AT-REST AND COMPACTION-INDUCED LATERAL EARTH PRESSURES OF MOIST SOILS

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CIVIL ENGINEERING

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Civil Engineering

(ABSTRACT)

An instrumented oedometer was designed and constructed for the purpose of investigating at-rest and compaction-induced earth pressures in moist soils. The device has a split oedometer ring, and horizontal stresses are measured using load cells which support one half of the ring. Rapid cyclic loading was applied to compacted soil specimens, using a digital pressure regulator and a computer-based data acquisition system. The performance of the device was validated by performing tests on silicon rubber and Monterey sand.

Instrumented oedometer tests were performed on specimens of compacted Yatesville silty sand, with various values of water content and dry unit weight. The results of the instrumented oedometer tests showed that (1) the value of the total stress earth pressure coefficients at rest, $K_0^T$, is about 0.4 for specimens compacted one percent or more dry of optimum, (2) that the value of $K_0^T$ increases very rapidly as degree of saturation increases in the neighborhood of optimum water content, and (3) at water contents above optimum, values of $K_0^T$ are 0.9 to 1.0. Using these results, value of $K_0^T$
for Yatesville silty sand can be estimated based on degree of saturation and relative compaction. Simplified design diagrams were developed for estimating values of $K_0^T$.

Filz's (1992) method for effective stress analysis of compaction-induced earth pressures in partly saturated soils was compared to the results of the instrumented oedometer tests. A method for estimating pore pressures based on the compressibility of the soil and the degree of saturation was used in the analyses. Applying Filz's (1992) modification of Duncan and Seed's (1986) effective stress $K_0$ model, both total and effective lateral pressures were estimated. The calculated values of lateral earth pressure were found to be in fairly good agreement with the results of the instrumented oedometer tests.

Tests were also performed in which specimens were soaked while loaded for the purpose of evaluating the effects of changes in soil moisture on the $K_0$ behavior. Soaking had the effect of releasing soil suction in specimens compacted dry of optimum and allowing positive pore pressures to dissipate in specimens compacted wet of optimum. Significant decreases in total stress $K_0$ values due to soaking were observed for specimens compacted either wet or dry. Changes in effective stress during soaking were inferred from the test results.

The instrumented retaining wall tests on Yatesville silty sand performed by Filz (1992) were analyzed using the proposed models. Total stress analyses provided good agreement with the measured results. Effective stress analyses were also in good agreement with experimental results when a limiting condition was imposed with respect to the ratio between total horizontal and total vertical stress.
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Financial support from Nikken Sekkei and Virginia Tech are gratefully appreciated.

I would like to express my thanks to my wife, Mieko, and my son, Takahisa. Their patience and dedication have made my studies possible. Lastly, this work is dedicated to the memory of my mother who passed away while I was in this course of study.
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CHAPTER 1

INTRODUCTION

Stiff retaining walls on non-yielding foundations are usually designed for at-rest earth pressures, equal to 0.4 to 0.6 times the vertical pressure. However, lateral earth pressures induced by compaction of backfill are sometimes much greater than the at-rest pressures. As a result, retaining structures designed to resist only at-rest earth pressures may be damaged, or they may undergo large lateral deflections, when the backfill is compacted.

A theory for estimating compaction-induced lateral earth pressures was developed by Duncan and Seed (1986) and modified by Filz (1992). This theory has been successfully applied to dry sand, but earth pressures calculated by Filz (1992) for moist Yatesville silty sand did not agree with values measured in instrumented retaining wall tests. It was surmised that the discrepancy between the calculated and measured earth pressures was caused by inaccurately estimated values of pore pressure in the partly saturated soil during loading. The engineering behavior of partly saturated soils were reviewed during this study to develop improved understanding of pore pressures in partly saturated soils and to devise better methods of estimating earth pressures exerted on walls by compacted moist soils.

Within the past thirty years, since Bishop first proposed a rational effective stress equation for partly saturated soils in 1959, progress in the use of effective stress theory for partly saturated soils has been slow. This slow progress is mainly due to the difficulties involved in measuring soil suction. One of the purposes of this research was to develop a method of estimating compaction-induced lateral earth pressures which would not require estimation or measurement of soil suction.
This dissertation consists of nine chapters and seven appendices. Chapter 2 describes and summarizes previous studies of at-rest and compaction-induced lateral earth pressures. The effective stress $K_0$ model developed by Duncan and Seed (1986) and extended by Filz (1992) are described in Chapter 2.

The behavior of partly saturated soils was a very important aspect of this research. Chapter 3 describes and summarizes previous studies of the behavior of partly saturated soils.

Chapter 4 describes the instrumented oedometer that was designed and constructed during this study. Tests performed on silicon rubber and dry Monterey sand to validate the operation of the apparatus are also described in Chapter 4.

Chapter 5 presents the results of the instrumented oedometer tests performed on specimens of moist Yatesville silty sand, and their interpretation with respect to total stresses. Relationships between the value of total stress $K_0$ and compaction density, and water content are discussed. Based on the test results, a method for estimating the total stress value of $K_0$ is proposed.

Filz's (1992) extension of Duncan and Seed's (1986) $K_0$ model is evaluated in Chapter 6 by analyzing the results of the instrumented oedometer tests. The $K_0$ behavior of partly saturated soils is discussed in terms of the concept of effective stress, and a method for estimating pore pressures during one-dimensional compression is presented. Applying the proposed method to Filz's (1992) effective stress $K_0$ model, both total stress behavior and pore pressures during one-dimensional loading can be estimated.

It has been reported that changes in moisture content affect the value of $K_0$ after compaction. The results of tests in which specimens were soaked after compaction to examine the effects of changes in moisture content are described in Chapter 7.
Comparison between calculated earth pressures and the results of Filz's (1992) instrumented retaining wall tests on Yatesville silty sand are presented in Chapter 8.

The findings of this research are summarized in chapter 9. Possible future studies are also discussed.

Appendix A describes the data acquisition and test control system used for the instrumented oedometer tests. The computer program used for data acquisition and test control is described in Appendix B. Shop drawings for the instrumented oedometer are shown in Appendix C. Appendix D presents some mathematical forms for pore pressure parameters for partly saturated soils, and discusses the assumptions and the mechanical basis for the expressions. Appendix E describes VanGenuchten's model, which is used to relate soil suction to degree of saturation. The results of conventional laboratory tests on Yatesville silty sand and Monterey sand are summarized in Appendix F and Appendix G.
CHAPTER 2

REVIEW OF PREVIOUS STUDIES OF AT-REST AND
COMPACTION-INDUCED LATERAL EARTH PRESSURES

2.1 Introduction

Sehn's (1990) review of earth pressures has been brought up to date during the
course of this research. Table 2-1 contains a summary of the studies reviewed. For each
reference, the nature of the study and the findings and conclusions are summarized.

A clear picture of the mechanics of at-rest earth pressures and compaction-induced
earth pressures emerges from the information in Table 2-1. The most important facts
relating to these issues are summarized in following sections.

2.2 At-rest Earth Pressures

Almost all of the experimental studies that have been performed to investigate at-
rest pressures have resulted in essentially the same picture of the behavior of soils under
condition of no lateral strain (Kjellman and Jacobson, 1955; Bishop and Henkel, 1957;
Hendron, 1963; Brooker and Ireland, 1965; Wright, 1969; Campanella and Vaid, 1972;
Ofer, 1982; Senneset, 1989; Sehn, 1990)

The essential aspects of this behavior are illustrated, for example, by the results of
tests performed by Wright (1969) which are shown in Fig. 2-1. The tests were performed
on specimens of Monterey sand in a triaxial apparatus with a sensitive strain indicator
band around the specimen. As the axial stress increased, the lateral stress was adjusted to
maintain zero lateral strain to a high degree of accuracy.

From the results shown in Fig. 2-1, the following characteristics of the behavior of
Monterey sand under no lateral strain condition can be seen:
<table>
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<th>Year</th>
<th>Author</th>
<th>Nature of Study</th>
<th>Principal Findings and Conclusions</th>
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<tr>
<td>1920</td>
<td>Terzaghi, C.</td>
<td>Development of a retaining-wall facility to measure lateral pressures. Lateral</td>
<td>It was found that the coefficient of earth pressure at rest was equal to 0.42, and was independent of</td>
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<td>pressures were deduced from the friction force on a steel tape. Five types of</td>
<td>the density of the backfill. The coefficient of active earth pressure was found to be as small as 0.05.</td>
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<td></td>
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<td>sand were used for backfill materials in the tests.</td>
<td></td>
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<tr>
<td>1936</td>
<td>Kjellman, W.</td>
<td>Development of a cubical testing apparatus where loads acting in each principal</td>
<td>One-dimensional compression tests performed using the apparatus gave greater compression but less</td>
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<td>direction were controlled separately. The principal stresses were applied to a</td>
<td>permanent compression than conventional oedometer tests. It was considered that this was due to</td>
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<td></td>
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<td>specimen through plungers and reaction plates. Five types of tests, (1) triaxial</td>
<td>friction in the oedometer. Lateral pressures were measured in the one-dimensional compression tests.</td>
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<td></td>
<td></td>
<td>compression, (2) one-dimensional compression, (3) biaxial compression, (4)</td>
<td>It was found that the ratio of the lateral pressures to the vertical pressures varied from 0.5 to 1.5.</td>
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<td>two-dimensional rupture, and (5) three-dimensional rupture, were performed on</td>
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<td>specimens of dry sand.</td>
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<td>1941</td>
<td>Binnie, G. M. and</td>
<td>Development of an apparatus for measuring lateral</td>
<td>The lateral pressures increased as moisture content increased. Remoulded specimens gave a higher</td>
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<td></td>
<td>Price, J. A.</td>
<td>pressures in clay samples. The lateral pressures were deduced from the</td>
<td>ratio of lateral to vertical pressure than undisturbed specimens at the same water content. The</td>
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<td>resistance to rotation of a shaft between two bearing surfaces.</td>
<td>lateral pressure peaked when the degree of consolidation was about 85 to 90 percent, and it</td>
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<td>decreased to the initial value, approximately 60 percent of the vertical pressure.</td>
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<tr>
<td>1950</td>
<td>Bishop, A. W.</td>
<td>A triaxial testing apparatus for evaluating K₀ was described. Tests were</td>
<td>The loading ram of triaxial testing apparatus had the same diameter as that of specimens; therefore,</td>
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<td>conducted on specimens of saturated sand.</td>
<td>the specimens yielded laterally only if fluid was expelled from the cell. It was found that K₀</td>
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<td>tends to increase with increasing pressure.</td>
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Table 2-1. Cont. Literature Review on At-Rest and Compaction-Induced Lateral Earth Pressures

<table>
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<tr>
<th>Year</th>
<th>Author</th>
<th>Nature of Study</th>
<th>Principal Findings and Conclusions</th>
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<tbody>
<tr>
<td>1950</td>
<td>Rowe, P. W.</td>
<td>Studies of lateral earth pressures on a rigid wall. A 2x2x2 ft. cubic bin containing a 2 ft. square steel plate on which two pressure gauges were mounted. The plate was movable by adjusting screws. A backfill of dry sand was used for the tests. Surface loads were applied and earth pressures during application of the surface pressures and after their removal were measured.</td>
<td>It was concluded that Boussinesq's solution provides accurate estimates of vertical pressure distribution due to surface loads. Diagrams of lateral pressure distributions for various types of surface loads were developed. It was concluded that $K_0$ varies with depth and surcharge pressure.</td>
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<tr>
<td>1954</td>
<td>Rowe, P. W.</td>
<td>A new stress-strain theory for cohesionless soils was discussed. Results of shear box tests and triaxial tests on cohesionless soils were studied to evaluate the significance of various factors the stress-strain behavior. A new type of confined compression testing device was developed to investigate the variation of $K_0$ with depth. Air pressure was applied through a flexible membrane to specimens with a diameter of 10 inches and heights of 2 to 5 inches.</td>
<td>It was concluded that the mobilization of friction in cohesionless soils which are not in failure is a function of the strain on a &quot;slip plane&quot;. The proposed function depends on confining pressure, density, strain history, loading direction, grading and strain rate. It was concluded that the conventional oedometer does not provide good results for sands because of their rigid plate loading system and side friction. The new compression testing device provided the compression characteristics satisfactorily. It was concluded that friction angles for the state where the soil is not failing are smaller than those for failure problems. The value of $K_0$ was obtained by substituting the partially mobilized friction angle into the equation for $K_0$.</td>
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<tr>
<td>1955</td>
<td>Kjellman, W. and Jacobson, B.</td>
<td>A large compressometer and a large direct shear apparatus were developed to study stress-strain characteristics of coarse gravels. The large compressometer consisted of a series of steel rings with an inner diameter of 50 cm and a height of 5 cm. The specimens height was 100 cm maximum.</td>
<td>In the compressometer tests, horizontal stress was computed from the circumferential deformation of the steel rings measured either by dial gauges or by strain gauges. The measurement by the dial gauges was found to be inaccurate. Values of Poisson's ratio were higher for unloading and reloading than that for virgin loading, except for small loads.</td>
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</table>
Table 2-1. Literature Review on At-Rest and Compaction-Induced Lateral Earth Pressures

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<tr>
<td>1957</td>
<td>Bishop, A. W. and Henkel, D. J.</td>
<td>$K_0$ triaxial tests were classified into two types: (1) undrained tests on partly saturated specimens for evaluating Skempton's (1954) pore pressure parameter $B$, coefficient of earth pressure at rest $K_0$, and compressibility in one-dimensional compression; (2) drained tests on saturated or partly saturated specimens to determine the values of $K_0$ and $n_p$. A triaxial testing cell was modified for evaluating $K_0$ values. A simple lateral strain indicator was attached to the the mid-height of the specimen to register the change in diameter. The vertical stress and the horizontal stress were adjusted to maintain the no lateral strain condition.</td>
<td>It was found that the value of $K_0$ approached to unity on unloading. The compressibility under $K_0$ conditions was found to be less than that for isotropic conditions. The pore pressure behavior, effective stress path, and volume change behavior were found to be hysteric. The value of $K_0$ was calculated as the ratio of the increments in effective stress because of the initial soil suction existing in the specimens. The results were similar to those of undrained tests.</td>
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<tr>
<td>1957</td>
<td>Sowers, G. F., Robb, A. D., Mullis, C. H., and Glenn, A. J.</td>
<td>Residual lateral pressures induced by compaction were studied. Laboratory tests were performed on specimens of four different soils. The soils were compacted into a cylinder with a diameter of 4 inches by both static and dynamic compaction. Field tests on sand backfill were performed using a concrete lined test pit 8 feet long, 5 feet wide, and 5 feet deep. Field tests on clay backfill were performed using a retaining wall with a height of 6 feet.</td>
<td>It was found that the upward movement was restricted by friction when compactive pressures were removed. Therefore, higher residual lateral earth pressures were maintained. The laboratory tests showed that the residual pressures in clay increased as compactive effort increased and as moisture contents decreased. Field tests showed that the lateral pressures for compacted backfills were higher than those for the uncompacted backfills. The measured residual pressures for the compacted backfills were greater than the calculated at-rest pressures. The residual pressures in the sand backfills did not change significantly with time after compaction. However, those in clay backfills decreased with time.</td>
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<tr>
<td>1958</td>
<td>Bishop, A. W.</td>
<td>The author reviewed the laboratory testing methods to estimate the earth pressure coefficient $K_0$. The characteristics of $K_0$ obtained from laboratory tests were discussed.</td>
<td>It was considered that triaxial cell tests for evaluating $K_0$ by balancing the amount of drained water from the cell cannot evaluate $K_0$ value correctly, because considerable lateral yields occurred in spite of careful measurement. There is no correlation between $K_0$ and plasticity index. It was found that the correlation proposed by Jaky provides good agreement to the test data for both sand and clay.</td>
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<tr>
<td>1961</td>
<td>Skempton, A. W.</td>
<td>Investigation of a short-term slip failure in a 40-feet deep excavation of over-consolidated London clay. Its at-rest earth pressure coefficient $K_0$ was studied by taking into account the effects of capillary pressures. Methods for determining the capillary pressures in the soil were described.</td>
<td>It was concluded that capillary pressures can be evaluated by (1) swelling pressure tests, (2) finding a load which prevents volume change when an oedometer specimen is immersed, (3) measuring suction directly, (4) deduction from UU-triaxial test results with assumed value of $A_f$. The first two methods were based on the assumption that the capillary pressure is equal to the swelling pressure. The fourth method, provided consistent results with the other methods when $A_f$ was assumed to be 0.25. A relationship between $K_0$ and the ratio of the capillary pressure to the in-situ effective stress was developed.</td>
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<td>1963</td>
<td>Hendron, Jr., A. J.</td>
<td>Stress-strain behavior of a face-centered cubic array of uniform spheres under one-dimensional compression was discussed. An apparatus for measuring the stress-strain behavior and $K_0$ during one-dimensional compression was developed. The apparatus consisted of a thin steel ring surrounded by an annular space, a testing head applying a vertical load, a base plate, and a thick walled cylinder. Strain gauges were attached to the thin ring to measure lateral strain. Specimens had diameters of 6.812 inches and heights of 2.0 inches. Oil in the annular space was pressurized maintain no lateral strain. The apparatus was capable of applying vertical stress up to 3290 psi. Specimens of four different sands were tested by applying cyclic loads.</td>
<td>A theoretical relationship between friction angle and coefficient of earth pressure at rest $K_0$ was proposed, based on the study on uniform spheres. A relationship between vertical stress and vertical strain was dependent on initial relative density. The values of $K_0$ for three of the sands tested were approximated closely by the equation suggested by Jaeky (1944); in contrast, those for Minnesota sand agreed with the proposed theoretical relationship. Energy absorption was classified into three categories: (1) rearrangement of the grains, (2) crushing of grains, and (3) elastic hysteresis mechanism due to friction. Energy absorption in the first cycle was quite high due to crushing and rearrangement of the grains; those in the second and the third cycles were considerable due to the effect of hysteresis. It was concluded that a type of Coulomb damping element instead of a viscous damper must be used to model the behavior, because the latter does not dissipate energy under static loads.</td>
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<tr>
<td>1965</td>
<td>Aksai, K. and Adachi, T.</td>
<td>A triaxial testing apparatus was modified to conduct $K_0$ consolidation tests. Isotropic and $K_0$ consolidation undrained compression tests were conducted on saturated specimens of clay.</td>
<td>$K_0$ was found to remain approximately constant during consolidation. Pore pressures were found to dissipate more rapidly in $K_0$ consolidation than in isotropic consolidation. Anisotropic consolidation influenced the dilatant characteristics of clay, although there is no distinct difference in the failure envelopes for anisotropically consolidated specimens and isotropically consolidated specimens.</td>
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<td>1965</td>
<td>Brooker, E. W. and Ireland, H. O.</td>
<td>Investigation of the influence of stress history on the earth pressure at rest of remolded cohesive soils. The one-dimensional compression testing apparatus developed by Hendron (1963) was slightly modified. One-dimensional loading-unloading tests were performed on specimens of five different clays which covered a range of plasticity from high to low. All the specimens were remoulded at a liquidity index of 50 percent. A consolidation pressure of 2200 psi was applied, and then it was reduced to achieve various overconsolidation ratios.</td>
<td>It was found that the coefficient of earth pressure at rest $K_0$ was highly dependent on stress history. Values of $K_0$ appeared to approach the coefficient of passive earth pressure, as values of OCR increased. The highest value which $K_0$ reached was dependent on the effective stress friction angle and the plasticity index of the cohesive soils. The values of $K_0$ for normally consolidated condition were well approximated by an empirical relation $K_0 = 0.95 - \sin \phi$.</td>
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<tr>
<td>1968</td>
<td>Mackey, R. D. and Kirk, D. P.</td>
<td>At-rest, active, and passive pressures acting on a rigid wall were measured. A rigid steel tank 36 in. x 16 in. x 12 in. was backfilled with three different sands. Loose backfills were constructed by pouring the sands, and dense backfills were constructed by pouring, and by use of a vibrating hammer.</td>
<td>The measured at-rest pressures in loose backfills were found to be in good agreement with Jacey's (1944) correlation. The measured at-rest pressures in densely compacted sands were almost equal to the Rankine passive pressures. It was considered that compaction developed residual lateral stresses.</td>
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<td>1969</td>
<td>D'Appolonia, A. M., Whitman, R. V. and D'Appolonia, E. D.</td>
<td>Field tests of sand compaction using a vibratory roller were performed to investigate the effects of lift height, number of roller passes, and roller operating frequencies. Fills of uniformly graded dune sand were placed with a single large lift or with 2-ft lifts. Ground accelerations and dynamic stresses within the soil were measured.</td>
<td>It was found that density increased with increase in roller weight, operating frequency, and number of roller passes. $K_0$ after vibratory compaction was significantly greater than for normally consolidated fills. Horizontal stresses increased as the number of roller passes increased and as the operating frequency increased. The horizontal stresses on planes parallel to the roller path were smaller than those on planes perpendicular to the path.</td>
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<td>1969</td>
<td>Obrican</td>
<td>Development of a consolidation testing apparatus to measure the lateral pressure at rest. The apparatus consists of a bottom plate and a split ring. The lateral load exerted by the specimen was measured by two proving rings mounted on a cross beam. Zero lateral strain was maintained by adjusting two screws manually when strain gauges detected movement. The tests were conducted on four granular soils of different gradations.</td>
<td>Significant wall friction force was detected between the side wall and the specimen, especially for large vertical stresses. The conclusions derived from the test results were: (1) $K_0$ is significantly influenced by relative density. $K_0$ was higher for loose sand than for dense; (2) The finer the sand, the higher $K_0$; (3) $K_0$ was significantly influenced by stress history. $K_0$ was higher for unloading and reloading than for primary loading; (4) Granular soils can retain a significant part of the lateral stress induced by cyclic loading; (5) Higher stresses gave smaller values of $K_0$, because of changes in density.</td>
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<tr>
<td>1969</td>
<td>Wright, S. G.</td>
<td>Development of a $K_0$ triaxial testing device for investigating lateral stresses in overconsolidated soils. Circumferential strain was measured at mid-height of specimens by a sensitive strain indicator device. Cell pressures were adjusted to maintain zero lateral strain. $K_0$ loading and unloading tests were conducted on specimens of Monterey No. 20 sand and Eastern Silica sand. Effects of initial confining pressure, loading rate, void ratio, soil type, and stress history were studied.</td>
<td>1. The maximum circumferential deformation was less than 10 microns, which corresponded to approximately $2.5 \times 10^{-6}$ radial strain. 2. Initial confining pressures influenced the results; however, disturbance and subsequent changes in density during the assembly of the apparatus would be significant with a low confining pressures. The lowest value of the confining pressure providing reasonable results was determined to be 0.5 kg/cm². 3. The effect of loading rate on $K_0$ was considered to be negligible. 4. The denser or stronger the specimen was, the greater the variation in $K_0$ values with varying overconsolidation ratio. 5. The values of $K_0$ during unloading were dependent on the maximum past pressure. 6. During reloading values of $K_0$ are not uniquely related to OCR. 7. Laboratory tests can be used to estimate in-situ horizontal stress in overconsolidated soils only if the complete stress history is known.</td>
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<tr>
<td>1971</td>
<td>Broms, B. B. and Ingelson, I.</td>
<td>Measurement of earth pressures on a bridge abutment with a height of 10 feet, backfilled with uniform sand.</td>
<td>The backfill was compacted in layers by a vibratory roller. At shallow depths, less than 2 feet, earth pressure coefficients were greater than unity and even close to a passive coefficient in some cases. The earth pressure coefficients were affected by movements of the wall caused by changes in temperature of the bridge slab.</td>
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| 1971 | Broms, B. B. | Study of the factors affecting lateral earth pressures on nonyielding walls. A method for estimating the lateral earth pressures induced by compaction was proposed. | The behavior of the lateral earth pressures induced by compaction was modeled as follows:  
(1) $K_0$ during the first loading can be estimated by Jaky's correlation:  
(2) vertical pressures induced by compaction can be calculated by the Boussinesq's equation:  
(3) upon unloading, the lateral pressures are assumed to be constant until the vertical pressure reaches the value $\sigma_{v,max}/K_0'$ where $K_0'$ is a limiting lateral earth pressure coefficient for unloading:  
(4) for vertical pressures smaller than the value above, the lateral pressures are given by $K_0' \sigma_{v,max}$.  
The lateral pressure distribution after compaction was estimated to be trapezoidal. The lateral pressure increases linearly from the ground surface to a critical depth, and remains constant below that depth. |
<p>| 1971 | Moore, C. A. | Effects of mica content on stress-strain relationships under little or no lateral yielding condition were studied. A triaxial testing apparatus was modified to perform $K_0$ tests. A lateral deformation sensor consisting of a circumferential foil band fitted with strain gauges was used. $K_0$ compression tests were performed on specimens of a mixture of sand and mica, and a mixture of silt and mica. | The compressibility of the sand and mica mixture increased with increasing mica content. The compressibility of the silt-mica mixture was independent of the mica content. The value of $K_0$ for the sand-mica mixture was unaffected by the mica content. The value of $K_0$ for the silt-mica mixture increased with increasing mica content. |</p>
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<tr>
<td>1972</td>
<td>Campanella, R. G. and Vaid, Y. P.</td>
<td>A new triaxial testing system was developed to perform $K_0$ consolidation tests. The $K_0$ condition was maintained during the tests by preventing any volume change in the cell water surrounding the specimen. Pore water was expelled during consolidation only by change in height of the specimen. $K_0$ consolidation tests were performed by applying two cycles of loading and unloading, and drained and undrained compression and extension tests were conducted. The tests were performed on specimens of undisturbed saturated sensitive marine clay.</td>
<td>The developed testing system provided reproducible results. $K_0$ consolidation tests can only be performed on saturated specimens using this device. The cyclic $K_0$ consolidation tests showed that $K_0$ was constant for normally consolidated specimens, and that $K_0$ for overconsolidated specimens was dependent on OCR and independent of the peak stress of the rebound cycle. The undrained extension tests gave effective friction angle 5 degrees higher than the undrained compression tests, although the extension tests gave 65% of the undrained strength in compression. For undrained conditions, the active tests and the passive tests gave almost the same values of peak stress and stress ratio. The drained passive extension tests gave higher stress ratio than the active extension tests. The drained compression tests gave similar stress ratios for both active and passive conditions.</td>
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<td>1972</td>
<td>Janbu, N.</td>
<td>Earth pressures for design were discussed. Selection of proper safety factors and shear stresses acting between a wall and a backfill were studied.</td>
<td>It was concluded that classical earth pressure theories provide stresses at failure, whereas design conditions are in equilibrium. The at-rest lateral pressure coefficient, $K_0'$, was defined as an effective principal stress ratio with attraction added to both the major and the minor principal stresses.</td>
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<tr>
<td>1972</td>
<td>Moore, P. J. and Spencer, G. K.</td>
<td>Studies of lateral earth pressures exerted by backfills of soft clay. A 19-inch high, 63-inch wide retaining wall was constructed. The wall was divided into three sections to isolate end-effects, and the center panel was instrumented with 18 pressure cells. Piezometers were placed at various depth to measure pore pressure distributions. A kaolin clay was mixed to a slurry, and placed behind the wall. The backfill was consolidated by its own weight, and by surcharge pressures in some tests. A special type of triaxial cell was developed for measurement of $K_0$ values. Volume change of specimens were maintained equal to zero by means of a burette connected to the pore water back pressure supply. A torsion shear apparatus was developed for measuring residual shear strength.</td>
<td>It was found that lateral pressures decreased immediately as the wall moved away from the soil, and that the fully active condition was achieved by horizontal movement equal to 5 percent of the height. After movement stopped, the lateral pressures increased to the values which were observed before the movement occurred. The values of $K_0$ calculated from the triaxial test results and the torsion shear tests were in good agreement with those measured in the retaining wall tests.</td>
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<tr>
<td>1972</td>
<td>Rehnman, S. E. and Broms, B. B.</td>
<td>Lateral earth pressures were investigated using an instrumented wall 600 cm long and 250 cm high. Lateral earth pressures were measured using earth pressure cells mounted on the wall, and vertical pressures were measured using pressure cells mounted on the floor slab. The wall was capable of rotating using hydraulic jacks. The wall was backfilled with gravelly sand or silty fine sand. The backfill was placed loose or compacted in layers with a vibratory plate compactor. The increase in lateral earth pressures due to surface loads was studied.</td>
<td>For uncompacted backfills, measured earth pressures were between the calculated active pressures and the calculated at-rest pressures. Measured pressures after compaction were greater than the pressures calculated by Jaky's (1944) correlation for the upper 0.5 to 1.5 m of the fill. Below these depths the pressures were constant or decreased slightly with depth. It was considered that the wall deflected away from the backfill by a small amount, as the upper layers were placed and compacted. Residual pressures after removal of the loads were found to be 30 to 70 % of the peak pressures induced by the loads.</td>
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<tr>
<td>1973</td>
<td>Andrawes, K. Z. and El-Sohby, M. A.</td>
<td>Specimens of four cohesionless materials were tested in a triaxial cell to measure coefficients of earth pressure at rest and to investigate deformation characteristics in the at rest condition. The effects of strain conditions and material properties on $K_0$ were studied.</td>
<td>The value of $K_0$ was determined by interpolating the results of two tests in which $\epsilon_v / \epsilon_a$ were just smaller than unity and just greater than unity. The deviation from no lateral yielding did not significantly influence the measured values of $K_0$ for strain controlled tests. Anisotropy and the value of intermediate principle stress influenced the value of $K_0$. The value of $K_0$ was found to be a function of not only the friction angle but also the modulus of elasticity, porosity and crushing of particles. It was concluded that the elastic and sliding strains have to be taken into account to deduce the value of $K_0$.</td>
</tr>
<tr>
<td>1973</td>
<td>Janbu, N.</td>
<td>The behavior of Coulombian material was expressed in terms of friction angle and attraction, where attraction is the intercept of the failure envelope with the normal stress axis. Based on this concept, the failure criterion was discussed in terms of normal deviator stress. A test called &quot;cyclic triaxial test&quot; ($K_0$-consolidated undrained or drained triaxial shear test) was explained, and the interpretation of the results was discussed.</td>
<td>It was considered that the failure criteria in terms of deviator stress is advantageous for interpreting triaxial test results. A concept of degree of shear strength mobilization was proposed to deal with safety factors. The at-rest lateral pressure coefficient, $K_0^*$, was discussed based on the definition suggested previously by Janbu (1972).</td>
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<tr>
<td>1974</td>
<td>Aggour, M. S. and Brown, C. B.</td>
<td>The effects of compaction on the behavior of retaining walls was analyzed using a new incremental model. The model was used to analyze the behavior of cantilever and foundation walls. The effects of number of passes, end wall constraints, wall flexibility, and backfill geometry were studied. In the analyses, it was assumed that the walls rested on bedrock.</td>
<td>The proposed model for evaluating the effects of compaction was a sequence of linear analyses in which soil properties were modified after each lift. The finite element method was considered to be appropriate for implementation of this model. Plane strain finite element analyses were used to simulate the 3D compaction process.</td>
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| 1976 | Abdelhamid, M. S. and Krizek, R. J. | A consolidometer was modified for evaluating lateral earth pressures at rest. Two pressure transducers, one for lateral pressures and one for pore pressures, were installed on a rigid stainless steel cylindrical chamber of 50-cm height, 20-cm diameter, and 0.6-cm thickness. The samples tested were a dispersed slurry and a floculated slurry, with initial water contents were about 250% and final water contents were about 45%. The vertical stress were increased from 7 kN/m² to 1,760 kN/m². | It was found that:  
(1) the value of lateral earth pressure coefficient $K_0$ was constant in the entire range of normal consolidation;  
(2) the value of $K_0$ increased, during unloading, to be unity at an overconsolidation ratio of about 2, and approached the value of the passive earth pressure coefficient at an overconsolidation ratio of about 8;  
(3) the clay fabric did not appear to influence the value of $K_0$;  
(4) values of $K_0$ calculated by Jaeky's expression and Booker and Ireland's expression showed good agreement with those measured directly;  
(5) values of $K_0$ should be determined by using the mobilized frictional shear strength at rest, although it requires measurement of Hvorslev's parameters. |
| 1977 | Carder, D. R. Pocock, R. G. and Murray, R. T. | A 1 m thick reinforced concrete wall was constructed opposite a metal wall. The metal wall was movable and the concrete wall was considered to be rigid. The pit between the walls was filled with a uniformly graded washed sand in 150 mm-thick lifts and compacted with a 1.3 Mg vibrating roller. | It was found that compaction induced considerable residual lateral pressure in the backfills. However, it was concluded that the magnitude of the residual stresses cannot exceed the stress calculated using $1/K_0$. The fully active condition was reached with a translations of $A/H = 0.0012$. It was considered that the active stresses are appropriate stresses for use in design, provided the wall is sufficiently flexible.  
A peak lateral resultant force was observed when the wall with a height of 1 m moved into a backfill by 25 mm. The peak pressures were greater than the passive pressures calculated using the Coulomb theory. It was speculated that the reason was the high compaction-induced lateral stresses or the influence of wall friction. |
| 1977 | Sherif, M. M. and Mackey, R. D. | Investigation of lateral earth pressures induced by repetitive loading. Model tests were performed using a steel tank 1200mm×1200mm×470mm. Pressures were measured with eight pressure cells mounted on the wall. The wall was backfilled with a dense uniform sand. A line load was applied repetitively to the surface of the backfill at different distances from the wall. | (1) The lateral earth pressures induced by repetitive loading were considerably greater than those induced by the virgin loading.  
(2) The largest increase in lateral pressure was observed near mid-height of the wall.  
(3) The effects of the repetitive loading decreased when loads were applied at greater distances from the wall. |
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<tr>
<td>1979</td>
<td>Ingold, T. S. (1)</td>
<td>Analysis of a long reinforced concrete cantilever wall damaged during compaction. The wall was constructed with a backfill compacted by a vibratory roller. When the compaction was completed, the top of the wall had deflected 3.7 inches, which corresponded to Δ/H of 0.013. Horizontal tension cracks in the wall near its base were observed. Moments calculated by classical earth pressure theory were much smaller than the ultimate bending resistance of the stem. Effects of compaction on the lateral pressures had not been taken into account in the initial design.</td>
<td>A new method for estimating lateral pressures induced by compaction was proposed. An analytical result obtained by means of this model was compared with the observed performance of the wall. The proposed model was similar to Broms's (1971) model. It was considered that the compaction-induced lateral pressures were constant below a critical depth, and they were calculated by applying passive failure condition. The pore water pressures were assumed to be negligible. It was concluded that the proposed model provides good estimation of compaction induced lateral earth pressures, although the method is not rigorous.</td>
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<td>1979</td>
<td>Ingold, T. S. (2)</td>
<td>A method for estimating compaction-induced lateral earth pressures proposed by the author (1979) (1) was described, extended, and applied to three case histories. Conventional design procedures of retaining walls were reviewed using the proposed method.</td>
<td>It was found that the proposed model provides good agreement with the case histories. Safety factors in retaining wall designs using the proposed method were significantly smaller than those by conventional design methods.</td>
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<td>1980</td>
<td>Carder, D. R., Murry, R. T. and Krawczyk, J. V.</td>
<td>An instrumented retaining wall test was performed using the same facility as the one used by Carder, et. al. (1977). The wall was backfilled with silty clay. The backfill was placed, and then compacted into 125 mm thick layers with six passes of a 3.25 Mg smooth-wheeled roller. Pore water pressures were measured using mercury-filled manometers which have ceramic tips with a high air-entry value. Triaxial tests were performed to measure undrained and effective stress strength parameters. Consolidation tests conducted on the silty clay provided a consolidation coefficient of 0.6 m²/year.</td>
<td>Lateral earth pressure exerted by the backfill at the top of a rigid retaining wall was considerably higher than the at rest pressure. The authors proposed a procedure for estimating compaction-induced lateral earth pressure. The procedure is based on Broms's method (1971), and it accounts for the type of compaction equipment and for pore pressure effects. The calculated lateral pressure distribution showed good agreement with experimental data. High lateral pressure recorded after compaction decreased as excess pore pressures dissipated. It was found that a small rotation, less than 1/20 degree, was sufficient to reach the active condition for a 2 meter high backfill, and that a rotation exceeding 7.5 degree was required to reach the passive condition for a 1 meter high backfill. For the active case, the measured pressure distribution agreed well with that calculated by using the residual effective friction angle.</td>
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<td>Year</td>
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<td>1980</td>
<td>Ofer, Z.</td>
<td>A correlation between the amount of swell and lateral swelling pressures was proposed, based on the results of lateral swelling pressure ring tests. The apparatus used for the study was a modified oedometer ring instrumented with strain gauges. In situ lateral swelling pressures were measured by means of an in situ lateral swelling pressure probe that consisted of a cutting module, a wetting ring, and a measuring module.</td>
<td>The in situ testing probe gave higher lateral swelling pressures than measured in the laboratory tests. The author considered that the specimens used for the laboratory tests were disturbed. It was found that only the vicinity of the probe was saturated and that this portion contributed most to the lateral swelling.</td>
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| 1981 | Ofer, Z.| Development of an oedometer for measuring lateral pressures and swelling pressures exerted by soil specimens. | The tests showed that:  
(1) the value of $K_0$ determined from the no-lateral strain condition were significantly higher than that obtained from tests where the specimens under small lateral strains.  
(2) identical lateral pressures were measured at the end of each cycle where the vertical pressure was reduced to zero,  
(3) the load-deformation relation was similar after the first cycle. |
| 1981 | Sherif, M. A. and Ishibashi, I. | Development of "stress meter" for measuring lateral stresses on non-yielding structures. Tests were conducted on eighteen different cohesive soils and dry sands. | It was concluded that $K_0$ is a function of overconsolidation ratio and liquidity index. For clay, it was found that $K_0$ is a linear function of OCR. For sands, it was concluded that $K_0$ is an exponential function of OCR. |
| 1982 | Mayne, A. M. and Kulhawy, M. | Studies of the relationship between at-rest earth pressure coefficients and OCR. Results of laboratory tests conducted previously on 170 different soils were reviewed. Simple empirical correlations were suggested and examined by statistical analyses. | (1) Values of $K_0$ for normally consolidated soils calculated by Jaky's correlation were in good agreement with data for cohesive soils, and were in reasonable agreement for cohesionless soils.  
(2) Values of $K_0$ for overconsolidated soils during unloading appear to be a function of the effective stress friction angle.  
(3) Values of $K_0$ for overconsolidated soils during reloading are dependent on the effective stress friction angle and the stress history. |
Table 2-1. Cont. Literature Review on At-Rest and Compaction-Induced Lateral Earth Pressures

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| 1982 | Ofer, Z.        | Studies of the compaction-induced lateral earth pressures using the lateral soil pressure ring developed by Ofer (1981). The author conducted tests on Honeydew sand and Rustenburg black clay. Sand samples were placed into the apparatus in a loose condition and then vibrated under a small vertical load prior to the test. Tests were conducted by applying four cycles of loading and unloading between 12.7 kPa and 634.5 kPa. Clay samples were prepared at four different moisture contents, and compacted in place by means of static vertical loads. Four loading and unloading cycles from 12.7 kPa to 761.4 kPa were performed. Each load cycle took 10 minutes with 15 minutes relaxation between cycles. | The following observations were made from the laboratory tests.  
1. The value of $K_0$ was high after placement of a sample or under low pressure, and it decreased with increase in the vertical stress.  
2. The value of $K_0$ increased with increasing OCR for unloading. The relation between $K_0$ and OCR was approximated with a logarithmic function. For clay, this relation was influenced by moisture content and time; on the other hand, is was independent of those factors for sand.  
3. A small lateral strain reduced the measured $K_0$ value for sand; however it did not influence the measured total stress ratio during quick loading-unloading tests on the specimens of partly saturated clay.  
4. Repetitive loading reduced the value of $K_0$.  
5. The values of $K_0$ measured in triaxial tests using the types of $K_0$ belt developed by Bishop and Henkel (1962) were slightly higher than those measured with the apparatus used in this study. However, the $K_0$ belt was found to be inconvenient for testing clay. |
<p>| 1982 | Sherif, M. A., Ishibashi, I. and Lee, C. D. | At-rest and active earth pressures under static and dynamic conditions were studied. A soil box 6ftx8ftx4ft on a shaking table was used for studies of earth pressures against a movable rigid retaining wall. The wall had a length of 40 inches and a height of 41 inches, and was backfilled with dry Ottawa sand. | It was suggested that the active condition should be defined as the condition when the mobilized wall friction angle is maximized. Deflection of the wall required to reach the active condition based on the proposed definition was found to be $\Delta/H = 0.00017$. Densification by vibration increased the at-rest earth pressures and raised the point of application of the resultant force. |</p>
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| 1983 | DiBernardo, A. and Lovell, C. W. | Studies of the compressibility of laboratory compacted soils. Specimens of a highly plastic residual clay were prepared by kneading compaction, and trimmed to size for oedometer tests. | The compressibility behavior was discussed in terms of compactive prestress and compaction pressure.  
(1) It was considered that the compressibility of compacted soils can be explained by taking into account compacted clay macrostructure.  
(2) For soils dry of optimum, the compactive prestress increases as water contents decreases, and as compactive efforts increases. For soils wet of optimum, the compactive prestress was not affected by these variables.  
(3) The ratio of compactive prestress to nominal compaction pressure was found to be less than unity. It appeared to decrease with increasing degree of saturation.  
(4) A statistical model for describing the compressibility behavior was proposed. |
| 1984 | Sherif, M. A., Fang, Y. S. and Sherif, R. I. | Static at-rest and active earth pressures were studied using the same facility and backfill as those in Sherif et. al. (1982). | It was concluded with respect to at-rest pressures that:  
(1) The stress distribution behind a non-yielding wall is hydrostatic.  
(2) Jakey's (1944) correlation was valid for loose sand.  
(3) Densification of backfills by either compaction or vibration increases at-rest pressures.  
(4) At-rest pressures in densified soils are the sum of the stresses due to gravity effects and the locked stresses due to overcompaction or overstressing.  
(5) At-rest pressures for dense sand is a function of the friction angle, the minimum density, and the actual density of the backfill. |
| 1985 | Burland, J. B. and Fourie, A. | Passive stress relief tests were conducted, using a stress path triaxial apparatus. The cell pressure was reduced as the vertical stress was decreased. Tests were performed on undisturbed specimens of clay from Claygate Beds and London Clay which were allowed to swell after isotropic consolidation, and then subjected to anisotropic stresses. A remolded specimen of London Clay was tested in the same manner. | The tests on undisturbed specimens gave significant values of effective cohesion, and indicated large effective friction angles during the early stages of the tests. The test on the remoulded specimen showed a high value of effective friction angle, although the effective cohesion was zero.  
The authors speculated that the values of $K_0$ for heavily overconsolidated clays near ground surface are associated with the strength corresponding to the passive relief condition. |
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<td>1985</td>
<td>Dyvik, R. Lacasse, S. and Martin, R.</td>
<td>An oedometer ring was modified to measure lateral stresses during consolidation tests. The inside face of the oedometer ring at mid-height was made of teflon membrane, with a water-filled chamber behind. The lateral pressure at rest was measured by pressurizing the water inside the chamber. Specimens of four types of overconsolidated clay were tested. The specimens were loaded, unloaded, and reloaded.</td>
<td>Negative intercepts were obtained in plots of lateral stress with vertical stress. The authors considered that the reason for this might be (1) the small initial gap between a specimen and the ring, (2) radial deformation of the device, or (3) differences between field condition and laboratory condition. The results were corrected to pass through the origin. Some of the corrected results agreed with the relation proposed by Mane and Kulhawy, but some did not.</td>
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<td>1986</td>
<td>Duncan, J. M. and Seed, R. B.</td>
<td>A model for estimating peak and residual compaction-induced lateral earth pressures either in free field or adjacent to nonyielding walls was developed. The model described hysteretic stresses generated by cycles of loading and unloading. Two case studies, reported by Rehmans and Broms (1972) and Carder et al. (1977), were analyzed using the proposed model and compared with the measurements. It was suggested that compaction loads should be transformed to an equivalent vertical pressure for calculation of lateral pressures with earth pressure coefficients rather than used directly. Five parameters are required for the model.</td>
<td>The proposed model consists of the following rules: (1) a virgin loading path is defined by the $K_0$ line, where $K_0$ can be estimated by Jaky's (1944) correlation; (2) upon unloading, the stress path is nonlinear until the path reaches the passive failure limiting condition; (3) thereafter the unloading path follows the passive failure limiting line; (4) for reloading, the stress path is linear to a target point on the $K_0$ line, and thereafter it follows $K_0$ line. It was considered that model parameters should represent the post-compaction condition. Model parameters were suggested based on the empirical correlation for cases where $K_0$ tests are not conducted. The proposed model provides good agreement with multicycle $K_0$ test results. Based on the proposed model, incremental analyses simulating field compaction provide good agreement with the field data.</td>
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<td>1986</td>
<td>Kavazanjian, Jr., E. and Mitchell, J. K.</td>
<td>Studies of a change in the values of $K_0$ with time. $K_0$-triaxial test results on undisturbed and compacted specimens of clays were evaluated.</td>
<td>The isotropic stress state was considered to be a minimum energy state. It was speculated that earth pressure coefficients at rest approach unity over time.</td>
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<td>1986</td>
<td>Massarsch, R.</td>
<td>Errors in measurement of $K_0$ were studied. Lateral earth pressures in a normally consolidated clay layer were measured by means of hydraulic fracturing tests, total stress cells, conventional pressuremeter, self-boring pressuremeter, and total stress cell in a preexcavated borehole.</td>
<td>Values of $K_0$ are quite sensitive to accuracies in values of vertical pressures, lateral pressures, and pore pressures used for the calculations. It was considered that there is a correlation between $K_0$ and plasticity index. The hydraulic fracturing method provided too high values of $K_0$. Disturbance due to excavation of a borehole reduced $K_0$.</td>
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<td>1986</td>
<td>Seed, R. B. and Duncan, J. M.</td>
<td>A hysteretic model for estimating compaction-induced lateral earth pressures was described. Incremental placement and compaction are modeled in the proposed method. An instrumented retaining wall test and a long-span flexible culvert were analyzed using the finite element method employing the proposed analytical model.</td>
<td>Lateral stresses were separated into two components, geostatic lateral effective stress and compaction-induced lateral effective stress. A nonlinear unloading stress path was approximated by a bilinear path. It was considered that plane strain finite element analysis provides good results for three dimensional problems if pressure increments accounting for three-dimensional effects are used. Multiple compaction passes at a given depth are modeled as a single increment. It was considered that model parameters determined by empirical correlations provided good agreement with field data.</td>
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<td>1987</td>
<td>Clayton, C. R. I. and Hiedra-Cobo, J. C. and Symons, I. F.</td>
<td>The authors measured the compaction-induced lateral earth pressures on an unyielding wall at the T. R. R. L. experimental retaining wall facility (Carder and Krawczyk (1975), Carder et. al. (1977, 1980)). Lateral pressures were measured by means of pneumatic cells, hydraulic cells, and vibrating wire cells. A total of 26 layers of London clay was placed, and each layer was compacted to a thickness of 120 mm by vibrotamper for the first six layers and a 7 tonne vibratory roller for the rest.</td>
<td>The stiff clay backfill exerted high lateral pressures on a non-yielding wall as a result of compaction. Both classical analyses for earth pressure at rest, and compaction theories proposed previously, underestimated the magnitude of these pressures by a factor greater than two. The distribution of these pressures was not triangular. The authors speculated that the effect of swelling would be an important factor in estimating compaction-induced pressures. They speculated that even higher lateral pressures would be exerted by backfills of cohesive soils with high plasticity or high strength.</td>
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<td>1987</td>
<td>Nwabuokei, S. O. and Lovell, C. W.</td>
<td>Prestress in clay induced by compaction was discussed. Specimens with various water contents were prepared by applying three different compactive energies. The compacted specimens were loaded in an oedometer to determine compactive prestresses. Some specimens were saturated by applying back-pressures prior to loading.</td>
<td>An analytical model for estimating the compactive prestresses was proposed. The model is based on the energy transferred to and stored in soils. The parameters in the model depend only on water content. Compactive prestress depends on water contents and compactive energies. The amount of swell due to saturation increased with increasing compactive prestresses. It was found that prestress in the saturated condition increased with increasing confining pressure, and that they depend on compaction water content, compacted void ratios, and the compacted prestress prior to saturation.</td>
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<td>Year</td>
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<td>1988</td>
<td>Symons, I. F. and Murray, R. T.</td>
<td>Compaction-induced lateral earth pressures were studied using a series of pilot scale experiments and two field studies. The pilot scale studies previously conducted at the T. R. R. L. experimental retaining wall facility (Carder et al., 1977, 1980; and Clayton et al., 1987) were reviewed. A field study was performed on a cantilever wall with a height of about 8 meters. A well graded sand and gravel backfill was compacted with a vibrating roller. Another field study involved a retaining wall with a backfill of pulverized fuel ash. The backfill was compacted with 6 to 8 passes of a 1Mg double vibrating roller.</td>
<td>High lateral pressures due to compaction were induced in the upper 2 meter of the backfills. The pressures were reduced to active after movement of only a few millimeters. For the unrestrained wall, movement occurred progressively during filling, and the lateral pressures were decreased by the subsequent filling. The pressure distribution was highly dependent on the boundary conditions.</td>
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<td>1989</td>
<td>Mawditt, J. M.</td>
<td>Outward movement of retaining walls supporting the M3 motorway was investigated. The retaining walls, with a height of about 9 m and a thickness of 600 mm, were founded on two 600 mm diameter reinforced concrete piles, and were backfilled with London clay. The outward movement was found to be the order of 100 mm. Pressuremeter and dilatometer tests were performed to investigate the load acting on the retaining walls; in addition, spade cells and hydraulic piezometers were installed.</td>
<td>High lateral pressures were observed at a point 13 m distant from the retaining walls, and pressures decreased significantly towards the back of the wall. Measured pore pressures were very low, and sometimes negative. High soil suction appeared to have dissipated near the wall, however about half of the swelling potential remained. The author speculated that highly overconsolidated clay could generate considerable swelling pressures.</td>
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<td>1989</td>
<td>Senneset, K.</td>
<td>Development of a new oedometer ring which was separated into three parts. Lateral stress was measured by three LVDT's installed behind steel membranes. Specimens had a diameter of 54.3 mm and a height of 20 mm. Continuous loading tests and incremental loading tests were performed on undisturbed specimens of overconsolidated Glava clay.</td>
<td>The deformation of the steel membranes was 40 μm at 800 kPa, and the radial deformation measured outside of the ring was 8 μm at 800 kPa. For vertical pressures greater than the preconsolidation pressure, the ratio of horizontal stress to vertical stress was constant. An initial gap of 0.26 mm was introduced between the specimen and the ring in order to evaluate the influence of lateral deformation. With an initial gap, a negative intercept was obtained in the plot of horizontal stress versus vertical stress. The author speculated that similar results obtained by Dyvik et al. (1985) were caused by an initial gap.</td>
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Table 2-1. Cont. Literature Review on At-Rest and Compaction-Induced Lateral Earth Pressure

