MODULUS REDUCTION DYNAMIC ANALYSIS

by

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B.A.Sc., University Of British Columbia, 1982

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

Civil Engineering

We accept this thesis as conforming

to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

July 1985

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ABSTRACT

A semi-analytical method of dynamic analysis, capable of predicting both the magnitude and pattern of earthquake induced deformations, is presented. The analysis is based on a modulus reduction approach which uses a reduced modulus to simulate the softening induced in soils during cyclic loading. The effects of the inertia forces developed during dynamic loading on the induced deformations are also included through an appropriate selection of the reduced modulus.

The reduced modulus is utilized in a static stress-strain analysis to predict the magnitude and pattern of the deformations induced during earthquake loading. The appropriate modulus reduction is determined from laboratory tests on undisturbed soil samples. Three methods of computing a suitable post-cyclic modulus were investigated but only the cyclic strain approach, in which the modulus is determined from cyclic loading tests that duplicate the field stress conditions, yields reductions of sufficient magnitude to provide realistic estimates of earthquake induced deformations.

The modulus reduction analysis was used to predict the deformations occurring during dynamic loading of a model tailings slope in a laboratory shaking table test and of the Upper San Fernando Dam during the earthquake of February, 1971. These studies showed that the modulus reduction analysis is capable of reproducing the dynamically induced deformations and that reductions in the modulus of up to 1000 times may be

required. Unfortunately, limitations of the testing equipment and inadequacies in the available data required that the appropriate modulus reductions could not be determined entirely through laboratory and field investigations. Some assumptions were necessary in selecting the reduced modulus values used in the analyses. Although these case studies were, hence, unable to provide full verification of the proposed method, they do demonstrate the reliability and simplicity of the analysis as a method of assessing the performance of soil structures during earthquake loading.

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ACKNOWLEDGEMENTS

I would like to thank Dr. Peter M. Byrne for his guidance in initiating this study and for making critical contributions towards its completion. Dr. Yogi Vaid and Edwin Chung also require thanks for their direction and help in performing the laboratory soil tests.

The financial assistance provided by a Natural Sciences and Engineering Research Council of Canada postgraduate scholarship is gratefully appreciated.

CHAPTER 1

INTRODUCTION

The variety of methods used to assess the dynamic response of soil structures to earthquake loading has increased substantially in recent years. However, many of these current methods either make so many simplifying assumptions that they do not accurately or realistically predict soil behavior under dynamic loading or are so complex and costly that their use may be limited to critical structures. There appears to be a need for a relatively simple method for assessing earthquake performance that is based on actual soil behavior and is capable of predicting earthquake induced deformations of the correct magnitude and pattern. The proposed modulus reduction approach to dynamic analysis is presented as such a method.

From their rather rudimentary beginnings in the pseudo-static analysis, dynamic response analyses have increasingly tried to incorporate more realistic models of soil behavior. The "equivalent" permanent force used in the pseudo-static analysis was replaced by a temporary force in Newmark's analysis which recognized the transient nature of earthquake induced loadings. The non-linear strain dependent behavior of soils was initially accounted for in total stress finite element analyses by incorporating hyperbolic or other similar stress-strain models of soil behavior. The importance of pore pressure development and effective stresses on soil response during cyclic loading was then recognized and dealt with, first by Seed in his dynamic stress path approach and later by many others in various rigorous non-linear effective stress dynamic analyses. These recent methods are extremely complex, often requiring parameters not commonly used in geotechnical practice, and generally lacking sufficient verification to justify their use.

The modulus reduction method of dynamic analysis is presented as a simple and realistic model for predicting earthquake induced deformations. Although the reduced modulus is primarily intended to simulate the softening of a soil during dynamic excitation, it can also account for the effects of the inertia forces on the deformations that develop during cyclic loading. Because the stiffness degradation that results from the progressive rise in pore pressure during cyclic loading is represented by the reduced modulus, this method is especially suited for the analysis of loose to medium dense soils which may develop significant pore pressures in response to dynamic excitation. However, since the effects of the inertia forces created during cyclic loading are also included by an appropriate selection of the reduced modulus, the proposed method may be used to predict the deformations resulting from cyclic loading of soils that are expected to experience only limited pore pressure changes during cyclic shear.

The basic approach of the proposed modulus reduction analysis is similar to Seed's stress path method except that a reduced modulus is utilized in conjunction with a static stress-strain analysis to predict earthquake induced deformations.

This thesis explains the theory behind the development of the modulus reduction method and determines its validity on the basis of two case histories: a laboratory shaking table study intended to model the response of saturated tailings to earthquake loading, and the Upper San Fernando Dam which experienced substantial downstream movements during the earthquake of February, 1971.

CHAPTER 2

EFFECTS OF CYCLIC LOADING ON SOIL

An earthquake or any other dynamic loading creating cyclically reversing shear stresses in a soil structure basically has two effects: it generates transient inertia forces and may cause the development of excess pore pressures. Although both of these conditions will cause earthquake induced deformations, most of the severe earthquake damage has resulted from deformations associated with pore pressure rise.

The extent of deformations resulting from changes in pore pressures during cyclic loading depends on the magnitude of the pore pressure rise and on the mechanism of pore pressure generation. When undrained conditions prevail during cyclic loading of a saturated soil, pore water pressures will increase because of the tendency of the soil to contract when subjected to cyclic shear. As the pore pressures increase, the soil softens and strains will develop. However, significant strains only occur if the pore pressures increase enough to trigger the onset of either of two distinct phenomena: liquefaction or cyclic mobility.

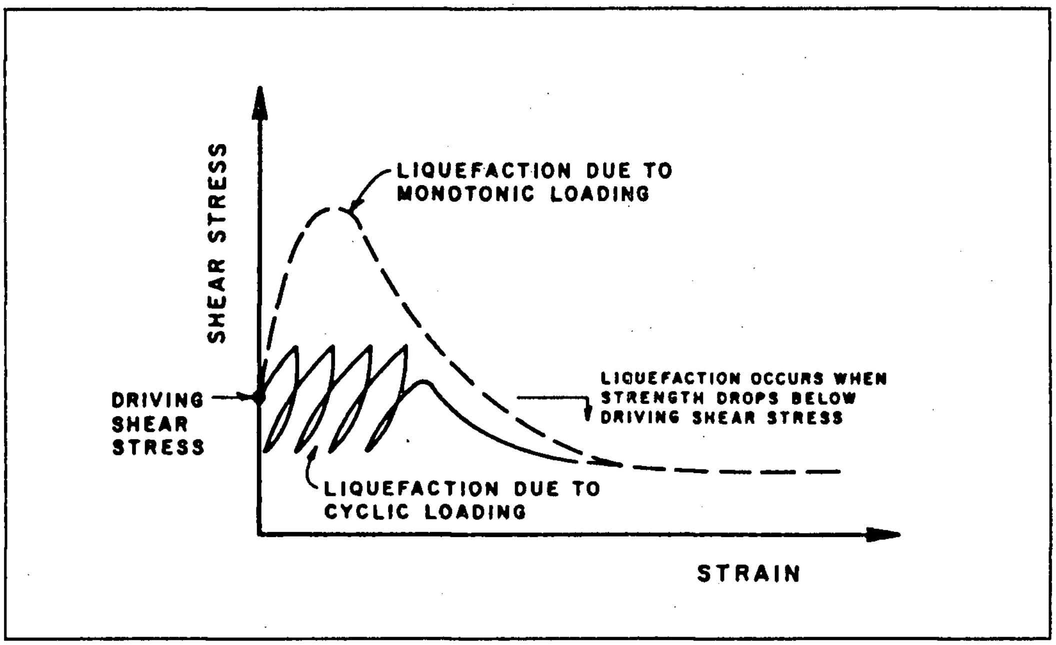
The two terms, "liquefaction" and "cyclic mobility", occur extensively in the literature and unfortunately have been used to describe a variety of phenomena by different investigators. Confusion particularly arises over the term "liquefaction". Although the term has traditionally referred to either a condition of zero effective stress within a soil or to slope failures which resemble the flow of a viscous fluid, it is now being used to describe several phenomena observed during laboratory cyclic loading tests. It has become customary to use the term "liquefaction" for the development of 5 or 10 percent strain in a cyclic load test. "Initial liquefaction" has been described by Seed and Lee (1966) as the stage in a cyclic load test when pore presssures momentarily become equal to the confining stresses. Castro and Poulos (1975), on the other hand, have defined liquefaction as a phenomenon wherein a saturated sand loses a large percentage of its shear resistance and flows in a manner resembling a liquid until the shear stresses acting on the soil mass are as low as the reduced shear resistance. Vaid and Chern (1983), likewise, referred to liquefaction as a contractive flow failure. For the purpose of this thesis, the term "liquefaction" will be reserved to describe the flow failures that result from a loss in strength during cyclic loading.

The term "cyclic mobility" was first introduced by Casagrande in 1965 to refer to the strains that accumulate in undrained cyclic load tests on sand. He described cyclic mobility as a progressive reduction in the stiffness of a saturated soil when subjected to cyclic loading. Unlike liquefaction, the strains associated with cyclic mobility develop progressively during each loading cycle. They may, however, become as significant as those resulting from liquefaction.

The distinction between liquefaction and cyclic mobility is necessary because the mechanism of strain development involved in each is entirely different. In addition, the development of the two phenomena are affected quite differently by the initial state of the soil, as defined by void ratio, confining pressure and static shear stress. To realistically simulate the soil response during cyclic loading and to predict the dynamically induced deformations, the different mechanisms of strain development involved in the two phenomena must be fully understood.

2.1 Liquefaction

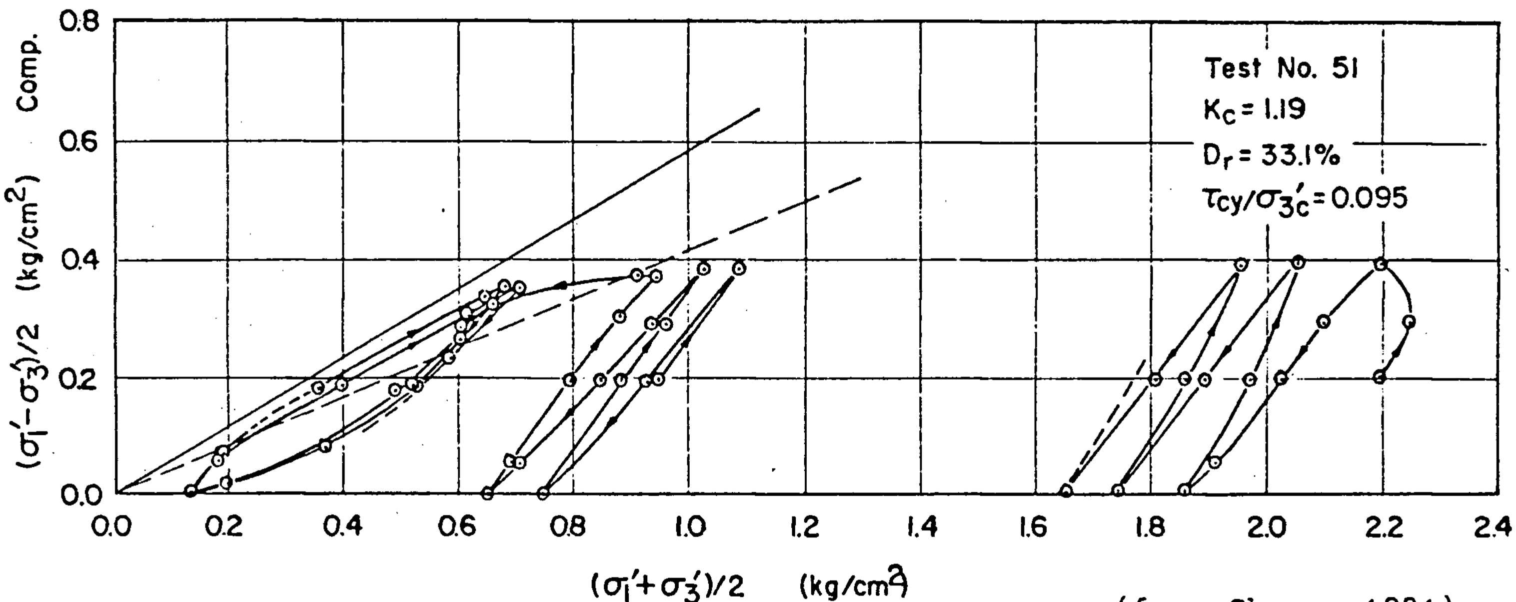
Liquefaction is a flow failure associated with contractive or strain softening soils. It can only occur in the presence of driving shear stresses and if a substantial proportion of the soil's shear strength is lost during undrained monotonic, cyclic or shock loading. The loss in strength results from a conversion of the soil mass from a practically drained condition to a practically undrained condition of shear. For contractive soils, the undrained shear strength may be substantially lower than the drained strength and strain softening occurs during undrained shear. Thus, although the soil mass is able to support the in-situ shear stresses under drained conditions, it fails when a sufficient triggering stress or cyclic loading is applied undrained. Figure 2-1 illustrates schematically the conditions required to initiate liquefaction failures. During liquefaction, the soil flows in a manner resembling a viscous fluid until the shear stresses are as low as the reduced strength. Because flow deformations involve large unidirectional displacements, it has been suggested (Castro and Poulos, 1975, and Poulos et al, 1985) that the reduced strength reached during liquefaction is the undrained steady-state shear strength. This strength is the strength that exists when the soil mass is continuously deforming at a constant volume, constant normal effective stress, constant shear stress and constant velocity. The steady-state strength is a function of the effective stresses reached during failure and is not zero.



<u>Figure 2-1</u>

Liquefaction Due to Monotonic or Cyclic Loading (Schematic) (from Poulos et al, 1985)

In laboratory cyclic load tests, liquefaction is associated with a rapid increase in pore pressures accompanied by a sudden development of axial strains. During cyclic loading, the progressive increase in pore water pressure causes the effective stress state of the soil to move steadily towards the failure envelope. For contractive soils, liquefaction is triggered when the effective stress state of the soil reaches a critical value of the effective stress ratio, σ_1'/σ_3', prior to the failure envelope or the steady-state line. Figure 2-2 illustrates a typical stress path followed during liquefaction. If the steady-state of deformation is reached during flow, the deformations may continue until the limits of the testing equipment are exceeded. Vaid and Chern (1983), however, found that flow deformations were often arrested when the triaxial samples had strained sufficiently to cause dilation on further straining. They referred to this type of behavior as limited liquefaction. The steady-state is not reached during limited liquefaction failures and although strains develop rapidly upon initiation of failure, they stop at the onset of dilation. The strain that develops during such limited flow failures depends on the initial state of the sample and the properties of the soil. Additional cyclic loading, after the arrest of liquefaction, was found (Vaid and Chern, 1983) to cause further straining accompanied by dilation and a reduction in pore pressures during the loading phase of each stress cycle. Significant accumulations of strain were only observed when the applied cyclic stress was of sufficient magnitude to exceed the static shear stress and cause stress reversals to occur. The stress path followed during these additional cycles of loading is shown on Figure 2-2.



(from Chern, 1981)

FIGURE 2–2 Effective Stress Path of Cyclic Loading Test on Anisotropically Consolidated Loose Sand

The magnitude and rate of pore pressure buildup during cyclic loading depends on the magnitude of the cyclic load, the number of cycles applied, the effective confining pressures and the level of static shear stress. The relative magnitudes of the cyclic deviator stress and the static shear stress are also significant. When the static shear stress is greater than the cyclic deviator stress, stress reversals during cyclic loading do not occur. When stress reversals do occur, they significantly increase the rate of pore pressure generation and the magnitude of the excess pore pressures reached during a specified number of cycles of loading. The magnitude of the pore pressures reached during liquefaction depends on the effective confining stresses and on the level of static shear stress in the soil. Chern (1981) showed that the residual value of the excess pore pressures, Ur, developed during liquefaction after the termination of cyclic loading is given by

$$Ur = \sigma_{3c}' \left(1 - \frac{(Kc - 1)}{2} \left(\frac{1 - \sin \phi'}{\sin \phi'}\right)\right)$$

Only in isotropically consolidated samples will the residual pore pressure become equal to the effective confining pressure. In the presence of static shear stresses, the residual pore pressure is always less than the effective confining pressure. However, the transient value of the pore pressure generated during cyclic loading will fluctuate about a mean value and will exceed the residual value periodically during cyclic loading as soon as liquefaction has occurred. Thus, the maximum value of the pore pressure during cyclic loading will be higher than the residual value given by Chern.

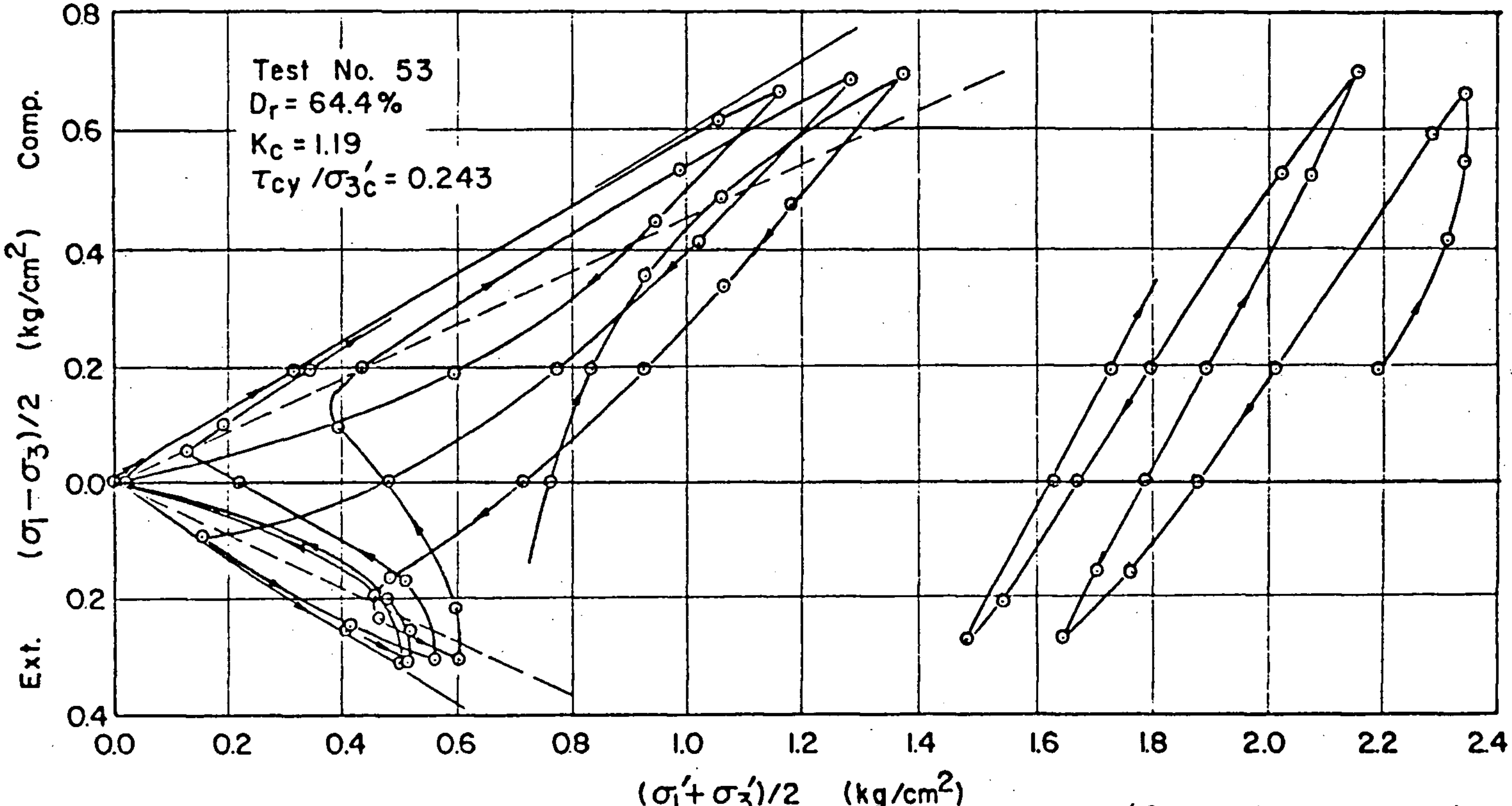
The susceptibility of a given soil at a given relative density to liquefaction is affected by the consolidation stress, σ_{3c}', and the initial static shear stress, τ_s. Higher effective consolidation pressures correspond to greater susceptibilities to liquefaction. For a fixed relative density, a soil will exhibit contractive behavior more readily at higher confining pressures. Thus, a soil that may be susceptible to liquefaction at high levels of confining stress will not be susceptible at lower confining stresses.

The influence of static shear on the liquefaction potential of loose sand varies with the level of static shear. At relatively low levels of static shear, the resistance to liquefaction, defined by the level of cyclic shear stress required to cause flow deformation in a fixed number of cycles of loading, generally increases as the static shear stress increases. This increase is apparently related to the reduced rate of pore pressure generation caused by a reduction in the magnitude of the shear stress reversals. For higher levels of static shear, however, increases in the static shear stress may be accompanied by substantial reductions in the resistance to liquefaction. The reduction in the resistance to liquefaction at high static shear stress levels has been attributed (Vaid and Chern, 1983) to the fact that the initial stress state of the sample is so close to the critical effective stress ratio line that only a small cyclic deviator stress or only a few cycles of loading are required for the stress state to reach the line and liquefaction to occur.

2.2 Cyclic Mobility

Cyclic mobility refers to the formation of large strains during undrained cyclic loading tests on sand. Unlike liquefaction, it is characterized by a gradual development of pore pressures accompanied by a progressive accumulation of strains during each cycle of loading. No sudden increase in the pore pressures or axial strains occurs. A loss in strength is not required to initiate the onset of cyclic mobility. Although cyclic mobility generally occurs in medium dense to dense sands which exhibit dilative rather than contractive behavior, it may also develop in loose sands after the arrest of liquefaction when sufficient strains have developed to cause dilation on further straining.

