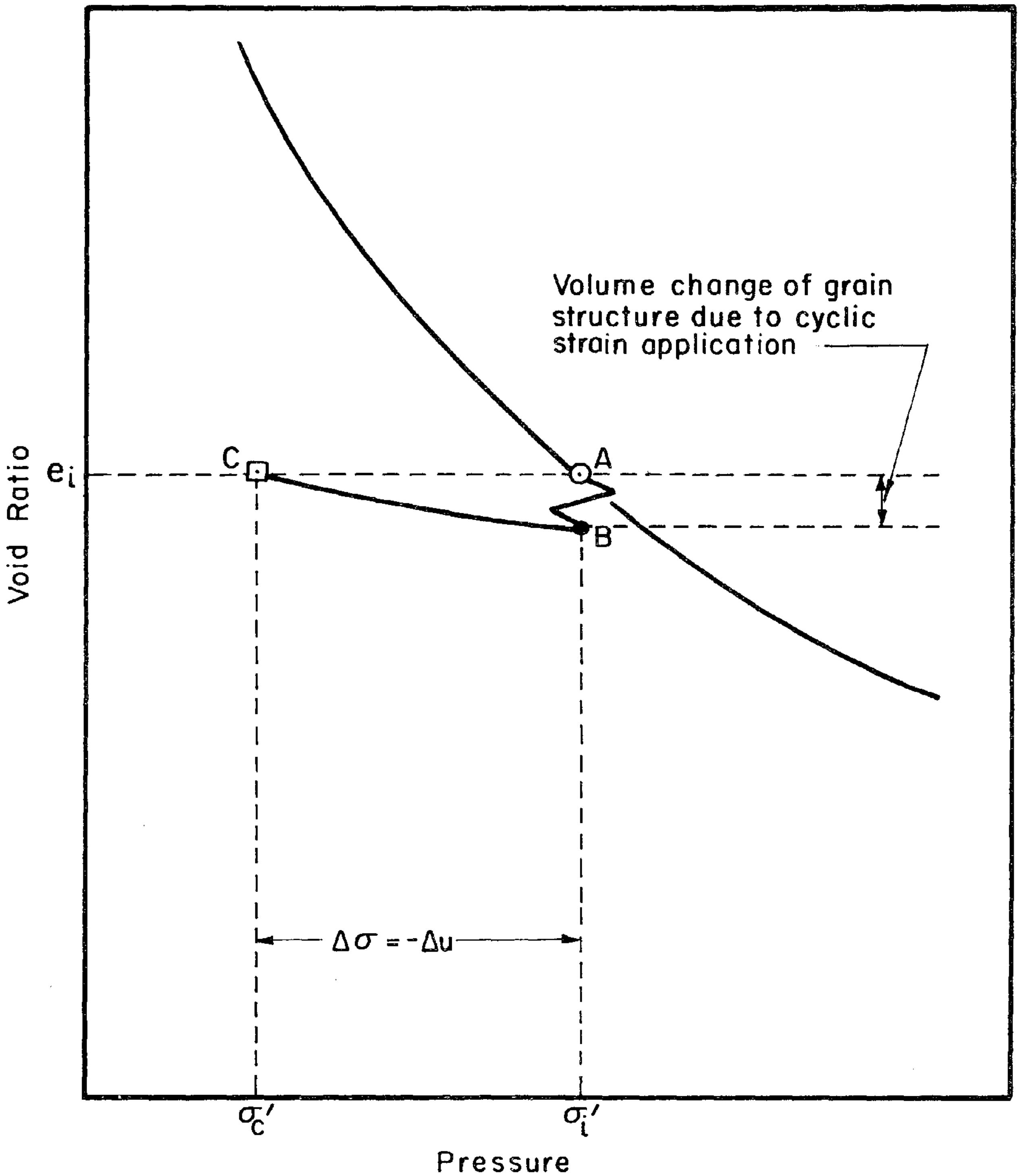

Fig. 3 SCHEMATIC ILLUSTRATION OF MECHANISM OF PORE PRESSURE GENERATION DURING CYCLIC LOADING

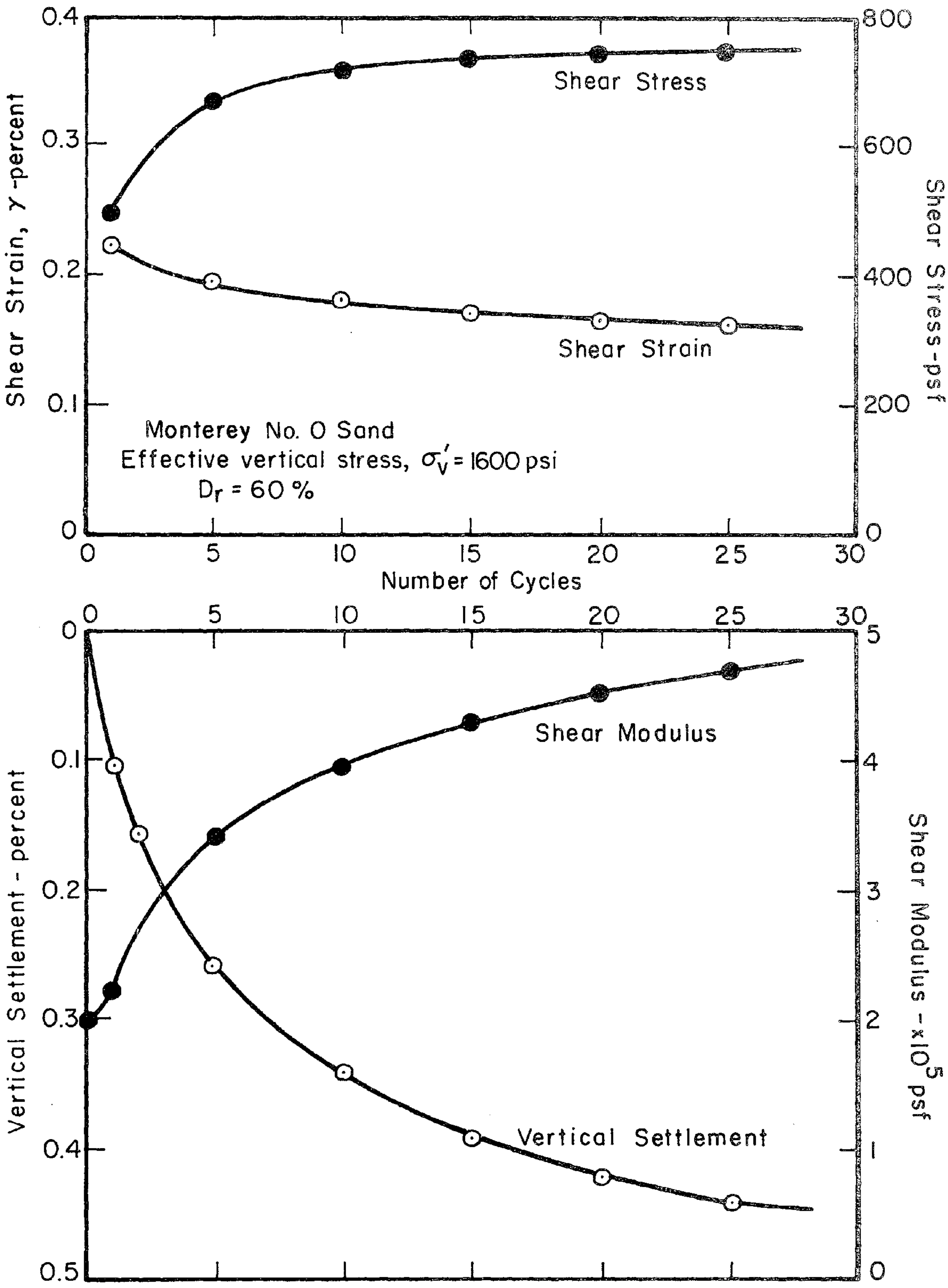

