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TECHNICAL DATA BASE FOR STABILIZATION OF DEEP SOIL LAYERS

Q. L. Robnett G. F. Jamison

M. R. Thompson

University of Illinois

TECHNICAL REPORT NO. AFWL-TR-70-84

April 1971

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AIR FORCE WEAPONS LABORATORY
Air Force Systems Command
Kirtland Air Force Base
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TECHNICAL DATA BASE FOR STABILIZATION
OF DEEP SOIL LAYERS

Q. L. Robnett G. F. Jamison
M. R. Thompson
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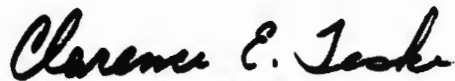
This report was prepared by the University of Illinois, Urbana, Illinois, under Contract F29601-70-C-0024. The research was performed under Program Element 63723F, Project 683M, Task 4.9.002.

Inclusive dates of research were January 1970 through September 1970. The report was submitted 12 March 1971 by the Air Force Weapons Laboratory Project Officer, Lt Richard A. Weismiller (DEZ).

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ABSTRACT

The majority of the airfield pavements supporting current Air Force operations were designed and built during the two decades spanning the 1940's and 1950's. The nature of the aircraft for which these early pavements were designed is substantially different than the planes currently in service and the much heavier aircraft, such as the C-5A, anticipated for use in the future. To accommodate the heavier repeated wheel loads of the future, many of the airfield pavements currently in service will have to be strengthened. Possible schemes for improving the load-carrying capacity and performance of existing pavements fall into two broad categories: (1) conventional overlaying operations and (2) strengthening of the soil beneath the existing pavement structure with a minimum of disturbance to the existing pavement structure. In this report, the ability of various stabilization procedures to provide strength improvement to in-place subgrades was examined. Included in the stabilization procedures examined were: electrical methods (electro-osmosis and electro-chemical), cement and chemical grouting procedures, lime stabilization procedures (drilled-hole and pressure injection) and a number of new techniques. A tentative evaluation, subject to review and revision upon completion of the scheduled laboratory studies, indicates that certain grouting procedures and lime stabilization procedures offer the most potential for accomplishing strengthening of in-place subgrade materials.

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SECTION I

INTRODUCTION

GENERAL

The majority of airfields supporting current Air Force operations were designed and built during the two decades spanning the 1940's and 1950's. These airfields are approaching the end of their design life and are consequently requiring increased maintenance and repair. The nature of the aircraft for which these early pavements were designed obviously is substantially different than the modern Air Force planes currently in service. Perhaps the most severe loading conditions to be anticipated in the future are associated with the very heavy multi-wheeled transport aircraft such as the C-5A. These large aircraft, with gross weights approaching one million pounds and landing gears with upward of 30 tires, are substantially different than the aircraft for which a large number of airfield pavements were originally designed.

It is obvious that many of the pavements currently in service will have to be strengthened in order to accommodate heavy repeated wheel loads such as those associated with the C-5A. Possible schemes for improving the load-carrying capacity and the performance of existing pavements fall into two broad categories. The first category is the conventional operation of overlaying the pavement with asphalt concrete or portland cement concrete. The second category, one which this study will consider in detail, is to stabilize the soil beneath the existing pavement structure. It is envisaged that deep layer stabilization techniques may be of value for strengthening isolated weak areas in an existing pavement and also in more extensive applications.

The objective of this project is to evaluate existing soil stabilization techniques and procedures and to attempt to develop a technique(s) for increasing the strength of the soil beneath existing airfield pavements without extensively damaging the upper pavement layers and surface.

REPORT OBJECTIVE

The objective of this report is to establish a technical data base detailing that soil stabilization technology having direct application to stabilizing deep soil layers in pavement systems without disturbing the pavement surfacing and base courses. Existing state of the art knowledge is reviewed and evaluated along with several new concepts of possible merit.

REPORT ORGANIZATION

A general evaluation of the deep-layer stabilization problem is discussed in Section II. Attention is focused on pavement structural behavior.

Basic approaches that may possibly provide a subgrade strength equivalent to a CBR of 20 percent are cited in Section III. General concepts of stabilizer distribution are discussed in light of pavement structural behavior.

Section IV contains a detailed review of various stabilization procedures, both current and new. Attention is given to both theoretical and laboratory findings and practical field application. Benefits and limitations of each technique are also reviewed.

The techniques discussed in the previous section are evaluated in Section V, in light of project objectives.

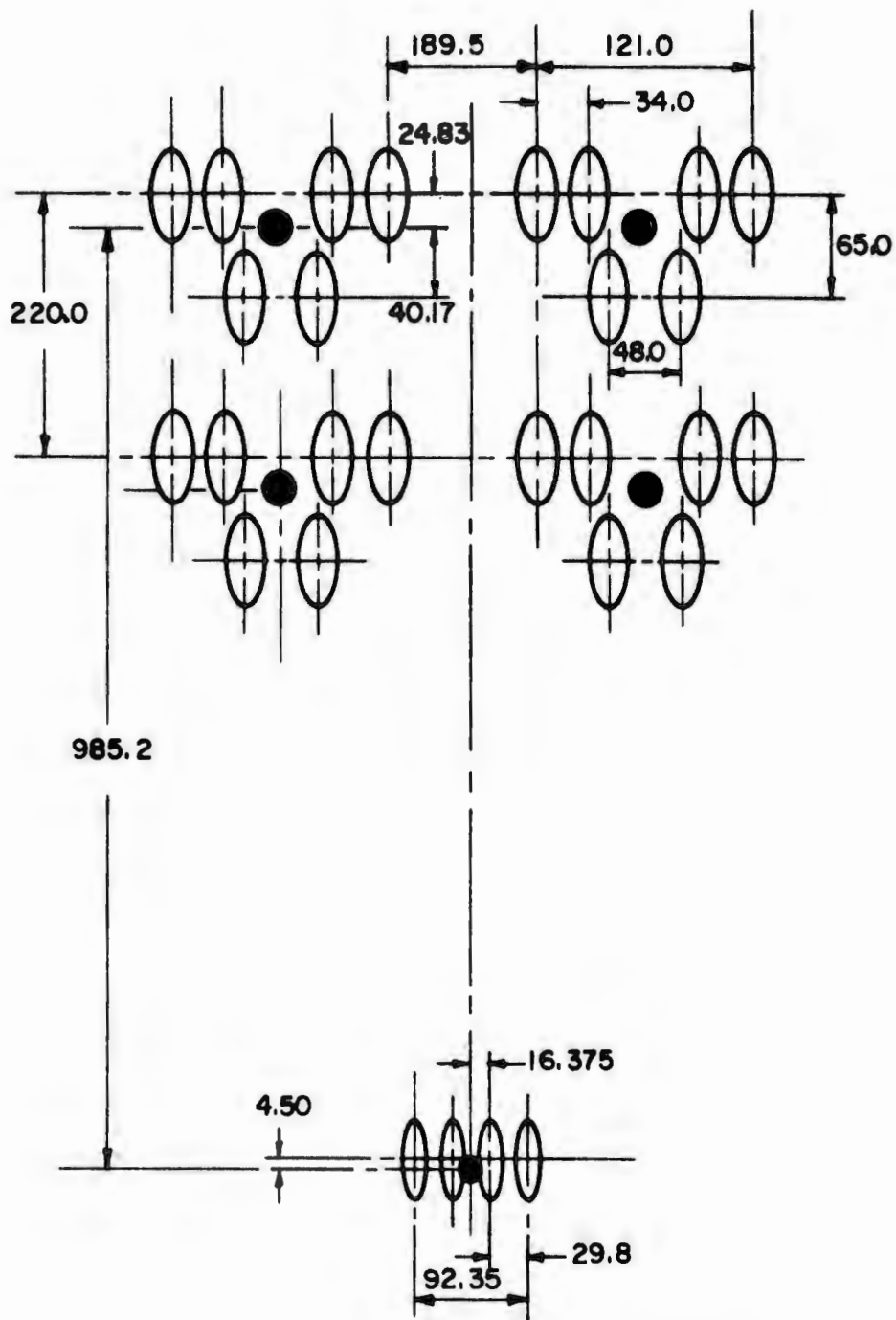
SECTION II
EVALUATION OF THE PROBLEM
GENERAL

It is anticipated that the typical subgrade soils beneath most existing inadequate pavements will probably be fine-grained materials with limited in-situ strength. Such subgrades obviously will be the most difficult to improve. The Air Force Weapons Laboratory (AFWL) has indicated that the technique developed for stabilizing the in-situ subgrade soils beneath the pavement structure should be capable of increasing the subgrade strength from a typical in-situ condition of CBR 2-4 to a strength equivalent to a CBR 20 subgrade. Based on Corps of Engineer criteria (1), CBR 4 corresponds to an unconfined compressive strength of approximately 25 psi and a CBR of 20 corresponds to an unconfined compressive strength of approximately 100 psi. Thus, an overall compressive strength increase of approximately 75 psi must be effected in the subgrade soil. It is assumed that any type of stabilization procedure which will provide pavement structural performance equivalent to that obtained from a pavement overlying a CBR 20 subgrade would also be an acceptable treatment procedure.

PAVEMENT STRUCTURAL BEHAVIOR

In order to determine the best type(s) of stabilization system for achieving subgrade strengthening and accompanying improved pavement performance, it is essential to examine the general behavior of typical flexible and rigid pavements when subjected to wheel loadings similar to those imposed by the C-5A. Theoretical analyses were conducted using a layered elastic system procedure. For general purposes, a wheel load of 30 kips and a tire pressure of 100 psi were assumed to be typical for the C-5A. A sketch showing the landing gear configuration of the C-5A is presented in Figure 1 (2).

The rigid pavement selected for evaluation was a 12-in. concrete slab overlying a CBR 4 subgrade. The concrete was assumed to have a modulus of elasticity of 3×10^6 psi, and a Poisson's ratio of 0.15. The CBR 4



**Main Landing Gear Configuration - Twin Delta Tandem
Nose Landing Gear - Twin - Twin**

Figure 1 C-5A Landing Gear Configuration (From Ref. 2)

subgrade was assumed to have a modulus of elasticity of 5,000 psi and a Poisson's ratio of 0.50.

The flexible pavement considered consisted of 3 in. of asphaltic concrete, 6 in. of base course aggregate, and 15 in. of subbase aggregate prepared for a CBR 4 subgrade. Cross sections of the rigid and flexible pavements and the assumed properties of the materials in the various pavement layers are shown in Figure 2.

The rigid and flexible sections considered above were actual test items included in the rigid and flexible pavement test evaluations sponsored by AFWL at the Waterways Experiment Station (WES), Vicksburg, Mississippi.

Pavement system response (vertical stress, deflection, radial stresses and strains) for one wheel load placed on the rigid or flexible pavement surface is depicted in Figures 3 and 4 for various depths beneath the surface of the pavement and for various offset distances from the center line of the wheel. Since the basic analysis assumed an elastic system, the pavement response effects of other wheels in the landing gear configuration can be determined by algebraically adding their effects to the stresses, displacements, etc. associated with the wheel initially considered.

Several significant points should be noted relative to the data presented in Figures 3 and 4.

1. Significant stresses and deflections are induced in the subgrade soil at substantial offset distances and depths from the point of wheel load application. For example, in the rigid pavement sections the deflection (@ $Z = 96$ in.) is 0.023 in. for $R = 0$ and 0.011 in. for $R = 150$ in. The surface deflection for this pavement (@ $Z = 0$, $R = 0$) is 0.034 in.
2. The surface deflection of both the rigid and flexible sections are quite large.
3. A high percentage of the surface deflection of the pavement system ($R = 0$) is contributed by the subgrade at depths below 96 in. (68 percent in the rigid and 21 percent in the flexible).

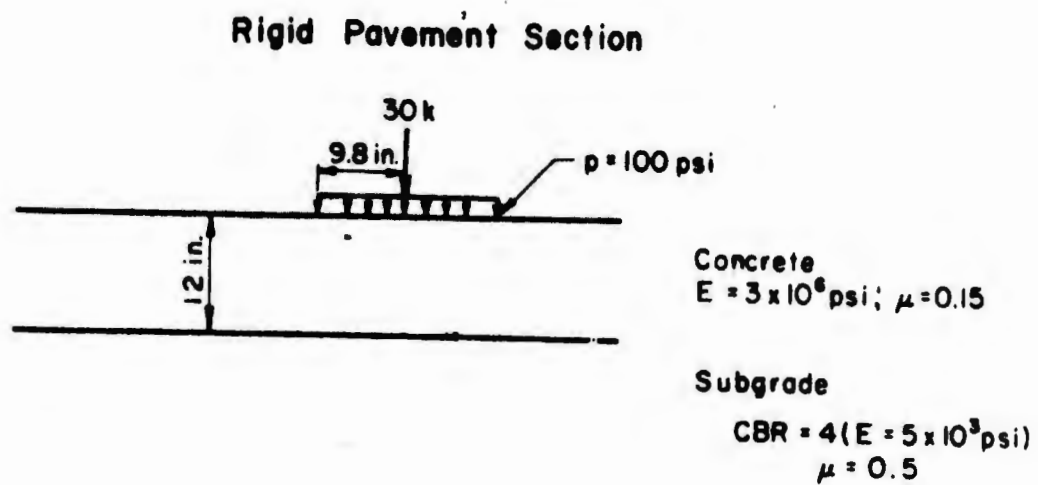
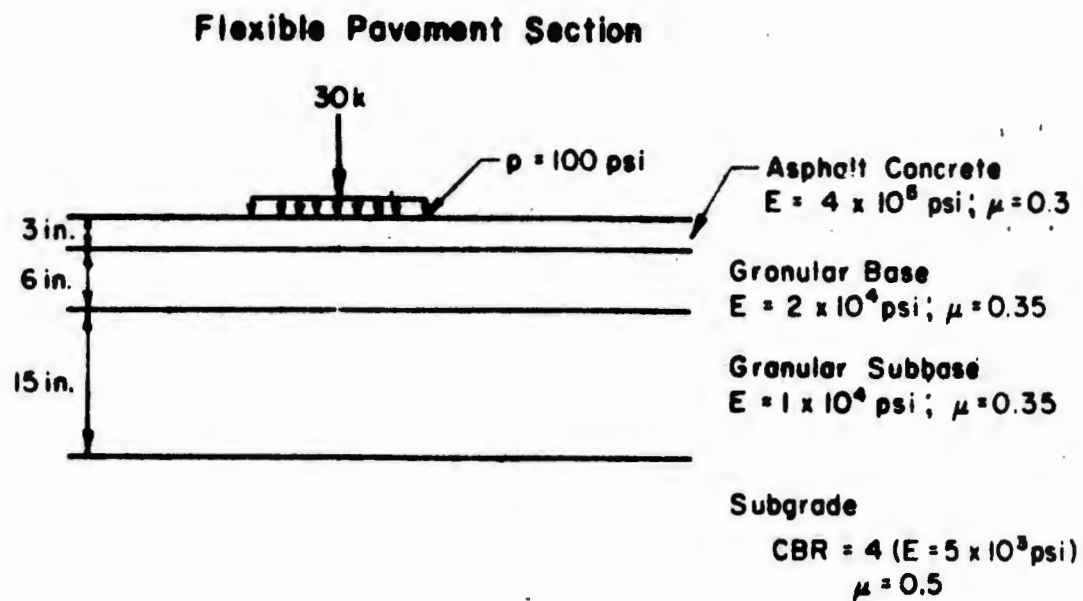


Figure 2 Flexible and Rigid Pavement Cross Sections and Assumed Material Properties

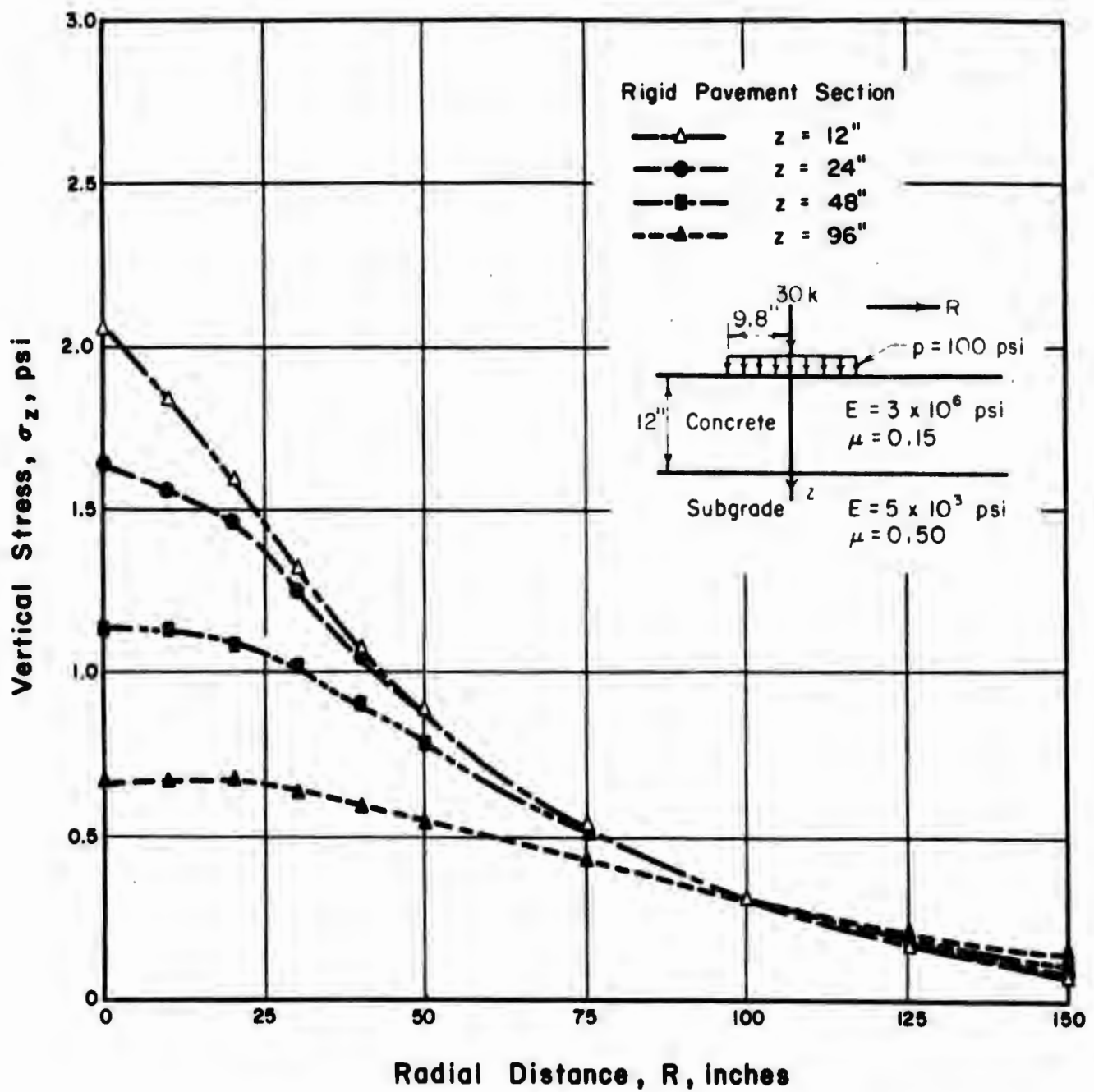


Figure 3a Rigid Pavement-Effect of Radial Offset Distance on Vertical Stress at Various Depths

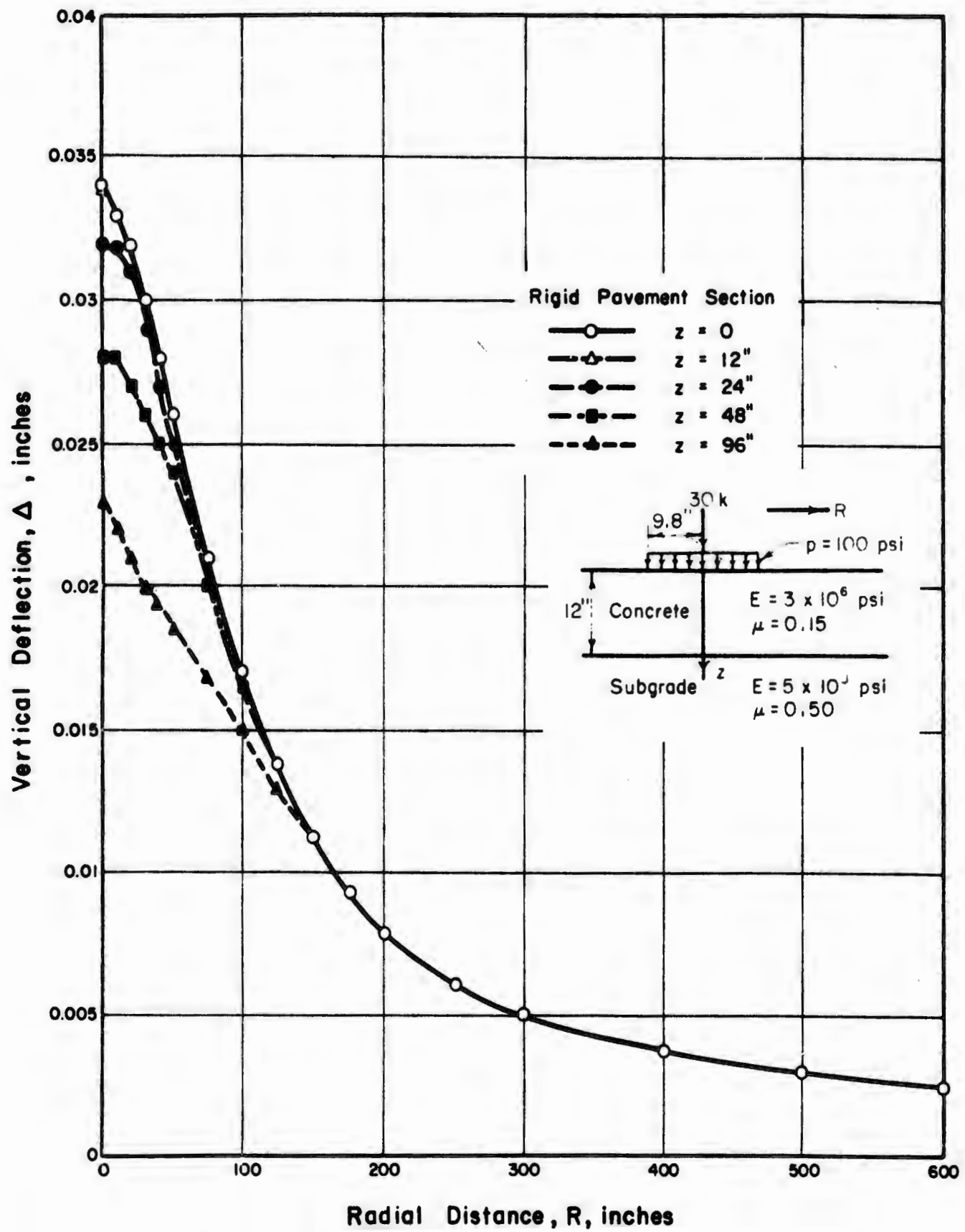


Figure 3b Rigid Pavement-Effect of Radial Offset Distance on Vertical Deflection at Various Depths

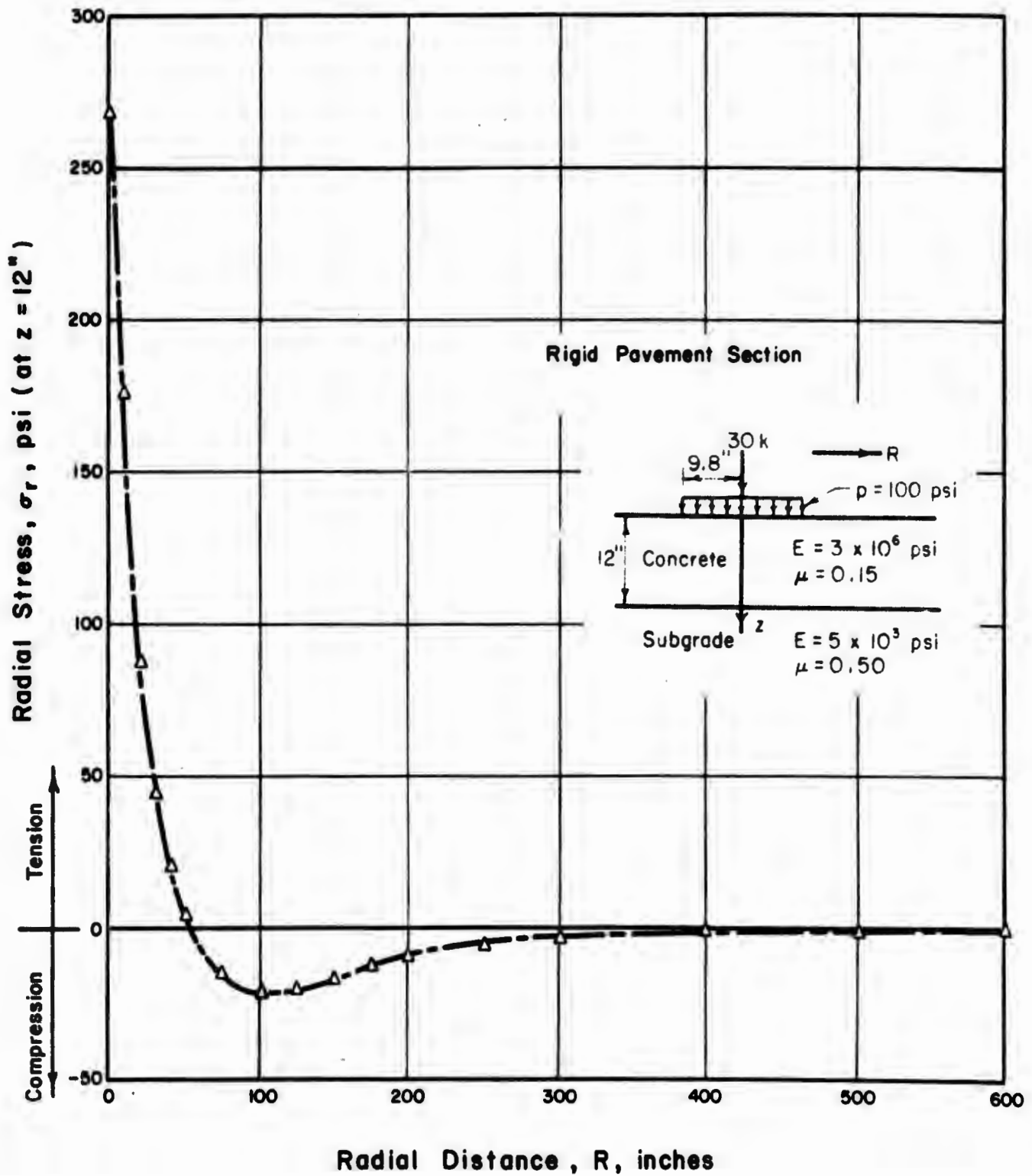


Figure 3c Rigid Pavement-Effect of Radial Offset Distance on Radial Stress at a Depth of 12 Inches (Z = 12 Inches)

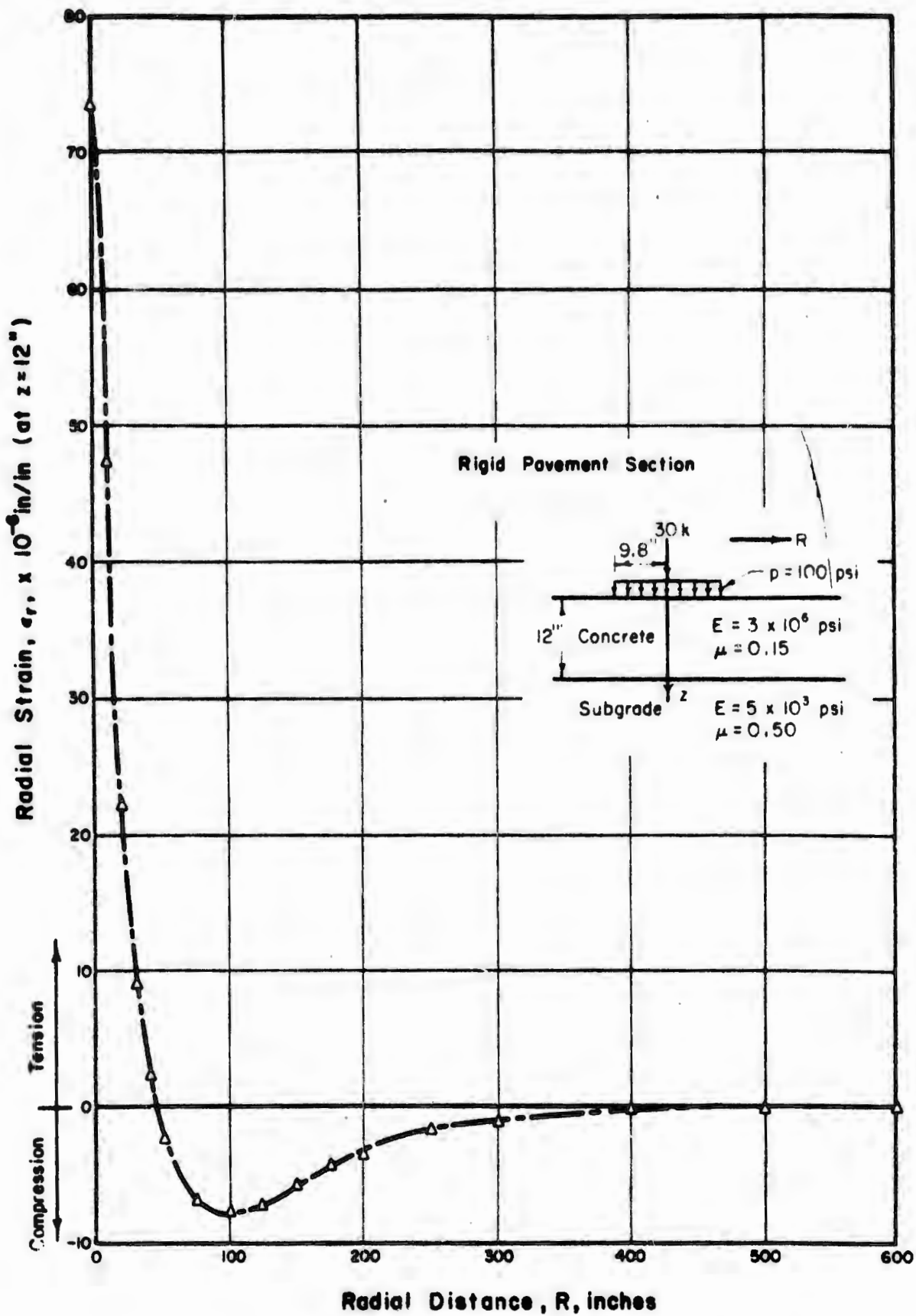


Figure 3d Rigid Pavement-Effect of Radial Offset Distance on Radial Strain at a Depth of 12 Inches (Z = 12 Inches)

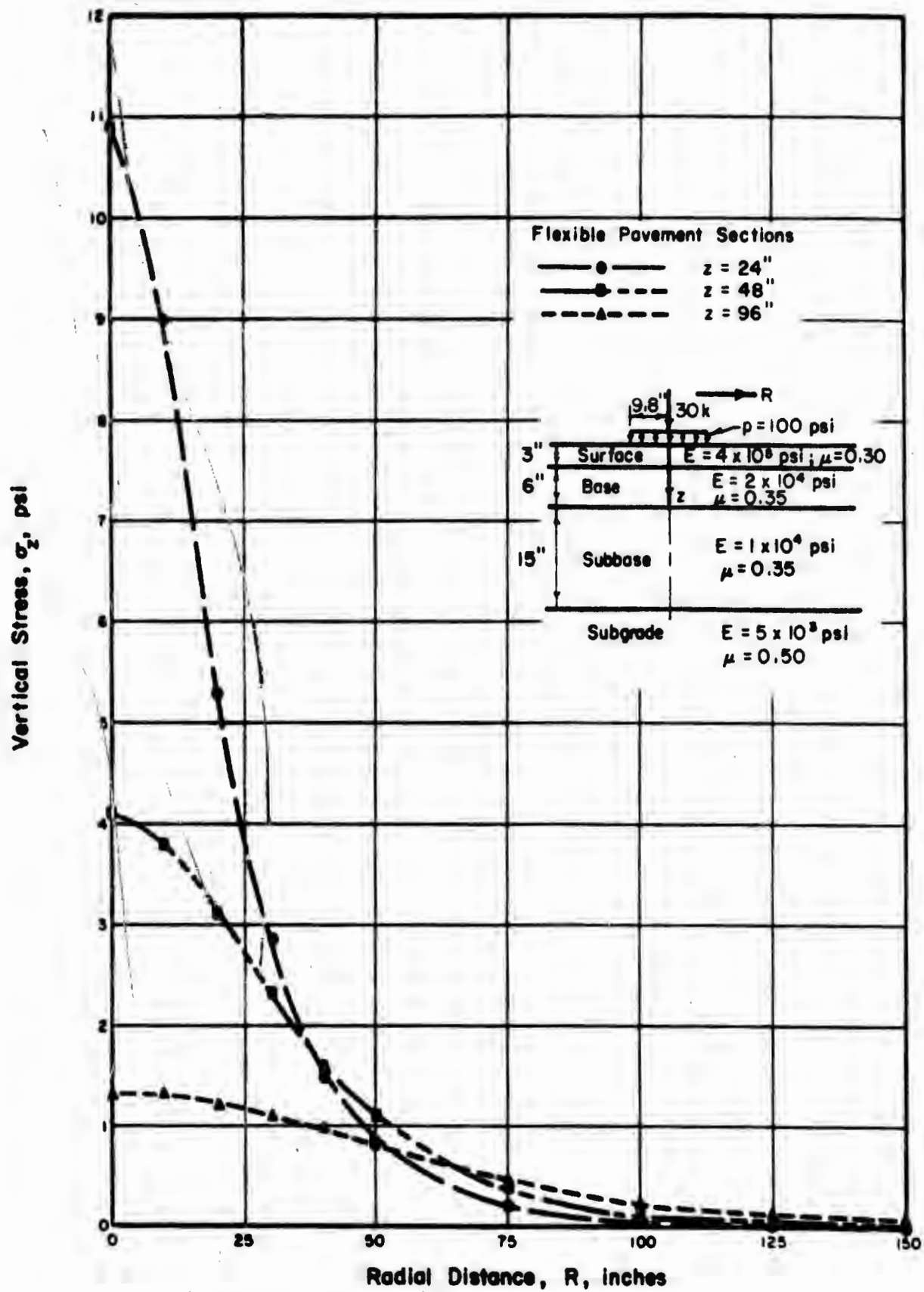


Figure 4a Flexible Pavement-Effect of Radial Offset Distance on Vertical Stress at Various Depths

