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Frank P. Mancini and Charles E. O'Bannon

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16. Abstract

A large scale soil stabilization project was initiated in July 1973 by the Arizona Department of Transportation. This project was sponsored by the Federal Highway Administration.

A 500-foot section, which is part of a 1300 foot cut on I-40 35 miles east of Holbrook, Arizona, was selected as the test site. The natural soil in the area is Chinle clay, a highly expansive Montmorillonite clay.

The stabilization technique used on the Chinle clay was treatment with a 0.4N solution of KCL and electrochemical action.

Results show that this method of soil stabilization produced a moderate decrease in the swelling characteristics of the Chinle clay, i.e., about a 50% decrease in expansive pressure and a 36% decrease in percent swell.

Moreover, the electron miciographic and x-ray diffraction data have indicated some probable causative factors for the decrease in the swelling characteristics of the Chinle clay.

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PREFACE

The contents of this report represent over two year's work conducted by the Arizona Department of Transportation, Research Section. This work entails the development and analysis of a field procedure for the stabilization of Chinle Clay using electro-chemical methods.

ACKNOWLEDGEMENTS

The prinicipal researchers on this study feel a deep sense of appreciation and gratitude to many individuals and Organization for the excellent, and continuing spirit of close cooperation that existed during the 2 year period of this study.

This project involved elements of difficult field construction work, sophisticated laboratory analysis and efforts of many highly qualified engineers and technicians without whose help little could have been accomplished.

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    Chemist

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    Drilling operations

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   District Engineer

   Mr. Rex Wolfe P.E.
   Assistant District Engineer

   Mr. Cliff Foster
   Laboratory Supervisor

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Charles E. O'Bannon
Frank P. Mancini

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CHAPTER 1

INTRODUCTION

Sponsorship

This report is based on work performed by the Arizona Highway Department, under contract with the Department of Transportation to investigate "Elctro-Chemical Hardening of Expansive Clays". Funds for the project were provided by the Federal Highway Administration through the Arizona Department of Transportation.

Nature of and Approach to Problem

Frequently the soils available for construction cannot meet the requirements, such as strength and incompressibility, imposed by their use in embankments or subgrades. The process of improving the soil so that it can meet the requirements is known as stablilization.

Many different methods of stabilization have been proposed. From the standpoint of their function or effect on the soil they can be classified as follows:

1. Moisture-holding: Retain moisture in soil.
2. Moisture-resisting: Prevent moisture from entering soil or from affecting clay materials.
3. Cementing: Binding the particles together without their alteration.

4. Void-filling: Plugging the Voids.

5. Mechanical Stabilization: Improving the soil gradation.

6. Diagenisis or Physiochemical Alteration: Changing the clay mineral or clay mineral absorbed-water system.

A satisfactory stabilizing technique must provide the required soil qualities and in addition must satisfy the following criteria: (1) Compatible with the soil material; (2) Permanent; (3) Easily handled and Implemented; (4) Low cost.

The problem of stabilizing Chinle Clay has been under consideration by A.D.O.T. since the early 1960's because of the vast deposits of Swelling Clay in Northern Arizona, some 6,000 sq. miles of surface exposure, as shown in figure 1.

The swelling characteristics of these Chinle Clay deposits are large and have caused excessive heaving in highway subgrades. This volumetric strain with resulting pavement stress has caused damage or failure to many miles of existing highways in this region of Northern Arizona.

Recognizing this condition the Arizona Department of Transportation and the F.H.W.A. supported the work of Dr. Charles E. O'Bannon, of A.S.U.'s Civil Engineering Department, on diagenisis of Chinle Clay by electrochemical methods.

MAP OF ARIZONA SHOWING MAJOR CHINLE OUTCROPS

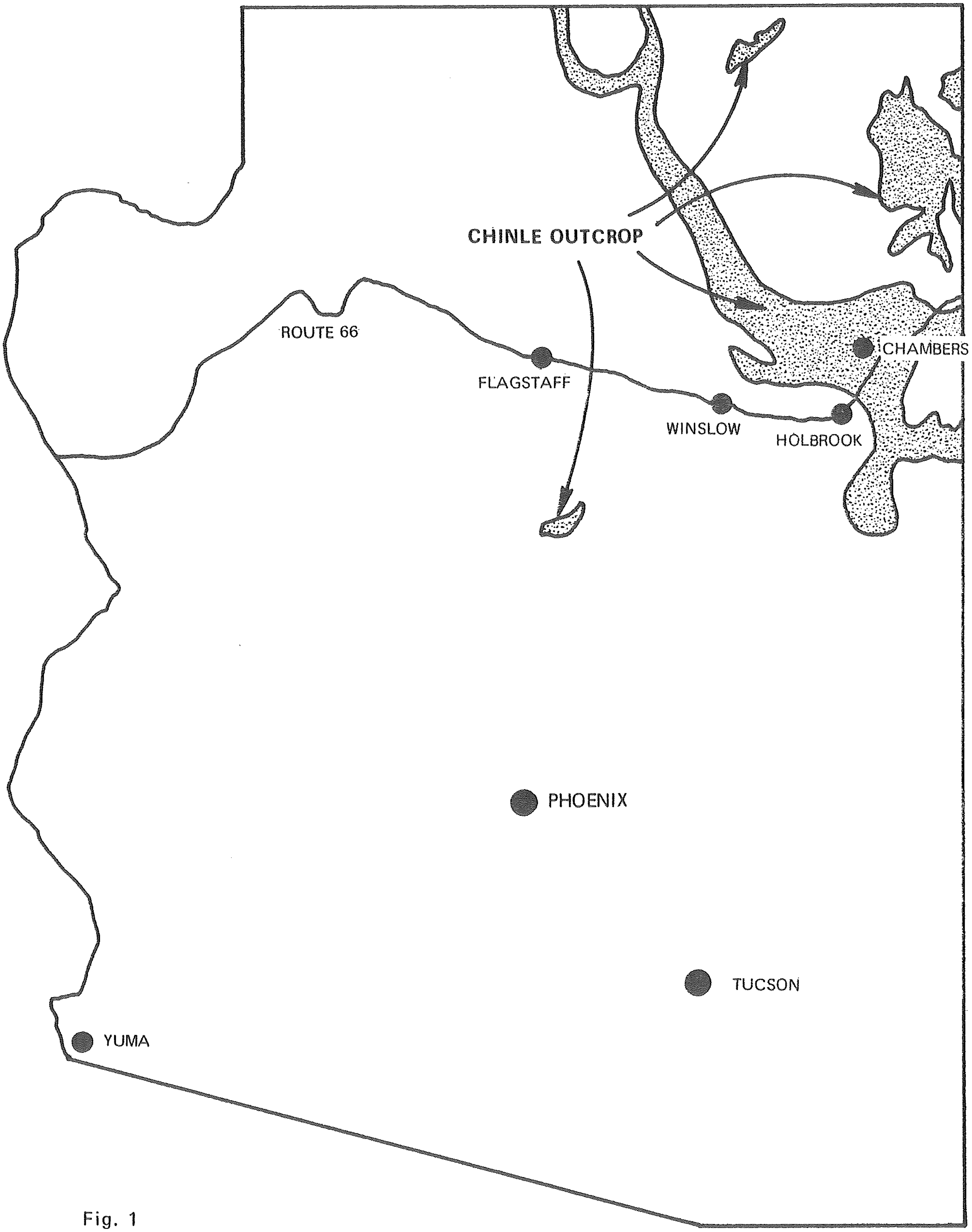


Fig. 1

Object of Research

The object of this research was to utilize Dr. Charles O'Bannon's electrochemical soil-treatment technology and, if necessary to modify portions of it for the express purpose of creating a viable soil stabilization technique that is implementable by Highway Division Maintenance personnel.

Scope of Report

This report on the Electro-Chemical hardening of Chinle Clay is presented in eight chapters. These chapters explain in detail the various areas of laboratory and field experimentation conducted since July 1, 1973. Chapter II discusses the Geology of site and the preliminary sampling layout. Chapter III is a review of some Electro-Chemical soil hardening concepts as it applies to the existing problem. Chapter IV presents the work conducted in the laboratory in preparation for the field test. Chapter V discusses the work performed in the field test in the spring and summer of 1974. Chapter VI presents the results of the field test with an analysis of the data. Chapter VII is a preliminary cost study for the method. Chapter VIII discusses the on-site moisture content measurements using Nuclear Gauges. Chapter IX is the conclusion and recommendation that have been reached during this study.

CHAPTER II

GEOLOGY AND PRELIMINARY SAMPLING OF THE AREA AND SITE

The site is located in Northeastern Arizona on the Colorado Plateau geological province, see figure 2. The nearly horizontal sandstones and shales of Triassic and Jurassic age in this area are intermittantly covered by thin-bedded Tertiary sandstones and shales. This region is characterized by low, broad mesas overlooking wide, flat, stream valleys containing Quaternary to Recent sands and silt alluvium with low terrances containing a very limited quantity of gravel.

U.S. Interstate 40 transverses primarily the Triassic Chinle formation as it crosses this area. The Chinle formation is composed of shale, clay sandstone, and minor amounts of limestone.

Characteristics of the Chinle formation vary greatly in both the physical and chemical aspects. Chinle formation colors include red, pink, brown green, purple, and grey tints. It has been previously mentioned that shale, clay, sandstone, and minor amounts of limestone are present. Physical and chemical characteristics of sedimentary deposits are dependent for the most part on three factors: 1) Of course, dependence on parent material, 2) method of transportation of sediments, and 3) changes after deposition. Several depositional environments are responsible for the characteristics of the Chinle formation. It is held that the conditions under which the

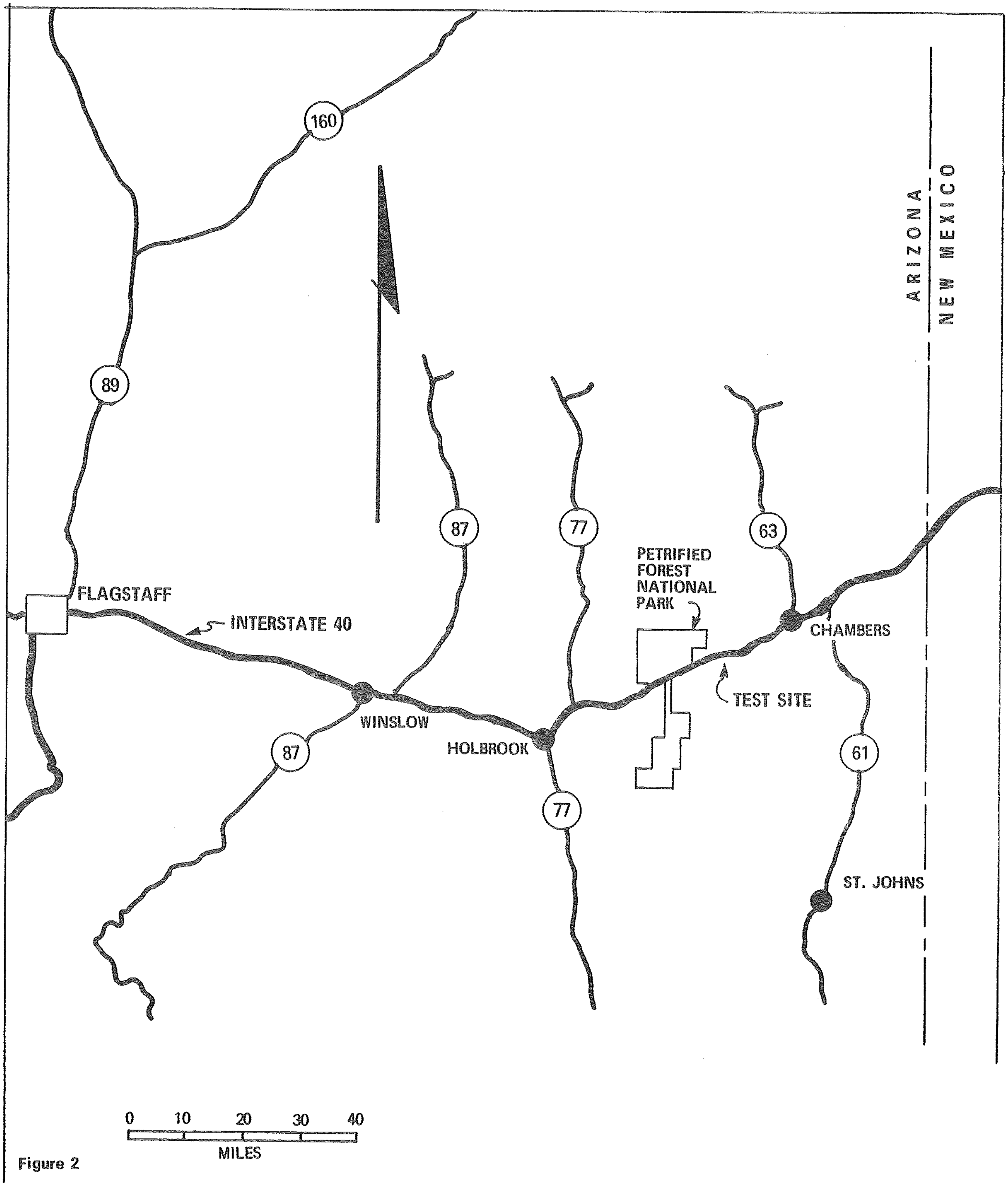

160
89
ARIZONA
NEW MEXICO
87
77
63
PETRIFIED FOREST NATIONAL PARK
FLAGSTAFF
INTERSTATE 40
CHAMBERS
TEST SITE
WINSLOW
HOLBROOK
87
61
77
ST. JOHNS
0
10
20
30
40
MILES

Figure 2

chinle was deposited were wholly continental--probably those of a well-graded but rather arid plain across which streams meandered and on which there were perhaps scattered lakes. Conglomerates of resistant materials transported from a great distance are scarce; hence, it is evident that stream gradients were low. Continental origin is evidenced by the presence of fresh-water fossils, land vertebrates and the trees of the petrified forest. Evidence is present for the existence of a warm marine environment in portions of the area in which the Chinle outcrops. Montmorillonoid and bentonitic clay minerals are suggestive of volcanic activity at the time of the warm marine climate. The alteration of volcanic ash in such an environment is favorable to the information of the aforementioned clays.

In the general area of the site, the Chinle varies in thickness and had a macimum depth of 1,500 feet at the test site on Interstate 40, the highway is in a 15-foot shale cut. The weathered material on the slopes of the cuts extends to a depth of about one foot. This material has the typical reticulated appearance of an expansive clay, weathered in place from the parent material. The underlying material is very hard and brittle. The out-crop is fractured, slickensided, and shows no visible bedding planes.

During the first weeks of July, 1973 A.D.O.T. District IV personnel set down engineering reference stations on the west bound lane of U.S. Interstate 40 at M.P. 323.8± for approximately 1200' of the cut.

After due consideration with an all-system go target date of September 4, 1973 in mind it was decided that the optimal pre-sampling procedure, both from an engineering and economic viewpoint, was to acquire all soil samples in a continuous straight line pattern. The samples would be taken only over a 500 foot section of the cut selected as the test site.

Taken in this manner the soil samples would provide adequate data on the soil characteristics of the section for the purposes of this study.

It should be pointed out that this sampling procedure was conceived under the constraints of minimal time and money expenditures which would be essential considerations for an implementable maintainence operation.

However, this type of sampling procedure is not recommended if the soil study is to be one of a general investigatory nature.

Preliminary Sampling

Samples were obtained from sta 7+00 to sta 12+00 on 20' intervals and tested for expansive characteristics and atterberg limits.

Based on the atterberg limits the soil specimens were classified via the Unified Soil Classifications Systems. The result of the classification and a soil profile are shown in figure 3.

SOIL CLASSIFICATION FOR I-40 E.O. SITE

MP 323.8±

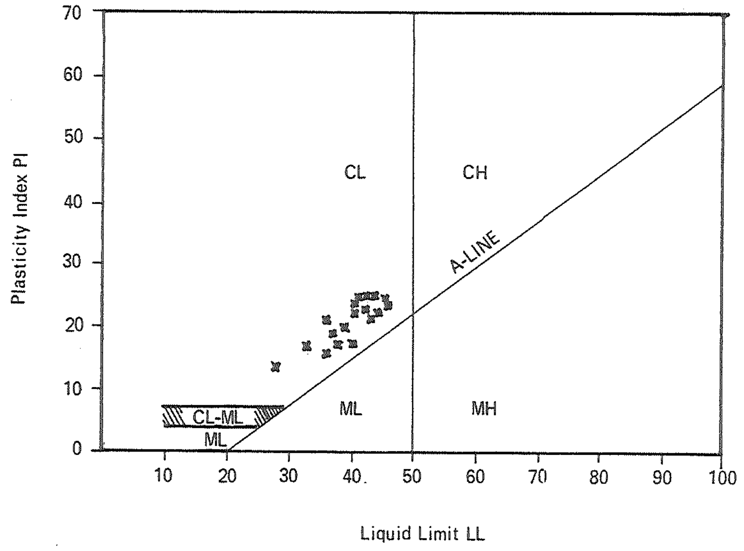
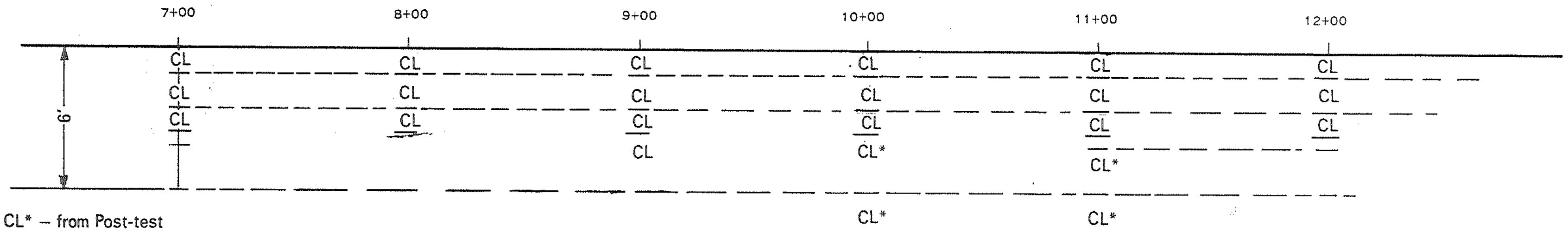


Fig 3

CRAZY CREEK
TEST – SECTION SOIL
CLASSIFICATION

SOME LENSES OF SANDY CLAYS, FINE SILTY SANDS
CLAYEY SANDS, PRIMARILY MEDIUM PLASTIC CLAY



CL* – from Post-test
Atterburg-limits

CHAPTER III

Physicochemical Aspects

Clay Structure

Because this work with electro-osmosis is primarily concerned with stabilization of Chinle Clay formations, a brief review of certain facts and models of clay minerology and electro-osmosis is fundamental.

It is well known that the soil particle size strongly affects the appearance and behavior of soils. These effects on soil behavior are due to the increasing magnitude of the forces among the molecules of adjacent particle surfaces as the size decreases. In silt or sand and larger soil sizes, the ratio of the area of the surface to the volume of the sample is relatively small, hence the intersurfaces forces small. In such cases the shape of the grain, surface roughness and resulting interparticle abrasive forces determine the overall behavior of the soil mass.

In the case of small soil particles, the molecules forming the particle surface constitute a large proportion of the total number of molecules, and the interparticle forces associated with these surface molecules have a significant effect on the behavior of the particle and hence, the mass of the soil.

It is found, in general, that allowance for the effect of surface forces must be made when the size of the particle becomes less than one micron (10^{-4} cm), a size which corresponds to the upper limit of colloids.

It was formerly believed that clay minerals were amorphous in structure, but investigations carried out with x-ray diffraction and electron microscopy since the 1920's have shown them to be predominately crystalline; moreover, it is found that clay minerals are crystalline hydrated aluminum silicates. Although the molecular structures are complicated, it has been shown that clay minerals are constructed essentially from two basic structural units: The silica tetrahedron (S_iO_2) and the octahedral aluminum hydroxide $Al(OH)_3$.

A silica tetrahedron consists of a central silicon atom surrounded by four oxygen atoms arranged at the apexes of equilateral triangles. The other structural element, hydrated aluminum, takes the form of an octahedral crystal, in which the aluminum atom occupies the center of the structure, above and below which the oxygen and hydroxyl ions are arranged.

The clay minerals designated Smectites are composed of basic units of two silica tetrahedral sheets with a central alumina octahedral sheet. These units are continuous in the a and b directions and are stacked one above the other in the c direction. These basic units are referred to as silica-alumina-silica units.

In the stacking of the basic units, one above the other, the 0-layers of each unit are adjacent to 0-layers of the neighboring units. Because of the proximitity of the 0-layers, there is a weak bonding energy which permits easy expansion of the lattice in the c-direction. It is this high swelling characteristic that is the outstanding feature of Smectites.

Chinle Clay is a random-mixed layered mineral containing minor to major amounts of the Smectite mineral; i.e., about 10% - 40% by wt. of Smectite.

Base Exchange

Since Dr. O'Bannon's treatment of swelling clays rests on base exchange phenomena, a short discussion of this phenomena is in order.

The absorbed ions on a clay surface are present in a diffuse double layer. Each of the ions required to neutralize the charge on the particle surface is oscillating due to Brownian motion, thus the ion is assumed to be oscillating in a cell, called an oscillation cell, adjacent to a charged area on the particle surface. Other ions from an added electrolyte may enter the oscillation cells or may remain in the external phase. A given ion with a large naked radius, i.e., K^+, will have a smaller hydration radius and thus be able to approach the charged surface closer than an ion with a smaller naked radius, i.e., N^+, and a corresponding larger hydration radius. Thus, the potassium ion will, on the average, bond to the surface with a correspondingly greater energy than a sodium ion.

Such concepts have been evolved into mathematical models which depict the ion exchange process as a stochastic process.

These theories, in particular the one by H. Jenny, lead to mass-law or mass-action equilibrium equations. However, these models of base-exchange imply the exchange phenomena is essentially a complex redistribution of ions both between an external phase and the ion swarm and also within the ion swarm. Unfortunately this process depends on several factors. The ion redistribution cannot be regarded as simple metathesis, precisely defined by a simple equation of the mass-law type. The mass-law equations often quoted in the literature, with regard to base exchange phenomena, must be considered as only approximations from which there may be considerable variation in "unfavorable" cases.

Because of the stochastic nature of base-exchange phenomena, any soil treatment based on this phenomena must therefore also be regarded as a stochastic process. Of course one may raise the "odds" in favor of a successful treatment along these lines by preparing the external phase to be "rich" in the preferred ions, e.g., utilize a high concentration KCl solution to treat the soil.

It must be borne in mind that an ion-exchange soil treatment can alter the physicochemical properties of the swelling clay and is one of the most effective ways to combat problem over a relatively small localized region on which an expensive inplace structure, say a pavement, rests.

The ability of a clay to absorb ions on its surfaces or edges is called its base or cation (anion) exchange capacity, which is a function of the surface chemistry of the clay and the size of the clay particles. Thus the term base exchange is widely employed, although in fact hydrogen ions and even organic ions may be involved in the exchange.

The base exchange capacity of a clay mineral is related both to the charge deficiency caused by isomorphous substitutions within the lattice of the mineral and also to the number of dangling bonds around the edges of particles. The absorbed ion may be N_a + C_a ++, K+, Li+, F_e ++ or other cations. These absorbed ions will not alter the basic clay chemical formula, but they significantly alter the engineering behavior of the clay mineral since they affect the bonding energy of the various basic structural sheets to one another. Moreover, in occupying spaces on the surface of the clay particles the ions interfere with development of water layers at the surface.

After analyzing this base-exchange phenomena, Dr. Charles O'Bannon of the A.S.U. Civil Engineering Department employed electro-chemical stabilization to the problem of swellings clays. Dr. O'Bannon felt that two primary effects could be relied upon to alter the clay properties.

(a) The release of ions from the anodes, an exchange of ions in the soil solution and the migration of solutes in the pore water or in the absorbed water layers. The activation of such a system can lead to new particle configurations (e.g., closing or opening the space between clay platelets and locking them together with different cations either face to face or edge to face.

(b) Mass-transfer phenomena, i.e., ionic transport and water transport to assist stabilizing solutions to penetrate into soils which may be otherwise nearly impermeable to the passage of solution.

In order to further illuminate the rationale behind Dr. O'Bannon's usage of electrochemical stabilization a short discussion of some fundamental ideas relating to swelling clays and effects of electrolytes on clay swelling is presented.

Classical Models of Swelling Mechanisms

The observed swelling pressure of montomorillonite is, of course, the difference between the attractive and repulsive forces operating between the layers per unit area.

At small interlayer distances two attractive forces predominate; namely, electrostatic forces between the negatively charged surfaces and the intervening exchangeable cations, and van der Waals forces. The magnitude of the former depends on the surface charge density resulting from isomorphous substitution. Thus, the forces between the charge surface and the cations depend on the composition of the mineral. The van der Waals forces are not greatly dependent of the material composition; however, they are influenced by the external environment.

The repulsive forces involved have been generally considered to be osmotic in nature. It is assumed that when montomorillonite is placed in contact with water, the water initially enters between the layers because the molar energy of the water is reduced by the hydration of the cations present between the layers and hydration of the layer surfaces.

Hence, a highly concentrated solution of cations is formed between the clay layers. Moreover, because of the negatively charged layers an electrostatic field is present which constrains the cationic thermal motion.

This thermal or Brownian motion of the cations is always present and without the constraint of the interlayer electrostatic field the cations between the layers would diffuse out to the exterior solution until an equilibrium concentration was established.

The osmotic model used to describe swelling phenomena is formulated around these facts as follows: Because the interlayer electric field acts as a semipermeable membrane and causes the interlayer solution to have a high concentration of cations, exterior water enters due to an osmotic gradient. The resulting repulsive pressures is, thusly, assumed to be an osmotic one.

This osmotic theory of swelling seems to be a plausible one and indeed some experimental evidence at least qualitatively, supports it. However, Dr. Phillip F. Low, among others, suggests that this is not the full story. It has been shown thermodynamically that the repulsive pressure has not only the osmotic contribution, but also a contribution from the molar potential energy of the water.

The molar potential energy is a factor because the interlayer water interacts with lattice structure of the clay crystal. That is the structures of the interlayer water and montmorillonite conform to one another, i.e., epitaxy occurs, because of the hydrogen bonding between them. What this means is that as the thickness of the interlayer water increases it's structure assumes a preferred configuration and the montmorillonite crystal adjusts accordingly. This adjustment continues until the water achieves its preferred configuration. An alternative interpretation is that the montmorillonite structure relaxes as stresses acting on it are relieved by the inclusion of water, and the water structure changes conformably. This relaxation continues until the water structure resists further strain.

Now as this phenomena occurs the b-dimension of the unit cell* increases due to an alternative clockwise and counterclockwise rotation of the clay's silica tetrahedra about their vertical axes. See figures 4 and 5. This rotation of the tetrahedra alters the b-dimension of the unit cell. Thus, all maximally swollen montmorillonites have the same b-dimension for their silica tetrahedra. This value is a little over 9 x 10^{-8} cm or 9Å.

It has been established that the presence of potassium ions in the interlayers of montmorillonite significantly influences the lattice structure of the clay. Work done by Dr. Phillip Low and others shows that the potassium ion increases the b-dimension, see figure 6, hence reduces the swell potential of the montmorillonite clay. Dr. O'Bannon has found that the required amount of potassium to significantly affect the swell of the Chinle Clay varies from about 0.008 gms K+/gm clay to about 0.02 gm K+/gm clay.

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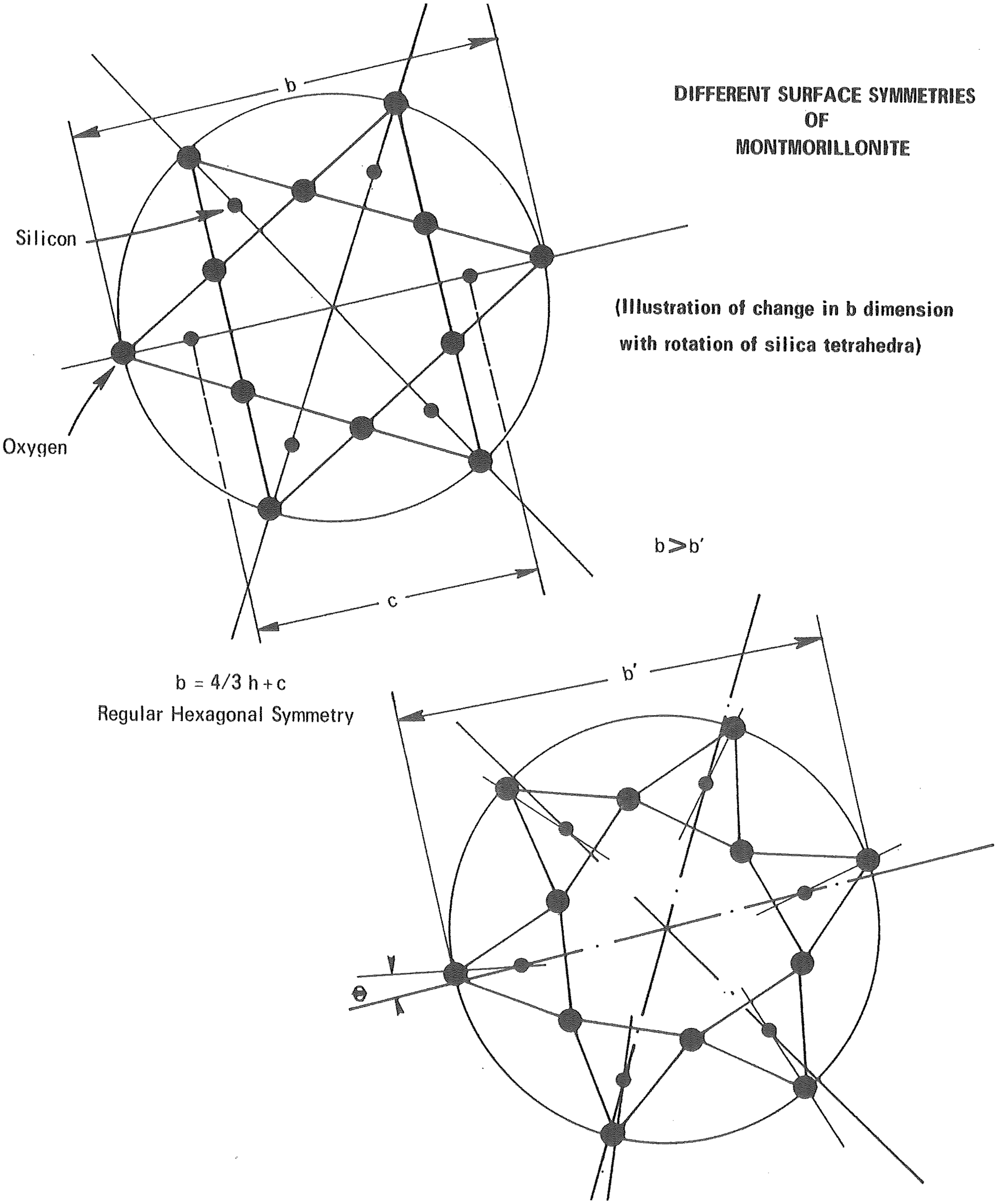
*The unit cell is the smallest structural unit which, when repeated in space, reproduces the given crystalline structure.