Fig. 4 TYPICAL RESULTS OF CYCLIC SIMPLE SHEAR TEST

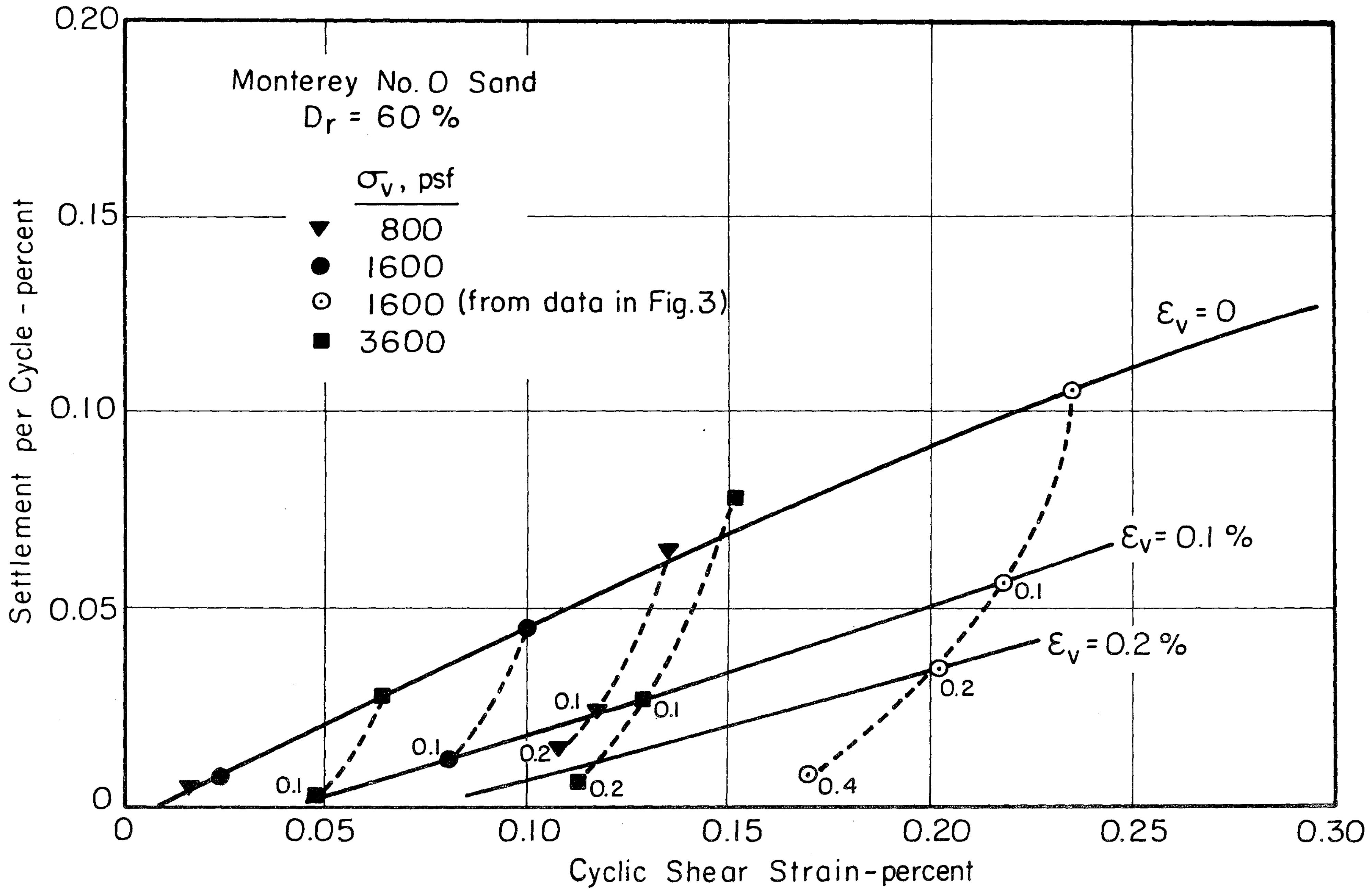

Fig. 5 SETTLEMENT DATA FROM CYCLIC SIMPLE SHEAR TESTS