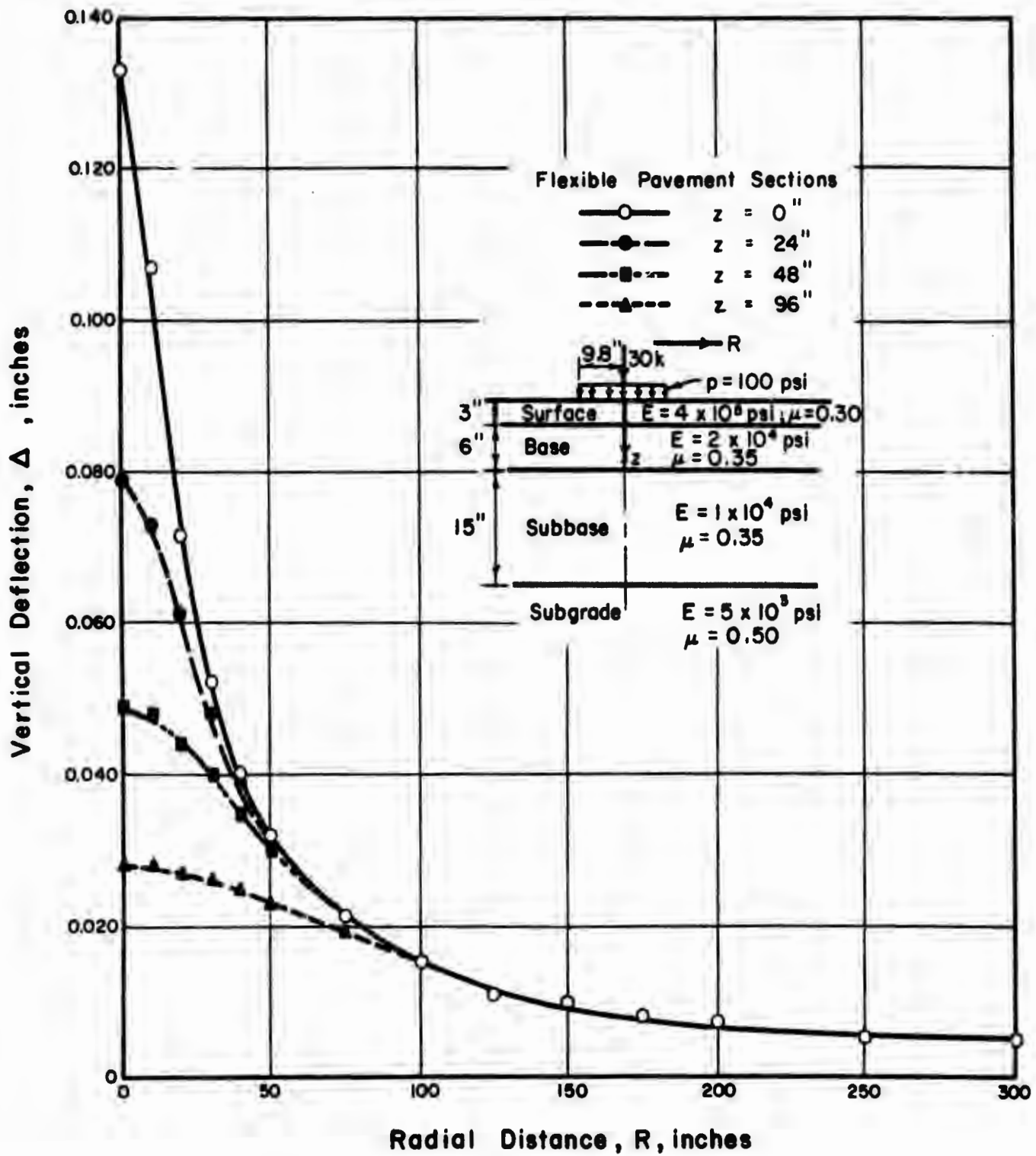


Figure 4b Flexible Pavement-Effect of Radial Offset Distance on Vertical Deflection at Various Depths

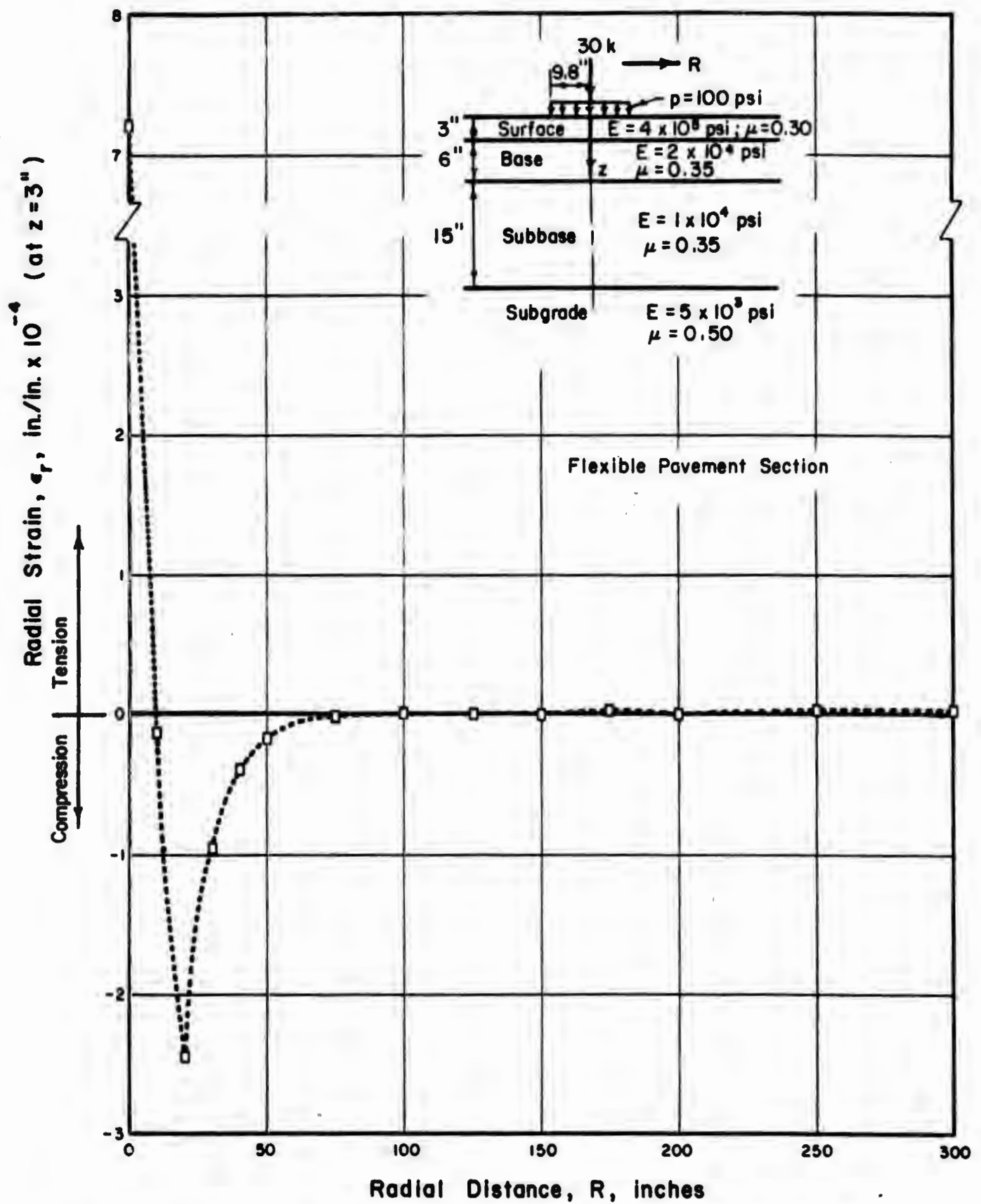


Figure 4c Flexible Pavement-Effect of Radial Offset Distance on Radial Strain at a Depth of 3 Inches ($Z=3.0$ Inches)

The rigid pavement system in particular displays this type of behavior.

4. The vertical compressive subgrade stresses beneath the flexible pavements and the radial tensile concrete stresses in the rigid pavements are quite high. If the effect of the other wheels in the gear configuration are considered, these stresses would further increase.

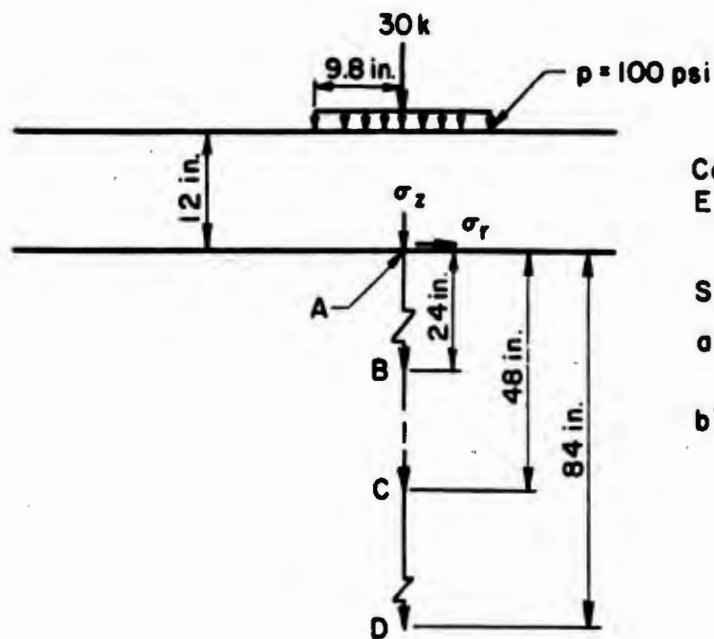
Structural pavement behavior response effected by increasing subgrade support to CBR 20 for the rigid and flexible pavements is indicated in Figures 5 and 6. The loading conditions were a 30-kip wheel load and 100-psi tire pressure. All stresses, deflections, etc. are for the center of the loaded area ($R = 0$). When subgrade support is increased, surface deflection is greatly decreased for the rigid pavement and the radial tensile stress in the concrete also is reduced. Surface deflection of the flexible section is substantially reduced, but the radial tensile strain in the asphalt concrete surface course, which closely relates to fatigue life and pavement performance, is approximately the same. Increased subgrade strength and stiffness effect an increase in vertical stresses in the subgrade mass for both the rigid and flexible pavements.

PERFORMANCE CRITERIA AND OVERLAYS

Performance criteria for pavements subjected to "Jumbo Jet" wheel loadings have not been established and validated. Vallerga and McCullough (3) have suggested the use of radial tensile strain in the asphalt concrete and vertical subgrade strain as design criterion for flexible pavements and a concrete flexural stress criterion for rigid pavements. Corps of Engineers' philosophy for flexible pavement design to accommodate "Jumbo Jet" loadings is based on the "cover thickness" concept that has historically been utilized by the Corps (4). Total thickness requirements based on the Corps' philosophy are quite large for low CBR subgrades as indicated by Figure 7.

Failure modes and mechanisms for "Jumbo Jet" loadings have not been well defined. Limited data from the WES Test Sections indicate that pumping and joint problems may be of great importance in rigid pavement

Rigid Pavement Section



Concrete
 $E = 3 \times 10^6 \text{ psi}$; $\mu = 0.15$

Subgrade

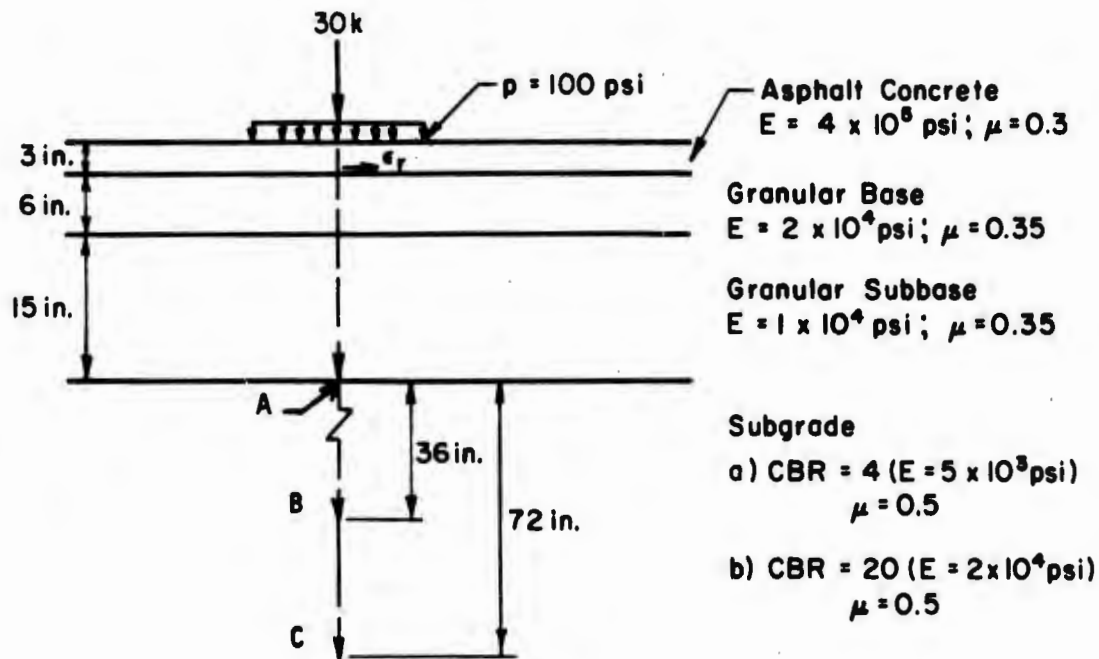
a) CBR = 4 ($E = 5 \times 10^3 \text{ psi}$)
 $\mu = 0.5$

b) CBR = 20 ($E = 20 \times 10^3 \text{ psi}$)
 $\mu = 0.5$

Subgrade Support CBR	Surface Deflection, in.	Radial Stress, σ_r , psi	Vertical Stress, σ_z , psi			
			A	B	C	D
4	.034	258	2.1	1.4	1.0	0.7
20	.013	209	5.2	2.8	1.8	1.0

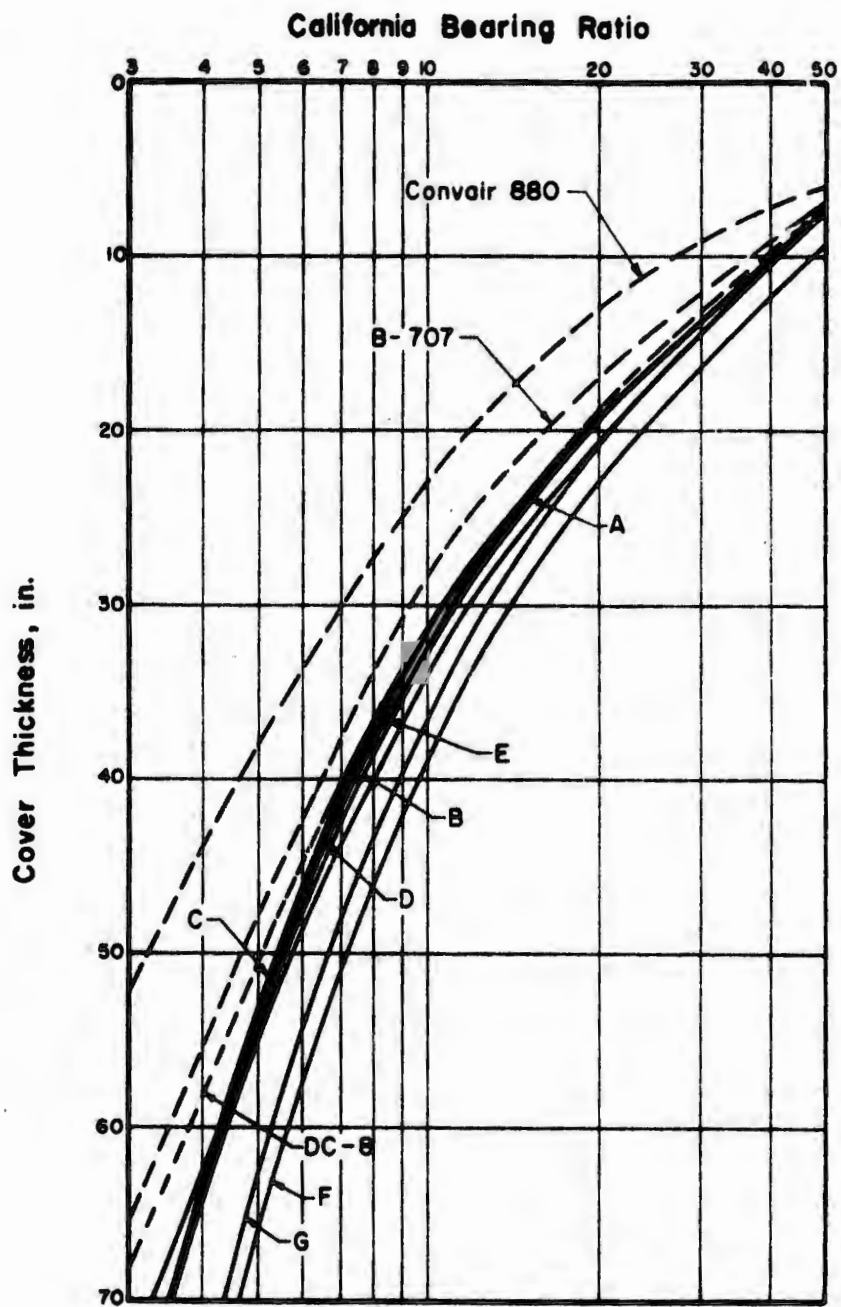
Figure 5 Subgrade Support Effect on Rigid Pavement Structural Response

Flexible Pavement Section



Subgrade Support, CBR	Surface Deflection, in.	Radial Strain in Surface Layer	Vertical Stress, σ_z , psi		
			A	B	C
4	.133	7.2×10^{-4}	10.9	2.9	1.3
20	.083	7.0×10^{-4}	17.4	4.0	1.6

Figure 6. Subgrade Support Effect on Flexible Pavement Structural Response



<u>Aircraft</u>	<u>Gross Load, kips</u>
Convair 880	188
Boeing 707	296
Douglas DC-8	310
A	350
B	367
C	409
D	410
E	683
F	750
G	818

Figure 7 Corps of Engineer's Cover Thickness Requirements for Various Aircraft (From Ref. 4)

design and performance. It is significant to note that the probable influence of deep layer stabilization on joint behavior, pavement faulting, and pumping is extremely difficult to assess. If the Corps' "cover thickness" concept is applicable to flexible pavement performance for "Jumbo Jet" loadings, any improved material (higher CBR than the subgrade) would be suitable for providing cover and protection to the underlying weaker subgrade.

Conventional overlay procedures have been used to upgrade existing pavements to accommodate "Jumbo Jet" loadings (3). Both rigid and flexible overlays have been utilized. Rigid overlay design criterion was flexural stress in the concrete. Criteria adopted for the flexible pavement for the flexible pavement sections were radial tensile strain in the asphalt concrete and vertical compressive strain in the subgrade soil.

The general effects of utilizing an asphalt concrete overlay over the flexible pavement and the basic 12-in. concrete slab previously considered are illustrated in Figures 8 and 9.

Surface deflection, radial strain in the asphalt concrete, and vertical subgrade compressive stress are substantially reduced by increasing the overlay thickness for the flexible section.

Radial tensile stresses in the portland cement concrete are considerably reduced by increased thickness of asphalt concrete overlay for the rigid pavement and the surface deflections and vertical subgrade compressive stresses are significantly decreased but not to the same extent as radial tensile stresses. An additional advantage of asphalt concrete overlays over rigid pavements is the reduction of temperature warping stresses in the concrete slab.

The effects of utilizing portland cement concrete to overlay an existing concrete slab are shown in Figure 10. Surface deflection and radial tensile stress in the concrete are substantially reduced. Vertical subgrade stresses at the slab-subgrade interface are quite low, 2 psi for the 12-in. concrete slab and 0.5 psi for the 30-in. slab.

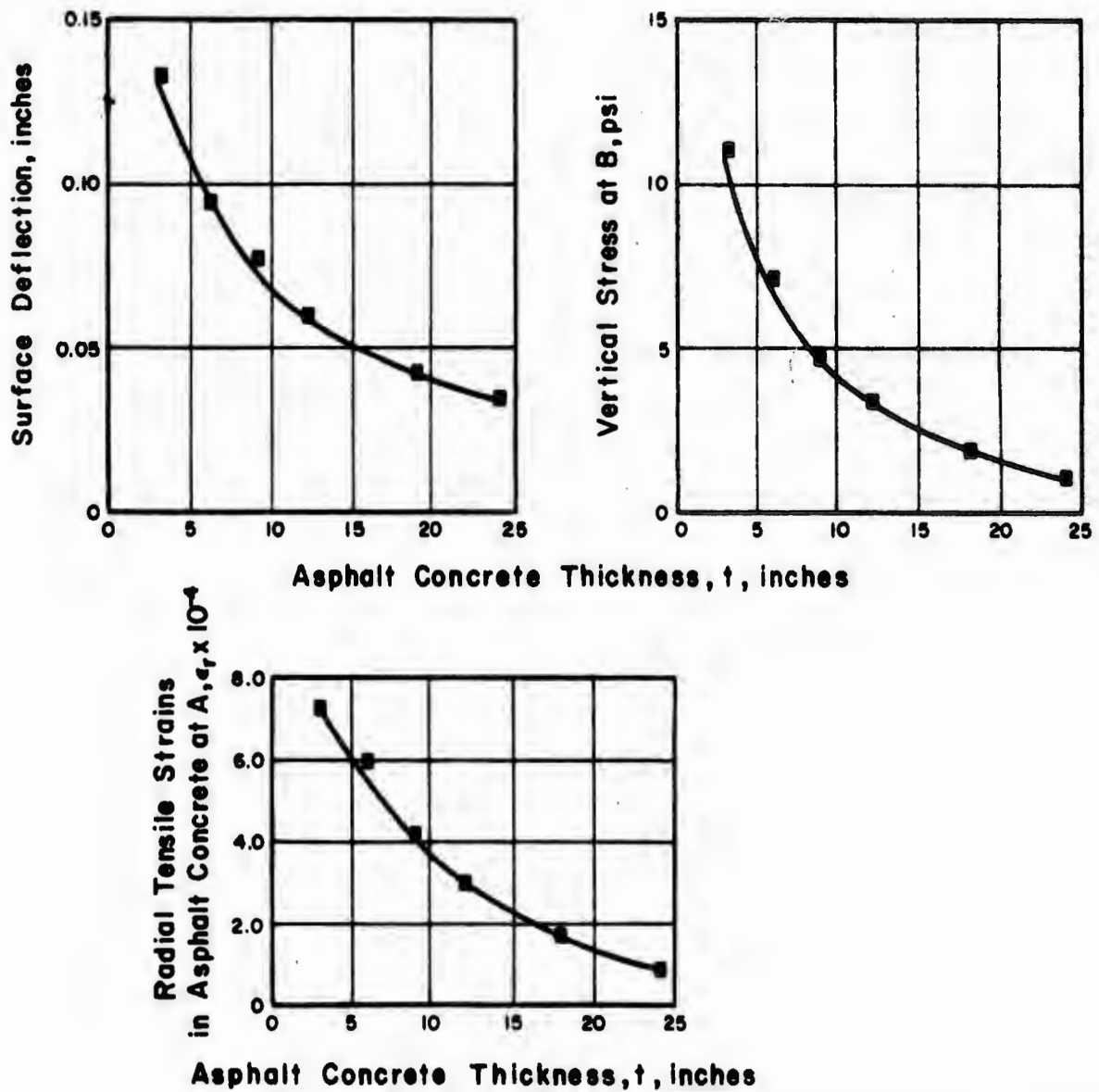
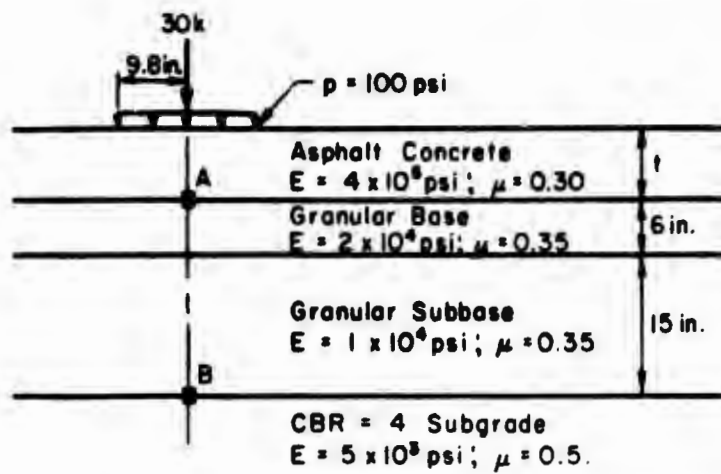


Figure 8 Effect of Using Various Thicknesses of Asphalt Concrete Overlay Over the Basic Flexible Pavement

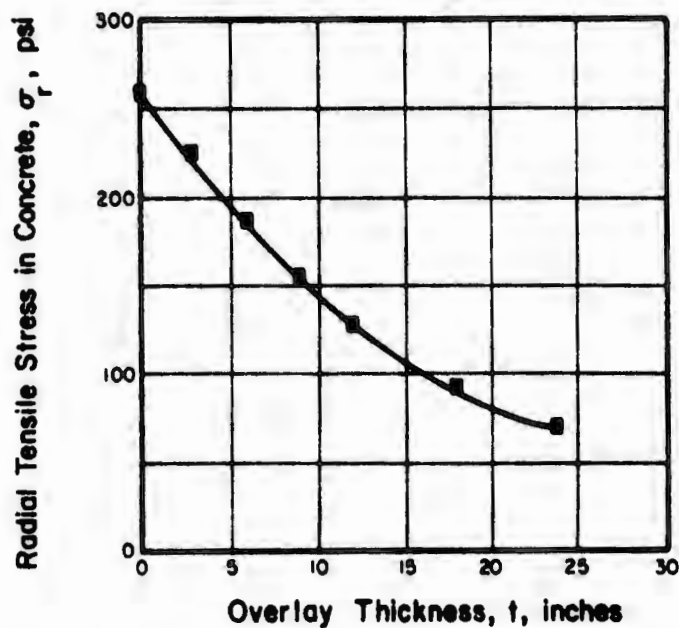
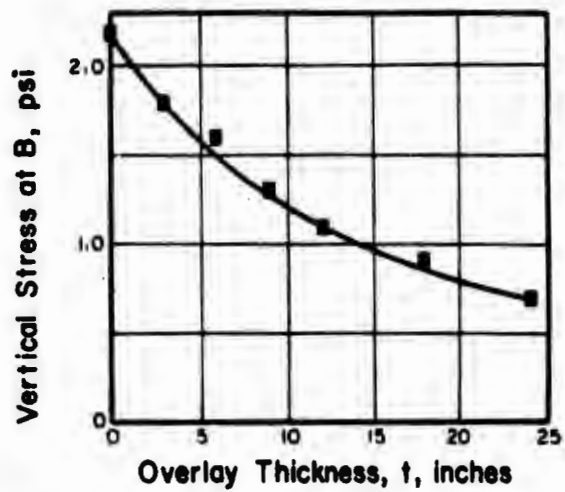
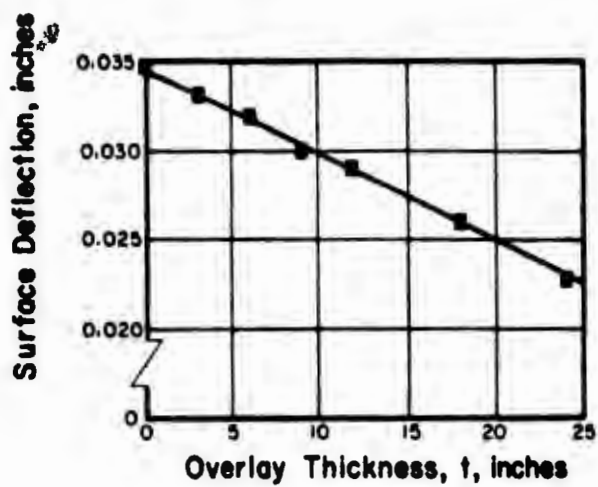
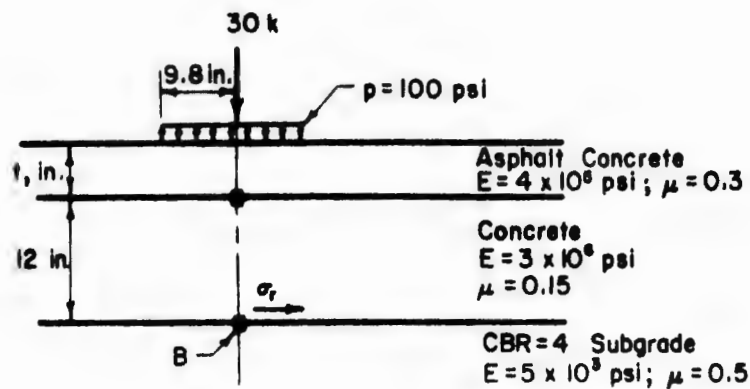


Figure 9 Effects of Using Various Thicknesses of Asphalt Concrete Overlay Over the Basic 12-Inch Concrete Pavement

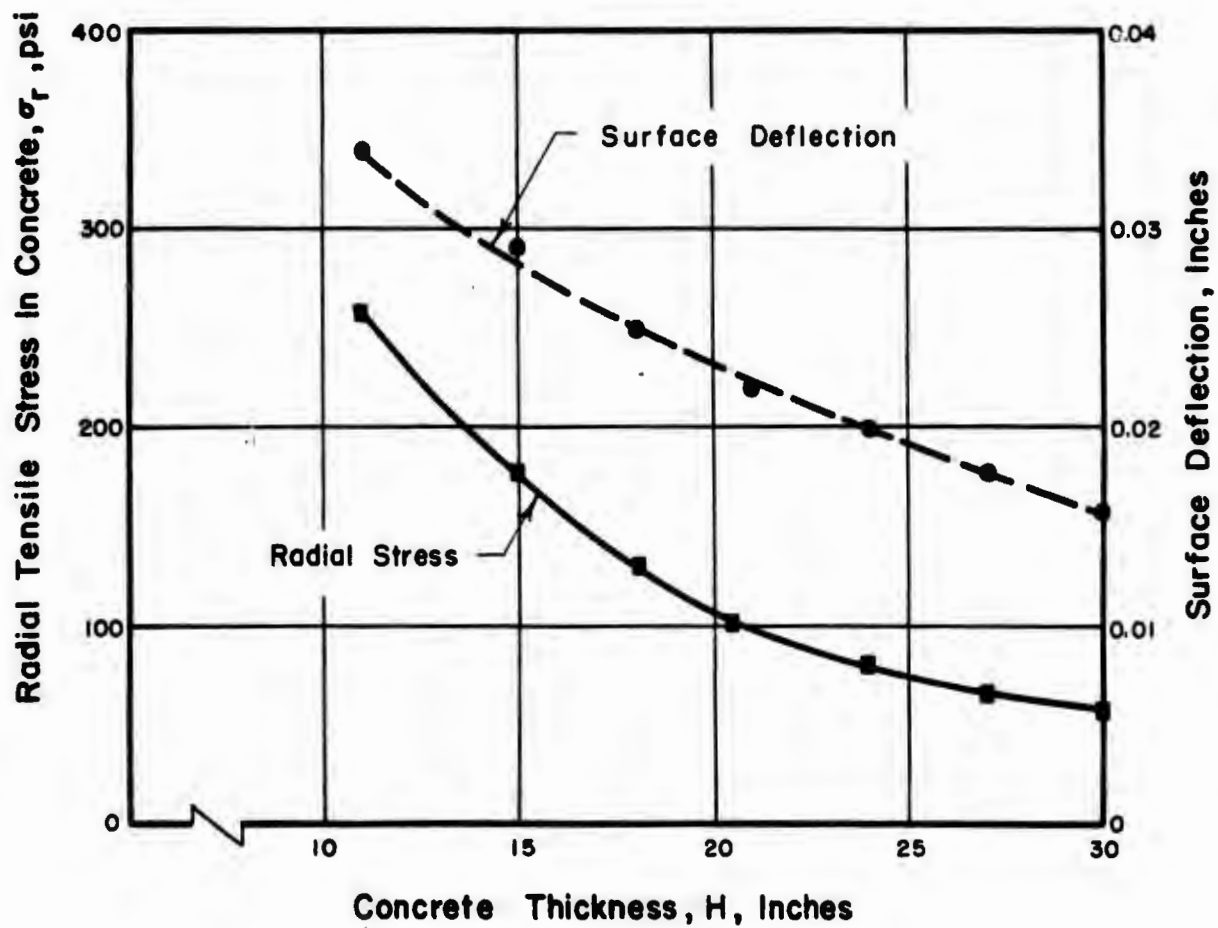
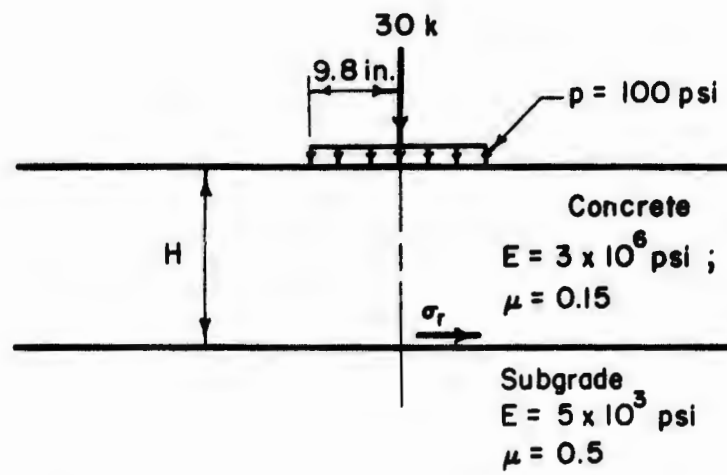


Figure 10 Effects of Using Various Thicknesses of Portland Cement Concrete Overlay Over the Basic 12-Inch Concrete Pavement

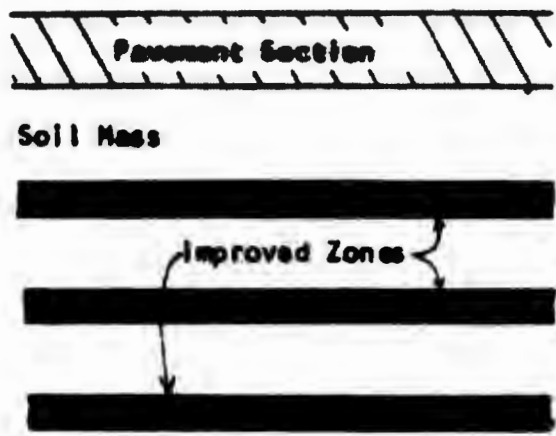
SECTION III
BASIC APPROACHES
GENERAL

The basic requirement for deep layer stabilization is to provide a subgrade mass equivalent to CBR 20 soil. This requirement may possibly be met in two ways. One approach is to stabilize the upper portion of the subgrade mass in a homogeneous fashion. A second feasible concept is to improve zones within the subgrade mass in such a way that the behavior of the "zone-stabilized" subgrade produces pavement structural performance equivalent to that attained from a CBR 20 subgrade. Zone improvement would include stabilizing strategic layers or columns (horizontal, vertical, slanted) or zones within the soil mass, or perhaps constructing mixed-in place stabilized soil columns within the subgrade soil mass, Figure 11.

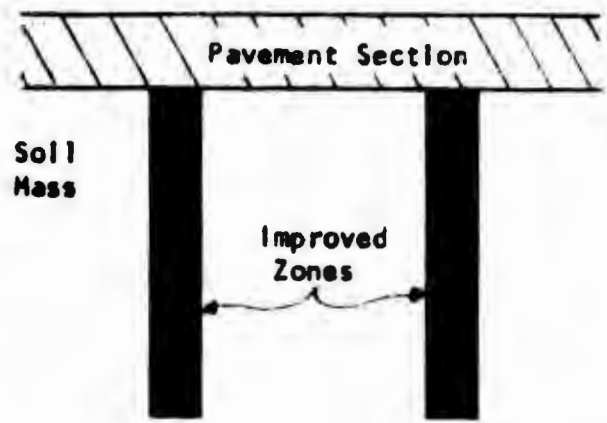
STABILIZER DISTRIBUTION

It is important to note that in the deep layer stabilization concept it is highly desirable to minimize the disturbance of the overlying pavement section. Consequently, mixing and manipulation of the soil and stabilizer may be quite limited. This limitation makes it very difficult to achieve intimate stabilized mixtures within the subgrade system although certain soils may be uniformly permeated by some stabilizers. Consequently it may be difficult to achieve uniform and homogeneous stabilization throughout the subgrade mass.