**DIFFERENT SURFACE SYMMETRIES
OF
MONTMORILLONITE**

**(Illustration of change in b dimension
with rotation of silica tetrahedra)**

$b > b'$

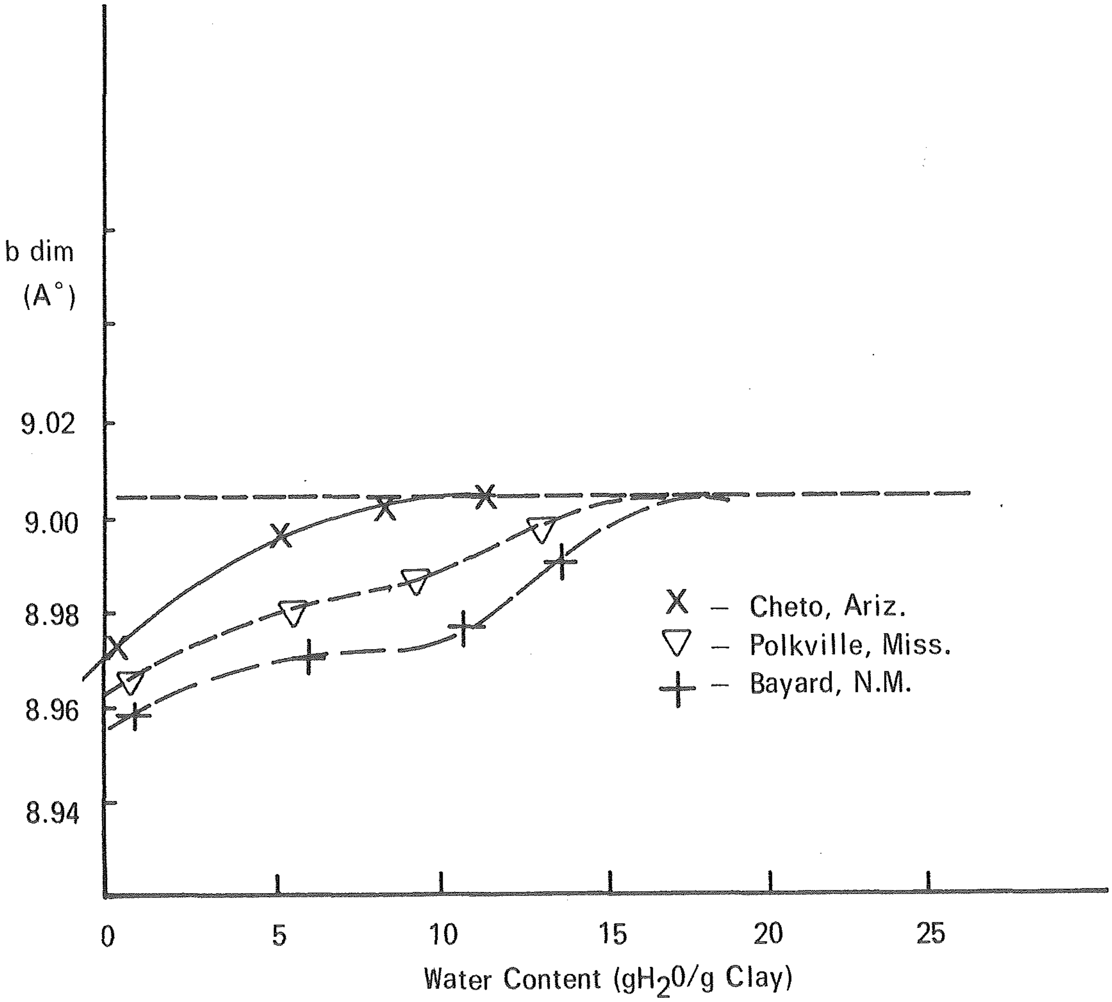
$b = 4/3\ h + c$

Regular Hexagonal Symmetry

$b' = 4/3\ h \cos \Theta + c$

Ditrigonal Symmetry

Fig. 4



Relation between b-dimension
and
Water content for $N_{\bar{a}}$ Saturated
Montmorillonites (Ravina & Low)

Figure 5

THE EFFECT OF THE EXCHANGEABLE CATION
ON THE B-DIMENSION OF NA-MONTMORILLONITE

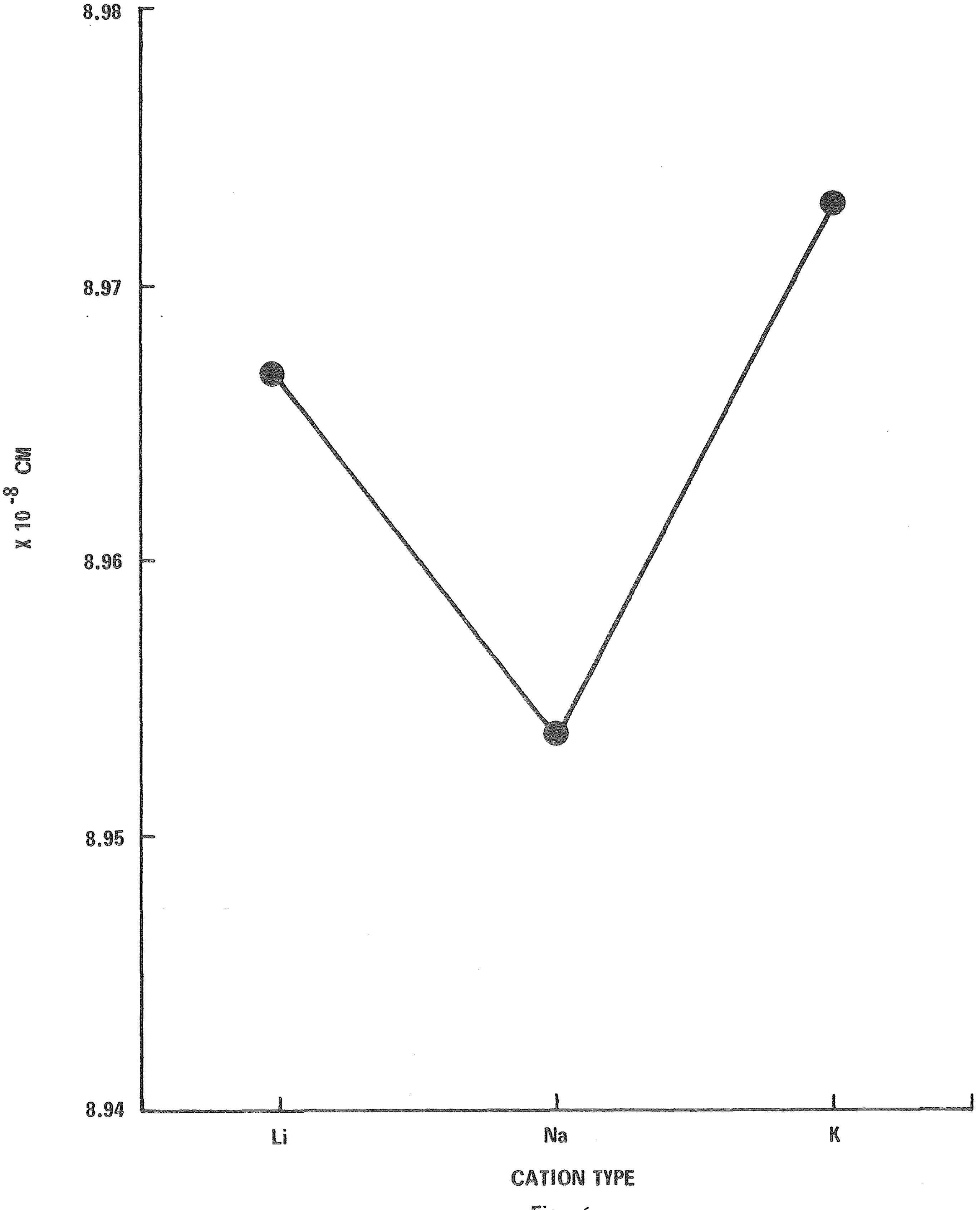


Fig. 6

Clay Stabilization by Electro-Osmosis and Base Exchange of Ions

It is generally believed that Dr. O'Bannon utilized an electric field to move a chemical solution through the clay region solely because of the low hydraulic permeability coefficient of Chinle Clay. This, however, is not strictly correct. It is well known** that electrolytes have an effect on the magnitude of the soil permeability. For example field & laboratory work for this project showed that a 0.4 N KCl solution moves through Chinle Clay, under a hydraulic gradient, at a significantly higher velocity than deionized water does (i.e., well over an order of magnitude increase in velocity).

Because of this, several attempts were made in previous studies, to introduce the KCl solution into the soil using only hydraulic gradients. Thus with a simple "ponding" technique one could rely on the base-exchange of ions to effect a change in the clay structure. While base exchange produces long-term effects on the clay structure it was found that application of an electric field induces additional electro-chemical phenomena which also greatly altered the soil structure.

With the application of the electric field to the KCl solution - saturated clay a variety of electro-chemical phenomena occur. Among the various mass-transfer effects are ion-transport and water-transport.

**Lutz, J.F., and W.D. Kemper, "Intrinsic Permeability of Clay as Affected by Clay-Water Interaction", Soil Science vol. 88, 1959, pg. 83.

O'Bannon's stabilization method relies primarily on ionic-transport to effectively move the potassium ions to the interstitial clay sites. However, in conjunction with the ionic-transport, water-transport occurs, and so O'Bannon's soil stabilization technique was initially categorized as employing electro-osmosis for the purpose of soil stabilization. Clearly, this is not the most comprehensive term to use for this process.

In fact as soil stabilization research work proceeded,it became clear that O'Bannon's electro-chemical technique can be characterized as basically the electrolysis of swelling clays.

MISCIBLE DISPLACEMENT

Introduction

Since the soil stabilization process as developed by Dr. O'Bannon consists of transporting KCl solution through the soil, it was found to be necessary to examine what conditions affected this mass transport and how they affected it.

The study of the movement of water, containing dissolved solutes, into and through the soil is called Miscible Displacement.

The term Miscible Displacement is a relatively new one, however, the technique is very old and has long been used. For example, reclaiming saline and alkali soils by leaching out the excess salts has been used for many years but the process has been examined analytically only recently.

Solute Displacement

Briefly, the following takes place in a porous medium as one fluid, say pure water, displaces a KCL solution.

The solutes, in this case KCl, in the displaced fluid are transferred through the soil by the mass transport of the moving fluid and by thermal agitation. Fluid flow through soil is, of course, very complex. The fluid velocity fluctuates rapidly due to the complex geometry of the medium. This complex flow causes the KCl in the solution to move into the pure water. The term for this mass-transport is dispersion, often called hydrodynamic dispersion, to distinguish this spreading mechanism from that due to diffusion. This distinction is necessary because diffusion is due to the random thermal motion of the solute molecules, whereas dispersion is due to the macroscopic fluctuations in the flow of the fluids through the complex pore systems. Thus, the movement of a solute across the initially sharp boundary between the displacing and the displaced fluid can be due to dispersion or diffusion or both.

The shape of a boundary between two different fluids and concentration gradients as they emerge from a soil column could yield information about the magnitude of viscous drag of the fluid on the pore walls, the velocity distribution of the fluid in the pores, the configuration of the pores, the amount of diffusion taking place in the pores, and even the degree of ion exchange taking place. However, such an experimental procedure is not easy to set up, in fact, little experimental work using such a procedure has even been attempted.

Another approach adopted by Drs. O'Bannon and Mancini for this project, is to examine the concentration change of a displaced solute in the effluent as the boundary of the displacing fluid and the displaced fluid emerges from a soil column. The manner in which the concentration changes can give us some information about the porous media and the physical behavior of the fluids moving through the media.

Experimental Procedure

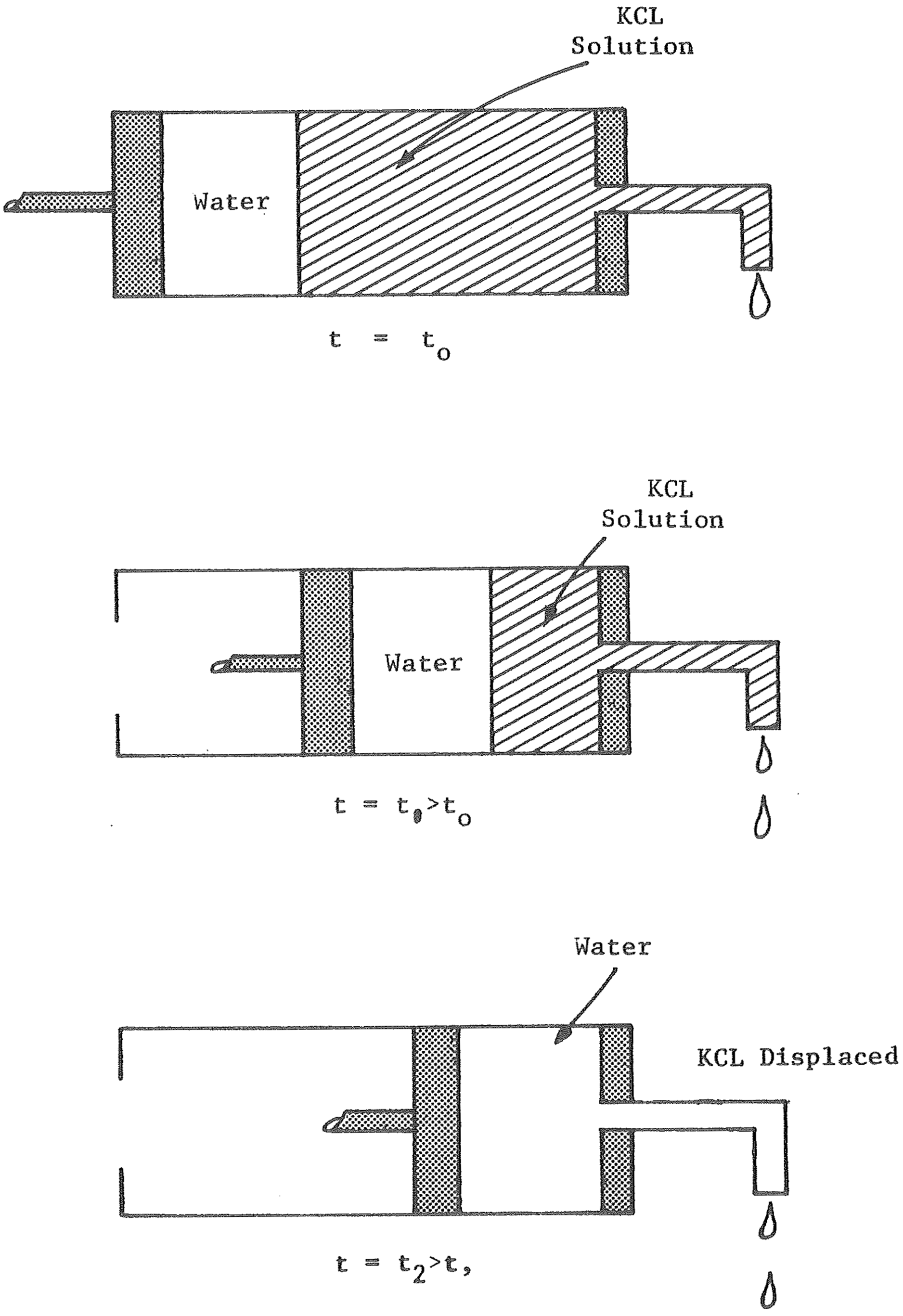
To analyze the concentration change of solute in the effluent from a soil column the data is presented in a standard form called a breakthrough curve.

A breakthrough curve is a graph of the ratio C/C_o versus the number of pore volumes of effluent collected. Here C is the concentration of the solute found in the effluent, C_o is the initial concentration of the solute in the displacing fluid and the pore volume V_o is the volume of the porous medium occupied by fluid. The number of pore volumes of effluent collected is given by Qt/V_o where Q is the quantity of flow per unit time and t the interval of time that has elapsed since the displacing fluid was added to the medium. The quantity Qt is also the volume of effluent collected. The ratio C/C_o will be zero at first and then approach 1 as the concentration of the effluent approaches that of the displacing fluid, that is when most of the displaced has been removed from the medium and the effluent is composed mostly of the displacing fluid.

There are two classes of Miscible Displacement, they are "piston flow" or flow without mixing and flow with mixing.

Piston flow is schematically shown in figure 7. In the upper part of that figure there is a tube with a piston at the left end and a small outlet tube at the right end. In the tube there is a salt solution, say a saturated KCl solution, initially separated from pure water by a membrane. If we pull out the membrane and push the piston fairly rapidly to the right so that there is no time for diffusion to occur across the boundary line between the salt solution and the water, the water will be moved to the right without any mixing at the interface. This is shown in the middle part of figure 7. The effluent will be KCl solution until the water front reaches the end of the tube. Then, only pure water will come out of the tube as shown in at the

Miscible Displacement



Schematic : Drawing of <u>Piston Flow</u> in a Tube, no mixing involved.

Figure 7

bottom of figure 7. Hence, in this type of flow the displaced fluid will move at the rate of the moving piston. In piston flow there is no viscous drag of the fluid along the walls of the tube and no turbulence, moreover piston flow almost never occurs in soils.

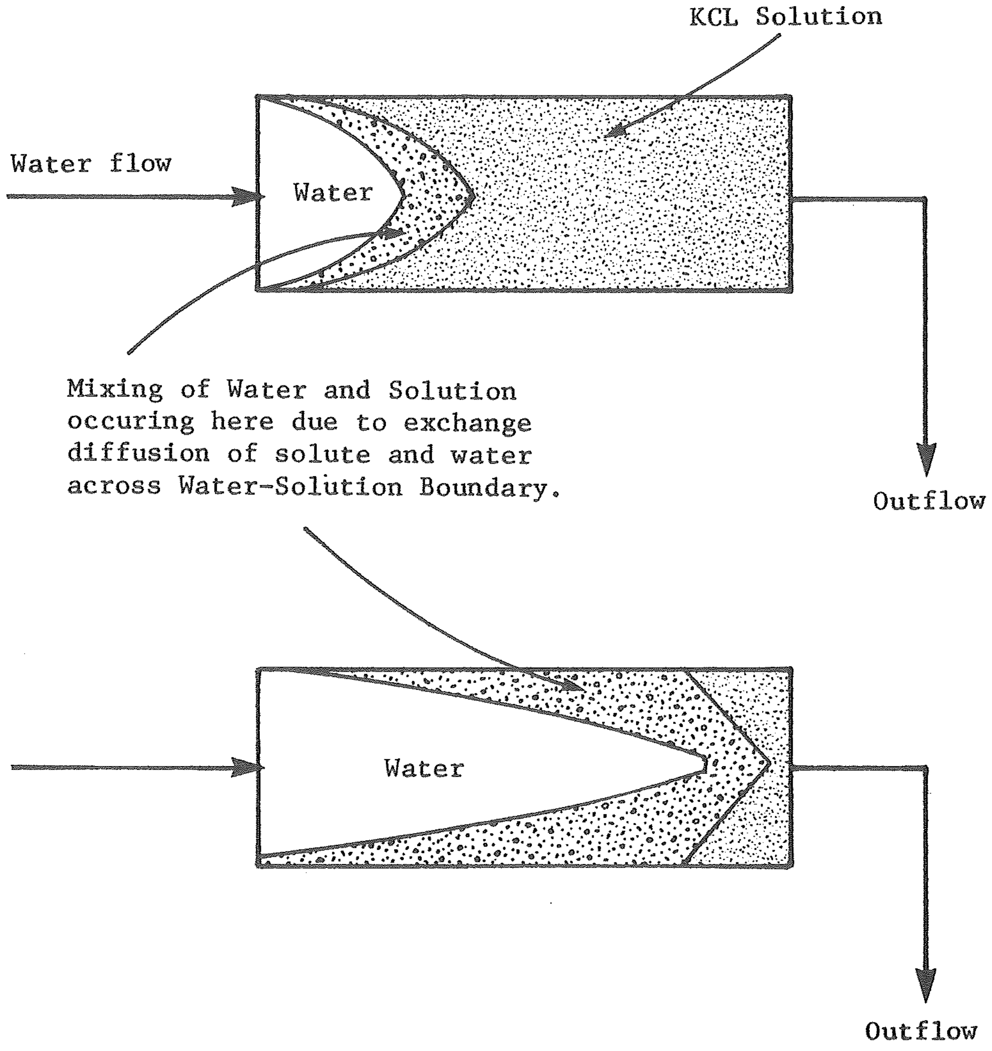
A more realistic type of flow through a tube is shown in figure 8-a. The tube is shown to exhibit a drag on the fluid. It is shown in the upper part of this figure that as the water is pushed into the tube at the left it moves faster down the center of the tube than it does along the walls. This is, of course, due to the viscous drag of the fluid near the wall which causes the velocity of the fluid next to the wall to be less than that near the center of the tube.

In the lower part of the figure it is shown that the salt front has advanced much further along the axis and it will not be long before the water will move out of the tube and be found in the effluent. Then as additional water is moved through the tube the concentration of the salt in the effluent will decrease. This is distinctive from the pure piston type flow since not just salt solution then water will move out of the tube, but an effluent with a measurably decreasing concentration of salt. It is shown in theoretical treatments that the smaller the diameter of the tube and the longer its length, the greater is the mixing.

Breakthrough Curves

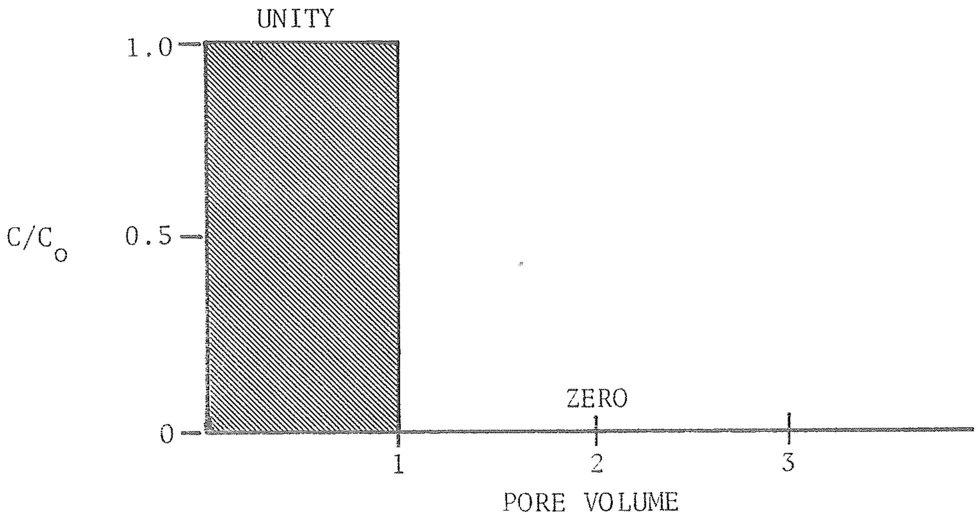
An example of breakthrough curves for both piston flow and flow in a tube are provided in figure 8-b. Notice that for piston flow one pore volume of effluent passes out of the medium before any change in the concentration

Miscible Displacement

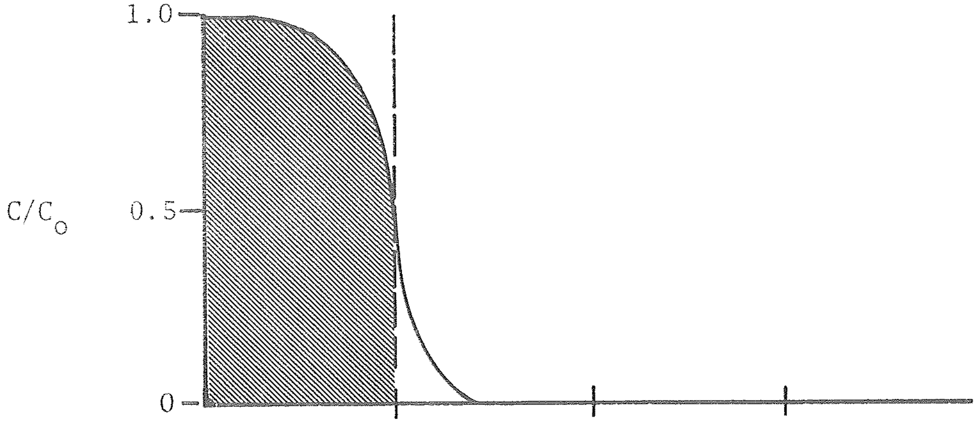


Schematic drawing of Actual flow in tube

Figure 8-a



BREAKTHROUGH CURVE FOR PISTON FLOW



BREAKTHROUGH CURVE FOR FLOW IN A TUBE

Figure 8-b

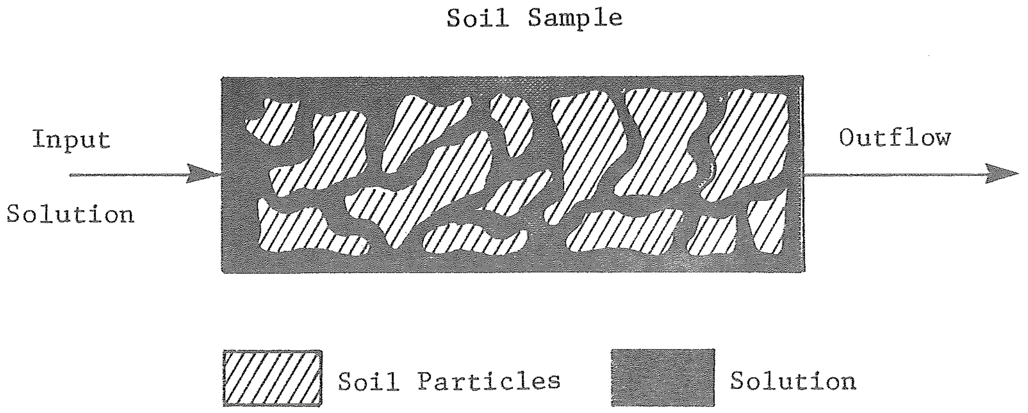
ratio C/C_0 takes place. Then, in a very short time interval the concentration ratio goes to zero. In the case of flow through a tube the concentration ratio remains constant until about a pore volume or so is collected then the concentration ratio begins to decrease in a relatively gradual manner.

In figure 9 one sees how a displacing fluid will move through a saturated non-swelling soil. The displacing fluid will tend to move mostly through the large pores and displacement will be slow in the smaller pores, moreover diffusion will occur.

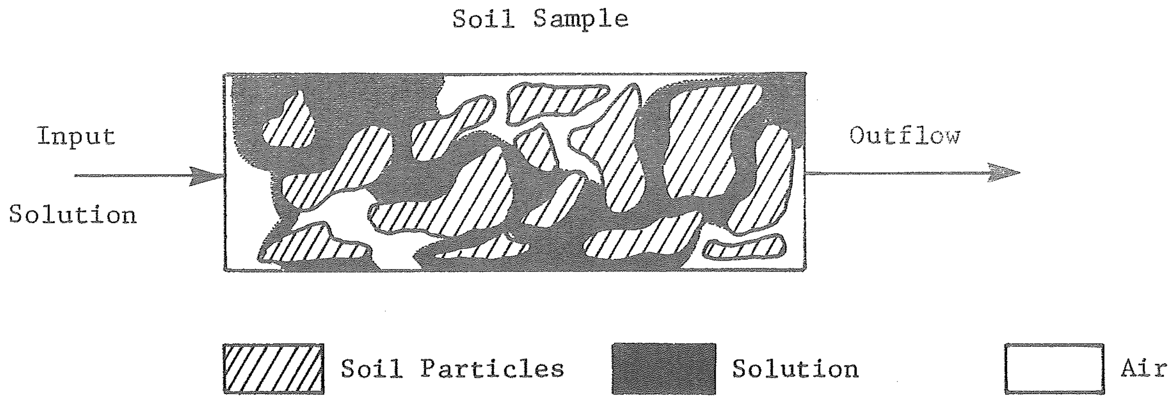
In laboratory experimental set-up the soil sample was not saturated, the moisture content being about 5 percent. Indeed it was found that if the soil was saturated with deionized water then the electrolyte would not move into the soil even under applied pressure gradients. With unsaturated soil the situation is more complex than for saturated soils. This case is shown in figure 10 where there are soil particles, water and air spaces. It should be stressed that this figure is drawn without taking into account any volumetric change due to swelling. In this situation the large pores, which, under saturated conditions conducted the salt solution easily, are now partly filled with air and have lost their conducting effectiveness. The salt solution will tend to go where the water films are the thickest.

Diffusion of a solute may take place during miscible displacement. If the fluids of the miscible displacement process move in the soil at practically zero velocity, some of the salt molecules may move forward and completely out of the soil just by the thermal motion of diffusion.

Miscible Displacement



Schematic drawing of flow in a Saturated Soil



Schematic drawing of flow in an unsaturated Soil

Figure 9 & 10

Under field conditions one will almost always have an unsaturated soil medium at hand. In this case the theory as developed can only serve as general guides for qualitative judgments. This is due to the wide range of pore velocities present, during the displacement process, in the unsaturated soil.

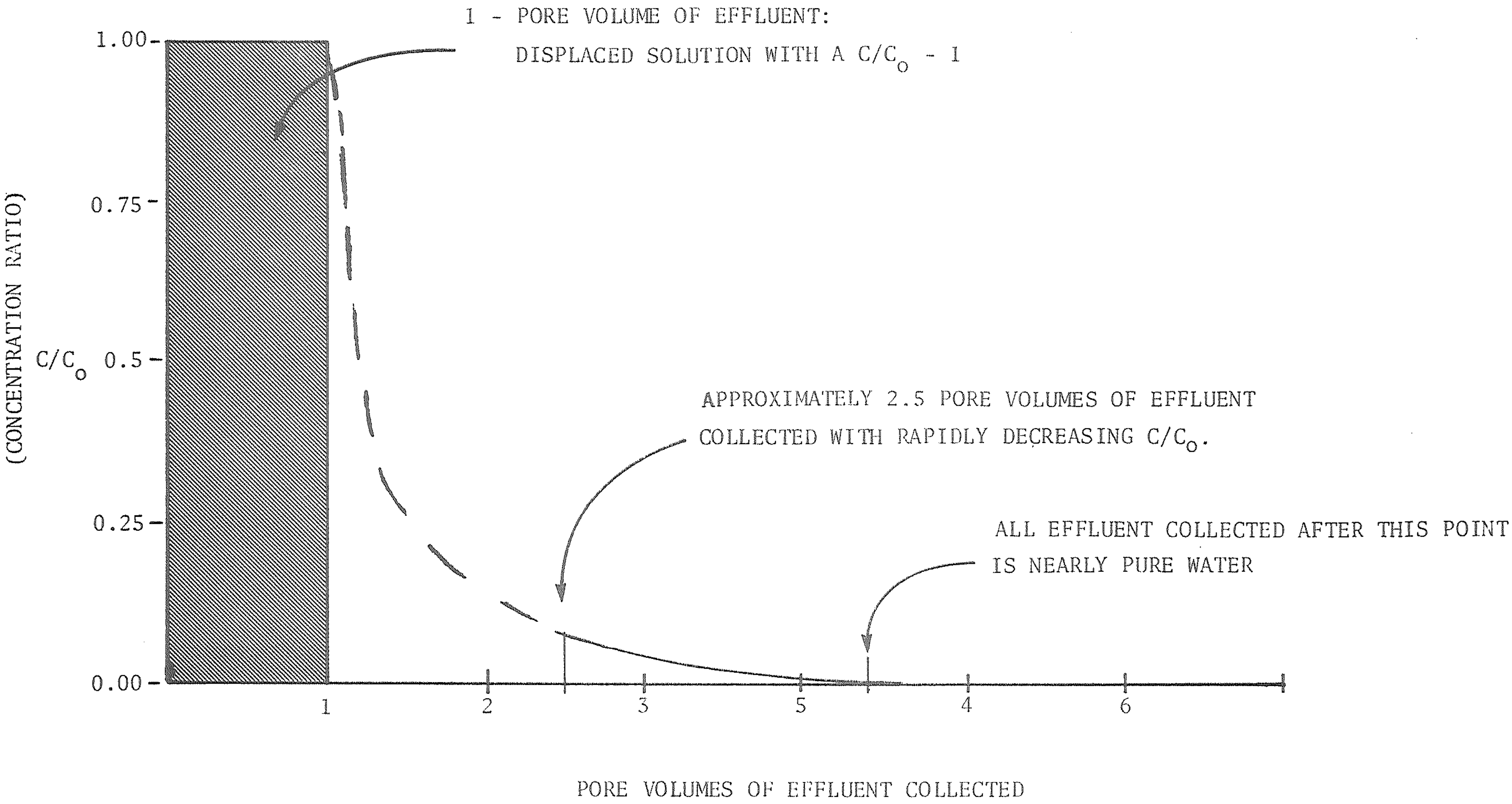
The miscible displacement experiments performed for this project were necessarily restricted in scope because of the limited time available.

It is apparent from figure 11 that after a Chinle Clay soil is saturated with an electrolyte (0.4 N KCl) and then the solution is displaced with deionized water the removal of the chlorides takes place very nearly like a classical "breakthrough" curve. This data suggests that the colloidal particles making up the soil have agglomerated, from the action of the electrolyte on the double layer, and created continuous pores so that the solution and water can pass through the clay column with relative ease.

More over even after several ml of deionized water were run through the sample and the effluent's salt concentration had reached an equilibrium value the clay's properties did not significantly alter and the sample retailed its relatively high permeability relative to the deionized water.

In attempting to run the experiment in the other direction, that is by using a sample saturated with deionized water and then using the electrolyte as the displacing fluid, our efforts were thwarted by the properties of the clay. That is, the clay so tenaciously held onto the deionized water that the 0.4 N KCl solution was not able to enter the soil and displace the water, this occurred even with applied pressure gradients.

MISCIBLE DISPLACEMENT OF KCL SOLUTION
FROM
CHINLE CLAY SOIL COLUMN



1 - PORE VOLUME OF EFFLUENT:
DISPLACED SOLUTION WITH A C/C_o - 1

APPROXIMATELY 2.5 PORE VOLUMES OF EFFLUENT
COLLECTED WITH RAPIDLY DECREASING C/C_o.

ALL EFFLUENT COLLECTED AFTER THIS POINT
IS NEARLY PURE WATER

These experiments clearly indicate that the electrolyte strongly interacts with dry Chinle Clay and is able to easily affect the permeability of the clay if clay has not been pre-saturated with pure water.

CHAPTER IV

Laboratory Work

General

The purpose of the laboratory work during Phase VI was to determine the expansive and swell characteristics of the untreated soil obtained from the test site.

Site Selection and Sampling

The field work for this study began in July, 1973, with the selection of the test site at mile post 328.8± on the westbound lane of Interstate 40. For a plan view of the site see figure 12. Since this project was initiated as an implementation study an engineering decision as to pretest sampling technique was made at the onset of the project.

This engineering decision is based on the following general considerations: most problems areas of swelling clays traversed by a roadway are easily identifiable visually. After an area has been indentified as a possible problem area, a quick economical way of sampling must be used. It is clear that in a study oriented towards purely investigatory goals of soil phenomena a sampling technique used over a selected region should be based on a random procedure. However, the primary goal was to utilize knowledge of past electrochemical work and apply the same techniques, properly modified, to enable a rapid evaluation of the swelling problems of a given area.