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<th>Author</th>
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| 1989 | Symons, I. F.                 | Continuation of studies reported by Clayton et. al. (1987). The experiments were divided into three stages: (1) construction of the fill, (2) the period after construction, (3) swelling of the fill. In the first stage, London clay was filled between a rigid concrete wall and a movable metal wall. In the second stage, a gradual reduction in total pressures with time was observed. In the third stage, sand drains with a diameter of 75 mm, which were replaced with 20 mm diameter perforated plastic pipes after about two months, were installed to accelerate the swelling process. | (1) Assuming that pore pressures were zero, the total lateral pressures exerted on both the nonyielding wall and the movable wall were greater than the calculated earth pressure at rest.  
(2) The lateral pressures decreased considerably, especially in the upper portion of the fill, during a short pause in the filling, and during the second stage. It was speculated that this was caused by the relaxation in the compacted clay.  
(3) The lateral pressures increased rapidly to the maximum values about 6 months after the start of swelling. Surface heave of 104 to 128 mm was measured over the 20 months period; in spite of that, considerable negative pore pressures remained in the lower portion of the fill.  
(4) When the lateral pressures stabilized after swelling, their values exceeded the estimated values of limiting passive earth pressure. The authors considered that the mobilized friction angles were greater than those obtained by conventional triaxial compression tests. |
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<td>1990</td>
<td>Sehn, A. L.</td>
<td>An instrumented oedometer for measuring lateral pressures was developed. The instrumented oedometer had a split ring, and the lateral pressure acting on the movable half was measured with two load cells. Tests were conducted on dry specimens of Monterey #30 sand by applying 1,000 cycles of loading and unloading. An instrumented retaining wall facility was constructed to investigate the factors affecting compaction-induced earth pressures. A 6.5-feet-deep backfill of Yatesville silty sand was constructed by placing and compacting the soil in 6-inch lifts.</td>
<td>It was found that the instrumented oedometer was useful for evaluating the effects of the number of load cycles on at-rest earth pressure coefficient. One thousand cycles of loading changed the value of $K_0$ very little at an OCR of 1.0, whereas the value of $K_0$ decreased by 10% at an OCR of 10. In the instrumented retaining wall tests, horizontal force measurement and earth pressure measurement gave good agreement during backfilling; however, they gave inconsistent results shortly after backfilling. The vertical shear forces on the wall at the end of construction agreed with values previously reported by Vogt (1986), and they increased after construction. The lateral pressures observed in the tests were well explained with the compaction induced lateral pressure theory by Duncan and Seed (1986), while the observed values were much higher than the theoretical values in one test conducted with a backfill compacted 2% wet of optimum. It was considered that this was due to the pore pressure developed during compaction. It was found that the measured earth pressure decreased after backfilling.</td>
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<td>1991</td>
<td>Clayton, C. R. I., Symons, I. F. and Hiedra-Cobo, J. C.</td>
<td>Study of lateral earth pressures exerted by clay backfill on rigid and moveable retaining walls. The process of development of lateral earth pressures was divided into three phases: (1) placement, compaction and burial, (2) stress relaxation at a constant moisture content, and (3) swelling or consolidation. London Clay was compacted into cylindrical molds with several sizes by impact in the laboratory. The authors studied the effect of energy per blow and of total energy on the magnitude of the residual total stress. The authors examined the influence of air void content on residual lateral pressure. Two series of pilot scale tests which had been reported previously (Carder et. al. 1980 and Symons et al. 1989) were expained and reanalyzed.</td>
<td>The authors concluded the followings from the laboratory tests. (1) The method of compaction was found to be important. However, when a sufficiently high energy per blow was used, the method of compaction did not influence the lateral earth pressures. (2) The lateral earth pressure induced by compaction was small when the air void content was greater than 15%. (3) Approximately one-half of the compaction-induced lateral pressure dissipated in 12 hours after compaction. (4) The maximum compaction-induced lateral earth pressure was found to be a function of the undrained strength and the plasticity of the clay. The maximum lateral pressures observed in laboratory were in the range of 20% to 100% of $S_u$. (5) When a specimen was inundated, the lateral earth pressure first increased due to swelling, and then decreased due to softening of clay. The authors concluded the following from the results of the pilot studies. (1) The total lateral earth pressures induced by compaction were likely to decrease with time, while both water content and applied total vertical stress remained constant. (2) Passive failure conditions were not reached during compaction. (3) The lateral pressures observed in the pilot studies at the end of compaction were in the range of 20% to 40% of $S_u$. The difference between these values and the results of the laboratory tests was attributed to higher side friction in laboratory tests. (4) A significant amount of compaction-induced lateral earth pressure appeared to dissipated during compaction. (5) Back calculated values of friction angle were much greater than the value obtained by conventional triaxial tests, and was similar to the values obtained by stress relief tests.</td>
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<td>1991</td>
<td>Garga, V. K., and Khan, M. A.</td>
<td>The authors proposed a new method to evaluate in situ horizontal stresses in heavily overconsolidated clays. The concept of the proposed method is that an undisturbed specimen which is isotropically consolidated to the in-situ vertical stress will experience significant strain only in vertical direction, unless the horizontal stress exceeds the in situ horizontal value. The proposed method was compared with some existing methods.</td>
<td>The authors concluded that: (1) The proposed method provided good results: (2) $K_0$ values estimated by the self-boring pressure meter were greater than those measured by the proposed method. $K_0$ values estimated by oedometer tests were smaller than those measured by the proposed method. The dilatometer gave similar values of $K_0$ to the proposed method. (3) Empirical correlations consistently underestimate $K_0$ values: (4) The $K_0$ - OCR relation is similar for remoulded and undisturbed samples.</td>
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<td>1992</td>
<td>Clayton, C. R. I. and Symons, I. F.</td>
<td>Discussion of pressures of compacted backfills on unyielding walls, based on recent experimental findings. Methods for estimating lateral pressures were reviewed.</td>
<td>It was considered that compaction-induced lateral pressures in granular soils are predictable using simple equations. It was observed that compaction-induced pressures did not exceed 20 to 30 kPa, and that the depth of influence did not exceed 3 to 4 m, even with heavy rollers. There is no quantitative way to calculate the compaction-induced pressures in cohesive soils. It was found that total lateral pressures may exceed 100 kPa, and may be greater than the passive values calculated using the effective friction angle obtained by conventional triaxial tests.</td>
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<td>1992</td>
<td>Diviney, J. G.</td>
<td>The author described case histories of the Eisenhower Lock and the Smell Lock, both of which are located on the Saint Lawrence Seaway. These Locks were designed in 1940's and constructed in the early to mid-1950's. Both of these locks consist of massive, unrefined concrete gravity structures with heights of approximately 110 feet.</td>
<td>The lateral earth pressures measured by in situ pressuremeters and hydrofracture tests were much higher than at-rest pressures, and corresponded to lateral earth pressure coefficients ranging from 0.7 to 2.0. The original designs used $K_0 = 0.25$. These high values of lateral earth pressures caused cracks and leakage in the locks, and also may lead to instability. The author described remediation designs for these locks.</td>
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Table 2-1. Cont. Literature Review on At-Rest and Compaction-Induced Lateral Earth Pressures

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<td>1992</td>
<td>Filz, G. M.</td>
<td>Instrumented retaining wall tests were performed on backfills of moist Yatesville silty sand and Light Castle sand using the same facility as Sehn (1990). The backfill was compacted in 6-inch lifts using a vibratory roller and a rammer compactor. The compactors were instrumented for measuring dynamic force and energy transferred to the backfill. A model for calculating dynamic compactor forces was developed based on Lysmer and Richart's (1966) model. The compaction-induced lateral earth pressure theory developed by Duncan and Seed (1986) was extended to consider lateral pressures in moist soils where pore pressures play a significant role. The revised model was applied to the case histories of Eisenhower and Snell Locks, and the calculated pressures were compared to field data. A model for estimating vertical shear forces on rigid walls with vertical backs was developed.</td>
<td>Significant drift and scatter were observed in the pressure cell measurements. Horizontal and vertical force measurement by load cells were stable and free from scatter. Measured peak dynamic forces of compactors were quite different from the manufacturer’s ratings. It was found that the measured peak forces increased with soil stiffness; however, measured transferred energy was independent of soil stiffness. The dynamic compactor forces calculated by the new model were in good agreement with the measurements. The compaction-induced lateral earth pressure theory was revised to include memory erasure. In the extended theory, the pore pressure behavior was modeled using Skempton’s (1954) pore pressure parameters, and the parameters were estimated from total and effective strength parameters. Compaction-induced lateral pressures measured in the instrumented retaining wall tests were found to be higher when the rammer compactor was used than when the vibrating plate compactor was used. The revised model provided good agreement with the measured lateral earth pressures in a backfill of Light Castle sand. Lateral pressures for the moist Yatesville silty sand with a degree of saturation of about 65% calculated by the revised model without considering pore pressure effects were found to be in good agreement in the measured pressures. For Yatesville silty sand with a degree of saturation equal to 85%, the extended theory provided better agreement; however, parameters giving good agreement were quite different from the values obtained from laboratory tests. It was concluded that the vertical earth force coefficient, $K_v$, is about 0.2 for walls which are over 20-feet high, rigid, with vertical back sides, and backfilled with cohesionless soil.</td>
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Fig. 2-1. Stress Paths Obtained by $K_0$ Triaxial Tests on Monterey #20 Sand - after Wright (1969)
(1) As the axial stress increases, the lateral stress increases. The ratio of lateral to axial stress is defined as $K_0$, the coefficient of earth pressure at rest:

$$K_0 = \frac{\sigma_h'}{\sigma_v'}$$  \hspace{1cm} (2.1)

where

$K_0$ = the coefficient of earth pressure at rest,
\(\sigma_h'\) = effective horizontal stress,
\(\sigma_v'\) = effective horizontal stress,

Once the effects of the initial isotropic stress have been overcome by application of higher pressures, the value of $K_0$ remains essentially constant as $\sigma_v'$ increases. This is true for both the dense specimen ($e_0 = 0.55$) and the loose specimen ($e_0 = 0.73$).

(2) The value of $K_0$ for loading is smaller for the dense specimen than for the loose specimen. The value of $K_0$ was equal to 0.33 for the dense specimen, and $K_0$ was equal to 0.47 for the loose specimen.

(3) During unloading, the magnitude of $\sigma_h'$ decreases as $\sigma_v'$ decreases. However, the value of $K_0$ ($= \sigma_h'/\sigma_v'$) increases continuously as the values of $\sigma_v'$ and $\sigma_h'$ decrease. This is shown in Fig. 2-2, where the value of $K_0$ is plotted as a function of the overconsolidation ratio, OCR, which is defined as:

$$OCR = \frac{\sigma_{v',\text{max}}}{\sigma_{v',\text{current}}}$$  \hspace{1cm} (2.2)

A number of researchers have found that the empirical correlation suggested by Jaky (1944) provides good approximation for at-rest earth pressure coefficients. Jaky's equation is

$$K_0 = 1 - \sin \phi' \text{ for } OCR = 1$$  \hspace{1cm} (2.3)
Fig. 2-2. Relationship Between $K_0$ and Overconsolidation Ratio (OCR) for Monterey #20 Sand - after Wright (1969)
Mayne and Kulhawy (1981) collected more than 170 values of $K_0$ for various types of soil which were reported in the literature, and evaluated the data statistically. They concluded that Jaky's (1944) equation provides good agreement with the data for both sands and clays during primary loading.

Brooker and Ireland (1965) suggested a slightly different relationship based on their laboratory tests on clay, as follows:

$$K_0 = 0.95 - \sin \phi' \quad \text{for OCR} = 1 \quad (2.4)$$

The differences between values of $K_0$ calculated using Eq. (2.3) and Eq. (2.4) are not very important under most conditions.

Brooker and Ireland (1965) showed that values of $K_0$ for clay during unloading vary systematically with overconsolidation ratio. Schmidt (1966) proposed the following equation based on Brooker and Ireland's (1965) data:

$$K_0 = (1 - \sin \phi') \cdot (OCR)^{\sin \phi'} \quad \text{for OCR} > 1 \quad (2.5)$$

Mayne and Kulhawy (1982) showed that Schmidt's equation provided good agreement with the data that they reviewed, for both sands and clays.

Duncan and Seed (1986) used the following, slightly different, relationship in their hysteretic $K_0$ model:

$$K_0 = (1 - \sin \phi') \cdot (OCR)^\alpha \quad \text{for OCR} > 1 \quad (2.6)$$

where $\alpha$ = constant which is a function of $\phi'$, but not equal to $\sin \phi'$ (dimensionless).

Duncan and Seed (1986) suggested the values of parameter $\alpha$ as shown in Fig. 2-3.

Sehn (1990) studied cyclic $K_0$ behavior by applying many cycles of loading and unloading to specimens of Monterey sand in an instrumented oedometer. Results from one of his tests are shown in Fig. 2-4.

During reloading, the value of $\sigma'_h$ increases again as $\sigma'_v$ increases. The rate of change in $\sigma'_h$ with change in $\sigma'_v$, (called $K_{0A}$), is smaller than the value of $K_0$ for primary
Fig. 2-3. Recommended Values of $\alpha$ - after Duncan and Seed (1986)
Fig. 2.4. Results of Multicycle Monterey #0/30 Sand - from Sehe (1990)
loading. The value of $K_{0\Delta}$ depends on both the maximum value of $\sigma'_v$ and the value of OCR at which reloading begins. Mayne and Kulhawy (1982) showed that $K_{0\Delta}$ remains approximately constant until the stress path reaches the primary loading path, and then, as the value of $\sigma'_v$ continues to increases, the tangent value of $K_{0\Delta}$ becomes equal to the primary loading value of $K_0$.

Only a few studies have been performed to investigate at-rest pressures in partly saturated soils (Binnie and Price, 1941; Ofer, 1982; Clayton et al., 1991). Ofer presented results of $K_0$ oedometer tests on compacted clay. From the results of his tests, the following characteristics of $K_0$ in partly saturated soils can be defined:

1. The primary loading path does not go through the origin, as shown in Fig. 2-5.
2. The secant value of $K_0$ with respect to total stress, $(K_0^T)$, first decreases and then increases during primary loading. The tangent value of $K_{0\Delta}$ with respect to total stress, $(K_{0\Delta}^T)$, increases continuously during primary loading, as shown in Fig. 2-6. At high vertical pressures the value of $K_{0\Delta}^T$ approaches unity. The wetter the sample is, the greater the rate of the increase in $K_{0\Delta}^T$.
3. During unloading, the value of $K_0^T$ increases with OCR, as shown in Fig. 2-7. The drier the sample is, the greater the rate of increase in $K_0^T$. This variation of $K_0^T$ with OCR follows the form of Eq. (2.6).

Clayton et al. (1991) did not evaluate the effects of variation in water content, but found a similar trend for the variation of $K_0^T$ with OCR.

2.3 Effects of Non-Zero Lateral Strain

Deviation from the ideal at-rest condition can be caused by small lateral strains. Little information is available with regard to the values of lateral strain that cause appreciable deviation from at-rest conditions. Ofer (1982) performed oedometer tests
Fig. 2-5: Result of K₀ Oedometer Test on Compacted Clay by Ofer (1982)

Initial density = 78 pct
Initial water content = 30%
Fig. 2-6. Relationship Between $K_{0\Delta{}^T}$ and Vertical Stress During Primary Loading

- after Ofer (1982)
Fig. 2-7. Relationship Between $K_0^T$ and OCR During Unloading - after Ofer (1982)
with and without lateral strain to determine how much lateral strain can be tolerated. It was found that lateral strain of $6.0\times10^{-5}$ reduced the value of $K_o$ by 5 percent for a uniform sand, and by 11 percent for a well graded sand. Also, lateral strain of $1.3\times10^{-4}$ in the well graded sand was found to cause a 17 percent reduction in the value of $K_o$. These values of strain conform to the values suggested by Bishop (1958) and Schmidt (1967), who suggested that if the lateral strains do not exceed $1.0\times10^{-6}$, the conditions are close enough to at rest so that accurate values of $K_o$ can be measured.

The maximum value of lateral strain in $K_o$ laboratory tests should be less than the values reported by Ofer (1982), to be accurate within 5 percent to 10 percent. Among the tests described in the literature, only a few $K_o$ tests satisfied this criterion. These include the tests performed by Bishop (1958), Brooker and Ireland (1965), Wright (1969), and Ofer (1982). The tests performed in this study also satisfy these criteria.

### 2.4 Compaction-Induced Lateral Earth Pressures

The fact that soils can retain a significant part of the lateral stress induced by compaction loading or cyclic loading has been reported based on laboratory tests (Obrican, 1969; Sehn, 1990), model wall tests (Sowers, et al. 1957; Carder et al., 1977; Sherif and Mackey, 1977; Carder et al., 1980; Symons et al., 1989; Sehn, 1990; Filz, 1992), and field measurements (D'Appolonia et al., 1969; Rehnman and Broms, 1972; Symons and Murray, 1988). Also, case histories where failure or distress was caused by compaction-induced lateral earth pressures has been reported by Ingold (1979$^1$, 1979$^2$) and Diviney (1992).

The behavior of sand under one-dimensional cyclic loading is illustrated by the results of a $K_o$-oedometer test performed by Sehn (1990) which is shown in Fig. 2-4. The important characteristics of this behavior are:
(1) The value of $K_0$ at peak vertical pressure decrease as the number of cycles increases,
(2) The value of $K_0$ at high values of OCR increases as the number of cycles increases, and
(3) After the first few cycles, there are no significant further changes.

The effects of compaction on lateral earth pressures can be clearly seen in results of a series of model wall tests performed in the TRRL model retaining wall facility (Carder et al., 1977; Carder et al., 1980; Symons et al., 1989). Fig. 2-8 shows pressure distributions measured immediately after compaction of sand, silty clay, and highly plastic clay ("heavy clay") backfill. The distributions shown in Fig. 2-8 (b) and (c) are total stress. Although different materials were used for these backfills, the compaction-induced lateral earth pressures were found to be much higher than normal at rest pressures.

Only a few studies have been performed to investigate the factors affecting compaction-induced earth pressures in clay. Sowers et al. (1957) found that the residual compaction-induced lateral pressure in clay increased with a decreasing moisture content around optimum water content, and that it remained nearly constant for conditions dry of optimum, as shown in Fig. 2-9 (a). Also, Sowers et al. found that the compaction-induced lateral pressures increased with increasing compactive effort, as shown in Fig. 2-9 (b), and Clayton et. al. (1987) reported the same trend.

Changes in lateral earth pressures after compaction have been studied by several investigators (Sowers et al., 1957; Carder et al., 1980; Ofer, 1982; Symons et al. 1989). These studies show that many factors, including (1) relaxation, (2) change in pore pressure, (3) change in $K_0$ caused by change in effective stress, and (4) swelling and shrinkage of backfill materials, all influence the way earth pressures change after compaction. Both increasing and decreasing values of $K_0$ with time have been reported.
Fig. 2-8. Compaction-induced Lateral Earth Pressures Measured in Model Wall Tests
(a) Water Content

(b) Compactive Effort

Fig. 2-9. Factors Affecting Residual Lateral Pressures Induced by Compaction for Sandy Silty Clay - after Sowers et al. (1957)
Carder et. al. (1980) measured changes in lateral pressures with time for a model wall backfilled with silty clay. High lateral pressures were recorded after compaction, as shown in Fig. 2-8 (b), and they decreased with dissipating excess pore pressures during consolidation as shown in Fig. 2-10 (a). It was found that the behavior could be explained as a result of dissipation of positive pore pressures.

Change in moisture content is one of the most important factors affecting changes in earth pressures with time after compaction. Change in lateral pressures in compacted clay due to swelling were studied by Ofer (1980, 1981, 1982) using a $K_0$-oedometer. Symons et al. (1989) measured changes in lateral pressure with time in a model wall backfilled with a "heavy clay". High lateral pressures were recorded after compaction as shown in Fig. 2-8 (c), and they increased considerably after compaction due to swelling, as shown in Fig. 2-10 (b). After increasing rapidly, the lateral pressures decreased gradually with time. Mawditt (1989) reported a similar case in which highly overconsolidated clay generated high lateral pressures due to swelling.

Clayton et. al. (1991) hypothesized the stress paths shown in Fig. 2-11 for earth pressures after compaction in backfills compacted wet and dry of optimum. In the backfill compacted wet of optimum, positive pore pressures are developed during compaction; subsequently, consolidation occurs. In the backfill compacted dry of optimum, negative pore pressures are developed during compaction; subsequently, swelling takes place. During swelling, it was suggested that the horizontal total stress would first increase, and then decrease after the stress path reaches the passive failure line.
(a) Decreasing Lateral Stress due to Dissipation of Pore Pressure - from Carder et al. (1980)

(b) Increasing Lateral Stress Due to Swelling - from Symons et al. (1989)

Fig. 2-10. Change in Lateral Stress After Compaction
Fig. 2-11. Hypothesized Stress Paths for a Clay Backfill Caused by Swelling and Consolidation After Compaction - after Clayton et al. (1991)
2.5 Analytical Models for Estimating Compaction-Induced Lateral Earth Pressures

2.5.1 Broms's (1971) Model

Broms (1971) developed a simple analytical model for compaction-induced earth pressures in free-draining granular materials, based on studies previously reported (Sowers et al., 1956; Rowe, 1950 and 1954). As shown in Fig. 2-12, Broms's model assumed that the lateral stress induced by the maximum compaction stress would be completely locked in until the principal stress ratio reached the limiting earth pressure coefficient. The characteristics of Broms's model are:

(1) The stress path for primary loading follows the normally consolidated $K_o$ stress line (AB or A*B).

(2) The maximum vertical pressure induced by compaction can be calculated using Boussinesq's equation:

(3) For a depth smaller than critical depth, the horizontal pressure remains constant (BD), and is equal to $K_o \sigma_v'_{\text{max}}$. The critical depth is given as $K_o \sigma_v'_{\text{max}}/K_{\text{lim}} \gamma$, where $K_{\text{lim}}$ is a limiting lateral earth pressure coefficient for unloading, and $\gamma$ is the moist unit weight of the backfill.

(4) For a point at a depth less than the critical depth, the stress path reaches the $K_{\text{lim}}$ line and then moves along this line (BCD*), as $\sigma_v'$ continues to decrease. The lateral pressure for this condition is given by $K_{\text{lim}} \sigma_v'$.

According to Broms's model, the lateral pressure distribution after compaction is trapezoidal. The lateral pressure increases linearly from the ground surface to the critical depth, and remains constant below that depth.

The model was slightly modified by Ingold (1979(1),(2)) and Carder et al. (1980). Carder et al. (1980) took into account pore pressure development during compaction.
**K_{lim}** line
\[ \sigma'_n = K_{lim} \sigma'_v \]

**K_{0}** line
\[ \sigma'_n \approx K_0 \sigma'_v \]

\(\sigma'_1\) = overburden pressure at a depth greater than the critical depth

\(\sigma'_2\) = overburden pressure at a depth less than the critical depth.

Fig. 2-12. Stress Paths During Compaction - after Broms (1971)
using Skempton's (1954) pore pressure parameters in order to estimate compaction induced lateral pressures in silty clay.

2.5.2 Duncan and Seed's (1986) Model

The Broms's model and the models based on it use simple linear segments to approximate a hysteretic stress path during compaction. This simplification leads to inaccurate values of $\sigma_\text{h}'$ in some cases. Duncan and Seed (1986) developed a hysteretic $K_0$ loading-unloading model for estimating lateral earth pressures induced by compaction. The proposed model is illustrated in Fig. 2-13. The characteristics of the model are:

1. The virgin loading path (OA) follows the normally consolidated $K_0$ line, and the value of $K_0$ is estimated using Jaky's (1944) equation (Eq. 2.3).

2. The unloading stress path (AB) is nonlinear until the path reaches the passive failure limiting condition. This relationship is described by Eq. (2.6).

3. Thereafter the unloading path (BC) follows the passive failure line.

4. For reloading, the stress path (CD) is linear toward a target point (R) on the $K_0$ line, which is determined from the past maximum vertical stress and the vertical stress at the reloading point (C). After the stress path intersects the normally consolidated $K_0$ line (RA), the stress path follows this line as $\sigma_\text{v}'$ continues to increase:

5. Upon unloading from point D above the normally consolidated $K_0$ line, the stress path (DE) is nonlinear.

6. The reloading path from point E (EF) is linear toward the unloading point (D).

It was found later that the model considerably overestimated compaction-induced pressures for conditions where very large compaction equipment was operated.
Fig. 2-13. Effective Stress $K_0$ Model by Duncan and Seed (1986)
immediately adjacent to a wall, and the stresses induced during compaction were very high. Based on this finding, Filz (1992) revised the model to eliminate the inaccuracy.

In Duncan and Seed's (1986) hysteretic $K_0$ model, precedent stress paths are memorized by the soil. Filz (1992) incorporated a rule of memory erasure into the $K_0$ model to improve the accuracy for very high stresses during compaction. Figure 2-14 (a) illustrates the rule of memory erasure upon reloading. If the soil is reloaded from point D to E and crosses a preceding unloading path at C, the exceeded loop CDC (a dotted line) is removed from the model memory. The model memory after the interior loop is removed is shown as a solid line in Fig 2-14 (a). Memory may be erased during unloading in the same manner.

A reloading path is defined by a target point, which is a function of the maximum past pressure, as explained previously. For the case in which an unloading stress path reaches the $K_{lim}$ line, the maximum past loading point is reset, as shown in Fig. 2-14 (b). If the soil is unloaded from point A to B and then from B to C on the $K_{lim}$ line, the stress path ABC is removed from the model memory. The maximum past loading point A is reset to a virtual point E, which is determined by a virtual unloading path intersecting the $K_{lim}$ line at the current stress state, point C. The reloading path from point C to D goes toward the target point R determined by the location of the virtual point E.

**2.5.3 Filz's (1992) Extended $K_0$ Model**

The effective stress $K_0$ model described in the previous section is applicable only to dry cohesionless soils, because it does not include pore pressures. However, in practice, moist soils with significant fine contents are often used as backfill materials. In some conditions, significant pore pressures are developed during compaction. Filz (1992) extended that the effective stress $K_0$ to consider the behavior of moist soils.
Fig. 2-14. Modification of Effective Stress $K_0$ Model - after Filz (1992)
The pore pressures developed during compaction can be related to the changes in total stress. To compute the total stresses exerted on wall, these pore pressures are added to the effective stress calculated by the effective stress $K_o$ model. The pore pressures can be calculated by adding the changes in pore pressure to the initial pore pressures. Filz (1992) made the following assumptions to estimate the pore pressures in the backfill.

(1) The pore air pressure during compaction is zero:

(2) The pore pressure parameters $A$ and $B$ are constant throughout compaction; the suggested value for $A$ was 0.5, and

(3) The value of the initial pore pressure, and the pore pressure parameter $B$, can be estimated based on the relationship between the total stress and effective stress strength envelopes.

Based on these assumptions, the values of Skempton's pore pressure parameter $B$ and the initial equivalent pore pressure $u_0$ can be calculated using these equations:

$$B = \frac{\sin \phi' - \sin \phi}{\{1 + (2 \cdot A - 1) \cdot \sin \phi\} \cdot \sin \phi'} \quad (2.7)$$

$$u_0 = \frac{1 + (2 \cdot A - 1) \cdot \sin \phi' \cdot \cos \phi \cdot \tan \phi' + \frac{c'}{\tan \phi'}}{1 + (2 \cdot A - 1) \cdot \sin \phi \cdot \sin \phi'} \quad (2.8)$$

Using these equations, the pore pressure at a given state of stress can be calculated.

Point B in Fig. 2-15 shows the initial stress state considered in the extended model. There is an offset between the total stress axis and the effective stress axis, because of the initial negative pore pressures. Filz (1992) considered that the initial total vertical stress and the initial total horizontal stress are zero before compaction, and assumed that a stress path OAB was followed to reach the initial state B. The stress path from the initial condition follows BR in the effective stress space. The total stress space shifts down
Fig. 2-15. Initial Stress of Moist Soils Considering the Effects of Negative Pore Pressures in Extended $K_0$ Model - after Filz (1992)
diagonally, and the offset between the effective stress axis and the total stress axis becomes smaller, as the pore pressures increase due to loading. Consequently, the corresponding stress path in the total stress space is steeper than that in the effective stress space.

2.5.4 Method for Estimating the Compaction-Induced Earth Pressures by the $K_0$ model

A method for estimating residual lateral pressures after compaction using the hysteretic $K_0$ model was proposed by Duncan and Seed (1986). The proposed method is described in this section. Duncan and Seed (1986) suggested the following procedure for estimating increases in lateral stresses induced by compaction.

(1) The lateral earth pressure increment at the depth of interest due to the surface applied compactor force is calculated by means of the Boussinesq elastic solution. The lateral stress increment is calculated by the most critical positioning of the compaction plant. The value of Poisson's ratio used in the Boussinesq equation $\nu'$ is estimated by the following empirical equation:

$$\nu = \frac{1}{2} (0.5 + \nu_0)$$

(2.9)

where

$$\nu_0 = \frac{K_0}{1 + K_0}, \quad \text{and}$$

(2.10)

$$K_0 = 1 - \sin \phi'$$

(2.11)

Substituting Eqs. (2.10) and (2.11) into Eq. (2.9), the following equation is obtained:

$$\nu = \frac{4 - 3 \sin \phi'}{8 - 4 \sin \phi'}$$

(2.12)
The calculated increase in the lateral pressure can be used directly for the free-field condition. The lateral pressure increment adjacent to the wall has to be doubled in order to account for the presence of a rigid wall.

(2) The lateral stress increment calculated in the previous step is converted to an equivalent vertical stress increment using Eq. (2.13).

$$\Delta \sigma_v = \frac{\Delta \sigma_h}{K_0}$$

(2.13)

where

$$\Delta \sigma_v = \text{increase in equivalent vertical stress},$$

$$\Delta \sigma_h = \text{increase in horizontal stress calculated by Boussinesq equation},$$

The calculated equivalent vertical stress is used in the $K_0$ model. Applying and removing an equivalent vertical stress represents an entire change in compaction force applied to the backfill in each lift.

Using this procedure, the same magnitude of compaction force may yield different values of change in lateral stress increment depending on the loading history. For uncompacted soils, the lateral stress increment calculated using the $K_0$ model are the same as those calculated using the Boussinesq solution, whereas, for previously compacted soils, they are smaller. Seed and Duncan (1986) concluded that using the suggested procedure in two-dimensional analyses provided good agreement with field data, even though the nature of compaction problems are three dimensional.

Duncan and Seed (1986) incorporated the $K_0$ model into an incremental analytical procedure in order to estimate the residual pressures induced by compaction. First, the soil parameters required for the $K_0$ model are determined. Subsequently, the following three computations are conducted for each lift of the backfill: (1) the increase in overburden pressure due to the self-weight of the fill lift is applied to the $K_0$ model; (2) the
equivalent vertical stress calculated by Eq. (2.13) is applied; and (3) the stress calculated using Eq. (2.13) is removed. The proposed incremental procedure requires such a large number of calculations that it was coded into a computer program, EPCOMPAC, in order to calculate the compaction-induced lateral earth pressures acting on a non-yielding wall (Filz, 1992).

2.6 Summary

Previous studies of at-rest and compaction-induced earth pressures provide considerable information and many concepts that are useful for the studies described in the following chapters:

- The behavior of dry cohesionless soils under at-rest conditions is well understood, and can be described with reasonable accuracy through the empirical relationships suggested by Jaky (1944), Schmidt (1967), and Duncan and Seed (1986).

- At-rest pressures in compacted clayey soils have been studied experimentally by investigators at TRRL (Carder et al., 1977; Carder et al., 1980; Symons et al., 1987), Sowers (1957), and by Ofer (1982). The effects of the changing compaction water content are shown clearly by the results of the tests performed by Ofer. Experiments show that earth pressures increase after compaction for swelling soils at low initial pressures, and decrease after compaction for non-swelling soils.

- The hysteretic model for analysis of compaction-induced earth pressures developed by Duncan and Seed (1986) has been found to work reasonably well for dry cohesionless backfills. The model has been extended in concept to moisture-sensitive soils with non-zero pore pressures by Filz (1992). To use
Filz's concepts for practical applications, it will be necessary to develop procedures for estimating the magnitudes of the pore pressures that develop during compaction.
CHAPTER 3

REVIEW OF PREVIOUS STUDIES OF PARTLY SATURATED SOILS

3.1 Introduction

The behavior of partly saturated soils has not been studied as much as the behavior of saturated soils, even though compacted fills are always only partly saturated. Compacted fills are often characterized in terms of total stresses, thereby circumventing the many problems involved in evaluating effective stresses in partly saturated soils. To develop a fundamental understanding of compaction-induced earth pressures in moisture-sensitive soils, however, it is important to examine their behavior in terms of effective stresses. The review described in this chapter was performed to provide a summary of previous work in this area, to serve as a basis for the experimental and analytical studies in later chapters. Table 3-1 contains a summary of the studies reviewed. For each reference, the nature of the study and the findings and conclusions are summarized.

The basic reason for the difficulties involved in the study of partly saturated soils is that the voids contain both air and water, as opposed to only water in saturated soils. Most investigators, notably Hilf (1956), and Bishop (1959), have treated partly saturated soils as a three-phase material consisting of solid particles, water, and air. Fredlund and Morgenstern (1977), and Allam and Sridhadrnan (1987), considered partly saturated soils to be four phase materials, with the additional component being an air-water interface (contractile skin). The fact that such a fundamental issue has not been resolved indicates that much is still to be learned about how best to characterize the behavior of partly saturated soils in terms of effective stresses.