The problem of uniformly distributing the stabilizing material throughout the fine-grained subgrade mass is very significant relative to accomplishing the objectives of deep layer stabilization. Stabilizing agents can be introduced into the soil mass in several ways, including a) permeation of the soil mass by the flow of a liquid stabilizer through the pores of the soil (injection process), b) migration of stabilizers from zones of high stabilizer concentrations (self-diffusion), c) utilization of electrical procedures for "pulling stabilizers" from point to point within the soil mass, and d) direct introduction of the stabilizer into a zone

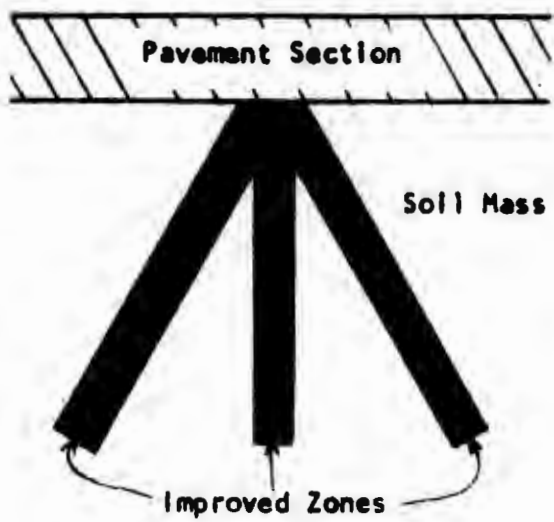


(a) Horizontal Zones

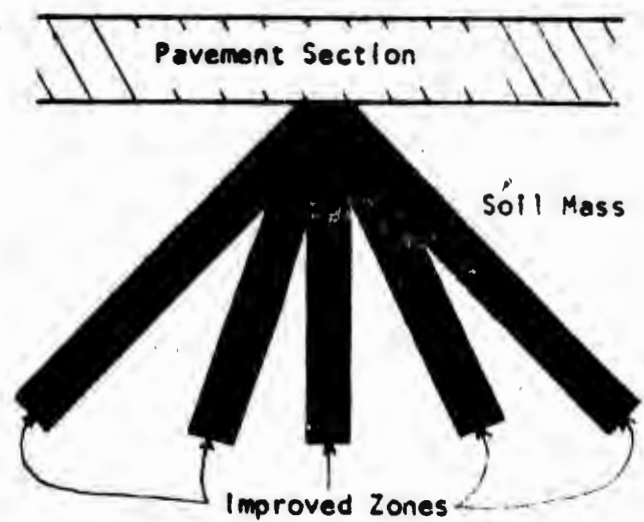


(b) Vertical Columns

NOT REPRODUCIBLE



(c) Vertical Column Plus One Row of Slanted Columns



(d) Vertical Column Plus Two Rows of Slanted Columns

Figure 11 Cross Section of Typical Zone Improvement Configurations

where mechanical mixing is being carried out. Various stabilizer distribution mechanisms may operate independently, or in some cases, perhaps in conjunction with one another. For example, permeation may take place if pressure injection techniques are utilized and migration of the stabilizer may occur following the initial pressure injection placement of the stabilizer. It also might be possible to utilize electrical techniques to accelerate and facilitate the movement of the stabilizer after placement. (See Electrical Methods, Section IV.)

Permeation Process

In order to examine the ability of various stabilization procedures such as the pressure grouting of cement and lime-water slurries, chemical grouts, etc. to intimately permeate a soil mass, it is helpful to examine the general nature of permeation. The theory of permeation (5) is normally discussed in terms of an ideal fluid. It is emphasized that many of the liquid stabilizers considered in this study are in reality a suspension of particles in a fluid transporting medium and thus will not exactly behave as an ideal fluid.

The relative ability of a soil mass to accept a fluid is often represented by a coefficient of permeability defined by the following equation:

$$k = \frac{\rho g}{\eta} K \quad (1)$$

where

ρ = density of fluid

η = viscosity of fluid

g = acceleration due to gravity

K = intrinsic permeability of medium; $K = Cd^2$,

C is a shape factor

d is average pore size of medium

k = coefficient of permeability.

Examination of Equation 1 reveals that as the viscosity of the fluid increases the coefficient of permeability decreases.

Relative to permeating a soil mass, it is desirable to be able to determine the rate of fluid flow into or through the soil mass. Darcy's equation, Equation 2, can be used to determine the velocity of flow:

$$V = k \frac{p}{l} \quad (2)$$

where

V = velocity of fluid flow

k = coefficient of permeability

p = pressure head

l = length over which pressure head acts.

The total quantity of fluid forced into a porous media can be determined by multiplying the velocity of fluid flow by the cross sectional area over which the pressure is applied and the length of time during which the pressure is sustained, Equation 3:

$$Q = V A t \quad (3)$$

where

Q = quantity of fluid flow

A = cross sectional area over which pressure acts

t = time of pressure application.

From examination of Equation 3, it becomes apparent that an increased quantity of fluid flow occurs as:

1. the viscosity of the fluid is reduced,
2. the pressure is increased,
3. the coefficient of permeability is increased, and/or
4. the time of pressure application (injection) is increased.

The distance to which the porous media has been permeated by the fluid after a given time of applied injection pressure is an important consideration. Karol (6) has suggested the use of the following equation for determining the radial travel distance of chemical grout:

$$r = 0.62 \sqrt[3]{\frac{Rqt}{n}} \quad (4)$$

where

- r = radial distance, ft
- R = ratio of water to grout viscosity
- g = rate of grout take, ft³/min.
- t = gel time, minutes
- n = soil porosity.

Diffusion-Migration Process

Diffusion or migration (used synonymously in this report) is a process by which ions, atoms, or molecules move randomly from one position to another (7). This process is analogous to the heat flow phenomena through a solid media, although diffusion is generally caused by a concentration gradient (8). The diffusion process can occur within a single phase or can occur between phases. Diffusion of gases in liquids or solids and diffusion of liquids in solids are examples of interphase diffusion while diffusion of gases within a gas or diffusion of liquids within a liquid are examples of single phase diffusion.

a) General Theory

Various attempts have been made to mathematically represent the diffusion process. Fick's first law, represented by Equation 5, has been suggested (8) for defining one-directional movement of a substance over a concentration gradient:

$$J = -D \frac{\partial c}{\partial x} \quad (5)$$

where

- J = quantity that flows per unit time
- D = diffusion coefficient
- $\frac{\partial c}{\partial x}$ = concentration gradient.

For steady state diffusion, the diffusion process can be represented by Equation 6 (9):

$$J = -D \frac{A}{L} (C_2 - C_1) \quad (6)$$

where

A = cross sectional area at right angles to the concentration gradient

L = length over which diffusion is occurring

C_1, C_2 = concentrations at either end of the specific length L.

The coefficient of diffusion, D, is often assumed to be approximately constant for a given medium at a given temperature and pressure although this is only approximately true (8). Talme (9) has used Equation 6 in connection with laboratory experiments to determine the coefficient of diffusion of various salt solutions in soil materials.

b) Diffusion in Clay-Water Systems

In deep layer stabilization applications, the diffusion process in clay-water systems is of primary concern. The movements of ions in soils have been divided into four groups according to mechanism (10): a) free diffusion of ion pairs in the intermicellar pores and channels, b) absorption of ions by colloidal particles which themselves diffuse as a result of Brownian movement or with the aid of some transporting agent, c) exchange of ions between surfaces and intermicellar liquids, and d) surface migration and contact exchange independent of the nature of the intermicellar liquid.

The diffusion process in clay-water systems is affected by the interactions between the clay particles and exchangeable cations. Lai and Mortland (7) state that the diffusion of cations within a clay-water system is an extremely complex process. Diffusing cations have been found to move by at least two different mechanisms: a) movement along the exchange sites on clay particle surfaces and jumping from exchange sites on one particle to exchange sites on adjacent particles; and b) diffusion of the cations through the pores of the clay-water system. The latter mechanism provides a faster rate of diffusion. Diffusion of anions takes place exclusively within the pores (7). However, if the soil is not saturated, diffusion through the pores may be greatly hindered.

Investigators thus far have succeeded only in determining the apparent diffusion coefficients of cations in various soil-water systems (7). Letey and Klute (11) have determined the apparent mobility of potassium and chloride ions in three different soil types and Talme (9) has determined the diffusion coefficient for a calcium chloride solution in various clay soils. Lai and Mortland (7) indicate that different diffusion coefficients may be acting in the clay-water system at the same time. Anions may have a different diffusion coefficient than cations (7). Factors such as differences in clay content, clay minerals, density, absorbed cations, and temperature have been found to affect the coefficient of diffusion and thus the rate of diffusion (12).

Davidson, Demirel, and Handy (12) have suggested that the diffusion of calcium cations in a soil-lime-water system is an example of the diffusion phenomena which is classified as a boundary process. The boundary process accompanying lime diffusion may include: a) transfer of lime into the soil, b) a chemical reaction between the lime and the soil, c) formation of nuclei and growth of the reaction product, and d) further diffusion of the lime into the soil from the reaction product layer (12).

Assuming a constant diffusion coefficient and a constant cross sectional area of diffusion layer the following expression has been suggested for determining the rate of growth of a product layer from the lime-soil reaction (12,13):

$$l = k_d \sqrt{t} \quad (7)$$

where

- l = distance of lime migration for a time t , in.
- k_d = diffusion constant, in./day
- t = elapsed time of diffusion, days.

Reported values of k in Equation 7 range from 0.081 to 0.63 in./day (12,13). If a value of 0.10 in./day is used, the required time for various distances of diffusion are as follows:

<u>l</u> , inches	<u>t</u>
1	100 days
6	3600 days
12	40 yrs

EFFECT OF SUBGRADE TREATMENT ON STRUCTURAL BEHAVIOR

Pavement structural behavior response effected by increasing the stability (higher strength, greater stiffness) of the upper portion of the subgrade beneath the rigid and flexible pavement sections is indicated in Figures 12 and 13. The modulus of elasticity of the stabilized zone was assumed to be 50,000 psi (a typical value for quality lime-soil mixtures) and the thickness of uniform stabilization was varied from 0 to 240 in. The layered elastic analysis program was utilized for analysis purposes.

The influence of stabilizing various horizontal layers within the subgrade mass beneath the flexible and rigid pavement sections was also determined. Twelve-inch incremental zones in the subgrade were assumed to include a stabilized layer (located at the top of each incremental zone) of variable thickness (2, 4, 8, and 12 in.). The 12-in. stabilized layer corresponded to uniform stabilization throughout the depth of the layer. The stabilized material was assumed to have a modulus of elasticity of 50,000 psi. Typical results for various numbers of incremental zones (12-in. thickness) and different thicknesses of stabilized layers are presented in Figures 14 and 15.

The data indicate:

1. Uniform (full depth) stabilization is more effective than the use of the stabilized layers.
2. Both stabilization concepts (uniform stabilization and stabilized layers) altered, to varying degrees, the structural response (surface deflection, concrete radial tensile stress) of the rigid pavement section. Increasing uniform stabilization thickness beyond 60 in. did not effect further reductions in radial tensile stress.

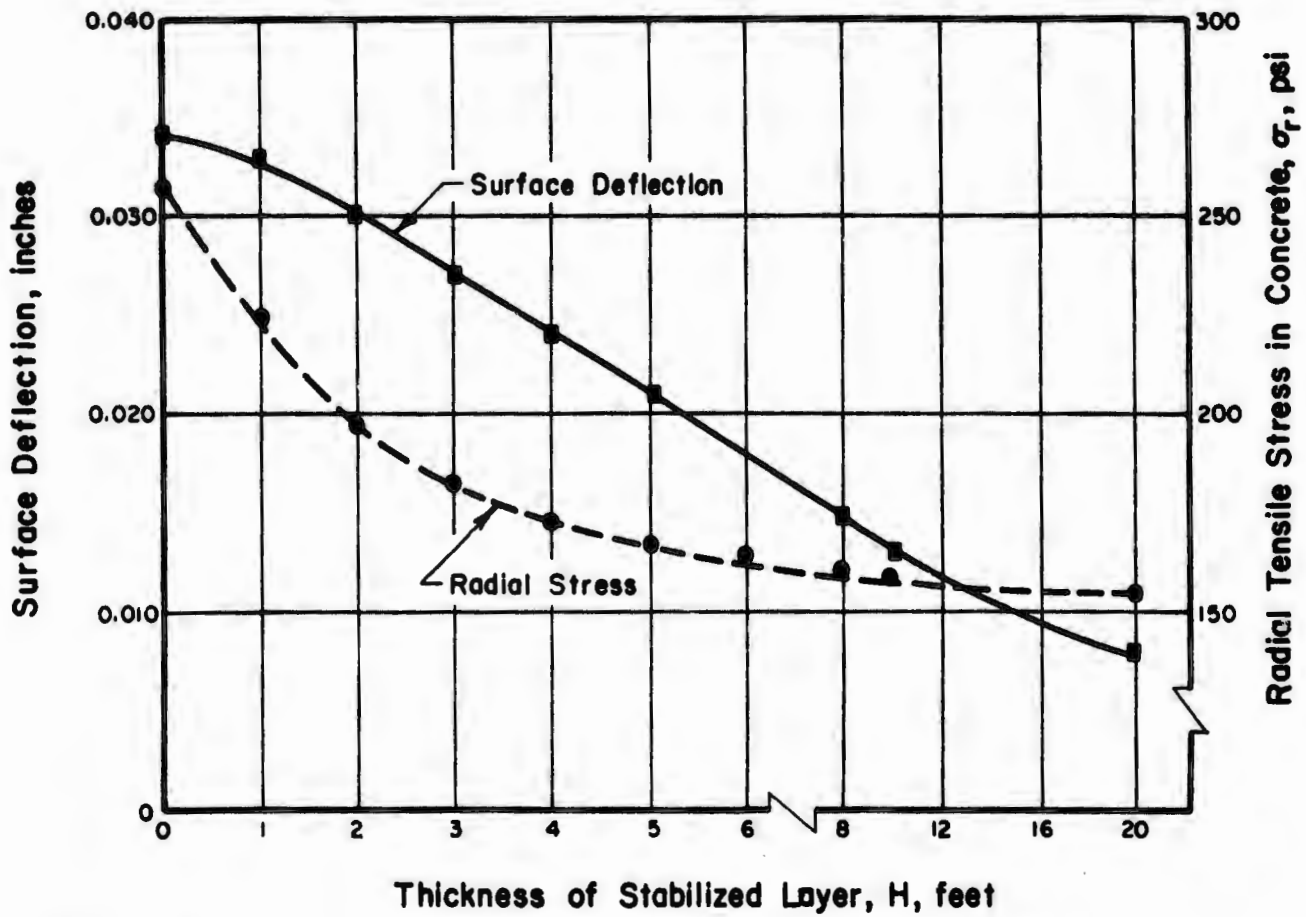
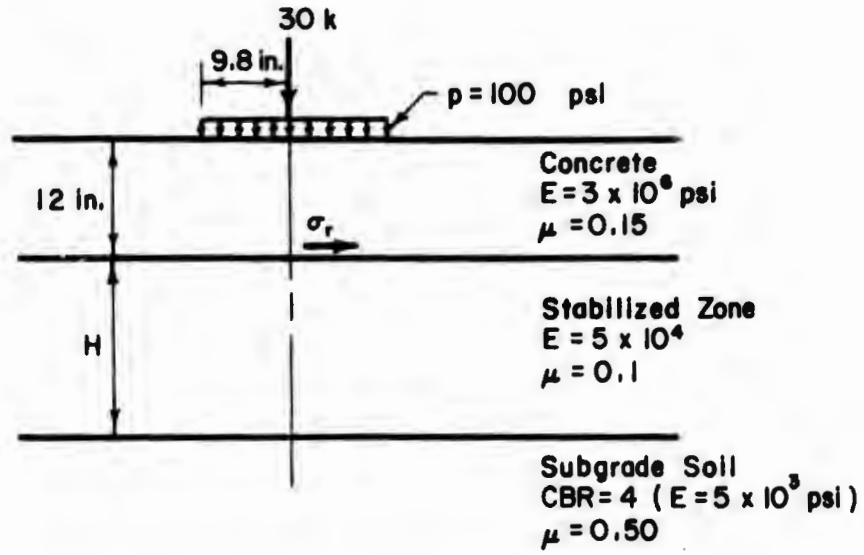


Figure 12 Effect of Stabilizing Upper Portion of Subgrade on Rigid Pavement Structural Behavior Response

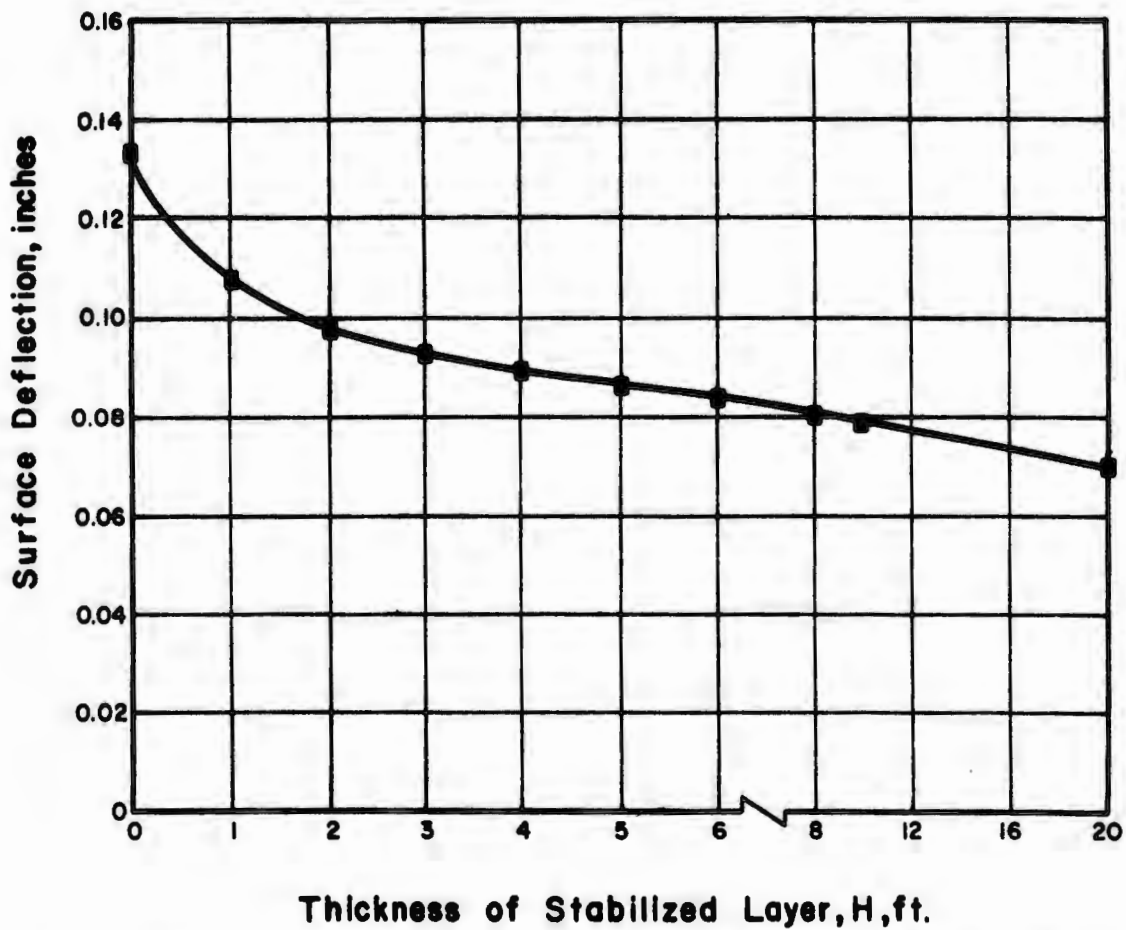
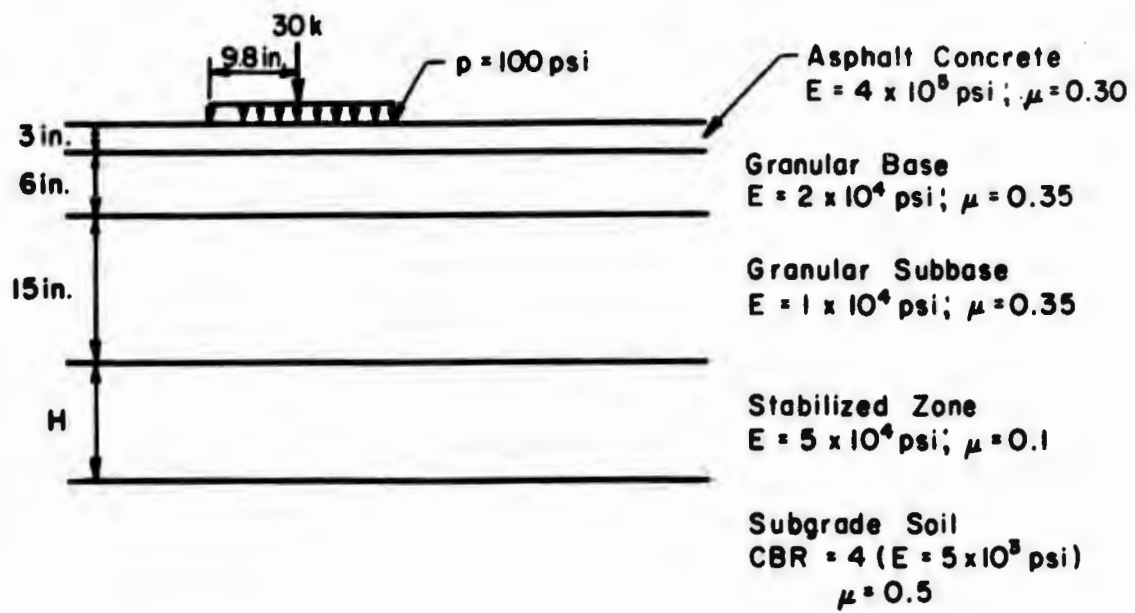
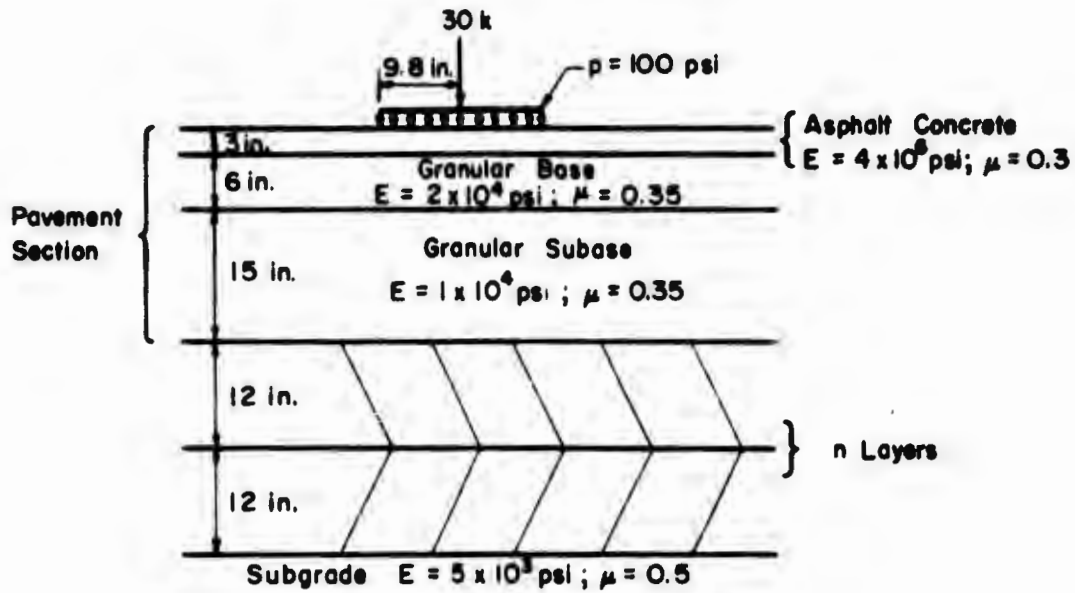
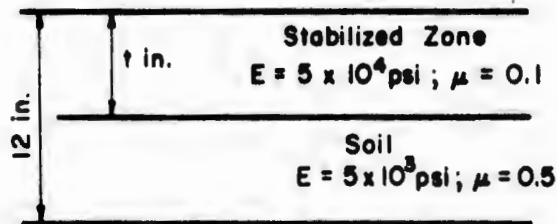


Figure 13 Effect of Stabilizing Upper Portion of Subgrade on the Surface Deflection of a Flexible Pavement

Flexible Pavement Zone Stabilization



Detail of Stabilized Layer



Note: n Designates Number of Stabilized Layers

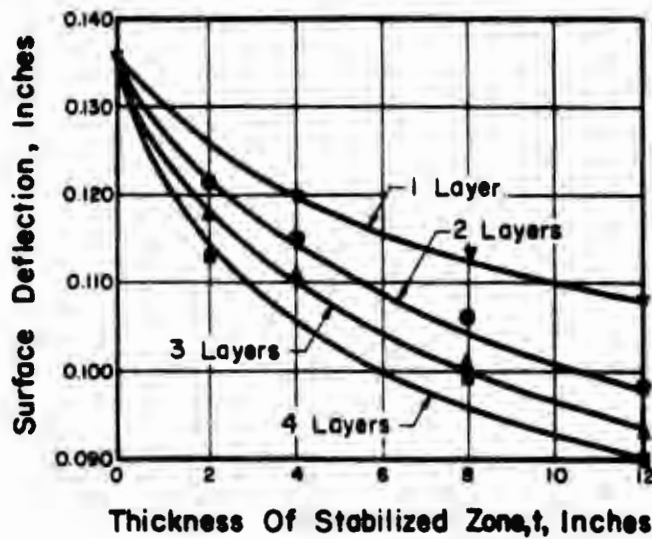
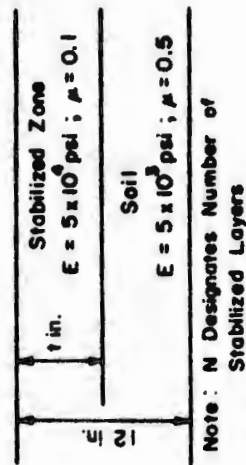
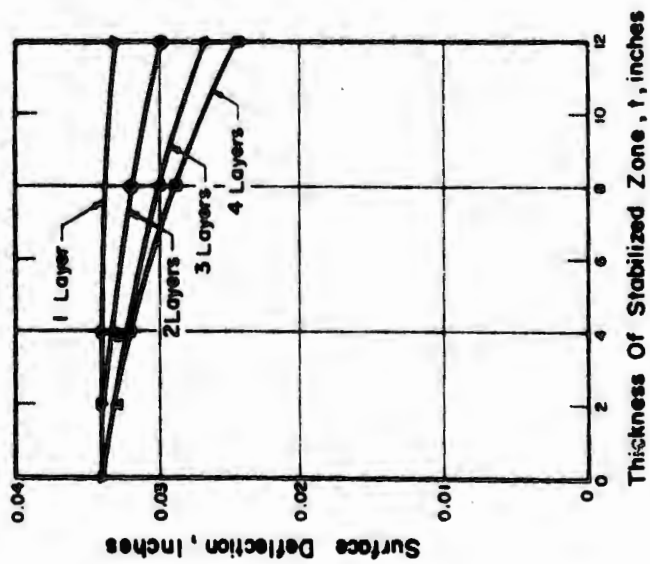
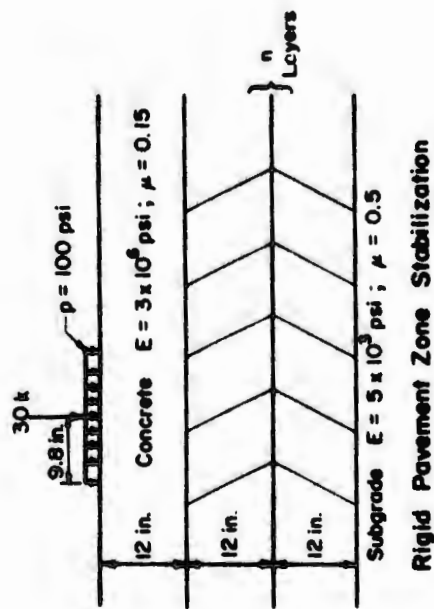


Figure 14 Effect of Thickness and Number of Incremental Stabilized Zones on the Surface Deflection of a Flexible Pavement



Detail of Stabilized Layer

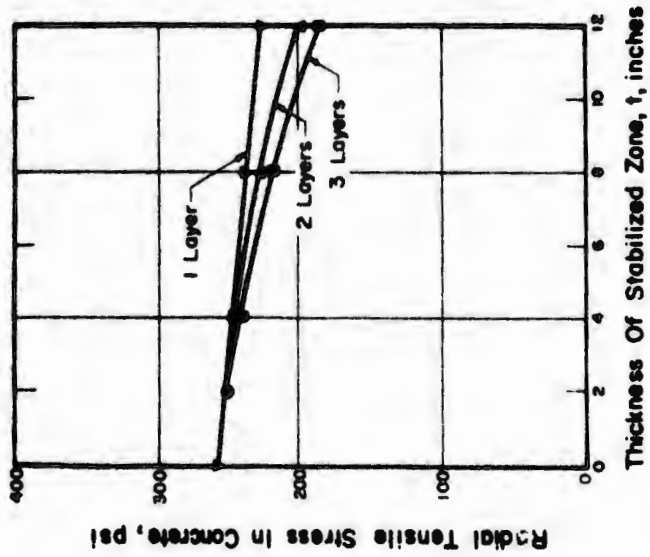


Figure 15 Effect of Thickness and Number of Incremental Stabilized Zones on Rigid Pavement Structural Response

3. For the flexible pavement section, both stabilization concepts effected a decrease in surface deflection, an increase in vertical subgrade stress, and a slight reduction in radial tensile strain at the bottom of the asphalt concrete layer. The range of radial tensile strains considering all stabilization procedures was only from 7.2×10^{-4} in./in. (no stabilization) to 6.5×10^{-4} in./in.

Subgrade stabilization systems incorporating stabilized vertical columns of materials, slanting stabilized layers, or random stabilization zones were not analyzed. A three-dimensional finite element solution would probably be required to approximate the behavior of such systems.

Comparison of the deep layer stabilization effects on pavement structural behavior with those obtained for conventional asphalt or portland cement concrete overlays (Section II) suggests that conventional overlay techniques are more effective. Corps of Engineers flexible pavement design theory (cover thickness concept) does not validate the correctness of the preceding statement.

It is important to recognize that such significant factors as pumping, faulting, and joint behavior of rigid pavements were not considered. It is possible that deep layer stabilization procedures may alleviate such problems.

SECTION IV

REVIEW OF A NUMBER OF POTENTIAL DEEP LAYER STABILIZATION PROCEDURES

INTRODUCTION

In order to evaluate the potential of various "deep layer stabilization" procedures currently available, this section examines a number of them in detail. The last subsection of the section is devoted to a short discussion of a number of new concepts and "far out" schemes that might possess some degree of relevance to deep layer stabilization.

In the subsections which follow, emphasis has been placed on discussion of:

1. theory of the procedure,
2. pertinent laboratory studies,
3. various field applications,
4. amenable soil types,
5. effects of treatment, and
6. cost of treatment.

Evaluations of the various procedures relative to their potential for deep layer stabilization is concluded in Section V.

ELECTRICAL METHODS FOR STABILIZING IN-SITU SOILS

Introduction

Electrical methods of improving the stability of in-situ fine-grained soils have been tried with varying degrees of success. Basically, three methods of electrical treatment have been utilized: electro-osmosis, electro-chemical, and electro-injection. Electro-osmosis has probably received the most attention as a result of its extensive use by Casagrande to facilitate construction of naval facilities in Germany during the 1930s. The use of electro-chemical and electro-injection procedures is reported much less frequently in the literature but they seem to have, at least conceptually, as much potential or possibly even greater potential than electro-osmosis. Karpoff (14) concluded that electro-chemical methods in

most instances make possible a greater improvement to physical properties than electro-osmotic treatment.

Theory

The exact mechanisms affecting stabilization are dependent on the particular electrical procedure being used.

A typical fine-grained soil-water system, Figure 16, consists of a large number of negatively charged clay particles intimately mixed with larger particle sizes. Cations typically attracted to the clay particles include Li^+ , Na^+ , NH_4^+ , K^+ , Mg^{++} , and H^+ . The type of cation(s) present depends on a number of factors but in general, the Lyotropic Series defines the relative affinity of cations for the surface of the clay particles. The relative affinity increases in the above list from $\text{Li}^+ \longrightarrow \text{H}^+$. The amount of cations present on the clay particle depends to a large extent, on the relative magnitude of negative charge that must be satisfied. The ability to hold and exchange cations is often referred to as the cation exchange capacity and increases for kaolinite, illite, and montmorillonite, respectively.

In the pore water that surrounds the clay particle-cation system, there exists more dissociated cations and anions. When an electrical gradient is imposed on the clay-water system, ions of opposite charge move in opposite directions. The cations (positively charged) move toward the cathode (negative electrode) and the anions (negatively charged) move toward the anode (positive electrode). It appears that as the ions migrate under the influence of the electrical potential, the positive ions transfer momentum or frictional forces to the dipolar water molecules. The force exerted by the cations apparently exceeds that exerted by the anions and a net movement of water toward the cathode occurs. Thus, as indicated by Winterkorn (15), an increased concentration of anions in the pore water will lessen the effectiveness of the electro-osmotic transport of the water toward the cathode.

Requisite to reducing the water content of a soil-water system by electro-osmosis is the provision of some means for removing the water as it

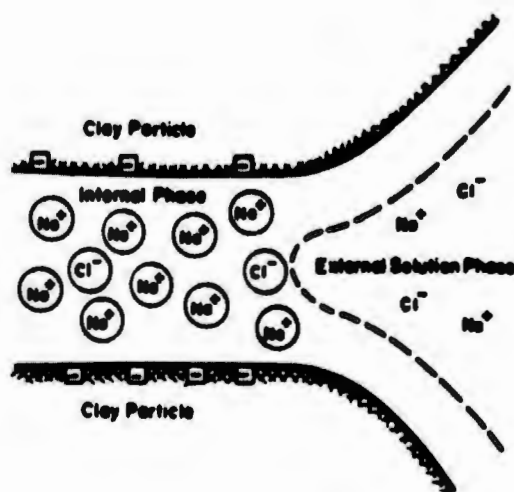


Figure 16 Schematic illustration of Donnan Distribution of Ions Between External and Internal Phases (From Ref. 18)

collects at the cathode. If some means such as a well point is provided, it is possible to gradually reduce the moisture content of the system, especially in the region of the anode.

Electro-chemical stabilization depends directly on the movement of ions and larger electrically charged molecules in the soil-water system. Under an electrical gradient, positively charged units (cations, molecules) move toward the cathode and negatively charged units (anions, molecules) move toward the anode. As the cations collect around the cathode the pH of this region increases and becomes quite alkaline. The high pH is partially attributed to the fact that hydrogen gas is liberated which produces an abundance of OH^- anions which can combine with the cations. On the other hand, at the anode, anions such as SO_4^{--} , CO_3^{--} , Cl^- , etc. are collecting with a resulting decrease in pH. Oxygen and chlorine gases are also liberated at the anode.

A study by Esrig (16) indicated that the pH may be as high as 11-12 in the region of the cathode and as low as 2-4 in the region of the anode, Figure 17. The solubility of silica and alumina increases significantly in high pH environments, Figure 18. In addition, alumina solubility increases markedly in low pH environments, Figure 18. Thus, the high pH in the cathode region and the low pH in the anode region may lead to partial degradation of the clay minerals and the liberation of alumina and silica in the system. If there is an abundance of Ca^{++} , Mg^{++} , etc. present in the system, it is possible that a pozzolanic reaction will occur which would provide cementing materials for the system and thus increase strength.

Winterkorn (15) suggests that chemicals might be introduced at both the cathode(s) and anode(s) and be allowed to migrate under the influence of an electrical gradient. These chemicals would then combine somewhere between the two electrodes to form insoluble compounds that would plug voids and increase strength. Winterkorn indicates that the formation of silicates, phosphates, chromates, etc. might be obtained by the movement of various chemicals. He also indicates that an electrical gradient might be used to permeate a soil system with polymer-forming liquid monomers (15).