Like liquefaction, cyclic mobility is initiated when the effective stress state of the soil reaches a critical value of the effective stress ratio. The critical value of the effective stress ratio for cyclic mobility is higher than that for liquefaction. Vaid and Chern (1983) showed that the effective stress ratios defining the onset of significant cyclic mobility in dense sands and the arrest of flow failures in loose sands coincide. Figure 2-3 shows a typical effective stress path followed during cyclic mobility. When the stress state passes the critical effective stress ratio line the soil experiences a



(from Chern, 1981)

FIGURE 2–3 Effective Stress Path of Cyclic Loading Test on Anisotropically Consolidated Dense Sand significant increase in the strain developed during a single loading cycle. Upon unloading, pore pressures rise substantially causing the minor effective stress to momentarily go to zero. However, little change in the axial strain occurs. Subsequent reloading results in the development of additional strains accompanied by dilation and the reduction of pore pressures. Repetition of cyclic loads, each causing additional strains, results in the large accumulation of strains associated with cyclic mobility.

The pore pressures that develop during cyclic mobility fluctuate around the residual pore pressure value defined by Chern. This residual pressure is determined by the level of static shear stress and is independent of whether it results from cyclic mobility or liquefaction (Vaid and Chern, 1983). During cyclic mobility, the pore pressures transiently exceed the residual value during each load cycle. The transient pore pressures may become equal to the confining pressures momentarily when the cyclic deviator stress is zero. When loading continues, the soil tends to dilate and the pore pressures reduce.

The susceptibility of a soil to cyclic mobility is influenced by the effective confining pressures and by the level of static shear. Although increased confining pressures generally increase the cyclic load necessary to cause cyclic mobility, the cyclic mobility ratio, defined as the cyclic deviator stress divided by twice the effective minor principal stress at the start of cyclic loading, usually decreases with increasing confining stress. Thus, the resistance to cyclic mobility generally decreases with increasing confining pressure. This same trend with confining pressure was observed for liquefaction.

Like the resistance to liquefaction, the resistance of a soil to cyclic mobility varies with the level of static bias in the soil but not in the same manner. Whereas high levels of static stress can reduce a soil's resistance to liquefaction, its resistance to cyclic mobility tends to increase with the level of static shear. This behavior is related to the occurrence and magnitude of shear stress reversals. Castro and Poulos (1977) found that without stress reversal, the larger strains that constitute cyclic mobility do not occur.

CHAPTER 3

CURRENT METHODS FOR EVALUATING EARTHQUAKE PERFORMANCE AND ESTIMATING EARTHQUAKE INDUCED DEFORMATIONS

3.1 Pseudo-static Method

The pseudo-static method was the original method of analysis used for assessing the performance of earth structures during seismic excitation. The method yields only a factor of safety against failure during seismic loading and does not consider the magnitude of deformations that may result. For over 40 years the pseudo-static method was the standard method of evaluating the safety of dams against sliding during earthquakes. Its predictive capabilities, however, came into doubt from its inability to predict several dam failures. Today its use is generally limited to preliminary design calculations for soils that are not expected to suffer from appreciable strength or stiffness loss during cyclic loading.

In the pseudo-static method the effect of the earthquake is represented by an equivalent horizontal static force calculated as a fraction of the weight of the assumed failure mass. The fraction of the weight, termed the seismic coefficient, is selected on the basis of the level or seismicity in the region of interest. The equivalent static force is asssumed to be permanent and to act in one direction only, through the centroid of the assumed failure mass. A conventional slope stability analysis is then performed to determine the factor of safety against failure. A factor of safety in the range of 1.0 to 1.2 is generally believed to be adequate (Seed, 1979).

The initial widespread acceptance of the pseudo-static method for seismic design resulted because it appeared to be an extremely simple and effective design approach. However, a wide variation in the choice of analytical details is available. Because each of these choices can have a significant effect on the computed factor of safety, a broad range for the factor of safety may exist. In addition, although the design effectiveness of the pseudo-static method seemed to be indicated by the fact that few dams designed under its guidelines had failed, this apparent design adequacy could be attributed to the lack of strong earthquake motions rather than to the predictive capabilities of the pseudo-static method.

The predictive capability of the pseudo-static method came under question for its inability to predict the 1925 Sheffield Dam failure, the failure of the Lower San Fernando Dam in 1971 and the failures of several tailings dams during the Izu Oshima Earthquake of 1978. Investigations into the inability of the pseudo-static method to predict these failures revealed several fundamental problems with such a method of analysis.

A major problem with the pseudo-static method of analysis is its use of the factor of safety. The significance of the computed value for the factor of safety in evaluating stability is not clear. Although a factor of safety less than one indicates failure in a standard static stability analysis, for the pseudo-static analysis it may indicate only the onset of plastic deformations rather than complete collapse. Because of the transient nature of the earthquake force, the peak seismic force will only act for brief instants of time during which small plastic movements may occur. The magnitude of these accumulated deformations can be significant in determining whether the soil structure may continue to be used for its design purpose. The inability of the pseudo-static method to predict the magnitude of such deformations represents a serious limitation of this method of analysis.

A factor of safety greater than one may fail to give a true indication of stability in pseudo-static analyses. Slopes having factors of safety greater than one may fail if the slope forming material is susceptible to strength and/or stiffness loss during cyclic loading. Such soil behavior cannot be represented in any rational manner by a permanent horizontal static force. The use of the pseudo-static method should thus be confined to the analysis of slope forming materials that are not susceptible to pore pressure development during cyclic loading.

The variability of the computed factor of safety and the difficulty in assessing its significance limit the usefulness of the pseudo-static method. Clearly, there are too many deviations from realistic conditions in such an analysis to determine the reliability of the result.

3.2 Newmark Analysis

The Newmark method of seismic analysis evaluates the performance of a soil structure during earthquake loading by computing the expected deformations rather than by evaluating a factor of safety. It was first proposed by N.M. Newmark in his Rankine Lecture of 1965 (Newmark, 1965). The method is based on the assumption that the failure mass can be represented as a rigid block on an inclined plane. Movements along the failure plane are assumed to occur whenever the inertia forces acting on the slide mass exceed the yield resistance along the failure plane. Typically, several periods of exceedance, representing only brief instants of time, will occur during a given earthquake and will result in the accumulation of small plastic deformations. This treatment of the effects of seismic loading appears to be more logical than the pseudo-static method of analysis and has the added benefit that the deformations resulting from seismic activity can be computed.

Newmark showed that the earthquake induced deformations can be calculated by integrating the earthquake acceleration in excess of the yield acceleration over the period in which the yield acceleration is exceeded. The yield acceleration is defined as the seismic coefficient, k, in a standard pseudo-static equilibrium stability analysis which yields a factor of safety against failure of 1.0. To facilitate such calculations and to eliminate the required integration, Newmark provided charts and equations that related the maximum displacement of the slide mass to the yield acceleration and to the maximum values of the surface acceleration and the surface velocity caused by the earthquake. Thus, Newmark's analysis is capable of providing a rapid estimate of earthquake induced deformations.

The Newmark analysis uses a single value of the earthquake induced acceleration for the entire failure mass. However, during an earthquake, the effective peak acceleration within an embankment decreases with increasing depth. Seed and Martin (1966) and Ambraseys and Sarma (1967) have suggested refinements to Newmark's analysis which will allow for variations in the acceleration throughout the embankment and slide mass. The variation in the effective acceleration on a potential slide mass may be estimated from curves such as those presented by Makdisi and Seed (1978) which show the variation in the peak acceleration with depth.

A major limitation of the Newmark analysis lies in its assumption that the soil behaves in a rigid plastic manner during seismic loading. Hence, it considers only the effects of the inertia forces and ignores any displacements that may result from softening or strength loss due to pore pressure changes during seismic loading. For loose to medium dense cohesionless soils that often do not exhibit a well defined yield strength and whose behavior during cyclic loading may be complicated by the generation of large excess pore pressures, Newmark's analysis will fail to provide realistic estimates of deformations. For this reason its use should be reserved for soils where pore pressures do not change significantly during an earthquake, that is, for situations where the rigid plastic assumption of soil behavior is reasonably valid.

Another problem with the rigid block method of analysis is that the pattern of deformations cannot be determined. The entire failure mass is assumed to move as a single unit for the entire computed distance. The direction of the computed movement is also not clear. Such deformations can be viewed as either horizontal movements or sliding movements along the slip surface of the failure mass. For a seismic model to be entirely useful it should not only give the magnitude of deformations expected but should also provide a realistic pattern of deformations throughout the failure mass.

3.3 Seed's Dynamic Stress Path Approach

The dynamic stress path approach, developed by H.B. Seed and his co-workers at the University of California (Seed et al, 1969, and Seed et al, 1973), is a semi-analytical method of analysis designed for analyzing soils that are susceptible to significant strength and stiffness reduction during cyclic loading. The procedure involves performing a simple equivalent linear dynamic analysis to estimate the dynamic stresses to be applied to representative samples in the laboratory. Earthquake induced deformations are then determined by observing the strains or pore pressures developed in the samples.

The steps involved in this dynamic stress path analysis may be summarized as follows:

1. select a design earthquake motion
2. determine the initial static stresses in the soil structure
3. determine the dynamic properties of the soils
4. perform a dynamic analysis to determine the time history of cyclic stresses and strains within the soil
5. apply the combined static and cyclic stresses to representative samples and observe the accumulated strains and pore pressures
6. estimate deformations using the observed strains or pore pressures

The strains referred to above are the axial strains that develop in triaxial test samples when subjected to the static and dynamic stresses which reproduce as accurately as possible the stresses in the field. Since these strains occur in isolated samples whose deformations are not restricted by surrounding soils, they represent potential strains rather than the strains that may be expected to occur in the field. These strain potentials must be manipulated to produce a set of compatible strains and deformations within the soil structure. Seed (1979) proposed a method in which the shear stress corresponding to the strain potential is determined from the static stress-strain curve. These shear stresses are subsequently converted to equivalent nodal forces and a static finite element program is used to determine post-cyclic deformations. Alternatively, the deformations may be computed by using the pore pressures observed in the cyclic loading tests. These excess pore pressures reduce the stiffness of the soil and allow it to deform under the in-situ stresses. Byrne and Janzen (1981) suggested that the observed pore pressures could be used in a static stress-strain analysis to predict the earthquake induced deformations.

Seed resorted to the semi-analytical stress path technique because of the failure of the total stress equivalent elastic method of dynamic analysis to account for the influence of pore pressure rise on soil behavior and because of its inability to predict permanent deformations. The equivalent-linear analysis is used only to determine the cyclic shear stress level that may be expected in the field. Such shear stresses, however, may not be an accurate reflection of the actual field stresses because soil response, which is controlled by effective stresses, may not be reasonably approximated by a total stress approach. Finn et al (1978) found that a total stress analysis would tend to overestimate dynamic response when pore water pressures exceeded about 30 percent of the effective overburden pressure. Additional overprediction of dynamic response may also result from the development of a pseudo-resonance response, characteristic of equivalent-linear analyses. When the fundamental period of the earthquake motion corresponds closely to the fundamental period of the soil, the amplification of the response due to pseudo-resonance may be as much as 50 percent

(Finn et al, 1978). Because the dynamic stresses tend to be overestimated by a total stress equivalent-linear analysis, when they are applied to representative soil samples in the laboratory they will cause deformations in excess of what may be expected in the field. The resulting error is, hence, on the conservative side.

The degree of approximation associated with the strain potential/nodal force conversion is not known. Although the approach appears plausible, there is no theoretical justification for its use. The method has been validated to some extent but as Seed stated in 1979 not nearly so thoroughly as one might wish for a procedure which could affect the safety evaluation of such a critical structure as a major dam.

3.4 Effective Stress Dynamic Analysis

Several effective stress dynamic analyses are available for calculating the response of saturated cohesionless soils to earthquake loading. All tend to be relatively rigorous analytical procedures in which the stress-strain and strength properties of the soil are modified to account for pore pressure changes during cyclic loading. Siddhartan (1984) recently presented a two-dimensional effective stress analysis in which the non-linear stress-strain behavior is modelled by an incremental elastic approach. Less complex, one-dimensional analyses such as those used in the computer programs DESRA (Lee and Finn, 1978) and CHARSOIL (Streeter et al, 1974) are also available. A comparison of these latter methods is given by Finn et al (1978).

The various effective stress analyses tend to differ in the simplifying assumptions made, the representation of the stress-strain relations of the soils, the method by which the development of pore pressures is taken into account and the procedures used to integrate the dynamic equations of motion. All models are capable of providing a time history of the displacements in the soil structure. Since the effects of pore pressure rise are included in the analysis, the resulting stresses, strains and displacements are reliable and hence there is no need to resort to the semi-analytical dynamic stress path approach.

To realistically model the nonlinear, hysteretic and effective stress dependent soil behavior, these dynamic analyses have become increasingly complex. Many require parameters not commonly used in geotechnical practice and most lack sufficient verification. Although such methods are clearly more fundamental than the other forms of dynamic analyses their complexity, associated high cost, and lack of verification generally limit their use.

CHAPTER 4

PROPOSED METHODS FOR COMPUTING MODULUS REDUCTION

The proposed modulus reduction approach to dynamic analysis was developed as a simple means of analysis intended to simulate actual physical changes in the soil under earthquake loading. Primarily intended for soils prone to pore pressure rise during cyclic loading, it is very similar to Seed's stress path approach but is physically more realistic as it eliminates the need to resort to an arbitrary procedure for converting strain potentials to compatible deformations through the use of "equivalent" nodal forces.

The proposed method is based on the observation that as the pore pressures rise during cyclic loading, the soil softens and deforms until the geometry of the structure reflects the altered stress-strain relationship of the soil. This behavior can be simulated through the use of a reduced value for the modulus in a static stress-strain analysis. A reduced strength may also be used to reflect the loss in strength resulting from liquefaction failures should they occur. For the dilative soils susceptible to cyclic mobility, no reduction in strength is used since the undrained strength does not decrease with strain and the steady-state undrained strength is greater than the drained strength. However, because the development of the negative pore pressure required to mobilize the full undrained strength of dilative soils should not be relied upon in the field, the drained strength is used in a conservative analysis. Significant strains may be required to fully mobilize this strength due to the decreased stiffness of the soil.

The magnitude of modulus reduction required to provide realistic estimates of seismically induced deformations and the method of determining such a reduction were not clear at the onset of this study. This thesis attempts to determine the method for calculating the modulus reduction that will not only provide deformations of an appropriate magnitude but will also give a realistic pattern of deformations throughout the soil structure.

4.1 Methods for Calculating Modulus Reduction

Three alternative theories were investigated for determining a suitable modulus reduction. The first approach, referred to as the post-cyclic modulus approach, was based on the theory that the earthquake induced deformations could be calculated from the difference between the pre-cyclic and post-cyclic stress-strain relationships of a soil. Figure 4-1 depicts the earthquake induced deformations as the difference between the pre- and post-cyclic stress-strain curves at the stress level existing in the field. The reduced modulus is calculated directly from the post-cyclic stress-strain curve.

The second theory for determining the extent of the modulus reduction, or cyclic strain approach, recognizes that the development of pore pressures during cyclic loading and the resulting soil behavior is directly influenced by the level of

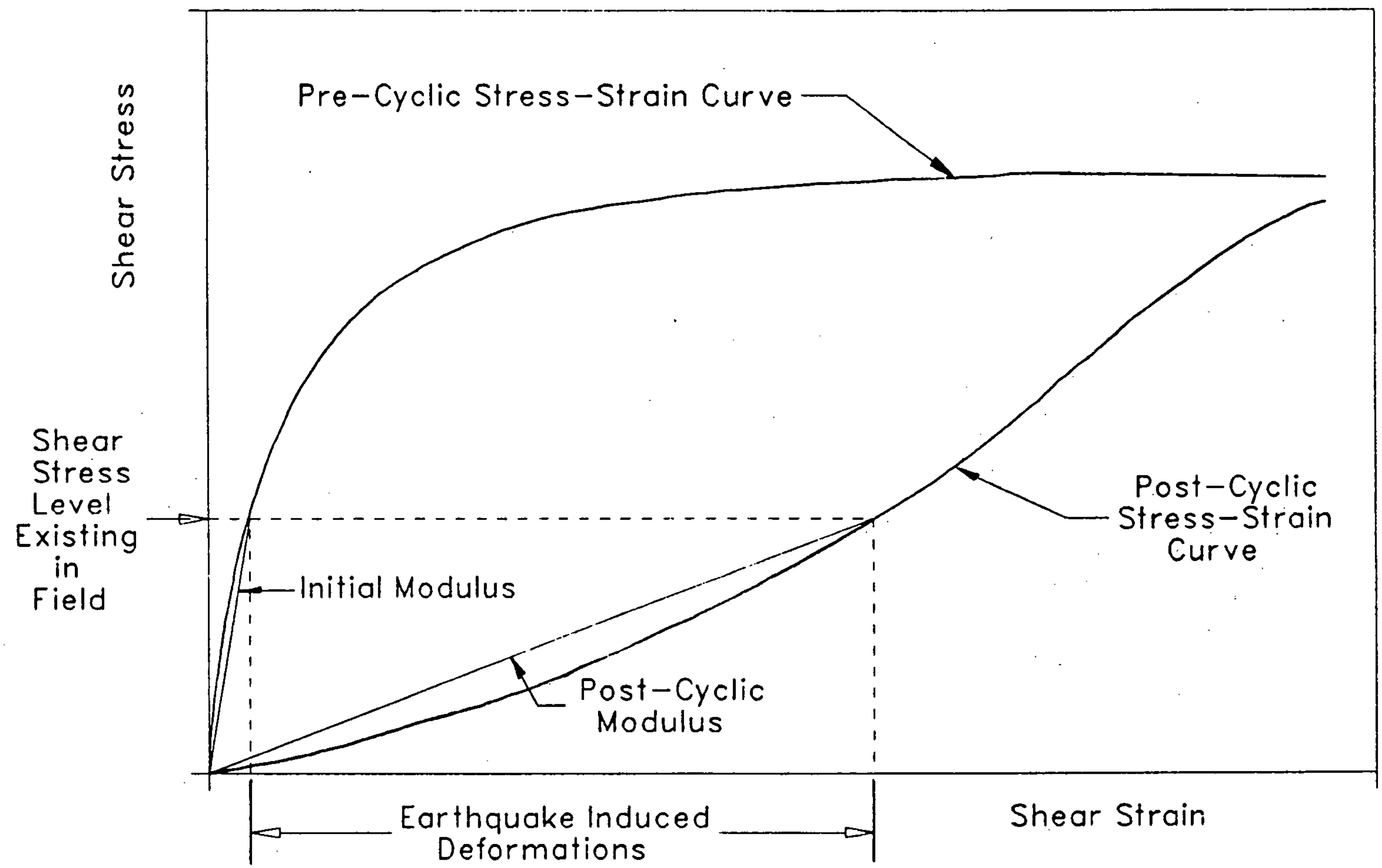


FIGURE 4-1 Post-cyclic modulus reduction approach for determining earthquake induced deformations

static shear stress in the soil and the duration of the earthquake. Hence, the appropriate modulus may be determined, as shown in Figure 4-2, from laboratory tests which duplicate the field static stress state and cyclic loading conditions as accurately as possible. The reduced modulus is given by the ratio of the in-situ shear stress and the shear strain produced during the combined static and cyclic loading. The earthquake induced deformations are calculated as the difference between the deformations computed using this reduced modulus and those found from a static analysis using the in-situ pre-cyclic modulus. The advantage of this method lies in its inclusion of both the effects of strain softening and inertia forces on the development of dynamically induced strains.

The third alternative, or the pore pressure approach, involves determining the pore pressures generated during cyclic loading and substituting them into a static analysis utilizing drained strength parameters. This procedure, shown in Figure 4-3, causes a reduction in strength as well as a reduction in modulus. The earthquake induced deformations are again taken to be the difference between the two stress-strain curves. The pore pressures to be used should be determined from cyclic loading tests that duplicate the field stress conditions as accurately as possible.