Partly saturated soils are often classified based on whether the air phase is occluded or is continuous (Bishop, 1959; Jennings and Burland, 1962; Barden, 1965). Barden's
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<th>Year</th>
<th>Author</th>
<th>Nature of Study</th>
<th>Principal Findings and Conclusions</th>
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<td>1956</td>
<td>Hilf, J. W.</td>
<td>The factors controlling pore water pressures in compacted cohesive soils were studied. Pore water pressure was separated into two components: capillary pressure and air pressure. A no-flow pore pressure measuring device was developed for testing partly saturated soils. Triaxial tests were performed on the specimens of an alluvial clay and a residual soil. The author proposed a new technique to test partly saturated soils. In this method, the pore water pressures to be measured were maintained positive by applying a positive ambient air pressure to an unsealed specimen (termed the axis translation technique). Compaction control in embankments constructed of cohesive soils was discussed.</td>
<td>The curvature of the menisci in partly saturated soils is independent of pore air pressure; therefore pore water pressure can be considered as the sum of capillary pressure and pore air pressure. The value of pore air pressure after compression without drainage was theoretically derived using Boyle's law and Henry's law. The measurement of negative pore water pressures less than one atmospheric was difficult, even with special porous tips. The axis translation technique was developed to overcome this difficulty, and was applied satisfactorily. The change in pore air pressures during compaction and the effect of capillary pressures were described. Settlement on inundation of soil loaded in an oedometer was attributed to shear failure caused by the reduction of capillary pressure. A new method for control of compaction was developed. The proposed method does not require measurement of water content, and it allows more rapid compaction control.</td>
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| 1959 | Bishop, A. W.| The author discussed the principle of effective stress, and extended it to partially saturated soils.                                                                                                                                                                                        | 1. The author considered that the stress-strain characteristics of unsaturated soils were governed by two factors, \((\sigma-u_a)\) and suction \((u_a-u_w)\), where \(\sigma\) = total stress, \(u_a\) = pore air pressure, and \(u_w\) = pore water pressure. Consequently, he developed an equation for effective stress in unsaturated soils, and in this equation he introduced a coefficient \(\chi\), a multiplier of the suction term.  
2. The coefficient \(\chi\) was considered to be unity for a saturated soil and zero for dry soil. The proposed equation reduces to Terzaghi's effective stress equation for a saturated soil. The effective stress of either a soil containing a large number of small bubbles or a soil containing a limited number of large bubbles can be approximated with the same equation. The parameter \(\chi\) is close to unity for the former case, and the term \((u_a-u_w)\) is small for the latter case. |
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<th>Year</th>
<th>Author</th>
<th>Nature of Study</th>
<th>Principal Findings and Conclusions</th>
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<td>1960</td>
<td>Bishop, A. W., Alpan, I., Blight, G.E. and Donald, I. B.</td>
<td>Review of the effective stress equation for unsaturated soils proposed by Bishop (1959), and application of this equation to soil strength. Triaxial compression tests were conducted on several types of soils in order to examine the relation between the coefficient ( \chi ) in the proposed effective stress equation and the degree of saturation.</td>
<td>1. Changes in negative pore water pressures were measured for remolded London clay. The pore pressures decreased with time after remolding. The authors concluded that this decrease is caused by changes in the clay-water system. 2. It was concluded that the effective stress equation proposed by Bishop (1959) described the behavior of partly saturated soils satisfactorily.</td>
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<tr>
<td>1961</td>
<td>Aichison, G. D.</td>
<td>A relationship between effective stress and moisture stress (suction) was proposed for partly saturated soils. In this relationship, the author introduced the pressure deficiency, ( p^* ), which is equivalent to matric suction.</td>
<td>The author concluded that the matric suction is the only significant component of suction up to total suction values of about 220 kPa. On the basis of a capillary model, the author proposed an effective stress equation, which includes the pore pressure deficiency. The author considered that the pressure deficiency is dependent on degree of saturation and void ratio. The relation between a parameter in the proposed equation, ( \psi ), and Skempton's pore pressure parameter, ( B ), was discussed.</td>
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<tr>
<td>1961</td>
<td>Bishop, A. W.</td>
<td>Pore pressure measurement techniques for use in triaxial tests were described, particularly for negative pore pressures and for pore air and pore water pressures in partly saturated soils. The limitations of the techniques were discussed.</td>
<td>For relatively permeable soils, pore pressure may be measured at the bases of specimens, because pore pressure gradients are negligible at ordinary rates of testing. Pore pressures in partly saturated soils compacted at or below the optimum water content must be measured using porous discs having high air entry values. The testing rate must be slow. A period of 6 hours was required for the pore pressures in specimens of clay with 8 inches height to reach 90 percent equilibrium after compaction. Pore air pressures were measured satisfactorily using a single layer of glass fiber cloth, unless water contents were below the optimum water content.</td>
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<tr>
<td>Year</td>
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<tr>
<td>1961</td>
<td>Bishop, A. W. and Donald, I. B.</td>
<td>A triaxial apparatus was developed for testing unsaturated soils. The authors conducted CD triaxial tests on unsaturated silt specimens in order to verify the accuracy of the effective stress equation for unsaturated soils developed by Bishop (1959).</td>
<td>The same stress-strain relation was obtained, provided both ($\sigma_3 - u_a$) and ($u_a - u_w$) were held constant, where $\sigma_3$ = confining stress, $u_a$ = pore air pressure, and $u_w$ = pore water pressure. If one of these stress differences varied, the stress-strain relation changed. Therefore, it was concluded that the stress-strain characteristics of unsaturated soils are represented well by the effective stress equation proposed by Bishop (1959).</td>
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<tr>
<td>1961</td>
<td>Croney, D</td>
<td>Several methods for soil suction measurement were explained, and results achieved using some of the methods were presented. The author described the significance of soil suction and negative pore water pressures in some aspects of soil mechanics.</td>
<td>The relationship between suction and moisture content depends on the grain size of the soil, its compressibility, its density, and its mineralogy.</td>
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<tr>
<td>1961</td>
<td>Jennings, J. E.</td>
<td>A wavy plane was drawn through the contact points of soil particles. On the basis of force equilibrium with respect to the wavy plane, the author proposed an effective stress equation for unsaturated soils, within which air voids are continuous and connected to the atmosphere. Triaxial tests were conducted to verify the proposed relation. Soil suction was measured with an ordinary pore pressure measuring system by applying a very high cell pressure.</td>
<td>The proposed equation is based on the concept that the sum of the pore pressure and the effective stress is equal to total stress. The author introduced a factor ($\beta$) which relates negative pore pressure to soil suction in the proposed equation, since soil suction is the only measurable quantity. The $\beta$ factor physically represents the proportion of the water phase on the wavy plane. If the air phase is occluded, the $\beta$ factor is equal to unity. The author stated that there is a relationship between Skempton's (1954) parameter, B, and the $\beta$ factor, if the air is occluded. It was considered that Skempton's parameter, B, cannot be applied for high negative values of pore water pressure or for soil having air voids connected to the atmosphere, because: (1) Skempton's parameter, B, is based on a single compressibility, whereas two fluid phases with different compressibilities exist in partly saturated soils, and (2) the effective stress equation on which Skempton's parameter, B, is based is equivalent to the $\beta = 1.0$ condition in the proposed equation.</td>
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<tr>
<td>1961</td>
<td>Lumbe, T. W.</td>
<td>The author measured negative pore pressures in partly saturated specimens of Vicksburg silty clay and kaolinite. Specimens were prepared by either tamping or static compaction. The pore water pressure was measured by two methods: a porous ceramic disk with a high air entry value, and a probe of stainless steel which was used by Hilf (1956).</td>
<td>The author made the following observation based on his test results: (1) The residual pore water pressure became less negative, as the molding water content increased. (2) The residual pore water pressure increased slightly with increasing compactive effort. (3) Cooling either the sample or the mold caused a decrease in pore pressure. (4) The samples which had once been wetter than the molding water content had higher residual pore water pressures, and (5) Removing confinement from specimens of kaolinite caused a reduction in pore pressure, but there was no change in pore pressure for silty clay specimens.</td>
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<tr>
<td>1962</td>
<td>Bishop, A. W. and Henkel, D. J.</td>
<td>The effective stress principles for partly saturated soils were reviewed. A triaxial testing apparatus for partly saturated soils was described in detail. The changes in pore water and pore air pressures caused by isotropic stress increments were shown for two different water contents; in addition, variation of initial pore water and pore air pressures with water content were shown. Pore pressure parameters in partly saturated soils were discussed.</td>
<td>The authors stated that: (1) a negative effective cohesion intercept is obtained if the effective stress equation proposed by Bishop (1959) is used for saturated soils, (2) a large positive cohesion intercept is obtained if pore water pressures measured with a coarse porous disc are used, (3) the actual cohesion is usually much smaller than the value commonly assumed in design.</td>
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<td>1962</td>
<td>Coleman, J. D.</td>
<td>A constitutive equation was proposed to evaluate effective stresses in unsaturated soils. The author stated that the effective stress in an unsaturated soil should not be considered only as a stress equilibrium condition; stress increments also need to be considered.</td>
<td>The author proposed a constitutive equation for unsaturated soils. The equation is in incremental form, in which a strain increment is expressed as the summation of $d(\sigma_3-u_3)$, $d(\sigma_1-\sigma_3)$, and $d(u_\alpha-u_\beta)$ each multiplied with a coefficient. It was suggested that these coefficients are functions of stress and soil properties, however, the characteristics of the coefficients were not explained.</td>
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<tr>
<td>1962</td>
<td>Jennings, J. E. B. and Burland, J. B.</td>
<td>The authors explained the mechanism of suction as an internal force and pointed out the following inconsistencies in the effective stress equation proposed by Bishop (1959): (1) $(\sigma-u_\theta)$ and $\chi(u_\theta-u_w)$ should be evaluated to verify the equation; however, this cannot be accomplished since $\chi$ is unknown, and (2) The equation cannot account for collapse due to the release of suction.</td>
<td>The authors suggested that suction should be treated as an internal force rather than an external force. The other component of stress in Bishop's (1959) equation $(\sigma_3-u_\theta)$ is an &quot;external force&quot;. Suction is generated due to the meniscus at the contact point of soil particles. The suction causes a normal stress at the contact point, such that the resistance against external forces increases. This means that the release of internal force reduces the internal resistance; consequently, collapse occurs. Collapse is more significant in oedometer tests than in isotropic consolidation tests because of the shear stresses in oedometer tests.</td>
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<td>1963</td>
<td>Bishop, A. W., and Blight, G. E.</td>
<td>The authors conducted triaxial compression tests to examine the characteristics of unsaturated soils, and to verify the effective stress equation proposed by Bishop (1959).</td>
<td>Difficulties were encountered when attempts were made to use Bishop's (1959) equation to explain volume changes of soils. The stress path in terms of both $(\sigma-u_\theta)$ and $(u_\theta-u_w)$ had to be taken into account to describe volume changes successfully. Suction is a function of soil type, water content, shear stress, and $(\sigma-u_\theta)$. The coefficient $\chi$ in the Bishop's effective stress equation for unsaturated soils is related to the degree of saturation; however, the relationship is not linear.</td>
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<td>1963</td>
<td>Olson, R. E.</td>
<td>The process of compaction was explained in terms of effective stress principles. The effects of foot pressure and number of stress applications on dry density, as well as the shape of the moisture-density curve, were studied theoretically on the basis of the effective stress theory.</td>
<td>As the moisture content of a soil on the dry side of optimum is increased, the pore pressure coefficient $B$ and pore water pressures increase. Also, for the dry of the optimum, the dry density obtained by compaction increases with increasing water content. This may be explained with respect to residual negative pore pressures and total lateral stresses. The increase in dry density at low water content was more rapid during the first few stress applications. The air permeability decreased to zero at the optimum water content, where the air phase became discontinuous. A soil on the wet side of optimum may have enough strength to resist the foot pressure because of (1) the negative pore water pressures resulting from negative pore-pressure coefficients $A$, and (2) a mechanism similar to bearing capacity.</td>
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<td>1963</td>
<td>Yoshimi, Y. and</td>
<td>Experimental study of the one-dimensional compression behavior of partially saturated silty clay. The equilibrium state of pore fluids, the movement of pore fluids caused by the applied loads, and the time-compression relationship were investigated. One-dimensional compression tests were conducted on compacted specimens of partly saturated Vicksburg silty clay. The samples were compacted and loaded in a compaction mold with a 2.0-in. height and a 4.44-in. diameter.</td>
<td>The authors found that: (1) virtually no outflow of water was observed during compression that changed saturation from 70 % to 97 %, (2) the air phase formed interconnected channels even in the case of high saturation, (3) the rate of compression was independent of sample thickness or drainage conditions, and (4) the strain rate for virgin compression increased with stress increment. The authors speculated that: (1) the pore water pressure is less than atmospheric at equilibrium and the pore water tends to fill smaller voids rather than large pore space, (2) the pore water remains in the soil when the applied stress does not increase the initially sub-atmospheric pore water pressure to exceed atmospheric pressure, and (3) the rate of compression is governed by the viscosity of the soil structure, if the movement of pore water is negligible.</td>
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<td>1965</td>
<td>Alpan, I.</td>
<td>Investigation of time rate of consolidation for unsaturated soil. A triaxial cell was modified to measure the expelled water and air separately.</td>
<td>A change in the degree of saturation caused by the drainage of water and air influenced the permeability of the soil significantly. The dissipation function for saturated soil was modified for unsaturated soil by means of a correction factor.</td>
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<td>1965</td>
<td>Burden, I.</td>
<td>Discussion of the following categories of compacted and unsaturated clays: (1) extremely dry clays (Sr&lt;0.5), (2) dry of optimum (0.5&lt;Sr&lt;0.9), (3) clays in the region of optimum water content, (4) wet of optimum (Sr&gt;0.9), (5) very wet clays (Sr&gt;0.95). The proposed governing equation was a partial differential equation which took into account the effects of decreasing permeability during consolidation and the compressibility of the pore fluid.</td>
<td>The air phase is continuous for clays dry of the optimum water content, and for this condition, the value of air permeability is too high to maintain high pore air pressures. The computed results using the proposed formulation showed that: (1) the settlement-square root of time relation is continuously curved, and (2) a plot of pore pressure dissipation at the mid-plane with time was flatter than the conventional curve for saturated soils. The author speculated that the deviation from the Terzaghi theory for compacted clay in the region of optimum or wet of optimum is caused by the decrease of permeability during consolidation.</td>
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<tr>
<td>1965</td>
<td>Blight, G. E.</td>
<td>The volume change characteristics of partly saturated soils were studied. The author studied two soils, an expansive clay and a clayey loess. Three types of tests were performed on partly saturated soils: (1) compression at constant water content, (2) desiccation or moistening under constant load, (3) increase of moisture content at constant volume. The author proposed a procedure to determine the swelling pressure from tests of the third type.</td>
<td>The author classified the volume change of partly saturated soils into the two following categories: (1) compression at constant water content, and (2) desiccation or moistening under constant load. The author concluded that: (1) Most of the volume change behavior of partly saturated clayey soils can be explained with the principle of effective stress, (2) However, collapse phenomenon is beyond the concept of the effective stress principle, (3) The effective stress function is dependent on the process of loading, (swelling at constant pressure showed the highest values of $\chi$, and compression at constant water content showed the lowest), and (4) The swelling pressure test may be effective for the determination of stresses in collapsing soils.</td>
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<td>1965</td>
<td>Burland, J. B.</td>
<td>Review of the effective stress principle for partly saturated soils, its limitations, and its practical application. Qualitative estimation of volume change in partly saturated soils was attempted by using the concept of grain contact stability.</td>
<td>The author concluded that: (1) the effective stress principle cannot be used to develop a theory that explains the behavior of partly saturated soils comprehensively, (2) however, the shear strength of partly saturated soils is explained well with the effective stress principle, (3) it is necessary to conduct a rigorous laboratory simulation of an in-situ stress condition in order to estimate the volumetric strain of partly saturated soils quantitatively, and (4) the behavior of partly saturated soils should be related to the two stress variables ($\sigma-u_n$) and ($u_n-u_w$).</td>
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<tr>
<td>1965</td>
<td>Olson, R. E. and Langfelder, L. J.</td>
<td>Measurement of negative pore water pressures for specimens of five different silty and clayey soils, using a pressure plate. The specimens were prepared by static and kneading compaction.</td>
<td>The measured pore water pressure was as low as -250 psi for samples 5% dry of optimum. The authors used microscopic analysis to examine the limiting value of pore water pressures in partly saturated soils. Theoretically, the actual pore water pressures in partly saturated soils vary from approximately -1 atmosphere to high positive values. They speculated that the measured pressure was the water pressure in a probe, not in the soil.</td>
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<tr>
<td>1966</td>
<td>Schuurman, Jr. E.</td>
<td>A relation between air and water pressures in a water/air mixture was derived, using Boyle's law, Henry's law, and the concept of surface tension. The compressibilities of air/water mixtures were discussed on the same basis. The proposed theory is valid for soils with degrees of saturation greater than 85%, in which the air forms occluded bubbles.</td>
<td>Pore air pressures were found to be greater than pore water pressures due to surface tension and vapor pressure. However, the effect of vapor pressure was found to be negligible. The value of the pore air pressure at which all the air in the mixture dissolves into water was theoretically determined. When pore pressures approaches a critical value, air bubbles in the air/water mixture collapsed. The author speculated that Skempton's (1954) pore pressure coefficients A and B could be estimated based on the compressibility of the air/water mixture and the relationship between pore air and water pressures.</td>
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<td>1967</td>
<td>Blight, G. E.</td>
<td>Triaxial tests on unsaturated specimens of a compacted silt and a compacted sandy clay. Variations of the effective stress parameter, $\chi$, with varying suction and strain were measured.</td>
<td>Two different methods for evaluating the effective stress parameter, $\chi$, from triaxial test data were proposed. The proposed methods give different values of the effective stress parameter, $\chi$. The author did not suggest which value is more accurate. For a given soil, the void ratio at failure under unsaturated conditions is quite different from that for saturated conditions. The value of the parameter, $\chi$, can be greater than unity in partly saturated soils close to saturation.</td>
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<td>1967</td>
<td>Sides, G. R. and Burden, J.</td>
<td>A series of long term compression tests was performed on specimens of partly saturated clay which were remolded at various water contents covering the range from continuous air phase to fully occluded air phase. Oedometer tests and triaxial tests were conducted using equipment modified to test partly saturated soils.</td>
<td>Measured pore water pressures decreased after reaching an initial peak value, and then increased to a final peak value that was sometimes higher than the initial peak value. The authors speculated that: (1) the initial peak was explained by Boyle's law and Henry's law, (2) the initial drop was caused by thixotropic action or the effect of Henry's law, and (3) the final peak was caused by structural viscosity of the soil skeleton.</td>
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<tr>
<td>1968</td>
<td>Leonard, J. Langfelder, A. Chen, C. F. and Justice, J. A.</td>
<td>Study of the factors that affect the air permeability of compacted cohesive soils. Specimens of five different cohesive soils with different percentage of fines were used for this study. The specimens were compacted using dynamic compaction, static compaction, and kneading compaction, with various values of compactive energy. Air permeability tests were performed using constant head or variable head.</td>
<td>(1) The air permeability of compacted cohesive soils changed by a small amount on the dry side of optimum, and changed several orders of magnitude at water contents close to optimum. The air permeability was approximately zero at optimum and higher water contents. (2) As compactive energy increased, the air permeability decreased. Kneading compaction resulted in smaller air permeability than dynamic compaction. (3) Air permeability increased with time after compaction. This appeared to be a thixotropic effect.</td>
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<td>1968</td>
<td>Matyas, E. I. and Radlakrishna, H. S.</td>
<td>A new theory replacing the conventional effective stress principle was proposed to explain the behavior of partly saturated soils. Triaxial tests were performed on statically compacted partly saturated specimens of a mixture of 80% non-plastic flint powder and 20% kaolin. A triaxial apparatus was modified, and pore air and water pressures as well as lateral strains were measured during the tests. Isotropic compression tests and $K_0$ compression tests were performed under conditions of either constant suction or decreasing suction to allow wetting.</td>
<td>The authors developed a new method to describe the behavior of partly saturated soils, where the soil behavior was expressed in terms of two state variables $(\sigma-u_a)$ and $(u_a-u_w)$. It was found that the proposed approach was more useful than previously used theories based on a single effective stress. The authors presented a concept of state surfaces representing the variations in void ratio and degree of saturation with respect to the two stress variables. The proposed method predicted fairly well the volumetric strains of partly saturated soils subjected to either isotropic or $K_0$ compression. The author concluded that the behavior of partly saturated soils is dependent on stress path.</td>
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<td>1972</td>
<td>Krahn, J. and Fredlund, D. G.</td>
<td>Two types of soils were tested: a glacial till of low plasticity and Regina clay with high plasticity. Total suction, matric suction and osmotic suction of partly saturated soils were measured independently. The matric suction was measured using a modified Anteus consolidometer with a high air entry porous stone. The osmotic suction was measured by using the electrical conductivity of either the extract of the pore fluid from a slurry paste or the pore fluid squeezed from the soil with a particular water content. The total suction was measured with a psychrometer.</td>
<td>The authors found significant differences between values of osmotic suction measured using saturation extract and using expelled pore water. The measured total suction was equal to the sum of the measured matric suction and the osmotic suction. This verified the subdivision of the total suction into matric and osmotic suction. The matric suction and the total suction are dependent on the water content and independent of the dry density.</td>
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<td>1975</td>
<td>Fredlund, D. G.</td>
<td>Description of a diffused air volume indicator which measures the amount of air diffused through a saturated high air entry porous disk.</td>
<td>A correction procedure for air diffusion was explained. The indicator was tested under high matric suctions (75 to 98 psi), and the observed diffused air volume was 0.23 cc per day. The author stated that most laboratory tests would likely be conducted under much lower suctions, so that the diffused air volume would be smaller than the values he observed.</td>
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<td>1976</td>
<td>Fredlund, D. G.</td>
<td>Analysis of the density and compressibility of air-water mixture based on the conservation of mass, using Boyle's Law and Henry's Law. The formula obtained was expected to predict the density and compressibility of the fluid phase of partly saturated soils.</td>
<td>The author concluded that: &lt;br&gt;1) the density of an air-water mixture is not influenced by surface tension effects; &lt;br&gt;2) the compressibility of an air-water mixture is a function of a pore pressure parameter $B_{aw}$ ($= \Delta U_w/\Delta u_w$) which is an experimentally measured parameter accounting for surface tension effects. $B_{aw}$ is less than unity for undrained loading, and it is equal unity when the air bubbles are occluded; &lt;br&gt;3) air that is only a small percentage of the total volume can increase the compressibility of the air-water mixture by several orders of magnitude.</td>
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1976 | Fredlund, D. G. and Morgenstem, N. R. | Development of constitutive equations to describe volume changes of partly saturated soils. Anteus oedometers were modified to test one-dimensional strain conditions, and triaxial cells were modified for isotropic consolidation. Air flushing systems were used in both types of tests to measure the volume of diffused air. Tests were conducted on specimens of overconsolidated undisturbed Regina clay and compacted kaolin. | The volume change constitutive relation for partly saturated soil consists of two equations: the equation for the soil structure, and the equation for the water phase. Tests results on compacted kaolin showed poor agreement with the calculated volume changes for the soil structure. However, correction for hysteresis improved the prediction. Prediction of behavior for the water phase is more difficult than for the soil structure. Corrections for nonlinearity and hysteresis were made. |
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<th>Year</th>
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<th>Principal Findings and Conclusions</th>
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<tr>
<td>1977</td>
<td>Fredlund, D. G. and Morgenstern, N. R.</td>
<td>Three possible sets of stress variables governing the behavior of partly saturated soils were proposed. The authors proposed considering a partly saturated soil as a four-phase material, including the air-water interface called the contractile skin as a separate phase. A force equilibrium equation was formulated for each phase, and the stress variables were extracted from these equations. The proposed stress variables were verified with experimental data. The authors performed laboratory tests on partly saturated kaolin, saturated kaolin, and saturated silt to verify the proposed stress variables. Anteus oedometers and triaxial cells were modified for the tests. In the tests, the overall volume change was kept zero, that is, the equilibrium of the soil structure and the contractile skin was maintained. These tests were called null tests.</td>
<td>Under an applied pressure gradient, the soil particles and the contractile skin come to equilibrium, but the air phase and the water phase flow.</td>
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<td>1978</td>
<td>Fredlund, D. G., Morgenstern, N. R. and Widger, R. A.</td>
<td>Review of triaxial tests on compacted shale, Boulder clay, and a mixture of Potters Flint and Peerless Clay. The data were analyzed by assuming constant values of the effective friction angle with respect to soil suction, $\phi_b$, for each test.</td>
<td>The authors proposed two shear strength equations which were extended from the conventional Mohr-Coulomb failure criteria. One equation is expressed in terms of $(\sigma-u_w)$ and $(u-a_{u_w})$, and the other is expressed with respect to $(\sigma-u_a)$ and $(u-a_{u_a})$. The authors concluded that the latter would be more useful for engineering purposes.</td>
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<td>1979</td>
<td>Bocking, K. A., and Fredlund, D. G.</td>
<td>Development and use of an osmotic tensiometer to measure negative pore pressure in soils directly. The osmotic tensiometer consists of a strain gage pressure transducer and a chamber where a semipermeable membrane contains a solution of polyethylene glycol.</td>
<td>The authors concluded that the osmotic tensiometer is only useful for research applications where temperature and pore pressure are constant, because: (1) The reference pressure used in data reduction drifts significantly with time, (2) The response of the osmotic tensiometer to the change in the pore pressure is slow, and (3) The effect of temperature varies with the reference pressure.</td>
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<td>1979</td>
<td>Fredlund, D. G.</td>
<td>State-of-the-art report regarding partly saturated soils. The author explained the nature of partly saturated soil. Two independent stress tensors proposed by Fredlund and Morgenstern (1977) were reviewed. Equations for describing the mechanical behavior of partly saturated soils were proposed.</td>
<td>The author considered that a partly saturated soil can be treated practically in the similar way as a saturated soil. The author proposed a logarithmic equation for the volume change in partly saturated soils, which was similar to that for saturated soils. Two compressive indices with respect to total stress and matrix suction respectively are used in the equation. The author considered that the three dimensional shear strength diagram for partly saturated soils can be considered as a series of conventional Mohr-Coulomb diagrams, each of which corresponds to a different value of matrix suction. For earth pressure problems and limiting equilibrium problems, partly saturated soil can be treated as saturated soils with apparent cohesion that is equivalent to the shear strength resulting from soil suction.</td>
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<td>1980</td>
<td>Bocking, K. A. and Fredlund, D. G.</td>
<td>Investigation of the axis translation technique proposed by Hilf (1956) for triaxial testing on partly saturated soils. A mathematical model was developed that described the triaxial testing system including a pore pressure measuring device, a porous disc with a high air entry value, and a specimen. Parametric studies were performed using the model.</td>
<td>The axis translation technique is theoretically correct for soils with inter-connected air bubbles. Soil suction is over-estimated for soils containing occluded air bubbles. Use of the axis translation technique results in volume change of specimens. This volume change is irreversible for soils with occluded air bubbles, and this leads to erroneous results.</td>
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<td>1980</td>
<td>Lloret, A. and Alonso, E. E.</td>
<td>Development of a constitutive model for one-dimensional consolidation of partly saturated soils. The proposed procedure can be used to calculate the instantaneous pore air and water pressures resulting from undrained loading. A finite element program was developed based on the proposed constitutive equations. The program was used to analyze (1) nonlinear saturated consolidation, (2) the evolution of the wetting front, and (3) collapse behavior upon wetting.</td>
<td>The proposed constitutive equations accounted for the volume change behavior of partly saturated soil including swelling and collapse behavior. The proposed procedure estimated that both initial pore air and water pressure increased to values very close to each other as a soil is subject to one-dimensional undrained compression. Knowledge of the state surface of partly saturated soil and that of the water and air permeabilities were required for the use of the proposed model.</td>
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<td>1980</td>
<td>Fredlund, D. G., Hasan, I. U., and Filson, H. L.</td>
<td>The authors reviewed methods of heave analysis proposed previously. A one-dimensional equation for heave analysis was developed based on the formulation by Fredlund and Morgenstern (1976). A new empirical method for predicting the amount of heave was proposed.</td>
<td>The proposed method required constant volume oedometer tests to determine the in-situ stress state. Correction for sample disturbance was discussed. It was found that interpretation of test results was quite difficult. Values of heave calculated using the newly proposed method slightly exceeded measured heaves.</td>
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<td>1981</td>
<td>Edil, T. B., Motan, S. E., and Toha, F. X.</td>
<td>The mechanical properties of partly saturated cohesive soils were studied using different types of tests: unconfined uniaxial compression tests and resonant column tests were conducted on laboratory consolidated kaolinite, and repetitive compression tests were conducted on laboratory compacted silt loam. Various values of the initial matric suction were applied to specimens using a pressure plate. The relation between the initial matric suction and the mechanical properties of partly saturated soils were discussed. Total suction was measured with a thermocouple psychrometer during the tests. The suction response under applied stress was studied by triaxial compression testing on specimens of low-plasticity silty clay compacted with a Harvard Miniature compactor. The effect of osmotic suction on response to repeated loading was studied by varying the concentration of NaCl in statically compacted Grundite clay specimens.</td>
<td>Matric suction reflects the mechanical behavior of partly saturated soils better than other indices such as water content or degree of saturation alone. From the unconfined compression tests, the authors found that: (1) The unconfined compressive strength increased as the initial suction increased, (2) The initial tangent modulus increased with increasing initial matric suction up to a critical value, 600 kPa, and decreased beyond that value, (3) The value of Poisson's ratio decreased with increasing initial matric suction and became less than 0.1. The authors found that the dynamic shear modulus and the resilient modulus were affected similarly to the static modulus by changes in initial matric suction. The critical values of the initial matric suction were found to be 400 kPa and 800 kPa respectively for these moduli. Matric suction rather than total suction controls the mechanical behavior of soils. Consequently, the use of psychrometers alone would give misleading information. The relationship between water content and suction shows hysteresis. Total suction decreases with increasing applied stress; however, this relationship is not simple, and depends on many factors.</td>
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<td>1981</td>
<td>Gulhati, S. K. and Satija, B. S.</td>
<td>Two types of triaxial tests, constant water content tests (CW) and drained tests (CD), were performed on partly saturated Dhanauri clay. In CW tests, pore air pressure were kept constant. In CD tests, both pore air pressure and pore water pressure were kept constant. Specimens were prepared by static compaction. The triaxial apparatus was modified to measure suction by using a ceramic disk with a high air entry value.</td>
<td>The shear strength of partly saturated soils can be expressed with two stress variables, ((\sigma_3-u_a)) and ((u_a-u_w)). These variables act independently. The authors concluded that there is no unique relation between saturation and the value of the parameter (\chi) introduced by Bishop (1959). The value of (\chi) was found to be greater than unity occasionally. Based on these results, they concluded that the equation proposed by Fredlund (1978) was preferable.</td>
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<td>1982</td>
<td>Ho, D. Y. F. and Fredlund, D. G.</td>
<td>A triaxial testing apparatus was modified to measure the strength of partly saturated soils. Pore water pressure and pore air pressure were controlled through a high air entry ceramic disc and a coarse corundum stone. A flushing system was used to get rid of the diffused air in the ceramic disc, and a diffused air indicator measured its volume.</td>
<td>A multistage testing procedure was used. The results were interpreted by using a known value of (\phi) obtained from triaxial tests on saturated specimens. The proposed multistage testing procedure can evaluate variations in friction angle with respect to soil suction, using a single test on a partly saturated specimen.</td>
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<td>1983</td>
<td>Chang, C. S. and Duncan, J. M.</td>
<td>Consolidation analysis for partly saturated clayey soils. The authors extended Biot's (1955) theory of consolidation to analyze partly saturated soils by employing an elasto-plastic constitutive model revised from a Cam Clay model to describe the behavior of partly saturated soils. The compressibility of partly saturated pore fluid and the permeability of partly saturated soils were studied.</td>
<td>The concept of an homogenized pore fluid was presented to describe the partly saturated pore fluid for soils with degrees of saturation high enough for air bubbles to be occluded. The compressibility of the homogenized pore fluid was derived by using Boyle's Law and Henry's Law. It was considered that surface tension effects are negligible for purpose of evaluating the compressibility of the homogenized pore fluid. An empirical equation for the permeability of partly saturated soils was used in the model. It was considered that the permeability of partly saturated soils is a function of the permeability in the fully saturated condition, the void ratio, and the degree of saturation. It was concluded that the proposed elasto-plastic constitutive model is appropriate for compacted clay with occluded air bubbles. A finite element program employing the model was found to be useful for consolidation analyses of partly saturated clay.</td>
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<td>1984</td>
<td>Edil, T. B. and Motan, S. E.</td>
<td>Construction and calibration of open-type thermocouple psychrometers. Statically compacted specimens of Grundite clay, which contains various amounts of soluble salts, were tested. Matric suction was induced in the specimens using a ceramic plate extractor. Total suction was measured with thermocouple psychrometers.</td>
<td>It was concluded that matric suction alone controls the strength behavior of partly saturated soils. Osmotic suction was deduced as the difference between the measured total suction and the induced matric suction. The deduced osmotic suction decreased with increasing matric suction, although the salt concentration retained by the specimens increased significantly. Two methods were used to measure osmotic suction directly: (1) The osmotic pressure of the pore-water extract from the soil was measured with a psychrometer, (2) A suspension of soil was made with distilled water. The conductivity of the supernatant was measured, and the osmotic suction was calculated from the measured conductivity. The psychrometer measurements gave more realistic values than the estimations based on the conductivity measurements. For specimens which contain very little salt in the pore water, psychrometric readings indicate the matric suction quite accurately.</td>
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<td>1985</td>
<td>Fredlund, D. G.</td>
<td>Stresses in unsaturated soils were divided into two matrices: [\sigma_{ij} - u_a d_{ij}] and [(u_a - u_w) d_{ij}]. Based on this definition, the following phenomena were discussed: (1) ground water and ground air flows, (2) shear strength and slope stability, (3) a constitutive equation based on linear elasticity, (4) one dimensional consolidation, and (5) pore pressure parameters.</td>
<td>(1) Flows of water and air were formulated separately, and the equation of water flow included a gravity term, (2) An unsaturated soil has two components of cohesion. The additional term is a function of ((u_a - u_w)), (3) The consolidation equation for unsaturated soil has two parts: one for the water phase and another for the air phase, (4) Skempton's (1954) pore pressure parameters can be defined for the water phase and the air phase separately in the case of an unsaturated soil, (5) Measurement of negative pore water pressure is very difficult, but improved techniques are being developed.</td>
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<td>1985</td>
<td>Lloret, A. and Alonso, E. E.</td>
<td>Mathematical functions to describe the state surfaces of void ratio and saturation, proposed previously by the authors, were discussed. The authors conducted several types of oedometer tests on specimens of compacted kaolin and Pisolitic clayey sand. A special oedometer developed for this purpose can control air and water pressure, and water content. The authors conducted isotropic compression tests on specimens of kaolin, using a modified triaxial cell. These results, and three other sets of results available in the literature, were analyzed.</td>
<td>The authors used least squares fitting techniques to compare a number of functions proposed to describe the state surface. These are linear and nonlinear functions with respect to pore air pressure, pore water pressure, and total stress. Some nonlinear functions showed good agreement with experimental results.</td>
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<td>1986</td>
<td>Escario, V. and Saez, J.</td>
<td>A direct shear apparatus was modified for shear strength tests on partly saturated soils. An air pressure chamber was constructed to enclose the shear box, and either a high air entry porous stone or a semipermeable membrane was used for pore water pressure measurement. Tests were conducted on statically compacted specimens of three different soils, Madrid grey clay, Red clay, and Madrid clayey sand.</td>
<td>The measured slopes of curves showing variations of shear strength with suction were not constant. Consequently, the equation proposed by Bishop (1959) is more realistic than Fredlund's equation, because the former is consistent with a nonlinear failure envelope with respect to suction. The effect of soil suction depends on the stress and capillary history of the soil, as well as on other factors.</td>
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<td>1986</td>
<td>Richards, B. G., Emerson, W. W. and Peter, P</td>
<td>The writers discussed the paper by Edil and Motan (1984).</td>
<td>Both matric suction and osmotic suction control the engineering behavior of dispersive clays. Total suction can be measured reliably, while matric and osmotic suctions are difficult to determine separately for dispersive soils.</td>
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<td>1987</td>
<td>Escario, V. and Saez, J.</td>
<td>The authors replied to the discussion on their 1986 paper.</td>
<td>The ratio of vertical stress to lateral stress in the oedometer changes as the specimen swells. The lateral stress goes to zero for high soil suction values.</td>
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<td>1987</td>
<td>Allam, M. M. and Sridhadran, A.</td>
<td>Derivation of an effective stress equation for partly saturated soils. The equation proposed by Bishop (1959) was modified by adding two terms, the osmotic suction and the effect of the contractile skin (the air-water interface).</td>
<td>Evaluation of the osmotic suction and the effect of contractile skin is desirable. The contractile skin stress was estimated theoretically based on the pore size distribution.</td>
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<td>1987</td>
<td>Josa, A Alonso, E. E. Lloret, A and Gens, A.</td>
<td>Development of a stress path triaxial cell for testing partly saturated soils. Pore air pressures were kept constant during the tests, and soil suction was controlled by means of pore water pressures applied through a porous stone with a high air entry value. Tests were performed on specimens of kaolin prepared by kneading compaction. Various stress paths were tested by maintaining shear stresses to be zero and by changing two isotropic stress variables, ( u_a - u_w ) and ( \sigma - u_a ), during the tests.</td>
<td>The authors concluded the followings based on the results of various stress path controlled tests. (1) The value of mean total stress influenced the development of plastic volumetric strain due to soil suction; therefore, the stress surface for partly saturated soils is not unique. (2) Yield loci dependent on both suction and mean stress might explain the stress-strain behavior of partly saturated soils, including collapse and swelling. (3) The volume change behavior for unloading or reloading was linearly proportional to the logarithm of soil suction or mean stress. (4) Irreversible swelling strain accumulated due to cycles of unloading and reloading.</td>
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<td>1988</td>
<td>Gan, J. K. M., Fredlund, D. G. and Rahardjo, H.</td>
<td>A conventional direct shear box was modified to test partly saturated soils. The modified apparatus can control pore air pressure and pore water pressure, and has a flushing system to control diffused air in a ceramic disc. Multistage direct shear tests were conducted on specimens of a glacial till which were compacted using the Standard AASHTO compaction test energy.</td>
<td>The authors considered that direct shear tests have merit in reducing the time required for testing, because of their short drainage path. The shear strength due to matric suction can be considered to be frictional or cohesive in nature. The authors found that the strength envelope with respect to suction was nonlinear, and they proposed several ways to handle this nonlinearity. As the degree of saturation becomes smaller due to higher matric suction, the angle of the envelope ( \phi ) decreases to a constant value.</td>
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<td>1988</td>
<td>Gan, K. J. and Fredlund, D. G.</td>
<td>A conventional direct shear box was modified to measure the shear strength of partly saturated soils. A chamber was constructed to enclose the shear box, and a ceramic disk with a high air entry value was installed on the base platen. Glacial till test specimens were statically compacted, and were sheared under either saturated or unsaturated condition. In tests on partly saturated specimens, various matric suctions were applied by controlling both pore water pressure and pore air pressure.</td>
<td>The strength versus suction envelope is nonlinear. At low matric suction, the slope of this envelope approaches the internal friction angle of the saturated soil, and the slope decreases significantly at high suction.</td>
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<td>1988</td>
<td>Gilii, J. A. and Alonso, E. E.</td>
<td>Discussion of a discontinuous constitutive model for partly saturated soils at low degrees of saturation. The proposed model incorporated liquid and gaseous phase into a discontinuous system.</td>
<td>The proposed model was capable of describing surface tension at gas-liquid interfaces, the stiffening effect of the menisci at the contact points between particles, the phase changes of water, and redistribution of water content. Transient flow problems as well as stress strain problems of partly saturated soils can be analyzed with the proposed model.</td>
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<td>1988</td>
<td>Karube, D.</td>
<td>Several equations for representing the characteristics of unsaturated soils were proposed. Compacted unsaturated kaolin clay was subjected to triaxial compression tests and anisotropic consolidation tests in order to evaluate the accuracy of the proposed equation. The results of the tests were analyzed in terms of ( (\sigma-u) ) and ( (u_0-u_w) ).</td>
<td>If the suction is kept constant, the effective stress can be defined by ( (\sigma-u) ), and, using some parameters dependent on the suction, the strains are expressed similarly to the saturated case. The suction influenced the friction angle as well as the cohesion. Plastic volumetric strain is a function of a change in ( (\sigma-u) ) with a coefficient which includes the suction term; on the other hand, elastic volumetric strain is a function of a change in ( (u_0-u_w) ) and a change in ( (u_0-u_w) ), which act separately.</td>
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<td>1988</td>
<td>McKeen, R. G.</td>
<td>Soil suction was defined as the sum of matric suction and osmotic suction, and the difference between negative pore pressure and soil suction was explained. The relationship between suction and moisture content was examined for a highly plastic clay (CH) and a clay of low plasticity (CL), and this relationship was supported by data from three different sites. Field suction profiles were shown for the active zones at four different sites, where soil suction is not in equilibrium.</td>
<td>The author discussed the characteristics of various methods for measuring total suction, matric suction, and osmotic suction, and described the thermocouple psychrometer and the filter paper method as the most practical and effective. The filter paper method gives reliable and repeatable data, although it is straightforward and fairly simple. The author showed linear relationships between the logarithm of total suction and water content. An upper limit and lower limit of suction were found to be ( 1,000 \text{ kg/cm}^2 ) and ( 0.1 \text{ kg/cm}^2 ) respectively. Soils cannot be drier or wetter than the water contents corresponding to these limits. Seasonal variations in total suction occur within an active zone near the ground surface. The author emphasized the importance of the suction measurements for predicting heaving and related behavior.</td>
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<td>1988</td>
<td>Peters, J. F., and Leavell, D. A.</td>
<td>Direct tensile tests and UU triaxial compression tests were conducted on Vicksburg silty clay compacted using kneading compaction. The relationship between suction and water content was measured using psychrometer tests. The direct tensile test apparatus consists of two gripping jaws, a rigid base, a slideable, and equipment for measuring force and displacement.</td>
<td>Soil suction decreases with increasing water content for compacted soil. Tensile strength gradually decreases with increasing water content until the optimum water content is reached, then sharply decreases as water content continues to decrease. The authors speculated that tensile strength is derived from the suction potential of the soil. They developed a general failure theory for partly saturated soils by studying the mechanism of the tensile strength due to suction. The proposed theory accounts for the effects of soil suction, water content, and void ratio.</td>
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<td>1988</td>
<td>Peterson, R. W.</td>
<td>Specimens of Vicksburg buckshot clay (CH) were remolded by kneading compaction. Constant water content tests (CW) were conducted on partly saturated specimens. CU tests were conducted on saturated specimens to provide a reference for evaluating the shear strength of unsaturated soil. One-dimensional consolidation tests were conducted to obtain the equivalent consolidation pressure as used in Hvorslev's theory of shear strength. A thermocouple psychrometer was installed in the top plate of a fixed ring consolidometer in order to measure suction during the consolidation tests. A thermocouple psychrometer was inserted into triaxial specimens through the base plate, and a cylindrical internal chamber was used inside the triaxial cell to measure the volume changes of unsaturated specimens.</td>
<td>The author proposed a modified Mohr-Coulomb strength relationship to describe the shear strength of unsaturated soils. In this relationship, the influence of suction is expressed as an apparent cohesion due to suction. This value becomes zero when a soil is fully saturated. The author mentioned the following limitations of the proposed relationship: (1) The strengths of saturated and partly saturated specimens should be compared at similar void ratios and stress states; otherwise, a procedure to normalize density differences must be used in order to evaluate the apparent cohesion due to suction, and (2) The proposed model should not be used for partly saturated specimens with high degrees of saturation, where the air phase is discontinuous. The apparent cohesion due to suction is approximately constant for the same water content, and its value decreases with increasing water content. On the other hand, the apparent cohesion is approximately independent of matric suction for soils in which the degree of saturation less than about 85 or 90 percent.</td>
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<td>1988</td>
<td>Schrefler, B. A. and Simoni, L.</td>
<td>Development of equations for saturated-unsaturated elastoplastic porous media, based on Biot's theory. Displacement and pressure fields are fully coupled in the proposed equation. The proposed governing equation is nonlinear because the material properties depend on the degree of saturation. Phase change in the pore fluid was taken into account. Several simplifications were made in the formulation: (1) it was assumed that pore air pressures are atmospheric everywhere, (2) capillary pressures were neglected, (3) Bishop's parameter $\chi$ was assumed to be equal to the degree of saturation.</td>
<td>Numerical difficulties were encountered in analyses performed using the equations: (1) the pressure field oscillated in space and time in an unrealistic manner, (2) technique to simulate movements of free surface. The authors discussed the difficulty in selecting an appropriate validation test for the behavior of partly saturated soils. They noted that incorporation of nonlinearities with respect to the Bishop's parameter $\chi$, modulus of soils, and porosity might improve the results.</td>
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<td>1989</td>
<td>Elfino, M. K., and Davidson, J. L.</td>
<td>Four subgrade soils were tested to study the effects of moisture content on resilient modulus. The soil-water retention characteristics of the soils were tested using a Tempe pressure cell. The resilient moduli were measured in triaxial tests using an MTS machine.</td>
<td>As the elevation above the water table increases: (1) the water content becomes smaller, (2) the pore water pressure becomes more negative, and (3) the resilient modulus becomes greater.</td>
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<td>1989</td>
<td>Escario, V. and Juca, J. F. T.</td>
<td>Direct shear and penetration tests were performed on statically compacted specimens of three soils. Soil suction was controlled during the tests. Moisture contents of the specimens were allowed to reach equilibrium with the applied suction prior to the tests.</td>
<td>The moisture contents of the soils reached equilibrium in five or six days in the pressure membrane apparatus under an applied soil suction. The measured modulus values were found to increase with increasing soil suction. Plots of the shear strength variation with soil suction, obtained from the direct shear tests, started with a slope equal to the effective stress angle of friction and gradually become flatter. The measured variation of shear strength with soil suction can be approximated by ellipses. For clays, both the apparent friction angle and the apparent cohesion increased with increasing soil suction. The apparent friction angle was essentially constant for clayey sand, although the apparent cohesion increased with suction.</td>
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<td>1989</td>
<td>Fredlund, D. G., and Wong, D. K. H.</td>
<td>Calibration tests were performed on thermal conductivity sensors used to measure soil suction. The device consisted of a pressure plate extractor, a temperature control box, and a data readout device.</td>
<td>The authors concluded that calibration of thermal conductivity sensors is important. The calibration curves for the sensors were found to be in good agreement with the manufacturer's calibration data for matric suctions ranging from 0 to 175 kPa, but the sensors showed nonlinearities for higher values of suction.</td>
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<td>1989</td>
<td>Krahn, J., Fredlund, D. G. and Klassen, M. J.</td>
<td>Investigation of the failure of a railroad embankment which occurred several years after construction. Triaxial tests were performed on both saturated and partly saturated specimens. Multistage loading tests were performed for the partly saturated specimens. In-situ suctions were measured with a tensiometer, and in-situ moisture contents were measured.</td>
<td>The variation of strength with suction was nonlinear. The authors observed that the strength of the soil was controlled by soil suction, not by water content. Water content is not a direct indicator of suction. Dissipation of negative water pressure played an important role in the slope failure: the strength decreased as the negative pore water pressure dissipated.</td>
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<td>1989</td>
<td>Sattler, P., and Fredlund, D. G.</td>
<td>Discussion of the mechanism, construction, and calibration of thermal conductivity sensors used for laboratory measurement of matric suction. Tests were conducted on undisturbed samples from several different sites and one set of compacted samples.</td>
<td>The sensor tip can be installed either dry or wet. The authors suggested using the dry method because less time is required for equilibration. A dry sensor installed in an undisturbed specimen required 3 to 4 days to reach equilibrium. The dry sensor showed greater values of suction, for example 105 kPa by the dry method, and 65 kPa by the wet method. The authors speculated that the value measured by the dry method was more accurate. Values of matric suction measured in laboratory tests were adjusted with respect to the overburden pressure by assuming that the pore pressure parameter, B, and the at-rest earth pressure coefficient were both equal to unity.</td>
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<td>1989</td>
<td>Sibley, J. W., and Williams, D. J.</td>
<td>A displacement method using Toluene was proposed to measure volumes of partly saturated soil specimens.</td>
<td>The toluene displacement method has a number of advantages over other methods. For instance, it can be used to measure irregular soil volumes without coating a soil, and can be used to measure the volume of a sample repeatedly.</td>
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<td>1989</td>
<td>Wheeler, S. J. and Sham, W. K.</td>
<td>Study of gas pressures in partly saturated offshore soils containing large discrete bubbles of gas. Both theoretical and experimental studies were conducted. Oedometer tests were conducted on specimens of clayey silt reconstituted from slurry. The reconstituted samples contained a controlled distribution of methane bubbles. Pore water pressures were measured at the bottom of the specimens and the pore pressures at other locations were calculated by assuming a parabolic distribution. Pore air pressures were calculated using Boyle's law and Henry's law.</td>
<td>The authors considered that: (1) there are upper and lower limits for pore air pressures, (2) the value of pore air pressure must always be greater than that of pore water pressure, (3) the difference between the values of pore air pressure and pore water pressure is limited by surface tension, and (4) the difference between the values of pore air pressure and total stress are limited by expansion and contraction of air bubbles. When the pore air pressures were between the proposed limits, they were independent of changes in total stress.</td>
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<td>1990</td>
<td>Alonso, E. E. and Josa, A.</td>
<td>A constitutive model for describing the stress-strain behavior of partly saturated soils was developed. The proposed model was based on a hardening plasticity model. The isotropic stress state and the triaxial stress state were analyzed and compared with the results of suction controlled test conducted on specimens of a compacted kaolin and a sandy clay.</td>
<td>The proposed model is based on a conventional critical state model, and is identical to that model for saturated conditions. The proposed model requires nine parameters, five more than the original Cam Clay model. The model uses two yield surfaces: a loading-collapse surface and a suction-increase surface. The proposed model showed good agreement with the test results. The model was developed for partly saturated soils that are not highly expansive.</td>
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<td>1990</td>
<td>Charlie, W. A., Ross, C. A., and Pierce, S. J.</td>
<td>The split-Hopkinson pressure bar (SHPB) was used to evaluate the influence of saturation on stress wave velocity, stress transmission, and attenuation. Tests were conducted on compacted specimens of silica sand.</td>
<td>Wave velocity, transmitted stress, and the value of quasi static modulus was found to increase with increasing degree of saturation, S, up to values of S equal to 30 to 40 %, and then decreased as S continued to increase. The authors did not suggest the cause of this behavior. They recommended further tests using specimens with carefully controlled water contents and dry densities.</td>
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<td>1990</td>
<td>Sibley, J. W., Smyth, G. K., and Williams, D. J.</td>
<td>Filter papers used for soil suction measurement were calibrated. The variability of the relationship between soil suction and moisture content was studied.</td>
<td>If filter papers from several boxes are used in the measurement of soil suction, those boxes should be from the same batch, and should be purchased from the same outlet at the same time.</td>
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<td>1990</td>
<td>Sibley, J. W., and Williams, D. J.</td>
<td>Cellulose seamless tubing and Millipore MF filtration membrane were used to measure soil suction. Their characteristics were compared with those of conventional filter paper.</td>
<td>Millipore filtration membrane showed greater sensitivity than either filter paper or cellulose tube over the middle suction range from 2.5 to 4.0 pF.</td>
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<tr>
<td>1990</td>
<td>Toll, D. G.</td>
<td>Triaxial tests were conducted on Kiunyu gravel, a lateritic gravel of Kenya, which was compacted using static and impact compaction. Six specimens were tested in a saturated condition, and twenty-three were partly saturated. Pore water pressures were measured through a ceramic disc with a high air entry value (300 kPa), and using a pore pressure probe at mid-height of the specimen. Specimens were sheared at constant water content, and pore air pressure was kept constant.</td>
<td>The author extended Fredlund's (1985) shear strength equation for unsaturated soils. The proposed relationship is expressed in terms of stress invariants, and is based on the coupling of volumetric and shear behavior. The influence of total stress and suction on shear strength depends strongly on degree of saturation. There is no unique set of soil parameters which define the critical state for partly saturated soils, because the initial soil fabric is not destroyed by shearing to large strains. The author speculated that the degree of saturation could be a useful indicator of fabric in compacted soils.</td>
</tr>
<tr>
<td>1990</td>
<td>Pande, G. N. and Pietruszczak, S.</td>
<td>The authors studied the relation between Skempton's (1954) pore pressure parameters and constitutive laws based on the theory of plasticity.</td>
<td>The authors showed that the pore pressure parameter A can be defined for any elasto-plastic constitutive law. The pore pressure parameter A at failure cannot be defined uniquely, if perfect plasticity is assumed.</td>
</tr>
<tr>
<td>1991</td>
<td>Fredlund, D. G.</td>
<td>Examination of two concepts concerning the lower limit of negative pore pressure: (1) the vapor pressure of water is the lowest pore water pressure, and the pore water pressure approaches one atmosphere negative, and (2) a total suction computed from the relative humidity of an air dried sample is possibly minus 10,000 atmospheres. The author measured suction in ceramic cylinders with a variety of pore sizes using a thermocoupler.</td>
<td>Total suction is the sum of matric suction and osmotic suction. The latter can be neglected for sandy soils. The author used Lord Kelvin's equation (based on thermodynamics) to explain the limit of negative pore water pressure; this equation indicates that pore water pressure can potentially decrease to extreme negative values. Testing on a cylindrical ceramic stone did not show cavitation of the water, even under highly negative pore water pressure. The author speculated that this fact indicates that there is another component of suction, or that there is change in the behavior of pore water in very small pores. The total suction of soil decreases to approximately minus 10,000 atmosphere as the water content decreases. All of the components of suction change as the water content or the total stress changes.</td>
</tr>
<tr>
<td>Year</td>
<td>Author</td>
<td>Nature of Study</td>
<td>Principal Findings and Conclusions</td>
</tr>
<tr>
<td>------</td>
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</tr>
<tr>
<td>1991</td>
<td>Tadepalli, R. and Fredlund, D. G.</td>
<td>The phenomenon of collapse in compacted soils was discussed, in the context of the theory for partly saturated soils developed by Fredlund and Morgenstern (1977). A partial differential equation governing the one-dimensional collapse behavior was formulated; in addition, a finite difference program was coded based on that equation. Collapse tests with matric suction measurement were conducted on compacted specimens of Indian Head silt, using a modified oedometer. Specimens were statically compacted into the oedometer ring, and tensiometers were installed in holes drilled into the specimens.</td>
<td>The governing equation of one-dimensional collapse behavior was formulated based on several assumptions; for instance, (1) pore air pressures remain constant during collapse, (2) modulus and permeability are functions of matric suction, (3) phase change can be ignored. The authors concluded that: (1) the collapse behavior depends on the matric suction during inundation, (2) there is a unique relationship between matric suction and total volume change of collapsible soils, (3) the coefficient of consolidation increases during inundation, (4) the coefficient of volume change with respect to matric suction either remains constant or decreases during inundation.</td>
</tr>
<tr>
<td>1991</td>
<td>Yoshida, Y. Kuwano, J. and Kuwano, R.</td>
<td>Undrained triaxial tests were performed on specimens of six different soils which were classified as SM or CH. All of the samples were taken from sites where heavy rainfalls had caused slope failures. Specimens were compacted to the same densities as measured in situ at water contents of 10 to 15 % for each soil. The compacted specimens were placed in a triaxial cell, and permeated to achieve the desired degree of saturation.</td>
<td>The measured value of cohesion intercept decreased with increasing degree of saturation, and were essentially zero for all of the samples at full saturation. As degrees of saturation increased, friction angles decreased at a nearly constant rate.</td>
</tr>
<tr>
<td>1991</td>
<td>Wheeler, S. J.</td>
<td>The framework proposed by Toll (1991) was modified, using an empirical curve for the critical state relationship.</td>
<td>It was found that a polynomial function of third order approximated the variation of deviator stress with soil suction quite accurately. The author suggested that the model can be used for two cases, (1) constant water content and air phase drained, and (2) both water and air phases drained. Only the first of these conditions was studied by Toll (1990).</td>
</tr>
<tr>
<td>Year</td>
<td>Author</td>
<td>Nature of Study</td>
<td>Principal Findings and Conclusions</td>
</tr>
<tr>
<td>------</td>
<td>--------</td>
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<td>-----------------------------------</td>
</tr>
<tr>
<td>1992</td>
<td>Day, R. W.</td>
<td>Drained compression tests were used to study the effective cohesion of a compacted silty clay. The remoulded specimens were submerged in water, and then loaded with zero confining pressure. Some of the specimens were cured in moisture-resistant containers prior to being submerged.</td>
<td>Specimens compacted at water contents above optimum and cured may not slake apart in water. The effective cohesion can be measured by using a slow strain rate which allows negative pore pressures to dissipate. It was found that curing reduced swelling and increased the effective cohesion significantly. The effective stress envelope is linear in the range of high stresses. However, at lower stresses, the envelope is curved, with a significantly lower cohesion intercept, and higher friction angle.</td>
</tr>
<tr>
<td>1992</td>
<td>Dhowian, A. W.</td>
<td>A suction-potential model was developed to estimate the behavior of suction on the basis of the diffusion equation. Laboratory tests, including oedometer tests and permeability tests, were conducted on undisturbed specimens of expansive shale. Field measurements were also carried out using thermocouple psychrometer stacks, moisture excess tubes, surface heave plates, and deep heave plates.</td>
<td>The proposed soil suction model uses a linear relation between swelling and water content, and an inversely linear relation between swelling and soil suction. It was estimated that suction decreased exponentially with time, and the decay rate was large near a water source. The model showed good agreement with the suction behavior observed in the laboratory, it did not agree with the field data, however. The author speculated that this discrepancy may be attributed to the simplified representation of the ground conditions which was used in the calibration.</td>
</tr>
<tr>
<td>1992</td>
<td>Ho, D. Y. F., Fredlund, D. G. and Rahardjo, H.</td>
<td>The volume change behavior of partly saturated soils was discussed in terms of void ratio and water content. Three types of tests: oedometer tests, pressure-plate tests, and shrinkage tests, were performed on specimens of silt and glacial till.</td>
<td>Oedometer tests, pressure-plate tests, and shrinkage tests are required to estimate the volume change behavior of partly saturated soils subject to loading. The rebound of oedometer tests and free swell tests were required for unloading conditions. A complete understanding of the relationship between the volume change and water contents is required for application of critical state soil mechanics to partly saturated soils.</td>
</tr>
<tr>
<td>1992</td>
<td>Moriwaki, T., Sato, T and Nakanodo, H.</td>
<td>Two case histories involving consolidation of partly saturated soils were discussed: the test embankment for Tomei highway, and the embankment for the Tokaido bullet train. The authors discussed the consolidation characteristics of partly saturated soils, using previously published results.</td>
<td>The authors classified the consolidation mechanism of partly saturated soils into two categories: (1) consolidation due to cyclically acting external loads (ex. traffic loads), and (2) consolidation due to changes in suction (e.g. drying and wetting caused by rainfalls). Case histories show that cyclic loads can cause increased settlement in embankments.</td>
</tr>
</tbody>
</table>
(1965) classification is shown in Table 3-2. Jennings and Burland (1962) considered that air in soil voids takes the form of separated, occulded bubbles at degrees of saturation above 20% for sands, 40 to 50% for silts, and 85% for clays.