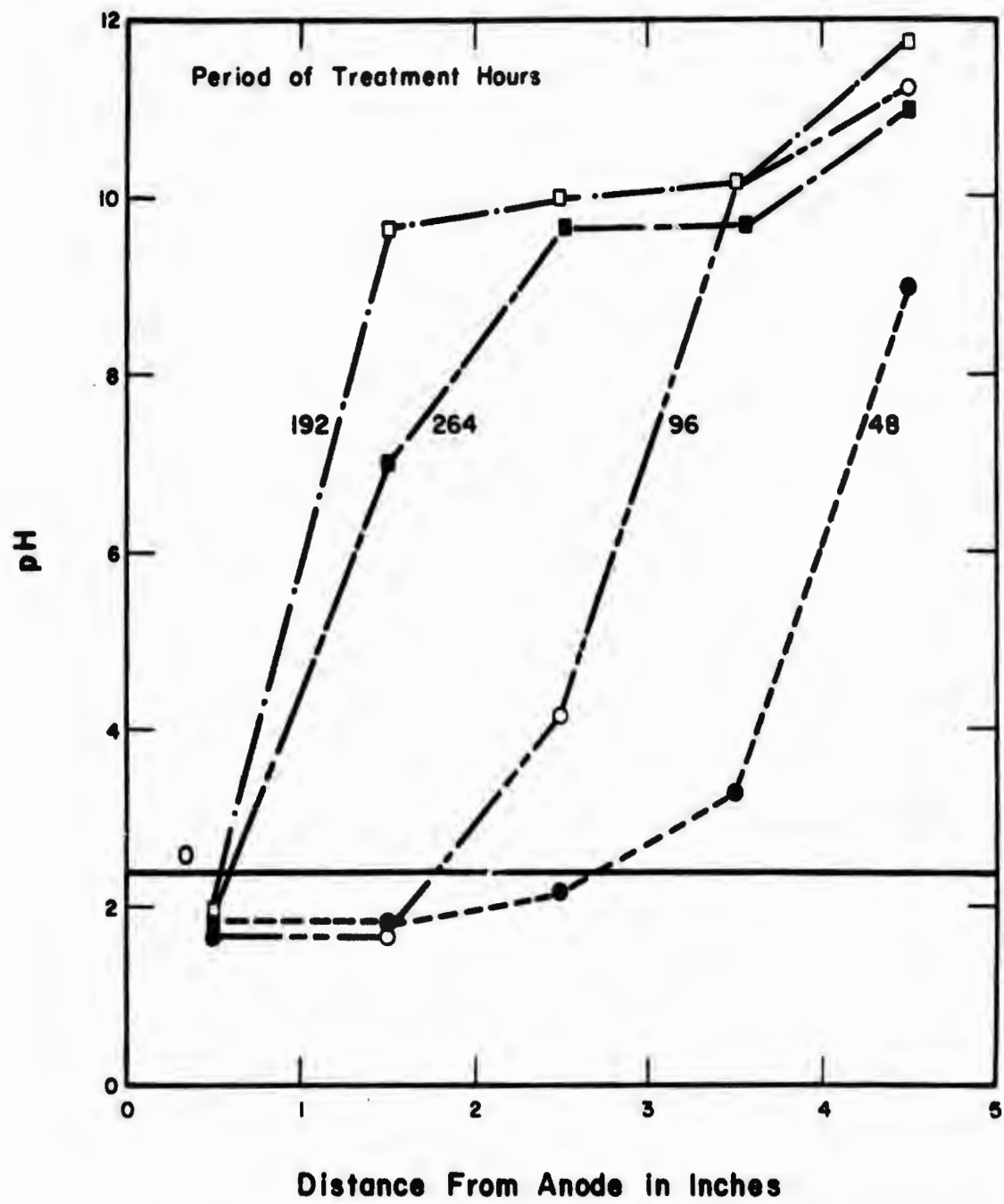


Figure 17 Variation of pH with Time and Distance from the Anode (From Ref. 16)

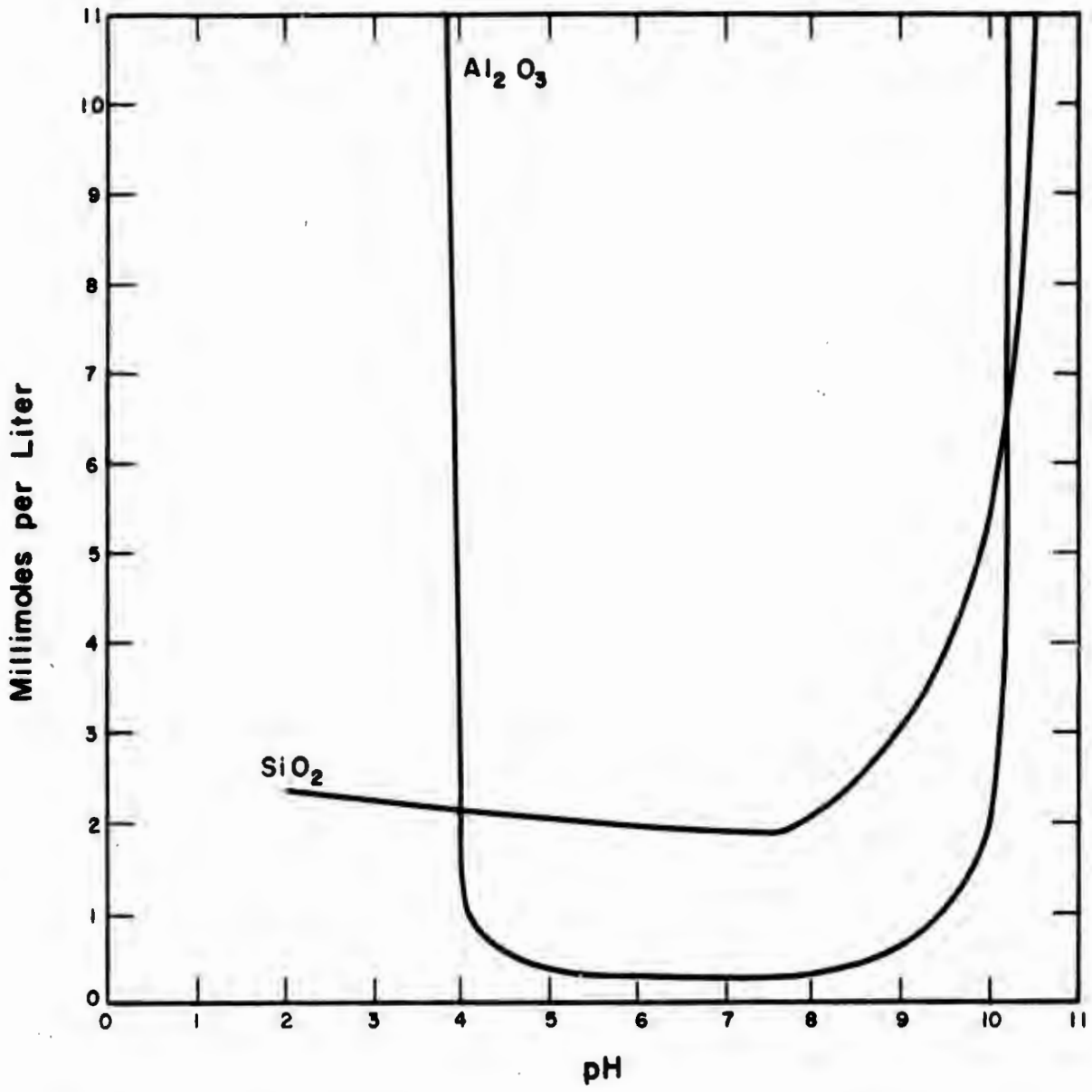


Figure 18 The Solubility of Silica and Alumina as a Function of pH
(From Ref. 37)

New chemicals can be introduced into the system by two general procedures. A common procedure is to use sacrificial electrodes which are dissolved. An example is the use of aluminum electrodes. A second procedure which lends itself to greater flexibility relative to the type of chemical that can be used is the use of porous reservoirs as the electrodes. Solutions of the chemicals can be placed in the reservoirs and allowed to migrate into the soil system under the influence of the electrical gradient. Application of a positive pressure at the reservoirs (electro-injection) may tend to increase the rate of migration of the chemicals.

As the concentration of cations around the cathode increases due to the migration of the dissociated cations in the pore-water (new or high concentrations of cations might also be introduced to the system) not only does the pH increase, but cation exchange may occur on the surface of the clay particles. The general order of replaceability of the common cations is given by the Lyotropic Series: $\text{Li}^+ < \text{Na}^+ < \text{NH}_4^+ < \text{K}^+ < \text{Ca}^{++} < \text{H}^+$. Any cation will tend to replace the cations to the left of it in the above series, and monovalent cations are usually replaced by multivalent cations. Cation concentration can however, alter the normal Lyotropic replacement series. It is a well known fact that cation exchange can cause a change in the properties (strength, Atterberg Limits, etc.) of clays. Esrig (16) attributed strength increases after electrical treatment to water content change and ion exchange, Figure 19. He found that the latter had the greatest influence on strength improvement. Some investigators indicate that after cation exchange, a flocculation of clay particles may result. However, it is difficult for an in-place material to undergo substantial structural changes without any form of manipulation of the soil particles.

The use of aluminum sacrificial anodes to provide Al^{+++} cations to the system has been a common procedure. Precipitation of aluminum salts may occur in the region where a pH of 7 is encountered. The presence of these salts may tend to increase the stiffness of the mass (17).

Electro-osmosis

Many theories have been advanced to explain complex electro-osmotic flow conditions on a macroscopic scale. Such theories have provided

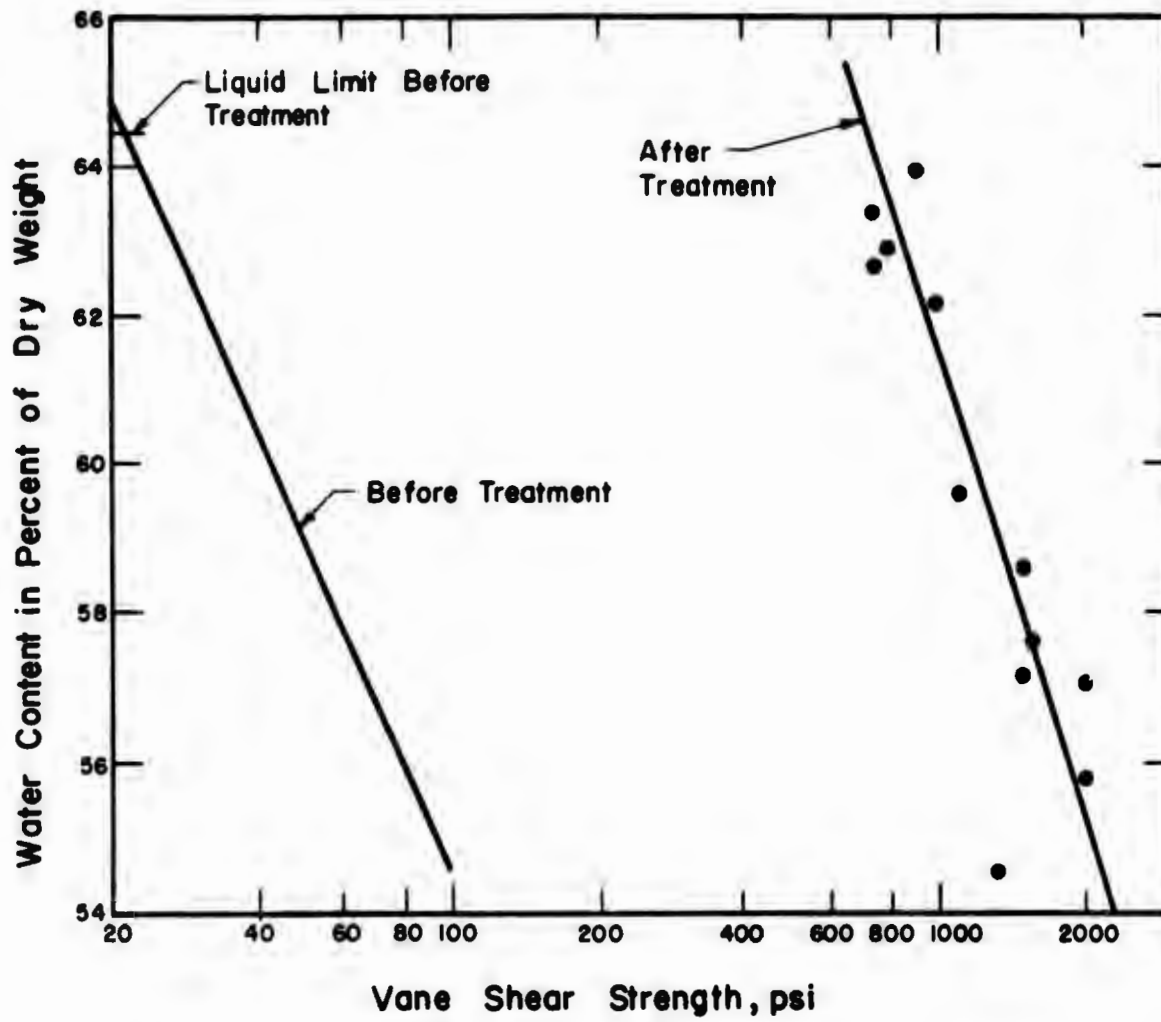


Figure 19 Water Content Versus Logarithm of Vane Shear Strength
(From Ref. 16)

mathematical expressions for water-ion flow through soil capillaries under an electrical gradient.

The classical models describing electro-osmosis assume that negatively charged anions and the water molecules migrate at the same velocity (18). The Helmholtz model (19) assumes large pore diameters while the Schmid model (20) deals with much smaller pores. Esrig's theory unifies the classical behavior concepts and includes the Helmholtz and Schmid models as special cases (21). Spiegler's friction model (22) and an ion hydration model (23) have been developed more recently. Both nonclassical models involve lengthy laboratory experimentation to derive meaningful expressions describing the observed electro-osmotic flow. Although such models take a more realistic view of flow conditions and are conceptually useful, the mathematical expressions derived are somewhat empirical in nature since much depends on extensive experimental observations and lengthy calculations. None of these models have yielded any simple, useful correlation between electro-osmotic flow and fundamental soil parameters (18).

Casagrande observed that electro-osmotic permeability is nearly constant (5×10^{-5} cm/sec per volt/cm) for saturated soils of widely varying hydraulic permeability (24). The rate of electro-osmotic water flow as developed from Darcy's equation is:

$$Q_e = k_e i_e A \quad (8)$$

where

Q_e = electro-osmotic flow, cm^3/sec

k_e = electro-osmotic permeability, cm/sec per volt/cm

i_e = electrical potential gradient (applied voltage/electrode spacing), volts/cm

A = cross sectional area, sq. cm.

This expression describes the maximum electro-osmotic flow that can be expected in any saturated soil (24,25). This application of Darcy's equation is one of the few mathematical expressions verified by most researchers as a valid statement for the maximum electro-osmotic water flow for a variety of soils.

Overall, it can be said that electro-osmotic flow is extremely complex and even highly idealized laboratory studies have failed to thoroughly explain the flow of water and the associated property changes occurring within the soil mass.

a) Laboratory Studies

Although the exact flow mechanism is not yet fully understood, extensive laboratory investigations have been made in an effort to predict property changes and power requirements.

Early work by Casagrande and various other investigators clearly indicates a decrease in water content and a drop in pore water pressures in the anode region. Consolidation of the soil mass in the region of the anode normally occurs -- the degree and extent of which depends on soil properties and duration of electrical potential application (24). Strength increases associated with electro-osmosis were initially thought to be a direct result of the degree of consolidation only.

Further experimentation indicated that shear strength increases were greater than predicted from consolidation theory alone. Physico-chemical property changes in the soil structure were found to have taken place (24,26). Especially when aluminum anodes were used, metal derivatives were deposited within the pore structure and permanent cementation occurred (24,27,28). Corrosion of the anodes accompanies significant durations of treatment. The cathodes are not normally deteriorated during treatment although the high pH environment may in some cases tend to erode the cathode. Heating, drying, and shrinkage take place when a high electrical gradient is used. Electrical current interruptions often occur when the soil shrinks away from the electrode (24,26,29). In addition, the decreased water content in the anode region will tend to increase the electrical resistivity of the soil mass and as a result, reduce electro-osmotic efficiency. At the cathode, increased water content and pore water pressure are usually noted, accompanied by a decrease in shear strength. The increased concentration of cations in the cathode region and the tendency for cation exchange to occur may tend to offset the reduced strength.

After the current is stopped, the pore water pressure at the electrodes normally tends to dissipate and strength changes noted during electro-osmotic treatment may be reduced. The exact magnitude of permanent strength improvement effected by electro-osmotic treatment (and the electro-chemical reactions) is difficult to predict. One of the major problems is that the water content is non-uniform between electrodes and as a result, consolidation and settlement between electrodes is erratic and somewhat unpredictable.

Changes in electro-osmotic permeability normally accompany water content changes. Generally, the permeability for a given soil type decreases with duration of treatment, except in the region of the cathode (18,24,26,29). Thus, flow rates also vary with time (21).

A recent publication by Gray and Mitchell (18) not only describes the fundamental theories of electro-osmosis, but is a valuable attempt to correlate common soil parameters with the flow efficiency and economy (as related to field application).

The efficiency and economy of electro-osmosis depends on the amount of water transferred per unit charge, e.g. gallons per ampere-hour. According to Gray and Mitchell (18), this quantity can vary by several orders of magnitude, depending upon water content, cation exchange capacity of the clay fraction, and pore water electrolyte concentration. Based upon the Donnan thermodynamic theory of irreversible flow (30), past research by others, and laboratory testing, Gray and Mitchell were able to predict the effect of increases in the pore water electrolyte concentration upon electro-osmotic flow, Figure 20.

Extensive laboratory investigations by Olsen (32), Carr (33), Gray (34), Gray and Mitchell (18), Ballou (35) and others have yielded a simple relationship between electro-osmotic water transport (gallons per ampere hours) and water content. Figure 21 indicates the wide range of soils (varying cation exchange capacity and pore water electrolyte concentration) for which the relationship is valid (18). Additional experimentation will be required before curve parameters (slope and intercept) can be directly calculated from electrolyte concentration and exchange capacity. Such a plot at least can be used to estimate the electro-osmotic flow efficiency.

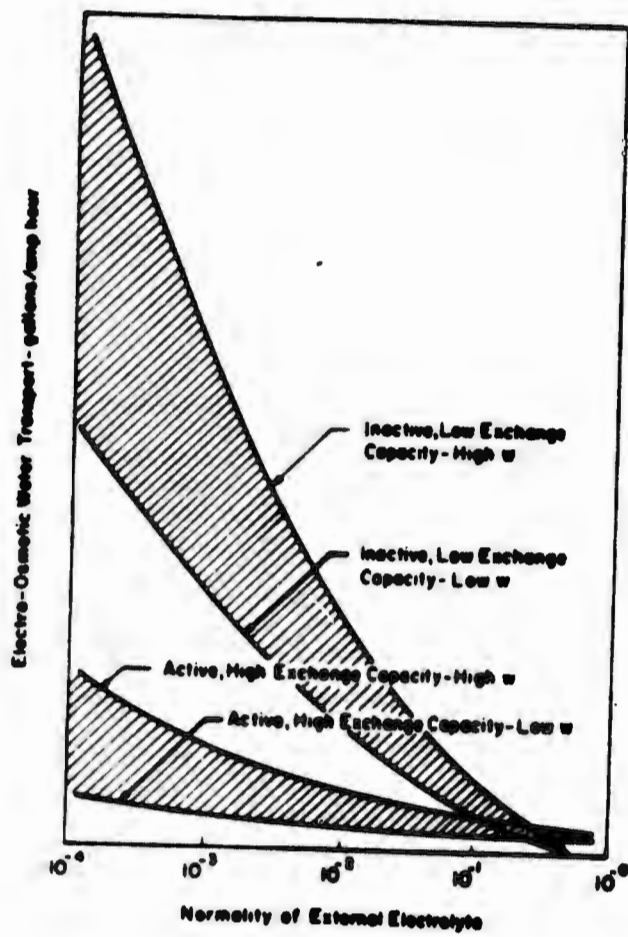


Figure 20 Schematic Prediction of Electro-Osmosis in Various Clays According to the Donnan Concept (From Ref. 18)

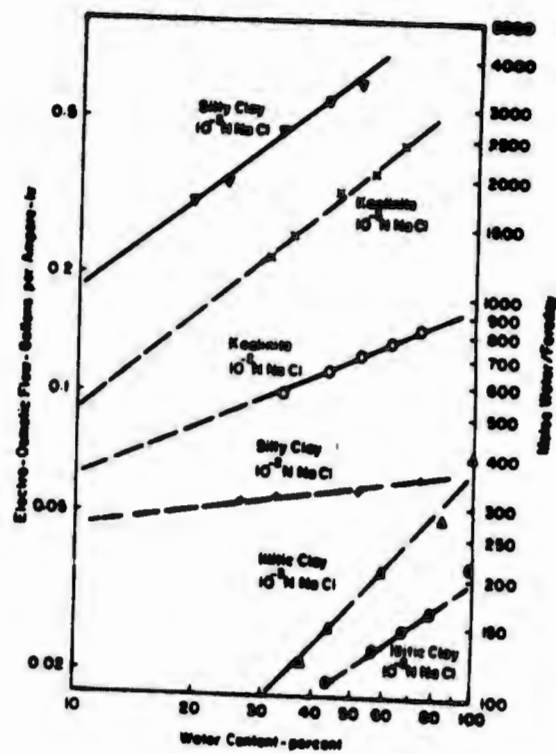


Figure 21 Electro-Osmotic Flow Versus Water Content in Clay-Water-Electrolyte Systems (From Ref. 18)

Alternately, the flow can be calculated from or measured directly from streaming potential data during a hydraulic permeability test. (Streaming potential is the voltage induced by the forced flow of a fluid through a porous membrane.) The electro-osmotic transport coefficient, k_i , expressed in gallons per ampere-hour, is related to k_e (previously defined) by the following equation:

$$k_i = 0.94 \left(\frac{k_e}{\sigma} \right) \quad (9)$$

where

σ = measured specific conductivity of the saturated soil,
ohms per cm.

Using a constant k_e of 5×10^{-5} cm/sec per volt/cm as suggested by Casagrande (or from measured k_e values at the field water content) along with a measured value of σ , k_i can be calculated.

If the streaming potential is measured, then the following relation holds:

$$k_i = 0.0094 \times E_s / P \quad (10)$$

where

E_s = streaming potential, millivolts
 P = pressure drop, atmospheres.

The above relation is an application of Saxon's Law, named after the original discoverer.

Gray and Mitchell (18) found good agreement between measured and calculated values of k_i at water contents between 50 to 60 percent. At higher water contents, "calculated" k_i values (Equation 9) are underestimated. At lower water contents, "calculated" k_i is greatly overestimated (18).

Having determined k_i by one of the above methods, the electrical energy requirements per gallon of water discharged, ϵ , in kilowatt hours, can be calculated from (18):

$$\epsilon = (\Delta E/k_i) \times 10^{-3} \quad (11)$$

where

ΔE = applied potential difference, volts.

b) Field Applications

Field applications in North America of electro-osmotic stabilization have been somewhat limited (30). Electro-osmotic methods of drainage were first recognized and developed in Europe. Casagrande is often credited with the first full-scale application of such methods for drainage and slope stabilization (25). The vast majority of field applications of electro-osmosis in the United States have been for slope stabilization in silty materials (6,29,36). A recent summary by Zeiler (29), revealed that most field applications were employed to stabilize slopes during or prior to excavation. Numerous examples of successful slope stabilization both in this country and abroad have been documented.

One of the few applications for embankment stabilization took place on the West Branch Dam (38). Excessive settlement and spreading were halted by temporarily reducing the embankment height and then decreasing excessively high pore pressures by electro-osmotic treatment. After decreasing the pore pressures the embankment was completed to original design levels. The shear strengths increased from 0.4-0.7 tsf to about 0.6-1.0 tsf. As predicted from laboratory studies, the greatest strength increase occurred at the anode with little or no change at the cathode. A general decrease in water content (4 to 9 percent) was also noted after electro-osmotic treatment of the clayey embankment material. As expected, the maximum decrease occurred at the anode.

A recent application of electro-osmosis for stabilization of a deep excavation in Norway has shown that the strength of quick clays can be measurably improved (39). Permanent shear strength increases from 1 to 4 t/m² were noted. The gain in shear strength exceeded the anticipated values (based on consolidation theory alone). Not only the water content decreased as expected, but physico-chemical properties apparently changed, resulting in the unexpected strength increase. The needed

reduction in water content to obtain a given strength increase was estimated from the relationship between water content, consolidation pressure, and undrained shear strength. Due to property changes causing strength increases, the predicted "required" change in water content was overestimated.

Electro-Chemical Treatment

Electro-chemical treatment of fine-grained materials has in some cases proven to be a satisfactory method of stabilizing a soil mass for particular job requirements. In general, laboratory applications indicate better results than do field studies, but reported field applications are limited.

In order to develop a better appreciation for electro-chemical procedures, a short discussion of a number of laboratory studies and field applications will be presented.

a) Laboratory Studies

A number of laboratory studies have been conducted in an effort to identify the significant parameters of electro-chemical methods.

Adamson, et al. (40) present a fine discussion of electro-kinetic procedures. They conducted laboratory tests to determine the effectiveness of dewatering clayey soils, high in montmorillonite, by an electro-osmotic procedure. Aluminum anodes were used during the electrical treatment and calcium chloride, aluminum acetate, and aluminum sulfate were added to the system. The unconfined compressive strength of the soils increased from about 1.5 psi prior to treatment to about 3 psi after treatment. The authors concluded that aluminum anodes should be used during electrical treatment so that Al^{+++} cations are provided to the system which increase strength through base exchange and cementation (40). In another study Adamson, et al. (41) found that electro-chemical treatment of sands containing as little as 1.5 to 3.5 percent clay could increase strength. The clay fraction consisted of either pure montmorillonite, illite, or kaolinite clay. A solution of $CaCl_2$ and $Al_2(SO_4)_3$ was added to the sand mixture during the electrical treatment. X-ray analysis of the electrically

treated fine-fraction indicated that new minerals had been formed including: gibbsite, limonite, calcite, allophane, hematite, gypsum, and others. Compressive strength increases noted in this study ranged from 1.5 to 3.0 psi.

Nettleton (42), Esrig (16), and Nettleton (43), respectively, investigated the effects of electro-chemical treatment of the three common clay minerals -- kaolinite, illite, and montmorillonite. In all three cases nominal strength increases were noted after treatment. In the two studies by Nettleton, an aluminum anode was used which might explain the increased strength. Esrig (16) used a carbon rod as the anode. The nominal strength increase noted in this case was attributed mainly to the ion exchange phenomenon, although some strength increase was due to a moisture content reduction. In all three cases, no new chemicals were added to the system; rather cations and anions present in the pore water contributed to ion exchange.

Geuze, et al. (44), attributed electro-chemical hardening of the fine-grained soils investigated to a decreased moisture content and to the formation of aluminum hydroxide when aluminum electrodes were used and eventually to a flocculation of the clay particles. They also hypothesized that the use of copper electrodes may introduce cupric ions (Cu^{++}) to the system and flocculate clay particles.

In a laboratory study conducted by Karpoff (14) the effect of electro-chemical treatment on a sandy silt and a medium fat clay was evaluated. Aluminum electrodes were used and calcium chloride and aluminum chloride chemical were introduced through perforated metallic anodes. About five gallons of 2 percent CaCl_2 was used per cubic yard of soil. Compressive strength increases ranged from 5 to 13 psi. Karpoff (14) concluded that electro-chemical methods in most instances make possible a greater improvement to physical properties than electro-osmotic treatment.

Murayama and Mise (17) also found that the use of aluminum electrodes during electrical treatment improves the post-treatment strength. They attributed the nominal strength increase noted (< 1.0 psi) to the formation of aluminum salts and bauxite in the soil.

By using perforated aluminum discs as the anode and a copper plate as the cathode, Shukla (45) found that plasticity and $\frac{\text{SiO}_2}{\text{R}_2\text{O}_3}$ (silica sesquioxide ratio) and $\frac{\text{SiO}_2}{\text{Al}_2\text{O}_3}$ ratios for a group of fine-grained soils from India decreases with increased electrical treatment. A nominal compressive and shear strength increase (< 3 psi) also were exhibited after treatment.

Esrig (21) probably conducted one of the most extensive laboratory analyses relative to the influence of various chemicals on electro-chemical hardening. The various chemicals were introduced at the anode and allowed to permeate the soil under the electrical gradient. Table 1 summarizes the chemicals that were used. Maximum compressive strength increases of about 4 psi were noted.

One of the major problems associated with electro-chemical treatment is the nonuniform strength increase and nonuniform chemical distribution between the electrodes. For example, Esrig (16) has shown that a large pH and strength differential can occur between the anode and cathode after treatment. Boiko (46) has suggested a method by which the region of electro-chemically associated strength increase and high pH noted at the cathode can be extended by using a CaO-CaCl₂ solution in a reservoir at the anode. Frequent replacement of this solution will extend the region of high pH and strength increase. Using this procedure a strength increase of 50 to 90 psi was noted for specimens submerged in water for 13 months after treatment. Adamson, et al. (41) suggested frequent reversals of polarity to improve uniformity of strength and chemical distribution.

Talme (9) has reported results from laboratory studies in which electro-chemical treatment was used to improve the shear strength of a number of Swedish "quick clay" soils. An electrical gradient ranging from 0.4 to 1.8 volts/cm was used to facilitate the permeation of the soils with calcium and magnesium chloride solutions of various concentration (concentration ranged from one percent to saturated). Both remolded and undisturbed shear strength measurements were made by a Swedish fall-cone procedure.

Table 1

Summary of Types of Chemicals Esrig (21) Electro-
Chemically Injected Into Soils

Type of Chemical Solution

Distilled Water

KOH

CaCl₂

NH₄ Cl

KH₂PO₄

Fe SO₄

Na₄SiO₄

Na₂ SO₄

Al₂ (SO₄)₃

AM-9

Arquad 2 HT-75

Aliquat H226

Chempact

The duration of treatment of 30 cm long samples ranged from about 150 minutes to about 4,800 minutes with an average duration of slightly greater than 1,000 minutes. Examination of the results of this experiment indicated that improved shear strength, as measured by the Swedish fall-cone procedure occurred throughout the entire length of the specimen although the location of the maximum shear strength increase, relative to the anode and cathode was erratic. In some cases maximum shear strength improvement occurred in the cathode region; while in others the maximum shear strength improvement occurred midway between the electrodes.

The maximum percentage shear strength increase was displayed by the remolded specimens where up to a tenfold increase occurred; however, the magnitude of the post-treatment strength was less than one psi. The undisturbed samples displayed post-treatment shear strengths ranging from 1.5 to 2.0 psi; untreated shear strengths were about one psi. No conclusions were made relative to the influence of type of chemical or the solution concentration used in the treatment.

The Louisiana Highway Department (47) has conducted a limited laboratory study relative to permeating sandy clay, medium silty clay, and heavy clay soils with a 10 percent (by volume) lime-water slurry by imposing an electrical gradient upon the system. In this study steel electrodes (pipe and angle iron) were spaced at 16.5 to 80 in. depending on the test and an electrical gradient ranging from approximately 0.75 to 2.0 volts/cm was applied. The lime slurry was placed in a trench between the electrodes and the electrical gradient applied for durations ranging from 75 to 1,584 hours. An evaluation of the study indicated that no significant changes in engineering characteristics occurred. It would appear that little or no lime migration was effected. A limited degree of electro-chemical hardening was noted around the cathode however.

b) Field Applications

Early field applications of electro-chemical treatment were almost solely restricted to treatment of piles to increase their bearing capacity. However, recently a number of applications have been made relative to foundations. An electric potential has been used to increase the rate of

permeation of sodium silicate into a loess deposit and to help permeate the subgrade under a runway with a commercial waterproofer.

(1) Treatment of Friction Piles

Results of full-scale field tests conducted by Casagrande have been reported in Reference 48. In one application, six wooden piles about one foot in diameter were sheathed with aluminum one millimeter thick and driven to a depth of 20 ft at 4- to 6-ft spacings. Load tests indicated that the bearing capacity of the piles was between 7 and 9 tons per pile prior to treatment. A 220-volt potential was applied between the piles until 60, 260, and 1,000 kwh of electrical energy had been applied respectively to the three pairs of piles.

A maximum bearing capacity of about 40 tons per pile was obtained after about 30 kwh of energy had been applied. Further application of electrical energy tended to reduce the bearing capacity. Examination of the soil around the piles indicated that no apparent change in the soil appeared at distances greater than 12 in. Increased pile bearing capacity was attributed to an increase in effective pile size and an increased skin friction due to the corrosion of the aluminum sheathing.

Soderman and Milligan (49) reported that electrical treatment of 12-in., 53-lb steel H piles ranging in length from 55 to 166 ft increased their bearing capacity. The piles were embedded in soft varved clay. A test pile was found to exhibit a bearing capacity increase of 30 tons (initial bearing capacity = 30 tons) after 3 hours of treatment with a potential of 115 volts. The increase in bearing capacity was attributed to electro-osmosis rather than electro-chemical treatment, however.

Spangler (48) reported the results of a small-scale study directed toward investigating the various aspects of increasing the bearing capacity of friction piles by electrical treatment. Aluminum rods were driven into various types of soils and electrical potential applied. The results indicated the following relevant facts:

1. Maximum bearing capacity resulted after an optimum amount of treatment;

2. The time or energy required to obtain maximum bearing capacity increased with increased clay content;
3. The maximum pile bearing capacity increased with increased clay content;
4. The bearing capacity of the cathode pile was greater than that of the anode pile but the bearing capacity of both increased; and
5. Based on limited data, the maximum bearing capacity did not seem to be related to the type of clay mineral present.

(ii) Electro-Chemical Treatment of Foundations

Guseva (50) reported that an electrical treatment procedure was used for arresting the progressive deformation of damp soil under an existing building. The method consisted of driving a series of electrodes into the soil, 2.5 meters deep and 1.5 meters apart; 5 amps DC were applied continuously for 3 days at a density of 10 mA/cm^2 at 10 volts/cm. In addition, by introducing calcium and magnesium salts (10 percent solution) into the soil, the load bearing capacity of the soil was increased from 1.5 kg/cm^2 to $5\text{-}10 \text{ kg/cm}^2$ (70-140 psi).

In a limited field investigation of electro-kinetic processes, Esrig (51) found that the major factor improving strength of a silty soil with low plasticity (ML) was consolidation that accompanied dewatering although some increase appeared to be due to electro-chemical hardening when a solution of CaCl_2 was used in conjunction. Esrig concluded that relatively uniform shear strength could be obtained by periodic reversals of polarity (51).

Bally and Antonescu (52) reported the use of electricity in an "electro-silicification process" to treat loess soils. A sodium silicate solution was injected under pressure and an electrical potential applied between electrodes about one meter apart. The quantity of silicate solution was halved when an electrical potential was used as compared to pressure injection alone. About 200 liters of solution were injected per cubic meter of soil. The electrical treatment reduced the treatment period

from seven days to one day. The total electrical energy used was about 10 to 15 kwh per cubic meter.

Dearstyne and Newman (53) have reported on the attempted use of an electro-kinetic procedure to stabilize a water-saturated fine-grained clayey subgrade under an airport runway at the Seattle-Tacoma International Airport. A 2-percent solution of commercially available Aliquat H226 was introduced through 1 1/2-in. diameter aluminum pipes on 5-ft centers at one side of a 150-ft wide runway. A copper wire was buried at a depth of 4 ft on the opposite side of the runway and a 270-volt electrical potential was applied for 18 days. Pavement deflection measurements were made prior to and after the treatment. Under a 45,000-lb crash truck, 0.2-in. surface deflection was noted prior to treatment. After treatment the deflection was reduced to 0.010 in. The total cost of treatment was about \$35 per longitudinal linear ft of the runway (includes \$10 per linear ft post-treatment subgrade grouting cost).

Chemical tests were made after the treatment period. pH values displayed an almost immeasurable increase from an initial 8.9 to a post-treatment value of 9.0. However, attempts to determine the amount of Aliquat H226 in the soil were unsatisfactory. It was found however, that about 1.5 lb of the aluminum electrodes were eroded.

The cause of marked reduction in deflection is difficult to explain. Esrig and Gemeinhardt (54) in a discussion of the article show that theoretically, the Aliquat H226 probably moved only about 2 in. from the aluminum pipes and that the stabilization effected was probably solely due to an electro-osmotic reduction in moisture content. However, a reduction in surface deflection of the magnitude reported, from the standpoint of a pavement behavior analysis, would require a tremendous increase in subgrade strength. Such an increase due to electro-osmosis is highly unlikely.