4.2 Deformation Analysis Procedure

The analysis procedure used in the modulus reduction

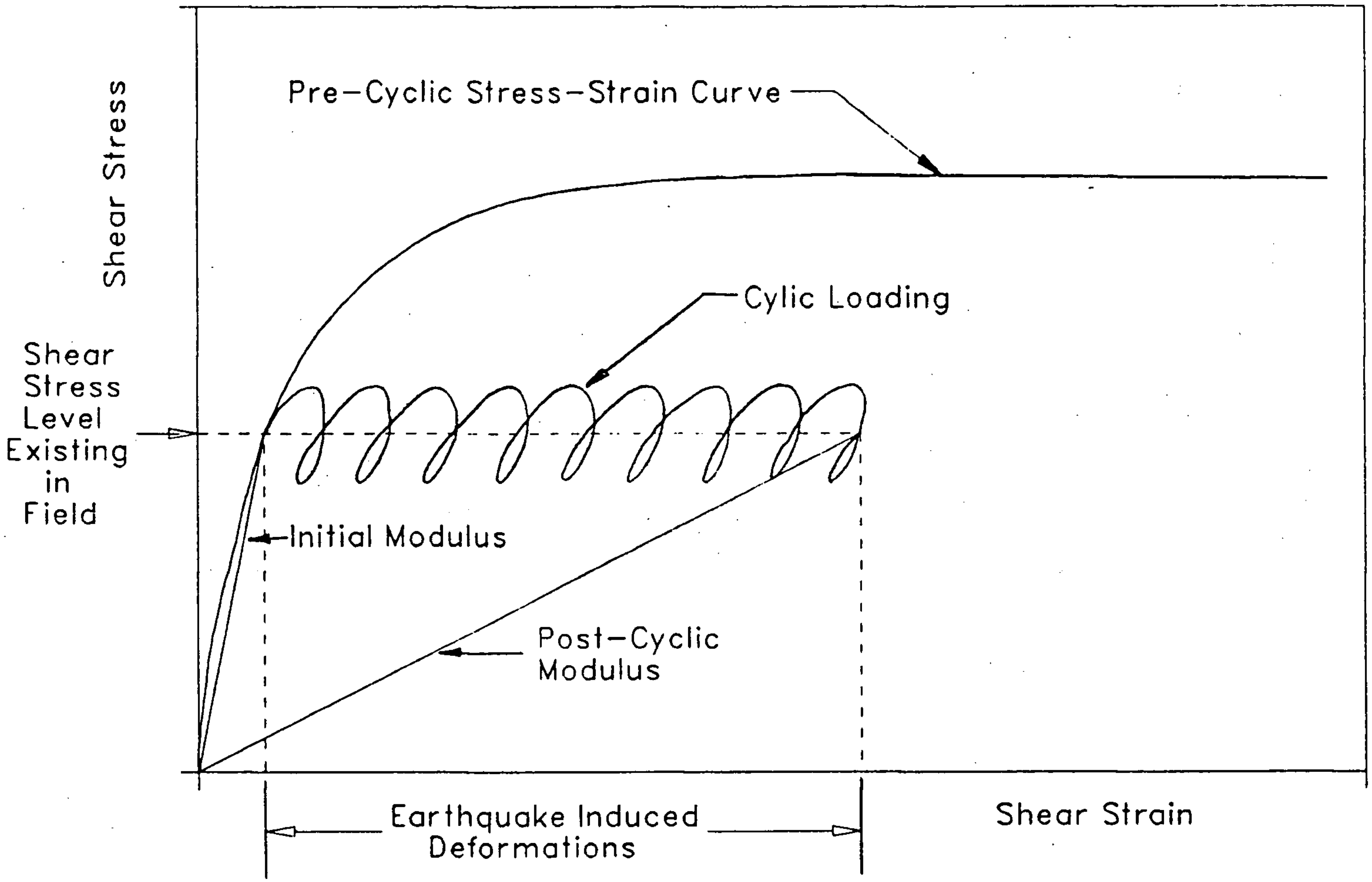


FIGURE 4-2 Cyclic strain modulus reduction approach for determining earthquake induced deformations

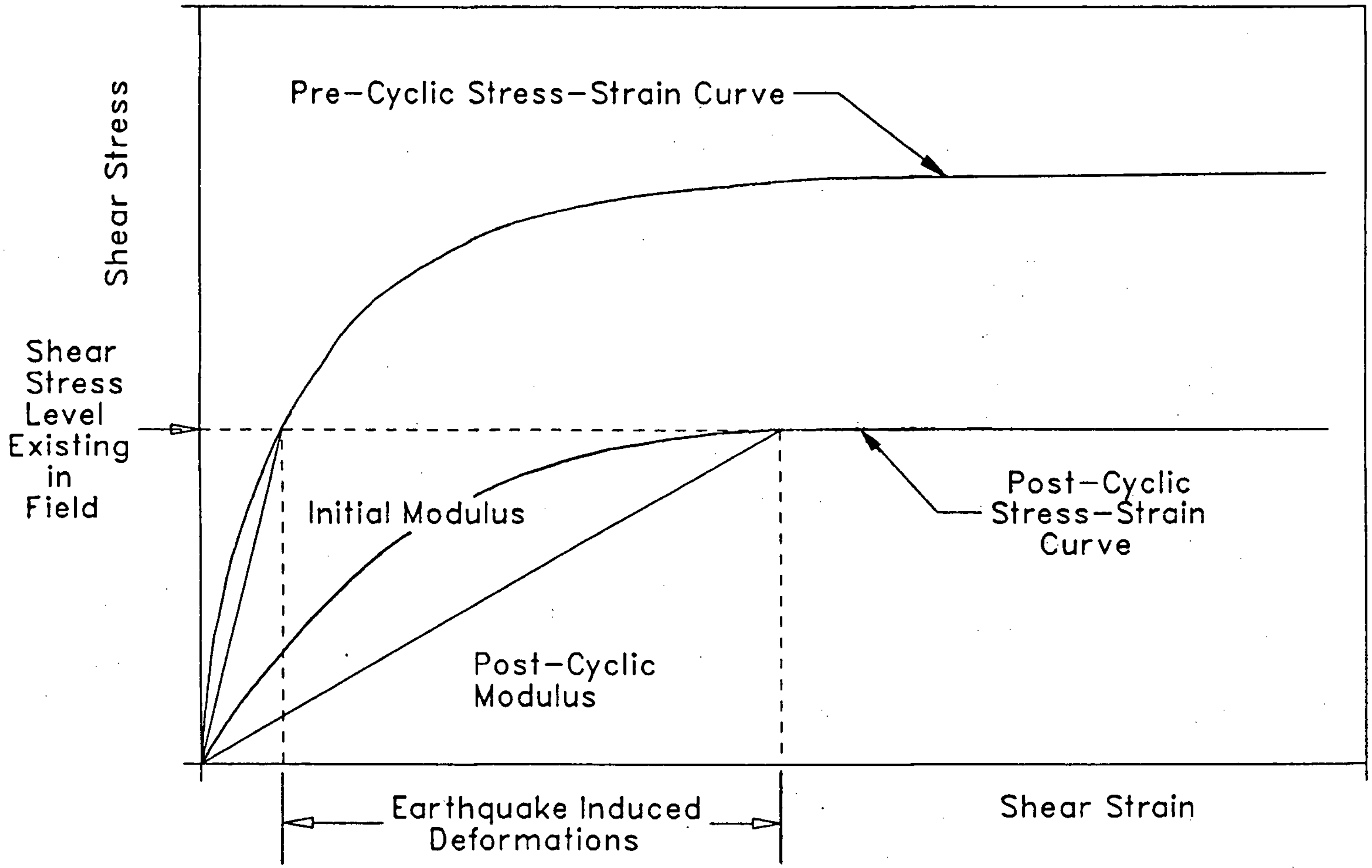


FIGURE 4–3 Pore Pressure approach for determining earthquake induced deformations

approach to dynamic analysis consists of the following basic steps.

1. select a design earthquake motion and a cross-section of the soil structure
2. determine the initial effective stresses in the soil structure by a static stress analysis
3. determine the time history of cyclic stresses throughout the soil structure by an equivalent-linear dynamic analysis
4. apply the combined static and cyclic stresses to representative samples in the laboratory and observe the soil response
5. calculate the appropriate modulus reduction
6. determine the earthquake induced deformations using a static analysis and the reduced modulus

The steps outlined above illustrate that the fundamental requirements of the modulus reduction approach are the static stability analysis, the dynamic analysis and the laboratory soil tests. The static stability analysis is used to determine the initial effective stresses in the soil structure and to evaluate the magnitude of the earthquake induced deformations through the use of a reduced modulus. The most convenient method of evaluating the initial stresses and deformations is through a finite element stability analysis such as the computer program SOILSTRESS (Byrne and Janzen, 1981). This program uses a plane strain finite element formulation in which the soil skeleton is modelled as a non-linear elastic continuum. The non-linear stress-strain behavior of soils is represented by equivalent-linear or secant moduli compatible with the level of induced stress and strain. A complete description of the program and its hyperbolic stress-strain formulation is given by Byrne and Janzen, (1981).

Experimental studies have shown that the shear modulus of a soil at any strain level is mainly a function of the initial shear modulus and the strength of the soil. If a hyperbolic relationship is assumed to exist between shear stress and shear strain, then the shear modulus, G, is given by

$$G = G_i \left(1 - \frac{\tau R_f}{s}\right)$$

where G_i = initial shear modulus

τ = shear stress developed

s = shear strength of soil

R_f = ratio of soil strength to the ultimate shear stress predicted by the hyperbolic relationship

The initial shear modulus, G_i, is a function of the mean confining stress, σ_m', and may be expressed as

$$G_i = k_g \, Pa \left(\frac{\sigma_m'}{Pa}\right)^n$$

where Pa = atmospheric pressure

k_g = shear modulus factor

n = shear modulus exponent

k_g and n are empirical parameters that vary with soil type and with loading condition. They may be determined from conventional drained or undrained triaxial tests or estimated from published parameters for similar soils. Because they are not fundamental soil properties, but are rather empirical coefficients whose values represent the behavior of the soil under a limited range of conditions, they should be evaluated from laboratory tests which duplicate field conditions as accurately as possible. During the modulus reduction dynamic analysis the value of k_g will be reduced to reflect the influence of dynamic loading.

The dynamic analysis utilized in the modulus reduction approach for predicting earthquake induced deformations is required to determine the level of cyclic shear stresses that is likely to be induced in the soil by the design earthquake. Because the evaluation of the design earthquake is difficult and subject to many uncertainties, the use of a rigorous analysis is generally not justified. In addition, the soil parameters may not be known to sufficient accuracy to reasonably approximate soil behavior in the more rigorous analyses. The selection of a total stress equivalent-linear dynamic analysis is consistent with both the level of accuracy of the available data and the desire for simplicity in avoiding the additional complexity of determining an appropriate pore pressure generation model.

Non-linear dynamic material properties are incorporated into the dynamic analysis using strain dependent modulus and damping values. Seed and Idriss (1970) showed the shear modulus to be a function of the square root of the mean confining stress and presented modulus reduction curves showing the decrease in modulus with strain level. Such relationships are used to determine the relevant shear moduli and damping ratios during the dynamic analysis.

The dynamic analysis is performed by an iterative approach in which the initial values for the shear modulus and damping values are assumed and used to compute the strains in each element. New shear moduli and damping values are calculated based on these strains and the analysis is repeated until the values of modulus and damping are compatible with the strains developed in each element.

Such a response analysis provides a time history of stresses and strains within the soil structure. These dynamic stresses may be converted to an equivalent series of uniform stress applications. Lee and Chan (1972) presented a method of conversion by appropriate weighting of the ordinates of the stress time history. The equivalent cyclic stress applications can then be applied to representative samples in the laboratory to determine appropriate modulus reductions for computing deformations.

In addition to cyclic tests, standard monotonic loading tests are required to evaluate soil behavior during drained and undrained loading and to determine the appropriate hyperbolic parameters. Standard triaxial and cyclic triaxial tests are most convenient because of their relative simplicity and wide availability of equipment.

Evaluation of the earthquake induced displacements by the post-cyclic modulus approach requires the determination of stress-strain curves for the soils before and after cyclic loading. Although, theoretically, this approach appears simple, it is in fact very difficult to determine the post-cyclic stress-strain relationship exactly as represented in Figure 4-1 which shows the curve beginning at zero axial strain. To determine such a curve from triaxial tests, the accumulation of permanent axial strains during cyclic loading would have to be prevented by consolidating and loading the sample under isotropic conditions. However, since isotropically consolidated samples generally fail during the extensional phase of cyclic loading, any subsequent reloading in compression during post-cyclic monotonic testing would not reflect the soil behavior expected in the field. To ensure failure on the compression side, triaxial samples generally have to be tested under anisotropic conditions with Kc values that depend on the charateristics of the soil. Under such anisotropic conditions permanent axial strains will develop during cyclic loading. If liquefaction occurs, the very large axial strains that develop may exceed the limits of the testing equipment. To ensure that large axial strains do not occur and post-cyclic monotonic tests can be performed, the development of axial strains during liquefaction must be halted at some small strain level. The sample may then be loaded monotonically and the post-cyclic stress-strain curve determined. The reduction in the modulus factor is determined by comparing the initial part of the post-cyclic stress-strain curve to that of the pre-cyclic stress-strain curve.

To determine the magnitude of modulus reduction from the cyclic strain method, both standard triaxial tests and cyclic loading tests must be performed on representative soil samples. The standard triaxial tests are required to determine the hyperbolic stress-strain parameters used to compute the initial effective stresses within the soil structure. These stresses are then applied to representative soil samples in the laboratory in combination with appropriate cyclic stresses. The strains resulting from these tests are used to compute the effective shear modulus during cyclic loading. This cyclic loading shear modulus is compared to the pre-cyclic value determined from the static analysis and an appropriate reduction in the modulus factor is determined.

The pore pressure method of dynamic analysis requires the same laboratory tests as the cyclic strain method. However, unlike the cyclic strain approach, the value of the modulus factor is not reduced. The reduction in the shear modulus results from the inclusion of the excess pore pressures generated during cyclic loading in the static analysis. The pore pressures cause a decrease in the mean confining stress and hence, a decrease in the effective shear modulus. Unlike the two preceding methods of modulus reduction, a decrease in strength also results from this approach.

CHAPTER 5

VERIFICATION OF PROPOSED METHOD

The validity and applicability of the three proposed methods for computing the post-cyclic modulus can only be evaluated by comparing predicted deformations to actual field deformation histories. Unfortunately, few earth structures have been subjected to severe earthquake shaking and for the few that have, insufficient data is available to provide accurate soil and earthquake parameters for use in such an analysis. Because of the lack of a suitable field case history, the three proposed methods for computing modulus reductions were investigated using the results of shaking table tests performed at the University of British Columbia on a model that was intended to represent a sloped saturated tailings deposit. The reliability of each of the proposed methods of analysis was evaluated by comparing the predicted magnitude and pattern of deformations to those observed in the model. The method of analysis that yielded the most accurate deformations was then used to predict the deformations that occurred in the Upper San Fernando Dam during the earthquake of February, 1971.

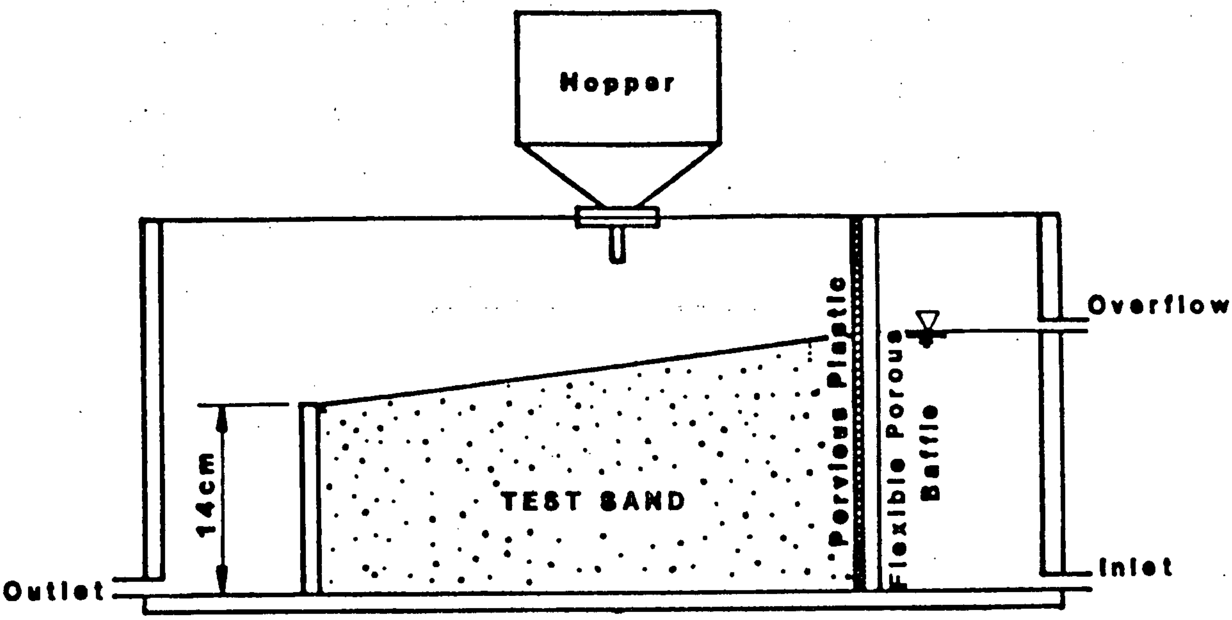
5.1 Tailings Model Tests

The results of shaking table tests on saturated model tailings slopes were used to determine the correct procedure for evaluating the appropriate magnitude of modulus reduction due to cyclic loading. The tests were part of a study on the prediction of deformations of liquefied tailings deposits by viscous flow theory. They were performed by B. Stuckert at the University of British Columbia as part of his master's degree research. A complete description of the testing equipment, procedure, and results is given by Stuckert, (1982).

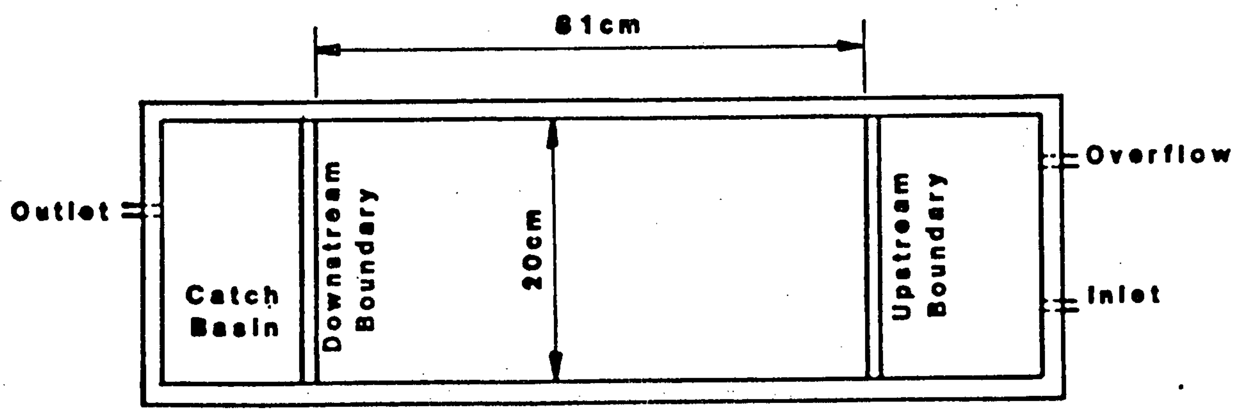
The model tailings slopes were constructed in the plexiglass container shown in Figure 5-1. The slopes were 81 cm long and 20 cm wide. Although various slope configurations were tested, only slopes having a fixed depth of 14 cm at the downstream boundary and rising with a 8 degree slope are considered for this thesis.

The slopes were formed by allowing dry sand to fall from a hopper into the plexiglass container that was partially filled with deaired water. The hopper was used to distribute the sand evenly across the slope. A scraper then smoothed the slope to the desired angle. During deposition silica beads were placed adjacent to the plexiglass wall in a grid pattern. The displacements of these beads were monitored during the test to determine the magnitude and pattern of the dynamically induced deformations.

The sand used in the study was a fine Ottawa sand. This sand is a clean silica sand having rounded to subrounded grains and a specific gravity of 2.67. Figure 5-2 shows the grain size distribution for the sand. The maximum and minimum void ratios were determined as .86 and .56 respectively. The method of deposition used for forming the model slopes resulted in void



ELEVATION



PLAN VIEW

(from Byrne, Vaid and Stuckert, 1981)