The study of partly saturated soils has led to the use of terminology not used in reference to saturated soils. Bishop (1959) suggested that effective stresses in partly saturated soils could be expressed in terms of total stress, pore air pressure and pore water pressure. Subsequently, most researchers have followed this suggestion, and they have commonly used the term 'soil suction' for the difference between pore air pressure and pore water pressure. Negative pore water pressure is thus different from soil suction. It is commonly considered that soil suction consists of 'matric suction' and 'osmotic suction'. The sum of the matric suction and the osmotic suction is often referred as total suction (Krahn and Fredlund, 1972; McKeen, 1988). Definitions of these terms are shown in Table 3-3.

Soil suction may be highly negative. As a result, a logarithmic scale is often used to express soil suction. The logarithmic unit pF is used for this purpose. The pF value is equivalent to the logarithm to base ten of the suction expressed in centimeters of water. The relationship between the pF unit and pressure expressed in psi is:

\[
[u \text{ expressed in psi}] = \frac{10^{u \text{ expressed in pF}}}{70.34} \tag{3-1a}
\]

or

\[
[u \text{ expressed in pF}] = \log_{10}(70.34 \times [u \text{ expressed in psi}]) \tag{3-1b}
\]

It can be seen that 3 pF corresponds to 1 atmosphere (14.7 psi).

PREVIOUS STUDIES OF PARTLY SATURATED SOILS
<table>
<thead>
<tr>
<th>Classification</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Extremely dry clays</td>
<td>i) effective stress equation can be approximated by $\sigma' = \sigma - u_a$</td>
</tr>
<tr>
<td>(2) Clays dry of optimum</td>
<td>i) $u_w$ rarely exceeds zero (atmospheric pressure) due to large suction</td>
</tr>
<tr>
<td></td>
<td>ii) only air flows from the voids when the clay is compressed</td>
</tr>
<tr>
<td></td>
<td>iii) the effective stress equation cannot be simplified by ignoring water pressure</td>
</tr>
<tr>
<td>(3) Clays in the region of the optimum water content</td>
<td>i) $u_w$ may be greater than zero</td>
</tr>
<tr>
<td></td>
<td>ii) both air and water are expelled, in roughly equal quantities, when the clay is compressed</td>
</tr>
<tr>
<td>(4) Clays wet of optimum</td>
<td>i) most of the air is in the form of separated bubbles</td>
</tr>
<tr>
<td></td>
<td>ii) the effective stress equation can be approximated by $\sigma' = \sigma - u_w$</td>
</tr>
<tr>
<td>(5) Very wet clays.</td>
<td>similar to saturated clays</td>
</tr>
</tbody>
</table>
Table 3-3. Definitions of Suctions - after Review Panel (1965)

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Suction</td>
<td>The negative gage pressure, relative to the external gas pressure on the soil water, to which a pool of pure water must be subjected in order to be in equilibrium with the soil water through a semipermeable membrane (a membrane permeable to water molecules only).</td>
</tr>
<tr>
<td>Osmotic Suction</td>
<td>The negative gage pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the water in the voids of the soil.</td>
</tr>
<tr>
<td>Matric Suction</td>
<td>The negative gage pressure, relative to the external gas pressure on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium with the soil water through a porous permeable membrane.</td>
</tr>
</tbody>
</table>
3.2 Pore Pressure Measurement Techniques

Hilf (1956) observed that measurement of negative pore pressures in partly saturated soils is quite difficult, and techniques for negative pore pressure measurement have not been improved significantly in the nearly 40 years since Hilf's investigation. This can be seen from the fact that Fredlund (1985) made the same statement less than 10 years ago. Recognizing the difficulties involved in negative pore pressure measurements, Hilf (1956) proposed an "axis translation" technique for evaluating pore pressures in partly saturated soils. The axis translation technique involves application of positive air pressures to increase water pressures to positive values, thereby avoiding the necessity of measuring negative pore pressures. However, Bock and Fredlund (1980) showed that the use of this technique may be erroneous for partly saturated soils containing occluded air bubbles.

A considerable number of techniques for suction measurement have been introduced in the literature. A number of these are summarized in Table 3-4. As shown in the table, each technique covers a limited range, and no single technique can cover the full range of interest.

Porous discs with high air entry values and psychrometers are often used in laboratory tests. Peterson (1988) measured total suction during triaxial tests with a psychrometer. The use of porous discs to measure suction during triaxial tests was described in detail by Bishop (1961). Since then, this method has been the most common technique for measuring negative pore pressures in laboratory tests on partly saturated soils. However, dissolved air diffuses fairly rapidly into the saturated porous disc when the pore water pressures decrease to -14.7 psi, and it diffuses slowly at even less negative pressures. Fredlund (1975) studied this problem and developed an air flushing system
<table>
<thead>
<tr>
<th>Technique</th>
<th>Measureable Suction</th>
<th>Range (pF)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suction plate</td>
<td>Matric suction</td>
<td>0 to 3.0</td>
<td>Croney et. al. (1952)</td>
</tr>
<tr>
<td>Continuous flow</td>
<td>Matric suction</td>
<td>0 to 3.0</td>
<td>Croney et. al. (1952)</td>
</tr>
<tr>
<td>Rapid method</td>
<td>Matric suction</td>
<td>0 to 3.0</td>
<td>Croney et. al. (1952)</td>
</tr>
<tr>
<td>Tensiometer</td>
<td>Matric suction</td>
<td>0 to 3.3</td>
<td>Black et. al. (1958)</td>
</tr>
<tr>
<td>Pressure plate</td>
<td>Matric suction</td>
<td>0 to 3.0</td>
<td>Croney et. al. (1965)</td>
</tr>
<tr>
<td>Pressure membrane</td>
<td>Matric suction</td>
<td>0 to 6.2</td>
<td>Croney et. al. (1958)</td>
</tr>
<tr>
<td>Centrifuge</td>
<td>Matric suction</td>
<td>3.7 to 4.1</td>
<td>Croney et. al. (1952)</td>
</tr>
<tr>
<td>Heat dissipation in a ceramic</td>
<td>Matric suction</td>
<td>0 to 4.2</td>
<td>Croney et. al. (1952)</td>
</tr>
<tr>
<td>Thermal conductivity sensor</td>
<td>Matric suction</td>
<td>0 to 3.3</td>
<td>Fredlund and Wong (1989)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sattler and Fredlund (1989)</td>
</tr>
<tr>
<td>Mechanical tests using a porous disc with a high air entry value</td>
<td>Matric suction</td>
<td>0 to 3.0</td>
<td>Hilf (1956)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bishop (1961)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bishop and Donald (1961)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Krahn and Fredlund (1972)</td>
</tr>
<tr>
<td>Osmotic cell</td>
<td>Total suction</td>
<td>2 to 4.1</td>
<td>MacKeen (1988)</td>
</tr>
<tr>
<td>Vacuum desiccator</td>
<td>Total suction</td>
<td>5 to 7</td>
<td>Croney et. al. (1952)</td>
</tr>
<tr>
<td>Sorption balance</td>
<td>Total suction</td>
<td>5 to 7</td>
<td>Coleman and Farrar (1956)</td>
</tr>
<tr>
<td>Thermocouple psychrometer</td>
<td>Total suction</td>
<td>2.5 to 4.8</td>
<td>Krahn and Fredlund (1972)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>MacKeen (1988)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Peterson (1988)</td>
</tr>
<tr>
<td>Filter paper</td>
<td>Total suction</td>
<td>0.1 to 7</td>
<td>MacKeen (1988)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sibley et. al. (1990)</td>
</tr>
<tr>
<td>Electric conductivity sensor</td>
<td>Osmotic suction</td>
<td>No description</td>
<td>Krahn and Fredlund (1972)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Edil and Motan (1984)</td>
</tr>
<tr>
<td>Osmotic tensiometer</td>
<td>Pore water pressure</td>
<td>0 to 4.2</td>
<td>Bocking and Fredlund (1979)</td>
</tr>
</tbody>
</table>
which may be used for measuring the volume of the diffused air to correct the results (Fredlund and Morgenstern, 1976; Ho and Fredlund, 1982).

### 3.3 Pore Pressure Behavior

Two aspects of the pore pressure behavior of partly saturated soils are important: (1) the individual values of pore air and pore water pressure, and (2) the relationship between them. In partly saturated soils with low degrees of saturation, pore pressures can be highly negative, and large (negative) values of soil suction exist. Fredlund (1991), using principles of thermodynamics, calculated that the total suction may be as great as 10,000 atmosphere. In partly saturated soils with high degrees of saturation, high positive pore pressures may develop in response to changes in total stress.

The factors affecting soil suction have been studied by many investigators (Krahn and Fredlund, 1972; Edil et. al., 1981; Peters and Leavell, 1988; and Mckeen, 1988). Fig. 3-1 shows a typical relationship between degree of saturation and soil suction. VanGenuchten's (1980) model is widely used to relate suction to degree of saturation, although hysteretic behavior is neglected in the model. VanGenuchten's model, and analyses using the model, are described in Appendix E.

The air pressures inside air bubbles are always greater than the pore water pressures outside the bubbles, because of surface tension. The form of the air bubbles varies, depending on whether the air phase is occluded or continuous. In partly saturated soils wet of optimum, the air bubbles are occluded. Dry of optimum, the air phase tends to be continuous. The pressures in occluded air bubbles has been studied by several researchers (Hilf, 1956; Barden, 1965; Schuurman, 1966; Fredlund, 1976). It was found that the air pressures in occluded bubbles are not much higher than the pore water pressures in the surrounding water. As a result, the value of soil suction for this condition can be assumed
Fig. 3-1. Typical Moisture-Capillary Relationship
to be zero with little inaccuracy, and pore air pressures change by the same amount as pore water pressures in response to changes in total stress. For a soil dry of optimum, the radii of the air-water interfaces decrease as the water phase retreats into smaller interstices, and the air bubbles become continuous as the degree of saturation decreases. In this condition, the pore water pressure can be highly negative, and highly negative values of soil suction can exist in the soil.

Fig. 3-2, from Fredlund and Rahardjo (1993), illustrates the manner in which pore pressures in partly saturated soils change in response to changes in total stress. A dry soil under zero total stress has continuous pore air. The pore water pressures are highly negative, and high soil suction exists in the dry soil. As the total stress increases, the degree of saturation increases and the radii of the air bubbles become larger. Both the air and the water pressures increase with increasing total stress. However, the change in pore water pressure is greater than the change in pore air pressure. As a result, the soil suction decreases with increasing total stress. When the degree of saturation becomes high enough, the pore air bubbles become occluded (Zone 2 in Fig. 3-2). In this condition, as noted previously, the pore water pressures are essentially equal to the pore air pressures, and soil suction is essentially equal to zero.

For saturated soils, Skempton (1954) proposed pore pressure parameters which represent the ratios of pore pressure increments due to changes in total stress, to the magnitude of the changes in total stress. It is convenient to extend the concept of pore pressure parameters to partly saturated soils. Pore pressure parameters for partly saturated soils have been studied by several researchers (Bishop, 1960; Aichison, 1961; Jennings, 1961; Fredlund, 1976; Fredlund and Rahardjo, 1993). As shown in Fig. 3-3, the magnitude of the pore pressure parameter B for the water phase is always greater than that for the air phase at the same pressure.
Fig. 3-2. Pore-Air and Pore-Water Pressure Responses to a Change in Total Stress During Undrained Compression

- from Fredlund and Rahardjo (1993)
Fig. 3-3. Tangent and Secant Pore Pressure Parameters - from Fredlund and Rahardjo (1993)
The pore pressure parameter B can be defined as a tangent value or as a secant value, as shown in Fig. 3-3. Both the tangent value of B and the secant value of B increase with increasing total stress. The tangent value of B becomes unity when the soil becomes saturated. The secant value of B is always less than unity.

The previous discussions is concerned with undrained conditions, where the quantities of air and water (by weight) are constant. However, partially drained conditions are often encountered in practice. In partially drained conditions, the magnitude of the air permeability influences the behavior of the soil. For soils dry of optimum, the air phase is continuous, the air permeability is high, and changes in pore air pressures are small (Barden, 1965; Yoshimi and Osterberg, 1965; Leonard et. al., 1968). For soils in which the air phase is in the form of occluded air bubbles, the air permeability is very low (Leonard et. al., 1968), and pore air pressures can be quite high.

3.4 Strength and Constitutive Relationship for Partly Saturated Soils

In early studies, the strengths of partly saturated soils were related to effective stress in the same way as for saturated soils. Bishop (1959) suggested the following equations to relate the strength of partly saturated soils to total stress, pore air pressure, and pore water pressure.

$$\tau_f = \sigma' \cdot \tan \phi'$$  \hspace{1cm} (3.2.a)

$$\sigma' = (\sigma - u_a) + \chi \cdot (u_a - u_w)$$  \hspace{1cm} (3.2.b)

where \( \tau_f = \) shear strength,

\( \sigma' = \) effective stress,

\( \phi' = \) effective stress friction angle

\( \sigma = \) total stress,

\( u_a = \) pore air pressure,
\( u_w \) = pore water pressure, and

\( \chi = \text{coefficient (1.0 for saturated soils and 0 for perfectly dry soils).} \)

Other equations proposed by Aichison (1960), Jennings (1961) are essentially the same as Bishop's (1959). The equation proposed by Bishop (1959) was evaluated by himself and other researchers (Bishop et. al., 1960; Bishop and Donald, 1961; Bishop and Henkel, 1962; Jennings and Burland, 1962; Bishop and Blight, 1963; Blight, 1967). It was found that:

1. The values of \( \chi \) does not vary linearly with degree of saturation (Bishop and Blight, 1963). Gulhati and Satija (1981) found that there is no unique relation between \( \chi \) and \( S \).
2. The value of \( \chi \) may be greater than unity for high values of degree of saturation (Blight, 1967; Gulhati and Satija, 1981):
3. Values of strength estimated using the equation are satisfactory (Burland, 1965):
4. The volume change behavior of partly saturated soils cannot be explained using effective stress expressed by Eq. (3.2.b) (Jennings and Burland, 1962; Bishop and Blight, 1963; Blight, 1965).

Fig. 3-4 illustrates changes in effective stress defined by Eq. (3.2.b), and changes in the terms in the equation, with increasing total stress.

Burland (1965) suggested that two stress variables are required to describe the volume change behavior of partly saturated soils. An equation of this type was proposed by Matyas and Radharishna (1968). This approach has been used subsequently in a number of research studies (Fredlund and Morgenstern, 1977; Fredlund et al., 1978; Gulhati and Satija, 1981; etc.).

Using two stress variables, Fredlund et al. (1978) suggested the following shear strength equation for partly saturated soils:
Fig. 3-4. Changes in Effective Stress and in Other Components of Bishop's (1959) Equation with Increasing Total Stress
\[ \tau_f = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi_b \]  
\[ \text{where} \]

\[ \phi_b = \text{friction angle with respect to changes in soil suction } (u_a - u_w), \text{ when } (\sigma - u_a) \text{ is held constant.} \]

Based on this equation, the strength of partly saturated soils can be represented in three dimensions, with a series of conventional Mohr-Coulomb diagrams, each corresponding to a different value of matric suction. Comparing Eq. (3.11) with Eq. (3.10), the following relation is obtained:

\[ \chi = \frac{\tan \phi_b}{\tan \phi'} \]  
\[ \text{(3.4)} \]

It has been found that the strength envelope with respect to soil suction is nonlinear. In other words, the value of \( \phi_b \) is not constant (Escario and Saez, 1986; Gan et al., 1988; Gan and Fredlund, 1988; Krahn et al., 1989). At low values of matric suction, the value of \( \phi_b \) is close to the value of \( \phi' \) (Gan and Fredlund, 1988). These findings are in agreement with the previous studies of the value of Bishop's (1959) coefficient \( \chi \).

Soil strength can be related to suction. Edil et al. (1981) showed that unconfined compressive strength increased with increasing soil suction. Peters and Leavell (1988) showed that tensile strength increased with increasing suction.

A number of studies have been performed to investigate volume change behavior and collapse phenomena in partly saturated soils (Jennings and Burland, 1962; Bishop and Blight, 1963; Yoshimi and Osterberg, 1963; Barden, 1965; Blight, 1965; Matyas and Radharishna, 1968; Fredlund and Morgenstern, 1976; Josa et al., 1987; Tadepalli and Fredlund, 1991; Ho et al., 1992). These studies indicate that it is necessary to consider the hysteresis in the volume change behavior of partly saturated soils in order to represent their volume change and collapse characteristics accurately (Fredlund and Morgenstern,
However, the effects of hysteresis can be neglected in some cases for practical purposes, and volume changes in partly saturated soils can be approximated to be linearly proportional to both the logarithm of soil suction and the logarithm of mean stress (Josa, et. al., 1987; Ho et al., 1992). This relationship, shown in Fig. 3-5, is similar to an e-log p diagram, but with two stress variables.

Several researchers have suggested elasto-plastic constitutive models for partly saturated soils (Lloret and Alonso, 1980; Gili and Alonso, 1988; Karube, 1988; Alonso et al., 1990). These models are more complicated than those for saturated soils (Alonso et al., 1990; Ho et al., 1992).

Numerical analyses of the behavior of partly saturated soils have been performed using elasto-plastic constitutive models (Lloret and Alonso, 1980; Schrefler and Simoni, 1988). Because of the complexities of constitutive model for partly saturated soils, these analyses have often involved sweeping and unrealistic assumptions. For example, Schrefler and Simoni (1988) assumed that (1) the pore air pressures were equal to one atmosphere, (2) the capillary pressures were negligible, (3) the values of Bishop's parameter $\chi$ was equal to the degree of saturation. It is clear that there are still a number of difficulties in numerical analyses for partly saturated soils using effective stress. Their use in practice will require continuing research.

3.5 Earth Pressure Problem

Most of the studies of partly saturated soil behavior have focused on volume change behavior or strength characteristics. Few investigators have been concerned with earth pressures exerted by partly saturated soils. Fredlund (1979) suggested that a partly saturated soil could be treated as a soil with apparent cohesion equal to the shear strength caused by soil suction. Peterson (1988) made a similar suggestion.
(a) Void Ratio Relationship

(b) Intersection Curves Between Void Ratio Surface and $\log (\sigma - u_a)$ or $\log (u_a - u_w)$ Plane

Fig. 3-5. Void Ratio and Water Content Relationship for an Unsaturated Soil
- from Ho et al. (1992)
3.6 Summary

Previous studies of the behavior of partly saturated soils provide a good understanding of the behavior of partly saturated soils, and also shows that considerable difficulties are involved in understanding and predicting the physical behavior of partly saturated soils. The information obtained from this review provides a good background for investigation of at-rest and compaction-induced pressures in partly saturated soils:

- Pore water pressures in partly saturated soils can be highly negative at water contents below optimum. They can be much below minus one atmosphere, or zero absolute pressure. These highly negative pore pressures play a significant role in the behavior of partly saturated soils at water contents below optimum.

- Pore water pressures are smaller than pore air pressures, because of surface tension. The concepts of Skempton's (1954) pore pressure parameters can be extended to express changes in pore pressure caused by changes in total stress for partly saturated soils.

- It has proven to be very difficult to measure negative pore pressures, and to relate them quantitatively to strength and deformation properties. This can be seen from the fact that many papers addressing new techniques for pore pressure measurement (or soil suction measurement) are still being published, and that all of the published techniques involve difficult procedures, and provide results of limited applicability. Soil suction measurements are the subject of continuing research at the present time, and are not used to any significant extent in engineering practice.

- The form of the air in soil voids, whether occluded or continuous, affects the behavior of partly saturated soils significantly, as Barden (1965) and other investigators reported. Soil suction can often be assumed to be zero for soils...
with water contents above optimum, which almost always contain occluded air bubbles.

- Bishop (1959) proposed a form of the effective stress equation for partly saturated soils. Many investigators have shown that the equation provides useful approximation for the purpose of characterizing the strength of partly saturated soils; however, it cannot be used to characterize collapse phenomena, which are important in the volume change behavior of partly saturated soils.
CHAPTER 4
INSTRUMENTED OEDOMETER

4.1 Introduction

This chapter describes the instrumented oedometer developed for this study, and the preliminary tests that were performed to validate its effectiveness for measuring at-rest pressures under single-cycle and multi-cycle loadings. The device and the data acquisition system are described, and the results of tests on silicon rubber and Monterey sand are discussed.

Bishop (1958), Schmidt (1967), Wright (1969), and Sehn (1990) have discussed the requirement for measurement of at-rest earth pressures. The requirements are:

(1) The lateral deformation of the specimen must be small enough for one-dimensional compression to be achieved for practical purposes. It is well recognized that even small lateral deformations can significantly influence the magnitude of the measured earth pressures:

(2) The effective stresses and pore pressures should be uniform throughout the specimen. Shear stresses along the horizontal and vertical boundaries create non-uniform stresses within in the specimen, and should therefore be of negligible magnitudes:

(3) The test specimen should be homogeneous.

Most of the tests performed to study at-rest earth pressures have used triaxial samples with strain-sensing belts, or instrumented oedometers. Although many of the published results from tests performed using triaxial devices appear to be reliable and accurate, $K_o$ triaxial tests require great care and involve quite complicated procedures. In
addition, it is difficult to conduct rapidly cycling $K_0$ compression tests with triaxial apparatus. Oedometer tests are easier to perform, and are more suitable for rapid cyclic $K_0$ tests.

A split-ring oedometer was designed and constructed for this study. Similar devices have been developed by Obrican (1969) and previous researches at Virginia Tech (Samad, 1988; Knight, 1988; Sehn 1990).

Sehn (1990) evaluated the effect of vibratory compaction on at-rest lateral pressures by applying large numbers of cyclic loads. He measured the lateral force acting on a split ring using S-type load cells. The performance of the device proved to be sensitive to setup because of torsional forces acting on the load cells. Sehn's (1990) experience helped to guide the design of the instrumented oedometer used in this study, and also the testing procedures. The important lessons learned from Sehn's (1990) work are:

1. A thin filler disk at the top of the specimen improved the accuracy of the results.
   The filler disk consisted of a narrow outer band of Plexiglas and center of formed silicon rubber;

2. A silicon rubber specimen, with Poisson's ratio equal to one-half, was useful for calibration of the device;

3. Lubricant at the top and the bottom of the specimen improved the accuracy of the results, and

4. The large number of load cycles required a computer-based test control and data acquisition system.

4.2 Instrumented Oedometer

The instrumented oedometer which was designed and constructed in this study is shown in Fig. 4-1. It consists of a base plate, an oedometer ring, and a cap. The
Fig. 4-1. Assembled Instrumented Oedometer
apparatus was made of 17-4 PH stainless steel, which is hard and stable. Fig. 4-1 shows the assembled apparatus. Fig. 4-2 shows plan views and cross sections. Drawings used for construction of device are shown in Appendix D.

The base module of the instrumented oedometer consists of a base plate, an end wall, two side walls, a disk insert, and a porous disk. Fig. 4-3 shows these pieces. High rigidity of the testing device is an essential requirement for \( K_0 \) testing. To provide rigidity, the end wall and the two side walls are fixed to the base plate to form a rigid box. The side walls also help to maintain good alignment of the oedometer ring during tests. The 5.5 in. diameter porous disk fits into the 6-inch diameter disk insert, and the disk insert fits into the base plate. For undraind tests without pore pressure measurement, the reverse side of the disk insert is used by inserting it upside down. All of the tests performed in this study were performed in this mode, without pore pressure measurements.

The oedometer ring module consists of two halves, a fixed half and a split half, as shown in Fig. 4-4. These two halves form the space for the specimen, which is 1.0 inch high and 6.0 inches in diameter. The fixed half of the oedometer is fixed to the base plate with four bolts and two dowel pins, as shown in Fig. 4-5. The split half is supported by two flat cage bearings (FF2515, manufactured by INA Bearing Company), which are mounted on the base plate, and two hardened steel ball bearings, 0.250 inches in diameter, on each side wall. Ideally, no lateral forces should be exerted on the side bearings during the tests.

The lateral load exerted on the split half when the soil specimen is loaded is measured using two button-type load cells on the end wall. Consequently, the lateral loads are measured as compressive forces. Three bolt screws attached to the split half through the end plate. One is used for in-place calibration of the load cells. The other
Fig. 4.2. Part I - Plan View and Cross-Sections of Instrumented Oedometer
Fig. 4-2. Part 2 - Plan View and Cross-Sections of Instrumented Oedometer
Fig. 4-3. Base Module of the Instrumented Oedometer
Fig. 4-4. Oedometer Ring Module of the Instrumented Oedometer
Fig. 4-5. The Fixed Half of Oedometer Ring Bolted to the Base Module of the Instrumented Oedometer
two, which bear against compression springs, are used to preload the load cells and assure a snug fit, even with zero pressure on the test specimen.

The cap module of the instrumented oedometer consists of the cap, an O-ring, and a retaining ring, as shown in Fig.4-6. A rubber membrane 0.012 inches thick is fixed to the bottom of the cap, using the O-ring and the retaining ring. Air pressure is applied through a port in the center of the cap. The pressure loading is applied to the top of the specimen through the membrane. Following the finding by Samad (1988), a thin filler disk was used between the specimen and the cap module in order to prevent deformation of the edges of the specimen and improve the accuracy of the results. The filler disk consists of a 0.5 in. wide plastic ring, 5.98 in. outer diameter, and 0.125 in. thick, with a solid circular silicon rubber sheet 0.125 in. thick filling the center of the ring.

Friction between the sides of the specimen and the walls of the oedometer ring needs to be considered. Hendron (1963) calculated the error due to the friction for his apparatus, and found error in vertical stress was 5.4 percent. Brooker and Ireland (1965) used silicon vacuum grease to reduce the friction, and obtained successful results. Consequently, vacuum grease was used in this study. In addition, silicon vacuum grease was used on the top and the bottom of the test specimens, based on the finding in previous studies at Virginia Tech. Schmidt (1967) and Wright (1969) speculated that lubricants may be squeezed out during long-term tests.

As the specimen is compressed vertically during a test, the height of the specimen in contact with the split half decreases. The amount of compression during the tests is small for sand specimens and well-compacted moist soils, and this effect would not significantly influence the results in those cases. On the other hand, a very loose specimen can undergo significant vertical compression. To accommodate this compression, a "settlement ring" with a height of 0.24 inches was added between the oedometer ring and the cap module.
Fig. 4-6. Cap Module of the Instrumented Oedometer and Thin Filler Disk
during tests on loose specimens. The height of the specimens could be maintained greater
than 1.0 inch through use of the settlement ring. As a result, the entire inner vertical face
of the split half stayed in contact with the side of the specimens during the tests.

The rapid cyclic loading applied during some of the tests required a computer-based
data acquisition and test control system. Those systems are shown in Figs. 4-7 and 4-8.
The systems use a high speed data acquisition card in a PC-compatible microcomputer.
They are described in detail in Appendix A. Both systems are controlled by means of a
computer program "ACQOED" developed for this research study. The program is
described in Appendix B.

The test specimens were formed by pouring dry sand or compacting moist soil in
place in the oedometer. Two to three hours were required to set up the apparatus and
prepare a specimen.

4.3 Lateral Deformation during the Tests

The compressibility of the load cells was measured by applying dead loads, and was
found to be 2.0\times10^{-6} \text{ inches/lb}. It was considered that the split half of the instrumented
oedometer might also be deformable. The deformation of the split half and the end wall
was measured using a dial gauge with an accuracy of 1/10,000 inch. The measured
deformation of the split half was found to be about 2.3\times10^{-6} \text{ inches/psi}, very nearly the
same as the compressibility of load cells. The deformation of the end wall was too small
to measure.

The average horizontal strain across the six-inch diameter specimen is:

\[ \varepsilon = \frac{\Delta D}{D} \]  

(4-1)
Fig. 4-7. Data Acquisition System
Compressed Air

Manual Pressure Regulator
  - Supply pressure

PAR-15 Digital Pressure Regulator
  - Specified pressure

Instrumented Oedometer and Pressure Transducer

Digital Interface
  - Make digital signals suitable to PAR-15

PC Compatible Microcomputer
  - Send signals for pressure control
  - Check supply pressure

Data Acquisition System

Air Pressure Line

Signal Transfer Circuit

Fig. 4-8. Test Control System
where

\[ \varepsilon = \text{horizontal strain}, \]
\[ \Delta D = \text{change in diameter, and} \]
\[ D = \text{diameter}. \]

With \( \Delta D = 2.3 \times 10^{-6} \) inches per psi and \( D = 6 \) inches, the strain is:

\[ \varepsilon = \frac{2.3 \times 10^{-6}}{6} = 0.4 \times 10^{-6} \text{ per psi} \tag{4-2} \]

Ofer (1982) suggested that for accurate measurement of \( K_0 \) for soils, the lateral strain should be less than \( 6.0 \times 10^{-5} \). For lateral pressures less than 150 psi, this requirement is satisfied by the device used in this study. Even in the high pressure tests where the maximum vertical pressure (70 psi) was applied, the maximum lateral pressure was less than half of the values corresponding to the strain limit suggested by Ofer. Therefore, it is clear that an acceptable at-rest condition was satisfied throughout the instrumented oedometer tests.

### 4.4 Preliminary Test Results

Preliminary tests were performed on silicon rubber specimens in order to examine the performance of the instrumented oedometer. The silicon rubber specimens were cast in place by pouring and aging gel-form silicon rubber. Silicon rubber is nearly incompressible, with a value of Poisson's ratio equal to one half, if it does not contain air bubbles. As a result, the tests on silicon rubber specimens should ideally result in values of \( K_0 \) very nearly equal to 1.00.

Fig. 4-9 shows the variation of lateral pressure with vertical pressure for tests performed without a settlement ring. The rubber specimen appeared to shrink very slightly as it cured, and a small gap was observed between the cured specimen and the
Fig. 4-9. Preliminary Test Results on Silicon Rubber Specimen, without Settlement Ring
oedometer ring. This is believed to be the reason that the initial slope in Fig. 4-9 is 0.94, rather than unity. The slope for pressures above 10 psi is 0.98, which indicates acceptable accuracy. The difference between the measured value \( K_0 = 0.98 \) and the ideal value \( K_0 = 1.0 \) may indicate that the silicon rubber contains some air bubbles and is not perfectly incompressible. However, regardless of the reason that the measured value of \( K_0 \) is not exactly equal to 1.00, it can be concluded that the instrumented oedometer without the settlement ring operates well enough to achieve useful results.

Two tests were performed on silicon rubber specimens to evaluate the performance of the instrumented oedometer with the settlement ring. The first was performed using the 1.00 in. high silicon rubber specimen used in the tests with the settlement ring. Test results are shown in Fig. 4-10. It can be seen that the results were almost identical to those described previously. In the second test, a silicon rubber specimen with a height of 1.24 inches was used. This specimen contained many air bubbles because insufficient vacuum was applied during curing. The initial gap between the specimen and the ring was negligibly small. As shown in the Fig. 4-11, the slope of the \( K_0 \) line is 0.96. The deviation in \( K_0 \) value from unity is believed to be due to the air bubbles in the silicon rubber specimen. Based on these results, it was concluded that the apparatus with the settlement ring works properly.

4.5 Results of Tests on Monterey Sand

To further evaluate the performance of the instrumented oedometer, a series of tests were performed on Monterey sand. Index properties and strength parameters of Monterey sand are summarized in Appendix G.

Test specimens of Monterey sand were formed by pouring sand into the instrumented oedometer from a stoppered bottle through a small-diameter tube.
Fig. 4-10. Preliminary Test Results on Silicon Rubber Specimen, with Settlement Ring, and 1.00 in. High Specimen
Fig. 4-11. Preliminary Test Results on Silicon Rubber Specimen, with Settlement Ring, and 1.24 in. High Specimen
Specimens of different densities were formed by adjusting the height of fall and the size of the tube.

The results of the tests are shown in Table 4-1 and Table 4-2. Eleven tests, M1 thorough M8 and M15 through M17, were performed without the settlement ring, and six tests, M9 through M14, were performed using the settlement ring. In test M15 thorough M17, one thousand cycles of loading and unloading with a peak vertical pressure of 30 psi were applied, and in the other tests a single cycle of loading and unloading was applied.

Sehn (1990) also performed $K_o$ oedometer tests on Monterey 0/30 sand. Wright (1969) performed $K_o$ triaxial tests on Monterey #20 sand. The results of these tests are shown in Table 4-3.

Figs. 4-12 to 4-14 show stress paths for the cyclic tests. As for the tests performed by Sehn (1990) (see Fig. 2-4), the values of $K_o$ at peak load decreased, and the stress paths become more hysteretic, as the number of cycles increased. The values of $K_o$ measured at 30 psi in the first loading are plotted against relative density in Fig. 4-15. These results indicate that the device worked properly and that the results are reproducible, because:

(1) The measured values of $K_o$ were found to vary no more than ±0.02 for a given relative density:

(2) The values of $K_o$ are in close agreement with values of $K_o$ calculated by Jaky's equation:

(3) The observed cyclic stress paths were in similar shape to those measured by Sehn (1990).

It can be seen that Wright (1969) measured slightly greater values of $K_o$ for a given value of relative density. Ofer (1982) reported that $K_o$ triaxial tests often result in values
Table 4-1. Instrumented oedometer test results for Monterey sand and the comparison to the Jaky's empirical correlation (1944)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry density (pcf)</th>
<th>Void ratio</th>
<th>Relative density (%)</th>
<th>Effective friction angle (deg)</th>
<th>$K_0$ calculated by Jaky's eqn.</th>
<th>$K_0$ obtained by experiment</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>92.4</td>
<td>0.790</td>
<td>13</td>
<td>35.3</td>
<td>0.422</td>
<td>0.379</td>
<td></td>
</tr>
<tr>
<td>M2</td>
<td>92.7</td>
<td>0.784</td>
<td>16</td>
<td>35.5</td>
<td>0.419</td>
<td>0.366</td>
<td></td>
</tr>
<tr>
<td>M3</td>
<td>93.0</td>
<td>0.778</td>
<td>18</td>
<td>35.7</td>
<td>0.416</td>
<td>0.374</td>
<td></td>
</tr>
<tr>
<td>M4</td>
<td>93.8</td>
<td>0.763</td>
<td>23</td>
<td>36.3</td>
<td>0.408</td>
<td>0.392</td>
<td></td>
</tr>
<tr>
<td>M5</td>
<td>98.1</td>
<td>0.687</td>
<td>49</td>
<td>39.2</td>
<td>0.368</td>
<td>0.376</td>
<td></td>
</tr>
<tr>
<td>M6</td>
<td>99.9</td>
<td>0.655</td>
<td>60</td>
<td>40.5</td>
<td>0.351</td>
<td>0.338</td>
<td></td>
</tr>
<tr>
<td>M7</td>
<td>101.9</td>
<td>0.623</td>
<td>71</td>
<td>42.0</td>
<td>0.331</td>
<td>0.345</td>
<td></td>
</tr>
<tr>
<td>M8</td>
<td>102.1</td>
<td>0.619</td>
<td>73</td>
<td>42.1</td>
<td>0.330</td>
<td>0.340</td>
<td></td>
</tr>
<tr>
<td>M9</td>
<td>92.0</td>
<td>0.797</td>
<td>11</td>
<td>35.1</td>
<td>0.425</td>
<td>0.368</td>
<td>specimen height = 1.25&quot;</td>
</tr>
<tr>
<td>M10</td>
<td>94.2</td>
<td>0.756</td>
<td>25</td>
<td>36.5</td>
<td>0.405</td>
<td>0.355</td>
<td>specimen height = 1.25&quot;</td>
</tr>
<tr>
<td>M11</td>
<td>97.1</td>
<td>0.704</td>
<td>43</td>
<td>38.5</td>
<td>0.377</td>
<td>0.339</td>
<td>specimen height = 1.170&quot;</td>
</tr>
<tr>
<td>M12</td>
<td>100.0</td>
<td>0.654</td>
<td>61</td>
<td>40.6</td>
<td>0.349</td>
<td>0.367</td>
<td>specimen height = 1.140&quot;</td>
</tr>
<tr>
<td>M13</td>
<td>101.2</td>
<td>0.634</td>
<td>67</td>
<td>41.5</td>
<td>0.337</td>
<td>0.319</td>
<td>specimen height = 1.235&quot;</td>
</tr>
<tr>
<td>M14</td>
<td>102.9</td>
<td>0.607</td>
<td>77</td>
<td>42.7</td>
<td>0.322</td>
<td>0.311</td>
<td>specimen height = 1.210&quot;</td>
</tr>
<tr>
<td>M15</td>
<td>91.2</td>
<td>0.813</td>
<td>6</td>
<td>34.6</td>
<td>0.432</td>
<td>0.410</td>
<td>1000 cycles of loading</td>
</tr>
<tr>
<td>M16</td>
<td>100.7</td>
<td>0.642</td>
<td>65</td>
<td>41.1</td>
<td>0.343</td>
<td>0.370</td>
<td>1000 cycles of loading</td>
</tr>
<tr>
<td>M17</td>
<td>103.6</td>
<td>0.596</td>
<td>81</td>
<td>43.2</td>
<td>0.315</td>
<td>0.317</td>
<td>1000 cycles of loading</td>
</tr>
</tbody>
</table>

- Relative densities were calculated by $e_{max} = 0.829$ and $e_{min} = 0.540$
- Effective friction angles were determined based on the correlation $0.56 = e \cdot \tan \phi'$.
- Tests from M19 through M14 were performed using a settlement ring.
Table 4-2. Values of $K_0$ Obtained by Instrumented Oedometer Tests for Monterey Sand

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial dry unit weight (pcf)</th>
<th>Initial relative density (%)</th>
<th>Maximum vertical Pressure (psi)</th>
<th>1st Cycle</th>
<th>2nd Cycle</th>
<th>100th Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>M1</td>
<td>92.4</td>
<td>13</td>
<td>30.6</td>
<td>0.379</td>
<td>0.487</td>
<td>0.538</td>
</tr>
<tr>
<td>M2</td>
<td>92.7</td>
<td>16</td>
<td>30.1</td>
<td>0.366</td>
<td>0.468</td>
<td>0.505</td>
</tr>
<tr>
<td>M3</td>
<td>93.0</td>
<td>18</td>
<td>30.1</td>
<td>0.374</td>
<td>Data are not available.</td>
<td></td>
</tr>
<tr>
<td>M4</td>
<td>93.8</td>
<td>23</td>
<td>30.4</td>
<td>0.392</td>
<td>0.515</td>
<td>0.593</td>
</tr>
<tr>
<td>M5</td>
<td>98.1</td>
<td>49</td>
<td>30.1</td>
<td>0.376</td>
<td>0.517</td>
<td>0.614</td>
</tr>
<tr>
<td>M6</td>
<td>99.9</td>
<td>60</td>
<td>30.1</td>
<td>0.338</td>
<td>0.462</td>
<td>0.545</td>
</tr>
<tr>
<td>M7</td>
<td>101.9</td>
<td>71</td>
<td>30.4</td>
<td>0.345</td>
<td>0.469</td>
<td>0.561</td>
</tr>
<tr>
<td>M8</td>
<td>102.1</td>
<td>73</td>
<td>30.1</td>
<td>0.340</td>
<td>0.491</td>
<td>0.608</td>
</tr>
<tr>
<td>M9</td>
<td>92.0</td>
<td>11</td>
<td>30.0</td>
<td>0.368</td>
<td>0.463</td>
<td>0.527</td>
</tr>
<tr>
<td>M10</td>
<td>94.2</td>
<td>25</td>
<td>30.0</td>
<td>0.355</td>
<td>0.454</td>
<td>0.523</td>
</tr>
<tr>
<td>M11</td>
<td>97.1</td>
<td>43</td>
<td>30.0</td>
<td>0.339</td>
<td>0.451</td>
<td>0.538</td>
</tr>
<tr>
<td>M12</td>
<td>100.0</td>
<td>61</td>
<td>30.0</td>
<td>0.367</td>
<td>0.512</td>
<td>0.628</td>
</tr>
<tr>
<td>M13</td>
<td>101.2</td>
<td>67</td>
<td>30.0</td>
<td>0.319</td>
<td>0.438</td>
<td>0.532</td>
</tr>
<tr>
<td>M14</td>
<td>102.9</td>
<td>77</td>
<td>30.0</td>
<td>0.311</td>
<td>0.432</td>
<td>0.540</td>
</tr>
<tr>
<td>M15</td>
<td>91.2</td>
<td>6</td>
<td>30.0</td>
<td>0.410</td>
<td>0.526</td>
<td>0.594</td>
</tr>
<tr>
<td>M16</td>
<td>100.7</td>
<td>65</td>
<td>30.0</td>
<td>0.370</td>
<td>0.512</td>
<td>0.617</td>
</tr>
<tr>
<td>M17</td>
<td>103.6</td>
<td>81</td>
<td>29.7</td>
<td>0.317</td>
<td>0.434</td>
<td>0.525</td>
</tr>
</tbody>
</table>

One cycle of loading-unloading was applied.
Table 4-3. $K_0$ test results for Monterey sand published by other researchers and the comparison to the Jaky's empirical correlation (1944)

<table>
<thead>
<tr>
<th>Reference and Type of tests</th>
<th>Dry density (pcf)</th>
<th>Void ratio</th>
<th>Relative density (%)</th>
<th>Effective friction angle (deg)</th>
<th>$K_0$ calculated by Jaky's eqn.</th>
<th>$K_0$ obtained by experiment</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sehn (1990) Instrumented oedometer</td>
<td>91.6</td>
<td>0.806</td>
<td>9</td>
<td>34.8</td>
<td>0.43</td>
<td>0.44</td>
<td></td>
</tr>
<tr>
<td></td>
<td>101.8</td>
<td>0.625</td>
<td>75</td>
<td>41.9</td>
<td>0.33</td>
<td>0.44</td>
<td></td>
</tr>
<tr>
<td></td>
<td>102.2</td>
<td>0.619</td>
<td>77</td>
<td>42.1</td>
<td>0.33</td>
<td>0.42</td>
<td></td>
</tr>
<tr>
<td></td>
<td>102.7</td>
<td>0.611</td>
<td>80</td>
<td>42.5</td>
<td>0.32</td>
<td>0.42</td>
<td></td>
</tr>
<tr>
<td></td>
<td>103.5</td>
<td>0.598</td>
<td>84</td>
<td>43.1</td>
<td>0.32</td>
<td>0.37</td>
<td></td>
</tr>
<tr>
<td>Wright (1966) K$_0$ triaxial</td>
<td>94.0</td>
<td>0.76</td>
<td>23</td>
<td>36.4</td>
<td>0.41</td>
<td>0.426</td>
<td>$\sigma_{con, ini}=7.11\text{psi}$</td>
</tr>
<tr>
<td></td>
<td>95.6</td>
<td>0.73</td>
<td>33</td>
<td>37.5</td>
<td>0.39</td>
<td>0.425</td>
<td>$\sigma_{con, ini}=1.42\text{psi}$</td>
</tr>
<tr>
<td></td>
<td>95.6</td>
<td>0.73</td>
<td>33</td>
<td>37.5</td>
<td>0.39</td>
<td>0.441</td>
<td>$\sigma_{con, ini}=7.11\text{psi}$</td>
</tr>
<tr>
<td></td>
<td>95.6</td>
<td>0.73</td>
<td>33</td>
<td>37.5</td>
<td>0.39</td>
<td>0.454</td>
<td>$\sigma_{con, ini}=14.2\text{psi}$</td>
</tr>
<tr>
<td></td>
<td>106.7</td>
<td>0.55</td>
<td>93</td>
<td>45.0</td>
<td>0.29</td>
<td>0.333</td>
<td>$\sigma_{con, ini}=28.4\text{psi}$</td>
</tr>
</tbody>
</table>

- Relative densities for the results by Sehn (1990) were calculated by $e_{\text{max}}=0.829$ and $e_{\text{min}}=0.540$.
- Effective friction angles for the results by Sehn (1990) were determined based on the correlation $0.56 = e \cdot \tan\phi'$.
- Effective friction angles for the results by Wright (1966) were determined by the triaxial test results by Marachi et. al. (1981).
Fig. 4-12. Instrumented Oedometer Test Result for Monterey Sand (Test M15)

Initial Condition of Specimen
- dry unit weight = 91.2 pcf
- void ratio = 0.813
- relative density = 6%
Fig. 4-13. Instrumented Oedometer Test Result for Monterey Sand (Test M16)

Initial Condition of Specimen
- dry unit weight = 100.7pcf
- void ratio = 0.642
- relative density = 65%
Fig. 4.14. Instrumented Oedometer Test Result for Monterey Sand (Test M17)

Initial Condition of Specimen
- dry unit weight = 103.6 pcf
- void ratio = 0.596
- relative density = 81%

No other textual content is visible in the provided image.
Fig. 4-15. Results of Instrumented Oedometer Tests on Monterey Sand
of $K_o$ that are larger than values measured in oedometer tests. It can be speculated that $K_o$ triaxial tests may overestimate values of $K_o$ due to small fluctuations in lateral strain around zero, and that $K_o$ oedometer tests are likely to underestimate values of $K_o$ due to small deformations of the oedometer ring, and due to side friction effects.