Talme (9) has reported on a field study conducted in Sweden concerning electro-chemical treatment in which a 35-percent solution of CaCl_2 was electro-injected into a fine-grained soil. In this experiment, steel electrodes, 20 mm in diameter and 5 meters long, were spaced on 300-cm centers. A 46-mm diameter, PVC plastic pipe, perforated at the lower end

was driven to 4.6 meters prior to placement of the steel electrodes. This plastic pipe served as a reservoir in which the calcium chloride solution and the steel electrodes were placed.

The electrical gradient averaged about 0.5 volt/cm during the 150-hour duration of treatment. After treatment, a piston sampler was used to sample the soil to a depth of 4 meters. A 4-meter deep profile sample was taken every 0.5 meter over the distance between the electrodes. Remolded and "undisturbed" shear strength measurements were made with the Swedish fall-cone procedure. On the average, at a depth ranging from 2.3 to 3.81 meters, a 28-percent shear strength increase was noted with a maximum undisturbed shear strength of about 1.3 psi being found. Remolded shear strengths showed increases on the average of about 375 percent although the maximum shear strength noted was less than one psi.

Chemicals Involved in Electrical Chemical Treatment

Esrig (16) attributed the strength increase after electrical treatment to ion exchange and water content decrease; not to the formation of new chemical compounds or the introduction of new chemicals into the system.

Probably one of the most common procedures is to introduce aluminum cations into the system by dissolving an aluminum anode electrically. Murayama and Mise (17) hypothesize that the Al^{+3} cations increase strength and decrease plasticity by forming aluminum salts and/or bauxite.

Another procedure has been tried in which $CaCl_2$ is introduced at the anode and the Ca^{+2} cations are electrically pulled toward the cathode. When excess Ca^{+2} cations are present in the system, cation exchange can occur. Boiko (46) has used a modified version of this procedure in which he first saturates the system with $CaCl_2$. Initially, the pH at the cathode rises due to the presence of Ca^{++} and OH^- . At the anode the tendency is for the pH to drop as a result of the absence of cations and the abundance of anions. However, by introducing a solution of $CaO-CaCl_2$ at the anode and frequently replacing the solution, it was possible to increase the extent of the high pH region. With reactive soils, it is possible to obtain a lime-soil pozzolanic reaction if a high pH environment is

maintained. By increasing the region of high pH with the CaO-CaCl₂ solution at the anode it was possible to extend the region of possible lime-soil pozzolanic reaction.

Adamson, et al. (41) used an electrical potential to saturate a fine-grained kaolinite soil with CaCl₂ and then reversed the polarity and introduced Al₂SO₄. X-ray analysis indicated that destruction of the silicates occurred and that a group of new chemical compounds was formed. In another study, with a montmorillonite, Adamson, et al. (40) introduced CaCl₂, Al₂SO₄, and an aluminum acetate solution.

Esrig (21) examined the effectiveness of various chemicals by introducing a number of chemical solutions at the anode. The following list summarizes the chemicals that were used:

distilled water

KOH

CaCl₂

NH₄Cl

KH₂PO₄

Fe SO₄

Na₂SO₄

Na₄SiO₄

Al₂(SO₄)₃

It was found in this study, that from the standpoint of strength improvement, KOH, CaCl₂, and Na₄SiO₄ were the most effective chemicals. In addition, Esrig (21) introduced the following commercial chemical stabilizers and waterproofers at the anode:

AM-9

Arquad 2HT-75

Aliquat H226

Chempact

None of these commercial chemical stabilizers or waterproofing agents produced increases in strength that were significantly different from increases obtained by common chemicals.

Winterkorn (15) indicates that polymer-forming liquid monomers might be "pulled-around" in the soil by use of an electrical gradient. Field applications are reported (52,53) in which an electrical gradient was used to permeate a soil with sodium silicate and Aliquat H226.

Power Requirements

In general, the electrical current required for electrical treatment of fine-grained soils is obtained from a direct current source. Alternating current can be used but generally it must be passed through a transformer first in order to convert it to direct current. Zaslavsky and Ravina (55) have suggested the use of alternating current for inducing electro-kinetic phenomena.

The level of applied electrical voltage and the duration of application are dependent on soil properties (water content, electrolyte concentration, activity, etc.), electrode spacings, purpose of electrical treatment (51), and degree of stabilization required. Water, ion, and/or molecule migration is directly linked with current flow. Current flow is highly dependent on soil properties and their relation to electrical resistance. Generally, water and ion flow can be expected to increase as the electrical gradient is increased. However, high applied voltages cause heating and drying at the anodes that result in reduced current flow or even current interruptions. Thus, a power level must be selected low enough to avoid excessive drying at the anode, yet high enough to maintain a reasonable potential gradient.

Long periods of treatment may be required for prolonged electro-osmotic drainage, permeation of large quantities of chemicals, and/or treatment of a large volume of soil drainage, and large strength or water content changes. In many cases the duration of treatment must be determined from field observations and testing as the stabilization progresses although valuable information can be obtained from laboratory studies.

Table 2 summarizes typical electrical requirements used in electro-chemical laboratory studies. Voltage requirements normally depend on the electrode spacing and the desired electric potential or gradient. Typical

Table 2
 Reported Electrical Requirements
 for a
 Number of Laboratory Electro-Chemical Investigations

<u>Investigator</u>	<u>Potential Gradient, volt/cm</u>	<u>Energy, kwh/yd³</u>	<u>Treatment Duration</u>
Erig (16)	1	-	9 hrs-11 days
Murayama and Mise (17)	3	-	20 hrs
Adamson, et al. (40)	3-10	130	140-160 hrs
Adamson, et al. (41)	1.5-8	-	10 days
Shukla (45)	12	-	5 hrs
Nettleton (43)	3-60	100-350	-
Nettleton (43)	0.6-1.5	8-12	-
Geuze, et al. (44)	-	-	-
Boiko (46)	-	1125	30 days

voltage requirements range from 30 to 270 volts. In electro-osmotic treatment a typical potential gradient is 0.5 volt/cm to 2.0 volts/cm. Initially greater potential is applied and later it is reduced. This may cause a rapid build-up of pore water tension (51). In electro-chemical studies, the gradient has varied from 1 volt/cm to 60 volts/cm with 10 volts/cm being the normal maximum gradient.

The amount of electrical energy consumed can vary greatly depending on the resistance of the soil, the amount of water to be moved, the amount of chemical to be moved, the degree of stabilization required, etc. Based on a limited number of embankment stabilization studies, electro-osmosis may require 15 to 25 kwh/yd³ of material. For electro-osmotic stabilization of excavations 0.4 to 2.3 kwh/cu.yd have been reported (51). Normally, the literature indicates that electro-chemical treatment requires more electrical energy. Laboratory studies, Table 2, have shown that anywhere from 10 kwh/yd³ to 1,125 kwh/yd³ of treated material may be required for electro-chemical stabilization. About 10 to 15 kwh/yd³ was reported for an electro-silicification job on an in-place loess (52). Treatment of friction piles has required as much as 1,000 kwh of energy per pile (48).

The duration of field treatment depends on a number of factors including: type of treatment, magnitude of electrical gradient, electro-osmotic permeability of the soil, volume of material to be treated, degree of stabilization required, etc. The duration of electro-osmosis has been reported to vary from a month to a year (29). The treatment duration in laboratory studies concerning electro-chemical treatment has ranged from 8 hours to 30 days, Table 2. Only a very limited amount of data is available concerning the duration of treatment in field applications of electro-chemical treatment. Guesva (50) reported a 3-day treatment period and Dearstyne and Newman (53) reported an 18-day treatment period.

Theoretical methods for estimating the power required per unit volume in electro-osmotic applications are not always applicable to field soil conditions, however, techniques developed by Gray and Mitchell (18) at least provide an indication as to overall feasibility.

Power consumption can basically be computed from:

Power (Watts) = Current (Amperes) x Applied Electrical Potential (Volts).

The Casagrande formula for calculating current requirements for electro-osmotic treatments is as follows (29):

$$I = \pi \tau LEN \left(\frac{1}{\log d/r_1} + \frac{1}{\log d/r_2} \right) \quad (12)$$

where

- I = current (amps)
- L = length of electrodes (cm)
- E = potential difference (volts)
- N = # of anode-cathode pairs in parallel arrangement
- d = distance between anode and cathode (cm)
- r_1 = anode radius (cm)
- r_2 = cathode radius (cm)
- τ = specific conductivity of the soil ($\text{ohm}^{-1} \text{cm}^{-1}$)

No such theoretical approach was found for determining power or current for electro-chemical treatments. However, in general, the current or power requirements would be higher than indicated by the above procedures.

Types of Electrodes

The type of electrode used in electrical treatment procedures depends to a certain extent on the particular type of procedure but normally consists of some type of metal pipe, rod, rail, etc. capable of conducting an electric current with a minimum of resistance. Probably, the most commonly used electrodes in electro-osmotic stabilization are iron pipes or steel rails. The electrodes must possess sufficient rigidity to be driven into the embankment although jetting is also used to place the electrodes. A metallic well point is often used as the cathode in order that the excessive water which collects at the cathode can be removed.

Electro-chemical treatment procedures, depend primarily on the migration of ions and/or the introduction of new cations. Several types of

electrodes have been tried. Table 3 summarizes a number of materials that have been used as electrodes. Cathode materials have included steel, iron, copper, and aluminum. The anode which is often eroded during electrical treatment is normally composed of aluminum, although laboratory procedures have used nonmetallic reservoirs filled with a solution such as CaCl_2 . Friction piles are often sheathed with thin aluminum which serves as the electrical conductor and also may contribute aluminum cations to the soil mass.

Electrode Spacing

The optimum electrode spacing is dependent on a number of factors including: soil properties (electro-osmotic permeability, activity, pore water electrolyte concentration, etc.), the desired amount and rate of water removal in electro-osmotic applications, the magnitude of energy input, the types and characteristics of chemicals used, etc.

Casagrande has used optimum spacings of 10 to 15 ft for cut slope electro-osmotic stabilization in silty soils. For electro-osmotic embankment stabilization, and for more clayey materials, lesser distances of 5 to 10 ft might prove more efficient (29).

Reported electrode spacings in electro-chemical treatments ranged from 3 to 5 ft indicating that possibly a closer spacing is required for electro-chemical stabilization procedures (50,52,53).

Types of Soils Treated

Silty and clayey soils have been successfully treated by electro-osmosis. The method is best suited to silts, as a much higher electrical consumption can be anticipated in clays. Most applications to date have been with saturated soils with no pre-loading history. Extremely high initial water contents and sensitivity explain the electro-osmotic related strength increases of 10 to 150 times reported in the literature for extremely weak soils (29).

A number of fine-grained soils have been treated in the laboratory by electro-chemical means. Table 4 summarizes general information and common

Table 3
 Reported Types of Materials Used
 for Electrodes in
 Electro-Chemical Laboratory Studies

<u>Investigator</u>	<u>Cathode</u>	<u>Anode</u>
Esrig (16)	Steel or brass grid	Carbon rod reservoir with CaCl ₂ solution
Murayama and Mise (17)	Aluminum	Aluminum
Adamson, et al. (40)	Iron	Aluminum
Adamson, et al. (41)	Iron	Aluminum
Shukla (45)	Copper	Perforated aluminum discs
Nettleton (43)	Copper	Aluminum
Nettleton (42)	Copper	Aluminum
Geuze, et al. (44)	Aluminum	Aluminum, Copper
Boiko (46)	-	Hollow, porous, non-metallic reservoir with iron anode
Esrig (21)	Steel or brass grid	Carbon rod

Table 4

Summary of Various Types of Soils Used in
Electro-Chemical Laboratory Investigations

<u>Investigator</u>	<u>Soil</u>	<u>Properties</u>		
		<u>LL (a)</u>	<u>PI (b)</u>	<u>2μ (c)</u> <u>pH</u>
Nettleton (43)	Pure Kaolinite	70	40	6.4-7.3
Nettleton (42)	Pure Montmorillonite	870	800	8.3
Esrig (16)	Pure Illite (Grundite)	64	35	2.5
Adamson, et al. (40)	Montmorillonite, Illite, and Kaolinite	-	-	-
Adamson, et al. (41)	Soils high in Montmorillonite	-	-	-
Shukla (45)	Fine-grained soils from India	33-75	7-3	-
Murayama and Mise (17)	Clay soils from Japan	39-76	10-47	6.2-7.7
Genze, et al. (44)	Heavy clay with trace of peat	-	-	-
Boiko (46)	Argillaceous soil	-	-	-
Esrig (21)	Illite	64	35	2.5

(a) Liquid Limit, %.

(b) Plasticity Index, %.

(c) Clay content, < 0.002 mm size, %.

physical properties of a number of the soils that have been treated. In many cases, pure clay minerals (montmorillonite, illite, and kaolinite) have been treated. Other soils treated have included fine-grained soils from India and Japan. Geuze, et al. (44) treated a heavy clay that contained traces of peat.

Electro-chemical treatment has been tried in the field on various soil types including: soft varved clay (49), silty soil of low plasticity (ML) (51), loess (52) and a fine-grained clayey subgrade (53).

Effects of Electrical Treatment

a) Electro-Osmosis

As previously indicated, few applications of electro-osmosis to embankment stabilization have been made. The West Branch Dam offers typical results. The shear strengths were increased from 0.4 to 0.7 tsf to about 0.6 to 1.0 tsf. Shear strengths greater than about 20 psi have not been achieved. In addition, those shear strengths noted during and immediately after treatment are not permanent. Some permanent strength increases have however been noted.

b) Electro-Chemical Procedures

The results obtained from various investigations have in most cases not been extremely encouraging. A summary of the findings of a number of laboratory investigations concerning electro-chemical treatment may be found in Table 5 and Table 6.

A number of investigators (14,16,17,40,41,42,43,45) have reported increased shear strength values (reported as cohesion interception and vane shear strength) after electro-chemical treatment, Table 5. Shear strength improvements attributed to the electro-chemical treatment ranged from 0.2 psi to 13 psi. However, typical shear strength improvement averaged about 1 psi, Table 5. Esrig (16) reported a vane shear strength of 8 to 11 psi at the cathode but only 0.2 psi at the anode after electro-chemical treatment. Boiko (46) reported a soaked (13 months) compressive strength after electro-chemical treatment of 70 to 140 psi.

Table 5

Summary of Typical Strength Improvements
Obtained by Electro-Chemical Treatment
of Laboratory Specimens

Investigator	Type of Strength	Strength after Treatment, psi	Magnitude of Strength Improvement, psi
Karpoff (14)	Cohesion	45-53	5-13
Esrig (16)	Vane Shear Strength	Cathode 8-11 Anode 0.2	-
Murayama and Mise (17)	Cohesion	-	0.4
Adamson, et al. (40)	Cohesion	3 (a)	1.4 (a)
Adamson, et al. (41)	Cohesion	0.8-1.4	0.8-1.4
Shukla (45)	Cohesion	4-16	1
Nettleton (43)	Cohesion	0.7-4.5	0.2-3.8
Nettleton (42)	Cohesion	2.0	1.3
Boiko (46)	Unconfined Compression	70-140 (b)	-

(a) Soaked 72 hours prior to testing.

(b) Soaked 13 months prior to testing.

Table 6

Typical Results of Esrig's (21) Laboratory
Study of Electro-Chemical Treatment with
Various Chemical Solutions

<u>Type of Chemical Solution</u>	<u>Shear Strength, psf</u>		<u>Strength Due to Electro-Chemical Treatment, psf</u>
	<u>Initial</u>	<u>Final</u>	
Distilled water	17	155	25
KOH	25-50	160-450	40-320
CaCl ₂	3-4	255-405	240-400
NH ₄ Cl	4	110	100
KH ₂ PO ₄	4	210	200
FeSO ₄	4	35	30
Na ₄ SiO ₄	3-4	200-680	190-640
Na ₂ SO ₄	5	265	260
Al ₂ (SO ₄) ₃	3-4	65-75	60-70
AM-9	4-20	60-95	10-85
Arquad 2 HT-75	4	48	40
Aliquat H226	3-4	50-235	40-225
Chempact	4	40-45	35-40

In another study (21) in which Esrig examined the effect of introducing a number of chemical solutions at the anode, limited strength increases were obtained, Table 6. Maximum shear strength improvements of 640 psf (5⁺ psi) were attributed to electro-chemical treatment. However, in general, shear strength increases were limited to less than 200 psf (1.5 psi).

Where examined, the remolded values of liquid limit and plasticity index decreased after electro-chemical treatment.

Field studies have indicated that a slight increase in local strength may be obtained. The bearing capacity of friction piles has been significantly increased by the use of electrical treatment (17,43). Guseva (50) reported that electro-chemical treatment (calcium and magnesium chloride solution) improved the load bearing capacity of a foundation. The load bearing capacity increased from about 20 psi to 70-140 psi. Talme (9) has reported a slight increase in shear strength after electro-injection of a calcium chloride solution. Maximum shear strengths displayed however were less than 2 psi.

Costs Involved in Electrical Treatment

Due to lack of application, few cost figures are available concerning electro-osmotic stabilization of embankments. About \$2.50 per cubic yard of treated material was the estimated cost on the West Branch Dam (29). It can however, be stated that, compared to other methods of stabilization, electro-osmosis may be quite expensive. This factor alone has severely limited its usage (6,25,29,36).

Unfortunately, only one instance was found concerning direct cost data for electro-chemical treatment. Dearstyne and Newman (53) reported that the treatment of a runway embankment costs about \$35 per linear ft. The cost of treatment would include such items as:

1. electrical energy
2. cost of chemicals
3. cost of electrodes
4. labor costs

The amount of electrical energy as measured in KWH/yd³ might be used as a guide to the cost of electrical energy. Chemical costs can range from inexpensive for low concentration treatments with CaCl₂ to quite expensive if high concentrations of such chemicals as aluminum acetate are used.

Electrode cost would depend on the type of electrodes and the ability to salvage the electrodes.

CHEMICAL GROUTING

Introduction

The primary objective of the grouting process is to inject a grout into the pores of a soil system in order to increase the in-place stability and/or to decrease the permeability of the system. For these purposes, chemical grouts have a distinct advantage over particulate grouts in that the grouting material is dissolved in a liquid and may have a viscosity approaching that of water, thereby enhancing the ability to inject the grout into small pores.

A number of chemical grouts have been used in the construction industry, but in general, silicates, resins, chrome-lignin, and AM-9 have been the most widely used. Chemical grout applications include foundations, tunnels, dam sites, sewers, concrete structures, etc.

References 56 and 57 contain extensive bibliographies concerning chemical grouting. Reference 56 also contains a substantial listing and accompanying discussion of a number of field applications of chemical grouting. Poliuka, et al. (58) relate various field experiences with various chemical grouts. A general discussion of chemical grouting is included in References 6 and 59. Reference 60 is a chemical grout field manual that is primarily devoted to AM-9 type grout but contains discussion of the various aspects of chemical grouting.

Chemical Grouts

A number of chemical grouts have been developed. A majority of the early chemical grouts were developed for use in oil wells. However, the construction industry realized the applicability of various grouting

techniques and a number of specialized grouts were developed. A brief discussion of a number of the available chemical grouting materials is presented in Reference 56.

The properties of chemical grouts can vary considerably depending on the particular grout. Karol (61) has listed characteristics of an ideal chemical grout as follows:

1. ability to beneficially modify strength and permeability,
2. water soluble powder,
3. non-toxic and non-corrosive,
4. density and viscosity similar to water,
5. no change in density and viscosity until gelation,
6. instantaneous gelation (but controllable induction time),
7. no influence of soil chemicals, minerals, and impurities on the reaction, and
8. low cost and high availability.

Basically, there are two groups of chemical grouts. One group of chemical grouts contains those chemical grouts which form chemical compounds through metathetical precipitation. This grout includes the sodium silicate variety of grouts. The second group of grouts contains those chemical grouts which form chemical compounds through a polymerization process in which organic polymers are formed. Certain chemical grouting processes such as dissolution do not realistically belong in either of the aforementioned groups. Figure 22 depicts a few types of structures that are produced in the chemical compounds of certain chemical grouts. Table 7 lists a number of the chemical grouts and the chemicals involved.

a) Silicates

The sodium silicate chemical grouting process was probably first widely used under the name of the Joosten process. In this process, sodium silicate is combined with other chemicals to form an insoluble hydrated silicate. An acid such as hydrochloric may be used to neutralize the basic sodium silicate, and cause precipitation of the silicate; addition of a divalent or trivalent cation can also be used to cause precipitation of the

Table 7

A Number of Chemical Grouts and Their
Constituent Chemical Compounds

<u>Chemical Grout</u>	<u>Chemicals Involved</u>
Sodium Silicate	Sodium Silicate plus Calcium Chloride or Copper Sulfate
	Sodium Silicate plus Hydrochloric Acid
Chrome-Lignin	Lignin (Ligno Sulfito) plus Sodium Dichromate
Resins	
Epoxy	Resin base plus Hardener or Catalyst
Polyester	Polyester resin base plus Catalyst or Hardener
AM-9	Acrylamide N,N'- Methylenebis Acrylamide plus Activator Catalyst and Initiator Catalyst
Phenoplasts	Pheno plus Formaldehyde

silicate (62). Calcium chloride is often used as a source of the divalent cation but copper sulfate has also been used. In general, the rate of precipitation or gelation is controlled by the degree of dilution of the silicate.

b) Chrome-Lignin

In the manufacture of paper, lignosulphite is produced as a byproduct. Upon adding a dichromate solution to the lignosulphite liquor, precipitation or gelation occurs as the result of an oxidation process.

c) Polymers

By combining single organic molecules, referred to as monomers, in a polymerization process, organic macromolecules are formed. In some processes, polymers are injected; in others, monomers are injected; and in still others, partially polymerized monomers are injected. Heat, pressure, and catalysts are the three means used to initiate polymerization; the latter being the common means for grouting (56).

One of the greatest advantages of the polymer type grouts is the fact that immediately after mixing, the monomer mixtures often have viscosities approaching that of water. At least one polymer grout, AM-9, is dissolved in water prior to injection.

Included in the list of polymer type chemical grouts are: resins, phenoplasts, and AM-9. In resin grouts, a resin base (epoxy or polyester) is combined with a hardener or catalyst. Limited experience with resin chemical grouting is described in References 63 and 64.

Phenoplast grouts consist of a combination of phenol and formaldehyde (62). AM-9, a proprietary chemical grout, has received extensive discussion in the literature (65,66,67). Essentially, AM-9 consists of acrylamide N, N'-methylenebis acrylamide. It is combined with an activator or initiator catalyst prior to injection.

The dissolution chemical grouting procedure consists of injecting acid into a soil system. The acid is used to dissolve silicates in the soil and produce silicate cementing materials (56).

The Chemical Grouting Process

The normal process involved in chemical grouting consists of either mixing the chemicals and then injecting them, referred to as a "one-shot" process or to inject one chemical or group of chemicals and then inject another chemical or group of chemicals into the mass in what is often referred to as a "two-shot" process. Probably the first process used was the Joosten process (early 1900s) which was a two-shot sodium silicate grouting process (58,66).

In the one-shot process the major advantages are: minimum handling of materials and more complete penetration and greater grouting radius. The major disadvantage is related to the grout mixture setting or gelling prior to injection and clogging injection equipment. The two-shot process has disadvantages in that no control over the uniformity of mixing can be obtained and the radius of grouting is often limited by the rapid setting time of the materials close to the grout point. The major advantage is related to the ease of handling of the chemicals without the worry of set or gel prior to injection.

When chemicals are mixed together to be injected in a "one-shot" process or injected separately in a "two-shot" process, the materials must be in a liquid form. Many liquid chemical solutions have a viscosity close to that of water. In general, the lower the viscosity, the greater the ease of injection -- especially into materials with a low permeability. Figure 23 illustrates the viscosity of various chemical grouts. For comparative purposes, the viscosity of a bentonite grout has also been plotted in Figure 23.

Subsequent to mixing, the chemical solutions possess a fairly constant viscosity for a period of time, often referred to as the induction period. This is the period prior to set or gelation during which injection is normally attempted. As time progresses however, the viscosity of the chemical grout solution normally increases until an effective set or gelation has occurred. Figure 24 illustrates the effect of time upon the viscosity of a number of chemical grouts.

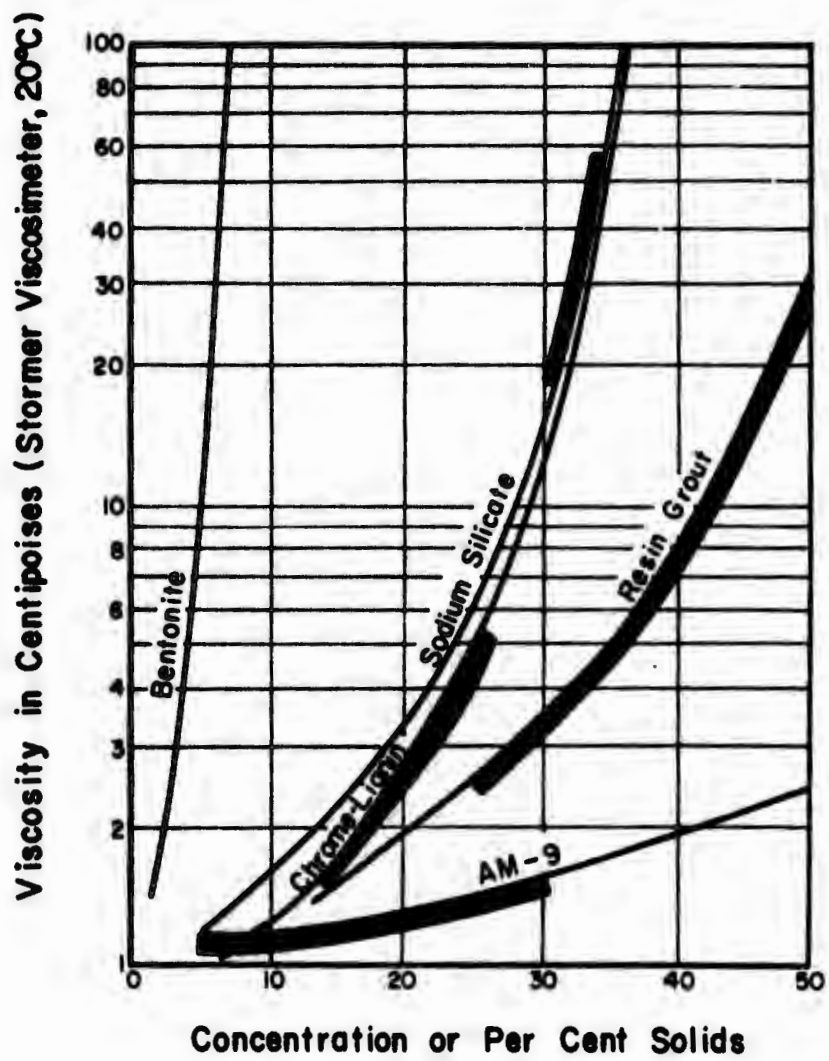


Figure 23 Viscosities of Various Grouts
(From Ref. 69)

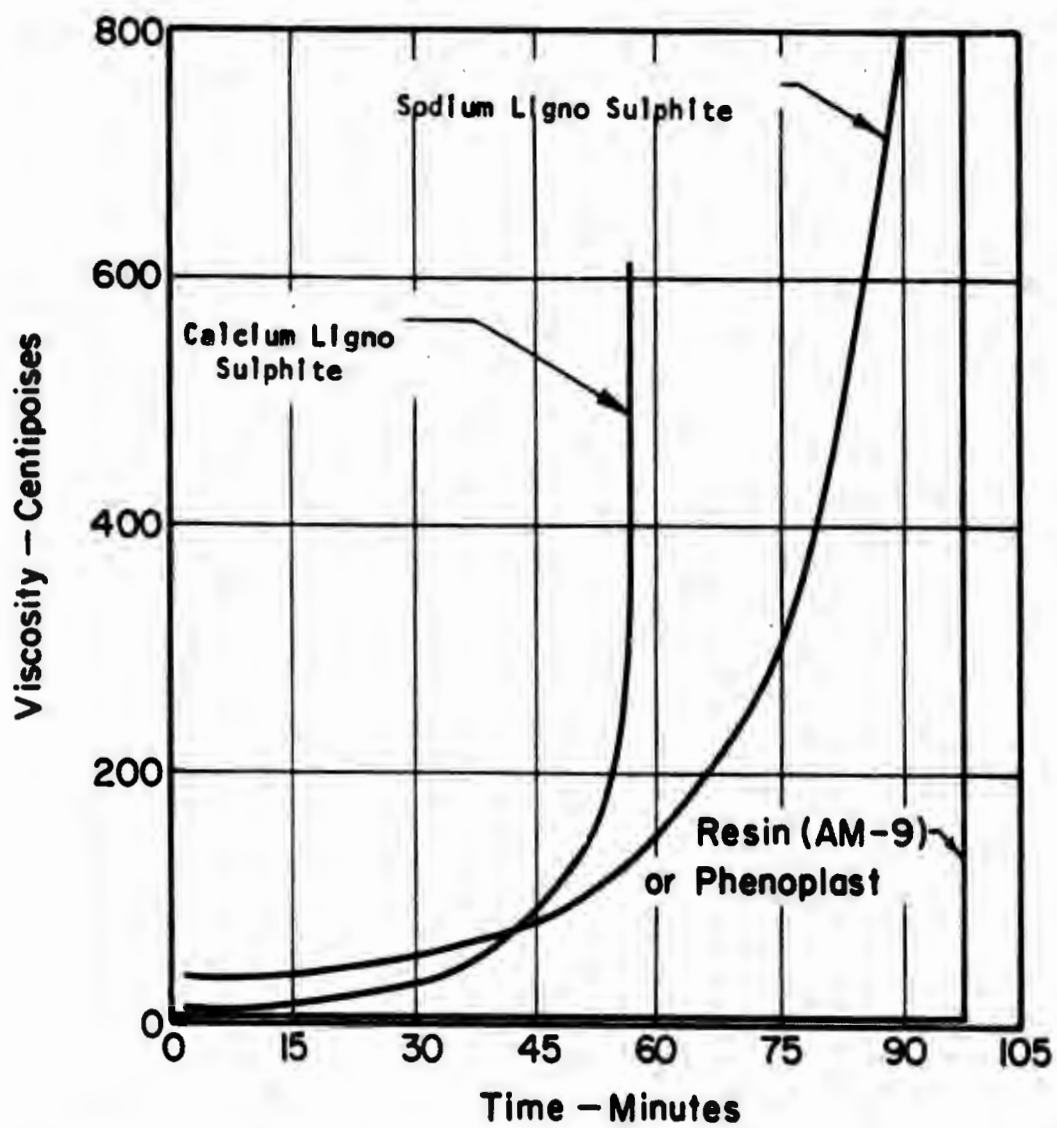


Figure 24 Viscosity Versus Time for Some Chemical Grouts (From Ref. 73)

Injection or grouting pressures are normally limited by the amount of overburden and commonly is limited to about one psi per ft of overburden.

Range of Materials Successfully Treated

Normally, successful chemical grouting is limited to usage with coarser textured materials such as sands, gravels, and fissured rock formations. In general, great difficulty is encountered in applying chemical grouting techniques to the treatment of cohesive finer-grained soils.

The main problem associated with chemical grouts relative to grouting of fine-grained materials is the inability to inject them into the system. Normally, the permeability is very low and the soil pores may already be filled with water. Even though the viscosity of a few chemical grouts approaches the viscosity of water, it may be difficult to displace the water in a saturated system and replace it with chemical grout.

Karol (6,66) has suggested an equation which can be used to determine the radial distance from a grout point to which a chemical grout will penetrate:

$$r = 0.62 \sqrt[3]{\frac{Rgt}{n}} \quad (4)$$

where

- r = radial distance, ft
- R = ratio of water to grout viscosity
- g = rate of grout take, cu. ft per minute
- t = gel time, minutes
- n = soil porosity

From this equation, it is noted that the radial distance and rate of grout take might be increased by increasing the time of injection or decreasing the viscosity of the grout. In addition, increased grouting pressure will increase grout take thereby increasing the radial distance; however, grouting pressure is normally limited in embankment grouting operations due to the shallow depths of grouting.

Figure 25 illustrates typical ranges in soil particle size in which various chemical grouts have been successfully used. Silicate, resin, and chrome-lignin grouts are normally restricted to use with soils in which the minimum particle sizes are larger than 0.07 to 0.15 mm. Caron, et al. (63) have reported the successful use of resins in soils with as much as 60 percent passing the number 200 sieve (0.074 mm), Figure 26. Abelev and Askalonov (68) reported the use of sodium silicate grouts with silty materials with plasticity indices as great as 18 percent which would indicate a fine-grained texture.

Although isolated cases of use with quite fine-grained materials have been reported, normally in the application of silicate, resin, and chrome-lignin grouts, a D_{10} of the soil of greater than 0.1 mm is desirable.

AM-9, due to its lower viscosity, reportedly can be successfully injected into finer-grained materials than other chemical grouts (69). Figure 27 illustrates the particle size distribution of soils in which AM-9 has been used.

Effect of Treatment

In this project the major objective is to improve the strength of an in-place fine-grained soil. Most chemical grouting processes have been used to fill the voids and decrease permeability. The strength of the material which has been treated with grout is primarily controlled by three factors. First, the strength of the natural in-place material; secondly, the degree of uniformity to which the grouting material has been injected into the system; and thirdly, the inherent strength of the grouts. In fine-grained soils natural strength may be quite low. In addition, in the fine-grained soils it is very difficult to inject any type of grout whether chemical or other types to create a uniformly treated mass. As a result, the strength increase obtained from chemical grouting fine-grained cohesive soils may be rather limited.