FIGURE 5–1 Model Container

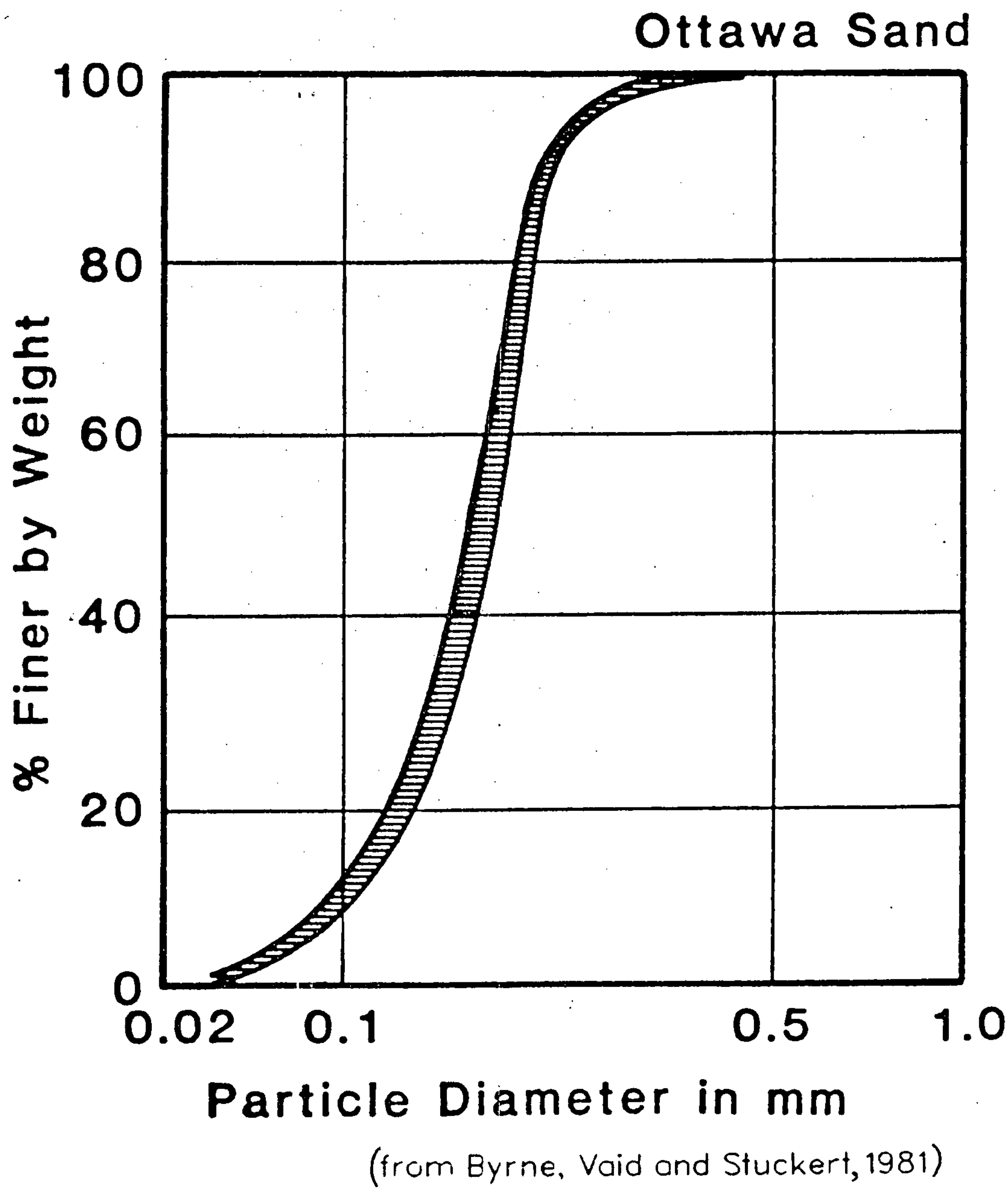


FIGURE 5–2 Grain Size Distribution of Test Sand

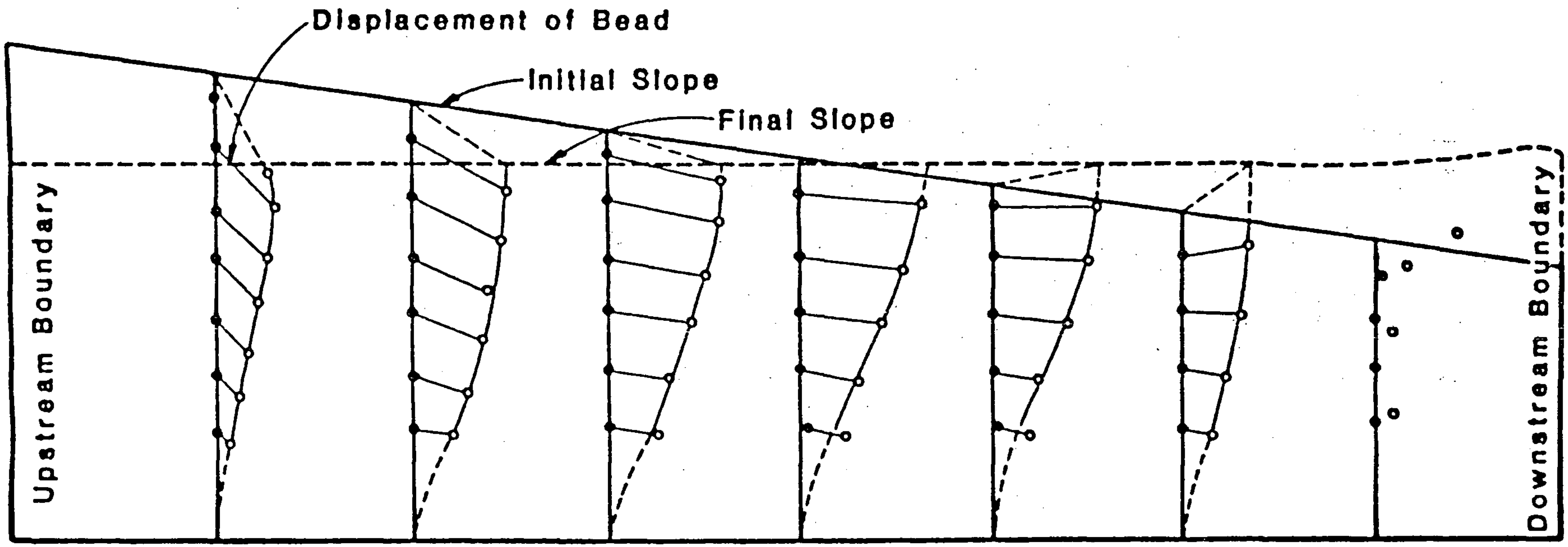
ratios of approximately .77 or about 30 percent relative density.

The tests were performed on a 1.2m by 2.7m shaking table. The table motions were controlled by an MTS Earthquake Simulator console which provided sinusoidal motions at a frequency of 5 Hz. This frequency was selected to ensure that practically undrained conditions existed for the 20 cycles of shaking applied. Maximum accelerations of .03 to .1g were used.

During shaking the model slopes experienced large movements which resulted in substantial slope flattening. For tests in which the accelerations exceeded about .05g, final slope angles were less than half a degree. The displacement pattern illustrated by the silica beads revealed that the deformations resulted from deep-seated movements rather than from merely a surface transport of material. Figure 5-3 shows the deformations which occurred in the tailings slope model during a test in which the maximum acceleration was .08g. The displacements are typical of those occurring when a significant portion of the slope appeared to liquefy. The displacements are greatest at the surface and decrease with depth to zero along the model base. Maximum movements of approximately 6 to 7 cm occur near the center of the slope. Such displacements indicate a shear strain level of 30 to 40 percent.

5.1.1 Laboratory Tests

Three sets of laboratory triaxial tests were performed to



(from Byrne, Vaid and Stuckert, 1981)

FIGURE 5–3 Observed Dynamically Induced Deformations for Model Test for 8 degree Slope and .08g Maximum Acceleration provide the soil parameters necessary for the various modulus reduction analyses. Drained and undrained monotonic triaxial tests were performed to determine soil response during drained and undrained loading. Cyclic loading tests followed by undrained monotonic loading tests were performed to evaluate soil behavior after cyclic loading.

For all of the tests, a relative density of approximately 30 percent was used to simulate the conditions in the model. Because of the very low stress levels existing in the model, it was impossible to perform the triaxial tests over the model stress range. All tests were performed at confining pressures between 50 and 200 kPa and the results were assumed to be appropriate for the lower stress levels in the model.

Monotonic Tests

The response of the fine Ottawa sand to drained and undrained monotonic loading is shown in Figures 5-4, 5-5, 5-6 and 5-7 for confining pressures of 50, 100, 150 and 200 kPa, respectively. For both the drained and undrained tests, the samples were consolidated isotropically and then loaded axially to failure. The behavior is typical of most granular soils. After an initial period of compression, the drained samples began to dilate. This volume change behavior is reflected in the undrained samples by the initial rise in pore pressure followed by a reduction at higher strain levels. The stress-strain curves for the drained samples are nearly hyperbolic. The undrained curves rise steeply initially, level off or reduce

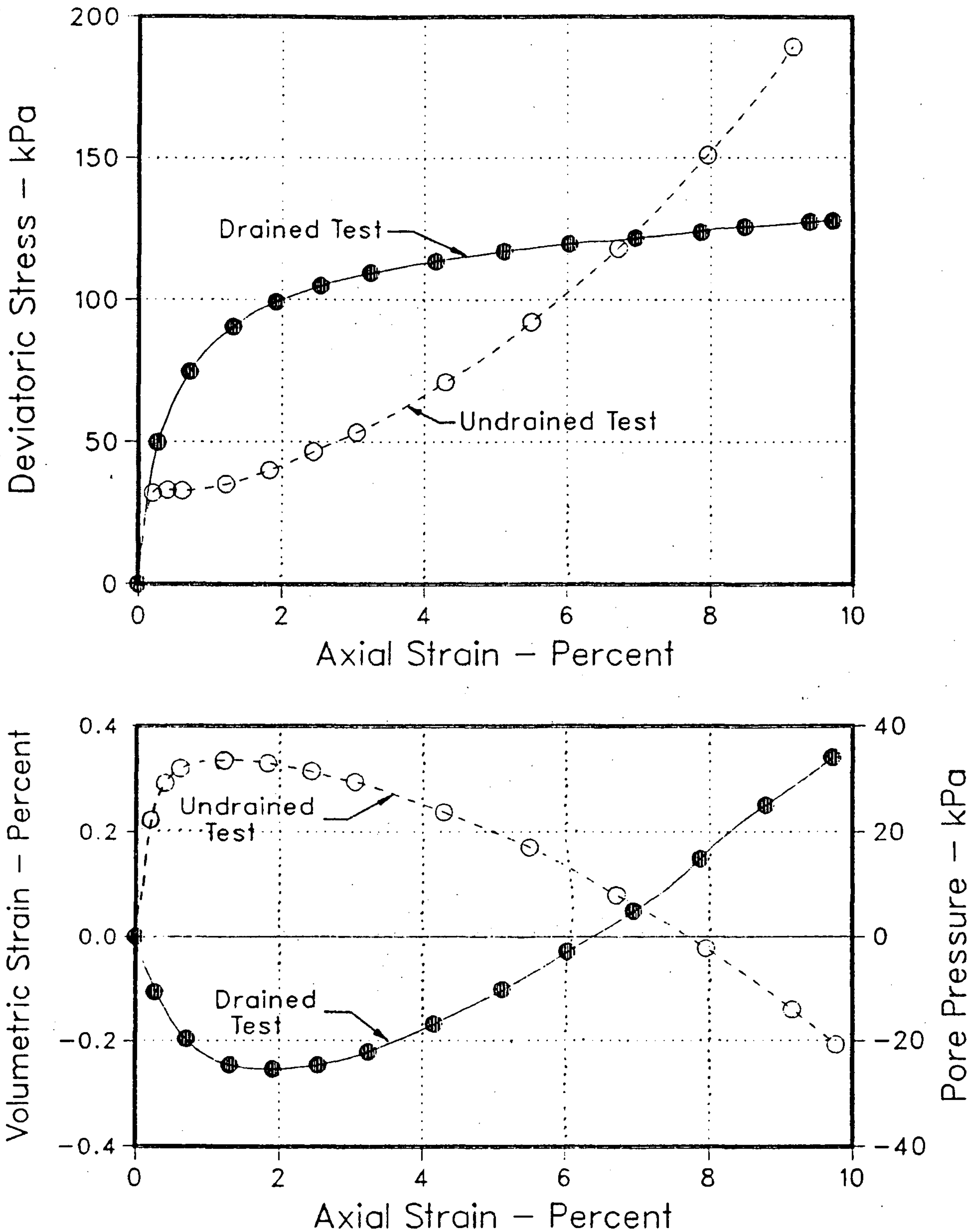


FIGURE 5–4 Soil Response During Monotonic Triaxial Loading Tests – Fine Ottawa Sand

Confining Pressure = 50 kPa

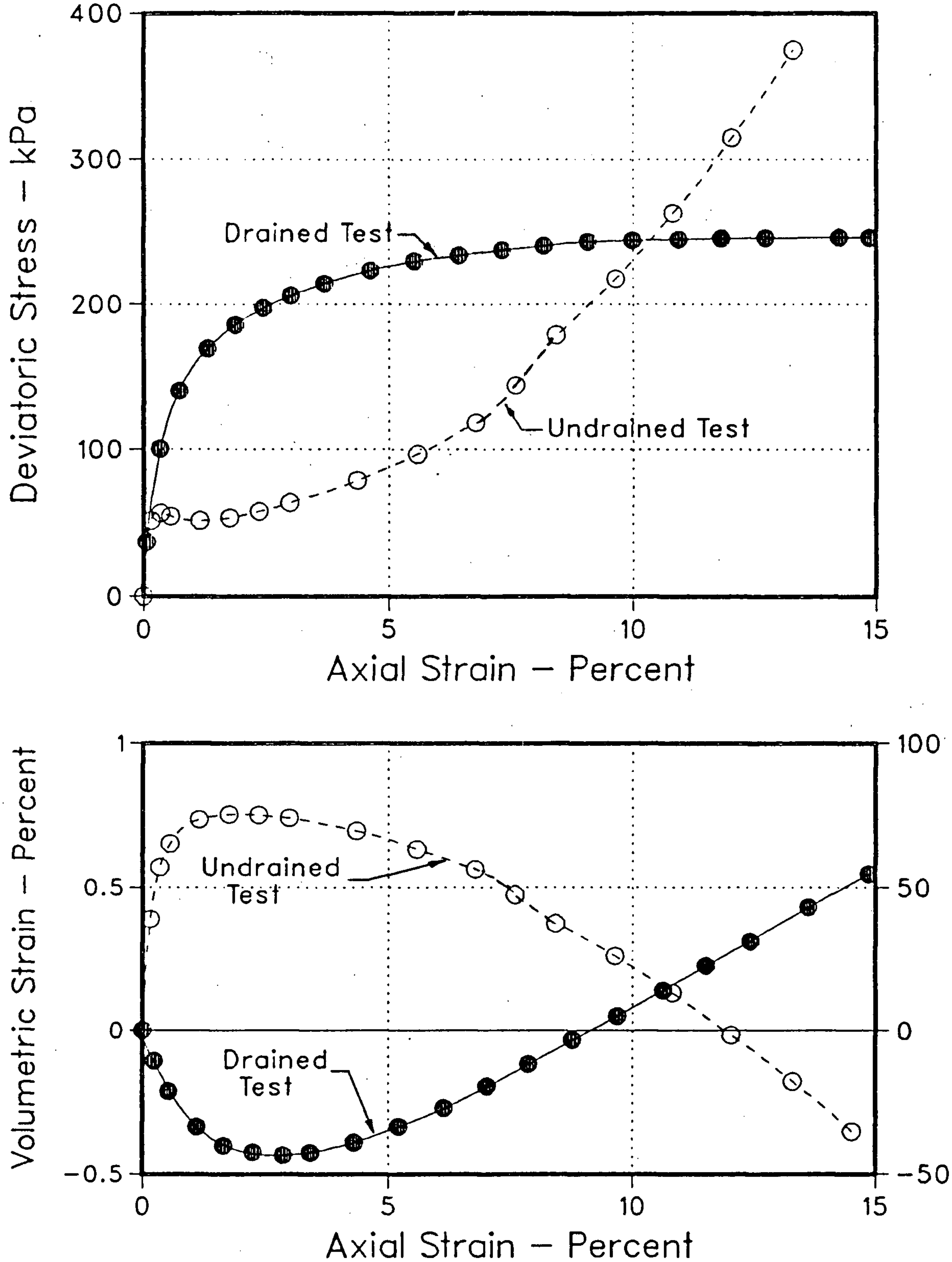


FIGURE 5–5 Soil Response During Monotonic Triaxial Loading Tests – Fine Ottawa Sand

Confining Pressure = 100 kPa

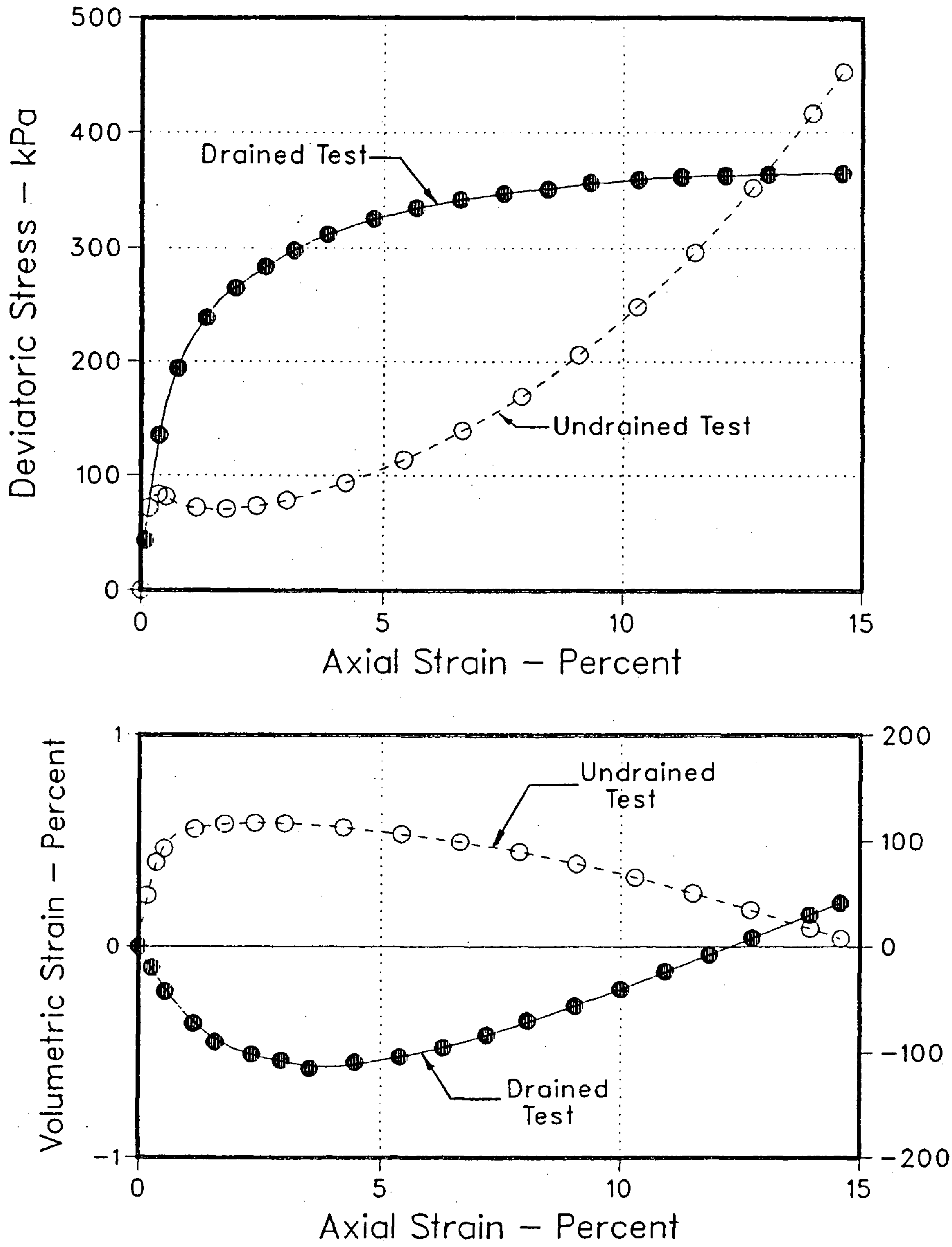


FIGURE 5–6 Soil Response During Monotonic Triaxial Loading Tests – Fine Ottawa Sand

Confining Pressure = 150 kPa

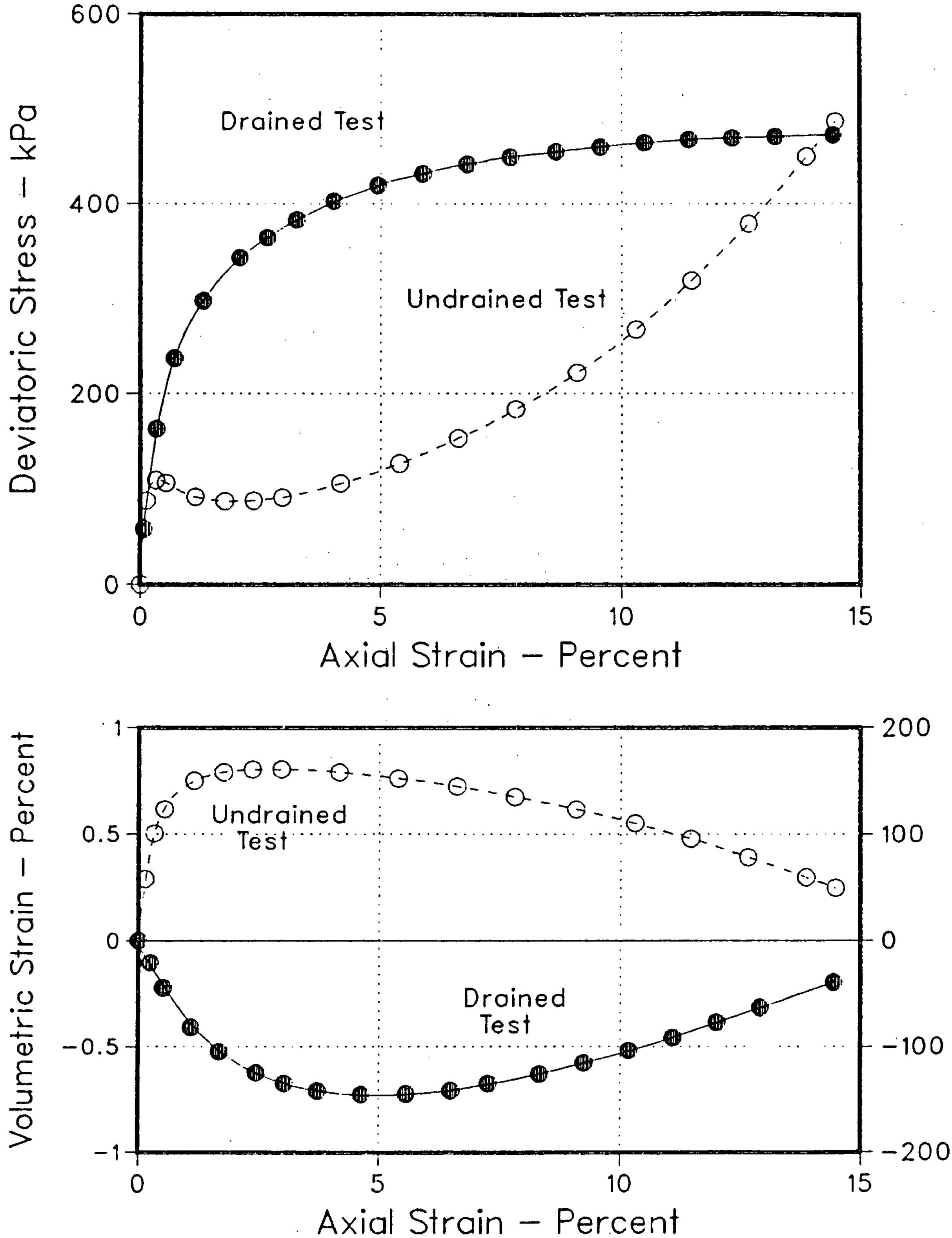


FIGURE 5–7 Soil Response During Monotonic Triaxial Loading Tests – Fine Ottawa Sand

Confining Pressure = 200 kPa

slightly, and then start to rise again as the sand begins to strain harden. For the undrained test at a confining pressure of 50 kPa, no significant amount of strain softening occurs after the initial peak in the stress-strain curve. Although the tests at higher confining pressures did show a small but distinct peak, the behavior illustrated in Figure 5-4 for a confining pressure of 50 kPa is believed to be more indicative of the soil response at the stress level existing in the model slopes. Hence, no significant loss in strength is anticipated within the model. The large displacements observed in the model tests were thus a result of cyclic mobility behavior rather than a true liquefaction failure which requires a loss in strength after the peak in the undrained stress-strain curve has been reached.