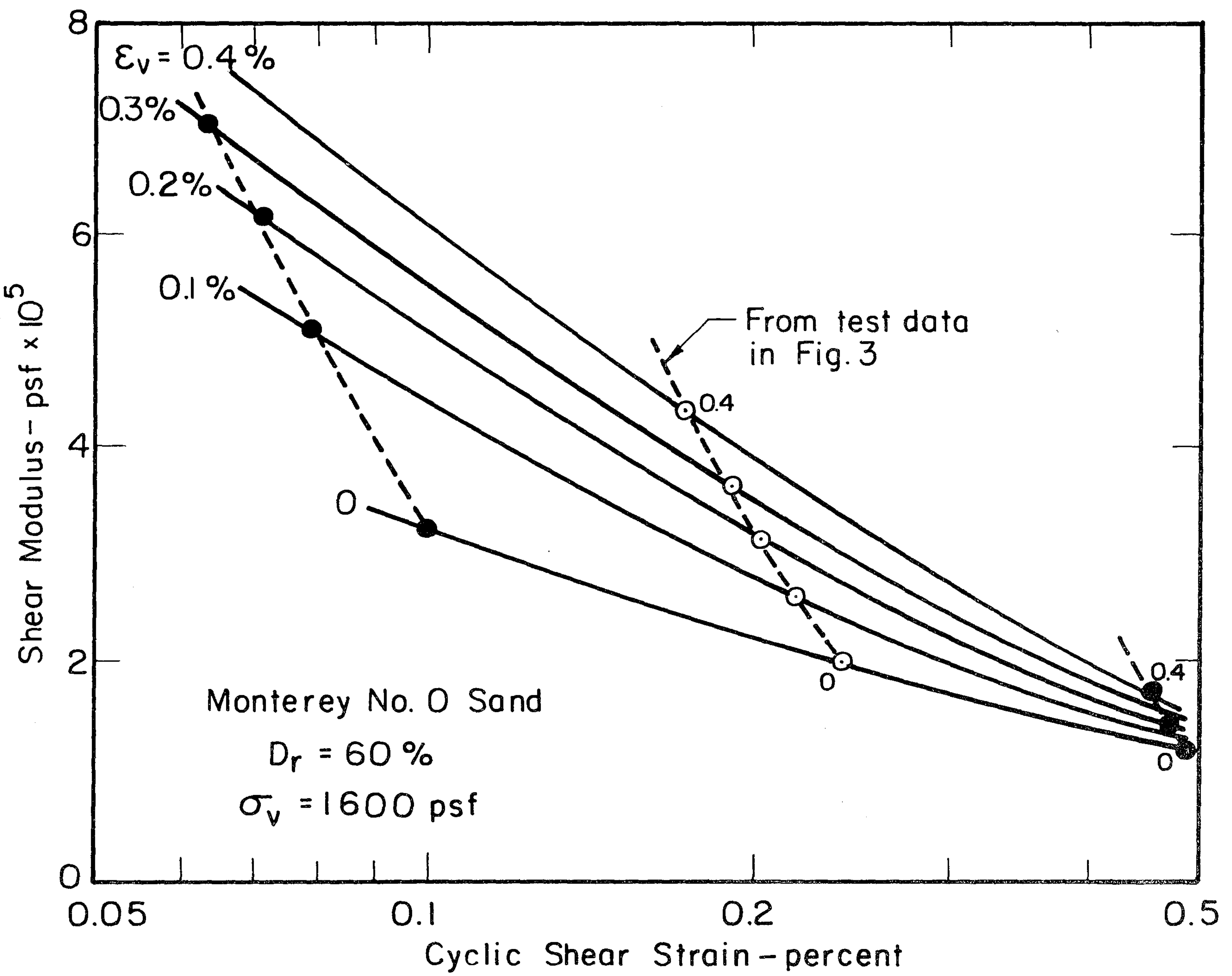

Fig. 6 SHEAR MODULUS DATA FROM CYCLIC SIMPLE SHEAR TESTS

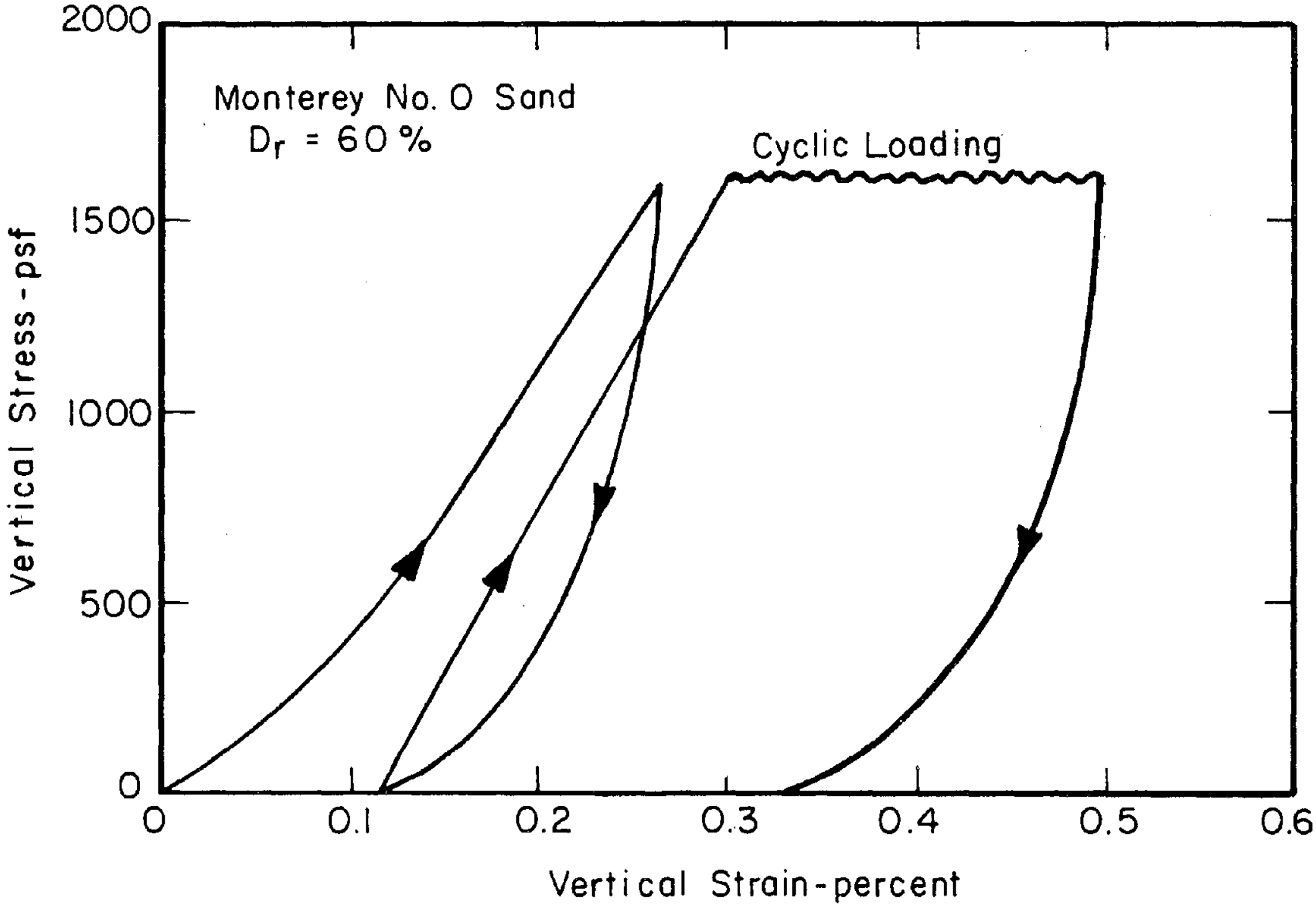

Fig. 7 ONE-DIMENSIONAL LOADING AND UNLOADING