The inherent strength of a chemical grout may be rather high. Although not typical of all chemical grouts, Erickson (64) has reported that various resin grouts (epoxy and polyester) can have tensile strengths on the order

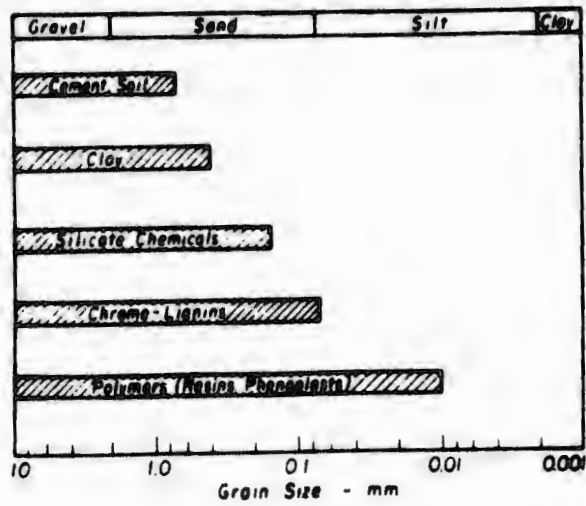


Figure 25 Soil Grain Size Ranges Over Which Different Grout Types are Useful (From Ref. 73)

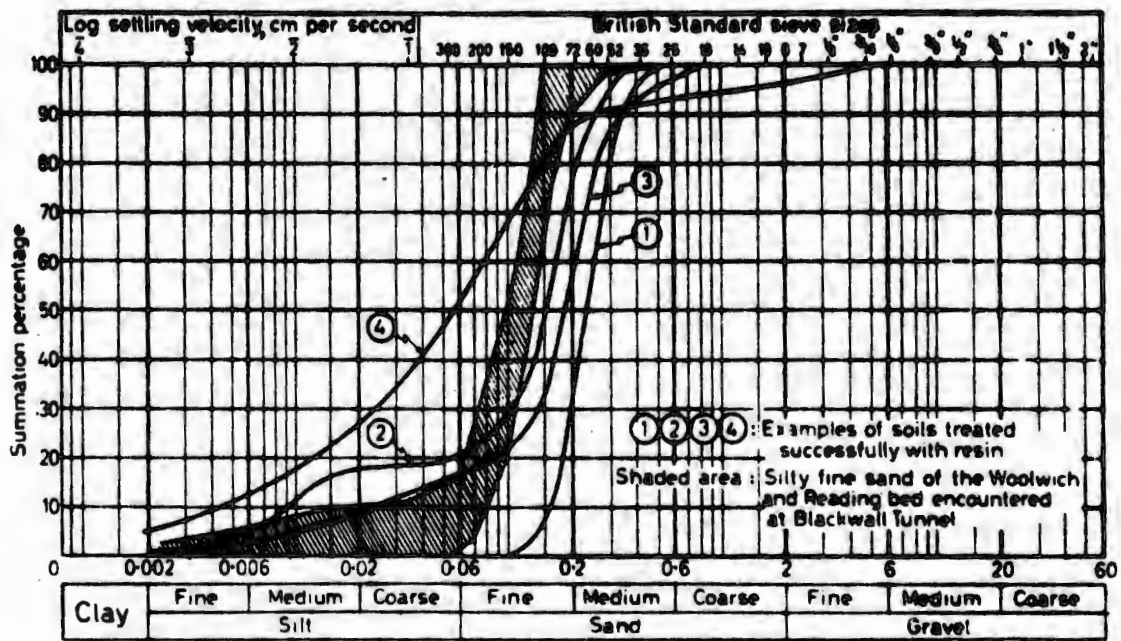


Figure 26 Grain Size Distribution of Soils in Which Resin Grouts Have Been Successfully Used (From Ref. 63)

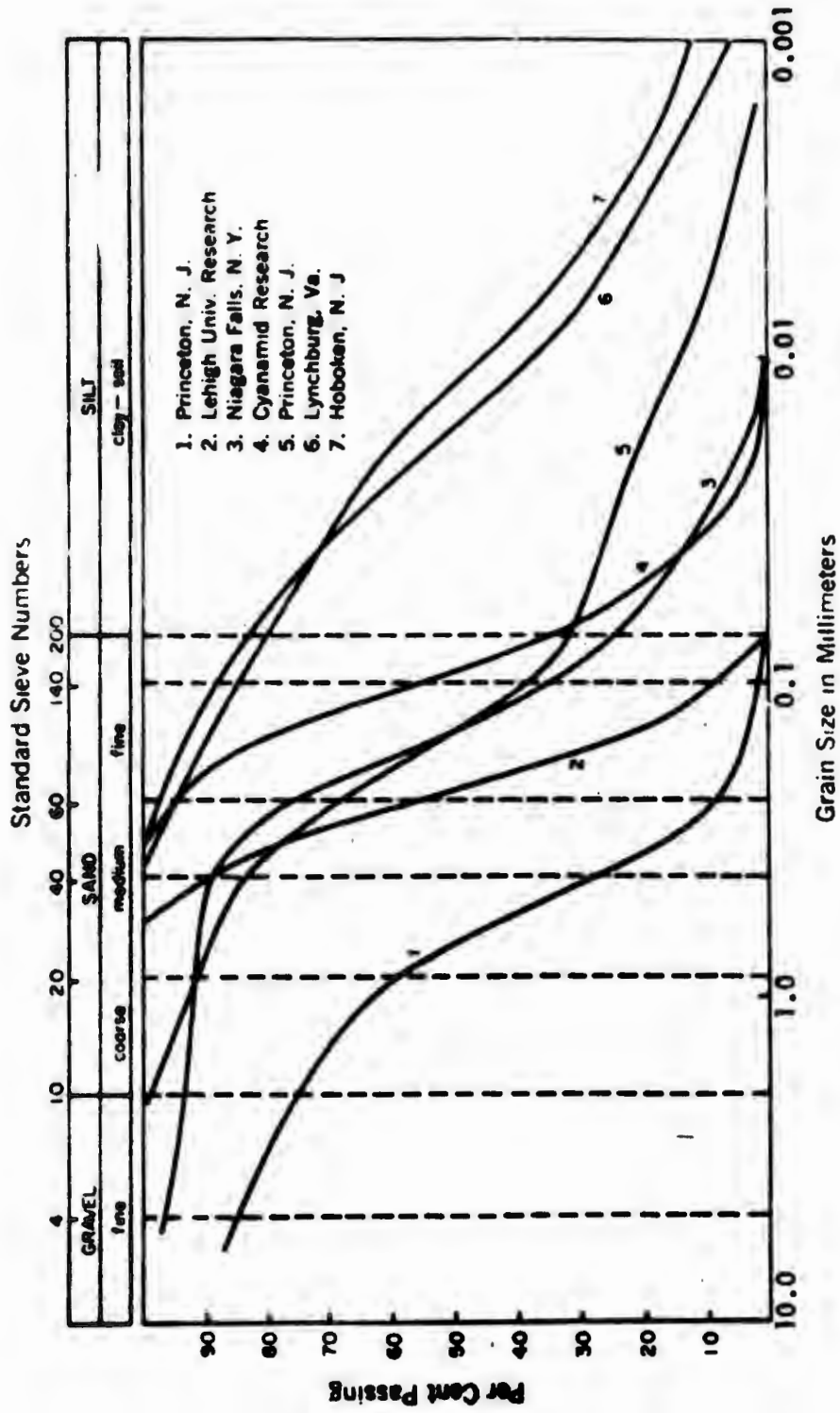


Figure 27 Typical Soils Injected With AM-9 Chemical Grout (From Ref. 69)

of 6,000 to 9,000 psi, flexural strength on the order of 9,000 psi, and compressive strengths ranging from 10,000 to 20,000 psi. With grout strengths of this magnitude, if uniform and thorough injection of the grout into the pores could be accomplished a high strength mass could be obtained. Abelev and Askalonov (68) have reported the successful treatment of loess with a sodium silicate chemical grout. Reported compressive strengths (unsoaked) for the treated loessial soils ranged from 85 psi to 350 psi. Polivka, et al. (58) reported compressive strengths of a fine sand treated with sodium silicate to range from 100 to 500 psi. Tschebotarioff (70) has reported the use of sodium silicate to control the vibration of a gas compressor foundation on fine sand. Gnaedinger (71) has presented results of strength studies on sodium silicate and AM-9 treatment of a sand which indicated that the modulus of elasticity ($\sigma_3 = 5$ psi) was increased approximately four times over the natural soil. Schiffman, et al. (67) reported the results of sand treated with AM-9, Figure 28, where increased relative density decreased the strain at maximum stress and increased the maximum stress. The AM-9 manual (69) indicates that AM-9 grouts are rather rubbery elastic materials and for dense soils the stabilized strength is normally primarily governed by the strength of the soil.

Caron reported shear strengths of 7 to 10 psi for ligno-sulphite-chromate-sand mixtures (62).

Cost of Chemical Grouting

The cost of chemical grouting depends on a number of factors including: type of material to be grouted, type and amount of chemical grout, complexity of grouting process, and size of project.

Reference 56 indicates that the labor costs in chemical grouting may be greater than the chemical costs. In general, silicate and chrome-lignin grouting materials will be less expensive than the resins, phenoplast, and polymer grouting materials. Fern (72) reports that the total cost of AM-9 grout per cubic ft of grout ranges from \$4 to \$8. References 56 and 73 indicate that normal costs of chemical grouting range from \$25 to \$100 per cubic yard of treated material.

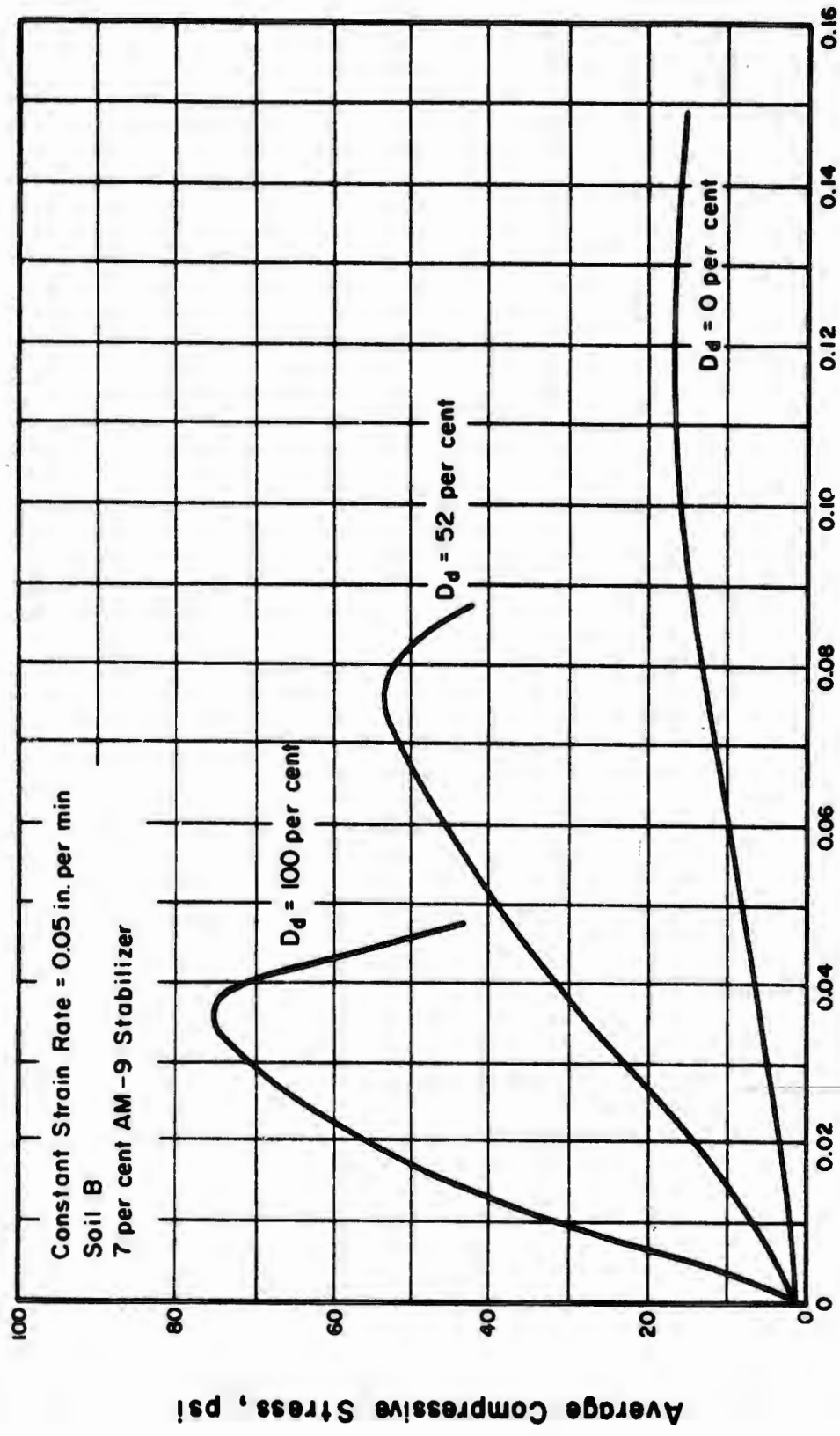


Figure 28 Typical Unconfined Compression Stress-Strain Behavior of a Sand Treated with 7 Percent AM-9 (From Ref. 67)

CEMENT GROUTING

Introduction

Cement pressure grouting has been used extensively in the construction industry since about 1900 (75). Cement grout consists of a mixture of cement particles and water. Upon being mixed with water, a hydration process is initiated. Injection of the grout must be made prior to "set" of the grout. Applications of cement grouting have ranged from cut-off walls and void filling to mud jacking, strength improvements, and mixed-in-place soil-cement piles or intrusion grouting.

Pertinent Applications

The cement grouting task committee of ASCE has prepared an excellent bibliography (76) concerning cement grouting. ASCE also has published results of a symposium on cement grouting (77).

A large volume of the literature has been devoted to cement grouting as it pertains to grouting of rock excavations and foundations. Other reported applications include grouting of permeable soil deposits under dams in order to minimize water leakage. In the latter, often referred to as impermeabilization applications, cement grout curtains or cut-off walls are constructed with the grout. Cement grout has been used to increase the strength of foundations under buildings, dams, heavy machines, bridge piers, etc.

Applications of most pertinence to this project consist of grouting in roads, pavements, and railroad embankments.

a) Roads and Pavements

Unfortunately the literature indicates that most of the cement grouting applications for roads and pavements have been concerned with "mud jacking." In this type of application, grout is pumped under a faulted pavement at sufficient pressure and in sufficient quantity to literally jack the pavement back into position by the increased volume caused by the cement grout. In most mud jacking applications, very little grout permeates the subgrade. In some cases, the high pressure grout injection may be used to densify the

soil surrounding the bulb of cement grout (Compaction Grouting). In general, however, saturated clay soils cannot be densified by this procedure (78). Typical descriptions of mud jacking may be found in References 79, 80, and 81. A discussion of the theory of mud jacking and compaction grouting may be found in Reference 78.

b) Railroad Embankments

Cement grouting has been used to increase the stability of soft, squeezing embankments and ballast roadbeds of railroads (76). Peck (82) and Smith, et al. (83,84,85,86) have reported that varying degrees of stabilization were obtained when cement grout was pressure injected into the roadbeds. In some cases, grout seams were formed in the fine-grained embankments under the ballast, Figure 29. In a majority of the cases, however, the pressure injected grout formed a discontinuous seam at the ballast-embankment interface. Substantial penetration of the grout into the more permeable ballast was experienced in many cases, Figure 30.

The cement grout used on these jobs normally contained a sizeable quantity of sand filler (asphalt emulsion or flyash was also added in certain cases) and was pressure injected to typical depths ranging from 3 to 10 ft. Grout-take normally ranged from 1 to 3 cu.ft per linear ft of track although in one case the take was 19 cu.ft per linear ft of track.

The effect of the grouting was normally measured in terms of reduced maintenance expenses, although a limited deflection study was conducted (82). In most cases, track maintenance expenses decreased. The deflection was reduced after grouting from 0.090 in. to 0.050 in. under a "light" wheel load and from 0.215 in. to 0.162 in. for the "heavy" wheel loads. This would indicate that the stability of the ballast-embankment system had been improved.

c) Intrusion Mixed-in-Place Cement Grouting Procedure

Mixing of cement grout and soil to form pile-like grout columns is a relatively common procedure. Intrusion-Prepakt, Inc. has a patented procedure in which the cement grout is mixed with the soil by employing a section

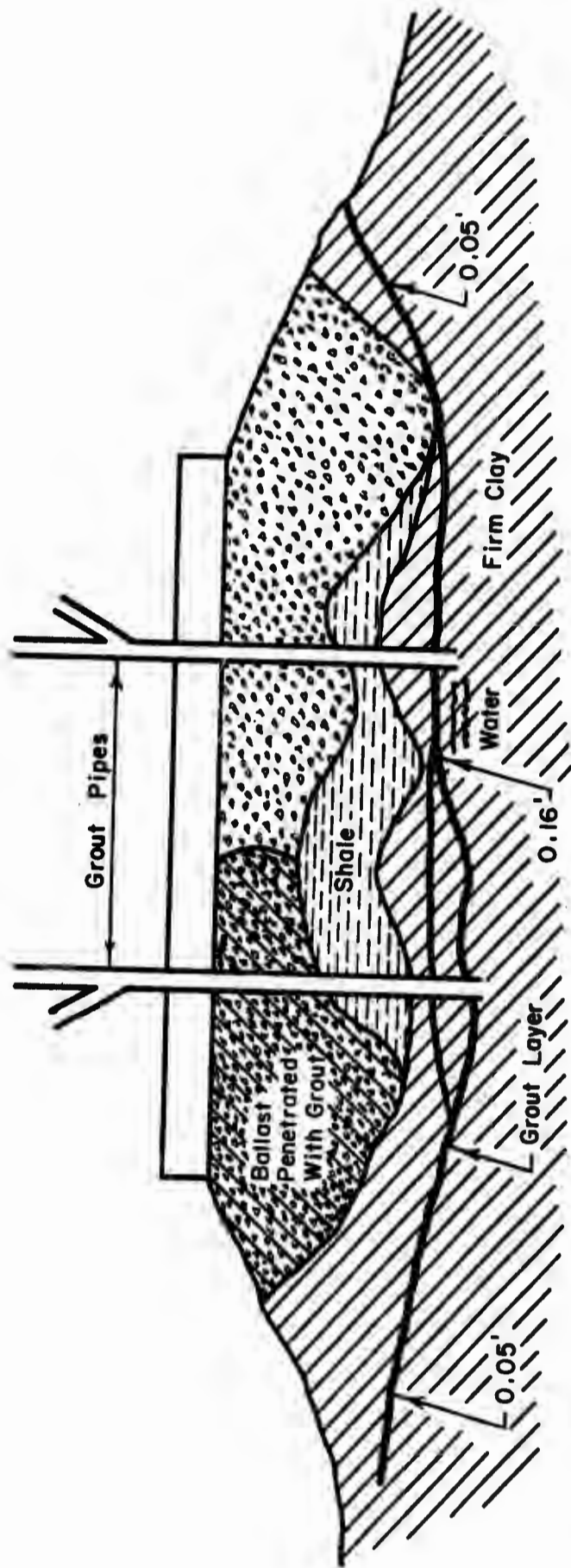


Figure 29 Cross Section of Railroad Embankment Illustrating Results of Cement Grouting Operation
 (From Ref. 82)

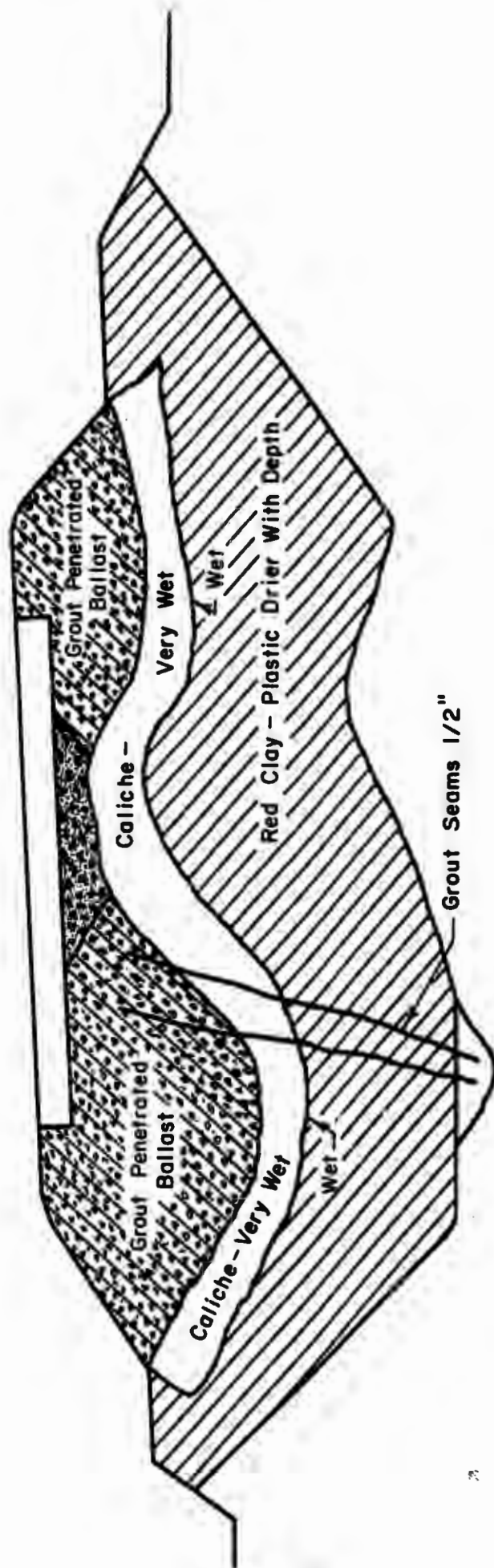


Figure 30 Cross Section of Railroad Embankment Illustrating Extensive Cement Grout Penetration of the Ballast (From Ref. 83)

of continuous flight auger or a mixing blade with grout outlet ports in the lower end and a hollow stem, Figure 31. As the auger or mixing blade is advanced into the soil, a controlled quantity of grout is pumped through the hollow shaft and distributed through the outlet ports.

The Intrusion-Prepakt cement grout is a mixture of cement, fly ash, intrusion aid, and water (87,88). The strength of the cement grout-soil columns produced by the intrusion procedure depends on: 1) type of soil, 2) volume of grout, 3) water/cement (W/C) ratio of grout, 4) intimacy of mixing, and 5) curing time. Figure 32 depicts typical stress-strain characteristics and the influence of grout volume on the strength of Mugu Mud mixed with cement grout. It may be noted that grout volumes in excess of 50 percent of the total volume have been used. The intimacy of mixing, which is normally controlled by the rate of advancement of the auger or mixing blade into the ground, has a marked effect upon compressive strength, Figure 33.

A number of field applications of the Prepakt procedure have been reported (87). A field intrusion cement grouting experiment was performed in natural mud deposits at San Francisco Naval Shipyard (87). It was concluded from the study that the process of mixing grout in-place with soft soils to form pile-like elements is feasible within limits, but is more adaptable to repair jobs and small-scale construction than to large permanent works (87).

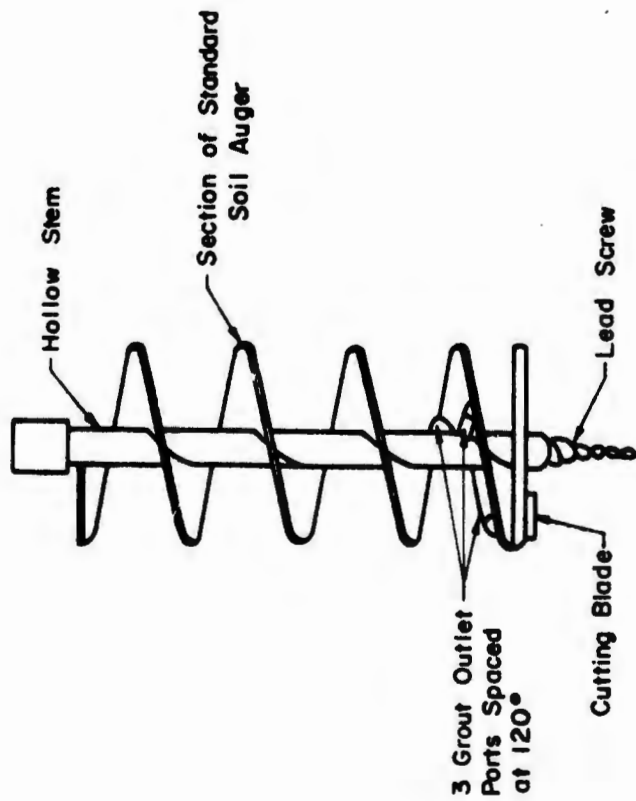
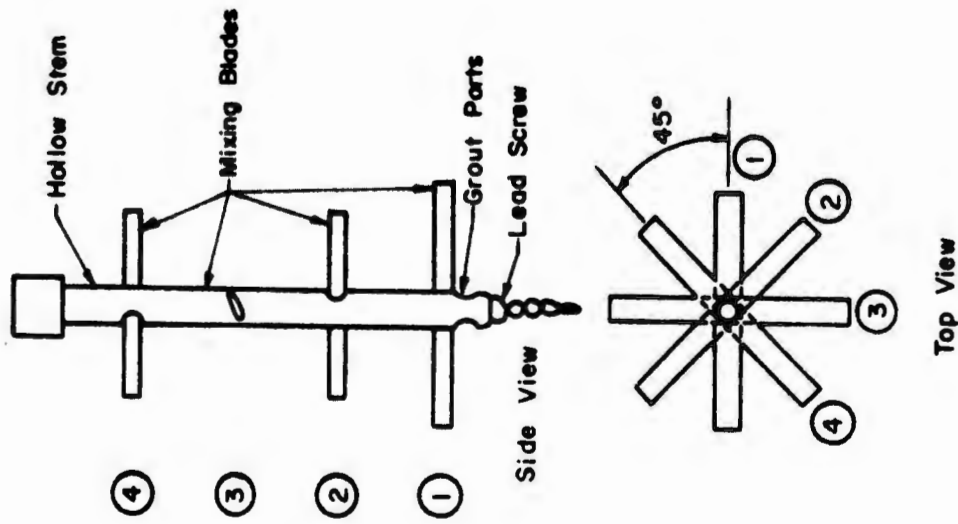
The penetration of the cement grout into the soft mud was negligible -- strength improvement to the mass was accomplished by the formation of grout piles.

d) Other Pertinent Applications

Cement grout has been used to arrest slope failures along highway (89) and railroad embankments (84,85).

Important Aspects of Cement Grouting

Cement grout consists of a suspension of cement particles in water. The water is used both as a transporting medium and to effect cement



(a) Auger Type Mixing Head

(b) Blade Type II Mixing Head

Figure 31 Schematic Diagram of Two Types of Mixing Heads Used in Intrusion-Prepakt Cement Grouting Procedure (From Ref. 88)

Sand Intrusion Grout
Mixed With Mugu Mud

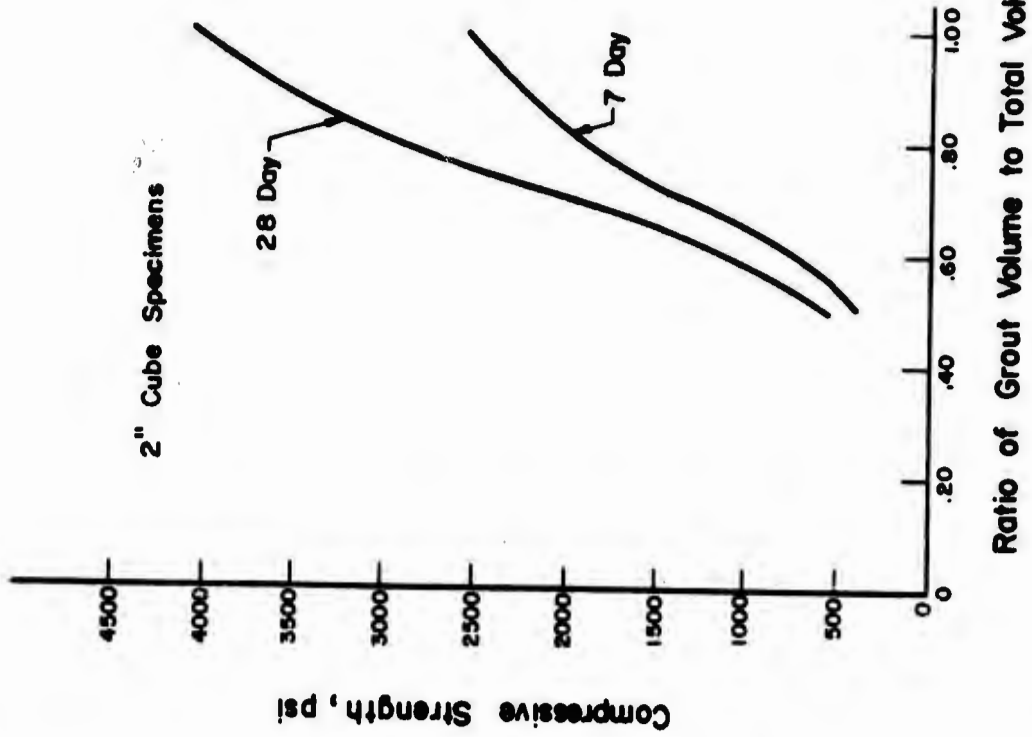
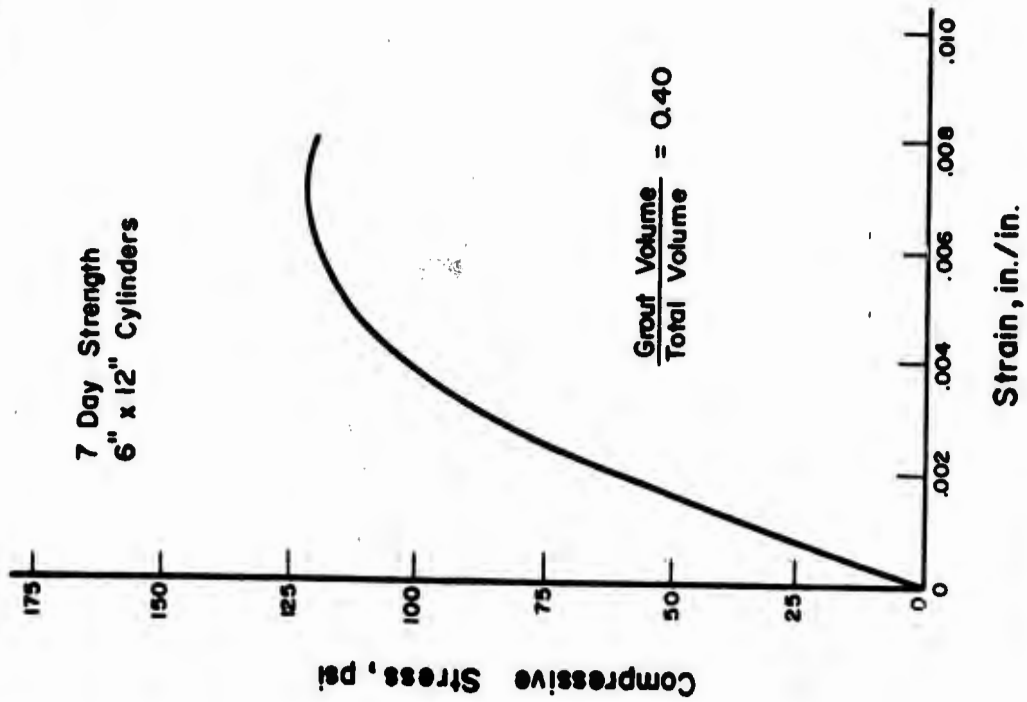


Figure 32 Typical Stress-Strain and Strength vs Grout Volume Relationships for Intrusion-Prepakt Cement Grout-Soil Mixtures (From Ref. 88)

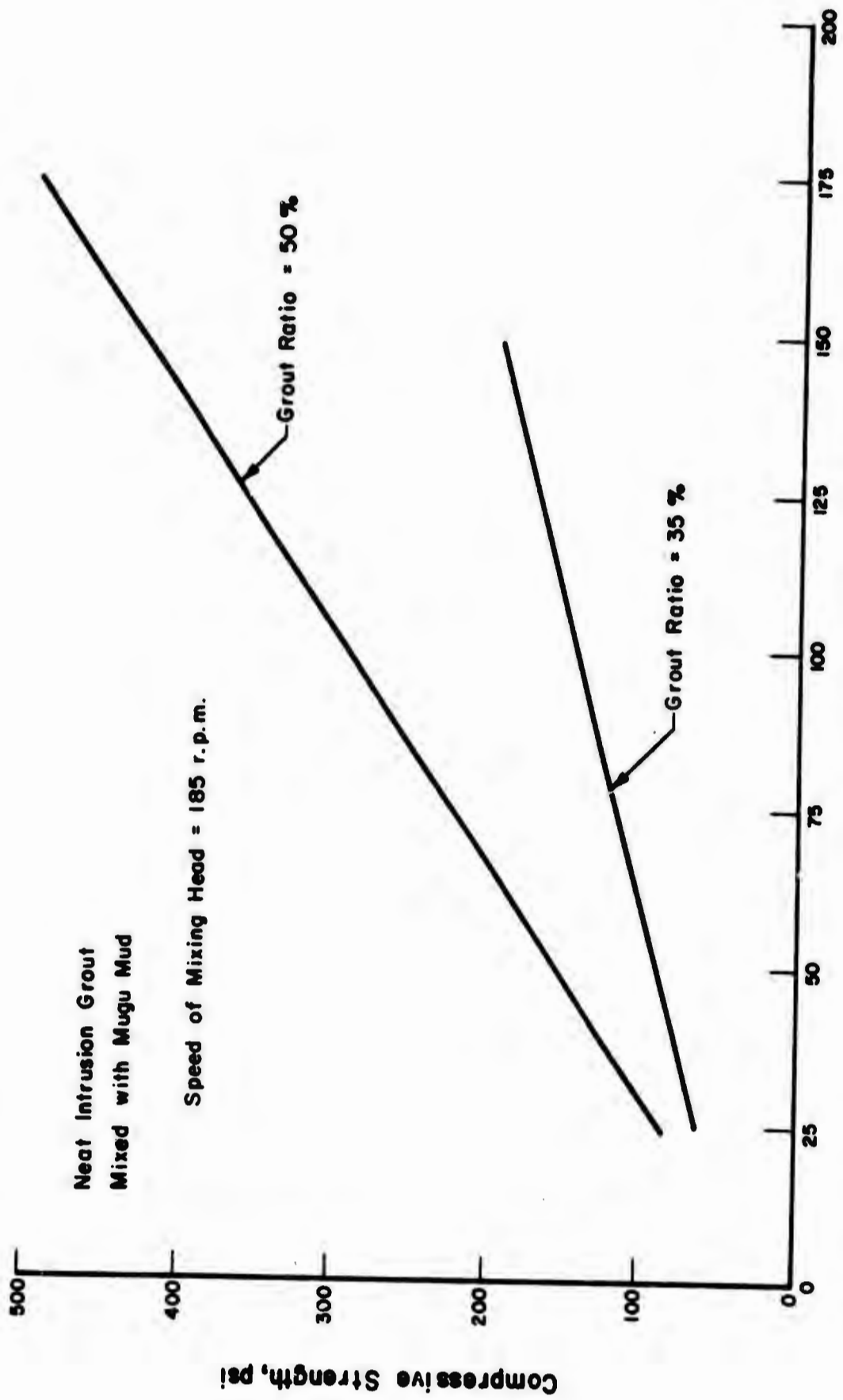


Figure 33 Typical Influence of Amount of Mixing of Cement Grout-Soil System on Compressive Strength (From Ref. 88)

hydration. Additives and fillers such as calcium chlorides, sand, fly ash, and emulsified asphalt may be added to the grout mixture, depending on job requirements. Type I cement is normally used although Type III has been used because of its greater relative fineness (90). The grout is normally mixed to the thickest consistency that will allow ease of injection into the mass to be grouted. Air pressure or positive displacement pumps are normally used to force the grout into the ground. There are a number of important factors that affect the cement grouting processes.

a) W/C Ratio

The fluidity of a cement grout is controlled by the amount of cement added. Typical water:cement ratios range from 10:1 to 4:1 (by volume) (90). Kennedy (90) stated that for finer-textured materials, if W/C ratios of less than 3:1 are used, pressures greater than 450 psi can not inject the grout into the system. Kravetz (91) indicates that cement grouts may not set up if W/C ratios are greater than 10:1. He further indicates that the elapsed time before setting increases with an increasing W/C ratio (91).

b) Strength

The strength of cement grout is very dependent on the W/C ratio; normally decreased strength is associated with increased W/C ratios. Johnson (92) indicates that strength is normally determined for the as-mixed grout neglecting any loss of moisture. Typical unconfined compressive strengths are depicted in Figure 34.

c) Grouting Pressures

Grouting to shallow depths may limit the grouting pressure. Lippold (93) reports that a "rule of thumb" used for pressure grouting is that the pressure should be limited to 0.75 to 2.5 psi per ft of cover. However, cement grouting pressures of up to 100 psi were reported for the railroad roadbed stabilization projects where cover thickness was approximately 5 to 10 ft (82,83).