Six tests, test M9 through M14, were performed using the settlement ring. As shown in Fig. 4-16, the tests using the settlement ring resulted in slightly smaller values of $K_o$. Most of the results of tests with the settlement ring plotted close to the lower boundary of $\pm 0.02$ trend band obtained previously. There may higher side friction with the settlement ring, which results in smaller values of $K_o$.

4.6 Comparison of Measured and Calculated Values of Lateral Pressure for Monterey Sand

The effective stress $K_o$ model developed by Duncan and Seed (1986) and modified by Filz (1992) has been applied successfully to dry soils. The lateral pressure measured in tests on Monterey sand were compared to values calculated using this model, to examine the accuracy of the model.

The $K_o$ model calculations were based on these assumptions: (1) $K_o$ can be estimated using Jaky's (1944) correlation, (2) the values of $\phi'$ can be calculated by the empirical correlation, $0.56 = e \cdot \tan \phi'$, and (3) the model parameters $\alpha$ and $\beta$ can be determined using the empirical correlation suggested by Duncan and Seed (1986). Figs. 4-17 through 4-19 show the stress paths calculated using the effective stress $K_o$ model with these assumptions, together with test results. The agreement between the measured and calculated stress paths is quite reasonable, but not exact.

As shown in Fig. 2-3, the suggested values of the parameters $\alpha$ and $\beta$ are mean values of scattered data. A second set of calculations was made, using the values of the
Numbers next to symbols indicate the heights of the specimens used for the tests with a settlement ring. The heights of the specimens used for the tests without a settlement ring was 1.0 inch.

Fig. 4-16. Results of Instrumented Oedometer Tests on Monterey Sand (Evaluation of Settlement Ring Effects)
\[ e = 0.56 \tan \phi', \text{ corresponding to } \phi' = 34.6 \text{ deg.} \]
\[ K_0 = 1 - \sin \phi' = 0.432 \text{ (Jaky, 1944)} \]
\[ \alpha = 0.62 \text{ (Duncan and Seed, 1986)} \]
\[ \beta = 0.60 \text{ (Duncan and Seed, 1986)} \]

Fig. 4-17. Comparison Between \( K_0 \) Model and Experimental Result for Monterey Sand (Test M15)
\( e = 0.56 \tan \phi', \) corresponding to \( \phi' = 41.1 \text{ deg.} \)

\( K_0 = 1 - \sin \phi' = 0.343 \) (Jaky, 1944)

\( \alpha = 0.74 \) (Duncan and Seed, 1986)

\( \beta = 0.60 \) (Duncan and Seed, 1986)

Fig. 4-18. Comparison Between \( K_0 \) Model and Experimental Result for Monterey Sand (Test M16)
Fig. 4-19. Comparison Between $K_0$ Model and Experimental Result for Monterey Sand (Test M17)

\[
\theta = 0.56 \tan \phi', \text{ corresponding to } \phi' = 43.2 \text{ deg.}
\]

\[
K_0 = 1 - \sin \phi' = 0.315 \text{ (Jaky, 1944)}
\]

\[
\alpha = 0.75 \text{ (Duncan and Seed, 1986)}
\]

\[
\beta = 0.60 \text{ (Duncan and Seed, 1986)}
\]
parameter α selected to give the best fit to the test results at OCR = 3. The values of the other parameters (β and K₀) were not changed. Figs. 4-20 and 4-21 show the stress paths calculated for tests M15 and M17. It can be seen that the agreement is improved.

Fig. 4-22 shows the relationship between OCR and K₀ plotted on log-log and semi-log scales. A linear relationship between log(OCR) and log(K₀) is widely accepted; however, the relationship found in Fig. 4-22 (a) is slightly nonlinear for loose specimens of Monterey sand. This trend was also found in Wright's (1969) results, shown in Fig. 2-2. The results shown in Fig. 4-22 (b) indicate that a linear relationship between log(OCR) and K₀ might be more reasonable for loose specimens.
$\phi = 0.56 \tan \phi'$, corresponding to $\phi' = 34.6$ deg.

$K_0 = 1 - \sin \phi' = 0.432$ (Jakay, 1944)

$\alpha = 0.41$ (adjusted to the experimental data at OCR = 3.0)

$\beta = 0.60$ (Duncan and Seed, 1986)

Fig. 4-20. Comparison Between $K_0$ Model with an Adjusted $\alpha$ Parameter and Experimental Result for Monterey Sand (Test M15)
\[ e = 0.56 \tan \phi', \text{ corresponding to } \phi' = 43.2 \text{ deg.} \]

\[ K_0 = 1 - \sin \phi' = 0.315 \text{ (Jaky, 1944)} \]

\[ \alpha = 0.69 \text{ (adjusted to the experimental data at } OCR = 3.0) \]

\[ \beta = 0.60 \text{ (Duncan and Seed, 1986)} \]

Fig. 4-21. Comparison Between \( K_0 \) Model with an Adjusted \( \alpha \) Parameter and Experimental Result for Monterey Sand (Test M17)
Fig. 4-22. Relationship Between $K_0$ and OCR for Monterey #30 Sand
CHAPTER 5

INSTRUMENTED OEDOMETER TESTS ON YATESVILLE SILTY SAND

5.1 Introduction

The results of instrumented oedometer tests on partly saturated specimens of Yatesville silty sand are described in this chapter. The results are discussed with respect to total stress values of $K_0$, called $K_0^T$. A simple procedure for estimating $K_0^T$ is suggested, based on the results of the tests conducted during this study.

5.2 Test Procedure and Test Program

Instrumented oedometer tests were conducted on compacted specimens of moist Yatesville silty sand. The water contents of the samples were adjusted before compaction, and clods were broken up. The soil was placed in the oedometer, and was compacted using a Harvard miniature compactor. The weight of the specimen was determined by subtracting the weight of soil left after compaction from the weight of sample prepared in advance. The water content prior to testing was determined by taking an average of the water content measured before compaction and the water content of the excess material remaining after compaction. These values usually differed by about 0.2 percent to 0.4 percent. After testing, the specimen height, weight and water content were also measured.

Two values of maximum vertical pressure were used in the tests - 30 psi and 70 psi. For the tests with 30 psi maximum pressure, either 100 or 1000 cycles of loading were applied to the previously compacted specimens. For the tests with 70 psi maximum pressure, application of a large number of cycles of loading tended to rupture the rubber membrane used to apply pressure to the top of the specimen. Consequently, ten or fewer cycles of loadings were applied in most of the high-pressure tests.
The properties of the test specimens are shown in Table 5-1. It can be seen that the values of dry unit weight and degree of saturation increased slightly during loading, and the water content decreased very slightly. The changes in water content indicate that the tests were performed under partially drained conditions. Figs. 5-1 and 5-2 show the initial conditions of the specimens on moisture-density diagrams.

The number of loading cycles, the maximum vertical stresses, and the unload vertical stresses are shown in Table 5-2 and Table 5-3. The procedures used in these tests is described in detail in Appendix A.

5.3 Results of $K_0$ Cyclic Loading Tests

Values of $K_0^T$ for various values of OCR during the 1st, 2nd and 100th cycles for tests with 30 psi maximum vertical stress are listed in Table 5-2. Table 5-3 shows the values of $K_0^T$ for various values of OCR in the 1st, 2nd and 10th cycles for tests with 70 psi maximum vertical stress. It can be seen from Table 5-2 and 5-3 that the results for specimens with similar densities and water contents were quite similar, indicating good repeatability.

Fig. 5-3 shows stress paths for the 1st, 2nd, 100th, and 1000th cycles in Test Y8. The degree of saturation of this specimen at the beginning of the test was 91 percent. The slope for each of the stress paths is close to 1.0, except for the 1000th cycle. For the 1st through 100th cycle, almost all of the increase in horizontal stress resulted from increase in pore pressure. The amount of hysteresis in the stress paths is small. About three hours was required to apply one thousand cycles of loading, and significant drainage occurred during this time period, as evidenced by the decrease in water content from 14.9 percent
Table 5-1. Conditions of Specimens for Instrumented Oedometer Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Condition</th>
<th>Condition after testing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water content (%)</td>
<td>Dry unit weight (pcf)</td>
</tr>
<tr>
<td>Y1</td>
<td>14.47 116.74</td>
<td>93.4 0.427 90.4 13.99</td>
</tr>
<tr>
<td>Y2</td>
<td>12.62 117.72</td>
<td>94.2 0.415 81.1 11.68</td>
</tr>
<tr>
<td>Y3</td>
<td>11.65 115.41</td>
<td>92.3 0.444 70.1 11.35</td>
</tr>
<tr>
<td>Y4</td>
<td>12.62 116.69</td>
<td>93.4 0.428 78.8 11.37</td>
</tr>
<tr>
<td>Y5</td>
<td>13.36 115.10</td>
<td>92.1 0.448 79.7 11.66</td>
</tr>
<tr>
<td>Y6</td>
<td>11.83 118.67</td>
<td>94.9 0.404 78.2 11.11</td>
</tr>
<tr>
<td>Y7</td>
<td>8.81 110.10</td>
<td>88.1 0.513 45.8 8.91</td>
</tr>
<tr>
<td>Y8</td>
<td>14.91 115.82</td>
<td>92.7 0.439 90.8 13.52</td>
</tr>
<tr>
<td>Y9</td>
<td>8.65 116.75</td>
<td>95.0 0.403 57.3 8.29</td>
</tr>
<tr>
<td>Y10</td>
<td>8.42 109.24</td>
<td>87.4 0.525 42.8 8.23</td>
</tr>
<tr>
<td>Y11</td>
<td>8.51 116.20</td>
<td>93.0 0.434 52.4 8.35</td>
</tr>
<tr>
<td>Y12</td>
<td>7.94 116.82</td>
<td>93.5 0.426 49.7 7.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b) maximum total vertical stress = 70 psi or higher</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y21</td>
<td>10.86 113.44</td>
<td>90.8 0.469 61.9 10.42</td>
</tr>
<tr>
<td>Y22</td>
<td>12.38 118.31</td>
<td>94.6 0.406 81.0 11.78</td>
</tr>
<tr>
<td>Y23</td>
<td>12.41 119.90</td>
<td>95.9 0.390 85.1 11.65</td>
</tr>
<tr>
<td>Y24</td>
<td>14.86 115.62</td>
<td>92.5 0.441 90.0 14.17</td>
</tr>
<tr>
<td>Y25</td>
<td>12.60 118.87</td>
<td>95.1 0.402 83.8 12.17</td>
</tr>
<tr>
<td>Y26</td>
<td>11.91 118.44</td>
<td>94.8 0.407 78.2 11.66</td>
</tr>
<tr>
<td>Y27</td>
<td>8.50 108.44</td>
<td>86.8 0.536 42.3 8.19</td>
</tr>
<tr>
<td>Y28</td>
<td>9.01 108.99</td>
<td>87.2 0.529 45.5 8.79</td>
</tr>
<tr>
<td>Y29</td>
<td>4.46 105.15</td>
<td>84.1 0.584 20.4 4.06</td>
</tr>
</tbody>
</table>
Fig. 5-1. Compaction Conditions for Instrumented Oedometer Specimens of Yatesville Silty Sand Tested at 30 psi Maximum Pressure.
Fig. 5-2. Compaction Conditions for Instrumented Oedometer Specimens of Yatesville Silty Sand Tested at 70 psi or Higher Maximum Pressure
### Table 5-2. Results of Low Pressure Instrumented Oedometer Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial dry unit weight (pcf)</th>
<th>Initial water content (%)</th>
<th>Number of loading cycles</th>
<th>Peak vertical stress (psi)</th>
<th>Unload vertical pressure (psi)</th>
<th>Total stress earth pressure coefficient at rest, $K^T_{oc}$</th>
<th>OCR</th>
<th>1st Cycle</th>
<th>OCR</th>
<th>2nd Cycle</th>
<th>OCR</th>
<th>100th Cycle</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y1</td>
<td>116.7</td>
<td>14.47</td>
<td>100</td>
<td>30</td>
<td>5 *</td>
<td>0.956 0.995 0.997 0.994 1.005 1.055</td>
<td>0.958</td>
<td>1.004</td>
<td>1.023</td>
<td>0.914</td>
<td>1.060</td>
<td>1.060</td>
<td></td>
</tr>
<tr>
<td>Y2</td>
<td>117.7</td>
<td>12.62</td>
<td>1000</td>
<td>30</td>
<td>5 *</td>
<td>0.877 0.998 1.037 1.050 1.049 1.058</td>
<td>0.875</td>
<td>1.052</td>
<td>1.045</td>
<td>0.907</td>
<td>1.065</td>
<td>1.063</td>
<td></td>
</tr>
<tr>
<td>Y3</td>
<td>115.4</td>
<td>11.65</td>
<td>1000</td>
<td>30</td>
<td>5 *</td>
<td>0.455 0.576 0.675 0.795 0.943 1.146</td>
<td>0.434</td>
<td>0.784</td>
<td>1.100</td>
<td>0.352</td>
<td>0.712</td>
<td>1.040</td>
<td></td>
</tr>
<tr>
<td>Y4</td>
<td>116.7</td>
<td>12.62</td>
<td>10000</td>
<td>30</td>
<td>5</td>
<td>0.806 0.941 0.991 1.005 1.014 1.037</td>
<td>0.792</td>
<td>1.010</td>
<td>1.033</td>
<td>0.827</td>
<td>1.075</td>
<td>1.088</td>
<td></td>
</tr>
<tr>
<td>Y5</td>
<td>115.1</td>
<td>13.36</td>
<td>100</td>
<td>30</td>
<td>5</td>
<td>0.887 0.990 1.016 1.012 1.014 1.035</td>
<td>0.884</td>
<td>1.029</td>
<td>1.040</td>
<td>0.940</td>
<td>1.078</td>
<td>1.069</td>
<td></td>
</tr>
<tr>
<td>Y6</td>
<td>118.7</td>
<td>11.83</td>
<td>1000</td>
<td>30</td>
<td>5</td>
<td>0.535 0.668 0.768 0.882 0.985 1.134</td>
<td>0.511</td>
<td>0.884</td>
<td>1.160</td>
<td>0.414</td>
<td>0.770</td>
<td>0.969</td>
<td></td>
</tr>
<tr>
<td>Y7</td>
<td>110.1</td>
<td>8.81</td>
<td>1000</td>
<td>30</td>
<td>5</td>
<td>0.312 0.393 0.462 0.561 0.678 0.880</td>
<td>0.308</td>
<td>0.580</td>
<td>0.878</td>
<td>0.278</td>
<td>0.575</td>
<td>0.866</td>
<td></td>
</tr>
<tr>
<td>Y8</td>
<td>115.8</td>
<td>14.91</td>
<td>1000</td>
<td>30</td>
<td>5</td>
<td>0.954 0.995 1.003 1.006 1.009 1.031</td>
<td>0.948</td>
<td>1.012</td>
<td>1.032</td>
<td>0.951</td>
<td>1.038</td>
<td>1.048</td>
<td></td>
</tr>
<tr>
<td>Y9</td>
<td>108.8</td>
<td>8.65</td>
<td>1000</td>
<td>30</td>
<td>5</td>
<td>0.319 0.409 0.484 0.613 0.737 0.963</td>
<td>0.314</td>
<td>0.622</td>
<td>0.998</td>
<td>0.283</td>
<td>0.618</td>
<td>1.012</td>
<td></td>
</tr>
<tr>
<td>Y10</td>
<td>109.2</td>
<td>8.42</td>
<td>100</td>
<td>30</td>
<td>5</td>
<td>0.308 0.425 0.528 0.674 0.862 1.050</td>
<td>0.303</td>
<td>0.685</td>
<td>1.042</td>
<td>0.277</td>
<td>0.657</td>
<td>1.207</td>
<td></td>
</tr>
<tr>
<td>Y11</td>
<td>116.2</td>
<td>8.51</td>
<td>100</td>
<td>30</td>
<td>5</td>
<td>0.341 0.489 0.618 0.803 1.007 1.203</td>
<td>0.333</td>
<td>0.816</td>
<td>1.190</td>
<td>0.322</td>
<td>0.827</td>
<td>1.528</td>
<td></td>
</tr>
<tr>
<td>Y12</td>
<td>116.8</td>
<td>7.94</td>
<td>100</td>
<td>30</td>
<td>5</td>
<td>0.351 0.506 0.650 0.883 1.143 1.476</td>
<td>0.342</td>
<td>0.914</td>
<td>1.464</td>
<td>0.328</td>
<td>0.851</td>
<td>1.627</td>
<td></td>
</tr>
</tbody>
</table>

* Specimen was unloaded to vertical stress = 0 during the first load cycle, and to vertical stress = 5 psi during subsequent load cycles.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial dry unit weight (pcf)</th>
<th>Initial vertical stress (psi)</th>
<th>Number of cycles</th>
<th>Unload vertical stress (psi)</th>
<th>1st Cycle OCR</th>
<th>Total stress earth pressure coefficient at rest, K₀</th>
<th>10th Cycle OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y21</td>
<td>113.4</td>
<td>10.86</td>
<td>15</td>
<td>19</td>
<td>0.5324</td>
<td>0.388</td>
<td>1.00</td>
</tr>
<tr>
<td>Y22</td>
<td>118.3</td>
<td>12.38</td>
<td>10</td>
<td>20</td>
<td>0.321</td>
<td>0.547</td>
<td>0.607</td>
</tr>
<tr>
<td>Y23</td>
<td>119.9</td>
<td>12.41</td>
<td>10</td>
<td>21</td>
<td>0.638</td>
<td>0.647</td>
<td>0.747</td>
</tr>
<tr>
<td>Y24</td>
<td>115.6</td>
<td>14.86</td>
<td>10</td>
<td>22</td>
<td>0.536</td>
<td>0.641</td>
<td>0.733</td>
</tr>
<tr>
<td>Y25</td>
<td>118.9</td>
<td>12.60</td>
<td>10</td>
<td>23</td>
<td>0.328</td>
<td>0.550</td>
<td>0.667</td>
</tr>
<tr>
<td>Y26</td>
<td>118.4</td>
<td>11.91</td>
<td>10</td>
<td>24</td>
<td>0.328</td>
<td>0.550</td>
<td>0.667</td>
</tr>
<tr>
<td>Y27</td>
<td>108.4</td>
<td>8.50</td>
<td>10</td>
<td>25</td>
<td>0.339</td>
<td>0.419</td>
<td>0.672</td>
</tr>
<tr>
<td>Y28</td>
<td>109.0</td>
<td>9.01</td>
<td>3</td>
<td>26</td>
<td>0.339</td>
<td>0.419</td>
<td>0.672</td>
</tr>
<tr>
<td>Y29</td>
<td>105.2</td>
<td>4.46</td>
<td>3</td>
<td>27</td>
<td>0.339</td>
<td>0.419</td>
<td>0.672</td>
</tr>
</tbody>
</table>

* The maximum vertical stress for the 2nd cycle was 119 psi.
** The maximum vertical stress for the 2nd cycle was 101 psi.
Fig. 5-3. Instrumented Oedometer Test Result (Test Y8)
to 13.5 percent. It can be seen that the value of $K_0^T$ decreased significantly from the 100th to the 1000th cycle, as drainage occurred.

Fig. 5-4 shows stress paths for Test Y24, with 70 psi maximum vertical stress. The initial degree of saturation was 90 percent, nearly the same as that for Test Y8. It can be seen that the results are very similar to the results of test Y8, with values of $K_0^T$ approximately nearly equal to 1.0. The test was stopped after 10 cycles, and there was thus not time for significant drainage.

The results shown in Figs. 5-5 and 5-6 were obtained from tests on specimens with initial water contents close to optimum and initial degrees of saturation equal to 79 percent and 81 percent. The results for Test Y4, with 30 psi maximum vertical stress, are shown in Fig. 5-5, and the results for Test Y22, with 70 psi maximum vertical stress, are shown in Fig. 5-6. Values of $K_0^T$ during primary loading obtained from these tests were 0.7 to 0.8. These values are less than the values measured for Test Y8 and Y24, but considerably higher than would be expected if pore pressures were zero. If the pore pressures were zero, the value of $K_0$, estimated using Jaky's equation, would be about 0.40. It seems clear that significant values of positive pore pressure were developed during loading in these tests. The stress paths obtained from these tests showed small amounts of hysteresis. The values of $K_0^T$ at the maximum stress increased slightly with number of cycles up to 100 cycles, and then decreased as a result of drainage during the period of two hours required to apply 1000 cycles of loading to specimen Y4.

Fig. 5-7 shows the stress paths obtained from Test Y3, in which the specimen was compacted dry of optimum, and the initial degree of saturation was 70 percent. The initial total horizontal stress was found to be non-zero at zero total vertical stress. It seems likely that this non-zero intercept resulted from soil suction in the specimen, which gave it appreciable apparent cohesion. The value of $K_0^T$ was about 0.46 during primary loading,
Fig. 5-4. Instrumented Oedometer Test Result (Test Y24)
Fig. 5-5. Instrumented Oedometer Test Result (Test Y4)
Fig. 5-6. Instrumented Oedometer Test Result (Test Y22)
Fig. 5-7. Instrumented Oedometer Test Result (Test Y3)
and decreased continuously with increasing number of loading cycles, reaching $K_0^T = 0.35$ after 1000 cycles. The stress paths exhibit hysteresis, and the amount of hysteresis decreases with increasing number of loading cycles.

Fig. 5-8 shows stress paths for Test Y21, with similar initial conditions, but with a higher maximum stress (70 psi). It can be seen that the result are quite similar to those shown in Fig. 5-7.

Fig. 5-9 shows the result of Test Y11. The specimen used in this test was very dry, and its initial degree of saturation was 52%. Values of $K_0^T$ at the maximum stress were low, about 0.34, and they did not change appreciably with number of cycles. A significant amount of initial total horizontal stress was observed.

Fig. 5-10 shows the result of Test Y28 in which a higher maximum stress was applied to a specimen having initial conditions similar to those in Y11, but a lower dry unit weight. The initial total horizontal stress at zero vertical stress was smaller than for Y11, which probably indicates smaller initial soil suction. The slope of the primary loading path shown in Fig. 5-10 increases with increasing vertical stress, and the value of $K_{0\alpha}^T$ increases to 0.42 at a vertical stress of 120 psi. This value of $K_{0\alpha}^T$ is close to the value of $K_0$ calculated using Jaky's equation, using the value of $\phi'$ for Yatesville silty sand at this density ($\phi' = 33.7$ deg). The values of $K_{0\alpha}^T$ may increase due to (1) increase in degree of saturation at higher vertical stress, or (2) transition from a reloading path to a primary loading path with respect to effective stress.

The values of $K_0^T$ at vertical stress = 30 psi during primary loading are plotted with values of dry unit weight in Fig. 5-11. For the sake of convenience, values of $K_0^T$ at vertical stress = 30 psi during primary loading are denoted by $K_{30}^T$. The values of $K_0^T$ obtained from soaking tests that are described in Chapter 7 are also shown in Fig. 5-11. It
Fig. 5.8. Instrumented Oedometer Test Result (Test Y21)

Initial Condition of the Specimen
water content = 10.86 %
dry unit weight = 113.4 pcf
relative compaction = 90.8 %
void ratio = 0.469
degree of saturation = 61.9 %

INSTRUMENTED OEDOMETER TEST RESULTS
Fig. 5-10. Instrumented Oedometer Test Result (Test Y28)
Fig. 5-11. Relationship Between $K_{30}^T$ and Initial Dry Unit Weight
can be seen that the results are scattered, and that there is a bi-modal distribution of $K_{30}^T$ with dry density.

Fig. 5-12 shows the variation of $K_{30}^T$ with compaction water content. It can be seen that there is some scatter, but a clear trend of variation of $K_{30}^T$ with water content. However, water content is not very useful for correlation, because its values vary from soil to soil, and the same value of water content does not necessarily mean the same degree of saturation or deviation from optimum water content for different soils.

Fig. 5-13 shows the variation of $K_{30}^T$ with degree of saturation. It can be seen that the trend is similar to that shown in Fig. 5-12. The values of $K_{30}^T$ increase sharply as the degree of saturation increases to 70 or 80 percent. For degrees of saturation less than 60 percent, $K_{30}^T$ is nearly constant.

Fig. 5-14 shows a plot of $K_{30}^T$ with deviation from optimum water content. It can be seen that the trend is similar to those in Figs. 5-12 and 5-13. It can also be seen that a significant change in the value of $K_{30}^T$ occurs about one percent below optimum water content. It seems likely that it is at this water content that the air in the soil voids becomes occluded.

Contours of $K_0^T$ at 30 psi vertical stress during primary loading ($K_{30}^T$) are shown in Fig. 5-15. It can be seen that the contours of $K_{30}^T$ are nearly parallel to the contours of degree of saturation for relative compaction greater than 90 percent, and that the value of $K_{30}^T$ decreases with decreasing degree of saturation. Therefore, it seems reasonable that $K_{30}^T$ can be correlated with fair accuracy to a single parameter, degree of saturation. However, it can be noted that the values of $K_{30}^T$ decrease slightly with increasing dry unit weight at constant degree of saturation. Also, it can be seen that the contours are very close in the range of 70 to 90 percent saturation. Fig. 5-15 can be used for estimating values of $K_0^T$ for primary loading.
Fig. 5-12. Relationship Between $K_{30}^T$ and Initial Water Content
Fig. 5-14. Relationship Between $K_{30}^T$ and Deviation from Optimum Water Content
Fig. 5-15. Total Stress At-rest Earth Pressure Coefficient at Vertical Pressure = 30 psi
Similar studies were made for the ratio of values of $K_0^T$ at 5 psi vertical stress ($K_3^T$) to values of $K_{30}^T$. However, no clear relationships were found between $K_3^T/K_{30}^T$ and dry unit weight, water content, degree of saturation, or deviation from the optimum water content. The relationships were all found to be quite scattered. Based on the findings, it was concluded that values of $K_3^T/K_{30}^T$ cannot be correlated with a single variable. A correlation of $K_3^T/K_{30}^T$ with several variables is discussed in section 5.4.

Fig. 5-16 shows contours of $K_0^T$ at 5 psi vertical stress during unloading ($K_3^T$). It can be seen that, for conditions dry of optimum, the values of $K_3^T$ increase with increasing dry unit weight and with decreasing water content. However, for conditions wet of optimum, values of $K_3^T$ are independent of dry unit weight and water content, and close to 1.0.

Fig. 5-17 shows contours of the ratio of $K_3^T$ to $K_{30}^T$. For soils with values of relative compaction larger than 90%, the ratio $K_3^T / K_{30}^T$ increases with decreasing water content, and it is nearly independent of dry unit weight.

### 5.4 Simple Method for Estimating the Value of $K_0^T$ for Yatesville Silty Sand

It was found that the correlation between the value of $K_{30}^T$ and degree of saturation is consistent, as shown in Fig. 5-13. The value of $K_{30}^T$ for a specific initial condition can thus be determined from Fig. 5-13 or Fig. 5-15. Alternatively, the value of $K_{30}^T$ can be approximated by the following equations.

\[
K_{30}^T = 0.35 \quad \text{ (S<0.7)} \quad \text{(5.1.a)}
\]

\[
K_{30}^T = 3.7 \cdot S - 2.24 \quad \text{ (0.7<S<0.85)} \quad \text{(5.1.b)}
\]

\[
K_{30}^T = \frac{2}{3} S + \frac{1}{3} \quad \text{ (S>0.85)} \quad \text{(5.1.c)}
\]
Fig. 5-16. Total Stress At-rest Earth Pressure Coefficient at Vertical Pressure = 5 psi During Unloading
Fig. 5-17. Ratio of Total Stress At-rest Earth Pressure Coefficient at Vertical Pressure of 5 psi ($K_s^T$) to that at 30 psi ($K_{s0}^T$)
Eq. (5.1.a) applies to conditions dry of optimum, Eq. (5.1.b) to conditions near optimum, and Eq. (5.1.c) to conditions wet of optimum. The variation of $K_{30}^T$ with $S$ described by these equations is compared to the experimental data in Fig. 5-18.

Values of $K_0^T$ for primary loading, $K_{0,NC}^T$, at any value of vertical pressure are essentially the same as values of $K_{30}^T$ for the range of vertical stress used in this study. However, when very large values of vertical stress are applied to partly saturated soil, the soil is compressed, and the degree of saturation increases, ultimately to 100 percent. In this condition, values of $K_{0,OC}^T$ become equal to 1.0. This trend can be seen in the results of $K_0$-laboratory tests performed by Ofer (1982) (Fig. 2-5) and Clayton et al. (1991). Figs. 5-13 and 5-15, and Eqs. (5.1) would not be applicable to this situation.

It is desirable also to be able to estimate values of total stress earth pressure coefficient, $K_0^T$, during unloading. A power function relating the effective stress value of $K_0$ to OCR was proposed by Schmidt (1967), and has been widely accepted. It seems logical to examine whether a similar relationship may exist for total stress $K_0$. It would have the form:

$$K_{3,OC}^T = K_{0,NC}^T \cdot (OCR)^\alpha$$  \hspace{1cm} (5.2)

where

$K_{0,NC}^T = K_0^T$ during primary loading ($= K_{30}^T$),

$K_{0,OC}^T = K_0^T$ during unloading, and

$\alpha =$ unloading exponent.

As a first step in examining the usefulness of Eq. (5.2), diagrams like the one shown in Fig. 5-17, which applies to OCR = 6, were drawn for OCR = 1.5, 2.0, 3.0, and 4.0. Values of the ratio $K_{0,OC}^T/K_{30}^T$ at values of relative compaction = 95 and 90 percent, and degree of saturation = 50, 60, 70, 80, and 90 percent were determined. Fig. 5-19 shows the variation of $K_0^T / K_{30}^T$ during unloading with OCR on a log-log plot. It can be seen
Fig. 5-18: Variation of $K_{30}$ with Degree of Saturation Described by the Proposed Equations (5.1)
Fig. 5-19. Relationship Between $K_{0,OC}/K_{0,NC}$ and OCR
that there is a different linear relationship between the logarithm of $K_{o}^{T}/K_{30}^{T}$ and the logarithm of OCR for each value of S and RC. This shows that values of $\alpha$ (the slopes of the lines in Fig. 5-19) vary with degree of saturation and relative compaction.

The values of $\alpha$ determined from Fig. 5-19 are plotted against degree of saturation in Fig. 5-20. The plot shows clearly that the values of $\alpha$ depend on degree of saturation and relative compaction. To make the diagram more easily useable, data for values of relative compaction equal to 92.5 percent and 87.5 percent were added to the plot. The values of $\alpha$ decrease with increasing degree of saturation and with decreasing values of relative compaction. For the fully saturated condition, the value of $\alpha$ is equal to zero. The values of $\alpha$ are smaller for loose soil than for dense soil.

Values of total stress earth pressure coefficient, $K_{o}^{T}$, during unloading can be calculated using Eq. (5.2). The value of $K_{o,NC}^{T}$ can be obtained from Fig. 5-13 or Eq. (5.1), and the value of $\alpha$ can be obtained from Fig. 5-20. These simple relationships incorporate all of the factors that have a major effect on lateral earth pressures in compacted Yatesville silty sand.

5.5 Design Diagram for Estimating At-Rest Earth Pressures in Moist Soils

The method described in the previous section can be used to estimate the most probable values of at-rest lateral earth pressures in moist soils. However, as discussed previously, the values of $K_{30}^{T}$ are quite sensitive to change in degree of saturation in the range of optimum water content. Accurate estimation of values of $K_{o,NC}^{T}$ is thus quite difficult. Therefore, it is recommended to use an envelope of the experimental results for design. Values of $K_{o,NC}^{T}$ can be assumed to be equal to 0.40 for $S < 70$ percent and to be equal to 1.0 for $S > 70$ percent, as shown in Fig. 5-21. Similarly, an envelope of the experimental values of unloading exponent $\alpha$ is presented in Fig. 5-22. Design values of
Fig. 5-20. Relationship Between Unloading Parameter $\alpha$ and Degree of Saturation for Different Values of Relative Compaction
Deviation from Optimum Water Content at RC = 95% 

Fig. 5.21. Design Diagram for Estimating $K_{0,NC}$ from Degree of Saturation

INSTRUMENTED OEDOMETER TEST RESULTS
Fig. 5-22. Design Diagram for Estimating $\alpha$ from Degree of Saturation
$K_{0,OC}^T$ can be calculated using values of $K_{0,NC}^T$ obtained from Fig. 5-21 and values of the unloading exponent $\alpha$ obtained from Fig.5-22. It is believed that the proposed diagram can be used for other moist soils, as well as Yatesville silty sand. This simple procedure provides conservatively large values of $K_{0,OC}^T$. 
CHAPTER 6
PORE PRESSURE ANALYSES FOR ESTIMATING
COMPACTION-INDUCED LATERAL EARTH PRESSURES

6.1 Introduction

The results shown in Chapter 5 and results reported previously (Carder et al., 1980; Ofer, 1983; Symons et al., 1987; Filz, 1992) indicate that both positive and negative pore pressures affect the behavior of partly saturated soils under one-dimensional loading. It was shown in Chapter 5 that behavior with respect to total stress can be estimated fairly well using the results of instrumented oedometer tests. However, it is necessary to be able to estimate pore pressures in order to predict compaction-induced lateral pressures without performing $K_0$ tests.

Filz (1992) extended Duncan and Seed's (1986) model for compaction-induced earth pressures to moisture-sensitive soils, and proposed a method for estimating the magnitudes of pore pressures during compaction. He analyzed the results of tests on an instrumented retaining wall backfilled with Yatesville silty sand. Although the model provided good agreement with the test results for backfills of dry sand, the lateral pressures estimated using the extended $K_0$ model were not in good agreement with the instrumental retaining wall test results for moist backfills.

In this chapter, the results of instrumented oedometer tests are analyzed using Filz's extended model, and the method for estimating pore pressure parameters proposed by Filz (1992). Subsequently, a method is discussed and proposed for estimating pore pressures based on the volume change characteristics of partly saturated soils. Applying the proposed method to the effective stress $K_0$ model, pore pressure response and total stress behavior under one-dimensional loading are estimated. The estimated stress paths are compared with the results of the instrumented oedometer tests.
6.2 Evaluation of Filz's (1992) Extended $K_o$ Model

Filz (1992) extended the $K_o$ model for use in estimating compaction-induced lateral earth pressures in moist soils. The extension consisted of:

(1) Suggestion of a method for evaluating initial pore pressures and the values of Skempton's (1954) pore pressure parameters $A$ and $B$, and

(2) Estimation of effective and total stress paths for moist soils using the initial pore pressures and pore pressure parameters.

Filz (1992) suggested using the relationship between the effective stress and total stress strength envelopes to estimate the initial pore pressures $u_0$ and the value of pore pressure parameter $B$, as described in chapter 2. Assuming a value for Skempton's pore pressure parameter $A$, the values of $u_0$ and $B$ can be calculated. Skempton (1961) used the same approach to estimate capillary pressures in situ. The expressions for $u_0$ and $B$ are:

\[
u_0 = \frac{1 + (2 \cdot A - 1) \cdot \sin \phi' \cdot \cos \phi}{1 + (2 \cdot A - 1) \cdot \sin \phi \cdot \sin \phi'} \cdot c + \frac{c'}{\tan \phi'} \tag{6.1}\]

\[
B = \frac{\sin \phi' - \sin \phi}{\{1 + (2 \cdot A - 1) \cdot \sin \phi\} \cdot \sin \phi'} \tag{6.2}
\]

in which,

$u_0 = \text{initial pore pressure (pressure unit)},$

$\phi' = \text{effective stress friction angle (dimensionless)},$

$c' = \text{effective stress cohesion (pressure unit)},$

$\phi = \text{total stress friction angle (dimensionless)},$

$c = \text{total stress cohesion (pressure unit)},$
A = pore pressure parameter with respect to changes in deviator stress 
(dimensionless), and

B = pore pressure parameter with respect to changes in all-around stress 
(dimensionless).

Stress paths for Test Y4 and Y12 were estimated using the procedure suggested by Filz (1992). In the first series of analyses, the results of UU triaxial tests with low cell pressures (Filz, 1992) were used, and Skempton’s pore pressure parameter A was assumed to be equal to 0.5. The specimen of Test Y4 was compacted at a water content close to optimum (degree of saturation = 79 percent). Using the total stress strength parameters from low-pressure UU triaxial tests (ϕ = 28.4 deg and c = 8.9 psi), the value of u₀ was calculated to be -13.2 psi, and Skempton’s pore pressure parameter B was calculated to be 0.20. The total stress path calculated using these values of u₀ and B are compared to the measured total stress path in Fig. 6-1. The calculated total stress path differs significantly from the measured path. It can be seen that the calculated effective stress path is offset from the axes by the amount of the initial pore pressure.

A second comparison, for Test Y12, is shown in Fig. 6-2. The specimen of Test Y12 was compacted dry of optimum (degree of saturation = 43 %). Using the total strength parameters from low-pressure UU triaxial tests (ϕ = 35.5 deg and c = 11.5 psi), values of u₀ and B were calculated to be -15.7 psi and 0.024. As can be seen in Fig. 6-2, the calculated total stress path is not in good agreement with the measured total stress path. The measured path shows much greater hysteresis than the calculated one, and exhibits non-zero total horizontal stress at the end of compaction.

Filz (1992) found similar lack of agreement between his calculations and instrumented retaining wall test results. He speculated that improved agreement between calculated and measured results might be achieved by using results of UU tests performed
Fig. 6-1. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y4) - [1]
Fig. 6-2. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y12) - [1]
using higher confining pressures, which would be more representative of pressures during compaction. To examine this possibility, UU triaxial tests with high cell pressures were performed on Yatesville silty sand during this study. The results of the tests are shown in Appendix F.

Figs. 6-3 and 6-4 show the stress paths calculated using values of \( u_0 \) and B based on the results of the UU triaxial tests with high cell pressures. As before, the value of Skempton's pore pressure parameter A was assumed to be 0.5. For Test Y4, the values of total stress strength parameters were \( \phi = 16.2 \text{ deg} \) and \( c = 10.3 \text{ psi} \). The corresponding values of \( u_0 \) and B were calculated to be -16.7 psi and 0.53. As can be seen in Fig. 6-3, the use of high pressure UU test results improved the agreement between calculated and measured values slightly, but the estimated total stress path is still quite different from the measured stress path. For Test Y12, the total strength parameters from high-pressure UU triaxial tests were \( \phi = 35.7 \text{ deg} \) and \( c = 11.3 \text{ psi} \) and the corresponding values of \( u_0 \) and B were \( u_0 = -15.4 \text{ psi} \), \( B = 0.019 \). Using these values did not improve the agreement between calculation and measurement.

Filz (1992) suggested that it was reasonable to assume that the Skempton's pore pressure parameter A is equal to 0.5, because changes in the values of A did not affect his calculation much. Skempton (1961) assumed A = 0.25 in a similar model, and concluded that the assumption was a good approximation. The effect of the value of the pore pressure parameter A on the agreement between measured and calculated results for instrumented oedometer tests was examined by performing analyses using A = 0.0 and A = 1.0. The results of the high-pressure UU triaxial tests were used for the analyses to calculate the corresponding values of \( u_0 \) and B. Figs. 6-5 and 6-6 show the results for analyses A equal to 0.0, and Figs. 6-7 and 6-8 show the results for A equal to 1.0. It can
Fig. 6-3. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y4) - [2]
Fig. 6-4. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y12) - [2]
Fig. 6-5. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y4) - [3]
Fig. 6-6. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y12) - [3]
Fig. 6-7. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y4) - [4]
Fig. 6.8: K₀ Model Analysis of Instrumented Oedometer Test (Test Y12).
be seen that the value of pore pressure parameter A does not influence the calculated behavior significantly.

It is clear that estimating values of \( u_0 \) and B from total and effective stress strength parameters did not work satisfactorily. As Filz (1992) suggested, it seems likely that the inaccuracy is caused by the difference between pore pressures at failure and those at the non-failure conditions in the instrumented oedometer tests. An alternate approach to estimating pore pressures is discussed in the following section.

6. 3 Theoretical Procedure for Estimating Pore Pressure Parameters

The concept of pore pressure parameters is useful for estimating the magnitude of changes in pore pressure caused by changes in total stress. Skempton (1954) defined two parameters (A and B) which are relate to changes in pore pressures caused by deviator stress and changes in all-round stress. Values of A and B are related for one-dimensional compression because horizontal stress varies with vertical stress under one-dimensional conditions. In one-dimensional compression, it is more convenient to use a single parameter \( \bar{B} \), which is defined with respect to change in total vertical stress.