The hyperbolic stress-strain parameters were evaluated following the procedure described by Duncan et al (1980), and are summarized on Table 5-1. For the drained tests, the parameters are based on effective stresses, while for the undrained tests, they are a function of the initial effective stresses. For the undrained tests the bulk modulus factor may be assumed to be very high to simulate the lack of volume change that would occur under undrained conditions for fully saturated samples. The bulk modulus parameters for the drained tests were determined from the volume change measurements.

Cyclic Tests

During the cyclic loading tests the samples were initially

Table 5-1

Summary of Hyperbolic Parameters
Fine Ottawa Sand

	drained loading	undrained loading
shear modulus factor k_g	135	320
shear modulus exponent n	.62	.68
bulk modulus factor k_b	157	
bulk modulus exponent m	.35	
reduction ratio R_f	.95	.71
angle of friction ϕ	33.5	32.0
change in friction angle $\Delta\phi$	2.1	4.0

consolidated anisotropically with a Kc value of 1.2 to ensure that failure would occur in compression. Because the entire model slopes appeared to liquefy within the first few cycles of loading during the shaking table tests, the cyclic stress ratio was selected as the value that would cause liquefaction in less than 10 cycles. If rigid body motion is assumed in the model, the cyclic stress ratio occurring during the shaking table tests may be evaluated as .16. The cyclic stress ratio used in the laboratory tests was slightly lower at .15. The development of large strains was prevented by stopping the tests after initial liquefaction had occurred at a strain level of approximately 2.5 percent. Samples were then monotonically loaded to provide post-liquefaction stress-strain curves. During these tests no additional pore pressures were generated. Instead the pore pressures began to drop, slowly initially and then at an increasing rate in the later stages of the tests. Because the reduced pore pressures led to increased confining stresses, the strengths at large strains tended to be similar to those determined from the pre-cyclic undrained tests. Figures 5-8, 5-9 and 5-10 show the similarity between the pre- and post-cyclic response of the sand at various confining pressures during undrained loading.

5.1.2 Determination of Reduced Moduli

The reduced modulus factor for the post-cyclic modulus approach was determined from the difference in soil response before and after cyclic loading. The initial shear modulus for each curve was evaluated as the secant to the curve connecting the origin to the initial peak in the pre-cyclic curve. For the post-cyclic curve, the 2.5 percent axial strain that developed during initial liquefaction was included in the calculation of the initial shear modulus. A comparison of the two moduli yielded an apparent modulus reduction of approximately 50 times, corresponding to a hyperbolic modulus factor of 6.

The modulus reduction for the cyclic strain approach was not determined directly from laboratory tests. Ideally, the reduction in the modulus would be determined by subjecting undisturbed samples to the combined static and cyclic stresses believed to exist in the field and observing the accumulated strains. However, laboratory tests at the very low stress

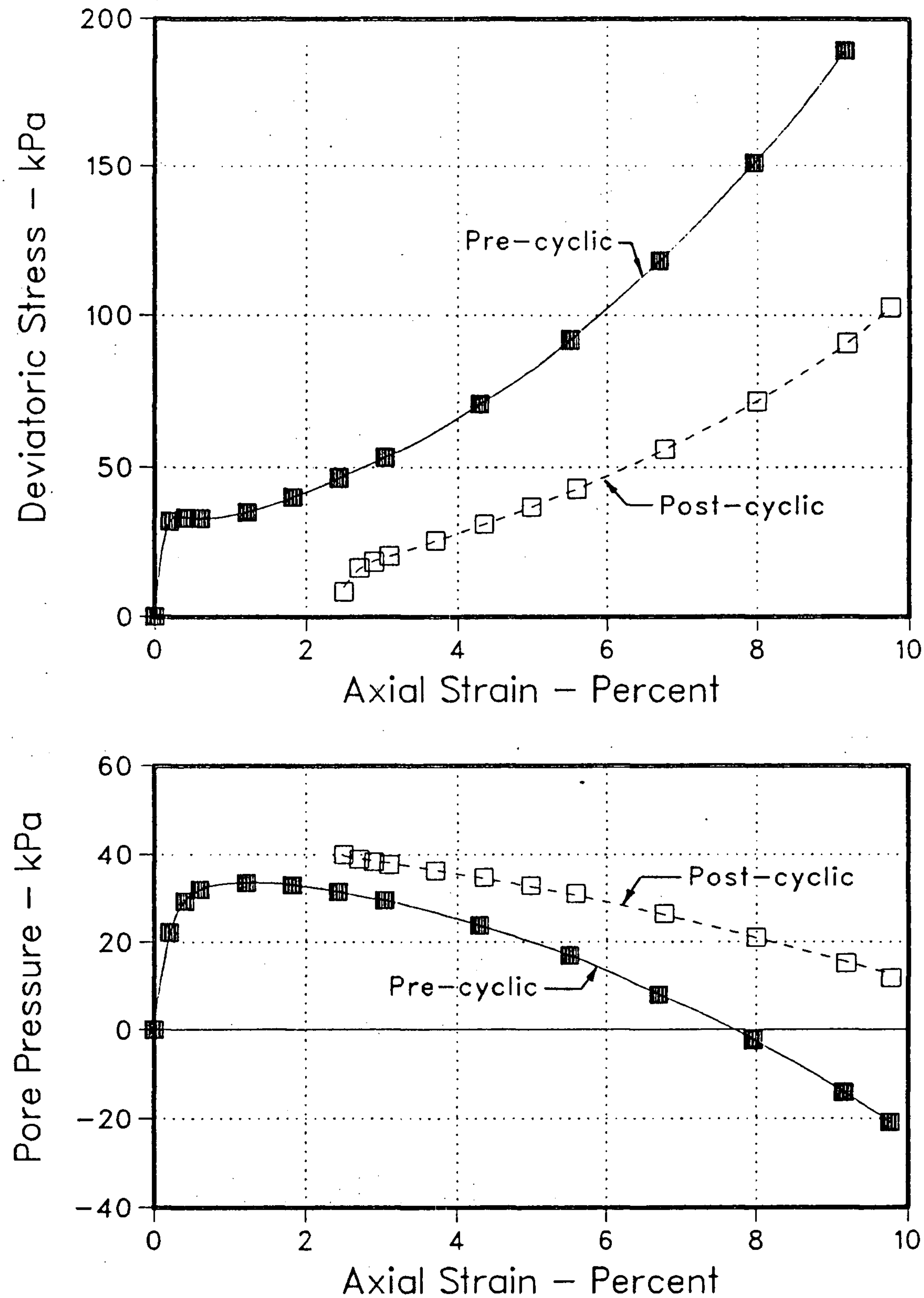
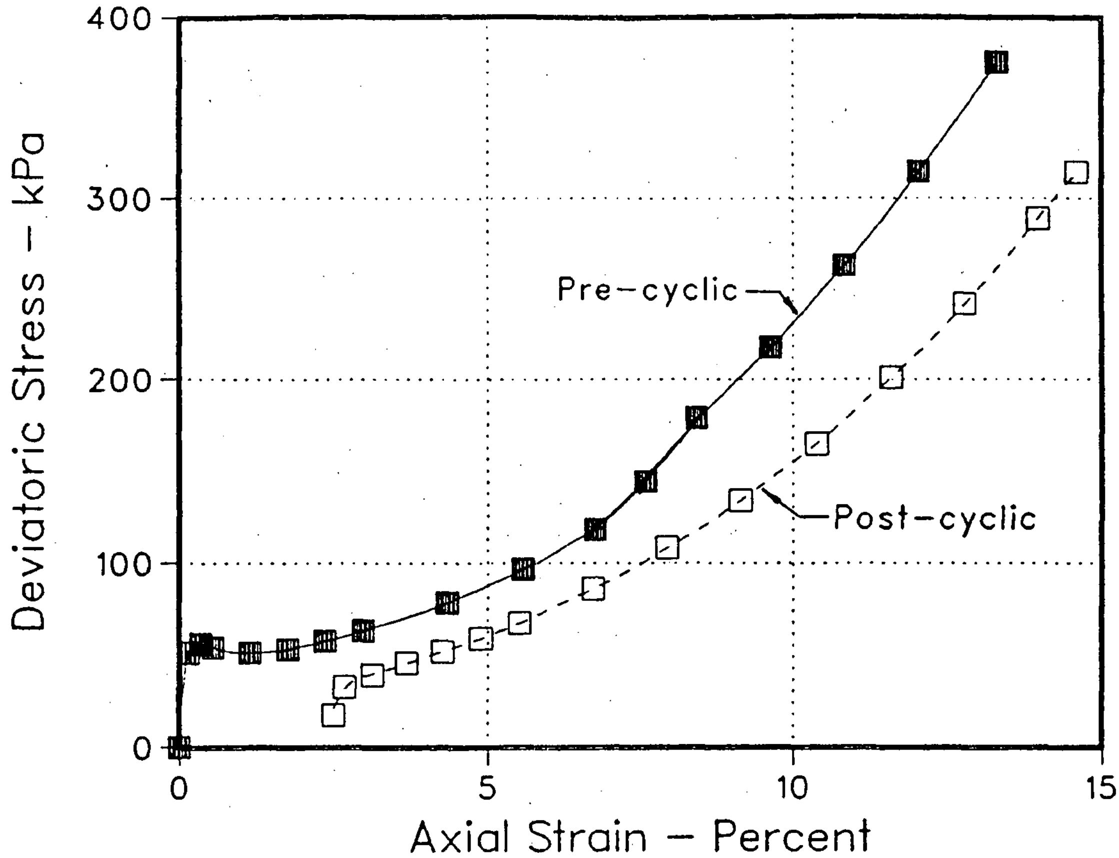


FIGURE 5–8 Stress–Strain Behavior Before and After Cyclic Loading – Fine Ottawa Sand

Confining Pressure = 50 kPa



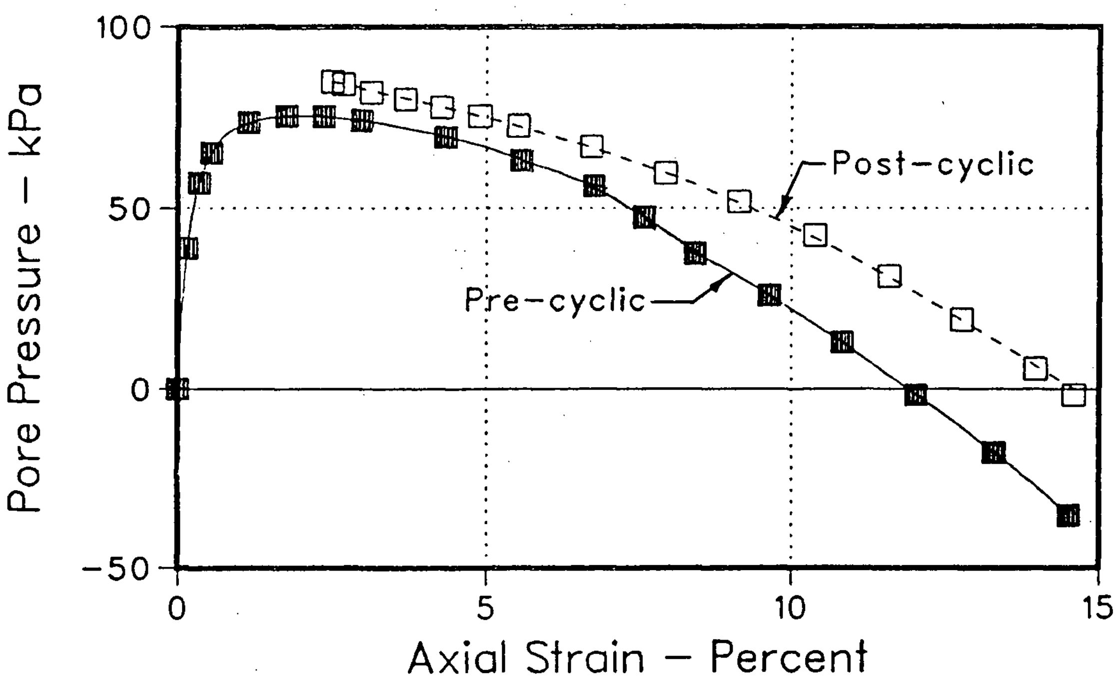
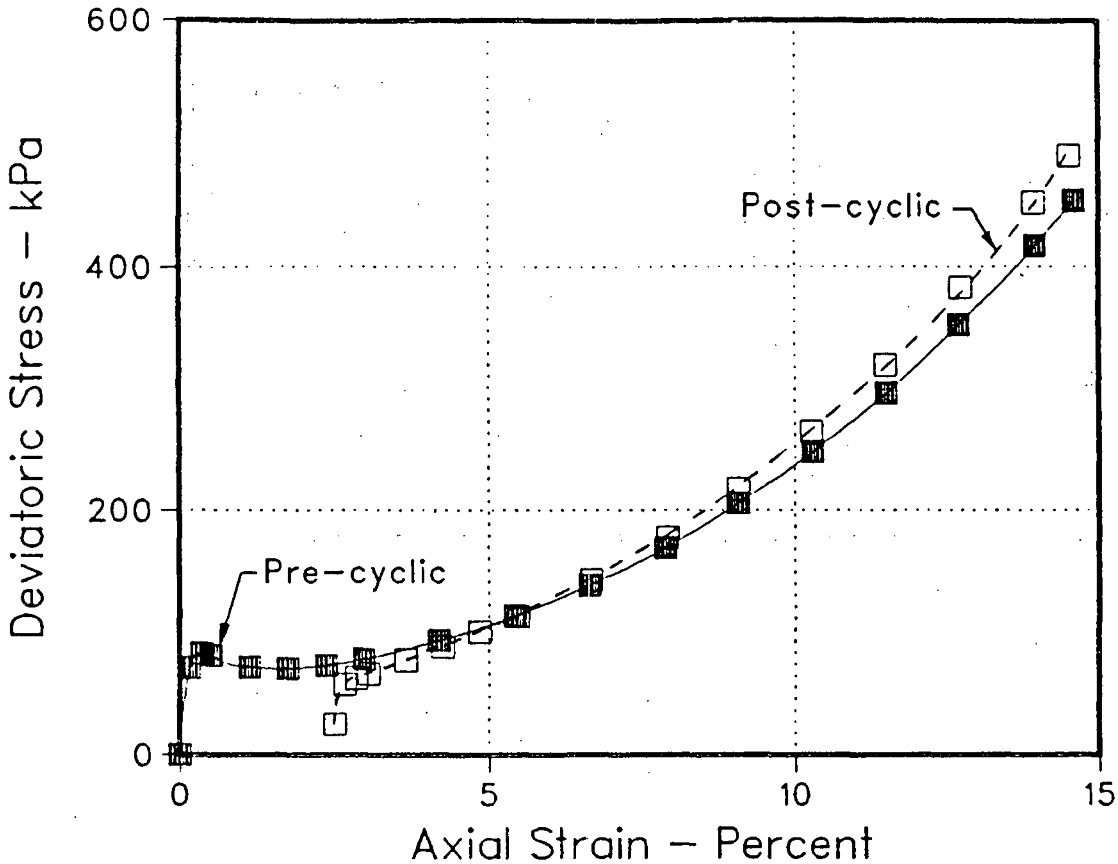


FIGURE 5-9 Stress-Strain Behavior Before and After Cyclic Loading – Fine Ottawa Sand

Confining Pressure = 100 kPa



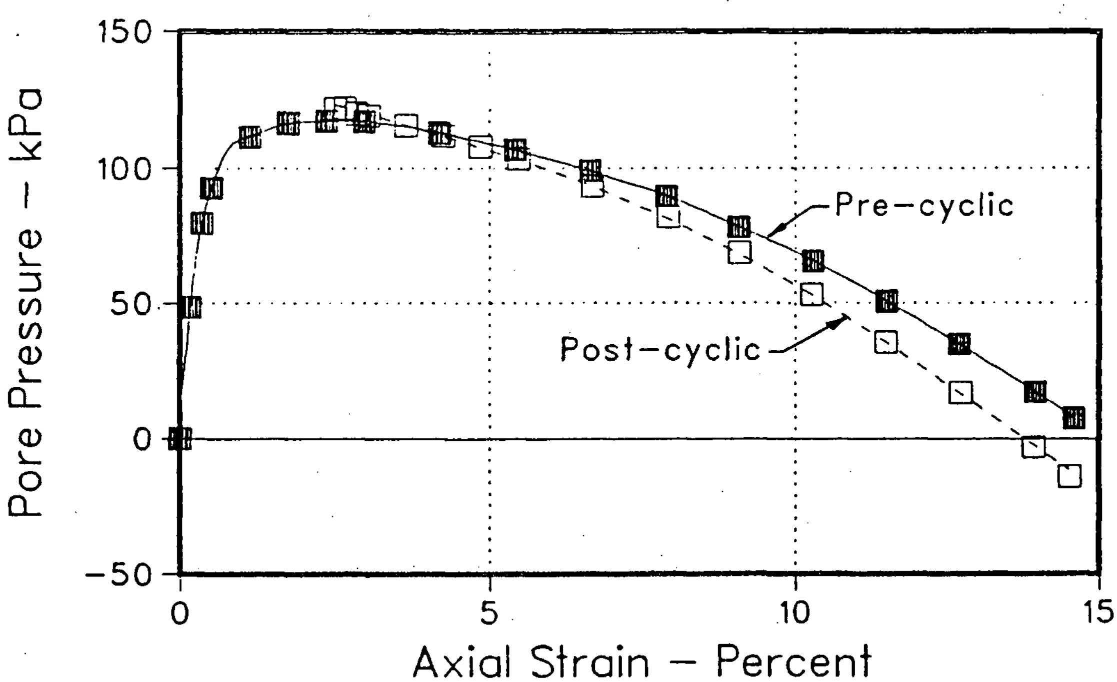


FIGURE 5-10 Stress-Strain Behavior Before and After Cyclic Loading – Fine Ottawa Sand

Confining Pressure = 150 kPa

levels existing in the model are extremely difficult to perform. Hence, any cyclic loading tests would have to be performed at stress levels significantly higher than those in the model. Because the susceptibility of a soil to liquefaction increases with increasing confining stress, such tests would not reproduce the mechanism of strain development thought to occur in the model. The monotonic loading tests performed on the test sand at the appropriate relative density indicated an increasing tendency for dilation at the lower confining pressures. For the triaxial test at the lowest confining pressure of 50 kPa, the sample began to dilate at strains of between 1 and 2 percent. The stress-strain behavior of the sand at this confining pressure was also characterized by the absence of the strain softening that had occurred during the tests at the higher confining pressures. Because the average confining pressure in the model slope was about 1 kPa - 50 times less than the confining pressure used for the monotonic triaxial test exhibiting no strain softening - the sand at the model stress conditions would have been dilative rather than contractive. Thus the development of strains during cyclic loading would have resulted from cyclic mobility rather than from liquefaction. The large deformations observed in the model slopes represent the accumulation of the strains occurring during the loading phase of each stress cycle. The speed at which the deformations occurred can be attributed to the high frequency of cyclic loading that was used to ensure that undrained conditions would prevail during shaking.

Although the appropriate cyclic loading tests could not be performed, the potential of the soil to undergo large deformations resulting from the progressive development of strains during each cycle of stress application was investigated. A cyclic loading test was performed to determine the cyclic stress amplitude required to produce appreciable strains during each cyclic load application. The cyclic stress ratio that would produce the magnitude of the strains observed in the model slope during the 20 cycles of shaking applied was also determined. The test was performed at an overall effective confining pressure of 100 kPa on a sample consolidated anisotropically to a Kc value of 1.2. At these conditions the sample would be susceptible to liquefaction and would develop cyclic mobility only after an initial flow failure. During the initial part of the test in which liquefaction was induced by applying a cyclic stress ratio of .15, the strains were limited to approximately 2.5 percent. The cyclic stresses were then reapplied but with a reduced amplitude. The axial strains resulting during each loading cycle were observed. The variation in the axial strains generated during each loading cycle with the cyclic stress ratio was determined by progressively increasing the cyclic stress amplitude. The test indicated that significant strains developed only when the cyclic stress amplitude exceeded the initial deviator stress used to create anisotropic conditions. Hence stress reversals were necessary for significant development of strain. The test also showed that a cyclic stress ratio of about .10, rather than

.16 as was characteristic of the model tailings tests, would be required to generate the same cumulative strains as those observed in the model tests for the 20 cycles of loading applied. However, since the model slopes were subjected to a much lower confining pressure and a somewhat higher level of static bias than the laboratory triaxial sample, a higher cyclic stress level would be required in the model tests to cause a comparable level of strain. Although the results of this cyclic loading test cannot be used quantitatively to predict the modulus reduction in the model tailings slope tests due to the differing stress conditions, they do confirm that substantial strains will develop during cyclic loading from the accumulation of strains occurring during each load cycle.

The shear strains used to compute the modulus reduction for the cyclic strain approach were assumed to be approximately 30 to 40 percent, corresponding to those observed in the model slopes. These strains may be slightly lower than those determined from appropriate laboratory tests because the static shear stresses existing in the model slopes reduced during shaking as a result of slope flattening. The assumed strain levels caused reductions in the modulus factor of 1000 to 2000 times, resulting in k_g values in the .05 to .20 range.