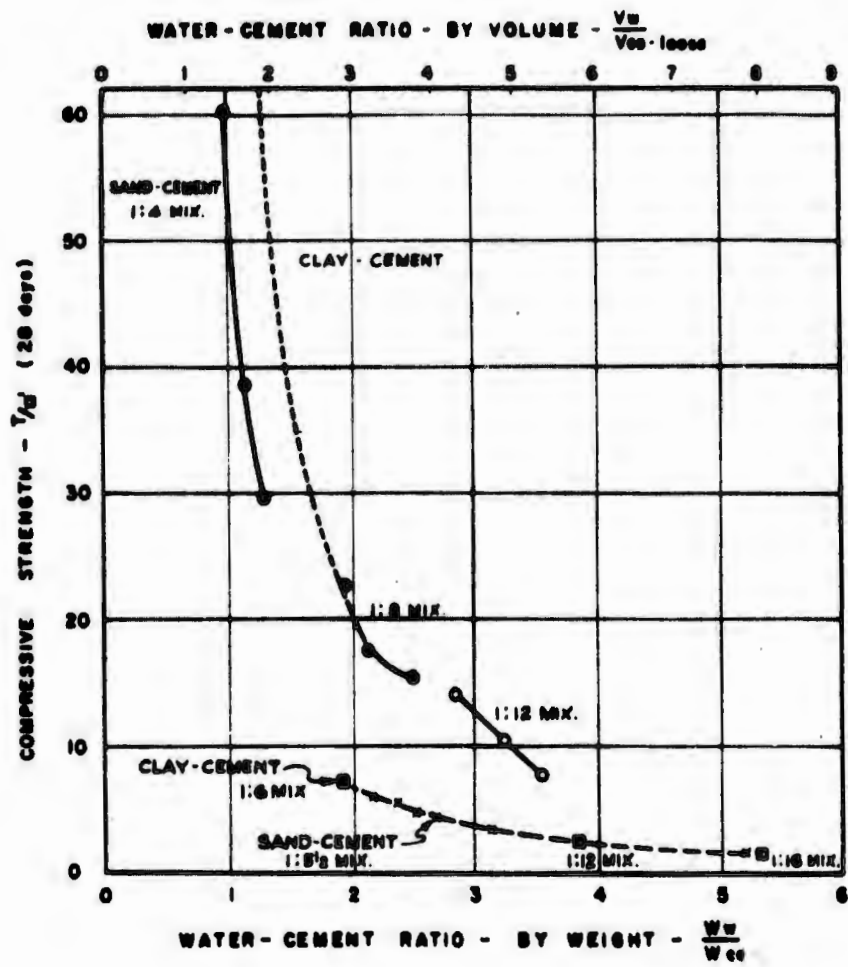


Figure 34 Typical Compressive Strengths of Clay-Cement Grouts at Various W/C Ratios (From Ref. 92)

Groutable Materials

Cement grout can be pressured-injected into the void spaces in a soil or rock mass. Owing to the particulate nature of cement grout, the minimum pore size that the grout will penetrate is limited. A basic parameter that indicates the ability of a material to accept grout is its permeability. Kollbrunner and Blatter (94) indicate that a coefficient of permeability greater than $k = 1 \times 10^{-1}$ cm/sec is necessary for the penetration of cement grout. This would limit the use of cement grouting to materials more permeable than fine sand. Machis (95) states that "no amount of pressure can make a cement slurry pass through sands with grains finer than 0.59 mm." Kennedy (90) does indicate that by using a Type III specially scalped cement, the D_{10} of the material to be grouted can be as small as 0.29 mm.

Probably one of the most useful criterion for determining cement groutability is the "groutability ratio" suggested by Johnson (92). The groutability ratio for cement grouting is represented in the following equation:

$$\text{Groutability Ratio} = \frac{(D_{15}) \text{ soil}}{(D_{85}) \text{ grout}}$$

Johnson (92) indicates that this ratio should be greater than 20 to 25 for successful cement grouting. This criterion requires that the D_{15} of the soil must be larger than 1.6 to 2.0 mm since typically the D_{85} of Type I cement is on the order of 0.08 mm. For special Type III cement ($< 30\mu$), the D_{15} of the soil must be larger than 0.6 to 0.75 mm (silt size).

Effects of Cement Grouting

The stability or strength of a grouted mass is dependent on a number of factors including: inherent strength of the soil, inherent strength of the cement grout, the amount of cement grout placed in the system, and the intimacy attained in the grouting process.

Due to the fine-grained nature of many low-strength subgrades, it may be virtually impossible to intimately permeate the soil with cement grout; although in some cases, fine sands and coarse silts might be successfully

injected (see section on groutability). As a result, cement grout injection into fine-grained soils probably will produce layers, seams, bulbs, spikes, etc. of grout rather than an intimate mixture. When this happens, strength improvement may be effected in stabilized zones and the degree of strength improvement will depend upon the relative strength of the grout and the relative amount of grout in the system.

The strength of the cement grout is dependent on the W/C ratio and if used, the amount of sand filler. Figure 34 illustrates 28-day compressive strength for various cement grouts. From this figure, it is noted that at W/C ratios greater than about 3:1 compressive strengths of the grout are less than about 140 psi. Cedergren (96) indicates that the strength of sand-cement grouts normally ranges from 100 to 700 psi. For cement grouts containing clay filler average grout strengths are about 100 psi (96).

It is apparent, that unless large quantities of cement grout are injected into the system to form improved zones, the strength of the cement grout-soil system may be only slightly improved over the ungrouted strength.

Costs

The costs involved in cement grouting can vary greatly depending on the type of material to be grouted, the amount of grout used, the extent of the job, and various other job requirements. Smith (84) reported cost figures for cement grouting of soft yielding railroad subgrades that ranged from \$0.25 to \$2.13/ft³ of grout placed with an average of \$0.53/ft³. Sherard (97) indicated that typical cement grouting costs will average \$8 to \$15/yd³ of treated material.

LIME STABILIZATION PROCEDURES

Introduction

Lime has been widely and successfully used as a stabilizing agent for fine-grained plastic soils. Most applications, however, have been directed toward improving the properties (strength, plasticity, workability, etc.) of the fine-grained soils in order that the material can be used as pavement structural layers (base, subbase), an improved subgrade, or a construction "working table." In most conventional applications, lime (3 to 10 percent by weight) is added to the soil and an intimate blending is effected by

various pulverizing and mixing operations. Following the blending operations, the lime-soil mixture is compacted.

The usual pulverization, mixing, and compacting operations, common to "original construction," are not possible when stabilizing in-situ fine-grained subgrades beneath an existing pavement. Requisite to deep layer stabilization, any stabilization scheme chosen must not cause significant disturbance to the existing pavement surface. As a result, certain of the advantages gained by using lime, as in conventional construction, cannot be obtained in the stabilization of in-situ embankments. However, a number of laboratory studies and field applications have indicated that lime can still be effectively used to stabilize in-place embankments under existing pavements. Procedures that have been or might be used to introduce lime into the subgrade include: (a) introducing quick or hydrated lime into a small diameter shaft drilled into the embankment (drilled-hole lime), (b) high-pressure injection of a lime slurry into the embankment, and (c) mixing in-place cylindrical zones of lime and soil.

Stabilization Mechanisms

Lime is an "active" stabilizing agent -- that is, it reacts with fine-grained soils causing physical and chemical changes during the stabilizing process. Thus, physical and chemical soil properties are very important. The fine-grained soil material is actually a reactant and not simply an inert mass.

When lime is mixed with a fine-grained soil in the presence of water, the pH of the system increases to about 12.3, the pH of a saturated lime-water solution. Although lime is not highly soluble in water (0.1 to 0.2 gm per 100 gm of saturated solution), excessive Ca^{++} cations are introduced into the system. Once the pH is raised and excessive calcium cations are present, occurrence of the common stabilizing mechanisms may proceed.

a) Cation Exchange and Flocculation/Agglomeration

Two mechanisms, cation exchange and flocculation/agglomeration, are initiated soon after the addition of lime. Cation exchange occurs rapidly

as the Ca^{++} cations replace other cations on the surface of the clay particles. The result is primarily a calcium clay which exhibits reduced plasticity index and swell characteristics. Although the plasticity index is reduced substantially, no significant strength increase is attributed to cation exchange.

Cation exchange will occur if excessive calcium cations are available regardless of the soil "state." That is, a soil need not be pulverized and manipulated for cation exchange to occur. For example, if a lime slurry is dumped into a hole drilled in an embankment, exchange will occur readily around the periphery of the hole and outward in fissures, etc., in the soil mass. As the calcium cations migrate through the soil, cation exchange will progress even in the undisturbed soil state.

In systems where manipulation and pulverization are possible, flocculation and agglomeration of the soil particles are initiated soon after the lime is introduced. Conceptually, the mechanism can be viewed as a reorientation of soil particles from the dispersed state to random (flocculated) orientation. The flocculated soil grains also cluster together into agglomerate particles forming a secondary, aggregate-like structure. Immediate changes in soil plasticity, workability, swell, and strength characteristics occur. Plasticity and swell are reduced and workability is substantially improved because of the low plasticity and the friable nature of the mixture. Thompson (98) and Neubauer and Thompson (99) have shown that significant improvements in strength (modulus of deformation, CBR, etc.) are effected immediately after lime is mixed with a fine-grained soil, Table 8 and Figure 35. Immediate unsoaked CBR values ranging from 4 to 25 have been noted. Stability improvement displayed by lime treated soils at very early ages has been attributed to the flocculated structure which is formed.

In confined systems where no mechanical manipulation or mixing can be accomplished, it is highly unlikely, though not impossible, that particle flocculation and agglomeration can occur. Such a reorientation of particles may occur at disturbed, unconfined interfaces such as the periphery of a drill hole. It is difficult to conceive how such structural changes can occur in a mass with overburden and confining pressures. Although cation

Table 8 CBR Values For Selected Soils And Lime-Soil Mixtures (From Ref. 98)

Soil	Natural Soil		Lime	No Curing (a)		Lime-Soil Mixtures	
	CBR, %	Swell, %		CBR, %	Swell, %	CBR, %	Swell, %
<u>Good Reacting Soils</u>							
Accretion Clay 2	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Clay 3	3.1	1.4	5	88.1	0.0	370.0	0.1
Byce B	1.4	5.6	3	20.3	0.2	197.0	0.0
Champaign Co. Till	6.8	0.2	3	10.4	0.5	85.0	0.1
Clans B	2.1	0.1	5	14.5	0.1	150.0	0.1
Cowden B	7.2	1.4	3	--	---	98.5	0.0
Cowden B	4.0	2.9	5	13.9	0.1	116.0	0.1
Cowden C	4.5	0.8	3	27.4	0.0	243.0	0.0
Darwin B	1.1	8.8	5	7.7	1.9	13.6	0.1
East St. Louis Clay	1.3	7.4	5	5.6	2.0	17.3	0.1
Fayette C	1.3	0.0	5	32.4	0.0	295.0	0.1
Illinoian B	1.5	1.8	3	29.0	0.0	274.0	0.0
Illinoian Till	11.8	0.3	3	24.2	0.1	193.0	0.0
Illinoian Till	5.9	0.3	3	18.0	0.9	213.0	0.1
Sable B	1.8	4.2	3	15.9	0.2	127.0	0.0
<u>Non-Reactive Soils</u>							
Fayette B	4.3	1.1	3	10.5	0.0	39.0	0.0
Miami B	2.9	0.8	3	12.7	0.0	14.5	0.0
Tenn B	2.6	2.0	3	4.5	0.2	9.9	0.1

(a) Specimens were placed in 96 hour soak immediately after compaction

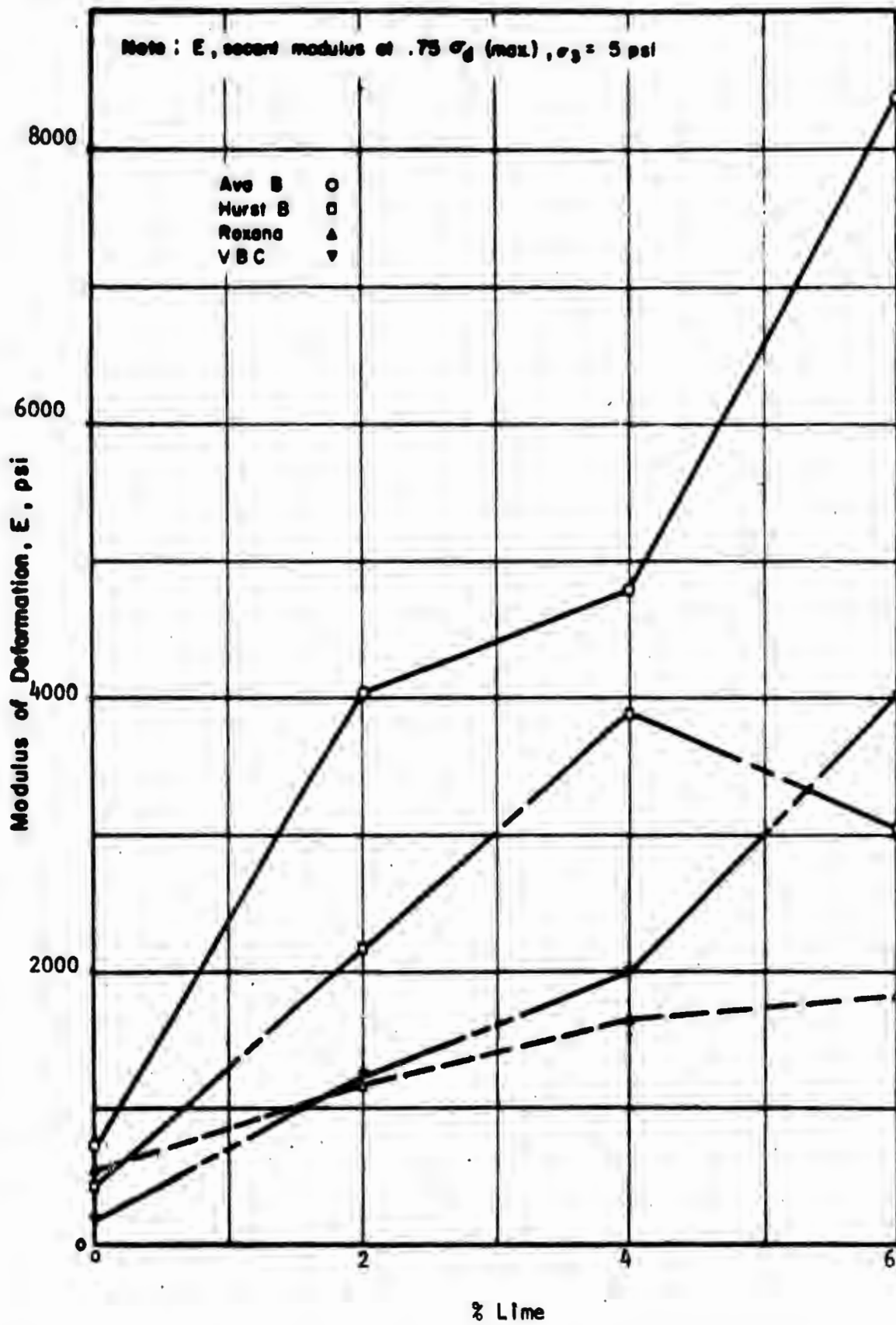


Figure 35 "Immediate Effects" of Lime Treatment on Modulus of Deformation (From Ref. 99)

exchange takes place, the improved strength properties that result from flocculation and agglomeration in an open system cannot be obtained throughout the soil mass. Thus, significant early gains in strength cannot be anticipated due to the flocculation/agglomeration mechanism.

b) Carbonation

Unlike cation exchange and flocculation/agglomeration this reaction is not immediate. Carbonation refers to the combination of calcium hydroxide or calcium oxide in the lime with carbon dioxide from the air or ground water to form calcium carbonate, a reversal of the lime-producing process. Although weak cementing agents are formed, the carbonation reaction depletes the "supply" of lime needed for the pozzolanic reaction (discussed below), eventually leading to a drop in pH. In conventional stabilization, care is taken to seal the pulverized mix and thus avoid prolonged exposure to CO_2 from the atmosphere. If such precautions are not taken, especially in industrial areas, normal strength increases may be reduced.

Carbonation of lime used in in-situ embankment stabilization due to CO_2 in the air would be less extensive than in conventional practice. However, CO_2 may be dissolved in the ground water, resulting in carbonation within the treatment area and surrounding soil already penetrated by lime. No quantitative data are available to establish what strength increases may be attributed to the weak cementing agents. Proposed techniques whereby carbon dioxide is "pumped" into the soil mass to react with the lime and form calcium carbonate are presently unproven. It seems highly unlikely that CO_2 and lime could be uniformly distributed throughout a confined soil mass. Likewise, the weak cementing agents formed could hardly be depended upon to substantially increase subgrade CBR values.

c) Lime-Soil Pozzolanic Reaction

The most important lime-soil reaction mechanism (with respect to strength increases) is the pozzolanic reaction. The reaction involves Ca^{++} contributed by the lime and silicate and/or aluminate liberated from the soil at high pH levels. They combine to form various types of hydrated

calcium silicates and/or calcium aluminates. These strong cementing products are similar to those formed in the hydration of portland cement.

The magnitude of strength gain displayed by a lime-soil mixture is a measure of the "lime-reactivity" of the particular soil. The degree of lime-reactivity has been found to be controlled by various physical and chemical soil properties. Thompson (100) conducted an extensive study concerning the lime-reactivity of Illinois soils. He found that such factors as natural soil pH, degree of weathering, clay mineralogy, organic matter content, soil horizon, and type of natural drainage have a significant influence on the degree of lime-reactivity of fine-grained soils (100). Since the soils in Thompson's study (100) were quite limited in geographical extent, the preceding list of soil properties found to influence lime-reactivity of Illinois soils is by no means complete for all fine-grained soils of the world. For example, a study (101) presently in progress concerning the lime-reactivity of highly weathered residual soils from various areas of the world indicates that highly weathered residual soils can be very lime-reactive (highly weathered Illinois soils are not).

Assuming that a soil is lime-reactive, a number of other factors tend to influence the magnitude of strength that is developed. The lime-pozzolanic reaction is time and temperature dependent. Below temperatures of about 40°F to 50°F, the pozzolanic reaction is retarded and may be dormant but will continue when the temperature is elevated to higher temperatures. In some geographical locations, a low ground temperature (< 50°F to 60°F) may exist in the subgrade (depths > 3 to 4 ft below surface) throughout the year. In such cases, the rate of strength gain resulting from the lime-pozzolanic reaction may be quite slow. Elevated temperatures greatly accelerate the rate of reaction, although extremely high temperatures cannot be expected in the subgrade.

An increased length of curing period (at a given temperature above 40° to 50°F) produces higher strength mixtures as a result of an increased quantity of reaction product.

In general, assuming a satisfactory temperature regime exists, the pozzolanic reaction will continue as long as lime is available to the system and the pH remains high.

Translocation of Reactants

In deep layer stabilization, the region of improvement that can be effected by the lime-soil pozzolanic reaction and other mechanisms is dependent on the ability to blend or permeate (lime slurry) the soil with lime or the ability of the lime to migrate away from a free source of lime such as a lime seam or a drilled hole. Due to the constraints inherent in deep layer stabilization, lime distribution by blending or permeation processes may be limited. However, due to the nature of lime, it is possible to obtain increased distribution of the lime through migration or self-diffusion processes.

Diffusion of lime through the soil structure is an extremely slow process (see Section III for a more complete discussion of the diffusion process). Water provides a medium through which the lime can diffuse in the soil-water system. Because of interactions between the clay particles and exchangeable cations and the clay and water the diffusion process is very complex (12). The intricate pore structure of an in-place clayey soil tends to retard migration in much the same way as electro-osmotic flow is resisted. Cations (such as Ca^{++}) generally migrate faster than negatively charged ions.

Davidson, Demirel, and Handy (12) studied lime migration characteristics in large lumps of pulverized soil taken from compacted lime-soil mixtures. Quantitatively, they found that the rate of hydrated calcitic lime penetration by diffusion into the soil system could be expressed by:

$$l = k_d t^{1/2} \quad (7)$$

where

- l = lime penetration distance, in.
- t = time, days
- k_d = diffusion constant, in./day.

The value of k_d for their particular soil system (a plastic loess) was 0.081 in./day.

Differences in clay content, clay mineralogy, density, absorbed cations, and curing temperature will affect the value of the diffusion constant. In addition, carbonation of the lime within soil pores may reduce the rate of diffusion. Temperature also exhibits a marked influence on lime migration with increased temperature associated with higher rates of diffusion (12).

A drilled-hole application for slope stabilization in Iowa (13) indicated that the migration or diffusion constant, k_d in Equation 7, was 0.63 in./day. The soil treated was less dense than the laboratory specimens previously mentioned and at a higher water content. These factors, along with differing clay content and mineralogy, temperature, and amount of lime carbonation, may account for nearly an order of magnitude difference in k_d values. (Quicklime was used on the above referenced project. It was speculated that the heat of hydration and pressures occurring when quicklime was added to water may have forced lime into fissures and accelerated the diffusion and pozzolanic reaction at elevated temperatures.)

As previously mentioned, many factors influence the diffusion of lime through a soil mass. Basic properties such as soil permeability, temperature, density, etc., have a profound effect on the rate of diffusion. The Iowa study (13) indicated that if a lime-soil mixture can be mixed and compacted, diffusion of the lime into the larger soil lumps may be expected. However, to eliminate pulverization (along with mixing and compaction) and depend on diffusion alone to effect the translocation of lime might prove ineffective. For example, a penetration of 4 in. into a soil mass would take over 50 days, if $k_d = 0.36$ in./day.

Current Applications of Lime in In-Situ Stabilization

A number of agencies have reported the use of lime to treat in-situ fine-grained soils. Two techniques -- drilled-hole and high pressure injection -- have been the most common procedures for introducing the lime into the in-situ fine-grained soils. However, a laboratory study has been

reported (47) where an electrical potential was applied to a soil mass to increase the rate of lime migration.

a) Drilled-Hole Lime Applications

(i) Procedure

Drilled-hole lime stabilization is a technique that has been used rather extensively to treat in-situ fine-grained soils. The degree of success derived by using this procedure has been quite erratic, however.

The technique basically consists of introducing quick or hydrated lime into a soil mass by placing the lime in holes drilled in the soil mass. Once placed in the holes, the lime (usually hydrated lime in a slurry form) migrates or diffuses into the soil system thereby initiating the lime-soil reactions previously discussed. If sufficient migration or diffusion occurs, it is possible that the properties of a sizeable quantity of the soil mass around the drilled hole will be improved. However, lime migration or diffusion is a very slow process and substantial time may be required before a substantial quantity of soil is affected.

The drilled-hole technique has been used for remedial measures and in new construction. A number of highway agencies (102,103,104,105,106,107, 108,109) have used the drilled-lime technique to treat in-situ embankments under pavements. A minimum of disturbance to the pavement results since only small diameter holes spaced at wide intervals are needed.

In highway and slope stability applications, small diameter holes (6 to 12 in. diameter) are advanced into the soil by using suitable apparatus such as a power post-hole digger equipped with a continuous flight auger. In highway applications, provision must be made to construct a hole in the pavement structure in order to gain access to the subgrade. Typically, the depth of drilled holes ranges from 30 to 50 in. in highway embankments and up to 20 ft in slopes. The exact hole depth depends largely on the depth and nature of material to be treated. Typical hole spacings are about 5 ft center to center.

After the hole has been made, it is partially filled with either quick or hydrated lime. In some cases, water is added to the lime in order to

create a slurry. However, dry lime (especially quicklime) is thought to act as a drying agent that absorbs soil moisture, thereby reducing the moisture content of the surrounding soil. The use of a lime-water slurry may, however, tend to increase the mobility of the lime since water acts as a medium for migration.

Backfilling the hole and patching the pavement is normally required. Both soil (from the hole) and aggregate have been used. The backfill should be tamped into the hole. The holes in the pavement may be patched with portland cement concrete or asphaltic concrete.

ii) Applications

The Oklahoma Highway Department (102) has reported numerous instances of successful use of drilled-hole lime to stabilize clayey subgrades. Success, however, has been reported in qualitative terms with little physical or chemical data presented. The typical Oklahoma procedure consists of: 1) drilling 9-in. diameter holes 30 in. deep; 2) adding 25 lbs of lime per hole with additional water to create a slurry; 3) backfilling holes and patching the pavement surface. The center-to-center hole spacing has been varied, but 5-ft centers have been found to be the most practical. Typical costs range from \$1.50 to \$2.00 per hole.

The Puerto Rican Highway Department has also reported (103) successful use of drilled-hole lime on three highway sections although no chemical or physical data were presented. Only a qualitative "giving good performance" rating was reported. The drilled-hole procedure was very similar to that reported for the Oklahoma Highway Department. The cost was about \$2.30/hole.

The Alabama Highway Department has reported (105) successful use of drilled-hole lime. The procedure was similar to that used by Oklahoma except the hole depth ranged to 20 ft in one instance. In one application, the stability of a "quaky embankment" was increased by drilled-hole lime. Although Alabama Highway Department personnel cannot explain why the procedure works, they are thoroughly convinced of its benefits. Drilled-hole lime was also used as spot maintenance in Cherokee County, Alabama (105), where only dry lime was placed in the holes. Benkelman beam rebound deflection measurements (9-kip wheel load) were made on the test section prior to and one year after treatment. In most instances, reduced rebound deflections were noted. It was reported that the treated sections appeared to be performing satisfactorily.

Drilled-hole lime was used to treat 1.5 miles of hydraulically placed highway fill on Route 148 in southern Illinois (108). Approximately 50 to 100 lbs of lime were tamped into 30- to 50-in. deep holes. Six hundred tons of lime were used on the job; the total cost of the job (including resurfacing of the pavement) was \$58,000. This application has not been highly successful. After one year, little change in the pH or calcium content of the soil was noted (108). Approximately 5 years after treatment, road signs adjacent to the section warned of "rough road."

The Bureau of Reclamation has reported that their experiences with drilled-hole lime have proven unsuccessful (109). Likewise, a study (104) conducted by the Louisiana Highway Department in cooperation with the Bureau of Public Roads, indicated the following concerning drilled-hole treatment of an unstable highway embankment:

1. Properties such as pH, calcium content, and plasticity index, in general, showed little or no significant change;
2. Dynaflect deflection at 8 ft either side of the centerline showed no significant change;
3. No improvements were noted in vane shear strength data for the embankment;
4. The rate of subsidence of the pavement actually increased after treatment; and
5. After one year, no apparent lime movement or pH change had occurred when the drilled holes were examined from a test pit.

The drilled-hole lime procedure has been successfully used to arrest the movement of a slope in Des Moines, Iowa (13). Five hundred holes were placed to various depths (up to about 9 ft) on 5-ft centers. Dry quicklime was placed in the holes. An area 75 ft x 250 ft was treated with 20 tons of lime. The soil encountered was a glacial till overlain by loess. After treatment, the treated portion of the slope ceased moving. One year after treatment, examination of the drilled holes indicated a "pozzolanic reaction" had occurred to a distance of one foot from the holes; unconfined compressive strength had increased from less than 10 psi to an average of

17 psi. A pH of 8.2 to 8.4 was found in the area of the "pozzolanic" reaction. Normally a pH of greater than those reported are considered necessary for the classic lime-soil pozzolanic reaction indicating that possibly the strength increase was due to changes other than a pozzolanic reaction.

b) Pressure Injection Applications

The Louisiana Highway Department has reported results (110) of a high pressure lime slurry injection experiment used on a section of Interstate 55. A lime-water slurry was injected into the embankment on 5-ft centers to depths of 5, 10, and 20 ft. Three 44-ft x 50-ft sections were treated with 0.5-percent lime (dry weight basis) and three sections with 1.5-percent lime. The injector consisted of a tapered steel rod equipped with a mechanical sealing device at the surface to prevent feedback during pressure injection. Injection was made every 8 1/2 in. of depth. It was estimated that during pressure injection that 2 to 30 percent of the lime slurry was lost at "breakout" points. During the injection a definite bulging of the soil near the injection point was noticed.

Both disturbed and undisturbed samples were taken of the embankment sections at one month, 2 years, 4 years after injection. The samples were taken approximately one foot from the injection holes. Inspection test pits were also dug at the injection sites.

A number of observations were made from a visual examination of the soil around the injection holes (110). The lime distribution was stratified in nature. It appeared that the high pressure method operated by creating enough pressure to physically tear the silty soils apart and force the slurry in the aperture thus formed. However, in the heavy clayey soils, no tearing was evident; rather it was assumed that a bulging of the soil had occurred. After the 4-year period it was estimated that approximately 1/2 to 1 1/2 in. of lime migration had occurred. It was found that the outer part of the lime seam consisted of a hard, white material which was assumed to be calcium carbonate. It was found that there was no active lime available in the lime seam after the 4-year period. Statistical examination of

test results (Atterberg limits, pH, and available CaO) on the disturbed samples indicated that no definite lime movement in the soils had occurred. However, there was a trend toward higher pH in the soil. There was no general tendency for increased unconfined compressive strength. It was found that little if any lime was distributed into the heavier clayey soil materials. In some of the silty materials, however, some lime distribution did occur.

A high-pressure lime injection procedure has been discussed in a recent issue of Construction Methods and Equipment (April, 1969) (111). International Soil Stabilizers of Dallas, Texas, has devised a procedure whereby lime is injected under high pressure into the soil by multi-injection points. A hydrated lime slurry consisting of 2 to 3 lbs of lime per gallon of water and a surface active agent (wetting agent) at a rate of 1 part per 1,500 parts of water is injected into the soil. An injection rate of approximately 10 gals./ft of injection and pressures ranging from 200 to 600 psi with a maximum of 1,000 psi are used. The lime is normally injected at center-to-center spacings of 3 to 5 ft to a depth of up to 10 ft although a normal depth is 5 to 7 ft.

The injection rod is a 1-in. diameter steel rod with forty 1/8-in. diameter perforations per rod. The injector rod is constructed so that water can be used to advance it into the ground prior to lime slurry injection. It was reported (111) that optimum pressures used during injection are best determined in the field because of the variable soil conditions that may be encountered. Blowbacks are minimized by close control of the pressure.

The cost reported for this lime slurry injection procedure (111) ranged from approximately \$0.20 to \$0.40 per sq.ft or about \$1.80 to \$3.60 per sq.yd for a typical 5- to 7-ft depth of treatment or about \$0.90 to \$1.80 per cu.yd of treated material.

The results of a high-pressure lime slurry injection experiment on U.S. 422 in west central Pennsylvania have been reported by Lundy and Greenfield (112). The materials that were pressure injected with the lime slurry consisted of a 6- to 8-ft layer of post glacial AASHO A-7-6 clayey

soil with a cohesion = 1,200 psf and a 60-ft layer of soft, wet varved clayey silt glacial material classified as an A-4(8) with a cohesion = 200 psf.

The procedure used to inject the lime slurry to depths of up to 20 ft was as follows:

A pressure of 300 to 600 psi was used to inject a lime-water slurry (30 percent lime, 70 percent water by weight) to depths of up to 20 ft. The slurry was forced horizontally into the soil by a nozzle similar to the one shown in Figure 36. Injection was started at a depth of 18 in. and approximately 8 to 12 gals. of slurry were injected every 8 1/2 in. of depth as the nozzle was moved downward.

It is noted in Figure 36 that the nozzle is tapered so that a self-sealing hole is formed as the nozzle is advanced into the soil. It was reported the lime slurry "breakouts" occurred as far as 10 ft away from the injection hole. Lime slurry losses of up to 25 percent were reported during the pressure injection operation (112).

A sampling program was conducted after periods of 1 month, 3 months, and 1 year following the lime injection. Undisturbed samples were taken and trenches were dug to permit visual observation of the lime slurry seams. After 1 month it was observed that approximately 1/4- to 1/2-in. migration of the lime had occurred above and below the lime seams. A pH indicator (phenolphthalein solution) was used to facilitate determination of the extent of lime migration. After one year, approximately 3/4 to 1 1/2 in. of lime migration had occurred above and below the horizontal seams.

Results of the sampling program indicated that the lime slurry had easily penetrated the post glacial soil in consistent horizontal and vertical patterns up to 5 ft from the injection holes.