LAYOUT OF TEST SITE FOR SOIL STABILIZATION PROJECT
LOCATION: I-40 WESTBOUND LANE
(CRAZY CREEK)

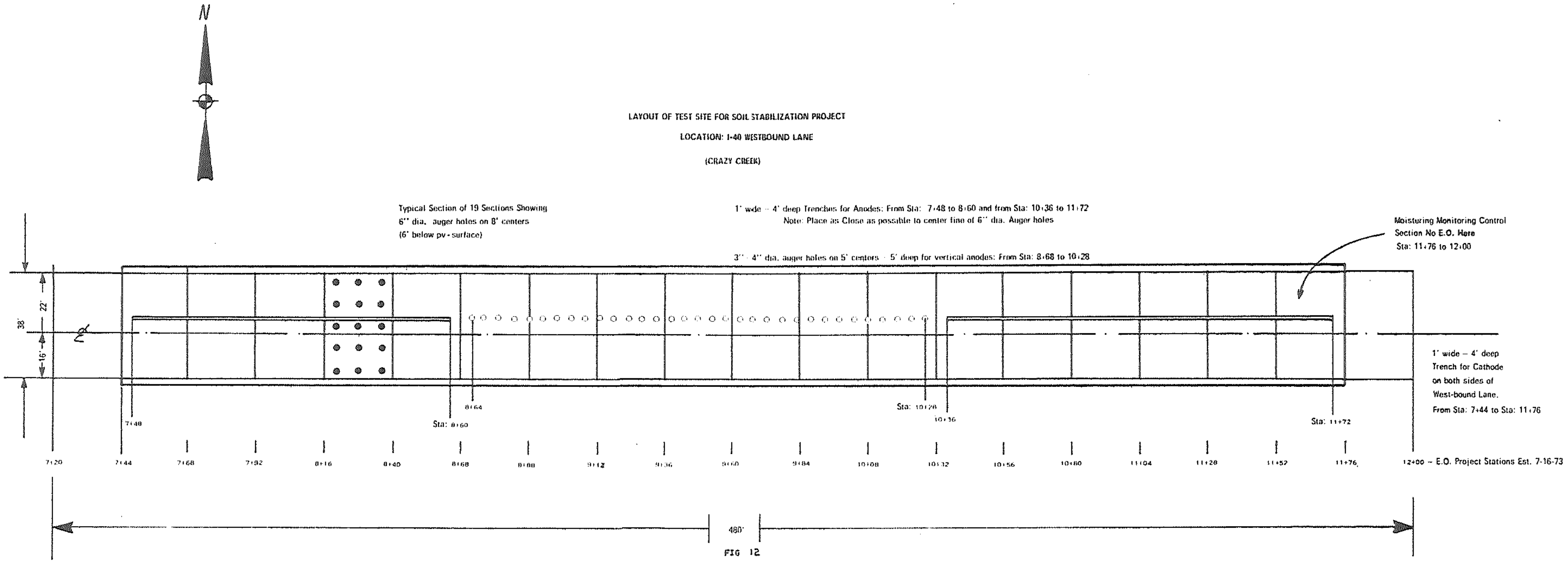


FIG 12

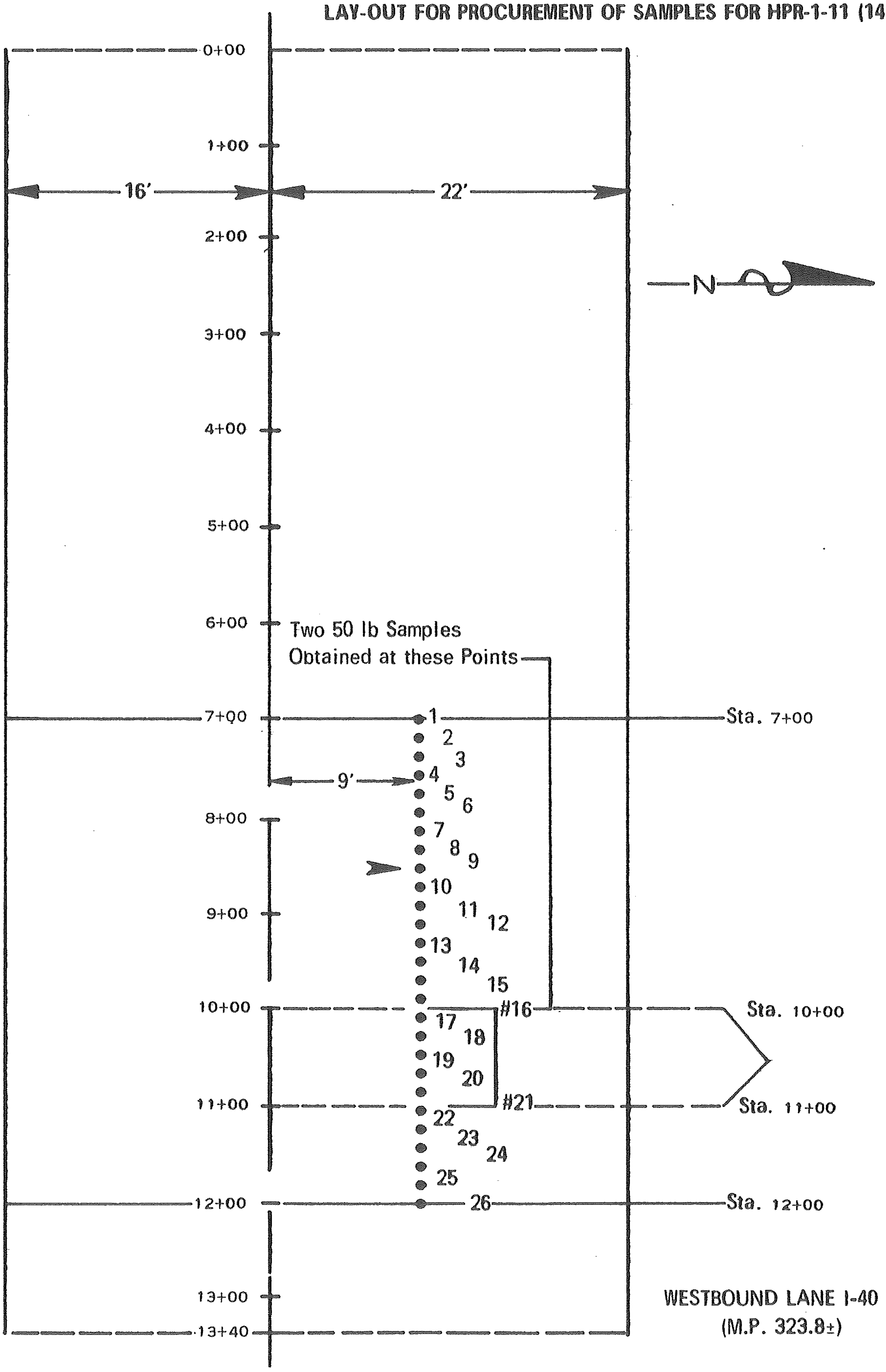
It was decided that the sampling procedure would be undertaken so that the drill rig would not have to move laterally but only longitudinally, this would save a great deal of time, hence money. Moreover, the rig was positioned as close to the center of the interstate as possible without interfering with traffic in one travel lane of the westbound roadway. Twenty-six test holes were drilled and samples were obtained to a depth from zero to 15 feet in the clay subgrade. See figure 13. The samples were transported to the Arizona Department of Transportation Materials Services where they were prepared and used in laboratory studies.

Sample Preparation

The samples were prepared in the following manner, see figure 14. The soil was sieved through a #4 sieve. All material passing the #4 sieve was stockpiled. The material retained on the #4 was crushed in a jaw crusher. The material was again sieved and the minus #4 added to the stockpile. This crushing and sieving process was repeated until all the material had passed the #4 sieve. The minus #4 material was then mixed and stored until needed for laboratory testing. In this manner, a homogeneous sample could be obtained for all further testing operations. See figure 14 for the flow diagram of laboratory work.

Note Stations 0 + 00 – 13 + 40 were set down July, 1973 by Dist. IV for Project HPR-1-11 (145)

LAY-OUT FOR PROCUREMENT OF SAMPLES FOR HPR-1-11 (145)



Pre-samples Taken Approx. July 16, 1973

WESTBOUND LANE I-40
(M.P. 323.8±)

Fig. 13

FLOW CHART FOR EXPANSIVE PRESSURE
(% SWELL FOR UNTREATED SAMPLES)

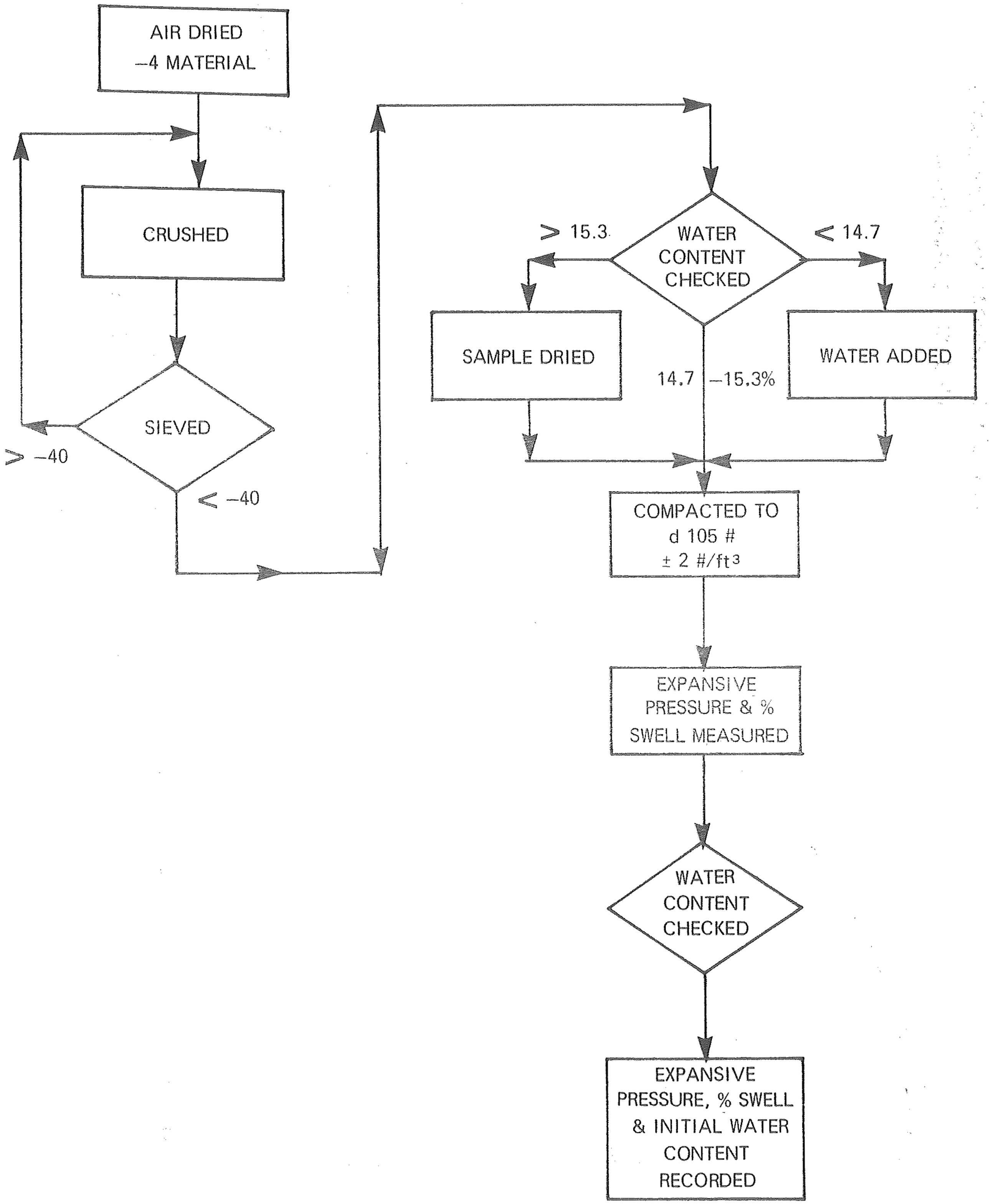


Figure 14

I-40 ELECTRO CHEMICAL SOIL STABILIZATION SITE
GRAIN SIZE DISTRIBUTION

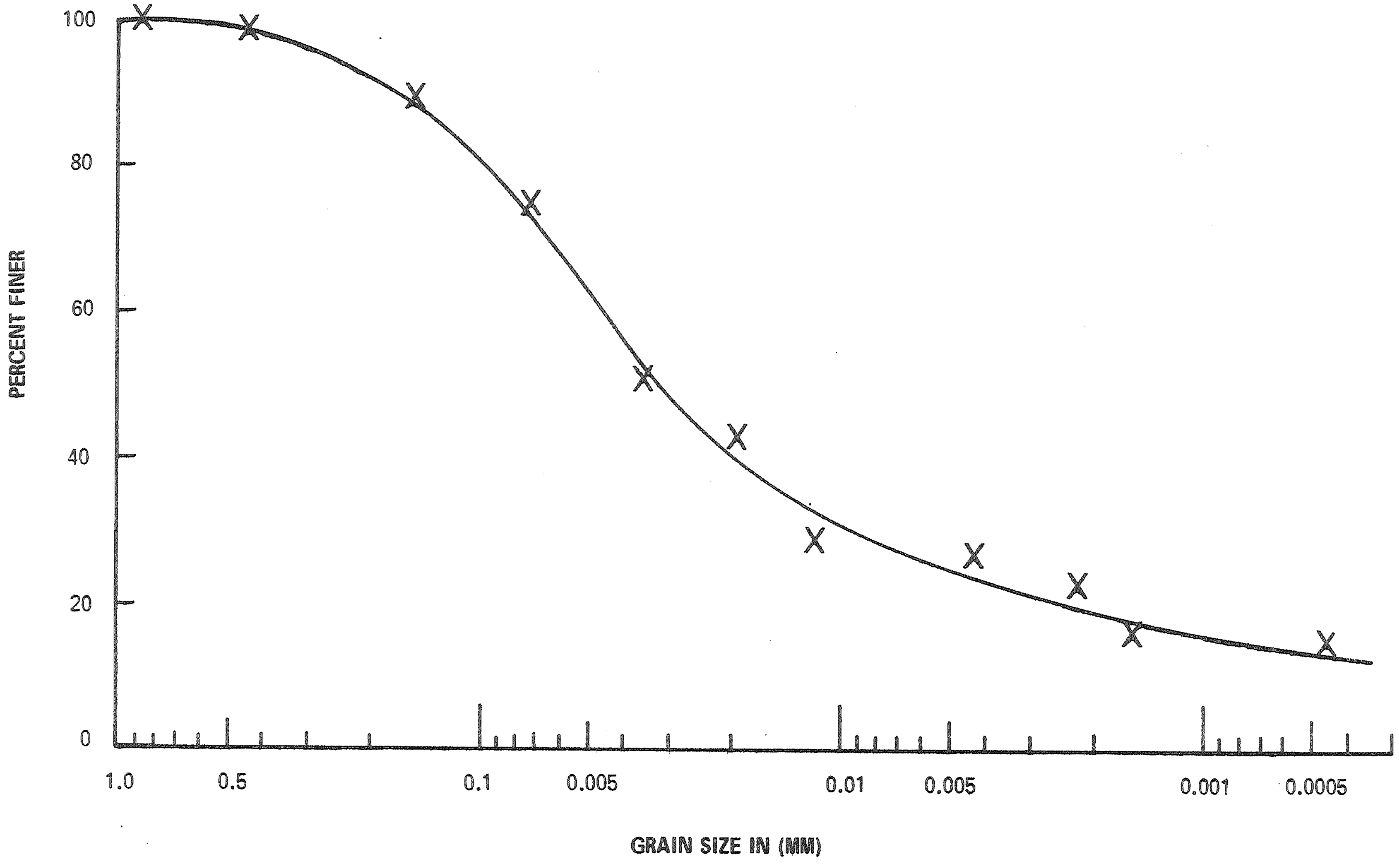


Fig. 15

Soil Properties of Untreated Material

A series of tests were conducted to determine the index properties and grain size distribution of the soil. The average values for the atterberg limits on the samples were liquid limit 39, plastic limit 17, and plastic index 22. The average percent passing the #200 sieve was 70% and its grain size distribution is shown in figure 15. The specific gravity of the soil was 2.75. The soil would classify as a CL material based on the unified soil classification system. See table No. 1 and figure 3 for the results of the atterberg limits tests.

Expansive Pressure and Percent Swell Tests

The expansive pressure of the untreated soil was determine for material passing a #40 sieve in a standard R-value testing apparatus. The density of the soil was approximately 106 $\#/ft^3$ and the moisture contents were 10 and 15 percent. The results of these tests are shown in table No. 2. The percent swell was determined on the untreated soil using a modified clock house apparatus. The density was maintained at 106 $\#/ft^3$ and the moisture contents were approximately 10 and 15 percent. The results of these tests are also shown in table No. 2.

These tests indicate that the selected area is representative of a region with moderate swelling characteristics.

Table No. 1

INDEX PROPERTIES

Sample No.	Location	Depth ft.	LL	PL	PI	%-#200	Unified Classification Symbol
40133	7+00	0-1.5	43	22	21	92	CL
40134	7+00	1-5-2.5	37	18	19	78	CL
40135	7+00	2.5-3.5	35	17	18	60	CL
40149	8+00	0-1.5	42	16	26	76	CL
40150	8+00	1.5-2.5	40	15	25	60	CL
40151	8+00	2.5-3.5	43	17	26	76	CL
40165	9+00	0-1.0	28	14	14	48	SC
40166	9+00	1.0-2.0	36	14	22	66	CL
40167	9+00	2.0-3.0	32	15	17	61	CL
40181	10+00	0-1.0	37	19	18	75	CL
40182	10+00	1.0-2.0	42	19	23	83	CL
40183	10+00	2.0-3.0	40	17	23	80	CL
40197	11+00	0-1.5	38	17	21	65	CL
40198	11+00	1.5-2.5	44	19	25	66	CL
40199	11+00	2.5-3.5	44	20	24	77	CL
40213	12+00	0-1.5	36	16	20	57	CL
40214	12+00	1.5-2.5	41	16	25	70	CL
40215	12+00	2.5-3.5	41	19	22	75	CL

Table No. 2

EXPANSIVE PRESSURES AND PERCENT SWELL

Sample No.	Location	Depth ft.	Density #1ft^3	% - Water	Expansive Pressure #1 ft^2	% - Swell
40133	7+00	0-1.5	106.5	9.9		17.35
"	"	"	-	10.1	4914	
"	"	"	106.4	15.0		12.50
"	"	"	106.0	15.6	4664	
40134	7+00	1.5-2.5	106.4	10.1		9.72
"	"	"	106.4	10.1	2912	
"	"	"	106.4	15.0		5.81
"	"	"	106.0	15.8	1866	
40135	7+00	2.5-3.5	106.5	9.3		5.06
"	"	"	106.5	10.2	1001	
"	"	"	106.4	15.3		4.96
"	"	"	106.3	15.5	819	
40149	8+00	0-1.5	107.1	9.2		9.00
"	"	"	106.6	10.1	2412	
"	"	"	106.6	15.1		6.60
"	"	"	106.6	15.1	2457	
40150	8+00	1.5-2.5	106.3	9.9		11.99
"	"	"	106.8	9.6	2710	
"	"	"	106.9	14.6		8.70
40151	8+00	2.5-3.5	106.4	9.3		9.06
"	"	"	106.9	9.9	1820	
"	"	"	106.5	15.2		4.3
40165	9+00	0-1.0	106.5	9.9		4.78
"	"	"	106.4	10.5	523	
"	"	"	106.4	15.4		2.50
40166	"	1.0-2.0	106.4	9.9		7.37
"	"	"	106.8	9.9	1183	
"	"	"	106.6	14.9		4.90
"	"	"	106.6	14.9	837	
40167	"	2.0-3.0	105.8	9.9		6.99
"	"	"	106.8	10.2	1456	
"	"	"	108.1	13.3		4.40
"	"	"	106.7	15.1	319	
40181	10+00	0-1.0	106.0	9.9		9.66
"	"	"	106.4	9.2	2730	
"	"	"	106.5	15.4	1866	
40182	"	1.0-2.0	106.2	9.2		15.51
"	"	"	106.4	10.5	4368	
"	"	"	106.8	14.6	2730	
40183	"	2.0-3.0	106.1	9.9		10.59
"	"	"	106.2	10.4	3117	
"	"	"	106.4	14.8		10.00
"	"	"	106.4	15.2	2480	
40197	11+00	0-1.5	106.4	10.2		9.49
"	"	"	106.8	9.8	2093	
"	"	"	106.4	15.0		7.60
"	"	"	106.6	15.2	1570	
40198	"	1.5-2.5	106.4	10.2		9.17
"	"	"	106.8	10.0	2275	
"	"	"	106.7	14.9		7.50
"	"	"	106.6	15.2	1456	
40199	11+00	2.5-3.5	107.3	10.5		17.77
"	"	"	106.1	10.4	2975	
"	"	"	106.7	14.8		15.70
"	"	"	106.2	15.8	3822	
40213	12+00	0-1.5	106.7	10.2		6.69
"	"	"	107.4	9.3	1722	
"	"	"	107.0	14.8		3.90
"	"	"	106.7	15.2	728	
40214	"	1.5-2.5	106.7	10.2		11.62
"	"	"	106.5	10.2	2002	
"	"	"	106.0	15.5		8.80
"	"	"	106.4	15.1	1138	
40215	"	2.5-3.5	106.5	10.1		11.05
"	"	"	106.9	10.0	3640	
"	"	"	106.4	15.1		9.5
"	"	"	106.6	15.0	1547	

Determination of Optimum gmKCl/gm Clay

The next important determination to make was the amount of KCl per unit wt. of clay necessary for proper stabilization of this particular clayey mass. Because of the large mass of data from previous soil stabilization work using this technique it was relatively straightforward to closely estimate the amount of KCl required to reduce the expansive pressure by some reasonable factor.

It was decided to use an expansive pressure reduction factor of 2 as a guideline in determining the overall amount of KCl needed. To use higher factors, say 4 or 5, would result, when coupled with the electrochemical effects induced by the elctrical current in the soil, in a very expensive overkill.

Thus, based on previous soil stabilization work a decision as to the gm. wt. of KCl per gm. wt. of clay was made during the logistical build up period.

It was determined from figure 16 that a best estimate from both an engineering and economic standpoint was about 0.02 gm KCl/gm clay. This amount of KCl added to the clay in addition to the fabric changes in the clay, induced by the elctrochemical process, was adjudged to be sufficient for our purposes.

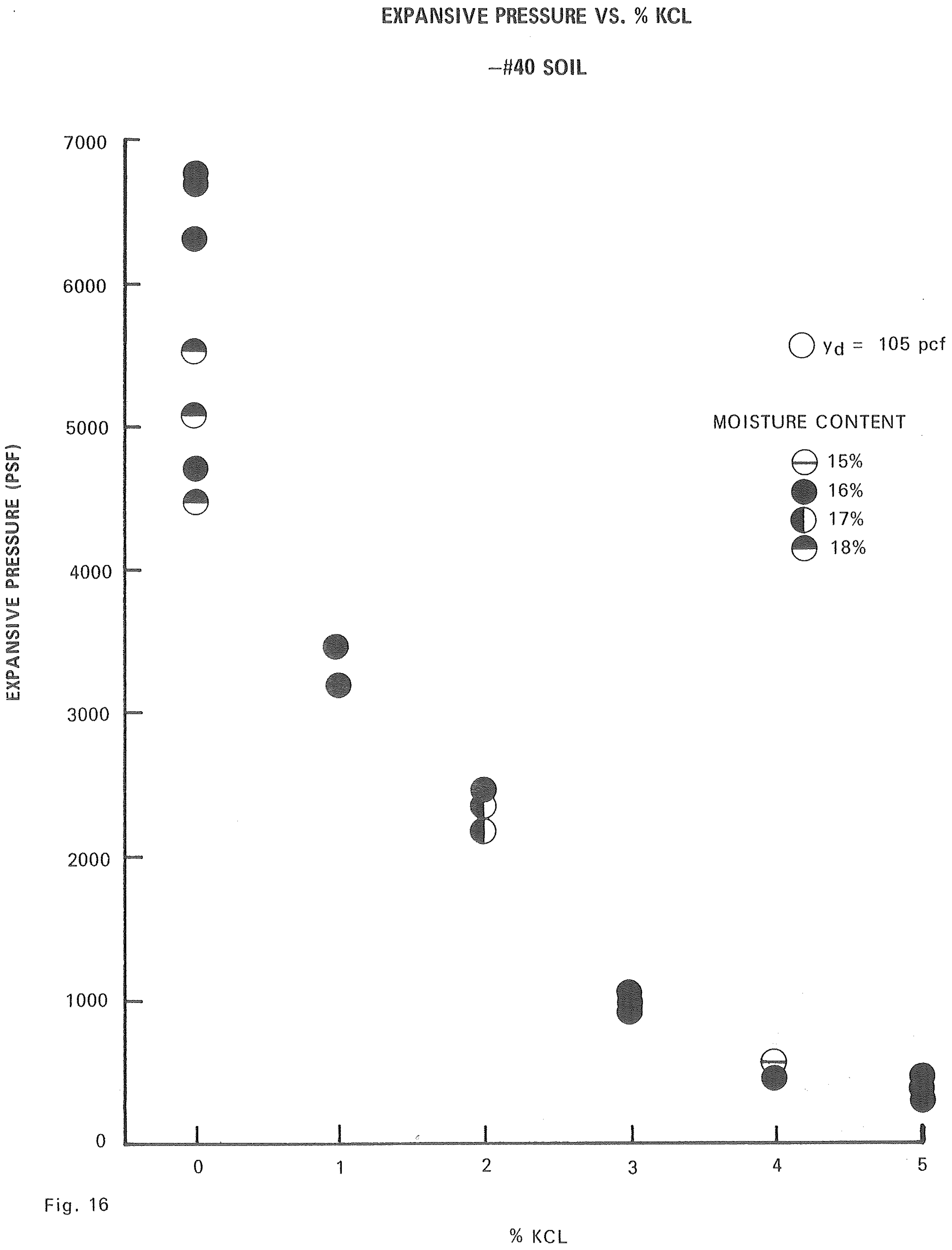

EXPANSIVE PRESSURE VS. % KCL
—#40 SOIL
EXPANSIVE PRESSURE (PSF)
7000
6000
5000
4000
3000
2000
1000
0
0
1
2
3
4
5
% KCL
γd = 105 pcf
MOISTURE CONTENT
15%
16%
17%
18%

Fig. 16

CHAPTER V

FIELD WORK

General

The scope of the field work of the electrochemical stabilization of Chinle clay was

(1) To choose a test site on a paved section of I-40 near Holbrook, Arizona and obtain representative samples of the expansive clay

(2) To design, install and operate a full-scale field test on the test section.

(3) To sample the electrochemically treated section.

(4) Evaluate the ability of existing maintainence resources to carry out this work with a minimum of specialized personnel in attendence.

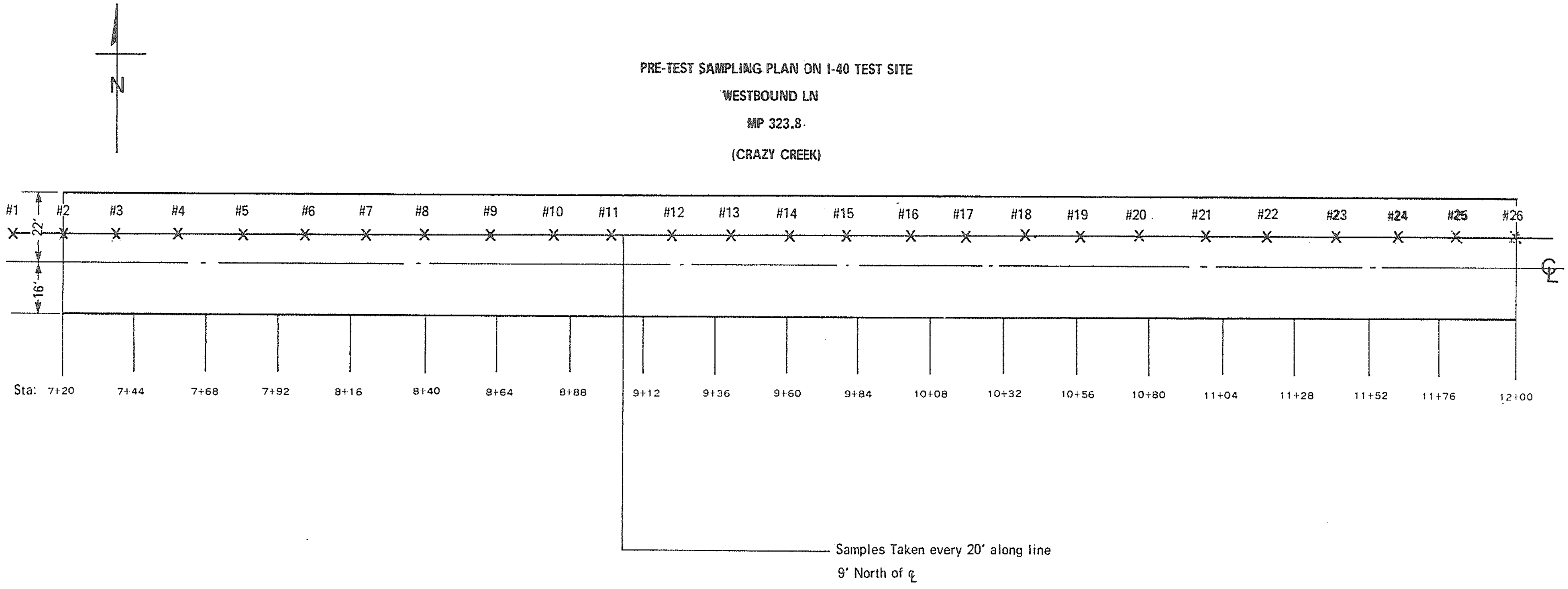
Site Section and Sampling

The field work of this project began in July, 1973 with the selection of a new test site east of Holbrook. This site is located approximately on MP 323.8± in the westbound lane of I-40. Engineering stations were set up and representative samples of the soil were taken from the test section, see figure 17. The Department of Transportation Materials Services Division transported the material to the main laboratory where it was prepared and used in laboratory studies.

Field Test Operations

Design - The highway field installation was designed to reproduce the simple electrode configurations used in previous studies.

PRE-TEST SAMPLING PLAN ON I-40 TEST SITE
WESTBOUND LN
MP 323.8
(CRAZY CREEK)



Samples Taken every 20' along line
9' North of ℄

Fig 17

The electrical design was such that three separate sections of the 500' test section could be electrified simultaneously with the same voltage gradient.

The electrode configurations for the whole test site is shown in Figure 18. It should be noted that the center section has, as anodes, verticle #8-1" (.0254 M) rebar about 5' (1.53 M) long. This is in contrast to the other two sections which have horizontal anodes made up of 20' (6.10 M) section of #8 rebar welded together end to end to form the anodes. After the horizontal anodes were formed in this manner they were manually placed into the previously prepared 4' (1.22 M) deep trench.

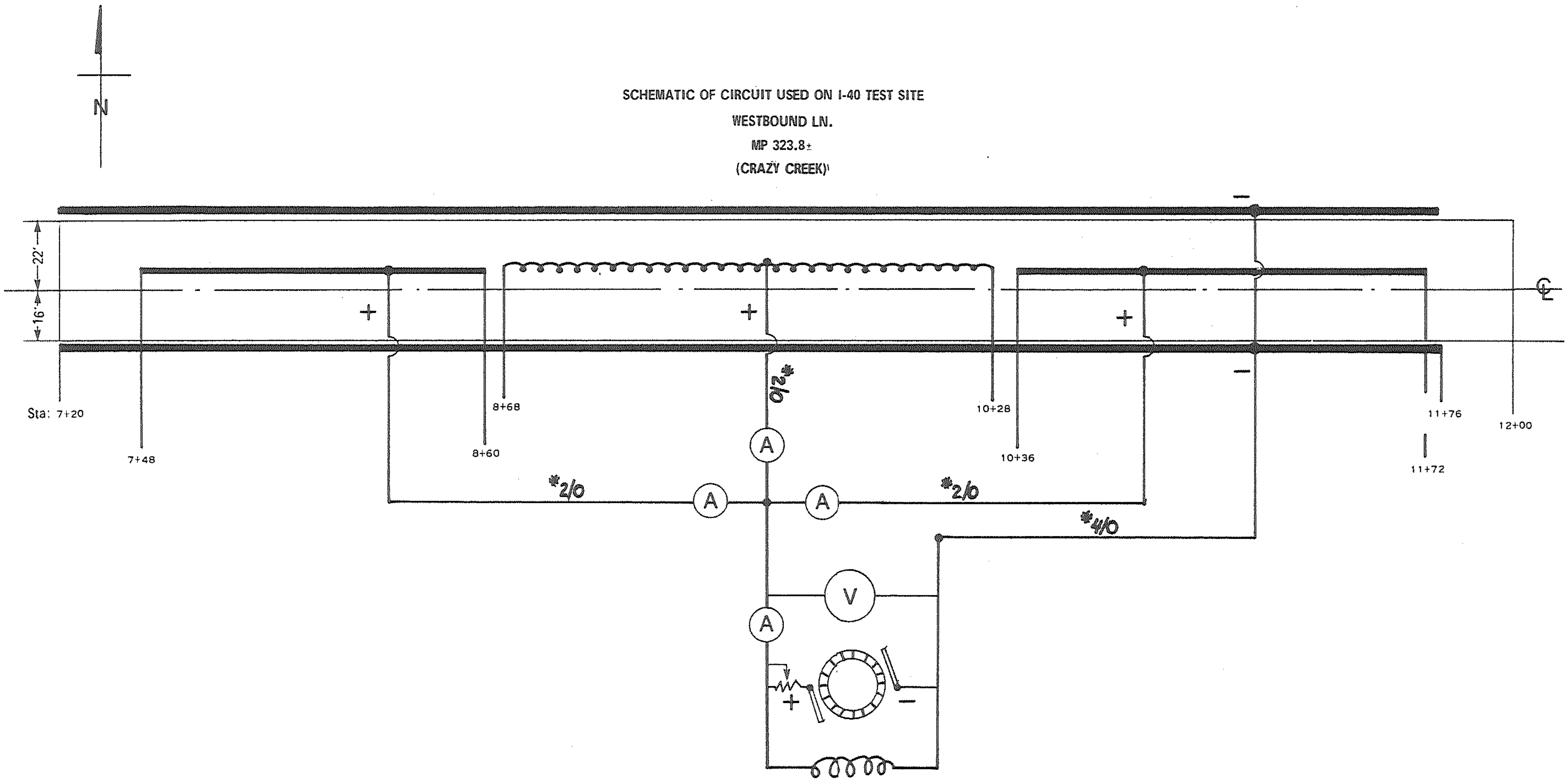
The cathodes, shown in Figure 19 alongside I-40 was formed of 20' (6.10 M) sections of #8 rebar welded end to end and then carefully lowered into the trench.