The reduction in the modulus for the pore pressure approach is determined from the magnitude of the pore pressure rise resulting during cyclic loading. The development of pore pressures during dynamic loading depends on both the properties of the soil and the characteristics of the dynamic excitation. The magnitude of the residual pore pressures developed during either liquefaction or cyclic mobility is a function of the effective confining stresses during consolidation and the level of static bias. For much of the model a Kc value of 1.8 prevailed. Thus the relationship for the residual pore pressures suggested by Chern (1981) would predict excess pore pressures of approximately 75 percent of the effective confining pressure. These pore pressures are utilized in a static stress analysis to determine the modulus reduction and the earthquake induced deformations.

The reduced modulus factors or the pore pressures determined for each of the three modulus reduction approaches were substituted into a finite element static stress stability analysis to calculate the dynamically induced deformations. Figure 5-11 shows the finite element grid used conjunction with the computer program SOILSTRESS to perform the analyses. During the analyses, the bulk modulus factor was kept high to simulate the undrained behavior of the model sand. The effect of changes in the geometry of the slope on the driving shear forces during deformation was included by modifying the element geometry during the analysis.

5.1.3 Results of Deformation Predictions

The deformations of the model tailings slope predicted by each of the three modulus reduction approaches for the model

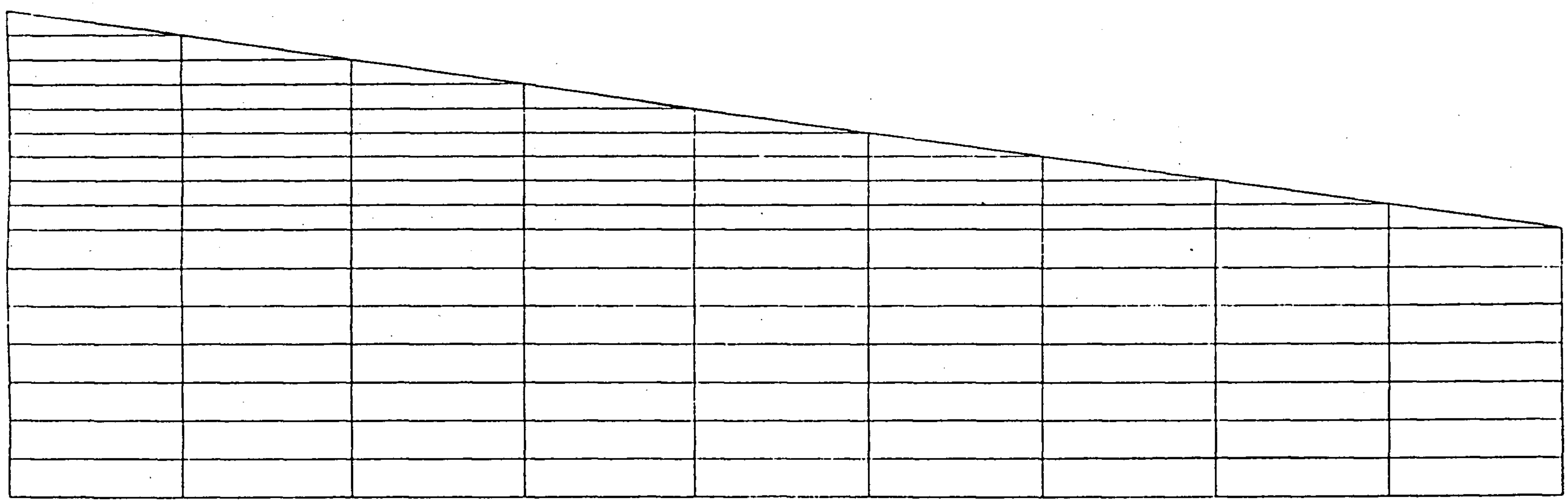


FIGURE 5-11 Finite Element Grid of Tailings Model used in Static-Stress Analysis

test with an 8 degree slope and a peak acceleration of .08g are shown in Figures 5-12, 5-13 and 5-14. The deformations determined by the post-cyclic modulus approach and the pore pressure approach are much smaller that those observed in the model slope and have been multiplied 30 and 300 times, respectively, to arrive at comparable deformations. All approaches reproduce approximately the pattern of deformations observed in the model.

The deformations predicted by the cyclic strain approach using an average k_g value of .10 are very similar to the model deformations. The horizontal displacements agree very well but the slope flattening movements involving settlements in the upper slope and vertical rise in the lower slope are only about half of those observed in the model. This lack of agreement results because totally undrained conditions apparently did not exist within the model slopes, allowing some volume changes to take place. The modulus reduction analysis was performed assuming that no volume changes occurred. This assumption initially appeared reasonable because the volume of sand undergoing downward movements in the upper part of the model slope was comparable to that undergoing upward movements in the lower slope. However, the displacement pattern depicted by the silica beads showed a decrease in volume in the upper slope of approximately 6 percent. For the entire model, the volume reduction was only about 1 percent. Hence an increase in the volume of sand in the lower slope must also have occurred. Figure 5-15 shows the deformations predicted when the bulk

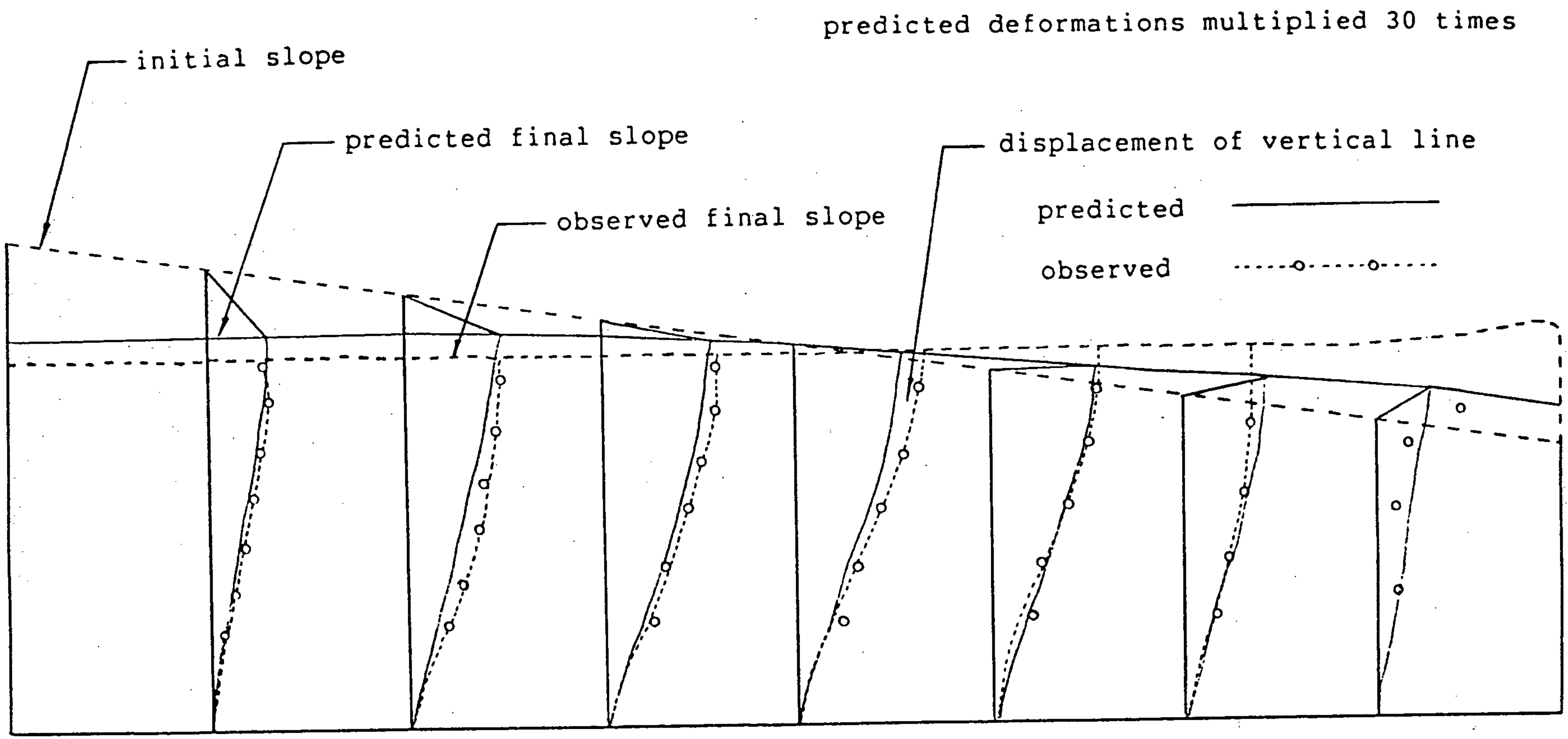


FIGURE 5-12 Deformations Predicted by Post-Cyclic Modulus Approach

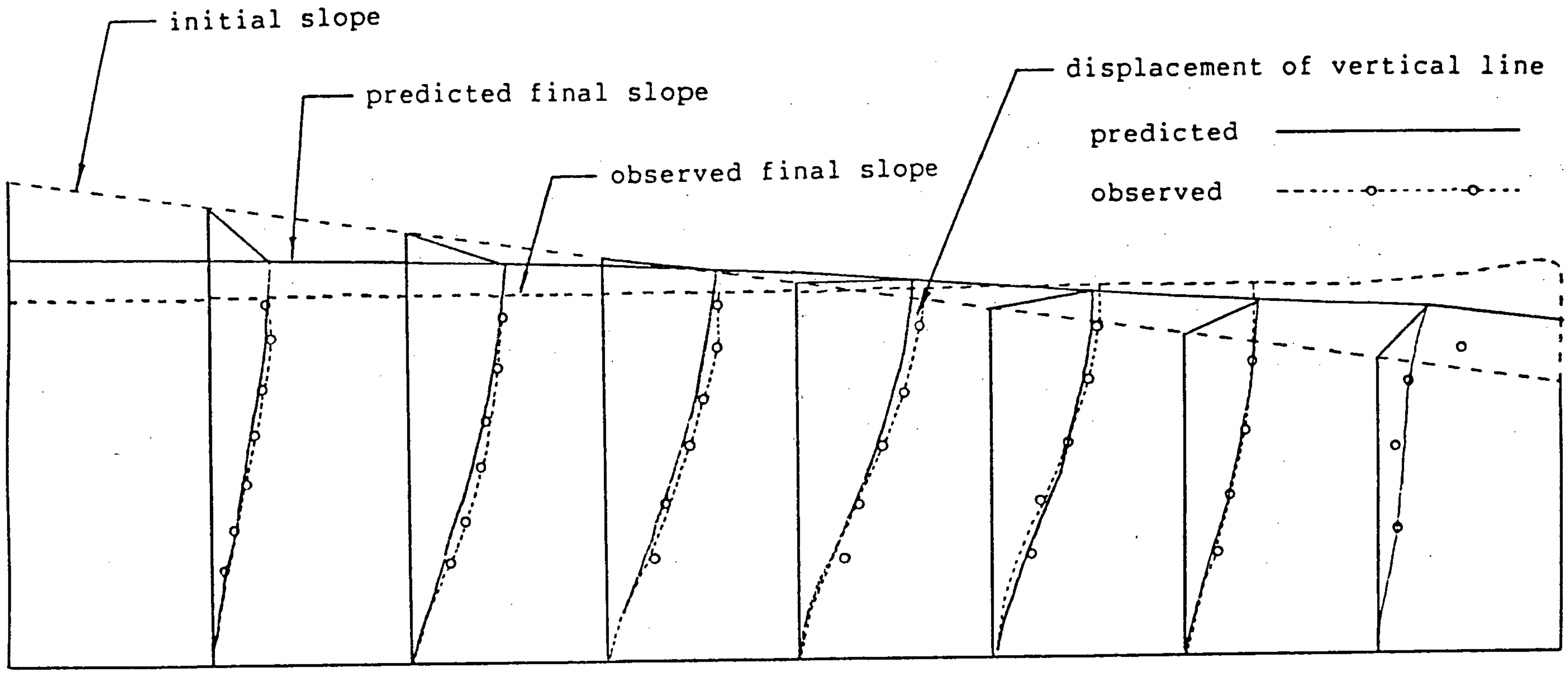


FIGURE 5–13 Deformations Predicted by Cyclic Strain Approach

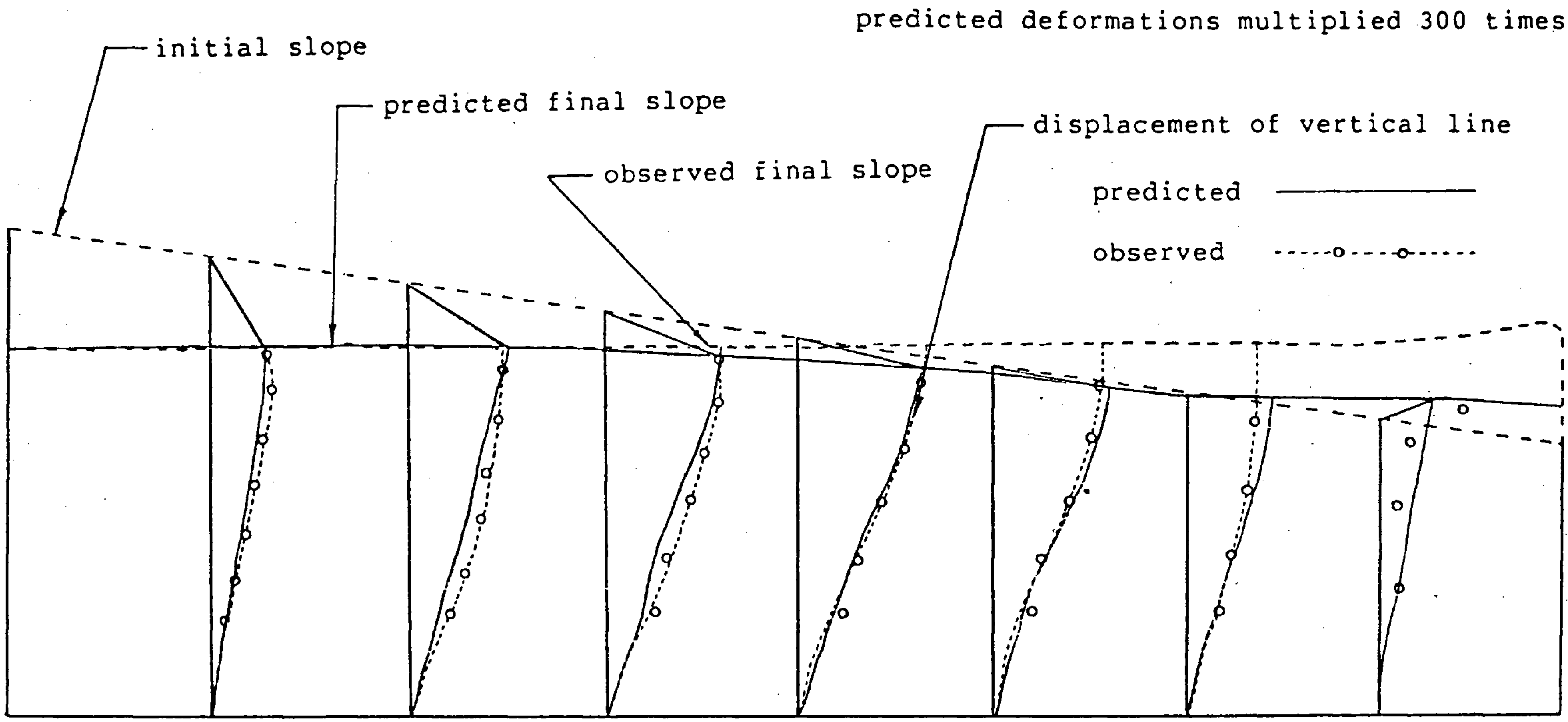


FIGURE 5–14 Deformations Predicted by Pore Pressure Approach

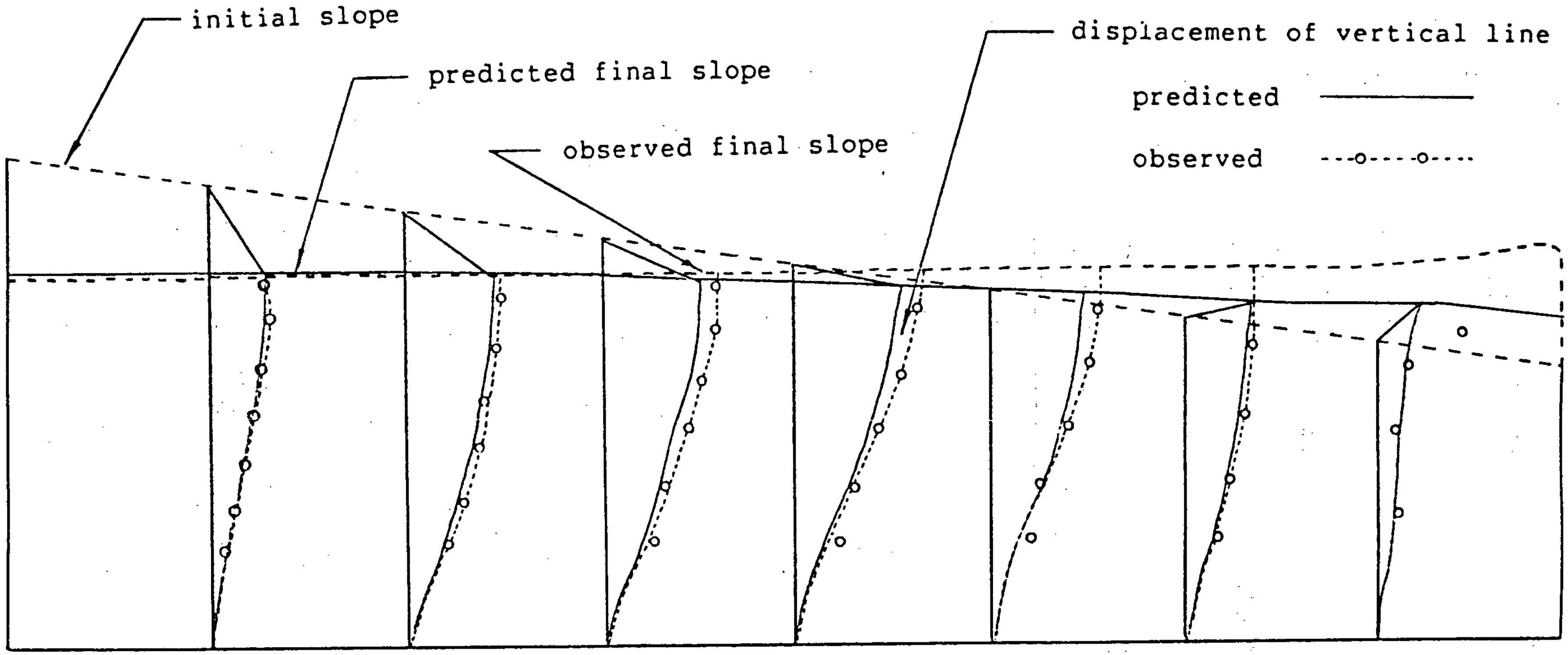


FIGURE 5-15 Deformations Predicted by Cyclic Strain Approach with Volume Change Correction

modulus was permitted to reduce in the upper slope to account for the observed changes in volume. The bulk modulus used in the lower slope was not permitted to reduce but no attempt was made to simulate the observed volume expansion. The deformations predicted using this correction for volume change simulate the slope flattening movements in the upper slope but cannot duplicate the rise of sand in the lower slope. Apart from this minor problem of predicting volume changes when partially undrained conditions exist, the cyclic strain approach appears capable of providing very good predictions of the observed displacements, both in the magnitude of the movements and in the pattern of deformation. The failures of the other two approaches to provide realistic deformation predictions result from their inability to account for fundamental aspects of the earthquake loading or soil behavior.

The failure of the post-cyclic modulus approach to predict deformations of the appropriate magnitude results because such a method ignores the influence of earthquake duration on the generation and accumulation of strains. Implicit in the use of the post-cyclic stress-strain relationship is the assumption that the observed strains are caused by the static shear stresses during a single phase of strain development. However, the reduction in the stiffness of the soil resulting from cyclic loading is not sufficient to reproduce the observed strains. Although significant pore pressures had developed during the cyclic triaxial tests, they began to decrease immediately during post-cyclic monotonic loading. Thus the initial modulus for the post-cyclic test did not reduce significantly. By moderate strain levels, the pore pressures existing in the post-cyclic test were very similar to those existing in the pre-cyclic monotonic test and enough strain hardening had occurred that the post-cyclic stress-strain relationship did not differ significantly from the pre-cyclic relationship.

The sequence of loading used in the cyclic triaxial tests was intended to reflect the assumption that the transmission of the earthquake induced shear stresses through the soil would cease when the pore pressures has increased sufficiently to substantially reduce the stiffness of the soil. The observed deformations would, hence, be caused entirely by the static shear stresses and would result from the reduction in the strength and/or stiffness of the soil. However, Finn et al (1976) found through an examination of an accelerogram of the Niigata Earthquake that dynamic accelerations, and hence shear stresses, continue to be transmitted, even after liquefaction, although at a much lower frequency. The Niigata accelerogram, shown in Figure 5-16, indicates that the amplitude of the acceleration after liquefaction approaches the acceleration amplitude prior to liquefaction. This ability of the soil to transmit shear stresses indicates a regain in strength during shear, presumably as a result of a reduction in pore pressures. As the soil shears in response to the reduced modulus, it dilates, regains strength and is again capable of transmitting the earthquake induced shear stresses. Thus the observed deformations result from the accumulation of strains that occur during several cycles of loading in which the pore pressures alternatively rise and fall. The strains occurring during each loading cycle will depend on both the static and cyclic shear stress levels. The magnitude of the cumulative strain depends on the number of cycles of loading which is directly related to the duration of the earthquake.