Expressions for the pore pressure parameters B and \( \bar{B} \) were derived by Fredlund and Rahardjo (1993). The expressions derived by Fredlund and Rahardjo involve parameters which cannot be determined from conventional tests, as described in Appendix D. However, the equations can be simplified, and these parameters can be eliminated, by making assumptions that are reasonable for the Yatesville silty sand test results that are of interest here:

(1) the pore water is incompressible, and

(2) the compression of soil due to change in soil suction is negligible.
Based on these assumptions, the value of $\overline{B}$ can be estimated as a function of the compressibility of the soil, the degree of saturation, the porosity, and the pore air pressure, as shown by Eq. (6.3). The derivation of Eq (6.3) is described in Appendix D.

\[
\overline{B} = \frac{1}{1 + \frac{n \cdot (1 - S + S \cdot H)}{m_v \cdot u_a}} \tag{6.3}
\]

where

$\overline{B}$ = tangent pore pressure parameter with respect to change in vertical stress (dimensionless),

$n$ = porosity (dimensionless),

$S$ = degree of saturation (dimensionless),

$H$ = solubility of air in water = 0.02 (dimensionless),

$m_v$ = compressibility of soil with respect to a change in vertical stress (reciprocal of pressure units), and

$u_a$ = pore air pressure [absolute pressure] (pressure units).

Based on the theory of elasticity, values of compressibility $m_v$ can be related to Young's modulus by the following equation.

\[
m_v = \frac{(1 + \nu) \cdot (1 - 2 \nu)}{(1 - \nu) \cdot E'} \tag{6.4}
\]

where

$\nu$ = Poisson's ratio (dimensionless), and

$E'$ = effective stress value of Young's modulus (pressure units).

Values of Poisson's ratio can be calculated using Eq. (2-12), which is used in the effective stress $K_o$ model.
In order to apply Eq. (6.3), changes in values of modulus, degree of saturation, and porosity with changes in stress must be known. Values of undrained (total stress) initial modulus can be expressed by the following equation, as shown in Appendix F.

\[ E_i = K \cdot P_a \cdot \left( \frac{\sigma_3}{P_a} \right)^n \]  

(6.5)

where

\( E_i \) = initial modulus (pressure units),

\( K \) = modulus number (dimensionless) (shown in Fig. F-13),

\( n \) = modulus exponent (dimensionless) (shown in Fig. F-14),

\( \sigma_3 \) = total minor principal stress (pressure units), and

\( P_a \) = atmospheric pressure (pressure units).

The values of \( K \) and \( n \) shown in Figs. F-13 and F-14 can be used to calculate undrained modulus values, whereas Eq. (6-3) requires values of drained modulus. However, as discussed in Appendix F, values of undrained and drained modulus differ by only about 13 percent for the saturated condition and 0 percent for the dry condition, and can be assumed to be equal for practical purposes.

Values of tangent modulus change with changing deviator stress. Values of tangent modulus can be evaluated as:

\[ E_t = E_i \cdot \left( 1 - \frac{R_f \cdot (\sigma_1 - \sigma_3) \cdot (1 - \sin \phi)}{2\sigma_3 \cdot \sin \phi + 2c \cdot \cos \phi} \right)^2 \]  

(6.6)

where

\( E_t \) = tangent modulus (pressure units), and

\( R_f \) = ratio of ultimate strength to failure strength (dimensionless) \((R_f = 0.8 \text{ was used in this study})\).
The value of modulus for unloading and unloading, \( E_{ur} \), is reported to be in the range between 1.5 \( E_i \) for dense sand and 3.0 \( E_i \) for soft clay (Duncan et al., 1980). For the Yatesville silty sand of interest here, it is reasonable to assume that:

\[
E_{ur} = 1.5 \cdot E_i
\]  \hfill (6.7)

Degree of saturation and porosity change as pressure is applied and the soil compresses. The current values of degree of saturation and porosity can be calculated using the estimated values of modulus. As seen in Eq. (6.3), the value of \( \bar{B} \) is a function of the values of \( u_r \) and the modulus value, which varies with the value of the minor principal stress. Consequently, it was necessary to iterate in order to calculate values of \( \bar{B} \).

Results of parametric studies using Eq. (6-3) are shown in Figs. 6-9 and 6-10. The effect of dissolving air in water are negligible except when the degree of saturation is very close to 100 percent, as shown in Fig. 6-9. Consequently, the effect of dissolving air in water is neglected in subsequent discussions. As values of modulus decrease, the values of \( \bar{B} \) increase significantly, as shown in Fig. 6-10. For stiff soils (high modulus values), it can be seen that changes in values of \( \bar{B} \) with degree of saturation are very large for values of degree of saturation greater than about 80 percent. The relationship between \( \bar{B} \) and \( E' \) is highly non-linear. For less stiff soils, the values of \( \bar{B} \) do not change so rapidly with degree of saturation.

6.4 Effective Stress Analysis of At-Rest Earth Pressures

Values of initial pore pressure must be known in order to estimate values of \( K_0 \) for moist soils. The values of initial pore pressures can be inferred without measurements, using either a moisture-capillary relationship like VanGenuchten's model, or the procedure
Fig. 6-9. Effects of Dissolving Air in Water on Pore Pressure Parameter

- Neglect dissolving of air in water
- Take into account dissolving air in water in accordance with Henry's Law

Young's Modulus \( E = 500 \text{ psi} \)

Poisson's ratio \( \nu = 0.3 \)

Initial pore air pressure = 0.0
Fig. 6-10. Effects of Values of Modulus on Pore Pressure Parameter
suggested by Filz (1992), which makes use of total and effective stress strength envelopes. It is difficult to apply VanGenuchten's model, because:

(1) The calculated values of suction are quite sensitive to the values of the parameters used in the model, and the values of the parameters cannot be determined accurately based on the visual description of a soil, as discussed in Appendix E, and,

(2) The model estimates only the capillary pressures (suction). The effects of the capillary pressures on effective stress can be evaluated by multiplying the suction by Bishop's (1959) coefficient $\chi$. In addition, values of pore air pressure must be assumed or measured when the model is used for estimating the effects of initial pore pressures.

On the other hand, Filz's method directly estimates the effects of pore pressures on effective stress, not the suction values. Also, the parameters required for Filz's method are the total stress and effective stress strength parameters, which can be obtained from conventional triaxial tests. Consequently, Filz's method appears to be more useful for practical purposes, and it was used in the following analyses. Fig. 6-11 shows the values of initial pore pressure calculated using Eq. (6-2) with the results of the high-pressure UU triaxial tests presented in Appendix F. The value of $A$ was assumed to be equal to 0.0 for the calculations, because higher values of $A$ provided unreasonably high values of initial pore pressures, even for high values of degree of saturation.

In the extended $K_0$ model, Filz (1992) considered that prestress induced by soil suction could be evaluated by assuming that the horizontal total stress was zero at zero vertical total stress. However, the results of the instrumented oedometer tests performed on specimens with initial water content dry of optimum showed that significant amount of total horizontal stress was locked within the specimens at the end of compaction. Filz's
Fig. 6-11. Initial Pore Pressure $u_0$ Calculated by Filz's (1992) Method Using Results of High Pressure UU Triaxial Tests
method provides the minimum possible prestress (zero). On the other hand, the maximum possible prestress is determined by assuming that the stress state at the end of compaction is located on the effective stress $K_{lim}$ line.

In this section, initial total horizontal stresses at zero vertical stress were determined based on the test results, rather than using Filz's assumption or the $K_{lim}$ line assumption. Contours of values of the measured initial horizontal stress are drawn on a moisture-density diagram, in Fig. 6-12.

Summarizing the preceding discussions, the analyses which follow involve these main points:

1. Values of effective stress strength parameters are obtained from Fig. F-3.
2. Values of total stress strength parameters are obtained from Fig. F-12.
3. Values of modulus are calculated using modulus number and modulus exponent obtained from Figs. F-13 and F-14 in Eqs. (6-5), (6-6), and (6-7):
4. Values of the pore pressure parameter $\bar{B}$ are calculated using Eq. (6-3):
5. Values of initial pore pressure $u_0$ are calculated using Eq. (6-1) (shown in Fig. 6-11): and
6. Values of initial horizontal stress are obtained from the experimental results (shown in Fig. 6-12).

It was found that the pore air pressures calculated using this procedure were sometimes greater than total stress, because of the hysteretic pore pressure response incorporated in the model. This condition was avoided by incorporating the following constraint within the model: When values of pore air pressure become equal to values of total stress, values of effective stress remain constant, and the changes in pore pressure are equal to the changes in total stress.
Fig. 6-12. Initial Horizontal Stress Measured in the Instrumented Oedometer Tests
Using this model, loading to vertical stress equal to 30 psi and subsequent unloading were simulated for various values of relative compaction and degree of saturation. Values of total stress earth pressure coefficient, $K_0^T$ at 30 psi vertical stress during primary loading were calculated, as shown in Fig. 6-13. Plots of calculated values of $K_0^T$ at OCR = 2 and 6 during unloading are shown in Figs. 6-14 and 6-15. The results show the effects of relative compaction and degree of saturation. The increases in the values of $K_0^T$ at low degrees of saturation resulted from the non-zero initial values of horizontal total stress.

Values of $K_0^T$ at vertical stress of 30 psi during primary loading for each instrumented oedometer test was calculated using the proposed model. The difference between the calculated and measured values of $K_0^T$ for each test ($\Delta K_0^T$) are plotted in Fig. 6-16. The values of $\Delta K_0^T$ range from -0.06 to +0.22, with the biggest deviations around degree of saturation equal to 80 percent. This difference is quite small, considering the fact that the measured values of $K_0^T$ change very quickly with degree of saturation in this range. The measured values of $K_0^T$ changed by about 0.5 as degree of saturation changed by about 6 percent.

In addition, the values of degree of saturation can not be determined with perfect precision. Assuming that errors in values of dry unit weight, specific gravity, and water content are $\pm 0.5$ pcf, $\pm 0.01$ and $\pm 0.3$ percent, the corresponding error in the calculated degree of saturation is about $\pm 3$ percent. Taking into account these uncertainties in the values of the parameters involved in the calculations, it can be concluded that the proposed method provides excellent agreement with the test results.

Figs. 6-17 and 6-18 show the results of similar analyses for OCR = 2 and OCR = 6 during unloading. As seen in the figures, the agreement is not as good as for primary loading. The estimated values of $K_0^T$ tend to be smaller than the measured values of $K_0^T$, and they differ by as much as 40 percent at degree of saturation near 60 percent.
Fig. 6-13. Theoretical Values of $K_{30}^T$ Calculated Using the Proposed Method for Estimating Pore Pressure Responses.

PORE PRESSURE ANALYSES
Fig. 6-14. Theoretical Values of $K_{0,oc}^T$ Calculated Using the Proposed Method for Estimating Pore Pressure Responses at OCR = 2
Fig. 6-15. Theoretical Values of $K_{0,oc}^T$ Calculated Using the Proposed Method for Estimating Pore Pressure Responses at OCR = 6
Fig. 6-16. Difference Between Theoretical $K_{30}^T$ and Experimental $K_{30}^T$, $\Delta K^T$
Fig. 6-17. Ratio of Difference Between Theoretical and Experimental Values of $K_0^T$, Divided by Experimental $K_0^T$ at OCR = 2
Fig. 6.18. Ratio of Difference Between Theoretical and Experimental Values of $K_0^T$ Divided by Experimental $K_0^T$ at OCR = 6.
Using the proposed method, total and effective stress paths were calculated for Test Y4. As shown in Fig. 6-19, the estimated values of $K_0^T$ are somewhat smaller than the experimental values. Assuming that the values of dry unit weight and water content are equal to the measured values plus 0.5 pcf and 0.3 percent, the calculated stress paths shown in Fig. 6-20 were obtained. It can be seen the calculated stress path is very close to the experimental results, indicating that differences between calculated and measured values of $K_0$ could be caused mainly by errors in calculating density and water content.

Fig. 6-21 shows the estimated stress paths for Test Y12. The measured stress path is more hysteretic than the calculated stress path. As a result, there are appreciable discrepancies in $K_0^T$ during unloading. It was speculated that the difference might be due to differences in the value of the unloading exponent $\alpha$. However, it was found that increasing the values of $\alpha$ by one standard deviation did not improve the accuracy significantly.

The causes of the discrepancies between measured and calculated values of $K_0^T$ during unloading may be related to hysteretic stress-strain behavior, or to changes in soil suction with pressure that are not represented in the calculations. Accurate estimation of unloading stress paths for soils compacted with low degree of saturation will require continuing studies.
Test Y4

Initial degree of saturation = 78.8%
Initial relative compaction = 93.4%

Effective and Total Lateral Pressure (psi)

Effective and Total Vertical Pressure (psi)

Measured total stress path

Calculated total stress path

Calculated effective stress path

$U_0 = -91.2$ psi
$B_{sec} = 0.547$
Fig. 6-20. $K_0$ Model Analysis of Instrumented Oedometer Test (Test Y4) Assuming Errors in Measurements of Initial Condition of the Specimen.
Fig. 6-21. Ko Model Analysis of Instrumented Oedometer Test (Test Y12)
CHAPTER 7  
RESULTS OF SOAKING TESTS ON YATESVILLE SILTY SAND

7.1 Introduction
The test results described in chapter 5, and the analyses shown in chapter 6, are concerned with the short term behavior of soils during and immediately after compaction. Understanding long term behavior is also important. As shown by the literature review in chapter 2, values of total horizontal earth pressure at rest may change appreciably with time after compaction, and changes are caused by dissipation of pore pressures, release of soil suction, and stress relaxation.

The studies described in this chapter were performed to investigate the effects of drainage and release of suction on compaction-induced earth pressures in Yatesville silty sand. To investigate these effects, specimens of Yatesville silty sand were soaked (submerged) while still under load. The soil suction or positive pore pressures in the specimens were reduced essentially to zero by submerging the specimens. In the following sections, the results of these tests are presented, and the effects of long-term changes in moisture on at-rest earth pressures are discussed.

7.2 Test Program and Test Procedures
The instrumented oedometer device was submerged after the test specimens had been loaded in order to release soil suction, or to allow pore pressures to dissipate. Because the load cells used to measure lateral forces are not submergible, the specimens were loaded with the device set in upright position, with the load cells on top. The oedometer, containing the test specimens, was submerged after loading by placing it in a water-filled container. Fig. 7-1 compares the stress path obtained from a test performed
Fig. 7-1. Influence of the Orientation of the Instrumented Oedometer on the Test Results (Test Y31 Compared with Test Y10)
with the oedometer in an upright position with the stress path from a test with the oedometer in the normal horizontal position. It can be seen that the orientation of the device did not influence the results.

Table 7-1 shows the conditions of the soaking test specimens before and after testing. Soaking resulted in increased values of degree of saturation; however, it did not result in complete saturation of the specimens. The final values of degree of saturation were about 80 percent for specimens which had initial degrees of saturation lower than 80 percent. It seems likely that small occluded air bubbles remained in the soil voids after soaking.

7.3 Test Results

Fig. 7-2 shows the initial conditions of the specimens used for the soaking tests. Data relating to density, water content, loading and values of $K_0$ for all of the soaking tests are shown in Table 7-2.

Figs. 7-3 through 7-5 show the results of Test Y34, in which a specimen was compacted wet of optimum, loaded, and soaked. Fig. 7-3 shows the total stress path for the test. It seems likely that air bubbles in the soil voids were occluded in this specimen, which was compacted wet of optimum, and that the soil suction at the end of compaction was essentially zero. This is consistent with the fact that the total horizontal stress was zero after compaction. Values of total stress earth pressure coefficient, $K_0^T$, are close to unity during loading and unloading, as can be seen in Fig. 7-3. The specimen was soaked after unloading to a total vertical stress of 5 psi (OCR = 6.0).

Fig. 7-4 shows the progressive changes in the value of $K_0^T$ with time after the specimen was submerged. The value of $K_0^T$ first decreased from 0.99 to 0.60 and then increased gradually to 0.69. It seems logical that the first decrease in the value of $K_0^T$
Table 7-1. Conditions of Specimens for Soaking Tests

<table>
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<tr>
<th>Test No.</th>
<th>Initial Condition</th>
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<th>Condition after testing</th>
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<tr>
<td></td>
<td>Water content (%)</td>
<td>Dry unit weight (pcf)</td>
<td>Relative compaction (%)</td>
<td>Void ratio (%)</td>
<td>Degree of saturation (%)</td>
<td>Water content (%)</td>
<td>Dry unit weight (pcf)</td>
<td>Relative compaction (%)</td>
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<td>Y31</td>
<td>8.73</td>
<td>110.16</td>
<td>88.1</td>
<td>0.512</td>
<td>45.5</td>
<td>15.11</td>
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Fig. 7-2. Compaction Conditions of Soaking Test Specimens of Yatesville Silty Sand
Table 7-2. Results of Soaking Tests

<table>
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<tr>
<th>Test No.</th>
<th>Initial dry unit weight (pcf)</th>
<th>Initial water content (%)</th>
<th>$K_o$ during loading-unloading</th>
<th>OCR</th>
<th>1st Cycle</th>
<th>$K_o$ during soaking</th>
<th>OCR</th>
<th>$K_o$ at 1440 min</th>
<th>End of the test time (min)</th>
<th>Minimum $K_o$ time (min)</th>
<th>$K_o$</th>
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<td>Y31</td>
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<td>0.305</td>
<td>1.0</td>
<td>0.412</td>
<td>0.508</td>
<td>0.647</td>
<td>0.841</td>
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<td>6</td>
<td>0.500</td>
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<td>0.632</td>
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* The test ended at 1440 minutes after soaking.
Fig. 7-3. Total Stress Path Obtained from Soaking Test (Test Y34)
Vertical pressure was maintained at 5.0 psi.

$K_0^T$ prior to soaking = 0.99

$K_0 = K_0^T = 0.69$

Initial Condition of Specimen
water content = 16.04 %
dry unit weight = 112.74 pcf
relative compaction = 90.2% (Std. Proctor)
void ratio = 0.478
degree of saturation = 89.6 %

Fig. 7-4. Change in $K_0$ with Time Due to Soaking After Unloading (Test Y34)
Initial soil suction and initial pore pressure were assumed to be negligible.
The pore pressure prior to soaking was assumed to be 90% of total vertical stress.

Fig. 7-5. Effective Stress Path During Soaking (Test Y34)
resulted from dissipation of positive pore pressure, and that the subsequent increase was caused by stress relaxation.

Although pore pressures were not measured in the test, the effective stress behavior during the test can be estimated fairly well for soils wet of optimum. Based on the discussion in chapter 6, it can be inferred that the value of the pore pressure parameter $\bar{B}$ during primary loading for specimen Y34 was about 0.9. Consequently, the value of pore pressure at a vertical stress of 5 psi was probably about 4.5 psi. Assuming that the pore pressures dissipated completely during soaking, the effective stress path during soaking can be drawn, as shown in Fig. 7-5. The estimated effective stress path is slightly steeper than the $K_0$ line for primary loading. This can be explained by some amount of relaxation proceeding simultaneously with dissipation of the pore pressures.

Figs. 7-6 through 7-8 show the results of Test Y32, performed on a specimen compacted dry of optimum. Fig. 7-6 shows a total stress path for the test, and also the estimated position of effective stress axis. It seems likely that the air in the soil voids was continuous, and that the soil suction at the end of compaction was not zero. As a result, a significant amount of total horizontal stress was locked within the specimen at the end of compaction. The value of $K_0^T$ was high ($K_0^T = 1.63$) when the specimen was soaked after unloading to a vertical stress of 5 psi (OCR = 6.0). The value of $K_0^T$ decreased from 1.63 to 0.74 during soaking, as shown in Fig. 7-7. It seems likely that this reduction in the value of $K_0^T$ resulted from release of suction.

To estimate the effective stress path during soaking for a specimen with appreciable initial soil suction, it is necessary to know both the equivalent initial pore pressure and changes in pore pressure during loading and unloading. It is reasonable to assume that the initial pore air pressure is equal to zero for specimens with low initial degree of saturation because the air phase is likely to be continuous and connected to the atmosphere. The
Fig. 7.6. Stress Path Obtained from Soaking Test (Test Y32)
Fig. 7-7. Change in $K_0$ with Time Due to Soaking After Unloading (Test Y32)
Fig. 7-8. Effective Stress Path During Soaking (Test Y32)
initial soil suction, or the initial equivalent pore pressure, $u_a - \chi (u_a - u_w)$, can be estimated using VanGenuchten's moisture-capillary relationship, or Filz's (1992) method. Although these methods may be useful under some conditions, they are not considered to be accurate enough to provide good estimations of equivalent initial negative pore pressures for purposes of predicting effective stress paths during compaction, as discussed in chapter 6. For purposes of estimating the effective stress paths for the instrumented oedometer tests, it was assumed that the stress state after compaction and before loading was located on the $K_{lim}$ line, as shown in Fig. 7-6. The values of equivalent pore pressure estimated by this method represent upper limits (least negative values consistent with the data) and the real values could be more negative. However, the estimated position of the effective stress axis shown in Fig. 7-6 appears to be quite reasonable. In other cases, where the specimen is not unloaded far enough to reach the $K_{lim}$ line, the same assumption might not provide such reasonable results.

As discussed in chapter 3, changes in soil suction with changes in total stress for soils that are dry of optimum can be approximated to be zero over a fairly wide range of water content. As a result, change in the term related to soil suction, $\chi (u_a - u_w)$, would be negligible for soils significantly drier than optimum. Also, as discussed in chapter 6, values of the pore pressure parameter $\bar{B}$ for soils with degrees of saturation below 70 percent are very small. Furthermore, the values of pore air pressure are likely to remain essentially constant ($u_a = atmospheric$ pressure) during loading. Therefore, the offset between the total stress axis and the effective stress axis would remain constant during loading and unloading. It can be concluded that the effective stress axis estimated for the initial condition, shown in Fig. 7-6, should provide a reasonable estimate of effective stress before soaking for soils with initial degrees of saturation less than 70 percent, which have been unloaded sufficiently to reach the $K_{lim}$ line.
Fig. 7-8 shows the estimated effective stress path during soaking for Test Y32. The value of the effective stress $K_0$ decreased from 1.35 to 0.74. The value after soaking was still higher than the value of $K_0$ calculated by Jaky's equation, showing that the sample retained some compaction-induced lateral pressure after soaking.

Figs. 7-9 through 7-11 show the results of Test Y47. The specimen used for this test was compacted wet of optimum. The specimen was loaded to 30 psi vertical stress, and was soaked at that pressure. Fig. 7-9 shows the total stress path measured during the test, and the estimated position of the effective stress axis. As for Test Y34, shown in Fig. 7-3, the total horizontal stress was zero at the end of compaction, and values of total stress earth pressure coefficient, $K_0^\top$, were very close to unity during loading and unloading.

Fig. 7-10 shows the change in value of $K_0^\top$ with time during soaking. The values of $K_0^\top$ first decreased from 0.93 to 0.57 within about 8 hours, and thereafter increased slightly to 0.59 after 38 hours. It seems likely that the initial decrease in the values of $K_0^\top$ resulted from dissipation of positive pore pressure, and that the subsequent increase was caused by stress relaxation. The observed behavior was quite similar to the results of Y34, shown in Fig. 7-4, although the values of OCR when the specimens were soaked are different. It had been expected that the value of $K_0^\top$ might decrease to the value of $K_0$ calculated by Jaky's equation. However, the minimum value of $K_0^\top$ was higher, indicating a residual effect of earth pressures due to compaction.

It appears that the value of $\bar{B}$ for specimen Y47 was about 0.9. Consequently, the pore pressure at vertical stress of 30 psi would be about 27 psi. The effective stress axis for this condition is shown in Fig. 7-9. Assuming that the pore pressures dissipated completely during soaking, the effective stress path during soaking can be drawn, as
Fig. 7-9. Stress Path Obtained from Soaking Test (Test Y47)
Vertical pressure was maintained at 30 psi.

\[ K_0^T \text{ prior to soaking} = 0.93 \]

\[ K_0 = K_0^T = 59 \]

Initial Condition of Specimen
- water content = 14.93 %
- dry unit weight = 114.8 pcf
- relative compaction = 91.8 % (Std. Proctor)
- void ratio = 0.451
- degree of saturation = 88.3 %

Elapsed Time after Soaking (min)

Fig. 7-10. Change in $K_0$ with Time Due to Soaking (Test Y47)
Fig. 7-11. Effective Stress Path During Soaking (Test Y47)
shown in Fig. 7-11. The effective stress value of \( K_0 \) increased from 0.33 to 0.59 during soaking, as noted previously.

Figs. 7-12 through 7-14 show the results of Test Y34, in which a specimen was compacted dry of optimum, loaded to 30 psi, unloaded to 5 psi, reloaded to 30 psi, and soaked. Fig. 7-12 shows the total stress path for the test, and the estimated position of the effective stress axis. A significant amount of horizontal stress was locked within the specimen at the end of compaction. The value of \( K_0^T \) was 0.305 before the specimen was soaked at a vertical stress of 30 psi, and it decreased slightly to 0.291 during soaking.

Fig. 7-13 shows the change in the value of \( K_0^T \) with time during soaking. The values of \( K_0^T \) initially decreased from 0.305 to 0.277, and subsequently increased slightly to 0.291. It seems likely that the initial decrease in the values of \( K_0^T \) resulted from release of soil suction, and that the subsequent increase was caused by stress relaxation.

Fig. 7-14 shows the effective stress path during soaking for Test Y46. The offset between the effective stress axis and the total stress axis was determined in the same manner as for Test Y32, shown in Fig. 7-12. It is estimated that the effective stress ratio decreased from 0.37 to 0.28, which is very close to the active failure line, although the total stress ratio remained nearly constant during soaking.

7.4 Summary

To investigate the effects of soaking on compaction-induced earth pressures, contours of \( K_0^T \) after soaking were drawn on the moisture-density diagram for tests where the specimens were soaked at OCR=6, as shown in Fig. 7-15. Fig. 7-16 shows contours of changes in the values of \( K_0^T \) during soaking. From these figures, the following characteristics of the behavior can be noted:
SOAKING TESTS
Fig. 7-13. Change in $K_o$ with Time Due to Soaking (Test Y46)
Fig. 7-14. Effective Stress Path During Soaking (Test Y46)
Fig. 7-15. Total Stress At-rest Earth Pressure Coefficients Before and After Soaking
Fig. 7-16. Change in Total Stress At-rest Earth Pressure Coefficients Due to Soaking

Values next to symbols are changes in $k_0^T$ due to soaking.
(1) The values of $K_0^T$ decreased during soaking in all cases. Although other investigators have reported increased values of $K_0^T$ due to swelling, this is not the case for Yatesville silty sand, which has very low plasticity, and is not a swelling soil.

(2) The values of $K_0^T$ after soaking (which are also effective stress values of $K_0$) were between 0.7 and 0.8 for soils wet of optimum.

(3) For soils dry of optimum, the values of $K_0^T$ after soaking (effective stress values of $K_0$) increased with increasing values of dry unit weight. For dry soils with dry unit weights greater than 90 percent of the Standard Proctor maximum, the values of $K_0^T$ after soaking were greater than unity. It appears that the magnitude of the initial soil suction affects the values of $K_0^T$ after soaking for soils compacted dry of optimum.

(4) The decrease in the value of $K_0^T$ during soaking is small for samples compacted at water contents close to optimum and densities less than 90 % of the Standard Proctor maximum. As the dry density increases, or as the deviation from optimum water content increases (plus or minus) the change in the value of $K_0^T$ increases.

Fig. 7-17 illustrates estimated effective stress paths during soaking for various compaction conditions. As shown in the figure, the effective stress paths approach the active failure line due to soaking in some cases. When specimens are soaked after unloading, the effective stress values of $K_0$ decreased in all cases. For wet soils, the values of $K_0$ were found to be greater than the values calculated using Jaky's equation.
(a) Dry of optimum

(b) Wet of optimum

Fig. 7-17. Effective Stress Path During Soaking (Trends of Experimental Results)
CHAPTER 8

ANALYSES OF INSTRUMENTED RETAINING WALL TESTS

8.1 Introduction

Filz (1992) performed instrumented retaining wall tests using backfills of moist Yatesville silty sand. The tests were performed in the Virginia Tech instrumented retaining wall facility, which was designed and constructed by Sehn (1990). In this chapter, the results of these tests are compared to results calculated using the proposed methods for estimating compaction-induced pressures, which were described in Chapter 5 and Chapter 6.

8.2 Description of the Instrumented Retaining Wall Tests Conducted by Filz (1992)

The Virginia Tech retaining wall facility consists of a stiff reinforced concrete U-frame structure which supports a 10-foot long, 7-foot high instrumented wall. The instrumented wall is divided into four separate panels, each 2.5 ft long by 7 ft high. The panels are supported by load cells reacting against the U-frame structure. Backfill was placed and compacted to a depth of 6.5 feet against the wall.

Filz (1992) performed 14 tests using Yatesville silty sand backfill. Compactor forces, which are required as input data for analyses of compaction-induced earth pressures, were measured in three tests (EP12, EP13, and EP14). The results of these tests are analyzed in following sections.

The conditions of tests EP12, EP13, and EP14 are summarized in Table 8-1. The batch of Yatesville silty sand used for tests EP13 and EP14 (batch YSS2) was same as that used for the instrumented oedometer tests. The material used for test EP12 (YSS1) had moisture-density characteristics which were slightly different from those of batch YSS2. In test EP12, the backfill was compacted by 5 passes of a vibrating plate.
compactor with a frequency of 98Hz. In tests EP13 and EP14, the backfill was compacted by 2 passes of a rammer compactor with a frequency of 10.6Hz, after having been compacted by 2 passes of the same vibrating compactor as used for EP12. The measured impact force of the rammer compactor was much larger than that of the vibrating compactor. Therefore, in the analyses of EP13 and EP14, only the compaction by the rammer compactor was simulated.

8.3 Total Stress Analyses

The total stress analyses described in this section were performed following the concepts discussed in section 5-4. The procedures for the total stress analyses were quite simple, as described below:

1. Values of $K_{0,NC}$ were determined from Fig. 5-15, and values of $\alpha$ were determined from Fig. 5-20;

2. Values of virtual friction angle were calculated as $\phi = \sin^{-1} (1 - K_0)$; and

3. Those values were applied to the effective stress $K_0$-model developed by Duncan and Seed (1986) and modified by Filz (1992).

The values of the parameters used for the total stress analyses are shown in Table 8-2. The sources of these values are indicated in the table.

Fig. 8-1 shows the results of the total stress analyses. For all three cases, the calculated lateral pressure distributions are in good agreement with the experimental results. Consequently, it appears that the instrumented oedometer test is suitable for estimating compaction-induced lateral earth pressures, and that it provides a basis for accurate estimates of compaction-induced lateral earth pressures.
Table 8-1. Conditions of Instrumented Retaining Wall Tests

<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td>Soil Type</td>
<td>YSS1</td>
<td>YSS2</td>
<td>YSS2</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>110 pcf</td>
<td>120 pcf</td>
<td>119 pcf</td>
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<tr>
<td>Relative Compaction</td>
<td>88 %</td>
<td>96 %</td>
<td>95 %</td>
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<tr>
<td>Moist Unit Weight</td>
<td>124 pcf</td>
<td>135 pcf</td>
<td>131 pcf</td>
</tr>
<tr>
<td>Compaction Water Content</td>
<td>12.3 %</td>
<td>12.7 %</td>
<td>10.1 %</td>
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<tr>
<td>Degree of Saturation</td>
<td>65 %</td>
<td>87 %</td>
<td>67 %</td>
</tr>
<tr>
<td>Compactor Type</td>
<td>Wacker BPU2440A, Vibrating Plate</td>
<td>Wacker BS60Y, Rammer</td>
<td>Wacker BS60Y, Rammer</td>
</tr>
<tr>
<td>Compactor Width</td>
<td>15 in</td>
<td>10.6 in</td>
<td>10.6 in</td>
</tr>
<tr>
<td>Compactor Length</td>
<td>15 in</td>
<td>7 in</td>
<td>7 in</td>
</tr>
<tr>
<td>Distance from Edge of Compactor to Wall</td>
<td>1 in</td>
<td>1 in</td>
<td>1 in</td>
</tr>
<tr>
<td>Total Compactor Force</td>
<td>1210 lbs</td>
<td>5040 lbs</td>
<td>7330 lbs</td>
</tr>
<tr>
<td>Lift Thickness</td>
<td>6 in</td>
<td>6 in</td>
<td>6 in</td>
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</tbody>
</table>
Table 8-2. Soil Properties Used in Total Stress Analyses of Instrumented Retaining Wall Tests

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Moist Unit Weight</td>
<td>124 pcf</td>
<td>135 pcf</td>
<td>131 pcf</td>
</tr>
<tr>
<td>$K_0^T$ (Fig. 5-15)</td>
<td>0.38</td>
<td>0.88</td>
<td>0.39</td>
</tr>
<tr>
<td>Effective Stress Friction Angle [$\sin'(1-K_0^T)$]</td>
<td>38.3°</td>
<td>6.9°</td>
<td>37.6°</td>
</tr>
<tr>
<td>Effective Stress Cohesion Intercept</td>
<td>0 psf</td>
<td>0 psf</td>
<td>0 psf</td>
</tr>
<tr>
<td>Unloading exponent $\alpha$ (Fig. 5-20)</td>
<td>0.35</td>
<td>0.09</td>
<td>0.68</td>
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</tbody>
</table>

ANALYSES OF RETAINING WALL TESTS 234
Fig. 8-1. Results of Total Stress Analyses for Instrumented Retaining Wall Tests
8.4 Effective Stress Analyses

Effective stress analyses were performed using the concepts discussed in Chapter 6. The analyses used the proposed pore pressure response model, and the effective stress $K_0$-model developed by Duncan and Seed (1986) and modified by Filz (1992). In the analyses, changes in vertical stress were divided into small increments, and the changes in pore pressure and effective vertical stress were calculated iteratively. Subsequently, the calculated changes in effective vertical stress were applied to the effective stress $K_0$-model.

The effective stress analyses do not require results of instrumented oedometer tests. They are based on values of effective and total stress strength parameters, and hyperbolic stress-strain parameters. The information required for the analysis can be obtained from conventional triaxial tests. Values of the parameters used for the effective stress analyses are listed in Table 8-3.

Fig. 8-2 shows the results of the effective stress analyses. It can be seen that the analyses provided good agreement with the experimental results for tests EP12 and EP13, but not for test EP14.

Figs. 8-3 and 8-4 show the stress paths for EP14 estimated by the total stress analysis and the effective stress analysis. The stress path shown in Fig. 8-3, for the total stress analysis, is governed by the limiting line. The lateral earth pressures corresponding to this stress path are in good agreement with the experimental results. The stress path shown in Fig. 8-4, for the effective stress analysis, is not controlled by a limiting line. The lateral earth pressures at low values of vertical stress are much higher than those calculated by the total stress analyses, and much higher than the experimental results. It thus seemed likely that imposing the total stress limiting line in the effective stress analyses would improve the accuracy of the results. Fig. 8-5 shows the stress paths estimated by
**Table 8-3. Soil Properties Used in Effective Stress Analyses**
for Instrumented Retaining Wall Tests

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>Soil Type</td>
<td>YSS1</td>
<td>YSS2</td>
<td>YSS2</td>
</tr>
<tr>
<td>Dry Unit Weight</td>
<td>110 pcf</td>
<td>120 pcf</td>
<td>119 pcf</td>
</tr>
<tr>
<td>Moist Unit Weight</td>
<td>124 pcf</td>
<td>135 pcf</td>
<td>131 pcf</td>
</tr>
<tr>
<td>Effective Stress Friction Angle (Fig. F-3)</td>
<td>34.5°</td>
<td>37.3°</td>
<td>37.0°</td>
</tr>
<tr>
<td>Effective Stress Cohesion Intercept</td>
<td>0 psf</td>
<td>0 psf</td>
<td>0 psf</td>
</tr>
<tr>
<td>$K_0 \ (= 1 - \sin \phi)$</td>
<td>0.438</td>
<td>0.391</td>
<td>0.395</td>
</tr>
<tr>
<td>Total Stress Friction Angle (Fig. F-12)</td>
<td>18°</td>
<td>19°</td>
<td>32°</td>
</tr>
<tr>
<td>Total Stress Cohesion Intercept (Fig. F-12)</td>
<td>1200 psf</td>
<td>1630 psf</td>
<td>1670 psf</td>
</tr>
<tr>
<td>Modulus Number (Fig. F-13)</td>
<td>55</td>
<td>18</td>
<td>250</td>
</tr>
<tr>
<td>Modulus Exponent (Fig. F-14)</td>
<td>0.37</td>
<td>0.41</td>
<td>0.12</td>
</tr>
<tr>
<td>Initial Total Horizontal Stress (Fig. 6-12)</td>
<td>40 psf</td>
<td>14 psf</td>
<td>300 psf</td>
</tr>
</tbody>
</table>
Fig. 8-2. Results of Effective Stress Analyses for Instrumented Retaining Wall Tests
Fig. 8.3. Total Stress Path for EP14 Estimated by Total Stress Analysis.
Fig. 8-4. Stress Paths for EP14 Estimated by Effective Stress Analysis
Fig. 8-5. Stress Paths for EP14 Estimated by Effective Stress Analysis Imposing Limiting Line
the effective stress analysis using a $K_{\text{lim}}^T$-line with a slope of 2.5, which corresponds to
zontal stress distribution calculated in these analyses is shown in Fig. 8-6. It can be seen
that imposing the $K_{\text{lim}}^T$-line resulted in considerably better agreement between the
calculated and measured results for EP14. The estimated results for EP12 and EP13 were
not affected.

It appears that the proposed method of effective stress analysis of compaction-
induced earth pressures is useful, because these analyses are based on properties that can
be determined from conventional tests. Further research will be required to provide a
basis for understanding why the total stress limit line is needed in these effective stress
analyses. Also, it seems possible that using the effective stress analyses for soils with
water contents further dry of optimum could be inaccurate, because the assumptions
employed in the effective stress analysis (constant soil suction and undrained conditions)
may not be appropriate for very low water contents. However, the water contents of soils
compacted fills are usually not more than a few percent above or below optimum, and the
assumptions appear to be valid for those conditions.
Fig. 8-6. Results of Effective Stress Analyses Imposing $K_{\text{lim}}$ line in Total Stress Space
CHAPTER 9

CONCLUSION

The purpose of this research was to investigate at-rest and compaction-induced lateral pressures exerted in partly saturated compacted soils, in which pore pressures may affect behavior. The studies performed provide the basis for the following conclusions:

Reviewing previous studies of at-rest and compaction-induced lateral pressures provided useful information for this research. Theories of at-rest and compaction-induced pressures in dry soils are well-developed and reliable. The effective stress $K_0$ model developed by Duncan and Seed (1986), and modified by Filz (1992), has been successfully applied to dry sands. No model that can be used to estimate compaction-induced earth pressures in partly saturated soils has been presented previously. Based on previous studies, it is clear that pore pressures and soil suction affect the at-rest earth pressures in moist soils significantly.

Previous studies of the behavior of partly saturated soils also provided useful information for this research. It has proven to be very difficult to measure negative pore pressures, and to relate them quantitatively to strength and deformation properties. Soil suction measurements are a subject of continuing research at the present time, and are not used to any significant extent in engineering practice. It is important that a practical procedure for estimating earth pressures in moist (partly saturated) soils should not require measurement of soil suction. Skempton's (1954) concept of pore pressure parameters can be applied effectively to partly saturated soils, and Bishop's (1959) expression of effective stress in partly saturated soils is sufficiently accurate for most practical purposes.
The instrumented oedometer which was designed and constructed during this study is useful for investigating at-rest and compaction-induced pressures. The device is a splitting oedometer with very stiff load cells. Lateral strains in the device are sufficiently small for accurate-measurement of at-rest pressures. Uniform vertical stresses were applied to the specimens using air pressure applied through a rubber membrane. A computer-base data acquisition and test control system allows rapid cyclic loading tests to be performed. The results of tests performed using the device were found to be reproducible, and they appear to be quite accurate.

Instrumented oedometer tests on Monterey sand were good agreement with Jaky's (1944) equation for primary loading. Measured values of $K_o$ decreased with the number of loading cycles, consistent with the findings of Sehn (1990). The measured stress paths were similar to those reported by Wright (1969) and Sehn (1990). The stress paths for dense specimens were found to be more hysteretic than those for loose specimens. The effective stress model proposed by Duncan and Seed (1986), and modified by Filz (1992), provided good agreement with the measured stress paths, provided the value of the unloading exponent $\alpha$ was selected appropriately.

The instrumented oedometer tests performed on specimens of partly saturated Yatesville silty sand provided much improved understanding of the factors that control at-rest and compaction-induced earth pressures in partly saturated soils:

- For specimens with very low degrees of saturation, measured values of $K_{0,NC}^T$ (the total stress coefficient of earth pressure at rest) were smaller than the value of $K_o$ estimated using Jaky's equation and did not change appreciably with number of loading cycles. The measured total stress paths exhibited a significant amount of hysteresis.
- For specimens with initial water content slightly dry of optimum, the measured values of $K_{0,NC}^T$ decreased continuously with increasing number of loading cycles. The stress paths exhibited marked hysteresis.

- For specimens with water contents near optimum, values of $K_{0,NC}^T$ were much greater than would be expected if pore pressures were zero. The values of $K_{0,NC}^T$ increased slightly up to 100 cycles, and then decreased as additional cycles were applied. It appears that this behavior is caused by gradually increasing positive pore pressures, followed by dissipation during the period of approximately two hours required to apply 1000 cycles of loading.

- For specimens with initial water content wet of optimum, values of $K_{0,NC}^T$ were very close to one, and did not change appreciably with the number of loading cycles during the first 100 cycles. The stress paths for this condition were nearly non-hysteretic.

The results of the instrumented oedometer tests showed that the values of $K_{0,NC}^T$ increase with degree of saturation, and that most of the change occurred at degrees of saturation between 70 and 85 percent. The values of $K_{0,NC}^T$ were slightly affected by relative compaction.

A method of total stress analysis was proposed based on the results of the instrumented oedometer tests. Diagrams and equations were presented for estimating the most probable values of $K_{0,NC}^T$. It was shown that values of $K_{0,OC}^T$ can be expressed in the same form as used for expressing effective stress $K_{0,OC}$. Values of unloading exponent $\alpha$ vary with degree of saturation and relative compaction. A diagram for estimating the most probable values of $\alpha$ for Yatesville silty sand was developed.

Accurate estimation of values of $K_0^T$ is quite difficult, because they change rapidly with degree of saturation around optimum water content. Consequently, design diagrams for estimating values of $K_{0,NC}^T$ and $\alpha$ were presented that can be used to estimate upper-
bound values of $K_{0,NC}^T$. It seems likely that these design diagrams can be used to estimate design values of $K_0$ for Yateville silty sand and other partly saturated compacted soils as well.

The pore pressure parameter $\overline{B}$ was found to be useful for estimating pore pressures during one-dimensional loading and unloading of partly saturated specimens. Pore pressure parameters for partly saturated soils can be calculated using theoretical relationships developed by Fredlund and Rahardjo (1993). The theoretical equation relates values of $\overline{B}$ to soil modulus and degree of saturation. The modulus was expressed as a function of all-around stress and deviator stress, using the hyperbolic model. Values of $\overline{B}$ calculated using the theoretical relationship were employed in Filz's effective stress $K_0$ model. This method was found to provide good agreement with the test results for primary loading, and to be useful for estimating at-rest earth pressures in partly saturated soils. Appreciable discrepancies between measured and calculated values of $K_0$ were found for unloading. It appears that these discrepancies may be related to hysteretic stress-strain behavior, or to changes in soil suction during unloading.

Soaking tests were performed on partly saturated specimens of Yatesville silty sand to investigate the effects of changes in moisture content. Specimens were soaked under load at OCR = 1 or OCR = 6. Values of $K_0^T$ were found to decrease due to soaking at all values of compaction water content. Effective stress paths were inferred from the results. The values of effective stress $K_0$ after soaking were close to the active earth pressure coefficient $K_a$ in some cases.

Three of the instrumented retaining wall tests performed by Filz (1992) were analyzed using the proposed total stress and effective stress analysis methods. The proposed total stress analysis procedure provided good agreement with the measured lateral earth pressures. The proposed effective stress analysis procedure provided good
agreement with the results of two tests, but not the third. Imposing a limiting line in total stress space improved the agreement considerably.

Based on the findings of this study, it appears that the following additional studies would be useful:

(1) Instrumented oedometer tests using higher values of vertical stress, and modification of the apparatus for this purpose;

(2) Instrumented oedometer tests with pore pressure measurement;

(3) Instrumented oedometer tests on other soils with significant fines contents, and on cohesive soils,

(4) Studies to clarify the mechanism of the limiting total stress lateral earth pressure line; and

(5) Evaluation of the reasons that the values of $K_o$ in Yatesville silty sand after soaking are very low, approximately equal to $K_a$. 
REFERENCES


REFERENCES

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REFERENCES


REFERENCES


Jennings, J. E. B., and Burland, J. B. (1962). "Limitation to the use of effective stresses in partly saturated soils." Geotechnique, 125-144.


REFERENCES


Appendix A

DATA ACQUISITION AND TEST CONTROL SYSTEM

Data Acquisition System

Outline

The data acquisition system used for the instrumented oedometer tests is described in this appendix. Accurate measurements of lateral and vertical pressure were required for the instrumented oedometer tests. Therefore, an advanced data acquisition system was used. This system is shown in Fig. 4-7, and a list of the components of the system is contained in Table A-1. The data acquisition system is controlled by means of a computer program "ACQOED" coded in Microsoft QuickBASIC 4.5. The program "ACQOED" is used for both data acquisition and loading control. It is described in Appendix B.

Instrumentation

Load Cell

Two low profile load buttons, LB-1K, manufactured by Transducer Techniques were used for measuring lateral loads in the instrumented oedometer. The LB-1K load cell is designed for compression-only applications. The LB-1K load cell is made from heat treated 17-4 ph stainless steel. A connection to a cable is sealed for use in industrial environment. However, this load cell is not submergeable because the cable connection is not welded. The characteristics of the load cell are shown in Table A-2.