The electrode configuration was designed to be very easy to assemble and yet be an effective item. While many sophisticated design patterns exist(1) it was obvious that the simple design used in this project would require the minimum amount of expert man hours to properly install on site.

Figure 20 provides an idealized pattern of the electric field created by this electrode pattern. It must be noted that in this illustration one has not taken into account the effects of the auger hole steel sleeve, on the electric field. These sleeves would not only distort the field, but can even act as partial screen to the field thus reducing the overall effect of the electrical field.

(1)See: "Innovations in Ground Stabilization" by James K. Mitchess; ASCE 1972, Chicago Soil Mechanics Lecture Series, "Innovations in Foundation Construction."

SCHEMATIC OF CIRCUIT USED ON I-40 TEST SITE
WESTBOUND LN.
MP 323.8±
(CRAZY CREEK)



DC Shunt Generator
60 KW Capacity
(Circuit Not to Scale)

Fig 18

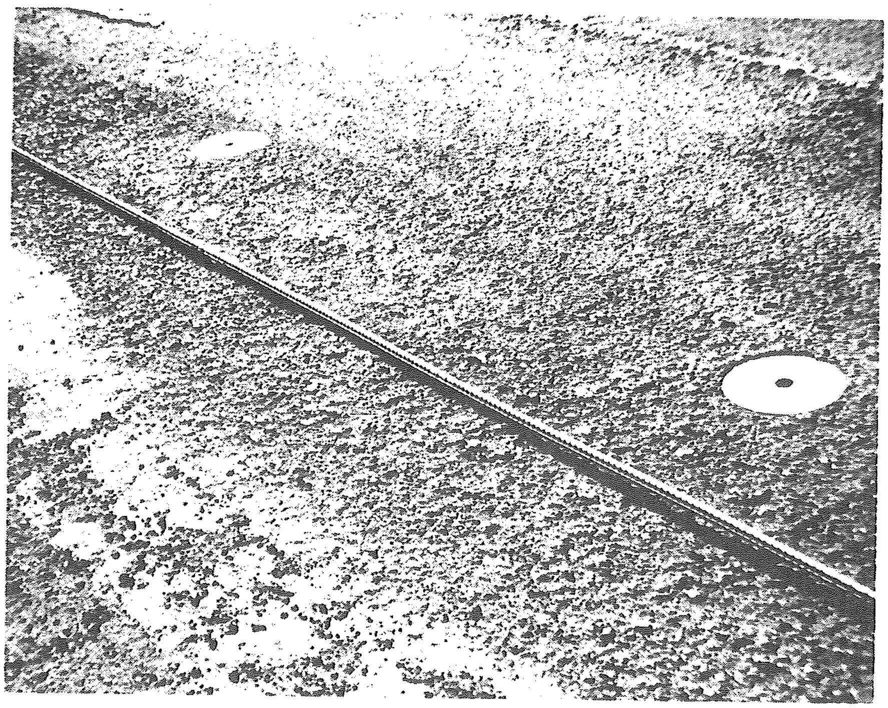
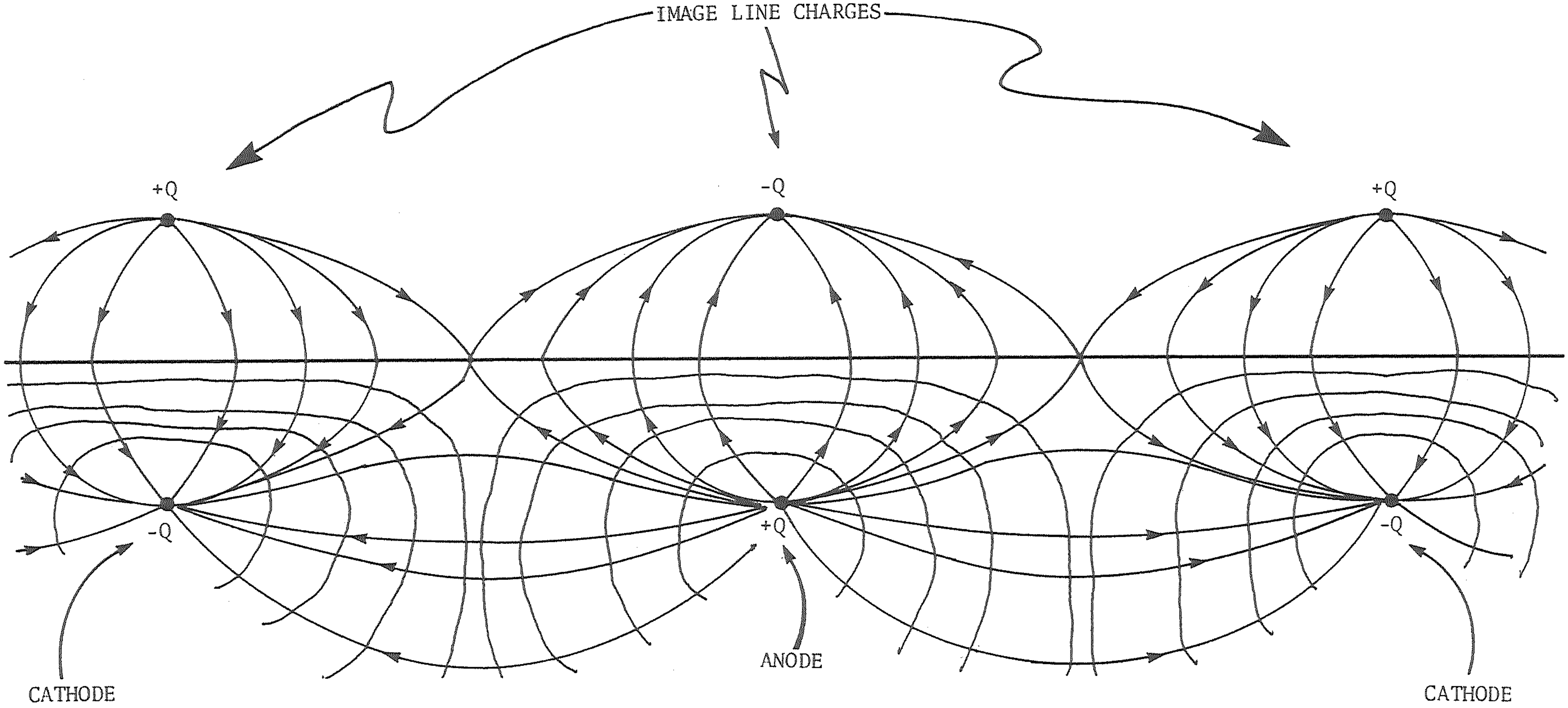


Figure 19

Figure 19 shows a section of the cathode before being placed into the trench seen near the left-hand corner of the picture. Note the sleeved auger holes clearly showing the 10" (.254 M) plate with the solution input opening in the center.

SCHEMATIC OF IDEALIZED ELECTRIC FIELD
ESTABLISHED WITH ELECTRODE CONFIGURATION USED
ON I-40 SITE
NOT TO SCALE



NOTE: CATHODE AND ANODE POSITIONS REPRESENTED AS LINE CHARGES.
SURFACE OF CLAY ASSUMED TO BE CONDUCTING. PLANE

Figure 20

As a result of past field tests using electrochemical methods for stabilization of Chinle clay it was decided to design the field installations around the desire that the clay should be treated to a depth of only 3 feet (.915 M). This estimate was obtained based on laboratory tests which showed that if the clay were effectively treated to this depth the site could be judged as stabilized. Moreover, because of these previous study results, it was obvious that to attempt to treat the clay to a greater depth than 3' (.915 M), say 4' (.122 M) would result in an "overkill" of the first 3' (.915 M).

In order to properly suffuse the soil pores with the KCL solution it was decided to drill 6" (.1524 M) diameter auger holes on 8' (2.44 M) centers, approximately 5.5 (1.68 M) deep throughout the test section. The positioning of the auger holes was based on previous studies.(2) There was a total of 285 of these auger holes position throughout the site, see Figure 21.

To discourage caving of the blow sand subbase material each auger hole was sleeved with a 6" (.152 M) O.D. steel pipe, 27" (.686 M) long, topped with a 10" (.254 M) diameter, 1" (.0254 M) thick steel plates. The plate had a 1-1/2" (.0381 M) hole in the center to permit introduction of the KCL solution into the auger hole. The 10" (.254 M) diameter steel plate was obviously necessary to give a stable platform for the steel and to provide a sufficiently rigid surface for vehicular traffic. See Figure 22 for an example of one of the steel sleeves.

The 10" (.254 M) diameter, 1" (.0254 M) thick steel plate welded to the top of each steel sleeve certainly would give a rough surface to the

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(2) See: "Stabilization of Chinle Clay by Electro Osmosis and Base Exchange of Ions" by Charles O'Bannon, Feb. 1973, Final Report for Arizona Department of Trans. Research Section.

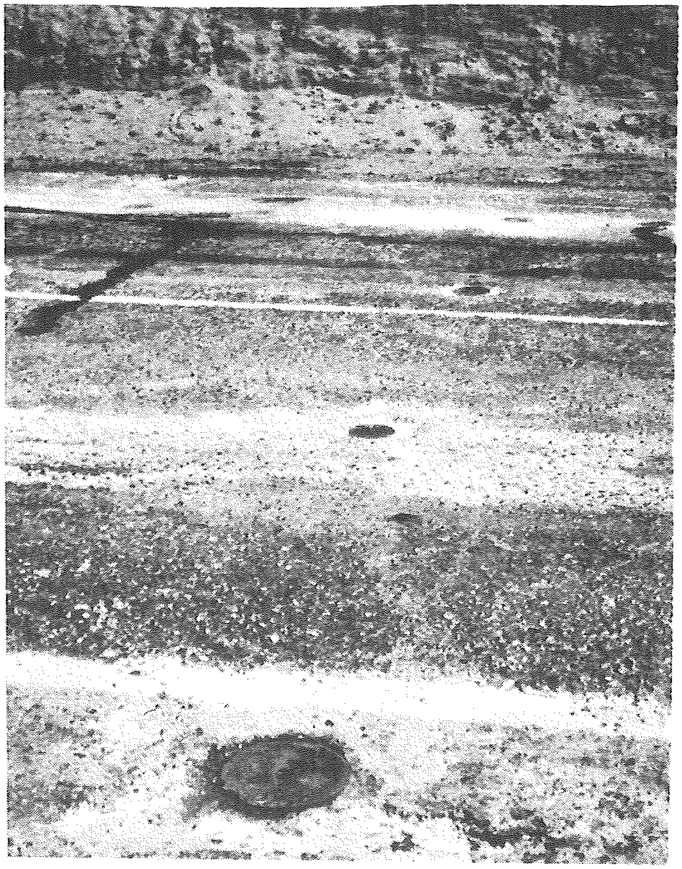

Figure 21. View of auger holes on 8' centers with steel sleeves in place. View is transverse to roadway looking north.

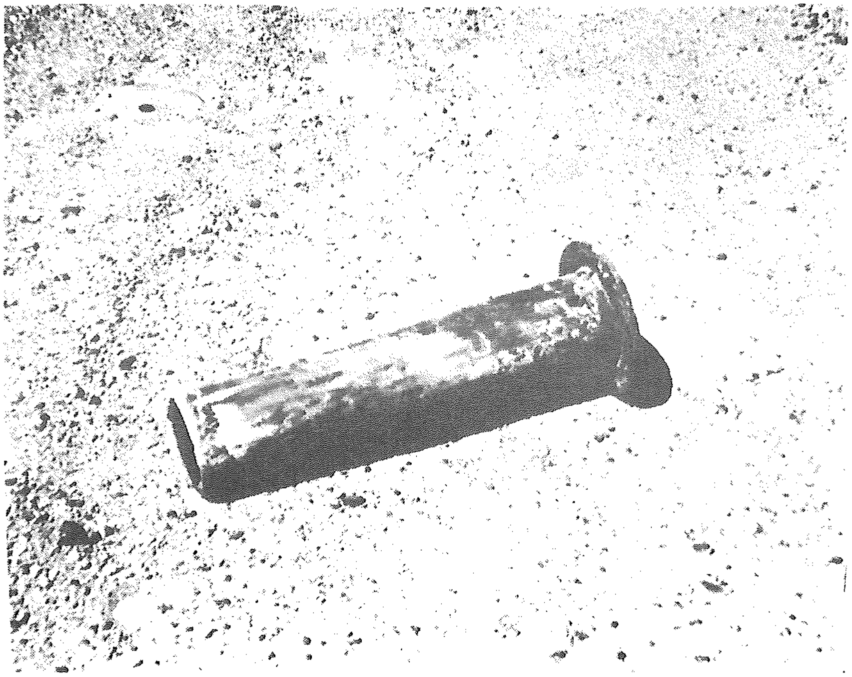

Figure 22 shows 27" (.686 M) long steel sleeve 6" (.152 M) O.D. topped with a 1" (.0254 M) thick steel plate. The plate is 10" (.254 M) in diameter with a 1-1/2" (.0381 M) hole in the center to permit the input of KCL solution into the auger hole.

roadway throughout the length of the 500' (152.5 M) test section after the sleeves were in place. Obviously, 285 protruding 10" (.254 M) steel plates presented a potential traffic problem and made it necessary to countersink each auger hold with a concentric circular 10" (.254 M) diameter, 1" (.0254 M) deep depression. The countersinking operation was accomplished easily by welding to the top of the auger drill stem one of the 10" (.254 M) steel plates.

After this was accomplshed, six old drilling teeth were set flat against the bottom of the plate, spaced about 60° apart in a symmetrical pattern and then welded into place. The teeth then provided an abrasive surface for the countersinking operation.

For the auger hole drilling operation see Figure 23. It should be noted in that figure the countersink apparatus is shown welded onto the top of the auger drill stem.

Each of the a85 auger holes were sleeved down to 27" (.686 M) below the asphalt surface. The sleeving did prevent extensive caving of the base course and KCL solution from wetting the base course. However, there was some caving and wetting. The figure of 27" (.686 M) of base course material was supplied by District IV personnel as a best estimate to use throughout the site. In the future it is recommended that the sleeving be placed about 6" (.152 M) below the estimated base course material.

See Figures 24 and 25 for a view of the site after installations of the casings.

The sleeves were so designed that their reuse was possible after completion of the project. Moreover, only a minor amount of corrosive action due to the salt solution was noted on the sleeves at the end of the field work.

Figure 23. Auger hole drilling. The round plate shown on the asphaltic surface is the countersink plate. It is welded onto the top of the auger drill stem.

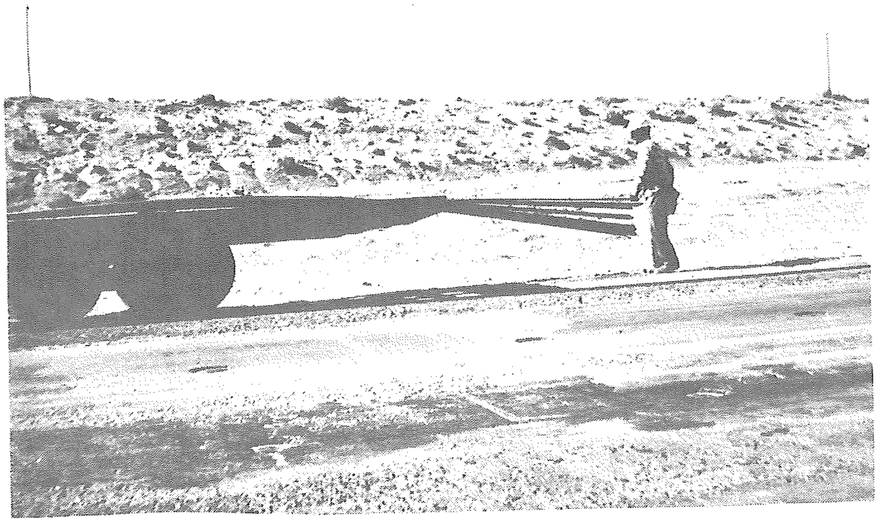

Figure 24. Site view showing auger holes with steel sleeves in plate. Crewman is unloading 20' (6.10 M) sections of #8 rebar, to be used for electrodes.

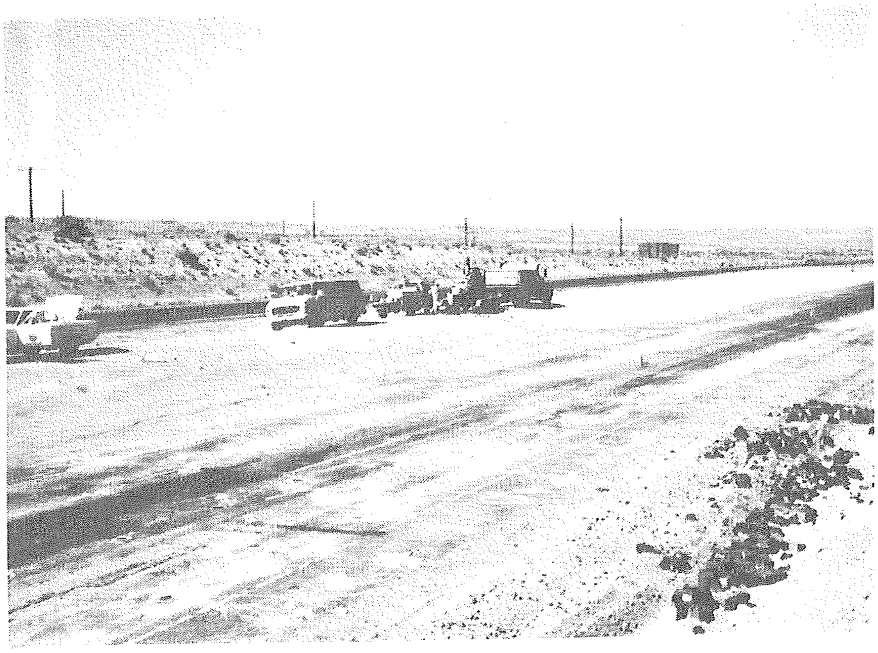

Figure 25. Site view looking southwest along westbound land of I-40.

The total drilling and sleeving operation lasted approximately 2 working weeks of the drill crew's time.

Upon completion of the drilling and sleeving the electrical installation was initiated.

This operation consisted of trenching for horizontal electrodes and drilling for a section of vertical anodes. See figures 26 and 27 for an illustration of this operation.

The electrical connections are shown in figures 28 thru 31.

The manner of mixing and placement of solution was accomplished by District IV personnel using a modified 2500 gal. (9463 litre) capacity "goose neck," water truck. The water and KCL was mixed so that a 0.4 N solution was obtained. See figures 32 and 33 for this operation.

A 0.4 N solution was used in order to introduce the salt solution into the site at a maximum rate. The 0.4 N solution is equivalent to about 300 gms KCL per litre of water.

Given the outside ambient water temperatures in the Holbrook area at that time of year (May thru August about 23^{o}C) this was the maximum amount of KCL soluable in water, at that temperature. To attemp to dissolve more KCL in the water would not have been possible. So with the given ambient water temperatures the optimal KCL concentration possible, in order to deliver the required amount of salt to the site in the shortest time, was about a 0.4 N solution.

After mixing in the tanker truck the 0.4 N KCL solution was placed into the auger holes under pressure using ordinary gasoline nozzel and hose fixtures leading from the solution truck.

Figure 26 shows cathode trenching operation on North side of westbound lane of I-40. The cathode trench was formed on both sides of the westbound lane and each was 480'(146.4 M) long, 4' (1.22 M) deep.

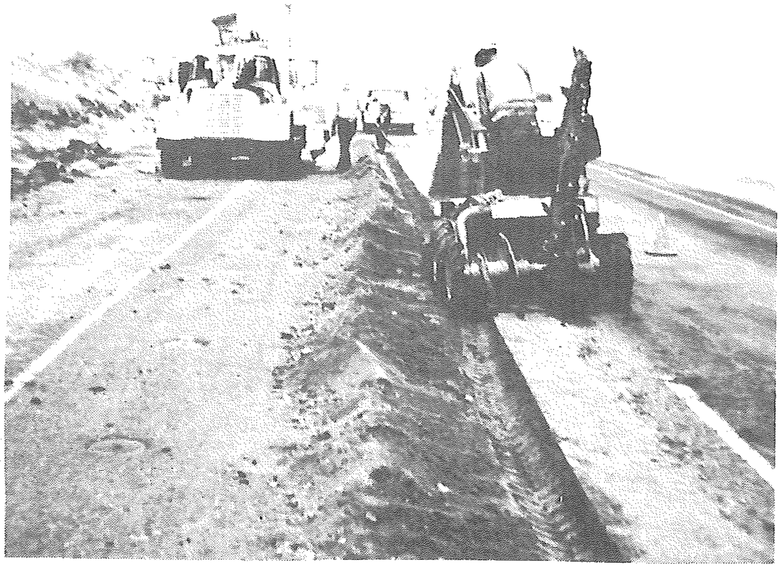

Figure 27 shows preparation of one of the anode trenches. The view is east on I-40.

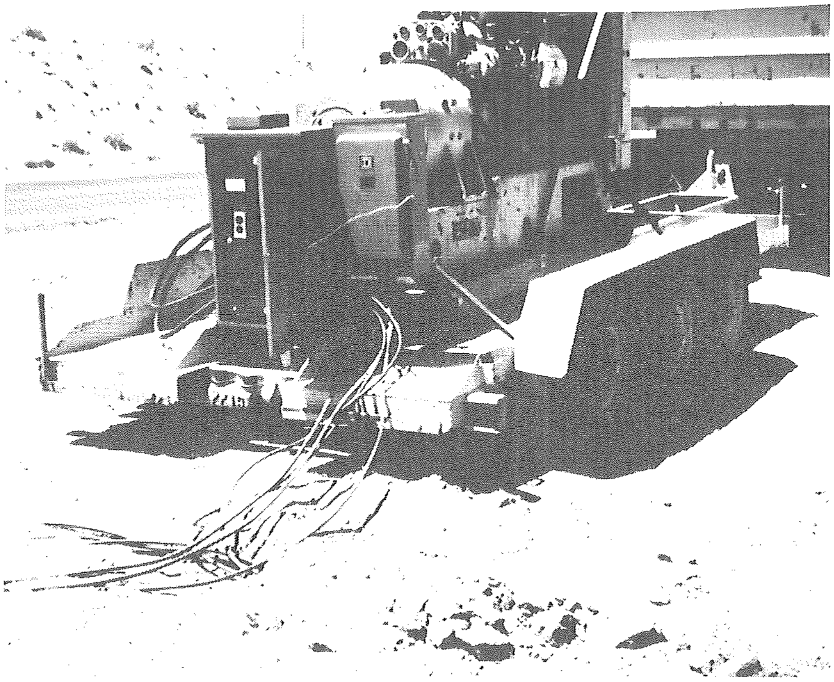

Figure 28 shows 60 KW D.C. generator with #4/0 anode and #2/0 cathode cables connected.

Figure 29. 3/4 view of generator with monitoring-control panel shown open.

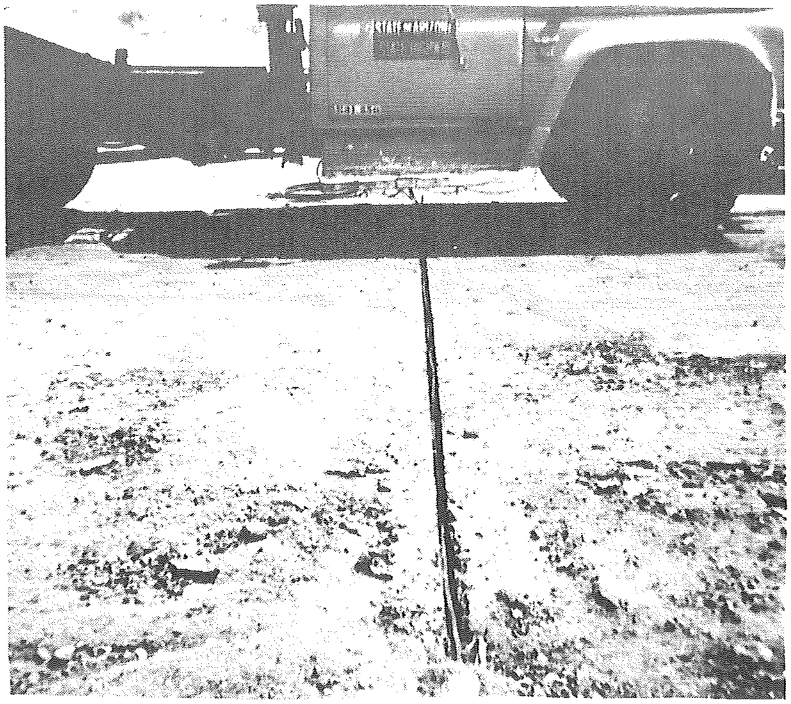

Figure 30 shows cable in transverse cut across roadway. The cable is #2/0 wore connecting the cathodes.

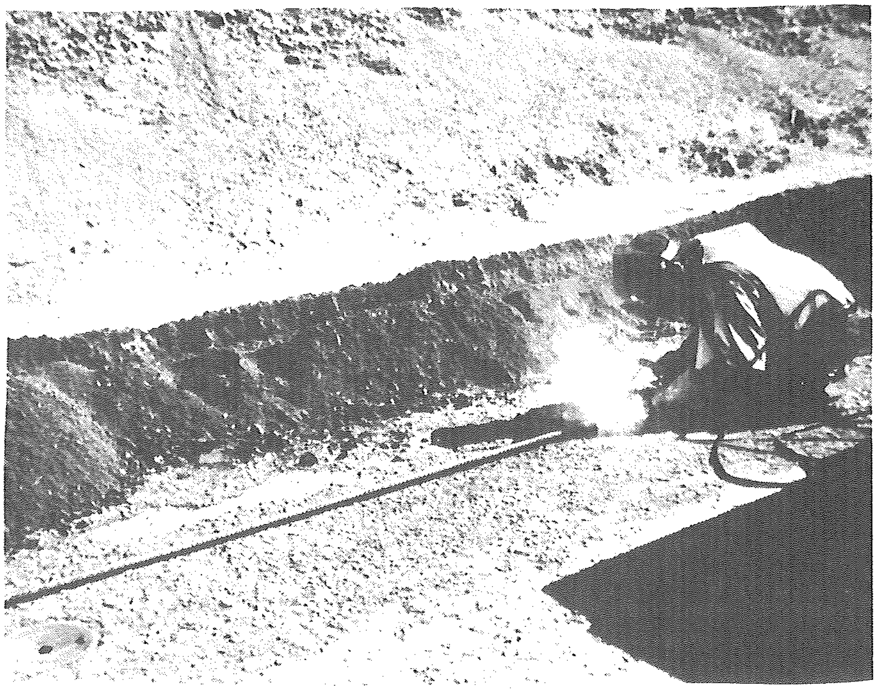

Figure 31 shows cathode welding operation. Each cathode consisted of 24 20' (6.10 M) Sections of 1" (.0254 M) rebar welded together for electrical continuity.

Figure 32 shows 2500 gallon (9463 litre) tanker being filled with water in preparation for the KCL solute.

Figure 33 shows one of the sixty 100 pound (45.4 KG) KCL bags being emptied into the water filled tanker.

In this manner the auger holes were filled twice a day for a period of 0.4 N solution was introduced into the site region. See figures 34 thru 37 for this operation.

It was determined early in the project that prior to the introduction of the electric field the solution would be delivered to the site for about 30 to 35 days. With this procedure the clay could be pre-saturated with the KCL solution by utilizing the relative ease with which an electrolyte moves through the clayey material and simultaneously avoid the expense of running an electrical generator during the initial soil saturation period. During this period about 16,894 gallons of solution was delivered to the site.

On July 8, 1973 the 60 KW D.C. generator was started and an overall current of about 400 amps was recorded with a voltage gradient of about 0.3 volt/cm. On the average about 133 amps flowed through each of the three sections during the field test. After approximately 40 days continuous operation an electrode polarization phenomena was noted which caused a rapid power loss. Electrical operations were discontinued immediately thereafter.

After the electrical system was shut down sufficient soil samples were obtained and shipped to the Materials Services Laboratories for testing and evaluation. See figure 38 for the sampling plan used. This sampling plan was randomized in contrast to the pre-test sampling plan. This was possible since the swelling had been sufficiently characterized by the pre-test sampling, therefore it was then decided that randomizing the post-test sampling would give a much better picture of how the site was affected overall.

Figure 34 shows converted gasoline hoses and nozzels for solution delivery system.

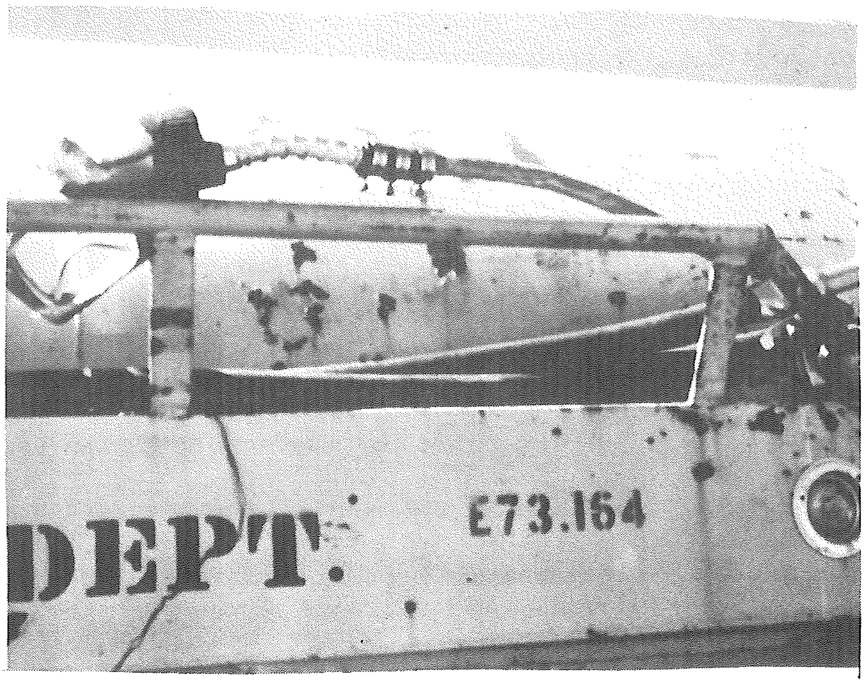

Figure 35. Close up of converted gasoline nozzel.

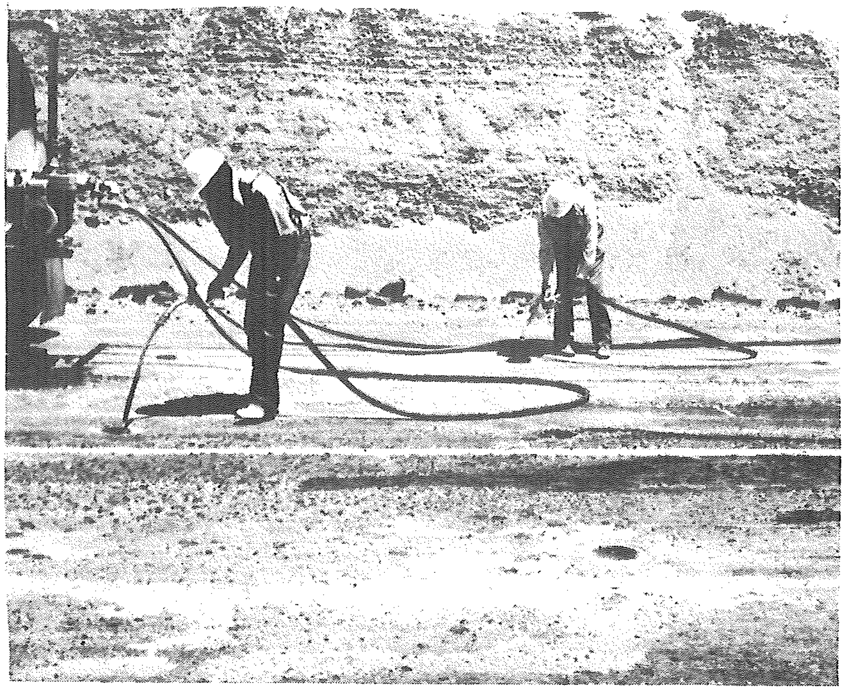

Figure 36 shows 0.4N KCL solution being introduced into the sleeved auger holes.