DATA OBTAINED IN SIMPLE SHEAR APPARATUS

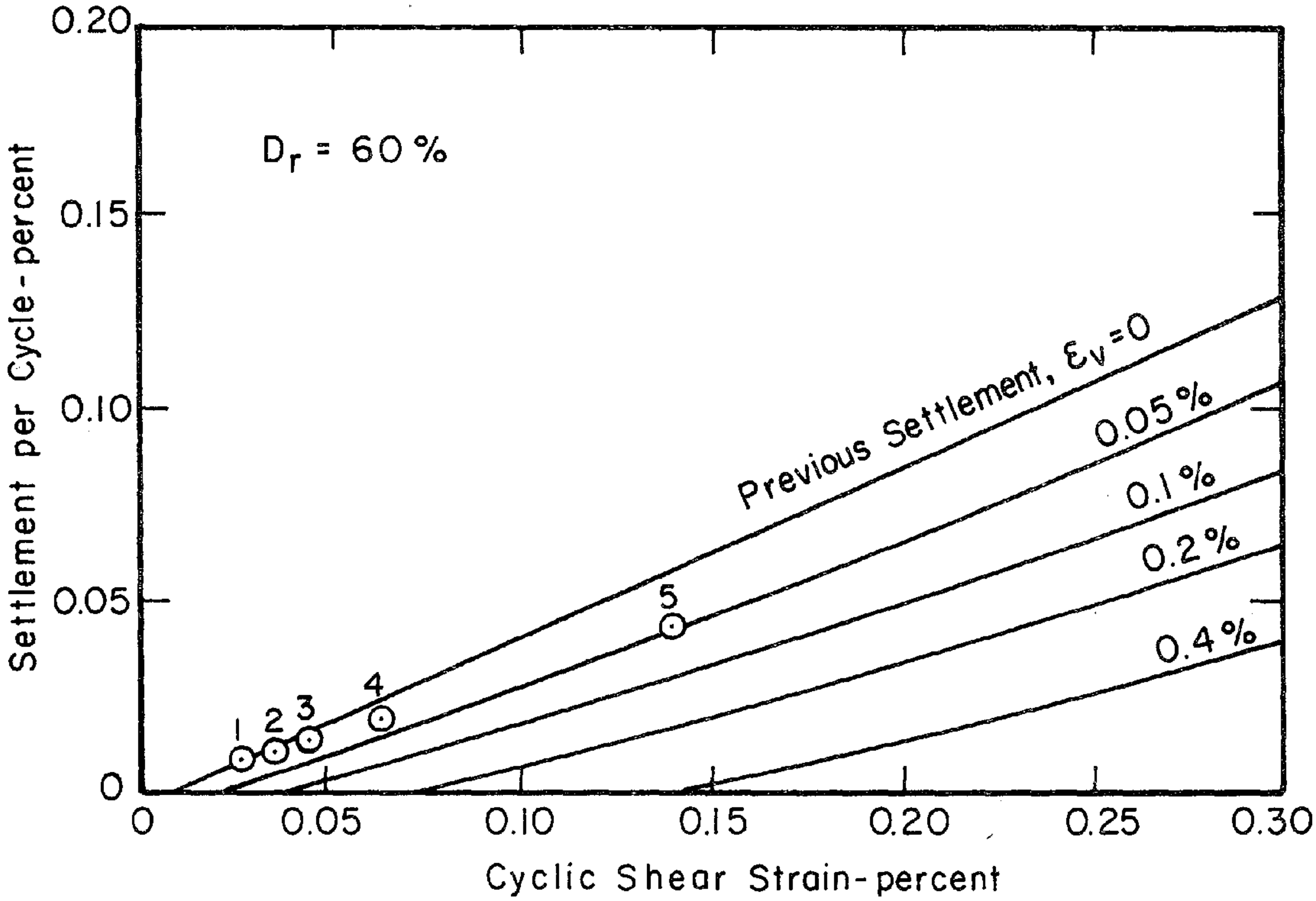

Fig. 8 SETTLEMENT DATA FOR MONTEREY NO. 0 SAND

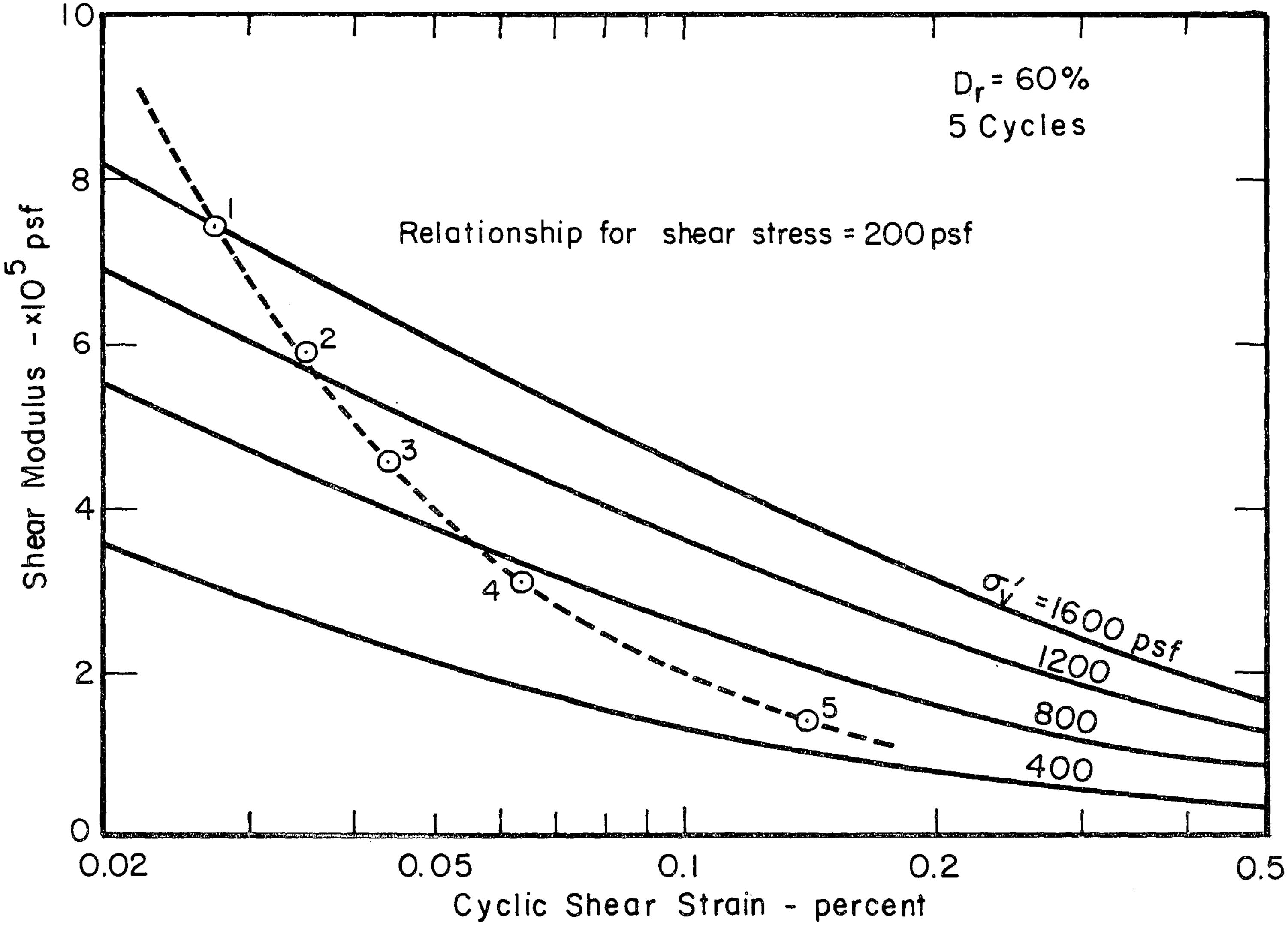