Pressure Transducer

Pressure transducers, Type P723 manufactured by Schaevitz Engineering were used to measure the vertical pressure and the pore pressure. All parts of the transducer are welded, and it offers an environmentally immune measurement. The characteristics of the pressure transducer are shown in Table A-3.
Table A-1. Instruments Used in the Data Acquisition System 
and the Test Control System

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Manufacturer</th>
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</thead>
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<tr>
<td>Load cells</td>
<td>LB-1K (1000 lbs capacity)</td>
<td>Transducer INC.</td>
</tr>
<tr>
<td></td>
<td>Low profile load button</td>
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</tr>
<tr>
<td>Pressure transducers</td>
<td>Type P723 (150 psi capacity)</td>
<td>Schaevitz</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Engineering</td>
</tr>
<tr>
<td>Excitation power supply</td>
<td>PWR-51A (+5VDC, 1000mA)</td>
<td></td>
</tr>
<tr>
<td>Signal conditioning module</td>
<td>MB38-02 (for load cell measurement)</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td></td>
<td>MB38-05 (for pressure transducer measurement)</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td></td>
<td>MB01 (board for MB38)</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td>Cables</td>
<td>C20MB1</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td></td>
<td>CDAS-2000</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td>Adapter</td>
<td>ADP5026</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td>Data acquisition board</td>
<td>DAS-HRES</td>
<td>Keithly MetraByte</td>
</tr>
<tr>
<td>Digital pressure regulator</td>
<td>PAR-15 (30 psi capacity)</td>
<td>Schrader Bellows</td>
</tr>
<tr>
<td></td>
<td>PAR-15 (150 psi capacity)</td>
<td>Schrader Bellows</td>
</tr>
<tr>
<td>Pressure regulator</td>
<td>150 psi capacity</td>
<td>Fairchild</td>
</tr>
<tr>
<td>Signal interface</td>
<td></td>
<td>Custom made</td>
</tr>
<tr>
<td>Microcomputer</td>
<td>PC compatible, 12MHz</td>
<td>EPSON</td>
</tr>
</tbody>
</table>

Schaevitz Engineering  
U.S. Route 130 & Union Ave., Pennsauken, NJ  
Tel: (609) 662-8000  
(Mail Address) P.O. Box 505, Camden, NJ 08101

Transducer Techniques  
43178 Business Park Drive, B101, Temecula, CA 92590  
Tel: (714) 676-3965, Fax: (714) 676-1200

Keithly MetraByte  
440 Myles Standish Blvd., Tauton, MA 02780  
Tel: (508) 880-3000, Fax: (508) 880-0179

Fairchild  
1501 Fairchild Drive, Winston-Salem, N.C. 27105  
Tel: (919) 767-6010

Schrader Bellows  
200 West Exchange Street, Akron, OH 44309-0631  
Tel: (216) 923-5202
Table A-2. Characteristics of Load Cells

<table>
<thead>
<tr>
<th>Model Name</th>
<th>LB-1K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity</td>
<td>1,000 lbs</td>
</tr>
</tbody>
</table>
| Dimension      | diameter = 1.25 inches  
|                | height = 0.40 inches    |
| Rigidity       | greater than $1.0 \times 10^6$ lbs/inches |
| Rated Output (R.O.) | 2 mV/ V    |
| Excitation Voltage | 10 VDC    |
| Natural Frequency | 32,000 Hz   |
| Nonlinearity   | 2.0 lbs for full scale |
| Hysteresis     | 0.7 lbs for full scale |
| Nonrepeatability | 0.1 lbs for full scale |
| Temperature Effect on Zero | 0.1 lbs/°F |
| Temperature Effect on Output | 0.005 % of Load/°F |

(1) The values in this table are given in the manufacturer's catalog.
Table A-3. Characteristics of Pressure Transducer

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Model Name</strong></td>
<td>P723</td>
</tr>
<tr>
<td><strong>Capacity</strong></td>
<td>150 psi</td>
</tr>
<tr>
<td><strong>Dimension</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>diameter = 1.22 inches</td>
</tr>
<tr>
<td></td>
<td>length = 2.83 inches</td>
</tr>
<tr>
<td><strong>Combined Nonlinearity, Hysteresis, and Nonrepeatability</strong></td>
<td>0.75 psi for full scale</td>
</tr>
<tr>
<td><strong>Rated Output (R.O.)</strong></td>
<td>2.5 mV/ V</td>
</tr>
<tr>
<td><strong>Excitation Voltage</strong></td>
<td>10 VDC</td>
</tr>
<tr>
<td><strong>Temperature Effect on Zero</strong></td>
<td>0.0225 psi/°F</td>
</tr>
<tr>
<td><strong>Temperature Effect on Output</strong></td>
<td>0.0225 psi/°F</td>
</tr>
</tbody>
</table>

(1) The values in this table are given in the manufacture's catalog.
Data Acquisition

A multi-function, 16-bit resolution analog and digital I/O board (DAS-HRES) was used for the data acquisition system. The DAS-HRES was directly plugged into an I/O slot of the microcomputer (EPSON-286 12MHz), and connected to the signal conditioning module outside the computer.

The DAS-HRES divides the full-scale voltage into $2^{16}(=65,536)$ segments, and the resolution is from 19.07 microvolts/bit for 0 to 1.25V input, to 305 microvolts/bit for $\pm$ 10 V input). The full scale input range to the A/D board should match the full scale input signal from the signal conditioning module. The differential input configuration used in the DAS-HRES reduced the electrical noise caused by taking data from several devices and by the difference of ground conditions between the signal sources and the computer. The DAS-HRES offers a maximum sampling rate of 47,600 per second, using a high quality sampling converter with a 17.25 microsecond conversion time and a 3.75 microsecond acquisition time. The performance of the data acquisition board would not be an obstacle to rapid-cyclic testing.

Signal Conditioning

Load cells with high capacity (1,000 lbs) were used to assure at-rest conditions during the tests, although the load to be measured by each load cell was not more than about 200 lbs. Consequently, the signals from the load cells were too small to be measured with a data acquisition card directly, and the ratio of electrical noise to signal was quite large. Signals from the signal sources were amplified and modified, so that the A/D conversion could perform accurately.

There are two ways to perform signal conditioning using hardware. First, signal conditioning can be accomplished by means of the input gain on the A/D board. The
DAS-HRES offers two options with regard to input gain: (1) switch selectable input gain, and (2) software programmable input gain. In this study, the input gain was selected by the software program, since that allowed independent gain for each channel. However, the DAS-HRES is limited to input gains less than or equal 8, much less than required.

Additional signal conditioning was achieved using an external signal conditioning modules (MB38-05 and MB38-02). These modules were powered by a 5 volt supply providing +10 V excitation to full bridge transducers. The transducers (the load cells and the pressure transducer) output 2 mV/V and 3 mV/V signals. The MB38-05 amplified the signal from the load cells by a factor of 250, and the MB38-02 amplified the signal from the pressure transducers by a factor of 167. The MB38 modules provided up to 1,500 Vrms of electrical isolation from input to output as well as 240 Vrms isolation from input to ground. Isolating the signals was very effective in eliminating ground loops. Isolation reduced excessive ground noise that is often seen in systems operating at high gain. The accuracy of the measurement was significantly improved by isolating the signals from the noisy ground. The MB38 modules also have signal filtering, with a cut-off frequency equal to 10 kHz. Table A-4 shows how the signals were amplified in the data acquisition system, and the corresponding resolutions.

A considerable amount of electrical noise was still measured, even after the signals were conditioned. Taking a large number of samples in a short period with the data acquisition board, and calculating a mathematical average of the data was found to reduce electrical noise significantly. Two hundred readings were averaged to reduce the electrical noise. Fig. A-1 is a histogram of load cell readings under constant load. This figure shows that the scatter due to the electrical noise is only ±0.05 lbs, and that 95 percent of the readings were within ±0.03 lbs. Fig. A-2 is a histogram of pressure readings under constant pressure. It was found that the accuracy of pressure readings was ±0.004 psi,
Table A-4. Signal Conditioning and Resolution for $\sigma_V = 70$ psi and $\sigma_H = 30$ psi

<table>
<thead>
<tr>
<th>Signal source</th>
<th>Signal from signal source</th>
<th>Gain in signal MB38</th>
<th>Signal from MB38</th>
<th>Gain in DAS-HRES</th>
<th>Signal for A/D converter</th>
<th>Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load cell</td>
<td>0 to 1.8 mV</td>
<td>250</td>
<td>0 to 0.45 V</td>
<td>8</td>
<td>0 to 3.6 V</td>
<td>0.0038 lbs</td>
</tr>
<tr>
<td>Pressure transducer</td>
<td>0 to 11.67 mV</td>
<td>167</td>
<td>0 to 1.95 V</td>
<td>4</td>
<td>0 to 7.79 V</td>
<td>0.0014 psi</td>
</tr>
</tbody>
</table>
Fig. A-1. Accuracy of Load Cell Measurement with Respect to Electrical Noise
Fig. A-2. Accuracy of Pressure Measurement with Respect to Electrical Noise
and that 95 percent of the readings were within ±0.003 psi. These variations are small enough to provide acceptable accuracy for the tests.

**Calibration**

The load cell LB-1K is quite sensitive to the texture of the surface against which it reacts and the shear force acting on the load cell. To standardize these factors, the load cells were calibrated in place. The instrumented oedometer apparatus was set upright, and dead weight loads were to apply forces to the split half of the oedometer. This method provided consistent and reproducible readings.

Two calibrations, one for a low load range (0 to 70 lbs) and another for a higher load range (0 to 180 lbs), were performed. Fig. A-3 shows a linear regression line and residuals for the high load-range calibration. The maximum residual was found to be only ±0.34 lbs which corresponds to a lateral pressure of 0.057 psi. This residual was considered to be the combined result of electrical noise, nonlinearity, nonrepeatability, and hysteresis. This value is much smaller than the value from the manufacturer's catalog. The reason for this may be the fact that the values given by the manufacturer are obtained from the full-scale calibration, and only 20 percent of the full scale was used in this study.

The pressure transducer was calibrated with an air pressure gauge by increasing and decreasing the pressure several times. For the range of pressure from 0 to 30 psi, a linear regression provided accurate results, with residuals of only ±0.05 psi, as shown in Fig A-4. For the range of pressure from 0 to 70 psi, the maximum residual for a linear regression was ±0.13 psi, as shown in Fig. A-5. Therefore, a third order polynomial nonlinear function was used. Fig. A-6 shows the nonlinear regression curve and corresponding residuals. As shown in Fig. A-6, the residuals for the nonlinear regression were only
Fig. A-3. Calibration of Load Cells for the High Load Range (0 - 180 lbs)
(a) Plot of Residuals, (b) Regression Line
Fig. A-4. Calibration of the Pressure Transducer for the Low Pressure Range (0 - 30 psi)
(a) Plot of Residuals, (b) Regression Line
Fig. A-5. Calibration of the Pressure Transducer for the High Pressure Range (0 - 70 psi)
(a) Plot of Residuals, (b) Regression Line
Fig A-6: Calibration of the pressure transducer for the high pressure range (0-70 psi) using nonlinear regression (a) Plot of Residuals, (b) Regression line
± 0.05 psi. The accuracy of the pressure transducer was also found to be better than the data provided in the manufacturer's catalog.

**Accuracy of the System**

The accuracy of the measurement was evaluated in terms of the error in measured values of $K_0$, by means of the following equation:

$$
K_{0,\text{err}} = \left( \frac{\sigma_{h,\text{max}} - \sigma_{h,\text{ave}}}{\sigma_{v,\text{min}}} \right) \frac{\sigma_{h,\text{ave}}}{\sigma_{v,\text{ave}}} 
$$  \hspace{1cm} (A-1)

Using this equation, normalized errors resulting from electrical noise were calculated. These are shown in Fig. A-7. It was found that the error due to electrical noise is significant only when the vertical pressure is less than about 1.0 psi.

A similar plot for all errors combined is shown in Fig. A-8. The normalized error due to the combined error is appreciable for vertical pressures less than about 5 psi, especially for the low $K_0$ condition. Small values of $K_0$ in the low stress range are only observed in virgin loading; as a result, the system provides accurate results for all conditions except virgin loading in the low stress range.

**Test Control System**

The instrumented oedometer tests were conducted by applying vertical pressures and reading lateral forces on the split half of the oedometer. The loading system consists of a digital pressure regulator and a custom digital interface. The system is shown in Fig. 4-8. The test control system was operated by a computer program "ACQOED" which
Fig. A-7. Normalized Error of $K_0$ Due to Electrical Noise

$K_{0,\text{err}} = \left( \frac{\sigma_{h,\text{max}}}{\sigma_{v,\text{min}}} - \frac{\sigma_{h,\text{ave}}}{\sigma_{v,\text{ave}}} \right) / \left( \frac{\sigma_{h,\text{ave}}}{\sigma_{v,\text{ave}}} \right)$
Fig. A-8. Normalized Error of $K_0$ Due to Combined Error

$$K_{0,\text{err}} = \left( \frac{\sigma_{h,\text{max}}}{\sigma_{v,\text{min}}} - \frac{\sigma_{h,\text{ave}}}{\sigma_{v,\text{ave}}} \right) \left/ \left( \frac{\sigma_{h,\text{ave}}}{\sigma_{v,\text{ave}}} \right) \right.$$
was coded in Microsoft QuickBASIC 4.5. The program "ACQOED" is a conjunctive program for load control and data acquisition. It is described in Appendix B.

The pressure regulators used in the loading system were Schrader Bellows PAR-15 digital pressure regulators. Two of PAR-15s were used, one with a low-range spring for low pressures (0 to 30 psi) and one with a standard spring for high pressures (0 to 70 psi). A Fairchild pressure regulator (0 to 150 psi) was used to supply regulated pressure to the inlet port of the PAR-15s. Regulating the inlet pressure reduced the effects of inlet pressure fluctuation. The inlet pressure may be set between 15 psi and 30 psi for the low pressure regulator, and between 15 psi and 150 psi for the high pressure regulator. The PAR-15 pressure regulator PAR-15 divides this inlet pressure into fifteen segments, making the increments of output pressure equal to one-fifteenth of the inlet pressure.

The signals from the microcomputer were +5 VDC. The 120 VAC input signals needed by the PAR-15 were generated by an interface unit.

**Pressure Stabilization**

Some time is required for the vertical pressure readings and lateral load readings to stabilize after a signal is sent to the digital pressure regulator. This time lag is due to the response time of the instrumentation and the pressure regulator, and the time required for the volume of air to be pressurized. Although the pressure line was made as short as possible to reduce the air volume so that the pressure would stabilize quickly, a time lag still remained, and it had to be taken into account. Fig. A-9 (a) shows the relationship between the elapsed time and measured vertical pressure, when the vertical pressure was increased from 0 psi to 30 psi in one step. It was found that the vertical pressure did not stabilize until 0.7 seconds after the signal had been sent, although the manufacturer indicates the pressure should stabilize in less than 20 milliseconds. The variations of the
Fig. A-9. Stabilization of Values of Vertical Pressure, Horizontal Pressure and $K_0$
After Vertical Pressure is Increased from 0 to 30 psi
measured lateral pressure and $K_0$ with elapsed time are shown in Fig. A-9 (b) and (c). It can be seen that about 0.7 seconds is also required for these quantities to stabilize.

Fig. A-10 shows how the measurements stabilize with time for the unloading case. It can be seen that the measurement registered an over-reduction in vertical pressure initially, and about 2 to 3 seconds were required for stabilization. The final rise in $K_0$ value was due to a decrease in the vertical pressure, which appeared to have resulted from a fluctuation in supply pressure.

The similar plots for cases where the vertical pressures were increased and decreased by 2 psi. are shown in Figs A-11 and A-12. It can be seen that a period of about 3 seconds is required for the measurement to stabilize. Therefore, it was concluded that the vertical pressure and the lateral load should be measured at least 3 seconds after the pressure was changed. Each loading step must be longer than 3 seconds for taking an average of 200 data points.

**Loading Pattern**

Tests were performed using single cycle and multiple cycles of loading. A single cycle of loading was used for most of the preliminary tests on dry sand specimens. Low pressure tests on Yatesville silty sand with maximum vertical pressures of 30 psi. were usually 1,000 cycles or 100 cycles. Ten cycles of loading were applied in high pressure tests on Yatesville silty sand with maximum pressures of 70 psi. Because of the characteristics of the digital pressure regulator, the pressures were changed in steps.

Fig. A-13 shows the loading patterns schematically. Cyclic loading consisted of two types of slow and fast cycles. In tests with slow cycle loading, the vertical pressure was increased in increments of one-fifteenth of the supply pressure, and was subsequently decreased with the same increment. At least 3 seconds are required to obtain stabilized
Fig. A-10. Stabilization of Values of Vertical Pressure, Horizontal Pressure and $K_0$
After Vertical Pressure is Decreased from 30 to 5 psi
Fig. A-11. Stabilization of Values of Vertical Pressure, Horizontal Pressure and $K_0$
After Vertical Pressure is Increased from 0 to 30 psi in 15 Steps. Each Step is 4 sec.
Fig. A-12. Stabilization of Values of Vertical Pressure, Horizontal Pressure and $K_0$
After Vertical Pressure is Decreased from 30 to 5 psi in 12 Steps. Each Step is 4 sec.
Fig. A-13. Loading Pattern for the Instrumented Oedometer Tests
data, as described previously. Therefore, each loading step was maintained for 4 seconds, and readings were taken after 3.5 seconds. In tests with fast cycle loading, the vertical pressure was increased to the peak pressure in a single increment, and was decreased to the lowest pressure in the same way. The peak pressure and the lowest pressure were maintained for 4 seconds, and data were read for only these two pressure levels. The combination of slow and fast cycles in a typical test was followed by slow unloading:

- 1st cycle --- slow loading from zero to the peak pressure, to an intermediate pressure:
- 2nd, 3rd, 10th, 100th, and 1000th cycles --- slow cycles between the intermediate pressure and the peak pressure:
- 4th through 9th, 11th through 99th, and 101th through 999th cycles --- fast cycles between the intermediate pressure and the peak pressure:
- The last cycle was unloaded to zero unless a creep test was performed.
Appendix B

DATA ACQUISITION AND TEST CONTROL PROGRAM
FOR THE INSTRUMENTED OEDOMETER

Introduction

A computer program ACQOED was used to control and record data from the instrumented oedometer tests. ACQOED controls the vertical pressure and records data from the pressure transducer and the load cells during the test. ACQOED was modified from the program KOPAR developed by Sehn (1990). The changes were made in order to use a new data acquisition system, and also to change both the loading pattern and the data acquisition sequence.

ACQOED was written in Microsoft Quick Basic 4.5 for IBM PC compatible computers operating under DOS. The program was linked to the subprograms HRES and HRESPCF, which were provided by Kethley Metrabyte. These subprograms address the MetraByte data acquisition board DAS-HRES. The linked program was named ACQOED.EXE.

The operator starts the instrumented oedometer test by running ACQOED. The operator adjusts the inlet pressure to the PAR-15 pressure regulator by looking at the pressure readings shown on the monitor. The operator can set a loading sequence interactively. Once the test starts, the program acquires and stores the test data automatically.

Functions of Subroutines

1. ACQOED.BAS (main program)

The main program ACQOED calls the subroutines shown below. ACQOED displays prompts on the screen for input of the loading sequence data.
2. ALARM

This subprogram sounds an alarm signal to notify the operator when attention is needed for the test.

3. ANYKEY

This subprogram halts the program until the operator presses any other keys.

4. CALIBRAT

This subprogram applies vertical pressures equal to the inlet pressure for the digital pressure regulator. This pressure level is kept for a time period specified by input data.

5. HRES

The object module of this subprogram is provided by Keytly Metrabyte. This controls the A/D conversion and the built-in amplifier of the DAS-HRES card. The source code of this subprogram is not available.

6. ERRORCODES

This subprogram outputs error messages on the monitor, if an error is detected by the program.

7. FLOOPS

This subprogram controls fast loading-unloading cycles, which are specified in the main program.

8. INITDAS

This subprogram initializes the data acquisition DAS-HRES board.

9. LOOPS

This subprogram controls slow loading-unloading cycles, which are specified in the main program.
10. OUTFILE

This subprogram shows a message for the operator to input the name of the output file, and writes that name as a heading in the output file.

11. PAR

This subprogram sends control signals to the pressure regulator PAR-15.

12. PAUSE

Calling this subprogram halts the operation of the program for a period of time specified by the operator.

13. RANGECHECK

This subprogram checks whether the digital values coming from the data acquisition board DAS-HRES exceed the pre-defined limit values.

14. READDATA

This subprogram reads the output signals two hundred times for each time this subroutine is called. Prior to taking an average of these readings, each reading is checked by the subprogram RANGECHECK. Zero readings are subtracted from the averaged values, and then the averaged values are converted to engineering units. The converted values are written to the output file.

15. READZERO

This subprogram reads the initial output signals that are subtracted from the data read by the subroutine READDATA. Two hundred data points are averaged to determine the zero value.

16. SCANLIM

This subprogram specifies the upper and the lower limit of the channels to be scanned.
17. SETSUPPLY

This subprogram lets the operator adjust the supply pressure for the pressure regulator interactively. Two hundred readings are averaged prior to conversion to engineering units. Each reading is checked by calling the subprogram RANGECHECK.

Input Data

The following data are required for the operation of the instrumented oedometer control program. All the data are defined interactively.

1. heading:
2. gain code for each channel:
3. lower and upper scan limits:
4. loading patterns consisting of (a) number of cycles, (b) starting stress, (c) peak stress, (d) ending stress, and (e) period of cycles:
5. data for creep test consisting of (a) vertical stress level, (b) time period of creep test.

Listing of Program ACOOED

DECLARE SUB alarm()
DECLARE SUB anykey()
DECLARE SUB calibrat()
DECLARE SUB chgain()
DECLARE SUB creep (ct%, pc%)
DECLARE SUB errorcodes (FLAG%)
DECLARE SUB floops (n%, st%, pt%, nd%, tp!)
DECLARE SUB hres (MD%, BYVAL dummy%, FLAG%)
DECLARE SUB initias()
DECLARE SUB loops (n%, st%, pk%, nd%, tp!)
DECLARE SUB outfile()
DECLARE SUB par (%)
DECLARE SUB pause (ct!)
DECLARE SUB rangecheck (i%, bit&), flag2%)
DECLARE SUB readdata()
DECLARE SUB readzero()
DECLARE SUB scanlim()
DECLARE SUB setsupply()
DIM SHARED D%(0 TO 15), G%(0 TO 15)
DIM SHARED cat(0 TO 3), zero(0 TO 3), plast%, %
DIM nlowe%, peab%, ppeb%, peab%, fmax%, psf%, psa%
DIM pps(0), pes0%, mslowe%, pse%, ppsa%, ppea%, pes%
DIM psbl(20), psof(20), pset(20)
COMMON SHARED n%, nu%, tstart, m%
CLS
CALL par(0)
CALL outfile

******* Calibration Factors
cal(0) = .0031765
cal(1) = .0031765
cal(2) = .0013764
cal(3) = .0013764

CLS
CALL initdas
CALL chgain
CALL scanlim
CALL readzero
CALL setsupply
CALL readzero
CLS

INPUT "Press: cyclic test = 0, static test (calibration) = 1 ": ical
IF ical = 1 THEN
CALL calibrat
ELSE
LOCATE 5, 1
PRINT " Connect a pressure line, and keep valve close"
PRINT
PRINT " Press Any Key To Continue"
anykey
CLS

i% = 0
DO
i% = i% + 1
CLS
INPUT "Enter Number of Slow Cycles to Begin Test ": nslowb%(i%)
IF nslowb%(i%) > 0 THEN
INPUT "Pressure Start Setting (0 to 15)": pssb%(i%)
INPUT "Pressure Peak Setting (0 to 15)": ppsb%(i%)
INPUT "Pressure End Setting (0 to 15)": pesb%(i%)
INPUT "Period of Loading Cycles ": tpsh%(i%)
END IF
PRINT
INPUT "Enter Number of Fast Cycles During Test ": nfast%(i%)
IF nfast%(i%) > 0 THEN
INPUT "Pressure Start Setting (0 to 15)": pssf%(i%)
INPUT "Pressure Peak Setting (0 to 15)": ppsf%(i%)
INPUT "Pressure End Setting (0 to 15)": pesf%(i%)
INPUT "Period of Loading Cycles ": tpshf%(i%)
END IF
PRINT
INPUT "Enter Number of Slow Cycles to End Test ": nslove%(i%)
IF nslove%(i%) > 0 THEN
INPUT "Pressure Start Setting (0 to 15)": pssve%(i%)
INPUT "Pressure Peak Setting (0 to 15)": ppsve%(i%)
INPUT "Pressure End Setting (0 to 15)": pesve%(i%)
INPUT "Period of Loading Cycles ": tpshve%(i%)
END IF
PRINT
PRINT "Press M to Enter More Cycles OR B to Begin the Test"
DO
k$ = UCASE$(INKEY$)
LOOP UNTIL k$ = "M" OR k$ = "B"
CLS
LOOP UNTIL k$ = "B"
CALL readdata
INPUT "Creep Testing Time ": ctt
IF ctt <> 0 THEN
INPUT "Pressure for Creep Test (0 to 15)": pcr%
END IF
CALL readdata
plast% = 0
ncycles% = i%
tstart = TIMER
FOR %I = 1 TO nocycles%  
IF nslowe%%(%I) > 0 THEN  
CALL loops(nslowe%%(%I), psab%%(%I), ppab%%(%I), pesb%%(%I), tspb%%(%I))  
END IF  
IF nfast%%(%I) > 0 THEN  
CALL loops(nfast%%(%I), pssf%%(%I), ppaf%%(%I), pesf%%(%I), tspf%%(%I))  
END IF  
IF nslowe%%(%I) > 0 THEN  
CALL loops(nslowe%%(%I), pse%%(%I), ppse%%(%I), pes%%(%I), tspse%%(%I))  
END IF  
CLOSE #2  
IF %I < nocycles% THEN  
OPEN #5 FOR APPEND AS #2  
END IF  
NEXT %I  
IF c% > 0 THEN  
CALL creep(c%, pcr%)  
END IF  
CALL per(0)  
pause (60)  
OPEN #5 FOR APPEND AS #2  
CALL readdata  
CLOSE #2  
CLS  
END IF  
quit:  
END

SUB alarm
******************************************************************************
** SUBPROGRAM ALRM
******************************************************************************
SOUND 100, .5
SOUND 200, .5
SOUND 300, .5
SOUND 400, .5
SOUND 500, .5
SOUND 600, .5
SOUND 700, .5
SOUND 800, .5
SOUND 900, .5
SOUND 1000, .5
SOUND 1100, .5
SOUND 1200, .5
SOUND 1300, .5
SOUND 1400, .5
SOUND 1500, .5
SOUND 1600, .5
SOUND 1700, .5
pause .25
FOR %I = 1 TO 2
SOUND 1200, 3
SOUND 70, 6
pause .1
NEXT
END SUB

SUB anykey
******************************************************************************
** SUBPROGRAM ANYKEY
******************************************************************************
DO  
LOOP UNTIL INKEY$ <> " "  
A$ = UCASE$(INKEY$); IF A$ = "Q" THEN STOP  
END SUB

SUB calibrat
******************************************************************************
** SUBROUTINE CALIBRAT
******************************************************************************
DIM s(n% TO nu%), flag2%(n% TO nu%), bit&(n% TO nu%)  
INPUT "Calibration Time ": t0  
t3 = 0!

APPENDIX B
m% = 0
i1 = TIMER
par (15)
dt1 = 5!
i3 = TIMER
FOR i% = 0 TO 3
s(i%) = 0
flag2%(i%) = 0
NEXT i%
FOR n% = 1 TO 200
FOR i% = n% TO n%/2
MD% = 3
' mode 3 - do one A/D conversion
CALL hres(MD%, VARPTR(D%(0)), FLAG%
IF flag2%(i%) < 0 THEN PRINT "Error #": FLAG%; " in mode 3": STOP
bit&%(i%) = D%(0)
IF bit&%(i%) < 0 THEN bit&%(i%) = bit&%(i%) + 65536
CALL rangecheck(bit&%(i%), bit&%(0), flag2%(i%)
s(i%) = s(i%) + bit&%(i%) / 200!
NEXT i%
NEXT n%
IF flag2%(0) OR flag2%(1) OR flag2%(2) OR flag2%(3) THEN
FOR i% = 0 TO 3
IF flag2%(i%) THEN
PRINT USING "Channel # was out of range.", i%
ENDIF
NEXT i%
PRINT
PRINT "PRESS ANY KEY TO CONTINUE"
anykey
CLS
ELSE
i2 = TIMER - i1
m% = m% + 1
hp = ((s(0) - zero(0)) * cal(0) + (s(1) - zero(1)) * cal(1)) / 6!
v3 = (s(2) - zero(2)) * cal(2)
IF vp = 0 THEN np = .0001
Ko = hp / vp
PRINT #2, USING "### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ###
(s(0), s(1), s(2), hp, vp, Ko)
PRINT USING "### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ### ###
(s(0), s(1), s(2), hp, vp, Ko)
ENDIF
par (0)
END SUB

SUB chgain
*****************************************************************************
** SUBPROGRAM CHGAIN
*****************************************************************************
' Select Channel Gains with mode 21 command
*****************************************************************************
FOR i% = 0 TO 15
G%(i%) = 0
NEXT i%
G%(0) = 3
G%(1) = 3
G%(2) = 2
G%(3) = 3
FOR i% = 0 TO 3
PRINT USING "Default Gain of Channel ## = $$": i%; G%(i%)
NEXT i%
PRINT "Do You Want to Change Channel Gains? (Y or N)" DO
k$ = UCASE$(INKEYS)
LOOP UNTIL k$ = "Y" OR k$ = "N"
IF k$ = "Y" THEN
PRINT
INPUT "Gain Code of Channel 0 (default value = 3)", G%(0)
INPUT "Gain Code of Channel 1 (default value = 3)", G%(1)
INPUT "Gain Code of Channel 2 (default value = 3)", G%(2)
INPUT "Gain Code of Channel 3 (default value = 3)", G%(3)
*****************************************************************************
ELSE
END IF

MD% = 21
CALL hires(MD%, VARPTR(G%(0)), FLAG%)
IF FLAG% <> 0 THEN PRINT "Error #", FLAG%; " in mode 21": STOP
END SUB

SUB creep (ct%, por%)  
******************************************************************************
** SUBPROGRAM CREEP
******************************************************************************
OPEN 15 FOR APPEND AS #2
PRINT #2, "Start Creep Test"
PRINT "Start Creep Test"
m% = 1
start = TIMER
dt = 0!
readdata
CLOSE #2
DO
OPEN 15 FOR APPEND AS #2
pause (dt - 10!)  
par (pcr%)  
pause (10!)
readdata
dt = dt * 2!
IF dt > 900! THEN
dt = 900!
END IF
CLOSE #2
cm% = cm% * 1.4 + 3
IF m% = cm% THEN
EXIT DO
END IF
m% = m% + 1
LOOP
END SUB

SUB errorcodes (FLAG%)  
******************************************************************************
** SUBPROGRAM ERRORCODES
******************************************************************************
SELECT CASE FLAG%
CASE 1
CLS
LOCATE 20, 10
PRINT "Failure to initialize base address using mode 0"
END
CASE 2
CLS
LOCATE 20, 10
PRINT "Mode number out of range"
END
CASE 3
CLS
LOCATE 20, 10
PRINT "Base address out of range"
END
CASE 4
CLS
LOCATE 20, 10
PRINT "Scan limits out of range"
END
CASE 5
CLS
LOCATE 20, 10
PRINT "Multiplexer channel address out of range"
END
CASE 6
CLS
LOCATE 20, 10
PRINT "Analog to digital timeout (not good)"
END SELECT

APPENDIX B 290
END
CASE 7
CLS
LOCATE 20, 10
PRINT "Interrupt level out of range"
END
CASE ELSE
END SELECT
END SUB

SUB loops (n%, st%, pk%, nd%, tpl)
******************************************************************************
** SUBPROGRAM FLOOPS
******************************************************************************

CLS
T1 = TIMER
dt1 = tpf / 2!

FOR t% = 1 TO n%
m% = m% + 1
IF (t% = 1 AND st% <> plast%) OR (t% > 1 AND nd% <> st%) THEN
CALL par(st%)
CALL readdata
END IF
DO
LOOP UNTIL TIMER - t1 >= dt1
T1 = TIMER
CALL par(pk%)
DO
LOOP UNTIL TIMER - t1 >= dt1 - .5
readdata
DO
LOOP UNTIL TIMER - t1 >= dt1
T1 = TIMER
CALL par(nd%)
DO
LOOP UNTIL TIMER - t1 >= dt1 - .5
readdata
NEXT t%
plast% = nd%
CLS
END SUB

SUB initdas
******************************************************************************
** SUBPROGRAM INITDAS
******************************************************************************

' Initialize DAS-HRES with mode 0 command

OPEN "hres.cfg" FOR INPUT AS #1 'get base I/O address
INPUT #1, D%(0)
CLOSE #1

D%(1) = 7 'interrupt level
D%(2) = 3 'D.M.A. level
D%(3) = 2 'auto calibrate
FLAG% = 0 'error variable
MD% = 0 'mode 0 - initialize

CALL hres(MD%, VARPTR(D%(0)), FLAG%)
IF FLAG% <> 0 THEN PRINT "MODE 0 ERROR...": END 'Halt on error
END SUB

SUB loops (n%, st%, pk%, nd%, tpl)
******************************************************************************
** SUBPROGRAM LOOPS
******************************************************************************

CLS
T1 = TIMER
dt1 = tpf / (pk% - st%) / 2
dt1 = 600!

FOR t% = 1 TO n%
m% = m% + 1

APPENDIX B 291
IF (% = 1 AND st% <> plast%) OR (% > 1 AND st% <> st%) THEN
  DO
  LOOP UNTIL TIMER - t1 >= dt1
  t1 = TIMER
  CALL par(st%)
  DO
  LOOP UNTIL TIMER - t1 >= dt1 - .5
  CALL readdata
  END IF
  FOR j% = (st% + 1) TO pk% STEP 2
  DO
  LOOP UNTIL TIMER - t1 >= dt1
  t1 = TIMER
  CALL par(j%)
  IF j% = 14 THEN j% = j% - 1
  DO
  LOOP UNTIL TIMER - t1 >= dt1 - .5
  readdata
  NEXT j%
  FOR j% = (pk% - 1) TO nd% STEP -2
  DO
  LOOP UNTIL TIMER - t1 >= dt1
  t1 = TIMER
  CALL par(j%)
  IF nd% = 3 AND j% = 4 THEN j% = j% + 1
  DO
  LOOP UNTIL TIMER - t1 >= dt1 - .5
  readdata
  NEXT j%
  NEXT j%
  plast% = nd% 
CLS
END SUB

SUB outfle
*****************************************************************************
** SUBPROGRAM OUTFILE
*****************************************************************************
CLS
PRINT " ENTER NAME FOR OUTPUT FILE:"
INPUT " ", $ 
CLS
OPEN $ FOR OUTPUT AS #2
PRINT #2, " Cycle/Time/Channel/Channel/Channel/Lateral/Vertical/Ko"
PRINT #2, " No. / /0/1/2/Press./Press/
PRINT #2, " /min.)/bits)/bits)/bits)/bits)/bits)/bits)/bits)/bits)/bits)/bits)/bits)
PRINT #2, 
END SUB

SUB par (%)
*****************************************************************************
** SUBPROGRAM PAR
*****************************************************************************
OUT &H78, 1%
END Sub

SUB pause (dt)
*****************************************************************************
** SUBPROGRAM PAUSE
*****************************************************************************
t = TIMER
DO
  IF TIMER < t THEN t = TIMER - 86400!
  LOOP UNTIL TIMER - t >= dt
END SUB

SUB rangecheck (1%, bit&(), flag2%() )
*****************************************************************************
** SUBPROGRAM RANGECHECK
*****************************************************************************
IF bit&(1%) >= 65535 OR bit&(% < 0 THEN
  flag2%(1%) = 1
END IF
END SUB
SUB readdata

```vbp
DIM s(n% TO nu%), flag2%(n% TO nu%), reading(n% TO nu%), bit&(n% TO nu%)
DO
  FOR i% = n% TO nu%
    s(i%) = 0
    flag2%(i%) = 0
  NEXT i%
FOR n% = 1 TO 200
  FOR i% = n% TO nu%
    MD% = 3 'mode 3 - do one A/D conversion
    CALL hrs(MD%, VARPTR(D%(0)), FLAG%)
    IF FLAG% <> 0 THEN PRINT "Error #; FLAG%; " in mode 3": STOP
    bit&(i%) = D%(0)
    IF bit&(i%) < 0 THEN bit&(i%) = bit&(i%) + 65536
    CALL rangecheck(i%, bit&(i%), flag2%(i%))
    s(i%) = s(i%) + bit&(i%) / 200
  NEXT i%
NEXT n%
IF flag2%(0) OR flag2%(1) OR flag2%(2) OR flag2%(3) THEN
  FOR i% = 0 TO 3
    IF flag2%(i%) THEN
      PRINT USING " Channel # was out of range."; i%
      alarm
    END IF
  NEXT i%
  PRINT
  PRINT " PRESS ANY KEY TO CONTINUE"
  anykey
  CLS
ELSE
  t2 = (TIMER - tstart) / 60
  hp = ((s(0) - zero(0)) * cal(0) + (s(1) - zero(1)) * cal(1)) / 6!
  vp = s(2) - zero(2) * cal(2)
  IF vp < -0.0001 THEN vp = -0.0001
  Ko = hp / vp
  PRINT "2. USING ", m%, t2, s(0), s(1), s(2), hp, vp, Ko
  PRINT USING " "; m%, t2, s(0), s(1), s(2), hp, vp, Ko
EXIT DO
END IF
END LOOP
END SUB

SUB readzero

```
NEXT n%

IF flag2%(0) OR flag2%(1) OR flag2%(2) THEN
FOR i% = 0 TO 2
  IF flag2%(i%) THEN
    PRINT USING "* Channel # was out of range during zero readings,"; i%
  END IF
NEXT i%
PRINT
PRINT "PRESS ANY KEY TO CONTINUE"
anykey
CLS
ELSE
  EXIT DO
END IF
LOOP UNTIL UCASE$(INKEY$) = "Q"
CLS
END SUB

SUB scanlim
SUBPROGRAM SCANLIM

'--- Prompt for scan limits and set using mode 1 -------
'Scan limits wil' default to 0-7 (8 channel) or 0-15 (16 channel) if you
'skip this step
INPUT "Enter lower channel scan limit number : " ; n%
INPUT "Enter upper channel scan limit number : " ; nu%
PRINT D%(0) = n%
PRINT D%(1) = nu%
MOD% = 1
CALL hres(MD%, VARPTR(D%(0)), FLAG%)
IF FLAG% <> 0 THEN PRINT "Error #": FLAG%; " In setting scan limits": STOP
END SUB

SUB setsupply
SUBPROGRAM SETSUPPLY

DM s(n% TO nu%), reading(n% TO nu%), flag2%(n% TO nu%), bit&(n% TO nu%)

CLS
LOCATE 5, 1
PRINT * Connect Pressure Transducer to Digital Pressure Regulator*
PRINT
PRINT * PRESS ANY KEY TO CONTINUE*
anykey

LOCATE 15, 1
FOR i% = 0 TO 15
par (i%)
NEXT i%
DO
PRINT "Press C to Continue"
FOR i% = 0 TO 3
s(i%) = 0
flag2%(i%) = 0
NEXT i%
FOR n% = 1 TO 200
FOR i% = n% TO nu%
  'mode 3 - do one A/D conversion
  CALL hres(MD%, VARPTR(D%(0)), FLAG%)
  IF FLAG% <> 0 THEN PRINT "Error #: FLAG%: " in mode 3": STOP
  bit&(i%) = D%(i%)
  IF bit&(i%) < 0 THEN bit&(i%) = bit&(i%) + 65536
  CALL rangecheck(i%, bit&, flag2%) (s(i%) = s(i%) + bit&(i%) / 200!
NEXT i%
NEXT n%

IF flag2%(0) OR flag2%(1) OR flag2%(2) OR flag2%(3) THEN

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FOR % = 0 TO 3
    IF flag2(%) THEN
        PRINT USING " Channel # was out of range."; %
    END IF
NEXT %
PRINT
PRINT * PRESS ANY KEY TO CONTINUE*
anykey
CLS
ELSE
    s(0) = (s(0) + zero(0)) * cal(0)
    s(1) = (s(1) - zero(1)) * cal(1)
    hp = (s(0) + s(1)) / 6!
    vp = (s(2) - zero(2)) * cal(2)
LOCATE 10, 1
PRINT USING " LOAD 1 = ###..#. lbs"; s(0)
PRINT USING " LOAD 2 = ###..#. lbs"; s(1)
PRINT USING " Pressure = ###..#. psi"; vp
PRINT USING " Pore Pressure = ###..#. psi"; s(3)
k$ = UCASE$(INKEY$)
if = TIMER
DO
    LOOP UNTIL (TIMER - if) > 1
END IF
LOOP UNTIL k$ = "C"
CLS
par 0
END SUB
Appendix C

MACHINE DRAWINGS
FOR THE INSTRUMENTED OEDOMETER

The instrumented oedometer consists of eleven components. The specifications for the stainless steel, bolts, pins, springs, bearings, and valves are shown in Table C-1. Figs. C-1 through C-12 are reduced-scale machine drawings. A plan view, side views and cross-sections of the assembled instrumented oedometer are shown in Fig. 4-2. The numbers shown for each piece in the machine drawings correspond to the numbers in Fig. 4-2.
Table C-1. Specifications for the Instrumented Oedometer

<table>
<thead>
<tr>
<th>Item</th>
<th>Specifications</th>
<th>Manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oedometer</td>
<td>17-4 PH Stainless Steel</td>
<td>Not specified</td>
</tr>
<tr>
<td>Bolt</td>
<td>60 Series Socket Head Cap Screws 1/4&quot;-28</td>
<td>Not specified</td>
</tr>
<tr>
<td></td>
<td>Flat Head Hex-Socket Cap Screws 1/4&quot;-28</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flat Head Hex-Socket Cap Screws #4-28</td>
<td></td>
</tr>
<tr>
<td>Dowel Pin</td>
<td>Diameter 0.250&quot; (All pins are slide-in type)</td>
<td>Not specified</td>
</tr>
<tr>
<td>Aluminum Pin</td>
<td>Diameter 0.0625&quot; with enlarged head</td>
<td>Not specified</td>
</tr>
<tr>
<td>Spring</td>
<td>F-37 (Hard Drawn)</td>
<td>Century Spring Corporation</td>
</tr>
<tr>
<td></td>
<td>OD = 0.375&quot;, Wire Diameter = 0.046&quot;</td>
<td>PO. 15287</td>
</tr>
<tr>
<td></td>
<td>Free Length = 0.750&quot;, Solid Height = 0.368&quot;</td>
<td>222 East 16th street</td>
</tr>
<tr>
<td></td>
<td>Length after being tightened = 0.550&quot;</td>
<td>Los Angeles, CA 90015</td>
</tr>
<tr>
<td></td>
<td></td>
<td>213-749-1466, 1-800-237-5225</td>
</tr>
<tr>
<td>Flat Bearing</td>
<td>Single Row Plastic Flat Cage (FF2515)</td>
<td>INA Linear Technik, INC.</td>
</tr>
<tr>
<td></td>
<td>Weight = 3.9 g, Width = 0.591&quot;,</td>
<td>3399 Progress street</td>
</tr>
<tr>
<td></td>
<td>Length = 1.772&quot;, Roller diameter = 0.984&quot;,</td>
<td>Bensalem, PA 19020</td>
</tr>
<tr>
<td></td>
<td>Load ratings - Dynamic = 4000lbs</td>
<td>215-245-8033, 1-800-INA-3399</td>
</tr>
<tr>
<td></td>
<td>- Static = 9300 lbs</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mounting height = 0.087&quot;</td>
<td></td>
</tr>
<tr>
<td>Ball Bearing</td>
<td>Precision Steel Balls (Diameter = 0.250&quot;)</td>
<td>Not specified</td>
</tr>
<tr>
<td>Valve</td>
<td>40 Series 2-Way Ball Valve</td>
<td>Whitey Company</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5679 Landregan street</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Oakland, CA 94662</td>
</tr>
</tbody>
</table>
Fig. C-1 (1) Cap
Fig. C-2 (2) Retaining Ring
Fig. C-3 (3) Fixed Half of the Oedometer
Fig. C-4  (4) Split Half of the Oedometer
Fig. C-5 (5) Base Plate - 1
Fig. C-6 (5) Base Plate-2
Fig. C-7 (6) End Wall
Fig. C-8 (7) Side Wall
Fig. C-9 (8) Drainage Insert
(a) Plate for Valve

(b) Drainage System

Fig. C-10. (9) Plate for Valve and Drainage System
Fig. C-11. (10) Settlement Ring
Appendix D

PORE PRESSURE PARAMETERS FOR PARTLY SATURATED SOILS

Introduction

In order to use the compaction-induced earth pressure theory developed by Filz (1992) to calculate earth pressures due to compaction of partly saturated soils, it is necessary to know the value of Skempton's (1954) pore pressure parameters $B$ and $A$ for the soil. Filz (1992) suggested the possibility of evaluating Skempton's parameter $B$ using the effective and the total strength parameters for the soil; however, that approach did not result in reasonable values of $B$. This appendix presents a derivation of the equation that relates the value of $B$ to the compressibility of the soil skeleton and the degree of saturation of the soil. The final expression (Eq. D.7 below) was used in the analyses discussed in Chapter 6.