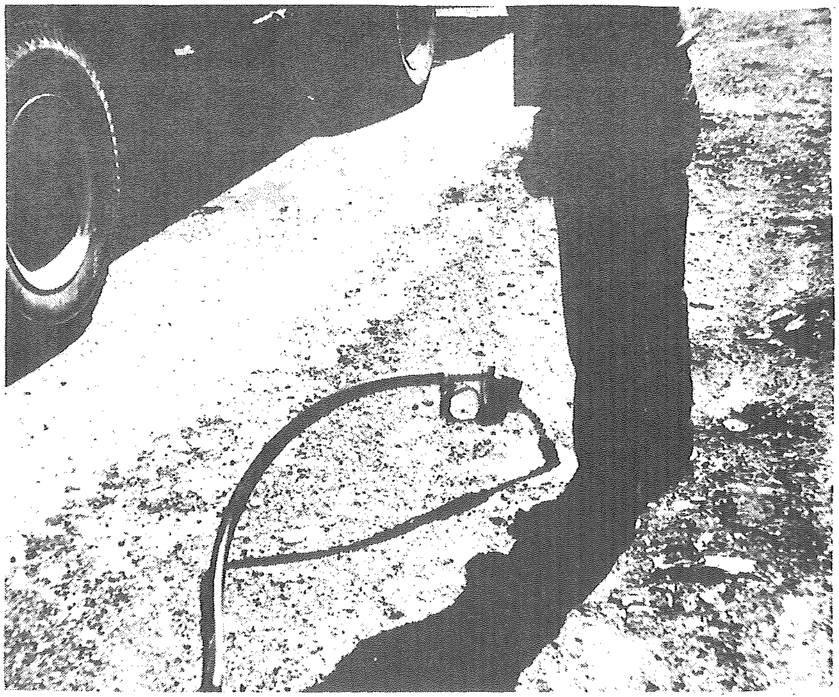

Figure 37. Close up of solution hose and nozzle. Note that the nozzle's orifice is fitted into the 1½" (.0318 M) diameter hole in the 10" (.254 M) diameter plate.

POST-TEST SAMPLING PLAN ON I-40 TEST SITE
WESTBOUND LN
MP 323.8±
CRAZY CREEK

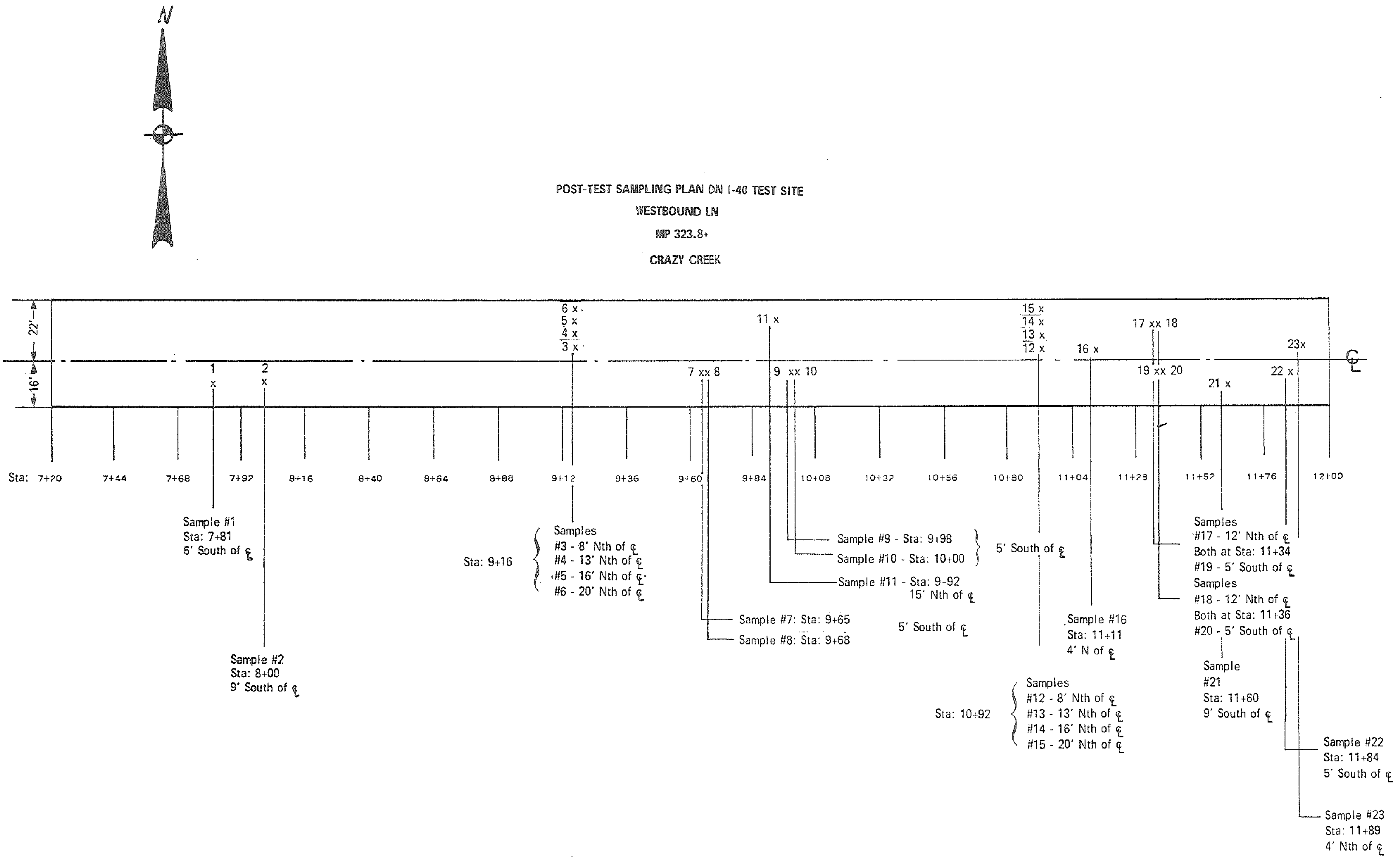

Figure 38

DISCUSSION OF OPERATIONAL CHARACTERISTICS OF ELECTROCHEMICAL SOIL STABILIZATION

The stabilization of soil utilizing a KCL solution is strongly dependent on the ambient water temperature since the water temperature determines the solubility of the KCL solute hence the solution strength, hence the effectiveness of the solution as a stabilizing agent.

Moreover, as Figure 16* hows, the greater the weight of KCL introduced into the soil the greater the reduction in the swell potential. Figure 16 is a representative behavioral curve of swelling chinle clay when subjected to treatment with a KCL solution.

If a swell reduction factor of say 3 for a section of chinle clay is desired, Figure 16 can be used to give an estimate of the amount required for the task at hand. However, it is stressed that the significant variances in the physio-chemical makeup of chinle clay make it imperative to sufficiently sample any chosen site of chinle clay and subject the samples to a laboratory swell test before making a final decision on the amount of KCL required.

If the tests indicate that an amount of KCL greater than than shown in Figure 16 is needed to give a reduction factor of 3 then more solution is required. That is, given the usual water temperatures encountered in northeastern Arizona during the summer (23°C, 73°F) it is not possible to greatly increase the amount of KCL in a 30-35 percent by weight solution without heating the water (see plate 1) which could be an expensive process unless solar heat were used. Hence, the only way to efficiently get more KCL into the soil is to use more solution or to heat the water using solar energy (i.e., painting the water tank black).

During the course of soil treatment about 36,000 gallons (136,000 liters) of 0.4N KCL solution was delivered to the site. The introduction of this large quantity of liquid into the site on I-40 apparently did not have any adverse effects on the carrying capacity of the pavement over a two-month

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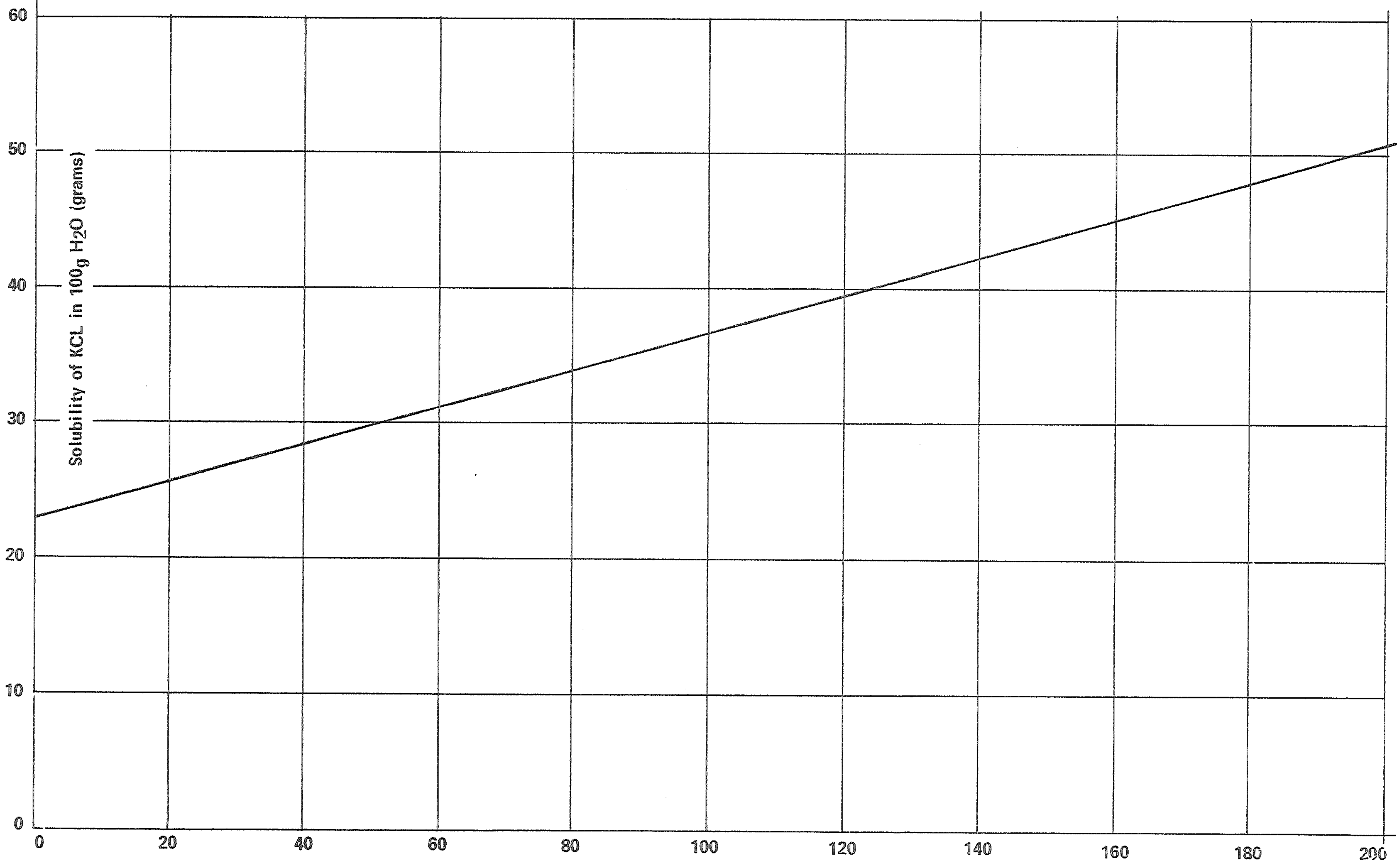
*See Page 45

WATER SOLUBILITY OF KCl.



Temperature (Fahrenheit)

Temperature (Celsius): $t_c = (t_f - 32) / 1.8$

Plate No. 1

period. However, during the last week of the project some rutting was noted at the west end of the test section. Undoubtedly, if the solution delivery had continued at the avg. rate of about 600 gallons per day for a significantly longer period of time, the carrying capacity of the pavement would have been impaired. This factor does add a constraint to the KCL + water treatment of soil problems beneath a roadway surface. However, most roadway sections of swelling chinle clay encountered during the ten years that aqeuous KCL solution plus electro-osmosis was employed by ADOT did not require an amount of solution, for satisfactory treatment, to disrupt the carrying capacity of the roadway. Because of this past-performance record it can be said that most sites located beneath a roadway surface can be treated by this method without significantly impairing the carrying capacity of the pavement.

During the actual field operation using electrochemical methods several components of the system must have a high reliability for the treatment to be satisfactory. They are: (1) the electrical circuitry, (2) the D.C. generator and (3) the solution delivery system. If (1) or (2) were to fail, the treatment would not cease, however, its efficiency would be significantly curtailed due to the lack of the electrochemical action. If (3), the solution delivery system, were to fail some soil treatment could still be carried out by maintaining an electrical current through the soil mass, however, after a 24-hour to 48-hour period it would be very inefficient since the soil would begin to dry out with a corresponding increase in soil resistivity.

The particular circuitry pattern used on this project is a very simple one which is almost fool-proof to install and relatively efficient. Care must be taken, however, in checking the rebar welds and cable-rebar connections for electrical integrity. These are the only possibile trouble

trouble points for the circuit set-up. Since these particular operations are relatively easy for one skilled maintenance man to perform, this portion of the operation is not difficult. Moreover, since the circuit is in use for only a matter of a few weeks corrosion of the electrical connections will be little or no problem. The only major electrical problem encountered during this test was caused by inclement weather. That is during a period of heavy rains, while the electrical components were in operation, the resistivity of the soil would necessarily be significantly lowered with a corresponding surge in current of 50 percent to 100 percent. This, of course, will destroy the safety fuses which were installed in the circuit to protect the generator. Of course, when this occurs the electrical operation is brought to a halt until new fuses are installed, but solution delivery may continue.

Generally speaking, barring a sudden cold front, heavy rains are the only form of adverse weather conditions which can bring an electrochemical operation to a halt. This is because of the resulting changes in soil conductivity which cause overly large current surges that can overload the circuitry and even damage the generator unless it is protected by the proper safety fuses (200-amp fuses in this case).

The power source used, a 60-KW D.C. generator, functioned without mishap during the approximately 40 days of continuous 24-hour operation, disregarding a several hour shut down during heavy rains because of blown electrical 200-amp safety fuses. One major safety device used on the generator was a r.p.m. regulator for its engine. The r.p.m. regulator prevented a "run-away". That is, it prevents the generator's r.p.m. from increasing too far above the safe operational rate of 1200 r.p.m. If a run-away were to occur, the generator's engine would literally shake itself to pieces. Also, a fuel cut-off was installed as a back-up safety device for this purpose.

If the generator were to fail for any other reason, i.e., a broken rod, piston or fuel exhaustion, solution delivery could still be continued with a loss of treatment efficiency due to the loss of the electrochemical action in the soil.

After completion of the electrochemical treatment, assuming that the results were proven to be satisfactory via laboratory soil tests, the only remaining task is to remove the auger hole casings and fill in the holes.

It must be pointed out, however, that the procedure as utilized on this project, i.e., employment of auger holes as solution wells does cause unacceptable damage to the roadway surface. Hence, it must be stressed that this method of soil treatment should only be used over a section of roadway that is scheduled for a pavement overlay.

Labor requirements, after such a treatment and after the scheduled overlay, are non-existent except for a yearly elevation run. Since the site treated on I-40 was shortly thereafter overlayed and, moreover, at the time of this report writing only a few months have passed since the overlay no meaningful elevation data can be included in this report.

The long term effects of this kind of treatment have been examined in the laboratory by using samples taken from a small site treated in 1968. Through the use of electron micrographs and electron diffraction these samples were shown to be structurally identical to those from the recently treated site on I-40 (see figures 46 and 51a). Moreover, District IV personnel are watching for any signs of heaving of the site on I-40. To date none have been reported.

In comparison to other kinds of treatments such as the one used by Ion-Tech it appears that the electrochemical treatment, while more expensive, seems to be much more effective in reducing swell. Clearly, if one were to simply overlay a swelling site without any kind of treatment, this would be akin to placing a bandage on a festering wound because as soon as the soil moisture equilibrium is disturbed the clay would swell and another overlay would be needed. However, as of late, membrane encapsulation of the roadway subgrade to stabilize the subgrade soil moisture content coupled with an overlay could provide a viable solution to the swelling clay problem. This method, if proven successful, would undoubtedly be the preferred solution since it would be much less expensive if done over a long stretch of roadway than the electrochemical treatment would be.

Preliminary Conclusions:

At this time it is not possible to use documented survey measurements to determine, with assurance, if soil expansion at the site has been arrested because of the short span of time since the treatment was concluded. However, it is certainly possible to state that the effects of the electrochemical treatment will have a life span at least as long as ten years as evidence by the electron microscopic work done with ten year old treated chinle clay samples.

In addition, the findings of this project suggest that electro-osmotic soil stabilization, using a 0.4N KCL solution can be applied efficiently only on a highly localized mass of swelling clay, i.e., electro-osmosis is apparently not efficient when applied to large masses of swelling clays because of the inherent and hence unavoidable difficulty in uniformly indurating the clay mass with the electrochemical treatment. The project findings also indicated that the solution-well-net used was too coarse, i.e., the 8' (2.4M) centers forced each well to inundate soil up to 4' (1.2M) from the well's ℄. A finer net would have been more efficient; however, the cost of installing a solution-well-net based on say 5' (1.5M) centers would have been nearly double the one based on 8' (2.4M) centers. Moreover, because of the wide variation in the behavior of the solution flow through Chinle Clay a relaible design figure for the solution-well-net can only be obtained after appropriate laboratory tests; hence, this parameter will, therefore, be tailored to each particular region one is working in and could conceivable vary from 4' (1.2M) to 8' (2.4M).

As to the physical effects on the clay structure induced by electro-osmosis one fact is a certainty: no measureable diagenesis of the chinle clay occurs, that is, no measurable amount of chinle clay goes over into an Illitic Clay. The effects apparently are confined to altering the stacking configurations of the unit cells making up the clay particles as shown in Figure 44 III (i.e., reorienting the unit cells). This conclusion was arrived at by the following rationale: While O'Bannon had shown in previous work that the Montmorillonite's x-ray intensity was reduced after E.O. treatment it was noted by Dr. Cowley of ASU that the elctron micrographs showed the stacking number of the clay to be markedly reduced.

It is well documented in x-ray diffraction theory that the intensity of any peak is proportional to the square of the number of planes contributing to this intensity peak. This fact is illustrated in Figure 44 II. Since Cowley's electron micrographs indicated that the number of these planes was reduced because of the disruption of the stacking geometry it appears that the reduction in the x-ray diffraction peak follows because of this reorientation of the unit cells and not because of any diagenetic action on the clay.

The underlying fact for recommending that E.O. operations be confined solely to small regions* is that the treatment is fundamentally not uniform in its effects, which is the causative factor for requiring very fine solution-well nets; i.e., very closely spaced solution-well holes.

Moreover, because of this, the cost of E.O. operations is not small.

* A roadway section 40' x 200' (12.2M X 61M)

Based on our operations it was calculated that the cost of stabilizing one cubic yard is approximately $9 or $11.80 per M^3. Also, it should be noted that the swell pressure was reduced by only 50 percent and the percent swell by 36 percent. To get better reduction, with solution-well-net used, would have raised the cost much higher because more solution would have been required.

Weather conditions also play a role in determining when to use or not use this electrochemical method of soil stabilization. Primarily, it should only be utilized in mild or warm weather and should never be attempted when the ambient water temperatures fall below 10°C (50°F) otherwise the resulting KCL - water solution strength would be relatively weak and, hence, inefficient to use. Moreover, during heavy rains problems may arise from current surges due to rain - increased soil conductance. Because of the very low voltage gradients used during work of this kind, 0.3v/cm, the danger to personnel is minimal unless, of course, the bare "hot" line from the generator was touched while standing in very wet soil. However, since this electrode is well insulated this kind of mishap is not going to occur by chance. In fact, during this project largely unskilled maintenance personnel were able to fuel the generator and start it, even during some very heavy rains, without any problems.

In general, the main problem encountered with the electrical system was overloaded electrical fuses (200 amps) caused by heavy rains. These problems amounted to only an annoyance since as soon as the solution delivery was made or the generator was refueled the fuses would be replaced. This would give a maximum possible shutdown period of about 12 hours, more likely much less.

Also during the time of solution in-put to the site (approximately 36,000 gallons) elevation checks were made over the site which showed that the pavement surface remained largely stable during this period with the exception of minor ruttings less than 1/2" deep, at the west end of the section which extended for about 5' in the right wheel path.

Because of these considerations it follows that while E.O. does moderately reduce the swell-potential of chinle clay for best efficiency of operation it should only be employed on localized masses** of swelling clay where installations of membranes may be impractical [e.g., a clay section 10 yd.(9.14M) wide x 100 yd.(91.4M) long and 1 yd. (.914M) deep].

**Not more than 1000 yard^3 (765 M^3) of soil at one time, since this will allow usage of a finer solution well net e.g. 4' centers (1.2M); hence, more efficient inundation of the soil by the solution and consequently more uniform induration of the soil.

CHAPTER VI

RESULTS AND ANALYSIS OF FIELD TEST

Introduction

After completion of the field test operations on August 16, 1974 soil samples were gathered for laboratory analysis.

The laboratory analysis were broken down into three parts:

(1) X-ray diffraction analysis

(2) Transmission Electron Microscope Techniques

(3) Expansive pressure and % swell tests

Data obtained from (1), will yield information as to the effects on the lattice structure of the clayey soil. Part (2) compliments part (1) and will determine any effects on the soil structure arising from the electrochemical treatment that are too gross (i.e., Macromicro effects) upon the crystalline structure, for the X-ray diffraction to resolve.

The information obtained from (3) will give data as to the effects of the electrochemical treatment on the engineering properties of the soil mass.

X-Ray Diffraction Analysis

The identification of the crystalline soil minerals has been performed by X-ray diffraction analysis. Both the clay and non-clay minerals which are crystalline have a long range order in their atoms or ions. By bombarding a mount containing the sample with high energy X-rays, the spacing of the crystalline structure can be determined from the wavelength of the X-ray and the angle of inclination of the X-ray path to the sample using Bragg's law. The various crystalline clay and non-clay minerals are then identified from their various characteristic spacings.

Two types of sample mounts are used in order to identify the non-clay and clay minerals. A powder mount which is a randomly oriented finely ground portion of the sample pressed into a plastic mount is used to identify the non-clay minerals.

A treatment by which an oriented mount is glycolated will cause the swelling components, smectite and vermiculite, to increase their spacing. A 24-hour 300°C or 525°C heat treatment of a dry oriented mount is useful to identify various clay minerals from the characteristic spacings which result from the different water layer thicknesses at different temperatures and by the collapse of the structure of certain minerals at these temperatures. The glycolated and heat treated specimens are not run past 15°, 2θ, since at that point the spacing is approximately 6 Å which is considerably less than the predominant characteristic spacings of the clay minerals.

The <u>relative amounts</u> of the various clay and non-clay minerals are identified by the headings <u>major</u>, <u>minor</u>, or <u>trace</u>. All principal peaks are identified by letter abbreviations which correspond to the minerals given in the key on the following page. The random-mixed-layer clay minerals are composed predominatly of smectite (montmorillonite).

A rough idea of the <u>relative amounts</u> of the various minerals is obtained from a comparison of the <u>relative intensities</u> of the characteristic peaks.

Usually the non-clay mineral fraction is more abundant in accordance with the majority of sample having a grain size larger than 2μ, the start of the clay mineral size range. The clay mineral component is usually more important than the non-clay mineral component since clay minerals are orders of magnitude more reactive the non-clay minerals due to their large specific surface area.

A interpretation of the results so as to predict the effects on the clay resulting from the electrochemical treatment can be had by noting the intensity of the diffracted radiation relative to the incident radiation. It is well known that the relative intensities is a function of the physical constant of the X-ray apparatus, the type of atoms comprising the diffraction planes and the relative number of planes oriented so that the diffracted radiation from these planes reinforce the observed intensity pattern.

It is to be noted that in all of the pre-test samples, from the I-40 site, checked by X-ray diffraction the presence of Smectite or Montmorillonides ranges from one sample with minor amounts to 3 samples with major amounts.

However, in the post-test samples checked, only 2 out of 7 readings show minor amounts of Smectite, the other 5 showing only a trace. A significant example of this given in figures 39 and 40 where the post test sample shows only a trace of montmorillonoids present.

This work is comparable with previous X-ray work performed on electrochemically treated Montmorillonite-Kaolin samples as shown in figures 41 thru 44.

Table 3 gives, in tabular form, the results of the X-ray data.

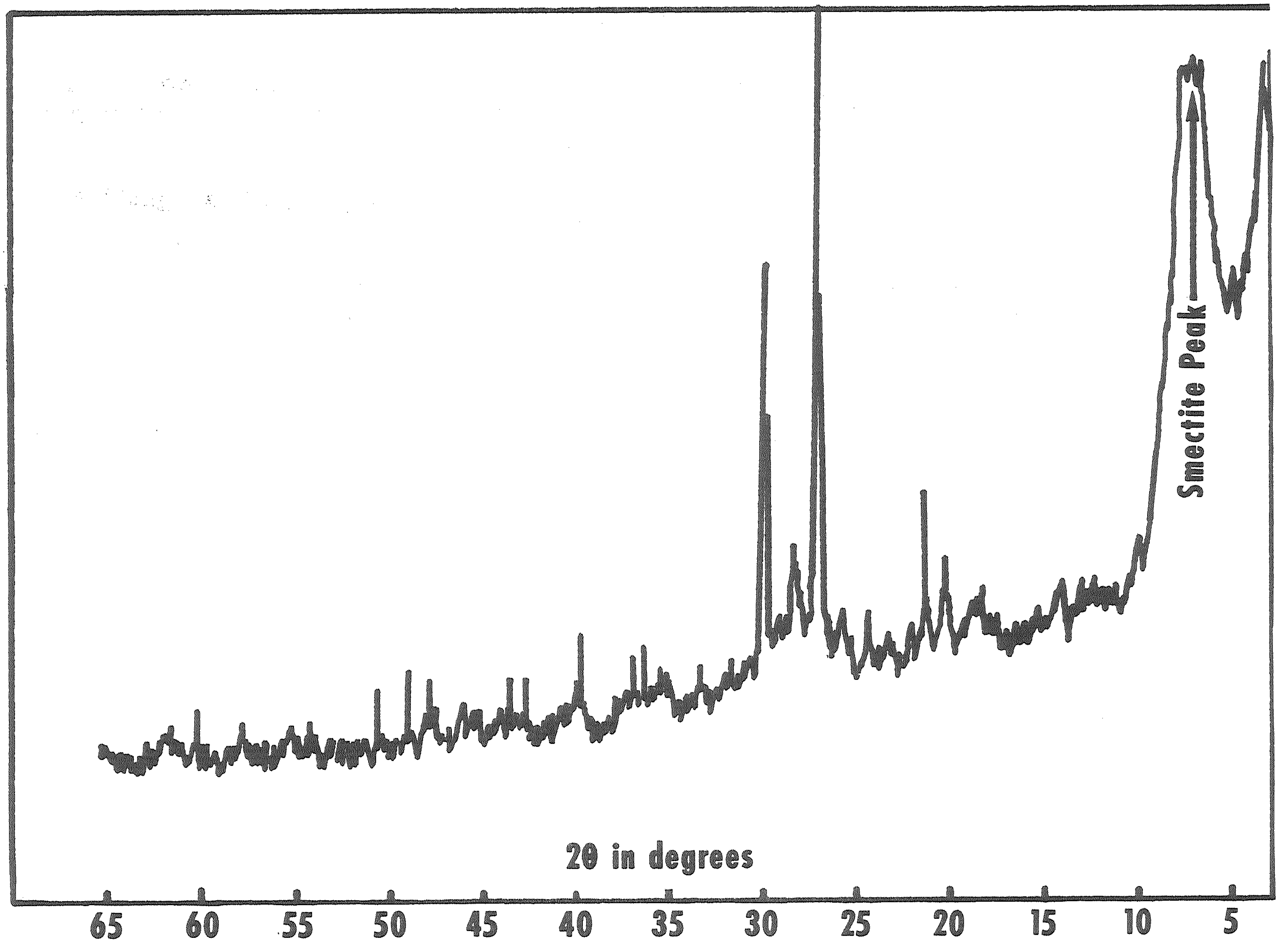


FIG. 39

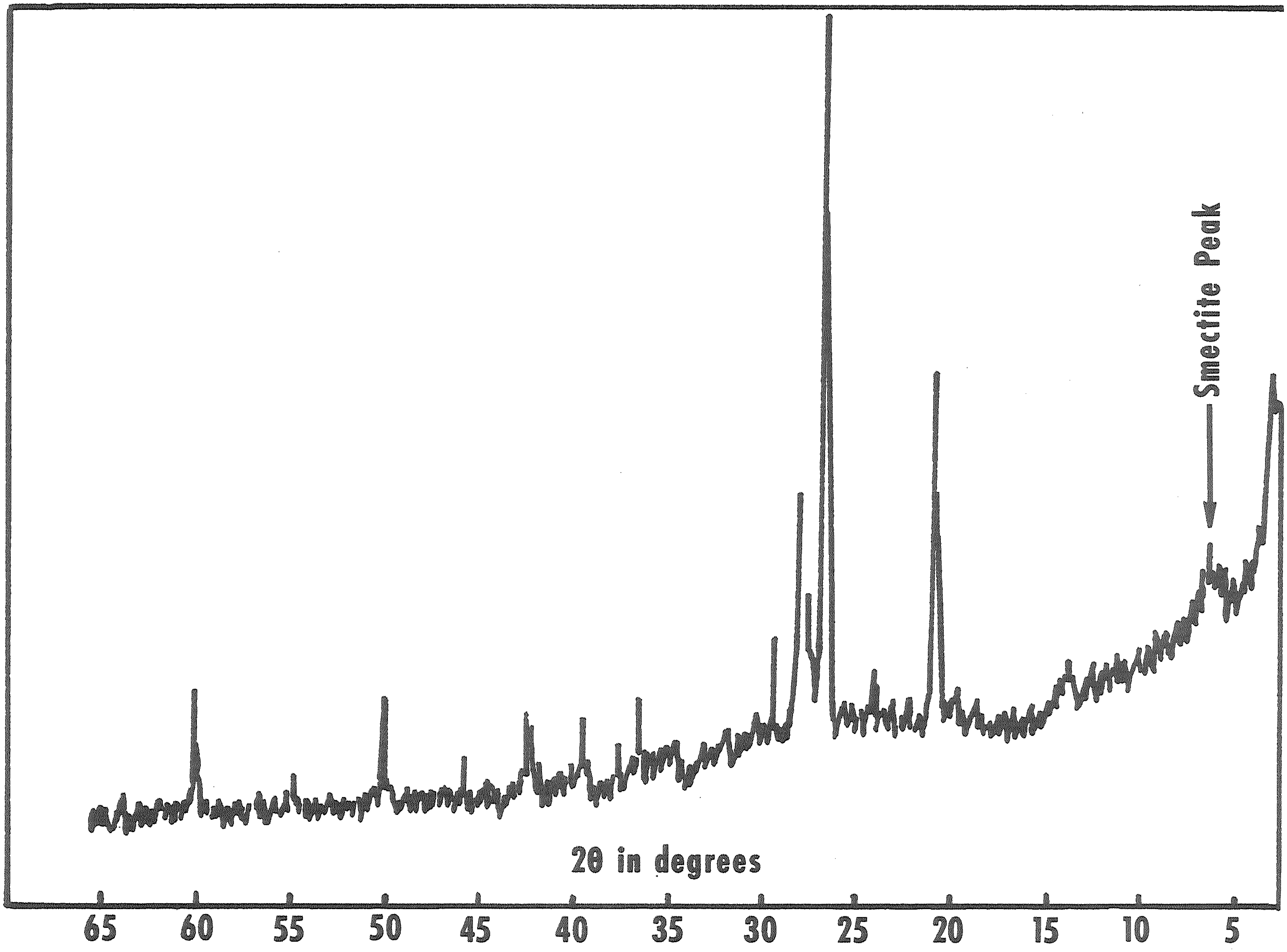
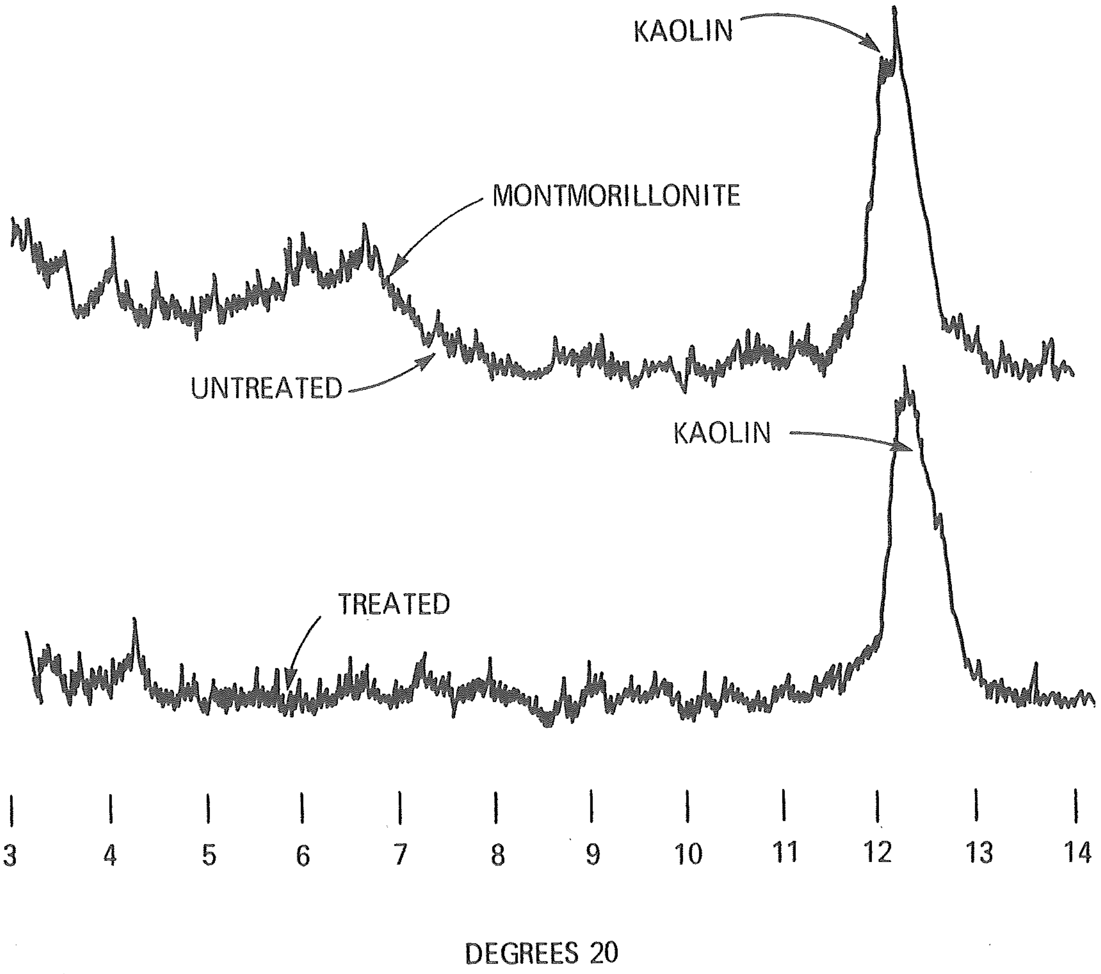