Fig. 9 RELATIONSHIP BETWEEN SHEAR MODULUS AND SHEAR STRAIN FOR MONTEREY NO. 0 SAND

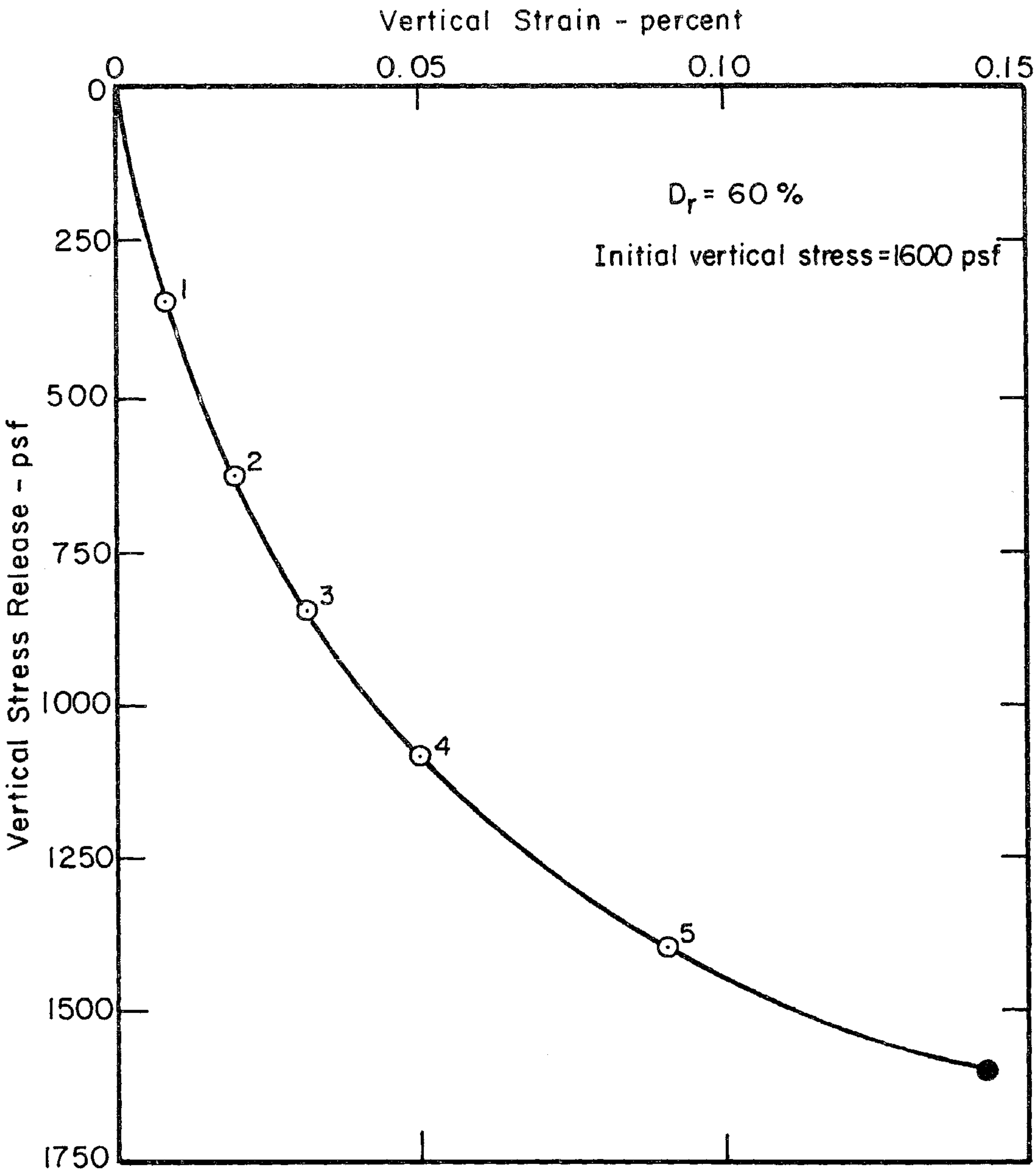

Fig. 10 STRESS – STRAIN RELATIONSHIP FOR