Skempton's Parameter $B$

Skempton (1954) defined a pore pressure parameter $B$ which represents the ratio of the change in pore pressure ($\Delta u$) caused by an equal all-around change in total stress ($\Delta \sigma_3$) to the magnitude of the change in total stress:

$$B = \frac{\Delta u}{\Delta \sigma_3} \quad \text{(D.1)}$$

where

$B = \text{Skempton's pore pressure parameter for pore pressures induced by an isotropic change in total stress (dimensionless)}$,

$\Delta u = \text{change in pore pressure induced by the change in total stress (pressure units)}$,

and
\( \Delta \sigma_3 = \text{change in total minor principal stress (pressure units)}. \)

Skempton showed that the value of \( B \) can be related to the compressibility of the soil skeleton and the degree of saturation of the soil, as follows:

\[
B = \frac{1}{1 + \frac{n \cdot c_m}{c_b}} \quad \text{(D.2)}
\]

where

\( n = \text{porosity (dimensionless)}, \)
\( c_m = \text{compressibility of the pore fluid (reciprocal of pressure units), and} \)
\( c_b = \text{coefficient of volume compressibility of the soil skeleton subjected to isotropic change in pressure (reciprocal of pressure units)}. \)

The value of \( c_b \) can be measured directly using triaxial specimens, or its value can be estimated as discussed in Chapter 5.

For one-dimensional compression, vertical pressures are known more precisely than horizontal pressures. Bishop (1954) suggested the use of a pore pressure parameter (\( \overline{B} \)) with respect to major principal stress (\( \sigma_1 \)), and Fredlund and Rahardjo (1993) suggested that \( \overline{B} \) should be defined in terms of changes in pore pressure and vertical stress, as follows:

\[
\overline{B} = \frac{\Delta u}{\Delta \sigma_v} \quad \text{(D.3)}
\]

where

\( \overline{B} = \text{pore pressure parameter for pore pressures induced by a change in total vertical stress under one-dimensional loading conditions (dimensionless), and} \)
\( \Delta \sigma_v = \text{change in total vertical stress (pressure units)}. \)
Using this equation, it is not necessary to estimate the value of horizontal stress in order to estimate the increase in pore pressure.

Several studies of Skempton's (1954) pressure parameters have been made for partly saturated soils (Bishop, 1960; Aichison, 1961; Jennings, 1961; Bishop and Henkel, 1962); however, these studies did not present analytical procedure for evaluating the magnitude of the pore pressure parameters. Schuurman (1966) suggested that Skempton's (1954) pore pressure parameters for partly saturated soils could be estimated based on the compressibility of the mixture of air and water in the soil voids, and the relation between pore air and water pressures, but he did not develop a quantitative procedure for applying this concept.

As shown by Eq. (D.2), the compressibility of the pore fluid in partly saturated soils must be known in order to estimate the value of the pore pressure parameter B analytically. Several simple expressions for the compressibility of a mixture of air and water have been proposed. Bishop and Eldin (1950), Skempton and Bishop (1954), and Chang and Duncan (1983) assumed zero compressibility of water and zero soil suction. Each derived a slightly different expression, because each used a different reference state for volumetric strain. Chang and Duncan (1983) assumed that an air-water mixture can be treated as a homogenized fluid, and derived an equation Eq. (D.4), which expresses the compressibility of an air-water mixture as a function of current void ratio and current degree of saturation. (The following equation is different from the original form presented by Chang and Duncan, in which total volume of soil was used as a reference for defining the volumetric strain. The volume of the air-water mixture is used as the reference in Eq. (D.4).)
\[ c_m = \frac{(e - Se + HSe)^2 / e}{(e_0 - S_0e_0 + HS_0e_0) / u_{a0}} \]  
\hspace{1cm} (D.4)

where

\[ e = \text{void ratio at a given pressure (dimensionless)}, \]
\[ e_0 = \text{initial void ratio (dimensionless)}, \]
\[ S_0 = \text{initial degree of saturation (dimensionless)}, \]
\[ S = \text{degree of saturation at a given pressure (dimensionless)}, \]
\[ u_{a0} = \text{initial absolute air pressure (pressure units), and} \]
\[ H = \text{solubility of air in water (dimensionless)}. \]

As Schuurman (1966) suggested, the relationship between pore air pressures and pore water pressures must be known for purposes of estimating the value of the pore pressure parameter B unless simplifying assumptions are made. The relationship between pore air and pore water pressures may be described by considering pore air pressures and pore water pressures separately. Bishop (1960) introduced the following pore pressure parameters for partly saturated soils:

\[ B_a = \frac{\Delta u_a}{\Delta \sigma_3} \]  
\hspace{1cm} (D.5.a)

\[ B_w = \frac{\Delta u_w}{\Delta \sigma_3} \]  
\hspace{1cm} (D.5.b)

where

\[ B_a = \text{Skempton's pore pressure parameter for pore air pressure induced by a change in minor principal stress (dimensionless)}, \]
\[ B_w = \text{Skempton's pore pressure parameter for pore water pressure induced by a change in minor principal stress (dimensionless)}. \]
\[ \Delta u_a = \text{change in pore air pressure induced by a change in total stress (pressure units), and} \]

\[ \Delta u_w = \text{change in pore water pressure induced by a change in total stress (pressure units).} \]

Similarly to equation (D.3), pore pressure parameters for air and water can be defined for one-dimensional compression. They are:

\[ \bar{B}_a = \frac{\Delta u_a}{\Delta \sigma_v} \quad \text{(D.6.a)} \]

\[ \bar{B}_w = \frac{\Delta u_w}{\Delta \sigma_v} \quad \text{(D.6.b)} \]

where

\[ \bar{B}_a = \text{pore pressure parameter for pore air pressure induced by a change in vertical total stress (dimensionless), and} \]

\[ \bar{B}_w = \text{pore pressure parameter for pore water pressure induced by a change in vertical total stress (dimensionless),} \]

A rigorous expression for the compressibility of the air/water mixture has been derived by Fredlund (1976), using the pore pressure parameters for air and for water separately. This approach was also discussed by Fredlund (1985), and Fredlund and Rahardjo (1993).

Fredlund and Rahardjo (1993) used this approach to derive expressions for the pore pressure parameters \( B \) with respect to pore water and pore air pressures. However, the solutions are quite complicated, and involve several parameters that are not commonly used in practice and not easily measured. In addition, it was found that values of modulus with respect to \( (\sigma-u_a) \) are affected by soil suction \( (u_a-u_w) \) (Edil et al., 1981), and that
values of volumetric strain due to soil suction are affected by total stress (Josa et al., 1987). Therefore, it can be concluded that the equations derived by Fredlund and Rahardjo (1993) are not useful in practice, at the present time.

The equations derived by Fredlund and Rahardjo can be simplified by making the following reasonable assumptions:

(1) the pore water is incompressible, and

(2) the compression of the soil skeleton due to change in soil suction is negligible.

Zero compressibility of pore water has been commonly assumed in practice (Bishop and Eldin, 1950; Bishop and Skempton, 1954; Chang and Duncan, 1983). The compressibility of water is a function of temperature and pressure. It is $4.57 \times 10^{-5}$ m$^2$/kN at 20°C under atmospheric pressure. Fredlund (1976) showed that the effects of compressibility of water on the compressibility of an air-water mixture is significant only when the degree of saturation is very close to 100 percent.

The second assumption is accurate for cases where the change in soil suction is small. For soils in which the air phase is occluded, pore air pressures are approximately equal to pore water pressures; as a result, soil suction is constantly zero. Jennings and Burland (1962) stated that the air phase in sand becomes occluded at a degree of saturation of about 20%, the air phase of silt becomes occluded at a degree of saturation equal to 40 to 50%, and the air phase in clay becomes occluded at a degree of saturation of 85%. In general, the degree of saturation corresponding to the optimum water content is about 80 to 90 percent. Consequently, it is reasonable to assume that the air phase of soils is occluded when their water content is around optimum. Even for soils that are drier, the change in soil suction may be also zero, even though the soil suction is non-zero.
The second assumption is also reasonable for the case where the compressibility of
the soil with respect to soil suction \((u_a - u_w)\) is much smaller than that with respect to \((\sigma - u_a)\). Volume changes due to changes in soil suction are only large for expansive clays.

Using the assumptions discussed above, the following simple forms of Fredlund and
Rahardjo's equations can be obtained:

\[
B_a = \frac{1}{1 + \frac{n \cdot (1 - S + S \cdot H)}{c_{ii} \cdot u_a}} \tag{D.7}
\]

\[
\bar{B}_a = \frac{1}{1 + \frac{n \cdot (1 - S + S \cdot H)}{m_v \cdot u_a}} \tag{D.8}
\]

\[c_{ii} = \text{compressibility of soil with respect to a change in the minor principal stress (reciprocal of pressure units)},\]

\[m_v = \text{compressibility of soil with respect to a change in vertical stress (reciprocal of pressure units)},\]

Although equations (D.7) and (D.8) take into account the solubility of air in water,
this factor is negligible where (1) the degree of saturation is less than 80 %, or (2) there is
not enough time for air to go into solution. In the case where the effects of air solubility
are negligible, equations (D.7) and (D.8) can be simplified to:

\[
B_a = \frac{1}{1 + \frac{n \cdot (1 - S)}{c_{ii} \cdot u_a}} \tag{D.9}
\]
\[
\bar{B}_a = \frac{1}{1 + \frac{n \cdot (1 - S)}{m_v \cdot u_a}}
\]  \hspace{1cm} (D.10)

These equations involve assumptions. However, the assumptions are quite reasonable for most compacted backfills.

It should be noted that equation (D.7) through (D.8) provide pore pressure parameter for the air phase, \(B_a\). For soils with occluded air, \(B_a\) is very nearly equal to \(B_w\); therefore, \(B_a\) can be considered to represent the change in both air and water pressure for soils with occluded air bubbles. For drier soils, the pore pressure parameter with respect to water (\(B_w\)) is slightly greater than \(B_a\). Therefore, the pore pressure parameter for the average pore fluid pressure must be slightly greater than the value obtained by the equations above, if the amount of air in the soil does not change due to air flow in or out of the voids.

As described in chapter 3, the value of \(B\) increases with increasing total stress due to increase in degree of saturation, and the secant value of the pore pressure parameter \(B\) is different from tangent value. Hilf (1948) developed the following equation expressing the secant value of \(B\) for one-dimensional compression:

\[
B_{as} = \frac{1}{1 + \frac{(1 - S_0 + H \cdot S_0) \cdot n_0}{m_v \cdot u_a}}
\]  \hspace{1cm} (D.11)

where

\(B_{as}\) = secant value of the pore pressure parameter \(B\) for air pressure (dimensionless), and

\(n_0\) = initial porosity (dimensionless).
It was found that use of Eq. (D.11) results in pore pressures that are about 10 percent higher than those calculated using Eqs (D.7) and (D.8). The reason for this discrepancy is not known.
Appendix E

INITIAL PORE PRESSURES

In order to use the compaction-induced earth pressure theory to calculate earth pressures for moisture-sensitive partly saturated soils, the value of initial pore pressures must be known. Filz (1992) suggested that the initial pore pressures might be inferred from the values of the effective and total strength parameters for the soil. However, it was found that the method did not yield reasonable results. This appendix describes a method for calculating the initial capillary pressures using an empirical correlation between water content and capillary pressure head. The relationship is frequently used for estimating unsaturated hydraulic conductivity. Several researchers have proposed empirical curves to relate capillary pressure head to water content. Many investigators believe that the model proposed by Van Genuchten (1980) describes the relationship between water content and capillary head fairly well. The Van Genuchten's model relates the effective saturation of a soil to the capillary head; as follows:

\[ S_e = (1 + \alpha |h|^n)^{-\left(1 - \frac{1}{n}\right)} \quad \text{for } h < 0 \]  
\[ S_e = 1.0 \quad \text{for } h = 1.0 \]

where

- \( S_e \) = effective saturation defined by Eq. (E.2) (dimensionless),
- \( h \) = capillary pressure head (length units),
- \( \alpha \) = curve fitting parameter relating to the position of the water-retention curve (reciprocal of length units), and
- \( n \) = curve fitting parameter relating to the slope of the water-retention curve (dimensionless).
The effective saturation ($S_e$) is defined as:

$$S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r}$$  \hspace{1cm} (E.2)

where

$\theta$ = volumetric moisture content, which is the ratio of volume of water to total volume of soil (dimensionless),

$\theta_s$ = saturated volumetric moisture content, which is identical to porosity (dimensionless), and

$\theta_r$ = residual volumetric moisture content, which is a curve fitting parameter for the water-retention curve (dimensionless).

Fig. 3-1 shows a typical soil-water retention curve obtained through laboratory tests. As shown in the figure, the soil-water retention curve is hysteretic. However, most researchers use a single set of parameters ($\alpha$, $n$, and $\theta_r$) for both drying and wetting, for the sake of simplicity. Table E-1 shows values of parameters reported previously for various types of soil.

Since no laboratory tests for measuring the water-retention curve were performed during this study, values of initial capillary pressure were calculated using the parameters for sandy loam, silt loam and sand listed on the Table E-1. The calculated values of capillary pressure are shown in Table E-2.

Two things about the values of capillary pressure listed in Table E-2 can be noted:

(1) The values calculated using the three sets of parameters vary widely. Therefore, it must be concluded that it is not possible to make highly accurate estimates of capillary head based on Van Genuchten's equation with parameter values estimated on the basis of a visual description.
Table E-1. Parameters for Van Genuchten's Model

<table>
<thead>
<tr>
<th>Reference</th>
<th>Soil</th>
<th>$\alpha$ (m$^{-1}$)</th>
<th>$n$</th>
<th>$\theta_s$</th>
<th>$\theta_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Van Genuchten</td>
<td>Hygiene sandstone</td>
<td>0.79</td>
<td>10.4</td>
<td>0.612</td>
<td>N.A.</td>
</tr>
<tr>
<td>(1980)</td>
<td>Touchet silt loam</td>
<td>0.5</td>
<td>7.09</td>
<td>0.405</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>Silt loam G.E. 3</td>
<td>0.423</td>
<td>2.06</td>
<td>0.331</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>Beit Netofa clay</td>
<td>0.152</td>
<td>1.17</td>
<td>0.00</td>
<td>N.A.</td>
</tr>
<tr>
<td>Parker et al.</td>
<td>Sandy loam*</td>
<td>1.53</td>
<td>1.265</td>
<td>0.074</td>
<td>0.355</td>
</tr>
<tr>
<td>(1985)</td>
<td>Silty loam*</td>
<td>3.46</td>
<td>1.289</td>
<td>0.103</td>
<td>0.388</td>
</tr>
<tr>
<td></td>
<td>Silty clay loam</td>
<td>0.82</td>
<td>1.275</td>
<td>0.112</td>
<td>0.402</td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>0.07</td>
<td>1.419</td>
<td>0.109</td>
<td>0.589</td>
</tr>
<tr>
<td></td>
<td>(weathered shale)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nishigaki et al.</td>
<td>Decomposite granite</td>
<td>7.0</td>
<td>1.82</td>
<td>0.11</td>
<td>0.338</td>
</tr>
<tr>
<td>(1992)</td>
<td>Toyoura sand*</td>
<td>2.2</td>
<td>12.32</td>
<td>0.00</td>
<td>0.411</td>
</tr>
</tbody>
</table>

* Parameters for these soils were used for the calculation of the initial capillary pressures in the specimens.
Table E-2. Capillary Pressures Estimated by Van Genuchten's Model (1980) for Specimens of Yatesville Silty Sand

<table>
<thead>
<tr>
<th>Parameters based on</th>
<th>Parameter values</th>
<th>Test No.</th>
<th>Effective saturation</th>
<th>Capillary pressure head (m)</th>
<th>Capillary pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silty loam</td>
<td>α = 3.46 n = 1.289 θ_r = 0.103</td>
<td>Y2</td>
<td>0.819</td>
<td>0.972</td>
<td>16.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y3</td>
<td>0.708</td>
<td>2.693</td>
<td>45.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y9</td>
<td>0.437</td>
<td>105.4</td>
<td>1796</td>
</tr>
<tr>
<td>Sandy loam</td>
<td>α = 1.53 n = 1.265 θ_r = 0.074</td>
<td>Y2</td>
<td>0.819</td>
<td>1.600</td>
<td>27.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y3</td>
<td>0.708</td>
<td>41.5</td>
<td>70.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y9</td>
<td>0.437</td>
<td>83.6</td>
<td>1425</td>
</tr>
<tr>
<td>Sand</td>
<td>α = 2.2 n = 12.32 θ_r = 0.0</td>
<td>Y2</td>
<td>0.819</td>
<td>0.836</td>
<td>14.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y3</td>
<td>0.708</td>
<td>0.880</td>
<td>15.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y9</td>
<td>0.437</td>
<td>0.967</td>
<td>16.5</td>
</tr>
</tbody>
</table>
(2) The suction values are quite high in some cases, ranging up to 1800 psi for the dry specimen of Test Y9.

The significance of these widely varying and very large suction values is mitigated by the fact that the suction is only high when the degree of saturation is low. Accordingly, the factor \( \chi \) in Bishop's pore pressure equation is very small:

\[
\sigma' = \sigma - u_s + \chi (u_s - u_w)
\]

Thus, although the values of suction \((u_s - u_w)\) may be large, the effects on the value of \(\sigma'\) is greatly mitigated because the value of \(\chi\) is so small.
Appendix F

LABORATORY TESTS ON YATESVILLE SILTY SAND

Description of Soil

Yatesville silty sand is brown to dark brown silty sand of alluvial origin. The sample used in this study is from the foundation of Yatesville Lake Dam in Lawrence County, Kentucky. The sample was sieved by Sehn (1990) using a commercial-grade wire mesh with 0.20-inch openings, and oversized particles were discarded. Roots, shells, and other non-mineral substances were removed.

Yatesville silty sand has been used by Sehn (1990), Brandon et al. (1990), and Filz (1992) for previous research studies. Strictly speaking, the Yatesville silty sand tested at Virginia Tech comes from two different batches. Filz used soil from the same batch as Sehn (1990) for most of his instrumented retaining wall tests. During Filz's research, more soil was added to make up for losses of material during early tests. The added soil came from another batch which Brandon et al. (1990) had used. The volume of soil added was approximately 10 percent of the original volume. The mixed soil was found to have different compaction characteristics from that tested earlier. The maximum dry density for the mixed soil was about 4 to 5 lb/ft³ higher than for the batch of Yatesville silty sand tested earlier by Sehn (1990) and Filz (1992). Using the mixed soil, Filz performed two additional instrumented retaining wall tests. The soil used in this research was taken from the mixed soil prepared by Filz (1992). It has been found that all of the samples of Yatesville silty sand have essentially the same soil properties, except for moisture-density characteristics.
**Particle Size Distribution and Specific Gravity**

Particle size analyses were performed in accordance with ASTM D 422-63. Fig. F-1 shows the test results, together with the gradation curves obtained by Brandon et al. (1990) and Filz (1992). It can be seen that all the curves are similar. About 40 to 50 percent of the material passes the No. 200 sieve. The material is classified as silty sand, with a group symbol SM, based on the Unified Soil Classification System (UCSC).

Sehn (1990) reported that the specific gravity was 2.66. Brandon et al. (1990) reported that the specific gravity of the added soil was 2.67. Filz (1992) reported that the specific gravity for the mixture of these soils which he prepared was 2.67. The soil used in this study was the same mixed soil as Filz (1992) used; therefore, Gs = 2.67 was used throughout this study.

**Moisture-Density Relations**

Filz (1992) performed Standard Proctor tests and Modified Proctor tests on Yatesville silty sand in accordance with ASTM D-698 and ASTM D-1557. In addition, he developed two other moisture-density curves using lower compactive efforts. The test results are shown in Fig. F-2. Based on the Standard Proctor test results, a maximum dry density of 125pcf was obtained at an optimum water content of 10.9%.

**Consolidated-Undrained Triaxial Tests**

Brandon et al. (1992) performed consolidated-undrained triaxial compression tests on laboratory compacted specimens of Yatesville silty sand.

Fig. F-3 shows the variation of effective stress friction angle with dry unit weight obtained from the results of these tests. It was found that the effective stress friction
Fig. F-1. Gradation Curves for Yatesville Silty Sand
Fig. F-2. Moisture Density Relationship for Yatesville Silty Sand - after Filz (1992)
Fig. F-3. Effective Friction Angle from Consolidated-Undrained Tests for Yatesville Silty Sand - after Filz (1992)
angle increased with increasing dry unit weight. The effective stress cohesion intercept was found to be zero.

**Low Pressure Undrained Triaxial Tests**

Filz (1992) performed unconsolidated-undrained triaxial compression tests on specimens of Yatesville silty sand. Samples were taken from the compacted backfill of the instrumented retaining wall tests using Shelby tubes. The Shelby tube samples had been stored for about one month at the time they were tested.

The tests were conducted on short specimens with a height of 1.5 inches and a diameter of 2.8 inches, using lubricated end platens. The cell pressures used for the tests were 0, 2, and 4 psi. It was considered that these small cell pressures reflected the condition of the backfills after compaction. The test conditions and results are shown in Table F-1. Failure was defined as 15 percent axial strain, unless the peak deviator stress occurred at smaller strain. Fig. F-4 shows the variations of the values of the total strength parameters with dry unit weight and water content obtained from the low-pressure UU-triaxial test results. The effective strength parameters obtained by Brandon et al. (1990) are shown together for the purpose of comparison. It was found that both the total stress friction angle and total stress cohesion intercept increased with increasing dry unit weight and with decreasing water content.

It was reported that most of the stress-strain curves were linear up to about 7 percent axial strain. The undrained initial tangent modulus values from the low-pressure UU-triaxial tests are listed in Table F-1. The values are scattered. Contours of undrained initial tangent modulus were drawn on the compaction diagram, as shown in Fig. F-5, so that the trend of changes in the initial moduli could be identified. It was found that the undrained initial tangent moduli increased with decreasing water content.
Table F-1. Results of Low Pressure Unconsolidated-Undrained Triaxial Tests Performed by Filz (1992)

<table>
<thead>
<tr>
<th>Water Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Cell Pressure (psi)</th>
<th>Deviator Stress at Failure* (psi)</th>
<th>Initial Tangent Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>106.7</td>
<td>12.1</td>
<td>0</td>
<td>11.9</td>
<td>145</td>
</tr>
<tr>
<td>114.5</td>
<td>10.1</td>
<td>0</td>
<td>33.3</td>
<td>800</td>
</tr>
<tr>
<td>115.1</td>
<td>13.2</td>
<td>0</td>
<td>33.1**</td>
<td>425</td>
</tr>
<tr>
<td>116.4</td>
<td>9.9</td>
<td>0</td>
<td>35.8</td>
<td>730</td>
</tr>
<tr>
<td>116.5</td>
<td>12.1</td>
<td>0</td>
<td>29.7</td>
<td>560</td>
</tr>
<tr>
<td>119.2</td>
<td>12.2</td>
<td>0</td>
<td>33.4</td>
<td>470</td>
</tr>
<tr>
<td>121.4</td>
<td>11.8</td>
<td>0</td>
<td>39.2</td>
<td>390</td>
</tr>
<tr>
<td>108.0</td>
<td>12.2</td>
<td>2</td>
<td>16.9</td>
<td>385</td>
</tr>
<tr>
<td>109.0</td>
<td>12.4</td>
<td>2</td>
<td>19.0</td>
<td>256</td>
</tr>
<tr>
<td>115.3</td>
<td>9.9</td>
<td>2</td>
<td>36.3</td>
<td>1200</td>
</tr>
<tr>
<td>116.0</td>
<td>10.0</td>
<td>2</td>
<td>43.5</td>
<td>960</td>
</tr>
<tr>
<td>116.5</td>
<td>12.3</td>
<td>2</td>
<td>34.3</td>
<td>444</td>
</tr>
<tr>
<td>118.0</td>
<td>9.6</td>
<td>2</td>
<td>41.2</td>
<td>850</td>
</tr>
<tr>
<td>119.3</td>
<td>12.5</td>
<td>2</td>
<td>39.6</td>
<td>440</td>
</tr>
<tr>
<td>120.0</td>
<td>12.2</td>
<td>2</td>
<td>42.4</td>
<td>485</td>
</tr>
<tr>
<td>120.7</td>
<td>11.9</td>
<td>2</td>
<td>45.7</td>
<td>440</td>
</tr>
<tr>
<td>121.7</td>
<td>10.1</td>
<td>2</td>
<td>55.4</td>
<td>1030</td>
</tr>
<tr>
<td>106.4</td>
<td>12.3</td>
<td>4</td>
<td>16.4</td>
<td>396</td>
</tr>
<tr>
<td>111.8</td>
<td>12.3</td>
<td>4</td>
<td>27.4</td>
<td>353</td>
</tr>
<tr>
<td>112.2</td>
<td>9.7</td>
<td>4</td>
<td>34.8</td>
<td>1050</td>
</tr>
<tr>
<td>114.4</td>
<td>12.4</td>
<td>4</td>
<td>32.4</td>
<td>294</td>
</tr>
<tr>
<td>118.6</td>
<td>12.7</td>
<td>4</td>
<td>42.2</td>
<td>480</td>
</tr>
<tr>
<td>118.7</td>
<td>9.9</td>
<td>4</td>
<td>49.8</td>
<td>730</td>
</tr>
<tr>
<td>120.2</td>
<td>12.2</td>
<td>4</td>
<td>49.0</td>
<td>495</td>
</tr>
<tr>
<td>120.6</td>
<td>9.6</td>
<td>4</td>
<td>57.9</td>
<td>910</td>
</tr>
</tbody>
</table>

* Failure was defined by 15% axial strain unless denoted by **.

** Failure was defined by peak deviator stress.
Fig. F-4. Total Stress Strength Parameters from Low Pressure UU-Triaxial Tests for Yatesville Silty Sand - after Filz (1992)
Fig. F-5. Initial Tangent Modulus Obtained from UU Triaxial Tests for Yatesville Silty Sand
High Pressure Undrained Triaxial Tests

Uncconsolidated-undrained triaxial compression tests were performed on partly saturated specimens of Yatesville silty sand during this study, using higher cell pressures than used by Filz (1992). The specimens were prepared by compacting the soil with a Harvard Miniature Compactor.

The specimens (initial height = 2.8 inches and initial diameter = 1.4 inches) were sheared in their as-compacted conditions at a strain rate of 1.0 percent per minute, using cell pressures of 10, 30, 50, and 70 psi. It should be noted that the specimens were sheared immediately after compaction. The cell pressures used in this series of tests cover the stress range induced during compaction in the instrumented retaining wall tests performed by Filz (1992). Test conditions and results are shown in Table F-2.

Figs. F-6 through F-9 show the variations of deviator stress at failure with dry unit weight and water content obtained from the test results summarized in Table F-2. Each figure shows values of deviator stress at failure for a different value of cell pressure (Fig. F-5, $\sigma_3$=10 psi; Fig. F-6, $\sigma_3$=30 psi; Fig. F-7, $\sigma_3$=50 psi; Fig. F-8, $\sigma_3$=70 psi).

Figs. F-10 and F-11 show the failure mechanisms of the specimen and idealized shapes of the stress-strain curves. In general, drier specimens exhibited more brittle behavior.

Fig. F-12 shows variations of the values of the total stress strength parameters with dry unit weight and water content which were determined from Figs. F-6 through F-9. Values of effective stress friction angle obtained by Brandon et al. (1990) are also shown for purposes of comparison. It was found that both the total stress friction angle and total stress cohesion intercept increased with increasing dry unit weight and decreasing water content.
Table F-2. Summary of UU Triaxial Tests on Yatesville Silty Sand (1/4)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cell pressure (psi)</th>
<th>Dry density (pcf)</th>
<th>Water content (%)</th>
<th>Degree of saturation (%)</th>
<th>Void ratio</th>
<th>Deviator stress at 10 % strain (psi)</th>
<th>Deviator stress at 15 % strain (psi)</th>
<th>Initial tangent modulus by hyperbolic fitting method (psi) (8) ***</th>
<th>Secant modulus at 1 % strain (psi) (9)</th>
<th>Modulus used for analysis (psi) (10) ***</th>
</tr>
</thead>
<tbody>
<tr>
<td>UU10-1</td>
<td>10</td>
<td>120.04</td>
<td>12.19</td>
<td>83.9</td>
<td>0.388</td>
<td>37.14</td>
<td>54.96</td>
<td>379</td>
<td>348</td>
<td>379</td>
</tr>
<tr>
<td>UU10-2</td>
<td>10</td>
<td>121.48</td>
<td>11.66</td>
<td>83.8</td>
<td>0.371</td>
<td>54.45</td>
<td>66.46</td>
<td>719</td>
<td>422</td>
<td>719</td>
</tr>
<tr>
<td>UU10-3</td>
<td>10</td>
<td>120.18</td>
<td>11.11</td>
<td>76.8</td>
<td>0.386</td>
<td>42.58 P</td>
<td>42.58 P</td>
<td>3984</td>
<td>1471</td>
<td>3984</td>
</tr>
<tr>
<td>UU10-4</td>
<td>10</td>
<td>109.56</td>
<td>11.12</td>
<td>57.0</td>
<td>0.521</td>
<td>34.21</td>
<td>37.42 P</td>
<td>1029</td>
<td>1213</td>
<td>1213</td>
</tr>
<tr>
<td>UU10-5</td>
<td>10</td>
<td>117.35</td>
<td>9.30</td>
<td>59.2</td>
<td>0.420</td>
<td>60.58 P</td>
<td>60.58 P</td>
<td>7847</td>
<td>3461</td>
<td>7847</td>
</tr>
<tr>
<td>UU10-6</td>
<td>10</td>
<td>118.11</td>
<td>9.82</td>
<td>63.9</td>
<td>0.411</td>
<td>61.50 P</td>
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<td>3230</td>
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<td>61.9</td>
<td>0.411</td>
<td>59.62 P</td>
<td>59.62 P</td>
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<td>3337</td>
<td>6201</td>
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<td>UU10-8</td>
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<td>10.80</td>
<td>49.2</td>
<td>0.586</td>
<td>31.53</td>
<td>37.52</td>
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<td>91.3</td>
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<td>11.13</td>
<td>18.73</td>
<td>89</td>
<td>135</td>
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<td>95.1</td>
<td>0.407</td>
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<td>70</td>
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<td>13.44</td>
<td>91.4</td>
<td>0.393</td>
<td>16.52</td>
<td>27.85</td>
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* 'P' indicates that the stress shown in the column is a peak value of deviator stress reached at a smaller value of strain.

** Initial tangent moduli were calculated using a hyperbolic fit at 95% and 70% of the deviator stress at 15% strain.

*** Values of modulus for analysis were chosen as the greater of the initial tangent modulus by the hyperbolic fitting method or the secant value at 1% strain.
Table F-2. Summary of UU Triaxial Tests on Yatesville Silty Sand (2/4)

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* 'P' indicates that the stress shown in the column is a peak value of deviator stress reached at a smaller value of strain.

** Initial tangent moduli were calculated using a hyperbolic fit at 95% and 70% of the deviator stress at 15% strain.

*** Values of modulus for analysis were chosen as the greater of the initial tangent modulus by the hyperbolic fitting method or the secant value at 1% strain.
Table F-2. Summary of UU Triaxial Tests on Yatesville Silty Sand (3/4)

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<th>Degree of saturation (%)</th>
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* 'P' indicates that the stress shown in the column is a peak value of deviator stress reached at a smaller value of strain.

** Initial tangent moduli were calculated using a hyperbolic fit at 95% and 70% of the deviator stress at 15% strain.

*** Values of modulus for analysis were chosen as the greater of the initial tangent modulus by the hyperbolic fitting method or the secant value at 1% strain.
Table F-2. Summary of UU Triaxial Tests on Yatesville Silty Sand (4/4)

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<th>Degree of saturation (%)</th>
<th>Void ratio</th>
<th>Deviator stress at 10 % strain (psi)</th>
<th>Deviator stress at 15 % strain (psi)</th>
<th>Initial tangent modulus by hyperbolic fitting method (psi)</th>
<th>Secant modulus at 1 % strain (psi)</th>
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* 'P' indicates that the stress shown in the column is a peak value of deviator stress reached at a smaller value of strain.

** Initial tangent moduli were calculated using a hyperbolic fit at 95% and 70% of the deviator stress at 15% strain.

*** Values of modulus for analysis were chosen as the greater of the initial tangent modulus by the hyperbolic fitting method or the secant value at 1% strain.
Fig. F-6. Deviator Stress at Failure from UU-Triaxial Tests on Yatesville Silty Sand
(Cell Pressure = 10 psi)

Failure is defined by peak deviator stress or 15% strain in case of no peak in stress-strain curve.
Fig. F-7. Deviator Stress at Failure from UU-Triaxial Test on Yatesville Silty Sand
(Cell Pressure = 30 psi)
Failure is defined by peak deviator stress or 15% strain in case of no peak in stress-strain curve.

Fig. F-8. Deviator Stress at Failure from UU-Triaxial Test on Yatesville Silty Sand (Cell Pressure = 50 psi)
Fig. F-9. Deviator Stress at Failure from UU-Triaxial Test on Yatesville Silty Sand
(Cell Pressure = 70 psi)

Failure is defined by peak deviator stress or 15% strain in case of no peak in stress-strain curve.
Fig. F-J0. UU-Triaxial Test Results - Failure Pattern and Type of Stress-Strain Curve
(Cell Pressure = 10 psi)
Fig. F-11. UU-Triaxial Test Results - Failure Pattern and Type of Stress-Strain Curve
(Cell Pressure = 70 psi)
Fig. F-12. Total Stress Strength Parameters from Unconsolidated-Undrained Tests for Yatesville Silty Sand (10 < σ < 70 psi)

Failure is defined by 15% strain.
Values of undrained initial tangent modulus were calculated using the hyperbolic fitting described by Duncan et al. (1986). The calculated values are shown in column (9) of Table F-2. Since the hyperbolic fitting method is based on the assumption that the stress-strain curve can be approximated as a hyperbola, the calculated values of modulus are not meaningful for the cases where the stress-strain curves deviated appreciably from hyperbolic shapes. Therefore, if the values of secant modulus at 1.0 percent strain exceeded those calculated using the hyperbolic fitting procedure, the secant modulus values were considered to be more representative of the behavior of the soil at low strains, and were used in calculating pore pressure parameters as explained in Chapter 5. Values of undrained secant modulus at 1.0 percent strain are shown in column (10) of Table F-2. A value of axial strain equal to 1.0 percent was chosen because data for strain values smaller than 1.0 percent strain involve relatively large inaccuracies due to seating load effects.

The values of undrained modulus used in the analyses are shown in column (11) of Table F-2. They were used to evaluate the modulus number (K) and the modulus exponent (n) in the following equation (Duncan and Chang, 1970):

\[
\left( \frac{E_i}{P_a} \right) = K \left( \frac{\sigma_3}{P_a} \right)^n \tag{F.1}
\]

where

\( E_i = \) undrained initial modulus (pressure units),
\( \sigma_3 = \) minor total principal stress (pressure units),
\( P_a = \) atmospheric pressure (pressure units),
\( K = \) modulus number (dimensionless), and
\( n = \) modulus exponent (dimensionless).
Figs. F-13 and F-14 show contours of \( K \) and \( n \) drawn on moisture-density diagrams. The contours indicate that the value of \( K \) decreases with increasing water content, and \( n \) increases with increasing water content. Both \( K \) and \( n \) are affected little by changes in density at the same water content.

**Direct Shear Tests**

Filz (1992) performed direct shear tests on specimens of Yatesville silty sand. The specimens were compacted in a 4 by 4-inch shear box, and shearing started within minutes after compaction. The normal loads were kept constant during the tests. Failure was defined by the occurrence of the peak shearing force. Cohesion intercepts obtained from the tests were plotted with dry unit weights in Fig. F-15. It was found that the values of total stress cohesion obtained from direct shear tests were smaller than those obtained from low-pressure UU-triaxial tests. Filz suggested that this might be due to aging of the triaxial specimens. It was also found that the values of total stress friction angle determined from the direct shear tests were larger than the values of effective stress friction angle obtained from CU-triaxial tests.

**Consolidation Tests**

Filz (1992) performed one-dimensional consolidation tests on specimens of Yatesville silty sand. The samples were taken from the compacted backfills of instrumented retaining wall tests using Shelby tubes. Some specimens were inundated after the seating loads were applied, and others were tested in the as-compacted conditions. Also, one test was conducted on a specimen consolidated from surry.

The void ratio-pressure relationships from these tests are shown in Fig.F-16 and F-17. Fig. F-16 shows results of tests on the inundated specimens, and Fig. F-17 shows the
Fig. F-13. Contours of Modulus Number K Obtained from High Pressure UU Triaxial Tests for Yatesville Silty Sand
Fig. F-14. Contours of Exponential Number n Obtained from High Pressure UU Triaxial Tests for Yatesville Silty Sand
Fig. F-15. Total Stress Cohesion Intercepts Interpreted from the Direct Shear Test Results
for Yatesville Silty Sand - from Filz (1992)
Fig. F-16. Consolidation Test Results for Inundated Yatesville Silty Sand

- from Filz (1992)
Fig. F-17. Consolidation Test Results for Moist Yatesville Silty Sand - from Filz (1992)
others. It was found that the consolidation curves the specimens that were not inundated crossed the normally consolidation line for the inundated specimens as shown in Fig. F-17. Filz considered that the negative pore pressures in the partially saturated specimens resist particle movement due to applied load. It is expected also that the specimens were stiffer as a result of aging of the specimen, as described in the previous section.

Values of constrained modulus were calculated using the results of the oedometer tests performed on the specimens with as-compacted water contents. Filz (1992) drew contours of constrained moduli, as shown in Fig. F-18.

**Summary**

Total stress strength parameters obtained from three types of tests are described in this appendix. The values of $c$ and $\phi$ determined from these tests differ as a result of: (1) strength gain due to aging, (2) confining pressure, and (3) type of test (triaxial or direct shear). Both the low-pressure UU tests and direct shear tests should provide the strength in a low stress range; however, it was found the strength by the low pressure UU tests were greater than those by the direct shear tests. This difference is believed to be due mainly to the strength gain resulting from aging of the low-pressure UU-triaxial test specimens. The difference in the values of the strength parameters between the low-pressure UU tests and the high-pressure UU tests was due primarily to the differences in degrees of saturation and pore pressures which resulted from the difference in confining pressure. As expected from classical soil mechanics, UU strength envelopes become flatter ($\phi$ decreases) as confining pressure increases. There may also be differences due to the differing ages of the low-pressure and the high-pressure test specimens, but these differences are considered to be a secondary factor.
Fig. F-18. Constrained Modulus Contours - from Filz (1992)
It is difficult to interpret the difference in moduli obtained by the low-pressure UU tests, the high-pressure UU tests, and the oedometer tests. Young's moduli obtained by UU triaxial tests are undrained moduli. It is necessary to know drained moduli for calculating Skempton's pore pressure parameter B analytically, as described in Chapter 5. Assuming isotropic elasticity, a relationship between undrained moduli and drained moduli for partly saturated soils can be developed. The relationship is:

\[ E' = (1 - \overline{A} + 2 \cdot \nu' \cdot \overline{A}) \cdot E_u \]  

(F.2)

where

- \( E' \) = drained Young's modulus (pressure units),
- \( E_u \) = undrained Young's modulus (pressure units), and
- \( \nu' \) = Poisson's ratio (dimensionless),
- \( \overline{A} \) = Skempton's pore pressure parameter with respect to a change in deviator stresses (\( = AB \)) (dimensionless).

For saturated soils, assuming \( \overline{A} = 0.33 \), and \( \nu' = 0.3 \),

\[ E' = 0.87 \cdot E_u \]  

(F.3)

For dry soils (\( \overline{A} = 0 \)),

\[ E' = E_u \]  

(F.4)

Most cases fall between these values. Therefore, values of undrained modulus obtained from UU triaxial tests can be considered to be approximately equal to values of drained modulus.

Second, it is convenient to use constrained moduli for calculating Skempton's pore pressure parameter B in one-dimensional compression, as described in Appendix D. The relationship between effective stress Young's modulus (\( E' \)) and effective stress constrained modulus (\( M' \)) can be expressed as:
\[ E' = \frac{(1 + v')(1 - 2v')}{(1 - v')} M' \]  \hspace{1cm} (F.5)

where

\[ M' = \text{effective stress constrained moduli (pressure units)}. \]

Assuming Poisson's ratio is equal to 0.3:

\[ E' = 0.74 M' \]

All three tests show similar trends in which values of modulus increase with decreasing water contents. However, the values of modulus determined from the three types of test are not the same. The values of constrained modulus obtained from the oedometer tests are greater than the values from triaxial tests, even when the relationship described by equation (F.5) is taken into account. The modulus values obtained from the low-pressure UU tests are smaller than those from the high-pressure UU triaxial tests.

Modulus values increase with increasing effective confining stress. This is the principal reason for the low values of moduli obtained by the low pressure triaxial tests. However, the values of undrained modulus obtained from the low-pressure UU tests were smaller than values of modulus for low stresses calculated using modulus numbers obtained from the high pressure UU tests, especially for dry specimens. The stubby shapes of the specimens used for the low pressure UU tests could have affected the values of modulus derived from these tests, as could aging of the specimens.
Appendix G

LABORATORY TESTS ON MONTEREY SAND

Description of Soil

Monterey sand #0/30 is a commercially available, clean, uniformly graded, fine to medium coarse sand. It consists of sub-rounded to rounded grains of quartz, with a small amount of feldspars.

K₀ test results and other test results have been reported for Monterey sand; for this reason, it was an appropriate material for evaluating the performance of the instrumented oedometer developed in this study. Sehn (1990) performed K₀ instrumented oedometer tests using a sample from the same shipment of sand as was used for this study. Milstone (1985) reported index properties for the same sample. Muzzy (1983) also reported index properties for Monterey sand #0/30. Wright (1969) performed K₀ triaxial tests, and Marachi et. al. (1969) reported strength parameters for a slightly different Monterey sand (Monterey #20).

Particle Size Analysis

Particle size analyses were performed in accordance with ASTM D 422-63. Fig. G-1 shows the test results. Monterey sand #0/30 is quite uniform, with about 100 percent passing the No. 20 sieve and about 100 percent retained on the No. 70 sieve. Monterey sand #0/30 is classified as SP by the Unified Soil Classification System. Values of C_u and C_c obtained from the particle size analysis are shown in Table G-1.

Another grain size distribution curve shown in Fig. G-1 is from a test reported by Milstone (1985). The two curves are almost identical. In addition, the values of C_u and C_c obtained by Muzzy (1983) were quite similar to the values for the sample tested in this study and the one tested by Milstone. Muzzy (1983) and Milstone (1985) reported that
Table G-1. Index Properties for Monterey #0/30 Sand

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<tr>
<td>$G_s$</td>
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<td>2.65</td>
<td>2.65</td>
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<tr>
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<td>0.83</td>
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<td>—</td>
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<td>0.45</td>
<td>0.45</td>
<td>0.55</td>
</tr>
<tr>
<td>$C_u$</td>
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<td>1.60</td>
<td>1.37</td>
<td>—</td>
</tr>
<tr>
<td>$C_c$</td>
<td>1.10</td>
<td>1.00</td>
<td>0.95</td>
<td>—</td>
</tr>
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</table>
the specific gravity of Monterey #0/30 is 2.65, and this value was adopted for this study.

**Minimum and Maximum Densities**

Minimum density tests and maximum density tests were performed in accordance with ASTM D-4254, and ASTM D-4253, respectively. Minimum density tests were conducted by method (A) and method (C). Method (A) provided a minimum density of 90.4 pcf, which was slightly higher than the value of 89.4 pcf obtained by method (C). Following the recommendation in ASTM D-4254, the value of 90.4 pcf obtained by method (A) was used in this study. The corresponding maximum void ratio is 0.829.

Maximum density tests were performed by the dry method and the wet method. The dry method provided a higher maximum density of 107.4 pcf, which corresponds to a minimum void ratio is 0.540.

The minimum density and the maximum density obtained in this study were slightly different from those reported by Muzzy (1983).

**Strength Parameters**

The effective friction angle of Monterey #0/30 sand can be represented by the following empirical relationship obtained by Knight (1988).

\[ 0.56 = e \cdot \tan \phi' \]  
\[ (G.1) \]

where

- \( e \) = void ratio, and
- \( \phi' \) = effective stress friction angle.

The equation above was adopted by Sehn (1990), and has also been used in this study.
Fig. G-2 shows results of vacuum triaxial tests and consolidated undrained triaxial tests conducted on Monterey #30 sand by Virginia Tech graduate students in the graduate laboratory class during the 1993 spring semester. The results are in good agreement with the form of Eq. (G.1), but with a different value of the constant (0.52 is opposed to the value of 0.56 representing the tests performed by Knight (1988)). The difference is probably due to the fact that a sample used by Spring, 1993 laboratory class was slightly better graded than the samples tested by Knight and used in this study. It was reported that both $C_u$ and $C_c$ for the lab class sample were equal to 1.0. Therefore, Eq. (G.1) has been used for estimating effective stress friction angles in this study.

Marachi et. al. (1969) performed triaxial tests and plane strain tests on Monterey #20 sand. Fig. G-3 (a) shows a plot of varying effective stress friction angles with initial void ratio for a confining pressure of 10 psi. Fig. G-3 (b) shows a plot of varying effective stress friction angles with confining pressures. The values of $\phi'$ estimated by Eq. (G.1) are in good agreement with data shown in Fig. G-3 (a).
Fig. G-2. Effective Stress Friction Angle for Monterey Sand
(a) Effective Friction Angle - Void Ratio Relationship

(b) Variation of Effective Friction Angle with Confining Pressure

Fig. G-3. Effective Friction Angle of Monterey #20 Sand
(from Marachi et. al., 1969)
VITA

Katsuji Ishihara was born on Nov. 15, 1960, in Kyoto, Japan. In 1979, he enrolled at Kyoto University, where he received his Bachelor of Science degree in 1983. Upon graduation, he continued studying geotechnical engineering in graduate school at the same university. After receiving a Master of Science degree in 1985, he was employed as a civil engineering designer by Nikken Sekkei, an architecture-engineering firm with offices in Tokyo and Osaka. He started his graduate study in Virginia Tech in the Fall of 1990, while on study leave from Nikken Sekkei.

Katsuji Ishihara