FIG. 40

UNTREATED
BLANK NO. 1 – 10% MONTMORILLONITE – 90% KAOLIN

TREATED
SAMPLE S-9
10% MONTMORILLONITE – 90% KAOLIN
80 LBS./FT.^{3}
3 DAY TREATMENT
80% SATURATION

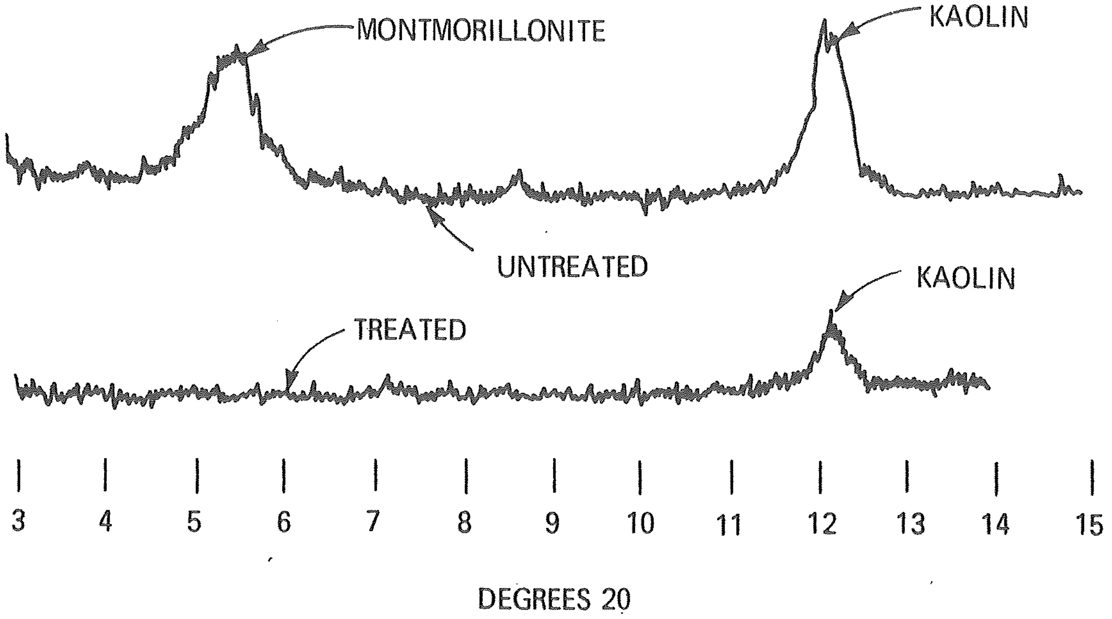


X-Ray Diffraction on Treated and Untreated Samples

Figure 41

UNTREATED
BLANK NO. 2 – 20% MONTMORILLONITE – 80% KAOLIN

TREATED
SAMPLE S-31
20% MONTMORILLONITE – 80% KAOLIN
80 LBS./ FT^3
4 DAY TREATMENT
80% SATURATION

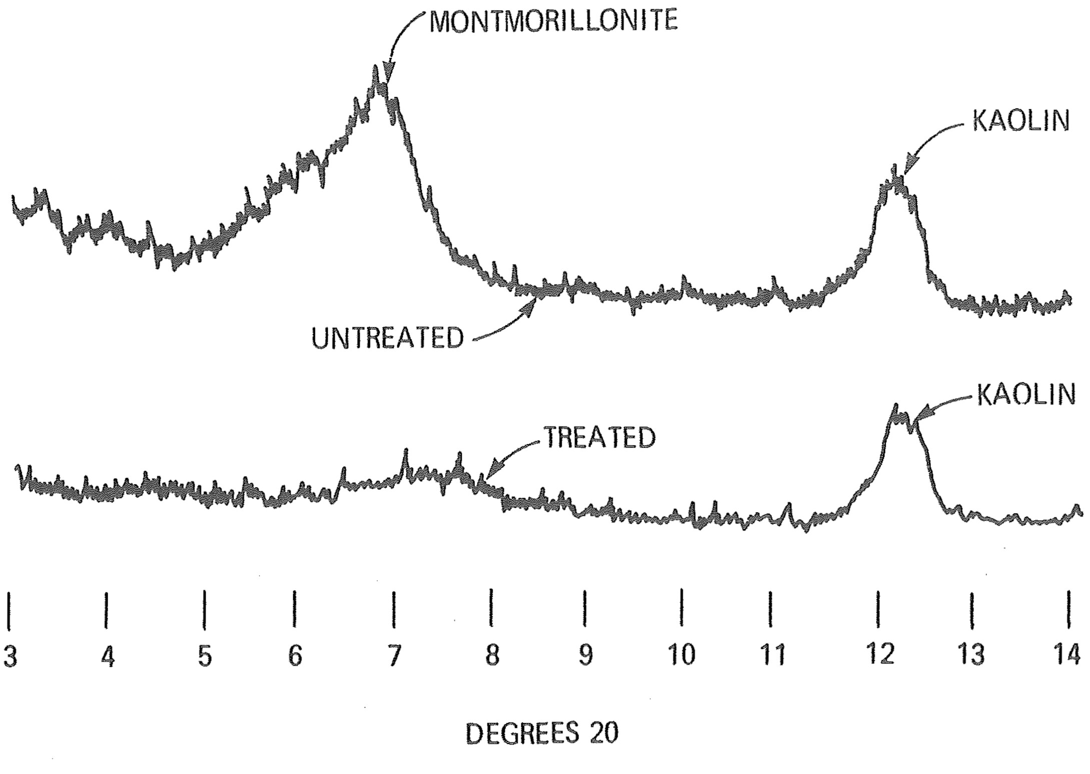


X-Ray Diffraction on Treated and Untreated Samples

Figure 42

UNTREATED
BLANK NO. 3 – 30% MONTMORILLONITE – 70% KAOLIN

TREATED
SAMPLE S-39
30% MONTMORILLONITE – 70% KAOLIN
80 LBS./FT^3
1 DAY TREATMENT
80% SATURATION

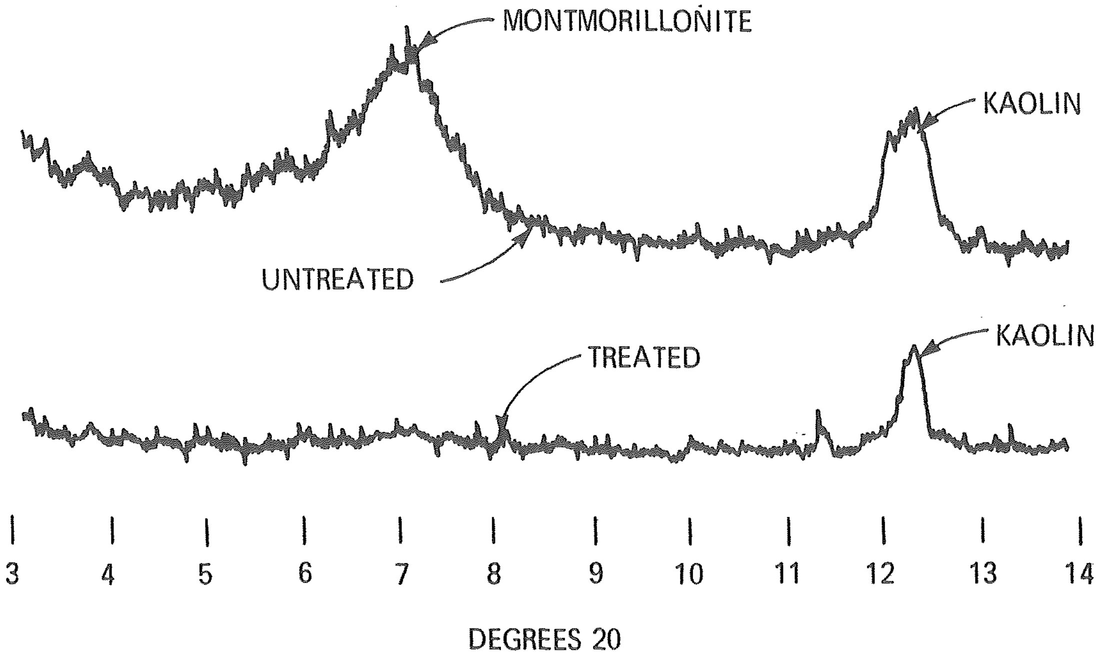


X-Ray Diffraction on Treated and Untreated Samples

Figure 43

UNTREATED
BLANK NO. 4 - 40% MONTMORILLONITE - 60% KAOLIN

TREATED
SAMPLE S-54R
40% MONTMORILLONITE - 60% KAOLIN
80LBS./FT^3
2 DAY TREATMENT
80% SATURATION



X-Ray Diffraction on Treated and Untreated Samples

Figure 44

TABLE 3

X-ray Diffraction Data

X-ray Sample	I-40 Sample Site Iden.	Relative Amounts of Constituents (Major, Minor, Trace)								
		Quartz	Plagioclase	Smectite	Calcite	Chlorite	Dolomite	Illite Mica	Kaolinite	Cristobalite
#1 Pre-Test	Auger Hole #12 0'-3'	Major	Minor	Minor	Trace	Trace				
#2 Pre-Test	Auger Hole #16 2'-3'	Major	Major	Major		Trace	Trace	Trace		
#3 Pre-Test	Auger Hole #21 0'-3'	Major	Minor	Major	Major					
#4 Pre-Test	Auger Hole #26 0'-3 1/2'	Major	Minor	Major	Minor	Trace				Major
#5 Post Test	Auger Hole # 3 0'-3'	Major	Minor	Trace	Trace		Trace			
#6 Post Test	Auger Hole # 4 0'-3'	Major	Minor	Trace	Trace		Trace			
#7 Post Test	Auger Hole #11 0'-3'	Major	Minor	Minor						
#8 Post Test	Auger Hole #12 0'-3'	Major	Minor		Major	Trace		Trace		
#9 Post Test	Auger Hole # 3 0'-3'	Major	Minor	Minor	Minor					
#10 Post Test	Auger Hole #16 0'-3'	Major	Trace	Minor	Major					
#11 Post Test	Auger Hole #23 0'-3'	Major	Minor	Trace	Trace					
#12 Not from Test Site	Old E.O. Site at Holbrook 0'-3'	Quartz	Minor	Minor	Trace				Trace	

CONCLUSION FROM X-RAY DIFFRACTION DATA

Clearly to attempt a rigorous crystallographic analysis of the treated and untreated material from this test would be beyond the scope, time, and budgetary limintations of the project. However, some definite inferences can be drawn from this data.

The X-ray data clearly indicates that the clay has been affected by the electro-chemical treatment, moreover, this data is supported by electron micrographs and tests on the engineering properties of the oil. Based on both electron micrograph and X-ray diffraction data, it appears that the stacking geometry of the clay particles has been modified by the electro-chemical treatment.

Each clay particle consists of from one to hundreds of stacked basic units as shown in figure 44II. The number of basic units comprising the particle will be called the stacking number. It is also depicted in figure 44II how the X-rays, from a series of equally spaced parallel planes of atoms, will add together to produce detectable X-ray diffraction patterns. Moreover, it should be noted that the maxima of intensity of the X-ray pattern will be roughly proportioned to the square of the number of parallel planes contributing to the particular diffraction maxima under observation. Now the electron micrograph data indicated that the stacking number of basic units has decreased and this decrease would contribute to the reduction in the intensity peak. A schematic of this stacking number decrease is shown in figure 44III.

GEOMETRIC CONDITIONS FOR SCATTERING OF X-RAYS

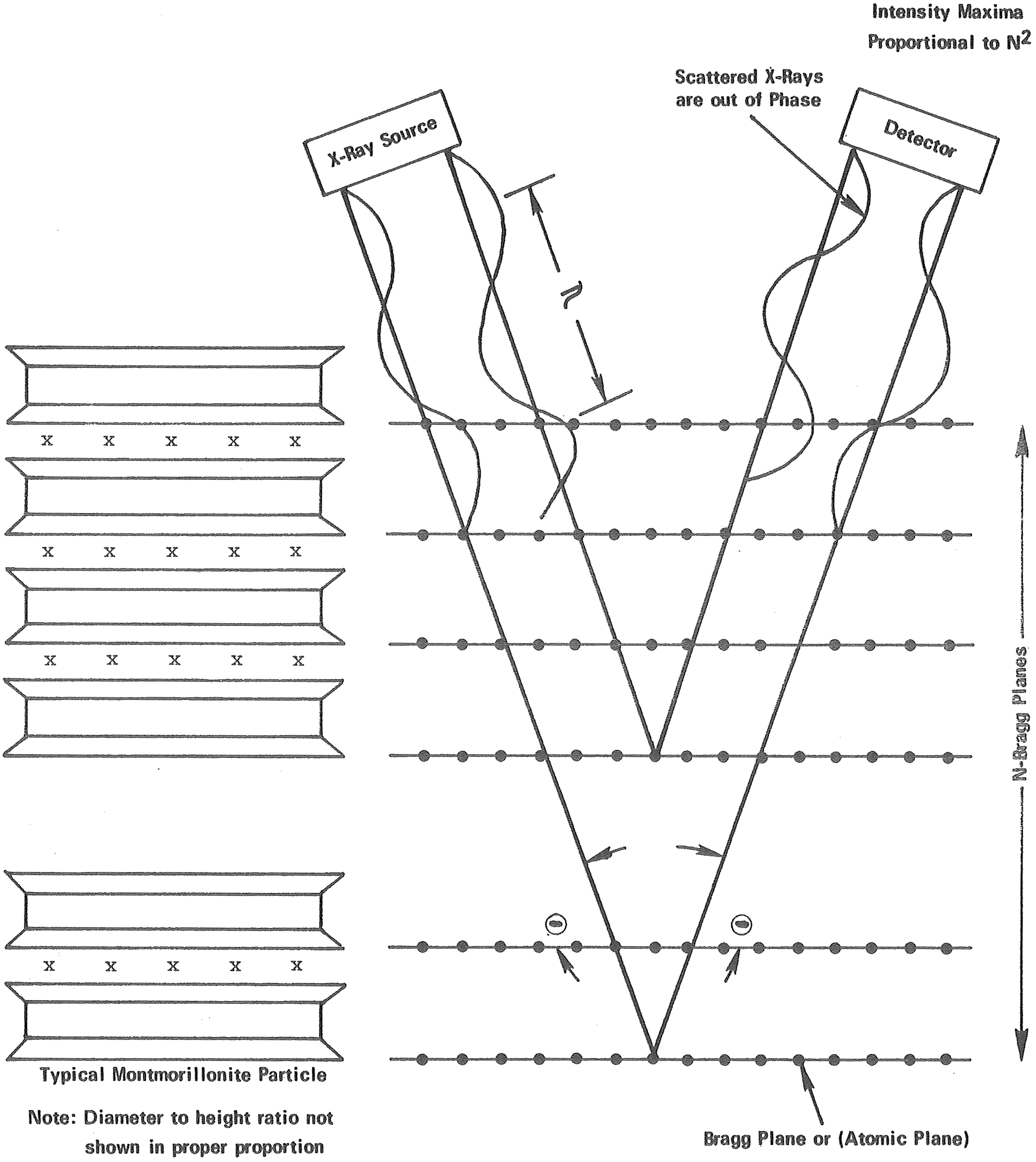


Fig 44-II

SCHEMATIC
OF
STACKING GEOMETRY
FOR
CHINLE CLAY PARTICLE BEFORE & AFTER
ELECTRO CHEMICAL TREATMENT

(NOTE: DIAMETER TO HEIGHT RATIO IS NOT SHOWN IN PROPER PROPORTION)

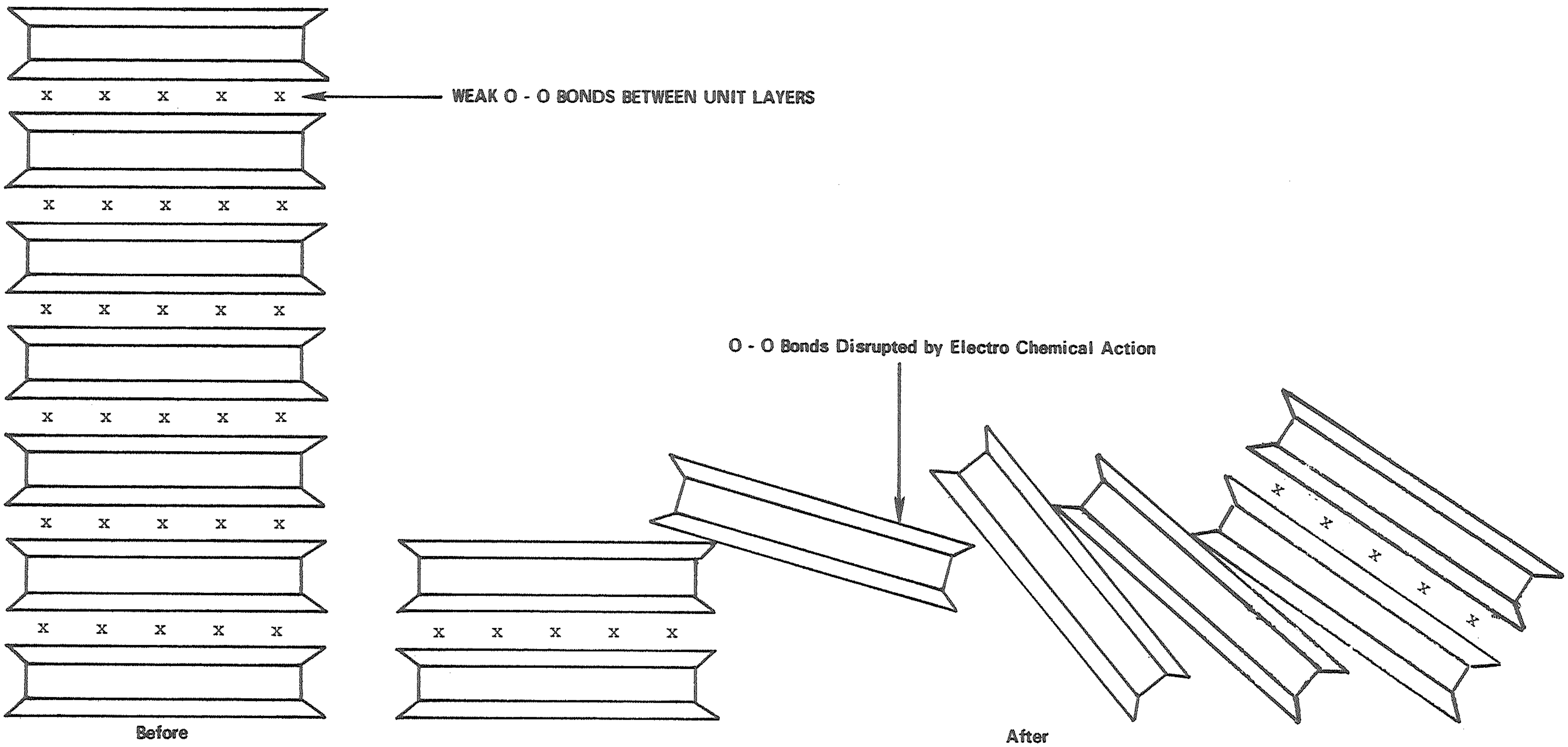


Fig. 44-III

Since no additional minerals were found in the treated materials as opposed to the untreated, it is not possible to conclude that the smectites were transformed into an illitic-like clay. Thus, the fact of reduction in the diffracted intensity of the treated clay coupled with the reduction in the stacking number, as shown by the electron micrograph, indicates that the treatment affected primarily the 0 - 0 bonds between the basic units but did not cause significant diagenisis of the clay. Moreover, since the engineering properties of the clay are highly dependent on the nature and stability of the interlayer environment between the unit cells, it is aparant, from the soil tests, that the electrochemical treatment of chinle clay also alters the interlayer complex, which is composed of inorganic ions, water, and even organic complexes in addition to interferring with the basic 0 - 0 bounding between the unit cells.

Transmission Electron Microscope Analysis

Bentonite-type materials were observed by using Transmission Electron Microscope techniques. Standard Bentonite minerals (Wyoming Bentonite and USP Bentonite) were compared to pre-test clay samples (40133 and 40188) and post-test clay samples (40415A).

A sample which has been treated with potassium chloride several years prior to the treatment of 40415A was also observed to see what the long range effects of treatment were.

All specimens were initially suspended in acetone, sonicated in an ultrasonic cleaner to break up large clumps and then dispersed onto electron microscope prids which had thin carbon film supports to hold the sample.

The acetone extracted an organic substance from the samples which showed up on the micrographs as a dark amorphous spotty substance.

The first samples were discarded because of the extracted organic material and new samples were suspended in distilled water rather than acetone. The same procedure for preparing the specimen was carried out as outlined above.

The preliminary results showed that the pre-test materials (40133 and 40188) were not as well dispersed as was the post-test sample (40415A).

The Electron Microscope Photographs for the post-test materials show the thin crystal layers to be dispersed better than for the pre-test samples for which the unit cells tended to remain stacked into thicker crystals.

The diffraction patterns for the post-test materials, figure 45 and 45, shows diffraction sponts on the circles where the pre-test diffraction pattern, figures 47 and 48, shows that some of the diffraction spots are displaced from the circles. A general interpretation of this is: When diffraction spots are displaced from the diffraction rings, this indicates that the unit cells are stacked up on top of each other to form well ordered three-dimensional crystals.

Another sample run was made to see if the preliminary results could be duplicated. This time exact amounts of sample were placed in 6 ml of distilled water for each specimen. In each case 0.5 gms of sample were suspended in 6 ml of distilled water. The water solution was sonicated in an ultrasonic cleaner for three minutes.

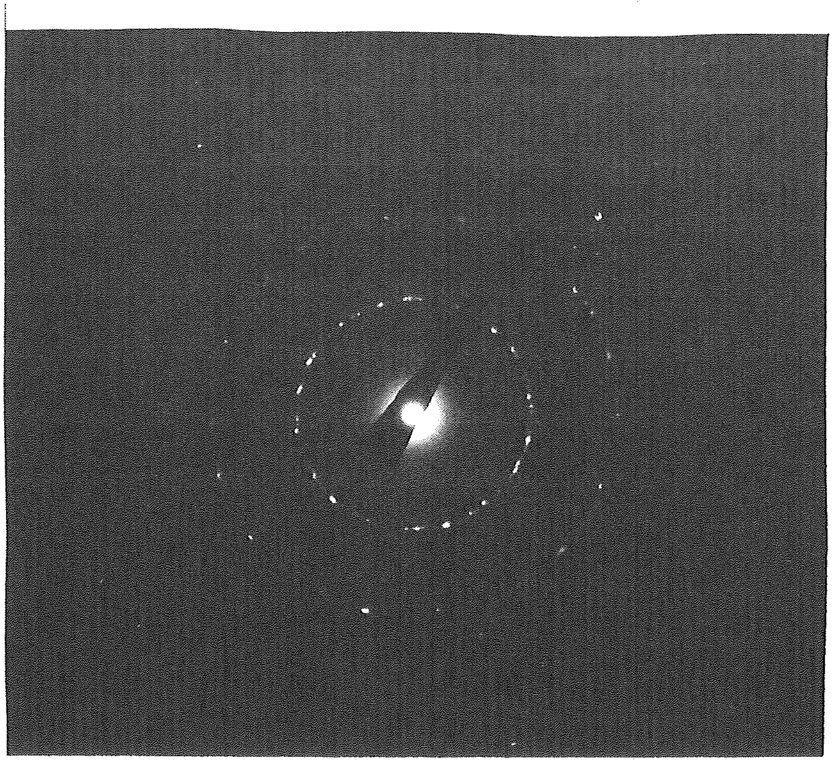


Figure 45
Electron diffraction from treated test site
Chinle clay

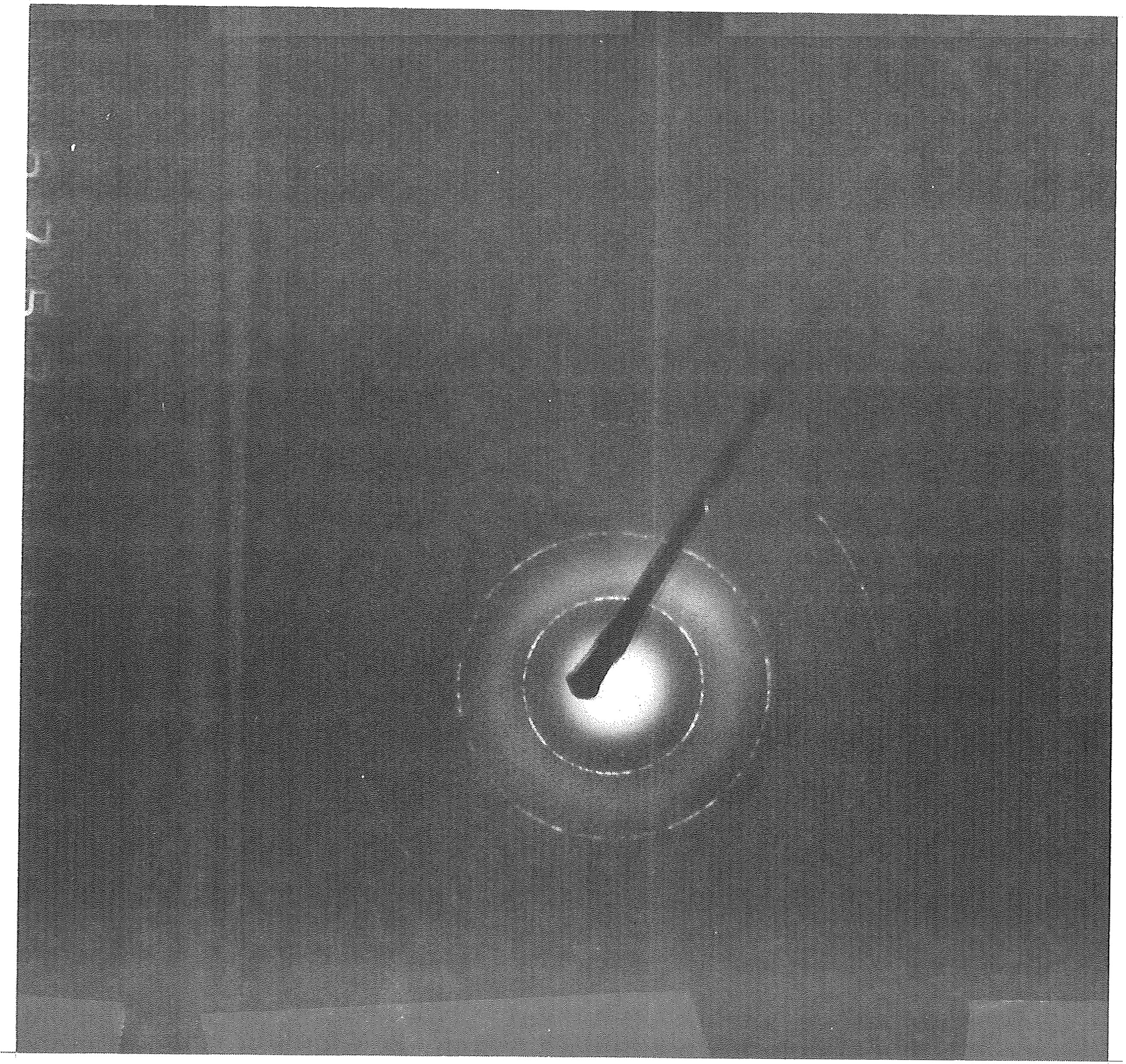


Figure 46
Electron diffraction from treated test site
Chinle clay

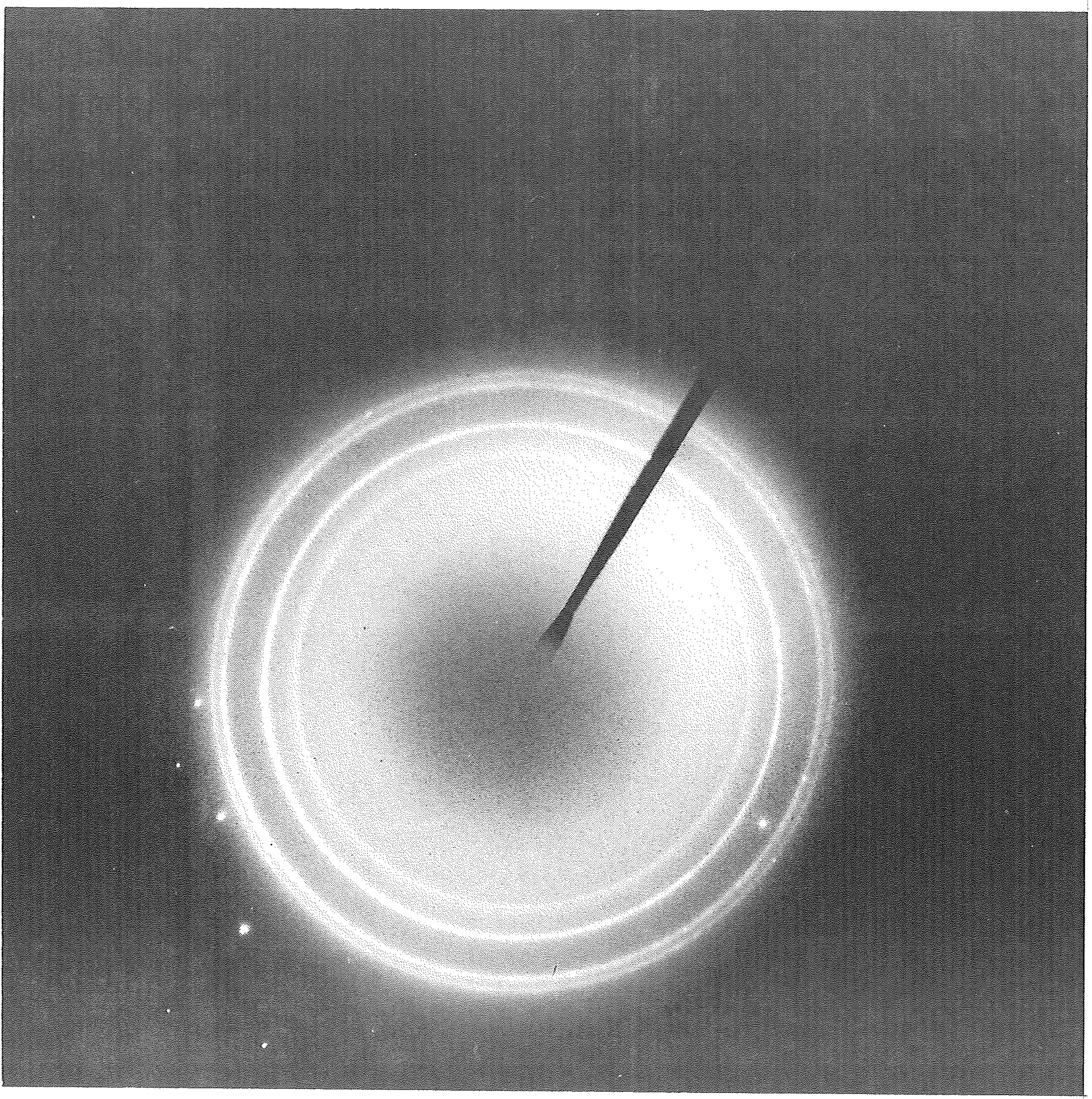


Figure 47
Electron diffraction from untreated test site
Chinle clay

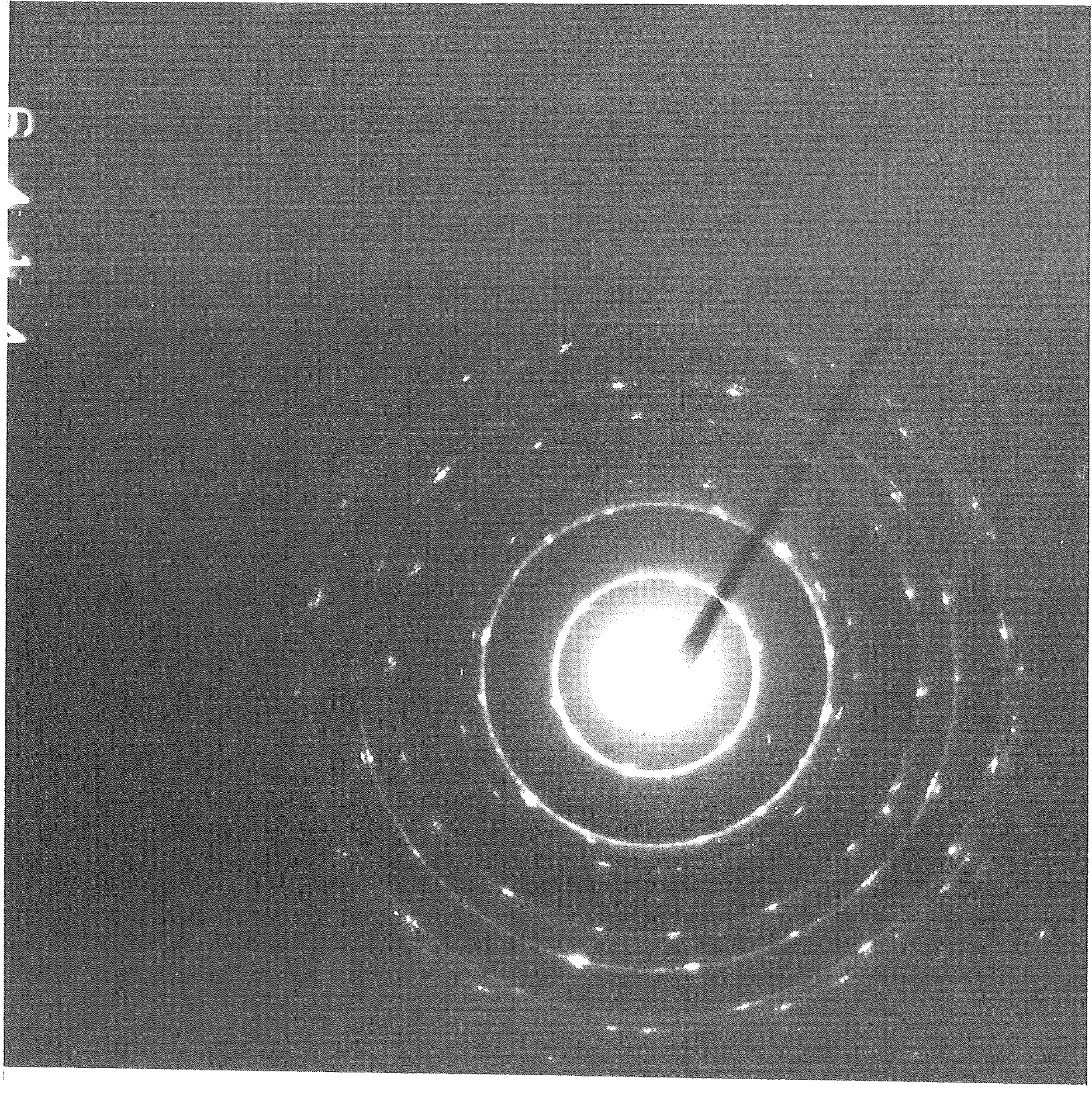


Figure 48
Electron diffraction from untreated test site
Chinle clay

Each sample was allowed to settle for ten minutes so the larger particles would go to the bottom of the sample vial. A 0.3 ml aliquot of sample suspension was then suspended in another six ml of distilled water. This technique helped to dilute the suspensions. This was necessary to keep the crystals from stacking on top of each other after drying. The diluted suspension would let the specimen dry so one could distinguish between crystals which were dispersed from each other and crystals which naturally piled on each other due to forces which kept the crystals together.