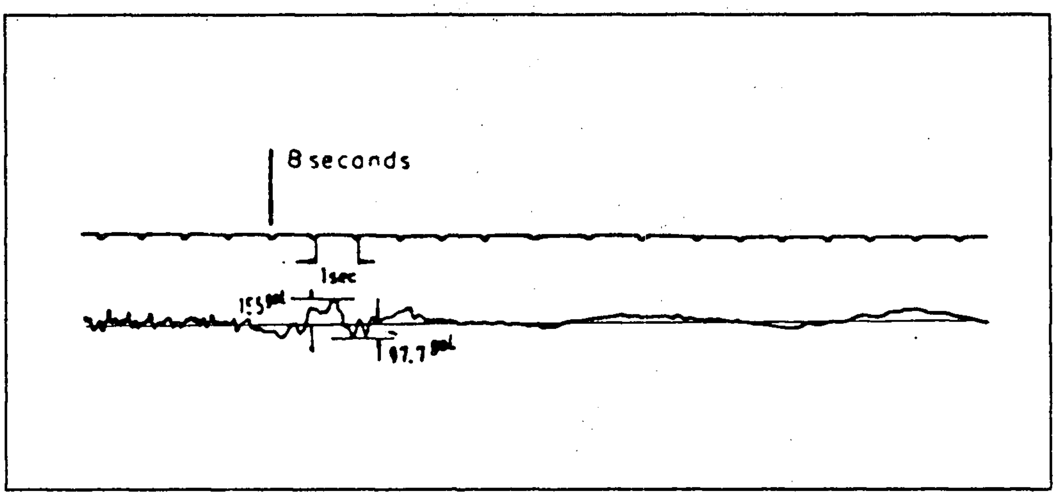


Figure 5-16

Earthquake Accelerogram of Niigata Earthquake (from Finn et al, 1976)

The post-cyclic modulus approach attempts to reproduce the accumulation of strains by using only a single phase of loading. The extent of the deformations that occur during a single phase of loading, however, is limited by strain hardening resulting from dilation and a reduction in pore pressures. It is only when the pore pressures rise again during unloading of the cyclic stress that the soil will soften sufficiently to allow the generation of additional strains during reapplication of the cyclic stress. To reproduce the strain behavior during cyclic loading, an analysis must be able to simulate the accumulation of strains resulting from several cycles of loading. The post-cyclic modulus approach fails to model this progressive strain development because it does not consider the entire cyclic stress history and hence ignores the successive development of strains. Realistic estimates of deformations can only be achieved when the entire duration of the earthquake is considered in the analysis.

An additional source of error in the post-cyclic modulus approach arises because it fails to account for the inertia forces. Although generally considered less significant than the deformations resulting from stiffness degradation, the inertia forces may become important when pore pressures rise enough to bring the soil to a near failure condition. When these conditions develop, any small increase in the shear stress may result in significant deformations. The failure of the post-cyclic modulus approach to include the effects of the inertia forces on the development of strains will, thus, contribute to its inability to predict the observed deformations.

The pore pressure approach also fails to predict the observed deformations. The significant error involved occurs because the shear modulus does not reduce substantially in response to the changes in the confining pressures created by the inclusion of the excess pore pressures. Whereas reductions in the modulus of 1000 times are required to predict the observed deformations of the tailings model, the pore pressure approach predicts reductions of only about 5 times. Even with the reduced strength that also results from the reduction in the effective stresses, the predicted deformations are 300 times less than those observed. As for the post-cyclic modulus approach, the problems with the pore pressure approach appear to lie in its failure to model the successive development of strains during cyclic loading, and in its neglect of the effects of the inertia forces. At the reduced strength used in the pore pressure approach, the inertia forces become significant. If the increase in pore pressure is enough to make the soil just barely stable, any small increase in shear stresses can cause substantial deformations. Repeated cycles of loading during which the strength of the soil is reached would lead to the accumulation of the significant strains observed during cyclic loading. The failure of the pore pressure approach to model this progressive strain development and to include the influence of the inertia forces during the successive periods of loading causes the large error in the predicted deformations.

The effectiveness of the cyclic strain approach appears to be related to its ability to model the gradual accumulation of strains caused by successive cycles of loading and by its inclusion of both the effects of soil softening and inertia forces on the development of the dynamically induced strains. Because the laboratory tests are performed at the in-situ stress levels, the pore pressure response and the resulting soil behavior is indicative of the soil response expected in the field. In addition, the influence of the inertia forces and the earthquake duration are included in the analysis by applying the equivalent number of uniform stress cycles to the laboratory samples and using the observed strains to compute the modulus reduction. The approach, thereby, accounts for the significant aspects of the soil response expected in the field and provides realistic predictions of the earthquake induced deformations.

5.2 Analysis of the Upper San Fernando Dam

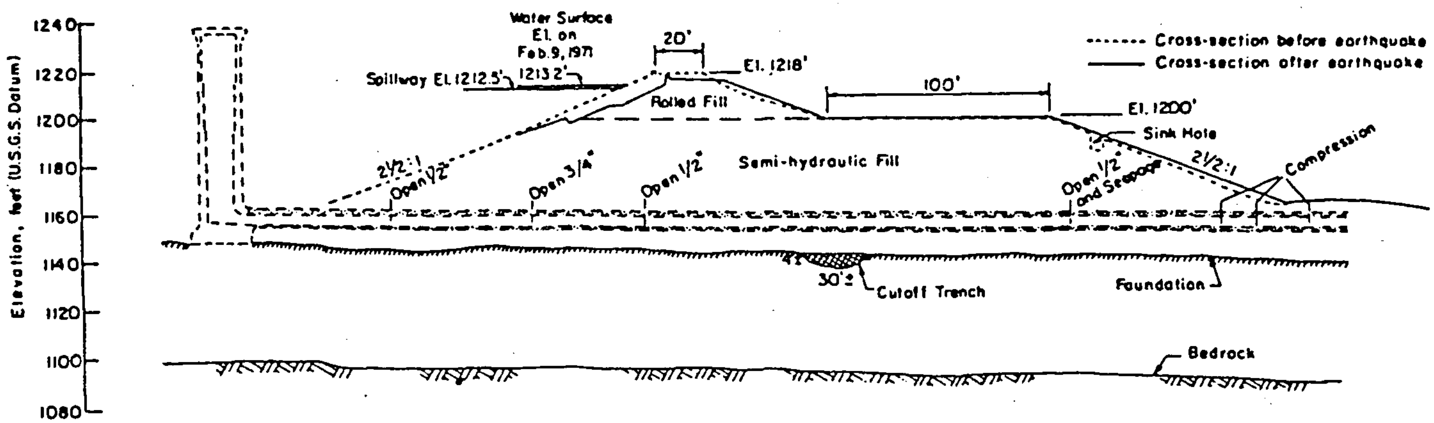
The analysis of the tailings slope model indicated the ability of the modulus reduction approach to predict deformations of earth structures resulting during earthquake excitation. However, such an analysis was relatively simple since it involved only one type of soil whose properties were uniform throughout the model slope. In addition the entire slope experienced substantial pore pressure rise and the softened soil was not constrained by soils which retained a significant portion of their stiffness during the dynamic excitation. The ability of the modulus reduction approach to predict the earthquake induced deformations of earth structures comprising several different soil types with varying properties and differing response to earthquake excitation should be examined. The movements of the Upper San Fernando Dam during the San Fernando Earthquake of 1971 provide an appropriate field history.

5.2.1 Description of Dam and Earthquake Deformations

The Upper San Fernando Dam is an hydraulic fill dam located in the San Fernando Valley of southern California. It was constructed between 1915 and 1925 directly on top of alluvial deposits of stiff clays and clayey gravels. The dam is 58 feet high, consisting of 40 feet of hydraulic fill material topped with an 18 foot cap of rolled fill. The completed section of the dam has a 2.5:1 concrete paved upstream slope, a 20 foot wide crest and a 2.5:1 downstream slope. A 100 foot long berm comprised of hydraulic fill material forms a substantial part of the dam between the crest and the downstream toe.

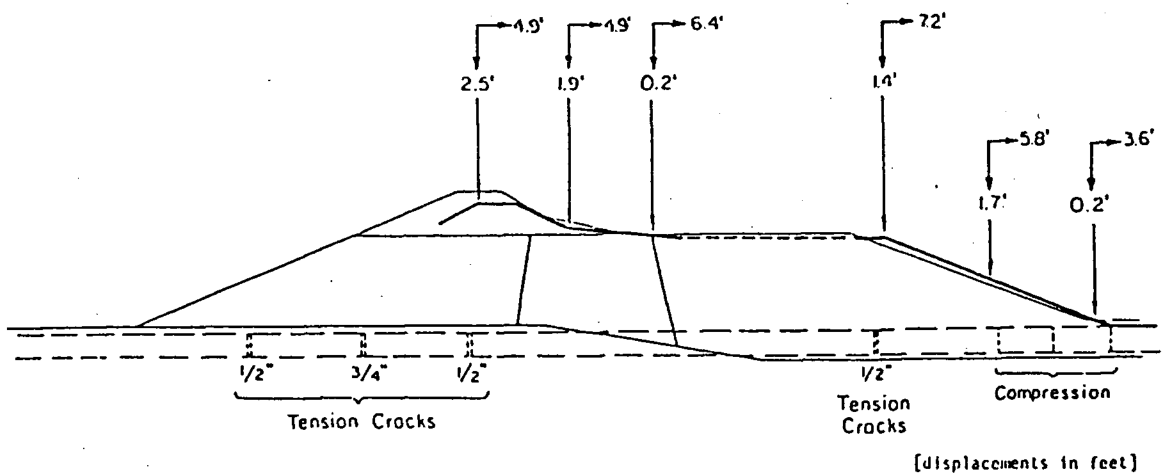
During the early morning of February 6, 1971, the San Fernando Valley region was shaken by an earthquake having a Richter magnitude of 6.6. The epicenter of the earthquake was located 8.5 miles to the northeast of the dam and had a focal depth of 8 miles. Although no major failure of the Upper San Fernando Dam occurred, substantial downstream movements involving the entire downstream slope and the visible part of the upstream slope were observed. The crest of the dam moved 5 feet laterally downstream and settled 3 feet. Measurements of deformations made from surface monuments and from the outlet conduit at the base of the dam indicated that the entire upper part of the dam participated in the downstream movement. Figure 5-17, which shows two cross-sections of the dam, indicates the extent of the movements which occurred.

The substantial downstream movements of the dam created



(from Seed et al, 1973)

(a) Cross-section showing displaced profile of dam



(b) Displacements measured after earthquake

FIGURE 5–17 Deformations of Upper San Fernando Dam during Earthquake several large vertical longitudinal cracks in the upstream slope that extended over almost the entire length of the dam. Another large vertical crack was observed midway down the downstream slope of the upper rolled fill section of the dam. These cracks are extensional features resulting from the greater downstream displacements within the berm relative to the crest region of the dam.

On the downstream slope beyond the berm, deformations tended to decrease towards the toe of the slope. A 2 foot high pressure ridge developed at the downstream toe as a result of the movements. A survey of the interior of the outlet conduit indicated compressional failures near the toe of the dam. Extensional features consisting of several one half to three quarter inch cracks in the conduit were observed in the central and upstream portions of the dam. Measurements of movements at the conduit level were considerably less than those observed on the surface. This reduction appears to indicate that the major movements occurred within the fill above the conduit, although slippage of the fill along the outside edge of the conduit could also account for the observed relative displacements.

Some lateral spreading of the dam was indicated by a transverse compression crack in the paved roadway across the crest of the dam and by damage to the concrete spillway wall at the east abutment. The lateral displacements, however, were considerably less significant than the transverse displacements.

Piezometer readings taken after the earthquake indicated maximum pore pressure increases of 8.5 to 17 feet of water. However, the first readings were not taken until 24 hours after the earthquake, allowing some dissipation of the excess pressures. In addition, in the center of the dam, the pore pressure increases were so large that the water spilled over the tops of the piezometer casings. Thus the maximum pore pressures reached during the earthquake could have been significantly higher than those recorded. In the area below the downstream toe, very high pore pressures apparently occurred in the loose silty sand fill located in that area, causing the formation of several sand boils.

5.2.2 Previous Investigations

After the earthquake, an extensive study of both the Upper and Lower San Fernando Dams was made by Seed et al (1973). In-situ and laboratory soil tests were performed to determine the characteristics of the soils forming the dams and their foundations. Soil response to dynamic loading was investigated by cyclic triaxial tests. A dynamic analysis was then performed to identify possible areas within the dam where liquefaction or cyclic mobility would occur and to estimate the earthquake induced deformations.

Figure 5-18 presents an idealized section of the Upper San Fernando Dam delineating the four major zones of soil in the dam. Because the alluvium and the hydraulic fill soils formed the major part of the dam and because their characteristics

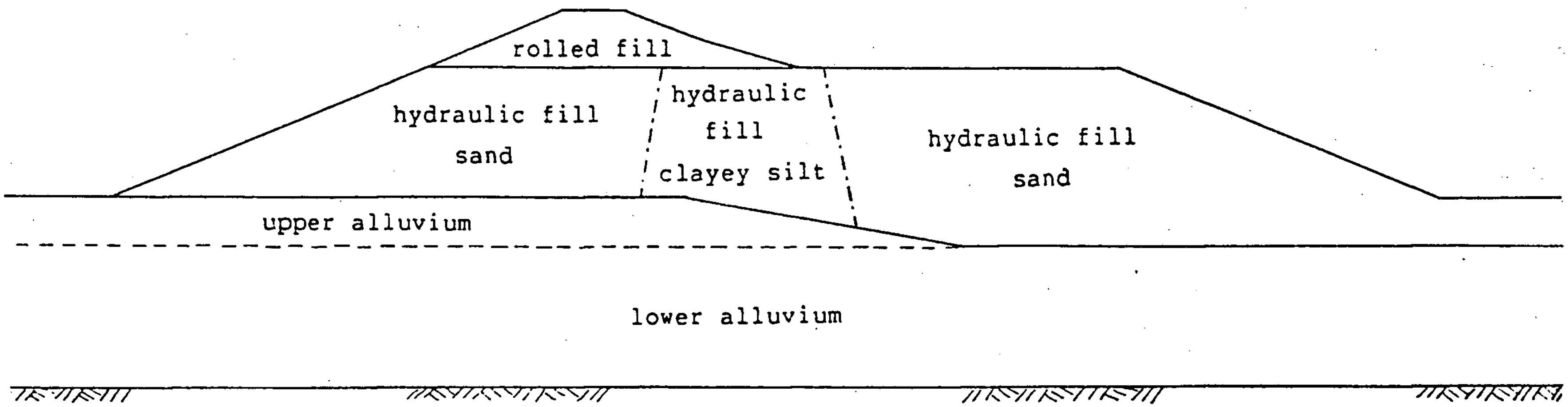


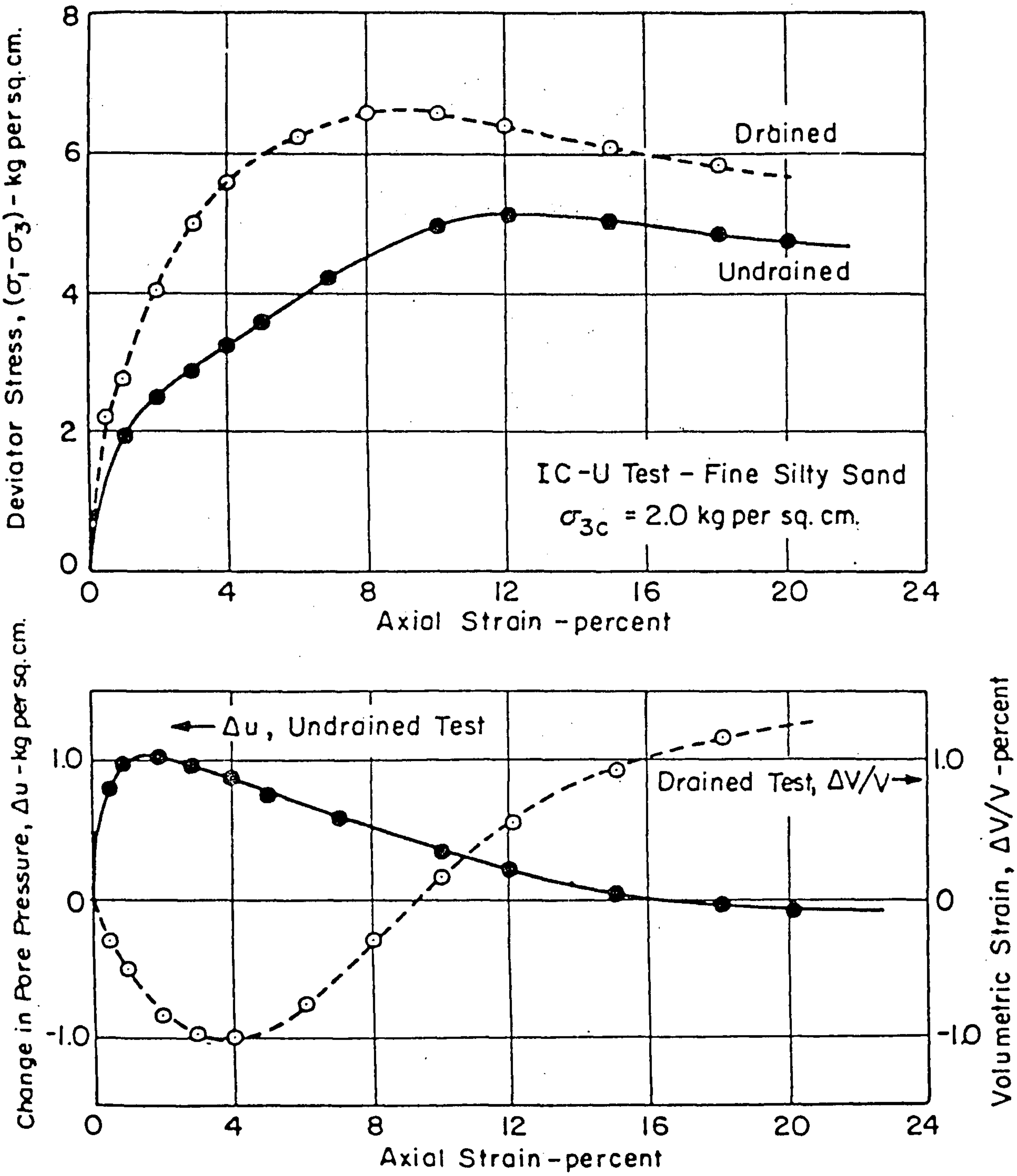
FIGURE 5–18 Major Soils Types in Upper San Fernando Dam

would tend to control the deformations of the dam, the soil testing program was confined to these soils.

Both the hydraulic fill and the alluvium consisted primarily of fine to coarse sand, although clay layers were common in both soils. The hydraulic fill consisted of layers of fairly clean sands and silty to clayey sands with occasional layers of clay. The layering was most pronounced in the outer parts of the embankment, becoming less distinct near the central part of the dam. The outer embankment materials tended to be the coarsest, with the grainsize decreasing towards the fairly homogeneous fine sandy silt of the inner core. The hydraulic fill was loose to medium-dense with an average relative density determined from field and laboratory compaction tests of 54 percent. N values were very low in the silt and clayey core of the central portion of the hydraulic fill, but were somewhat higher in outer sands and gravels of the shell.

The alluvium tended to be slightly coarser than the hydraulic fill and was better graded. It had an average relative density of 67 percent, some 10 percent higher than the hydraulic fill. Shelby tube samples of the alluvium identified it as a very heterogenous soil ranging from layers or pockets of clay to layers or pockets of sands and gravels.

Static loading triaxial tests were performed on undisturbed samples of both the hydraulic fill and the alluvium. Figure 5-19 shows a typical response of the hydraulic fill to drained and undrained loading. After an initial period of compression



(from Seed et al, 1973)

FIGURE 5–19 Typical Response of Hydraulic Fill in Drained and Undrained Triaxial Tests the fill tended to dilate near and after failure. No significant strength loss after failure was indicated by the tests. Table 5-2 presents the soil parameters determined from the test data obtained by the State of California, Department of Water Resources. The parameters were determined from drained triaxial tests on undisturbed samples of the hydraulic fill and alluvium.

Table 5-2

Soil Parameters for Upper San Fernando Dam Soils

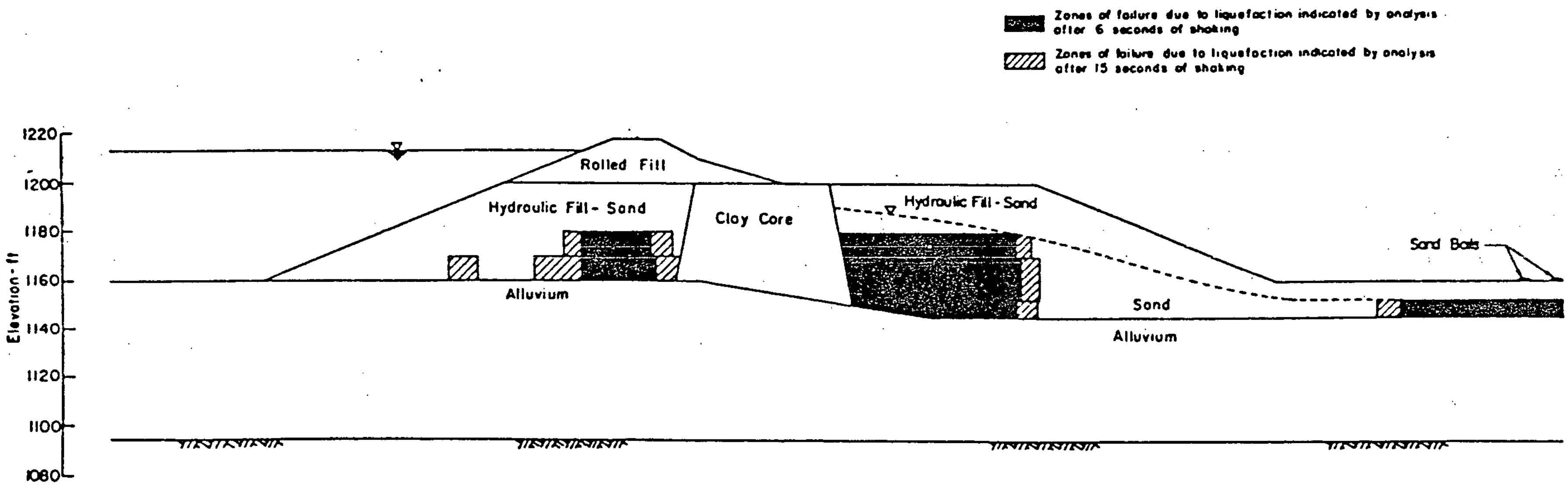
	fill	alluvium
shear modulus factor k_g	105	120
shear modulus exponent n	.85	.64
reduction ratio R_f	.73	.71
angle of friction ϕ	38.0	40.0
change in friction angle $\Delta\phi$	1.4	2.0

Cyclic loading tests were also performed on both soils. However, because the majority of the observed movements appeared to occur within the hydraulic fill and because the fill was more susceptible to pore pressure rise than the alluvium, the testing program concentrated on determining the response of the hydraulic fill to cyclic loading. None of the samples of the hydraulic fill experienced a true liquefaction failure involving a rapid development of large strains. Instead, a transient liquefaction condition resulting in the progressive development of large strains occurred. This behavior is characteristic of cyclic mobility rather than liquefaction which involves a loss in strength. Seed presented extensive data on the levels of cyclic shear stresses required to cause specific strain levels within the soil for various levels of confining pressure and static bias.