Results of the laboratory testing program indicated that a very nominal plasticity reduction had occurred in the post glacial soil deposits. Where complete lime migration had occurred, a significant shear strength increase had occurred, Figure 37. As shown in Figure 38, at approximately

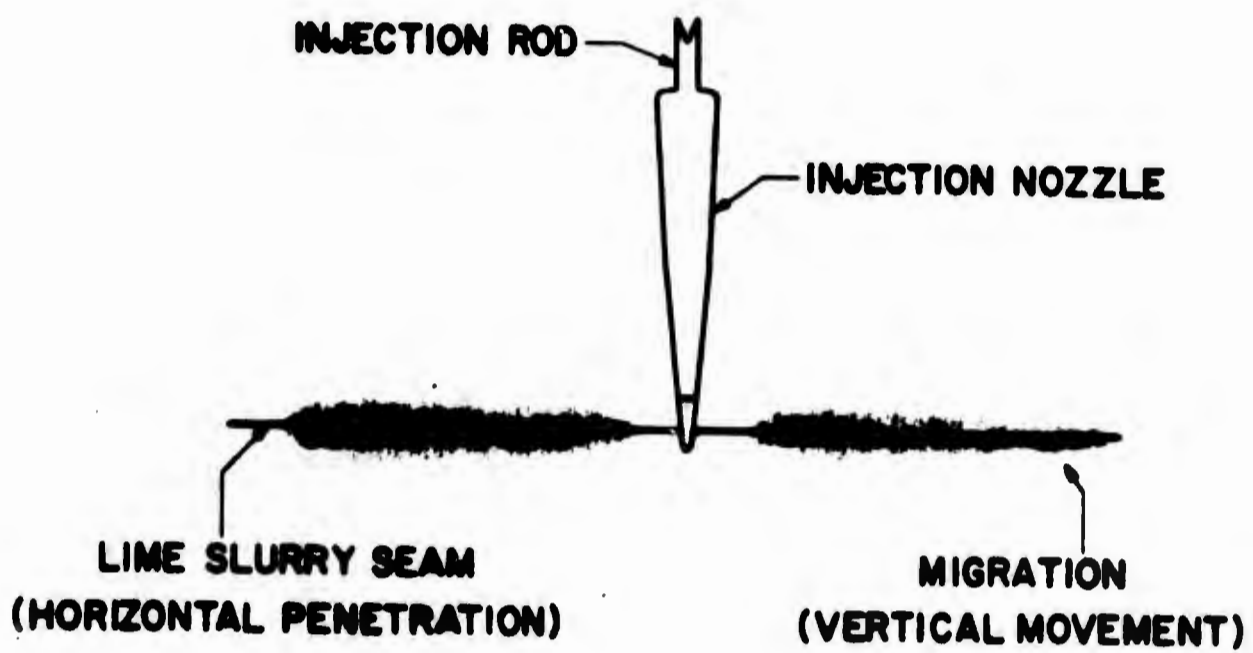


Figure 36 Schematic Diagram of a High Pressure Lime Injection Nozzle
(From Ref. 112)

Post Glacial Deposit
General AASHO Class A-7-6

--- Untreated Soil
— Lime-Treated Soil

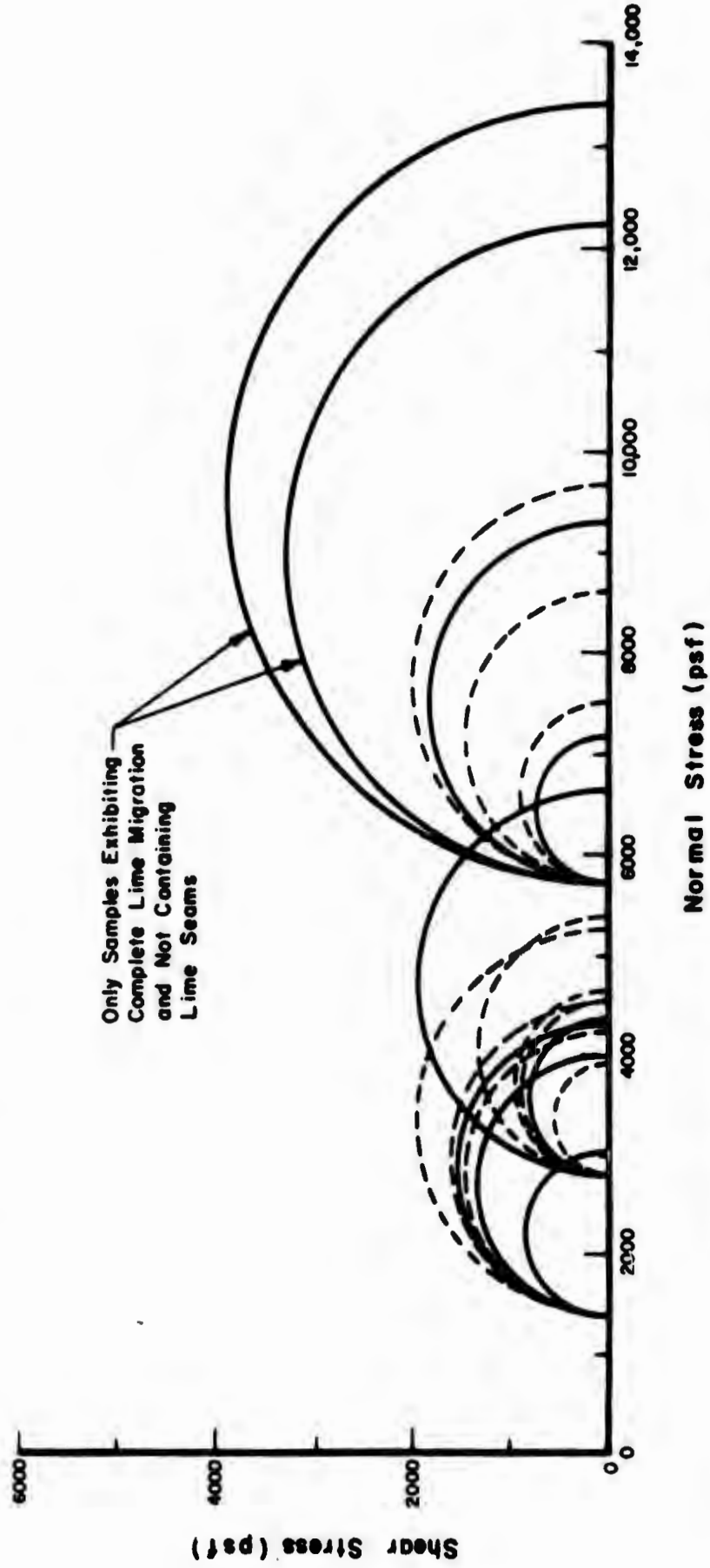


Figure 37 Plot of Shear Stress vs Normal Stress for Post Glacial Soil Samples Exhibiting Complete Lime Migration (From Ref. 112)

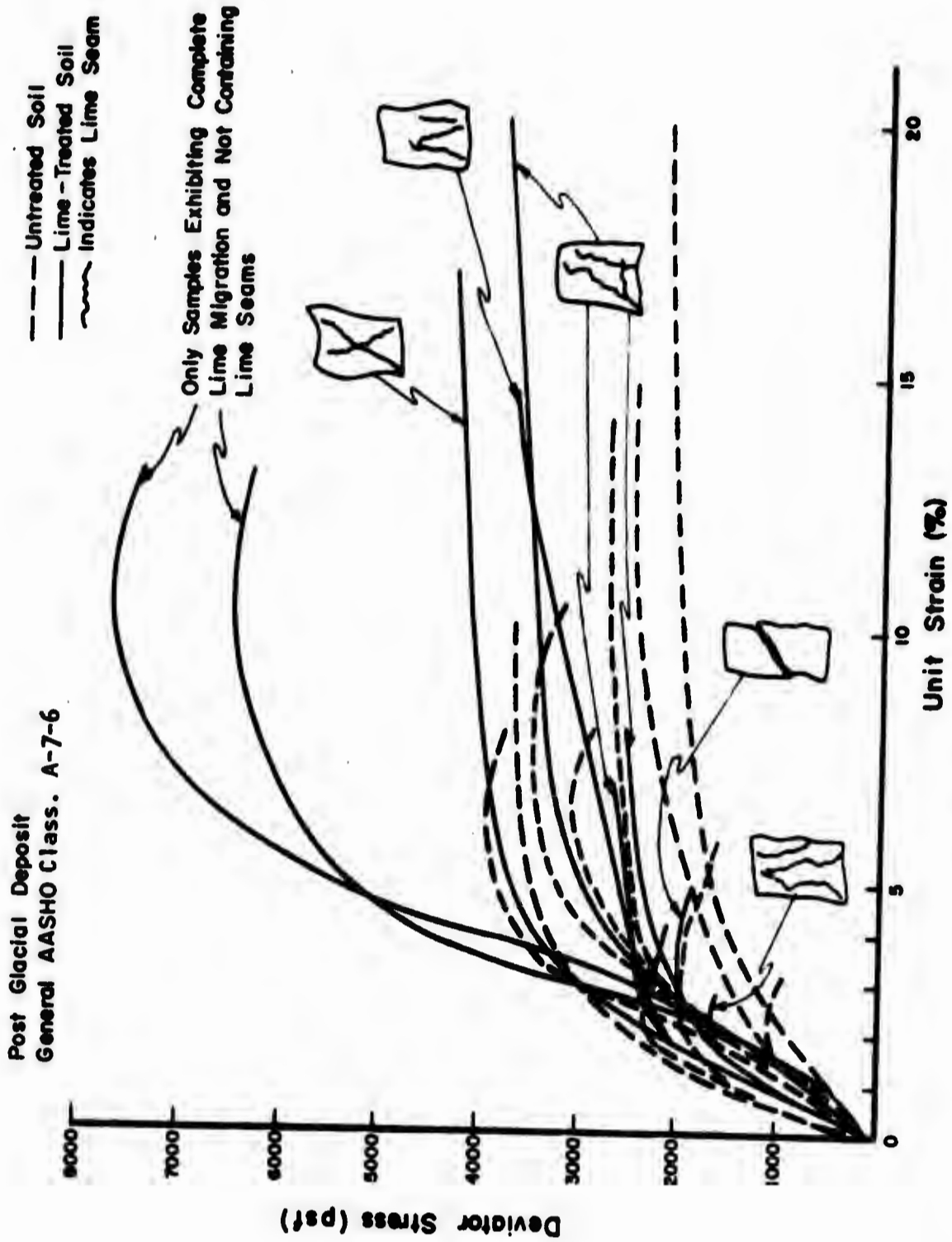


Figure 38 Stress-Strain Plot for Untreated and Treated Post Glacial Soil Samples (From Ref. 112)

10 percent strain the deviator stress increased from 2,000 to 4,000 psf for the untreated specimens to 8,000 psf for the specimens which experienced complete lime migration. However, where lime seams occurred in the test specimens no significant shear strength increases were observed, Figure 37.

In the glacial soil deposit, the plasticity remained essentially unchanged. Shear strength increases of approximately twofold were noted, Figure 39. In addition, a much greater stiffness was observed from the lime treated specimens, Figure 40.

Significant observations concerning this lime injection job (112) indicate that a definite reaction and strength increase occurred in those areas where lime had migrated into the post glacial deposit. It appeared from visual observations that better lime migration had occurred in the glacial deposit than in the post glacial deposit possibly because of the coarser texture of the former.

c) Use of an Electrical Gradient to Facilitate Movement of Lime

The Louisiana Highway Department (47) has conducted a limited laboratory study relative to permeating sandy clay, medium silty clay, and heavy clay soils with a 10 percent (by volume) lime-water slurry by imposing an electrical gradient upon the soil-water-lime system. The lime slurry was placed in a trench between steel electrodes and an electrical gradient ranging from 0.75 to 2.0 volts/cm was applied for durations ranging from 75 to 1,584 hrs. An evaluation of the study indicated that no significant changes in engineering characteristics occurred. It would appear that little or no lime migration was effected.

New Technique for Deep Layer Stabilization with Lime

The slow diffusion process is instrumental in drilled-hole or injection techniques as the method of lime translocation. Obviously, if the lime could be mixed with the soil and mechanically distributed further into the mass from the lime source, then reactions could occur much sooner and a larger volume of soil could be improved.

Glacial Deposit
General AASHO Class A-4 (8)

--- Untreated Soil
— Lime-Treated Soil

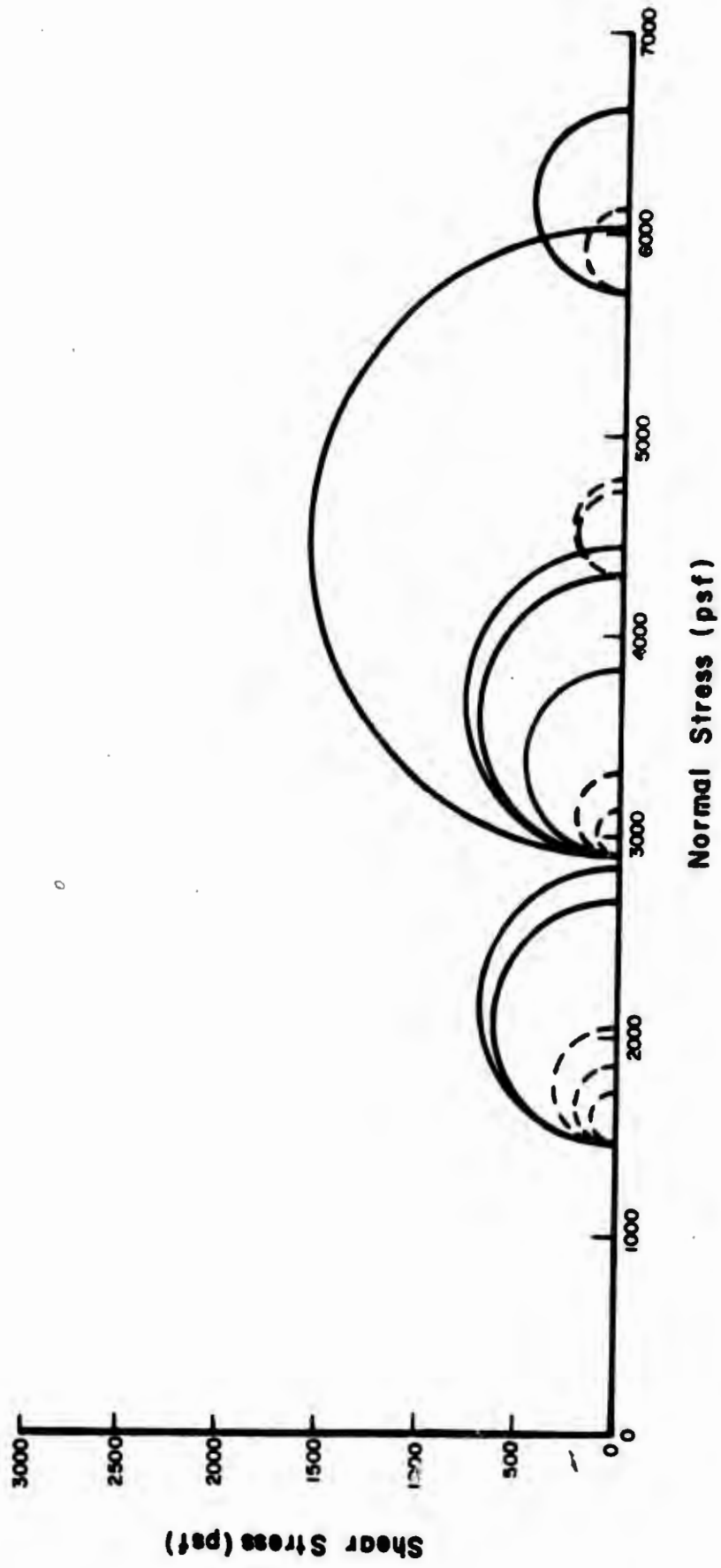


Figure 39 Plot of Shear Stress vs Normal Stress for Untreated and Treated Glacial Soil Samples
(From Ref. 112)

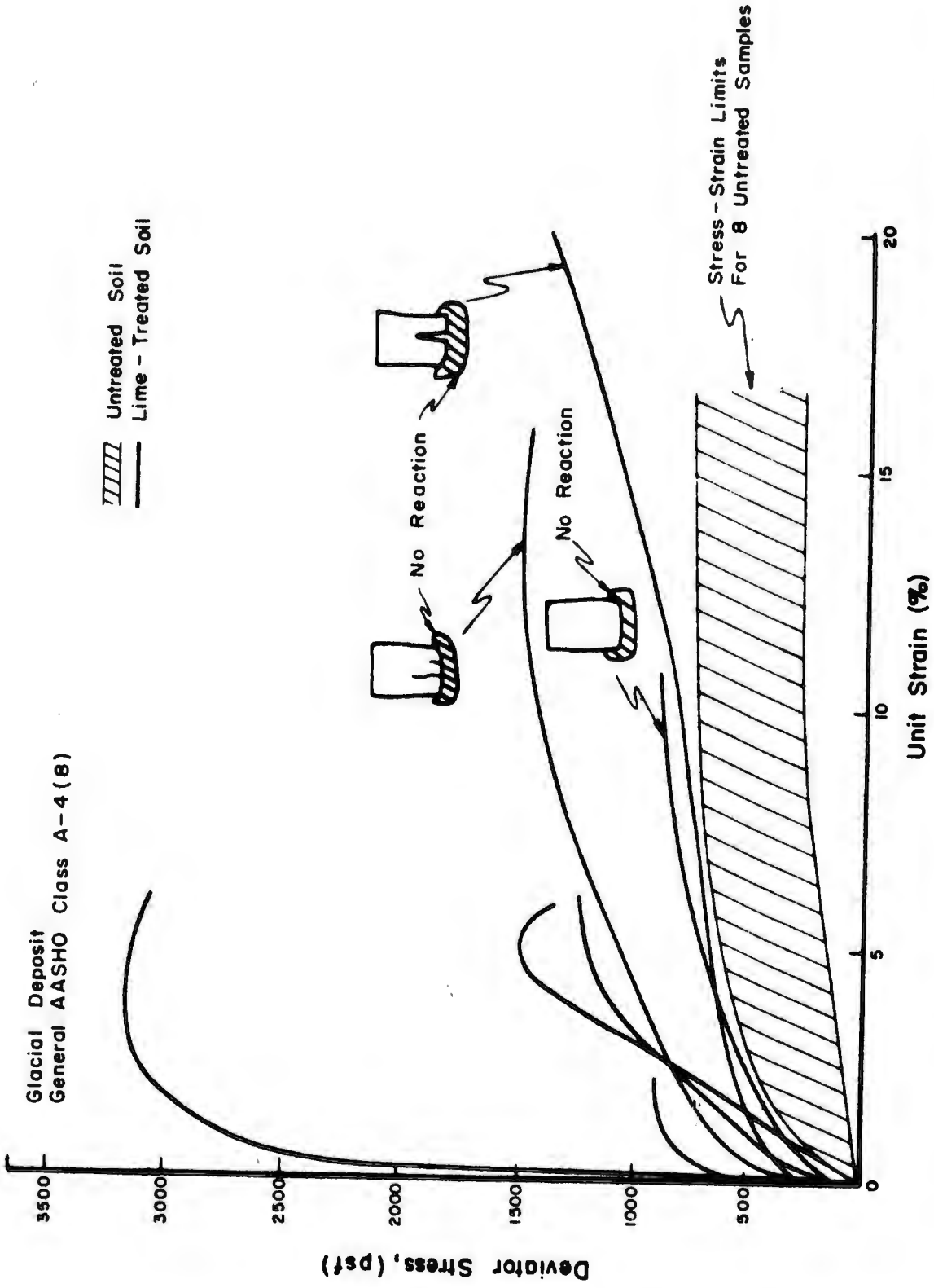


Figure 40 Stress-Strain Plot for Untreated and Treated Glacial Soil Samples (From Ref. 112)

Field application of such a technique, herein referred to as mixed-in-place, has not been found to be documented. Thus, no quantitative data or performance records are available.

The end result of this technique would be a mixed-in-place lime-soil mass with minimal surface disturbance. Larger zones can be treated by this technique than with drilled-hole or pressure injection. The mixing would increase the availability of lime to the soil and possibly allow for some flocculation/agglomeration. The soil would not, of course, be finely divided, and diffusion of lime into the larger lumps would be necessary to stabilize them. Concurrently lime would be diffusing outward into the undisturbed soil mass. Further discussion of the mixed-in-place technique can be found later in this section.

Effect of Lime Treatment

For a reactive soil system, lime could be used to effect localized or zonal improvements. Significant increases in shear strength are possible, although they would be limited largely to the immediate zone of treatment. Diffusion of the lime through an in-place embankment is a very slow process. The added benefit of mixing the soil and lime together is not currently a reality for stabilizing in-situ subgrades.

The mixed-in-place technique appears promising. Some mixing is possible and larger zones can be treated while still restricting the size of opening made in the pavement. If the lime-soil mixture could be adequately compacted to avoid pavement subsidence, then a fan-shaped cluster of zones could be treated at each point. The success of the scheme would still depend on the diffusion of lime, although not to the same degree.

Cost of Lime Treatment

The cost of treating fine-grained in-situ soils with lime typically ranges from about \$1.50 to \$2.00 per hole for drilled-hole lime and \$0.90 to \$1.80/cu.yd of treated material for lime slurry injection.

NEW DEEP LAYER STABILIZATION SCHEMES

Introduction

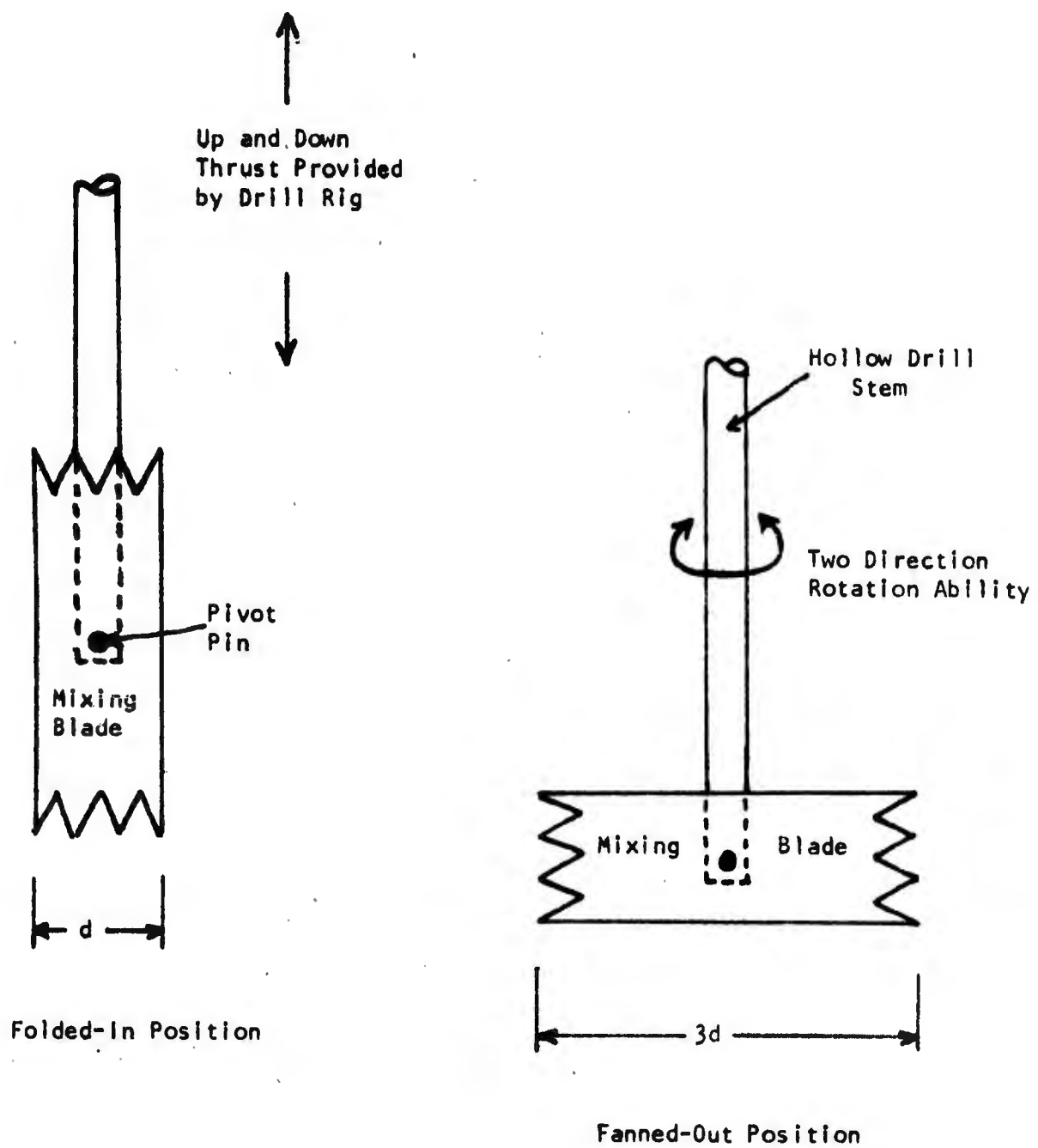
Stabilization of in-situ subgrades below existing pavements (deep layer stabilization) is a relatively new concept. In order to effect adequate stabilization under the restraints inherent in deep layer stabilization, it may be that conventional stabilization procedures will not be satisfactory. The development of new procedures and techniques may be necessary. In this section, a number of new deep layer stabilization schemes which may have some potential are briefly discussed.

Mixed-in-Place Procedures

In deep layer stabilization applications, conventional one-pass stabilizers, pulvi-mixers, etc., cannot be used to accomplish intimate mixing of the stabilizer and soil. In order to accomplish mixing in deep layer stabilization work, a method that provides subsurface manipulation with a minimum of disturbance to the pavement structure is desirable. Various schemes for accomplishing subsurface stabilizer-soil manipulation have been tried.

Mixing of cement grout and soil to form pile-like grout columns is a relatively common procedure. Intrusion-Prepakt, Inc., has a patented procedure in which the cement grout is mixed with the soil by employing a section of continuous flight auger or a mixing blade with grout outlet ports in the lower end and a hollow stem, Figure 31. As the auger or mixing blade is advanced into the soil, a controlled quantity of grout is pumped through the hollow shaft and distributed through the outlet ports. A number of field applications of the Prepakt procedure have been reported (88). A typical stress-strain curve and a plot showing the influence of amount of injected grout on the compressive strength of a fine-grained wet soil mixed with Prepakt cement grout are illustrated in Figure 32.

Another scheme that might be used is a modified version of an under-reaming apparatus. A conceptual schematic diagram of this apparatus is shown in Figure 41. In the "folded-in" position, it is possible for this apparatus to be placed through a hole as small as 6 in. in diameter. In



Note: Mixing blade may be positioned and held in the fanned out position by either mechanical or hydraulic means. Grout ports in the mixing blade allow injection of fluid stabilizer.

Figure 41 Conceptual Schematic Diagram of Apparatus for Accomplishing Mixed-in-Place Stabilization Below Pavements

the "fanned-out" position, mixing and blending of a column of material up to three times the folded-in diameter might be accomplished. This apparatus might be equipped with a hollow stem and blade pipes so that stabilizer can be pumped into the soil zone being manipulated.

Conceptually, various degrees of mixing and manipulation might be accomplished with these types of apparatus. The rate of advancement of the mixing head could probably be used to control the degree of mixing and manipulation accomplished. Figure 33 illustrates the influence of the rate of advancement of a Prepakt mixing blade on the compressive strength of a Prepakt grout-fine-grained, wet soil mixture. In some cases, it might be desirable to effect an intimate blending of the stabilizer and the soil while with active stabilizers such as lime it might be satisfactory to effect something less than intimacy and depend upon self-diffusion or migration for further stabilizer movement.

The angle of advancement of these types of apparatus could be controlled so that not only vertical columns or zones could be placed but also columns or zones at various angles from the vertical.

This mixed-in-place scheme could be used to distribute lime, cement, or chemical stabilizers.

PneumaGopher (a)

A procedure which might be used to facilitate introduction of stabilizers (lime, cement, chemicals) into the subgrade under a pavement without disturbance to the pavement surface is the PneumaGopher. With this apparatus, it is possible to enter the subgrade from the side of the pavement and create horizontal cylindrical tunnels of 4 to 6 in. diameter (113). Stabilizers could then be forced into the cylindrical tunnels to effect zone improvements.

The PneumaGopher (113) is a missile-shaped, subsurface piercing tool, designed with a bit in the nose and powered by compressed air. It is

(a) A patented apparatus (Pat. Pend. No. 3,137,483); manufactured by Schramm, Inc., West Chester, Pennsylvania, 19380.

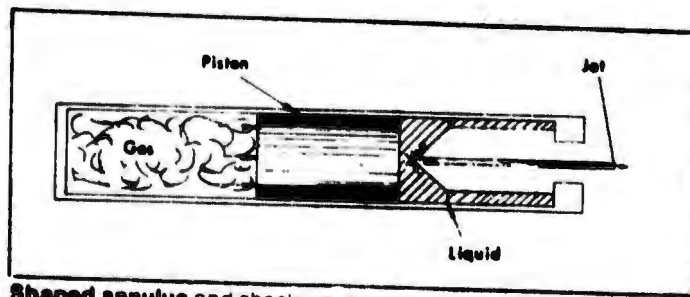
capable of piercing holes from 3 3/4 in. up to 5 7/8 in. in diameter for distances of 100 ft or more. An air compressor capable of providing working pressure of 90 psi and a volume capacity of 60 cfm is required to operate the PneumaGopher. Rapid hammer blows (600 per minute) from an internal piston against the base of the bit, force the PneumaGopher forward through the earth leaving in its wake a clean, compacted hole. The hole is created as a result of compacting the surrounding soil. Soil composition will vary the speed of the tool from one inch per minute in hard cemented soils to four feet per minute in sandy soils. A level is often used when starting a PneumaGopher in order that correct aim is obtained. Deflection off the initial line may be caused by rocks, roots, etc. Soft, wet, or rocky soils will limit the use of the PneumaGopher.

By using the PneumaGopher, it is not necessary to disturb the pavement surface. For a typical job, only two access pits are required, an entry pit long enough (about 5 ft long) to start the tool; and a second pit large enough to retrieve the PneumaGopher (113).

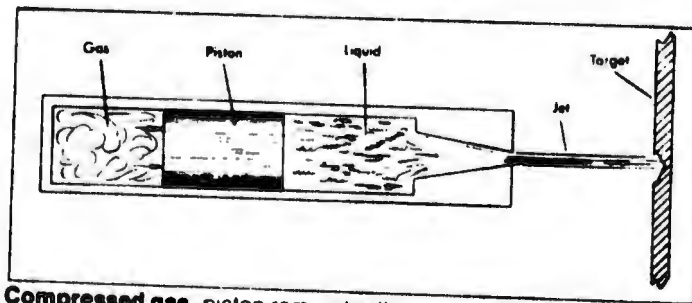
Water Cannon

A new technique for tunneling in rock has been developed (114). The technique basically consists of shooting a stream of high velocity water against the rock to fracture it. The water is compressed in a nozzle chamber by a gas driven piston until extremely high pressures are attained (up to 300,000 psi) and then the water is released through a jet in the end of the nozzle, Figure 42. In the pulse sequence, energy is stored in the nozzle for one second and then the water is released in 10 milliseconds. By using an annulus cumulation technique, pressures up to 5 million psi have been produced (114).

By using a technique such as this, it might be possible to impregnate the subgrade of a pavement system to a considerable distance by high pressure jettings of a liquid stabilizer (cement or chemical grout, lime, etc.) either vertically through a very small hole in the pavement surface or radially from a vertical auger or injection hole.



Shaped annulus and shock wave create high-intensity jet.



Compressed gas, piston ram water through nozzle in jet.

Figure 42 Schematic Diagram of Water Cannon (From Ref. 114)

High-Pressure Impulse Injection

The use of high-pressure stabilizer injection below pavements is normally limited by overburden. Grouting pressures as high as 600 psi have been used to inject lime-water slurries into fine-grained soil embankments. However, with injection pressures of these high magnitudes, blowouts of the embankment are frequently experienced. When the blowouts occur, this provides a path of low resistance through which stabilizer (grout, lime slurry, etc.) can easily flow and as a result, additional pressure grouting will not improve distribution of the stabilizer within the soil mass.

In order to minimize the formation of the undesirable blowouts, a high pressure cyclic impulse type injection procedure might have merit. In this procedure, a short impulse of high pressure would be applied followed by a period of reduced pressure.

SECTION V

EVALUATION OF THE DEEP LAYER STABILIZATION POTENTIAL OF VARIOUS PROCEDURES

INTRODUCTION

The basic objective of deep layer stabilization is to substantially improve the stability of fine-grained subgrade soils beneath existing pavement structures. AFWL has indicated that the stabilization technique developed should be capable of increasing the subgrade strength from a typical in-situ condition of CBR 2-4 to a strength equivalent to a CBR 20 subgrade, corresponding to an equivalent unconfined compression strength increase of approximately 75 psi (1). Thus, the basic criterion used in evaluating various procedures should be stability. If a particular stabilization procedure is not capable of providing a substantial stability increase to the fine-grained soil, then it is obvious that the procedure is unsatisfactory. However, if satisfactory stability improvement can be effected by the stabilization procedure, the potential of the particular procedure may be controlled by other considerations such as cost, construction considerations, permanency of treatment, etc.

EVALUATION OF PROCEDURES

In order to facilitate evaluation of the potential of various deep layer stabilization procedures, Table 9 has been prepared. This table contains a summary of pertinent characteristics of various stabilization procedures and is based on the discussion contained in Section IV. It should be noted that much of the information contained in Table 9 is qualitative in nature although quantitative data have been included where possible.

From an examination of available data (laboratory and field) concerning stability improvements, it is concluded that the electrical methods (electro-osmosis and electro-chemical) are not capable of providing the magnitude of stability increase required for adequate deep layer stabilization. The various stability data indicate, however, that cement and

Table 9
Summary of Pertinent Characteristics of Various Stabilization Procedures

Procedure	Stability		Permanency Of Treatment	Amenable Soil Types	Required Construction and Treatment Time	Construction Considerations				Costs
	Region Of Improvement	Degree Of Regional Improvement				Ability To Obtain Equivalent Embankment CBR-20	Degree Of Pavement Disturbance	Disruption To Runway U/S	Special Equipment Requirements	
ELECTRICAL METHODS Electro-Osmosis Electro-Chemical	STABILITY GRADIENT (Decreasing with distance from anode to cathode region.) STABILITY GRADIENT (Variable with chemical type, treatment time, etc.)	VERY LOW TO LOW (Possible stability reduction in cathode region.) LOW (Depends on type of chemical grout and total amount of grout used.)	No	Fine-grained soils (silt and clays)	Very Long Long	MODERATE (Depends on electrode spacing.)	Substantial	Electro-Osmosis 1. Electrical power source. 2. Dewatering electrodes. Electro-Chemical 1. Electrical power source. 2. Holes, perforated electrodes. 3. Amalgamated electrodes.	SAFETY (Shielding of cables and electrodes; protective wear.)	VERY HIGH (Large power consumption per cubic yard.)
CHEMICAL GROUTING	ZONE IMPROVEMENT (Depends on permeability of soils.)	MODERATE TO VERY HIGH (Depends on type of chemical grout and total amount of grout used.)	Feasible	Minimum allowable grain size 0.01-0.1mm (Depends on type of chemical grout.)	Moderate	MODERATELY HIGH (Requires relatively close grout hole spacing.)	Substantial	Pressure grouting equipment; Special equipment required to handle certain chemicals.	Control of gel time. Possible blow outs or feed back at high pressures.	25-100 dollars/cubic yard of treated material.
CEMENT GROUTING	ZONE IMPROVEMENT (Depends on permeability of soils.)	HIGH TO V. HIGH (Depends on w/c ratio, additives and total amount of grout injected.)	Feasible	For injection Dis of soil > 1.6-2.0 mm For mixed-in-place, possibly all fine-grained soils.	Moderate	For injection MODERATE For mixed-in-place HIGH	Substantial	For injection Pressure grouting equipment.	For injection Possible blow outs or feed back at high pressures.	For injection 8-15 dollars/cubic yard of treated material.
LIME PROCEDURES Drilled Hole Pressure injected	ZONE IMPROVEMENT (Zone size increases with time.)	MODERATE TO HIGH (If soil is lime reactive.)	Feasible	Any fine-grained soil with greater than 10% clay.	Moderate	Drilled-hole HIGH Pressure injection MODERATE	Substantial	For injection High pressure grouting equipment.	For injection Possible blow outs or feed back at high pressure.	Drilled hole 1.50-2.00 dollars per hole. For injection 0.30-1.80 dollars/cubic yard of treated material.

chemical grouting and lime procedures (drilled-hole and pressure injection) may be capable of providing substantially increased stability. For these promising procedures, the exact nature of stability improvement is controlled by a number of factors.

Examination of various construction considerations, Table 9, of those stabilization procedures which appear to satisfy the stability improvement criterion reveals that cement and chemical grouting and lime procedures require moderate treatment time, cause a moderately high to high degree of pavement disturbance, and create a substantial amount of disruption to the use of the pavement.

Typical pressure type grouting equipment is required for the cement and chemical grouting and high pressure lime injection procedures. A capability for higher pressure is required for the latter procedure.

For the chemical grouting procedure special equipment may be needed to handle the chemicals involved. Intrusion cement grouting will require special equipment similar to that pictured in Figure 31, in addition to the grouting equipment. Drilled-hole lime can be accomplished with probably the minimum amount of equipment.

An examination of cost data reveals that chemical grouting may be quite expensive; the cost involved may range from \$25 to \$100 per cubic yard of treated material. The literature indicates that the costs involved with the cement grouting are substantially less, typically ranging from \$8 to \$15 per cubic yard of treated material. Drilled-hole lime literature indicates that the typical cost ranges from \$1.50 to \$2.00 per hole which, on a cubic-yard-of-treated-material basis, would be significantly less costly than any of the other procedures. Pressure injection lime literature indicates that the typical cost ranges from \$0.90 to \$1.50 per cubic yard of treated material which is also substantially less costly than other procedures.

As indicated by Table 9, the region of stability improvement provided by cement and chemical grouting and lime procedures is a zonal improvement area. Zone-type improvement is a result of the inability of the stabilizers involved to permeate or diffuse to any great extent in fine-grained soils

within a reasonable length of time. Techniques for increasing the region of zone improvement of the conventional procedures discussed above (cement and chemical grouting and lime procedures) would obviously increase their potential. A number of new concepts and/or techniques which might have potential when used in conjunction with conventional procedures have been discussed in Section IV. The mixed-in-place concept might be used with cement, chemical, or lime stabilizers to increase the size of the treatment zone over that accomplished by drilled-hole or pressure injection. Similarly, the "water cannon" or high pressure pulse injection techniques could be used to increase the size of the treatment zone. The "PneumaGopher scheme" might be used as a procedure for introducing stabilizers into zones in the subgrade soil without causing disturbance to the pavement surface.

SECTION VI

CONCLUSIONS

Final evaluation of the potential of a particular promising procedure should be deferred until further studies (theoretical and/or laboratory experiments) are conducted concerning:

1. Determination of the minimum quantity of stabilizer(s) required to effect the desired strength improvements.
2. Evaluation of stabilizer "distribution effects" and their influence on strength development. It is highly improbable that "uniform stabilizer distribution" can be obtained under field conditions.
3. Determination of the extent to which soil structure and accompanying strength changes can be effected within an in-situ soil mass without the benefit of mixing and manipulation of the stabilizer and the soil.
4. Identification of those factors that control stabilizer movement in the in-situ mass. Typical factors will include soil type, physical properties, chemical properties, mineralogical properties, nature of stabilizer, in-situ properties of the soil mass, injection pressures, etc.
5. Development of qualitative, and if possible quantitative, characterizations of the phenomena associated with stabilizer movement in the soil mass.
6. Evaluation of the influence of the following factors, if appropriate, on the effectiveness of the selected technique(s):
 - a. time,
 - b. temperature,
 - c. moisture (degree of saturation), and
 - d. freeze-thaw.

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13. ABSTRACT (Distribution Limitation Statement No. 3) The majority of the airfield pavements supporting current Air Force operations were designed and built during World War II and are presently approaching the end of their design life. The nature of the aircraft for which these early pavements were designed is substantially different than the planes currently in service and the much heavier aircraft, such as the C-5A, anticipated for use in the future. In order to accommodate the heavier repeated wheel loads of the future, many of the airfield pavements currently in service will have to be strengthened. Possible schemes for improving the load-carrying capacity and performance of existing pavements fall into two broad categories: (1) conventional overlaying operations and (2) strengthening of the soil beneath the existing pavement structure with a minimum of disturbance to the existing pavement structure. In this report, the ability of various stabilization procedures to provide strength improvement to in-place subgrades was examined. Included in the stabilization procedures examined were electrical methods (electro-osmosis and electro-chemical), cement and chemical grouting procedures, lime stabilization procedures (drilled-hole and pressure injection) and a number of new techniques. A tentative evaluation, subject to review and revision upon completion of the scheduled laboratory studies, indicates that certain grouting procedures and lime stabilization procedures offer the most potential for accomplishing strengthening of in-place subgrade materials.			

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