ONE-DIMENSIONAL UNLOADING OF MONTEREY NO. 0 SAND

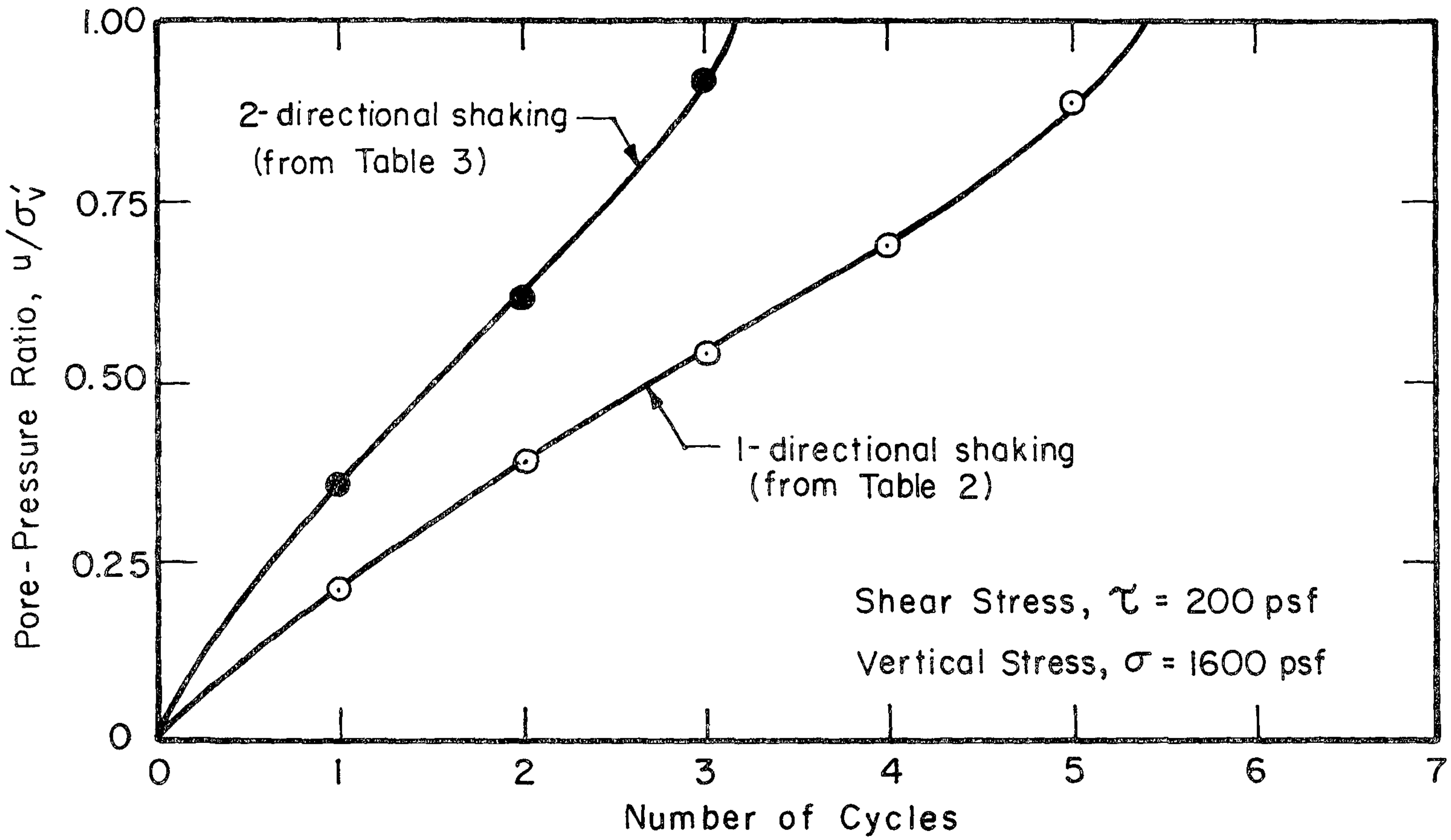

Fig. 11 COMPUTED RELATIONSHIPS BETWEEN PORE-PRESSURE BUILD-UP AND NUMBER OF STRESS CYCLES