The suspended samples were placed on thin carbon films which had a thin layer of SiO evaporated on both of its sides, see figure 49.

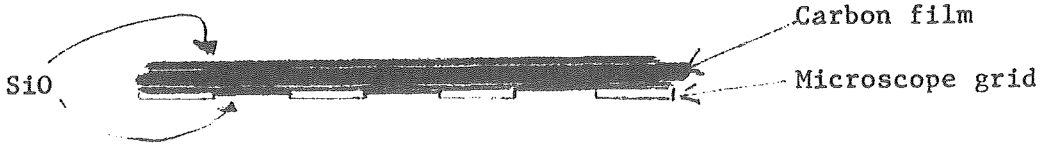


Fig. 49

The SiO evaporated on the carbon film causes the carbon surface to be hydrophillic. Carbon tends to be hydrophobic. A hydrophobic film will cause the sample to coalesce into pooled areas as the water evaporates from the grid. The SiO prevents the specimen from coalescing and thus allows the evaporation to proceed throughout the area of the whole grid. The specimen will remain dispersed on the carbon because the crystals will not be pooled as the surface tension of the water pulls the crystals into themselves.

Samples 40133, 40188, 40415A and the Old E.O. site were observed.

The evidence seems to show that the treated sample has mineral flakes which are less thick compared to the pre-test samples. This is presumably because the nature and stability of the bonding between the successive basic structural units (Lamella) has been altered causing the stacking of the units to changes geornetrically.

The USP and Wyoming bentonite samples resembled each other morphologically. These samples also more closely resembled the pre-test samples because they showed the thin crystals to be stacked up on each other, see figure 50.

The Old E.O. Site Crystals appeared to look like the post-treated samples as shown in figure 51-a.

Conclusions from Transmission Electron Microscope Data

As was pointed out in the discussion of the Electron Microscopic procedure, the evidence clearly indicates that the crystalline fabric of the clay has been affected by the treatment. In particular it appears that the Smectite layers have been dispersed after the treatment. This is indicated by the electron-diffraction patterns which is pre-treated material depicts a thick layered structure. This is noted by the wider diffraction rings in the pre-treated material as opposed to the relatively thin diffraction rings obtained from the treated material. The wide rings are characteristic of a many layered crystal. If the layers are dispersed, a pattern of thin diffraction rings is obtained. Moreover in the electron transmission photographs figures 51-b and 51-c it is shown how the layers of the treated material appear much more transparent to the electrons (51-b) than does the untreated material (51-c), indicating, respectivily, very thin crystalline layers as opposed to very thick crystalline layers.

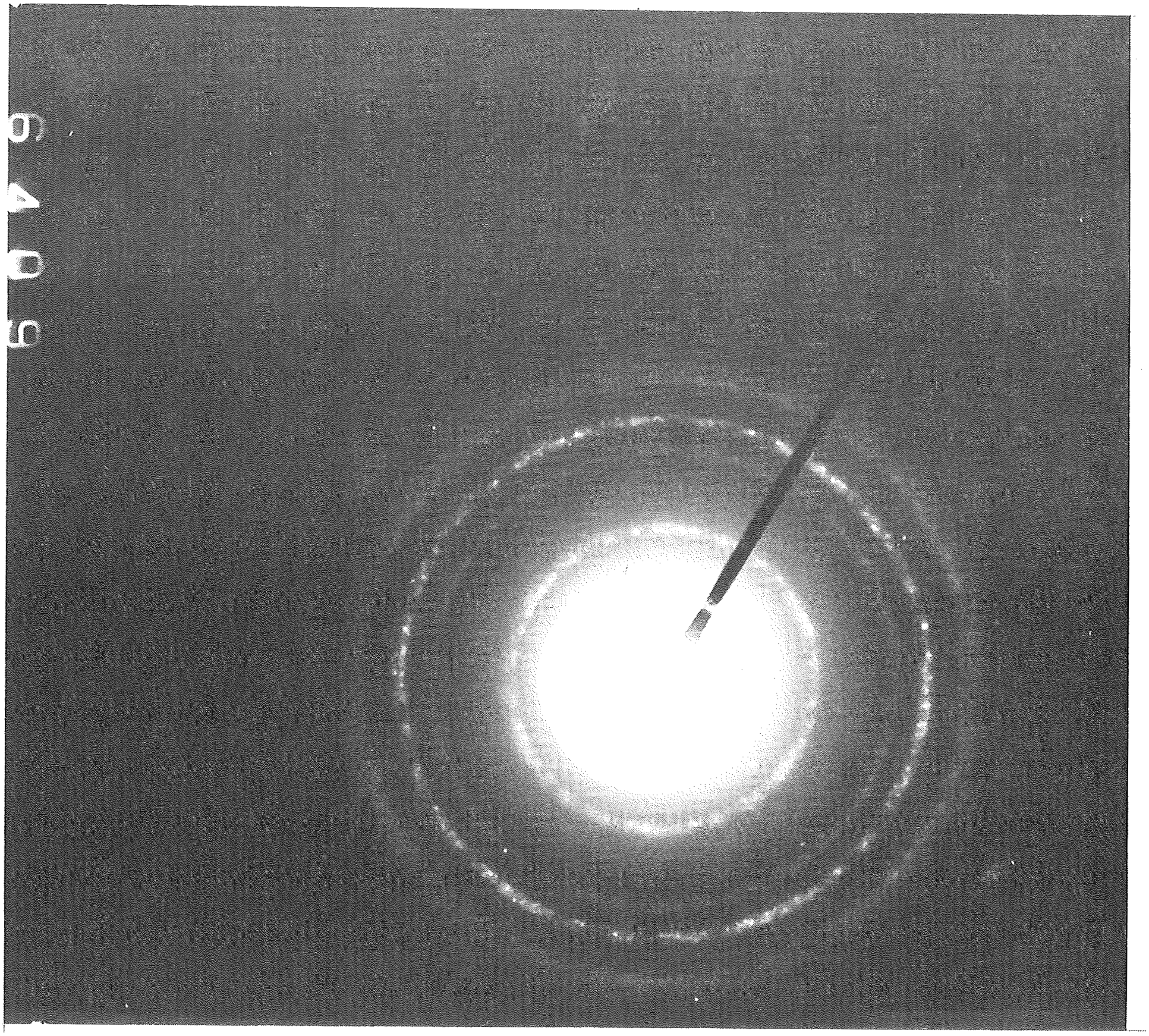


Figure 50
Wyoming Bentonite Diffiaction Pattern

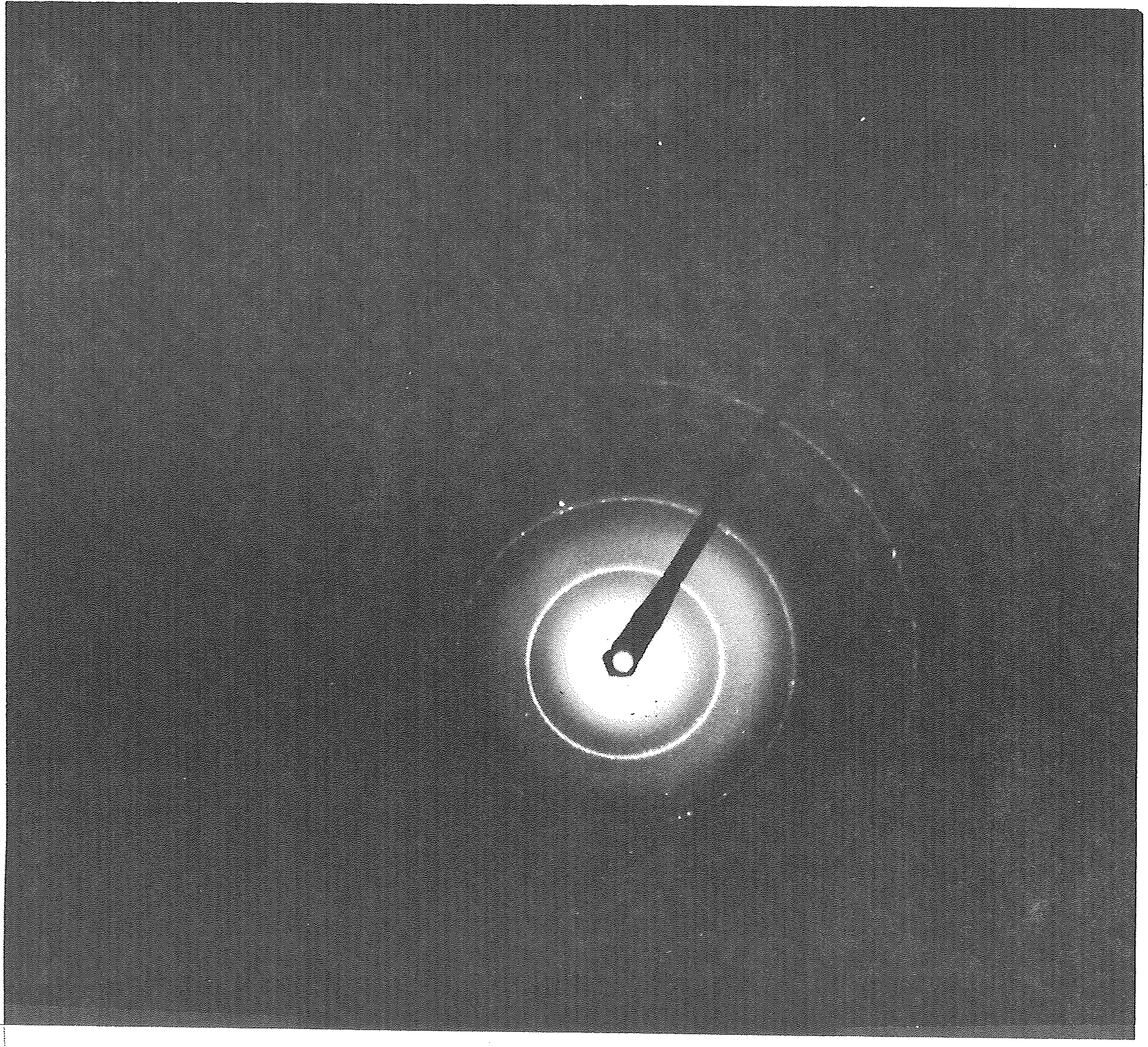


Figure 51-a
Electron Diffiaction from Old E.O.
Chinle Clay

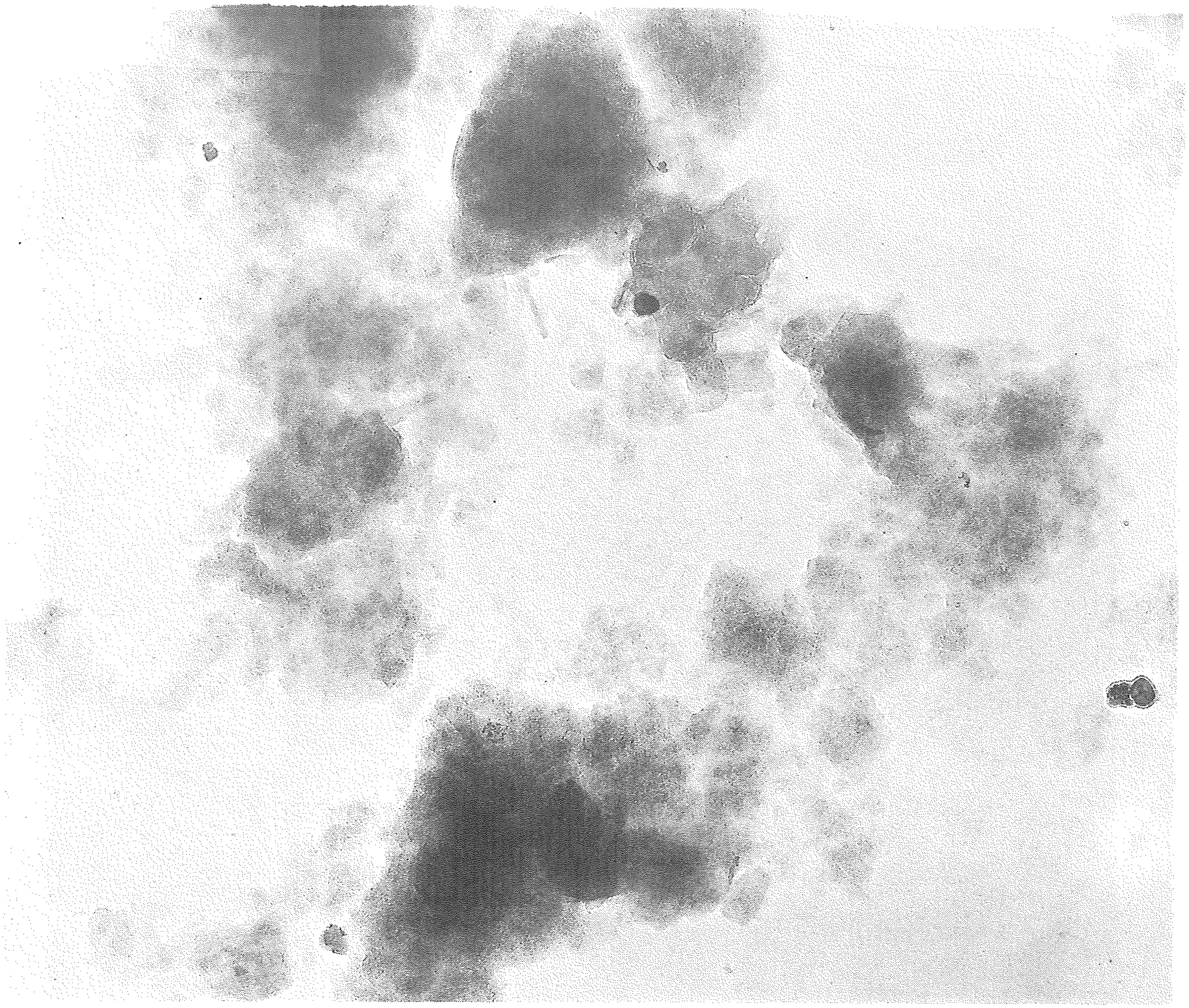


Figure 51-b
Electron Transmission of Treated Test Site
Chinle Clay
33000X

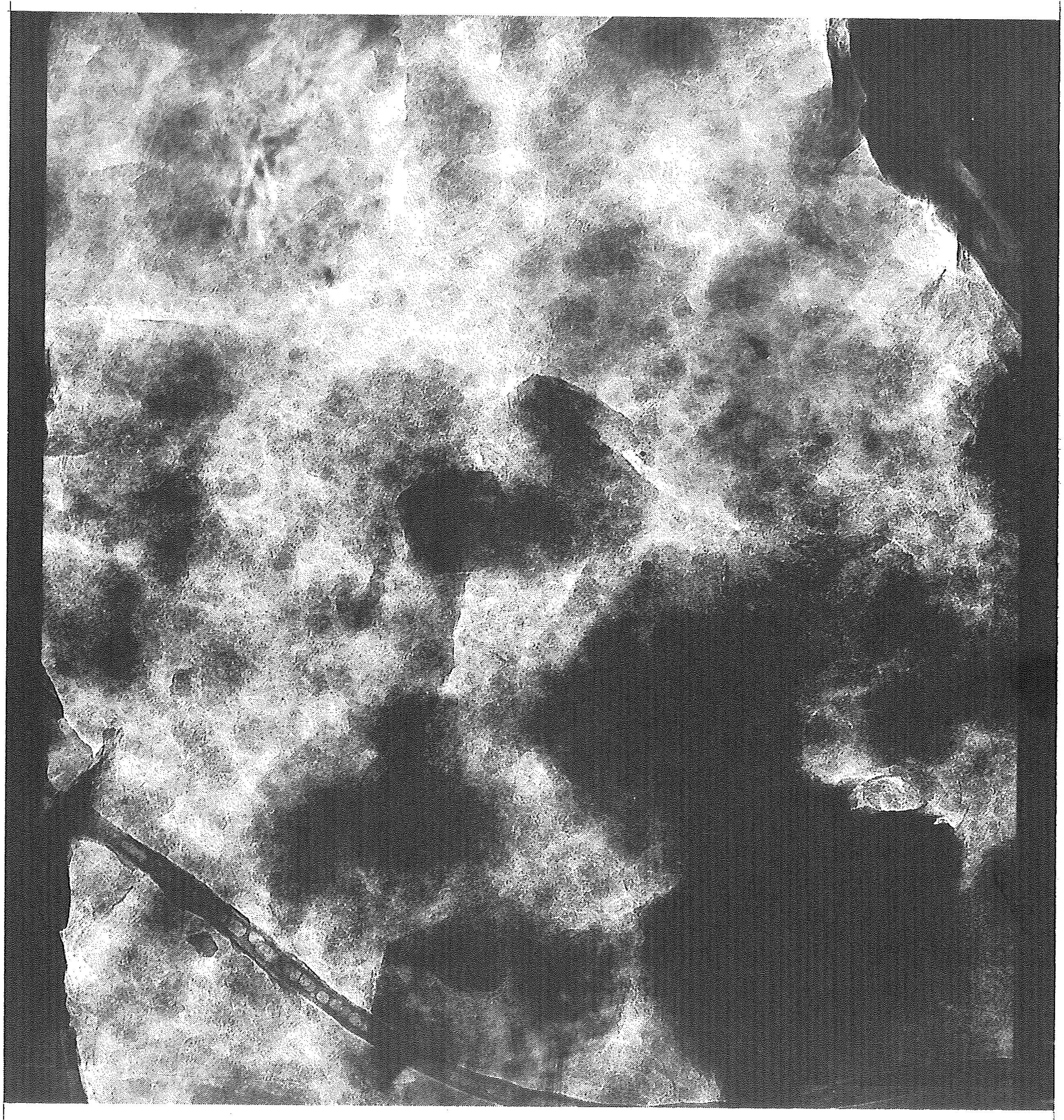


Figure 51-c
Electron Transmission of Untreated Test Site
Chinle Clay
33000X

Expansive Pressure and % Swell of Post-Test Samples

Upon completion of the electrochemical treatment samples were obtained and returned to the Materials Services Laboratory in Phoenix where expansive pressure and % swell tests were performed in the standard Clock-House Apparatus. The results of these tests are shown in Table 4.

TABLE 4
ENGINEERING PROPERTIES OF TREATED SOIL

SAMPLE #	HOLE #	DEPTH	LL	PL	PI	PERCENT SWELL	W_c	r_d	EXPANSIVE PRESSURE (LBS/FT^2)	W_c	r_d
40415A	9	0'-1'	34	20	14	1.84	13.9	106.7	1001	12.3	107.2
40415B	9	1'-2'	28	23	5	6.30	12.9	108.7	2503	11.7	111.1
40415C	9	2'-3'	38	22	16	6.70	14.2	107.5	3275	13.8	108.0
40415D	9	3'-4'	35	21	14	4.93	12.5	108.8	1775	13.9	107.8
40415E	9	4'-5'	35	20	15	2.58	12.7	108.9	1092	13.8	107.7
40415F	9	5'-6'	38	20	18	0.95	13.2	108.4	273	14.5	108.1
40416A	10	0'-3'	35	21	14	1.80	13.3	108.3	0	20.5	102.0
40416B	10	3'-6'	34	21	13	3.43	12.7	108.6	592	14.9	106.9
40416C	10	6'-9'	37	21	16	1.55	12.3	109.3	744	13.4	108.2
40416D	10	9'-15'	49	23	26	2.95	13.9	107.9	683	13.9	106.8
40417A	11	0'-3'	39	20	19	7.04	14.9	106.9	2912	15.4	106.5
40417B	11	3'-6'	38	19	19	5.49	14.7	106.7	1820	15.5	106.4
40417C	11	6'-9'	43	19	24	1.98	14.2	107.5	1727	11.4	110.9
40417D	11	9'-15'	50	23	27	11.72	15.1	106.5	4504	14.6	107.4
40418A	12	0'-3'	36	21	15	0.91	15.1	106.4	455	13.8	106.3
40418B	12	3'-6'	37	23	14	5.57	14.5	107.4	2184	15.3	106.6
40418C	12	6'-9'	44	24	20	10.29	13.8	107.9	4095	14.5	107.2
40418D	12	9'-15'	38	23	15	8.87	13.7	107.9	2457	14.5	107.0
40419A	13	0'-3'	36	19	17	5.44	14.1	107.4	2184	15.1	106.6
40419B	13	3'-6'	45	23	22	9.24	14.7	106.8	3822	15.3	106.2
40419C	13	6'-9'	45	23	22	9.18	14.2	107.4	3231	-	-
40419D	13	9'-15'	39	21	18	6.78	14.9	106.8	2321	14.4	107.5

SAMPLE #	HOLE #	DEPTH	LL	PL	PI	PERCENT SWELL	W_c	r_d	EXPANSIVE PRESSURE (LBS/FT^2)	W_c	r_d
40420A	14	0'-3'	34	22	12	2.60	16.0	105.8	571	14.2	105.4
40420B	14	3'-6'	26	23	3	4.15	15.6	106.0	2002	-	106.5
40420C	14	6'-9'	38	25	13	5.51	16.1	105.7	2821	15.1	106.2
40420D	14	9'-15'	34	22	12	5.25	15.1	106.5	2275	14.6	106.5
40421A	15	0'-3'	37	19	18	4.43	15.5	106.3	887	15.4	105.8
40421B	15	3'-6'	39	22	17	3.33	15.1	106.5	1183	15.3	106.0
40421C	15	6'-9'	41	22	19	4.18	15.6	106.2	1547	15.0	105.6
40421D	15	9'-15'	41	20	21	7.46	15.8	105.9	2548	15.1	105.4

ASU DESIGNED EXPANSOMETER

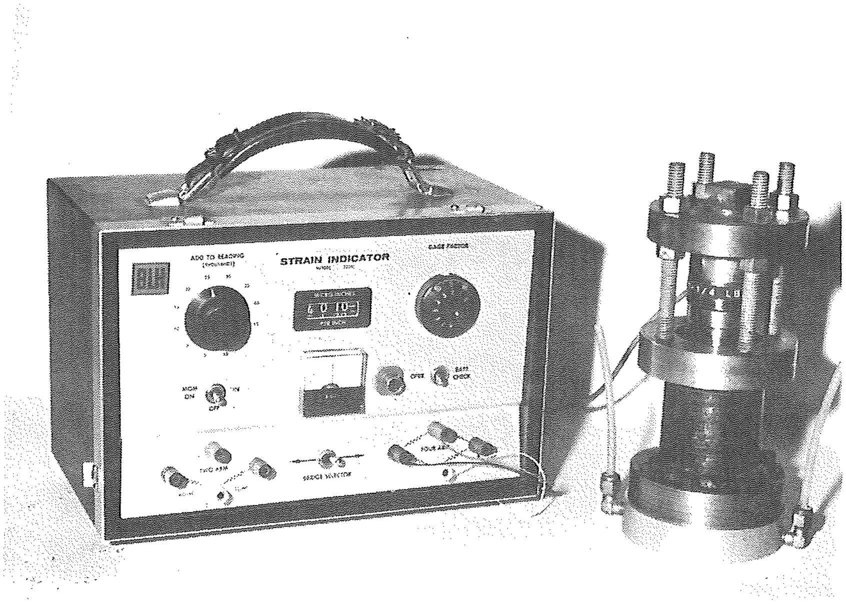


Figure 52

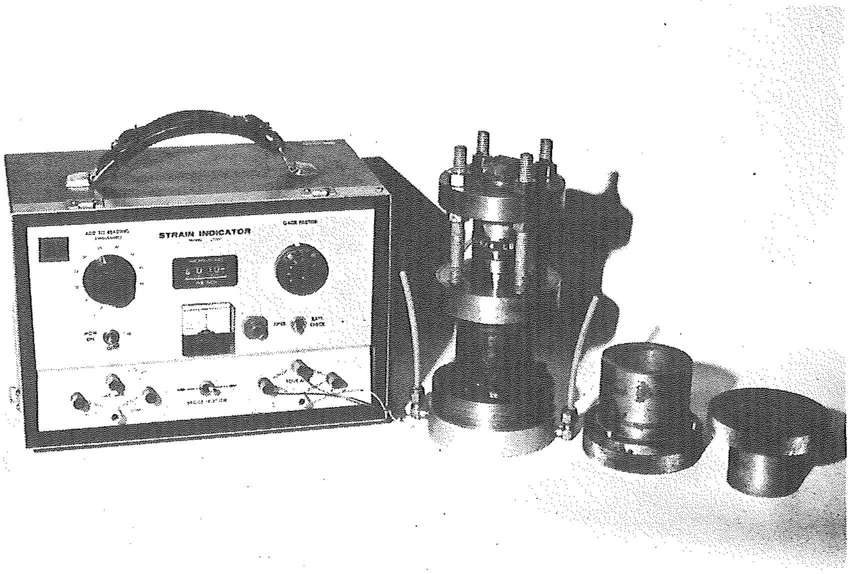


Figure 53

ASU DESIGNED EXPANSOMETER

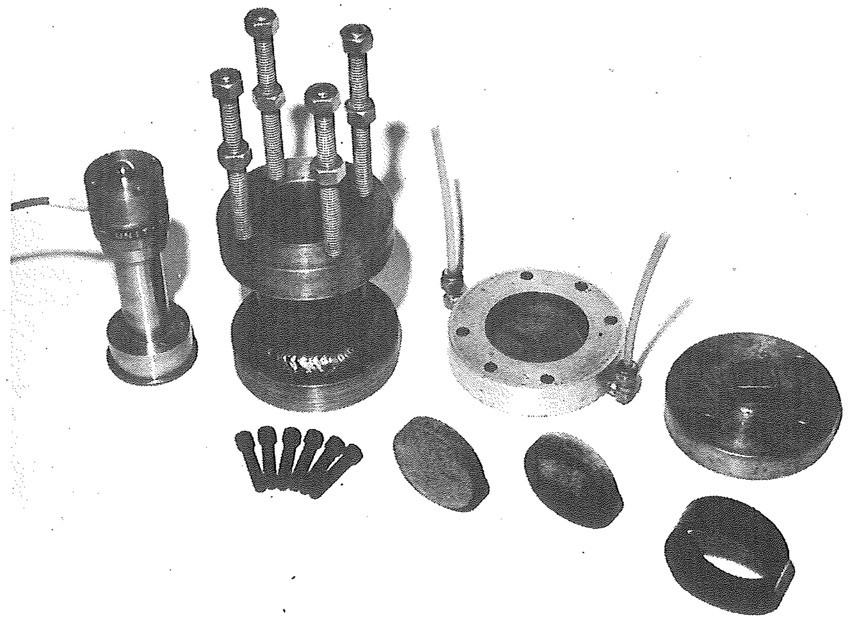


Figure 54

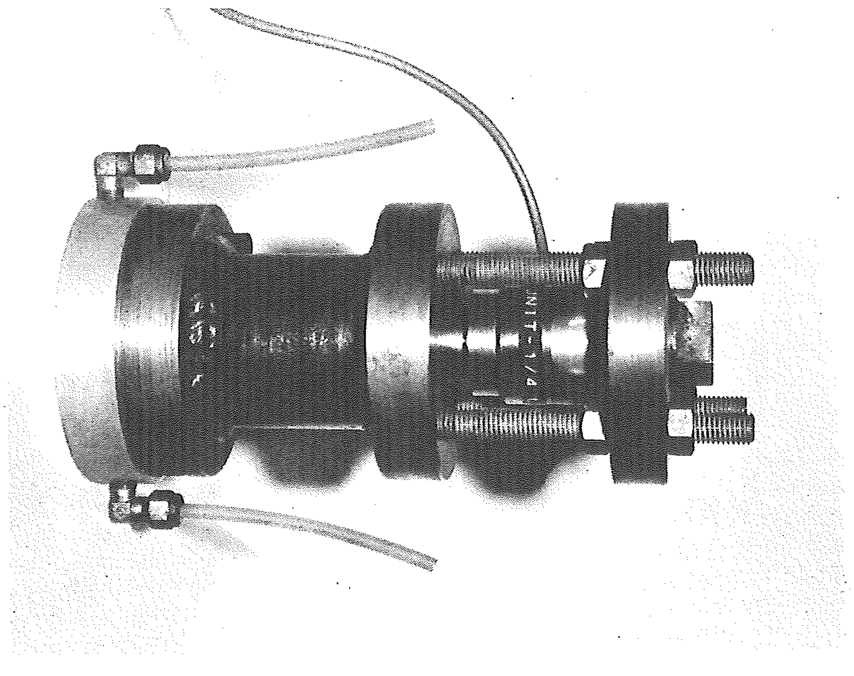


Figure 55

After the % swell and expansive pressure tests were completed in Clock-House Apparatus it was determined that the sample could possibly swell and flake off because it was not properly confined. In addition, a question of reliability of the R-value method of obtaining expansive pressure was raised. It was decided to use the A.S.U. designed expansometer and % swell apparatus as a means of control. See figures 52 thru 55 for illustrations of this apparatus.

The results of these tests are shown in Table 5.

This data indicates that the expansive pressure was decreased somewhat over 50%. The % swell was decreased by over 36%. This is a significant reduction and is an indication of the success of the electrochemical treatment.

CONCLUSIONS FROM SOIL TEST DATA

The electro chemical treatment moderately decreased both expansive pressure and % swell, it did not have any apparent affect on the atterburg limits of the soil.

It is also apparent that the soil mass treated by the method was interspersed with "hot" and "cold" spots. That is, some parts of the soil mass were significantly affected by the treatment and others were largely untouched, moreover these "hot" and "cold" spots were randomly distributed throughout the soil mass.

TABLE 5

POST TEST SAMPLES

Hole #	Depth	Sample #	Initial Moisture Content	Initial Dry Density	% Swell	Initial Moisture Content	Initial Dry Density	Expansive Pressure (PSF)
9	0-1	40415A	14.9	106.9	1.8	15.5	105.9	1022
9	1-2	40415B	15.2	106.8	4.5	15.5	106.4	1449
9	2-3	40415C	15.2	106.7	6.2	15.5	106.0	2353
11	0-3	40417A	14.5	107.2	6.7	15.4	105.8	2426
13	0-3	40419A	14.5	107.1	4.8	16.3	105.8	1635
15	0-3	40421A	14.7	107.3	4.8	15.3	106.3	1904
					ave. 4.8			ave. 1798

PRE TEST SAMPLES

16	2-3	40183	13.9	107.9	6.9	--	--	--
18	0-3	40188	13.9	107.8	9.7	15.1	106.3	5088
21	0-3	40197	14.2	107.4	6.0	14.2	107.2	2471
					ave. 7.53			ave. 3779

CHAPTER VII

MONETARY AND ENERGY EXPENDITURES FOR ELECTROCHEMICAL OPERATIONS

The following itemized costs are presented that typify the monetary and energy expenditures to optimally prepare and treat, electrochemically, a 480' long, 40' wide, in-place section of asphaltic paved interstate highway to a depth of 3' below the select material.

It must be stressed that these 1973-1975 money figures are subject to both monetary and wage inflationary forces and their usage at any future date should be adjusted accordingly.