On the basis of the cyclic triaxial tests and a dynamic analysis, Seed identified several areas where initial liquefaction and 5 percent strain would occur within the dam and concluded that the development of large strains would be expected even in the outer part of the embankment. Figure 5-20 shows the areas that Seed considered to have liquefied within the hydraulic fill. The term "liquefaction" as used by Seed refers to the development of 5 percent strain in triaxial test samples. The alluvium was not considered to have developed significant pore pressures because of its higher density and was not considered in Seed's analysis. Based on the results of these tests, Seed determined the strains that would occur in the hydraulic fill if samples were subjected to cyclic triaxial tests that reproduced the field stress conditions.

5.2.3 Modulus Reduction Analysis

The analysis of the movements of the Upper San Fernando Dam was made by the proposed cyclic strain method of modulus reduction. Although extensive investigations and soil tests were made by Seed et al (1973) after the earthquake, their data is not in the appropriate form for such an analysis. Since no



(from Seed et al, 1973)

FIGURE 5–20 Liquefied Areas of Dam

further testing could be performed, the analysis relied on the strain potentials presented by Seed to estimate the modulus reduction by the cyclic strain approach.

To determine the appropriate modulus reduction by the cyclic strain approach the in-situ shear stresses determined from a static stress analysis were divided by the shear strain potentials recommended by Seed. These strain potentials were assumed to represent the shear strains which would occur in laboratory tests performed at the field stress conditions. Although Seed performed many cyclic loading tests, both under isotropic and anisotropic conditions, he was not primarily concerned with determining the distribution and magnitudes of the strains that would potentially occur throughout the dam. To determine such strains the stress conditions existing at specific locations in the dam would have to be duplicated in the cyclic loading tests. The strain potentials that Seed determined to occur throughout the dam are shown on Figure 5-21. The criteria that Seed used in selecting these strain potentials were not specified. In the central part of the dam, use of strain potentials in the 50 to 70 percent range resulted in modulus reductions of 500 to 3000 times. Elements in the outer embankment on the downstream slope were not saturated and, hence, not expected to experience any modulus reduction as a result of soil softening. No modulus reduction was used in the alluvium because the deformations observed in the outlet conduit indicated that the majority of the movement was confined to the hydraulic fill region of the dam. The alluvium was also 25 to

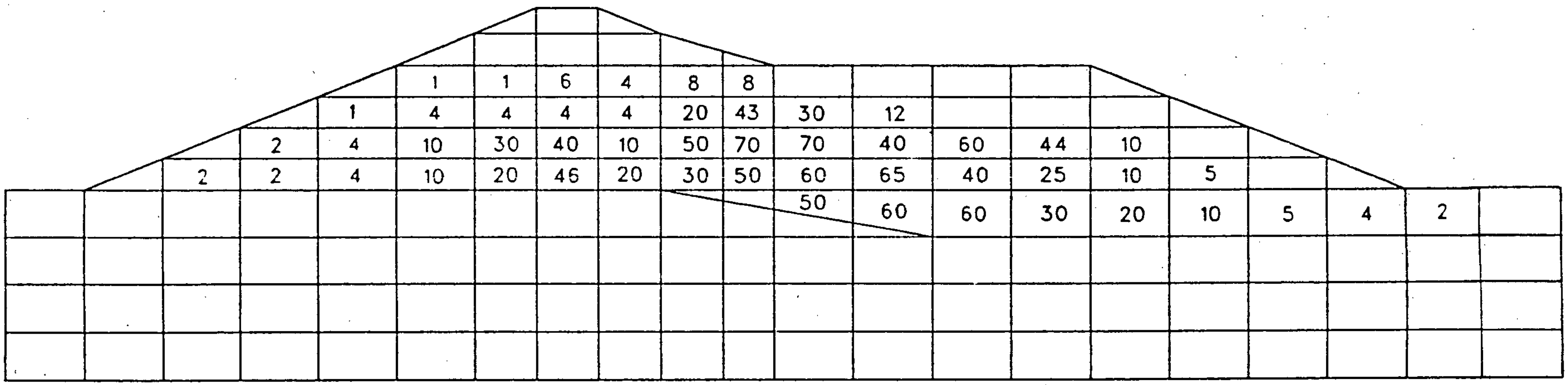


FIGURE 5-21 Shear Strain Potentials

30 percent stronger than the hydraulic fill under cyclic loading. Hence, the magnitude of any modulus reduction in the alluvium would be much less significant than in the fill, although some reduction could occur. No data was available for the upper rolled fill and thus no modulus reduction was assumed in this area.

The finite element program SOILSTRESS was again used to perform the static-stress analysis required for computing the earthquake induced deformations. Figure 5-22 shows the finite element grid used in the analysis. The reduced modulus factors, computed for each element by dividing the in-situ static shear stress by the recommended strain potential, were used in the static stress analysis. Because undrained shear was assumed to occur during the earthquake, the bulk modulus was not permitted to reduce from its initial pre-cyclic value. The geometry of the elements was corrected throughout the analysis to account for any small reduction in the driving stresses that might occur due to slope flattening.

The results of the deformation analysis are shown on Figure 5-23. The crest of the dam was predicted to move downstream 2.5 feet and settle 2.5 feet. Horizontal displacements increased downstream of the crest to 4.3 feet at the start of the berm where an area of extension was indicated by still greater movements further down the berm. After reaching a maximum of 5.0 feet the predicted horizontal displacements decreased to 3.9 feet at the edge of the berm and continued to decrease to 2.3

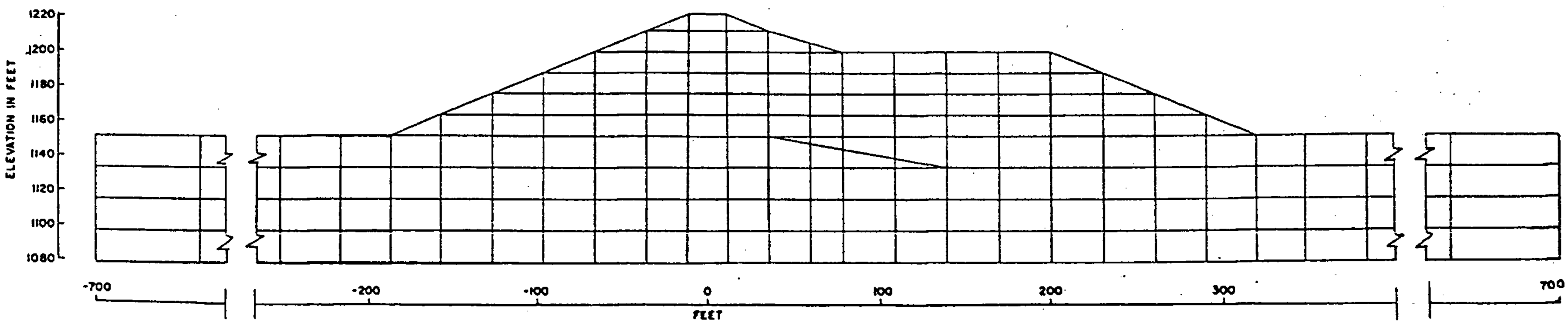


FIGURE 5–22 Finite Element Grid of Upper San Fernando Dam used in Static Stress Analysis

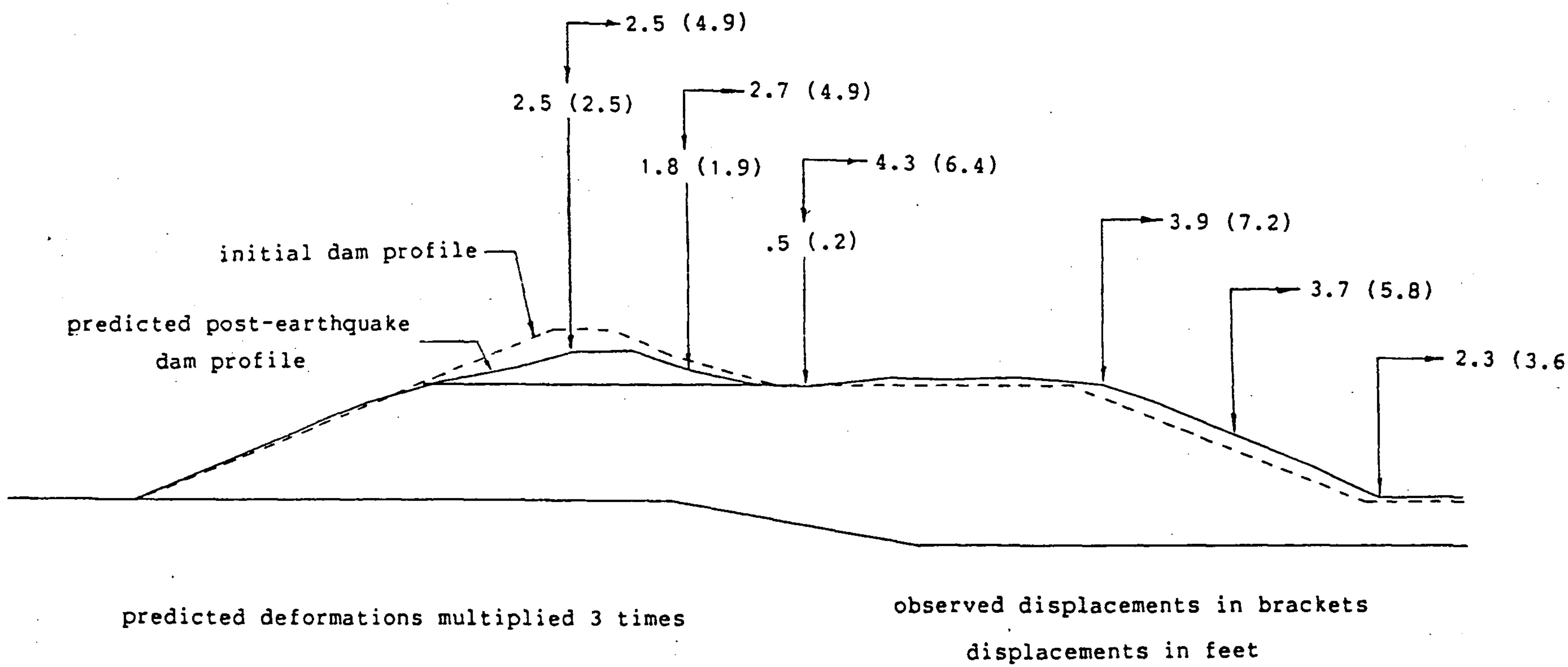


FIGURE 5-23 Predicted Deformations of Upper San Fernando Dam

feet at the toe. Although these deformations are about half of those observed, the pattern of deformation duplicates that of the dam. An area of compression is indicated at the toe of the berm while extension is indicated in the central and upper parts of the berm and dam.

To predict the deformations observed in the dam, the strain potentials used to determine the necessary reduction in the modulus factor have been modified as shown in Figure 5-24. Only elements in which the strain potentials were changed are shown with their new assumed values. The increases in strain potential were only required for a very few elements located near the toe of the dam. The use of higher strain potentials in this area may be justified by the high level of pore pressure development observed to occur just beyond the downstream toe of the dam and because of the higher static shear stresses occurring within the downstream slope of the berm. The deformations predicted using these minor changes are shown in Figure 5-25. The predicted displacements of the dam agree very well with the observed displacements except at the crest. Apart from minor discrepancies, the modulus reduction dynamic analysis predicts deformations that agree with those observed, both in the magnitude of the displacements and in their pattern. Better agreement, even in the crest area, may have been achieved if the appropriate laboratory tests were performed to determine the actual strains to be used for calculating the modulus reduction.

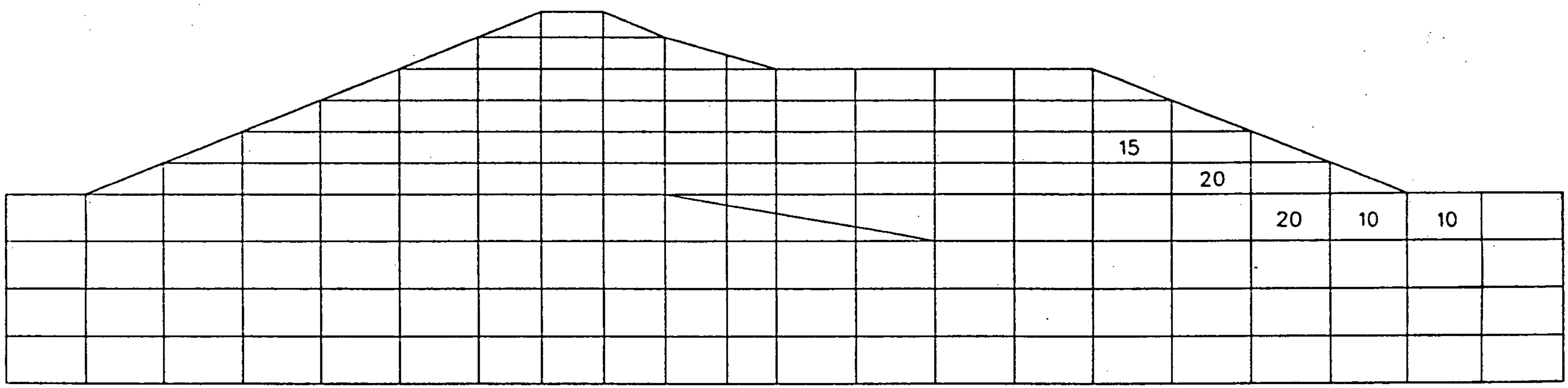


FIGURE 5–24 Required Shear Strain Potentials

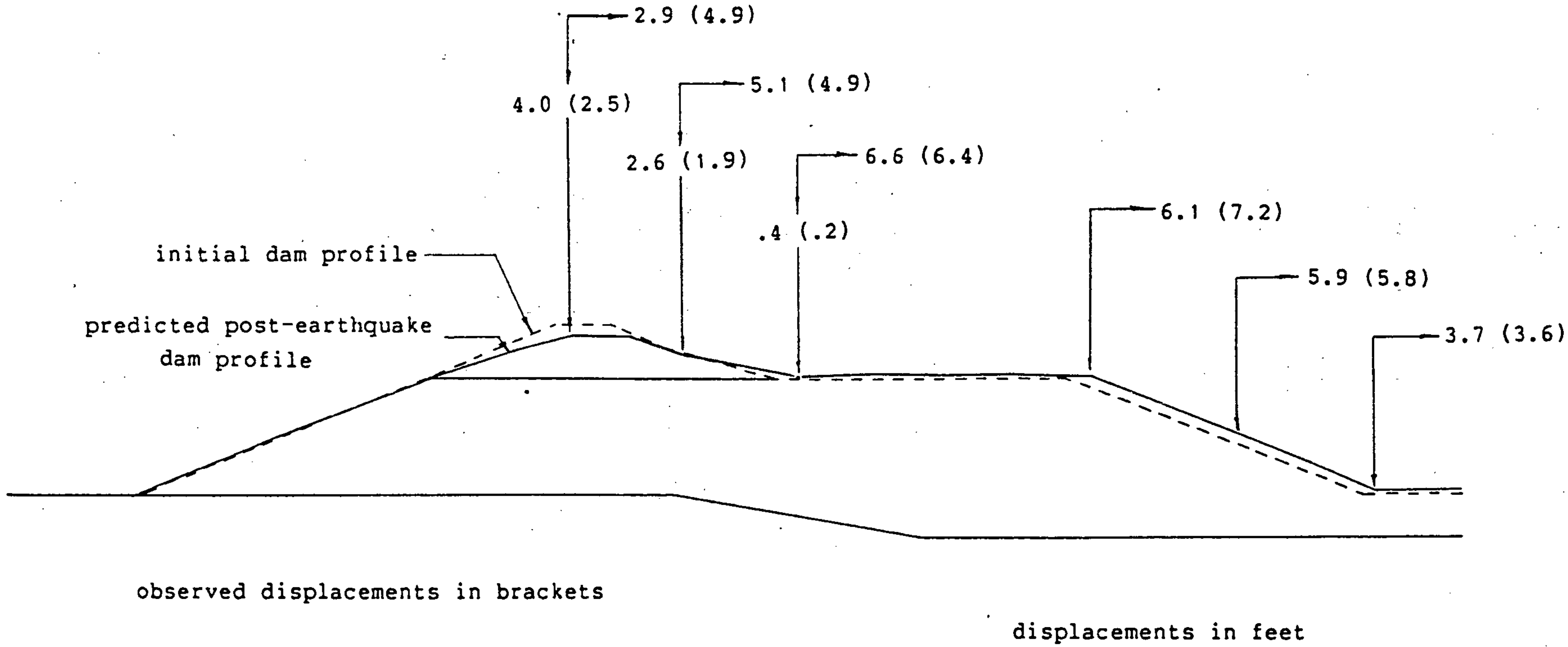


FIGURE 5-25 Predicted Deformations using Required Shear Strain Potentials

CHAPTER 6

SUMMARY AND CONCLUSIONS

The modulus reduction analysis is an effective semi-analytical method of dynamic analysis capable of predicting earthquake induced deformations of a realistic magnitude and pattern. The effects of an earthquake on a soil are represented as a reduction in the stiffness properties of the soil. The analysis is primarily intended for saturated soils which experience significant softening as a result of pore pressure rise. However, because the effects of the inertia forces are also incorporated into the modulus reduction analysis, it can be used effectively for soils which are expected to experience only limited changes in pore water pressure during cyclic loading.

The reductions in the modulus that are required to predict the deformations observed during cyclic loading may be as large as 1000 to 3000 times for soils that develop liquefaction or cyclic mobility as a result of their susceptibility to substantial pore pressure rise. Of the three proposed methods of computing a reduced modulus, only the cyclic strain approach was capable of predicting reductions of such a magnitude. Both the post-cyclic modulus approach and the pore pressure approach failed to predict reductions of sufficient magnitude.

The post-cyclic modulus approach, in which the modulus reduction is determined by comparing the stress-strain curves before and after cyclic loading, fails to predict deformations of sufficient magnitude. This failure is presumably related to the inability of such a method to model the successive development of strains during cyclic loading. The post-cyclic stress-strain relationship that is used for this analysis cannot account for the increase in pore water pressure that develops during the unloading phase of each cyclic stress application. The strains that develop during a single load cycle will be limited by strain hardening, resulting from dilation and a reduction in pore pressures. Only by considering successive periods of strain development in which the pore pressures alternatively rise and fall will an analysis be able to model the significant accumulation of strains observed during cyclic loading.

The pore pressure approach also fails to predict realistic displacements of the tailings model. Even though the inclusion of the excess pore pressures created near failure conditions in the model, the reduction in the shear modulus was substantially lower than that required to predict the observed deformations. The failure of the pore pressure approach to predict realistic deformations results because it ignores the significance of the inertia forces on the development of strains when substantial increases in pore water pressure occur during cyclic loading. When only a minor increase in the pore water pressure occurs, the inertia forces generally do not cause substantial deformations unless they are very large or the soil is very sensitive. However, the inertia forces will become significant when the increase in pore water pressure brings the soil so near the failure condition that any small increase in the driving forces causes substantial deformations. Failure to include the effects of the inertia forces during successive periods of loading results in the inability of the pore pressure approach to predict sufficient modulus reductions and, hence, to reproduce the observed deformations.

The success of the cyclic strain approach in predicting a sufficient modulus reduction lies in its incorporation of the effects of earthquake duration on soil softening and in its inclusion of the inertia forces in the analysis. The strains observed in the laboratory tests performed at the in-situ stress levels with the equivalent number of shear stress cycles, depend on both the magnitude and the duration of the cyclic loading applied. Because the entire effects of the earthquake are represented by an equivalent cyclic loading, the observed behavior will reproduce the soil response expected in the field. The modulus reduction computed from the observed strains will include the effects of soil softening and inertia forces and will be of sufficient magnitude to predict realistic deformations.

The advantage of the modulus reduction analysis is that it uses relatively simple dynamic and static-stress analyses to predict earthquake induced deformations. Only common geotechnical parameters that may be determined from simple triaxial and cyclic triaxial tests are required. The semi-analytical technique permits the inclusion of the effects of pore pressure rise on soil behavior without having to resort to a rigorous and complex effective stress analysis. The proposed method, thus, appears to be an effective and relatively simple method for predicting the deformations of soil structures during earthquake loading.

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