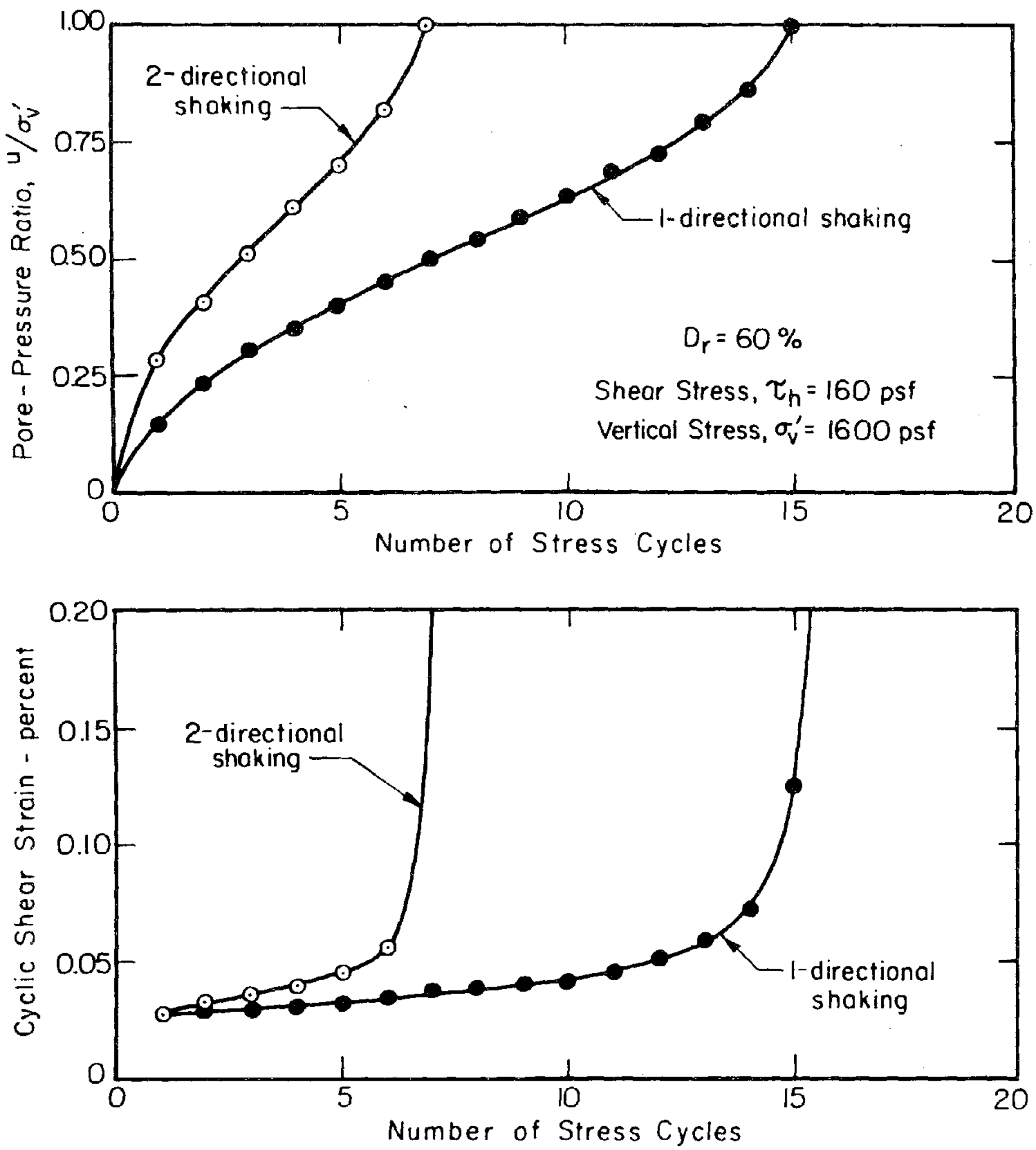

Fig. 12 COMPARISON OF COMPUTED RATES OF DEVELOPMENT OF INITIAL LIQUEFACTION FOR MONTEREY NO. 0 SAND

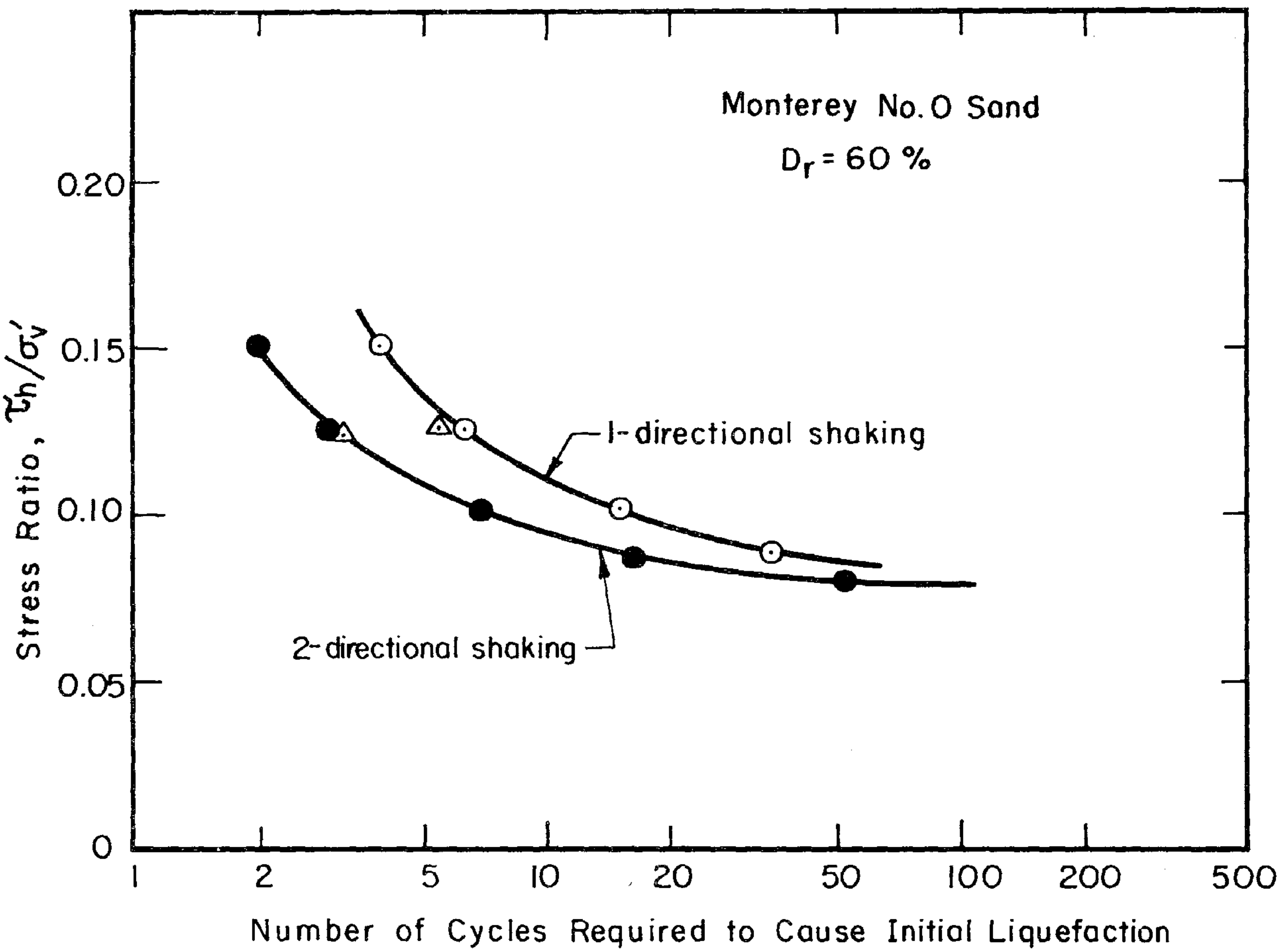

Fig. 13 COMPARISON OF STRESSES CAUSING INITIAL LIQUEFACTION UNDER ONE AND TWO-DIRECTIONAL SHAKING

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