A. Cost for Site Preparation

1. Drill 285 6" diameter, 6' deep auger holes on 8' centers and sleeve with 6" O.D. 27" long steel casing, - $4,000.

2. Dig two trenches 4' deep, 6" wide and 460' long, along with three trenches in center of roadway surface 6" wide and 138' long each - $1,000.

3. Installation of electrical components: $700.

B Cost of Expendable Items

(Note: Although the steel sleeves were designed to facilitate their reusage on future projects, circumstances may render it too expensive to attempt salvaging the sleeves. Thus we are considering the sleeves as expendable items).

1. Cost for 300, 6" O.D., 27" long casings each topped with a 10" diameter 1/2" thick steel plate with a 1-1/2" diameter hole in center of plate - $2,000.

2. Cost for KCl solution delivery system (excluding tanker truck) - $400.

3. Cost for 1500', 1" rebar steel (#8 grade 60) to use for electrodes - $570.

4. Electrical wiring components: 60' of 4/0 insulated wire (stranded copper) suitable for burial - $65.
   500' of 2/0 insulated wire (stranded copper) suitable for burial - $354.

5. KCl (potassium chloride), 90,000 lbs. granular agricultural grade, in 100# multi-wall paper bags on wood pallets - $3,100.

6. 860 lbs of aerosol C-61 wetting agent (Surfactant) in 430 lb drums - $860.

7. 3120 gallons of diesel fuel at 40¢/gal, to operate 60 KW D.C. generator for 40 days continuous operation - $1,248.

<u>Equipment</u>

1. 2,500 gallon converted water truck, ADOT equipment.

2. 60 KW D.C. generator, ADOT equipment.

Solution Delivery Cost

1. 200 man hours at $3.50/hr. - $700.

Hence the total cost to prepare and optimally treat a 480' long, 40' wide inplace, asphaltic paved, roadway section 3' below select material was $14,997.

Based on the assumption that the material is *treated uniformly giving a swell reduction factor of 2 or more*, it is estimated that the cost per ft^3 is approximately 26¢/ft^3. Clearly if the material mass is not uniformly treated by the electrochemical process this cost/ft^3 figure will increase correspondingly.

Energy Expenditure:

The energy in-put from the start of the electrochemical treatment to the finish was obtained from the energy rating of the 3120 gallons of diesel fuel consumed by the 60 KW generator. Each gallon of diesel fuel has an energy rating of 140,000 B.T.U.

A. Energy in-put - 4.368×10^8 B.T.U.

B. Total generator output - 19,657,296 watt hours. In terms of Joules this figure is $1.9657296 \times 10^7 \times 3.6 . 10^3 = 7.076626560 \times 10^{10}$ Joules. So using a B.T.U. rating the generator output was

$$\frac{7.076626560 \times 10^7}{1.05587} \text{ B.T.U.} = 6.702175988 \times 10^7 \text{ B.T.U.}$$

This gave a usable energy factor of about 15.34 percent. So about 84.66 percent of the available energy was lost because of generator efficiency and electrical losses.

With a generator in top running condition it is estimated that it would be possible to raise the usable energy factor to over 40%.

Although the last figures given here appear to reflect excessive costs, it must be borne in mind that this field work has a first time operation. Correspondingly the man hours involved were much longer than would be the case in any future work utilizing personnel familiar with the technique.

CHAPTER VIII
NUCLEAR MOISTURE MONITORING

Midway in the project the Materials Services of the Arizona Department of Transportation set up two experimental sites located at Station 12+00 - 11+76 and Station 11+76 - 11+52 for the purposes of monitoring soil moisture movement during the actual field work.

Troxler nuclear depth moisture gauges were employed during the field work to determine the moisture movement during the solution input period. Moreover, ion movement determination was also made since it was felt that potassium K^+ or chloride Cl^- ions could have interferred with nuclear readings, this was done chemically.

To monitor the moisture movement from the post holes aluminum access tubes were placed in a pattern as shown in figure 56. in all 38 tubes were used. The AL tubes allowed the nuclear moisture probes to be inserted into them to any depth from 12" below the pavement surface to a depth of nearly 96". A typical access tube installation is shown in figure 57.

Laboratory tests were made on soil samples, after the field work was completed, to determine the total distance of ionic movement vs concentration level from the center line of a post hole. The results of these tests are shown in figures 58 and 59 for both K and CL.

It sould be noted that the K concentration rapidly diminished with distance from the post hole, indeed concentration greater than one percent by weight was not found two feet from the post hole. At first glance it may appear that the CL concentration is much greater at any distance than that of K, however, note that the CL concentration is plotted on a different scale than it is for K, however the CL seemed to move further thant the K^+ ions.

NUCLEAR MOISTURE MONITOR LAY-OUT FOR HPR-1-11 (145)

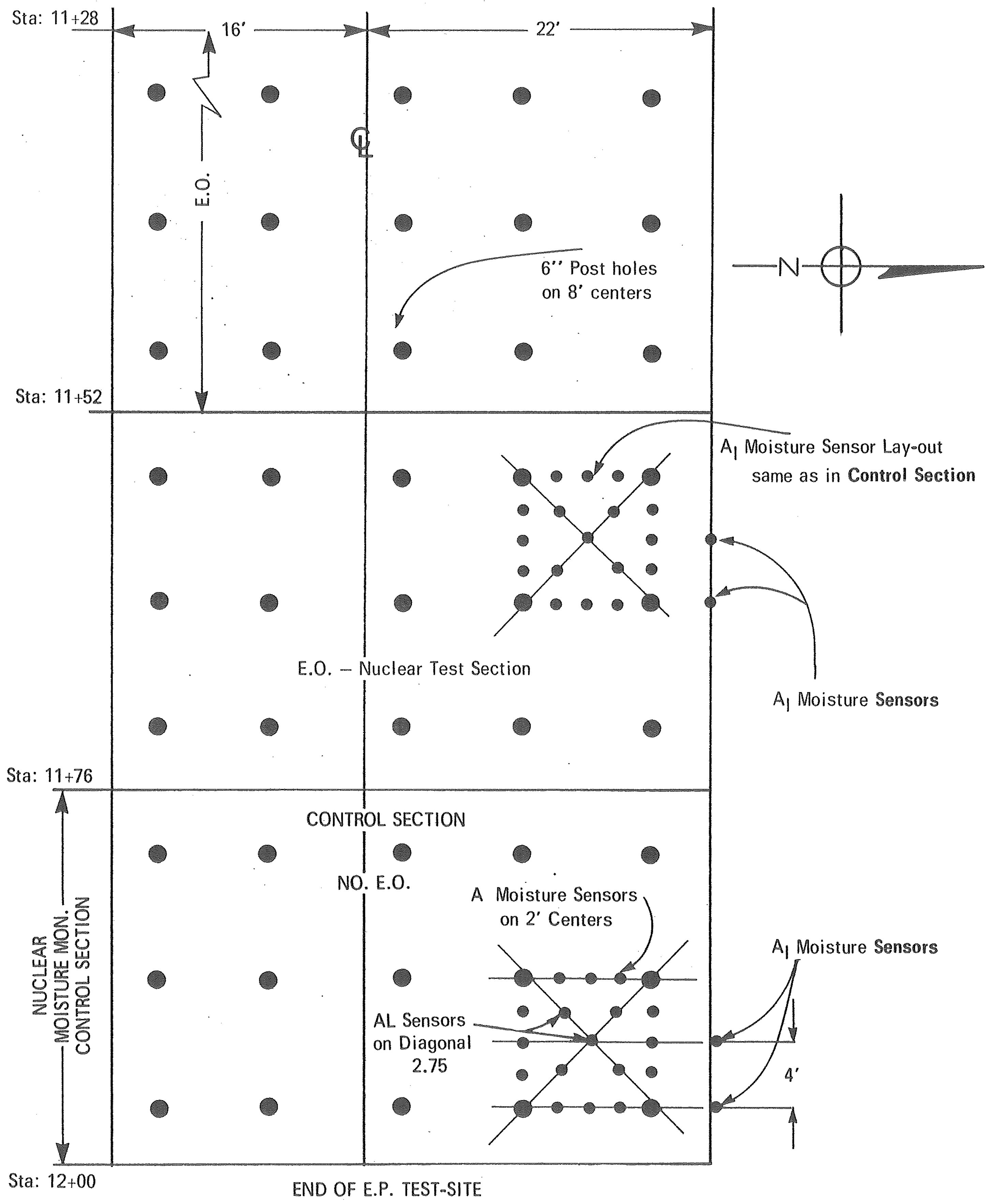
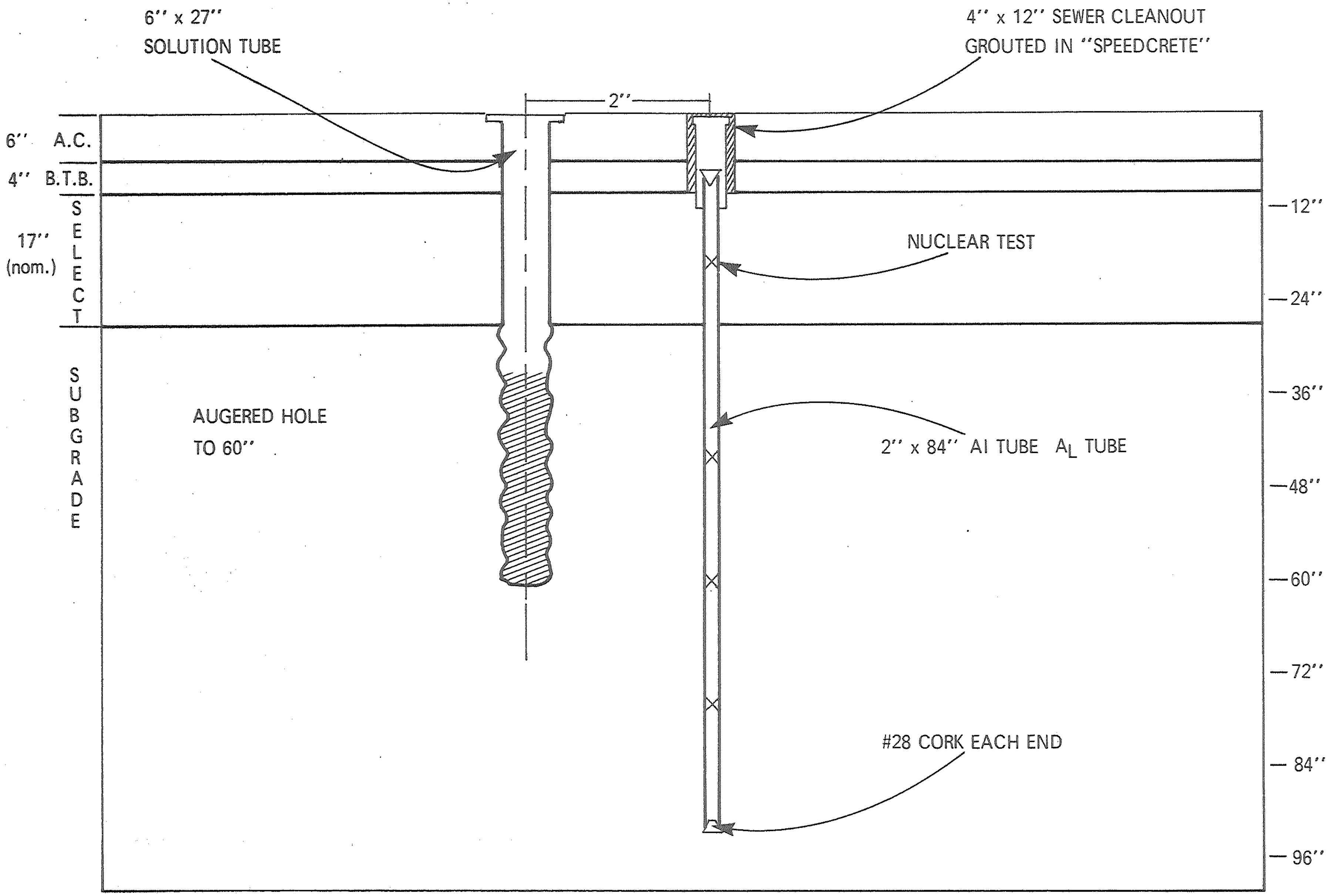
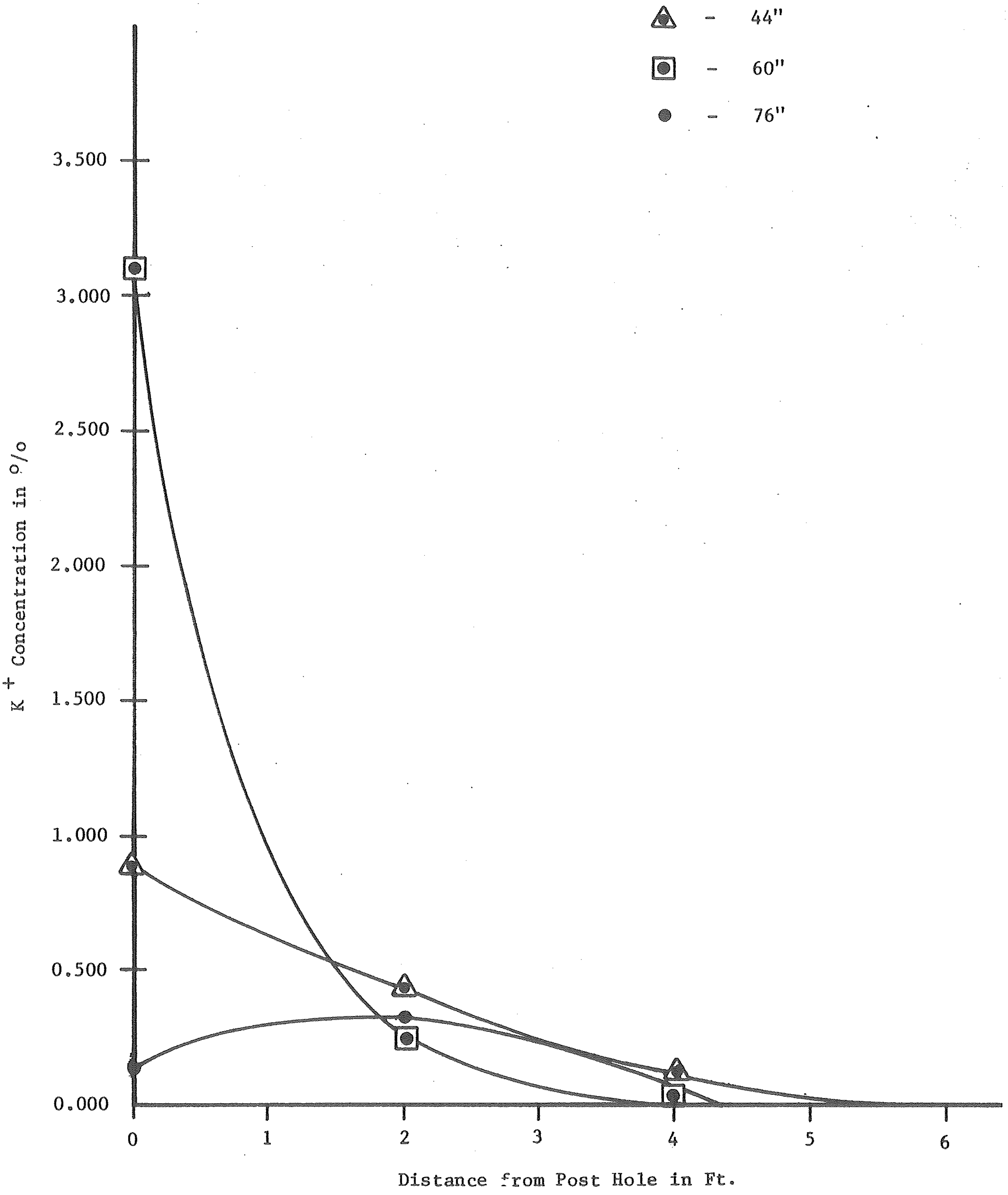


Figure 56



TYPICAL ACCESS TUBE INSTALLATION
HPR-1-11 (145) CRAZY CREEK CUT
Figure 57



K^+ Ion Concentration
VS Distance from ℄ of Auger Hole
for Various Test Depths

Figure 58

CL - Concentration vs Distance from
℄ of post hole for various test depths

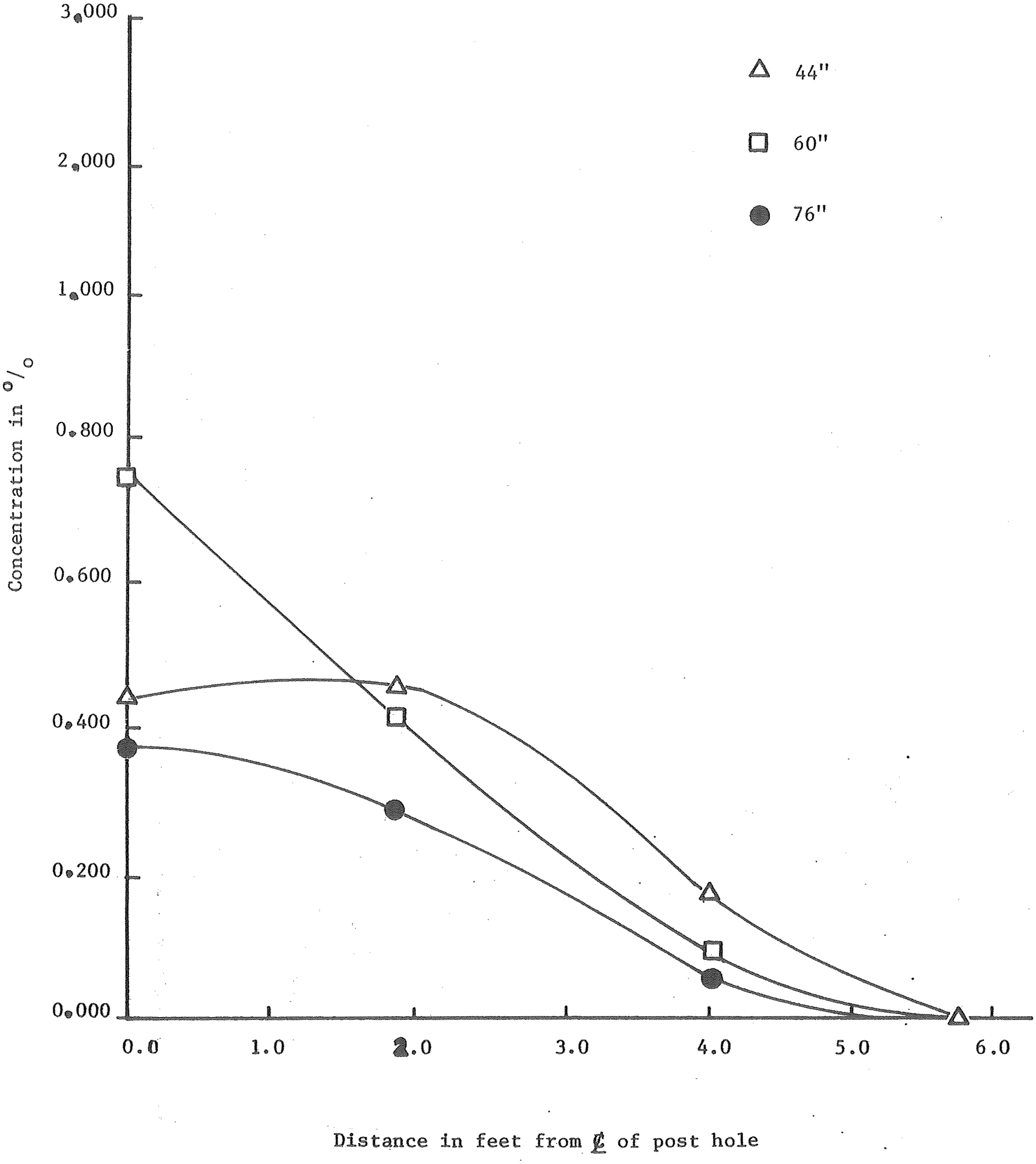


Figure 59

TYPICAL DIFFUSION CURVES UNDER ACTION OF CONSTANT FORCE

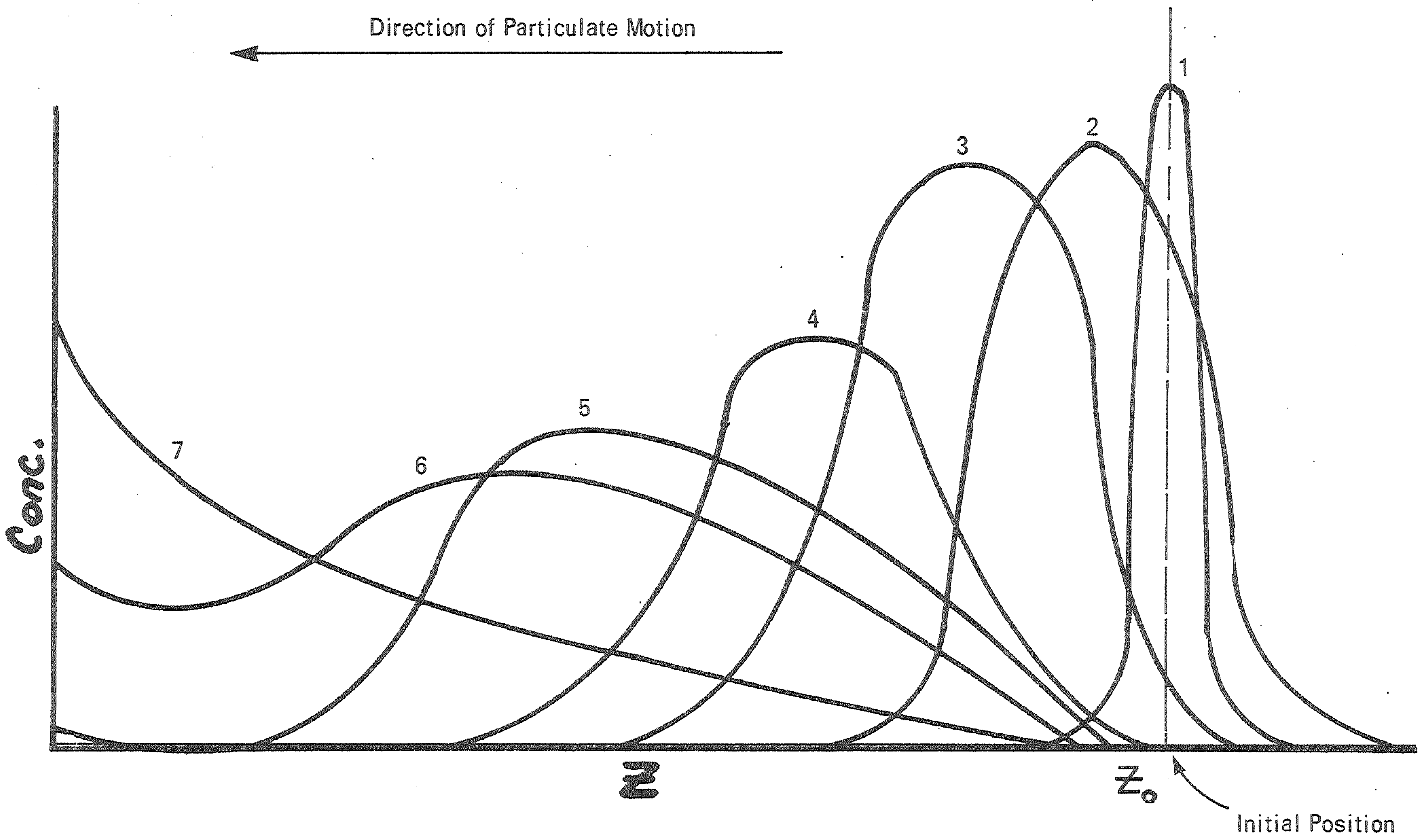


Fig. 60

It is interesting to note that these curves are similar to standard one given for diffusion of particles in force fields as shown in figure 60.

Note that both curves given for K and CL at a depth of 60" appear to be similar to the "tails" of curves 2, 3, or 4 shown in figure 60. The curve for CL at a depth of 44" seems like curve 5 or 6 of figure 60 while the curve for K at a depth of 76" seems like 5 or 6. This strongly indicates that the ions were indeed acted on by an electric field during their migration through the soil.

Conclusions from Nuclear Moisture Monitoring:

The test results indicated an overall drying trend for the test site during the course of the solution input. That is to say, the nuclear gauge information implied that the soil mass, comprising the site region, was drying out during the course of the field work. Moreover, the Materials Services of Arizona Department of Transportation feels that the nuclear gauge has proven itself to be a useful tool in monitoring long-term moisture changes in subgrade or base materials.

CHAPTER IX

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

This study focused primarily on the implementing Electrochemical soil treatment procedures developed in previous F.H.W.A. sponsored work.

Initially a site for the field work was chosen along a section of I-40 that was experiencing swelling problems. The site was selected along a roadway of high traffic volume so that traffic control problems during an operation of this sort could be fully considered since this method of soil stabilization is intended to become part of A.D.O.T. Maintenance procedures.

In this work the primary considerations were:

(1) Relative ease of field implementation of established electrochemical soil treatment technology.

(2) The effectiveness of soil treatment by electrochemical methods when the field operations are conducted largely by unskilled and semi-skilled maintenance personnel.

(3) Cost of field operation.

Based on this study it can be stated that the use of electrochemical soil treatment technology can be effectively implemented utilizing existing manpower and material resources of A.D.O.T. supplemented by minor purchases of specialized hardware and software, however, it is to be stressed that the method is an expensive one, costing about $9 per cubic yard of soil treated.

Moreover, this study has indicated that electrochemical stabilization is a viable method which will moderately reduce excessive swelling pressures if they are encountered in a localized area over which an existing pavement rests; however, using the same resources as were used on this project, treatment of a volume of material greater than 1800 cubic yards (approximately 1380 cubic meters) should not be attempted.

Conclusions

(1) Based on this study electrochemical soil treatment can be successfully completed by A.D.O.T. Maintenance personnel using the previously established field procedures.

(2) From the x-ray diffraction and transmission electron microscope data it is evident that the electrochemical treatment with KCL solution is effective in altering the physical characteristics of Montmorillonites and thereby reducing the expansive pressure of the soil.

This evidence, both x-ray and electron micrographic, indicates that, generally, the environment of the Chinle Clay's particles interlayer space has been altered which, of course, significantly affects the engineering properties of the clay.

As to the effects on the lattice structure of the clay, i.e. diagenesis, there is no conclusive evidence wihch clearly shows that significant amounts of the Chinle Clay are electrochemically converted into an illitic clay.

(3) Based on previous work it is apparent that the lower the moisture content at the beginning of treatment the more effective the treatment.

(4) The higher the percentage of Montmorillonite the more effective the treatment.

(5) The most effective electrode configuration for field installation is one in which both the anode and cathodes are placed in a horizontal position.

(6) Potassium chloride is a water soluble metallic salt that will effectively reduce the swelling of Montmorillonite.

(7) It is recommended that the average voltage gradient (potential/distance between electrodes) for field projects lie between 0.2 volts/cm and 0.4 volts/cm. Below 0.2 volts/cm the time requirement is prohibitive. Above 0.4 volts/cm there is a noticable heating of the soil which is an indication of high energy loss and waste associated with this loss.

(8) Because of the inherently random nature of the electrochemical process when used on heterogeneous material, like Chinle Clay, it is necessary to construct a very fine net of solution wells to satisfactorily inundate the soil with a stabilizing chemical. It was found that the net as used on this project, i.e., 6" (15.24cm) post holes on 8' (2.4m) centers, was not fine enough. Moreover, the fineness of the solution well net will be a function of the soil characteristics and hence must be determined by laboratory tests.

(9) Electrochemical soil stabilization is most effective on a highly localized clay mass which has high swell potential.

(10) The nuclear moisture monitoring gauge has proven itself to be a useful tool in monitoring long-term moisture changes in subgrade

or base materials. However, it should not be used in the presence of significant concentrations of neutron poisons, i.e., those elements with a high neutron absorptive cross section.

Recommendations

Electrochemical soil stabilization is recommended for use whenever a highly localized mass of high-swell clay is encountered. In such cases it is quick acting, efficient and has long lasting effects.

This study has produced several operational techniques that will facilitate the field treatment of expansive clays over which a roadway is built to reduce the swelling potential. It is recommended that the process be implemented using the following procedure:

(1) Execute preliminary sampling plan as given in Chapter II of this report.

(2) After identification of degree of swelling determine depth of stabilization required based on the expansive pressure if in-situ material. This will range from 3' (.912m) to 5' (1.5m). A typical design figure would be 4' or about 1.2 meters.

(3) Determine volume of soil to be treated. Based on this figure obtained required amount of KCL by using a design figure of 1.6% - 2% KCL per unit weight of soil.

(4) Prepare site for electrochemical treatment using 6" (15.24cm) diameter auger holes sunk approximately 6' (1.8m) below surface on 4' (1.2m) to 8' (2.4m) centers (Note: The exact centering distance can only be determined by solution movement tests in soil samples). Sleeve auger holes with 6" (15.24cm) O.D. steel pipe with a 10" (25.4cm) diameter plate will have a 1.5" (3.81cm) diameter hole in center to permit the introduction of KCL solution. The pipe should have a length which is the depth of the base course plus 6" (15.24cm).

(5) Place horizontals anodes and cathodes and make electrical connections.

(6) Place solution in wells twice a day for 30 days.

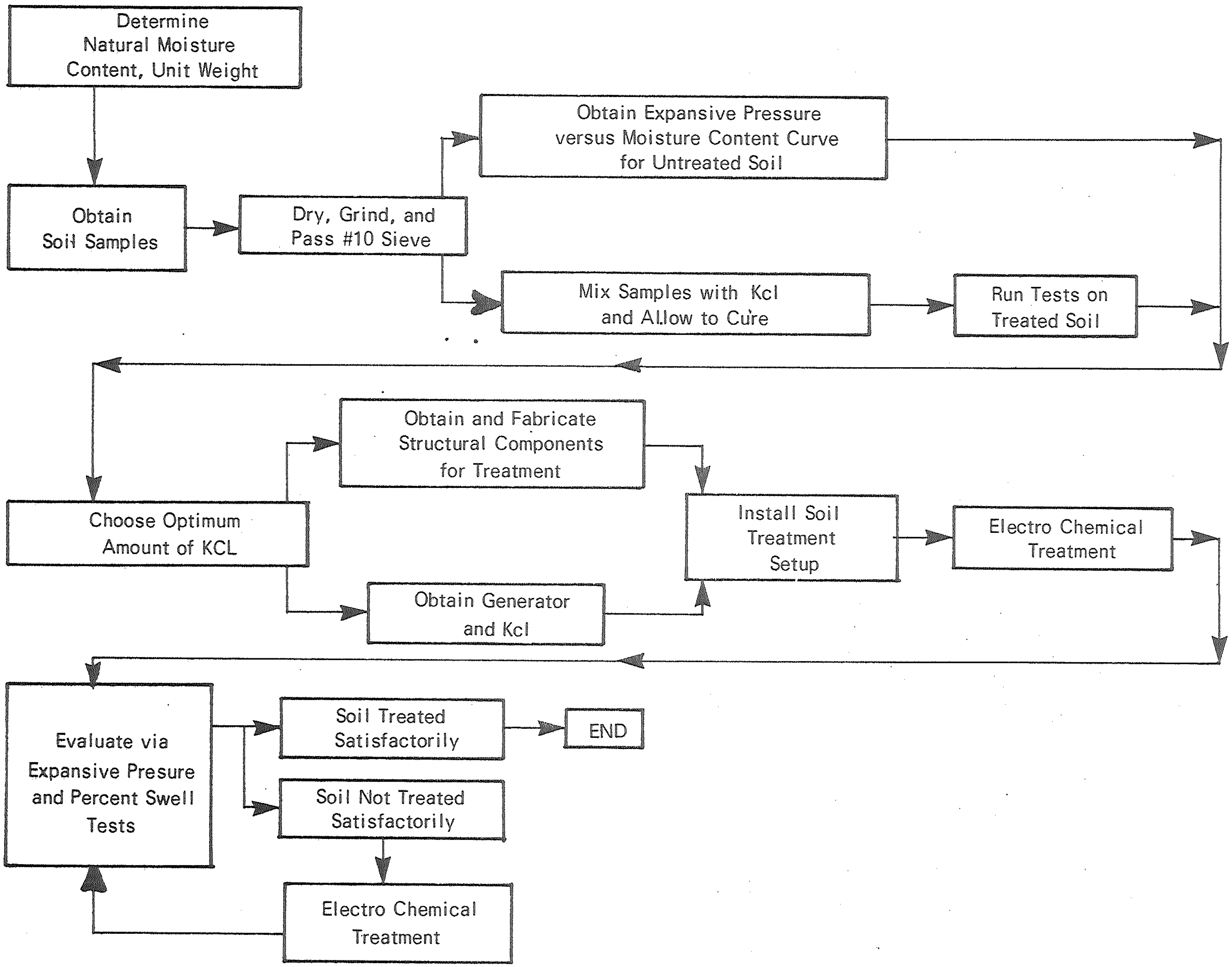
(7) At the end of 30 days turn on a voltage gradient of 0.2 volts/cm and continue operation with applied voltage gradient for 3 to 4 weeks depending on electrode polarization phenomena.

(8) Collect sufficient post-test samples to determine effectiveness of electrochemical treatment.

A flow chart illustrating the essential procedure steps, comprising an electrochemical treatment, is given in Figure 61.

(9) To avoid moisture measurement errors during the operation of Nuclear Moisture Monitoring gauges in an environment containing elements with a high Neutron Absorption cross section, i.e., 10 barns, continual calibration of the gauges relative to the chemically inundated soil must be carried out.

ELECTROCHEMICAL SOIL STABILIZATION
FLOW – CHART



Figure